Quantifying the Fatigue Damage Accumulation in Bridges

by

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Abstract

Many of the existing bridges in the United States were built during the interstate era and are reaching the end of their life cycle. Traffic-induced loadings are one of the primary factors affecting the service and fatigue life of bridges and could accelerate bridge deterioration, making them structurally deficient or obsolete. According to U.S. DOT's report to Congress on condition and investment requirements of the nation's highway and bridges in 2015, 25% of the 607,380 bridges in the U.S. are either structurally or functionally deficient. They projected a cost of $123.1 billion for replacement or rehabilitation of these deficient bridges.

State transportation agencies are interested in knowing the damage caused by overloaded vehicles (permit loads and illegal loads) to bridges for the potential uses in weight limit enforcement, budgeting, maintenance, and planning inspection intervals. The weigh-in-motion (WIM) database is the major source of information about traffic loads. An added benefit of the WIM system is that it can measure detailed vehicle weight information of the vehicles traveling on highways without the knowledge of drivers.

This dissertation first proposes a data-driven decision support tool that: (1) evaluates the quality of WIM traffic data to avoid misinterpretation of traffic load effects, (2) identify permitted and illegally loaded vehicles and (3) develop procedures to quantify the fatigue damage caused by traffic loads to steel bridges. The procedures are demonstrated using traffic data collected in the state of Alabama. Second, the adequacy of the current AASTHO fatigue design truck for the state of Alabama is checked.

The developed quality control procedure can interpret inconsistency in recording due to communication failure, operational problems with the sensor, and drift in the calibration of WIM systems. Two novel techniques are proposed to sort legal, permit, and illegally overloaded vehicles in the accumulated traffic data.

The procedure to quantify fatigue damage allows comparisons of the impacts of truck traffic on various routes and also for a specific fatigue prone detail in a bridge. The results show that approximately 20% of trucks in Alabama that are overloaded create more than 50% of the total damage based on the combined data from all the WIM locations in the state. A typical steel bridge with bottom flange cover plates was evaluated for a heavily traveled route. This analysis shows that the fatigue life of the bridge was consumed at an annual rate consistent with a mean life of 100 years. Computer apps AL_WIM_QC and AL_WIM_DAI were then developed using the developed procedures to check the quality of WIM data and to quantify the fatigue damage accumulated in bridges. Also, the developed procedures may be incorporated into the National Bridge Inventory (NBI) to assess the knowledge of current loads on each bridge, evaluation of current and future conditions of highway infrastructure and budget allocation for maintenance and improvement.

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and

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List of Abbreviations

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ALDOT	Alabama Department of Transportation
API	Application Programming Interface
ASCE	American Society of Civil Engineers
ATR	Automatic Traffic Recorder
CAFT	Constant Amplitude Fatigue Threshold
CDF	Cumulative Distribution Function
CV	Coefficient of Variation
DOT	Department of Transportation
FHWA	Federal Highway Administration
GDF	Girder Distribution Factor
GIS	Geographic Information System
GVW	Gross Vehicle Weight
HPMS	Highway Performance Monitoring System
KML	Keyhole Markup Language
LRFD	Load and Resistance Factor Design
LTPP	Long Term Pavement Performance Program
MBE	Manual for Bridge Evaluation

MEPDG	Mechanistic-Empirical Pavement Design approach
NBI	National Bridge Inventory
OS/OW	Oversize/Overweight
PDF	Probability Distribution Function
TS&W	Truck Size and Weight
TMG	Traffic Monitoring Guide
WIM	Weigh-in-Motion

List of Symbols

γ	load factor
(Δf)	force effect, live load stress range due to passage of fatigue load
$(\Delta F)_n$	nominal fatigue resistance
α	damage accumulation index
D_m	fraction of mean fatigue life expended at a specific fatigue prone detail
λ	fatigue damage ratio
σ	standard deviation

Chapter 1: Introduction

1.1 Background

Bridges and pavements constitute a vital part of the highway infrastructure. The socio-economic well-being of a region or a state is directly dependent on its infrastructure. The core role of state transportation agencies is to maintain the safety and usability of the infrastructure within their mandate. Many bridges in the U.S were built before 1975, in the post-interstate era (Mohl and Rose 2012), where bridges were designed for 50 years of design life. Currently, many of these bridges are approaching the end of their life cycle. It was not until the early 2000s that bridges were designed for a 75-year design life after the bridge design specifications were calibrated according to Load and Resistance Factor Design (LRFD) (Nowak 1999; Nowak and Young-Kyun 1991). However, it was mandated by the Federal Highway Administration (FHWA) that after October 1, 2007, the new bridges should be designed according to *AASHTO LRFD Bridge Design Specifications* (Tobias 2011).

According to U.S. DOT's 2015 report to Congress on condition and investment requirements of the nation's highway and bridges, 25% of the 607,380 bridges in the U.S. are either structurally or functionally deficient, and $123.1 billion is required for replacement or rehabilitation (FHWA 2017). ASCE's *Infrastructure Report Card* states that the average age of bridges in the U.S is 43 years old, and grades bridge infrastructure as C+, indicating it is mediocre and requires attention (ASCE 2017).

The service life of a bridge is affected by many factors such as, but not limited to, traffic loads, natural hazards, and defects in material production. Traffic-induced loads cause damage to a bridge by either fatigue and overload or a combination of two. Steel bridges are more prone to fatigue cracking compared to other types of bridges (Azizinamini et al. 2013).

To perform bridge management tasks, bridge owners must know the actual traffic loads or live loads. Most of the damage to bridges is caused by overloaded vehicles (Ghosn et al. 2015; Nassif et al. 2015). The magnitude of traffic loads is controlled by:

- Legal load limits
- Permit loads, numbers and weights
- Control of illegally overloaded vehicles

Fatigue is addressed in *AASHTO LRFD Bridge Design Specifications* by designing the bridge structures for fatigue and fracture limit state to limit the crack growth under repetitive loads to prevent fracture, thereby capable of safely carrying design loads for a specified lifetime (AASHTO 2017). The AASHTO LRFD fatigue design truck should envelop the fatigue loads caused by the current traffic. Fatigue in steel bridges is a major concern and a recently released *Innovative Bridge Design Handbook* (Pipinato 2015) states that "*ASCE Committee on Fatigue and Fracture Reliability* (1982a, 1982b,1982c, 1982d) reported that 80%–90% of failures in steel structures are related to fatigue and fracture."

Knowledge of the actual loads, including illegally overloaded vehicles, can help in day-to-day and planned maintenance procedures and law-enforcement effort. The primary source of information about bridge traffic loads is weigh-in-motion (WIM) data. There is an enormous WIM database collected by states, for various locations, practically covering the whole nation, but this valuable resource is underutilized by bridge engineers. There is a need to assess the periodic evaluation of traffic-induced loads and fatigue evaluation for maintaining the safety of the bridges.

1.2 Service Life and Design Life of Bridges

In *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017) the terminologies are defined as:

Service Life: The period of time that the bridge is expected to be in operation.

Design Life: The period of time on which the statistical derivation of transient loads is based: 75 years for these specifications.

Fatigue Design Life: The number of years that a detail is expected to resist the assumed traffic loads without fatigue cracking. In the specifications, it has been taken as 75 years.

In Federal Highway Administration's (FHWA) *Bridge Preservation Guide* (FHWA 2018) the terminologies are defined as:

Service Life: It is the period for which a component, element, or bridge provides the desired function and remains in service with appropriate preservation activities.

Design Life: The design life is the period for which a component, element, or bridge is expected to function for its designated purpose when designed, constructed, and maintained as per standards.

Figure 1.1 shows a typical life cycle condition of a bridge. Over time, the condition of the structure is deteriorated, and the life of the bridge is consumed. Preventive maintenance is carried out to increase the performance of the bridge. Life of the bridge is consumed further over time until it reaches the minimum acceptable performance, at which point there is a need for major rehabilitation or replacement. The service life of the bridge is reached when the bridge must undergo major rehabilitation or replacement. If the service life of the bridge is consistent with the design life, then the bridge has provided the desired level of performance.

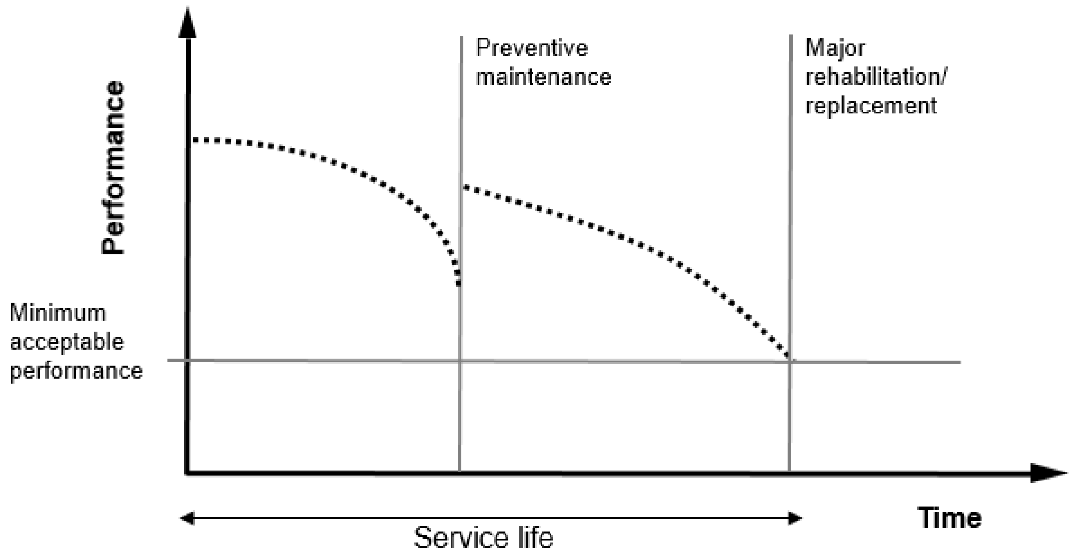


Figure 1.1: Bridge condition life cycle.

The service life and rate of consumption of bridges are affected by many factors. Overall, the bridge system is affected by various degrees of internal and external factors that influence the rate of consumption that impacts the service life of the bridge. Recently, *Design Guide for Bridges for Service Life* (Azizinamini et al. 2013) discussed the factors affecting the service life, using experience and data collected from local and state agencies. Figure 1.2 shows a fault tree diagram that identifies the factors that affect the service life of a bridge. At the highest level, it can be attributed to obsolescence or deficiency. Obsolescence refers to outdated, attributing from capacity to accommodate traffic, bridge physical issues, or due to an increase in design live load. Deficiency refers to damage or deterioration of a bridge consisting of deck, superstructure, and substructure. In a typical bridge, the deck supports the live load, and superstructure supports the deck and transmits the load to across the span. The superstructure is supported by substructure. The factors that contribute to the superstructure component can be due to loads, natural or production defects. The latter two can be controlled to some extent. Traffic-induced factors are mainly fatigue, overload and wear and tear of the superstructure. Here, the discussion is limited to superstructure components, with a focus on girder type bridges.

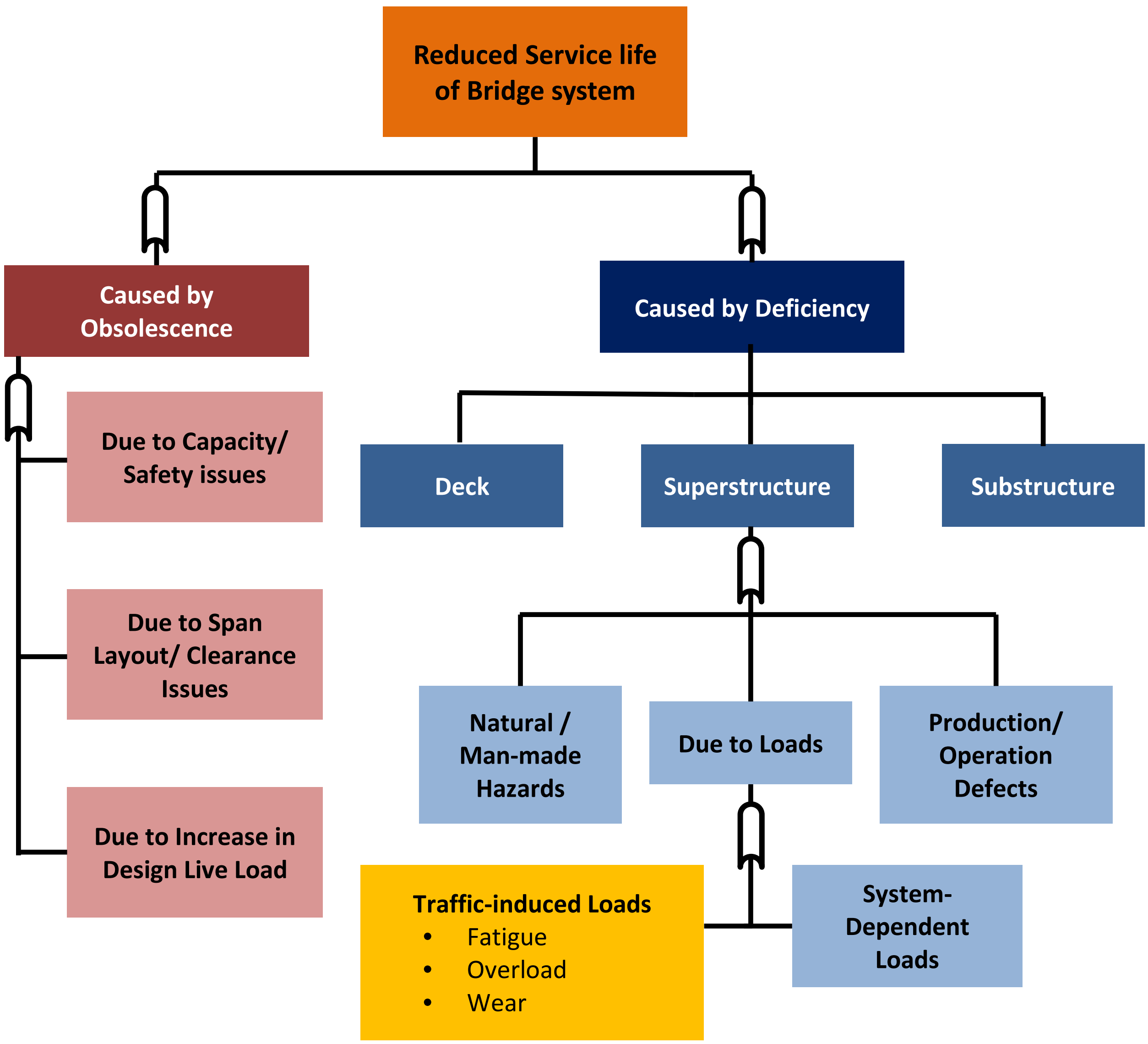


Figure 1.2: Factors affecting the service life of bridges. (Azizinamini et al. 2013)

1.3 Justification of Research

Bridges are affected by heavy traffic, and the major factors are traffic volume, weight and axle configuration of the vehicles, and multiple presence, i.e., the occurrence of multiple vehicles in the lane and adjacent lanes. The WIM traffic data is being collected by local and state agencies and is used for engineering, planning, enforcement, finance (estimating economic benefits), and monthly submittal to FHWA (Vandervalk-Ostrander 2009). Engineering uses of WIM data include design and management for pavement, and evaluation, assessment, and design code calibration in bridges (Hans van Loo and Aleš Žnidarič 2019; Nowak 1999; Ramesh Babu et al. 2019a).

The modern design of flexible and rigid pavements uses the Mechanistic-Empirical Pavement Design approach (MEPDG), where the physical cause of stresses is identified and calibrated with observed performance (AASHTO 2015; ARA, Inc. 2004). Performance indicators such as rutting, fatigue, transverse, and longitudinal cracking are developed by using axle load spectra using the WIM database. These damage and performance indicators are updated periodically as WIM data becomes available. These types of damage indicators for fatigue in the bridges are useful for understanding the rate of fatigue damage accumulation or fatigue life. They can also be included in the National Bridge Inventory (NBI) for each bridge to estimate the rate of damage or damage accumulated annually.

In section 7.2.2 of *Manual for Bridge Evaluation* (MBE) (AASHTO 2018), it is specified that WIM data can be used alternatively for estimating stress ranges. However, the procedure to use WIM data to estimate stress ranges is not discussed. Any analytical use of WIM data would first require checking for adequate quality.

The problem of illegal overloading of trucks goes far beyond the safety of the roads and bridges. The violators create a high competition in the transportation service market, where the operators that follow the permitted limits are disadvantaged. Most state transportation agencies are interested to know when and where the illegally overloaded trucks travel. Knowledge of the actual loads, including illegally overloaded vehicles, can help in the development of a live load model for Strength I (Iatsko 2018; Nowak 1999), Strength II limit state (Lou et al. 2018) and law-enforcement effort.

In Alabama, there are 9,785 bridges with superstructure and the average life of bridges is 44 years (NBI 2018). Of these, 45% of the bridges are in a satisfactory condition with at least some minor deterioration, and 3% of the bridges are structurally deficient with advanced section loss. There are 2,917 steel bridges and of this 72% or 2,086 are below satisfactory condition. The availability of an enormous WIM database collected by states has made it possible to analytically evaluate the damaging effect of the overweight trucks on infrastructure. Alabama Department of Transportation (ALDOT) collects traffic data on a continuous basis and uses for traffic studies and monthly submittal to FHWA. Also, there were no in-house quality control procedures to evaluate the quality of WIM data. Errors in WIM data may be due to WIM system malfunction, sensors needing recalibration, or irregular vehicle position on the sensor.

Steel bridges are more prone to fatigue cracking compared to other types of bridges (Azizinamini et al. 2013). Every passage of a truck across a bridge creates one or more stress cycles in the structural components, which results in the accumulation of fatigue damage over time. Thus, there is a need to quantify the damage produced by an individual truck and the accumulated damage resulting from many trucks.

Beginning in the 21st century, the study on the impact of overloaded vehicles by states has been topical. A study in Arizona estimated that overweight vehicles cause between $12 million to $53 million in uncompensated damages to its highway infrastructure (Straus and Semmens 2006). In Ohio, it was estimated that the impact of overloaded vehicles on bridges costs $22 million annually (Ohio Department of Transportation, 2009). In New York, estimated overloaded vehicles cause $95 million in damage to bridges and $145 million for pavements (Ghosn et al. 2015).

The current study was aimed to alleviate the above-mentioned knowledge gaps by developing a data-driven procedure that analyzes the quality of WIM traffic data, identifies permitted and illegal vehicles, and uses a mathematical model to calculate the damage caused by traffic loads. The goal is to quantify the damage produced by an individual truck and the accumulated damage resulting from many trucks. These procedures have applications in planning weight limit enforcement, budgeting, and maintenance, and they have the potential for future use in planning inspection intervals. These procedures are demonstrated using Alabama traffic data. Increasing the efficiency of design for new bridges requires a more accurate fatigue truck that envelops the fatigue load of the current traffic. To address this, a check was done of the adequacy of the current AASHTO fatigue design truck in Alabama.

The developed procedures are implemented in the form of standalone applications. This modular design enables use by any state transportation agency for processing traffic data and allows for periodic evaluation of results.

1.4 Research Objectives

The primary objective is to develop practical procedures for processing WIM and issued permit data for the evaluation of traffic-induced damage to bridges. The procedures are demonstrated using Alabama WIM data but can be used by any transportation agency.

Objectives:

- Improve the procedures used to process WIM data from the raw measurements.
- Develop a Quality Control (QC) procedure to evaluate the quality of the WIM data and routinely maintain the "health" of WIM systems.
- Develop an analytical procedure to identify legal, permitted and illegal vehicles in the traffic.
- Develop a procedure to convert the raw measurements into an index of accumulated damage for the bridges along the route. It will provide an excellent planning tool for transportation agencies for an understanding of the significance of the truck traffic along various routes and the impact of illegal and permitted overweight trucks on the bridges.
- Check the adequacy of the current AASHTO fatigue design truck for the state of Alabama.

1.5 Dissertation Outline

The research approach, developed procedures, practical examples and corresponding results are documented in this dissertation. Some of the results and procedures in this dissertation were made possible through the outcome of the project

ALDOT 930-947 Application of WIM and Permit data (Ramesh Babu et al. 2019c). This dissertation is divided into 8 chapters and 8 appendices:

Chapter 1: Introduction - This chapter is an introductory chapter providing the background, problem statement and research objectives.

Chapter 2: Traffic Monitoring Devices and Databases - In this chapter, a literature review of the state of practice on the development and practice of WIM systems are discussed. The advantages and disadvantages of many types of WIM systems in existence are also discussed. Later, the WIM systems that are used in the state of Alabama are discussed. Mainly, the WIM database and ALDOT issued permit database were used. Alabama WIM data from 12 traditional WIM stations were used. Various formats of WIM data and the conversion process and summary of available WIM data are discussed.

Chapter 3: Quality Control Procedure for WIM Traffic Data - This chapter discusses a proposed procedure to check the quality of the traffic data and detect the root cause of questionable recorded traffic data. Inconsistency in recording due to communication failure, operational problems with the sensor and drift in calibration can be interpreted from this proposed procedure. The proposed procedure consists of a completeness check, logical checks, and statistical checks. A review of the literature to identify the state-of-the-art was performed and the database of issued permits is used to establish limits for threshold parameters. A computer app “AL_WIM_QC v1.0” was developed to process the WIM data using the developed QC procedures.

Chapter 4: Bridge Live Load Models – In this chapter, traffic data that are deemed to have good quality after processing through the developed QC procedures are presented. The distribution of traffic loads among each WIM location and years are shown. Moment and shear ratios are calculated by normalizing the individual WIM vehicle moment and shears to AASHTO HL-93 design live load. Axle load distributions are shown.

Chapter 5: Identification of Issued Permit Vehicles in WIM Traffic Database - This chapter discusses a procedure to identify the permit vehicles in the WIM data. The first step is the separation of legal traffic so that the remaining file includes only permit vehicles and illegal traffic. Then WIM data without legal vehicles are sorted out using the parameters of issued permits to identify vehicles that have a permit. The remaining vehicles can be considered as illegal traffic. Two procedures – Geographic Information System (GIS) routing procedure and data-driven procedure is presented to identify issued permit vehicles in the WIM database. Issued permit data from Alabama for years 2014 and 2015 were available, so those years are used to demonstrate the procedure.

Chapter 6: Bridge Damage Accumulation - In this chapter, a procedure to quantify damage accumulated on different components of a bridge is presented. Every passage of a truck creates stress cycles in the bridge components and damage is accumulated at fatigue prone details. The procedure allows the damage induced by a single truck alone to be evaluated, or the damage from all trucks in the WIM database or for only a desired category of trucks in the traffic stream. Examples of what can be assessed include damage due to the overloaded trucks in the WIM traffic, damage due to the trucks with issued permits, and the damage caused by different FHWA vehicle classes. Comparisons of the damage at various WIM sites are possible and are reported for years 2014 and 2015. Most of the comparisons reported here are for generic steel bridges, but the procedure can be applied to a particular bridge. An example showing the application of the procedure to assess the damage specific to a particular bridge is included in Chapter 6. A computer app “AL_WIM_DAI v1.0” was developed and delivered to ALDOT for the processing of WIM data using the damage accumulation procedures.

Chapter 7: Adequacy of AASHTO Fatigue Design Truck - Fatigue loads that are experienced by bridges are addressed by fatigue limit state in *AASHTO LRFD Bridge design specifications* (AASHTO 2017). The actual fatigue damage is calculated at each WIM station from the available Alabama WIM data and a check is run to test whether the recently updated AASHTO fatigue truck envelopes the fatigue loads.

Chapter 8: Summary, Conclusions, and Recommendations - This chapter contains the summary of the overall dissertation, conclusions and discussions of future research in the area.

Chapter 2: Traffic Monitoring Devices and Databases

2.1 Introduction

WIM systems and Continuous Count Stations (CCS) are the two primary sources that collect traffic data (Hallenbeck, M. and H. Weinblatt 2004). CCS is also referred to as Automatic Traffic Recorders (ATR) in many publications. WIM systems can collect both traffic volume and load spectra, whereas the CCS can collect only traffic volume. Since the load spectra are important for bridge load assessment, the scope of this dissertation is limited to WIM systems only. Alabama traffic databases used in this dissertation are the WIM database and the issued permit database by ALDOT. The WIM database consists of traffic data from 12 WIM stations for the years 2014 to 2016 and the issued permit database consists of data for the years 2014 and 2015. The use of WIM systems in the State of Alabama dates back to as early as 1986 (Cunagin 1986).

2.2 Weigh-in-Motion Systems

Weigh-in-motion (WIM) is defined in *ASTM E1318-09 Standard Specification for Highway Weigh-In-Motion (WIM) Systems with User Requirements and Test Methods* (ASTM International 2009) as – ‘‘the process of estimating a moving vehicle’s gross weight and the portion of that weight that is carried by each wheel, axle, or axle group, or combination thereof, by measurement and analysis of dynamic vehicle tire forces.”

WIM data collection provides a powerful tool for traffic load assessment (Ramesh Babu et al. 2019a). Each traffic record collected at the WIM site includes a detailed description of the vehicle configuration (*Traffic Monitoring Guide* 2016a). The information recorded for each vehicle in the WIM database includes the exact time and date, lane and direction code, speed, Gross Vehicle Weight (GVW), individual axle loads, individual axle spacing and a class of vehicle based on FHWA Classification scheme (Cambridge Systematics, Inc. 2007). The functioning principle in WIM is the WIM sensors measure the axle loads of a vehicle once it moves on sensors through signals such as voltage, strain, and resistance. Since the WIM systems are entrenched in pavements the accuracy of WIM systems depends on pavement roughness, speed and vehicle suspension. Factors like installation, calibration, and maintenance also contribute to the accuracy of measured data.

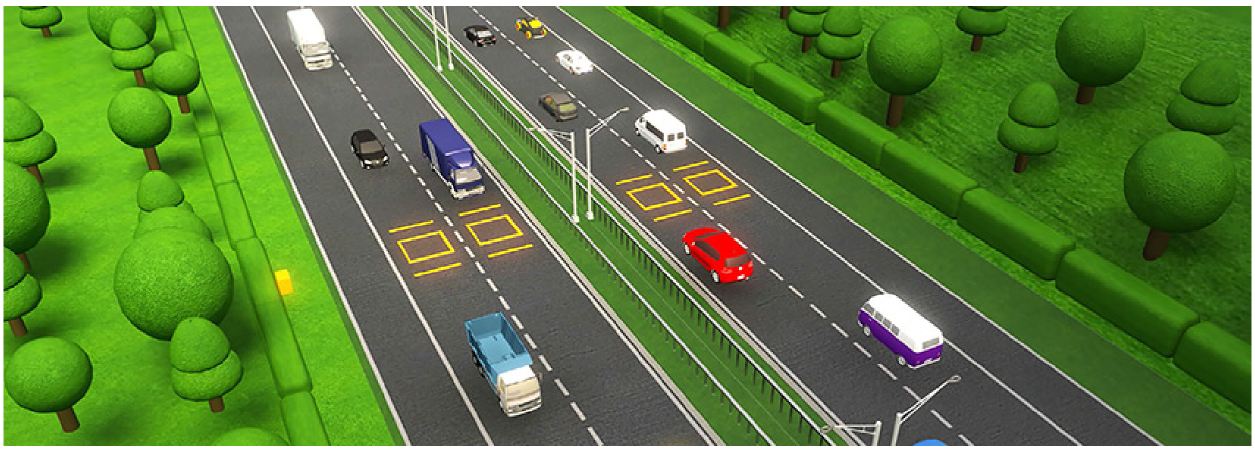


Figure 2.1: A typical WIM system on a highway. (HI-TRAC® TMU4)

One of the first WIM systems was developed in 1952 by the United States Bureau of Public Roads (predecessor of FHWA) (Norman and Hopkins 1952). It was just a reinforced concrete platform instrumented with resistance wire strain gauges. The vehicle weight was calculated manually by making use of the output from the oscilloscope attached to strain gauges. Contemporary WIM systems are very different from the sensors developed in the 1960s.

Recently, FHWA along with State DOTs have collected a substantial weigh-in-motion (WIM) database. Many states collected the data as a part of the Highway

Performance Monitoring System (HPMS) monthly submittal and various internal purposes. As of 2016, there are 1,276 WIM stations in operation in U.S and Figure 2.2 shows the locations (Steven Jessberger, personal communication, June 2017).

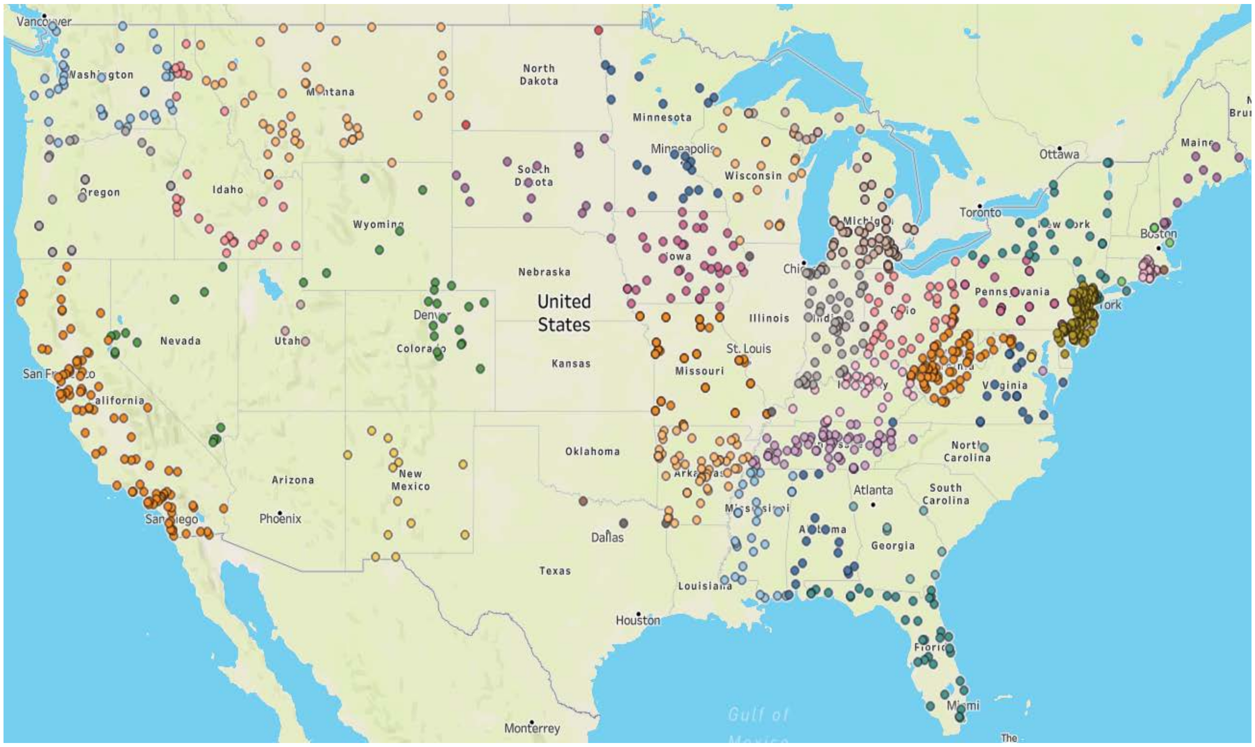


Figure 2.2: WIM station locations in the U.S.

There are a variety of weigh-in-motion technologies available for permanent or temporary traffic data collection. In general, a WIM system includes a set of weighing sensors, roadside unit and other miscellaneous sensors (Al-Qadi et al. 2016a; Hans van Loo and Aleš Žnidarič 2019). Weighing sensors are installed on the road that can consist of scales, plates, bars, strips, strain gauges or pressure transducers. Different technologies used in WIM systems are shown in Figure 2.3. Roadside units can be a data storage medium, power source, or communication device. Miscellaneous sensors are cameras or license plate readers.

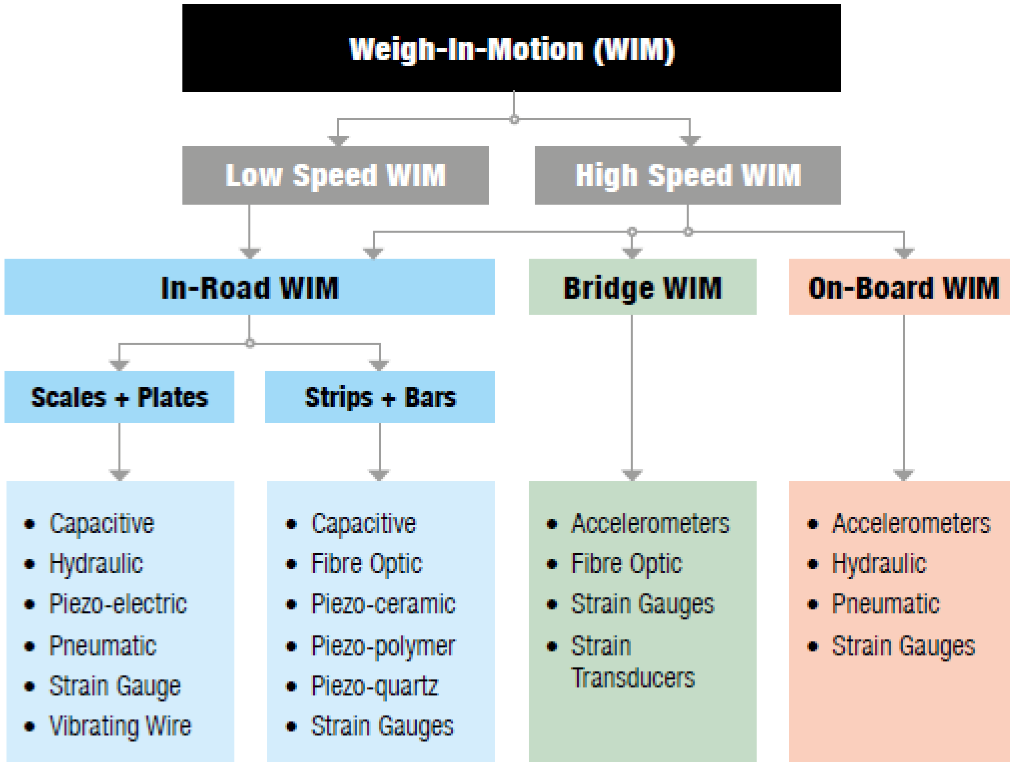


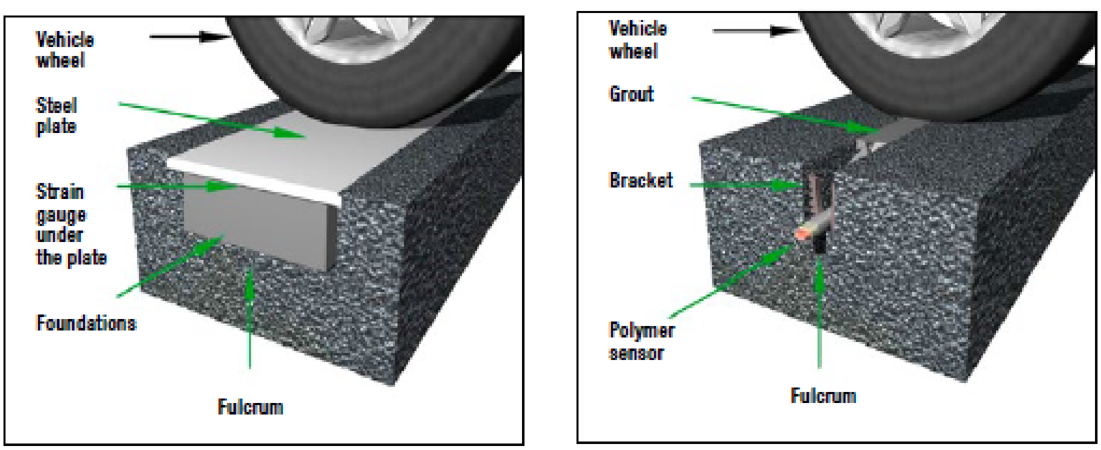
Figure 2.3: Different technologies used in WIM systems. (Hans van Loo and Aleš Žnidarič 2019)

WIM sites across the US are equipped with the following types of weigh-in-motion (WIM) systems and/or sensors: bending plate, piezoelectric sensor, single load cell, and B-WIM systems (Al-Qadi et al. 2016a; McCall and Vodrazka 1997). WIM systems are classified into Type I to Type IV systems depending on the performance requirements of the WIM systems in ASTM E1318-09 (ASTM International 2009).

Bending plate sensor works on the principle that bend of a plate can be related to axle loads as the vehicle moves due to the wheel pressure. The pressure is measured by strain gauges and axle loads are computed from the strains measured. These systems were designed for monitoring of traffic moving with speeds from 3 to 124 mph. The expected accuracy of measurement for Type I WIM sensors is 10% for GVW and 25%

for axle load and axle spacing group. A typical bending plate WIM sensor is shown in Figure 2.4 (a).

Piezoelectric WIM sensors can be classified based on the type of material: piezoceramic sensors, piezopolymer sensors, and piezo quartz sensors. A principle of this type of sensor is based on the difference in voltage due to the applied force. This type of system is only accurate in case of dynamic load, while there is a substantial error for static or slow-motion speed moving vehicles. Piezoceramic sensors and piezopolymer sensors are highly temperature-dependent and mostly used for vehicle count and classification (Al-Qadi et al. 2016b). Piezopolymer sensors are used in the regions prone to frequent freeze-and-thaw cycles because of low sensitivity to temperature fluctuations (White et al. 2006). These sensors belong to ASTM E1318 Type I WIM systems and, thus, can be used for measuring vehicle weight with sufficient accuracy of 10% for GVW and 25% for axle load and spacing. A typical piezoelectric WIM sensor is shown in Figure 2.4 (b).



(a) (b)

Figure 2.4: WIM sensor technology (a) Bending plate (b) Piezo-electric. (Hans van Loo and Aleš Žnidarič 2019)

Load cell-based WIM systems are based on the principle that load or force is converted to an electronic signal (Hans van Loo and Aleš Žnidarič 2019). The load cell WIM systems are commonly used along with inductance loops to eliminate incorrect records (Al-Qadi et al. 2016b). These systems have good accuracy, but they are above average cost.

The B-WIM systems work on the principle that the existing bridge is used as a weighing scale. The sensors, such as strain gauges and strain transducers, are attached to the bottom of the bridge, and strains induced by the vehicles are recorded. The bending moments are calculated from the mechanical properties of structural members and the recorded strains. B-WIM has a capacity to measure the traffic on a whole width of a bridge. One of the first B-Wim systems was developed in the 1970s in the U.S. (Moses 1979). The use of this type of system is limited and depends mainly upon factors such as bridge geometry, in-the-lane and multiple presence of trucks and dynamic interactions of truck and bridge.

Virtual Weigh Stations are just the high-end version of the existing type of WIM systems. Additional digital cameras and software to process the visual information in real-time are used. Some of the systems can detect vehicle license plates. Many factors cannot be controlled that affect the quality of WIM data. Factors related to site conditions such as, but not limited to, roadway geometry, pavement conditions, WIM sensors, routine maintenance, and pavement maintenance affecting the quality of WIM data (Quinley 2010b).

The basic principle and objective of WIM systems remain the same, and there are many WIM technologies available in the market and each has its advantages and disadvantages. A comparison of WIM technologies is discussed in detail in *Montana Weigh-in-Motion (WIM), and Automatic Traffic Recorder (ATR) Strategy* (Qi et al. 2013) and is shown in Figure 2.5. The advantage, disadvantages, accuracy and sensor life span comparisons are shown.

The sensor life span relatively varies from WIM technology. The bending plate technology has many advantages, and few disadvantages and sensor life span are above average when compared to other WIM technologies. Instrumented WIM technologies do not require a road closure, giving an advantage on highways where there is a high volume of traffic.

Despite the advantages of WIM technologies, a decrease in WIM research has been observed since 2000 (Pigman et al. 2012). One of the reasons is that the setting of permanent WIM devices, as well as the following service, is quite costly. Therefore, the WIM systems are usually installed on busy state roads or interstate highways.

Technology	Advantage	Disadvantage	Accuracy (GVW)	Sensor Life Span (years)
Bending Plate	• Well understood, mature technology. • High accuracy of wheel load due to whole footprint of wheel is on the plate at one time. • Resistant to environmental changes.	• Requires lane closure for installation and maintenance. • Requires other sensors to classify vehicles.	±10% for 95% of vehicles	15
Piezoelectric	• Low cost compared to other WIM systems. • Accurate vehicle classification.	• Low accuracy due to tire bridging over sensor. • Installation requires pavement cut. • Temperature sensitive (except quartz systems)	±10% for 95% of vehicles	6-10
Capacitance Mat	• Highly portable.	• Causes dynamic motion, thus, decreasing accuracy. • Highly visible to passing trucks.	±10% not better than ±660-lb (300-kg)	20
Instrumented Bridge	• Some systems do not require sensors in road surface. • Does not require lane closure for installation or maintenance. • Highly accurate vehicle classification. • Low visibility from the road.	• Requires a bridge at the WIM site.	±10% for 95% of vehicles	10
Instrumented Culvert	• Does not require lane closure for maintenance. • Low visibility from the road.	• Requires other sensors to activate system. • Requires installation of culvert at WIM site.	±10% for 95% of vehicles	10

Figure 2.5: Comparison of WIM technologies. (Austroads et al. 2010; Vandervalk-Ostrander 2009)

2.3 Alabama WIM Systems and database

There are 12 WIM sites for collection of weigh-in-motion data in Alabama, and each site is equipped with one of these systems: traditional WIM, SiWIM or Bridge WIM, and Virtual Weigh Station. The visual information collected at Virtual Weight Station at WIM site 965 (Shorter, I85) has a technology to record the following information: license plate, a picture of the vehicle, axle configuration, axle weight, time and speed of the vehicle. The bending plate systems at WIM site 965 (Shorter, I-85) are shown in Figure 2.6. However, the data is not stored and thus, not available for analysis. Therefore, the sources of data included herein are the 12 traditional WIM stations. All the traditional WIM stations except one are equipped with a permanent bending plate system consisting of two scales and inductive loops (*ASTM E1318 - 09,* 2009). However, there is a future possibility that data in real-time may be obtained from both the traditional WIM and Virtual Weigh Stations. ALDOT uses WIM systems from International Road Dynamics Inc. (IRD).

Figure 2.6: Bending Plate Systems in WIM Location 965 (Shorter, I-85).

The location of each traditional WIM station in the state of Alabama is shown in Table 2.1, along with their respective latitude and longitude coordinates. The direction of travel in all the WIM locations is North-South or Northeast-Southwest (N-S or NE-SW) and for lane of travel, 1 indicates the rightmost lane and 2, 3 and 4 are other lanes. Also, the location of the WIM stations on the map of the state of Alabama is shown in Figure 2.7. Summary of available WIM data is discussed in subsequent sections.

Table 2.1: WIM station locations in the State of Alabama.

Station code	Name	Location	Latitude	Longitude	Direction of Travel	Lane of Travel			
911	Alex City	US280 Coosa Co.	32.449819	-87.492372	N-S or NE-SW	1	2	3	4
915	Sunflower	US43 Washington Co.	31.367501	-88.032962	N-S or NE-SW	1	2	3	4
918	Bucksville	I20 Tuscaloosa Co.	33.276556	-87.099040	N-S or NE-SW	1	2	3	-
931	Athens	I65 Limestone Co.	34.844252	-86.933136	N-S or NE-SW	1	2	3	4
933	Muscle Shoals	AL157 US72 Colbert Co.	34.693714	-87.622924	N-S or NE-SW	1	2	3	4
934	Sumiton	US78 Walker Co.	33.74022	-87.033022	N-S or NE-SW	1	2	3	4
942	Pine Level	US231 Montgomery Co.	32.015582	-86.030366	N-S or NE-SW	1	2	3	4
960	Whatley	US84 Clark Co.	31.646739	-87.705445	N-S or NE-SW	1	2	-	-
961	Mobile	I65 Mobile Co.	30.889663	-88.023691	N-S or NE-SW	1	2	3	4
963	Grand Bay	I10 Mobile Co.	30.499070	-88.321659	N-S or NE-SW	1	2	3	4
964	Ozark	US231 Dothan Co.	31.370076	-85.565963	N-S or NE-SW	1	2	3	4
965	Shorter	I85	32.392695	-85.984328	N-S or NE-SW	1	2	3	4

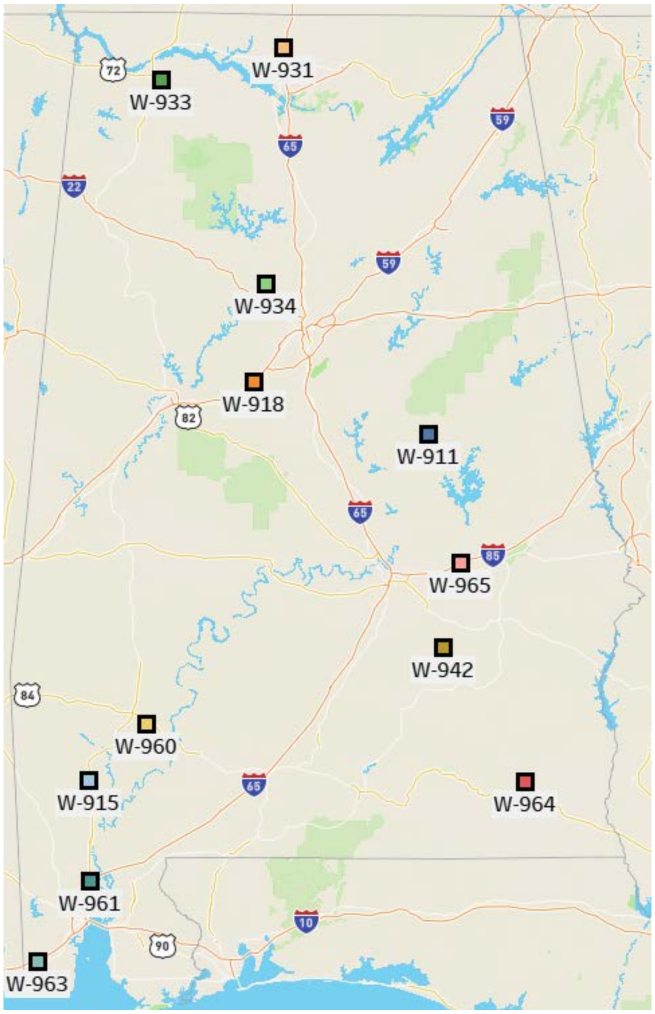


Figure 2.7: Locations of WIM stations on the map of state of Alabama.

2.3.1 Raw WIM records

Once the vehicles are recorded by WIM sensors, the records (or data) are transferred over a dial-up line using cell modems and stored in ALDOT's data storage medium. For this study, data was uploaded to Auburn University data storage medium by ALDOT personnel. All the data are in an encrypted format in the so-called "Raw" format. The raw format can be defined as data free from QC and just downloaded from the storage medium (Pelphrey, J and C. Higgins 2006).

2.3.2 WIM data formats

The traffic data collected by WIM systems are available in different data formats. For instance, in TMG there is Station Description format, Traffic Volume format, Vehicle Classification format, Weight format, and five other formats (*Traffic Monitoring Guide* 2016a). In LTPP, depending on the type of software that processes the WIM data, it has different formats (Federal Highway Administration 2015a; Office of Federal Highway Administration n.d.). LTPP Traffic Quality Control (LTQC) software has 4-card (Classification card) and 7-card (Weight card) data formats. At many WIM locations, data is processed by vendor's software that can produce data in a variety of formats (Office of Federal Highway Administration). WIM system vendor of ALDOT, IRD has an option to choose from a variety of data formats (International Road Dynamics Inc. 2017). The next section of the dissertation discusses in more detail the formats used to process WIM data in this dissertation.

2.3.3 WIM Data conversion

The WIM data contains data from 12 traditional WIM sites from Jan 2014 until December 2016. All the data were encrypted and in Raw data format. WIM vendor software "iAnalyze" was required to decrypt to the data. A license was shared for iAnalyze software. Alabama uses the FHWA 13 vehicle classification system with small modifications and class 0 as a bin to classify the records that have improperly recorded vehicles, axles greater than 13, and vehicles outside threshold limits of axle spacing and weight of classes 1-13. FHWA 13 vehicle classification system is shown in Figure 2.8. Improperly recorded vehicles can be those vehicles that are not appropriately positioned on the sensor and have other potential violation conditions (in appendix f of (International

Road Dynamics Inc. 2017)). Vendor-provided software has a built-in algorithm to flag that kind of vehicle. A detailed description of ALDOT's vehicle classification scheme based on axle configuration and weight is shown in Appendix A and Appendix B.

The WIM data containing properly recorded Class 0 and Class 4-13 is of interest in this dissertation. Classes1-3 are eliminated since these records are mostly cars and motorcycles. The flowchart in Figure 2.9 shows the step by step procedure of data conversion. The Class 0 data was decrypted using iAnalyze by selecting IRD ASCII Raw Data format. The initial step after decrypting Class 0 was to eliminate the records that had improperly recorded vehicles. The Class 4-13 data was decrypted using iAnalyze by selecting TMG 2001 Truck Weight Data format. Therefore, the remaining database contains properly recorded Class 0 vehicles and Classes 4-13. For the efficient processing of WIM data, it was decided to use two different kinds of data formats for decrypting the Raw WIM data. Special Matlab routines were used to convert data to user-friendly Matlab table format.

The summary of the WIM data received for each month for years 2014-2016 is shown in Table 2.2. The data for a few months in some of the WIM locations was missing. The summary of a number of records available in each year and WIM location is shown in Table 2.3.

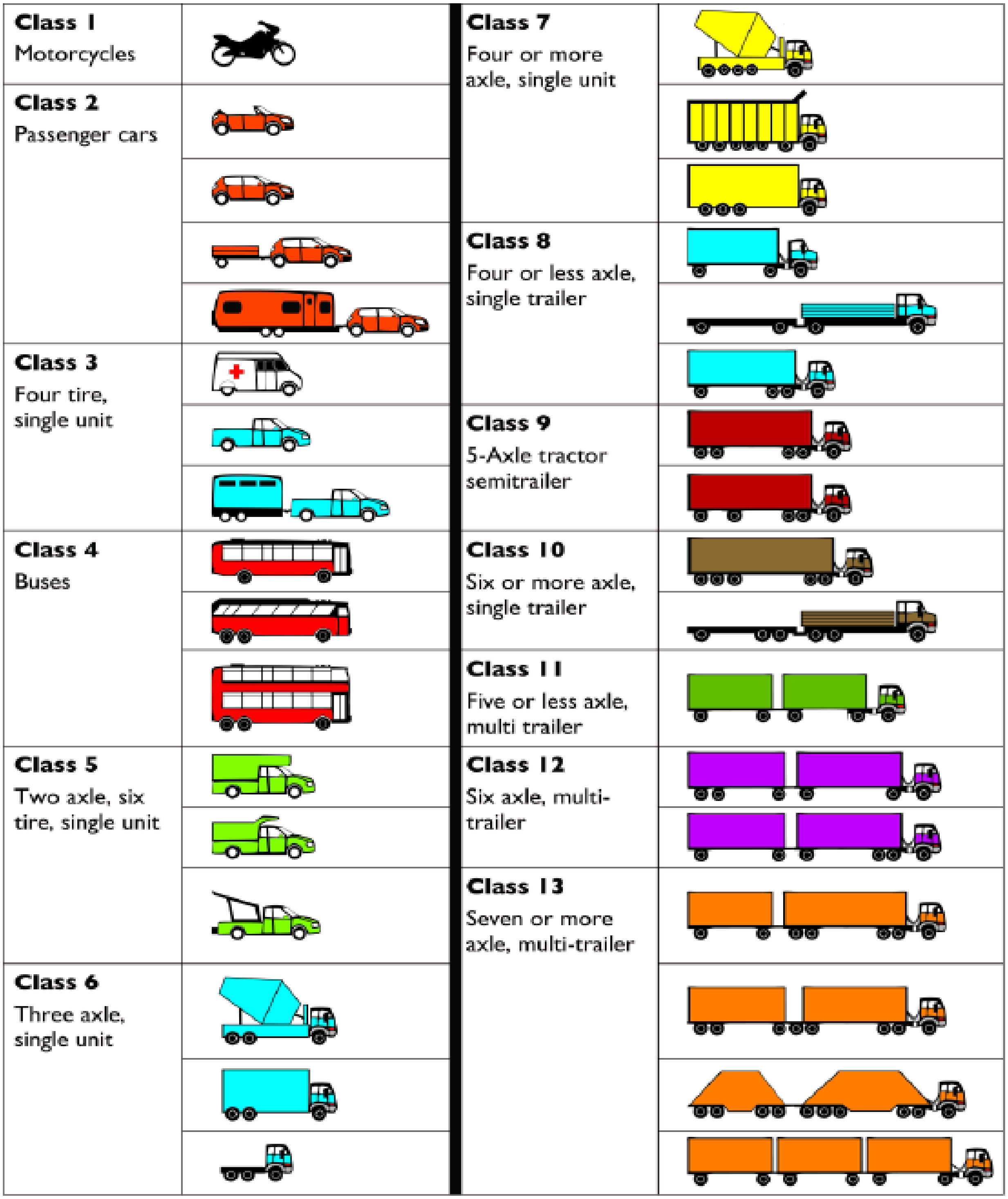


Figure 2.8: FHWA 13 Vehicle Category Classification (*Traffic Monitoring Guide* 2016a).

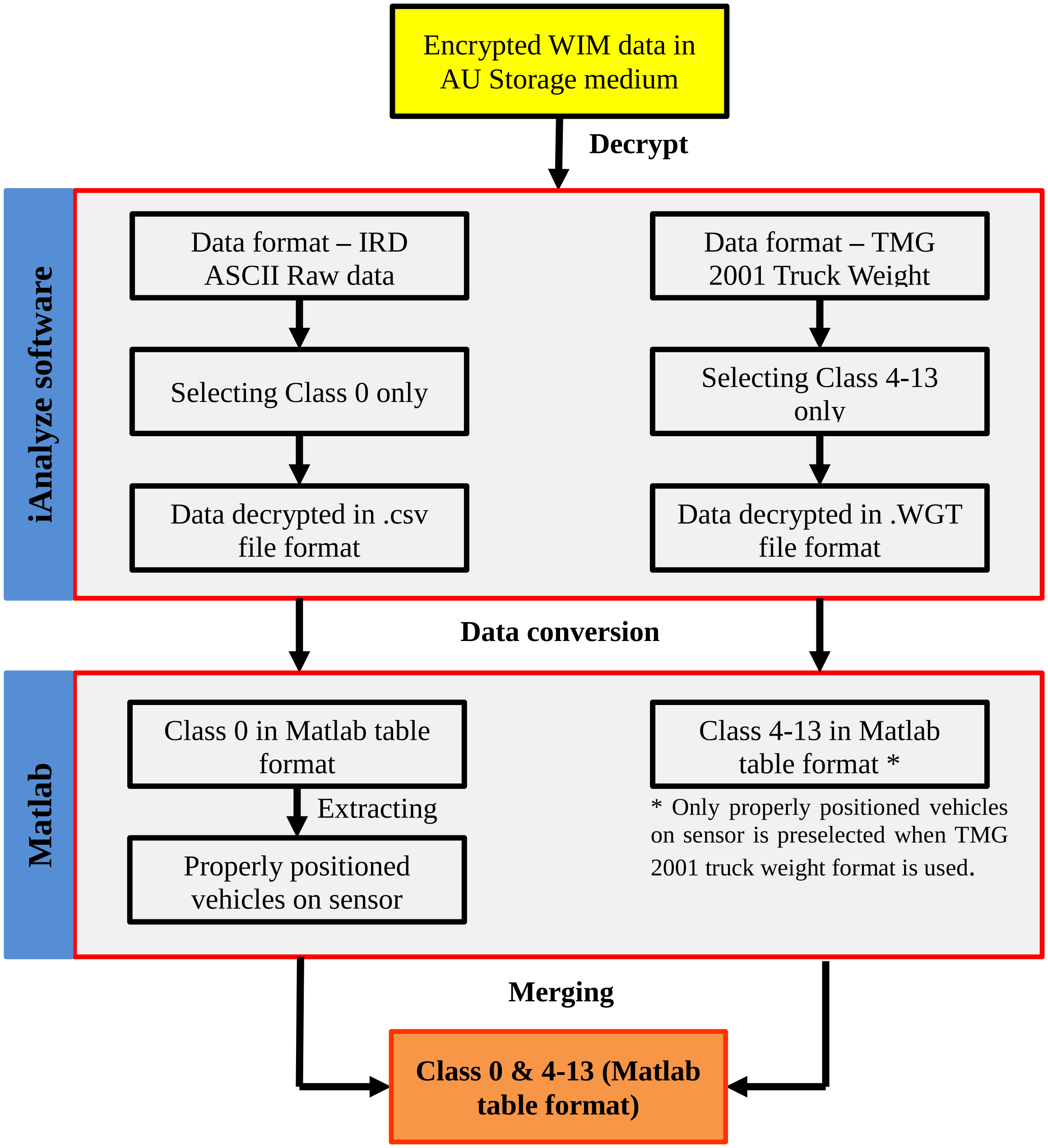


Figure 2.9: WIM Data conversion flowchart.

Table 2.2: Summary of received WIM data for years 2014-2016.

Station code	2014												2015												2016											
	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
911	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
915	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
918	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
931	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
933	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
934	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
942	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
960	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
961	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	x	x	x	x	x	x	x	x	x	x	x	x	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
963	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	x	x	x	x	x	x
964	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	x	x	x	x	x	x	x	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
965	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	x	x	x	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

Legend: ✓ – Data is present; x – Data not available

Table 2.3: Summary of number of records for years 2014-2016.

Station code	Name	Location	2014	2015	2016	Total
911	Alex City	US280 Coosa Co.	1,092,751	863,592	1,262,220	3,218,563
915	Sunflower	US43 Washington Co.	652,295	676,997	771536	2,100,828
918	Bucksville	I20 Tuscaloosa Co.	1,163,845	119,302	-	1,283,147
931	Athens	I65 Limestone Co.	3,655,980	4,024,460	4,260,765	11,941,205
933	Muscle Shoals	AL157 US72 Colbert Co.	977,580	931,817	826,870	2,736,267
934	Sumiton	US78 Walker Co.	688,388	516,595	649,083	1,854,066
942	Pine Level	US231 Montgomery Co.	1,262,375	1,074,754	1,145,221	3,482,350
960	Whatley	US84 Clark Co.	521,484	509,497	555,826	1,586,807
961	Mobile	I65 Mobile Co.	2,136,008	191,853	1,821,562	4,149,423
963	Grand Bay	I10 Mobile Co.	6,088,720	7,503,103	3406241	16,998,064
964	Ozark	US231 Dothan Co.	1,217,687	278,020	1149733	2,645,440
965	Shorter	I85	2,441,637	1757523	2,593,647	6,792,807
TOTAL						58,788,967

2.4 ALDOT Issued Permit Database

The Maintenance Bureau of Alabama DOT issues about 500-600 permits per day. About 200 of them are permits for overweight. The issued permit data for the years 2014 and 2015 was made available as part of the project (Ramesh Babu et al. 2019c). The annual reports are in the form of tables and they include permit ID, the validity of the permit, original and final destination, authorized roads, description and FHWA class of vehicle, GVW, axle load and axle spacing. The data also includes information about the size of the vehicle (e.g., over width or length). Annual permits are issued, but each trip accomplished within the annual permit is also listed as a separate row in the database. The total number of issued permits is 123,602 for 2014 and 122,539 for 2015.

To process the database, special Matlab routines were developed. A more detailed discussion of the permit database is provided in Chapter 4 of the dissertation.

2.5 Summary

In this chapter, a literature review of the state of practice on the development and use of WIM systems is discussed. The WIM systems that are used in the state of Alabama are described. Also, the databases that were used in this dissertation are described. Mainly, the WIM database and ALDOT issued permit database were used. WIM data from 12 traditional WIM stations were available. The various formats of WIM data and the conversion process and summary of available WIM data are discussed.

Chapter 3: Quality Control Procedure for WIM Traffic Data

3.1 Introduction

Traffic-induced loadings are one of the primary factors affecting the service and fatigue life of bridges and pavements. The major source of information about traffic loading is the weigh-in-motion (WIM) database. However, poor quality of traffic data may lead to misinterpretation and incorrect estimation of the load effects. The errors may occur due to WIM system malfunction, out-of-calibration or irregular vehicle position on the sensor. If the error in recorded WIM data is not recognized and eliminated at the earlier stage, the quality of the entire data accumulated is questionable. Therefore, it is essential to use a Quality Control (QC) procedure.

3.2 Literature Review

So far, WIM records were used by ALDOT's Weight Enforcement team to screen weight violators and by the Transportation Planning Division for the statistical analysis of the traffic mix. Accurate traffic data from WIM stations are also needed for accurate bridge evaluation, design and fatigue analysis. For instance, a significant number of incorrectly recorded vehicles that create high load effects may lead to overconservative design or unrealistically high estimated fatigue damage. Therefore, the development of the detailed quality control procedure was an essential step in this study.

Two types of error occur in long-term WIM data collection: random (occurring individually), and systematic errors (occurring frequently and affecting some records). The errors are usually associated with the WIM system malfunctioning, mis-recording, non-typical vehicle configuration, vehicle position on the sensor, and other causes. However, there are calibration practices employed by states to meet the functional performance of the WIM systems. If the functional performance is not met, there is a need for calibration of WIM systems.

Table 3.1 shows the functional performance requirements for WIM systems. There are a number of case studies related to traffic data quality checks that are analyzed and employed by many state agencies in the US (Ramesh Babu et al. 2019b) (Turochy et al. 2015), (Elkins and Higgins 2008), (Southgate 1990), (Ramachandran et al. 2011), (Qu et al. 1997), (Quinley 2010b), (Kulicki et al. 2015), Sivakumar et al. (2011), etc.). However, there is no documented state-specific quality control (QC) procedure employed by Alabama DOT.

Table 3.1: Functional Performance Requirements for WIM Systems. (ASTM International 2009)

Function	Tolerance for 95 % Compliance				
	Type I	Type II	Type III	Type IV	
				Value $lb (kg)[B]	±lb (kg)
Wheel Load	±25 %		±20 %	5000 (2300)	300 (100)
Axle Load	±20 %	±30 %	±15 %	12 000 (5400)	500 (200)
Axle-Group Load	±15 %	±20 %	±10 %	25 000 (11 300)	1200 (500)
Gross-Vehicle Weight	±10 %	±15 %	±6 %	60 000 (27 200)	2500 (1100)
Speed	±1 mph (2 km/h)				
Axle-Spacing and Wheelbase	±0.5 ft (0.15 m)				

[A] 95 % of the respective data items produced by the WIM system must be within the tolerance.
[B] Lower values are not usually a concern in enforcement.

Thus, a comprehensive quality control (QC) procedure is of interest in this dissertation to ensure adequate quality of the data. A review of the literature to identify the state-of-the-art quality control and an assurance was performed to develop an effective QC procedure. The literature review was focused on various QC programs developed to monitor the quality of traffic data collected by WIM systems. As many states gather traffic data as part of FHWA's Highway Policy Management System (HPMS) submittal and traffic inputs for AASHTO's Mechanistic-Empirical Pavement Design Guide (MEPDG), the quality of the traffic data should meet minimum requirements prescribed in the respective guides (Quinley 2010a). For the benefit of each state Department of Transportation (DOT), FHWA and AASHTO have documents of guidelines to achieve maximum performance of their investment in traffic monitoring programs and equipment. Three important documents recommend the guidelines for WIM data QC: Traffic Monitoring Guide (TMG), AASHTO Guidelines for Traffic Data Programs (TDP) and HPMS field Manual (Vandervalk-Ostrander 2009). Apart from this, there is FHWA's Long-Term Pavement Performance Program (LTPP) that collects traffic data as a part of the pavement study (Office of Federal Highway Administration n.d.). Literature findings of QC checks in national standards and of common practices are discussed in the following sections.

3.2.1 Quality Control checks in national standards

The traffic data collected by WIM systems are available in different data formats. For instance, in TMG there is Station Description format, Traffic Volume format, Vehicle Classification format, Weight format and five other formats (*Traffic Monitoring Guide (2001)* . In LTPP, depending on the type of software that processes the WIM data it has different formats. LTPP Traffic Quality Control (LTQC) software has 4-card (Classification card) and 7-card (Weight card) data formats. At many WIM locations data is processed by vendor's software that can produce data in a variety of formats (Federal Highway Administration 2015a; Office of Federal Highway Administration n.d.). A brief compendium of literature findings of Quality Control checks in national standards is shown in Table 3.2. TMG contains a compendium of QC criteria used by various states and recommends the checks used in Traffic Monitoring Analysis System (TMAS). TMAS includes QC checks for Station, Classification, Volume and Weight data format (*Traffic*

Monitoring Guide 2001). Before data is updated in HPMS, it is filtered through TMAS checks (Office of Highway Policy Information 2017). TDP recommends minimum validation criteria for weight, classification, and vehicle count data. LTPP has the most rigorous quality control checks. Traffic data stored in a database known as the LTPP national information management system (IMS) and should comply with QC checks mentioned in IMS manual (Federal Highway Administration 2015a; Office of Federal Highway Administration n.d.).

3.2.2 Quality Control checks of common practices

As the use of WIM data is beyond just submitting data to HPMS and MEPDG, the quality control of the traffic data can be tailored according to customer needs. Some states have developed their QC programs to meet customer needs and achieve maximum performance (Vandervalk-Ostrander 2009). The need for adequate quality of traffic data in bridge design has been studied extensively in NCHRP report 683 (Sivakumar et al. 2008). The QC checks developed in state DOT’s QC programs, research papers, NCHRP reports, and journal articles are reviewed and listed in Table 3.3 to Table 3.6.

Table 3.2: Literature findings of Quality Control checks in national standards.

Reference	Description	Findings
Traffic monitoring analysis system (TMAS) (in Appendix J of TMG)	TMAS provides online data submitting capabilities to State traffic offices to submit data to FHWA.	Provides QC checks on Station, Classification, volume & Weight data format.
Traffic Monitoring Guide (TMG)	Provides guidance to state highway agencies related to equipment used in traffic monitoring programs consisting of procedures, standards and policies.	Contains a compendium of QC criteria used by various states. TMG does not dictate any QC checks but recommends check that are given in TMAS.
Highway performance monitoring system (HPMS)	HPMS is a national transportation information system. It consists of scope, condition and performance of National Highways.	WIM data is subjected TMAS checks before it is updated in the system.
AASHTHO guidelines for traffic data programs (TDP)	TDP provides guidelines to improve the quality of the traffic information for all kinds of traffic data programs.	TDP recommends minimum validation criteria for weight, classification and vehicle count data. Also, examples of data validation standards are listed.
Long-Term Pavement Performance (LTPP)	LTPP consists of database of traffic, environment, monitoring and materials of each test section of a pavement under study.	LTPP has most rigorous quality control checks. Stored in a database known as LTPP national information management system and should comply with QC checks mentioned in IMS manual.

Table 3.3: Literature findings of Quality Control checks of common practices (1994-2002).

Reference	Description	Findings
WIM data Quality Assurance (1994)	WIM data editing and the quality checks used on weight and classification cards is discussed.	Class 9 histogram check of the unloaded peak between 28- 34 kips and loaded peak between 74-84 kips was proposed.
States' Successful Practices Weigh-In-Motion Handbook (1997)	Purpose was to provide practical advice to the users of WIM technology.	The QC checks of LTPP's traffic quality control software (LTAS), FHWA's vehicle travel information system software (VRTIS) and Caltrans quality assurance programs is reported.
Quality Assurance of Weigh-In-Motion data (2000)	A personal spreadsheet program was developed to determine quality assurance and develop the firm guidelines of the WIM data.	A concept of "steering axle load per foot of spacing between front axle and following axle" combined with 12 kip limit was proposed. Used to check calibration of sensors.
Traffic Data Editing Procedures: Traffic Data Quality (2002)	Pooled fund study by 14 states and FHWA to document traffic data screening methods. Also, to develop "rule base" traffic data screening method.	120 QC checks rule list was developed for volume, weight, classification cards.

Table 3.4. Literature findings of Quality Control checks of common practices (2004-2007).

Reference	Description	Findings
Quality Control Procedures for Weigh-in-Motion Data (2004)	The accuracy of Indiana WIM data is improved by proposing more effective QC procedure.	The DMAIC model (tool in Six Sigma project model) is used to improve the accuracy of the data.
Equipment for Collecting Traffic Load Data (NCHRP 509) (2004)	Information useful for state agencies for selecting right equipment for traffic data collection is summarized.	The common checks used in quality assurance programs are reported.
Calibration of LRFR Live Load Factors for Oregon State-Owned Bridges using WIM Data (2006)	A rating factor for new Load and Resistance factor rating method is developed.	A set of QC checks for a dataset that is used in developed rating factors is proposed.
Enhancement of bridge live loads using weigh-in-motion data (2007)	This paper discusses the possible enhancement of bridge live load factors using WIM data.	The filtering criteria to eliminate unrealistic data used on New York WIM data is presented.
Quality Control Procedures for Archived Operations Traffic Data: Synthesis of Practice and Recommendations (2007)	The report recommends the QC procedures that be adopted or customized for system specific quality control issues.	QC criteria are established for archived data. Archived data from 9 states were surveyed.

Table 3.5: Literature findings of Quality Control checks of common practices (2008-2013).

Reference	Description	Findings
High Speed WIM System Calibration Practices (2008)	This report synthesizes the WIM system calibration practices in USA.	Reports the states that calibrate WIM systems using traffic stream data QC.
WIM Data Analyst's Manual (2010)	Recommends procedures to be utilized by state DOT's WIM data analyst to perform validation and QC checks of traffic data.	Extensive QC checks on WIM data and tasks of WIM Data analyst is reported.
NCDOT Quality Control Methods for Weigh-in-Motion Data (2011)	Development of North Carolina DOT WIM QC procedures.	Rule list of QC for Weight cards and Class cards which are like LTPP QC is used. Weight range peaks for Class 4-13 is established.
Protocols for Collecting and Using Traffic Data in Bridge Design (NCHRP 683) (2011)	Protocols of collection, processing and use of WIM data in Bridge design is addressed	Filters for data scrubbing and QC checks on reminder of scrubbed data is proposed.
Cleaning Weigh-in-Motion Data: Techniques and Recommendations (2011)	Data from 5 countries in Europe is used for simulating truck traffic. But as an initial step data cleaning is done.	Different filtering criteria is used for each country in the study. Unique way to detect ghost and split axles was developed.
Validation of TMG Traffic Data Check Algorithms (2013)	This paper validates the TMG data check algorithms using WIM data from LTPP WIM sites.	The paper concludes that TMG checks are still valid. Framework to check suitability of the data is presented in case of exceptional patterns.

Table 3.6: Literature findings of Quality Control checks of common practices (2014-2019).

Reference	Description	Findings
Calibration of AASHTO LRFD Concrete Bridge Design Specifications for Serviceability (2014)	This report presents the calibration of service limit states in concrete bridges.	A set of filtering criteria on WIM data for eliminating questionable records. Additional criteria for using data in fatigue limit state calibration is also used.
Development of Alabama Traffic Factors for use in Mechanistic-Empirical Pavement Design (2015)	Traffic inputs for MEPDG and QC for Alabama WIM data was developed.	A set of QC measures consisting of threshold and rational checks are proposed.
Bridge Live Load Models in U.S. and Europe (2018)	Study of the effects of traffic data in U.S and Europe using recent WIM data.	A comparison of QC checks in the USA and Europe is made.
Comparison of Bridge Live Loads in US and Europe (2019)	Statistical parameters for U.S WIM data and European WIM are developed in this paper.	An extensive WIM data set from U.S. is run through QC checks to develop statistical parameters that can be used for calibration.

3.3 Quality Control Algorithm

Based on the literature review, many QC checks were recommended for a data format (For Ex. TMG or LTPP formats). For bridge live load modeling, vehicle weight, configuration, traffic volume, and timestamp are essential. For example, TMG's Weight format or LTAS's 7-card format includes axle weight and configuration information but is limited to FHWA vehicle classes 4-13 (*Traffic Monitoring Guide* 2001). If the state uses another classification system than FHWA (13 vehicle category classification) the vehicles that do not meet the FHWA limits are categorized into an “unclassified” group, such as Class 0.

In the proposed procedure it is recommended to obtain data in a so-called “RAW format” rather than pre-processed. RAW format can be defined as data free from QC and just downloaded from the storage medium (Pelphrey, J and C. Higgins 2006). Many WIM system vendor’s software provides an alternative to extract this data in RAW format rather than one of the TMG or LTPP’s formats. For example, the Alabama WIM data is classified into FHWA Classes 0-13. However, when extracted using TMG’s weight data format, only Classes 4-13 are obtained. It matters what kind of data Class 0 contains (in this case, the records that have improper positioning of vehicles on sensors, axles greater than 14, and vehicles outside threshold limits of axle spacing and weight of Classes 1-13 are placed in Class 0). The vehicles outside threshold limits that are in Class 0 is of importance (Iatsko 2018).

Selection and sequential order of the quality control criteria are critical to ensuring only questionable records are eliminated. The proposed QC procedure consists of 3 sets of checks: completeness, logical and statistical. The proposed procedure is shown in the form of a flowchart in Figure 3.1. The logical checks are based on threshold limits. Selection of threshold limits is a critical factor so that the correct data is not eliminated. Some of the threshold limits were based on the limits recommended in previous studies (Ramesh Babu, A., et al. 2018; Sivakumar et al. 2008; Wagdy G. Wassef et al. 2014). However, after examination of filtered records, it was observed some records which appear to be real are being eliminated just because they are out of limits. So, the threshold limits are set by analyzing the source where accurate vehicle configuration information is available, such as issued permit data and police citation data. This is important because the vehicles exist which are not typical vehicle in traffic but are real and should not be eliminated.

The statistical checks are applied to the accumulated data set rather than individual vehicle record. Most of the statistical checks are applied to vehicle Class 9, as it is the most common vehicle class in the traffic stream. Minnesota DOT first developed checks on Class 9 vehicles and then used them in Long-Term Pavement Performance (LTPP) (Hellenbeck 1994). As of now, the developed statistics are used by many national (Office of Federal Highway Administration n.d.) and state agencies (*Traffic Monitoring Guide* 2001) as a way to maintain "health" of the WIM systems. LTTP's annual Standard Data Release can be used to compare statistical check limits as the LTTP WIM sites are regularly maintained ("LTPP InfoPave - Standard Data Release" n.d.). Statistical check limits reported in the literature were consistent in many cases. Therefore, standard limits are used in the proposed procedure.

3.3.1 Completeness check

This first set of checks is used to identify missing data in the accumulated database. The algorithm can be developed to check whether the data is present in each hour of the day or just in each day of a month based on user preference. The hour of the data with no records can be flagged, and the possible cause of missing data can be investigated. Probable causes of missing data may be communication failures or system malfunction.

3.3.2 Logical checks

These checks were developed based on the common practice reported in the literature. All the filtering criteria in the logical checks are applied to each individual vehicle record. The individual records containing obvious errors, such as but not limited to, empty rows, zero-weight vehicles are eliminated. The proposed set of filtering criteria in logical checks is shown in Table 3.7. Each criterion is categorized by the type and a unique error code is given. Each filtering criteria has a threshold limit(s) and if the records are outside the limit(s), then the records are eliminated from further analysis. The criterion that can be modified depending upon availability of issued permit data is indicated. The filtering criteria such as error code 3.c, 3.d, 3.e and 3.j can be modified by analyzing permit database. In case the issued permit database is not available, then the threshold limits mentioned in Table 3.7 can be used. Usually, there is a limitation on the number of axles that can be recorded by WIM sensors. That limit is part of the logical check filtering criteria.

3.3.3 Statistical checks

The statistical checks are applied to identify the anomalies in the traffic patterns and possible reasons causing the anomaly. Checks can be applied on accumulated data on a monthly basis to detect the possible malfunctions and their reasons, such as communication failures, operational problems with the sensor and drift in the calibration of the systems.

The flowchart of statistical checks is highlighted in Figure 3.1. First, in the vehicle class distribution check the percentages of vehicles distributed among the classes in the accumulated database is compared with historical data. If it is done on a monthly basis then it should be compared with the corresponding month of previous years if that data is available. As an alternative, it can also be compared with the data from ATR or any vehicle classification equipment that is available. If the statistics are not matching with the historical data, a possible cause would be operational problems in the sensor.

After this check, the rest of the checks are on vehicle Class 9. If the large percentage of trucks in vehicle Class 9 are above 100 kips, then there might be a problem with the sensor. It is hard to say what percentage of trucks should be above 100 kips because the truck statistics vary by region. One possible way to check is to compare the percentage to that for a nearby WIM station. Alternatively, the Class 9 records of all the WIM stations in the state can be plotted separately on the same Cumulative Distribution Function (CDF) plot for comparison. One such example is shown in the next section.

In the next statistical check, the gross vehicle weight (GVW) histogram, front axle weight histogram and CDF plot of tandem axle spacing are plotted, and the check fails if the peaks are out of limits. A possible reason would be a system out of calibration. In GVW histogram check, a 4-kip bin width histogram is plotted, if the data is correct, then there is an unloaded peak between 28 and 36 kips and a loaded peak between 72 and 80 kips. Then in front axle weight histogram check, a 1-kip bin width histogram is plotted, and usually, one peak between 8 and 12 kips is seen. In CDF plot of tandem axle spacing, a spacing of 4 feet is most common.

The last set of the statistical checks is based on tandem axle load and it is the only check that determines if the considered dataset should be eliminated from further analysis. This set of check was developed in Turochy et al. (Turochy et al. 2015a). A 2-kip bin width histogram of tandem axle load is plotted and compared to historical data of the corresponding month of previous years. The first peak should be between 14 and

16 kips and the second peak between 32 and 38 kips. If the peak is shifted out of these limits, it can be detected. A correlation analysis is performed by comparing to historical data, from the corresponding month of the previous year. If the Pearson correlation is less than 0.85, then it is statistically significant and is treated as failed (Everitt 2011). It is almost impossible that the considered dataset fails all the checks before the tandem axle check and can pass only tandem axle checks.

Table 3.7: Logical checks filtering criteria.

Type	Error code	Filtering criteria	Threshold limits
WIM station description	**1.a**	FIPs state code	≠ (01)*
	1.b	Station ID	Alabama WIM station ID*
	1.c	Direction of travel code	≠ (0-9)
	1.d	Lane of travel	≠ (0-9)
Period of travel	**2.a**	Invalid year	Null or irrespective year
	2.b	Invalid month	≠ (1-12)
	2.c	Invalid day	≠ (1-31)
	2.d	Invalid hour	≠ (0-23)
Vehicle configuration	**3.a**	Records with zero GVW	= 0
	3.b	Records with zero axle spacing	= 0
	3.c	Number of axle (Naxle)	≠ (2-14)**
	3.d	Axle weights (Waxle)	≠ (1 kips -70 kips)**
	3.e	Axle spacing (Saxle)	≠ (3.33 ft - 180 ft)**
	3.f	Number of axles = Number of axle spaces + 1	Naxle ≠ Saxle +1
	3.g	Number of axles = Number of axle weights	Naxle ≠ # of Waxle
	3.h	Sum of axle weights +/- 10% of GVW	> or < than 10% of GVW
	3.i	Minimum first axle spacing	< 6ft
	3.j	Length of the vehicle (L)	> 220 ft**
	3.k	Invalid vehicle class	≠ (0-13)*
Duplicates	**4.a**	Identical records (rows)	If duplicated

*Depending upon state

**Can be modified based on issued permit data

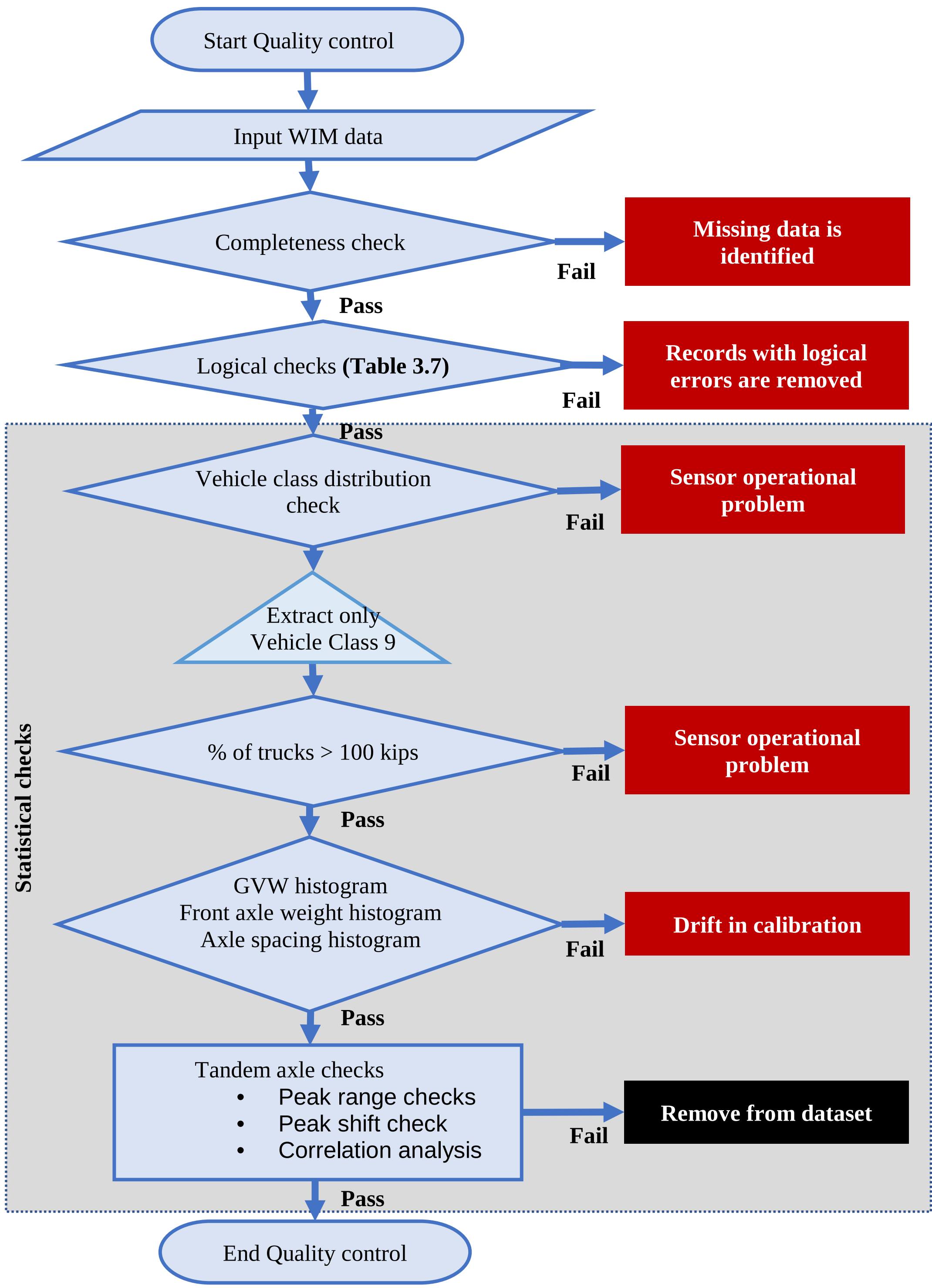


Figure 3.1: Flowchart of a Quality Control procedure.

3.4 Probability paper

Probability paper is a special scale for the statistical interpretation of data (Kulicki et al. 2015). Probability paper is defined in *Probability Concepts in Engineering Planning and Design: Basic principles* textbook (Ang and Tang 1975) as "Graph papers for plotting observed experimental data and their corresponding cumulative frequencies (or probabilities)." In the probability paper, usually, the x-axis is on a regular scale, and the y-axis is in units of probability for appropriate distribution. So, a different probability paper corresponds to a different probability distribution. It is constructed using a transformed probability scale such that a linear graph is seen between the cumulative probabilities of the corresponding distribution and the corresponding values of the variate. A vertical scale is redefined so that a regular Cumulative Distribution Function (CDF) (S-shaped) will plot as a straight line for the corresponding distribution and probability of that distribution. If a normal distribution is plotted on normal probability paper, it is seen as a straight line (Nowak and Collins 2012).

Normal probability paper is the most commonly used type of probability paper. Throughout this dissertation, the variable(s) (data such as GVW, axle loads) is shown as CDF on normal probability plot (hereafter, the CDF on normal probability paper is referred to as CDF). Construction of a normal probability paper is discussed in textbooks like *Reliability of Structures* (Nowak and Collins 2012). A graphical representation of straightening from the regular CDF scale to scale on normal probability paper is shown in Figure 3.2. The values in regular CDF are transformed into normal probability scale values. The origin of the x-axis is at the mean value.

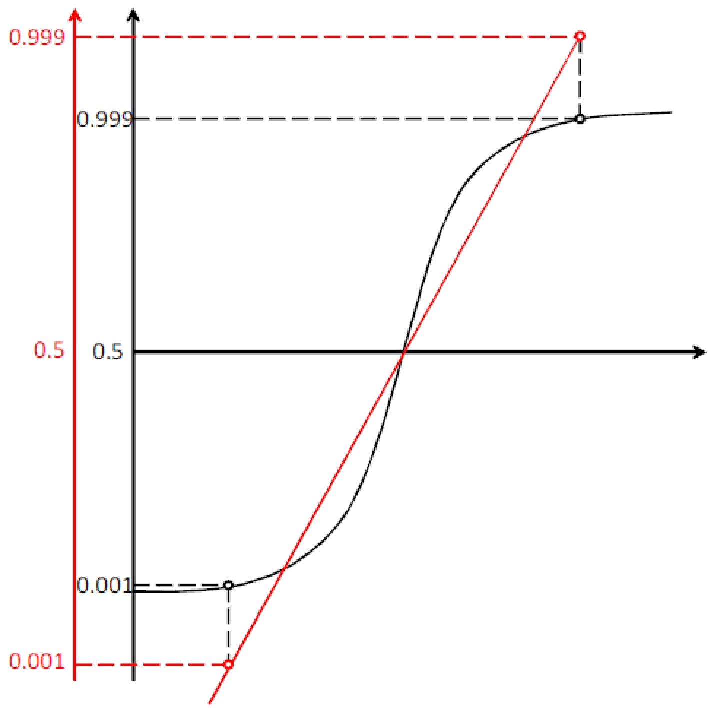


Figure 3.2: Graphical representation from regular CDF scale to scale on normal probability paper. (Rakoczy 2011)

The construction of normal probability is explained here.

1. From the N values, the individual data values (x) are arranged in ascending order first.
2. Each x_i value is associated with a cumulative probability p_i, as shown in Equation 3.1.

$$p_i = \frac{i}{N+1} \tag{3.1}$$

3. If the commercial normal probability paper is available then the x_i and p_i values are plotted directly on normal probability scale, as shown in Figure 3.3. In commercial normal probability paper that is available in analytical software like Matlab (*MATLAB 2018a* n.d.), only the input data, which is variables are needed and a built-in function in software can plot on normal probability paper (For example, plots in Figure 3.4 and Figure 3.5).

4. If the commercial normal probability paper is not available than the inverse CDF of standard normal variable z_i is calculated, as shown in Figure 3.3. Then the x_i and z_i values are plotted using a standard linear graph.

Using the normal probability scale plot in Figure 3.3, it is easy to interpret the probability of being exceeded. For instance, 0.841 on the vertical axis represents 84.1% of the variable(s) (such as GVW, axle loads) are below or 15.9% values are above that intersecting value on the x-axis. Also, on standard normal variable scale, 0.841 corresponds to 1, and it is interpreted as one standard deviation above the mean value.

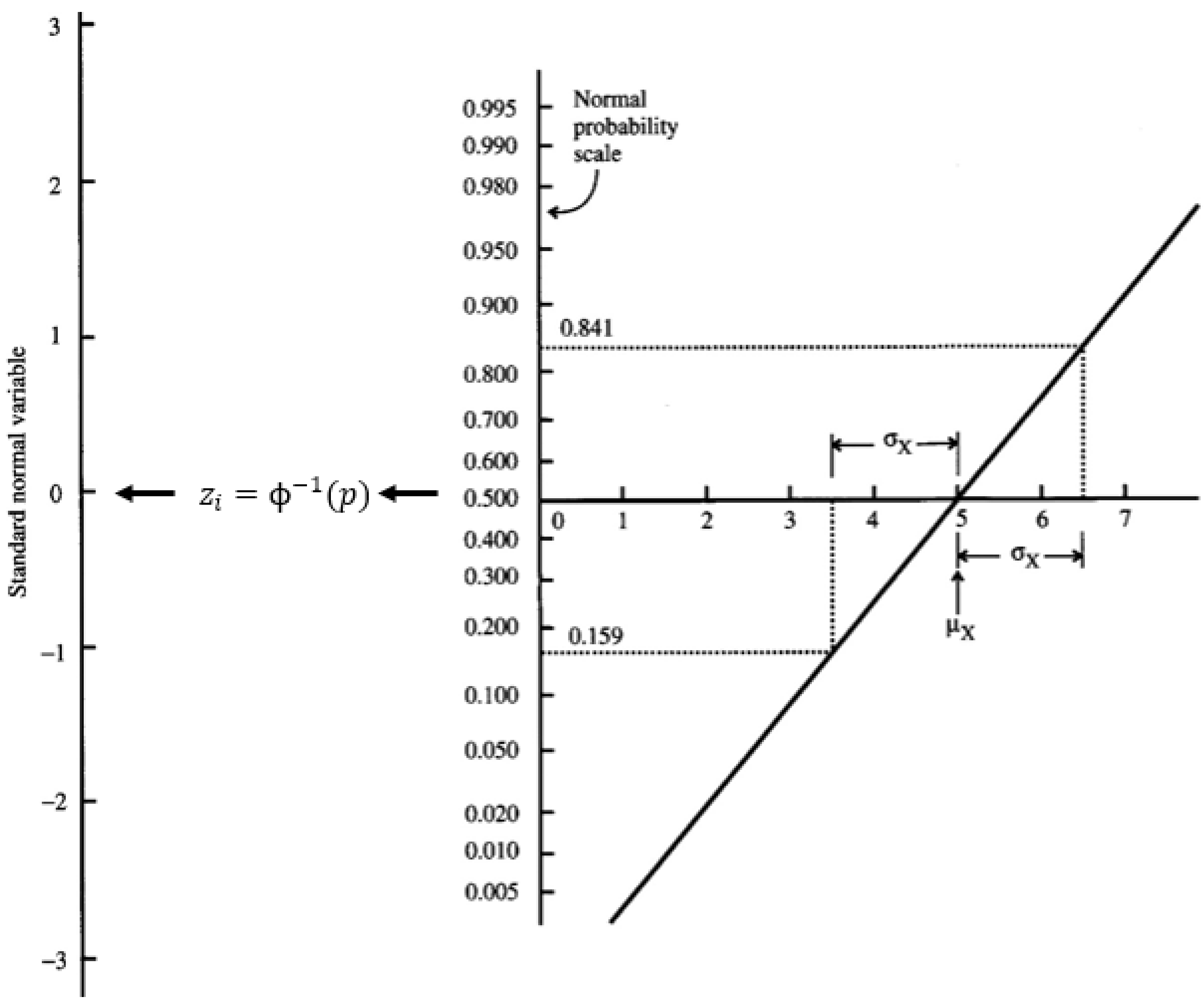


Figure 3.3: Normal probability paper. (Nowak and Collins 2012)

3.5 Quality Control Results

The WIM data from 12 WIM stations for the years 2014-2016 and issued permit data for the years 2014-2015 was obtained from Alabama DOT. The WIM data was obtained in Raw format, and it was encrypted initially. The data was decrypted and processed in Matlab table format as outlined in Figure 2.9. Therefore, the remaining database contains properly recorded Class 0 and Classes 4-13 vehicles. Classes 1-3 are eliminated since these records are mostly cars and motorcycles. Technical difficulties were encountered in the decryption of the data from WIM stations 915 and 965 using iAnalyze, so that data was not used further in this dissertation.

The remaining database is run through the proposed QC procedure shown in Figure 3.1. It was decided to use the threshold limits mentioned in Table 3.7 after the analysis of issued permit data of Alabama. Also, all vehicles of GVW less than 20 kips were eliminated in remaining classes before the QC procedure was performed due to limitations in the processing capacity. The upper tail of the traffic data is of importance in bridge live load modeling, so the elimination of these lightweight vehicles (lower tail) is not significant. The Class 0 contained some vehicles which were just outside the threshold limits of GVW of Classes 1-13 but were still correct records that contributed to the end of the upper tail.

To illustrate the importance of Class 0, the plot of “Class 0 & 4 -13” and “Class 4-13” is shown in Figure 3.4 for WIM station 911 of the year 2014 as an example after the data was processed through proposed QC procedure. It is clearly seen how the upper tail of the traffic data changes when Class 0 is included.

The completeness check indicated inconsistency in recording, missing some days of recording. The results of the completeness check is shown in Table 3.8. The total number of days in each month of the availability of the WIM data is listed. The summary of available WIM data before and after logical check filtering is shown in Table 3.9. Summary of available WIM data sorted based on FHWA vehicle class is shown in Appendix H.

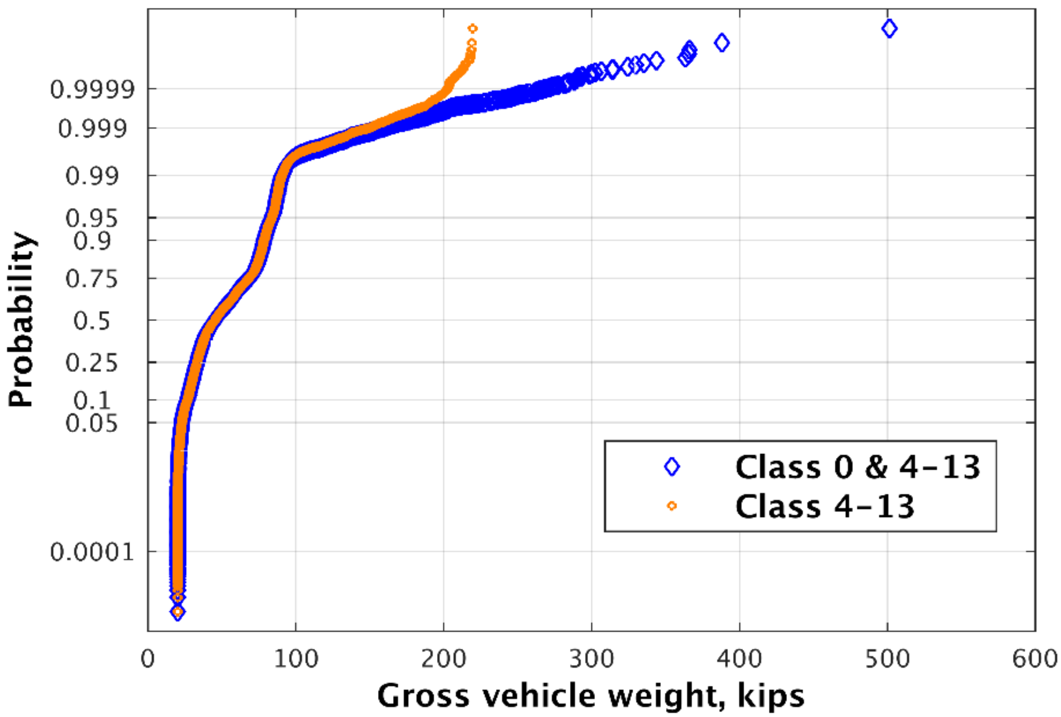


Figure 3.4: Cumulative Distribution Function of gross vehicle weight of "Class 0 & 4 -13" and "Class 4-13" for WIM station 911, year 2014.

Table 3.8: Completeness check of all WIM stations in state of Alabama for years 2014 to 2016.

WIM Location	Year	Month											
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
911	2014	30	28	30	29	30	29	30	30	29	30	29	30
	2015	30	27	30	29	30	29	30	30	29	30	29	30
	2016	30	28	30	29	30	29	30	30	29	30	29	30
931	2014	30	27	30	30	31	30	31	30	30	31	30	31
	2015	30	27	31	29	30	30	31	31	30	31	30	31
	2016	31	29	31	29	31	30	31	31	29	30	30	31
933	2014	30	27	30	29	31	30	31	30	30	30	29	30
	2015	30	27	31	30	30	30	31	30	29	30	30	31
	2016	30	28	30	29	30	29	30	31	29	15	29	30
934	2014	24	21	30	27	18	29	30	30	29	30	14	29
	2015	30	27	30	29	30	29	30	31	29	27	29	30
	2016	30	28	30	29	30	5	31	31	29	30	30	30
942	2014	28	23	30	29	30	30	30	30	29	30	29	30
	2015	30	27	31	29	30	29	30	30	29	30	30	31
	2016	30	28	30	29	30	29	30	30	29	30	29	30
960	2014	30	27	31	29	30	29	30	30	29	30	12	30
	2015	8	27	27	30	30	29	30	30	29	30	29	30
	2016	30	23	30	29	30	29	30	30	29	30	29	30
961	2014	31	28	30	30	30	29	30	30	29	7	0	0
	2015	0	0	0	0	0	0	0	0	0	0	0	30
	2016	30	28	25	29	30	29	30	2	0	13	29	30
964	2014	29	27	31	29	30	29	30	30	29	30	29	30
	2015	30	27	21	0	0	0	0	0	0	0	0	30
	2016	27	28	30	29	31	29	30	30	29	3	1	30

Table 3.9. Number of records in Alabama weigh-in-motion database.

WIM station	Before logical checks			After logical checks			Records eliminated by logical check (%)	Total records after logical check
	2014	2015	2016	2014	2015	2016		
911 (US280)	399,514	378,359	430,793	357,839	350,492	361,684	11%	1,070,015
918 (I20)	1,002,049	116,661	N/A*	743,287	33,739	N/A*	31%	777,026
931 (I65)	1,730,840	1,941,813	1,985,302	1,584,096	1,511,419	1,350,318	21%	4,445,833
933 (AL157)	524,116	456,251	382,906	427,474	395,916	350,085	14%	1,173,475
934 (US78)	180,634	113,529	148,192	169,251	112,105	134,012	6%	415,368
942 (US231)	806,305	707,222	733,913	786,932	688,980	713,436	3%	2,189,348
960 (US84)	317,502	292,802	313,075	305,353	282,213	301,933	4%	889,499
961 (I65)	1,298,636	115,589	1,150,865	829,946	115,338	1,101,595	20%	2,046,879
963 (I10)	7,936,829	8,481,882	3,669,721	4,972,917	5,284,795	2,283,603	38%	12,541,315
964 (US231)	660,591	148,357	607,532	642,038	135,810	587,857	4%	1,365,705

*Data not available

In Table 3.10, the example of error vehicles detected by QC procedure and in Table 3.11, the percentage of records eliminated by each logical check filtering criteria is shown. The minimum, maximum and average percentage of eliminated records of all the considered WIM stations combined is shown. Most of the records are eliminated by logical error code 3.d, 3.e and 3.i. Data from WIM station 918 was eliminated after logical checks as it contained many erroneous records and some corrupted files were found during decryption.

Table 3.10. Example of error vehicles detected by QC procedure.

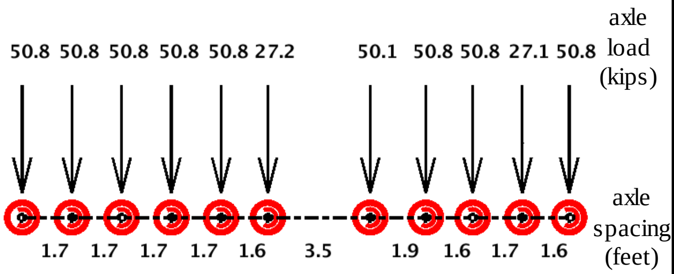
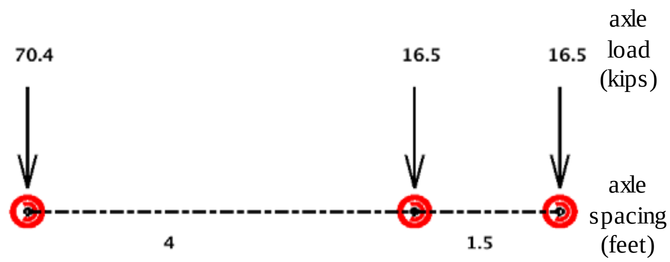
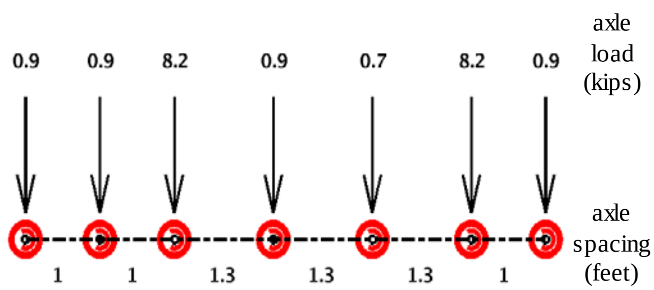
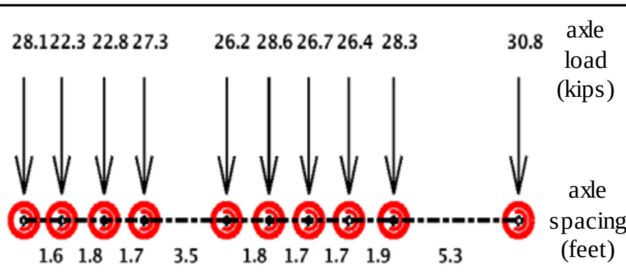
Error vehicle detected by QC procedure	Recorded by WIM sensor			Logical check error code
	GVW	No. of axles	Vehicle Class	
	513 kips	11	0	3.e, 3.i
	103.5 kips	3	6	3.d, 3.e, 3.i
	27.8 kips	7	10	3.d, 3.e, 3.h, 3.i
	269 kips	10	0	3.e, 3.i

Table 3.11: Summary of percentage of vehicles eliminated by each logical check filtering criteria.

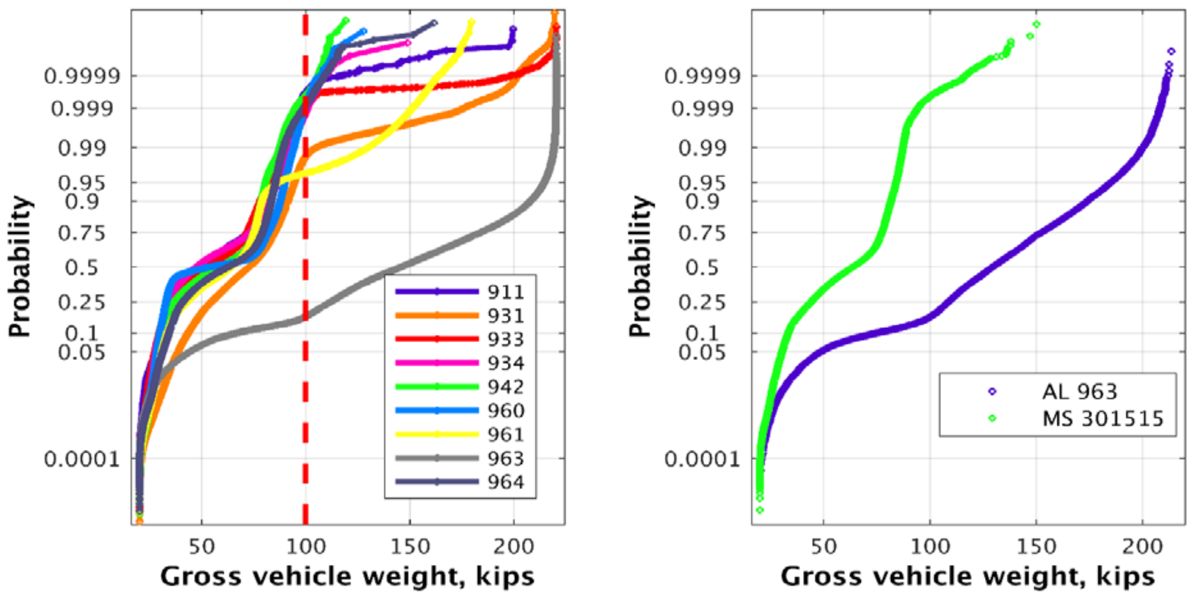
Error code	Filtering criteria	Threshold limits	Records eliminated (%)		
			Min.	Max.	Avg.
1.a	FIPs state code	Null or invalid state code*	0.0	0.0	0.0
1.b	Station ID	Null or invalid station ID*	0.0	0.0	0.0
1.c	Direction of travel code	≠ (0-9)	0.0	0.0	0.0
1.d	Lane of travel	≠ (0-9)	0.0	0.0	0.0
2.a	Invalid year	Null or irrespective year	0.0	0.0	0.0
2.b	Invalid month	≠ (1-12)	0.0	0.0	0.0
2.c	Invalid day	≠ (1-31)	0.0	0.0	0.0
2.d	Invalid hour	≠ (0-23)	0.0	0.0	0.0
3.a	Records with zero GVW	= 0	0.0	0.0	0.0
3.b	Records with zero axle spacing	= 0	0.0	0.0	0.0
3.c	Number of axle (Naxle)	≠ (2-22)**	0.0	0.0	0.0
3.d	Axle weights (Waxle)	≠ (1 kips -70 kips)**	1.0	61.5	10.3
3.e	Axle spacing (Saxle)	≠ (3.33 ft - 180 ft)**	6.0	93.0	55.9
3.f	Number of axles = Number of axle spaces + 1	Naxle ≠ Saxle +1	0.0	11.6	1.2
3.g	Number of axles = Number of axle weights	Naxle ≠ # of Waxle	0.0	0.0	0.0
3.h	Sum of axle weights +/- 10% of GVW	> or < than 10% of GVW	0.0	12.5	1.3
3.i	Minimum first axle spacing	< 6ft	0.7	67.3	30.6
3.j	Length of the vehicle (L)	> 220 ft**	0.0	7.6	0.8
3.k	Invalid vehicle class	≠ (1-13)*	0.0	0.0	0.0
4.a	Identical records (rows)	If duplicated	0.0	0.1	0.0

The statistical checks are performed on the remainder of the data. The results of some of the statistical checks are shown from Figure 3.5 to Figure 3.7. In Figure 3.5 (a), the CDF plot of Class 9 vehicles for all the WIM stations for the year 2014 are shown. This plot shows that WIM station 963 has a different traffic pattern than other WIM stations within the state. The WIM station 963 is located at 5.0 miles east of the Mississippi border on I-10 in Grand Bay.

Further investigation was made to validate the data by comparing it with Mississippi WIM station 301515 located at 3.7 miles west of Alabama state border on I-

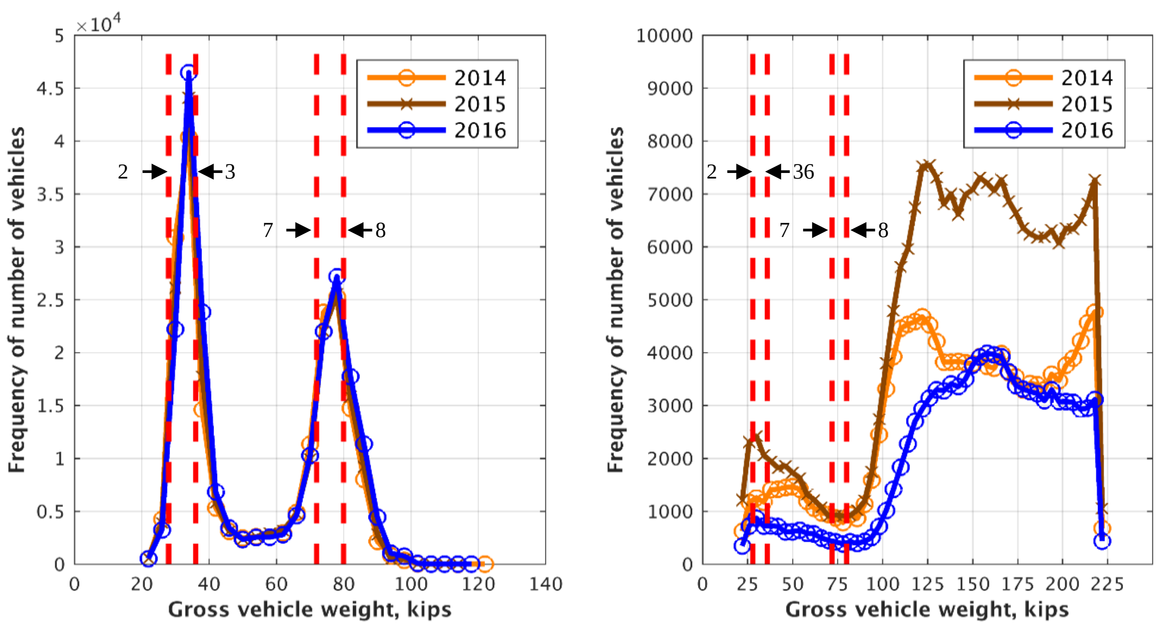
10 for the year 2013. The WIM station AL 963 and MS 301515 are on the same line on I-10 at 30.2 miles apart. Figure 3.5 (b) is a CDF plot to show the discrepancy in GVW of Class 9 trucks between station 963 of Alabama with station 301515 of Mississippi. The results of statistical checks for GVW of Class 9 are shown for some of the representative locations. In Figure 3.6 (a) for WIM station 915, the peaks are within limits, whereas in Figure 3.6 (b) for WIM station 963 it is clearly seen to be out of the limits. To determine whether the data set should be removed from further analysis, the tandem axle load check is performed. In Figure 3.7 (a), for WIM station 931 the peaks are within limits and calculated Pearson correlation coefficients shown in the top right corner of the figure are above 0.85. However, in Figure 3.7 (b) for WIM station 963 there are no peaks and calculated correlation coefficients are less than 0.85. The correlation coefficients of less than 0.85 are highlighted.

In summary, the data from WIM stations 918 and 963 were eliminated entirely. The statistical check and comparison of data with Mississippi data for WIM station 963 indicated the poor quality of the data. For other WIM stations, the data retained after logical check filtering is treated as good quality WIM data. The data was processed by the proposed QC procedure and shared with Alabama DOT. Alabama DOT confirmed the existence of a problem with WIM stations 918 and 963.



(a) (b)

Figure 3.5. Cumulative Distribution Function plot of Class 9 vehicles of (a) all WIM stations in Alabama for the year 2014 (b) WIM station AL 963 of Alabama and MS 301515 of Mississippi for year 2013.



(a (b

Figure 3.6. Histogram of gross vehicle weight of Class 9 vehicles for (a) WIM station 915 (b) WIM station 963.

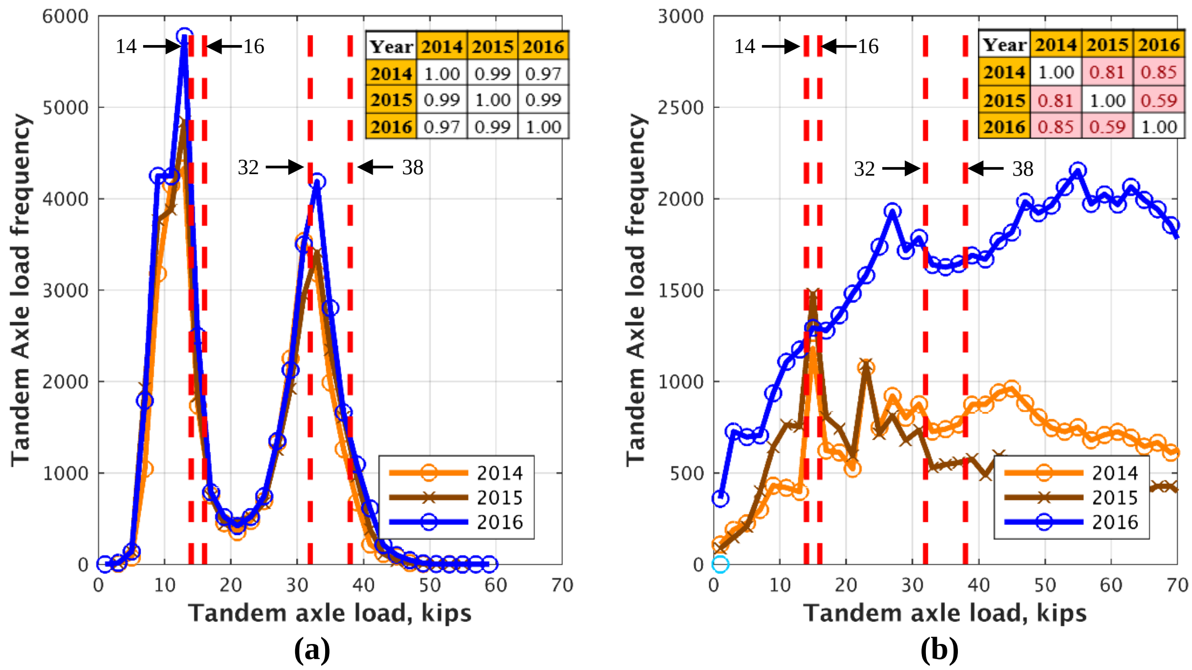


Figure 3.7. Histogram and Pearson correlation coefficients of tandem axle checks for January for (a) WIM station 931 (b) WIM station 963.

3.6 Computer App for Evaluating the Quality of WIM Data – AL_WIM_QC v1.0

A computer app was developed so that the developed QC procedure can be implemented on a routine basis. The computer app can process the data collected over a one-month period. The user can visualize the results of the completeness check, logical checks, and statistical checks. Also, the results are saved in the form of images and can be assessed or shared by the user. A more detailed discussion of the AL_WIM_QC v1.0 computer app is provided in Appendix E.

3.7 Summary

This chapter discusses a proposed procedure to check the quality of the traffic data and detect the root cause of questionable recorded traffic data. Inconsistency in recording due to communication failure, operational problems with the sensor and drift in calibration can be interpreted from this developed procedure. The proposed procedure consists of a completeness check, logical checks, and statistical checks. A review of the literature to identify the state-of-the-art was performed and a database of issued permits was used to establish limits for threshold parameters. The proposed QC procedure can verify the accuracy of unusual vehicle configurations that are categorized as “unclassified.” Some of the results are shown for the WIM database for years 2014 to 2016. This procedure can be useful in monitoring the health of WIM systems by performing the checks periodically on accumulated data.

Chapter 4: Bridge Live Load Models

4.1 Introduction

From the bridge engineer's perspective, the knowledge of bridge live loads is important for the design and evaluation of bridges. The safety reserve in the new generation of design codes is provided by means of load and resistance factors through a reliability based code calibration process (Nowak 1999). For the purposes of calibration, the statistical parameters of load and resistance are required (Nowak and Pipinato 2016; Nowak 1993, 1995; Nowak and Iatsko 2017). There is a need to evaluate bridge live loads on a continuous basis and update the statistical parameters if necessary. This chapter discusses bridge live loads in the state of Alabama that are processed and filtered through the procedures in Chapter 3.

For all the available WIM data, the load effects, moment and shear for spans ranging from 30 ft to 200 ft span are calculated for each traffic record and normalized to HL-93 loading. Results are shown in the form of Cumulative Distribution Function (CDF) plots on normal probability paper for better interpretation of results.

As discussed in Chapter 3, the state of Alabama has 12 WIM stations. The WIM data for only the years 2014 to 2016 is used in this section for the state of Alabama. The WIM data from all the stations are filtered through the QC procedure, as discussed in Chapter 3. The variation in bridge live loads among each WIM station is presented. Also, the variation of live loads among each year is shown. The results are shown in the form of cumulative distribution function (CDF) plots.

4.1.1 Gross vehicle weight

The CDF's of GVW for the years 2014, 2015 and 2016 for the considered WIM locations are shown from Figure 4.1 to Figure 4.3 The variation in traffic from each year at the WIM stations can be seen. The highest GVW of 608 kips is found in WIM station 933. The highest GVW varies from year to year. In all the WIM locations, there is variation in traffic load only in the top 0.1 % of the records.

In Figure 4.4 and Figure 4.5, the variation of GVW with respect to years for two of the WIM locations with heavy GVW is shown. There is a consistency in variation for different years in location 931. In location 933, there is an inconsistency only in the year 2016.

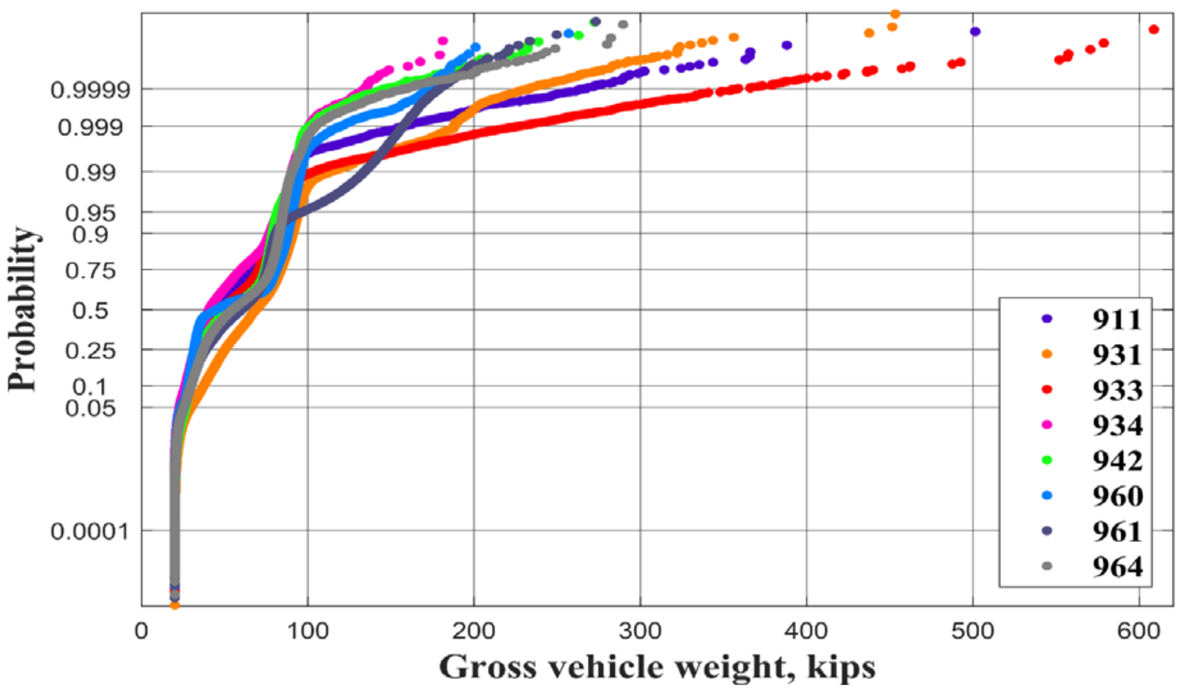


Figure 4.1: Cumulative Distribution Function plot for GVW of the WIM records of all WIM stations for the year 2014.

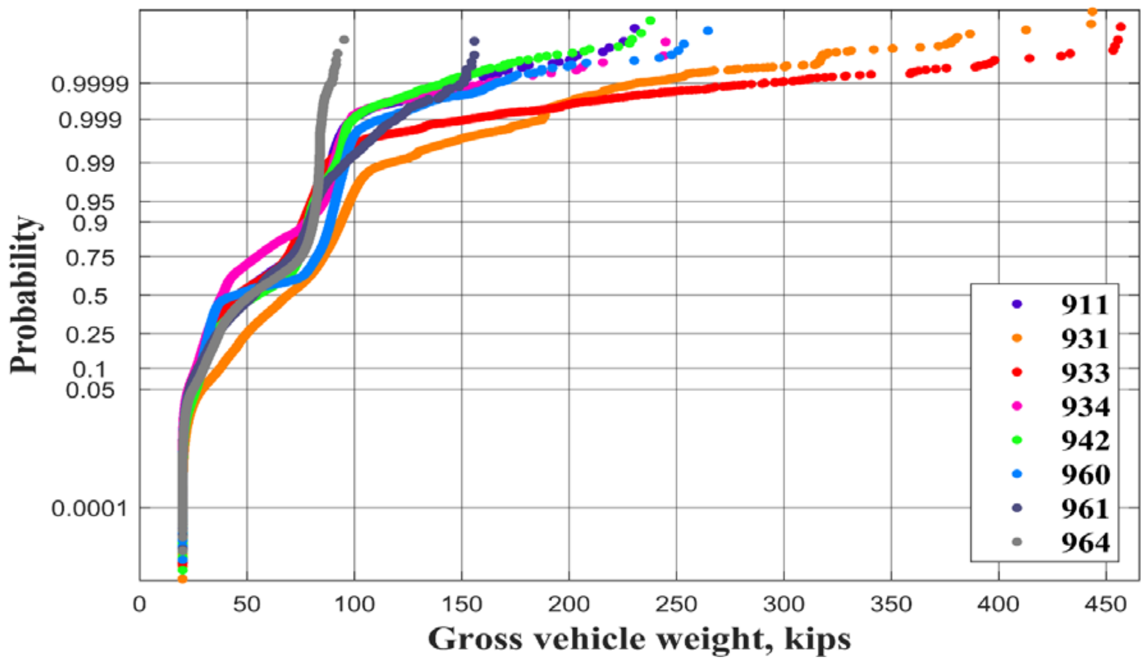


Figure 4.2. Cumulative Distribution Function plot for GVW of the WIM records of all WIM stations for the year 2015.

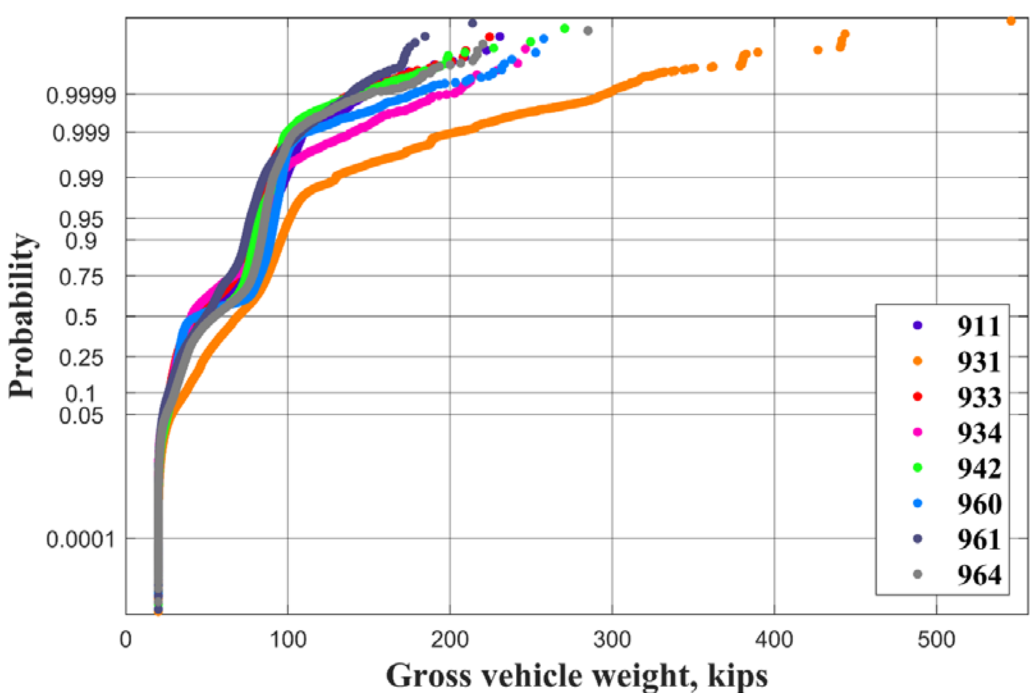


Figure 4.3. Cumulative Distribution Function plot for GVW of the WIM records of all WIM stations for the year 2016

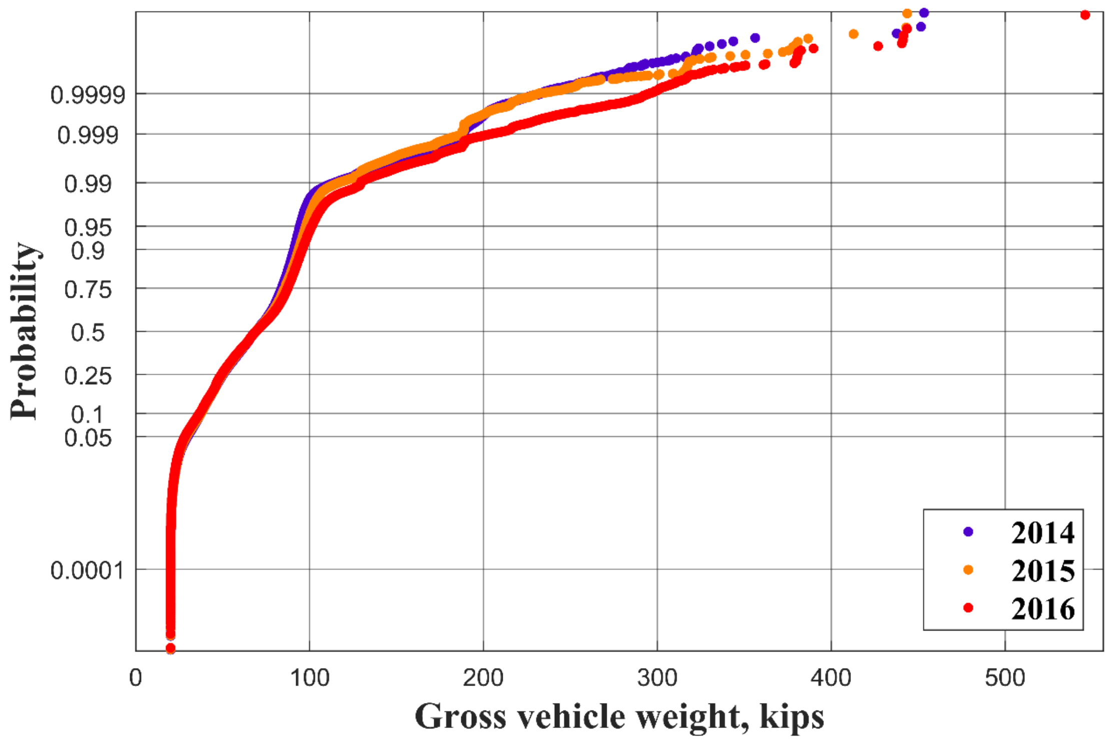


Figure 4.4: Cumulative Distribution Function plot for GVW for WIM location 931 for years 2014-2016.

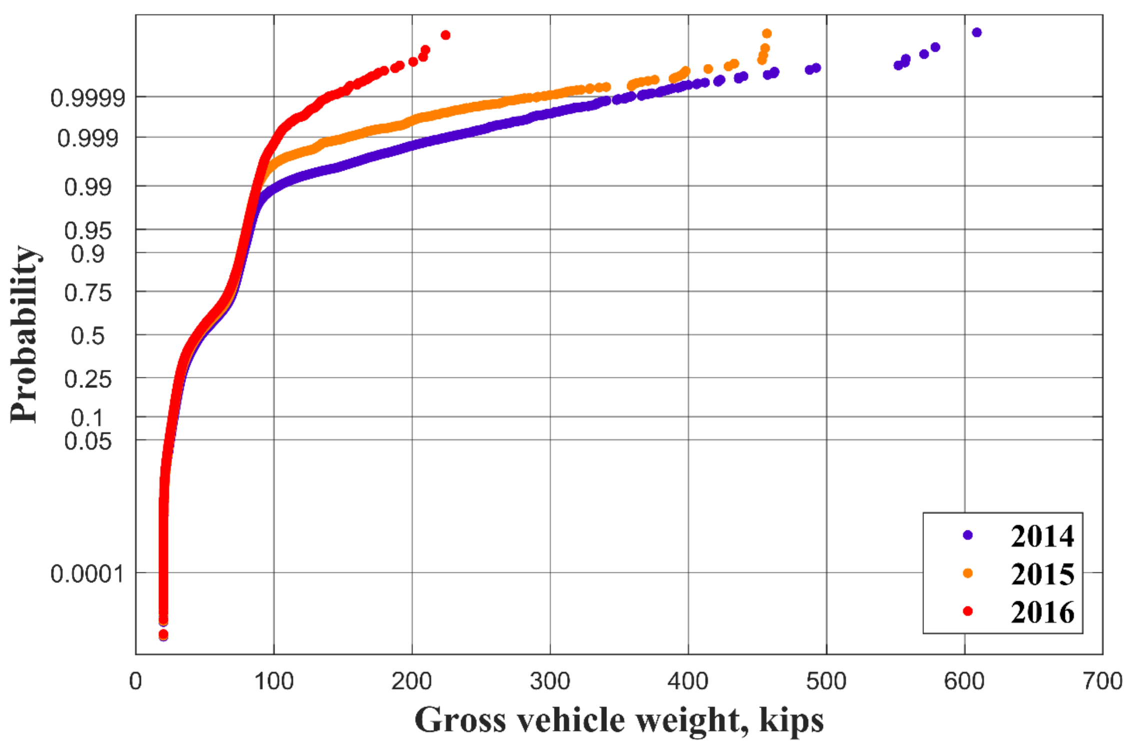


Figure 4.5: Cumulative Distribution Function plot for GVW for WIM location 933 for years 2014-2016.

4.1.2 Live load effects

This section discusses the results of the live load effects, i.e., moment and shear forces. For each of the WIM record in the database, the vehicle was run on an influence line of simply supported spans of 30 ft, 60 ft, 90 ft, 120 ft, and 200 ft. The maximum moment and shear for each span length are calculated for each WIM record. For better interpretation of how the load effects relate to design loads, the obtained moment and shear forces were divided by HL-93 moment and shear forces for respective span lengths. The HL-93 design load cases are shown in Figure 4.6.

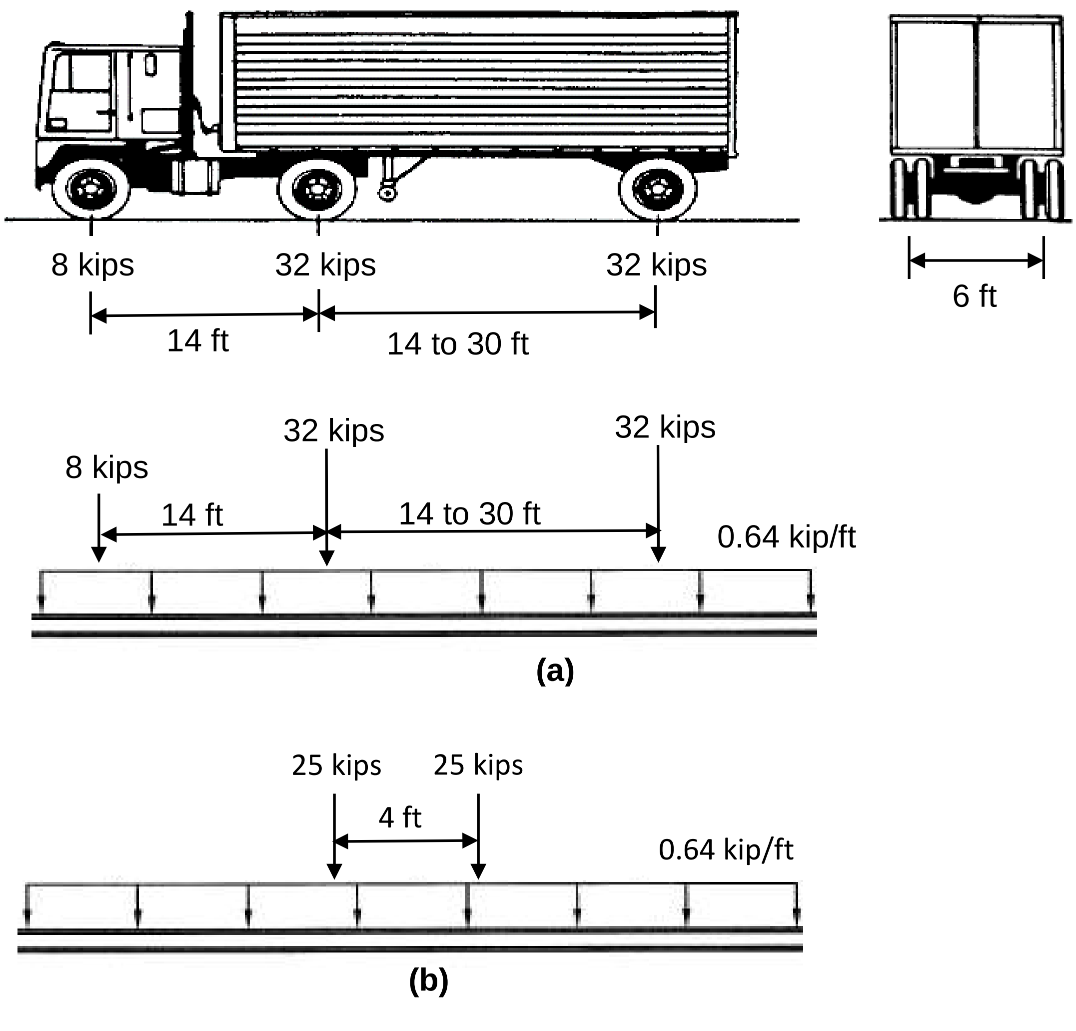


Figure 4.6: HL-93 load cases (a) Design truck + design lane load (b) Design tandem + design lane load

The moment ratio and shear ratio that is WIM truck divided by HL-93 loading, are shown in the form of CDF plots. In Figure 4.7 and Figure 4.8, the CDF plots for 30 ft span and 200 ft span for all the WIM locations in the year 2014 are shown. In almost all the WIM locations there is 0.1 % of the records have the moments above HL-93 design moments. In Figure 4.9, the shear ration of CDF plot for a 200 ft span for all the WIM locations in the year 2016 is shown. WIM location 931 creates high shear effects. Due to space constraints, only selected years and span lengths are shown.

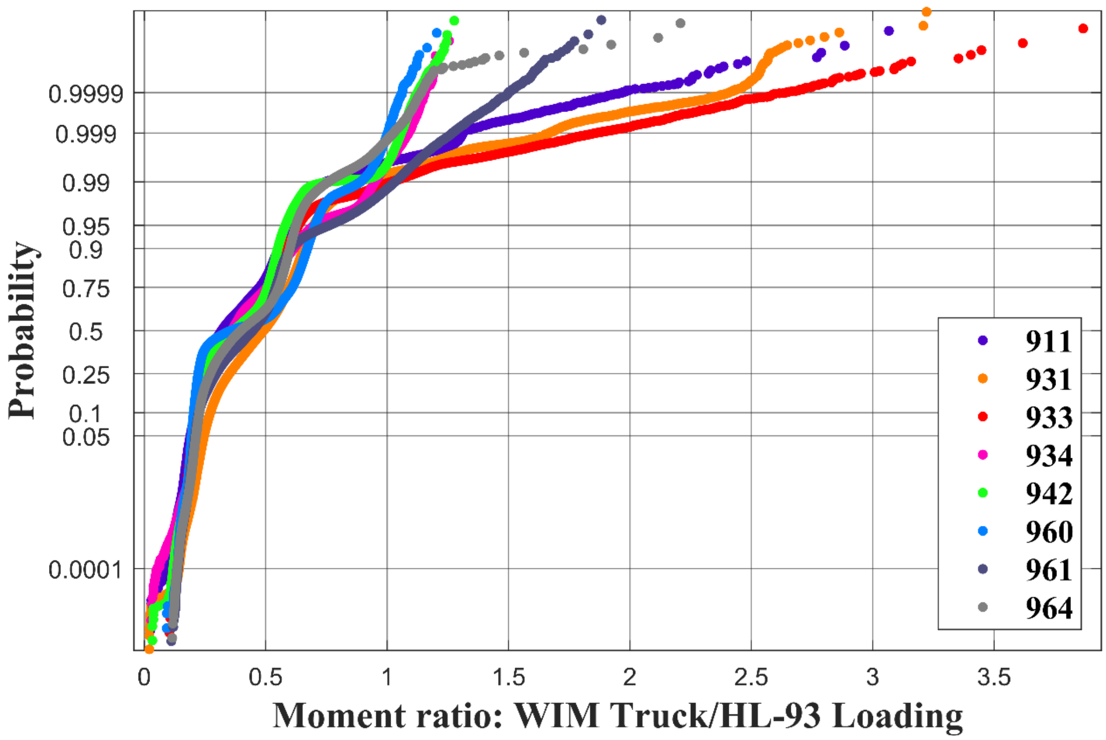


Figure 4.7: Cumulative Distribution Function plot for moment ratio for 30 ft span of all WIM stations for the year 2014.

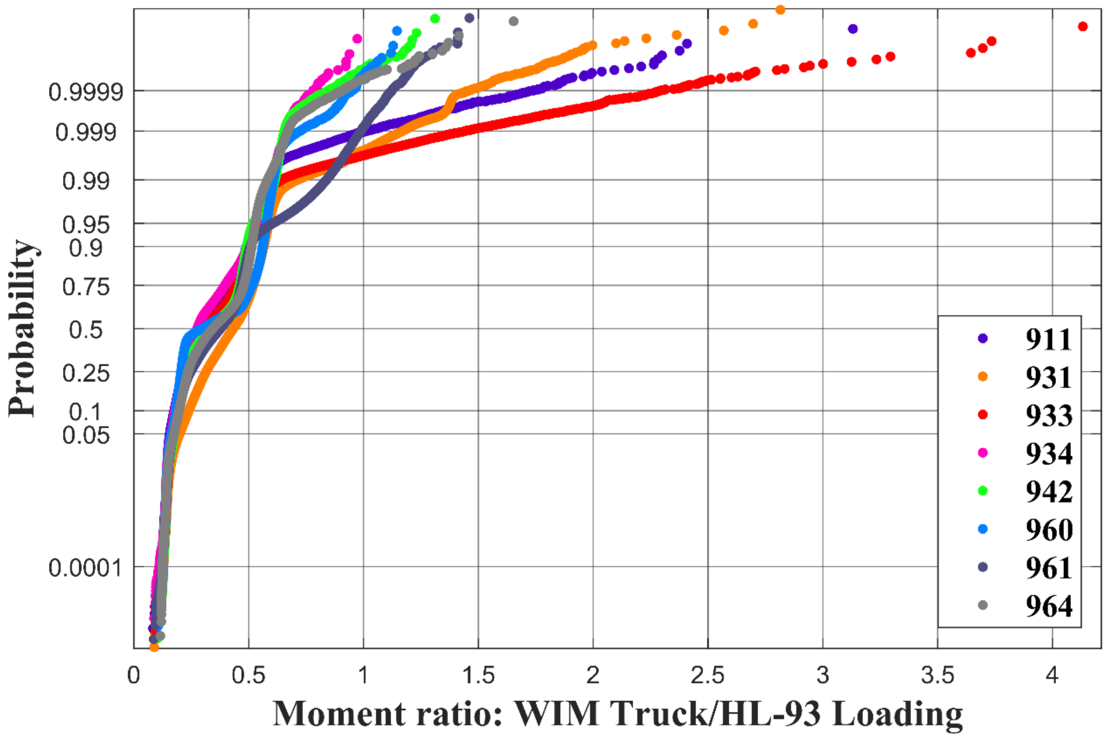


Figure 4.8: Cumulative Distribution Function plot for moment ratio for 200 ft span of all WIM stations for the year 2014.

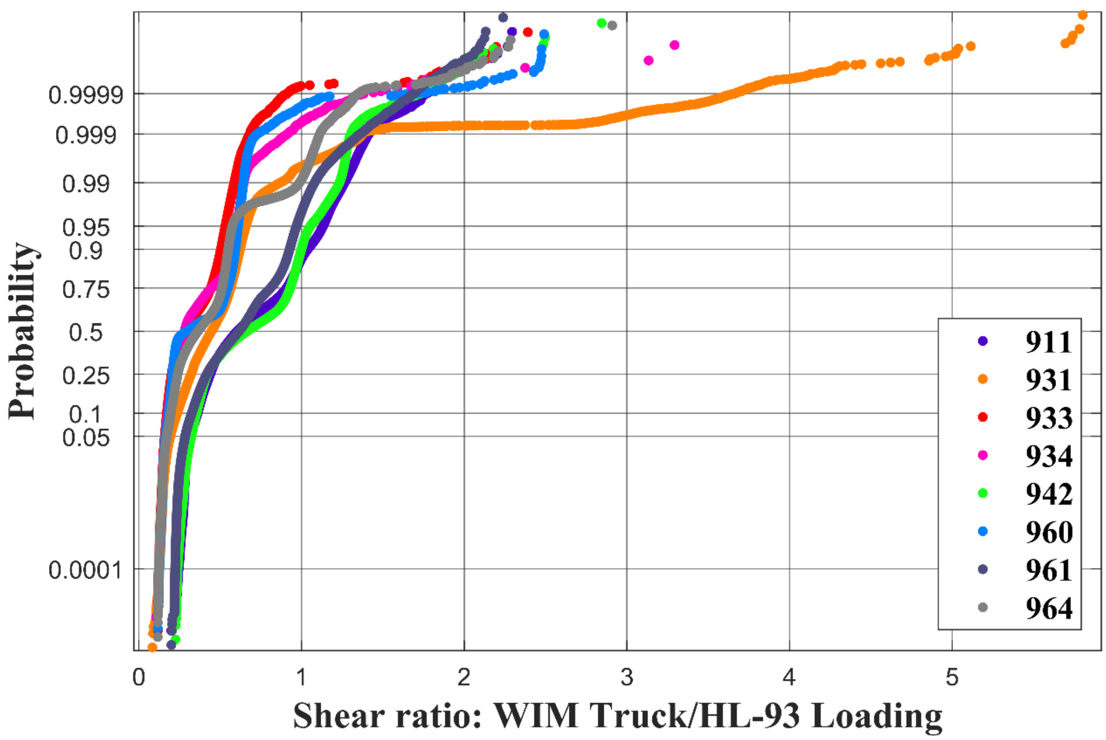


Figure 4.9: Cumulative Distribution Function plot for shear ratio for 200 ft span of all WIM stations for the year 2016.

4.1.3 Axle loads

The contribution of individual axle weight combines to form GVW in a vehicle. Heavy axle loads and frequency of occurrence contributes to faster deterioration of decks in the bridge. In some states the deign life of decks are 40 years (Ghosn et al. 2015). The replacement of decks without replacing the superstructure is not viable in some type of bridges like box girder bridges. It is necessary to have a knowledge of axle loads and frequency for adequate design and maintenance of bridges.

From Figure 4.10 to Figure 4.12, the CDF of axle loads of each axle is shown for years 2014 to 2016. Almost 99% (0.99 on probability scale) of the truck's axle loads are below 30 kips in all the years. The variation of axle loads is consistent in all the years.

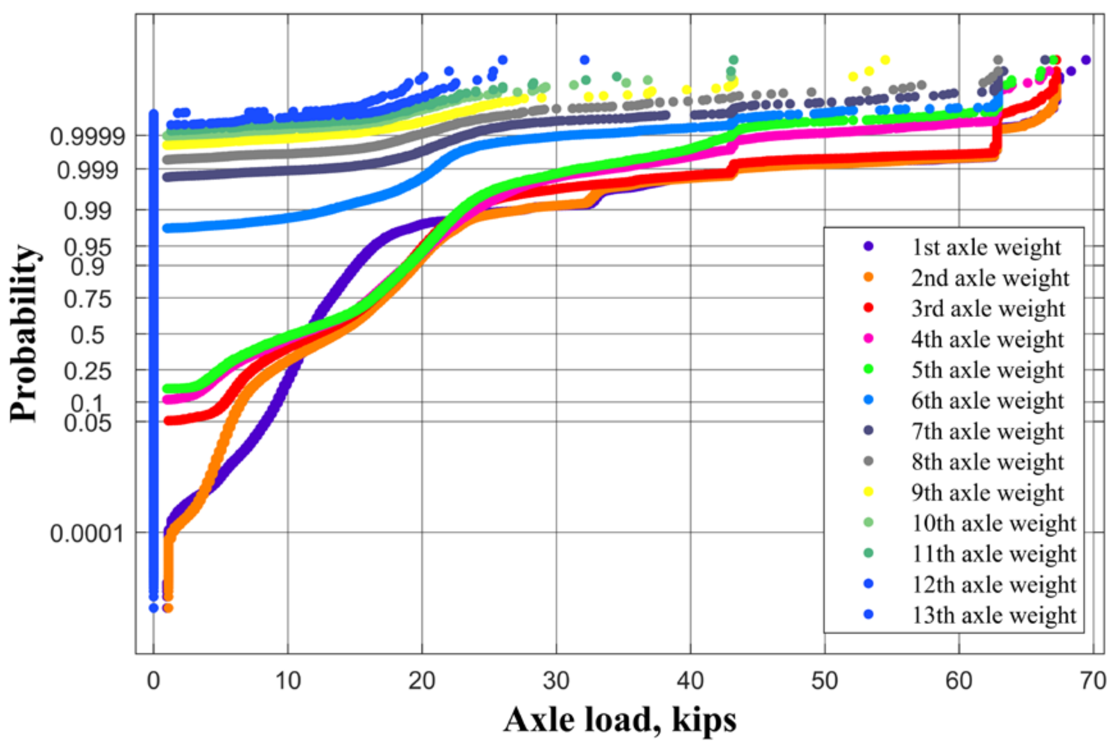


Figure 4.10: Cumulative Distribution Function plot of axle loads for all the WIM locations combined for year 2014.

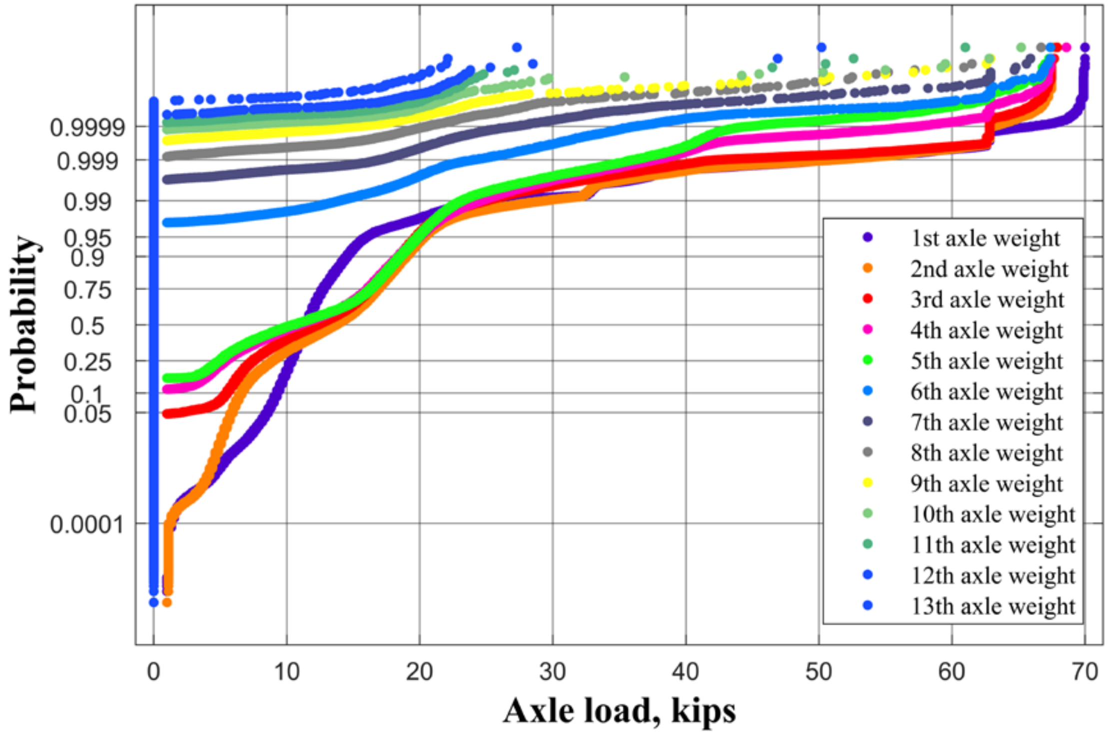


Figure 4.11: Cumulative Distribution Function plot of axle loads for all the WIM locations combined for year 2015.

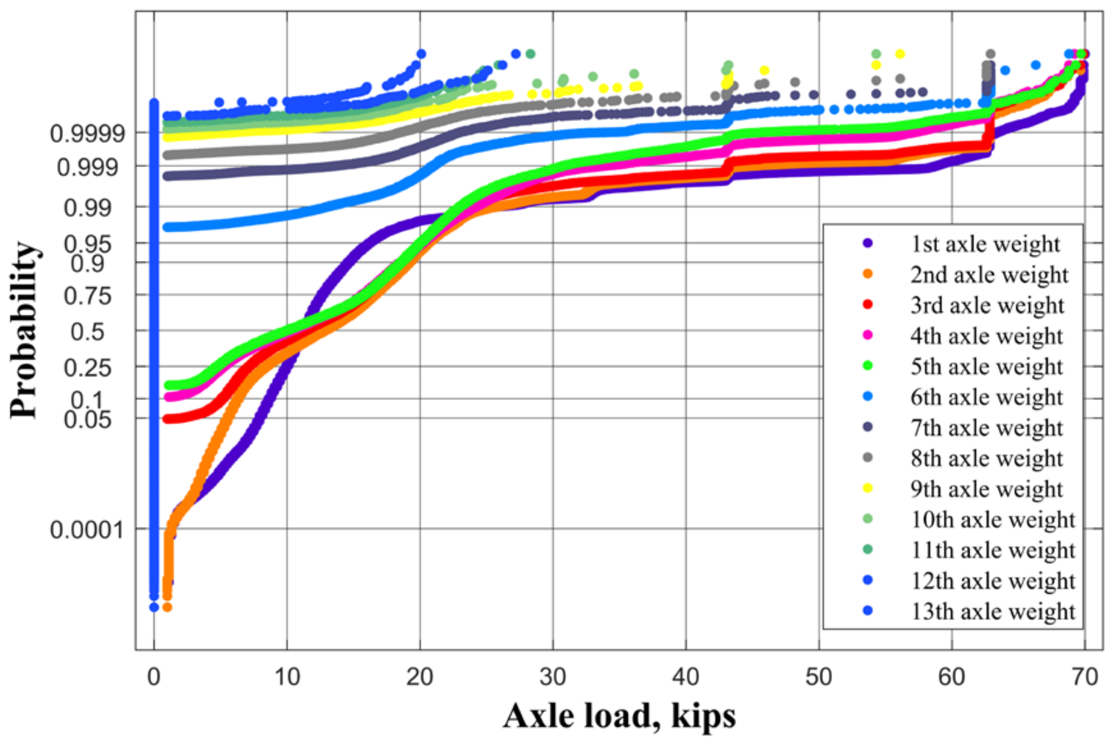


Figure 4.12: Cumulative Distribution Function plot of axle loads for all the WIM locations combined for year 2016.

4.2 Summary

This chapter shows the bridge live loads in the state of Alabama for the years 2014 to 2016. The WIM data filtered through the QC procedure, as discussed in Chapter 3, is used to develop bridge live load models.

The CDF of GVW shows the variation of GVW among each WIM location in the state and also for different years. Since the governing case for design is moment and shear, the moment and shear were calculated for each record in the WIM database by running on an influence line. Spans ranging from 30 ft to 200 ft were considered. For a better interpretation of results, the ratios are calculated for moment and shear produced by WIM records by dividing by HL-93 design moments for respective span lengths. If the ratio is greater than one, the respective WIM record is creating a moment or shear effect greater than design moment and shears. There are few vehicles in the Alabama database that have moment and shear ratios greater than one.

Chapter 5: Identification of Issued Permit Vehicles in WIM Traffic Database

5.1 Introduction

Evaluation of existing bridges requires the assessment and prediction of the load-carrying capacity and actual loads. Knowledge of the actual loads, including illegally overloaded vehicles, can help in day-to-day and planned maintenance procedures and law-enforcement effort. A major source of information about the bridge live load is the weigh-in-motion (WIM) database. Prediction of live load involves consideration of three groups of vehicles: legal, permit and illegally overloaded vehicles. Therefore, it is important to identify these three groups in the WIM data. It is easy to check if a recorded vehicle satisfies the requirements for legal vehicles. However, the major difficulty is to separate permit vehicles from the illegally overloaded ones.

Two procedures are introduced and discussed in this chapter: Geographic Information System (GIS) routing procedure and Data-driven procedure. The two procedures are demonstrated with the WIM data for the years 2014 and 2015. A simple revenue estimation of collected and uncollected revenue from overweight vehicles is made.

In the context of this dissertation, the identification of issued permit vehicles and illegal vehicles in the WIM database is useful to assess the relative damage caused by this group of vehicles to bridges. Other uses (but not limited) to are:

- Development of live load model for Strength I and Strength II limit state.
- Effective truck weight enforcement.
- Estimation of collected and uncollected revenue from overweight vehicles.
- Supporting information for modification in the state's permit fee structure.

According to section 3.4.1 of *AASHTO LRFD Bridge design specifications* (AASHTO 2017) the Strength I limit state is intended for ”normal vehicular use of bridge” and Strength II limit state is intended for ”owner-specified special design vehicles, evaluation permit vehicles, or both.” NCHRP Report 683 (Sivakumar et al. 2008) and Leahy et al. (Leahy et al. 2015) interpret the normal vehicular traffic as ”all legal trucks, illegal overloads and un-analyzed permits (all routine permits).” For the Strength II limit state, the different transportation agencies use state-specific design permit vehicles (Lou et al. 2018). So, to develop the live load model the overall traffic has to be categorized to normal vehicular traffic and traffic that has issued permits. A live load model for strength II limit state based on New Jersey permits data is developed by authors Lou et al. (Lou et al. 2018). Another important aspect of developing a live load model for the Strength II limit state is that not all issued permit trucks might travel on highways with the axle weight distributions for which they are authorized. So, the identification of this permit vehicle in WIM traffic data can help in developing an accurate live load model for Strength II limit state.

The state can deploy effective weight enforcement by knowing the summary of illegal trucks traveling on highways. A study conducted using a large truck traffic database in New York State concluded that by strategically planning effective enforcement, the state can realize a $16.0 to $ 53.2 million per year reduction in expenditures for pavement and bridge repair and maintenance (Fiorillo and Ghosn 2016). The estimation of collected and uncollected revenue from overweight vehicles can help the state in planning infrastructure maintenance and replacement budgets. Also, the identification of issued permit vehicles and illegal vehicles can provide supporting data for state transportation agencies to evaluate the state oversize/overweight (OS/OW) permit fees.

5.2 Literature Review

Identification of illegally overloaded vehicles in traffic is a trending topic in the transportation community. Nowadays, the traffic load monitoring systems are rapidly developing and are incorporated by State DOTs (Office of Freight Management and Operations 2017; *Traffic Monitoring Guide* 2016b). The effects caused by legal vehicles and permit vehicles can be assessed, but it is more important to evaluate the damage caused by illegally overloaded vehicles.

The problem of illegal overloading of trucks goes far beyond the safety of the roads and bridges. The violators create a high competition in the transportation service market, where the operators that follow the permit limits stay at a disadvantage. Most states follow the federal weight limits to protect the roads and bridges from progressive damage. However, requests to increase axle load limits to reduce transportation costs are reported (Luskin and Walton 2001; Stith 2006). A Texas Department of Transportation (TxDOT) study reported the consequences of a legal limit change to transportation infrastructure in the state of Texas are quite dramatic: $10 and $510 million for the replacement and repair of pavements and bridges, respectively. Moreover, the estimated annual savings on transportation costs from a repeal of the gross vehicle weight (GVW) limit of 355kN (80,000lb) in the state of Texas exceeds $2 billion (Luskin and Walton 2001).

Permit regulations and monitoring procedures were developed to provide the safe operation of the transportation structures. However, the problem of controlling the haulers violating the law remains unsolved, as well as the question: to what extent are the vehicles can be overloaded? Several sources reported about the relative proportion of illegal vs. law-abiding haulers (Enright et al. 2016; Fiorillo and Ghosn 2014; Luskin and Walton 2001; Stith 2006).

There is no exact method to distinguish permits and illegal vehicles in the collected WIM dataset (Enright et al. 2016). However, WIM records have been used to develop models of permit trucks for bridge rating and design. The benefits of separating the data and analyzing are shown in Caprani et al. (Caprani et al. 2008). In Wisconsin, individual vehicle records were used to evaluate the state-specific standard permit vehicles based on a statistical analysis of the load effects caused by the heaviest 5% of trucks in each class (Jian Zhao and Habib Tabatabai 2012). Similarly, both European and US WIM databases were analyzed to identify permit vehicles based on the state regulations and to produce an equivalent permit truck traffic using Monte-Carlo simulation (Enright et al. 2016). Fiorillo and Ghosn proposed a sorting procedure to define the proportions of illegally overloaded and permitted traffic based on WIM data collected by New York DOT (Fiorillo and Ghosn 2014).

5.3 Truck Size and Weight Regulations

It is required by law that vehicles exceeding the legal truck size and weight limits (TS&W) obtain the permits and pay permit fees to travel on highways. The state DOTs issue permits on a daily basis to oversize/overweight (OS/OW) vehicles that travel on highways that exceed the legal TS&W. The permits are to be obtained for vehicles that are oversized or overweight and the combination of both above legal limits. One of the first laws establishing limits on truck weight in the US was enacted in 1913 by many states (“Truck Size and Weight - FHWA Freight Management and Operations” n.d.). Later in

1956, the Federal Government started regulating the truck size and weight by the enactment of the Federal-Aid Highway Act of 1956 (Federal Highway Administration 2015b). As of today, all the states have laws to limit the truck size and weight. So, on highways, the traffic belongs to one of these three vehicle categories:

1. Legal loads - if it is within state's TS&W limits.
2. OW/OS Permit loads – if it is above state's TS&W limits and has authorization from state's permit issuing office.
3. Illegal loads - if it is above state's TS&W limits and does not have any authorization from state's permit issuing office.

The vehicles are allowed to operate without any permit and are considered as legal, as long as they satisfy the axle load limit, GVW limit and weight guidelines of Federal Bridge Formula Weights (Formula B) (Equation 4.2) ("Bridge Formula Weights-FHWA Freight Management and Operations" n.d.). The primary purpose of the formula is to reduce the risk of damage to highway bridges by the adequately distributed load by determining the optimum axle configuration and axle load distribution.

$$W=500\left[\frac{LN}{N-1}+12N+36\right] \tag{4.2}$$

where,

W – Gross vehicle weight of a group of axles under consideration, lbs

L – Distance between the outer axles of any group of two consecutive axles, ft

N – Number of axles in the considered group

However, this is applicable only on the Interstate network. For the state and local highway systems, each state has its set of weight guidelines. Many vehicles that do not obey the Federal Bridge Formula B but do obey the state's legal weight guidelines are commonly referred to as vehicles exempt with “grandfather rights” (Moses 2001). Weight limits that are in use now, along with Formula B and state-specific “grandfather” exceptions, were established in the mid-1970s (Federal Highway Administration and U.S. Department of Transportation 2015).

Figure 5.1 shows a graphical representation for sorting vehicles in traffic into different categories based on weights. Vehicles that are under legal weight limits in the jurisdiction and that satisfy Federal legal weight limit and “grandfathered rights” are considered as “Legal loads.” Otherwise, they require permits, either annual, single trip, or super load permit. Vehicles that require a permit but do not have it are considered as “Illegal Trucks.” Overloaded vehicles are those that require permits to travel. The database of permits and illegal vehicles together is overloaded vehicles.

According to *AASHTO LRFD Bridge design specifications* (AASHTO 2017), the normal vehicular live load for bridges (Strength I limit state) includes all legal trucks, “grandfathered” exceptions and vehicles permitted by routine permits. Illegally overloaded vehicles without permits belong to an unanalyzed portion of bridge live load that is more likely to create an extreme lifetime stress condition.

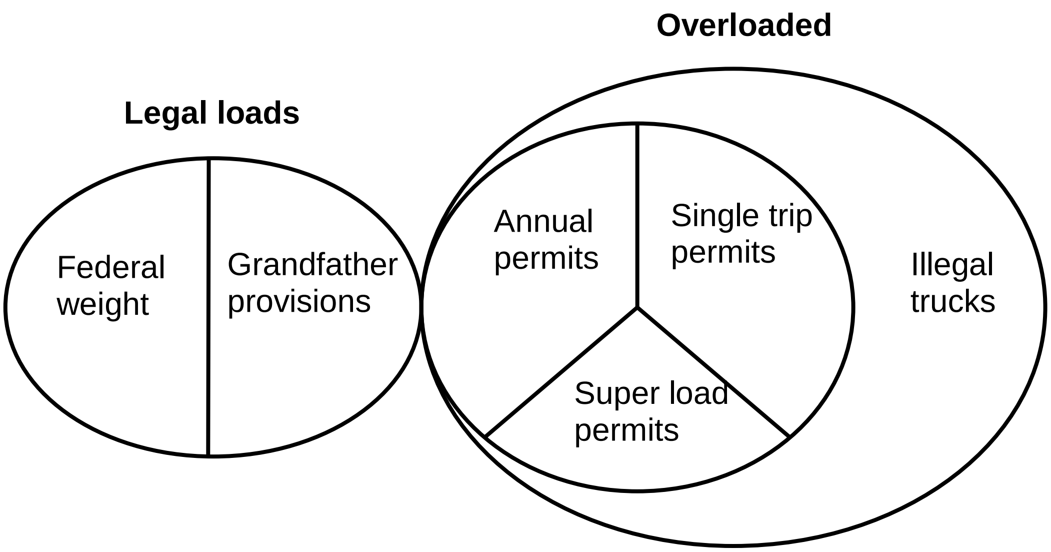


Figure 5.1: Vehicle categories.

Each state has specific permit regulations for the transportation of certain goods through the state. Truck weight regulations under Alabama jurisdiction ("Alabama Code Title 32. Motor Vehicles and Traffic" n.d.) are presented in Figure 5.2. Vehicles that satisfy the Federal legal weight limit and "grandfathered rights" in Figure 5.2 are considered as "Legal loads." Otherwise, they require permits, either annual, single trip, or super load permit. Vehicles that require a permit but do not have it are considered as "Illegal Trucks." Also, permits are issued for either overweight or oversize or both combined.

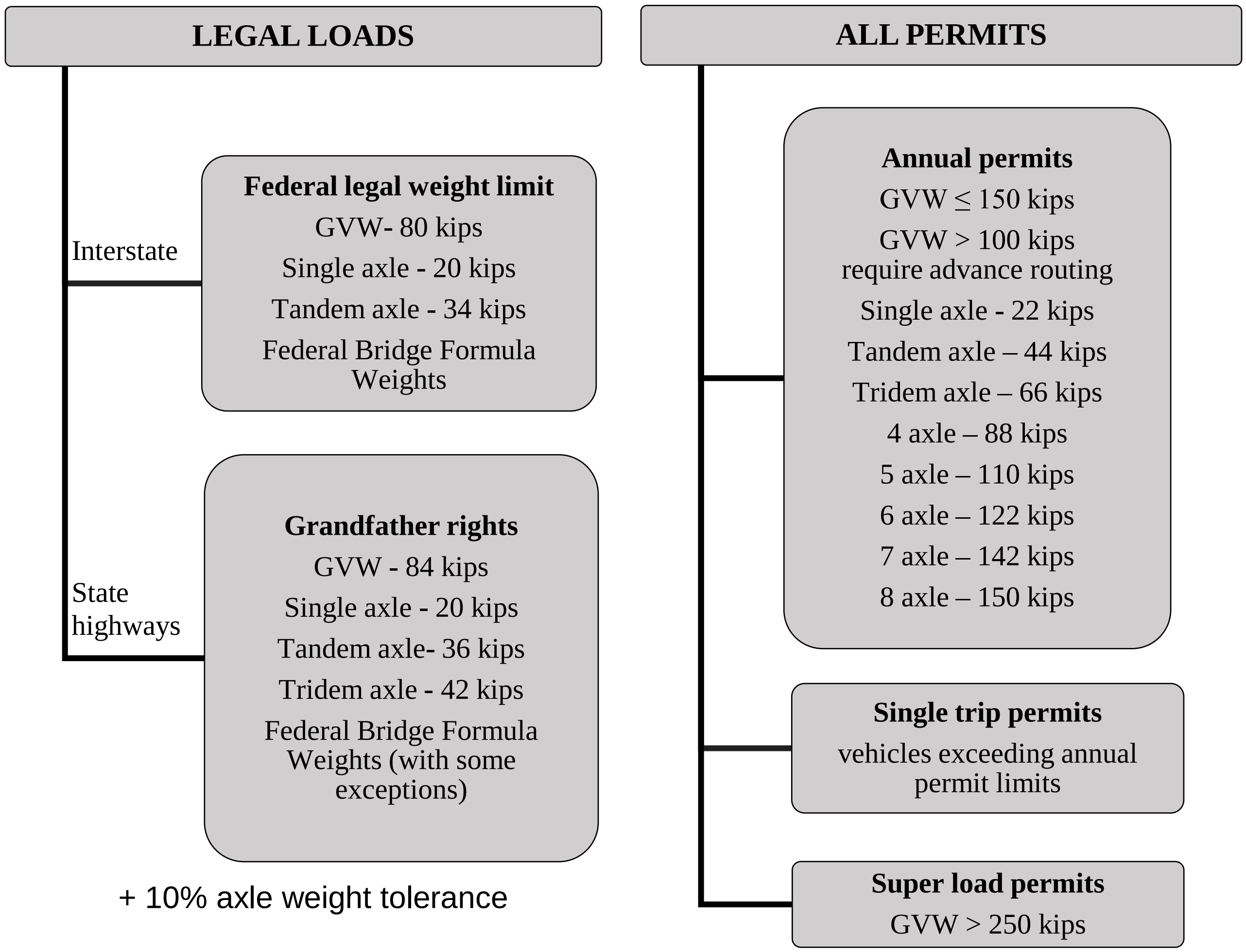


Figure 5.2. Truck weight regulations in the state of Alabama.

5.3.1 Vehicles that require permits in WIM database (Overloaded vehicles)

To identify permitted trucks, the first step is the elimination of legal traffic in the WIM traffic database, so that the remaining database includes only permit vehicles and illegal traffic, i.e., overloaded trucks. Based on the truck weight regulations for the state of Alabama that are shown in Figure 5.2, the traffic was sorted into legal and overloaded trucks.

Table 5.1 and Table 5.2 shows the summary of the WIM records that require permits for the year 2014 and 2015. Also, the WIM records that require permits are categorized further into the different kinds of permits required. Summary of legal and overloaded vehicles sorted based on FHWA vehicle class is shown in Appendix H.

Table 5.1: Vehicles that require permits in WIM database for the year 2014.

Year - 2014	WIM Location							
	911	931	933	934	942	960	961	964
Total records	**357,839**	**1,584,096**	**427,474**	**169,251**	**787,932**	**305,353**	**829,946**	**642,038**
GVW limit	1,553	30,388	4,850	192	862	1,039	37,986	1,109
Single Axle limit	7,059	297,015	9,695	9,910	13,372	15,432	62,848	10,886
Tandem Limit	5,599	296,030	4,621	6,512	9,526	28,093	43,538	9,622
Tridem Limit	576	-	346	760	4,353	1,492	-	1,791
Bridge Formula Weights	8,249	3,554	6,067	2,951	10,394	22,325	3,364	32,309
Permit/ Illegally overloaded	**23,036**	**626,987**	**25,548**	**20,325**	**38,507**	**68,381**	**147,736**	**55,717**
	6%	**40%**	**6%**	**12%**	**5%**	**22%**	**18%**	**9%**

Table 5.2. Vehicles that require permits in WIM database for the year 2015.

Year – 2015	WIM Location							
	911	**931**	**933**	**934**	**942**	**960**	**961**	**964**
Total records	**350,492**	**1,548,620**	**395,916**	**112,105**	**688,980**	**282,213**	**115,338**	**135,810**
GVW limit	320	53,168	1,558	88	664	713	410	239
Single Axle limit	5,665	350,002	7,437	6,094	11,746	13,689	2,625	862
Tandem Limit	6,582	240,637	3,319	4,766	8,968	30,497	6,187	475
Tridem Limit	494	0	511	328	3,550	1,062	-	299
Bridge Formula Weights	5,563	1,944	4,609	2,501	9,028	22,934	330	1,004
Permit/ Illegally overloaded	**18,624**	**645,751**	**17,434**	**13,777**	**33,956**	**68,895**	**9,552**	**2,879**
	5%	**43%**	**4%**	**12%**	**5%**	**24%**	**8%**	**2%**

5.4 Issued Permit Data

State DOT's issue permits on a daily basis to oversize/overweight (OS/OW) vehicles that go on highways that exceed the legal truck size and weight regulations (TS&W). Many OS/OW vehicles may pass more than one state from start point to end point. Some states require haulers to buy separate permits for each state they travel through. The permit fee structure varies by each state. The five basic permit fee structures currently used among the states are flat fees, distance-based fees, weight-based fees, weight-distance-based fees, and axle-based fees (Chowdhury et al. 2013). Figure 5.3 shows the permit fee structure that is adopted by different states in the whole U.S. (the state of Alabama has weight-based fees). Also, the ways of calculating the permit fee and reason for collecting permit fees change from state to state. The most common goal is to recover maintenance, repair, and construction cost of roads and bridges in the state.

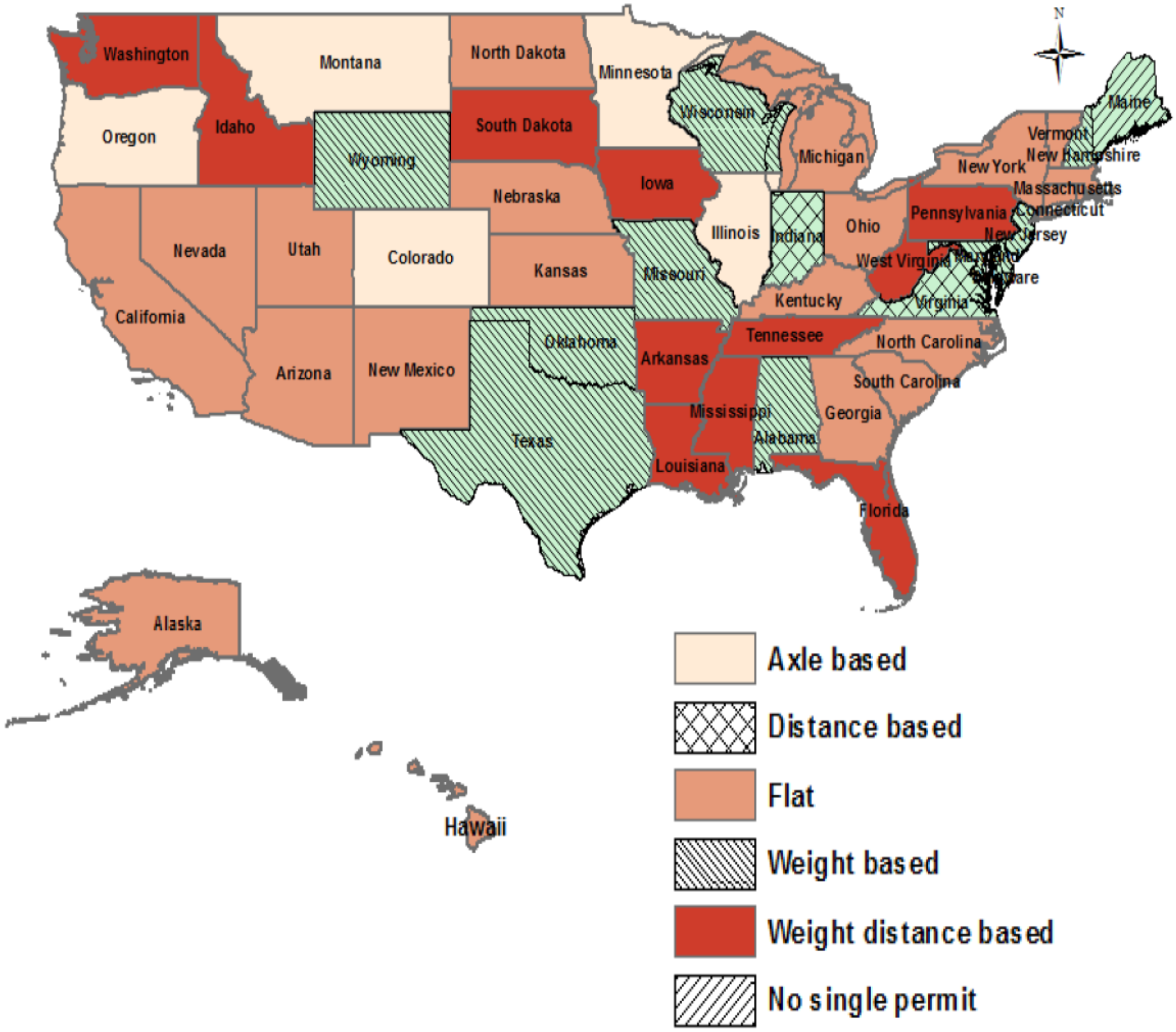


Figure 5.3: Permit fee structure of different states in the U.S. (Chowdhury et al. 2013)

In the state of Alabama, the Vehicle Enforcement office of Maintenance Bureau at ALDOT issues OS/OW permits on a daily basis. The permits in the state of Alabama are currently issued through ALPASS – Alabama's Online OS/OW Permitting system. Single trip permit, annual or blanket permit, and superload permit are the types of permits that are issued. Everyday Maintenance Bureau issues 500-600 permits. The TS&W regulations are dictated in Section 32-9-1 to 32-9-32, Code of Alabama 1975. The legislative history of the issuance of special permits is dated since 1932. However, the authorization to collect fees for issuance of permits was not in effect until 1977 ("A Legislator's Guide to Alabama Taxes" 2019). The current permit fees in the state are:

(1) Annual permits: $100 for annual permit.

(2) Single trip permits:

- (a) Mobile homes, modular homes, sectional homes, portable buildings, and boats:
  - (i) $10 - up to and including 12 ft. wide and 75 ft. long.
  - (ii) $20 - boats in excess of 12 ft. wide; and mobile homes, modular homes, sectional houses, and portable buildings in excess of 12 ft. wide and/or 75 ft. long.
- (b) Heavy commodities or equipment:
  - (i) $10 - over on any limitations as to length, height, or width.
  - (ii) $10 - over on weight from 80,001 lbs. up to 100,000 lbs.
  - (iii) $30 - over on weight from 100,001 lbs. up to 125,000 lbs.
  - (iv) $60 - over on weight from 125,001 lbs. up to 150,000 lbs.
  - (v) $100 - over on weight from 150,001 lbs. and over.
- (c) Miscellaneous:
  - (i) $20 for houses.
  - (ii) $10 for off-road equipment.
  - (iii) $20 for other oversized vehicles, loads, and equipment not otherwise specified.
  - (iv) $10 for other over height loads not otherwise specified.

(3) Superload permits: $100 + $10 if over on any limitations as to length, height, or width.

Currently, the state of Alabama operates mainly with six major funds (Operating Funds – Description – Executive Budget Office n.d.), and the revenue generated from OS/OW permit fees goes to Public Road and Bridge Fund (“A Legislator’s Guide to Alabama Taxes” 2019). Revenue collected over recent years is shown in Figure 5.4. The revenue is from both OW/OS permit vehicles. The change in revenue over the years is insignificant. Figure 5.5 shows OW/OS permit vehicle fee revenue based on the type of permits for the considered years. The revenue is almost similar in both the years.

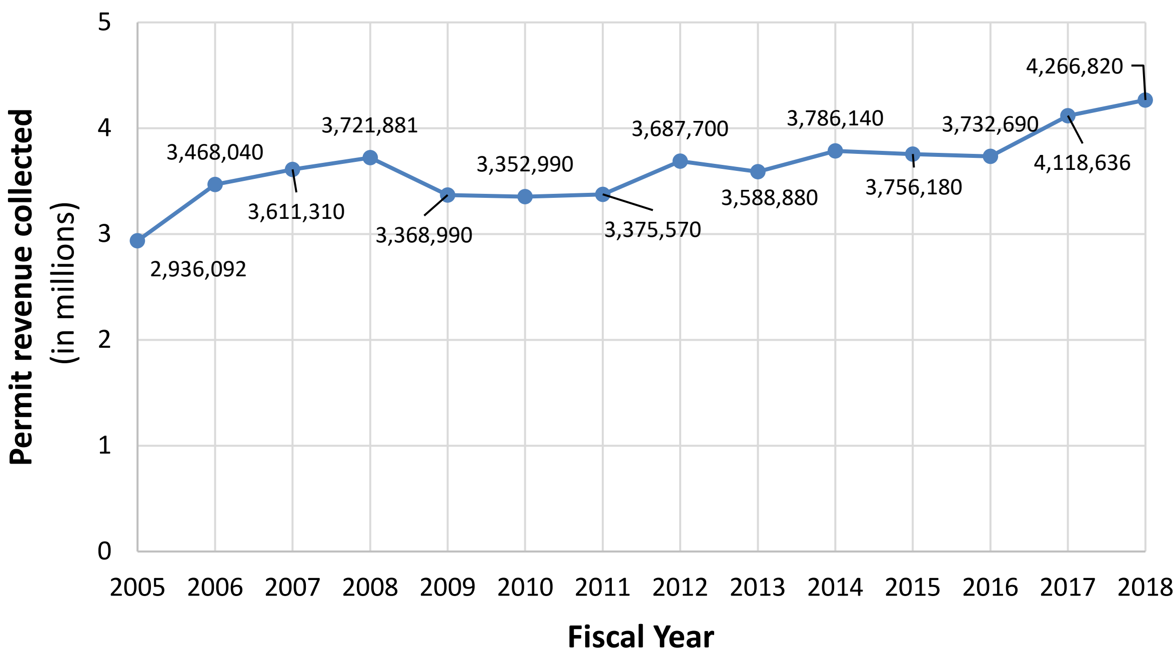


Figure 5.4: Revenue collected by issuing Overweight/Oversize permits in the state of Alabama. (“A Legislator’s Guide to Alabama Taxes” 2019)

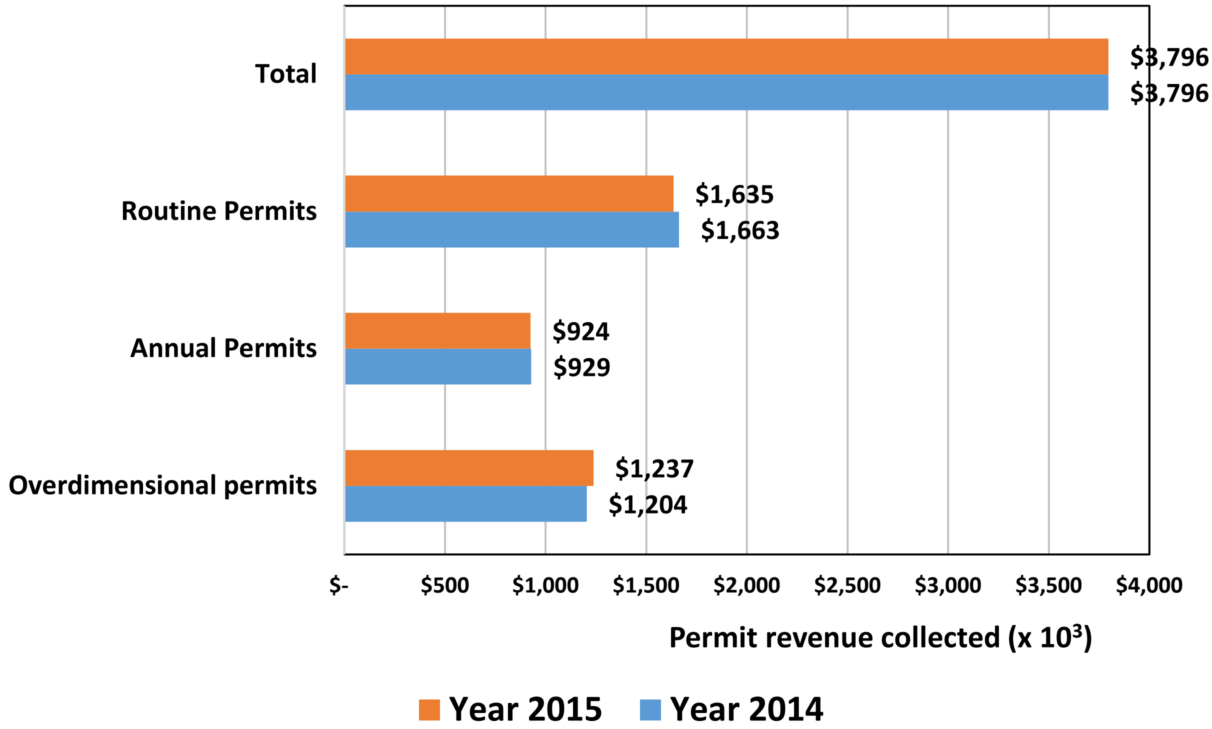


Figure 5.5: Overweight/ Oversize permit vehicle fee revenue based on the type of permits.

5.4.1 Permit Data Filtering

The total number of permits issued by the Maintenance Bureau is 123,602 for 2014 and 122,539 for 2015. Out of 500-600 permits per day, around 200 of them are permits issued for overweight. The permit data was available for the years 2014 and 2015. The permit data were in the form of tables in Excel format for each year. The data consisted of permit ID, the validity of the permit, original and destination, authorized roads, GVW, axle load and axle spacing. The data also included information about the size of the vehicle (e.g., abnormal width, length or length). The data for annual permits were included in the dataset; also, multiple trips of the vehicles that had annual permits are included in the database.

Initial analysis of data indicated some inconsistency in issued permit records. Also, the data contained all kinds of permits (Oversize/Overweight/Both combined). Since analyzing the oversize permits are beyond the scope of this dissertation, those were eliminated from the analysis. However, issued permits that had both oversize and overweight were retained along with only overweight permits. WIM systems used in the state of Alabama can record up to 14 axle vehicles only. So, permits issued to vehicles with more than 14 axles were excluded from the analysis. To eliminate oversize, annual trip permits and vehicles with more than 14 axles from the issued permit data, the following criteria were developed for filtering the data:

- GVW column is “0” or “LEGAL”
- “Number of axles” column is marked as “LEGAL” or blank “Axle Load” columns
- Number of Axles >14 (number of axles limit in WIM data)

The results of issued permit data filtering are summarized in Table 5.3. Also, the number of vehicles filtered based on number of axles is shown in Table 5.4.

Table 5.3: Summary of database of issued permits and filtering criteria.

		Year	
		2014	**2015**
	Total records	123602	122539
Filtering criteria	GVW = "LEGAL" and "0"	75833	75954
	Number of axles = "LEGAL" or "0"	6	15
	Number of axles > 14	28	18
	Data eliminated	75867	75987
	Data left after filtering	47735	46552
	% of data left after filtering	39%	38%

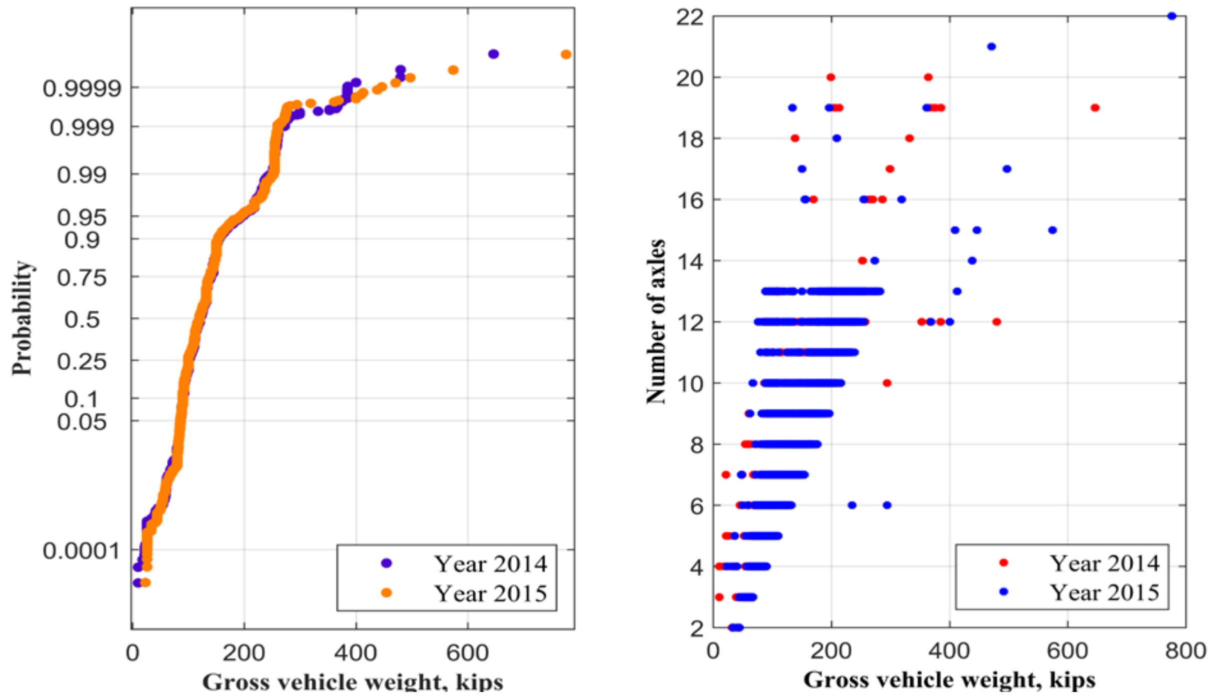
Table 5.4: Summary of database of issued permit based on number of axles.

Number of axles	Year 2014			Year 2015		
	Total data	Filtered data	Data left after filtering	Total data	Filtered data	Data left after filtering
0	27,172	27,172	0	29,248	29,248	0
2	18	2	16	24	1	23
3	147	2	145	177	0	177
4	330	3	327	220	6	214
5	6,671	5	6,666	6,885	4	6,881
6	11,440	2	11,438	1,1311	0	11,311
7	17,388	0	17,388	1,6274	2	16,272
8	6,480	0	6,480	6,742	0	6,742
9	1,288	0	1,288	1,315	0	1,315
10	824	0	824	704	0	704
11	973	0	973	807	0	807
12	1,065	0	1,065	893	0	893
13	1,114	0	1,114	1,206	0	1,206
14	11	0	11	7	0	7
15	0	0	0	3	3	0
16	7	7	0	7	7	0
17	4	4	0	2	2	0
18	3	3	0	1	1	0
19	10	10	0	3	3	0
20	3	3	0	0	0	0
21	0	0	0	1	1	0
22	0	0	0	1	1	0
LEGAL	48,654	48,654	0	46,708	46,708	0
Total	**123,602**	**75,867**	**47,735**	**122,539**	**75,987**	**46,552**
%	**100%**	**61%**	**39%**	**100%**	**62%**	**38%**

5.4.1 Issued permit data characteristics

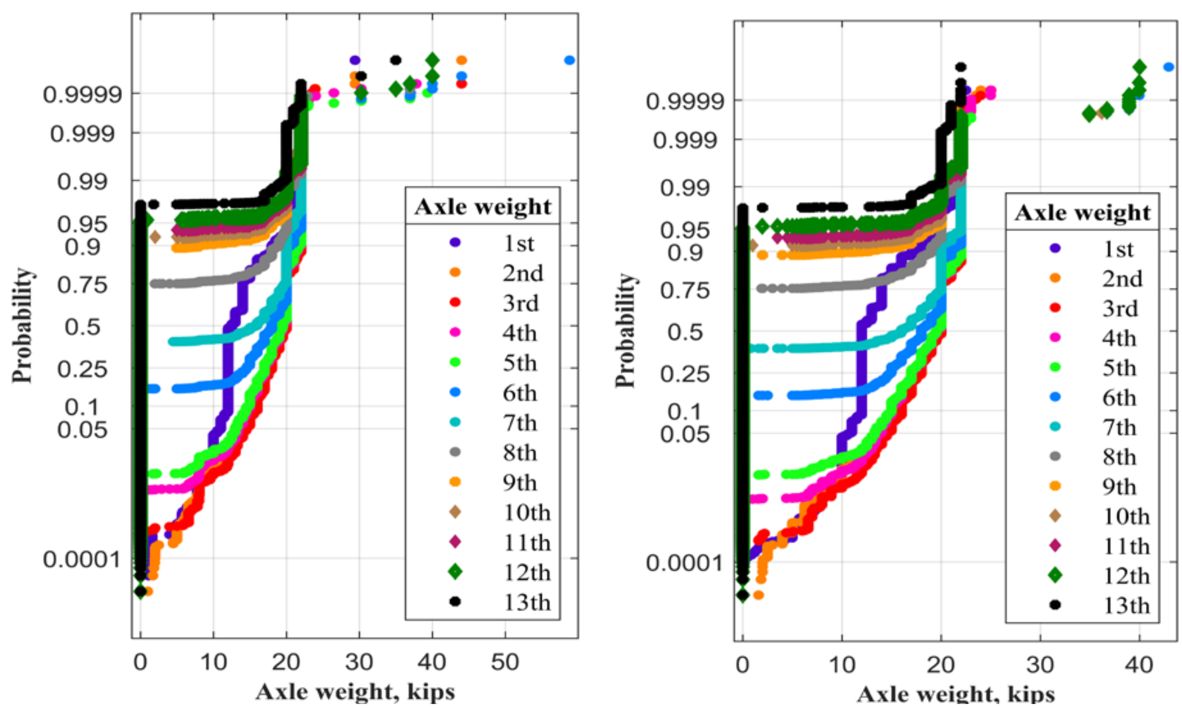
The issued permit data left after filtering is 47,735 in 2014 and 46,552 in 2015. However, for the purpose of visualization of the issued permit characteristics such as GVW, axle loads, moment and shear ratios, the issued permit with number of axles greater than 14 were included. The CDF’s of GVW for the years 2014 and 2015 is shown in Figure 5.6 (a). Almost 99.95 % (0.05 on probability scale) of issued permit data in both the years are above 80 kips (GVW legal limit). Both curves in the plots are for all WIM locations. The variation in traffic from each year at the WIM stations can be seen. The highest GVW varies from year to year. In all the WIM locations, there is variation in traffic load only in the top 0.1 % of the records. The highest GVW of an issued permit vehicle in 2014 is 646 kip and in 2015 is 776 kip. Figure 5.6 (b) shows the GVW versus number of axles of issued permit vehicles. A trend of increase in GVW as the number of axles increases is seen. There are a lot of issued permit vehicles that have 6 and 7 axles (count in Table 5.4). The highest GVW of an issued permit vehicle in 2014 is 646 kip, has 22 axles and in 2015, that is 776 kip has 19 axles.

Figure 5.7 (a) and (b) are the CDF plots of axle weight distribution for the years 2014 and 2015. Almost all the OW permits axle weight are below 22 kips except 2 to 3 vehicles above 22 kips. Figure 5.8 (a) and (b) are the CDF plots of moment ratio and shear ratio for the year 2014 and Figure 5.9 (a) and (b) show the same for 2015, respectively. These values are calculated as discussed in chapter 4.1.2.



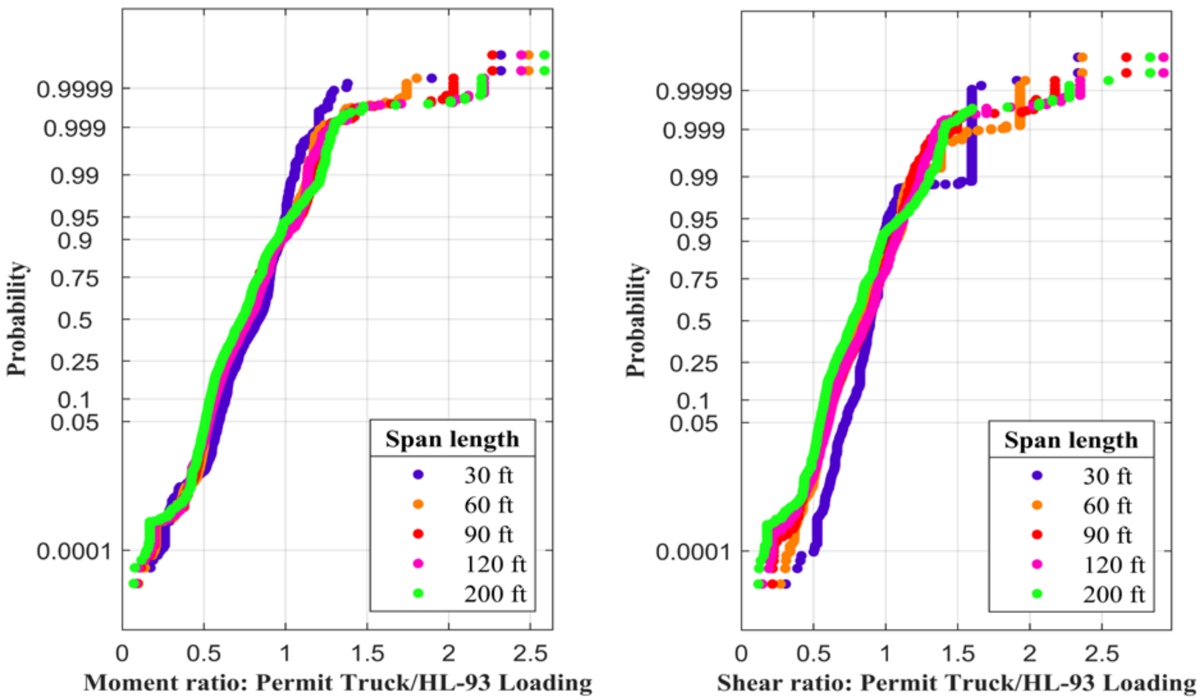
(a) (b)

Figure 5.6: Issued permit data for year 2014 and 2015 (a) Cumulative Distribution Function plot of GVW (b) GVW versus number of axles.



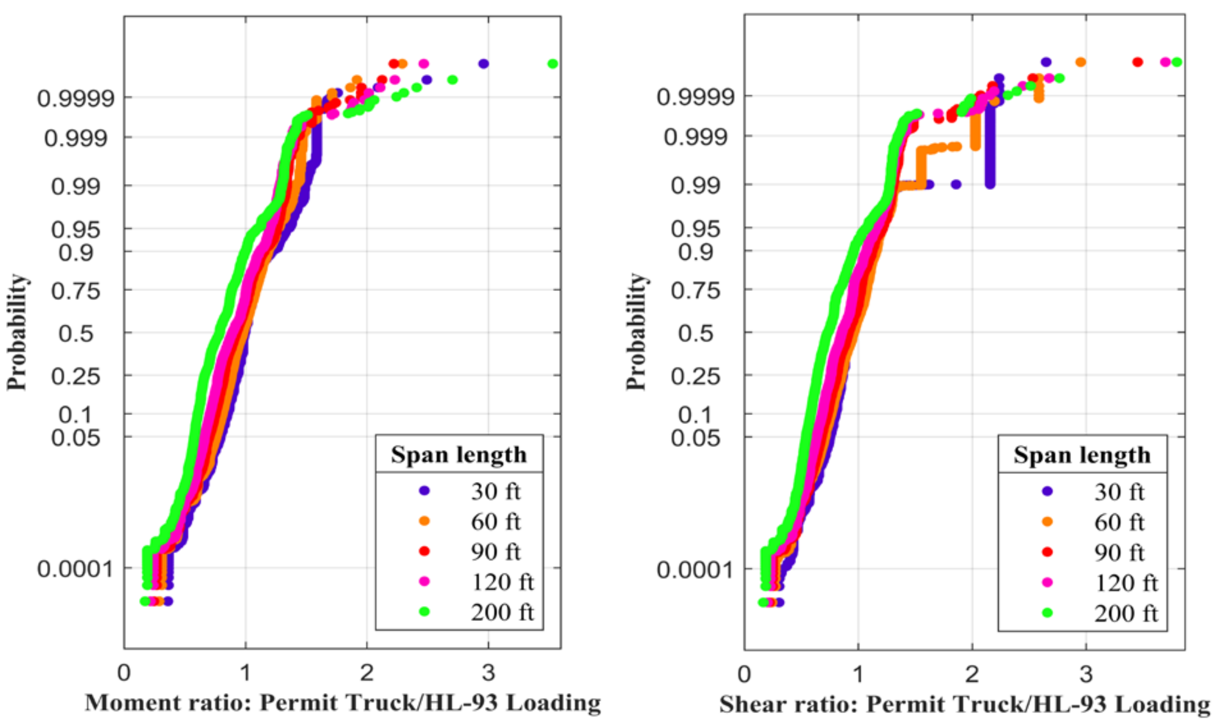
(a) (b)

Figure 5.7: Cumulative Distribution Function plot of axle weight of issued permit data (a) Year 2014 (b) Year 2015.



(a) (b)

Figure 5.8: Cumulative Distribution Function plot for year 2014 (a) Moment ratio (b) Shear ratio.



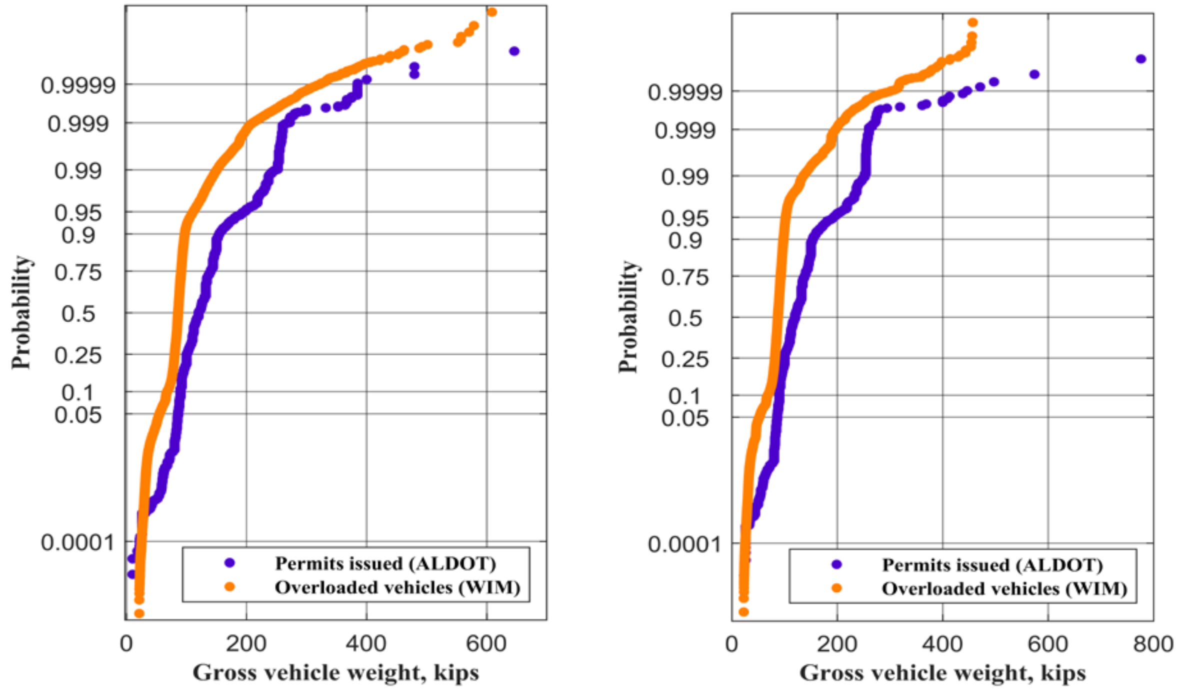
(a) (b)

Figure 5.9: Cumulative Distribution Function plot for year 2015 (a) Moment ratio (b) Shear ratio.

5.5 Identification of Permit Vehicles in WIM Data

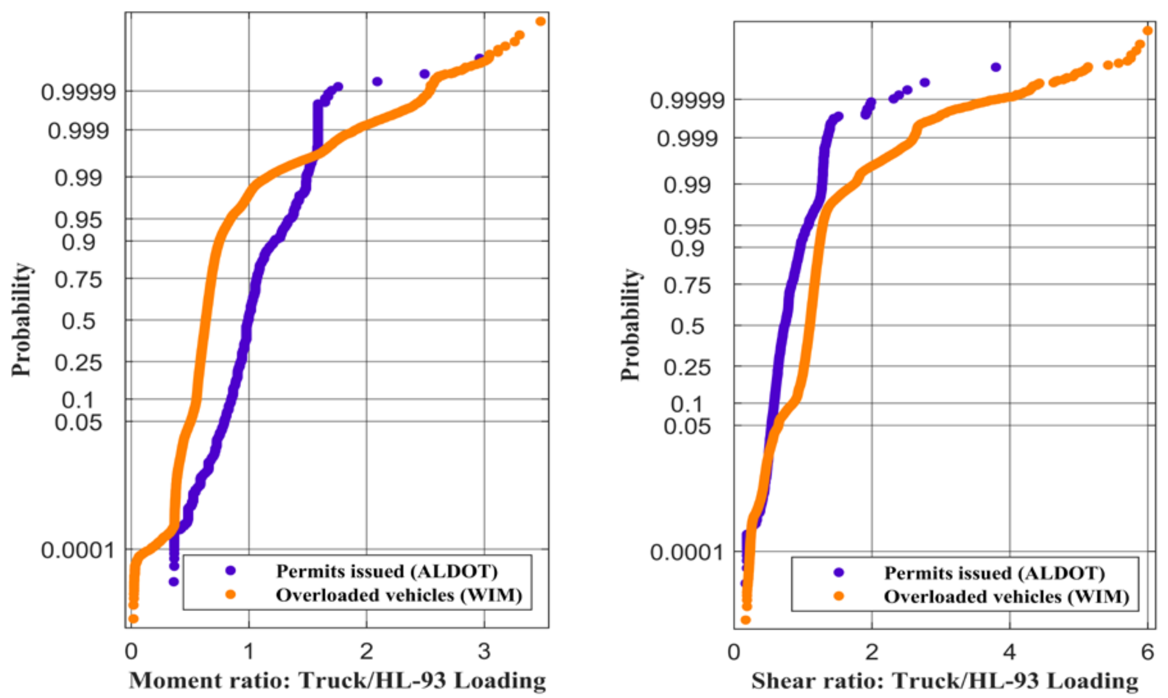
In the previous section, a summary of the overloaded vehicles (permit / illegally overloaded) in the WIM data database was presented. The main challenge is to determine if a vehicle with a permit passed a specific WIM station and was recorded by the WIM sensor. The procedure is demonstrated only for WIM data for years 2014 and 2015 since issued permit data was available only for those years. Before the developed procedure is demonstrated, some of the characteristics of the overloaded vehicles in WIM (a vehicle that requires a permit) and vehicles that have permits (ALDOT issued permits) are shown.

Figure 5.10 (a) and (b) show the CDF plot of GVW of overloaded vehicles in the WIM database and permit issued by ALDOT for years 2014 and 2015. The heaviest loads are seen in permit issued database for both considered years. From a fatigue point of view, vehicle count, and its weight and configuration are important. Figure 5.11 (a) and (b) shows the CDF plot of moment ratio of 30 ft span and the shear ratio of 200 ft span of overloaded vehicles and permit issued during the year 2015. In both the cases, there are a lot of overloaded vehicles that has moment and shear ratio above 2 compared to issued permit vehicles.



(a) (b)

Figure 5.10: Cumulative Distribution Function plot of GVW of overloaded vehicles and permit issued (a) Year 2014 (b) Year 2015.



(a) (b)

Figure 5.11: Cumulative Distribution Function plot of overloaded vehicles and permit issued for 2015 (a) Moment ratio of 30 ft span (b) Shear ratio of 200 ft span.

5.5.1 GIS routing procedure

Each vehicle in the database of issued permits by ALDOT has a detailed description of a route, including “original destination,” “final destination” and “authorized routes.” Using this information, it is possible to visualize the route of each trip on a web mapping service and check whether permit vehicles pass one or more WIM stations. However, ALDOT issues around 120,000 permits annually and it is difficult to track each permitted vehicle manually.

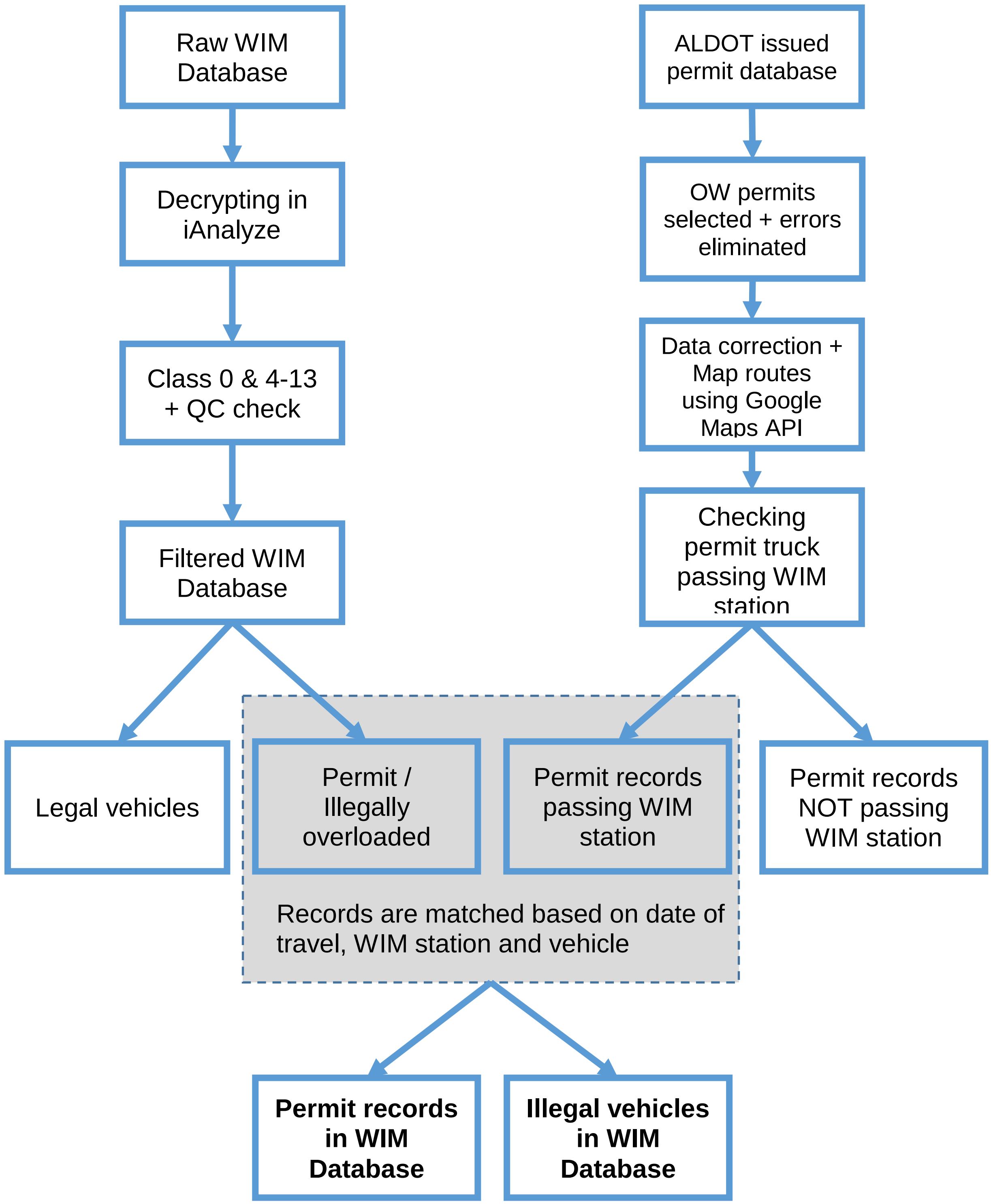
In this section, the procedure to detect permit vehicles passing WIM station is presented. An automated code was developed to analyze every trip route made by the vehicle in the permit database to identify the trip paths, and corresponding WIM stations passed. The algorithm is presented as a flowchart and is shown in Figure 5.12.

As shown in Figure 5.12, there are two databases used to sort permit vehicles in the WIM database. One is the WIM database, and other is issued permits database by ALDOT. Firstly, the WIM database is processed as shown in the left side of the flowchart by starting from decrypting in iAnalyze (Figure 2.9). Then, the Class 0 & 4-13 is selected, and QC checks are done to eliminate errors and determine the validity of the data (Figure 3.1). Later the filtered data is sorted into a legal vehicle group and permit/illegally overloaded group using Alabama weight regulations (Figure 5.2).

Next, the issued permits database by ALDOT is processed. On an everyday basis, each permit application is processed by ALDOT using Bentley software, where the trip route is entered manually by the hauler, and authorized routes are approved by ALDOT. So, in the provided accumulated permit database, there are permits issued for Oversize (OS) as well as Overweight (OW). The database of OW permits is extracted and further filtered to eliminate errors caused by manual entry. There are many possible ways the routes can be entered manually, and there are some routes that may not be recognized by Google Maps API. Therefore, some of these names and route descriptions must be manually corrected for more accurate mapping in Google Maps API.

Route mapping process is illustrated below:

- Asking Google Maps API for directions on the possible routes between the source and destination.
- Comparing the route descriptions produced by Google Maps API to the authorized route description and select the best match.
- Encoding coordinates from the Google Maps API path description string for the selected route.
- Building a string line based on the above-encoded path coordinates.
- Finding WIM station in a buffer zone around the string line (WIM station coordinates are mapped at first).
- Storing results in database and Keyhole Markup Language (KML) for future review.



***WIM – Weigh-in-Motion; QC – Quality Control; OW – Overweight; API – Application Programming Interface*

Figure 5.12: Flowchart of GIS routing procedure to identify issued permit vehicles.

The routes specified in the authorized route description of each permit application were matched with one of the possible routes of Google Maps API. All the routes of individual overweight permit vehicles are shown in Figure 5.13. As a result of this procedure, each record of the permit database is marked with the corresponding WIM station it passed. Then there is a set of permit records that pass-through WIM stations and others that do not pass. Lastly, the permit/illegally overloaded dataset from the WIM database and a dataset of permit records that pass-through WIM station is matched based on the date of travel, WIM station, and vehicle configurations. There might be more than one vehicle configuration matching, but only one of the vehicles was taken into consideration. By this matching process, the permit and illegally overloaded vehicles can be identified at each WIM station. Some of the permit vehicle routes go through more than one WIM station. Therefore, the same permit is compared with the WIM records collected from several WIM sites. The summary of the permit vehicles identified at all WIM sites is shown in Table 5.5. Also, in Table 5.5, the summary of a number of records that passed through a WIM station and number of records that passed through WIM station and were matched with one of the Overloaded WIM records is shown.

A comparison of the permits identified in the traffic stream (Table 5.5) to the vehicles that require permits (Table 5.1 and Table 5.2) for the respective WIM stations is possible. In the year 2014 and 2015 there were 1,006,237 and 810,868 vehicles that require permits, out of which 3,734 and 2,951 vehicles operated with a permit, respectively. These numbers indicate that less than 0.5% of overweight vehicles operate with a permit. The total number of WIM overloaded vehicles requiring permits of all the

WIM locations is substantially higher than the number of permits that were issued even though the permit trucks often pass more than one WIM location.

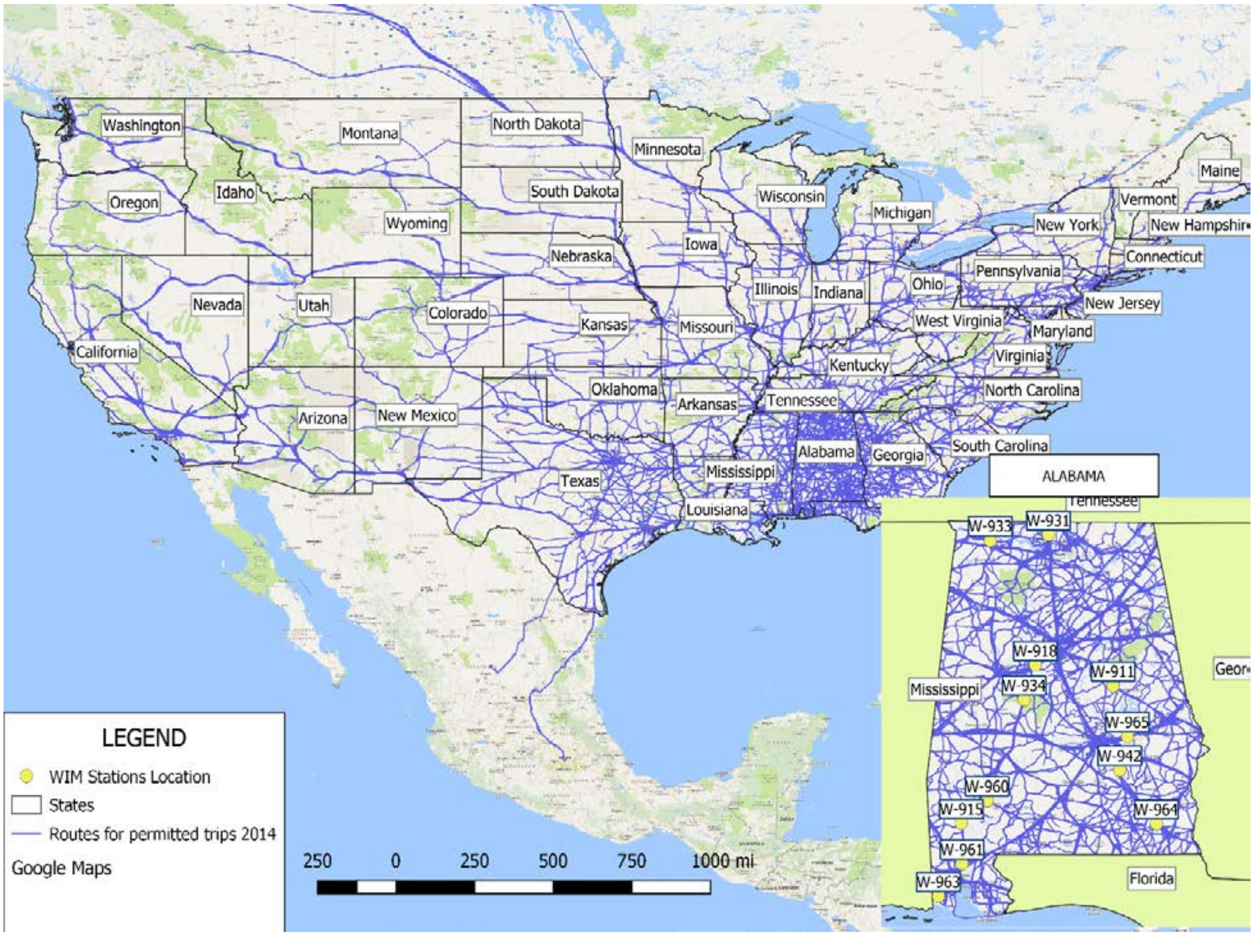


Figure 5.13. Routes of individual overweight permits issued by ALDOT for the Year 2014.

Table 5.5. Summary of ALDOT issued permits that passed through WIM stations and that that passed through WIM stations and matched with Overloaded WIM records.

WIM Station	Issued permits that passed through WIM stations		Issued permits that passed through WIM stations and matched with one of Overloaded WIM records	
	Year 2014	Year 2015	Year 2014	Year 2015
911 (US280)	575	448	225	165
915 (US43)	556	899	N/A	N/A
918 (I20)	8,086	6,963	N/A	N/A
931 (I65)	2634	2,479	1,000	934
933 (AL157)	200	216	69	73
934 (US78)	228	211	86	76
942 (US231)	526	530	206	193
960 (US84)	885	489	339	192
961 (I65)	4,381	3,055	1,696	1,193
963 (I10)	3,899	4,292	N/A	N/A
964 (US231)	282	349	113	125
965 (I85)	2,827	3,748	N/A	N/A
Total	25,079	23,679	3,734	2,951

5.5.2 Data-driven procedure

The data-driven procedure is a simple but effective procedure. This procedure uses very few conditions and simple filtering procedures. To develop this procedure, one should understand the state's TS&W and look carefully at the limitations on size and weights. Figure 5.14 shows the data-driven procedure to identify Illegal vehicles.

Firstly, an overloaded WIM database is sorted based on the TS&W of the state as discussed in section 5.3. Then each axle load is checked to see if it is greater than 30 kips. The limit of 30 kips was chosen because, according to Alabama's TS&W, all axle weights greater than 22 kips (+10%) variation are not allowed on highways nor permits are issued. Assuming there is a variation in a measurement by WIM sensors, it was rounded to 30 kips. If all the axle loads are within 30 kips, then the respective WIM records go to the next step. If any one of the axle loads is above 30 kips, then it is put into illegally overloaded vehicles database.

From the issued permit database, overweight permits are extracted, as discussed in section 5.4. From this overweight permit database, the mining parameters are extracted. First, the overweight permits are grouped based number of axles, and from each axle group, maximum and minimum wheelbase dimensions are extracted. Similarly, from each axle group, maximum and minimum inter-axle spacings are extracted. Table 5.6 and 5.7 shows the minimum and maximum wheelbase and inter-axle spacing for years 2014 and 2015 issued permit database. The vehicles in the WIM database that are less than 30 kips are checked to determine if the wheel base and axle spacing are within the mining parameters. If any WIM records are within the parameters, the respective WIM records go to the next step. If any is outside, then it is put into illegally overloaded vehicles database.

The last step in the procedure is to match the records in the WIM database that have passed the axle load check and mining parameters with the overweight permit database. In this step, it checked to see if any vehicle in the overweight permit database axle configuration and date of travel matches the record in the WIM database. The issued permit database is valid for five days, so it is checked whether a vehicle in the WIM database is within these five days of travel. The vehicles in the WIM database that are matched are the vehicles that traveled with authorized permits. Those vehicles that did not match are put into illegally overloaded vehicles database.

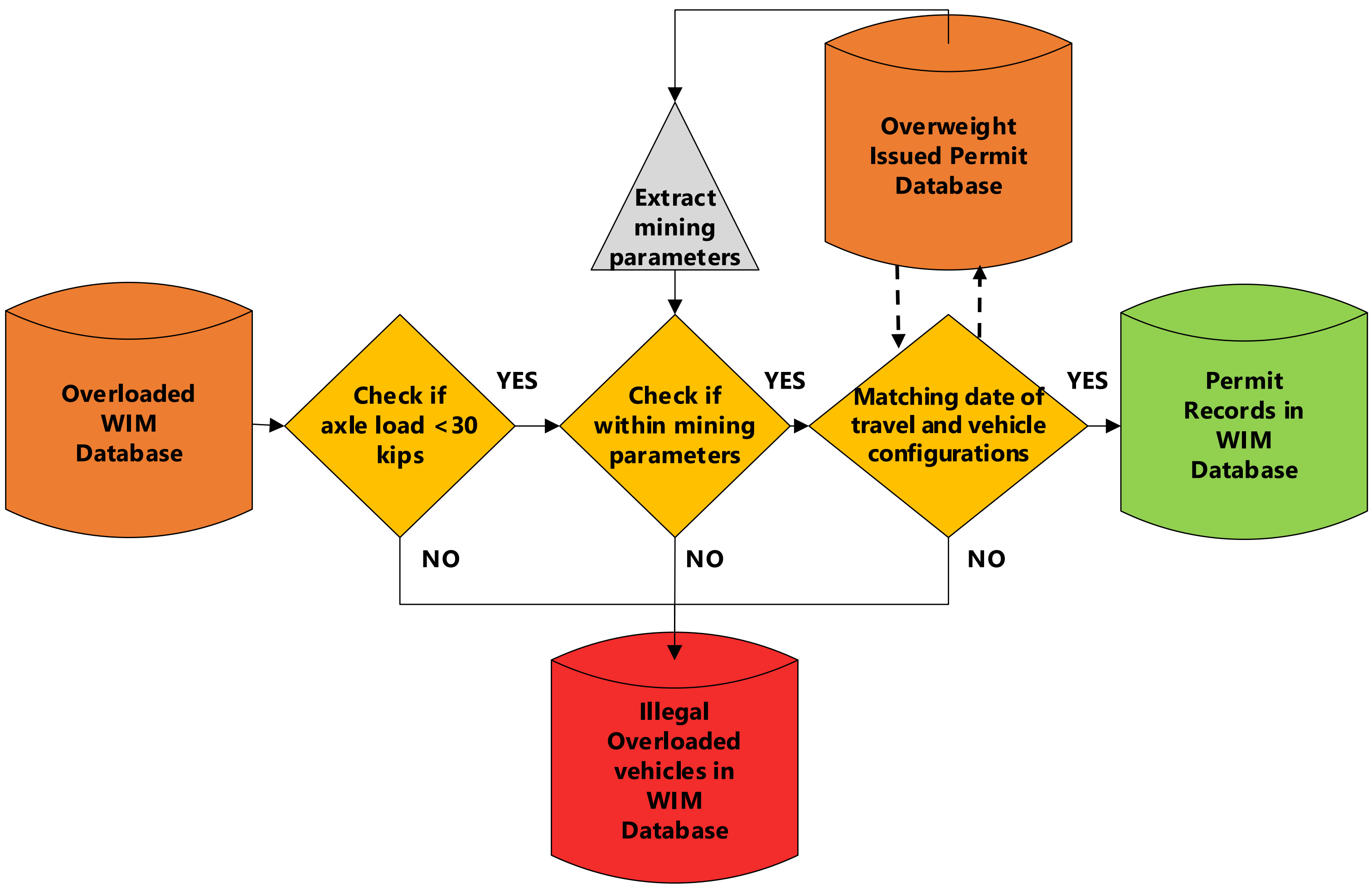


Figure 5.14: Flowchart of data-driven procedure to identify issued permit vehicles.

Table 5.6: Wheelbase and axle spacing parameters of issued permit database for year 2014 (a) Maximum and (b) Minimum

(a)

Year 2014				Max. axle spacing length (ft)												
No. of Axles	No. of records	Max GVW (kips)	Max length (ft)	Axle 1-2	Axle 2-3	Axle 3-4	Axle 4-5	Axle 5-6	Axle 6-7	Axle 7-8	Axle 8-9	Axle 9-10	Axle 10-11	Axle 11-12	Axle 12-13	Axle 13-14
2	16	44	27.583	27.6	-	-	-	-	-	-	-	-	-	-	-	-
3	145	67	42.5	26.3	23.5	-	-	-	-	-	-	-	-	-	-	-
4	327	89	76.834	26.3	36	51.7	-	-	-	-	-	-	-	-	-	-
5	6666	110	122.25	29	66	95.6	48.2	-	-	-	-	-	-	-	-	-
6	11438	132	152.333	29.5	33	119	68	46	-	-	-	-	-	-	-	-
7	17388	154	155.17	28	14.5	86	122	35.3	18.8	-	-	-	-	-	-	-
8	6480	174	166.167	35	35	63	102	99	36.2	18	-	-	-	-	-	-
9	1288	196	167.917	26	10.5	95	95	82	90.6	39.4	15.3	-	-	-	-	-
10	824	294	170.417	24.3	10.5	27.5	95	53.8	97	64.3	18.8	18	-	-	-	-
11	973	236	161.166	25.5	5.58	35.2	38.5	34.8	97.3	51.3	16.1	18	14.42	-	-	-
12	1065	479.6	171.5	22.4	5.58	42.5	42.5	9.08	80	60	42.7	16.8	17.5	9.083	-	-
13	1114	282	204	24	5.25	16.1	46	16.1	36.7	101	55	16.1	28.42	16.08	5.25	-
14	11	273	204	17.3	4.5	4.5	41.5	5	5	5	90	5	5	16	5	5

(b)

Year 2014				Min. axle spacing length (ft)												
No. of Axles	No. of records	Min GVW (kips)	Min length (ft)	Axle 1-2	Axle 2-3	Axle 3-4	Axle 4-5	Axle 5-6	Axle 6-7	Axle 7-8	Axle 8-9	Axle 9-10	Axle 10-11	Axle 11-12	Axle 12-13	Axle 13-14
2	16	32	12.25	12.3	-	-	-	-	-	-	-	-	-	-	-	-
3	145	10.02	2	2	2	-	-	-	-	-	-	-	-	-	-	-
4	327	10	4	4	4	2.83	-	-	-	-	-	-	-	-	-	-
5	6666	22	2.9167	2.92	2	2.75	1.17	-	-	-	-	-	-	-	-	-
6	11438	45.25	2.25	2.25	1.33	2.75	1.17	2.83	-	-	-	-	-	-	-	-
7	17388	21.6	1.5	1.5	3	3	3.5	3.5	3.5	-	-	-	-	-	-	-
8	6480	54	0	0	3.5	3	3	3	2.08	3	-	-	-	-	-	-
9	1288	60	4.9167	4.92	3.75	4.17	4	4	4	4	4	-	-	-	-	-
10	824	86.8	5.4167	5.42	4	4	4.08	3.42	4	4	4.08	4.08	-	-	-	-
11	973	80	9.6667	9.67	3.83	4.17	4.08	4	4.17	4.17	4.17	4.17	4	-	-	-
12	1065	80	1.5	1.5	3.5	4	4.08	4.08	4.17	4.17	4.17	4.25	4.167	4.167	-	-
13	1114	90	10.5	10.5	3.75	4	5	4.08	4.08	4.5	4.08	4.08	4.167	4.083	4.083	-
14	11	252	16	16	4.5	4.5	33.8	4.5	4.5	4.5	42.1	4.5	4.5	15.67	4.5	4.5

Table 5.7: Wheelbase and axle spacing parameters of issued permit database for year 2015 (a) Maximum and (b) Minimum.

(a)

Year 2015				Max. axle spacing length (ft)												
No. of Axles	No. of records	Max GVW (kips)	Max length (ft)	Axle 1-2	Axle 2-3	Axle 3-4	Axle 4-5	Axle 5-6	Axle 6-7	Axle 7-8	Axle 8-9	Axle 9-10	Axle 10-11	Axle 11-12	Axle 12-13	Axle 13-14
2	23	44	26	26	-	-	-	-	-	-	-	-	-	-	-	-
3	177	67	42.5	34	23.5	-	-	-	-	-	-	-	-	-	-	-
4	214	90	64.833	26.3	32.5	38.8	-	-	-	-	-	-	-	-	-	-
5	6881	110	215.92	178	36.7	114	38	-	-	-	-	-	-	-	-	-
6	11311	294	148	33	35.5	119	65	18.7	-	-	-	-	-	-	-	-
7	16272	154	142.75	31.1	30	79	110	34	16	-	-	-	-	-	-	-
8	6742	176	145.58	35	35	50.7	94	52	31.8	39.1	-	-	-	-	-	-
9	1315	196	135.25	24.8	11.5	58	86	63.5	42.5	39.4	22	-	-	-	-	-
10	704	216	175.5	26.2	10.5	40.1	88.5	69	89	48.7	20.9	16.2	-	-	-	-
11	807	239	175	25.5	5.25	18.2	35	35	104	60.1	16.8	18	13.83	-	-	-
12	893	400	191.5	24.7	5.25	42.5	26.9	9.08	90	59.9	42.7	22.5	17.5	12	-	-
13	1206	412.6	205.25	25.5	6	19.3	42	15.1	5.08	120	5.17	15.1	22.42	15.08	5.083	-
14	7	438	192.75	15.3	5	5	16.8	5.75	12.3	100	59.6	5.75	16.92	5.75	12.67	5.75

(b)

Year 2015				Min. axle spacing length (ft)												
No. of Axles	No. of records	Min GVW (kips)	Min length (ft)	Axle 1-2	Axle 2-3	Axle 3-4	Axle 4-5	Axle 5-6	Axle 6-7	Axle 7-8	Axle 8-9	Axle 9-10	Axle 10-11	Axle 11-12	Axle 12-13	Axle 13-14
2	23	33	16.5	16.5	-	-	-	-	-	-	-	-	-	-	-	-
3	177	44	4	4	2.25	-	-	-	-	-	-	-	-	-	-	-
4	214	23	4	4	3.42	3	-	-	-	-	-	-	-	-	-	-
5	6881	36.2	4	4	2.17	2	2.42	-	-	-	-	-	-	-	-	-
6	11311	50	4	4	1.67	4	2	-	-	-	-	-	-	-	-	-
7	16272	47.18	4.1667	4.17	3.5	3	2.5	3	3.67	-	-	-	-	-	-	-
8	6742	72	1.4167	1.42	3.08	3	3	3	3	1.5	-	-	-	-	-	-
9	1315	62.34	4.9167	4.92	4	4	4	4.17	4	4	4	-	-	-	-	-
10	704	67	5.4167	5.42	3.5	4.17	4.08	4	4.08	4.08	4.08	4	-	-	-	-
11	807	80	9.6667	9.67	3.67	4.17	4.08	4	4.5	4.17	4.17	4.33	4	-	-	-
12	893	76	10.5	10.5	3.5	4	4.08	4	4	4.5	4.33	4.25	4.333	4.083	-	-
13	1206	88.58	11.583	11.6	3.75	4	4.92	4.08	4.08	4.92	4.17	4.17	4.5	4	4.167	-
14	7	273	13	13	4.33	4.33	15.8	5	5	5.75	5	5	12.33	5	5	5

The WIM database for years 2014 and 2015 was run through the procedure, as shown in the flowchart of Figure 5.14. The results are summarized in Table 5.8 for the year 2014 and 2015. A comparison of the permits identified in the traffic stream (Table 5.8) to the vehicles that require permits (Table 5.1 and Table 5.2) for the respective WIM stations is possible. In the years 2014 and 2015, there were 1,006,237 and 810,868 vehicles that require permits, out of which 28,171 and 20,998 vehicles operated with a permit, respectively. These numbers indicate that less than 2.7% of overweight vehicles operate with a permit. The total number of WIM overloaded vehicles requiring permits of all the WIM locations is substantially higher than the number of permits that were issued.

Table 5.8. Summary of identification of issued permitted vehicles by the data-driven procedure for year 2014 and year 2015.

WIM station	Issued permits identified in WIM database	
	Year 2014	Year 2015
911	715	580
931	14,548	12,520
933	612	462
934	2,221	1,249
942	1,163	805
960	4,066	4,769
961	3,612	438
964	1,234	175
Total	**28,171**	**20,998**

5.5.3 Discussion of procedures to identify issued permit vehicles

Two procedures are presented and discussed in the previous section. GIS routing procedure is discussed in section 5.5.1 and data-driven procedure in section 5.5.2. GIS routing procedure uses the information of authorized routes and visualization of routes on the GIS mapping software such as Google Maps API. This procedure is more accurate as of the each issued permit truck routes and WIM stations it has passed through is identified. The main disadvantage is the complexity of the technology used and difficulty of adopting it in the routine basis of any DOT's.

The data-driven procedure is the more simplistic procedure, and the data can be analyzed with any analytical software such as Matlab (*MATLAB 2018a*). The disadvantage of this procedure is the inability to identify whether a particular issued permit truck traveled through a WIM station or not.

Table 5.9 shows the comparison of the procedures for the years 2014 and 2015. On average, 0.5 % of overladed vehicles by GIS routing procedure and 2.7 % of overloaded vehicles by the data-driven procedure are identified as vehicles with permits. Even though the relative difference of vehicles that have permits among the procedures is high, the percentage of vehicles that are illegal in each WIM station location is consistent. Since the GIS routing procedure is more accurate, in the rest of the dissertation the results of this procedure are used.

Table 5.9: Comparison of results of GIS routing and data-driven procedure (a) Year 2014 (b) Year 2015.

(a)

Year 2014	Legal vehicles	Overloaded vehicles	GIS – Permit vehicles		GIS – Illegal vehicles		Data-driven - Permit		Data-driven – Illegal vehicles	
			No.	%	No.	%	No.	%	No.	%
911	334,803	23,036	225	1.0%	22,811	99.0%	715	3.1%	22,321	96.9%
931	957,109	626,987	1000	0.2%	625,987	99.8%	14548	2.3%	612,439	97.7%
933	401,926	25,548	69	0.3%	25,479	99.7%	612	2.4%	24,936	97.6%
934	148,926	20,325	86	0.4%	20,239	99.6%	2,221	10.9%	18,104	89.1%
942	748,425	3,8507	206	0.5%	38,301	99.5%	1,163	3.0%	37,344	97.0%
960	236,972	68,381	339	0.5%	68,042	99.5%	4,066	5.9%	64,315	94.1%
961	682,210	147,736	1696	1.1%	146,040	98.9%	3,612	2.4%	144,124	97.6%
964	586,321	55,717	113	0.2%	5,5604	99.8%	1,234	2.2%	54,483	97.8%
Total	**4096,692**	**1006,237**	**3734**	**-**	**1002,503**	**-**	**28171**	**-**	**978,066**	**-**

(b)

Year 2015	Legal vehicles	Overloade d vehicles	GIS – Permit vehicles		GIS – Illegal vehicles		Data-driven – Permit vehicles		Data-driven – Illegal vehicles	
			No.	%	No.	%				No.
911	331,868	18,624	165	0.9%	18,459	99.1%	580	3.1%	18,044	96.9%
931	865,668	645,751	934	0.1%	644,817	99.9%	12,520	1.9%	633,231	98.1%
933	378,482	17,434	73	0.4%	17,361	99.6%	462	2.6%	16,972	97.4%
934	98,328	13,777	76	0.6%	13,701	99.4%	1,249	9.1%	12,528	90.9%
942	655,024	33,956	193	0.6%	33,763	99.4%	805	2.4%	33,151	97.6%
960	213,318	68,895	192	0.3%	68,703	99.7%	4,769	6.9%	64,126	93.1%
961	105,786	9,552	1193	12.5%	8,359	87.5%	438	4.6%	9,114	95.4%
964	132,931	2,879	125	4.3%	2,754	95.7%	175	6.1%	2,704	93.9%
Total	**2781,405**	**810,868**	**2951**	**-**	**807,917**	**-**	**20,998**	**-**	**789870**	**-**

5.6 Collected and uncollected revenue from overweight vehicles

A comparative analysis was made between the collected revenue by issuing a permit with uncollected revenue if all the overloaded traffic passing through that particular WIM location did not have permits. The total number of issued OS/OW permits is 123,602 and 122,539 in 2014 and 2015, respectively. Using the special data sorting technique, only OW permits were sorted (Chapter 5.4.1) and using the GIS routing technique (Chapter 5.5.1), the issued OW permit vehicles that passed through the WIM stations were identified. The WIM data was sorted using the state's TS&W to filter out only overloaded trucks (Table 5.1 and Table 5.2). The total permit fees that all the overloaded traffic is supposed to pay is calculated using the state's current permit fee structure (Chapter 5.4). The analysis results are shown in Figure 5.15 for the year 2014. This simple analysis shows the potential of increasing the revenue to state DOT by weight enforcement or developing a permit fee structure based on consumption cost.

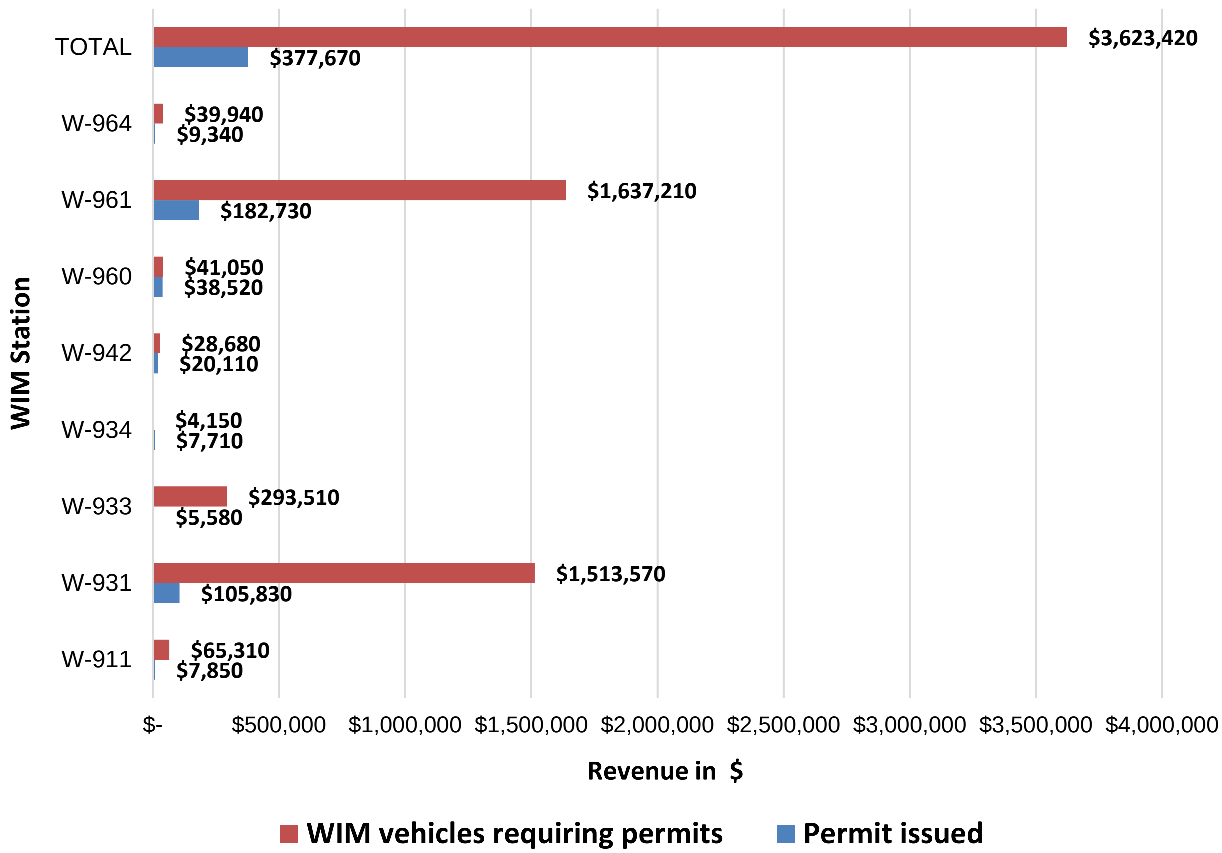


Figure 5.15: Collected and uncollected revenue from overloaded vehicles in year 2014.

5.7 Summary

This chapter discusses a procedure to identify the permit vehicles in the WIM data. Various literature related to the identification of issued permit data was reviewed. The first step is the separation of legal traffic so that the remaining file includes only overloaded vehicles, i.e. permit vehicles and illegal traffic. Then two procedures were introduced to identify permit vehicles in the WIM database. The remaining vehicles can be considered as illegal traffic. The issued permit truck identification procedures, GIS routing procedure and the data-driven procedure, is illustrated for the traffic data for the years 2014 and 2015. On average, 0.5 % of overloaded vehicles by GIS routing procedure is identified as vehicles with permits. The developed procedure depends primarily on the authorized routes of issued permit vehicles and the accuracy of WIM measurements. The soundness of the procedure can be improved if authorized routes of issued permit vehicles were available in the form of geo coordinates. This procedure is not practical for implementation by state transportation agencies such as ALDOT at this stage due to complexity in the technology and high computational skills.

The data-driven procedure is simpler and can be easily adopted by state transportation agencies. However, it is not accurate as a GIS routing procedure, but there is a vast majority of vehicles that can be easily identified as illegal. On average, 2.7% of overloaded vehicles by the data-driven procedure is identified as vehicles with permits. For the further chapters in this dissertation, the results of the GIS routing procedure are used.

Chapter 6: Bridge Damage Accumulation

6.1 Introduction

The service life of a bridge is affected by many factors such as, but not limited to, traffic loads, natural hazards, defects in material production. Traffic-induced loads may cause damage to a bridge by fatigue and/or overload. Steel bridges are more prone to fatigue cracking compared to other types of bridges, so steel bridges are the focus of this research.

Every passage of a truck across a bridge creates one or more stress cycles in the structural components, which results in the accumulation of fatigue damage over time. A steel bridge located on a busy highway experiences millions of cycles of fatigue loading by heavily loaded trucks. If these stress cycles are of sufficient magnitude and number, they will result in fatigue cracking.

The entire fatigue process in a member includes the formation of a fatigue crack, crack growth, and final failure (Fisher et al. 1998). The number of stress cycles required for the formation of a fatigue crack is typically much larger than the number of cycles required to grow a crack to a size that will cause failure. After formation, if a fatigue crack is not detected and properly repaired, it may lead to failure of the member. So, in broad terms, the passage of each heavy truck uses a tiny amount of the fatigue life of a bridge. In this chapter, the goal is to quantify the damage produced by an individual truck and the accumulated damage resulting from many trucks.

AASHTO LRFD Bridge design specifications (AASHTO 2017) have a design approach for fatigue. The stress range calculated for a code-specified fatigue design truck is limited to avoid fatigue cracking caused by the accumulation of damage from repetitive truck loading. The AASHTO fatigue design truck is intended to represent truck traffic. However, in the service life of a bridge, there is the uncertainty of the traffic loads that the bridge experiences. This chapter addresses the fatigue damage accumulated by bridges as a result of actual heavy truck traffic recorded at WIM sites. Background information is provided along with a review of the state-of-the-art literature and the practices in the other states. Further, the methodology used in this report and the implementation of the developed procedures are discussed.

6.2 Background

National Steel Bridge Alliance's (NSBA) *A Fatigue Primer for Structural Engineers* defines fatigue as the initiation and propagation of microscopic cracks into macro cracks by the repeated application of stresses (Fisher et al. 1998). Fatigue cracks tend to form at discontinuities or changes in geometry or cross section. Welded attachments such as web stiffeners or the end of a flange cover plate are considered fatigue prone details, or potential locations for the formation of fatigue cracks. Since a steel bridge experiences repeated cyclic stress, a suitable model of fatigue resistance is needed to evaluate the cyclic performance of the fatigue-prone details (referred to herein simply as details).

There are three most common approaches for the assessment and design of components to resist fatigue loading. The three approaches are the nominal-stress approach, local-stress life approach, and fracture mechanics approach. Nominal-stress is the simplest approach and is being used in many design specifications. By the use of simple equations, the fatigue resistance and applied stress are calculated using nominal stress near the detail for the bending and axial load of the member (Russo et al. 2016).

Stress range to a number of load cycles to failure (S-N) curves are developed based on testing for various detail categories.

The local-stress approach is similar to the nominal-stress approach, but the S-N curves are developed by either measuring using strain gauges or by refined finite element models, thus having a single baseline S-N curve. This approach increases the complexity of the analysis (Fisher et al. 1998; Russo et al. 2016).

The fracture mechanics approach is used to estimate the propagation life of the component and makes a distinction between crack initiation and propagation. In the case of bridge structures, a linear elastic fracture mechanics approach is used and is based on the theory of elasticity. It is difficult to estimate the initial crack size and the magnitude of stress intensity factors for complicated component geometry. The nominal-stress life approach is used in *AASHTO LRFD Bridge design specifications* (AASHTO 2017).

Two sources of fatigue, load-induced, and distortion-induced fatigue, are considered in *AASHTO LRFD Bridge design specifications* (AASHTO 2017). Load induced fatigue is described as "*The fatigue limit state shall be taken as restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles*" and further explained in commentary as "*The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge*" (AASHTO 2017).

Distortion-induced fatigue occurs when proper detailing practices are not followed, and fatigue cracking is seen due to strains that are not accounted for in the design process (AASHTO 2017). Usually, proper detailing practices are followed to avoid distortion-induced fatigue. However, this type of failure cannot be anticipated. Only load-induced fatigue is of interest in this dissertation, and it is discussed in detail.

In *AASHTO LRFD Bridge design specifications* (AASHTO 2017) the fatigue design is based on the following assumption (AASHTO 2017; Russo et al. 2016):

- Load-induced fatigue
    1. The force effect of live load stress range shall be considered in fatigue design.
    2. Residual stress shall not be considered in fatigue design.
    3. Only the details subjected to net applied tensile stress shall be designed for fatigue.
- Distortion-induced fatigue
    1. Proper detailing shall be provided to transmit all intended and unintended forces from transverse members by connecting to longitudinal members by means of welding or bolting to achieve the load path.

6.2.1 History of AASHTO/ AASHO fatigue design provisions

The evolution of fatigue design dates back to 1930's where the steel bridges were connected by rivets and extensive studies were done by Professor W.M. Wilson at the University of Illinois (Wilson and Coombe 1939). The fifth edition of the American Association of State Highway Officials (AASHO) Standard Specifications for Highway Bridges (AASHO 1949) in 1949 included provisions only related to welding on low carbon steel and wrought iron according to specification of American Welding Society (AWS 1936). The timeline of fatigue design provisions is shown in Figure 6.1.

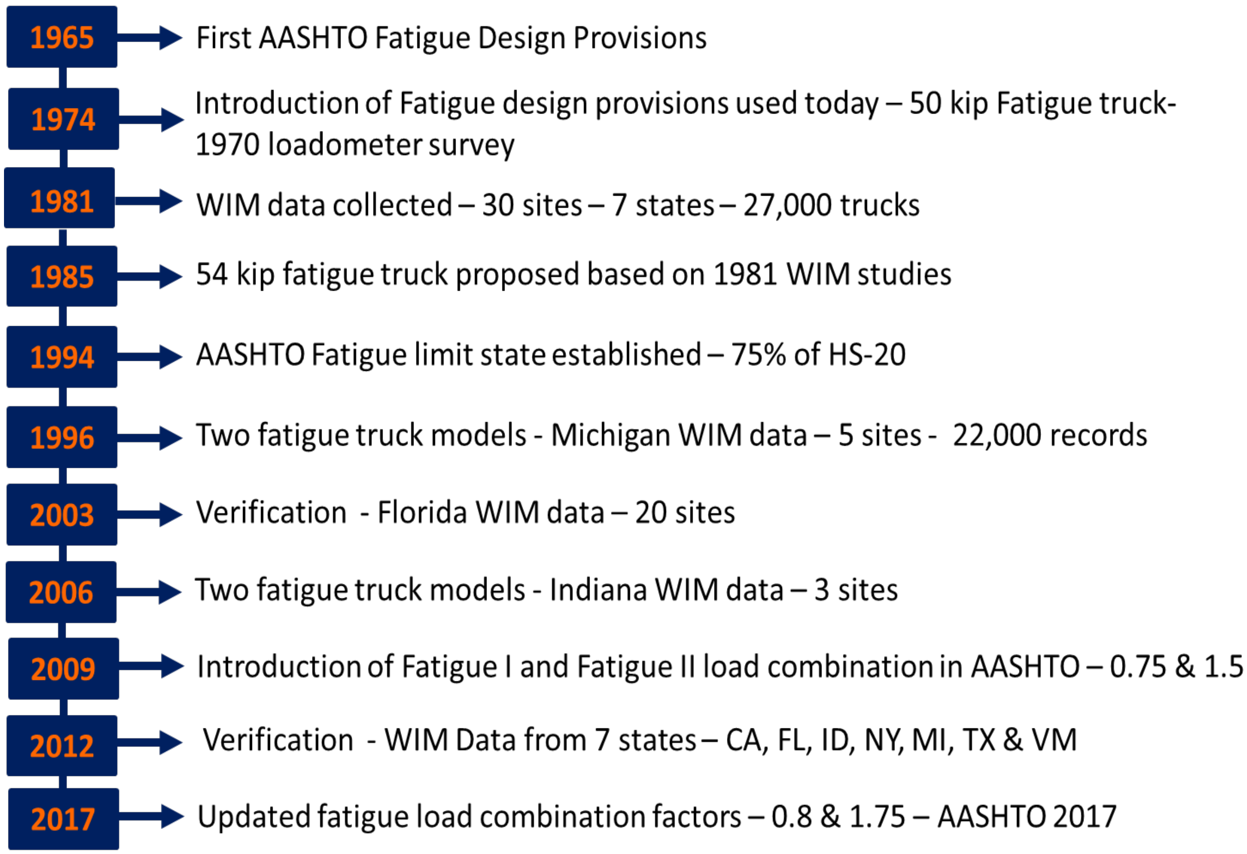


Figure 6.1: Timeline of fatigue design provisions in design specifications.

The first AASHO fatigue design provisions were introduced in the ninth edition of the AASHO Standard Specifications for Highway Bridges (AASHO 1965) in 1965. It had various fatigue categories and fatigue design provisions. From 1960's to 1972, there were many tests conducted by Professor John W. Fisher at Leigh University through NCHRP project 12-7 that resulted in NCHRP Report 102 (Fisher et al. 1970) and NCHRP Report 147 (Fisher et al. 1974). The fatigue strength of weldments was defined using the exponential relationship between stress range and fatigue life, as shown:

$$N = \frac{A}{S_r^3} \tag{6.3}$$

where,

N = Number of cycles to failure

A = constant for a given category of fatigue details

S_r= stress range

The equation shows a linear relationship on a log scale, as shown in Figure 6.2. The tests at a very low-stress range called threshold stress range, S_{Rth} revealed no cracking occurs, which is the important characteristic of fatigue resistance. There is no crack formation if the stress range is below this threshold, so it is deemed to have infinite fatigue life (Fisher et al. 1983).

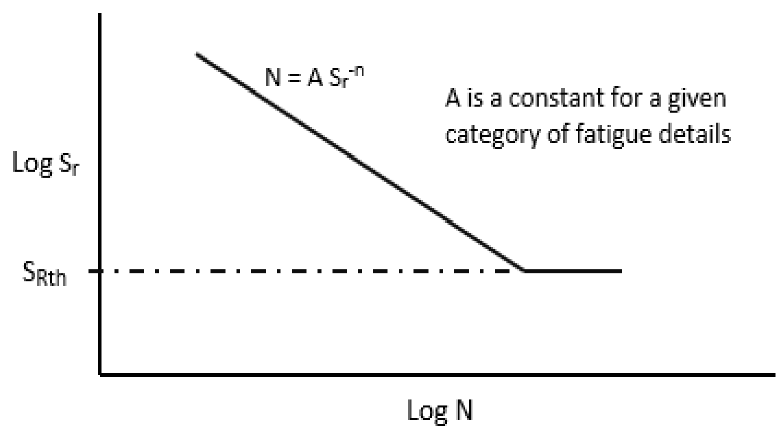


Figure 6.2: Fatigue S-N curve.

The current fatigue design concepts that are being used were introduced in 1974 (Fisher, J. W. 1974) interim specifications to the AASHTO Standard Specifications for Highway Bridges (AASHO 1974). The fatigue categories A through F for fatigue resistance were introduced then. From 1974 until 1994, the AASHTO standard specifications used HS-20 design vehicles without any modifications for both strength and service conditions.

In 1994, the fatigue was defined as its own limit state in the AASHTO LRFD Bridge Design Specifications (AASHTO 1994). The design vehicle was changed to 75 % of HS-20 vehicles which dates to 1978 (Schilling, C. G. and Klippstein, K. H. 1978) based on FHWA's loadometer survey (Fisher, J. W. 1974) measured in 1970. It was a 3-axle truck with 14 ft and 30 ft axle spacing and a GVW of 50 kips distributed at 0.122, 0.444 and

0.444 of GVW for axles 1, 2 and 3 respectively. Later in 1981, in NCHRP 299 (Moses et al. 1987) based on 27,000 WIM measurements from 30 sites nationwide (California, Oregon, Michigan & New York), the GVW was modified to 54 kips without any modification of axle configurations.

Equation 6.4 is the formula to calculate an effective truck weight for fatigue truck if the data from the WIM study is available for a particular bridge site (Fisher 1984; Fisher, J. W. 1974). The truck traffic, excluding panel, pickup, and other 2-axle/4-wheel trucks are considered for calculating effective GVW. Truck traffic from Class 6-13 is considered for calculating effective truck weight.

$$W = \left(\sum \frac{1}{T} * W_i^3\right)^{\frac{1}{3}} \qquad (6.4)$$

where,

W = gross weight of fatigue truck

T = total number of trucks

W_i = gross weight at mid-width of interval i

From 1994 until 2009, the fatigue truck was validated or was propsed by using state-specific data. Laman et al. (Laman and Nowak 1996) used Michigan WIM data from 5 sites containing 22,000 records to develop new fatigue trucks. Also, in 2006, two fatigue truck models were introduced by using Indiana WIM data (Chotickai Piya and Bowman Mark D. 2006). In 2009 interim changes to *AASHTO LRFD Bridge Design Specifications*, the Fatigue limit state I and II combinations were introduced. Later in 2012, by using WIM data from 7 states (California, Florida, Idaho, New York, Michigan, Texas & Vermont) (Bowman et al. 2012), the fatigue truck was validated.

In the modern design codes, the safety reserve is provided by load and resistance factors through a reliability-based calibration process (Nowak 1999). Through NCHRP project 12-33 resulting in NCHRP Report 368 (Nowak 1999), only strength limit states were calibrated. The fatigue limit state was not calibrated until recently. It was uncalibrated though a deemed level of safety and limit state was based on testing to define safety resistance values (Russo et al. 2016). Through the SHRP2 R19B project *Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Kulicki et al. 2015), the service and fatigue limit states were calibrated. The study used the WIM database to develop fatigue load models. More in detail about this study is discussed in Chapter 7.

In 2017, the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017) was updated with new factors for the fatigue limit state. 1.75 is used for Fatigue limit state I and 0.80 for Fatigue limit state II but still the HL-93 truck that is a 3-axle truck with 14 ft and 30 ft axle spacings is used. In the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017) for load-induced fatigue considerations, each detail should satisfy the fatigue limit state formulation as expressed in Equation 6.5 (AASHTO 2017, Equation 6.6.1.2.2-1) as:

$$\gamma(\Delta f) \leq (\Delta F)_n \quad (6.5)$$

For the Fatigue I limit state for a steel detail to have an infinite life, the factored design stress must be below the constant amplitude fatigue threshold (CAFT) as shown in Equation 6.6 (AASHTO 2017, Equation 6.6.1.2.5-1) as:

$$\gamma(\Delta f) \leq (\Delta F)_n = CAFT \quad (6.6)$$

For the Fatigue II limit state for a steel detail in the infinite life region, the factored design stress must be below the finite life resistance, as shown in Equation 6.7 (AASHTO 2017, Equation 6.6.1.2.5-2) as:

$$\gamma(\Delta f) \leq (\Delta F)_n = \sqrt[3]{A/N} \tag{6.7}$$

and

$$N = (365)(75)n\,(ADTT)_{SL} \tag{6.8}$$

where,

γ = load factor for fatigue load combination (AASHTO 2017, Table 3.4.1-1)

(Δf) = force effect, live load stress range due to passage of fatigue load (AASHTO 2017, Article 3.6.1.4)

$(\Delta F)_n$ = nominal fatigue resistance (AASHTO 2017, Article 6.61.2.5)

CAFT = constant amplitude fatigue threshold

A = detail category constant (AASHTO 2017, Table 6.6.1.2.5-1)

n = number of stress range cycles per truck passage (AASHTO 2017, Table 6.6.1.2.5-2)

$(ADTT)_{SL}$ = single-lane average daily truck traffic (AASHTO 2017, Article 3.6.1.4)

6.3 Literature Review

The study of the impacts of vehicular traffic on infrastructure has been conducted in many states. The earliest study dates back to the 1970s. Many states have sponsored studies to develop methodologies to quantify damage and do the cost analysis based on assumed cost models. The cost impact study for Indiana DOT was done in 1979 by Yoder et al. 1979 to study the impact on bridges and pavements due to a GVW limit increase from 73.28 kips to 80 kips. A study for New York State DOT was conducted in 1987 by the BTML Division of Wilbur Smith Associates 1987 on the effect of permit truck weights on bridges. In 1991 the Minnesota DOT (Minnesota DOT 1991) conducted a study in response to TRB Special report 225 (Board et al. 1990) to investigate bridge-related impacts. A study for Illinois DOT (Illinois Department of Transportation, 1992) was conducted to study the impact on bridges due to a weight limit change. Also in 1992, Sorensen and Robledo, 1992 conducted a study for Washington State DOT to estimate the impact of the Turner trucks on the state's bridges. A study for Ohio DOT by Moses, 1992, was done to develop a permit fee system based on bridge damage costs. Other studies were conducted on fatigue life of double angle tension members and diaphragm-girder connections (Stallings et al. 1996, 1997).

Beginning in the 21st century, many states sponsored overweight load studies. In 2004, Culmo et al. 2004 did a study on the behavior of steel bridges under specific permit trucks for Connecticut. Reisert and Bowman conducted a study on the fatigue response of older steel bridges to overweight and oversized loads in Indiana in 2005. Also, in 2005, a study for Louisiana was done by F. L. Roberts et al. 2005 on the effects of specific commodities transporting vehicles on Louisiana infrastructure. Later in 2012, a multi-phase study in Wisconsin was done by (H. Bae and M. Oliva 2009, 2012), where the impact of overweight vehicles was studied. Laboratory tests and numerical simulations were performed for deck deterioration as part of the study.

Almost all state DOT's have sponsored studies to determine the impact of overweight traffic on infrastructure. However, the recent studies done in Texas, South Carolina, New York, and New Jersey give more insight into the modern approach and use state-of-the-art practice. Also, the study sponsored by FHWA on the effect of truck weight on bridge network costs uses an innovative approach to envelope all states. These five sources are described in the following sections.

Effect of Truck Weight on Bridge Network Costs (Gongkang Fu et al. 2003)

This study was sponsored by AASHTO and FHWA with an objective to develop a methodology to estimate bridge network cost due to changes in truck weight limits. Based on the state of the practice literature review four-cost impact categories such as fatigue of existing steel bridges, decks, and deficiency due to overstressing. Also, deficiency due to overstress of new bridges was considered. Level I and Level II analyses were proposed based on the extent of data availability.

Oversize/Overweight Vehicle Permit Fee Study (Prozzi et al. 2012)

This study was done for the State of Texas and was sponsored by the Texas Department of Transportation (TxDOT) and the Federal Highway Administration. The objective of this study was to conduct a study of infrastructure damage caused by oversized and overweight vehicles (OS/OW) and to provide recommendations for permit fee adjustments if required. A methodology to quantify the pavement and bridge consumption rate per mile was developed as part of the project. Also, the new fee schedule was developed to account for the costs associated with OS/OW vehicles. Also, a revenue analysis was conducted to compare the revenue generated from permit sales and the revenue estimates from the new permit fee structure. It was concluded that from the permit sales of the financial year 2011 the revenue collected was $111.4 million compared to the estimated revenue of $671.4 million from the revenue estimates based on the new permit fee structure.

The rate of Deterioration of Bridges and Pavements as Affected by Trucks (Chowdhury et al. 2013)

This study was done for South Carolina DOT to analyze the impact of heavy vehicle traffic on infrastructure and develop policy recommendations. Several alternative fee structures were proposed, such as axle-based system and flat fee. Stake holder interviews were done as part of the study.

Effects of Overweight Vehicles on NYSDOT Infrastructure (Ghosn et al. 2015)

This study was focused on the development of a methodology for estimating effects caused by heavy trucks on New York State infrastructure sponsored by New York State Department of Transportation (NYSDOT). In modeling the effects of overweight trucks on bridges, the overweight WIM traffic data was categorized to probable divisible permits, special hauling permits, and Illegals. The structural response to overweight vehicles in the traffic data was considered using overstress of main bridge members and cyclic fatigue accumulation. For estimating the effects on pavements, an incremental cost approach was used to compare the options of using an increase in design thickness of pavement layers and a possible increase in the maintenance schedule. The cost effect was calculated considering the bridge material and construction. The cost effect was studied on a representative sample of 22 bridges along the I-88 corridor in New York State. Based on a cost allocation study it was found that the total cost for entire New York state infrastructure is $240 million per year, with $95 million per year for bridge network and $145 million per year for pavements.

Impact of Freight on Highway Infrastructure in New Jersey (Nassif et al. 2015)

This study was conducted with an objective to quantify the effects of overweight vehicles on the New Jersey Infrastructure sponsored by the New Jersey Department of Transportation (NJDOT). A model was proposed based on a literature review of the effects of overweight vehicles from other states and deterioration models. A tool ASSISTME-WIM software was developed to estimate the actual damage cost on NJ highways due to overweight trucks. The Life Cycle Cost Performance Analysis (LCCA) was done, and it was estimated that average cost of moving one ton of load by an overweight truck per mile in NJ is about $0.33, where 40% of damage is attributed to bridges and 60% to pavements.

Table 6.1 shows the overall summary of the relevant studies related to the impact of overweight vehicles on highway infrastructure. The type of highway infrastructure, their mechanisms and input datasets that were used by various researchers are listed. The inventory of infrastructure maintenance by state transportation agencies is a valuable asset in all these studies.

Table 6.1: Summary of relevant studies on estimated cost of damage due to overweight traffic.

Source	State	Highway infrastructure	Mechanism considered	Dataset – Load side	Dataset – resistance side	Dataset – Cost analysis
Oversize/ Overweight Vehicle Permit Fee Study (Prozzi et al. 2012)	Texas	Pavement	Rutting, fatigue cracking, and roughness	OS/OW issued permits	-	1. TxDOT's average low bid price portal (unit cost of materials)
Oversize/Overweight Vehicle Permit Fee Study (Prozzi et al. 2012)	Texas	Bridge	Fatigue and different fatigue curves depending upon type of material	1.OS/OW issued permits 2. Non-routed permits	FHWA's National Bridge Inventory (NBI)	1. Current asset value of bridge - Texas 2030 Committee 2. Permit fees collected for FY 2011
Effects of Overweight Vehicles on NYSDOT Infrastructure (Ghosn et al. 2015)	New York State	Bridge	1. Overstress of main members 2. Cyclic fatigue accumulation in main members and decks	Weigh-In-Motion (WIM) data	1. FHWA's National Bridge Inventory (NBI) 2. "WINBOLTS" – database assembled by NYSDOT 3. Detailed bridge plans	RSMeans - "Heavy Construction Cost Data"
Effects of Overweight Vehicles on NYSDOT Infrastructure (Ghosn et al. 2015)	New York State	Pavements	Incremental cost approach	Weigh-In-Motion (WIM) data	NYS pavement database	RSMeans - "Heavy Construction Cost Data"
Impact of Freight on Highway Infrastructure in New Jersey (Nassif et al. 2015)	New Jersey	Bridges	Fatigue in steel bridge girders, pre-stressed bridge girder tendons and RC decks	Weigh-In-Motion (WIM) data	FHWA's National Bridge Inventory (NBI)	Unit cost of bridge construction from FHWA

6.4 Technical Approach

The nominal-stress life approach from the *AASHTO LRFD Bridge design specifications* (AASHTO 2017) is used. Fatigue resistance of a material and connection detail is usually presented with an S-N curve like one shown in Figure 6.3**.** The resistance relates the magnitude of the applied constant-amplitude stress range (S) to the corresponding number (N) of cycles to failure of the detail.

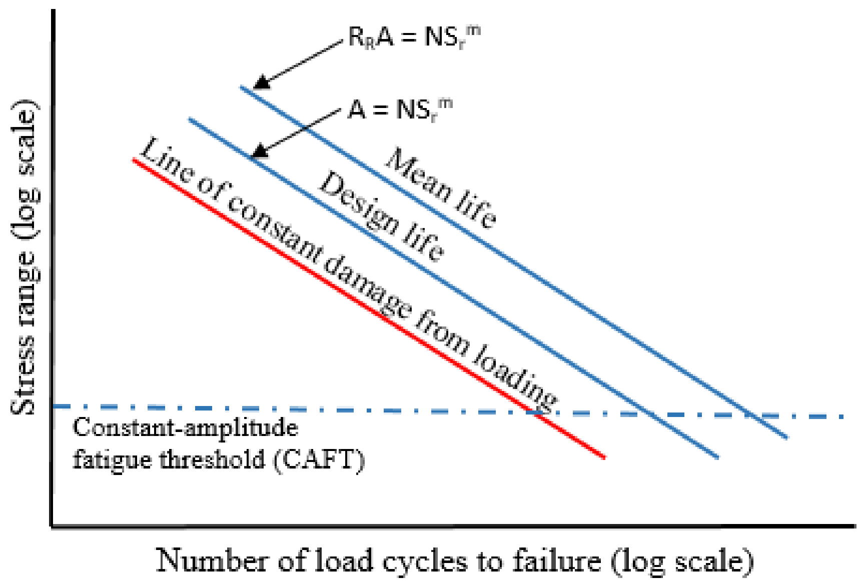


Figure 6.3: Fatigue life relationship.

A family of similar S-N curves for different detail categories was established by extensive laboratory testing and is included in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017) for various categories of fatigue details that are commonly used in bridge construction. These S-N relationships are shown in Figure 6.4. The stress range and fatigue life relationship is:

$$N = AS^{-m} \tag{6.9}$$

where,

m – slope constant (3 for steel)

S – nominal stress range

N – number of cycles to failure

A – constant for a given detail

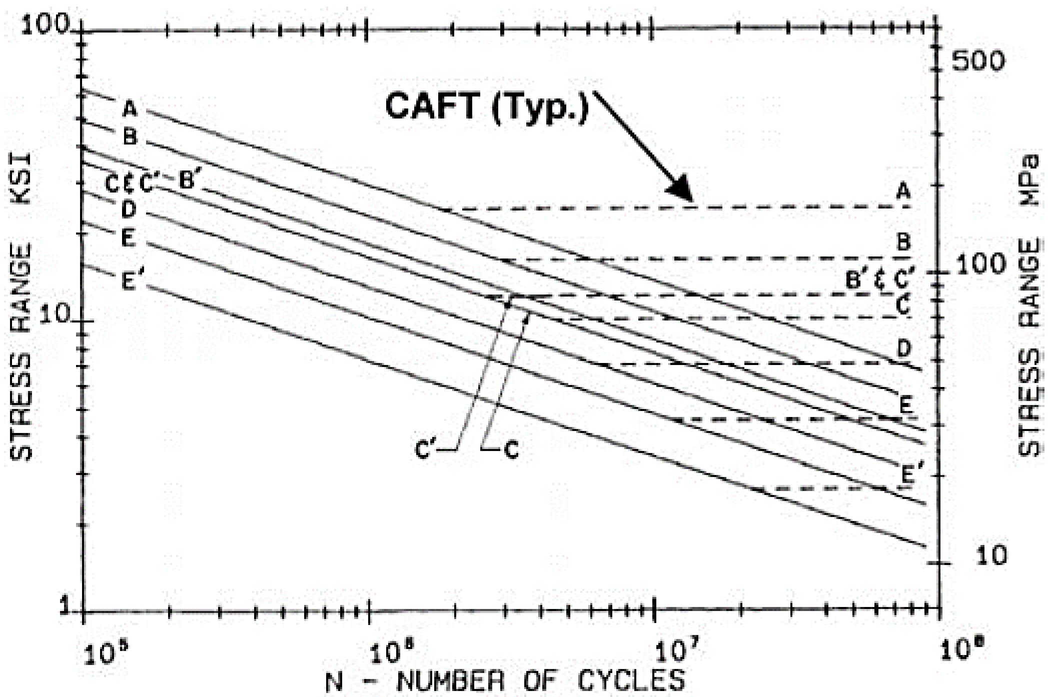


Figure 6.4: Stress Range versus Number of Cycles (S-N) curves (AASHTO 2017)

As illustrated by the categories in Figure 6.4, different details in a steel bridge have different lives, and as a result of the overall bridge geometry, individual details experience traffic-induced stress ranges of various magnitudes. So, some details accumulate fatigue damage faster, and their fatigue life is expended faster. (Franklin 2000) Performed an evaluation of a large number of steel bridge spans in Birmingham that are typical of the steel bridges on Alabama highways and interstates. Based on that study (Franklin 2000), the base metal at the end of the bottom flange cover plates was identified as the most fatigue prone detail of steel girder bridges. These are Category E' details and have the lowest fatigue resistance, and they also have relatively high applied stress ranges. In this chapter, much attention will be focused on the damage accumulated at the ends of the bottom flange cover plates on simple span girders. Traffic-induced stress ranges and damage accumulation at the upstream end of the cover plate is higher so that the end receives more attention. Also, for comparisons with past research and more generic comparisons, such as comparisons of damage from different classes of trucks, damage accumulation at midspan is considered. The most critical detail likely to be present at midspan would be a transverse stiffener-to-web fillet weld, or perhaps a transverse stiffener-to-flange weld in a newer bridge. These would be Category C' details as defined in *AASHTO LRFD Bridge Design Specifications* (AASHTO 2017).

An important question that must be addressed in the discussion of fatigue damage accumulation is: do all traffic-induced stress cycles contribute to the accumulation of damage and potential formation of a fatigue crack? Current U.S. practice is that all stress cycles are considered if even only a small percentage (Fisher et al. 1983) of the traffic-induced stress ranges are above the constant amplitude fatigue limit (CAFT) (see horizontal dashed lines in Figure 6.4). Pearson (2002) reported field measurements of stress ranges at cover plate ends of bridges in Birmingham that clearly show a sufficient number of stress cycles above the CAFT to cause fatigue cracking. It is assumed here that those bridges where the field measurements were made are representative of the steel bridges across the state, and all stress ranges at cover plate ends should be considered. But it is also common practice in the analysis of WIM data to omit light vehicles. In the work reported here, vehicles with gross vehicle weight less than 20 kips were omitted from the analyses. For simplicity, since all trucks heavier than 20 kips were considered in the damage accumulation calculations at cover plate ends, they were also considered at midspan locations.

The S-N curves for different detail categories are shown in Figure 6.4 (Figure C6.6.1.2.5-1 of (AASHTO 2017)). The S-N curves are like the S-N curve in Figure 6.3 and were developed using constant-amplitude stress range test data. However, bridges are subjected to variable amplitude stress cycles. A cumulative damage theory is used to calculate an effective stress range from variable amplitude stress cycles. The Palmgren-Miner (Miner 1945) rule provides a rational means to account for this cumulative damage for variable amplitude stress ranges as follows.

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \cdots + \frac{n_n}{N_n} = \sum \frac{n_i}{N_i} = 1 \qquad (6.10)$$

where, $\frac{n_i}{N_i}$ is the incremental damage that results from the stress range cycles with magnitude S_i that occur n_i times, and N_i is the number of cycles to failure at the constant amplitude stress range S_i. Some trucks in the WIM database create more than one cycle of loading as they cross a bridge. Processing of large amounts of WIM data is simplified by representing the fatigue damage produced by these multiple cycles of loading from a single truck as an equivalent single cycle of loading. In terms of stresses, the equivalent single cycle stress range S_{eff} for the set of truck is:

$$S_{eff} = \left[\sum \frac{n_i}{N} S_i^m\right]^{1/m} \qquad (6.11)$$

where,

m – slope constant (3 for steel)

n_i – number of cycles at i^{th} stress range, S_i

N – total number of stress cycles

The fatigue damage is the result of tensile stress ranges resulting from bending moment. The stress formulation can be extended to bending moment as shown in Eq. (6.12).

$$M_{eff} = \left[\sum \frac{n_i}{N} M_i^m\right]^{1/m} \quad (6.12)$$

where,

m – slope constant (3 for steel)

n_i – number of cycles at i^{th} moment range, M_i

N – total number of stress cycles

Each WIM record from the traffic database is run for different span lengths to obtain plots of bending moment versus time at a specific location as truck crosses along the span. Rainflow cycle counting (ASTM E1049—85 (Reapproved 2017)) was used to determine the number and magnitudes of the individual stress cycles resulting from the truck crossing. Rainflow counting is discussed in detail in section 6.4.1.

Various approaches were investigated for the calculation and reporting of fatigue damage accumulation so that the results would be useful for ALDOT in routine maintenance activities. Two approaches are used in this dissertation to quantify the damage. One approach is a WIM site-specific damage ratio, which is a relative measure of damage that can be generalized to any type of bridge. The other approach is specific to a particular bridge. Two approaches are discussed in detail in the next sections.

6.4.1 Rainflow cycle counting method

The purpose of the cycle counting methods is to reduce random and variable frequency amplitudes to the results that can be compared with the S-N curve curves obtained from constant amplitude testing (Cheng and Broz 1986). Bridges are subjected to cyclic loads that result in structural failures due to fatigue. The load cycles experienced by bridges due to traffic are random and have variable frequencies and amplitudes (Cheng and Broz 1986; Fisher, J. W. 1974). Determination of cycles and stress range for wide band stress histories that do not have a constant period is difficult. The bending moment time histories due to truck passage are wide band (Kulicki et al. 2015). A cycle counting method is required to count an actual number of cycles and stress range to be used in fatigue analysis.

ASTM has documented the methods for cycle counting in Fatigue Analysis in ATSM E1049-85 (ASTM E1049—85 (Reapproved 2017) n.d.). Some of the methods listed are range-pair counting, level-crossing counting, simple-range counting, peak counting, and rainflow counting. As the millions of WIM records are run over an influence line to obtain bending moment time histories, a robust procedure is required. Rainflow counting method was chosen as it is also easy to program on a computer.

A rainflow counting method was introduced in 1968 by Japanese scholars Matsuishi and Endo (Matsuishi and Endo 1968). It got its name from the metaphorical flow of rain drops down the many overlapping pagoda-like roofs. The edge of each roof is represented by peaks and valleys. A rainflow counting diagram ((Bannantine et al. 1990; Kulicki et al. 2015; Laman 1995) is shown in Figure 6.5 and rules are explained below :

1. Turn the sheet clockwise to 90^o such that the earliest time is at the top as shown in Figure 6.5.
2. Next, reduce the time history as a sequence of peaks (tensile) and throughs (compressive).
3. The number of half-cycles is counted by initiating a rainflow path from inside of each peak and through and imagined as a source of water that drips down and is allowed to flow unless:
    a. The rain began at a local maximum point (peak/tensile) and falls opposite a local maximum point greater than that it came.

b. The rain began at a local minimum point (valley /compressive), and it falls opposite a local maximum point greater (magnitude) than it came.

c. If it encounters a previous flow.

4. For each half-cycle, an amplitude (stress) difference between its start and termination is assigned as the magnitude (Table 6.2).
5. The complete cycle is counted by pairing up half cycles of identical magnitude (Table 6.3). Sometimes there are residual half cycles.

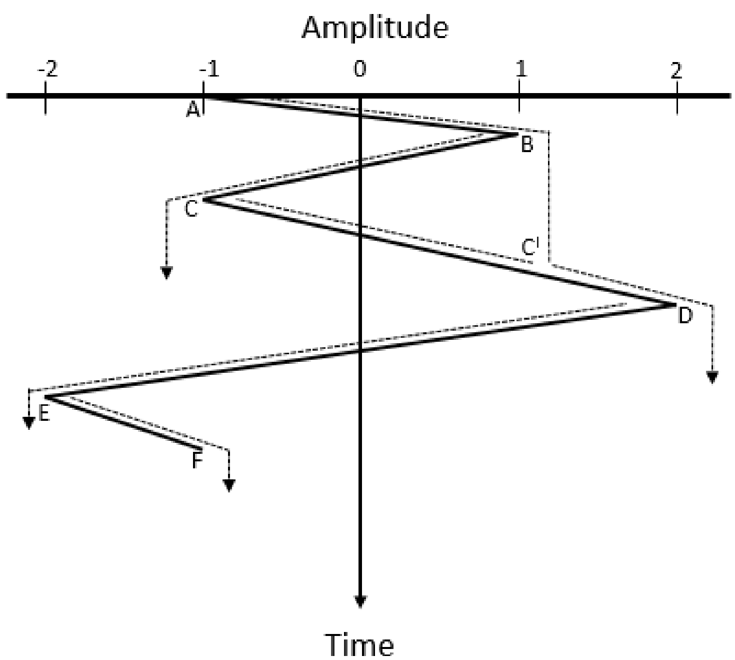


Figure 6.5: Illustration of a rainflow counting.

Table 6.2: Half cycle from rainflow counting.

Positive direction		Negative direction	
Range	**Amplitude**	**Range**	**Amplitude**
A-B-C'-D	3	B-C	2
C-C'	2	D-E	4
		E-F	1

Table 6.3: Complete load cycles.

Amplitude	No. of cycles
1	0.5
2	1
3	0.5
4	0.5

6.5 WIM Site-Specific Damage Ratio

This approach can help understand which WIM location experiences the most damaging traffic in the state of Alabama. Also, the damage caused by different groups of trucks in the traffic can be assessed. To provide comparisons using the largest amount of traffic data possible, the direction of traffic on both sides (Lane 1 & 2 and Lane 3 & 4) of WIM stations are combined. Each WIM record from the traffic database is run for 30, 60, 90, 120 and 200 ft span lengths to obtain plots of bending moment versus time at a specific location as truck crosses along the span. The specific location along the span length was limited to mid-span and upstream cover plate end along the span. The upstream cover plate end location rather than the downstream was used because the damage at the upstream cover plate end is higher. The amount of damage is calculated as shown in Equation 6.13 which is a modification of Equation 6.9.

$$\mathrm{D} = \mathrm{N}\mathrm{M}_{\mathrm{eff}}^{\mathrm{m}} \quad (6.13)$$

where,

D – amount of damage

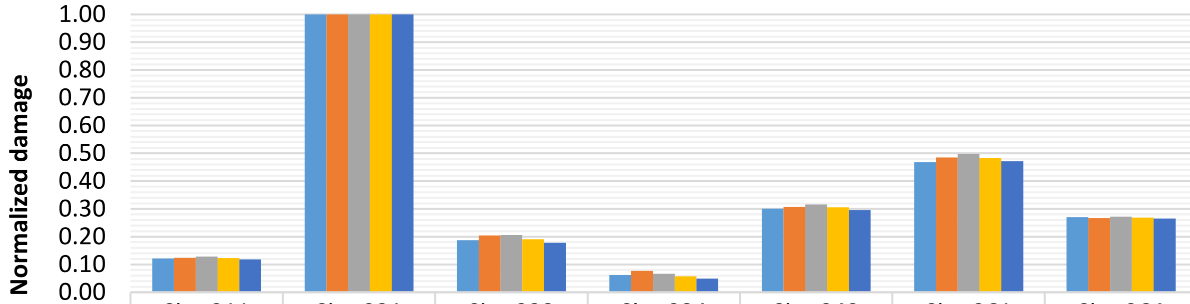
m – slope constant (3 for steel)

$\mathrm{M}_{\mathrm{eff}}$ – effective moment calculated as shown in Eq. 6.12

N – number of cycles

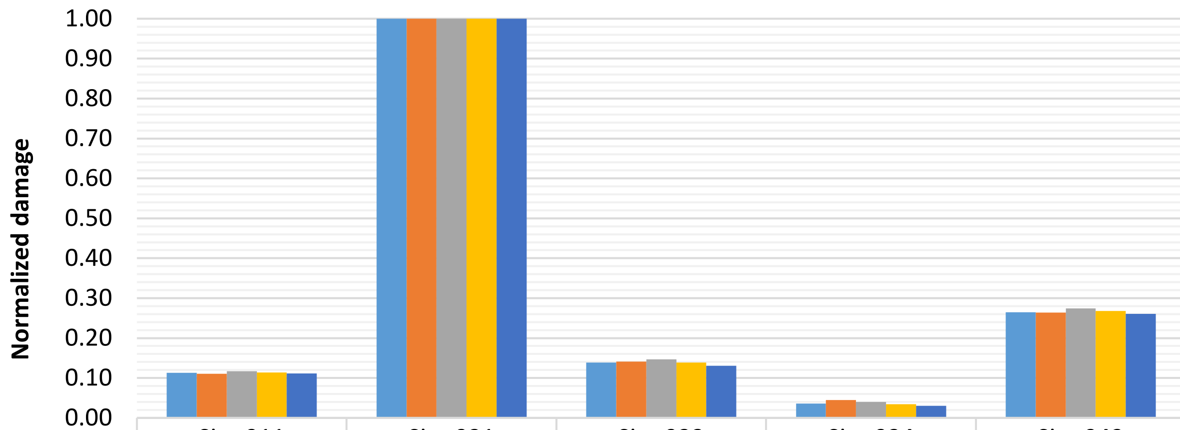
6.5.1 Comparison of damage of WIM Sites

The amount of damage (NM_{eff}^3) was calculated for each WIM site (911-964) for both the directions combined and for years 2014-2015 for upstream cover plate end and mid-span. The resulting damage (NM_{eff}^3) for each WIM location is calculated for 30, 60, 90, 120 and 200 ft span lengths and normalized to the WIM location with most damaging traffic. In this case, WIM location 931 has the most damaging traffic. The results for the upstream cover plate end are shown in Figure 6.6 and Figure 6.7 and for mid-span are shown in Figure 6.8 and Figure 6.9. The numbers of WIM records are shown in Figure 6.10 and Figure 6.11. In Figure 6.6 and Figure 6.8, for the year 2014, the most damaging traffic is WIM location 931 followed by WIM location 961 and 942. The damage accumulated by considering different span lengths is almost the same. In Figure 6.7 and Figure 6.9, for the year 2015, the most damaging traffic is WIM location 931 followed by WIM location 942 and 933. The data from WIM location 961 and 964 are eliminated for the year 2015 as data was not available for the whole year. Also, data for WIM location 960 is eliminated for the year 2014 and 2015 since the data was available only for one direction. The only consistent pattern in the relative damage at the various sites is that the damage at site 931 is at least twice that at the other sites for both 2014 and 2015. The normalized accumulated damage is practically independent of the location along the girder and span length.



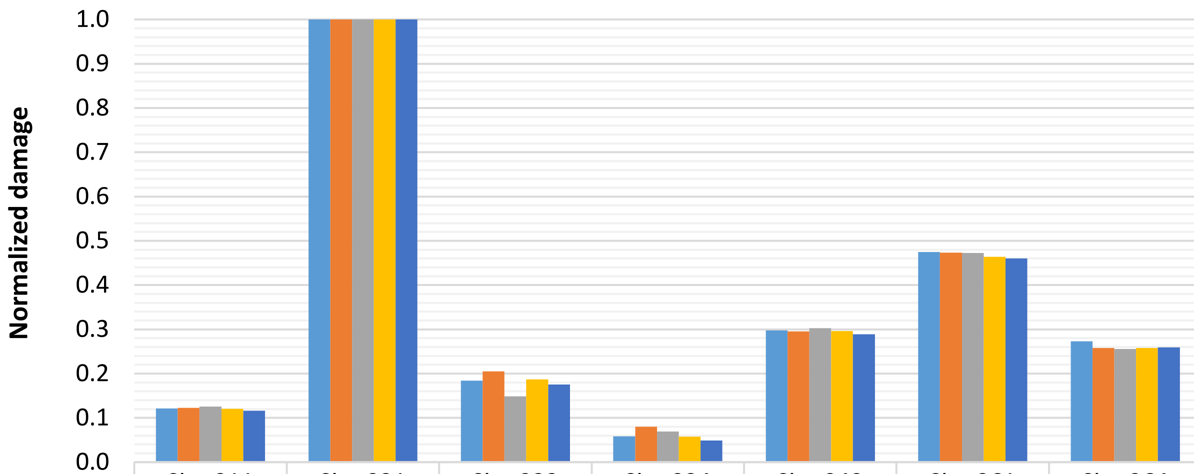
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 961	Site 964
30ft	0.12	1.00	0.19	0.06	0.30	0.47	0.27
60ft	0.12	1.00	0.20	0.08	0.31	0.48	0.27
90ft	0.13	1.00	0.21	0.07	0.32	0.50	0.27
120ft	0.12	1.00	0.19	0.06	0.31	0.48	0.27
200ft	0.12	1.00	0.18	0.05	0.29	0.47	0.27

Figure 6.6: Normalized accumulated damage in 2014 at the upstream cover plate end.



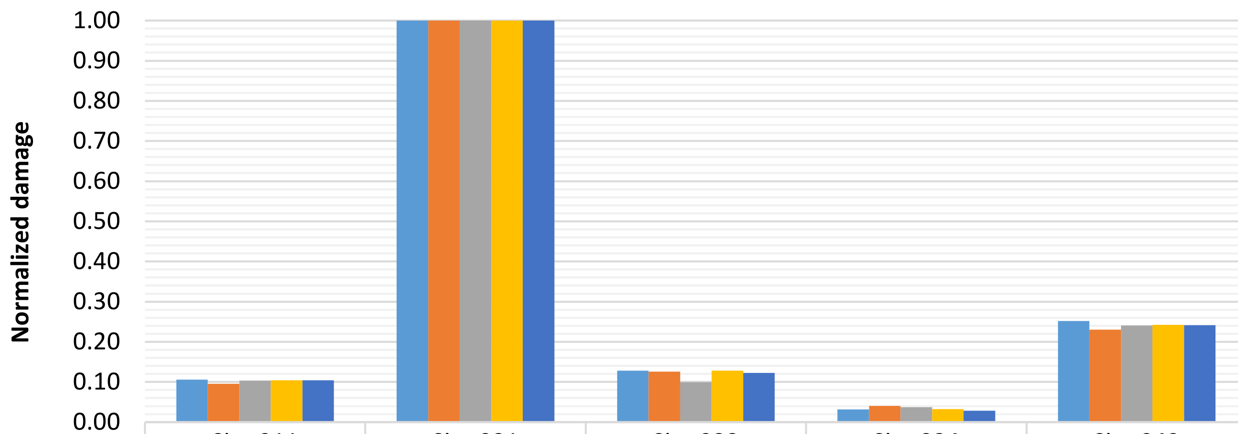
	Site 911	Site 931	Site 933	Site 934	Site 942
30ft	0.11	1.00	0.14	0.04	0.26
60ft	0.11	1.00	0.14	0.04	0.26
90ft	0.12	1.00	0.15	0.04	0.27
120ft	0.11	1.00	0.14	0.03	0.27
200ft	0.11	1.00	0.13	0.03	0.26

Figure 6.7: Normalized accumulated damage in 2015 at the upstream cover plate end.



	Site 911	Site 931	Site 933	Site 934	Site 942	Site 961	Site 964
30ft	0.12	1.00	0.18	0.06	0.30	0.47	0.27
60ft	0.12	1.00	0.21	0.08	0.30	0.47	0.26
90ft	0.13	1.00	0.15	0.07	0.30	0.47	0.26
120ft	0.12	1.00	0.19	0.06	0.30	0.46	0.26
200ft	0.12	1.00	0.18	0.05	0.29	0.46	0.26

Figure 6.8: Normalized accumulated damage in 2014 at mid-span.



	Site 911	Site 931	Site 933	Site 934	Site 942
30ft	0.11	1.00	0.13	0.03	0.25
60ft	0.10	1.00	0.13	0.04	0.23
90ft	0.10	1.00	0.10	0.04	0.24
120ft	0.10	1.00	0.13	0.03	0.24
200ft	0.10	1.00	0.12	0.03	0.24

Figure 6.9: Normalized accumulated damage in 2015 at mid-span.

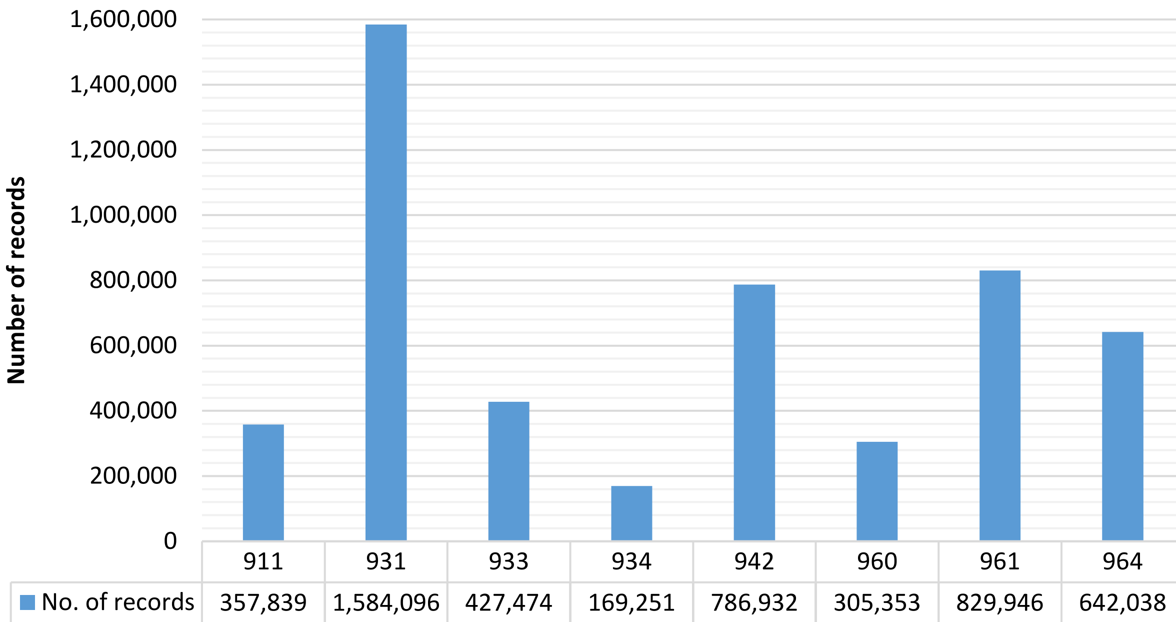


Figure 6.10: The number of WM records collected in 2014.

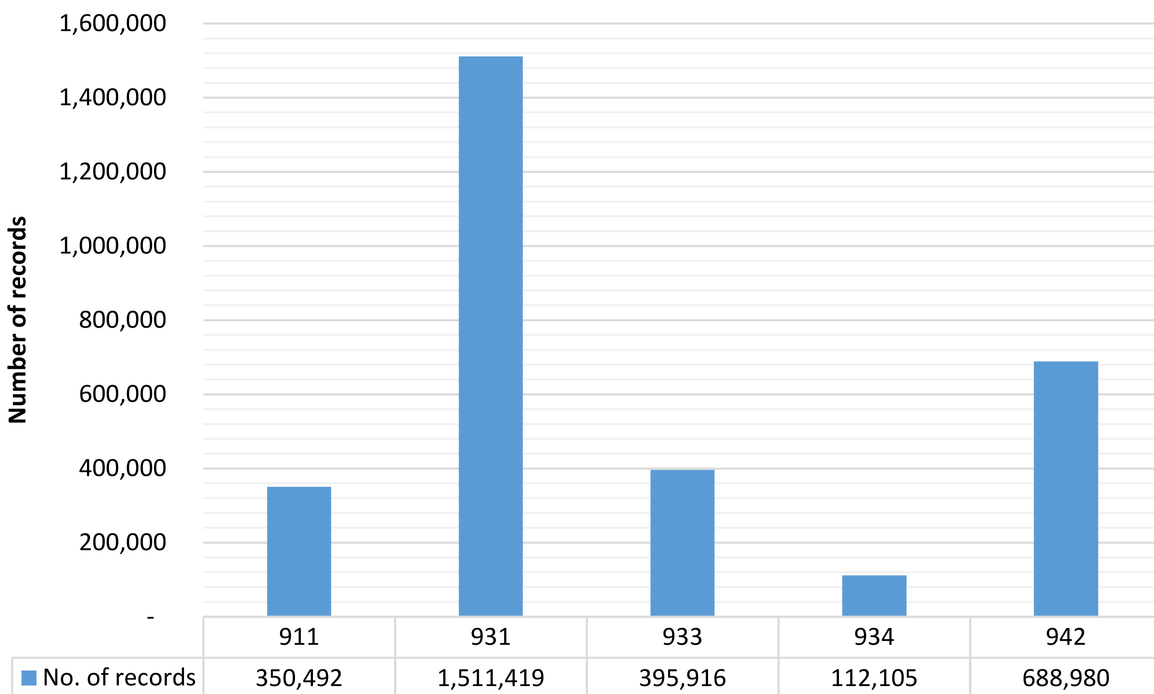
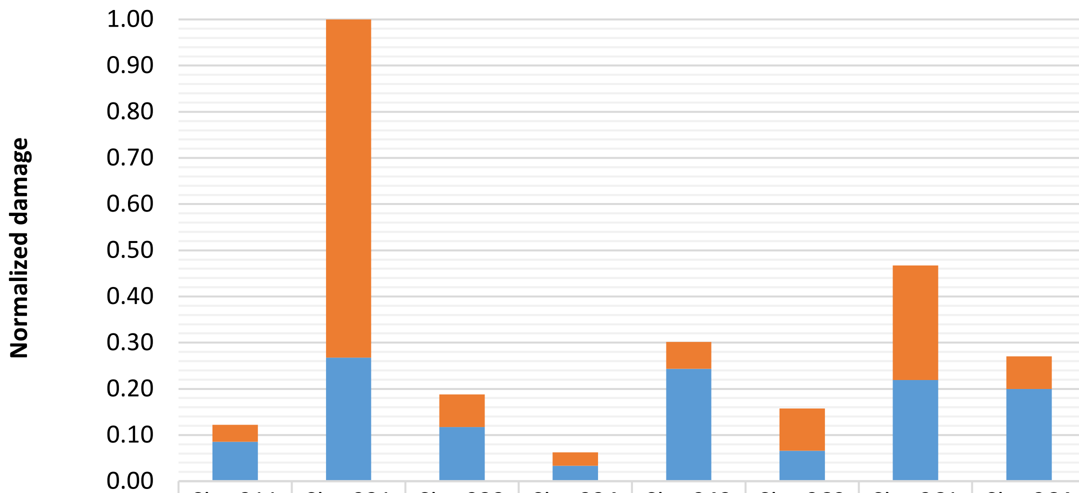


Figure 6.11: The number of WM records collected in 2015.

6.5.2 The impact of overweight vehicles on damage accumulation

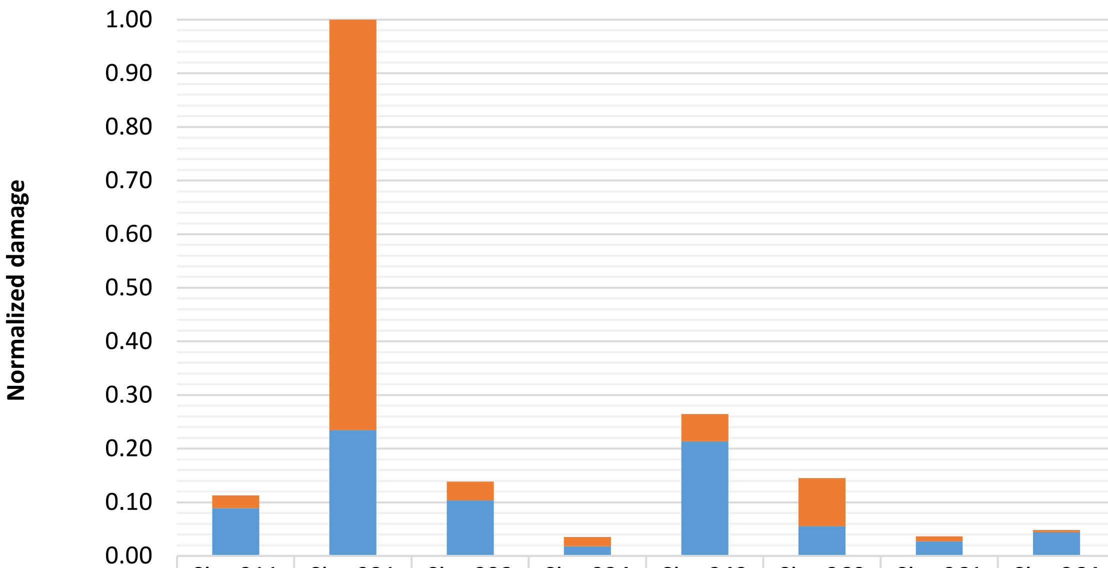
The truck traffic data was divided into two groups, legal and overweight, and the corresponding amount of damage (NM_{eff}^3) was computed for each group. Legal traffic includes vehicles that comply with Alabama's legal regulations, "grandfather exceptions" and annual permits that have GVW less than 100 kips. The overweight group covers vehicles that require an individual trip permit to travel legally due to its weight or axle load combination and annual permits that have GVW greater 100 kips. The proportion of the damage caused by these groups of vehicles (legal, overweight) is shown in Figure 6.12 and Figure 6.13 for the year 2014 and 2015. The corresponding number of legal and overloaded trucks are shown in Figure 6.14 and Figure 6.15. The results of the previous section show that the normalized fatigue damage is practically independent of the location along the girder and span length. Therefore, here and in further sections, the results are shown for the 30-ft span and upstream cover plate end only.

From Figure 6.12 and Figure 6.13, the amount of damage accumulated by overloaded traffic from WIM locations 931, 960, 961 is greater than for the legal traffic. However, at WIM location 961, the damage accumulated from overloaded traffic is greater than legal traffic only in the year 2014. It appears that damage caused by overweight vehicles is higher than legal vehicles when the number of overweight vehicles is greater than approximately 20% of all the vehicles. From Figure 6.16 and Figure 6.17 for the years 2014 and 2015, it can be concluded that the 20% of overloaded trucks create more than 50% of the total damage for the traffic combined from all the locations.



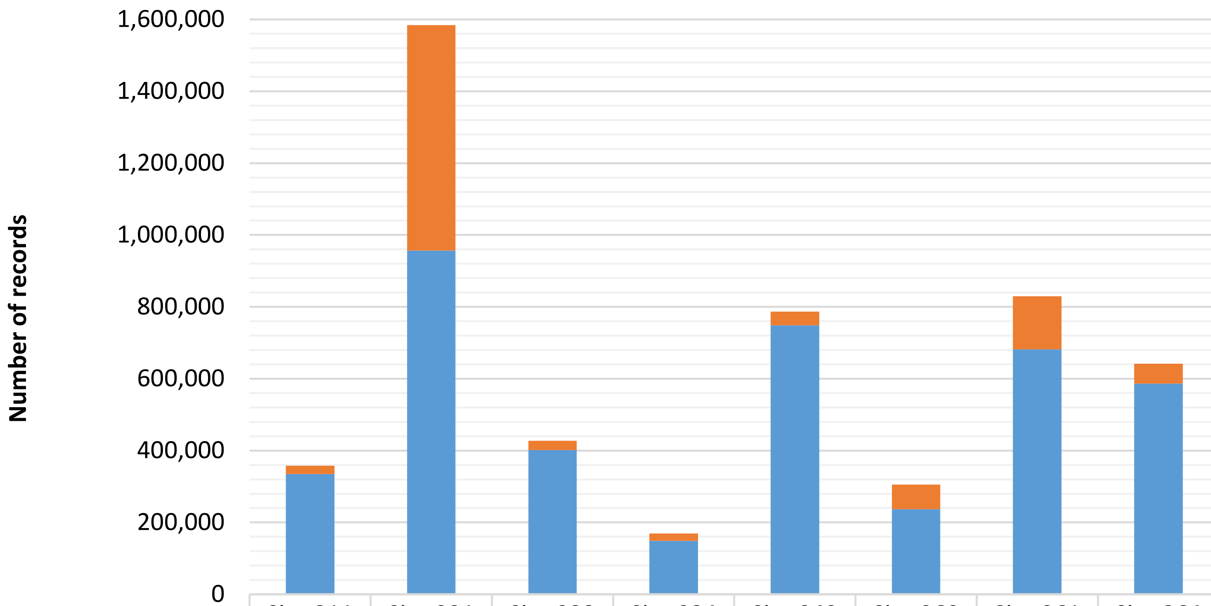
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Overweight traffic	0.04	0.73	0.07	0.03	0.06	0.09	0.25	0.07
Legal traffic	0.09	0.27	0.12	0.03	0.24	0.07	0.22	0.20

Figure 6.12: Normalized accumulated damage in 2014 at the upstream cover plate end of a 30-ft span due to legal and overweight truck traffic



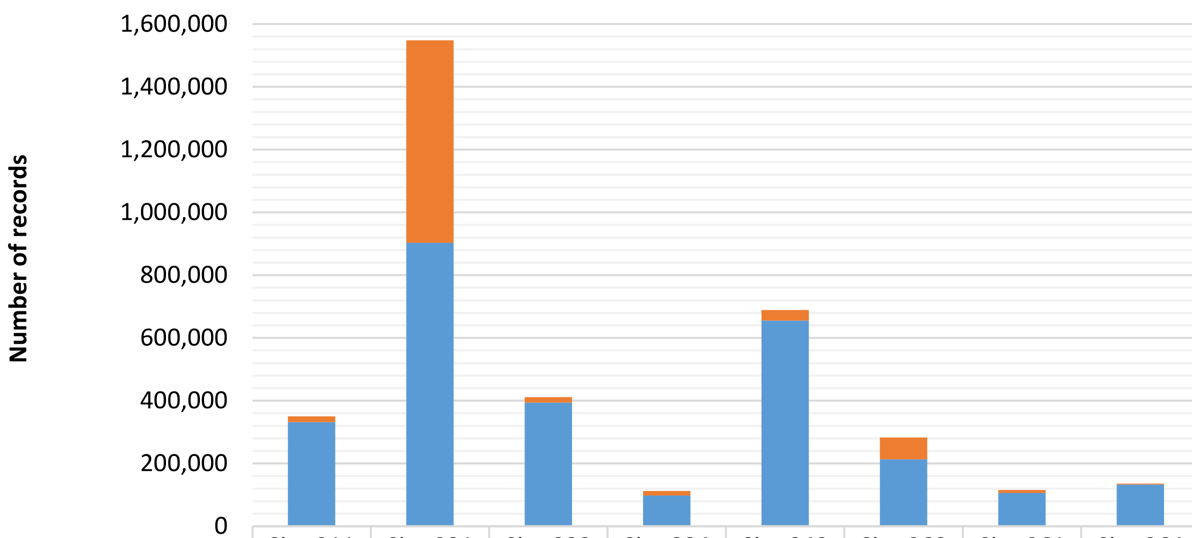
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Overweight traffic	0.02	0.77	0.04	0.02	0.05	0.09	0.01	0.00
Legal traffic	0.09	0.23	0.10	0.02	0.21	0.06	0.03	0.04

Figure 6.13: Normalized accumulated damage in 2015 at the upstream cover plate end of a 30-ft span due to legal and overweight truck traffic.



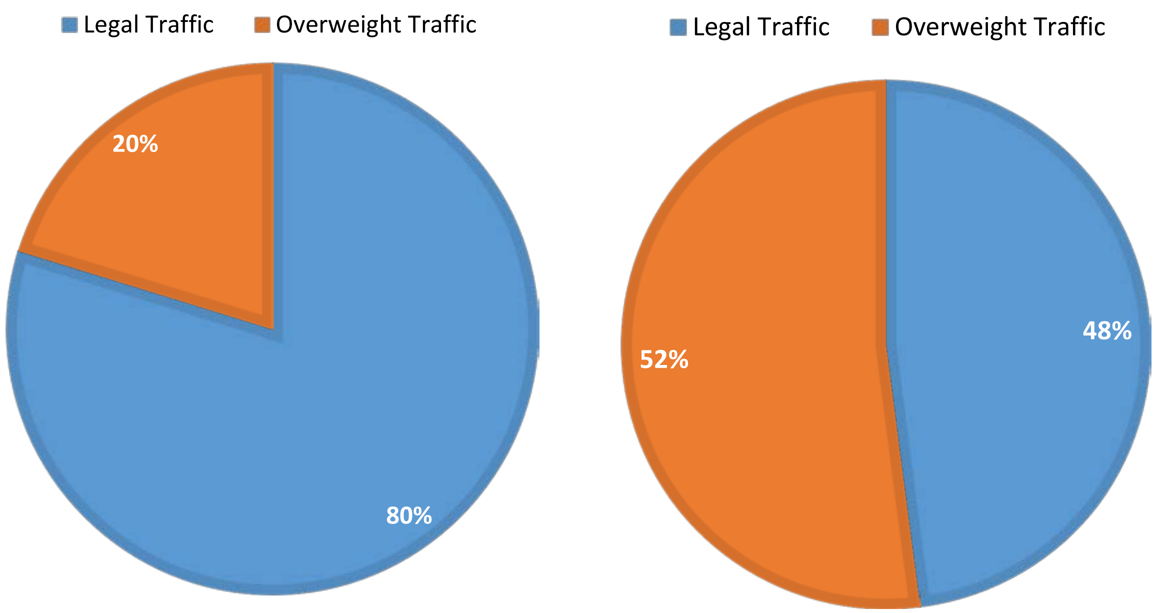
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Overweight traffic	23,036	626,987	25,548	20,325	38,507	68,381	147,736	55,717
Legal traffic	334,803	957,109	401,926	148,926	748,425	236,972	682,210	586,321

Figure 6.14: The number of legal and overweight WIM records collected in 2014



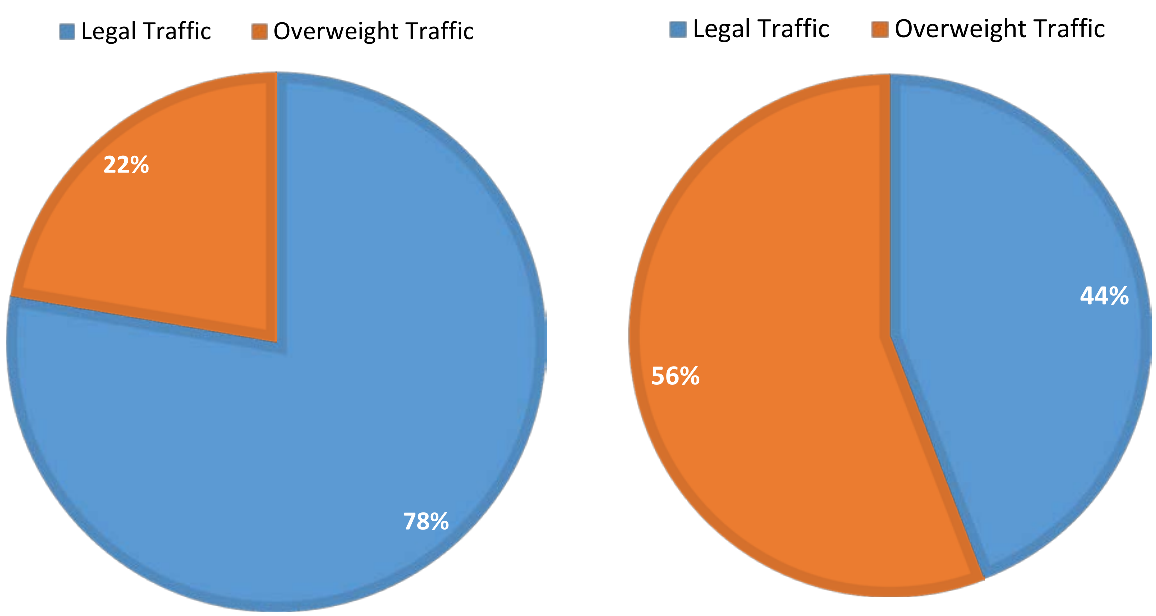
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Overweight traffic	18,624	645,772	17,436	13,792	34,102	68,993	9,578	2,894
Legal traffic	331,869	902,848	394,289	98,328	655,024	213,318	105,786	132,931

Figure 6.15: The number of legal and overweight WIM records collected in 2015.



a) b)

Figure 6.16: Contribution of the legal and overweight traffic to (a) the total number of records, and (b) the total amount of fatigue damage for all WIM locations in 2014



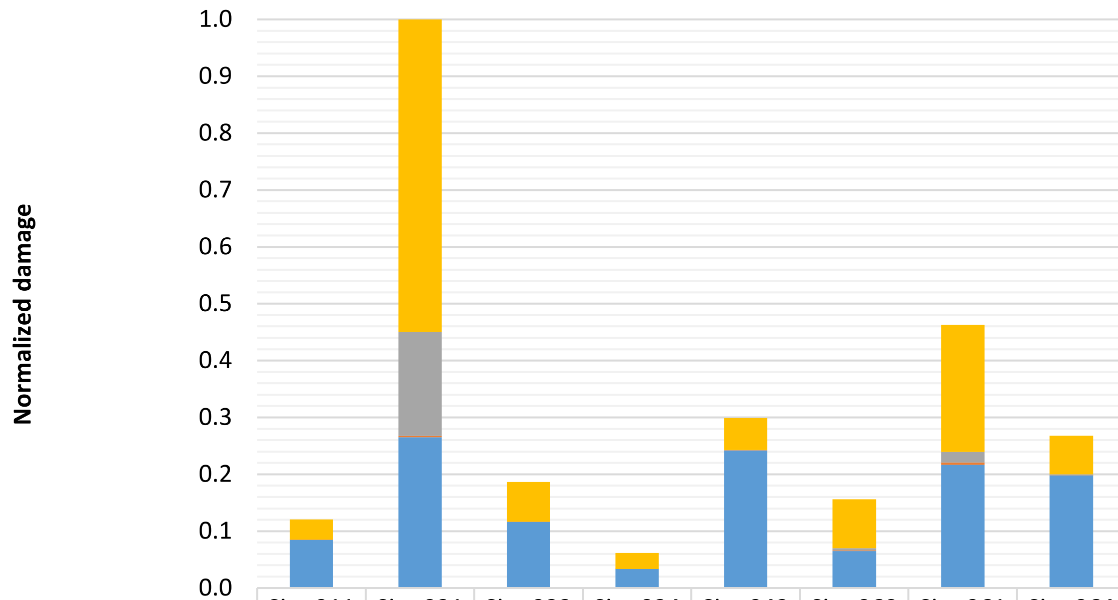
a) b)

Figure 6.17: Contribution of the legal and overweight traffic to (a) the total number of records, and (b) the total amount of fatigue damage for all WIM locations in 2015

6.5.3 The impact of vehicles with permits on damage accumulation

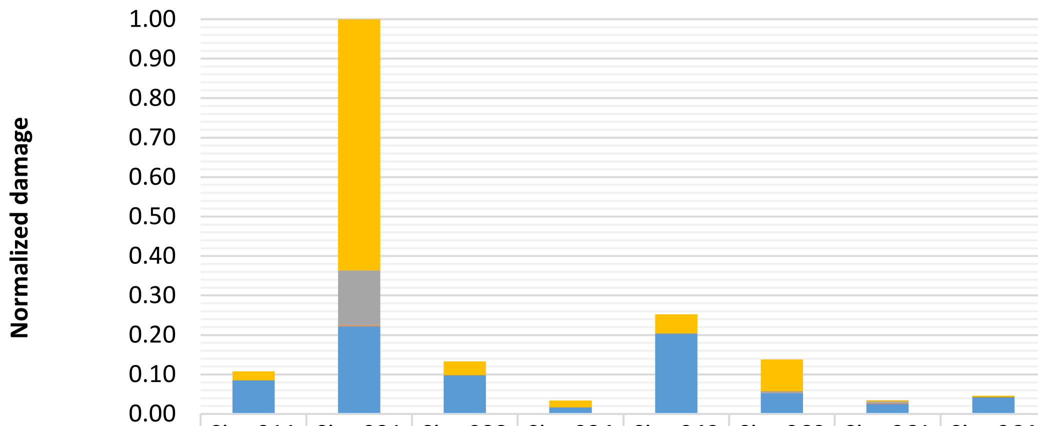
Some vehicles require an overweight permit to operate on a state or interstate road. However, several them do not have permits and, therefore, operate illegally. Others have an axle weight distribution and axle configuration like ones listed in the database of permits issued by ALDOT. The number of such trucks operating on the highways is substantially higher than permitted by ALDOT, but they meet permit criteria. The proportion of the damage caused by these groups of vehicles (legal, ALDOT permitted, illegal and those that meet ALDOT permit criteria) is shown in Figure 6.18 and Figure 6.19 for the year 2014 and 2015. The corresponding numbers of WIM records are shown in Figure 6.20 and Figure 6.21.

The Illegal vehicles cause the most damage at the WIM site 931 and 960 for the years 2014 and 2015 and at WIM site 961 for the year 2014. Vehicles with a permit and those that meet permit criteria do less damage than illegal vehicles or legal. From Figure 6.22 and Figure 6.23 where the traffic from all locations is combined for years 2014 and 2015, it can be concluded that the 2.5% of trucks that are illegally overloaded create more than 40% of the total damage.



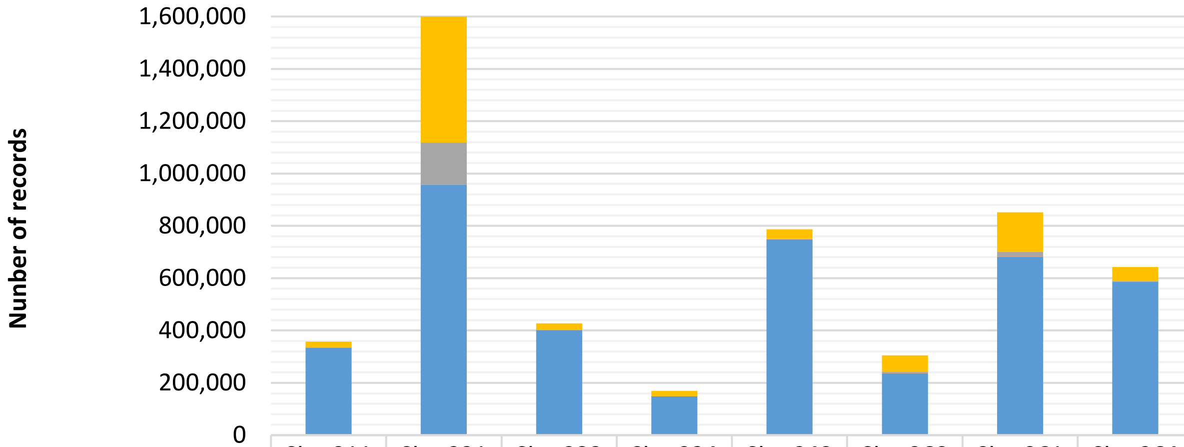
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
WIM Illegal traffic	0.04	0.55	0.07	0.03	0.06	0.09	0.22	0.07
Meet permit requirements	0.00	0.18	0.00	0.00	0.00	0.00	0.02	0.00
ALDOT Permits	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Legal traffic	0.08	0.27	0.12	0.03	0.24	0.07	0.22	0.20

Figure 6.18: Amount of damage accumulated in 2014 at the upstream cover plate end due to legal, permitted, and illegal truck traffic that meets permit requirement



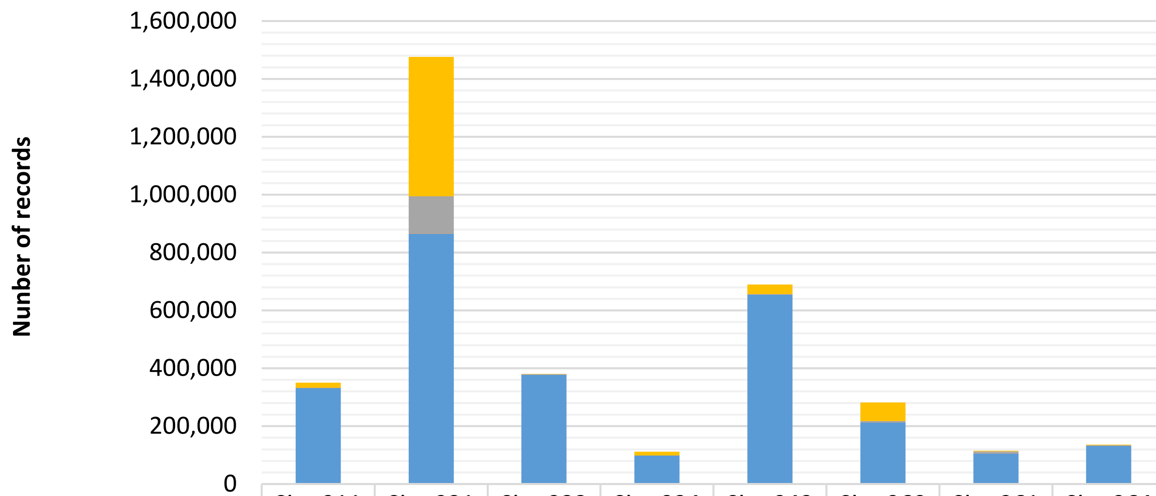
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
WIM Illegal traffic	0.02	0.64	0.03	0.02	0.05	0.08	0.00	0.00
Meet permit requirements	0.00	0.14	0.00	0.00	0.00	0.01	0.00	0.00
ALDOT Permits	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Legal traffic	0.08	0.22	0.10	0.02	0.20	0.05	0.03	0.04

Figure 6.19: Amount of damage accumulated in 2015 at the upstream cover plate end due to legal, permitted, illegal and truck traffic that meet permit requirement



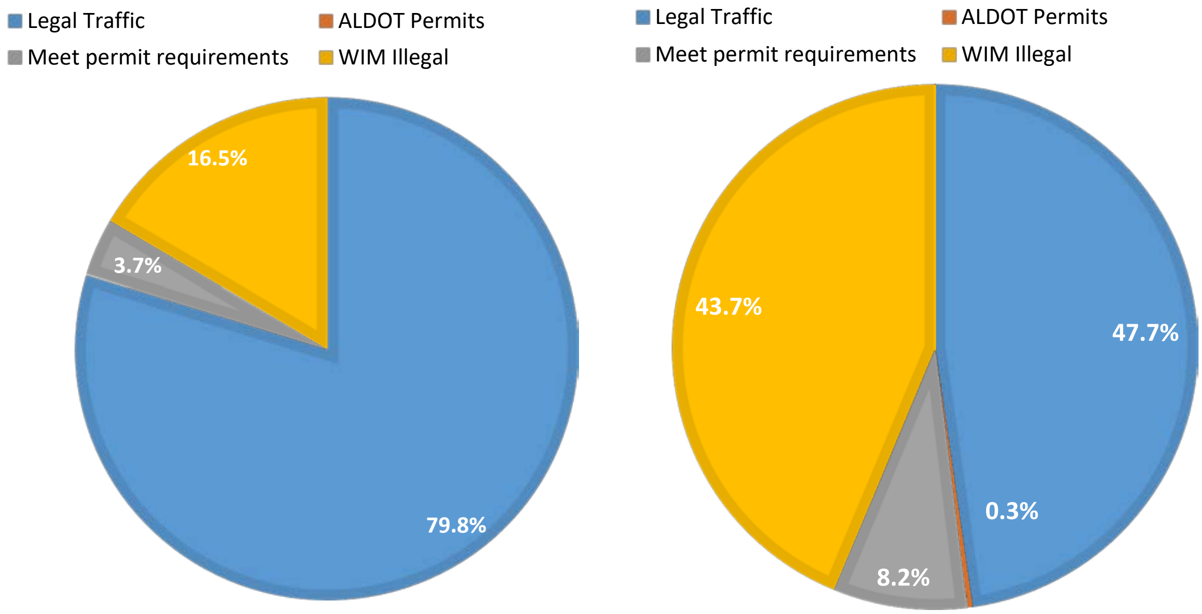
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
WIM Illegal traffic	22,131	501,628	25,188	20,288	37,588	64,483	149,809	53,702
Meet permit requirements	689	160,385	322	106	1,207	3,772	17,497	2,201
ALDOT Permits	225	1,000	69	86	206	339	1,696	113
Legal traffic	334,803	957,109	401,926	148,926	748,425	236,972	682,210	586,321

Figure 6.20: The number of legal, permitted, and illegal trucks that meet ALDOT permit criteria collected in 2014



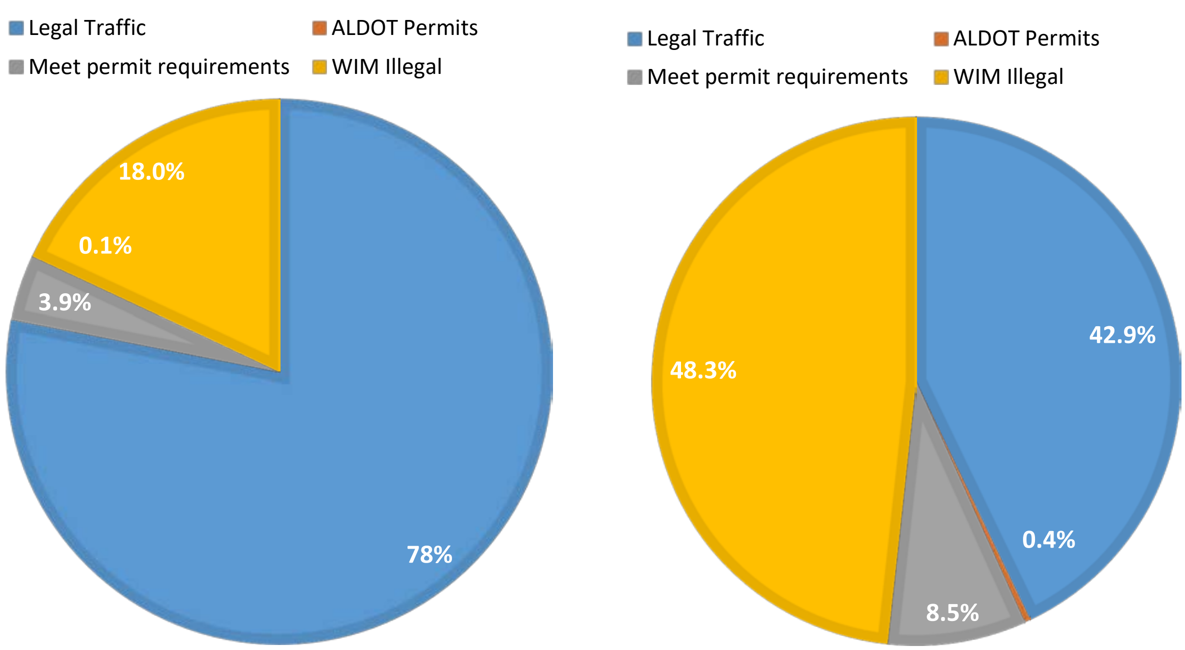
	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
WIM Illegal traffic	18,027	481,193	1,372	13,521	32,740	64,442	2,713	2,551
Meet permit requirements	432	127,918	211	195	1,169	4,359	5,673	219
ALDOT Permits	165	934	73	76	193	192	1,193	124
Legal traffic	331,868	865,668	378,482	98,328	655,024	213,318	105,786	132,931

Figure 6.21: The number of legal, ALDOT permitted, illegal and records that meet ALDOT permit criteria collected in 2015



a) b)

Figure 6.22: Contributions of the legal, permitted, illegal and trucks that meet permit criteria to (a) the total number of records, and (b) the total amount of fatigue damage for all WIM location in 2014



a) b)

Figure 6.23: Contributions of the legal, permitted, illegal and trucks that meet permit criteria to (a) the total number of records, and (b) the total amount of fatigue damage for all WIM location in 2015

6.5.4 Comparison of damage from different FHWA vehicle classes

The total amount of accumulated damage strongly depends on the traffic mix, in particular on the dominating vehicle types. The contribution of damage by different vehicle classes is shown in Table 6.4 and Table 6.5 for WIM location 931 and 960. These locations were selected because these WIM sites have the most damaging traffic compared to other sites. The results indicate that the Class 9 truck (5-axle, single trailer truck) contributes the most to the total amount of accumulated damage at WIM location 931 and 960. This is partly due to the high population of those vehicles. This type of vehicle is the most common in Alabama, as well as in the US. Above 70% of damage due to legal, permit and illegal vehicles are caused by 5-axle Class 9 trucks. At WIM location 960, other vehicle classes also contribute significantly to damage accumulation.

Table 6.4: Contribution to the total accumulated damage by different FHWA vehicle classes at WIM site 931 for the year 2014 and 2015.

WIM location 931							
Year 2014				**Year 2015**			
Vehicle Class	Legal (%)	Permit (%)	Illegal (%)	Vehicle Class	Legal (%)	Permit (%)	Illegal (%)
VC 0	0.0	0.0	3.1	VC 0	0.0	0.0	8.5
VC 4	1.5	0.0	2.1	VC 4	1.6	0.0	2.4
VC 5	0.6	0.0	7.1	VC 5	1.4	0.0	6.5
VC 6	0.5	0.0	3.6	VC 6	0.6	0.0	3.4
VC 7	0.0	0.0	0.8	VC 7	0.0	0.0	1.2
VC 8	2.5	0.0	3.3	VC 8	3.0	0.0	4.1
VC 9	91.4	98.2	76.6	VC 9	89.9	97.6	70.2
VC 10	1.7	0.9	1.0	VC 10	1.7	0.8	0.9
VC 11	1.7	0.0	1.6	VC 11	1.7	0.0	1.9
VC 12	0.0	0.0	0.1	VC 12	0.0	0.0	0.2
VC 13	0.1	0.8	0.7	VC 13	0.1	1.4	0.5
Total	**100**	**100**	**100**		**100**	**100**	**100**

Table 6.5: Contribution to the total accumulated damage by different FHWA vehicle classes at WIM site 960 for year 2014 and 2015.

WIM location 960							
Year 2014				Year 2015			
Vehicle Class	Legal (%)	Permit (%)	Illegal (%)	Vehicle Class	Legal (%)	Permit (%)	Illegal (%)
VC 0	0.0	0.0	0.2	VC 0	0.0	0.0	0.1
VC 4	0.7	0.0	0.2	VC 4	0.7	0.0	0.1
VC 5	0.8	0.0	0.1	VC 5	0.8	0.0	0.0
VC 6	8.1	0.0	4.2	VC 6	7.4	0.0	4.0
VC 7	0.3	0.0	9.7	VC 7	0.2	0.0	6.0
VC 8	0.9	2.3	0.0	VC 8	1.0	0.3	0.0
VC 9	84.6	69.8	67.1	VC 9	85.6	84.0	74.4
VC 10	4.3	27.2	15.0	VC 10	4.0	14.4	13.5
VC 11	0.0	0.0	0.0	VC 11	0.0	0.0	0.0
VC 12	0.1	0.0	1.9	VC 12	0.0	0.0	0.9
VC 13	0.3	0.8	1.6	VC 13	0.2	0.2	0.9
Total	**100**	**100**	**100**		**100**	**100**	**100**

6.6 Damage Accumulation Index to a Specific Bridge

The traffic carried by each bridge is different. Also, the bridges were built in different time periods and there is variation in the design of bridges. Additional assessments of damage can be calculated by considering a specific bridge and by knowing the traffic carried by that bridge. A procedure was developed to estimate the damage for a specific bridge. Fatigue crack initiation and propagation usually occur in a region of tensile stress at a welded attachment. Therefore, specific fatigue prone details are of interest in the damage assessment. Also, bridges are designed for fatigue, assuming the AASHTO fatigue design truck encompasses the traffic carried by that bridge during its service life. A primary interest here is knowing how much damage is accumulating over the service life of the bridge. All the scenarios mentioned above are considered to develop a practical procedure.

6.6.1 Damage at a specific fatigue prone detail

Different details in a steel bridge experience stress ranges of various numbers and magnitudes and, therefore, some accumulate fatigue damage faster than others. Based on the study performed by Franklin (Franklin 2000), the base metal at the end of a bottom flange cover plate is considered here as the most fatigue prone detail in Alabama's steel girder bridges. The bottom flange of the girder at the upstream cover plate end is a Category E' detail, and the base metal adjacent to a transverse stiffener weld near the bottom flange at the mid-span may be a Category C' detail as defined in *AASHTO LRFD Bridge design specifications* (AASHTO 2017).

For a specific fatigue prone detail, an index of the fatigue damage accumulated by a bridge along a route due to the truck traffic recorded by a WIM station can be calculated using Eq. 5.8. D_m is a Miner's fraction determined by dividing the fatigue damage accumulated over a specified period of time by the mean value of fatigue damage defining failure. This fraction is also equal to the fatigue life expended divided by the mean fatigue life as follows:

$$D_m = \frac{p * N * S_{eff}^3}{R_R * A} \quad (6.14)$$

where,

D_m – fraction of mean fatigue life expended at a specific fatigue prone detail

N – total number of cycles produced by truck traffic in a specific period of time (set of records in the WIM database) determined by counting bending moment cycles using rainflow counting

A – constant for a particular detail category (*AASHTO LRFD Bridge design specifications* Table 6.6.1.2.3.-1 (AASHTO 2017))

R_R – resistance factor for mean fatigue life (Manual for Bridge Evaluation (MBE), Table 7.2.5.1-1 (AASHTO 2018))

S_{eff} – effective stress range for a set of records in the WIM database determined based on Eq. 6.15

p – fraction of truck traffic in a single lane as specified in *AASHTO LRFD Bridge design specifications* Table 3.6.14.2.1, (Table 6.6 of this dissertation)

Table 6.6: Fraction of truck traffic in a single lane, p (*AASHTO LRFD Bridge design specifications,* Table 3.6.1.4.2.1 (AASHTO 2017))

Number of lanes available to Trucks	p
1	1
2	0.85
3 or more	0.8

The variable ‘p’ in Equation 6.14 is used for a fraction of truck traffic in a single lane depending upon the number of lanes available to trucks. Effective stress range, S_{eff} for a set of WIM records is calculated based on Equation 6.15.

The Equation 6.15 calculates stress range from the moment range created by the trucks in WIM records. All the vehicle records in WIM database contain the direction of travel and lane number information. The calculated S_{eff} for the set of WIM records is for the traffic in one direction and with traffic in all the lanes in that direction combined. The girder distribution factor (GDF) and dynamic load allowance (IM) are calculated according to *AASHTO LRFD Bridge design specifications*, section 4.6.2.2.2 and 3.6.2 respectively (AASHTO 2017). The variable ‘P’ is the ratio of measured to calculated stress range. A ‘P’ value of 0.6 is calculated by using the results from Pearson (2002) where the author had measurements of the stress range at cover plate ends of four typical steel girder bridges. The 'P' value is the ratio of stress range measured from the passage of a single truck of known weight and configuration to the stress range calculated by the simplified bridge analysis procedure of the *AASHTO LRFD Bridge design specifications.* The effective constant amplitude stress range at the detail is

$$S_{eff} = \frac{M_{eff} * GDF * (1+IM) * P * R_p}{S} \quad (6.15)$$

where,

M_{eff} – effective moment range for a set of WIM records calculated as shown in Eq. 6.12

GDF – girder distribution factor for a single loaded lane

IM – dynamic load allowance, 0.15 from *AASHTO LRFD Bridge design specifications,* Table 3.6.2.1-1 (AASHTO 2017)

P – ratio of measured to calculated stress range, 0.6 (determined based on the stress ranges measured by Pearson (Pearson 2002))

S – section modulus for the specific fatigue detail

R_p – multiple presence factor

The multiple presence factor as described in MBE Article 7.2.2.1 is given by

$$R_p = 0.988 + 6.87 \text{x} 10^{-5} \text{ (L)} + 4.01 \text{x} 10^{-6} \text{ [ADTT]}_{PRESENT} + \frac{0.0107}{n_L} \quad (6.16)$$

where,

L = span length in feet

$[ADTT]_{PRESENT}$ = present average number of trucks per day for all directions of truck traffic including all lanes on the bridge

n_L = number of lanes

6.7 Damage accumulated from WIM traffic relative to AASHTO fatigue design truck and ALDOT rating vehicles

As mentioned in the previous section the AASHTO fatigue design truck is intended to capture the fatigue damage caused by the traffic on the highways. The current AASHTO fatigue design truck has a GVW of 57.6 kips and the configuration is the same as the HS truck shown in Figure 6.24; except, the spacing between the last two axles is a constant 30 ft (AASHTO 2017). A fraction called damage accumulation index, α, is introduced in Equation 6.17, which is the ratio of the damage caused by WIM truck traffic relative to an equal number of crossings of the AASHTO fatigue design truck. Equation 6.13 can be used for a set of WIM trucks or a single truck alone. Values of NM_{eff}^3 for a fatigue truck on various span lengths are shown in Table 6.7. The damage accumulation index, α, is shown in the form of an equation as:

$$\alpha = \frac{D_{\text{due to WIM trucks}}}{D_{\text{due to fatigue design truck}}} \tag{6.17}$$

Further substituting Eq. 6.13 in Eq. 6.17, and if T is the number of trucks (WIM records) then the damage accumulation index is:

$$\alpha = \frac{\left(NM_{eff}^{m}\right)_{\text{due to WIM trucks}}}{T * \left(NM_{eff}^{m}\right)_{\text{due to fatigue design truck}}} \tag{6.18}$$

Similar to the relative damage calculated by using the AASHTO fatigue design truck, the relative damage of the ALDOT rating vehicles can be calculated. The ALDOT rating vehicles are shown in Figure 6.24. The corresponding ratios of D (using Equation 6.13) for a single passage of an ALDOT rating vehicle to a single passage of the AASHTO fatigue design truck for standard span lengths of 30, 60, 90, 120 and 200 ft are shown in Table 6.7 and Table 6.8.

Table 6.7: Ratios of D for ALDOT rating vehicles to AASHTO fatigue design truck for mid-span

Vehicle Type	Span length				
	30 ft	60 ft	90 ft	120 ft	200 ft
AASHTO Fatigue Truck	1	1	1	1	1
H Design	0.54	1.78	0.87	0.65	0.49
Two-Axle	0.85	4.30	2.33	1.84	1.44
Tri-Axle	2.54	10.22	5.23	4.03	3.08
Concrete	1.66	6.84	3.53	2.72	2.09
18-Wheeler 3S2	1.57	3.73	3.18	3.00	2.85
6 Axle	1.97	4.17	3.41	3.31	3.21
School Bus	0.13	0.25	0.15	0.12	0.10
Emergency Vehicle 2	0.77	3.94	2.14	1.69	1.33
Emergency Vehicle 3	3.16	14.28	7.52	5.87	4.56

Table 6.8: Ratios of D for ALDOT rating vehicles to AASHTO fatigue design truck for upstream cover plate end

Vehicle Type	Span length				
	30 ft	60 ft	90 ft	120 ft	200 ft
AASHTO Fatigue Truck	1	1	1	1	1
H Design	0.24	0.48	0.23	0.18	0.13
Two-Axle	0.72	1.29	0.64	0.49	0.39
Tri-Axle	0.86	2.86	1.40	1.05	0.81
Concrete	0.56	2.13	0.99	0.73	0.55
18-Wheeler 3S2	0.47	2.61	1.93	0.88	0.79
6 Axle	0.54	3.24	1.18	1.05	0.94
School Bus	0.05	0.21	0.05	0.04	0.03
Emergency Vehicle 2	0.58	1.10	0.61	0.49	0.38
Emergency Vehicle 3	2.52	4.18	2.03	1.55	1.22

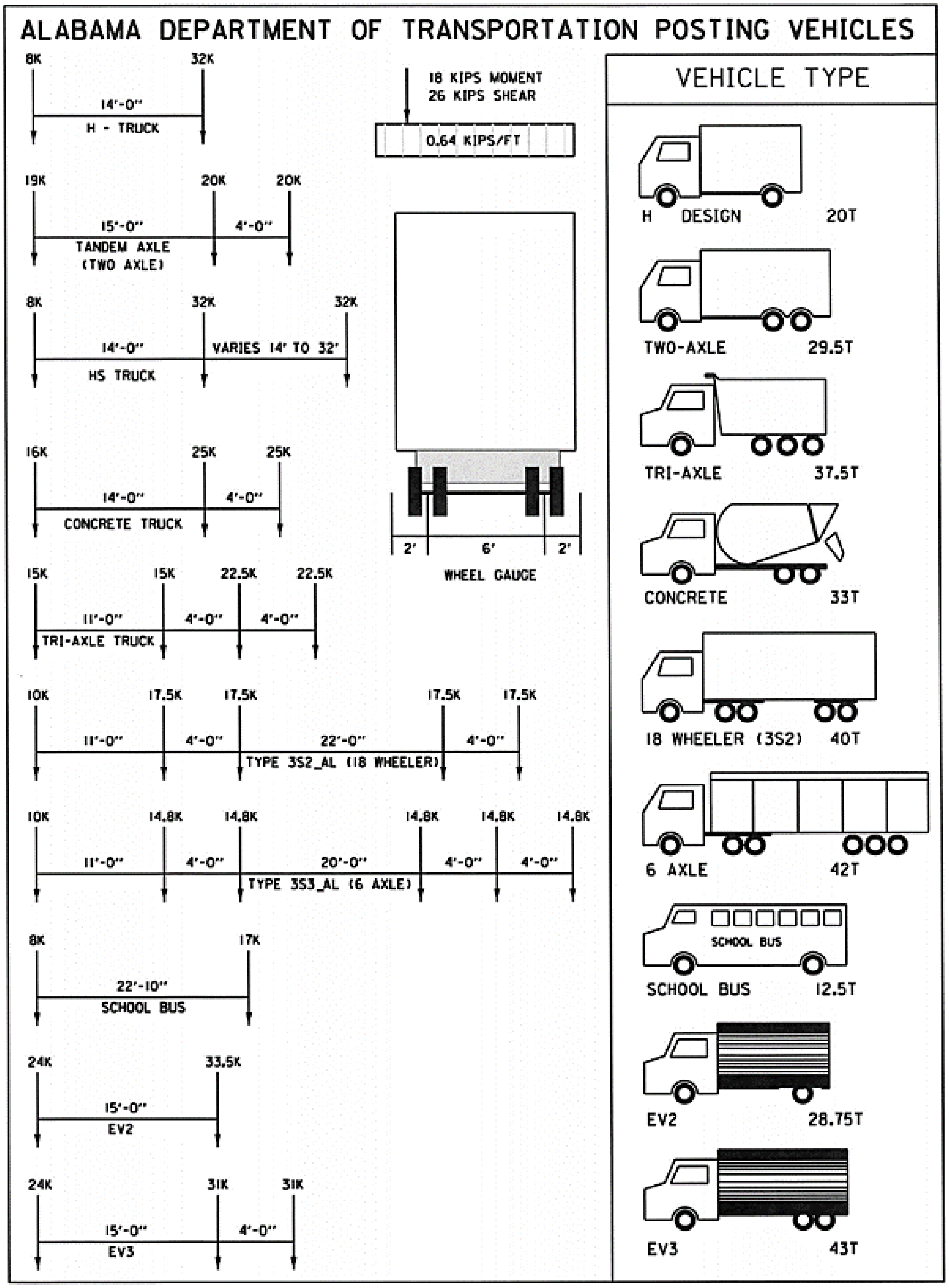


Figure 6.24: ALDOT rating vehicles

6.8 Evaluation of a Specific Bridge Using Traffic Data from WIM Site 931

A previous study funded by ALDOT and conducted by Auburn University Highway Research Center included evaluation and instrumentation of representative spans from among 58 simple-span rolled girder bridges, 18 continuous-span rolled-girder bridges and 6 plate-girder simple-span bridges in downtown Birmingham (Franklin 2000). Stress ranges were calculated at the following fatigue-prone details of the bridges: transverse diaphragm connections, longitudinal cover plate fillet weld connections, shear connectors, and cover plate ends at the upstream and downstream locations. The study concluded that the base metal at the end of the cover plate was identified as the most fatigue-detail of those steel girder bridges. The bottom flange of the girder at upstream cover plate end is detail category Type E' as defined in *AASHTO LRFD Bridge design specifications* (AASHTO 2017).

To demonstrate the procedures developed in this dissertation, the damage accumulated at the specific fatigue prone detail, E,' i.e., cover plate end for the Span 86-W described by Franklin (2000) is selected. Plan view and cross section view of Span 86-W is shown in Figure 6.25 and Figure 6.26. This bridge is selected because it is a real bridge typical of steel girder bridges on Alabama highways. Also, as an example, WIM data from site 931 is used. Damage accumulated from the passage of traffic in one of the directions (lane 1 and 2) at WIM station 931 for the year 2014 on the upstream cover plate end is shown here.

Span 86-W consists of eight W36x150 rolled section girders spaced 8.71 ft. On average the total length of the beams is 66.30 ft, and the approximate of average span length is 60 ft. The cover plate size is 10" x 0-15/16" x 41"-6". Based on this information the bridge data inputs to estimate the damage accumulation are calculated and listed in Table 6.9.

Table 6.9: Bridge data inputs for Span 86-W Bridge on Interstate I-59/20 in Birmingham, Alabama

Section modulus (S)	701.98 in^3
Span length (L)	60 ft
Girder distribution factor (GDF)	0.51
Dynamic load allowance (IM)	0.15
Location of upstream cover plate end	11.04 ft
x/L of upstream cover plate end	0.2
Location of downstream cover plate end	53.57 ft
x/L of downstream cover plate end	0.2
Number of traffic lanes	4
Direction of traffic	One-direction only

The traffic recorded at WIM site 931 is bi-directional with two lanes of traffic in each direction. Here fatigue damage is assumed to result only from traffic in one direction. N and M_{eff} for a span length of 60 ft and x/L location of 0.2 are calculated using the computer app - AL_WIM_DAI v1.0 for each month of the year 2014. (A more detailed discussion of the AL_WIM_DAI v1.0 computer app is shown in Appendix F). The output generated by the computer app by processing WIM station 931 traffic data is shown in Table 6.10. The table contains N, M_{eff} and D calculated for traffic in one of the directions (lane 1 and 2) for all the months in the year 2014.

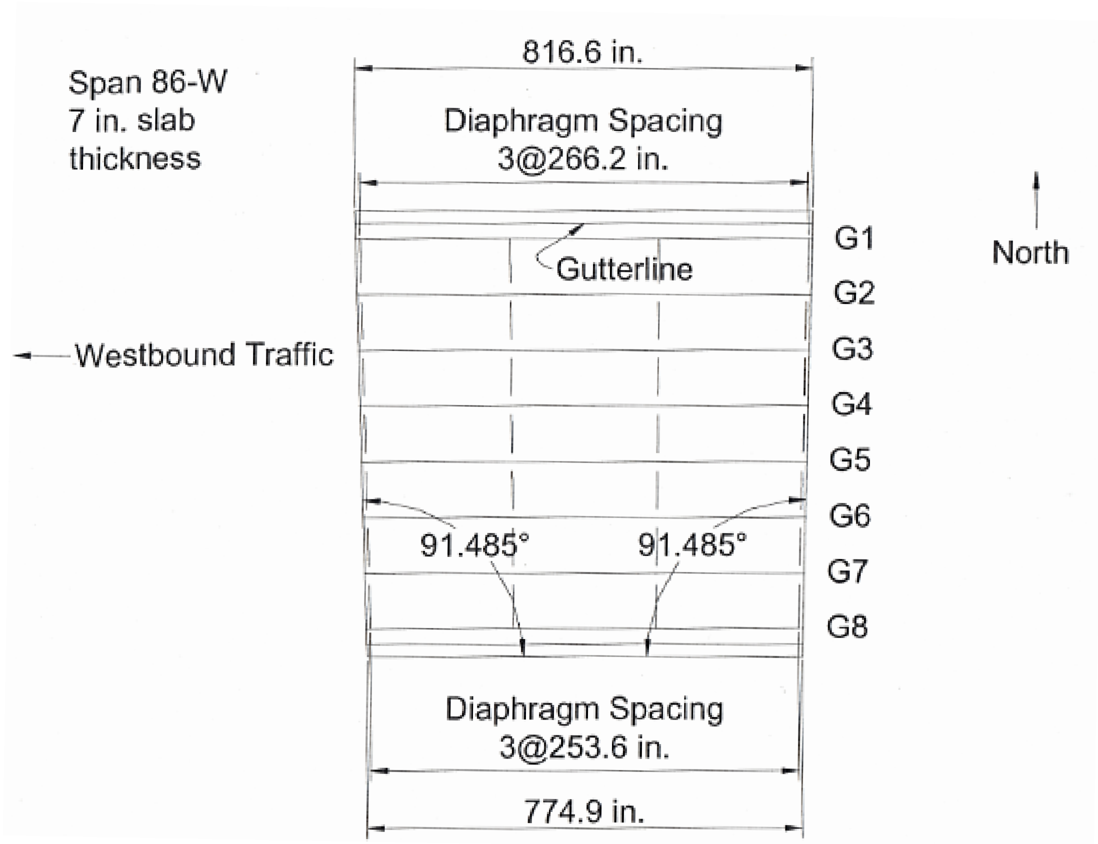


Figure 6.25: Plan View of Span 86-W

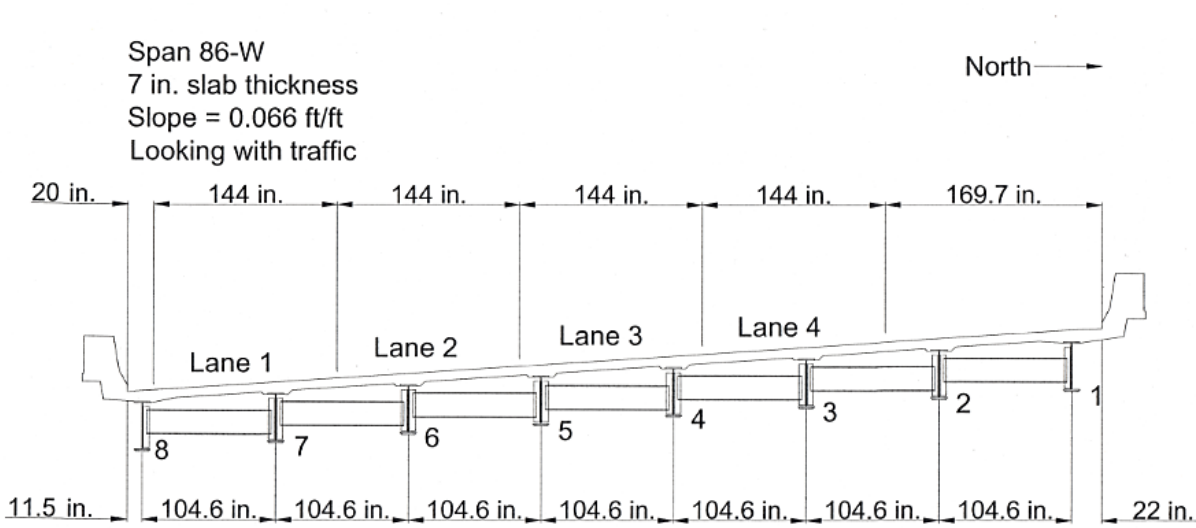


Figure 6.26: Cross Section View of Span 86-W

Table 6.10: Output of AL_WIM_DAI v1.0 for WIM station 931 and year 2014

Year 2014	For L = 60 ft and x/L = 0.2 (upstream coverplate end)			
Month	**No. of records**	**N (cycles)**	**M_{eff} (kip-ft)**	**$D=NM_{eff}^3$ (cycles(kip-ft)^3)**
Jan	66423	145916	257.64	2.50E+12
Feb	62731	137922	257.38	2.35E+12
Mar	71102	155941	260.24	2.75E+12
Apr	69661	153324	256.61	2.59E+12
May	73938	161858	266.31	3.06E+12
Jun	72429	154466	296.94	4.04E+12
Jul	73259	154400	315.29	4.84E+12
Aug	73270	157581	300.38	4.27E+12
Sep	67079	145985	272.55	2.96E+12
Oct	79038	167937	306.89	4.85E+12
Nov	66480	144871	268.98	2.82E+12
Dec	67562	145270	283.27	3.30E+12
	T = 842972			∑ D = 4.03E+13

Other variables used in equation 6.14 and 6.15 to calculate the fraction of mean fatigue life expended, D_m, are listed in Table 6.11.

Table 6.11: Variables for calculation of fraction of mean fatigue life expended at a specific fatigue prone detail, D_m

Fraction of truck traffic (p)	0.8
Resistance factor for mean fatigue life for E' detail (R_R)	1.9
Ratio of measured to calculated stress range (P)	0.6
Multiple presence factor (R_p)	0.998005
Average daily truck traffic (ADTT)	2809
Number of lanes (n_L)	4

By substituting Eq. 6.15 in Eq. 6.14, the D_m results in:

$$D_m = \frac{p}{R_R * A} * \left(\frac{GDF*(1+IM)*P*R_p}{S}\right)^3 * NM_{eff}^3 \tag{6.19}$$

By substituting all the values from Table 6.9, Table 6.11 and $\sum D$ for (NM_{eff}^3) for the year 2014 from Table 6.10 in Eq. 6.19, the fraction of mean fatigue life expended, D_m, at the upstream cover plate end is 0.0099. The calculated D_m can be interpreted by comparing with a design life of the bridge to the estimated amount of damage it accumulated in a year. For instance, if a bridge is designed for 75 years, then 1/75 (which is 0.0133) is more than the calculated damage 0.0099, indicating the rate of damage accumulation is less than anticipated during the design.

Also, a demonstration of damage accumulated from WIM traffic relative to AASHTO fatigue design truck for WIM station 931 for the year 2014 is shown here. The damage accumulation index, α is calculated from Eq. 6.17 for a Span 86-W that has an approximate span length of 60 ft. $\sum D$ for the year 2014 and T are shown in Table 6.10. NM_{eff}^3 values for the fatigue truck for various span lengths are shown in Appendix D, and the one corresponding to 60 ft span length is 7.23E+07. By substituting all these values in Eq. 6.17, the damage accumulation index, α is 0.66. This indicates that the damage accumulated from the WIM trucks is 34% less than the damage accumulated from an equal number of crossings of the AASHTO fatigue design truck. So, the AASHTO fatigue design truck captures the fatigue damage caused by the traffic at WIM site 931. D_m and α for the traffic from other WIM stations for the year 2014 and for the same Span 86-W bridge are shown in Table 6.12.

Table 6.12: D_m and α for the traffic in all WIM Sites in the state of Alabama for the year 2014

WIM Site	No. records (T)	ADTT	R_p	NMeff^3 (cycles(kip-ft)^3)	D_m	α
911	181387	755	1.000	5.26E+12	0.0013	0.40
931	842972	2809	1.006	4.05E+13	0.0099	0.66
933	358308	1216	1.000	9.18E+12	0.0022	0.35
934	84034	2092	1.003	3.60E+12	0.0009	0.59
942	376148	1667	1.001	7.58E+12	0.0018	0.28
960	628916	787	1.000	1.21E+13	0.0029	0.27
961	306640	2862	1.006	1.80E+13	0.0044	0.81
964	304904	829	1.000	8.32E+12	0.0020	0.38

Also, the effective moment, M_{eff} and a number of cycles, N, for all the WIM locations for the years 2014 to 2016 are listed in Appendix C. The tables in Appendix C are separated based on directions (Lane 1 & 2 in one direction, Lane 3 & 4 in other direction) and results are calculated for standard span lengths of 30, 60, 90, 120 and 200 ft. Also, results for Class 0 vehicles are listed in separate tables.

6.9 Fraction of Mean Fatigue Life Expended at a Specific Fatigue Prone Detail (D_m) for AISI Short Span Steel Bridges

AISI Short-Span Steel Bridges (American Iron and Steel Institute 1995) has real-life bridge design examples of composite rolled beams with welded cover plates. To evaluate the fraction of mean fatigue life expended at a specific fatigue prone detail (D_m), analyses were performed for the bottom flange of the girder at the mid-span (transverse stiffener fillet welds, detail category Type C') and cover plate ends (detail category Type E') at the upstream and downstream locations for some of these example bridge designs. Span lengths of 60, 90 and 120 ft were selected. Descriptive information for the composite rolled beams with a welded cover plate for the selected bridge span lengths is shown in Table 6.13. More in-depth design details of each span length are in Appendix G.

Table 6.13: Composite rolled beams with a welded cover plate from AISI (American Iron and Steel Institute 1995)

		Cover plate		
Span (ft)	**Beam cross-section**	**Thickness x Width (in)**	**Location (in)**	**Girder spacing (ft)**
60	W 33x118	3/4 x 9 -1/2	5.5	10
90	W 40x183	1-1/2 x 10	9.5	10
120	W 36x300	2 x 14	14.5	10

Using the designs from AISI and WIM data for all the WIM stations, the mean fatigue life expended by a specific fatigue prone detail (D_m) is calculated using the procedure shown in section 6.5. The results are shown in Figure 6.27 to Figure 6.34 for the traffic in two different directions (Lane 1 & 2 in one direction, Lane 3 & 4 in other direction) for the bottom flange of the girder at the mid-span (transverse stiffener fillet welds, detail category Type C') and cover plate ends (detail category Type E') for the year 2014 and 2015.

In Figure 6.27 to Figure 6.34, the dashed red colored line at 0.0133 marks the fraction 1/75 corresponding to one year of 75-year design life. If the calculated D_m is more than 0.0133, it indicates that the rate of damage accumulation is more than anticipated during the design. The plots show that the bridge design life is consumed up to four times faster than expected if the bridge experiences the truck traffic that was recorded at WIM site 931 (Athens). It is used up two times faster if the truck traffic is similar to WIM records collected at WIM sites 942 (Pine Level) and 961 (Mobile). For the traffic recorded at the other WIM sites, the fatigue life of the standard bridge details investigated is greater than 75 years.

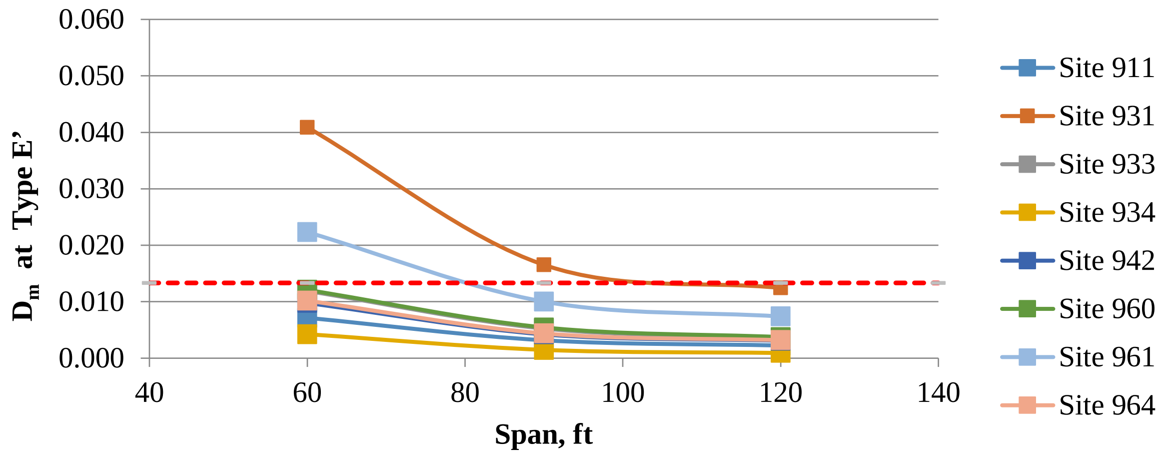


Figure 6.27: D_m at upstream cover plate end (Lane 1 & 2, year 2014)

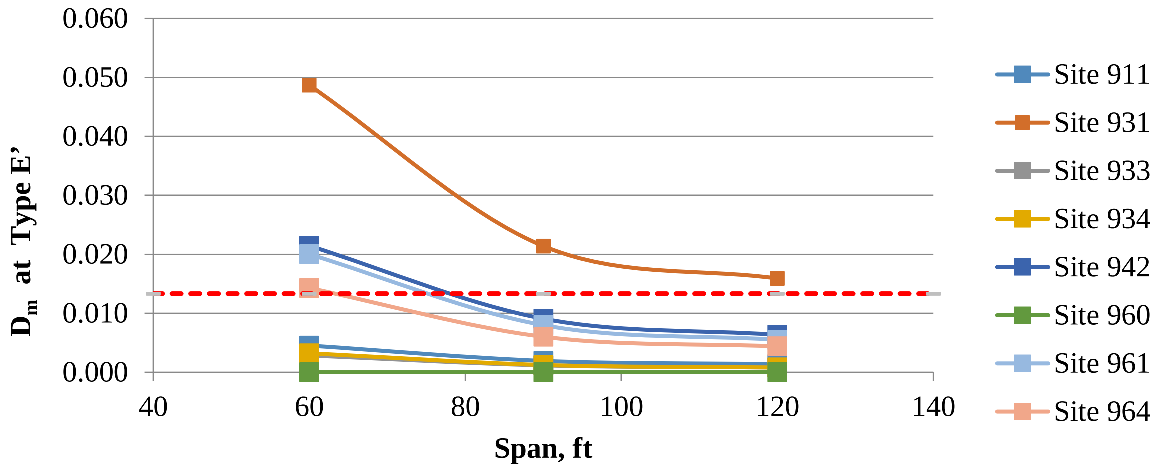


Figure 6.28: D_m at upstream cover plate end (Lane 3 & 4, year 2014)

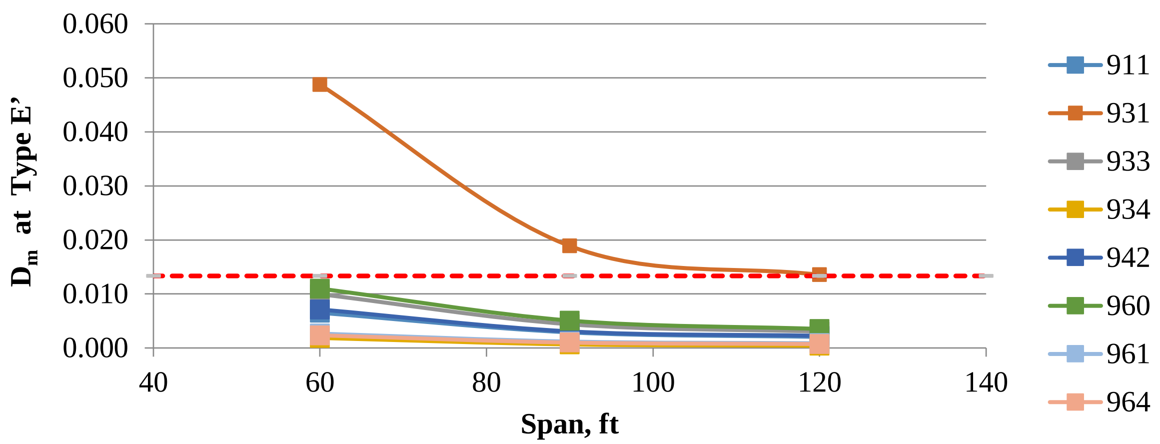


Figure 6.29: D_m at upstream cover plate end (Lane 1 & 2, year 2015)

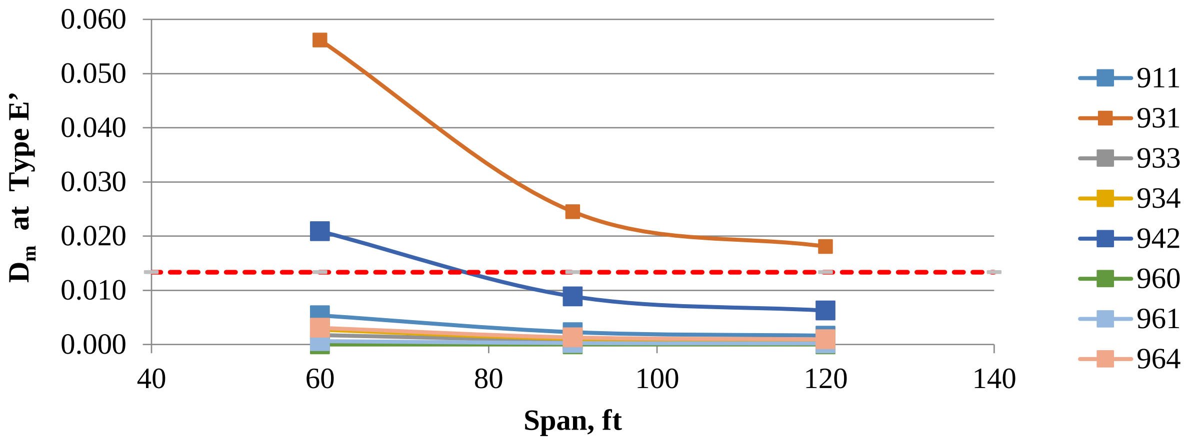


Figure 6.30: D_m at upstream cover plate end (Lane 3 & 4, year 2015)

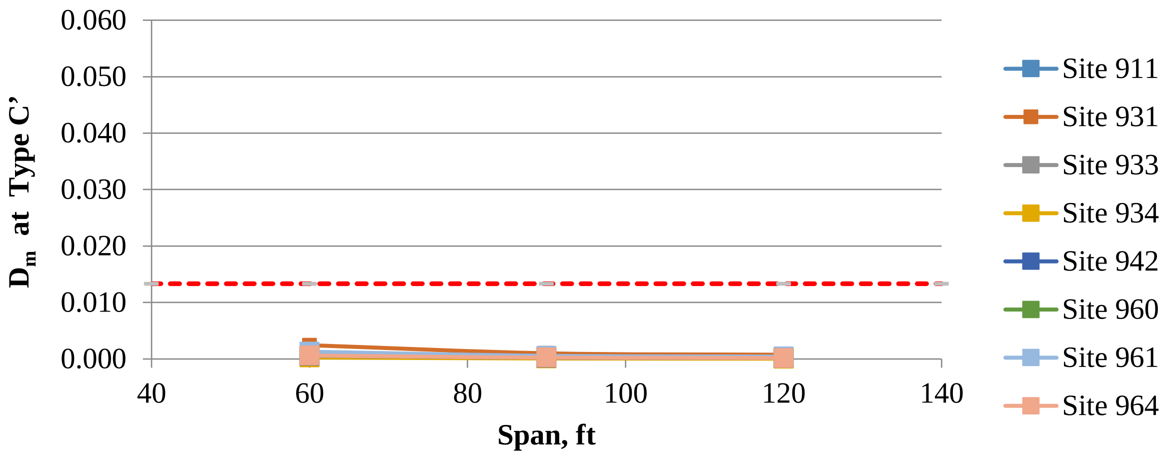


Figure 6.31: D_m at mid span (Lane 1 & 2, year 2014)

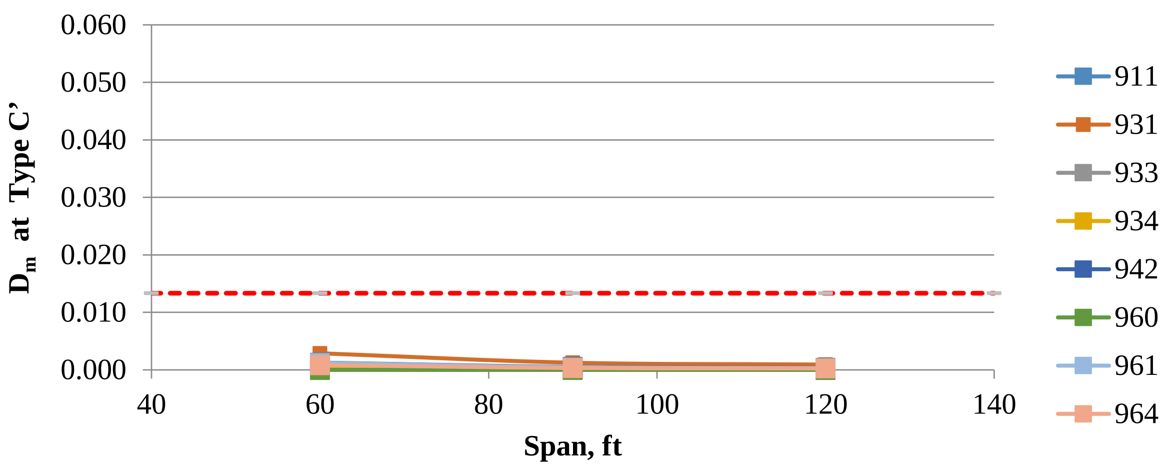


Figure 6.32: D_m at mid span (Lane 3 & 4, year 2014)

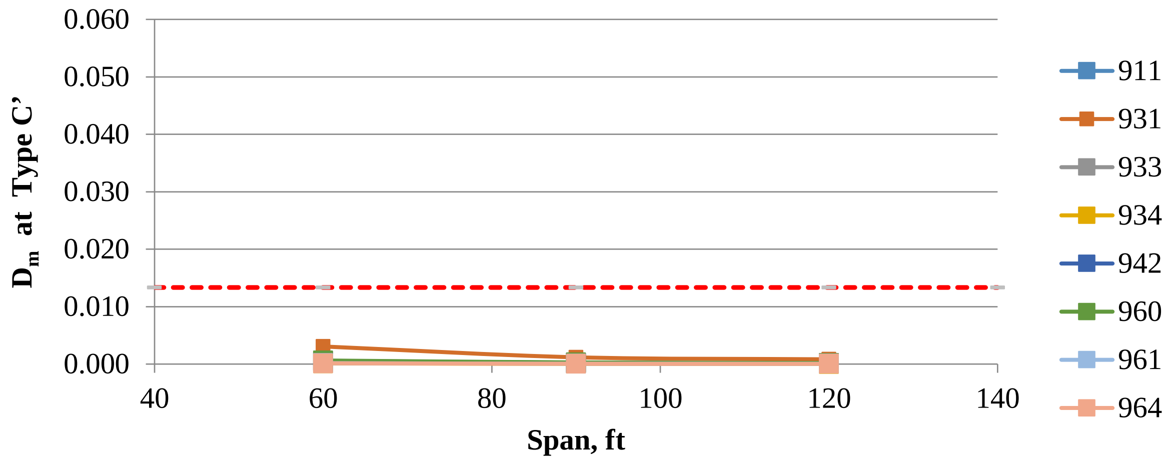


Figure 6.33: D_m at mid span (Lane 1 & 2, year 2015)

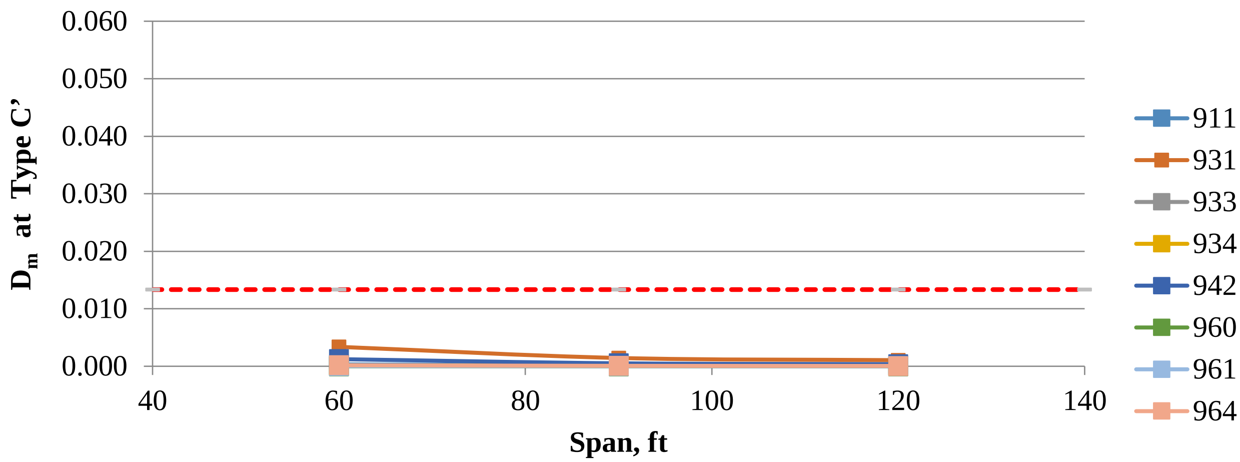


Figure 6.34: D_m at mid span (Lane 3 & 4, year 2015)

6.10 Summary

A methodology was developed for using WIM data to calculate the amount of fatigue damage accumulated in steel girder bridges due to real traffic. Previous research has shown that steel girder bridges on Alabama highways experience stress ranges at bottom flange cover plate ends that are large enough to eventually cause fatigue cracking. So, cover plate ends were a focus of the work presented here.

The methodology is robust because it allows comparisons of the traffic at various locations, comparisons of real traffic to design assumptions and comparisons of the damage caused by individual trucks. Results presented show that the traffic recorded at some sites are more damaging than assumed in design, especially at sites 931 and 961. Overloaded trucks are a significant source of damage. Trucks with overweight permits do not contribute significantly to the total accumulated damage. At sites where 20% or more of the trucks are overloaded, the overloaded trucks produce more fatigue damage than the legal traffic. Considering all data from all WIM sites for 2014 and 2015, 2.5% of the trucks are overloaded so that they do not meet ALDOT criteria for a permit, and those trucks produce more than 40% of the fatigue damage.

Chapter 7: Adequacy of AASHTO Fatigue Design Truck

7.1 Introduction

The fatigue in the steel bridges are major concern and a recently released *Innovative Bridge Design Handbook* (Pipinato 2015) states that ”ASCE Committee on Fatigue and Fracture Reliability (1982a, 1982b,1982c, 1982d) reported that 80%–90% of failures in steel structures are related to fatigue and fracture.”

In the most recent *AASHTO LRFD Bridge design specifications* (AASHTO 2017), the fatigue in steel bridge is addressed by two sources of fatigue: load-induced and distortion-induced fatigue. The load-induced fatigue is addressed by Fatigue I and Fatigue II limit state. The limit state is reached once the accumulated load spectra exceed the fatigue resistance of the material (Kulicki et al. 2015). Through NCHRP project 12-33 resulting in NCHRP Report 368 (Nowak 1999), only strength limit states were calibrated for LRFD design. Through the SHRP2 R19B project *Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Kulicki et al. 2015), the service and fatigue limit states were calibrated. The study used the WIM database to develop statistical models of fatigue load based on the WIM data from 15 states for 1 year. This chapter discusses the overview of the calibration framework and in detail about the fatigue limit state calibration. The methodology used to develop statistical parameters of fatigue load using the national WIM traffic database is discussed in detail. Using the available Alabama WIM traffic database, statistical parameters of fatigue load is developed and adequacy of the current AASHTO fatigue truck is checked. There are some of the reasonable assumptions made in the development of statistical parameters of fatigue load for Alabama.

7.2 Calibration Process

In the load and resistance factor design codes, safety reserve is provided in terms of load and resistance factors. The factors are determined through the reliability-based calibration procedure (Nowak and Collins 2012). The code calibration requires the knowledge of statistical parameters of load and resistance. Reliability is measured in terms of the reliability index. The load and resistance factors are selected so that the designed structures will have at least the minimum acceptable reliability, i.e., β will be at least equal to the target reliability index, β_T. The target reliability index depends on consequences of failure and relative cost (cost of a unit of safety).

The overview of the basic calibration framework is shown in Figure 7.1. The code calibration procedure (Kulicki et al. 2015; Nowak and Collins 2012) can be formulated as follows:

1. Formulate the limit state function and identify variables – For each considered limit state, the acceptability criteria are established. In this case, the fatigue limit state.
2. Identify and select representative structural types and design cases – in the case of fatigue limit state structural details prone to fatigue are considered.
3. Determine load and resistance parameters for the selected design cases – For the design of steel structures to fatigue, live load from the traffic and resistance of the material is considered.
4. Develop statistical models for load and resistance – Based on the variables determined, the statistical parameters for those variables are developed. In this case, statistical parameters for fatigue load are developed.
5. Develop a reliability analysis procedure - Reliability can be calculated using either a closed-form formula or simulation techniques like the Monte Carlo method. The reliability index for each case can be calculated using closed formulas available for particular types of probability distribution functions.
6. Calculate the reliability index for the current design code – The reliability index for the fatigue limit state in the current specification is calculated.
7. Review the results and select the target reliability index – Based on the calculated reliability index in the current specification and experience from the current engineering practice, the β_T, is selected.

8. Select potential load and resistance factors – The optimum values of load and resistance factors that correspond to the so-called “design point” are selected.
9. Calculate the reliability index for selected load and resistance factors – The reliability index corresponding to potential load and resistance factors is calculated for verification.

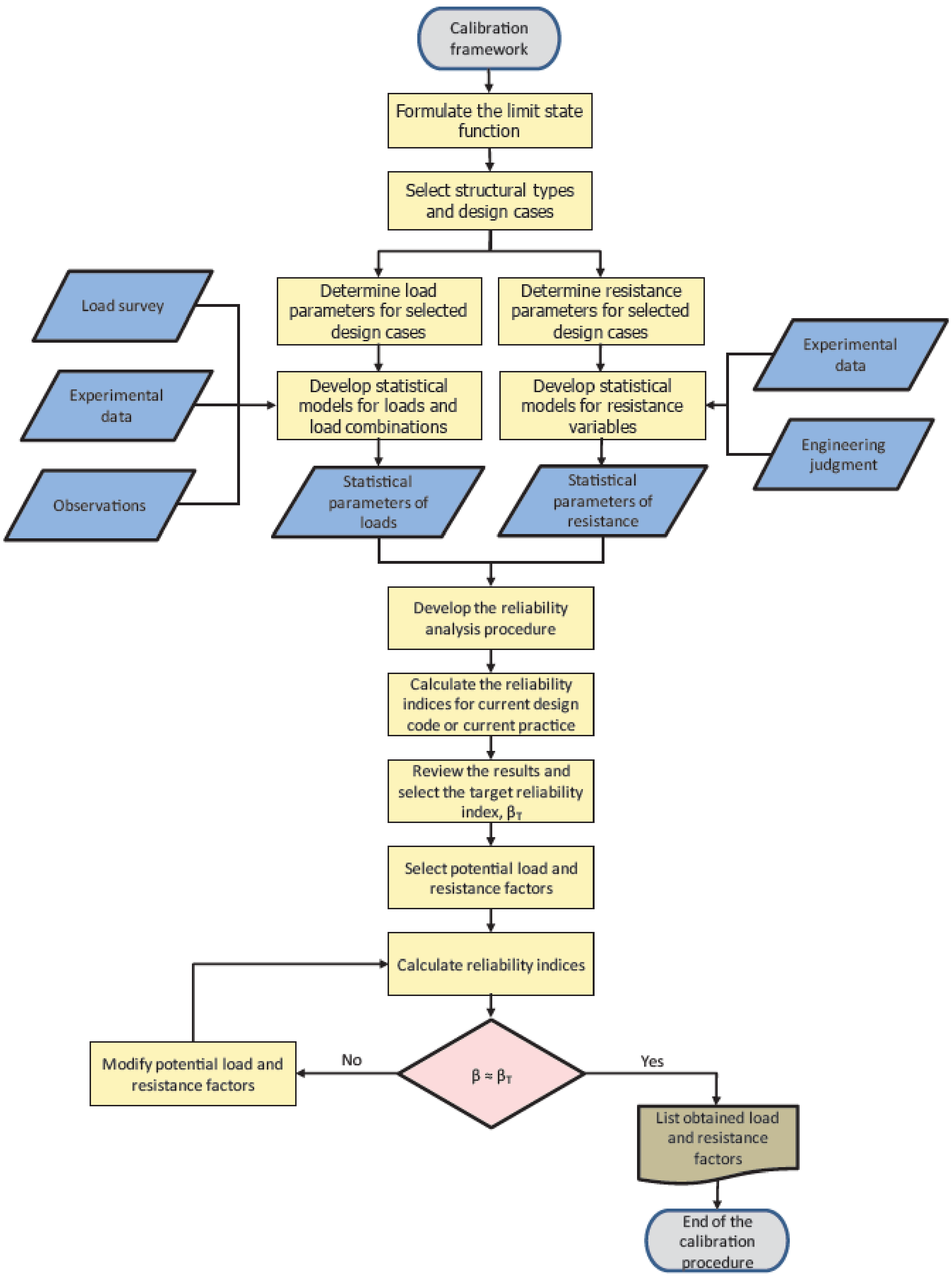


Figure 7.1: Basic calibration framework flowchart (Kulicki et al. 2015).

7.3 Statistical Parameters of Fatigue Load

For the design of new bridges and evaluation of existing bridges, the knowledge of load-carrying capacity and the accumulated loads is required. This section focuses on the development of statistical parameters of fatigue load. Firstly the methodology used in the SHRP2 R19B project - *Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Kulicki et al. 2015) is discussed. Later, the Alabama WIM traffic data is used, and the same methodology is followed to develop statistical parameters.

In Kulicki et al. 2015, the WIM data from 15 WIM sites for 15 different states were considered as a representative to develop statistical models for fatigue load. Each site had one year full of WIM recordings and for only one lane and in one direction. The filtering criteria were used to retain only regular truck traffic (only legal loads in the context of this dissertation), and only GVW greater than 20 kips were considered. Their cases were considered to develop fatigue load models. Three cases are:

1. Mid-span moment for a simply supported bridge.
2. Moment at the interior support of a two-span continuous bridge.
3. Moment at 0.4 of the span lengths of a continuous bridge.

The WIM trucks were run over an influence line to develop bending moment time history for span lengths of 30 ft, 60 ft, 90 ft, 120 ft, and 200 ft. The cycles produced by this are irregular with variable frequencies, so a suitable method is needed for cycle counting. Then the rainflow counting method was used to count cycles. Then Palmgren-Miner rule was used to account fatigue damage due to random variable amplitude loading. More details about the development of Fatigue Limit State I (called as Maximum moment range ratio) and Fatigue Limit State II (called Fatigue damage ratio) is discussed in the next sections.

7.3.1 Fatigue Limit State I

The Fatigue Limit State I refers to infinite load-induced fatigue life in *AASHTO LRFD Bridge design specifications* (AASHTO 2017). If most of the stress ranges due to traffic are below the threshold stress range (S_{Rth}) as shown in Figure 6.2, then the considered detail will have an infinite fatigue life. The philosophy that this limit state should have a probability of exceedance no more than 1:10,000 is used (Russo et al. 2016) and an assumption is made that the distribution of stress has the same shape of corresponding moments. The following steps were followed to obtain statistical parameters:

1. Run each truck over an influence line for various span lengths (30 ft to 200 ft) for different cases of fatigue detail under consideration.
2. The maximum moment range of each truck is noted, and CDF of the maximum moments is plotted.
3. The moment corresponding to the upper 0.01% or the probability of 0.9999 is considered. See Figure 7.2 for example.
4. The obtained values of the moment were divided by corresponding HL-93 fatigue moments to obtain maximum moment range ratio.
5. Then the ratios were fitted with a straight line to find distributions. The ratios followed a normal distribution, and statistical parameters mean (μ) and coefficient of variation (CV) were calculated, as shown in Table 7.1.
6. For reliability analysis procedure, the mean plus1.5 standard deviation ($\mu+1.5*\sigma$) was considered. This indicates the probability of exceeding the value is 5%, and 95% of the sites in the U.S. are below this value.
7. The statistical parameters were further simplified and mean of 2.0 HL-93, and the coefficient of variation of 0.12 was used for further calibration.

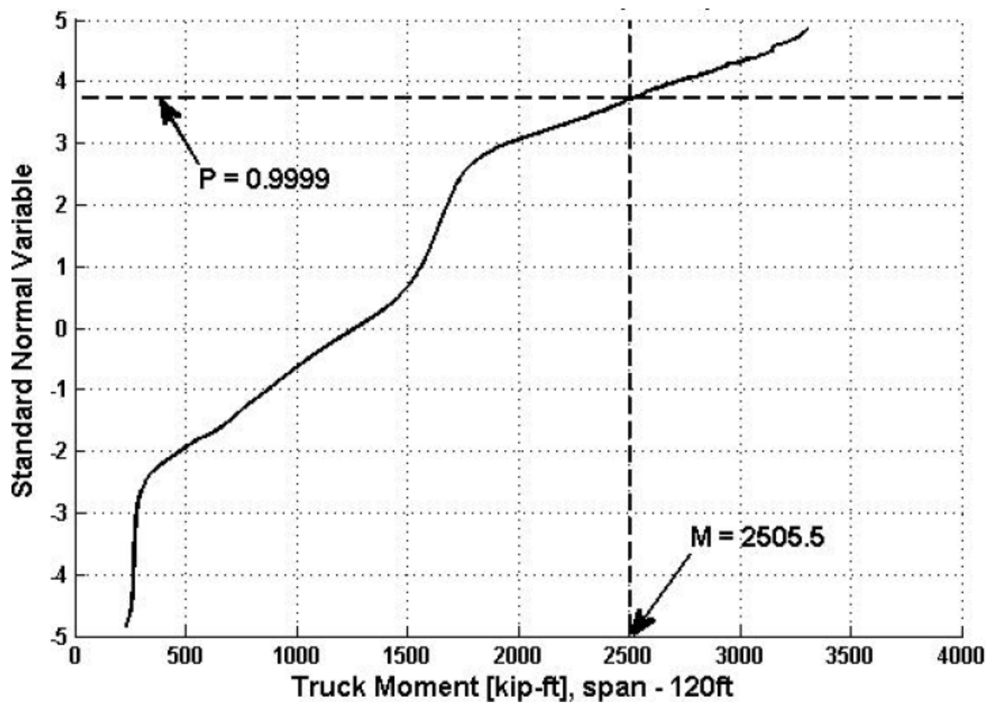


Figure 7.2: Moment corresponding to 0.01% for a span of 120 ft for Arkansas data. (Kulicki et al. 2015)

Table 7.1: Statistical parameters for Fatigue limit state I. (Kulicki et al. 2015)

Bridge Type	Span (ft)	Mean	Mean + 1.5σ	CV
Simple-supported midspan	30	1.60	1.90	0.13
	60	1.83	2.24	0.15
	90	1.60	1.96	0.15
	120	1.64	1.88	0.10
	200	1.70	2.15	0.18
Continuous middle support	30	1.35	1.61	0.13
	60	1.81	2.13	0.12
	90	1.92	2.18	0.09
	120	1.97	2.17	0.07
	200	2.27	2.47	0.06
Continuous 0.4 of the span length	30	1.54	1.86	0.14
	60	1.67	2.06	0.16
	90	1.60	1.92	0.13
	120	1.65	1.97	0.13
	200	1.72	2.11	0.15

7.3.2 Fatigue Limit State II

The Fatigue Limit State II refers to finite load-induced fatigue life in AASHTO LRFD Bridge design specifications (AASHTO 2017). The finite fatigue life depends on the magnitude and number of load cycles during the service life of the bridge. If the majority of the stress ranges due to traffic are above the threshold stress range (SRth) as shown in Figure 6.2, then the considered detail will have finite fatigue life. A ratio of fatigue damage due to the actual load to fatigue damage due to a design load called Fatigue damage ratio (λ) was used. This ratio is free from load factors, so it reflects the damage caused by actual traffic in terms of design load. The following steps were followed to obtain statistical parameters:

1. Run each truck over an influence line for various span lengths (30 ft to 200 ft) for different cases of fatigue detail under consideration to obtain bending moment time history.
2. Use of rainflow counting method (section 6.4.1) to count an actual number of cycles (N_R) and Palmgren-Miner rule of cumulative damage to find an equivalent moment (M_{eq}).
3. Then λ is calculated using Equation 7.20.

$$\lambda = \sqrt[3]{\frac{N_R}{N}} * \frac{M_{eq}}{M} \qquad (7.20)$$

where,

N_R = actual number of cycles

M_{eq} = equivalent moment from miners rule

M = moment due to fatigue design truck

N = number of cycles as shown in Chapter 6.8

4. Then λ was calculated considering number of cycles (n) in Equation 6.8 first according to previous *AASHTO LRFD Bridge design specifications* (AASHTO

2012) provisions as shown in Table 7.2 (a). Also, the λ was calculated based on the proposed number of cycles per truck passage, as shown in Table 7.2 (b).

Table 7.2: Number of cycles per truck passage (n) according to (a) *AASHTO LRFD Bridge design specifications* (AASHTO 2012) (b) Based on proposed fatigue design.

(a)

Longitudinal Members	Span Length	
	>40.0 ft	≤40.0 ft
Simple Span Girders	1.0	2.0
Continuous Girders		
1) near interior support	1.5	2.0
2) elsewhere	1.0	2.0
Cantilever Girders	5.0	
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling	5.0	
Trusses	1.0	
Transverse Members	Spacing	
	> 20.0 ft	≤20.0 ft
	1.0	2.0

(b)

Longitudinal Members		*n*
Simple-span girders		1.0
Continuous girders	Near interior support	1.5
	Elsewhere	1.0

5. The obtained values of the Fatigue damage ratio (λ) by considering the proposed number of cycles are used for the development of statistical parameters.
6. Then the fatigue damage ratios of the WIM stations were fitted with a straight line to find distributions. The ratios followed a normal distribution, and statistical parameters mean (μ) and coefficient of variation (CV) were calculated using the fatigue damage ratios calculated for each WIM station, as shown in Table 7.3.

7. For reliability analysis procedure, the mean plus1.5 standard deviation (μ+1.5*σ) was considered. This indicates the probability of exceeding the value is 5%, and 95% of the sites in the U.S. are below this value.
8. The statistical parameters were further simplified and mean of 0.8 HL-93, and the coefficient of variation of 0.07 was used for further calibration.

Table 7.3: Statistical parameters for Fatigue limit state II. (Kulicki et al. 2015)

Bridge Type	Span (ft)	Mean	Mean + 1.5σ	CV
Simple-supported midspan	30	0.79	0.87	0.07
	60	0.78	0.86	0.06
	90	0.73	0.81	0.07
	120	0.76	0.84	0.07
	200	0.78	0.86	0.07
Continuous middle support	30	0.59	0.65	0.07
	60	0.74	0.82	0.07
	90	0.69	0.77	0.07
	120	0.71	0.78	0.06
	200	0.79	0.87	0.07
Continuous 0.4 of the span length	30	0.73	0.81	0.07
	60	0.72	0.80	0.07
	90	0.68	0.75	0.07
	120	0.72	0.79	0.06
	200	0.76	0.84	0.07

7.4 Statistical Parameters of Fatigue Load for Alabama WIM Data

Using the procedure described in the above sections, the statistical parameters of fatigue load for Alabama WIM data were developed. Only one case that creates high effects, i.e. mid-span moment for a simply supported bridge, is considered.

Two assumptions are made to be consistent with the procedure followed in Kulicki et al. 2015. Alabama WIM data contained traffic in two lanes and two directions (refer Table 2.1), but the WIM data used to develop national fatigue load models contained WIM data in one direction and one lane. So, in the development of statistical parameters of fatigue load for Alabama WIM data, the lanes that create most damaging traffic are considered for developing statistical parameters.

Another assumption is WIM traffic data to be considered in developing statistical parameters. In Kulicki et al. 2015, the WIM records were filtered through multiple filtering criteria to eliminate inevitable errors, permit and illegally overloaded vehicles and vehicles less than 20 kips. In the development of statistical parameters of fatigue load for Alabama WIM data, the vehicles which were filtered through the state's TS&W limits (Figure 5.2) that are legal loads and represent regular truck traffic are used. The number of records considered for fatigue analysis is shown in Table 7.4. Table 7.5 shows the ADTT for each location.

Table 7.4: Summary of records for fatigue load analysis.

Station code	Name	Location	Number of records		
			2014	2015	2016
911	Alex City	US280 Coosa Co.	186,699	211,585	223,805
931	Athens	I65 Limestone Co.	561,597	560,269	544,804
933	Muscle Shoals	AL157 US72 Colbert Co.	280,197	290,050	291,102
934	Sumiton	US78 Walker Co.	121,357	67,394	73,422
942	Pine Level	US231 Montgomery Co.	372,205	391,658	408,512
960	Whatley	US84 Clark Co.	152,220	143,003	147,913
961	Mobile	I65 Mobile Co.	527,393	58,591	630,121
964	Ozark	US231 Dothan Co.	271,053	68,342	238,679
		Total	2,472,721	1,790,892	2,558,358

Table 7.5: ADTT of WIM locations in Alabama.

Station code	Name	Location	Average Daily Truck Traffic (ADTT)		
			2014	2015	2016
911	Alex City	US280 Coosa Co.	527	599	632
931	Athens	I65 Limestone Co.	1,556	1,552	1,501
933	Muscle Shoals	AL157 US72 Colbert Co.	785	808	856
934	Sumiton	US78 Walker Co.	390	192	220
942	Pine Level	US231 Montgomery Co.	1,070	1,100	1,154
960	Whatley	US84 Clark Co.	452	435	424
961	Mobile	I65 Mobile Co.	1,925	1,953	2,291
964	Ozark	US231 Dothan Co.	768	633	804

7.4.1 Fatigue Limit State I

For the development of statistical parameters for Fatigue limit state I, the procedure discussed in section 7.3.1 was followed but with the above-mentioned assumptions. 1/10,000 of maximum moment for simply supported spans at mid-span is shown in Table 7.6. The variation of 1/10,000 of the maximum moment is location specific, and it changes for each span length. However, there is a consistency in variation between years for each WIM location. Also, the Maximum moment range ratio, i.e. the ratio of 1/10,000 Moment Cycle to HL-93 Fatigue Moment, is shown in

Table 7.7. In almost all the locations, maximum moment range ratios for 60 ft span are highest. The ratios of 1/10,000 Moment Cycle to HL-93 Fatigue Moment is plotted on normal probability paper for the year 2014 in Figure 7.3. Each point in Figure 7.3 represents one of the WIM sites in Alabama for respective span lengths.

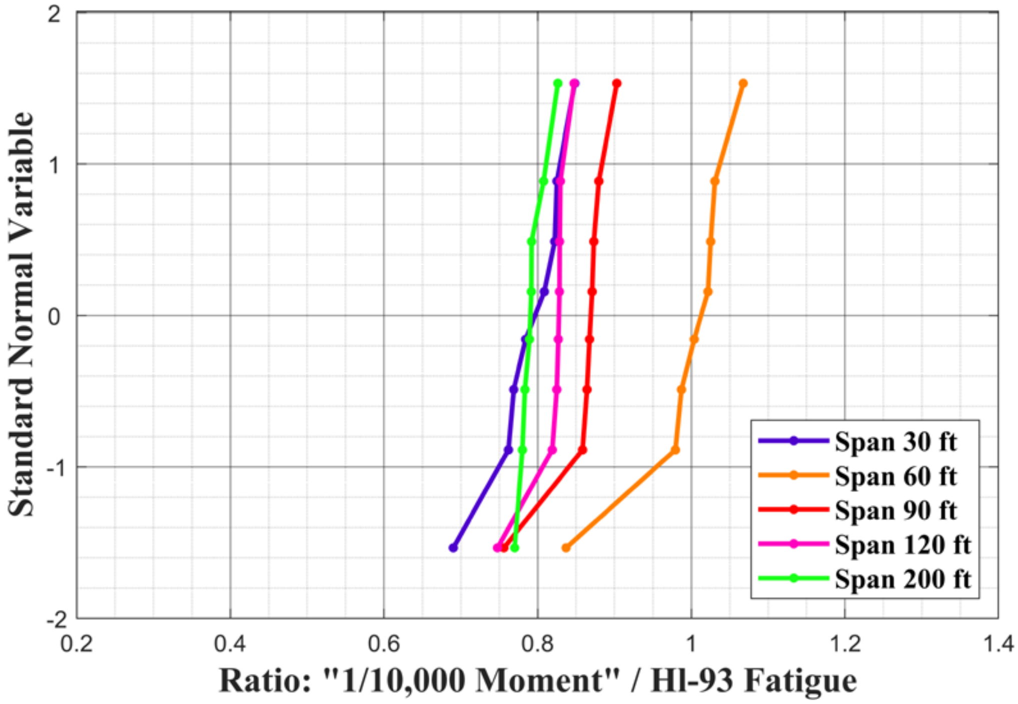


Figure 7.3: Maximum moment range ratio for simply supported bridges at mid-span for year 2014.

Table 7.6: 1/10,000 moment cycle for simply supported bridges at mid-span.

Station code	Name	Location	Span, ft	1/10,000 Moment Cycle (kip-ft)		
				2014	2015	2016
911	Alex City	US280 Coosa Co.	30ft	191	186	190
			60ft	548	526	543
			90ft	938	920	923
			120ft	1338	1313	1320
			200ft	2423	2378	2394
931	Athens	I65 Limestone Co.	30ft	168	166	172
			60ft	457	471	467
			90ft	820	832	820
			120ft	1213	1208	1207
			200ft	2356	2363	2363
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	201	207	196
			60ft	558	573	542
			90ft	947	964	930
			120ft	1341	1360	1322
			200ft	2388	2393	2382
934	Sumiton	US78 Walker Co.	30ft	197	189	196
			60ft	563	550	566
			90ft	954	947	962
			120ft	1346	1353	1358
			200ft	2424	2417	2417
942	Pine Level	US231 Montgomery Co.	30ft	186	194	191
			60ft	539	563	560
			90ft	941	959	957
			120ft	1344	1369	1359
			200ft	2416	2430	2427
960	Whatley	US84 Clark Co.	30ft	207	205	192
			60ft	583	581	560
			90ft	980	973	963
			120ft	1375	1368	1360
			200ft	2472	2460	2435
961	Mobile	I65 Mobile Co.	30ft	201	180	173
			60ft	560	516	490
			90ft	945	909	851
			120ft	1344	1303	1225
			200ft	2528	2355	2306
964	Ozark	US231 Dothan Co.	30ft	188	182	180
			60ft	535	513	525
			90ft	931	909	926
			120ft	1328	1310	1327
			200ft	2398	2370	2395

Table 7.7: Maximum moment range ratio for simply supported bridge at mid-span.

Station code	Name	Location	Span, ft	1/10,000 Moment Cycle / HL-93 Fatigue Moment		
				2014	2015	2016
911	Alex City	US280 Coosa Co.	30ft	0.78	0.76	0.78
			60ft	1.00	0.96	0.99
			90ft	0.86	0.85	0.85
			120ft	0.82	0.81	0.81
			200ft	0.79	0.78	0.78
931	Athens	I65 Limestone Co.	30ft	0.69	0.68	0.70
			60ft	0.84	0.86	0.86
			90ft	0.76	0.77	0.76
			120ft	0.75	0.74	0.74
			200ft	0.77	0.77	0.77
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	0.83	0.85	0.80
			60ft	1.02	1.05	0.99
			90ft	0.87	0.89	0.86
			120ft	0.83	0.84	0.82
			200ft	0.78	0.78	0.78
934	Sumiton	US78 Walker Co.	30ft	0.81	0.78	0.80
			60ft	1.03	1.01	1.04
			90ft	0.88	0.87	0.89
			120ft	0.83	0.83	0.84
			200ft	0.79	0.79	0.79
942	Pine Level	US231 Montgomery Co.	30ft	0.76	0.79	0.78
			60ft	0.99	1.03	1.02
			90ft	0.87	0.88	0.88
			120ft	0.83	0.84	0.84
			200ft	0.79	0.79	0.79
960	Whatley	US84 Clark Co.	30ft	0.85	0.84	0.79
			60ft	1.07	1.06	1.03
			90ft	0.90	0.90	0.89
			120ft	0.85	0.84	0.84
			200ft	0.81	0.80	0.80
961	Mobile	I65 Mobile Co.	30ft	0.82	0.74	0.71
			60ft	1.03	0.95	0.90
			90ft	0.87	0.84	0.78
			120ft	0.83	0.80	0.76
			200ft	0.83	0.77	0.75
964	Ozark	US231 Dothan Co.	30ft	0.77	0.75	0.74
			60ft	0.98	0.94	0.96
			90ft	0.86	0.84	0.85
			120ft	0.82	0.81	0.82
			200ft	0.78	0.77	0.78

7.4.2 Fatigue Limit State II

For the development of statistical parameters for Fatigue limit state II, the procedure discussed in section 7.3.2 was followed but with the above-mentioned assumptions. Since the maximum damage at each lane is considered, it is a function of both a number of cycles and effective moment, so NM_{eff}^3 is calculated and shown in Table 7.8. The NM_{eff}^3 is increasing as the span length increases. In Table 7.9, the fatigue damage ratio is shown. The variation is location specific, and there is a consistency in variation between years for each WIM location. The ratios of fatigue damage ratio are plotted on normal probability paper for the year 2014 in Figure 7.4. Each point in Figure 7.4 represents one of the WIM sites in Alabama for respective span lengths.

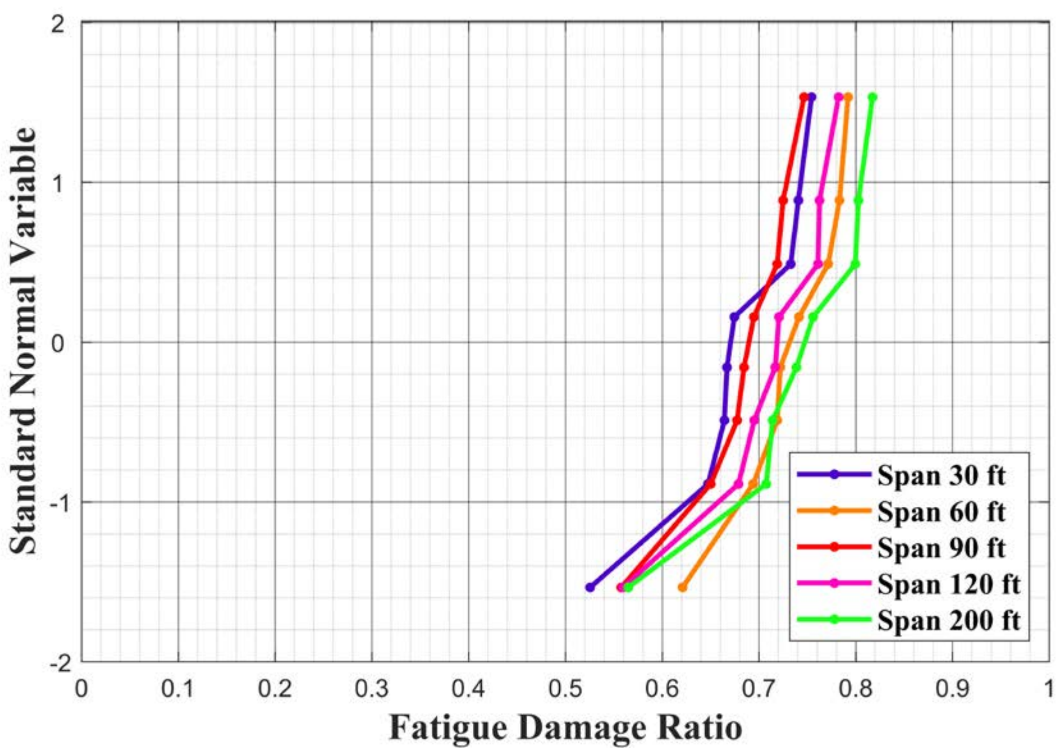


Figure 7.4: Fatigue damage ratio for simply supported bridges at mid-span for year 2014.

Table 7.8: NM_{eff}^3 for simply supported bridges at mid-span.

Station code	Name	Location	Span, ft	D=N*M_{eff}^3 [(kip-ft)^3 cycle]		
				2014	2015	2016
911	Alex City	US280 Coosa Co.	30ft	1.02E+12	9.79E+11	1.04E+12
			60ft	9.82E+12	1.02E+13	1.07E+13
			90ft	6.46E+13	6.74E+13	6.93E+13
			120ft	2.48E+14	2.54E+14	2.64E+14
			200ft	1.89E+15	1.90E+15	2.00E+15
931	Athens	I65 Limestone Co.	30ft	3.33E+12	2.73E+12	2.03E+12
			60ft	3.60E+13	3.01E+13	2.26E+13
			90ft	2.27E+14	1.88E+14	1.41E+14
			120ft	8.93E+14	7.34E+14	5.48E+14
			200ft	6.95E+15	5.68E+15	4.24E+15
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	1.74E+12	1.61E+12	1.37E+12
			60ft	1.66E+13	1.54E+13	1.37E+13
			90ft	1.18E+14	1.10E+14	9.61E+13
			120ft	4.39E+14	4.07E+14	3.57E+14
			200ft	3.24E+15	3.01E+15	2.64E+15
934	Sumiton	US78 Walker Co.	30ft	3.57E+11	2.06E+11	2.29E+11
			60ft	4.58E+12	3.17E+12	3.14E+12
			90ft	2.65E+13	1.79E+13	1.87E+13
			120ft	9.04E+13	5.90E+13	6.21E+13
			200ft	6.27E+14	3.96E+14	4.18E+14
942	Pine Level	US231 Montgomery Co.	30ft	3.23E+12	3.27E+12	3.44E+12
			60ft	2.91E+13	2.96E+13	3.12E+13
			90ft	1.95E+14	1.98E+14	2.08E+14
			120ft	7.58E+14	7.69E+14	8.10E+14
			200ft	5.82E+15	5.91E+15	6.23E+15
960	Whatley	US84 Clark Co.	30ft	9.57E+11	7.58E+11	7.84E+11
			60ft	9.33E+12	7.46E+12	7.77E+12
			90ft	6.24E+13	4.98E+13	5.18E+13
			120ft	2.28E+14	1.82E+14	1.90E+14
			200ft	1.66E+15	1.34E+15	1.40E+15
961	Mobile	I65 Mobile Co.	30ft	4.20E+12	4.51E+11	3.53E+12
			60ft	3.81E+13	4.07E+12	3.26E+13
			90ft	2.53E+14	2.68E+13	2.14E+14
			120ft	9.94E+14	1.05E+14	8.42E+14
			200ft	7.72E+15	8.19E+14	6.56E+15
964	Ozark	US231 Dothan Co.	30ft	2.23E+12	5.05E+11	1.83E+12
			60ft	2.05E+13	4.67E+12	1.71E+13
			90ft	1.27E+14	2.88E+13	1.06E+14
			120ft	5.08E+14	1.16E+14	4.24E+14
			200ft	4.02E+15	9.14E+14	3.35E+15

Table 7.9: Fatigue damage ratio for simply supported bridges at mid-span.

Station code	Name	Location	Span, ft	Fatigue Damage Ratio		
				2014	2015	2016
911	Alex City	US280 Coosa Co.	30ft	0.65	0.61	0.61
			60ft	0.69	0.67	0.67
			90ft	0.65	0.63	0.63
			120ft	0.68	0.66	0.65
			200ft	0.71	0.68	0.68
931	Athens	I65 Limestone Co.	30ft	0.66	0.62	0.57
			60ft	0.74	0.70	0.64
			90ft	0.68	0.64	0.59
			120ft	0.72	0.68	0.62
			200ft	0.76	0.71	0.65
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	0.67	0.65	0.61
			60ft	0.72	0.70	0.67
			90ft	0.69	0.67	0.64
			120ft	0.72	0.69	0.66
			200ft	0.74	0.71	0.68
934	Sumiton	US78 Walker Co.	30ft	0.53	0.53	0.54
			60ft	0.62	0.67	0.65
			90ft	0.56	0.60	0.59
			120ft	0.56	0.59	0.58
			200ft	0.56	0.59	0.58
942	Pine Level	US231 Montgomery Co.	30ft	0.75	0.74	0.75
			60ft	0.79	0.78	0.79
			90ft	0.75	0.74	0.74
			120ft	0.78	0.77	0.78
			200ft	0.82	0.81	0.81
960	Whatley	US84 Clark Co.	30ft	0.67	0.63	0.64
			60ft	0.72	0.68	0.69
			90ft	0.68	0.64	0.65
			120ft	0.70	0.65	0.67
			200ft	0.71	0.67	0.69
961	Mobile	I65 Mobile Co.	30ft	0.73	0.72	0.65
			60ft	0.77	0.76	0.69
			90ft	0.72	0.71	0.65
			120ft	0.76	0.75	0.68
			200ft	0.80	0.79	0.71
964	Ozark	US231 Dothan Co.	30ft	0.74	0.71	0.72
			60ft	0.78	0.76	0.77
			90ft	0.72	0.69	0.71
			120ft	0.76	0.74	0.75
			200ft	0.80	0.78	0.79

7.5 Checking the Adequacy of Alabama Fatigue Loads to Fatigue Loads in AASHTO

The objective of this chapter was to check whether the fatigue loads that are used in the development of fatigue limit states of *AASHTO LRFD Bridge design specifications* (AASHTO 2017) are adequate for the state of Alabama. If the statistical parameters (mean and CV) of fatigue load developed for Alabama is lesser than the statistical parameters of fatigue load of the national WIM data that is used in Kulicki et al. 2015, then it is adequate.

Table 7.10 shows the comparison for Fatigue Limit State I (maximum moment range), and all the statistical parameters for Alabama WIM data are less than national WIM data. The mean+1.5 σ of 2.0 HL93 and CV of 0.14 was obtained for simply supported midspan, but from Alabama WIM data mean+1.5 σ of 0.82 HL93 and CV of 0.05 was obtained. The results indicate AASTHO fatigue truck for this limit state is very conservative.

Table 7.11 shows the comparison for Fatigue Limit State II (fatigue damage ratio), and all the statistical parameters for Alabama WIM data are less than national WIM data. The mean+1.5 σ of 0.85 HL93 and CV of 0.07 was obtained for simply supported midspan but from Alabama WIM data mean+1.5 σ of 0.78 HL93 and CV of 0.09 was obtained.

Table 7.10: Comparison of maximum moment range ratio for Fatigue Limit State I for Alabama and National WIM data.

Year	Span	From Alabama WIM data			From National WIM data (Kulicki et al. 2015)		
		Mean	Mean + 1.5 σ	CV	Mean	Mean + 1.5 σ	CV
2014	30ft	0.70	0.78	0.07	1.60	1.90	0.13
	60ft	0.88	0.99	0.08	1.83	2.24	0.15
	90ft	0.76	0.83	0.06	1.60	1.96	0.15
	120ft	0.73	0.77	0.04	1.64	1.88	0.10
	200ft	0.70	0.73	0.02	1.70	2.15	0.18
2015	30ft	0.69	0.77	0.08	1.60	1.90	0.13
	60ft	0.87	0.98	0.08	1.83	2.24	0.15
	90ft	0.76	0.82	0.06	1.60	1.96	0.15
	120ft	0.72	0.77	0.05	1.64	1.88	0.10
	200ft	0.70	0.71	0.02	1.70	2.15	0.18
2016	30ft	0.68	0.74	0.06	1.60	1.90	0.13
	60ft	0.87	0.96	0.08	1.83	2.24	0.15
	90ft	0.75	0.82	0.07	1.60	1.96	0.15
	120ft	0.72	0.77	0.05	1.64	1.88	0.10
	200ft	0.69	0.71	0.02	1.70	2.15	0.18

Table 7.11: Comparison of fatigue damage ratio for Fatigue Limit State II for Alabama and National WIM data.

Year	Span	From Alabama WIM data			From National WIM data (Kulicki et al. 2015)		
		Mean	Mean + 1.5 σ	CV	Mean	Mean + 1.5 σ	CV
2014	30ft	0.68	0.79	0.11	0.79	0.87	0.07
	60ft	0.73	0.81	0.08	0.78	0.86	0.06
	90ft	0.68	0.77	0.09	0.73	0.81	0.07
	120ft	0.71	0.81	0.10	0.76	0.84	0.07
	200ft	0.74	0.86	0.11	0.78	0.86	0.07
2015	30ft	0.65	0.76	0.11	0.79	0.87	0.07
	60ft	0.71	0.78	0.06	0.78	0.86	0.06
	90ft	0.67	0.74	0.07	0.73	0.81	0.07
	120ft	0.69	0.78	0.09	0.76	0.84	0.07
	200ft	0.72	0.82	0.10	0.78	0.86	0.07
2016	30ft	0.64	0.74	0.11	0.79	0.87	0.07
	60ft	0.70	0.78	0.08	0.78	0.86	0.06
	90ft	0.65	0.73	0.08	0.73	0.81	0.07
	120ft	0.67	0.77	0.09	0.76	0.84	0.07
	200ft	0.70	0.81	0.10	0.78	0.86	0.07

7.6 Summary

The purpose of the AASHTO fatigue truck in *AASHTO LRFD Bridge design specifications* (AASHTO 2017) is to reflect the fatigue stress range caused by current legal truck traffic. The Fatigue Limit State I and Fatigue Limit State II was recently calibrated using the extensive WIM data through SHRP2 R19B project *Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Kulicki et al. 2015). Using the Alabama WIM data for years 2014 to 2016 the statistical parameters of fatigue load was calculated using the same procedure in Kulicki et al. 2015 but with the assumptions of taking traffic data from most damaging traffic lane and more realistic actual legal truck traffic. The results indicated that AASHTO fatigue truck is adequate for the state of Alabama. The Fatigue Limit state I is more conservative than Fatigue Limit state II based on the analysis.

Chapter 8: Summary, Conclusions, and Recommendations

8.1 Summary

Knowledge of current live loads helps in both law enforcement efforts and regular planned maintenance of bridges and pavements. Each heavy truck traveling across a bridge contributes to its accumulated damage or expenditure of its useful life. WIM data provides an excellent source to study truck loads and their effects on bridges.

The main objective of this dissertation is to develop procedures to evaluate the traffic-induced damage to bridges by the use of WIM data that can be implemented by ALDOT in day-to-day maintenance activities. The objectives also include the improvement of the procedures used to process raw WIM data, development of a procedure to evaluate the quality of the WIM data, development of a procedure to identify vehicles with permits issued by ALDOT in the WIM database, and development of procedures to convert the WIM measurements into an index of accumulated damage for bridges along the route.

WIM data from 12 WIM stations for the years 2014 to 2016 and issued permit data for the years 2014 and 2015 were provided by ALDOT. WIM data was encrypted and shared in raw format, which was free from any pre-filtering and representing the whole traffic database at the respective WIM station. The steps of WIM data conversion are shown in Figure 2.9. The first step of WIM data analysis is to evaluate quality. A QC procedure was developed to check the traffic data and detect the root cause of questionable recorded traffic data. Inconsistency in recording due to communication failure, operational problems with the sensor and drift in calibration can be interpreted using the proposed procedure. The proposed procedure consists of a completeness check, logical checks, and statistical checks. A review of the literature to identify the state-of-the-art was performed, and the database of issued permits was used to establish limits for threshold parameters.

Issued permit data from ALDOT for the years 2014 and 2015 were reviewed. The data contained both overweight and over-dimension permits. A filtering procedure was developed to filter over-dimensional permits and retain only overweight permits. Two analytical procedure to identify legal, permitted and illegal vehicles in the WIM records was developed. GIS routing procedure identified that 0.5 % of the overloaded vehicles had issued permits, whereas a data-driven procedure identified 2.7% of the overloaded vehicles had issued permits.

Traffic-induced loads can cause damage to bridge by fatigue and overload. Steel bridges are more prone to fatigue compared to other types of bridges. A procedure to evaluate damage accumulation in steel bridges was developed. The procedure can be used to assess WIM site-specific damage or bridge specific damage. WIM site-specific damage was used to make comparisons such as which of the WIM sites have the most damaging traffic and what types of trucks cause the most fatigue damage. Bridge specific damage includes assessing the damage at a fatigue prone detail in a particular bridge, and quantifying the relative damage of specific trucks such as the ALDOT rating trucks with respect to the AASHTO fatigue design truck.

Two computer apps were developed using developed procedures. The computer app, AL_WIM_QC v1.0, for evaluating the quality of the WIM data. Another application, AL_WIM_DAI v1.0, for estimating the fatigue damage in a steel girder bridge due to the traffic recorded at the WIM site. The results of the fatigue damage calculations can be used to evaluate the significance of the truck traffic along various routes and the impact of the various FHWA Vehicle Classes on the total damage.

Adequacy of current AASTHO fatigue design truck for Alabama was checked by using the procedures developed in SHRP2 R19B project *Bridges for Service Life Beyond 100 Years: Service Limit State Design* (Kulicki et al. 2015). The fatigue design truck was found to be suitably representative of the state of Alabama and envelopes the fatigue load of the current traffic.

8.2 Conclusions

Based on the results of analyses and procedures developed in this dissertation, the following conclusions are made:

1. The developed QC procedures identified malfunctioning of two WIM stations, 918 (Bucksville) and 963 (Grand Bay), and these stations are recommended for repair or replacement.
2. The developed logical check was found to be an effective QC procedure. It identified the malfunctioning of WIM site 918 (Bucksville). WIM data recorded at station 918 was excluded from further analysis.
3. The developed statistical check was found to be an effective procedure. It identified the malfunctioning of the sensor at WIM station 963 (Grand Bay). Truck statistics for that site were significantly different from other locations. Thus, WIM data recorded at this station is questionable and excluded from further analysis.
4. It was found that less than 0.5% of overweight vehicles operate with a permit based on the combined WIM and issued permit data for 2014 and 2015 using GIS routing procedure, whereas data-driven procedure identified 2.7% of overweight vehicles operate with a permit.

5. WIM location 931 (Athens) accumulated the most fatigue damage in 2014 and 2015 followed by 961 (Mobile) in 2014 and 942 (Pine Level) in 2015.
6. Based on the combined WIM data, 20% of the vehicles are overloaded (both permit loads and illegal loads), and they cause more than 50% of the total fatigue damage.
7. WIM sites 931(Athens), 960 (Whatley) and 961 (Mobile) have the highest percentage of overweight vehicles.
8. Group of vehicles with a permit and those that meet permit criteria do less fatigue damage than either legal or illegal group of vehicles.
9. The 16-18% of trucks that are illegally overloaded create more than 40% of the total damage.
10. Five axle Class 9 trucks cause more than 70% of fatigue damage.
11. For traffic recorded at WIM site 931 (Athens), the fatigue life of steel girder bridges is consumed four times faster than expected for a design life of 75 years. For traffic recorded at WIM sites other than 931 (Athens), 942 (Pine Level), and 961 (Mobile), the fatigue life of steel girder bridges is consumed slower than expected for a design life of 75 years.
12. The current AASHTO fatigue design truck is adequate for the design of new bridges in the state of Alabama. Based on the analysis, the Fatigue Limit state I was more conservative than Fatigue Limit state II.
13. Two computer apps were developed: (a) AL_WIM_QC for help in the timely identification of malfunctioning of WIM systems, (b) AL_WIM_DAI computer app provides significant information about the impact of traffic and damage accumulated on the bridge.

8.3 Recommendations for Future Research

The following recommendations are offered for potential future research based on the scope of this dissertation:

1. The procedure for quantifying the fatigue damage can be extended to other bridge types such as concrete and prestressed concrete bridges.
2. The damage assessment model can be linked to a cost model to obtain damage in terms of monetary value.
3. Identification of issued permit vehicles in the WIM database procedure can be further improved if the routes of travel are available in geo-coordinates.
4. The images from virtual WIM stations can be used to further improve the soundness of the QC procedure.
5. An extensive study by using the damage assessment of pavements can provide justifications for change in current permit fees and structure, thereby assigning fair costs for overweight transporters and to the public.

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Appendix A: Vehicle Class Description of State of Alabama on State Highways

Compliance data is available.
Tandem spacing is 8.00 ft or less.
Tridem spacing is 11.00 ft or less with equal spacing tolerance less than 0.33 ft and equal weight tolerance n/a.
Quadrem spacing is unused.
Class table has classes 0-13 and 27 vehicle type definitions. Error vehicles are class 0.
Unclassified vehicles are 0.
Autocal vehicle definition is type 18.

Weights are in pounds and lengths are in feet.

Axles	Class	Cpl-Table	Gross Weight	Loop Detuning	Overall length	Front overhang	Rear overhang
Axle	Type	SpacingRange	Axle Weight	Groups			Page 1
2	1	THE ACTUAL DAT	any	any	any	any	
	any A1	x	NOmin-3500				
A2	x	min-5.84	any				
2	2		any	any	any	any	any
A1	s	NO	min-3500				
A2	s	5.84-10.00	any				
2	3		any	any	any	any	any
A1	s	NO	min-3500				
A2	s	10.00-20.00	any				
3	2		any	any	any	any	any
A1	s	NO	min-3500				
A2	s	min-10.00	any				
A3	s	min-20.00	any				
4	2		any	any	any	any	any
A1	s	NO	min-3500				
A2	s	min-10.00	any				
A3	d	min-20.00	any				
A4	d	min-4.00	any				
3	3		any	any	any	any	any
A1	s	NO	min-3500				
A2	s	10.00-max	any				
A3	s	min-20.00	any				
4	3		any	any	any	any	any
A1	s	NO	min-4409				
A2	s	10.00-max	any				
A3	d	min-20.00	any				
A4	d	min-4.00	any				
2	5		any	any	any	any	
	any A1	s	YES/1	3500-max			
A2	s	min-20.00	any				
2	4		any	any	any	any	
	any A1	s	YES/1	3500-max			
A2	s	20.00-max	any				
3	6		any	any	any	any	any
A1	s	YES/1	any				
A2	d	min-20.00	any				
A3	d	min-5.84	any				
3	8		any	any	any	any	any
A1	x	YES/1	any				
A2	x	min-20.00	any				
A3	x	5.84-max	any				
3	4		any	any	any	any	any
A1	x	YES/1	any				
A2	x	20.00-max	any				
A3	x	any	any				
4	7		any	any	any	any	any

A1	s	YES/1	any
A2	x	any	any
A3	x	min-9.84	any
A4	x	min-5.84	any

Axles	Class Type	Cpl-Table SpacingRange	Gross Weight Axle Weight	Loop Detuning Groups	Overall length	Front overhang	Rear overhang	Axle Page
2								
4	7		any	any	any	any	any	
A1	d	YES/1	any					
A2	d	min-5.84	any					
A3	d	any	any					
A4	d	min-5.84	any					
5	7		any	any	any	any	any	
A1	s	YES/1	any					
A2	x	any	any					
A3	x	min-5.84	any					
A4	x	min-5.84	any					
A5	x	min-5.84	any					
6	7		any	any	any	any	any	
A1	s	YES/1	any					
A2	x	any	any					
A3	x	min-5.84	any					
A4	x	min-5.84	any					
A5	x	min-5.84	any					
A6	x	min-5.84	any					
4	8		any	any	any	any	any	
A1	x	YES/1	any					
A2	x	any	any					
A3	x	any	any					
A4	x	any	any					
5	9		any	any	any	any	any	
A1	s	YES/1	any					
A2	d	any	any					
A3	d	min-5.84	any					
A4	x	any	any					
A5	x	min-11.68	any					
5	11 Auto		any	any	any	any		
	any A1	x	YES/1	any				
A2	x	min-14.17	any					
A3	x	any	any					
A4	x	any	any					
A5	x	any	any					
5	9		any	any	any	any	any	
A1	x	YES/1	any					
A2	x	any	any					
A3	x	any	any					
A4	x	any	any					
A5	x	any	any					
6	10		any	any	any	any	any	
A1	s	YES/1	any					
A2	d	any	any					
A3	d	min-5.84	any					
A4	x	any	any					
A5	x	any	any					
A6	x	any	any					
6	10		any	any	any	any	any	
A1	s	YES/1	any					
A2	s	any	any					
A3	x	any	any					
A4	x	min-5.84	any					
A5	x	min-5.84	any					
A6	x	min-5.84	any					
6	12		any	any	any	any	any	
A1	x	YES/1	any					
A2	x	any	any					
A3	x	any	any					
A4	x	any	any					
A5	x	any	any					
A6	x	any	any					

Axles Axle	Class Type	Cpl-Table SpacingRange	Gross Weight Axle Weight	Loop Detuning Groups	Overall length	Front overhang	Rear overhang
7	13		any	any	any	any	any
A1	x	YES/1	any				
A2	x	min-14.17	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
7	13		any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
8	13		any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
A8	x	any	any				
9	13		any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
A8	x	any	any				
A9	x	any	any				

There are 8 subtables.

Weights are in pounds and lengths are in feet.

Distance Single Tandem Tridem Axle limit Group limit

Table 1 is used 1 time and has 1 line
and is named "Tandem Table - use 8ft value in 2 Axle table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
n/a	n/a	n/a	n/a	n/a	39600

Table 2 is used 1 time and has 1 line
and is named "Tridem Table - use 11ft value in 3 axle table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
n/a	n/a	n/a	n/a	n/a	48400

Table 3 is used 1 time and has 1 line
and is named "GVW Table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
n/a	n/a	n/a	n/a	n/a	92400

Table 4 is used 1 time and has 3 lines
and is named "2 Axle Table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
min-8.00	n/a	n/a	n/a	n/a	39600
8.00-9.00	n/a	n/a	n/a	n/a	41800
9.00-10.00	n/a	n/a	n/a	n/a	44000

Table 5 is used 1 time and has 22 lines
and is named "3 Axle Table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
min-8.00	n/a	n/a	n/a	n/a	46200
8.00-9.00	n/a	n/a	n/a	n/a	46748
9.00-10.00	n/a	n/a	n/a	n/a	47849
10.00-11.00	n/a	n/a	n/a	n/a	48400
11.00-12.00	n/a	n/a	n/a	n/a	49500
12.00-13.00	n/a	n/a	n/a	n/a	50049
13.00-14.00	n/a	n/a	n/a	n/a	51149
14.00-15.00	n/a	n/a	n/a	n/a	51700
15.00-16.00	n/a	n/a	n/a	n/a	52800
16.00-17.00	n/a	n/a	n/a	n/a	53349
17.00-18.00	n/a	n/a	n/a	n/a	54449
18.00-19.00	n/a	n/a	n/a	n/a	55000
19.00-20.00	n/a	n/a	n/a	n/a	56100
20.00-21.00	n/a	n/a	n/a	n/a	56649
21.00-22.00	n/a	n/a	n/a	n/a	57750
22.00-23.00	n/a	n/a	n/a	n/a	58300
23.00-24.00	n/a	n/a	n/a	n/a	59400
24.00-25.00	n/a	n/a	n/a	n/a	59950
25.00-26.00	n/a	n/a	n/a	n/a	61600
26.00-27.00	n/a	n/a	n/a	n/a	62700
27.00-28.00	n/a	n/a	n/a	n/a	64900
28.00-29.00	n/a	n/a	n/a	n/a	66000

Distance	Single	Tandem	Tridem	Axle limit	Group limit

Table 6 is used 1 time and has 37 lines
and is named "4 Axle Table"

min-8.00	n/a	n/a	n/a	n/a	46200
8.00-9.00	n/a	n/a	n/a	n/a	46748
9.00-10.00	n/a	n/a	n/a	n/a	47849
10.00-11.00	n/a	n/a	n/a	n/a	48400
11.00-12.00	n/a	n/a	n/a	n/a	55000
12.00-13.00	n/a	n/a	n/a	n/a	55549
13.00-14.00	n/a	n/a	n/a	n/a	56649
14.00-15.00	n/a	n/a	n/a	n/a	57200
15.00-16.00	n/a	n/a	n/a	n/a	57750
16.00-17.00	n/a	n/a	n/a	n/a	58850
17.00-18.00	n/a	n/a	n/a	n/a	59400
18.00-19.00	n/a	n/a	n/a	n/a	59950
19.00-20.00	n/a	n/a	n/a	n/a	61050
20.00-21.00	n/a	n/a	n/a	n/a	61600
21.00-22.00	n/a	n/a	n/a	n/a	62150
22.00-23.00	n/a	n/a	n/a	n/a	63250
23.00-24.00	n/a	n/a	n/a	n/a	63800
24.00-25.00	n/a	n/a	n/a	n/a	64350
25.00-26.00	n/a	n/a	n/a	n/a	65450
26.00-27.00	n/a	n/a	n/a	n/a	66000
27.00-28.00	n/a	n/a	n/a	n/a	66550
28.00-29.00	n/a	n/a	n/a	n/a	67650
29.00-30.00	n/a	n/a	n/a	n/a	68200
30.00-31.00	n/a	n/a	n/a	n/a	69848
31.00-32.00	n/a	n/a	n/a	n/a	70949
32.00-33.00	n/a	n/a	n/a	n/a	71500
33.00-34.00	n/a	n/a	n/a	n/a	72049
34.00-35.00	n/a	n/a	n/a	n/a	73149
35.00-36.00	n/a	n/a	n/a	n/a	73700
36.00-37.00	n/a	n/a	n/a	n/a	74800
37.00-38.00	n/a	n/a	n/a	n/a	75900
38.00-39.00	n/a	n/a	n/a	n/a	77000
39.00-40.00	n/a	n/a	n/a	n/a	78100
40.00-41.00	n/a	n/a	n/a	n/a	79200
41.00-42.00	n/a	n/a	n/a	n/a	80300
42.00-43.00	n/a	n/a	n/a	n/a	81400
43.00-44.00	n/a	n/a	n/a	n/a	82500

Table 7 is used 1 time and has 33 lines
and is named "5 Axle Table"

min-12.00	n/a	n/a	n/a	n/a	55000
12.00-13.00	n/a	n/a	n/a	n/a	55549
13.00-14.00	n/a	n/a	n/a	n/a	56649
14.00-15.00	n/a	n/a	n/a	n/a	57200
15.00-16.00	n/a	n/a	n/a	n/a	63800
16.00-17.00	n/a	n/a	n/a	n/a	64350
17.00-18.00	n/a	n/a	n/a	n/a	64900
18.00-19.00	n/a	n/a	n/a	n/a	66000
19.00-20.00	n/a	n/a	n/a	n/a	66550
20.00-21.00	n/a	n/a	n/a	n/a	67100
21.00-22.00	n/a	n/a	n/a	n/a	67650
22.00-23.00	n/a	n/a	n/a	n/a	68748
23.00-24.00	n/a	n/a	n/a	n/a	69300
24.00-25.00	n/a	n/a	n/a	n/a	69848
25.00-26.00	n/a	n/a	n/a	n/a	70400
26.00-27.00	n/a	n/a	n/a	n/a	71500
27.00-28.00	n/a	n/a	n/a	n/a	72049
28.00-29.00	n/a	n/a	n/a	n/a	72600
29.00-30.00	n/a	n/a	n/a	n/a	73149
30.00-31.00	n/a	n/a	n/a	n/a	73700
31.00-32.00	n/a	n/a	n/a	n/a	74800
32.00-33.00	n/a	n/a	n/a	n/a	75900
33.00-34.00	n/a	n/a	n/a	n/a	77000
34.00-35.00	n/a	n/a	n/a	n/a	78100
35.00-36.00	n/a	n/a	n/a	n/a	79200
36.00-37.00	n/a	n/a	n/a	n/a	80300
37.00-38.00	n/a	n/a	n/a	n/a	81400
38.00-39.00	n/a	n/a	n/a	n/a	82500
39.00-40.00	n/a	n/a	n/a	n/a	83600
40.00-41.00	n/a	n/a	n/a	n/a	84700
41.00-42.00	n/a	n/a	n/a	n/a	85800
42.00-43.00	n/a	n/a	n/a	n/a	86900
43.00-44.00	n/a	n/a	n/a	n/a	88000

Table 8 is used 1 time and has 29 lines
and is named "6 Axle Table"

Distance	Single	Tandem	Tridem	Axle limit	Group limit
min-16.00	n/a	n/a	n/a	n/a	63800
16.00-17.00	n/a	n/a	n/a	n/a	64350
17.00-18.00	n/a	n/a	n/a	n/a	64900
18.00-19.00	n/a	n/a	n/a	n/a	66000
19.00-20.00	n/a	n/a	n/a	n/a	72600
20.00-21.00	n/a	n/a	n/a	n/a	73149
21.00-22.00	n/a	n/a	n/a	n/a	73700
22.00-23.00	n/a	n/a	n/a	n/a	74800
23.00-24.00	n/a	n/a	n/a	n/a	75349
24.00-25.00	n/a	n/a	n/a	n/a	75900
25.00-26.00	n/a	n/a	n/a	n/a	76449
26.00-27.00	n/a	n/a	n/a	n/a	77000
27.00-28.00	n/a	n/a	n/a	n/a	78100
28.00-29.00	n/a	n/a	n/a	n/a	78649
29.00-30.00	n/a	n/a	n/a	n/a	79200
30.00-31.00	n/a	n/a	n/a	n/a	79749
31.00-32.00	n/a	n/a	n/a	n/a	80850
32.00-33.00	n/a	n/a	n/a	n/a	81400
33.00-34.00	n/a	n/a	n/a	n/a	81950
34.00-35.00	n/a	n/a	n/a	n/a	82500
35.00-36.00	n/a	n/a	n/a	n/a	83600
36.00-37.00	n/a	n/a	n/a	n/a	84700
37.00-38.00	n/a	n/a	n/a	n/a	85800
38.00-39.00	n/a	n/a	n/a	n/a	86900
39.00-40.00	n/a	n/a	n/a	n/a	88000
40.00-41.00	n/a	n/a	n/a	n/a	89100
41.00-42.00	n/a	n/a	n/a	n/a	90200
42.00-43.00	n/a	n/a	n/a	n/a	91300
43.00-44.00	n/a	n/a	n/a	n/a	92400

===

There is 1 structure.

Weights are in pounds and lengths are in feet.

Table 1 is used 20 times
Named "THE ACTUAL DATA IS STARTED HERE"
Steering axle maximum allowed weight of 22000 lb Single
axle maximum allowed weight of 22000 lb Maximum vehicle
length is unused.
Spacing tolerance of is unused.
Maximum kingpin distance is unused.
Maximum vehicle width is unused.

Tandem axle spacing type is 2 OUTER_AXLES_SPC_TYP
Tandem axle balance factor is unused.
Tandem axle spacing table is 1 "Tandem Table - use 8ft value in 2 Axle table"

Tridem axle spacing type is 2 OUTER_AXLES_SPC_TYP
Tridem axle balance factor is unused.
Tridem axle spacing table is 2 "Tridem Table - use 11ft value in 3 axle table" Quadrem axle

spacing type is 0 NO_SPC_TYP

Gross weight spacing type is 0 NO_SPC_TYP Gross
weight spacing table is 3 "GVW Table"

Axle grouping spacing type is 2 OUTER_AXLES_SPC_TYP Axle
grouping grouping type is 2 ADJ_AXLES_GRP_TYP Axle grouping
spacing table 3 axle is 4 "2 Axle Table" Axle grouping
spacing table 4 axle is 5 "3 Axle Table" Axle grouping
spacing table 5 axle is 6 "4 Axle Table" Axle grouping
spacing table 6 axle is 7 "5 Axle Table" Axle grouping
spacing table 7 axle is 8 "6 Axle Table"

Appendix B: Vehicle Class Description of State of Alabama on Interstate

Compliance data is available.
Tandem spacing is 8.00 ft or less.
Tridem spacing is 11.00 ft or less with equal spacing tolerance less than 0.33 ft and equal weight tolerance n/a.
Quadrem spacing is unused.
Class table has classes 0-13 and 27 vehicle type definitions. Error
vehicles are class 0.
Unclassified vehicles are 0.
Autocal vehicle definition is type 18.

Weights are in pounds and lengths are in feet.

Axles Axle	Class Type	Cpl-Table SpacingRange	Gross Weight Axle Weight	Loop Detuning Groups	Overall length	Front overhang	Rear overhang Page 1
2	1	Type 1	any	any	any	any	
	any A1	x	NOmin-3500				
A2	x	min-5.84	any				
2	2	Type 2	any	any	any	any	
	any A1	s	NOmin-3500				
A2	s	5.84-10.00	any				
2	3	Type 3	any	any	any	any	
	any A1	s	NOmin-3500				
A2	s	10.00-20.00	any				
3	2	Type 4	any	any	any	any	
	any A1	s	NOmin-3500				
A2	s	min-10.00	any				
A3	s	min-20.00	any				
4	2	Type 5	any	any	any	any	
	any A1	s	NOmin-3500				
A2	s	min-10.00	any				
A3	d	min-20.00	any				
A4	d	min-4.00	any				
3	3	Type 6	any	any	any	any	
	any A1	s	NOmin-3500				
A2	s	10.00-max	any				
A3	s	min-20.00	any				
4	3	Type 7	any	any	any	any	
	any A1	s	NOmin-4409				
A2	s	10.00-max	any				
A3	d	min-20.00	any				
A4	d	min-4.00	any				
2	5	Type 8	any	any	any	any	
	any A1	s	YES/1	3500-max			
A2	s	min-20.00	any				
2	4	Type 9	any	any	any	any	
	any A1	s	YES/1	3500-max			
A2	s	20.00-max	any				
3	6	Type 10	any	any	any	any	
	any A1	s	YES/1	any			
A2	d	min-20.00	any				
A3	d	min-5.84	any				
3	8	Type 11	any	any	any	any	
	any A1	x	YES/1	any			
A2	x	min-20.00	any				
A3	x	5.84-max	any				
3	4	Type 12	any	any	any	any	
	any A1	x	YES/1	any			
A2	x	20.00-max	any				
A3	x	any	any				
4	7	Type 13	any	any	any	any	any
A1	s	YES/1	any				

A2	x	any	any
A3	x	min-9.84	any
A4	x	min-5.84	any

Axles Axle	Class Type	Cpl-Table SpacingRange	Gross Weight Axle Weight	Loop Detuning Groups	Overall length	Front overhang	Rear overhang
4	7	Type 14	any	any	any	any	any
A1	d	YES/1	any				
A2	d	min-5.84	any				
A3	d	any	any				
A4	d	min-5.84	any				
5	7	Type 15	any	any	any	any	any
A1	s	YES/1	any				
A2	x	any	any				
A3	x	min-5.84	any				
A4	x	min-5.84	any				
A5	x	min-5.84	any				
6	7	Type 16	any	any	any	any	any
A1	s	YES/1	any				
A2	x	any	any				
A3	x	min-5.84	any				
A4	x	min-5.84	any				
A5	x	min-5.84	any				
A6	x	min-5.84	any				
4	8	Type 17	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
5	9	Type 18	any	any	any	any	any
A1	s	YES/1	any				
A2	d	any	any				
A3	d	min-5.84	any				
A4	x	any	any				
A5	x	min-11.68	any				
5	11 Auto	Type 19	any	any	any	any	any
A1	x	YES/1	any				
A2	x	min-14.17	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
5	9	Type 20	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
6	10	Type 21	any	any	any	any	any
A1	s	YES/1	any				
A2	d	any	any				
A3	d	min-5.84	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
6	10	Type 22	any	any	any	any	any
A1	s	YES/1	any				
A2	s	any	any				
A3	x	any	any				
A4	x	min-5.84	any				
A5	x	min-5.84	any				
A6	x	min-5.84	any				
6	12	Type 23	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				

Axles Axle	Class Type	Cpl-Table SpacingRange	Gross Weight Axle Weight	Loop Detuning Groups	Overall length	Front overhang	Rear overhang
7	13	Type 24	any	any	any	any	any
A1	x	YES/1	any				
A2	x	min-14.17	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
7	13	Type 25	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
8	13	Type 26	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
A8	x	any	any				
9	13	Type 27	any	any	any	any	any
A1	x	YES/1	any				
A2	x	any	any				
A3	x	any	any				
A4	x	any	any				
A5	x	any	any				
A6	x	any	any				
A7	x	any	any				
A8	x	any	any				
A9	x	any	any				

There are 9 subtables.

Weights are in pounds and lengths are in feet.

Distance	Single	Tandem	Tridem	Axle limit	Group limit

Table 1 is used 1 time and has 1 line
and is named "Tandem table - Value based on 8ft Tandem definition from Randy Braden"

n/a	n/a	n/a	n/a	n/a	34000

Table 2 is used 1 time and has 1 line
and is named "Tridem Table - Value based on 11ft Tridem Definition from Randy Braden"

n/a	n/a	n/a	n/a	n/a	44000

Table 3 is used 1 time and has 1 line
and is named "GVW Table - Max GVW 80,000 lbs"

n/a	n/a	n/a	n/a	n/a	80000

Table 4 is used 1 time and has 3 lines
and is named "2 axle group table"

min-8.04	n/a	n/a	n/a	n/a	38000
8.04-9.00	n/a	n/a	n/a	n/a	39000
9.00-10.00	n/a	n/a	n/a	n/a	40000

Table 5 is used 1 time and has 26 lines
and is named "3 axle group table"

min-8.00	n/a	n/a	n/a	n/a	34000
8.00-8.04	n/a	n/a	n/a	n/a	42000
8.04-9.00	n/a	n/a	n/a	n/a	42500
9.00-10.00	n/a	n/a	n/a	n/a	43500
10.00-11.00	n/a	n/a	n/a	n/a	44000
11.00-12.00	n/a	n/a	n/a	n/a	45000
12.00-13.00	n/a	n/a	n/a	n/a	45500
13.00-14.00	n/a	n/a	n/a	n/a	46500
14.00-15.00	n/a	n/a	n/a	n/a	47000
15.00-16.00	n/a	n/a	n/a	n/a	48000
16.00-17.00	n/a	n/a	n/a	n/a	48500
17.00-18.00	n/a	n/a	n/a	n/a	49500
18.00-19.00	n/a	n/a	n/a	n/a	50000
19.00-20.00	n/a	n/a	n/a	n/a	51000
20.00-21.00	n/a	n/a	n/a	n/a	51500
21.00-22.00	n/a	n/a	n/a	n/a	52500
22.00-23.00	n/a	n/a	n/a	n/a	53000
23.00-24.00	n/a	n/a	n/a	n/a	54000
24.00-25.00	n/a	n/a	n/a	n/a	54500
25.00-26.00	n/a	n/a	n/a	n/a	55500
26.00-27.00	n/a	n/a	n/a	n/a	56000
27.00-28.00	n/a	n/a	n/a	n/a	57000
28.00-29.00	n/a	n/a	n/a	n/a	57500
29.00-30.00	n/a	n/a	n/a	n/a	58500
30.00-31.00	n/a	n/a	n/a	n/a	59000
31.00-32.00	n/a	n/a	n/a	n/a	60000

Distance	Single	Tandem	Tridem	Axle limit	Group limit

Table 6 is used 1 time and has 46 lines
and is named "4 axle group table"

min-12.00	n/a	n/a	n/a	n/a	50000
12.00-13.00	n/a	n/a	n/a	n/a	50500
13.00-14.00	n/a	n/a	n/a	n/a	51500
14.00-15.00	n/a	n/a	n/a	n/a	52000
15.00-16.00	n/a	n/a	n/a	n/a	52500
16.00-17.00	n/a	n/a	n/a	n/a	53500
17.00-18.00	n/a	n/a	n/a	n/a	54000
18.00-19.00	n/a	n/a	n/a	n/a	54500
19.00-20.00	n/a	n/a	n/a	n/a	55500
20.00-21.00	n/a	n/a	n/a	n/a	56000
21.00-22.00	n/a	n/a	n/a	n/a	56500
22.00-23.00	n/a	n/a	n/a	n/a	57500
23.00-24.00	n/a	n/a	n/a	n/a	58000
24.00-25.00	n/a	n/a	n/a	n/a	58500
25.00-26.00	n/a	n/a	n/a	n/a	59500
26.00-27.00	n/a	n/a	n/a	n/a	60000
27.00-28.00	n/a	n/a	n/a	n/a	60500
28.00-29.00	n/a	n/a	n/a	n/a	61500
29.00-30.00	n/a	n/a	n/a	n/a	62000
30.00-31.00	n/a	n/a	n/a	n/a	62500
31.00-32.00	n/a	n/a	n/a	n/a	63500
32.00-33.00	n/a	n/a	n/a	n/a	64000
33.00-34.00	n/a	n/a	n/a	n/a	64500
34.00-35.00	n/a	n/a	n/a	n/a	65500
35.00-36.00	n/a	n/a	n/a	n/a	66000
36.00-37.00	n/a	n/a	n/a	n/a	66500
37.00-38.00	n/a	n/a	n/a	n/a	67500
38.00-39.00	n/a	n/a	n/a	n/a	68000
39.00-40.00	n/a	n/a	n/a	n/a	68500
40.00-41.00	n/a	n/a	n/a	n/a	69500
41.00-42.00	n/a	n/a	n/a	n/a	70000
42.00-43.00	n/a	n/a	n/a	n/a	70500
43.00-44.00	n/a	n/a	n/a	n/a	71500
44.00-45.00	n/a	n/a	n/a	n/a	72000
45.00-46.00	n/a	n/a	n/a	n/a	72500
46.00-47.00	n/a	n/a	n/a	n/a	73500
47.00-48.00	n/a	n/a	n/a	n/a	74000
48.00-49.00	n/a	n/a	n/a	n/a	74500
49.00-50.00	n/a	n/a	n/a	n/a	75500
50.00-51.00	n/a	n/a	n/a	n/a	76000
51.00-52.00	n/a	n/a	n/a	n/a	76500
52.00-53.00	n/a	n/a	n/a	n/a	77500
53.00-54.00	n/a	n/a	n/a	n/a	78000

54.00-55.00	n/a	n/a	n/a	n/a	78500
55.00-56.00	n/a	n/a	n/a	n/a	79500
56.00-57.00	n/a	n/a	n/a	n/a	80000

Table 7 is used 1 time and has 36 lines
and is named "5 axle table"

min-16.00	n/a	n/a	n/a	n/a	58000
16.00-17.00	n/a	n/a	n/a	n/a	58500
17.00-18.00	n/a	n/a	n/a	n/a	59000
18.00-19.00	n/a	n/a	n/a	n/a	60000
19.00-20.00	n/a	n/a	n/a	n/a	60500
20.00-21.00	n/a	n/a	n/a	n/a	61000
21.00-22.00	n/a	n/a	n/a	n/a	61500
22.00-23.00	n/a	n/a	n/a	n/a	62500
23.00-24.00	n/a	n/a	n/a	n/a	63000
24.00-25.00	n/a	n/a	n/a	n/a	63500
25.00-26.00	n/a	n/a	n/a	n/a	64000
26.00-27.00	n/a	n/a	n/a	n/a	65000
27.00-28.00	n/a	n/a	n/a	n/a	65500
28.00-29.00	n/a	n/a	n/a	n/a	66000
29.00-30.00	n/a	n/a	n/a	n/a	66500
30.00-31.00	n/a	n/a	n/a	n/a	67500
31.00-32.00	n/a	n/a	n/a	n/a	68000
32.00-33.00	n/a	n/a	n/a	n/a	68500
33.00-34.00	n/a	n/a	n/a	n/a	69000
34.00-35.00	n/a	n/a	n/a	n/a	70000
35.00-36.00	n/a	n/a	n/a	n/a	70500
36.00-37.00	n/a	n/a	n/a	n/a	71000
37.00-38.00	n/a	n/a	n/a	n/a	71500
38.00-39.00	n/a	n/a	n/a	n/a	72500
39.00-40.00	n/a	n/a	n/a	n/a	73000
40.00-41.00	n/a	n/a	n/a	n/a	73500
41.00-42.00	n/a	n/a	n/a	n/a	74000
42.00-43.00	n/a	n/a	n/a	n/a	75000
43.00-44.00	n/a	n/a	n/a	n/a	75500
44.00-45.00	n/a	n/a	n/a	n/a	76000
45.00-46.00	n/a	n/a	n/a	n/a	76500
46.00-47.00	n/a	n/a	n/a	n/a	77500
47.00-48.00	n/a	n/a	n/a	n/a	78000
48.00-49.00	n/a	n/a	n/a	n/a	78500
49.00-50.00	n/a	n/a	n/a	n/a	79000
50.00-51.00	n/a	n/a	n/a	n/a	80000

Table 8 is used 1 time and has 24 lines
and is named "6 axle table"

min-20.00	n/a	n/a	n/a	n/a	66000
20.00-21.00	n/a	n/a	n/a	n/a	66500
21.00-22.00	n/a	n/a	n/a	n/a	67000
22.00-23.00	n/a	n/a	n/a	n/a	68000
23.00-24.00	n/a	n/a	n/a	n/a	68500
24.00-25.00	n/a	n/a	n/a	n/a	69000
25.00-26.00	n/a	n/a	n/a	n/a	69500
26.00-27.00	n/a	n/a	n/a	n/a	70000
27.00-28.00	n/a	n/a	n/a	n/a	71000
28.00-29.00	n/a	n/a	n/a	n/a	71500
29.00-30.00	n/a	n/a	n/a	n/a	72000
30.00-31.00	n/a	n/a	n/a	n/a	72500
31.00-32.00	n/a	n/a	n/a	n/a	73000
32.00-33.00	n/a	n/a	n/a	n/a	74000
33.00-34.00	n/a	n/a	n/a	n/a	74500
34.00-35.00	n/a	n/a	n/a	n/a	75000
35.00-36.00	n/a	n/a	n/a	n/a	75500
36.00-37.00	n/a	n/a	n/a	n/a	76000
37.00-38.00	n/a	n/a	n/a	n/a	77000
38.00-39.00	n/a	n/a	n/a	n/a	77500
39.00-40.00	n/a	n/a	n/a	n/a	78000
40.00-41.00	n/a	n/a	n/a	n/a	78500
41.00-42.00	n/a	n/a	n/a	n/a	79000
42.00-43.00	n/a	n/a	n/a	n/a	80000

Table 9 is used 1 time and has 11 lines
and is named "7 axle table"

min-24.00	n/a	n/a	n/a	n/a	74000
24.00-25.00	n/a	n/a	n/a	n/a	74500
25.00-26.00	n/a	n/a	n/a	n/a	75000
26.00-27.00	n/a	n/a	n/a	n/a	75500
27.00-28.00	n/a	n/a	n/a	n/a	76500
28.00-29.00	n/a	n/a	n/a	n/a	77000
29.00-30.00	n/a	n/a	n/a	n/a	77500
30.00-31.00	n/a	n/a	n/a	n/a	78000
31.00-32.00	n/a	n/a	n/a	n/a	78500
32.00-33.00	n/a	n/a	n/a	n/a	79000
33.00-34.00	n/a	n/a	n/a	n/a	80000

==

There is 1 structure.

Weights are in pounds and lengths are in feet.

Table 1 is used 20 times
Named "THE ACTUAL DATA IS STARTED HERE"
Steering axle maximum allowed weight of 20000 lb
Single axle maximum allowed weight of 20000 lb Maximum
vehicle length is unused.
Spacing tolerance of is unused. Maximum
kingpin distance is unused. Maximum
vehicle width is unused.

Tandem axle spacing type is 2
OUTER_AXLES_SPC_TYP Tandem axle balance factor
is unused.
Tandem axle spacing table is 1 "Tandem table - Value based on 8ft Tandem definition from Randy Braden"

Tridem axle spacing type is 2 OUTER_AXLES_SPC_TYP
Tridem axle balance factor is unused.
Tridem axle spacing table is 2 "Tridem Table - Value based on 11ft Tridem Definition from Randy Braden"

Quadrem axle spacing type is 0 NO_SPC_TYP

Gross weight spacing type is 0 NO_SPC_TYP
Gross weight spacing table is 3 "GVW Table - Max GVW 80,000 lbs"

Axle grouping spacing type is 2 OUTER_AXLES_SPC_TYP Axle
grouping grouping type is 2 ADJ_AXLES_GRP_TYP
Axle grouping spacing table 3 axle is 4 "2 axle group table" Axle
grouping spacing table 4 axle is 5 "3 axle group table" Axle
grouping spacing table 5 axle is 6 "4 axle group table" Axle
grouping spacing table 6 axle is 7 "5 axle table"
Axle grouping spacing table 7 axle is 8 "6 axle table"

Appendix C: Effective Moment and Number of Cycles for the WIM Stations in Alabama for the Year 2014-2016

The effective moment (M_{eff}), kip-ft and number of cycles (N) at mid-span and cover plate end created by the traffic from the WIM stations in the state of Alabama are listed in this appendix. The M_{eff} and N are separated per direction. Lane 1 & 2 is one direction and Lane 3 & 4 is in other direction are listed for Class 4-13. Class 0 data is listed separately as lane and direction information is not available. Also, the M_{eff} and N are calculated for standard span lengths 30, 60, 90, 120 and 200 ft.

Table C.8.1: Number of cycles at mid-span for Lane 1 & 2 (Class 4-13)

Station code	Name	Location	Span, ft	Number of cycles (N), cycles			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	467,752	436,478	534,408	479,546
			60ft	332,033	310,030	375,315	339,126
			90ft	223,331	208,225	251,787	227,781
			120ft	181,515	169,576	207,773	186,288
			200ft	181,392	169,445	207,693	186,177
931	Athens	I65 Limestone Co.	30ft	2,286,106	1,962,140	1,620,244	1,956,163
			60ft	1,588,547	1,372,996	1,135,427	1,365,657
			90ft	1,057,172	927,143	782,738	922,351
			120ft	845,437	749,451	632,166	742,351
			200ft	843,014	744,517	625,253	737,595
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	898,813	885,405	846,717	876,978
			60ft	627,384	616,157	586,846	610,129
			90ft	434,334	423,975	402,340	420,216
			120ft	358,364	352,948	333,173	348,162
			200ft	358,308	352,888	333,093	348,096
934	Sumiton	US78 Walker Co.	30ft	186,779	76,982	117,924	127,228
			60ft	121,570	49,813	76,248	82,544
			90ft	94,269	38,464	58,655	63,796
			120ft	84,412	34,667	53,615	57,565
			200ft	84,169	34,649	53,571	57,463
942	Pine Level	US231 Montgomery Co.	30ft	1,039,188	730,094	753,761	841,014
			60ft	700,418	494,329	511,221	568,656
			90ft	474,271	339,196	360,320	391,262
			120ft	376,514	263,972	272,811	304,432
			200ft	376,243	263,811	272,607	304,220
960	Whatley	US84 Clark Co.	30ft	753,280	704,162	752,137	736,526
			60ft	518,186	481,924	512,209	504,106
			90ft	345,929	319,969	340,754	335,551
			120ft	305,711	282,387	302,203	296,767
			200ft	305,486	282,267	301,892	296,548
961	Mobile	I65 Mobile Co.	30ft	1,733,947	191,857	1,747,517	1,224,440
			60ft	1,178,581	131,306	1,184,616	831,501
			90ft	826,662	92,933	826,088	581,894
			120ft	629,633	70,228	634,264	444,708
			200ft	628,916	70,175	633,339	444,143
964	Ozark	US231 Dothan Co.	30ft	825,688	190,024	743,080	586,264
			60ft	561,416	129,713	504,760	398,630
			90ft	387,611	88,461	342,444	272,839
			120ft	307,010	69,722	273,831	216,854
			200ft	306,642	69,631	273,419	216,564

Table C.8.2: Effective moment at mid-span for Lane 1 & 2 (Class 4-13)

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	149	149	156	151
			60ft	355	354	372	360
			90ft	761	760	801	774
			120ft	1,266	1,263	1,328	1,286
			200ft	2,477	2,471	2,600	2,516
931	Athens	I65 Limestone Co.	30ft	171	176	184	177
			60ft	424	447	472	448
			90ft	875	907	944	909
			120ft	1,463	1,495	1,546	1,502
			200ft	2,872	2,917	3,009	2,933
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	142	139	136	139
			60ft	343	341	335	340
			90ft	741	729	719	730
			120ft	1,217	1,192	1,180	1,196
			200ft	2,363	2,309	2,291	2,321
934	Sumiton	US78 Walker Co.	30ft	162	163	174	166
			60ft	457	458	496	470
			90ft	854	865	945	888
			120ft	1,289	1,306	1,416	1,337
			200ft	2,381	2,418	2,613	2,470
942	Pine Level	US231 Montgomery Co.	30ft	130	130	126	129
			60ft	316	315	306	312
			90ft	667	662	637	655
			120ft	1,134	1,134	1,102	1,123
			200ft	2,243	2,243	2,182	2,223
960	Whatley	US84 Clark Co.	30ft	167	166	167	167
			60ft	404	401	405	403
			90ft	868	871	879	873
			120ft	1,386	1,395	1,404	1,395
			200ft	2,680	2,702	2,717	2,700
961	Mobile	I65 Mobile Co.	30ft	148	154	143	148
			60ft	351	362	341	351
			90ft	742	761	720	741
			120ft	1,278	1,313	1,235	1,275
			200ft	2,526	2,592	2,441	2,520
964	Ozark	US231 Dothan Co.	30ft	146	146	146	146
			60ft	350	348	351	350
			90ft	731	724	732	729
			120ft	1,241	1,239	1,243	1,241
			200ft	2,452	2,461	2,464	2,459

Table C.8.3: Number of cycles at mid-span for Lane 3 & 4 (Class 4-13)

Station code	Name	Location	Span, ft	Number of cycles (N), cycles 2014	2015	2016	Average
911	Alex City	US280 Cosa Co.	30ft	451,386	465,366	394,620	437,124
			60ft	318,730	322,260	269,301	303,430
			90ft	215,286	214,309	177,282	202,292
			120ft	177,176	181,108	154,030	170,771
			200ft	176,328	181,012	153,957	170,432
931	Athens	I65 Limestone Co.	30ft	2,015,074	2,087,000	1,953,681	2,018,585
			60ft	1,397,383	1,445,883	1,352,618	1,398,628
			90ft	1,035,292	1,069,849	1,006,747	1,037,296
			120ft	741,967	767,814	724,370	744,717
			200ft	740,688	766,772	722,292	743,251
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	163,037	105,119	41,943	103,366
			60ft	116,261	74,928	30,209	73,799
			90ft	80,097	50,525	19,621	50,081
			120ft	71,755	43,896	16,969	44,207
			200ft	68,735	42,825	16,964	42,841
934	Sumiton	US78 Walker Co.	30ft	197,628	177,690	187,492	187,603
			60ft	125,315	108,081	116,010	116,469
			90ft	97,186	85,592	91,945	91,574
			120ft	85,249	77,472	83,135	81,952
			200ft	85,202	77,432	82,110	81,581
942	Pine Level	US231 Montgomery Co.	30ft	1,081,652	1,125,704	1,168,382	1,125,246
			60ft	759,243	790,499	821,780	790,507
			90ft	527,995	548,212	571,376	549,194
			120ft	411,505	425,601	441,363	426,156
			200ft	411,116	425,261	441,061	425,813
960	Whatley	US84 Clark Co.	30ft	-	-	-	-
			60ft	-	-	-	-
			90ft	-	-	-	-
			120ft	-	-	-	-
			200ft	-	-	-	-
961	Mobile	I65 Mobile Co.	30ft	446,085	126,545	1,324,514	632,381
			60ft	317,625	86,946	908,147	437,573
			90ft	256,600	62,250	659,698	326,183
			120ft	202,922	45,080	469,896	239,299
			200ft	199,207	44,942	468,230	237,460
964	Ozark	US231 Dothan Co.	30ft	901,839	202,088	776,834	626,920
			60ft	623,047	139,235	535,174	432,485
			90ft	436,570	97,111	374,423	302,701
			120ft	335,181	74,642	288,163	232,662
			200ft	334,379	74,509	287,483	232,124

Table C.8.4: Effective moment at mid-span for Lane 3 & 4 (Class 4-13)

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft 2014	2015	2016	Average
911	Alex City	US280 Cosa Co.	30ft	138	134	136	136
			60ft	344	333	342	340
			90ft	711	711	736	719
			120ft	1,163	1,164	1,191	1,172
			200ft	2,258	2,270	2,316	2,281
931	Athens	I65 Limestone Co.	30ft	178	180	186	181
			60ft	427	431	447	435
			90ft	885	893	915	898
			120ft	1,558	1,574	1,603	1,578
			200ft	3,087	3,121	3,173	3,127
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	201	179	156	178
			60ft	510	451	380	447
			90ft	997	906	827	910
			120ft	1,528	1,418	1,337	1,428
			200ft	2,956	2,730	2,593	2,760
934	Sumiton	US78 Walker Co.	30ft	141	135	158	145
			60ft	387	384	445	405
			90ft	753	743	864	787
			120ft	1,177	1,141	1,321	1,213
			200ft	2,232	2,150	2,498	2,294
942	Pine Level	US231 Montgomery Co.	30ft	162	159	159	160
			60ft	398	389	392	393
			90ft	828	810	815	818
			120ft	1,385	1,360	1,370	1,372
			200ft	2,690	2,646	2,666	2,667
960	Whatley	US84 Clark Co.	30ft	-	-	-	-
			60ft	-	-	-	-
			90ft	-	-	-	-
			120ft	-	-	-	-
			200ft	-	-	-	-
961	Mobile	I65 Mobile Co.	30ft	211	109	109	143
			60ft	531	263	259	351
			90ft	1,024	546	535	702
			120ft	1,695	958	947	1,200
			200ft	3,326	1,901	1,884	2,370
964	Ozark	US231 Dothan Co.	30ft	157	156	157	157
			60ft	379	372	378	376
			90ft	779	768	780	776
			120ft	1,336	1,322	1,339	1,332
			200ft	2,644	2,624	2,653	2,641

Table C.8.5: Number of cycles at mid-span for Class 0

Station code	Name	Location	Span, ft	Number of cycles (N), cycles			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	434	190	163	262
			60ft	292	107	86	162
			90ft	235	90	75	133
			120ft	222	85	68	125
			200ft	195	52	39	95
931	Athens	I65 Limestone Co.	30ft	1,966	3,586	6,719	4,090
			60ft	1,287	2,611	4,882	2,927
			90ft	1,151	2,358	4,262	2,590
			120ft	1,052	2,271	3,693	2,339
			200ft	807	2,091	3,510	2,136
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	1,410	638	121	723
			60ft	961	432	63	485
			90ft	785	379	60	408
			120ft	673	338	46	352
			200ft	568	271	29	289
934	Sumiton	US78 Walker Co.	30ft	202	190	295	229
			60ft	104	83	203	130
			90ft	87	78	183	116
			120ft	97	98	181	125
			200ft	53	39	146	79
942	Pine Level	US231 Montgomery Co.	30ft	308	250	199	252
			60ft	151	120	98	123
			90ft	145	111	90	115
			120ft	130	105	84	106
			200ft	69	58	47	58
960	Whatley	US84 Clark Co.	30ft	275	173	173	207
			60ft	168	98	95	120
			90ft	154	86	88	109
			120ft	115	75	77	89
			200ft	80	46	41	56
961	Mobile	I65 Mobile Co.	30ft	763	50	273	362
			60ft	486	34	145	222
			90ft	357	24	137	173
			120ft	316	22	131	156
			200ft	256	17	69	114
964	Ozark	US231 Dothan Co.	30ft	230	44	73,586	24,620
			60ft	121	20	50,167	16,769
			90ft	108	18	34,910	11,679
			120ft	107	20	27,042	9,056
			200ft	55	10	26,959	9,008

Table C.8.6: Effective moment at mid-span for Class 0

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	502	203	150	285
			60ft	1,616	647	501	921
			90ft	3,121	1,197	953	1,757
			120ft	4,678	1,825	1,503	2,669
			200ft	9,160	4,602	4,232	5,998
931	Athens	I65 Limestone Co.	30ft	588	719	673	660
			60ft	1,715	2,281	2,054	2,017
			90ft	3,003	4,152	3,795	3,650
			120ft	4,473	6,079	5,822	5,458
			200ft	9,182	11,477	11,032	10,564
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	628	566	193	463
			60ft	2,005	1,818	619	1,481
			90ft	3,802	3,376	1,098	2,759
			120ft	5,872	5,124	1,902	4,300
			200ft	11,741	10,295	5,241	9,093
934	Sumiton	US78 Walker Co.	30ft	136	166	343	215
			60ft	463	547	1,068	693
			90ft	888	1,006	1,991	1,295
			120ft	1,283	1,451	2,961	1,898
			200ft	3,283	4,714	6,133	4,710
942	Pine Level	US231 Montgomery Co.	30ft	205	179	210	198
			60ft	673	584	688	649
			90ft	1,173	1,044	1,205	1,141
			120ft	1,926	1,645	1,902	1,824
			200ft	5,588	4,755	5,432	5,258
960	Whatley	US84 Clark Co.	30ft	259	232	245	245
			60ft	725	720	753	733
			90ft	1,274	1,302	1,334	1,304
			120ft	2,213	2,097	2,195	2,168
			200ft	5,451	5,498	6,179	5,709
961	Mobile	I65 Mobile Co.	30ft	224	106	167	165
			60ft	630	275	522	476
			90ft	1,285	586	915	929
			120ft	2,097	941	1,441	1,493
			200ft	4,609	2,163	4,185	3,652
964	Ozark	US231 Dothan Co.	30ft	293	160	152	202
			60ft	868	521	366	585
			90ft	1,534	978	755	1,089
			120ft	2,347	1,457	1,295	1,700
			200ft	6,498	4,356	2,573	4,476

Table C.8.7: Number of cycles at cover plate end for Lane 1 & 2 (Class 4-13)

Station code	Name	Location	Span, ft	Number of cycles (N), cycles			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	498,063	466,062	567,539	510,555
			60ft	375,824	351,282	428,317	385,141
			90ft	341,068	318,285	385,533	348,295
			120ft	328,463	305,785	371,237	335,162
			200ft	181,793	169,799	208,061	186,551
931	Athens	I65 Limestone Co.	30ft	2,411,960	2,074,272	1,717,320	2,067,851
			60ft	1,825,471	1,576,870	1,310,930	1,571,090
			90ft	1,609,157	1,392,089	1,151,846	1,384,364
			120ft	1,565,208	1,351,366	1,117,065	1,344,546
			200ft	847,826	752,881	636,467	745,725
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	974,421	958,386	908,780	947,196
			60ft	725,354	711,906	674,564	703,941
			90ft	672,263	660,306	623,283	651,951
			120ft	650,941	640,471	606,564	632,659
			200ft	358,553	353,134	333,443	348,377
934	Sumiton	US78 Walker Co.	30ft	201,114	82,519	126,292	136,642
			60ft	152,267	62,824	98,193	104,428
			90ft	128,650	52,524	81,183	87,452
			120ft	120,884	49,471	73,583	81,313
			200ft	84,622	34,721	53,683	57,675
942	Pine Level	US231 Montgomery Co.	30ft	1,089,886	764,923	791,234	882,014
			60ft	807,038	567,147	589,213	654,466
			90ft	721,439	506,416	522,842	583,566
			120ft	696,149	490,042	504,850	563,680
			200ft	377,490	264,890	273,904	305,428
960	Whatley	US84 Clark Co.	30ft	832,892	781,263	837,210	817,122
			60ft	606,754	567,514	604,305	592,858
			90ft	564,281	526,337	561,457	550,692
			120ft	552,979	517,831	552,118	540,976
			200ft	306,689	283,161	303,045	297,632
961	Mobile	I65 Mobile Co.	30ft	1,825,851	202,847	1,839,279	1,289,326
			60ft	1,335,050	147,876	1,339,453	940,793
			90ft	1,211,801	134,917	1,218,232	854,983
			120ft	1,173,173	130,861	1,181,595	828,543
			200ft	630,751	70,408	635,508	445,556
964	Ozark	US231 Dothan Co.	30ft	871,283	198,734	777,980	615,999
			60ft	649,683	149,887	588,690	462,753
			90ft	575,050	131,291	511,852	406,064
			120ft	554,345	126,820	494,026	391,730
			200ft	307,826	70,094	275,388	217,769

Table C.8.8: Effective moment at cover plate end for Lane 1 & 2 (Class 4-13)

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	96	96	100	97
			60ft	241	241	253	245
			90ft	458	457	482	466
			120ft	697	697	734	709
			200ft	1,623	1,619	1,703	1,648
931	Athens	I65 Limestone Co.	30ft	110	113	118	114
			60ft	281	293	308	294
			90ft	515	533	558	535
			120ft	787	809	843	813
			200ft	1,864	1,888	1,945	1,899
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	92	90	88	90
			60ft	233	230	226	230
			90ft	442	433	426	434
			120ft	669	654	645	656
			200ft	1,549	1,512	1,497	1,520
934	Sumiton	US78 Walker Co.	30ft	106	106	114	109
			60ft	287	290	312	296
			90ft	513	523	569	535
			120ft	752	768	845	788
			200ft	1,546	1,577	1,706	1,610
942	Pine Level	US231 Montgomery Co.	30ft	84	83	81	83
			60ft	211	211	205	209
			90ft	400	400	389	396
			120ft	616	615	599	610
			200ft	1,463	1,464	1,423	1,450
960	Whatley	US84 Clark Co.	30ft	106	106	107	106
			60ft	271	269	271	270
			90ft	511	511	514	512
			120ft	767	767	772	769
			200ft	1,761	1,774	1,782	1,772
961	Mobile	I65 Mobile Co.	30ft	94	97	91	94
			60ft	238	246	231	238
			90ft	454	468	438	454
			120ft	699	719	674	697
			200ft	1,657	1,701	1,599	1,652
964	Ozark	US231 Dothan Co.	30ft	93	93	93	93
			60ft	234	233	234	234
			90ft	445	443	445	444
			120ft	684	682	684	683
			200ft	1,605	1,610	1,610	1,609

Table C.8.9: Number of cycles at cover plate end for Lane 3 & 4 (Class 4-13)

Station code	Name	Location	Span, ft	Number of cycles (N), cycles			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	475,502	491,423	416,320	461,082
			60ft	362,417	370,784	313,151	348,784
			90ft	328,737	337,241	284,593	316,857
			120ft	315,385	324,780	274,478	304,881
			200ft	177,859	181,330	154,180	171,123
931	Athens	I65 Limestone Co.	30ft	2,120,833	2,195,102	2,053,937	2,123,291
			60ft	1,578,492	1,628,885	1,529,719	1,579,032
			90ft	1,419,448	1,466,632	1,371,008	1,419,029
			120ft	1,380,386	1,427,005	1,334,993	1,380,795
			200ft	742,536	768,605	727,468	746,203
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	175,891	113,593	45,968	111,817
			60ft	136,410	86,070	34,047	85,509
			90ft	125,473	79,303	31,717	78,831
			120ft	121,807	77,479	31,103	76,796
			200ft	73,734	44,448	16,981	45,054
934	Sumiton	US78 Walker Co.	30ft	208,900	186,877	199,321	198,366
			60ft	155,413	141,682	152,417	149,837
			90ft	131,345	116,339	125,097	124,260
			120ft	122,338	107,427	111,354	113,706
			200ft	85,352	77,524	83,668	82,181
942	Pine Level	US231 Montgomery Co.	30ft	1,159,041	1,199,338	1,244,432	1,200,937
			60ft	857,947	887,947	922,009	889,301
			90ft	769,238	801,368	828,734	799,780
			120ft	748,587	778,795	806,824	778,069
			200ft	412,444	426,548	442,531	427,174
960	Whatley	US84 Clark Co.	30ft	-	-	-	-
			60ft	-	-	-	-
			90ft	-	-	-	-
			120ft	-	-	-	-
			200ft	-	-	-	-
961	Mobile	I65 Mobile Co.	30ft	488,498	133,363	1,393,196	671,686
			60ft	374,624	102,622	1,073,332	516,859
			90ft	325,088	86,969	906,791	439,616
			120ft	309,691	84,199	879,641	424,510
			200ft	205,930	45,475	473,800	241,735
964	Ozark	US231 Dothan Co.	30ft	950,825	212,832	819,369	661,009
			60ft	723,292	161,682	622,154	502,376
			90ft	626,397	139,869	536,634	434,300
			120ft	604,542	135,367	519,768	419,892
			200ft	336,662	74,940	289,566	233,723

Table C.8.10: Effective moment at cover plate end for Lane 3 & 4 (Class 4-13)

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft 2014	2015	2016	Average
911	Alex City	US280 Cosa Co.	30ft	88	87	88	88
			60ft	226	221	226	224
			90ft	418	414	423	418
			120ft	634	632	645	637
			200ft	1,466	1,474	1,501	1,480
931	Athens	I65 Limestone Co.	30ft	114	116	119	116
			60ft	288	291	300	293
			90ft	549	555	569	558
			120ft	847	857	875	860
			200ft	2,018	2,042	2,072	2,044
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	127	114	100	114
			60ft	326	292	252	290
			90ft	580	524	471	525
			120ft	858	779	717	785
			200ft	1,908	1,766	1,680	1,785
934	Sumiton	US78 Walker Co.	30ft	91	89	100	93
			60ft	251	246	282	260
			90ft	465	457	529	484
			120ft	696	682	799	726
			200ft	1,459	1,405	1,625	1,497
942	Pine Level	US231 Montgomery Co.	30ft	104	102	102	103
			60ft	268	262	264	265
			90ft	504	492	496	498
			120ft	761	745	751	752
			200ft	1,763	1,733	1,745	1,747
960	Whatley	US84 Clark Co.	30ft	-	-	-	-
			60ft	-	-	-	-
			90ft	-	-	-	-
			120ft	-	-	-	-
			200ft	-	-	-	-
961	Mobile	I65 Mobile Co.	30ft	133	70	69	91
			60ft	348	177	174	233
			90ft	651	340	337	443
			120ft	988	523	518	676
			200ft	2,156	1,242	1,232	1,543
964	Ozark	US231 Dothan Co.	30ft	101	99	101	100
			60ft	253	249	253	252
			90ft	481	473	481	479
			120ft	738	729	739	735
			200ft	1,731	1,718	1,736	1,728

Table C.8.11: Number of cycles at cover plate end for Class 0

Station code	Name	Location	Span, ft	Number of cycles (N), cycles			Average
				2014	2015	2016	
911	Alex City	US280 Cosa Co.	30ft	497	207	172	121
			60ft	392	169	153	281
			90ft	309	117	89	510
			120ft	252	99	67	770
			200ft	213	67	56	1,480
931	Athens	I65 Limestone Co.	30ft	2,133	3,920	7,323	169
			60ft	1,744	3,175	5,926	392
			90ft	1,336	2,736	5,099	716
			120ft	1,226	2,558	4,794	1,090
			200ft	925	2,213	3,681	2,104
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	1,671	711	128	138
			60ft	1,295	563	101	320
			90ft	1,019	457	68	580
			120ft	906	400	56	872
			200ft	701	322	34	1,665
934	Sumiton	US78 Walker Co.	30ft	214	191	333	127
			60ft	195	177	280	321
			90ft	131	119	225	563
			120ft	95	81	190	820
			200ft	75	63	163	1,521
942	Pine Level	US231 Montgomery Co.	30ft	321	266	217	146
			60ft	278	237	177	340
			90ft	166	148	111	616
			120ft	141	115	93	924
			200ft	101	83	68	1,757
960	Whatley	US84 Clark Co.	30ft	308	180	183	224
			60ft	269	155	148	191
			90ft	178	100	89	122
			120ft	163	92	81	112
			200ft	89	63	56	69
961	Mobile	I65 Mobile Co.	30ft	815	52	283	127
			60ft	701	47	245	298
			90ft	507	37	162	536
			120ft	434	31	137	809
			200ft	304	24	107	1,557
964	Ozark	US231 Dothan Co.	30ft	236	46	77,246	144
			60ft	203	41	58,567	328
			90ft	123	24	50,529	595
			120ft	112	22	48,850	905
			200ft	83	16	27,153	1,750

Table C.8.12: Effective moment at cover plate end for Class 0

Station code	Name	Location	Span, ft	Effective moment (M_{eff}), kip-ft 2014	2015	2016	Average
911	Alex City	US280 Cosa Co.	30ft	317	131	95	137
			60ft	954	372	287	315
			90ft	1,838	708	603	580
			120ft	2,886	1,125	999	872
			200ft	5,734	2,787	2,468	1,666
931	Athens	I65 Limestone Co.	30ft	370	455	426	156
			60ft	1,013	1,395	1,268	370
			90ft	1,873	2,563	2,337	653
			120ft	2,784	3,777	3,467	989
			200ft	5,721	7,252	7,007	1,908
933	Muscle Shoals	AL157 US72 Colbert Co.	30ft	387	357	124	129
			60ft	1,189	1,095	367	301
			90ft	2,280	2,067	712	554
			120ft	3,468	3,144	1,214	827
			200ft	7,090	6,285	3,313	1,568
934	Sumiton	US78 Walker Co.	30ft	84	106	215	145
			60ft	246	296	628	360
			90ft	503	589	1,212	615
			120ft	841	1,006	1,899	886
			200ft	1,920	2,635	3,858	1,621
942	Pine Level	US231 Montgomery Co.	30ft	133	113	133	118
			60ft	372	317	384	270
			90ft	761	636	756	493
			120ft	1,285	1,089	1,246	751
			200ft	3,301	2,835	3,214	1,456
960	Whatley	US84 Clark Co.	30ft	162	149	157	153
			60ft	406	411	443	349
			90ft	839	832	917	644
			120ft	1,378	1,333	1,493	961
			200ft	3,557	3,346	3,772	1,815
961	Mobile	I65 Mobile Co.	30ft	142	69	109	134
			60ft	377	169	300	304
			90ft	776	351	588	561
			120ft	1,260	571	986	855
			200ft	2,849	1,279	2,444	1,650
964	Ozark	US231 Dothan Co.	30ft	187	102	97	132
			60ft	497	276	244	302
			90ft	996	588	464	550
			120ft	1,570	955	714	836
			200ft	3,791	2,490	1,683	1,618

Appendix D: Effective Moment and Number of Cycles of a Rating Truck for Different Span Lengths

The effective moment (M_{eff}), kip-ft and number of cycles (N) at mid-span (0.5 L), upstream cover plate end (0.2 L) and downstream cover plate end (0.8 L) created by the AASHTO fatigue design truck are listed in this appendix. Span lengths from 20 to 300 ft with varying increments are considered.

Span length, L [ft]	Location along girder								
	x/L = 0.5			x/L = 0.2			x/L = 0.8		
	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]
20	3	145.14	9.17E+06	3	90.35	2.21E+06	2	103.19	2.20E+06
22	2	177.40	1.12E+07	3	104.86	3.46E+06	2	103.19	2.20E+06
24	2	177.40	1.12E+07	3	119.47	5.12E+06	2	103.19	2.20E+06
26	2	179.44	1.16E+07	3	134.04	7.23E+06	2	129.02	4.29E+06
28	2	185.81	1.28E+07	3	143.10	8.79E+06	2	132.31	4.63E+06
30	2	218.01	2.07E+07	2	152.87	7.15E+06	2	132.31	4.63E+06
32	2	247.58	3.04E+07	2	152.70	7.12E+06	2	140.57	5.55E+06
34	2	240.60	2.79E+07	2	151.13	6.90E+06	2	150.84	6.86E+06
36	2	237.15	2.67E+07	2	151.80	7.00E+06	2	150.84	6.86E+06
38	2	239.63	2.75E+07	2	162.18	8.53E+06	2	157.88	7.87E+06
40	2	266.57	3.79E+07	2	173.97	1.05E+07	2	176.18	1.09E+07
42	2	297.09	5.24E+07	2	187.00	1.31E+07	2	170.36	9.89E+06
44	2	300.28	5.42E+07	2	201.19	1.63E+07	2	199.32	1.58E+07
46	2	304.94	5.67E+07	2	215.10	1.99E+07	2	222.48	2.20E+07
48	2	310.86	6.01E+07	2	233.18	2.54E+07	2	218.22	2.08E+07
50	2	339.59	7.83E+07	2	252.47	3.22E+07	2	243.18	2.88E+07
52	2	371.52	1.03E+08	2	271.92	4.02E+07	2	268.76	3.88E+07
54	2	377.64	1.08E+08	2	291.65	4.96E+07	2	268.31	3.86E+07
56	1	384.00	5.66E+07	2	307.24	5.80E+07	2	288.53	4.80E+07
58	1	400.00	6.40E+07	2	318.78	6.48E+07	2	318.89	6.49E+07
60	1	432.00	8.06E+07	2	330.68	7.23E+07	2	318.89	6.49E+07
62	1	464.00	9.99E+07	2	343.02	8.07E+07	2	333.34	7.41E+07
64	1	480.00	1.11E+08	2	355.74	9.00E+07	2	369.71	1.01E+08
66	1	512.00	1.34E+08	2	368.46	1.00E+08	2	374.88	1.05E+08
68	1	544.00	1.61E+08	2	390.05	1.19E+08	2	384.30	1.14E+08
70	1	576.00	1.91E+08	2	413.04	1.41E+08	2	420.66	1.49E+08
75	1	640.00	2.62E+08	2	470.45	2.08E+08	2	445.63	1.77E+08
80	1	720.00	3.73E+08	2	527.92	2.94E+08	2	496.78	2.45E+08
85	1	784.00	4.82E+08	2	559.53	3.50E+08	2	547.95	3.29E+08
90	1	864.00	6.45E+08	2	591.48	4.14E+08	2	599.12	4.30E+08
95	1	928.00	7.99E+08	2	642.67	5.31E+08	2	650.31	5.50E+08
100	1	1008.00	1.02E+09	2	700.26	6.87E+08	2	675.81	6.17E+08

Span length, L [ft]	Location along girder								
	x/L = 0.5			x/L = 0.2			x/L = 0.8		
	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]	N	M_{eff} [kip-ft]	$D=NM_{eff}^3$ [N(kip-ft)^3]
110	1	1152.00	1.53E+09	2	789.80	9.85E+08	2	778.21	9.43E+08
120	1	1296.00	2.18E+09	2	873.00	1.33E+09	2	880.61	1.37E+09
130	1	1440.00	2.99E+09	2	988.20	1.93E+09	1	957.40	8.78E+08
140	1	1584.00	3.97E+09	1	1052.20	1.16E+09	1	1059.80	1.19E+09
150	1	1728.00	5.16E+09	1	1161.00	1.56E+09	1	1136.60	1.47E+09
160	1	1872.00	6.56E+09	1	1250.60	1.96E+09	1	1239.00	1.90E+09
170	1	2016.00	8.19E+09	1	1333.80	2.37E+09	1	1341.40	2.41E+09
180	1	2160.00	1.01E+10	1	1449.00	3.04E+09	1	1418.20	2.85E+09
190	1	2304.00	1.22E+10	1	1513.00	3.46E+09	1	1520.60	3.52E+09
200	1	2448.00	1.47E+10	1	1621.80	4.27E+09	1	1597.40	4.08E+09
210	1	2592.00	1.74E+10	1	1711.40	5.01E+09	1	1699.80	4.91E+09
220	1	2736.00	2.05E+10	1	1794.60	5.78E+09	1	1802.20	5.85E+09
230	1	2880.00	2.39E+10	1	1909.80	6.97E+09	1	1879.00	6.63E+09
240	1	3024.00	2.77E+10	1	1973.80	7.69E+09	1	1981.40	7.78E+09
250	1	3168.00	3.18E+10	1	2082.60	9.03E+09	1	2058.20	8.72E+09
260	1	3312.00	3.63E+10	1	2172.20	1.02E+10	1	2160.60	1.01E+10
270	1	3456.00	4.13E+10	1	2255.40	1.15E+10	1	2263.00	1.16E+10
280	1	3600.00	4.67E+10	1	2370.60	1.33E+10	1	2339.80	1.28E+10
290	1	3744.00	5.25E+10	1	2434.60	1.44E+10	1	2442.20	1.46E+10
300	1	3888.00	5.88E+10	1	2543.40	1.65E+10	1	2519.00	1.60E+10

Appendix E: AL_WIM_QC v1.0 user manual

Contents

Introduction

Long-term WIM recording is often associated with errors that may occur due to WIM system malfunction, out-of-calibration or irregular vehicle position on the sensor. If an error in recorded WIM data is not recognized and eliminated at the earlier stage, the quality of entire accumulated data is questionable. Therefore, it is essential to use a Quality Control (QC) procedure. The procedure is developed by AU to check the quality of the traffic data and detect the root cause of questionable records. Inconsistency in recording due to communication failure, operational problems with the sensor and drift in calibration can be interpreted from this proposed procedure. The developed algorithm consists of three sets of QC checks: Completeness check, Logical checks and Statistical checks (Figure E.8.1).

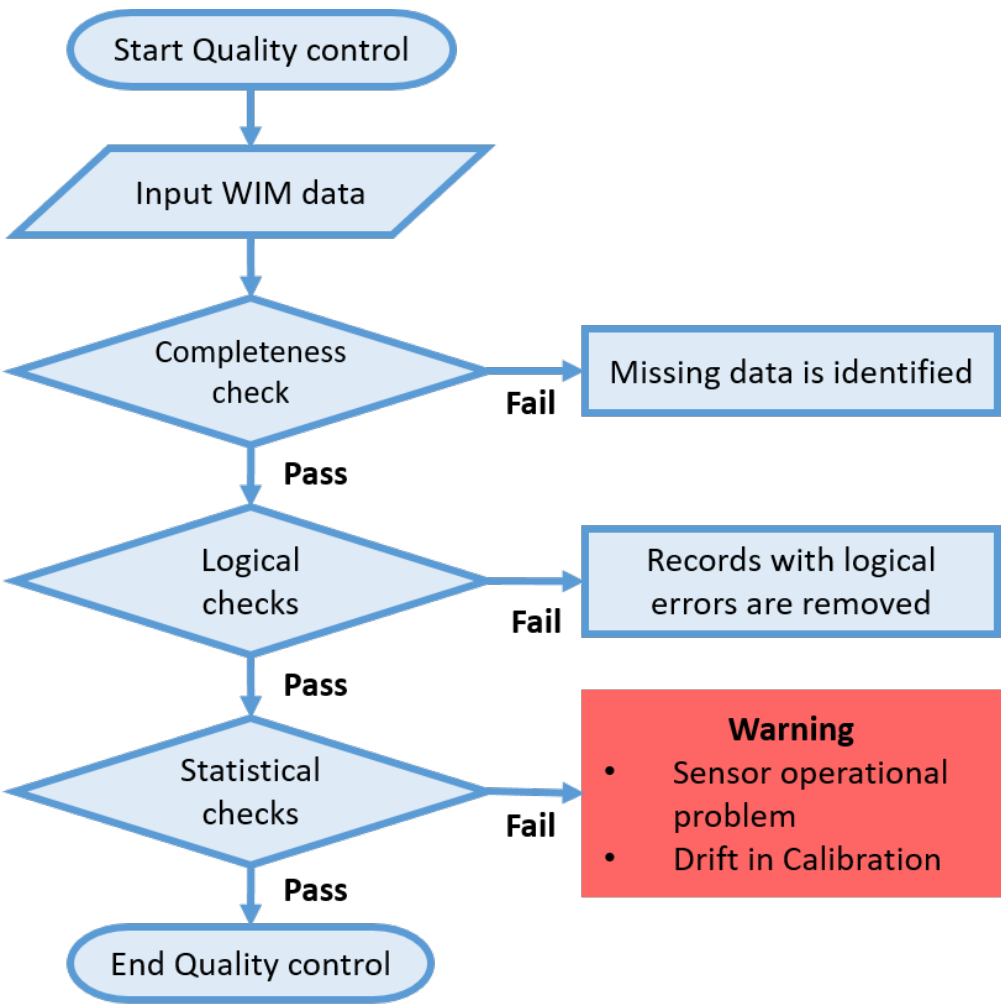


Figure E.8.1: Quality Control Procedure Flowchart

<u>Completeness check:</u> This missing data is identified and possible cause for missing data can be investigated.

<u>Logical check:</u> This set of filtering criteria, so-called “logical,” was developed based on the common practice reported in the literature and modified according to the database of permits issued by ALDOT. In some cases, the current research has identified combinations of QC criteria that can detect certain vehicle configurations that are always recorded incorrectly.

Logical check requirements can be applied instantly once the single-vehicle record is delivered to the database or after a number of vehicle records are accumulated over a period of time. Random and systematic errors are distinguished based on the frequency of violation of a single criterion or a combination of criteria. The systematic errors are usually associated with the malfunctioning of the WIM system, misrecording, non-typical vehicle configuration, or vehicle position with regard to the sensor, and other causes.

<u>Statistical checks:</u> As stated in Traffic Monitoring Guide (2001) the best way to have a reliable record from the WIM sensors is to calibrate the sensors and then compare the output from the sensors and expected weight and volume statistics of the recorded traffic data.

Statistical analysis helps to investigate the possible changes in the traffic mix, GVW and vehicle configurations, and helps to find a possible reason causing an error. Most of the checks are on vehicle class 9 as it is the dominating vehicle class for most of the WIM sites. Until today the statistics developed on vehicle class 9 are being used by many National and State agencies as a way to maintain “health” of the WIM systems.

Statistical checks include a vehicle class distribution check and checks on vehicle class 9 such as GVW check, front axle weight check, axle spacing check and tandem axle load spectra checks.

Chapter 1. Preparing Input Data

The data recorded by WIM sensors are stored in the data storage medium maintained by ALDOT in an encrypted format. The iAnalyze software provided by WIM system vendor International Road Dynamics Inc. (IRD) is used to decrypt WIM data to the desired format. For using it in this application, the Class 0 data is required to be decrypted in IRD ASCII Raw data format (as referred in section D.13.1 of iAnalyze Software operator's manual) and Class 4-13 in TMG 2001 Truck Weight format (as referred in section D.9.5 of iAnalyze Software operator's manual). For every WIM site for the selected month and year, there is always two input files, one containing Class 0 data and another containing Class 4-13.

Once the data is decrypted, it has to be renamed in the following format to use in the application and can be stored in the user desired folder.

Class 0 data:

Syntax: **<Year>_<Month>_<WIMID>.csv**

Where,

<Year> is the year of the data

<Month> is the month of the data in number

<WIMID> is WIM station ID used by ALDOT

.csv is an extension of the file type which is saved by default

For example, the WIM data for WIM location 911 for the Year 2018 and Month of January the filename will be **2018_1_911.csv**

Class 4-13 data:

Syntax: **<Year>_<Month>_<WIMID>.WGT**

Where,

<Year> is the year of the data.

<Month> is the month of the data in number.

<WIMID> is WIM station ID used by ALDOT.

.WGT is the extension of the file type which is saved by default

For example, the WIM data for WIM location 911 for the Year 2018 and Month of January the filename will be **2018_1_911.WGT**

Note: It is important that filename of Class 0 data is in .csv and Class 4-13 is in .WGT

Chapter 2. Installing the application

The users are provided with the installation file named "MyAppInstaller_web" as shown in Figure E.8.2. Once the user double-clicks the installation file, the installation processes start as shown in Figure E.8.3. The installer will request the path to the installation folder where the application package is stored as shown in Figure E.8.4. A desktop shortcut can also be created for easier assess. After that *Next* button is clicked.

A Matlab compiler is required to run the application so the dialog box as shown in Figure E.8.5 is requested and the user is required to click *Install*. After the installation is complete the dialog box as shown in Figure E.8.6 is seen indicating the successful completion of installation. Click *Finish* to complete the setup.

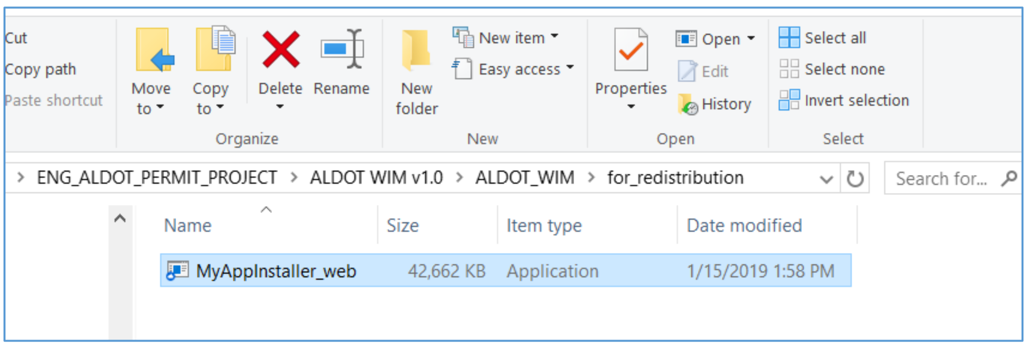


Figure E.8.2: Redistribution file provided to the user

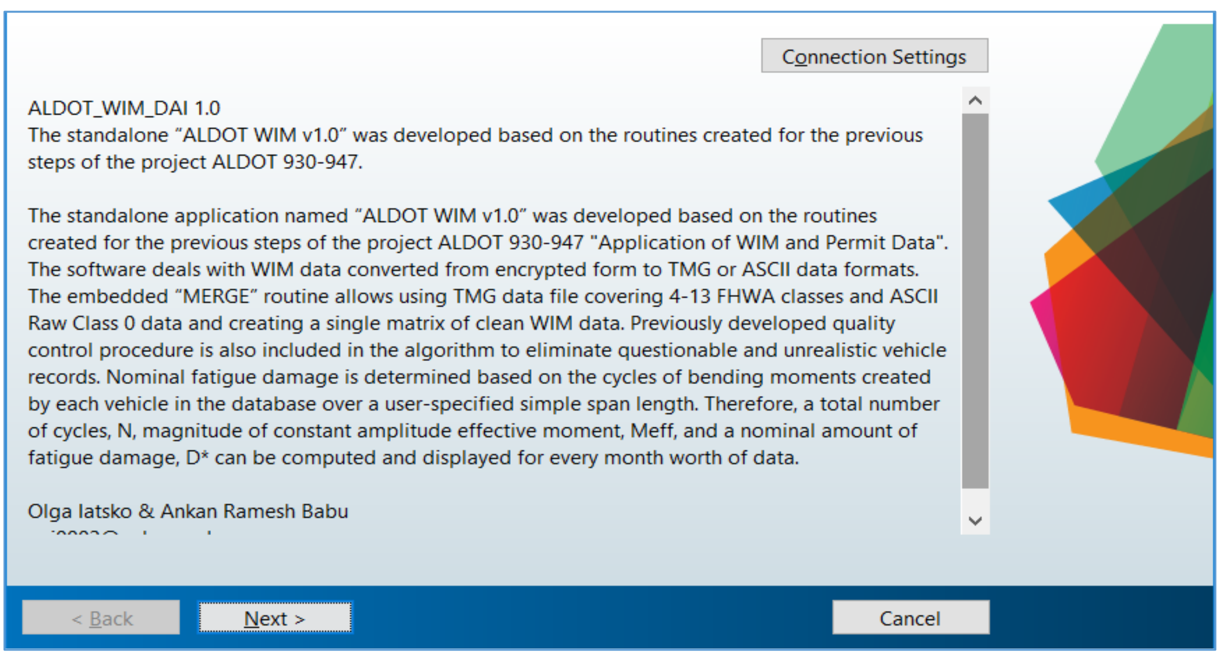


Figure E.8.3: Application installation startup screen

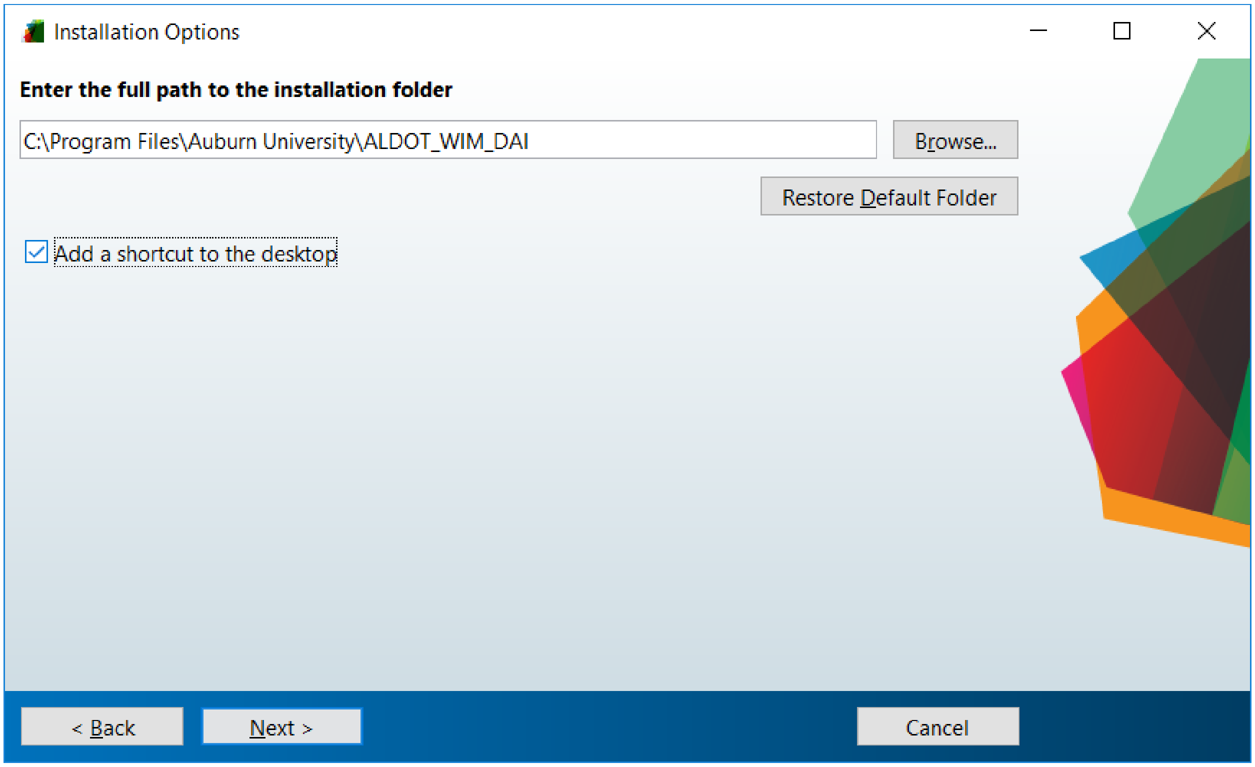


Figure E.8.4: Installation folder selection

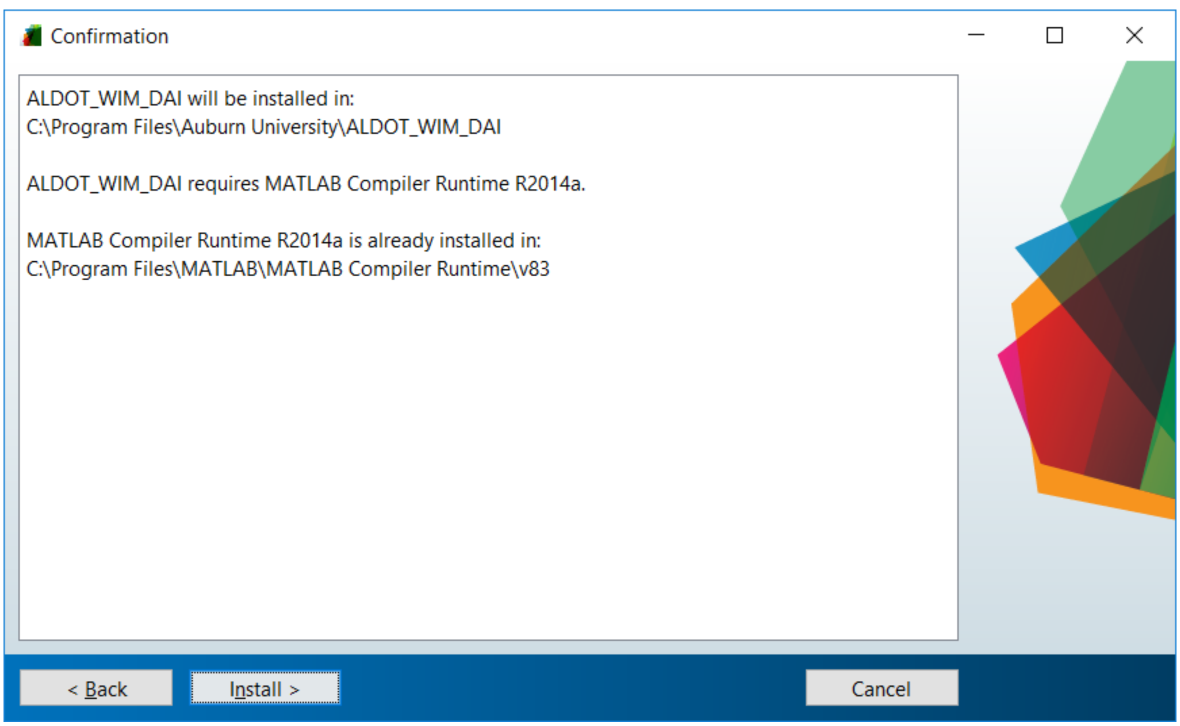


Figure E.8.5: Installation of MATLAB Compiler Runtime

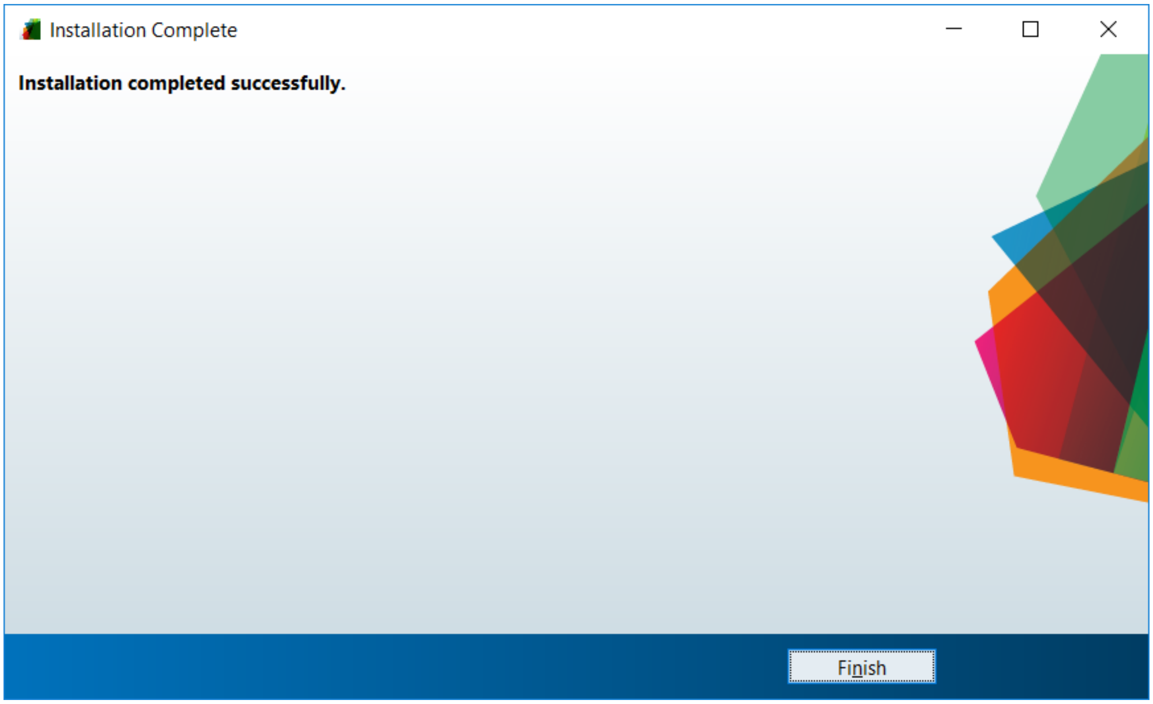


Figure E.8.6: Installation completed dialog box

Chapter 3. Step-by-step instructions to use application

After installing the application, double-clicking on the desktop shortcut icon opens the application. Alternatively, the application can also be opened by searching in the start menu. The startup screen of the application looks like it is shown in Figure E.8.7.

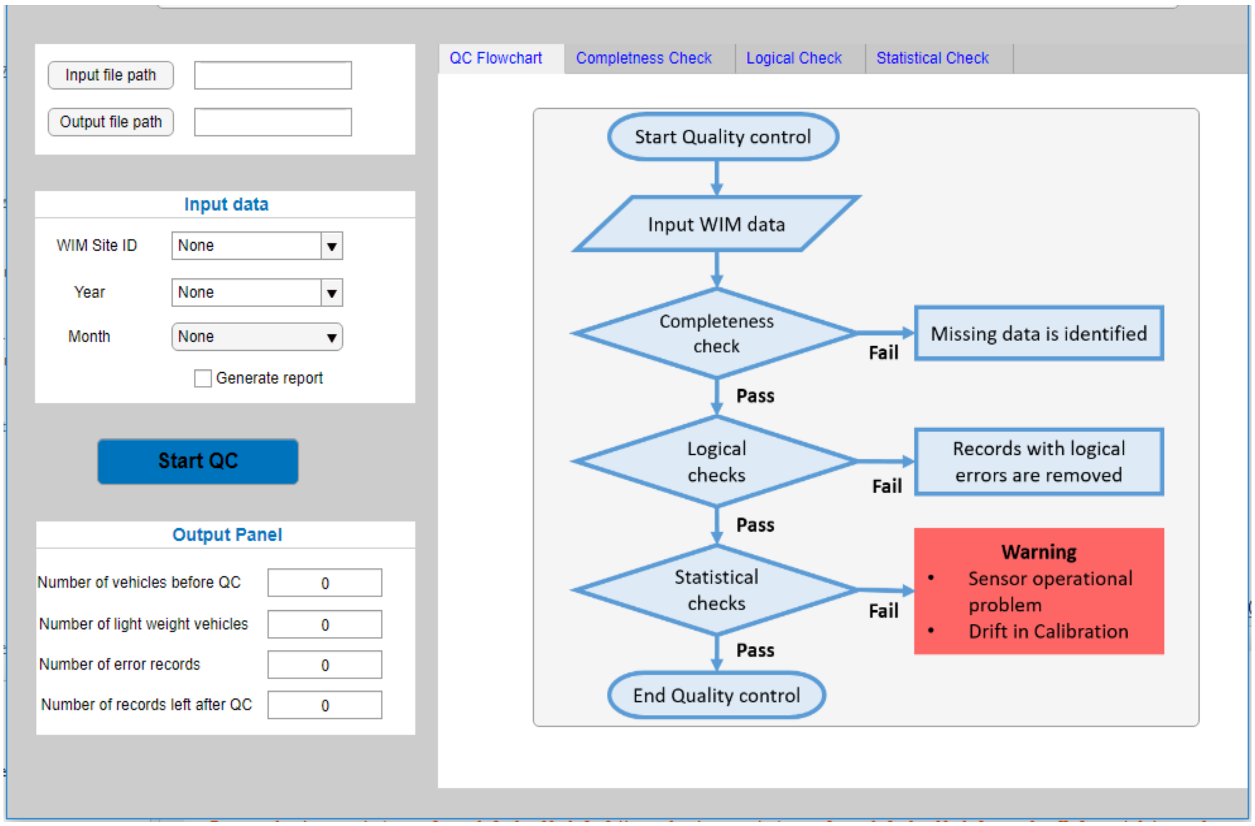


Figure E.8.7: AL_WIM_QC application startup screen

<u>Step 1:</u> Selecting the input and output file path.

The input and out file path have to be selected by clicking on the button as it is shown in Figure E.8.8. The *input file path* is the folder where the *renamed Class 0 data in a .csv* format *and Class 4-13 data in .WGT format* is stored. The output folder can be created by clicking on the "output file path" button and once the window pop-ups a new folder can be created by right-clicking the mouse and clicking on *New>>Folder* option. It is recommended to create output folder name in: **<Year>_<Month>_<WIMID>** format as shown earlier. The results of the QC of each WIM Site are stored in the form of images in the output folder. Also, the input data compiled of Class 0 and 4-13 are stored.

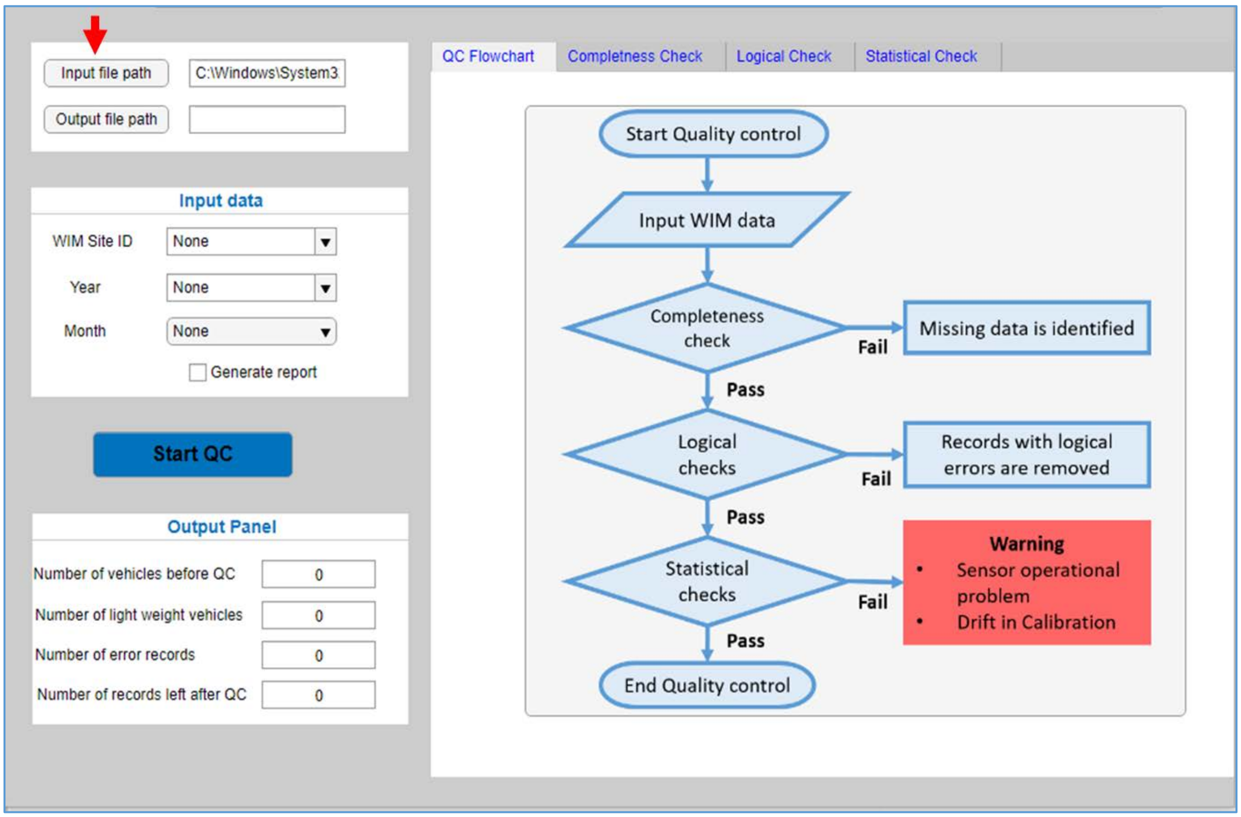


Figure E.8.8: Selecting input and output file path

Step 2: Inputting the data in *Input Data* panel

The Input Data panel is self-explanatory, the preferred *WIM Site ID* is selected from drop-down menu or by inputting the WIM Station ID. Also, preferred *Year* and *Month* is selected. *Generate report* is selected if needed. An example of a screen after inputting the data is shown in Figure E.8.9.

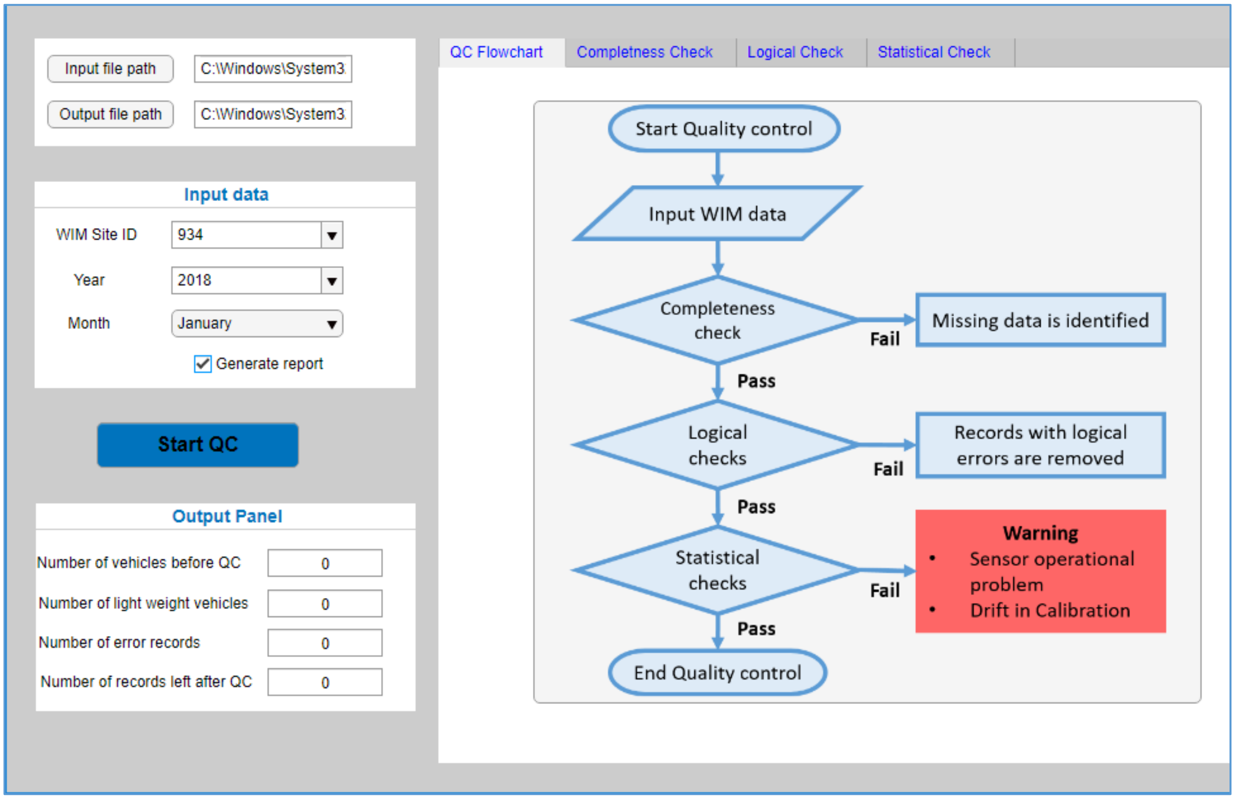


Figure E.8.9: Inputting the data in *Input Data* panel

Step 3: Starting the Quality Control (QC)

The QC is started once the *Start QC* button is pushed. The wait bar dialog box as shown in Figure E.8.10 is pop-upped once the *Start QC* button is pushed indicating the progress of the QC. Once the process is complete, the wait bar closed automatically indicating the QC is finished. The interpretation of results is shown in the next chapter.

In case the input data filename is in the wrong format or input data is not in the input file path for the selected WIM Site ID, Year or Month then the error window pop-ups as shown in Figure E.8.11. For every WIM site for the selected month and year, there is always two input files, one containing Class 0 data and another containing Class 4-13. In case there is no data in Class 0, the iAnalyze creates an empty file.

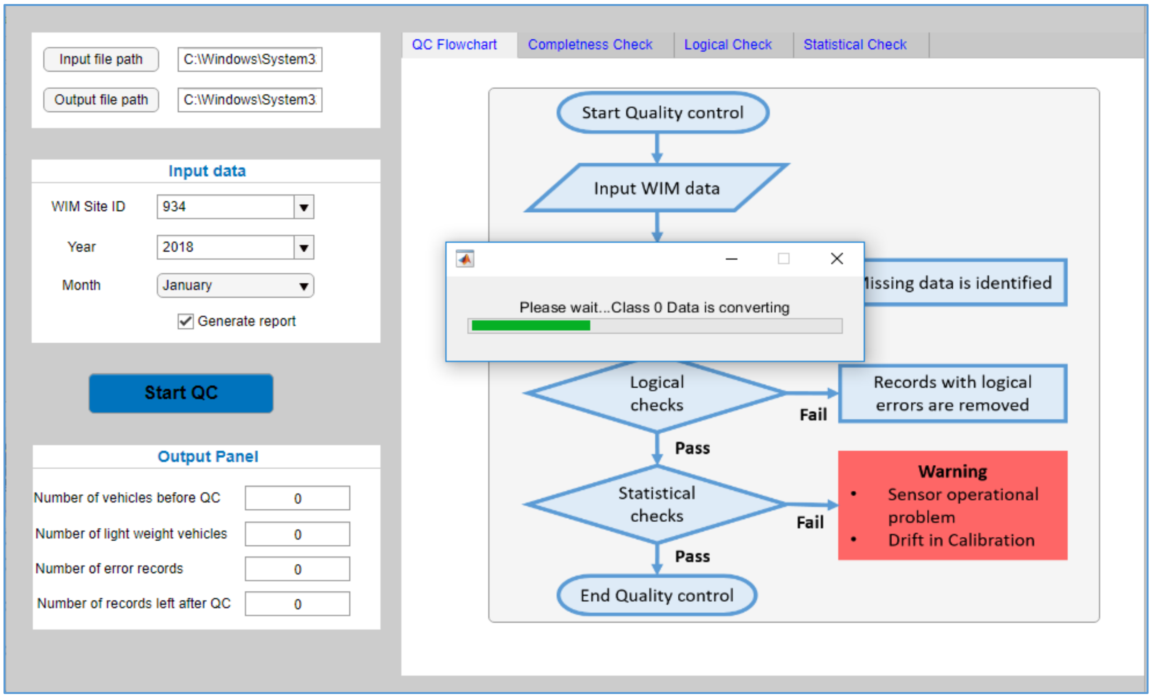


Figure E.8.10: Wait bar dialog box indicating the progress of QC

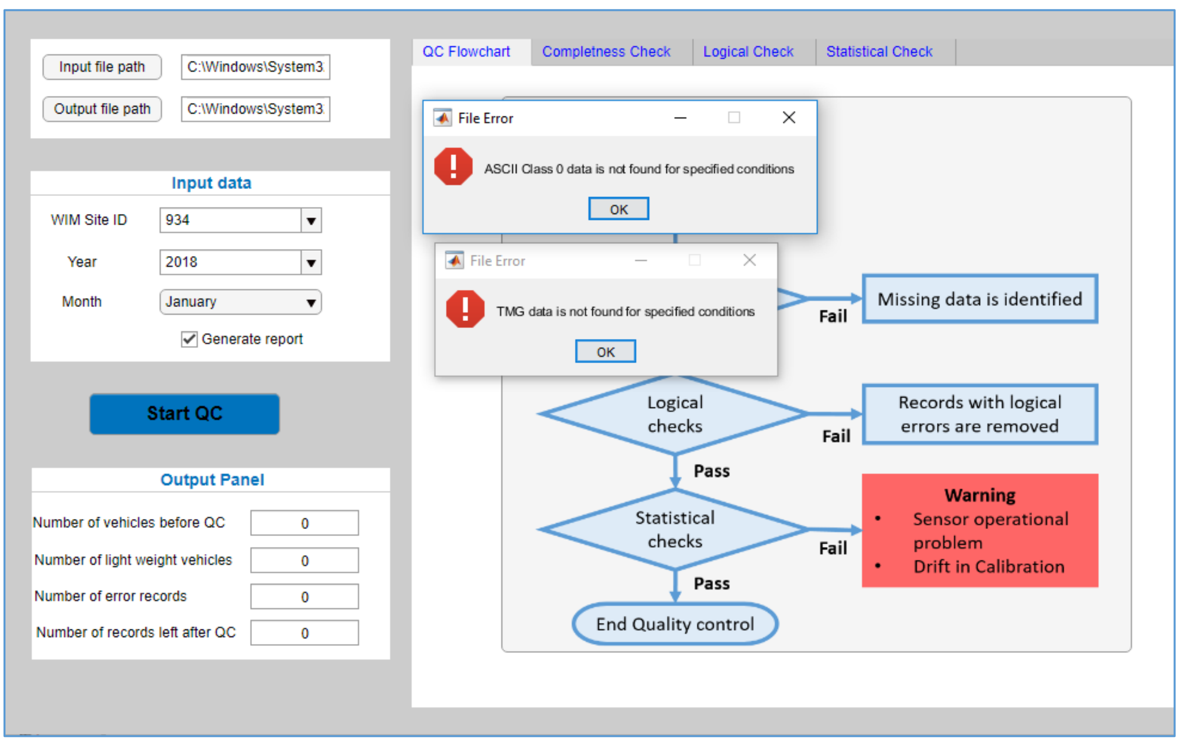


Figure E.8.11: An error dialog box indicating the problem with input data

Chapter 4. Interpretation of the results

After the QC is finished, the wait bar dialog box closes, and the results are displayed in the *output panel* as shown in Figure E.8.12. The completeness, logical and statistical checks are discussed in the following sections.

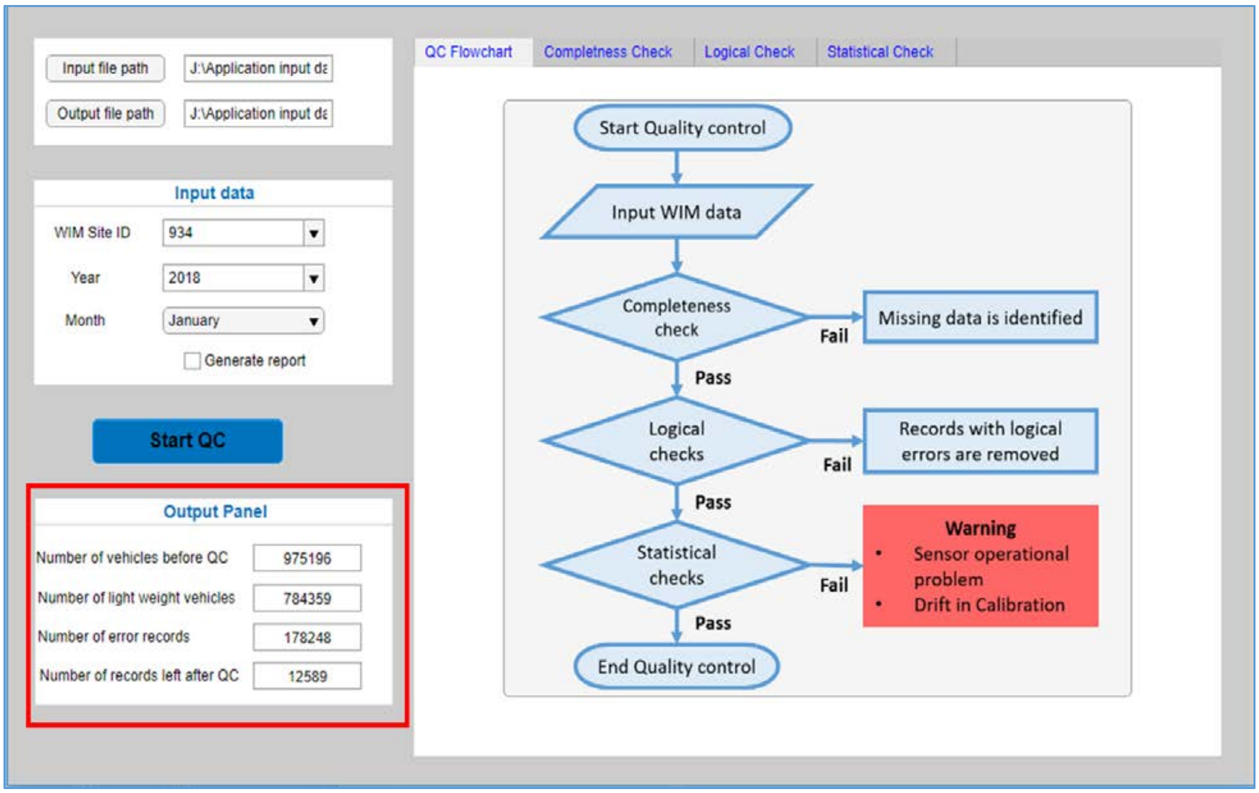
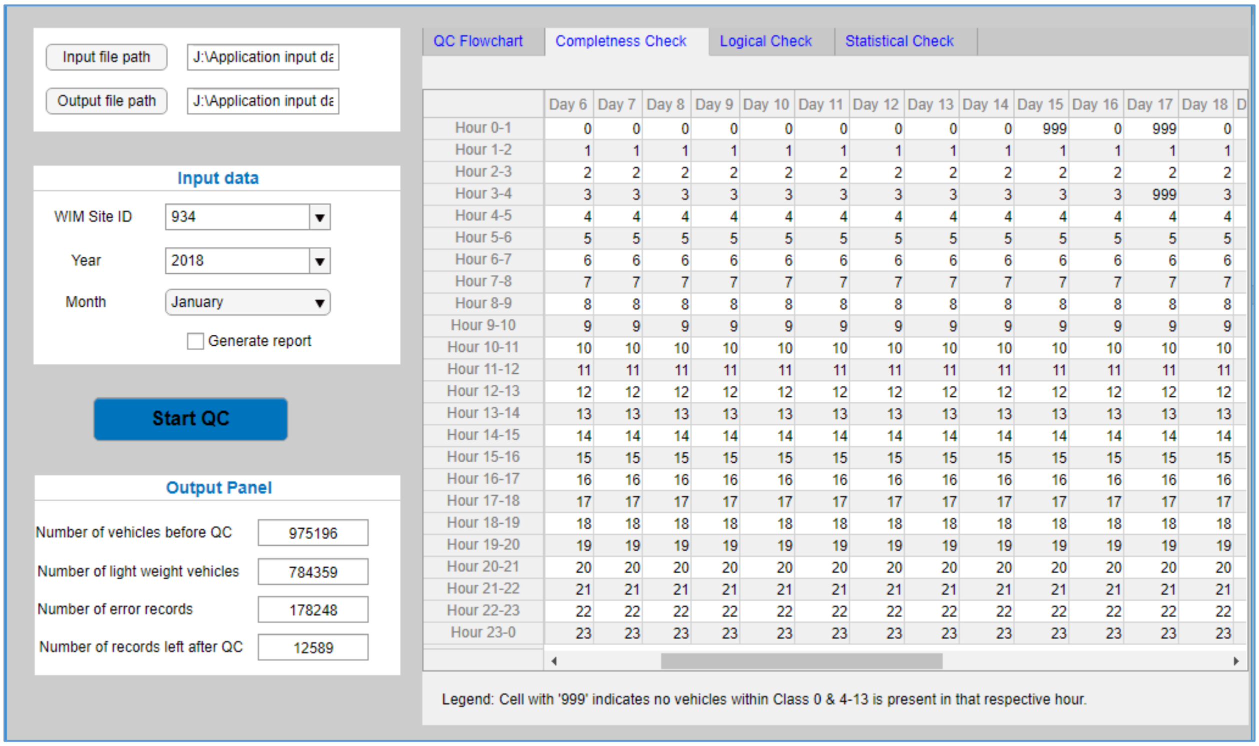


Figure E.8.12: QC results shown in output panel after completion of the QC procedure

1. Completeness Check

Objective: To check whether the data is present for each hour every day of the month. The inconsistency can be caused by a system malfunction or lack of maintenance.

When the data is present, the respective hour value is listed below. If no data is present in that particular hour then '999' is listed. Day 1 is the first day of the month, and hour zero is from midnight until 1 a.m. An example for WIM site 934 for the year 2018 and the month of January is shown in Figure E.8.13, where for Day 17 from Hour 3-4 the '999' indicates that no data was present.



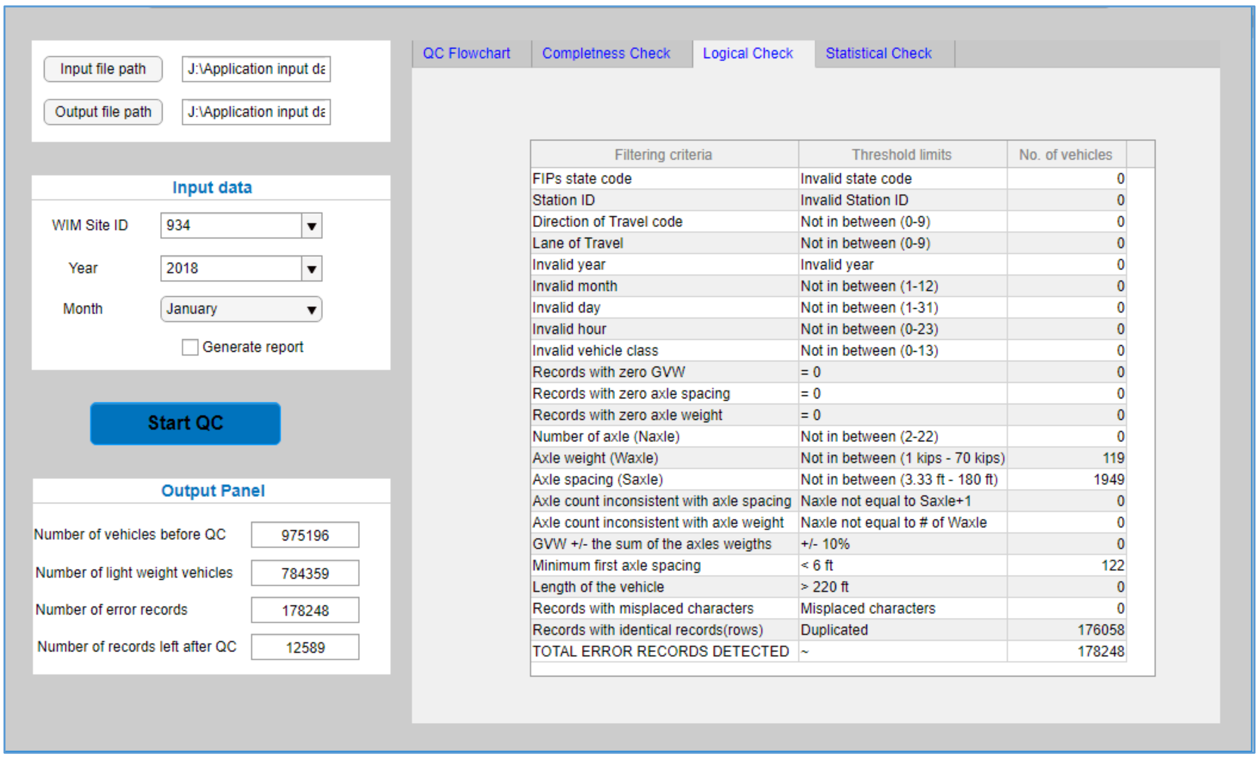
	Day 6	Day 7	Day 8	Day 9	Day 10	Day 11	Day 12	Day 13	Day 14	Day 15	Day 16	Day 17	Day 18	D
Hour 0-1	0	0	0	0	0	0	0	0	0	999	0	999	0	
Hour 1-2	1	1	1	1	1	1	1	1	1	1	1	1	1	
Hour 2-3	2	2	2	2	2	2	2	2	2	2	2	2	2	
Hour 3-4	3	3	3	3	3	3	3	3	3	3	3	999	3	
Hour 4-5	4	4	4	4	4	4	4	4	4	4	4	4	4	
Hour 5-6	5	5	5	5	5	5	5	5	5	5	5	5	5	
Hour 6-7	6	6	6	6	6	6	6	6	6	6	6	6	6	
Hour 7-8	7	7	7	7	7	7	7	7	7	7	7	7	7	
Hour 8-9	8	8	8	8	8	8	8	8	8	8	8	8	8	
Hour 9-10	9	9	9	9	9	9	9	9	9	9	9	9	9	
Hour 10-11	10	10	10	10	10	10	10	10	10	10	10	10	10	
Hour 11-12	11	11	11	11	11	11	11	11	11	11	11	11	11	
Hour 12-13	12	12	12	12	12	12	12	12	12	12	12	12	12	
Hour 13-14	13	13	13	13	13	13	13	13	13	13	13	13	13	
Hour 14-15	14	14	14	14	14	14	14	14	14	14	14	14	14	
Hour 15-16	15	15	15	15	15	15	15	15	15	15	15	15	15	
Hour 16-17	16	16	16	16	16	16	16	16	16	16	16	16	16	
Hour 17-18	17	17	17	17	17	17	17	17	17	17	17	17	17	
Hour 18-19	18	18	18	18	18	18	18	18	18	18	18	18	18	
Hour 19-20	19	19	19	19	19	19	19	19	19	19	19	19	19	
Hour 20-21	20	20	20	20	20	20	20	20	20	20	20	20	20	
Hour 21-22	21	21	21	21	21	21	21	21	21	21	21	21	21	
Hour 22-23	22	22	22	22	22	22	22	22	22	22	22	22	22	
Hour 23-0	23	23	23	23	23	23	23	23	23	23	23	23	23	

Figure E.8.13: Completeness check result

2. Logical Check

Objective: To identify and eliminate random and systematic errors.

After the lightweight vehicles (Gross vehicle weight < 20 kips) are eliminated, the remaining data is used for the logical check. Some quality control criteria have an acceptance range defined based on the physical limits and site-specific vehicle configurations. The *Filtering criteria* and *Threshold limits* for one of the location is shown in Figure E.8.14. Total error records detected is shown in the last row.



Filtering criteria	Threshold limits	No. of vehicles
FIPs state code	Invalid state code	0
Station ID	Invalid Station ID	0
Direction of Travel code	Not in between (0-9)	0
Lane of Travel	Not in between (0-9)	0
Invalid year	Invalid year	0
Invalid month	Not in between (1-12)	0
Invalid day	Not in between (1-31)	0
Invalid hour	Not in between (0-23)	0
Invalid vehicle class	Not in between (0-13)	0
Records with zero GVW	= 0	0
Records with zero axle spacing	= 0	0
Records with zero axle weight	= 0	0
Number of axle (Naxle)	Not in between (2-22)	0
Axle weight (Waxle)	Not in between (1 kips - 70 kips)	119
Axle spacing (Saxle)	Not in between (3.33 ft - 180 ft)	1949
Axle count inconsistent with axle spacing	Naxle not equal to Saxle+1	0
Axle count inconsistent with axle weight	Naxle not equal to # of Waxle	0
GVW +/- the sum of the axles weigths	+/- 10%	0
Minimum first axle spacing	< 6 ft	122
Length of the vehicle	> 220 ft	0
Records with misplaced characters	Misplaced characters	0
Records with identical records(rows)	Duplicated	176058
TOTAL ERROR RECORDS DETECTED	~	178248

Figure E.8.14: Logical check results

3. Statistical Check

Objective: To check the errors in the recorded data that are usually caused due to communication failures, operational problems with sensor and drift in calibration of the systems. Most of the checks are on Vehicle class 9 as it is the dominating vehicle class.

The statistical check flowchart is shown in Figure E.8.15 . Step by step interpretation of the results are shown in the following section.

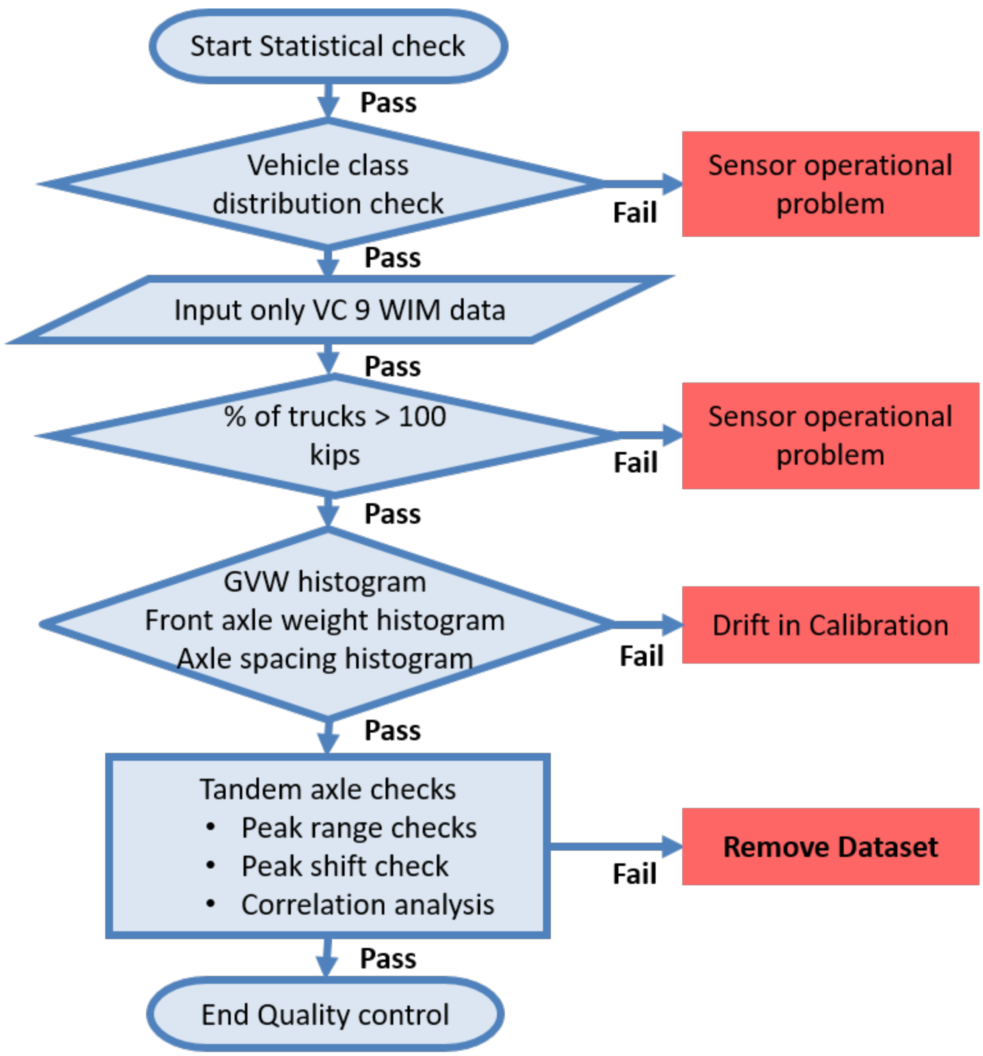


Figure E.8.15: Statistical check flowchart

3.1. Vehicle class distribution check

Objective: The percentage of vehicles in each class is calculated to check the health of the WIM sensors. Data can be compared for different years and months in each location. If an abrupt change is noticed when compared to historical data or consecutive months, it might indicate a problem with the sensor or vehicle classification algorithm.

An example of one of the WIM sites is shown in Figure E.8.16. The results are in percentage (%).

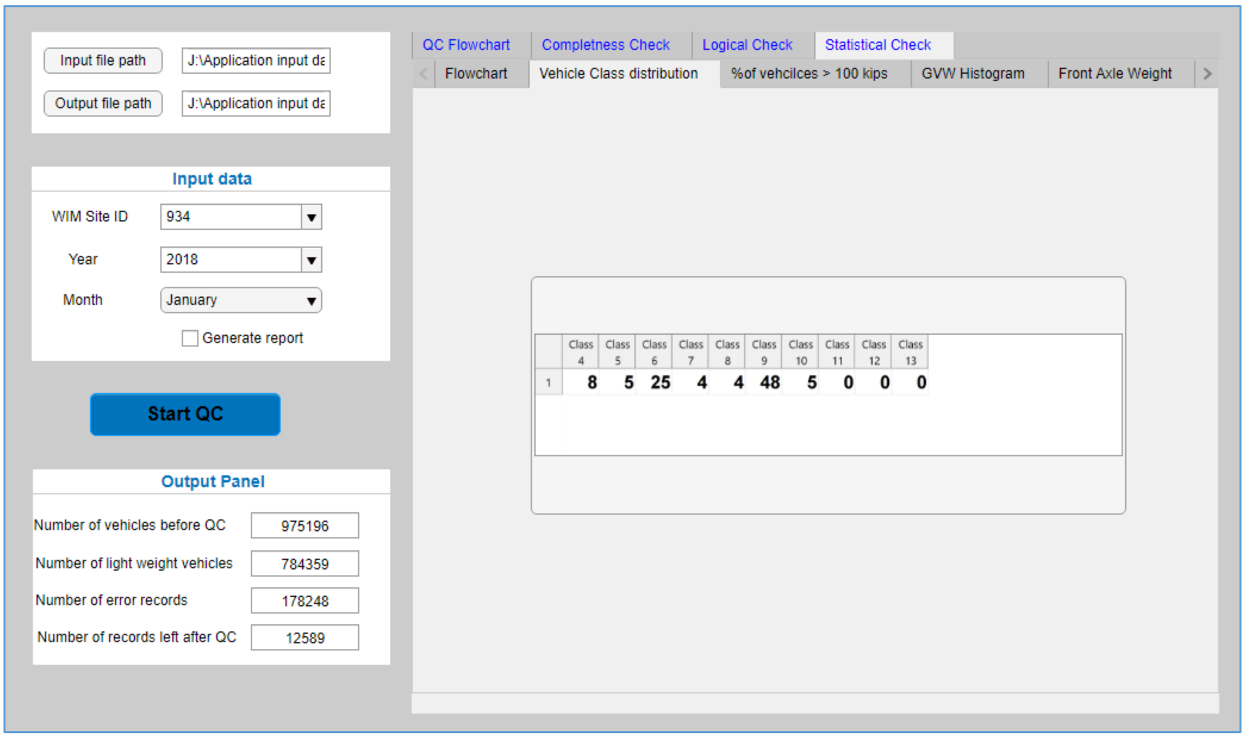


Figure E.8.16: Vehicle class distribution check result

3.2. Class 9 Check - Gross Vehicle Weight (GVW) > 100 kips

Objective: If there are a sensor operational problem, a large percentage of the GVW data of Class 9 vehicles are above 100 kips.

An example of one of the WIM Site is shown in Figure E.8.17. One way to interpret is to compare the results with other location. Usually, most of the locations have the same percentage Class 9 vehicles above 100 kips.

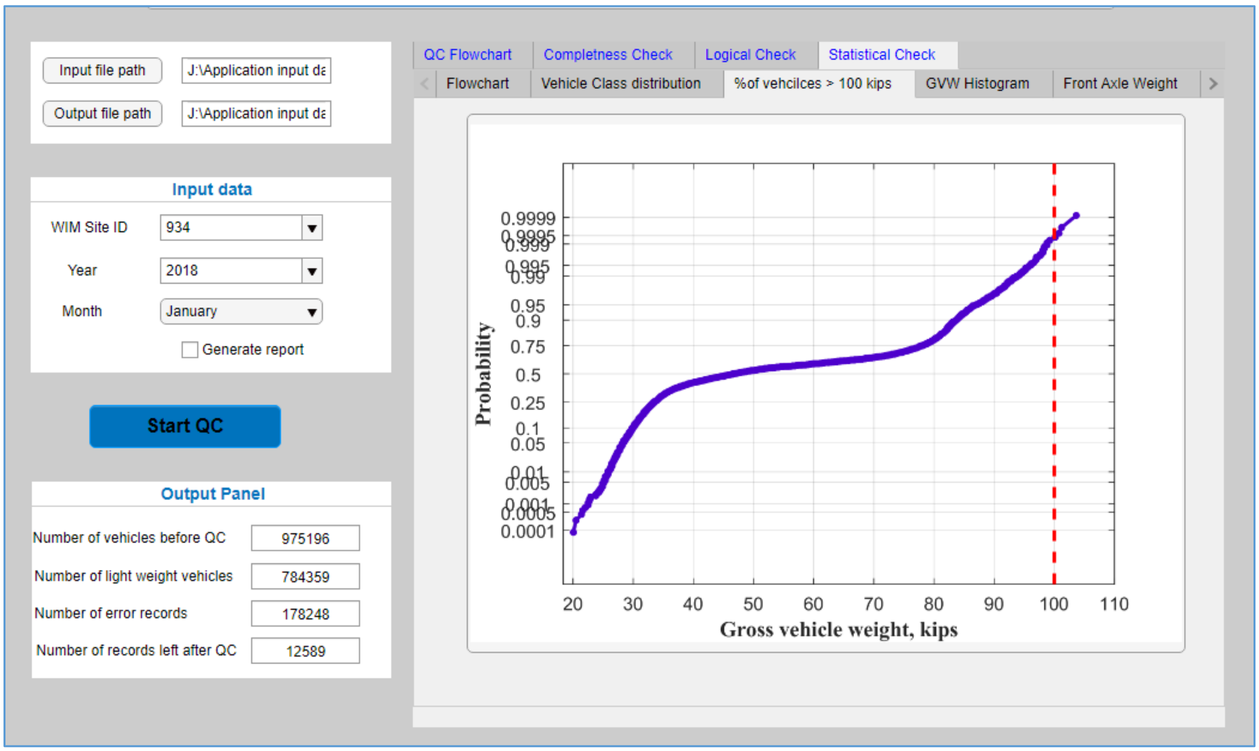


Figure E.8.17: Probability plot of GVW of Class 9 vehicles

3.3. Class 9 Check - GVW histogram check

Objective: A 4-kip bin width histogram is plotted and two peaks - unloaded peak between 28 and 36 kips range and a loaded peak between 72 and 80 kips range should be seen. A shift in the peak is of importance. Both peaks shifted in the same direction indicates most probably scale out of calibration. Single peak shift indicates an error, or it might be because of a change in traffic characteristics. If a valid reason can not explain a peak shift, then it is most probable that the sensor is out of calibration.

An example of one of the WIM Site is shown in Figure E.8.18.

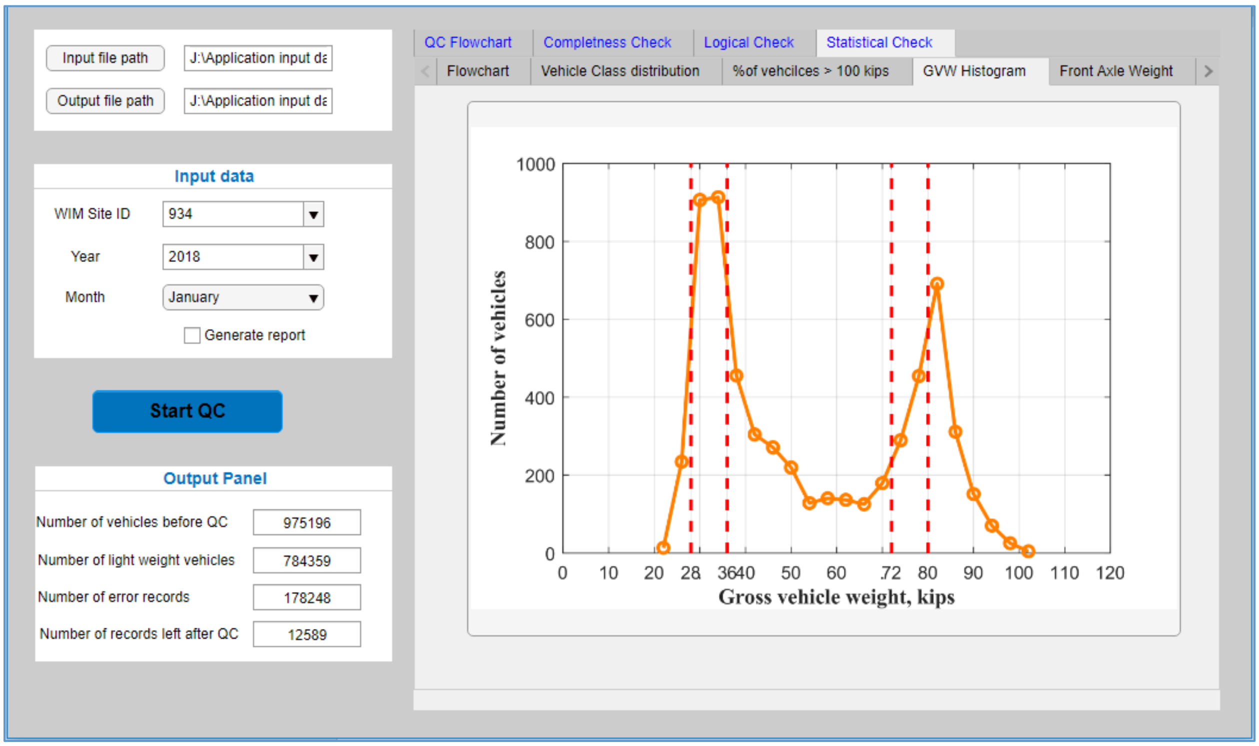


Figure E.8.18: Histogram of GVW of Class 9 vehicles

3.4. Class 9 Check - Front axle weight histogram check

Objective: A 1-kip bin width histogram is plotted and one peak between 8 and 12 kips range is seen. Front axle weight in most Class 9 vehicles are constant as its cabin part of a truck, and there cannot be much of weight difference irrespective of a truck loaded or unloaded. Alabama has front axle weight limit of 12 kips + 10% change on State highways. If a peak is not seen, the most probable cause is a problem with the sensor.

An example of one of the WIM Site is shown in Figure E.8.19.

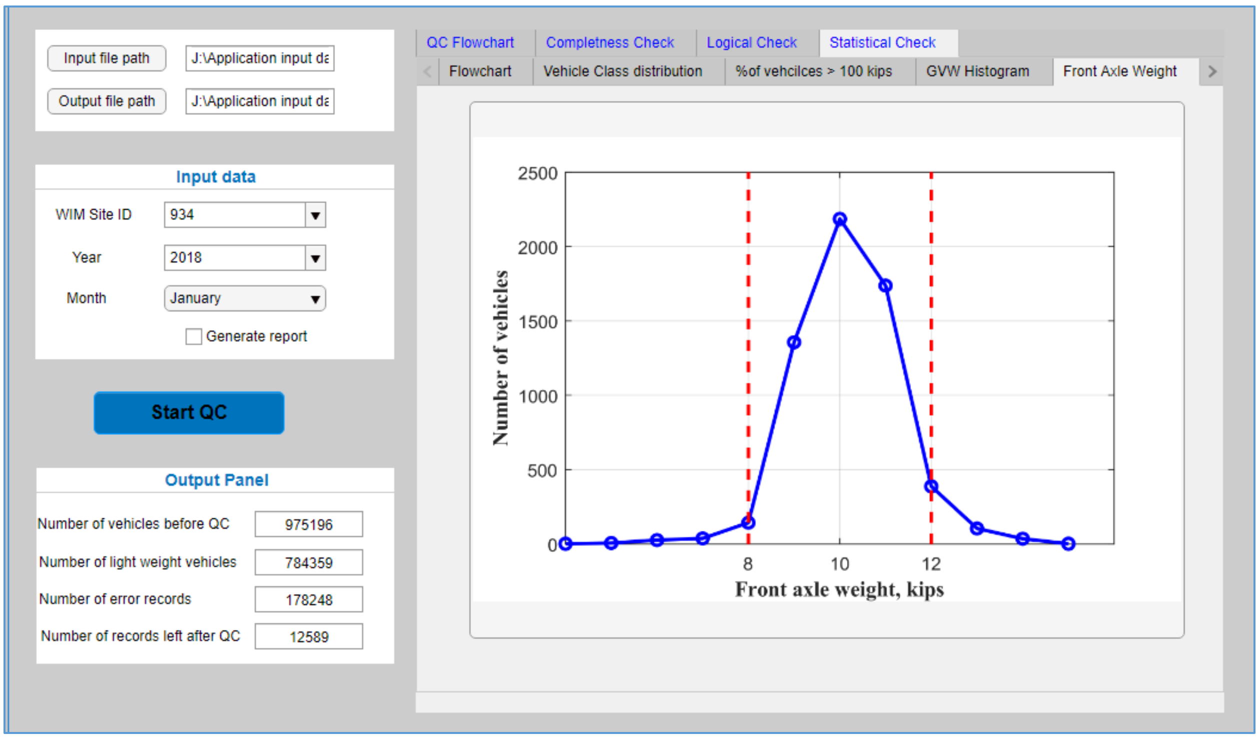


Figure E.8.19: Histogram of front axle weight of Class 9 vehicles

3.5. Class 9 Check – Tandem axle spacing histogram check

Objective: Probability plot of tandem axle spacing is plotted to check the condition of sensors. Class 9 trucks on the drive and rear tandem axle have almost constant spacing. Measurement of speed is an important factor to find the axle spacing in a WIM record. Also, inaccuracy in axle weight measurements is correlated to speed measurements. The results are not shown in the application but saved in the output folder.

3.6. Class 9 Check - Tandem Axle load spectra histogram

Objective: A 2-kip bin width histogram is plotted and one peak between 14 and 16 kips range and another peak between 32 and 38 kips range should be seen. Data are compared within each location for different years and months. If an abrupt change is noticed when compared to historical data, it indicates a problem with the sensor.

An example of one of the WIM Site is shown in Figure E.8.20.

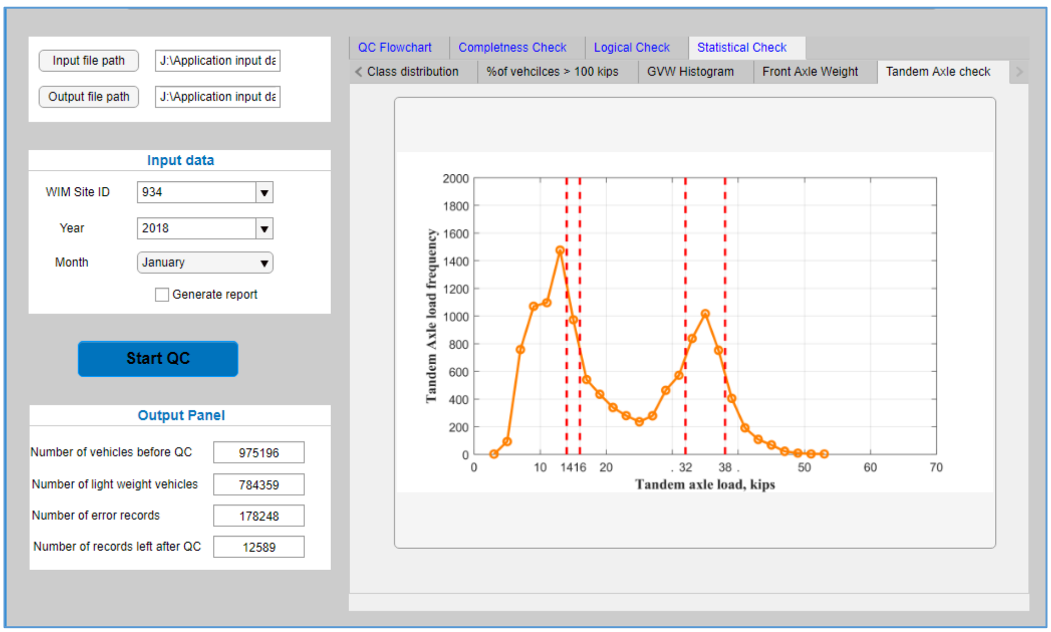


Figure E.8.20: Histogram of tandem axle loads of Class 9 vehicles

Appendix F : AL_WIM_DAI v1.0 user manual

Contents

Introduction

The service life of the bridge is affected by many factors such as traffic loads, natural hazards, defects in material production, etc. Traffic-induced loads by vehicular traffic cause damage to the bridge by fatigue and overload. Steel bridges are more prone to fatigue failure compared to other types of bridges. Every passage of a truck creates multiple stress cycles on a bridge and accumulates damage on a bridge. The entire fatigue process in a member includes the formation of a fatigue crack, crack growth, and final failure (Fisher et al. 1998). If the fatigue crack is not detected and properly maintained it might lead to the failure of the member. AASHTO LRFD Bridge design specifications (AASHTO 2017) has a design approach to design for fatigue. A code-specified fatigue design truck is used to restrict the stress range to address the fatigue. The AASHTO fatigue design truck used in the design is intended to control crack growth under repetitive loads and prevent fracture. However, in the service life of the bridge, there is an uncertainty of traffic loads the bridge experiences and damage accumulated on the bridge must be accessed periodically for proper maintenance and evaluation.

Long-term WIM recording provides an excellent tool to estimate the accumulated fatigue damage. AL_WIM_DAI v1.0 application provides a user-friendly interface to estimate the damage accumulated by the most fatigue prone details.

Firstly, this application checks the quality of WIM data and eliminates the questionable records, and a Quality Control (QC) procedure is shown in Figure E.8.1. Then the effective moment, M_{eff} and number of constant-amplitude cycles with the magnitude M_{eff}, N, is calculated to determine the nominal damage. The outputs from this application can be used to find site-specific and bridge-specific fatigue damage using the bridge-specific parameters, as demonstrated in the dissertation mentioned above.

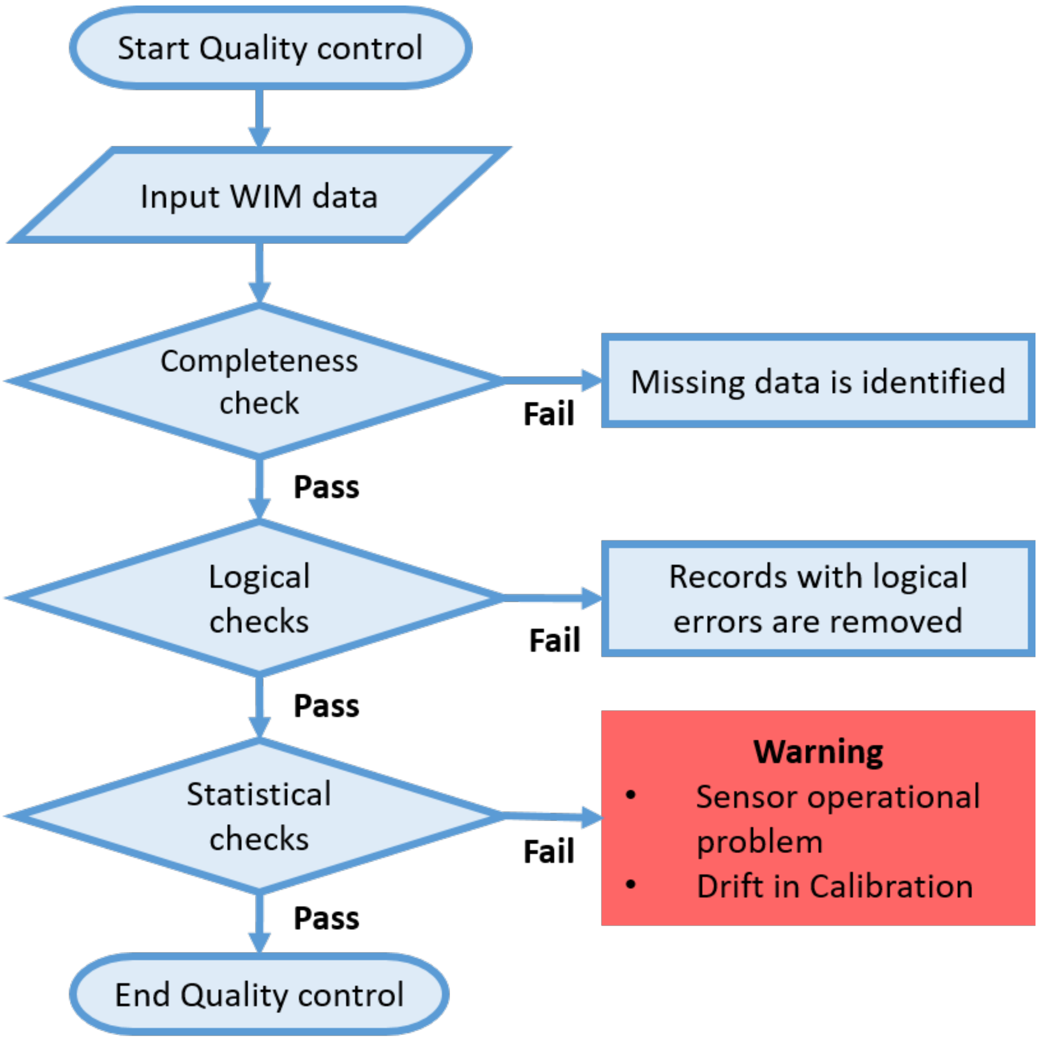


Figure F.8.1: Quality Control Procedure Flowchart

Chapter 1. Preparing Input Data

The data recorded by WIM sensors are stored in the data storage medium maintained by ALDOT in an encrypted format. The iAnalyze software provided by WIM system vendor International Road Dynamics Inc. (IRD) is used to decrypt WIM data to the desired format. For using it in this application, the Class 0 data is required to be decrypted in IRD ASCII Raw data format (as referred in section D.13.1 of iAnalyze Software operator’s manual) and Class 4-13 in TMG 2001 Truck Weight format (as referred in section D.9.5 of iAnalyze Software operator’s manual). For every WIM site for the selected month and year, there is always two input files, one containing Class 0 data and another containing Class 4-13.

Once the data is decrypted, it must be renamed in the following format to use in the application and can be stored in the user desired folder.

Class 0 data:

Syntax: **<Year>_<Month>_<WIMID>.csv**

Where,

<Year> is the year of the data
<Month> is the month of the data in number
<WIMID> is WIM station ID used by ALDOT
.csv is an extension of the file type which is saved by default

For example, the WIM data for WIM location 911 for the Year 2018 and Month of January the filename will be **2018_1_911.csv**

Class 4-13 data:

Syntax: **<Year>_<Month>_<WIMID>.WGT**

Where,

<Year> is the year of the data.
<Month> is the month of the data in number.
<WIMID> is WIM station ID used by ALDOT.
.WGT is the extension of the file type which is saved by default

For example, the WIM data for WIM location 911 for the Year 2018 and Month of January the filename will be **2018_1_911.WGT**

Note: It is important that filename of Class 0 data is in “.csv” and Class 4-13 is in “.WGT”

Chapter 2. Installing the application

The users are provided with the installation file named “MyAppInstaller_web” as shown in Figure E.8.2. Once the user double-clicks the installation file, the installation processes start as shown in Figure E.8.3. The installer will request the path to the installation folder where the application package is stored as shown in Figure E.8.4. A desktop shortcut can also be created for easier assess. After that ***Next*** button is clicked.

A Matlab compiler is required to run the application so the dialog box as shown in Figure E.8.5 is requested, and the user is required to click ***Install***. After the installation is complete, the dialog box as shown in Figure E.8.6 is seen indicating the successful completion of installation. Click *Finish* to complete the setup.

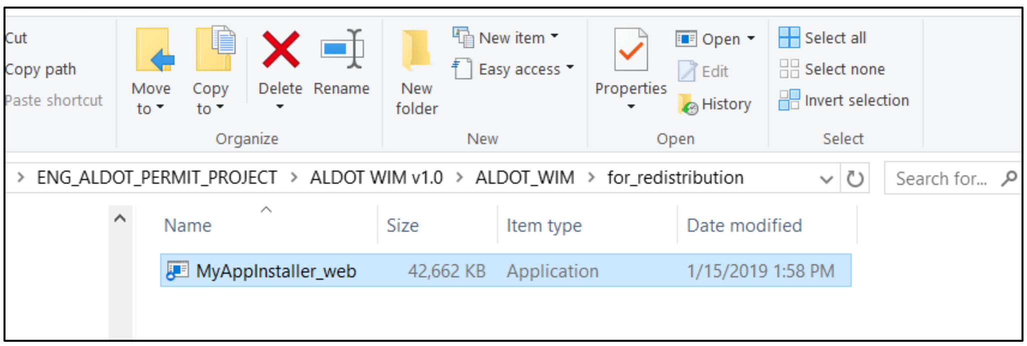


Figure F.8.2: Redistribution file provided to the user

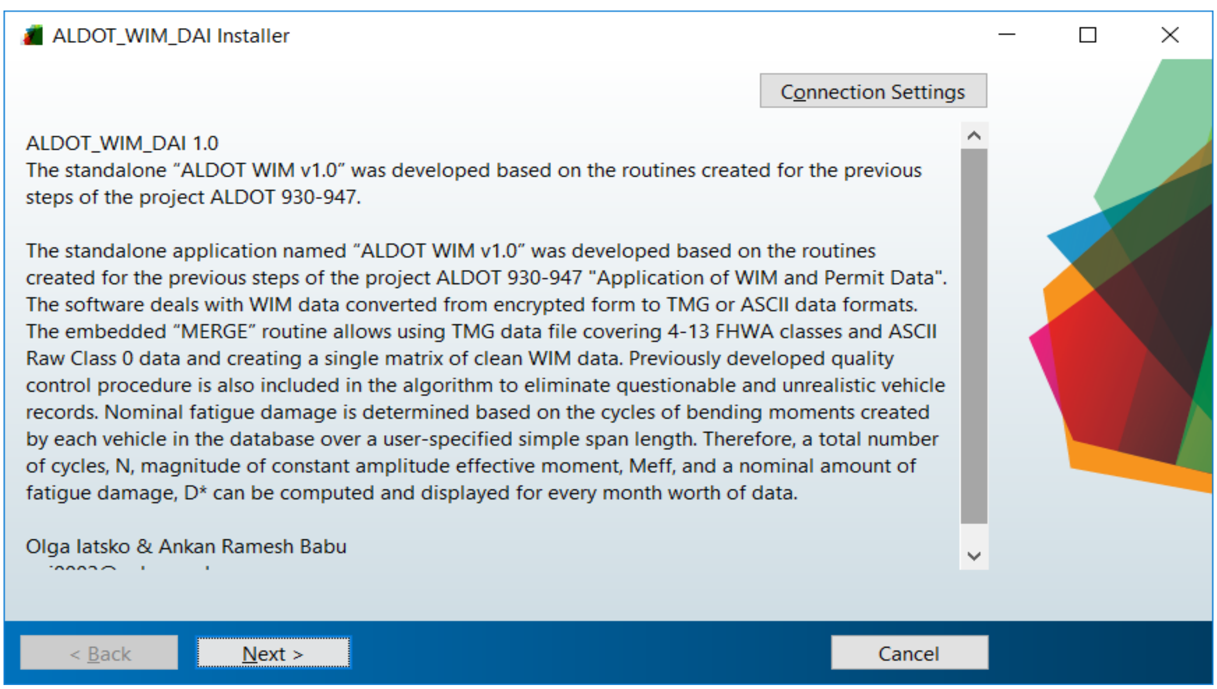


Figure F.8.3: Application installation startup screen

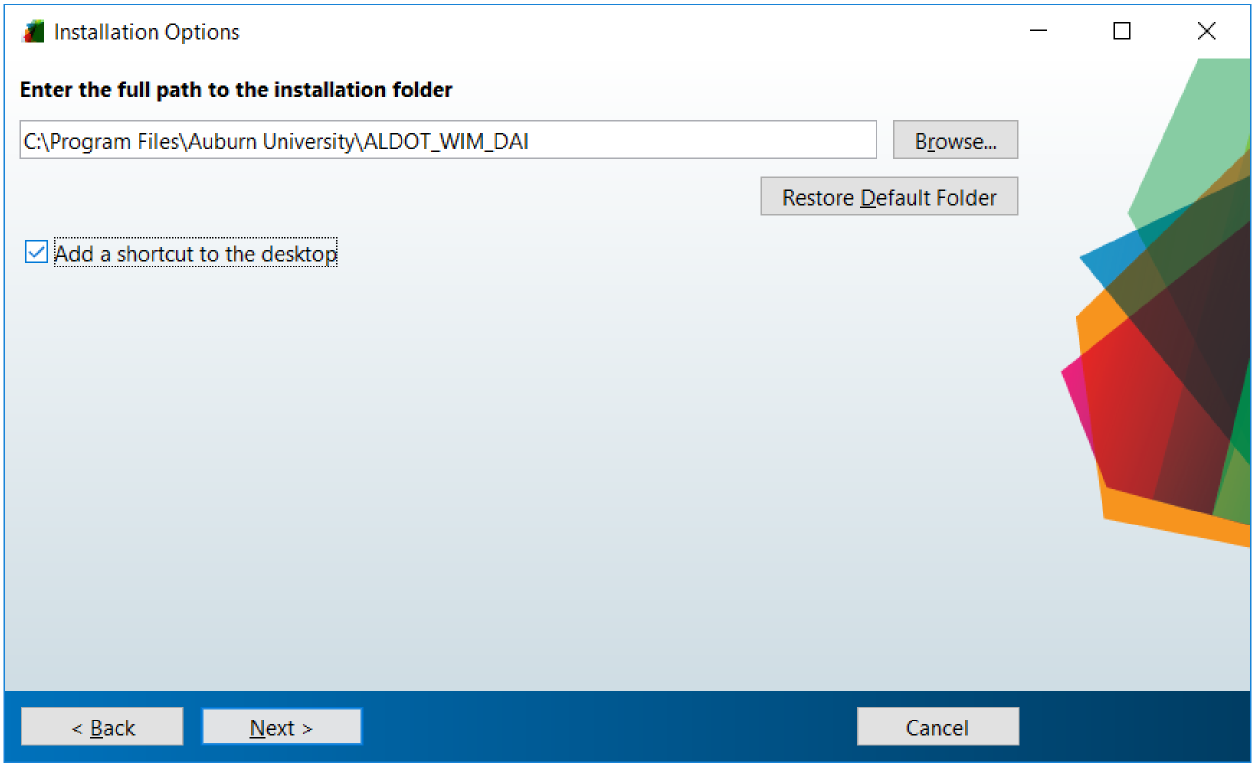


Figure F.8.4: Installation folder selection

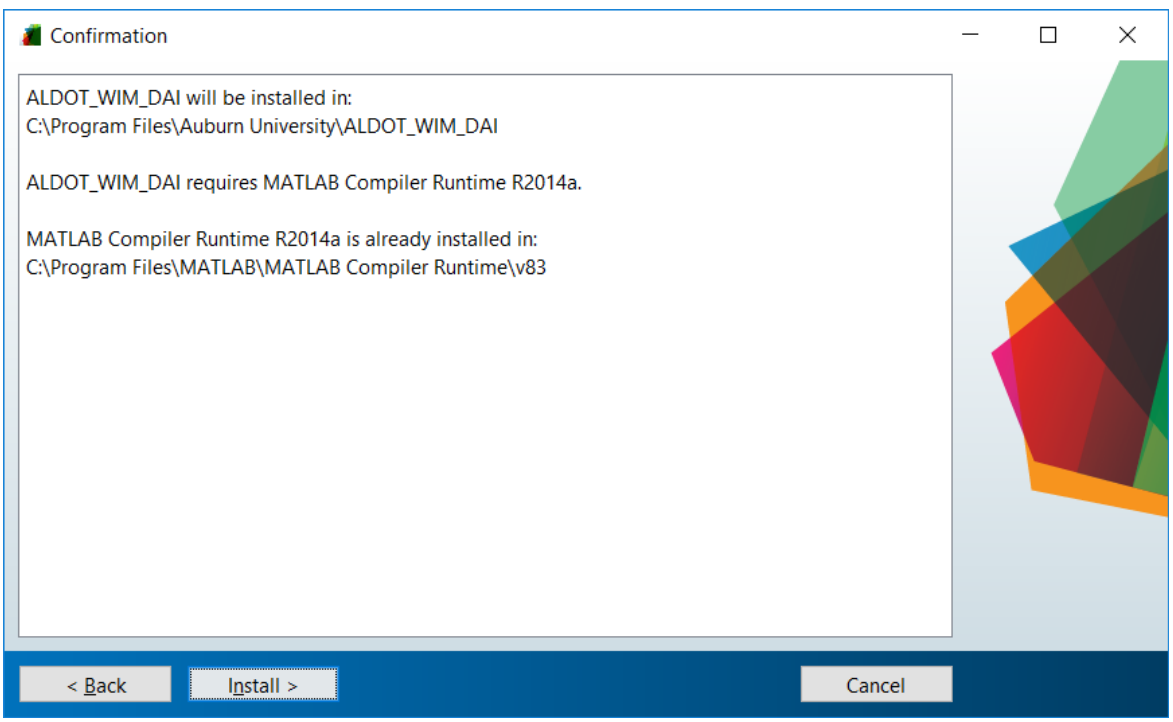


Figure F.8.5: Installation of MATLAB Compiler Runtime

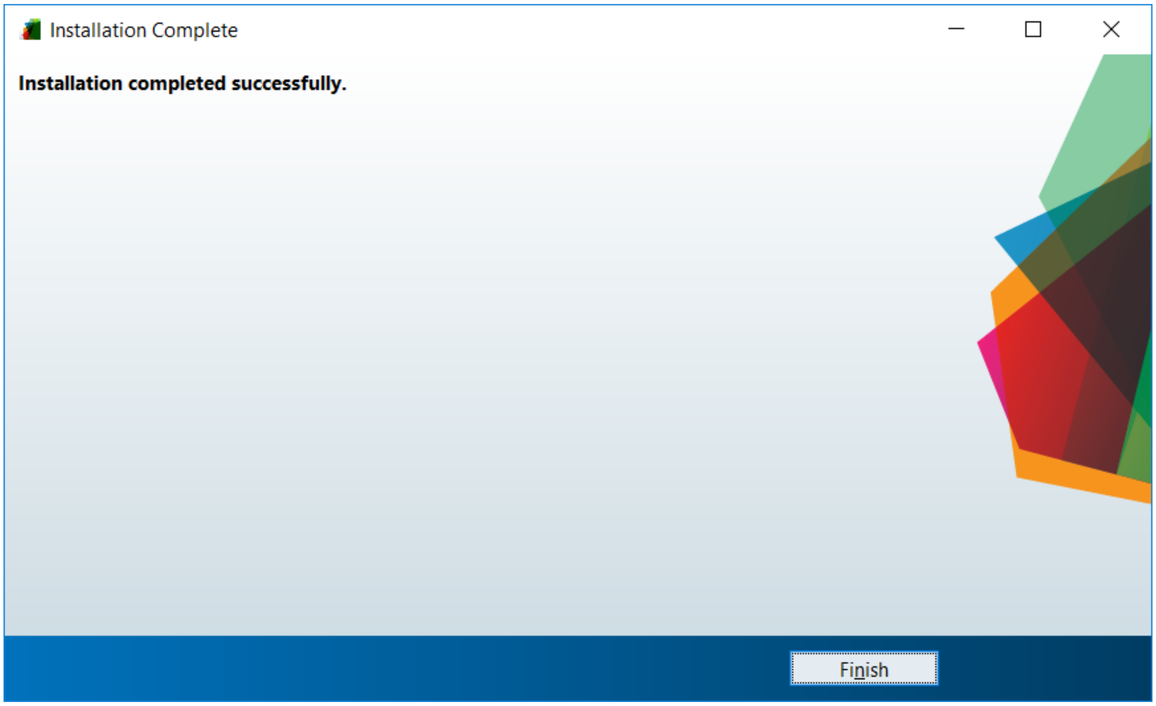


Figure F.8.6: Installation completed dialog box

Chapter 3. Step-by-step instructions to use application

After installing the application, double-clicking on the desktop shortcut icon opens the application. Alternatively, the application can also be opened by searching in the start menu. The startup screen of the application looks like it is shown in Figure E.8.7.

Figure F.8.7: AL_WIM_DAI application startup screen

__**Step 1:**__ Selecting the input (TMG and ASCII) and the output file path.

The input (TMG and ASCII) and out file path must be selected by clicking on the button as it is shown in Figure E.8.8. The ***Path to TMG files*** is the folder where the ***Class 4-13 data in .WGT format*** *and* ***Path to ASCII files*** where the ***renamed Class 0 data in a .csv*** **format** is stored. The output folder can be created by clicking on the "output file path" button and once the window pop-ups a new folder can be created by right-clicking the mouse and clicking on *New>>Folder* option. It is recommended to create output folder name in: **<Year>_<Month>_<WIMID>** format as shown earlier. The results of the DAI of each WIM Site are stored in the output folder. Also, the input data compiled of Class 0 and 4-13 are stored.

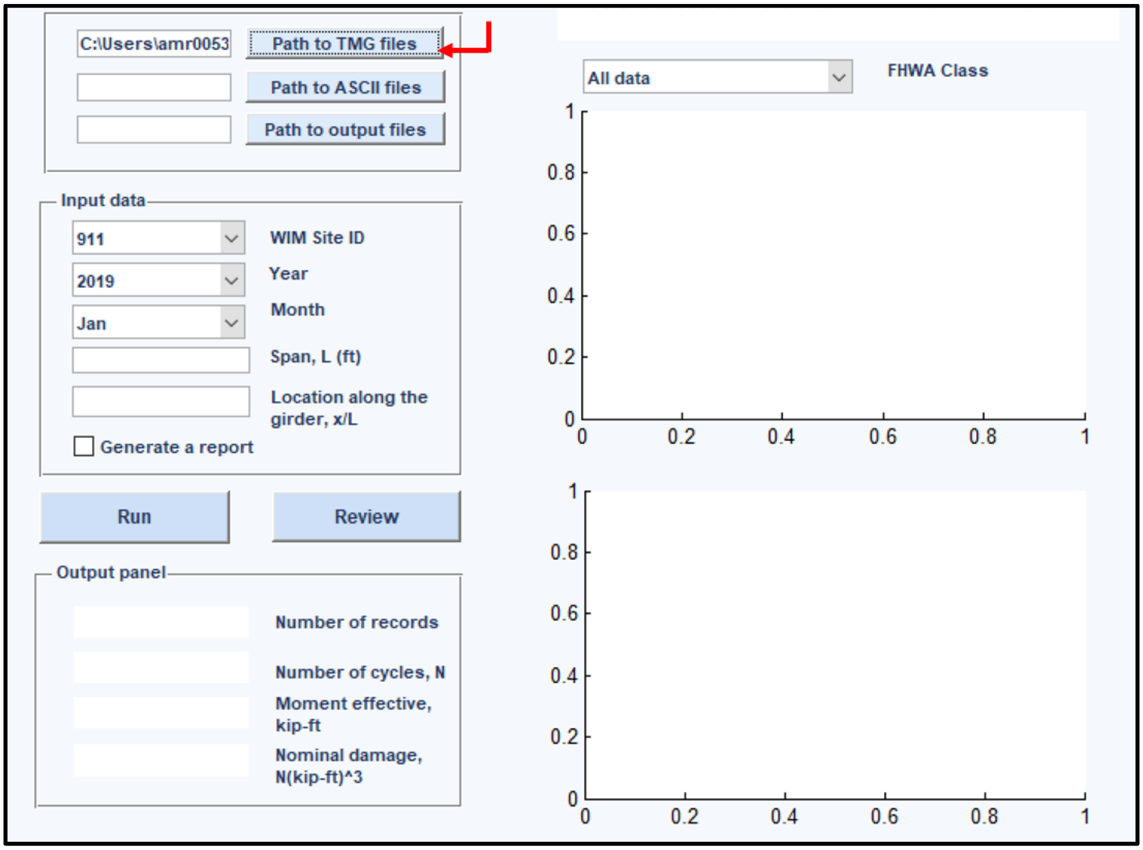


Figure F.8.8: Selecting input and output file path

<u>Step 2:</u> Inputting the data in *Input Data* panel

The Input Data panel is self-explanatory, the preferred *WIM Site ID* is selected from the drop-down menu or by inputting the WIM Station ID. Also, preferred *Year* and *Month* is selected. The span length and location along the span length can be specified. The effective moment and cycles are calculated for that location of the bridge. *Generate report* is selected if needed. An example of a screen after inputting the data is shown in Figure E.8.9.

Figure F.8.9: Inputting the data in *Input Data* panel

<u>Step 3:</u> Starting the DAI

The DAI is started once the ***Run*** button is pushed. The wait bar dialog box as shown in Figure E.8.10 is pop-upped once the ***Run*** button is pushed indicating the progress. Once the process is complete, the wait bar closes automatically indicating the process is finished. The interpretation of results is shown in the next chapter.

In case the input data filename is in the wrong format or input data is not in the input file path for the selected WIM Site ID, Year or Month then the error window pop-ups as shown in Figure E.8.11. For every WIM site for the selected month and year, there is always two input files, one containing Class 0 data and another containing Class 4-13. In case there is no data in Class 0, the iAnalyze creates an empty file.

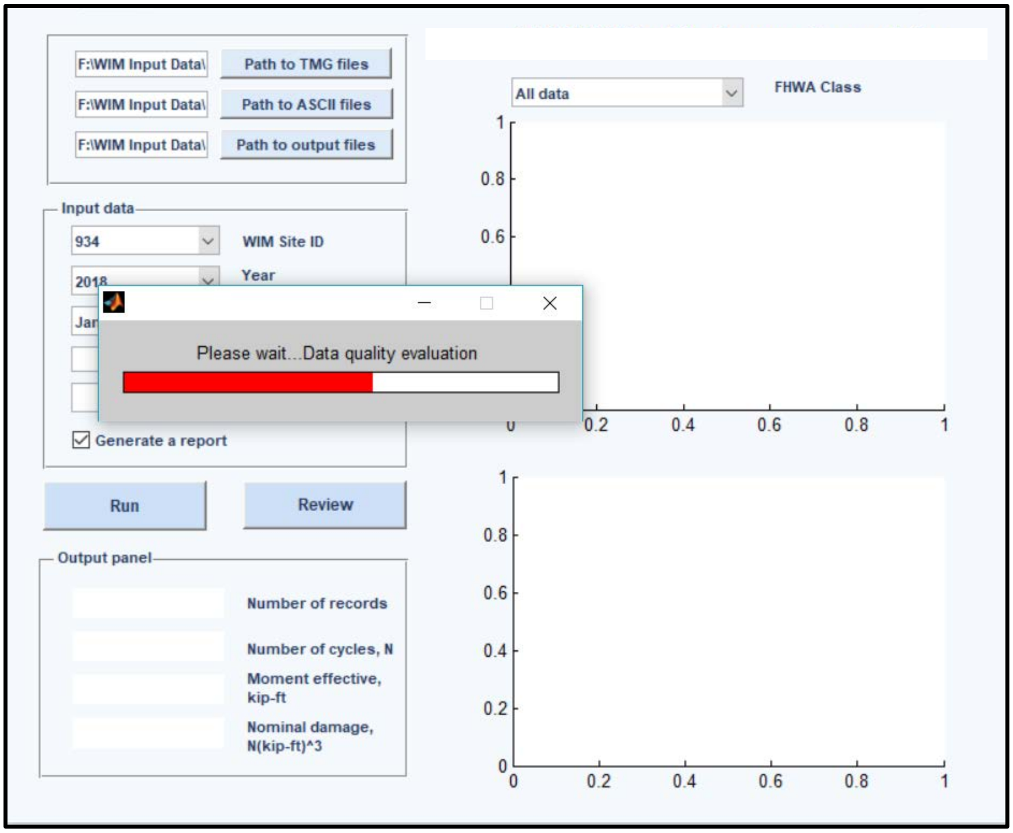


Figure F.8.10: Wait bar dialog box indicating the progress of DAI

Figure F.8.11: An error dialog box indicating the problem with input data

Chapter 4. Interpretation of the results

After the DAI calculations are finished, the wait bar dialog box closes, and the results are displayed in the *output panel* as shown in Figure E.8.12. The number of cycles, N and moment effective, M_{eff} for the selected location along the span length is calculated. Also, an interactive plot on the right side of the screen is generated showing the gross vehicle weight distribution among different FHWA classes and another plot showing the percentage of vehicles distributed among different FHWA classes.

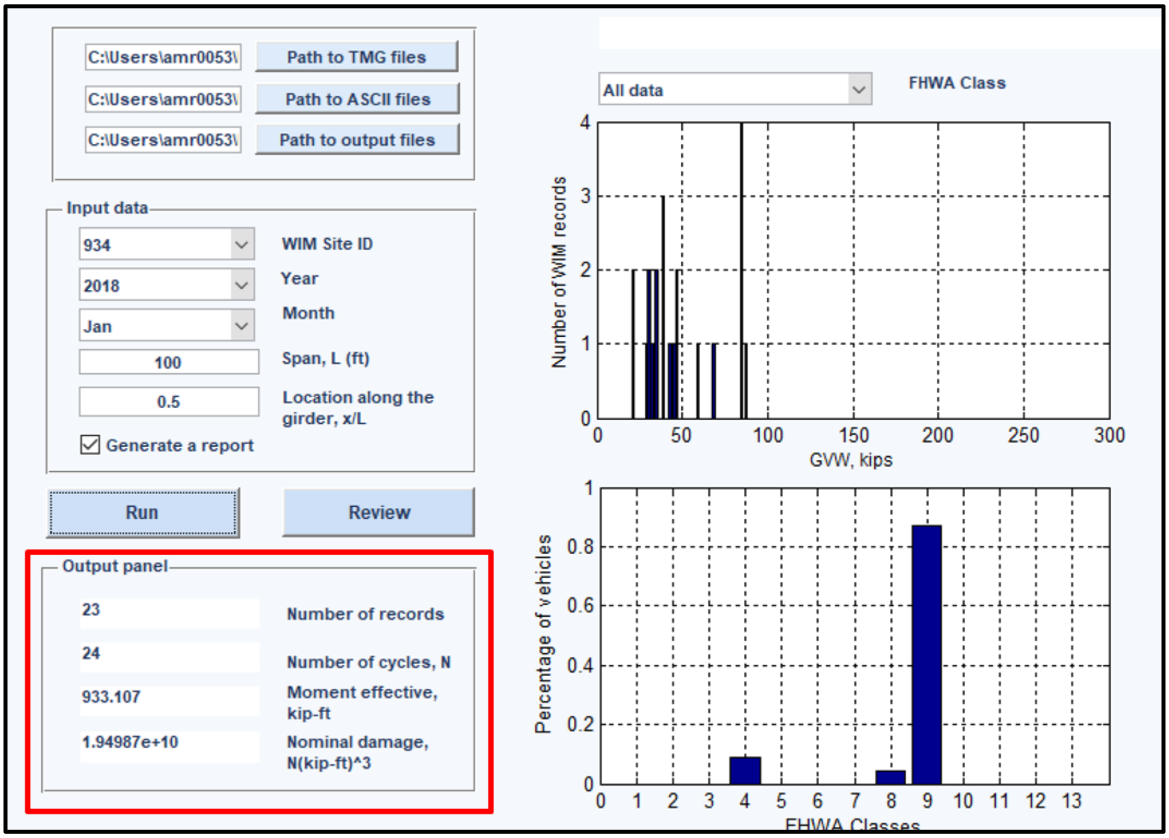


Figure F.8.12: DAI results shown in output panel after completion of the QC procedure

4. Viewing results for a particular vehicular class.

One the process has completed the results for all the FHWA vehicles together is shown by default. By selecting the drop-down menu as shown in Figure F.8.13 a particular class of interest can be selected.

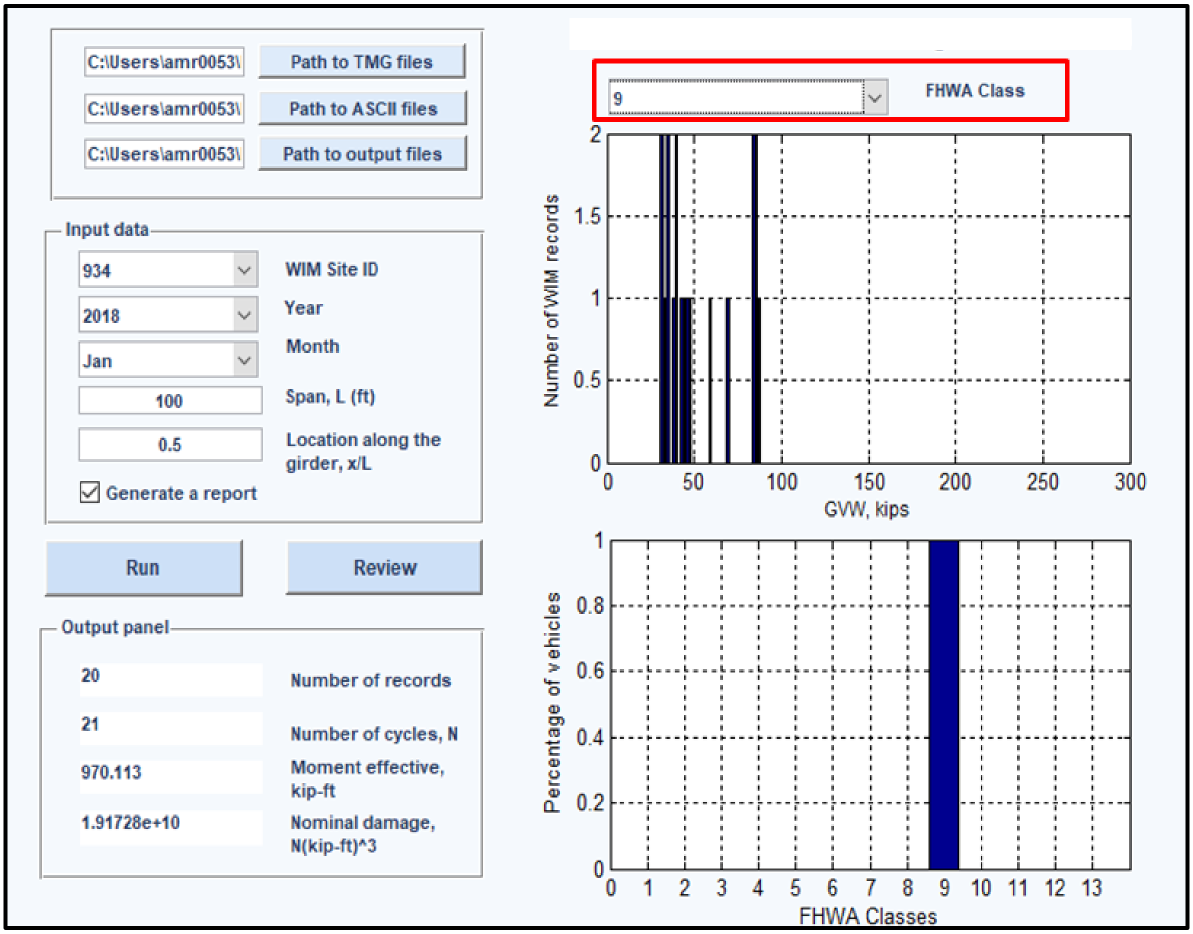


Figure F.8.13: DAI results selection for the particular Vehicle class

5. Viewing results for a particular direction of travel.

One the process has completed the results for all the directions together is shown by default. By selecting the drop-down menu as shown in Figure F.8.13 a direction of travel of interest can be selected.

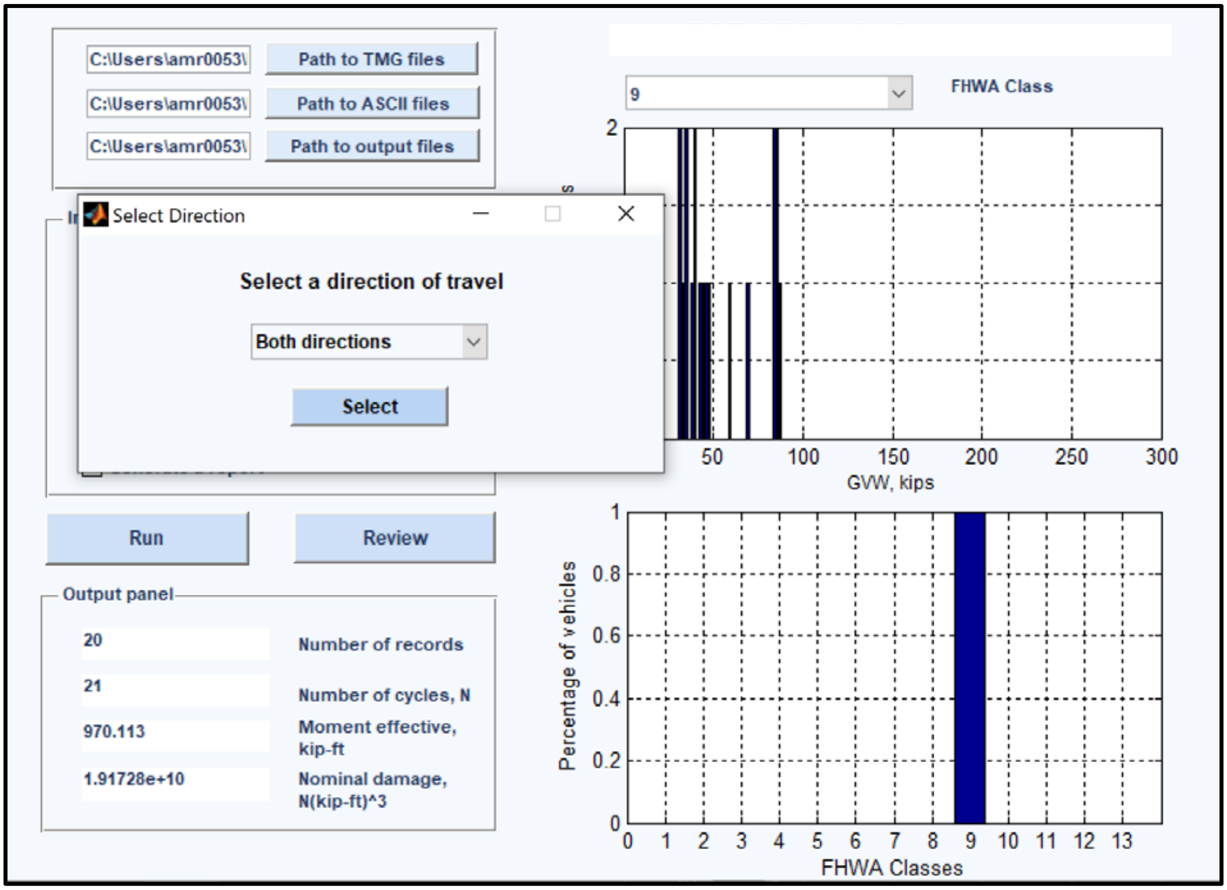


Figure F.8.14: DAI results selection for direction of travel

Chapter 5. Review of previously processed data.

Once the data for a particular WIM location, period, span length and location along span length is run, it is stored in the output folder. The results can be viewed later without running that whole process. Just the output folder location can be selected as shown in Figure F.8.15, and reminder input data is inputted to see the processed data. Rest of the feature such as the selection of class and direction of travel remains the same as discussed in Chapter 4.

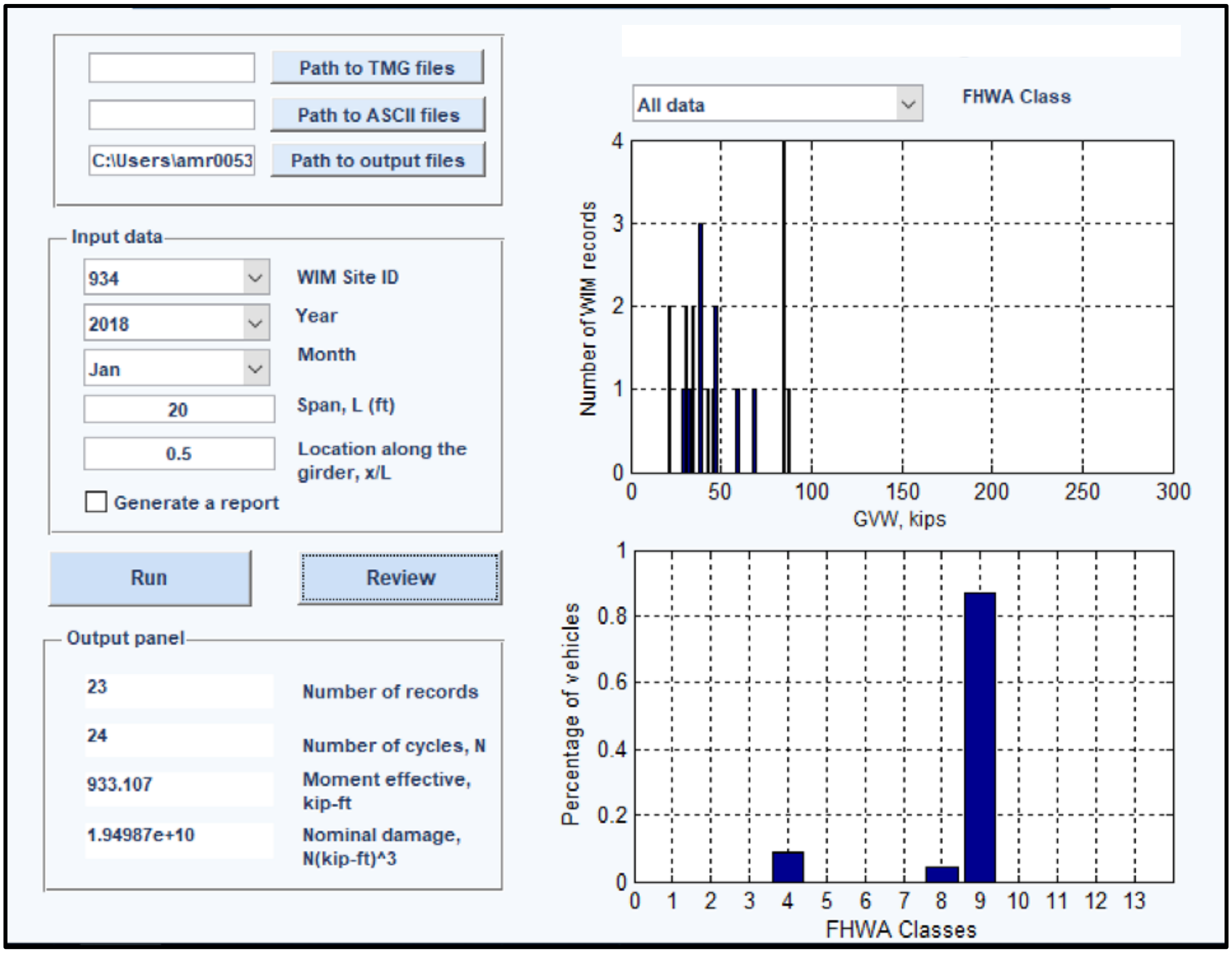
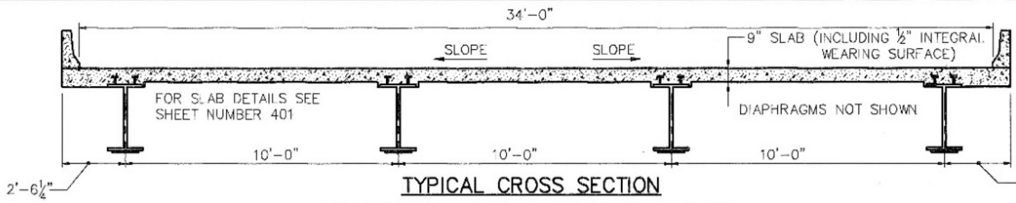
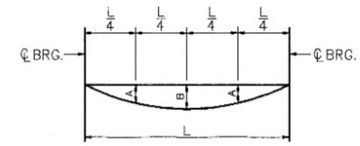


Figure F.8.15: Review of the processed data

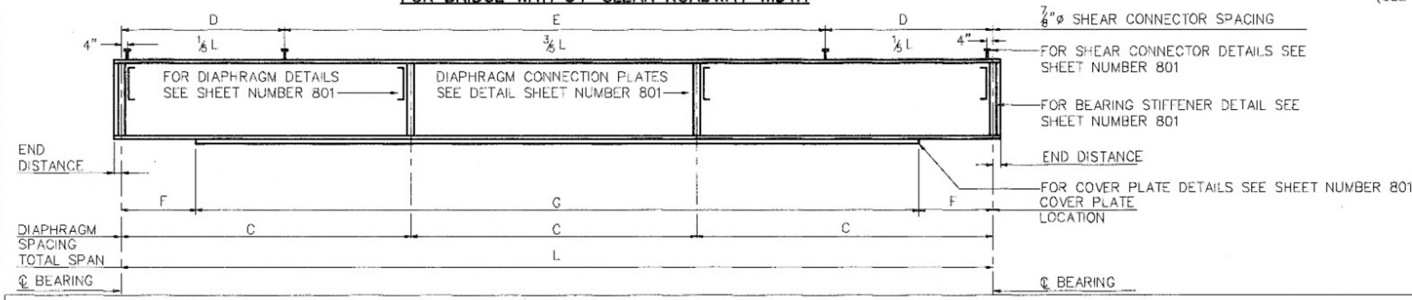
Appendix G : AISI Short-Span Steel Bridge Designs



TYPICAL CROSS SECTION
FOR BRIDGE WITH 34' CLEAR ROADWAY WIDTH



DEAD LOAD DEFLECTION DIAGRAM
(SEE TABLE BELOW)



COMPOSITE ROLLED BEAMS WITH WELDED COVER PLATE

SPAN (L) – Ft.	REQUIRED BEAM	REQUIRED COVER PLATE: THICKNESS X WIDTH	REQUIRED COVER PLATE: WELD SIZE	REQUIRED COVER PLATE: F	REQUIRED COVER PLATE: G	SIZE OF DIAPHRAGM	DIAPHRAGM SPACING (C) – Ft.	NUMBER OF DIAPHRAGMS	DEAD LOAD DEFLECTION – In.: A	DEAD LOAD DEFLECTION – In.: B	SUPERIMPOSED DEAD LOAD DEFLECTION – In.: A	SUPERIMPOSED DEAD LOAD DEFLECTION – In.: B	SHEAR CONNECTOR MAX. SPACING–In.: D	SHEAR CONNECTOR MAX. SPACING–In.: E	REACTION–Kips. (UNFACTORED): DEAD LOAD	REACTION–Kips. (UNFACTORED): SUPERIMPOSED DEAD LOAD	REACTION–Kips. (UNFACTORED): LIVE LOAD WITH IMPACT	MOMENTS Kips–Ft. (UNFACTORED): DEAD LOAD	MOMENTS Kips–Ft. (UNFACTORED): SUPERIMPOSED DEAD LOAD	MOMENTS Kips–Ft. (UNFACTORED): LIVE LOAD WITH IMPACT
55	W 30x99	15/16 x 8 1/2	5/16	4.5	46	C 15x33.9	18.33	4	1 7/16	1	1/8	3/16	5	7	41.8	11.7	91.5	574.8	160.7	1032.0
60	W 33x118	3/4 x 9 1/2	1/4	5.5	49	C 15x33.9	20.00	4	1 1/2	2 1/8	3/16	1/4	5	7	46.2	12.8	92.4	693.0	191.3	1155.9
65	W 33x130	7/8 x 9 1/2	5/16	6.0	53	C 15x33.9	21.67	4	1 13/16	2 9/16	3/16	5/16	5	7	50.4	13.8	93.1	818.6	224.5	1278.8
70	W 36x135	15/16 x 10	5/16	6.5	57	MC 18x42.7	23.33	4	2	2 7/8	1/4	5/16	6	8	55.0	14.9	93.7	961.6	260.3	1400.6
75	W 40x149	15/16 x 10	5/16	7.5	60	MC 18x42.7	25.00	4	2 5/16	3 3/16	1/4	3/8	6	8	60.0	15.9	94.1	1125.0	298.8	1521.4
80	W 40x167	15/16 x 10	5/16	8.5	63	MC 18x42.7	20.00	5	2 9/16	3 5/8	5/16	7/16	6	8	64.8	17.0	94.5	1296.0	340.0	1641.3
85	W 40x167	1 3/8 x 10	5/16	8.5	68	MC 18x42.7	21.25	5	3	4 1/4	3/8	1/2	6	8	70.6	18.1	94.7	1499.2	383.8	1760.4
90	W 40x183	1 1/2 x 10	5/16	9.5	71	MC 18x42.7	22.50	5	3 1/2	5	3/8	9/16	6	8	79.2	19.1	94.9	1782.0	430.3	1878.7
95	W 40x211	1 1/2 x 10	5/16	10.5	74	MC 18x42.7	23.75	5	4 1/16	5 9/16	1/2	11/16	7	9	83.6	20.2	95.1	1985.5	479.5	1996.3
100	W 40x211	1 7/8 x 10	5/16	10.5	79	MC 18x42.7	25.00	5	4 1/2	6 3/8	9/16	3/4	7	9	88.0	21.3	95.2	2200.0	531.3	2113.2
105	W 36x230	1 5/8 x 14	5/16	11.0	83	MC 18x42.7	21.00	6	5 5/8	7 15/16	11/16	1	6	8	92.4	22.3	95.2	2425.5	585.7	2229.5
110	W 36x245	1 7/8 x 14	5/16	11.5	87	MC 18x42.7	22.00	6	6 9/16	9 1/4	13/16	1 1/8	6	8	102.3	23.4	95.3	2813.3	642.8	2345.2
115	W 36x280	1 7/8 x 14	5/16	13.5	88	MC 18x42.7	23.00	6	6 13/16	9 11/16	7/8	1 1/4	7	8	107.0	24.4	95.3	3074.8	702.6	2460.3
120	W 36x300	2 x 14	5/16	14.5	91	MC 18x42.7	24.00	6	7 1/2	10 5/8	1	1 3/8	7	9	111.6	25.5	95.3	3348.0	765.0	2574.9

NOTES:
1. FOR BEARINGS AND END DISTANCES SEE SHEET NUMBER 807.
2. THE DEAD LOAD IS THE ADDITION OF THE LOADS IMPOSED BY THE BEAM/GIRDER, THE CONCRETE DECK SLAB, THE STAY-IN-PLACE DECK FORM AND THE CONCRETE HAUNCH. THE SUPERIMPOSED DEAD LOAD IS THE ADDITION OF THE LOADS IMPOSED BY THE CONCRETE PARAPETS AND THE FUTURE WEARING SURFACE.

STATE OF MARYLAND — CHARLES ROBERT KENNEDY — REGISTERED PROFESSIONAL ENGINEER

AISI SHORT SPAN STEEL BRIDGES

COMPOSITE ROLLED BEAMS WITH WELDED COVER PLATE NORMAL WEIGHT CONCRETE

DATE: SEPTEMBER, 1994		DO NOT SCALE	HS 25-44 LOADING
DRAFTED BY: S.L.P.	DESIGNED BY: S.T.	CHECKED BY: C.R.K.	34'-0" ROADWAY 406

Appendix H : Summary of Legal and Overloaded Vehicles Sorted Based on FHWA Vehicle Class

In this Appendix, the summary of legal and overloaded vehicles sorted based on FHWA vehicle class is shown in Table H.1 and H.2 for year 2014, and Table H.3 and H.4 for year 2015.

Table H.1: Summary for year 2014 for WIM sites 911 to 934.

WIM site	Vehicle Class	Legal Vehicles					Overloaded Vehicles				
		Lane					Lane				
		1	2	3	4	All	1	2	3	4	All
911	0	-	-	-	-	51	-	-	-	-	126
	4	6,312	551	1,527	5,956	14,346	96	106	1,655	68	1,925
	5	3,755	337	717	3,302	8,111	67	81	575	177	900
	6	5,338	364	637	5,352	11,691	313	15	96	405	829
	7	104	17	103	330	554	540	21	96	321	978
	8	7,708	582	1,302	8,195	17,787	56	36	908	114	1,114
	9	122,831	7,886	6,096	126,040	262,853	13,854	331	198	1,117	15,500
	10	2,791	203	218	3,193	6,405	630	33	43	361	1,067
	11	5,859	237	298	6,102	12,496	40	9	96	30	175
	12	3	7	62	107	179	6	3	42	3	54
	13	47	4	58	221	330	203	11	14	140	368
Total		154,748	10,188	11,018	158,798	334,803	15,805	646	3,723	2,736	23,036
931	0	-	-	-	-	46	-	-	-	-	501
	4	21,748	2,654	2,348	16,209	42,959	3,478	2,405	1,020	4,874	11,777
	5	7,413	984	1,076	10,163	19,636	752	18,550	149	3,707	23,158
	6	8,653	652	1,106	7,437	17,848	793	1,052	83	1,047	2,975
	7	109	12	15	83	219	147	182	27	273	629
	8	25,268	2,804	2,305	20,594	50,971	3,759	1,758	389	4,496	10,402
	9	430,244	38,904	39,508	280,462	789,118	218,286	22,695	32,229	280,960	554,170
	10	7,954	713	1,093	7,066	16,826	2,668	445	356	2,909	6,378
	11	8,782	368	633	8,956	18,739	7,091	350	361	6,862	14,664
	12	49	10	5	207	271	37	17	2	357	413
	13	179	27	36	234	476	890	90	75	865	1,920
Total		510,399	47,128	48,125	351,411	957,109	237,901	47,544	34,691	306,350	626,987
933	0	-	-	-	-	26	-	-	-	-	506
	4	4,445	3,487	2,250	1,332	11,514	116	125	73	6,103	6,417
	5	3,962	2,471	1,864	180	8,477	268	47	85	430	830
	6	10,598	4,094	4,257	25	18,974	385	94	177	67	723
	7	248	35	47	31	361	1,438	153	605	173	2,369
	8	11,055	4,778	2,472	179	18,484	179	122	112	882	1,295
	9	221,819	67,629	43,096	32	332,576	7,220	2,574	2,654	124	12,572
	10	2,588	734	314	3	3,639	302	111	83	6	502
	11	4,802	2,076	824	9	7,711	10	10	21	21	62
	12	12	2	12	12	38	10	1	1	25	37
	13	85	12	23	6	126	188	23	11	13	235
Total		259,614	85,318	55,159	1,809	401,926	10,116	3,260	3,822	7,844	25,548
934	0	-	-	-	-	31	-	-	-	-	11
	4	6,358	1,489	1,286	6,953	16,086	421	255	11	146	833
	5	3,512	574	763	3,970	8,819	163	48	18	259	488
	6	11,784	3,900	1,750	21,296	38,730	3,630	1,944	45	473	6,092
	7	172	10	6	48	236	4,155	180	95	1,166	5,596
	8	3,549	207	353	3,008	7,117	35	1	1	29	66
	9	34,267	1,349	2,043	33,141	70,800	1,450	74	110	4,604	6,238
	10	2,645	103	177	2,365	5,290	385	5	9	319	718
	11	665	13	21	633	1,332	7	0	0	4	11
	12	182	0	0	4	186	5	2	0	1	8
	13	278	0	0	21	299	218	6	0	40	264
Total		63,412	7,645	6,399	71,439	148,926	10,469	2,515	289	7,041	20,325

Table H.2: Summary for year 2014 for WIM sites 942 to 964.

WIM site	Vehicle Class	Legal Vehicles					Overloaded Vehicles				
		Lane					Lane				
		1	2	3	4	All	1	2	3	4	All
942	0	-	-	-	-	17	-	-	-	-	49
	4	7,466	1,129	1,493	11,335	21,423	159	36	20	273	488
	5	3,139	525	586	4,855	9,105	65	5	13	429	512
	6	10,640	1,517	818	4,761	17,736	138	13	21	377	549
	7	83	7	25	95	210	167	12	597	6,618	7,394
	8	18,237	1,718	1,291	17,390	38,636	73	2	22	495	592
	9	290,772	23,278	21,367	298,608	634,025	2,654	357	692	12,796	16,499
	10	4,327	306	500	5,110	10,243	864	129	1,230	8,775	10,998
	11	7,564	411	341	8,363	16,679	11	0	5	124	140
	12	14	1	1	13	29	1	0	0	4	5
	13	94	7	11	210	322	253	25	42	961	1,281
Total		342,336	28,899	26,433	350,740	748,425	4,385	579	2,642	30,852	38,507
960	0	-	-	-	-	6	-	-	-	-	61
	4	3,006	2,271	0	0	5,277	143	162	0	0	305
	5	2,664	2,643	0	0	5,307	121	91	0	0	212
	6	13,273	11,973	0	0	25,246	478	2,681	0	0	3,159
	7	90	171	0	0	261	1,279	2,364	0	0	3,643
	8	4,469	3,280	0	0	7,749	18	42	0	0	60
	9	106,421	75,026	0	0	181,447	7,172	41,428	0	0	48,600
	10	7,790	3,259	0	0	11,049	4,020	6,738	0	0	10,758
	11	6	21	0	0	27	1	0	0	0	1
	12	47	4	0	0	51	942	10	0	0	952
	13	296	256	0	0	552	315	315	0	0	630
Total		138,062	98,904	0	0	236,972	14,489	53,831	0	0	68,381
961	0	-	-	-	-	80	-	-	-	-	164
	4	13,514	2,579	1,258	7,374	24,725	1,956	518	492	8,392	11,358
	5	4,941	1,595	827	24,768	32,131	310	179	86	3,786	4,361
	6	9,509	1,858	1,311	12,488	25,166	1,660	245	266	2,486	4,657
	7	554	96	14	960	1,624	200	22	17	1,557	1,796
	8	17,253	3,246	1,061	17,893	39,453	469	215	70	23,158	23,912
	9	449,673	50,813	20,323	10,155	530,964	36,905	5,557	7,575	28,927	78,964
	10	9,451	1,346	594	1,440	12,831	1,148	190	771	8,332	10,441
	11	10,531	1,060	456	719	12,766	579	56	62	2,133	2,830
	12	54	7	6	760	827	13	4	2	3,103	3,122
	13	870	45	14	714	1,643	621	83	49	5,378	6,131
Total		516,350	62,645	25,864	77,271	682,210	43,861	7,069	9,390	87,252	147,736
964	0	-	-	-	-	12	-	-	-	-	42
	4	8,558	1,653	2,218	10,265	22,694	220	64	37	282	603
	5	5,319	813	1,184	4,830	12,146	296	28	51	277	652
	6	11,151	1,444	1,511	8,050	22,156	1,201	61	67	446	1,775
	7	113	17	30	61	221	977	67	470	2,596	4,110
	8	15,138	2,122	3,087	15,815	36,162	370	32	65	756	1,223
	9	195,859	31,101	35,789	201,546	464,295	14,761	1,261	1,840	24,878	42,740
	10	3,296	489	738	3,713	8,236	369	50	494	2,150	3,063
	11	8,471	1,246	2,037	8,349	20,103	134	7	28	293	462
	12	7	10	1	12	30	3	2	0	7	12
	13	48	10	32	176	266	117	18	71	829	1,035
Total		247,960	38,905	46,627	252,817	586,321	18,448	1,590	3,123	32,514	55,717

Table H.3: Summary for year 2015 for WIM sites 911 to 934.

WIM site	Vehicle Class	Legal Vehicles					Overloaded Vehicles				
		Lane					Lane				
		1	2	3	4	All	1	2	3	4	All
911	0	-	-	-	-	28	-	-	-	-	23
	4	6,119	493	838	6,472	13,922	83	29	57	356	525
	5	3,405	277	478	4,159	8,319	53	21	30	233	337
	6	5,504	362	740	6,948	13,554	210	17	16	510	753
	7	118	12	14	271	415	741	38	15	388	1,182
	8	7,142	592	493	7,483	15,710	63	16	34	184	297
	9	113,241	7,974	7,207	132,454	260,876	12,388	201	67	1,371	14,027
	10	2,851	242	190	2,726	6,009	528	16	27	487	1,058
	11	5,505	285	220	6,273	12,283	80	5	0	59	144
	12	313	6	8	47	374	37	1	2	1	41
	13	295	10	5	68	378	153	5	2	77	237
Total		144,493	10,253	10,193	166,901	331,868	14,336	349	250	3,666	18,624
931	0	-	-	-	-	55	-	-	-	-	554
	4	20,482	1,919	2,566	17,456	42,423	9,611	2,587	1,203	6,170	19,571
	5	12,320	866	1,277	10,993	25,456	17,052	15,796	151	2,606	35,605
	6	8,494	504	1,205	7,629	17,832	2,220	588	80	1,091	3,979
	7	160	12	17	85	274	588	71	25	240	924
	8	24,212	2,088	2,887	24,019	53,206	6,104	1,621	430	5,148	13,303
	9	355,408	23,704	40,196	275,236	694,544	197,433	14,128	34,653	301,383	547,597
	10	6,391	428	1,076	6,901	14,796	2,696	242	301	3,114	6,353
	11	7,231	245	584	8,249	16,309	7,327	369	376	7,596	15,668
	12	52	4	7	162	225	133	14	5	285	437
	13	218	22	30	278	548	736	47	66	911	1,760
Total		434,968	29,792	49,845	351,008	865,668	243,900	35,463	37,290	328,544	645,751
933	0	-	-	-	-	35	-	-	-	-	205
	4	3,736	3,490	1,220	452	8,898	108	117	47	1,536	1,808
	5	3,795	2,610	973	147	7,525	270	57	43	403	773
	6	10,266	4,746	2,006	33	17,051	336	152	77	57	622
	7	269	56	33	52	410	2,390	837	296	76	3,599
	8	12,210	4,783	1,391	112	18,496	140	99	55	287	581
	9	221,327	63,447	30,043	16	314,833	4,909	2,140	1,947	41	9,037
	10	3,018	794	575	5	4,392	254	84	130	3	471
	11	4,149	1,942	455	4	6,550	5	14	36	7	62
	12	15	1	52	10	78	11	3	18	5	37
	13	103	17	88	6	214	183	5	46	5	239
Total		258,888	81,886	36,836	837	378,482	8,606	3,508	2,695	2,420	17,434
934	0	-	-	-	-	12	-	-	-	-	27
	4	2,717	497	965	6,804	10,983	347	118	3	129	597
	5	1,547	234	765	3,989	6,535	74	12	10	154	250
	6	5,332	1,317	1,638	20,679	28,966	2,201	710	36	501	3,448
	7	48	3	8	44	103	1,096	36	74	1,016	2,222
	8	1,254	88	501	2,852	4,695	12	3	3	29	47
	9	13,448	575	2,207	26,764	42,994	1,196	81	216	5,014	6,507
	10	1,059	33	154	2,289	3,535	458	5	4	162	629
	11	92	6	59	284	441	0	0	0	0	0
	12	1	0	0	0	1	2	1	1	4	8
	13	28	0	3	32	63	15	0	1	26	42
Total		25,526	2,753	6,300	63,737	98,328	5,401	966	348	7,035	13,777

Table H.4: Summary for year 2015 for WIM sites 942 to 964.

WIM site	Vehicle Class	Legal Vehicles					Overloaded Vehicles				
		Lane					Lane				
		1	2	3	4	All	1	2	3	4	All
942	0	-	-	-	-	22	-	-	-	-	33
	4	5,356	396	1,257	11,987	18,996	96	4	16	250	366
	5	2,124	147	598	4,718	7,587	47	6	1	230	284
	6	7,652	539	866	5,148	14,205	150	6	21	296	473
	7	55	4	25	125	209	172	5	627	6,606	7,410
	8	14,385	608	1,381	18,758	35,132	46	1	8	549	604
	9	211,501	9,376	22,743	312,441	556,061	1,691	82	263	11,731	13,767
	10	2,534	116	633	4,257	7,540	673	54	830	8,560	10,117
	11	5,536	158	385	8,901	14,980	1	0	1	152	154
	12	9	1	1	10	21	0	0	1	9	10
	13	57	8	12	194	271	170	3	26	539	738
Total		249,209	11,353	27,901	366,539	655,024	3,046	161	1,794	28,922	33,956
960	0	-	-	-	-	9	-	-	-	-	35
	4	3,240	2,640	0	0	5,880	82	70	0	0	152
	5	2,343	2,291	0	0	4,634	73	61	0	0	134
	6	10,027	9,785	0	0	19,812	348	2,150	0	0	2,498
	7	81	118	0	0	199	1,062	839	0	0	1,901
	8	3,964	3,561	0	0	7,525	21	42	0	0	63
	9	100,843	64,513	0	0	165,356	7,453	46,920	0	0	54,373
	10	7,113	2,458	0	0	9,571	3,610	5,368	0	0	8,978
	11	23	15	0	0	38	3	0	0	0	3
	12	22	4	0	0	26	382	6	0	0	388
	13	132	136	0	0	268	212	158	0	0	370
Total		127,788	85,521	0	0	213,318	13,246	55,614	0	0	68,895
961	0	-	-	-	-	12	-	-	-	-	3
	4	1,453	313	141	725	2,632	193	96	8	6	303
	5	566	263	24	58	911	30	37	0	2	69
	6	1,157	192	178	227	1,754	310	49	4	6	369
	7	12	4	2	22	40	51	3	0	4	58
	8	2,178	518	179	1,048	3,923	68	65	1	4	138
	9	48,211	3,947	5,422	34,209	91,789	6,572	1,521	67	63	8,223
	10	1,009	127	283	1,311	2,730	153	64	7	9	233
	11	951	70	65	741	1,827	63	7	1	0	71
	12	1	0	2	3	6	1	1	0	0	2
	13	60	1	4	97	162	48	2	11	22	83
Total		55,598	5,435	6,300	38,441	105,786	7,489	1,845	99	116	9,552
964	0	-	-	-	-	2	-	-	-	-	3
	4	2,142	356	470	2,431	5,399	67	56	0	4	127
	5	1,019	152	256	962	2,389	10	22	0	0	32
	6	2,363	282	337	1,775	4,757	112	30	0	1	143
	7	33	0	4	14	51	49	3	0	3	55
	8	3,746	514	671	3,856	8,787	27	47	0	1	75
	9	45,169	6,746	7,689	45,771	105,375	1,262	922	9	0	2,193
	10	653	78	133	642	1,506	93	49	6	4	152
	11	1,897	296	439	1,983	4,615	14	6	0	0	20
	12	1	0	0	2	3	1	1	0	0	2
	13	7	3	7	30	47	46	2	11	18	77
Total		57,030	8,427	10,006	57,466	132,931	1,681	1,138	26	31	2,879