RELIABILITY EVALUATION OF WEIGHT-IN-MOTION DATA QUANTIFYING TRAFFIC LOAD DEMANDS AND ASSET USAGE FOR NATIONAL HIGHWAY SYSTEM BRIDGES IN GEORGIA TOWARDS DATA DRIVEN ASSET MANAGEMENT

by Alexander Trammell

(Under the Direction of Mi Geum Chorzepa)

ABSTRACT

This thesis aims to analyze the latest Weight-In-Motion (WIM) data available with the Georgia Department of Transportation. The performance and live load capacity of a bridge at its design stage cannot remain constant as time progresses, leading to the importance of measuring traffic live loads imposed on bridges. The research method involves the use of the National Bridge Inventory (NBI), NCHRP Reports, and Python Programming. The study focuses on the examination of live load demands and thus calculates maximum shear forces and moments for eleven WIM sites. These values are compared with current truck assessment models testing their reliability. Results indicate the need of a new truck configuration classified as the Notional Rating Load (NRL), concluded in NCHRP report 575. Additionally, bridge demand-to-capacity ratios are analyzed for future development through the inquiry of traffic load demand and condition reduction assessment. Finally, this thesis concludes with recommendations for future WIM sites.

RELIABILITY EVALUATION OF WEIGHT-IN-MOTION DATA QUANTIFYING TRAFFIC LOAD DEMANDS AND ASSET USAGE FOR NATIONAL HIGHWAY SYSTEM BRIDGES IN GEORGIA TOWARDS DATA DRIVEN ASSEST MANAGEMENT

by

ALEXANDER TRAMMELL

B.S., The University of Georgia, 2019

A Thesis Submitted to the Graduate Faculty of The University of Georgia in Partial Fulfillment of the Requirements for the Degree

MASTER OF SCIENCE

Athens, Georgia

2020

RELIABILITY EVALUATION OF WEIGHT-IN-MOTION DATA QUANTIFYING TRAFFIC LOAD DEMANDS AND ASSET USAGE FOR NATIONAL HIGHWAY SYSTEM BRIDGES IN GEORGIA TOWARDS DATA DRIVEN ASSET MANAGEMENT

by

ALEXANDER TRAMMELL

Major Professor: Mi G. Chorzepa

Committee: Stephan A. Durham

Sidney Thompson

Electronic Version Approved:

Ron Walcott

Interim Dean of the Graduate School

The University of Georgia

May 2020

ACKNOWLEDGEMENT

First, I would like to thank God for the opportunity to perform and write this thesis. I want to give a special thanks to my major professor, Dr. Mi Chorzepa, for all her hard work and dedication in helping me during this research project as I could not be more thankful for her support. Additionally, I want to thank my other two committee members, Dr. Stephan Durham and Dr. Sidney Thompson, for their guidance and helpful criticism. I also want to thank all the funding I received during this project from the School of Environmental, Civil, Agricultural, and Mechanical Engineering for the teaching assistantships during the Fall 2019 and Spring 2020 semesters as well as the Summer research funding (RP 17-28). Additionally, I would like to thank the Georgia Department of Transportation for their support and effort towards this thesis. I would like to thank a few colleagues, Abuzar Turabi and Hiwa Hamid, for their contributions in assisting me throughout this year. Finally, I want to thank my family for their constant support throughout my life and for pushing me to be the best of my abilities as I would not be where I am today without them.

Table of Contents

LIST OF TABLES

LIST OF FIGURES

1 INTRODUCTION

1.1 Background

There are over ten thousand bridges located in the state of Georgia that are used daily to get its ten million residents to their required destinations. The majority of these bridge structures were built in the prime of the transportation hub with the creation of paved highways and roads in the 1940s and 50s. The average life span of bridges typically last approximately 50 to 75 years as many factors cause the bridge and its elements to deteriorate to a point of functional obsolescence. Of these factors, the live load demand on a bridge is one of the most influential as traffic is continually moving along its span. This produces a dynamic magnification of stresses and deflections reducing its overall strength.

Weigh-In-Motion (WIM) sites managed by the Georgia Department of Transportation (GDOT) were investigated in order to better understand the load demand of Georgia's bridges. The WIM sites captured and recorded vehicle weight, class, axles, spacing, and etc. on both sides of the highway. The data recorded from the sites was correlated with bridges in close proximity. Two points of view from this WIM dataset were extracted including the daily count of vehicles and the load produced by the vehicles. With these two types of analysis, the load demand was determined and combined with condition ratings provided by the National Bridge Inventory (NBI). This demand-to-capacity analysis informs GDOT of which bridge to focus on next to predict its overstressed elements to rehabilitate or replace them beforehand.

The count of traffic is typically defined as average daily traffic (ADT) and average daily truck traffic (ADTT) for bridge design and are provided by the NBI for maintenance. The NBI is a domestic system that documents all the details of each bridge across the nation. These details include hundreds of items such as location, span length, average daily traffic, and much more. Inspections are bi-annually executed to provide ratings for these elements which can vary depending on the inspector. Due to such a large quantity of bridges in the country of approximately 116,000, the traffic data in this set is not always updated appropriately. In addition, many of the values found in the inventory appeared to be inaccurate estimates based upon the initial design state of each bridge. Therefore, some of this NBI data can be misleading as it may not be the true representation of what each bridge undergoes. Therefore, the WIM data provided a good test that compared its ADT and ADTT with NBI assessing its reliability.

As for truck load demand, GDOT follows a specific design and bridge evaluation method in order to determine the reliability of bridges. Through the use of WIM, the load of every truck was analyzed for shear and moment. This provided a representation of what is expected across specific routes and was compared to the standard configurations tested during a load-rating evaluation process determining their reliability. Additionally, a relatively new truck configuration was evaluated and compared to the Georgia state/legal loads as a better representation for examination. This along with the traffic counts provided by WIM embodied the load demand of Georgia bridges.

To fully understand and predict the capacity of a bridge, its condition was studied. The condition of a bridge is based upon inspection ratings that rank the overall bridge as well as its main components of deck, superstructure, and substructure. In evaluating bridge conditions for traffic, the deck condition is most essential as the deck is in direct contact with the vehicles crossing over it. The deck rating is provided by the NBI database and was compared with the traffic counts of WIM to assess the change in condition of a bridge over time based on ADTT. Due to the recent usage of WIM data and lack of accurate systems in its preliminary stages, only two years of ADTT data was utilized at this moment. Therefore, once more WIM data is gathered and more sites produced over the coming years, a more accurate prediction can be generated.

WIM data was utilized further to create state specific truck data as the sites captured trucks on the major routes within Georgia throughout the entire year. Currently, truck data being applied and used throughout the nation is based on a 2-week study in Ontario, Canada in the 1970s. This information is outdated and traffic in Georgia differs vastly from Ontario due to geographic and culture differences creating the need of new truck data. With this in mind, specific details within the Ontario data was compared with WIM data to see how much the two datasets were different. These details included the following probability and side-by-side probability of trucks on a bridge span. This information was helpful in determining the likelihood of multiple trucks being on the same bridge span to be used for calculating the maximum shear or moment being applied.

With the addition of WIM technology, a true understanding of vehicles on Georgia roads and bridges was analyzed properly. It classified the load demand on major routes through ADT/ADTT and provided truck weights in determining the reliability of GDOT condition assessment of bridge evaluation. The deck condition ratings can be predicted when combined with ADTT and used to determine which bridges GDOT should invest on first. Finally, through the calculation of the following probability and side-by-side probability, Georgia truck data was compared to the Ontario data determining their differences.

1.2 Literature Review

1.2.1 National Bridge Inventory

After the collapse of the Silver Bridge in 1967, inspections of three main bridge components became necessary for public safety. With mandated inspections, ratings of each component range from 0 to 9. These condition ratings deal with describing the “physical deterioration due to environmental effects and traffic” (Dunker, Rabbatt 1995). A rating of 9 represents a component with a high-quality condition while a rating of 0 indicates a component needing complete rehabilitation. This information on all public bridges openly became organized in the national bridge inventory (NBI) in 1983. Table 1 indicates a narrative of each condition rating described in the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges.

Table 1 – NBI Rating with Description.

Code	Description
N	Not Applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place
4	Meets minimum tolerable limits to be left in place
3	Basically intolerable, requiring high priority of corrective action
2	Basically intolerable, requiring high priority of replacement
1	This value of rating code not used
0	Bridge closed

Highway bridges that meet the standards of being displayed in the NBI are required to be at least 20 feet (6.1 meters) in span length {Dekelbab, 2008 #14}. The ratings for this inventory are classified as Good (G), Fair (F), or Poor (P). For a bridge component to be classified as good, it must range between the values of 7 and 9. If the values are either 5 or 6 then it can be deemed fair, and if the values are less than or equal to 4, a classification of poor is given. The three main condition ratings that symbolize the overall quality of a bridge are its deck (NBI Item 58), superstructure (NBI Item 59), and substructure (NBI Item 60).

A bridge component is considered to be structurally deficient (SD) whenever it produces a rating of 4 or below for the three main condition ratings of deck, superstructure, or substructure. It can additionally be considered SD if the structural condition rating is 2 or less. Whenever a bridge component undergoes routine maintenance, rehabilitation, replacement or other factors contributing to improvement of a component's condition, it is referred to as OI or observed improvement {Dekelbab, 2008 #14}. OI advancements should be carefully monitored especially when dealing with deterioration rates. Human error and negligence act a part in this, however, as some OIs are not recorded leading to incorrect data being displayed.

This NBI index provides over one hundred data areas detailing the entirety of each bridge. For this study, only certain information sets were analyzed which includes the following items in Table 2.

Table 2 – NBI Item Details.

Item Number	Name	Item Number	Name
28A&B	Lanes on Structure	59	Superstructure
29	Average Daily Traffic	60	Substructure
30	Year of ADT	62	Culverts
43A&B	Structure Type	67	Structural Evaluation
45	Number of Spans in Main Unit	106	Year Reconstructed
48	Length of Maximum Span	107	Deck Structure Type
49	Structure Length	108A&C	Wearing Surface / Protective System
52	Deck Width	109	Average Daily Truck Traffic
58	Deck		

NBI has provided useful information for multiple different programs over the years but mainly have been utilized by the Federal Highway Administration (FHWA) itself and the National Bridge Inventory Study Foundation (NBISF). The FHWA typically focuses on using NBI for congressional reports that document current bridge conditions and trends while the second use emphases on improvement and replacement funding of bridges through the Highway Bridge Replacement and Rehabilitation Program. Many of these bridges are mainly ordered based upon a health index rated from 0 to 100 with a value of 100 representing an entirely sufficient bridge. Funds are then distributed to the states according to their amount of insufficient bridges with the states having the most insufficient bridges receiving more funding {Stam, 2006 #17}. The data recorded within the NBI dataset is biannually updated by each state's department of transportation. This allows the state to correctly receive its necessary funding to create projects for bridges in need of either replacement or rehabilitation.

1.2.2 Weigh-In-Motion

In 2007, the I-35W Bridge in Minneapolis, MN collapsed killing 13 people and injuring more than 100. This bridge experienced a large increase of average daily traffic (ADT) over its life span, as it became one of the busiest bridges in Minnesota. It is led to believe this increase in loading caused such a collapse as its main spans failed during rush hour {Schaper, 2017 #29}. This indicates how crucial monitoring live load demands is for public safety. The usage of WIM systems across the state and nation are continuously gathering such data for this analysis.

1.2.2.1 Benefits

The addition of WIM sites in the state of Georgia provide a variety of benefits to the transportation system. The core benefit of this system is its ability to gather the entire database of what a highway experiences as both cars and trucks pass along its surface. This information is used for studies in pavement and structural fields, enforcement and inspection purposes, and analysis of truck transport practices {Wiegand, 2018 #19}. Additionally, the utilization of WIM systems help eliminate the need of weigh stations for trucks to stop at beside the highway. This saves truck drivers time and money by removing long waits while also reducing environmental hazards. According to the GDOT official website (2020), greenhouse gases have been shown to be condensed by 36 to 67 percent and a 57 percent increase in fuel economy in trucks with the arrival of WIM sites due to the removal of stops at a station. Highway safety is improved as well as fewer lane changes and traffic buildups are present.

1.2.2.2 Potential Sources of Error

However, WIM systems are not perfect and consist of trends classifying potential errors in its system. Whenever the vehicle count of a particular WIM site increases, the percentage of unclassified vehicles increase. As the amount of travel lanes increases in number, the more likely vehicles will change lanes over the sensors producing invalid data. This additionally can lead to changes in speed over the lane switch, increasing the percentage of unclassified vehicles. Accelerating and decelerating over the WIM sensor can cause fluctuations in the vehicles' recorded weight as well. The majority of unclassified vehicles tend to be single tractor-trailers and passenger cars {Li, 2010 #30}. Vehicles going at higher speeds also recorded an increase in axle weight.

1.2.2.3 Site Visit

In order to gain a greater knowledge of how WIM system work, a site visit was made to Gainesville, Florida. The trip was made to spectate the calibration process of a WIM system located on Interstate 75. The entire process takes a couple of days but due to time constraints, the team only viewed the site for a couple of hours and gathered as much information from the workers and system as possible. The team performing the calibration process was Southern Traffic, Inc.

Three trucks of known weights were utilized and drove between the two exits over the WIM sensors gathering its data. This process was tested multiple times to get an accurate result. This averaged weight value for each of the trucks was then compared to the actual known weight of the trucks. With this comparison, a calibration factor was then calculated to be applied to all vehicles crossing over this sensor to accurately record its weight. By viewing this firsthand, I could see how the weights of the same vehicle ranged as it passed over the sensor each time. The values varied each time by hundreds, so the process was not as accurate as I hoped. However, each value was within the tolerance, established by Florida DOT, allowing the system to provide an accurate estimate of its true loading for valuable data.

The system works by having one load cell and two loops within a traffic lane and the sensor and one loop are shown in the Figure 1 below. The two loops were box shaped and recorded the axle spacing of the vehicles passing over them. In-between these two loops is a transverse load cell that reads the weight of each of the axles. All of this data is then transferred over to the control station for managing and distribution shown in Figure 2. The data do not have to be retrieved on site each time as it is sent directly to the Drakewell's server database. This information was provided by Southern Traffic, Inc. workers on site assisting us on our visit.

When a single vehicle passes over each of these sensors, a data row is collected and recorded. This set of information includes site, time, lane number, lane name, vehicle class, temperature, number of axles, axle weights, and axle spacing. With this detailed description, a good representation of each vehicle can be analyzed and used for predictive analysis.

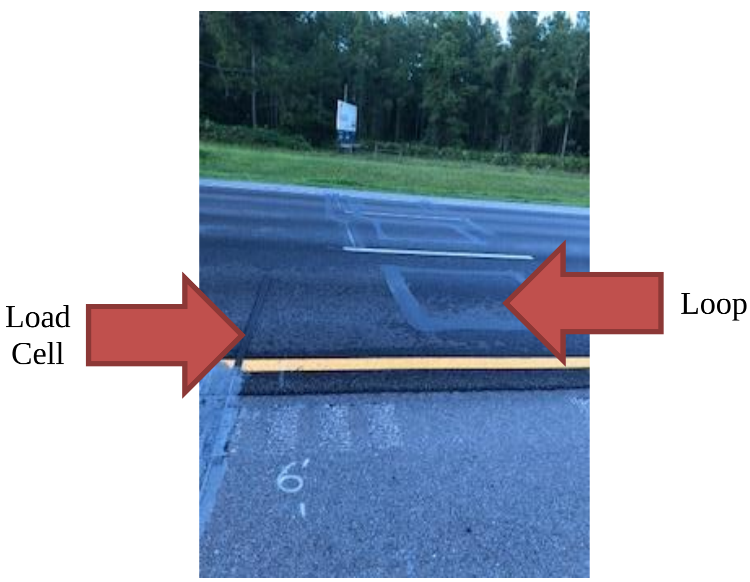


Figure 1 – WIM Street View.

Figure 2 – WIM Control Panel.

1.2.2.4 WIM Locations

Before WIM systems can begin gathering data, the location of the site must be chosen. The location is a significant variable in the information gathered as it should be able to efficiently represent a vast amount of area. Georgia contains 14 WIM stations with the majority of the sites located near its border. Sites must meet specific requirements in order to successfully be considered eligible. Some of these requirements include adequate environmental status, geometric design, and pavement conditions {McCall, #9}. These affect the dynamic behavior of vehicles which decreases accuracy of its static weight. Table 3 displaying site principals is provided below.

Table 3 – WIM Site Principals (McCall, 1997).

	WIM Site Principles
4.1	**Select the site based on the required site design life and accuracy performance level.**
4.2	**Evaluate the geometric design of the location on the following qualities.**
4.2.1	Determine if the horizontal curvature is acceptable.
4.2.2	Determine if the roadway grade is acceptable.
4.2.3	Determine if the cross slope is acceptable.
4.2.4	Determine if the lane is wide enough and marked properly.
4.3	**Determine if the pavement is adequate or if the pavement should be replaced.**
4.4	**Evaluate the site location on the following qualities.**
4.4.1	Determine the availability of access to power and phone.
4.4.2	Determine if there is an adequate location for the controller cabinet.
4.4.3	Determine if the site provides adequate drainage.
4.4.4	Determine the traffic condition at the site.

In order to determine a location based on traffic conditions, an area that has a free flow of traffic and a good sight distance is recommended. Additionally, it is recommended to avoid spots where there is stop and go traffic, slow moving traffic, large quantity of lane changing, and passing on two lane roads. Other constraints to consider for site location not related to traffic are to ensure area has access to power and phone, adequate location for controller cabinet, and drainage.

In addition to assuring the location meets the discussed criteria, high traffic volume spots were considered the leading variable in selecting an efficient WIM site location. However, these high traffic volume spots are no longer the main variable as axle weights do not change from the origin of a trip to its destination {Mahmoudabadi, 2013 #11}. If high volume spots were selected, then multiple trucks would be recorded twice over common routes reducing the efficiency of these sites. Therefore, WIM sites should be placed in areas to maximize the number of checked trucks. Figure 3 illustrates an example of how using high volume spots for WIM sites can be wasteful.

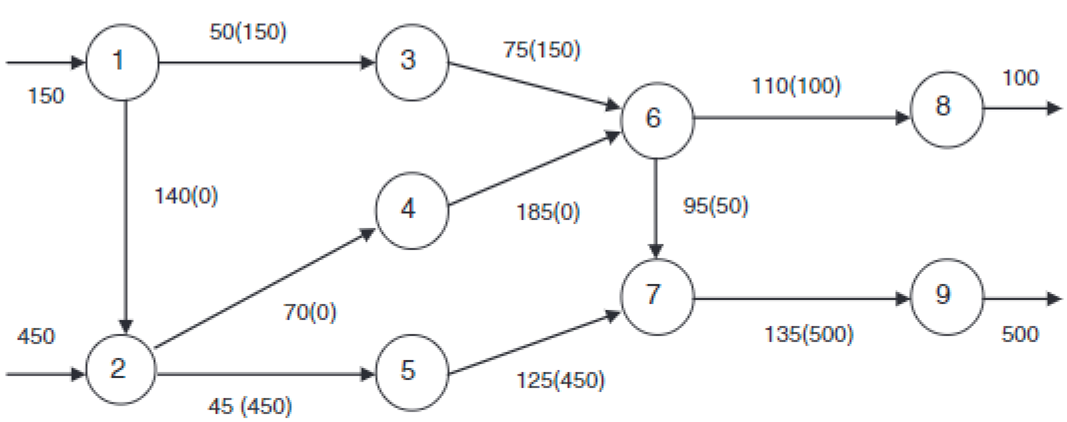


Figure 3 – Road Network Example (Mahmoudabadi, 2013).

WIM sites should be selected by a site providing the maximum number of once-checked trucks and minimum number of unnecessary checks (i.e. already checked). By viewing the GDOT WIM location map shown in Figure 4, the sites chosen match this recommendation as most of the sites are located on transportation routes near edges of state. However, the Northeast portion of the state lacks a relatable WIM site creating a difficult task to correlate bridges in this portion to a certain site. The same issue can be said about the Atlanta location as well as it proves to be difficult to distinguish which WIM site Atlanta bridges should be represented by. The addition of more WIM sites attest to being useful even if it disagrees with the concept to avoid multiple checked vehicles.

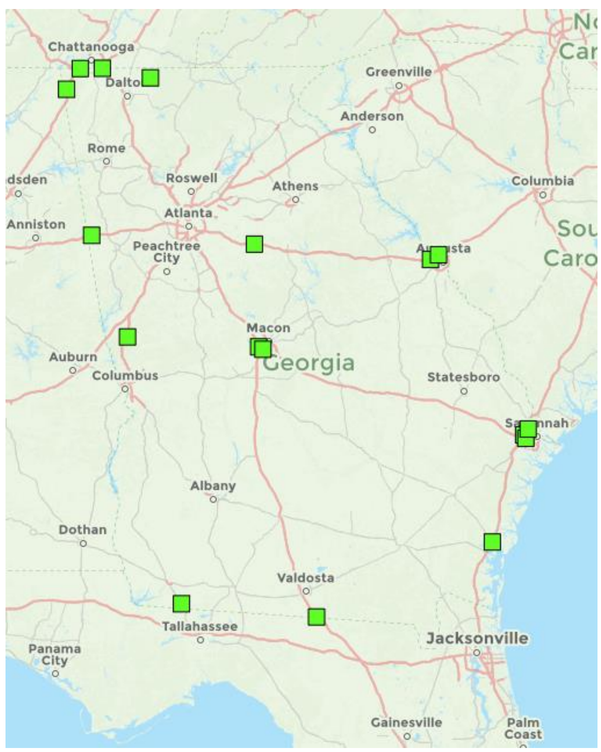


Figure 4 – 2019 Georgia WIM Site Locations.

1.2.2.5 Bridge weigh in motion technology

Bridge weigh-in-motion systems or BWIM is another option that can be explored for gathering traffic data. Instead of a typical WIM sensor load cell placed on a highway, BWIM techniques focus on utilizing instrumented bridges as the weighing scale. This type of system can be expressed using either a bridge or a culvert and consist of a data acquisition system, communication system, power supply system, and sensors {Yu, 2016 #20}. The BWIM is split into two categories of weighing sensors and axle-detecting sensors. The sensors measure the global bending strain of the bridge as a vehicle crosses over it and serves as the input calculation for determining the weight of the vehicle. For this to occur, the sensor should be placed at locations with the most pronounced responses which is typically at mid-span.

These BWIM systems can provide numerous advantages compared to the traditionally WIM systems. The BWIM sensors are more durable as WIM sensors are experiencing direct contact with the vehicles leading to more repairs as the BWIM sensors are located beneath the bridge safe from direct loading. Due to the location of the BWIM sensors, installation is easier as traffic does not have to be stopped like it does for WIM. Additionally, the accuracy of the BWIM system follows a more accurate trend. This is because vehicles only come in contact with the WIM sensors for a few milliseconds which can lead to over or under estimations with a dynamic axle force. The BWIM sensors record an entire time history of the bridge's response helping create an accurate estimation with post-processing. BWIM technologies provide very useful help in recording traffic data and bridge responses.

1.2.3 Georgia Freight and Logistics Research

Georgia truck traffic data was provided to identify truck patterns and volume throughout the state. The database consisted of both continuous counts of data and short duration counts of 48 hours that were extrapolated. The data is from 2009 and classifies trucks as spanning more than 40 feet in length. The database supplied very useful information as to where large amounts of trucks can be seen and what popular routes are taken. Figure 5 below illustrates truck average annual daily traffic (AADT) in the state of Georgia and shows high-volume areas on the major interstates. The highest volume area is located at the intersection of I-285 and I-75 North {GDOT Office of Planning, 2015 #28}.

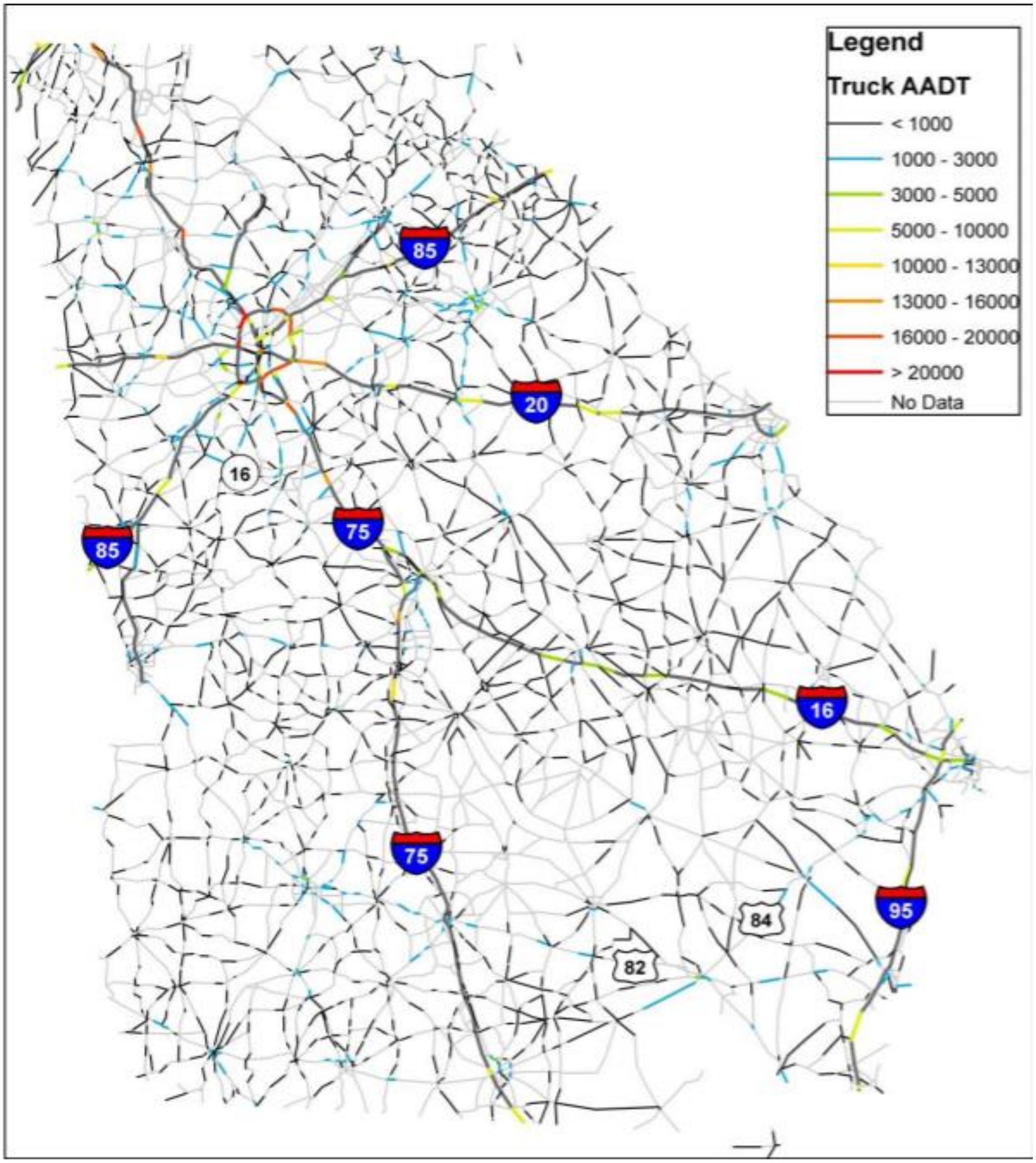


Figure 5 – 2009 Georgia Truck AADT (GDOT, 2015).

As you zoom in closer to just the Atlanta region, as shown in Figure 6, it can be observed that all the intersections with I-285 experience large truck counts of over 20,000 truck AADT. The largest value occurs on I-75 North with a truck AADT of approximately 25,000.

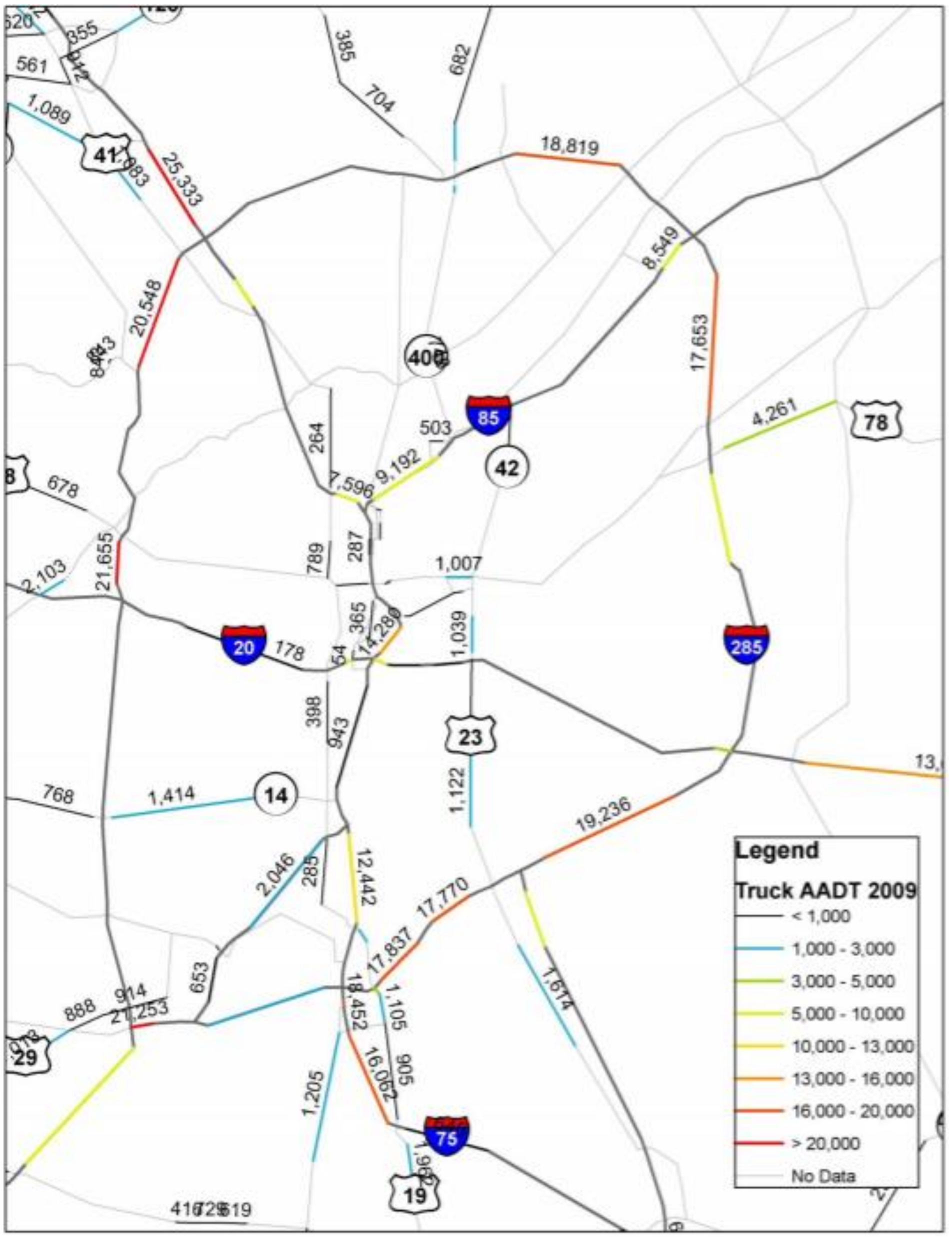


Figure 6 – 2009 Georgia Truck AADT in Metro Atlanta (GDOT, 2015).

The top 50 truck count locations in Georgia are plotted in Figure 7 below with the value 1 being the highest spot of truck traffic count and decreases as the value rises. At quick glance, Atlanta dominates the field but each of the other interstates carry a few of the locations except I-16. This is surprising as Savannah is a key location with such a large port for goods and services. During a survey, it was recorded that around 80 percent of trucks on I-16 were destined for a location within the state {GDOT Office of Planning, 2015 #28}. This indicates that the majority of trucks driving somewhere outside the state travel the I-95 route down to Florida or up to South Carolina. Additionally, I-85 South and I-20 East produced low truck volumes relative to their counterparts.

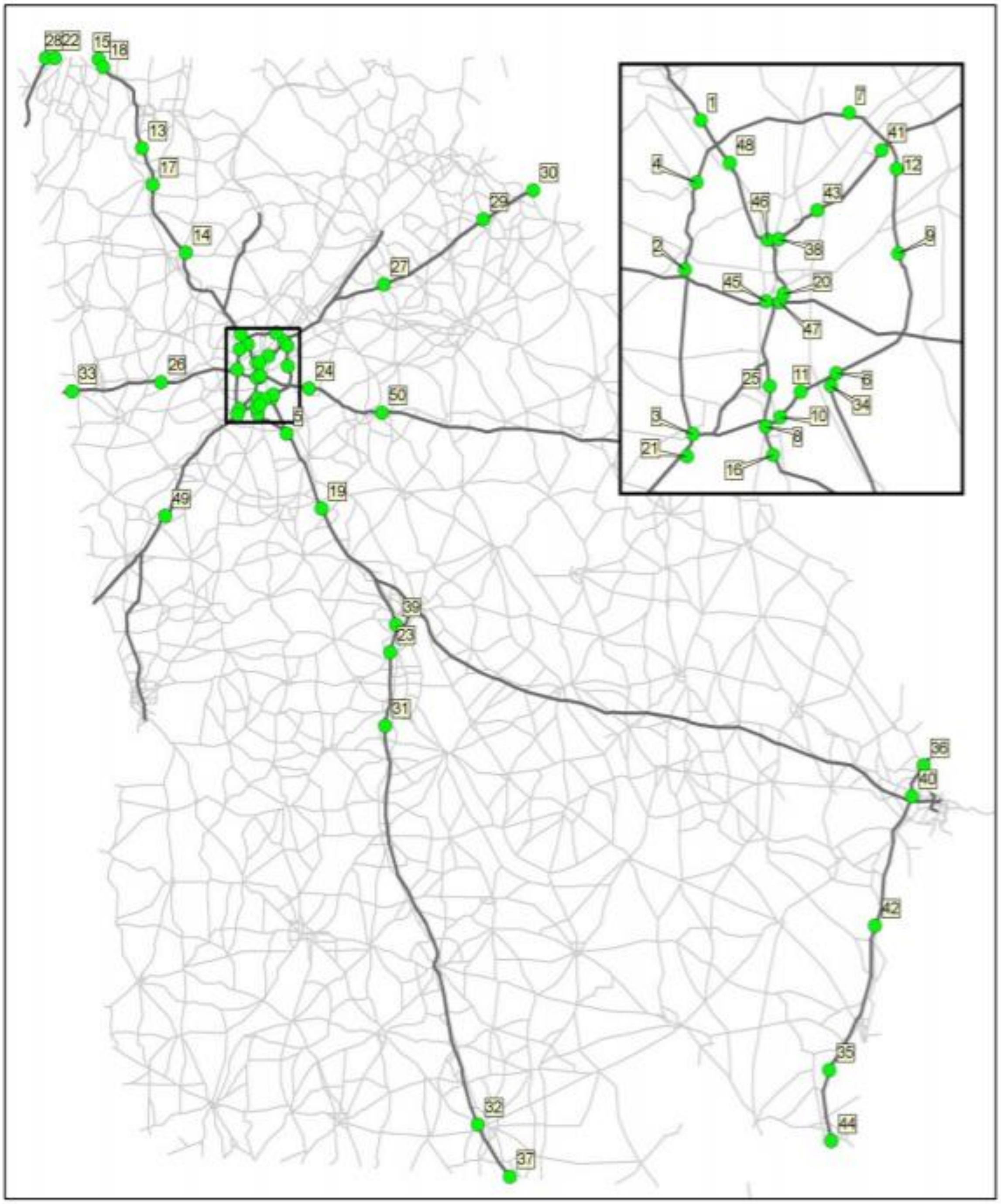


Figure 7 – Top 50 Highest Truck Count Locations in Georgia (GDOT, 2015).

The weights were taken into consideration as well and are illustrated in Figure 8 displaying the inbound and outbound truck tons from 2013. From this figure, it can be seen that two major areas stand out including Atlanta and the Savannah port. I-85 North indicates an important outbound route to the East as the Carolinas are Georgia's 2nd and 3rd top trading partners {GDOT Office of Planning, 2015 #28}. In terms of truck freight, Florida is a top trading partner with Georgia and highlights the significance of I-75 South and I-95 South as these routes carry the majority of truck traffic into Florida.

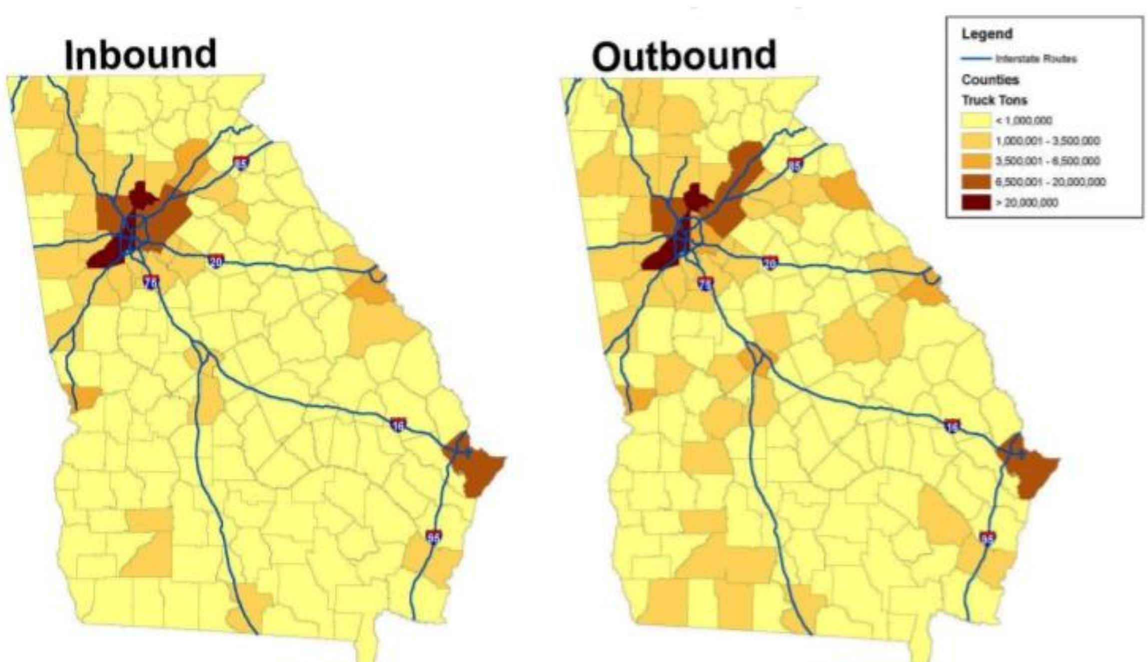


Figure 8 – 2013 Inbound vs Outbound Truck Tons per County (GDOT, 2015).

Figure 9 takes a deeper look into how important the Savannah port is in the transportation industry. It can be seen how far some of its trucks have reached after just 7 days of departure as some trucks have reach across the country up to Idaho and California. According to the Marine Modal Profile, the Savannah port generates 5,000 trucks per day.

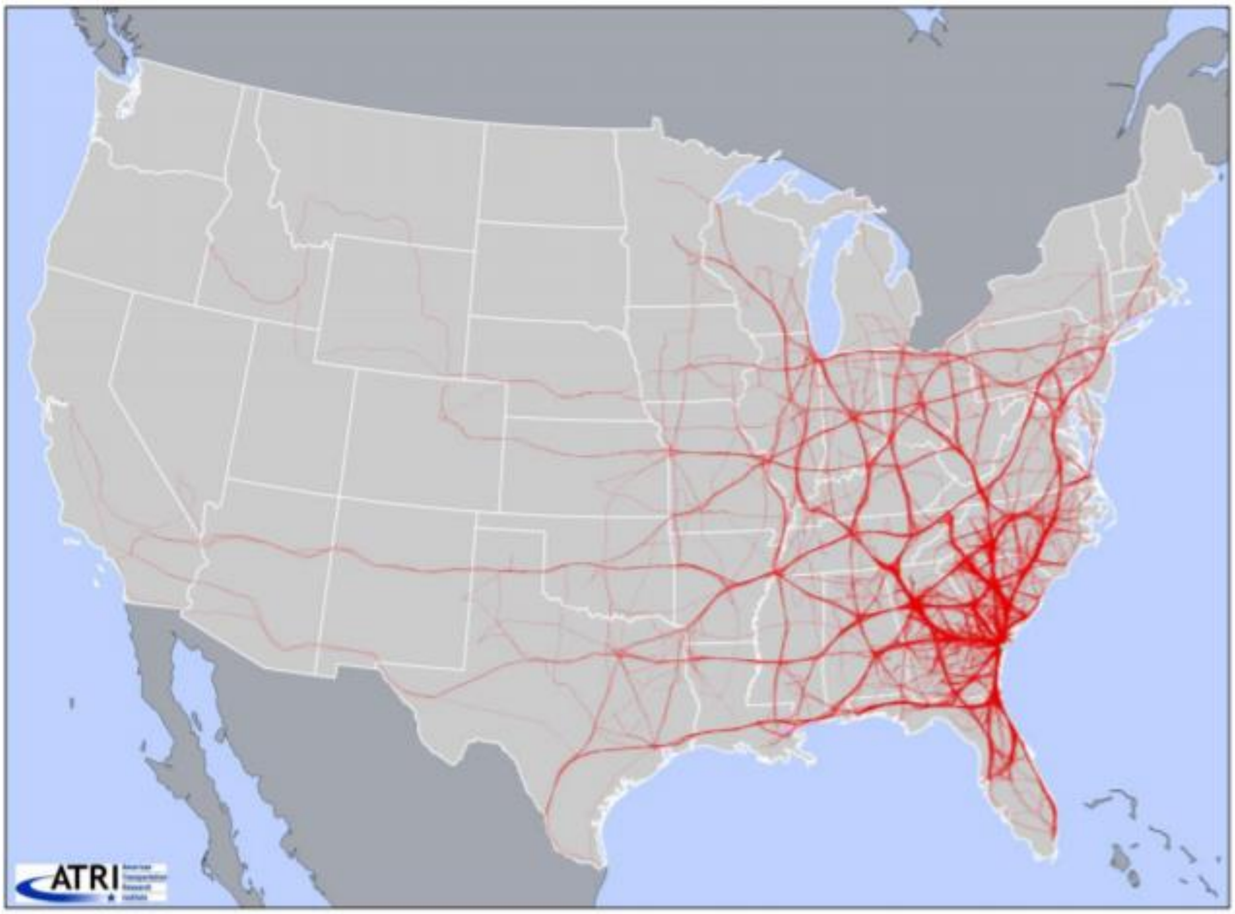


Figure 9 – Truck Patterns One Week from Departing Savannah (GDOT, 2015).

1.2.4 Kruskal-Wallis Test

The Kruskal-Wallis test is a rank-based nonparametric test that is used to determine if a significant difference between two or more independent groups is present based on a continuous or ordinal dependent variable {Statistics, #23}. It provides an alternative method to the one-way ANOVA and is an extension of the Mann-Whitney U test. The Kruskal-Wallis test cannot provide what variable creates a difference between groups but distinguishes if sets of data are indeed different by rejecting a null hypothesis.

Specific assumptions are checked before the performance of this test. The first assumption states that the dependent variable is a measure of ordinal or continuous level. The dependent variable for this study is continuous as it measures the traffic counts for ADT and ADTT recorded throughout the year. The second assumption is that the groups being compared must be categorical, independent groups. This study compares the ADT and ADTT between the NBI and WIM database for the state of Georgia meeting both of these criteria.

The principle of an independence of observations is another assumption and is met in this study for NBI and WIM data. The final assumption relates the shapes of each group's distribution. When the shapes are similar, the medians can be shifted and compared. This assumption does not relate to this study as the mean ranks between the two datasets will be compared.

Once the assumptions are determined and accepted, then the test is conducted to determine the relationship between the datasets. A null hypothesis states that the population means are all equal. This null is usually tested at a 95 percent confidence level indicating a risk 5 percent chance that if the null hypothesis is rejected that the datasets are actually relatable. After the null hypothesis has been stated, it is tested through the calculation of a p-value.

The methodology for calculating a p-value follows the approximation of a distribution of H. A higher H value typically indicates a greater chance of rejecting the null hypothesis. The equation for H is provided below in Equation 1. This distribution of H is found through the use of a chi-square distribution with k-1 degrees of freedom {Minitab, #25}. Equation 2 then defines the process of determining the p-value to be analyzed.

$$\boldsymbol{H = \frac{12\,\Sigma\, n_j(\overline{R_j} - \overline{R})^2}{N(N+1)}} \qquad \textbf{(Eq. 1)}$$

$$\boldsymbol{P - Value = 1 - CDF\,(\chi^2 H, df)} \qquad \textbf{(Eq. 2)}$$

If the resulting p-value is greater than the significance level of 0.05, then the null hypothesis cannot be rejected and the differences between the groups being compared are not statistically significant {Minitab, #24}. This does not determine similarity between the groups as it tests the difference be confirmed. However, if the p-value is less than or equal to the significance level of 0.05, then the differences between the datasets is verified statistically significant. Python programming was utilized throughout this study and was beneficial for calculating this test. A simple command, “stats.kruskal(Dataset 1, Dataset 2)” was applied that provided the p-value for this analysis.

1.2.5 Statistical Analytics

Many approaches are taken in dealing with so much information and determining what steps to take in describing the data accurately. Data mining is implemented to determine trends and correlations between various different categories for the "big" datasets being examined. "Big data is high-volume, high-velocity and high-variety information assets that demand cost-effective, innovative forms of information processing for enhanced insight and decision making" as stated by Gartner IT Glossary. WIM data is grouped into large volumes exceeding one terabyte and is constantly experiencing high velocities meaning it has a high rate of update as vehicles cross over the sensors often. NBI data does not experience high velocities as it is updated annually but does have a large volume set with over 14,000 bridges in the state. Nevertheless, both sets are defined as being "big" data due to their large sizes exceeding one terabyte {Gandomi, 2015 #2}.

Modeling, analysis, and interpretation are needed to correctly process the information given. Predictive analysis is conducted by capturing patterns or trends within the data either from current or historic data. Some methods commonly utilized include moving average based on historic data and linear regression based on current data being one of the most effective choices. In order to maximize time, a small sample size should be taken from the large dataset and tested accordingly before being applied to the entire population. The sample sizes should be examined properly to truly represent the population which includes avoiding heterogeneity as this will create a skewed sample. An example of an inaccurate sample would be taking bridge data from a coastal area while the majority of the population in Georgia is inland.

Other factors to avoid consist of noise accumulation, spurious correlations, and incidental endogeneity {Gandomi, 2015 #2}. Noise accumulation involves the amassing of multiple errors that lead to improper outcomes causing the failure to identify important explanatory variables. An example of this would include the accumulation of class 15, which is an error class used in the data. This class consists of the vehicles recorded by the sensor that were unable to be grouped within a specific class invalidating the data. A big dataset can additionally lead to spurious correlation which is defined as a factor being reasoned as an explanatory variable but in fact is not. Finally, an assumption of incidental endogeneity is applied meaning that the explanatory variables are independent of the residual. This is simplified by stating that the results do not affect the causing variables.

Two approaches should be taken to describe deck performance, which include either using a deterministic method or a probabilistic method. The deterministic approach deals with known outcomes and can be viewed as simply adding two numbers to a sum. This method does not provide detailed results as compared to its counter method. The probabilistic method deals with uncertain values and uses statistical terms such as mean to describe them. This method can be divided into state-based and time-based views. State-based views predict probability that bridge deck will change conditions during a fixed time frame. The time-based view predicts the time distribution it takes the bridge deck to change conditions.

1.2.6 Deterioration Types

Deterioration of a bridge deck can be distinguished by multiple different types. By looking more in depth at these types of deterioration, the factors leading to these types will be more easily understood. Six factors are considered as being the main types of deterioration viewed in bridge decks focused primarily on reinforced concrete decks. These types include cracking, spalling, efflorescence, honeycombing, breakage, and corrosion of rebar {Huang, 2010 #16}. These types can be caused by either physical or chemical attacks.

Cracking is the most known default in concrete and is visually seen. Cracking is significant as it can accelerate more issues such as efflorescence and corrosion. Cracking can be caused by multiple aspects with some including rainfall, expansion joints, live load, and number of lanes. Spalling occurs when the steel rebar within the concrete rusts increasing the cracking in concrete. For this to occur, water intrusion is necessary, so rainfall and humidity are factors that can lead to an increase in spalling. The distance located from the coast can be another factor.

Efflorescence is the process of the relocation of salt to the surface of the material and can be viewed as the white powdery constituent on the concrete surface. Similar to spalling, efflorescence is correlated with the occurrence of cracking. Rainfall and even the soil profile are factors leading to efflorescence. Honeycombing is the rough pitted surfaces or voids in concrete. This is usually the result of poor construction practices due to poor compaction or consolidation. This is not as common as the other factors and forms after the construction phase because of rainfall.

Breakage is one of the most common types of deterioration experienced in bridge decks. It is simply the breaking and separation of concrete. It is the easiest type to notice for inspectors and can be the most worrisome type from the public perspective. It is primarily caused by traffic volume but can additionally be a result of an increase in rainfall intensity. Corrosion of rebar is be a vital flaw in bridge elements as the rebar is necessary to provide the tensile strength of the beam and the rebar will deteriorate if corrosion sets in. The main two factors of this are traffic volume and rainfall. However, this is additionally affected by chemical reactions.

Every bridge undergoes depreciation throughout its lifespan as traffic and live loads play a vital role in its decline. Therefore, traffic should be analyzed in predicting future deterioration in bridges. The use of NBI and WIM can be allocated to perform such a task once enough data is gathered. This led to the creation of relationships between elements and ratings. Deck scores are focused on because it experiences direct contact with traffic. However, many other aspects should be considered when looking into deterioration of Georgia bridges.

1.2.7 Deterioration Factors

The use of WIM and NBI data to determine how traffic usage affects Georgia bridge conditions is very beneficial for predictive analysis of deteriorating elements. Many different factors play a role in the deterioration of decks including age, traffic, material, and etc. According to a study conducted in New Jersey, the two main contributing factors of bridge deck deterioration were average daily traffic (ADT) and axle counts per day {Lou, 2016 #5}. With this in mind, the data provided from the WIM sites deliver a valuable correlation between traffic loading and deterioration as illustrated in Chapter 4.3.

Deicing is another issue that affects deterioration and should be taken into consideration. However, due to the location of Georgia, it is not as big a factor. However, snow does occur in the northern parts of Georgia requiring the deicing process each winter.

Age is one of the leading causes of deck deterioration as with any material over time. Due to the majority of Georgia bridges being greater than 20 years old (85%) with most Georgia bridges falling between the 50 to 60 years old range (19.5%) as illustrated by LTBP InfoBridge analytics in Figure 10 below, aging infrastructure is a major concern for the state. Therefore, the relationship of the age of each bridge and change in deterioration should be studied.

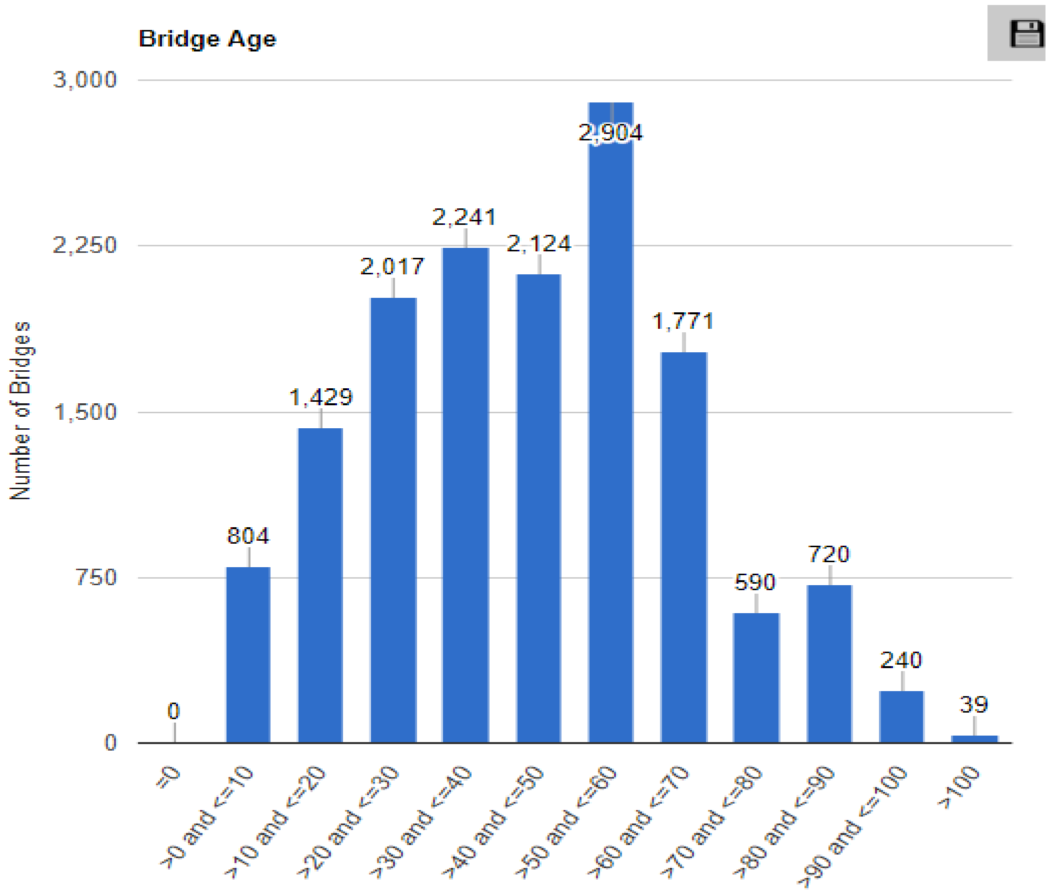


Figure 10 – Georgia Bridge Ages (LTBP InfoBridge, 2019).

Poor design work or construction practices additionally are an influential factor on deterioration rates. If cutting corners to decrease costs or construction errors in production exist, bridge performance will not be maximized.

Weather/climate is another factor that can control deterioration rates. A rapid change of temperature in the heat of hydration stage of concrete can bring about an increase in cracking and deterioration by exposing its reinforcement to the environment. The environment then creates corrosion and other degrading processes weakening the durability of the materials. The location of the bridges should be taken into consideration as well due to its proximity to salt water. Bridges that are close to the ocean cause more deterioration due to the amount of salt in those areas.

1.2.8 GDOT Load Rating Assessment of Existing Bridge Structures

The goal when verifying the structural reliability of a bridge is to determine its load rating and if it passes the necessary requirements to remain structurally sound. This load rating is increased due to traffic patterns or volume to adjust with the continual growth of transportation. Whenever a bridge produces a load rating below the assessment requirements, it is posted for public safety. This means the bridge cannot handle the load capacity of everyday traffic and is examined further to determine what strength capacity it can handle. Approximately, 1,982 out of 8,988 bridges within the jurisdiction of the Georgia Department of Transportation have been stated to require posting as more join the list each year {Ellingwood, 2009 #27}.

The Load and Resistance Factor Rating (LRFR) procedure provides the following Equation 3 for examining a bridge's capacity through the calculation of its load rating. This equation is simply defined as a ratio of the bridge capacity, minus its dead weight, divided by the live load demand. For a bridge to be considered structurally reliable, its rating factor should be greater than 1.0 stating its capacity exceeds the demand it is experiencing.

$$\boldsymbol{RF = \frac{C - \gamma_{CD}DC - \gamma DW \pm \gamma_p P}{\gamma_L LL(1+IM)}} \qquad \textbf{(Eq. 3)}$$

$$\boldsymbol{Where\colon C = \phi\phi_C\phi_S R_n}$$

$$\boldsymbol{\phi_C\phi_S \geq 0.85}$$

The bridge is initially checked for a HL-93 design load as illustrated in Figure 12 below. This design load is classified as being a HL-20 truck combined with a uniform lane load of a 0.64 kips per linear foot force. The HL-20 truck is a series of three vertical forces of 8, 32, and 32 kips applied at a spacing of 14 feet and 14 to 30 feet. Additionally, a 4-foot tandem of 25 kips each is examined with the uniform lane load. If a rating factor smaller than the required 1.0 is calculated, then further checks are necessary before posting. If the rating factor is indeed larger than 1.0 then the bridge is considered structurally reliable and does not need further checks. Each of these situations is displayed in Figure 11 below and will be checked at different span lengths to be compared with Georgia's WIM data for analytic use.

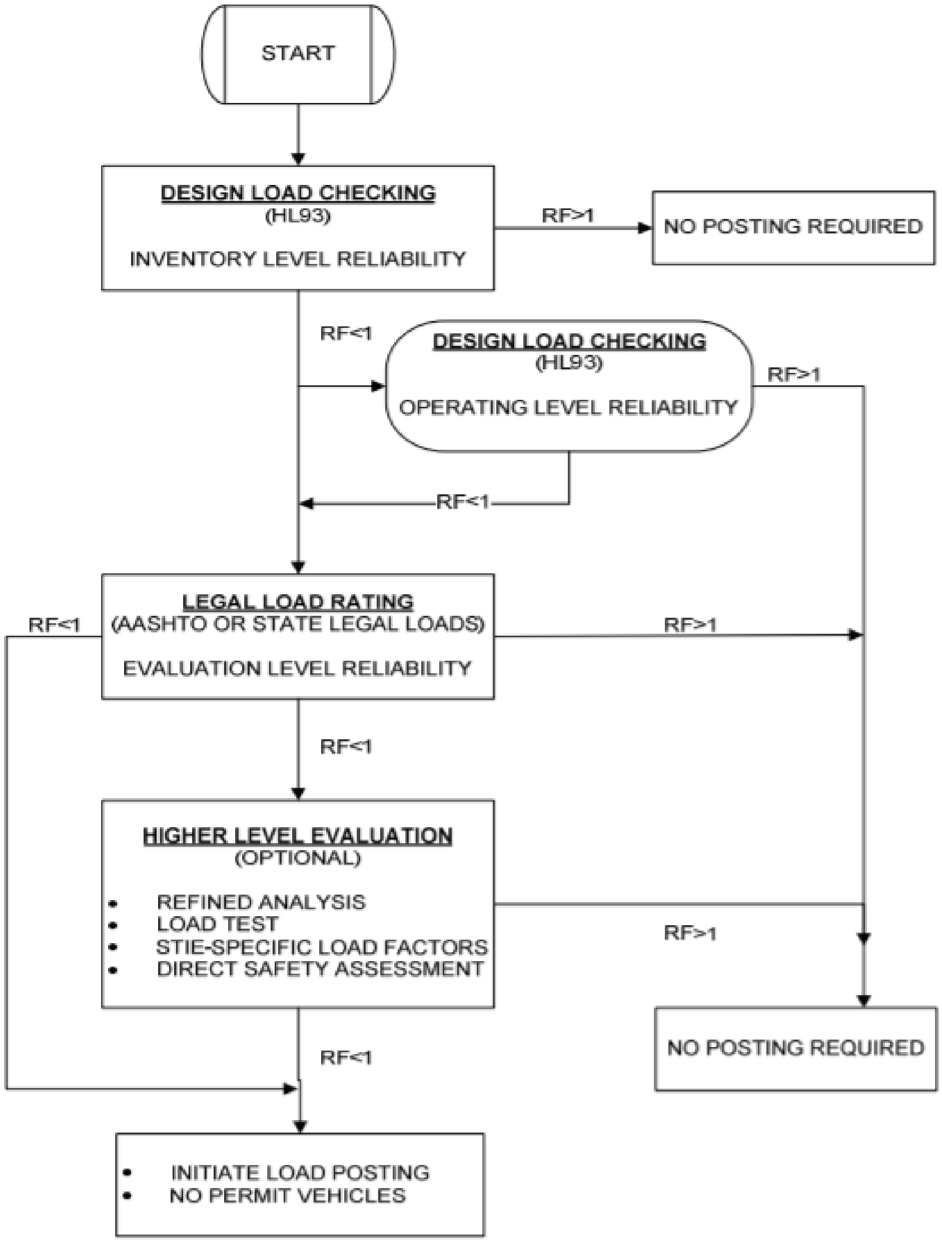


Figure 11 – Bridge Assessment Flowchart (Ellingwood, 2009).

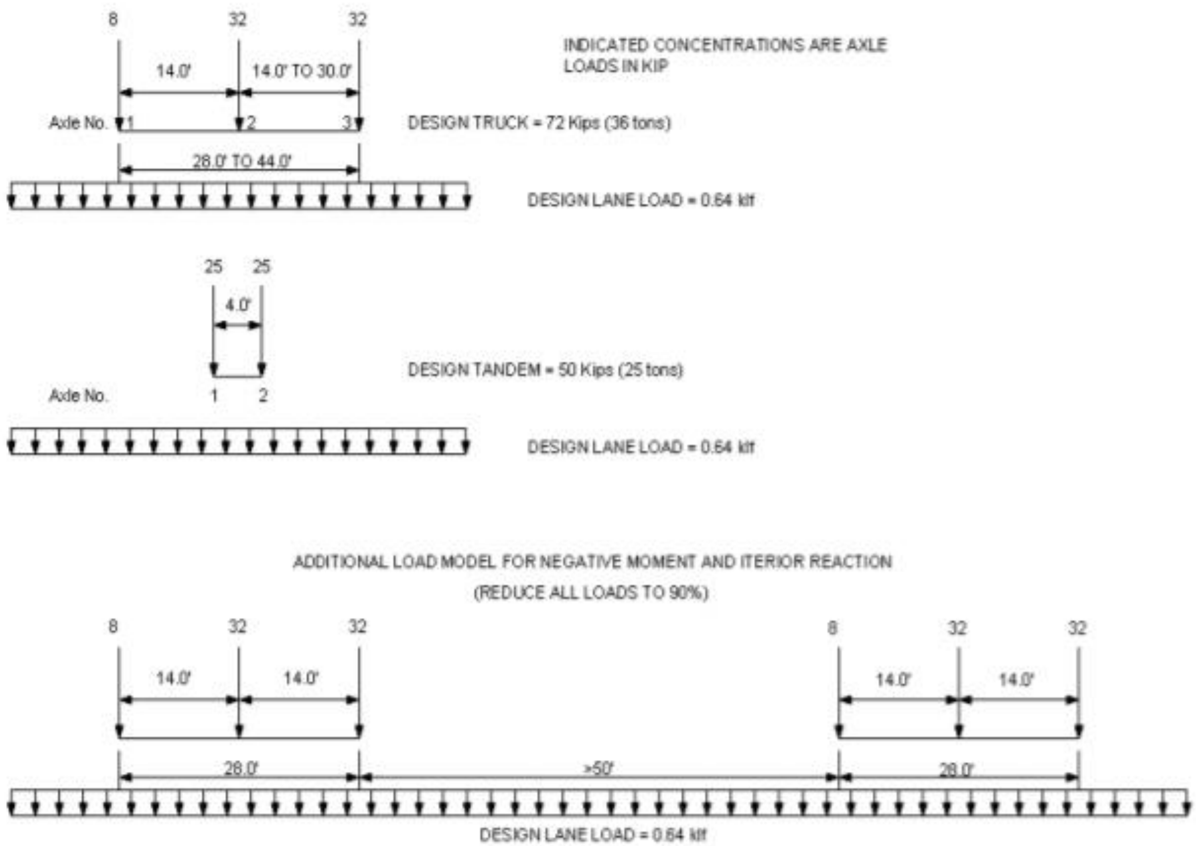


Figure 12 – HL-93 Design Load (Ellingwood, 2009).

If the bridge fails to produce a rating factor above 1.0 for the HL-93 design load test, then it is evaluated using AASHTO/State legal trucks. These trucks follow a specific arrangement based upon the states typical truck types. The arrangements for the state of Georgia are presented in Figure 13 below. If the bridge cannot handle the AASHTO/State legal trucks, then it undergoes the posting process. However, a new truck model with a different configuration has been started to be tested as well and represents new methods used to distribute heavier weights while still meeting the legal conditions. This type of truck called the Notional Rating Load (NRL) and is defined by NCHRP Report 575: Legal Truck Loads and AASHTO Legal Loads for Posting.

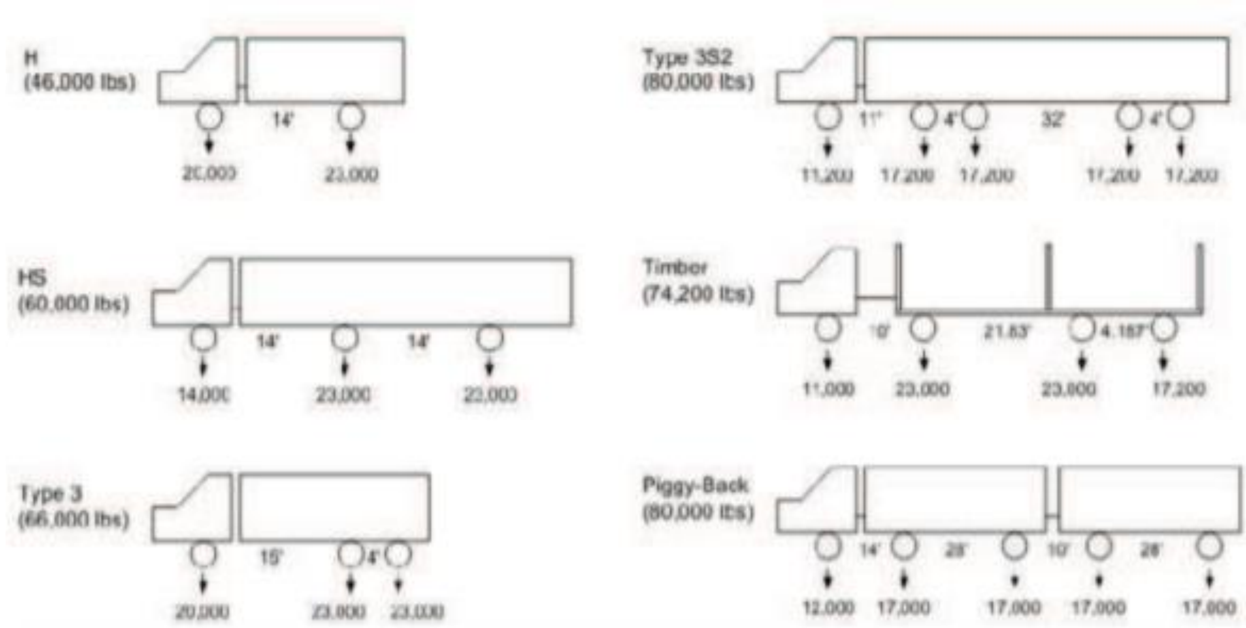


Figure 13 – AASHTO/State Legal Trucks (Ellingwood, 2009).

1.2.9 NCHRP Report 575

The NCHRP Report 575: Legal Truck Loads and AASHTO Legal Loads for Posting discusses the format described in the previous section. This includes the method for evaluating existing bridges through load factoring based upon a HL-93 design load. This design load is clarified again in Figure 14 below. If the bridge is not able to sustain this design load, then it will be tested for the AASHTO state/legal trucks regarding the need for posting.

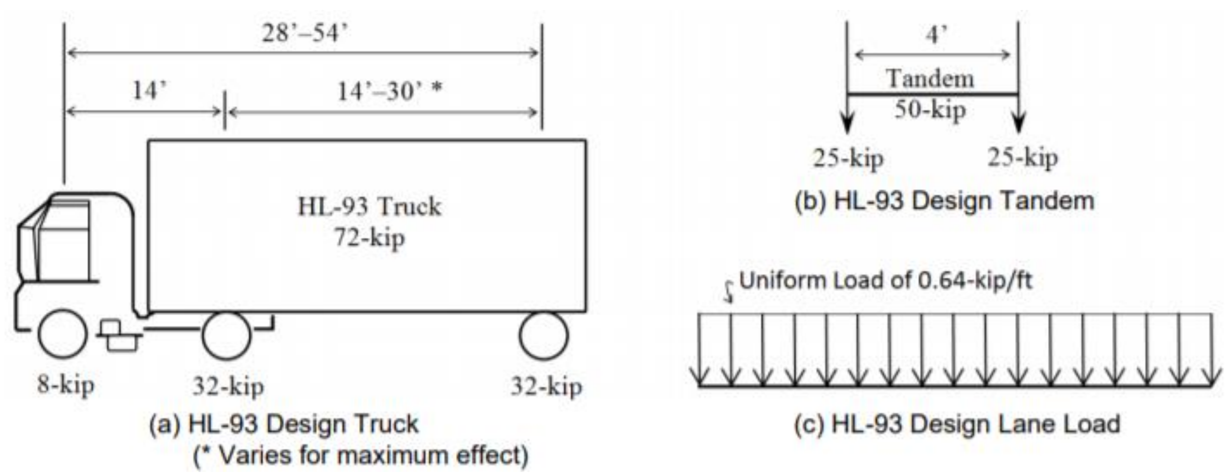


Figure 14 – HL-93 Evaluation Assessment (Waldron, 2010).

According to report 575, there are four basic federal weight limits under the Formula B requirements that are displayed in Table 4 below. The single axle weight is defined as total weight on one or more axles with a center no more than 40 inches apart. The tandem axle weight is explained as total weight of 2 or more consecutive axles with center distance between 40 inches to 96 inches. The maximum gross weight is classified as the entire weight of vehicle or vehicle combination.

Table 4 – Formula B Requirements (NCHRP Report 575, 2007).

Four Basic Federal Weight Limits	
1	Single Axles – 20,000 lbs
2	Tandem Axles – 34,000 lbs
3	Maximum Gross Vehicle Weight – 80,000 lbs
4	Application of the FBF B for each Axle Group up to Maximum Gross Vehicle Weight

Each of the existing bridge tests of HL-93 and AASHTO/State legal trucks meet these federal weight limits and represent their maximum cases. Therefore, if the bridge cannot handle these cases, it will be posted and receive a specific capacity limit. Many special hauling vehicles (SHV) containing dump trucks or construction vehicles have a configuration that includes multiple close spaced axles to maintain the weight limits listed in Table 4. Therefore, a new truck to represent this configuration is illustrated in Figure 15 below and is classified as the Notional Rating Load (NRL). NRL meets the Formula B requirements similar to the other two bridge tests. This report concludes that this NRL load configuration is a more suitable representation of trucks than the AASHTO state/legal loads.

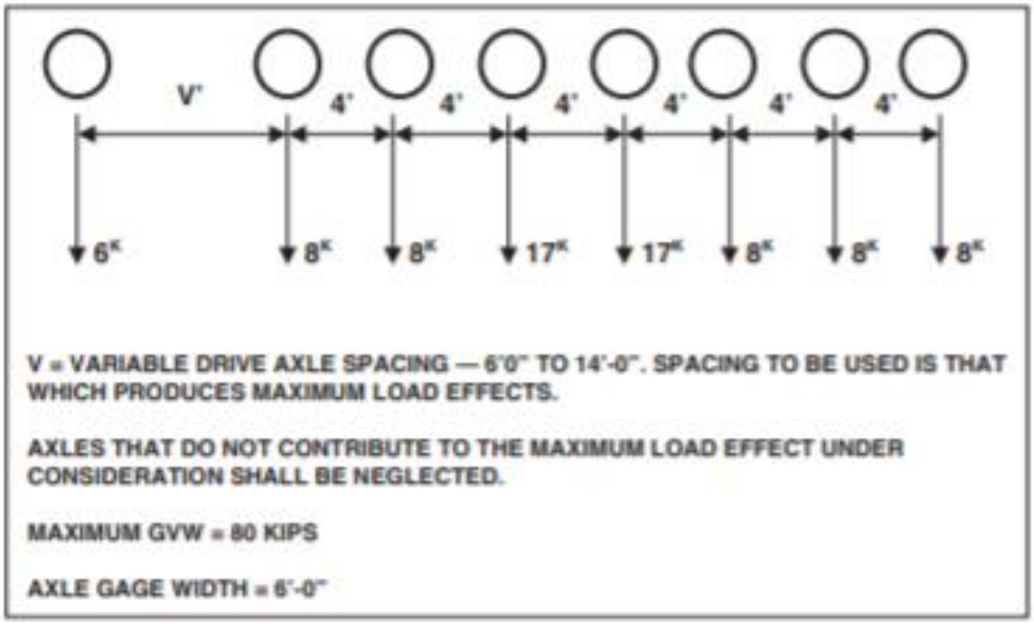


Figure 15 – Notional Rating Load (NCHRP Report 575, 2007).

1.2.10 NCHRP Report 454

The "NCHRP Report 454: Calibration of Load Factors for LRFR Bridge Evaluation" discusses the derivations of live load factors and its methodology of LRFR. When dealing with the evaluation or design of a load, its resistance factor needs to be considered. The load can be considered safe if it does not exceed the resistance. However, if the load indeed surpasses the resistance factor, then the structure will undergo failure. This process can be characterized in Figure 16 below.

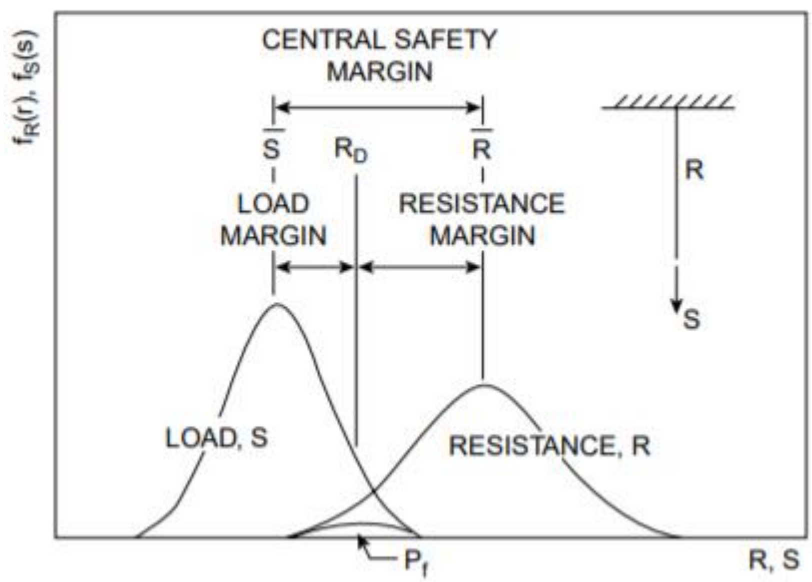


Figure 16 – Load vs Resistance Diagram (NCHRP Report 454, 2001).

The loading is defined by S and the strength or resistance of the material (or member) is defined by R and both distributions represent the uncertainty of their values at inspection. The distribution of S is usually based upon the largest expected load over an appropriate time length as it is currently at the national level according to Ontario data in 1975. As seen in Figure 16, there is an overlap between the loading and resistance. This length varies depending on distribution and safety factor applied and represents the probability of failure. This probability can be determined by integrating over the load frequency distribution as indicated in Equation 4.

$$\boldsymbol{P_f = P[R < S] = \int P[R < s] f_s(s)\, ds} \qquad \textbf{(Eq. 4)}$$

In Equation 4, the P_f represents the probability that failure does occur, and the $f_s(s)$ stands for the load probability according to the loading, s. Therefore, the failure probability is determined through integration or summation of the amount of times R is smaller than s. The reliability is considered to be 1 minus the failure probability. In order to increase its reliability or decrease this failure probability, the overlap of the load should be minimized either through high safety factors or steeper distribution curves. The removal of uncertainty in the curve creates this steeper shape. This shape is represented through its standard deviation or its coefficient of variation.

A time variable should be taken into consideration in these curves causing a shift as seen in Figure 17 which illustrates how the margins between load and resistance reduce. Due to an increase in ADTT and weights over time, the loading curve, S, will shift to the right while the resistance curve, R, shifts to the left. This is due to deterioration in elements of a bridge and creates a higher probability of failure as the bridge ages as well as reducing its structural reliability.

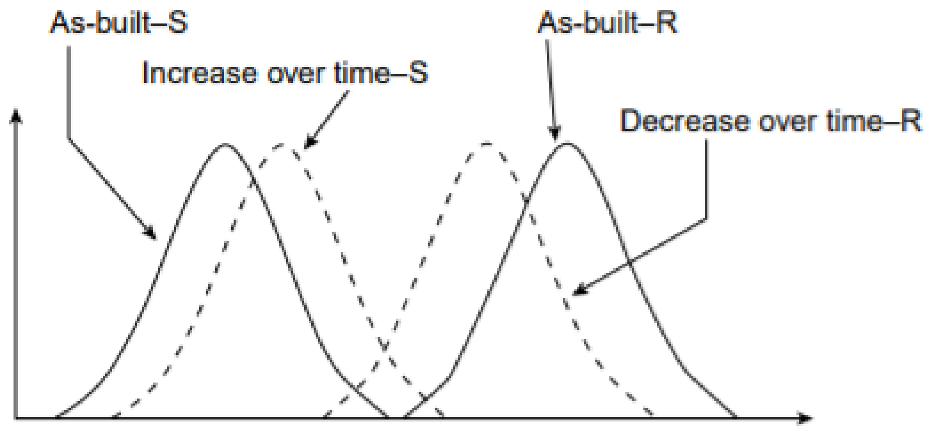


Figure 17 – Load vs Resistance Diagram over Time (NCHRP Report 454, 2001).

As seen above, the reliability factor plays a crucial part in the evaluation process acting as a lead variable in the determination of either bridge replacement, rehabilitation, or repair. Even small changes can lead to costly alterations; therefore, it is recommended in the Manual of Bridge Evaluation to gather site or state specific traffic data. WIM data is a primary option for this matter as its specific data will reduce uncertainties, possibly by steepening its distribution and reducing the failure probability. Its large dataset will provide strong evidence compared to the original Ontario data from 45 years ago and provide the state with a more accurate reliability factor.

According to the LRFR methodology, a component in the design or evaluation process is checked by Equation 5 below.

$$\boldsymbol{\phi R_n = \gamma_d D + \gamma_L L_n} \quad \textbf{(Eq. 5)}$$

The γ_L variable represents the live load factor and is the main focus of this study as WIM data is utilized for this calculation. It is multiplied by the L_n value which is characterized by a nominal live loading effect prescribed by a load model or in this case by WIM data. This live loading formation added with the dead load factor, γ_D, coupled with the dead load effect, D, combine to form the nominal component resistance, R_n. This resistance is multiplied by a resistance factor, ϕ, for safety precautions.

For evaluation purposes, the nominal resistance is determined from the inspection data and rating factor, R.F. This rating is simply multiplied by the loading term, L_n, and is calculated according to Equation 6.

$$\boldsymbol{R.F.} = \frac{\boldsymbol{\phi R_n - \gamma_d D}}{\boldsymbol{\gamma_L L_n}} \qquad \textbf{(Eq. 6)}$$

Truck traffic is the controlling element in bridge analysis as its massive size and weights influence the structural ability of bridge components. However, truck traffic is difficult to collect as its movement are varied according to politics, economy, region, technology, seasons, and weather. The AASHTO LRFR specifications uses heavy truck data according to information gathered from Ontario, Canada. This information was obtained in 1975 and includes data from a 2-week span through the process of weighing stations.

The Ontario data was gathered around 45 years and is still used today. WIM technology is an alternative method that should be used to create state specific truck data. This is only allowed if acceptable WIM technology is utilized and not including a heavy load of trucks being able to bypass the system. This would include the system producing a large amount of class 15 (unclassified) vehicles. It is additionally recommended to extract the permit vehicles in determining the truck distribution and calculating the maximum loading effects for the legal loading. A procedure for evaluating the live load factors is given in the Evaluation Manual and follows a format that focuses on the largest 20 percent of truck data available.

1.2.11 Manual for Bridge Evaluation

The calculation for a live load factor using the LRFR method for a WIM site is displayed in Equation 7 below. This is according to the AASHTO Manual of Bridge Evaluation, which allows for the reliability to be tested without dead load being taken into consideration. This is because of the proportion created by comparing the factor to a referenced live load factor.

$$\frac{\gamma_L LE_n}{\overline{LE}} = \frac{\gamma_{L,ref}\, LE_{n,ref}}{\overline{LE}_{ref}} \qquad \textbf{(Eq. 7)}$$

The left-hand side of Equation 7 represents the Georgia data while the right-hand side is in accordance to an existing case as a reference. The γ_L or live load factor for Georgia is the variable to be determined of Equation 7 and is rearranged into Equation 8. The $\gamma_{L\ ref}$ in Equation 7 is a known live load factor from the referenced source. The γ_L and $\gamma_{L\ ref}$ have to be representing the same legal load rating, span length, and bending moment. Continuing, the LE_n is the nominal load effect which will represent a bridge's spatial maximum moment for a specific span according to Georgia trucks while the $LE_{n\ ref}$ is for the referenced circumstance. $\overline{LE}$ and $\overline{LE}_{ref}$ represent the average of the maximum live load effect extrapolated over the next 5-years for loads of interest. The 5-year extrapolation can be predicted through the assumption that the tail end of the maximum load effect histogram approaches a Gumbel distribution and follows the assumption that the information is sufficient to represent a long period of time.

This 5-year extrapolation is not included in this study as the change in the live load factor will be represented by the difference in shear or moment calculations determined. Therefore, the equation is broken down into a simple proportion to determine the worst-case scenarios a bridge experiences within the annual time frame of the WIM data. Equation 8 below illustrates how the maximum live load factor for a specific bridge can be calculated by referencing a HL-93 truck driving over it through the use of influence lines.

$$\boldsymbol{\gamma_L} = \frac{(\gamma_{L,HL-93})(LE)}{LE_{HL-93}} \qquad \textbf{(Eq. 8)}$$

The generalized routine traffic live load factors are found in Figure 18 below collected from the Manual of Bridge Evaluation. These values are utilized to determine the referenced or HL-93 truck's live load factor for Equation 8. The ADTT of the site will determine the factor and linear interpolation is allowed for the ADTT values between 1,000 and 5,000.

Traffic Volume (One direction)	Load Factor
Unknown	1.45
$ADTT \geq 5{,}000$	1.45
$ADTT \leq 1{,}000$	1.30

Figure 18 – Live Load Factor per ADTT for Generalized Routine Traffic (AASHTO, 2018).

The Manual for Bridge Evaluation additionally discusses the requirement of NRL truck according to the NCHRP Report 575 recommendations. It mandates the testing of this model to act as a screening load for rating bridges. Therefore, it is classified as a conservative load confirming the few SHV and overweight trucks generating high stress.

1.2.12 Influence Lines

In order to efficiently determine the shear and moment that are experienced as each vehicle moves across a bridge's span, influence lines are often utilized. An influence line is defined as being a factor that represents the variation of the function being examined for any given point on a structure through the application of a unit load {Fanous, 2020 #26}. This process is presented in Figure 19 below. It can act for a shear force, axial force, or bending moment but cannot act as any two at the same time meaning each function is calculated separately.

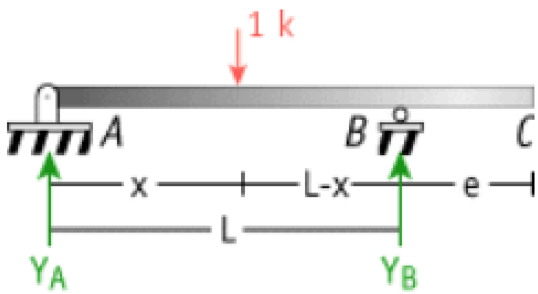


Figure 19 – Influence Lines Example (Fanous, 2020).

By calculating the function of a unit load across several locations along the span of the bridge, a list of factors is collected to represent any point load being applied to the structure. This is done by taking a dynamic load in a vehicle and making it static as it is treated in iterations as it moves. Maximum shear is typically calculated near the two end supports of a beam while the maximum moment will most likely occur at the midpoint of the bridge span but can vary slightly depending on the vehicle moving across it.

1.2.13 NCHRP Report 368

The live load is a crucial aspect of bridge performance as it covers a range of forces produced by vehicles moving across a bridge. The effect the live load plays on the bridge depends on multiple parameters from vehicle characteristics to bridge dimensions. The truck measurements and loading gathered from the Ontario Ministry of Transportation in 1975 are utilized in this report and represented the U.S. transportation loading. The survey included around 10,000 trucks and is predicted to act for 75 years or lifespan of a bridge even though it will have its uncertainties with such a large prediction. Current WIM data should be utilized for comparison and specific information removing some of these uncertainties.

When focusing on one lane maximum moment and shear force, probabilities of occurrence come into effect as the largest values depend on amount of trucks on span at the same time. This situation (see Figure 20) is classified as a following probability and is determined through the definition of headway distance which is displayed in Figure 20 below. This distance is the length between the last axle of the first truck crossing over the span to the first axle of the second truck. The headway distance decreases drastically when bumper to bumper traffic is present and is approximately 15 feet. This situation should be focused on as it can significantly affect the following probability statistic.

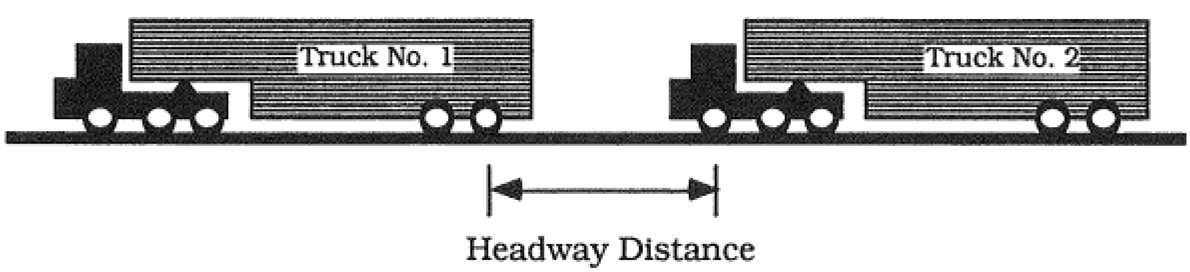


Figure 20 – Headway Distance Diagram (NCHRP Report 368, 1999).

This topic is additionally defined by correlation of truck weights. The three degrees of correlation include no correlation, partial correlation, and full correlation and are plotted in Table 5. A fully correlated value means the two trucks have similar characteristics including weight and spacing while a zero correlation value means the two trucks lack comparable features. Through a study conducted by Nowak, Nassif, and DeFrain, it was stated that about every 50th truck is followed by another truck on a 100-foot headway distance. Additionally, it states that about every 150th truck is followed by a partially correlated truck, and every 500th truck is followed by a fully correlated truck. However, this information is according to limited data leading to another use of WIM data for site specific values removing uncertainties and lack of extent of data.

Table 5 – Truck Probabilities (NCHRP Report 368, 1999).

Truck Probabilities			
Type	Degree	Following Probability	Side by Side Probability
No Correlation	P = 0.0	2.00%	6.67 %
Partial Correlation	P = 0.5	0.67%	10.00%
Full Correlation	P = 1.0	0.20%	3.33%

The degrees of correlation work for two lane moment and shear as well and is defined as side-by-side probability. It was determined that every 15th truck was on a bridge simultaneously with another truck and that every 10th truck was partially correlated at side-by-side probability. A fully correlated truck occurred side-by-side one out of every 30th truck. The probabilities are listed above in Table 5.

1.2.14 Literature Review Discussion

The purpose of this literature review was to provide information beneficial to aiding the contends of live load demand for this study. Initially, the review focused on the details relating to NBI and WIM data as these two sources were the key to the entire analysis. These sections explained the contents of each, and listed components of significance relating to traffic and condition ratings. Advantages as well as potential errors in both were additionally investigated, leading to the first section of this thesis. It tests the reliability of NBI traffic counts (ADT and ADTT) by the use of current WIM data placed along the bridge's routes.

The traffic volume of Georgia was then researched to locate major areas of interest. With this in mind, this study focused on the Savannah region in more detail as it is a controlling location in the state and is home to multiple WIM sites that were analyzed thoroughly. Additionally, I-75 North of Atlanta was further analyzed examining its live loads as it is home to the largest amount of truck traffic in the state. None of the existing WIM sites are located in Atlanta, however, as it is difficult to install one with its buildup of traffic.

Due to the large size of WIM data, a statistical analysis portion was examined in Chapter 1.2.5. This chapter provides helpful methods to analyze big data more efficiently. A Kruskal-Wallis test was needed for the NBI and WIM comparison and, therefore, was studied in Chapter 1.2.6. Its procedure was established as well as how to determine its results.

This thesis focused on how traffic affects the structural integrity of bridges, but other variables need to be considered as they also affect bridge performance. Therefore, different types and factors of bridge deterioration were studied in Chapters 1.2.7 and 1.2.8. WIM and NBI data can be coupled to create correlations between deterioration rates and factors to predict future bridge performance. This will help maximize budget spending and public safety as bridges with the highest risk can be targeted.

With the ability to create a load envelope with WIM data, bridge evaluation was examined. The GDOT Condition Assessment {Ellingwood, 2009 #27} and NCHRP Report 575 provided evidence of how bridges are tested according to their performance. These documents gave details on three loads examined in performance (HL-93 design load, Georgia state/legal loads, and NRL). It additionally classified permit trucks from allowable loads by Formula B limitations. In this study, trucks were separated into two categories according to these Formula B requirements.

NCHRP Report 454 and the Manual for Bridge Evaluation were researched next. These guide reports provided information relating to the purpose of determining load factors and how WIM data is used to assess these factors. The analysis methods were presented. The three testing loads defined in the GDOT Condition Assessment and NCHRP Report 575 are compared with WIM traffic data through load factors. Methods were broken down in Equation 8 illustrating how the WIM data are related to the testing loads. The shear and moment of each WIM site were calculated and compared as it is directly proportional to the load factors represented in Equation 8. Influence lines were then analyzed creating a more efficient method to calculate the moment demands.

NCHRP Report 368 was then analyzed discussing the currently used Ontario truck data and multiple truck probabilities. This Ontario data appeared unrelated to Georgia's traffic in 2020, and through the analysis of WIM data, a more accurate representation was computed. A state/route specific truck dataset should be created to replace the outdated Ontario set. The following probability and side-by-side probability were then studied for comparison to show the difference between the Ontario truck data and Georgia WIM data.

1.3 Problem Statement

Georgia bridges are undergoing more and more traffic each year deteriorating its components and overall strength capacity. Weigh-In-Motion (WIM) data has been placed throughout the state by GDOT to help characterize this increase in traffic volume. However, a use for this data has not be found as WIM sites continually collect large amounts of data. Therefore, this study was conducted to determine a valuable use for this WIM data in classifying Georgia's traffic demand. Multiple factors were analyzed providing insight into why WIM sites within the state need to be increased and assessed further. Asset usage, live load demands for bridge load rating, and the mobility of goods are all essential aspects provided by this WIM data.

The studied aimed to determine a use for the WIM data gathered as GDOT has invested in fourteen stations placed across the state. Research revolved around WIM/NBI history, freight logistics, Georgia bridge load rating procedure, and NCHRP reports. Techniques involved extracting the data into Python Programming for ease in filtering its large dataset and assumed its quality was assured eliminating uncertainties. The reliability of NBI data relating to Georgia bridges was tested through WIM traffic volume providing GDOT with dependable traffic demand. Bridge evaluation and design methods were assessed in comparison with WIM truck data through the creation of a live load envelope calculating maximum shear and moment values. Additionally, the approach for predictive methods in deterioration rates coupled with traffic demand to efficiently manage bridge improvement was provided. Finally, significant locations for future WIM site positioning were recommended to cover major interstate routes in a cost-effective process.

This study provided valuable analysis into WIM data and how it is extremely beneficial to bridge assessment for the state of Georgia. Its detailed database and ability to capture every vehicle assists GDOT with exact information of what their bridges are experiencing every day. Through the addition of even more sites, predictive analysis will become more accurate in helping shape the future of transportation in Georgia.

1.4 Scope of Study and Objectives

The study focused on quantifying live load demands for bridge asset management for GDOT as a result for finding a use for Georgia WIM data. Due to the large amount of bridges within the state, only the National Highway System (NHS) bridges were examined. The NBI database was utilized to characterize the bridges analyzed. The traffic volume of this database appeared falsely estimated especially in the truck traffic section. Therefore, ADT and ADTT values from WIM sites were determined through the use of Python Programming to assess the NBI's reliability. This analysis additionally provided awareness of heavy truck traffic routes within the state for GDOT. Once the traffic volume was studied then the live load demand was evaluated for bridge load rating.

This study reviewed NCHRP Reports (454, 575, 368) and the Georgia Condition Assessment of Bridge Evaluation for live load demand purposes. Truck models including the HL-93, Georgia state/legal loads, and new NRL truck were defined from these papers as representing truck traffic in Georgia and clarifying the need of posting. Therefore, a live load envelope calculating the shear and moment values of Georgia trucks based on the Federal Highway Administration (FHWA) was performed. This analysis was then compared with these truck models evaluating their reliability. Multiple truck probabilities were additionally tested that provided evidence in how Georgia truck data differs from the nationally used Ontario data from 1975, therefore, indicating a need of a state specific dataset centered around WIM. Permit vehicles percentages based on the 2019 WIM data were provided for GDOT exploration as well as recommendations for future WIM locations to maximum performance. On a larger scale, analysis on bridge deterioration rates associated with truck traffic demand was presented leading to improved bridge management that will be expanded on once more WIM information is available.

1.5 Summary of Research Work

This study consists of 8 chapters. Chapter 1 introduces the concept of WIM data and gives the reader the background information being analyzed in this thesis. Additionally, the research conducted to set up this work was provided along with the problem statement and scope of study addressing what is covered in this paper. Chapter 2 discusses the methodology taken to achieve the results including the use of Python Programming and ARC GIS. Chapter 3 focuses on the comparison of NBI and WIM data about the topic of traffic volume. ADT and ADTT values were calculated and an analysis of the results is provided. Chapter 4 gets into the load demand of bridge asset evaluation through the testing of multiple truck model reliabilities through the calculation of shear and moment values. It also provides intel in truck permit percentages, multiple truck probabilities, and deterioration ratings for future analysis. Chapter 5 ties everything together in forming the conclusion of the study. Chapter 6 provides recommendation mainly focusing on locations for future WIM sites. Finally, Chapter 7 and Chapter 8 cover the appendix and references utilized within the study.

2. METHODOLOGY

2.1 Comparative Analysis between NBI and WIM

2.1.1 Introduction

To create an accurate representation of traffic data of bridges in Georgia, WIM systems will be compared with NBI data to test how accurate and correlated the two datasets are. The NBI data lacks up-to-date traffic data and thus is not reliable for studying traffic usage. Some of the NBI bridge traffic data is based on percentage growth delineated from its initial design loading, which has changed drastically over the years.

The WIM sites should provide a higher quality of data as they gather all the live loads moving over the roads at specific sites. However, the WIM sites are only located on major transportation routes in fourteen different locations. The primary issue with so few WIM sites, is trying to associate them to more than ten thousand Georgia bridges. This study focuses on around 500 bridges distinguished by the National Highway System.

After the two sets, NBI and WIM, of data were managed, side-by-side plots were displayed comparing the two while additionally distinguishing the precision of each set. In order to put a specific value on the correlation between the NBI and WIM traffic data, a correlation test was performed. A Kruskal-Wallis test was conducted to check their relationship assuming the populations are not normally distributed.

Additionally, major regions in Georgia were analyzed due to their prime trucking locations. None of the fourteen WIM sites are located in the metro Atlanta area and therefore do not qualify. However, two sites consisting of heavy truck traffic were viewed in a detailed matter including the

Savannah port and Interstate 75 between Atlanta and the Tennessee border. The Savannah port generates over 5,000 trucks a day that spread out to the entire nation {Planning, 2015 #28} while I-75 has the highest recorded truck AADT count over the past few years.

2.1.2 NBI Database

The NBI database is used nationally as a resource for multiple different reasons due to its ability to offer distinct characteristics for every bridge in the United States. This information is updated annually according to each of the state's department of transportation. Georgia uses this database constantly for bridge design and inspections providing valuable information on every bridge. This data set is organized in a structured layout for easy access as shown in Figure 21. Only a small portion of this database is provided as it consists of over one hundred columns of material. This format allowed for simple steps to be taken to find a specific bridge or set of bridges based upon similarities used for this study creating an ease for coding purposes.

STATE_CO	STRUCTUR	RECORD_1	ROUTE_PF	SERVICE_L	ROUTE_NU	DIRECTIOI	HIGHWAY_	COUNTY_(	PLACE_CC	FEATURES	CRITICAL_	FACILITY_(	LOCATION	MIN_VERT	KILOPOIN	BASE_HW	LRS_INV_I	SUBROUTE	LAT_016	LONG_017
13	1GA39	1	8	8	0	0	0	39	0	'CREEK	'	'USS WOO	'SOUTH OI	99.99	0	0			30.79647	-81.5373
13	1GA39	1	6	0	0	0	0	39	0	'CREEK	'	'FORT PET	'600 M N C	99.99	0	0			30.77422	-81.5443
13	1GA49	1	8	0	0	0	0	39	0	'NORTH RIVER		'USS HENR	'90M E. OF	99.99	0	0			30.79842	-81.5335
13	2GA14	1	8	8	0	0	0	95	1052	'DRAINAGE DITCH		'DIRT ACCI	'0.1 KM E (	99.99	0	0			31.55111	-84.0929
13	2GA18	1	8	8	0	0	0	95	1052	'DRAINAGE DITCH		'WEST SHA	'1.0 KM S (	99.99	0	0			31.54764	-84.0951
13	25	1	8	1	27	0	0	53	30760	'BABBITT ROAD		'VICTORY I	'.1 KM NE	4.27	0	0			32.35669	-84.8403
13	36	1	8	1	0	0	0	53	30760	'OSWICHEE CREEK		'SUNSHINI	'3.7 KM E/	99.99	0	0			32.30611	-84.9386
13	41	1	8	1	0	0	0	53	30760	'BONHAM CREEK		'HOURGLA	'1.5 KM SV	99.99	0	0			32.42656	-84.7647
13	45	1	8	1	0	0	0	53	30760	'WEEMS POND SPILL'		'JAMESTO	'1.2 KM S/	99.99	0	0			32.30718	-84.8427
13	00000000C	1	8	1	0	0	0	53	30760	'PINE KNOT CREEK		'BUENA VI	'1.5 KM E/	99.99	0	0			32.43913	-84.733
13	00000000C	1	8	1	0	0	0	215	30760	'RANDALL CREEK		'BUENA VI	'1.5 KM W	99.99	0	0			32.45413	-84.7912
13	00000000C	1	8	1	0	0	0	215	30760	'UPATOI CREEK		'BUENA VI	'.1 KM W/	99.99	0	0			32.44518	-84.7566
13	00000000C	1	8	1	0	0	0	53	30760	'N.F. PINE KNOT CRE		'CACTUS R	'6.5 KM N/	99.99	0	0			32.43822	-84.6675
13	00000000C	1	8	1	0	0	0	215	30760	'N.F. RANDALL CREEK		'MIDWEST	'2 KM NE A	99.99	0	0			32.47894	-84.801

Figure 21 – 2018 NBI Database Example.

2.1.3 WIM Database

Georgia currently has 14 active WIM stations across the state recording traffic data every second over the entire year. Most of these stations are located on the major transportation routes to efficiently gather as much information as possible. Of these fourteen stations, a total of 19 sites exist as some stations include more than one site as Savannah, for example, includes 4 sites within the region represented by 1 station. The data from these WIM stations represent 2019, and the recently replaced and/or advanced sensors provide more precise information. For this study, some WIM data were excluded due to low traffic volume and sites inability to cover enough NBI bridges. Therefore, a total of 7 of the 19 WIM sites were utilized due to their locations and traffic volumes. These 7 sites and the remaining sites are displayed in Figure 22.

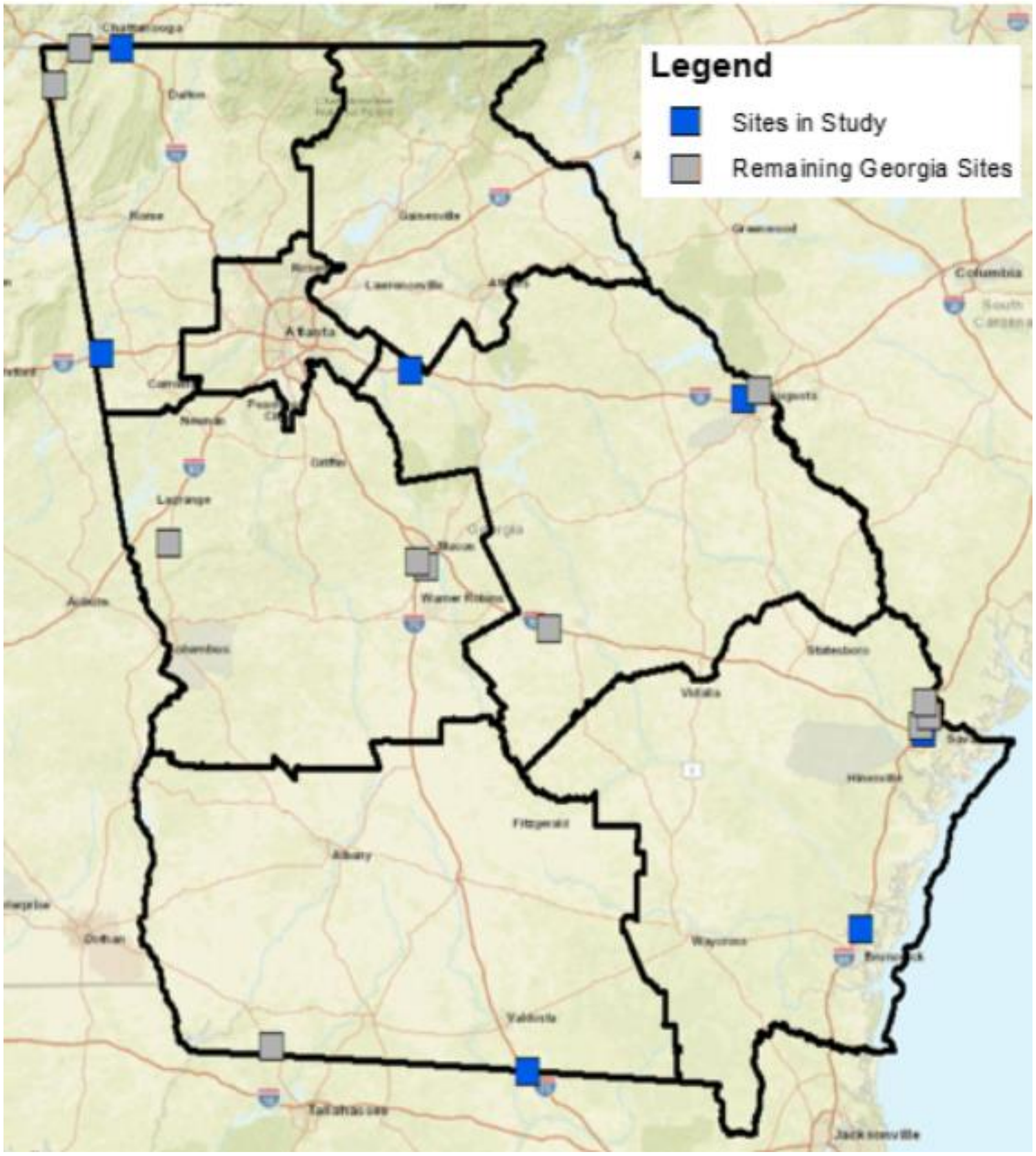


Figure 22 – Georgia WIM Sites.

These sites include a sensor that captures and records every vehicle that crosses over it. Most of the sites have sensors in each lane and on both sides travelling either direction. Whenever a vehicle crosses over the strip, multiple data variables characterizing the vehicle are collected within the system including class, time, axle weight, and speed. This information is piled up each day on all the sites in a format illustrated in Figures 23 and 24. This arrangement allows for easy understanding and ability, similar to the NBI data, to extract from in terms of coding. These files build up into large sets classifying it as being big data.

Node	Cosit	Time	Vehicle N	Lane	Lane Nam	Class Sche	Class Sche	Class	Class Nam	Length (m	Headway	Gap (s)	Speed (m
GDOT_CC!	217334	05:05.8	1207	2	Ln 1 SB	14	FHWA15	2	F2	3.74			59.78
GDOT_CC!	217334	05:13.7	1208	2	Ln 1 SB	14	FHWA15	3	F3	6.27	7.88	7.704	72.89
GDOT_CC!	217334	05:37.4	1209	2	Ln 1 SB	14	FHWA15	2	F2	5.47	23.725	23.477	70.28
GDOT_CC!	217334	05:41.8	1210	2	Ln 1 SB	14	FHWA15	3	F3	5.97	4.395	4.163	71.33
GDOT_CC!	217334	06:28.1	1211	2	Ln 1 SB	14	FHWA15	2	F2	5.81	46.264	46.02	69.9
GDOT_CC!	217334	06:50.4	1212	2	Ln 1 SB	14	FHWA15	2	F2	5	22.284	22.04	70.65
GDOT_CC!	217334	07:10.8	1213	2	Ln 1 SB	14	FHWA15	2	F2	5.97	20.392	20.176	56.55
GDOT_CC!	217334	07:13.7	1214	2	Ln 1 SB	14	FHWA15	2	F2	5.35	2.948	2.64	66.8
GDOT_CC!	217334	07:25.7	1215	2	Ln 1 SB	14	FHWA15	9	F9	20.4	11.977	11.737	63.69
GDOT_CC!	217334	07:29.0	1216	2	Ln 1 SB	14	FHWA15	2	F2	5.98	3.277	2.497	68.16
GDOT_CC!	217334	08:00.6	1217	2	Ln 1 SB	14	FHWA15	3	F3	11.05	31.576	31.32	65.56
GDOT_CC!	217334	08:15.0	1218	2	Ln 1 SB	14	FHWA15	2	F2	5.16	14.473	14.025	60.02
GDOT_CC!	217334	09:02.3	1219	2	Ln 1 SB	14	FHWA15	1	F1	1.75	47.267	47.007	63.13
GDOT_CC!	217334	09:16.4	1220	2	Ln 1 SB	14	FHWA15	2	F2	5.54	14.088	14.004	72.14

Figure 23 – WIM Format (Part 1).

Weight (k	Temperat	Duration (	Validity C	Chassis Cc	Class Inde	Loop Time	Chassis Pr	Num Axle	Axle Weig	Axle Spacings (m)	
1200		221	64	27	1	176	0817141B(	2	700|500	2.6	
2500		248		103	2	248	6.6E+10	2	1330|117(	3.5	
2070		233		132	1	232	057E4B84(	2	1120|950	2.74	
2550		244		104	2	244	07634268(	2	1310|124(	3.33	
2110		245		144	1	244	08816B90(	2	1200|910	3.04	
1050		216		139	1	216	083A2D8B	2	620|430	2.55	
1820		309		171	1	308	047831AB(	2	1060|760	2.9	
1620		240		128	1	240	06715380(	2	880|740	2.76	
14040		781		81	17	780	06490E511	5	4510|267(	5.89|1.32|7.23|3.09	
1700		256		149	1	256	8.88E+10	2	920|780	3.06	
2980		561	64	87	11	448	0742104A(	4	800|850|5	3.57|4.38|0.89	
1820		261		109	1	260	0550546D(	2	1070|750	2.83	
300		209	64	3	0	84	3.03E+10	2	100|200	1.4	
1440		229		167	1	228	063D47A7(	2	790|650	2.82	

Figure 24 – WIM Format (Part 2).

2.1.4 Pairing of Bridges with WIM Sites

In order to define each bridge with a WIM site, ARC GIS was utilized to map each accordingly. By extracting the latitudinal and longitudinal coordinates from the Drakewell Traffic Analysis and Data Application website, the 14 WIM locations were placed on a street view base map of the ARC GIS program. After the WIM sites were located on the map, the coordinates of each bridge referenced in the NBI data frame were established as well. The bridges are represented by the blue dots while the green squares are each of the WIM sites as illustrated below in Figure 25.

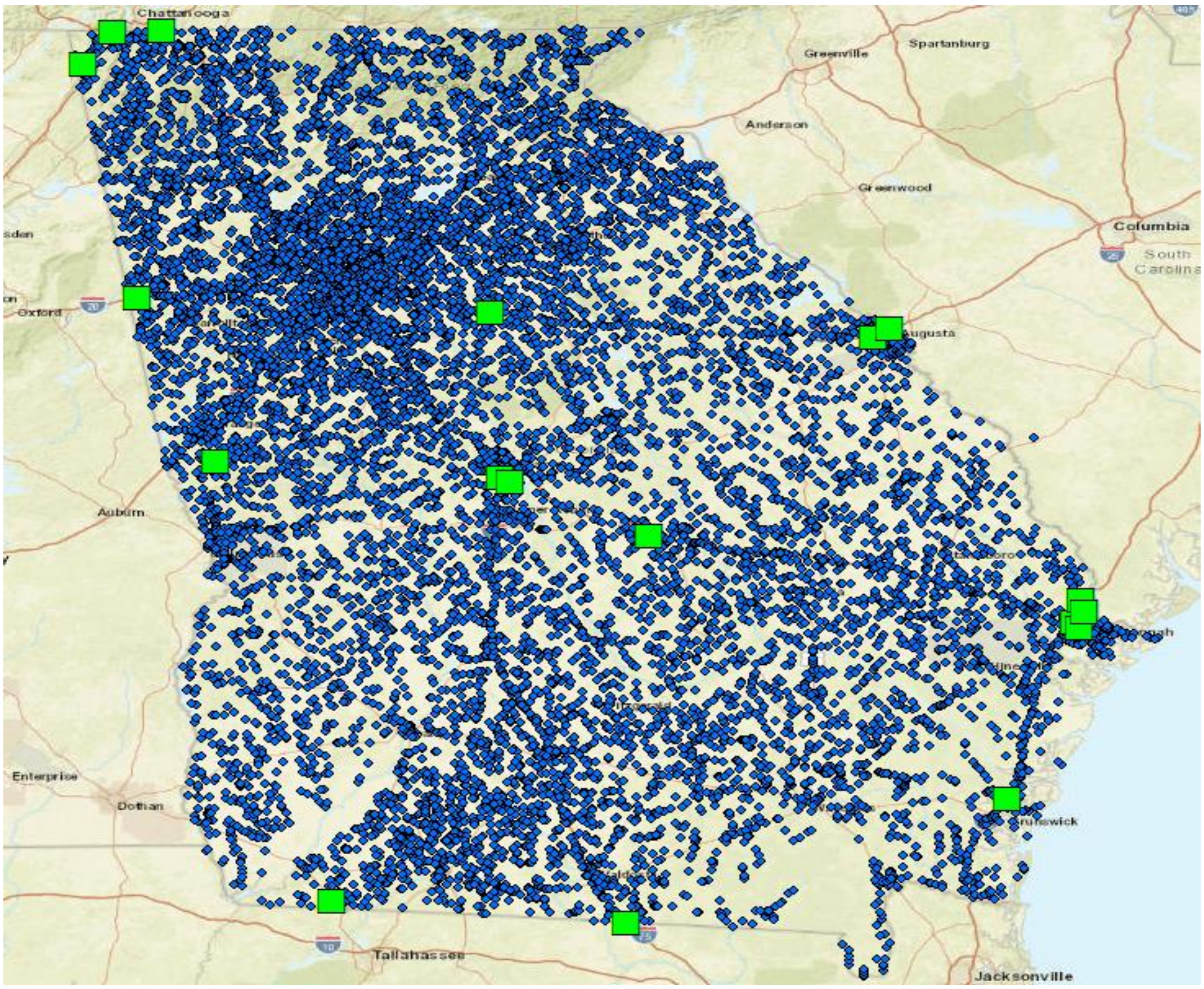


Figure 25 - ARCMAP View of Georgia Bridges with WIM Sites.

After viewing this overwhelming about of bridges, a new method was chosen to create a more organized and clear system of correlation. Instead of focusing on every bridge in the state of Georgia, only the bridges classified in the National Highway System or NHS were utilized. This allowed for an easier pathway to view and compare as well as eliminated those minor bridges that prove to be difficult to compare. Additionally, the bridges were segregated based off of GDOT sections for classification allowing for easier decisions in the decision of which site works with which bridges. Figure 26 represents the NHS bridges organized by color into their different GDOT sectors.

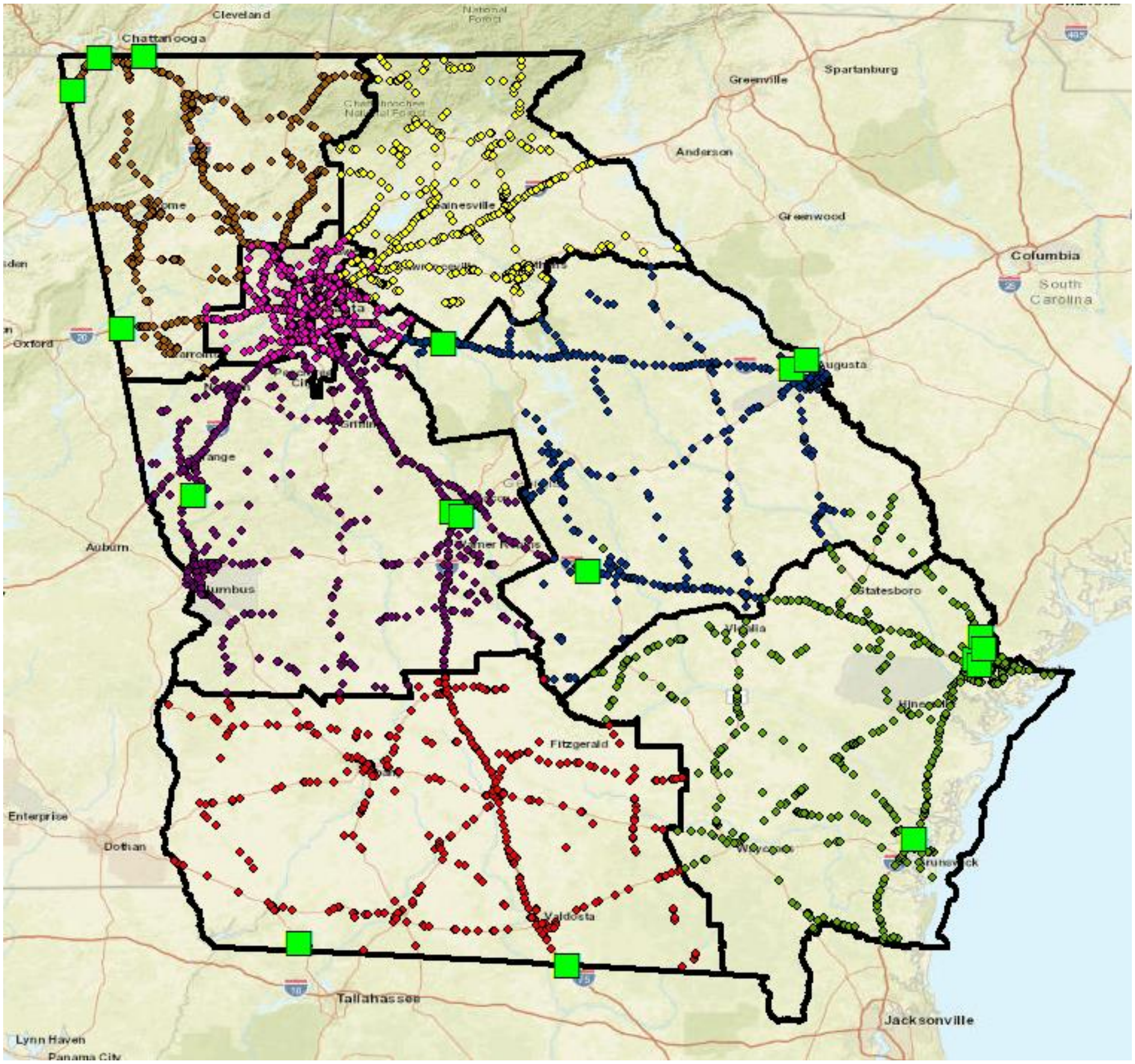


Figure 26 – ARCMAP View of NHS Bridges with GDOT Sections.

When comparing WIM sites to their associated bridges, a few tests were run to correlate the two. The major highway routes that pass along the WIM sites were focused on as they are best represented by that specific site's traffic flow. NHS bridges with traffic that does not directly flow into a nearby WIM site were not focused on in this study. However, these bridges could later be examined further and be applied a factor below a value of one as their traffic is assumed to be less than the data provided by the WIM sites.

To compare the two data sets, Python coding was utilized as it is effective in evaluating large amounts of data and can be used for multiple different tasks. As mentioned earlier, the low volume sites were removed from this study as their data would skew the results and only relate to a handful of bridges in its area. The remaining sites are displayed in Table 6 below presenting their characteristics and their exact locations were displayed earlier in Figure 22.

Table 6 – WIM Site Details.

Site ID	Site Name	Description	Speed Limit	Orientation	Latitude	Longitude
210378	021-0378	I-475 (SR 408) btwn I-75 & SR 22 South of SR 22	65.0mph	N	32.79937	-83.7212
470114	047-0114	I-75 btwn SR146 & Tennessee Line	65.0mph	N	34.98511	-85.2009
510368	051-0368	I-16 East of Dean Forest exit.	65.0mph	E	32.06899	-81.1928
510387	051-0387	I-95, 2 mi N. of SR-21 @ SC state line	70.0mph	NE	32.2002	-81.1877
830214	083-0214	I-24 bn TN State Line & SR299 W Side, Chattanooga	70.0mph	NE	34.98289	-85.4098
1270312	127-0312	I-95 btwn SR27 & Golden Isles Parkway SR 25 Spur M	70.0mph	NE	31.23438	-81.5093
1430126	143-0126	I-20 btwn Alabama State line & SR100 Veterans Mem Hwy	70.0mph	E	33.68077	-85.3022
1850227	185-0227	I-75/SR401 @FLA SL, Lake Park, Lowndes Co	70.0mph	NW	30.62671	-83.1731
2170218	217-0218	I-20 WEST OF SR 11 BTWN SR 142	70.0mph	E	33.61157	-83.7616
2450214	245-0214	I-20 Columbia Co Line & SR415 Bobby Jones Expressway	55.0mph	E	33.49057	-82.0949
2450218	245-0218	I-20 E of I-520 @SC State Line, Augusta	65.0mph	NE	33.52746	-82.0191

These 11 WIM sites were then broken down into just 7 sites. This was due to the 7 sites being situated on major highway routes creating a simple process in categorizing the site with specific bridges. By filtering the NBI dataset by county and location, specific bridge sets were created to represent each WIM site. The sets were then displayed on ARCGIS to illustrate how the WIM sites related to each set for this study in Figure 27.

Figure 27 – ARCMAP View of 7 WIM Sites with Coordinated NHS Bridges.

A primary code was generated as a general formula to be applied to each of the 7 sites in the study. A small sample of the population was used at first to check the accuracy and reliability of the code. Once the code was deemed correct, it was then applied to the 7 chosen sites for comparison. This code can be viewed in Appendix C.

2.1.5 NBI Traffic Count

The goal of this initial comparison is to focus on the amount of average daily traffic (ADT) and average daily truck traffic (ADTT) gathered in the NBI and WIM site data. NBI is organized into columns allowing it to be straightforwardly called out and utilized in the code. The NBI data was extracted and analyzed according to the specific bridges described earlier. The NBI data has a category labelled “ADT_029.” that was extracted for this procedure to determine the ADT for each bridge. However, by just viewing this column, it is determined that this data is estimated due to the rounding off of its numbers. Additionally, it is not updated properly as the ADT year classified in category “YEAR_ADT_030” is showing at least a 7-year difference for the majority of bridges as seen in Figure 28.

ADT_029	YEAR_ADT
73750	2011
31460	2011
31460	2011
32610	2011
32610	2011
32430	2012
32430	2012
36510	2012
38650	2011
56630	2011
56630	2011
68530	2011

Figure 28 – NBI ADT and ADT Year Sample.

After the ADT for the NBI data was determined, the ADTT was focused on. The ADTT is the most important information needed in this study as it represents the amount of trucks passing over these bridges which sets the standard for load distribution across its deck. ADTT was a simple process to determine as it was product of the "ADT_029" column and the "PERCENT_ADT_TRUCK_109" column. The truck percent like the ADT is rounded off and usually appeared in sets of 5 indicating its lack of precision.

2.1.6 WIM Traffic Count

After analyzing the NBI data individually, the WIM data then had to be described properly. This process proved to be more time consuming as instead of viewing around just 50 to 100 bridges like in the NBI data, the WIM data consisted of millions of values. The dataset of each WIM site includes every vehicle from large 18 wheelers to small compact cars that cross over its interstate marker in the year of 2018.

While dealing with this big data analysis, multiple techniques had to be tested. An initial procedure of dividing the entire vehicle count by 365 was used. Then the process of averaging the entire set by month was utilized then changed to be organized by day to provide a more precise value to represent the data. To organize this large set into a controllable daily value, a loop was created that provided a list of all 365 ADT and ADTT values. The ADT list included every vehicle listed that year while the ADTT was filtered by using only vehicles between Class 4 and Class 13 based upon the Federal Highway Administration (FHWA) to match how NBI determines its ADTT percent. Table 7 defines these FHWA classes as well as illustrates which classes are considered heavy trucks. This 8 to 13 class range was used for the load demand in this study to represent the heavier side of the scale which controls the loading distribution on the bridge decks. Once the two lists representing the ADT and ADTT were found, then an analysis of the data was performed.

Table 7 – FHWA Classification System (FHWA, 2017).

	CLASS GROUP		DESCRIPTION	NO. OF AXLES
	1		MOTORCYCLES	2
	2		ALL CARS CARS W/ 1-AXLE TRAILER CARS W/ 2-AXLE TRAILER	2 3 4
	3		PICK-UPS & VANS 1 & 2 AXLE TRAILERS	2, 3, & 4
	4		BUSES	2 & 3
	5		2-AXLE, SINGLE UNIT	2
	6		3-AXLE, SINGLE UNIT	3
	7		4-AXLE, SINGLE UNIT	4
HEAVY TRUCKS	8		2-AXLE, TRACTOR, 1-AXLE TRAILER (2&1) 2-AXLE, TRACTOR, 2-AXLE TRAILER (2&2) 3-AXLE, TRACTOR, 1-AXLE TRAILER (3&1)	3 4 4
HEAVY TRUCKS	9		3-AXLE, TRACTOR, 2-AXLE TRAILER (3&2) 3-AXLE, TRUCK W/ 2-AXLE TRAILER	5 5
HEAVY TRUCKS	10		TRACTOR W/ SINGLE TRAILER	6 & 7
HEAVY TRUCKS	11		5-AXLE MULTI-TRAILER	5
HEAVY TRUCKS	12		6-AXLE MULTI-TRAILER	6
HEAVY TRUCKS	13	ANY 7 OR MORE AXLE		7 or more
	14	NOT USED		
	15	UNKNOWN VEHICLE TYPE		

Now that the WIM data was condensed into a reasonable set, a histogram of this daily information was created to see how it resulted. The histograms were then placed next to the NBI data displayed as a scatter plot to create a side by side comparison of the two sets. Additionally, the outliner of the WIM set were removed for curiosity purposes and it did not vary the outcomes significantly, so the outliner were left in the data. A more detailed outlook of this process can be seen by viewing the python code in Appendix C.

2.1.7 Kruskal-Wallis

More steps were needed to help distinguish the correlation between the NBI and WIM counts. A t-test could not be conducted as not all the distributions were considered normal; therefore, the use of a Kruskal-Wallis test was performed instead. This type of test allows for two populations to be compared without the need to assume a normal distribution. A 95 percent confidence interval was conducted for both the ADT and ADTT results for WIM and NBI to see if any correlation between the two datasets matched. For this testing, a null hypothesis was created for evaluation. This null hypothesis stated that the two sets of data are considered to be similar. This testing was calculated in python and produced a p-value as its final outcome. If this value was small enough or being below 0.05 for a 95 percent significance interval, then the null hypothesis stating that the two sets of data are similar can be rejected.

2.1.8 Site Specific Analysis

Two specific locations were selected for a more detailed inspection due to their significance within the state. Interstate 75 between Atlanta and the Tennessee Border was classified as being the area to some of the largest traffic counts in the state and the Savannah port distributes thousands of trucks each day. Due to the ADT and ADTT of these two locations, the truck total weights and maximum axle loads were analyzed as it is believed to be controlling variables in the state representing Georgia's maximum quantities. The airport and more of I-285 would have been explored if WIM sites were able to be in these locations.

2.2 Live Load Demand Envelope

2.2.1 Introduction

Due to the availability of new 2019 WIM data for the state of Georgia, truck traffic was studied at a more accurate and in-depth knowledge. This information created the capability to evaluate every vehicle on Georgia roads and bridges with detailed statistics. A structural analysis of NHS bridges was investigated to see what shear and moment values were being applied to its components through the creation of a load envelope. This load envelope was utilized to increase efficiency and dependability on standards to precisely analyze every bridge within the state. The bridges being examined were assumed to be simply supported as most Georgia NHS bridges are. Additionally, this focused on the live load as the dead load is not necessary for the reliability evaluation due to the LRFR method.

According to the GDOT Condition Assessment of Existing Bridge Structures for determining its structural reliability, a few tests are performed to determine the capability of a bridge. These tests include the HL-93 design load and Georgia state/legal truck configurations if the design load fails. If a bridge passes the HL-93 design load, then it is deemed reliable and can withstand normal load limits as well as permitted trucks. If a bridge fails to resist a HL-93 design load criterion, then it is not allowed to provide access to permitted trucks. However, if the bridge can handle the Georgia state/legal loads then it is considered reliable to manage the allowable weight limits defined in Formula B. However, if the bridge fails to pass the Georgia state/legal loads then it cannot bear the normal traffic flow of trucks and is classified under the posted category. A bridge defined in this set is given a load limit based upon its strength capacity limiting the amount of traffic able to access it. Conversely, NCHRP Report 575 has researched the examination of a new type of truck configuration to represent the modifying of truck loads over the years. This new alignment is defined as the Notional Rating Load or NRL and is required according to the Manual of Bridge Evaluation but is considered a conservative approach. However, it was believed to be a more reliable depiction of trucks within the Formula B weight requirements when compared with the Georgia state/legal trucks.

For this study, the HL-93 design load, Georgia state/legal trucks, NRL truck were applied to different span lengths to determine the maximum shear and moment values for the condition assessment representation. Once these numbers were found, the entire truck dataset from WIM sites throughout the state were analyzed for comparison. A straightforward comparison between the shear and moment values was acceptable as the truck traffic between WIM and referenced data was assumed equal. The results of each were equated and discussed for recommendations on how the methodology for bridge condition assessment should be improved and how the NRL and Georgia state/legal trucks differ.

2.2.2 Truck Filtration

The first step was accurately classifying the WIM truck data. In the code, the data being examined was called in initially then filtered to remove uncertainties and classify the data properly. The dataset was reduced to include only heavy weighted trucks in the classes of 8 to 13. Then the truck set was condensed further to include only trucks with a maximum of 8 axles as this included the majority of the vehicles. Once the dataset had been edited to the demands, the values of axle weight and spacing were applied a factor to convert the units from metric to SI.

Once the dataset was modified, the WIM data had to be separated into two groups of Formula B trucks and Non-Formula B trucks. The Formula B trucks are the trucks that meet the four basic federal weight limits defined in Table 8 from NCHRP Report 575. These trucks do not need a permit to travel as their weights are not overly damaging to the structural ability of bridges. The second set of WIM trucks are Non-Formula B trucks and represent the trucks that do exceed at least one of the Formula B requirements in Table 8. These trucks legally need a permit to travel as their weights can be detrimental to bridge structures in the state. These trucks are usually given a specific route to avoid bridges that cannot handle their higher weights. A code was made to create these two categories and determined their maximum shear and moment values to be compared with the three truck models defined next.

Table 8 – Formula B Requirements.

Four Basic Federal Weight Limits	
1	Single Axles – 20,000 lbs
2	Tandem Axles – 34,000 lbs
3	Maximum Gross Vehicle Weight – 80,000 lbs
4	Application of the FBF B for each Axle Group up to Maximum Gross Vehicle Weight

2.2.3 HL-93 Design Load

The HL-93 design load is the combination of a HL-20 truck, lane load of 0.64 kips per linear foot, and tandem of two 25-kip forces at 4 feet apart as defined by AASHTO. An excel file representing this design load was created and called into the python code for its moment and shear calculations. Due to the variations of the length of the last two axles in the HL-20 truck, multiple trucks were simulated through the code at iterations of one foot between 14 and 30 feet to determine its true maximum. This design load is illustrated in Figure 29 below.

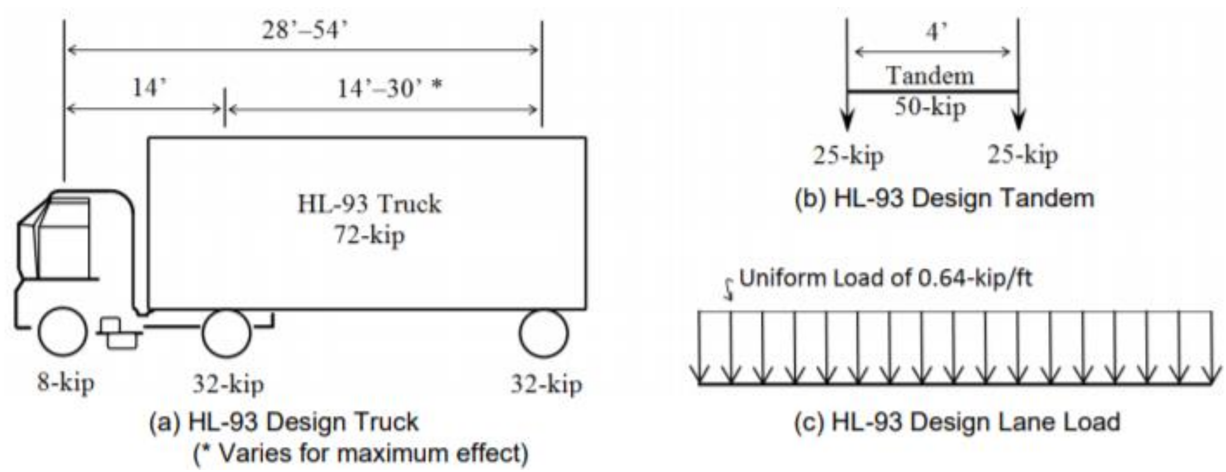


Figure 29 – HL-93 Design Load (Waldron, 2010).

This design load is the highest standard of the three truck models and was expected to produce the largest values for shear and moment. As a design load it should be considered the worst-case scenario and be able to represent every truck the bridge should experience. Therefore, it was compared with both Formula B and Non-Formula B trucks to examine its reliability in handling every WIM truck.

2.2.4 Georgia State/Legal Trucks

Once the HL-93 design load was generated and examined, then the AASHTO State/legal trucks were created as well. This inspection checks for 6 different types of trucks including H, HS, Type 3, Type 3S2, Timber, and Piggy-Back. Each of the 6 configurations are illustrated in Figure 30 below. An excel file representing each of these trucks was generated to be transported into the Python code.

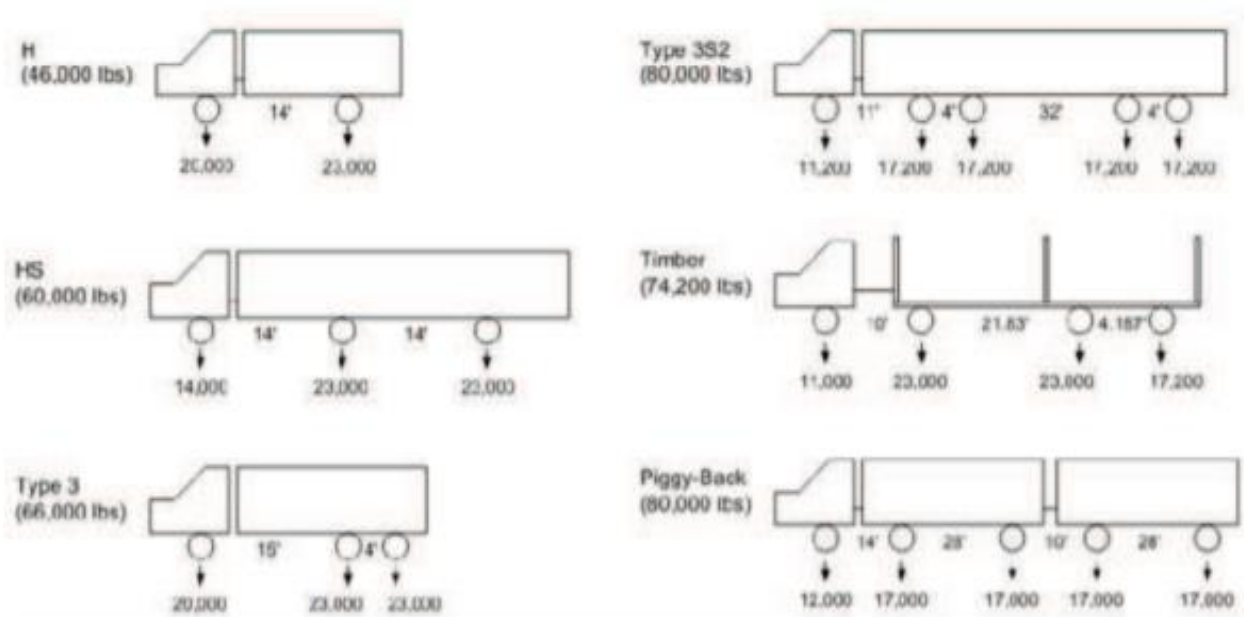


Figure 30 – Georgia State/Legal Trucks (Ellington, 2009).

The Georgia state/legal trucks are the second truck model tested in the GDOT condition assessment for bridge evaluation. It is a lower standard than both the HL-93 design load and NRL and, therefore, should produce the lowest shear and moment values. Its six truck configurations only represent trucks that do not need permit to travel and meet the Formula B requirements. So, its results were only compared with WIM Formula B trucks.

2.2.5 Notional Rating Load

Finally, the NRL truck load was developed for inquiry. This truck includes the configuration of 8 axles with a common spacing of 4 feet besides the initial spacing that varies between 6 to 14 feet. Similar to the HL-93 design load, an excel file was made to be transferred into the Python code that included multiple trucks with an initial spacing changing at one-foot intervals between the desirable range. This file is presented below in Figure 31.

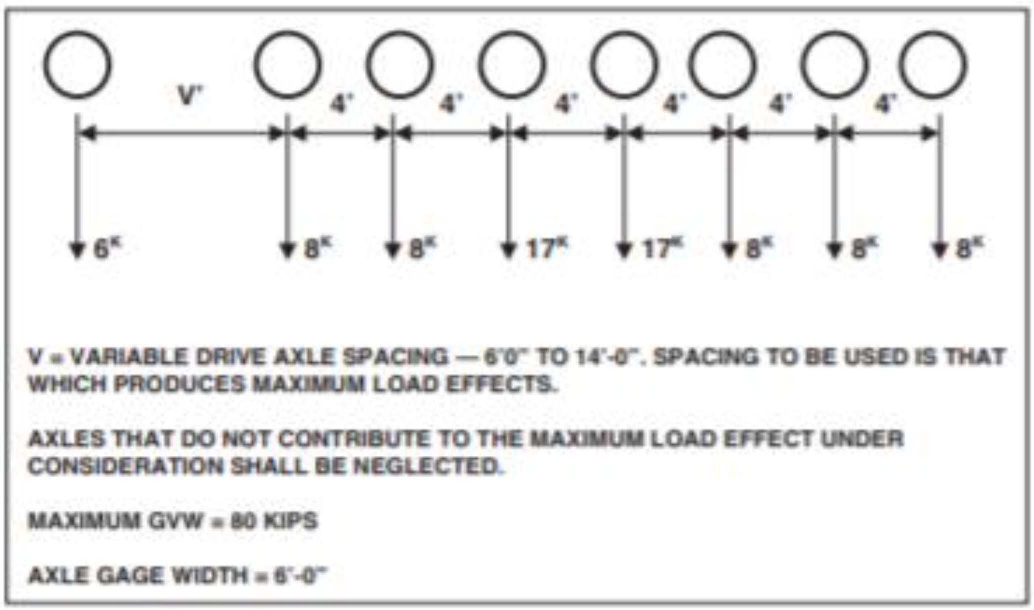


Figure 31 – Notional Rating Load Configuration.

This truck is a new model to represent special hauling vehicles and modifications trucks have experienced over the years. It is not part of the GDOT condition assessment of bridge evaluation but is required for the Manuel of Bridge Evaluation but considered as a conservative approach. It is believed to be a better representation of the WIM Formula B trucks and, therefore, was compared with the Georgia state/legal loads to see which is a more reliable representation. As a conservative approach it should produce greater shear and moment values than Georgia state/legal loads but not greater than the HL-93 design load.

2.2.6 Span Length Examination

The span lengths to be tested was determined based upon the 2018 NBI data. For a single span length, the values ranged from 3 to 1,250 feet for the 14,880 bridges. However, the larger span values provided represent cable stayed bridges that were not analyzed for this study. Figure 32 illustrates the adjusted range of maximum span lengths with the majority of bridges having a maximum span length below 50 feet. Very few bridges are seen above the 180 feet mark and were set as the maximum length tested for this study. For the lowest value, 30 feet was chosen and was a significant length as it embodied the majority of bridges. Therefore, the span lengths examined ranged from 30 to 180 feet in iterations of 30 feet implying the values were 30, 60, 90, 120, 150, and 180 feet for this study.

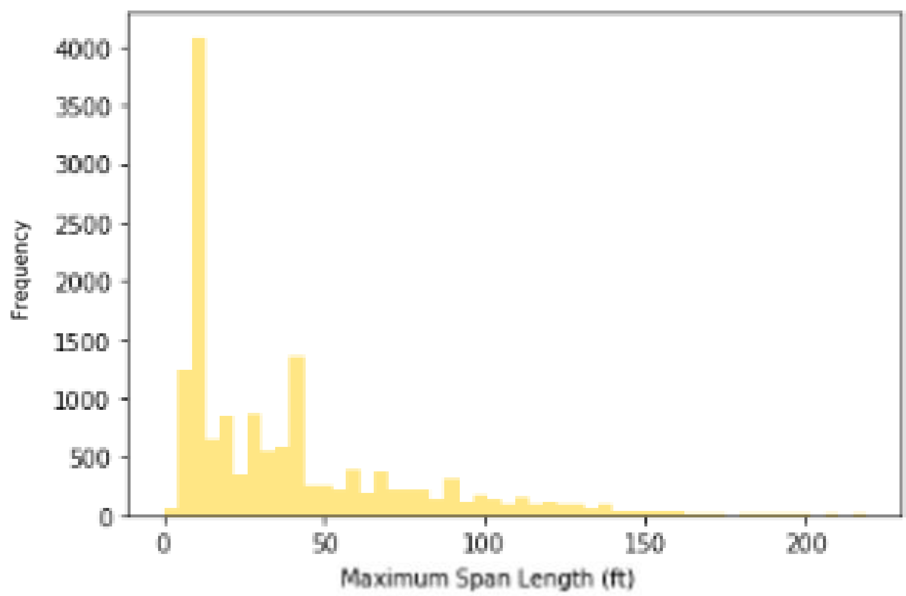


Figure 32 – Maximum Span Length of Georgia Bridges.

2.2.7 Shear and Moment

The maximum shear on a single span for moving loads on a bridge is found at the two end supports of the span. This is due to a beams usual uniform cross section and large ratio of length to height. Being able to condense the amount of locations down to two spots at each support reduced the run time necessary to determine maximum shear. Additionally, the maximum shear is whenever the vehicle being examined is near the supports. So, the duration of the code was reduced even more as it only calculated shear values when the truck length was fully present at the beginning and end support of the span. Once the shear values for each vehicle were appended into a list, the maximum value was identified and placed into another list comprising of the maximum shear of every truck in that WIM site. This process is verified in Appendix F as the shear maximum values for the HL-93 design load provided by the code matched with the verification calculation. The code analyzing Non-Formula B trucks for shear is provided in Appendix B.

Influence lines were utilized in order to compute the moment values in a more efficient way. Once the span length being examined was stated, then factors corresponding to a unit load at each location were established. These factors were recorded for every foot on the span length. For example, if the span was specified at 30 feet then 30 factors would be generated. This allowed for an effective procedure in reducing run time in each code.

The moment code was not as easy of a task as the maximum value did not occur in a known location as it does with shear. The maximum moment usually is found near the halfway or midpoint of the span but can fluctuate at times. Initially a code was created that produced the influence factors for every location at every foot of the specified span length. If the span length was 180 feet, then the total amount of influence factors generated would be 180 multiplied by 180 ensuing in 32,400 factors per truck. This resulted in a significantly long duration of code as the WIM dataset would take days to run just one site. In order to fix this dilemma, the moment at the midpoint was computed for each vehicle assuming it to be at or within a close range of the maximum. The centroid of the truck being examined would be calculated according to Equation 9 below applied to the midpoint moment code. With this new method, the total amount of influence factors applied to each truck for a 180-foot span was 180 factors compared to the initially method amount of 32,400 factors. The maximum moment per truck would finally be appended to a list similar to the shear maximum. A verification is provided in Appendix G illustrating the calculation of the maximum moment for a HL-93 truck matching the code's value. Additionally, the code determining the moment values for Formula B trucks is presented in Appendix A.

$$\boldsymbol{Centroid} = \frac{\sum(\boldsymbol{Weight} * \boldsymbol{Length\ from\ Front\ Axle})}{\sum \boldsymbol{Weights}} \qquad \textbf{(Eq. 9)}$$

Once the codes for determining the maximum shear and moment values were established, then each file representing the HL-93 design load, Georgia state/legal trucks, and NRL truck were applied. Afterwards, the 11 WIM sites chosen for this study were ran through the code for both state legal and Non-Formula B trucks. The 11 WIM sites are displayed from ARC GIS in Figure 33 below and were selected according to their high volume of traffic and prime location. After all the sites were applied, then the results were plotted comparing the Non-Formula B trucks with the HL-93 design load standard to test the reliability of its context. The shear and moment values for the state legal trucks meeting the Formula B limits for each WIM site were then assessed with the Georgia state/legal trucks and NRL truck.

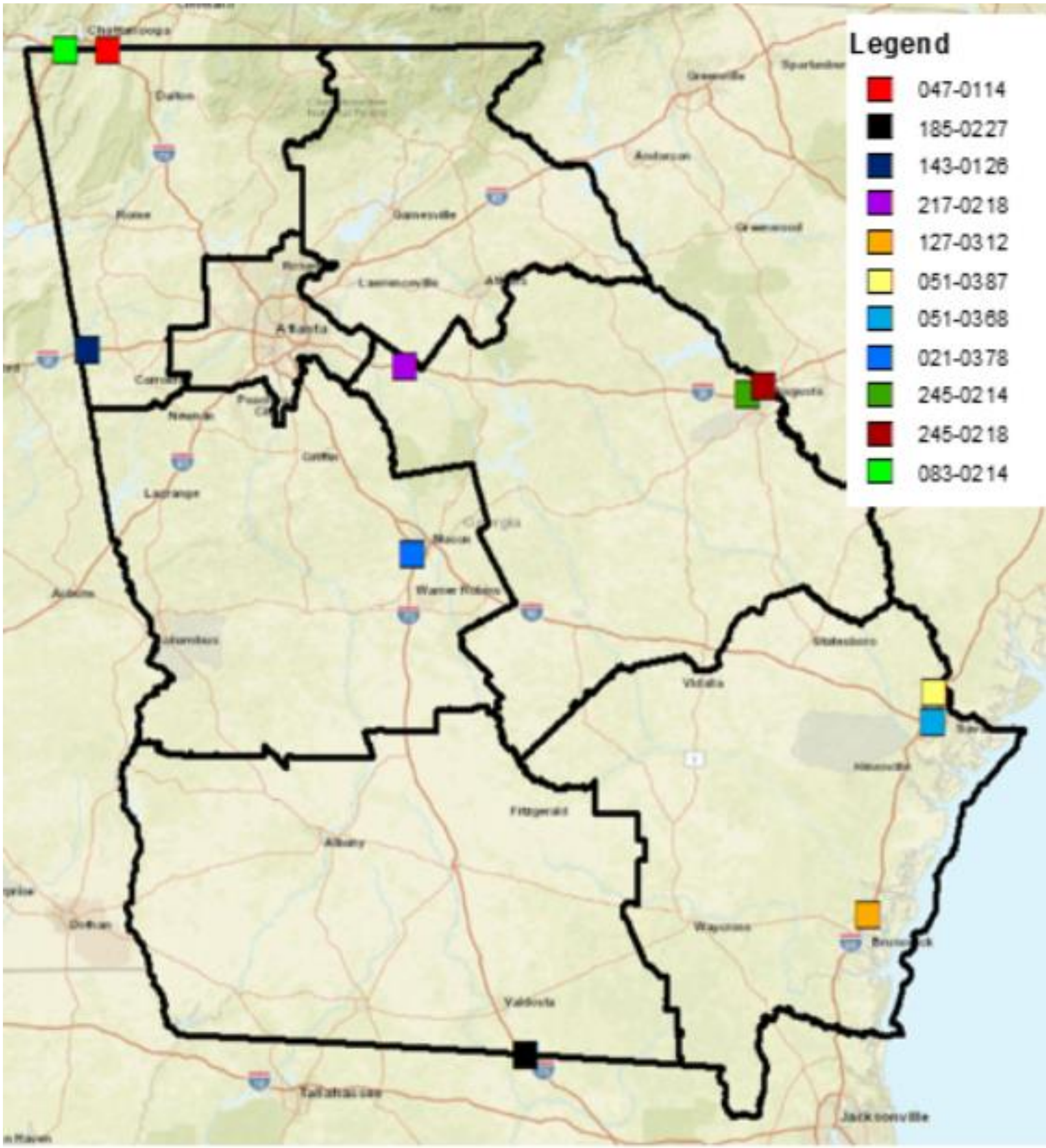


Figure 33 – High Volume WIM Sites.

2.3 Multiple Truck Statistics

2.3.1 Introduction

According to the NCHRP Report 368, the following probability and side-by-side probability are useful parameters when designing and calculating the live loads experienced on a bridge. Each probability is key in predicting the maximum shear and moment a bridge will undergo as its peak value occurs when multiple heavy trucks cross its span at the same time or when one lane is fully loaded while the other lane is unloaded. The following probability, as defined in this report, of two trucks following each other by a 100-foot headway distance is 1 out of every 50 trucks or 2.00 percent. The side by side probability for a two-lane calculation is considered to be 1 out of every 15 trucks or 6.67 percent. This information is based upon the Ontario truck data gathered in 1975 for a 2-week evaluation. Georgia traffic is believed to differ from this dataset due to the time frame of initial study and different cultures of the two areas. Therefore, the WIM data for 2019 was analyzed and compared to the Ontario data to view the differences for following probability and side-by-side probability.

2.3.2 Following Probability

A python code was made to determine the following probability of trucks for each WIM site and is provided in Appendix D. The sites utilized for this test are the same 11 sites studied for maximum shear and moment and are demonstrated in Figure 33. Once the specific site was called into the code, it was filtered to focusing on just the first or slow lane as this lane holds the majority of trucks on the interstates. The headway distance, defined as the length from the last axle of the first truck to the first axle of the second truck, was then determined for each passing truck. This was found through the multiplication of the change of time logged for each truck and the speed measured as shown in Equation 10. Finally, this value was subtracted by the length of the first truck providing the headway distance and was then checked to see if it met the criteria of being less than 100 feet. Once the probability of this occurrence for each site was calculated, it was recorded for comparison with the Ontario probability of 2.00 percent.

$$[\boldsymbol{Time}_{\boldsymbol{Truck\ 2}\ (\boldsymbol{seconds})} - \boldsymbol{Time}_{\boldsymbol{Truck\ 1}\ (\boldsymbol{seconds})}] * \boldsymbol{Speed}_{\boldsymbol{Truck\ 1}} \qquad \textbf{(Eq. 10)}$$

2.3.3 Side-by-Side Probability

The same sites were then applied to a secondary code that determined the side-by-side probability which is displayed in Appendix E. Based on the NCHRP Report 368, the side-by-side probability focuses on the first two lanes of the site. After the filtration process, the same formula based on Equation 10 was recorded providing the distance between the two front axles of each truck being analyzed. If the distance calculated was less than length of the first truck, it was considered to be defined as side-by-side. This side-by-side probability was then recorded and compared to the 6.67 percent from the Ontario dataset.

Once the possibilities of each case were determined, a concluding argument defining if the Ontario dataset is accurate was stated. If concluded inaccurate, state specific values can be found using its WIM data to enhance the reliability of Georgia's data in load rating analysis. Additionally, the probabilities can be broken down further to be quantified by major routes as many interstates differ depending on location and truck traffic flow.

3.2 COMPARISON OF NBI AND WIM DATASETS

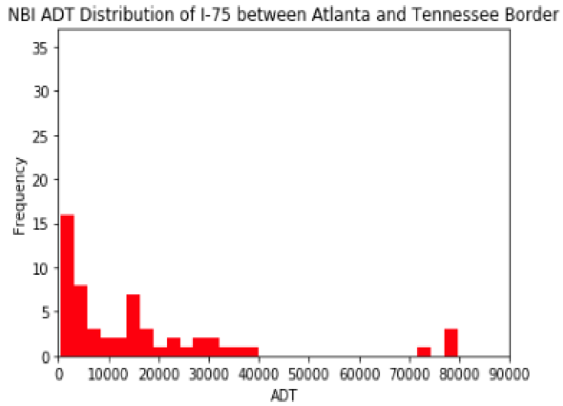
3.1 Introduction

This chapter provides valuable knowledge in comparing the traffic usage observed in the WIM and NBI data in Georgia over seven interstate routes. A hypothesis was conducted stating that the NBI database fails to produce accurate and updated ADT and ADTT values. This study focused on analyzing the traffic counts of NBI data with the traffic counts of WIM data to check the reliability of this NBI data through a correlation test. The NBI data was determined and analyzed first with its methodology listed in Chapter 2.1.5. The WIM data was then calculated and compared to the NBI data with its procedure provided in Chapter 2.1.6. Each route followed a python code with an example code provided in Appendix C for additionally understanding. The results of this comparison proved to be valuable in the use of WIM data for characterizing traffic usage over Georgia bridges.

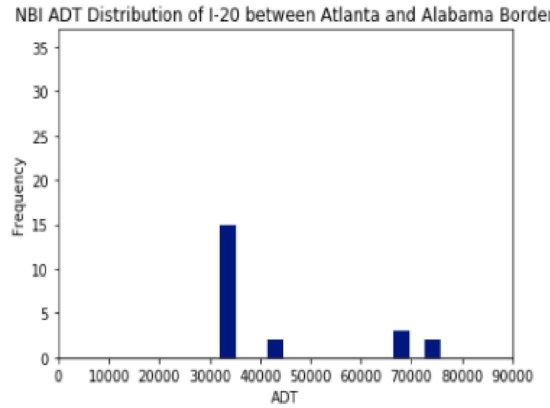
3.2 NBI Average Daily Traffic

Initially just the NBI data was analyzed. The NBI average daily traffic or ADT was determined and histograms of each of the seven routes examined were plotted. The seven routes being studied are illustrated in Figure 34 below and are color coordinated with their plots. The square shapes represent the WIM sites and the smaller circles are the NBI bridges associated with the WIM sites. Its distribution is an important tool in determining the consistency of the bridges as a normal distribution should be present. The NBI ADT distribution of all seven interstates is shown in Figure 35.

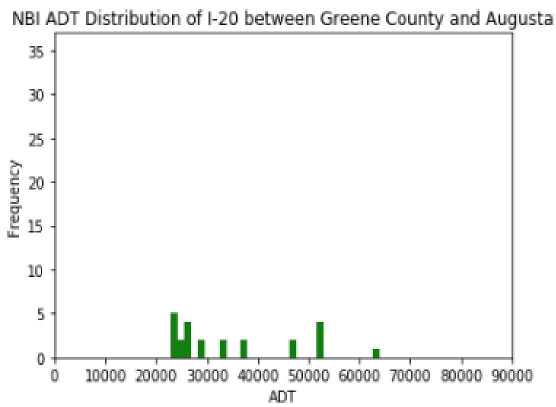
Figure 34 - ARCMAP View of 7 WIM Sites with Coordinated NHS Bridges.



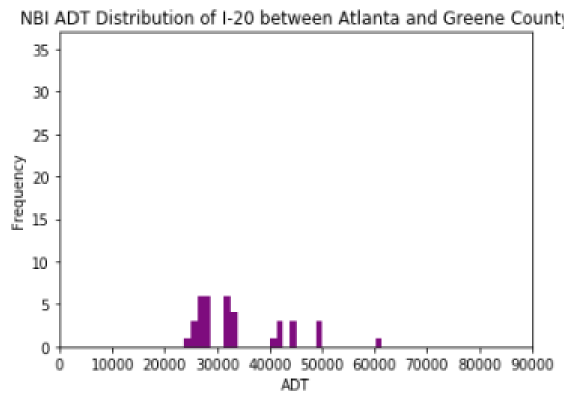
(a) NBI I-75 between Atlanta and Tennessee Border



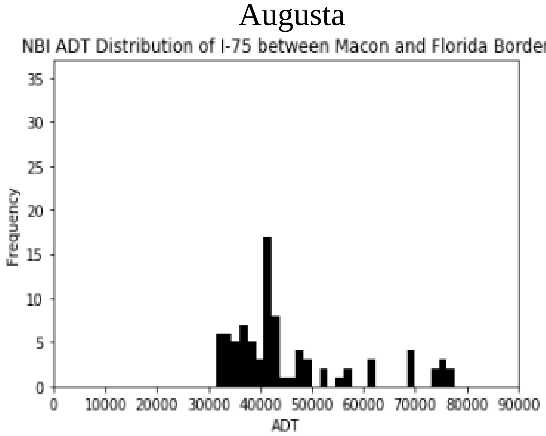
(b) NBI I-20 between Atlanta and Alabama Border



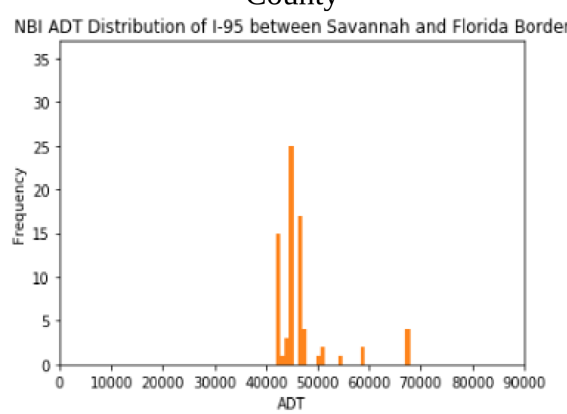
(c) NBI I-20 between Greene County and Augusta



(d) NBI I-20 between Atlanta and Greene County

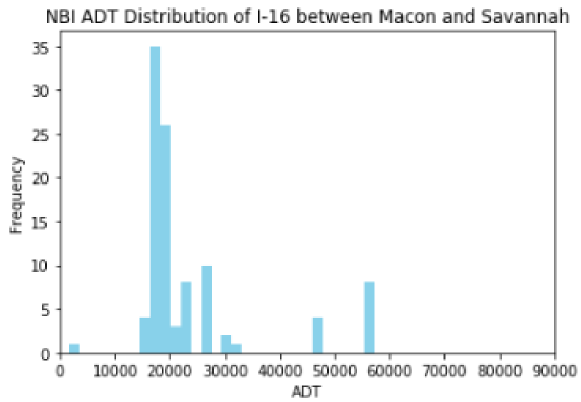


(e) NBI I-75 between Macon and Florida Border



(f) NBI I-95 between Savannah and Florida Border

Figure 35 – NBI ADT Distribution for WIM Sites.



(g) I-16 between Macon and Savannah

Figure 35 Continued – NBI ADT Distribution for WIM Sites.

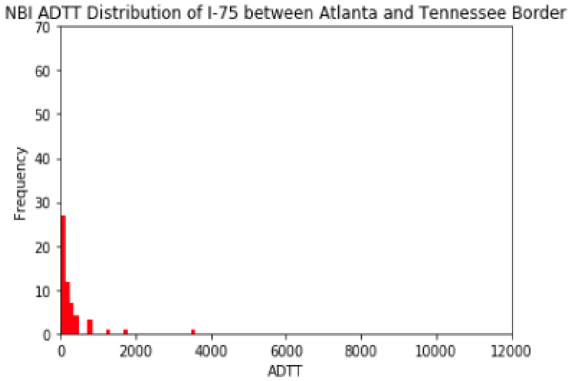
The data determined in each of the seven routes was expected to follow a normal distribution as this would typically represent the usage of traffic flow. However, none of the interstates displayed a normal distribution as the plots seem to be random and lack the ability to form a defined shape. Each of the graphs showed multiple outlier on the higher end of the scale with some ranging up to near 80 thousand vehicles a day. The majority of routes displayed the bulk of their distributions around the 30 to 50 thousand ADT range exempting two routes of I-75 (Atlanta and Tennessee Border) and I-16 (Macon and Savannah) with lower values. This is expected for I-16 (Macon and Savannah) but I-75 (Atlanta and Tennessee Border) is a major route anticipating a high traffic volume. Instead, it produced a NBI ADT average of 16,239 which was well below the other routes in this study shown in Table 9. I-75 (Atlanta and Tennessee Border) was expected to yield the highest ADT. These values as well as the lack of a normal distribution in each route question the accuracy of the NBI database and, therefore, was examined in more detail with the WIM data. I-20 (Atlanta and Alabama Border) produced the largest ADT according to the NBI data of 52,980 and did not have any days that recorded a value lower than 30 thousand vehicles as seen in its plot (see Figure 35). Other high-volume routes included I-75 (Macon to Florida Border) and I-95 (Savannah to Florida Border) which is reasonable as Florida is Georgia's top trading partner.

Table 9 – NBI ADT Distribution Averages.

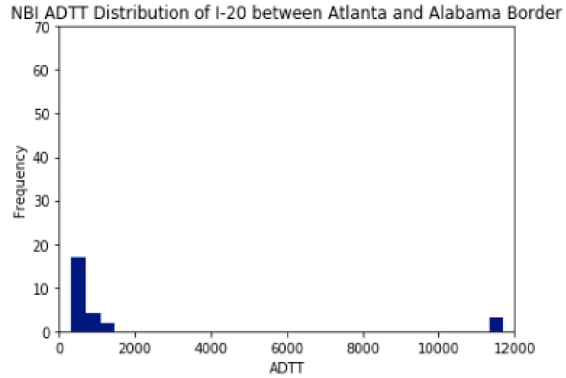
NBI ADT Distribution Averages	
Route	ADT
I-75 (Atlanta and Tennessee Border)	16,239
I-20 (Atlanta and Alabama Border)	52,980
I-20 (Greene County and Augusta)	34,605
I-20 (Atlanta and Greene County)	34,244
I-75 (Macon and Florida Border)	45,741
I-95 (Savannah and Florida Border)	46,693
I-16 (Macon and Savannah)	23,615

3.3 NBI Average Daily Truck Traffic

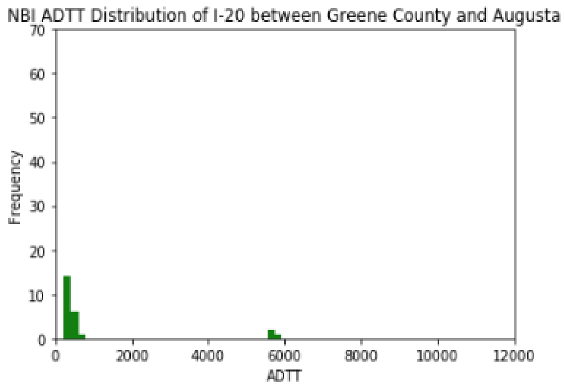
Once the ADT was computed and plotted, the NBI average daily truck traffic or ADTT for each of the seven routes was analyzed. This data is more important than the ADT as truck traffic controls design and structural analysis purposes. Its shape was additionally check in following a normal distribution. The results of the NBI ADTT is shown in Figure 36 below and follows the same color coordination shown in Figure 34.



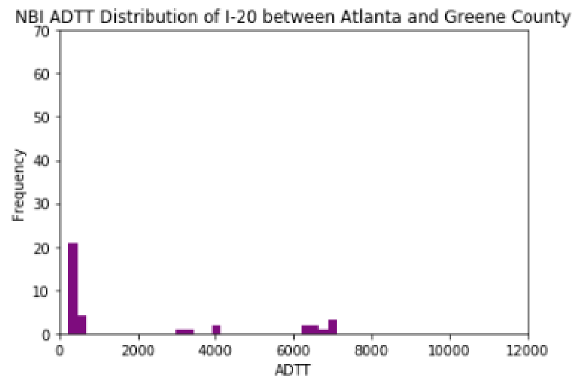
(a) NBI I-75 between Atlanta and Tennessee Border



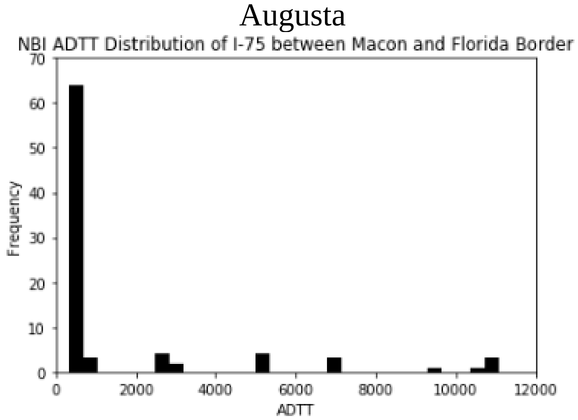
(b) NBI I-20 between Atlanta and Alabama Border



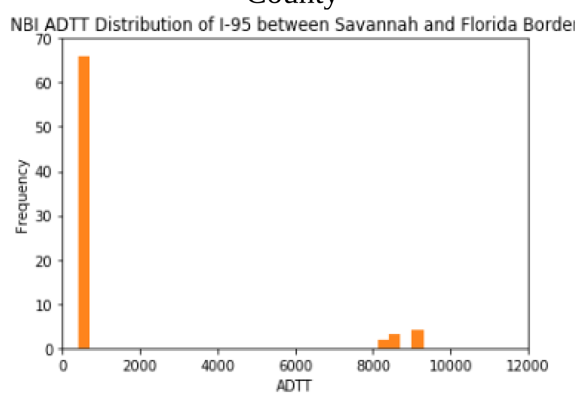
(c) NBI I-20 between Greene County and Augusta



(d) NBI I-20 between Atlanta and Greene County

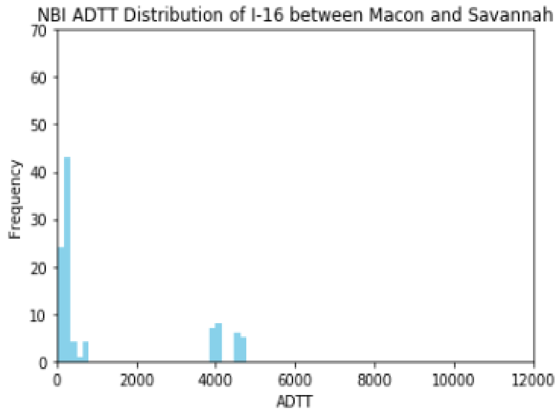


(e) NBI I-75 between Macon and Florida Border



(f) NBI I-95 between Savannah and Florida Border

Figure 36 – NBI ADTT Distribution for WIM Sites.



(g) I-16 between Macon and Savannah

Figure 36 Continued – NBI ADTT Distribution WIM Sites.

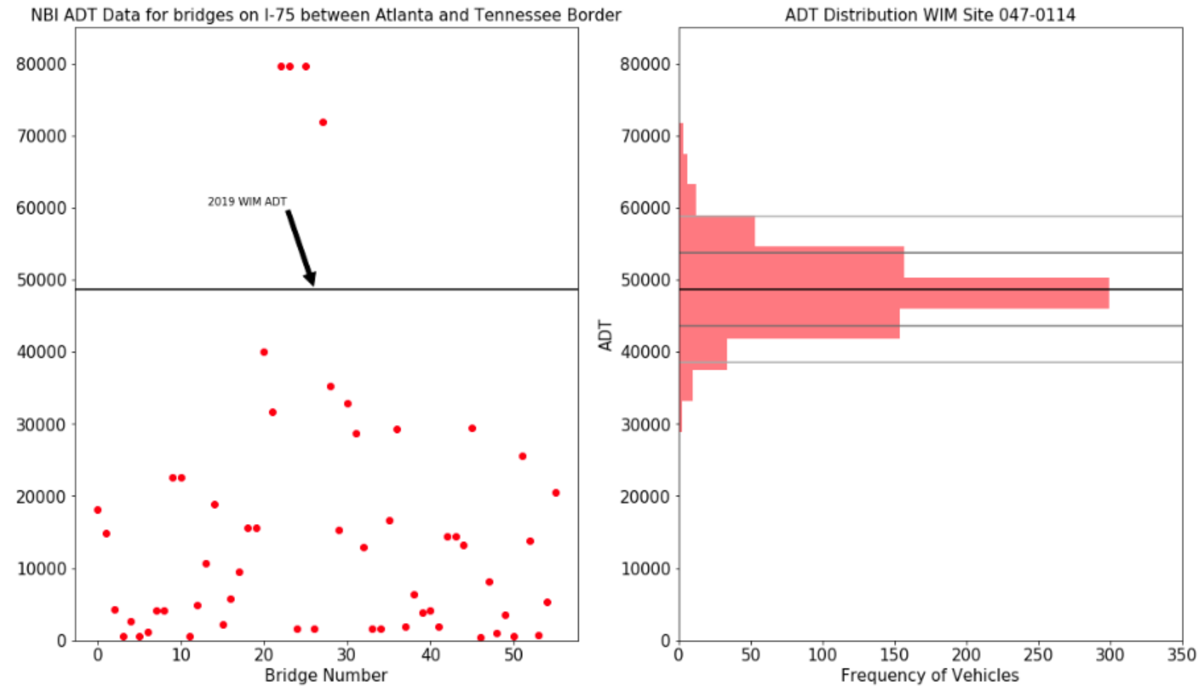
Similar to the ADT distribution, the goal is to have a normally distributed shape indicating an accurate dataset. The NBI ADTT data did not follow this trend and instead produced a heavily skewed to the right format. For each of the sites, the majority of traffic counts were below 2 thousand trucks. As displayed in Table 10 below, route I-20 (Atlanta to Greene County) recorded the highest average truck counts per day of 2,079 and was around the average for ADT data shown in Table 9. An average of only 283 ADTT was found for I-75 (Atlanta and Tennessee Border), which was surprising, following the same trend of ADT as the lowest value. This further points to a worrisome analysis as this route is expected to control truck traffic for the state. The reliability of the data must be checked so that this outcome does not contradict with current mobility. The skewness and random high ADTT frequencies give the impression that only a few bridges along the route have been updated properly, while the remaining bridges for each route have yet to be changed to the accurate value. With this assumption, it can be estimated that the route most likely experiences higher averages closer to the top ADTT values indicated in each of the figures above. The WIM data was analyzed next and provided good evidence for this in representing the true ADTT on these routes.

Table 10 – NBI ADTT Distribution Averages.

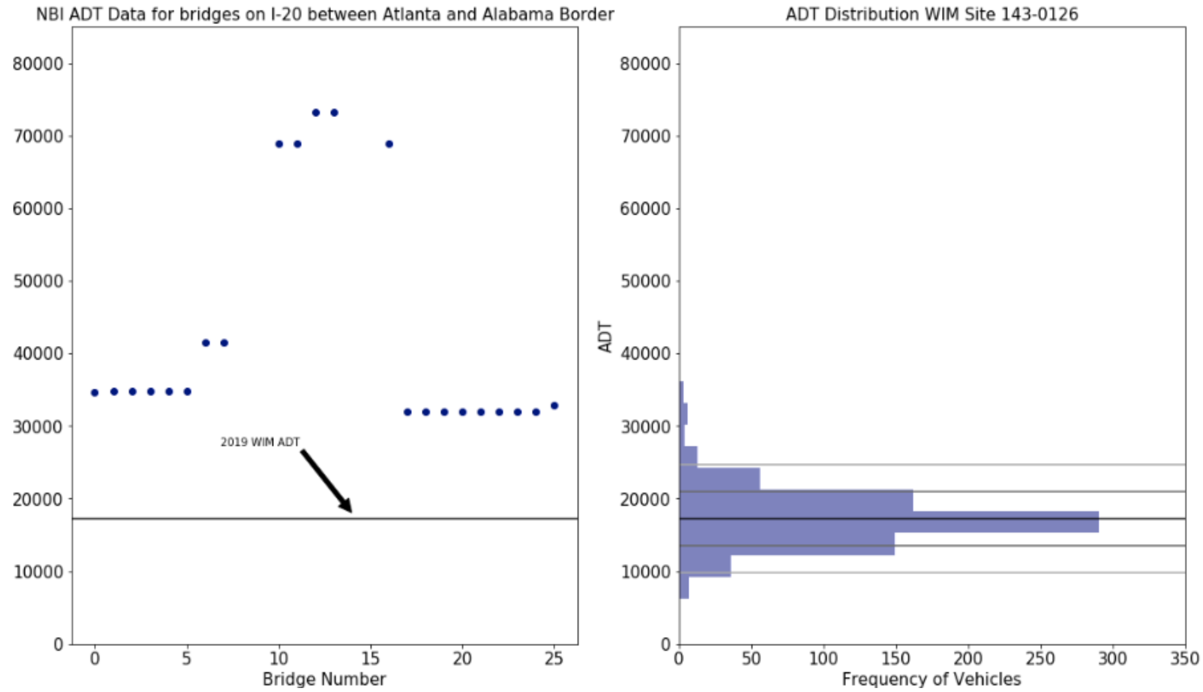
NBI ADTT Distribution Averages	
Route	ADTT
I-75 (Atlanta and Tennessee Border)	283
I-20 (Atlanta and Alabama Border)	1,803
I-20 (Greene County and Augusta)	1,040
I-20 (Atlanta and Greene County)	2,079
I-75 (Macon and Florida Border)	1,656
I-95 (Savannah and Florida Border)	1,473
I-16 (Macon and Savannah)	1,281

3.4 NBI vs WIM Comparison

The WIM data was then analyzed and transformed into histograms for a side-by-side comparison with the NBI data to observe how the daily information between the two vary. The ADT plots are presented in Figure 37 and discussed. Then, the ADTT data was assessed and demonstrated in Figure 38. The following ADT and ADTT plots are color coordinated and illustrate the NBI bridge data on the left side of each plot in a scatter plot format. The right side of each plot is the WIM data distribution turned sideways for comparison purposes. Daily count values are represented by the y axis for both NBI and WIM data. The WIM site's mean is indicated by the horizontal black line stretching across both the NBI scatter plot and WIM histogram. The additional grey and light grey lines represent the first and second standard deviations from the mean of the WIM data.

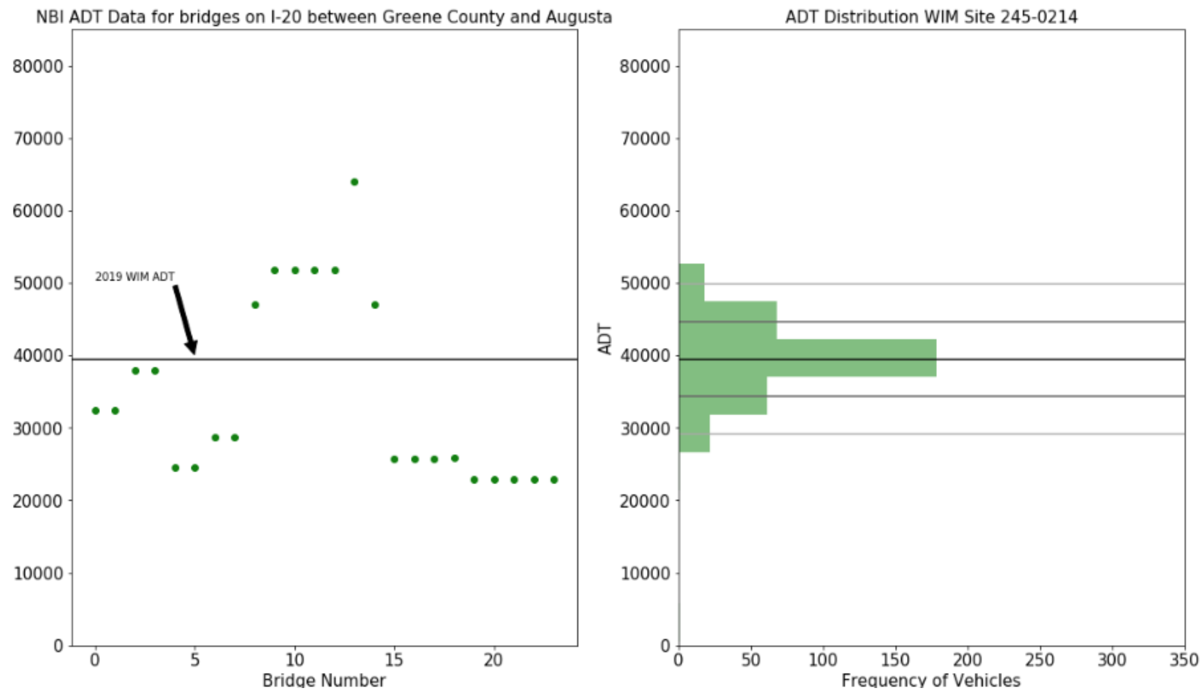


(a) WIM Site 047-0114

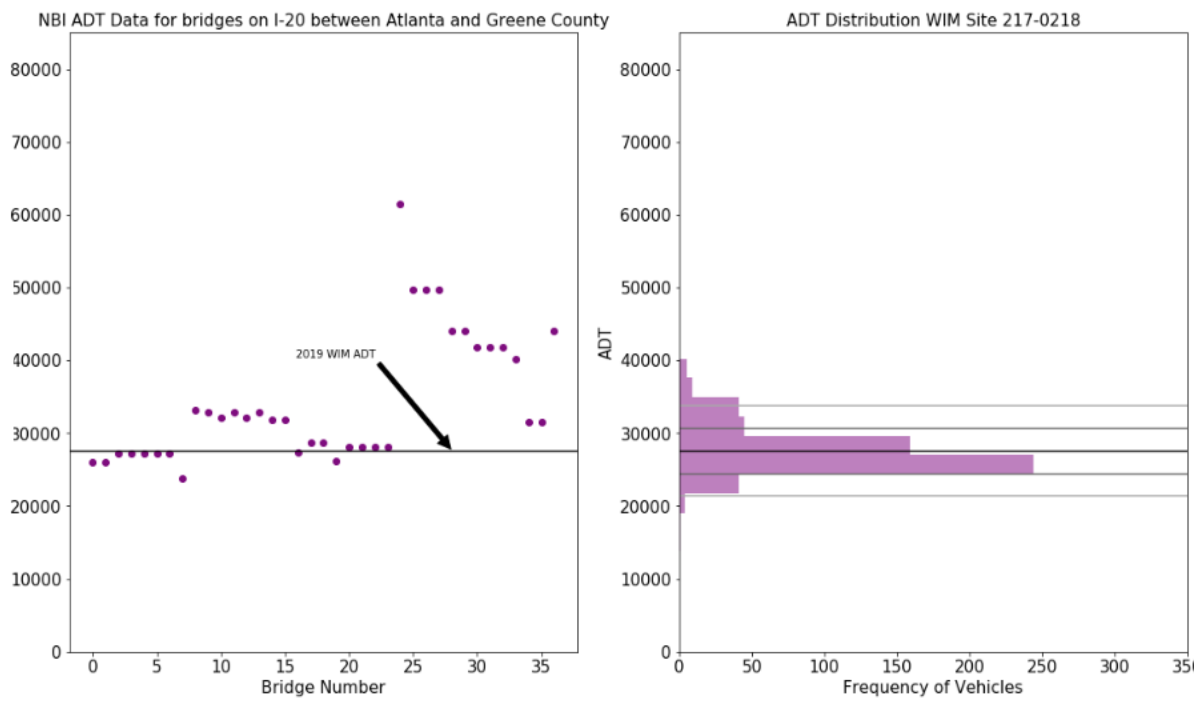


(b) WIM Site 143-0126

Figure 37 – NBI vs WIM ADT.

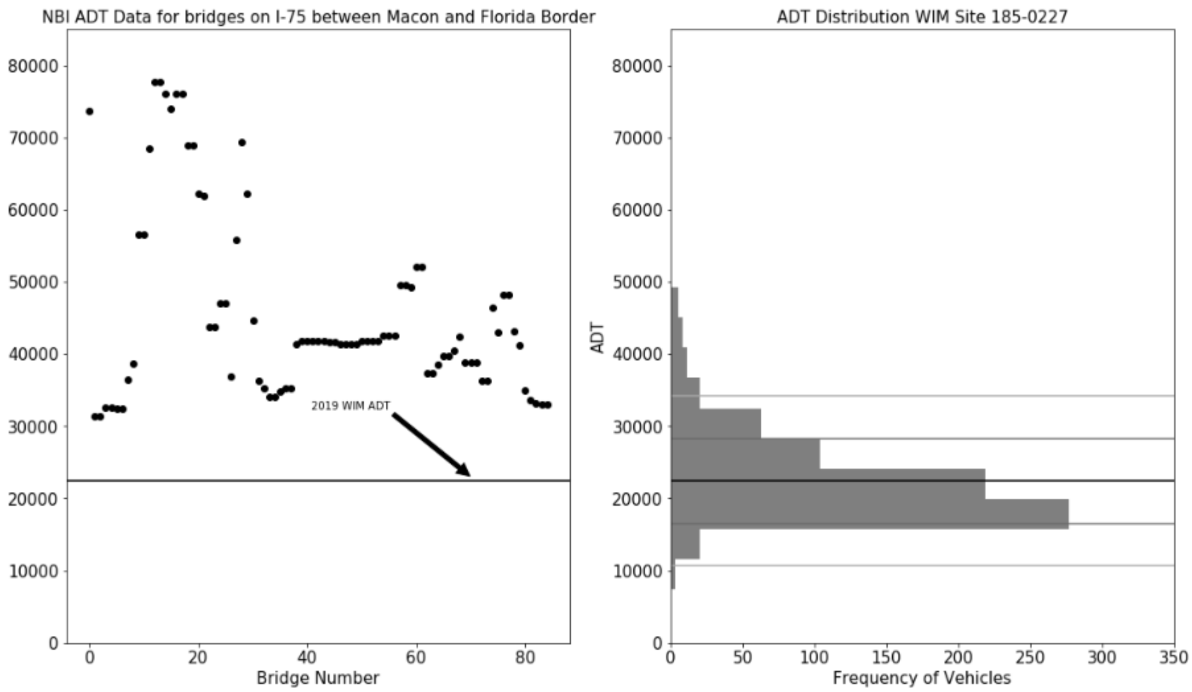


(c) WIM Site 245-0214

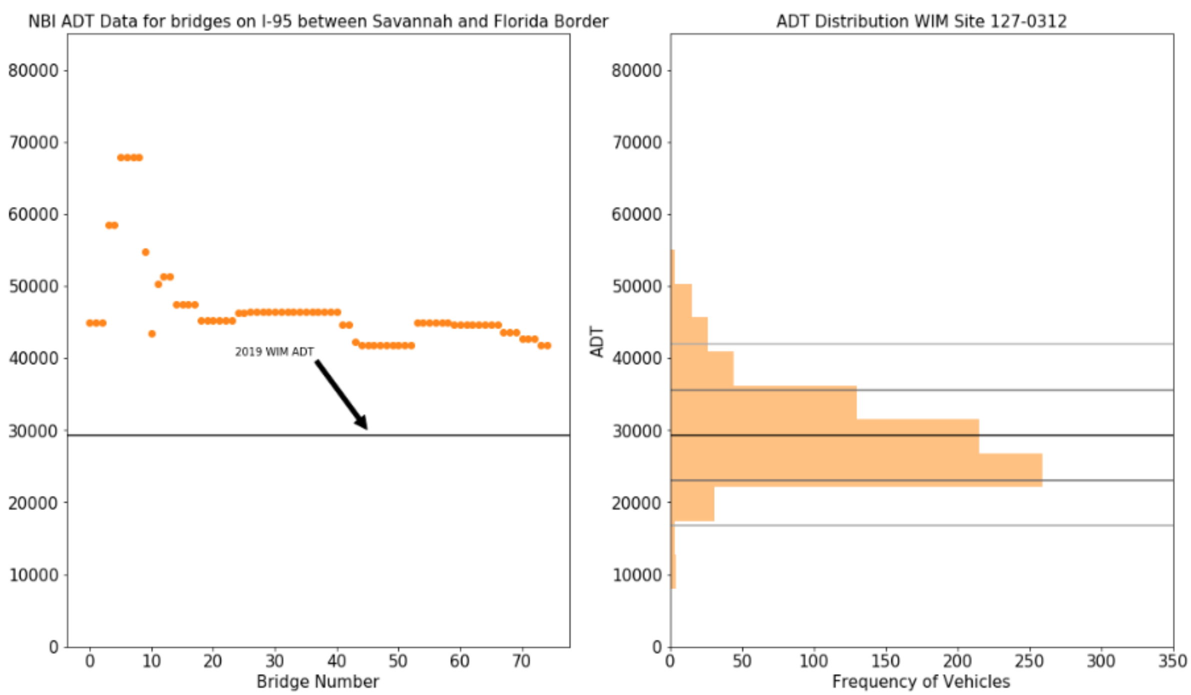


(d) WIM Site 217-0218

Figure 37 Continued – NBI vs WIM ADT.

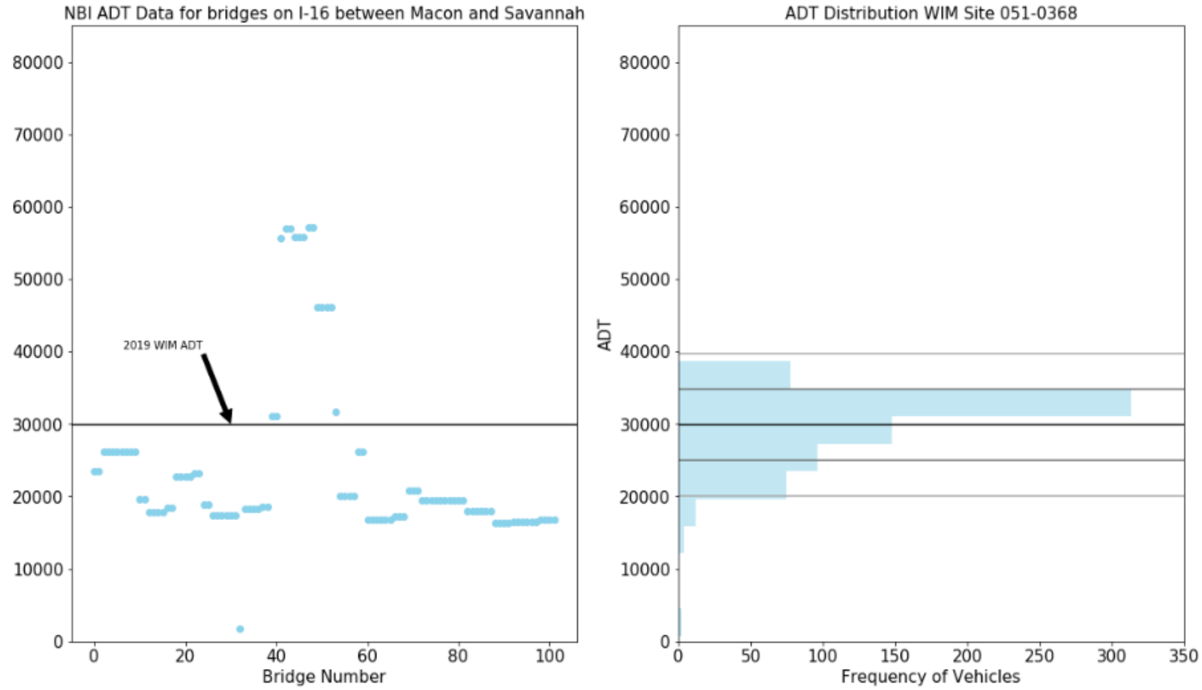


(e) WIM Site 185-0227



(f) WIM Site 127-0312

Figure 37 Continued – NBI vs WIM ADT.



(g) WIM Site 051-0368

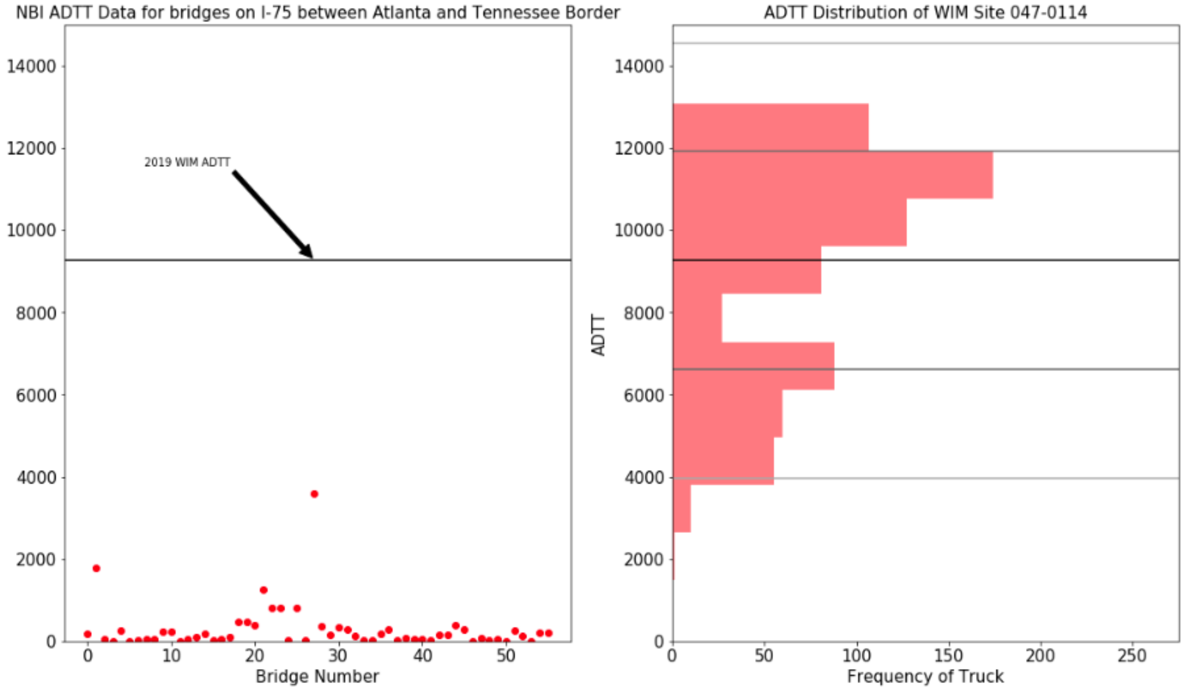
Figure 37 Continued – NBI vs WIM ADT.

By viewing the graphs demonstrated in Figure 37, the WIM distribution shown on the right side of each plot formed reasonable shapes following a normal distribution. This indicates a strong precision in the WIM technology. Some of the plots illustrated some skewness as this is most likely due to different seasons with varying flows of traffic. By looking closer into each route, the NBI and WIM data did not follow similar patterns and fluctuated depending on each route. A few sites did show some comparison as the I-20 (Atlanta and Greene County) had relatable data and decently close averages as shown in Table 11 below but did include large NBI values exceeding the WIM values. Considering the ADT averages listed in Table 11, most of the data between the two sets were extremely different as many sites produced a difference of up to approximately 32,000 vehicles a day, while the other sites had data off by around 5,000. Four of the sites had NBI data greater than the WIM ADT values with the remaining three sites being lower indicating no clear factor between the two datasets.

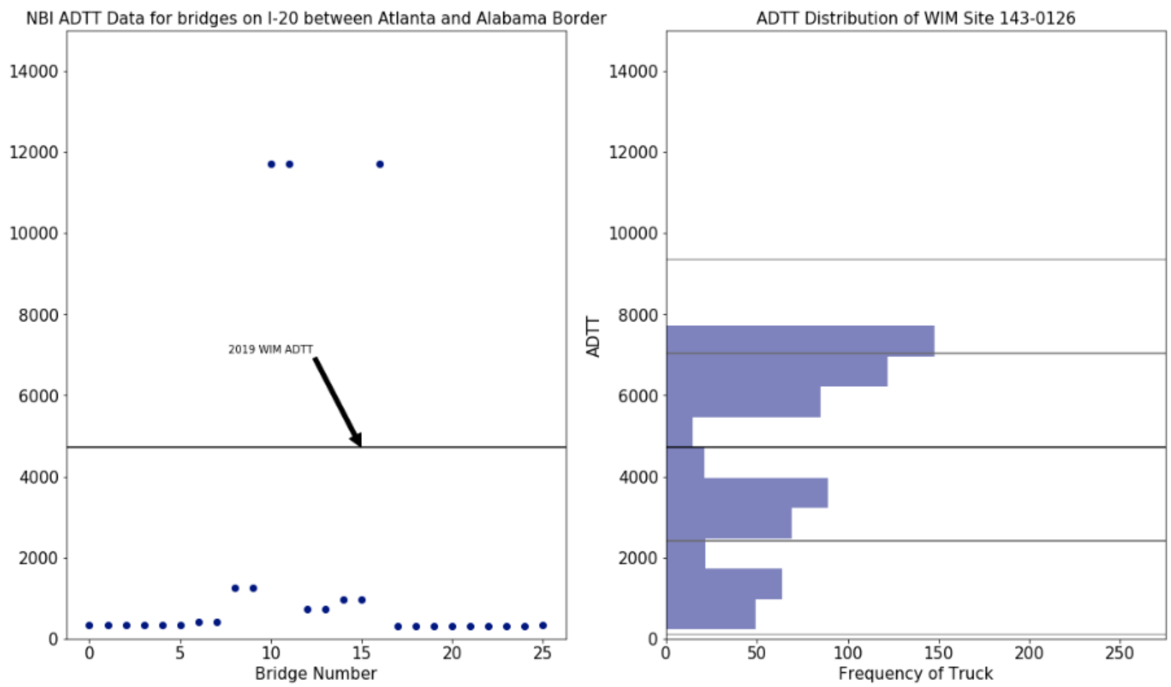
The WIM data, as predicted, for I-75 (Atlanta and Tennessee Border) matched the high-volume characteristics discussed in the literature review and produced the highest WIM ADT average of 48,675. This confirmed the NBI inconsistency as its average for this site was the lowest at 16,239. Some of the sites produced similar results downgrading NBI’s reliability and were tested further as it could not be confirmed by just viewing the plot. Therefore, a Kruskal-Wallis test was conducted to provide a single value validating the true relationship between the two. This was necessary for a few sites including I-20 (Atlanta and Greene County) and I-20 (Greene County and Augusta) although most of the sites indicated strong evidence for no correlation. To check further into this comparison, the ADTT between the two datasets was analyzed.

Table 11 - ADT Distribution Averages (NBI vs WIM).

ADT Distribution Averages (NBI vs WIM)		
Route	NBI	WIM
I-75 (Atlanta and Tennessee Border)	16,239	48,675
I-20 (Atlanta and Alabama Border)	52,980	17,273
I-20 (Greene County and Augusta)	34,605	39,492
I-20 (Atlanta and Greene County)	34,244	27,541
I-75 (Macon and Florida Border)	45,741	22,440
I-95 (Savannah and Florida Border)	46,693	29,340
I-16 (Macon and Savannah)	23,615	29,870

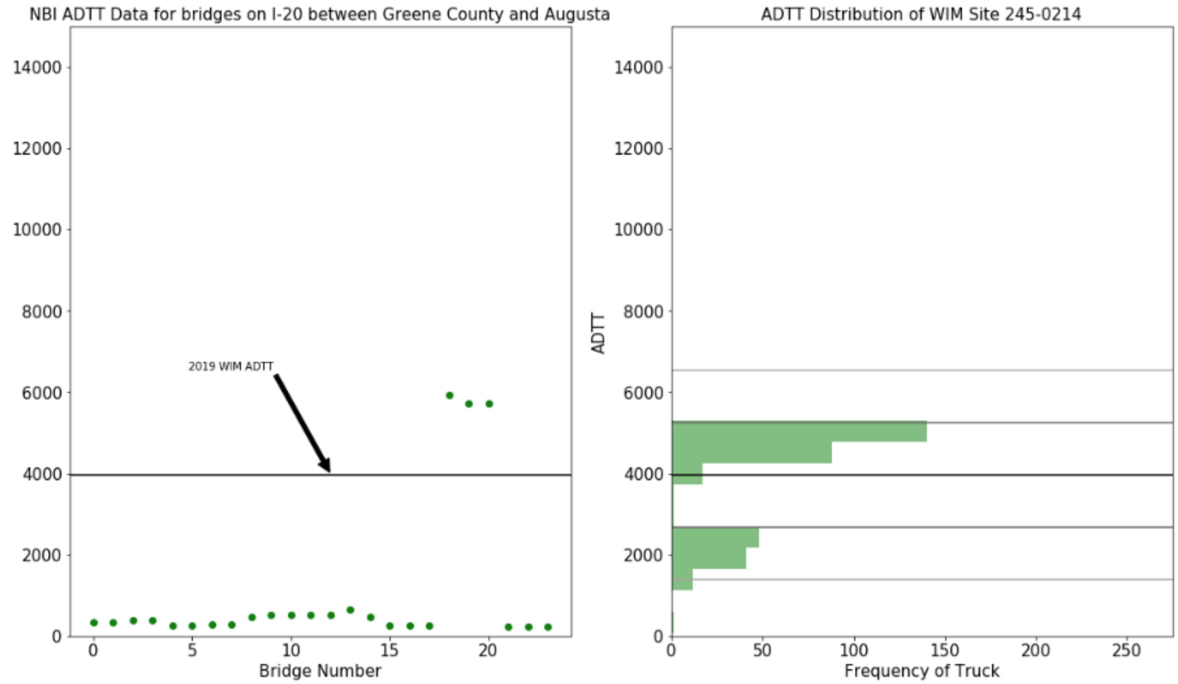


(a) WIM Site 047-0114

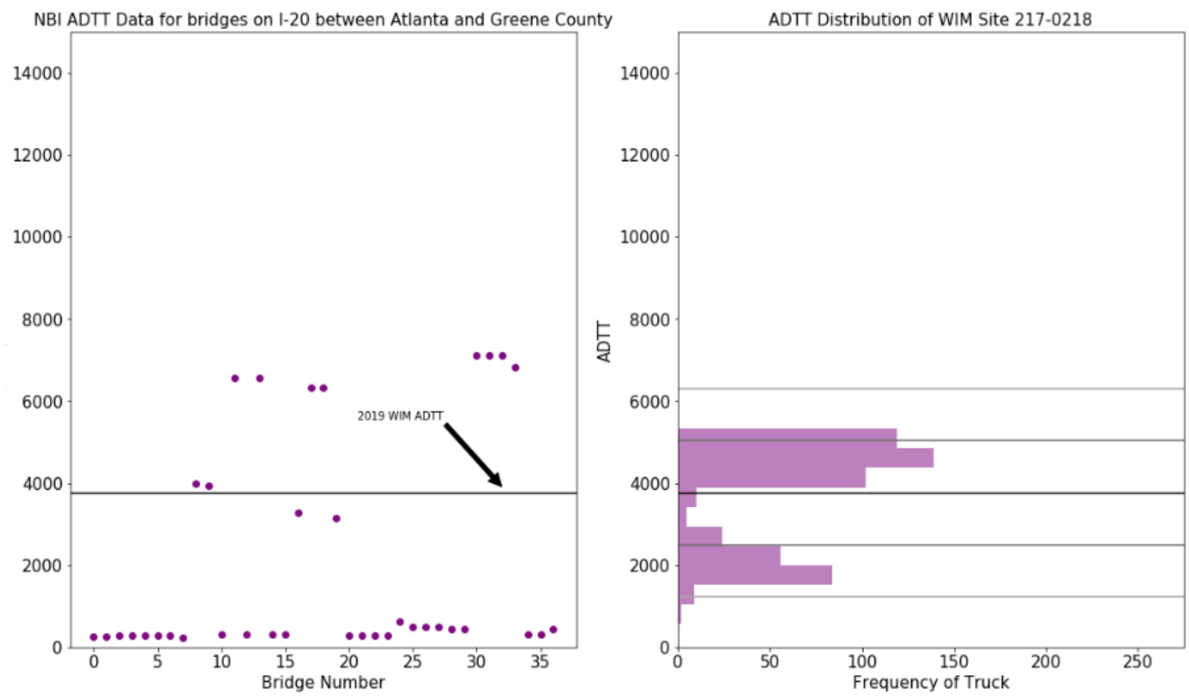


(b) WIM Site 143-0126

Figure 38 – NBI vs WIM ADTT.

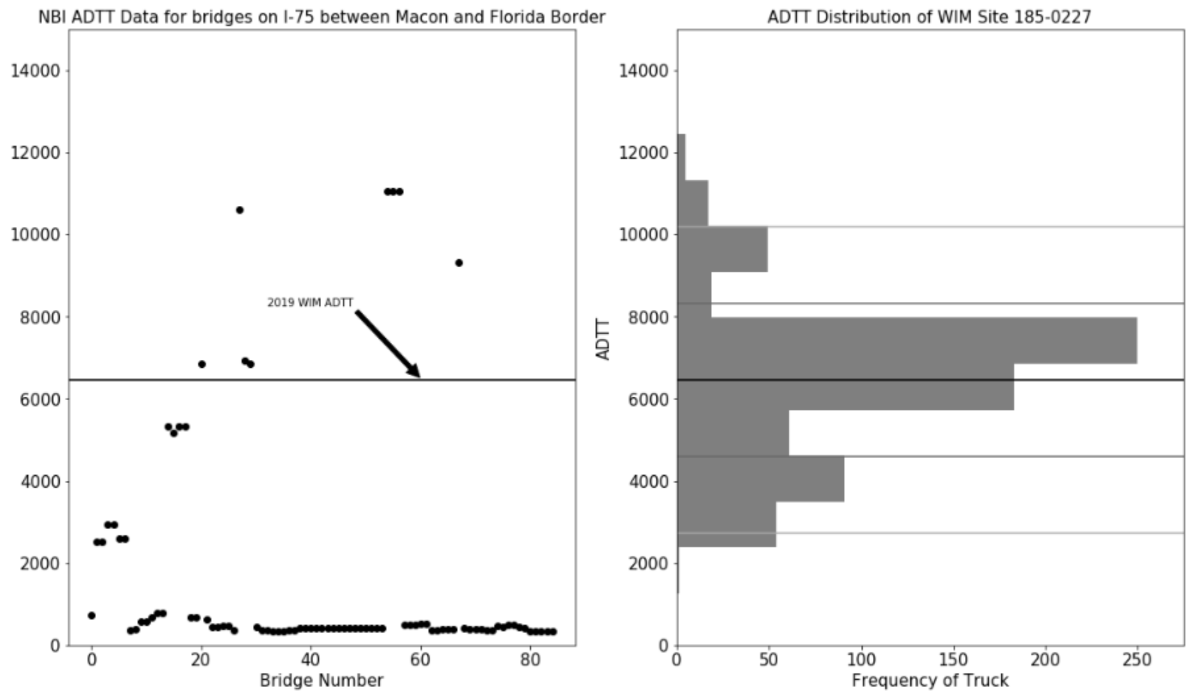


(c) WIM Site 245-0214

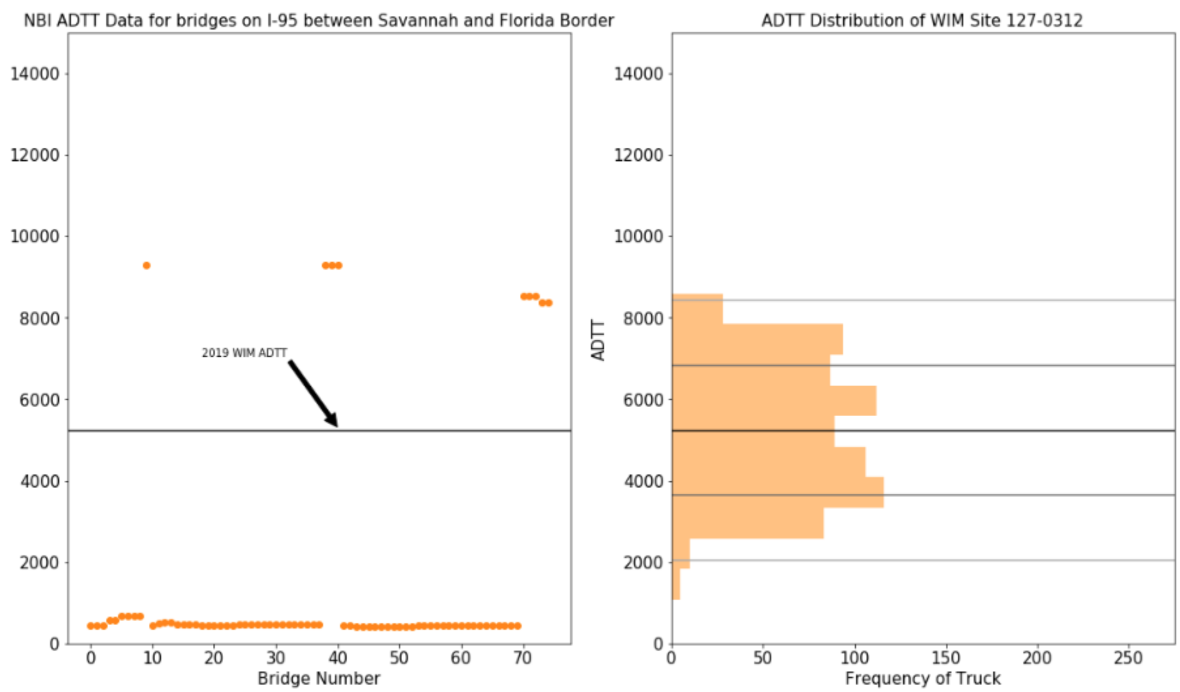


(d) WIM Site 217-0218

Figure 38 Continued - NBI vs WIM ADTT.

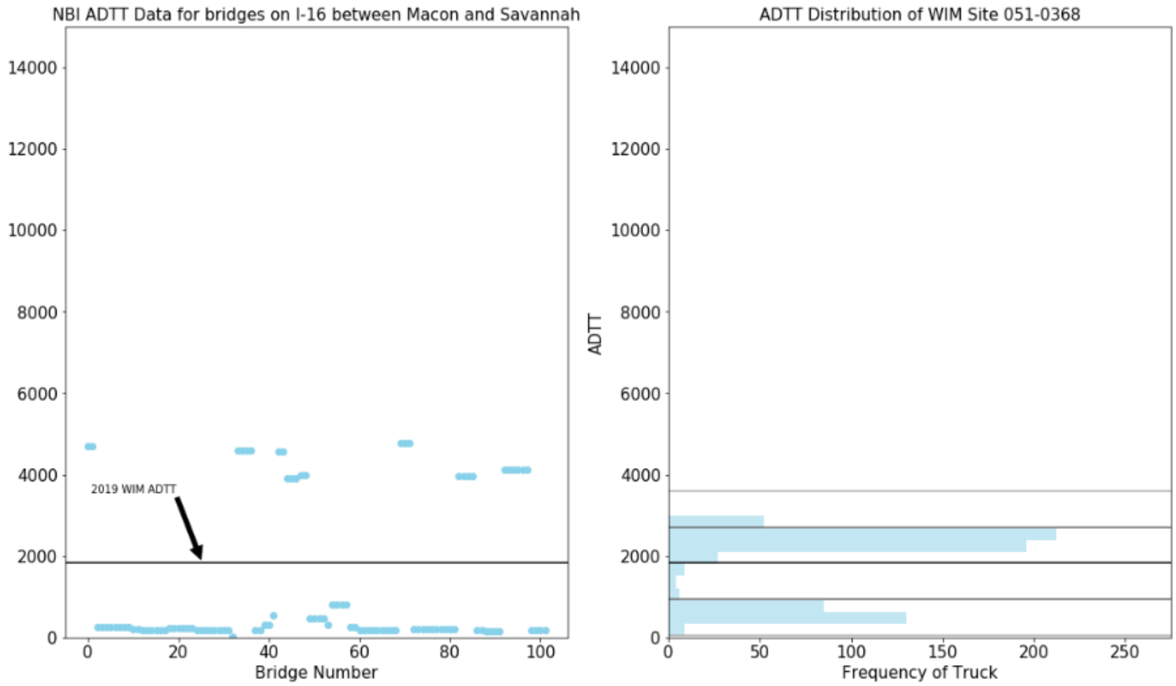


(e) WIM Site 185-0227



(f) WIM Site 127-0312

Figure 38 Continued - NBI vs WIM ADTT.



(g) WIM Site 051-0368

Figure 38 Continued - NBI vs WIM ADTT.

The ADTT information is a crucial aspect as its data controls the bridge design and evaluation standards. By observing the NBI dataset, it was seen that the ADTT percentage was inaccurately estimated and was further proven by viewing the plots displayed in Figure 38. The ADT plots produced a few scenarios that appeared slightly relatable, however, in the case of ADTT, each of the routes provided strong evidence of visually different datasets. The NBI data had results leading to the conclusion of being underestimated by a large margin. Table 12 shows the ADTT averages. By looking at this data, only one route in I-16 Macon to Savannah had averages within 500 trucks per day while most routes were off by double to 30 times the NBI ADTT. The largest WIM ADTT was, as assumed, in I-75 (Atlanta to Tennessee Border) with an average of 9,267 trucks per day with the lowest being I-16 (Macon to Savannah) as the research suggested. A Kruskal-Wallis test was performed on this analysis to numerically confirm that inaccurate data exists in the NBI dataset.

The WIM ADTT data was expected to follow a normal distribution compared to what the ADT experienced; however, a bimodal shape was produced. Due to the fact that every route produced this shape, it was assumed consistent. The main source of the bi-modal distribution is due to the change of seasons throughout the year. The flow of truck traffic might be different in the summer compared to the winter with its holiday shipping and increase in distribution.

Table 12 - ADTT Distribution Averages (NBI vs WIM).

ADTT Distribution Averages (NBI vs WIM)		
Route	NBI	WIM
I-75 (Atlanta and Tennessee Border)	283	9,267
I-20 (Atlanta and Alabama Border)	1,803	4,716
I-20 (Greene County and Augusta)	1,040	3,970
I-20 (Atlanta and Greene County)	2,079	3,762
I-75 (Macon and Florida Border)	1,656	6,453
I-95 (Savannah and Florida Border)	1,473	5,234
I-16 (Macon and Savannah)	1,281	1,824

With this data, Table 13 shows the truck percentages calculated for additional exploration. This table provided the most effective indication of improper data being presented in the NBI as the WIM truck percentages are significantly larger. The I-16 (Macon to Savannah) route showed less than a 1 percent difference, but the remaining 6 routes are ominously different. It can be stated that I-75 (Macon to Florida Border) had the highest percentage of trucks on its route at approximately 29 percent with the second largest percentage at 28 percent for the I-20 (Atlanta to Alabama Border) route. This illustrated the importance of WIM data as many of the truck percentages per route were underestimated.

Table 13 - Truck Percentage (NBI vs WIM).

Truck Percentage (NBI vs WIM)		
Route	NBI	WIM
I-75 (Atlanta and Tennessee Border)	1.74%	19.00%
I-20 (Atlanta and Alabama Border)	3.40%	27.30%
I-20 (Greene County and Augusta)	3.00%	10.06%
I-20 (Atlanta and Greene County)	6.07%	13.66%
I-75 (Macon and Florida Border)	3.62%	28.76%
I-95 (Savannah and Florida Border)	3.15%	17.84%
I-16 (Macon and Savannah)	5.42%	6.11%

3.5 Kruskal-Wallis Results

This chapter numerical quantified the correlation between the NBI and WIM counts. A Kruskal-Wallis test was conducted in place of a t-test because the data did not satisfy the normality requirement. Additionally, the sample size was different. The results for ADT and ADTT are displayed in Table 14 and Table 15, respectively. The tables provide a p-value that was calculated through a python code. This p-value was the deciding factor (or significance level) on being able to reject the null hypothesis. The null hypothesis stated that the two datasets are similar. It can be rejected if the p-value is lower than the significance interval of 95 percent. Therefore, if the p-value is less than 0.05, then the hypothesis was rejected and concluded as there being no significant evidence that the datasets were correlated at a 95 percent confidence. If the p-value was greater than 0.05, there was evidence that they were similar.

Table 14 - ADT Kruskal-Wallis Results.

ADT Kruskal-Wallis Results		
Route	P-Value	Null Rejected?
I-75 (Atlanta and Tennessee Border)	1.30e-26	Yes
I-20 (Atlanta and Alabama Border)	6.61e-18	Yes
I-20 (Greene County and Augusta)	3.30e-3	Yes
I-20 (Atlanta and Greene County)	1.44e-8	Yes
I-75 (Macon and Florida Border)	1.18e-47	Yes
I-95 (Savannah and Florida Border)	1.95e-41	Yes
I-16 (Macon and Savannah)	1.50e-26	Yes

Table 15 - ADTT Kruskal-Wallis Results.

ADTT Kruskal-Wallis Results		
Route	P-Value	Null Rejected?
I-75 (Atlanta and Tennessee Border)	9.72e-36	Yes
I-20 (Atlanta and Alabama Border)	3.85e-10	Yes
I-20 (Greene County and Augusta)	1.02e-9	Yes
I-20 (Atlanta and Greene County)	4.63e-7	Yes
I-75 (Macon and Florida Border)	2.45e-35	Yes
I-95 (Savannah and Florida Border)	1.93e-27	Yes
I-16 (Macon and Savannah)	3.00e-13	Yes

The results for the Kruskal-Wallis test provided numerical evidence in terms of a correlation factor between the NBI and WIM traffic usage data. Only one route produced a p-value close to the 0.05 significance level for ADT at 0.0033 in the I-20 (Greene County to Augusta) route as seen in Table 14. However, this route as well as the other 6 routes were below the significance level, and thus the null hypothesis is rejected. As for ADTT, the p-values were all considerably low especially the I-75 (Atlanta to Tennessee Border) at a value of 9.72e-36 as seen in Table 15, concluding the rejection of the null hypothesis as well. Therefore, this test outcome confirmed the assumption that the NBI and WIM datasets were indeed different.

3.6 Specific Site Analysis

As mentioned in the methodology, two sites were investigated further due to their heavy traffic and importance to the state. WIM Site 047-0114 collecting data from I-75 between Atlanta and the Tennessee border produced the largest amount of ADT and ADTT data in the state at 48,675 and 9,267 compared to the other WIM sites studied. Additionally, Georgia is home to the Savannah port that produces over 5,000 departing trucks each day. These trucks deliver cargo all across the state driving over a multitude of bridges along the way to their destinations. Since these two locations are significant in the bridge analysis of Georgia, their maximum axle loads and total weights were calculated.

The I-75 route only consists of one WIM site while the Savannah port is home to six WIM sites. However, due to inaccuracies that may exist with old sensor technology, only four of the WIM sites were able to be examined. The plots expressed in Figure 39 below illustrate the distribution of the maximum axle loads per truck recorded in the 2019 WIM data for these two sites. Figure 40 presents the total weight per truck distribution of the 2019 WIM data at these two sites. The red plots represent the I-75 route while the frequencies of the 4 Savannah sites are combined by the colors of blue, orange, green, purple in the following figures.

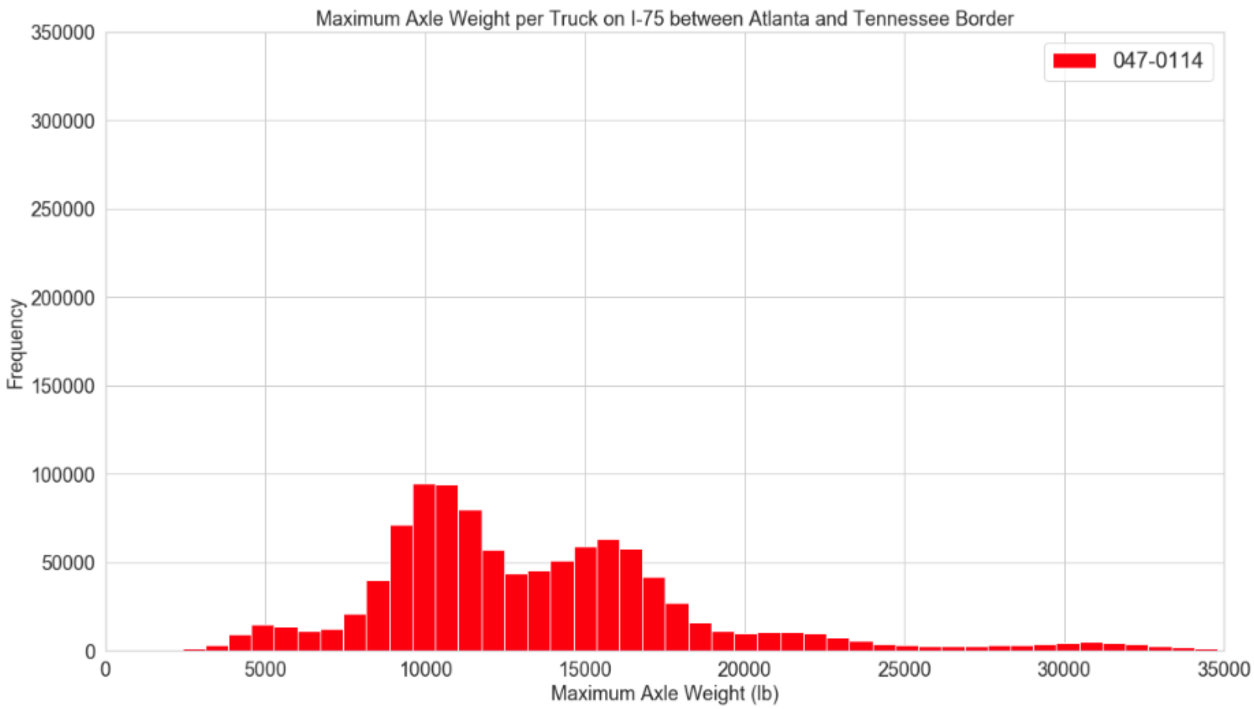


Figure 39a – Maximum Axle Weight per Truck of I-75 North of Atlanta.

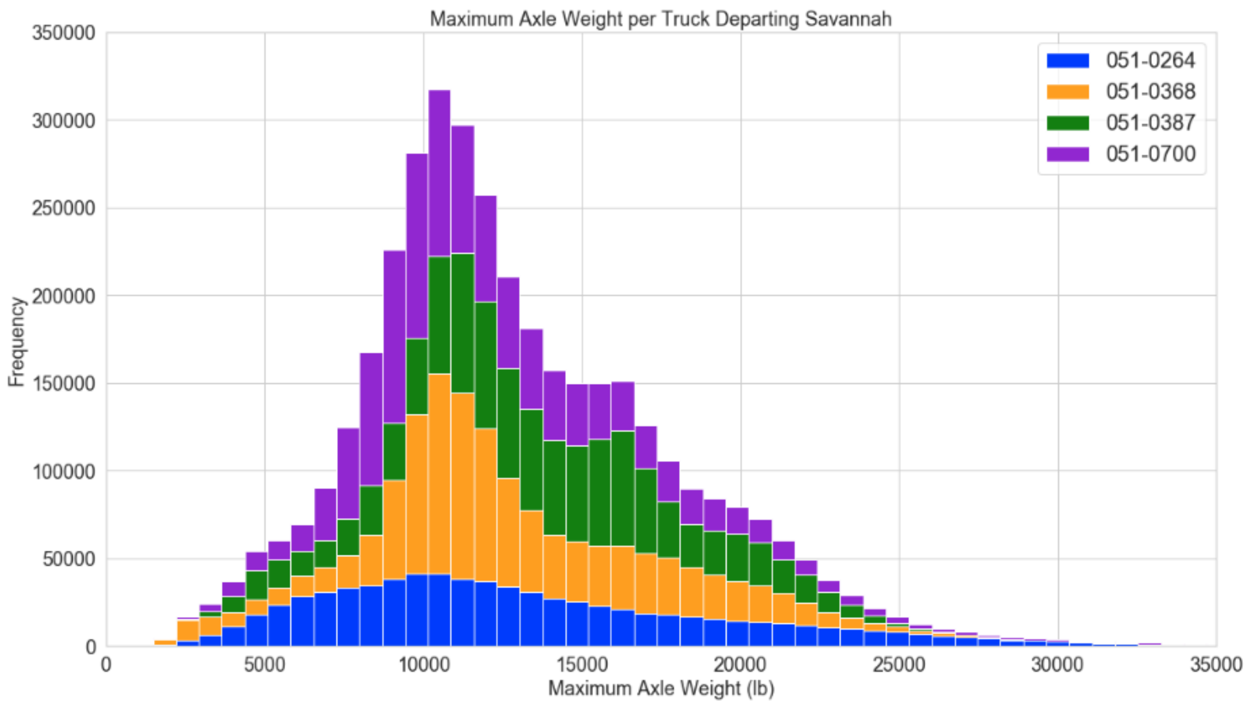


Figure 39b – Maximum Axle Weight per Truck of Savannah Port.

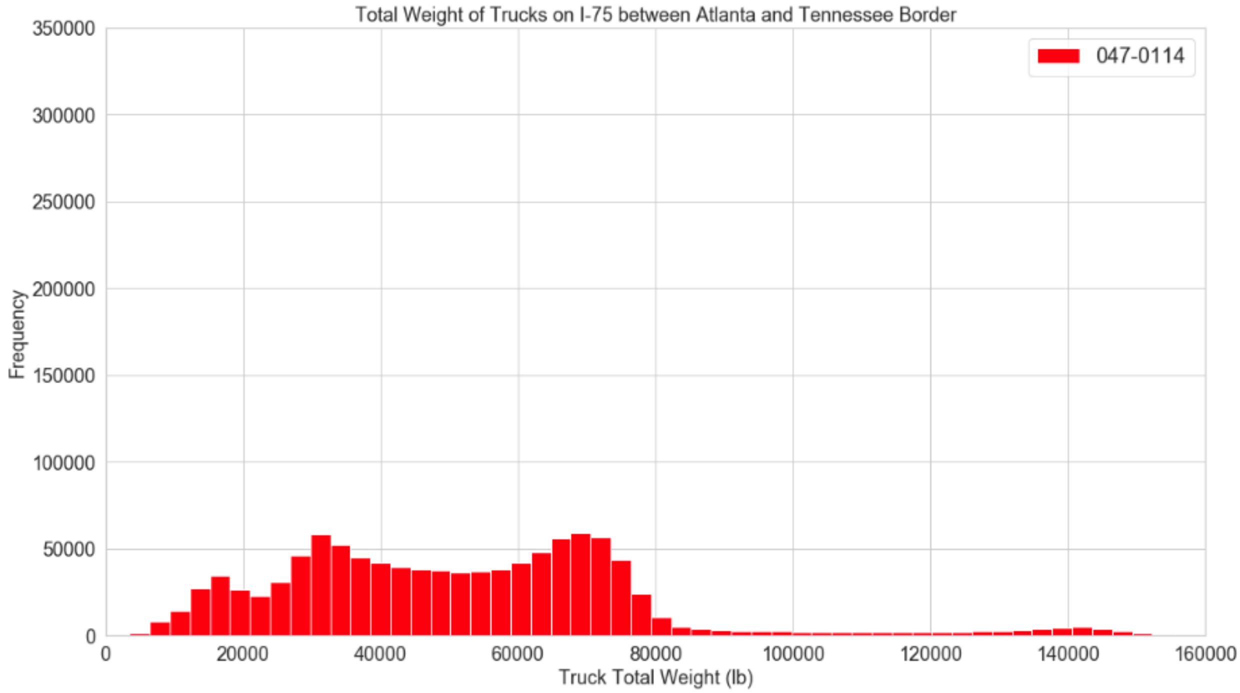


Figure 40a - Total Weight of Trucks of I-75 North of Atlanta.

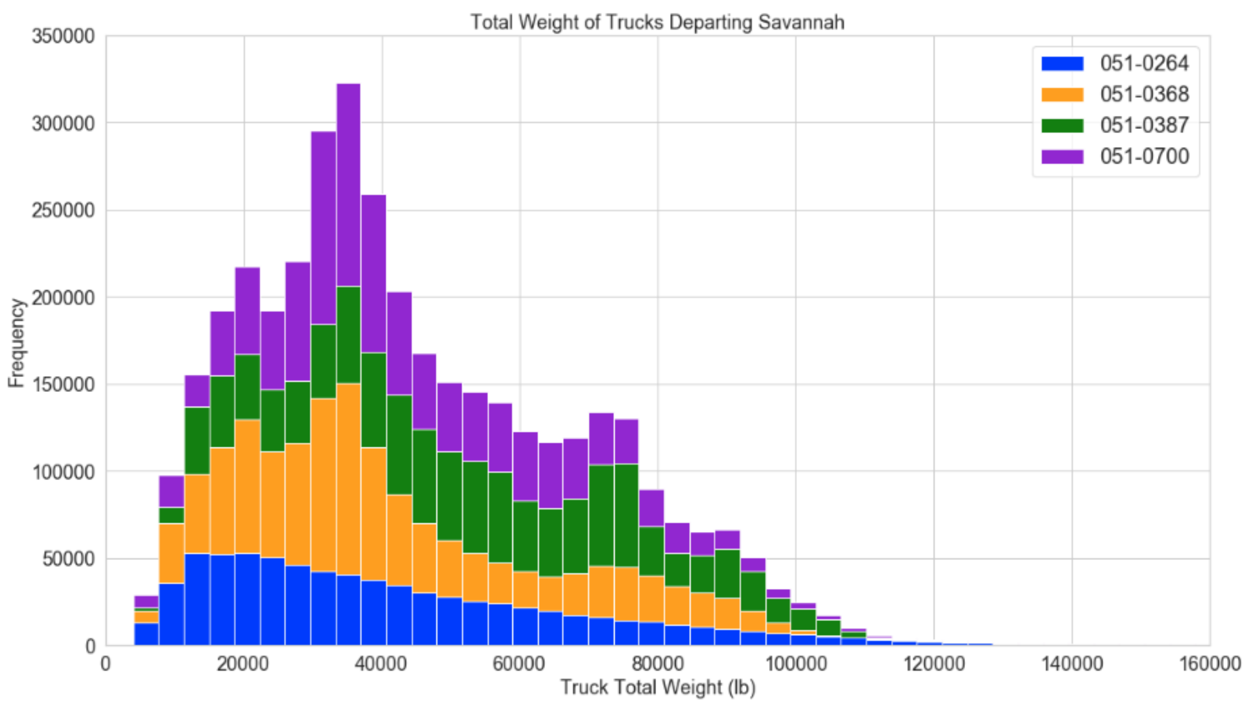


Figure 40b – Total Weight of Trucks of Savannah Port.

Figure 39 shows that the maximum axle per truck on each site ranges from approximately 2,000 lb to 35,000 lb. Both sites had similar shapes with a major peak loacted around the 10,000 lb to 13,000 lb range and a minor peak near the 17,000 lb mark. This can be defined as a bimodal distribution and indicated two different sets of axle data. One set is light weight estimated to be below the 14,000 lb weight and included the majority of trucks visited by these two sites. The second set was not as frequent and is classfied as covering the heavier trucks.

The total weights illustrated in Figure 40 for each of the two sites followed similar shapes as well. This time, the plots each had three peaks clasifying it as a multi-modal. However, the second peak for the Savannah port was its maximum frequency while the I-75 route had similar frequencies for the second and third peak. These graphs breaked the trucks total weight into three different sections of lightweight, midweight, and heavyweight trucks. Both plots were skewed to the right ranging from around 5,000 lb to approximately 150,000 lb for the I-75 route.

It should be stated that these plots excluded outlier and had a few trucks beyond the scope indicated in the range. For the maximum axle weights, both sites produced the same maximum value at 72,240 lb for a single axle. This large value was concerning considering the maximum axle weight per Formula B is 20,000 lb. For the total gross vehicle weight, the I-75 route presented a maximum total weight of 292,398 lb while the Savannah port produced a larger value in 366,717 lb. Again, these values well exceeded the Formula B requirements of 80,000 lb and must have required a special permit for travel. This is assuming the WIM data to be fully accurate and should be stated that these high values could be due to increased speeds/temperatures or other factors causing an increase in its true load. However, these values are concerning and should be analyzed further for safety reasons.

These sites offered very similar and interesting results, created specific sections within the loads, and produced large maximum values for both axle and total weight. These two sites were of importance to the state of Georgia; however, increased truck weights observed in the data must be further investigated. Chapter 4 investigated the live load demands of Georgia bridges through WIM analysis.

3.7 Conclusion

In conclusion, the NBI ADT and ADTT counts were examined and compared to the 2019 WIM data gathered. It was determined that the ADT and ADTT NBI data did not follow a normal distribution. The WIM ADT, on the other hand, followed a normal distribution but had a bi-modal shape for ADTT. This bimodal shape was attributed to the fluctuations in truck traffic due to the seasons and holidays. Additionally, the two datasets were compared and indicated no correlation as proven by the Kruskal-Wallis test rejecting the null hypothesis in every case studied. The ADT plots showed some areas of similarity but were still significantly different whereas the ADTT plots demonstrated no relation between the two. The NBI data was much lower than the WIM data gathered. Further analysis on vehicle weight distributions was provided defining sites on I-75 North of Atlanta and the Savannah port.

It is concluded based on the findings of this chapter that WIM data provided beneficial information on bridge usage as it captured every vehicle traveling on its roads. It additionally indicated the need for an improvement in NBI data as it was not consistent with the WIM data. This is an issue as it is misleading to engineers in design and evaluation assuming the traffic flow is lower than it actually is as proven by WIM. The WIM information gathered should be utilized to update NBI annual information increasing its reliability and removing the uncertainties within the dataset. The utilization of WIM data has created a clearer image of what goes across Georgia bridges and should be expanded to improve public safety in the state.

4. LIVE LOAD DEMAND ENVOLOPE

4.1 Introduction

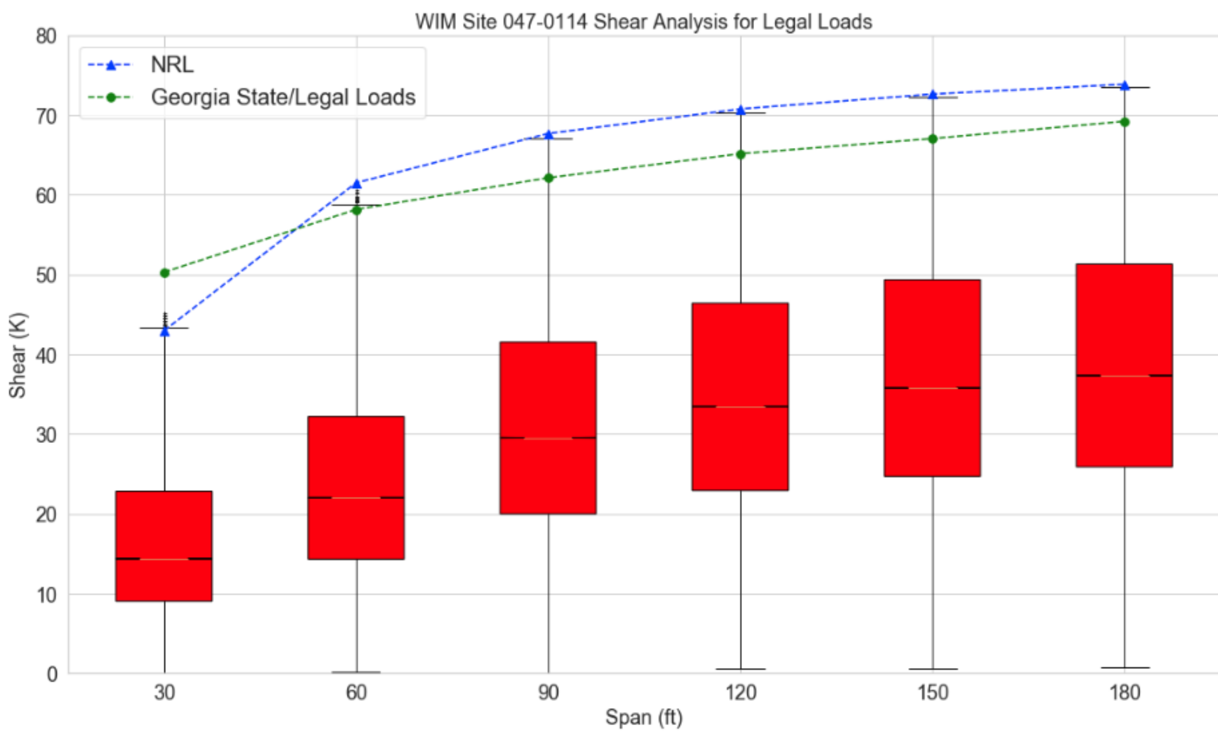
In Chapter 4, a truck live load envelope was developed to classify the bridge demand across the state on major interstates. In order to determine this, the maximum shear and moment demands were calculated per truck and organized for two weight categories: 1) trucks that meet and 2) do not meet the Formula B requirements. These results were then compared with the GDOT condition assessment defined in Chapter 1.2.8. This assessment focused on the HL-93 design load and Georgia state/legal trucks while the NCHRP Report 575 provided an additional assessment of the NRL truck. Once the analysis was assembled, the live load demand was evaluated. An output on the condition segment of the analysis was provided in Chapter 4.3 and is tentative because not enough traffic data was available for a full study. Additionally, the probabilities of multiple trucks were examined and compared with the original Ontario truck data from 1975.

4.2 Live Load Analysis

4.2.1 NRL vs Georgia State/Legal Loads

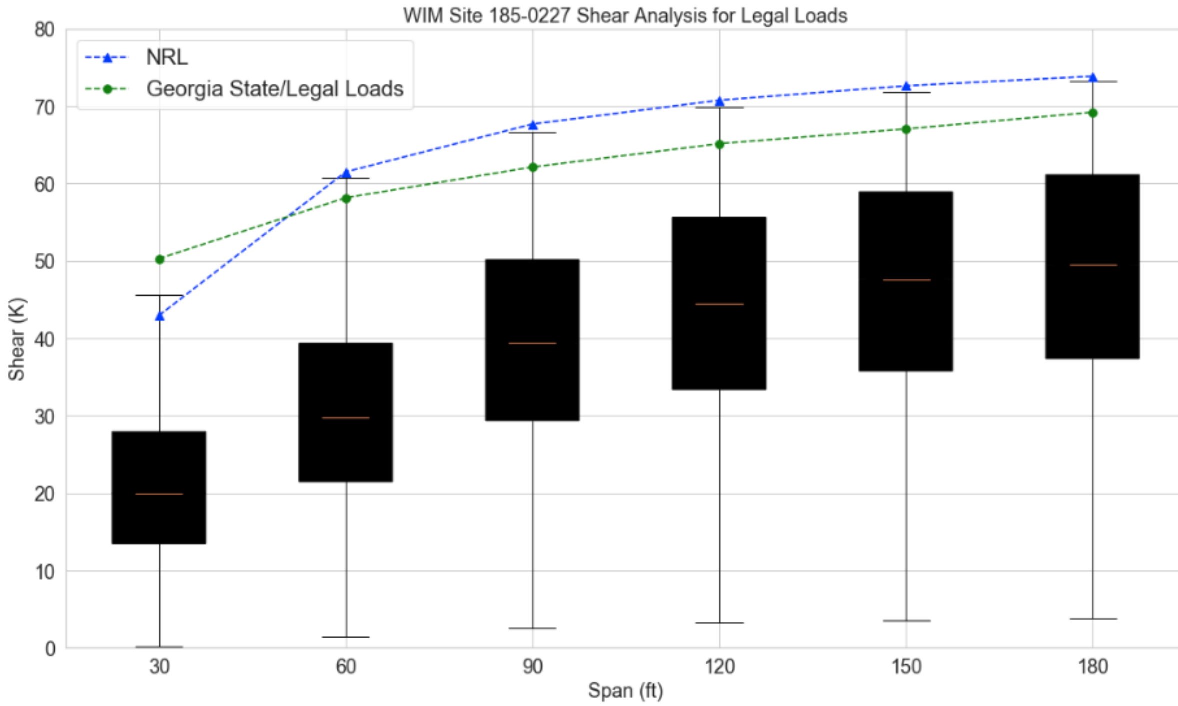
The truck live load demand of each WIM site was studied and compared with condition assessment techniques to determine its reliability and safety. As explained in the methodology section, the Georgia state/legal trucks and NRL truck maximum shear and moment values were determined and compared with the maximum shear and moment demands evaluated for 11 WIM sites. The trucks being examined at these WIM sites meet the Formula B requirements stating that the trucks are state legal (not needing an overweight vehicle permit to travel).

According to the GDOT condition assessment in the evaluation process of deeming a bridge for posting, only the Georgia state/legal loads are necessary for bridge load. However, the NCHRP Report 575 published in 2016 states that a new configuration of NRL is essential as trucks have transformed over the years. Therefore, the two truck models will be analyzed alongside the WIM Formula B data in order to see if the NRL is necessary for Georgia bridges. If so, this would require an update in the bridge load rating evaluation. The live load shear force results of this study are presented in Figure 41 below. A total of 11 figures are shown for each of the WIM sites and are color coordinated with the WIM sites in Figure 33. Each plot illustrates the WIM maximum shear per Formula B truck distributed through box plots at each of the tested span lengths. The maximum shear values for the NRL are presented by the dotted blue line while the maximum shear values for the Georgia state/legal loads are shown by the dotted green line per span length.

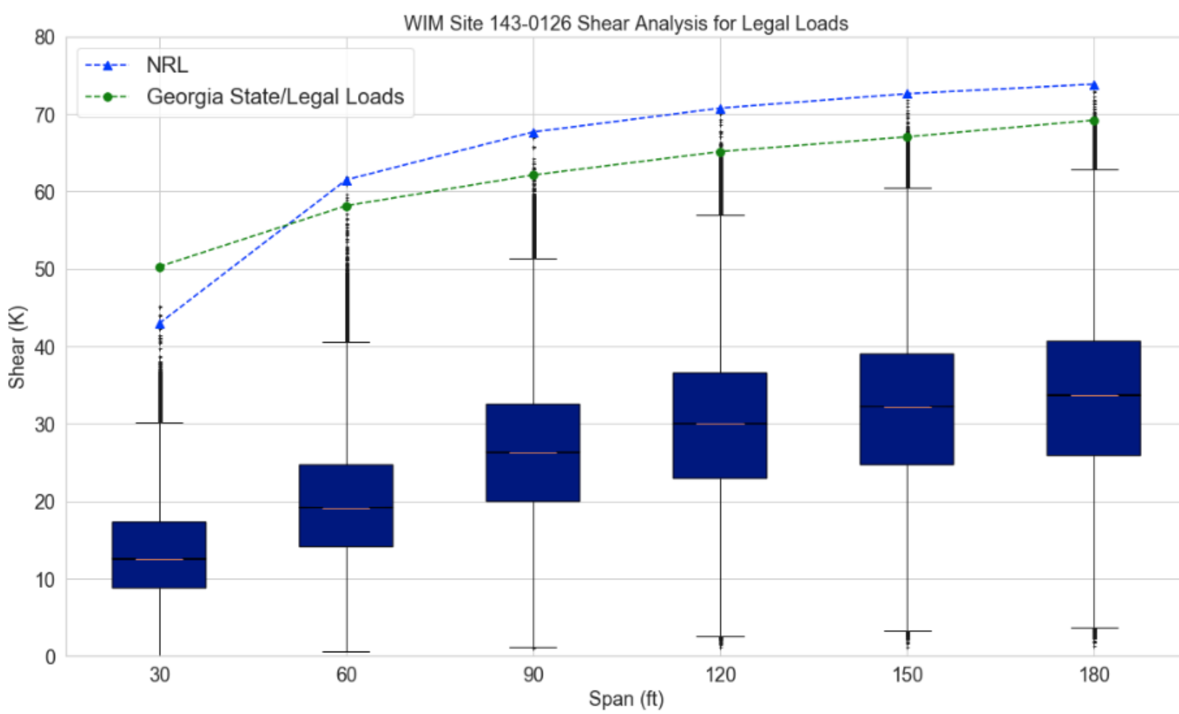


(a) WIM Site 047-0114

Figure 41 – Shear Analysis for Trucks Meeting Formula B.

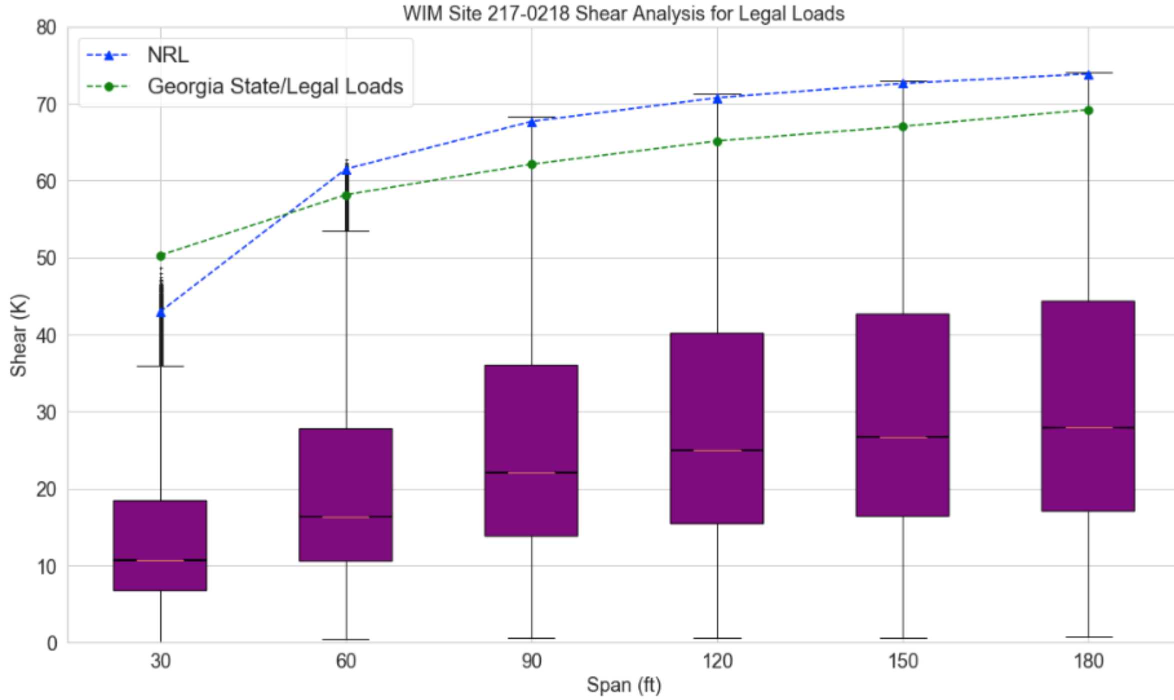


(b) WIM Site 185-0227

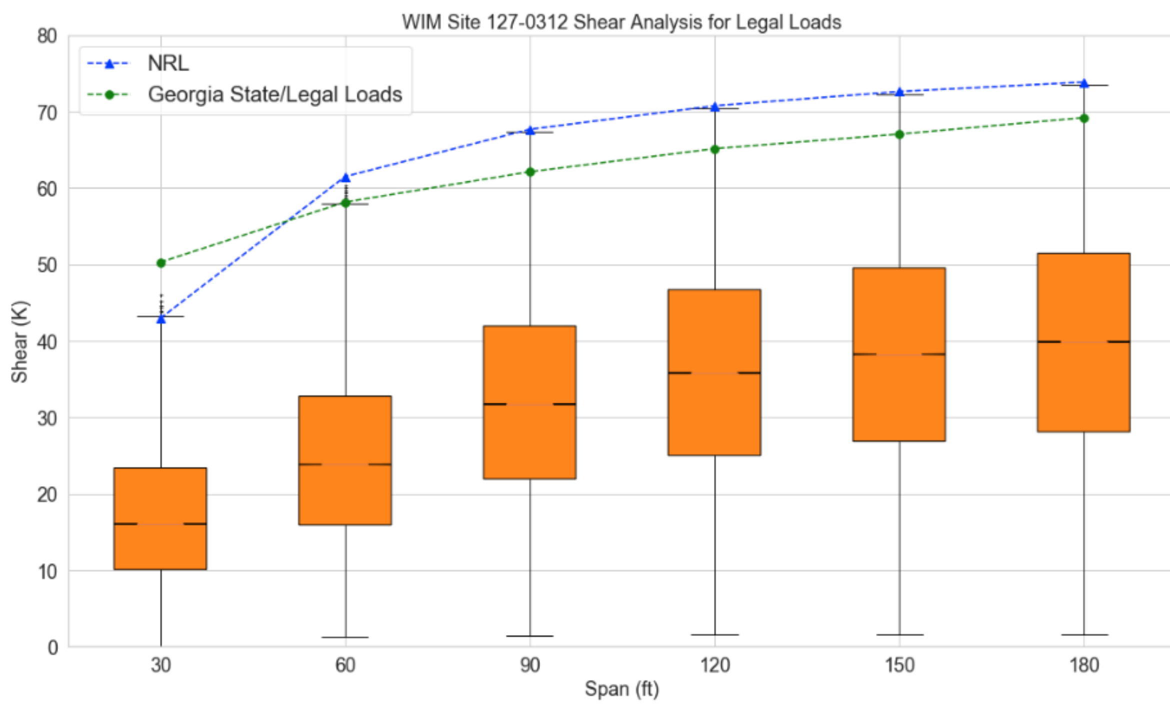


(c) WIM Site 143-0126

Figure 41 Continued – Shear Analysis for Trucks Meeting Formula B.

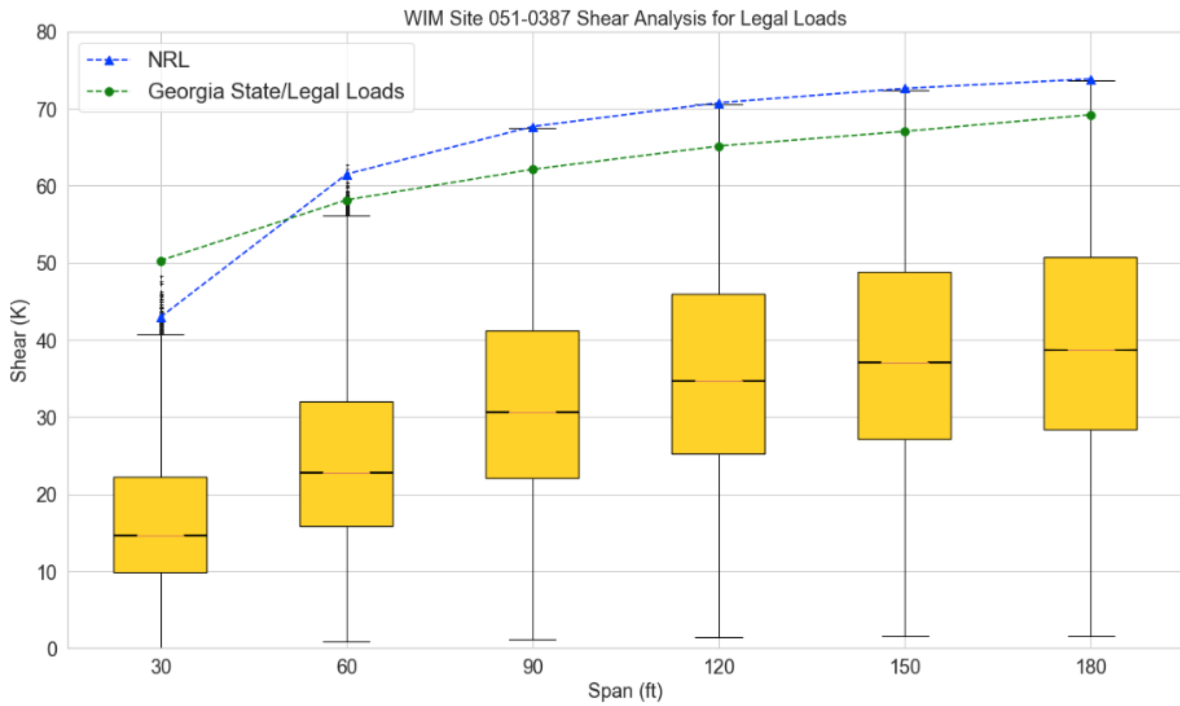


(d) WIM Site 217-0218

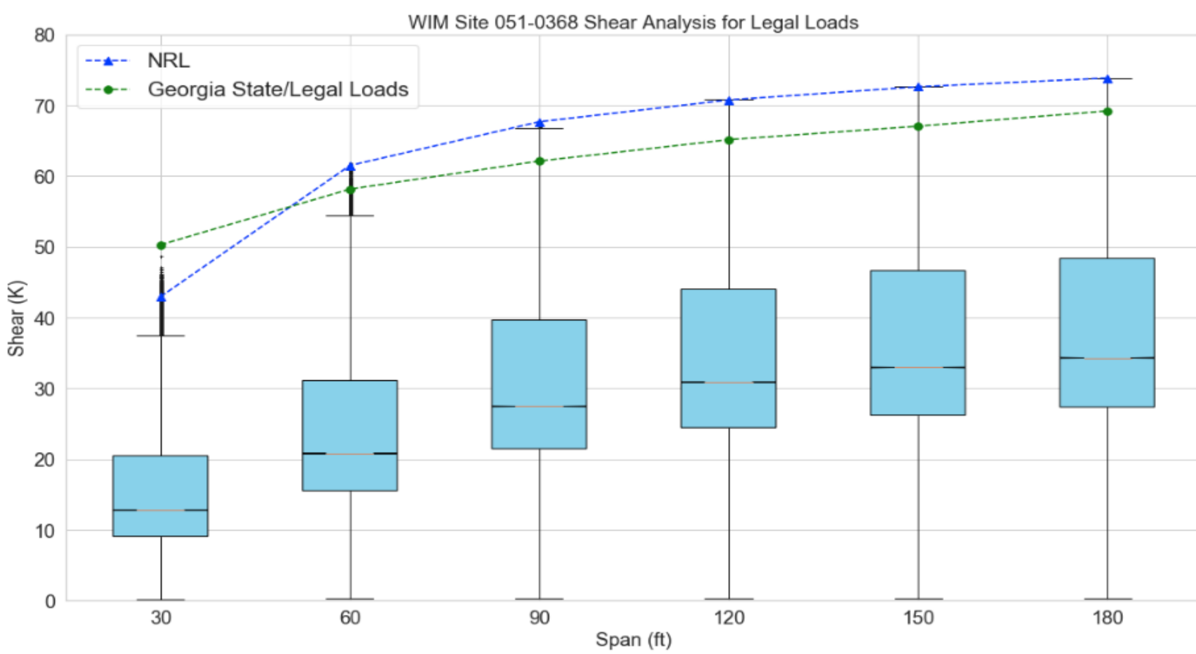


(e) WIM Site 127-0312

Figure 41 Continued – Shear Analysis for Trucks Meeting Formula B.

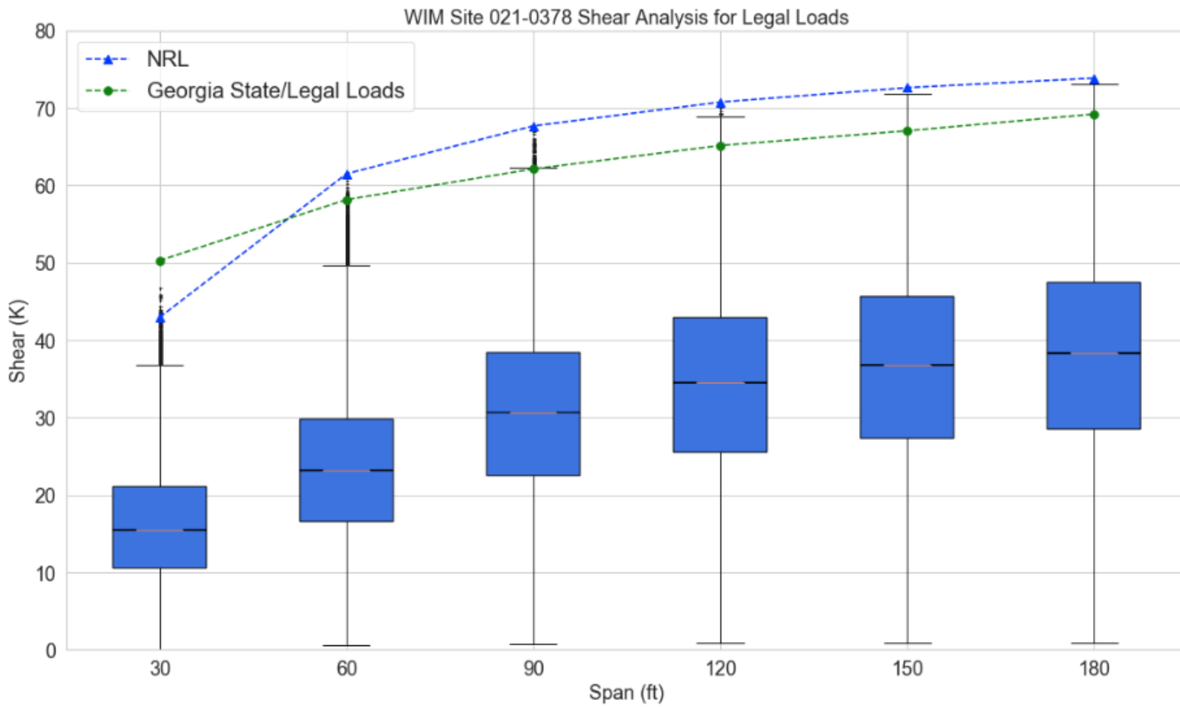


(f) WIM Site 051-0387



(g) WIM Site 051-0368

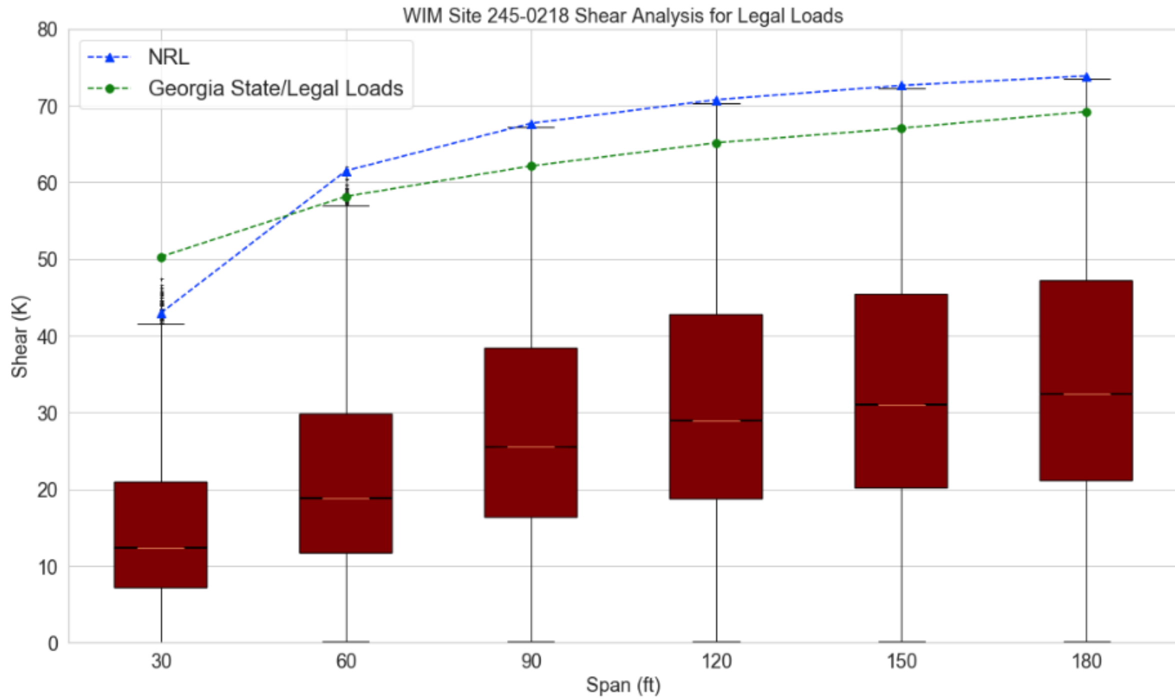
Figure 41 Continued – Shear Analysis for Trucks Meeting Formula B.



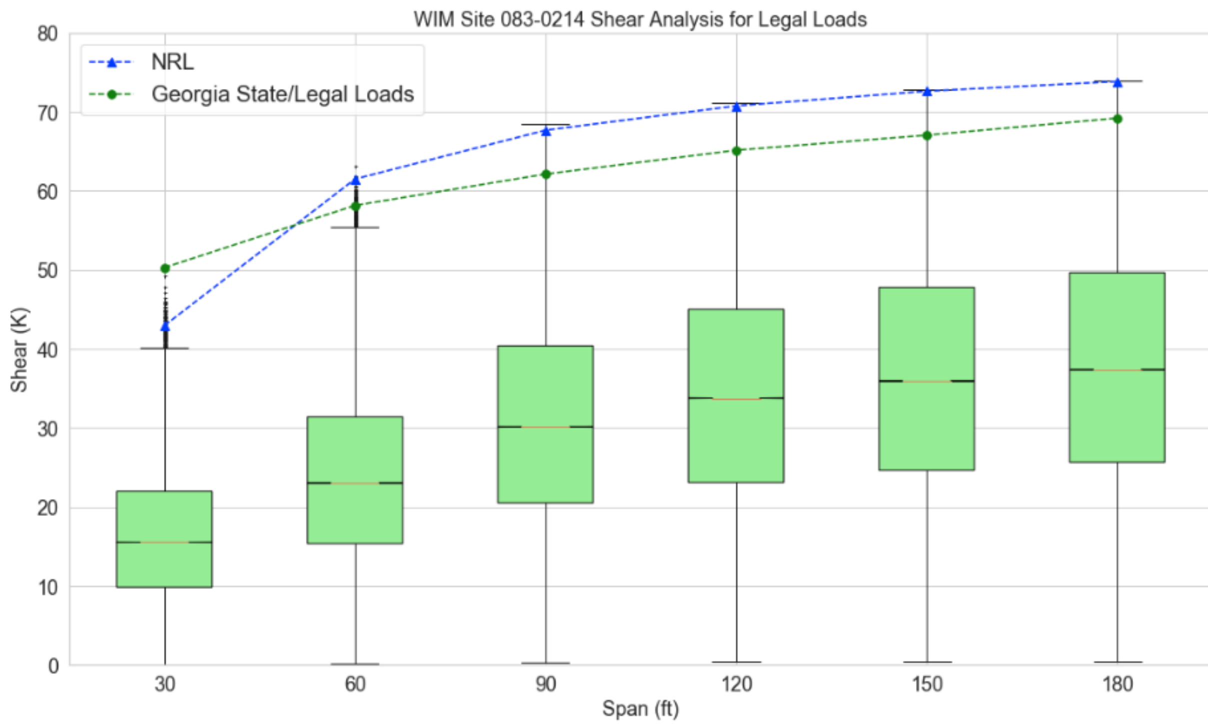
(h) WIM Site 021-0378

(i) WIM Site 245-0214

Figure 41 Continued – Shear Analysis for Trucks Meeting Formula B.



(j) WIM Site 245-0218

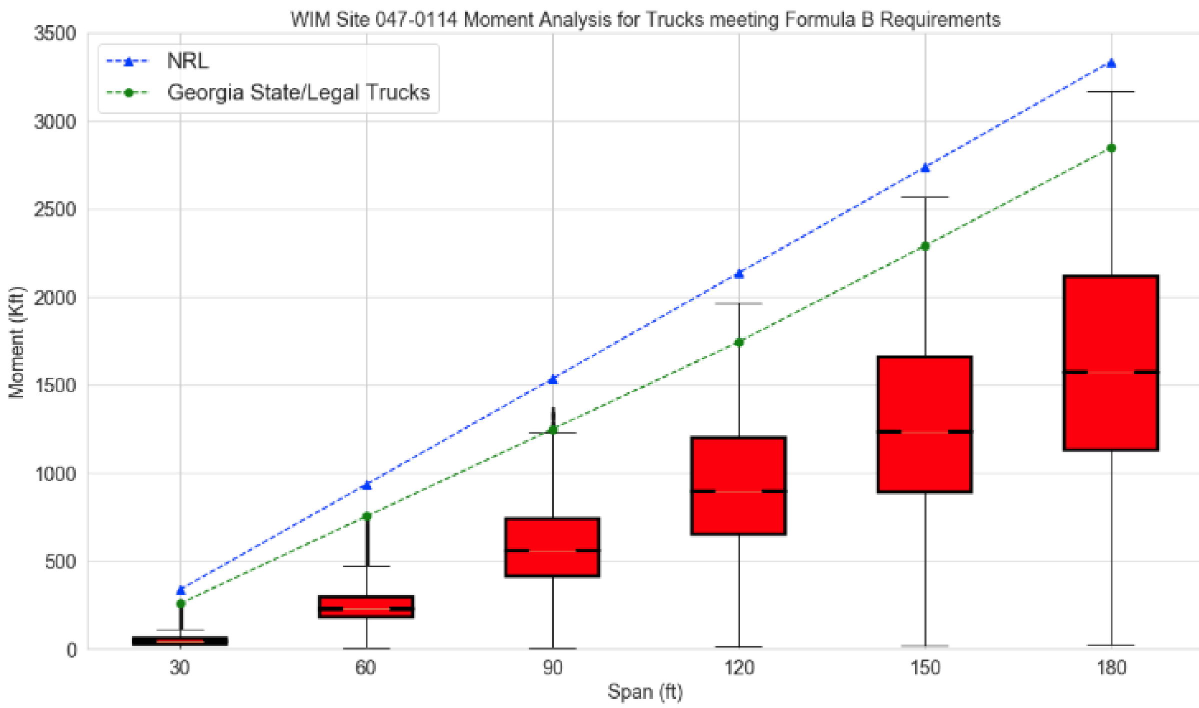


(k) WIM Site 083-0214

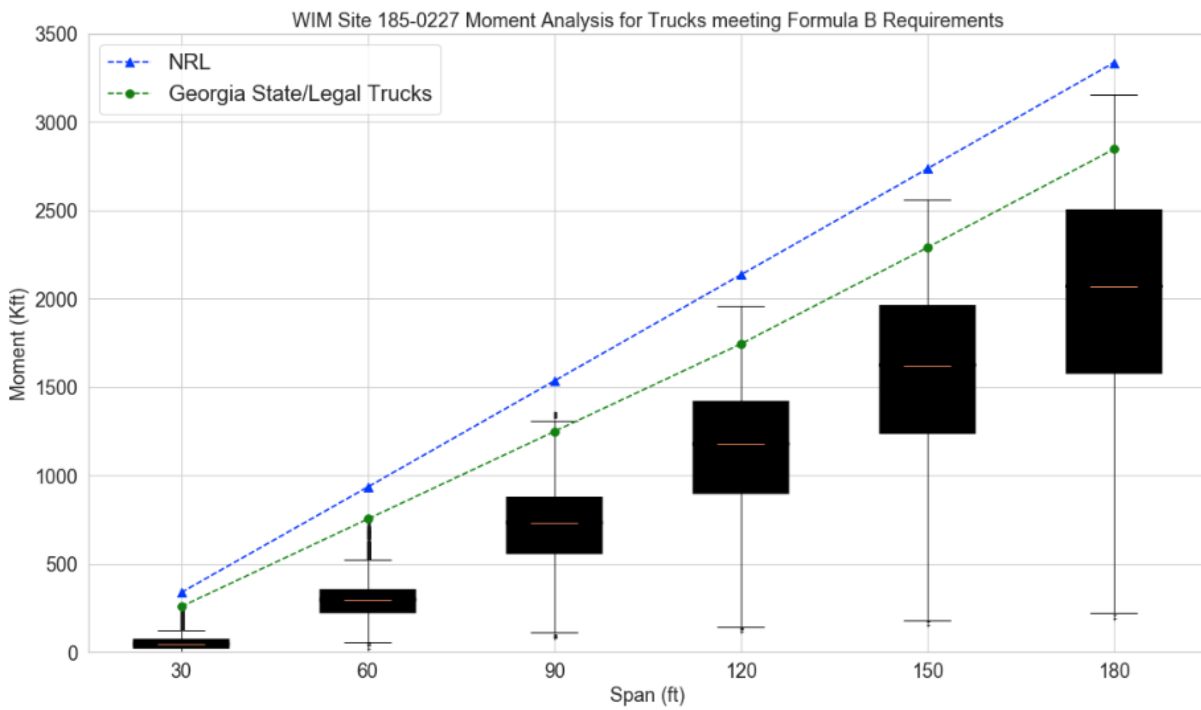
Figure 41 Continued – Shear Analysis for Trucks Meeting Formula B.

As indicated in the live load shear demands calculated in Figure 41 above, most of the boxplots followed a similar pattern and shape due to the truck weight limitations of Formula B. The Georgia state/legal loads exceeded the 30-foot span results for all 11 sites but failed to surpass all the other span lengths. This brings up concern as the condition assessment relies on these loads to characterize the non-permit trucks throughout the state. As for the NRL shear values, it was almost completely opposite as it did not surpass the 30-foot span length values and exceeded remaining span length values at the majority of sites. As for the 30-foot span length, many of the axles on the trucks and legal loads did not fit on the span at the same time due to its short length. This was most likely the reasoning for the discrepancies in the NRL and Georgia state/legal loads as the axles with the greatest weights spaced near each other controlled its result on this span. Due to this assumption, it was decided that the NRL does indeed yielded greater shear demands than the Georgia state/legal loads.

The NRL shear results did not surpass all of the WIM data on specific sites though, including WIM Sites 217-0218 and 083-0214. This raised some issues as it was desired that an examining load be able to handle every truck the bridge undergoes. However, most sites illustrated the NRL exceeding WIM Formula B truck maximum shear values by a few kips. This shows that the NRL is a better representation of the Formula B data as it handles a larger portion than the Georgia state/legal loads but could be adjusted more to ensure its capability to cover every Formula B truck. Therefore, it is recommended, according to the shear results, to adjust the NRL truck model to withstand every shear value from WIM Formula B trucks and additionally couple the NRL model with a tandem portion, similar to the HL-93 design load, to increase its maximum value at the 30 foot or lower span lengths. In addition to the shear force demands evaluated for truck traffic, the moment analysis was conducted and plotted in Figure 42 below.

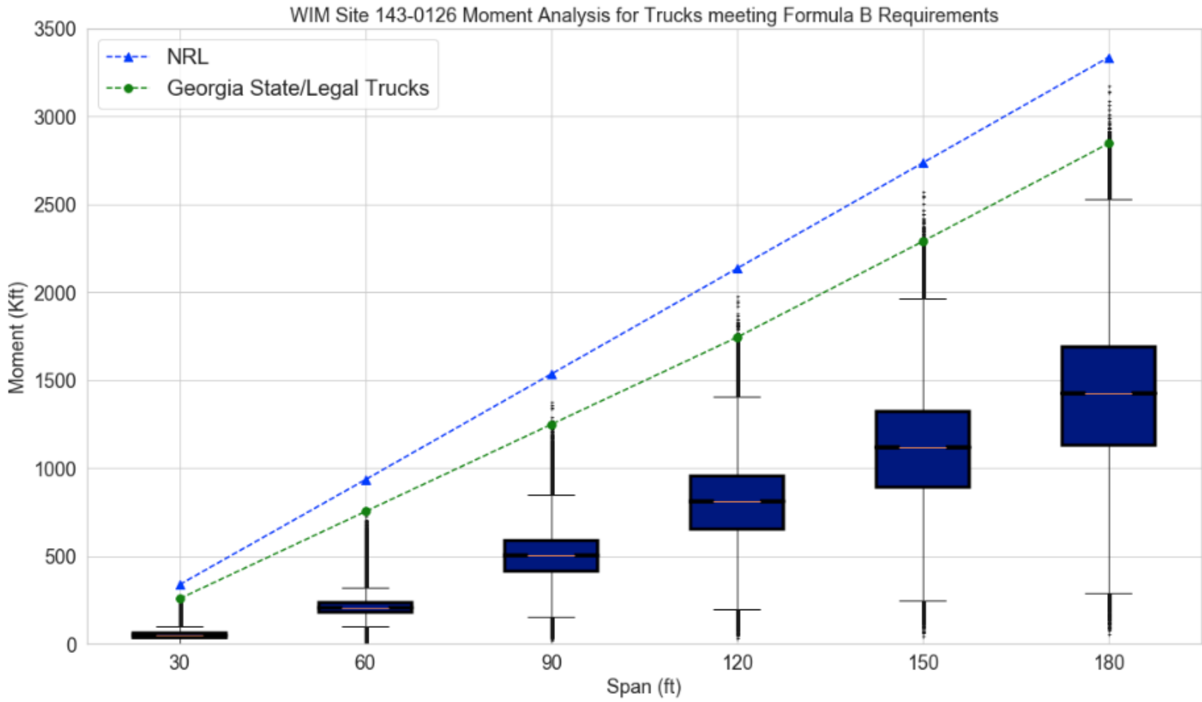


(a) WIM Site 047-0114

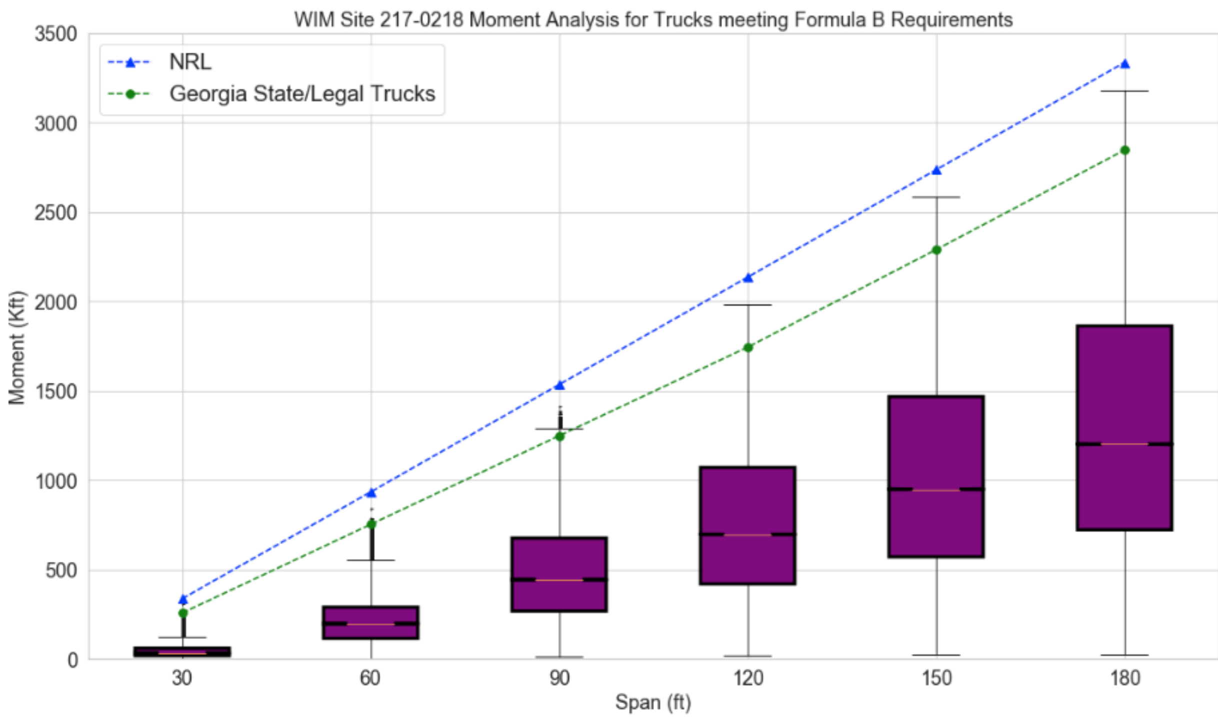


(b) WIM Site 185-0227

Figure 42 – Moment Analysis for Trucks Meeting Formula B.

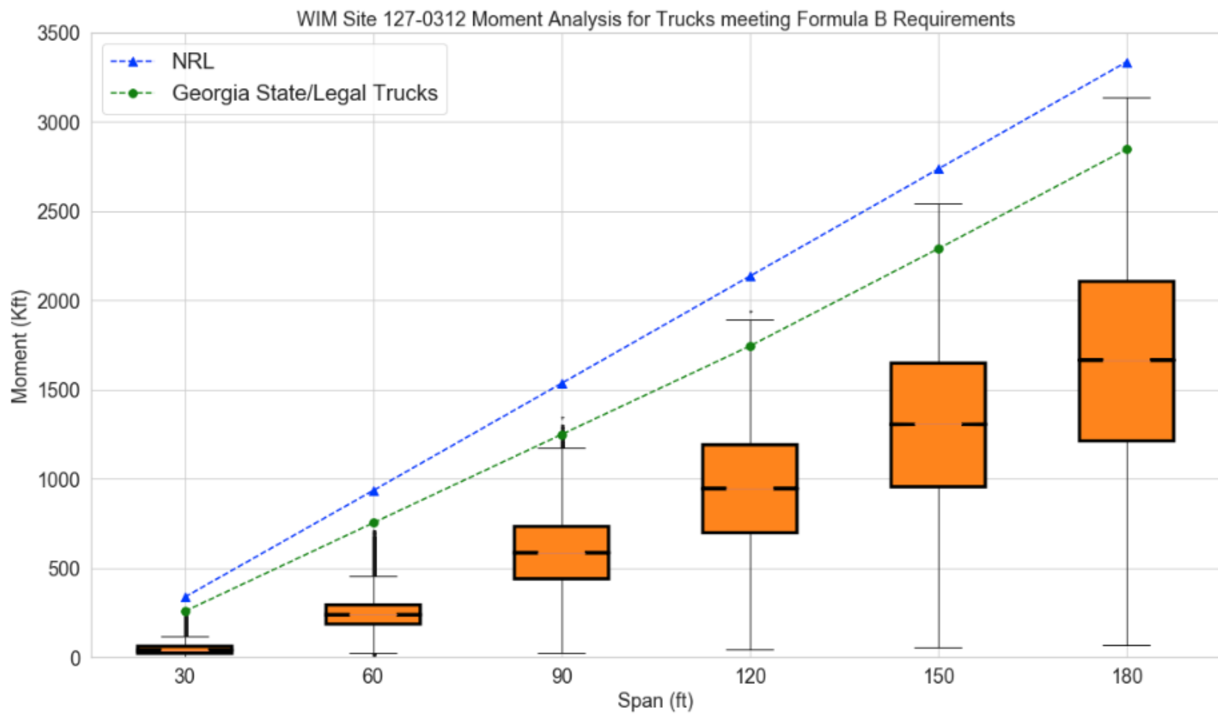


(c) WIM Site 143-0126

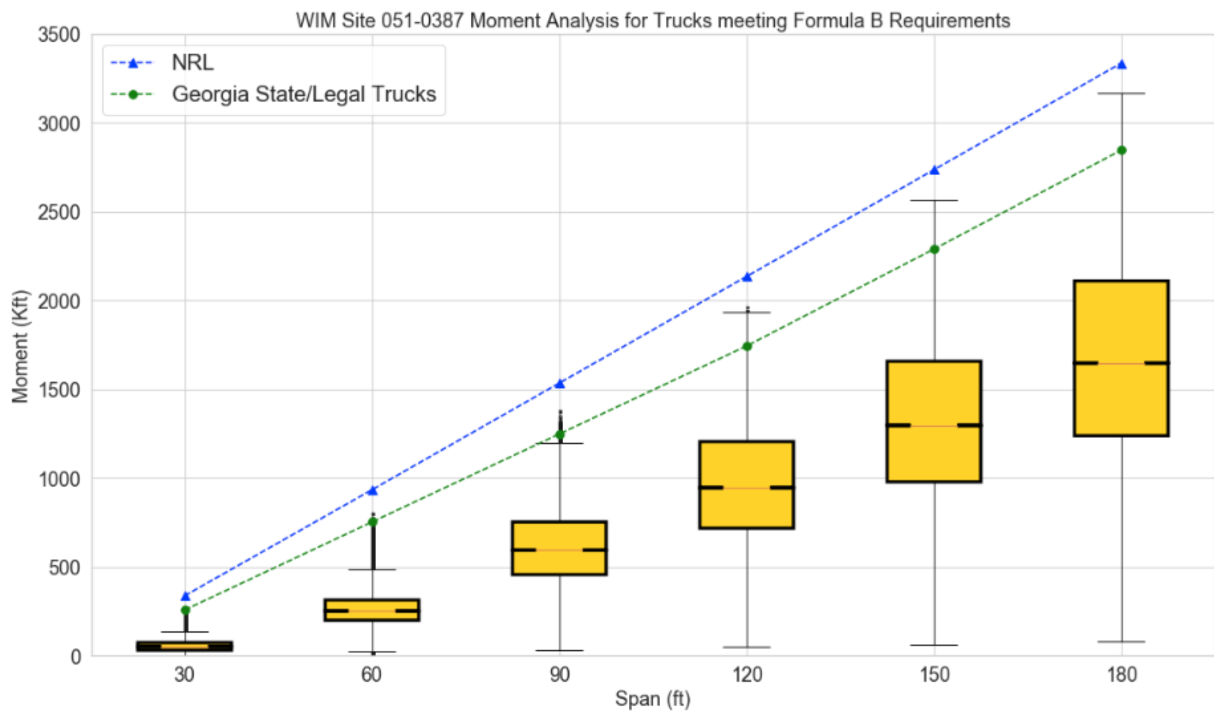


(d) WIM Site 217-0218

Figure 42 Continued – Moment Analysis for Trucks Meeting Formula B.

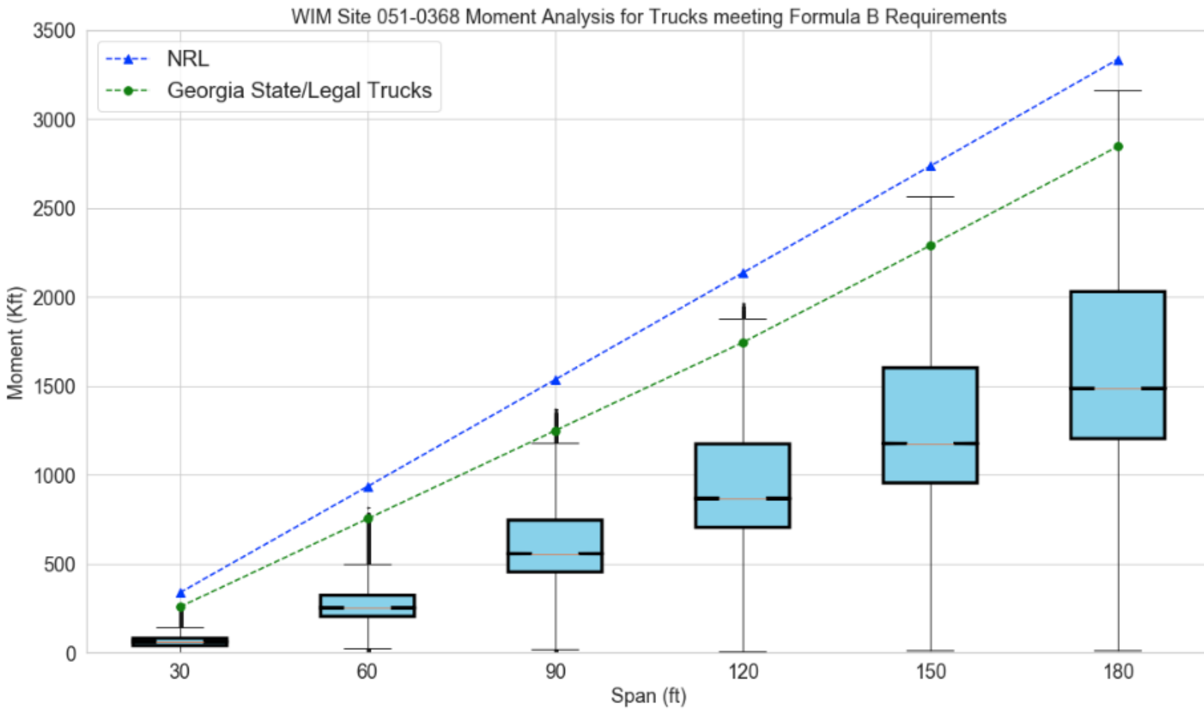


(e) WIM Site 127-0312

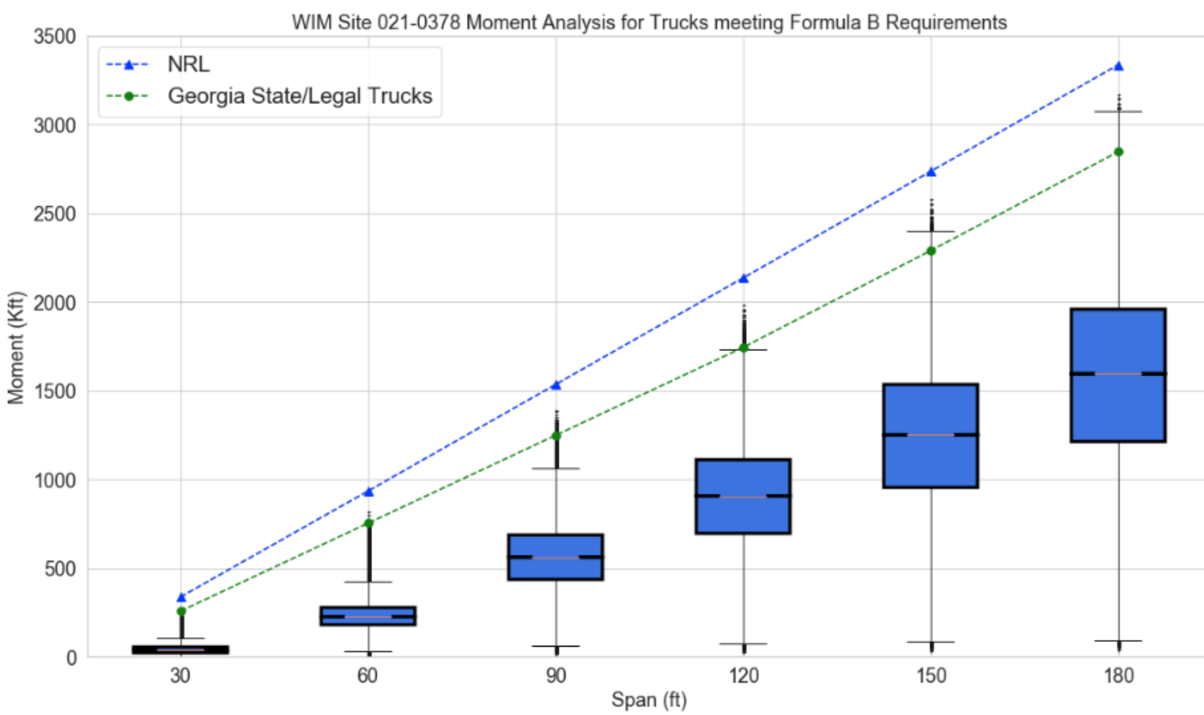


(f) WIM Site 051-0387

Figure 42 Continued – Moment Analysis for Trucks Meeting Formula B.

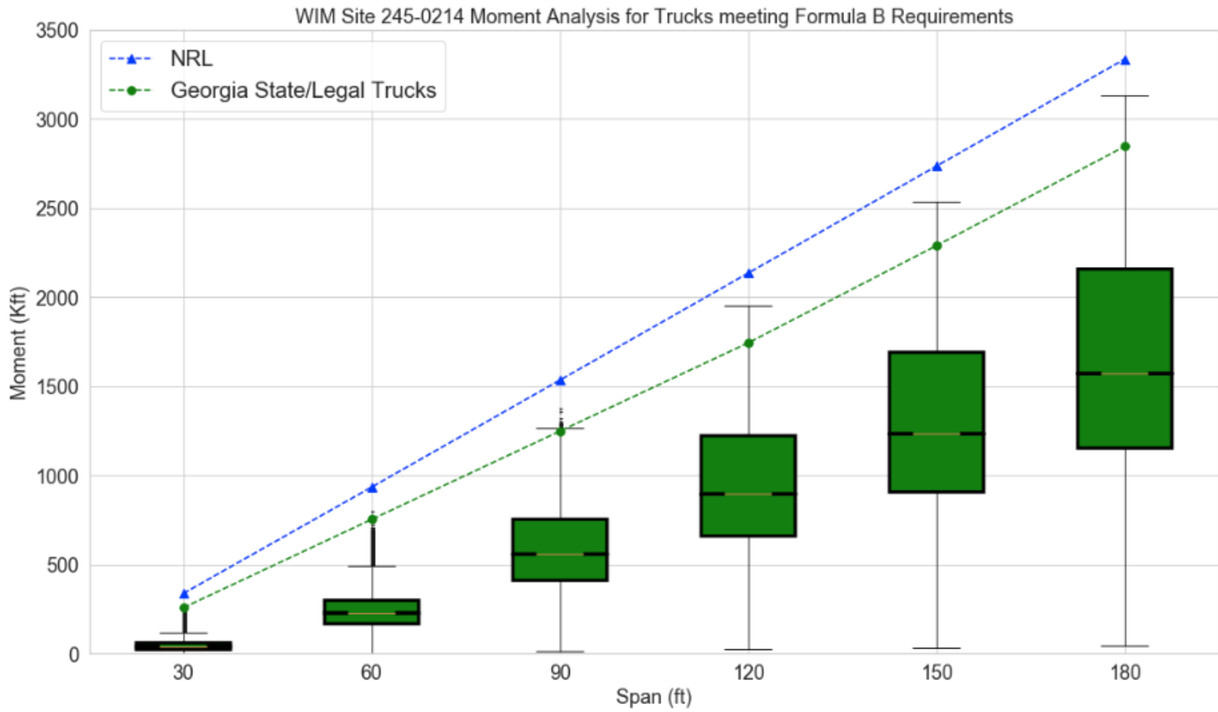


(g) WIM Site 051-0368

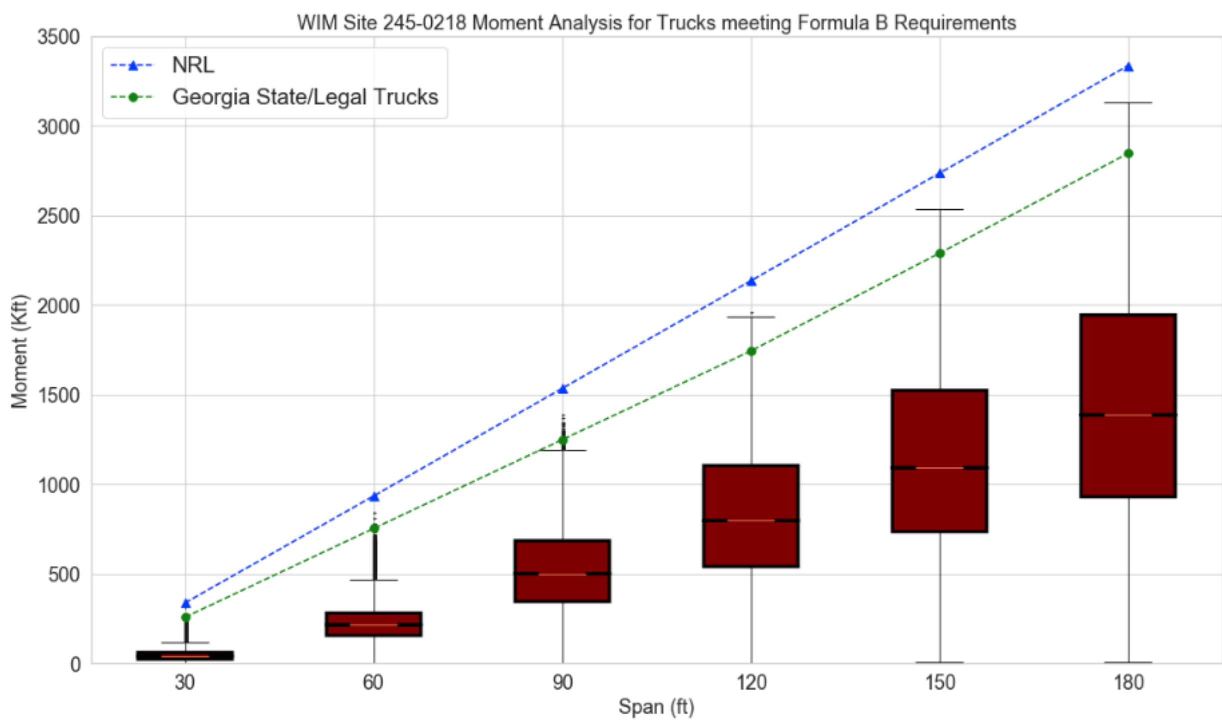


(h) WIM Site 021-0378

Figure 42 Continued – Moment Analysis for Trucks Meeting Formula B.

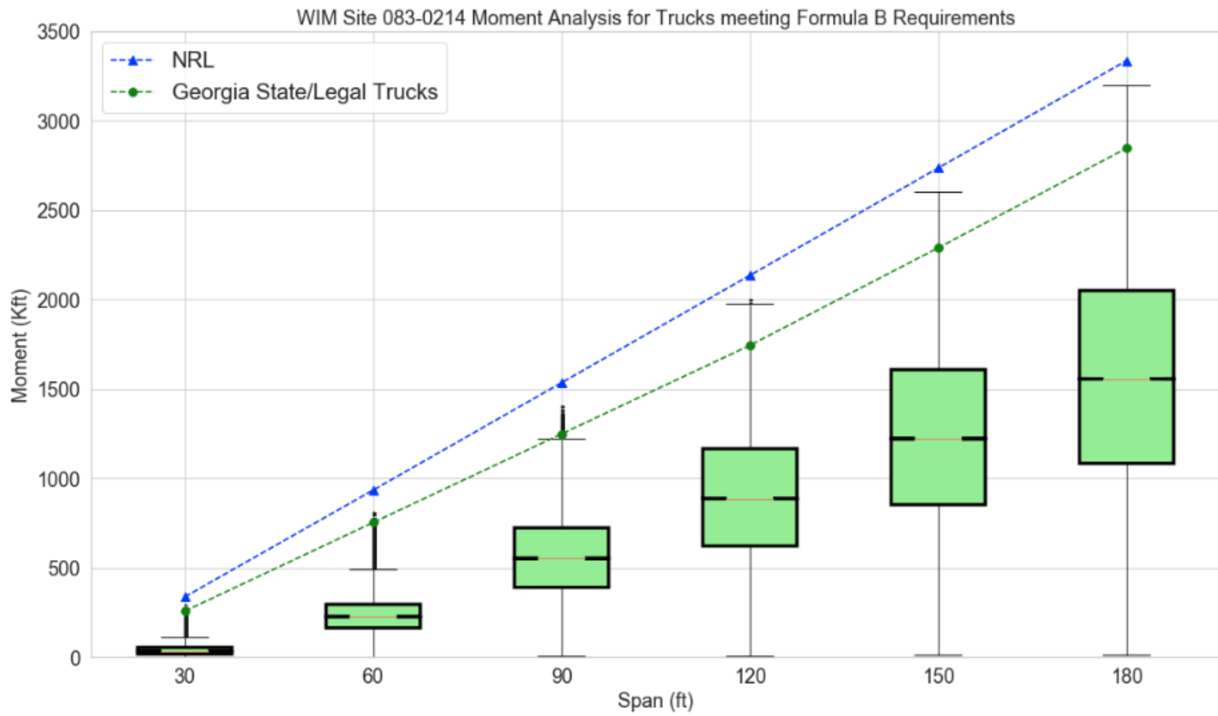


(i) WIM Site 245-0214



(j) WIM Site 245-0218

Figure 42 Continued – Moment Analysis for Trucks Meeting Formula B.



(k) WIM Site 083-0214

Figure 42 Continued – Moment Analysis for Trucks Meeting Formula B.

As indicated by Figure 42, each of the 11 plots followed a very similar appearance with very adjacent moment values in the WIM data and bridge rating loads (i.e., NRL and Legal truck loads). This similar appearance was experienced in the shear as well due to the Formula B limitations. By viewing the Georgia state/legal trucks maximum moment for each span, indicated with the dotted green line, the analysis was about the same as it was for shear indicating its inability to envelope the live load demands computed for all of the Formula B trucks. This is a problem because its use in bridge load rating evaluation leads to a bridge passing tests due to the Georgia state/legal loads and risks the chance that bridges will be exposed to heavier trucks. Figure 42 illustrated that the moments under NRL are slightly greater than the live moment demands evaluated for each of the eleven sites tested. As for moment analysis, this NRL configuration was able to handle every live load moment the sites are experiencing.

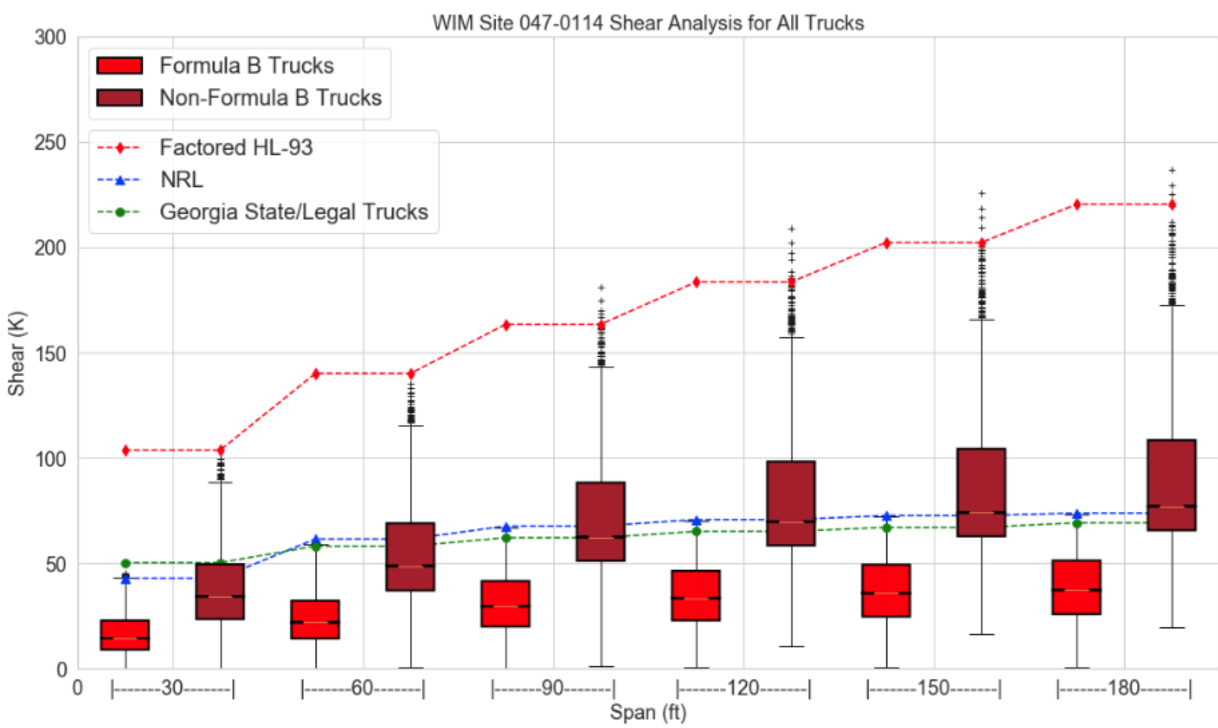
Special Hauling Vehicles (SHVs) generally comply with Bridge Formula B and are considered legal in Georgia. Weight exemptions and special operations are only allowed on non-Interstate highway in Georgia. Thus, weight limits may be exceeded in Georgia without a permit for vehicles transporting commodities such as forest products and live poultry {Administration, 2019 #36}. Otherwise, a special hauling permit is necessary. As a result, the state bridge load rating model should envelope the applicable SHV configurations. Georgia must rate bridges for SHVs as defined in the 2018 AASHTO Manual for Bridge Evaluation (MBE) {AASHTO, 2019 #35}, and the manual mandates the use of NRL. Since this provisional change was made following the NCHRP recommendations, state agencies have been re-rating bridges. This study confirmed that the notional rating load (NRL) enveloping truck load configurations, including SHVs, should serve as screening load for rating bridges. SHVs and overweight trucks created higher force effects than the legal truck loads used by GDOT in the past. NRL is often considered a conservative rating load model. However, NRL closely envelops truck loads in Georgia and thus should not be considered conservative for load rating but rather be regarded as a minimum requirement.

It was established that for both shear and moment results, the NRL was a more reliable test than the previously used Georgia state/legal load model. It was able to surpass all the moment values at every span length for the sites studied as the Georgia state/legal loads did not for any of the sites. The NRL handled most of the WIM shear results although a small percentage did exceed its values. It would be recommended to use the NRL truck configuration, as concluded in NCHRP Report 575.

4.2.2 HL-93 Design Load

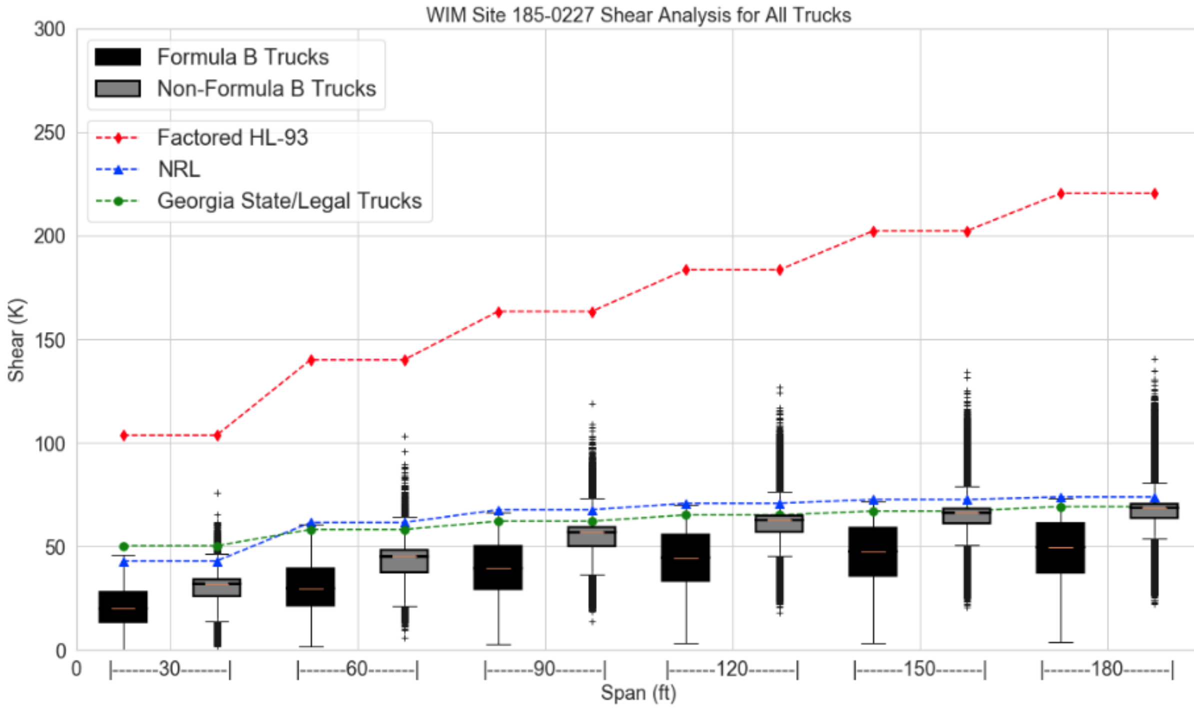
Both Formula B and Non-Formula B trucks were analyzed with the HL-93 design load as it should be able to handle all trucks due to its design functions. The HL-93 data in this examination was factored by a value of 1.75 based on the limit state of Strength I to provide credible results to be analyzed. Equation 11 is provided illustrating this process for shear. The shear results of this study are indicated in Figure 43 below. The boxplots of this figure represent the maximum shear per WIM truck for both Formula B and Non-Formula B trucks. The dotted red line illustrates the maximum shear for the factored HL-93 design load and should embody all the maximum shear values for both Formula B and Non-Formula B trucks.

$$\emptyset SS_{HL-93} = (1.75)(SS_{HL-93}) \qquad \textbf{(Eq. 11)}$$

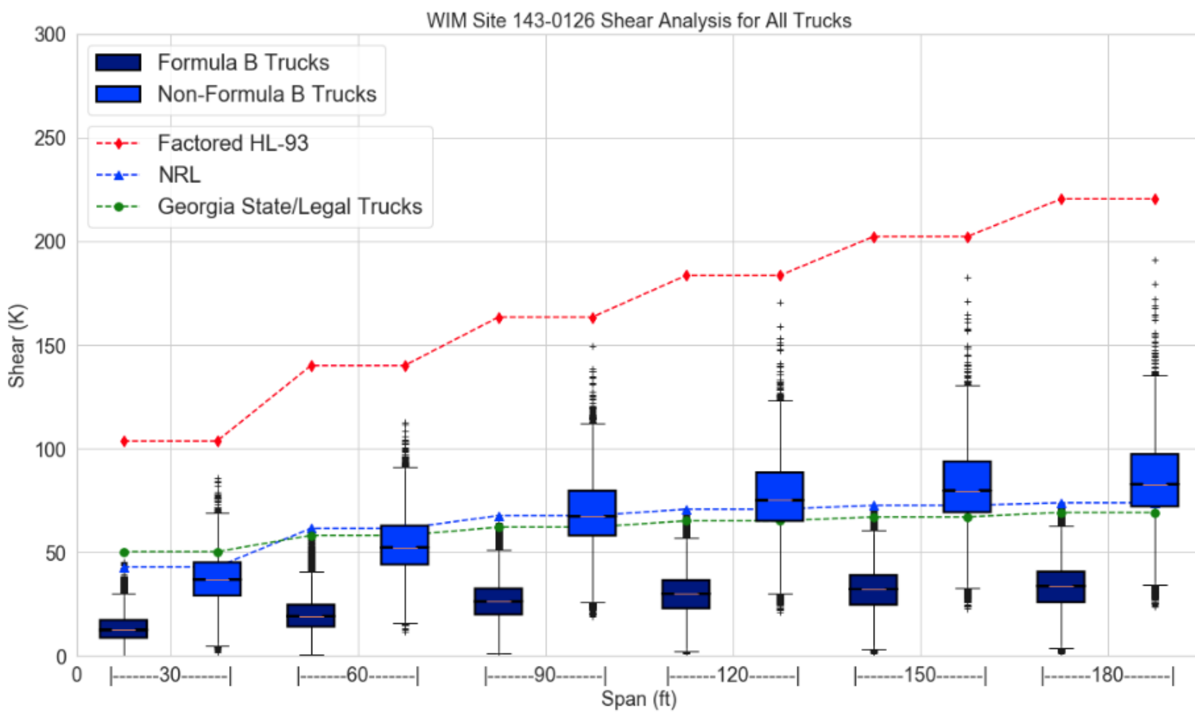


(a) WIM Site 047-0114

Figure 43 – Shear Analysis for All Trucks.

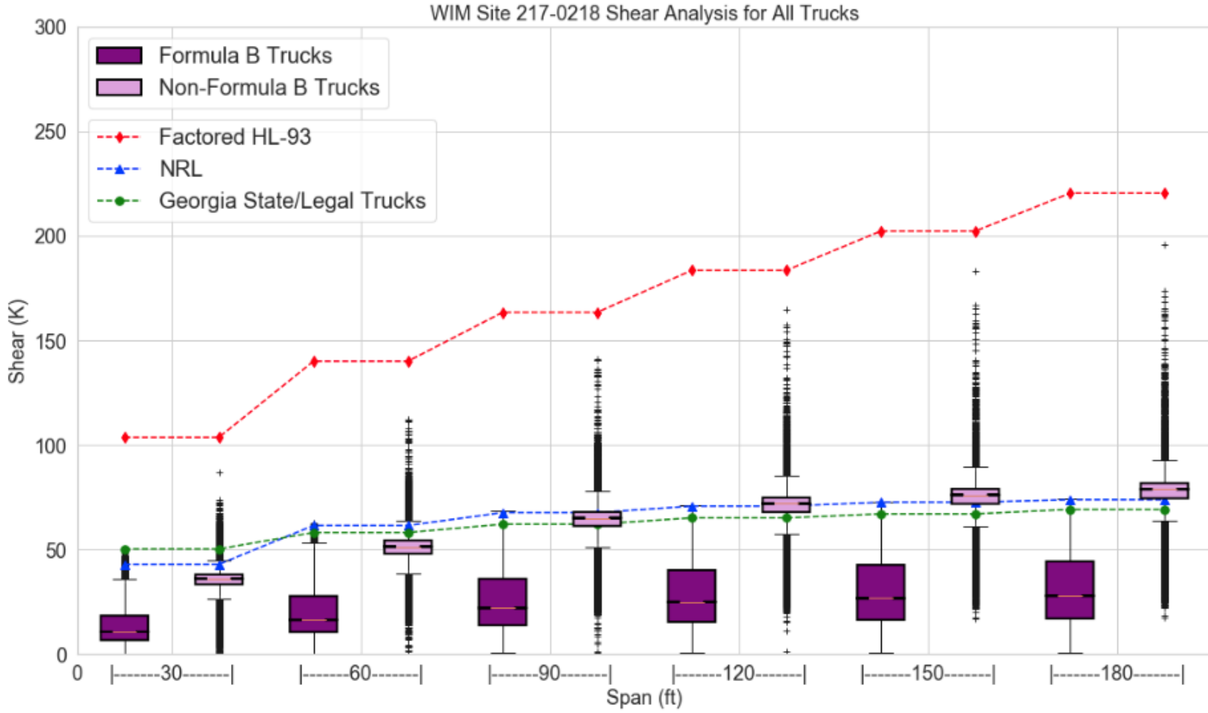


(b) WIM Site 185-0227

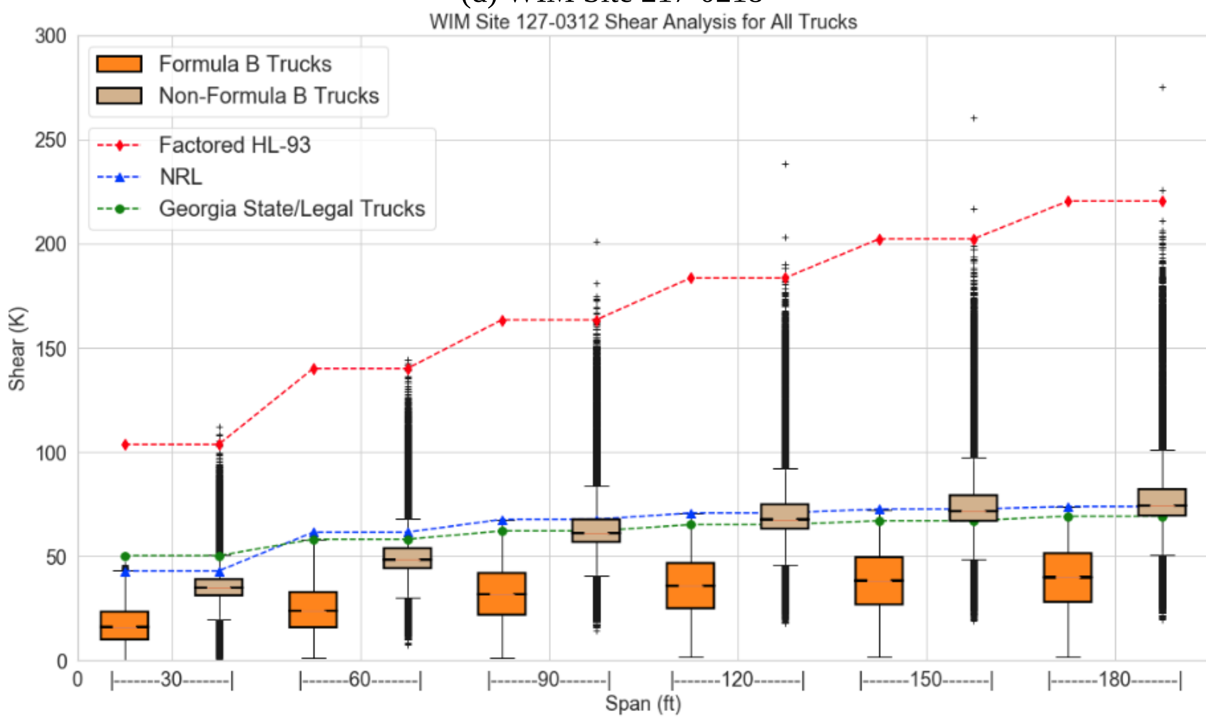


(c) WIM Site 143-0126

Figure 43 Continued – Shear Analysis for All Trucks.

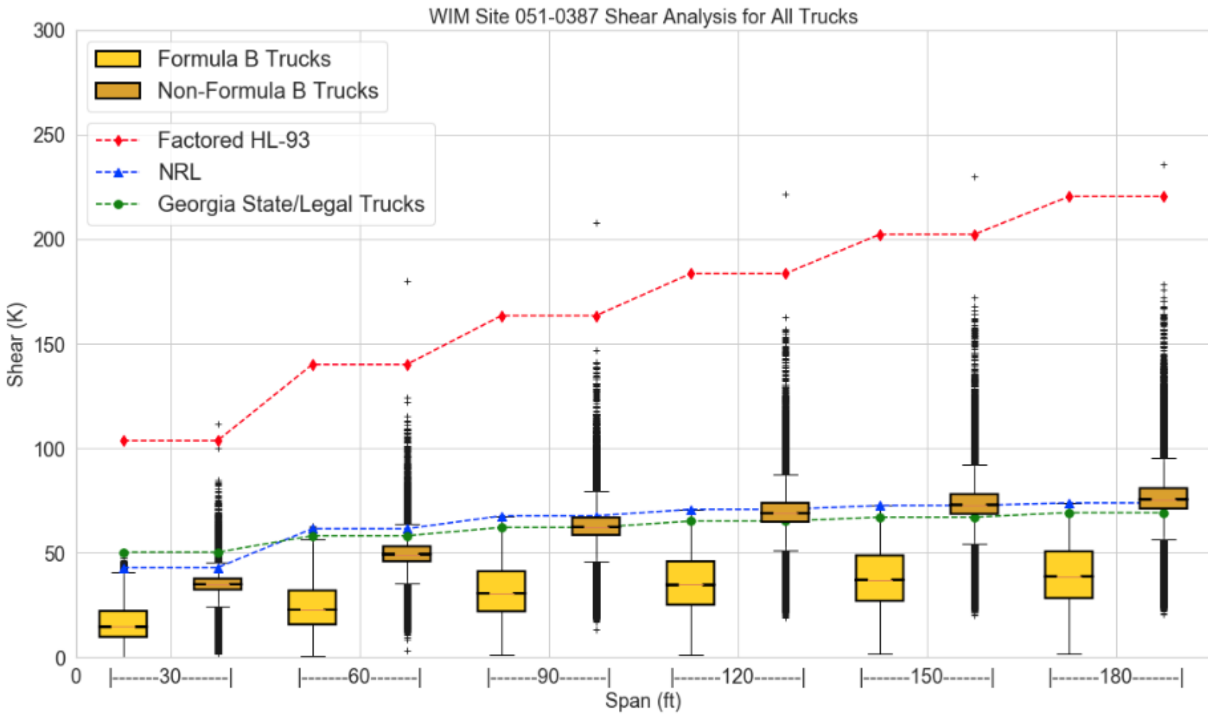


(d) WIM Site 217-0218

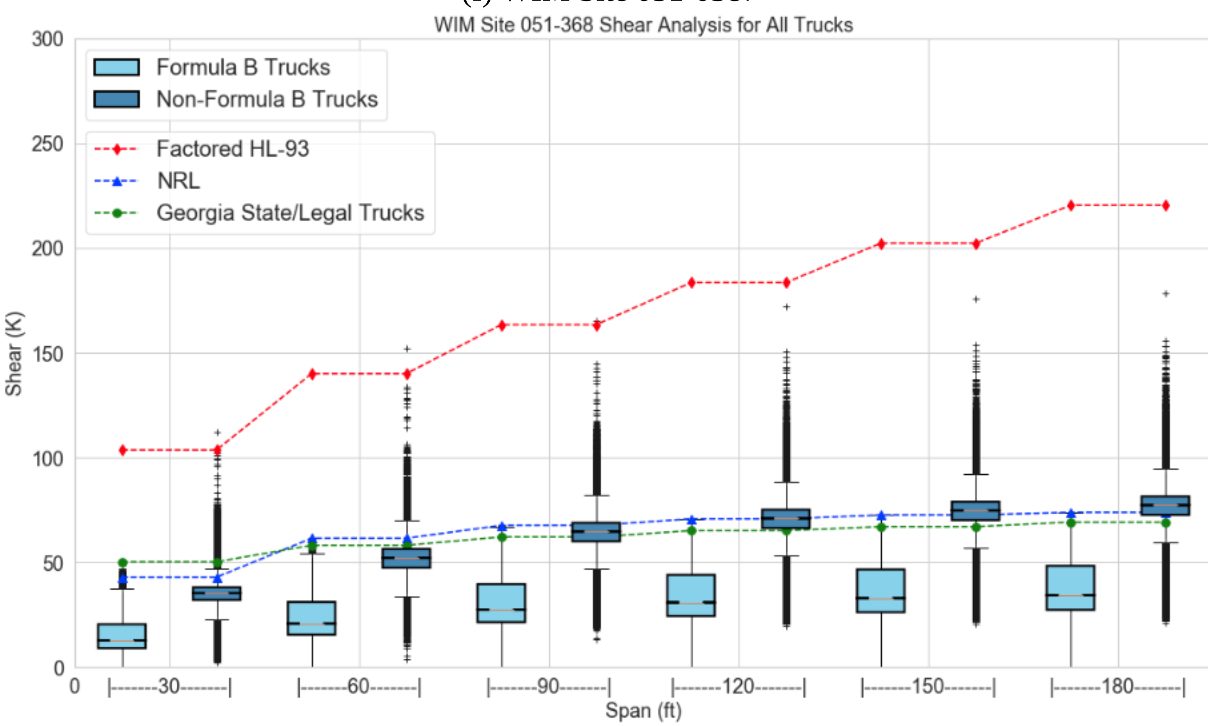


(e) WIM Site 127-0312

Figure 43 Continued – Shear Analysis for All Trucks.

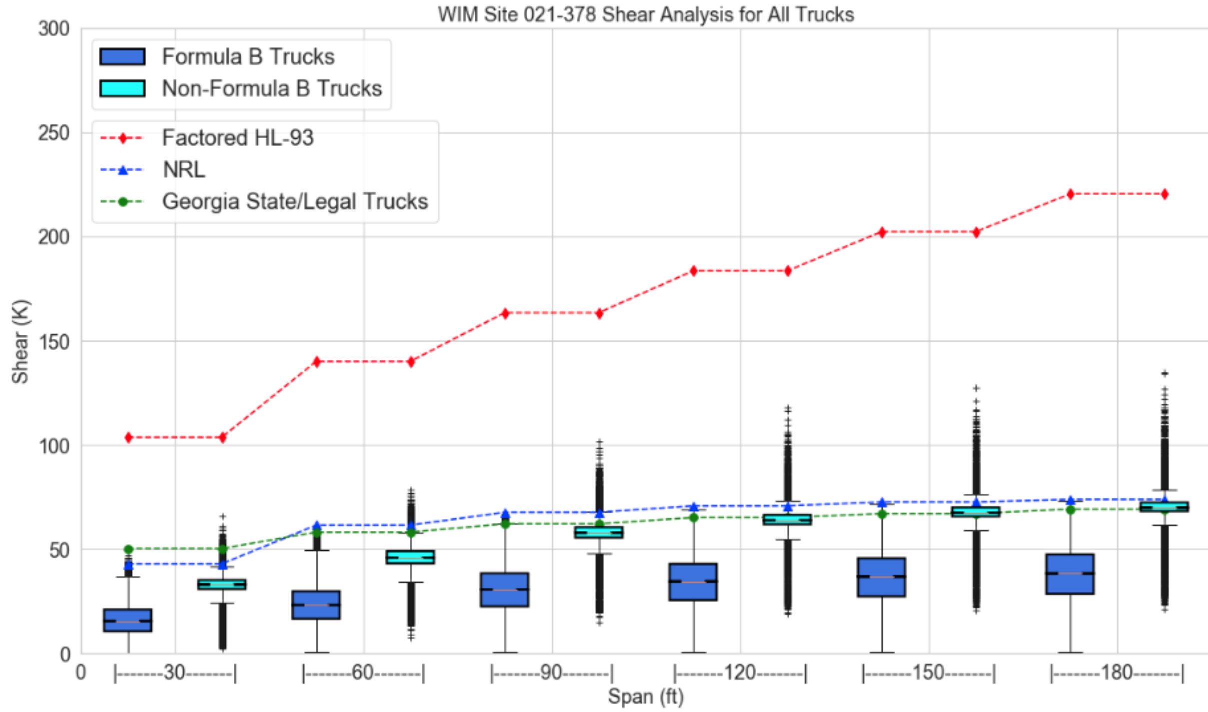


(f) WIM Site 051-0387

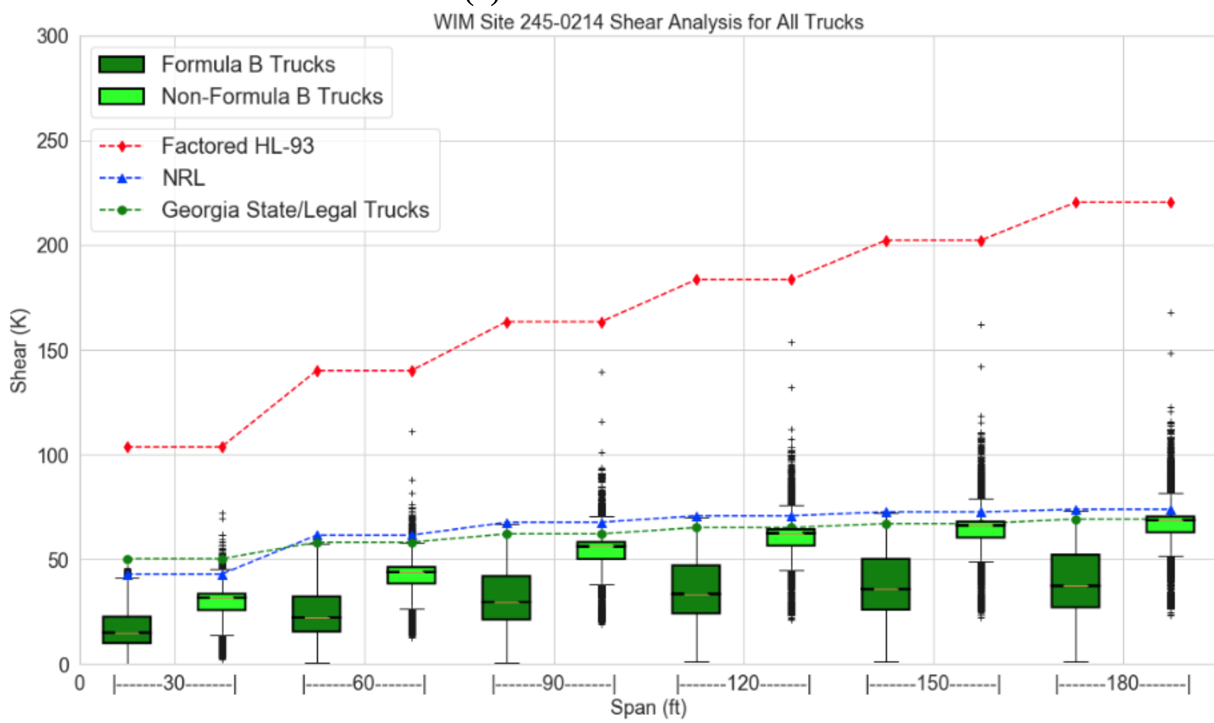


(g) WIM Site 051-0368

Figure 43 Continued – Shear Analysis for All Trucks.

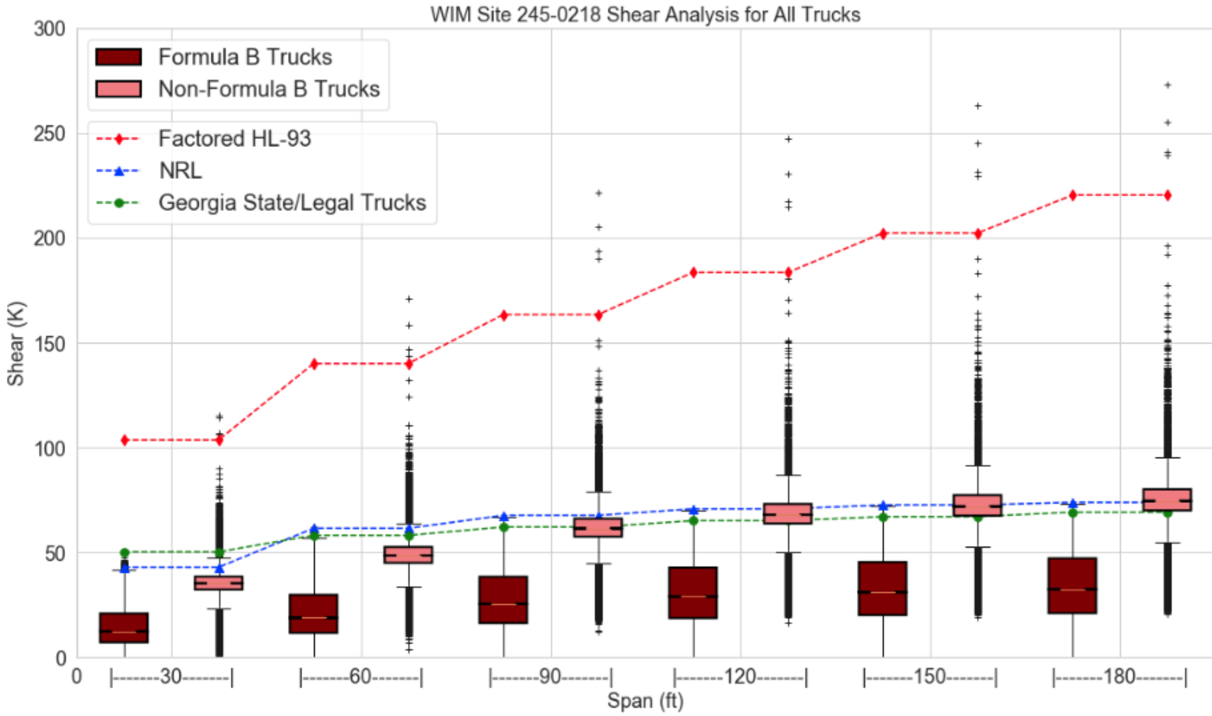


(h) WIM Site 021-0378

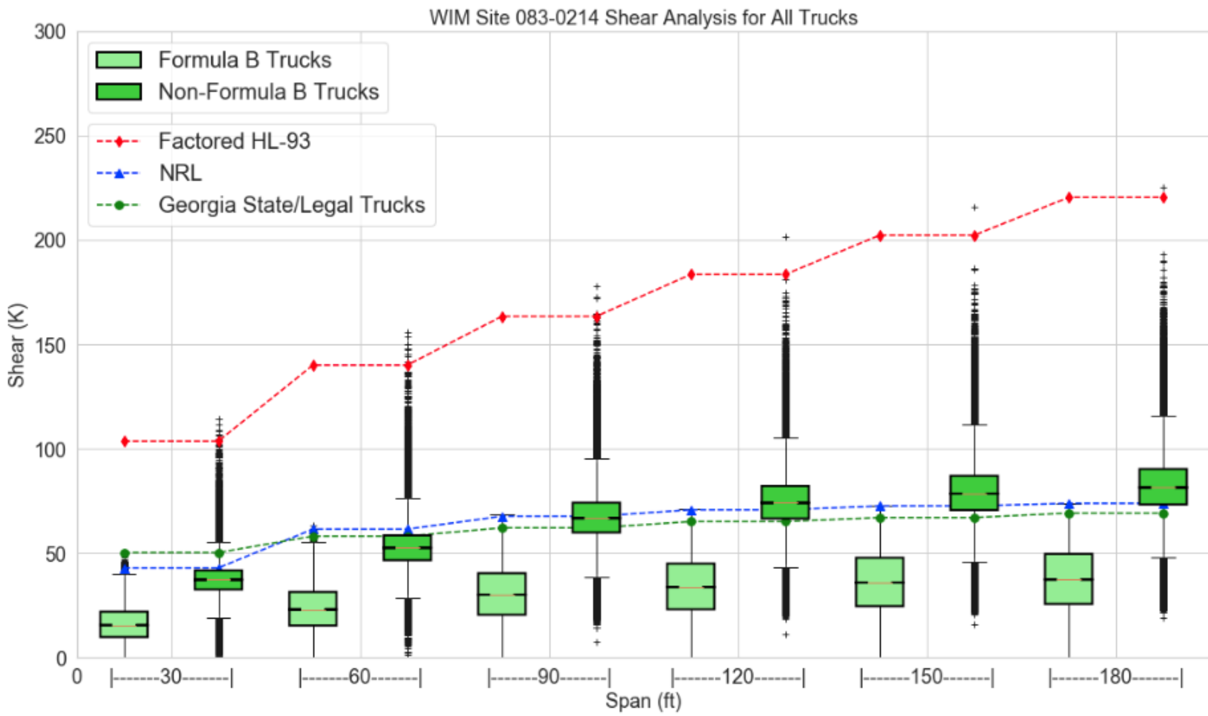


(i) WIM Site 245-0214

Figure 43 Continued – Shear Analysis for All Trucks.



(j) WIM Site 245-0218



(k) WIM Site 083-0214

Figure 43 Continued – Shear Analysis for All Trucks.

The figures above illustrated how the factored HL-93 design load compared with the WIM shear data. The majority of the sites had a comparable shape in the Non-Formula B trucks as the data was more compact than the Formula B trucks and had multiple outlier. These outlier specified the greatest forces that bridges along the route experienced over the entire year. As for the HL-93 design load, its factored values exceeded the force demands in the majority of the sites over each of the span lengths tested. However, some sites did have values above the factored HL-93 design load and were analyzed further. This analysis is provided in Table 16 below, and Figure 44 compares the factored HL-93 shear force with the maximum single shear demand found per WIM sites per span.

Table 16 – HL-93 Factored Shear.

Span Length	HL-93 Factored Shear	WIM Sites that Exceed
30 feet	103.60 k	047-0114, 127-0312, 051-0387, 051-0368, 245-0218, 083-0214
60 feet	140.00 k	127-0312, 051-0387, 051-0368, 245-0218, 083-0214
90 feet	163.33 k	047-0114, 127-0312, 051-0387, 051-0368, 245-0218, 083-0214
120 feet	183.40 k	047-0114, 127-0312, 051-0387, 245-0218, 083-0214
150 feet	202.16 k	047-0114, 127-0312, 051-0387, 245-0218, 083-0214
180 feet	220.27 k	047-0114, 127-0312, 051-0387, 245-0218, 083-0214

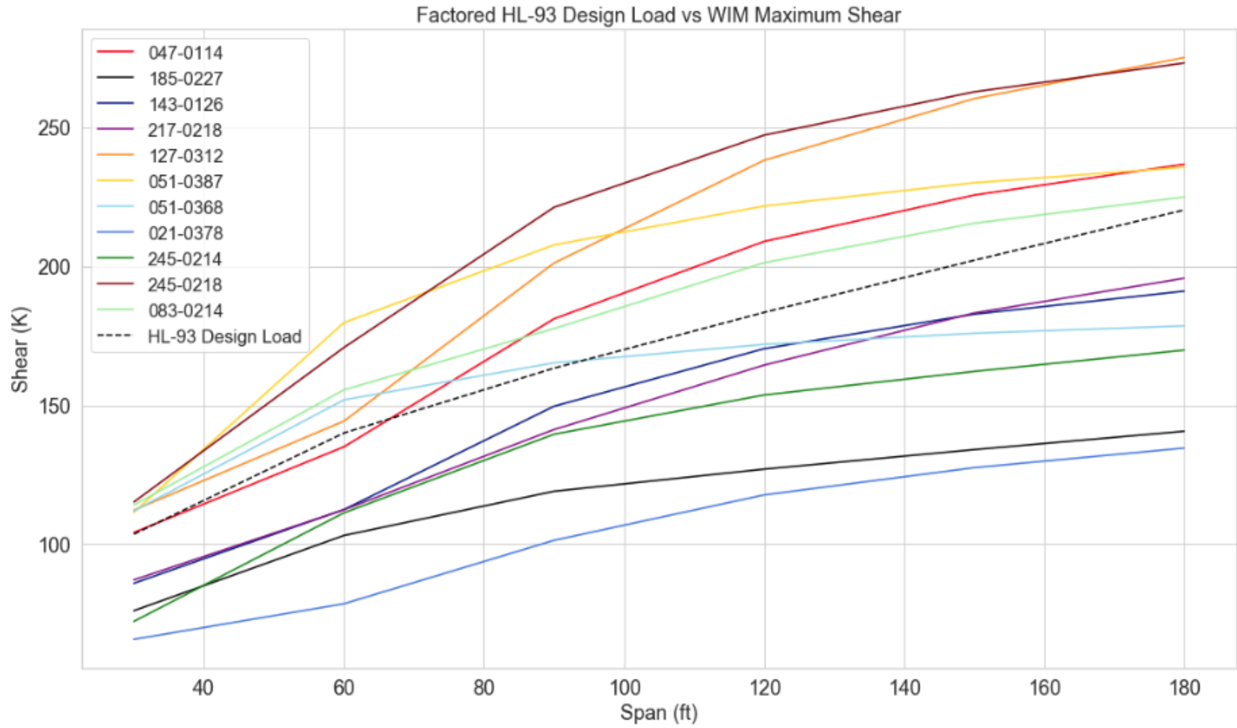
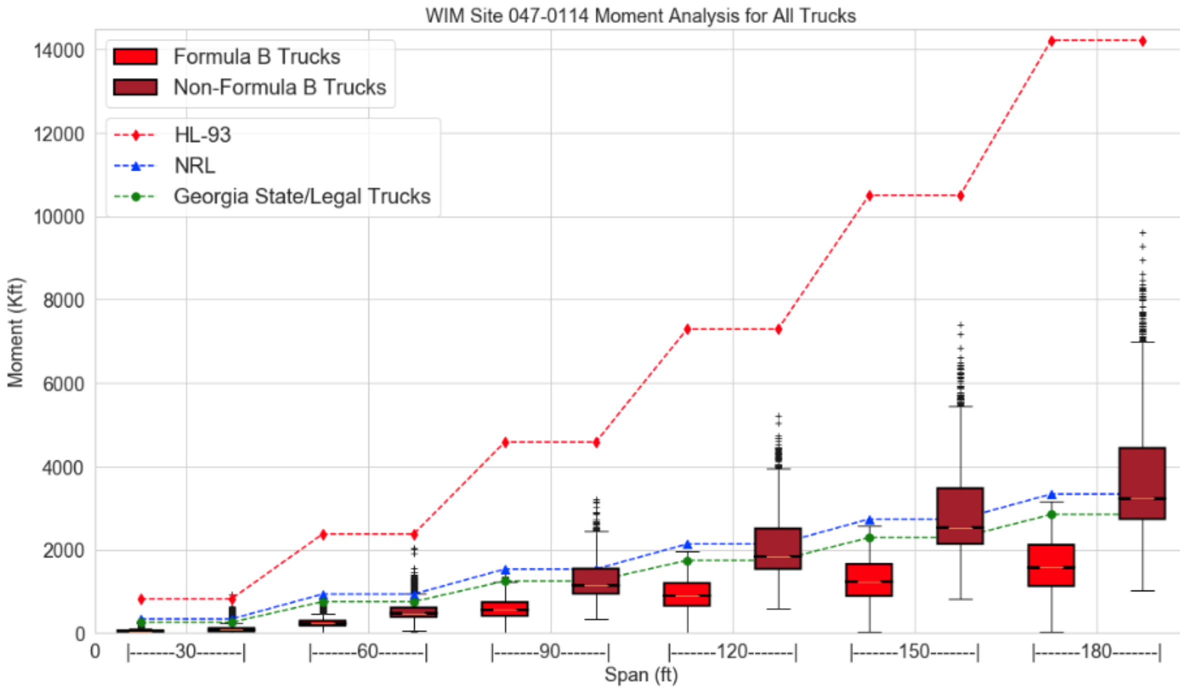


Figure 44 – Factored HL-93 Design Load vs WIM Maximum Shear.

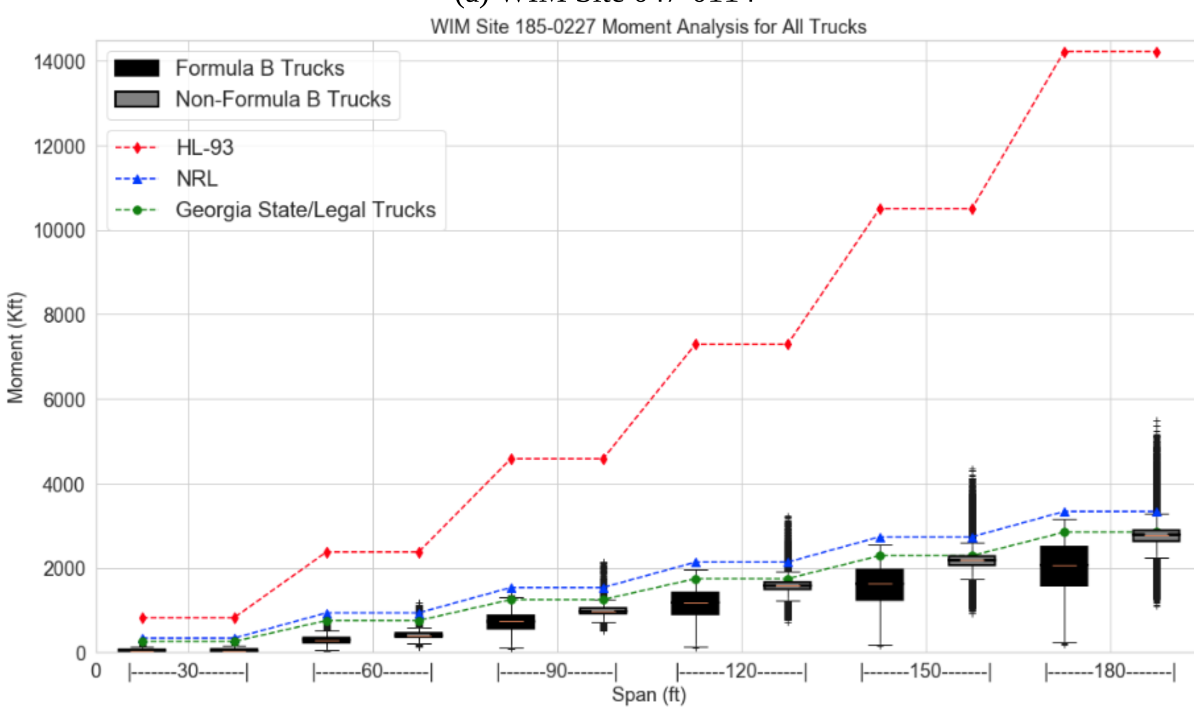
The line plots above indicated that six sites have experienced trucks causing maximum shear values above the factored HL-93 design loads. These values brought up some concern as the HL-93 is used for design. Luckily, the design process of a bridge undergoes more conservative steps, creating a capacity that exceeds the maximum forces it should experience by a safety margin. When designing a bridge, multiple HL-93 truck models are assessed creating a larger strength capacity. However, the HL-93 design load is utilized to represent every truck a bridge will encounter. Two sites (127-0312 and 217-0218) did produce shear maximums 50 kips greater than the HL-93 at 180-foot span length, being a significant difference.

Every site listed above the HL-93 design load maximum in Table 16 only had a few trucks causing maximum shear forces above the HL-93 design load while a few sites including sites 051-0387 and 051-0368 only had one truck surpassing the mark. Additionally, these results could have be swayed due to load calibration and/or inherent sensor errors within the WIM systems including speed/temperature as indicated in the literature review, Chapter 1.2.2.2. Therefore, for shear, it was stated that the HL-93 design load is still a good standard to follow as it represented the bulk of basic live loads affecting Georgia bridges as an insignificant percentage surpassed its values. However, this method should be studied further to ensure its reliability of design and evaluation purposes as some trucks did produce shear results that were greater than HL-93 design. The moment analysis on the HL-93 design load was then performed and presented in Figure 45 below. The figure follows a similar format to the shear except its y axis is a moment in kip feet. Its HL-93 design load factored values were calculated similarly to shear, and its method is provided in Equation 12.

$$\emptyset \boldsymbol{MM_{HL-93}} = (\boldsymbol{1.75})(\boldsymbol{MM_{HL-93}}) \qquad \textbf{(Eq. 12)}$$

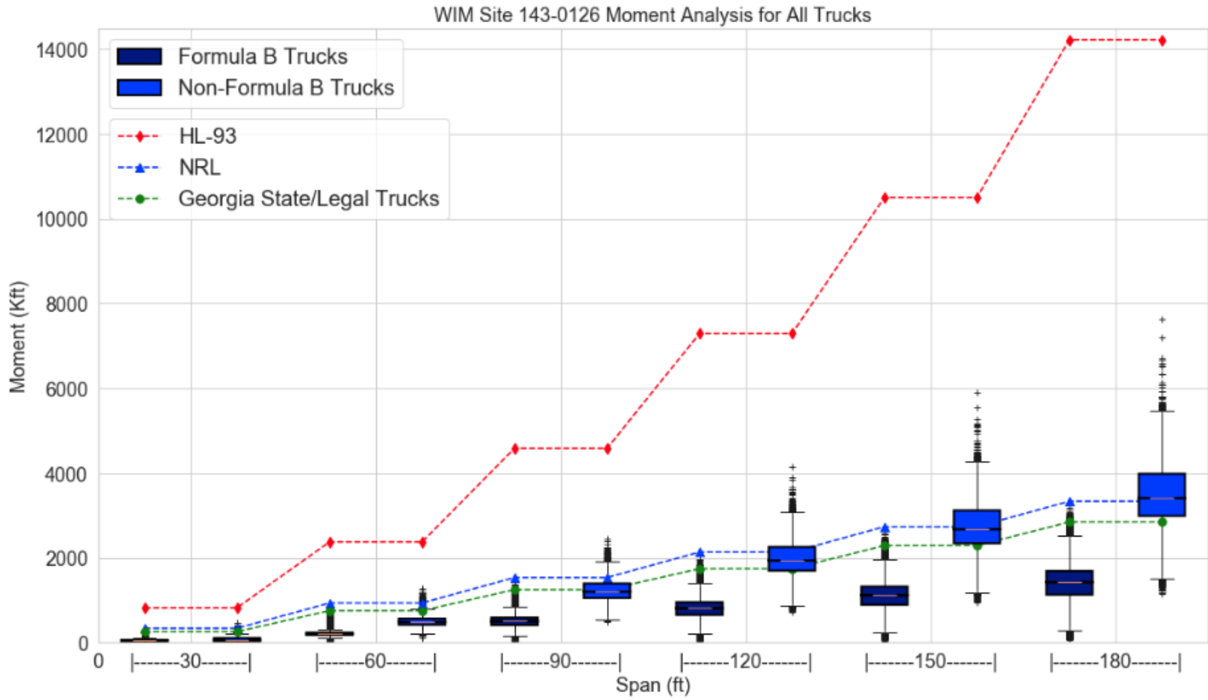


(a) WIM Site 047-0114

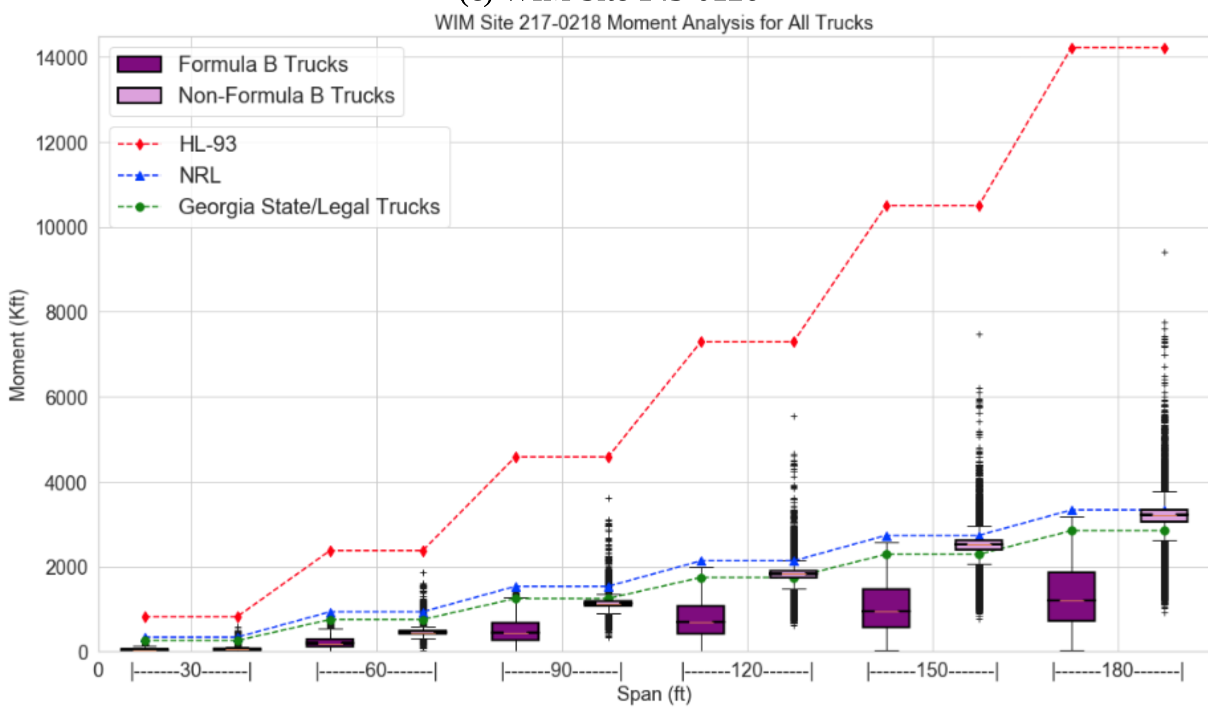


(b) WIM Site 185-0227

Figure 45 – Moment Analysis for All Trucks.

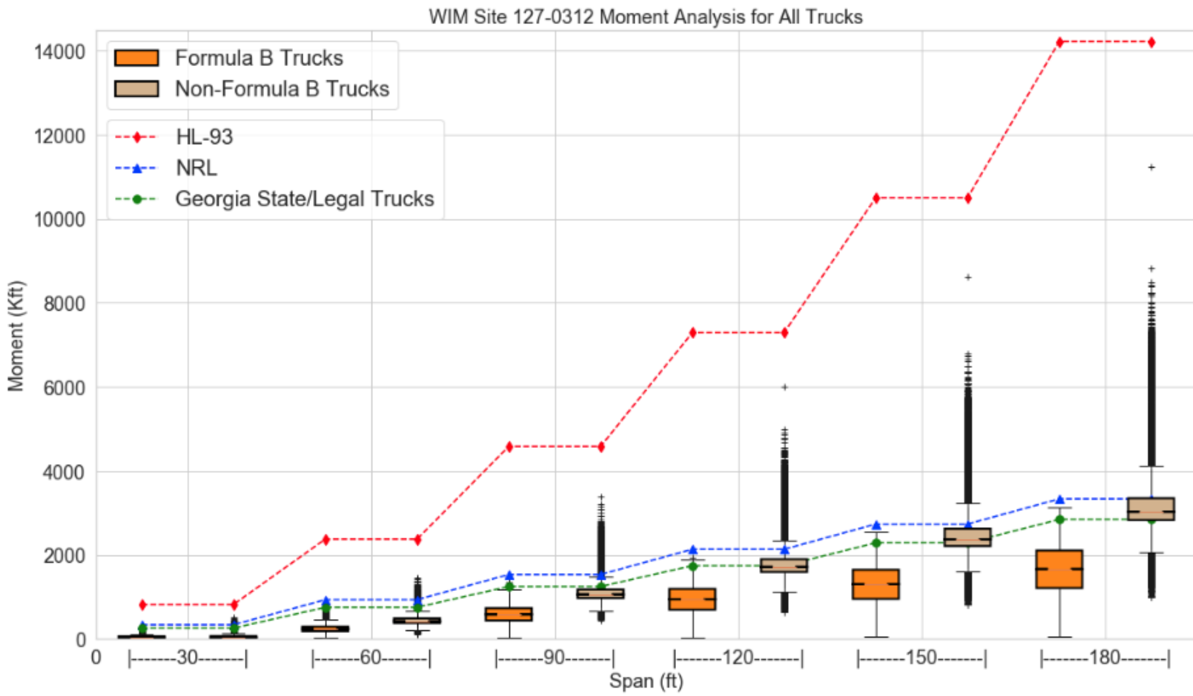


(c) WIM Site 143-0126

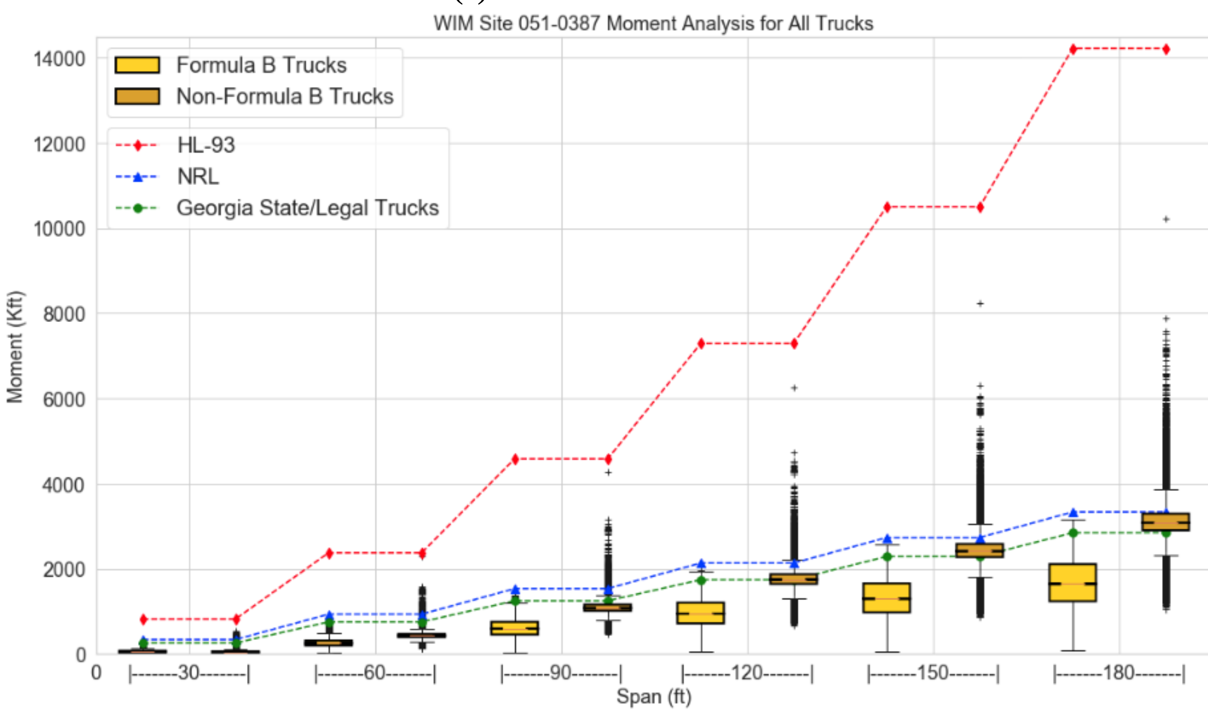


(d) WIM Site 217-0218

Figure 45 Continued – Moment Analysis for All Trucks.

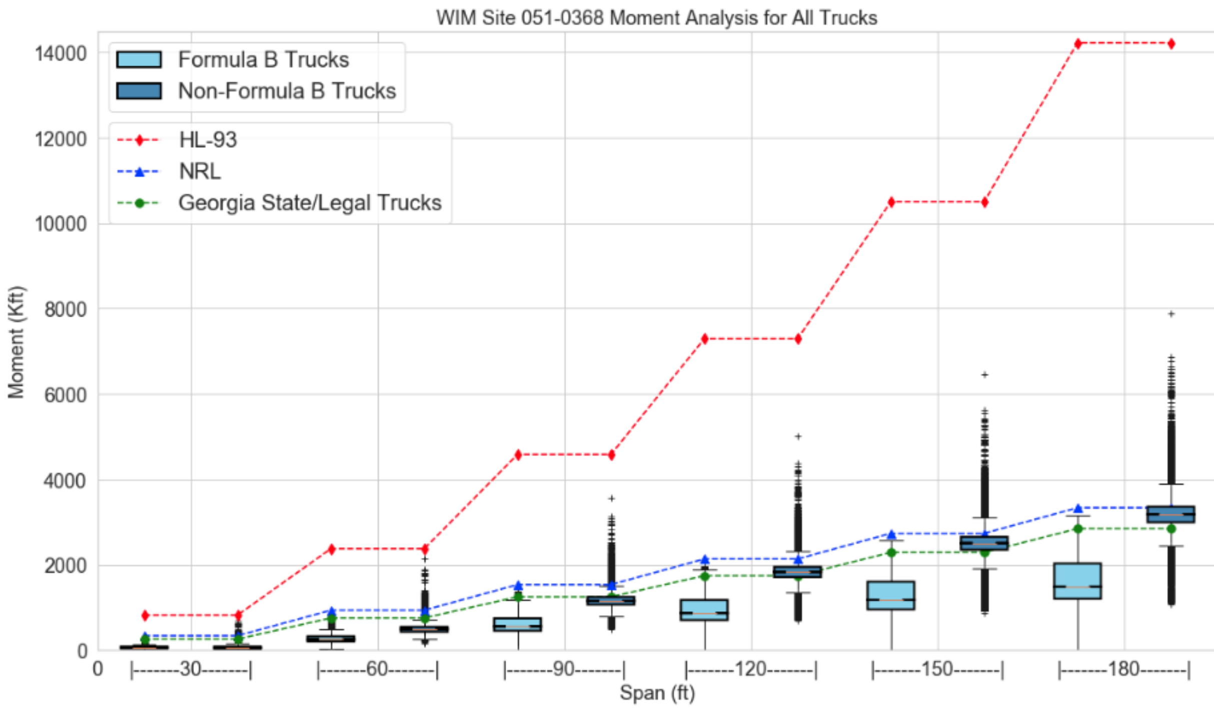


(e) WIM Site 127-0312

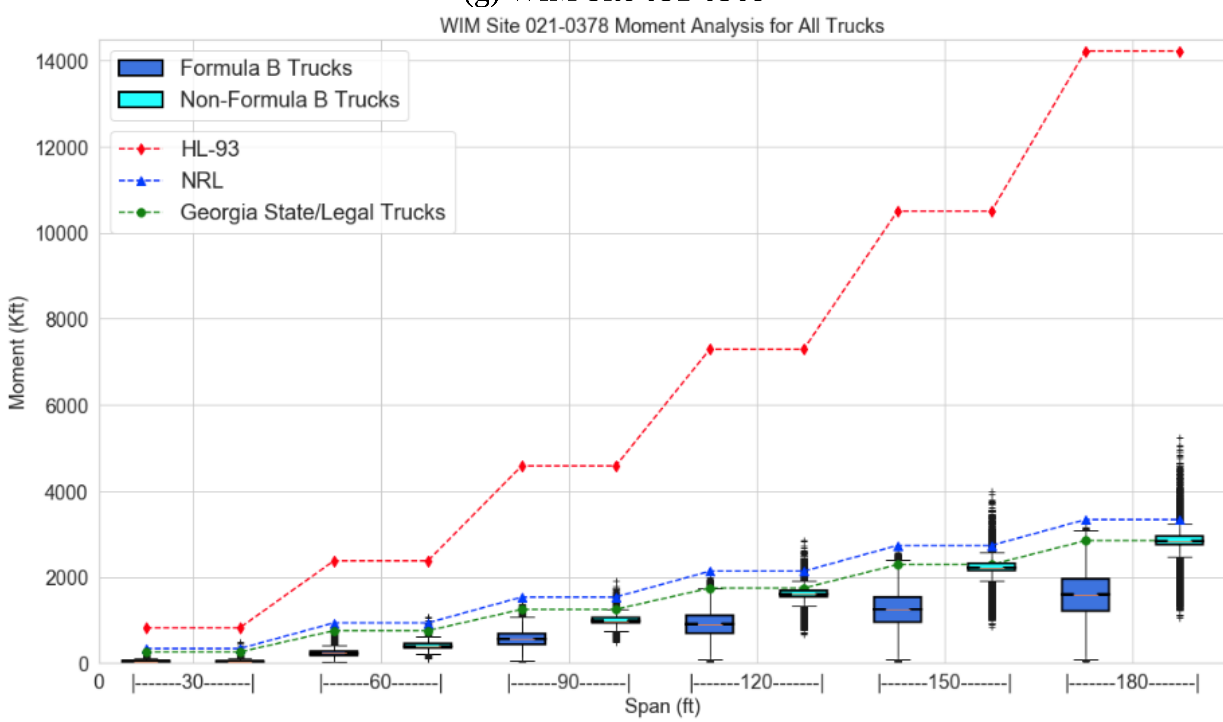


(f) WIM Site 051-0387

Figure 45 Continued – Moment Analysis for All Trucks.

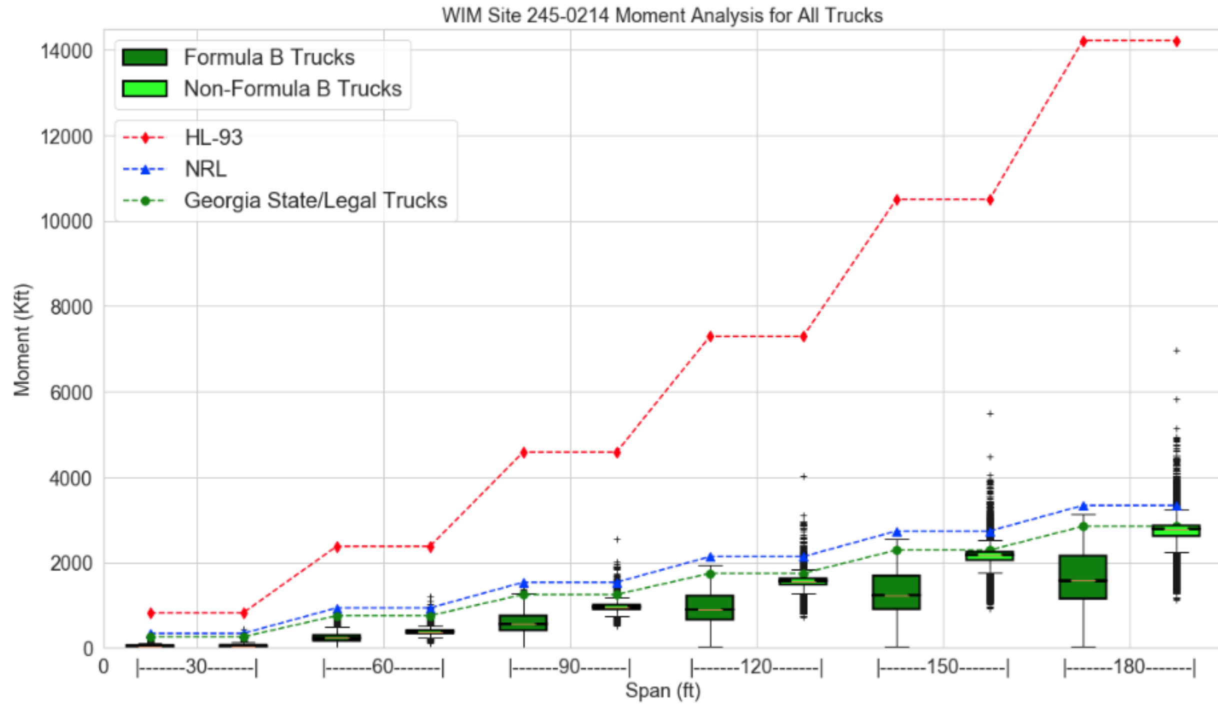


(g) WIM Site 051-0368

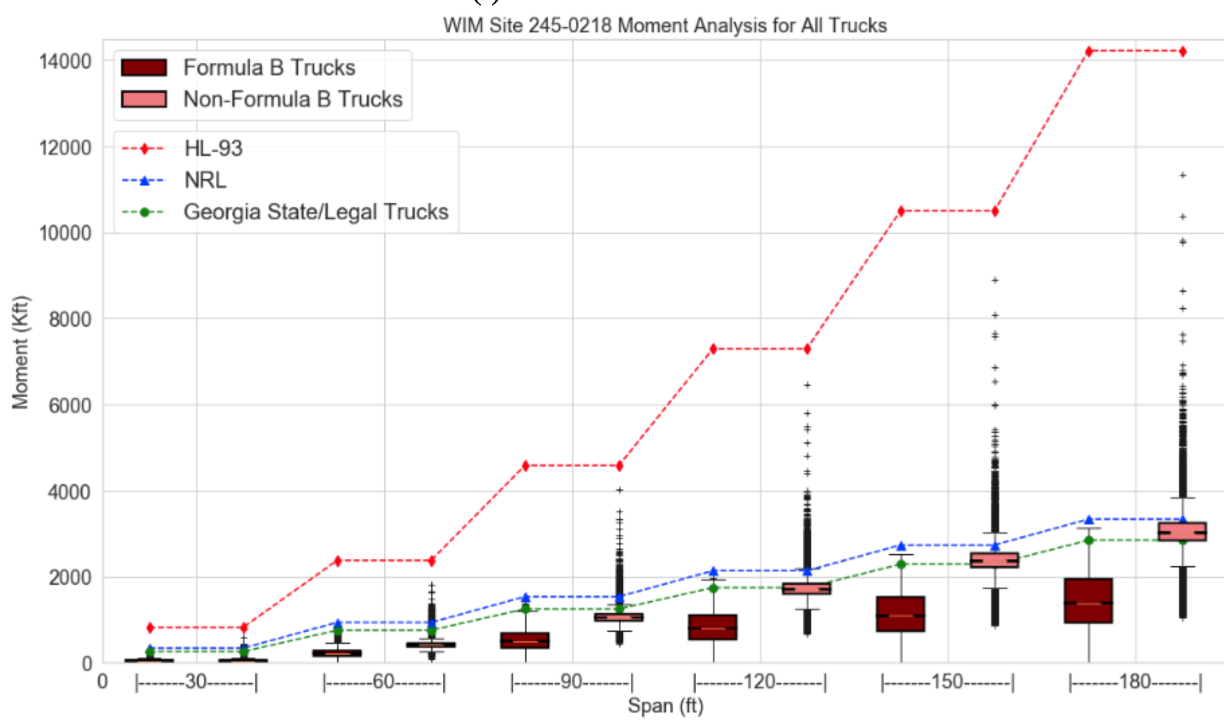


(h) WIM Site 021-0378

Figure 45 Continued – Moment Analysis for All Trucks.

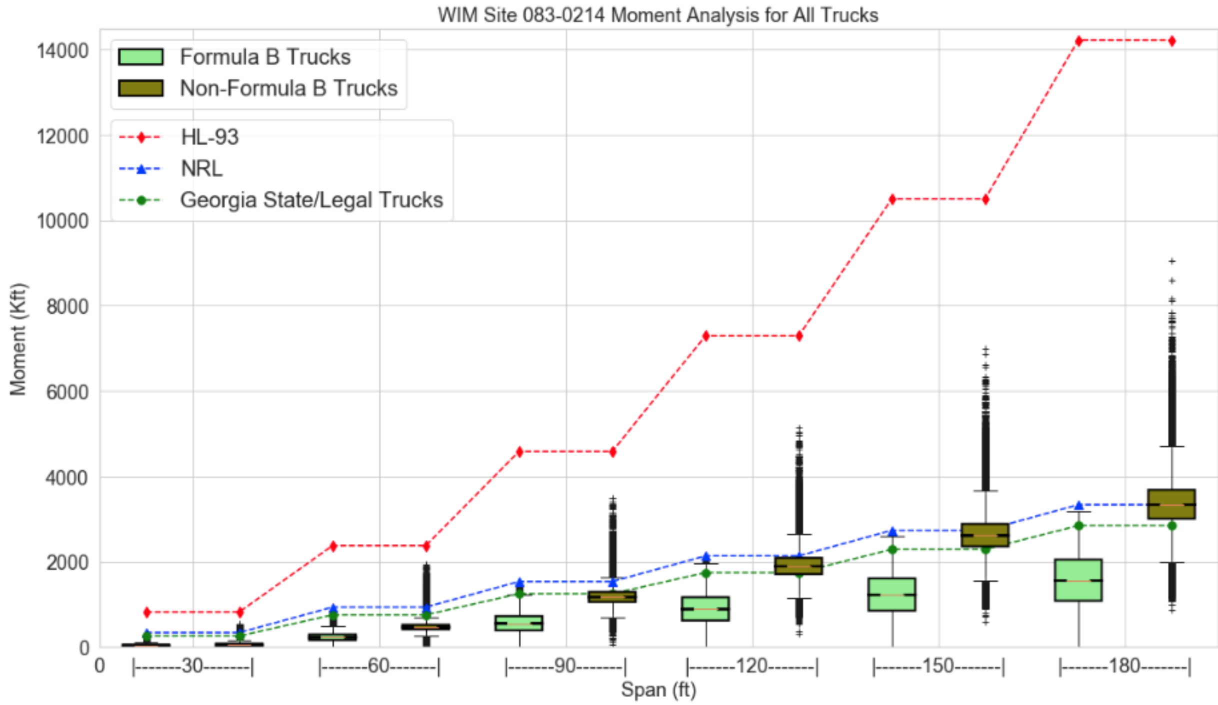


(i) WIM Site 245-0214



(j) WIM Site 245-0218

Figure 45 Continued – Moment Analysis for All Trucks.



(k) WIM Site 083-0214

Figure 45 Continued – Moment Analysis for All Trucks.

The moment analysis of HL-93 design load followed similar trends to the shear results as the Non-Formula B plots were more compacted and presented multiple outliers. However, its maximum WIM shear results did not, as only one site exceeded the HL-93 design load when factored. The rest of the WIM data fell well below the factored HL-93 line signifying that shear controlled as it did for the Formula B truck analysis testing NRL and Georgia state/legal loads. The sites were then applied the same furthered examination as the shear and is provided in Table 17 and Figure 46 below.

Table 17 – HL-93 Factored Moments.

Span Length	HL-93 Factored Moment	Sites that Exceed
30 feet	820.75 kft	W047-0114
60 feet	2,375.33 kft	N/A
90 feet	4,580.33 kft	N/A
120 feet	7,289.33 kft	N/A
150 feet	10,502.33 kft	N/A
180 feet	14,219.33 kft	N/A

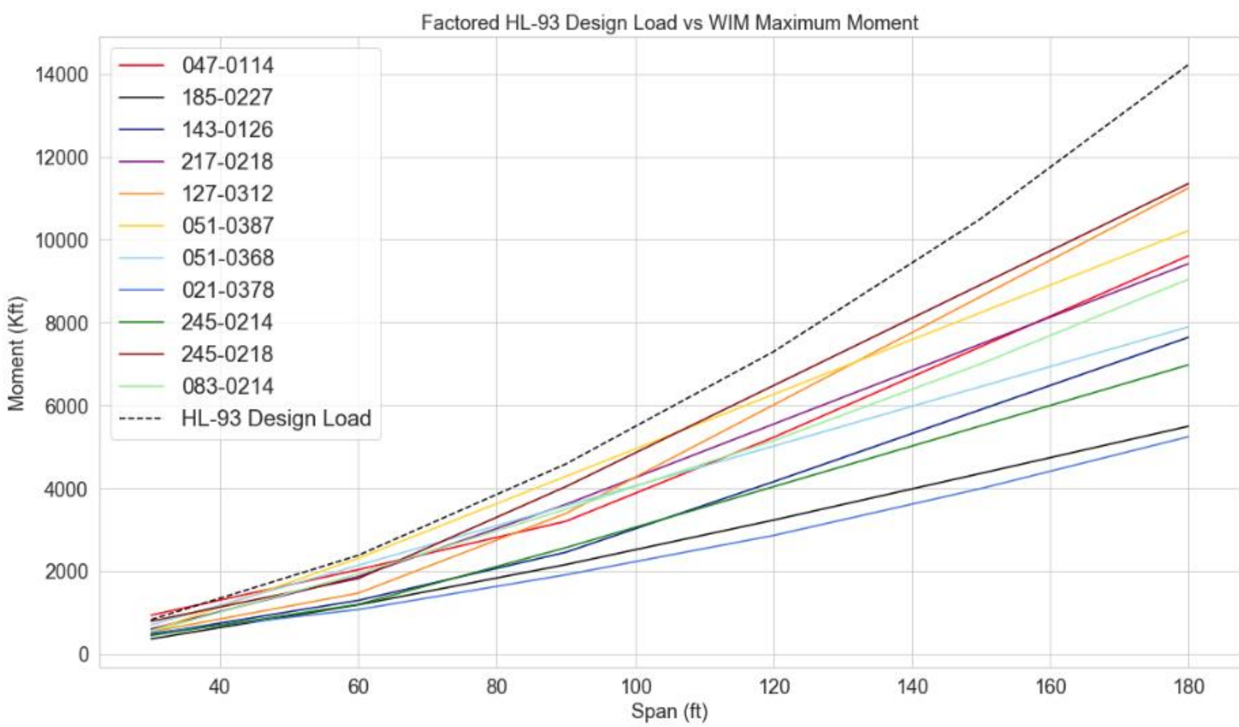


Figure 46 – Factored HL-93 Design Load vs WIM Maximum Moment.

As the results indicated in both Table 17 and Figure 46, it was concluded that the majority of bridges along the routes meet the factored design criteria of HL-93 and were determined safe for all spans ranging from 0 to 180 feet based on moment. However, one of the sites did produce a specific value that did not pass the factored loading standard. WIM sites W047-0114 produced a maximum moment greater than the HL-93 factored design load at a span length of 30 feet. The site's maximum moment exceeded the HL-93 factored moment by approximately 200 kft. Therefore, the HL-93 design load was concluded as a good representation of all truck basics based on moment as it embodied a significantly large percentage of WIM trucks but should be certified according to shear as some results exceeded the HL-93 design basis.

4.2.3 Permit Trucks

Trucks that exceed Formula B requirements need a permit to legally drive on state roads and bridges. However, the results of this study indicated a large portion (3%-31% per WIM site) of Non-Formula B trucks, and many of these trucks barely exceeded a specific category of the Formula B requirements providing the theory that many of these trucks are not permitted either by mistake, WIM error, or illegal motives. Therefore, the trucks under this category were investigated further in this chapter to review truck weights and provide GDOT with data to explore further. Figure 47 is provided first giving information on the total number of Formula B and Non-Formula B trucks per site in 2019.

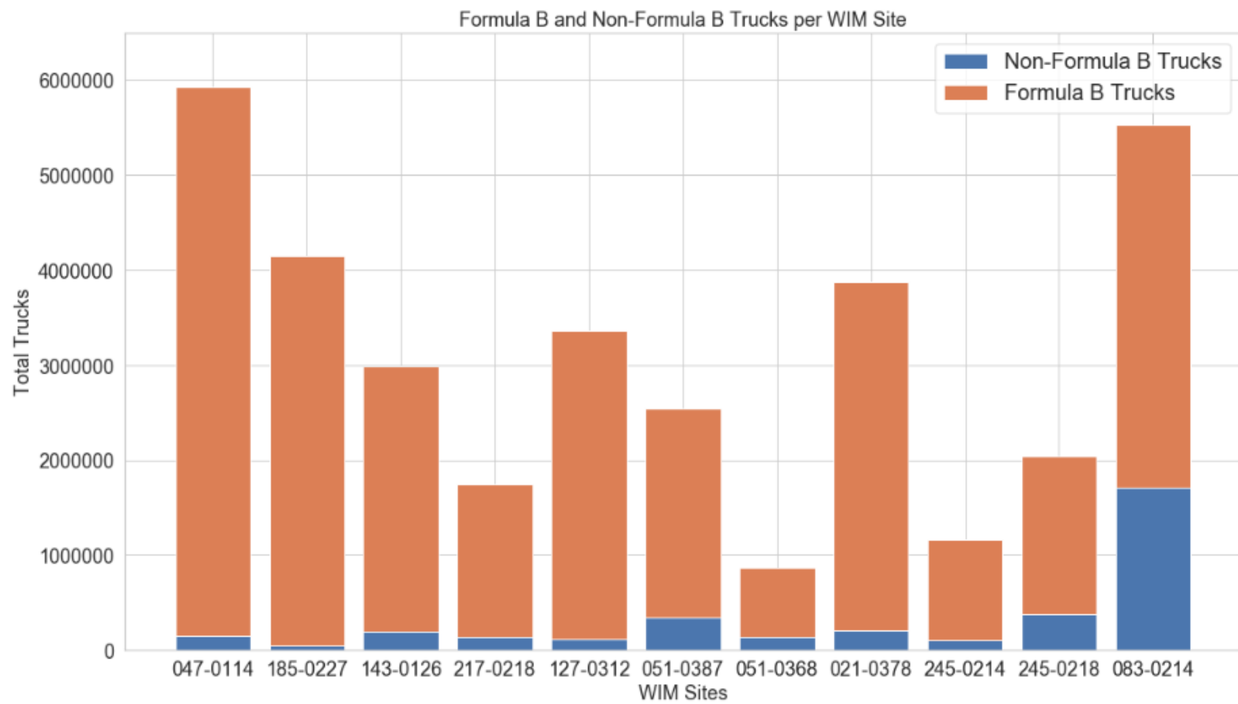


Figure 47 – Formula B and Non-Formula B Trucks per WIM Site.

It was seen that the sites vary in the total truck range of around 1 million to 6 million trucks in the year 2019. WIM site 047-0114 provided the largest amount of total trucks while site 083-0214 was a close second. WIM site 083-0214 did provide the most crucial evidence as a large portion of around 31 percent of its total trucks were Non-Formula B. That means that a little less than 2 million trucks should have received permits for that specific route just in 2019. A total of 3,529,696 trucks were recorded as being Non-Formula B and needed a permit to perform its travel. That was a total of 12.32 percent of the total truck population for just the 11 sites involved in this study.

Many of these trucks barely exceeded the requirements leading to the issue of WIM calibration. Therefore, the trucks were examined another step in comparison to NRL to see the percentages per span length that caused a significant effect to need a permit to avoid posted bridges. The results for shear and moment are shown in Figure 48 and 49 respectfully. The WIM sites in the figures are color coordinated with Figure 33. In Figure 48, each line represents the percentage of trucks that exceed the maximum shear values of the NRL truck model as this model represents the largest values for Formula B trucks. Figure 49 does the same but for moment. Therefore, the truck percentage of each of the span lengths in these two figures illustrate the amount of Non-Formula B trucks that need a permit and can produce a dangerous amount of shear and moment values.

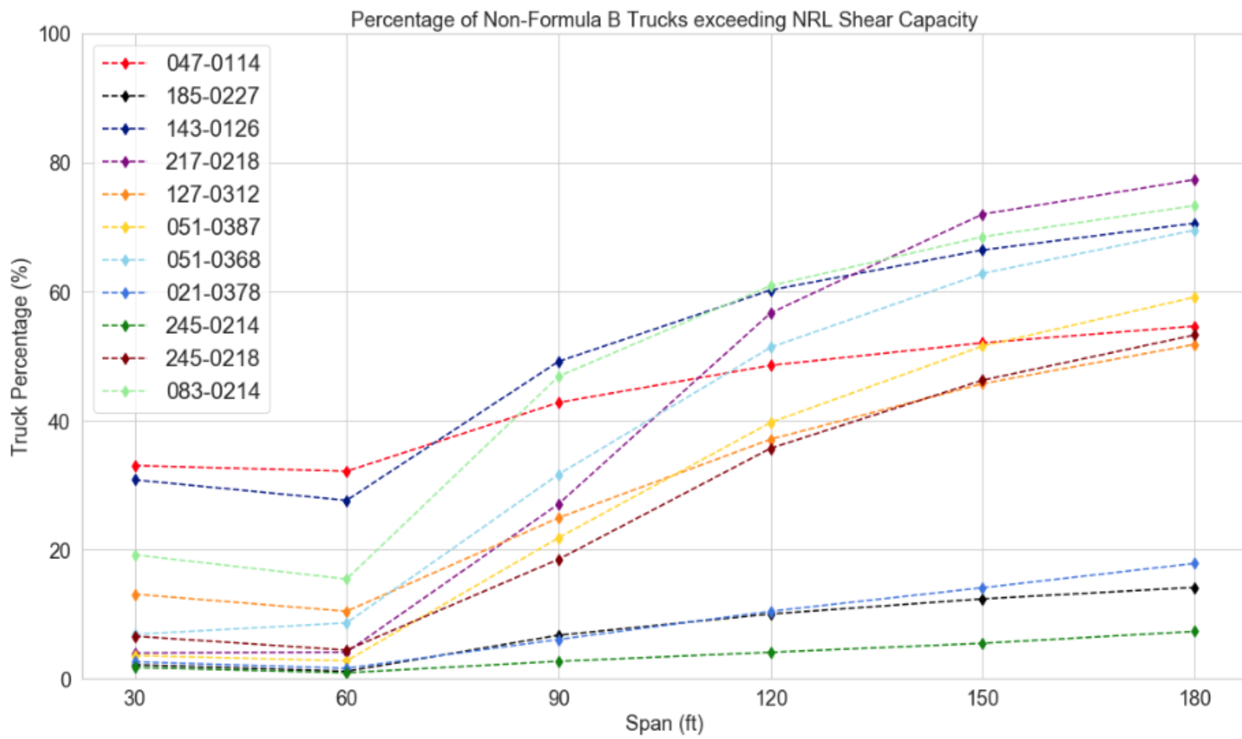


Figure 48 – Percentage of Non-Formula B Trucks exceeding NRL Shear Capacity.

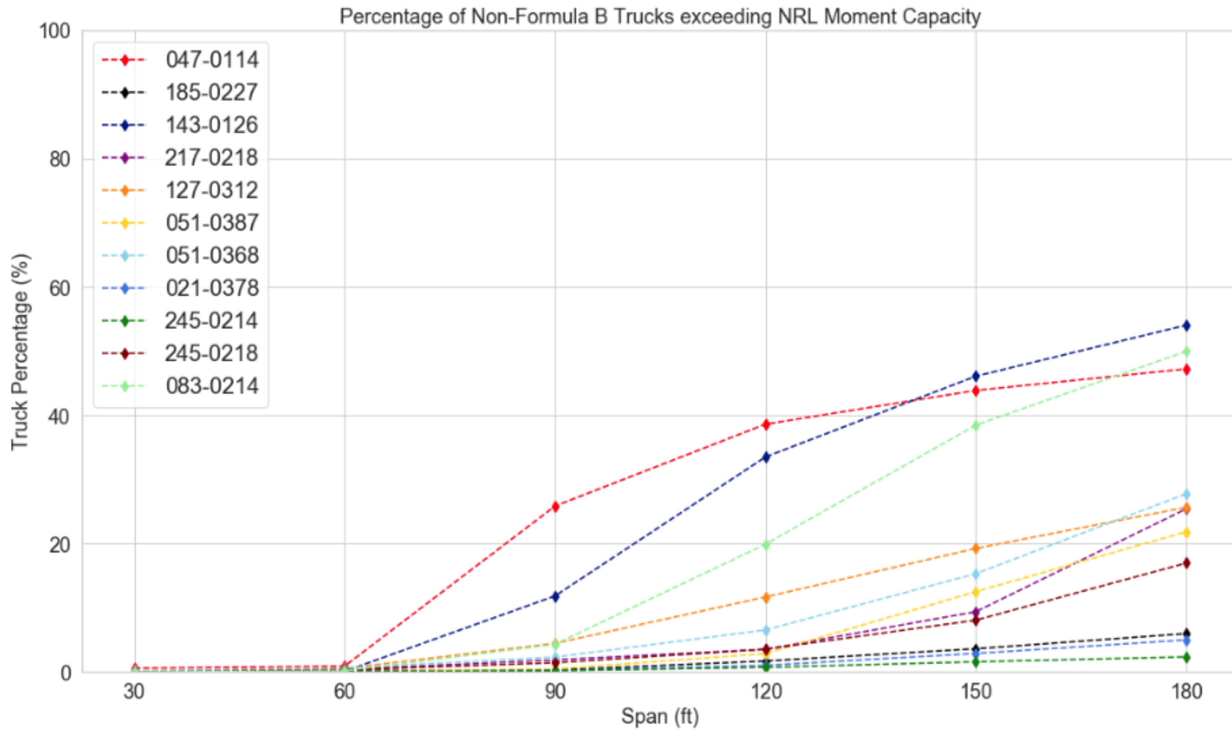


Figure 49 – Percentage of Non-Formula B Trucks exceeding NRL Moment Capacity.

Figure 49 illustrated how three WIM sites consisting of 047-0114, 143-0126, and 083-0214 provided the largest percentage of trucks exceeding NRL moment needing to avoid posted bridges at 47.20, 54.01, and 49.93 percent for 180 span length. It can additionally be analyzed that percentages were insignificant in the span range of 0 to 60 feet as values did not increase until the 90-foot length. Therefore, it was assumed that bridges below the 60-foot mark are highly unlikely to experience issues with permit trucks due to moment.

This is not the case for shear shown in Figure 48 as the percentages ranged up to near 40 percent in the 30- and 60-foot span lengths. As the span length increased up to 180 feet for shear, the range then became between about 50 to 80 percent. The largest percentages at the 180-foot span were found at sites 217-0218, 083-0214, and 143-0126. These values indicated that a large amount of the 12.32 percent of the Non-Formula B Trucks found in Figure 47 weighed enough to exceed NRL and needed to avoid posted bridges throughout the state.

WIM site 083-0214 recorded a large percent of Non-Formula B trucks and is among the highest percentages exceeding NRL in both shear and moment. This route should be analyzed further as many of its trucks are considerably large and cause significant damages to bridges if not routed the correct way with a permit. Fortunately, this route is located in Northeast Georgia and only crosses a few bridges on its way through the state which are not listed under the posted category. This information was provided for GDOT to ensure the correct percentages of permitted trucks and to help identify routes experiencing large portions of overweight vehicles.

4.3 Deck Condition Assessment

Once the load demand was determined and discussed, the condition analysis was conducted. This analysis features how the truck traffic count can affect deck condition ratings over time. Due to the lack of accurate and available WIM data in Georgia so far, only two years of data for ADTT was examined. Therefore, only the last two years of NBI data was used as well. Hypothetical data was illustrated to provide an idea of how this assessment would look once more WIM data is gathered in the coming years. Figure 50 below presents this data as the dotted lines represent hypothetical or made up data to provide an idea of how this process would look in years to come. The black line represents how truck traffic demand or ADTT will increase with time while the remaining line plots are individual deck scores for randomly chosen bridges. This figure shows how as ADTT increases in time, the deck scores will deteriorate.

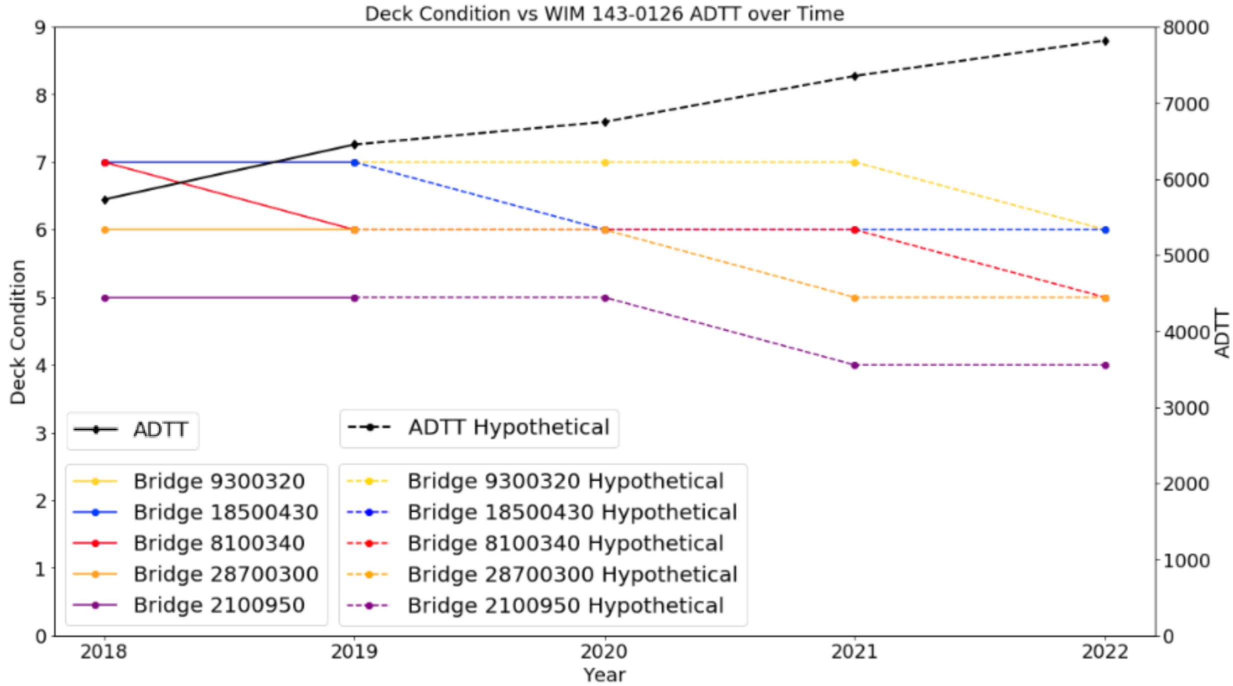


Figure 50 – Deck Condition vs WIM 143-0126 ADTT Over Time.

WIM Site 143-0126 was chosen due to its large volume of traffic and availability of ADTT and NBI data. Once more data is gathered in the years to come, more sites can be graphed with more accurate data. The goal of this procedure was to provide GDOT with an approach to determine how deck ratings will deteriorate over time due to the amount of truck traffic it is experiencing. With this system, a more efficient technique can predict how bridges will deteriorate in the future and begin to allocate resources for maintenance, rehabilitation, and repair by reviewing traffic reflected in the WIM data and forecasting traffic growth. This assessment will begin as the framework for GDOT through the combination of WIM and NBI data to accurately measure bridge performance.

This assessment coupled with the load demand of WIM data can create demand-to-capacity ratios quantifying the reduction of bridge capacity due to truck traffic. This process will prove to be very beneficial as bridges highlighted for capacity reduction will be selected and receive rehabilitation or replacement years before signs of deterioration are detected. This not only increases safety but additionally decreases the number of posted bridges within the state as bridge posting is growing each year {GDOT, 2019 #37}. This analysis can be extended even further to quantify bridge asset usage and growth in usage through the installation of more WIM sites as larger portions of the state can be covered.

4.4 Following and Side-by-Side Probabilities

Through the use of WIM data, new truck data should be formed being state/route specific instead of the current data for load rating based on a 2-week study from Ontario in 1975 as defined by NCHRP Report 454 discussed in Chapter 1.2.10. This data does not represent the same traffic experienced within the state of Georgia because trucks, particularly special hauling vehicles, have been modified over the years, which is proven by the NRL. In order to investigate if Georgia undergoes different truck traffic than the Ontario data, the following probability and side-by-side probability of both sets were compared. These two possibilities were chosen as it details the likelihood of multiple trucks on the same bridge span at the same time leading to important variables in determining the maximum forces a bridge experiences each year.

The following probability was calculated for all 11 sites and was averaged based on each direction of traffic flow per site. These values were compared with the Ontario following probability of 2.00 percent according to NCHRP report 368. These probabilities are based on the headway distance being 100 feet and the results of the procedure are illustrated in Figure 51 below.

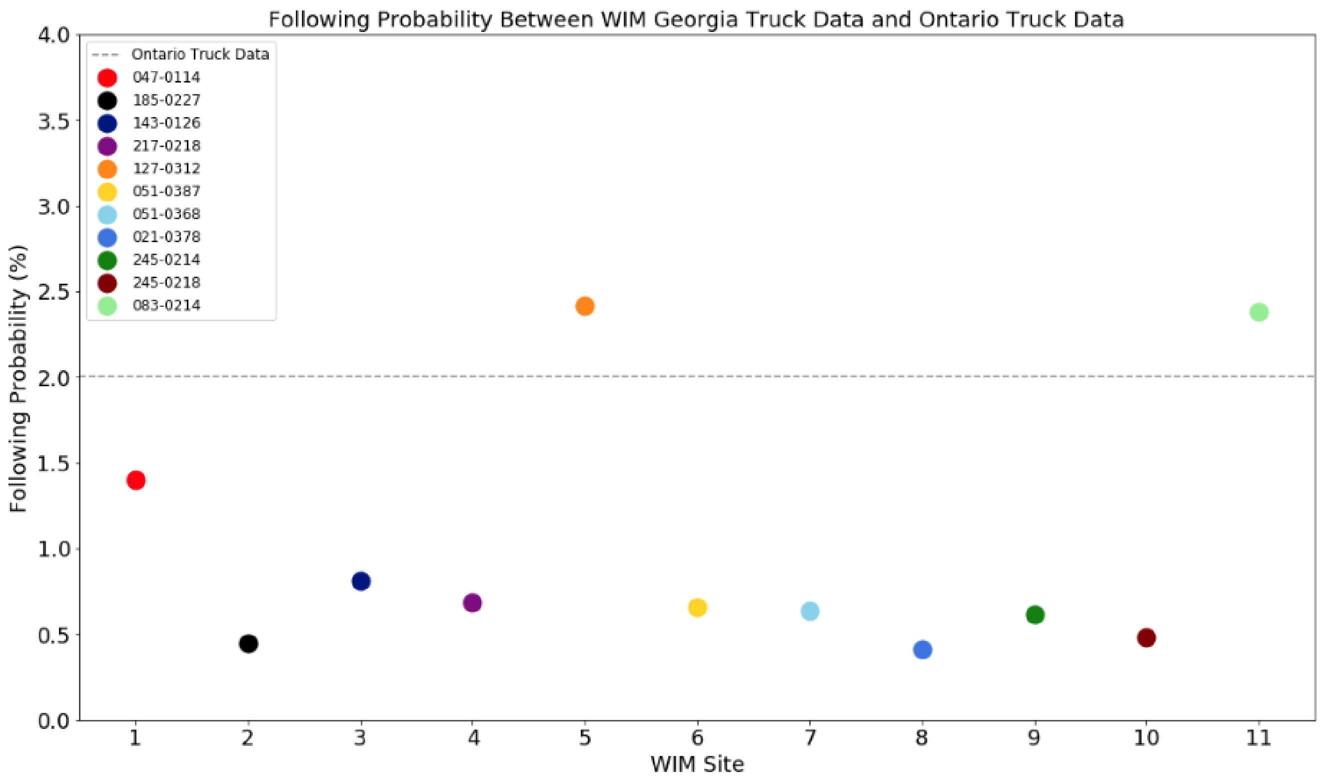


Figure 51 – Following Probability between WIM Georgia Truck Data and Ontario Truck Data.

The results of the 11 WIM sites were as expected as a 2.00 percent following probability (represented by the dotted line) seemed overestimated. The majority of the Georgia sites produced values below 1.0 percent while the heavily populated I-75 North of Atlanta 047-0114 WIM site provided a 1.40 percent following probability. However, two sites did surpass the 2.00 percent Ontario truck data at the I-95 South of Savannah 127- 0312 WIM site and the I-24 South of Tennessee border 083-0214 WIM site. The I-95 route is a major truck hub for transporting goods to Florida leading to its high following probability. The I-24 route connects Alabama to Tennessee and passes through Georgia for small period; therefore, it only represents a couple of bridges within the state.

The side-by-side probability was computed next for each of the 11 sites and averaged on each direction of traffic flow similar to following probability. These results are illustrated in Figure 52 below and are compared to the 6.67 percent of the Ontario dataset. The Ontario 6.667 percent is plotted by the dotted line.

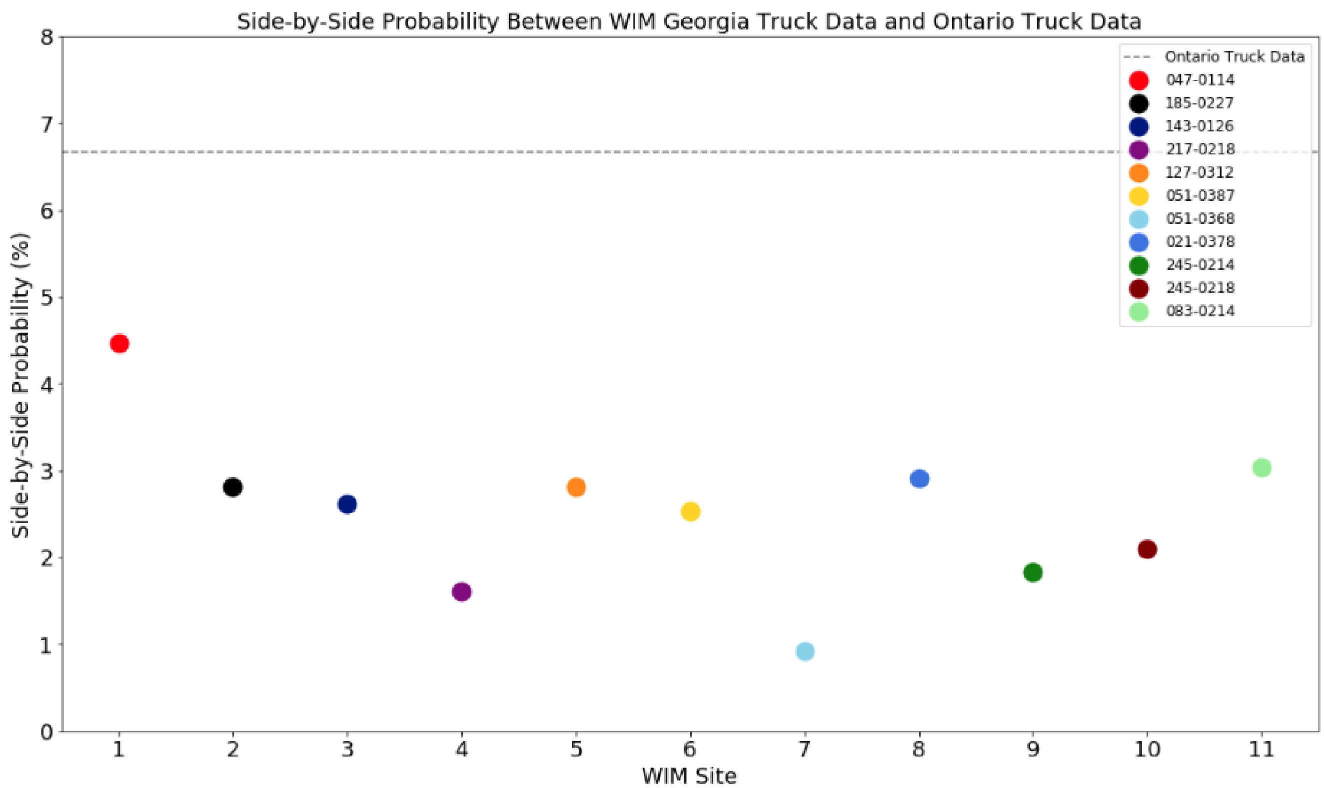


Figure 52 – Side-by-Side Probability between WIM Georgia Truck Data and Ontario Truck Data.

The side-by-side probability resulted as predicted as every Georgia WIM site produced low values when compared to the Ontario truck data. The largest side-by-side probability was found at the highest volume site in the state of I-75 North of Atlanta 047-0114 WIM site at 4.47 percent. This indicated how much different the Georgia data are compared to the 1975 Ontario data as the average of the Georgia WIM data is around 2.50 percent compared to the Ontario average of 6.67 percent.

Moreover, it was concluded that Georgia following probability is less than the Ontario truck data as Georgia's overall average is approximately 1.00 percent being half of the Ontario's average. The side-by-side probability was additionally much smaller than the Ontario data of 6.67 percent as Georgia resulted in 2.50 percent. This provides strong evidence that Georgia should utilize WIM data to create a state specific truck data frame to be used for design and examination in load rating. Sites do differ as displayed in Figures 51 and 52, therefore, it is recommended to use the truck data to create route specific truck data, instead of just state truck data, to increase its reliability in representing the bridge data per state location. Additionally, this data can be used to indicate which bridges are experiencing the worst-case scenario of loading due to multiple trucks to increase lifespan and safety of bridges. It can further locate where traffic congestions occur and mitigate the problem.

5. CONCLUSION

As traffic volume is increasing each year on Georgia bridges, its components are deteriorating faster reducing its overall strength capacity. Weigh-In-Motion sites have been placed throughout the state by GDOT to gain insight on what Georgia bridges are undergoing. However, a use for this mass amount of WIM data has not been provided, therefore, leading to the purpose of this report. This study was conducted to find a use of this WIM data and help classify traffic demand in Georgia. The WIM data was assumed to be accurate and the scope was adjusted to just National Highway System (NHS) bridges as the WIM data best represented major routes. This process was performed under the resources of the NBI database and NCHRP Reports 454 and 575 {Board, 2001 #32; Board, 2007 #34} while being composed through Python Programming and ARC GIS.

Initially, the NBI database was investigated as its traffic count per bridge is given, although not updated. This study tested the theory that the traffic counts in the NBI are different from those observed in the WIM data. Seven WIM sites were associated with specific NHS bridges in the evaluation of this notion as it should provide precise counts of both ADT and ADTT. The results of the data proved to meet the proposition as both cases of ADT and ADTT did not correlate between NBI and WIM data as confirmed by a Kruskal-Wallis test. The ADT counts for NBI data were either overestimated or underestimated compared to the WIM data. However, the ADTT for NBI were significantly underestimated compared to WIM data. This was a concern as designers and engineers refer to these values for their work, especially the truck traffic data which controls the live loads on bridges. With this in mind, it was concluded that the NBI dataset should contain as-measured WIM ADT and ADTT data as they were used to quantify asset usage and deterioration.

Additionally, WIM site 047-0114 representing I-75 between Atlanta and the Tennessee border produced the largest ADT and ADTT counts of 48,675 and 9,267, respectively leading to the perception that its route yields the largest amount of traffic, which is consistent with the previous findings presented in Georgia's Freight and Logistics Research. Due to this analysis, this site was explored further as well as four sites representing the Savannah port as these two areas characterize maximum truck traffic within the state. The maximum axle weight per truck and the total truck weight were determined and plotted for each site for comparison. Both sites resulted in similar distribution in shape and range. The maximum axle weight for each site produced a bimodal distribution while the total weight plots were multimodal with three peaks leading to the ability to classify each group with different sections depending on weight. The range on both sites for maximum axle weight was between 2,000 lb to 35,000 lb and was between approximately 5,000 lb to 150,000 lb for total weight. The sites did include a few trucks acting as major outliers with the maximum axle weight resulting in 72,240 lb for each site. As for total weight, the I-75 route produced a maximum value of 292,398 lb while the Savannah sites had an even greater weight in 366,717 lb. These values were extremely high and should be examined in more detail as a truck with such large magnitude may cause a catastrophic failure. However, it should be noted that the WIM data can fluctuate due to high speeds or temperatures and could be the reason for these large maximums.

The live load demand of trucks within the state was subsequently evaluated. This evaluation included the use of eleven WIM sites and associated data collected in 2019. The trucks on these sites were run through a program calculating its maximum shear and moment quantities for varying span lengths. The truck load demands were compared with HL-93 design load and Georgia state/legal loads. The shear and moment analysis indicated that the Notional Rating Load truck configuration, as concluded in NCHRP Report 575, is a more reliable option than the Georgia state/legal loads as it encompassed a larger percentage of the WIM truck weights. The shear forces did show some trucks exceeding this NRL truck; therefore, it should be considered a minimum requirement instead of a conservative check for load rating. As for the HL-93 design load, only one site produced greater moment demands than its factored form while a small percentage of trucks on some sites surpassed its shear force demands. This proves that the HL-93 design load is still a quality representation of all basic truck loading as only a significantly small percentage exceeded its values, however, a couple of the trucks exceeding its factored shear forces were around 50 kip larger. This leads to the idea to either further analyze the HL-93 design basis for Georgia or validate the WIM data more in order to confirm its reliability. Around 3.50 million trucks examined in this study were considered to not meet the Formula B requirements, which means they required the use of a permit to travel. Non-Formula B trucks exceeding NRL values for both shear and moment were screened to analyze the amount of these trucks.

A condition assessment was performed next linking ADTT counts with bridge deck condition scores to analyze how traffic demand affects the deterioration of bridge performance. However, only two years of WIM data was available for use in this category. Therefore, hypothetical data was added to this section to provide an idea of how this study would look once more WIM data is gathered. The overall goal of this process is to provide GDOT with the framework to test how bridge capacities undergo reduction over time due to load demand and eventually add other variables that affect deterioration. This will create a system predicting failure in certain bridge elements in less time and ultimately real-time. This will allow quicker action to be taken upon these elements to replace its deteriorating components reducing the amount of posted bridges and increasing cost efficiency in the department.

Finally, the following probability and side-by-side probability of Georgia WIM data was calculated for comparison with current truck data used from a two-week study conducted in Ontario data in 1975. The purpose of this analysis was to provide evidence that Georgia truck data differs from Ontario data, as well as to provide additional statistics for multiple-lane calculations and accounting for multiple vehicle presence within a lane. The two probabilities were much greater than what Georgia trucks experience. Thus, it was concluded that Georgia truck data is very different from the Ontario dataset. The following probability for Georgia resulted in an average of 1.00 percent between the eleven WIM sites which is half of the Ontario percentage. As for the side-by-side probability, Georgia WIM data averaged 2.50 percent while Ontario data averaged 6.67 percent. Therefore, for future reference, Georgia should utilize WIM data for state/route specific truck data as its numbers are more specific to the state and records vehicles over the entire year as compared to just a two-week study performed 45 years ago by the Ontario dataset.

6. RECOMMENDATIONS AND FUTURE WORK

This thesis provided strong evidence that more WIM sites are needed throughout the state in order to provide the most effective way to define truck traffic throughout the state. Georgia currently has around 19 WIM sites in the state while other states including Indiana and Michigan have over 40 WIM sites according to the Federal Highway Administration (FHWA WIM System Technology, 2017). New Jersey has around 75 based on NJDOT intelligence (NJDOT, 2018). In order to capture the full picture of live load demands on bridges in Georgia, additional sites need to be placed in strategic locations to collect valuable information for traffic usage and asset management. The current and recommended WIM sites are displayed in Figure 53 below.

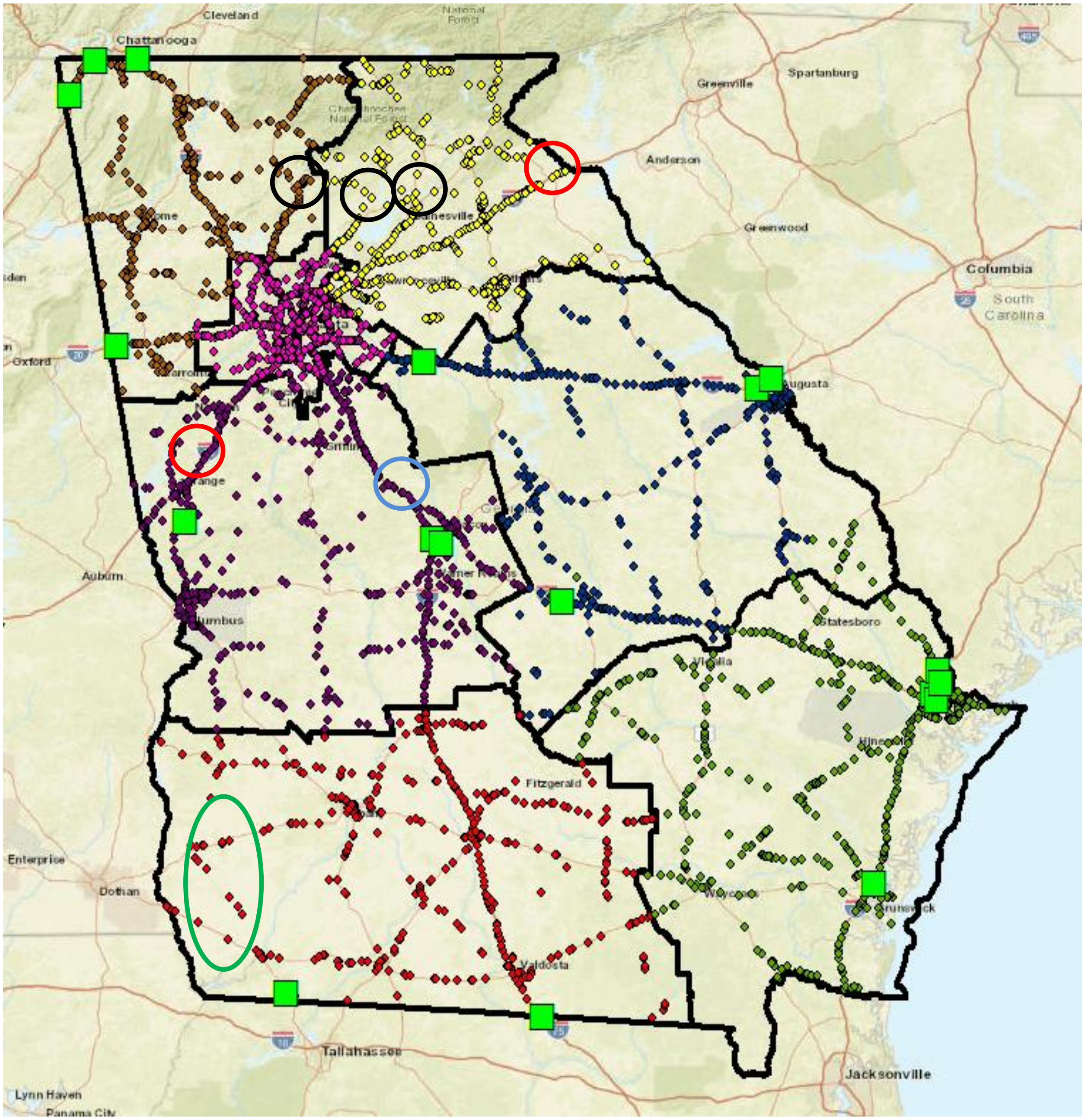


Figure 53 – ARCGIS View of Current and Recommended WIM Sites in Georgia.

Figure 53 shows that the majority of major highways are captured by at least one WIM site for analysis. However, a few areas representing crucial bridges throughout the state lack their own WIM site and are represented by the circles on the figure. The main interstate in need of a WIM site is I-85 as the load on these bridges cannot be determined for structures above and below Atlanta. A WIM site is located around I-85 near Lagrange, Georgia but does not represent the bulk of vehicles on its route. Therefore, the circles signified by the color red are where I-85 placement of sites would be recommended to maximize its potential.

As for bridges on I-75 located below Atlanta and above Macon, a light blue circle has been marked since the current WIM sites in Macon cannot represent this area as they are located after the split, resulting in only a portion of the traffic being recorded. Another site or sites should be placed in the southwestern portion of Georgia to gain more data dealing with the transportation of heavy timber products flowing in and out of Alabama. Cases involving the illegal (i.e., overweight) shipping of timber have been discovered causing concern in the amount of weight bridges within this area might be experiencing. A green circle represents this area as multiple sites can be placed here to record this timber data. Finally, a few more sites are recommended by the black circles all located above Atlanta. These circles are placed on routes consisting of I-575, SR 400, and SR 985 as bridges on these sites do not experience the amount of truck traffic found on neighboring routes of I-75 and I-85, but do undergo large percentages of truck loading.

The vast majority of bridges in the state are found within metro Atlanta and no WIM sites represent this center hub. This is due to the fact that WIM sites should not be placed along roads facing congested traffic and need a flow of vehicles to record properly. However, Atlanta contains some of the most vital bridges found in the state and need a practical way to determine its traffic demand. Therefore, it is recommended to invest in Bridge Weight-In-Motion (BWIM) technology as its system does not require the need of constant moving traffic and will not need the direct contact of WIM system making it more durable. It additionally does not need the halting of traffic for installation as it is placed underneath the bridge. This means it will record the exact forces the bridge is undergoing but does have a higher cost when compared with WIM systems. More information of this type of technology is discussed in the literature review in Chapter 1.2.2.5.

Another recommendation would be to use WIM data to update NBI annually for ADT and ADTT. This study indicated how the NBI does not provide accurate evidence to represent the traffic flow on its bridges especially in the ADTT category as the NBI underestimates its amount. Through the use of WIM annual data, it can be analyzed to properly update the NBI for engineers and designers to use as a source for bridge work.

Additionally, a suggestion to increase the awareness of the importance on the NRL configuration beyond the AASHTO-MBE requirement in the bridge load rating evaluation is recommended. Its load configuration covers the forces all Formula B trucks created on each of the eleven WIM sites tested. The current procedure calls for the evaluation compared to Georgia state/legal trucks which did not cover all the non-permit trucks as indicated in Chapter 4.2.1. The NRL rating model, in addition to the Georgia/AASHTO legal loads, provides a more reliable screening load in Georgia and characterizes the modifications trucks have experienced over the years. A recommendation for the NRL truck model would be to add a tandem force, similar to the HL-93 design load, to increase the maximum shear values for lower span lengths as indicated in Figure 41. The final recommendation would be to investigate the HL-93 design load further as a few values for shear exceeded its maximum as seen in Figure 43 and to adjust it accordingly to handle all trucks.

The future work to be conducted after this study should primarily focus on the condition assessment discussed in Chapter 4.4 as it could not be conversed completely due to the lack of WIM data. The WIM data in Georgia have only been around for two years making it difficult to predict future trends and deterioration patterns. Furthermore, the reliability of the data must be assessed. Once the reliability is verified and more sites placed, a full assessment can be directed to effectively manage bridge analysis and provide valuable usage information for asset management. The goal would be to use the available truck traffic data provided by WIM to estimate how condition ratings in the deck change over time. This examination combined with the load demand provided by WIM will unlock the ability to assess the demand versus capacity, which is calculated by quantifying the reduction in capacity a bridge undergoes based on traffic, loading, and time.

Other bridge elements can be taken into consideration as well including both superstructure and substructure. The deck condition was the main feature focused on as it is in direct contact with vehicles crossing over the bridge. Once other elements are added, many factors including age, weather/climate, and construction practices can be analyzed along with traffic volume. This will open the capability to see what factor causes which effect on the deterioration of a bridge. Therefore, bridges can become more predictable allowing GDOT to estimate when it will need restoration beforehand and create an order for their future work. This will help GDOT maximize their efficiency by saving time and money, improving safety, and reducing the amount of posted bridges in the state.

Additionally, ideas for future work would be to create and test a refined NRL truck configuration that is able to handle the maximum shear values of every WIM truck recorded in Georgia. This new truck configuration would ensure the reliability of the condition assessment for bridge evaluation. The HL-93 design load can also be investigated further to see if it truly represents the entire population of Georgia trucks and if new modifications to its loading is necessary. Finally, the accuracy of vehicle weights in the WIM data should be investigated.

7. APPENDIX

Appendix A: Python Code – Formula B Trucks Moment Analysis Example

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
WIMMM = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\W_WI
M Data\2019 Data\2019-000000830214.csv', low_memory=False)
WIMN = WIMMM['Lane Name'].str.contains('EB')
WIMs = WIMMM[WIMN]
WIM7 = WIMs['Class'] > 7
WIM8 = WIMs[WIM7]
WIM13 = WIM8['Class'] < 14
WIMM = WIM8[WIM13]
WIM6 = WIMM['Num Axles'] < 9
WIM = WIMM[WIM6]
Weight = WIM['Axle Weights (kg)'].str.split('|', expand = True)
Weight = Weight.astype(float)
Spacingss = WIM['Axle Spacings (m)'].str.split('|', expand = Tru
e)
Spacingss = Spacingss.astype(float)
Weightss = Weight.replace(np.NaN, 0.0)
Spacings = Spacingss.replace(np.NaN, 0.0)
Weightst = Weightss.multiply(other = 2.20462)
Spacingst = Spacings.multiply(other = 3.28084)
WW1 = Weightst[0] < 20000
Weights11 = Weightst[WW1]
Spacing11 = Spacingst[WW1]
WW2 = Weights11[1] < 20000
Weights22 = Weights11[WW2]
Spacing22 = Spacing11[WW2]
WW3 = Weights22[2] < 20000
Weights33 = Weights22[WW3]
Spacing33 = Spacing22[WW3]
WW4 = Weights33[3] < 20000
Weights44 = Weights33[WW4]
Spacing44 = Spacing33[WW4]
WW5 = Weights44[4] < 20000
```

```
Weights55 = Weights44[WW5]
Spacing55 = Spacing44[WW5]
WW6 = Weights55[5] < 20000
Weights66 = Weights55[WW6]
Spacing66 = Spacing55[WW6]
WW7 = Weights66[6] < 20000
Weights77 = Weights66[WW7]
Spacing77 = Spacing66[WW7]
WW8 = Weights77[7] < 20000
Weights = Weights77[WW8]
Spacing = Spacing77[WW8]
Totalwe = Weights.sum(axis=1)
NewW = Totalwe < 80000
Weights = Weights[NewW]
Spacing = Spacing[NewW]
span = 180
Moment = []
MCount = []
MLength = []
count = span/2
for length in range(0, span+1):
    if length <= (span/2):
        B1 = 1-(length/span)
        B2 = length/span
        Y1 = B1 * length
        Y2 = B2*(count - length)
        M = Y1-Y2
        Moment.append(M)
        MCount.append(count)
        MLength.append(length)
        print(count, length, M)
    elif length > (span/2):
        B1 = 1-(length/span)
        B2 = length/span
        Y1 = B2 * (span-length)
        Y2 = B1*(length - count)
        M = Y1-Y2
        Moment.append(M)
        MCount.append(count)
        MLength.append(length)
        print(count, length, M)
MomMax = []
trr=len(Spacing.axes[0])
```

```
half = int(span/2)
for count in range(0, trr):
    print(count)
    Momo= []
    SM1 = []
    R = Weights.iloc[count]
    S = Spacing.iloc[count]
    Rtotal = R[0]+R[1]+R[2]+R[3]+R[4]+R[5]+R[6]+R[7]
    Stotal = S[0]+S[1]+S[2]+S[3]+S[4]+S[5]+S[6]
    Cent = ((R[1]*S[0]) + (R[2]*(S[0]+S[1]))+ (R[3]*(S[0]+S[1]+S
[2]))+ (R[4]*(S[0]+S[1]+S[2]+S[3]))+ (R[5]*(S[0]+S[1]+S[2]+S[3]+
S[4]))+ (R[6]*(S[0]+S[1]+S[2]+S[3]+S[4]+S[5]))+ (R[7]*(S[0]+S[1]
+S[2]+S[3]+S[4]+S[5]+S[6])))/Rtotal
    Momo =(((R[0])*(np.interp(((span/2) + Cent), MLength , Momen
t)) + (R[1])*(np.interp(((span/2) + Cent - S[0]), MLength , Mome
nt)) + (R[2])*(np.interp(((span/2) + Cent - S[0]- S[1]), MLength
 , Moment)) + (R[3])*(np.interp(((span/2) + Cent - S[0]- S[1]- S
[2]), MLength , Moment)) + (R[4])*(np.interp(((span/2) + Cent -
S[0]- S[1]- S[2]- S[3]), MLength , Moment)) + (R[5])*(np.interp(
((span/2) + Cent - S[0]- S[1]- S[2]- S[3]- S[4]), MLength , Mome
nt)) + (R[6])*(np.interp(((span/2) + Cent - S[0]- S[1]- S[2]- S[
3]- S[4]- S[5]), MLength , Moment)) + (R[7])*(np.interp(((span/2
) + Cent - S[0]- S[1]- S[2]- S[3]- S[4]- S[5]- S[6]), MLength ,
Moment))) / 1000)
    MomMax.append(Momo)
```

Appendix B: Python Code – Non-Formula B Trucks Shear Analysis Example

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
WIMMM = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\W_WI
M Data\2019 Data\2019-000000830214.csv', low_memory=False)
WIMN = WIMMM['Lane Name'].str.contains('EB')
WIMs = WIMMM[WIMN]
WIM7 = WIMs['Class'] > 7
WIM8 = WIMs[WIM7]
WIM13 = WIM8['Class'] < 14
WIMM = WIM8[WIM13]
WIM6 = WIMM['Num Axles'] < 9
WIM = WIMM[WIM6]
Weight = WIM['Axle Weights (kg)'].str.split('|', expand = True)
Weight = Weight.astype(float)
Spacingss = WIM['Axle Spacings (m)'].str.split('|', expand = Tru
e)
Spacingss = Spacingss.astype(float)
Weightss = Weight.replace(np.NaN, 0.0)
Spacings = Spacingss.replace(np.NaN, 0.0)
Weightst = Weightss.multiply(other = 2.20462)
Spacingst = Spacings.multiply(other = 3.28084)
WW1 = Weightst[0] > 20000
WW2 = Weightst[1] > 20000
WW3 = Weightst[2] > 20000
WW4 = Weightst[3] > 20000
WW5 = Weightst[4] > 20000
WW6 = Weightst[5] > 20000
WW7 = Weightst[6] > 20000
WW8 = Weightst[7] > 20000
Totalwe = Weightst.sum(axis=1)
NewW = Totalwe > 80000
mask = [any(tup) for tup in zip(WW1, WW2, WW3, WW4, WW5, WW6, WW
7, WW8, NewW)]
Weights = Weightst[mask]
Spacing = Spacingst[mask]

For Spans 120’ to 180’
Span = 180
SheMax = []
tr=len(Spacing.axes[0])
```

```
for count in range(0, tr):
    print(count)
    R = Weights.iloc[count]
    S = Spacing.iloc[count]
    Rtotal = R[0]+R[1]+R[2]+R[3]+R[4]+R[5]+R[6]+R[7]
    Stotal = S[0]+S[1]+S[2]+S[3]+S[4]+S[5]+S[6]
    S1 = Rtotal-((R[7] + (R[6]*S[6]) + (R[5]*(S[6]+S[5])) + (R[4
]*(S[6]+S[5]+S[4]))+ (R[3]*(S[6]+S[5]+S[4]+S[3]))+ (R[2]*(S[6]+S
[5]+S[4]+S[3]+S[2]))+ (R[1]*(S[6]+S[5]+S[4]+S[3]+S[2]+S[1]))+ (R
[0]*(S[6]+S[5]+S[4]+S[3]+S[2]+S[1]+S[0]))))/Span)
    S1k = S1/1000
    SheMax.append(S1k)

For Spans 30’ to 90’
Span = 90
SheMax = []
tr=len(Spacing.axes[0])
for count in range(0, tr):
    print(count)
    R = Weights.iloc[count]
    S = Spacing.iloc[count]
    Rtotal = R[0]+R[1]+R[2]+R[3]+R[4]+R[5]+R[6]+R[7]
    Stotal = S[0]+S[1]+S[2]+S[3]+S[4]+S[5]+S[6]
    if Stotal < Span:
        S1 = Rtotal-((R[7] + (R[6]*S[6]) + (R[5]*(S[6]+S[5])) +
(R[4]*(S[6]+S[5]+S[4]))+ (R[3]*(S[6]+S[5]+S[4]+S[3]))+ (R[2]*(S[
6]+S[5]+S[4]+S[3]+S[2]))+ (R[1]*(S[6]+S[5]+S[4]+S[3]+S[2]+S[1]))
+ (R[0]*(S[6]+S[5]+S[4]+S[3]+S[2]+S[1]+S[0]))))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]) < Span:
        S1 = (Rtotal-R[0])-((R[7] + (R[6]*S[6]) + (R[5]*(S[6]+S[
5])) + (R[4]*(S[6]+S[5]+S[4]))+ (R[3]*(S[6]+S[5]+S[4]+S[3]))+ (R
[2]*(S[6]+S[5]+S[4]+S[3]+S[2]))+ (R[1]*(S[6]+S[5]+S[4]+S[3]+S[2]
+S[1]))))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]-S[1]) < Span:
        S1 = (Rtotal-R[0]-R[1])-((R[7] + (R[6]*S[6]) + (R[5]*(S[
6]+S[5])) + (R[4]*(S[6]+S[5]+S[4]))+ (R[3]*(S[6]+S[5]+S[4]+S[3])
)+ (R[2]*(S[6]+S[5]+S[4]+S[3]+S[2]))))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]-S[1]-S[2]) < Span:
        S1 = (Rtotal-R[0]-R[1]-R[2])-((R[7] + (R[6]*S[6]) + (R[5
]*(S[6]+S[5])) + (R[4]*(S[6]+S[5]+S[4]))+ (R[3]*(S[6]+S[5]+S[4]+
S[3]))))/Span)
```

```
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]-S[1]-S[2]-S[3]) < Span:
        S1 = (Rtotal-R[0]-R[1]-R[2]-R[3])-((R[7] + (R[6]*S[6]) +
 (R[5]*(S[6]+S[5])) + (R[4]*(S[6]+S[5]+S[4]))))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]-S[1]-S[2]-S[3]-S[4]) < Span:
        S1 = (Rtotal-R[0]-R[1]-R[2]-R[3]-R[4])-((R[7] + (R[6]*S[
6]) + (R[5]*(S[6]+S[5]))))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    elif (Stotal-S[0]-S[1]-S[2]-S[3]-S[4]-S[5]) < Span:
        S1 = (Rtotal-R[0]-R[1]-R[2]-R[3]-R[4]-R[5])-((R[7] + (R[
6]*S[6]))/Span)
        S1k = S1/1000
        SheMax.append(S1k)
    else:
        S1 = (Rtotal-R[0]-R[1]-R[2]-R[3]-R[4]-R[5]-R[6])-((R[7])
/Span)
        S1k = S1/1000
        SheMax.append(S1k)
```

Appendix C: Python Code – WIM vs NBI ADT and ADTT Calculation

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
WIM227 = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\W_W
IM Data\126.csv', low_memory=False)
WIMln = WIM227['Lane Name'].str.contains('EB')
WIMLN = WIM227[WIMln]
WIMlno = WIM227['Lane Name'].str.contains('WB')
WIMLNO = WIM227[WIMlno]
WIM8 = WIMLN['Class'] > 3
WIM8T = WIMLN[WIM8]
WIM88 = WIMLNO['Class'] > 3
WIM88T = WIMLNO[WIM88]
WIM15 = WIM8T['Class'] < 15
WIMT = WIM8T[WIM15]
WIM155 = WIM88T['Class'] < 15
WIMTO = WIM88T[WIM155]
d = {}
t = {}
z = {}
k = {}
ADTlist = []
ADTTlist = []
a = '2018-'
for count in range(10, 13):
    aa = a + str(count) + '-0'
    for count in range(1, 10):
        aaa = aa + str(count)
        d[str(aaa)] = WIMLN['Time'].str.contains(aaa, regex=Fals
e)
        t[str(aaa)] = WIMLN[d[str(aaa)]]['Node'].count()
        ADTlist.append(t[str(aaa)])
        z[str(aaa)] = WIMT['Time'].str.contains(aaa, regex=False
)
        k[str(aaa)] = WIMT[z[str(aaa)]]['Node'].count()
        ADTTlist.append(k[str(aaa)])
```

Etc. - X4 per monthly

```
ADTfinal = list(filter(lambda a: a != 0, ADTlist))
ADTTfinal = list(filter(lambda a: a != 0, ADTTlist))
ADTd = pd.DataFrame(ADTfinal)
```

```
ADTTd = pd.DataFrame(ADTTfinal)
NBI75 = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\N_NB
I\Python Ref\0368 NBI.csv')
ADT = NBI75['ADT_029']
TP1 = NBI75['PERCENT_ADT_TRUCK_109']
TP = TP1/100
ADTT = ADT*TP
from scipy import stats
stats.kruskal(ADTT, ADTTfinal)
```

```
KruskalResult(statistic=53.21025751609799, pvalue=2.996897746782
8024e-13)
```

```
stats.kruskal(ADT, ADTfinal)
```

Appendix D: Python Code – Following Probability

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
WIM1 = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\W_WIM
 Data\2019 Data\2019-000000830214.csv', low_memory=False)
WIM2 = WIM1['Lane Name'].str.contains('WB')
WIMNL = WIM1[WIM2]
WIM4 = WIMNL['Lane Name'].str.contains('Slow')
WIM = WIMNL[WIM4]
WIM7 = WIM['Class'] > 7
WIM8 = WIM[WIM7]
WIM13 = WIM8['Class'] < 14
WIM = WIM8[WIM13]
WIM6 = WIM['Num Axles'] < 7
WIM = WIM[WIM6]
Spacing = WIM['Axle Spacings (m)'].str.split('|', expand = True)
Spacing = Spacing.astype(float)
Spacing = Spacing.replace(np.NaN, 0.0)
Spacing = Spacing.multiply(other = 3.28084)
Spacing = Spacing.append(pd.Series([0.0, 0.0, 0.0, 0.0, 0.0], in
dex=Spacing.columns ), ignore_index=True)
Spacing = Spacing.append(pd.Series([0.0, 0.0, 0.0, 0.0, 0.0], in
dex=Spacing.columns ), ignore_index=True)
Speed = WIM['Speed (mph)']
Speed = Speed.multiply(other = 1.46667)
Time = WIM['Time']
trr=len(Spacing.axes[0])
g = []
xxx = 1000.0
for num in range(0, trr-2):
#      (Day, Clock) = Time.loc[num].split(' ')
    (hh, m, sx) = Time.iloc[num].split(':')
    (s, sxx) = sx.split('.')
    (Day, h) = hh.split()
    result = int(h)*3600 + int(m)*60 + int(s)
    g.append(result)
g.append(xxx)
d = []
trr=len(Spacing.axes[0])
```

```
for count in range(0, trr-2):
    S = Spacing.iloc[count]
    SS = Spacing.iloc[count+1]
    SSS = Spacing.iloc[count+2]
    Stotal = S[0]+S[1]+S[2]+S[3]+S[4]
    Change = g[count+1] - g[count]
    SP = Speed.iloc[count]
    Distance = (Change * SP) - Stotal
    if Distance < 100:
        d.append(count)
        dlen = len(d)
FP = (dlen/(trr-2))*100
print(FP)
```

Appendix E: Python Code – Side-by-Side Probability

```
import pandas as pd
import numpy as np
import matplotlib.pyplot as plt
WIM1 = pd.read_csv(r'Z:\RP 18-36 Bridge Asset (Alex Happy)\W_WIM
 Data\2019 Data\2019-000000830214.csv', low_memory=False)
WIM2 = WIM1['Lane Name'].str.contains('WB')
WIMNL = WIM1[WIM2]
WIM7 = WIMNL['Class'] > 7
WIM8 = WIMNL[WIM7]
WIM13 = WIM8['Class'] < 14
WIM = WIM8[WIM13]
WIM6 = WIM['Num Axles'] < 7
WIM = WIM[WIM6]
Spacing = WIM['Axle Spacings (m)'].str.split('|', expand = True)
Spacing = Spacing.astype(float)
Spacing = Spacing.replace(np.NaN, 0.0)
Spacing = Spacing.multiply(other = 3.28084)
Spacing = Spacing.append(pd.Series([0.0, 0.0, 0.0, 0.0, 0.0], in
dex=Spacing.columns ), ignore_index=True)
Spacing = Spacing.append(pd.Series([0.0, 0.0, 0.0, 0.0, 0.0], in
dex=Spacing.columns ), ignore_index=True)
Speed = WIM['Speed (mph)']
Speed = Speed.multiply(other = 1.46667)
Time = WIM['Time']
trr=len(Spacing.axes[0])
g = []
xxx = 1000.0
for num in range(0, trr-2):
#     (Day, Clock) = Time.loc[num].split(' ')
    (hh, m, sx) = Time.iloc[num].split(':')
    (s, sxx) = sx.split('.')
    (Day, h) = hh.split()
    result = int(h)*3600 + int(m)*60 + int(s)
    g.append(result)
g.append(xxx)
r = []
trr=len(Spacing.axes[0])
for count in range(0, trr-2):
```

```
    S = Spacing.iloc[count]
    SS = Spacing.iloc[count+1]
    SSS = Spacing.iloc[count+2]
    Stotal = S[0]+S[1]+S[2]+S[3]+S[4]
    Change = g[count+1] - g[count]
    SP = Speed.iloc[count]
    Distance = (Change * SP)
    if Distance < Stotal:
        r.append(count)
        rlen = len(r)
FP = (rlen/(trr-2))*100
print(FP)
```

Appendix F: HL-93 Moment Check

Mid Moment for HL-20 + 0.64 lane load

	Span	Shear L	Shear R	MidMoment		Python Moment Value			
	30	36.62222	46.57778	400		469	Does not match as the tandem		
	60	55.2	55.2	1357.333333		1357.33	controlled this span		
	90	64.8	64.8	2617.333333		2617.33			
	120	74.4	74.4	4165.333333		4165.33			
	150	84	84	6001.333333		6001.33			
	180	93.6	93.6	8125.333333		8125.33			
	Centroid	18.66667							

***Moment values match python code results**

Appendix G: HL-93 Shear Check

Shear for HL-20 + 0.64 lane load

	Span	Shear L	Shear R	Lane Load	Shear Max	Python Shear Value
	30	49.6	22.4	9.6	59.2	59.2
	60	60.8	11.2	19.2	80	80
	90	64.53333	7.466667	28.8	93.33333333	93.33
	120	66.4	5.6	38.4	104.8	104.8
	150	67.52	4.48	48	115.52	115.52
	180	68.26667	3.733333	57.6	125.8666667	125.87

***Shear values match python code results**

8. REFERENCES

Chi, J. (2019). WIM Based Live Load Factors for Consistent Illinois Bridge Reliability, Illinois Institute of Technology.

Dekelbab, W., et al. (2008). "History lessons from the national bridge inventory." **71**(6): 30.

Ellingwood, B. R., et al. (2009). Condition assessment of existing bridge structures, Georgia Institute of Technology.

Fanous, D. F. (2020). "Inluence Lines."

Gandomi, A. and M. Haider (2015). "Beyond the hype: Big data concepts, methods, and analytics." International Journal of Information Management **35**(2): 137-144.

Huang, R.-Y., et al. (2010). "Exploring the Deterioration Factors of RC Bridge Decks: A Rough Set Approach." Computer-Aided Civil and Infrastructure Engineering **25**(7): 517-529.

Kwon, O.-S., et al. (2010). "Calibration of live-load factor in LRFD bridge design specifications based on state-specific traffic environments." **16**(6): 812-819.

Lou, P., et al. (2016). "Effect of Overweight Trucks on Bridge Deck Deterioration Based on Weigh-in-Motion Data." Transportation Research Record: Journal of the Transportation Research Board **2592**(1): 86-97.

Mahmoudabadi, A. and S. M. Seyedhosseini (2013). "Improving the efficiency of weigh in motion systems through optimized allocating truck checking oriented procedure." IATSS Research **36**(2): 123-128.

McCall, B. The U.S. States Successful Practices Weigh-In-Motion Handbook, CEC.

Minitab "Interpret the key results for Kruskal-Wallis Test."

Minitab "Methods and formulas for Kruskal-Wallis Test."

Nowak, A. S. and P. J. K. J. o. C. E. Rakoczy (2013). "WIM-based live load for bridges." **17**(3): 568-574.

Planning, G. O. o. (2015). "Georgia Statewide Freight and Logistics Plan: Truck Model Profile."

Stam, A., et al. (2006). "An Exploratory Analysis of NBI Data."

Statistics, L. "Kruskal-Wallis H Test using SPSS Statistics."

Wiegand, K. (2018). "Georgia's Traffic Monitoring Guide."

Yu, Y., et al. (2016). "State-of-the-art review on bridge weigh-in-motion technology." **19**(9): 1514-1530.