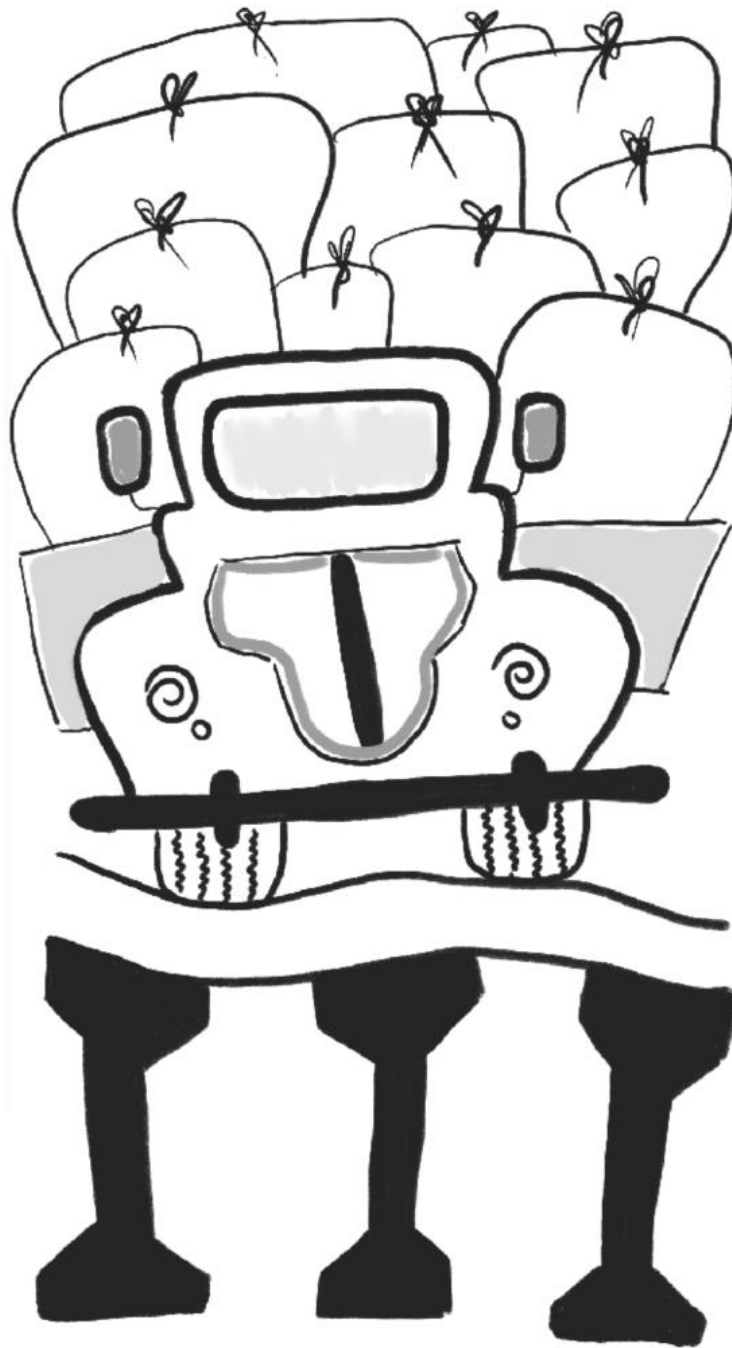


Impact of Overloaded Vehicles on Bridges



Impact of Overloaded Vehicles on Bridges

by

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Abstract

Excessive weight and a growing number of overloaded vehicles can cause an accelerated consumption of bridge service life. Therefore, there is a need to control the operation of heavy traffic. This study considered the following issues: a growing number of issued permit vehicles, outdated permit fee regulations, lack of uniform provisions throughout the United States regarding notional permit vehicles, and no provisions for bridge rating under legally overloaded grandfather vehicles.

This dissertation is divided into three parts (papers), with a common topic being the impact of overloaded vehicles on bridge structures. It considers issues with bridge design and evaluation procedures under heavy traffic, and assessment of bridge damage under overloaded permit vehicles.

The objective of the first paper is to assess bridge life consumption by the permit vehicles. The economic lifetime of a bridge is considered in terms of the number of load cycles to failure so that the structure has to be replaced. Each truck's passage causes one or more load cycles and reduces or consumes a part of the remaining fatigue life of the bridge. The consumption by a permit vehicle can be considered as a superposition of two parts. The first one is caused by the vehicle loaded up to the legal limit, which would not require any permit, and the other part is the additional consumption caused by the weight exceeding the legal limit. Thus, a formula is derived for the incremental consumption to quantify the damage caused by the permit overload. Furthermore, the fatigue damage is converted to dollar damage based on the bridge construction cost, traffic volume, and state-specific bridge parameters. This approach allows state agencies to establish a rational and fair permit fee structure. The methodology is transparent and can be applied by other states using the available state-specific database.

The objective of the second paper is to propose a reliability-based methodology to determine a notional permit vehicle(s) for bridge design and evaluation analysis. Notional vehicle is developed to represent the existing traffic and assure the minimum required safety along with specific live load factors. Over 300,000 notional trucks with different axle weight and spacing combinations were generated to develop a candidate for the notional permit vehicle. The analysis led to the selection of a notional permit vehicle with eight axles and gross vehicle weight (GVW) of 94.8 tons (209 kips) and a length of 33.83 m (111 ft). The selected notional permit vehicle can serve as a basis for verification of the design provisions for the Strength Limit State II and rating under permit vehicles. It can also simplify the permit issuance procedure and help to control permit vehicle operations.

The third paper aims to determine if the current live load factors provide adequate safety for exempted legally overloaded vehicles under grandfather provisions. Presently, there are no guidelines to evaluate bridges for grandfather vehicles. A reliability-based analysis procedure is developed to determine the adequacy of the state-specific live load factor for bridge rating under exempted vehicles. The procedure is demonstrated in the example of grandfather agricultural vehicles in Montana. The developed approach does not require any re-analysis of the state bridge inventory. Linear scaling of existing ratings from bridge records can be used to determine ratings for legally overloaded agricultural vehicles. The study provides a calibration method to maintain the required safety level with the minimum cost and effort.

The research presented in this dissertation advances the state of the art and practice in understanding and assessment of the impact of overloaded vehicles on bridge structures. The proposed approach helps to improve and enhance control of overloaded vehicles and assure safety of bridges.

Abstract

Presented studies focus on heavy traffic and its impact on bridges. This issue may raise the following questions: Are bridges adequately designed for overloaded vehicles? Does the bridge code respond to an increasing number of heavy vehicles? What is the safety level for heavy vehicular traffic? Do bridge design and evaluation provisions account for permit traffic and grandfather exemptions? What is the damage caused by overloaded permit vehicles? Is the permit fee schedule adequate and consistent with bridge consumption? What is the monetary consumption of bridges?

Paper 1

Assess monetary service life consumption by overloaded permitted vehicles on bridges. Develop a method to measure a dollar consumption that provides a rational basis to propose a new permit fee structure.

Paper 2

Propose a reliability-based method to determine a notional permit vehicle(s) based on issued permits and weigh-in-motion (WIM) databases. Check if the selected notional permit vehicle can serve as a basis for verifying the design provisions for the Strength Limit State II and rating of permit vehicles. Verify if it can simplify the permit issuance procedure and help to control permit vehicle operations.

Paper 3

Determine if the current live load factors provide adequate safety for exempted vehicles under grandfather provisions. Develop a reliability-based analysis to determine state-specific live load factor for bridge rating under legally overloaded grandfather vehicles.

Conclusions

- Incremental consumption equation was developed to assess fatigue damage caused by overloaded permit vehicles. It is an efficient tool to assess dollar damage consumption.
- Incremental bridge consumption approach gives a rational basis to develop fair and justifiable permit fees.
- Reliability-based method was developed to find a representative notional permit vehicle, which can be used in bridge design for Strength Limit State II, bridge evaluation under permit loading, and it can help with the issuance of permits.
- Development of the notional permit vehicle offers a very beneficial and cost-effective process for the state DOTs as it helps to reduce individual permit analyses when issuing a permit.
- Bridge evaluation procedure under legally overloaded grandfather vehicles was assessed to help bridge owners account for excessive load effects and implement it in a new rating policy. Linear scaling of an existing rating was proposed to minimize a cost and effort in making necessary posting decisions.

Keywords: overload, heavy traffic, permit, live load, reliability, bridge design, bridge evaluation, damage, permit fee, consumption, reliability index, safety, LRFD, LRFR.

Dedication

To my parents, I love you.

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CHAPTER 1 INTRODUCTION

1.1.MOTIVATION

Many bridges in the United States are approaching the end of their design life, resulting in an increasing number of bridges requiring major repair, rehabilitation, or replacement. Based on the National Bridge Inventory database (2020), approximately 7.2% of bridges in the USA are in poor, serious, or critical condition. Excessive number and load effects caused by overloaded vehicles have not been explicitly quantified. Numerous bridges are not designed and evaluated for extreme loading. It leads to a need to verify the American Association of State Highway and Transportation Officials (AASHTO) bridge design and evaluation provisions for overloaded vehicles.

A growing number of overloaded vehicles can cause an accelerated consumption of bridges. Therefore, it is required to monitor operation of heavy traffic. The traffic loads include three groups of vehicles: (1) legal vehicles which do not exceed state and federal provisions; (2) illegal vehicles exceeding the weight, size, or weight and size limits operating without permits; and (3) permitted vehicles, which can legally exceed the legal limits after purchasing the permit. The number of issued permits is growing every year, and the impact of overloaded vehicles has not been measured.

In most states, the load regulations pertaining to permit vehicles are outdated, and they have not been changed for decades. The permit fee schedule in the U.S. varies significantly from state to state. The permit fees and permit criteria are very different. The permit fees can be based on axle weight, gross vehicle weight, distance, weight and distance, and flat fee. State agencies seek to establish a rational and fair permit fee structure based on damage assessment analysis. The permit fee schedules are developed to control the heavy

traffic and maintain the good condition of bridges and roads, recover damage costs, and provide a possibility for efficient and economically justified transport. Therefore, it is necessary to provide a method to assess the monetary consumption of overloaded vehicles on bridges.

Furthermore, a growing number of overloaded vehicles need to be quantified in terms of load effects on bridges to verify the AASHTO design and evaluation provisions. The design provisions for overloaded vehicles are not uniform throughout the United States. Every state uses its own notional permit vehicles to evaluate the live load effect. Bridge specifications are based on load and resistance factor design, which is a reliability-based method that assesses the safety level determined for each considered limit state function. The limit state function is defined as a function of load and resistance parameters, and it describes the borderline between acceptable and unacceptable performance. The required safety level in the form of the target reliability index is assured by the load and resistance factors. At present, the notional permit vehicles were not determined using a reliability-based approach. There is a need for a standardized method to help state DOTs in the development of notional permit vehicles representing the traffic-induced load spectra and provide a minimum safety level.

Quantified load effects and notional representative vehicles can then be used in code calibration, for design (Strength Limit State II), and evaluation (operating rating under permit loading). The provisions for overloaded vehicles need to be verified and re-evaluated to make sure new and existing bridges are safe for traffic. Also, the impact of permit vehicles and damage should be quantified to help in the development of an equitable and justifiable permit fee structure.

1.2.BACKGROUND

1.2.1. Bridges condition in the U.S.

Infrastructure Report Cards (ASCE Cards 2021) rated the US bridge structures at mediocre condition, which requires attention. It confirms that bridge infrastructure is in fair to good condition, but it shows signs of deterioration and needs attention. The American Society of Civil Engineers (ASCE) assessment revealed bridge deficiencies in conditions and functionality. There is over 600,000 bridges in the U.S., and many are approaching the end of their design life, resulting in an increasing number of bridges requiring maintenance activities. Currently, AASHTO Load and Resistance Factor Design (LRFD) assumes the service life of the bridge as 75 years. Based on the National Bridge Inventory database (2020), approximately 13% of bridges are over 75 years. **Figure 1.1** presents a map with bridges of age over 75 years and their percentage in every state. The percentage of old bridges varies from 3-30 %, depending on location. An increasing number of old bridges requires more attention. In the past, bridges were designed for a service life of 50 years. Now, 42% of all US bridges are 50 years old or more, which yields an average bridge age of 44 years.

To protect the public's safety, the federal government requires regular bridge inspections. The bridges are evaluated every 12 to 48 months based on the bridge condition, type of bridge, traffic, location, and age of the structure. In case of bridge deficiency in the structural capacity or with an element of the bridge, the inspector can post the bridge for load, weight, speed restrictions, order repair, or close the bridge to ensure safety. Besides bridge safety concerns, posted bridges can dramatically increase driving time for larger vehicles such as ambulances, fire trucks, school buses, and delivery trucks.

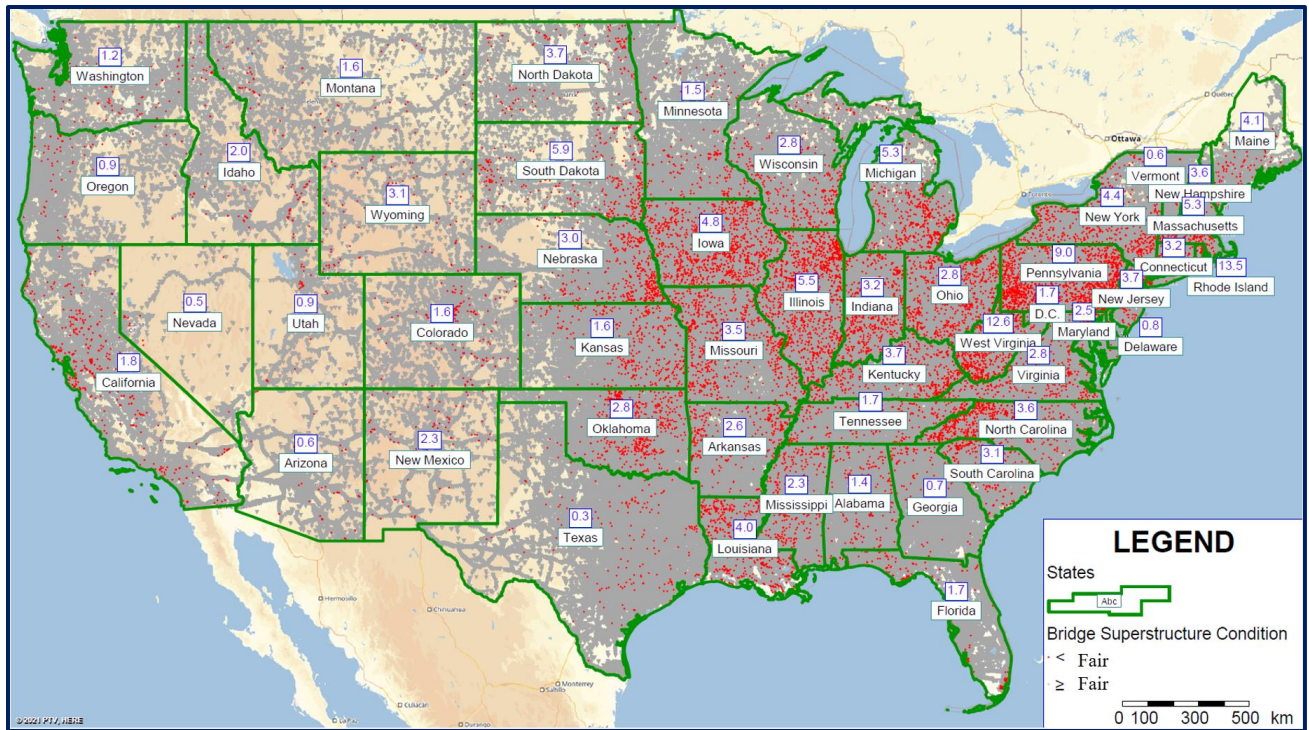


Figure 1.2. Bridge superstructure condition in the U.S.

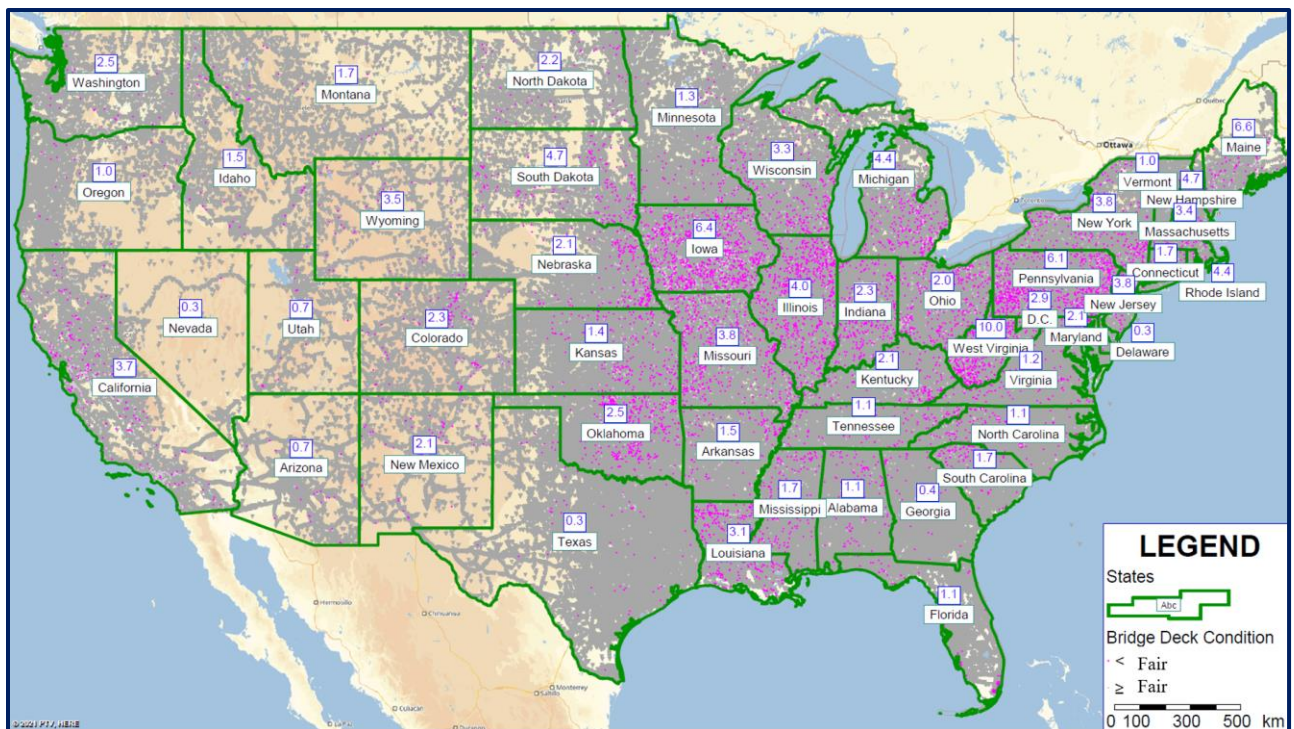


Figure 1.3. Bridge deck condition in the U.S.

It was noticed that number of bridges transitioning from good to fair condition is increasing annually. However, bridges categorized as fair are potentially one inspection from being downgraded in classification. Still, they can also be preserved at a fraction of the cost required to address a structurally deficient bridge. ASCE Cards reported that it would take 50 years to make all of the necessary repairs for the present rate of investment. Therefore, the federal government requires a systematic program for bridge preservation, prioritizing existing deterioration, but also focus on preventive maintenance. Bridges are costly and complex assets, which makes it difficult to maintain them in good condition.

It is crucial to consider a life-cycle cost analysis, which can minimize long-term costs over the infrastructure's lifetime. It is essential to identify problems earlier, increase the lifespan of the nation's bridges, stretch limited resources, and prioritize public safety.

Additionally, bridges are being subjected to trucks that are heavier than those they were initially designed to withstand. Heavy truck traffic may cause overstress of bridge elements, causing fatigue and cracking, consequently decreasing bridges' service life. Excessive number and load effects caused by overloaded vehicles have not been explicitly quantified. As a result, numerous bridges are not designed and evaluated for excessive loading. It leads to a need to verify the American Association of State Highway and Transportation Officials (AASHTO) bridge design and evaluation provisions for overloaded vehicles.

1.2.2. Traffic composition and legal limits

Traffic flow consists of various vehicle types and configurations, including a number of axles, their spacings, axle weights, and consequently various impact on the road and bridge infrastructure.

Vehicles can be grouped as:

- 1) legal vehicles that do not exceed general regulations (federal truck weight and size law and state laws) for axle spacings and weights and do not require a permit. Special vehicles have exemptions under grandfather provisions;
- 2) grandfather vehicles are legally overloaded vehicles under grandfather provisions, which are old rules that remain unchanged after a new rule was introduced. Based on that, some vehicles can operate above the federal truck weight and size law;
- 3) permit vehicles, which can legally exceed the legal limits after purchasing the permit;
- 4) illegal vehicles, which do not meet the regulations exceeding the weight, size, or weight and size limits.

Truck weight and size limits are enacted to ensure the safety of roads and bridges. The impact of heavy traffic needs to be monitored to control and wear and tear caused by heavy traffic. Federal truck weight and size law prevent states from imposing vehicle weight limits on interstate highways that deviate from established federal weight limits in the US. The traffic on an interstate highway is subjected to the standard federal weight limits ("23 U.S. Code § 127 - Vehicle weight limitations—Interstate System," 1974). There are also state-specific exceptions from standard law, called grandfather provisions.

Grandfather provisions allow exceptions to the federal limits on vehicle weight and size. These provisions are exempt from previously existing rules. The first, enacted in 1956, deals primarily with gross vehicle weights, axle weights, and permitting practices. In 1975, federal law employed a bridge formula that limits the axle configuration and axle load distribution. Many states adopted their interpretations of weight laws under grandfather provisions, depending on local traffic conditions. The most common exemptions

include vehicles carrying agricultural and farm products and commodities. Federal Highway Administration (FHWA) reports that 41 states provide exemptions for “agricultural vehicles” (FHWA, Freight Management, and Operations, 2019).

Permit vehicles are overloaded vehicles that can operate legally prior to purchasing the permit. Overloaded permit vehicles can be oversized, overweight, or both. Permit vehicles need to follow the limitations specified in their permit, which may restrict the gross, single axle, and group axle weights. In the US, every state has its own policies on issuing permits but must follow federal rules. Permits allow vehicles of specific configurations and sizes to exceed the standard vehicle size and weight limitations. Permits can be issued for single or multiple trips. The permit may have limitations on designated routes, the number of trips, times of operation, and the necessity, or not, for escort vehicles. The movement of permit oversized or overweight vehicles must also comply with the requirements and safety considerations specified by state law.

Illegally overloaded vehicles, with or without permits, belong to an unanalyzed portion of bridge traffic load that is more likely to create an extreme loading case. The traffic flow, with presented types of vehicles, is shown in **Figure 1.4**.

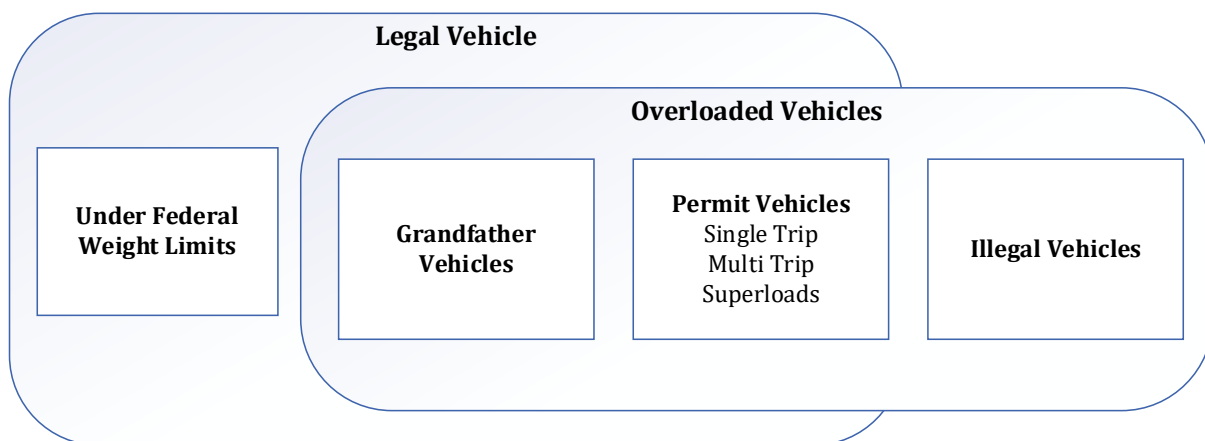


Figure 1.4. Traffic flow – types of vehicles.

Federal truck weight and size law consist of four conditions, which state that the vehicles are legal if its maximum Gross Vehicle Weight does not exceed 36.3 tones (80 kips), maximum single axle weight is 9.1 tones (20 kips), and maximum tandem axle weight of 15.4 tones (34 kips), and the vehicle does not exceed Federal Bridge Formula (FBF), shown in **Equation (1.1)**. US Federal Bridge Formula is used to check the axle configuration and axle load distribution. The formula limits the weight of any set of consecutive axles to, and can be expressed as:

$$W = 500 \left[\frac{(L \cdot N)}{N - 1} + 12N + 36 \right] \quad (1.1)$$

where:

W – the overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds [lbs.],

L – the distance in feet between the outer axles of any group of two or more consecutive axles [ft],

N – the number of axles in the group under consideration.

In general, bridge design and evaluation codes specify a notional load model for normal traffic, which is deemed to represent the extremes of legal vehicle loading. Vehicles seeking permits are compared to abnormal vehicles that the bridge has been found to have the capacity to carry. The overloaded vehicles are considered in AASHTO for Strength Limit State II, and AASHTO Load and Resistance Factor Rating (LRFR) for operating rating under permit vehicles. Illegally overloaded vehicles without permits belong to an unanalyzed portion of the bridge live load that is more likely to create an extreme lifetime stress

condition. The impact of traffic and live load model development is presented in the next sections.

1.2.3. Traffic data

The monitoring of traffic-induced load effects is essential to maintain the safety of infrastructure. Therefore, the existing traffic volume and load effects must be continuously monitored and evaluated. Traffic data can be utilized in the design of new bridge structures and evaluation of existing ones, live load modeling, reliability assessment, etc. Nowadays, there are several techniques to measure traffic-induced loading, including static and in-motion measurements.

The static methods are local, selective and measure only a small fraction of the highway network traffic. The static measurements can be performed by portable scales and Weigh Stations. The weigh stations are located off-road and typically have static scales built into the pavement. In these systems, an operator checks if the legal weight limits have been violated. Furthermore, weigh station locations are known by truck drivers, and illegally overloaded vehicles may try to avoid them, resulting in biased truck weight data.

Weigh-in-Motion (WIM) measurements enable continuous recording of vehicles passing a measurement site. The WIM systems can collect traffic volume, vehicle configurations, and load spectra. It is a powerful tool to collect a massive traffic database. Data is recorded for every vehicle, including a detailed description of vehicle configuration, vehicle class, measurement date and time, occupied lane, trip direction, moving speed, and truck axle weights and spacings.

To accurately assess traffic-induced load effects, it is required to verify the data quality. There are uncertainties involved in the measurement process that must be

considered while dealing with big data. Several factors can affect the accuracy of the weigh-in-motion measurements, such as pavement roughness (causing bouncing axle movement or dynamic impact), temperature effects, multiple presences, incorrect vehicle position, etc. Assessment of the live load effect plays a key role in designing and evaluating roads and bridges to maintain the infrastructure's safety. Hence, it is important to adequately assess the load effects not to underestimate or overestimate the load effects. The underestimation of live load effect can cause premature damage to bridges and roads, and overestimation can cause a significant cost increase.

There are many studies considering quality control checks of traffic (Anitori et al. 2017; Elkins and Higgins 2008; Fiorillo and Ghosn 2014; Kulicki et al. 2015; Liao et al. 2015; Nichols and Bullock 2004; Quinley 2010b; Ramachandran et al. 2011a; Ramesh Babu et al. 2019; Sivakumar et al. 2008). However, there is no uniform Quality Control procedure. All states gather traffic data as part of FHWA's Highway Policy Management System, and the quality has to meet the minimum requirements prescribed in the guides (Quinley 2010). The quality and quantity of traffic data for bridge design and evaluation purposes have been examined extensively (Ghosn et al. 2011b; Sivakumar et al. 2008).

Many researchers utilized WIM data to account for the site-specific nature of traffic and develop more efficient bridge designs (Ali et al. 2020; Anitori et al. 2017; Leahy et al. 2015) to evaluate existing bridges (Ghosn et al. 2011a; Hayworth et al. 2008; Lawson et al. 2013; Minervino et al. 2004; Nowak and Tharmabala 1988; Sivakumar et al. 2007, 2008; Zhao and Tabatabai 2012) and for fatigue studies (Chotickai and Bowman 2005; Iatsko et al. 2020; Laman and Nowak 1996; Ramesh Babu 2019; Treacy and Brühwiler 2012).

The changes in truck traffic volume, axle load and configuration in recent decades are reviewed by (Anitori et al. 2017; Ghosn et al. 2011b; Liao et al. 2015; Ramesh Babu et al. 2019; Treacy and Brühwiler 2012).

The application of big traffic data allows controlling traffic volume, evaluate trends and fluctuation, and assessing load effects. It can be used in reliability-based calibration procedures to develop a live load model with specific statistical parameters and to calculate optimal live load factors for bridge design and evaluation procedures to achieve the desired safety level under existing traffic.

1.2.4. Impact of overloaded trucks

The impact of overloaded vehicles on bridges is essential to assess the damage and behavior of bridges under excessive loading. To protect the public's safety and maintain the bridge condition, regular bridge inspections, traffic analysis, and improvement of the bridge code provisions regarding the everchanging live load effect are crucial. This section presents significant studies on the impact of overloaded vehicles are presented in chronological order.

Behavior of Steel Bridges under Superload Permit Vehicles

(Culmo et al. 2004)

Culmo et al. investigate different types of overload vehicles to verify if simplified methods of analysis of the superloads are appropriate. The longitudinal and lateral load distribution over different bridge spans for various truck trailers was checked. The study was based on the one passage of a large permit vehicle used to transport the power plant equipment across a specific bridge in Connecticut. The strain gauges were used to check the stresses and dynamic impact on the bridge girders. Dynamic allowance for fatigue design is

1.15 times the static load effects and 1.33 for normal design. It was proved that if vehicles crawl across the bridge, dynamic load allowance can essentially be reduced to 1.00. The measured stresses from superload vehicles were significantly lower than calculated by AASHTO Guide Specification for Distribution of Loads for Highway Bridges. The conservative consumptions in the AASHTO guide are appropriate for the superloads. This is a very important and practical conclusion that there is no need to change the method for superload analysis.

Fatigue of older bridges in northern Indiana due to overweight and oversize loads

(Reisert and Bowman 2005)

This study was conducted to evaluate the fatigue behavior of steel bridges on the heavy-duty corridor between Indiana and Michigan. This research is presented in two volumes. The first one is to determine heavy load spectra in the considered corridor and check the bridge response on that traffic. The second volume aimed to estimate the remaining fatigue life of steel bridges along the heavy-weight corridor. The major contributions for both volumes are presented.

Volume 1: Bridge and Weigh-in-Motion Measurements

(Reisert and Bowman 2005)

Reisert and Bowman assessed the magnitude of the traffic-induced load on the heavy-duty corridor to determine the effects on the fatigue strength of the steel bridge structures. The representative bridge structures were selected within the extra heavy-duty highway. The Weigh-in-Motion system situated in proximity to bridges were used to evaluate the truck loads. Also, strain gauges were installed to assess the response of the bridge under the traffic loading. The structural analysis models (2D and 3D) of the bridge

were presented. The predictive analysis was conducted, structural models' behavior on known truck loading was compared to the strain measurements. It was found that a three-dimensional finite element model was accurately predicting the bridge response. In the analysis, class 9 and class 13 vehicles were chosen as representative heavy trucks. The legal limit for class 9 trucks is 90,000 lb and 134,000 for class 13. The results indicate that 15% of the Class 9 trucks and 26% of the Class 13 trucks travel heavier than their respective legal limits. Extreme weights of more than 200,000 lbs. were observed. The analytical model on the heavy-weight corridor does not appear to be a fatigue issue. However, a second stage of the study is to develop a more thorough analytical model.

Volume 2: Analysis Methods and Fatigue Evaluation

(Chotickai and Bowman 2005)

Chotickai and Bowman analyzed the fatigue strength of steel bridges on extra heavy corridors. The fatigue truck model was developed for the 3-axle and 4-axle vehicles that accurately represent the existing traffic for 2-5 axle trucks and 8-11 axle vehicles. The study introduced a fatigue reliability model. The developed procedure allows determining the remaining fatigue life of in-service steel bridge structures. The reliability model may provide a more accurate estimate of fatigue life evaluating uncertainties involved in the fatigue damage assessment method. The reliability model was evaluated through field measurements of two bridges using the strain gauge measurements and the development of 2D and 3D analytical models. The fatigue evaluation of the remaining fatigue life for selected bridges was obtained on the heavy-weight corridor. The remaining fatigue life for critical details was estimated as 25 years.

***Effects of Hauling Timber, Lignite Coal, and Coke Fuel
on Louisiana Highways and Bridges***

(F. L. Roberts et al. 2005)

Roberts et al. performed a study to assess the economic impact of overweight permitted vehicle hauling timber, lignite coal, and coke fuel on pavements and bridges. The highway roads and bridges utilized by the hauls were identified. About 2,800 bridges are considered in the study. Three different weight scenarios were selected for the research: 80,000 lbs (legal limit) 86,600 lbs or 88,00 lbs (permit practice), 100,000 lbs (proposed permit limit). Also, additional analysis for a tandem load of 48,000 lbs was considered since it was the maximum permissible tandem axle load in Louisiana. The bridge analysis was considered for simple and two equal span continuous bridges, span length varied from 20 ft to 120 ft. Moments, shear load effects and the deflection were computed for H15, HS-2044, and 3S2 loading. The analytical model for the bridge deck was developed, and the stresses caused by the considered truck were checked. The cost to repair fatigue damage for each vehicle passage was calculated.

The study shows that permit fees paid by timber trucks should increase from \$10 per year to around \$346/year/truck for a GVW of 86,600 lb. If 48-kip tandem is allowed, the permit price should be increased to \$4,377/year/truck. The current permit fee for lignite coal should remain at current levels. The DOT should not raise the GVW level to 100,000 lb; such a change from 86,600 lb would double the cost of pavement overlays. In many cases, the bridge costs per passage of a loaded truck amount to \$8.90, meaning that the cost of bridge damage per truck per year can easily exceed \$3,560.

Legal Truck Loads and AASHTO Legal Loads for Posting

(Sivakumar et al. 2007)

Sivakumar et al. developed notional vehicles representing legal trucks for AASHTO LRFR bridge evaluation. By federal law, legal vehicles are those that do not violate the requirement of maximum single axle load (20,000 lb), tandem axle load (34,000 lb), and Gross Vehicles Weight (80,000 lb). Also, vehicles must comply with Federal Bridge Formula (FBF). Specialized hauling vehicles meet FBF requirements but still cause excessive load effects on bridges. The study presents the developments of notional vehicles that can better represent legal loads for bridge evaluation. The notional vehicle development was based on Weigh-in-Motion data and state DOT survey to capture unusual vehicle configurations that meet legal requirements but caused overstress in bridges. Single-unit multi-axle trucks were identified as legal load candidates for bridge posting. Then calibration of live load factors for the vehicle candidates was conducted to provide uniform safety. Notional vehicles that imitate specialized hauling vehicles, which cause a bridge to overstress were developed with live load factors for bridge evaluation procedures.

From that study, it is seen that states do not have a guide for legal vehicles that do not meet Federal Formula limits, such as grandfather vehicles. State DOT's can develop their own notional vehicles that account for grandfather provisions. That method can be bothersome for DOT Engineers since it adds a new rating vehicle that might not be included in a rating software such as AASHTOWare Bridge (<https://www.aashtoware.org/>). There are many variations from federal weight law among the states; that is why evaluation of state-specific grandfather legal loads is important to maintain the safety of bridges along with the simple implementation of LRFR procedures for bridge inspectors.

State-Specific LRFR Live Load Factors Using Weigh-in-Motion Data

(Pelphrey et al. 2008)

The first site-specific live load factors developed for Load and Resistance Factor Rating (LRFR) were conducted in Oregon. The paper summarizes the development of site-specific load factors for bridge evaluation. The factors were calibrated using the same statistical methods that were used in the original development of LRFR. This procedure provides a uniform structural reliability index for bridge evaluation. A large WIM database from four WIM sites was used in the calibration. The WIM database utilized in that study was much larger than in the original LRFD or LRFR calibration. The considered WIM sites were different in truck volumes, and also, the seasonal and directional variations were noticed. The data collection period varied from site to site. The specific live load factors for legal load Oregon were smaller than those in the LRFR specification. In all cases, the largest computed live load factor from each data set was used to describe the WIM site. Developed site-specific load factors were rounded to the nearest 0.05 and implemented in state rating policy in Oregon.

Protocols for Collecting and Using Traffic Data in Bridge Design

(Sivakumar et al. 2008)

This study provides a methodology to calculate site-specific live load factors using the Weigh-in-Motion database. The traffic volume, vehicle weight, and configuration have changed since 1975 when the vehicle survey was conducted to represent the live load on a bridge in the US. The goal of this project was to develop a set of protocols and methodologies to enable development of site-specific live load factors. The available truck traffic data collected at different WIM sites around the US can be utilized, as it includes

detailed information about vehicle weight and configuration. The study provides a step-by-step procedure that can be followed to obtain site live load models for LRFD bridge design. The methodology includes requirements for WIM data collection, quality, and quantity. The traffic data need to be verified to eliminate questionable records. Quality Control procedures are introduced to eliminate bias records. WIM data validation and system calibration must be checked to recognize eventual issues. The multiple presence of vehicles is presented, and actual percentage of side-by-side multiple truck event must be verified. The study shows the multiple presence is significantly lower than assumed in the AASHTO LRFD code. Live load factor calibration begins with a calculation of the maximum expected load effect on bridge superstructure. The live load model is based on extreme value theory, where 5% of load effects from Cumulative Distribution Function (CDF) are considered to determine the live load effect. The protocols provide a recommendation on the evaluation of bridge superstructure, bridge deck, and fatigue assessment. It is a complex procedure that can be utilized to update the live load factors for existing traffic load based on Weigh-in-Motion data. The protocols are developed for bridge design, but AASHTO Guide Manual provides the same procedure in a commentary to determine load factor for bridge rating.

Bridge analysis and evaluation of effects under overload vehicles, Phase 1

(Bae and Oliva 2009)

Han-Ug Bae and Michael Oliva developed a user guide to calculating the impact of overloaded vehicles for single and dual trailer configuration. A procedure was introduced to assess the impact of vehicles on multi-girder bridges. The number of possible special vehicle configurations of vehicles is significant, and it is problematic for state DOTs to assess the damage for every individual vehicle. The overloaded vehicles need to be properly evaluated

to keep bridge structures safe. Special vehicle permits are evaluated individually for the vehicles and the route. It is very time-consuming and vulnerable to errors. Simplified multi-girder bridge analysis was introduced. Girder distribution factor for moment and shear for a single and dual trailer is provided. Limitations of using this method are provided. A minimum number of girders, bridge span, girder spacing, and deck thickness ranges are given. This method does not include multi-presence and dynamic allowance. The deck investigation is also included that considers punching and flexural failure. Punching shear is determined by the correlation between the single wheel and two-wheel set space 6 ft apart, with interpolated factors for minimum longitudinal and transverse wheel spacing for overload vehicles. The example calculations for simplified methods are presented in that study.

Bridge analysis and evaluation of effects under overload vehicles, Phase 2

(Ug-Bae and Oliva 2012)

Ug-Bae, and Oliva analyzed the effect of overload vehicles on bridges, specifically on concrete decks and steel girders. The study examined long-term effects on bridge behavior. Bridge life cycle cost was checked along with the development of mean costs assign per overload of vehicle. The analysis considers the effect of design truck HL-93 and overload vehicle per single and dual lanes for the most severe permit case noticed for ten years of permit data in WisDOT. The effects of selected trucks were checked for two types of decks and steel girders. Then bridge life cycle considers the design and construction cost of a bridge and maintenance and rehabilitation costs. The Net Present Value calculation was introduced, which accounts for the discount rate, cash flow period, and present value. A procedure to calculate the cost assigned to overloaded vehicles to deck and steel girders was presented for long-term fatigue damage. The approach includes moment calculations for overload

vehicle, a number of cycles to failure for at the considered stress level and cost of deck and steel girder. For future recommendations, the simplified approach for common bridges is to be developed to be able to assess the cost associated with permit vehicles to bridges.

Evaluation of a Permit Vehicle Model Using Weigh-in-Motion Truck Records

(Zhao and Tabatabai 2012)

This research utilizes 5% of the heaviest WIM data for each vehicle class to verify the adequacy of the notional permit vehicles in Wisconsin. The live load analysis for simple span and 2-3 continuous span bridges was used. It was concluded that 5-axle, short trucks might cause greater load effects in bridge girders than standard permit vehicles. Therefore, a new 5-axle truck model was proposed to supplement the standard permit vehicle for Wisconsin's bridge design and rating procedures. The state-specific traffic model was used to adjust the code provisions.

Development of state-specific load and resistance factor rating method

(Ghosn et al. 2013).

Ghosn et al. calculated site-specific load factors for bridge evaluation based on the example of the reliability analysis of New York State bridges. Presented methodology was applied for a rating of existing bridges, posting of understrength bridges, and checking of permit trucks. The WIM data was utilized to develop live load models used in calibration. The assumption in the state-specific LRFR method is consistent with those from the original AASHTO LRFD. It was noticed that in New York, for some instances, the load effects caused by the vehicles are larger than the loading considered in AASHTO LRFR calibration. Calibration for legal and permit vehicles for New York was developed, and recommendations for load factors that provide a uniform reliability index were proposed. The live load factor

for legal loading had to be increased, but the live load factor decreased for permit vehicles. The procedure presented in the paper provides a tool for possible calibration of site-specific live load factors adjusted to New York State example traffic.

WIM-based Live load for Bridges

(Nowak and Rakoczy 2013)

Calibration of site-specific live load statistical parameters can be determined by using the Weigh-in-Motion database. Still, the prediction of maximum expected loading in the service life of a bridge is not feasible. Calibration for Strength Limit State I is specified for 75 years. There is not enough data to determine the maximum expected load over the economic bridge life. Other limit states such as Strength Limit State II require one year of data. The rating factor for bridge evaluation, depending on the source, requires two or five years of data. Even having sufficient one or two years of data, it needs to be collected in several cycles. The study presents a method to extrapolate the maximum expected live load values depending on the considered time period and Average Daily Truck Traffic. Therefore, the statistical parameters of live load are determined for time periods from 1 day through 100 years. Live load statistical parameters are developed for traffic collected from several WIM sites. The statistical analysis of load effects provides a basis for the development of national live load parameters and live load factors for the bridge design and evaluation.

Bridge Load Rating of a Superload using AASHTO LRFR

(Lawson et al. 2013)

The study involved a rating analysis of over 50 bridges for the superload vehicle with a GVW of 715 tons, traveling from California to Utah. It contained AASHTO rating procedures, with adjustments in analyzing old bridges and their material deficiencies.

Concrete and steel strength was adjusted for bridges built before 1980. Also, additional bridge restrictions and specialized inspections were considered, including bridge closing, speed limits temporary shoring. The movement of superload over the bridges is changing and required special analysis that is time-consuming, costly, and damaging for infrastructure. The suggested approach includes the use of screening vehicles, which guide permitting procedures. A permit can be safely operating as a single trip if its axle weights and spacings fall within any of the proposed screening vehicles (without considering types, capacities, or span configurations of the bridges that a permit truck will pass on its route). Otherwise, if a vehicle needs to be further evaluated by other methods. This method offer sets of screening trucks, which then can be checked against rating trucks. The goal is to make the permitting process easier and propose as many as possible screening vehicles that reflect the state-specific permit vehicles.

The effect of truck permitting policy on US bridge loading

(OBrien et al. 2013)

The study analyzes the effect of heavy permit vehicles by utilizing WIM data from Arizona, Illinois, and Indiana. The accurate effects of traffic loading are essential for bridge management and safety. The three configurations of heavy vehicles are analyzed in this paper. It was established that the live load model provides a good agreement when compared against the load effects calculated directly from the WIM data. WIM data may be an excellent source to evaluate the load effects caused by the overloaded vehicles and provide a basis to improve the permit fee structures.

Procedure for Statistical Categorization of Overweight Vehicles in a WIM Database

(Fiorillo and Ghosn 2014)

The traffic is constantly changing, and the number of issued permitted, overloaded vehicles increase. Overloaded vehicles contribute significantly to bridge life consumption. The cost attributed to the repair and maintenance of the highway infrastructure system due to the overloaded vehicles can be appalling. The WIM data may be used to verify the number and load effects of overloaded vehicles operating on bridges. Detailed information provided by the WIM system allows evaluating the effects caused by existing traffic.

The study presents the data mining procedure to identify overloaded vehicles, categorized as permits and illegal vehicles. The algorithm was developed to distinguish the heavy vehicles in upstate New York. The algorithm validation is compared with the results of a truck survey performed by the New York State Department of Transportation (NYSDOT). A parametric analysis is executed to assess the sensitivity of the results and the level of accuracy of the WIM system. It is concluded that the algorithm produces statistically robust overweight truck and permit categorizations, which can eventually help highway agencies establish rational permit issuance policies, weight enforcement strategies, and cost allocation studies. Assessment of load effects caused by overloaded vehicles is essential in bridge posting decisions. The maximum expect load effects from permit and illegal vehicles can be distinguished from the WIM database and analyzed separately to update bridge evaluation policies.

Assessment and Evaluations of I-80 Truck Loads and Their Load Effects 2016

(Barker and Puckett 2016)

Wyoming DOT conducted a study to assess the safety of the bridges on the I-80 corridor. Wyoming's I-80 carries a large volume of heavy trucks compared to many states. Hence, the study verifies if the AASHTO LRFD provisions provide adequate provisions for heavy traffic. The reliability-based analysis was done to examine if the current provisions provide a minimum safety level at the target reliability index. The WIM data was used to develop a live load model. The traffic-induced loads were computed on a simple span and two equal continuous spans for 30-200 ft. The live load was extrapolated for 75-years economic bridge lifetime. Reliability analysis was conducted for steel bridges, with varying dead, wearing surface, and live load ratios. It was concluded that the AASHTO LRFD does not provide adequate safety, and it was proposed to increase the live load factor from 1.75 to 2.0 to achieve the minimum required safety set by the target reliability index for design 3.50.

Proposed AASHTO Load Rating Provisions for Implements of Husbandry

NCHRP Report 951

(Fu et al. 2020)

The study was aimed to assess the impact of the farm vehicles, usually called implements of husbandry (IoH). IoH load effects are considered for bridge design code AASHTO LRFD and bridge evaluation by AASHTO Manual Bridge Evaluation. The increase in the number of IoH vehicles and their weight and configuration are critical for bridge safety. The study examined the impact of the IoH vehicles on bridges, notional load models, live load, and dynamic analysis. The final product is a calibration of live load factor for IoH load rating and proposes revisions for AASHTO LRFD.

Assessment and Evaluations of I-80 Truck Loads and Their Load Effects

Phase 2: Service

(Barker et al. 2020)

Phase 2 of the project on assessment and evaluations of I-80 truck loads was conducted to assess the serviceability of existing Wyoming bridges. Reliability-based analysis was performed for 112 steel bridges and 60 prestressed concrete bridges. The considered limit state functions are Service II and Service III. The aim was to verify the adequacy of safety levels for serviceability limit states. The analysis shows that the current AASHTO LRFD provisions are deficient for I-80 Wyoming traffic. The live load factor for Service II limit state (yielding) and Service III (prestressed concrete cracking). It was recommended to increase the live load factors for Service II to 1.45 (from 1.30) and for Service III to 1.00 (from 0.80) to provide a minimum acceptable safety level.

Load-carrying capacity evaluation of girder bridge using moving vehicle

(Sun et al. 2021)

Sun et al. developed a procedure to evaluate the load-carrying capacity of the bridges based on the displacement of moving vehicles by using a radar measurements system. The method was verified by the FEM model, and it was proven that the presented bridge evaluation is effective and can be used instead of the conventional bridge testing method.

1.2.5. Permit fee structure

Permit regulations and monitoring procedures are developed to provide safety to the road and bridge infrastructure. Nevertheless, the issue of controlling the drivers violating the law remains unresolved, as well as the question to what extent the vehicles can be

overloaded. The law intends to protect motorists from traffic hazards caused by the movement of overweight and oversized vehicles or loads on state highways to minimize damage to infrastructure, thus protecting the investment in the highway system.

It is required by federal truck weight and size law for every state jurisdiction that vehicles exceeding the legal limits on size and/or weight must purchase permits to legally operate within that jurisdiction. State DOT's issue permits on a daily basis to oversize and overweight or both oversize and overweight vehicles. The permit fee structure varies significantly by state. There are typically single and annual multi trip permits. The annual multi-trip permits are valid for 12 months and for an unlimited number of trips. Single trip permits are valid from one point of origin to one destination. In the US, there five basic permit fee structures, including flat fees, distance-based fees, weight-based fees, weight-distance-based fees, and axle-based fees. **Figure 1.5** shows the permit fee structure that is adopted by different states (Chowdhury et al. 2013).

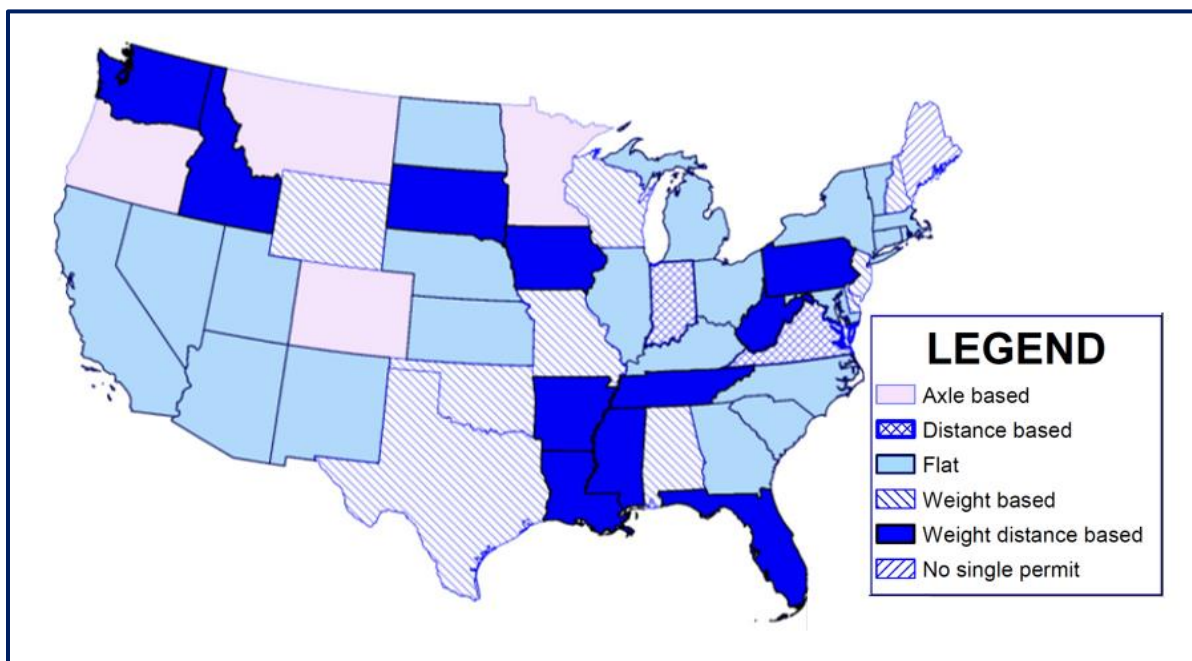


Figure 1.5. Permit fee schedule types in the U.S.

The comparison of the permit fees in neighboring states of Florida was presented in the study (Ali et al. 2020). **Figure 1.6** shows the single trip permit fee for the vehicles with a total gross weight of 43t (95 kips) in terms of a miles traveled. It can be noticed that fees are very different. Also, **Figure 1.7** presents the comparison for the same states but for fixed price multi trip permit vehicles. The permit fee schedule may be that distinct due to the fact that some of the permit fee schedules are outdated and have not been changed for many decades. A growing number of permit vehicles raise concerns, and there is an evident need to develop a method to assess the state-specific impact of permit vehicles. Several states conducted damage assessment analysis on pavement and bridges, and state of the art is listed in chronological order.

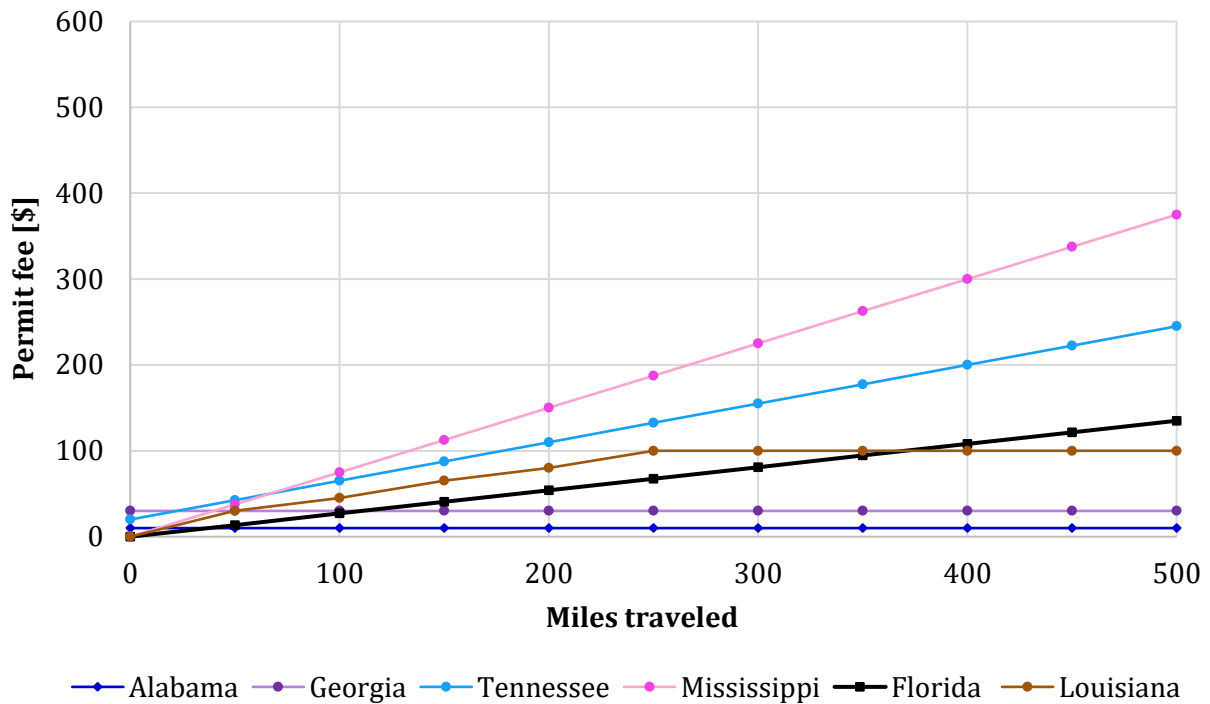


Figure 1.6. Permit fees vs. miles traveled for a single vehicle for considered states.

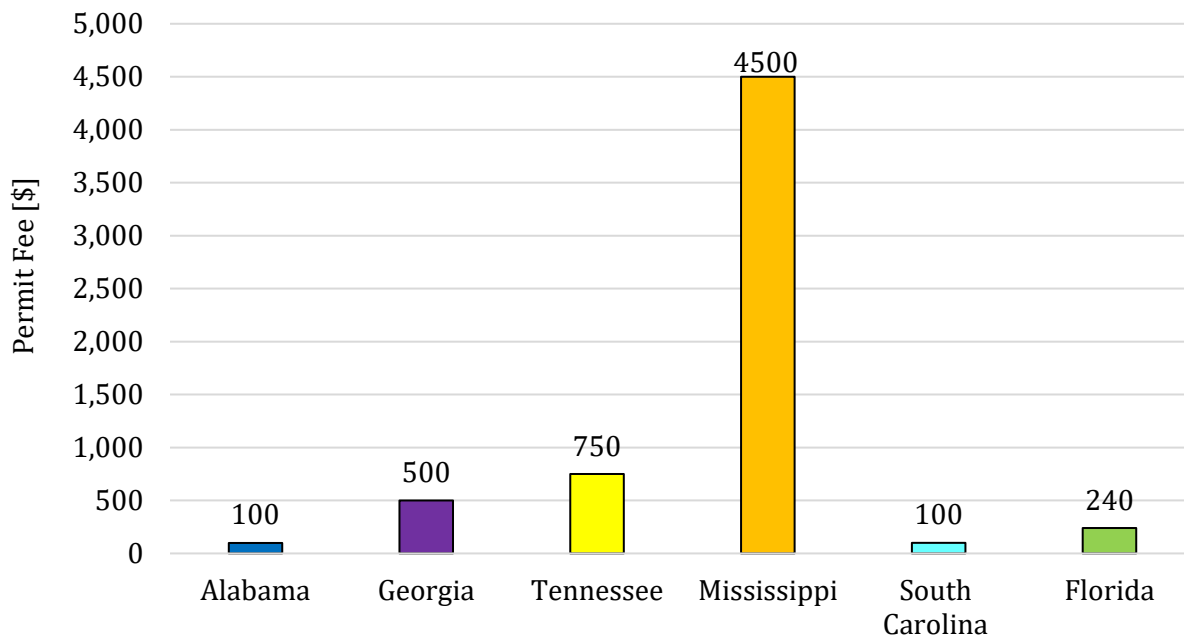


Figure 1.7. Permit fees for annual multi trip for various states.

Estimating the Cost of Overweight Vehicle Travel on Arizona Highways

(Straus and Semmens, 2006)

Straus and Semmens assessed the impact of overweight vehicles on Arizona state highways. It was reported that damage caused by overweighed vehicles consumes millions of dollars for maintenance, rehabilitation, and replacements. Overweight vehicles impose approximately \$12-\$53 million of damage on Arizona roadways. The Arizona enforcement budget is \$5.8 million per year. It was calculated if doubling the enforcement budget, where 50% of that budget was aimed towards the elimination of illegal vehicles from roadways, the savings would be at a range of \$6-\$27 million per year. The enforcement personnel survey was conducted, and the recommendation for the state's enforcement were made. It was concluded that for every dollar invested in motor carrier enforcement efforts, there would be \$4,50 in pavement damage avoided.

Impact of permitted trucking on Ohio's transportation system and economy

(Ohio Department of Transportation 2009)

Ohio Department of Transportation presented a study on the impact of permitted trucking on Ohio's transportation system and economy. Combined bridge and pavement impact cost was established \$144 million annually, where \$122 million resulting in pavement and \$ 22 million in bridges. For bridge structures, the study used the incremental method to quantify the damage directly in dollar terms. The impact was assessed for various roads, using bridge asset value per square footage, assuming the following bridge maintenance activities \$75/sq. foot for deck replacement, and \$20 /sq. foot for steel beam painting. Annual bridge preservation cost assigned to overloaded vehicles was calculated, and total asset consumption to find annual bridge costs assigned to overweight vehicles. The multi trip permit survey was conducted to find the number of trips for annual permits and the average length of the trip. The number of trips was estimated as 24.8 trips per year, and the average length of the trip 98.8 miles. The permit cost was computed based on permit vehicles, GVW, Equivalent Single Axle Load (ESAL)/mile, and miles traveled.

Oversize/Overweight Vehicle Permit Fee Study

(Prozzi et al. 2012)

Prozzi et al. performed a study to assess the damage caused by oversize and overweight vehicles on bridges and pavements in Texas, along with enforcement and management cost. The methodology was developed to find consumption in dollars per mile. A new permit fee was proposed for 34 different rate categories. Also, the administrative cost of \$10 was added as required for each issued permit and TxDOT Base Fee of \$40 to help recover fee revenues. Permit fees from 2011 generated \$111.4 million in revenue, the new

permit fee structure will generate \$521.4 million, and considering permits for exempt vehicles, the revenue is estimated as \$671.4 million. The analysis results shows that the current permit fee schedules may be inadequate and disproportional to the damage caused by overloaded vehicles.

Aligning oversize/overweight fees with agency cost: critical issues

(Adams et al., 2013)

Adams et al. reviewed the permit practices and provided guidance on fee structure and permit demand management. It was noticed that some of the freight is favorable by the agencies, but others are the only possible mode of transport. Certain industries are crucial for the state's economic growth, and specific legislative provisions consider it. DOTs have to protect infrastructure first and later promote commerce. The recommendation from the study is to unify the permit fee system, reduce the time required to obtain a permit, and simplify the administrative burden. A new permit system should improve the fee structure, recognize, and adapt to trends, and contribute to overall management and uniformity. The study concludes that permit fees do not recover the issuance costs, but for some agencies, the fees are not designed to recover infrastructure consumption and accelerated damage on roads and bridges.

Review and Revision of Overload Permit Classification

(Barker et al., 2013)

Michigan DOT requested a study to create a more robust system, which ensures the safety and control of the operation of overloaded vehicles. It was aimed to develop a system that assesses the impact of vehicles requesting a permit to more efficient using the computing capacity, to reduce the number of manual bridge structural analyses. Moreover,

it was intended to check the live load capacity of the bridges and flag those that are not able to carry overloaded traffic. The study created software that enhances the state permitting system. It was developed together with AASHTOWare and Vitris software.

Rate of Deterioration of Bridges and Pavements as Affected by Trucks

(Chowdhury et al. 2013)

South Carolina DOT commissioned a study to investigate the impact of heavy vehicles on pavement and bridges to develop the policy recommendations. The pavement and bridge deterioration models were introduced, along with trucking industry response to recommended permit fee schedule changes. The developed permit fee varies between \$24 - \$175 per trip, and the flat fee is charged for all overweight trucks of \$65 (including a \$10 administrative fee). This research utilized WIM data to assess the live load effect. Finite Element bridge models were created to assess the stress range caused by heavy traffic. The fatigue damage and cost estimation allowed the development of a new permit fee schedule.

Assessment of Current Design Loads for Permit Vehicles

(Laman and Shah, 2016)

This study was designed to evaluate the current PennDOT Live Load Design Permit Vehicle, P-82 (8-axle, 204-kip truck configuration) and PHL-93 for adequacy as a model to predict loading effects of current special hauling permit vehicles, specifically from the heaviest Pennsylvania-issued superload permits. The study objective was to develop the analytical tools needed to evaluate the truck data files, both WIM, and superloads. The study proposed a permit design vehicle model that envelopes all or most WIM and superload vehicles in the PennDOT-provided databases. The developed permit design vehicle was

presented for PennDOT consideration. Notional permit vehicles can be used in bridge design, evaluation, and simplification of permit issuance.

Development of a proposed overweight vehicles permit fee structure in Illinois

(Al-Qadi et al., 2017)

Al-Qadi et al. performed an analysis to update the current permit system by evaluating the impact of overweight vehicles. The study assesses the damaging impact on pavements, bridges, and traffic safety. The bridge damage assessment is based on bridge load carrying capacity and vehicle weight frequency. National Bridge Inventory Database (NBI) and Weigh-in-Motion were utilized to develop the prediction engine to calculate the bridge fees. Extensive data cleaning procedures were introduced. I was a first permit study that contains a large bridge data sample. Previous studies considered only a few bridge structures, providing the FEM model, which involves high computational cost. The limited scope of samples raised the discussion about the bias associated with the research. This study considered the NBI database that includes inventory rating for each bridge, which was account as bridge strength. Also, the vehicle's weight frequency was also calculated for available WIM sites and then combined by the location with the bridges.

Expected bridge life calculation considering overweight impact was performed. Bridge life-cycle cost analysis and damage quantification were conducted. It led to calculation of average damage cost per mile for representative bridge and traffic sample for Illinois. Permit fees were calculated as a summation of damage on pavements, bridges, and safety and introduced in few scenarios compared with neighboring states for DOT consideration.

***Impact of Heavy Trucks and Permitted Overweight Loads on Highways and Bridges
Now and in the Future versus Permit Fees, Truck Registration Fees, and Fuel Taxes***

(Ali et al., 2020)

Ali. et al. developed a damage assessment approach to calculate the monetary consumption caused by overweight permit vehicles on bridges and pavements in Florida. The bridge damage assessment was based on the fatigue damage measured by the equivalent bending moment on a representative bridge. Results of the bridge consumption analysis are given in \$/miles. To pavement damage, the cost was presented in terms of equivalent single axle loads (ESALs). The pavement and bridge damage costs were combined and presented in the same format as the existing overweight permit fee structure. Proposed fees are higher than the existing ones; they reflect the actual cost of damage to Florida's roads and bridges. A new permit fee schedule may increase the permit revenue by a factor of 1.6-2.7.

1.2.6. Reliability-based methods

The current design codes are based on the reliability approach defined by the limit state function and load and resistance factors. AASHTO defines four types of limit states: ultimate limit state (ULS), serviceability limit state (SLS), fatigue limit state, and extreme event limit state. ULS considers moment carrying capacity, shear capacity, axial compression, and axial tension. SLS includes cracking, deflection, and excessive vibration. The fatigue limit state is defined as when the number of load cycles is required to reach a limiting value. The extreme event limit state applies to earthquakes and other natural disasters.

Load and resistance safety factors are applied to nominal load effects to obtain factored design values. In many design codes, the SLS factors are taken equal to 1.00.

A mathematical representation of the border between acceptable (safe) performance and unacceptable (failure) performance is the limit state function. If variable R represents resistance, and Q is load effect, the limit state function can be expressed by **Equation (1.2)**:

$$g(R, Q) = R - Q \quad (1.2)$$

Load, Q , resistance, R , and limit state function $g(R, Q)$ are presented on the probability density functions (PDF) in **Figure 1.8**. The structure is considered safe if $g(R, Q)$ is zero or positive and unsafe if it is negative. The graphical representation limit state function shown as shaded area represents the probability of failure, $g(R, Q) < 0$.

The limit state function, g is the difference of two random variables, R and Q , its mean is the difference of their means, expressed in **Equation (1.3)**:

$$\mu_g = \mu_R - \mu_Q \quad (1.3)$$

where:

μ_g – mean value of limit state function,

μ_R – mean value of resistance,

μ_Q – mean value of load.

The standard deviation of the limit state function is shown in **Equation (1.4)**:

$$\sigma_g = \sqrt{\sigma_R^2 + \sigma_Q^2} \quad (1.4)$$

where:

σ_R – standard deviation of resistance,

σ_Q – standard deviation of load.

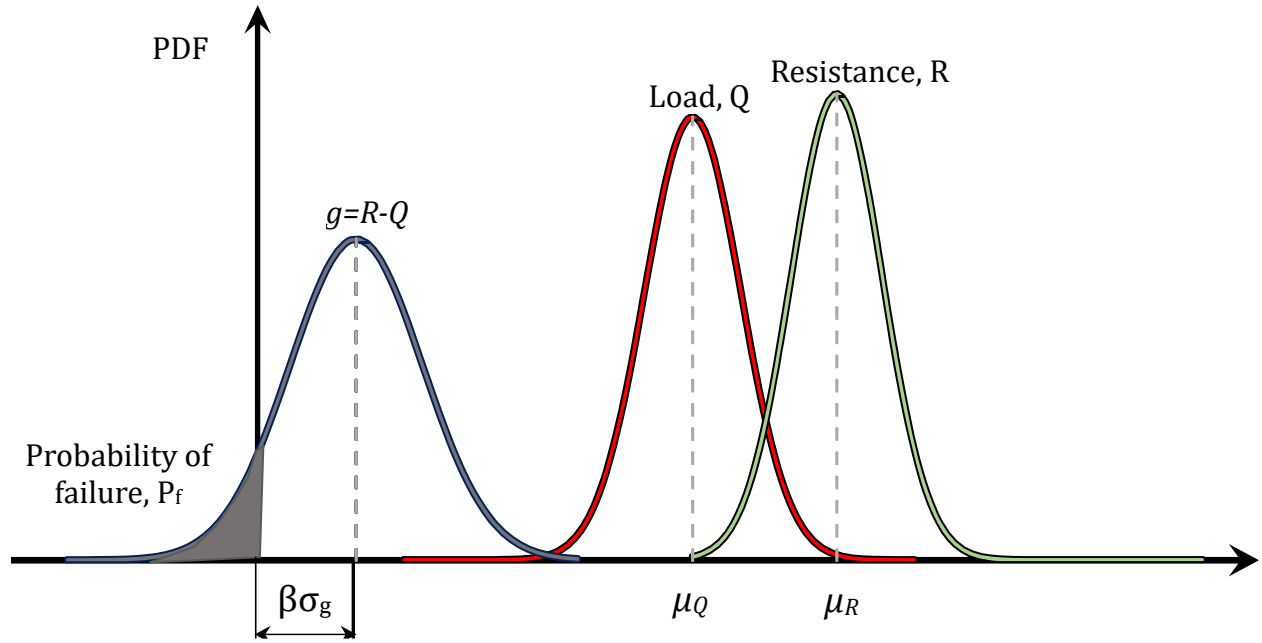


Figure 1.8. Probability density function (PDF) for load, resistance, and limit state, g .

In Load and Resistance Factor Design (LRFD), the acceptability criterion for the minimum safety margin is specified in terms of a reliability index (an indicator of the probability of failure. Load and resistance parameters are treated as random variables, and the reliability index is calculated accordingly. The reliability index may be defined by **Equation (1.5)** (Cornell 1968):

$$\beta = \frac{\mu_g}{\sigma_g} \quad (1.5)$$

Hence, it can be expressed by **Equation (1.6)**:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (1.6)$$

The probability of failure, P_f , can be calculated using the CDF of limit state function, g. If load, Q and resistance, R are both normal random variables, then it can be expressed as

Equation (1.7):

$$P_f = \Phi(-\beta) \quad (1.7)$$

where:

Φ – standard normal variable.

In the LRFD codes, the acceptance criterion for load and resistance factors is to keep the probability of failure at an acceptably low level. In practice, rather than a probability of failure, the structural condition can be defined in terms of a reliability index, β . Calculation of the reliability index requires knowledge of statistical parameters of load and resistance. The load and resistance factors are selected so that the designed structures have at least the minimum acceptable reliability, i.e., β being not less than the target reliability index, β_T . The target reliability index depends on the consequences of bridge failure by exceeding the limit state and increased safety costs. Bridge design and evaluation requirements are established by American Association of State Highway and Transportation Officials, AASHTO. Currently, code provisions are based on the LRFD for design and LRFR for rating. These two approaches were developed and documented in NCHRP Report 368 and NCHRP Report 454.

Calibration of LRFD Bridge Design Code. NCHRP Report 368

(Nowak 1999)

NCHRP 368 presents the load and resistance factors calibration for the AASHTO bridge design code. The calibration procedure required developing load and resistance models, selecting reliability analysis methods, and calculating reliability indices for various

bridges and span lengths. Statistical parameters of the dead, live, and dynamic load were reviewed. The resistance model was developed for steel, reinforced concrete, and prestressed concrete girder bridges. A new live load model was based on truck Ontario survey data. The maximum expected load effects were determined. The design live load provides a uniform safety level for a wide variety of bridge structures. The probability-based approach was developed to verify structural performance in terms of reliability index. The acceptance criterion for design provisions is closeness to the target safety level. The target reliability index for bridge design is 3.50. Reliability indices calculated for developed load and resistance factors using LRFD code are close to the target value for all materials and spans. AASHTO LRFD code calibration from 1999 is the basis for developing the new AASHTO Manual for Bridge Evaluation. All new calibration methods are consistent with presented in NCHRP 368 calibration method.

Calibration of load factors for LRFR bridge evaluation NCHRP Report 454

(Moses 2001)

AASHTO Manual for Bridge Evaluation (MBE) provides procedures to evaluate bridge live load capacity using the Load and Resistance Factor Rating (LRFR) analysis. Bridge evaluation procedures have evolved through the years and adapted to changing traffic volume and its parameters, like increased vehicle size and weight, axle loads, etc. The development of AASHTO guides for bridge evaluation was summarized by (Gao 2012). In 2003, the “AASHTO Guide Manual” was introduced in the NCHRP 12-46 (Moses 2001) to supplement already implemented in the US Load and Resistance Factor Design provisions. The first edition of the AASHTO Manual for Bridge Evaluation was released in 2008. It was developed to help bridge owners to maintain safety under a considerable variation of traffic

loads. MBE provides rating procedures by utilizing notional vehicles representing design, legal, and permit loads. The notional vehicles were developed to represent traffic across the US, and live load factors are used to envelop the load effects and provide uniform safety for a wide range of bridges.

1.3. PROBLEM STATEMENT

Numerous bridges in the United States require major repair, rehabilitation, or replacement. The growing number of overloaded vehicles and more irregular vehicle configurations used to transport heavy loads significantly affect the infrastructure system compared to standard vehicles. They have to be considered in the permitting and operation process. Their load effects have not been explicitly quantified; therefore, it is recommended to verify AASHTO design and evaluation provisions for overloaded vehicles, including grandfather and permit vehicles.

AASHTO live load analysis uses notional trucks to represent the traffic-induced load spectra. Every state uses its own unique truck(s) for permit vehicles, developed by different methods. Hence, it is important to propose a reliability-based procedure to develop notional permit vehicles that provide uniform safety for a wide variety of bridge types and is consistent with the AASHTO specifications.

Currently, there are no rating provisions for legally overloaded vehicles under grandfather provisions. The grandfather provisions are old rules which remain unchanged after a new rule was introduced. Therefore, some vehicles can operate above the federal truck weight and size law. The load effect caused by legally overloaded vehicles is not reflected in bridge rating provisions. Thus, it is essential to provide a methodology to

validate if the load effect caused by exempted vehicles should be considered to adjust the live load factor for bridge rating.

Moreover, to manage the operation of overloaded permit vehicles, it seems necessary to evaluate the damage to bridges caused by overloaded vehicles and measure the monetary consumption. The damage assessment can serve as a basis for the development of a new permit fee schedule. The major contributions of this dissertation provide the following to evaluate the impact of overloaded vehicles:

1. Assess bridge life consumption based on fatigue damage analysis. The monetary consumption approach is developed to help state agencies to establish a rational and fair permit fee structure.
2. Propose a reliability-based method to determine a notional permit vehicle(s) based on issued permits and weigh-in-motion (WIM) data. Notional permit vehicles aim to represent existing traffic to simplify the design and evaluation analysis of bridges.
3. Use a notional permit vehicle(s) to simplify the permit issuance procedure and help to control permit vehicle operation and maintain bridge safety.
4. Provide a method for verification if the current live load factors provide adequate safety for exempted legally overloaded grandfather vehicles.

1.4. DISSERTATION OUTLINE

This paper-based dissertation consists of five chapters and four appendices. The first chapter of the dissertation consists of the introduction, motivation, literature review, and problem statement. Chapters 2-4 contain three papers, with the overall topic being the impact of overloaded vehicles on bridges. Each of these chapters begins with an abstract, introduction, literature review, research approach, results, discussions, and conclusions.

Chapter 5 is a summary of the dissertation with overall findings and a future research section. Tables and figures are embedded in the text. Appendix 1 includes the data sources summary and developed algorithms. In Appendices 2-4, there are additional supporting figures and tables for papers in Chapters 2-4.

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CHAPTER 2 BRIDGE LIFE CONSUMPTION BY PERMIT VEHICLES¹

ABSTRACT

Excessive weight and a growing number of permit vehicles can cause an accelerated consumption of bridge life. Therefore, there is a need to better control operation of permit vehicles. The objective of this paper is to present an approach for assessment of bridge life consumption. An economic lifetime of a bridge is considered in terms of a number of load cycles to failure so that the structure has to be replaced. Each truck's passage causes one or more load cycles and thus reduces or consumes a part of the total remaining fatigue life. The consumption by a permit vehicle includes the legal weight of the vehicle that does not require a permit and the weight representing the load that exceeds the legal limit. An incremental consumption equation is derived that converts fatigue damage to dollar damage by considering the bridge construction cost, traffic volume, and state-specific parameters. The consumption analysis is presented for single and multiple trip permits using Florida as an example. This approach can help state agencies to establish a rational and fair permit fee structure. The methodology is transparent and can be used by any state utilizing the available databases.

¹ under review in Structure and Infrastructure Engineering Journal
Part of the project Ali, H., Nowak, A. S., Stallings, J. M., Chmielewski, J., Stawska, S., Babu, A. R., and Haddadi, F. (2020). "Impact of Heavy Trucks and Permitted Overweight Loads on Highways and Bridges Now and in the Future versus Permit Fees, Truck Registration Fees, and Fuel Taxes."

2.1.INTRODUCTION

Highway infrastructure is a major part of the national investment. A good condition of roads and bridges is vital for efficient transportation and economic growth. According to the ASCE Infrastructure Report Card from 2017, there are approximately 600,000 bridges in the United States, and over half of them are approaching the end of their design life. About 25% of bridges are not adequate for normal traffic, including 9% that are structurally deficient and 13% being functionally obsolete (ASCE Cards, 2017). The trucking industry is crucial for economic growth, but operation of heavy vehicles has to be managed and controlled to prevent premature damage to roads and bridges.

A poor condition of bridges can result in premature needs for bridge repair, rehabilitation, and/or replacement. In addition to deteriorating condition of bridges, there is a growing number of heavy trucks. Based on the weigh-in-motion (WIM) data analysis, it was observed that an annual truck traffic growth could be over 3%. Heavy trucks impact the service life of bridges and contribute significantly to accumulated damage. The traffic loads include three groups of vehicles legal, illegal, and permit vehicles, which can legally exceed the legal limits after purchasing the permit. The number of issued permits is growing every year, and their impact has not been quantified. In most states, the load regulations pertaining to permit vehicles are outdated, and they have not been changed for decades.

In this study, bridge consumption is considered in terms of service life or the number of load cycles until the structure reaches the fatigue limit state. The aim is to provide a simple method that captures the relative consumption or deterioration of the AASHTO bridge design life caused by a wide range of overloaded permit vehicles. A fundamental point of the proposed method is that the life of the bridge is defined by 75 years of crossings by

the AASHTO fatigue design truck. Each passage of a vehicle contributes to the accumulation of fatigue damage that eventually results in the total consumption of the ability to carry the loads. The percentage of contribution depends on the vehicle weight, axle loads, and axle configuration. However, the fatigue life can be considered as corresponding to the cost of a bridge replacement. Therefore, each vehicle passage can be attributed a monetary value. The objective of the present study is to propose a methodology for assessing the fatigue and dollar damage to bridges caused by permit vehicles.

The proposed approach involves permit data, and traffic volume analysis, load effects calculations, assessment of bridge construction cost, and other state-specific parameters. The developed procedure is to help bridge owners to establish a rational and fair permit fee structure. It is applicable for all states, and it is demonstrated for the selected single and multiple trip permits using Florida as an example.

2.2.LITERATURE REVIEW

The number of issued overloaded permit vehicles is growing. In Florida, it is about 5% every year. Based on Florida WIM data, about 10-12% of WIM records are overloaded vehicles. The Departments of Transportation (DOT's) are concerned about permit load effect on infrastructure. To determine the impact of overloads, it is necessary to evaluate the traffic data and its effect on bridge structures. The WIM and permit database are essential to assess the load effects on bridges. The calculation of the load effect can be challenging when dealing with massive traffic data. Currently, the WIM station data is the main source of information about the traffic and its attributes. Several studies utilized the WIM data to identify specific overloaded vehicles in the traffic flow to assess their load effects (Ghosn et al., 2015; Bae & Oliva, 2012; Chotickai & Bowman, 2005; Ghosn et al., 2015; Reisert & Bowman, 2005).

The changes in truck traffic volume, weight, and configuration in recent decades were reviewed by (Anitori et al., 2017; Ghosn et al., 2011; Liao et al., 2015; Ramesh Babu et al., 2019; Treacy & Brühwiler, 2012). A set of protocols and techniques for collecting and analyzing WIM data and methods for calculating traffic load factors to be used in LRFD design are presented by (Ghosn et al., 2011).

The impact of various types of overloaded vehicles on bridges was studied to assess the damage and behavior of bridges under excessive loading. In Louisiana, the impact of overweight permitted vehicles hauling timber, lignite coal, and coke fuel was conducted to assess the economic impact on pavements and bridges. The analysis estimated permit fees and provided the recommendation for timber trucks in Louisiana (Roberts et al., 2005). In Wisconsin, a special user guide was developed to calculate the impact of overloaded vehicles for single and dual trailer configuration (Bae & Oliva, 2009, 2012). In Connecticut, an overload vehicle's impact was assessed by checking the effect of one passage of a large permit vehicle across a specific bridge. The measured stress from superload vehicles was significantly lower than calculated by AASHTO Guide Specification (Culmo et al., 2004).

Fatigue of older steel bridges in northern Indiana due to overweight and oversize loads were evaluated on the heavy-duty corridor. It was found that 15% of class 9 trucks and 26% of class 13 trucks travel heavier than their respective legal limits. The remaining fatigue life of in-service steel bridge structures for critical details was estimated as 25 years (Reisert & Bowman, 2005).

A number of studies sponsored by state DOTs attempted to evaluate the impact of overloaded permit vehicles on bridges. Wisconsin presented a review of permitting fee structure for several states (Adams et al., 2013). In many instances, overloaded trucks are

operating on highways, roads, and bridges that are not designed for such a weight or traffic volume. A new permit fee structure calculated based on the actual damage caused by permit vehicles can help state agencies with administering permits and enforcing permit regulations to recover the costs associated with overload and oversize vehicles operation.

South Carolina DOT performed a study to investigate the impact of heavy vehicles on pavement and bridges. The pavement and bridge deterioration models were introduced to verify and propose the policy changes. A bridge deterioration model was based on WIM data analysis and FEM models of steel, reinforced concrete, and prestressed concrete bridges. The fatigue damage was calculated based on stress cycle analysis and compared to an allowable number of stress cycles (Chowdhury et al., 2013).

Texas Department of Transportation (TxDOT) and the Federal Highway Administration completed a study to calculate the dollar damage caused by oversized and overweight vehicles to provide recommendations for permit fee schedules (Prozzi et al., 2012). In New Jersey (Nassif et al. 2015) performed a study on the damaging effects of heavy truck traffic. A deterioration model was proposed based on the effects of overweight vehicles. A special software tool was created to estimate the actual damage costs on New Jersey highways. Illinois Department of Transportation examined the permit system by evaluating the impact of overweight vehicles. The study assessed the damaging impact on pavements, bridges, and traffic safety (Al-Qadi et al., 2017).

Florida DOT sponsored a study to assess the damage caused by permit vehicles. An approach was developed to calculate the monetary consumption caused by overweight permitted vehicles on bridges and pavements in Florida. The pavement and bridge damage costs were calculated and combined to propose a new permit fee structure (Ali et al., 2020).

The monetary bridge damage assessment procedure utilized in the Florida study is shown in this paper. The proposed methodology can be applied to any state.

Permit vehicle regulations and adequate permit fees are essential to keep bridges in good condition and ensuring safety. Excessive damage caused by heavy permitted vehicles may have a critical effect on the condition of the bridge. That is why state DOTs evaluate the permit fee schedules according to their state-specific bridge and traffic parameters. In the present study, a methodology is proposed to calculate dollar damage using the permit database. The fatigue incremental consumption approach requires permit data and state-specific parameters to assess the monetary consumption of overloaded vehicles.

2.3.CONSUMPTION BY PERMIT VEHICLES

Permit operation and their impact on the infrastructure has not been explicitly quantified. DOT's are interested in determining the degree of damage caused by overweight vehicle operation. To reasonably define the agency and user costs, it is essential to charge permit vehicles with a fair permit fee that corresponds to the damage they cause. **Figure 2.1** presents the overall procedure to develop a permit fee schedule based on the monetary consumption caused by overloaded permit vehicles.

To assess the effect of overloaded permit vehicles, traffic data is a key input. Heavy vehicle configurations can then be used to determine their impact on the roads and bridges. In this study, the impact of overloaded permit vehicles is defined as fatigue bridge life consumption. The permit fee schedule is meant to reflect the damage caused to roads and bridges, and to some degree, the other additional administrative costs, services such as interruption in the traffic, escorts, temporary structures strengthening, etc.

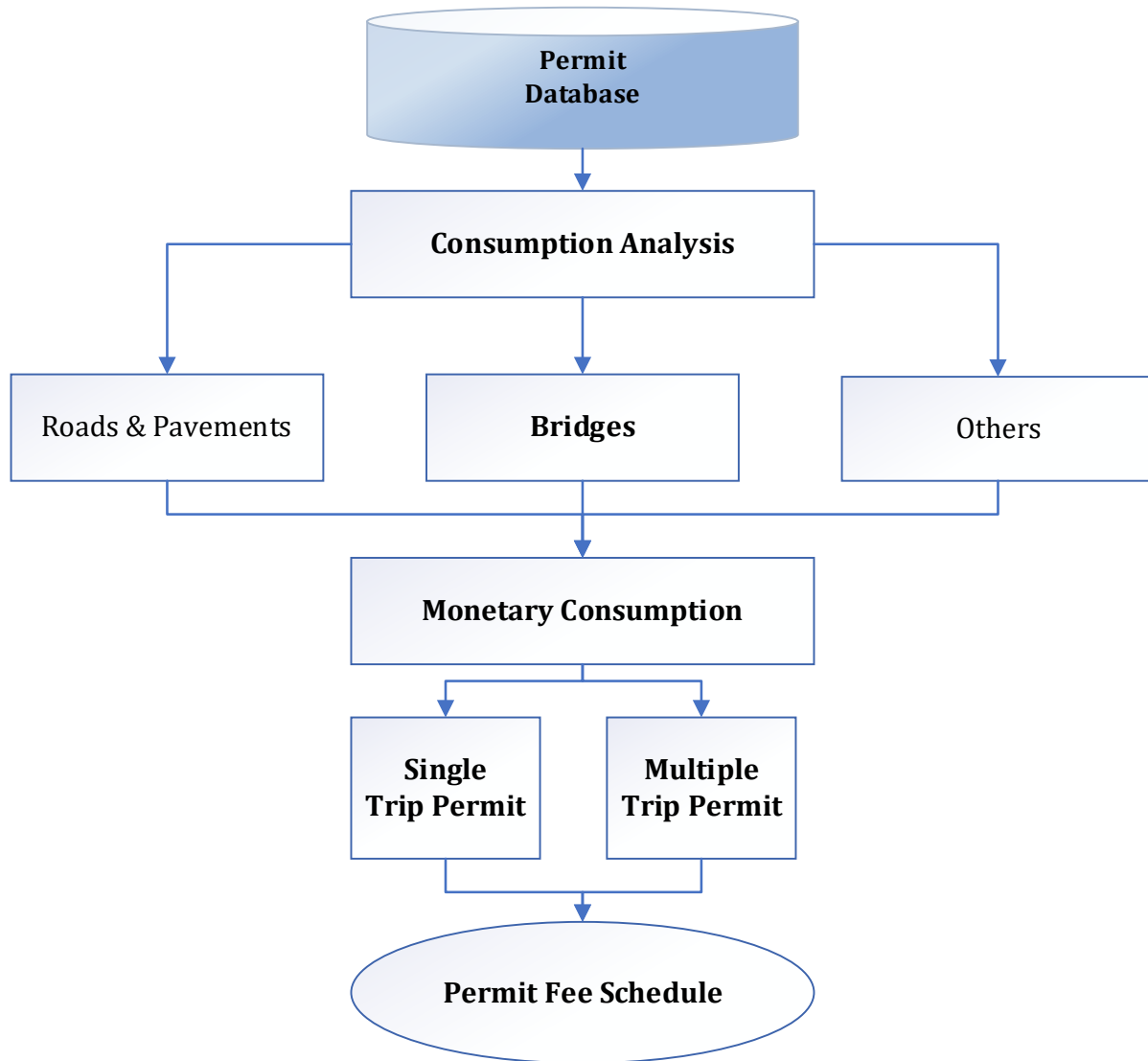


Figure 2.1. Permit fee schedule's development based on the infrastructure consumption.

The overall damage caused by the permit vehicles can then be presented in terms of dollars. It is a common practice that permits are issued for a single trip or multiple trips. That is why the permit fee schedule needs to be considered separately for a single trip and multiple trips permit due to their distinctive characteristics. Consumption assessment provides the basis for a fair approximation of damage caused by excessive load and size of operating vehicles. This study presents only the bridge consumption analysis.

The service life of a bridge depends on many factors such as climate, traffic loads, natural hazards, defects in material production, extreme events, etc. The damage assessment of permit vehicles only considers the impact of excessive loading, includes the permit load effects above the federal weight and size limits. So, the monetary consumption by a permit vehicle is calculated only for the overload part of the vehicle's weight since bridges are not designed for excessive permit loading. The climate impact and natural hazards are not considered in the permit damage assessment since unpredictable events contribute to the infrastructure damage, independently of the traffic. Similarly, material, production, and analysis assumptions are not considered because the uncertainty of random variables is assessed in bridge specification safety factors.

Traffic-induced loads may cause damage to a bridge by fatigue and overstress. Every passage of a truck across a bridge creates one or more stress cycles in the structural components, which result in the accumulation of fatigue damage over time. A bridge may experience many fatigue load cycles by heavily loaded trucks over its lifetime. If the stress cycles are of a certain number and magnitude, they will result in fatigue damage. Therefore, the damage caused by permit vehicles has to be assessed to calculate the consumption of bridge structures (Ramesh Babu, 2019). In the current bridge design code AASHTO LRFD 2020, a bridge's economic design life is defined as 75 years. The stress range is calculated for the code-specified fatigue design truck to prevent fatigue cracking caused by the accumulation of damage from cyclic truck loading. The AASHTO fatigue design truck is 0.80 of the design truck HL-93 (AASHTO LRFD 2020), and it is intended to represent truck traffic. However, there is a considerable degree of uncertainty in predicting the traffic loads during the service life of a bridge.

2.3.1. Bridge fatigue consumption

Bridge consumption is considered as a fatigue process in which each permit vehicle passing over a bridge consumes a part of its design life. The damage caused by a vehicle depends on each axle load and axle spacing. WIM database contains millions of records on vehicles, including permit ones. In the fatigue damage assessment used in this study, the permit database serves as a primary source of information. The data includes detailed information about the type of permit, planned route, and, more importantly, about axle spacings and axle loads for each issued permit. It is also possible to use the WIM data, but it may be difficult to separate permit vehicles from WIM's general traffic flow.

The fatigue resistance of material and connection detail is presented as a relationship between stress range and the number of load cycles to failure on a logarithmic scale, so-called the S-N curve. The resistance relates the magnitude of the applied constant-amplitude stress range (S) to the corresponding number (N) of cycles to failure. **Equation (2.1)** shows a relationship between the stress range and the number of cycles to failure. AASHTO Specifications provides S-N curves for different bridge detail categories, which were developed based on extensive laboratory tests.

$$A = N \cdot S^m \quad (2.1)$$

where:

N – number of cycles to failure,

A – constant for a given category of fatigue details [MPa/ksi],

S – stress range [MPa/ksi],

m – fatigue exponent, material dependent, as shown in **Table 2.1**.

Table 2.1 Values of m for bridge fatigue analysis (Prozzi et al. 2012)

Bridge Structural Material and Type	m
Concrete Slab	4.1
Concrete Girder	3.5
Concrete T Beam	4.1
Concrete Box Beam	4.1
Concrete Continuous Slab	4.1
Concrete Continuous T Beam	4.1
Steel Girder	3.0
Steel Continuous Girder	3.0
Steel Continuous Box Beam	3.2
Prestressed Concrete	3.5
Prestressed Concrete Slab	3.5
Prestressed Concrete Girder	3.5
Prestressed Concrete Box Beam	3.5
Prestressed Concrete Continuous	3.5

Bridges are subject to variable amplitude stress cycles. A cumulative damage theory, Palmgren-Miner (Miner 1945) rule, is used to calculate the effective stress range, **Equation (2.2)** from variable amplitude stress cycles as follows:

$$S_{eff} = \left[\sum \frac{n_i}{N} S_i^m \right]^{1/m} \quad (2.2)$$

where:

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

S_i – constant amplitude stress range [MPa/ksi],

Constant, m , for the fatigue analysis, depends on bridge material and structural type (Prozzi et al. 2012). **Table 2.1.** presents various magnitudes of the m constant. Depending on the bridge, the constant can take a value of 3.0, 3.2, 3.5, or 4.1. At a specific point along a bridge, the applied range of bending moment can be determined by multiplying the applied stress range by the section modulus. Multiplying each side of **Equation (2.2)** by the section modulus results in the effective moment range, **Equation (2.3)**.

$$M_{eff} = \left[\sum \frac{n_i}{N} M_i^m \right]^{1/m} \quad (2.3)$$

where:

M_i – i^{th} moment range [kNm/kip-ft].

Each permit truck creates a bending moment that changes as the truck crosses the bridge. This change in a bending moment may result in a single cycle or multiple cycles of different magnitudes depending on the truck load distribution, axle spacings, and bridge geometry. In this study, the bending moment at midspan due to a permit truck crossing bridge is determined using an influence line on a simply supported beam.

The rainflow counting method ASTM E1049-85 (American Society for Testing and Materials 2017) is employed to determine the number and magnitude of moment cycles resulting from each permit truck. **Equation (2.4)** is then used to determine the equivalent single moment range that accounts for all of the cumulative fatigue damage due to multiple cycles caused by a truck passage. If the vehicle creates one cycle while crossing the bridge, an effective moment equals the equivalent single moment. If the vehicle creates more than one cycle, the magnitude of the equivalent single moment differs from an effective moment.

$$M_{eq} = \left[\sum n_i M_i^m \right]^{1/m} \quad (2.4)$$

The presented fatigue damage assessment procedure uses the equivalent single moment range caused by permit vehicles to determine the bridge consumption caused by one crossing of a permit vehicle. In AASHTO, the number of fatigue load cycles N , **Equation (2.5)** defining the service life is represented by the relationship between the

economic bridge design life and the single lane average daily truck traffic $(ADTT)_{SL}$. According to AASHTO, this number of cycles in the 75-year service life is:

$$N = (365)(75)n (ADTT)_{SL} \quad (2.5)$$

where:

365 – days in a year,

75 – design life of a bridge in years,

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

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

To determine the damage caused by one crossing of a permit vehicle, the fatigue damage by one crossing of a permit truck is set to be equal to the damage caused by the AASHTO fatigue truck. **Equation (2.6)** compares the damage caused by a permit vehicle to the fatigue design truck effect during the design lifetime.

$$(1) (M_{permit})^m = (N_f) (M_{fatigue\ truck})^m \quad (2.6)$$

where:

M_{permit} – equivalent moment range due to the considered permit truck [kip-ft];

$M_{fatigue\ truck}$ – equivalent moment range due to the fatigue design truck [kip-ft];

N_f – number of fatigue truck crossings in the design lifetime.

The bridge consumption ratio between the considered permit vehicle and the fatigue truck can be expressed as:

$$N_f = \left(\frac{M_{\text{permit}}}{M_{\text{fatigue truck}}} \right)^m \quad (2.7)$$

Hence, **Equation (2.7)** can be interpreted as how much more damage was caused by the permit vehicle in comparison to the AASHTO fatigue truck for the same bridge type and material. It provides a rational basis to assess the damage caused by permitted vehicles. This fundamental relationship was used to assess the fatigue damage caused by a permit truck in terms of the design fatigue truck in AASHTO.

2.3.2. Monetary bridge consumption

It is assumed that when a bridge reaches its economic lifetime of 75 years, it has to be replaced at a certain construction cost that depends on its structural type, material, geometry, etc. The bridge consumption is represented by **Equation (2.8)**, which relates an equivalent moment by a permit vehicle to fatigue truck and bridge construction cost.

$$Consumption_{\text{permit}} = \frac{(Construction\ Cost)(N_f)}{N} \quad (2.8)$$

The consumption caused by a single permit vehicle can be calculated by substituting **Equation (2.7)** to **Equation (2.8)**. It provides the monetary consumption by a passage of a permit for representative state-specific parameters, shown by **Equation (2.9)**:

$$Consumption_{\text{permit}} = \left(\frac{Construction\ Cost}{(365)(75)n (ADTT)_{SL}} \right) \left(\frac{M_{\text{permit}}}{M_{\text{fatigue truck}}} \right)^m \quad (2.9)$$

Bridge construction cost varies for different bridges, depending on their structural type, geometry, length, and material type. For a general analysis, it is convenient to consider

representative state parameters such as construction cost, ADTT per single lane, the maximum span length, and also a bridge material and structural type.

The consumption **Equation (2.9)** is the total dollar damage caused by permit vehicles, and it includes the consumption that would be allowed without a permit if the vehicle met the legal load limits. In fairness, it is necessary to calculate the incremental consumption, which is defined as a difference between bridge consumption by a permit and legal vehicle, expressed by **Equation (2.10)**:

$$Incremental\ Consumption = Consumption_{permit} - Consumption_{legal} \quad (2.10)$$

The consumption caused by a typical legal vehicle can be expressed by **Equation (2.11)**:

$$Consumption_{legal} = \left(\frac{Construction\ Cost}{(365)(75)n\ (ADTT)_{SL}} \right) \left(\frac{M_{legal}}{M_{fatigue\ truck}} \right)^m \quad (2.11)$$

where:

M_{legal} – equivalent moment range due to a typical legal vehicle in traffic [kNm/kip-ft].

The proposed approach allows for computing the monetary consumption for a large permit dataset. It provides an advantage over previous research projects considering only a limited number of heavy representative vehicles or by using WIM overloaded traffic database, which does not confirm a permit purchase. The presented fatigue approach sets an amount of damage caused by permit vehicles and compares it to the AASHTO fatigue truck. This allows for a calculation of the consumption ratio, which is a sensible estimation of damage caused by the operation of overloaded permitted vehicles. The assessment of fatigue damage by setting the strain gauge on representative structures and checking stresses

caused by different types of permit vehicles is not possible. Moreover, FEM analyses of stresses caused by the permit vehicles can be numerically intense and inefficient.

The proposed methodology is useful in assessing damage for a broad spectrum of state-specific permit vehicles using the available permit database. The methodology proposed in this study is not intended to be an exact analysis of the consumption of the remaining life until structural failure. However, the goal is to provide a simple approach that captures the relative amounts of the consumption or deterioration of the AASHTO bridge design life caused by a wide range of overloaded permit vehicles.

2.4. RESEARCH APPROACH

The presented monetary consumption approach can be utilized to calculate dollar damage for each permit vehicle in the state-specific database. Such analysis can serve as a basis to change the permit fee structure. The consumption analysis requires determining state-specific representative parameters. It includes bridge construction cost, average daily truck traffic per lane, maximum bridge span length, and bridge material and structural types. The monetary consumption can be assessed as follows:

1. Determine representative bridge parameters.
2. Find a representative legal truck for state-specific traffic.
3. Run permit vehicles over an influence line to obtain a bending moment history at midspan. Use a rainflow counting to determine the number and magnitude of load cycles from the bending moment history and calculate the single cycle equivalent moment.

4. Determine the equivalent single moments for the AASHTO fatigue truck and the selected legal vehicle. Calculate the incremental consumption for each permit vehicle in the database.
5. Determine a monetary bridge consumption.
6. Propose a new fee schedule for the considered permit types.

2.4.1. Representative state-specific parameters

Representative state-specific parameters are required to calculate incremental consumption of permit vehicles. **Figure 2.2** presents the necessary database to determine the state-specific representative parameters: ADTT per single lane, maximum span length, bridge type, bridge cost per mile of the road.

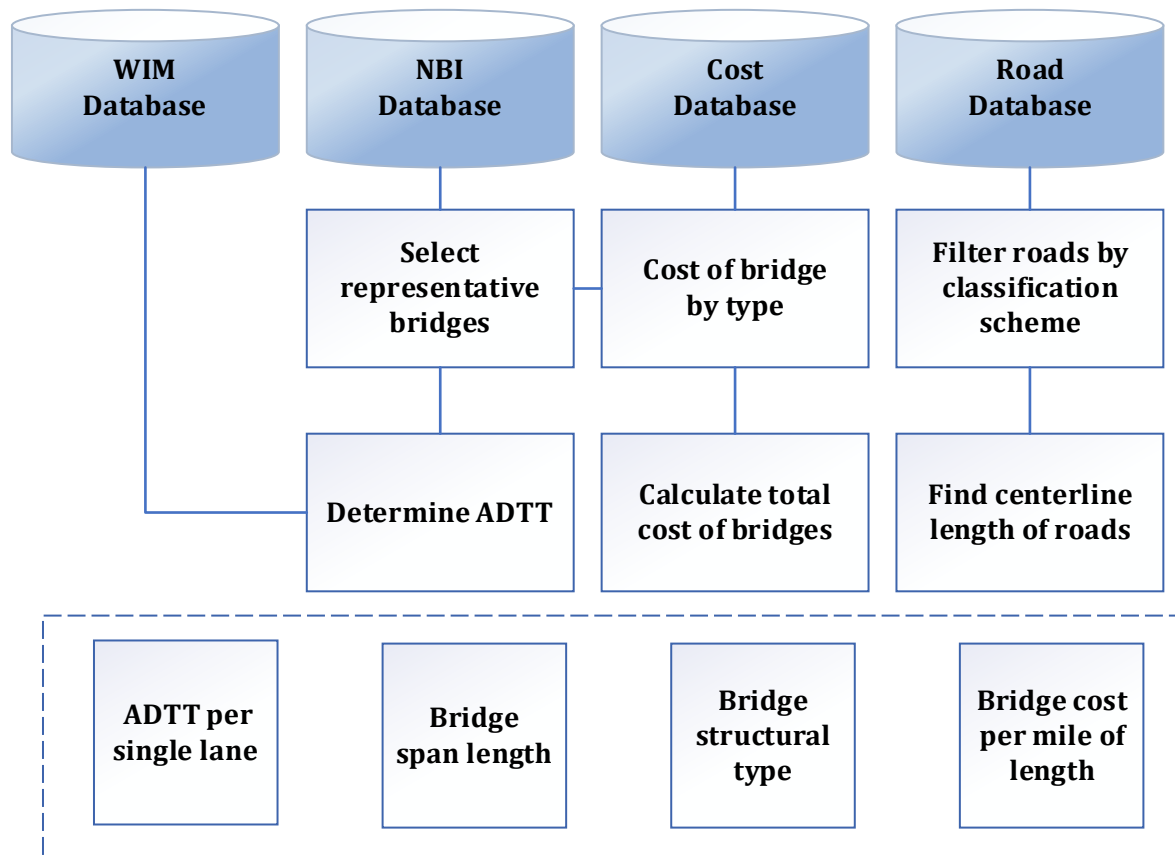


Figure 2.2. Flowchart for selection of representative state-specific parameters.

The WIM data allows finding distribution of traffic per single lane. The NBI database enables determining the representative bridge types, material, and geometry. The bridge construction cost is collected uniquely for each state and can be used depending on availability. The road data is necessary to determine the total length of roads.

The initial step is to find routes and bridges used by heavy permit vehicles. For Florida, the NBI database was used to determine the number of bridges in each route class. Based on that and an average ADTT per single lane, it was concluded that most of the bridges are located in the principal arterials with the highest average annual daily traffic and hence the percentage of truck traffic. **Table 2.2.** summarizes the number of bridges and average daily truck traffic in each inventory route. Therefore, only bridges on principal arterials are

Table 2.2. Bridges in Florida based on the functional classification of the inventory route.

Functional Classification of Inventory Route	No. of Bridges	% of bridges	Average ADTT
Principal Arterial – Interstate	1,675	17%	4,346
Principal Arterial – Other Freeways or Expressways	1,969	20%	2,263
Other Principal Arterial	1,129	11%	1,388
Minor Arterial	1,286	13%	771
Major Collector	453	5%	160
Minor Collector	1,243	12%	420
Local	2,215	22%	225

2.4.2. Bridge construction cost

To find the bridge construction cost per mile, the length of principal arterials is required. The Florida Transportation Trends and Conditions report (FDOT 2013) documented total centerline length for principal routes as 8,532 miles, with 1,496 miles on the interstates, 747 miles on turnpikes and freeways and 6289 miles on other principal arterials.

Bridge construction costs vary from state to state, and they can be affected by many factors. In 2014, Florida DOT released the transportation cost report with a construction cost of a new bridge per square foot, for short-span, 6 to 13.7 m (20 to 45 feet), medium-span, 13.7 to 45.7 m (45 to 150 feet), and long-span bridges, over 45.7 m (150 feet). The costs are grouped by material and structural type of bridges. The cost per square foot is given as a low and high price. The bridge construction cost was estimated based on the FDOT experience and the contract/bid database.

The average construction unit costs were used along with the NBI database to find the dollar amount for a representative bridge in Florida. The NBI data for 2018 was used to identify bridges by inventory route, span length, structural type, and material to match with cost categories listed by FDOT (**Table 2.3**).

Table 2.3. New bridge construction cost by FDOT (FDOT 2014)

Bridge Type	Low Cost [\$ /sq. ft]	High Cost [\$ /sq. ft]
Short Span Bridges		
Reinforced Concrete Flat Slab Simple Span	115	160
Pre-cast Concrete Simple Span	110	200
Reinforced Concrete Flat Slab Continuous Span	-	-
Medium and Long Span Bridges		
Concrete Deck / Steel Girder – Simple Span	125	142
Concrete Deck / Steel Girder – Continuous Span	135	170
Concrete Deck / Pre-stressed Girder – Simple Span	90	145
Concrete Deck / Pre-stressed Girder – Continuous Span	95	211
Concrete Deck / Steel Box Girder – Span Range from 150-280 in	140	180
Segmental Concrete Box Girders – Span Range 150-280 in	140	160
Movable Bridge – Bascule Span and Piers	1800	200
*Increase the cost by 20% for phased construction		

The total number of considered bridges is 4,773. These bridges are called the *bridge sample database* and are used to determine the cost per mile of principal arterials. Each bridge's average cost can be calculated using the bridge sample database and corresponding asset values in dollars per square foot. The average area of the bridge is computed using deck width and the total length of the bridge. Bridge cost calculation needs to be adjusted to the centerline length of the roads.

In NBI, direction of traffic for bridges is given, so the bridges can be divided into non-parallel bridges, which carry traffic in two directions, and parallel bridges that carry traffic in one direction, with separately considered right and left sides. The construction cost was computed for each group. Bridges on principal arterials that were not assigned to any cost category are referred to like others. The dollar asset value of “Others” bridges was calculated as the weighted average of known bridge asset values. The total calculated costs of representative bridges in Florida in dollars are presented in **Table 2.4**.

The total construction cost was calculated for the considered bridge sample data. **Table 2.5** summarizes the results of computations. The total bridge construction cost is presented in the last column, where non-parallel bridges, which carry traffic in two directions, are taken as half of the price since the centerline length of the road is considered. The weighted average based on the bridge area is considered with traffic in two directions for parallel bridges.

For Florida, the bridge construction cost is over \$9 billion, which gives the average construction cost per mile of road approximately \$1M/mile. For the permit vehicle operation, this value can be considered as a representative bridge construction cost of bridges per mile of principal arterials.

Table 2.4. Total cost of selected bridge structures in Florida

Bridge Type	No. of Bridges	Asset Value [\$ /ft2]	Average Area [ft2]	Construction Cost [\$]
Non-parallel (two direction traffic) Bridges (Group A)				
1	2	3	4	2*3*4
RC Slab, Simple Span	90	165	7,180	106,500,000
Precast Slab	199	186	12,420	459,674,600
Steel Girder, Simple Span	92	160	29,860	439,538,300
Steel Girder, Continuous Span	151	183	52,200	1,442,241,200
P/C Girder, Simple Span	773	118	30,280	2,761,432,200
P/C Girder, Continuous Span	25	153	141,550	541,429,900
Steel Box	82	160	41,590	545,588,200
Box Girder	15	150	28,220	63,494,000
Others	269	143	54,180	2,084,121,500
Total	1,696	-	397,460	8,444,019,500
Parallel right (one direction traffic) Bridges (Group B)				
RC Slab, Simple Span	56	165	5,700	52,635,400
Precast Slab	181	186	8,060	271,135,900
Steel Girder, Simple Span	91	160	33,340	485,398,100
Steel Girder, Continuous Span	53	183	71,590	694,280,900
P/C Girder, Simple Span	973	118	21,410	2,457,432,900
P/C Girder, Continuous Span	18	153	133,620	367,977,900
Steel Box	13	160	60,820	126,503,800
Box Girder	18	150	14,490	39,101,000
Others	130	143	19,300	358,683,300
Total	1,533	-	368,290	4,853,148,900
Parallel left (one direction traffic) Bridges (Group C)				
RC Slab, Simple Span	48	165	6,250	49,425,700
Precast Slab	200	186	7,590	282,312,700
Steel Girder, Simple Span	86	160	35,600	489,747,400
Steel Girder, Continuous Span	42	183	65,620	504,337,000
P/C Girder, Simple Span	1,001	118	19,680	2,323,928,900
P/C Girder, Continuous Span	20	153	186,350	570,207,400
Steel Box	13	160	59,990	124,762,100
Box Girder	12	150	15,160	27,288,500
Others	122	143	21,310	371,654,500
Total	1,544	-	417,510	4,743,663,900

Table 2.5. Construction cost of bridge sample per direction.

Type	Construction Cost [\$] Group A	Construction Cost [\$] Group B	Average Area B [ft ²]	Construction Cost [\$] Group C	Average Area C [ft ²]	Construction Cost [\$] $\frac{0.5*2+(3*4+5*6)}{(4+6)}$
1	2	3	4	5	6	
RC Slab, Simp. Span	\$106,500,000	\$52,635,400	5,700	\$49,425,700	6,250	\$104,207,400
Precast Slab	\$459,674,600	\$271,135,900	8,060	\$282,312,700	7,590	\$506,395,600
Steel Girder, Simp. Span	\$439,538,300	\$485,398,100	33,340	\$489,747,400	35,600	\$707,412,900
Steel Girder, Cont. Span	\$1,442,241,200	\$694,280,900	71,590	\$504,337,000	65,620	\$1,324,558,600
P/C Girder, Simp. Span	\$2,761,432,200	\$2,457,432,900	21,410	\$2,323,928,900	19,680	\$3,774,206,500
P/C Girder, Cont. Span	\$541,429,900	\$367,977,900	133,620	\$570,207,400	186,350	\$756,470,500
Steel Box	\$545,588,200	\$126,503,800	60,820	\$124,762,100	59,990	\$398,433,100
Box Girder	\$63,494,000	\$39,101,000	14,490	\$27,288,500	15,160	\$64,806,600
Others	\$2,084,121,500	\$358,683,300	19,300	\$371,654,500	21,310	\$1,407,550,500
Total	\$8,444,019,500	\$4,853,148,900	368,290	\$4,743,663,900	417,510	\$9,044,041,400

2.4.3. Average Daily Truck Traffic

The ADTT for each bridge is given in the NBI database. The ADTT was measured for different years between 1992 and 2018, and, if needed, for 2018, it was extrapolated. The average daily truck traffic in the NBI database is provided for the cross-section of the road. Weighted average ADTT was calculated for two-direction bridges and then divided by two to get the ADTT per one direction. The weighted average ADTT per direction for principal roads and bridges equals 1394 vehicles/day. The results of the analysis are shown in **Table 2.6**.

The WIM data can be used to assess the actual percentage of truck traffic per one lane. The traffic distribution per one lane in Florida was determined using data from 33 WIM stations and records from 2012 and 2014-2017. WIM sites record information about

occupied lanes by measured vehicles. Each WIM station records traffic for a different number of lanes. Based on the NBI database, the average number of lanes in a bridge is three; therefore, the traffic distribution for all WIM stations was considered only for three lanes.

Table 2.6. Weighted average ADTT for a representative bridge in Florida.

Type	No. of Bridges	Average ADTT
1	2	3
RC Slab, Simple Span	194	1,806
Precast Slab	580	1,742
Steel Girder, Simple Span	269	3,595
Steel Girder, Continuous Span	246	2,081
P/C Girder, Simple Span	2,747	3,337
P/C Girder, Continuous Span	63	1,670
Steel Box	108	1,274
Box Girder	45	2,839
Others	521	1,778
Weighted Average	-	2,787
ADTT per direction	-	1,394

The fraction of the truck traffic for the most occupied lane was found as 67%. For consistency of the results, only WIM stations with three lanes were considered, and the fraction of the truck traffic was 66%. Finally, based on the WIM database analysis, the percentage of truck traffic in one-way traffic per one lane is assumed conservatively as 70%. Thus, the ADTT per single lane for a representative bridge can be obtained, and in Florida, it was taken as 1000 trucks per day.

2.4.4. Bridge type and maximum span length

The maximum span length for representative bridges is summarized in **Table 2.7**. The weighted average maximum span length is taken as 27.1 m (90 ft). The most common bridge type in Florida is a prestressed concrete (P/C) structure. They constitute about 60% of all bridges. The consumption equation includes the fatigue exponent m that is dependent

on material and structural types as shown in **Table 2.1**. In the consumption analysis, the value of m for Florida is taken as 3.50.

Table 2.7. Weighted average span length of a representative bridge in Florida.

Type	No. of Bridges	Average Max Span [m (ft)]
1	2	3
RC Slab, Simple Span	194	8.8 (29)
Precast Slab	580	10.1 (33)
Steel Girder, Simple Span	269	45.4 (149)
Steel Girder, Continuous Span	246	54.2 (178)
P/C Girder, Simple Span	2747	27.1 (89)
P/C Girder, Continuous Span	63	39.3 (129)
Steel Box	108	64.9 (213)
Box Girder	45	37.2 (122)
Others	521	23.2 (76)
Weighted Average		27.1 (89)

2.4.5. Typical legal vehicle

A typical legal vehicle was determined using the WIM data to calculate the incremental bridge consumption. First, the distribution of vehicle classes had to be verified for selected WIM sites. Based on WIM data from FHWA, class 9 vehicles, 5-axle semi-trailer, are the most common trucks. Florida 2012 and 2014-2017 WIM data was used, including over 150 million records, where vehicle class 9 is the majority, and they are approximately 57% of all WIM records. A typical legal vehicle (class 9) is then filtered based on GVW at a federal limit of 36.3 tones (80 kips) with +/- 5% tolerance. For the selected typical legal vehicle (**Figure 2.3.**), an equivalent single moment is calculated.

The mean equivalent moment for the typical class 9 vehicle in Florida is equal to 1397 kNm (1030 kip-ft). In the analysis, the 95th percentile of the equivalent moment was chosen as a representative legal vehicle. The equivalent single moment is 1532 kNm (1130 kip-ft) for a mid-span simple span bridge analysis. A typical legal vehicle load effect is

used to determine legal consumption defined in **Equation (2.11)** and then assess the incremental permit vehicle consumption. This approach allows for calculating a monetary damage of permit vehicles attributed only to the overloaded part (above the legal limit). Such an approach ensures the fairness of the charged permit fee.

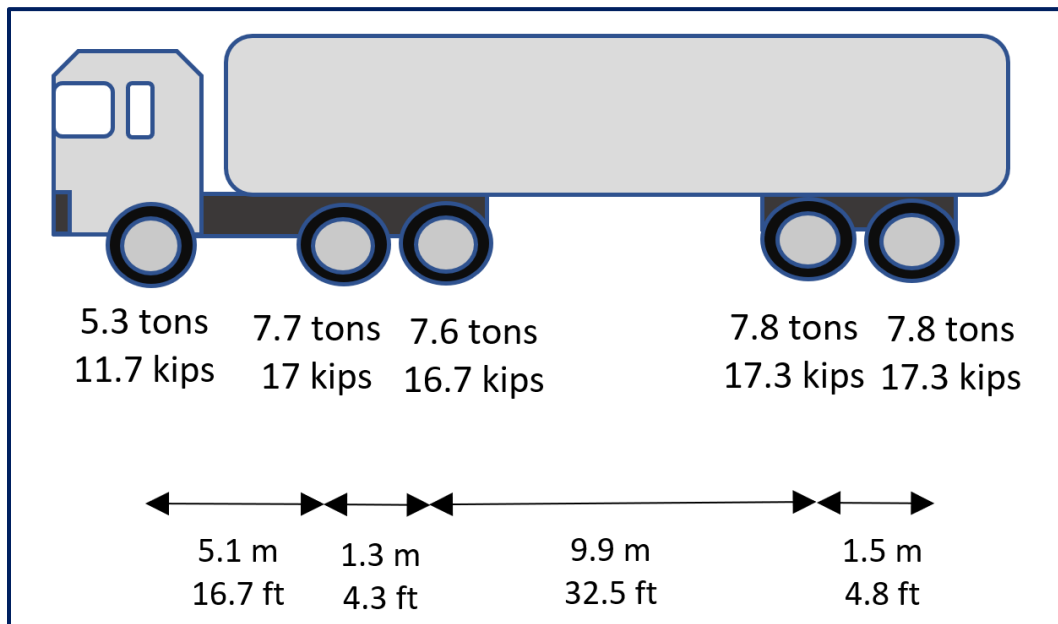


Figure 2.3. Typical legal class 9 vehicle configuration in Florida.

2.5. RESULTS AND DISCUSSION

The described fatigue-based approach can be used to determine the monetary consumption caused by permit vehicles. The incremental consumption computations are demonstrated for Florida, using the representative state-specific parameters. The relative dollar amount attributed to various types of permits is computed using Florida bridge construction cost in dollar per mile, ADTT per single lane, the maximum span length, and the typical legal vehicle damage, along with equivalent single moment effects caused by over 350 thousand permit vehicles.

2.5.1. Consumption for single trip permits

The damage caused by permit vehicles is evaluated based on the single equivalent moment caused by a permit truck, AASHTO fatigue truck, and a typical legal state-specific truck. The MATLAB code was developed to calculate the bending moment at midspan due to a truck crossing bridge on a simply supported beam. The selected bridge span length in Florida is 27.4 m (90 ft), and material constant m is taken as 3.50. The bending moment computation is used to find the equivalent single moment range due to a permit truck using the rainflow count analysis method ASTM E1049-85 (American Society for Testing and Materials, 2017).

An example calculation of equivalent single moment is presented for a permit vehicle with the GVW of 57.7 tons (120.7 kips), distributed to 6 axles, and a total vehicle length of 30.5m (100ft). **Figure 2.4** shows the rainflow histogram created by MATLAB.

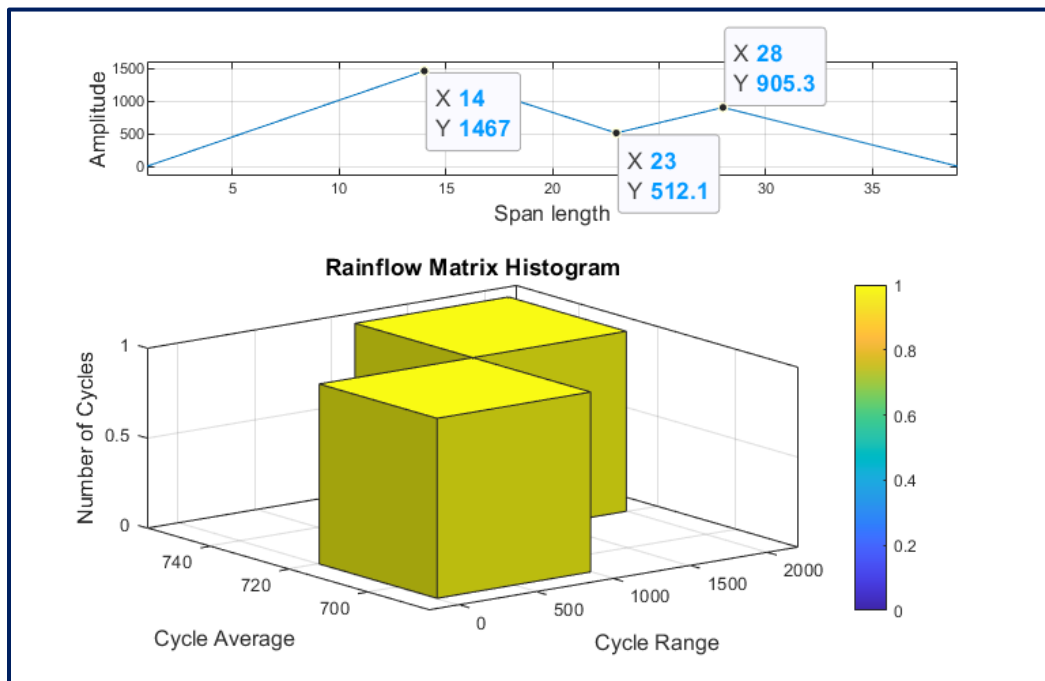


Figure 2.4. Rainflow histogram for equivalent moment made by MATLAB (in kip-ft).

Sample calculation of equivalent single moment range **Equations (2.4)**, using reinflew histogram shown in **Figure 2.4**. is presented by **Equation (2.12)**:

$$M_{eq} = [1467^{3.5} + (905.3 - 512.1)^{3.5}]^{\frac{1}{3.5}} = \quad (2.12)$$

$$= 1471 \text{ kip} - \text{ft} \rightarrow 1994 \text{ kNm}$$

The equivalent single moment for the considered permit truck is equal to 1994 kNm (1471 kip-ft). The permit vehicle creates two cycles, in which the first one creates a moment of 1989 kNm (1467 kip-ft) and the second cycle moment is equal to 533 kNm (393.2 kip-ft). The equivalent single moment range depends on the number of stress cycles caused by the vehicles crossing the bridge and can be different than the effective moment for the number of cycles different than one.

In comparison, the effective moment calculated using **Equation (2.12)**, which is equal to 1637 kNm (1207 kip-ft), and the maximum bending moment at midspan is equal to 1989 kNm (1467 kip-ft). Using the same algorithm, the equivalent single moment was calculated at midspan for the AASHTO fatigue truck, for the representative span length of 27.4 m (90 ft), and it is equal to 1171 kNm (864 kip-ft). The typical legal vehicle selected from an extensive WIM database caused the moment of 1532 kNm (1130 kip-ft).

For every permit vehicle, the incremental monetary consumption can be found using state-specific parameters, such as cost \$1M/mile and ADTT per single lane 1000 trucks/day. For a demonstration, ten random permit vehicles with varying GVW, total length, and a number of axles present the magnitudes of an equivalent single moment and, most importantly, the incremental consumption in dollars per mile. **Table 2.8.** shows magnitudes of the equivalent moments and incremental consumptions for selected permit

vehicles in Florida. The incremental consumption varies from \$0.06/mile to \$2.64/mile. That proves that GVW is not the most appropriate damage parameter that should determine the permit fee. The bridge fatigue consumption depends on a combination of axle loads, a number of axles, and spacings between them.

Table 2.8. Equivalent moment and incremental consumption for selected permit vehicles

No.	GVW [ton (kip)]	Total Length [m (ft)]	Number of Axles	Equivalent Moment [kNm (kip-ft)]	Consumption Ratio	Incremental Consumption [\$/mile]
1	39.9 (88)	17.7 (58)	5	1779 (1312)	4.3	0.06
2	54.8 (120.7)	30.5 (100)	6	1994 (1471)	6.4	0.14
3	58.9 (130)	22.9 (75)	7	1900 (1402)	5.4	0.11
4	59.8 (132)	28.9 (95)	7	1886 (1391)	5.3	0.10
5	63.5 (140)	22.9 (75)	8	2245 (1656)	9.7	0.26
6	75.5 (166.4)	22.9 (75)	8	3372 (2487)	40.5	1.38
7	84.8 (187)	29.4 (96.6)	9	2682 (1978)	18.1	0.57
8	105 (231.5)	24.7 (81)	13	4020 (2965)	74.9	2.64
9	108.9 (240)	36 (118)	12	2988 (2204)	26.5	0.88
10	113.8(251)	37.8 (124)	12	3041 (2243)	28.2	0.94

2.5.2. Consumption for multi trip permits

The proposed methodology can also be used to assess the damage caused by multiple trip permits. The incremental consumption **Equation (2.10)** provides the results in \$/mile, but the multiple trip permits are commonly charged by the fixed price. To determine the fixed fee, the number and length of the trip are required. The permit vehicle operation survey can be done to find the estimated number of permit trips. Trucking companies or individual permit haulers need to be asked to record the number of permit vehicle trips in the agreed time period.

The state of Ohio conducted such a survey (Ohio DOT 2009), and it was estimated that the annual number of multiple trip permits is 24.8 with an average trip length

is 98.8 miles. These parameters are state-specific, and they should be assessed individually. However, the average permit trip length can be determined by using a permit database and GIS database. Florida Permit Application System (<https://pas.fdot.gov>) includes an assigned origin and destination information for every issued permit. Using origin and destination coordinates along with Geographic Information System (GIS), possible routes can be determined. The special algorithm was developed to find the possible routes for permits in Florida and calculate the trips' length. The calculations were conducted for all types of permits in Florida, including single and multiple permits. In conclusion, an assessment of the average length of the trip for multiple permits in Florida provides an estimated length of 135 miles. For a single trip permit, the average trip length is approximately 190 miles. The average length of the multiple trip permits can then be used in the monetary consumption calculations.

For instance, if the permit vehicles from **Table 2.8** are considered as multiple trips, then the incremental consumption in \$/mile can be converted to the fixed price by multiplying the number of trips and trip length. In Florida, using the average trip length of 135 miles, and for the annual multiple trip permit assuming 25 trips per year as shown in the Ohio study, the annual multiple trip cost for selected permit vehicles would vary from \$215 up to almost \$9k. A specific discount is often assigned to multiple trip permits to allow for flexibility in freight transport operations and economy growth, depending on state policy.

2.6.IMPLEMENTATION

The proposed methodology allows for the computation of the monetary consumption for a significant number of permit vehicles. It provides a rational basis to update the state-specific permit fee structure. The approach is not intended to be an exact

analysis of the consumption of the remaining life until structural failure. In fact, not many bridges experience fatigue failures that cause closures or collapse. Although, the goal is to provide a simple methodology that captures the relative amounts of the consumption or deterioration of the AASHTO bridge design life caused by a wide range of overloaded permit vehicles. A fundamental assumption of the proposed method is that the life of the bridge is defined by 75 years of crossings by the AASHTO fatigue design truck.

Individual monetary consumption can be assessed for every permit vehicle. Every permit fee price can be calculated and charged to the individual permit applicant using the computerized permit applications system. In the United States, the tabularized permit fee structures are commonly used instead of vehicle and route-specific fees. The proposed consumption approach allows for flexibility and fairness in the implementation process by computing the dollar consumption caused by a large group of permit vehicles.

Individual permit dollar consumptions can be used to determine the new permit fee structure. The permit vehicles can be grouped by different parameters such as GVW, number of axles, etc. The permit structure is adaptable and depends on state-specific preferences and a transportation policy. By using the permit vehicle state-specific selection criteria, the permit fees for single and multiple trips can be determined to adequately represent the damage caused by groups of issued permits. The most common permit fee structure is GVW-based. Tabularized prices can be grouped by specific GVW ranges, as it is currently in Florida and many other states. However, the permit database and fatigue damage analysis showed that the bridge damage is not strongly correlated with the GVW. The number of axles, axle spacings, and axle weight distribution throughout the vehicles is impacting the damage to bridges. Therefore, it would be recommended to consider these

factors for fairness in the case of tabularized permit fee structure.

The goal is to create a permit fee system that effectively charges overloaded vehicles for the damage caused to the infrastructure. This approach enhances the technology using the available permit databases. It provides a methodology that captures the relative amounts of AASHTO bridge design life consumption caused by a wide range of overloaded permit vehicles. The method has its limitations as it does not consider the selected route-specific costs, but it is the weighted average value for the state's representative bridges. Also, it does not consider the number of bridges crossed since it is not route specific.

The consumption-based approach can be expanded using more advanced analysis. Individual permit fees can be calculated based on a vehicle configuration and a permit route. The individual consumption of the permit vehicles can be computed by the presented consumption formula and assess a cost for a specific route. The selected route has a specified number of bridges, which allows determining a total permit fee. The GIS could offer the flexibility for the trucking industry to manage the permit fee cost, by suggesting optimal routes. The GIS-based permit fee schedule may be a great solution for a fair permit fee structure based on the actual damage caused to the roads and bridges. Also, the advanced GIS technology could allow for monitoring bridge and pavement conditions depending on the number and weight of issued permits crossing the bridges within the state.

2.7.CONCLUSIONS

A methodology was developed to assess the proportion of the bridge design life that is consumed by permit vehicles. To quantify the relative amounts of the design life consumed by vehicles of different weights and configurations, a bridge fatigue analysis serves as the framework of the method. The bridge life is defined by 75 years of crossings of

the AASHTO fatigue design truck. Therefore, the monetary loss is represented by the cost of new bridge construction. Each passage of a permit vehicle adds to the accumulated fatigue damage. The fundamental part of the proposed approach is an incremental consumption formula. The consumption is presented in terms of the equivalent single moment caused by permit trucks. The incremental fatigue damage is calculated only for the overload share of a permit vehicle. Monetary consumption can be calculated for a wide range of permit vehicles, and this provides a basis for a fair and equitable permit fee structure.

The procedure can be applied by any state using the available state-specific database. The incremental consumption cost can be assessed for any vehicle based on axle loads and axle spacings. The permit fee structure can be tabularized according to the DOT preferences by GVW, a number of axles, permit type, etc. The proposed approach provides flexibility for the development of the permit fee schedule. It can also be adjusted to meet the state-specific expectations. The proposed damage assessment methodology is relatively simple. It uses available databases, and it does not require a high budget. NBI data can be used to select state-specific representative bridge parameters, information on the new construction cost, and the WIM records to find typical legal vehicle configuration and percentage of heavy traffic in one lane, and a road and permit database for all other required parameters.

The proposed methodology is an efficient tool to assess the fatigue and dollar damage caused by the permit vehicles operating in the state. It gives a rational basis to develop fair and justifiable permit fees.

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CHAPTER 3 DEVELOPMENT OF NOTIONAL PERMIT VEHICLES ²

ABSTRACT

Bridge live load is specified in the form of notional vehicle(s) that are expected to represent the traffic-induced load spectra. The permit traffic is also represented by notional permit vehicle(s), which are state-specific. The study objective is to propose a reliability-based method to determine notional permit vehicle(s) based on issued permits and weigh-in-motion (WIM) data. Notional permit vehicles are selected so that the corresponding reliability index is close to the target value. The proposed approach is demonstrated for Florida traffic. Over 300,000 candidates for notional permit vehicles were generated and considered in the analysis. It led to the selection of one notional permit vehicle with 8 axles and gross vehicle weight (GVW) of 94.8 tons (209 kips), and a length of 33.83 m (111 ft). The selected notional permit vehicle can serve as a basis for verification of the design provisions for the Strength Limit State II and rating of permit vehicles. It can also simplify the permit issuance procedure and help to control permit vehicle operations.

² under review in Reliability Engineering and System Safety Journal.

3.1.INTRODUCTION

The number of issued permits is growing every year, and there is a need to assess their impact on bridges. For instance, in Florida, the analysis of weigh-in-motion (WIM) data showed that an annual truck traffic growth is over 3%, and an increase in the number of issued permits is about 5%. The operation of heavy trucks is justified from the transportation efficiency point of view, but excessive weight and the number of permit vehicles can result in accelerated damage to bridges. An important question is, if bridge design and evaluation provisions account for permit traffic?

AASHTO live load analysis uses notional trucks to represent the traffic-induced load spectra. There are several notional vehicles, beginning with the HL-93 design truck used to bridge design and evaluate existing bridges for inventory and operating rating. Every state uses its own unique truck(s) for permit vehicles, developed by different methods. The objective of the present study is to propose a reliability-based procedure to develop notional permit vehicles based on WIM records and an issued permit database that provides uniform safety for a wide variety of bridge types and is consistent with the AASHTO specifications. The analysis involves the derivation of statistical parameters of moment and shear caused by permit vehicles and calculation of the corresponding reliability indices for the considered notional permit vehicle candidates. The reliability-based analysis is performed for single and multi-trip permits. The procedure is demonstrated using issued permit records and WIM data in Florida.

The candidates for notional permit vehicles are considered as a combination of axle load and axle spacings. Overloaded traffic characteristics were obtained from the available traffic database. Typical heavy vehicles are grouped by the number of axles, axle

loads, and spacings. Over 300 thousand combinations of candidates for notional permit vehicles were generated and analyzed. Using influence lines, load effects due to permit vehicles were calculated for a wide range of simple span and continuous bridges. Statistical parameters of permit-caused live load were computed for every candidate for a notional permit vehicle. In the reliability-based analysis, load and resistance variables were generated using Monte Carlo simulation, and the corresponding reliability index was calculated. Based on the available data, a notional permit vehicle was developed for Florida.

The presented approach can also be applied to calibrate the Strength Limit State II design provision and bridge rating under permit vehicles. The developed notional permit vehicles and selected live load factor provide a uniform safety level for design and evaluation. Application of the developed notional permit vehicles can also help streamline the permit vehicle issuance. All special permit vehicles that produce a load effect lower than those corresponding to the notional permit vehicles would not have to be processed using a refined numerical analysis. To further simplify the effort for state DOT, the notional permit vehicles can be used to validate issued permits and provide efficiency by saving cost and time.

3.2. LITERATURE REVIEW

Further referred to as permits, the permit vehicles are legally overloaded vehicles that purchased a permit to operate on roads and bridges. Permits are site-specific thus, they are represented by various notional vehicles developed for state-specific traffic. To assure the safety of new and existing bridges under permit vehicular traffic, there is a need for a representative notional permit vehicle, which envelops the state-specific load spectra. In many instances, overloaded trucks are operating on roads and bridges that are not

designed for either the weight and/or traffic volume. Therefore, it is important to control the operation and impact of permit vehicles on road infrastructure.

Many researchers analyzed the impact of overloaded vehicles since the increase in the numbers of permit and illegally overloaded trucks traveling on U.S. highways has raised concerns. Therefore, the heavy traffic flow and its load effects were studied by (Fiorillo and Ghosn 2014). The study developed a robust algorithm, which can assist in establishing permit issuance policies. The effect of heavy vehicles on bridges was studied in a heavy-duty corridor between Indiana and Michigan. The load spectra were checked for selected representative bridges to verify the bridge response to that traffic (Chotickai and Bowman, 2005; Reisert and Bowman, 2005). Michigan DOT requested a study to create a more robust system, which ensures the safety and control of the operation of overloaded vehicles. It was aimed to assess the impact of vehicles requesting a permit to more efficiently using the computing capacity to reduce the number of manual bridge structural analyses (Barker 2013). Another study utilized 5% of the heaviest WIM data to validate the adequacy of the notional permit vehicles in Wisconsin. It was concluded that 5-axle, short trucks might cause greater load effects in bridge girders than standard permit vehicles (Zhao and Tabatabai 2012). Several state DOTs sponsored studies to update the policy for overloaded vehicles to maintain the safety of bridges (Chowdhury et al. 2013b; Culmo et al. 2004; Ghosn et al. 2015; Nassif et al. 2015; Ohio Department of Transportation 2009; Prozzi et al. 2012; Wu et al. 2019).

AASHTO live load analysis requires a notional vehicle that, along with the calibrated single live load factor, provides a required minimum safety defined as a target reliability index. There are several notional vehicles, such as design truck HL-93 used for

new bridge design and existing bridge evaluation for inventory and operating rating. Existing bridges are also checked for legal loads that represent vehicles that do not exceed Federal Bridge Formula. The legal loading trucks are Type 3, Type 3S2, and Type 3-3 to account for short, medium, and long-span bridges. For the legal loads, it was noticed that some specialized hauling vehicles meet Federal Bridge Formula requirements but cause excessive load effects on bridges. In 2007, Sivakumar et al. developed notional vehicles that better represent legal loading for bridge evaluation. The notional vehicle development was based on Weigh-in-Motion data and state DOT survey to capture unusual vehicle configurations that meet legal requirements but cause overstress in bridges. Single and multi-unit trucks were identified as legal load candidates for bridge rating. Notional vehicles that imitate specialized hauling vehicles were proposed along with live load factors required for bridge evaluation procedures SU4, SU5, SU6, and SU7 (Sivakumar et al. 2007).

Existing bridges are also checked for permit loads, but the Manual Bridge Evaluation (MBE) does not specify notional permit vehicles. So, every state uses its own unique truck(s) with the manual specific live load factor provided for single and multi-trip permits.

Notional permit vehicle(s) can be used in the permit issuance process. A special analysis of a superload vehicle was conducted with a Gross Vehicle Weight (GVW) of 715 tons (1576 kips), traveling from California to Utah over 50 bridges. This superload analysis was time-consuming and costly; therefore, the alternative method based on screening vehicles was used. It was shown that the superload vehicle could safely operate if its axle weights and spacings fall within any of the proposed screening vehicles (without considering types, capacities, or span configurations of the bridges that a permit truck will

actually pass on its route). Otherwise, a vehicle has to be further evaluated by other analytical methods. The goal was to make the permitting process easier and recommend as many as possible screening vehicles that reflect the state-specific permit vehicles (Lawson et al. 2013). In another study, the adequacy of the Tennessee formula, which determines if the permit vehicles can purchase a permit without refined numerical analysis, or it must be analytically checked to enable permit issuance (Milutinovic 2016). The formula was based on the allowable stress design, so it is inconsistent with the currently used load and resistance factor design approach. It was concluded that some vehicles pass the Tennessee formula but exceed the Load Factor Rating (LFR) or the Load and Resistance Factor Rating (LRFR) provisions. Therefore, a notional screening vehicle was developed to help to make permitting decisions. This shows that notional permit vehicles can also be used in the permit decision-making processes to help determine if a refined analysis is required.

Pennsylvania DOT's Live Load Design Permit Vehicle, P-82, and PHL-93 were checked for adequacy to represent the effects of current special hauling permit vehicles. The study developed analytical tools needed to evaluate heavy truck data, both WIM, and superloads. The objective was to propose a permit design vehicle model that envelopes all or most WIM and superload vehicles in the PennDOT databases (Laman and Shah 2016). Wyoming DOT performed a study to assess the safety of bridges on I-80, which carry a large volume of heavy trucks. The goal was to check if AASHTO LRFD provisions provide adequate safety. The reliability analysis showed that AASHTO does not provide adequate safety, and it was proposed to increase the live load factor from 1.75 to 2.0 to achieve the minimum required safety set by the target reliability index for design, 3.50. (Barker and Puckett 2016).

The presented literature review emphasizes the importance of the development of representative notional permit vehicles. The application of notional permit vehicles is in design and evaluation provisions and also permit issuance policies.

3.3. TRAFFIC DATA

The traffic data is essential to evaluate load effects on bridge structures. The major database is Weigh-in-Motion (WIM). It is not only an excellent source of information, but it is widely available all over the United States. The WIM sites continuously collect massive traffic data, which includes the weight and configuration of passing vehicles. The WIM data contains (1) legal vehicles which do not exceed state and federal provisions; (2) illegal vehicles exceeding the weight, size, or weight and size limits; and (3) permitted vehicles, which can legally exceed the legal limits after purchasing the permit. To determine the impact of permit vehicles on bridge structures, it is necessary to filter these vehicles from WIM traffic flow. A filtering procedure can be developed, but it will not guarantee that the vehicles captured by WIM operate based on the issued permit. However, another available source is the state-specific permit data that contains information about all issued permits. Permit systems record detailed information about every issued permit based on submitted applications. The permit data includes the axle weights and spacings, which are crucial to determine the load effects. The permit data include unmeasured values provided by the permit applicant. The trucking company declares axle spacings and the expected axle load distribution of a vehicle.

In this study, Florida permit and overloaded WIM databases are used to present the reliability-based methodology. **Table 3.1** presents a summary of utilized traffic databases. There are three sources of data: raw WIM data from Florida DOT, years 2012-

2017, Federal Highway Administration (FHWA) Florida WIM, years 2016-2019, and Florida permit database, years 2016-2019 (with no data for December 2019). The available permit data contain oversize (overlength, overwidth, overweight, or any combination of the three) and overweight vehicles, and the combination of overweight and oversize vehicles. In this analysis, only overweight permit data is used to assess load spectra of permit vehicles operating on Florida bridges and roads.

Table 3.1. Summary of available Florida traffic data.

Years	Raw FDOT WIM	FHWA, FL WIM	FL Permit
2012	40,515,575	-	-
2013	43,633,337	-	-
2014	42,004,700	-	-
2015	44,384,906	-	-
2016	43,282,959	33,677,086	88,390
2017	41,625,652	29,069,820	91,293
2018	-	29,301,658	95,257
2019	-	39,168,609	94,023
Total	255,447,129	131,217,173	368,963

To process and use WIM data, the quality of the dataset needs to be checked by a special Quality Control (QC) procedure. The WIM checks aim to eliminate erroneous records. It was noticed that FHWA WIM data for Florida was pre-filtered, but the filtering procedure is unknown. The analyzed data shows that the maximum Gross Vehicle Weight (GVW) recorded is 72.6 tones (160 kips). It may be because all heavier trucks were considered outliers and were discarded from this dataset. Therefore, it was decided not to consider FHWA data for the analysis. The QC was applied to FDOT WIM data, and approximately 15% of WIM data was rejected.

The analysis of the WIM data is summarized in **Table 3.2**. It presents the number of WIM records before and after QC and the number of filtered overloaded vehicles.

To determine WIM overloaded vehicle load effects, the WIM data were filtered to select only vehicles that exceed the federal law, including four criteria: maximum GVW of 36.3 tones (80 kips), maximum single axle weight of 9.1 tones (20 kips), maximum tandem axle weight of 15.4 tones (34 kips), and Federal Bridge Formula (FBF). The algorithm was developed to capture vehicles that exceed at least one of the federal law criteria. These records were considered as overloaded, meaning that they require a DOT's permit to legally operate within the state. The analysis shows that approximately 10-12% of the vehicles recorded by Florida WIM stations exceed at least one of the truck size and weight federal law criteria.

Table 3.2. Overloaded FDOT WIM traffic data summary.

FDOT WIM records	2012	2013	2014	2015	2016	2017
Total WIM	40,515,575	43,633,337	42,004,700	44,384,906	43,282,959	41,625,652
Overloaded WIM	4,951,194	4,866,929	4,686,438	4,620,372	5,038,266	5,068,402
Overloaded WIM [%]	12.2%	11.2%	11.2%	10.4%	11.6%	12.2%
Total QC WIM	35,109,586	37,045,048	36,055,609	36,861,700	35,614,773	33,953,884
QC, Overloaded WIM	4,875,314	4,774,376	4,573,573	4,526,731	4,937,816	4,963,449
QC, Overloaded WIM [%]	12.0%	10.9%	10.9%	10.2%	11.4%	11.9%

Additional analysis was completed to find the most common cause of federal law violation. The results showed that over 40% of the overloaded WIM vehicles exceed the tandem axle weight of 15.4 t (34,000 lbs). The traffic data considered in this study consists of overloaded WIM data for the years 2012-2017, with a total number of 28,651,259 available records. For the years 2016-2019, the permit data was filtered for overloaded vehicles and separately for single and multi-trip permits. The total number of multi-trip permits is 38,544 and 94,750 for single trip permits.

A wide range of the bridge span lengths was considered to validate the notional permit vehicles. Load effects were computed for simple span lengths 9, 18, 27, 36, 61 m

(30, 60, 90, 120, 200 ft) for moment and shear, and similarly for two equal continuous span lengths of 9-9, 18-18, 27-27, 36-36, 61-61 m (30-30, 60-60, 90-90, 120-120, 200-200 ft) for positive, negative moments and shear effects. Fifteen load cases of moment effects were considered, including five simple span moments, five continuous positive, and five continuous negative moments. Moreover, the analysis includes ten shear load cases for simple and continuous spans, which gives twenty-five live load results for every WIM and permit database record.

3.4. RESEARCH APPROACH

The reliability-based procedure to develop representative notional permit vehicles that provide uniform safety is shown in **Figure 3.1**. It utilizes permit and WIM databases to capture overloaded vehicles. Load effects in terms of moment and shear are calculated for a wide range of bridges, including simple and continuous spans. Collected heavy traffic data can be used to determine the characteristics of overloaded vehicles, such as the number of axles, total weight, individual axle weight, and geometry. The overloaded traffic attributes are applied to selected groups of notional permit vehicle candidates. The reliability-based analysis starts with the development of statistical parameters of permit live load using the notional permit candidates. Permit vehicle candidates with low variation of statistical parameters are selected for the reliability analysis, which involves the selection of a single live load factor and reliability index calculation for the considered permit vehicle candidates. The analysis aims to determine a notional permit candidate for single and multi trip permits, which provides the desired safety together with a single live load factor for a wide variety of bridge types.

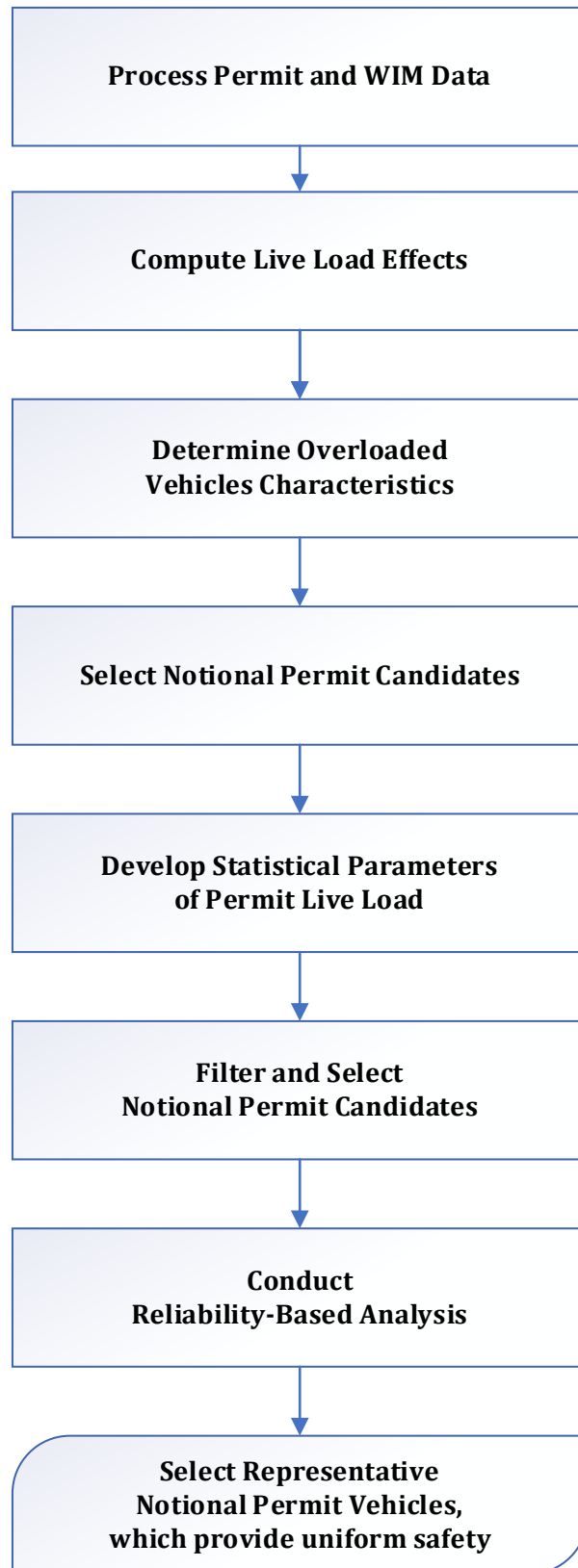


Figure 3.1. Flowchart of development of a state-specific notional permit vehicle.

3.4.1. Permit vehicles characteristics

State-specific overloaded vehicle characteristics are defined based on available WIM and permit databases. Typical vehicle characteristics such as GVW, number of axles, axle spacings are evaluated. WIM, multi, and single trip permits are considered separately since they represent different traffic attributes. In this study, vehicle characteristics histograms of GVW, number of axles, and separately for nine consecutive axle weights and eight axle spacings were examined. The analysis was performed for all WIM and permit data, assuming that more overloaded vehicle configurations considered in the analysis help to find the vehicle configuration that can be overloaded. Also, the load effect for the considered span length for simple and continuous spans was used to determine permit vehicle characteristics that cause maximum load effects. This way the critical vehicle configurations were captured. The vehicle characteristics with the largest moment and shear effects were used as a basis to determine notional permit vehicle candidates for the reliability-based analysis.

Figure 3.2 and **Figure 3.3** present histograms of GVW and a number of axles for overloaded WIM, multi trip, and single trip data. **Figure 3.2** shows the similarities of the peak GVW, 36.3-49.9 tones (80-110 kips) for WIM and multi trip permits. It was observed that the single trip permit vehicles are heavier than WIM vehicles and multi-trip permit vehicles. Single trip permit differs, what is shown in **Figure 3.3**, the number of axles is larger, so the load is distributed on more axles. It is important to note that the total weight of the vehicle is not a crucial parameter, but the load distribution on axles and the axle spacing are critical since they decide about the load effect. Based on the overloaded vehicle characteristics, including the number of axles, single and tandem axle weight, and spacings, permit vehicle candidates for Florida traffic were developed.

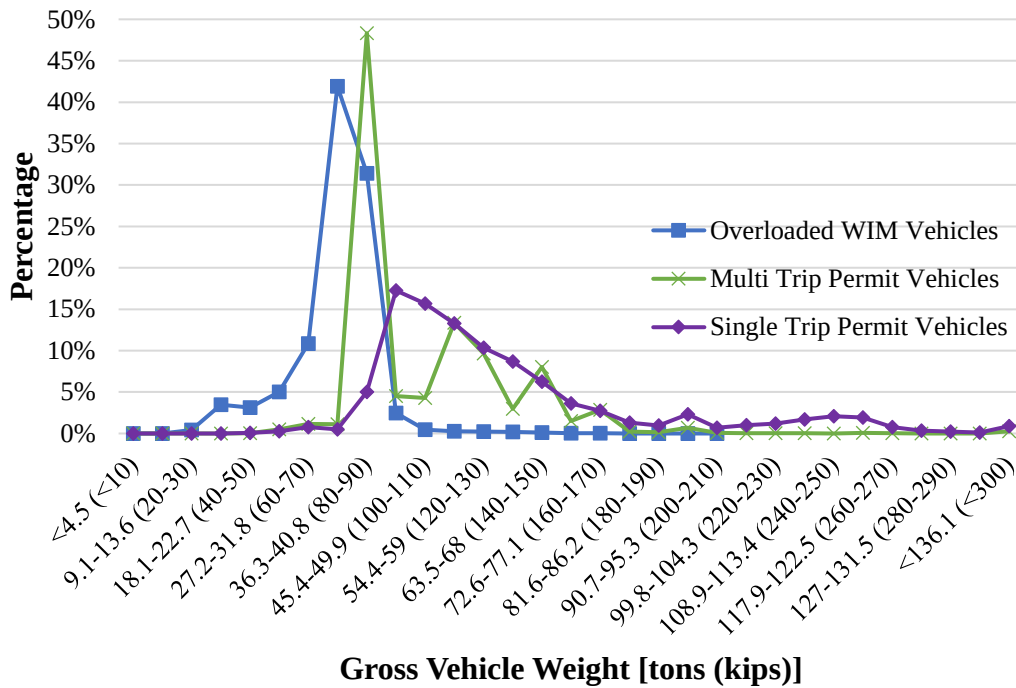


Figure 3.2. Histogram of Gross Vehicle Weight range for overload WIM, multi trip, and single trip vehicles.

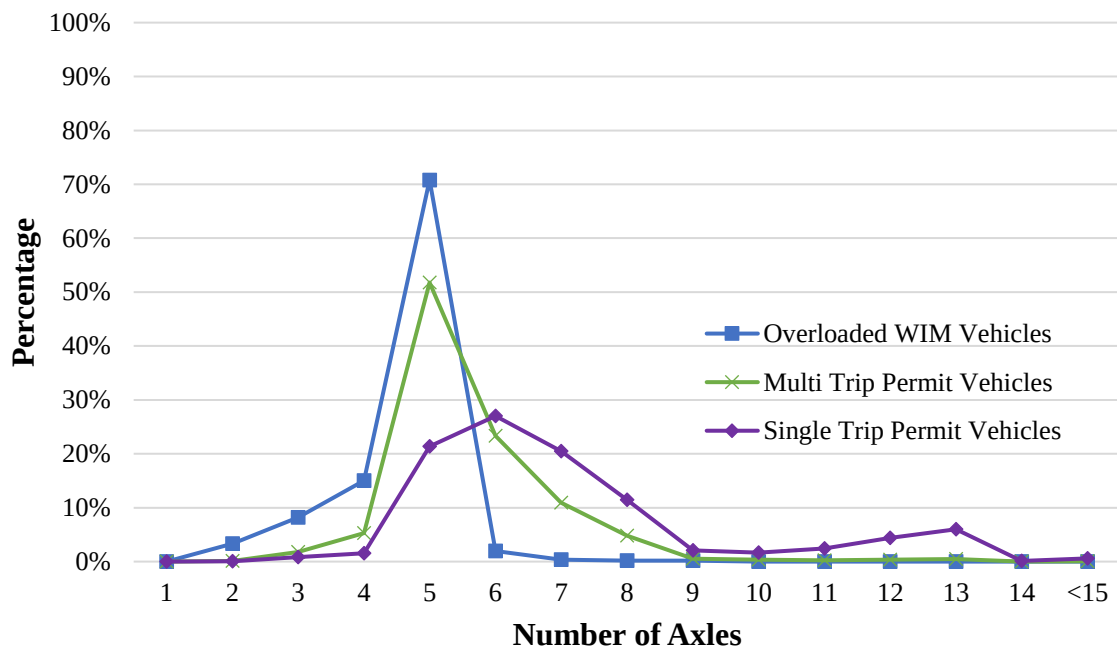


Figure 3.3. Histogram for number of axles for overloaded WIM, multi and single trip vehicles.

3.4.2. Groups of notional vehicle permit candidates

Analysis of vehicle characteristics using available overloaded traffic data allowed selecting combinations of notional permit vehicle candidates. Based on created histogram data, nine different groups of notional permit vehicle candidates were developed. A summary of the notional permit vehicle candidates is shown in **Table 3.3**. There are nine groups of candidates, where the number of axles varies from 5-13 axles. Various combinations of axle loads and spacings were considered. The axle weights are provided in metric tons, and meters, and in parenthesis in US units: (kips) and (ft).

The first axle weight is the same for all groups of candidates, which varies from 8.16-11.79 tons (18-26 kips). The axle weight is a variable that changes in the iterative procedure for every 0.91 ton (2 kips). It means that there are the following values for the first axle weight: 8.16, 9.07, 9.98, 10.89, 11.79 tons (18, 20, 22, 24, 26 kips). The first axle spacing range is 4.57-8.23 m (15-27 ft), and the iterative step is 0.91 m (3 ft). That provides the following first axle spacing combinations: 4.57, 5.48, 6.40, 7.31, 7.92 m (15, 18, 21, 24, 27 ft). The other axle load combinations have an iterative step of 0.91 ton (2 kips) and the tandem axle spacing of 0.15 m (0.5 ft).

Based on the presented load and spacing ranges and iterative procedure, notional permit vehicle candidates' combinations were created. The last column of **Table 3.3**, presents a number of created combinations of notional permit vehicle candidates. The total number of notional permit vehicle candidates is over 300,000.

Figure 3.4 presents the first candidate and its axle load and axle spacing range to visualize the procedure which generates the groups of notional permit vehicles. This

approach allows considering a wide range of possible notional permit vehicle candidates to find representative ones.

Table 3.3. Groups of load and spacing ranges for notional permit vehicle candidates.

ID	No of Axles	First Axle Load [t/(kips)]	First Axle Spacing [t/(kips)]	Axle Loads [t/(kips)]	Tandem Spacing [m/(ft)]	MidAxle Spacing [m/(ft)]	BackAxle Spacing [m/(ft)]	No of Combs
1	5	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	6.10÷9.75 (20÷32)	-	15,925
2	6	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	6.10÷9.75 (20÷32)	-	15,925
3	7	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	6.10÷9.75 (20÷32)	1.22÷2.13 (4÷7)	111,475
4	8	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	10.97÷12.19 (36÷40)	1.22÷2.13 (4÷7)	79,625
5	7	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	8.53÷12.19 (28÷40)	-	15,925
6	8	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	8.53÷12.19 (28÷40)	-	15,925
7	12	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	8.53÷12.19 (28÷40)	-	15,925
8	13	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	8.53÷12.19 (28÷40)	-	15,925
9	13	8.16÷11.7 (18÷26)	4.57÷8.23 (15÷27)	8.16÷13.60 (18÷30)	1.22÷2.13 (4÷7)	8.53÷12.19 (28÷40)	-	15,925

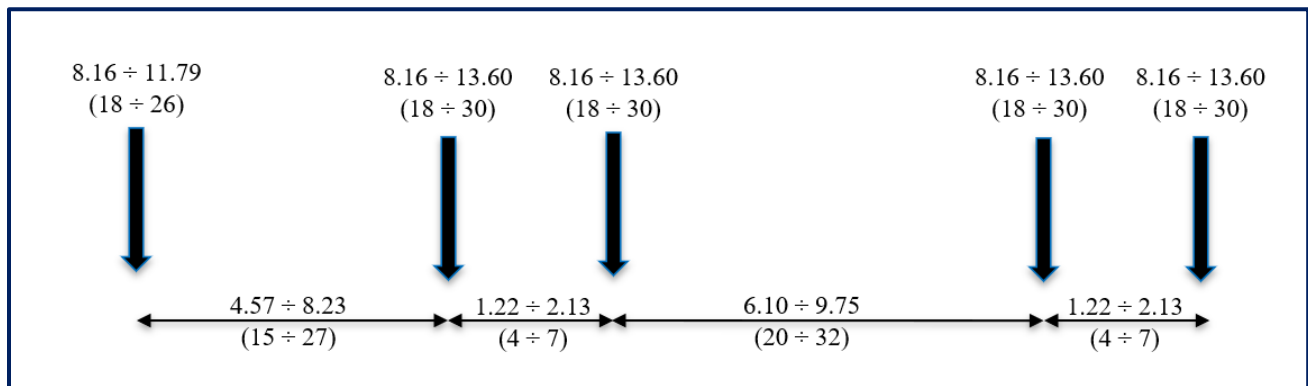


Figure 3.4. Considered range of axle load and spacing for candidates in group 1.

3.4.3. Statistical parameters of live load

Statistical parameters are essential in reliability-based methods as they define the level of uncertainty of random variables. The required statistical parameters include a bias

factor (ratio of mean-to-nominal/design value) and a coefficient of variation (ratio of standard deviation and the mean). In this study, the notional permit vehicle candidates are checked by a reliability-based procedure to examine the consistency of provided safety. Therefore, the statistical parameters of the live load are developed for every notional permit vehicle candidate and every load case. The load effects of notional permit vehicle candidates are compared to single and multi-trip permit vehicles.

The development of statistical parameters for live load models is a complex process. In this case, it required an analysis of the traffic database and calculation of load effects. Load effects were computed for all permit vehicles for simple and continuous span bridges to capture a broad range of load effects. Also, the load effects in terms of moment and shear were computed for all possible combinations of notional permit candidates.

As shown in **Table 3.3**, there are over 300,000 permit candidates for which live load effects were computed. Candidates for groups 1 to 4 were determined based on multi-trip permit vehicles' characteristics, and candidates for groups 5 to 9 represent single trip permit vehicle configurations. To find the bias factor for live load, it is necessary to determine moment and shear ratios for every candidate in comparison to all vehicles recorded in a permit database for a single and multi-trip.

Figure 3.5 presents the CDF plots for (a) simple span moment ratio for the length of 9 m (30 ft), for the randomly chosen candidate from group 7, and (b) moment ratio for all fifteen considered load cases. For each permit vehicle candidate, the moment and shear ratios were computed to determine bias factors for every notional permit candidate and load case.

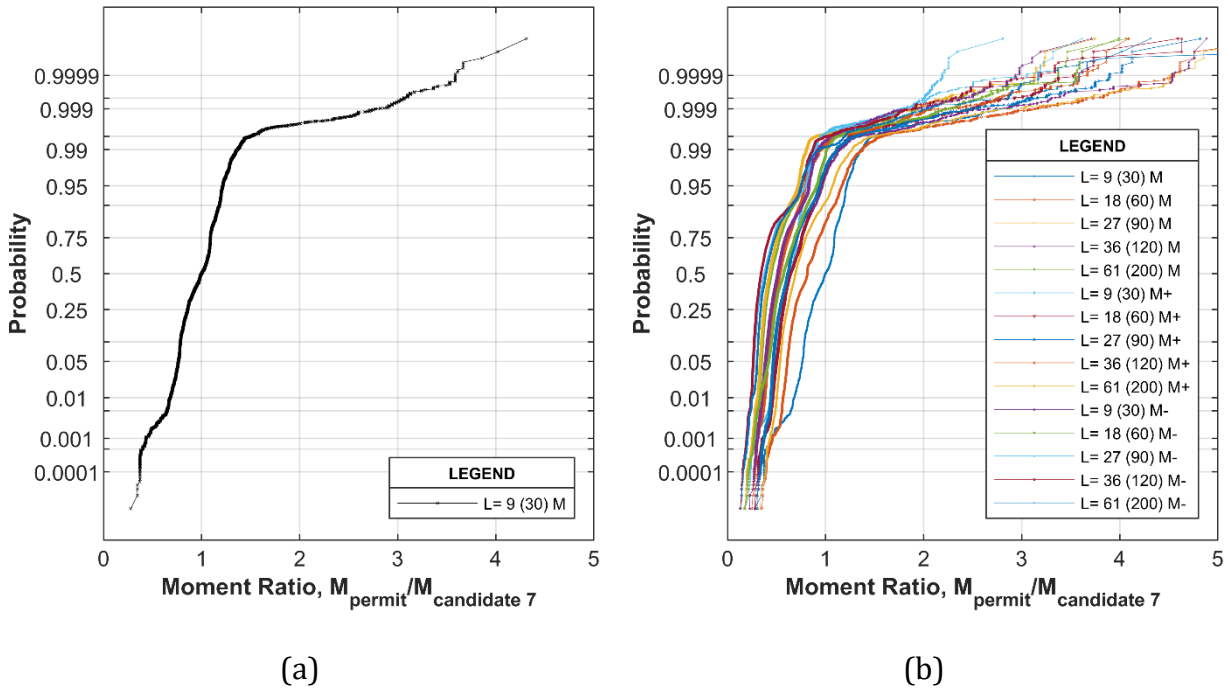


Figure 3.5. Moment effects for simple span (M), positive (M+) and negative (M-) continuous ratios for single trip permit vehicles and a random permit vehicle candidate from group 7 for (a) simple span moment ratio for span length 9 m (30 ft), (b) all moment load cases.

Live load is a time-varying variable, so its characteristics depend on time period or a specific return period. To determine the bias factor for an extended return period, the method of extrapolation is required. Since this study is not a calibration but a presentation of the reliability-based method to develop notional permit vehicles, extrapolation is not performed. A one-day return period is considered to find a bias factor for multi and single trip permits, which allows for the elimination of the data extrapolation analysis. Moment and shear ratios for the notional permit candidates are used to determine a bias factor for the specified return period and traffic volume, usually defined by the Average Daily Truck Traffic (ADTT).

Bridge service life is specified as 75 years by AASHTO LRFD; this means that the return period is 75 years. If the expected ADTT for a newly built bridge is 1,000 trucks per day, the number of trucks expected to cross the bridge within 75 years can be expressed as (365 days) (75 years) (1000 trucks/day), which gives 27,375,000 trucks. For this example, the probability (P) of exceeding the maximum load is expressed in **Equation (3.1)**:

$$P = \frac{1}{(365 \text{ days})(75 \text{ years})(1000 \text{ trucks/day})} = (3.65)(10^{-8}) \quad (3.1)$$

The probability depends on the number of vehicles that cross the bridge. Florida WIM data was used to find the ADTT per single lane for multi and single trip permits to determine the expected number of overloaded vehicles crossing the bridge. Only a single-lane loading is considered since the multiple presence analysis is not part of this procedure.

Data recorded by 33 of Florida's WIM sites was used to find the Average Daily Overloaded Truck Traffic (ADOTT) per lane. All overloaded vehicles were considered, where single trip permits were filtered using two criteria of GVW above 40.82 tons (90 kips), and at least one axle load above 9.07 tons (20 kips). These criteria were determined based on analyzed issued permit vehicle characteristics. The overloaded WIM data was filtered for multi and single trip permits, and then ADOTT per each WIM site, and one lane was captured.

Figure 3.6 shows the distribution of ADOTT per lane in Florida for multi and single trip permits. The 95th quantile of the ADOTT per lane was considered in the analysis. It gives 22 single trip permit vehicles and 492 multi trips permit vehicles per one day. These vehicle counts can be used to compute the probabilities, which are 0.9(54) for single trip permits and 0.997(9674) for multi trips. Calculated probabilities are used to find bias factors for every permit vehicle candidate and load case.

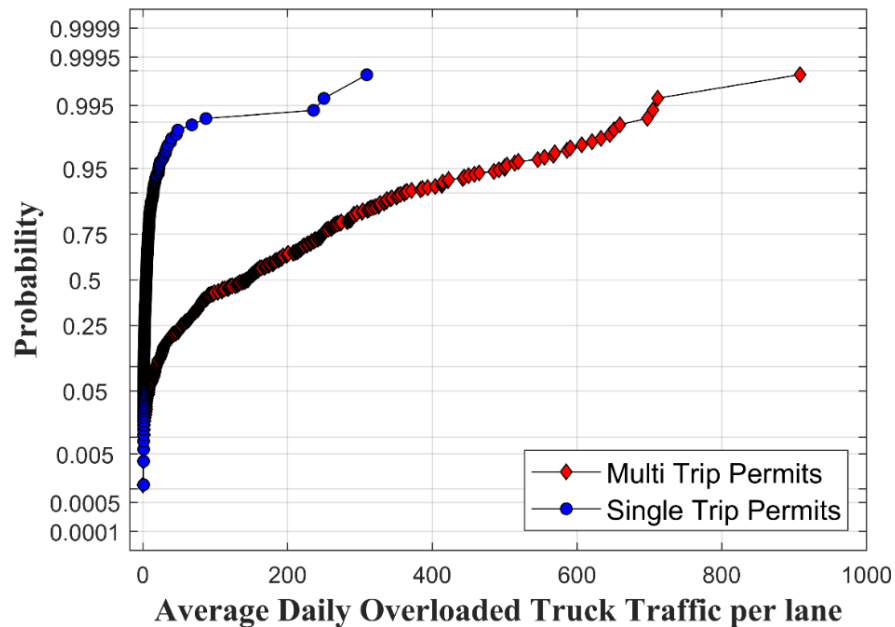


Figure 3.6. ADOTT per lane for multi and single trip permit vehicles.

A special procedure was developed to find bias factors for all moments and shear load effects for every possible notional permit vehicle candidate. Such analysis allows calculating bias factors for a wide range of bridges and load effects. The coefficient of variation of computed biases is taken as the coefficient of variation of moments and shears. Bias and coefficient of variation are then used to select the representative notional permit vehicles to form each group.

3.4.4. Selection of notional permit vehicle candidates

Notional permit vehicle candidates were created, and their statistical parameters of live loads, such as bias factors and coefficients of variation, were computed. Furthermore, the percentage of vehicles that exceed the bias factor of 1.0 was determined for every candidate and load case. The computed parameters provide a basis to select notional permit vehicles that provide uniform safety for different bridges and load cases.

The filtering criteria for the notional vehicle selection are shown in **Figure 3.7**. The lowest variation of statistical parameters for moment and shear is captured to select vehicles with consistent reliability. The three filtering criteria are considered (1) the lowest variation of bias factor for a moment, (2) the lowest variation of bias factor for shear, and (3) the lowest percentage of vehicles above 1.0. It was decided to reduce the number of possible permit candidates by taking 90th quantiles for the lowest variance for moments, shears, and percentage of vehicles with bias above 1.0. These criteria allowed to greatly reduce the number of permit vehicle candidates.

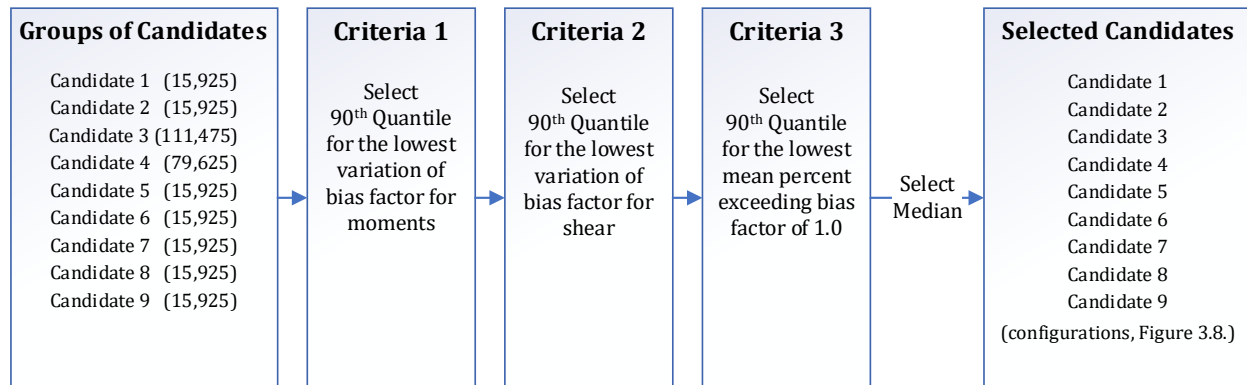


Figure 3.7. Notional Permit Vehicle candidate's selection criteria.

The remaining vehicles were checked. It was concluded that they do not differ significantly from each other and provide very similar statistical parameters of live load. To select one permit vehicle candidate from each group, the median for each group was taken since the variation of the remaining vehicles was insignificant. Based on the presented filtering method, one representative vehicle was selected from each permit candidate group.

Figure 3.8 shows the configuration of the selected notional permit candidates from each group. The axle weights are shown in tones and meters, and in parenthesis in kips and feet.

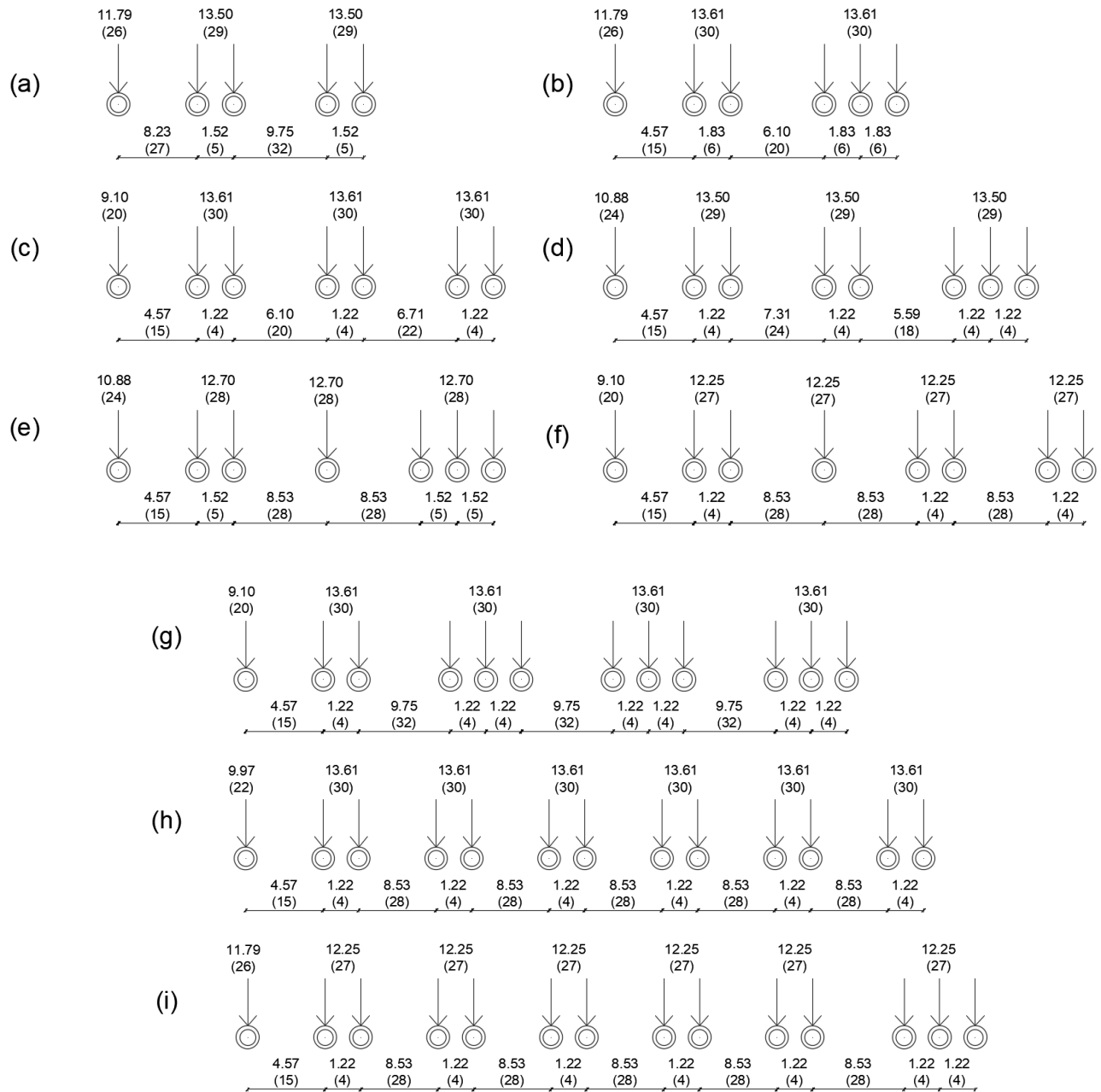


Figure 3.8. Selected notional permit candidates from groups (a) Candidate 1, (b) Candidate 2, (c) Candidate 3, (d) Candidate 4, (e) Candidate 5, (f) Candidate 6, (g) Candidate 7, (h) Candidate 8, (i) Candidate 9 (axle weights tons (kips), spacings m (ft)).

3.4.5. Reliability-based method

Bridge design and evaluation procedures are based on the load and resistance factor approach. The load and resistance are considered random variables. Each random variable can be described by statistical parameters that account for its variation, a coefficient of variation, and a bias factor, which defines a ratio of mean maximum expected value to the design value. The reliability-based calibration procedure allows for determining appropriate load and resistance factors that provide desired safety. The acceptance criterion for load and resistance factors is to keep the probability of failure at an acceptably low level. In practice, rather than a probability of failure, the structural condition can be defined in terms of a reliability index, β . Therefore, load and resistance factors are selected so that the designed structures have β that is not less than the predetermined target reliability index, β_T . Since the design and evaluation procedures were derived using reliability methods, also a notional permit vehicle should be developed based on the reliability-based criteria to assure consistent safety for the wide range of bridges. Therefore, the proposed procedure to develop a representative notional permit vehicle is consistent with the LRFD philosophy.

Figure 3.9 Figure 3.9 presents a proposed reliability-based procedure. It begins with the selection of notional permit vehicle candidates based on state-specific heavy traffic attributes. For Florida example, nine groups of notional permit vehicles were selected, and they are summarized in **Table 3.3**. The next step is the calculation of load effects for permit vehicles and permit vehicle candidates for simple span bridges for length 9, 18, 27, 36, 61 m (30, 60, 90, 120, 200 ft), and continuous span for equal span lengths 9-9, 18-18, 27-27, 36-36, 61-61 m (30-30, 60-60, 90-90, 120-120, 200-200 ft). Load effects are then used to determine statistical parameters of live load.

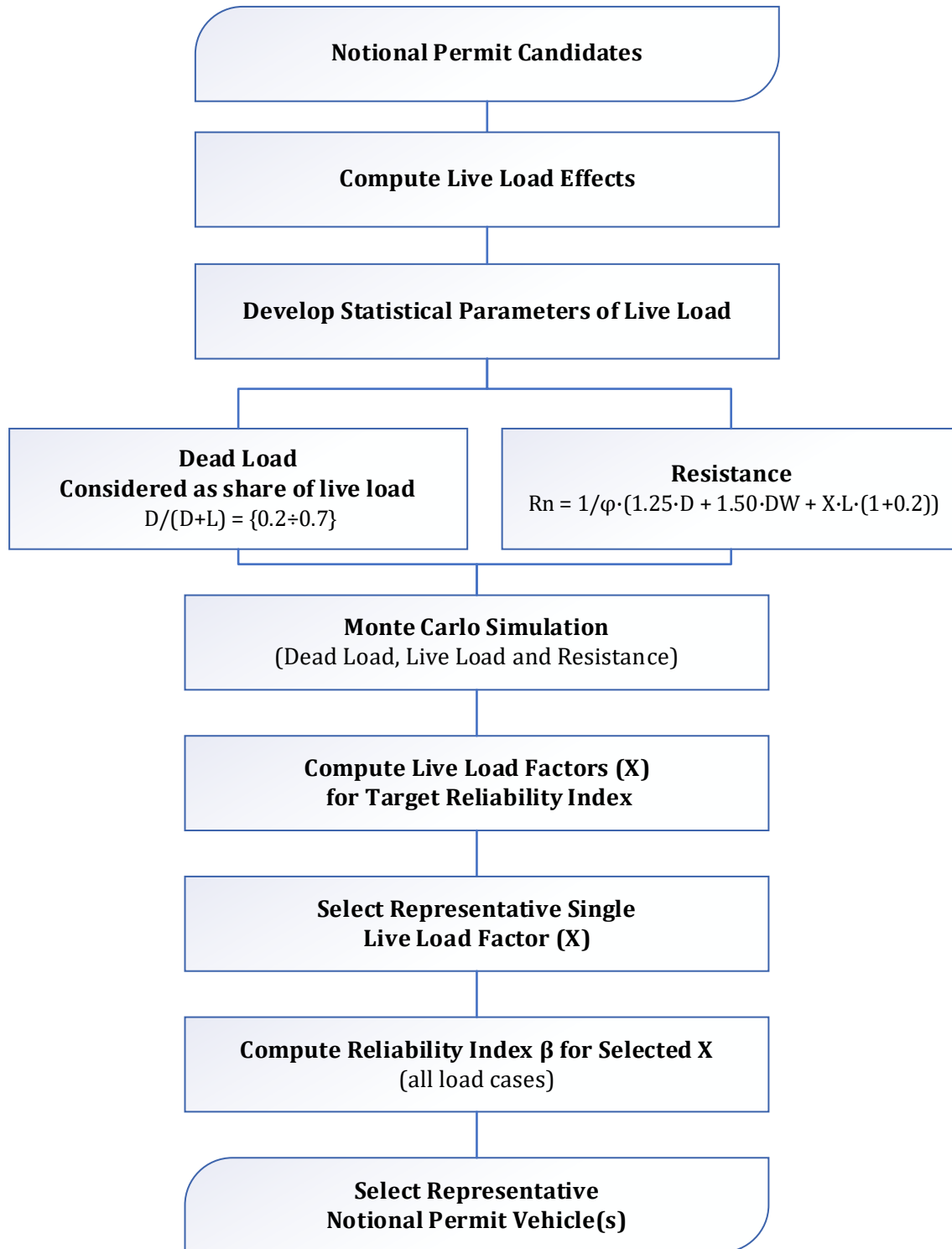


Figure 3.9. Reliability-based method to select the representative notional permit vehicles which provide uniform safety.

The bias factors are determined for all considered load cases and candidates. The coefficient of variation of computed biases is taken as the coefficient of variation of considered moment and shear cases. Also, the dynamic load effect is considered. There is not enough data about the dynamic impact of overloaded vehicles. Hence, the dynamic load is assumed as in the original AASHTO calibration (NCHRP Report 368), where the derivations of dynamic load effects were based on the numerical simulations using an equivalent static load effect and further confirmed by field testing (Nowak 1999). In this study, the coefficient of variation of live load with dynamic load is 18-21%. Live load is represented by a lognormal distribution.

The dead load is considered as 20-70% of the expected total loading. The dead load ranges were developed based on over 100 representative bridge structures with different geometries presented in NCHRP 368. It allowed accounting for a majority of dead load cases in bridge engineering structures. The statistical parameters of dead load are available for prefabricated steel and prestressed concrete elements, site-cast concrete, and wearing surfaces. The values of statistical parameters are taken from the original AASHTO calibration (Nowak 1995). The bias factor and coefficient of variation are 1.03, 0.08 for prefabricated elements, 1.05, 0.10 for site-cast concrete, and 1.0, 0.25 for wearing surface. Dead load is assumed to be normally distributed (NCHRP 368).

Resistance is calculated as the minimum required load-carrying capacity for a given total factored load with a specified resistance factor. The resistance factors for steel bridges are taken as 1.00. For reinforced concrete, the resistance factor is 0.90, and for prestressed concrete, it is 1.00 for flexure and 0.90 for shear. Statistical parameters of resistance depend on material properties, quality of workmanship, and analytical model.

Resistance is considered a product of three factors: material properties, fabrication (dimensions), and analytical model. Tests, simulations, and observations determine the statistical parameters of resistance. The bias factor and coefficient of steel variation in flexure are 1.12 and 0.10, and shear 1.14, 0.105. For reinforced concrete in flexure 1.14, and 0.13, and shear 1.20, 0.155. In flexure, the prestressed concrete structure is 1.05, 0.075, and shear 1.15, 0.14 (NCHRP 368).

Reliability indices were calculated using statistical parameters listed above and Monte Carlo simulation. Live load factors were determined for the selected notional permit candidates that produce reliability close to the target reliability index. In this study, the target reliability index was selected as 2.50. The goal is to use a single live load factor for permit traffic that produces desired safety level set by the target reliability index.

Table 3.4 presents the live load calculations for moment cases for Candidate 6. It shows the calculated bias factor for different bridge span lengths and live load moment effects. Dead load for this case 20% of the total load. The dead load is considered 85% is self-weight, and 15% wearing surface, based on the representative bridge dataset from NCHRP 368. The resistance is calculated as the minimum required load-carrying capacity based on load demand. Values of the considered random variables are then simulated using previously established statistical parameters and distribution types, where the dead load is assumed to be a normal distribution and the live load and resistance as a lognormal distribution.

The Monte Carlo method was used, and one million simulations were applied for every load case and random variable. An iterative procedure was used to find the optimal live load factor, which produces a target reliability index, β_T . Load and resistance factors are

to be rounded to the nearest 0.05. Therefore, the iteration step is taken as 0.05. The last two columns in **Table 3.4** show reliability indices β , and calculated live load factors (γ_{LL}).

Table 3.4. Load and resistance moment effects for Candidate 6.

Load Case	Bias λ	Live Load [kNm (kip-ft)]	Dead Load [kNm (kip-ft)]	Wearing	Resistance [kNm (kip-ft)]	Reliability index, β	Live
				Surface [kNm (kip-ft)]			load factor γ_{LL}
30 M	1.34	4,387 (301)	932 (64)	165 (11)	10,888 (746)	2.38	1.80
60 M	1.28	13,251 (908)	2,816 (193)	497 (34)	32,887 (2253)	2.58	1.80
90 M	1.33	24,070 (1649)	5,115 (350)	903 (62)	59,739 (4093)	2.42	1.80
120 M	1.28	37,891 (2596)	8,052 (552)	1,421 (97)	94,041 (6444)	2.59	1.80
200 M	1.17	97,735 (6697)	20,769 (1423)	3,665 (251)	219,110 (15014)	2.45	1.60
30-30 M+	1.08	4,251 (291)	903 (62)	159 (11)	9,020 (618)	2.57	1.50
60-60 M+	1.23	10,726 (735)	2,279 (156)	402 (28)	25,333 (1736)	2.53	1.70
90-90 M+	1.31	18,999 (1302)	4,037 (277)	712 (49)	47,154 (3231)	2.48	1.80
120-120 M+	1.18	31,901 (2186)	6,779 (465)	1,196 (82)	71,519 (4901)	2.42	1.60
200-200 M+	1.12	78,843 (5402)	16,754 (1148)	2,957 (203)	167,294 (11463)	2.37	1.50
30-30 M-	1.10	4,242 (291)	901 (62)	159 (11)	9,000 (617)	2.47	1.50
60-60 M-	1.28	11,115 (762)	2,362 (162)	417 (29)	27,587 (1890)	2.60	1.80
90-90 M-	1.23	22,103 (1515)	4,697 (322)	829 (57)	52,205 (3577)	2.53	1.70
120-120 M-	1.25	28,892 (1980)	6,139 (421)	1,083 (74)	68,238 (4676)	2.46	1.70
200-200 M-	1.18	48,885 (3350)	10,388 (712)	1,833 (126)	109,595 (7510)	2.41	1.60

Every candidate was checked for considered dead load ratio, and bridge material types to determine live load factor. **Table 3.5** shows the calculations of live load factors for different moment load cases for Candidate 6. Based on these calculations, the average live load factor is selected for further analysis. The single live load factor is then used to calculate reliability index, β for steel, P/C, and R/C bridges for different dead load share, separately for moment and shear.

This approach allows for finding a notional permit vehicle resulting in a uniform safety consistent with the load and resistance factor philosophy. Monte Carlo simulation was

conducted for all nine considered notional permit vehicle candidates and various dead load percentages varying from 20 to 70% for every 10%.

Table 3.5. Live load factors produce a target reliability index of 2.50 for different dead load ratio and moment load cases.

Load Case	Dead Load Share D/(D+L)											
	0.20		0.30		0.40		0.50		0.60		0.70	
	β	γ_{LL}	β	γ_{LL}	β	γ_{LL}	β	γ_{LL}	β	γ_{LL}	β	γ_{LL}
30 M	2.38	1.80	2.38	1.80	2.40	1.80	2.41	1.80	2.42	1.80	2.43	1.80
60 M	2.58	1.80	2.60	1.80	2.60	1.80	2.61	1.80	2.62	1.80	2.63	1.80
90 M	2.42	1.80	2.42	1.80	2.44	1.80	2.46	1.80	2.47	1.80	2.48	1.80
120 M	2.59	1.80	2.60	1.80	2.61	1.80	2.62	1.80	2.63	1.80	2.63	1.80
200 M	2.45	1.60	2.47	1.60	2.47	1.60	2.48	1.60	2.49	1.60	2.49	1.60
30-30 M+	2.57	1.50	2.56	1.50	2.58	1.50	2.59	1.50	2.60	1.50	2.58	1.50
60-60 M+	2.53	1.70	2.54	1.70	2.56	1.70	2.56	1.70	2.57	1.70	2.57	1.70
90-90 M+	2.48	1.80	2.50	1.80	2.50	1.80	2.52	1.80	2.53	1.80	2.54	1.80
120-120 M+	2.42	1.60	2.42	1.60	2.44	1.60	2.44	1.60	2.45	1.60	2.46	1.60
200-200 M+	2.37	1.50	2.38	1.50	2.39	1.50	2.41	1.50	2.42	1.50	2.42	1.50
30-30 M-	2.47	1.50	2.48	1.50	2.49	1.50	2.50	1.50	2.51	1.50	2.50	1.50
60-60 M-	2.60	1.80	2.60	1.80	2.61	1.80	2.61	1.80	2.63	1.80	2.63	1.80
90-90 M-	2.53	1.70	2.54	1.70	2.56	1.70	2.56	1.70	2.57	1.70	2.57	1.70
120-120 M-	2.46	1.70	2.46	1.70	2.48	1.70	2.49	1.70	2.50	1.70	2.50	1.70
200-200 M-	2.41	1.60	2.42	1.60	2.44	1.60	2.44	1.60	2.46	1.60	2.46	1.60

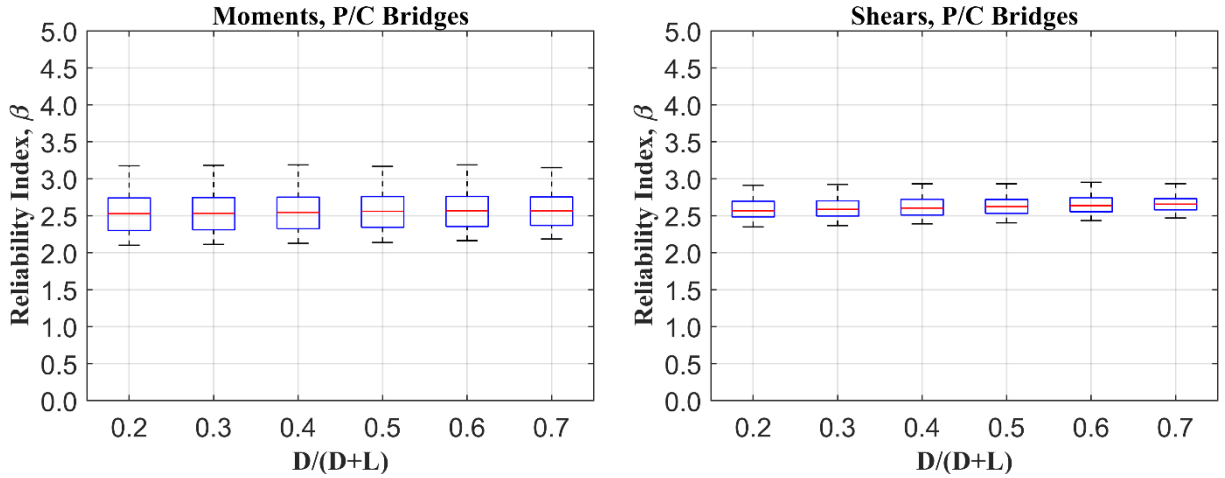
3.5. RESULTS AND DISCUSSION

The presented reliability procedure determines notional permit vehicles, which provides a uniform safety for a broad spectrum of bridges. Over 300,000 notional permit candidates were considered and filtered out to select one vehicle from nine different groups using Florida traffic data. All nine notional permit candidates were checked using the reliability-based method. The aim was to use a single candidate for all types of bridges that yields desirable safety for selected live load factors adjusted to the state-specific permit live load.

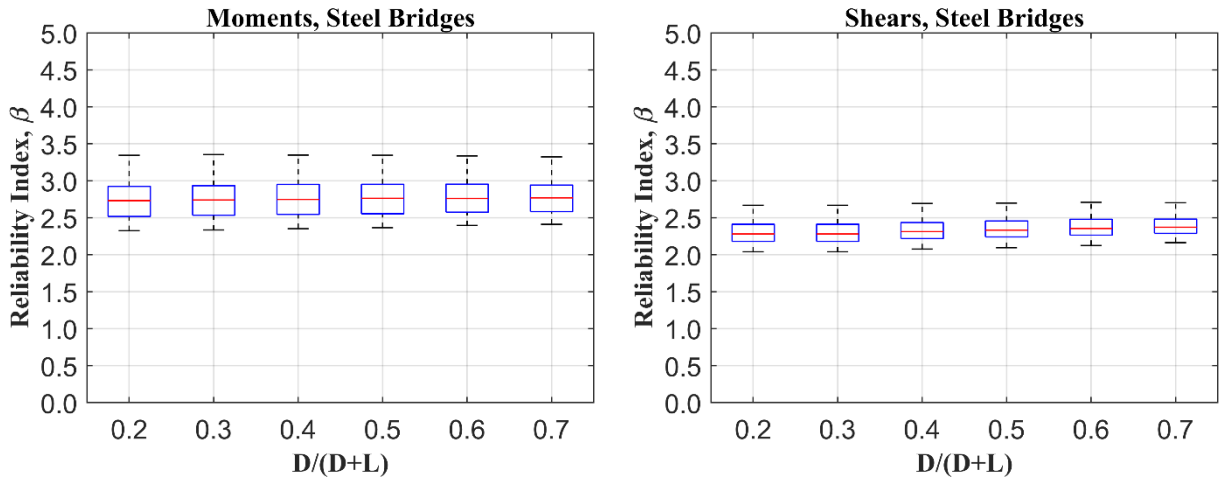
A separate analysis was conducted for simple and multi trip permits. Candidates 5-6 were checked for all dead load ratios and different bridge material types for moment and shear in case of single trip permits. The analysis proved that the notional permits Candidate 6 is adequate for a selected live load factor of 1.70 for a moment, and 1.40 for shear. **Figure 3.10** shows the reliability indices for moment and shear for dead load percentage from 20-70% for prestressed concrete, reinforced concrete, and steel bridges. The box plots indicate that the median for all cases is at required safety level defined by target reliability index, β_T .

Candidates 1 to 4 were checked for the multi trip permits but the selected for single trip permits Candidate 6 was also verified. It was concluded that Candidate 6, with eight axles, a total GVW of 94.8 t (209 kips), and a length of 33.83 m (111 ft), is also the best representation for multi trip permits. **Figure 3.11** presents the reliability indices for multi trip for the moment and shear for various percentages of dead load for prestressed concrete, reinforced concrete, and steel bridges. The live load factor for Candidate 6 was 2.10 for a moment, and 1.40 for shear. The box plots indicate that the median for all cases is above the minimum required safety. This notional vehicle also provides consistent safety for Florida multi trip permits.

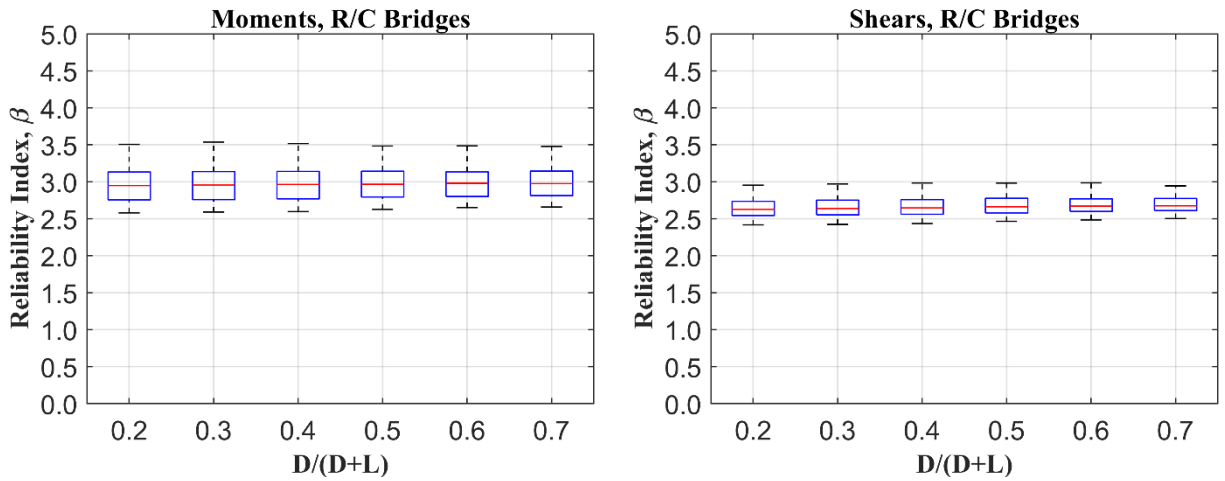
Figure 3.12 shows the range of reliability indices for selected Candidate 6 for single and multi-trip permits for prestressed concrete, steel, and reinforced concrete bridges for various moments and shear dead load ratios. It confirms that the selected notional permit vehicle Candidate 6 for the chosen single live load factor provides consistent safety for a wide bridge spectrum.



(a)

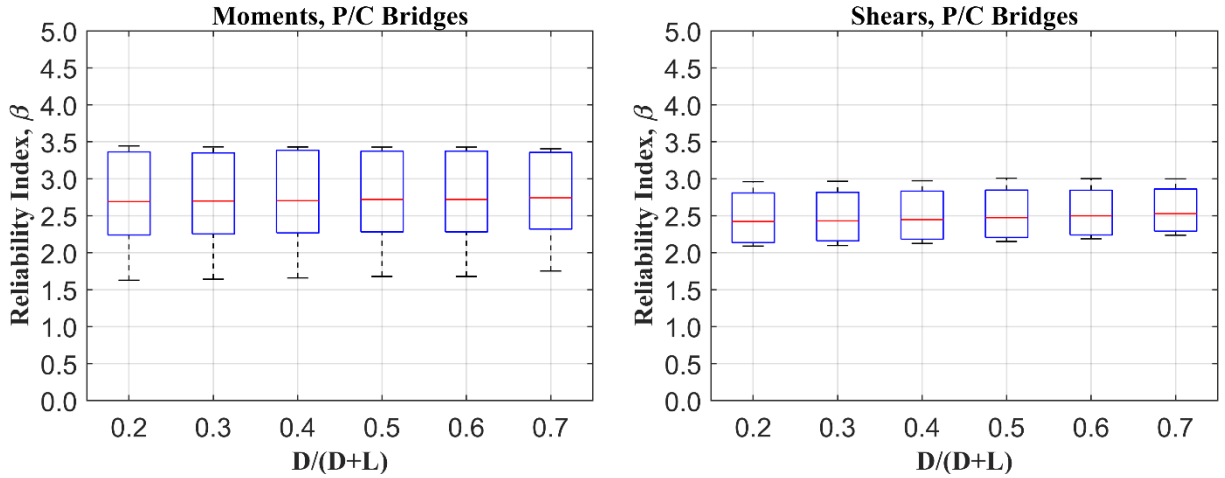


(b)

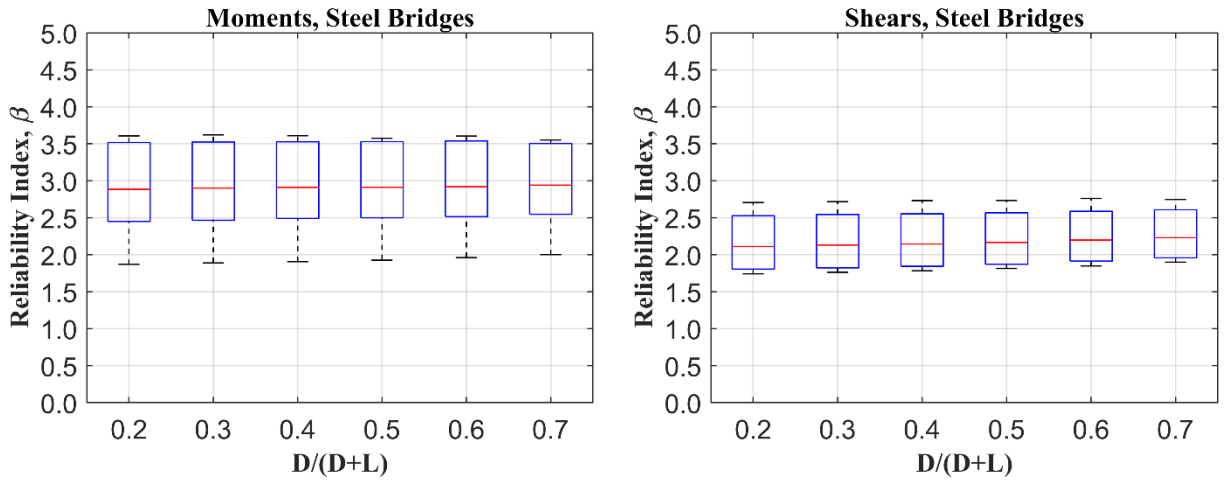


(c)

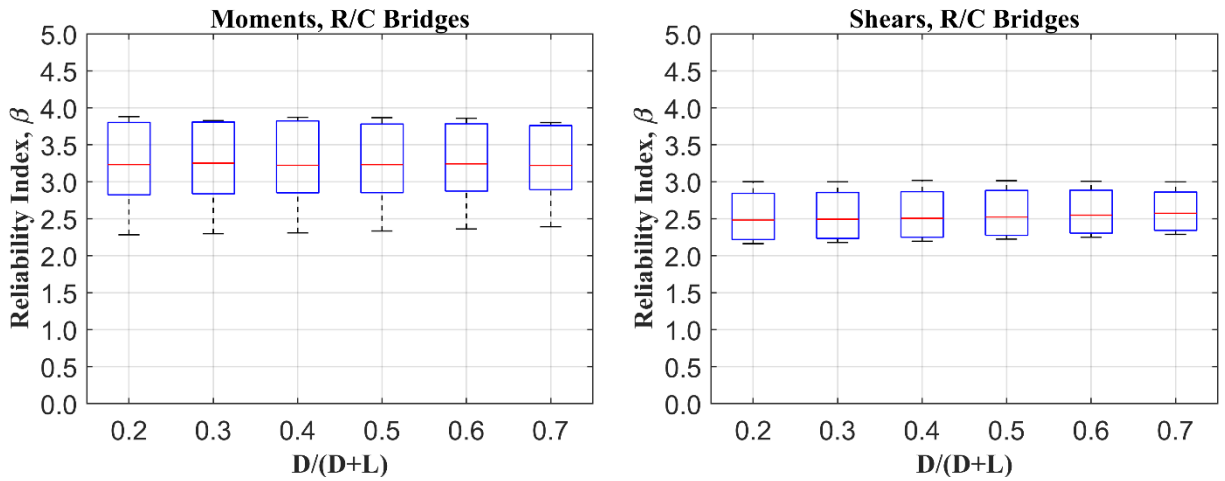
Figure 3.10. Reliability index for single trip permits and Candidate 6 (a) P/C bridges, (b) Steel bridges, and (c) R/C bridges for various moment and shear dead load ratios.



(a)

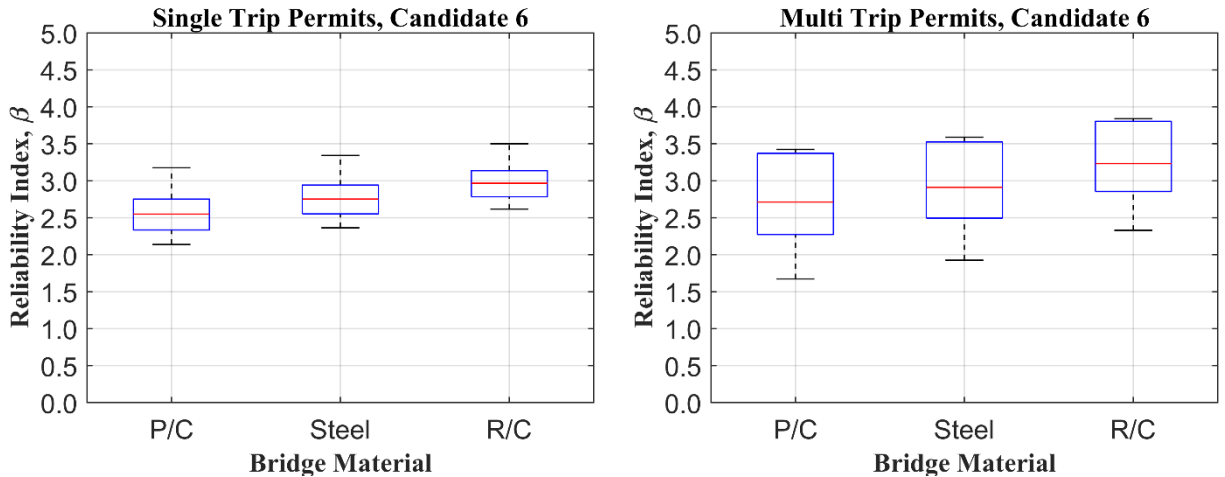


(b)

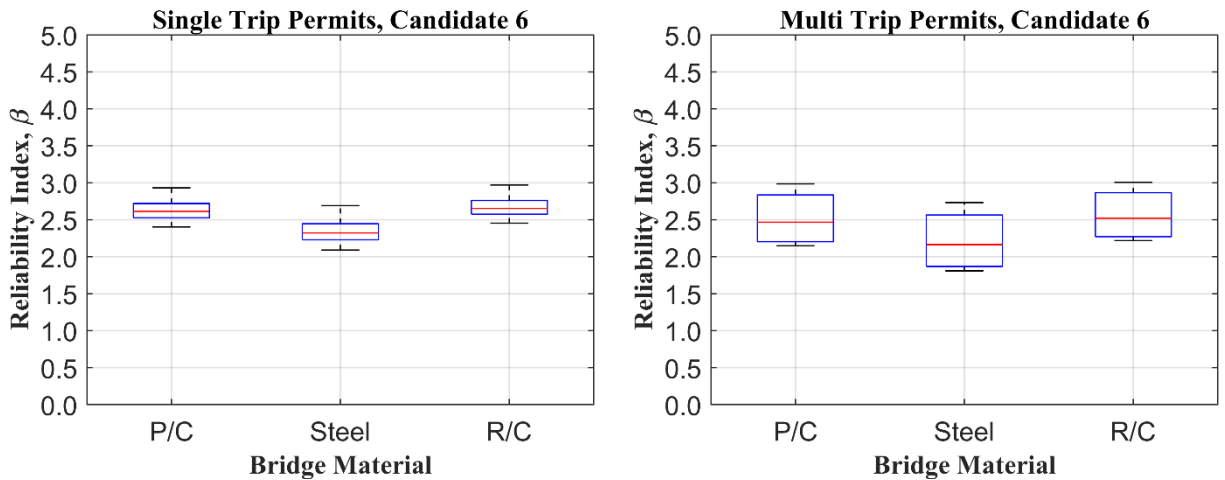


(c)

Figure 3.11. Reliability index for multi trip permits and Candidate 6 (a) P/C bridges, (b) Steel bridges, and (c) R/C bridges for various moment and shear dead load ratios.



(a) Moment



(b) Shear

Figure 3.12. Reliability index for Candidate 6, (a) P/C bridges, (b) Steel bridges, and (c) R/C bridges for various moment and shear dead load ratios.

3.6. CONCLUSIONS

This study presents a reliability-based method for developing representative notional permit vehicles that provide uniform safety for a wide spectrum of bridges. The developed approach is consistent with AASHTO design and evaluation provisions. Based on the available WIM and permit data, state-specific permit live load spectra are determined with overloaded vehicle characteristics to create notional permit vehicle candidates. They are then used to select representative state-specific permit vehicle(s).

The developed reliability-based procedure was demonstrated using Florida as an example. Overload permit and WIM data were used to compute live load effects for simple span and continuous span bridges. The permit vehicle characteristics for single and multi-trips permits allowed for the development of nine groups of permit candidates. Over 300,000 notional permit candidates were generated and considered in the analysis. Nine notional permit vehicles were selected based on the low variability of permit statistical parameters. Selected notional permit vehicles with a number of axles from 5 to 13 and GVW from 64.41-158.76 tons (142-350 kips) were checked using the reliability-based approach. Monte Carlo simulations were used to select live load factors that produced the desired target reliability index. For a single selected live load factor, reliability indices were calculated for various bridge types and materials. The analysis concluded that Candidate 6, with eight axles and a total GVW of 94.8 t (209 kips), and a length of 33.83 m (111 ft), is the best representation for both single and multi-trip permits. That vehicle provides uniform safety for all considered load cases.

The procedure can be numerically intense because it requires extensive traffic data analysis and simulation techniques. However, it enables the development of suitable notional permit vehicles that can be used in bridge design for Strength Limit State II, bridge evaluation under permit loading, and even help with permit vehicle issuance. All issued overload permits that produce a load effect lower than the developed notional permit vehicles can apply for a permit without any complex and time-consuming numerical analysis. The proposed method offers a very beneficial and cost-effective process for the state DOTs to reduce individual permit analyses to issue a permit.

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CHAPTER 4 BRIDGE RATING FOR GRANDFATHER VEHICLES³

ABSTRACT

To ensure bridge safety under vehicular traffic, owners must systematically verify the adequacy of bridge live load-carrying capacity. Bridges are evaluated for design, legal, and permit loading. The legal load rating uses AASHTO legal trucks, which meet the Federal Bridge Formula requirements. However, each state has a federal exemption for truck weight and size law. The state-specific grandfather provisions allow operation of legally overloaded vehicles. Thus, this study aims to determine if the current live load factors provide adequate safety for exempted vehicles since there are no provisions for overloaded vehicles under grandfather provisions. This study presents a reliability-based analysis to determine state-specific live load factor for bridge rating on an example of Montana's 20% overload allowance for agricultural vehicles. This approach does not require a re-analysis of the state bridge inventory. Linear scaling of existing rating results from bridge records can be used to determine ratings for agricultural vehicles. The study provides a calibration method to maintain the required safety level with the minimum cost and effort.

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4.1.INTRODUCTION

Many bridges in the United States are approaching the end of their design life, resulting in an increasing number of bridges requiring major repair and rehabilitation. To ensure the safety of existing bridges under vehicular traffic, there is a need for systematic evaluation procedures to verify the live load carrying capacity, determined by the rating factor. The rating factor is calculated for the design load, legal load, and permit vehicle load. The design load is represented by the design truck, and it is used to check the inventory and operating rating. The legal load is defined by vehicles that meet the requirements under federal law for truck weight and size. The permit vehicles are legally overloaded vehicles that purchased a permit to operate on bridges and roads. Permits are site-specific and are represented by state-specific notional vehicles.

Currently, there are no rating provisions for legally overloaded vehicles under grandfather provisions. The grandfather provisions are old rules that remain unchanged after a new rule was introduced. Therefore, some vehicles can operate above the federal truck weight and size law. The load effect caused by legally overloaded vehicles is not reflected in bridge rating provisions. Hence, this study provides a methodology to validate if the load effect caused by exempted vehicles should be considered to adjust the live load factor for bridge rating. The proposed approach includes a procedure to identify legally overloaded vehicles in the Weigh-in-Motion (WIM) database and calculate their load effects on representative bridges. The reliability-based calibration is presented to calculate optimal live load factors for bridge evaluation procedures to achieve the desired safety level under vehicles with grandfather provisions. The proposed method for bridge evaluation is not state-specific, and it can be used to verify the impact of any exempted vehicles. It is essential

to maintain the safety of bridges under existing traffic. Excessive load effects not considered in bridge rating procedures can cause accelerated wear and tear of bridge structures. Therefore, the proposed methodology can help bridge owners assess the load effect caused by legally overloaded vehicles and adjust the rating policy accordingly. The approach is demonstrated in an example of vehicles carrying agricultural products with a 20% overload allowance in Montana.

4.2. LITERATURE REVIEW

AASHTO Manual for Bridge Evaluation (MBE) provides procedures to evaluate bridge live load capacity using the Load and Resistance Factor Rating (LRFR) analysis. Bridge evaluation procedures have evolved throughout the years and adapted to changing traffic volume and parameters, like increased vehicle size and weight, axle loads, etc. The development of AASHTO guides for bridge evaluation was summarized by (Gao 2012). In 2003, the “AASHTO Guide Manual” was introduced in the NCHRP 12-46 (Moses 2001) to supplement already employed in the US Load and Resistance Factor Design provisions. The first edition of the AASHTO Manual for Bridge Evaluation was released in 2008. It was developed to help bridge owners to maintain safety under a considerable variation of traffic loads. MBE provides rating procedures by utilizing notional vehicles representing design, legal, and permit loads. The notional vehicles were developed to represent traffic across the US, and live load factors are used to envelop the load effects and provide uniform safety for a wide range of bridges.

MBE development was based on the original AASHTO LRFD calibration (Nowak 1999). The calibration procedure requires the development of load and resistance models, selection of reliability analysis methods, calculation of load and resistance factors, and

corresponding reliability indices for various bridge types. AASHTO LRFD live load model was based on truck survey data from 1975 in Ontario, Canada. In MBE, the live load model was derived from the WIM database.

The live load model that accurately represents the traffic and considers the uncertainty in predicting the maximum bridge loading varies from site to site and often depends on the traffic volume. Therefore, the WIM database plays a key role in developing the live load model required for reliability-based calibration. The WIM data includes detailed information about vehicle weight and configuration. The vehicle-specific parameters can be used to assess the load effects on bridges. The WIM data is a massive database that needs to be validated to eliminate questionable records and accurately quantify the live load effects. The protocols for WIM data collection, cleaning, and processing were presented by (Moses 2001). Many state Department of Transportation (DOT) developed their unique quality assurance procedures to satisfy the federal requirements (Ramachandran et al. 2011, Vandervalk-Ostrander, A., 2009). The need for adequate traffic weight data in bridge design has been studied extensively in NCHRP report 683 (Sivakumar et al. 2008). Comprehensive information provided by the WIM system allows evaluating the effects caused by existing traffic, as utilized by (Caprani et al., 2008; Nowak and Rakoczy, 2013; OBrien and Enright, 2013). The traffic is constantly changing, and WIM data helps identify the trends in heavy trucks to evaluate their effects on infrastructure. The development of live load factors using the WIM data was studied by (Fu and Hag-Elsafi 2000; Moses 2001, O'Brien, and Enright 2013).

WIM data can serve as a source to adjust the live load factor for site-specific traffic. A set of protocols to calculate site-specific live load factors using the Weigh-in-Motion

database was developed (Sivakumar et al. 2008). The available truck traffic data was collected at different WIM sites around the US. The complex reliability-based procedure was developed to determine site-specific load factors. It involves a list of requirements for WIM data collection, quality, and quantity. Live load factor calibration is based on the extreme value theory, where the top 5% of load effects is considered to determine the expected live load effect. The protocols are developed for bridge design, but AASHTO Guide Manual utilizes the same procedure to determine the bridge rating load factor.

The first site-specific live load factors for LRFR were developed in Oregon. The live load factors were calibrated using the same statistical methods as in the original calibration of LRFR. The study showed that state-specific live load factors for legal load were smaller than those provided by the AASHTO Manual (Pelphrey et al. 2008). Later on, Ghosn et al. performed a site-specific calibration of load factors for bridge evaluation in New York (Ghosn et al., 2013). In some instances, the New York traffic load effects were larger than the loading considered in AASHTO LRFR calibration. The live load factor for legal loading had to be increased. It was concluded that for the WIM sites that have significantly higher load effects than those used in LRFR, calibration could be required to adopt new, more conservative factors to maintain bridge safety. Also, the development of site-specific loading can be used when a load is much lower than represented by the national posting vehicles. Traffic is site-specific, and the national live load model development can lead to the local live load effects under or overestimation. Consequently, a traffic variation makes it challenging to envelop all possible load effects in one simple live load model.

The development of a live load model can data-intense and difficult to implement by DOTs. Therefore, the AASHTO Manual provides simplified methods to calculate the site-

specific live load factor based on Gross Vehicle Weight (GVW) (Mlynarski et al., 2011; Moses 2001) and load effects (Sivakumar et al. 2008). Recently, (Siavashi and Eamon, 2020) illustrated the simplified method to develop site-specific live load factors for bridge evaluation. The approach reduces the amount of WIM data with a minimal loss of accuracy. The proposed method provides similar results to standard calibration with reduced effort and computational capacity. It appears that the WIM data truncation approach to determine state-specific live-load rating factors offers accurate results.

Additionally, several studies utilized the WIM data to identify specific overloaded vehicles in the traffic flow to assess their load effects (Fiorillo and Ghosn 2014). The overloaded vehicle evaluation may be essential in bridge posting decisions. The assessment of the heavy vehicles on bridges was studied on a heavy-duty corridor between Indiana and Michigan. The load spectra were checked for selected representative bridges to check the bridge response to that traffic (Chotickai and Bowman, 2005; Reisert and Bowman, 2005). Another study in Wisconsin was conducted to analyze the effect of overloaded vehicles on bridges, specifically on concrete decks and steel girders. This study examined long-term effects on bridge behavior and costs assigned per vehicle overload (Ug-Bae and Oliva 2012). A number of state DOTs sponsored research to update the policy for overloaded vehicles to maintain safety of bridges (Chowdhury et al., 2013; Ghosn et al., 2015; Nassif et al., 2015).

Another issue is the development of notional vehicles, which are used in bridge rating. AASHTO LRFR provides legal trucks for bridge evaluation, which meet the federal law requirements. It was noticed that specialized hauling vehicles meet Federal Bridge Formula requirements but cause excessive load effects on bridges. In 2007, Sivakumar et al. developed notional vehicles that better represent legal loading for bridge evaluation.

The notional vehicle development was based on Weigh-in-Motion data and state DOT survey to capture unusual vehicle configurations that meet legal requirements but cause overstress in bridges. Single-unit multi-axle trucks were identified as a legal load candidate for bridge rating. Notional vehicles that imitate specialized hauling vehicles were proposed, along with live load factors required for bridge evaluation procedures (Sivakumar et al. 2007). From that study, it is seen that states do not have a guide for legal vehicles that do not meet Federal Formula requirements, such as legally overloaded vehicles with grandfather provisions. There are many variations from a federal weight law among the states; that is why evaluating state-specific vehicles with grandfather provisions is important to maintain the safety of bridges.

This study considers the impact of state-specific federal truck size and weight law exemptions on bridge rating procedures. Various vehicle types and commodities can legally operate above federal weight limits without issuing permits. Every state can have different grandfather provisions that can be expressed in the form of additional permissible, load per axle, Gross Vehicle Weight, or exemptions on Federal Bridge Formula. The most common exemptions include vehicles carrying agricultural and farm products and commodities. Federal Highway Administration (FHWA) reports that 41 states provide exemptions for “agricultural vehicles” (FHWA, Freight Management, and Operations, 2019).

The objective of this study is to verify if the MBE accounts for legally overloaded vehicles with grandfather provisions. The state-specific calibration analysis is demonstrated on the example of vehicles carrying agricultural products with a 20% overload allowance in Montana. It provides a methodology to assess effect of legally overloaded vehicles, and provisions to adjust by live load factor(s) for bridge rating. The present study involves WIM

database analysis, identification of vehicles with grandfather provisions, reliability-based calibration, calculation of live load, and proposal for new policy implementation.

4.3.DATA

WIM data is an excellent source of information to evaluate traffic-induced load effects on bridge structures. Nowadays, many WIM sites are operating all over the United States that collect massive traffic data. The weight and configuration of passing vehicles are recorded continuously. The resulting database can be used to develop a live load model for bridge design and evaluation.

In order to assess the impact of state-specific legally overloaded vehicles, there is a need to find WIM stations with the highest level of confidence to capture exempted vehicles. For each considered WIM station, the data has to be filtered to find the vehicles of interest. The filtering criteria depend on grandfather provisions, which can be expressed as an additional axle weight, GVW, percentage of allowable overload, exemptions on Federal Bridge Formula, etc. Based on grandfather provisions, the WIM data can be filtered to find possible legally overloaded vehicles. The state officials can also provide more specific vehicle characteristics such as a typical number of axles and wheelbase range to narrow down the search and capture the vehicles with weight exempts.

4.3.1. Identification of grandfather vehicles

In this study, Great West Engineering Inc. was responsible for identifying the vehicles transporting the agricultural commodities, particularly utilizing the 20% overload provisions in Montana. The goal was to develop a search criterion to help isolate the targeted agricultural commodities from traffic data. Non-interstate WIM stations, which most likely

record targeted traffic, were investigated. The variables yielding the highest probability to obtain the desired vehicular data were compiled together to determine the most feasible WIM locations. **Figure 4.1** presents the map with the location of non-interstate WIM stations in Montana, some agricultural commodity storage facilities, and bridges on the presented road network.

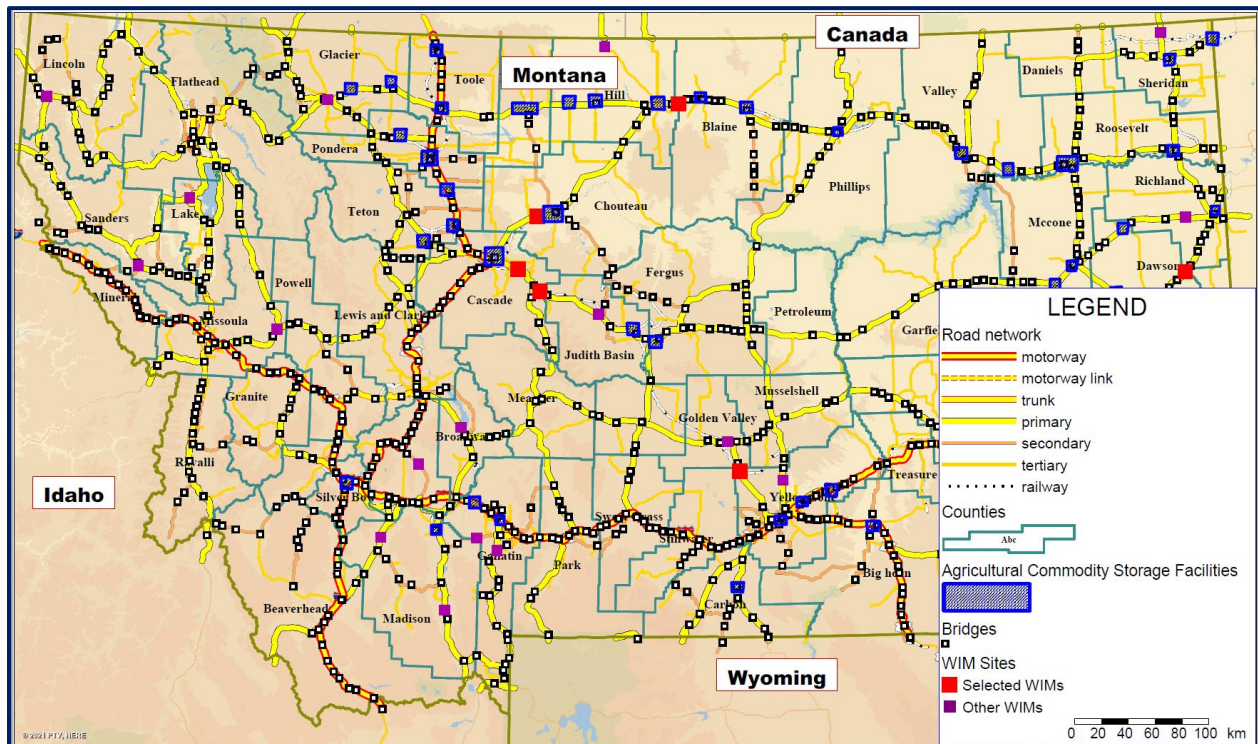


Figure 4.1. Map of selected non-interstate WIM sites and agricultural commodity storage facilities in Montana.

The 26 non-interstate WIM sites were considered to identify sites most likely with vehicles carrying agricultural commodities (alfalfa, hay, wheat, barley, sugar beets, and potatoes). The crop and distribution location after harvesting were considered in WIM site selection. The volume of planted acres of an agricultural commodity within each studied maintenance district was correlated with the probability the commodity will be transported within that district. Therefore, each commodity was assigned with a confidence ranking

based on the likelihood that commodity would be transported within a maintenance district. The confidence ranking and location of agricultural commodity storage facilities locations in proximity to the non-interstate WIM stations were considered. The comparative analysis of location and carried commodities allowed to narrow down the number of WIM sites. The final selection criterion was to compare the Weighted Commercial Annual Average Daily Traffic (CAADT) for selected WIM stations. It was assumed that the higher traffic volume received at the WIM stations provides a greater probability of obtaining the desired vehicular data. Due to a significant range of CAADT within the targeted non-interstate WIM locations, it was assumed that stations with the largest CAADT would be considered for the analysis. The six WIM stations, W-110, W-115, W-116, W-126, W-127, and W-145, were selected for further evaluation (indicated as red squares in **Figure 4.1**).

4.3.2. Weigh-in-Motion data

After selecting WIM sites, the candidates for common truck and trailer configurations capable of transporting the targeted agricultural commodities were examined. The wheelbase, defined as the distance between the center of the front and rear axle(s) of each truck type were researched individually through publicly available DOT information, truck manufacturer websites, and online information. The wheelbase dimensions for each type of truck and trailer are variants. Motor Carrier Services provided wheelbase ranges, and consensus wheelbase dimensions were obtained from online trucking forums. Several truck types were considered and described by Great West Engineering Inc. (Great West Engineering Inc. 2019). Ultimately, two vehicle types were considered, type 6 vehicles with 3-axles and wheelbase in between 4.85–7.32 m (16–24 ft),

and type 9 vehicles with 5-axles and wheelbase between 15.55-21.03 m (51–69 ft). Selected vehicles are shown in **Figure 4.2**.

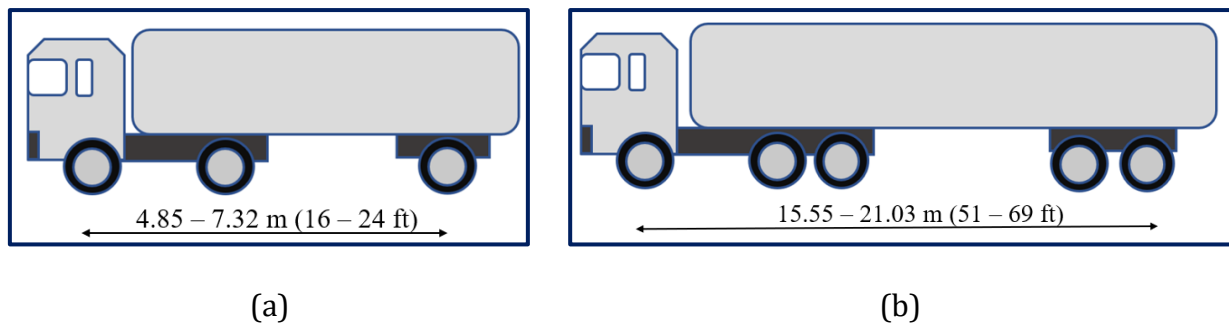


Figure 4.2. Wheelbase range for selected agricultural vehicles (a) Type 6, 3-axle truck, (b) Type 9, 5-axle truck.

The summary of agricultural vehicle search criteria is shown in **Table 4.1**. It includes WIM sites, agricultural commodity type, wheelbase range of selected trucks, and harvesting season.

Table 4.1. Recommended search criteria for agricultural vehicles with 20% overload.

WIM Sites	Agricultural Commodity	Truck Type	Wheelbase range	Harvesting Season
W-115 W-116 W-126 W-127	Wheat / Barley	Type 6	4.85–7.32 m (16–24 ft)	July 22nd – October 4th
W-145	Sugar Beets / Potatoes	Type 6	4.85–7.32 m (16–24 ft)	Sep 19th – Nov 4th
W-110 W-115 W-116 W-126 W-127 W-145	Alfalfa / Hay	Type 9	15.55–21.03 m (51–69 ft)	June 18th – Oct 7th

For selected WIM sites, the data for the years 2007-2019 was used. The availability of data varies each year for different WIM stations. The overall number of available raw records is 28,297,123. The raw WIM data was processed and decrypted to a

user-friendly format. A set of Quality Control (QC) checks was applied to the Montana WIM data. The QC criteria were selected based on the literature review and recommended DOT practices (Nichols and Bullock 2004; O'Brien 2011; Pelphrey et al. 2008; Ramachandran et al. 2011; Sivakumar et al. 2008). The QC criteria include a completeness check to identify any missing data, as some days and months might not be available due to system malfunction.

Additionally, the logical tests were used to capture errors, including axle weight tolerances, minimum and maximum weight, and axle spacing, etc. The last group of checks includes sensor operational problems. Overall, twenty-one QC checks were used to identify questionable records in databases. Each WIM station was checked separately, and the percentage of errors detected by each filtering criteria out of the total number of available records was calculated. Based on the QC results, only records with indicated sensor errors and identical records were eliminated.

The overall number of records available for analysis is 28,297,123. After Quality Control, 7% of the data was discarded. The data used for further analysis includes 26,407,231 records. **Table 4.2** summarizes number of records before and after QC procedure.

Table 4.2. Summary of WIM records before and after Quality Control procedure.

WIM site	Number of WIM records	Number of filtered WIM records	Percentage of discarded WIM records
110	4,414,469	4,100,358	7%
115	3,540,382	3,232,157	9%
116	6,107,141	5,751,983	6%
126	3,004,127	2,681,167	11%
127	5,664,079	5,345,600	6%
145	5,566,925	5,295,966	5%
Total	28,297,123	26,407,231	7%

The detailed summary of selected WIM data for the analysis is shown in **Table 4.3**. The data availability varies from site to site and year to year. It can be caused by system/sensor failure, maintenance and calibrations, changes in weather conditions, detours, road accidents, road closures, and roadworks, etc.

Table 4.3. Availability of WIM records within selected WIM sites for years 2007-2019.

Years	WIM sites					
	110	115	116	126	127	145
2007	125,645	70,356	114,045	84,402	91,585	-
2008	112,883	74,245	122,746	-	-	-
2009	-	66,077	115,066	76,734	80,491	-
2010	-	91,045	132,679	77,066	64,866	-
2011	65,425	102,444	-	86,034	66,338	-
2012	355,253	350,353	-	167,447	320,580	-
2013	589,072	115,405	443,916	251,825	-	341,878
2014	590,027	-	958,691	383,454	340,973	1,069,540
2015	559,799	324,094	1,023,056	393,600	1,093,981	962,136
2016	535,727	921,786	990,376	405,835	1,075,041	772,192
2017	-	-	-	-	-	763,365
2018	625,195	888,230	960,225	395,810	1,097,856	769,649
2019	492,707	170,512	770,843	330,058	894,381	617,206

Also, the selected WIM sites were checked for activity within the specified harvesting season. **Figure 4.3** presents the seasonality of traffic for selected WIM stations. It was noticed that the number of vehicles in the harvesting season is increased, which confirms that targeted agricultural commodities are most active in harvesting season for selected WIM stations. The potential 3-axle and 5-axle agricultural vehicles were filtered from the WIM database records by a number of axles and provided wheelbase ranges. The 3-axle vehicles were captured at five WIM sites, excluding W-110, but 5-axle vehicles were found in all six WIM sites. It was classified by specific agricultural commodities carried by vehicles passing the WIM site, as shown in **Table 4.1**.

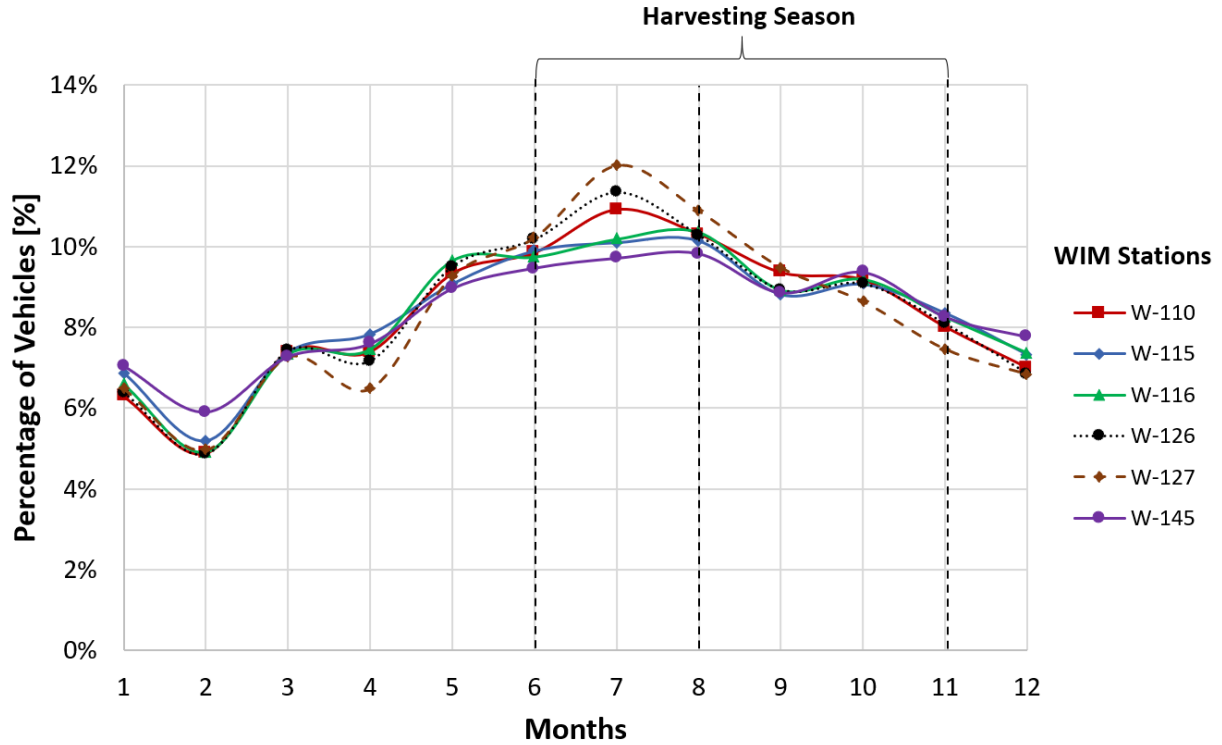


Figure 4.3. Seasonality of traffic for selected WIM stations in 2018

The next step of the data analysis was to capture possible agricultural vehicles with up to 20% overload provisions. To find those vehicles, a four-step classification procedure was developed. Every vehicle was checked by federal law criteria, considering the maximum Gross Vehicle Weight of 36.3 t (80,000 lbs), single axle weight of 9.1 t (20,000 lbs), and tandem axle weight of 15.4 (34,000 lbs).

These weight criteria were checked with a 20% overload allowance to capture vehicles with 20% overload provisions. Then all possible agricultural vehicles were verified by the Federal Bridge Formula (FBF), **Equation (4.1)** (FHWA 2019).

$$W = 500 \left[\frac{(L \cdot N)}{N - 1} + 12N + 36 \right] \quad (4.1)$$

where:

W – the overall gross weight on any group of two or more consecutive axles to the nearest 500 pounds [lbs.],

L – the distance in feet between the outer axles of any group of two or more consecutive axles [ft],

N – the number of axles in the group under consideration.

Federal Bridge Formula was checked for all possible axle combinations. The exemptions of FBF listed in Permissible Gross Load for Vehicles in Regular Operation were considered. **Figure 4.4** summarizes the WIM data analysis to capture possible agricultural vehicles with up to 20% overload provisions.

The identification procedure begins at WIM data collection and cleaning. Afterward, processed data is used to filter 3-axle and 5-axle common agricultural vehicles. These vehicles are grouped based on common wheelbase criteria for vehicles carrying agricultural commodities. Selected vehicles are checked by GVW, single and tandem axle federal law requirements plus 20% overload to identify vehicles above 20% overload allowance. FBF validates vehicles that passed federal weight checks for all possible combinations to filter possible agricultural vehicles with up to 20% overload provisions.

The number of identified 3-axle and 5-axle possible agricultural vehicles is 15,714 and 321,459, respectively. The identified vehicles based on wheelbase criterion and 20% overload allowance are considered in the calibration analysis.

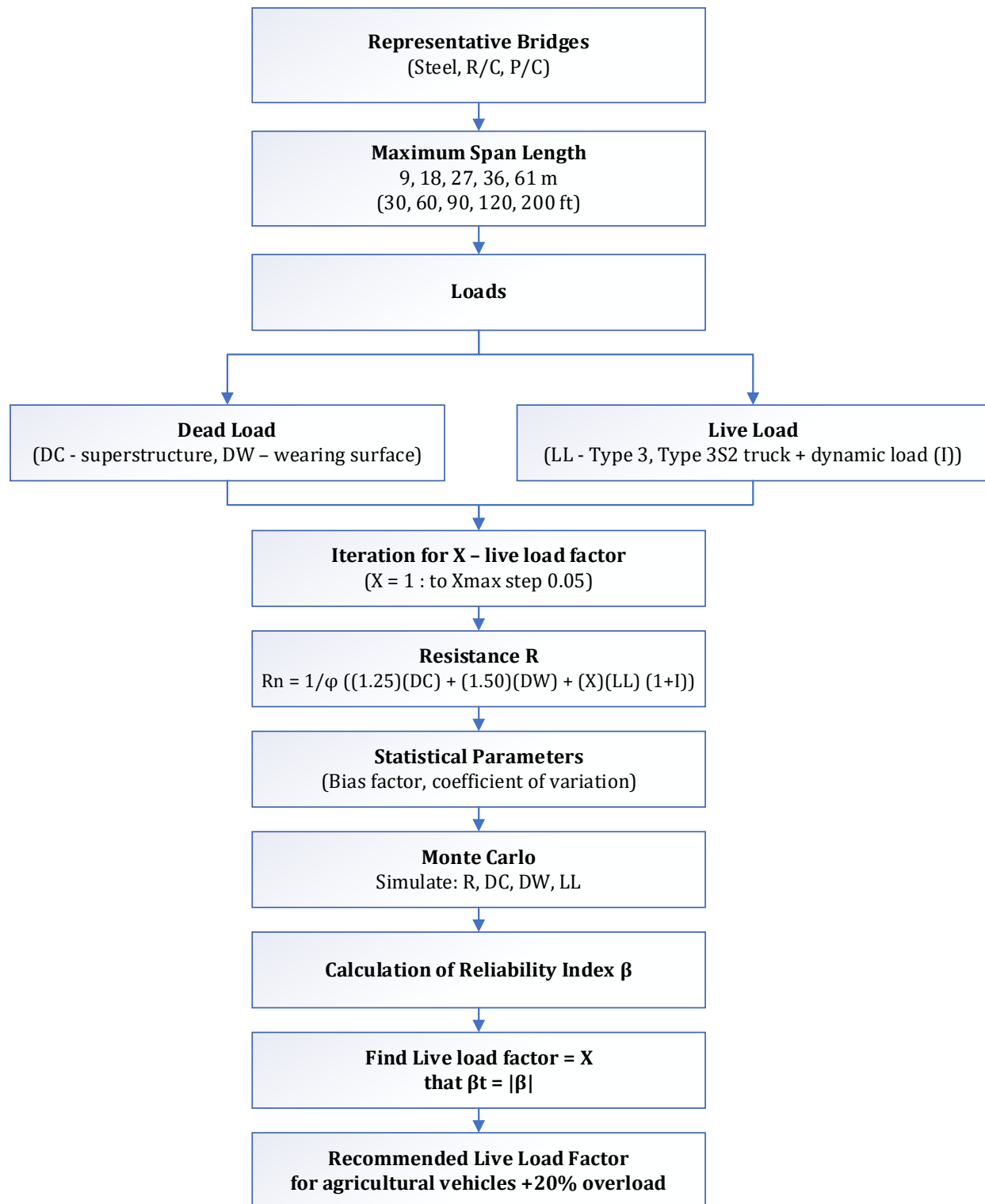


Figure 4.4. Identification of possible agricultural vehicles with 20% overload.

4.4. RESEARCH APPROACH

As presented in the Montana Code, agricultural trucks currently operating in Montana benefit from a 20% overload allowance. However, it is necessary to understand better this allowance's implications on Montana bridges continued safe operation. Analysis of the impact of overload allowance can help bridge owners confidently implement a policy for load rating. It may require the use of new appropriate truck loads and load factors to consider this overload allowance. Initially, the possible agricultural vehicles with 20% grandfather provisions need to be identified in the WIM database. Then, the load effects caused by these vehicles will help to develop recommendations for bridge load rating policy.

The load effects of identified possible agricultural vehicles operating on Montana bridges are computed using the influence line analysis. The influence line analysis was used to find the vehicle's position that causes the maximum load effect. The load effects, i.e., moments and shear, were calculated for representative simply supported bridges with span lengths of bridges 9, 18, 27, 36, 61 m (30, 60, 90, 120, 200 ft). The loading is compared to AASHTO typical legal vehicles to check if agricultural vehicles produce greater load effects. Such comparison allows for verifying if the AASHTO notional load rating is a good representation of exempted vehicles. That may be used to confirm if bridges can support the loads from agricultural vehicles. The methodology is to verify how to use the legal load rating to adjust for grandfather provisions.

The reliability-based calibration analysis starts with the selection of representative bridge structures. Then, the limit state function and the load and resistance random variables have to be identified. To account for the variation of random variables, statistical parameters are determined. The live load statistical parameters include load effect

assessment to predict the mean maximum expected loading. Multiple presence effects for agricultural vehicles are checked to evaluate the governing case for maximum expected load effect. The reliability-based calibration requires selection of the reliability analysis procedure and target reliability index. In this study, the Monte Carlo simulation is used to calculate the live load factor that envelopes load effects caused by agricultural vehicles with a 20% allowance in Montana.

4.4.1. Multiple presence

The multiple presence is a simultaneous occurrence of vehicles on a bridge. It can be a side-by-side event on adjacent lanes or one vehicle following another within one lane. The multiple presence can cause significant load effects, which can be a governing case in the live load model. Therefore, the multiple presence analysis of agricultural vehicles with a 20% load allowance on bridges in Montana was performed. The WIM data provides information about an occupied lane and vehicle time stamp, enabling the multiple presence analysis. The multiple presence is considered for two scenarios: the simultaneous occurrence of vehicles in one and two lanes. The multiple presence analysis was conducted separately for 3-axle and 5-axle possible agricultural vehicles. The multiple presence was checked when two vehicles occur simultaneously on a bridge and when an agricultural vehicle occurs on the bridge along with a random truck with GVW greater or equal to 27.2 t (60,000 lbs). The GVW of the random truck used for multiple presence analysis was selected based on the average inventory rating from the Montana National Bridge Inventory database since inventory rating represents a safe load level on existing bridge structures. The multiple presence analysis for agricultural vehicles with 20% overload provisions is shown in **Figure 4.5**.

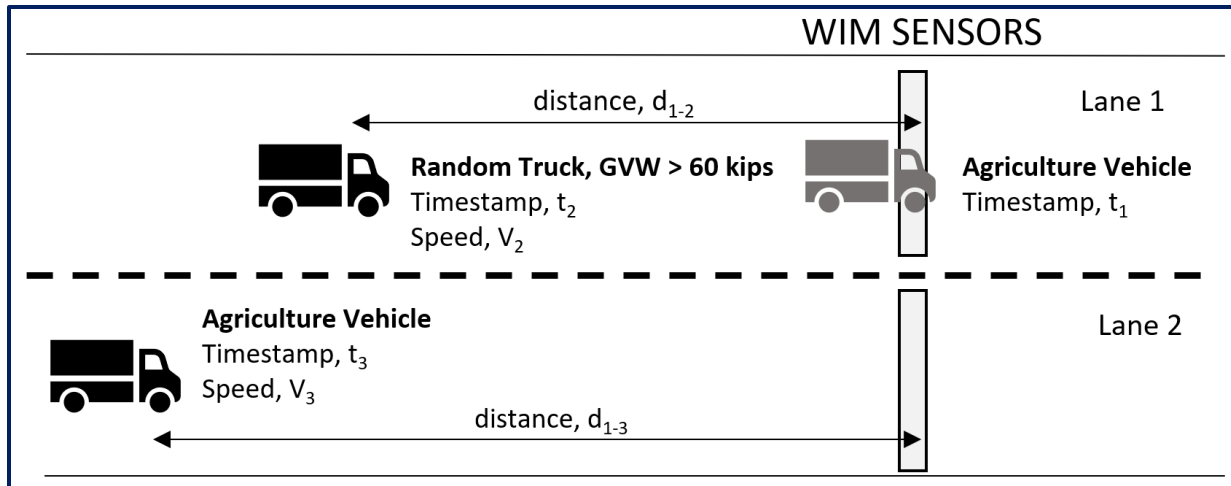


Figure 4.5. Multiple presence for agricultural vehicles with 20% overload.

The multiple presence analysis was based on time stamps recorded by WIM. The timestamp of agricultural vehicles with 20% overload provisions was recorded, and then the timestamp of the following vehicle on the same lane or adjacent lane was considered. The speed of the following vehicles was used to compute the distance between the two vehicles. The speed and distance were used to classify the multiple presence based on the time intervals. The 1, 2, 3, 4 seconds intervals were considered. The traffic in one direction was taken into account in this analysis. It was concluded that the probability of multiple presence of two typical 3-axle and 5-axle vehicles traveling the same direction being on the same bridge is exceptionally low. There were no cases reported for time intervals 1 and 2 seconds. The maximum probability of occurrence of an agricultural vehicle with another random truck with GVW greater than 27.2 t (60,000 lbs) for time intervals of 3 and 4 seconds was at maximum of 0.65%. That indicates that the probability of two agricultural vehicles simultaneously on the bridge is low, and it is not considered in this study.

4.4.2. Load effects

The analysis of load effects is essential in the assessment of a bridge's live load capacity. The traffic load effects depend on many attributes, including bridge span length, vehicle GVW, axle loads, axle spacings, truck traffic volume (ADTT), number of vehicles on the bridge (multiple presence), etc. All identified 3-axle and 5-axle vehicles with 20% allowance were run on an influence line to find the absolute maximum moment and shear for the representative bridge span lengths: 9, 18, 27, 36, 61 m (30, 60, 90, 120, 200 ft).

The load effects for identified possible agricultural vehicles were compared with AASHTO legal trucks: the 3-axle agricultural vehicles were compared with Type 3 trucks, and 5-axle agricultural vehicles were compared with Type 3S2 trucks. The load effects of agricultural vehicles were computed separately for flexure and shear. The analysis of load effects allows for determining the envelope of agricultural vehicles' load effects, and it is used to establish live load statistical parameters for bridge rating.

Figure 4.6 and **Figure 4.7** present an example of a Cumulative Distribution Function (CDF) on a normal probability plot for the ratio of 3-axle agricultural vehicles to Type 3 vehicles and 5-axle agricultural vehicles to Type 3S2 vehicles for a bridge span length of 18 m (60 ft). The distributions of moment and shear ratios were generated for all representative span lengths to determine statistical parameters of the live load. It is observed that the moment ratios plotted for different WIM station data have almost the same distribution. It concludes that agricultural vehicles do not vary from site to site.

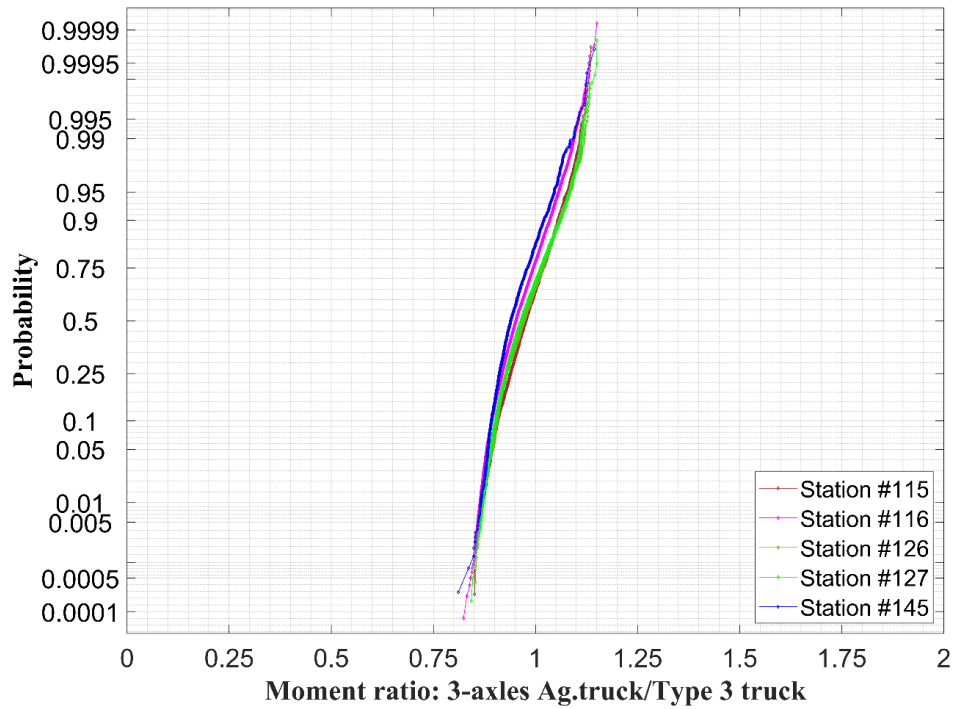


Figure 4.6. CDF of moment ratio of 3-axle agricultural vehicles to Type 3 vehicle for a bridge span length of 18 m (60 ft).

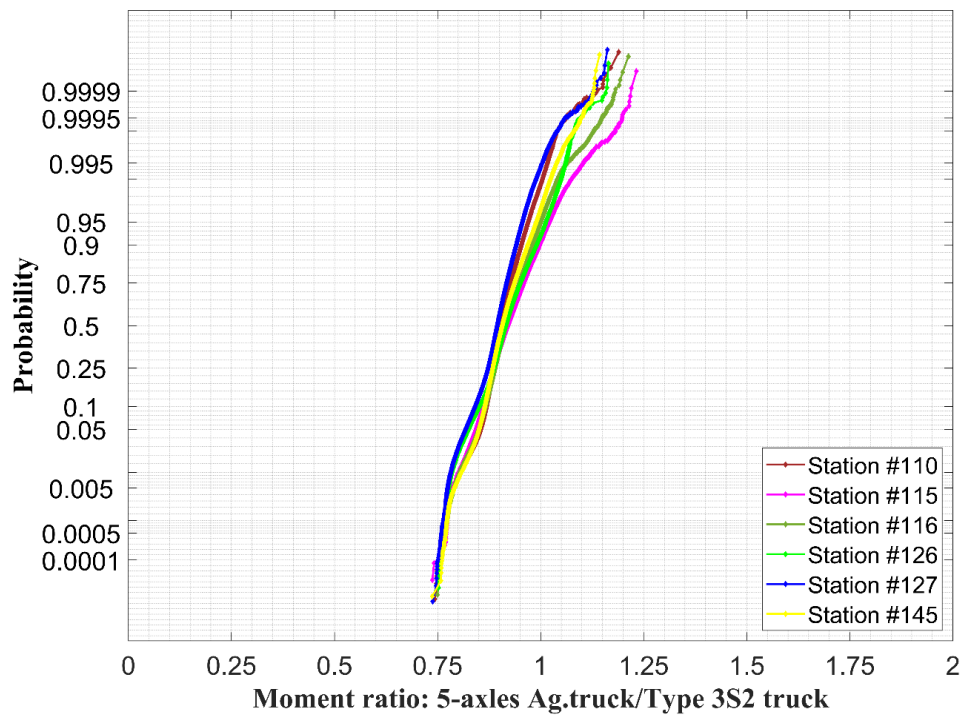


Figure 4.7. CDF of moment ratio of 5-axle agricultural vehicles to Type 3S2 vehicle for a bridge span length 18 m (60 ft).

4.4.3. Statistical parameters of load and resistance

Statistical parameters are essential in reliability-based calibration. Hence, they define the level of uncertainty of random variables that appear in the limit state function. In this study, Strength Limit State I for bridge rating is considered. The random variables considered in the limit state function are dead load, live load, dynamic load, and resistance. The statistical parameters, such as a bias factor (ratio of mean-to-nominal/design value) and a coefficient of variation (ratio of standard deviation and the mean), are to be determined for all considered random variables. The statistical parameters of dead load are grouped as prefabricated steel and prestressed concrete elements (D1), site-cast concrete (D2), and wearing surface (D3). The values of statistical parameters are taken from the original AASHTO calibration (NCHRP 368). The dead load is assumed to be normally distributed.

The statistical parameters of live load are determined for the considered agricultural vehicles with a 20% overload allowance. The development of statistical parameters for live load models is a complex process. It required analysis of a large traffic database, calculations of load effects, and the projection of maximum expected load effect. It also includes the evaluation of the dynamic load allowance. The live load is a time-varying variable, so its characteristics as a random variable depend on time period or a specific return period. In this research, the live load model is determined for a 2-years return period since it is a typical period between Montana's bridge inspections. The statistical parameters are determined separately for 3-axle and 5-axle agricultural vehicles.

Based on the WIM data analysis, approximately 15,000 of 3-axle agricultural vehicles were found for the 12-years' worth of WIM data. Five WIM stations were considered, which means that on average, there are 3,000 agricultural vehicles per one WIM station.

For the 12 years of data, there are 3,000 vehicles, so for a 2-years period, there are about 500 vehicles. Considering a 2-year return period, the probability of occurrence can be expressed as $1-(1/500)$, which gives a probability of 0.998. For each WIM station and bridge span length, the moment and shear ratio were calculated, corresponding to the probability of 0.998. Those moments and shear ratio represent the maximum expected load effects of 3-axle agricultural vehicles with 20% overload provisions.

Similarly, the statistical parameters of 5-axle agricultural vehicles were developed. The WIM data analysis captured approximately 320,000 of 5-axle vehicles for the 12-years' worth of WIM data. The six WIM stations were considered, which means that there are 53,000 agricultural vehicles recorded per one WIM station. For the 12 years of data, there are about 53,000 vehicles, so for the 2-years period, there are about 9,000 vehicles. Considering a 2-years period, the probability of occurrence can be expressed as $1-(1/9,000)$, which gives a probability of 0.999(8). For each WIM station and bridge span length, the moment and shear ratio were calculated corresponding to the probability of 0.999(8) probability were found. Those moment and shear ratios represent the maximum expected load effects of 5-axle agricultural vehicles with 20% overload provisions.

The obtained load effect ratios for the 3-axle and 5-axle agricultural vehicles are compared to Type 3 and 3S2 trucks, and the ratios are taken as a bias factor of live load. For 3-axle agricultural vehicles, the bias factor varies from 1.13-1.18 and 1.17-1.20 for flexure and shear, respectively. For 5-axle agricultural vehicles, the bias factor varies from 1.16-1.26 and 1.13-1.28 for flexure and shear, respectively. Computed bias factors for different bridge span lengths are used in the calibration of the live load factor.

The coefficient of variation is another parameter required to define live load as a random variable. To find a live load variation, the upper tail of the CDF plots is considered. The upper tail's standard deviation and mean value were found for each WIM station database and bridge span length. Knowing those parameters, the coefficient of variation of the live load was computed.

Also, the dynamic load effect was considered. The dynamic impact depends on road surface roughness, bridge dynamics (frequency of vibration), and vehicle dynamics (suspension system). The derivations of dynamic load effects were based on the numerical simulations using an equivalent static load effect (Nowak 1999), further confirmed by field testing (Nowak and Nassif 1995). Based on NCHRP 368, the mean value of the dynamic load is taken as 20% of the mean value of the live load. The standard deviation of the dynamic load is 80% of the mean value of the live load. The standard deviation of dynamic load and live load is calculated as the square root of the sum of the variance of live load and dynamic load. For all bridge span and WIM site cases, the coefficient of variation of live load with dynamic load is 14%. Live load is represented by Extreme Type I (Gumbel) distribution. This distribution represents fluctuating traffic-induced load effects to characterize the maximum of sample data.

Statistical parameters of resistance depend on material properties, quality of workmanship, and analytical model. Thus, resistance is a product of material properties, fabrication, and analysis factors. The variation of resistance is evaluated by tests, simulations, and observations. The uncertainties in a random variable representing resistance are determined to ensure an adequate safety margin. In this study, the statistical parameters of resistance are taken from NCHRP 368. The resistance parameters account for

three main bridge materials: steel, reinforced concrete, and prestressed concrete. **Table 4.4** summarizes the statistical parameters for random variables along with the distribution type.

Table 4.4. Summary of statistical parameters of load and resistance random variables.

Random variables	Bias factor	COV	Distribution	Source
Dead load, prefabricated	1.03	8%	normal	NCHRP 368
Dead load, site-cast	1.05	10%	normal	NCHRP 368
Dead load, wearing surface	1.00	25%	normal	NCHRP 368
Live load + dynamics, 3-axle vehicles for flexure	1.14-1.21	14%	Extreme Type I	this study
Live load + dynamics, 3-axle vehicles for shear	1.15-1.22	14%	Extreme Type I	this study
Live load + dynamics, 5-axle vehicles for flexure	1.16-1.26	14%	Extreme Type I	this study
Live load + dynamics, 5-axle vehicles for shear	1.10-1.28	14%	Extreme Type I	this study
Steel resistance for flexure	1.12	10%	Lognormal	this study
Steel resistance for shear	1.14	10.5%	Lognormal	NCHRP 368
Reinforced concrete resistance for flexure	1.14	13%	Lognormal	NCHRP 368
Reinforced concrete resistance for shear	1.20	15.5%	Lognormal	NCHRP 368
Prestressed concrete resistance for flexure	1.05	7.5%	Lognormal	NCHRP 368
Prestressed concrete resistance for shear	1.15	14%	Lognormal	NCHRP 368

4.5. RESULTS AND DISCUSSION

The current bridge rating procedures provide safety in terms of load and resistance factors determined using reliability-based calibration analysis. The calibration procedure allows for determining appropriate load and resistance factors that provide a uniform safety level. The acceptance criterion for load and resistance factors is to keep the probability of failure at an acceptably low level. In practice, rather than a probability of failure, the structural condition can be defined in terms of a reliability index, β . Calculation of the reliability index requires knowledge of statistical parameters of load and resistance.

The load and resistance factors are selected so that the designed structures have at least the minimum acceptable reliability, i.e., β being not less than the target reliability index, β_T . The target reliability index depends on the consequences of bridge failure by

exceeding the limit state and increased safety costs. The cost associated with an increased safety level for existing bridges may be greater than for the design of new bridges.

4.5.1. Reliability-based calibration

The reliability-based calibration procedure is used to determine load and resistance factors that result in a desirable safety level defined by the target reliability index.

The calibration procedure can include the following steps:

1. Select representative bridges.
2. Formulate the limit state function.
3. Determine the statistical parameters of load and resistance.
4. Select the target reliability index.
5. Choose and perform the reliability analysis procedure.
6. Select appropriate load and resistance factors.

As presented in the calibration steps, representative bridge structures need to be selected. In this study, over one hundred representative steel, reinforced concrete (R/C), and prestressed concrete (P/C) bridges were used in the calibration procedure.

The nominal dead load effects for representative bridges for flexure and shear are taken from NCHRP 368. The representative bridges are grouped by lengths: 9, 18, 27, 36, 61 m (30, 60, 90, 120, 200 ft). The limit state function has to be formulated, and random variables representing load and resistance need to be identified. The Strength Limit State I for bridge rating evaluation for legal loads is considered. The summary of statistical parameters of load and resistance factors is presented in **Table 4.4**.

The target reliability index needs to be selected to establish the minimum required safety. The performance of existing bridges, consequences of exceeding the limit

state function, and increased safety cost were considered to determine the target reliability index. The target reliability index for bridge evaluation is set as 2.50, which corresponds to a 1% probability of failure (Moses 2001).

Reliability is calculated using the Monte Carlo simulation. Live load factors are then determined that envelope load effects caused by agricultural vehicles with a 20% overload allowance in Montana. The Monte Carlo simulation is an efficient tool for complex problems, where closed-form solutions are either not possible or extremely difficult. It allows considering the distribution of the load and resistance random variables, and more precise results can be obtained. The mean values of random variables are simulated using suitable distribution and statistical parameters. The nominal value of dead load was taken from NCHRP 368. The nominal live load effects for representative bridge span lengths were computed for Type 3 and Type 3S2 trucks. The nominal resistance was calculated from **Equation (4.2)** using the appropriate resistance factor depending on material type for flexure and shear (AAHSTO LRFD 2020). The resistance factors for steel bridges are taken as 1.00. For reinforced concrete, the resistance factor is 0.90, and for prestressed concrete, it is 1.00 for flexure and 0.90 for shear. Nominal resistance is computed based on the load demand as:

$$R_n = \frac{1}{\phi} [1.25(DC) + 1.50(DW) + X(LL)(1 + I)] \quad (4.2)$$

where:

R_n – nominal resistance,

ϕ – resistance factor,

DC – weight of the dead load,

DW – weight of the wearing surface,

LL – live load (Type 3 and Type 3S2 trucks),

I – dynamic load (33%),

X – live load factor for agricultural vehicles.

The Monte Carlo simulation is used to calculate the mean values of random variables. For each considered bridge, one million simulations were generated. The iterative procedure was utilized to find the optimal live load factor. An algorithm was developed to find the live load factor that produces a reliability index of 2.50. Load and resistance factors are to be rounded to the nearest 0.05. Thus, the iteration step is taken as 0.05. **Figure 4.8** presents the calibration procedure to find the live load factor that envelopes load effects caused by agricultural vehicles with up to 20% overload in Montana. Agricultural vehicles are compared with 3-axle (Type 3) and 5-axle (Type 3S2) trucks for flexure and shear. For each considered case, the optimum live load factor and the corresponding value of the reliability index were calculated.

4.5.1. Live load factor selection

The reliability-based calibration analysis was applied to determine the optimum live load factors corresponding to the target reliability index of 2.50. **Table 4.5** presents calculated live load factors and reliability indices for 3-axle and 5-axle agricultural vehicles. The live load factors were calculated separately for representative bridge groups based on material type (steel, R/C, and P/C) and span lengths 9, 18, 27, 36, 61 m (30, 60, 90, 120, 200 ft). The statistical analysis of Montana bridges was conducted using the National Bridge Inventory database for the year 2019.

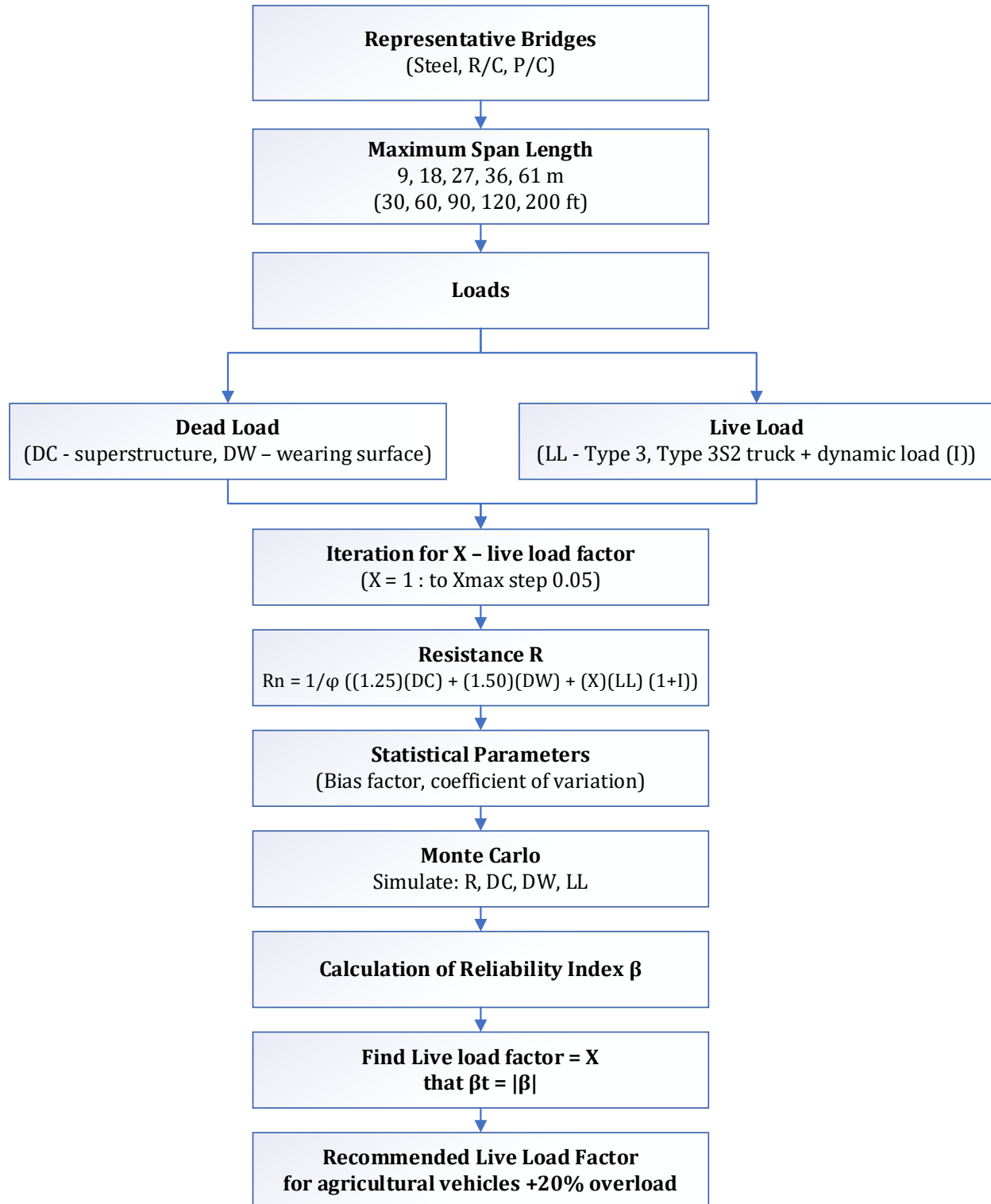


Figure 4.8. Reliability analysis procedure based on the Monte Carlo Simulation.

Table 4.5. Calculated live load factors for target reliability index of 2.50.

Span Length [m]	Percentage of bridges in Montana	3-axle Agricultural Trucks				5-axle Agricultural Trucks			
		flexure		shear		flexure		shear	
		Live load factor	Reliability index [β]	Live load factor	Reliability index [β]	Live load factor	Reliability index [β]	Live load factor	Reliability index [β]
Steel Bridges									
9	5.4%	1.40	2.51	1.40	2.54	1.40	2.45	1.35	2.44
18	9.6%	1.35	2.50	1.35	2.50	1.40	2.55	1.35	2.49
27	5.2%	1.30	2.48	1.30	2.50	1.35	2.49	1.30	2.50
36	2.5%	1.30	2.53	1.30	2.54	1.35	2.53	1.30	2.55
61	5.9%	1.45	2.47	1.45	2.53	1.45	2.48	1.45	2.53
R/C Bridges									
9	7.7%	1.25	2.50	1.25	2.46	1.25	2.43	1.25	2.49
18	3.4%	1.20	2.51	1.25	2.56	1.25	2.56	1.25	2.56
27	1.3%	1.15	2.54	1.15	2.48	1.20	2.53	1.15	2.47
36	0.1%	1.10	2.53	1.10	2.47	1.15	2.50	1.10	2.47
61	0.1%	1.25	2.51	1.15	2.50	1.25	2.47	1.15	2.49
P/C Bridges									
9	4.3%	1.45	2.45	1.30	2.53	1.50	2.53	1.30	2.56
18	20.9%	1.45	2.55	1.25	2.50	1.45	2.48	1.25	2.50
27	23.1%	1.40	2.52	1.20	2.53	1.45	2.54	1.20	2.53
36	8.4%	1.40	2.55	1.15	2.51	1.40	2.44	1.15	2.50
61	2.1%	1.60	2.54	1.20	2.50	1.55	2.47	1.20	2.50
Weighted average		1.40	2.55	1.30	2.55	1.45	2.50	1.30	2.55

Bridges were selected based on material type and maximum span length. The second column in **Table 4.5** presents the percentages of representative bridges in Montana for each material type and span category. The statistics of bridges in Montana were used to calculate the weighted average live load factor to reflect the share of representative bridge categories.

The optimum live load factors vary from 1.10 to 1.60 for flexure and 1.10 to 1.45 for shear. From the weighted average, the maximum live load factor is 1.45. To maintain a uniform safety level for bridges in Montana, the reliability indices for live load factor of

1.45 are calculated for all considered cases. **Table 4.6** presents the reliability index for 3-axle and 5-axle vehicles for flexure and shear.

Table 4.6. Reliability index for a live load factor of 1.45 for representative bridges.

Span length [m]	Percentage of bridges in Montana	3-axle Agricultural Trucks		5-axle Agricultural Trucks	
		flexure	shear	flexure	shear
		Reliability index [β]			
Steel Bridges					
9	5.4%	2.65	2.68	2.58	2.71
18	9.6%	2.75	2.75	2.68	2.75
27	5.2%	2.83	2.84	2.73	2.84
36	2.5%	2.83	2.84	2.75	2.84
61	5.9%	2.48	2.53	2.48	2.54
R/C Bridges					
9	7.7%	3.00	2.96	2.94	2.98
18	3.4%	3.10	2.99	3.03	2.99
27	1.3%	3.16	3.03	3.08	3.03
36	0.1%	3.16	3.01	3.09	3.01
61	0.1%	2.85	2.80	2.85	2.80
P/C Bridges					
9	4.3%	2.45	2.91	2.38	2.93
18	20.9%	2.55	2.96	2.47	2.97
27	23.1%	2.65	3.02	2.54	3.02
36	8.4%	2.66	3.01	2.56	3.00
61	2.1%	2.25	2.77	2.25	2.77
Weighted average		2.70	2.95	2.60	2.95

It is noticed that for almost all considered cases, the reliability index is above the target reliability index of 2.50. Only for 5-axle vehicles on P/C bridges in flexure, the calculated reliability index is slightly below 2.50. Also, for some cases, for a span length of 61 m (200 ft), the corresponding values of the reliability index are below the target reliability index. In conclusion, live load factor of 1.45 for most cases results in a reliability index above the target reliability index of 2.50, but it does not drop below the minimum required reliability index value of 1.50. Therefore, live load factor of 1.45 is recommended for bridge evaluation for agricultural vehicles, with a 20% overload allowance in Montana.

4.6.IMPLEMENTATION

Posting decisions are made based on rating factor calculations. If the rating factor is less than 1.0, a bridge posting may be necessary. The proposed calibration methodology includes identification of possible legally overloaded vehicles, calculations of load effects, and live load statistical parameters development. The calibration is to determine a live load factor that envelope load effects caused by exempted vehicles. The proposed methodology is applicable only if legally overloaded vehicles produce load effects greater than the effect caused by existing rating vehicles. The proposed calibration procedure is complex. It requires the identification of vehicles with grandfather provisions utilizing overload exempts. This process involves the input from Motor Carrier Services, truck manufacturers, and DOT to find the WIM sites with the highest level of confidence to capture exempted vehicles. The grandfather vehicle type, dimensions, and weight characteristics need to be determined. The traffic data analysis can be numerically intense to process a large database, compute load effects, and conduct the Monte Carlo Simulation. However, the implementation of state rating policies is simple. The suggested approach was developed in a manner that would minimize cost and effort. To achieve that, a single factor is proposed.

The live load factor used for AASHTO legal vehicles in Montana is 1.30 (based on an ADTT of 1,000 or less, ref. MBE Table 6A.4.4.2.3a-1), and live load factor for agricultural vehicles determined in the calibration procedure is 1.45. Therefore, a typical live load factor of 1.30 can be compared with the calibrated live load factor, which gives a “scale factor.” That can serve as the evaluation tool for the adequacy of a bridge to carry legally overloaded vehicles. If the recommended live load factor for both 3-axle and 5-axle agricultural vehicles (1.45) is divided by the typical live load factor for the Type 3 and 3S2 vehicles (1.30), the

resulting scale factor is 1.12. To compute the LRFR rating factor for the 3-axle or 5-axle agricultural vehicles, the Type 3 or Type 3S2 rating factors can be divided by 1.12. The resulting rating factor can then be multiplied by the standard truck tonnage for the Type 3 (25 tons) or Type 3S2 (36 tons) vehicle to determine Safe Load Capacity (SLC).

Examples of a rating factor calculation according to LRFR for agricultural vehicles are provided. For instance, the bridge's most recent load rating is 1.10 for the Type 3 vehicle and a 1.29 rating factor for the Type 3S2 vehicle. To check the rating factor and live load capacity for legally overloaded 3-axle vehicles, the following calculation needs to be completed:

- Type 3 vehicle LRFR Rating Factor / 3-axle agricultural vehicle LRFR Rating Factor, that is: $1.10 / 1.12 = 0.98$;
- 3-axle agricultural vehicle LRFR Rating Tonnage (SLC) = $0.98 \times 25 \text{ tons} = 24.5 \text{ tons}$.

The rating factor and SCL calculation for legally overloaded 5-axle agricultural vehicles is:

- Type 3S2 vehicle LRFR Rating Factor / 5-axle agricultural vehicle LRFR Rating Factor, that is: $1.29 / 1.12 = 1.15$;
- 5-axle Ag Vehicle LRFR Rating Tonnage (SLC) = $1.15 \times 36 \text{ tons} = 41.4 \text{ tons}$.

The presented example of LRFR calculations shows that state DOTs can easily implement this method. The calibrated live load factors may be utilized along with typical legal vehicles used in bridge rating procedures. This approach allows bridge owners to simply evaluate bridges under excessive load effects caused by legally overloaded vehicles. It can serve as a tool for checking the adequacy of a bridge to carry vehicles with grandfather provisions and help to make bridge posting decisions and maintain bridges in good condition at required safety level.

4.7.CONCLUSIONS

This study assesses the impact of state-specific federal law exemptions on vehicle weight limits on bridge rating procedures. Various types of vehicles can legally operate above federal weight limits without issuing permits. It was noticed that bridge owners do not have a guide for bridge rating under legally overloaded vehicles with grandfather provisions. Therefore, this study provides a methodology to verify if the load effect caused by exempted vehicles should increase the live load factor for a bridge rating to maintain the minimum required safety level. The reliability-based calibration methodology is demonstrated in an example of agricultural vehicles with a 20% overload allowance in Montana. The proposed procedure identifies legally overloaded vehicles in the WIM database and calculates their load effects on representative bridges. The Monte Carlo simulation technique is used to calculate the optimum live load factors for bridge evaluation procedures. The proposed calibration of legally overloaded vehicles under grandfather provisions leads to conclusions and recommendations for bridge posting practices. The procedure is transparent and can be used to verify the impact of state-specific load effects of any legally overloaded vehicles.

The reliability-based calibration led to calculation of live load factor for bridge rating under exempted loads. The calculated bias factor for agricultural vehicles varies from 1.13-1.26 and 1.13-1.28 for flexure and shear, respectively. The coefficient of variation of live load and dynamic load is 14%. The proposed reliability-based calibration uses the Monte Carlo simulation. An algorithm was developed to find the live load factor that produces a reliability index of 2.50. The optimal live load factor and corresponding reliability index need to be calculated for representative bridge types and span lengths to determine the

recommended live load factor for vehicles with grandfather provisions. For agricultural vehicles in Montana, with a 20% allowance, the recommended live load factor is 1.45.

The proposed calibration methodology is complex, but the implementation of state rating policies is simple. It is recommended to use the existing notional vehicles and adjust the live load factor to maintain the minimum required safety level established by the target reliability index. The suggested method uses a single scale factor. The AAHSTO live load factor of 1.30 is compared with the calibrated live load factor for legally overloaded vehicles, 1.45. This scale factor is a ratio of AASHTO legal load to the calibrated live load factor, which is 1.12. The AASHTO legal rating factor is then divided by the scale factor to obtain the rating factor. Rating tonnage is determined by multiplying the resulting rating factor by the tonnage of rating vehicles.

In summary, the proposed methodology provides a guide to assess the load effect caused by legally overloaded vehicles. It helps bridge owners to account for excessive load effects and implement it in a new rating policy. Montana's bridges are subjected to loads from agricultural vehicles that create load effects that exceed those caused by existing legal rating vehicles. However, consideration of these vehicles does not require a re-analysis of the state bridge inventory. Linear scaling of existing rating results from bridge records can be used to determine ratings for agricultural vehicles. These results can then be used in conjunction with the existing state posting policy to make the necessary posting decisions with the minimum cost and effort.

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CHAPTER 5 SUMMARY

5.1. Conclusions and Remarks

The presented research considered the impact of overloaded vehicles on bridge structures. The need for study is due to outdated permit fee regulations, lack of uniform permit provisions throughout the United States, no guidance for bridge evaluation under legally overloaded grandfather vehicles, and inconsistent procedures for developing notional permit vehicles for bridge rating.

The major research contributions of this dissertation include:

1. Incremental consumption formula was developed to assess fatigue damage caused by overloaded permit vehicles. It provides a rational basis and efficient tool to assess dollar consumption caused by overloaded permit vehicles.
2. Reliability-based method was developed for selection of a representative notional permit vehicle, which can also be used in bridge design for the Strength Limit State II, and bridge evaluation under permit loading, and help with permit vehicle issuance.
3. The notional permit vehicle development offers a beneficial and cost-effective process for the state DOTs to reduce individual permit analyses to issue a permit.
4. Bridge evaluation procedure under legally overloaded grandfather vehicles was presented to help bridge owners account for excessive load effects and implement it in a new rating policy. Linear scaling of an existing rating was proposed to minimize cost and effort in making necessary posting decisions.

The research presented in this dissertation shows a significant contribution to advance the state of practice on the impact of overloaded vehicles on the bridge structure. It provides an advanced methodology to improve monitoring and control of heavy traffic

operations, enhance enforcement, and maintain a good condition of bridges by improving design and evaluation procedures. These results are useful for state agencies and other bridge owners, bridge designers and inspectors, code developers, and the research community.

5.2. Future Research

Presented studies on the impact of overloaded vehicles on bridges provide a basis for improving the permit fee regulations, controlling the permit-induced load effects by representative notional permit vehicles, and utilizing them in design, evaluation, and permit issuance procedures. Also, it can be helpful in bridge evaluation under legally overloaded vehicles under grandfather exemption. The research provides a scientific basis to control the damage and maintain the required safety for bridge users. However, the studies presented in this dissertation have the potential to be expanded in future research. All research studies have limitations, but the step-by-step enhancement of the previous assumption allows for progress. The ideas for future research are presented for all three studies:

1. The proposed approach to assess the monetary consumption of permit vehicles, derived by the incremental consumption formula, has its limitations as it does not consider the selected route-specific costs, but it is the weighted average value for the state's representative bridges. Therefore, the consumption-based approach can be expanded using more advanced analysis. Individual permit fees can be calculated based on a vehicle configuration and a permitted route. The derived consumption formula can compute the individual consumption of the permit vehicles and assess a cost for a specific route. The selected route has a specified number of bridges, which allows for determining a total permit fee. This approach utilizes a Geographic

Information System (GIS), which could help to select optimal routes to help DOT maintain the bridge condition. Also, GIS technology offers the flexibility for the trucking industry to choose alternative routes and manage the permit fee cost. Moreover, it helps DOTs control the operation of permit vehicles and prevent premature and accelerated damage caused by permit vehicles, which consume bridge and pavement design life much faster than regular traffic. The individual permit fee schedule maximizes equity, so every permit pays for the actual damage caused to the infrastructure. The advanced technology can monitor bridge and pavement conditions based on the number of issued permits, vehicle configurations, and calculated monetary damage to infrastructure.

2. The proposed reliability-based method to find a notional permit vehicle(s), which provides uniform safety for a wide range of bridge structures, is complex and requires analyzing big datasets. The approach is consistent with the AASHTO LRFD and LRFR provisions. Future research can involve calibration for the selected notional permit vehicles for Strength Limit State II and Rating Factor for single and multi-trip permits. It would require multiple presence analyses of permit vehicles, dynamic impact testing, and the girder distribution factor analysis for permit vehicles. Additionally, it will involve selecting a target reliability index for permit loading for considered limit state functions. The target reliability index defines the minimum acceptable safety, which depends on the consequences of failure and cost analysis. The cost associated with increased safety for an existing bridge can be greater than for a new design. Control of permit vehicles' operation and careful consideration of their load effect in

designing and evaluating bridges are crucial to providing the minimum required safety of bridges.

3. There are no provisions for bridge evaluation under state-specific legally overloaded grandfather vehicles. Thus, the reliability-based method was developed to determine state-specific live load factors to maintain the required minimum safety level with the minimum cost and effort. Further research could consider the load effect on continuous bridges to capture negative moment effects. Furthermore, it would be beneficial to assess economic impact to justify the existence of grandfather exemptions.

APPENDIX 1

Data sources and developed algorithms

Data	Source
Florida Permit Data for, years 2016-2019	Florida Department of Transportation, Nov. 2019, from Permit Application System (PAS). Within the project Ali, H., Nowak, A. S., Stallings, J. M., Chmielewski, J., Stawska, S., Babu, A. R., and Haddadi, F. (2020). "Impact of Heavy Trucks and Permitted Overweight Loads on Highways and Bridges Now and in the Future versus Permit Fees, Truck Registration Fees, and Fuel Taxes."
Florida Weigh-in-Motion Data for 2012-2017	Florida Department of Transportation.
Florida Weigh-in-Motion Data for 2016-2020	Federal Highway Administration, September 2020.
Montana Weigh-in-Motion Data for 2007-2020	Montana Department of Transportation, through project "Agricultural loads on Montana bridges," conducted by AECOM and Auburn University, July 2020.
Agricultural vehicles characteristics in Montana	Great West Engineering Inc. prepared a memo to identify vehicles transporting the agricultural commodities, particularly utilizing the 20% overload provisions in Montana. The goal was to develop a search criterion to help isolate the targeted agricultural commodities from traffic data. Findings are documented in Green West Engineering Inc. (2019). "MDT Agricultural Loads – Amendment 1 Research."
National Bridge Inventory for 1992-2020	Public domain database with detail information about bridges in the US.

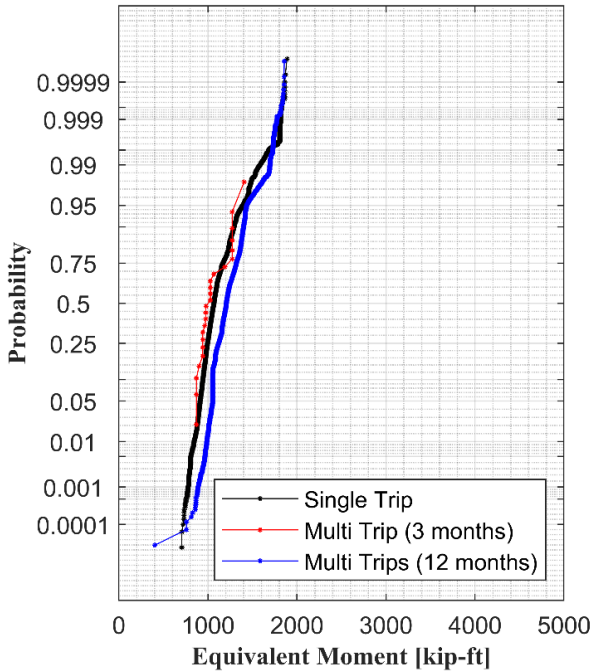
Algorithm	Description
WIM data processing	WIM data conversion to a user-friendly format.
Quality Control	A program to verify WIM records and eliminate errors, including data description, timestamp, duplicates, and logical checks.
Simple Span load effects	Analytical program based on influence line analysis to calculate moment and shear effects on simply supported span for various vehicle configurations.
Continuous Span load effects	Analytical program based on influence line analysis to calculate positive and negative moment and shear effects on simply supported span for various vehicle configurations.
Equivalent single moment - reflow	Algorithm calculates an equivalent single moment using a rainflow counting method based on ASTM E1049-85 to determine the number and magnitude of moment cycles resulting from vehicle passage.
Bridge Consumption	Code based on derived bridge consumption equation to find dollar damage caused by permit vehicle.
Incremental Consumption	Code based on derived bridge incremental consumption equation to find dollar damage caused by overloaded part of permit vehicle.
Overloaded WIM vehicles	Algorithm, which filters overloaded WIM vehicles based on federal truck size and weight law. It includes four criteria: maximum Gross Vehicle Weight of 36.3 tones (80 kips), maximum single axle weight of 9.1 tones (20 kips), maximum tandem axle weight of 15.4 tones (34 kips), and Federal Bridge Formula (FBF).
Vehicle histogram characteristics	Code allows creating a histogram for the selected ranges to evaluate different vehicle characteristics, including total weight, axle weight, axle spacing, and load effects.
Notional vehicles generator	Code generating notional vehicles candidates for various configurations, based on an iterative procedure.
Load effect ratio	Code which calculates load effects ratios for given traffic database and selected notional design and evaluation trucks.
Distribution fitting	Algorithm which determines best-fit distribution for load effect ratio based on a negative log-likelihood parameter, and Bayesian and Akaike criteria. The algorithm checked the best fit of the following distribution types: Beta, Exponential, Extreme Value, Gamma, Generalized Extreme Value (GEV), Generalized Pareto, Inverse Gaussian, Logistic, Loglogistic, Lognormal, Nakagami, Normal, Rayleigh, Rician, Weibull.

Development of statistical parameters	Extrapolation method to determine maximum expected live load ratio for considered traffic and notional vehicle. It is based on Average Daily Truck Traffic, return period, and distribution type.
Notional vehicles selection	Algorithm which select notional permit vehicle candidate based on: 1) the lowest variation of bias factor for a moment effect, 2) the lowest variation of bias factor for shear effect, 3) the lowest percentage of vehicles above 1.0 ratio.
Monte Carlo Simulation	The component of limit state function was simulated using their distribution type and statistical parameters (bias factor and coefficient of variation) to simulate the magnitude of load and resistance required for reliability-based analysis.
Reliability-based analysis	Simulation-based analysis to determine: a) reliability index for set live load factor, b) live load factor for target reliability index.
Filtering agricultural vehicles	Filtering algorithm to find agricultural vehicles based on developed wheelbase criteria and allowable 20% overload provisions.
Multiple presence	The multiple presence is a check of simultaneous occurrence of vehicles on a bridge. It can be a side-by-side event on adjacent lanes or one vehicle following another within one lane. The multiple presence analysis was based on time stamps recorded by WIM. The timestamp of a vehicle was recorded, and then the timestamp of the following vehicle, on the same lane or adjacent lane was considered. The speed of the following vehicles was used to compute the distance between the two vehicles. The speed and distance were used to classify the multiple presence based on the time intervals. The 1, 2, 3, 4 seconds intervals were considered. The traffic in one direction was taken into account in this analysis.

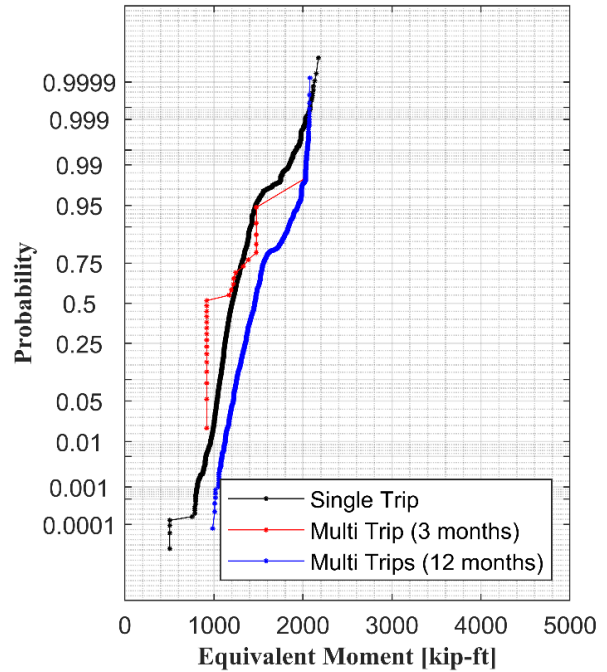
APPENDIX 2

Results for Bridge Consumption by Permit Vehicles

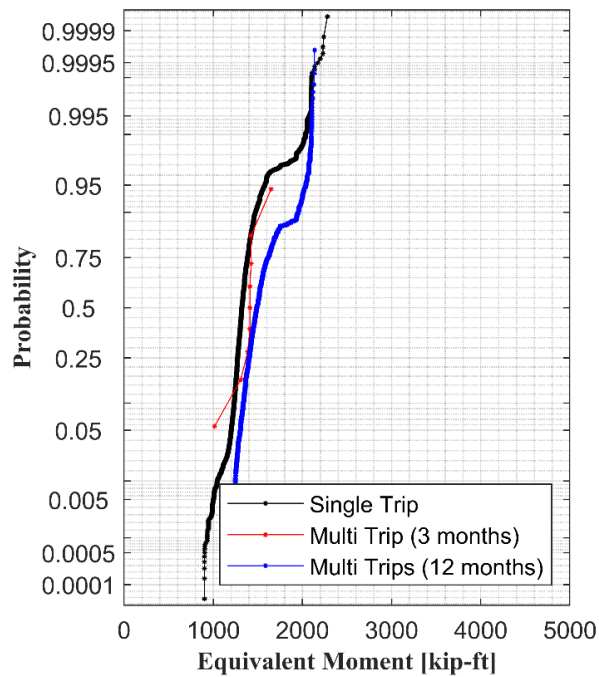
Appendix 2-A. Equivalent moment for permit vehicles with various GVW



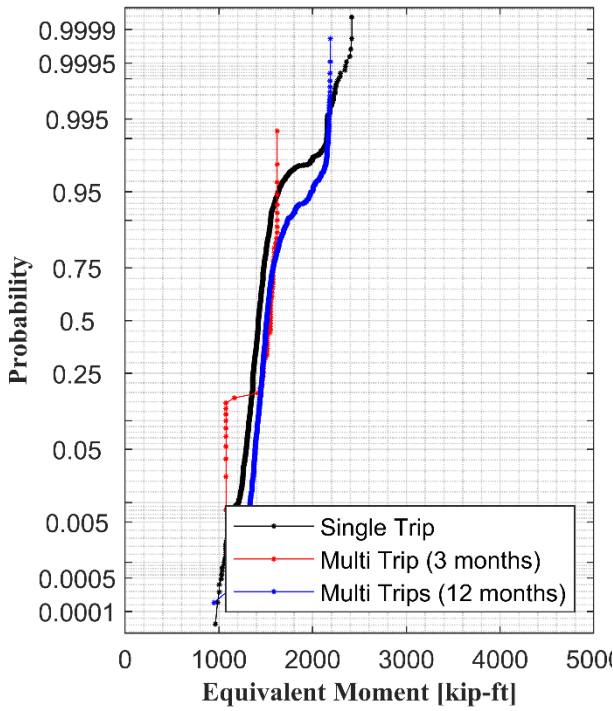
(a) Permit vehicles with GVW < 95 kips



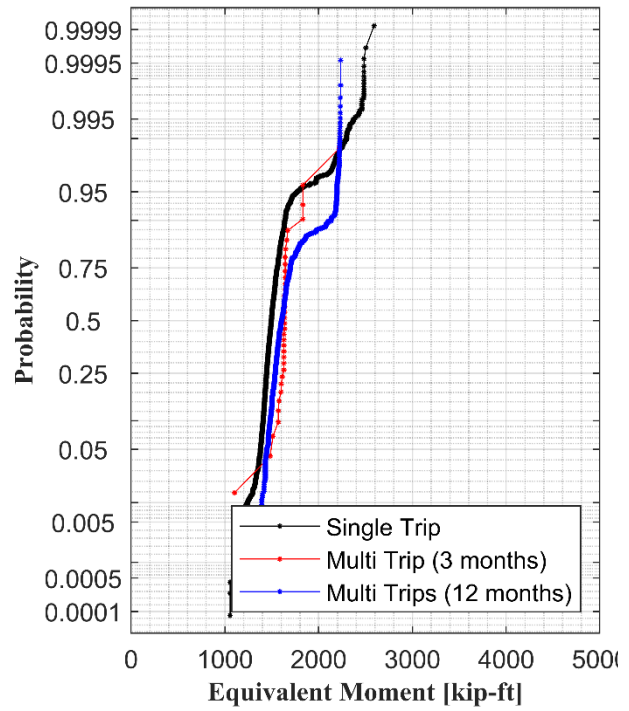
(b) Permit vehicles with GVW 95-112 kips



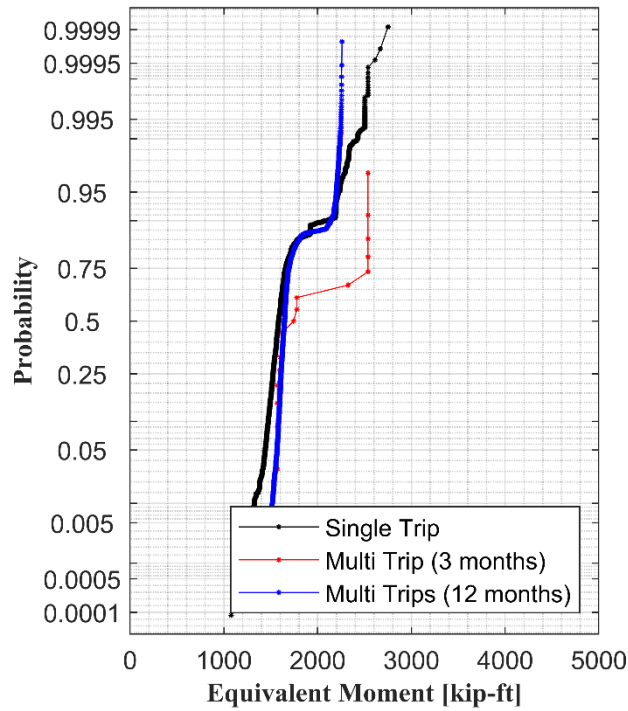
(c) Permit vehicles with GVW 112-122 kips



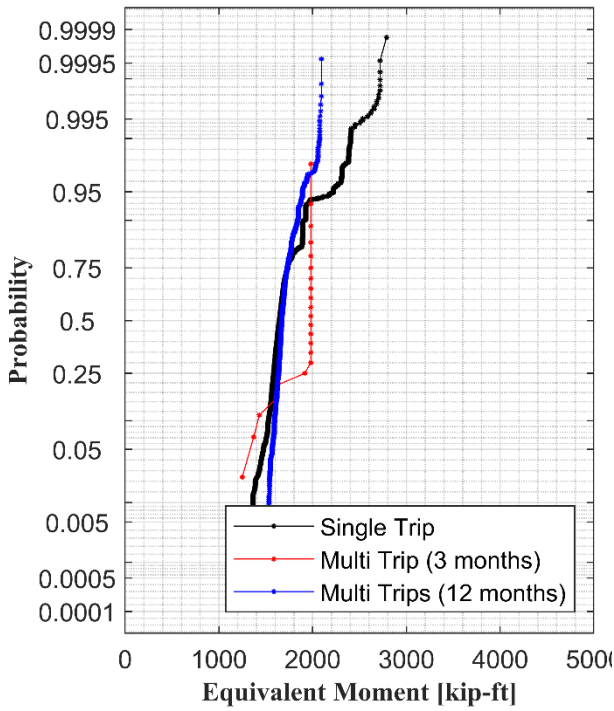
(d) Permit vehicles with GVW 122-132 kips



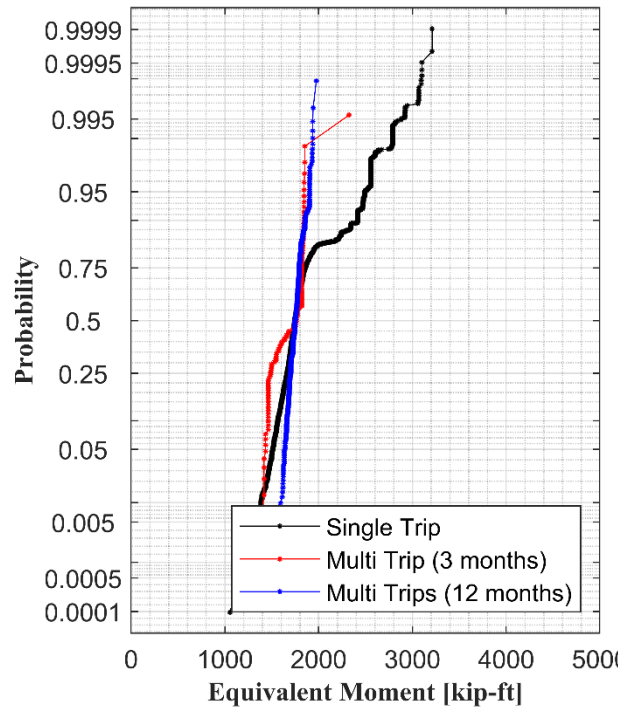
(e) Permit vehicles with GVW 132-142 kips



(f) Permit vehicles with GVW 142-152 kips



(g) Permit vehicles with GVW 152-162 kips



(h) Permit vehicles with GVW 162-199 kips

Figure 2-A.1 CDF of equivalent moment for permit vehicles with various GVW.

Appendix 2-B. Equivalent moment for multi and single trip vehicles

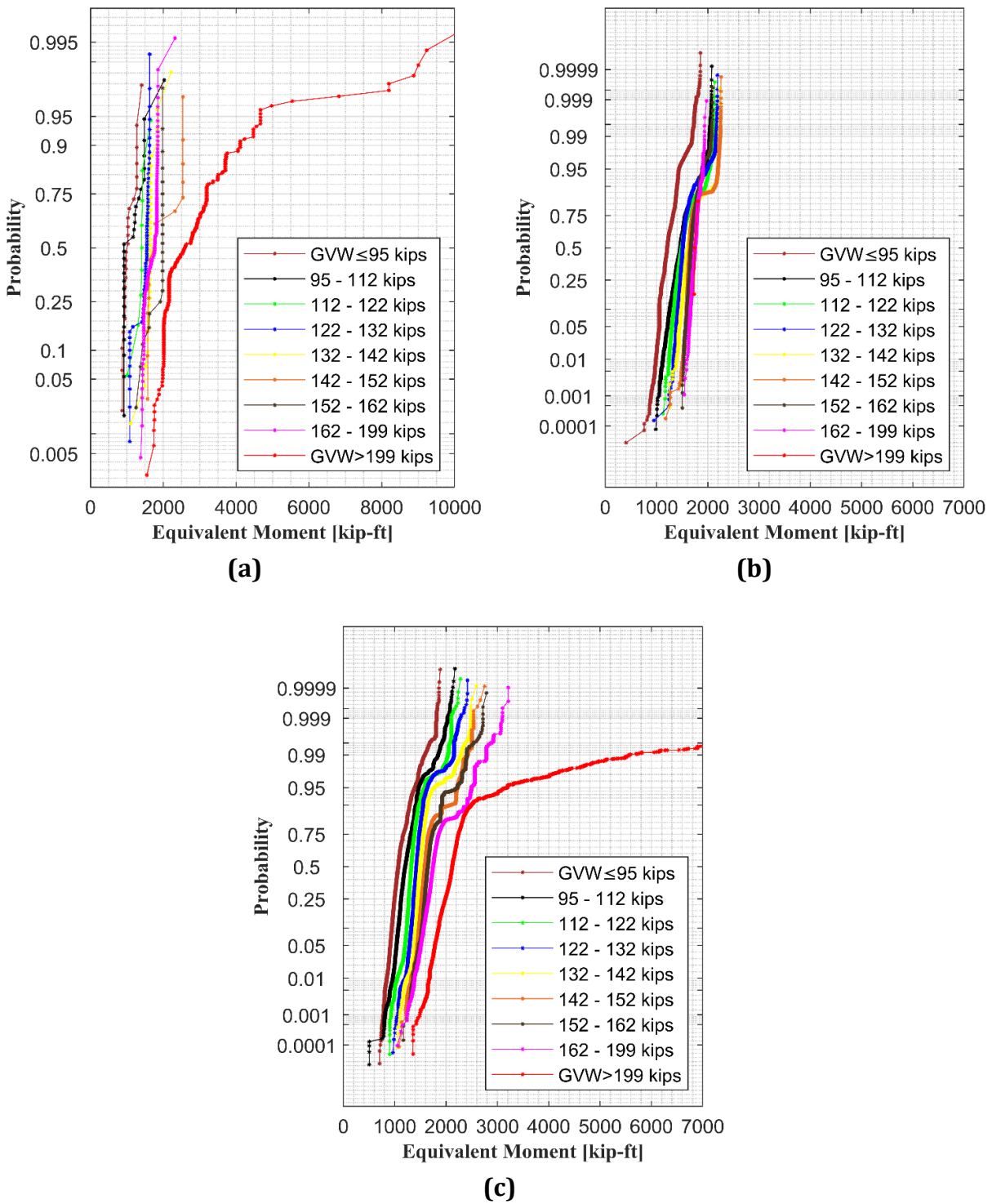


Figure 2-B.1 CDF of equivalent moment Bridge for (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

Appendix 2-C. Bridge Consumption in \$/mile of trip for permit vehicles

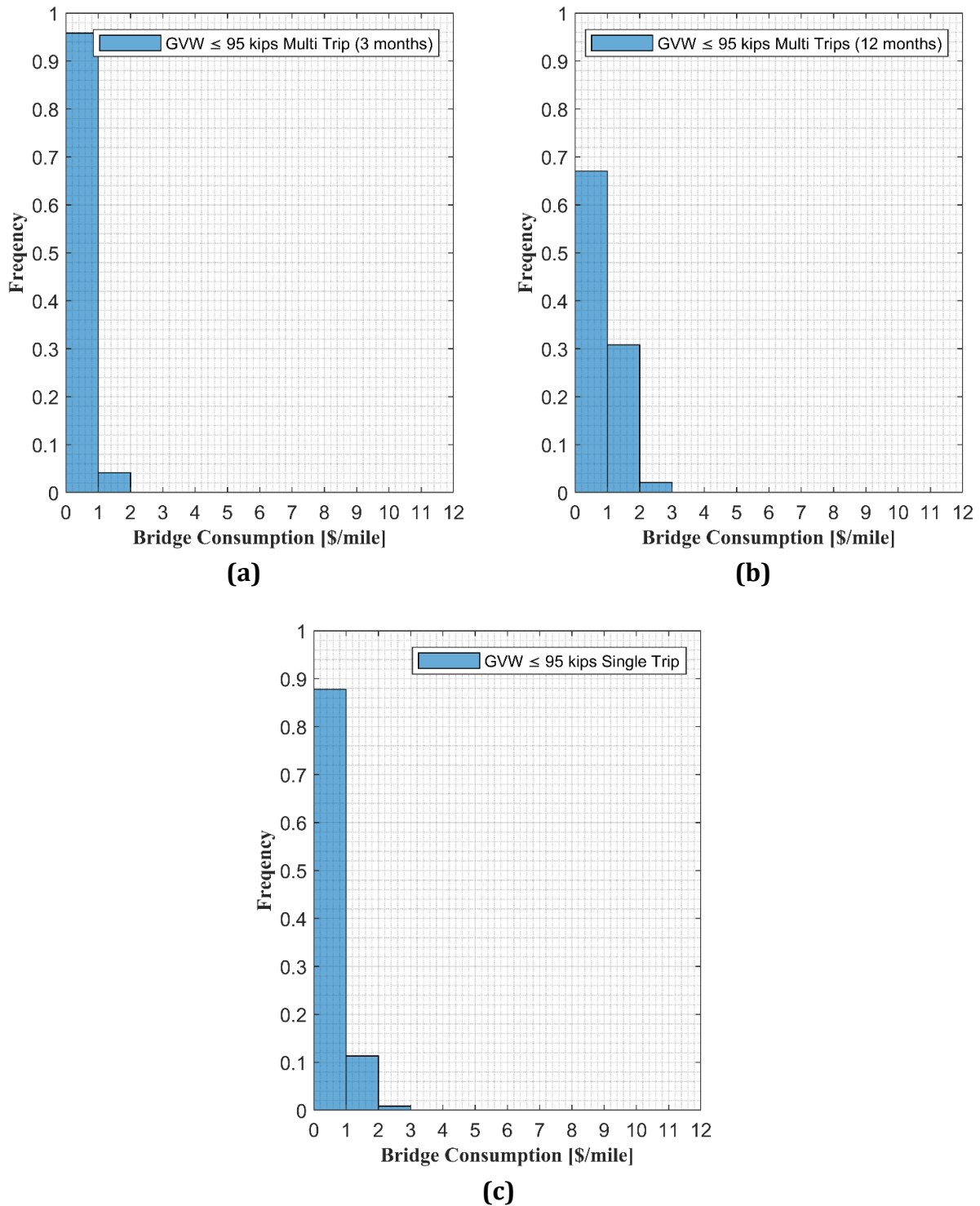


Figure 2-C.1. Bridge Consumption in \$/mile of trip for permit vehicles with GVW < 43.1 tons (95 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

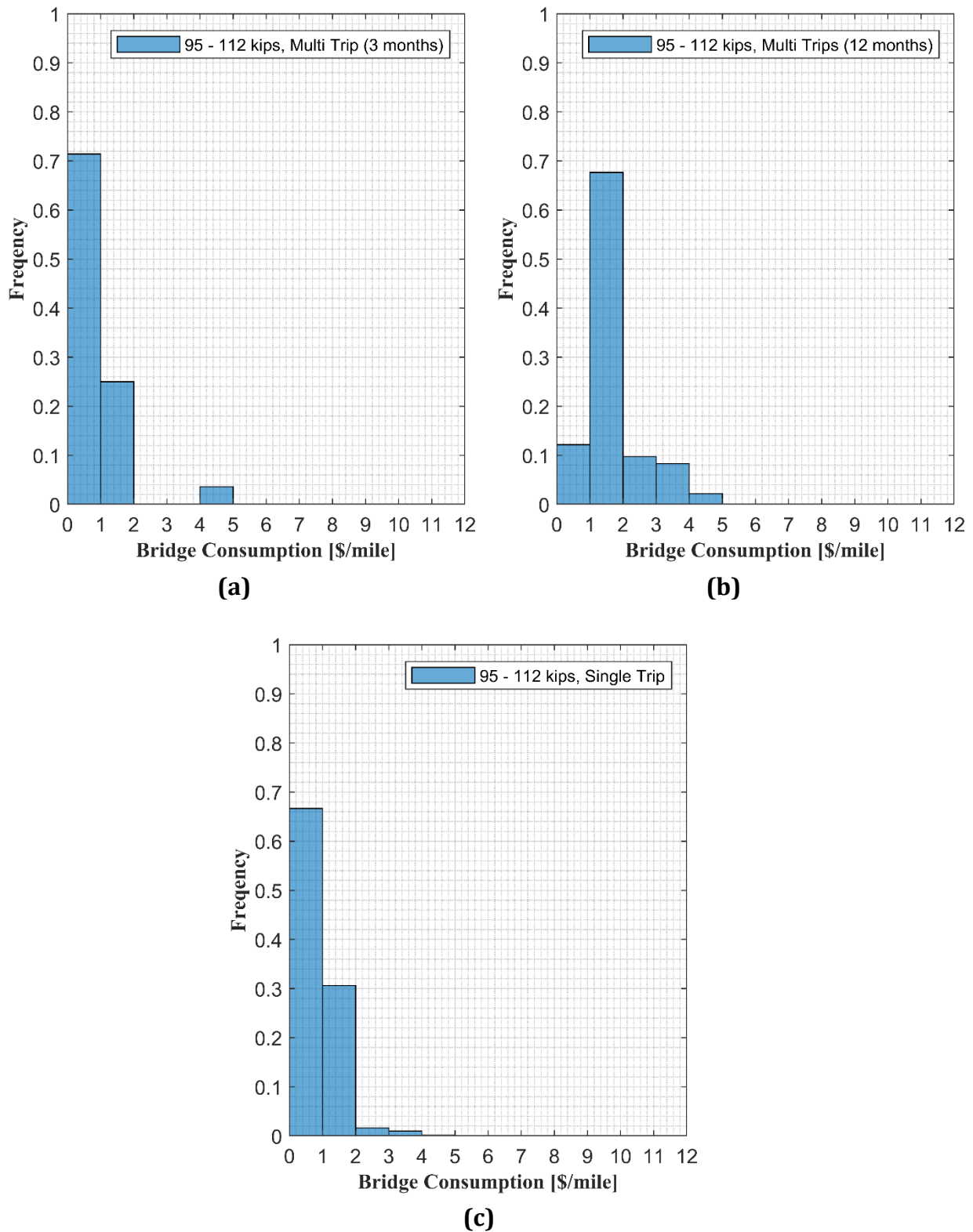
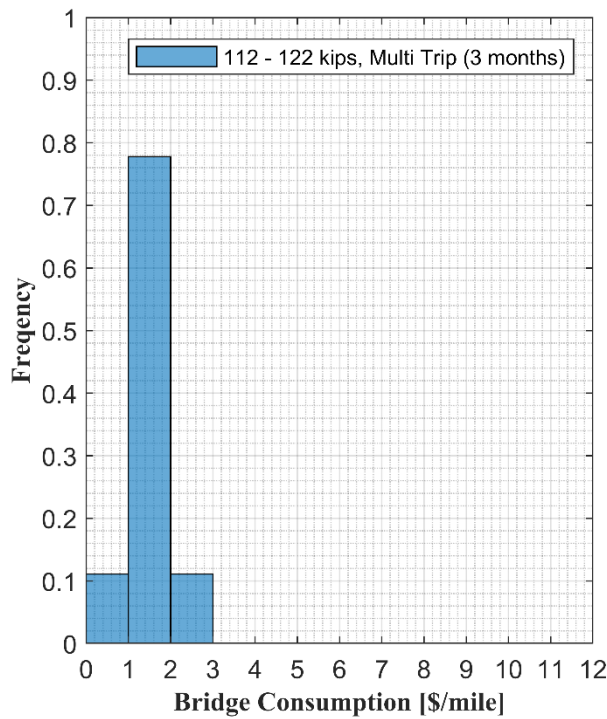
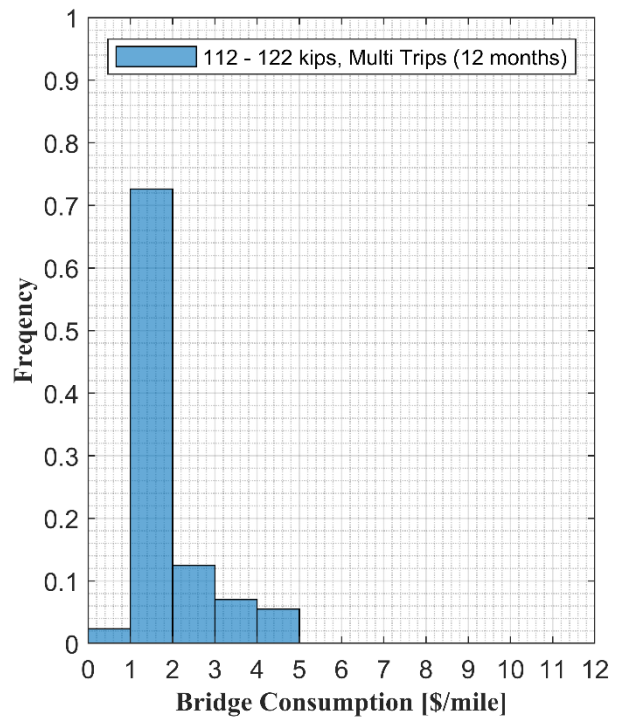


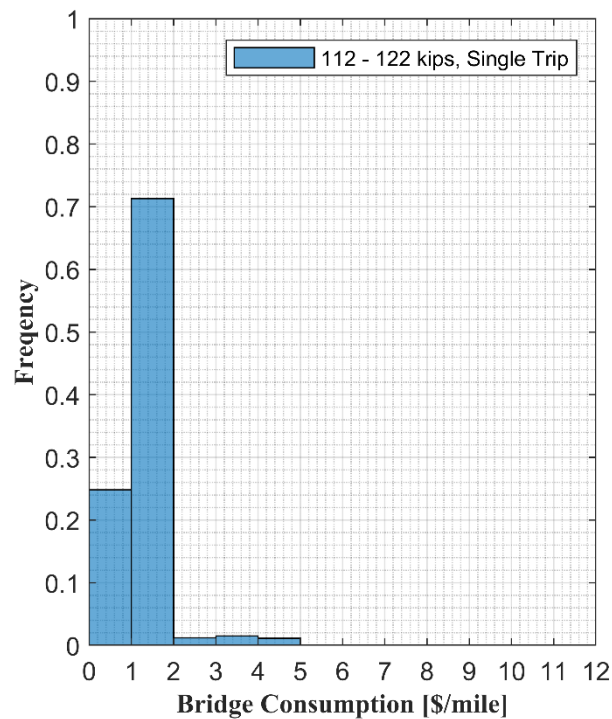
Figure 2-C.2. Bridge Consumption in \$/mile of trip for vehicles with GVW between 43.1-50.8 tons (95-112 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.



(a)



(b)



(c)

Figure 2-C.3. Bridge Consumption in \$/mile of trip for vehicles with GVW between 50.8-55.3 tons (112-122 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

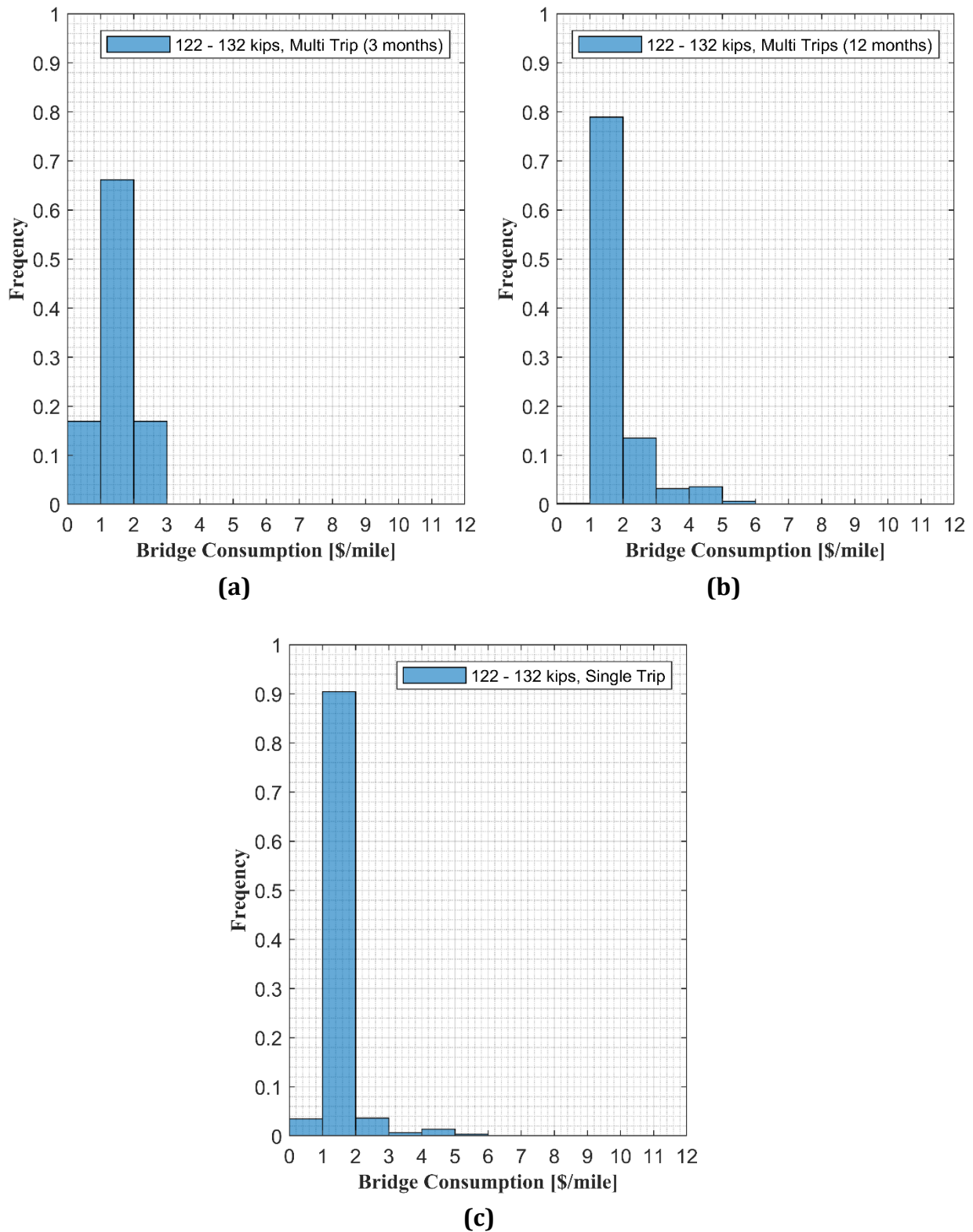
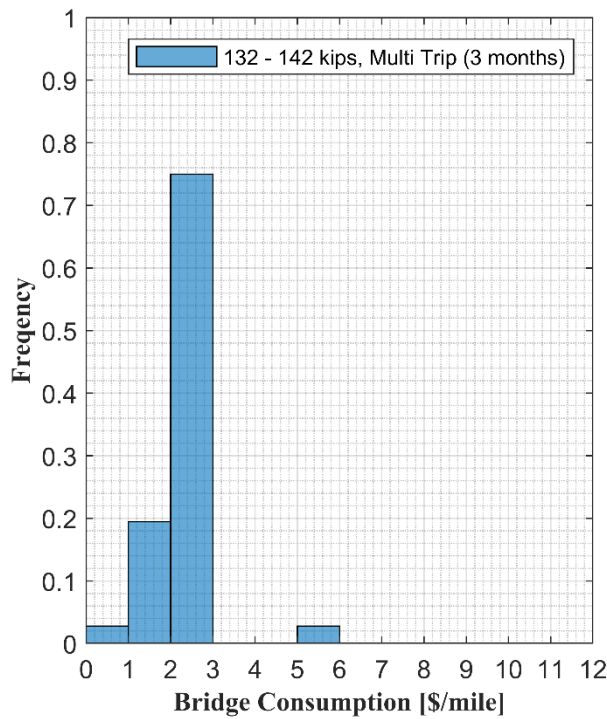
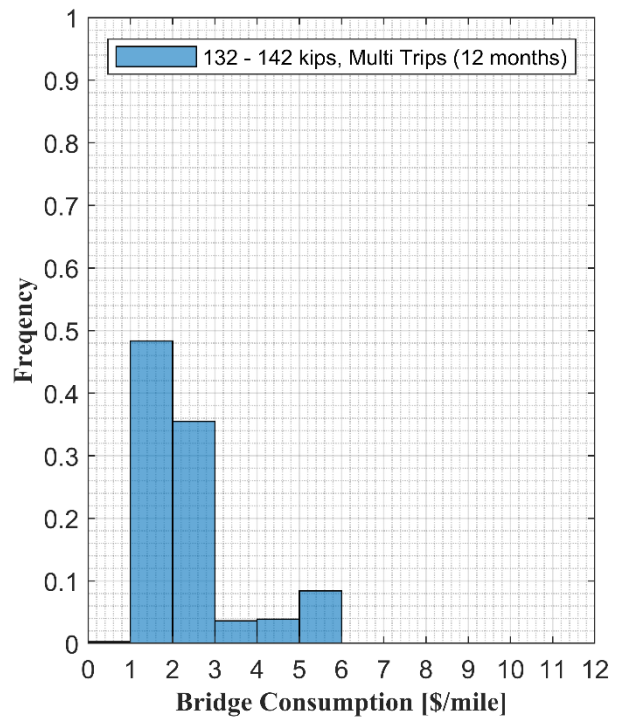


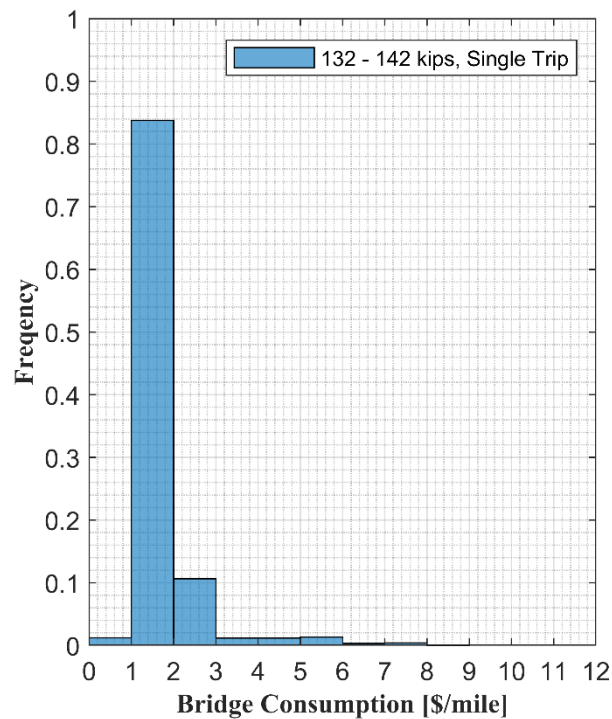
Figure 2-C.4. Bridge Consumption in \$/mile of trip for vehicles with GVW between 55.3-59.9 tons (122-132 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.



(a)

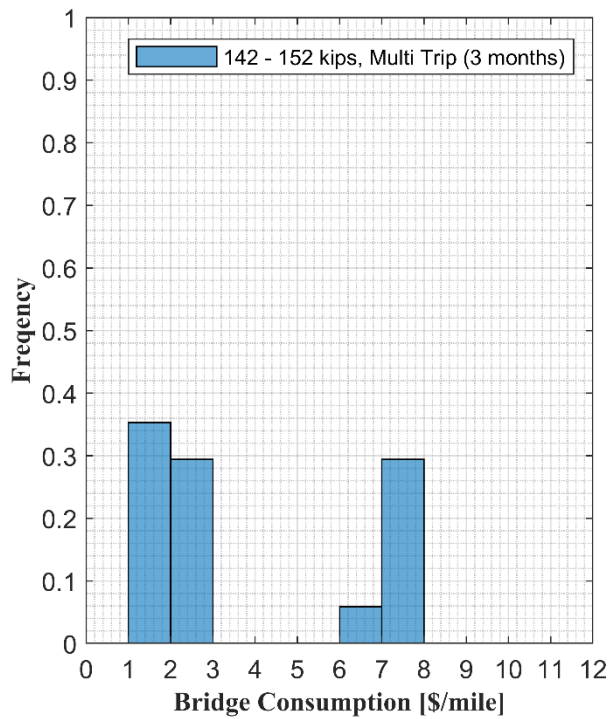


(b)

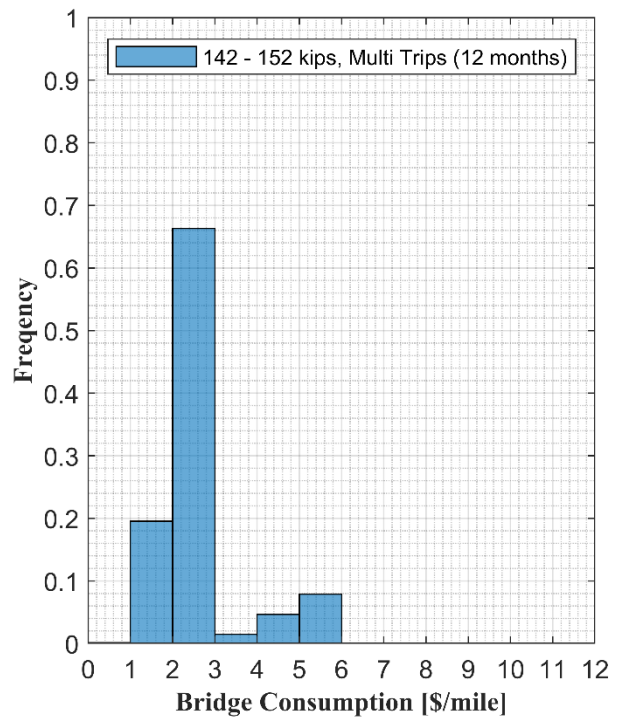


(c)

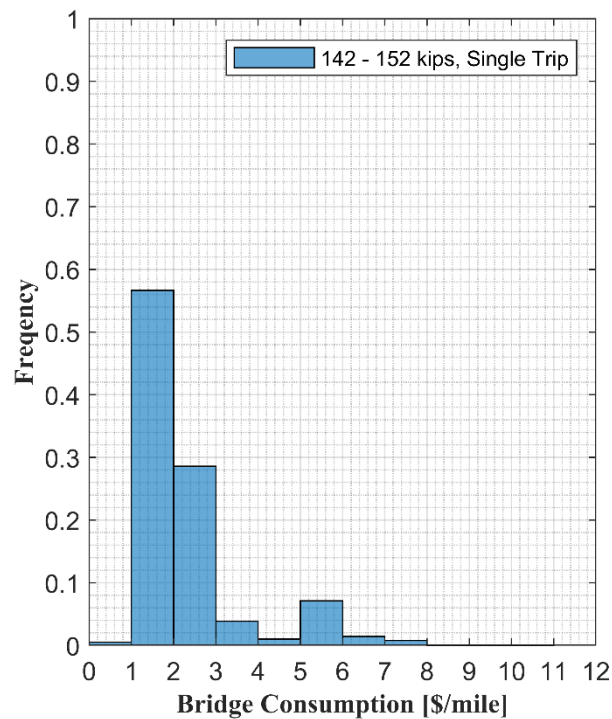
Figure 2-C.5. Bridge Consumption in \$/mile of trip for vehicles with GVW between 59.9-64.4 tons (132-142 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.



(a)



(b)



(c)

Figure 2-C.6. Bridge Consumption in \$/mile of trip for vehicles with GVW between 64.4-68.9 tons (142-152 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

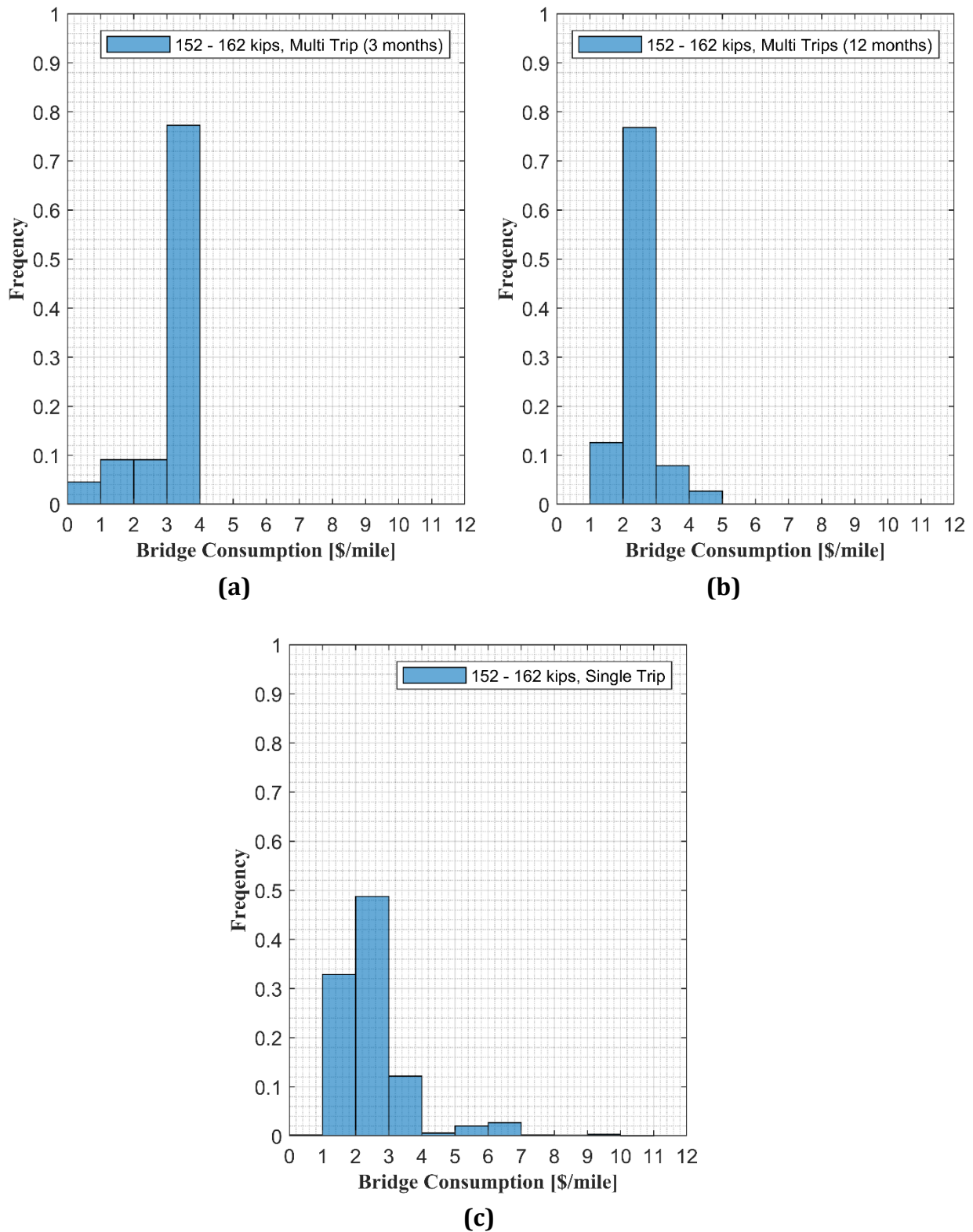
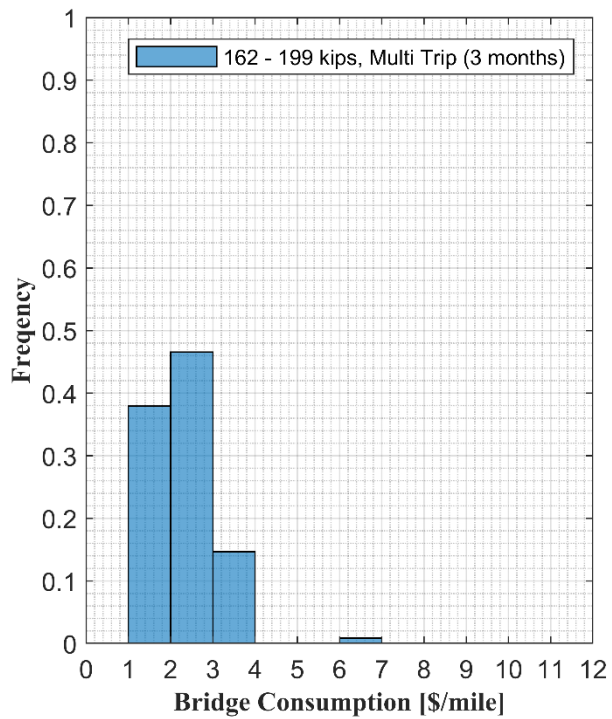
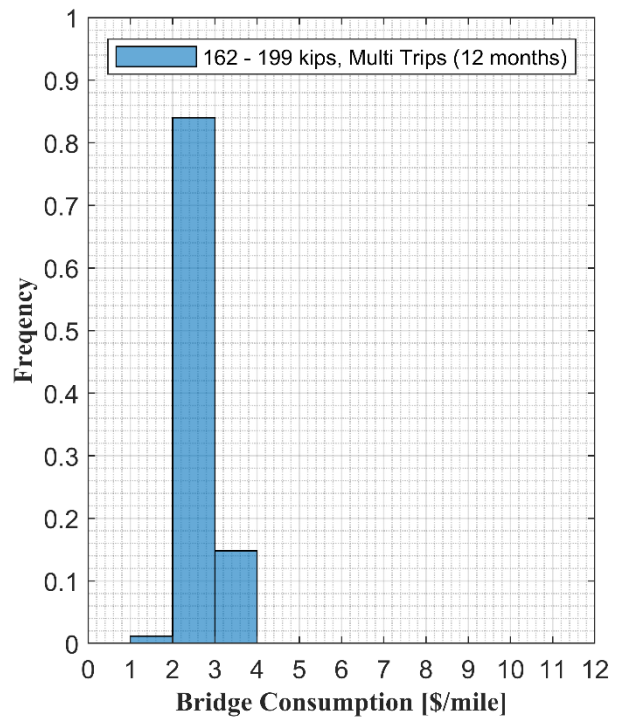


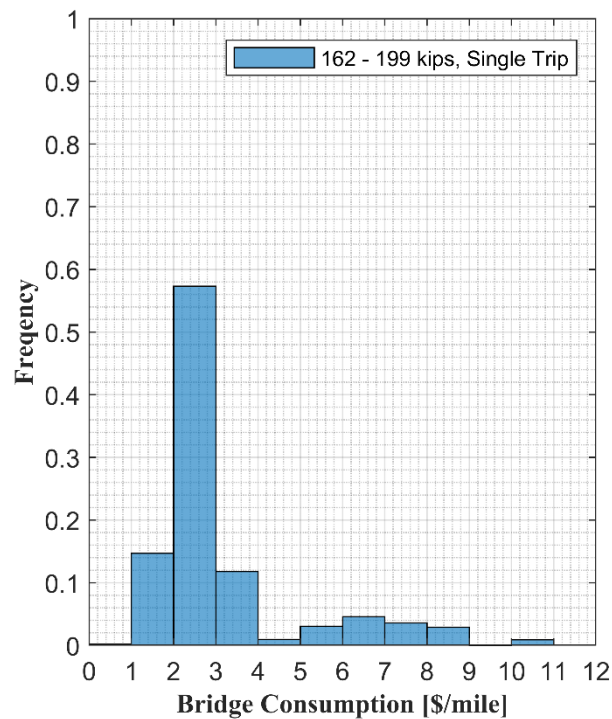
Figure 2-C.7. Bridge Consumption in \$/mile of trip for vehicles with GVW between 68.9-73.5 tons (152-162 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.



(a)



(b)



(c)

Figure 2-C.8. Bridge Consumption in \$/mile of trip for vehicles with GVW between 73.5-90.3 tons (162-199 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

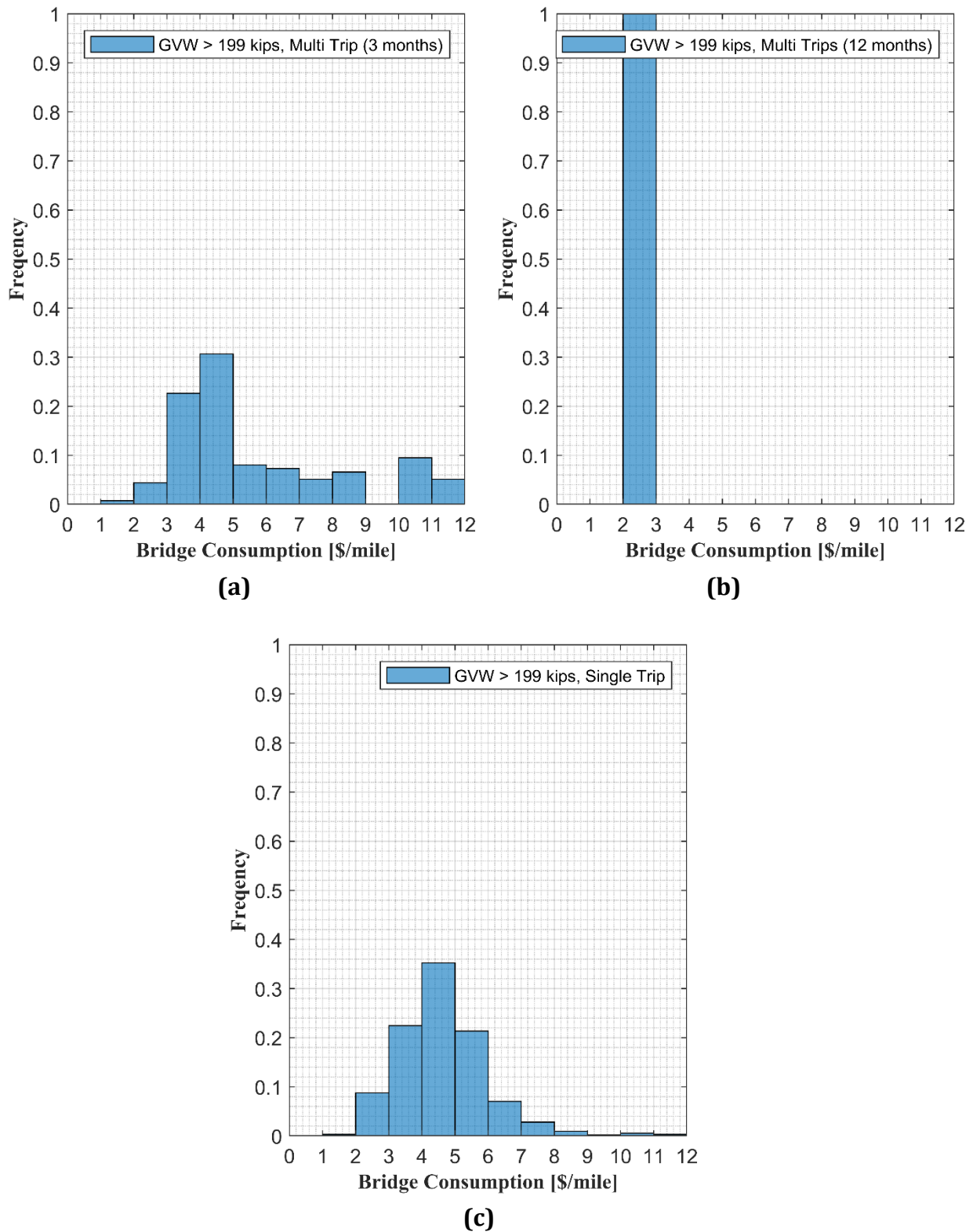
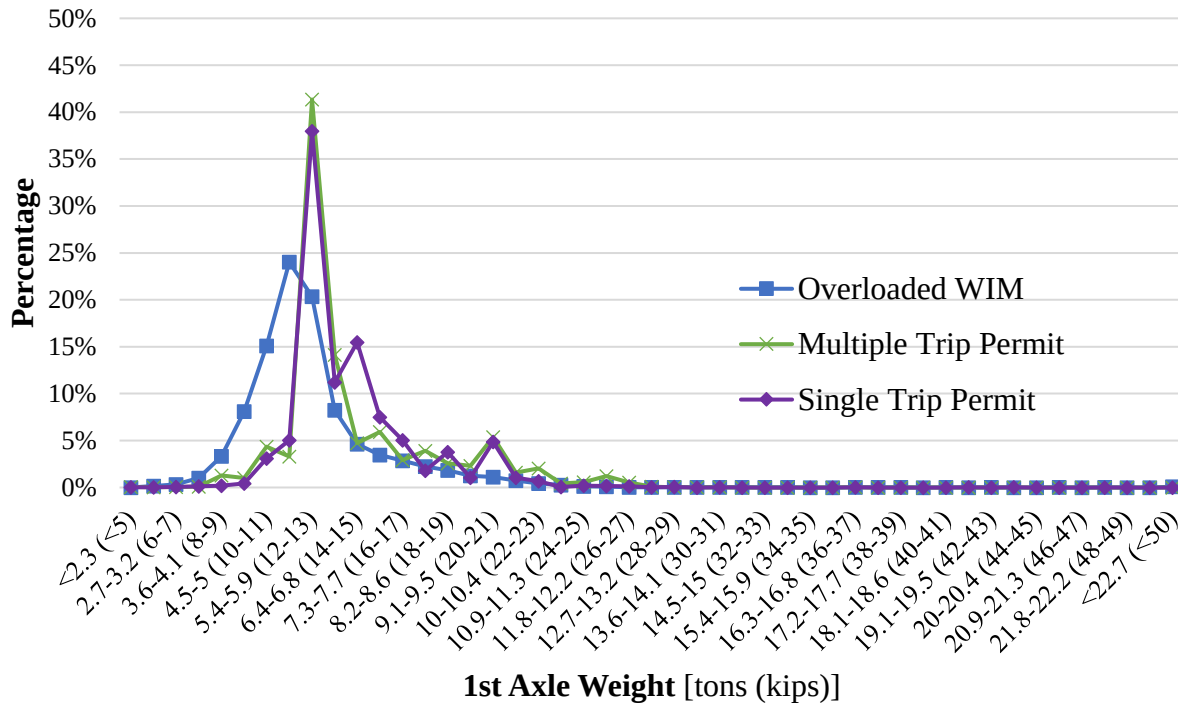


Figure 2-C.9. Bridge Consumption in \$/mile of trip for vehicles with GVW > 90.3 tons (199 kips) (a) multi trips permit for 3 months, (b) multi trips permit for 12 months (c) single trips permits in Florida.

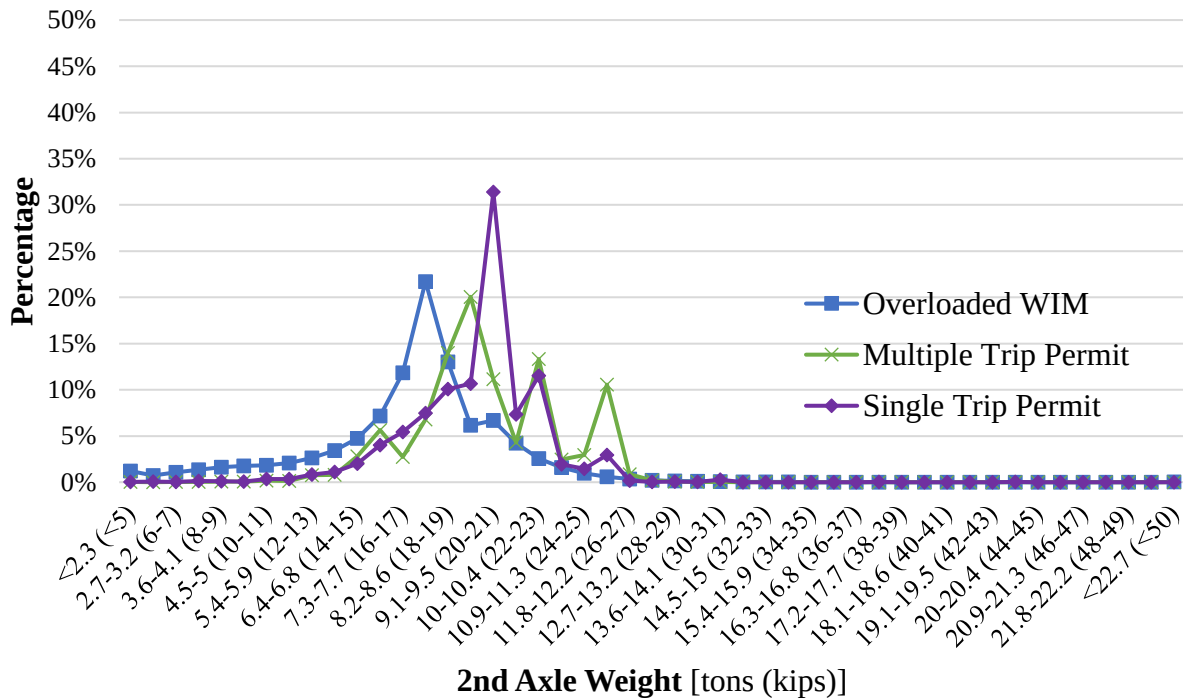
APPENDIX 3

Results for Development of Notional Permit Vehicles

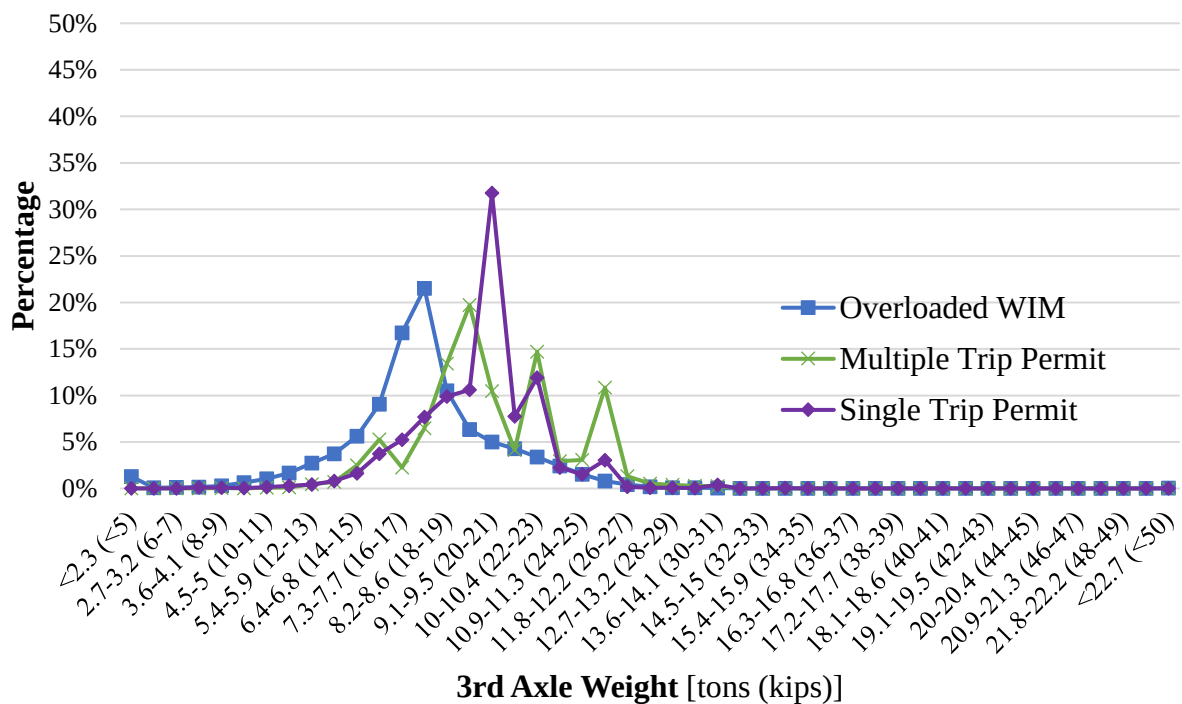
Appendix 3-A. Axle weight and spacings histograms for overloaded vehicles



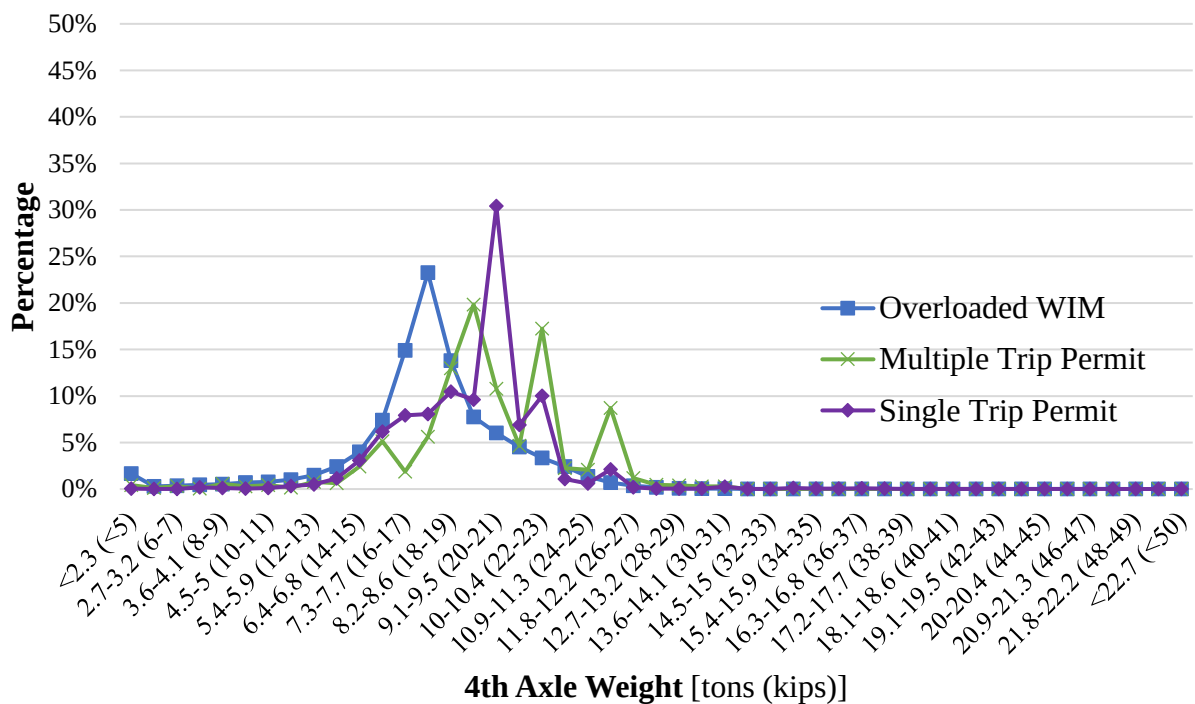
(a)



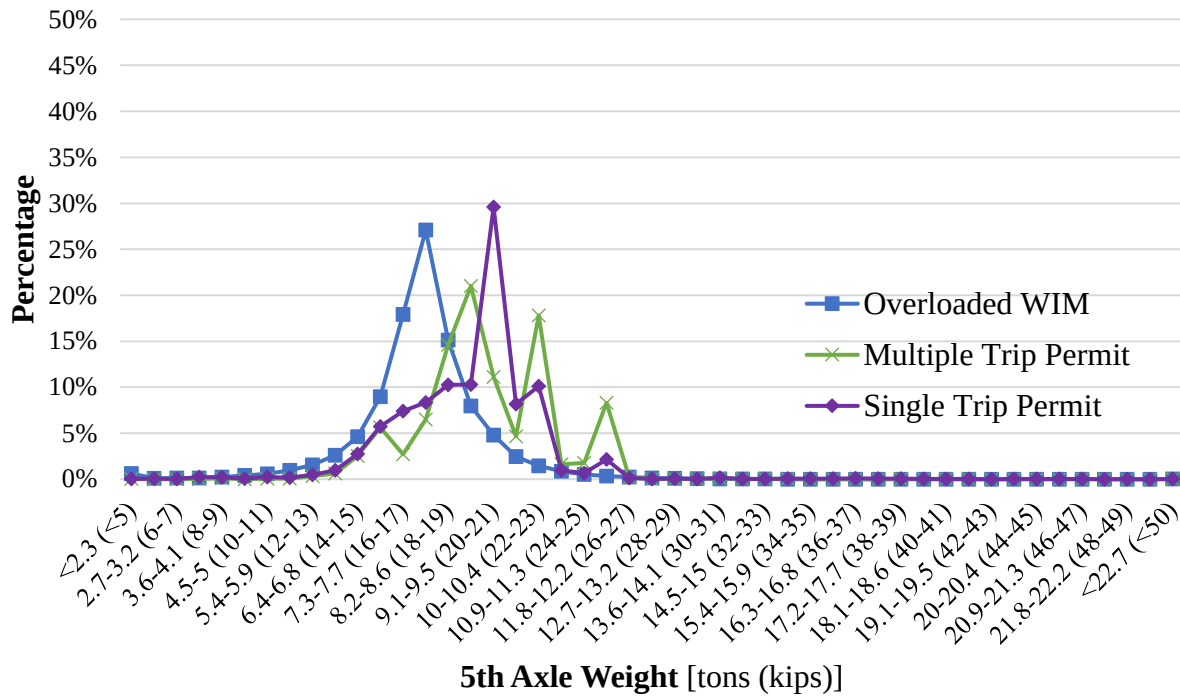
(b)



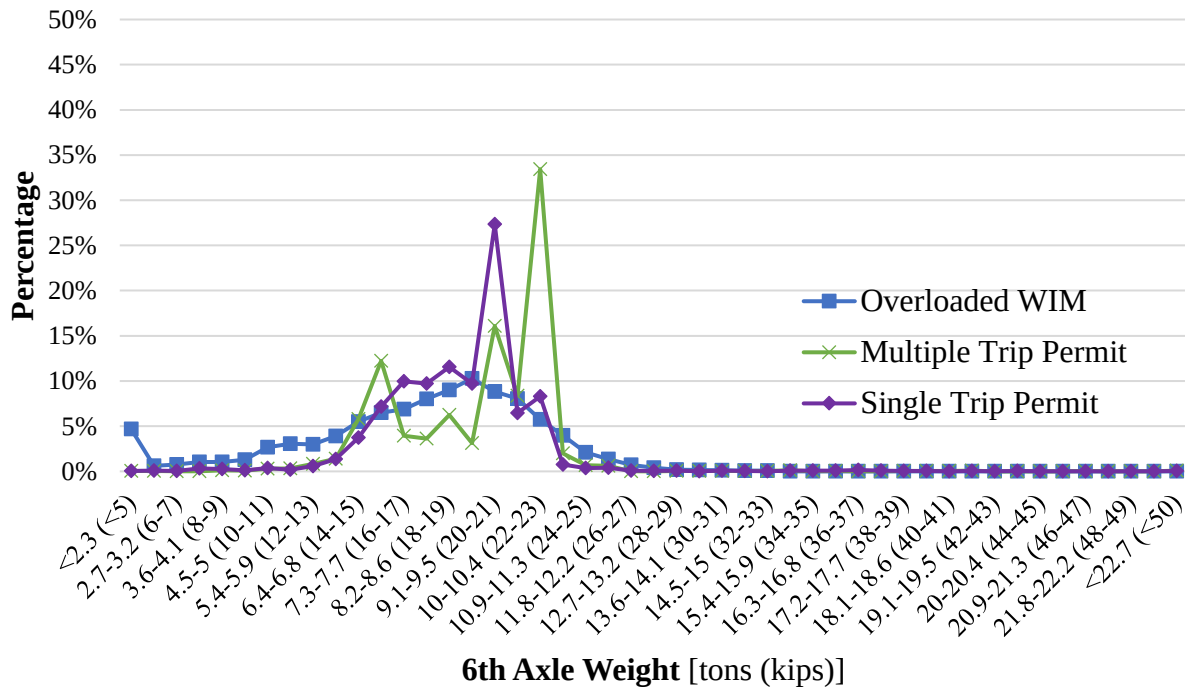
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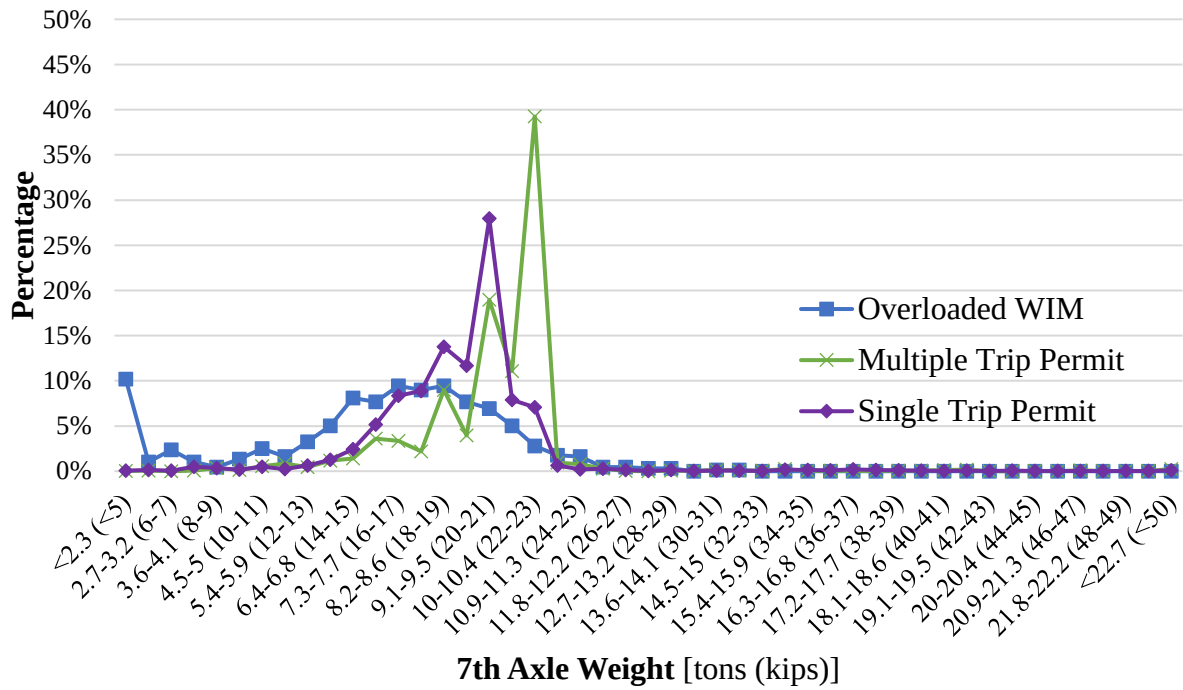
(d)



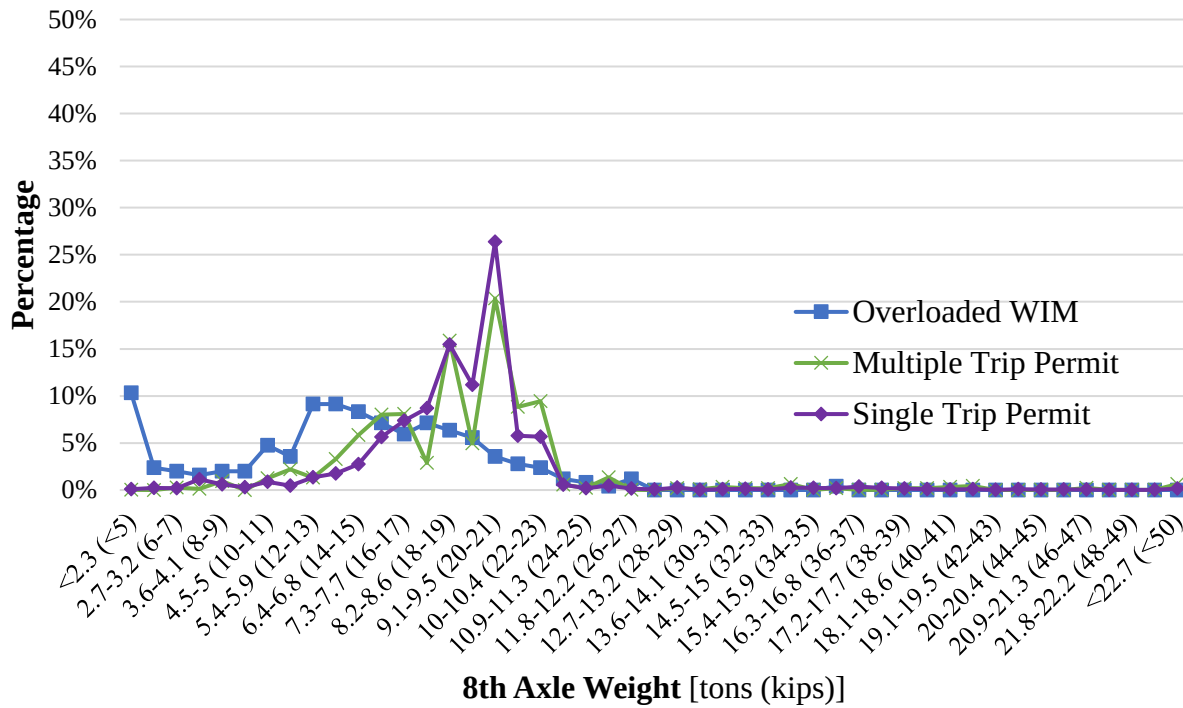
(e)



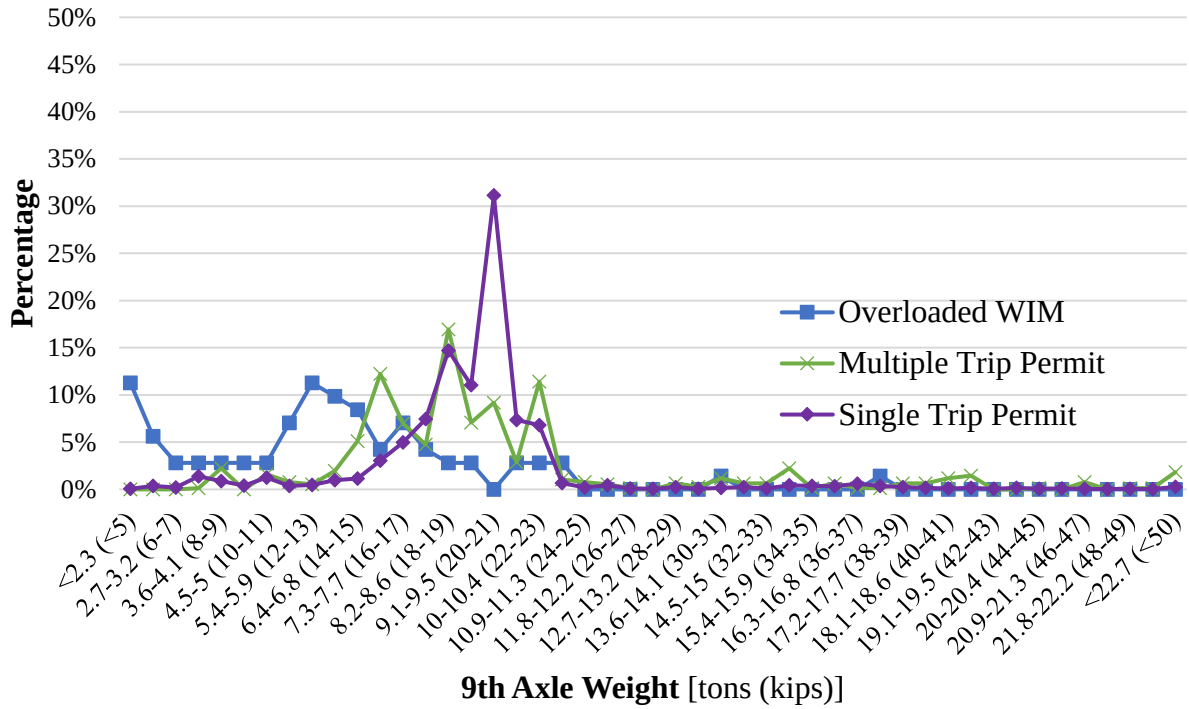
(f)



(g)

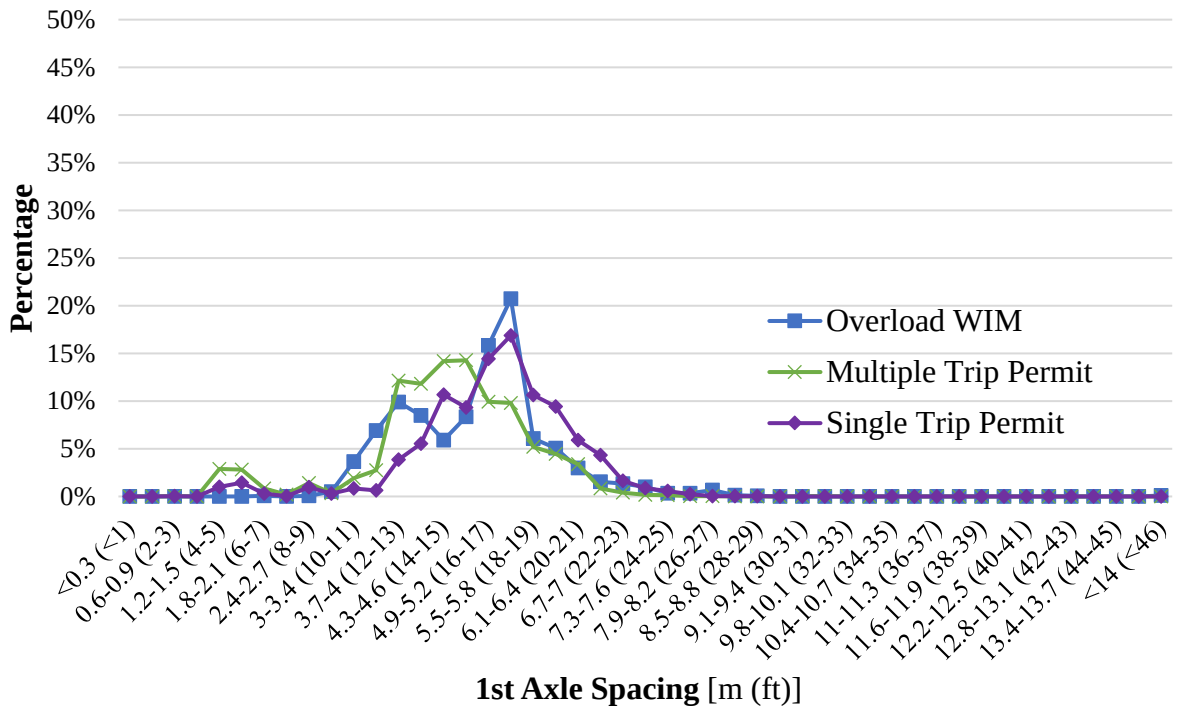


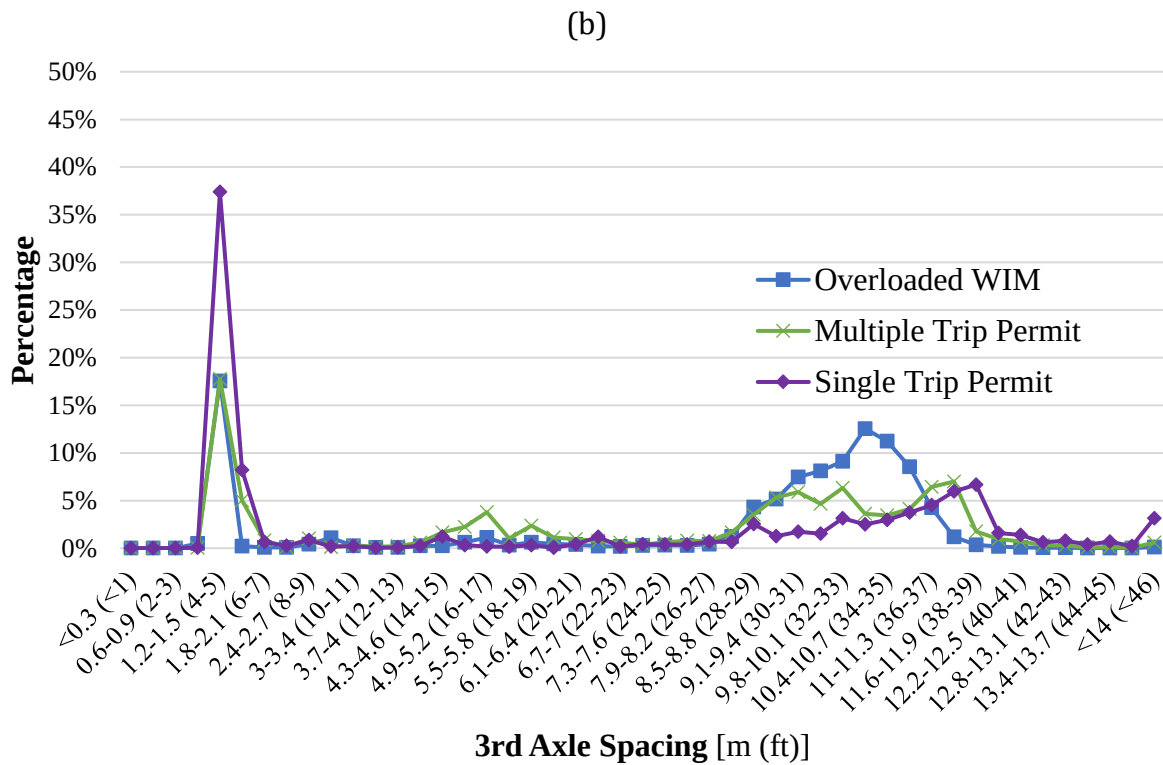
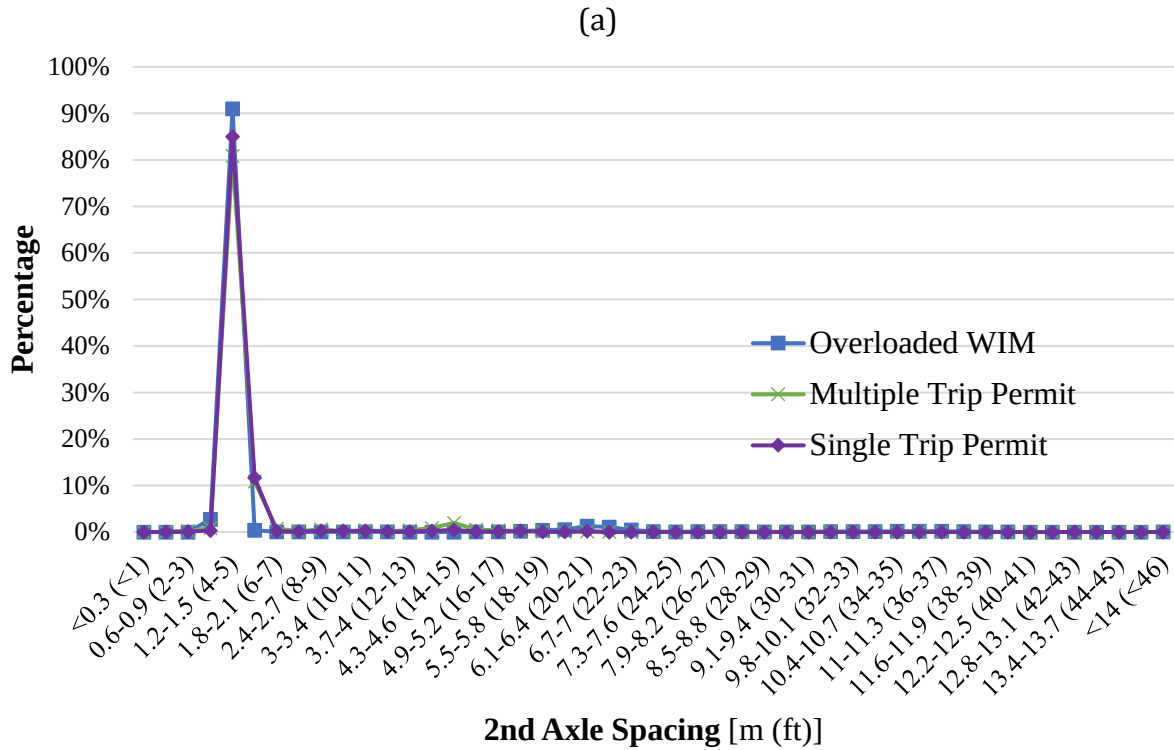
(h)



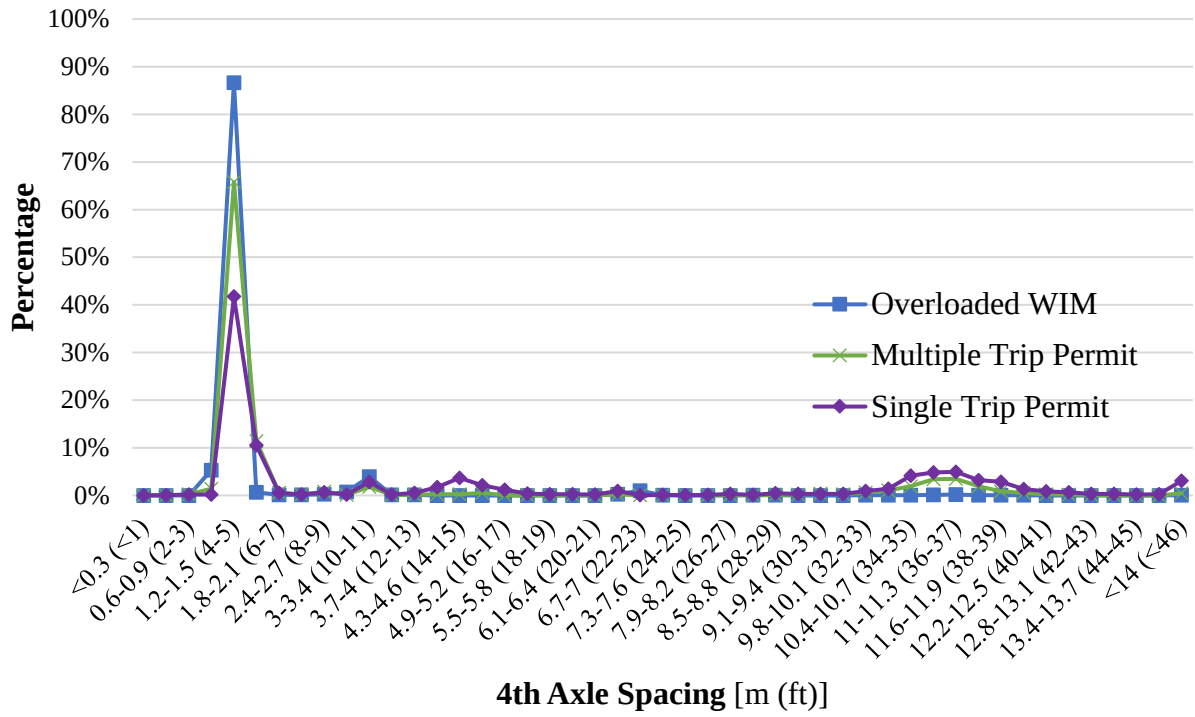
(i)

Figure 3-A.1. Axle weight histograms for (a) first axle, (b) second axle, (c) third axle, (d) fourth axle, (e) fifth axle, (f) sixth axle, (g) seventh axle, (h) eighth axle, and (i) ninth axle.

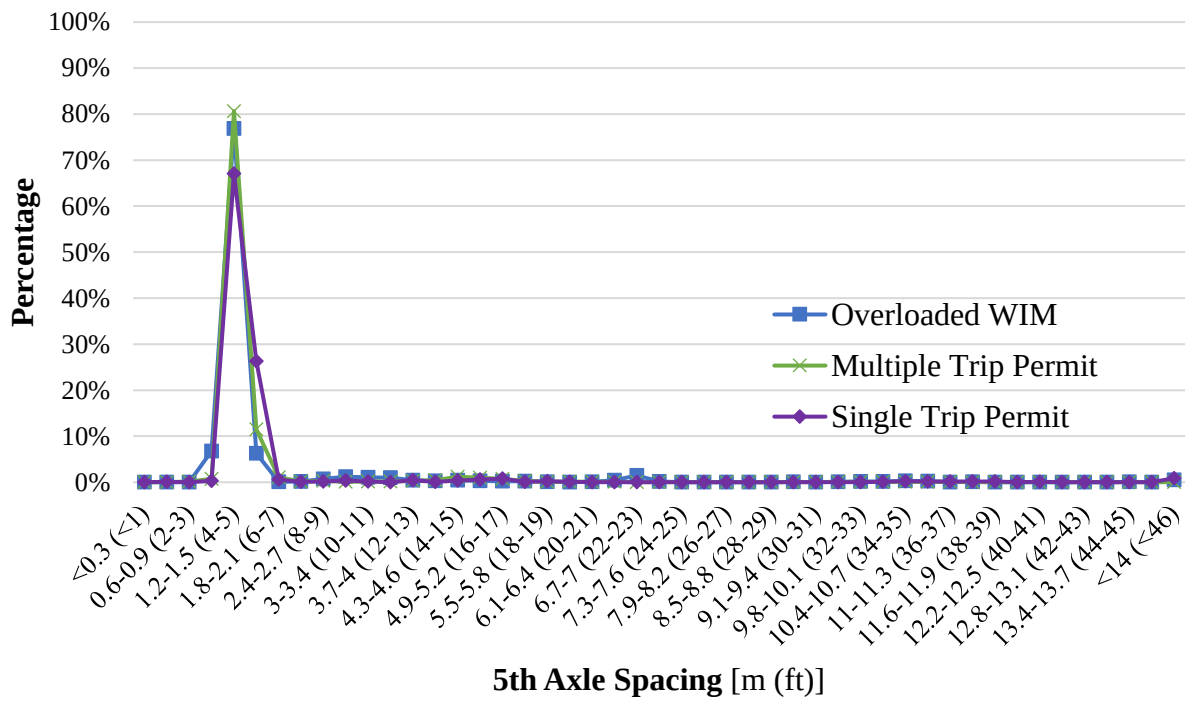




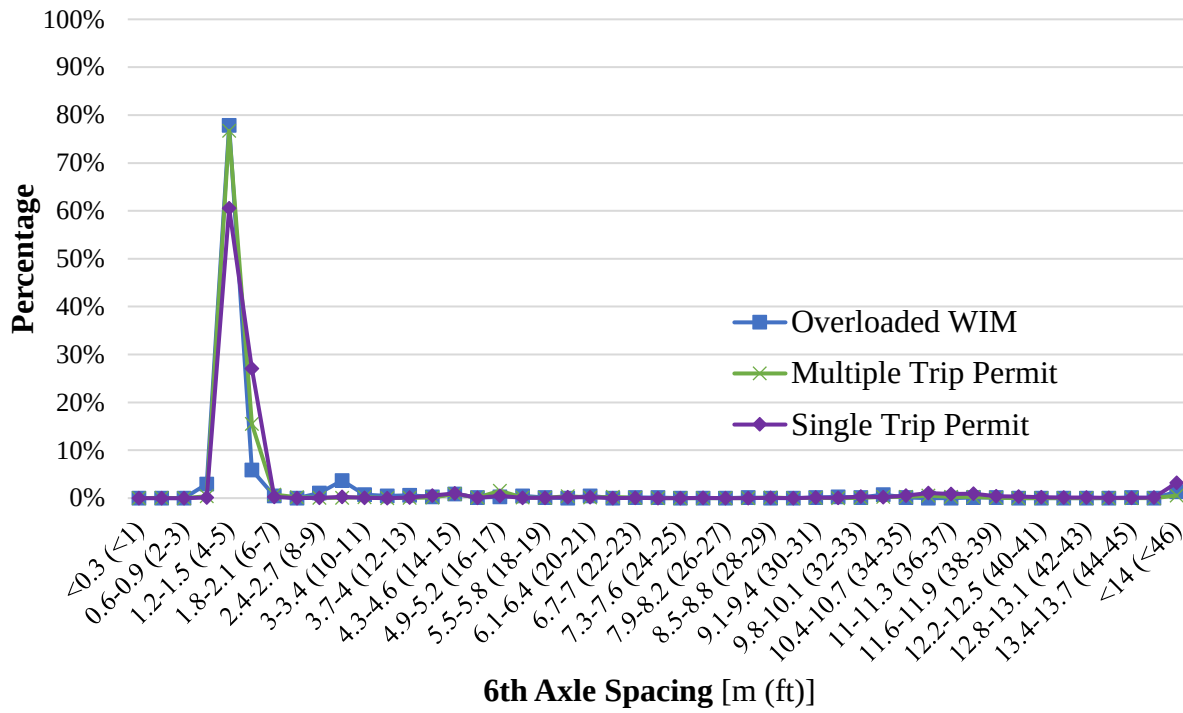
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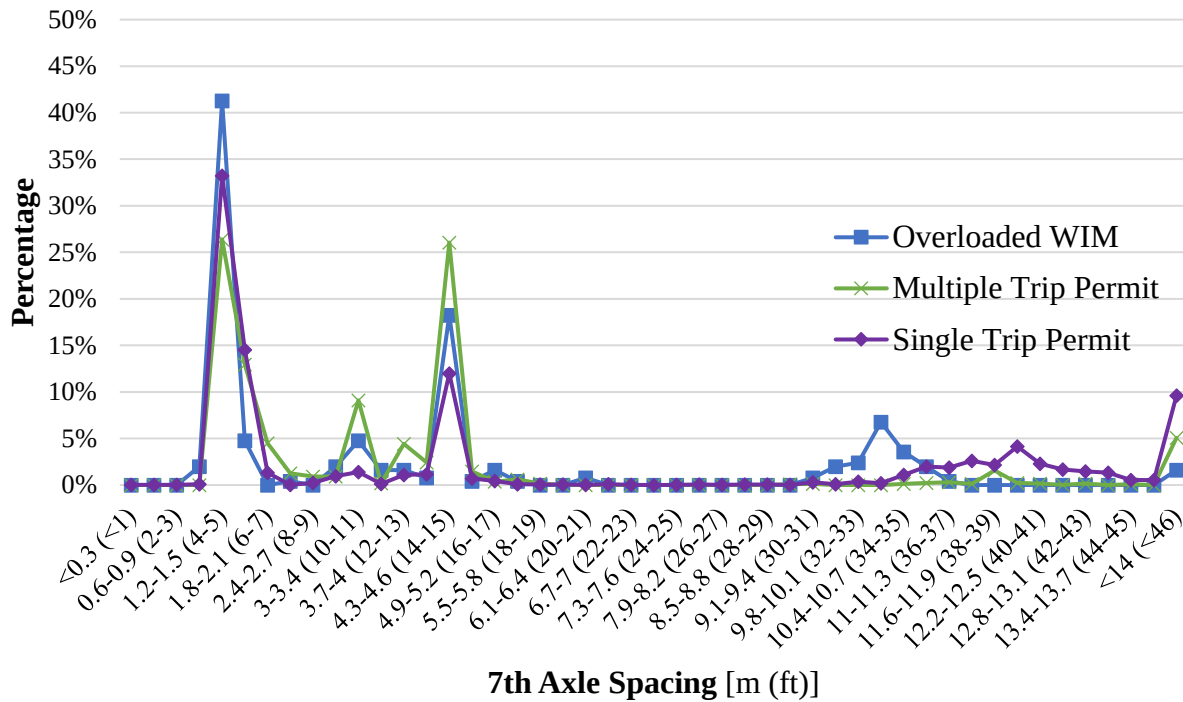
(d)



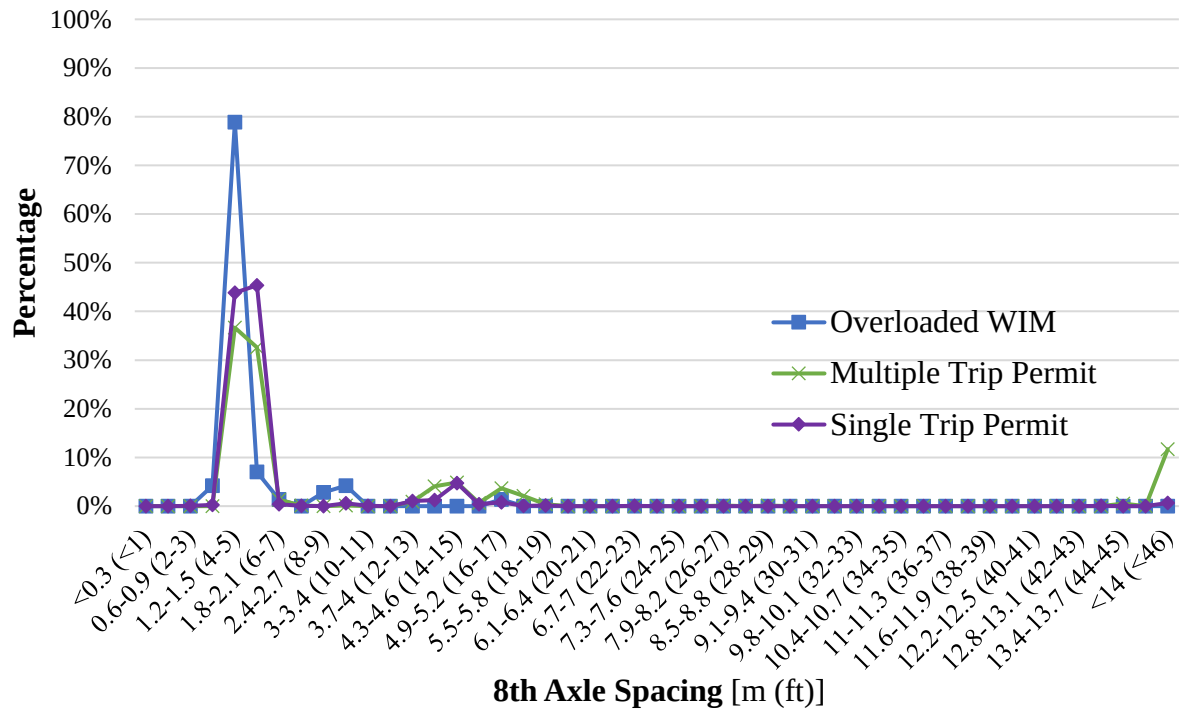
(e)



(f)



(g)



(h)

Figure 3-A.2. Axle spacing histograms for (a) first spacing, (b) second spacing, (c) third spacing, (d) fourth spacing, (e) fifth spacing, (f) sixth spacing, (g) seventh spacing, (h) eighth spacing, and (i) ninth spacing.

Appendix 3-B. Load ratios for groups of notional permit vehicle candidates

Load Ratios for Candidates in Group 1

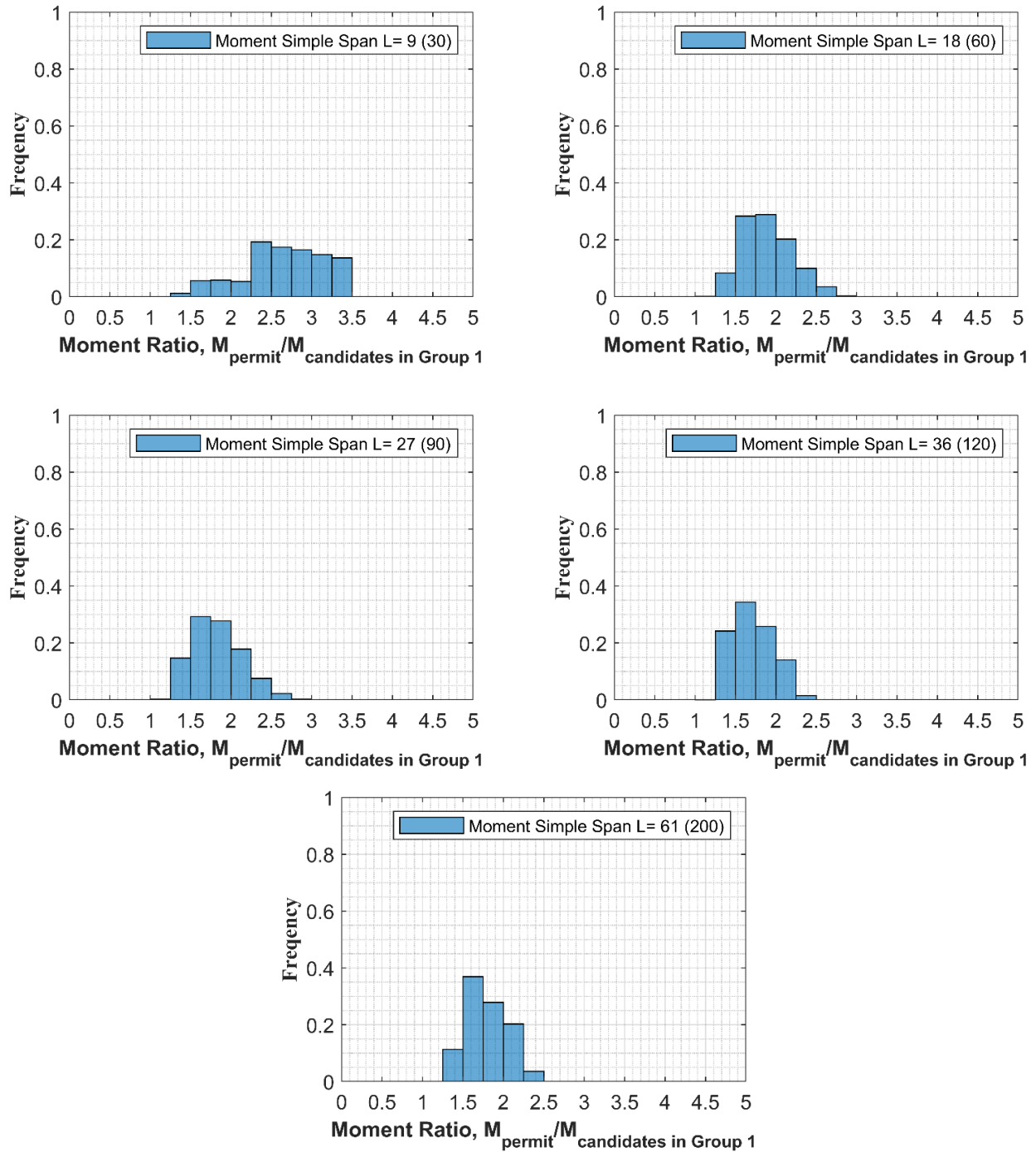


Figure 3-B.1. Load ratios of Candidates in Group 1 for various simple span moments.

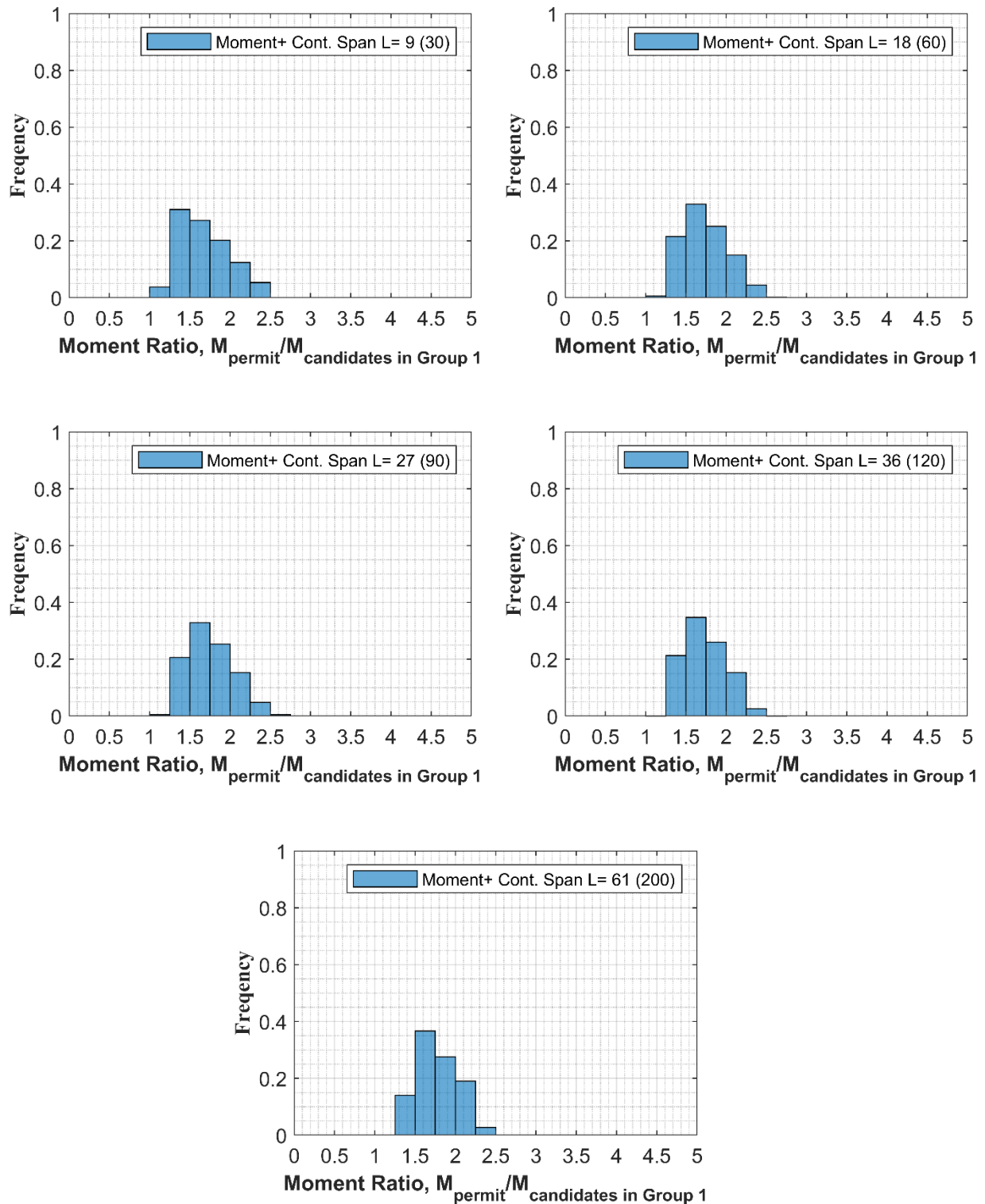


Figure 3-B.2. Load ratios of Candidates in Group 1 for various positive continuous span moments.

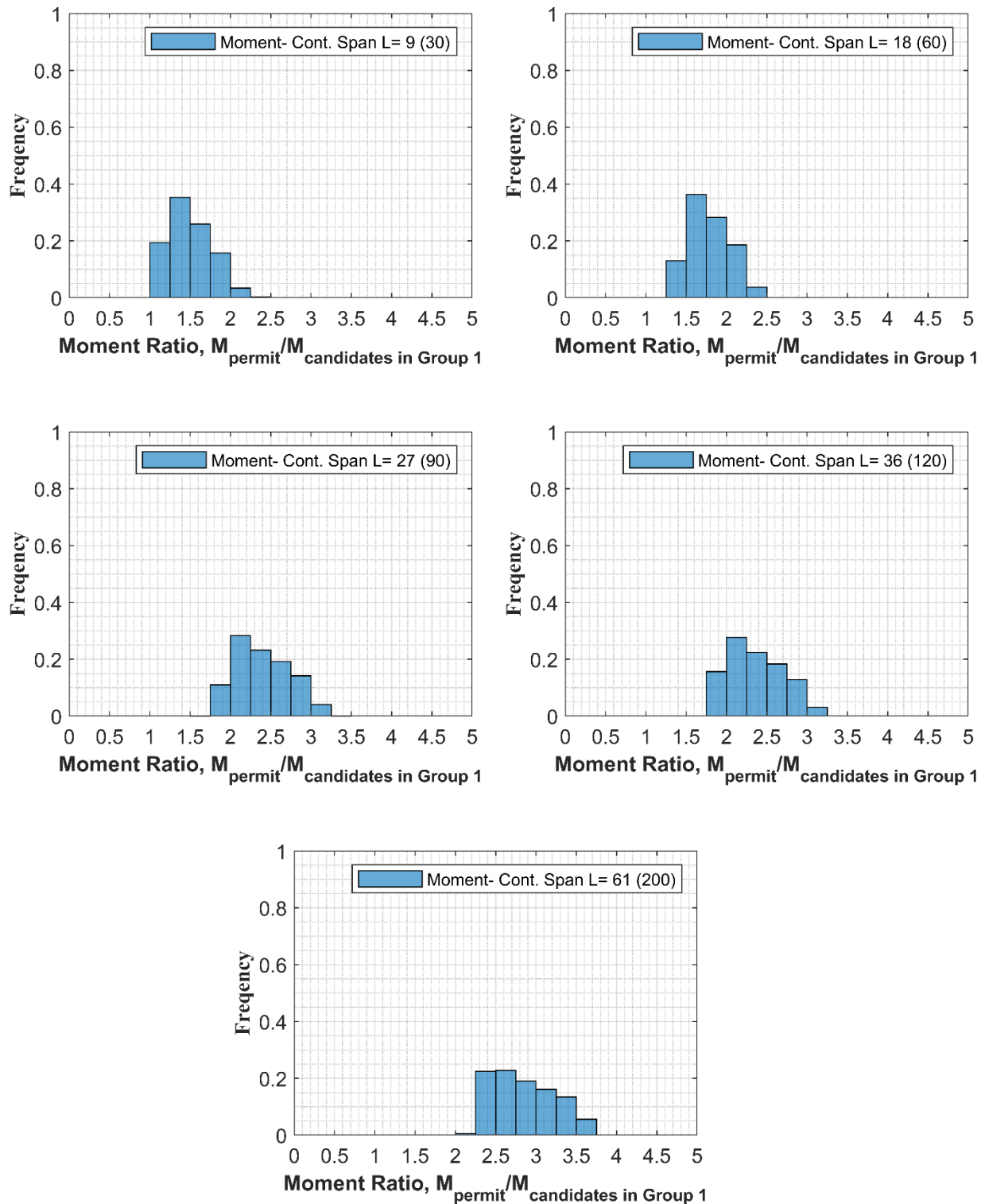


Figure 3-B.3. Load ratios of Candidates in Group 1 for various negative continuous span moments.

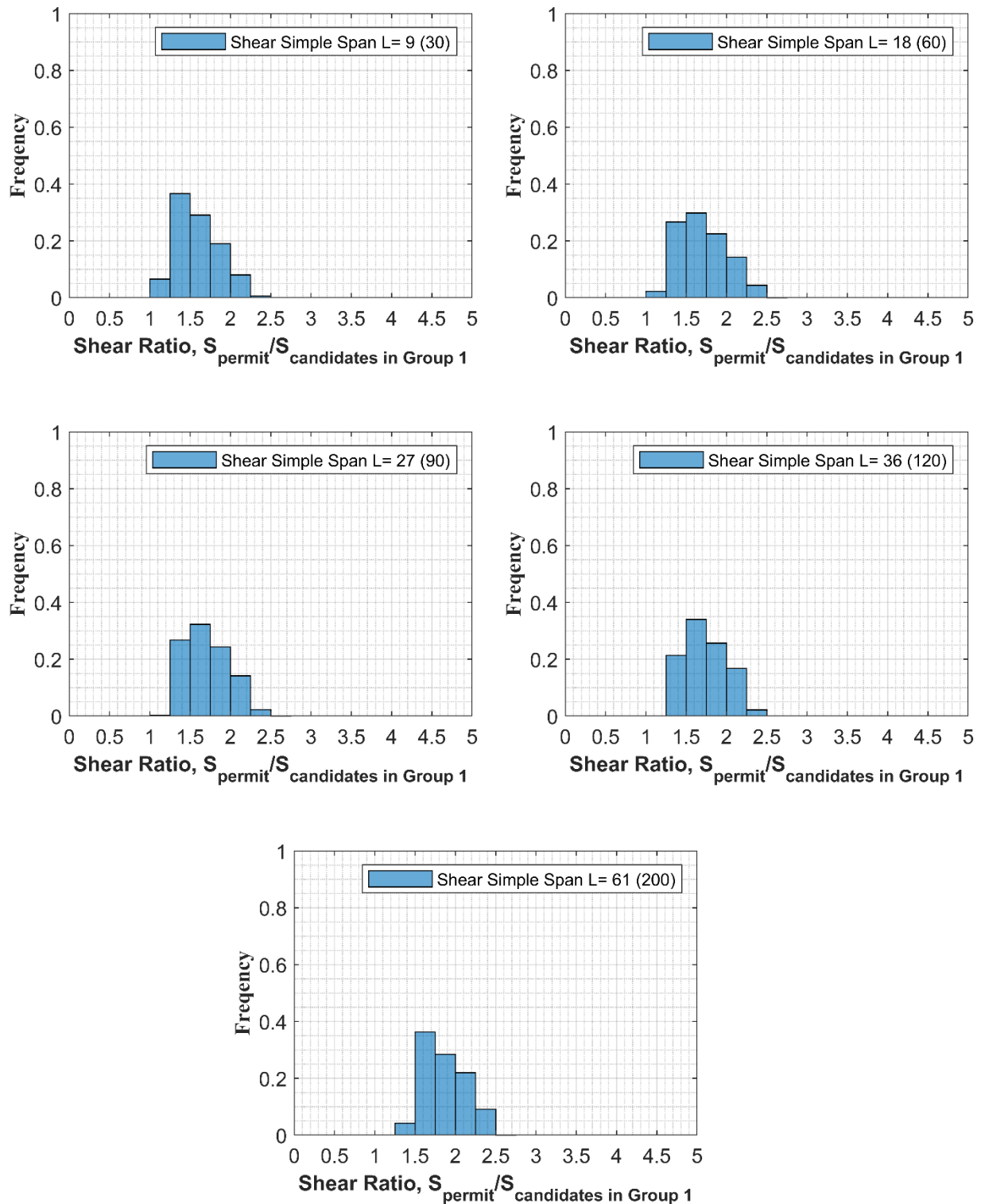


Figure 3-B.4. Load ratios of Candidates in Group 1 for various simple span shears.

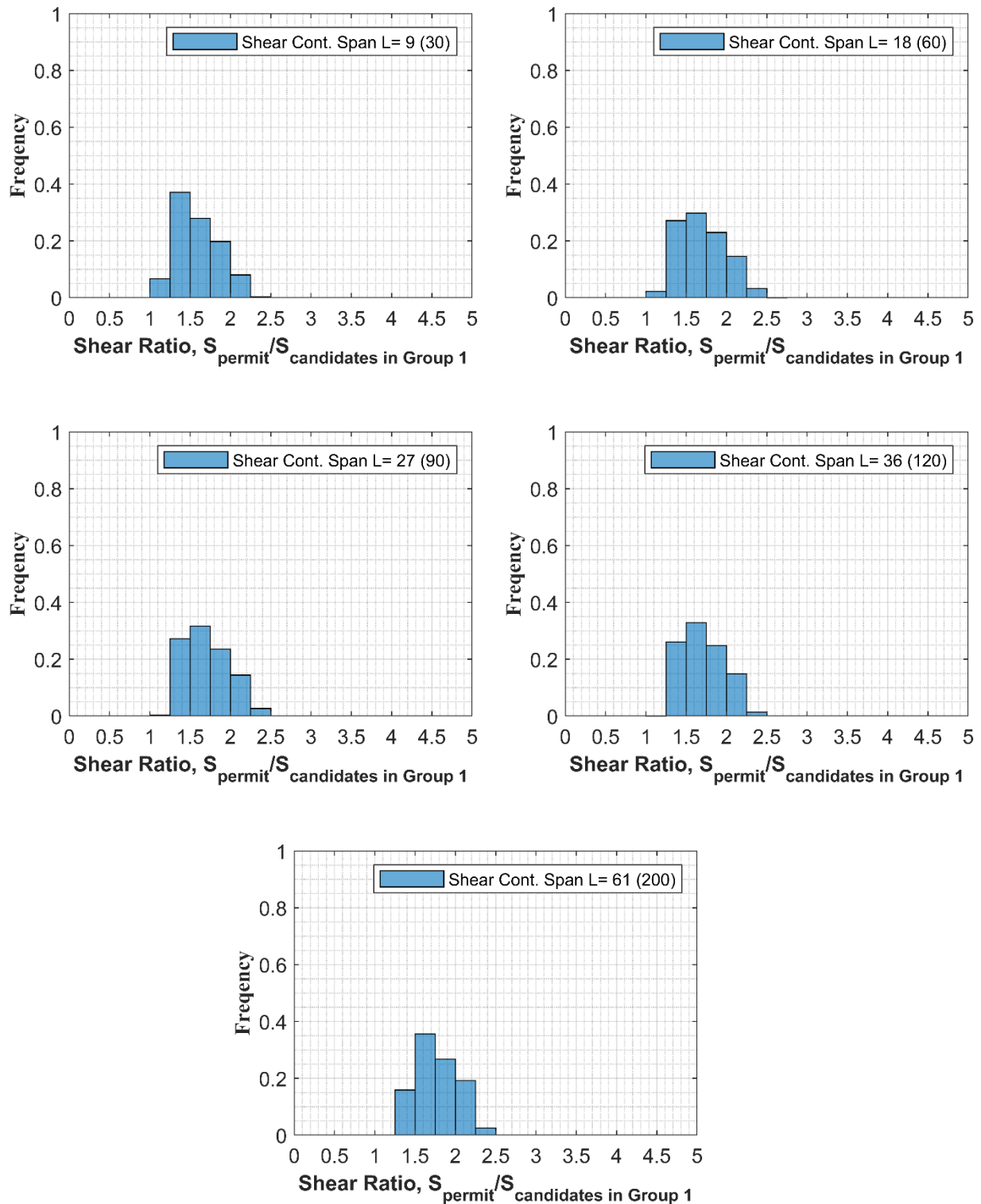


Figure 3-B.5. Load ratios of Candidates in Group 1 for various continuous span shears.

Load Ratios for Candidates in Group 2

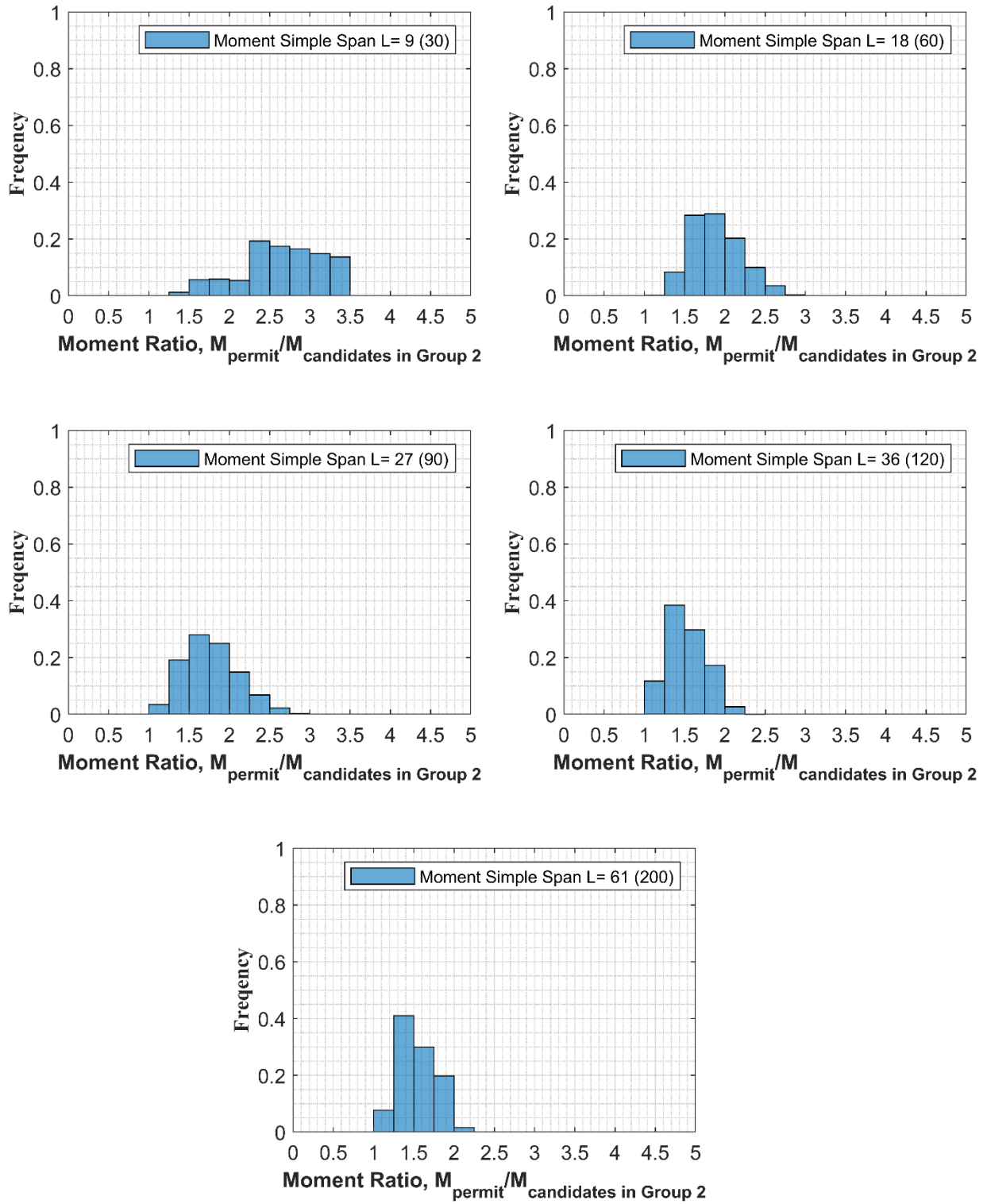


Figure 3-B.6. Load ratios of Candidates in Group 2 for various simple span moments.

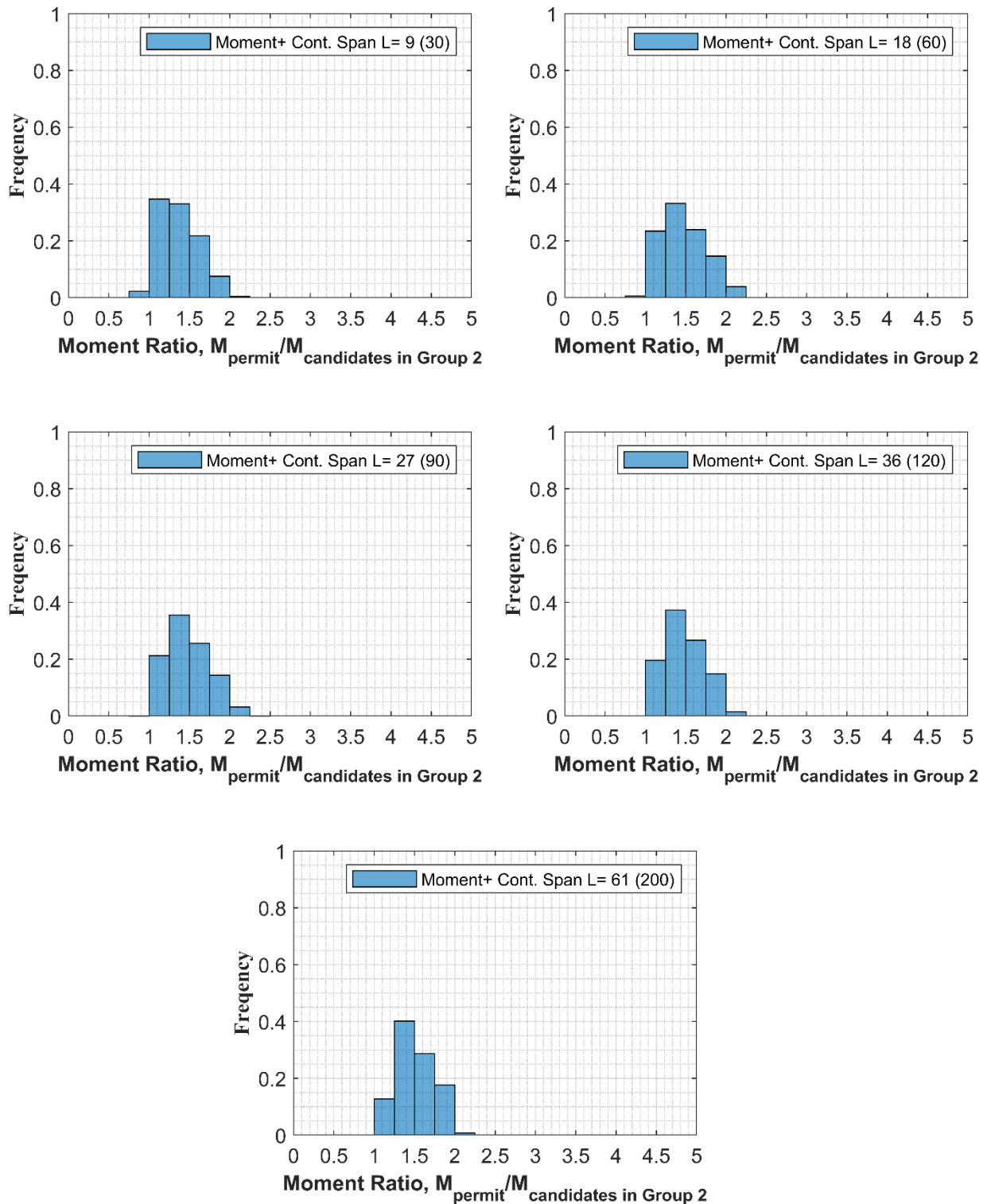


Figure 3-B.7. Load ratios of Candidates in Group 2 for various positive continuous span moments.

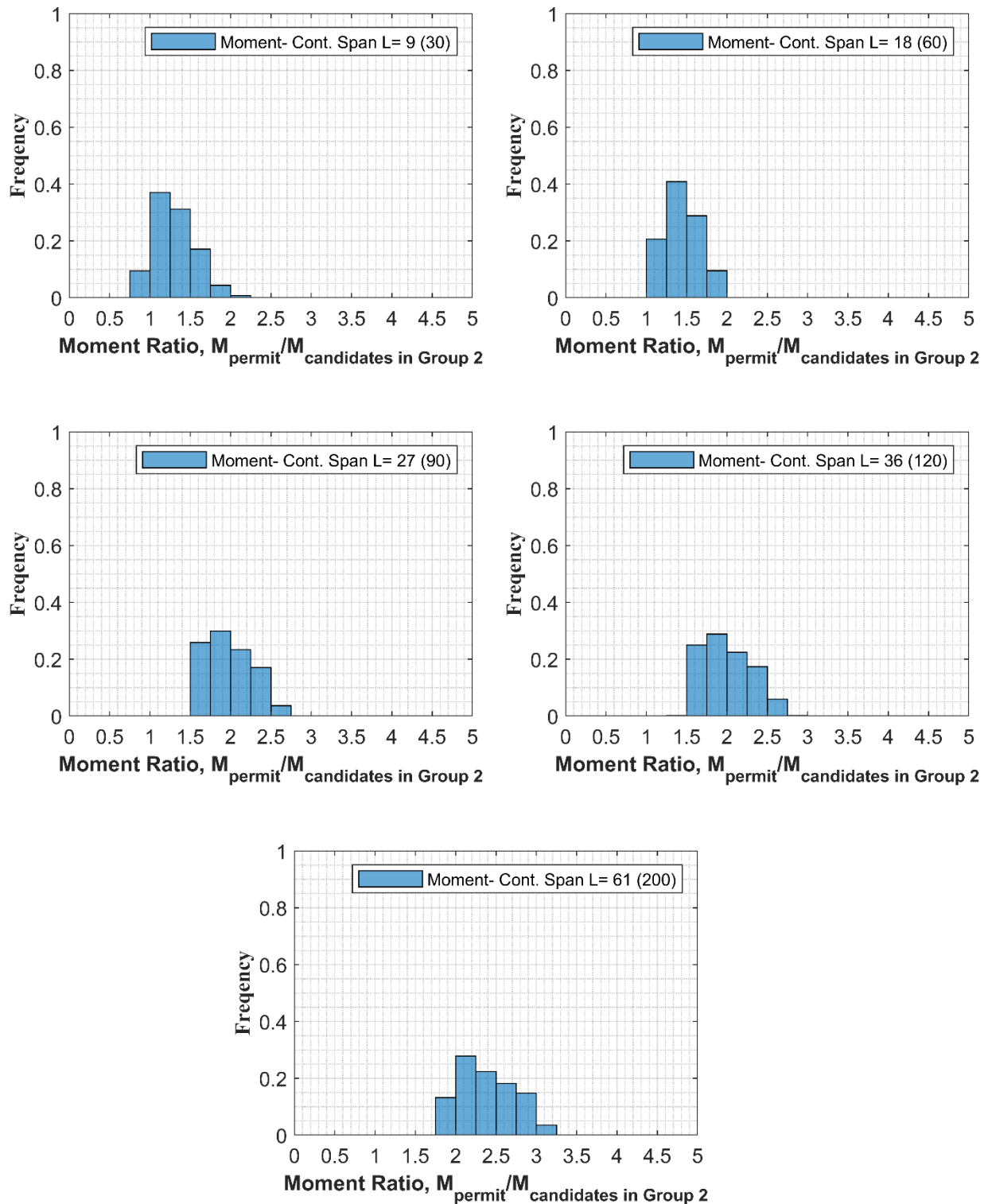


Figure 3-B.8. Load ratios of Candidates in Group 2 for various negative continuous span moments.

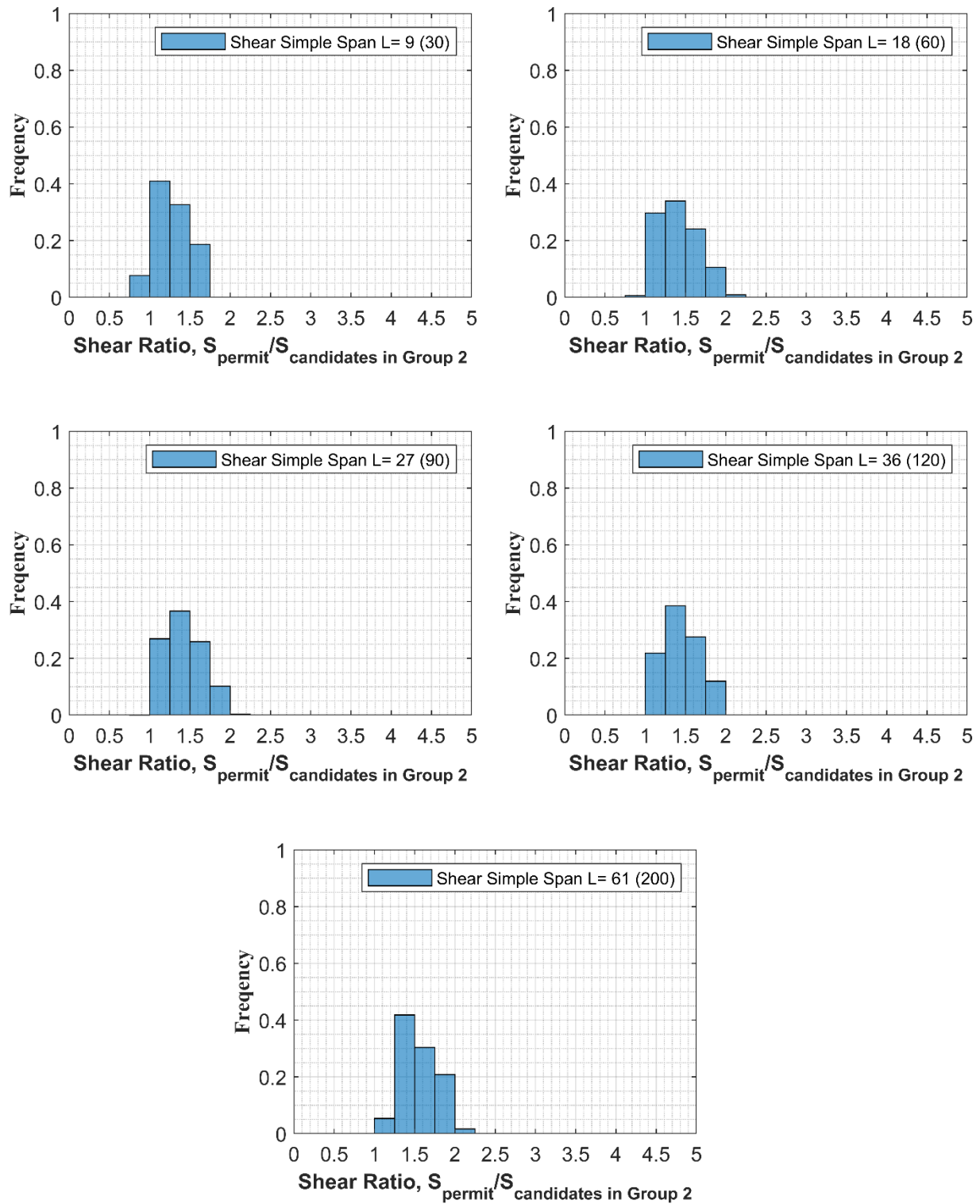


Figure 3-B.9. Load ratios of Candidates in Group 2 for various simple span shears.

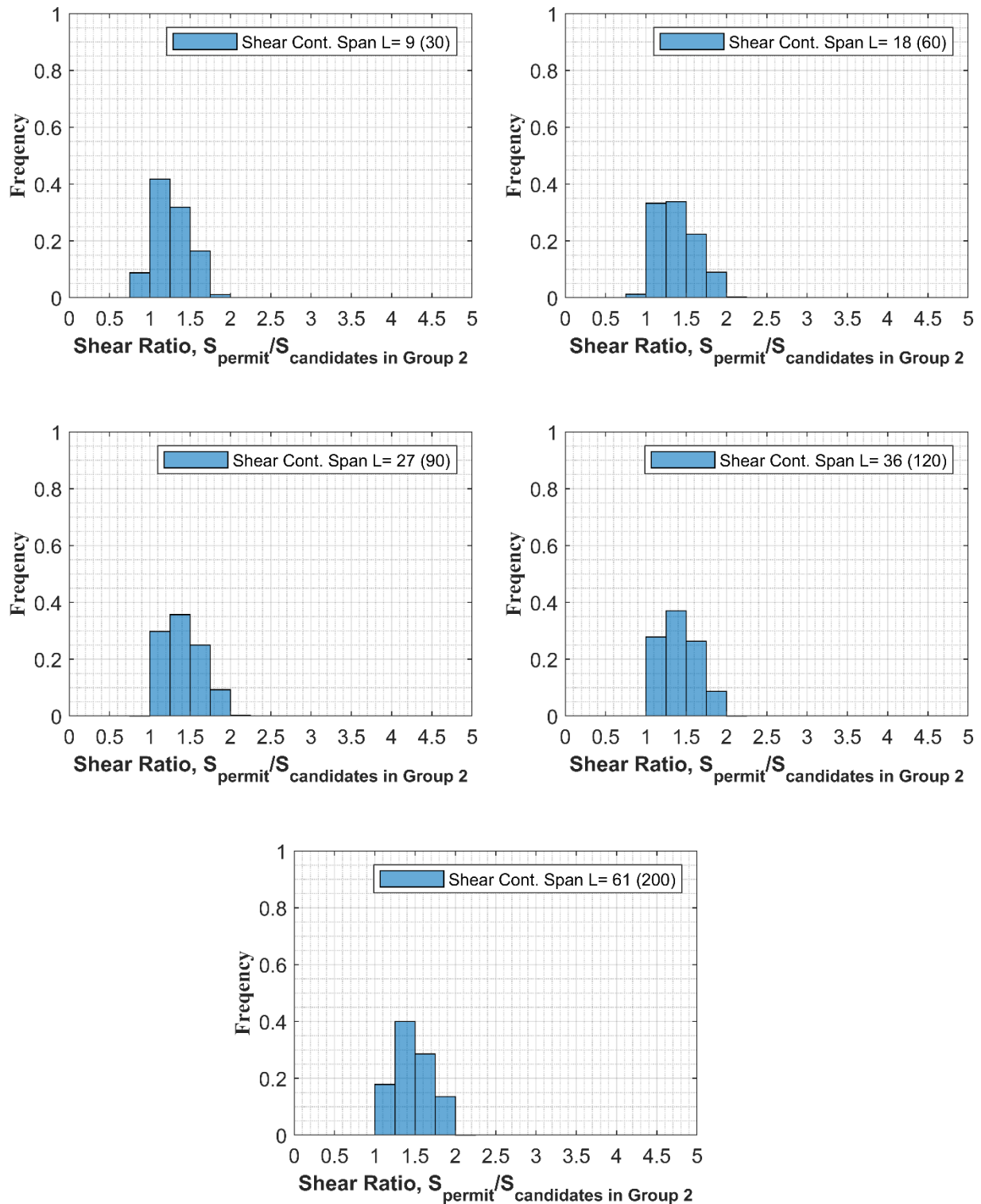


Figure 3-B.10. Load ratios of Candidates in Group 2 for various continuous span shears.

Load Ratios for Candidates in Group 3

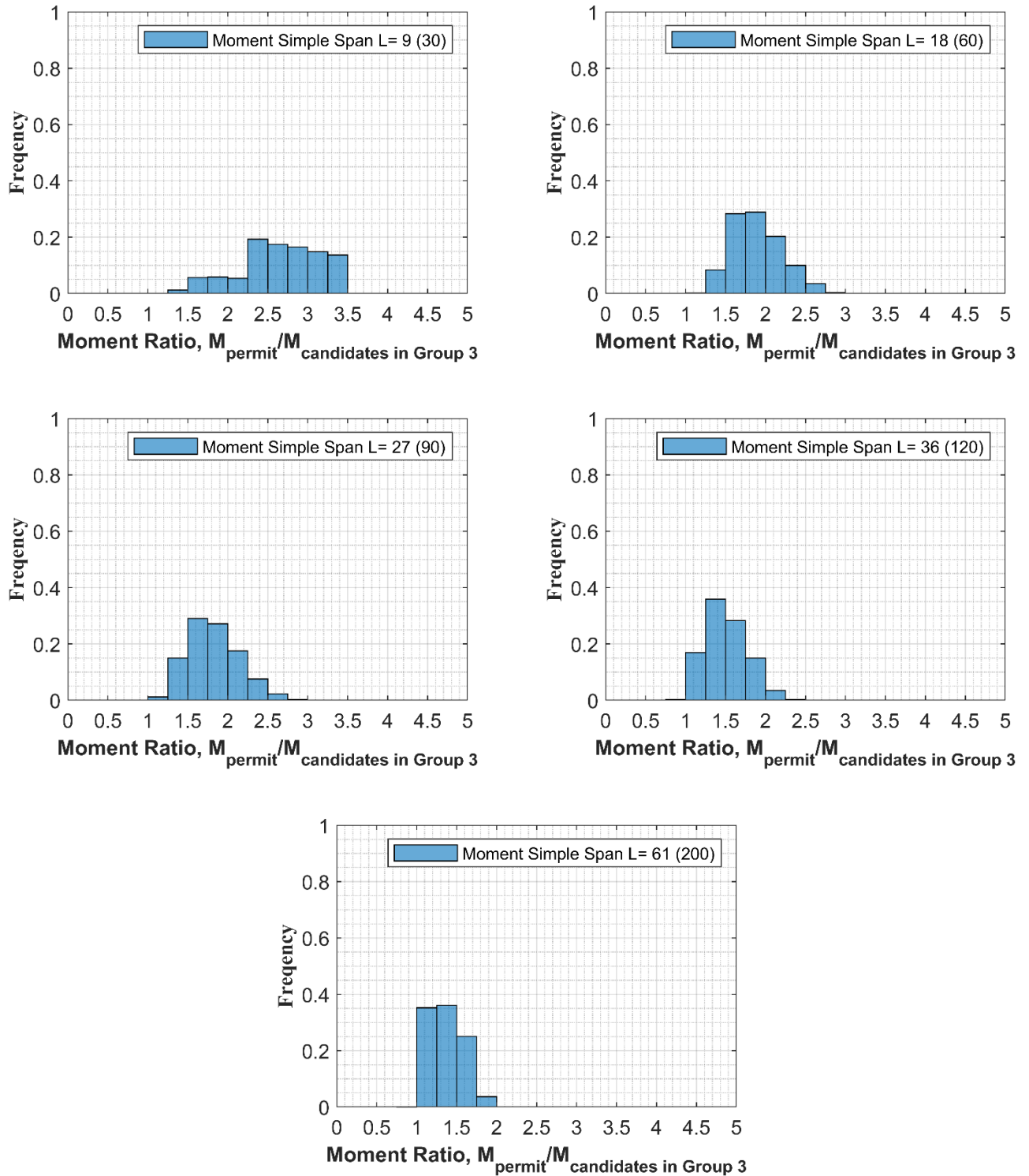


Figure 3-B.11. Load ratios of Candidates in Group 3 for various simple span moments.

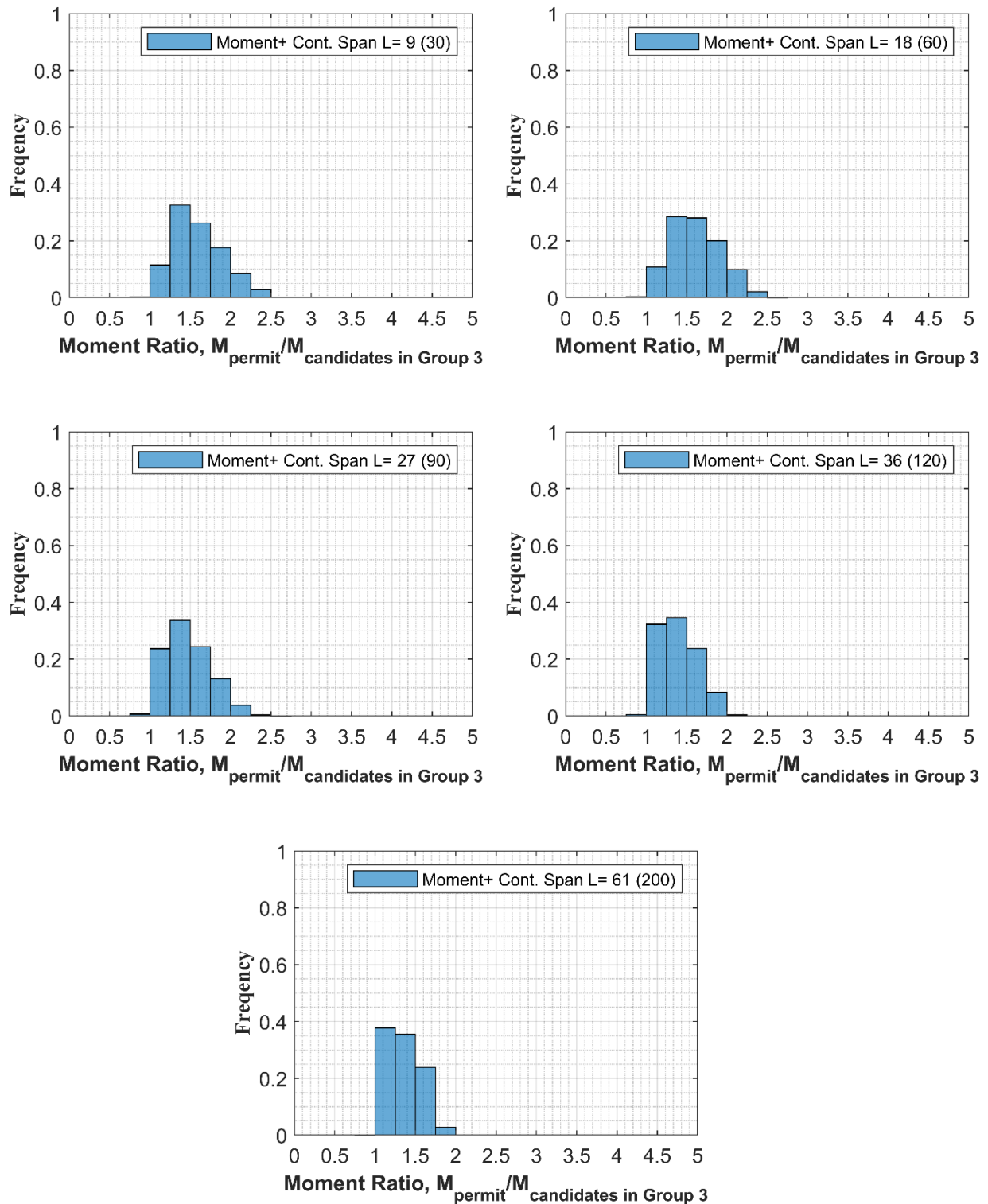


Figure 3-B.12. Load ratios of Candidates in Group 3 for various positive continuous span moments.

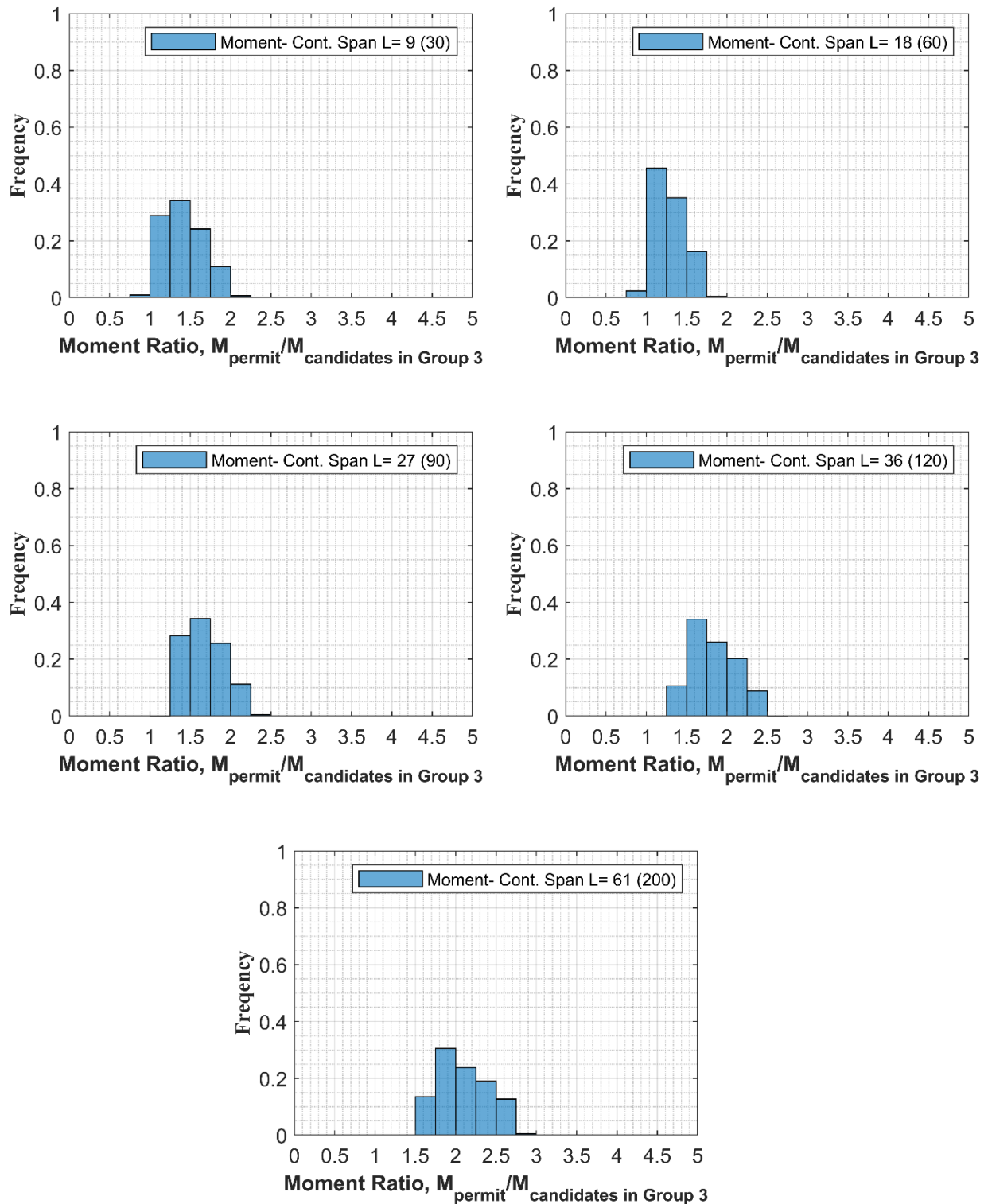


Figure 3-B.13. Load ratios of Candidates in Group 3 for various negative continuous span moments.

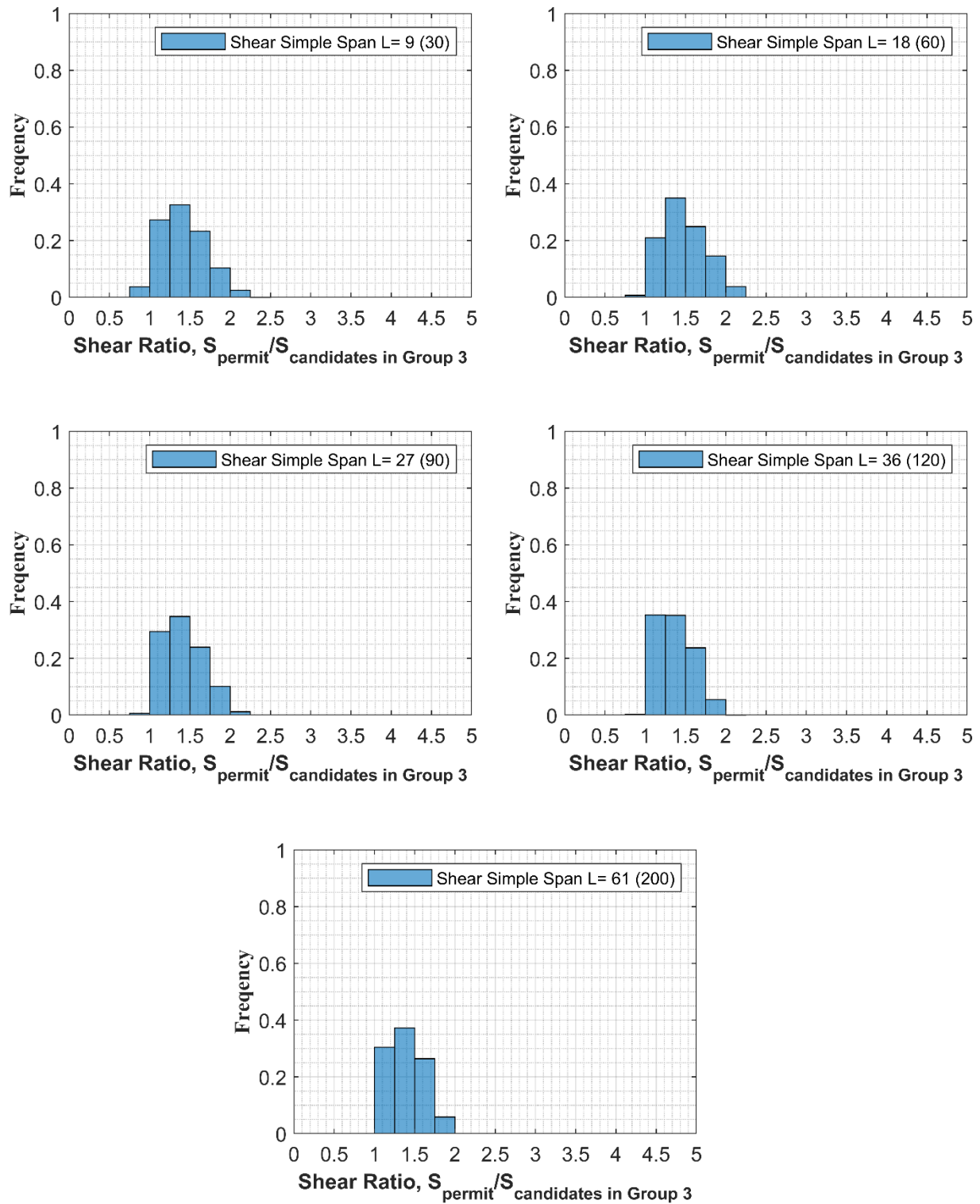


Figure 3-B.14. Load ratios of Candidates in Group 3 for various simple span shears.

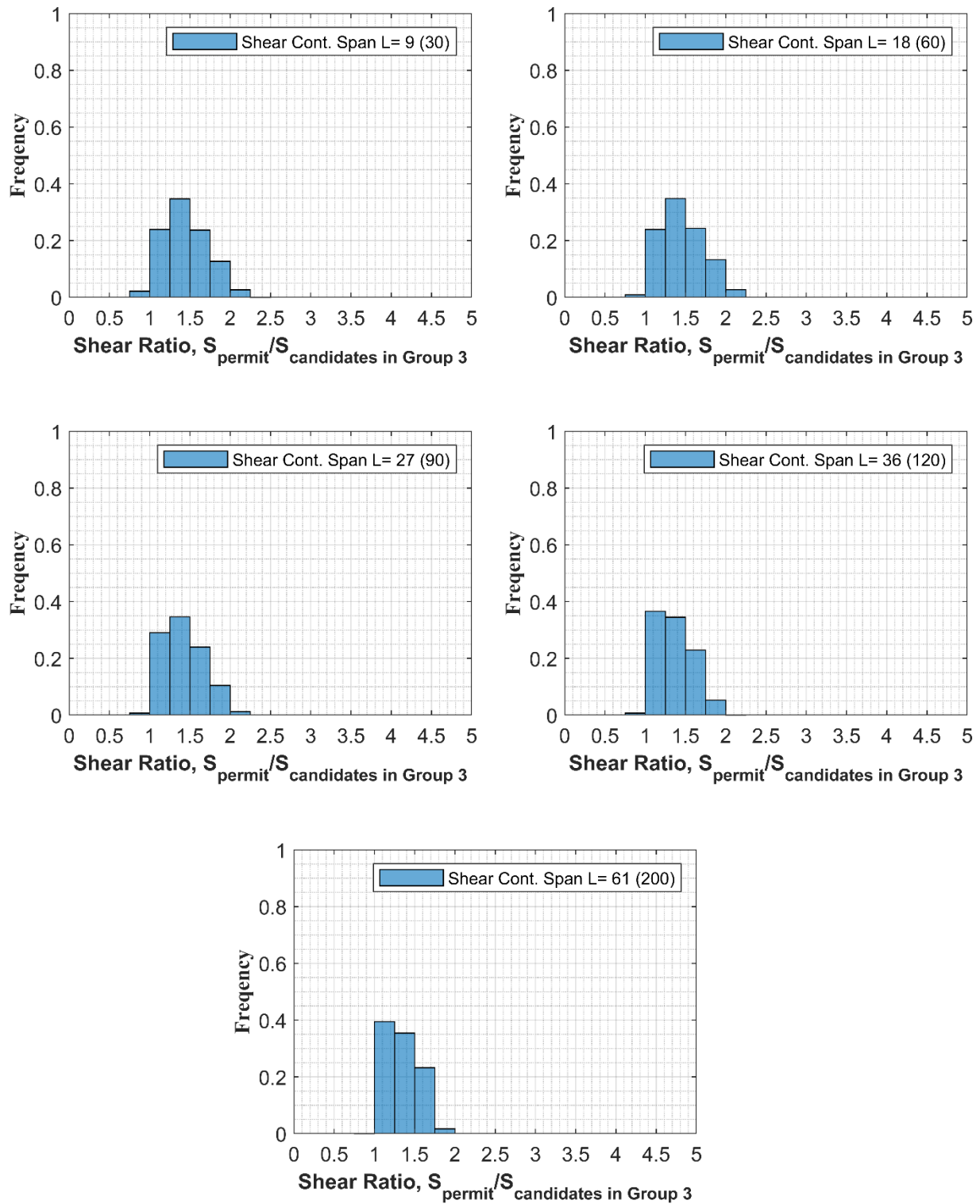


Figure 3-B.15. Load ratios of Candidates in Group 3 for various continuous span shears.

Load Ratios for Candidates in Group 4

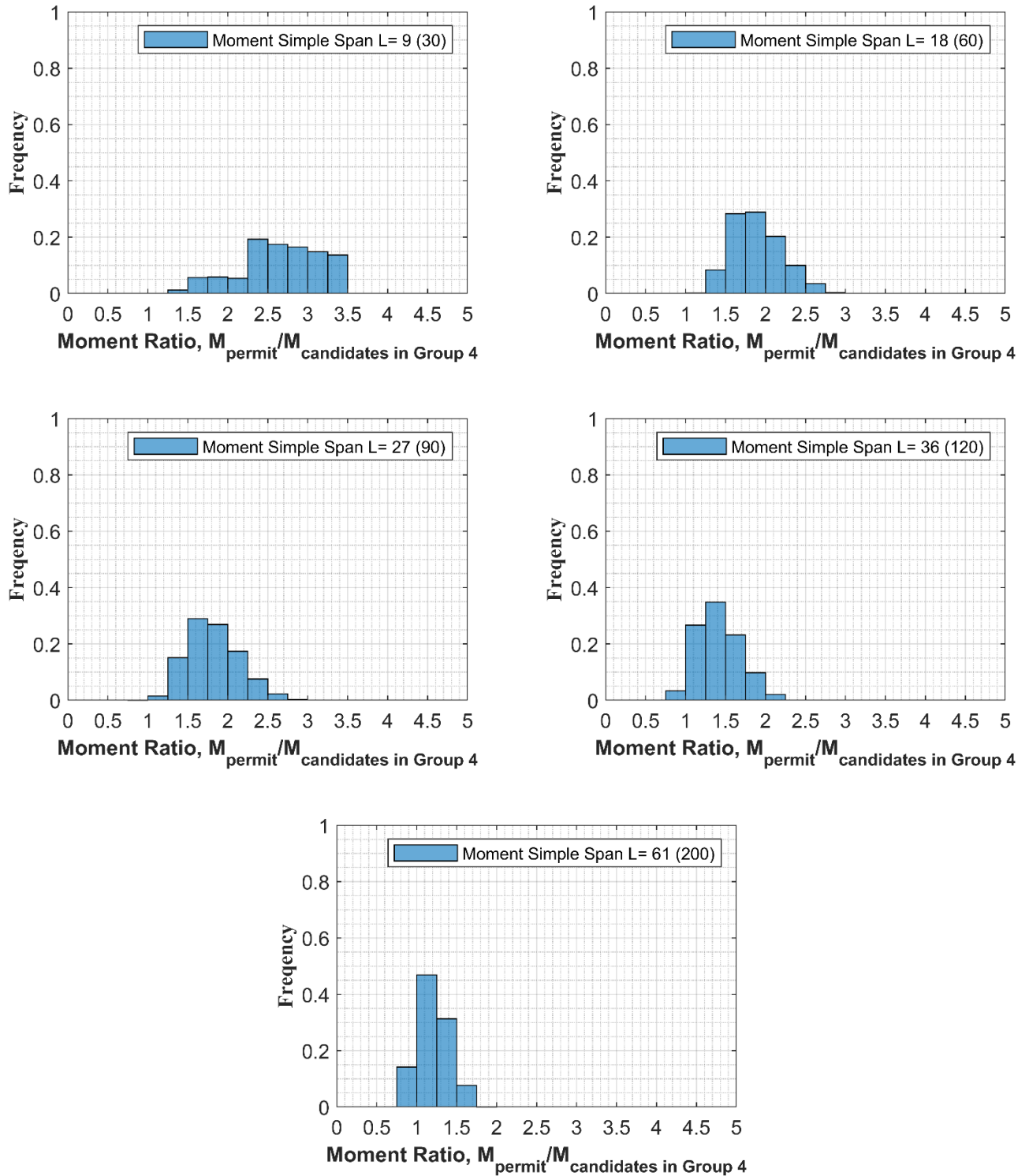


Figure 3-B.16. Load ratios of Candidates in Group 4 for various simple span moments.

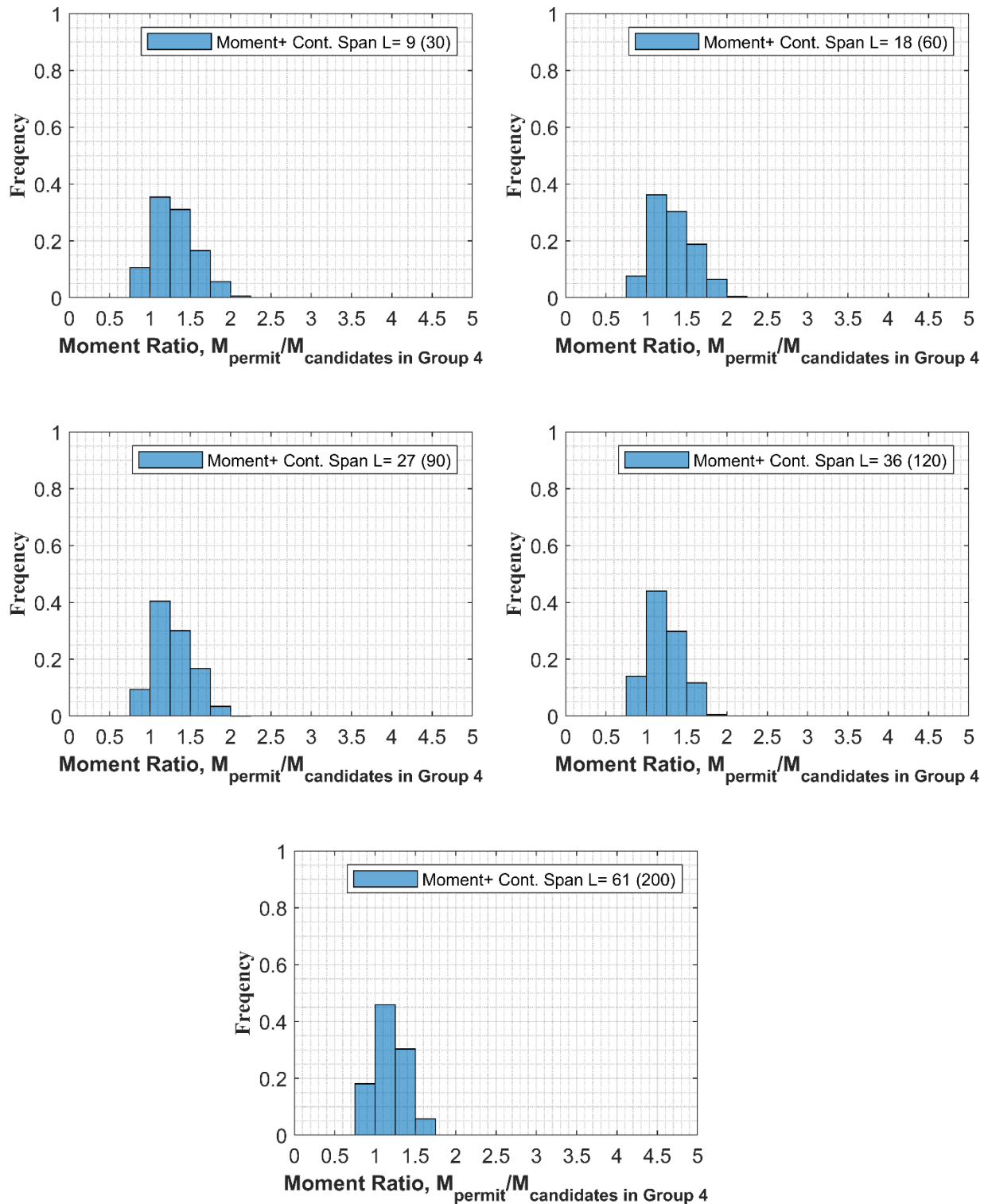


Figure 3-B.17. Load ratios of Candidates in Group 4 for various positive continuous span moments.

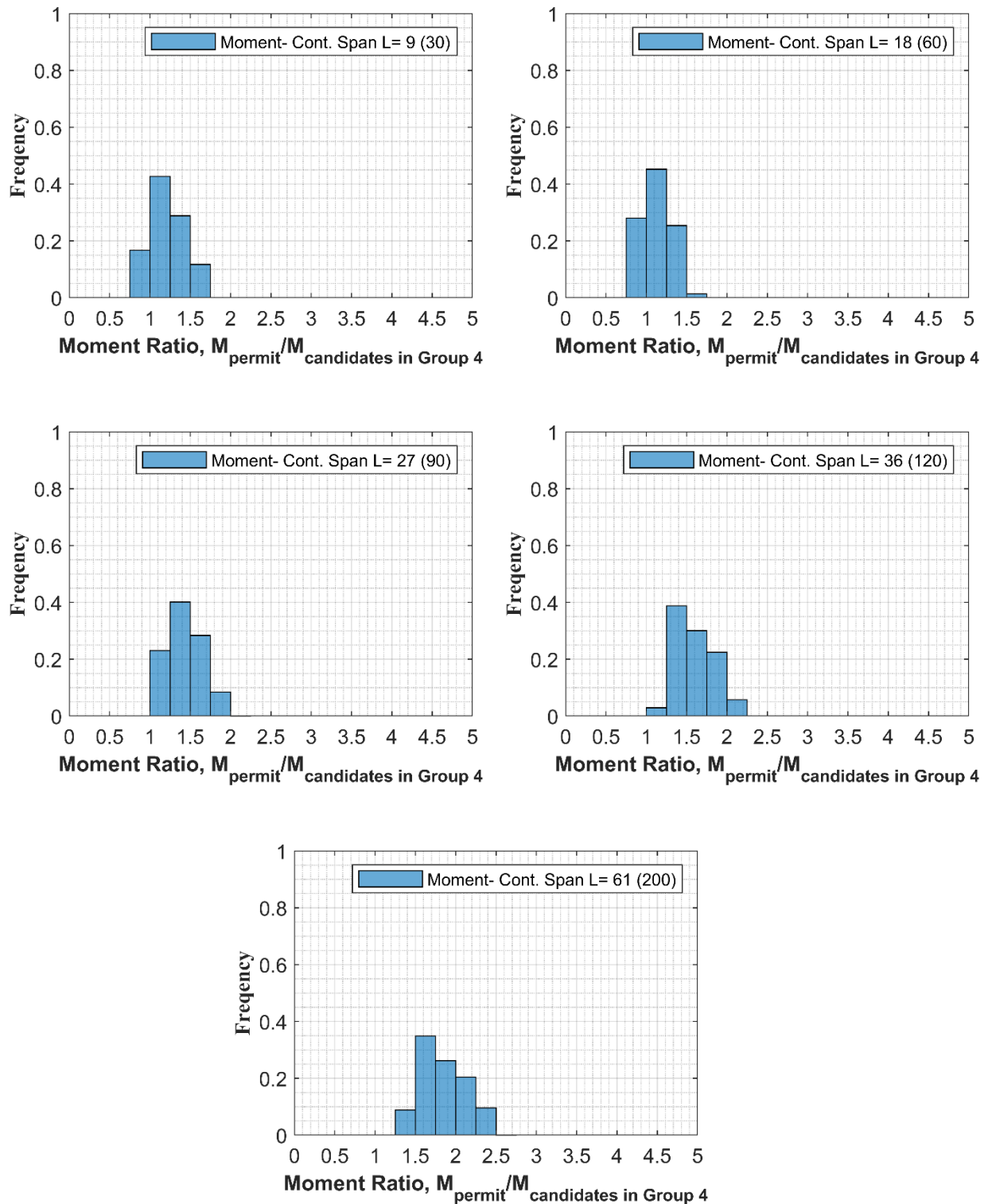


Figure 3-B.18. Load ratios of Candidates in Group 4 for various negative continuous span moments.

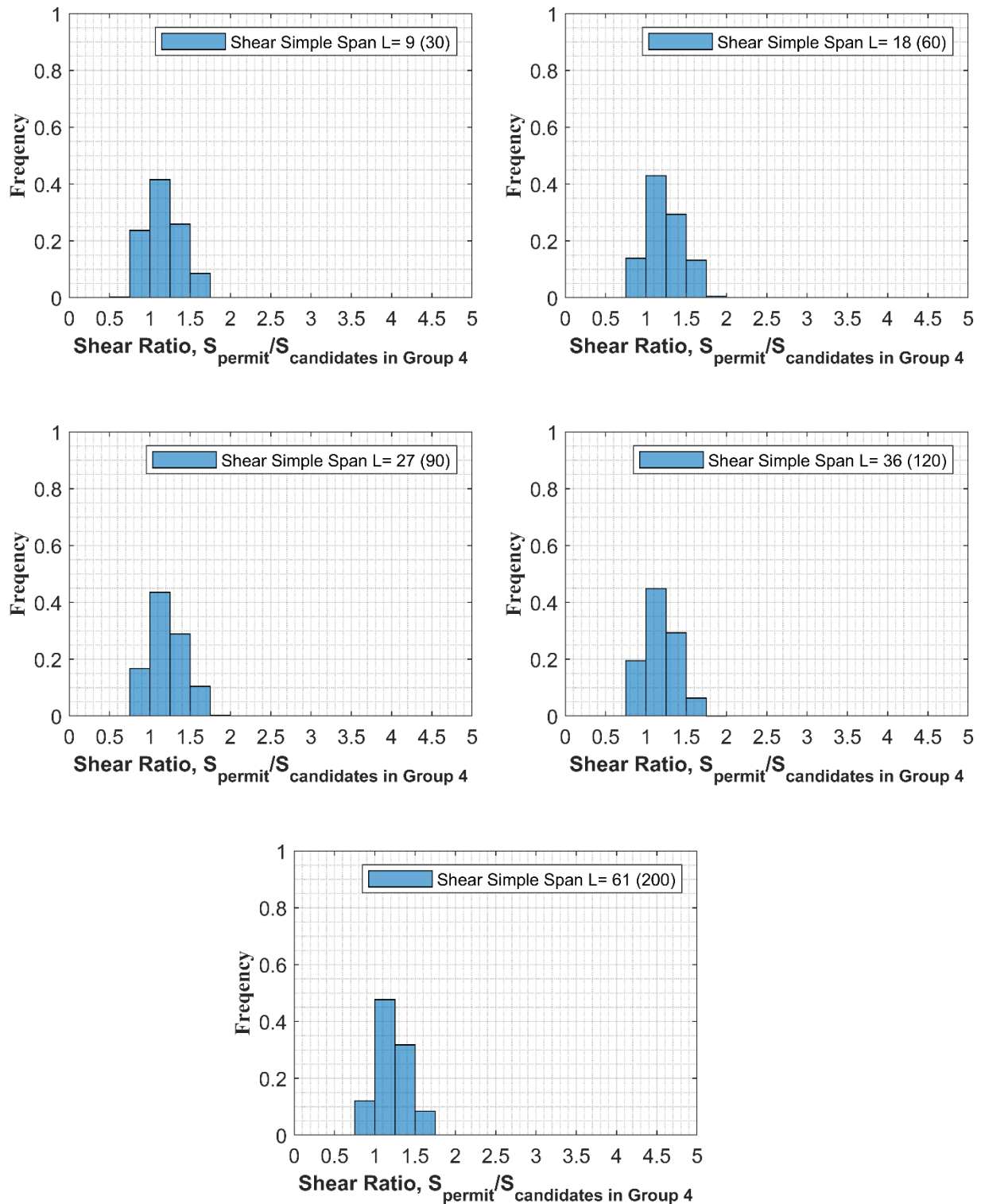


Figure 3-B.19. Load ratios of Candidates in Group 4 for various simple span shears.

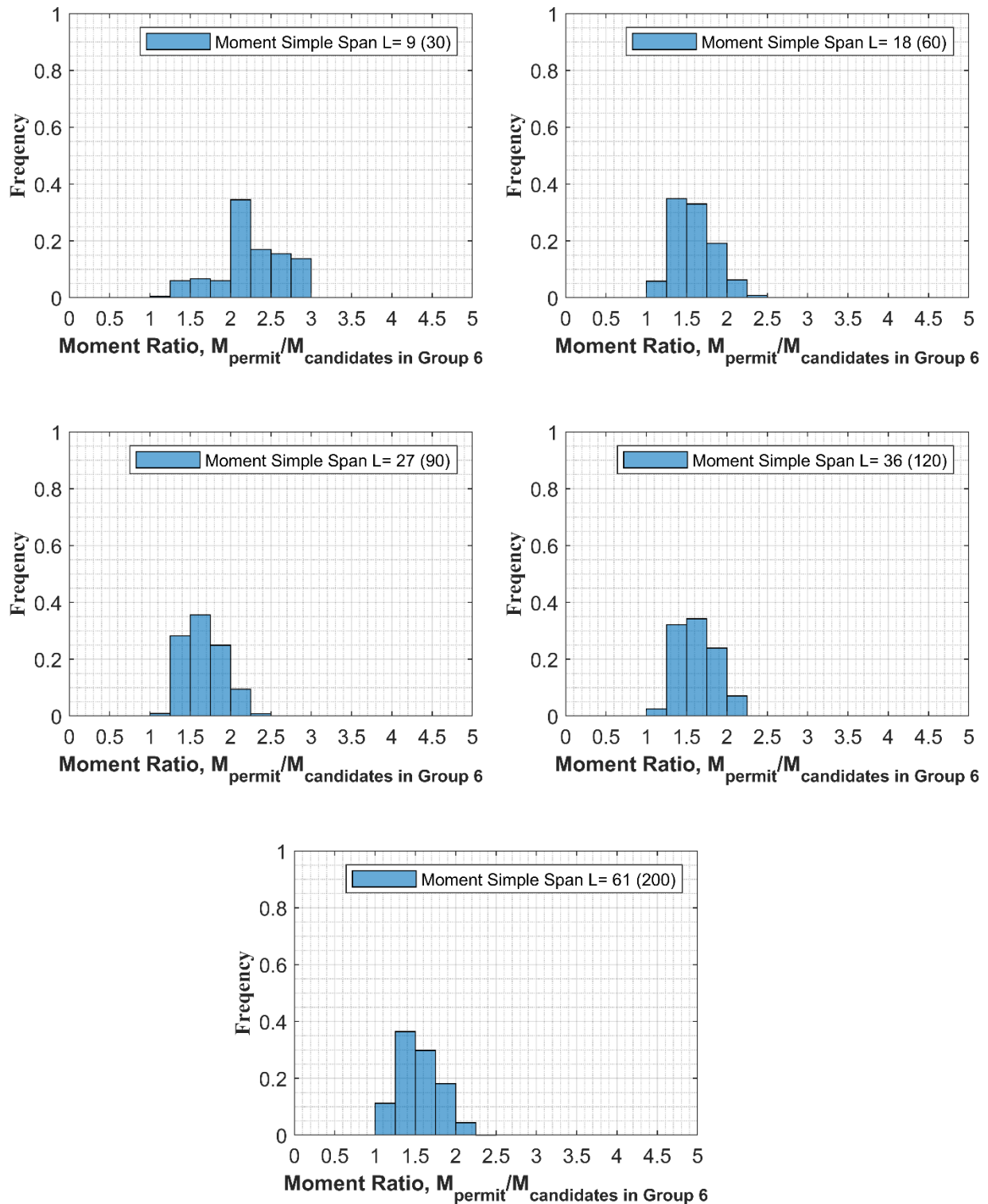


Figure 3-B. 20. Load ratios of Candidates in Group 4 for various continuous span shears.

Load Ratios for Candidates in Group 6 for Multi Trip Permits

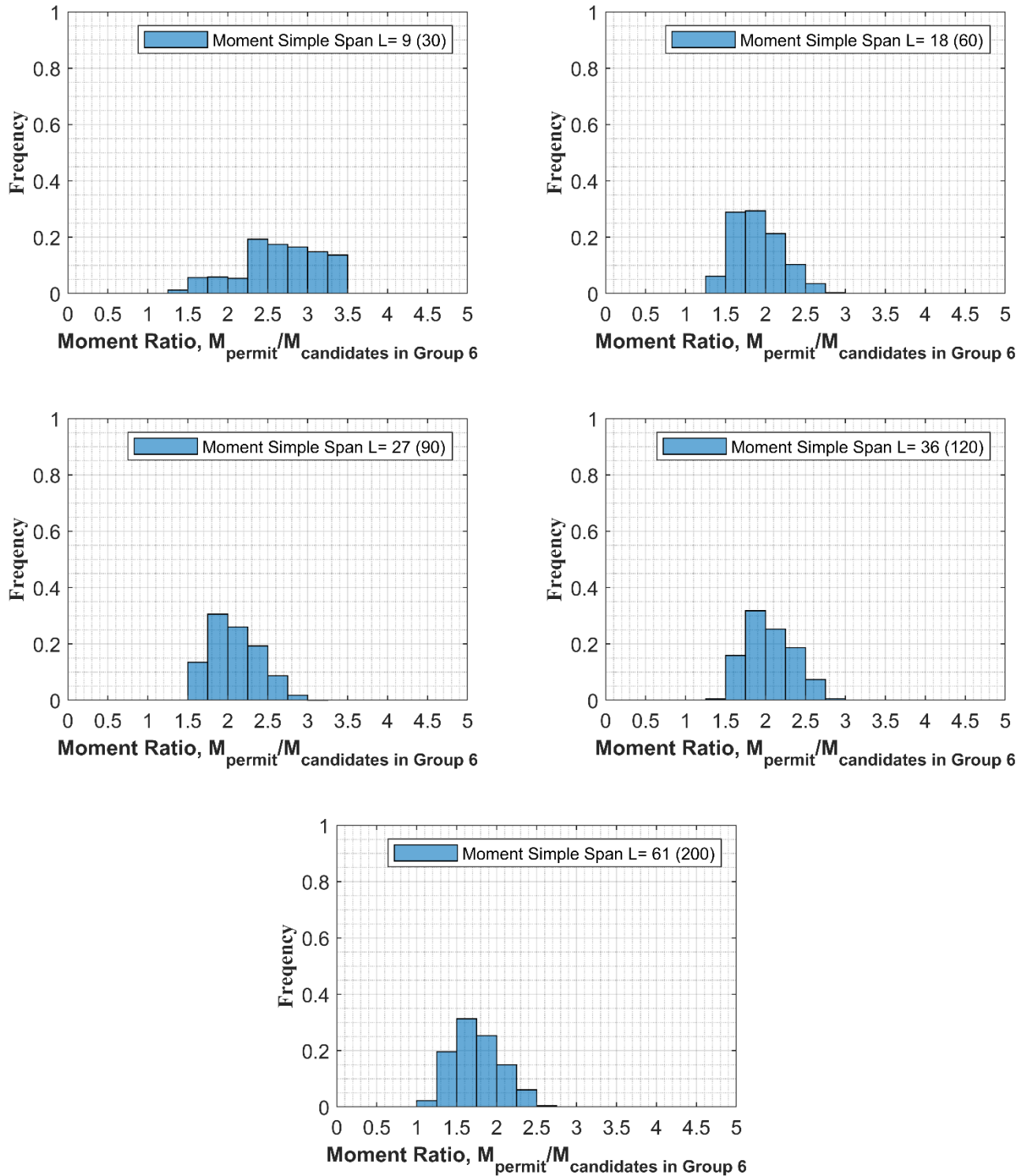


Figure 3-B.21. Load ratios of Candidates in Group 6 for various simple span moments.

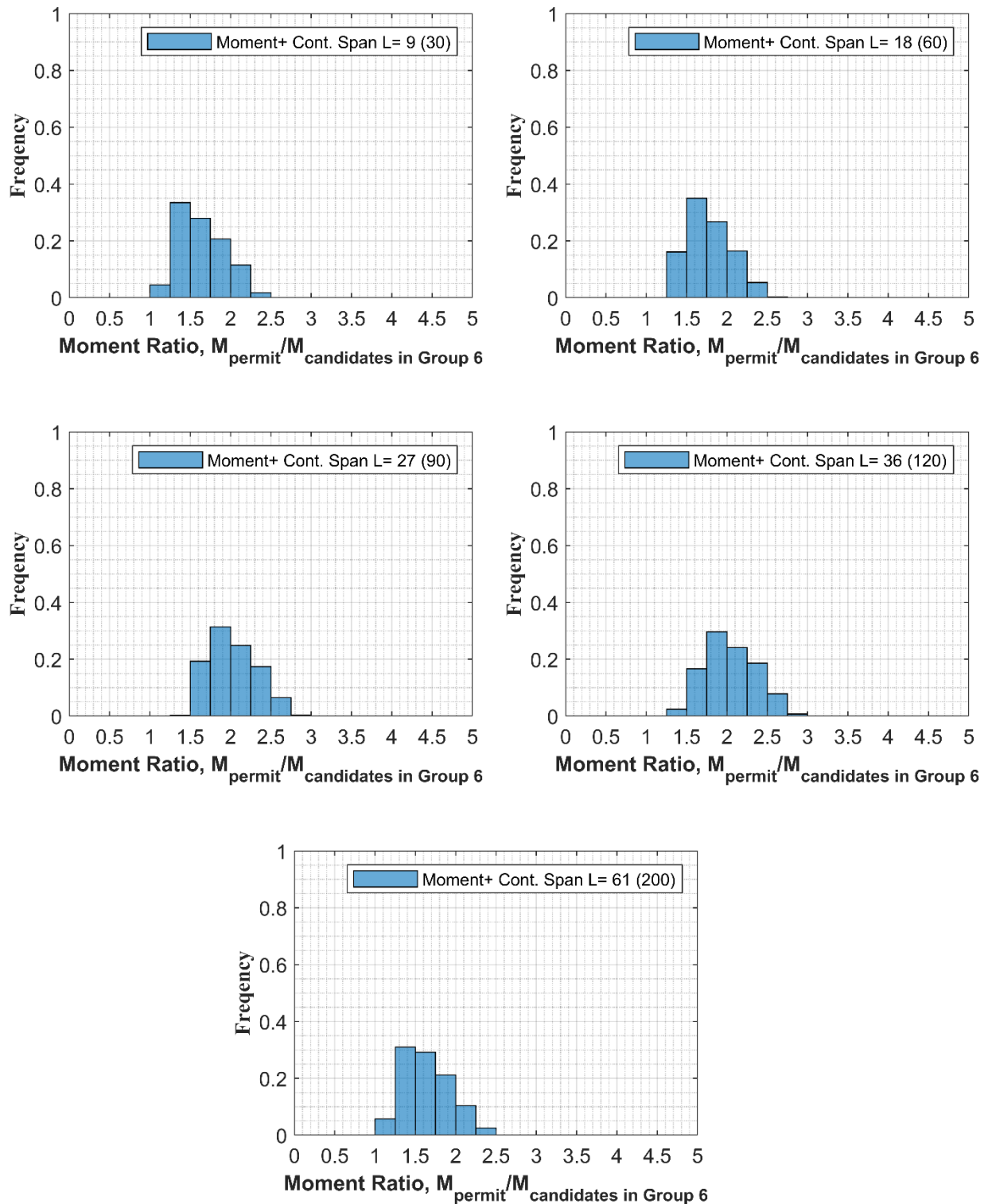


Figure 3-B.22. Load ratios of Candidates in Group 6 for various positive continuous span moments.

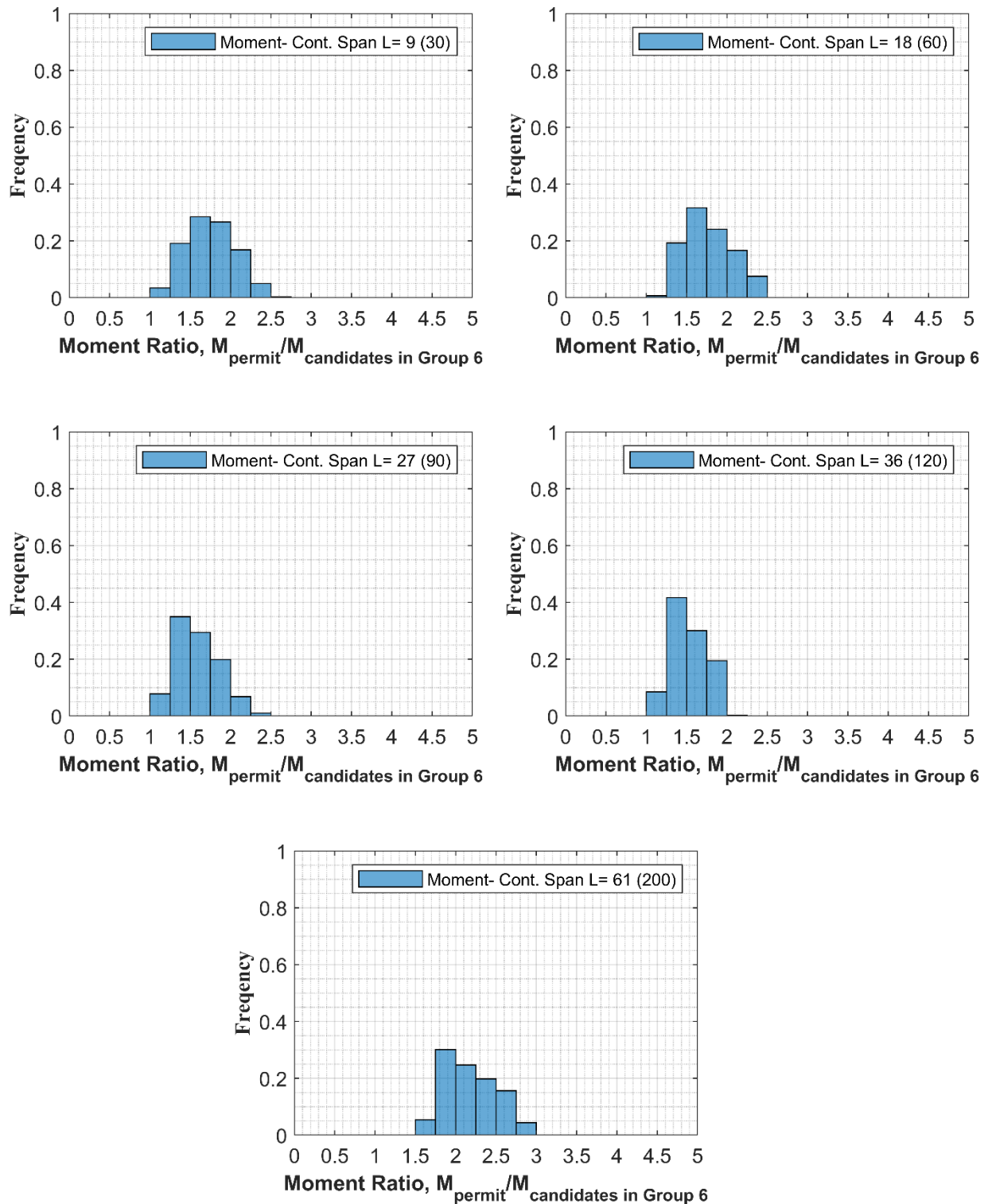


Figure 3-B. 23. Load ratios of Candidates in Group 6 for various negative continuous span moments.

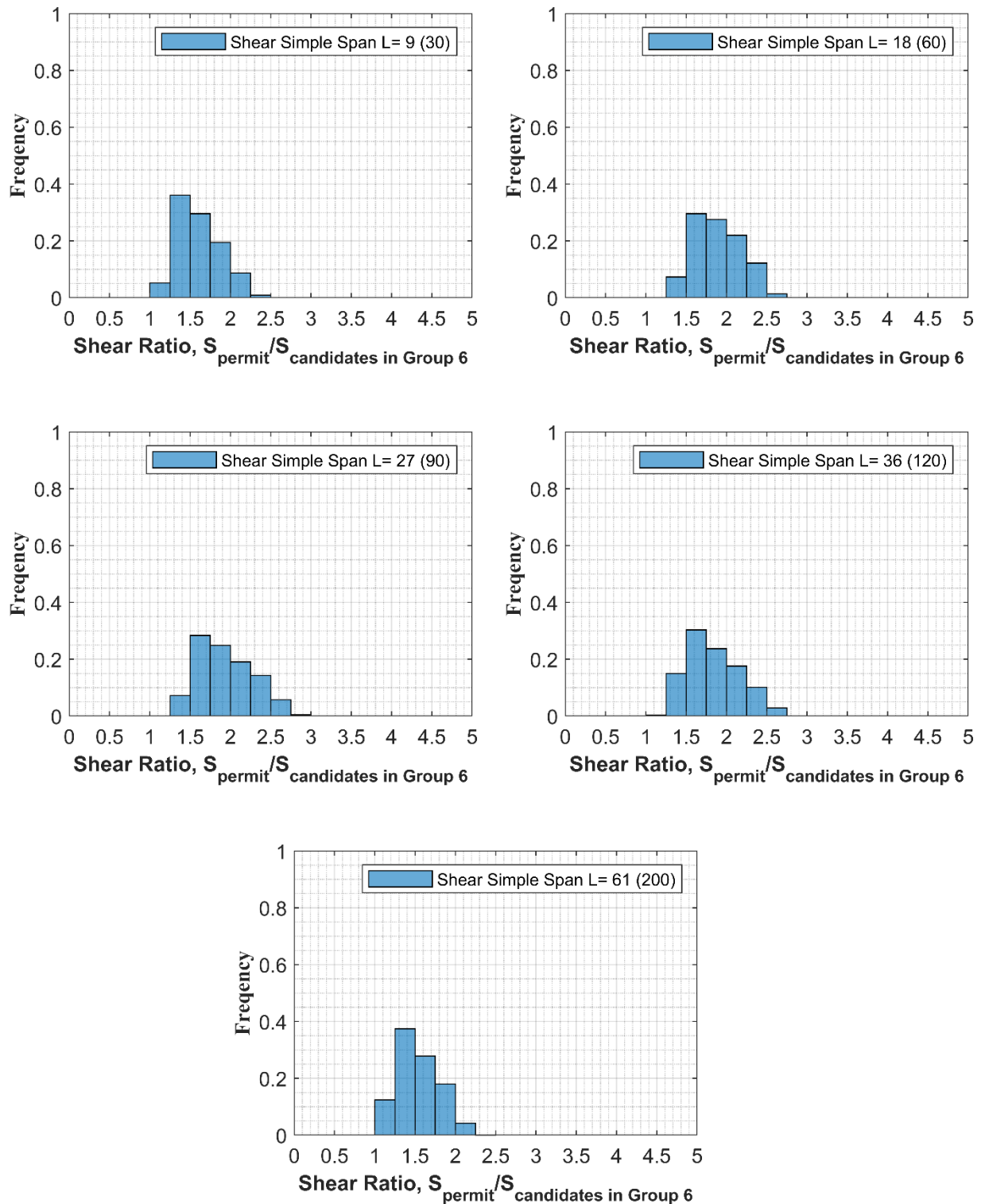


Figure 3-B.24. Load ratios of Candidates in Group 6 for various simple span shears.

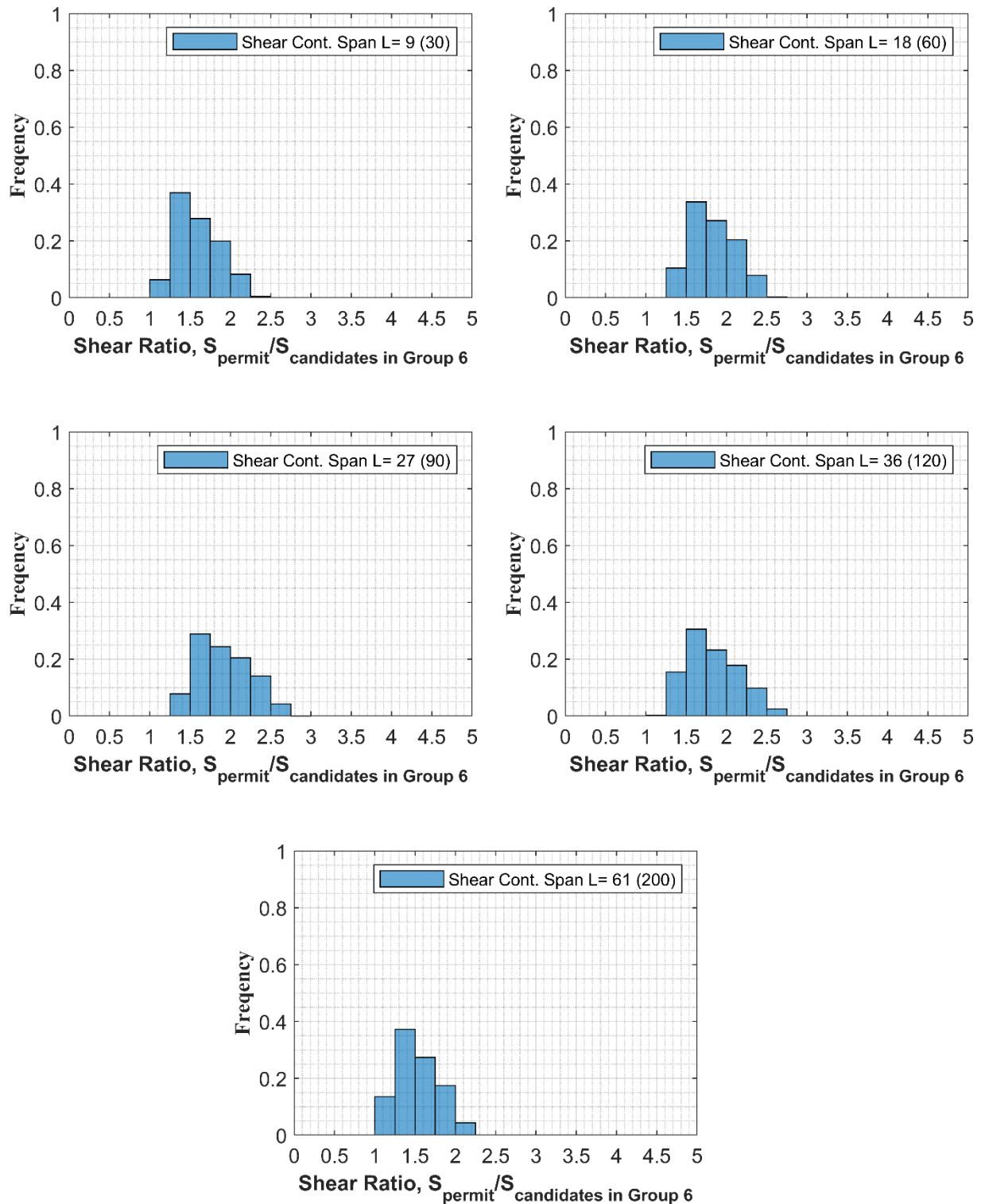


Figure 3-B.25. Load ratios of Candidates in Group 6 for various continuous span shears.

Load Ratios for Candidates in Group 5

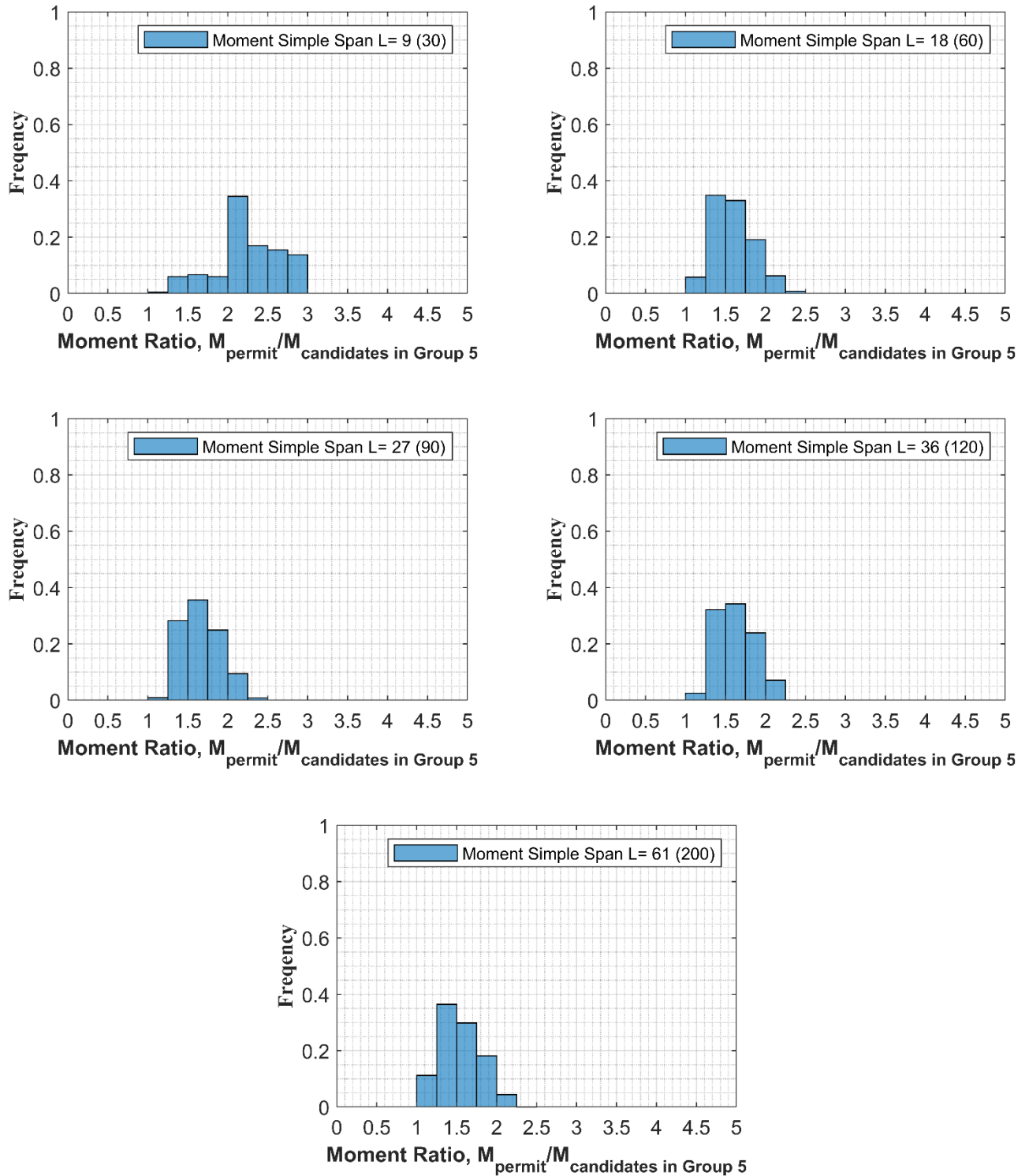


Figure 3-B.26. Load ratios of Candidates in Group 5 for various simple span moments.

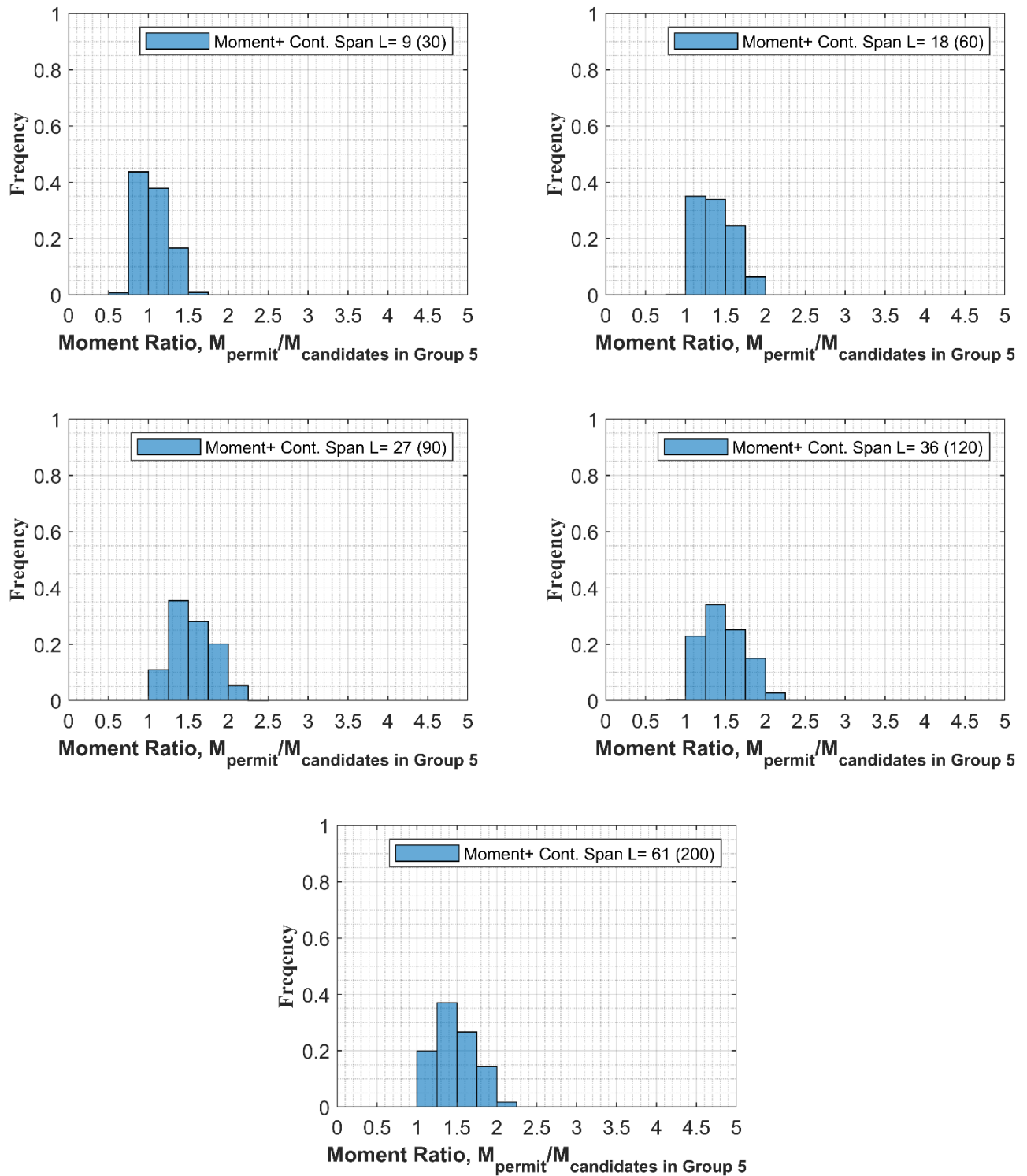


Figure 3-B.27. Load ratios of Candidates in Group 5 for various positive continuous span moments.

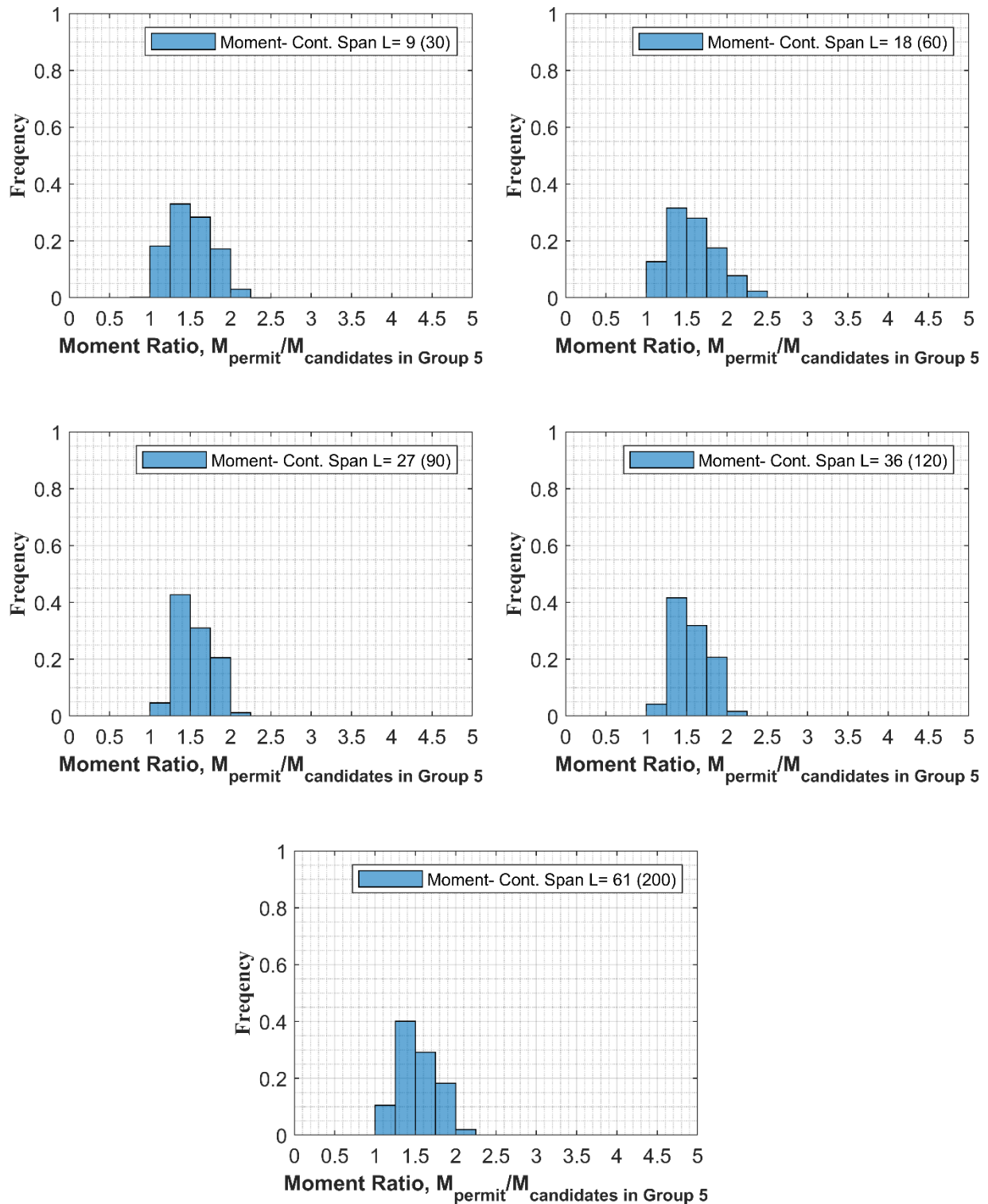


Figure 3-B.28. Load ratios of Candidates in Group 5 for various negative continuous span moments.

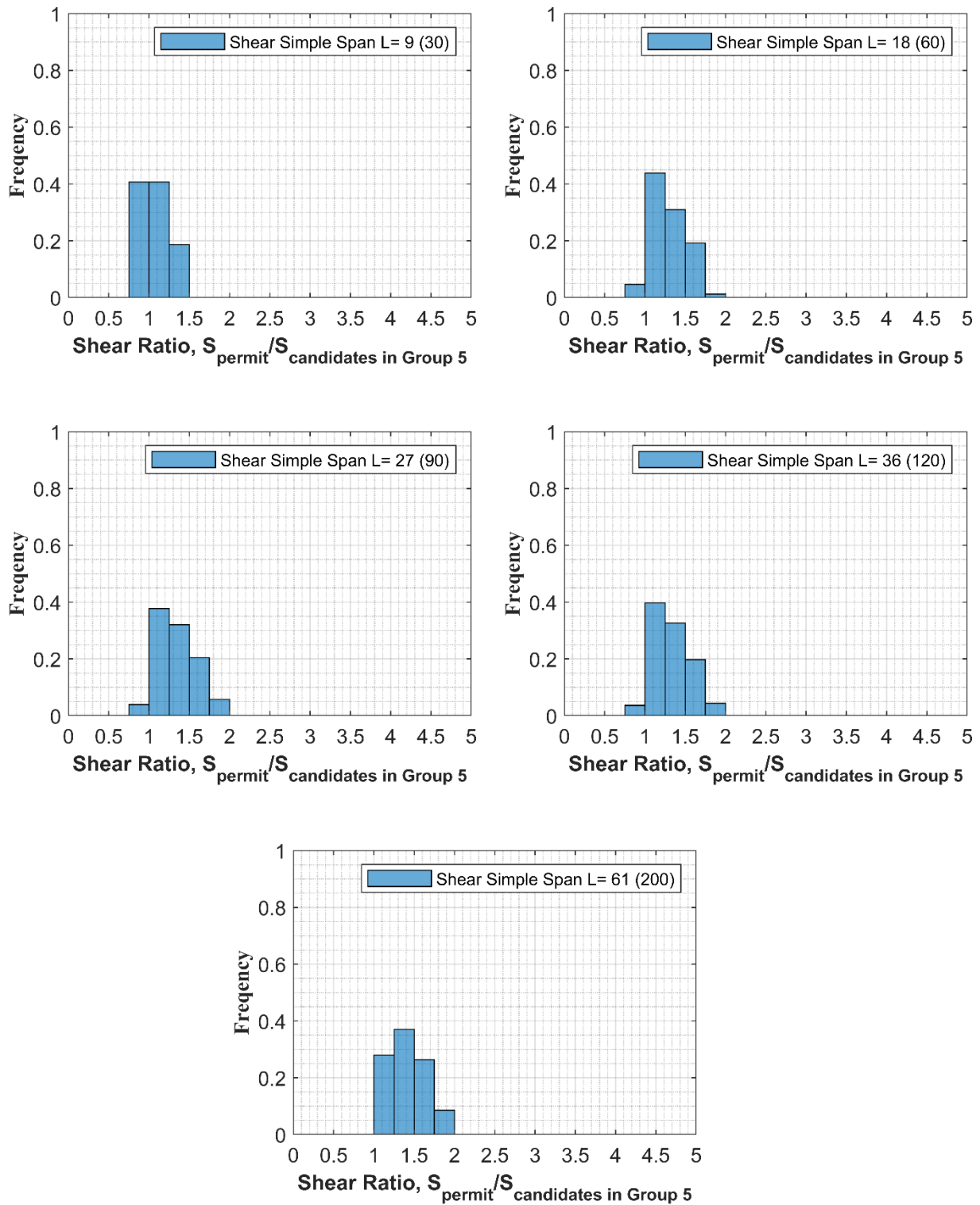


Figure 3-B. 29. Load ratios of Candidates in Group 5 for various simple span shears.

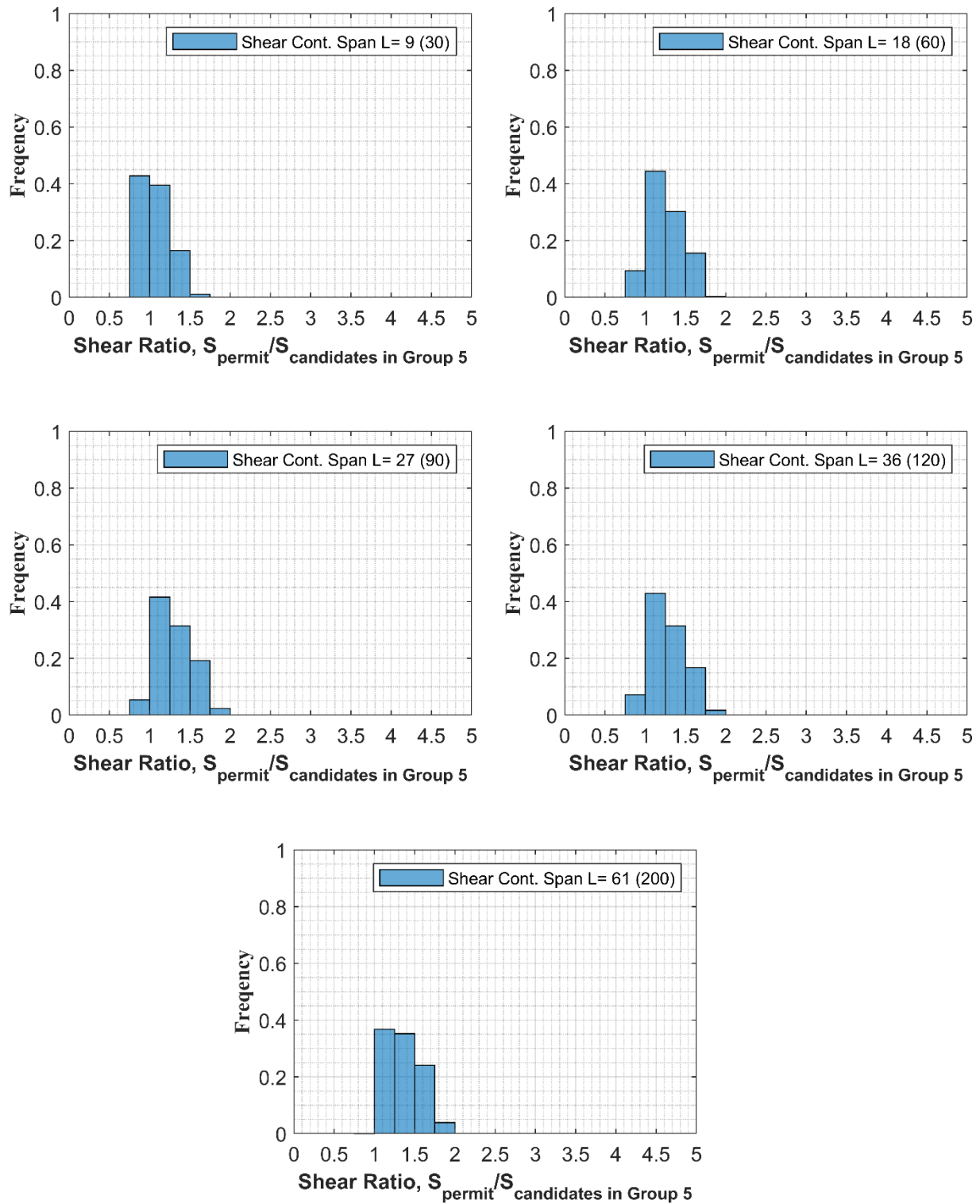


Figure 3-B. 30. Load ratios of Candidates in Group 5 for various continuous span shears.

Load Ratios for Candidates in Group 6

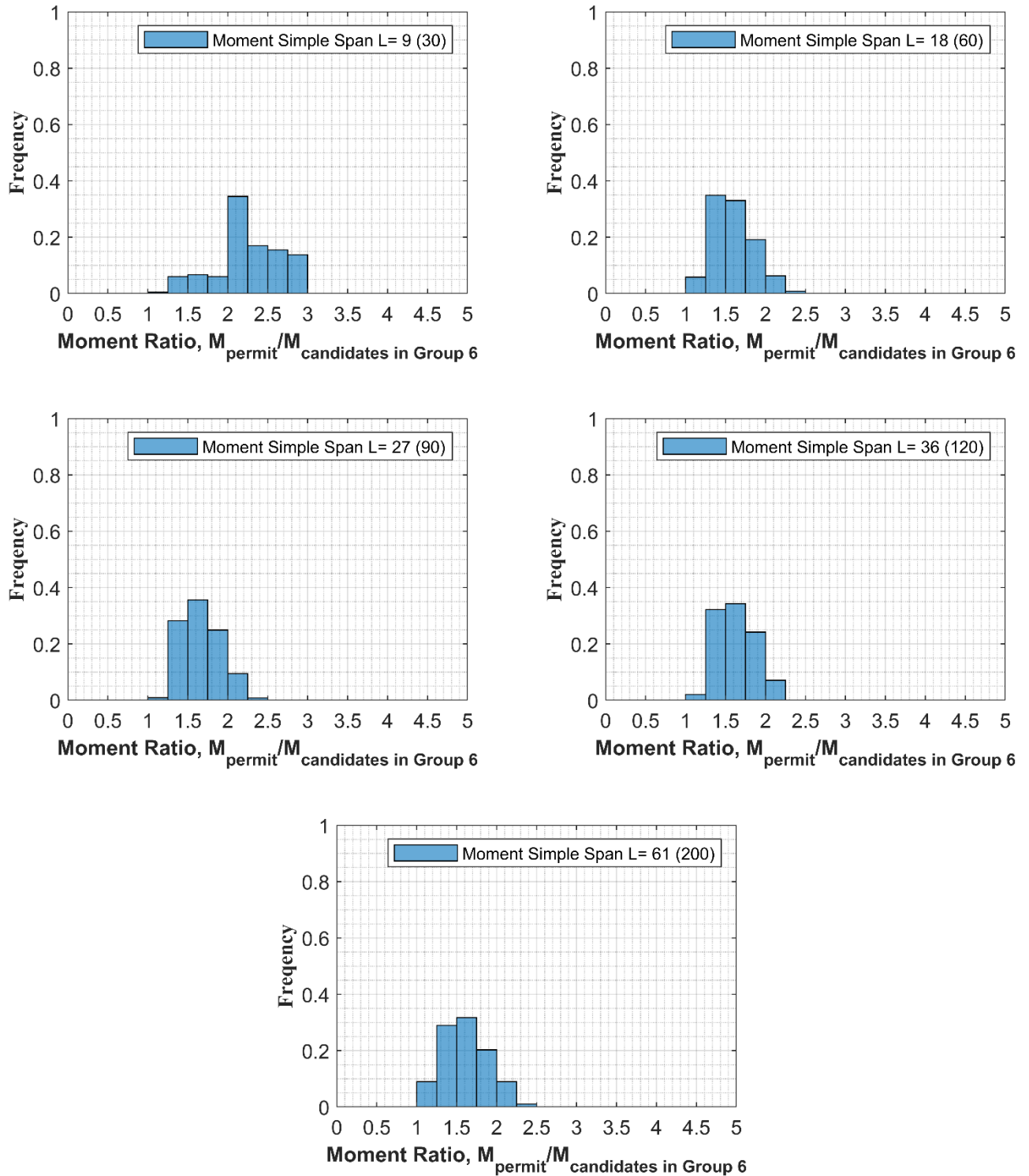


Figure 3-B. 31. Load ratios of Candidates in Group 6 for various simple span moments.

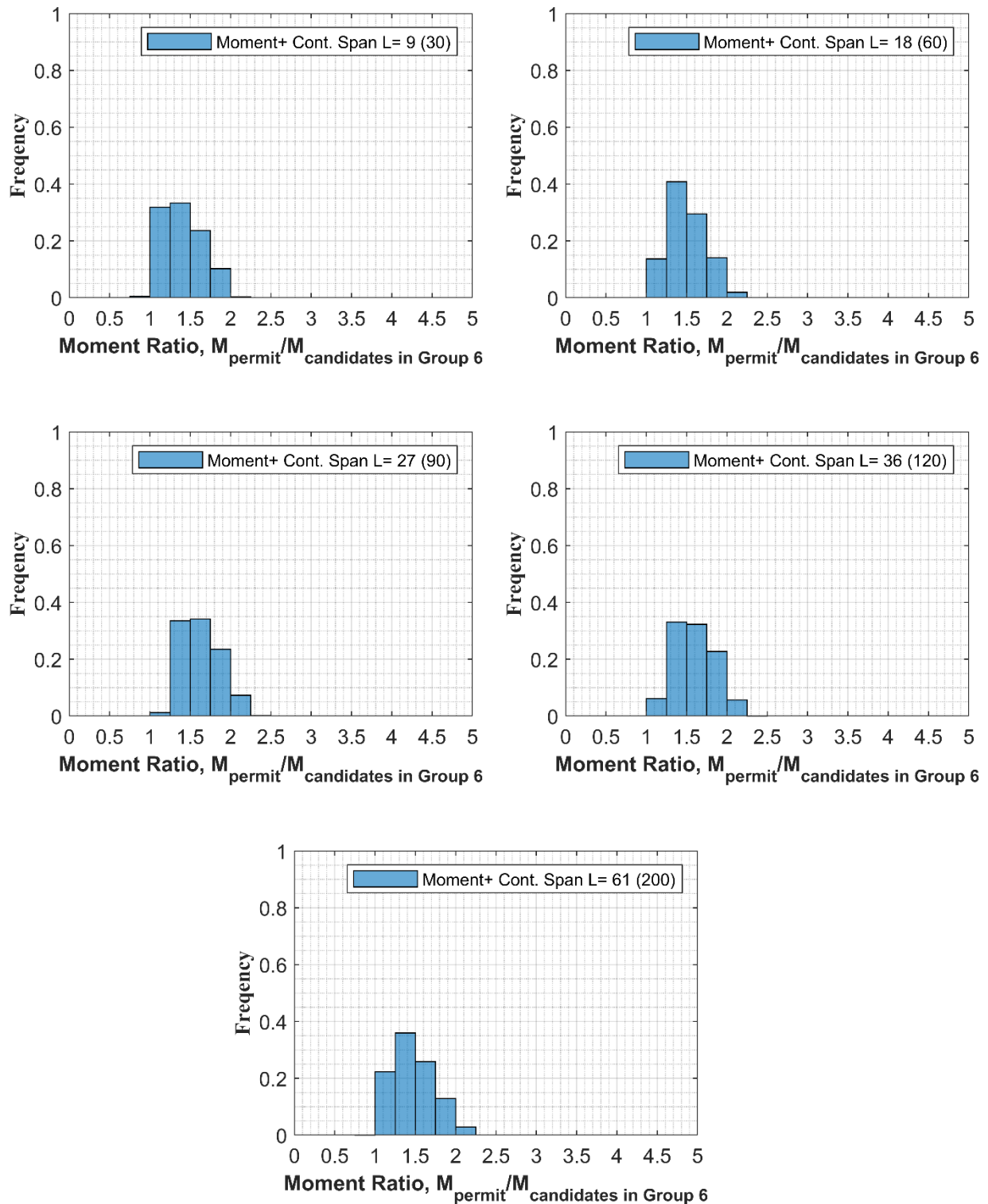


Figure 3-B.32. Load ratios of Candidates in Group 6 for various positive continuous span moments.

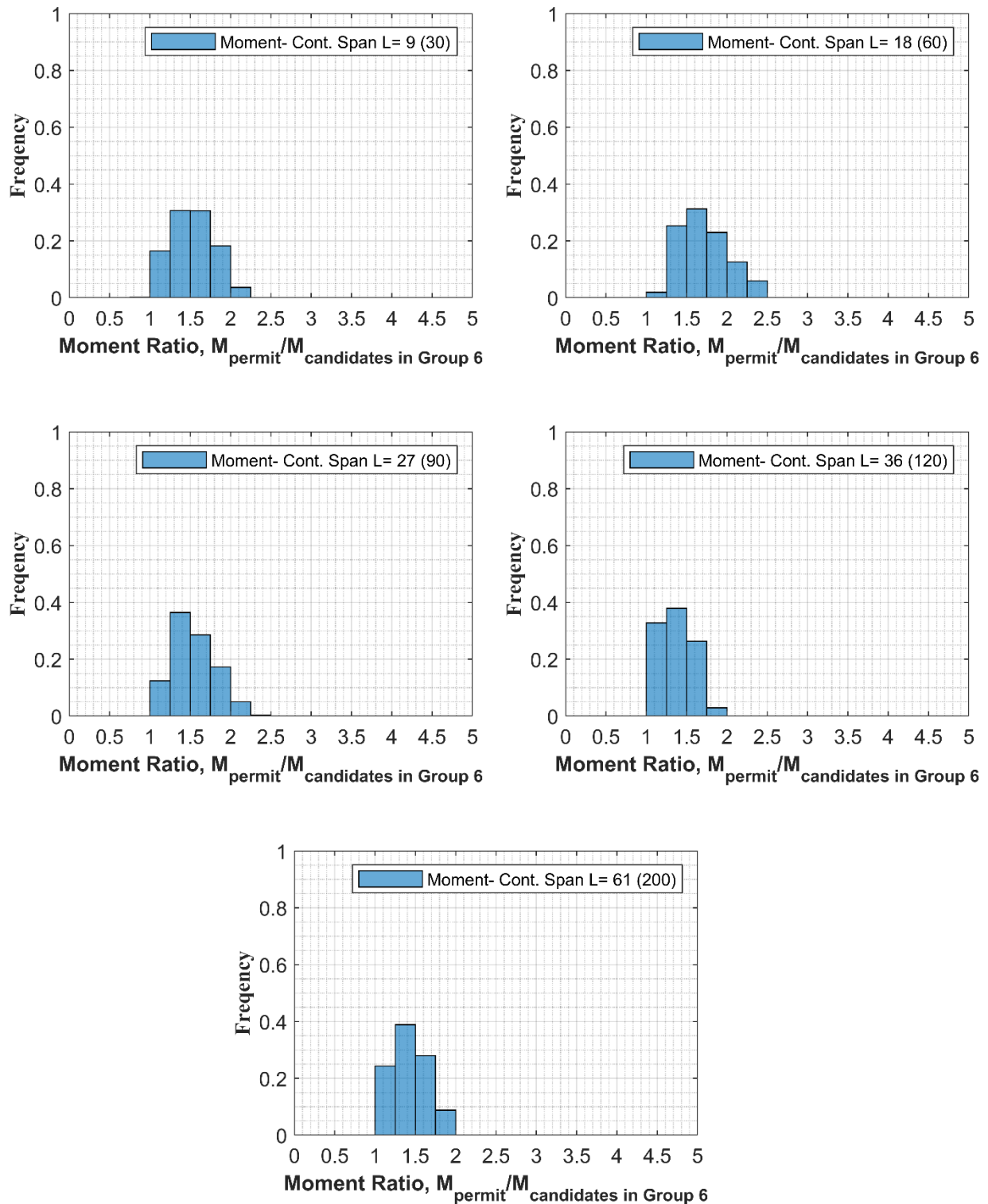


Figure 3-B.33. Load ratios of Candidates in Group 6 for various negative continuous span moments.

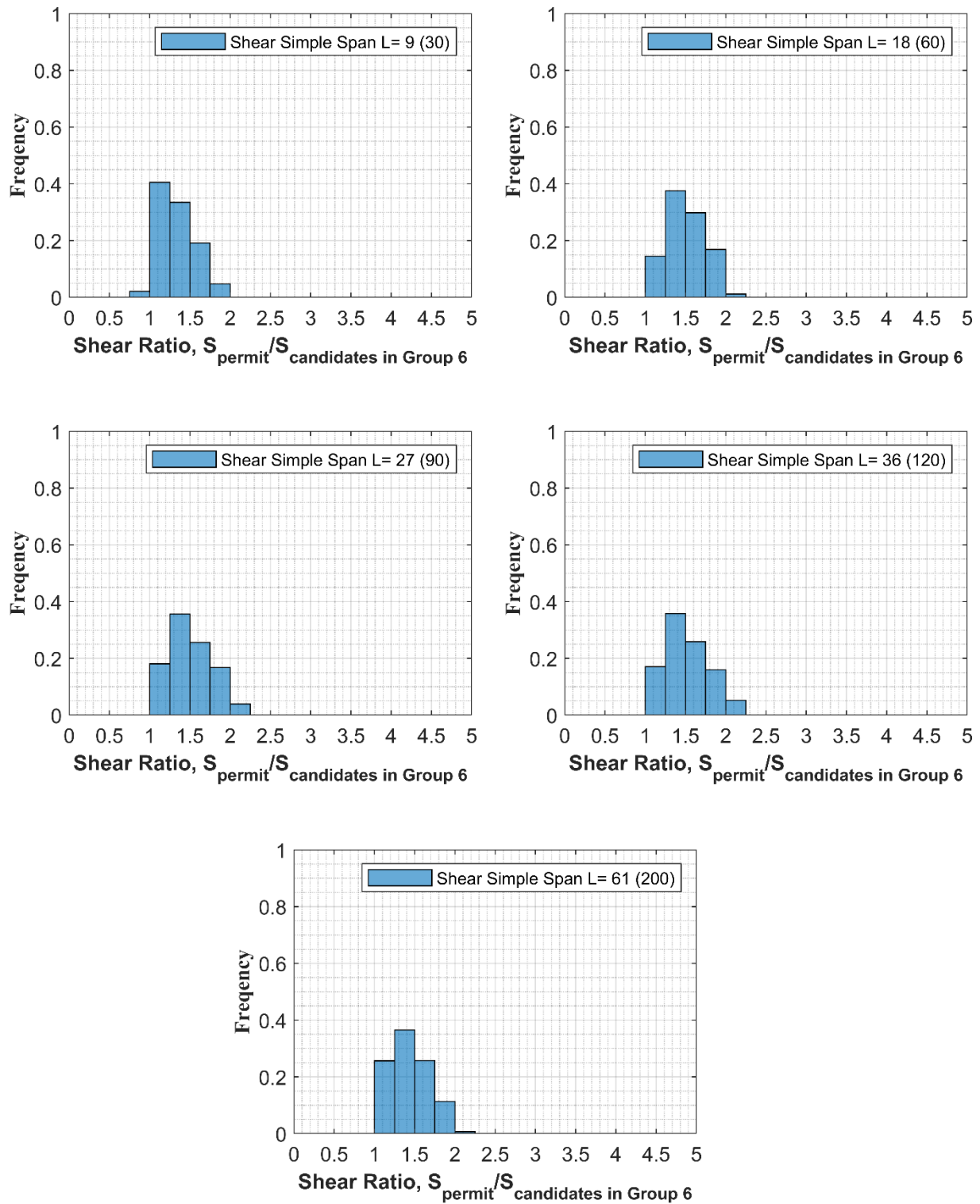


Figure 3-B.34. Load ratios of Candidates in Group 6 for various simple span shears.

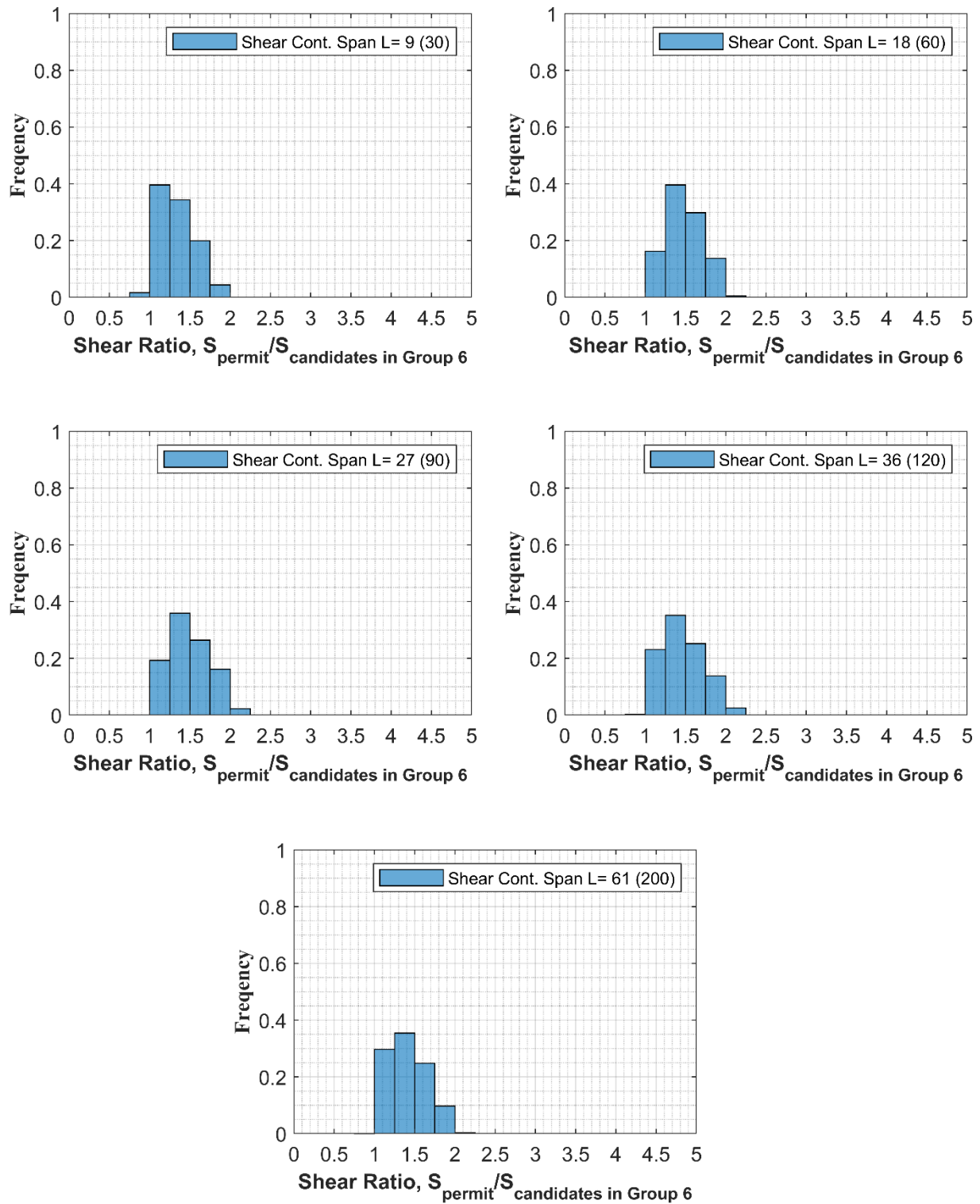


Figure 3-B.35. Load ratios of Candidates in Group 6 for various continuous span shears.

Load Ratios for Candidates in Group 7

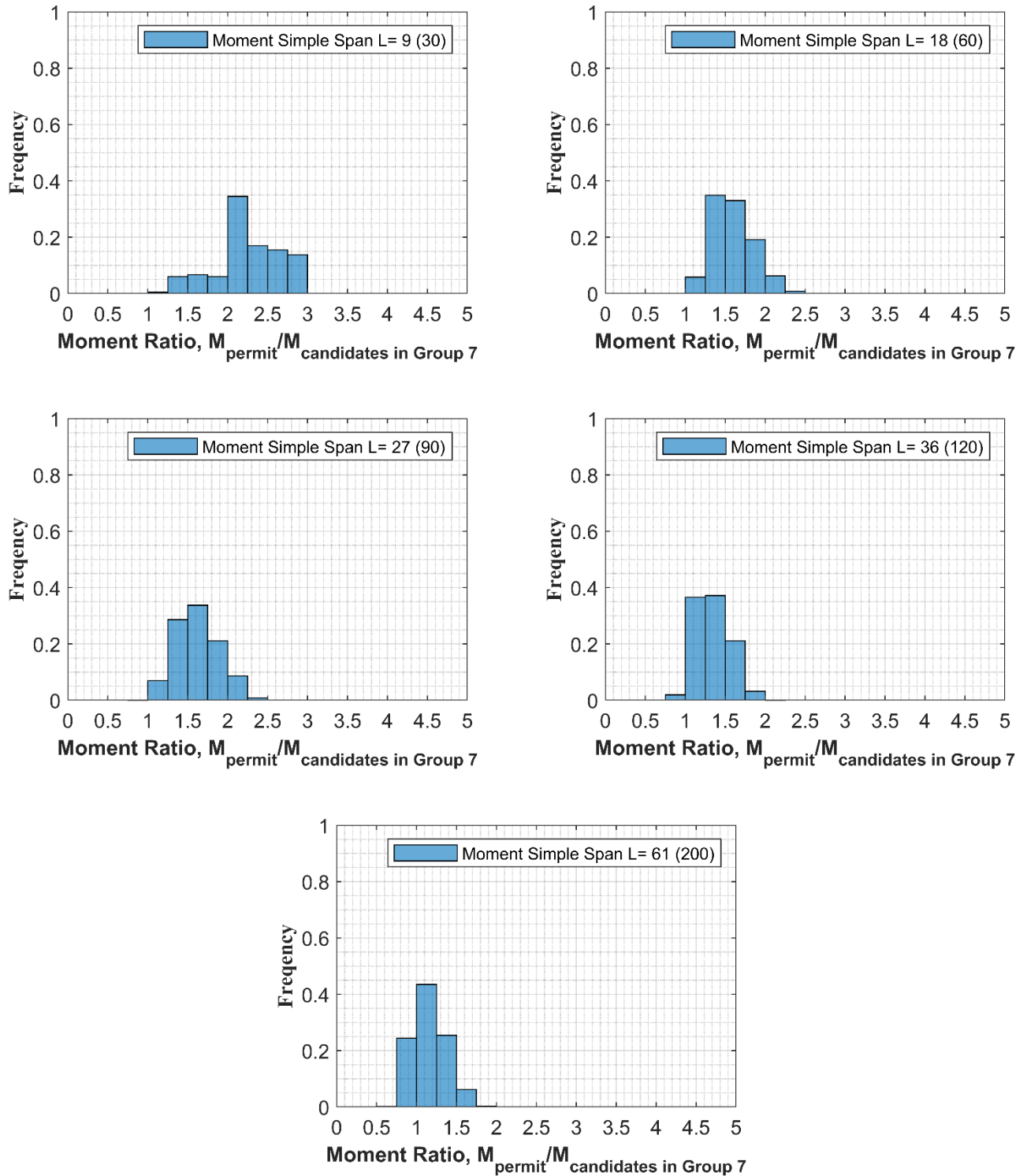


Figure 3-B. 36. Load ratios of Candidates in Group 7 for various simple span moments.

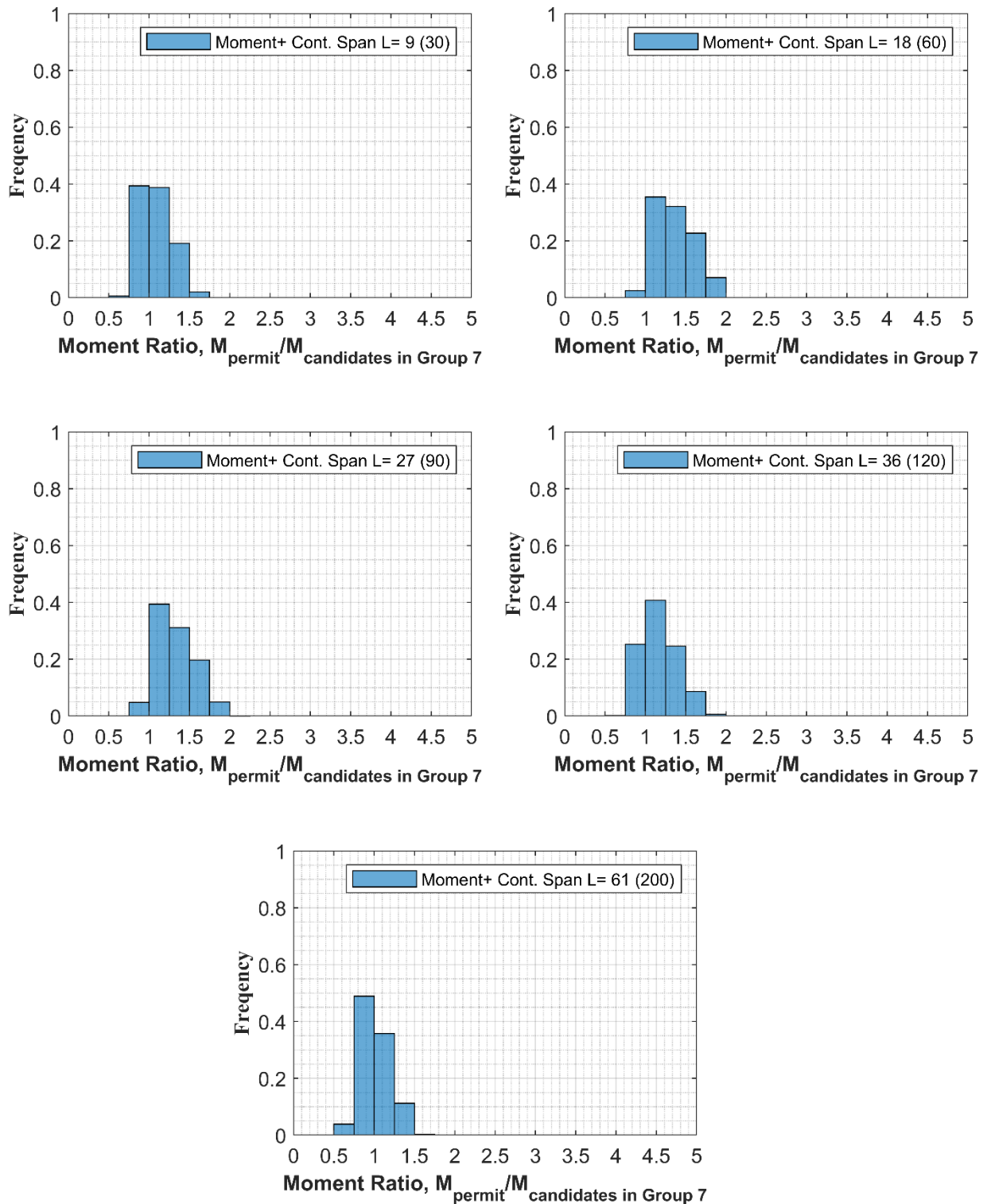


Figure 3-B. 37. Load ratios of Candidates in Group 7 for various positive continuous span moments.

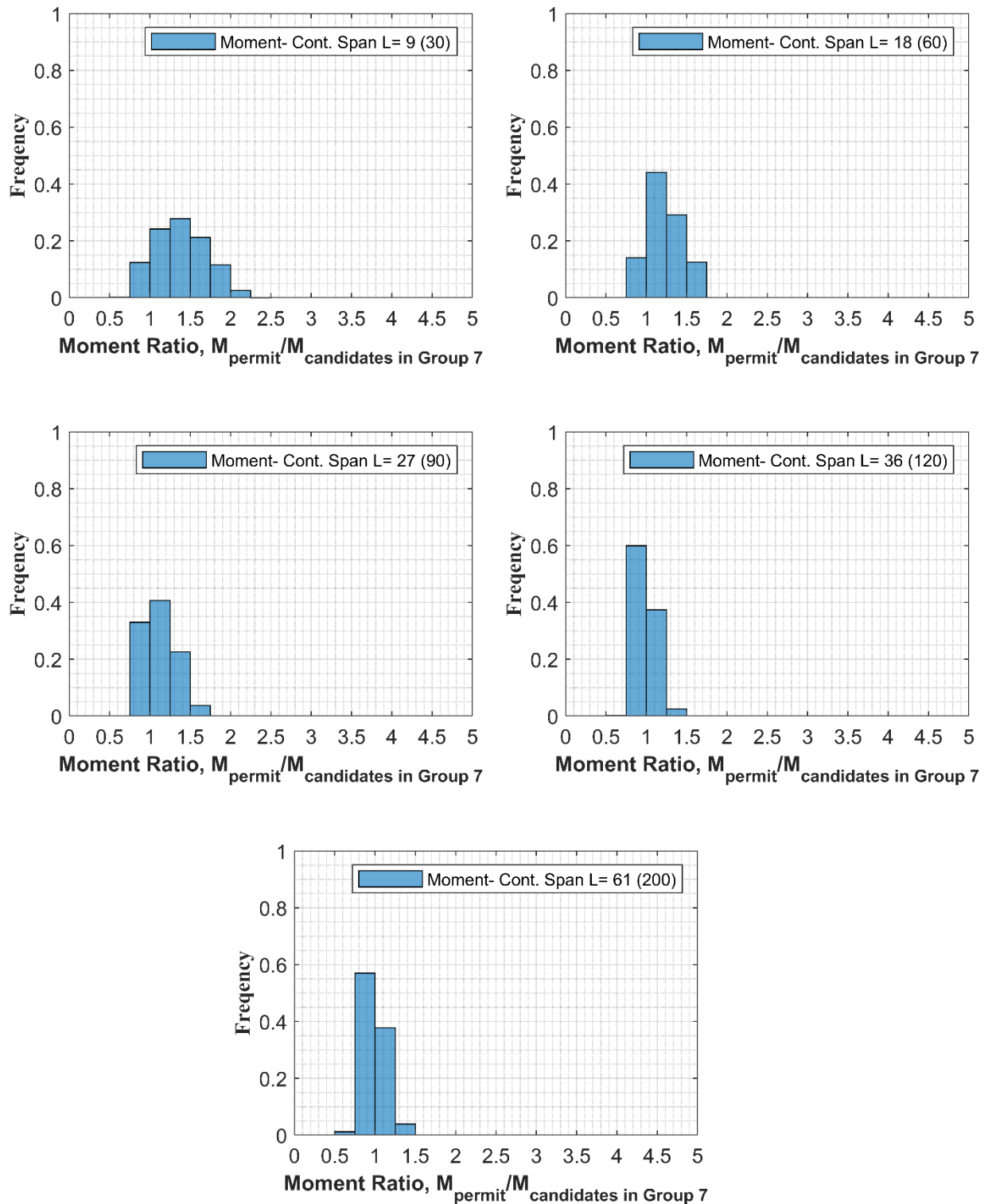


Figure 3-B.38. Load ratios of Candidates in Group 7 for various negative continuous span moments.

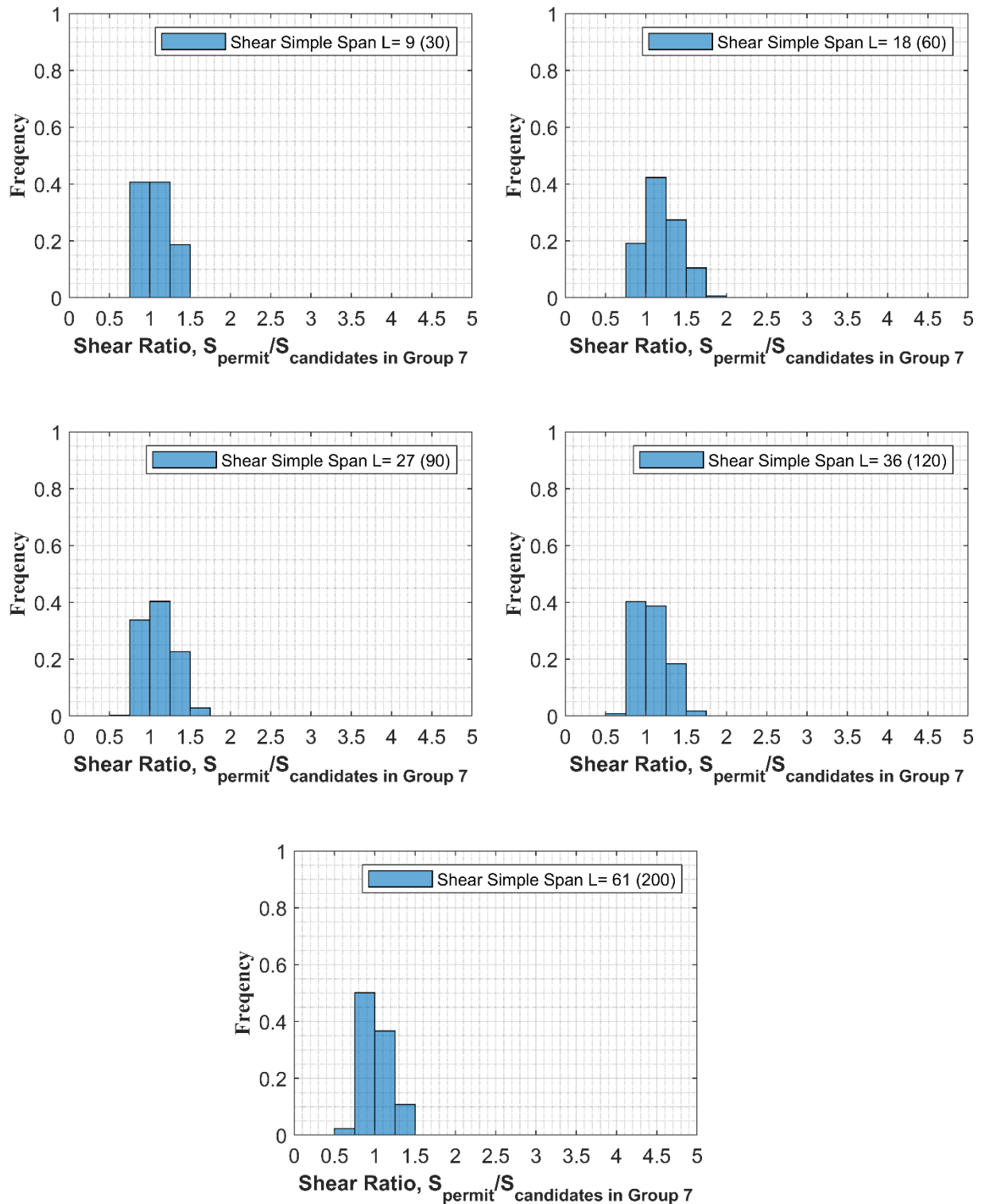


Figure 3-B.39. Load ratios of Candidates in Group 7 for various simple span shears.

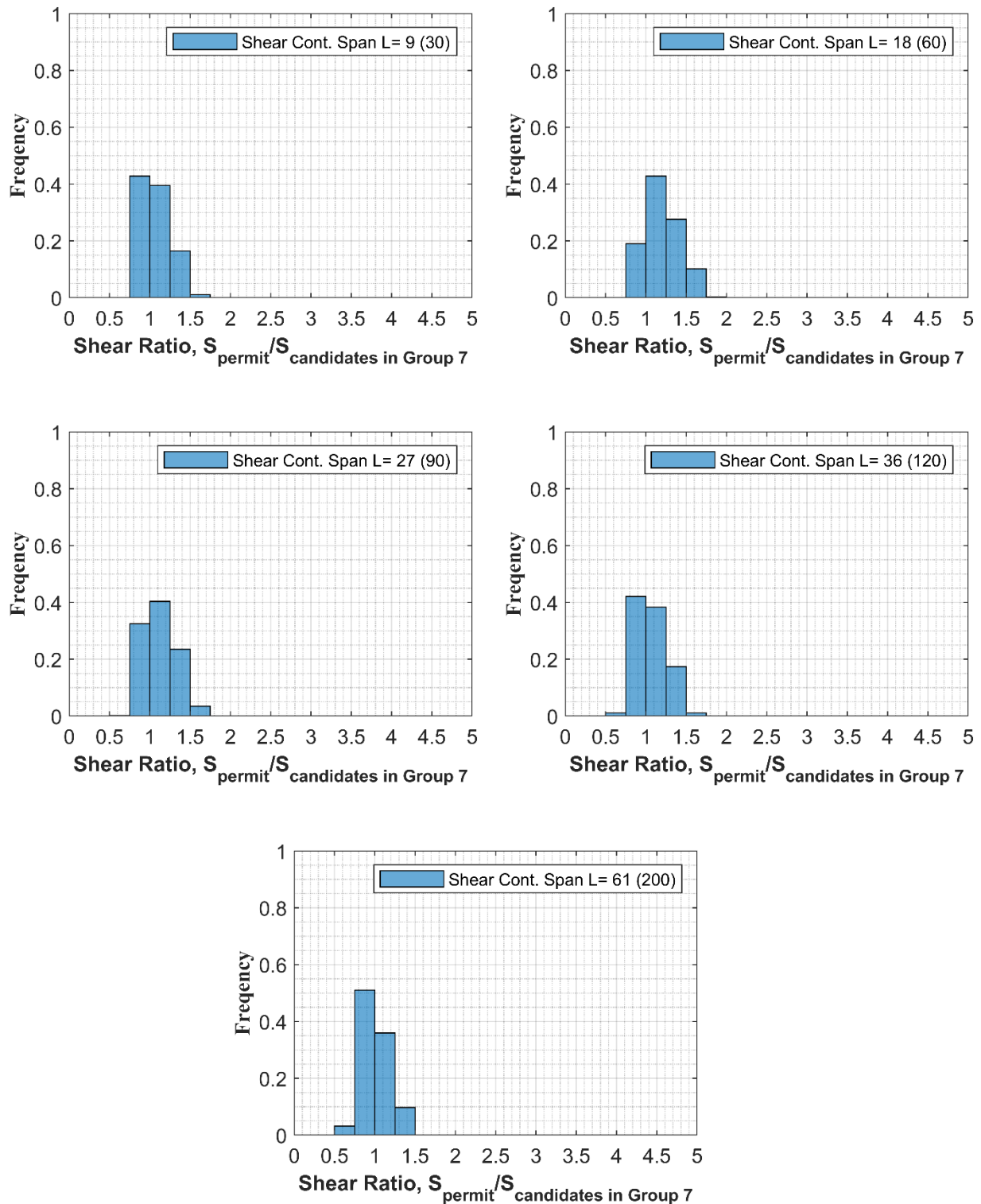


Figure 3-B.40. Load ratios of Candidates in Group 7 for various continuous span shears.

Load Ratios for Candidates in Group 8

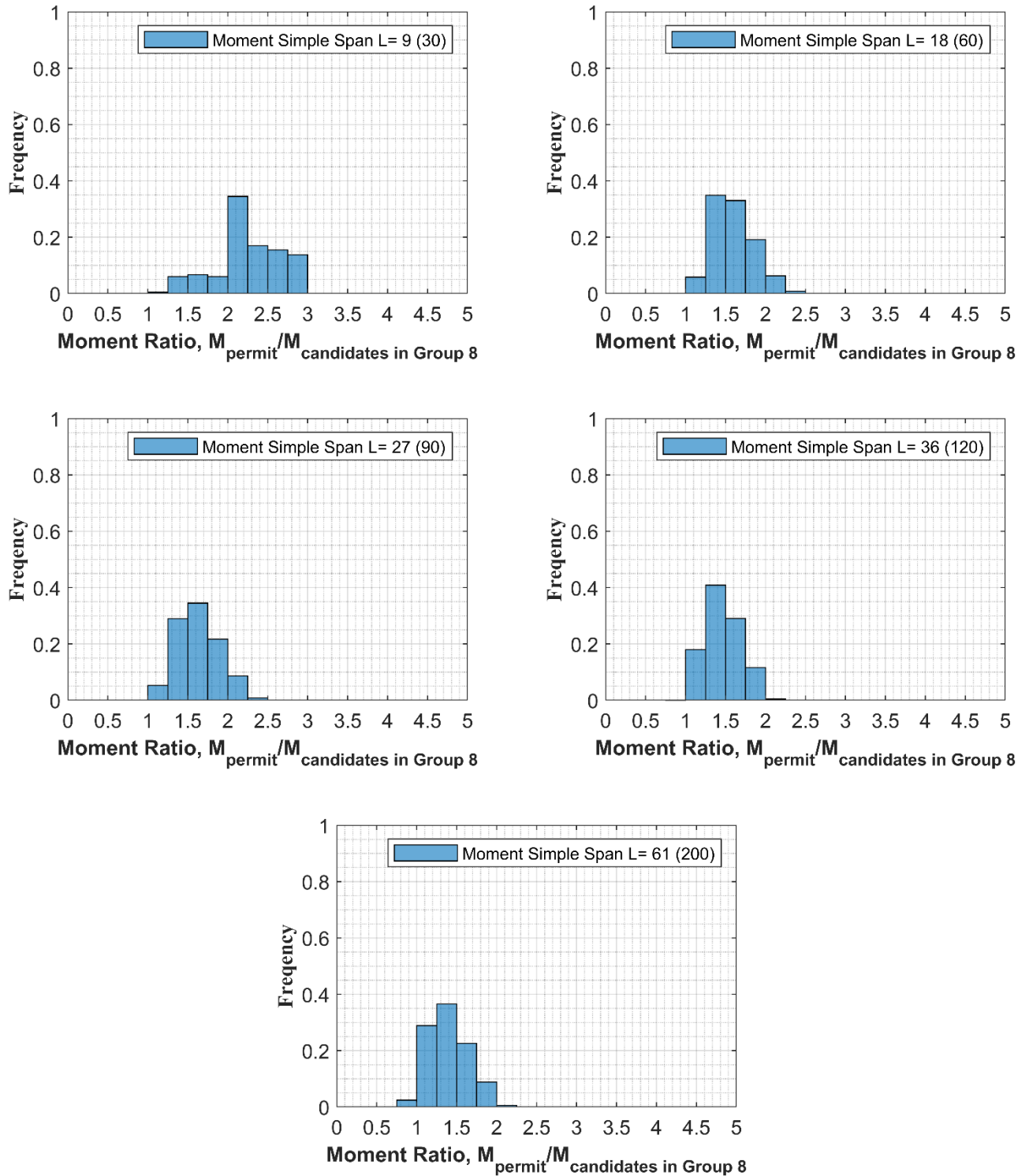


Figure 3-B.41. Load ratios of Candidates in Group 8 for various simple span moments.

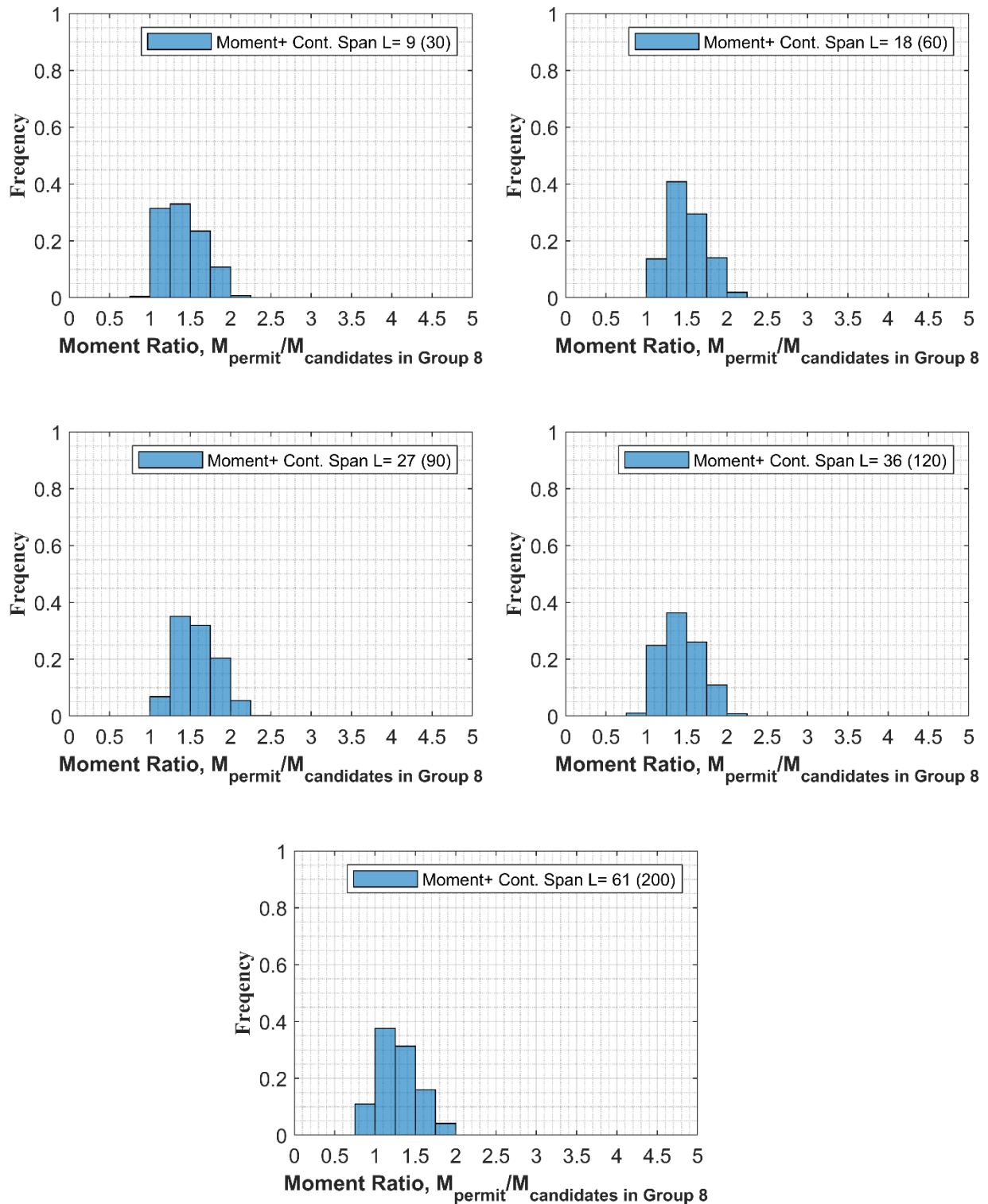


Figure 3-B.42. Load ratios of Candidates in Group 8 for various positive continuous span moments.

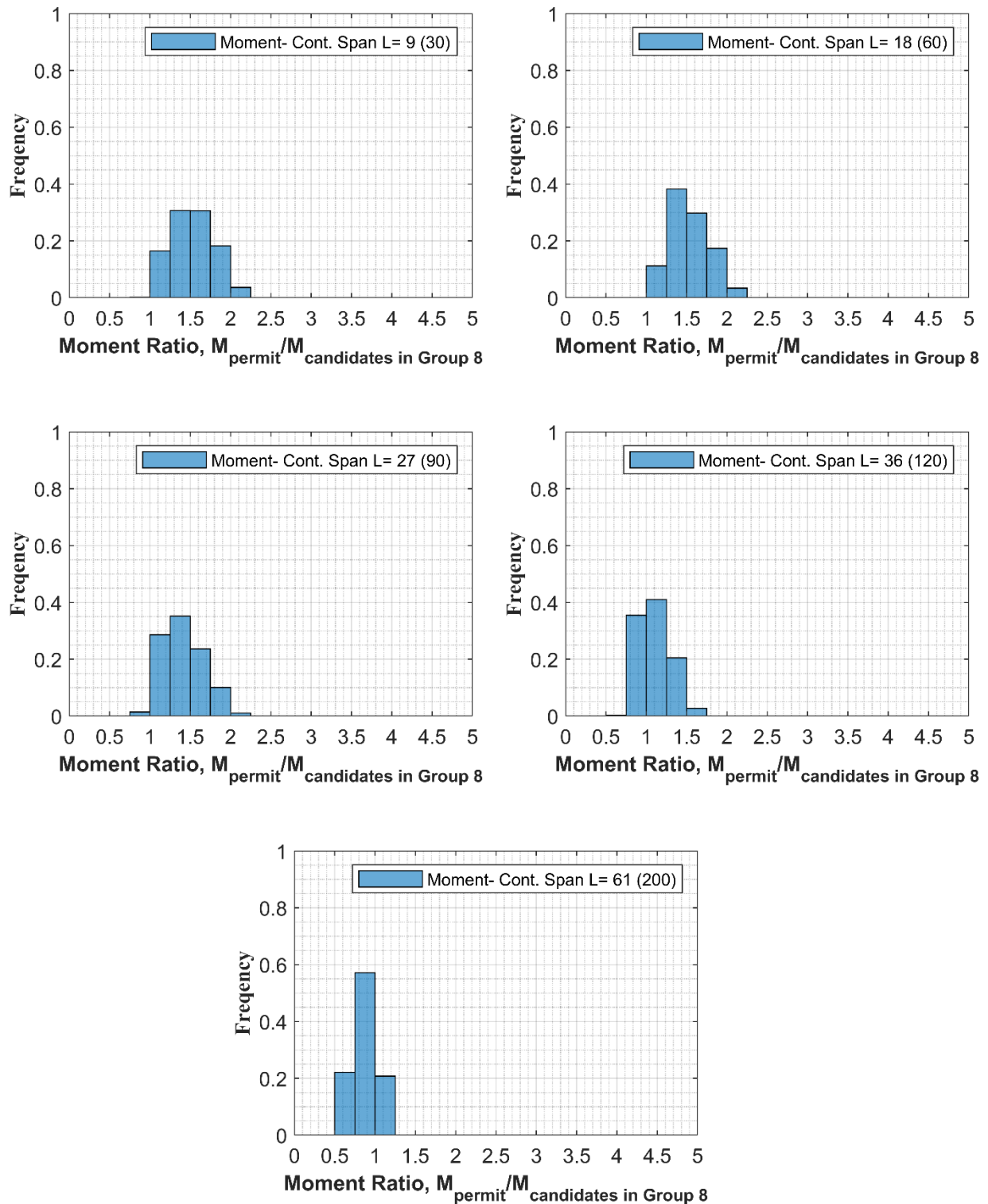


Figure 3-B.43. Load ratios of Candidates in Group 8 for various negative continuous span moments.

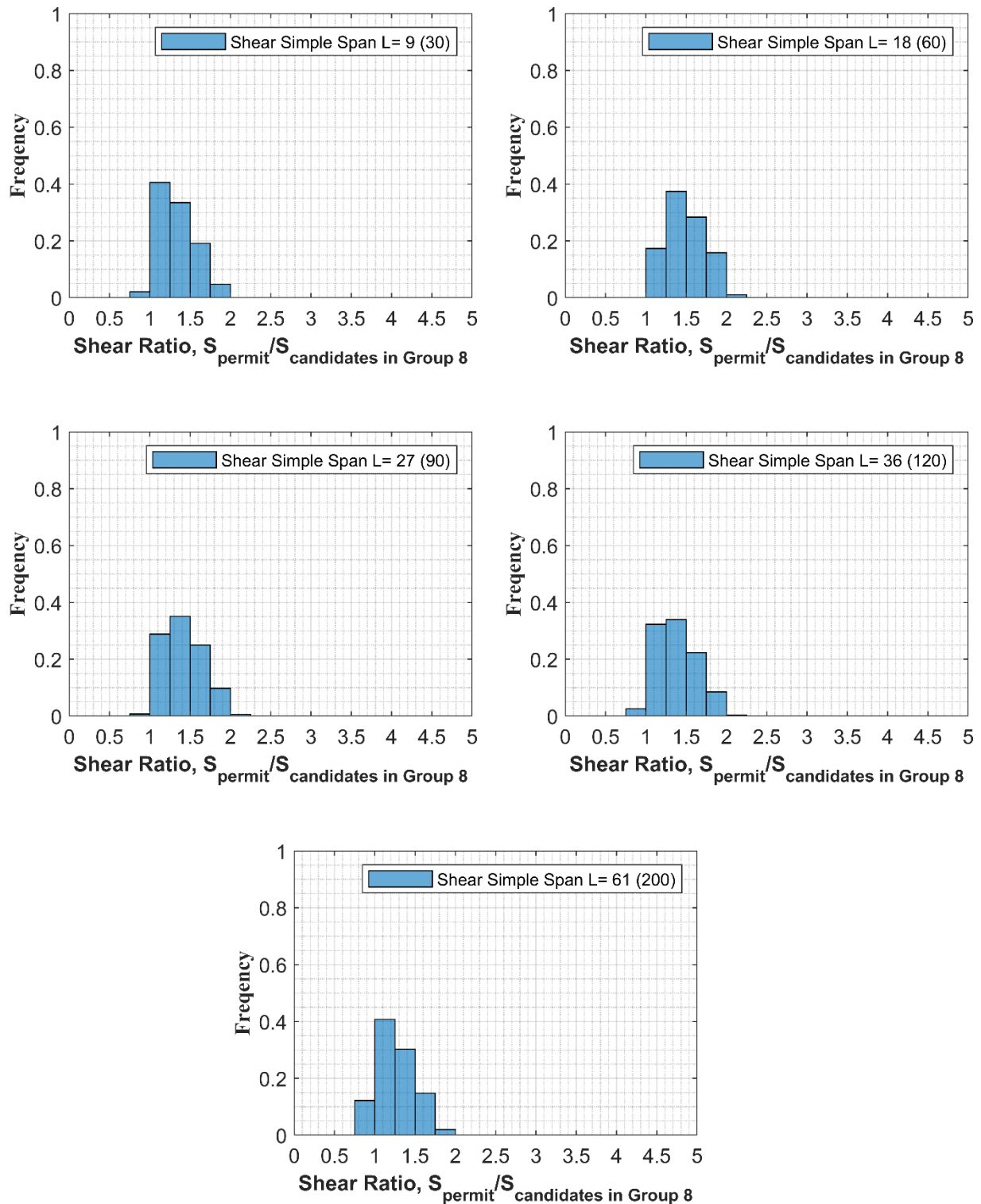


Figure 3-B.44. Load ratios of Candidates in Group 8 for various simple span shears.

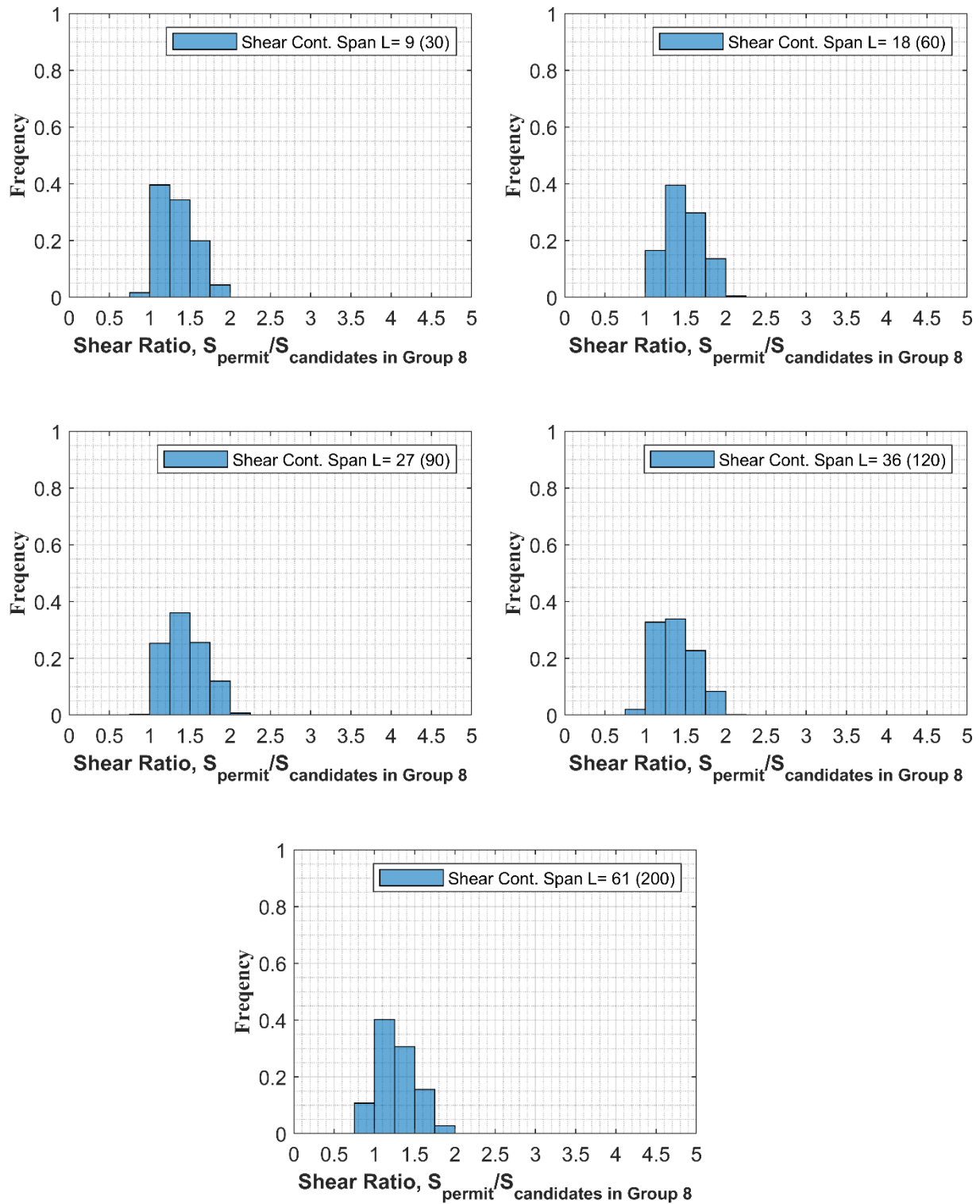


Figure 3-B.45. Load ratios of Candidates in Group 8 for various continuous span shears.

Load Ratios for Candidates in Group 9

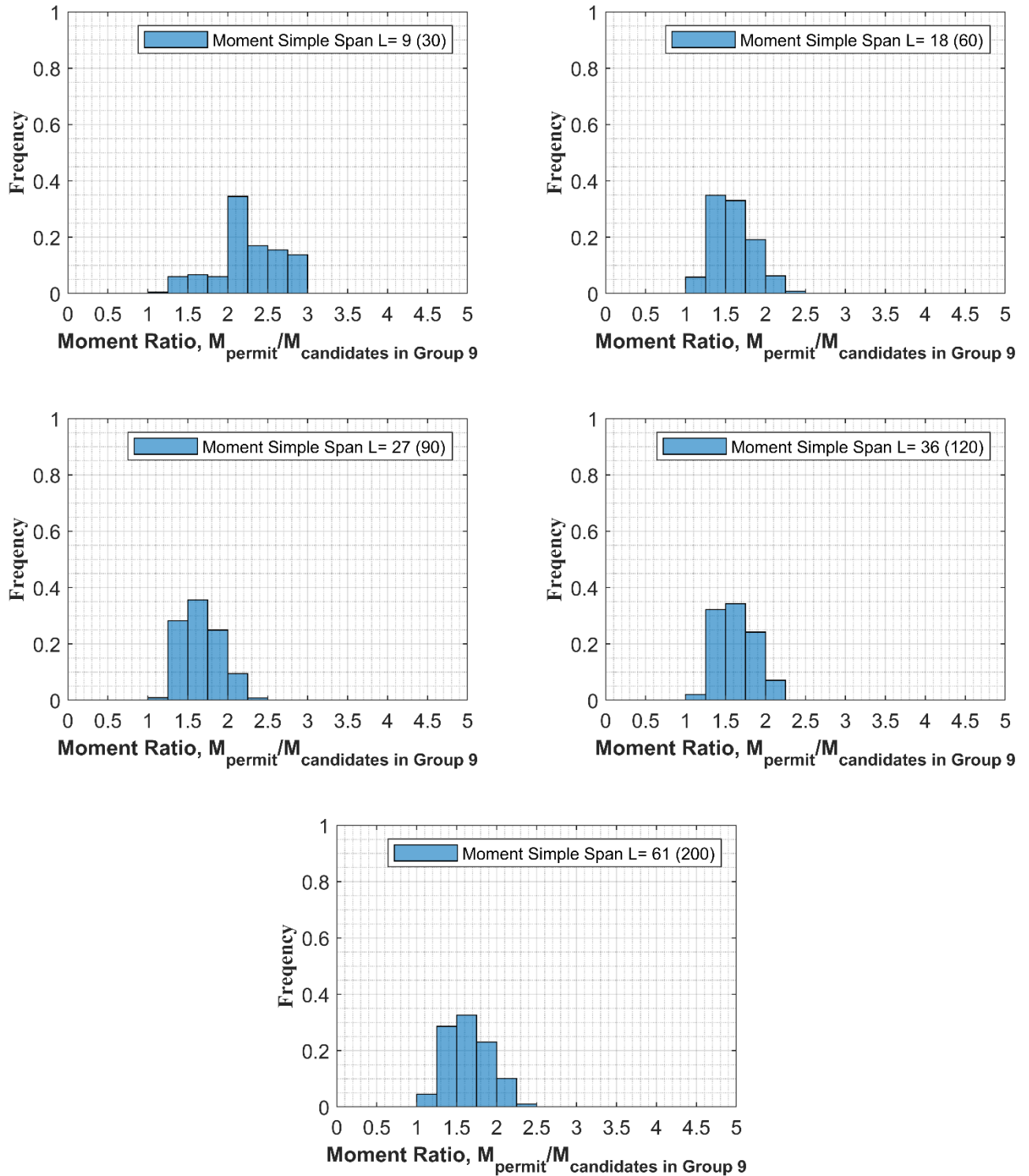


Figure 3-B.46. Load ratios of Candidates in Group 9 for various simple span moments.

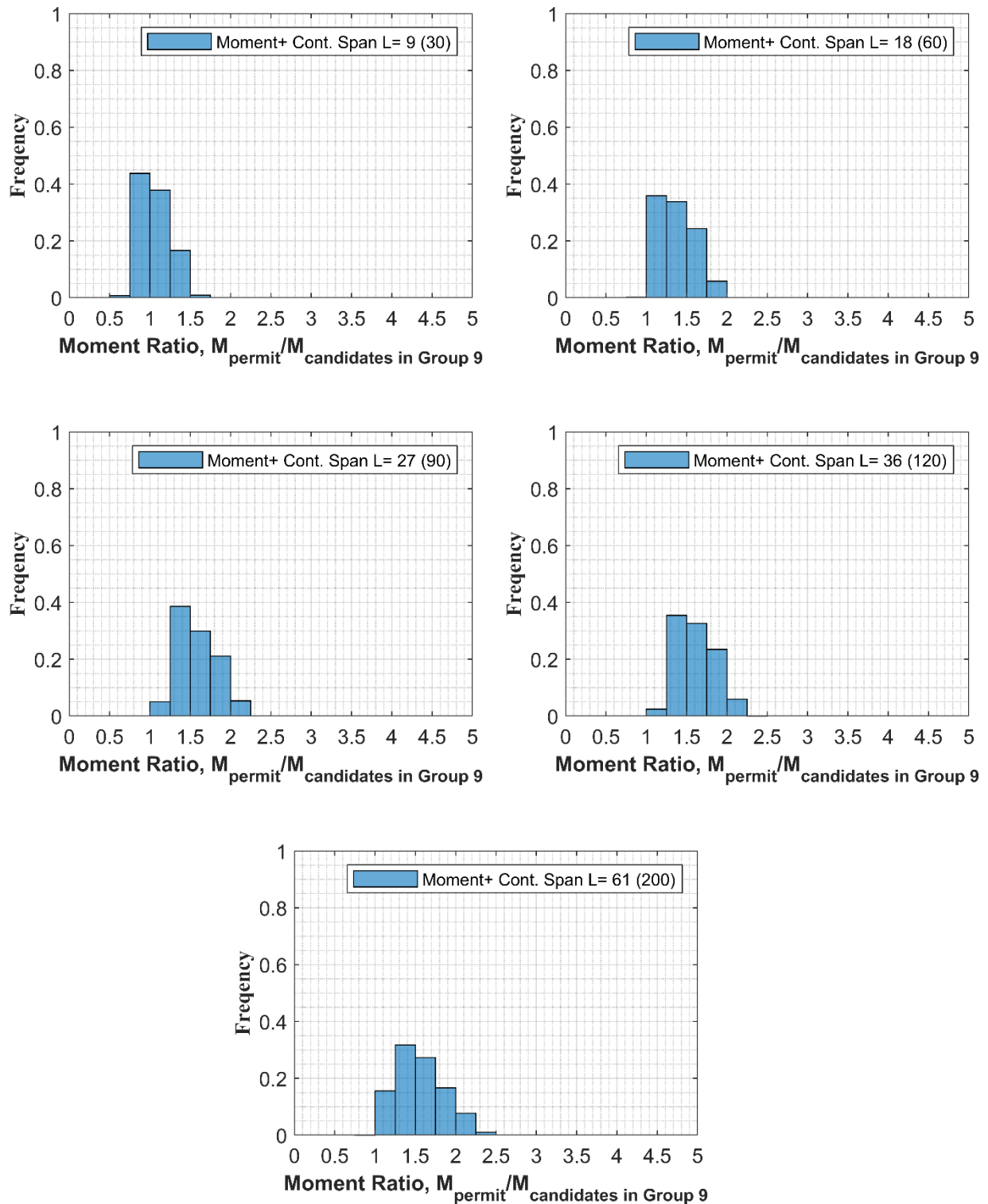


Figure 3-B.47. Load ratios of Candidates in Group 9 for various positive continuous span moments.

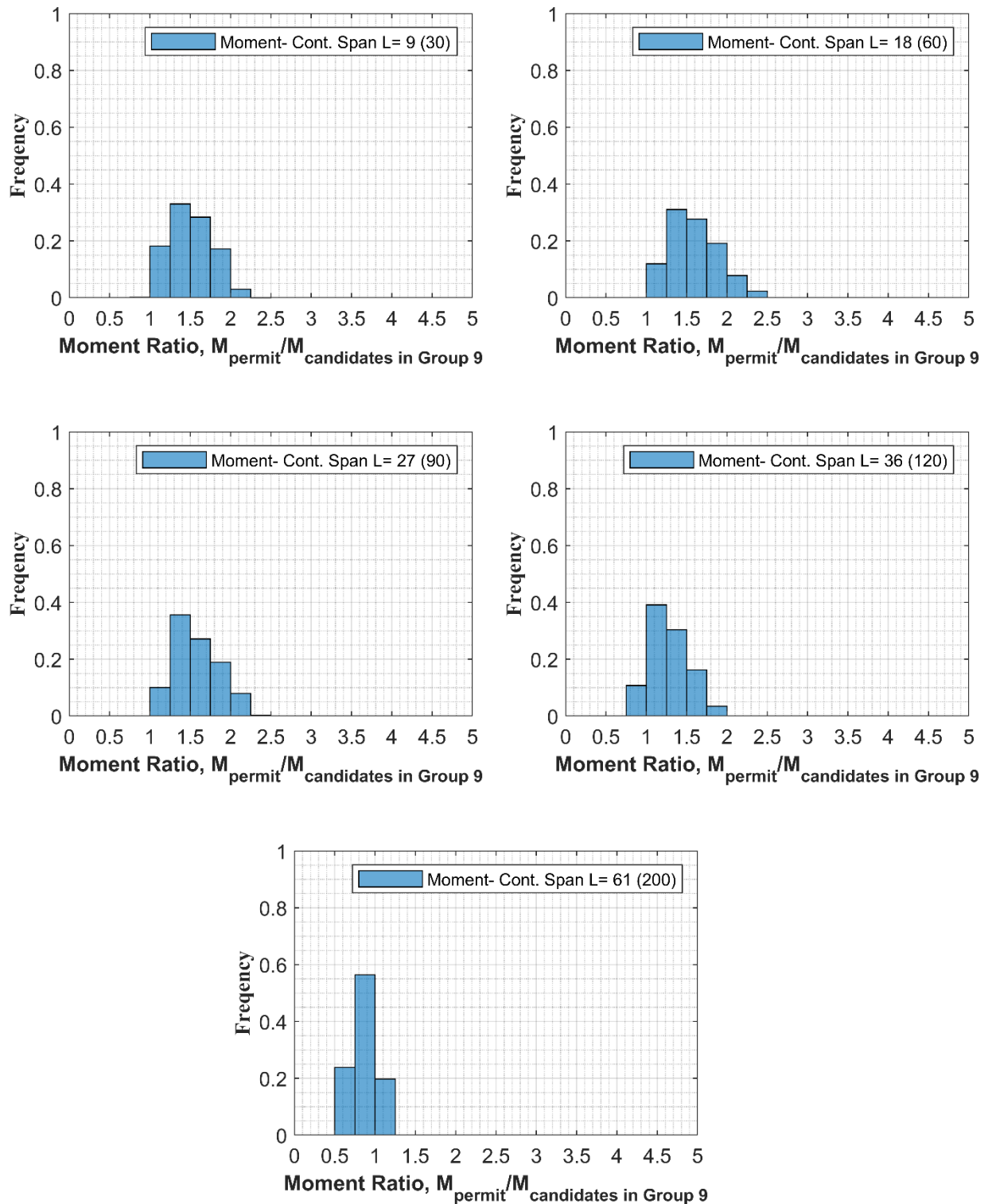


Figure 3-B.48. Load ratios of Candidates in Group 9 for various negative continuous span moments.

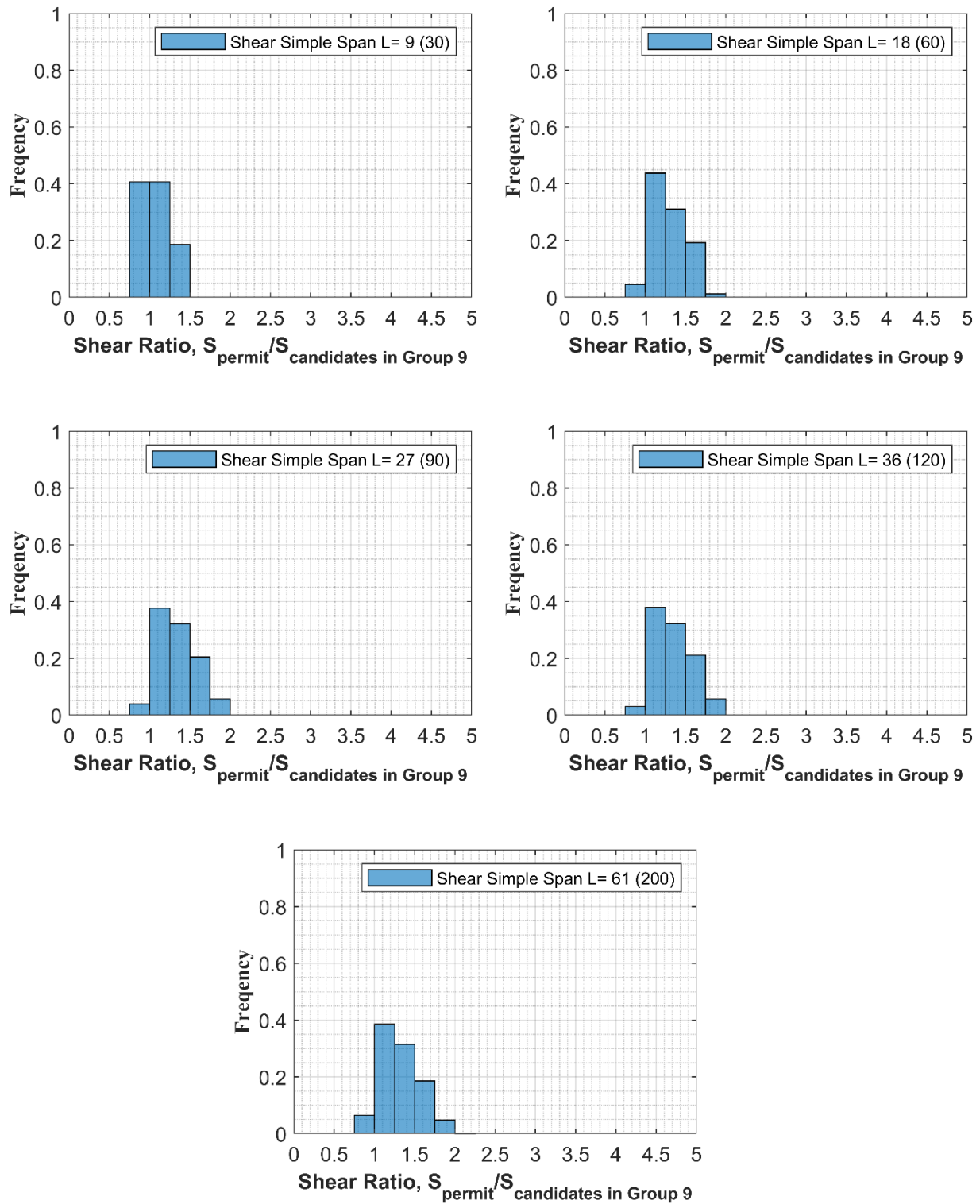


Figure 3-B.49. Load ratios of Candidates in Group 9 for various simple span shears.

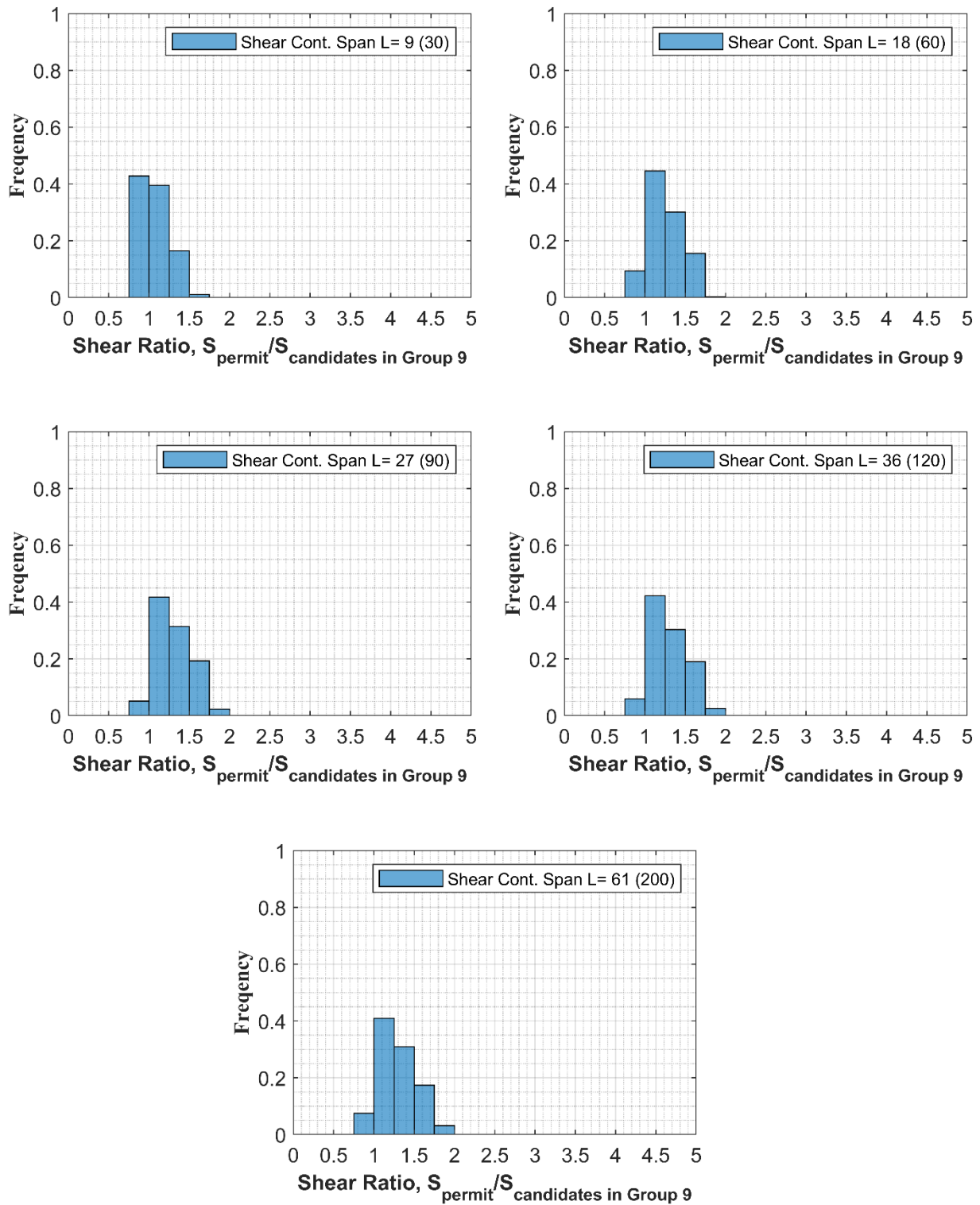
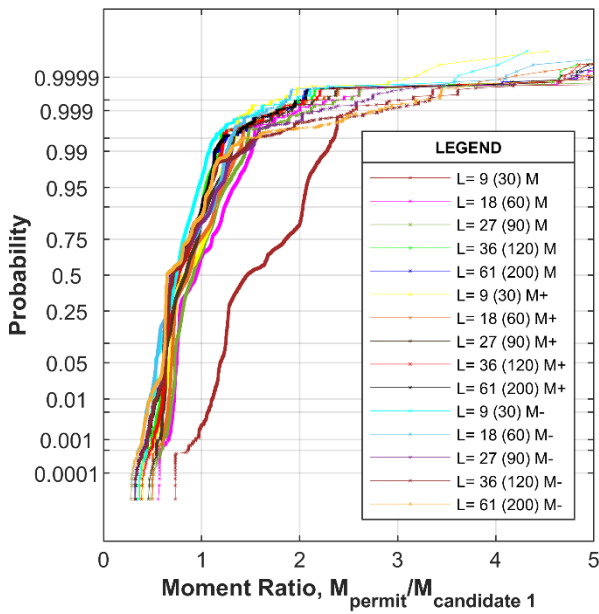
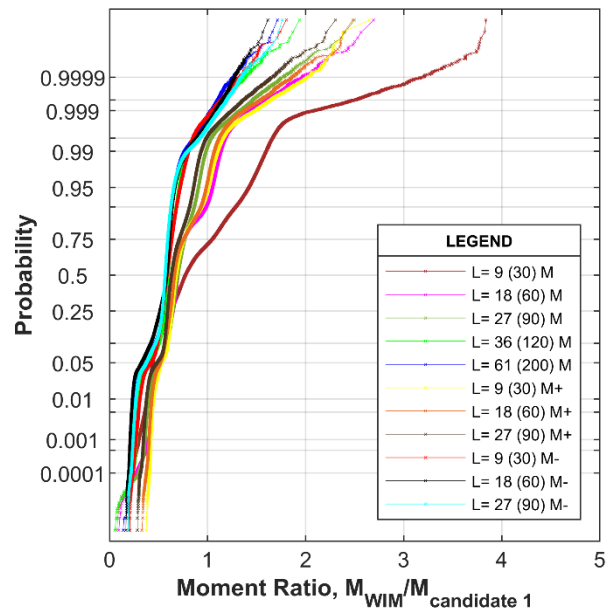


Figure 3-B.50. Load ratios of Candidates in Group 9 for various continuous span shears.

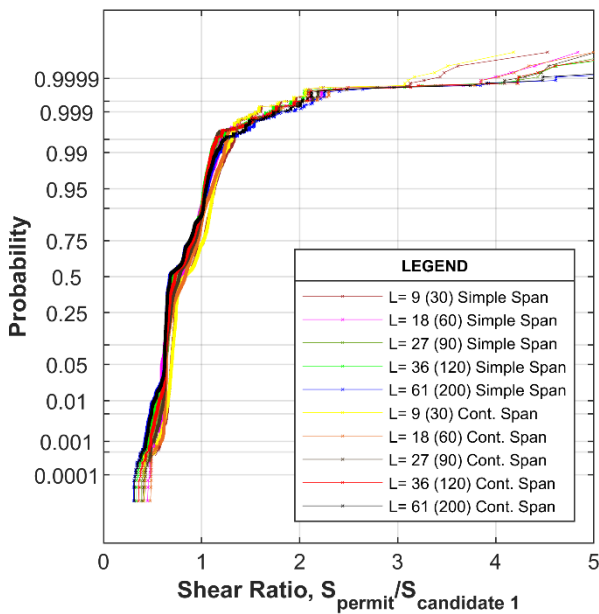
Appendix 3-C. Load effects ratios for notional permit vehicle candidates



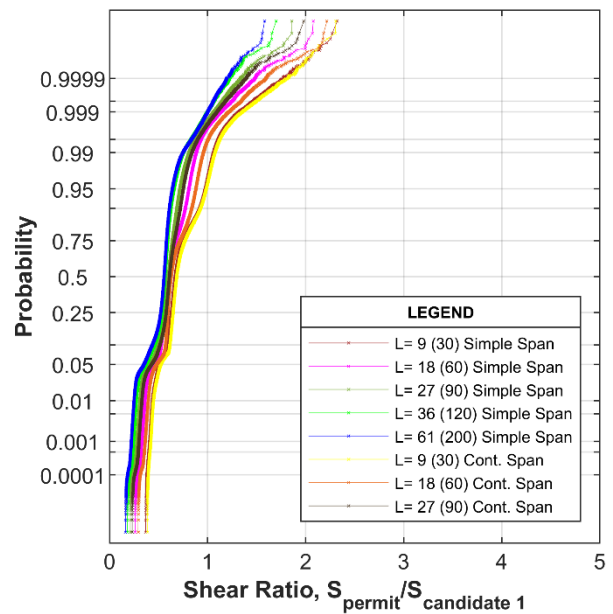
(a) Moment, Multi Trip Permits



(b) Moment, Overloaded, WIM

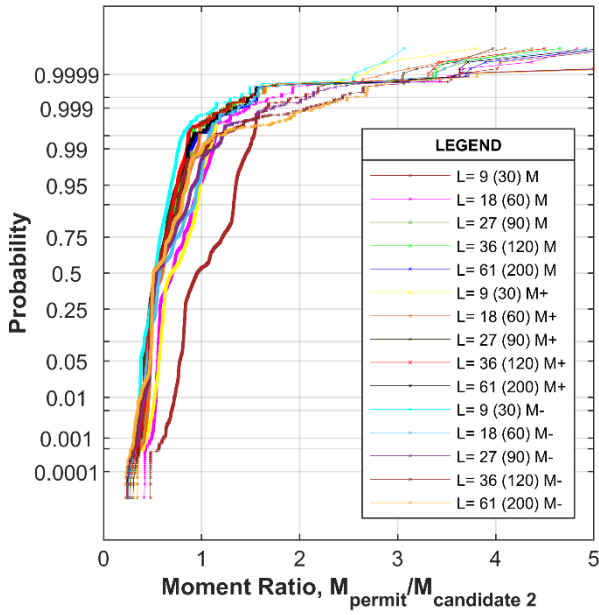


(c) Shear, Multi Trip Permits

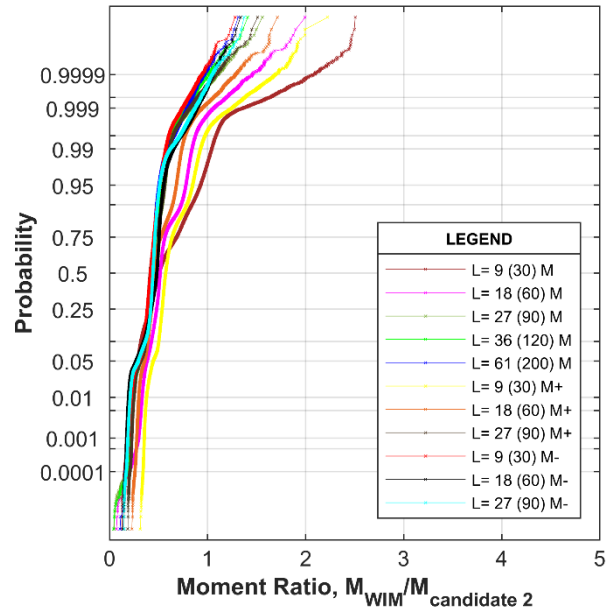


(d) Moment, Overloaded, WIM

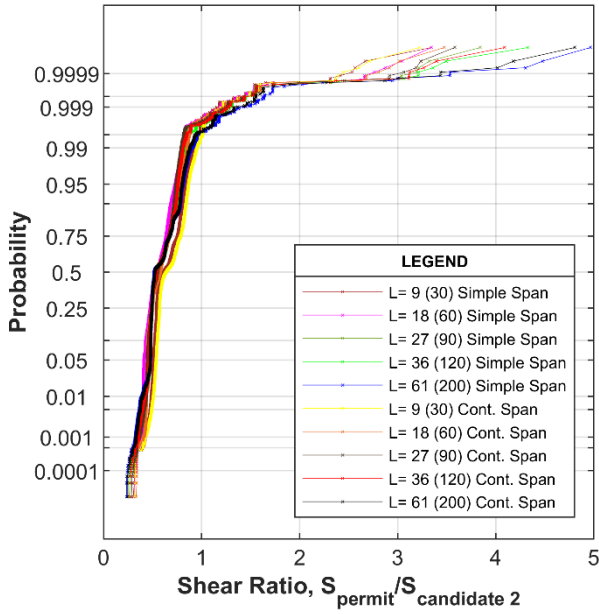
Figure 3-C.1. Load effects of Candidates 1 for (a) moment for multi trip permit data (b) moment for overload WIM data, (c) shear for multi trip permit data (d) shear for overload WIM data.



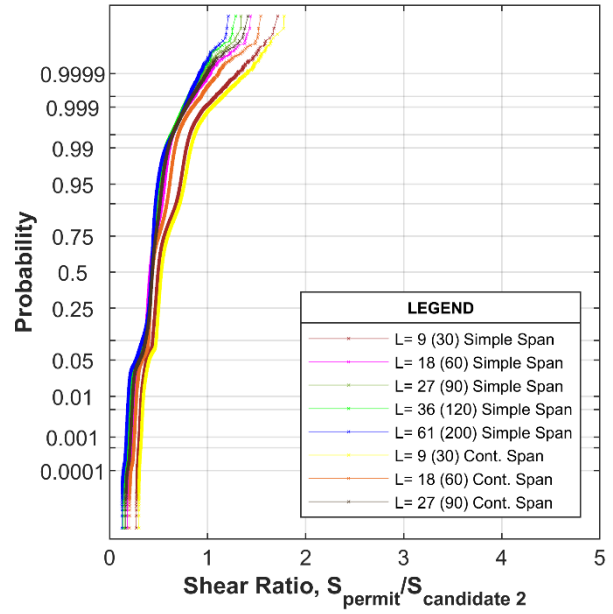
(a) Moment, Multi Trip Permits



(b) Moment, Overloaded, WIM

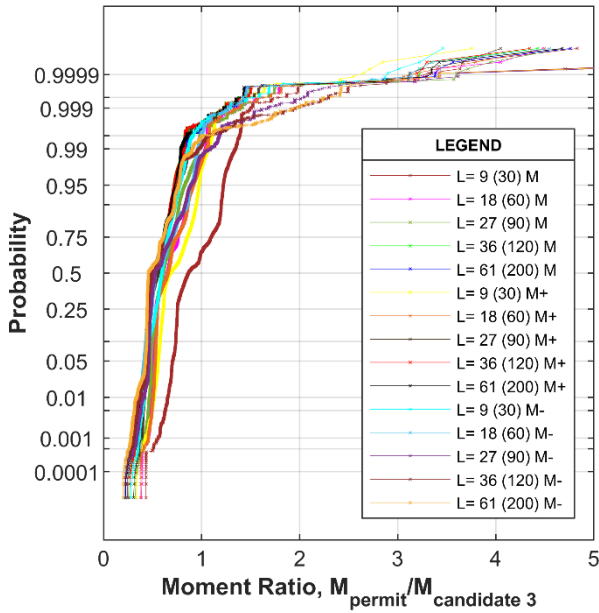


(c) Shear, Multi Trip Permits

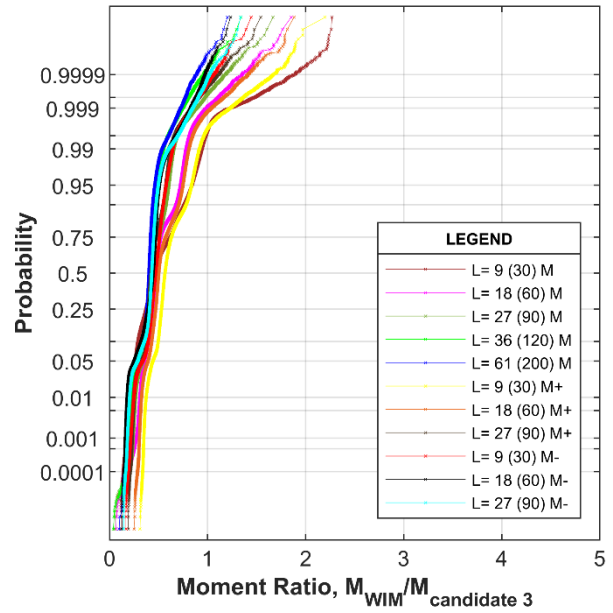


(d) Moment, Overloaded, WIM

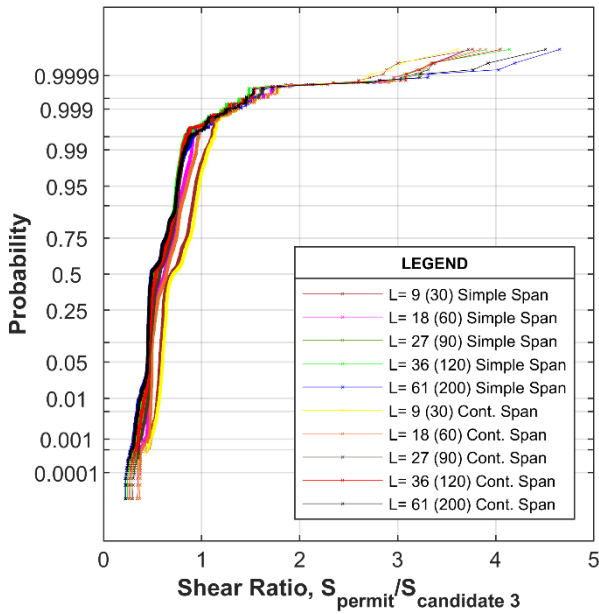
Figure 3-C.2. Load effects of Candidates 2 for (a) moment for multi trip permit data (b) moment for overload WIM data, (c) shear for multi trip permit data (d) shear for overload WIM data.



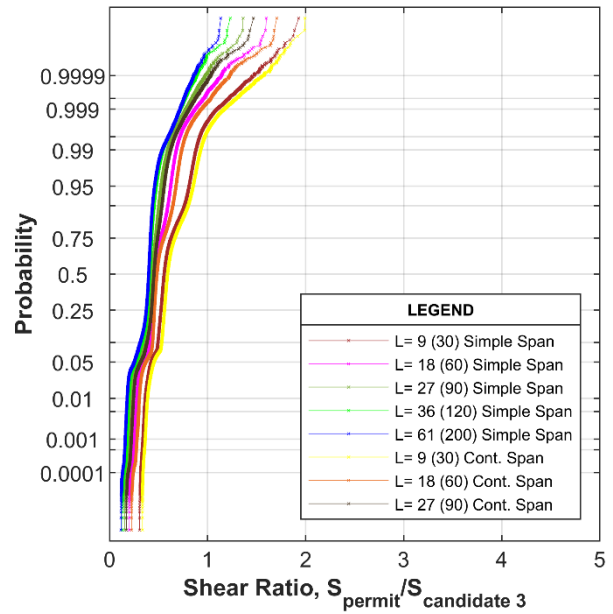
(a) Moment, Multi Trip Permits



(b) Moment, Overloaded, WIM

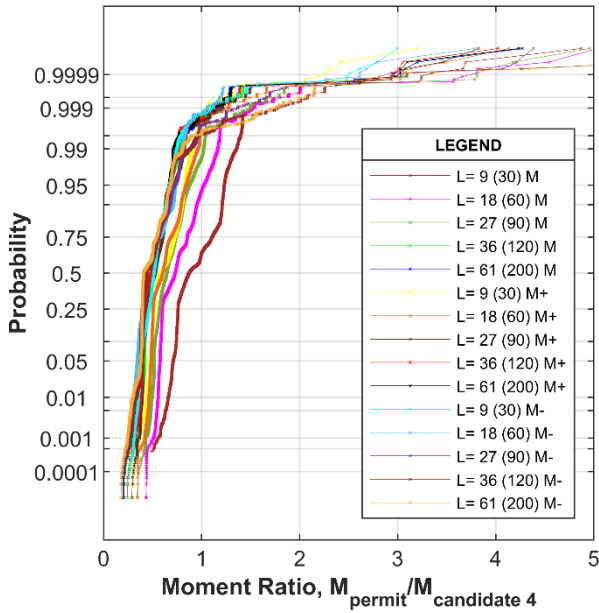


(c) Shear, Multi Trip Permits

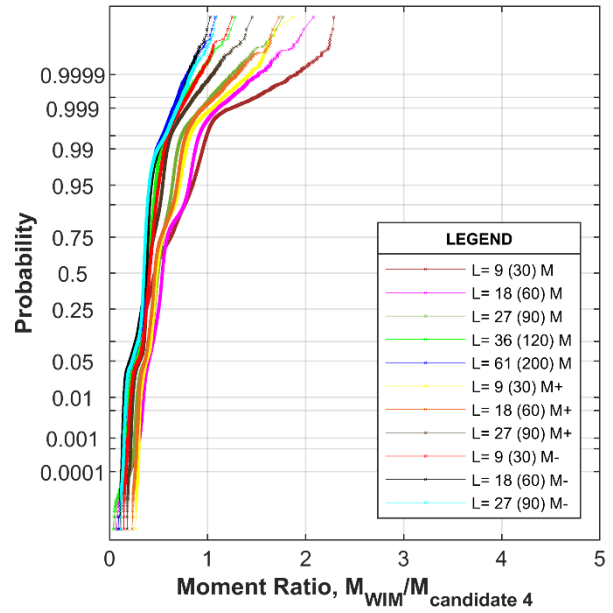


(d) Moment, Overloaded, WIM

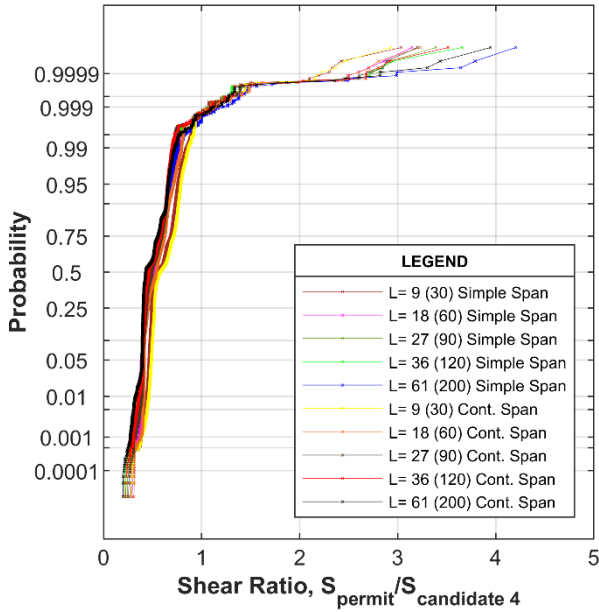
Figure 3-C.3. Load effects of Candidates 3 for (a) moment for multi trip permit data (b) moment for overload WIM data, (c) shear for multi trip permit data (d) shear for overload WIM data.



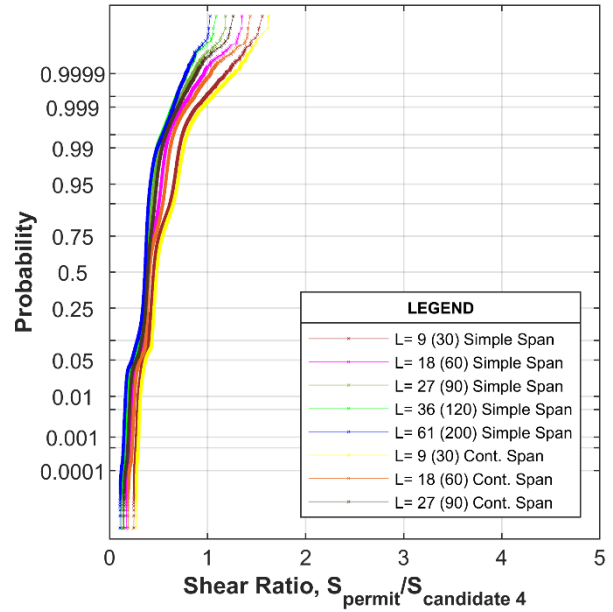
(a) Moment, Multi Trip Permits



(b) Moment, Overloaded, WIM

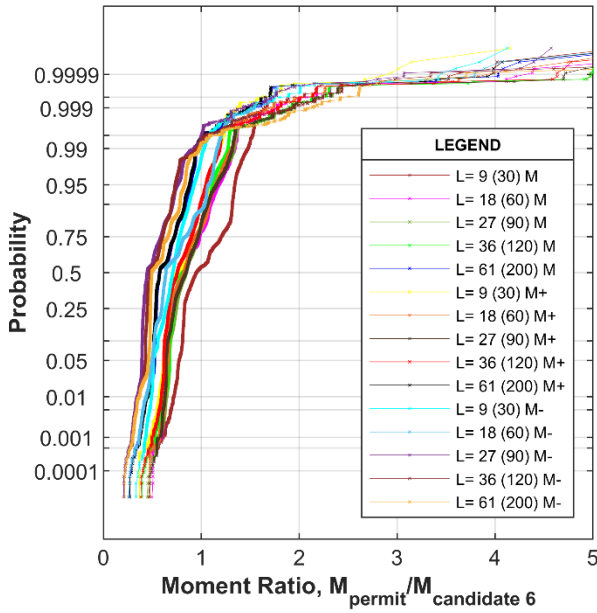


(c) Shear, Multi Trip Permits

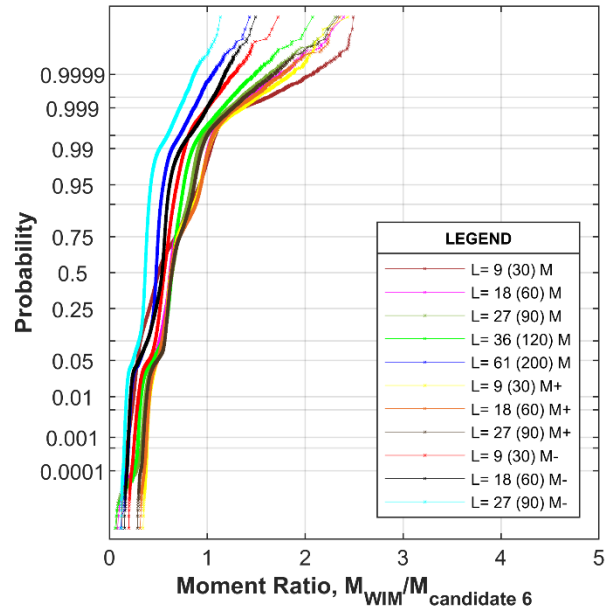


(d) Moment, Overloaded, WIM

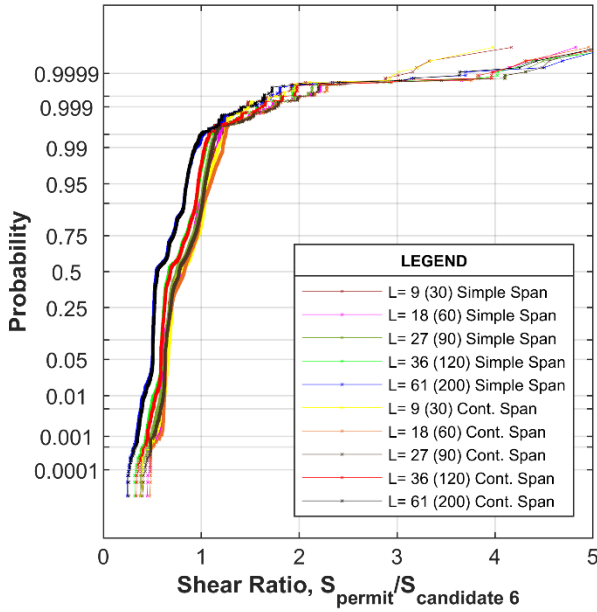
Figure 3-C.4. Load effects of Candidates 4 for (a) moment for multi trip permit data (b) moment for overload WIM data, (c) shear for multi trip permit data (d) shear for overload WIM data.



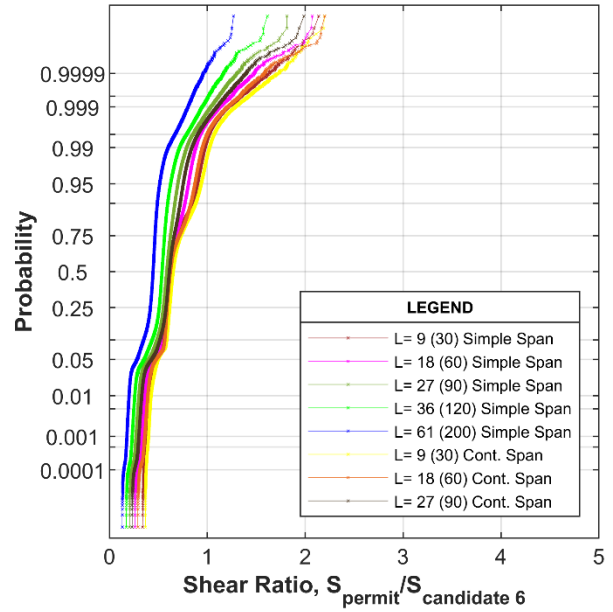
(a) Moment, Multi Trip Permits



(b) Moment, Overloaded, WIM

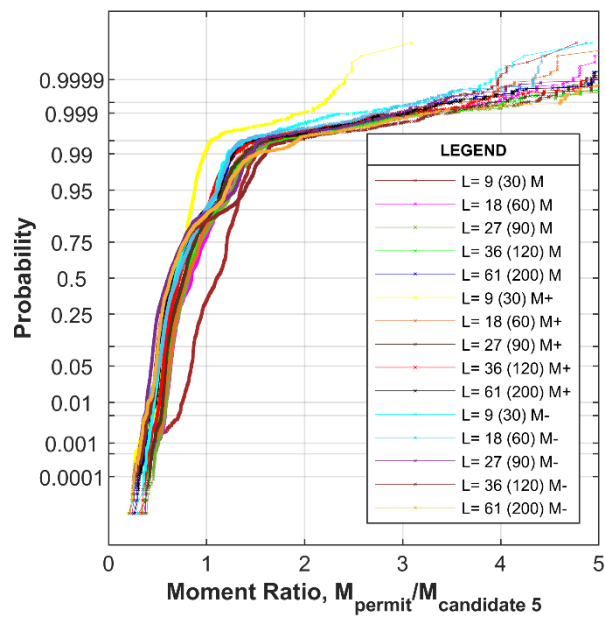


(c) Shear, Multi Trip Permits

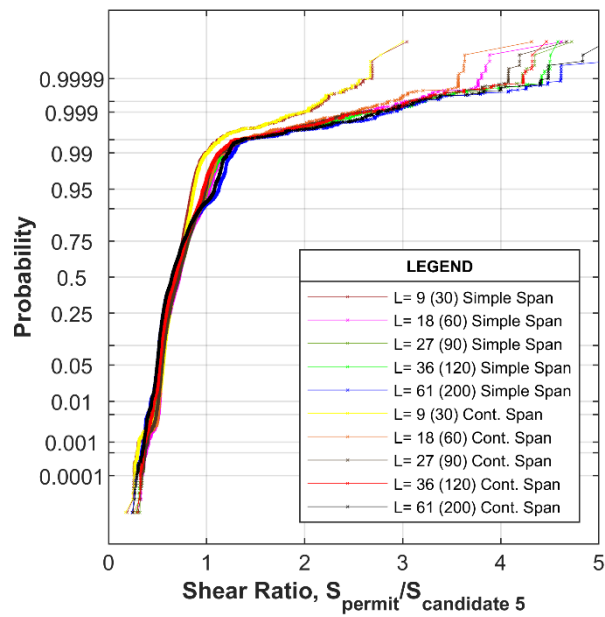


(d) Moment, Overloaded, WIM

Figure 3-C.5. Load effects of Candidates 6 for (a) moment for multi trip permit data (b) moment for overload WIM data, (c) shear for multi trip permit data (d) shear for overload WIM data.

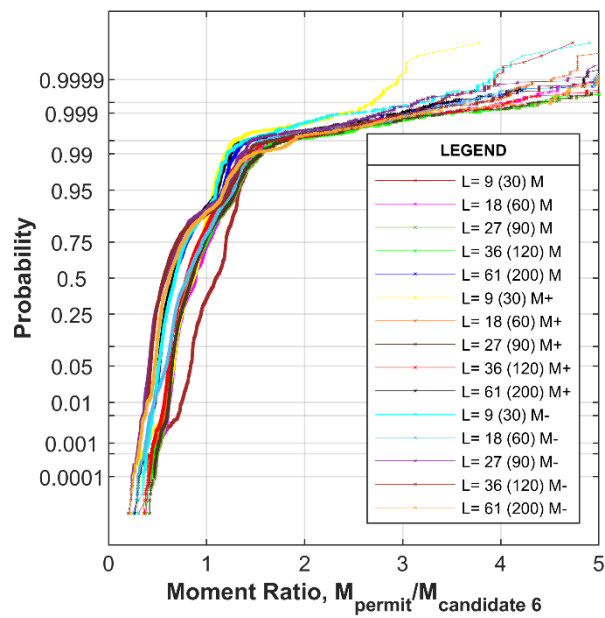


(a) Moment, Single Trip Permits

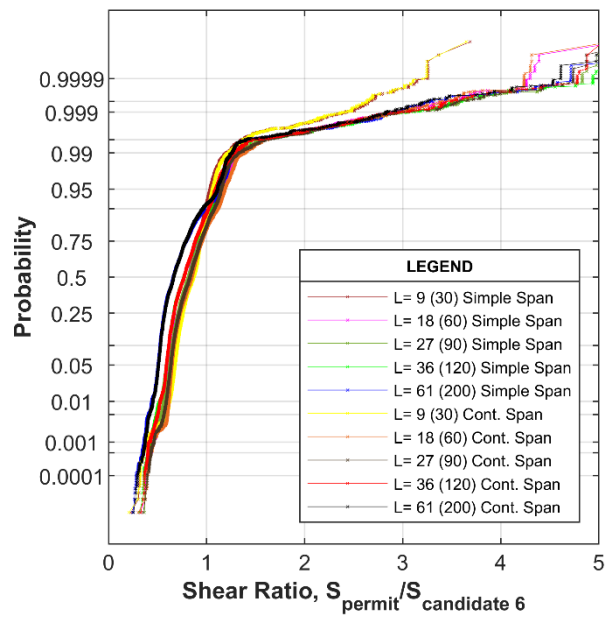


(b) Shear, Single Trip Permits

Figure 3-C.6. Load effects of Candidates 5 for (a) moment for single trip permit data (b) shear for single trip permit data (d).

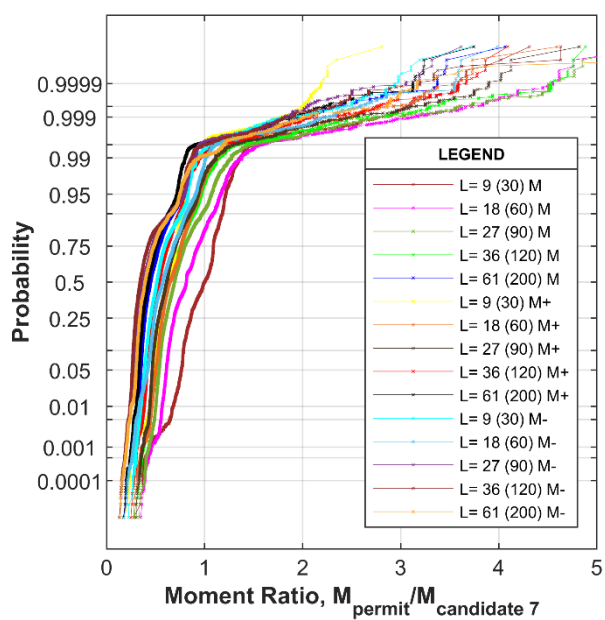


(a) Moment, Single Trip Permits

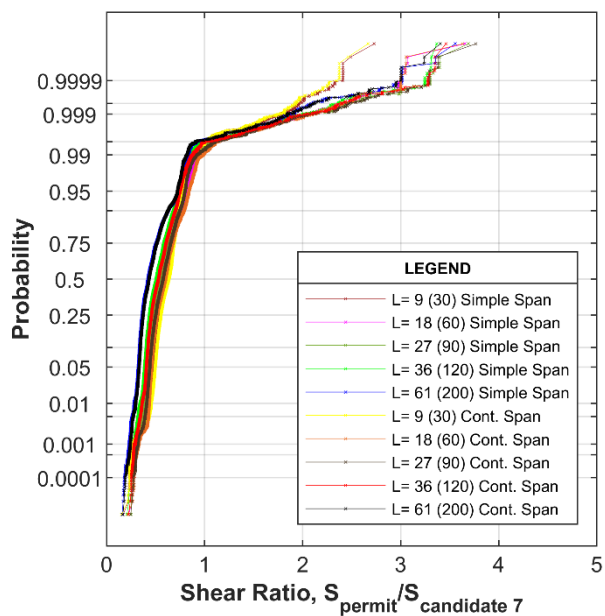


(b) Shear, Single Trip Permits

Figure 3-C.7. Load effects of Candidates 6 for (a) moment for single trip permit data (b) shear for single trip permit data (d).

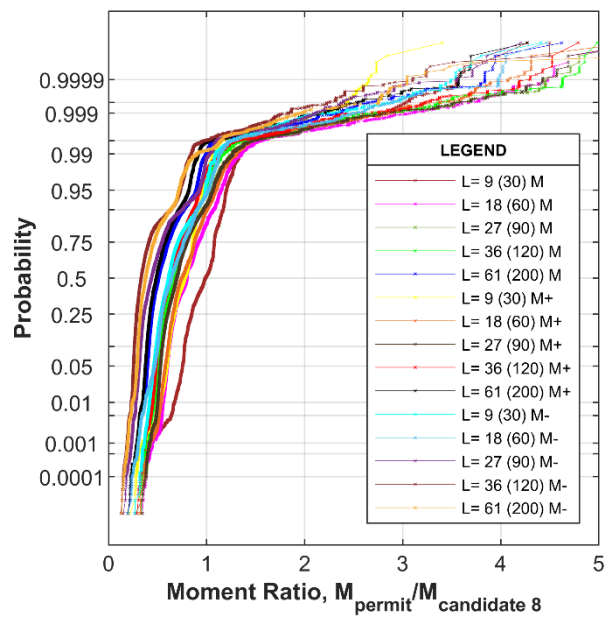


(a) Moment, Single Trip Permits

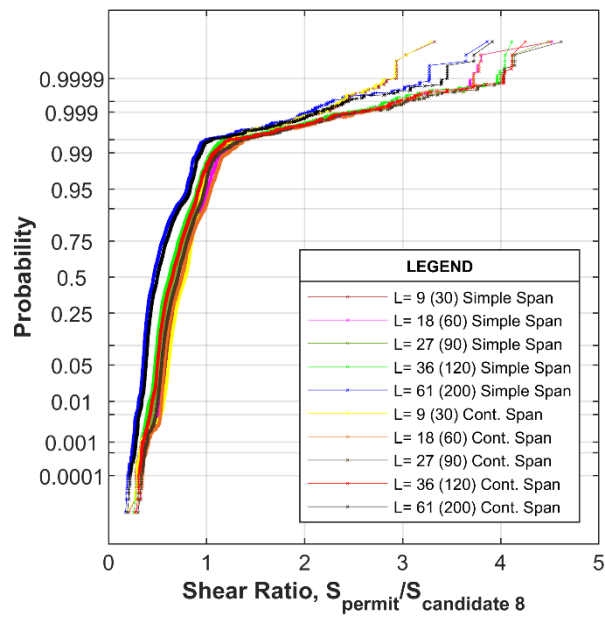


(b) Shear, Single Trip Permits

Figure 3-C.8. Load effects of Candidates 7 for (a) moment for single trip permit data (b) shear for single trip permit data (d).

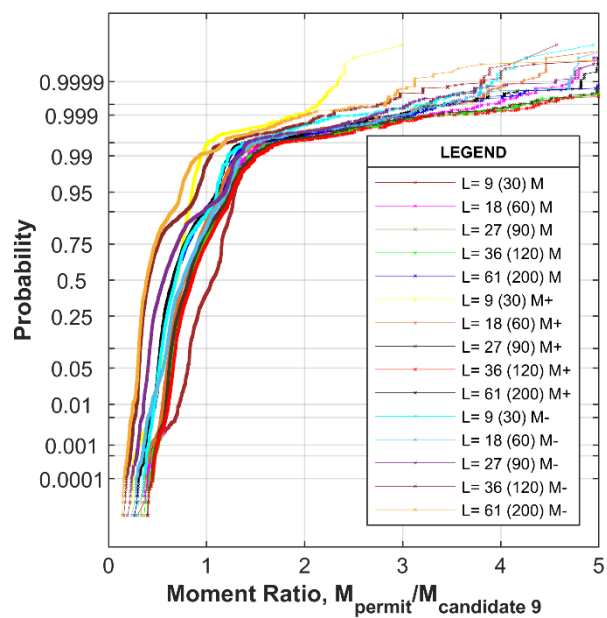


(a) Moment, Single Trip Permits

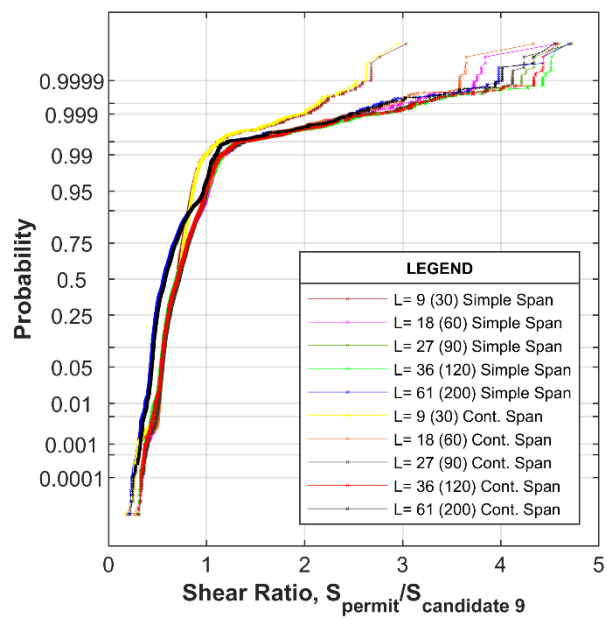


(b) Shear, Single Trip Permits

Figure 3-C.9. Load effects of Candidates 8 for (a) moment for single trip permit data (b) shear for single trip permit data (d).



(a) Moment, Single Trip Permits



(c) Shear, Single Trip Permits

Figure 3-C.10. Load effects of Candidates 9 for (a) moment for single trip permit data (b) shear for single trip permit data (d).

Appendix 3-D. Reliability index for notional permit candidates

Candidates 1-4, and 6 for multi trip permit vehicles

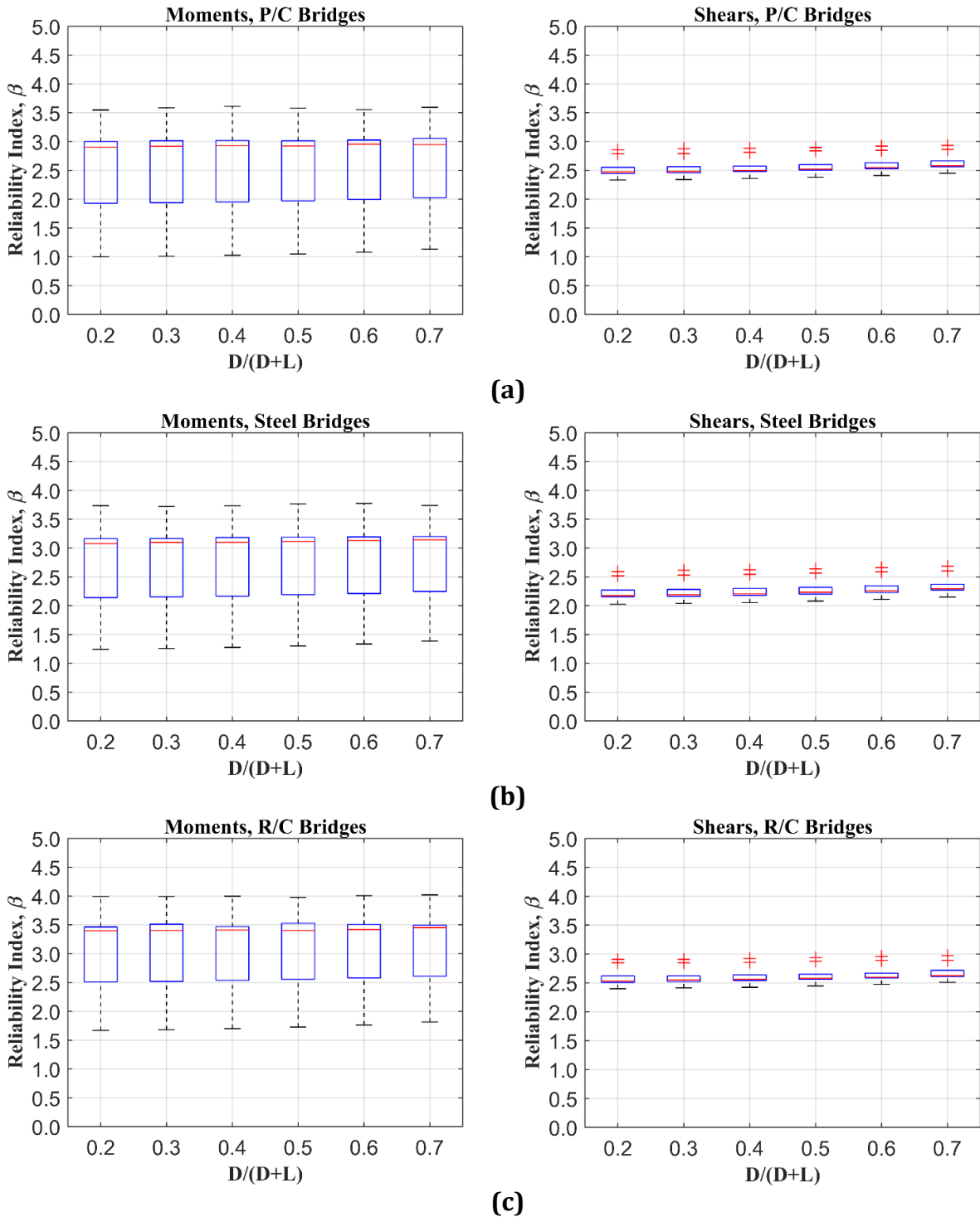
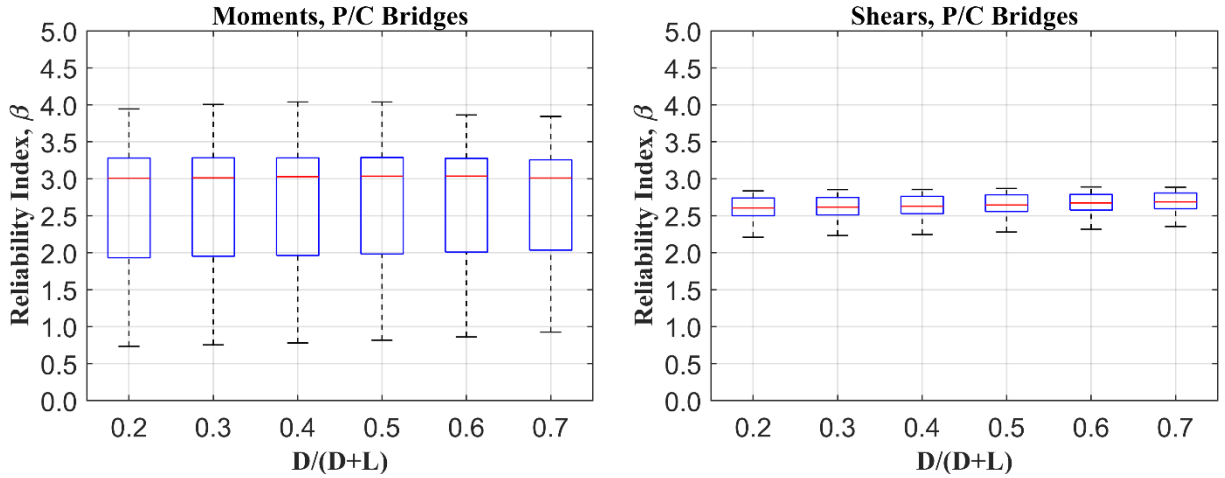
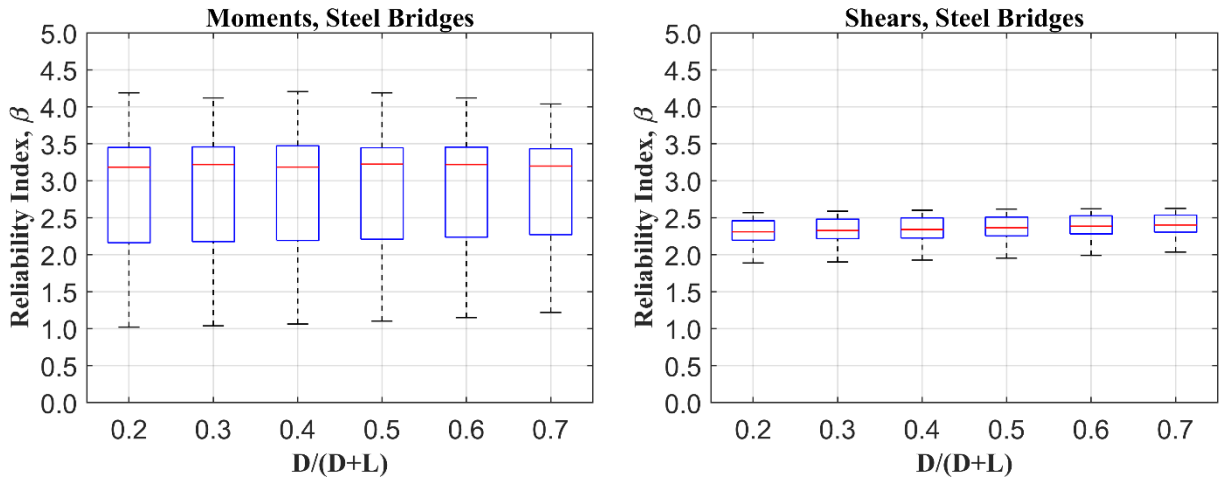


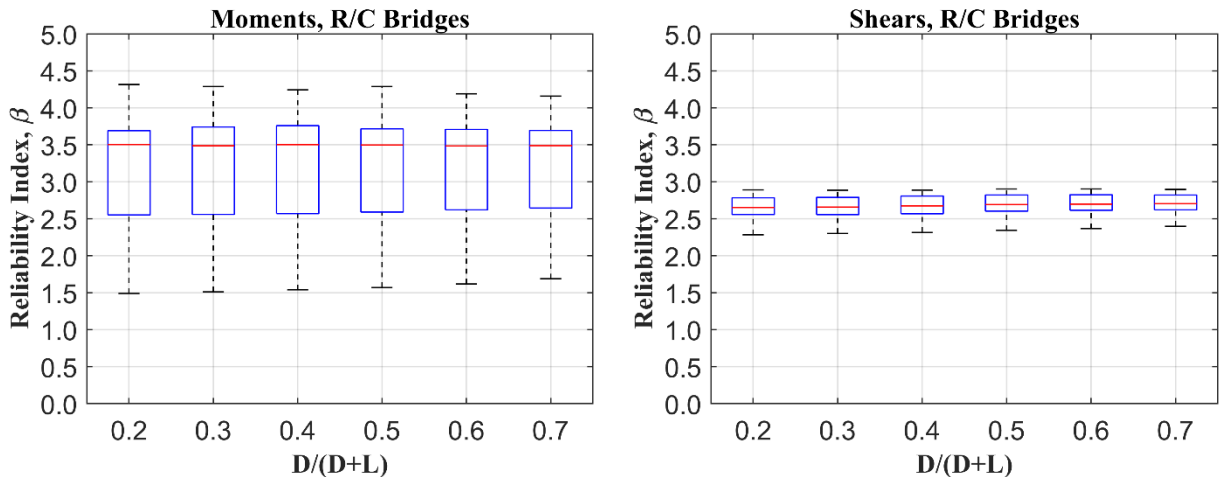
Figure 3-D.1. Reliability index for Candidate 1 for (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



(a)

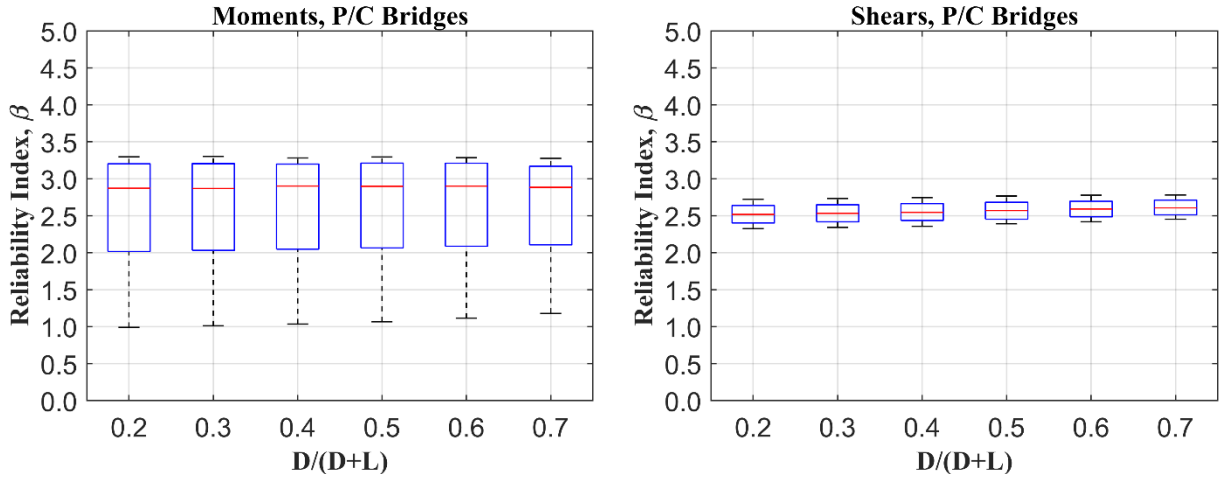


(b)

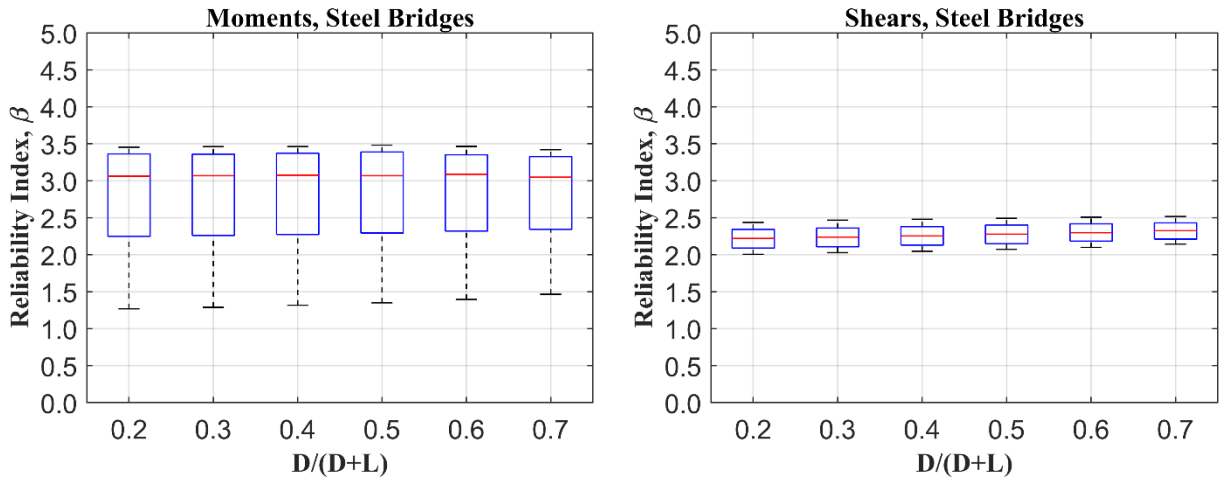


(c)

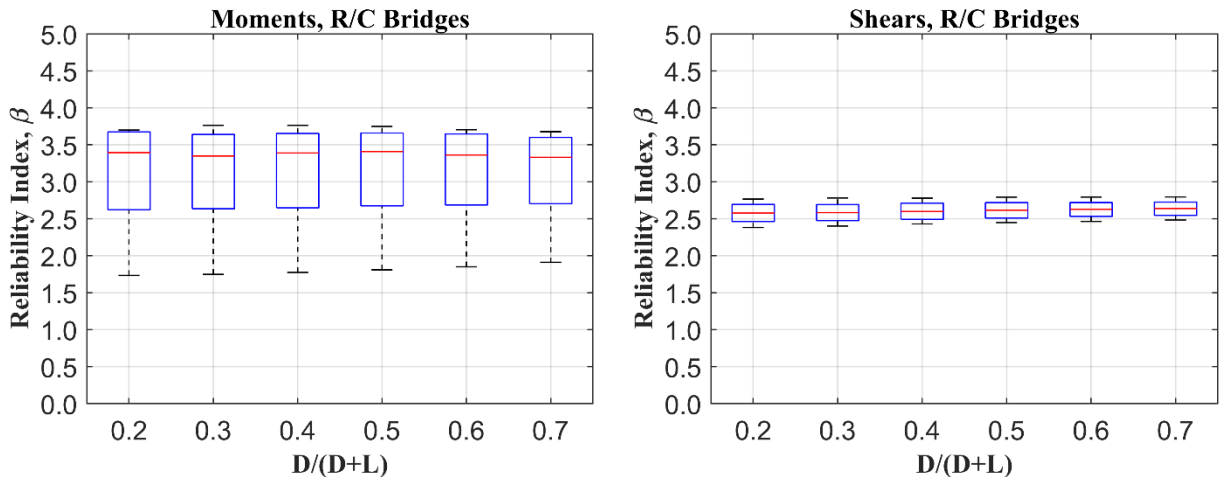
Figure 3-D.2. Reliability index for Candidate 2 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



(a)

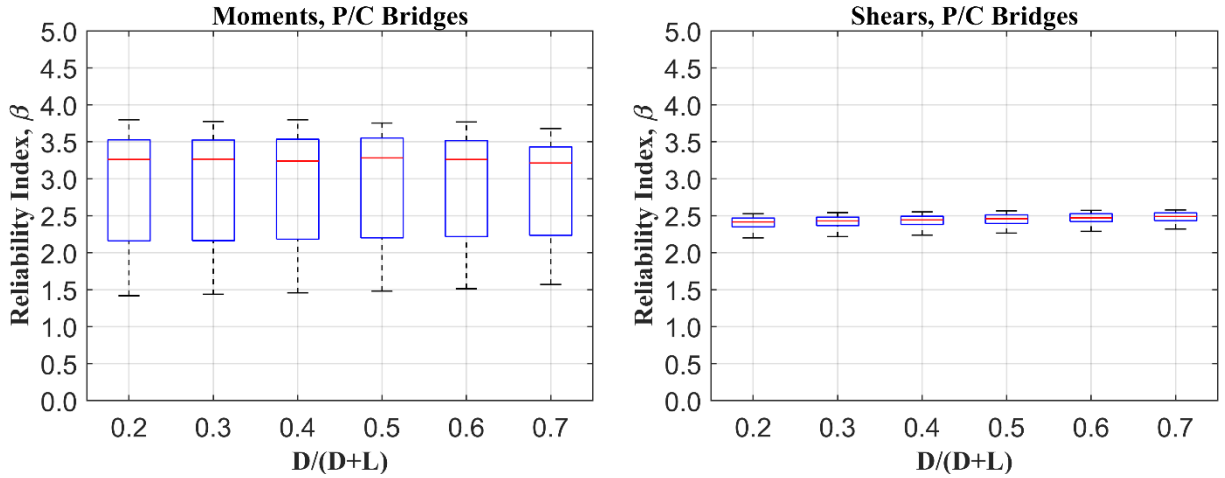


(b)

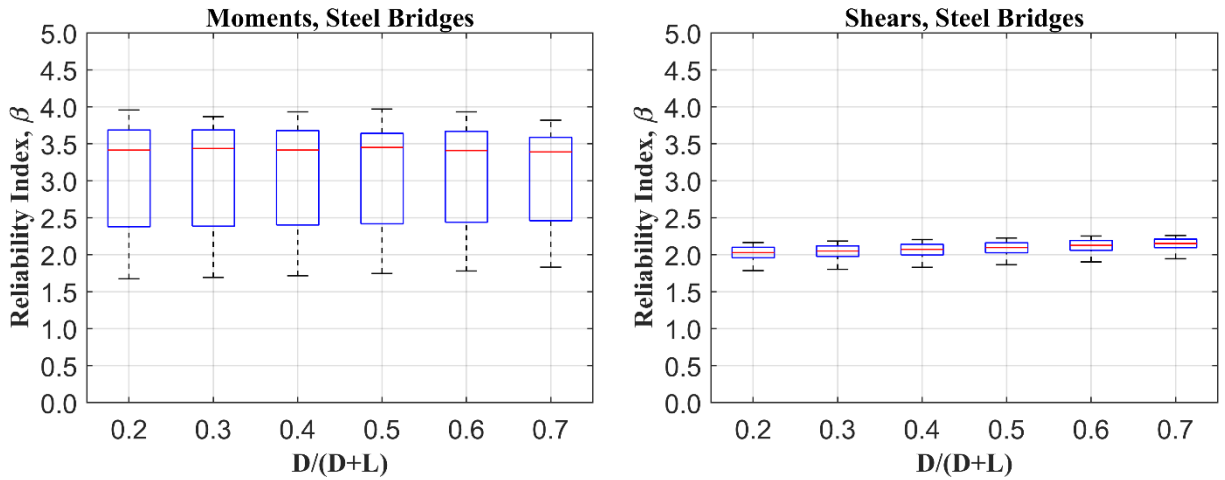


(c)

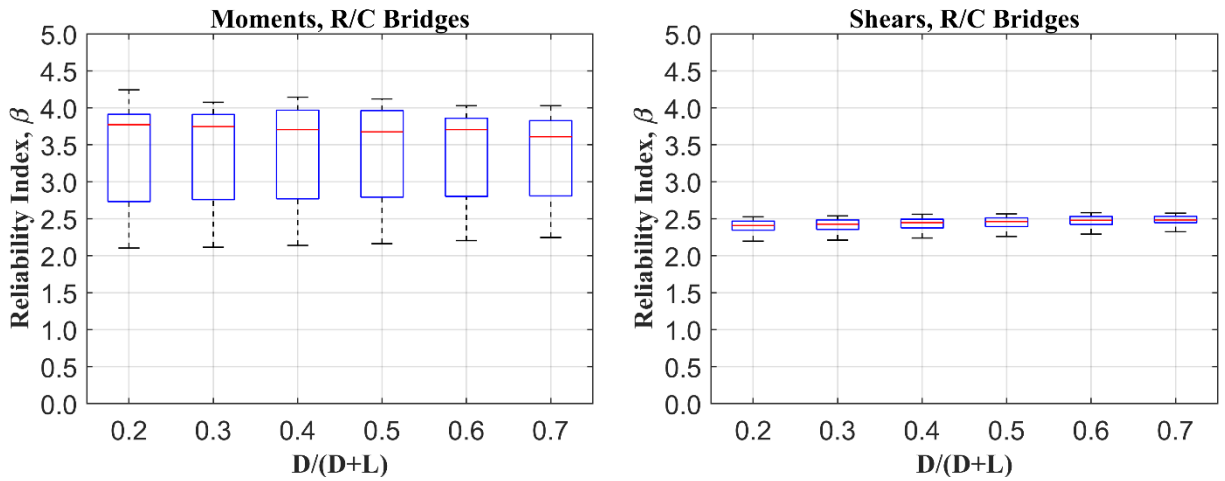
Figure 3-D.3. Reliability index for Candidate 3 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



(a)



(b)



(c)

Figure 3-D.4. Reliability index for Candidate 4 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.

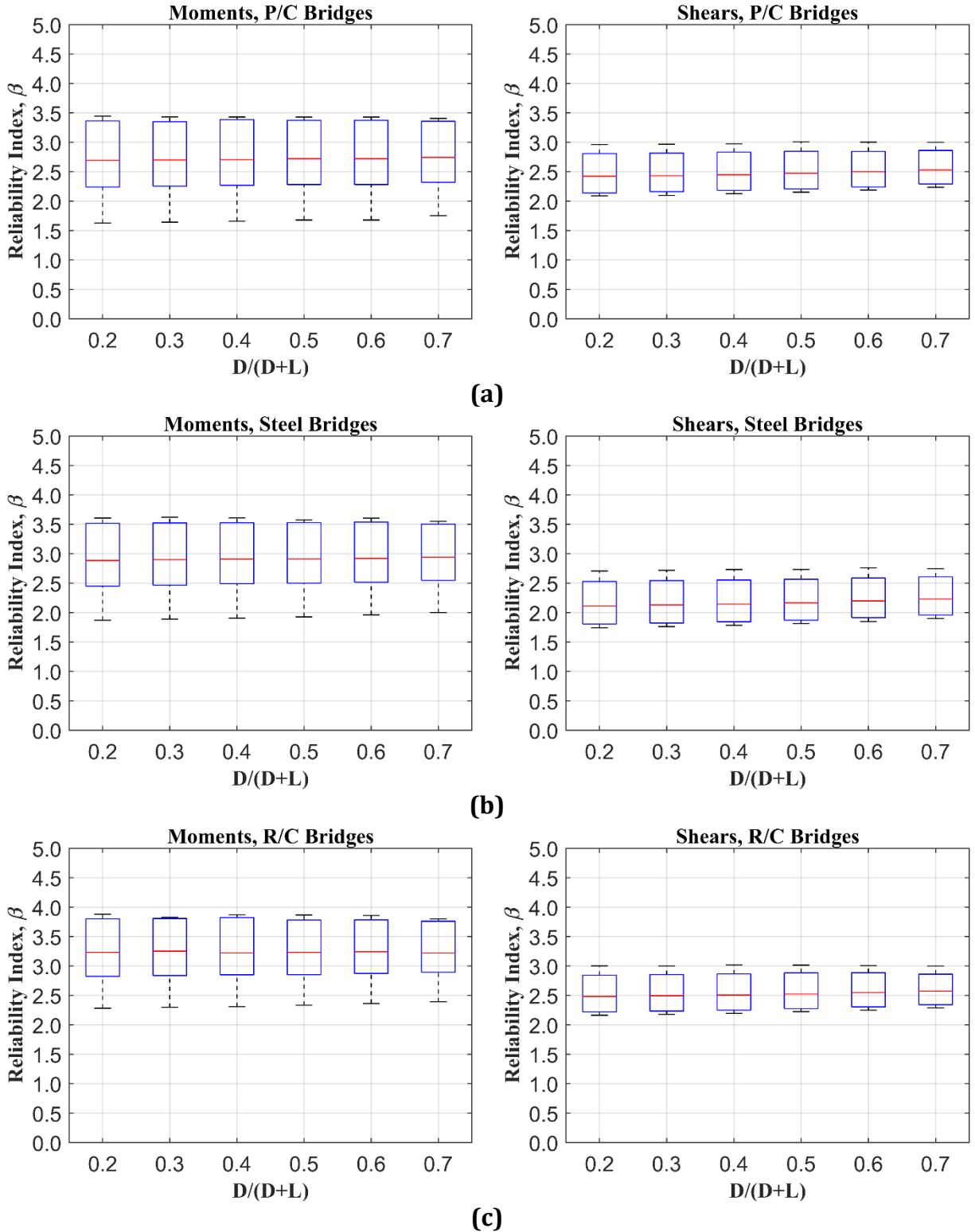
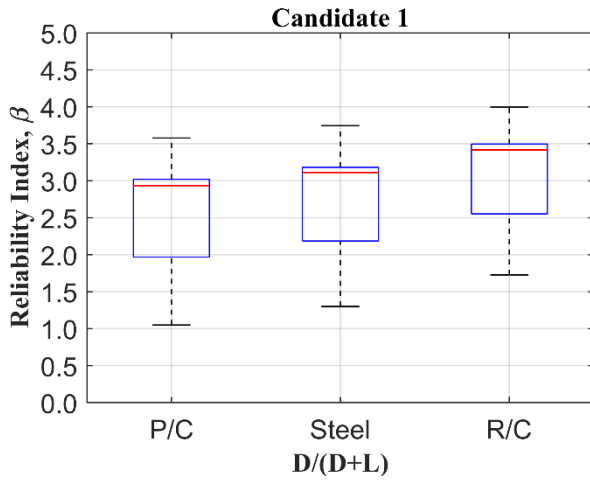
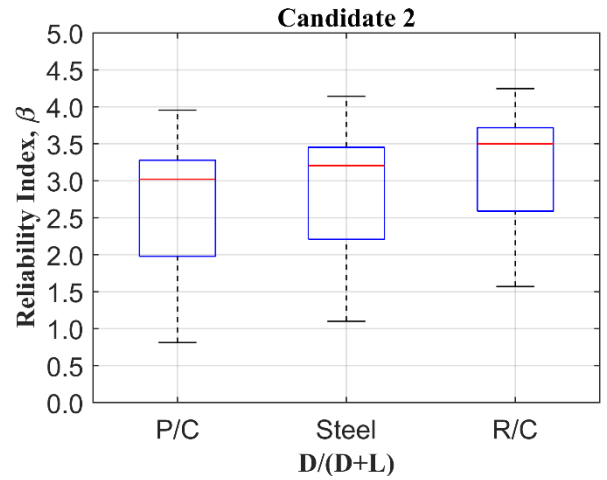


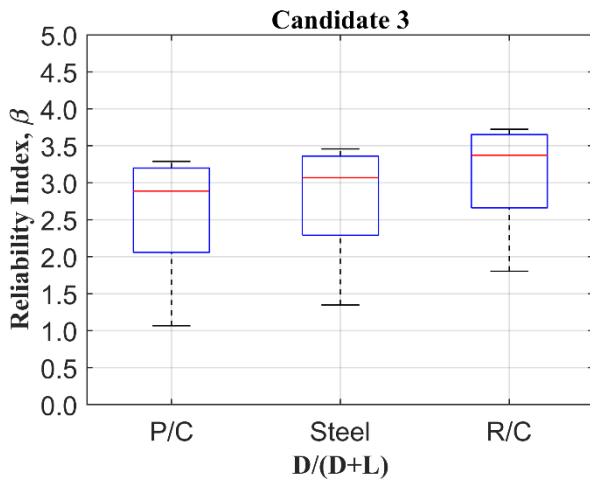
Figure 3-D.7. Reliability index for Candidate 6, (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



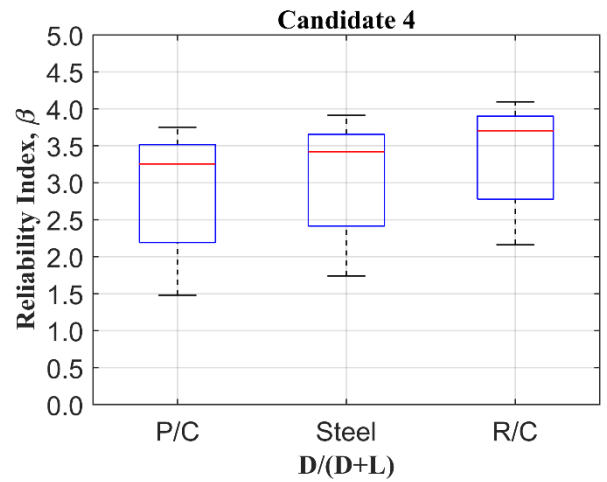
(a)



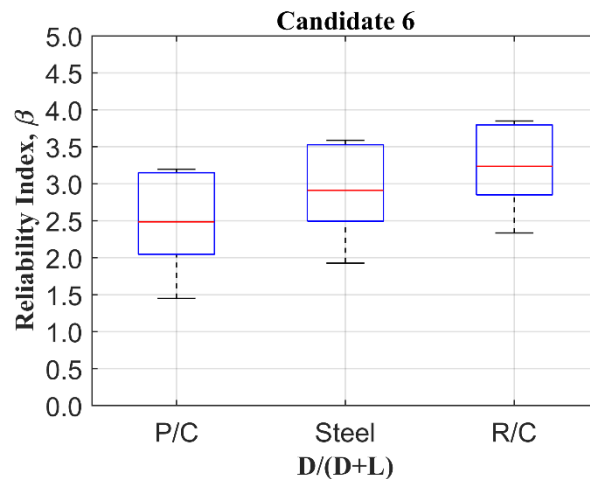
(b)



(c)

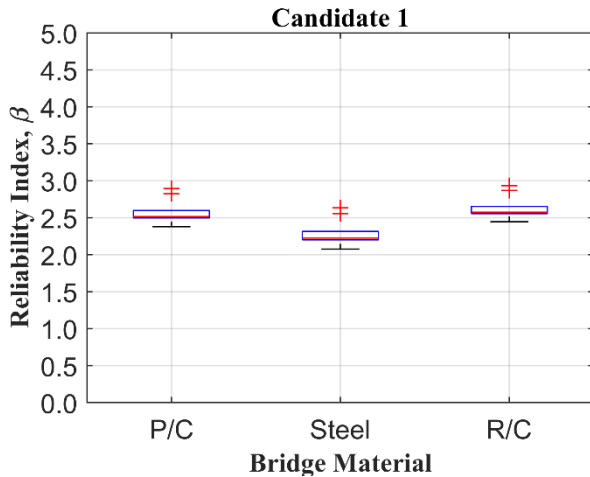


(d)

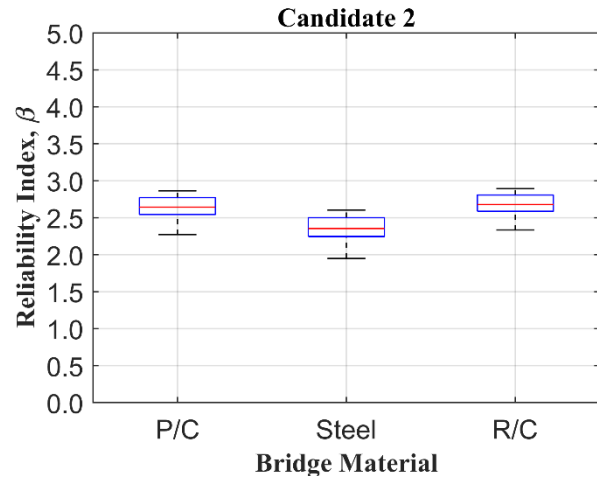


(e)

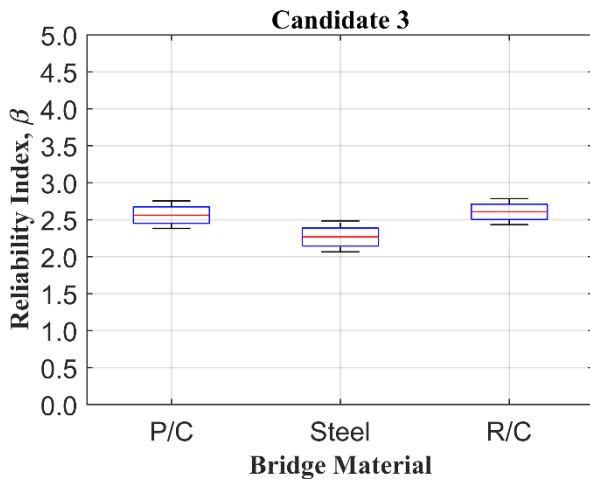
Figure 3-D.5. Average reliability index for moment (a) Candidate 1 (b) Candidate 2 (c) Candidate 3 (d) Candidate 4 (e) Candidate 6.



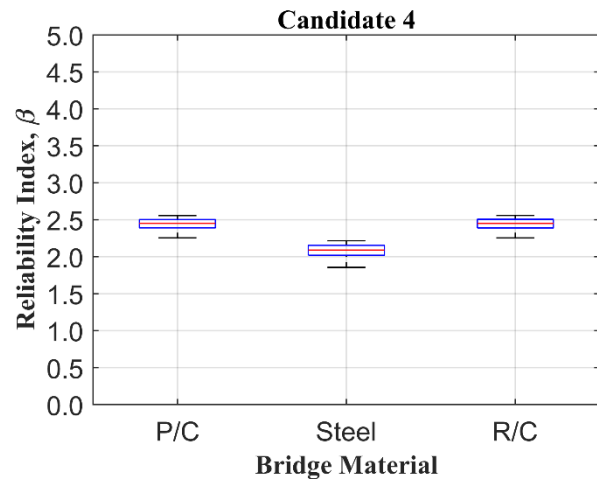
(a)



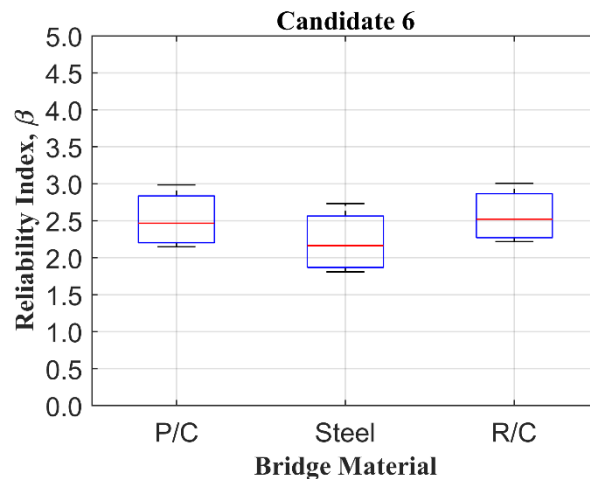
(b)



(c)



(d)



(e)

Figure 3-D.6. Average reliability index for shear (a) Candidate 1 (b) Candidate 2 (c) Candidate 3 (d) Candidate 4 (e) Candidate 6.

Candidates 5-9 for single trip permit vehicles

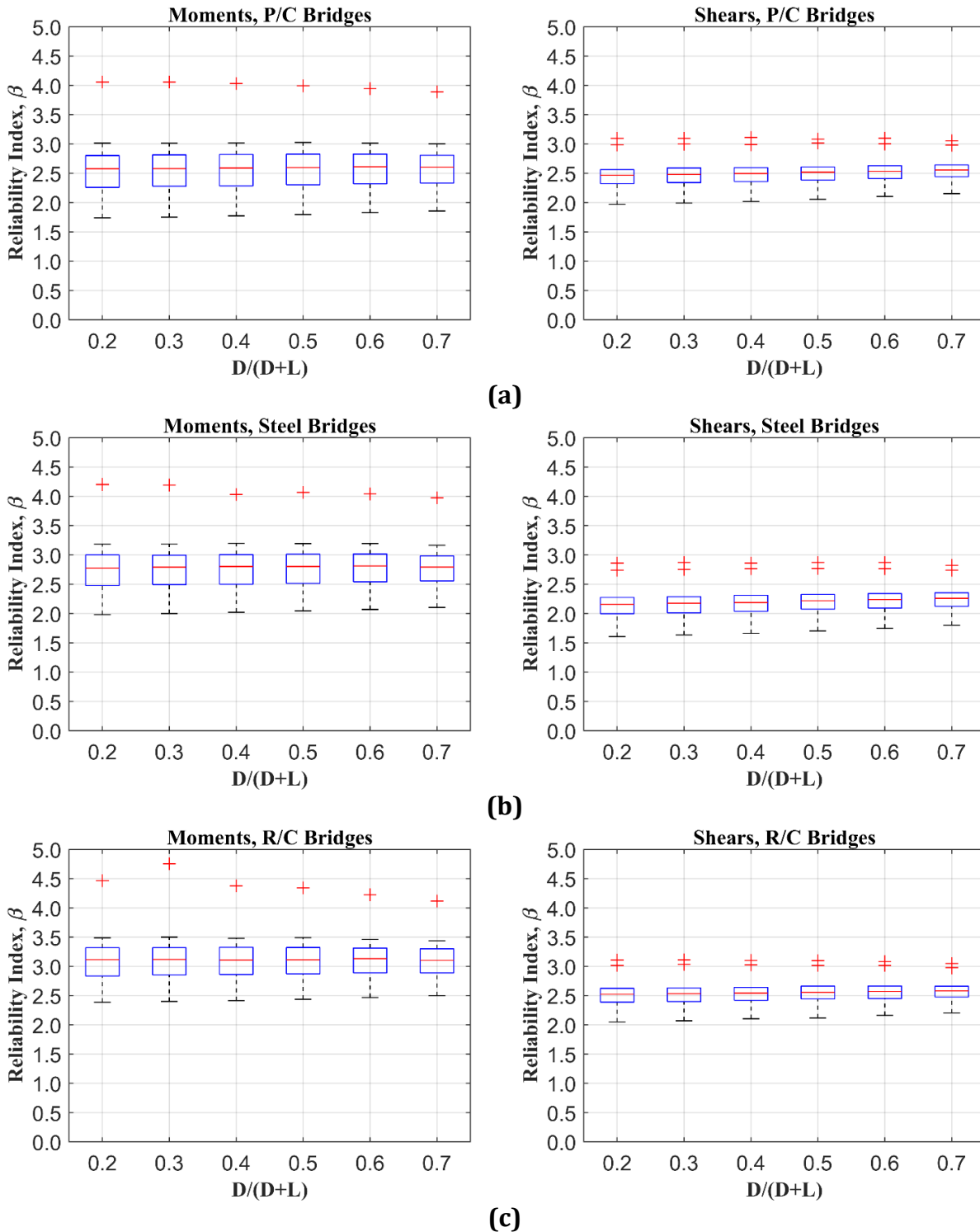
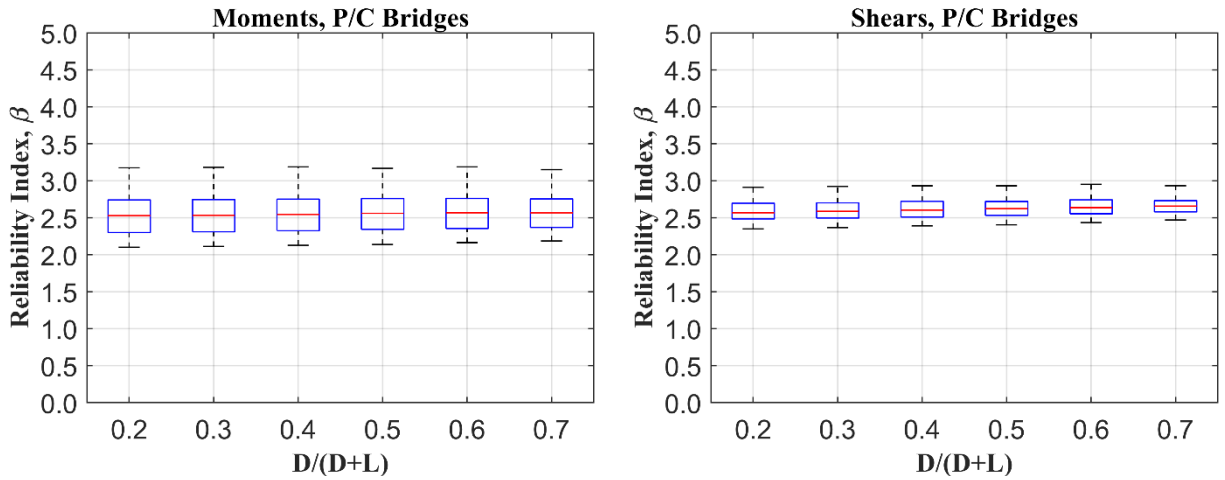
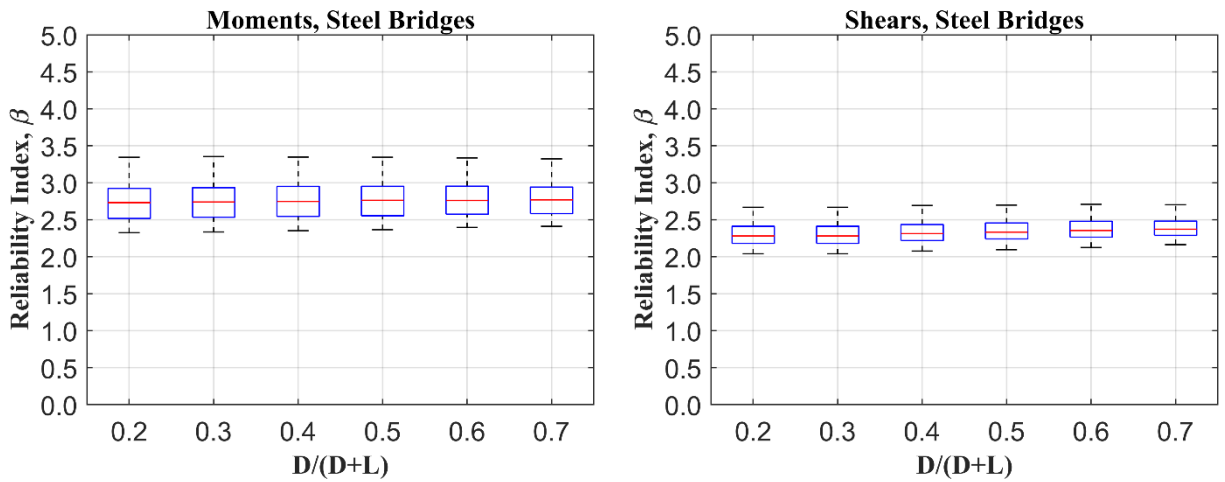


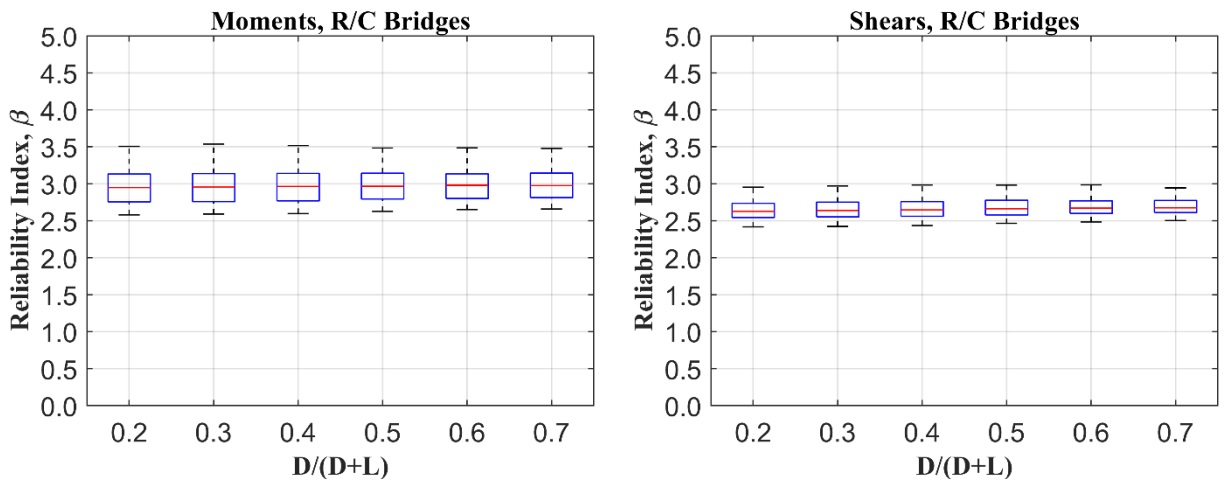
Figure 3-D.8. Reliability index for Candidate 5 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



(a)



(b)



(c)

Figure 3-D.9. Reliability index for Candidate 6 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.

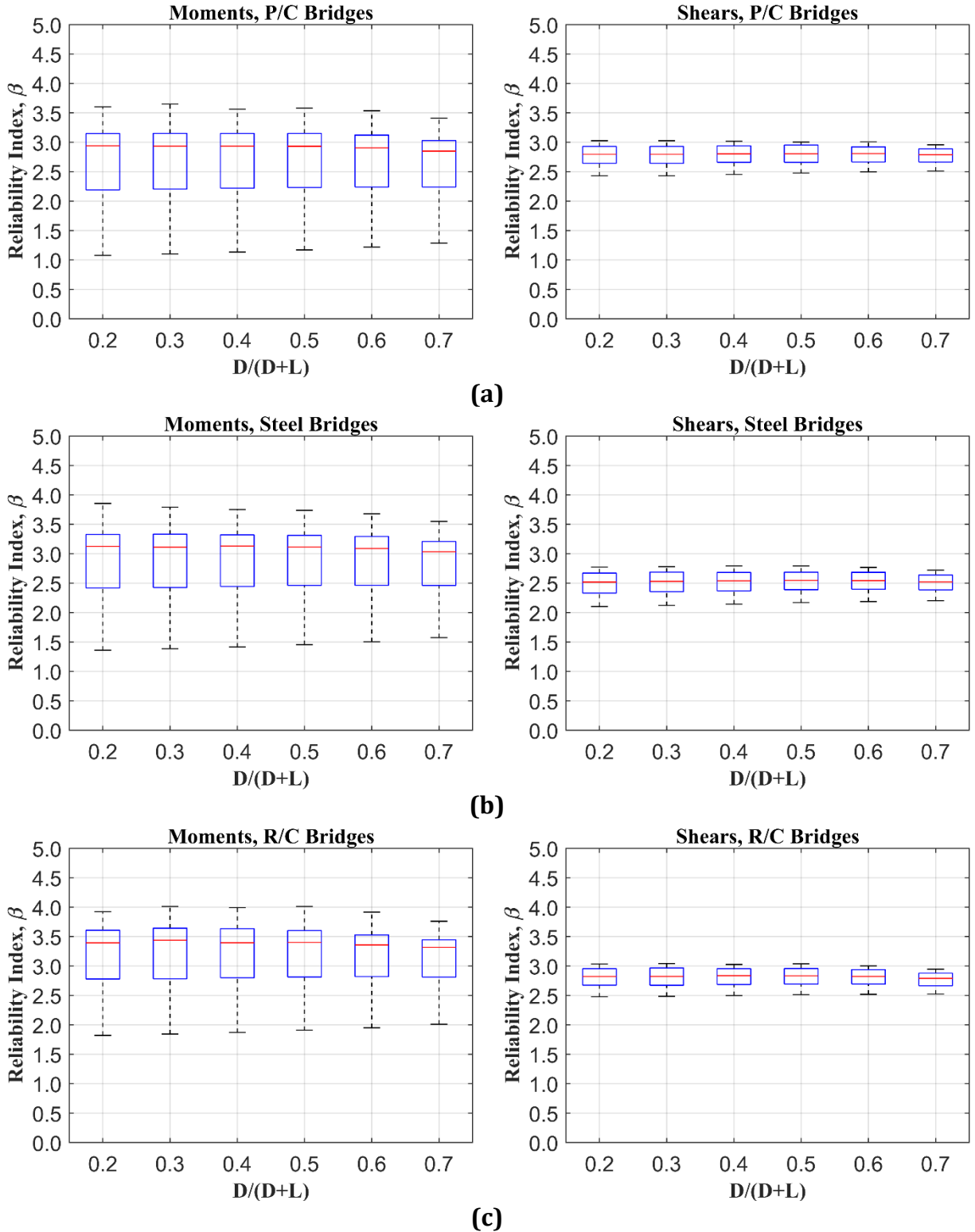


Figure 3-D.10. Reliability index for Candidate 7 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.

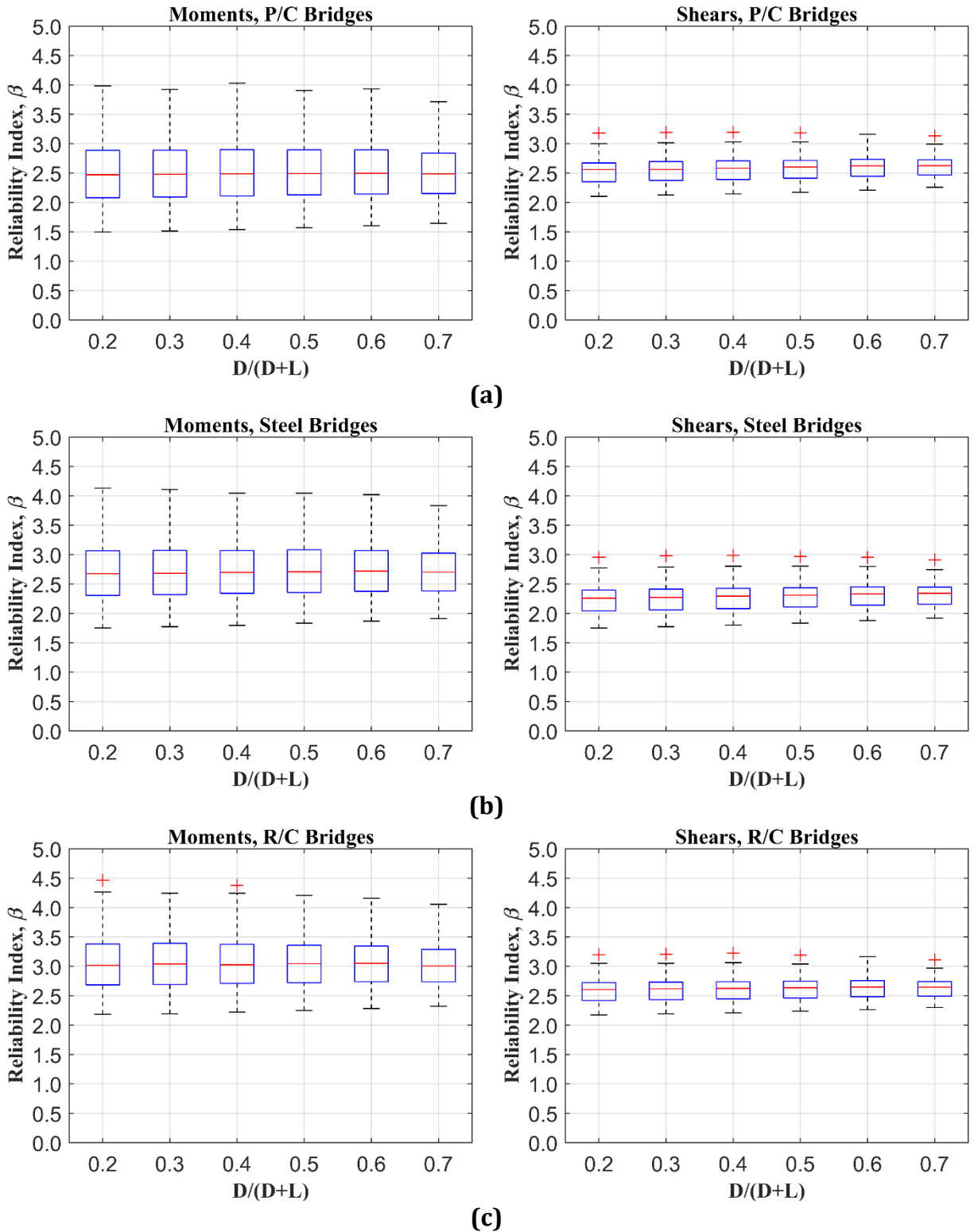
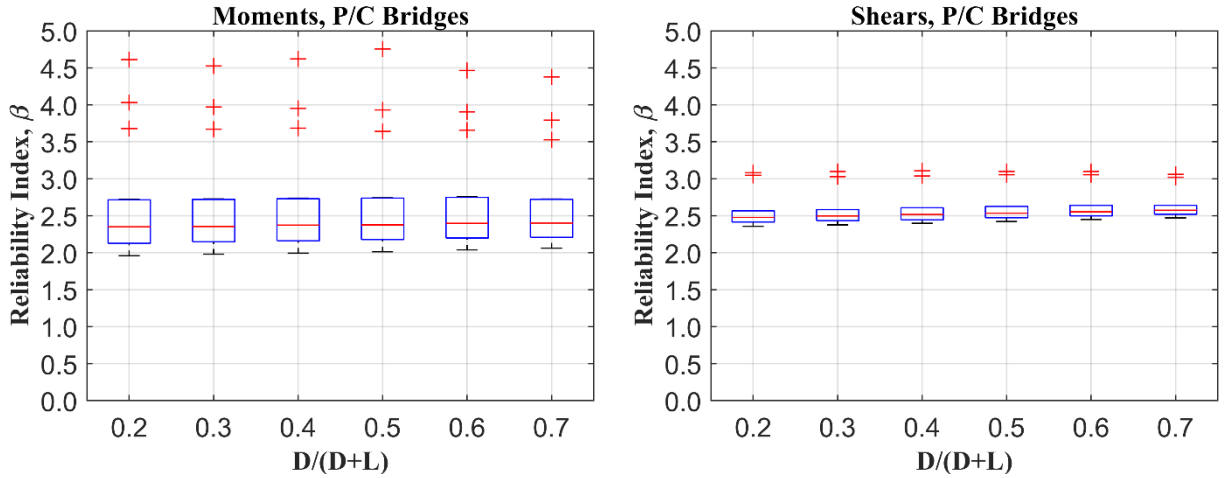
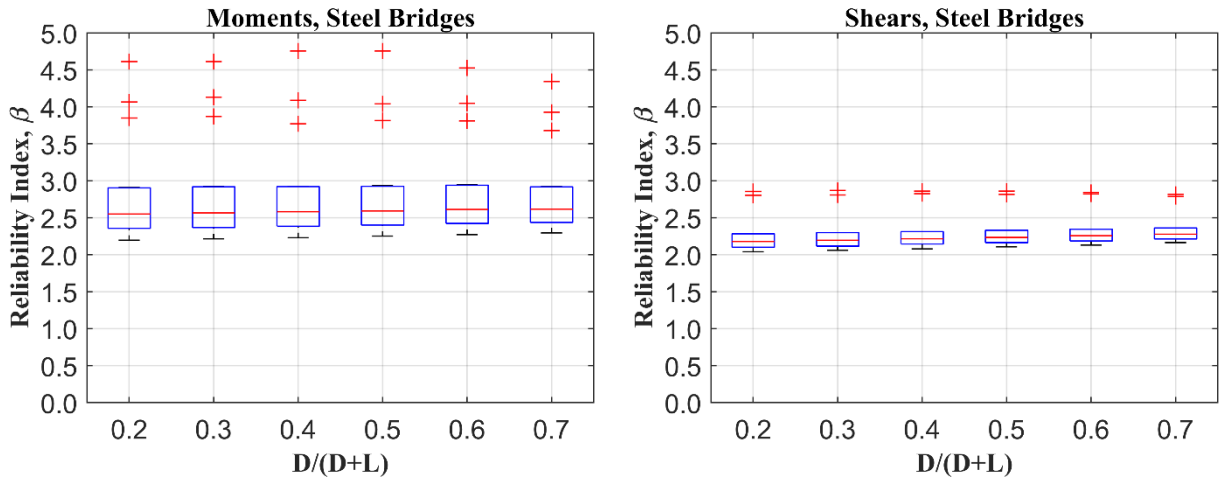


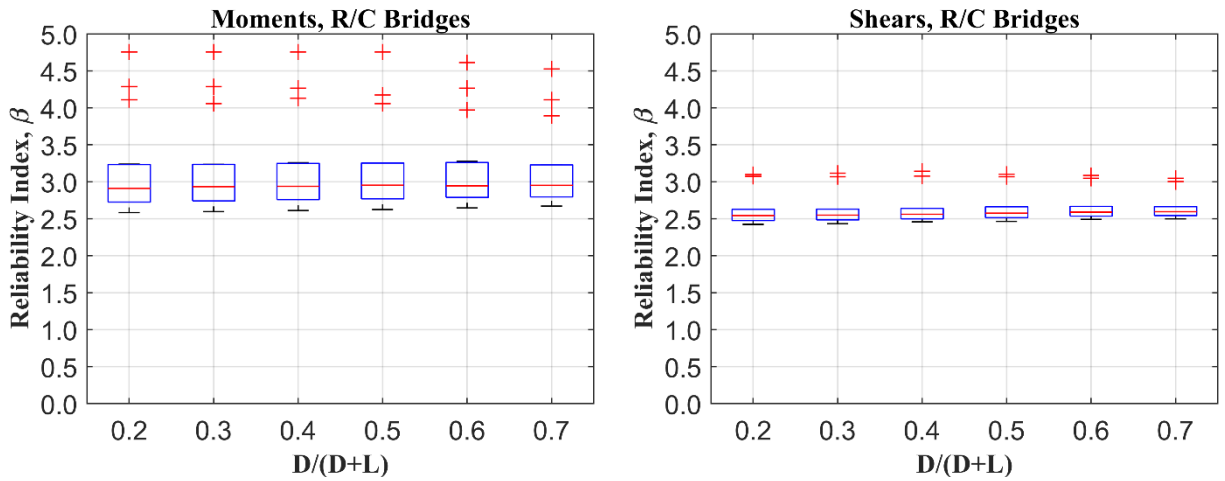
Figure 3-D.11. Reliability index for Candidate 8 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



(a)

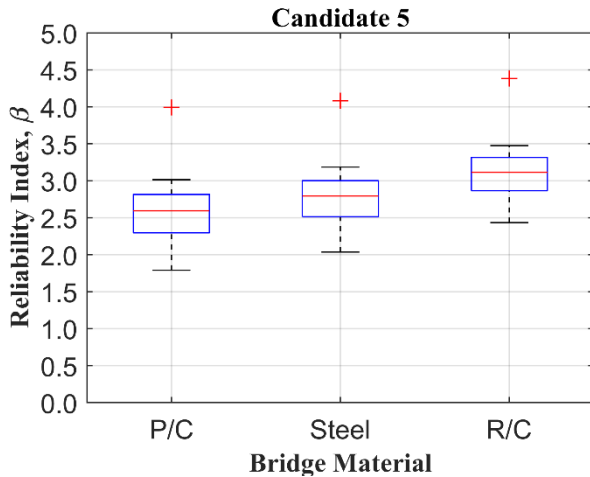


(b)

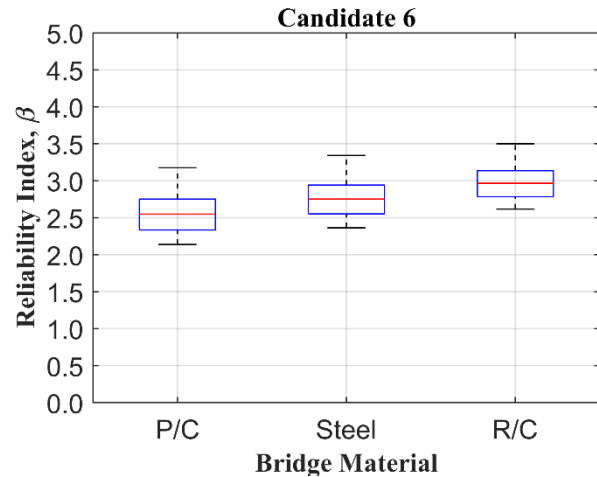


(c)

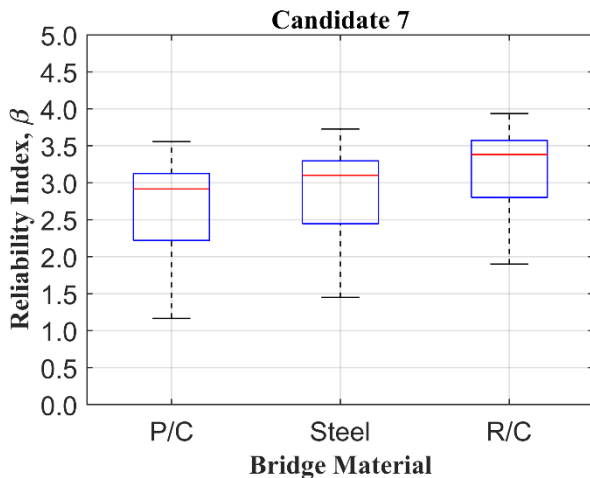
Figure 3-D.12. Reliability index for Candidate 9 (a) prestressed concrete (P/C), (b) steel, and (c) reinforced concrete (R/C) bridges for various moment and shear dead load ratios.



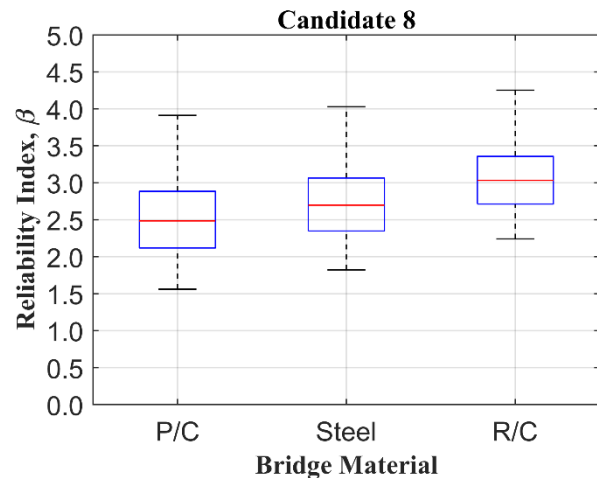
(a)



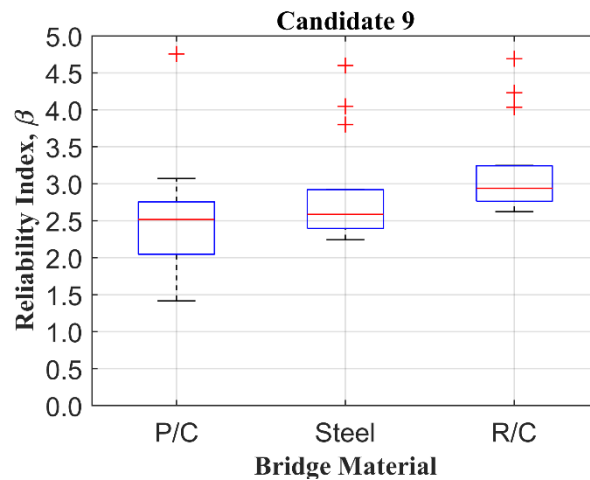
(b)



(c)

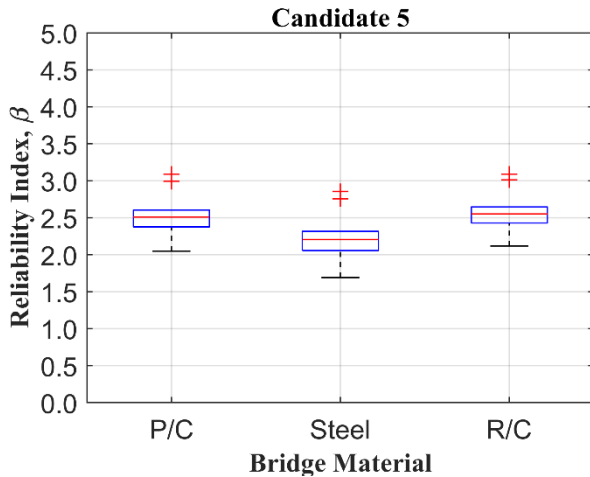


(d)

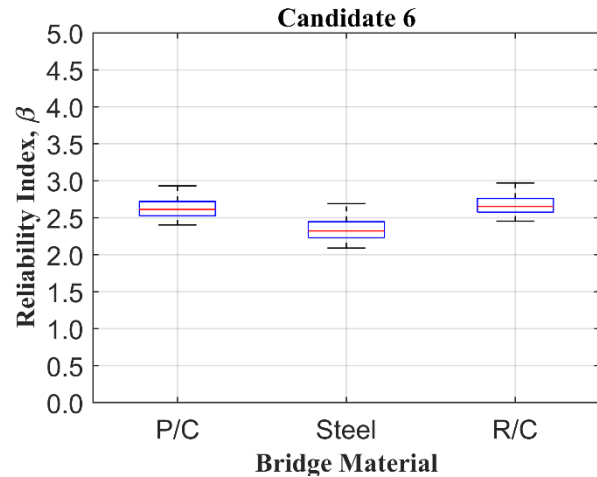


(e)

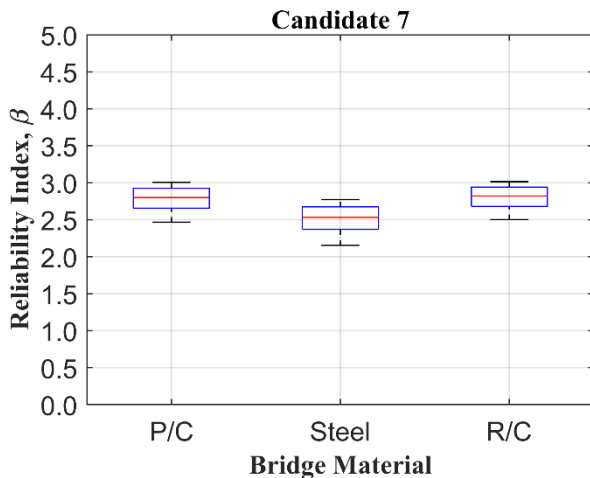
Figure 3-D.13. Average reliability index for moment (a) Candidate 5 (b) Candidate 6 (c) Candidate 7 (d) Candidate 8 (e) Candidate 9.



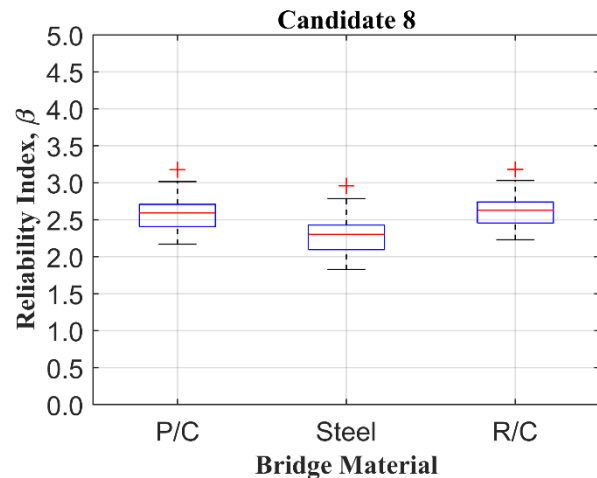
(a)



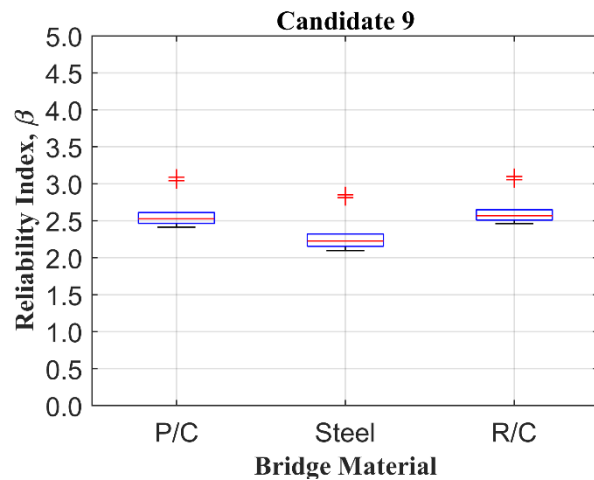
(b)



(c)



(d)



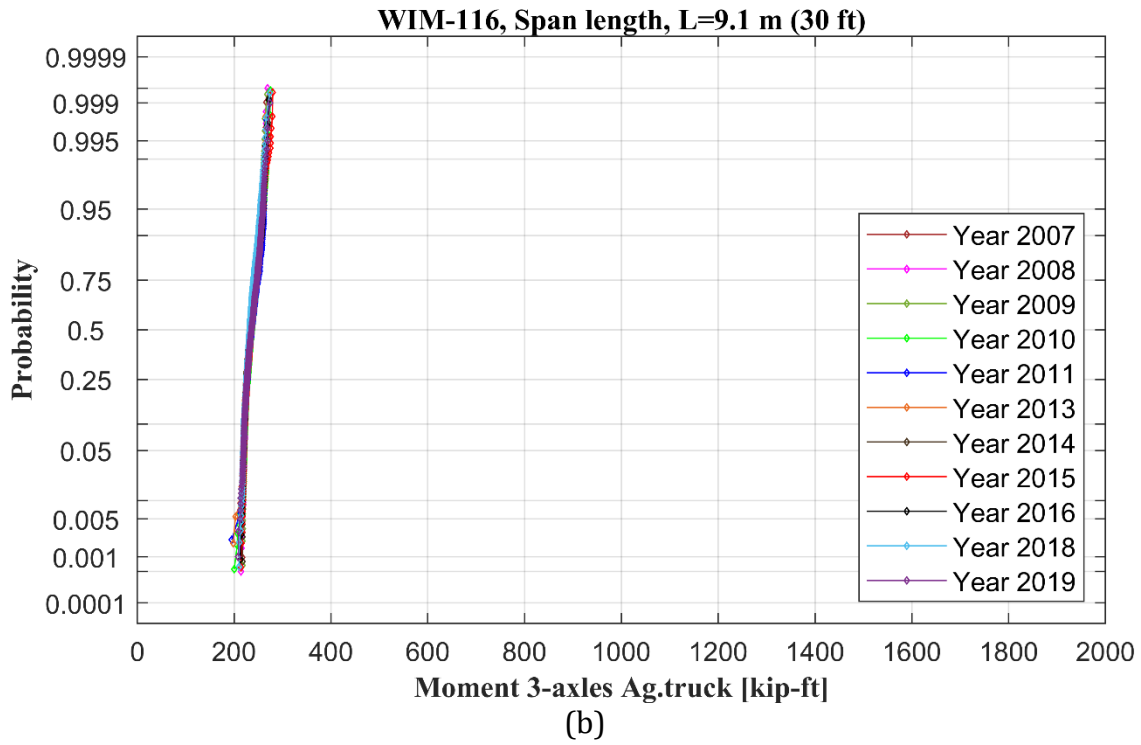
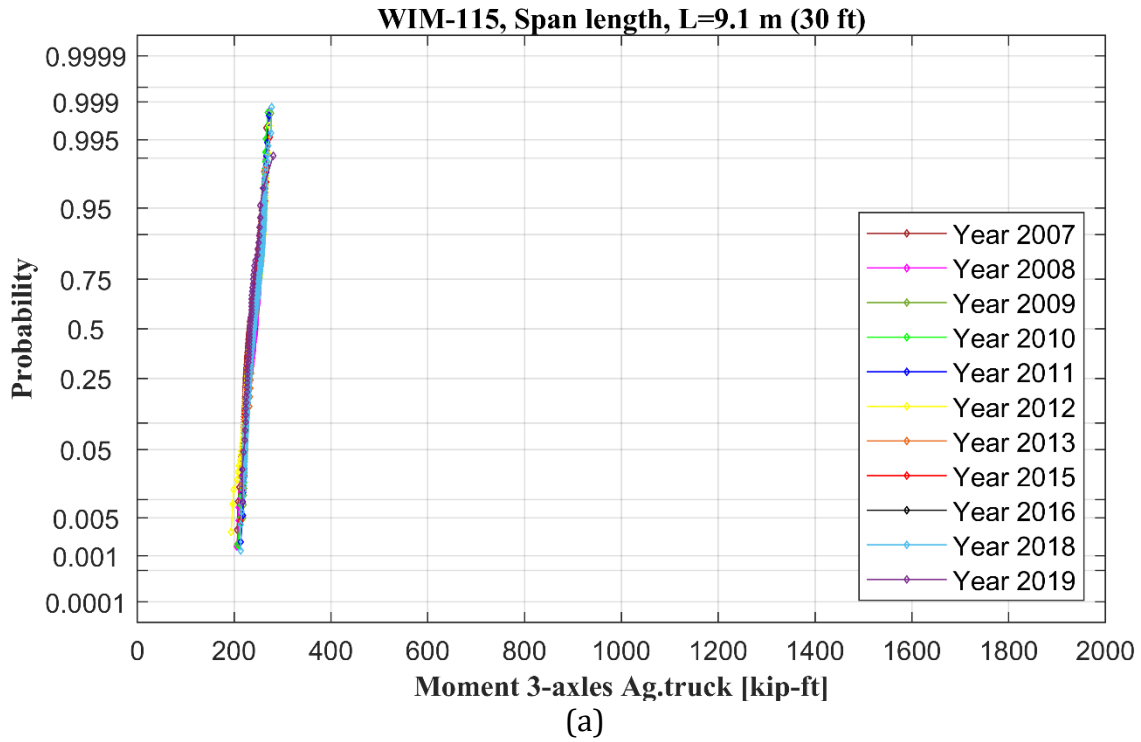
(e)

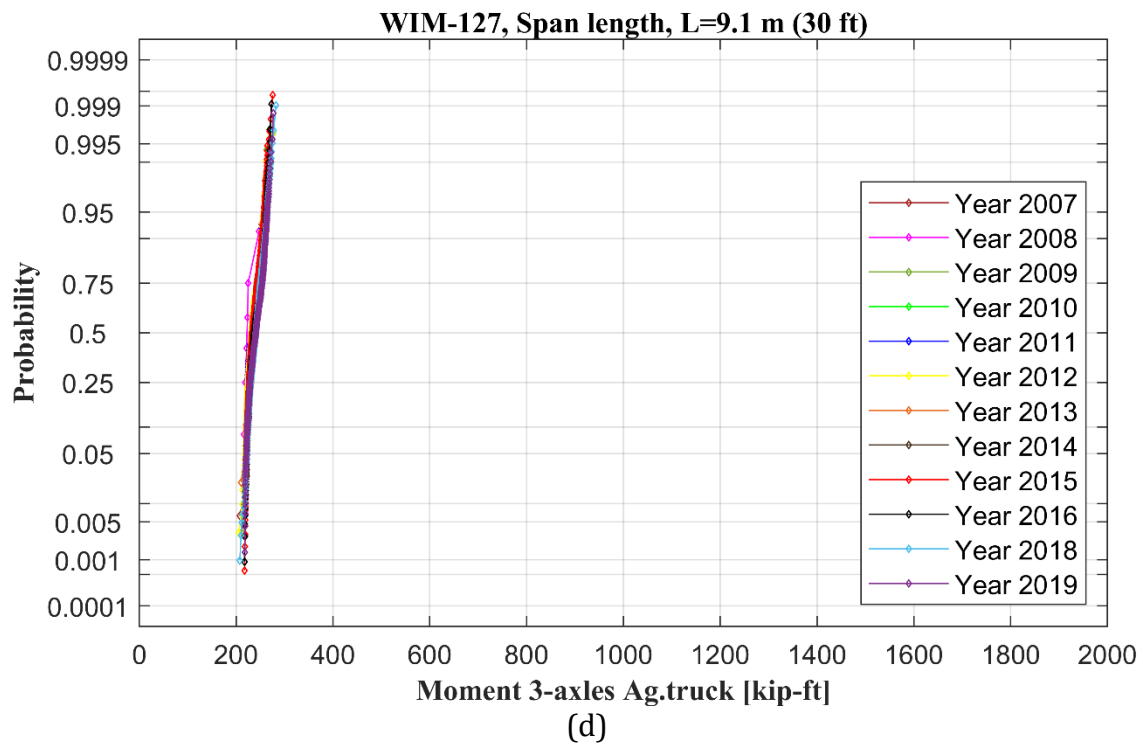
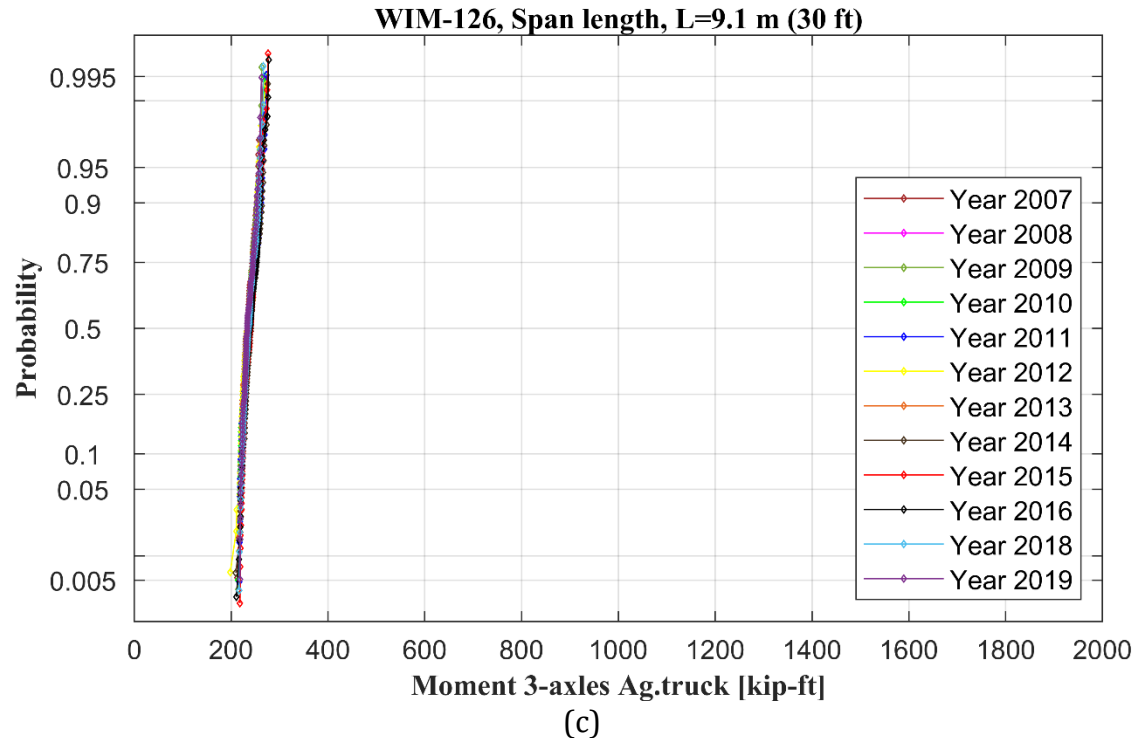
Figure 3-B.11. Average reliability index for shear (a) Candidate 5 (b) Candidate 6 (c) Candidate 7 (d) Candidate 8 (e) Candidate 9.

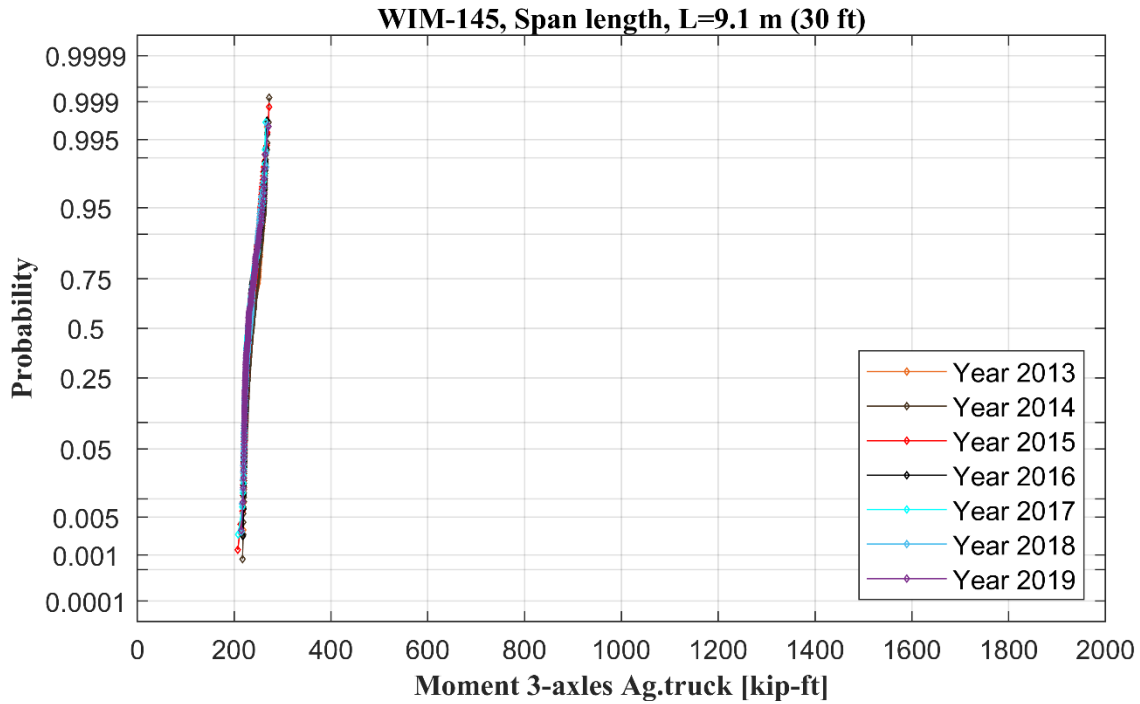
APPENDIX 4

Results for Rating for Grandfather Vehicles

Appendix 4-A. CDF of moment effects for 3-axle agricultural vehicles

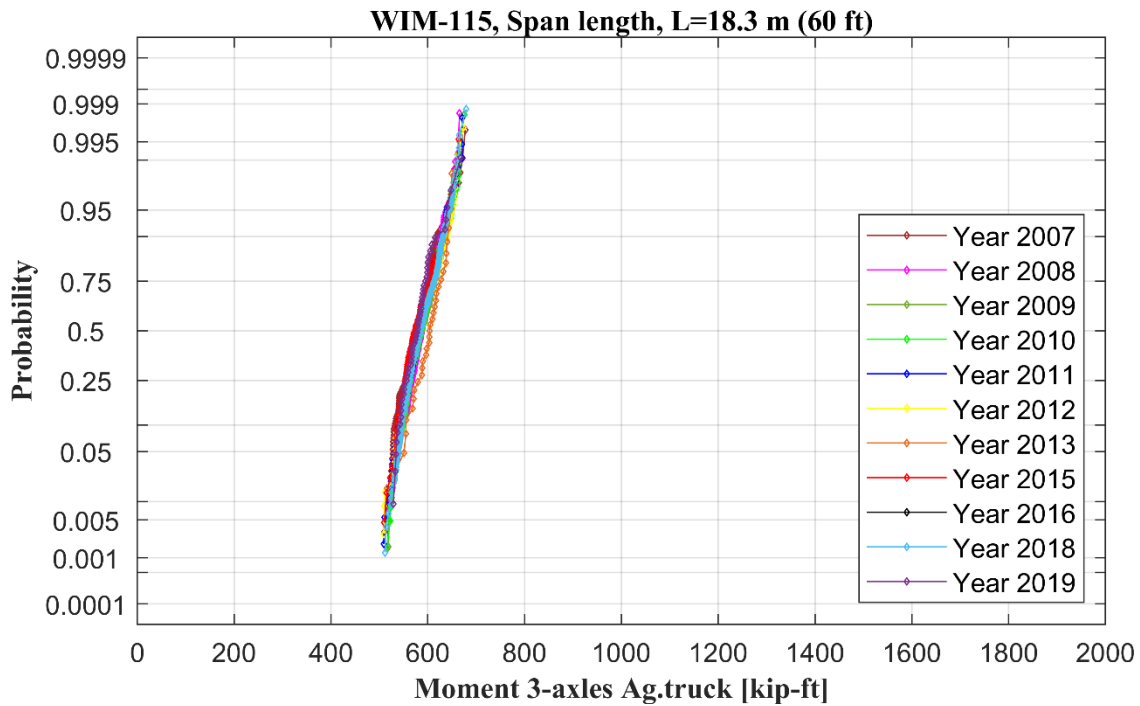




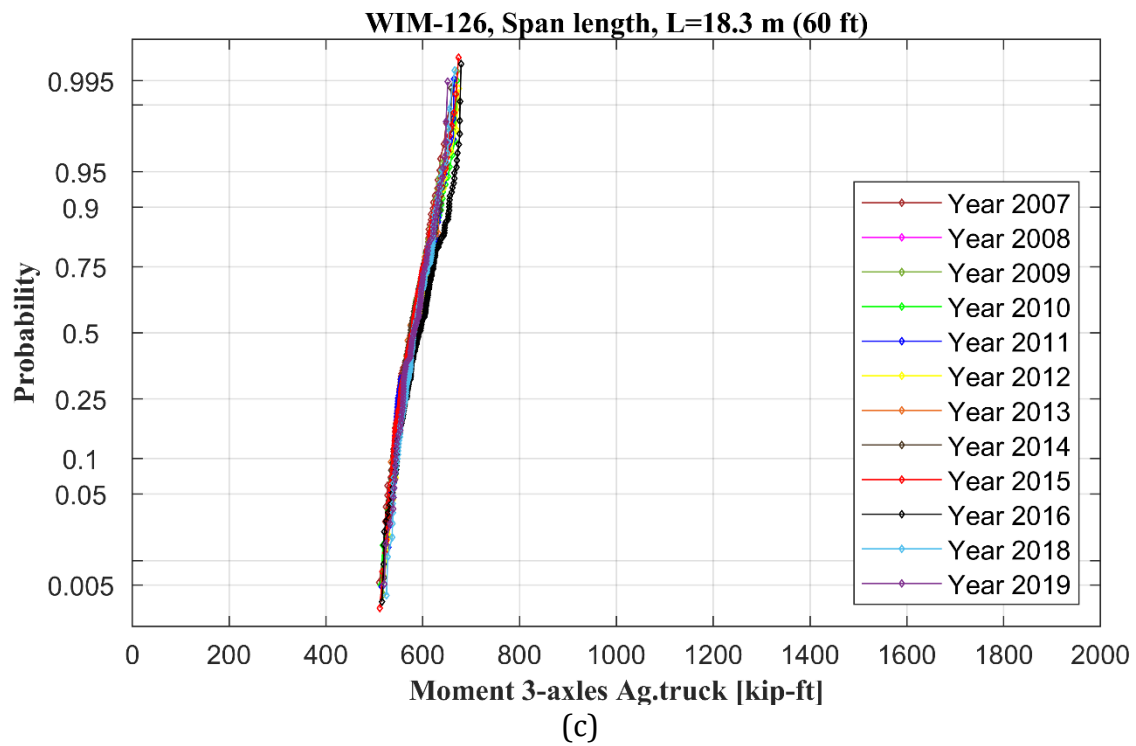
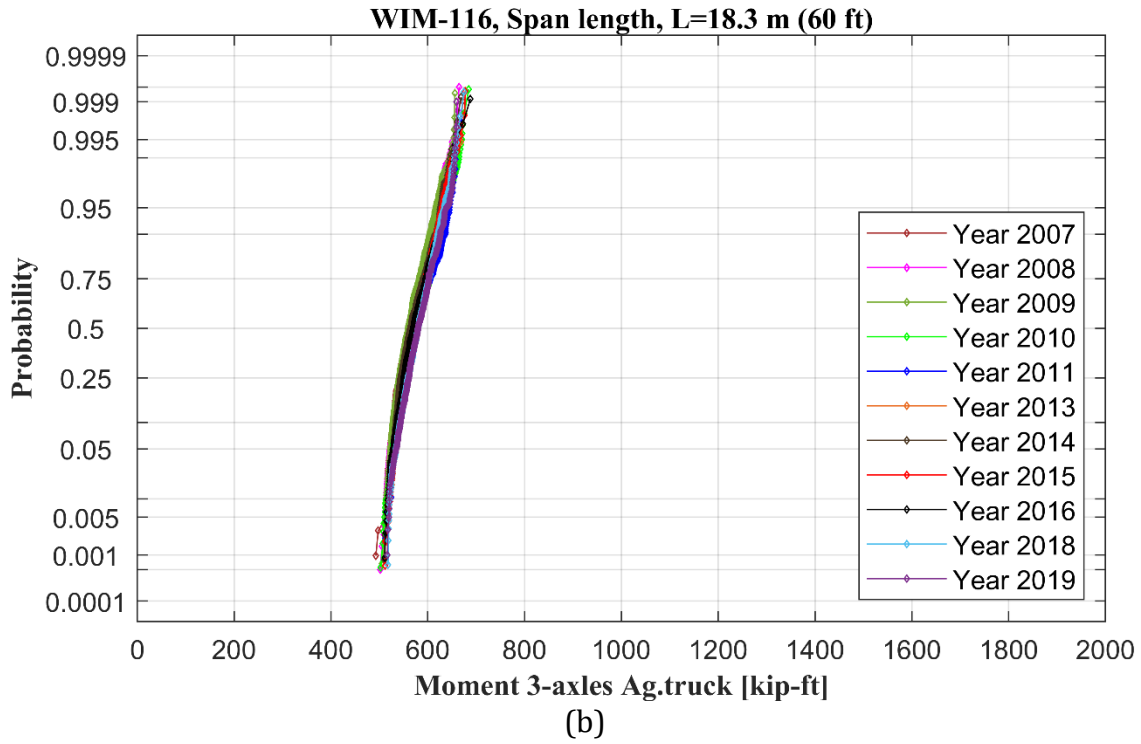


(e)

Figure 4-A.1. CDF of moment for 3-axes agricultural vehicles for span length of 9.1 m (30 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.



(a)



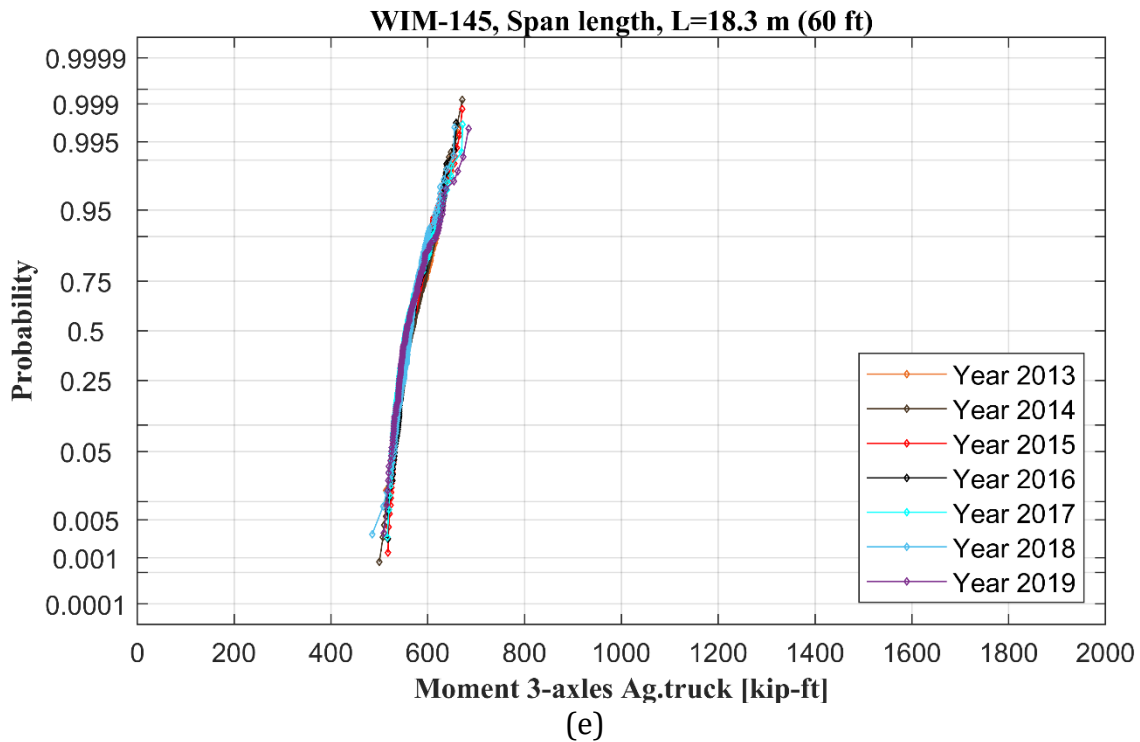
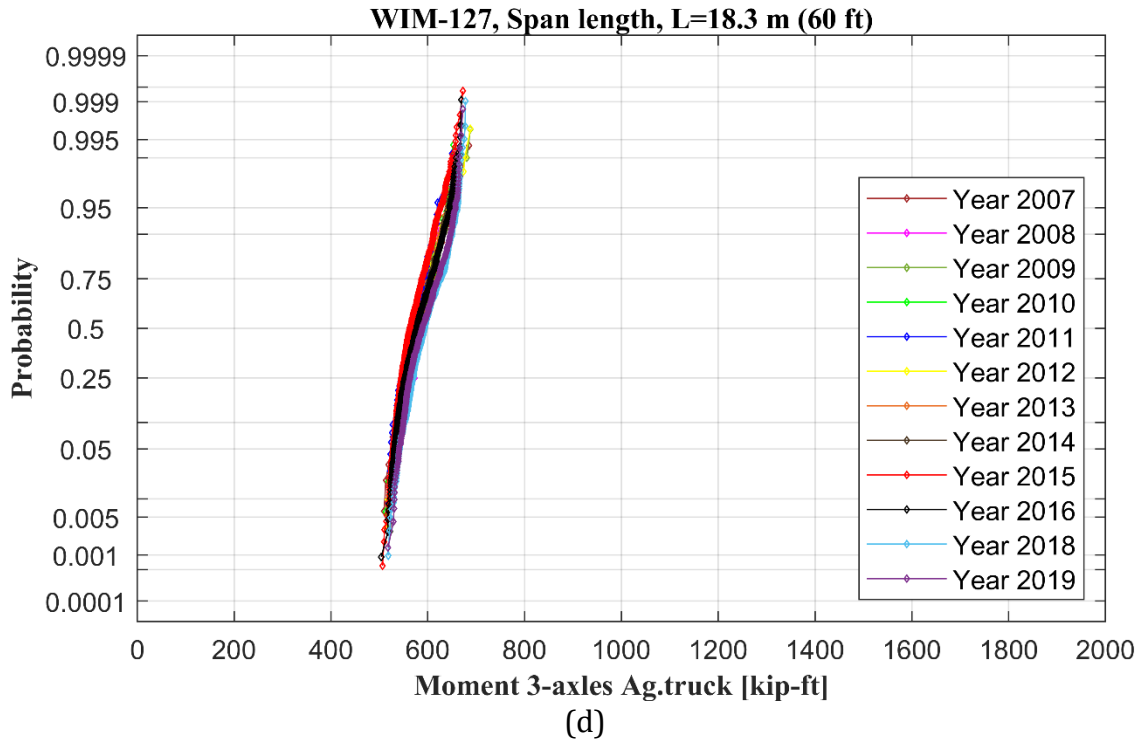
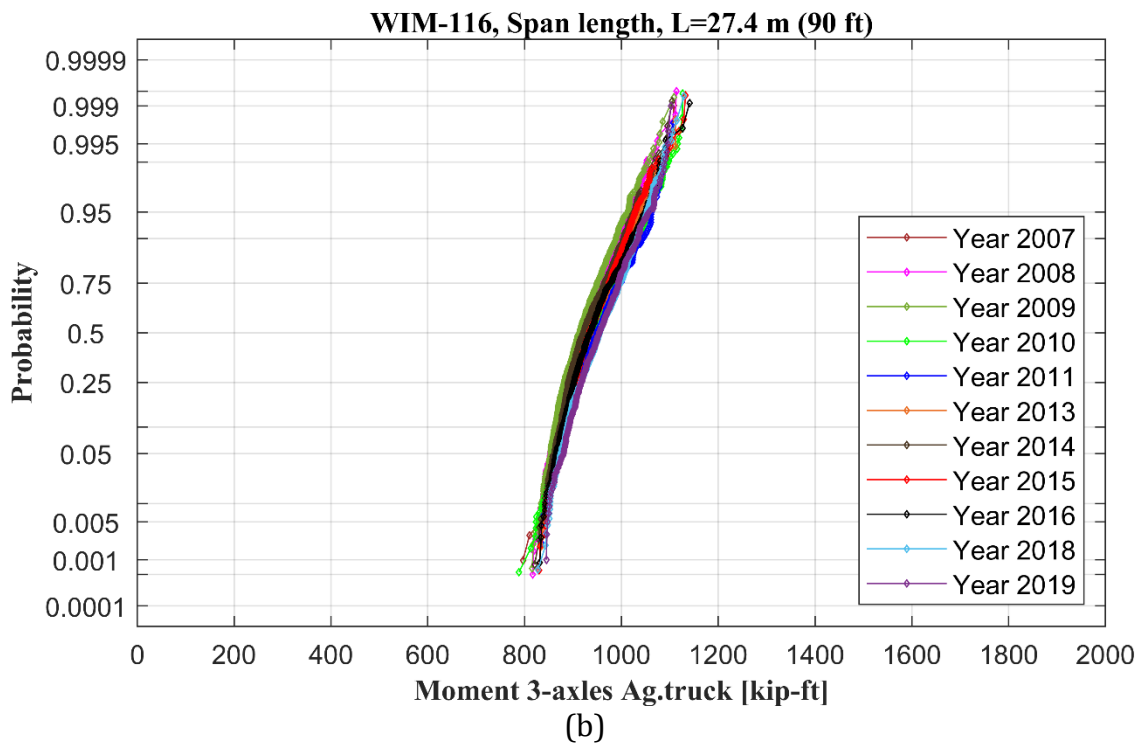
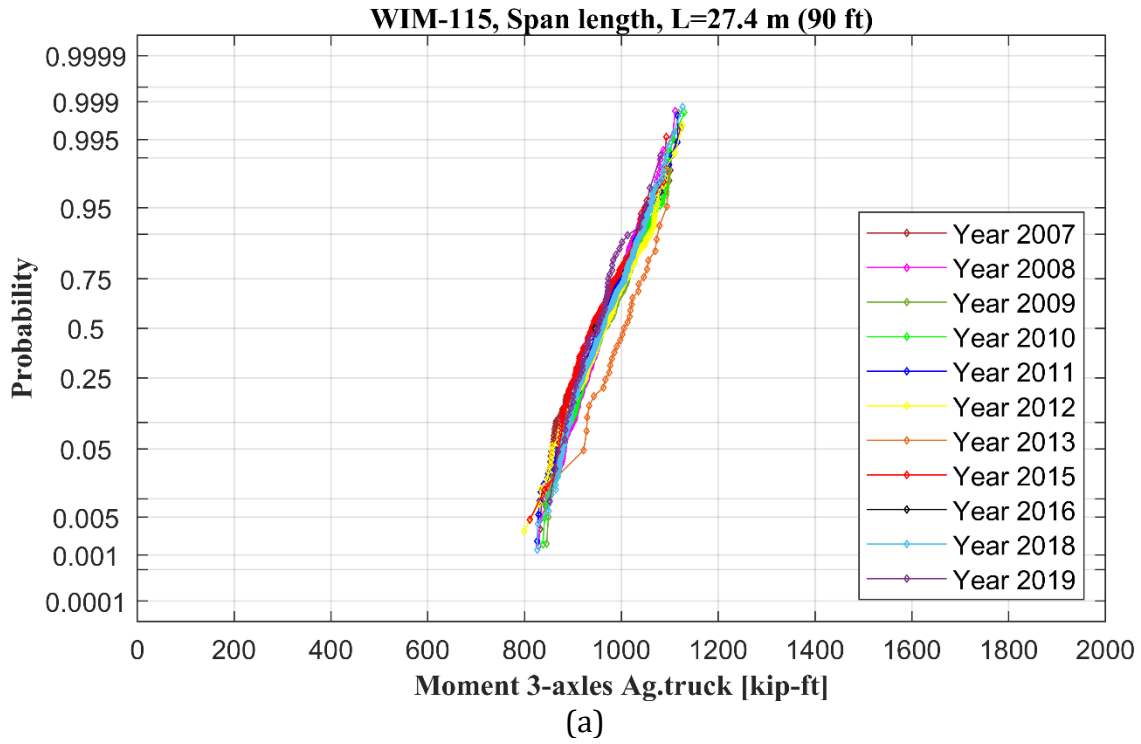
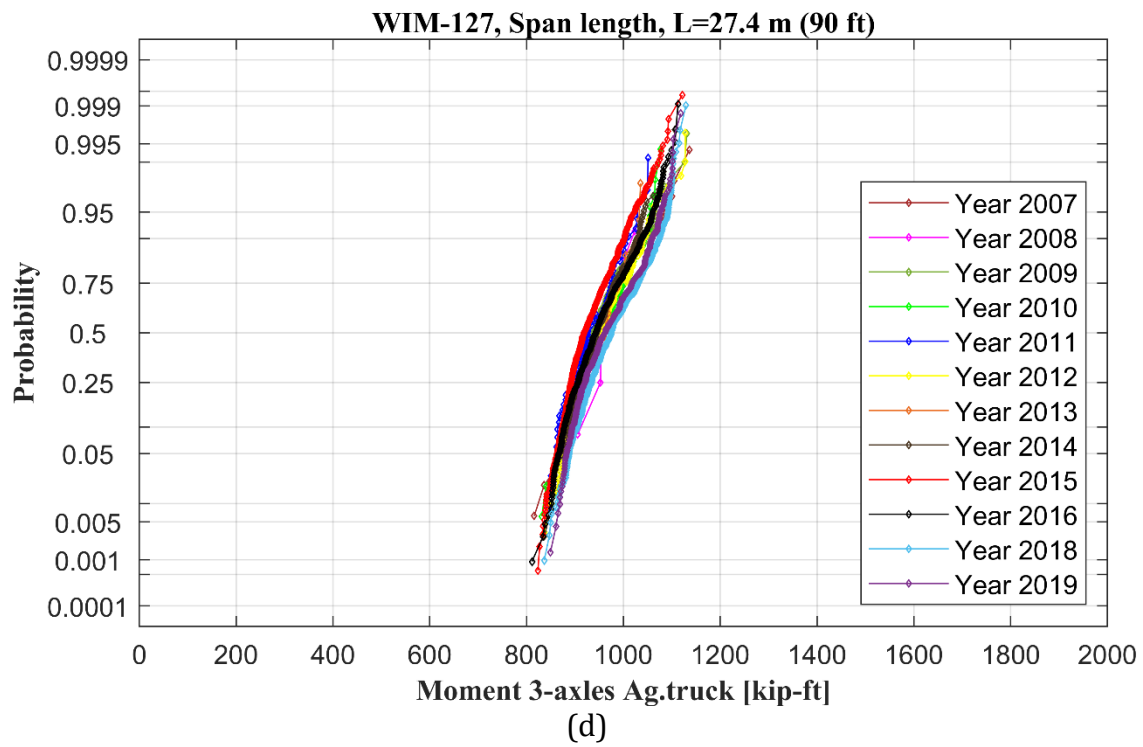
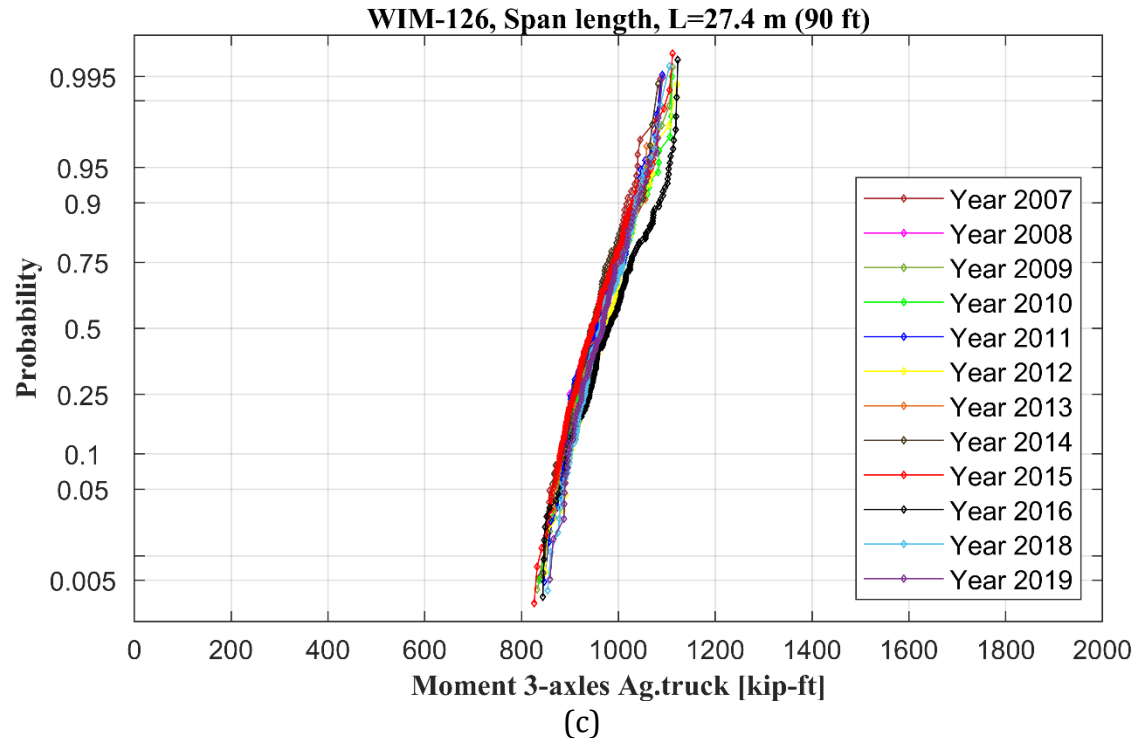
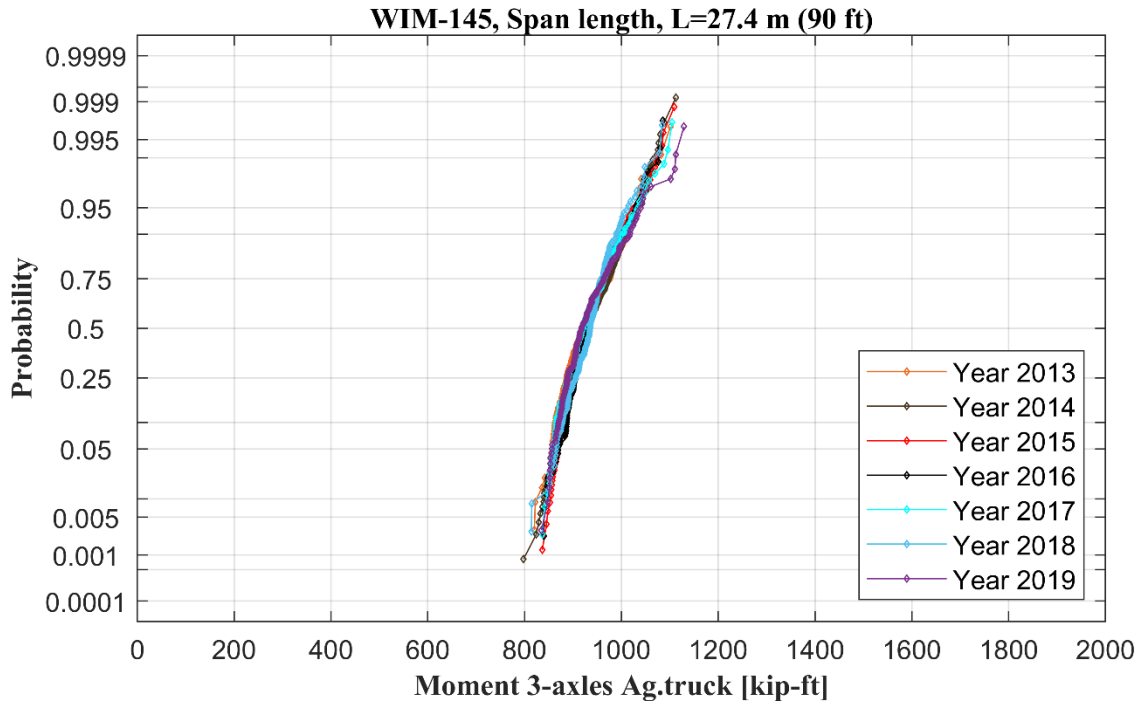


Figure 4-A.2. CDF of moment for 3-axes agricultural vehicles for span length 18.3 m (60 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

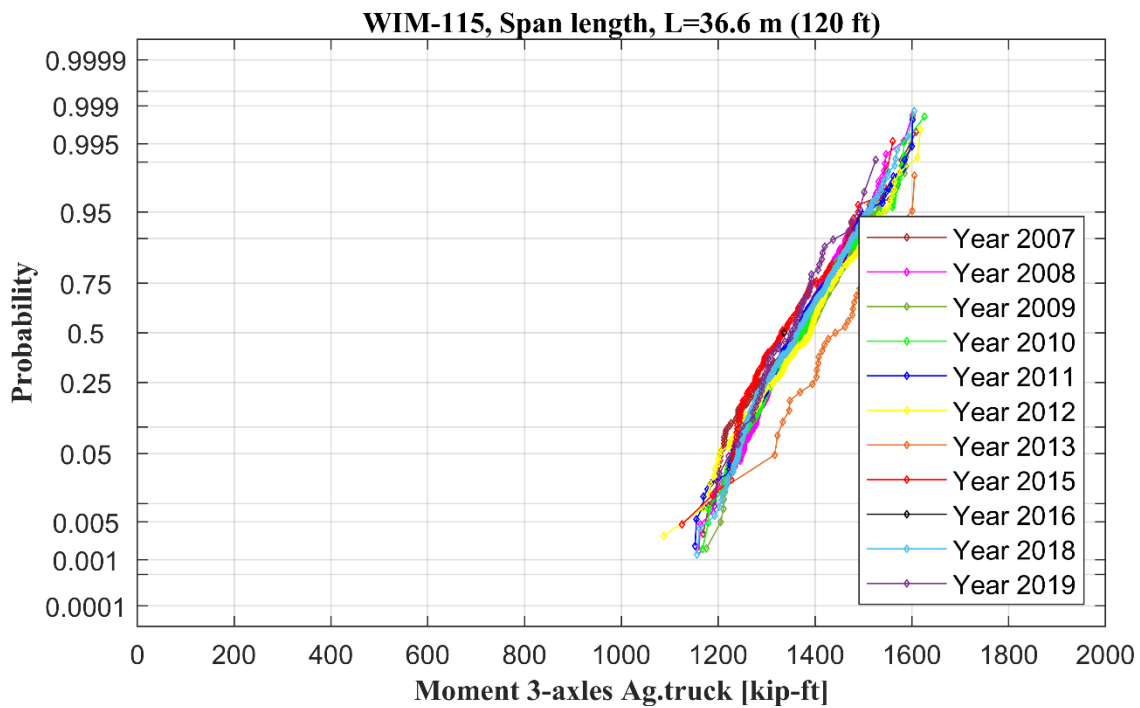




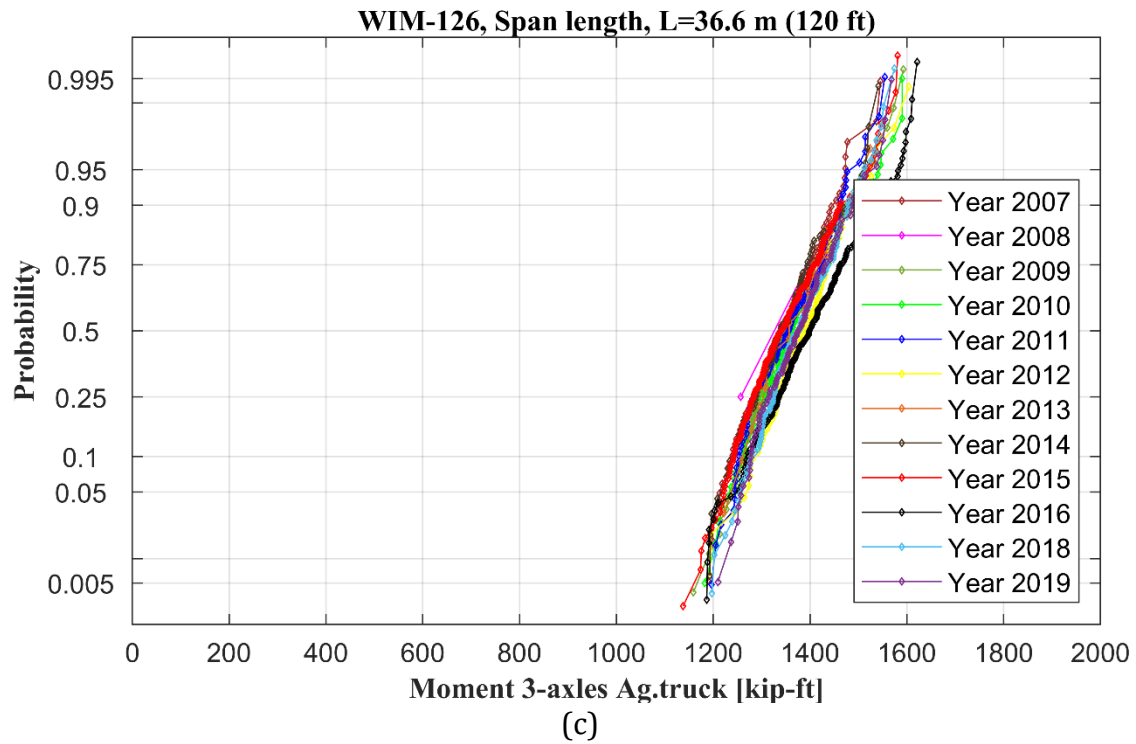
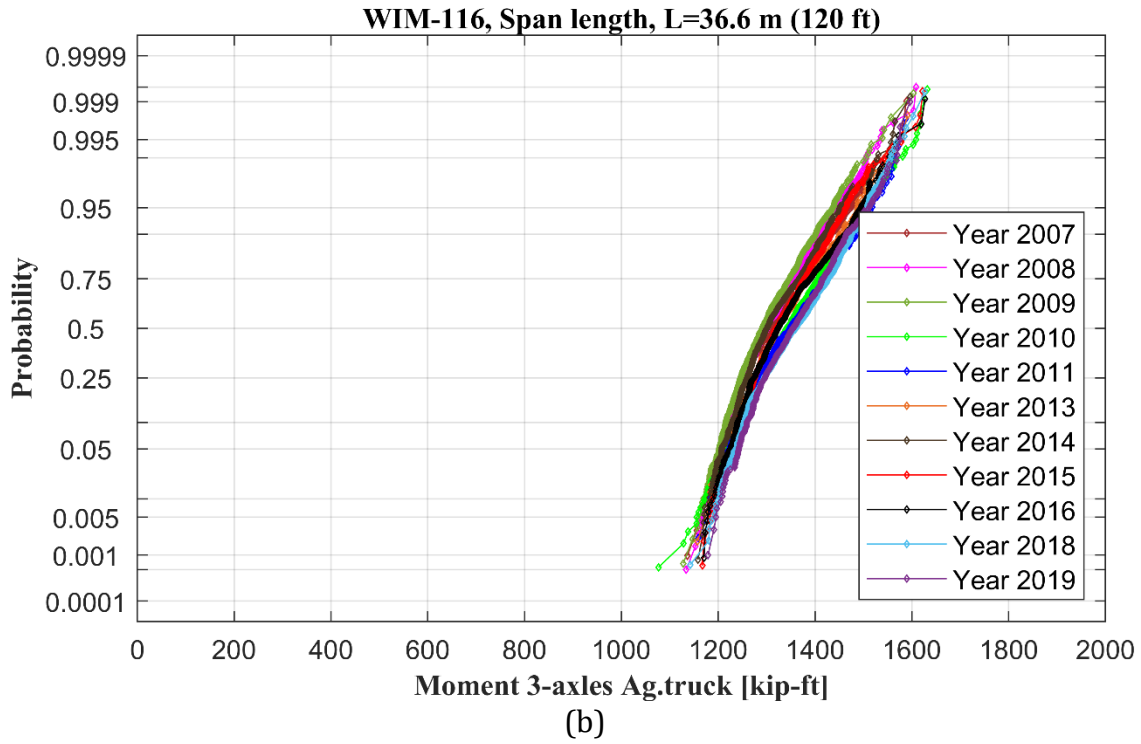


(e)

Figure 4-A.3. CDF of moment for 3-axes agricultural vehicles for span length 27.4 m (90 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.



(a)



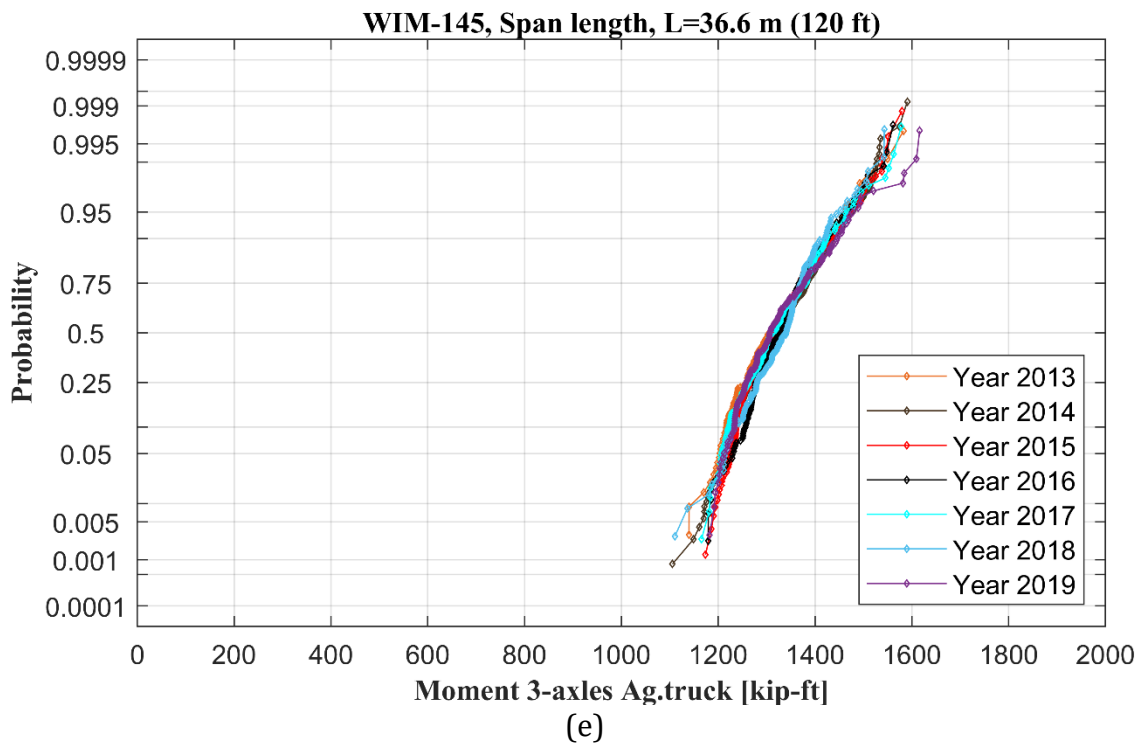
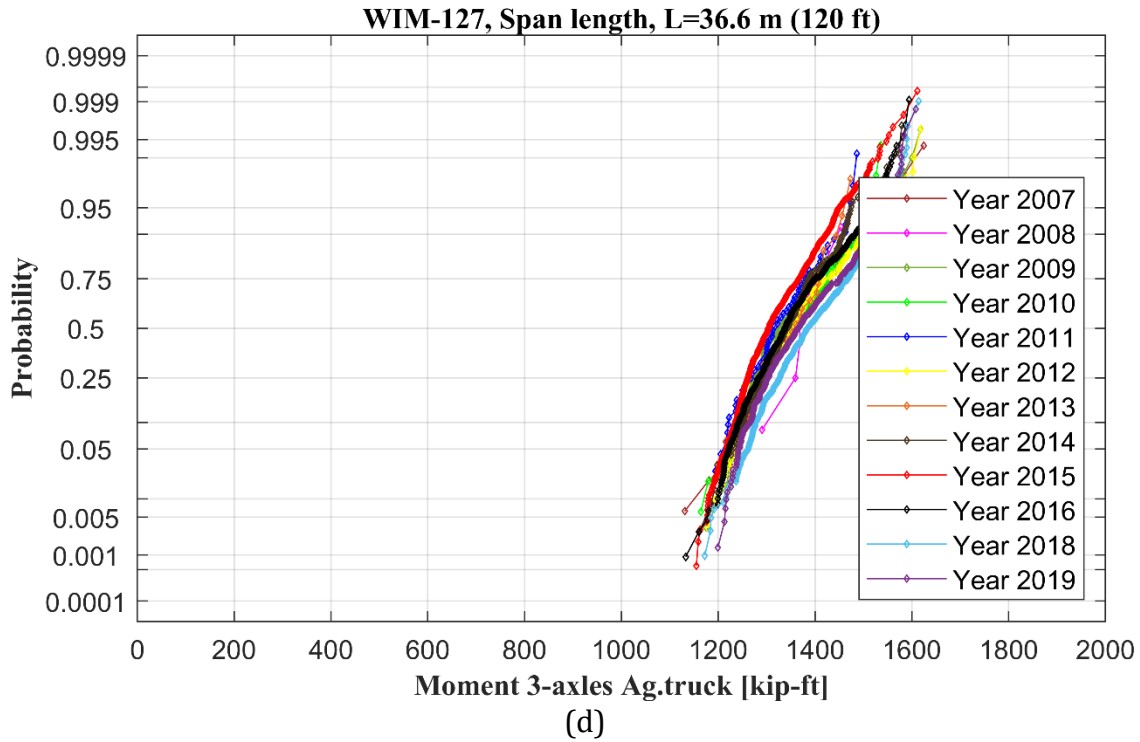
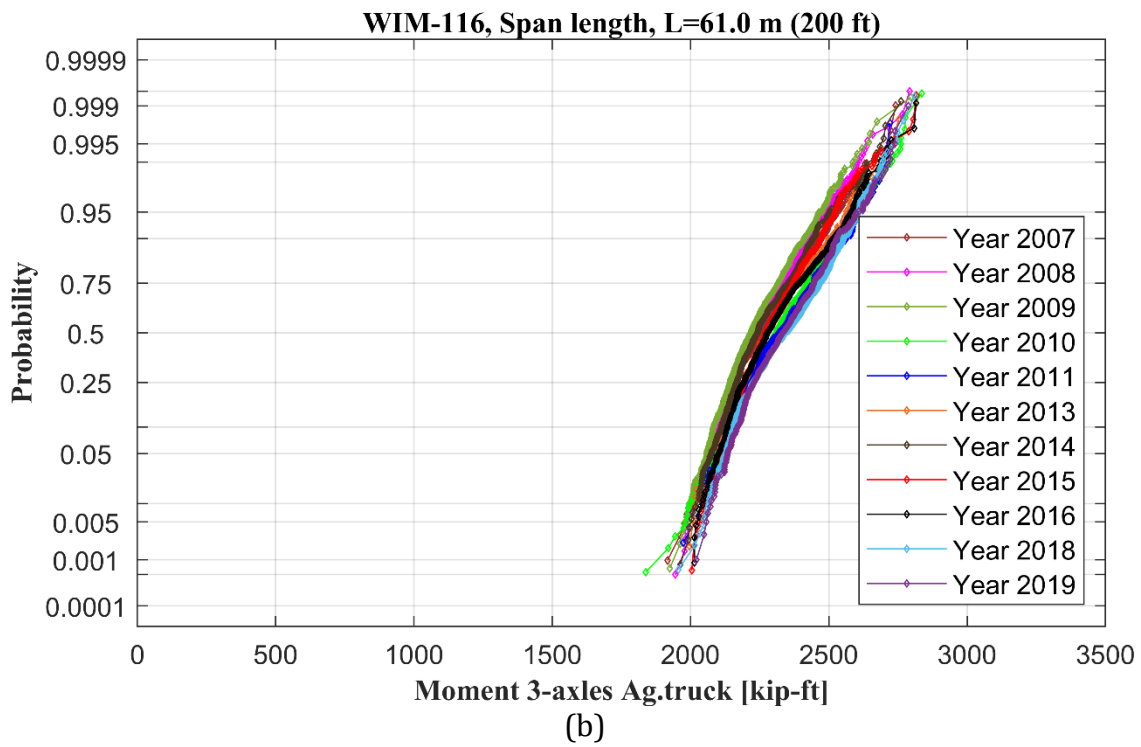
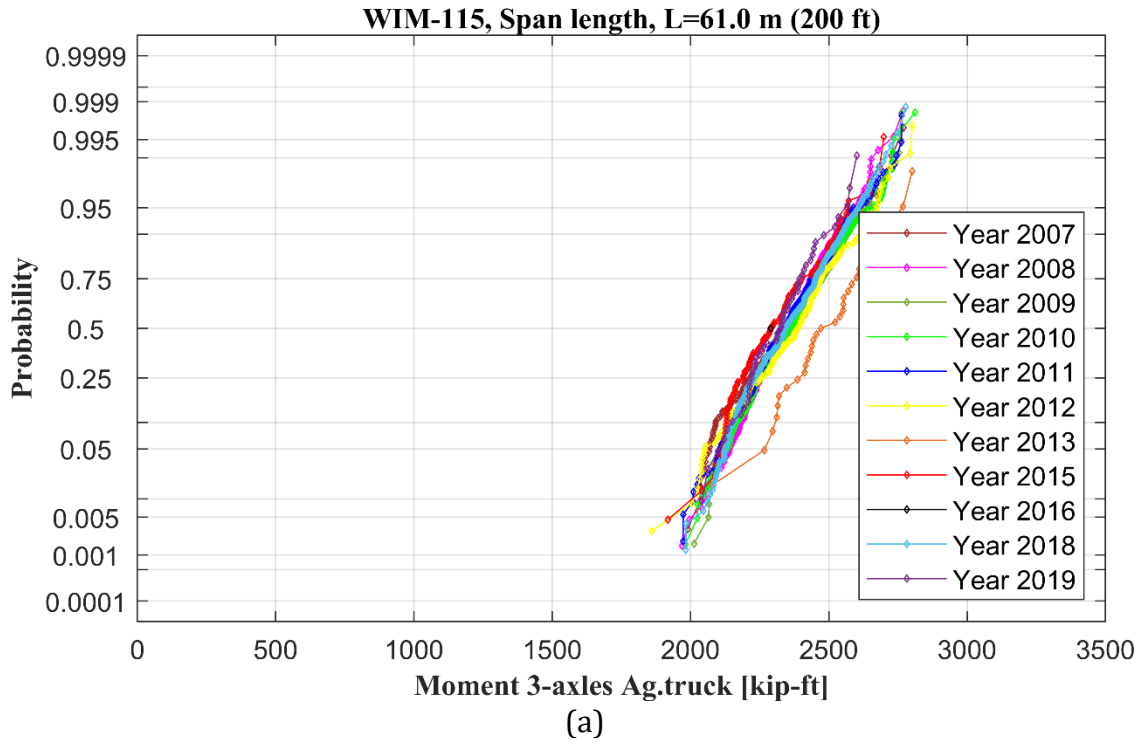
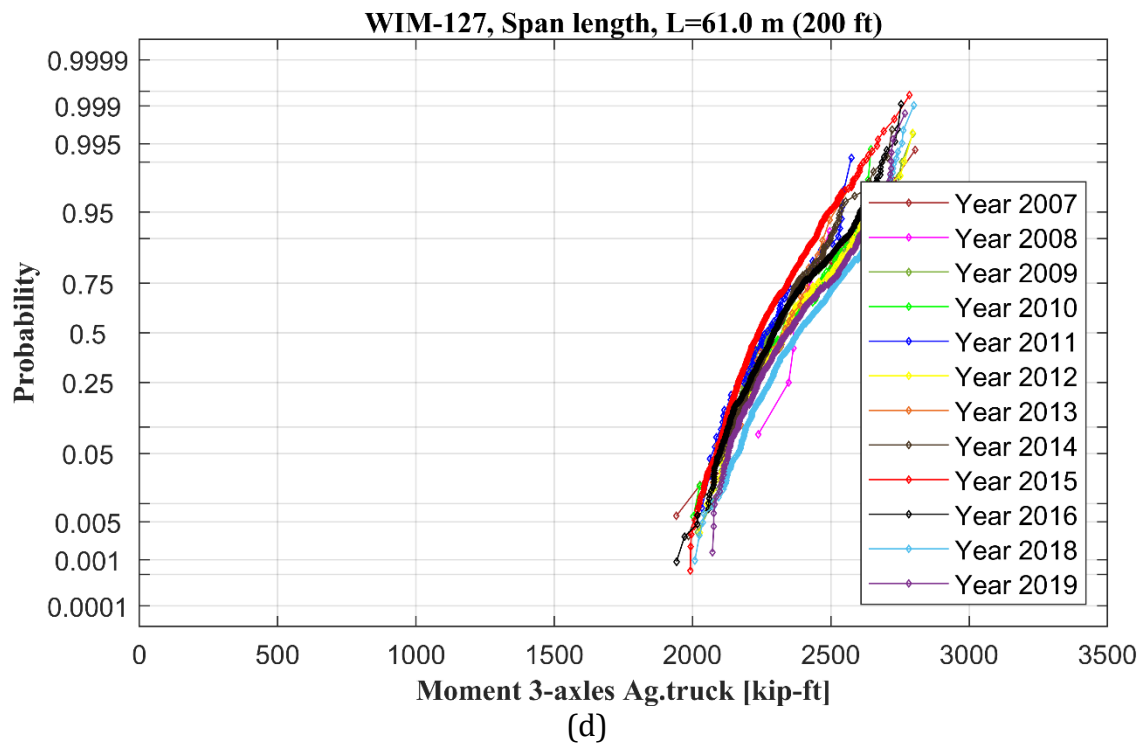
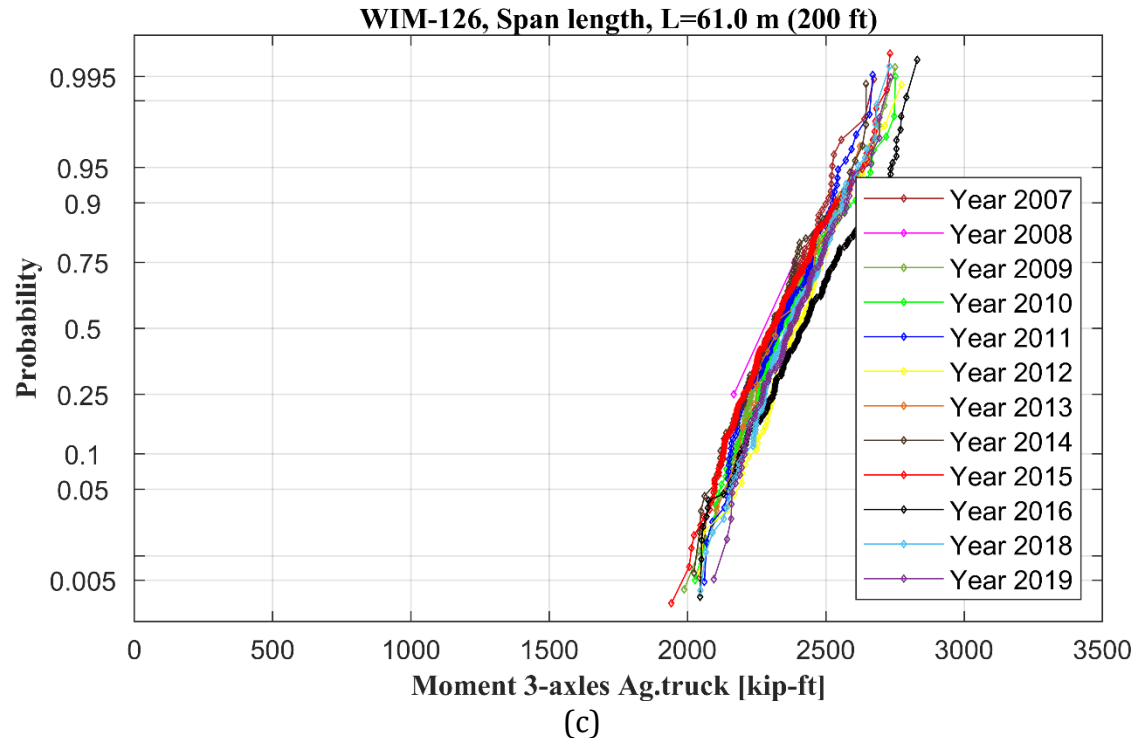


Figure 4-A.4. CDF of moment for 3-axes agricultural vehicles for span length 36.6 m (120 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





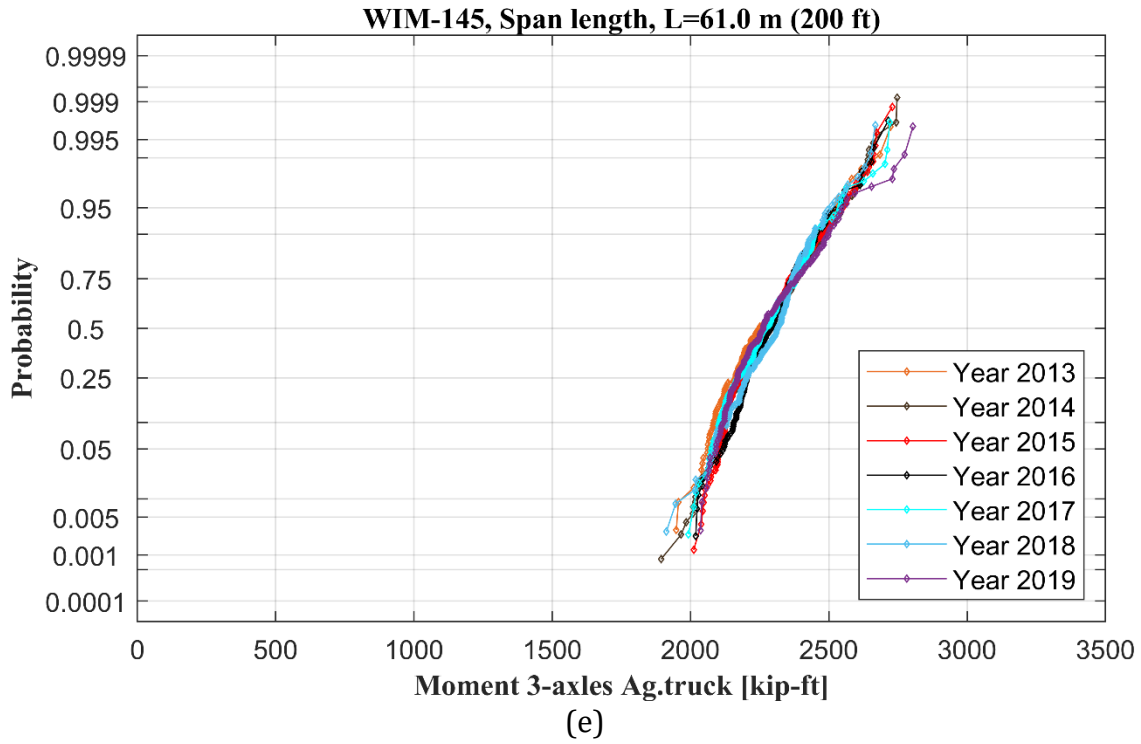
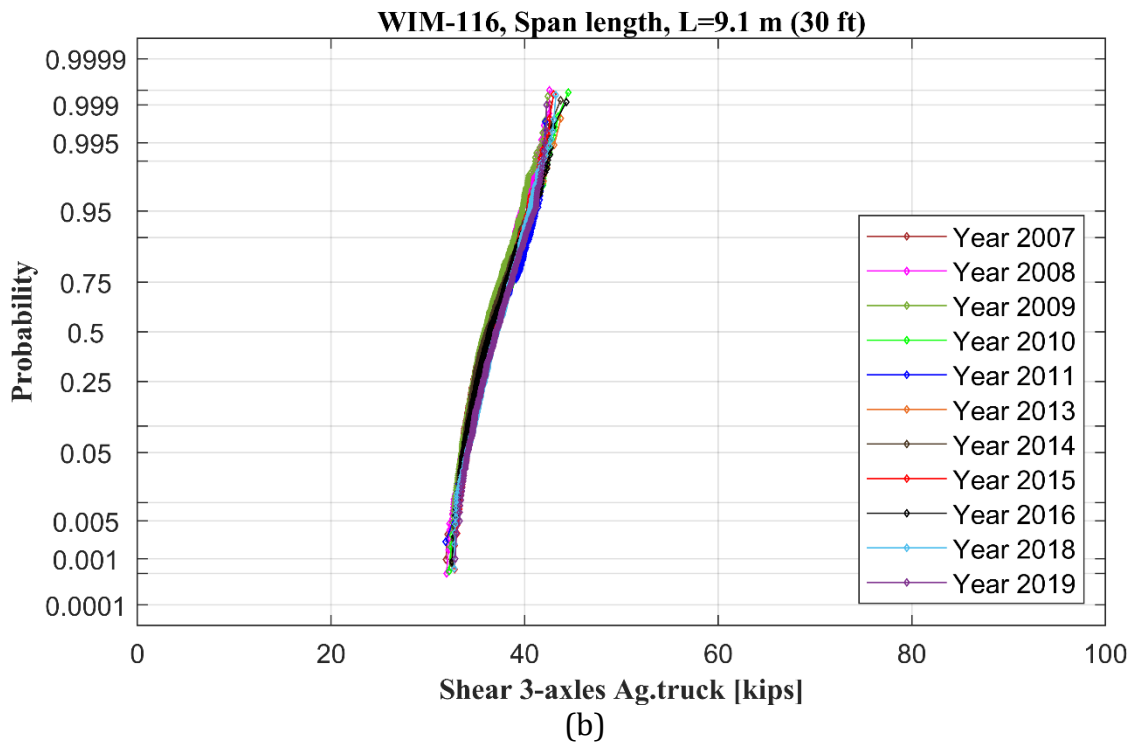
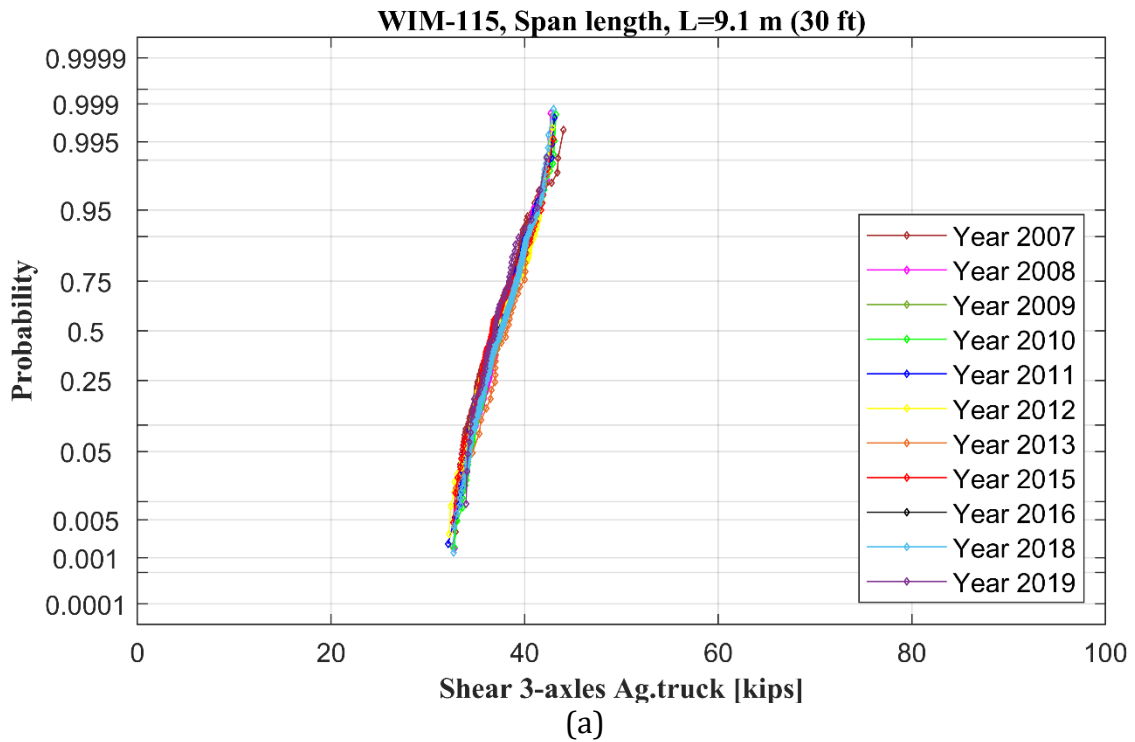
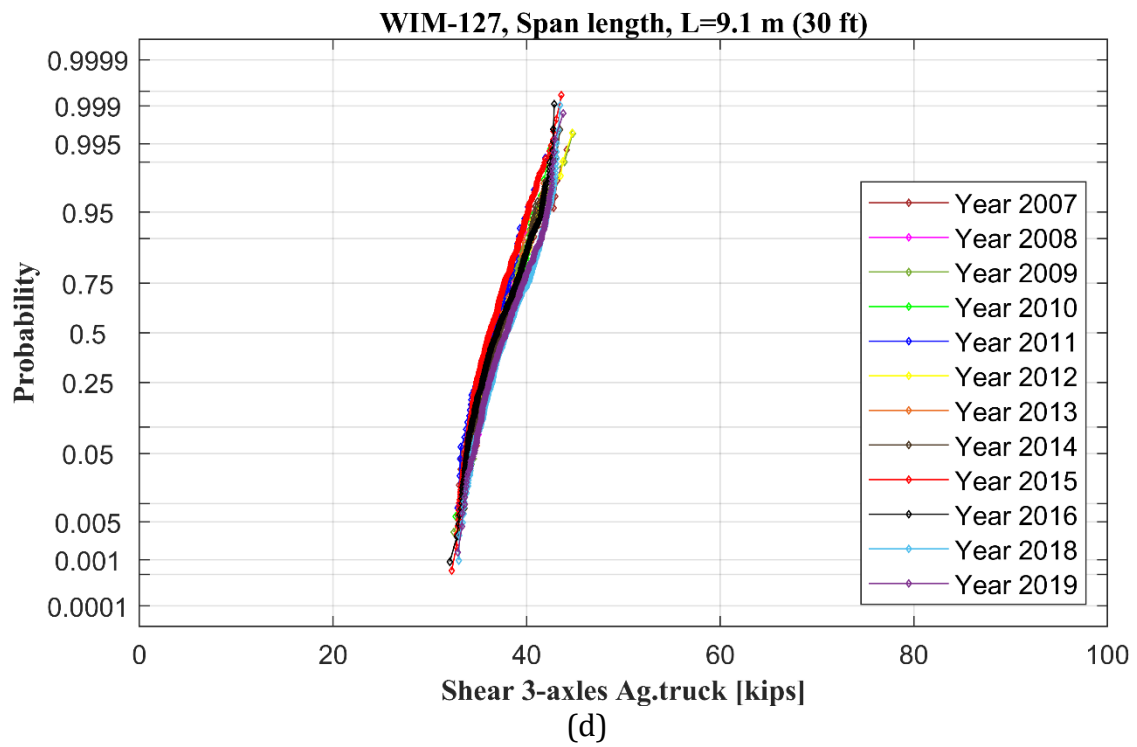
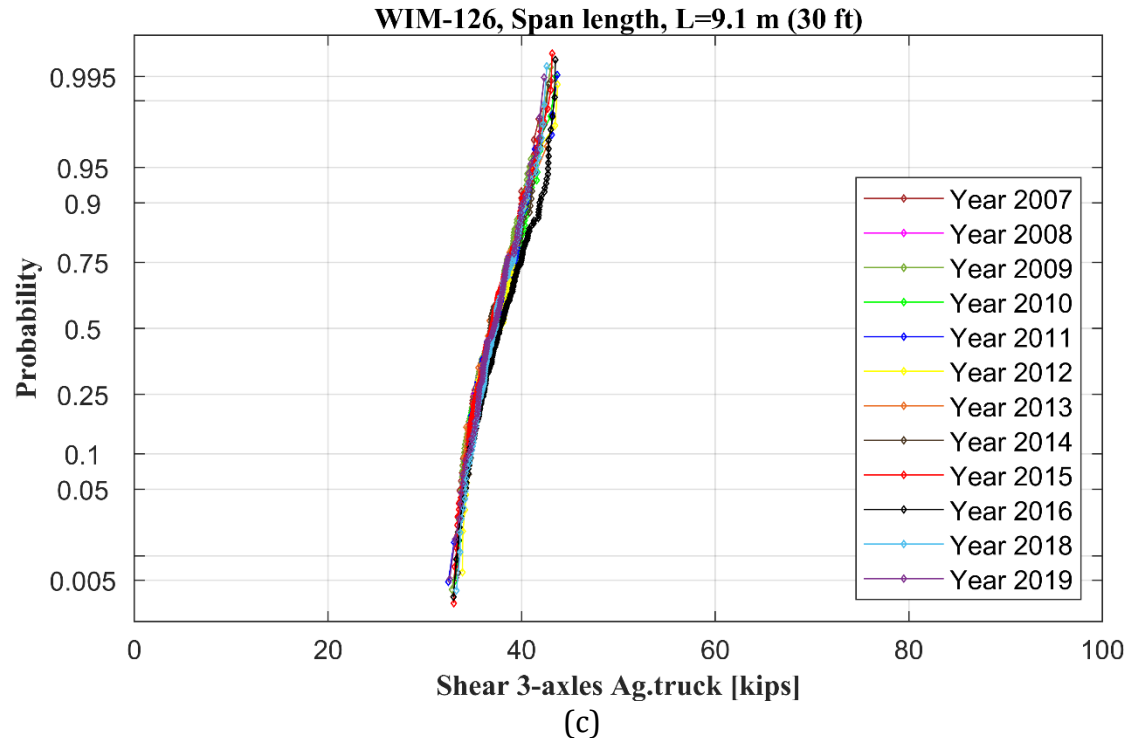
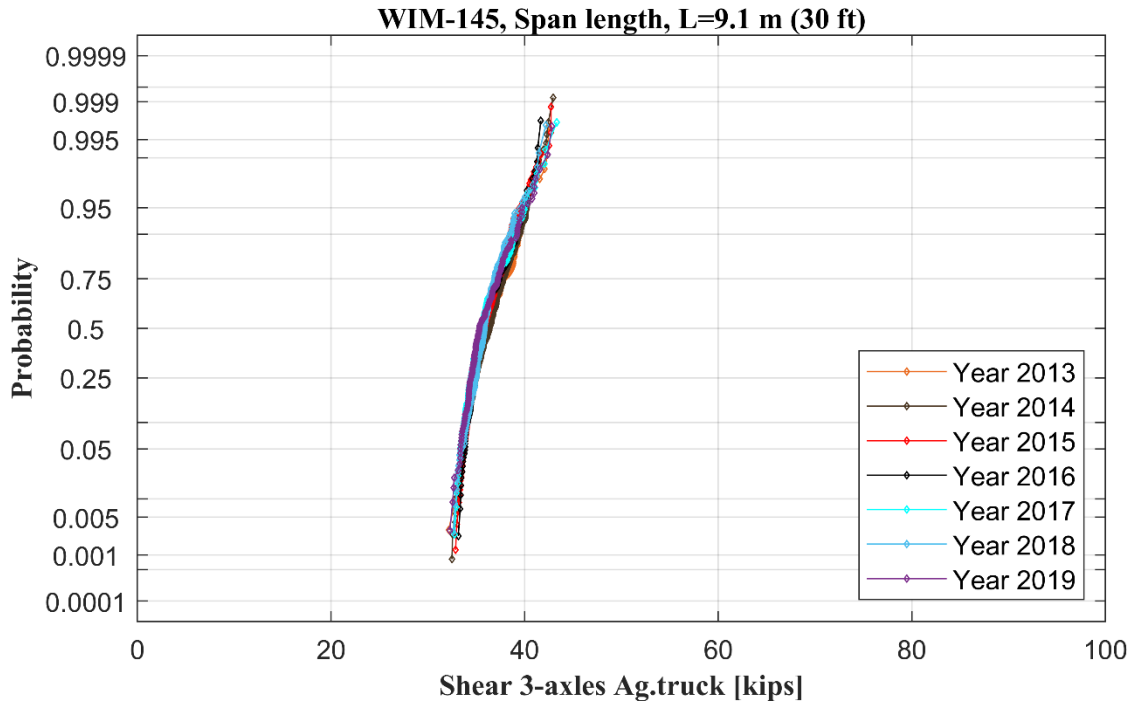


Figure 4-A.5. CDF of moment for 3-axes agricultural vehicles for span length 61 m (200 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

Appendix 4-B. CDF of shear force effects for 3-axle agricultural vehicles

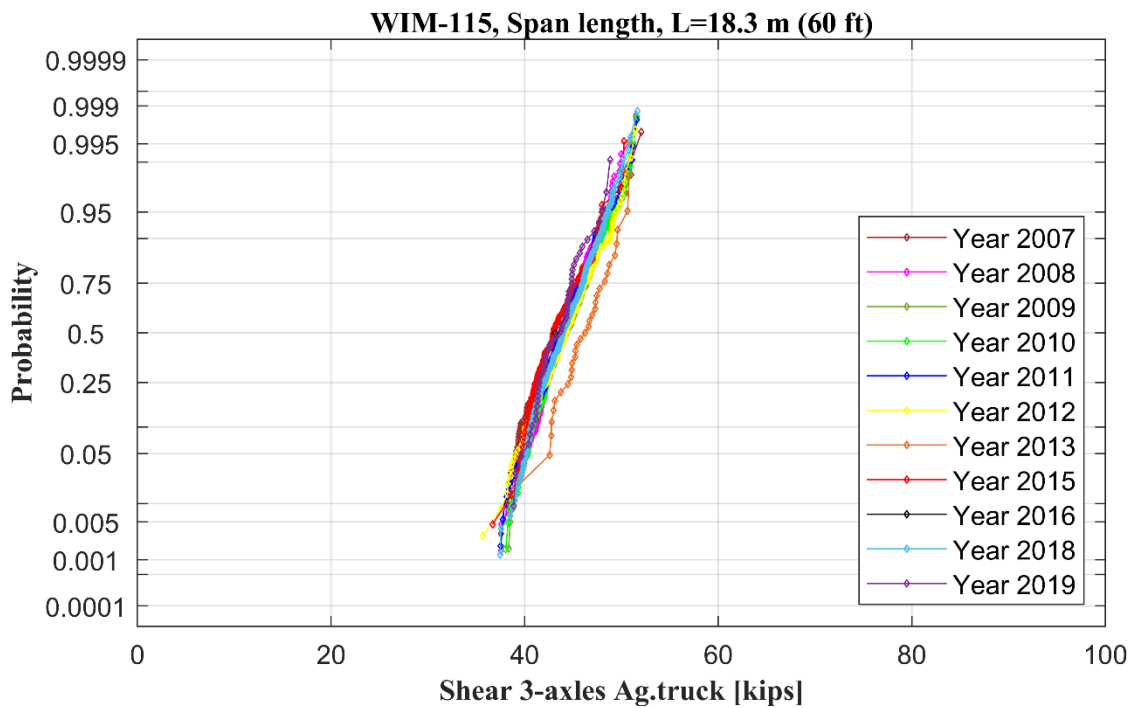




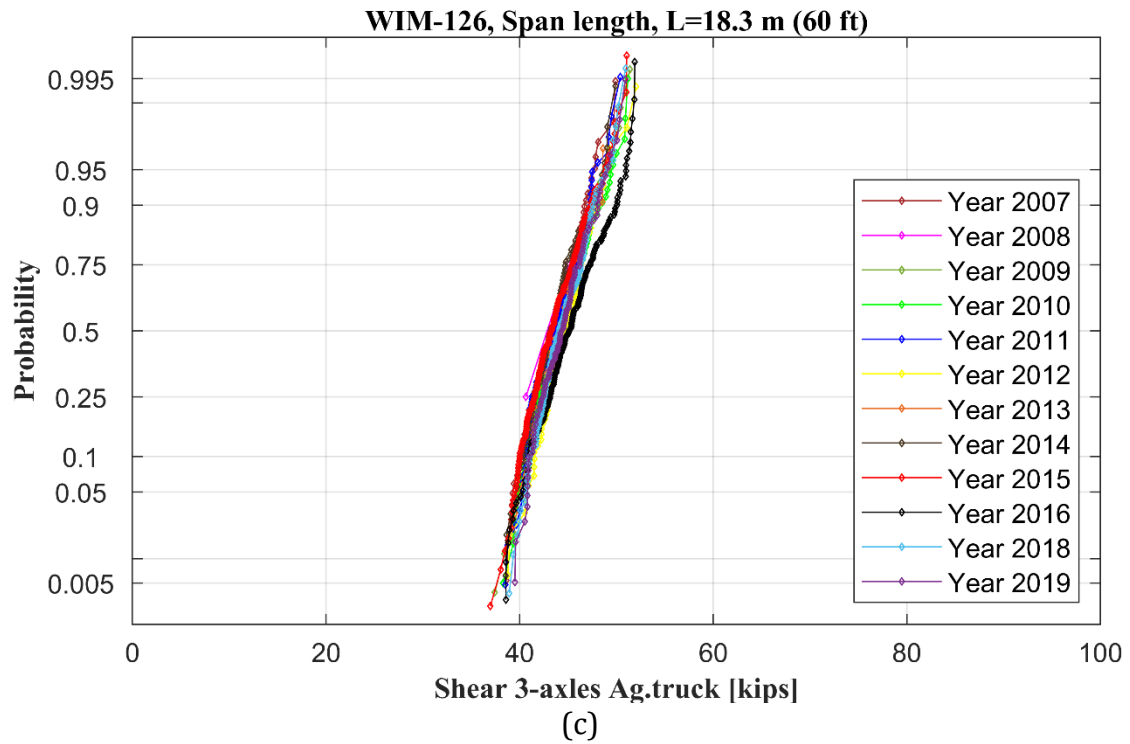
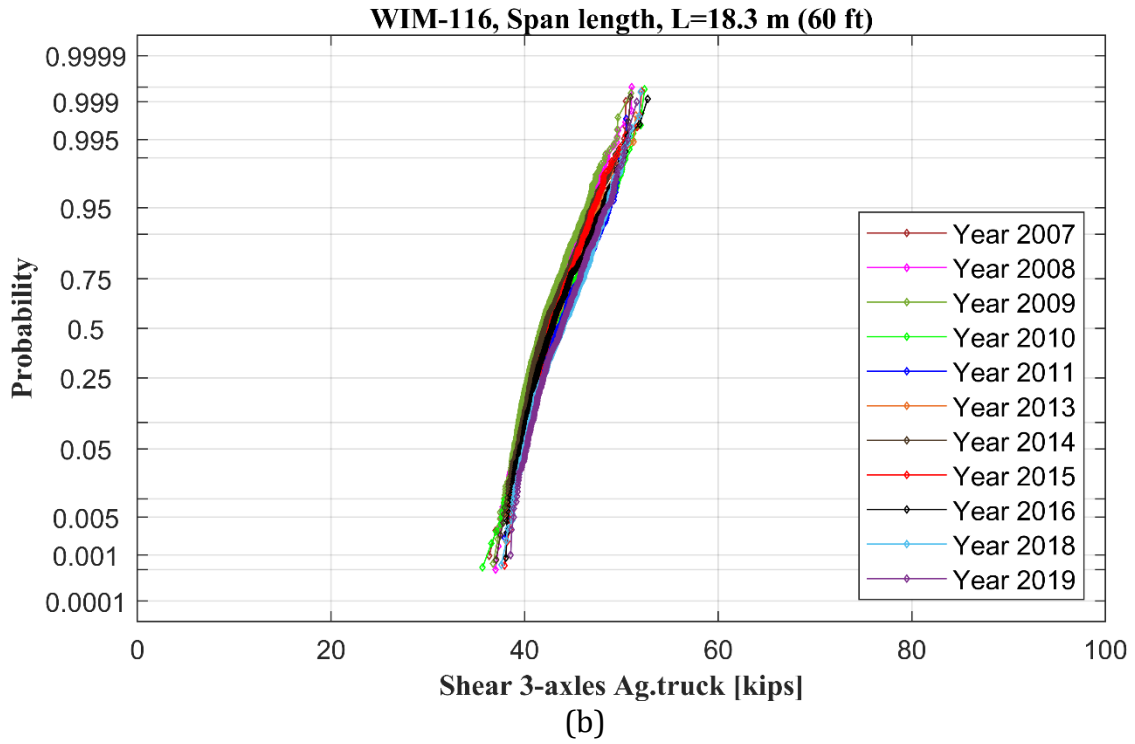


(e)

Figure 4-B.1. CDF of shear for 3-axes agricultural vehicles for span length of 9.1 m (30 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.



(a)



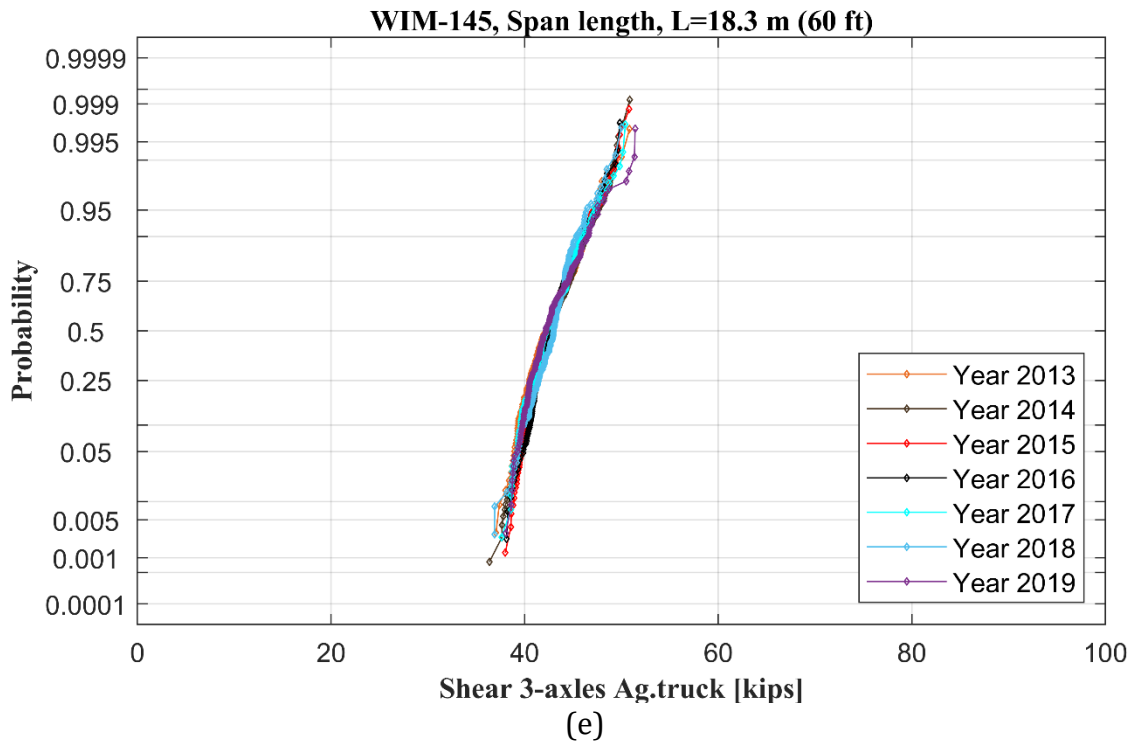
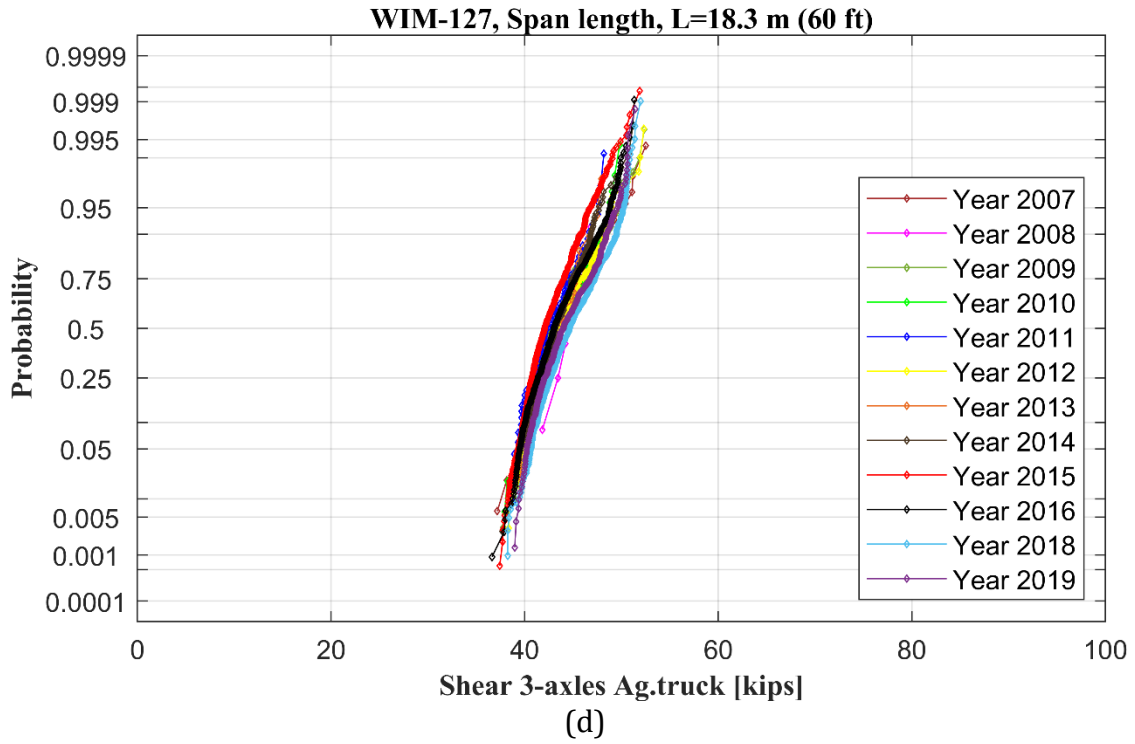
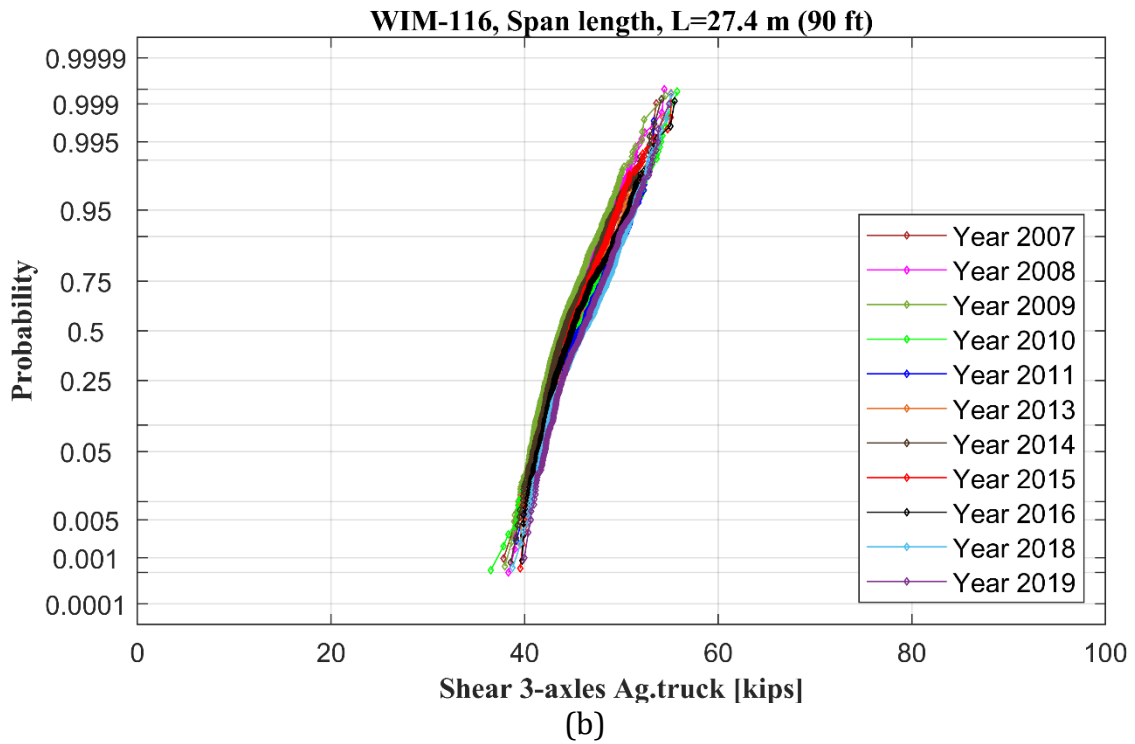
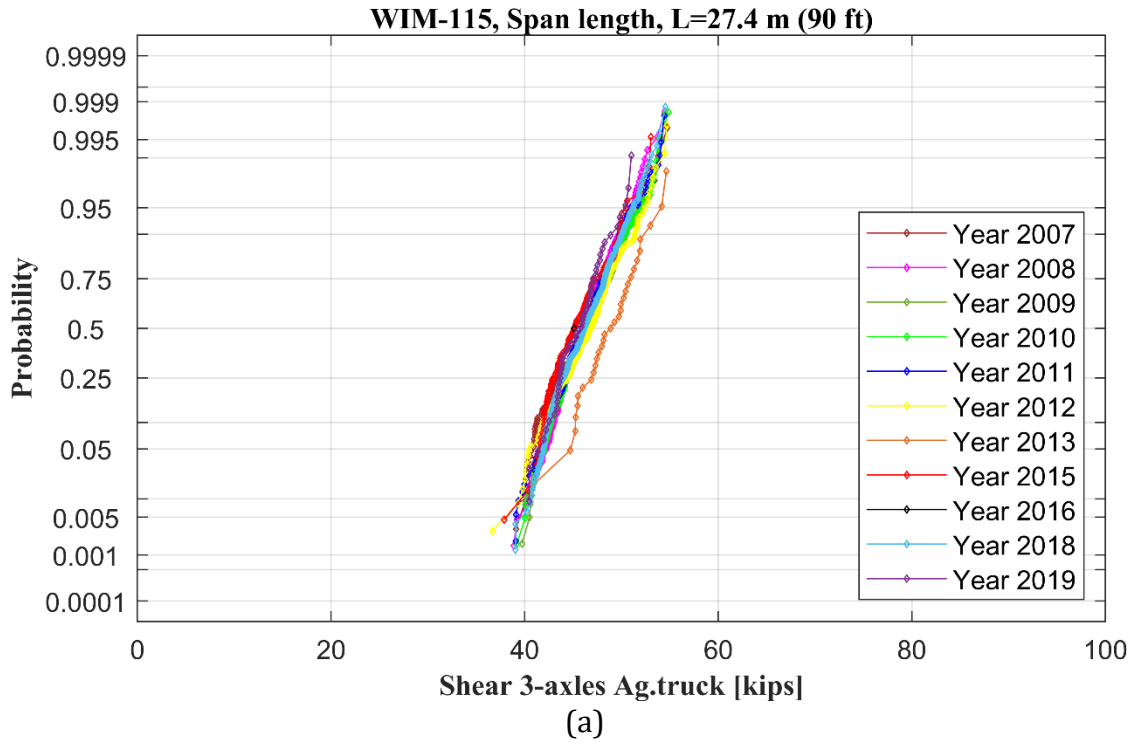
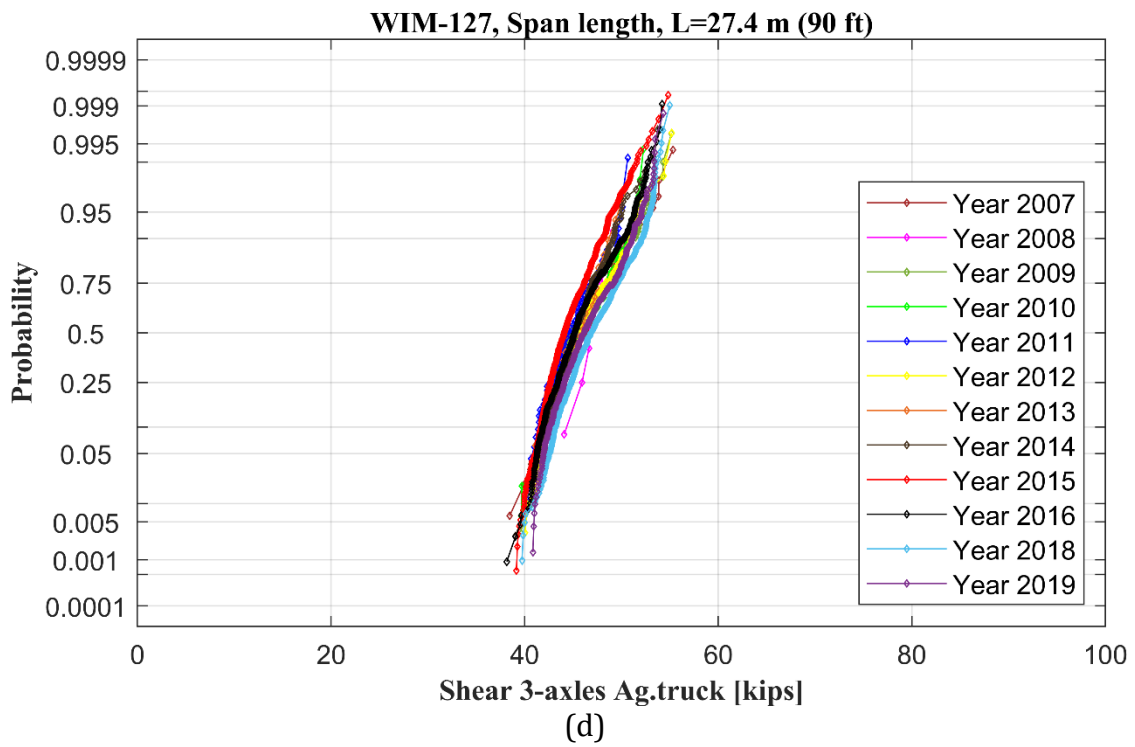
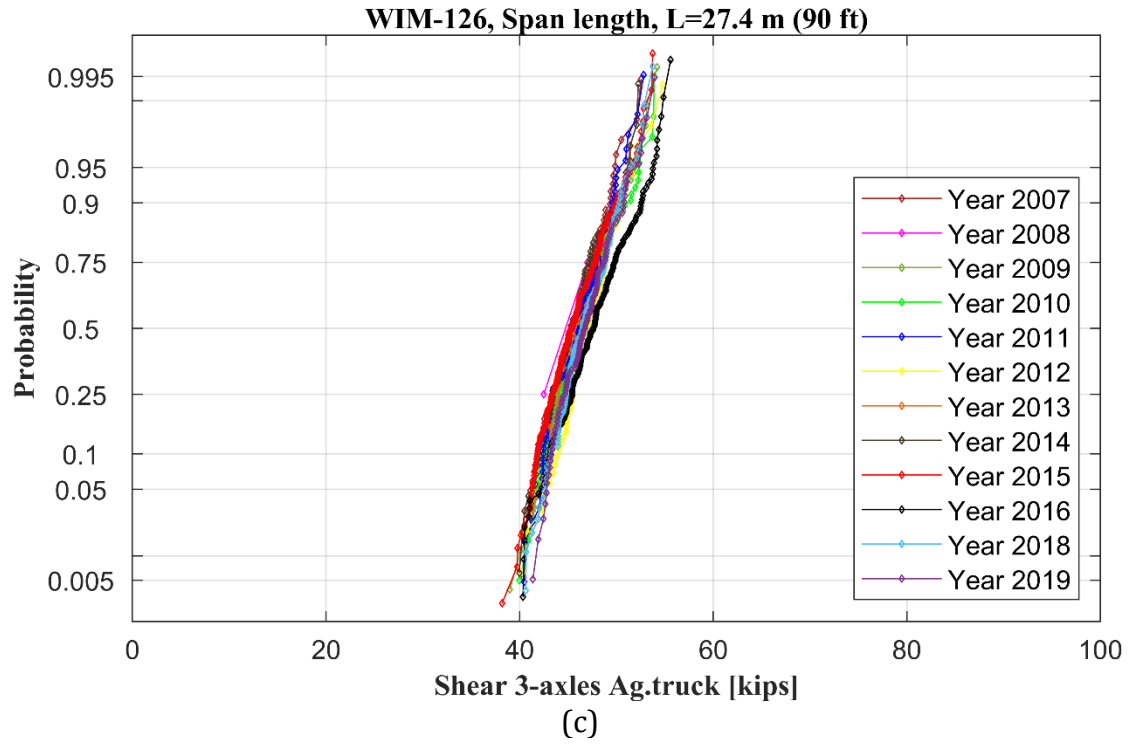
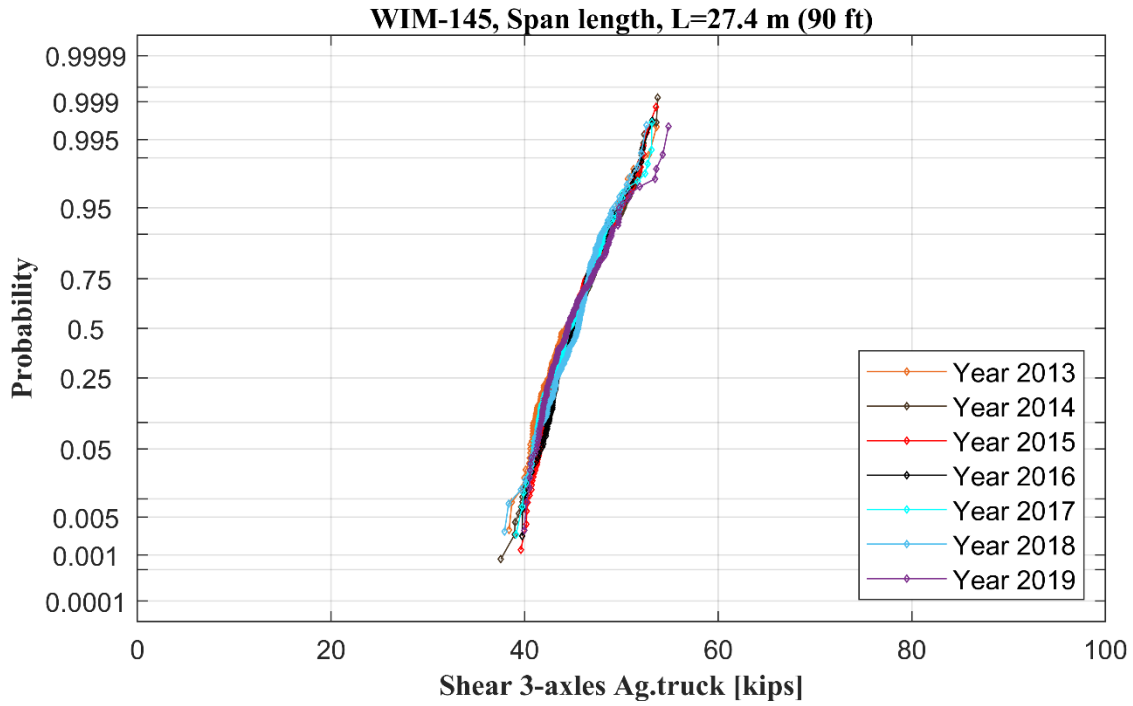


Figure 4-B.2. CDF of shear for 3-axes agricultural vehicles for span length 18.3 m (60 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

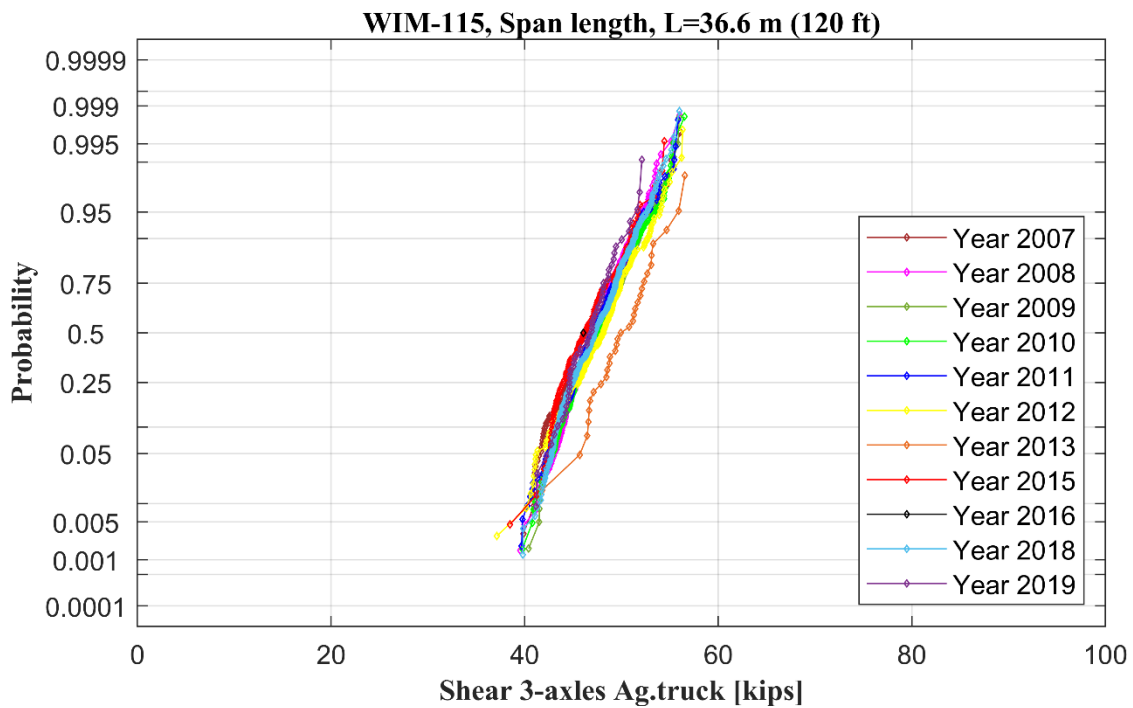




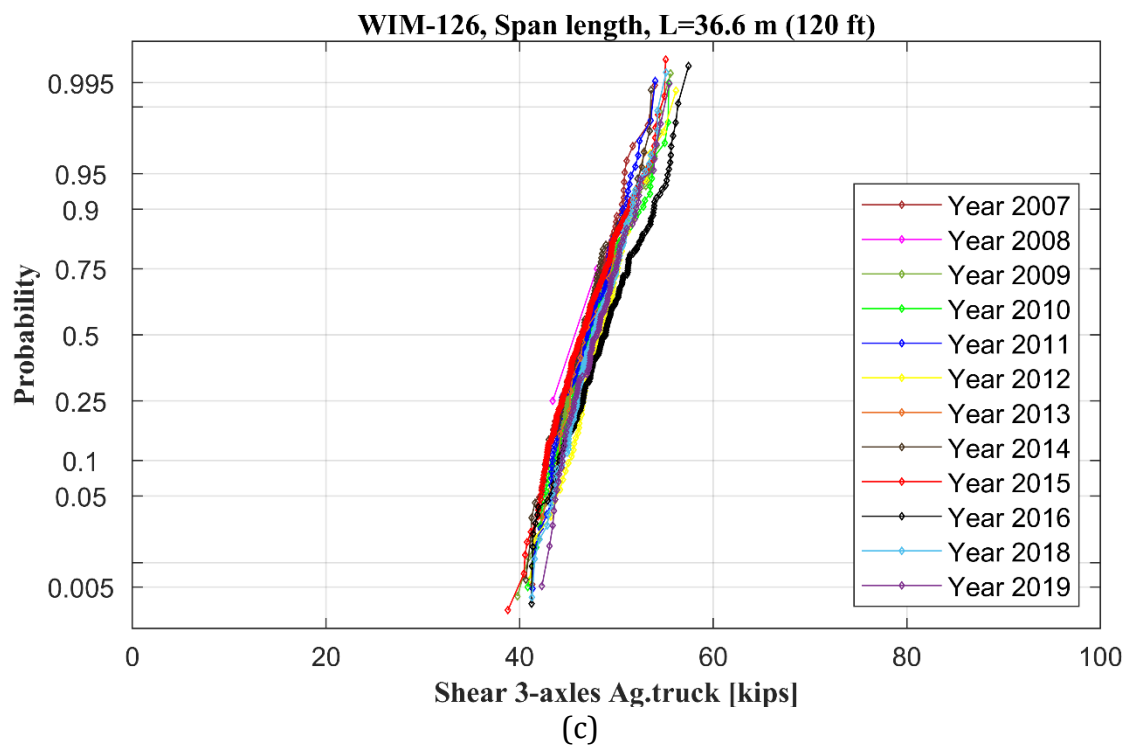
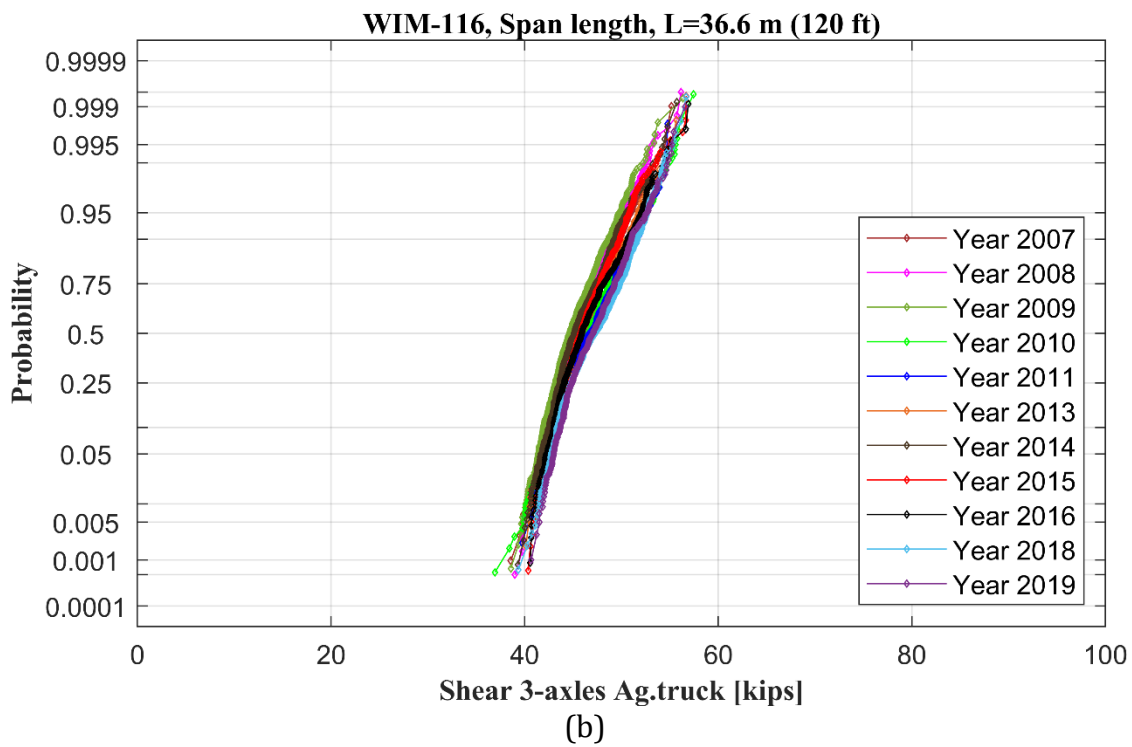


(e)

Figure 4-B.3. CDF of shear for 3-axes agricultural vehicles for span length 27.4 m (90 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.



(a)



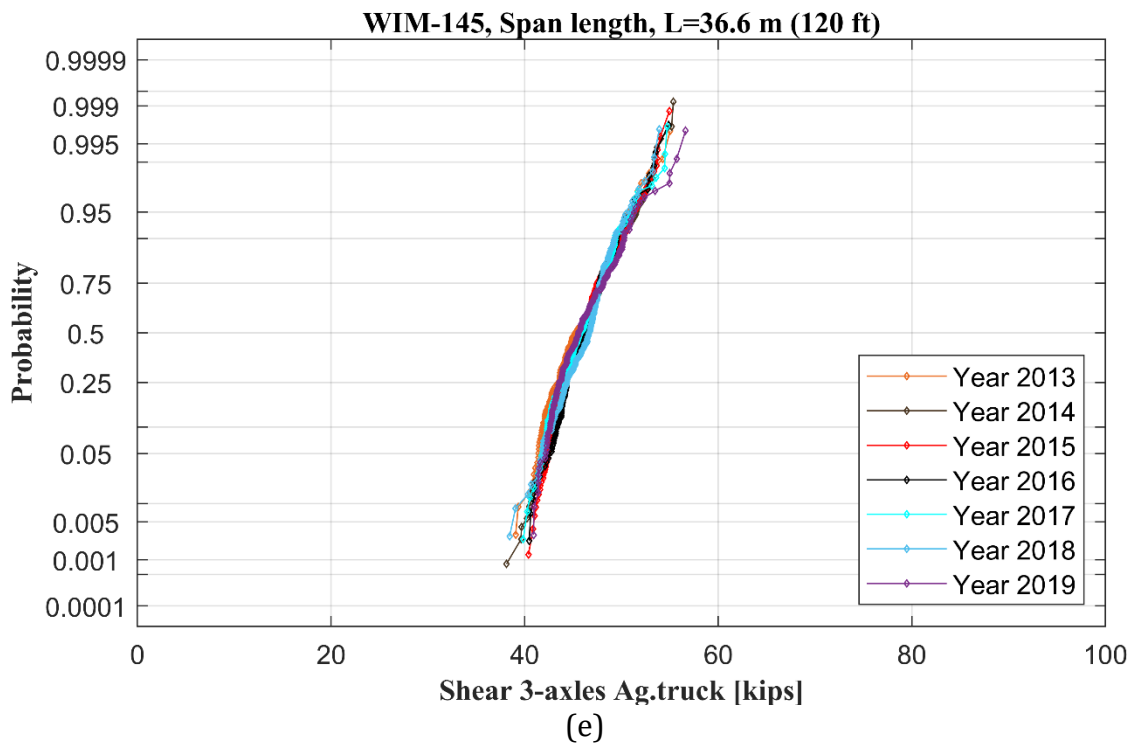
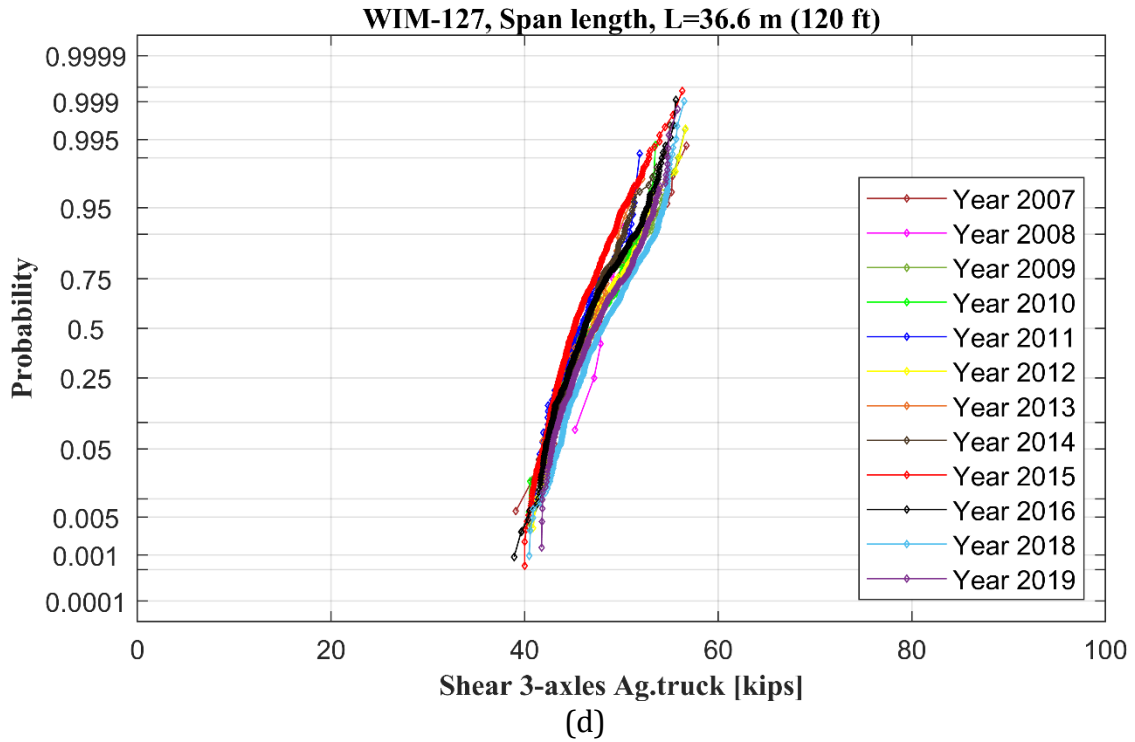
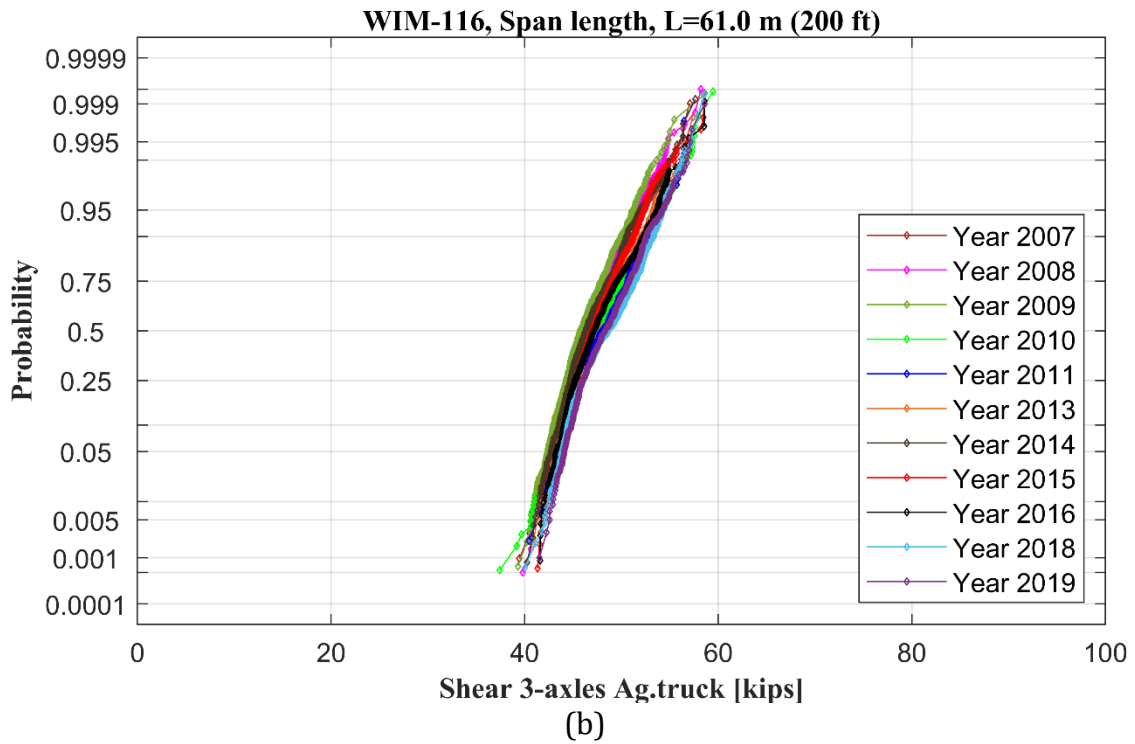
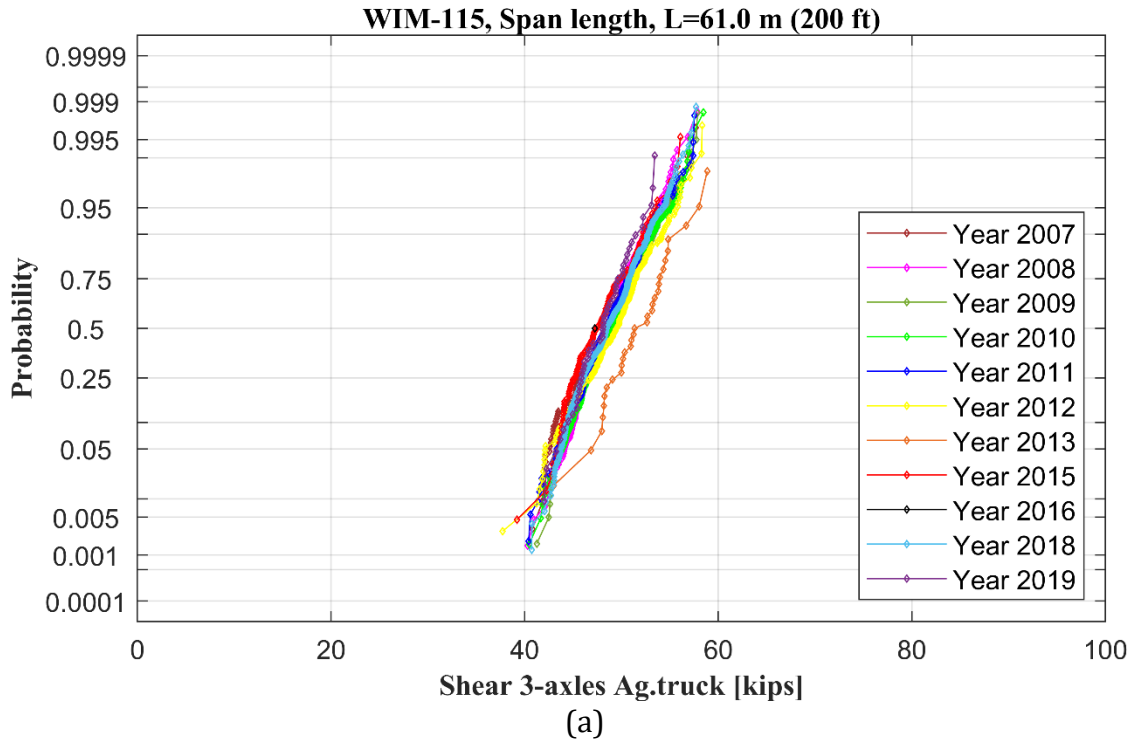
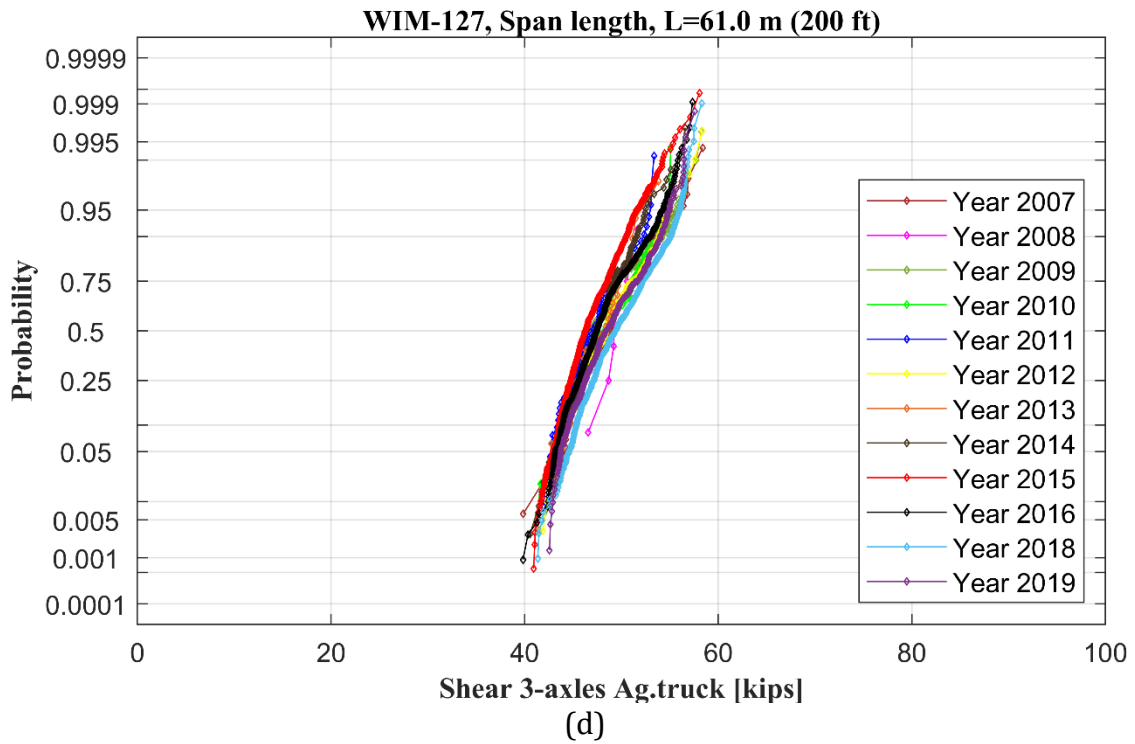
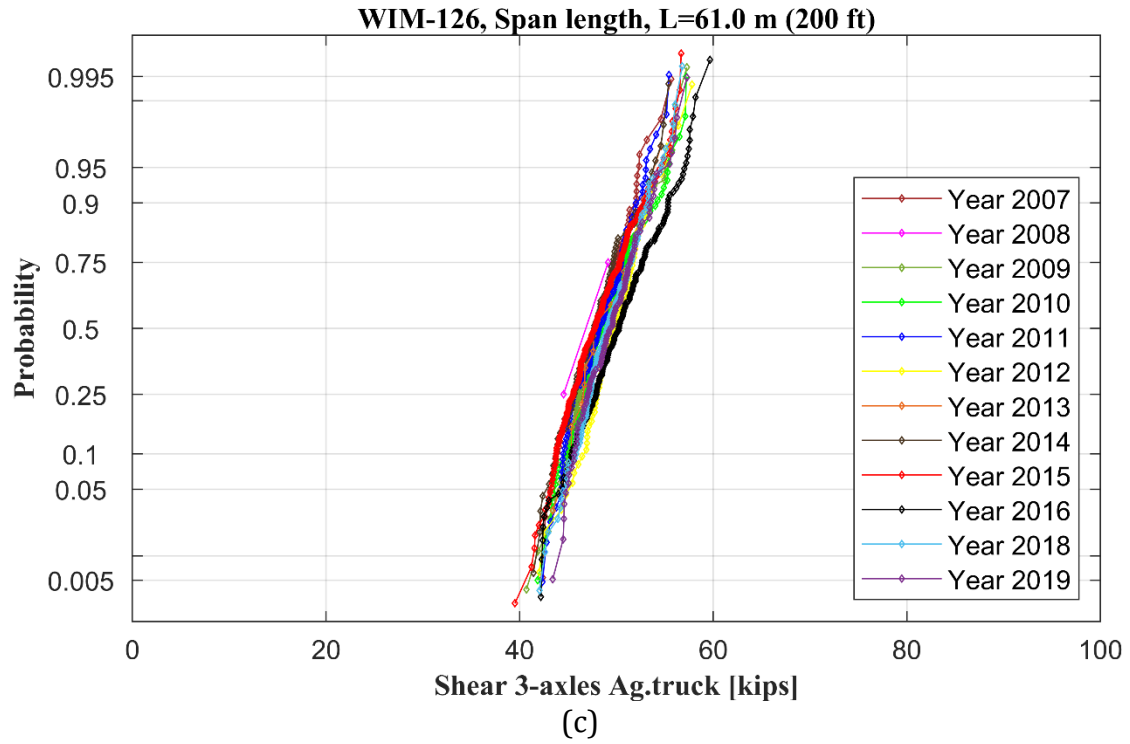


Figure 4-B.4. CDF of shear for 3-axes agricultural vehicles for span length 36.6 m (120 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





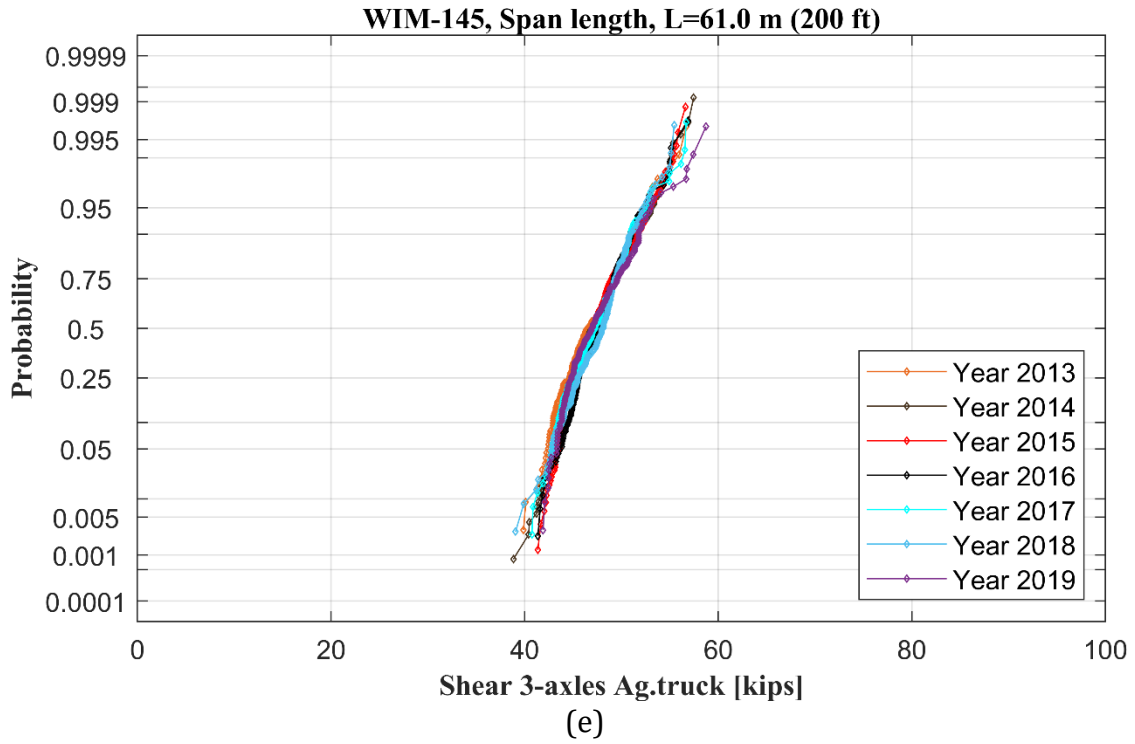
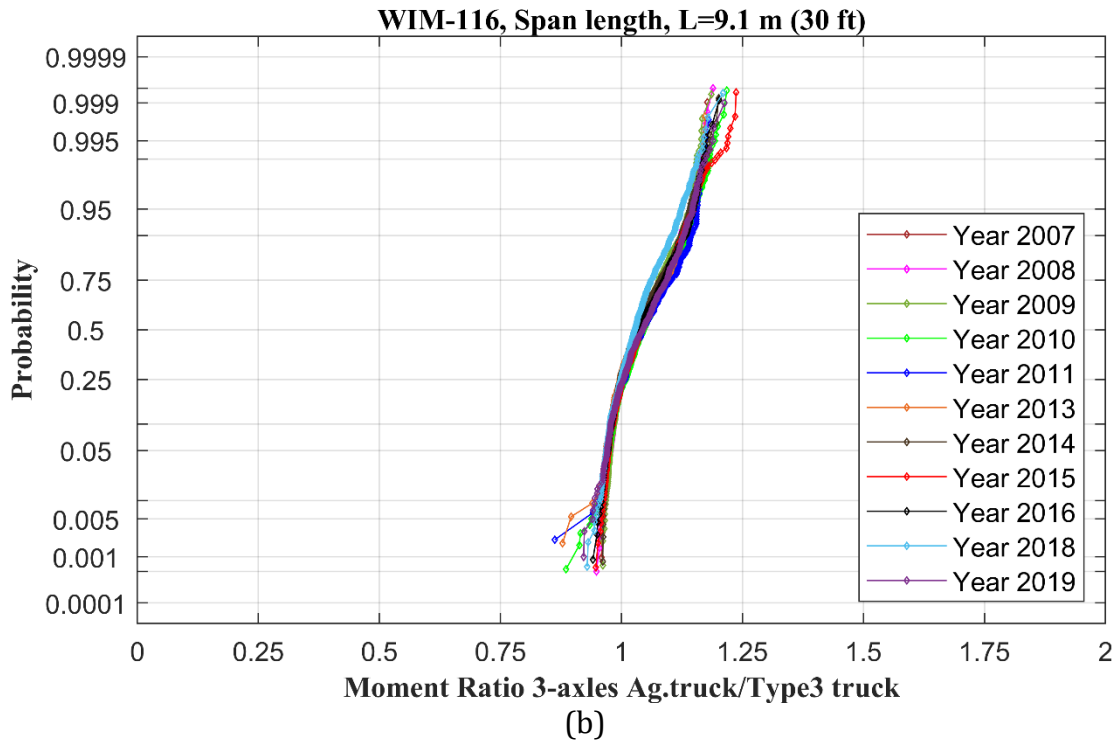
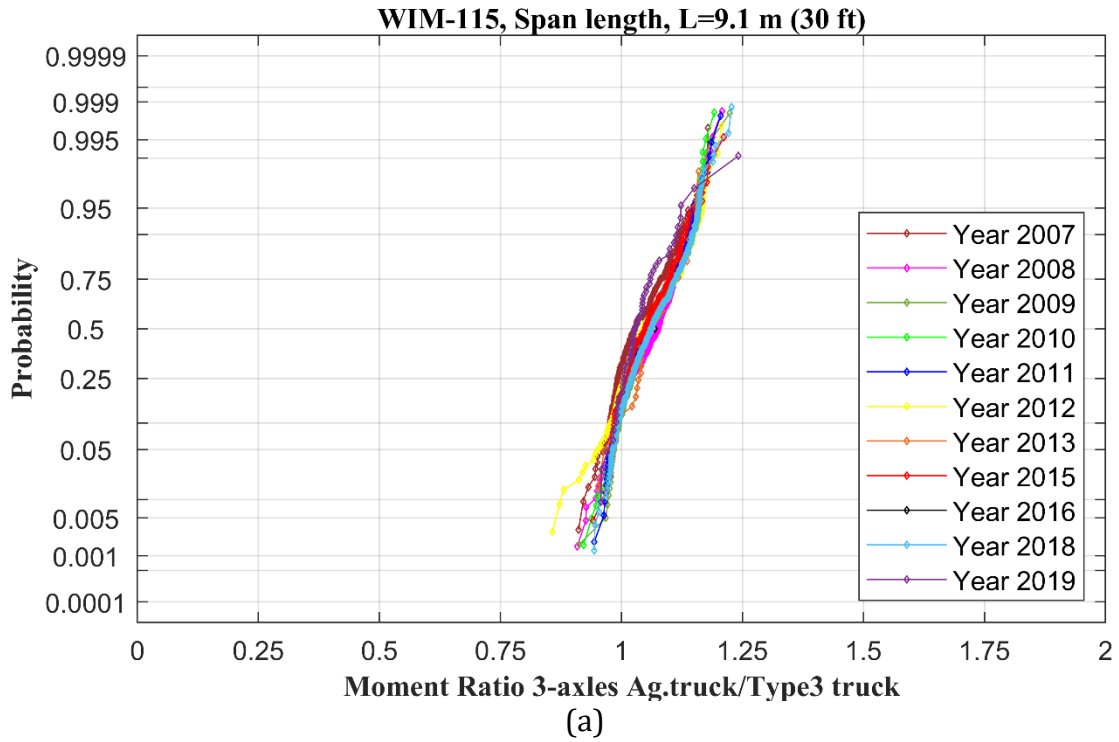
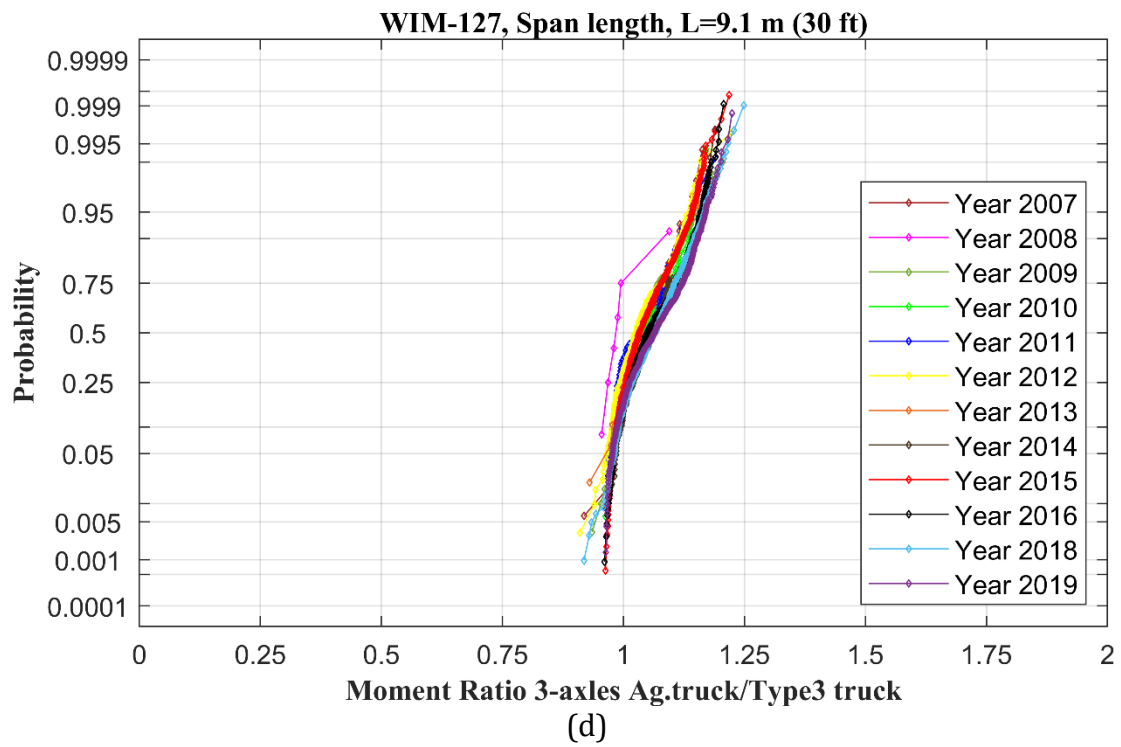
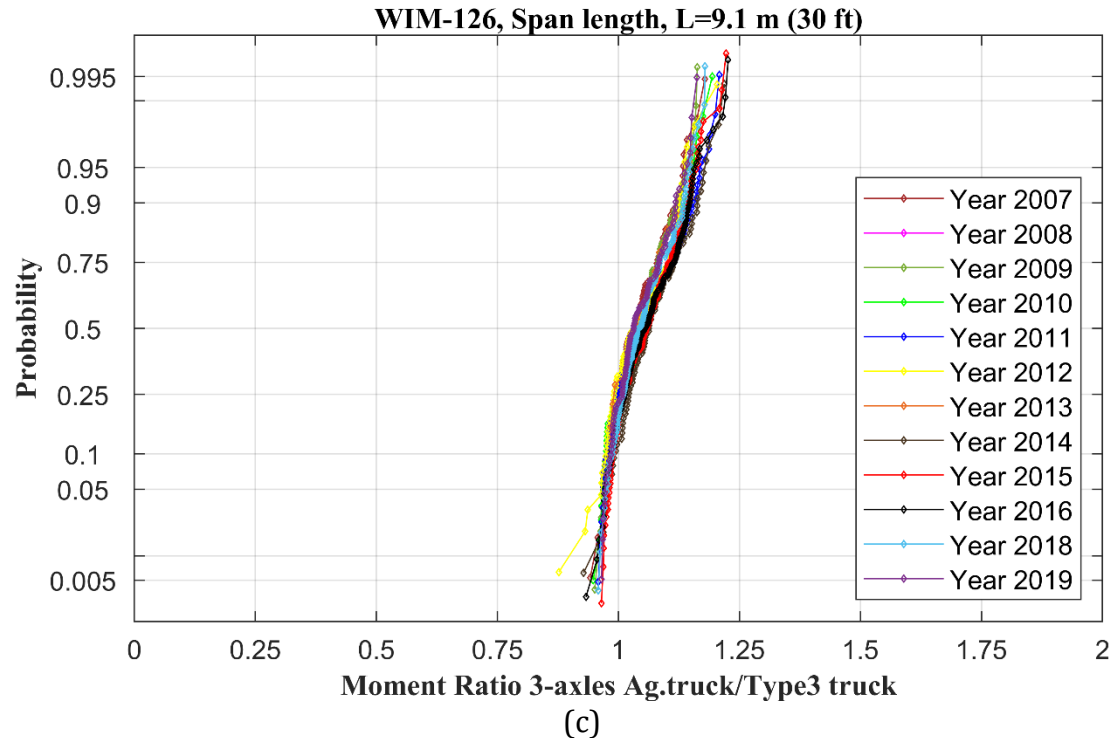


Figure 4-B.5. CDF of shear for 3-axles agricultural for span length 61 m (200 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

Appendix 4-C. CDF of moment ratio for 3-axle vehicles to Type 3 truck





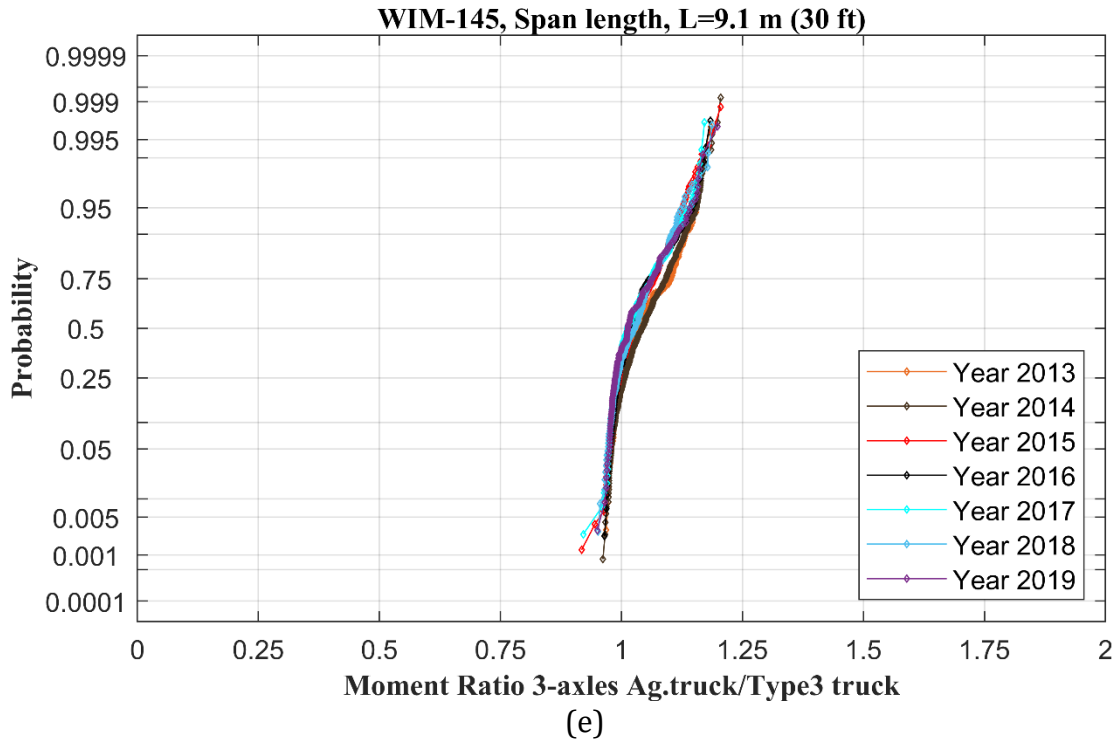
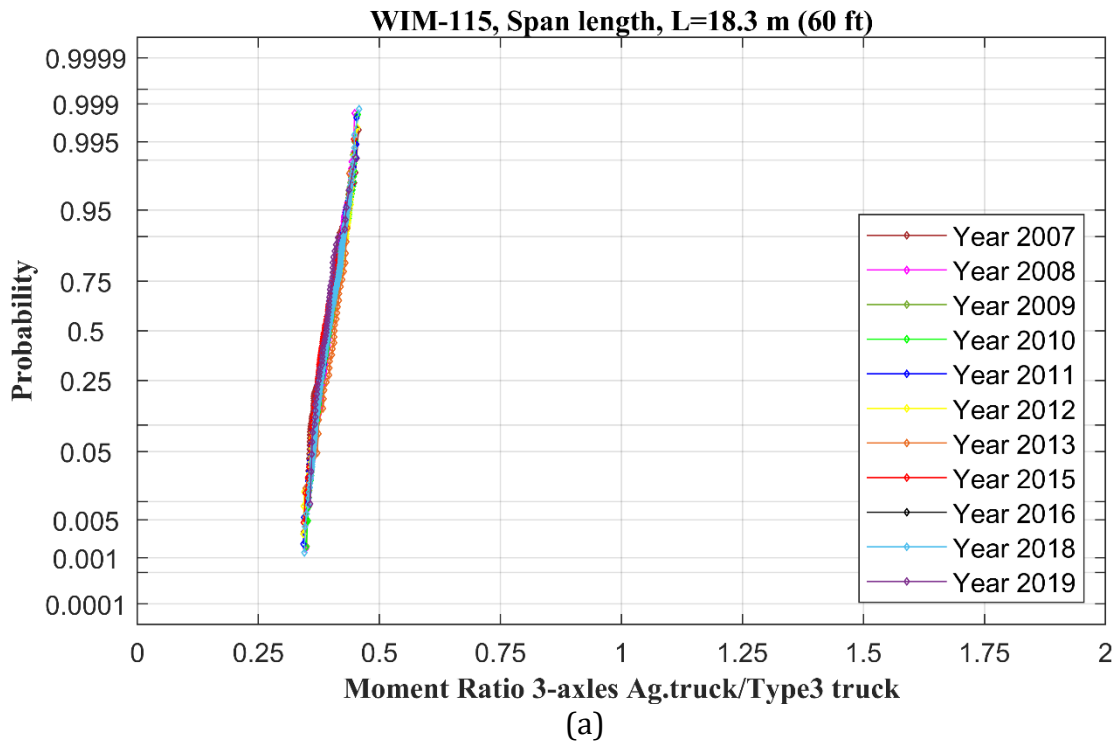
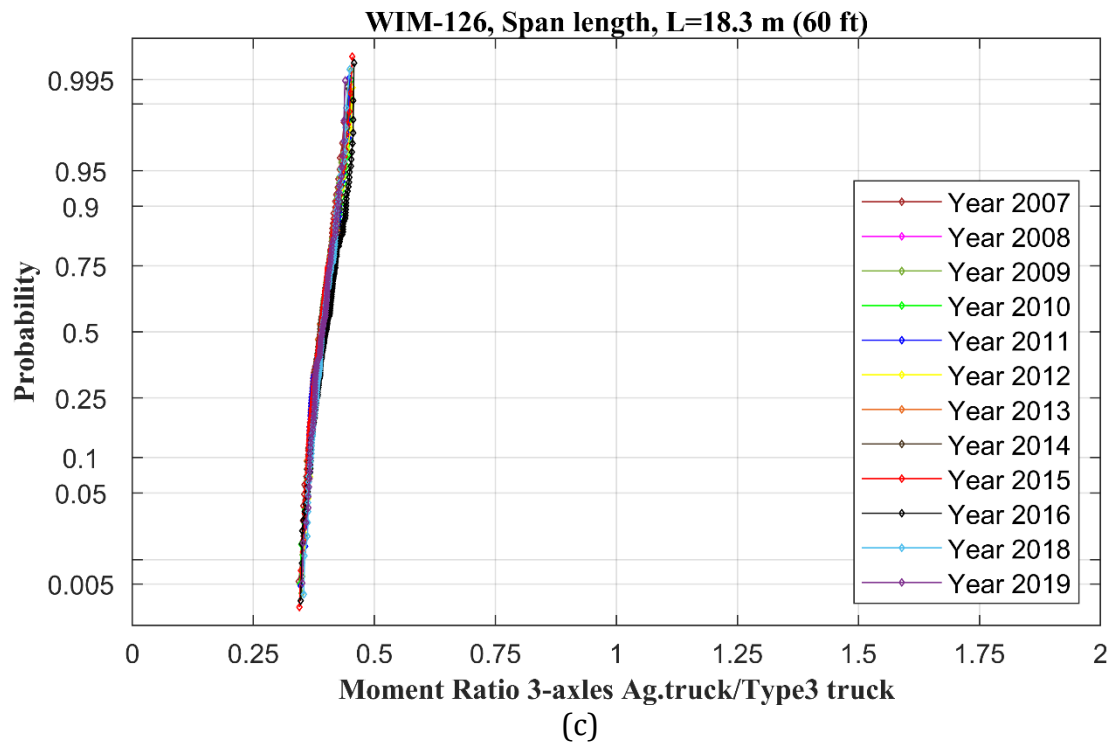
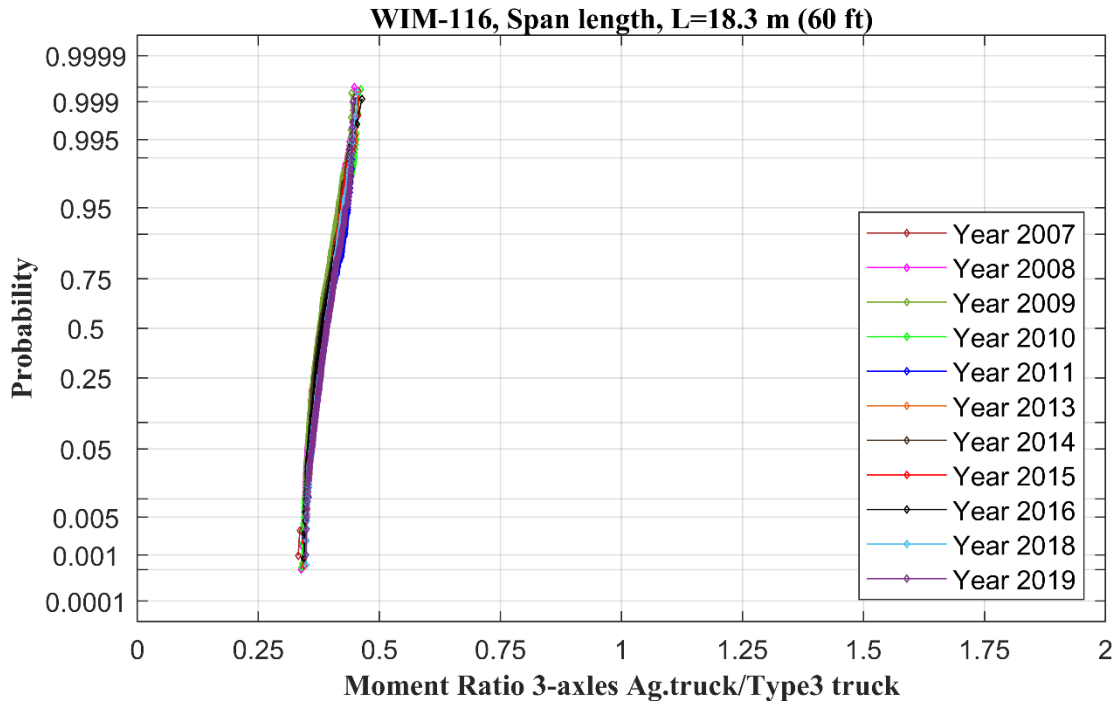


Figure 4-C.1. CDF of moment ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length of 9.1 m (30 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





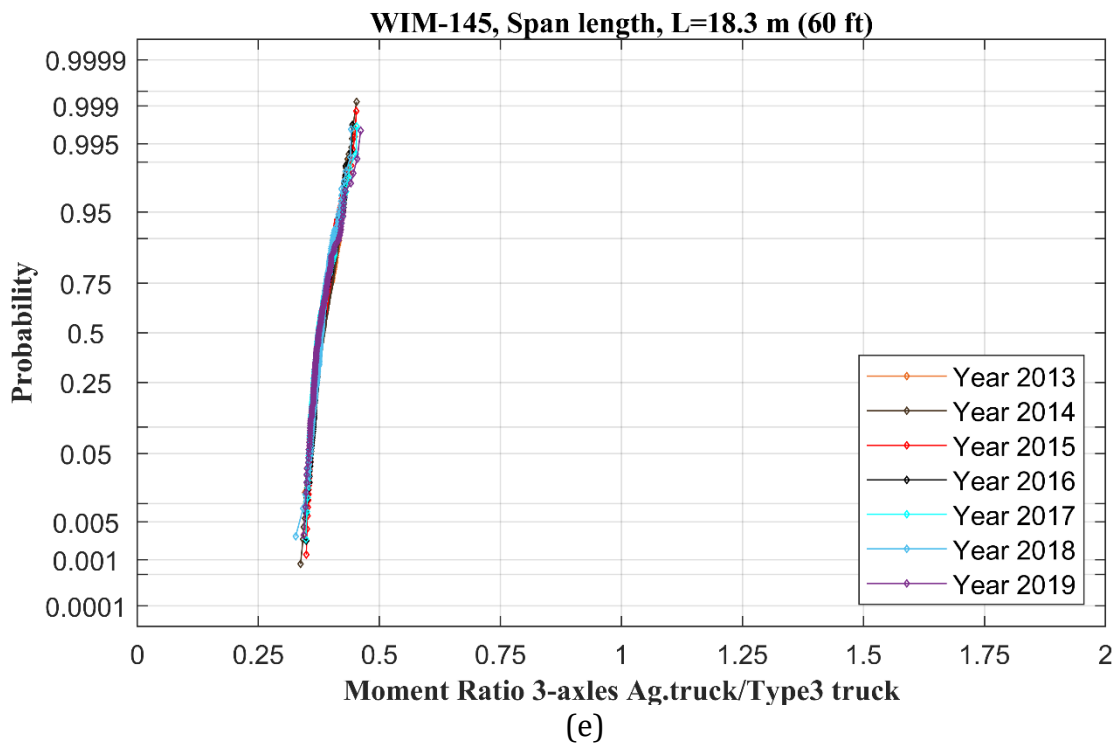
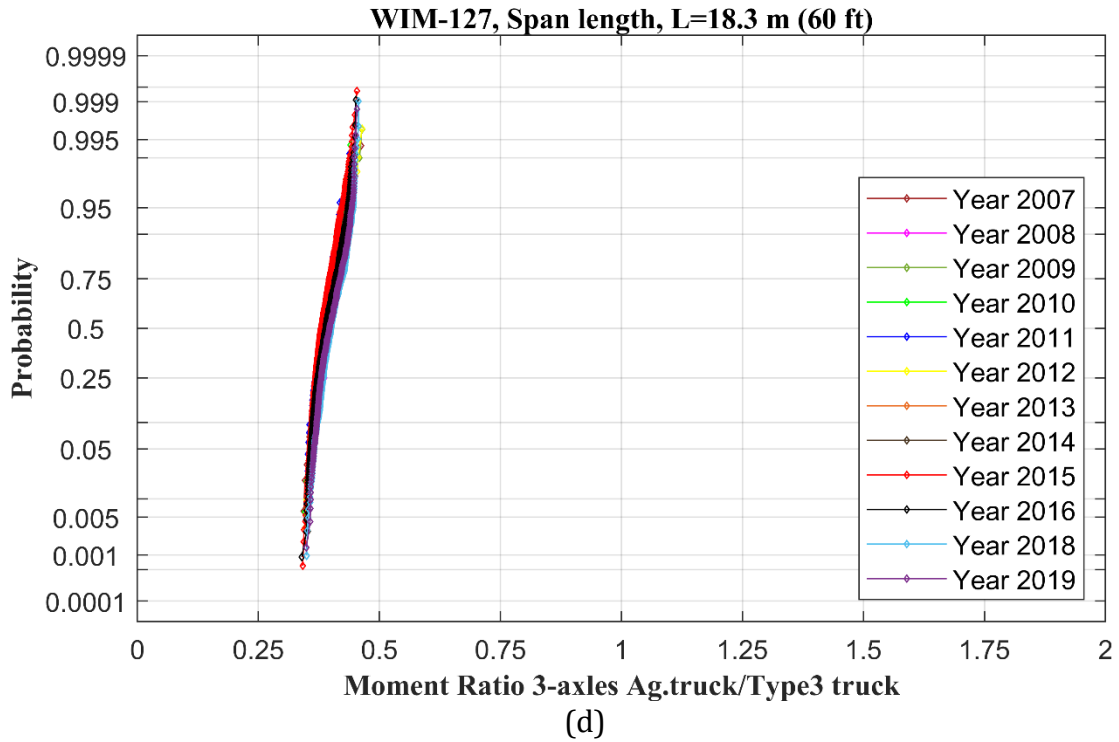
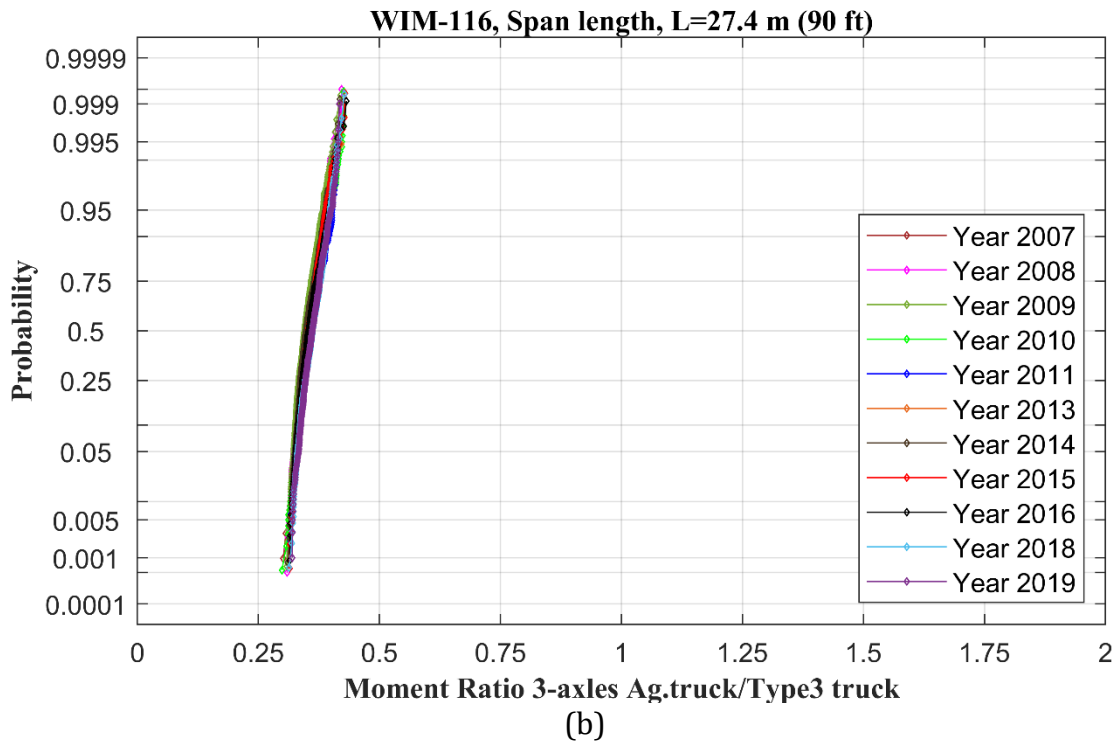
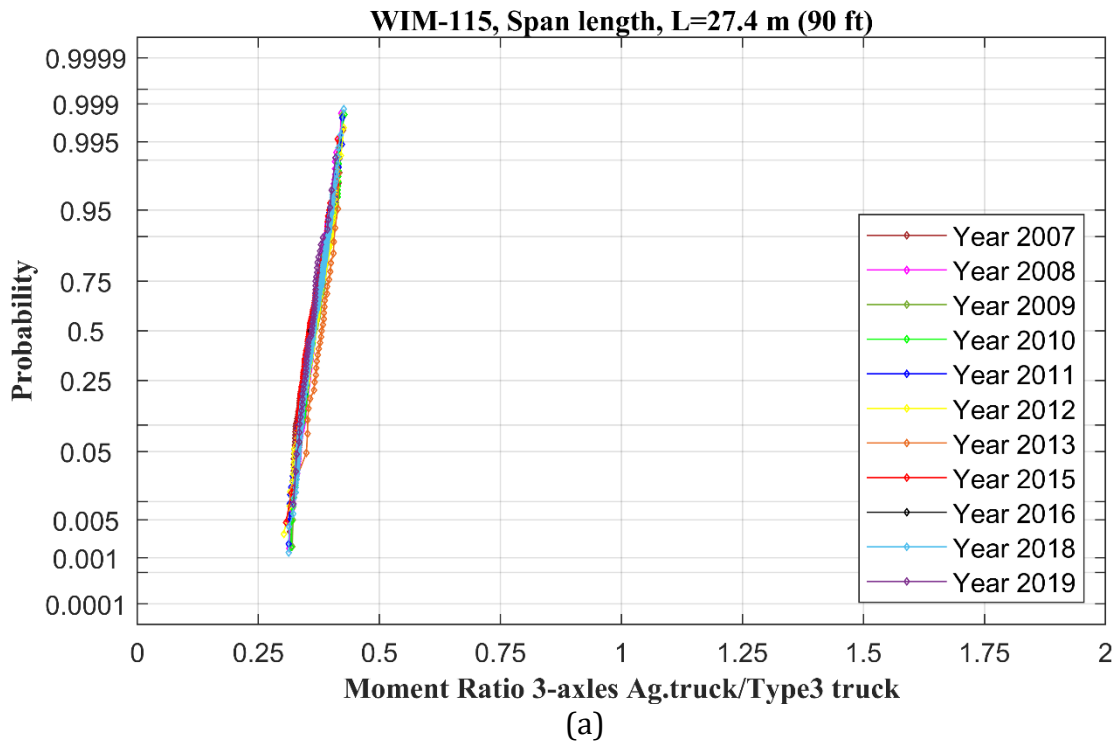
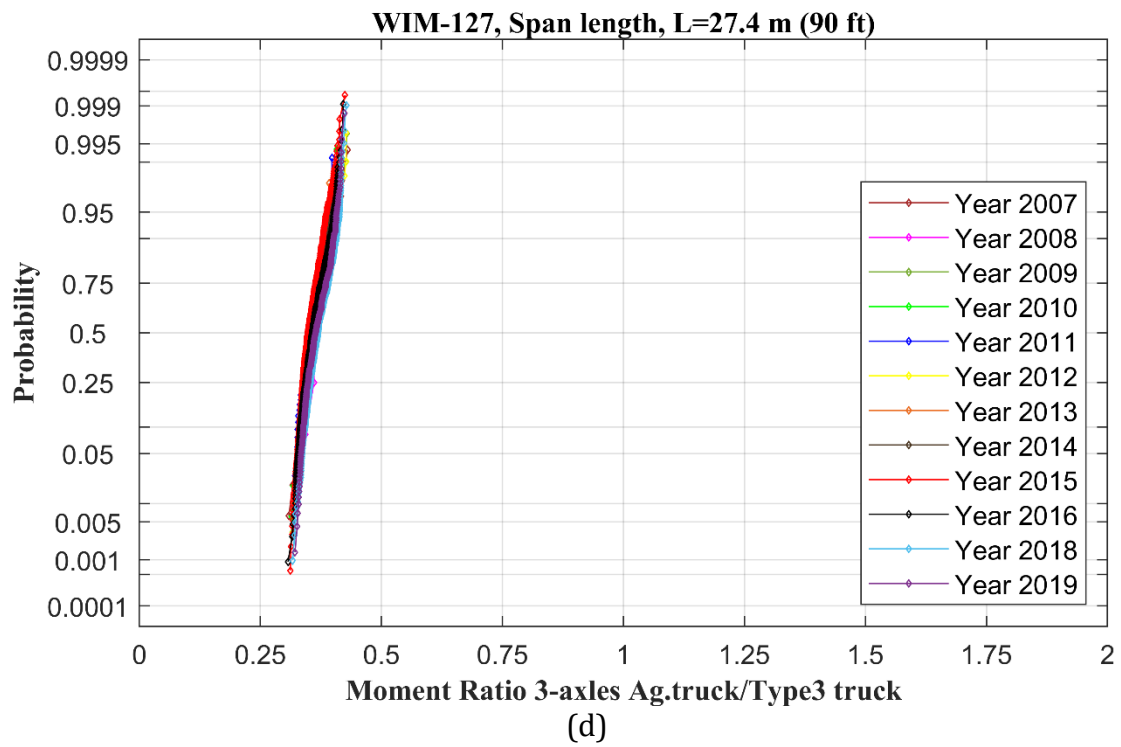
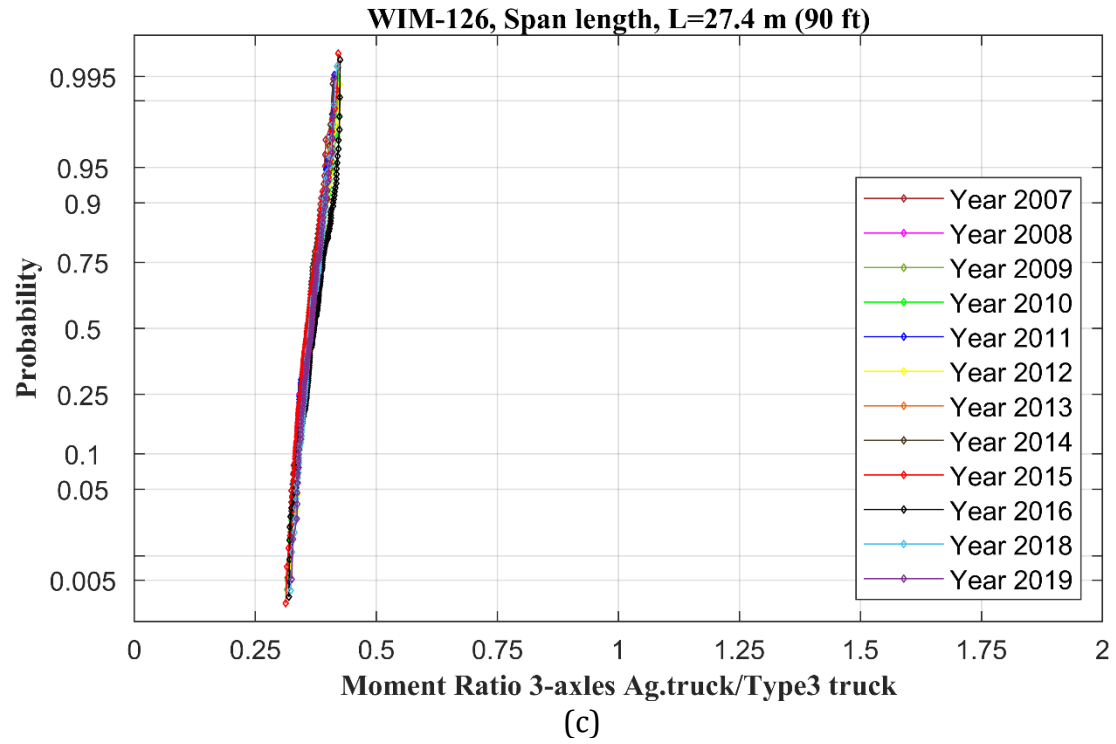


Figure 4-C.2. CDF of moment ratio for 3-axles agricultural vehicles to Type 3 vehicle for span length 18.3 m (60 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





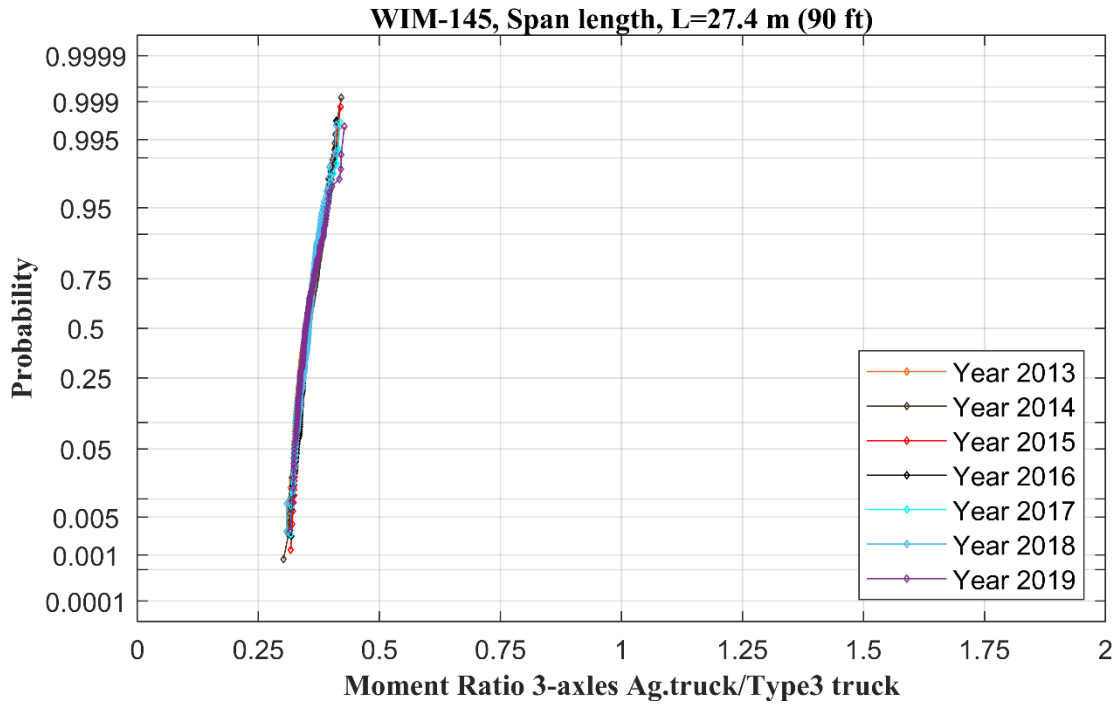
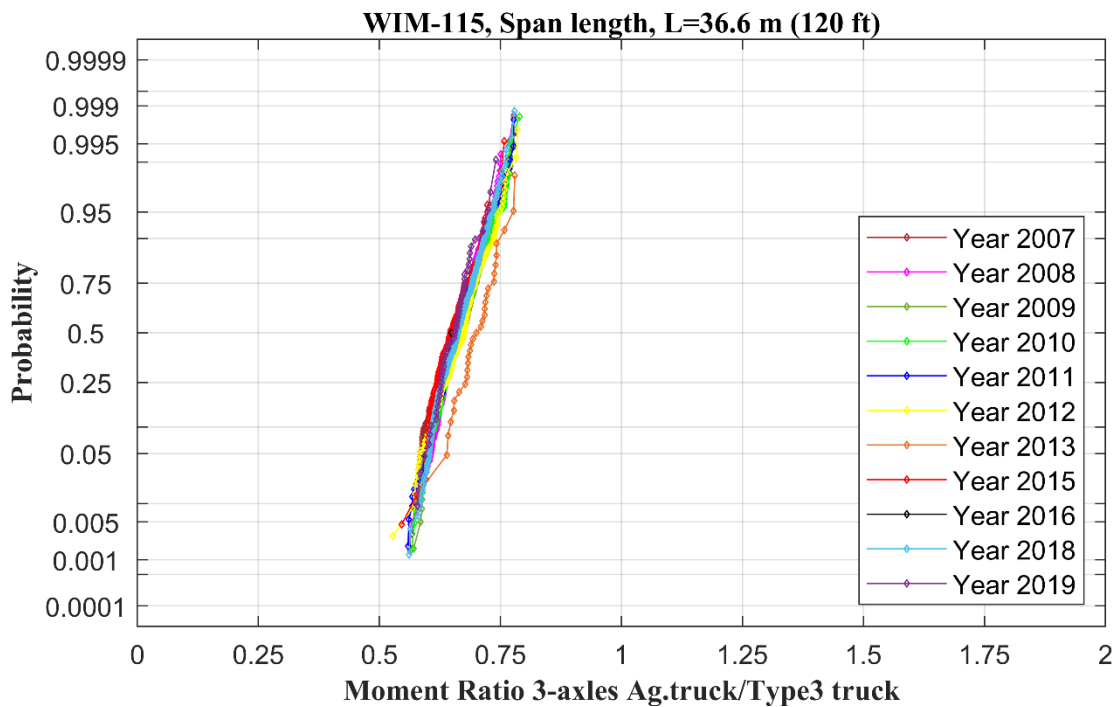
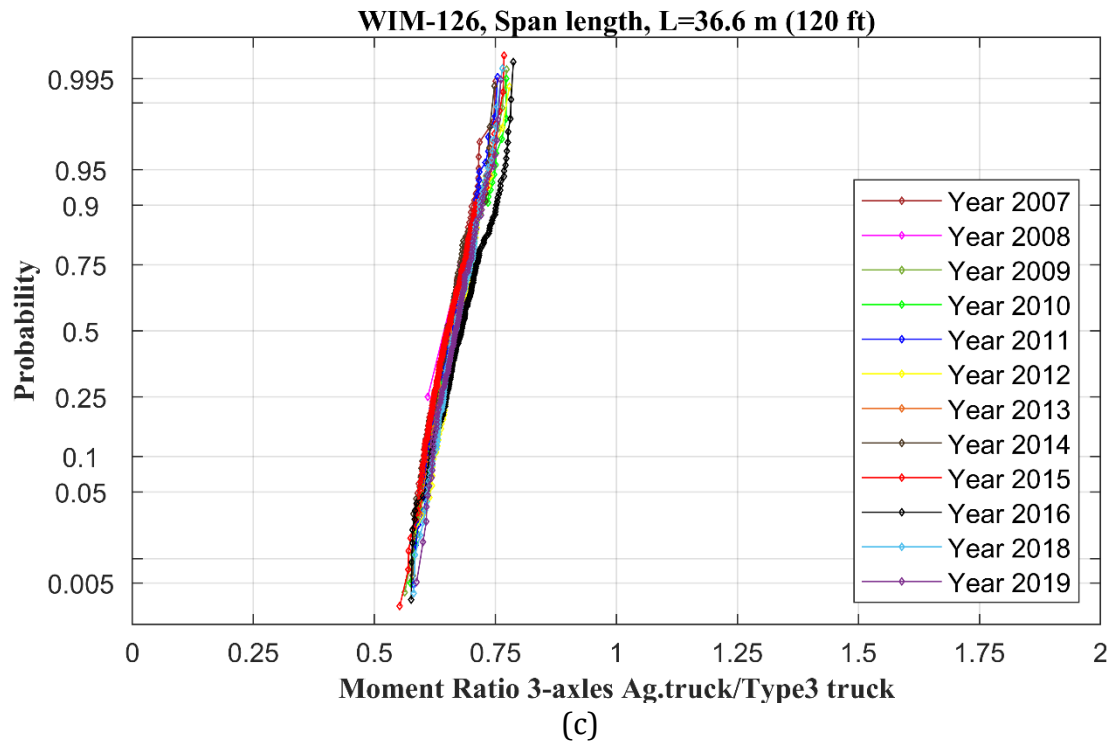
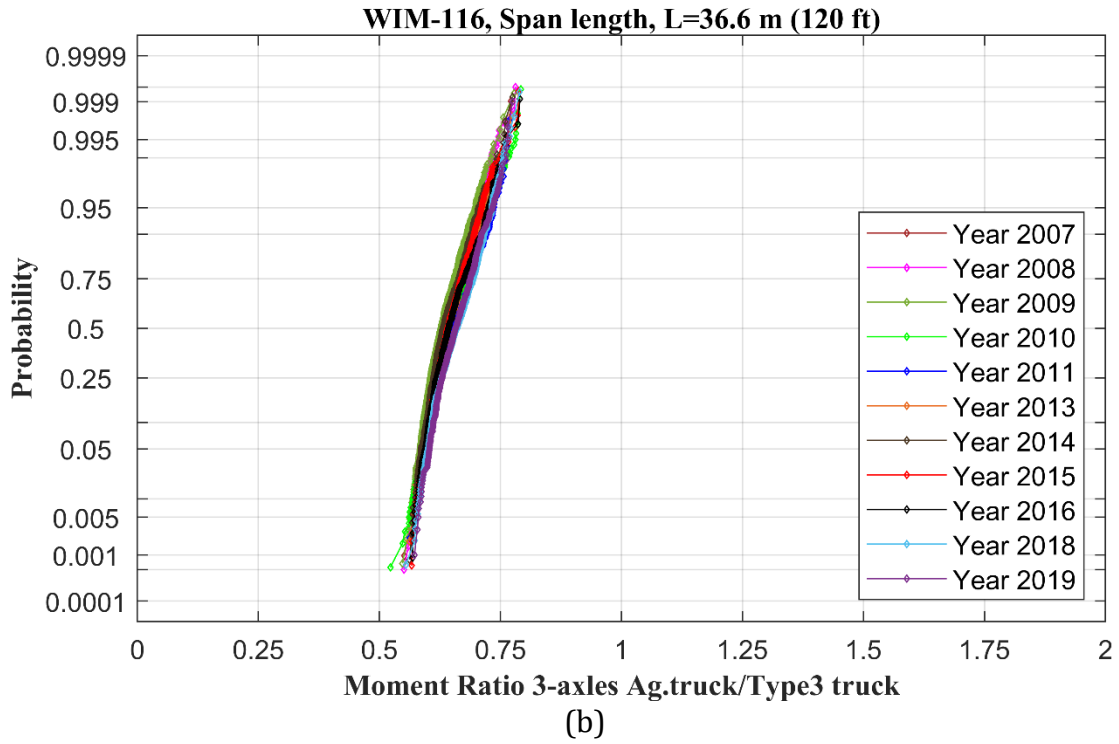


Figure 4-C.3. CDF of moment ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length 27.4 m (90 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





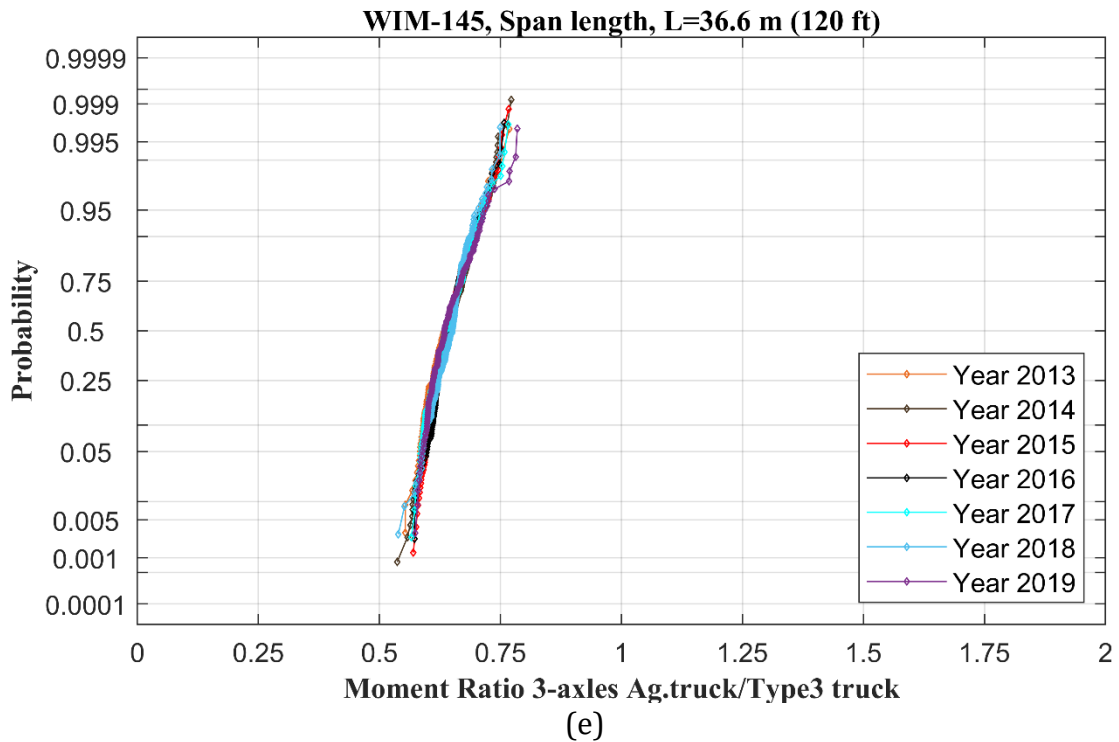
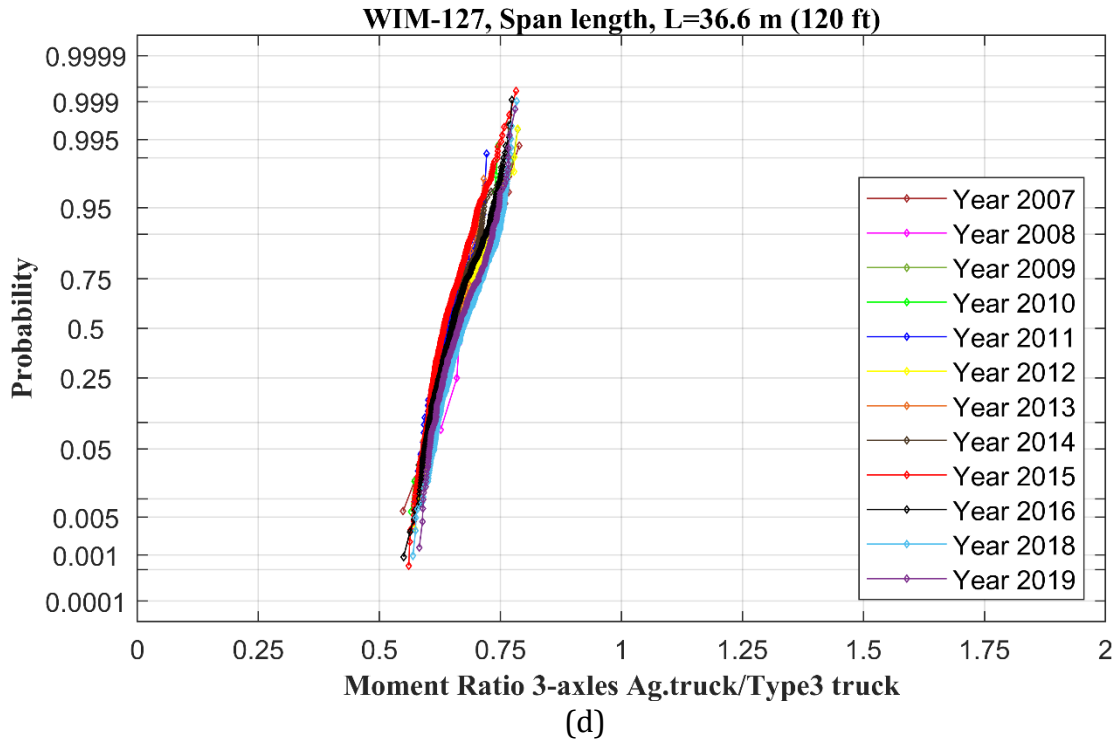
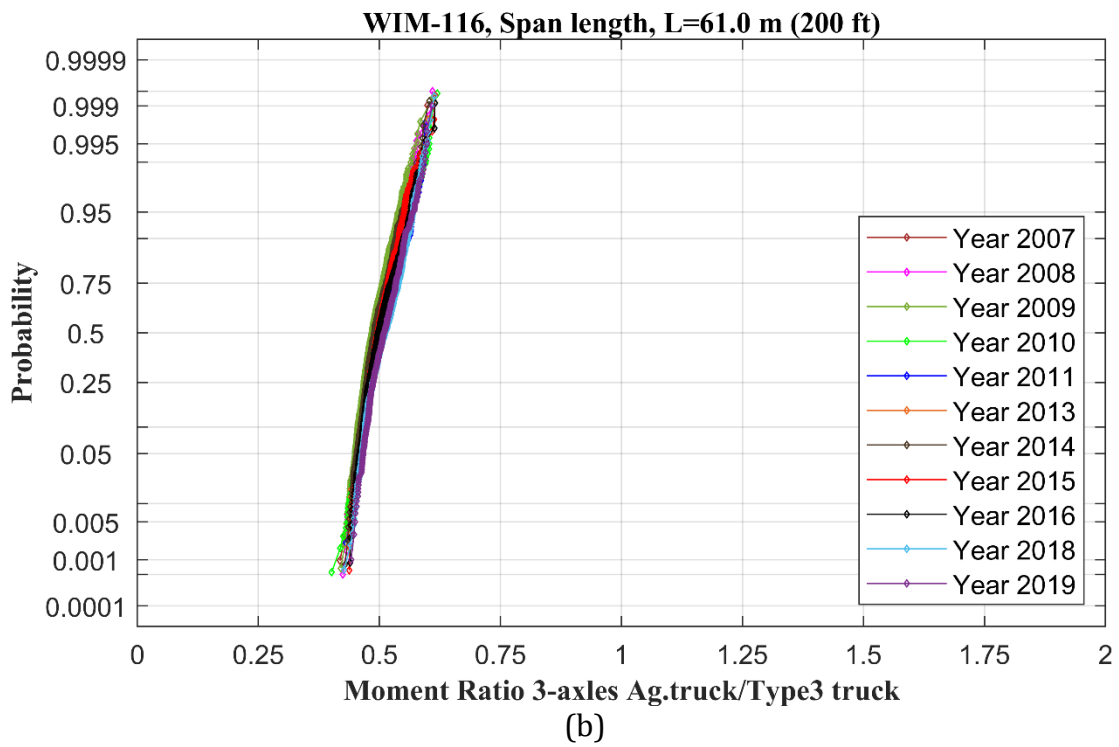
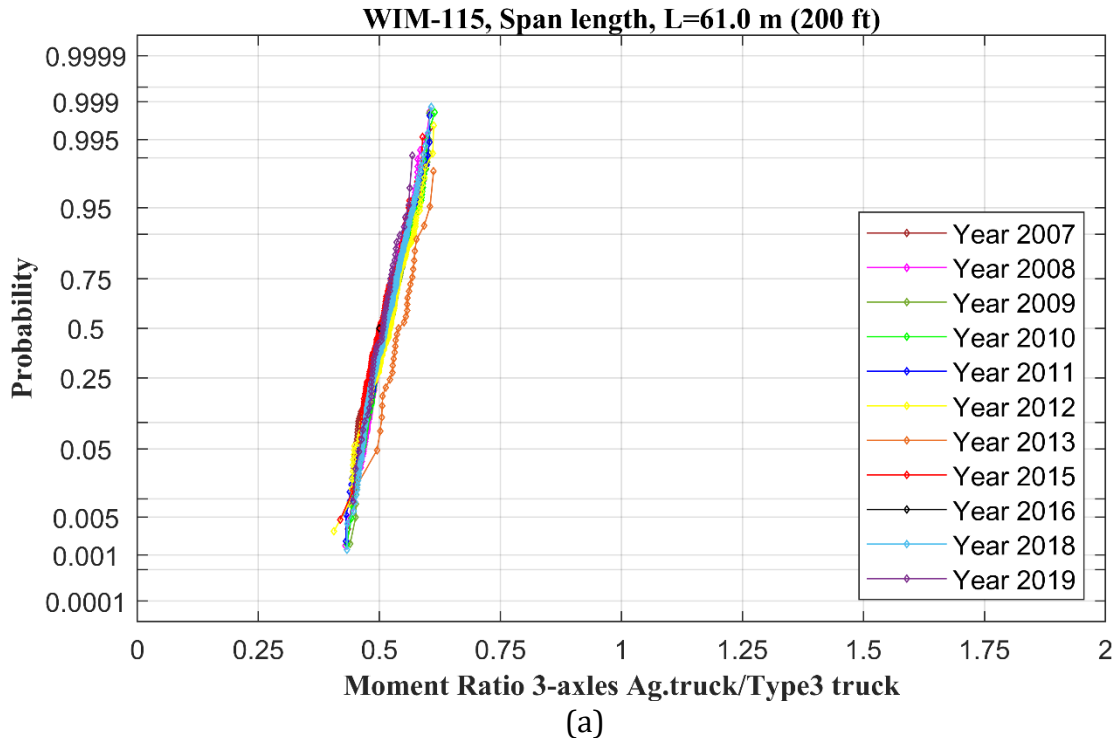
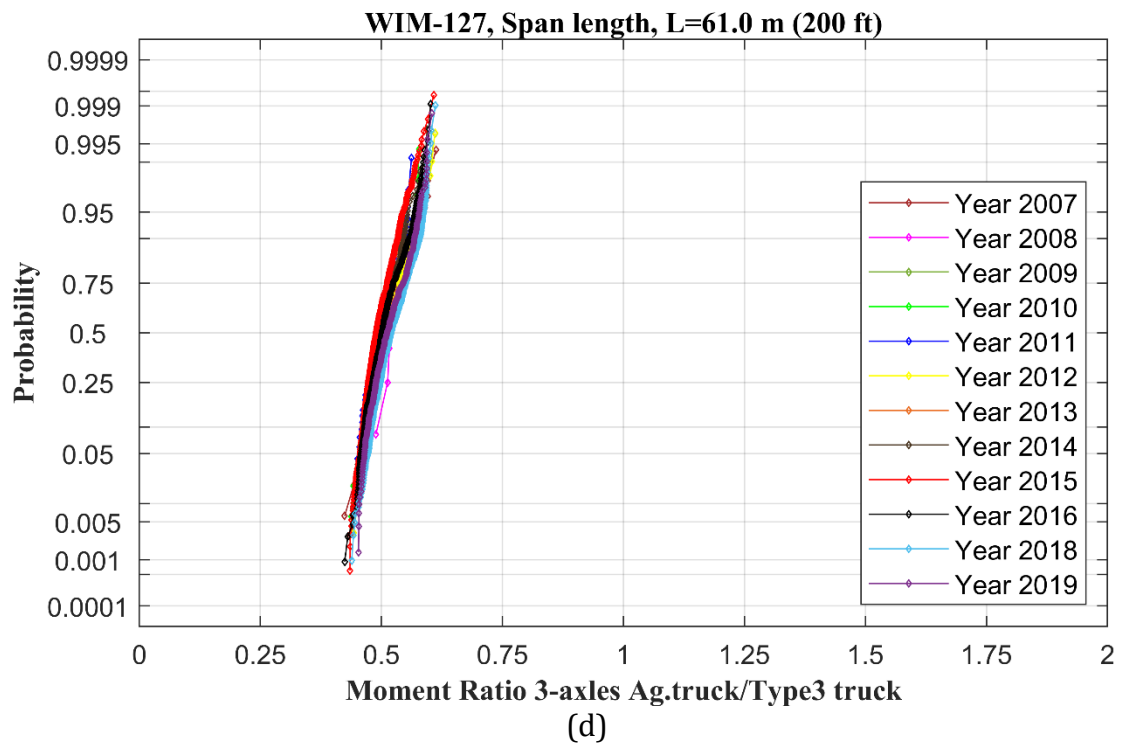
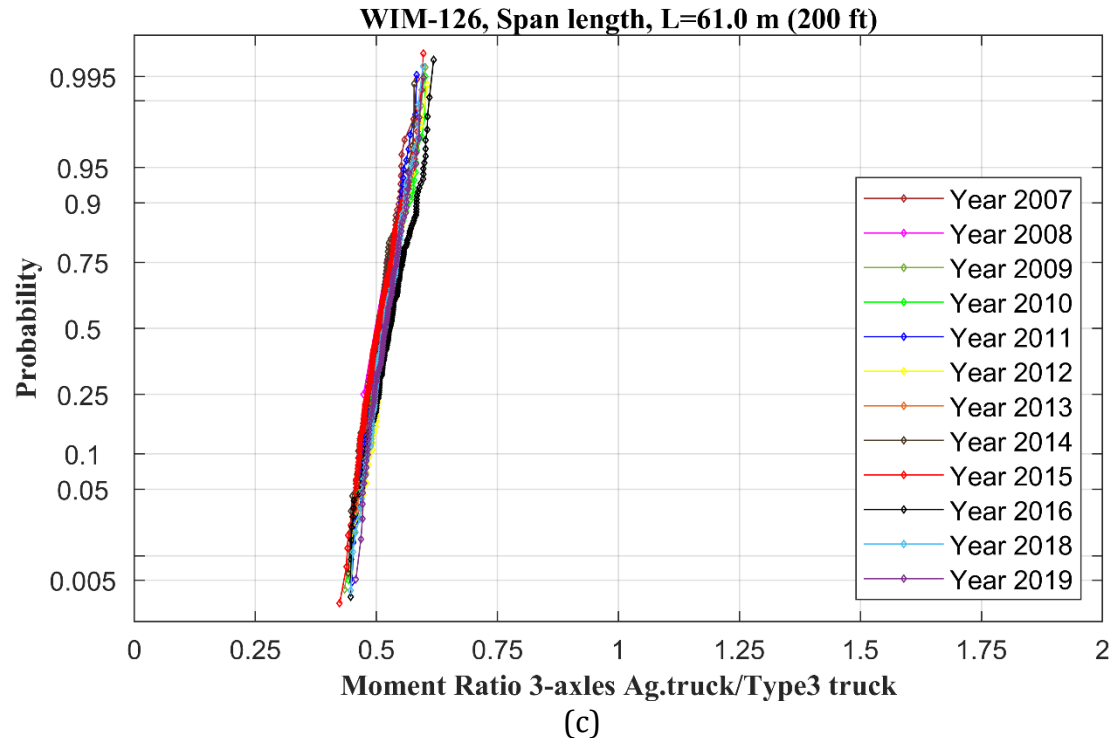


Figure 4-C.4. CDF of moment ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length 36.6 m (120 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





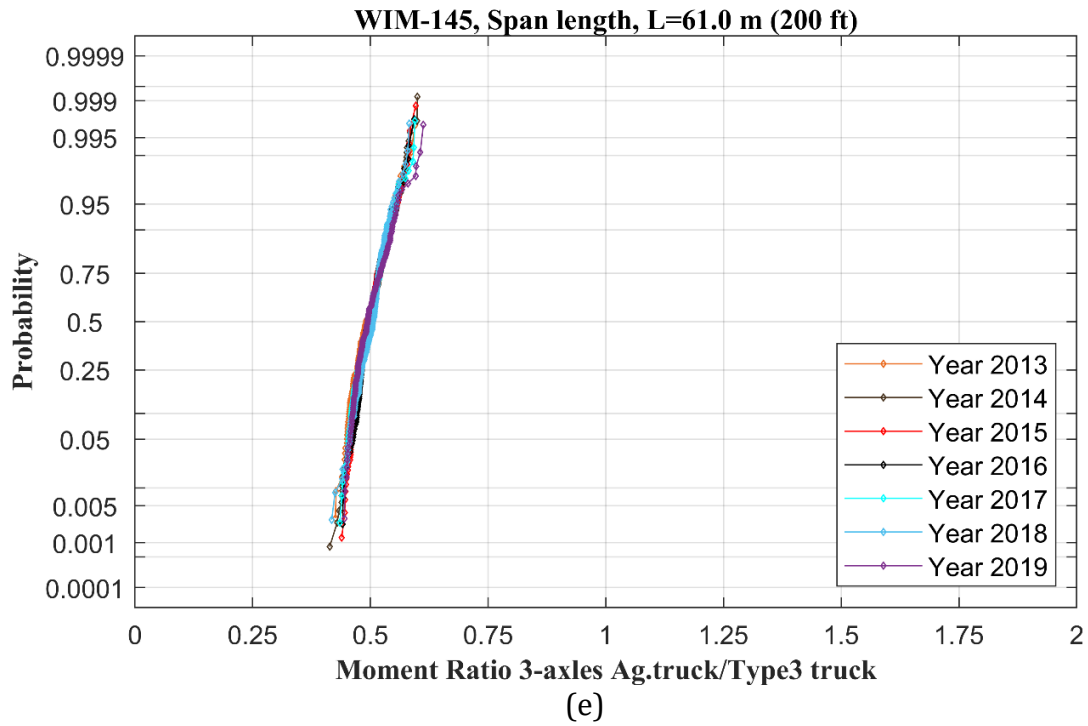
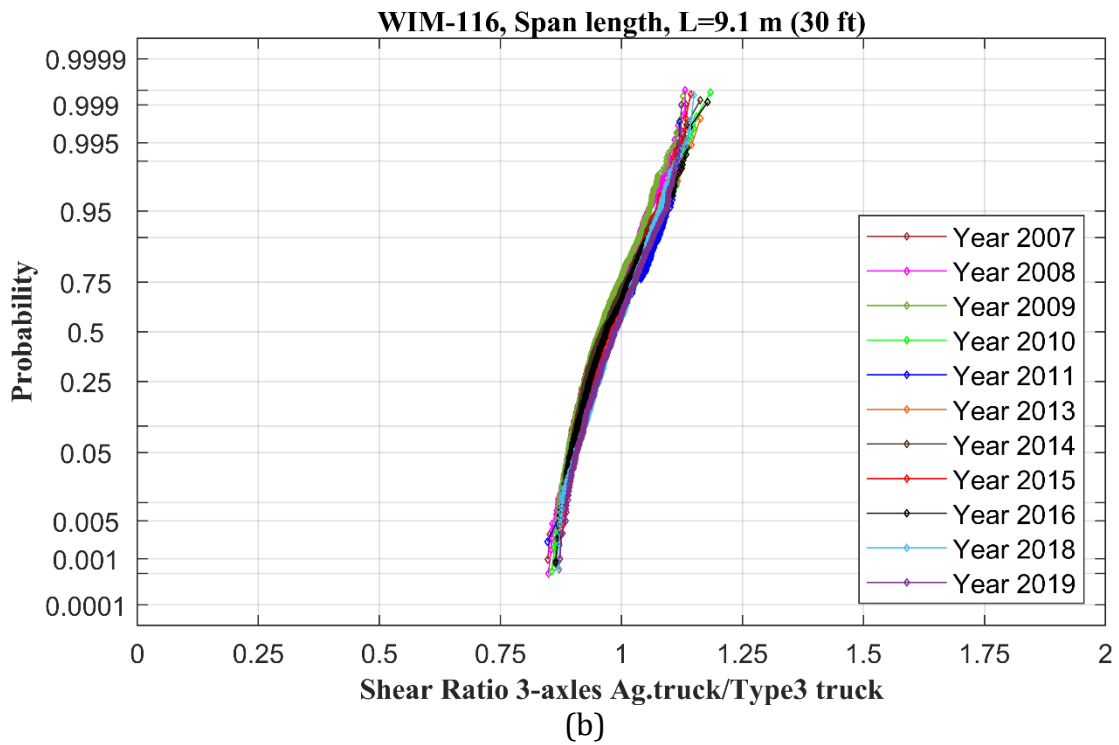
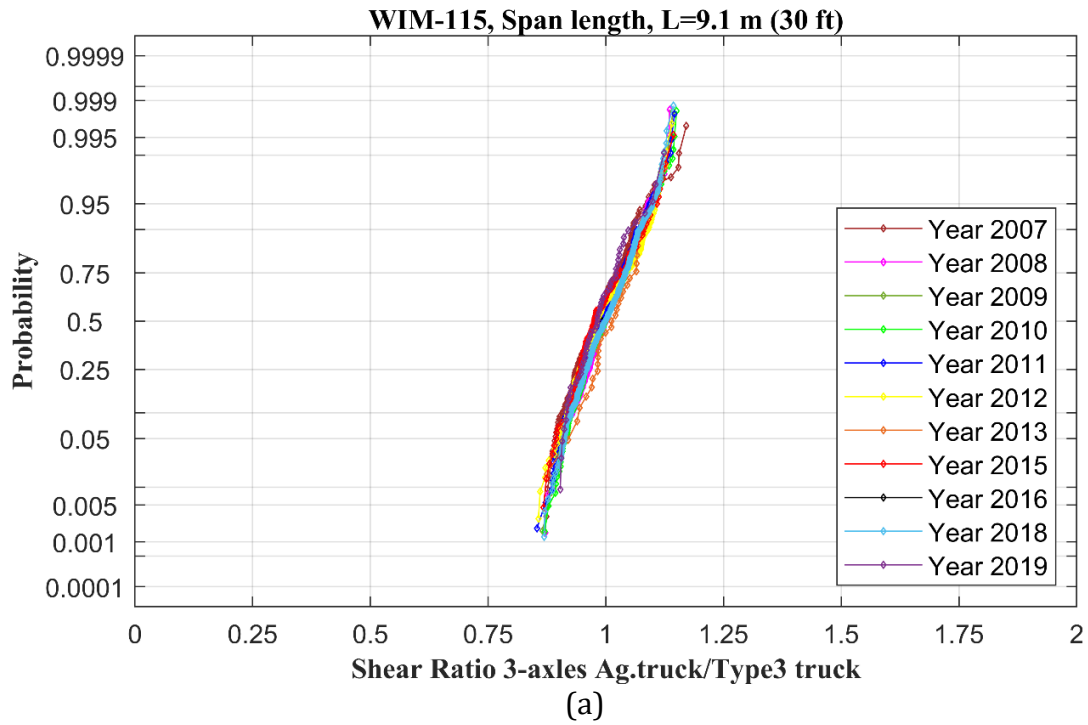
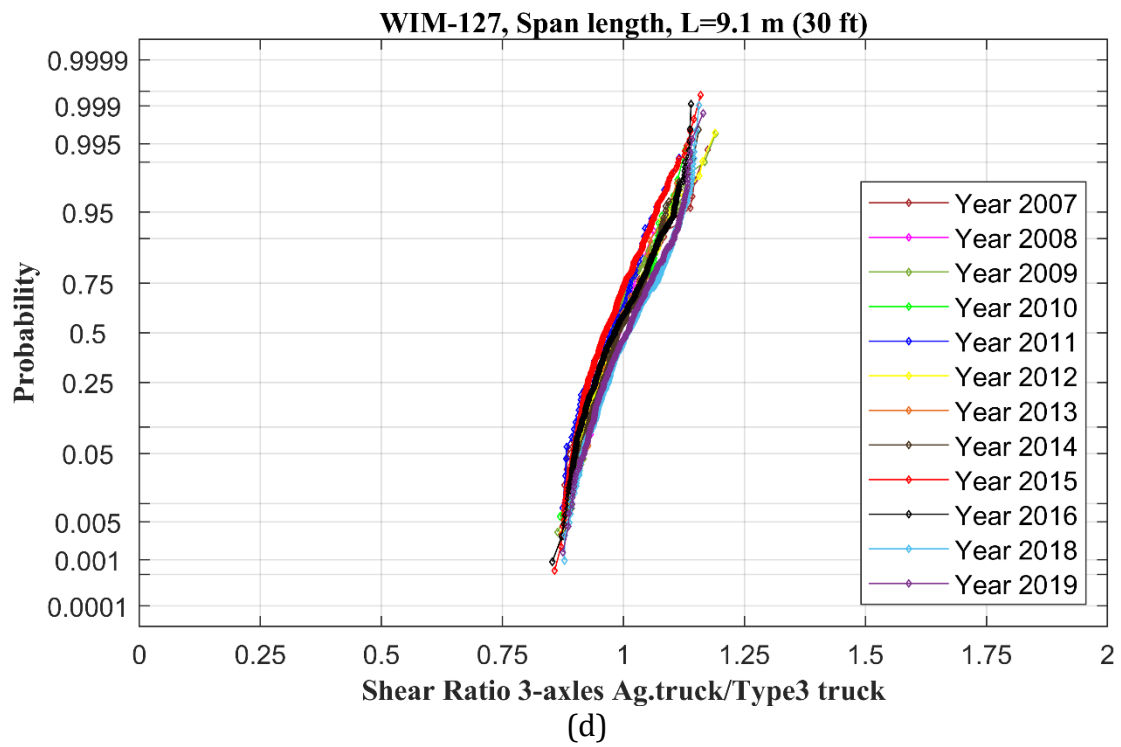
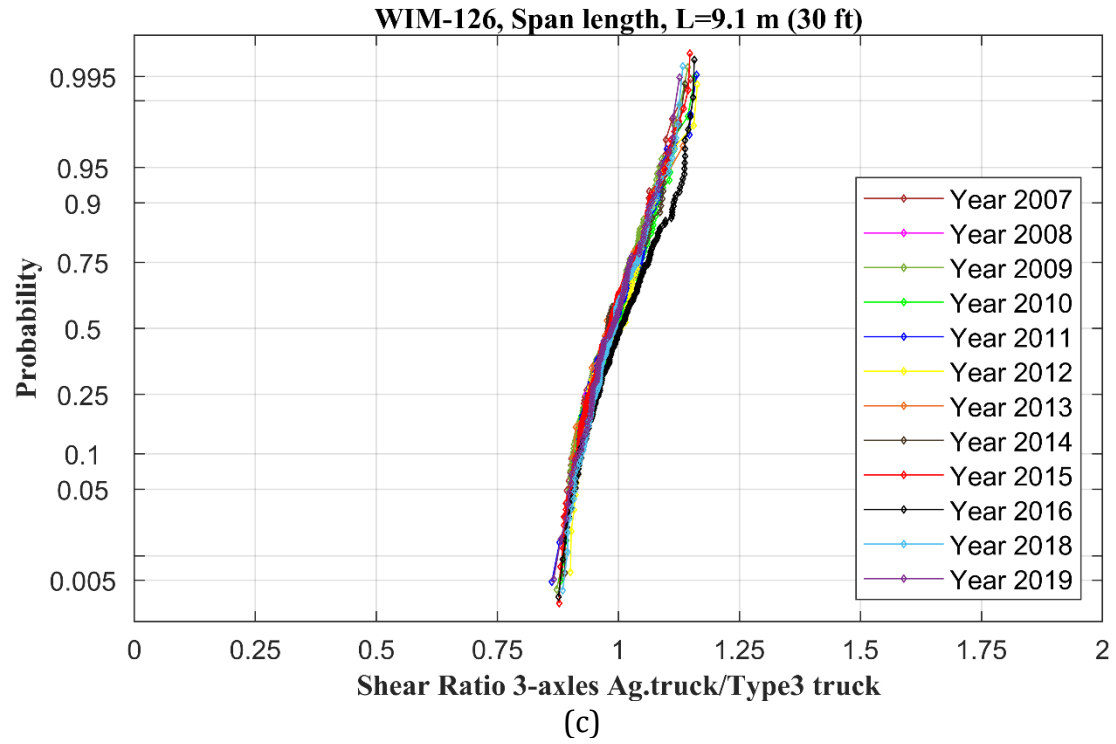


Figure 4-C.5. CDF of moment ratio for 3-axles agricultural vehicles to Type 3 vehicle for span length 61 m (200 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

Appendix 4-D. CDF of shear ratio for 3-axle vehicles to Type 3 truck





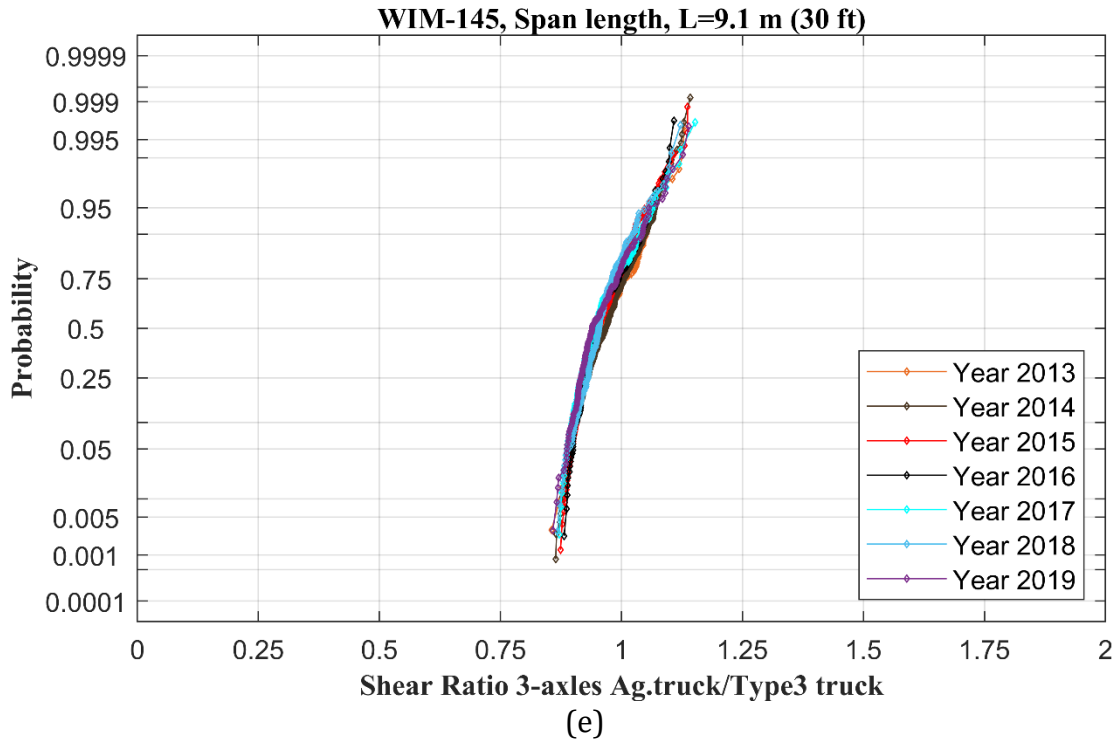
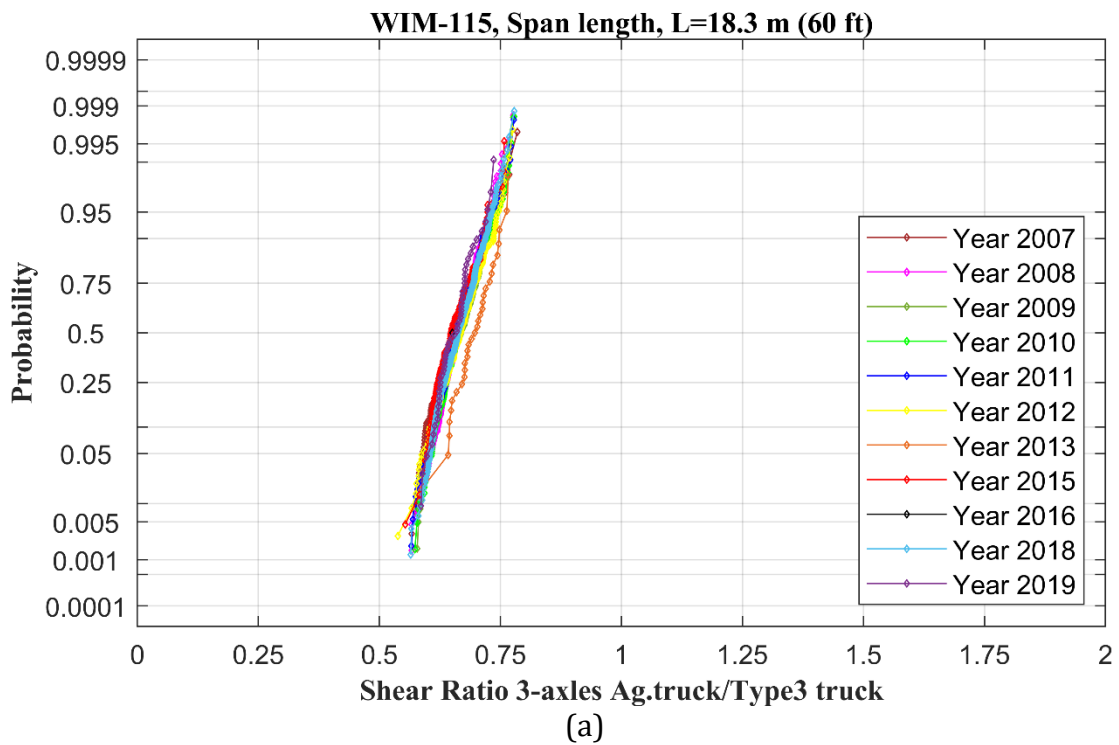
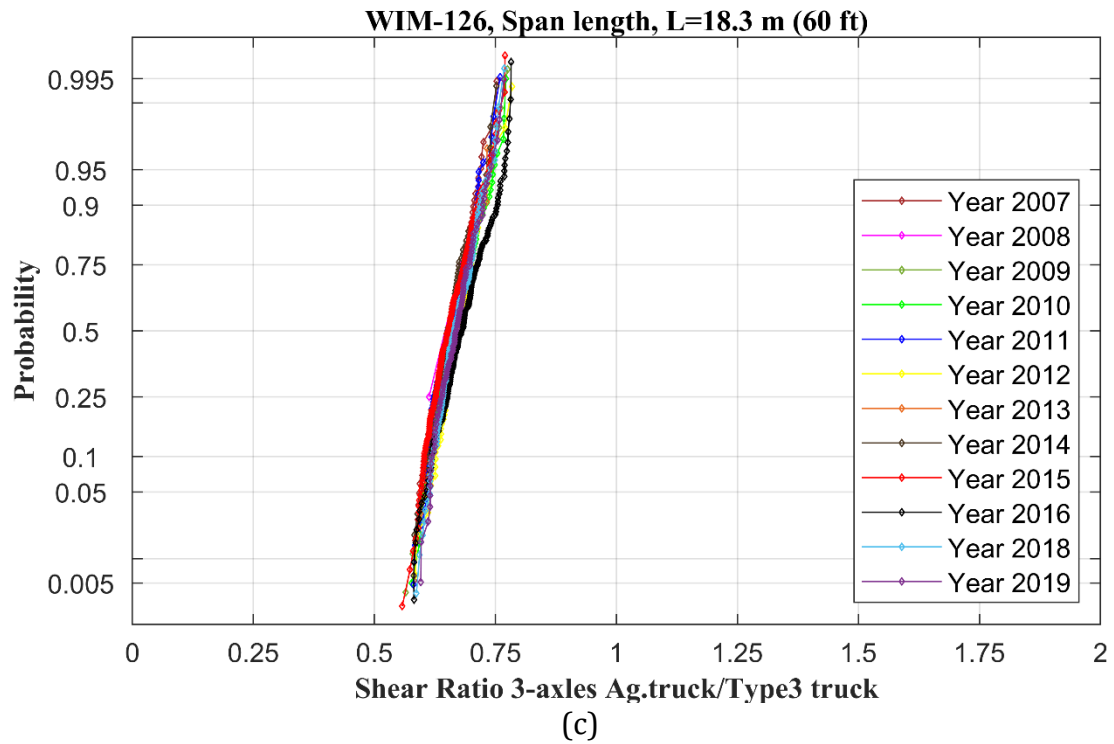
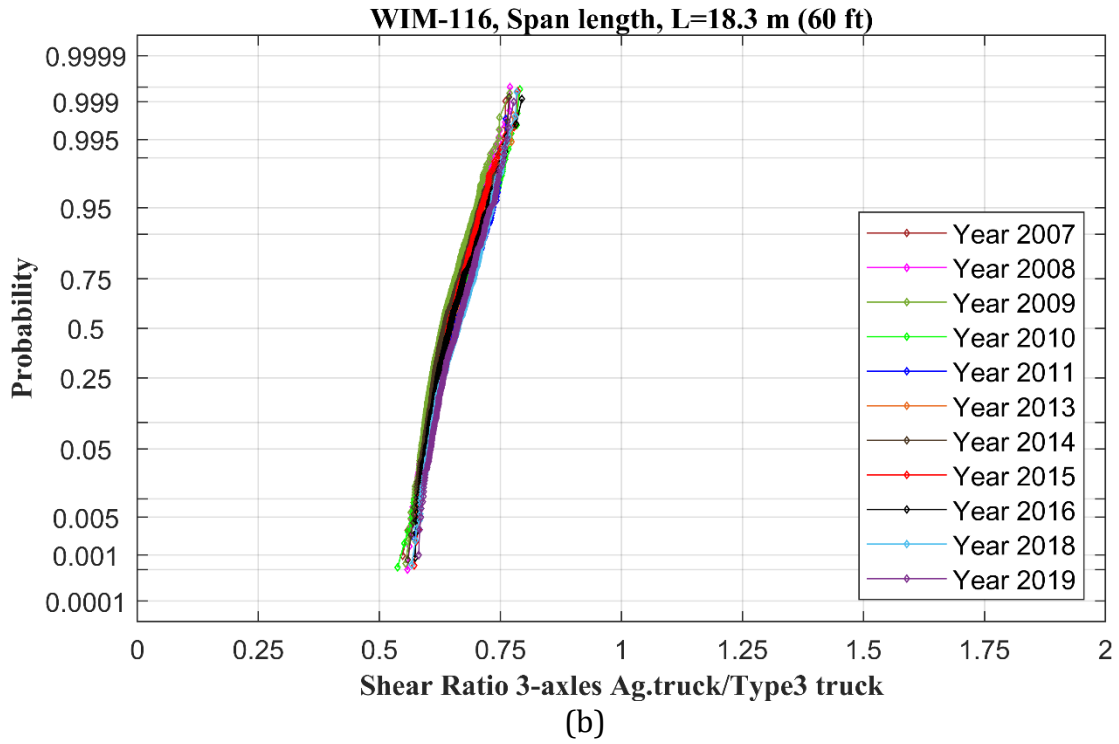


Figure 4-D.1. CDF of shear ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length of 9.1 m (30 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





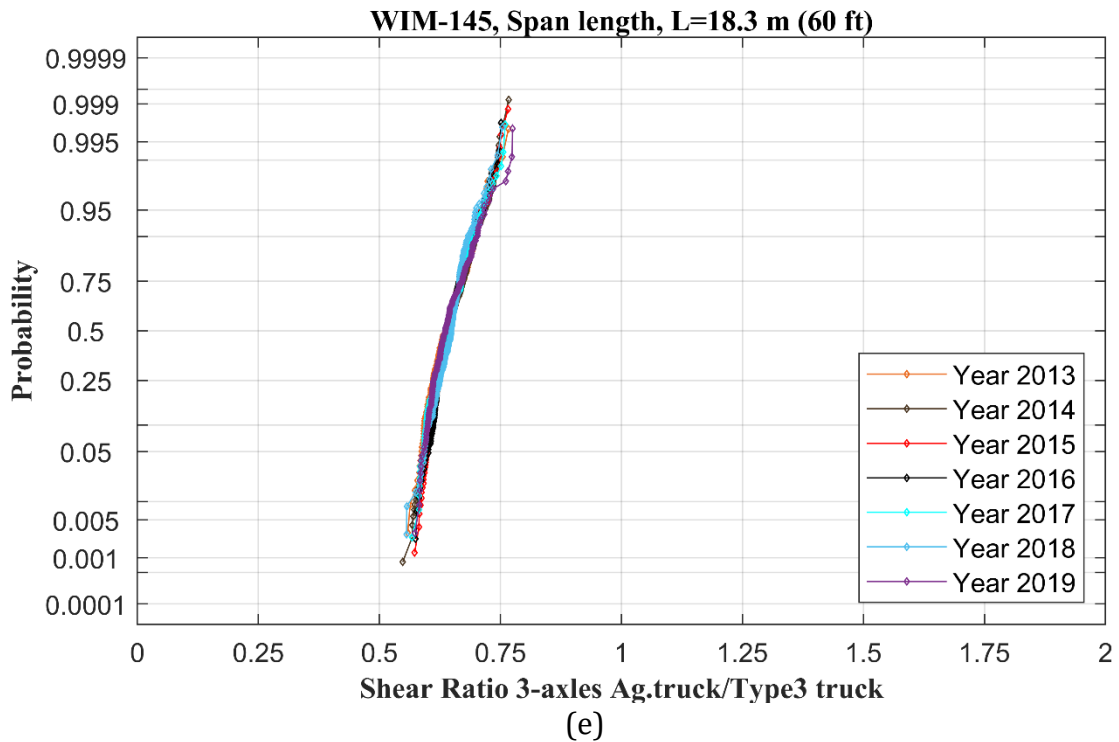
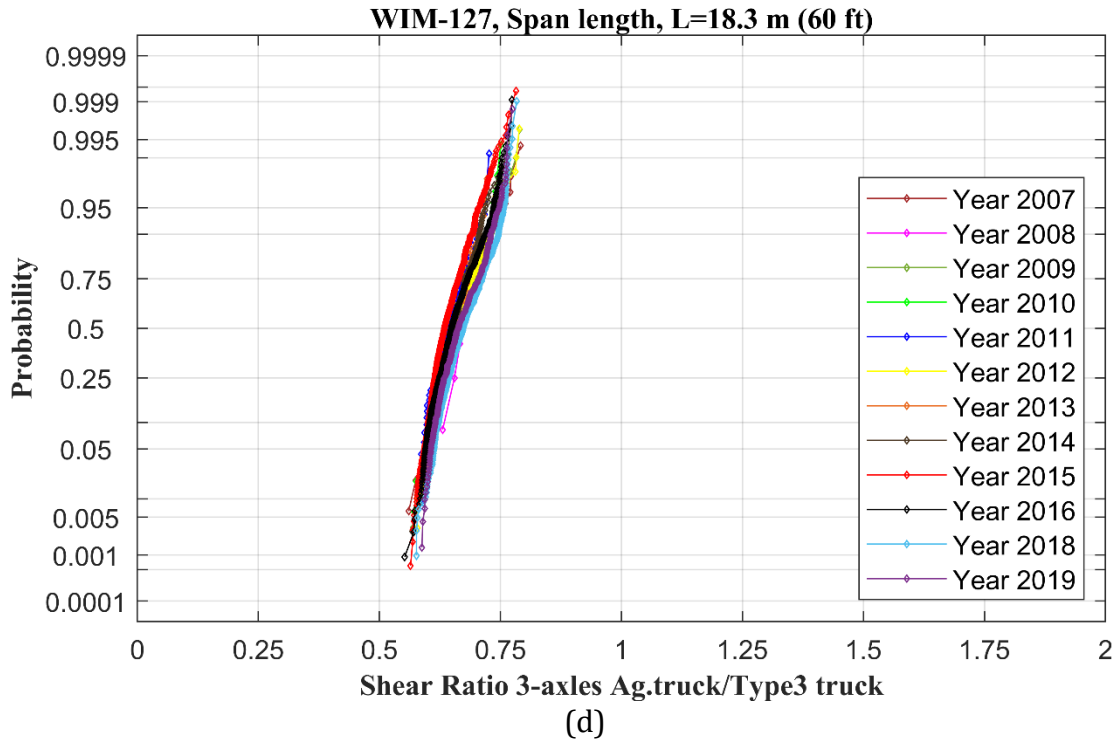
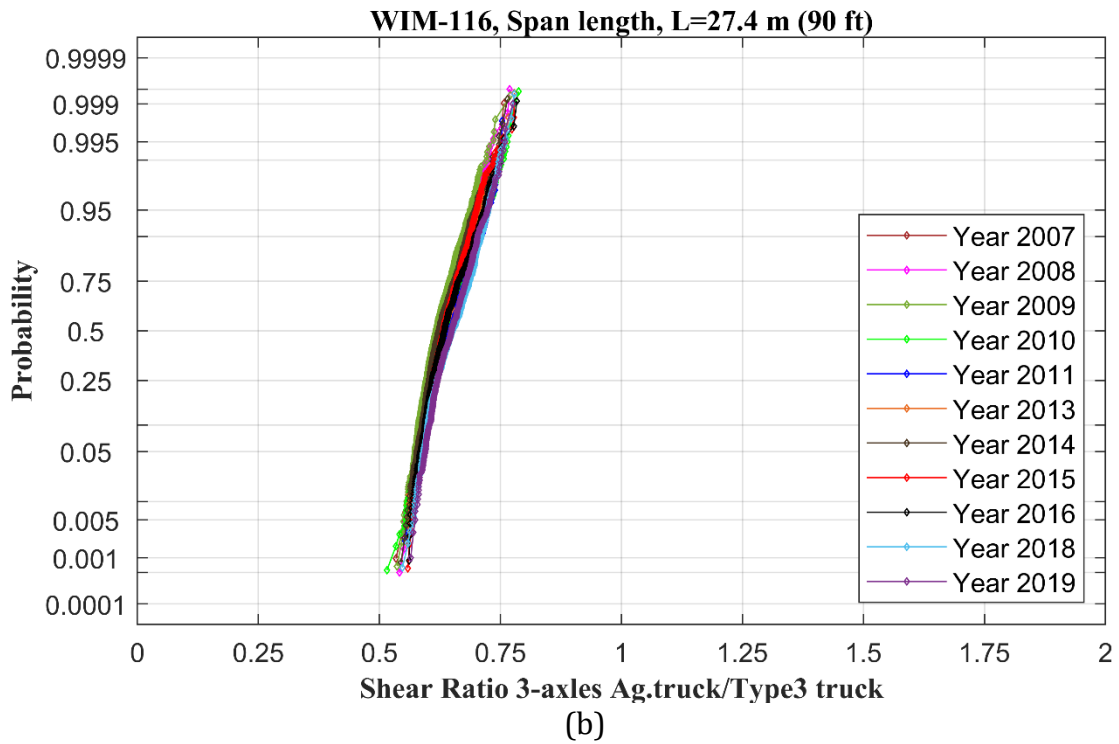
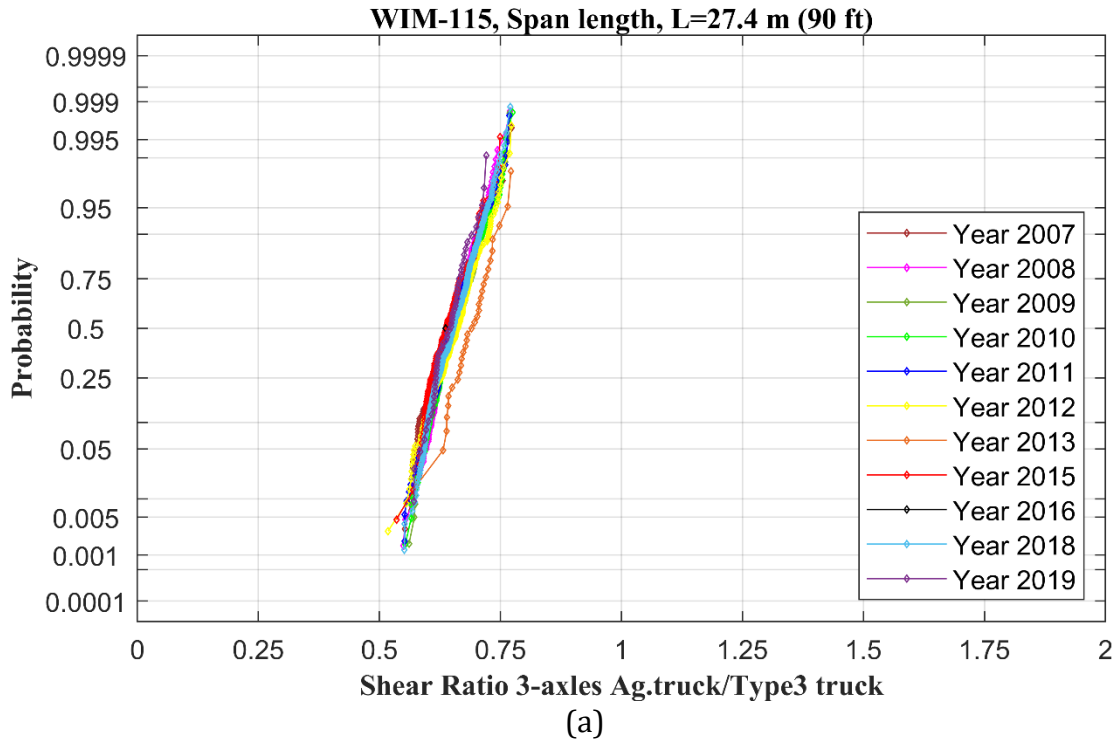
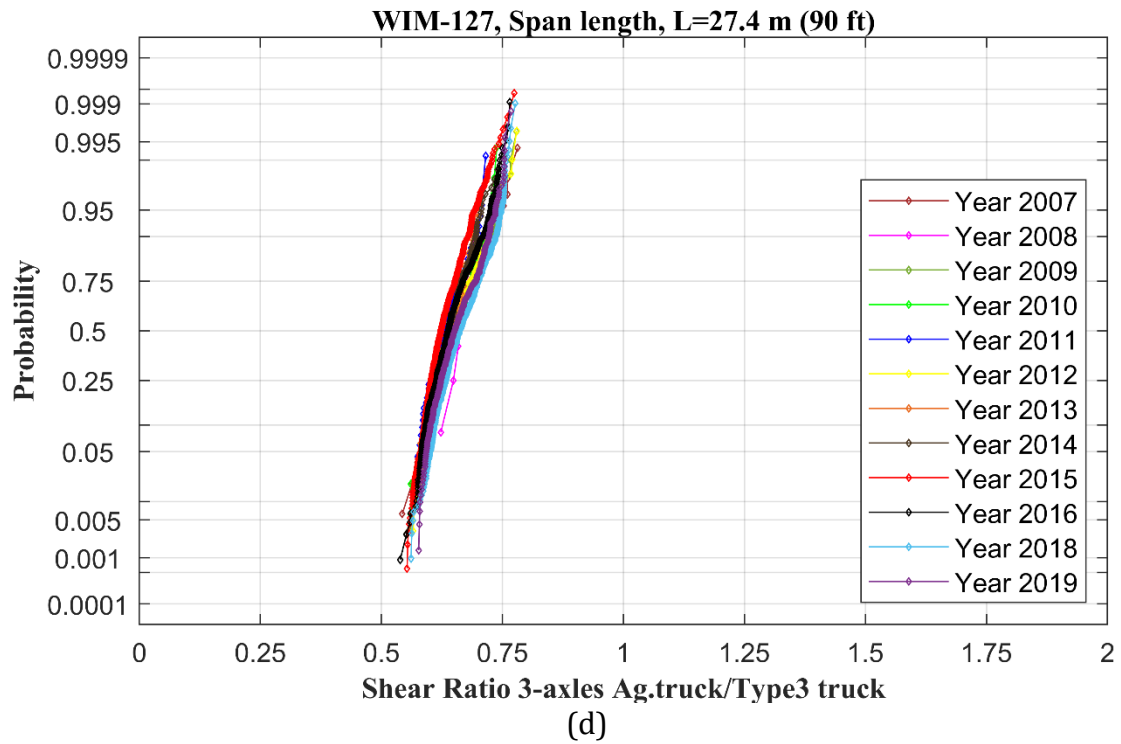
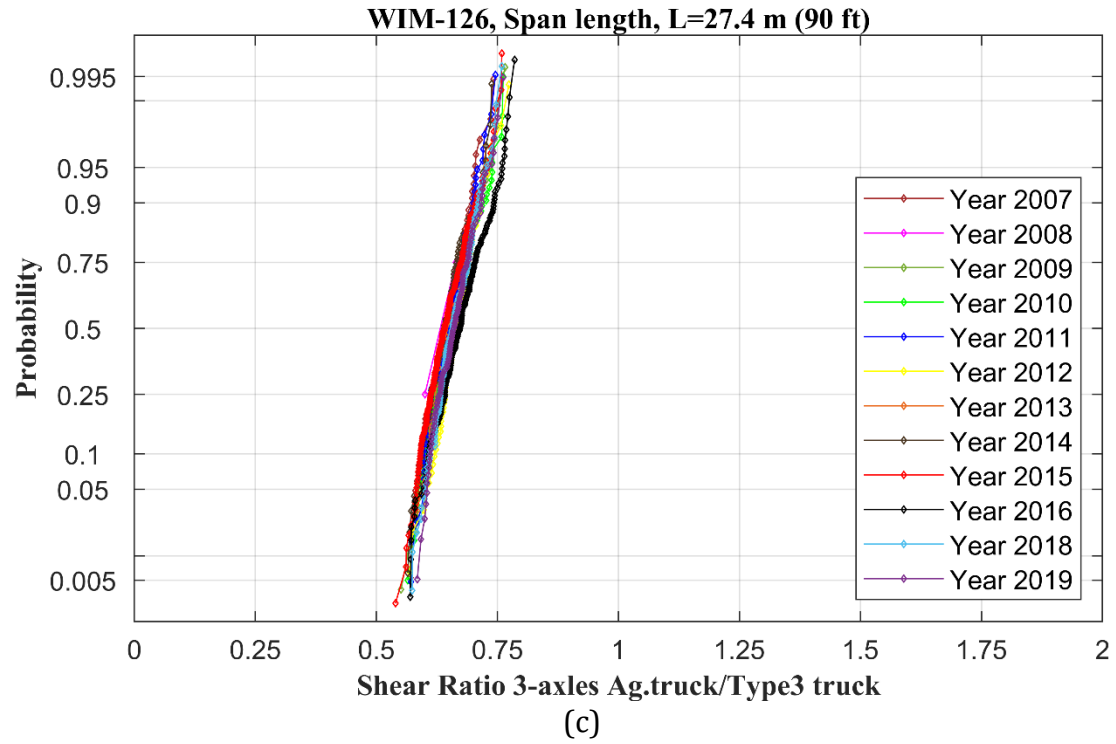


Figure 4-D.2. CDF of shear ratio for 3-axles agricultural vehicles to Type 3 vehicle for span length 18.3 m (60 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





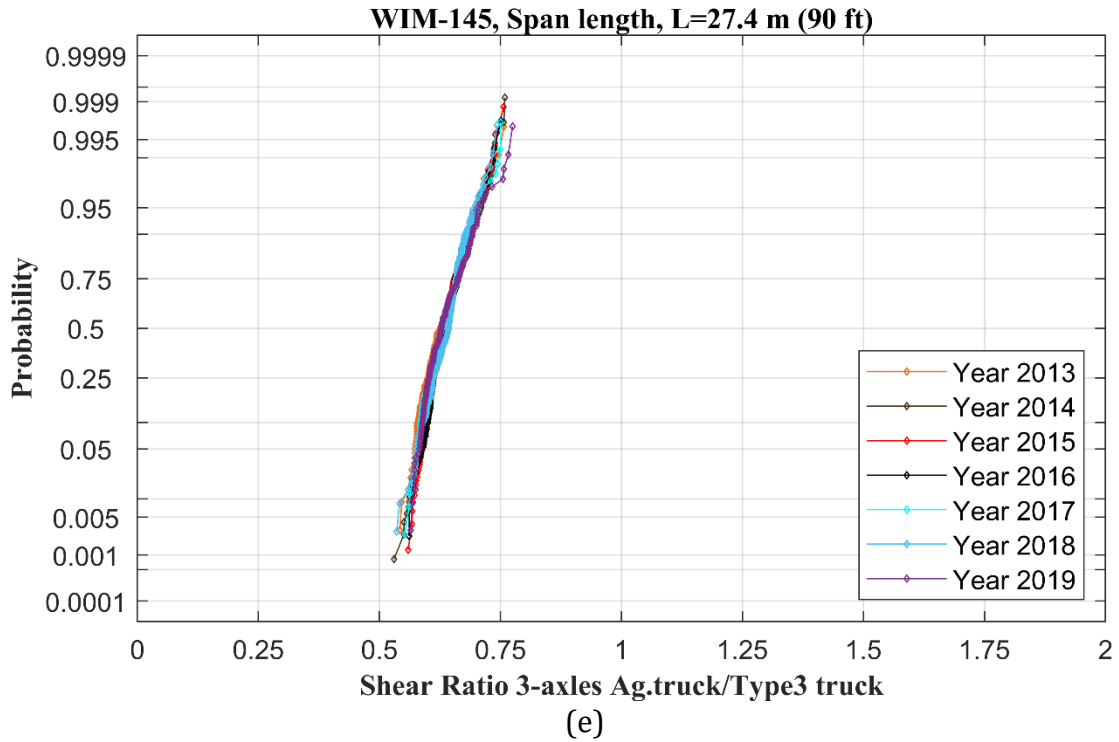
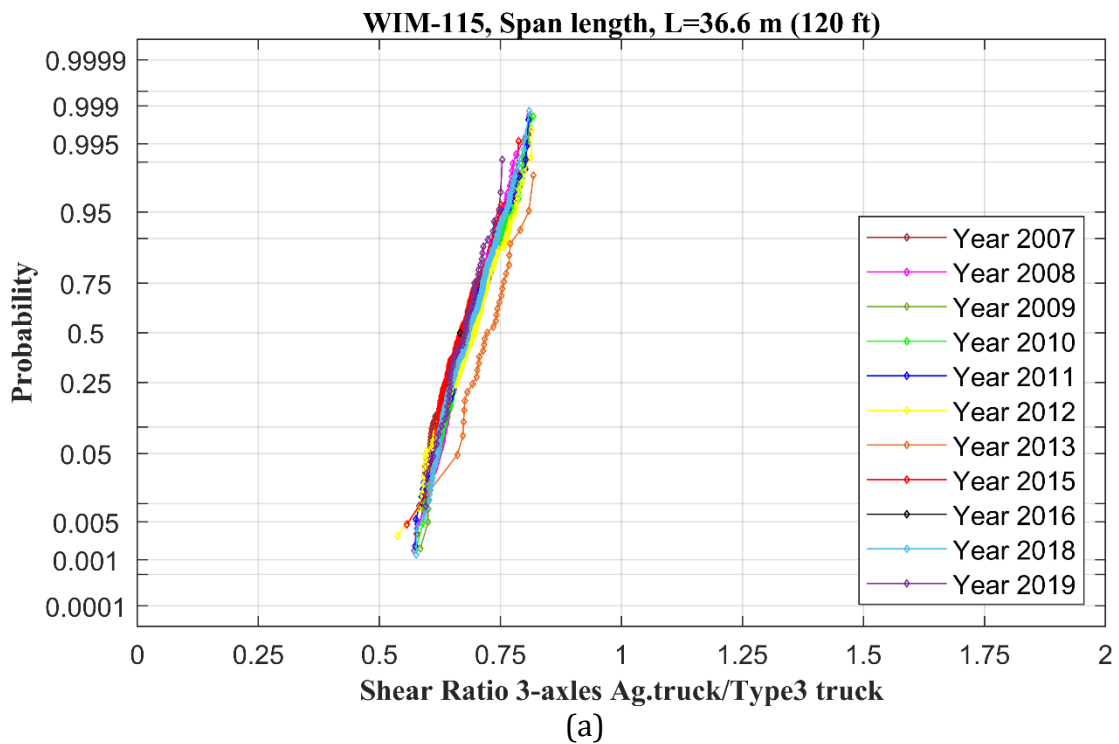
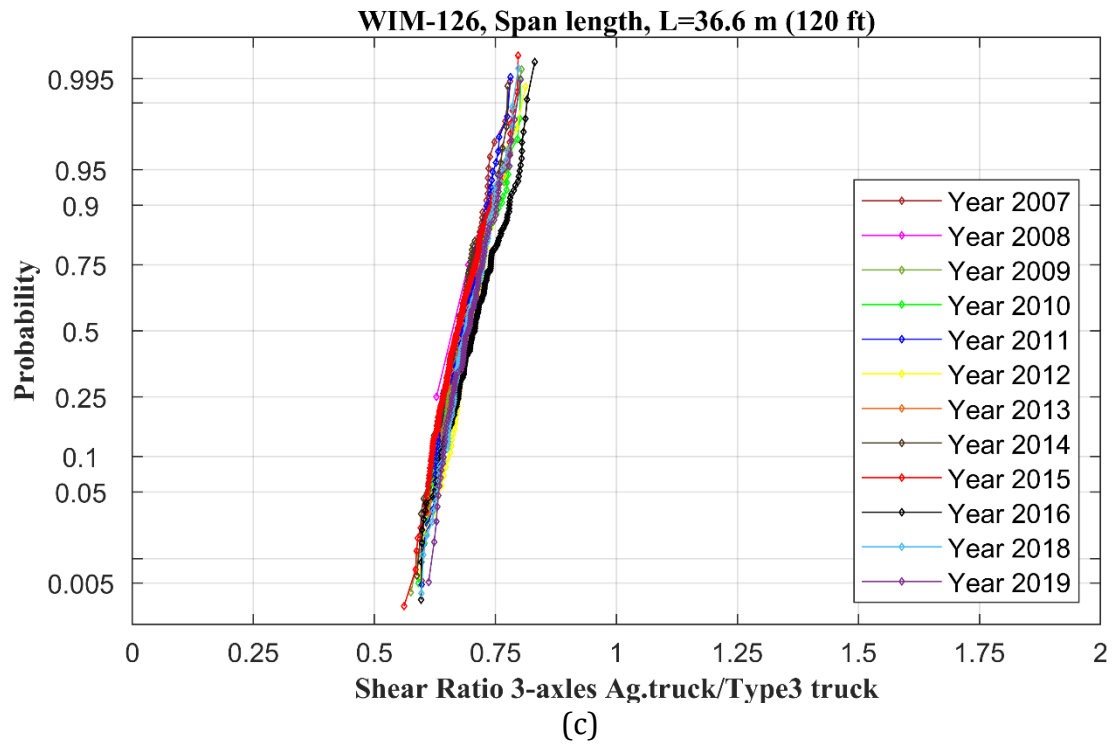
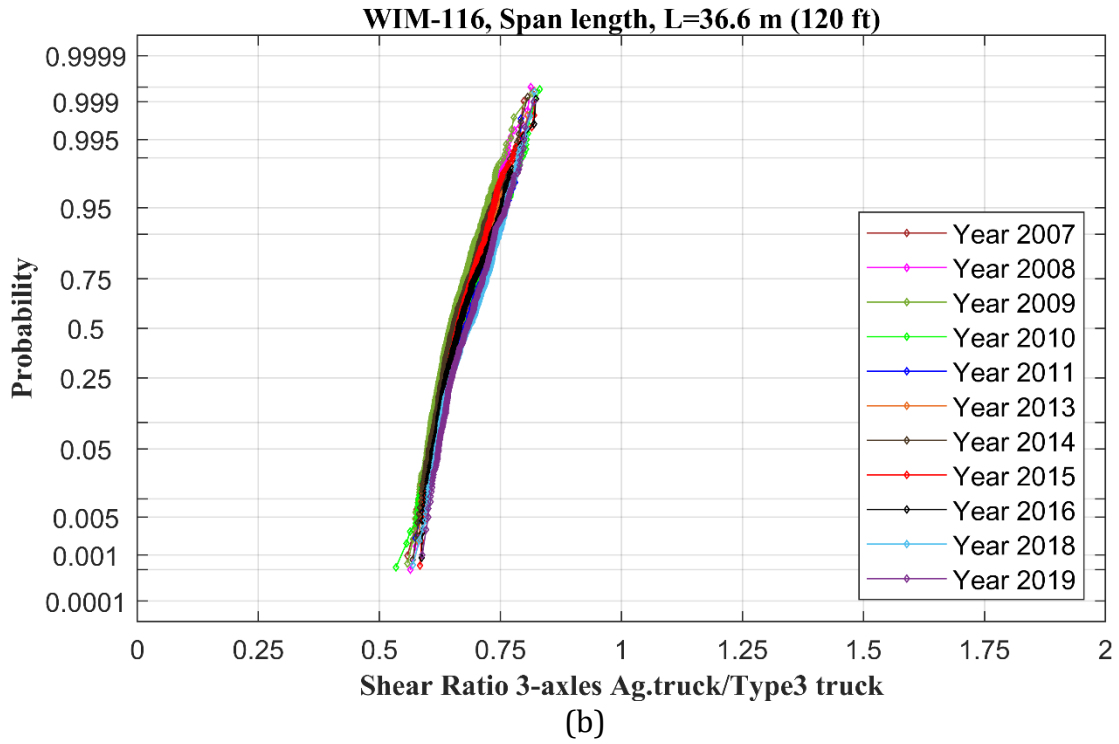


Figure 4-D.3. CDF of shear ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length 27.4 m (90 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





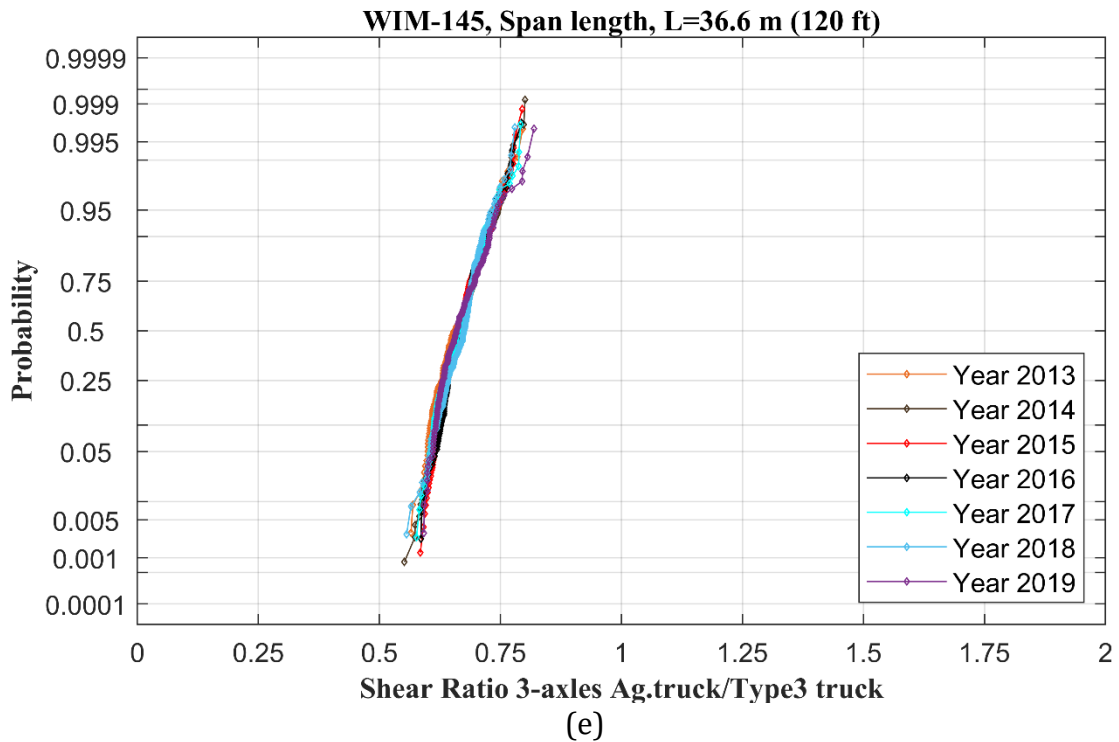
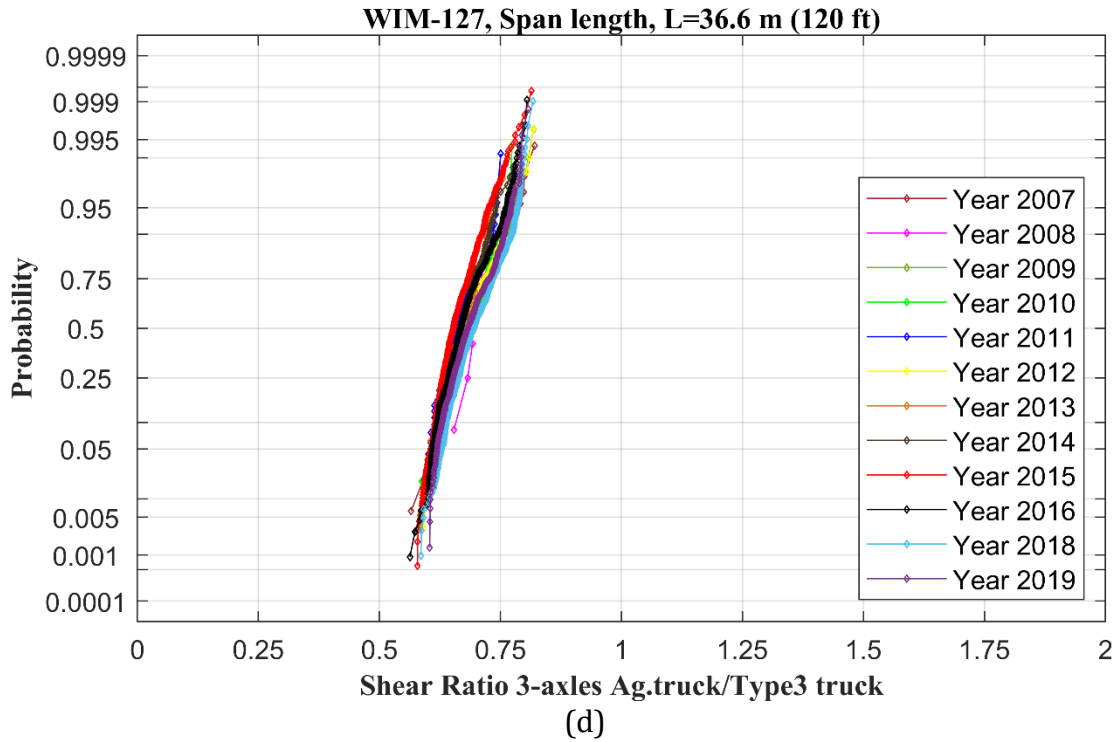
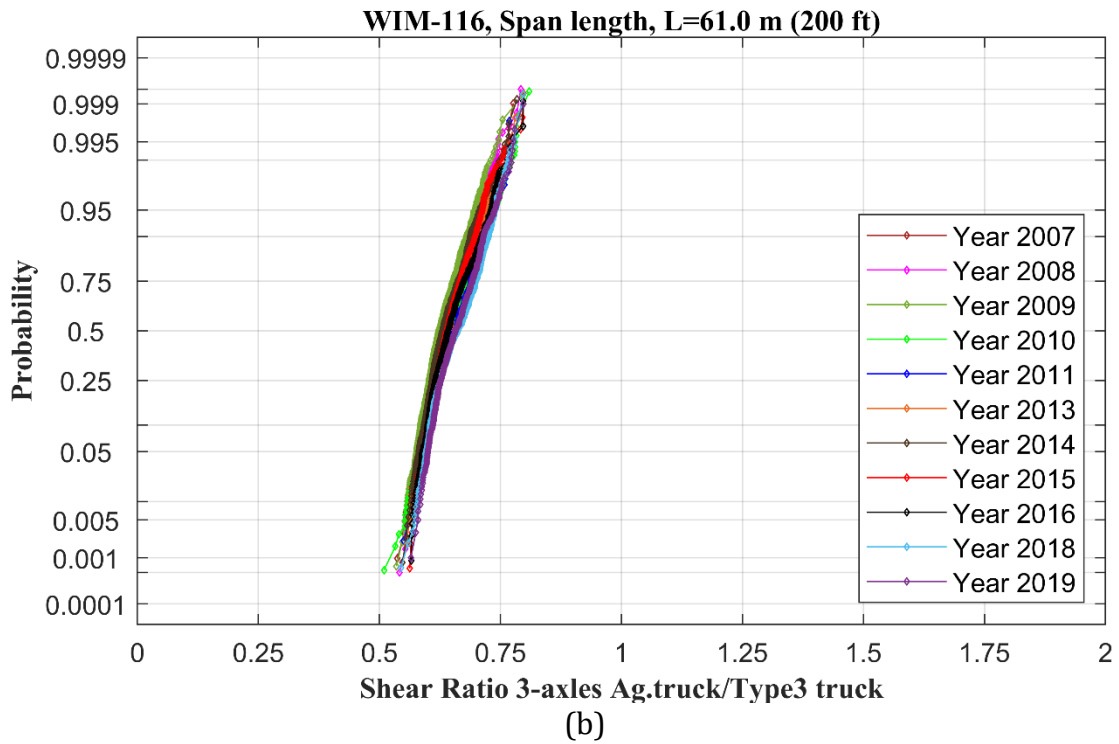
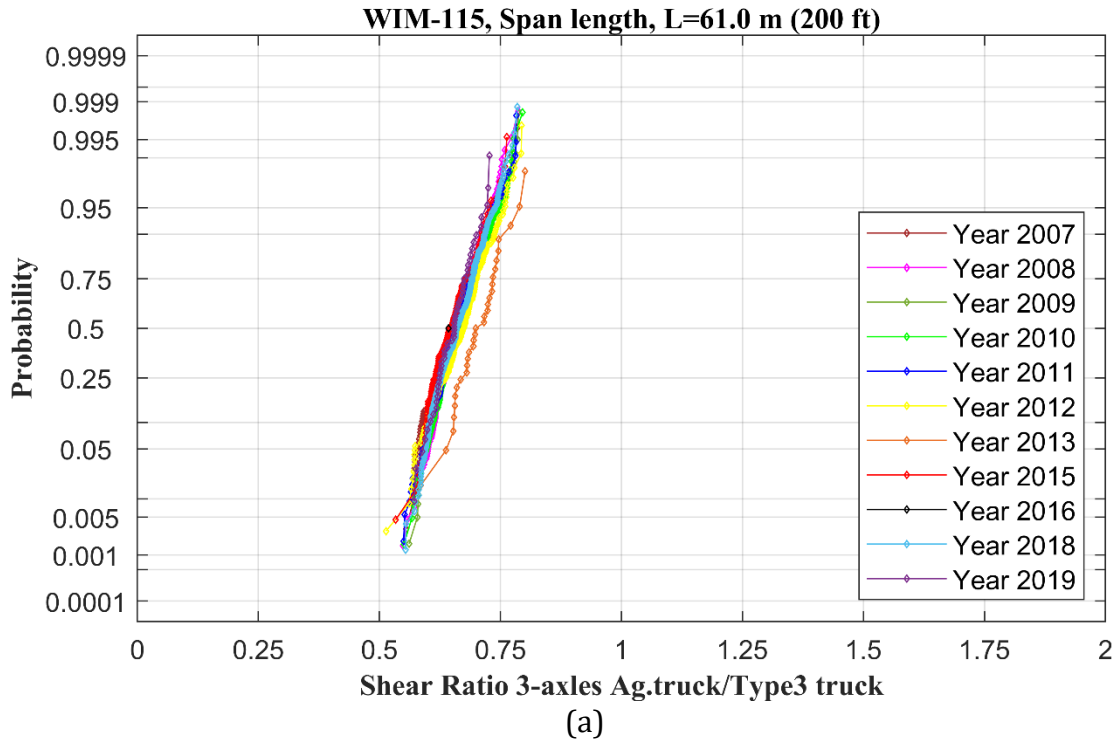
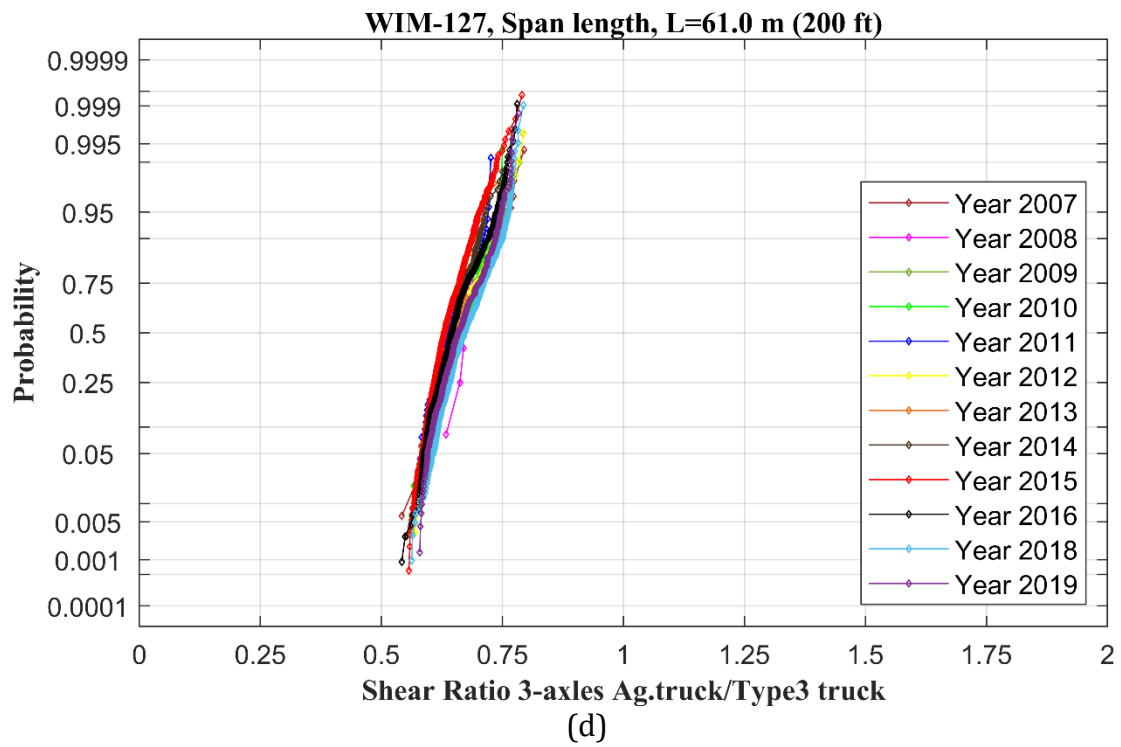
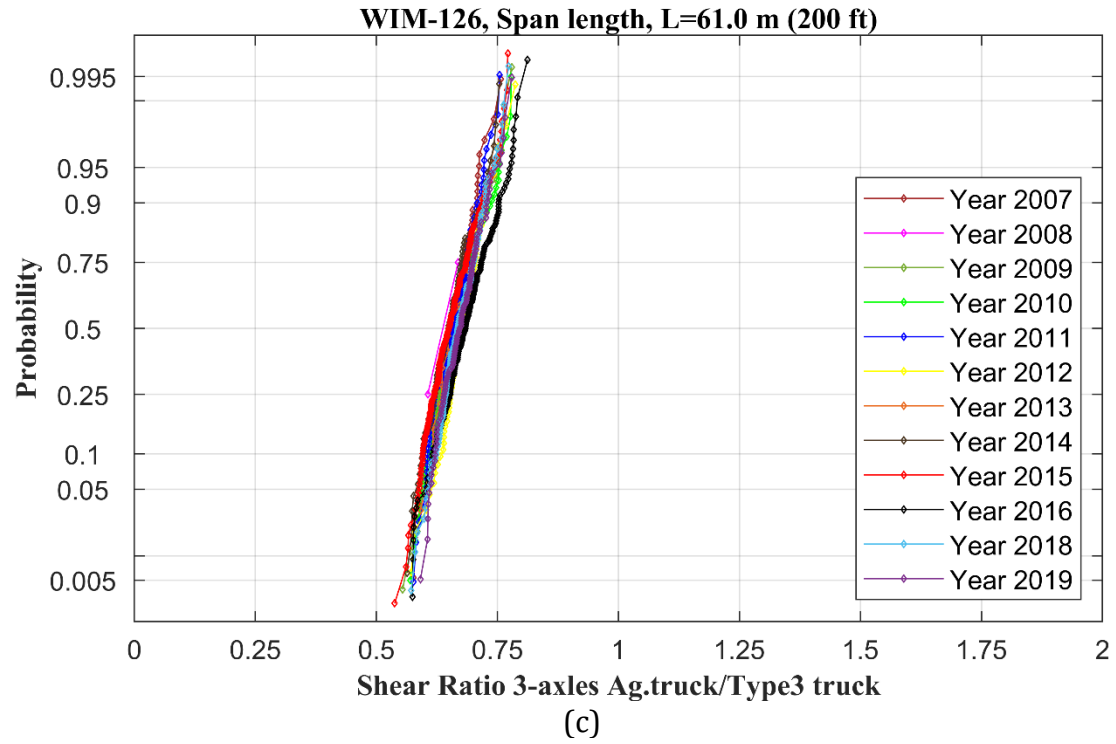


Figure 4-D.4. CDF of shear ratio for 3-axes agricultural vehicles to Type 3 vehicle for span length 36.6 m (120 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.





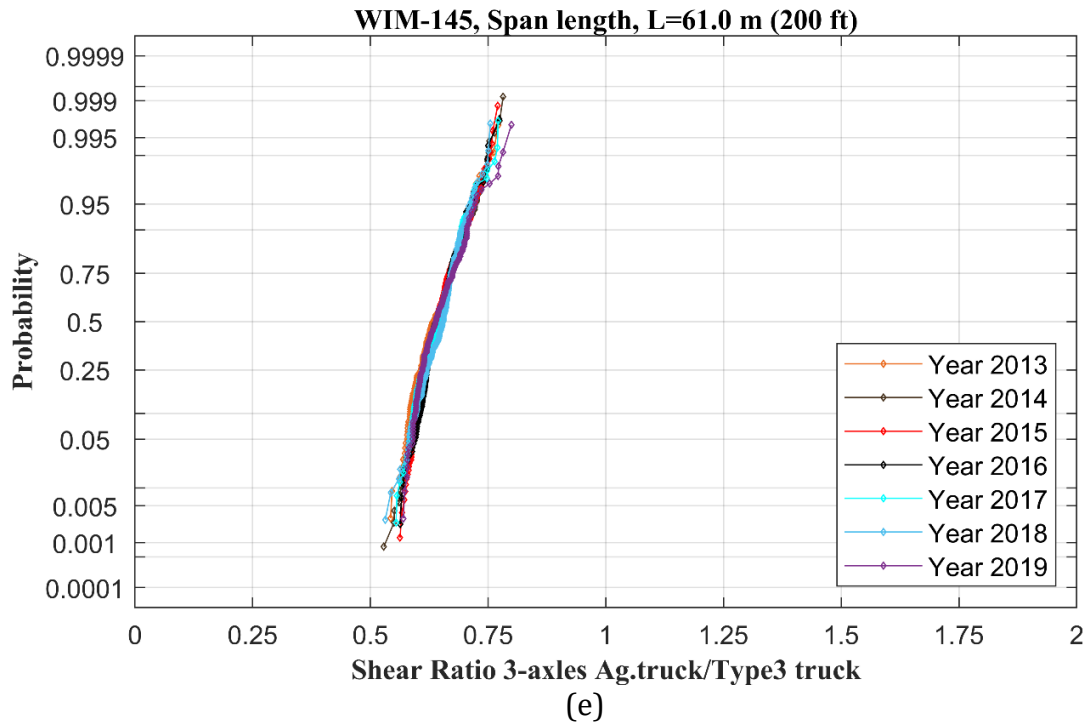
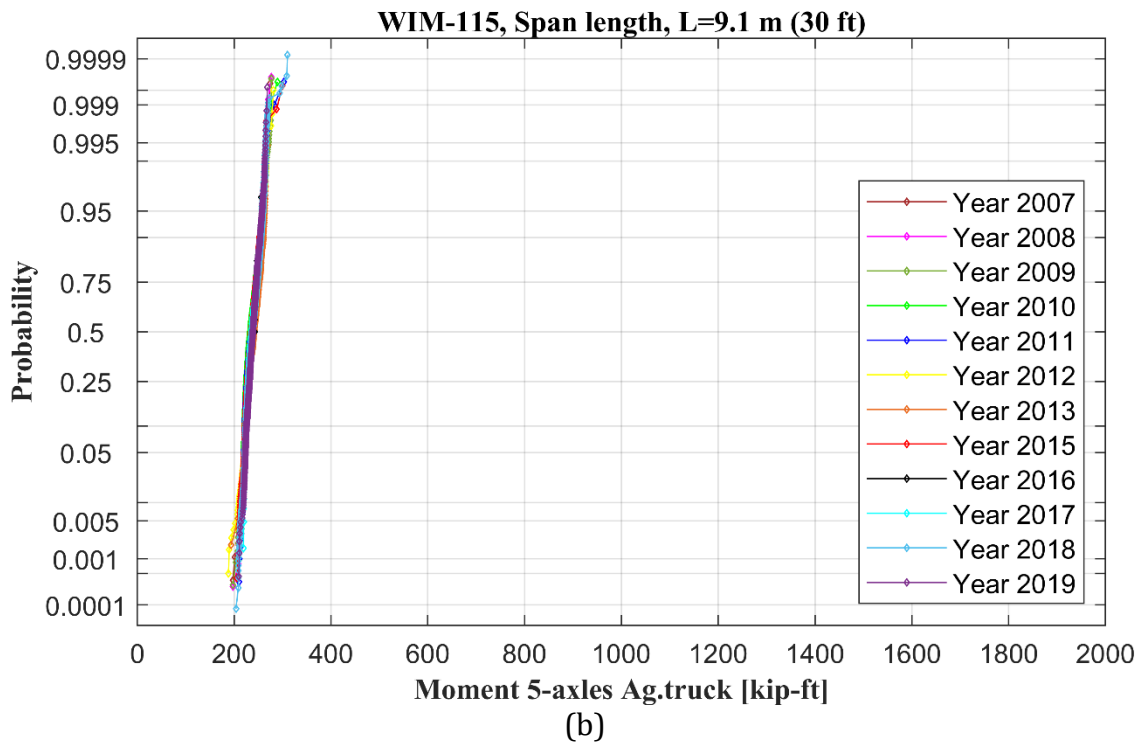
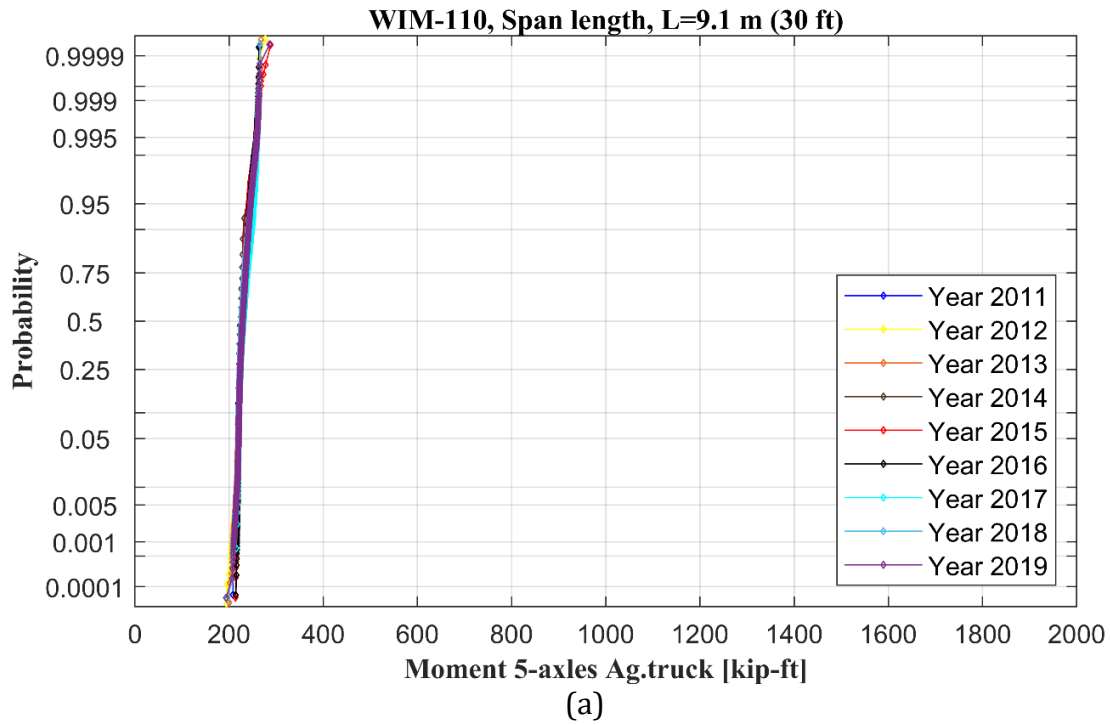
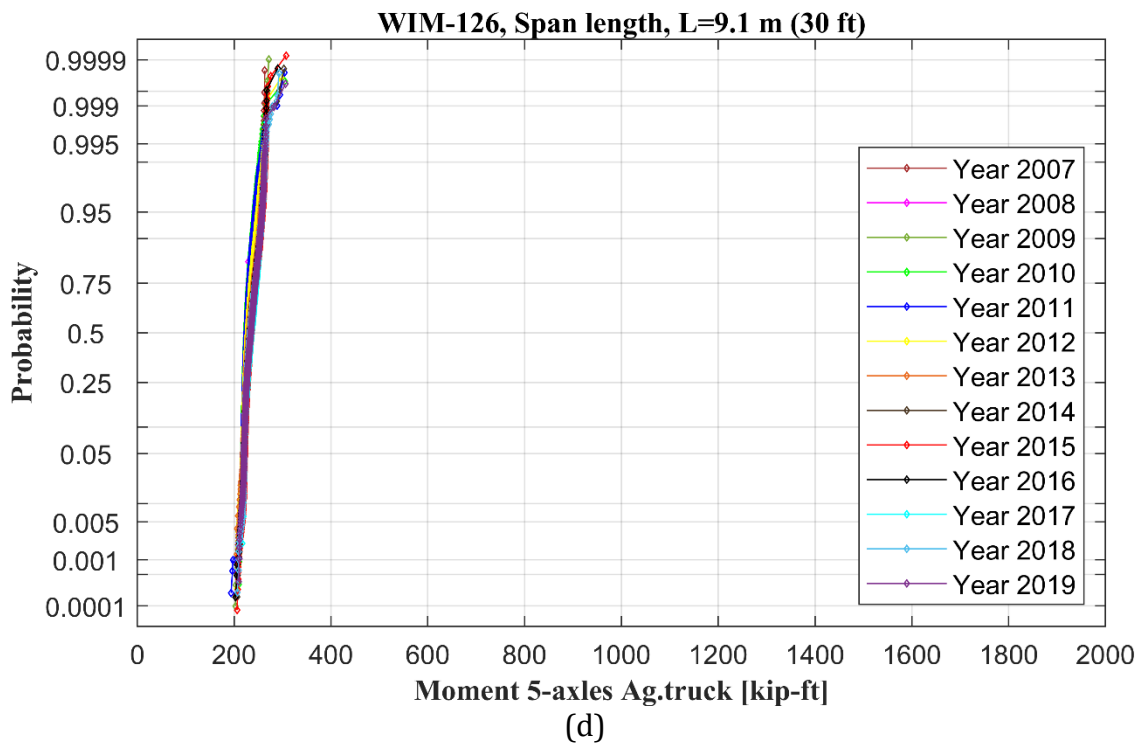
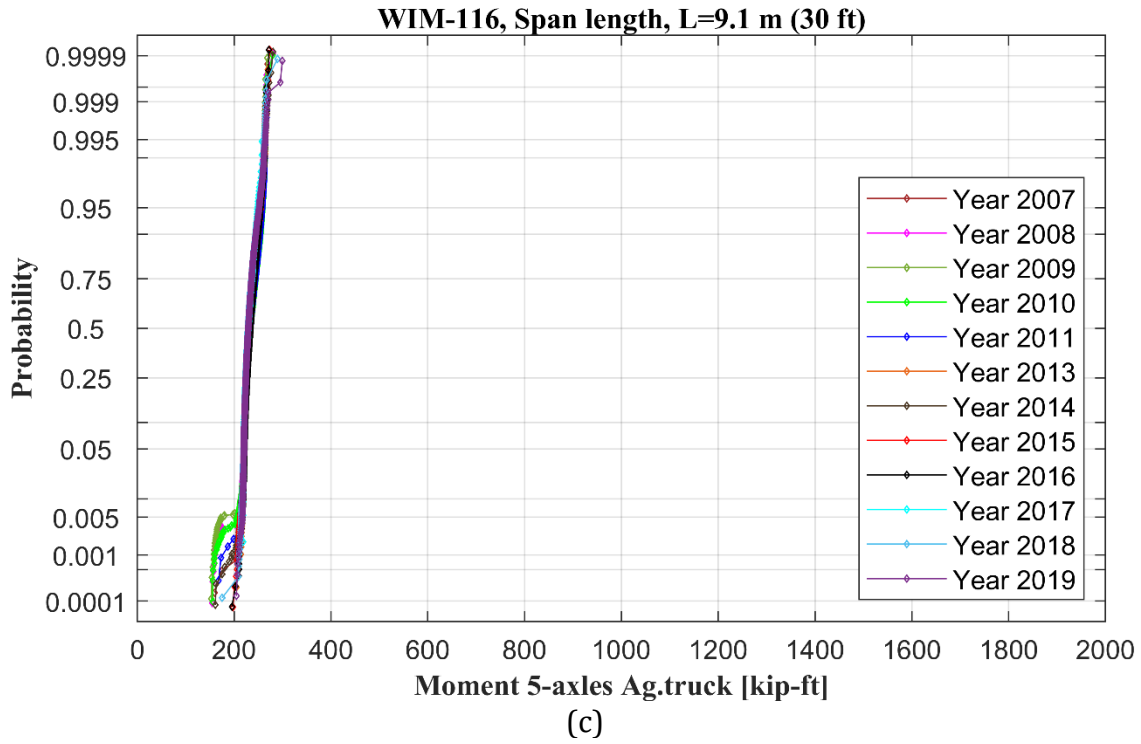


Figure 4-D.5. CDF of shear ratio for 3-axles agricultural vehicles to Type 3 vehicle for span length 61 m (200 ft) for (a) WIM-115 (b) WIM-116, (c) WIM-126, (d) WIM-127, and (e) WIM-145.

Appendix 4-E. CDF of moment effects for 5-axle agricultural vehicles





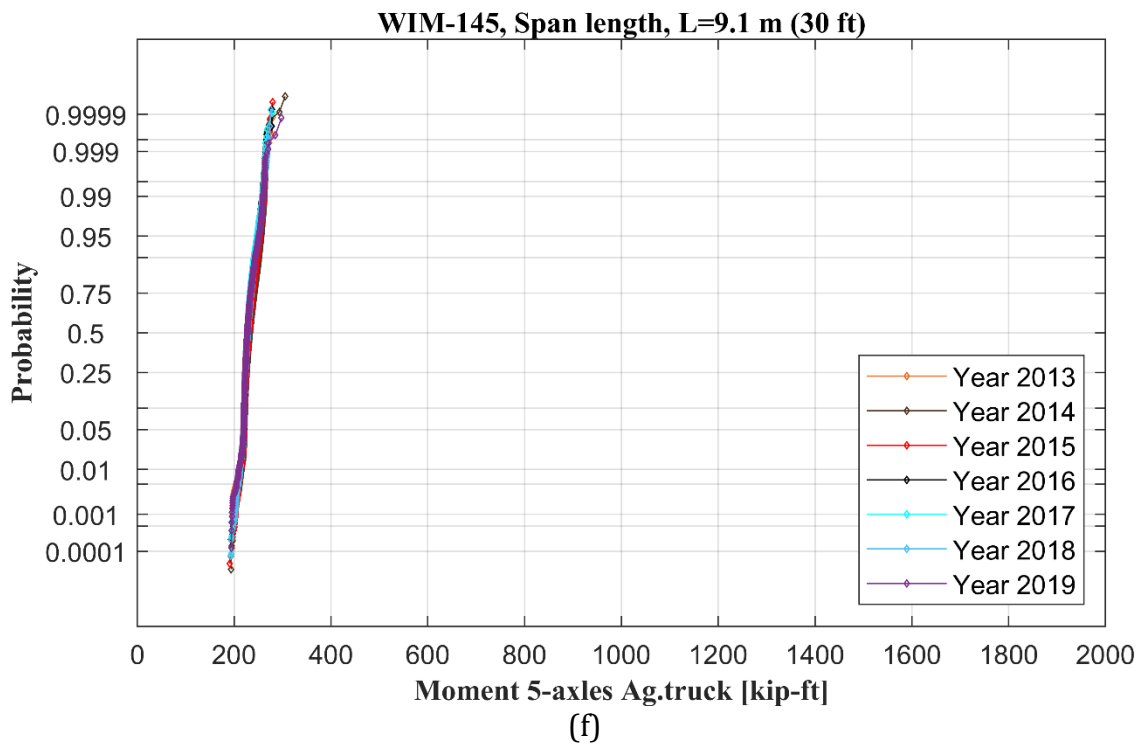
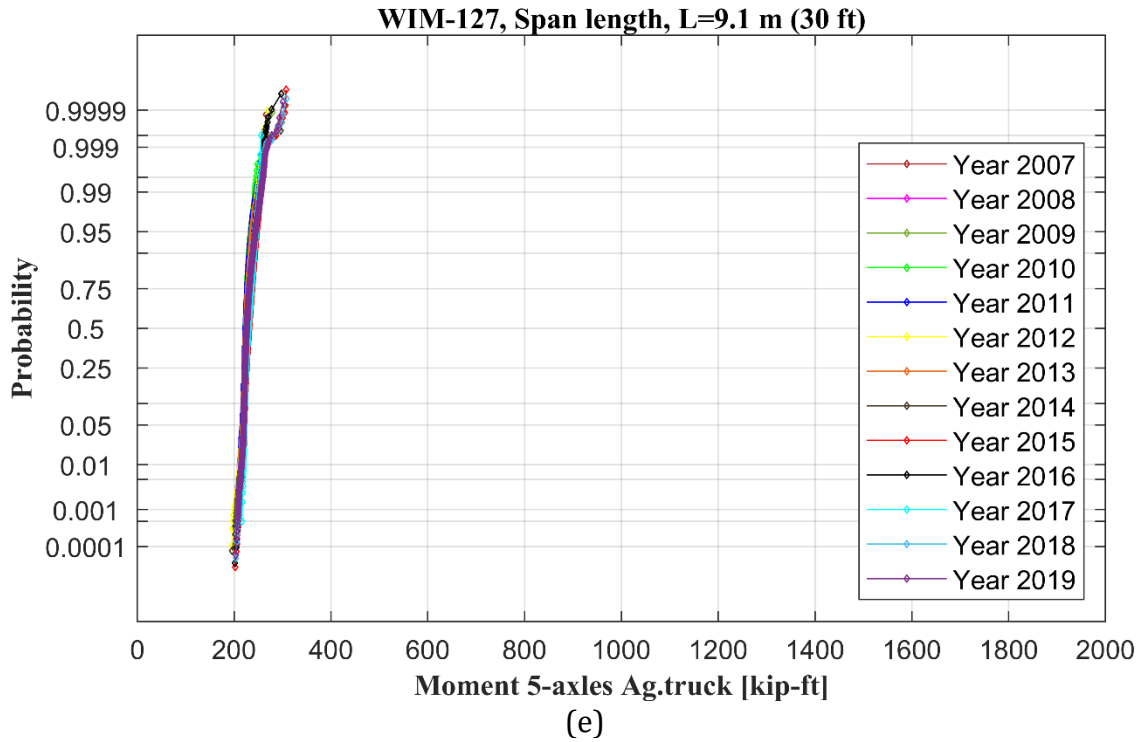
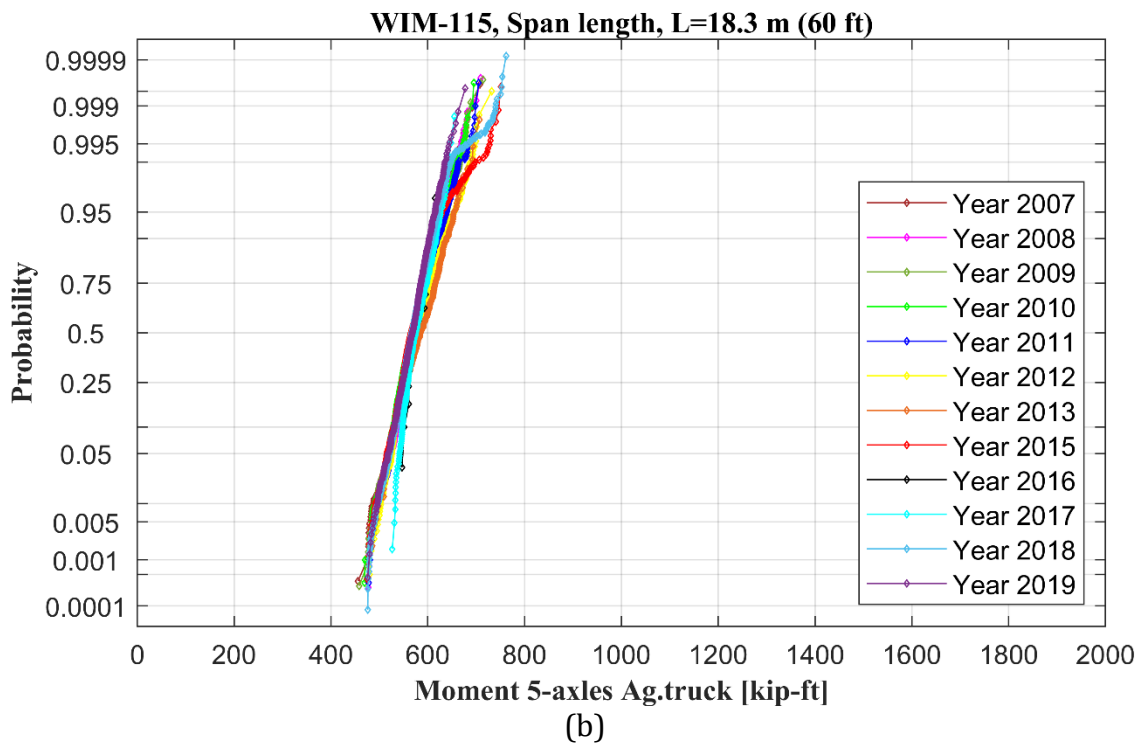
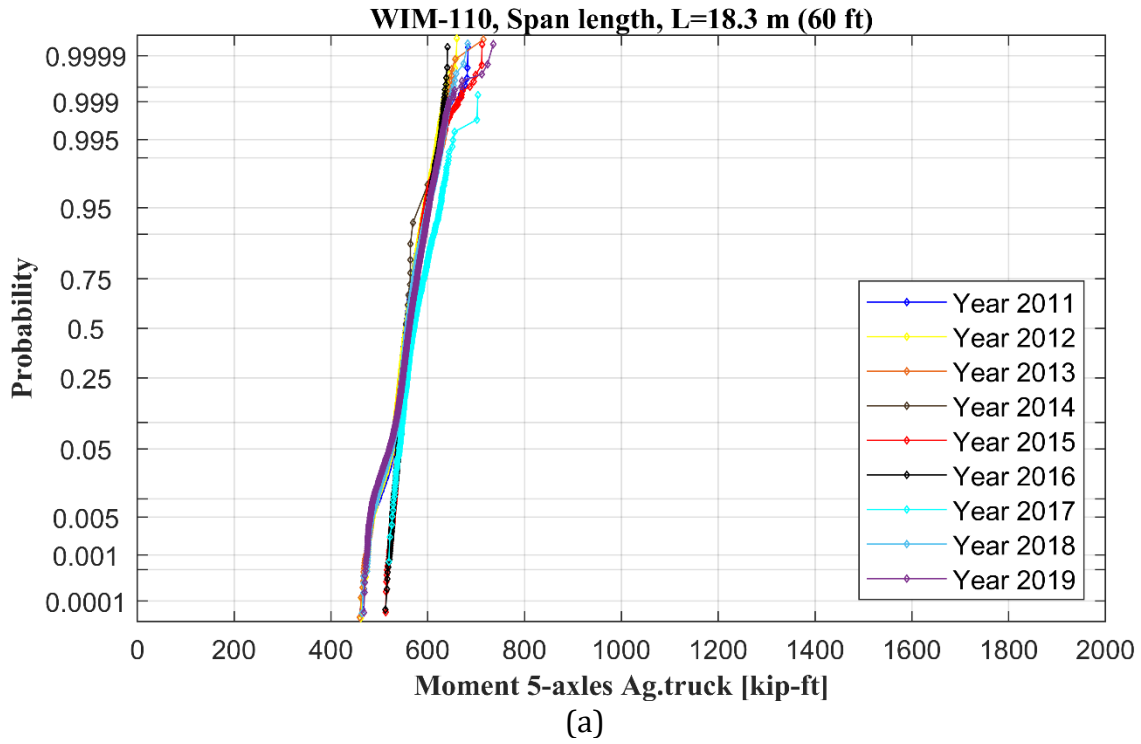
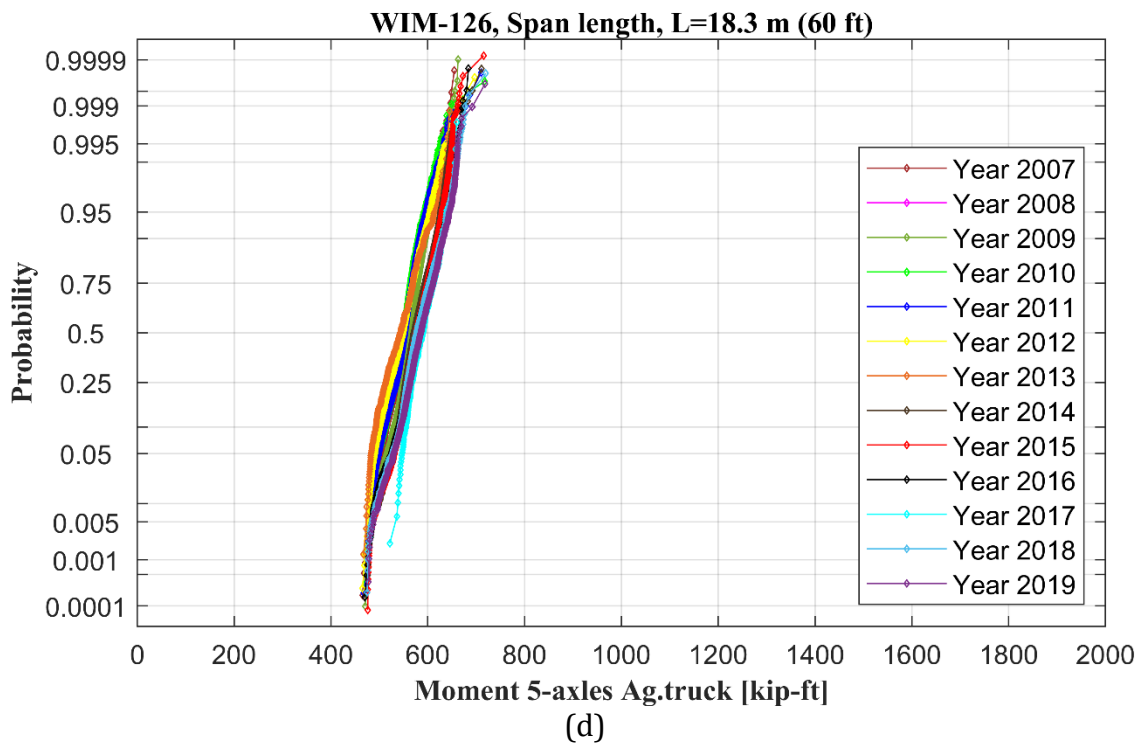
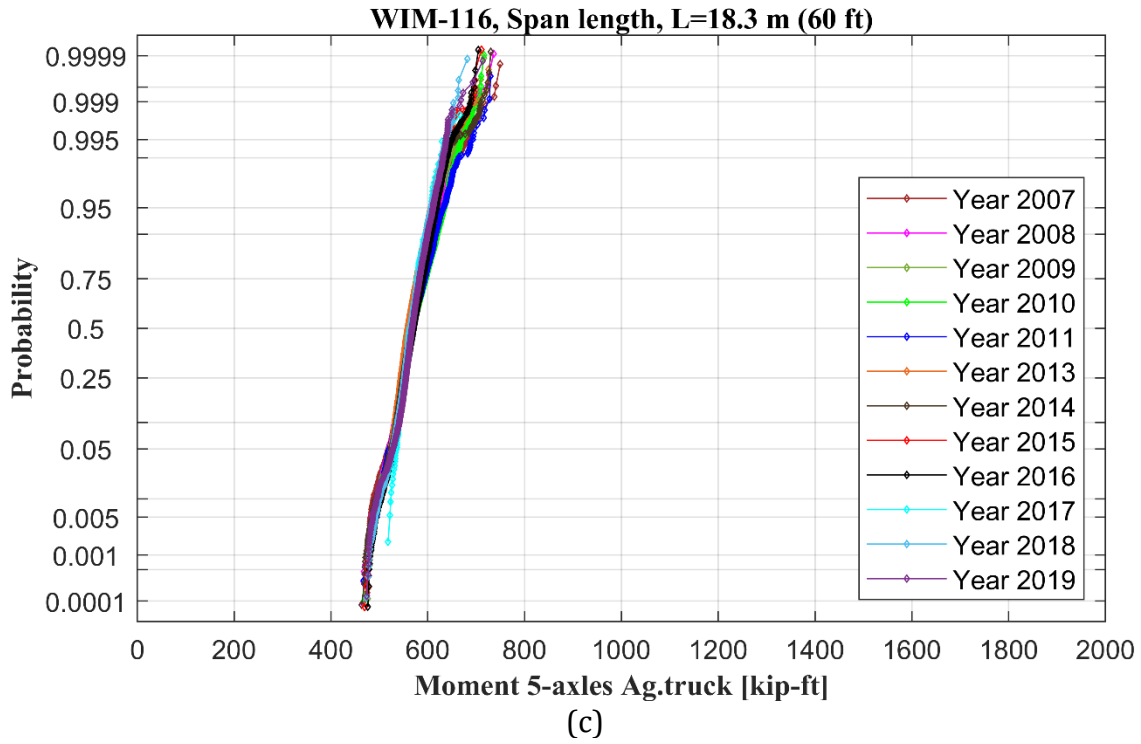


Figure 4-E.1. CDF of moment for 5-axes agricultural vehicles for span length of 9.1 m (30 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





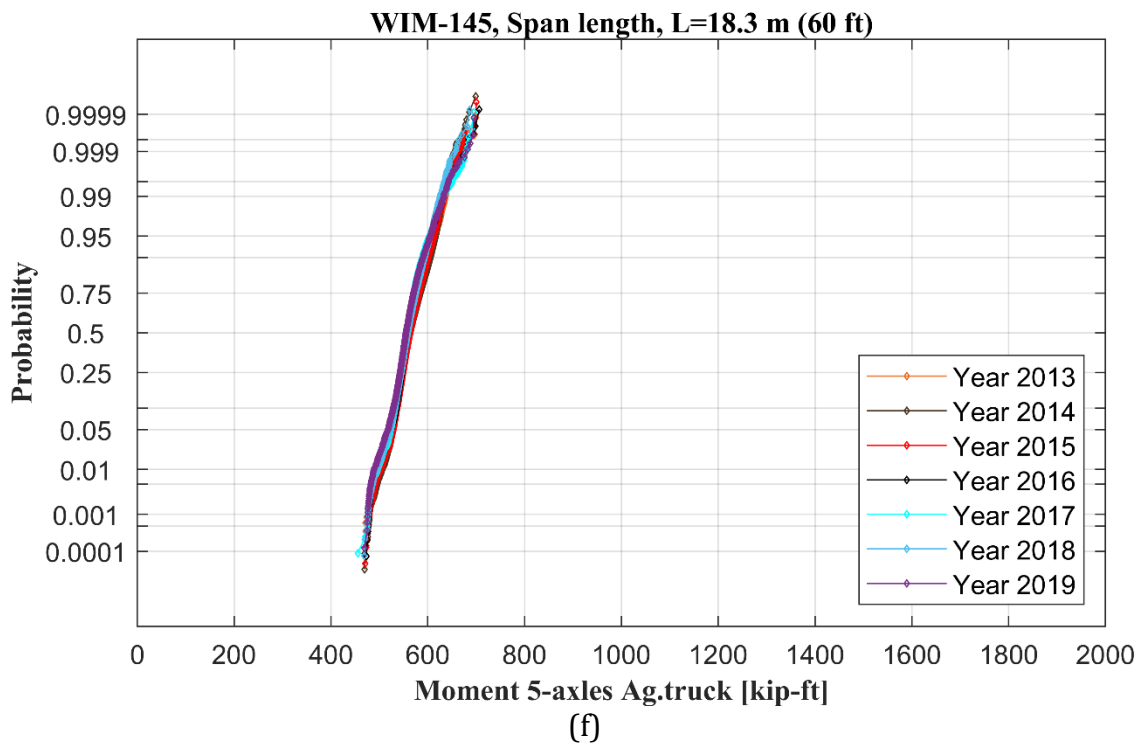
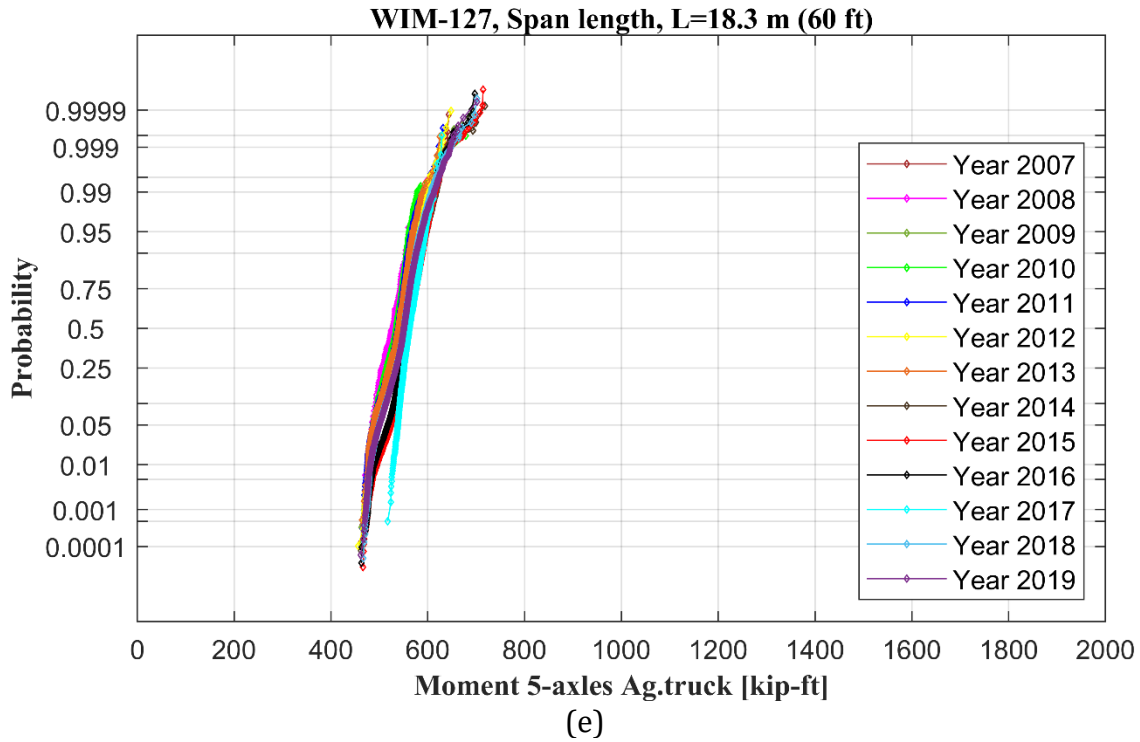
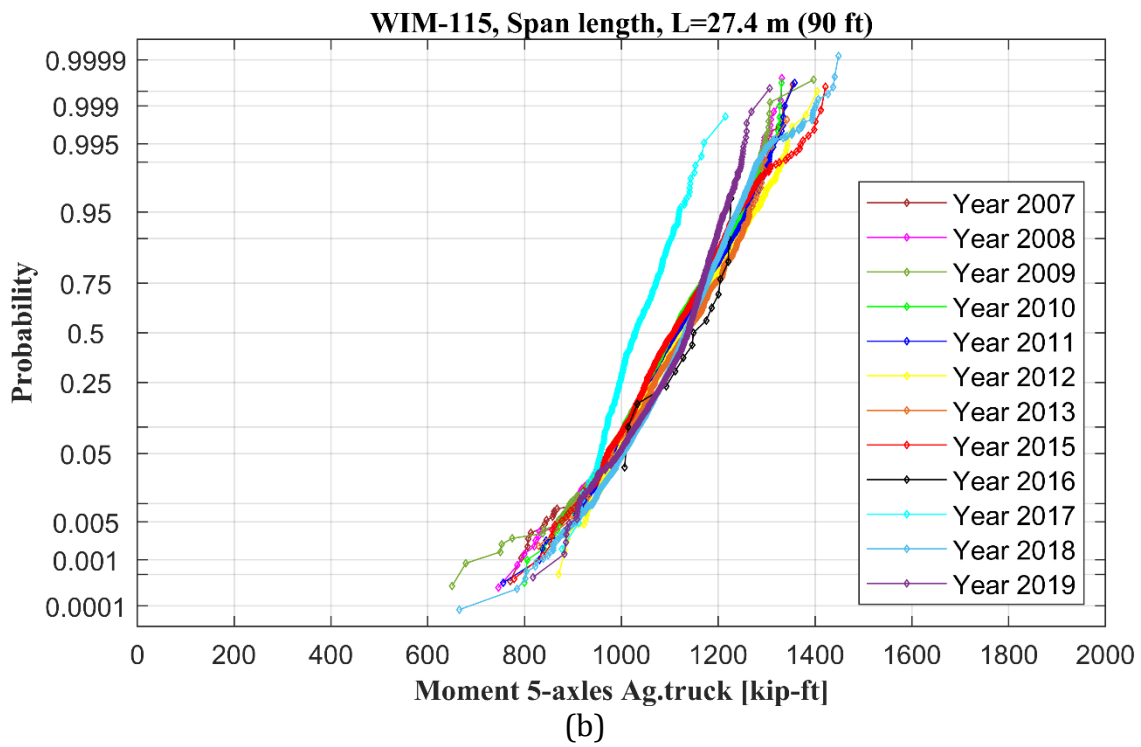
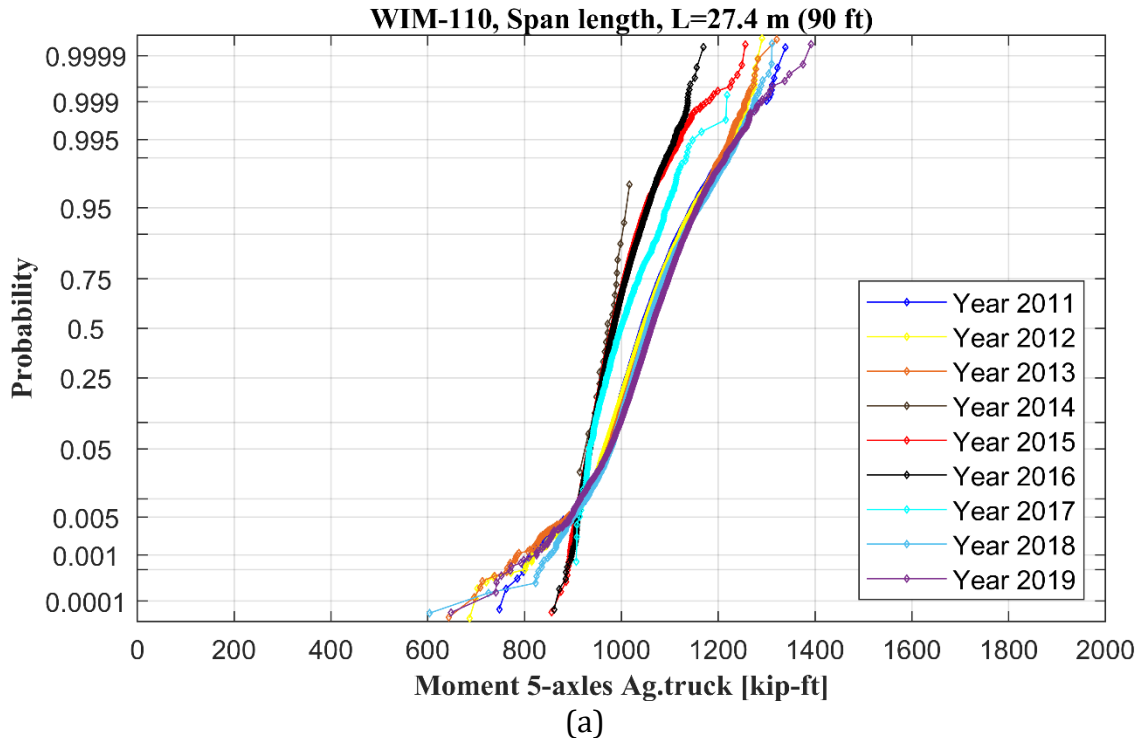
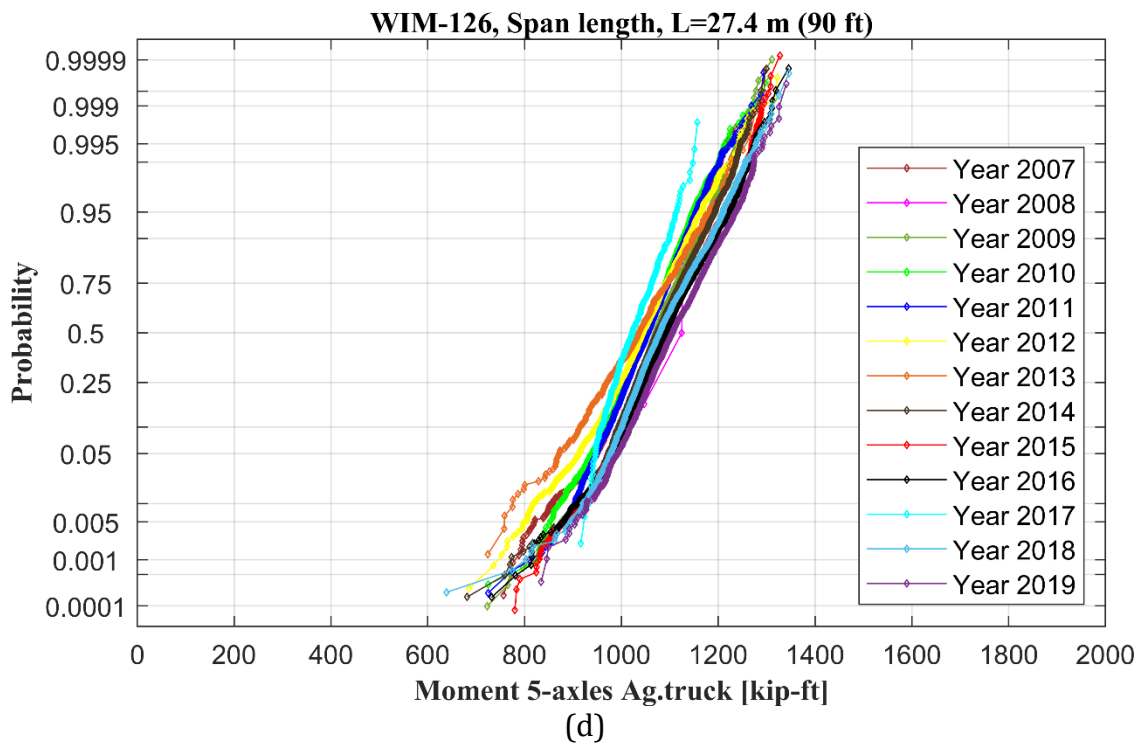
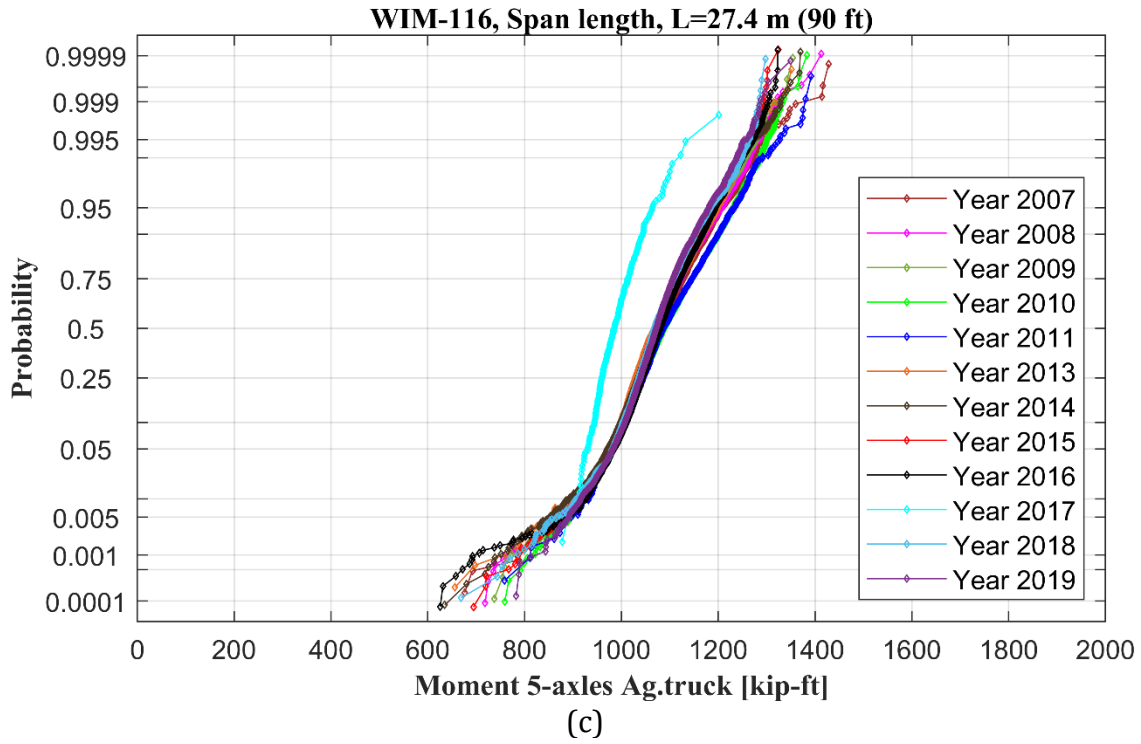


Figure 4-E.2. CDF of moment for 5-axes agricultural vehicles for span length of 18.3 m (60 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





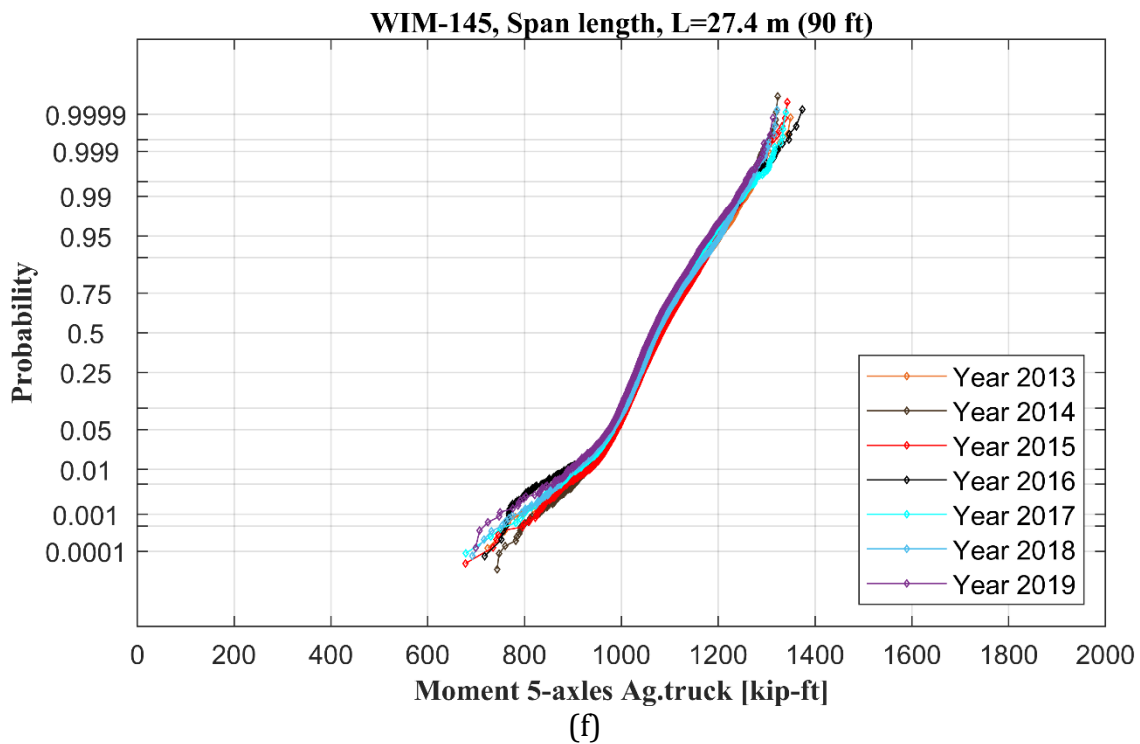
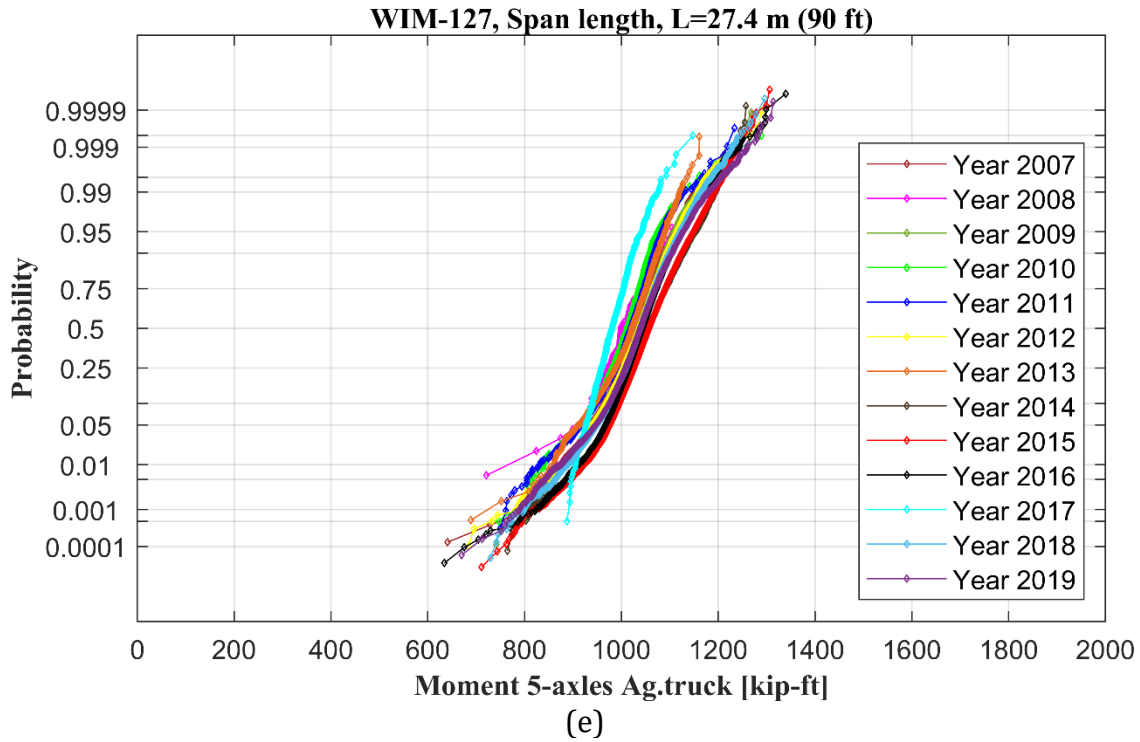
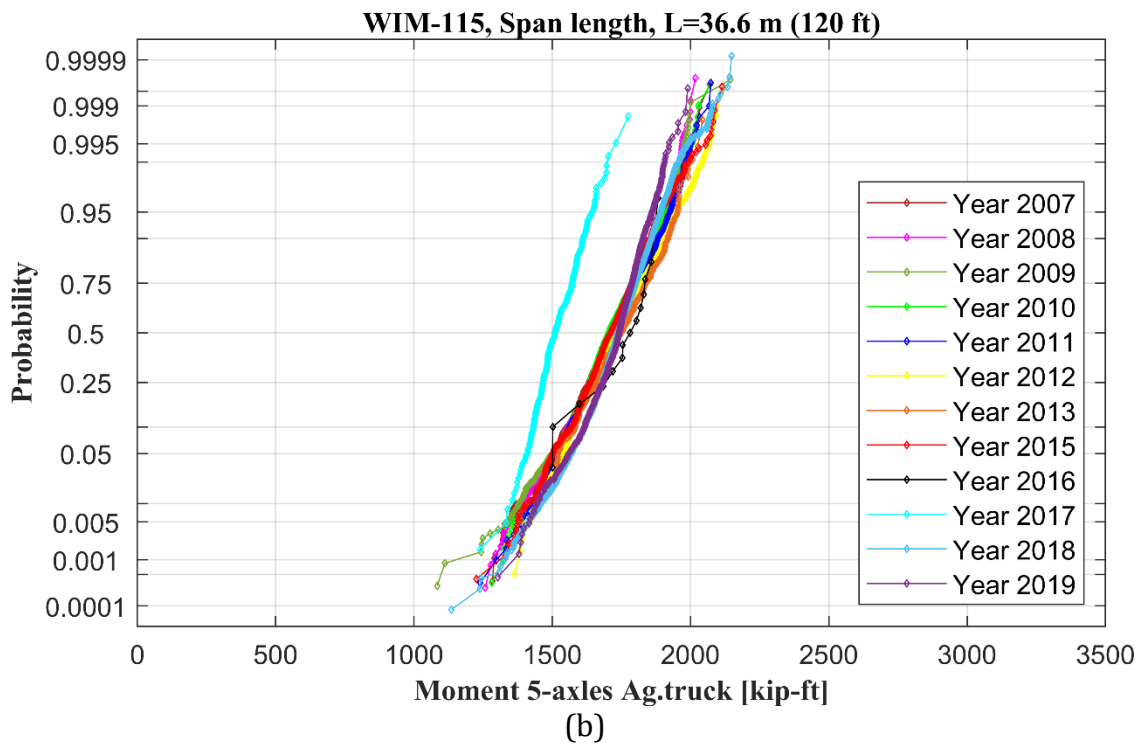
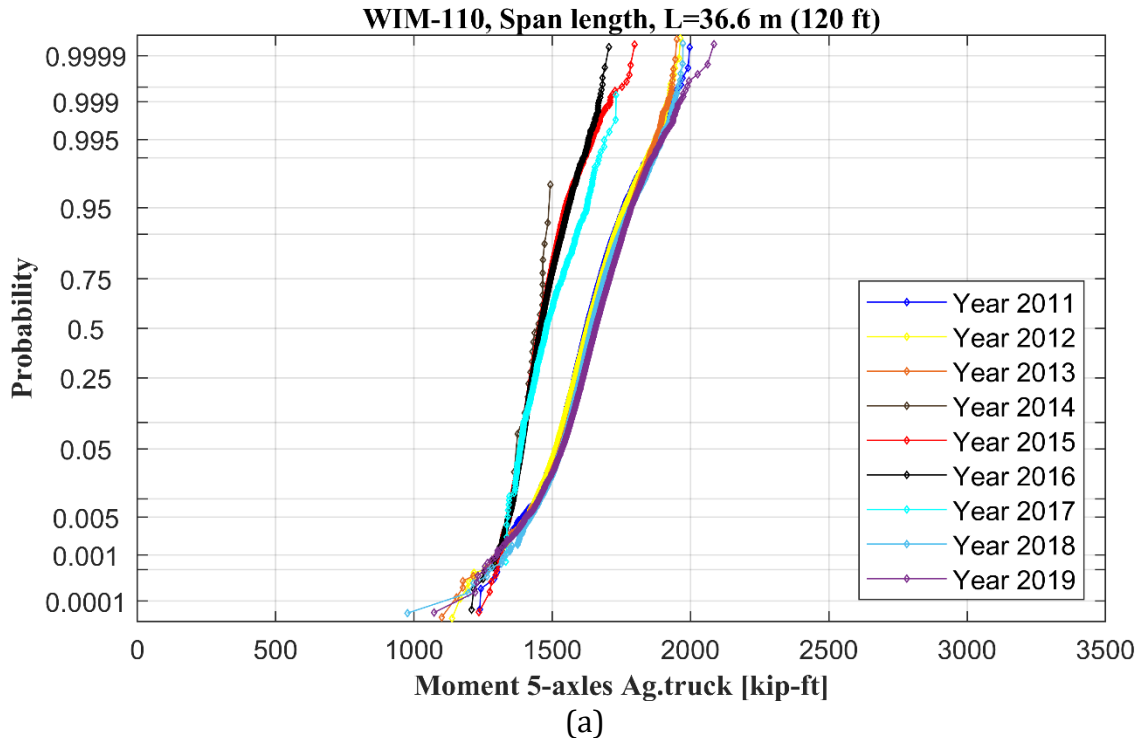
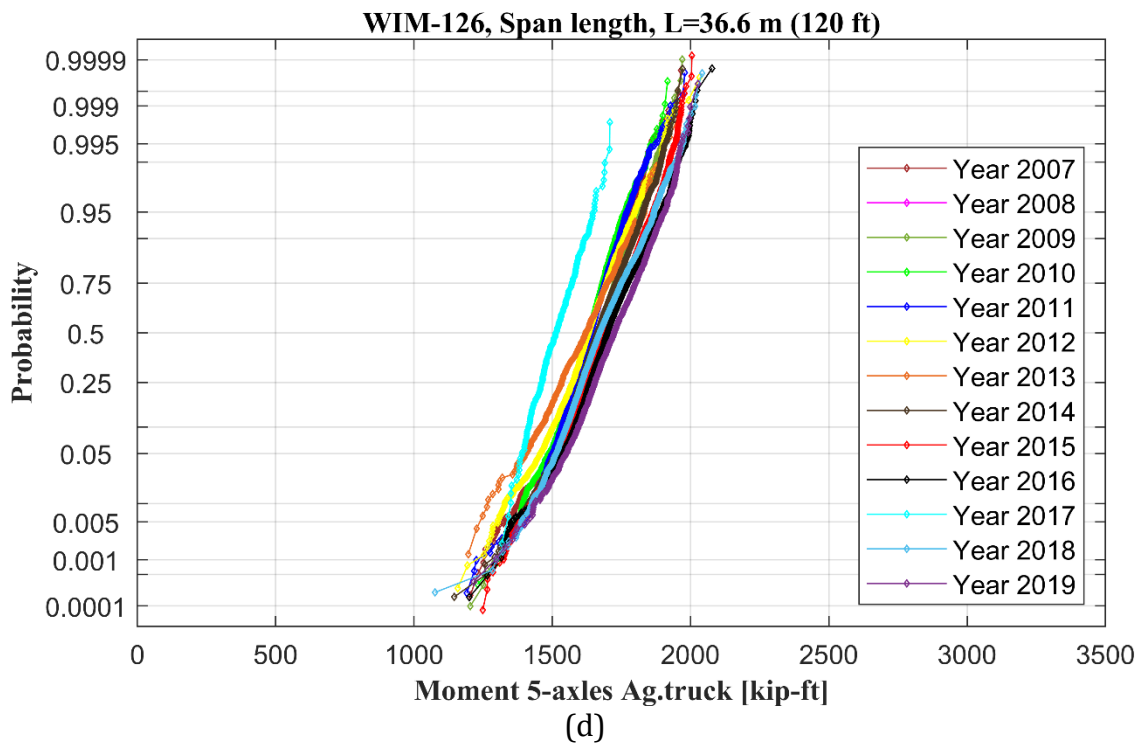
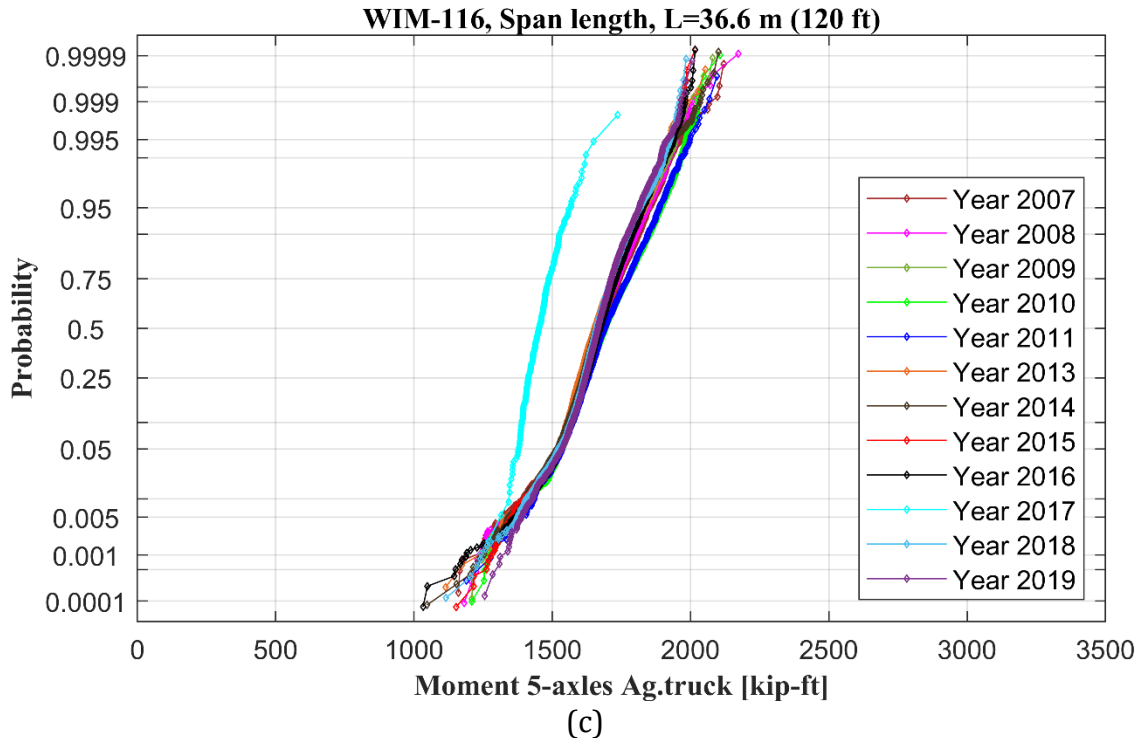


Figure 4-E.3. CDF of moment for 5-axes agricultural vehicles for span length of 27.4 m (90 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





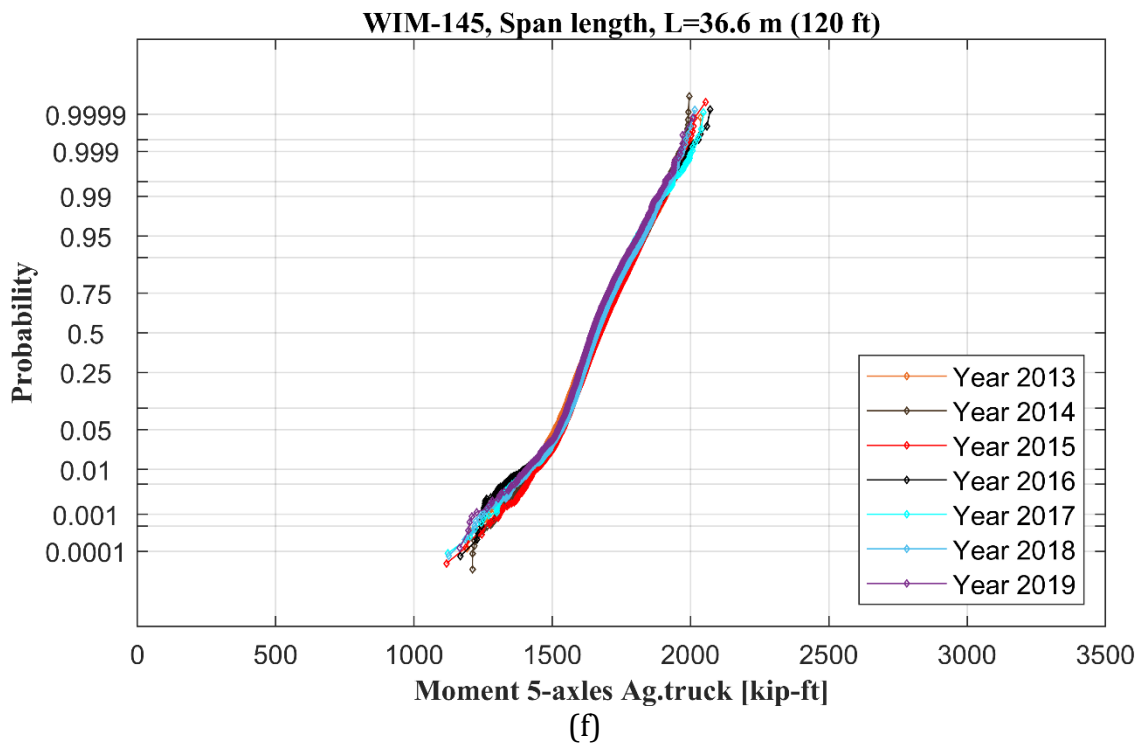
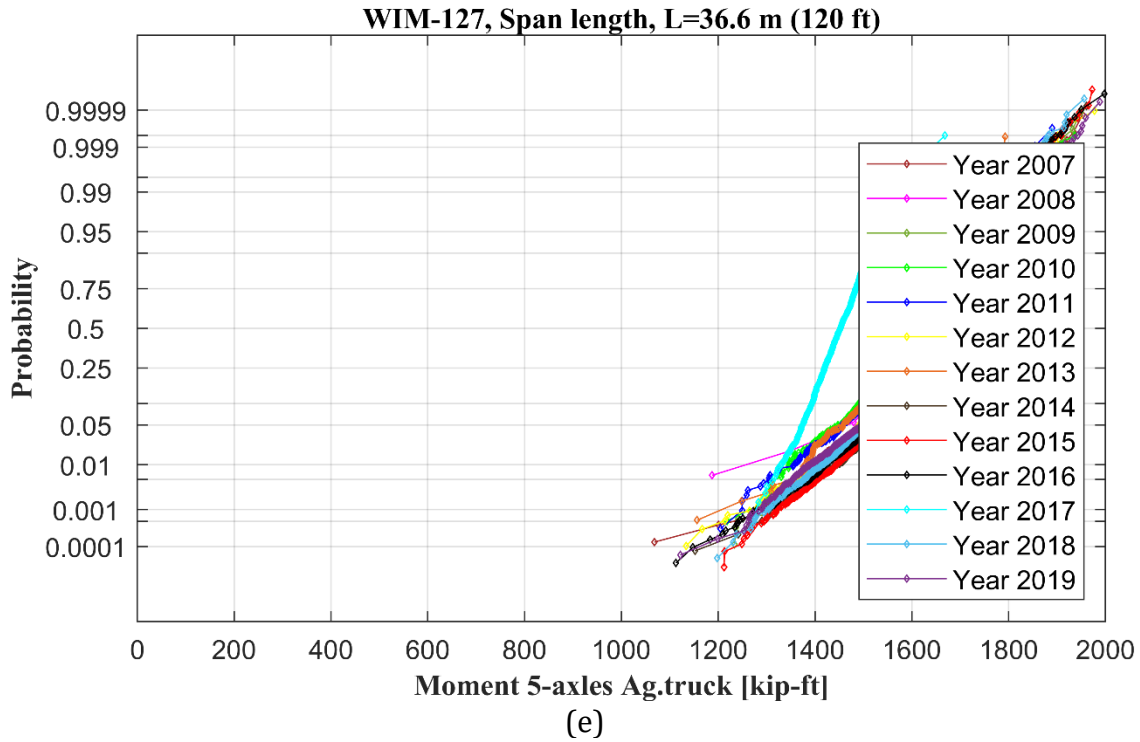
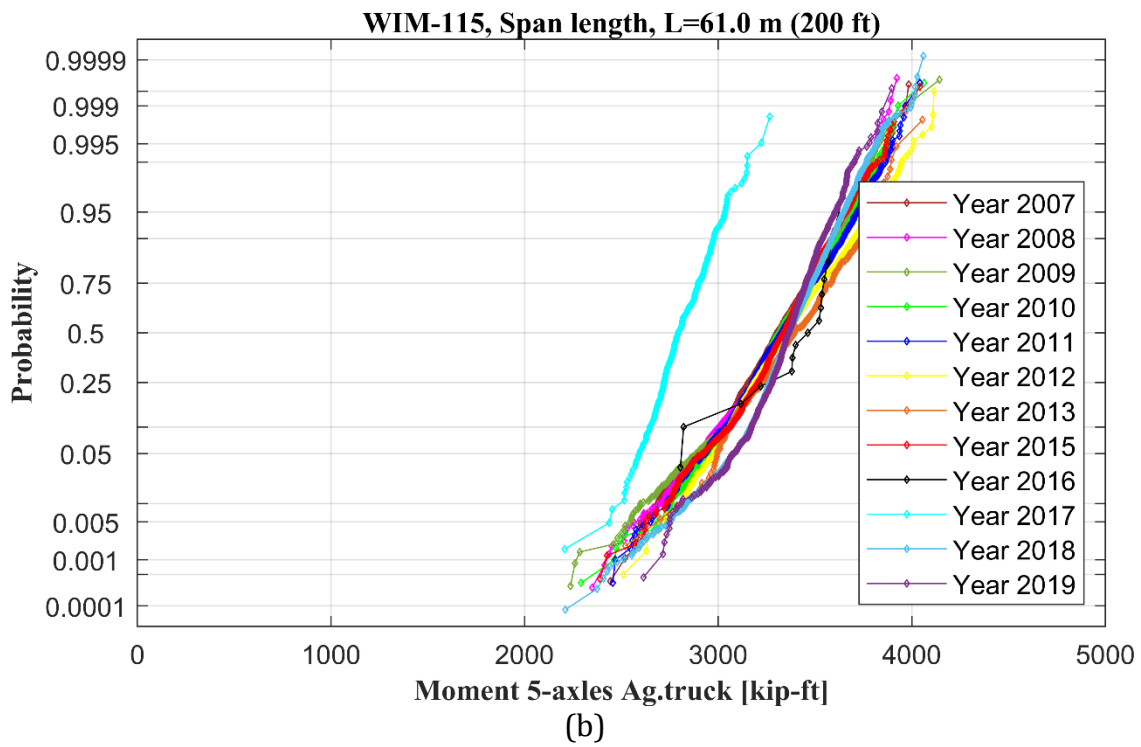
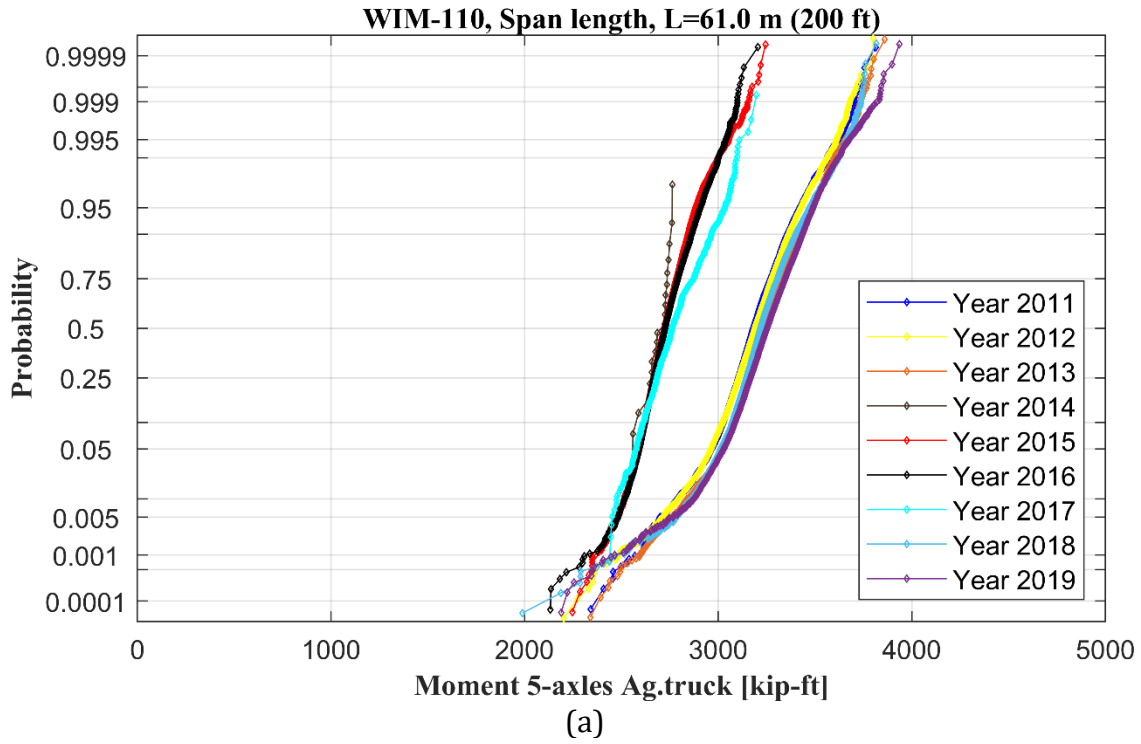
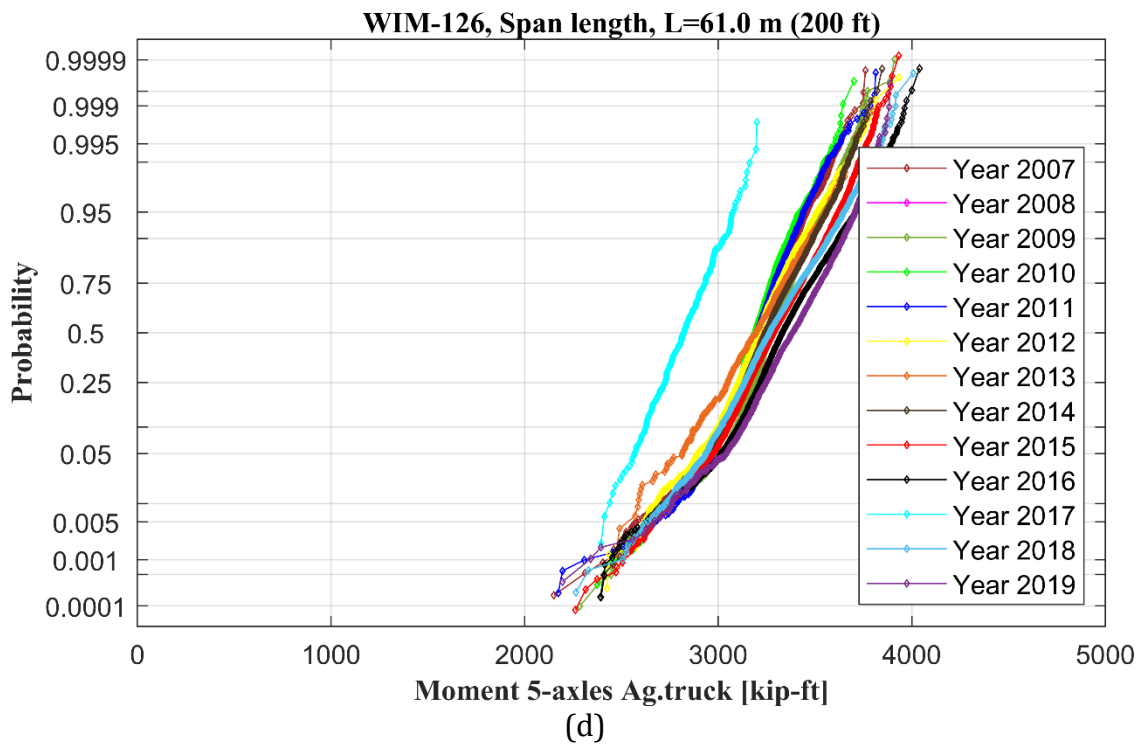
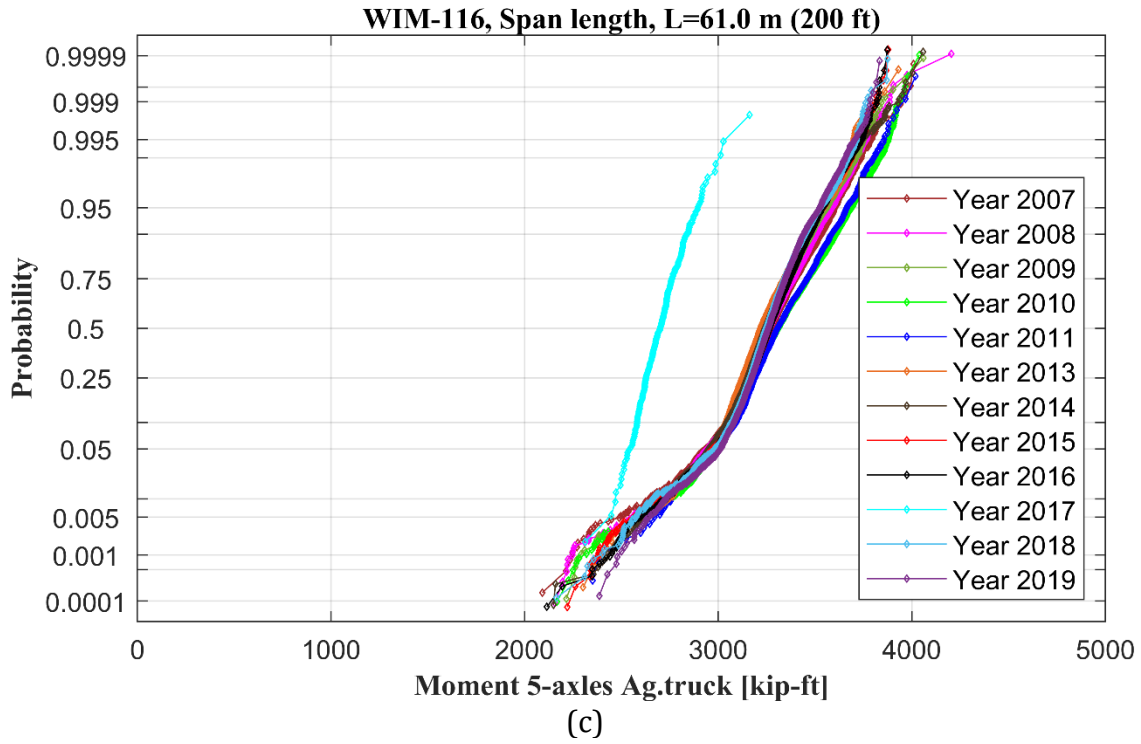


Figure 4-E.4. CDF of moment for 5-axes agricultural vehicles for span length of 36.6 m (120 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





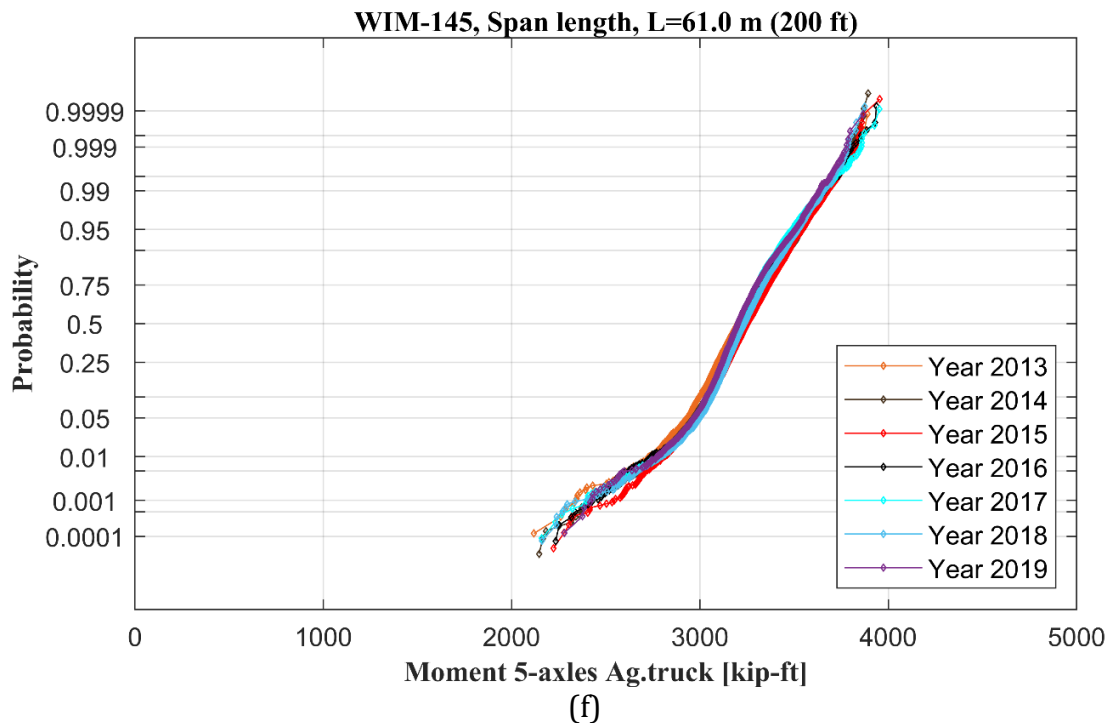
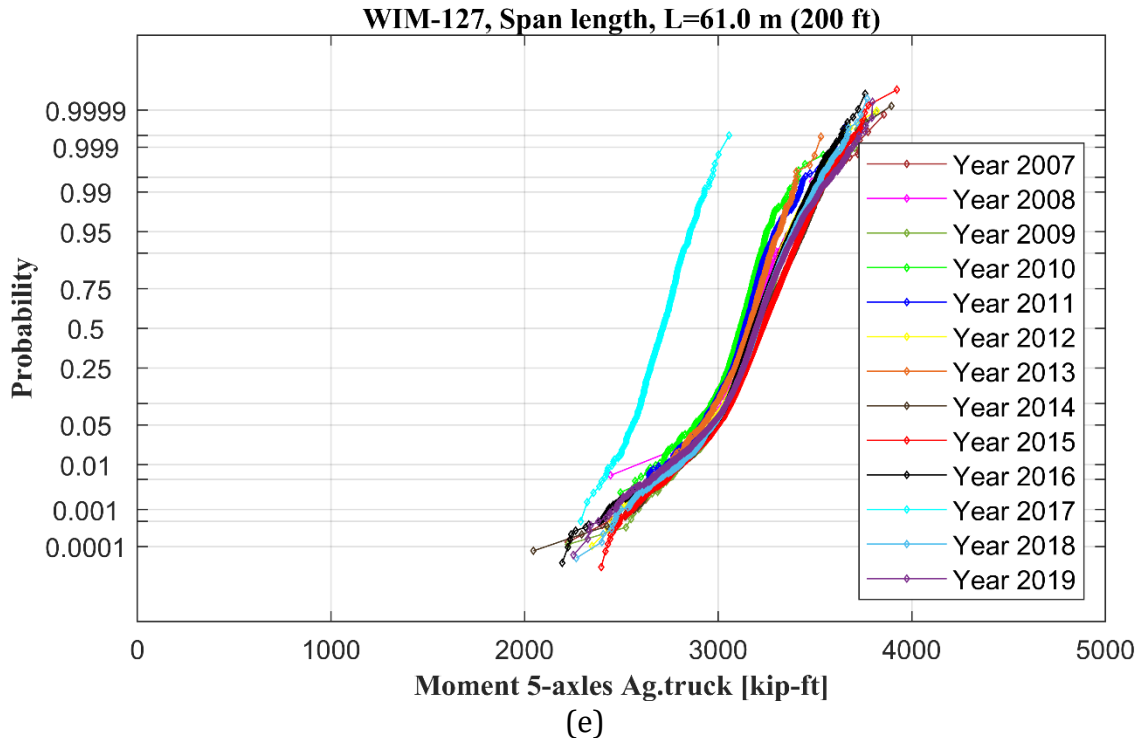
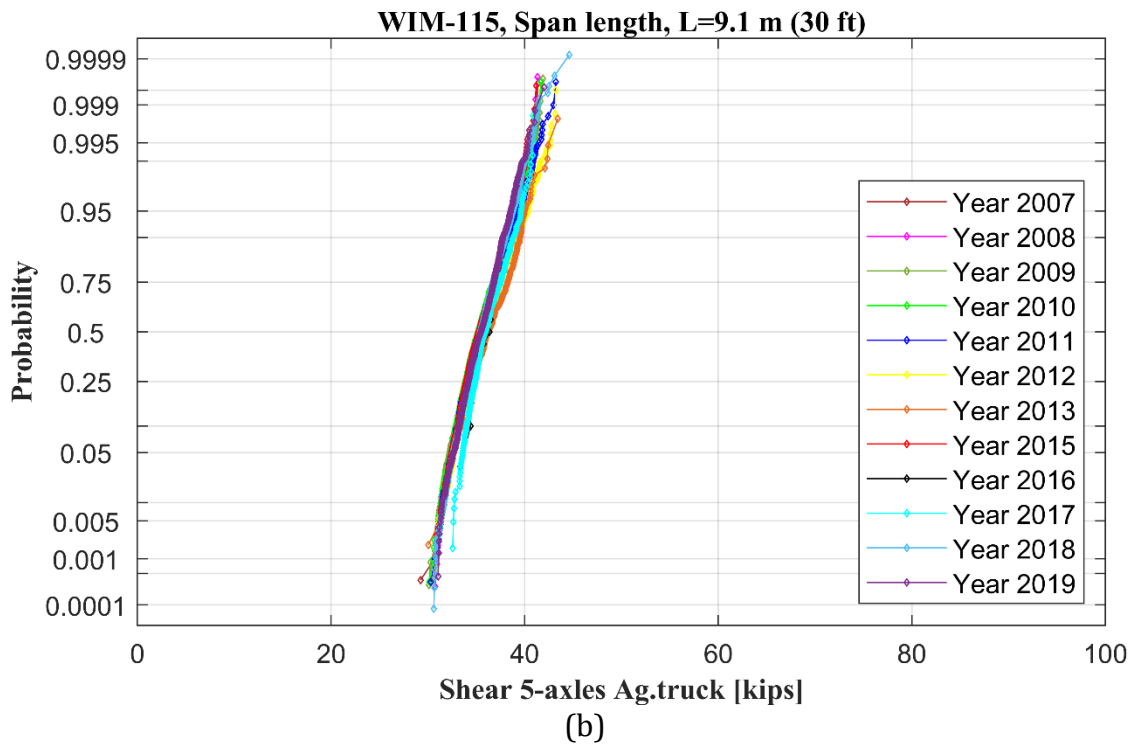
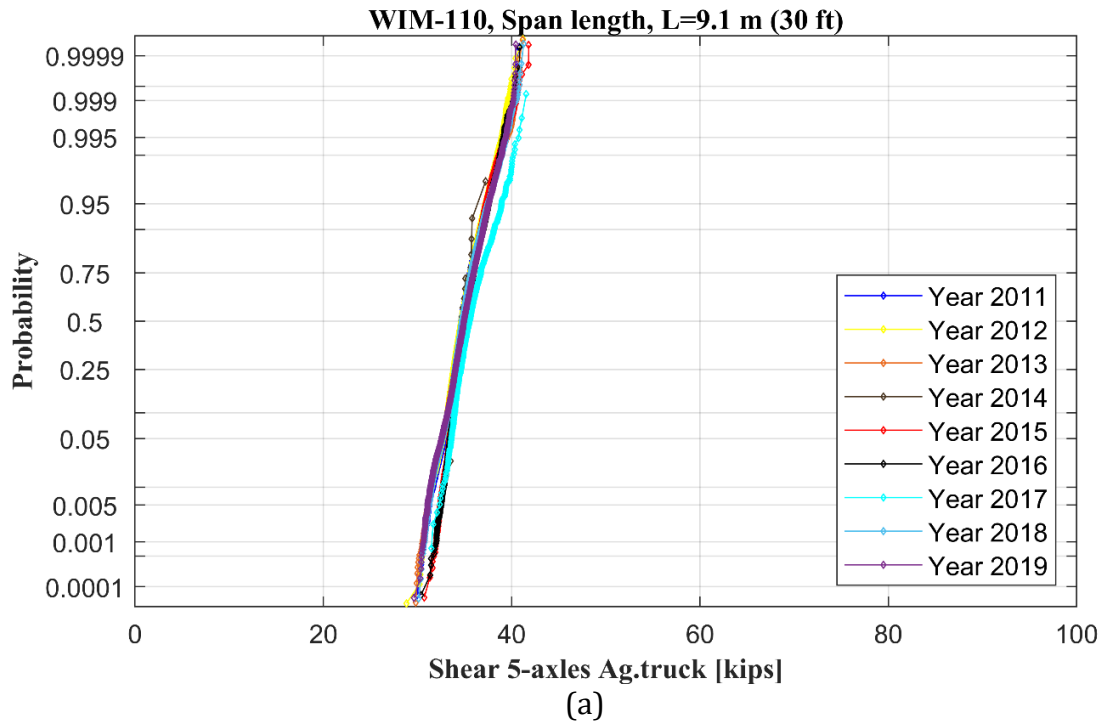
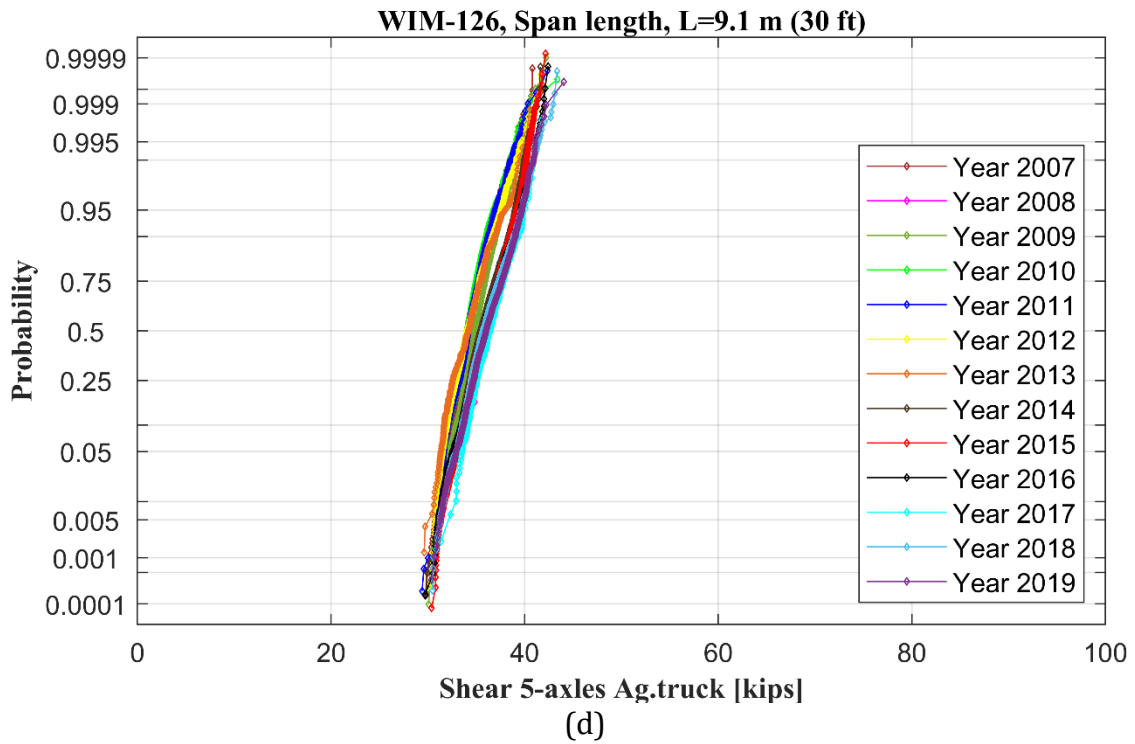
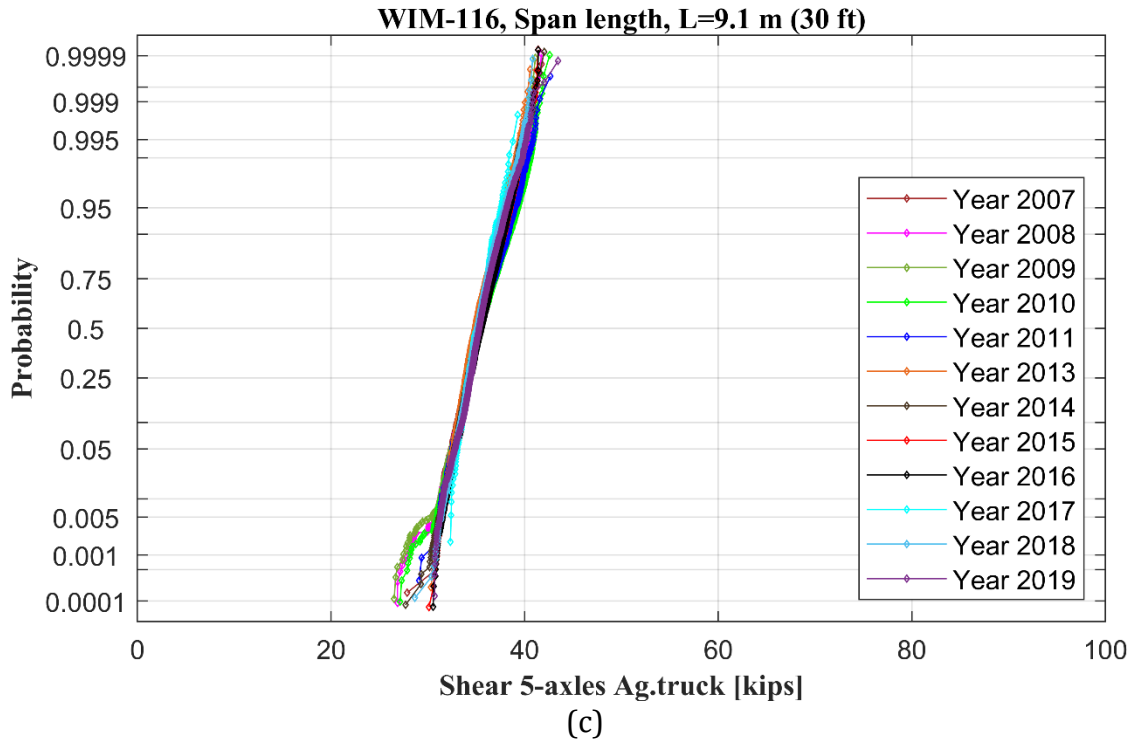


Figure 4-E.5. CDF of moment for 5-axes agricultural vehicles for span length of 61 m (200 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.

Appendix 4-F. CDF of shear force effects for 5-axle agricultural vehicles





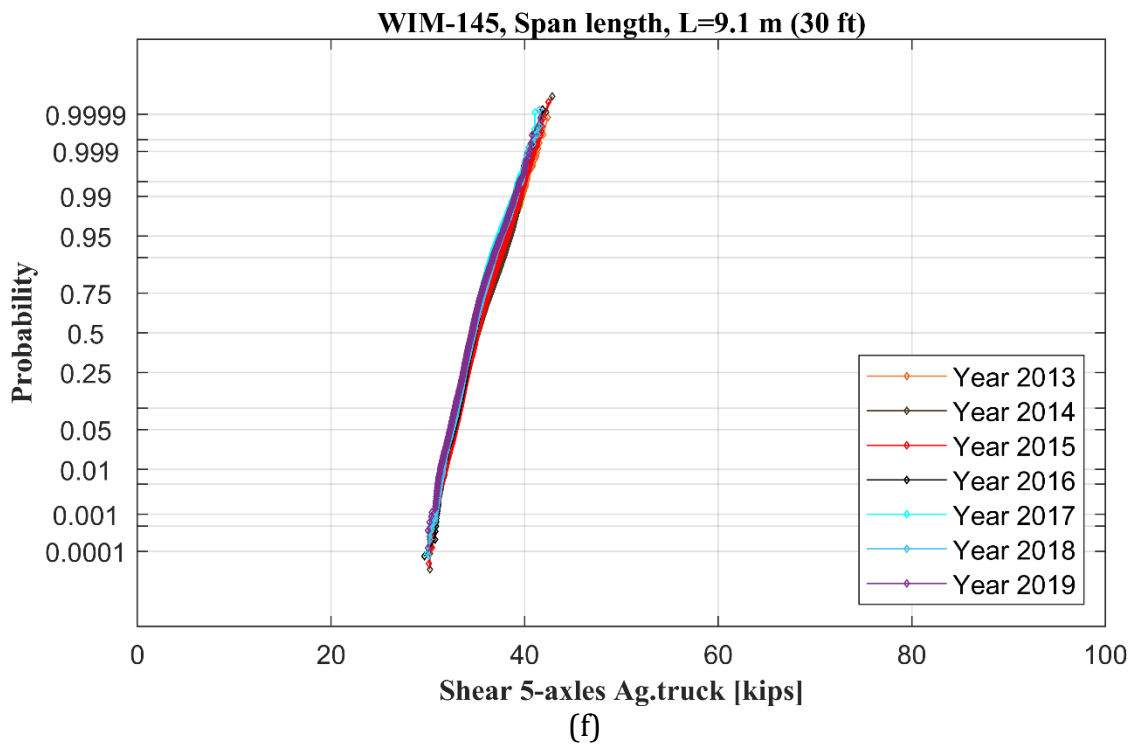
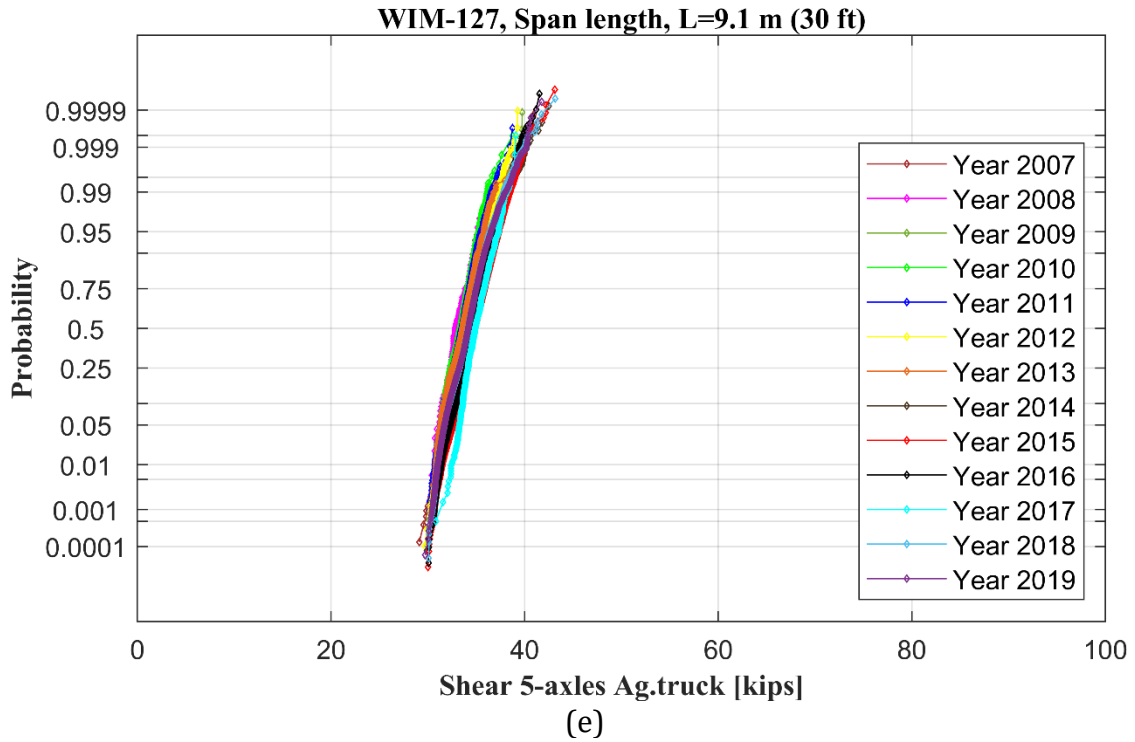
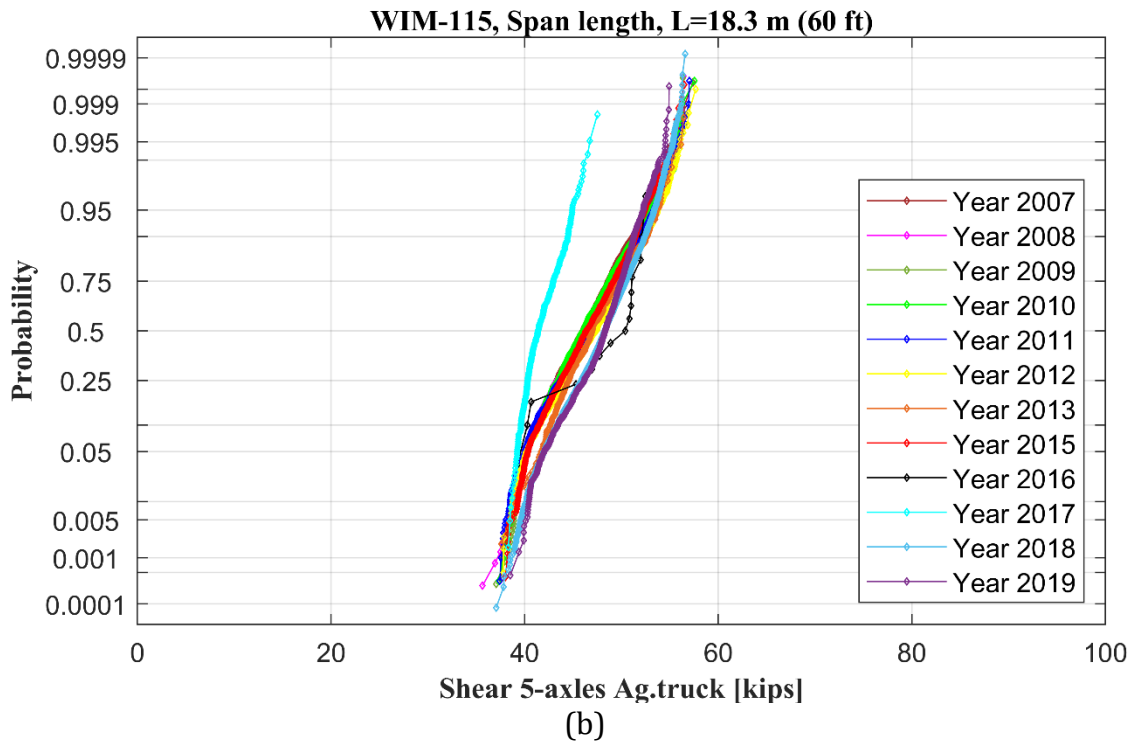
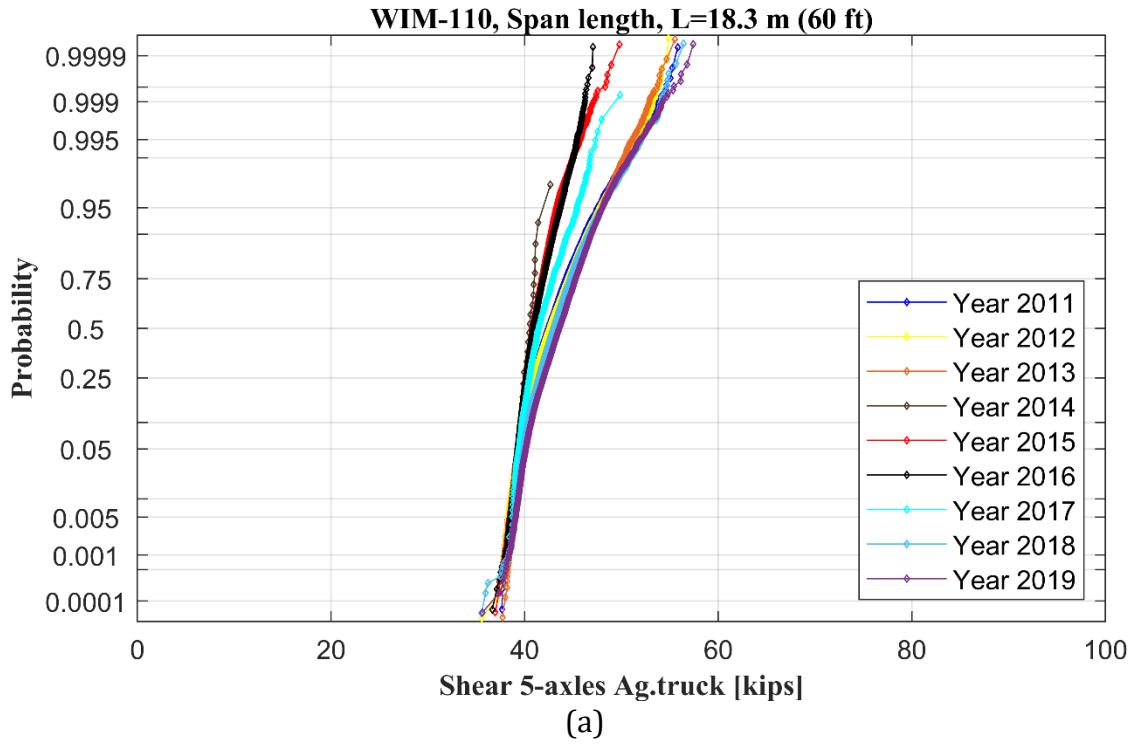
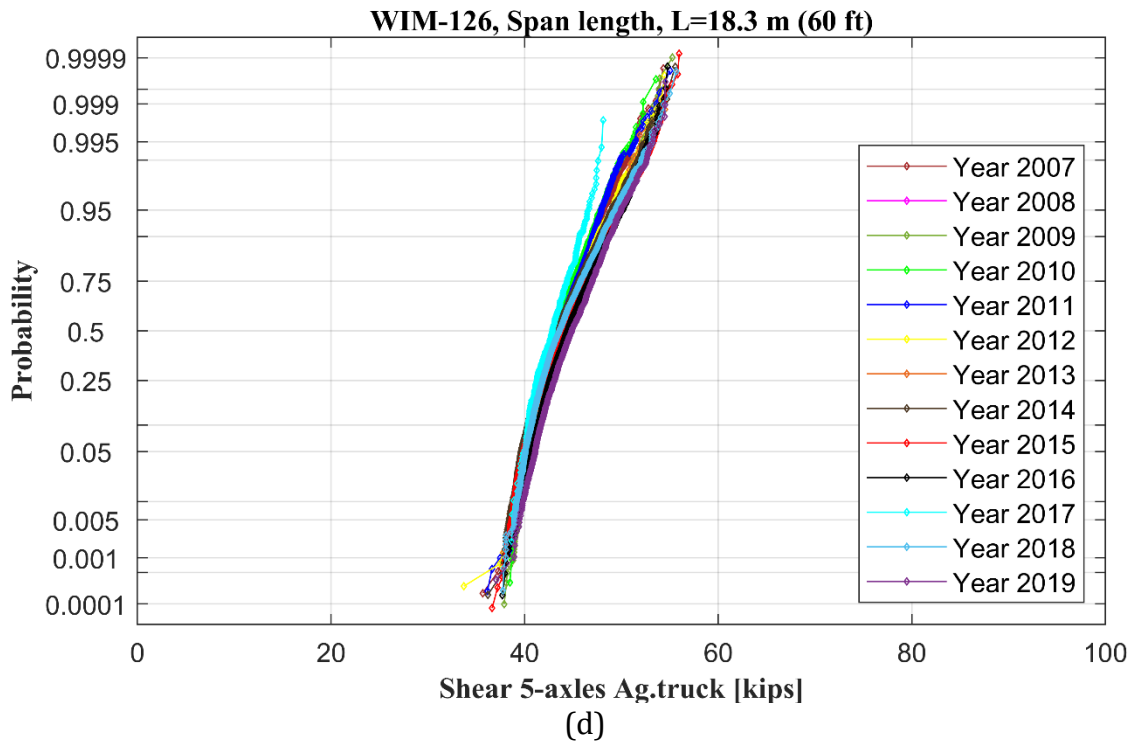
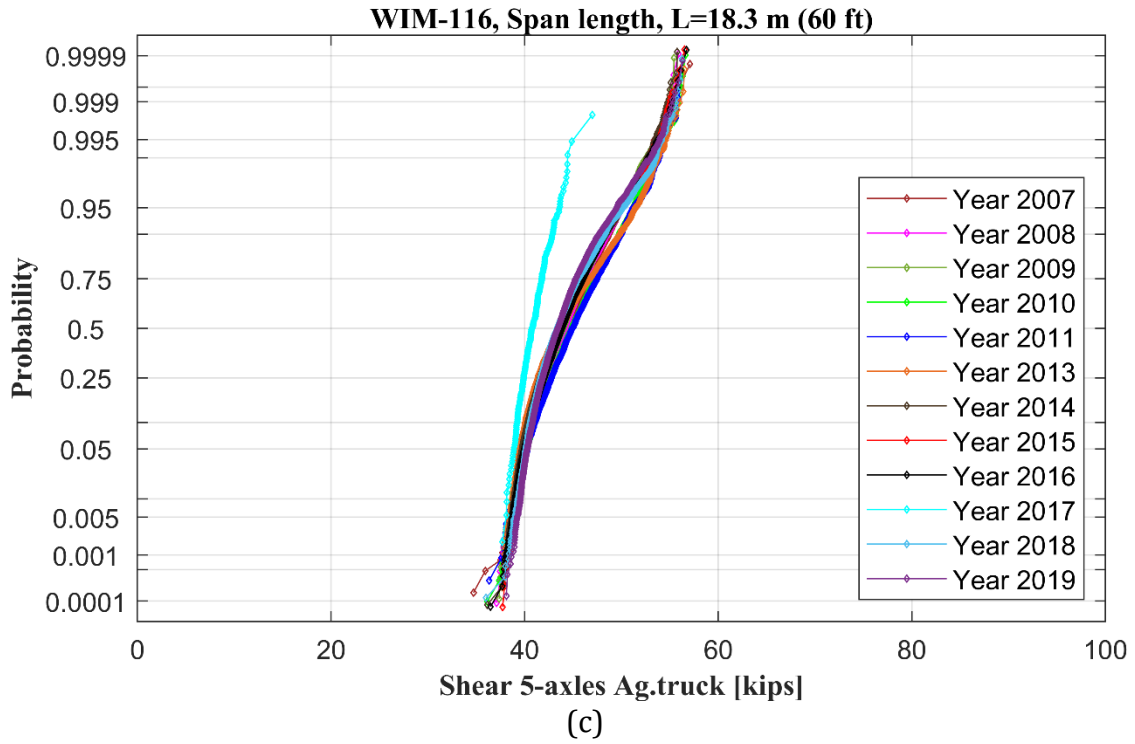


Figure 4-F.1. CDF of shear for 5-axes agricultural vehicles for span length of 9.1 m (30 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





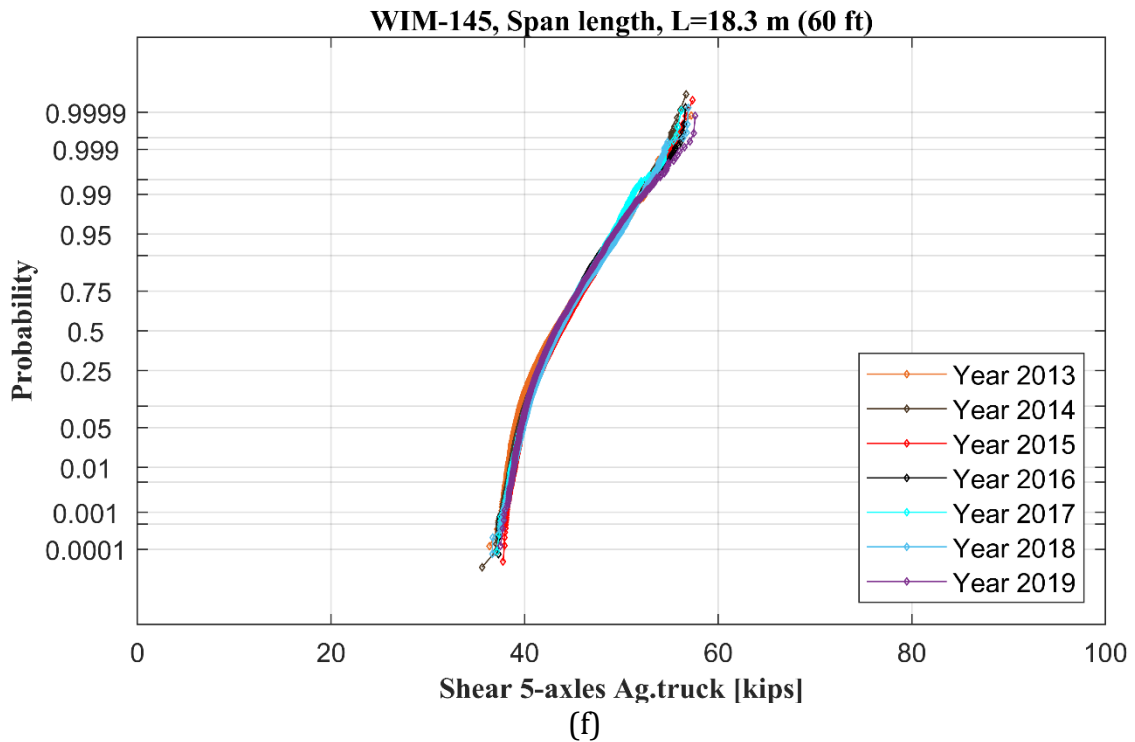
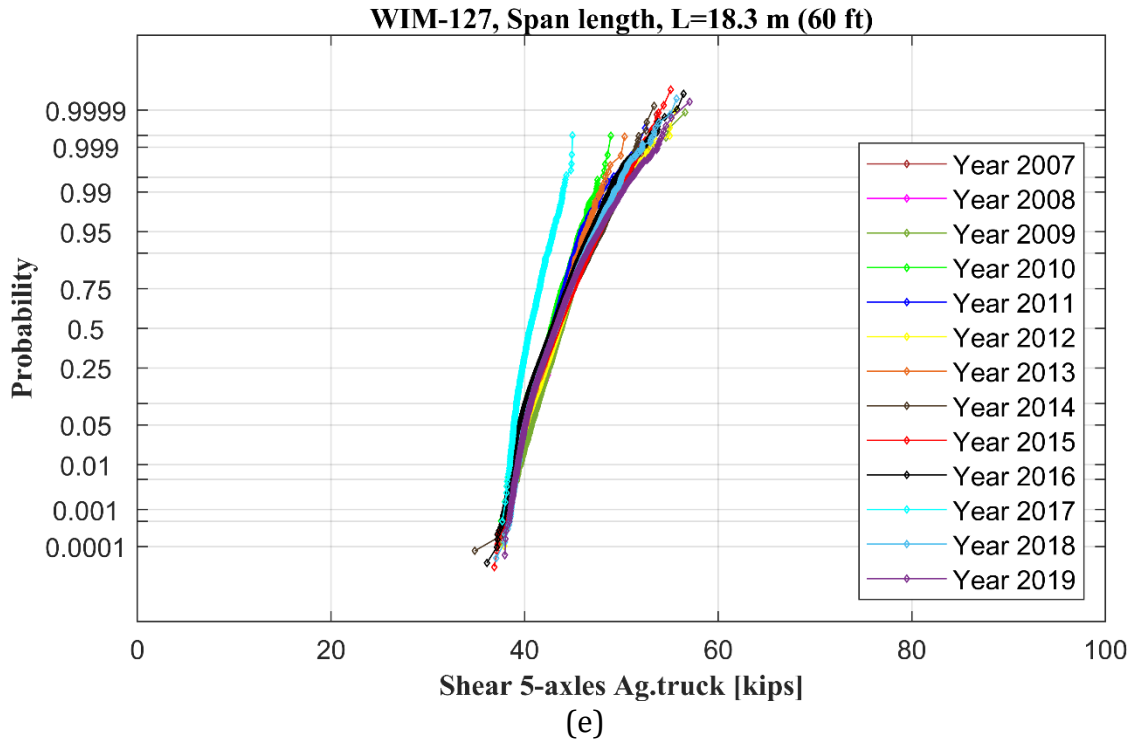
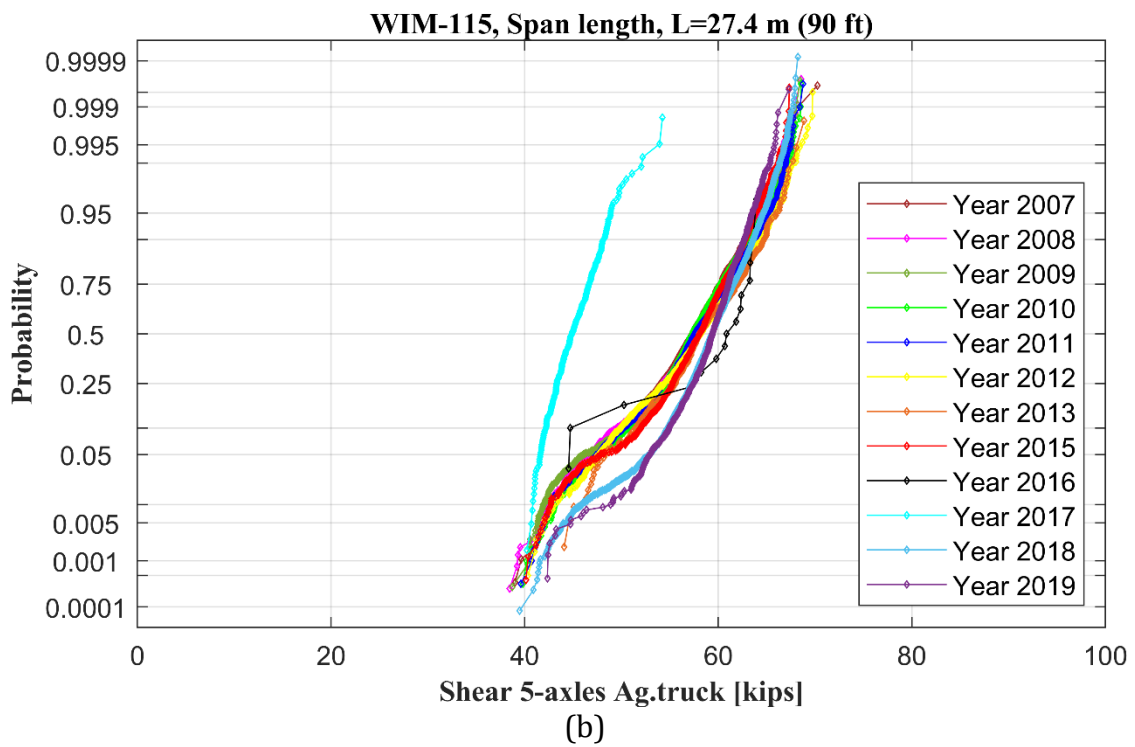
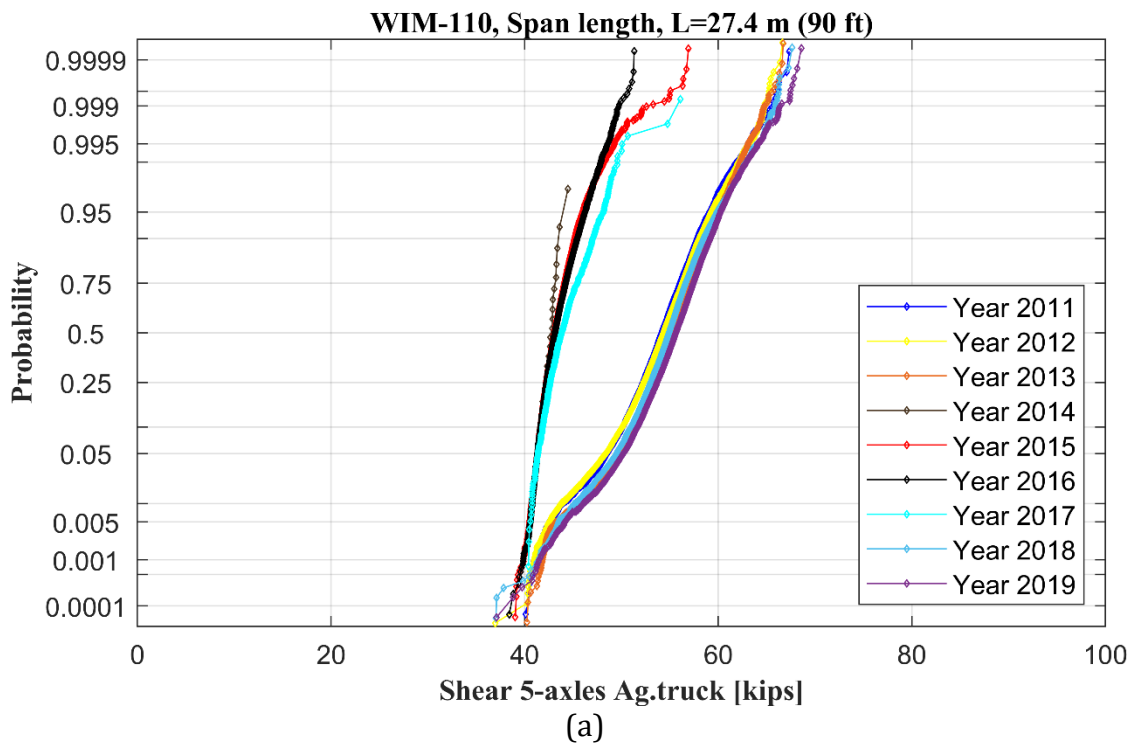
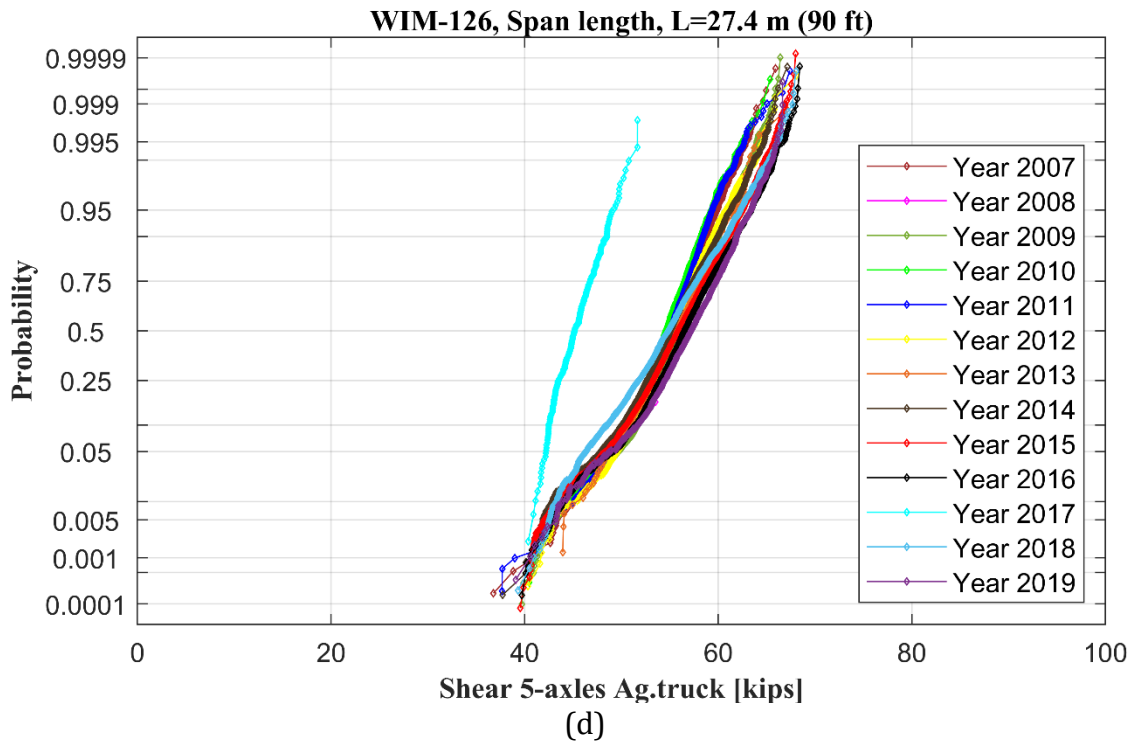
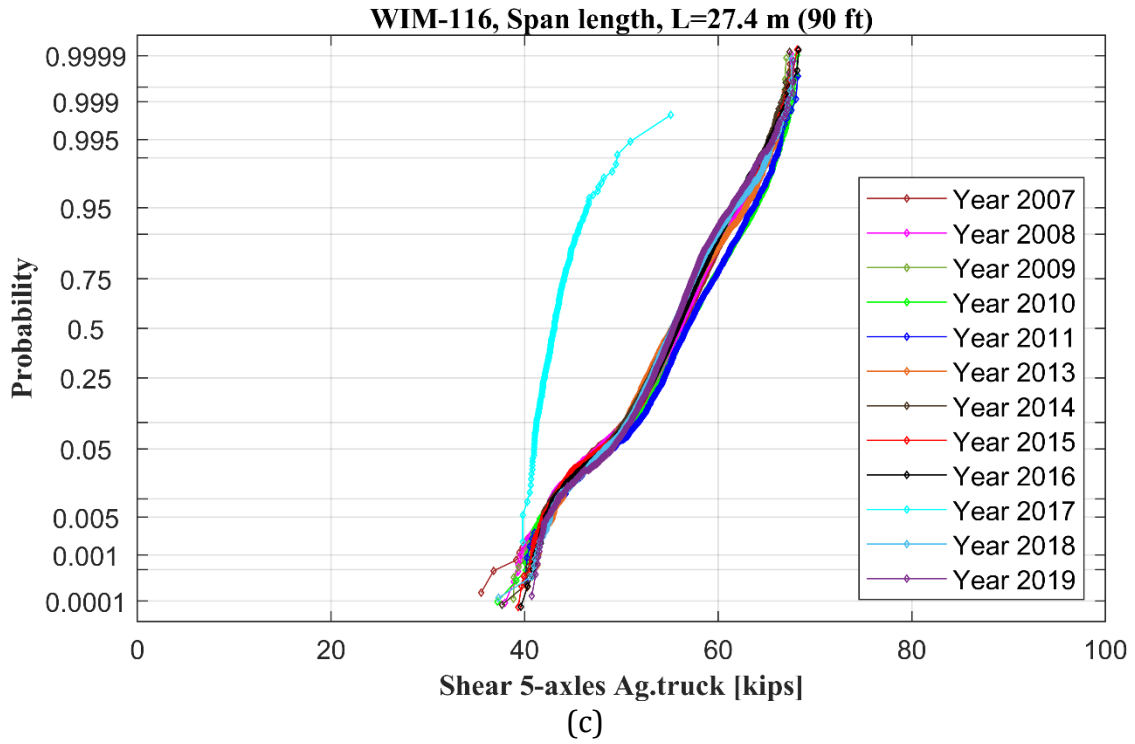


Figure 4-F.2. CDF of shear for 5-axes agricultural vehicles for span length of 18.3 m (60 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





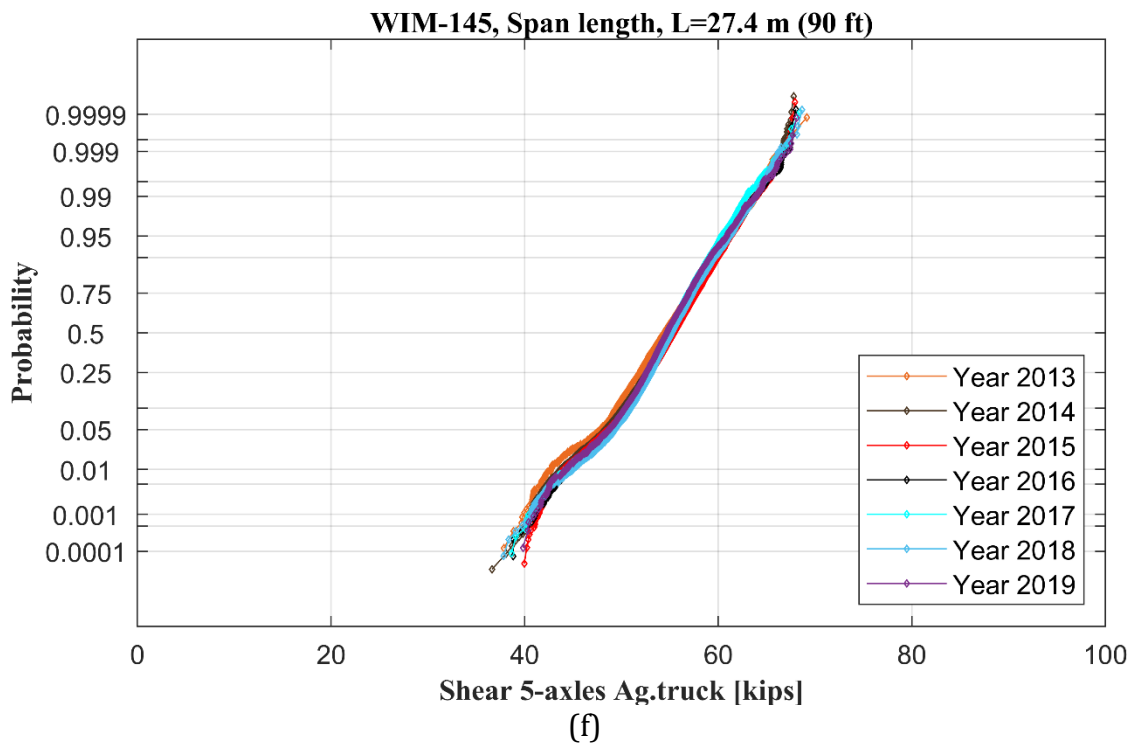
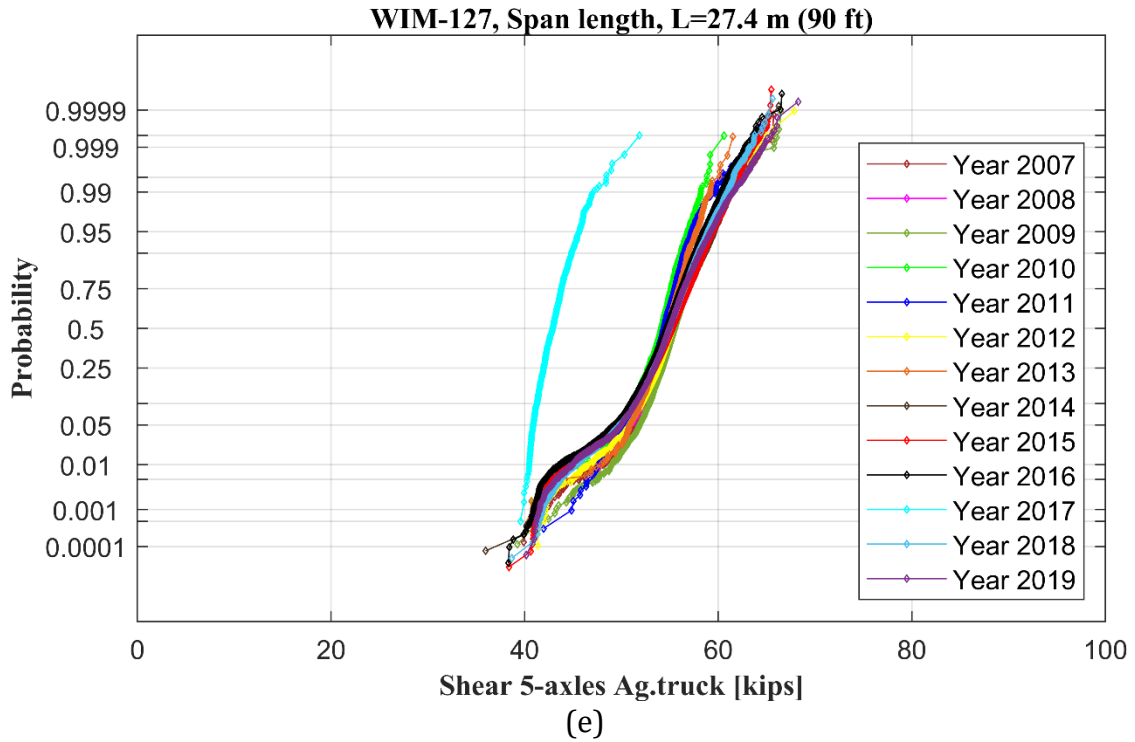
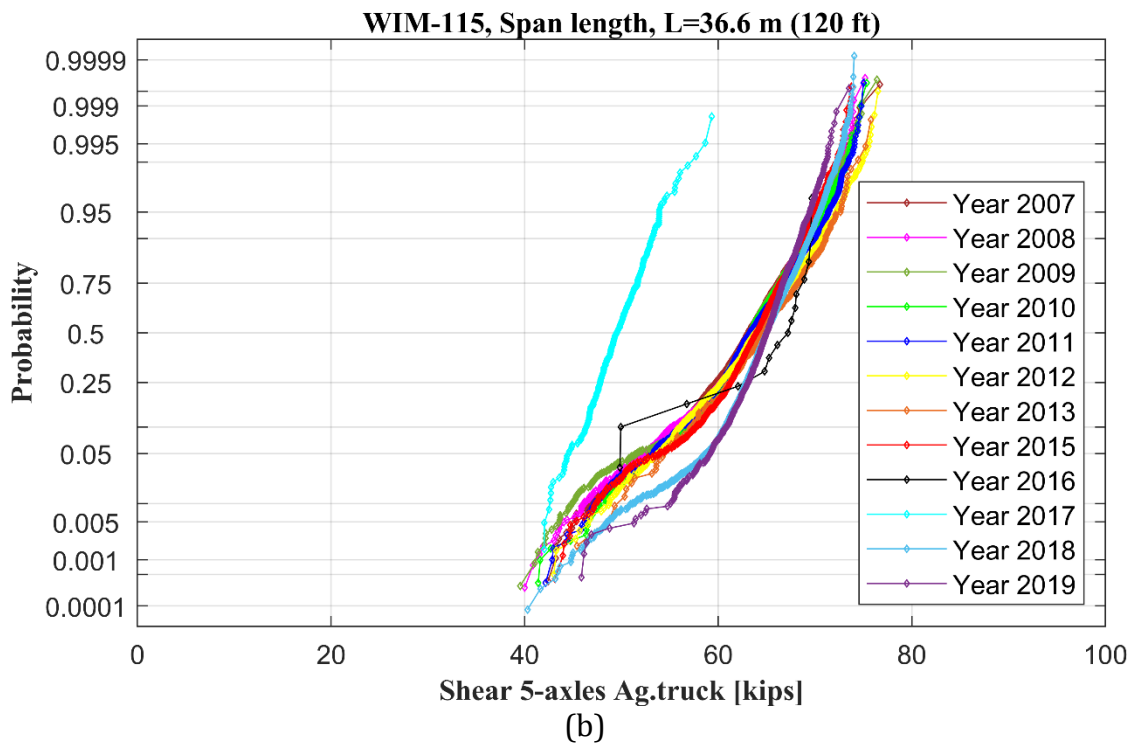
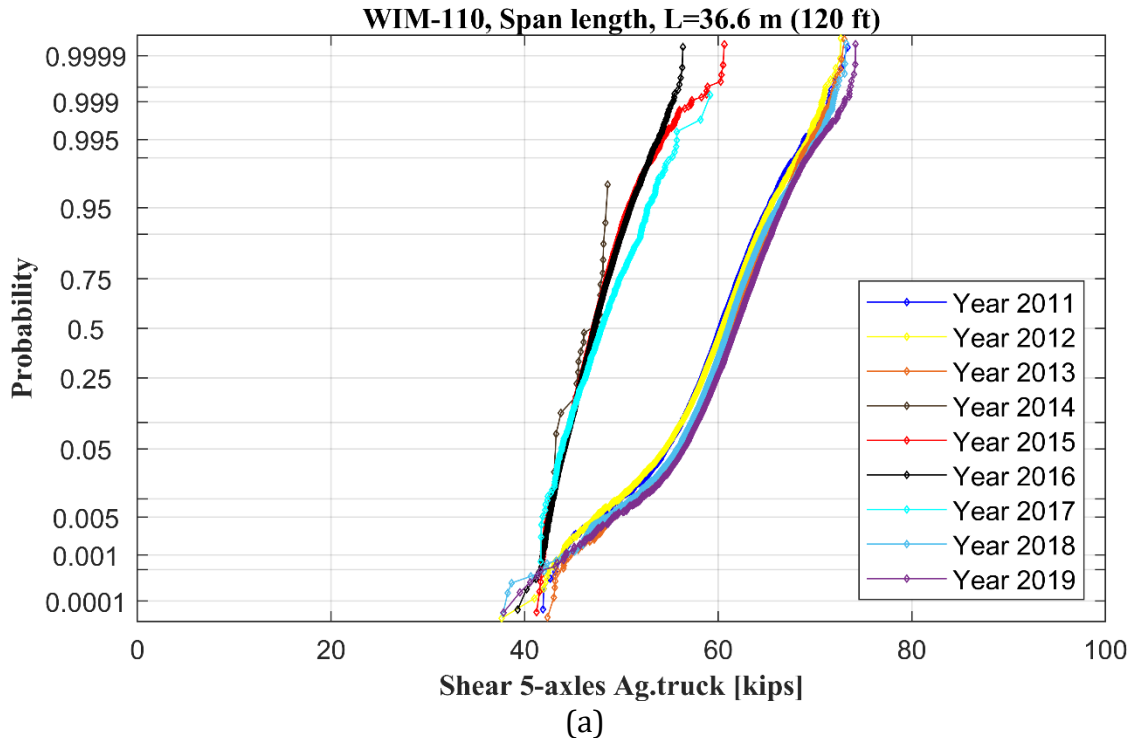
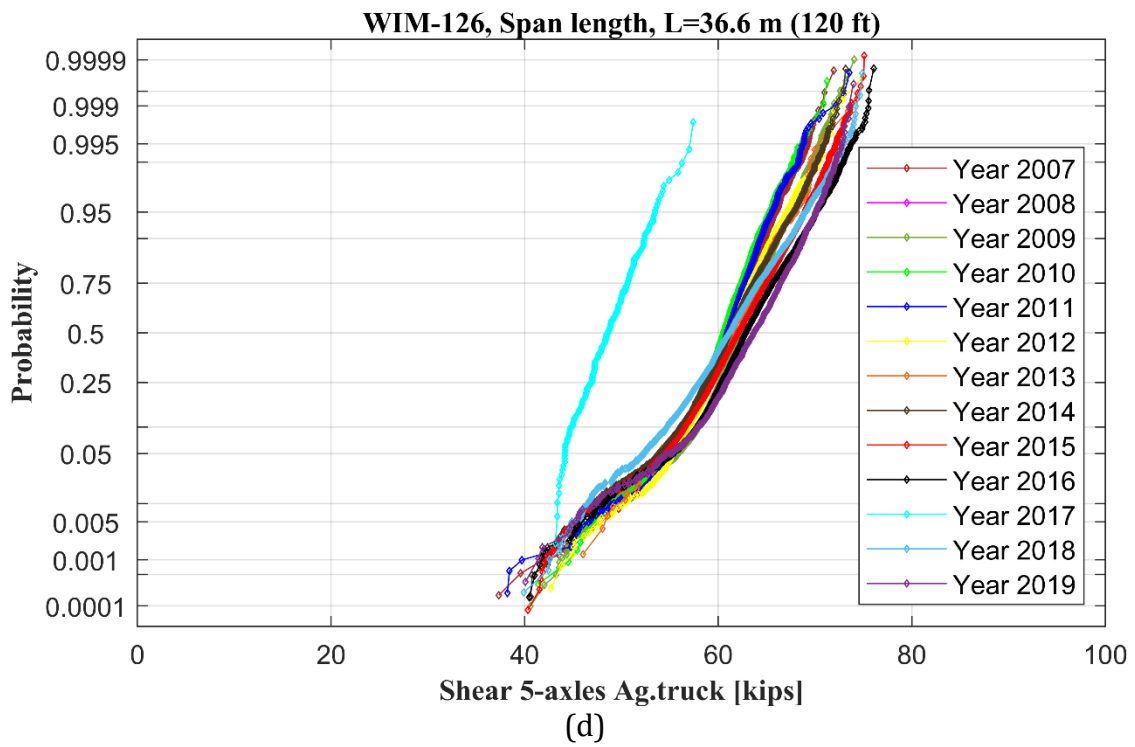
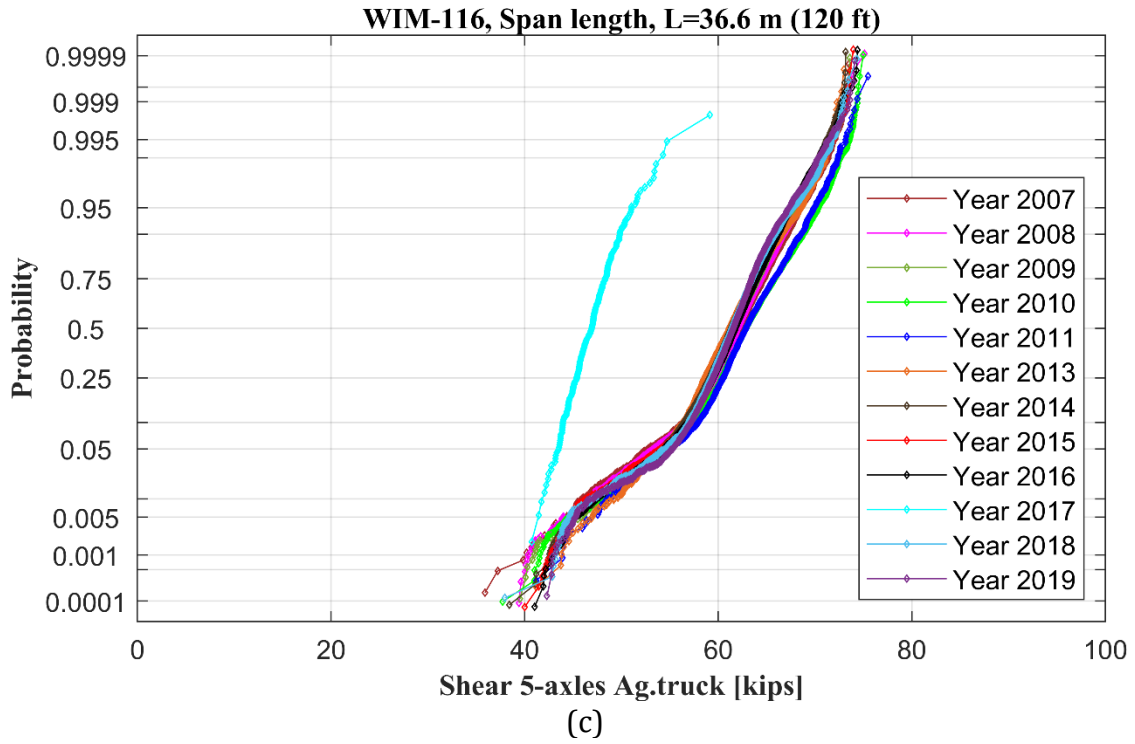


Figure 4-F.3. CDF of shear for 5-axes agricultural vehicles for span length of 27.4 m (90 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





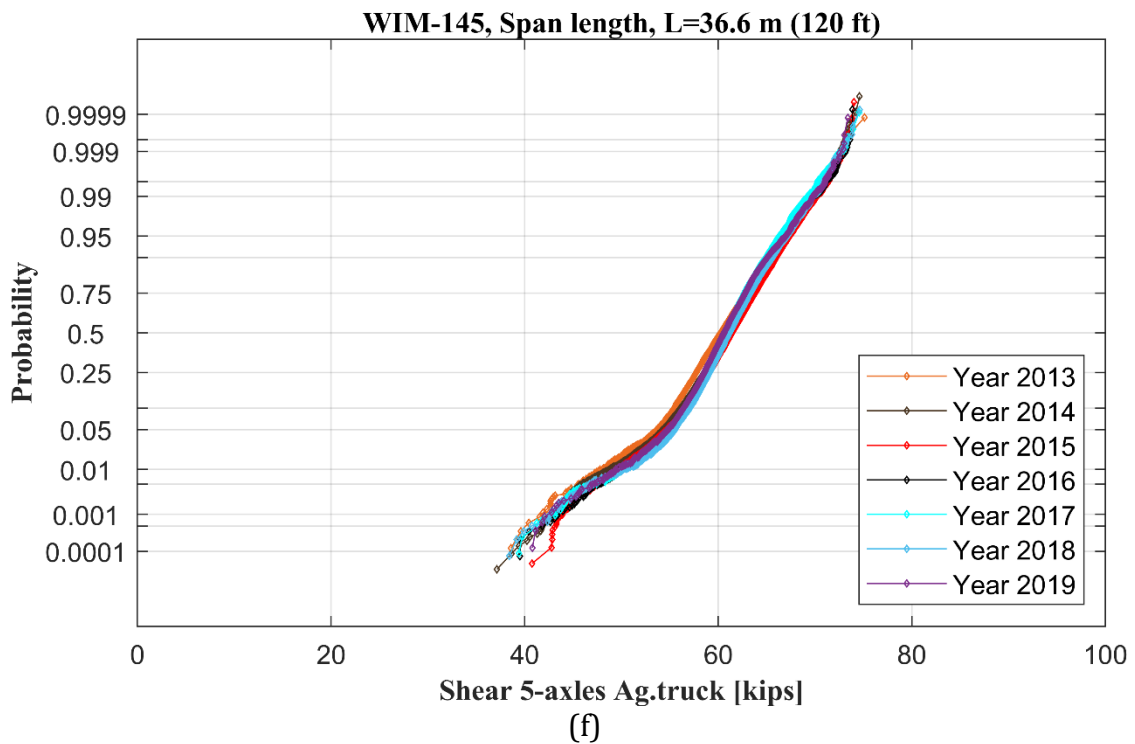
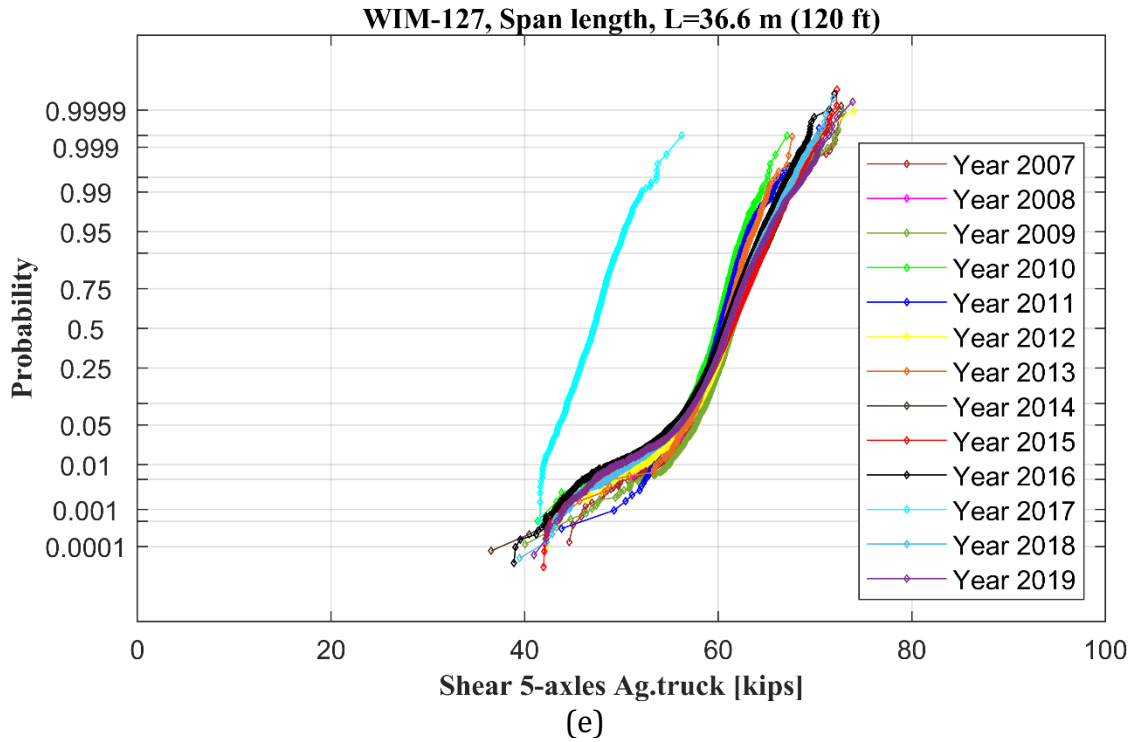
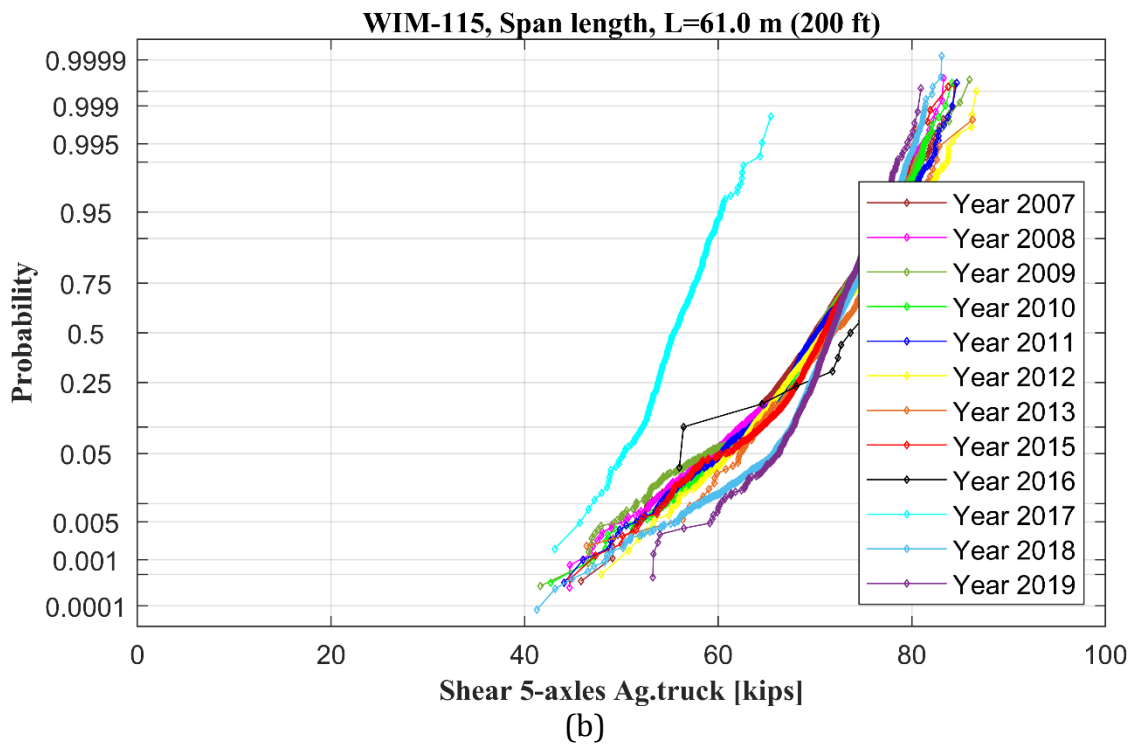
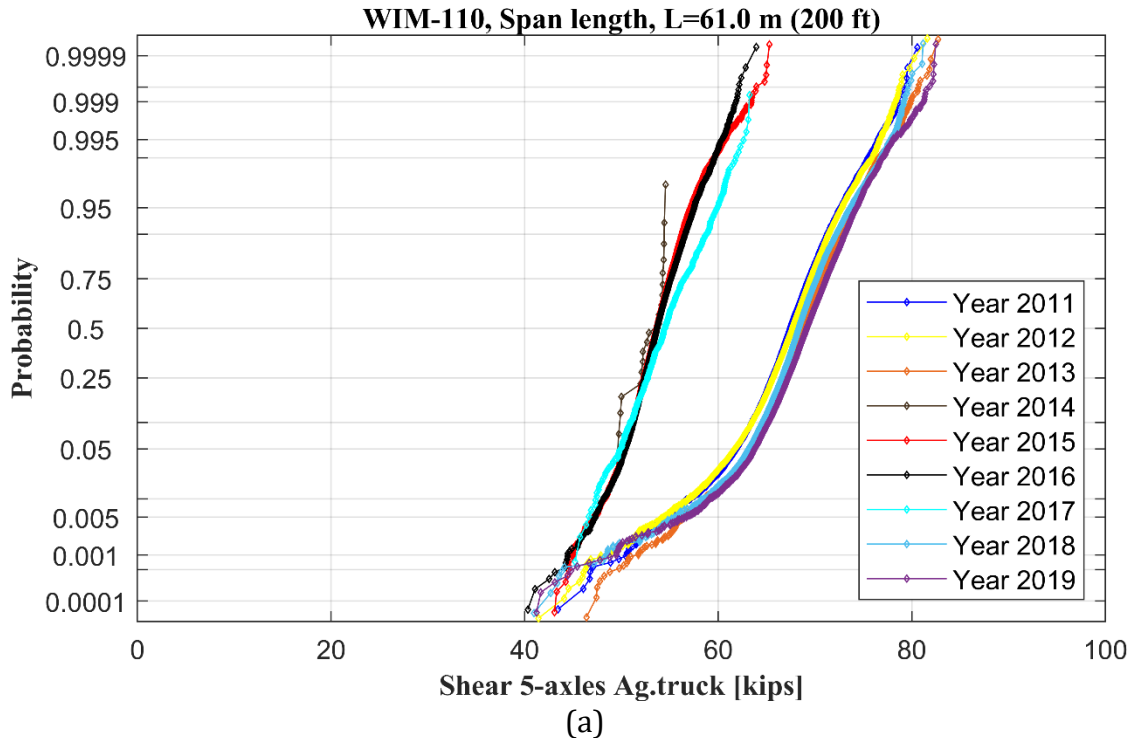
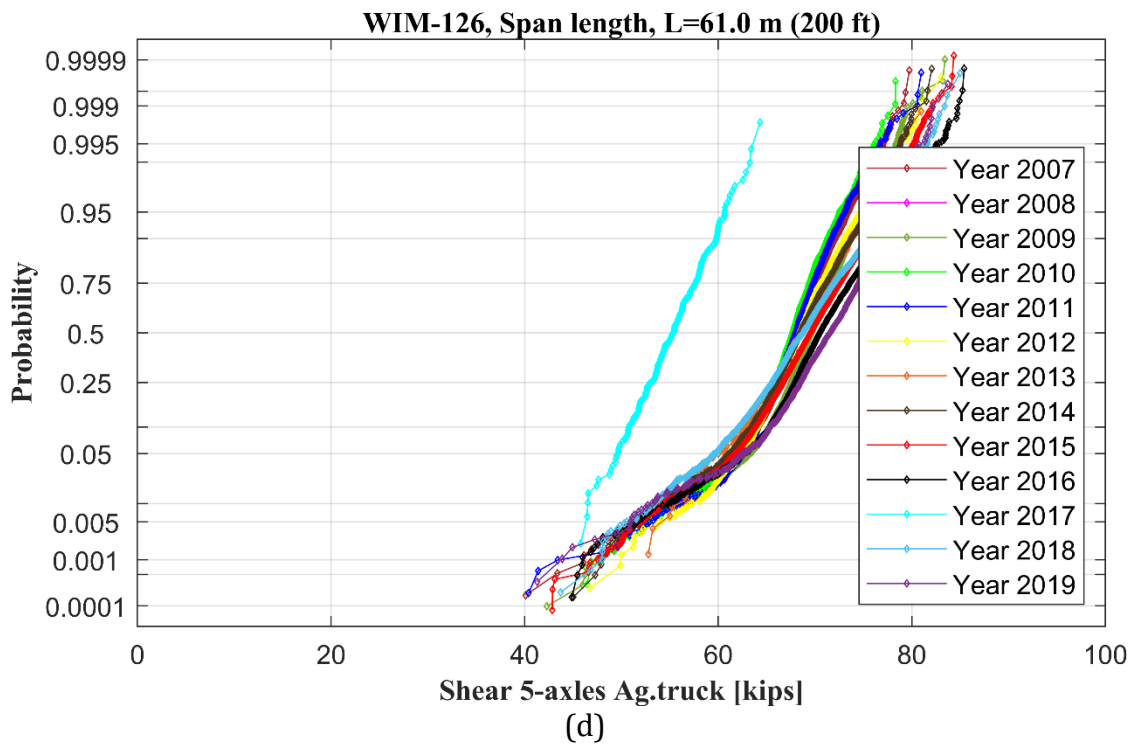
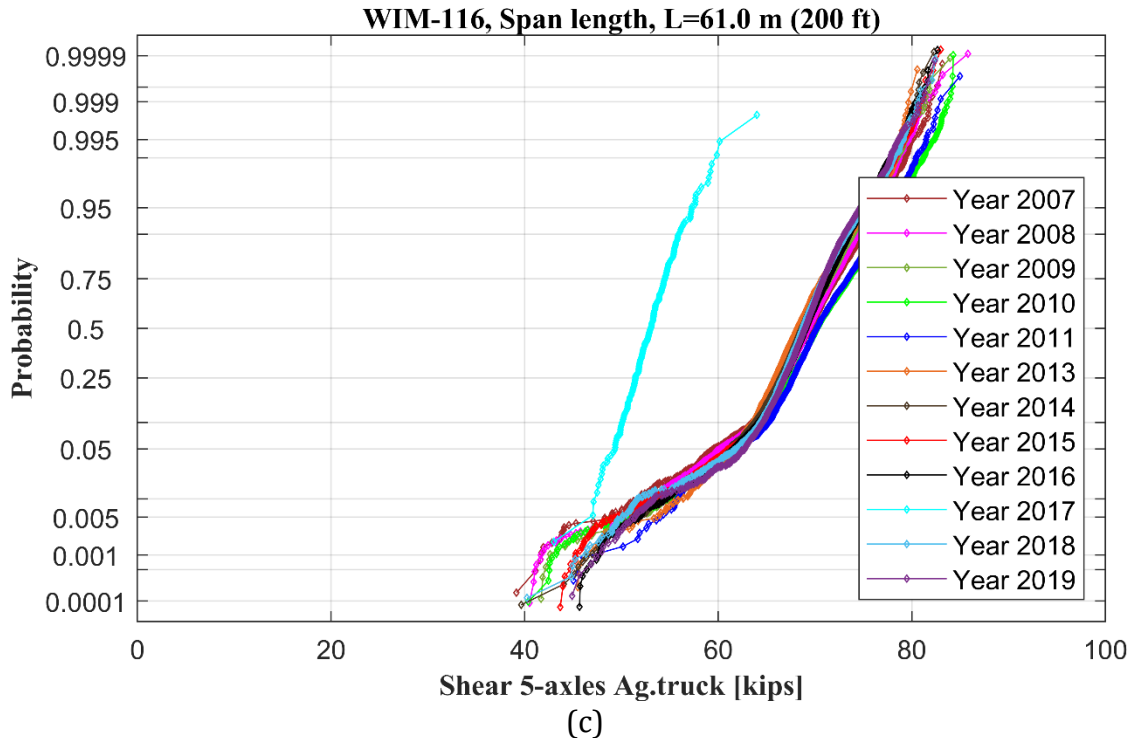


Figure 4-F.4. CDF of shear for 5-axes agricultural vehicles for span length of 36.6 m (120 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





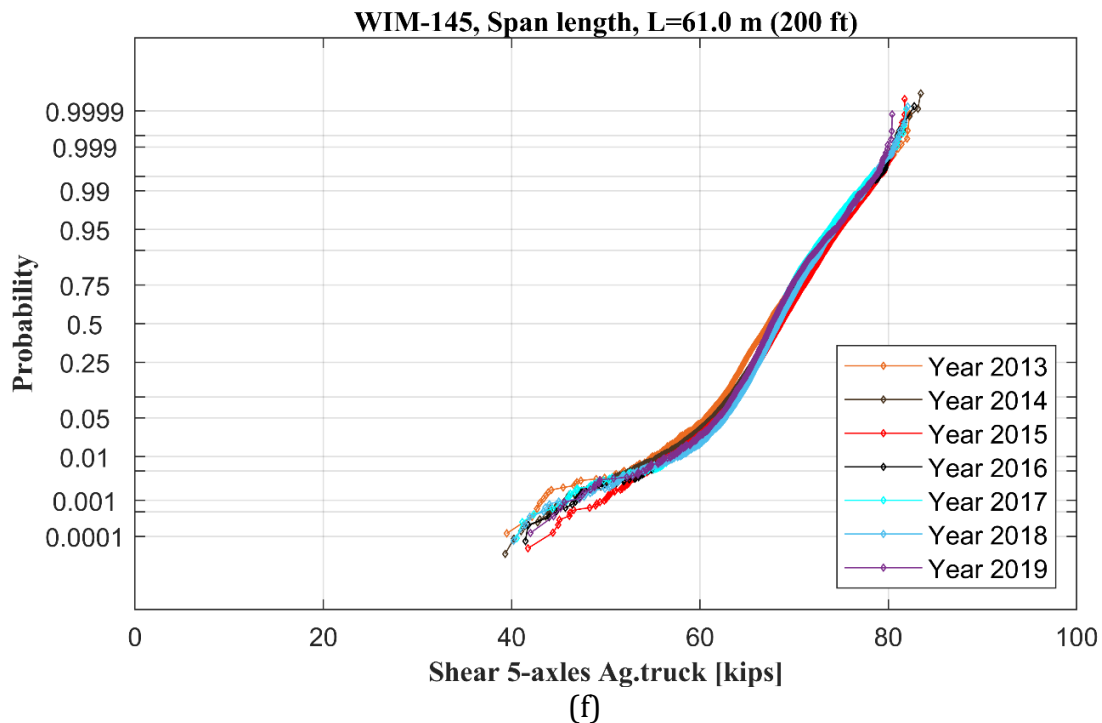
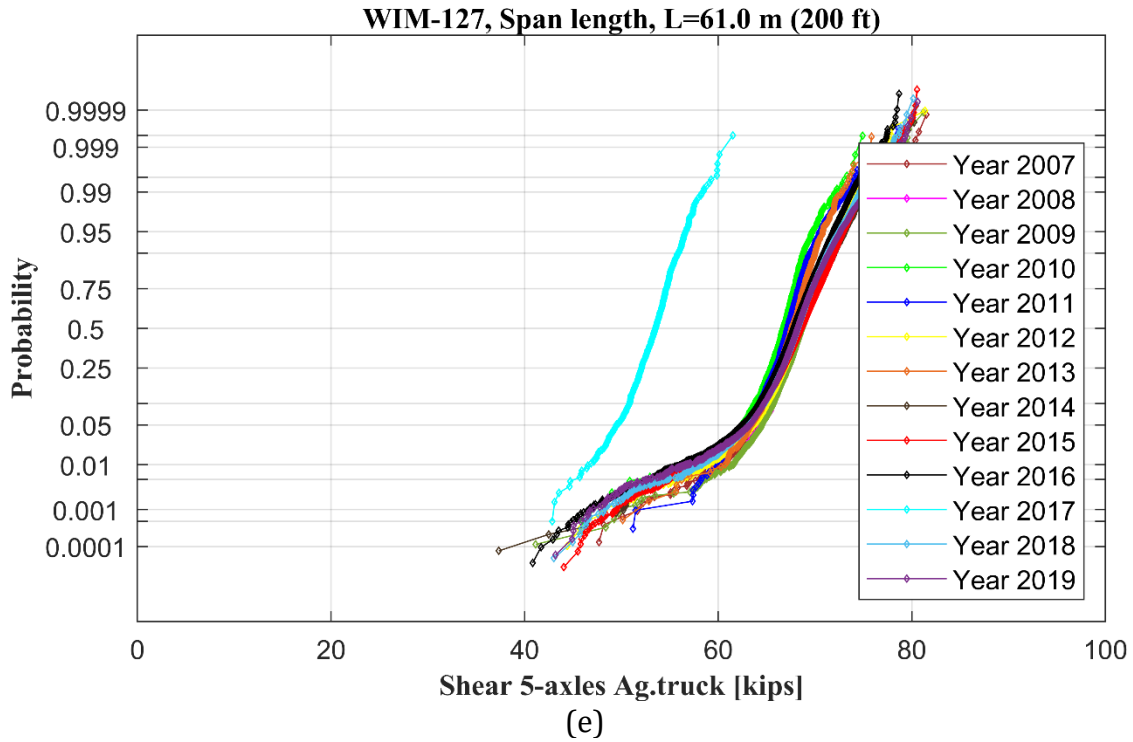
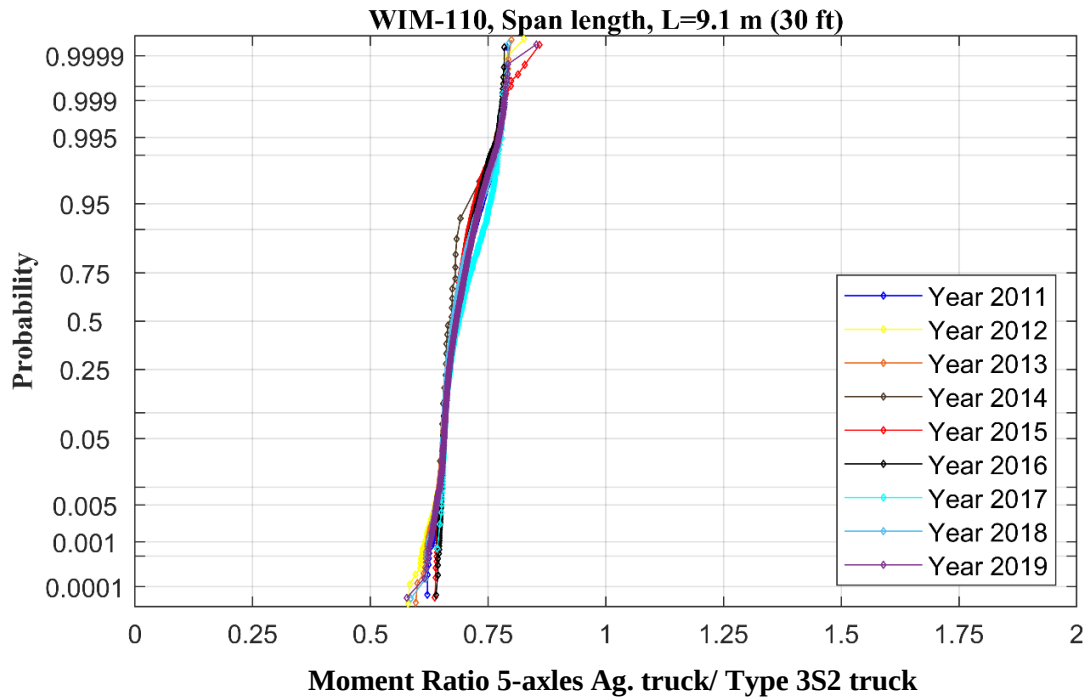
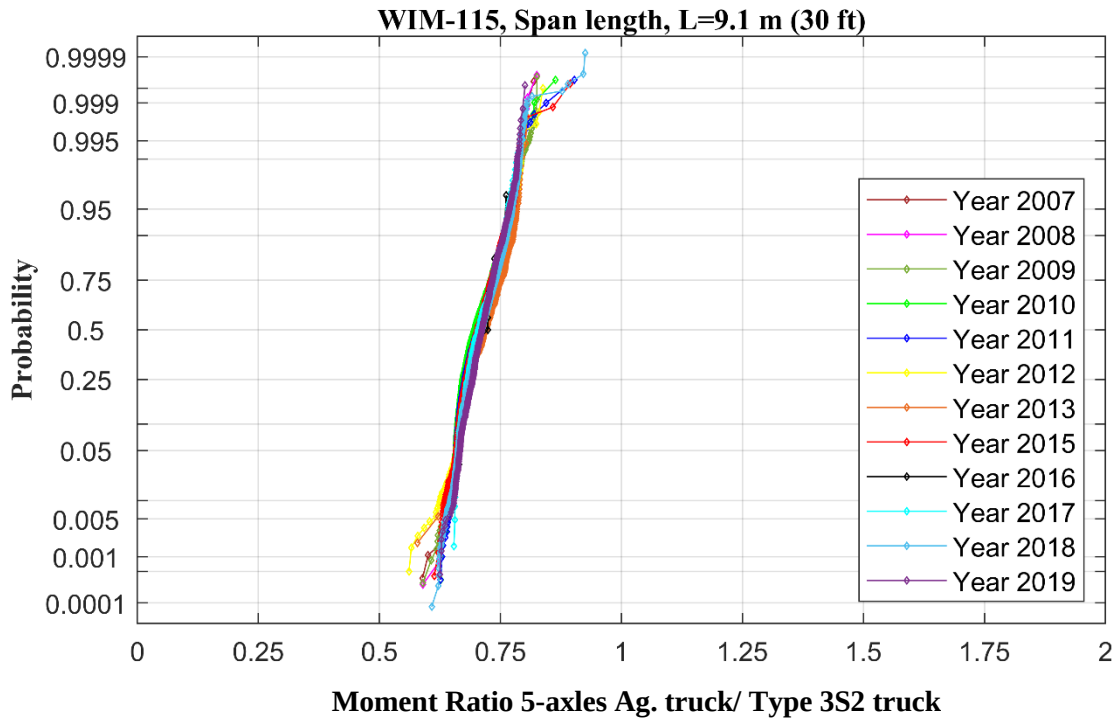


Figure 4-F.5. CDF of shear for 5-axes agricultural vehicles for span length of 61 m (200 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.

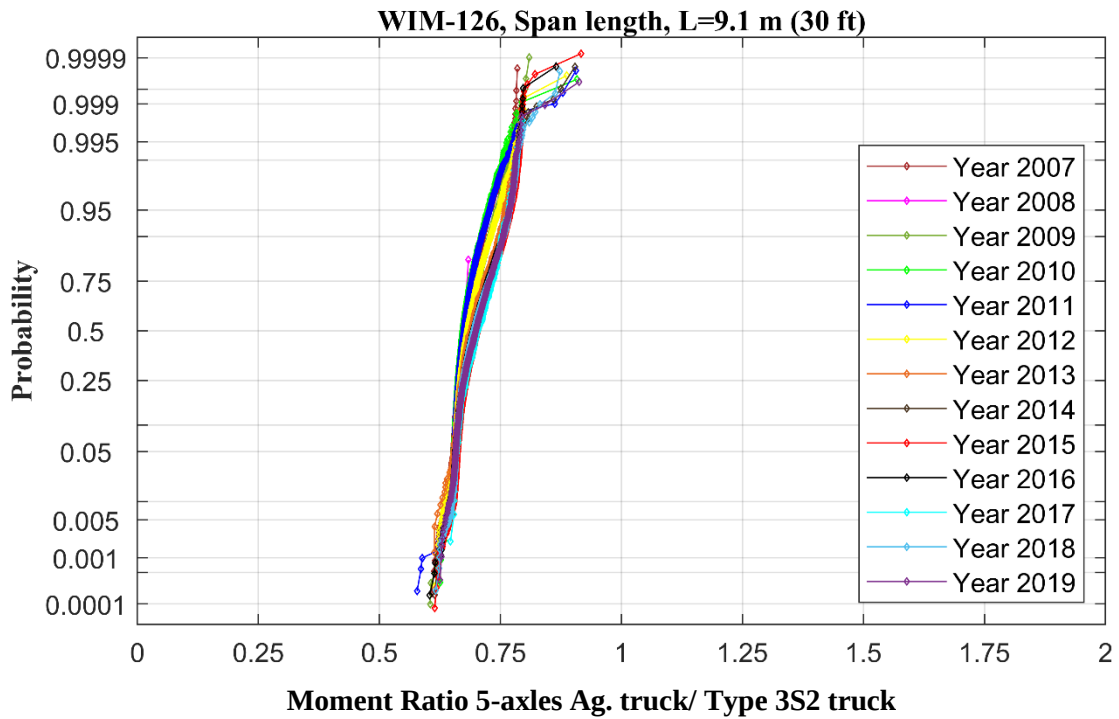
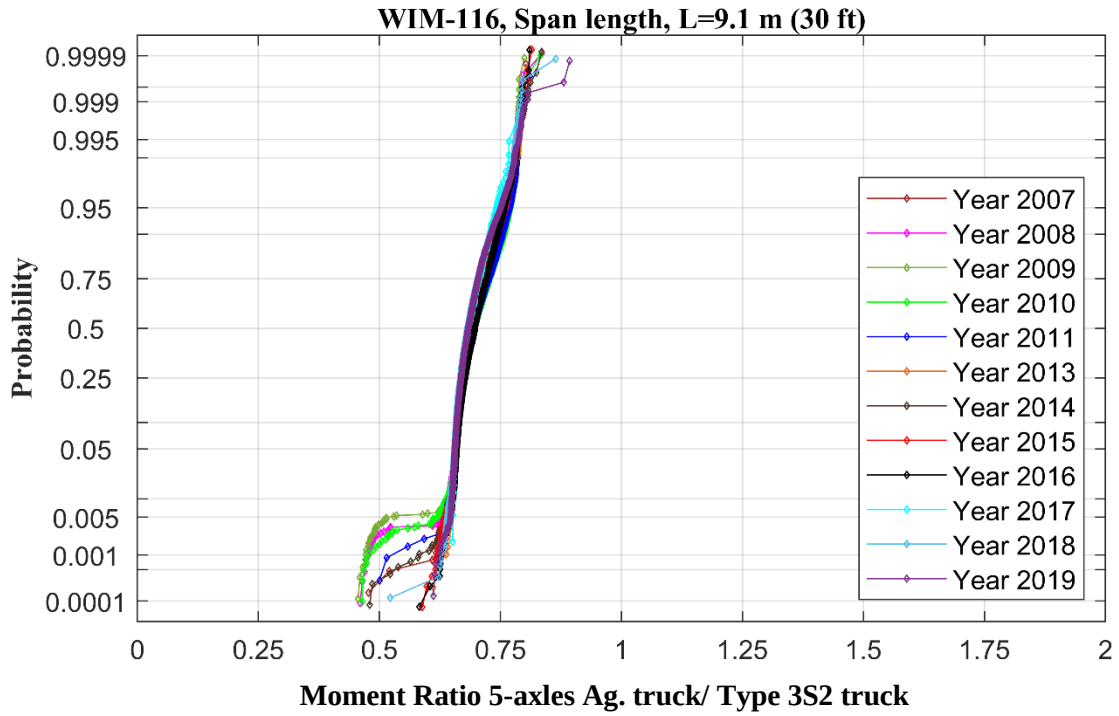
Appendix 4-G. CDF of moment ratio for 5-axle vehicles to Type 3S2 truck



(a)



(b)



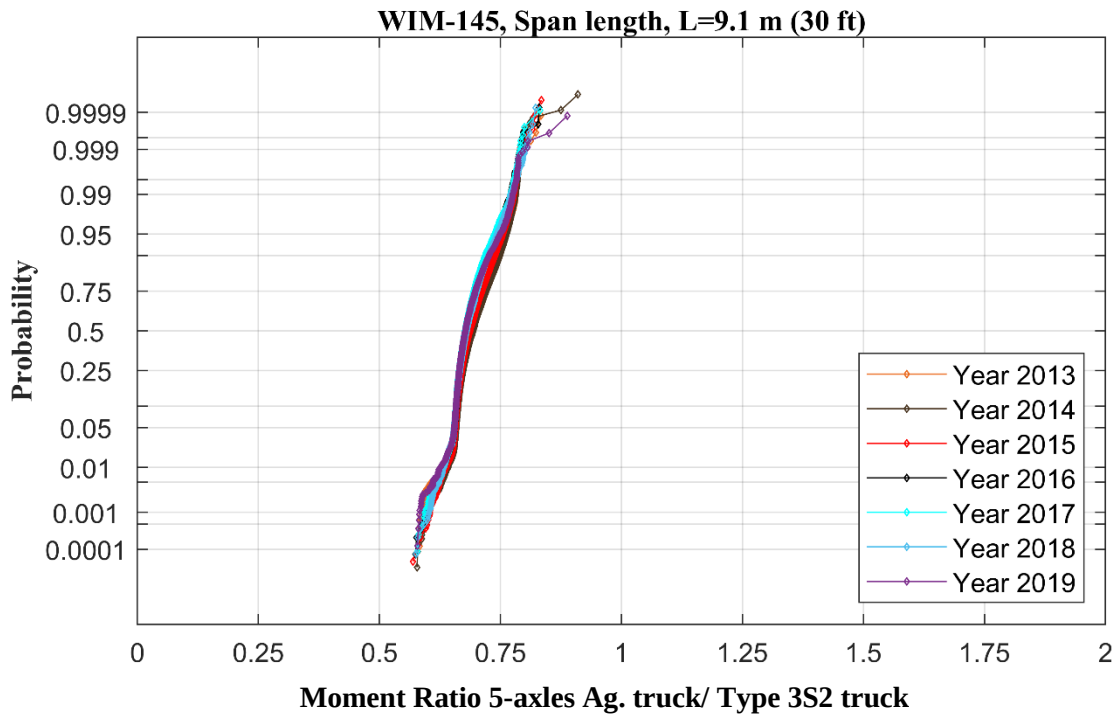
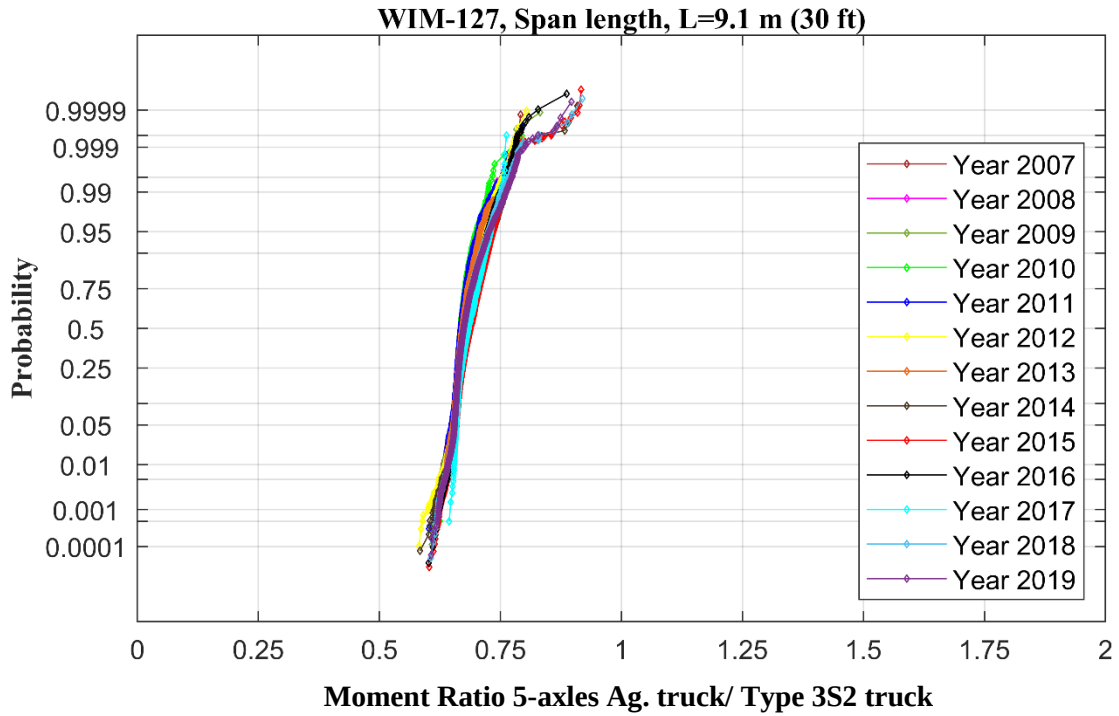
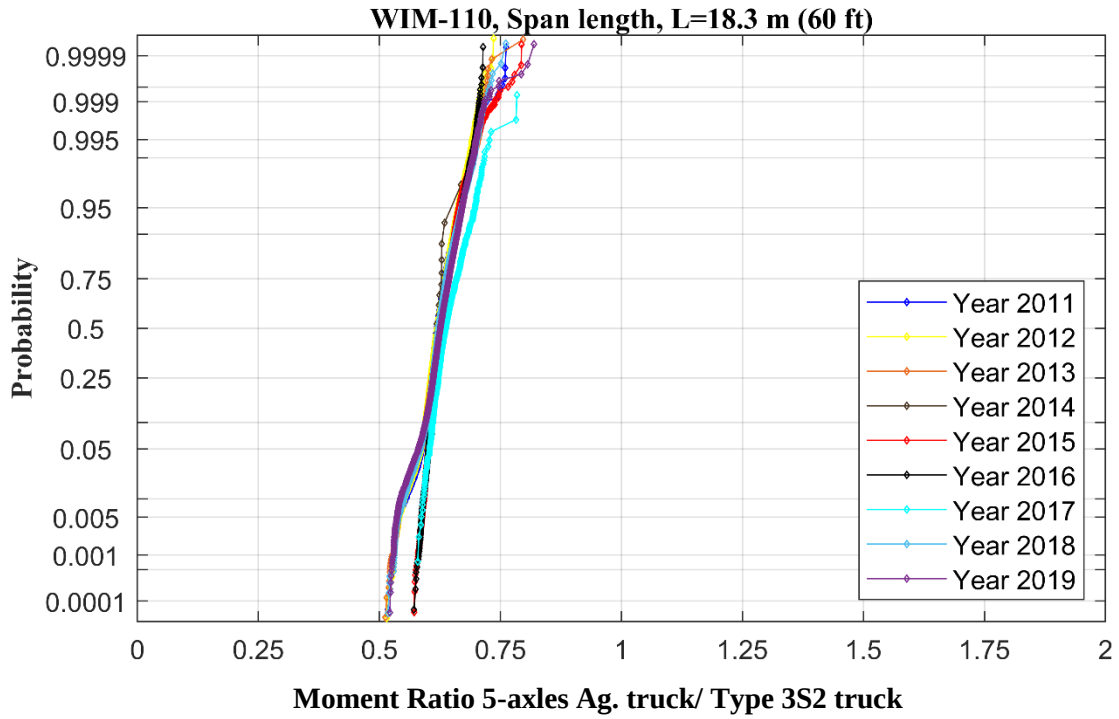
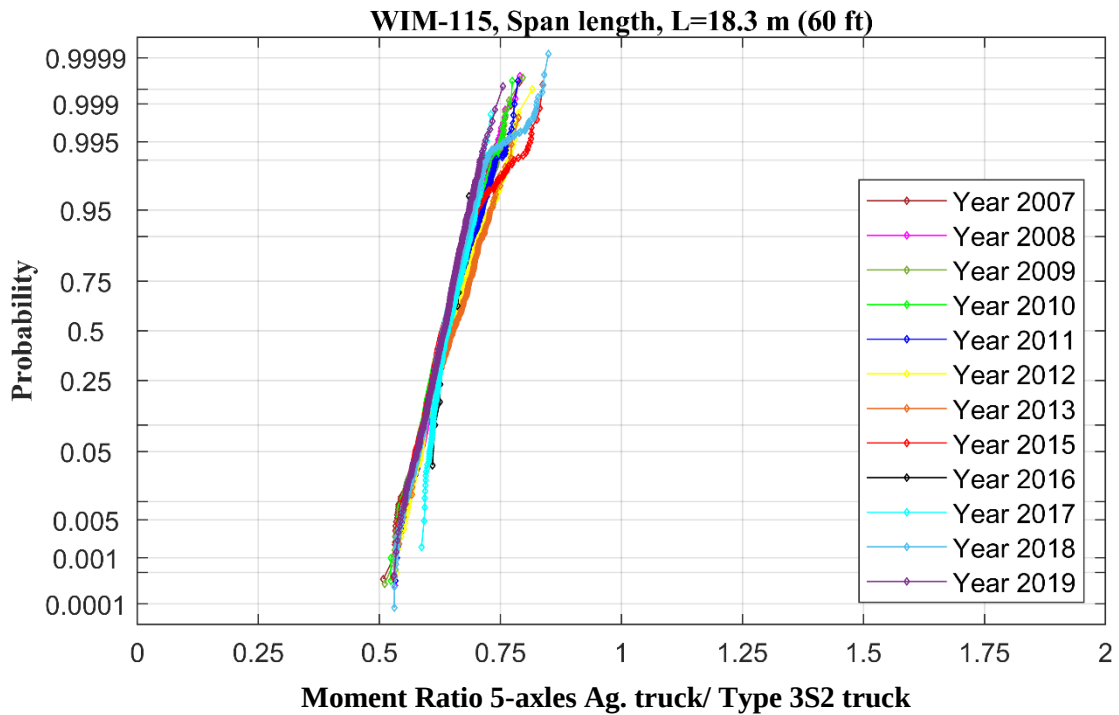


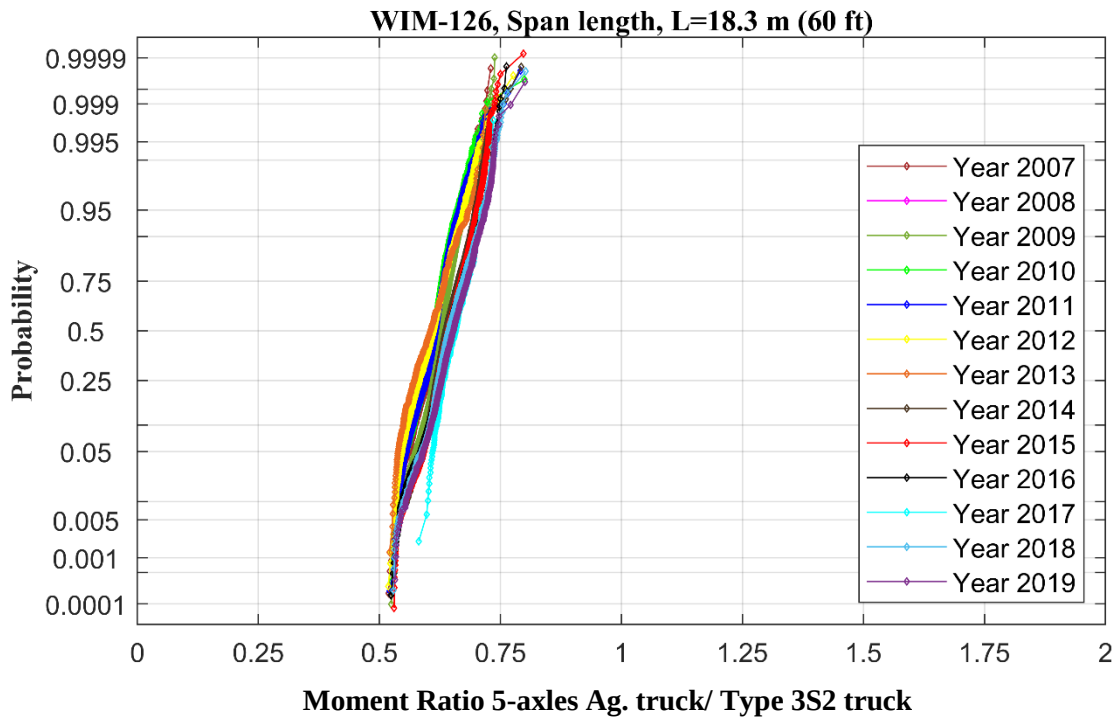
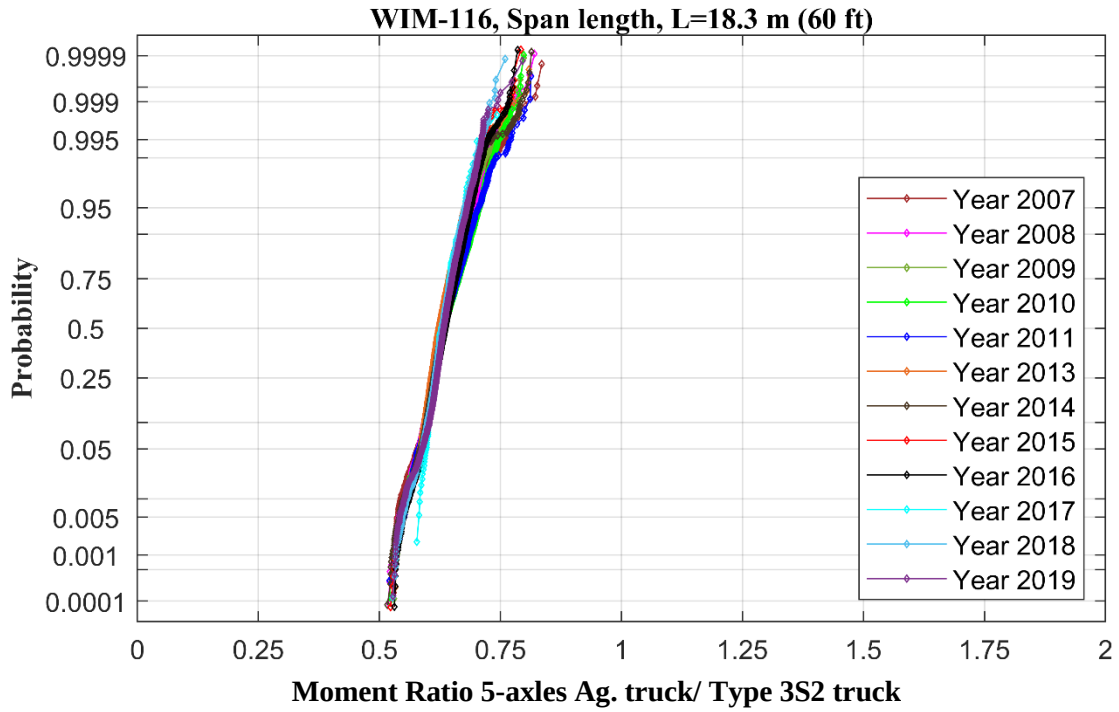
Figure 4-G.1. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 9.1 m (30 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



(a)



(b)



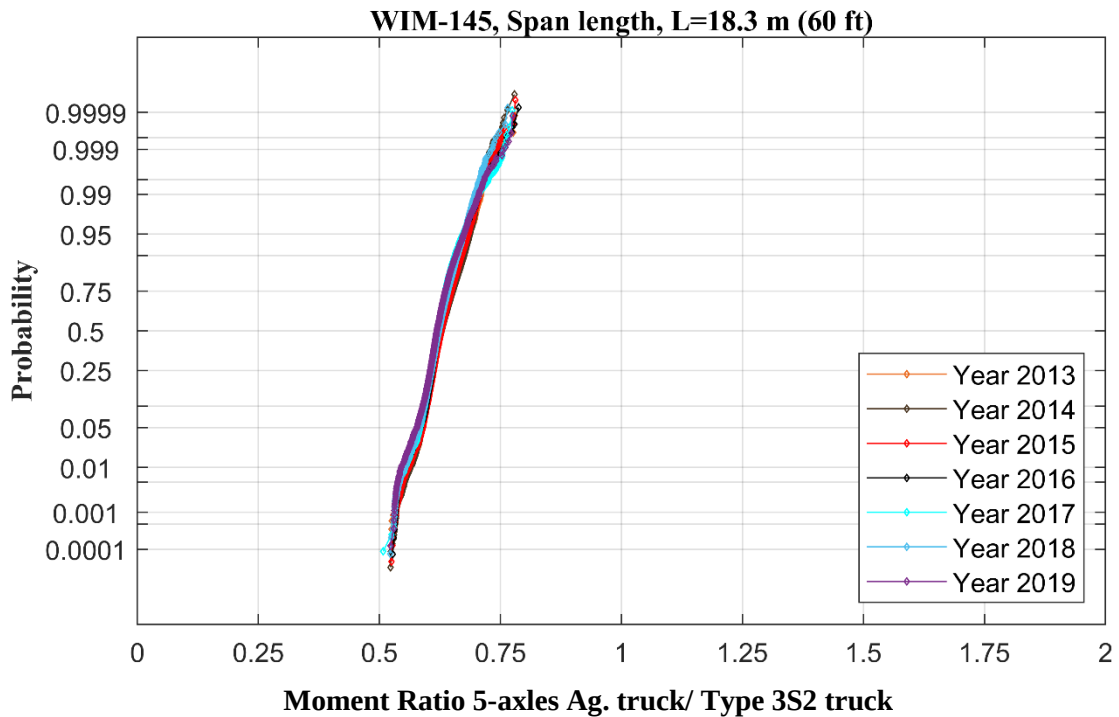
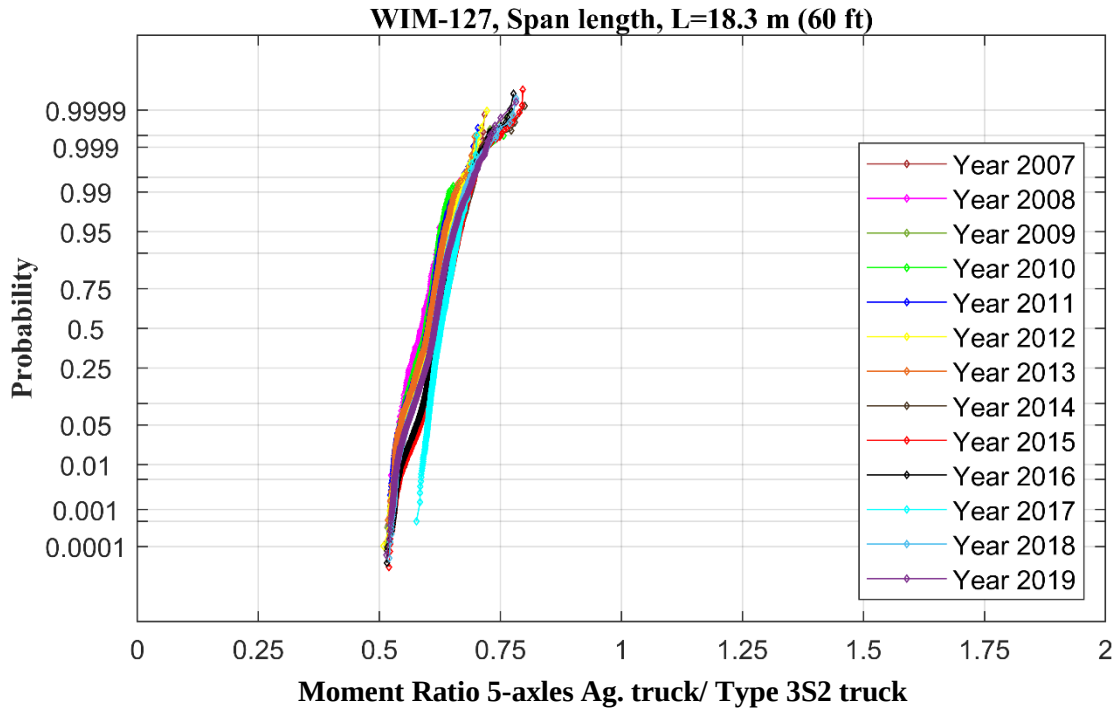
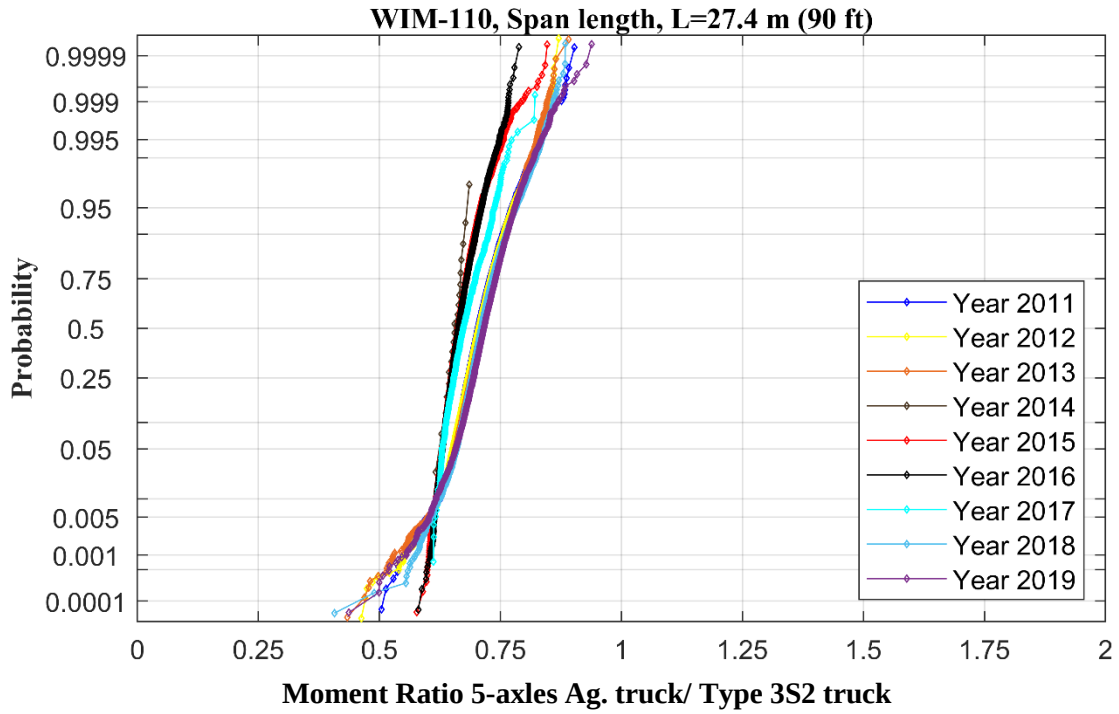
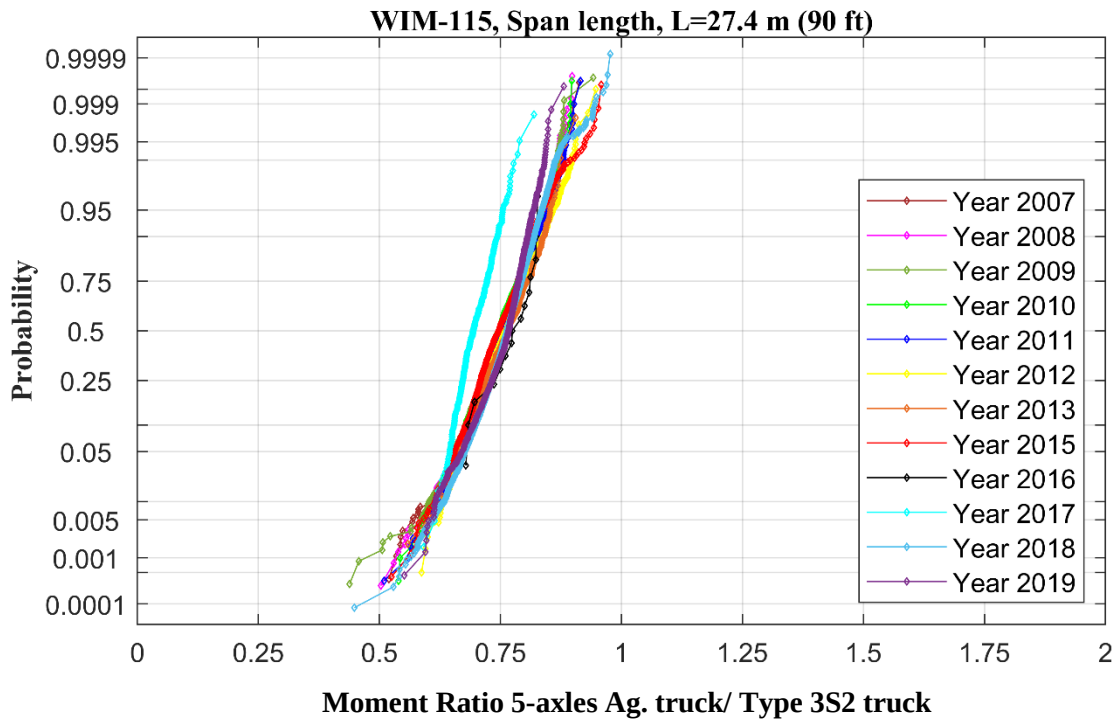


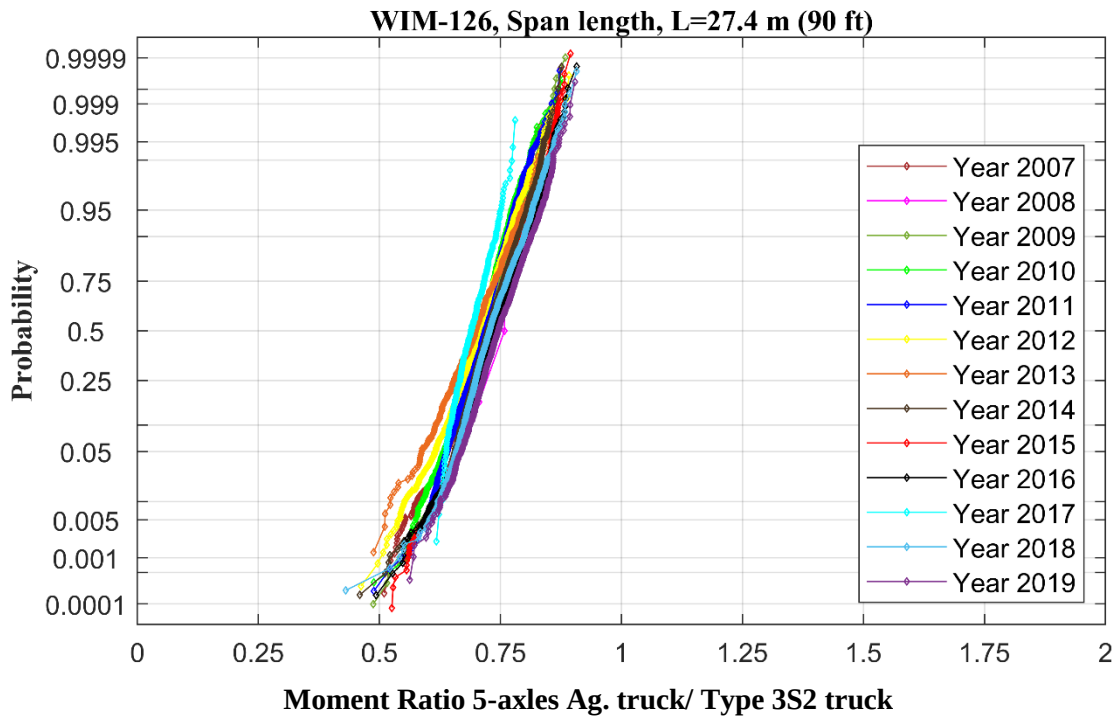
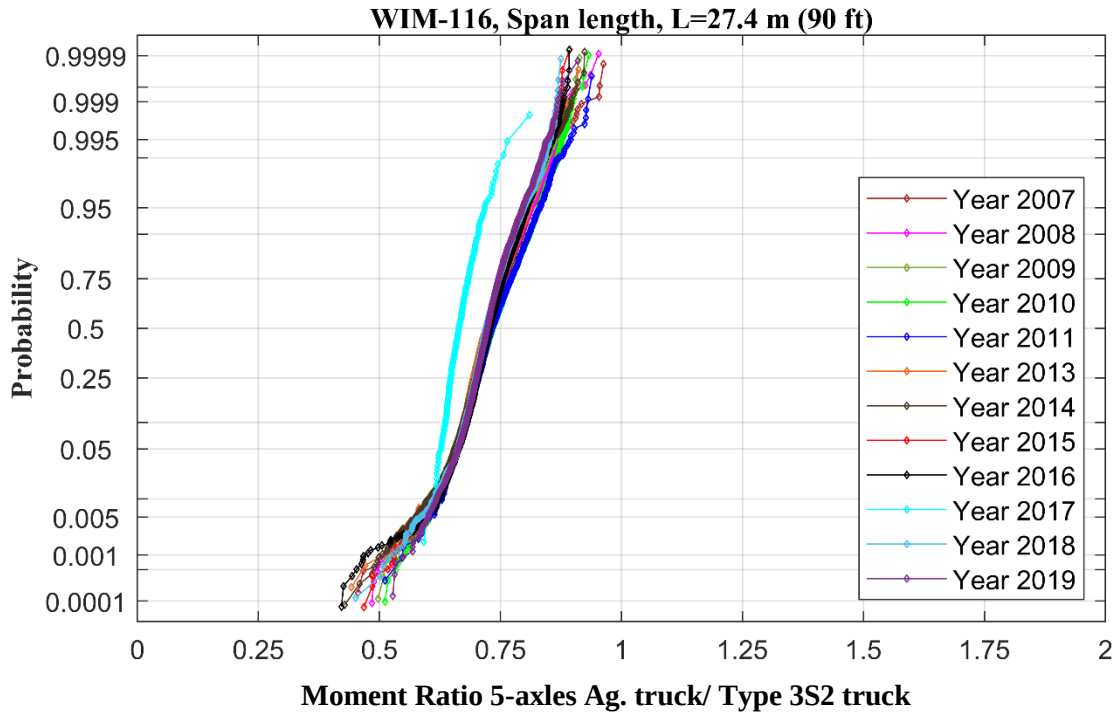
Figure 4-G.2. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 18.3 m (60 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



(a)



(b)



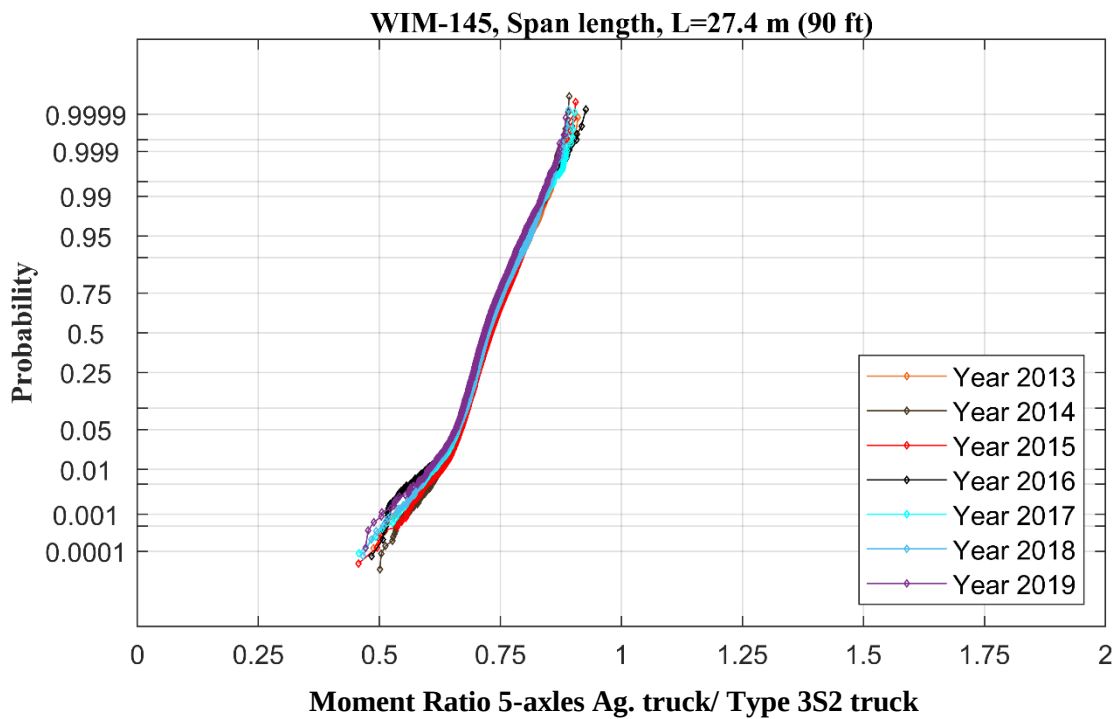
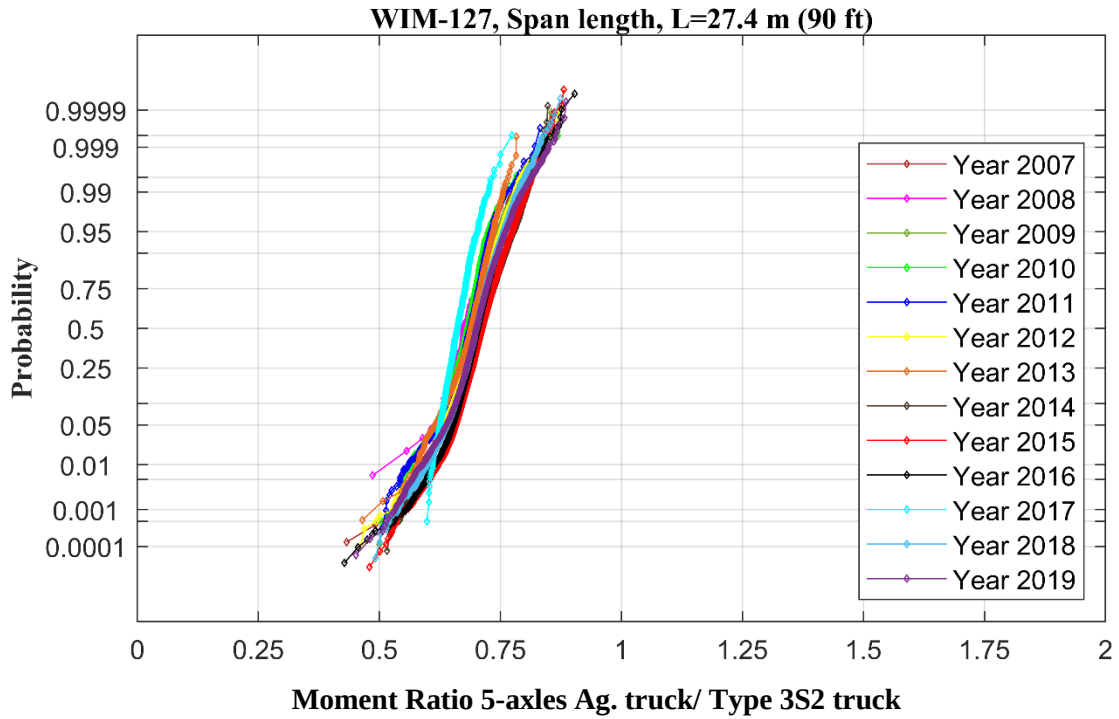
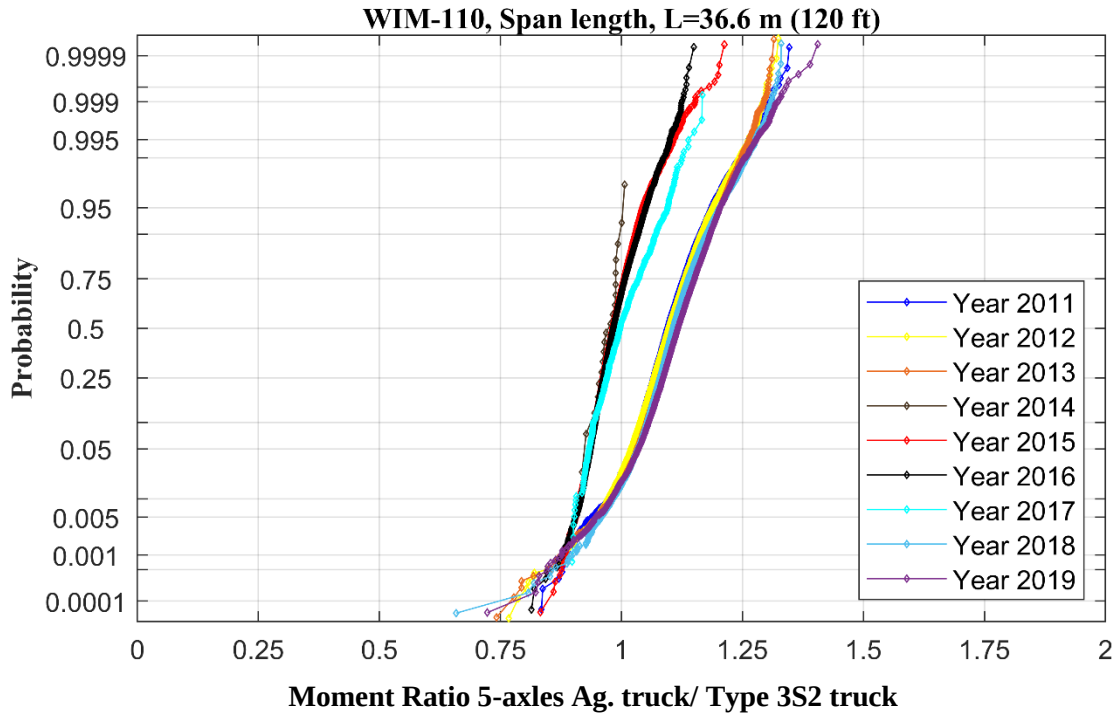
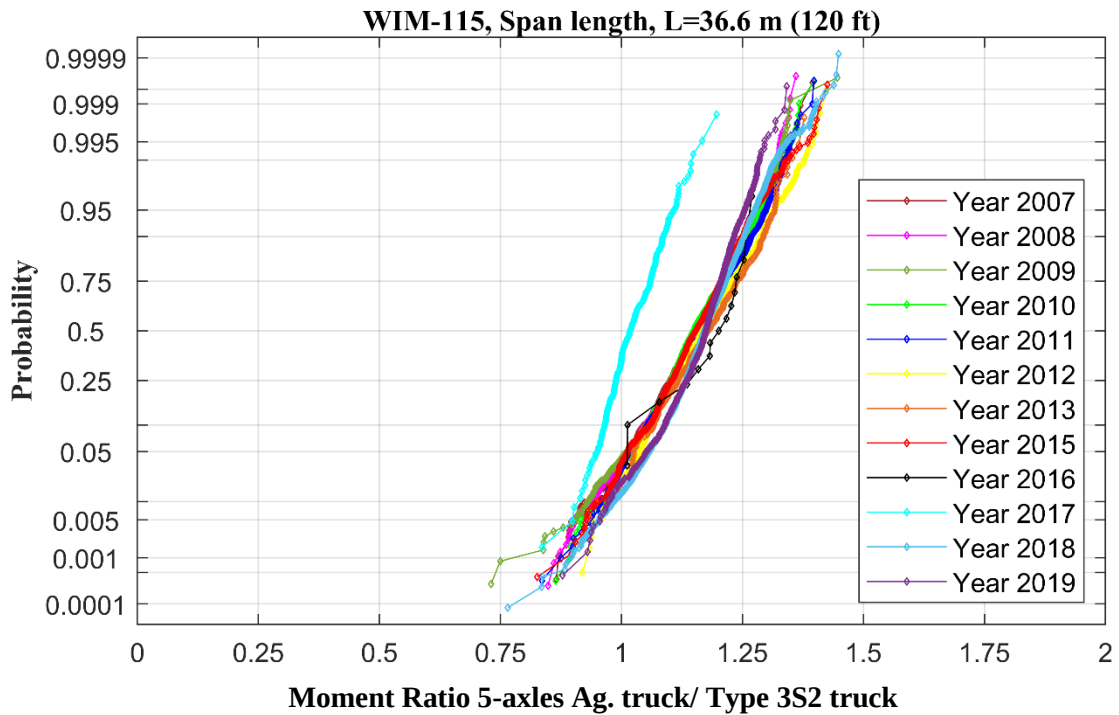


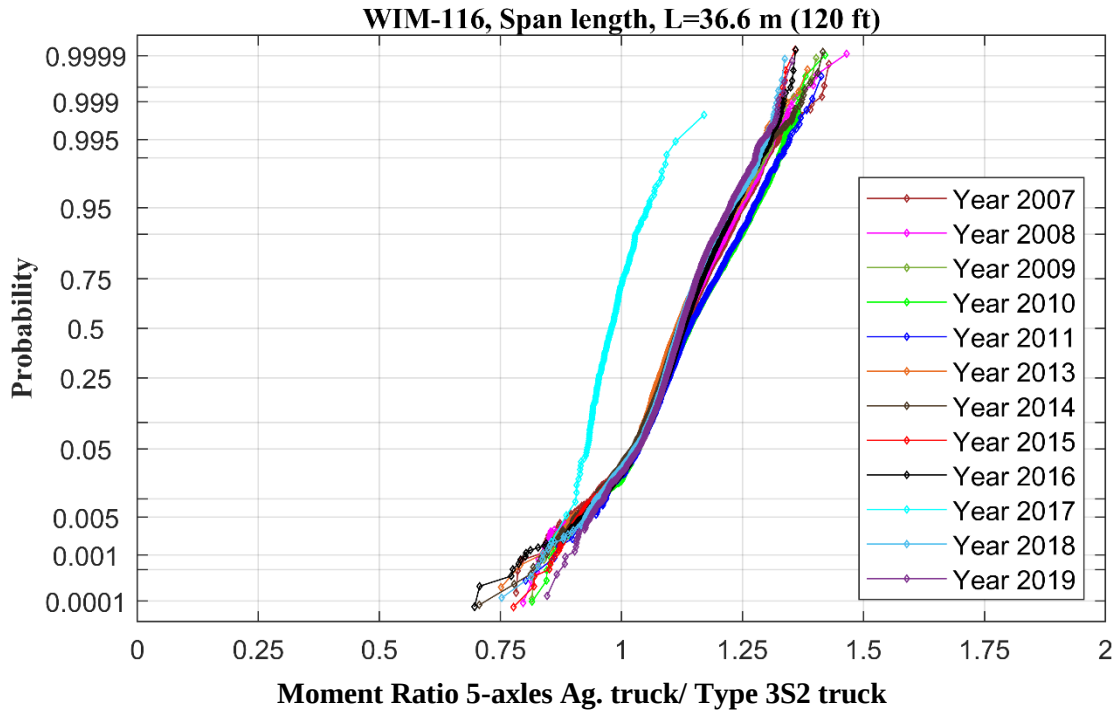
Figure 4-G.3. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 27.4 m (90 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



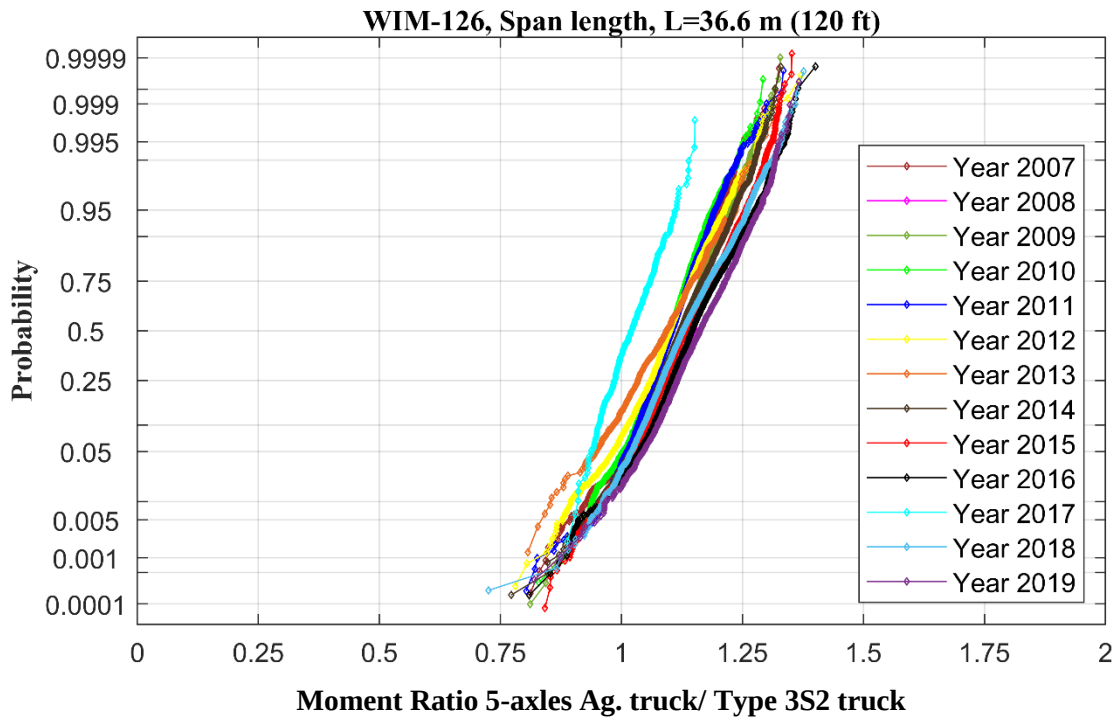
(a)



(b)



(c)



(d)

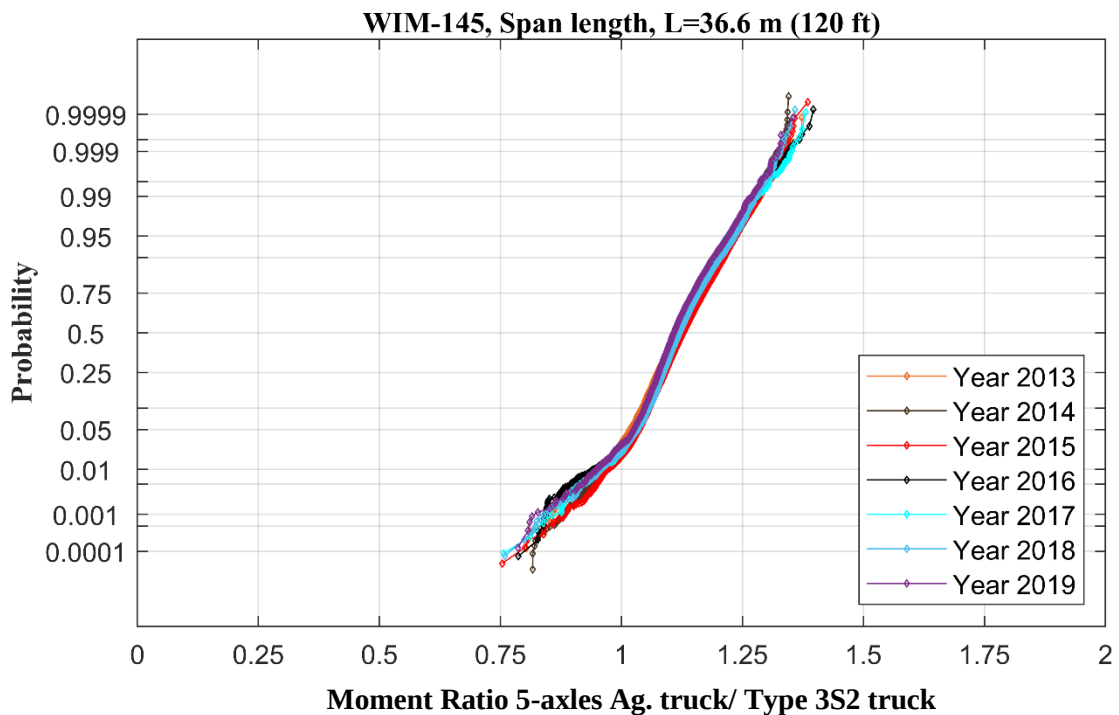
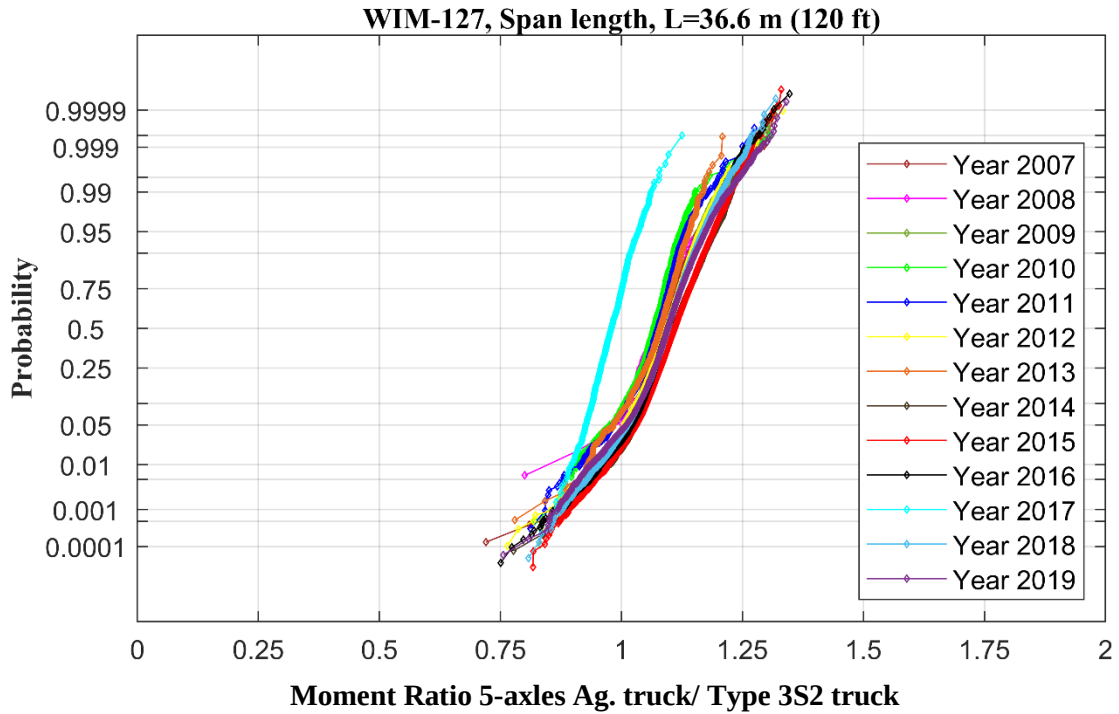
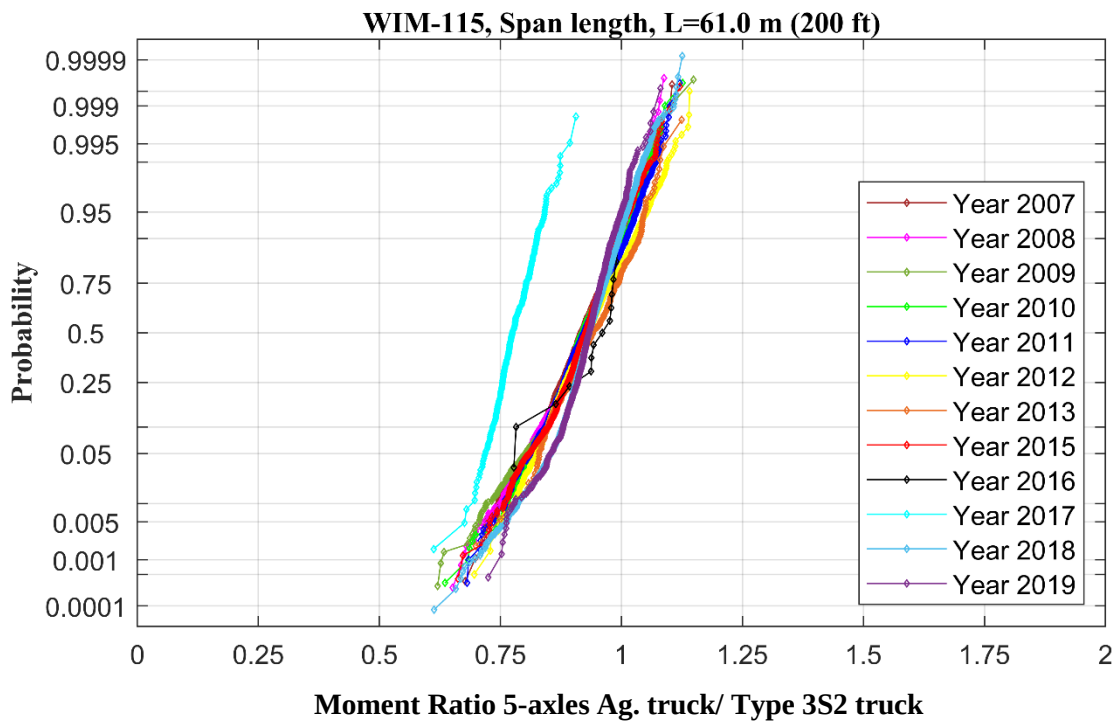
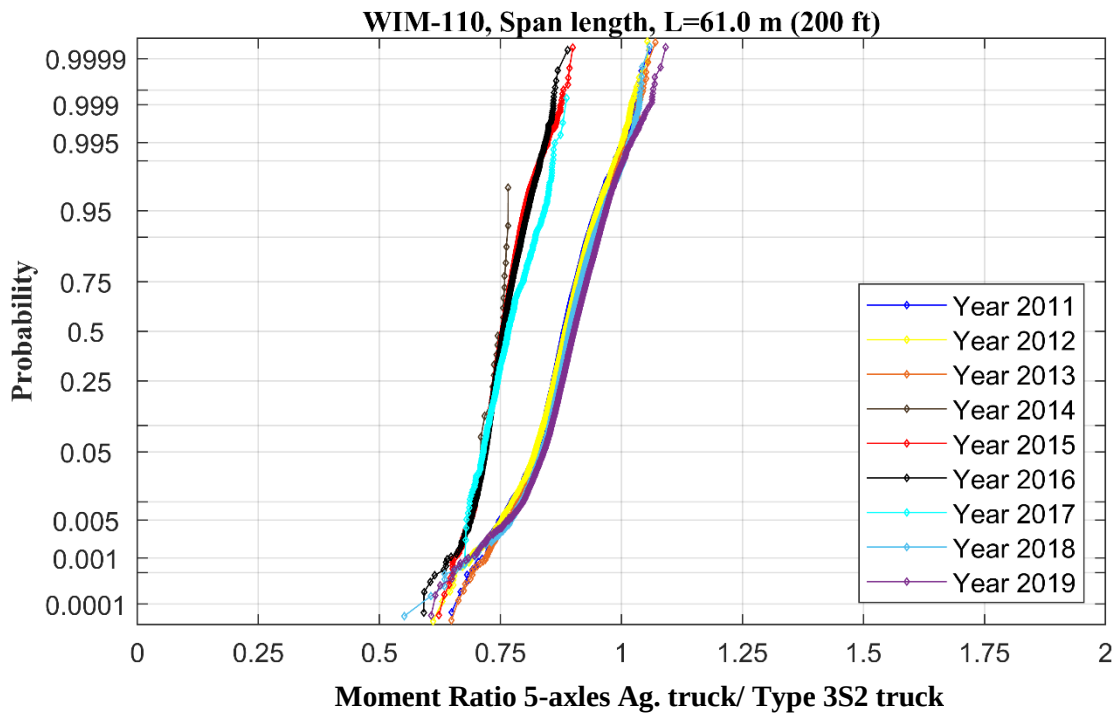
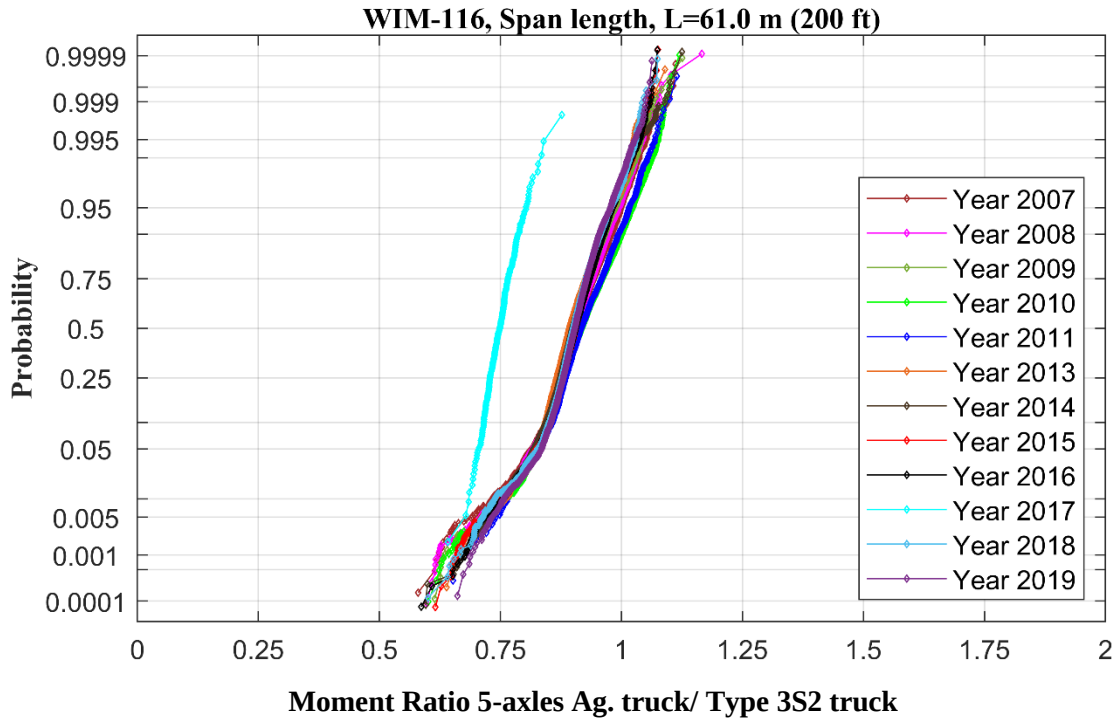
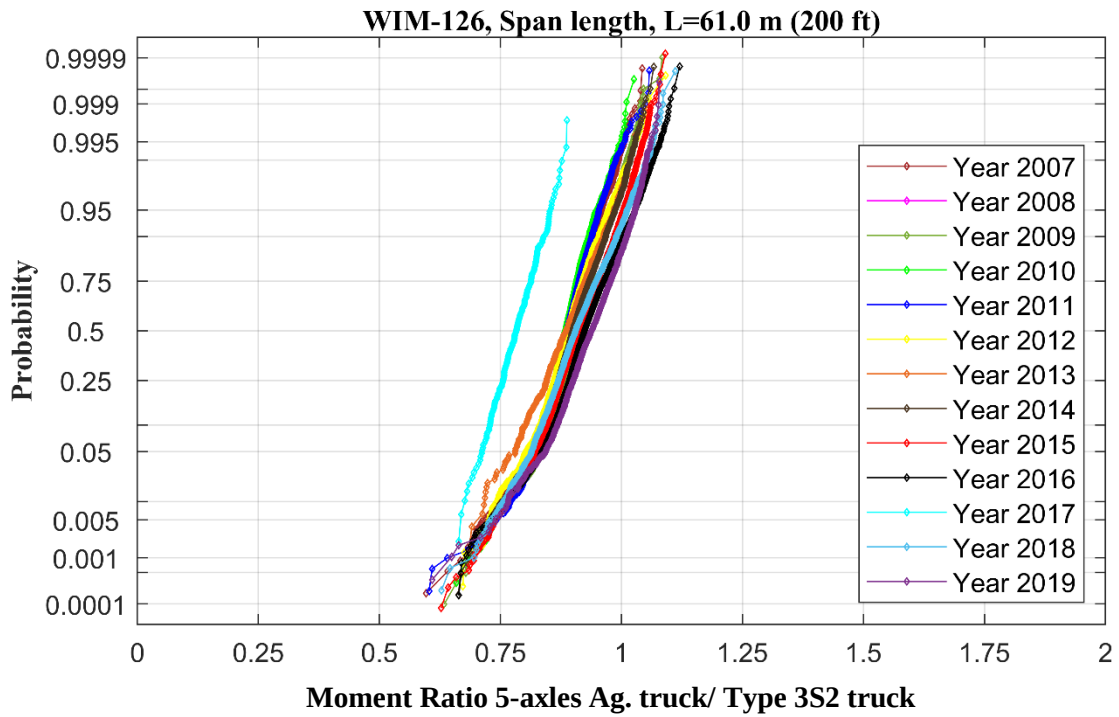


Figure 4-G.4. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 36.6 m (120 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





(c)



(d)

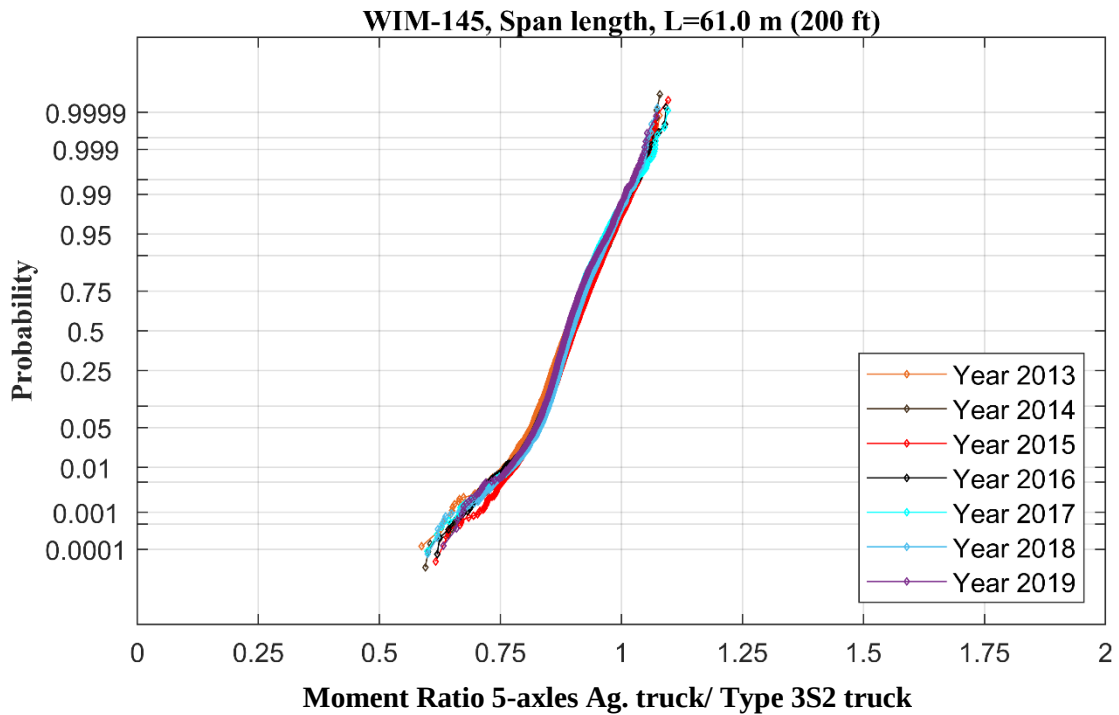
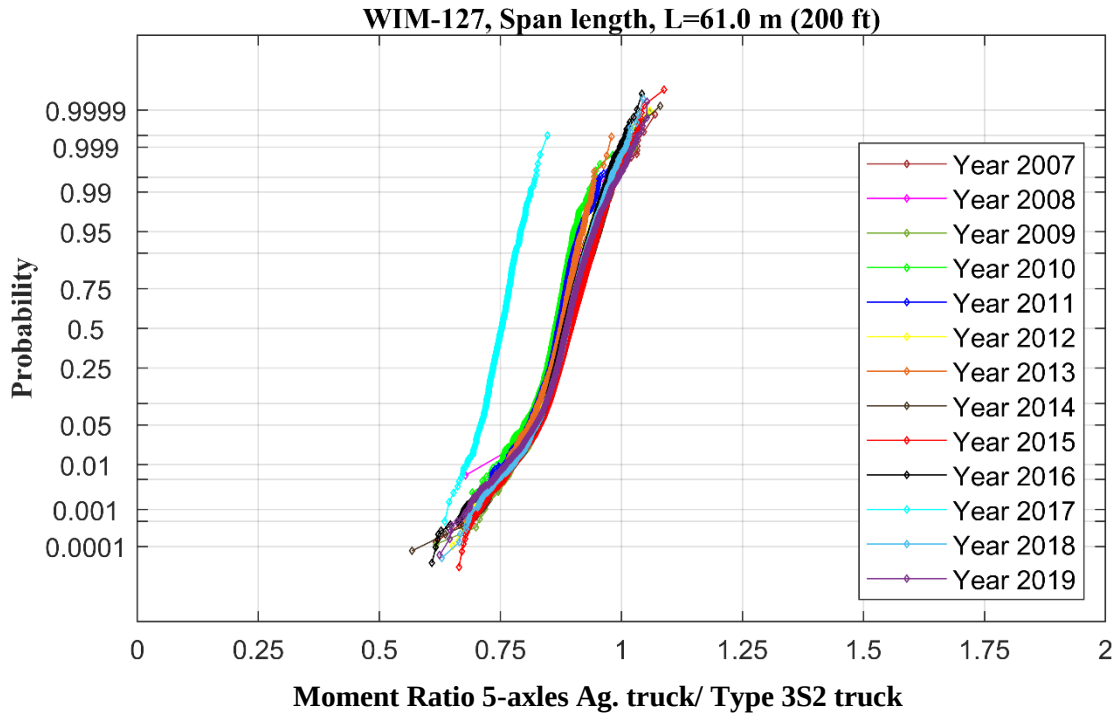
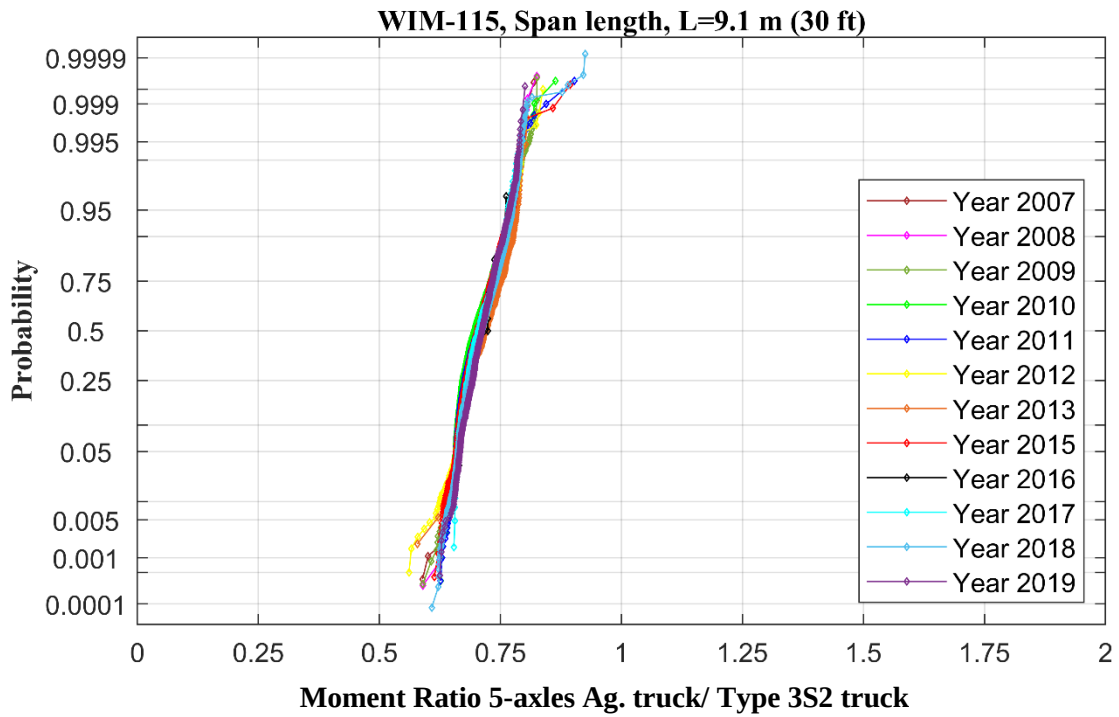
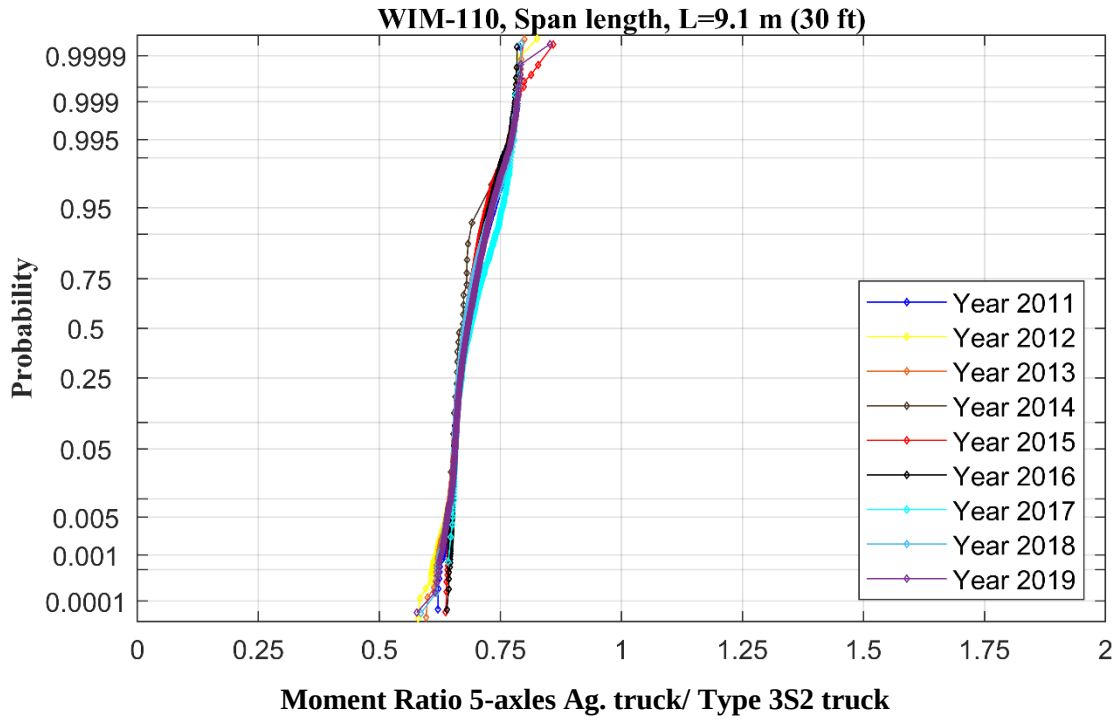
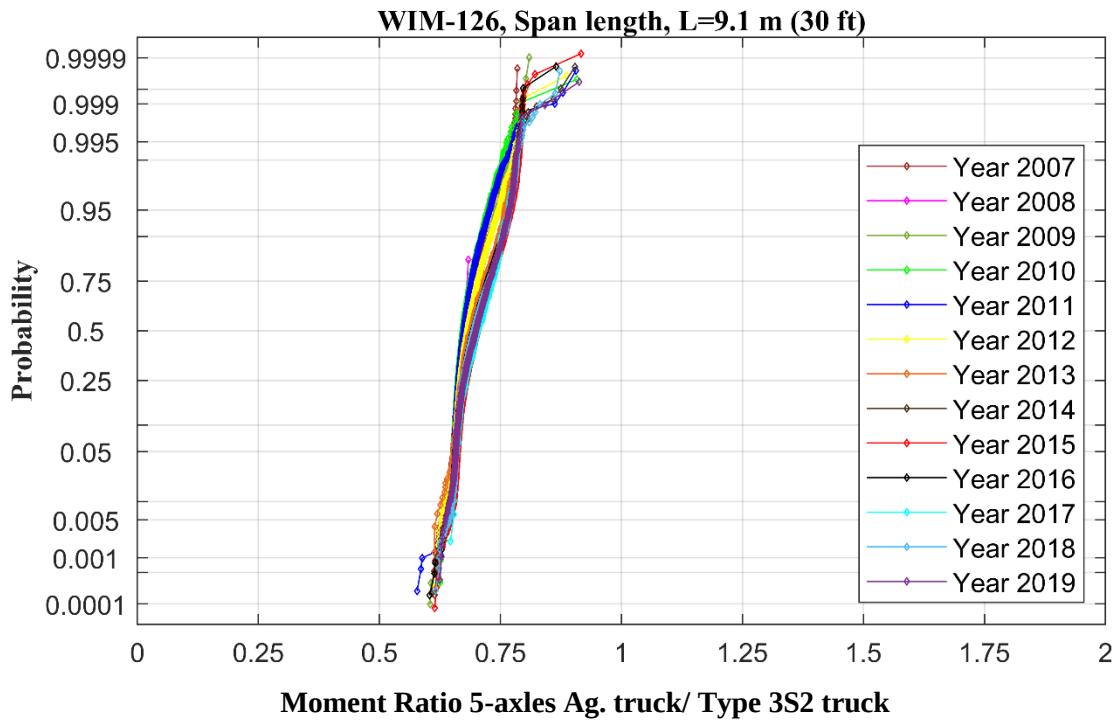
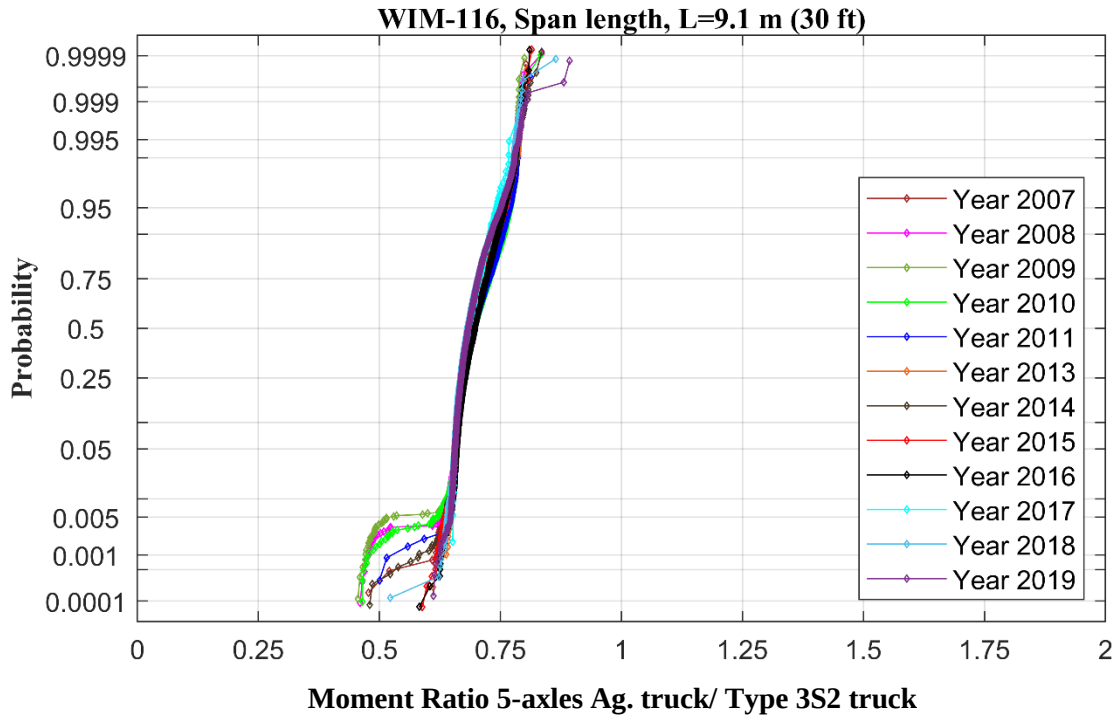


Figure 4-G.5. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 61 m (200 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





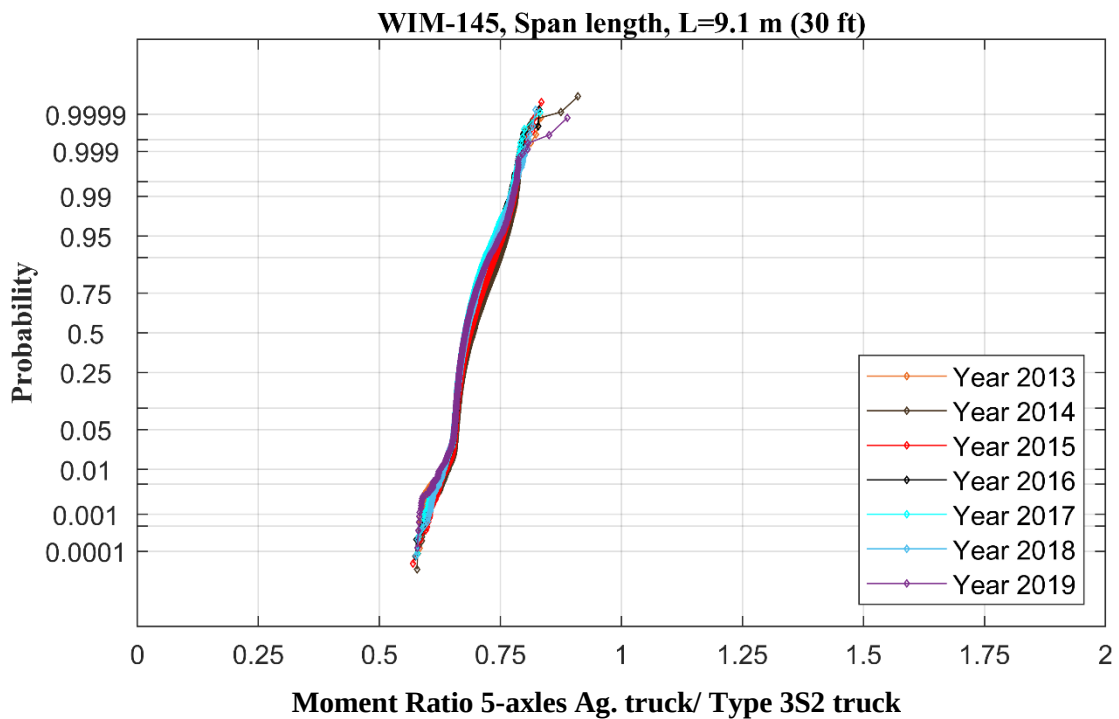
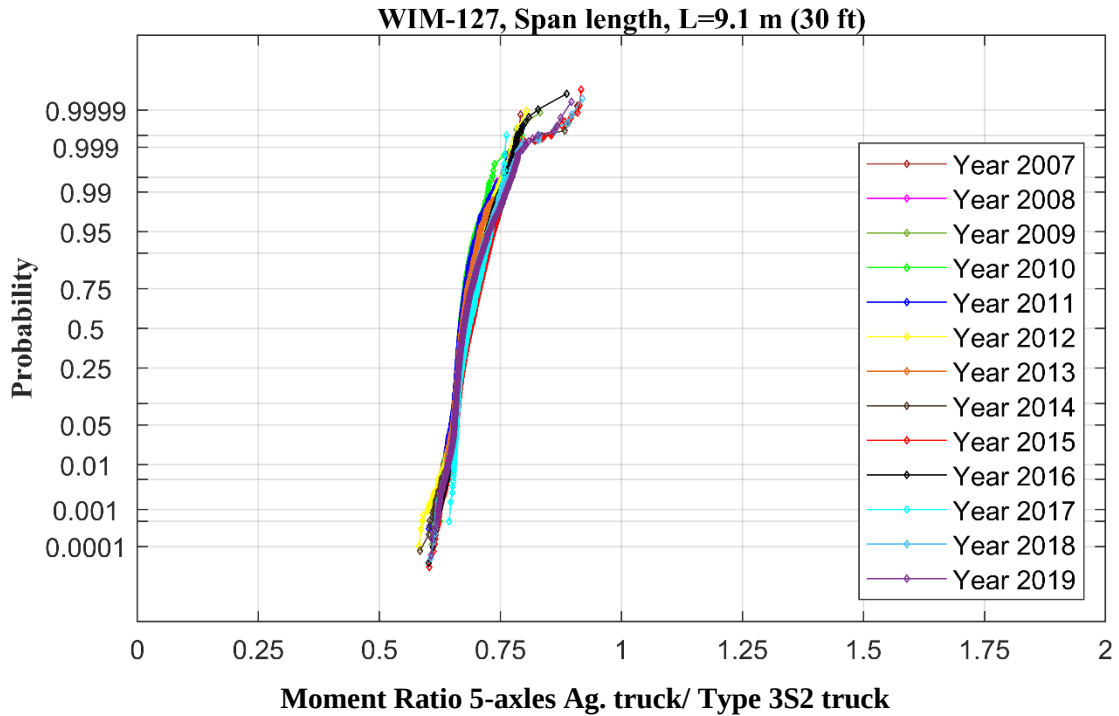
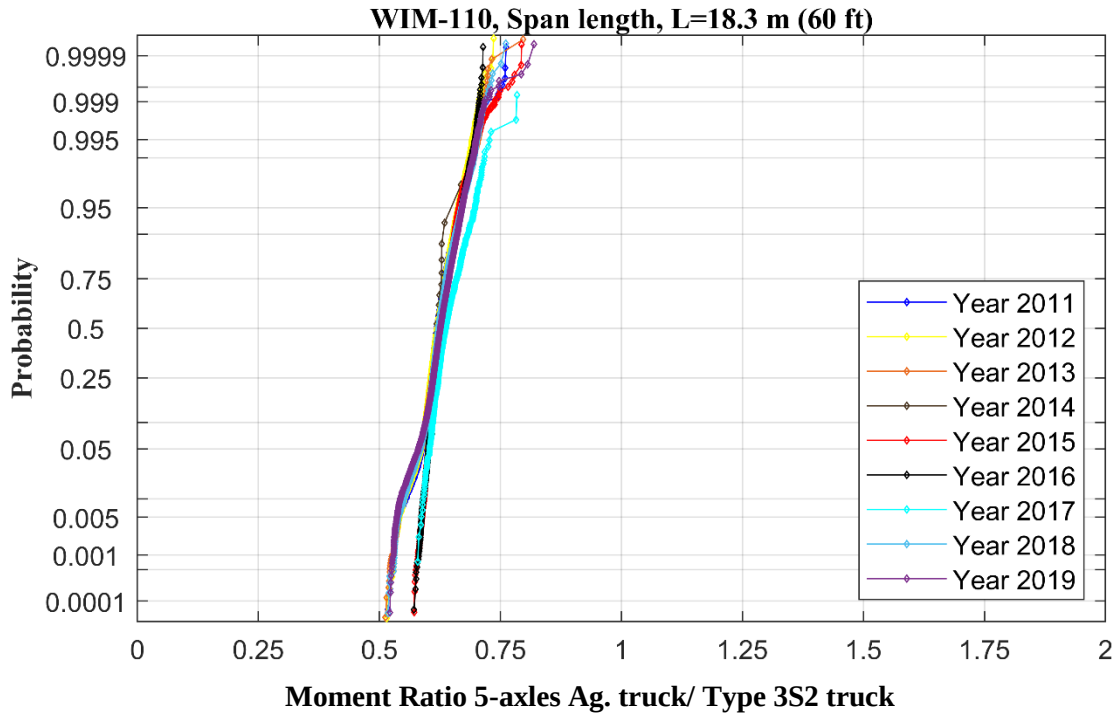
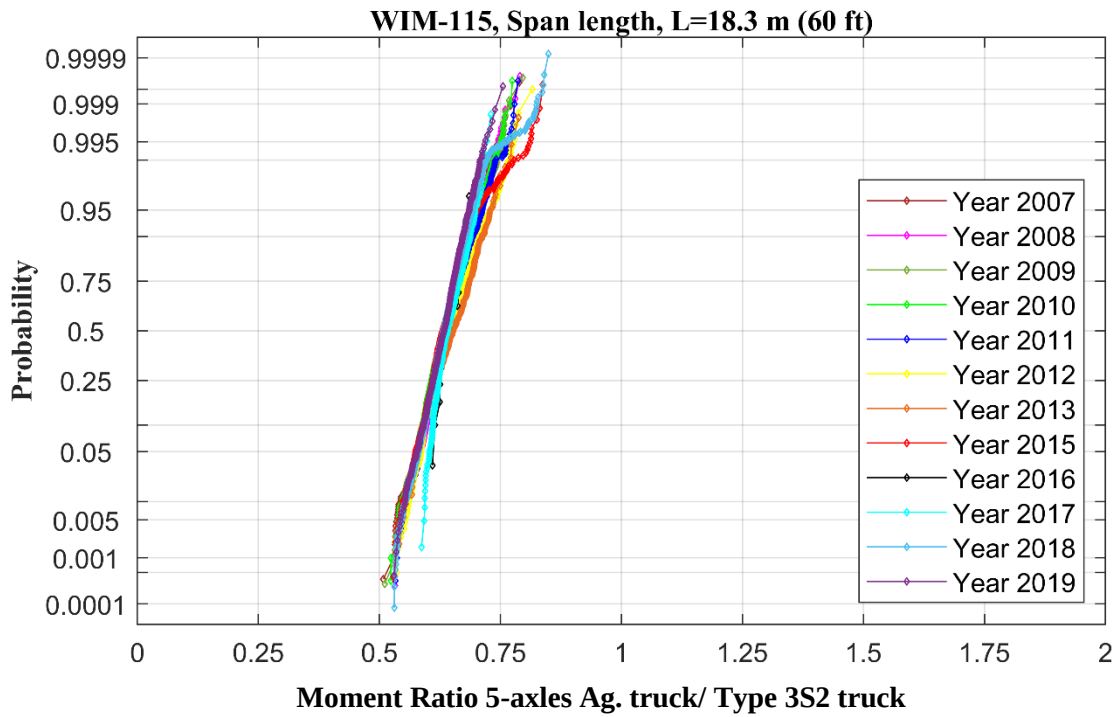


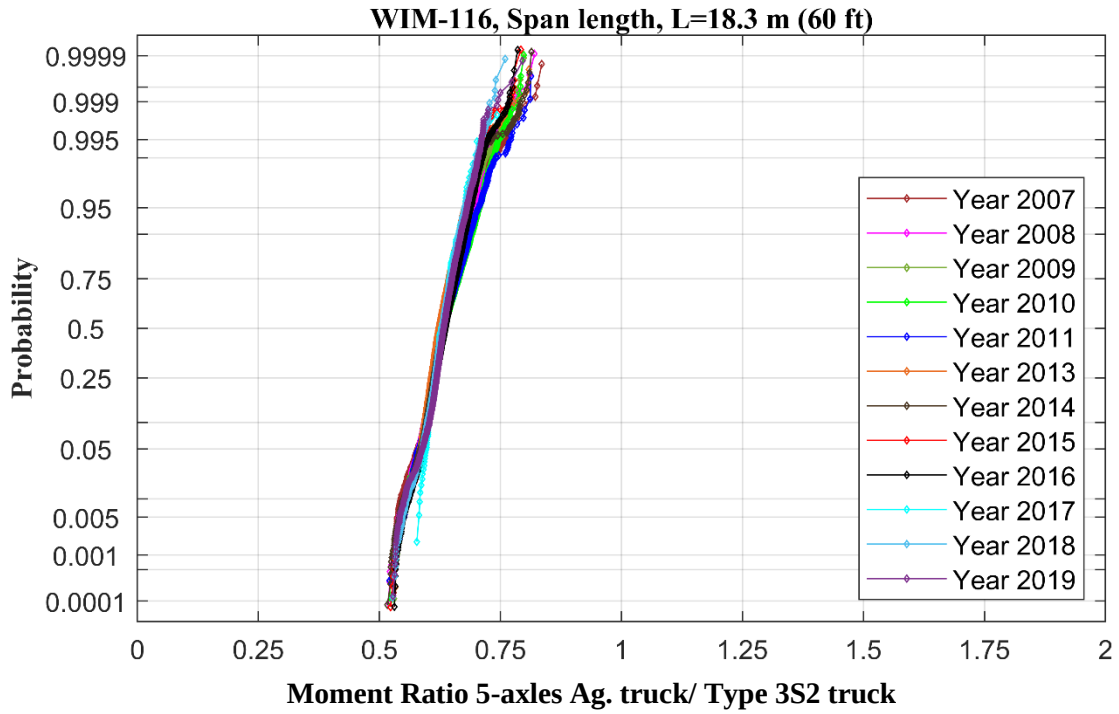
Figure 4-G.1. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 9.1 m (30 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



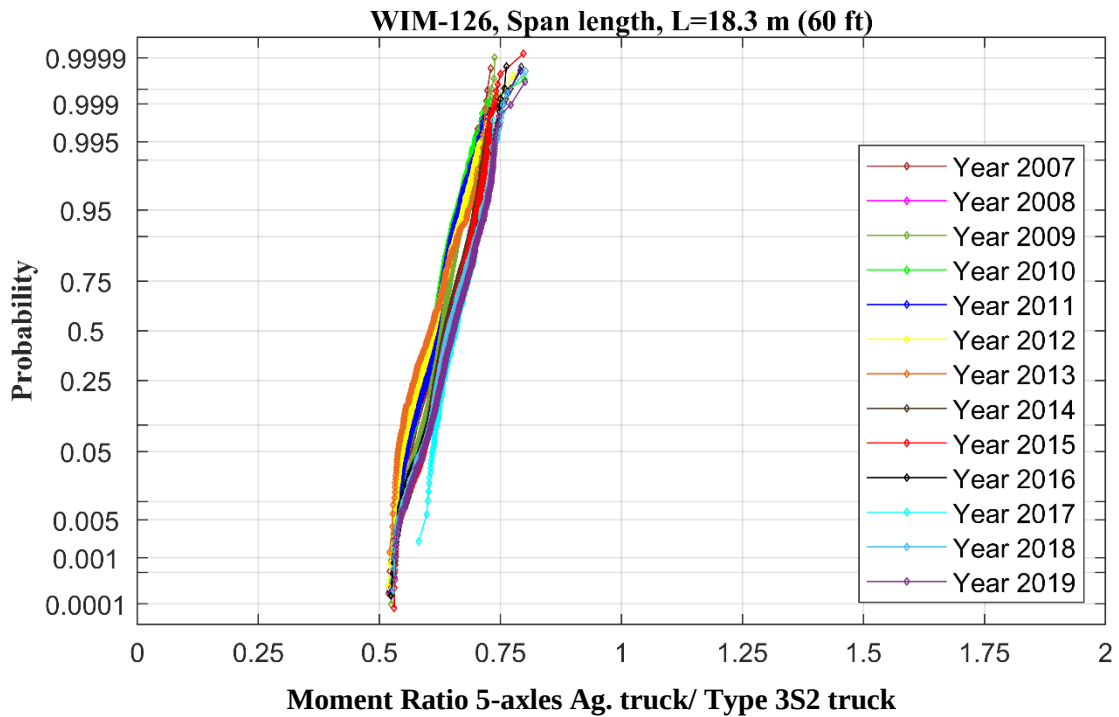
(a)



(b)



(c)



(d)

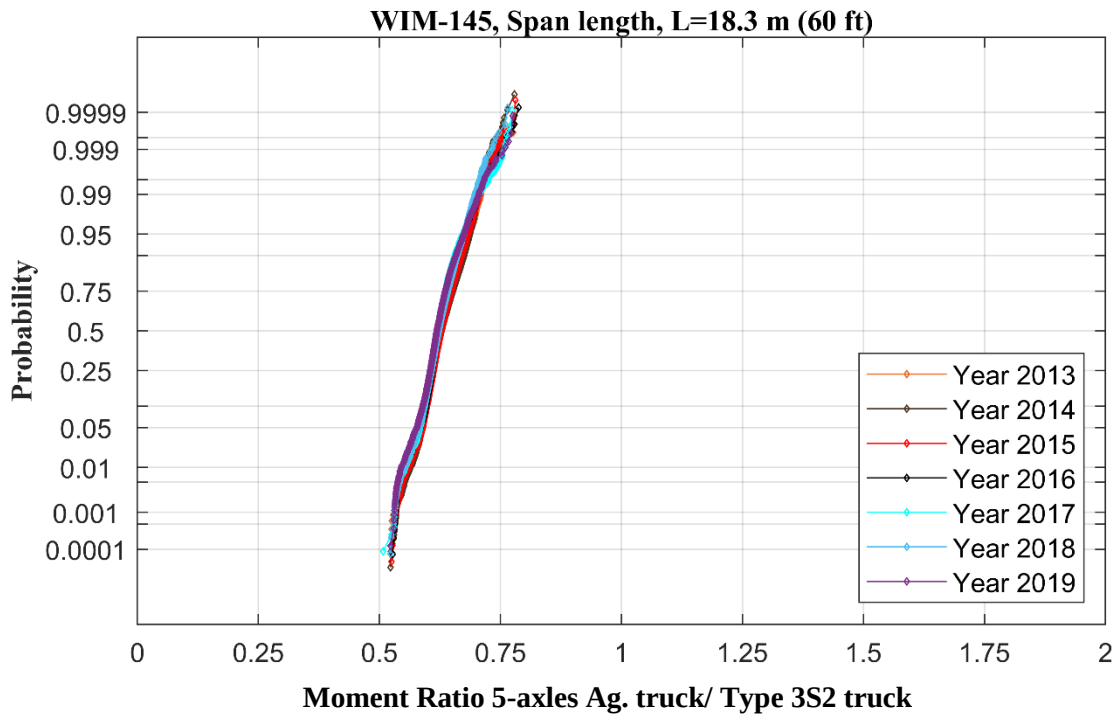
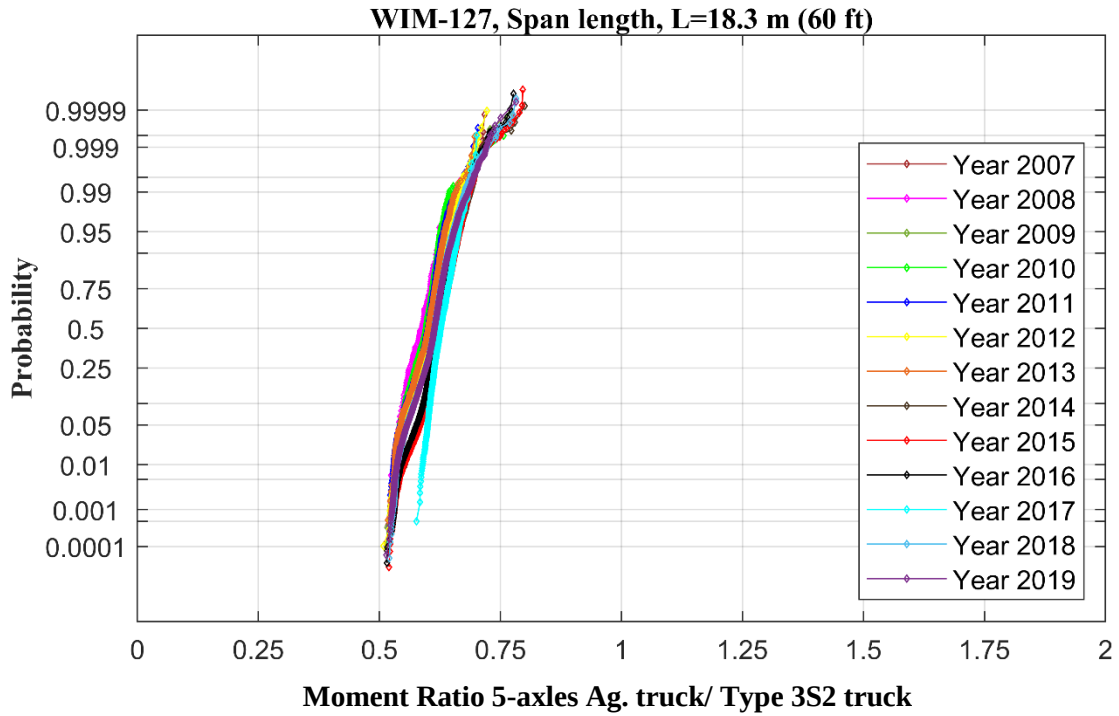
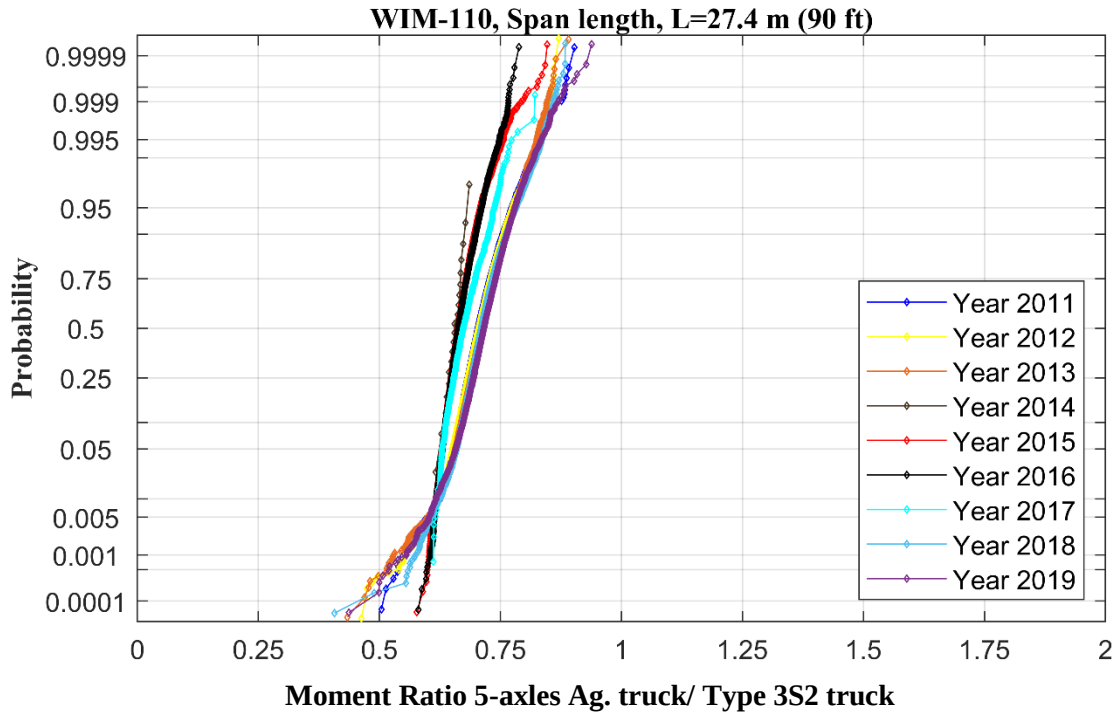
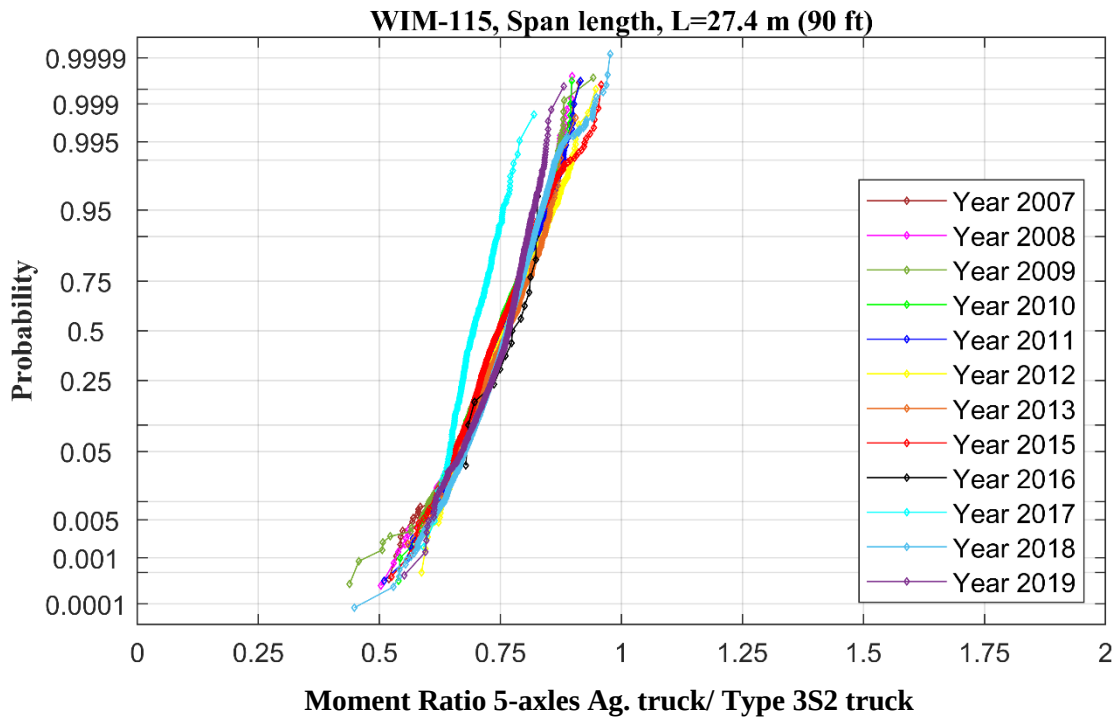


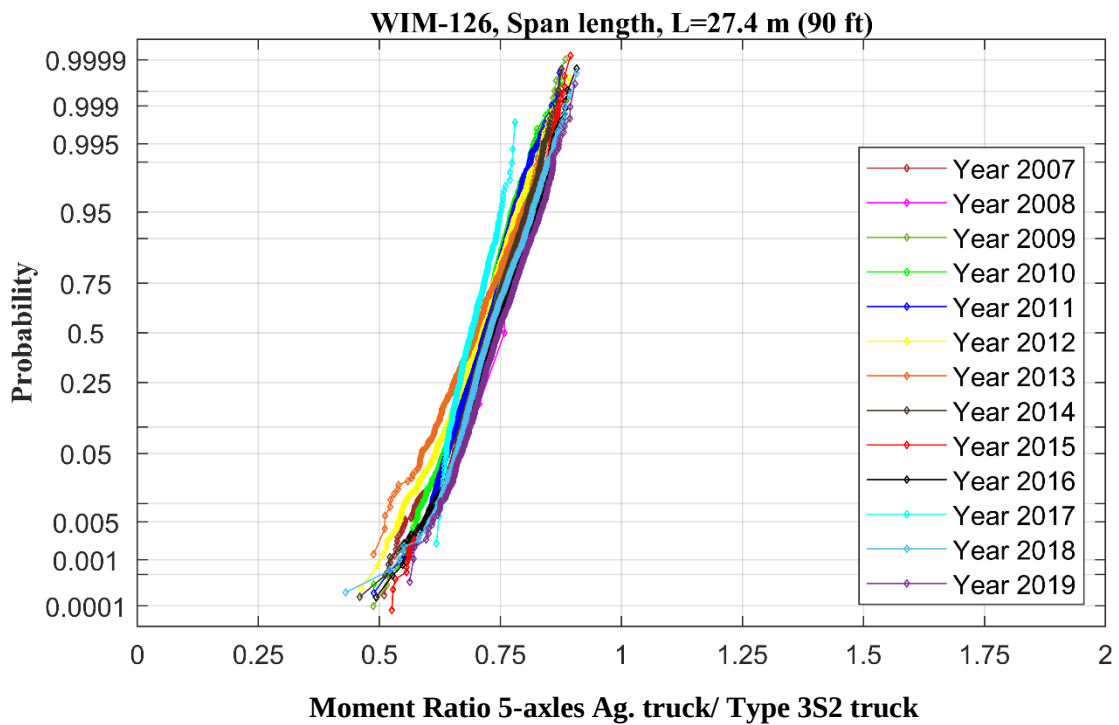
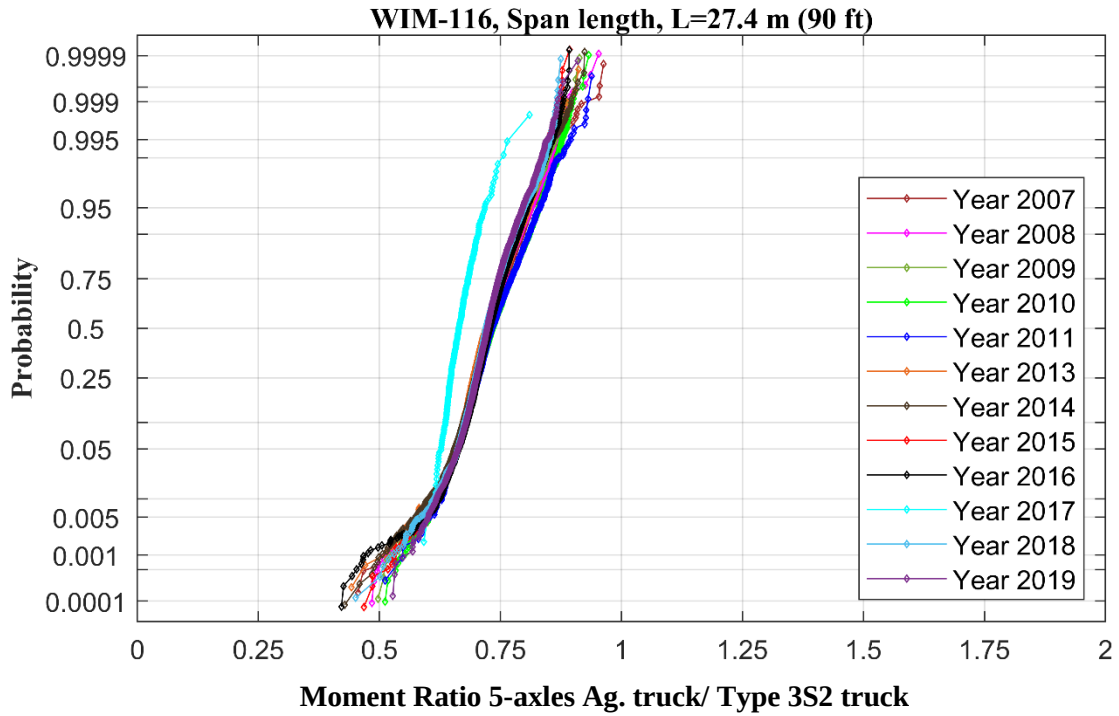
Figure 4-G.2. CDF of moment ratio for 5-axles agricultural vehicles to Type 3S2 vehicle for span length of 18.3 m (60 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



(a)



(b)



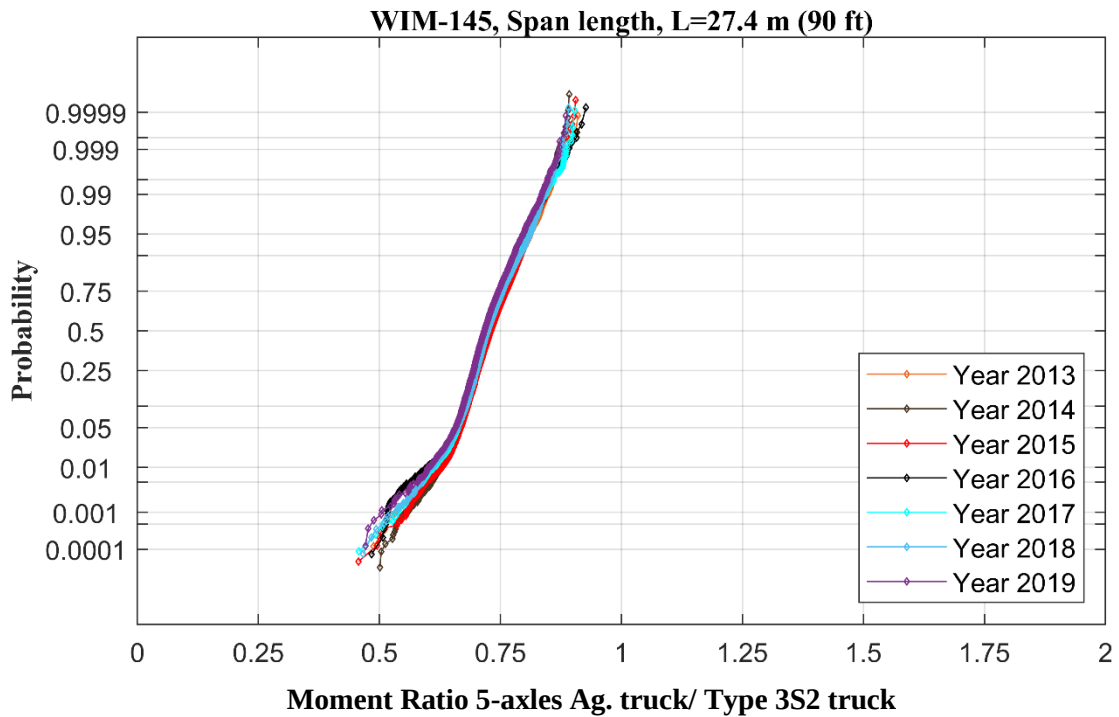
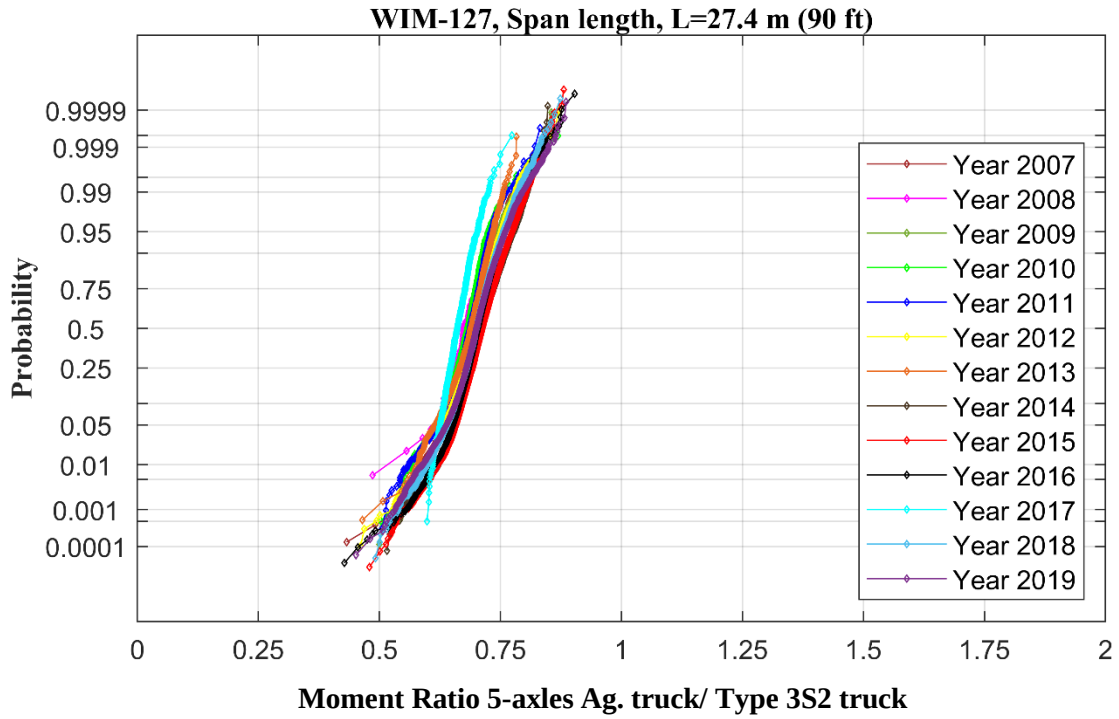
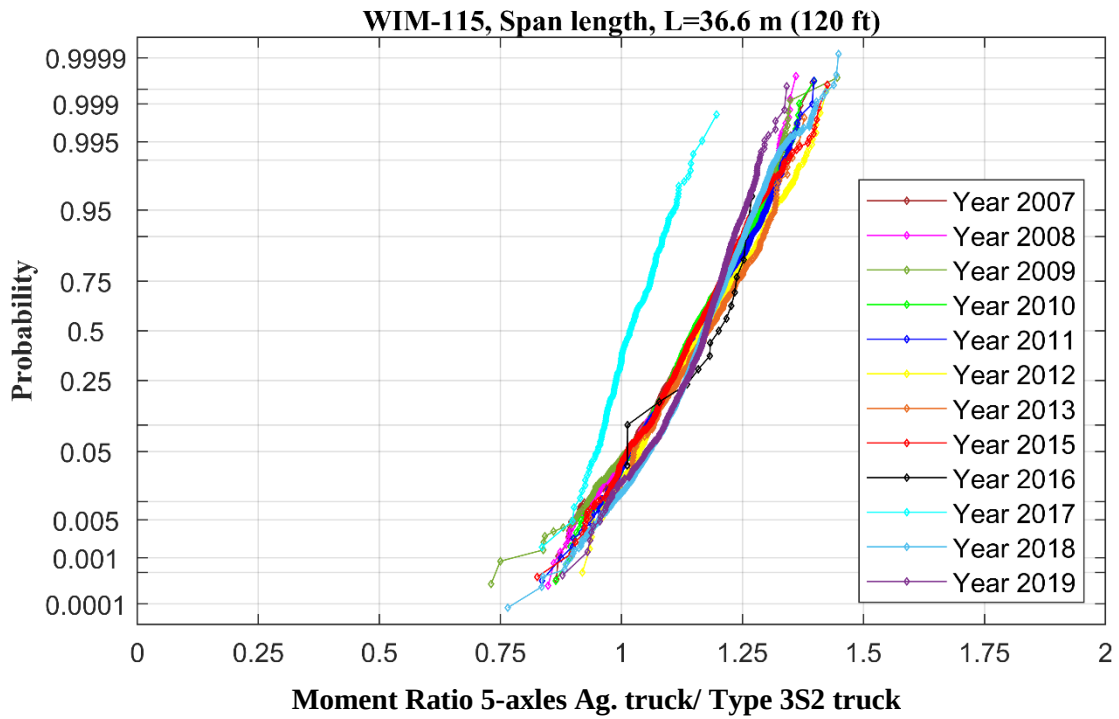
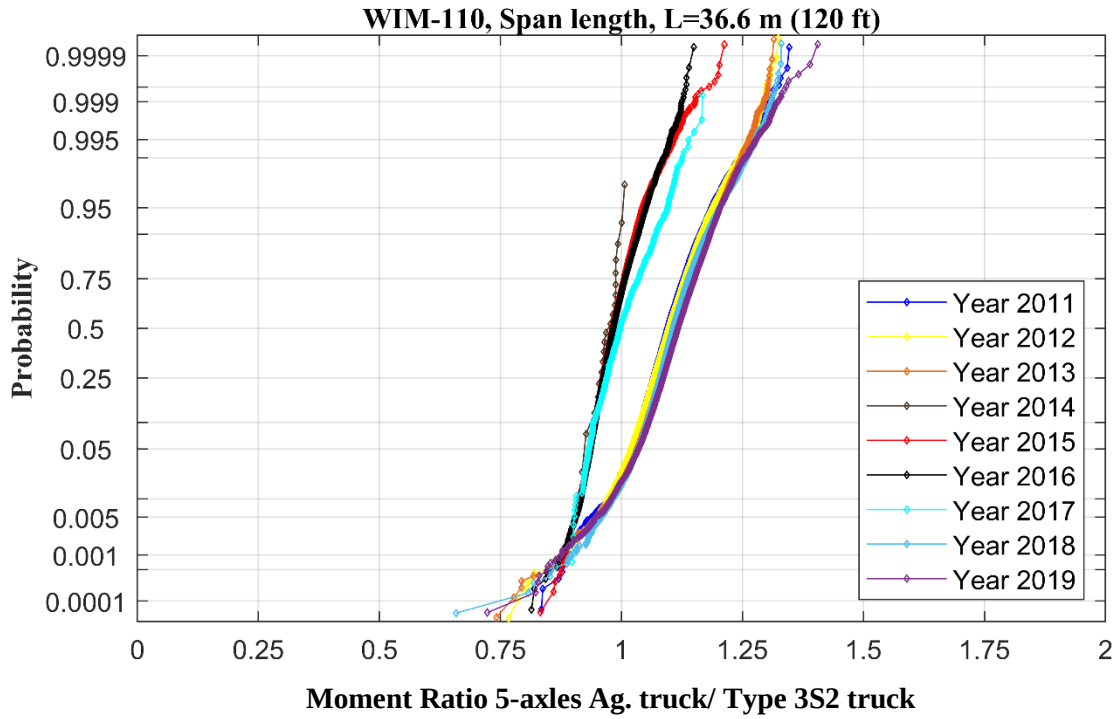
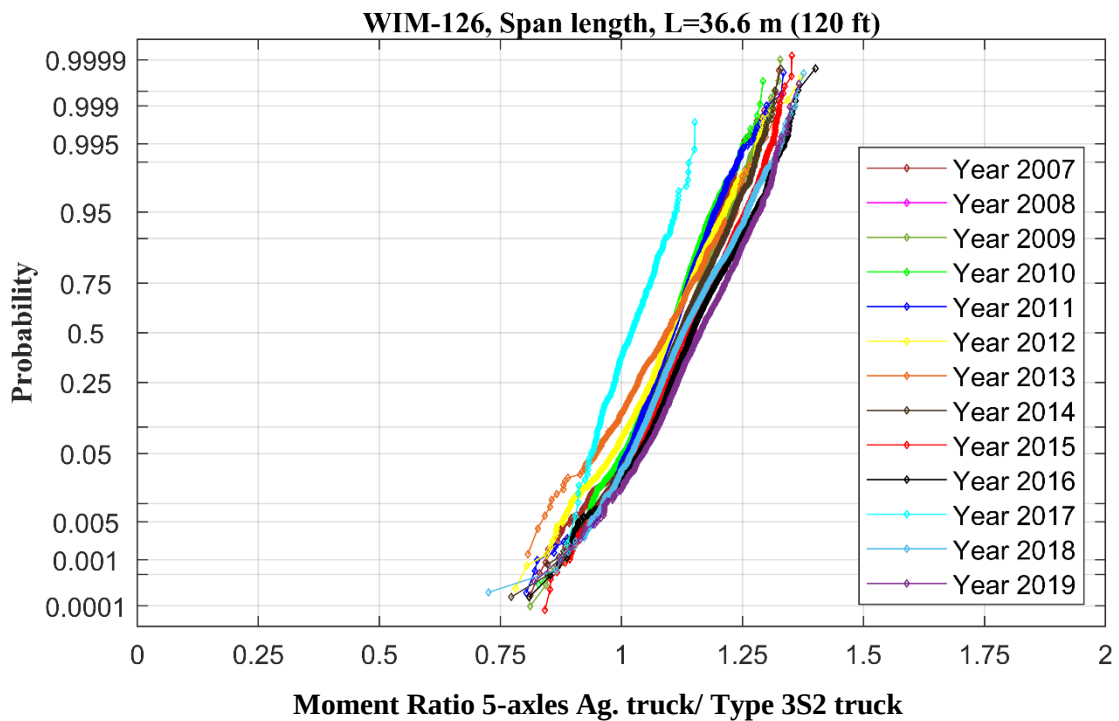
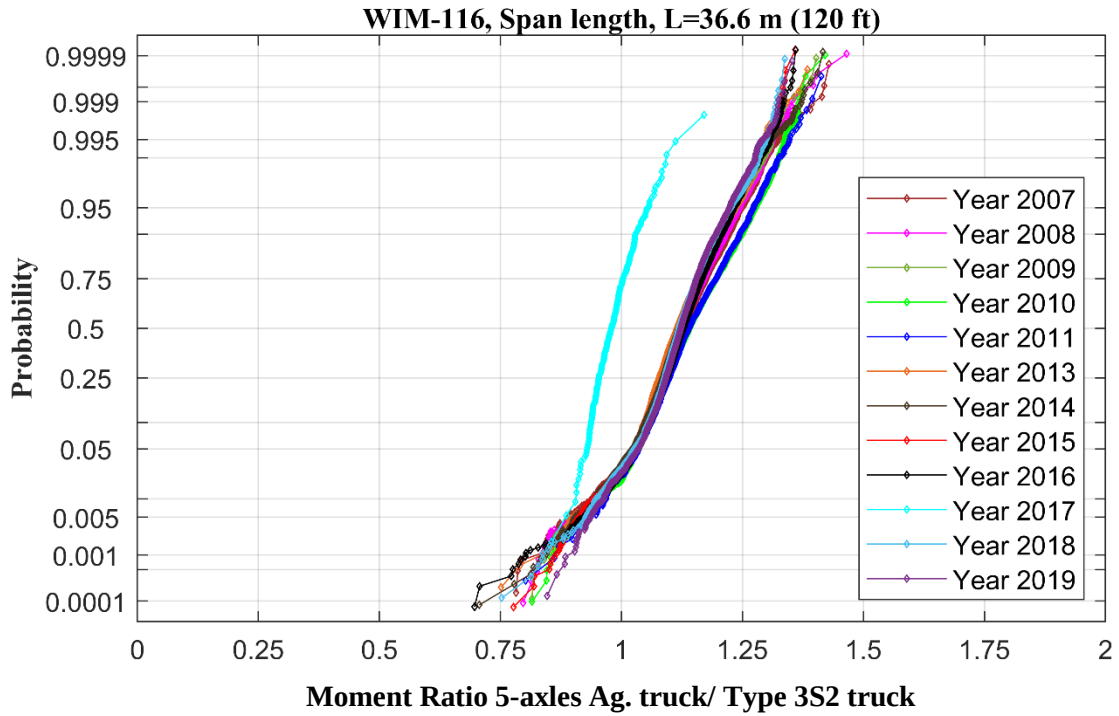


Figure 4-G.3. CDF of moment ratio for 5-axles agricultural vehicles to Type 3S2 vehicle for span length of 27.4 m (90 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





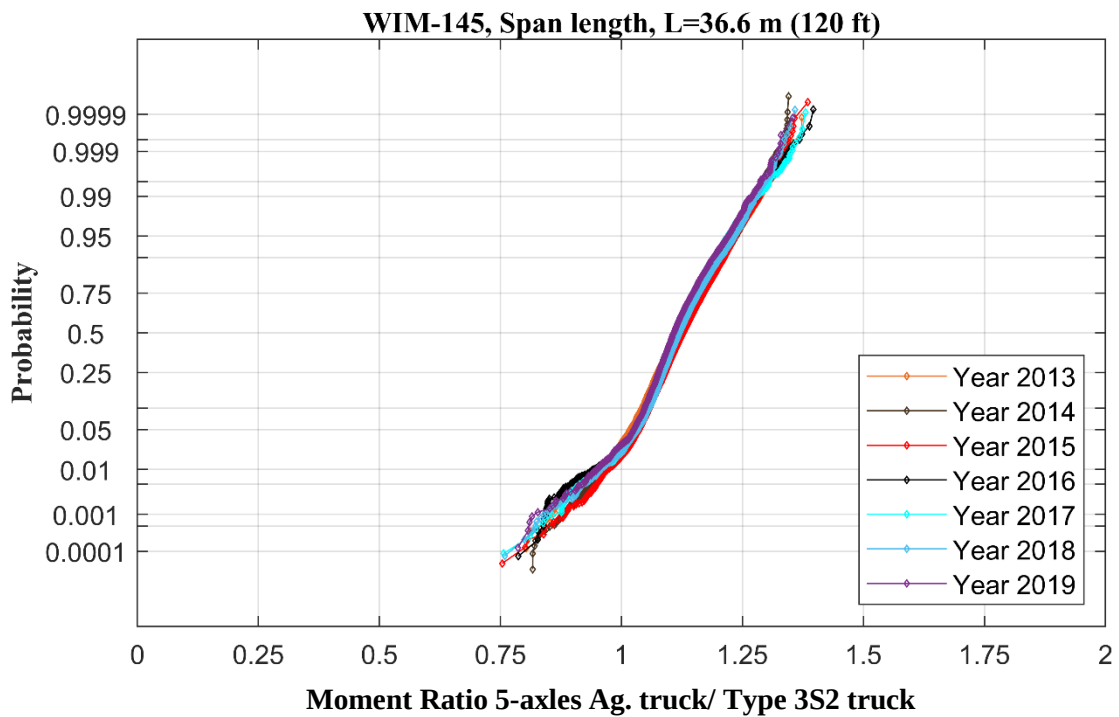
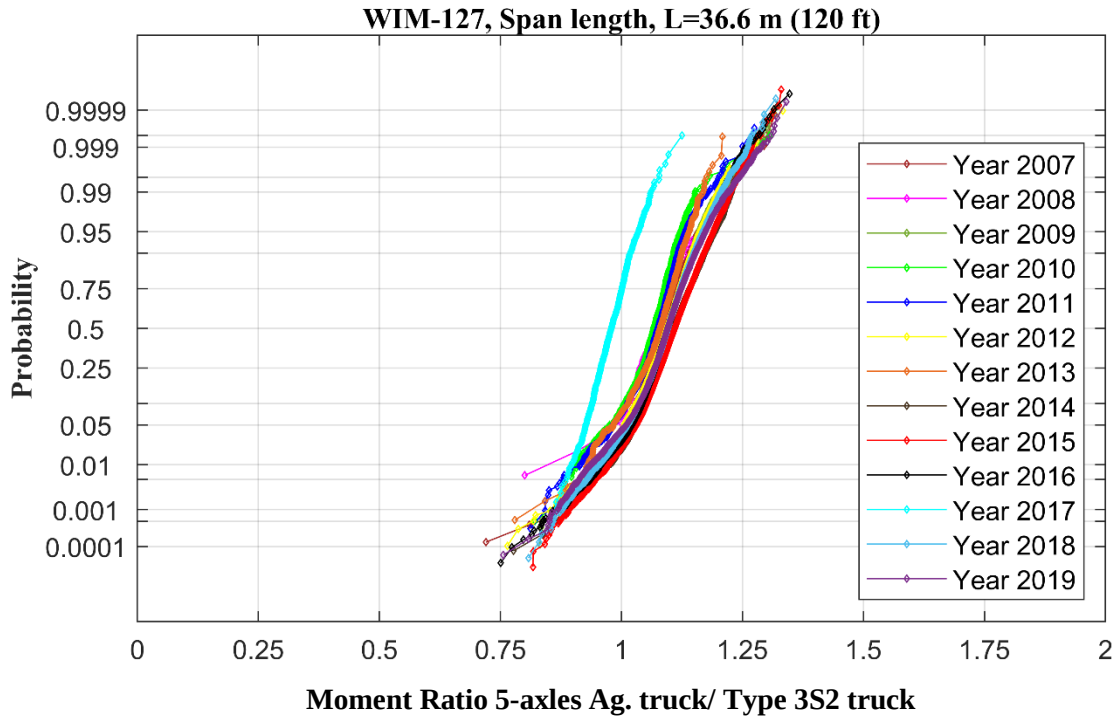
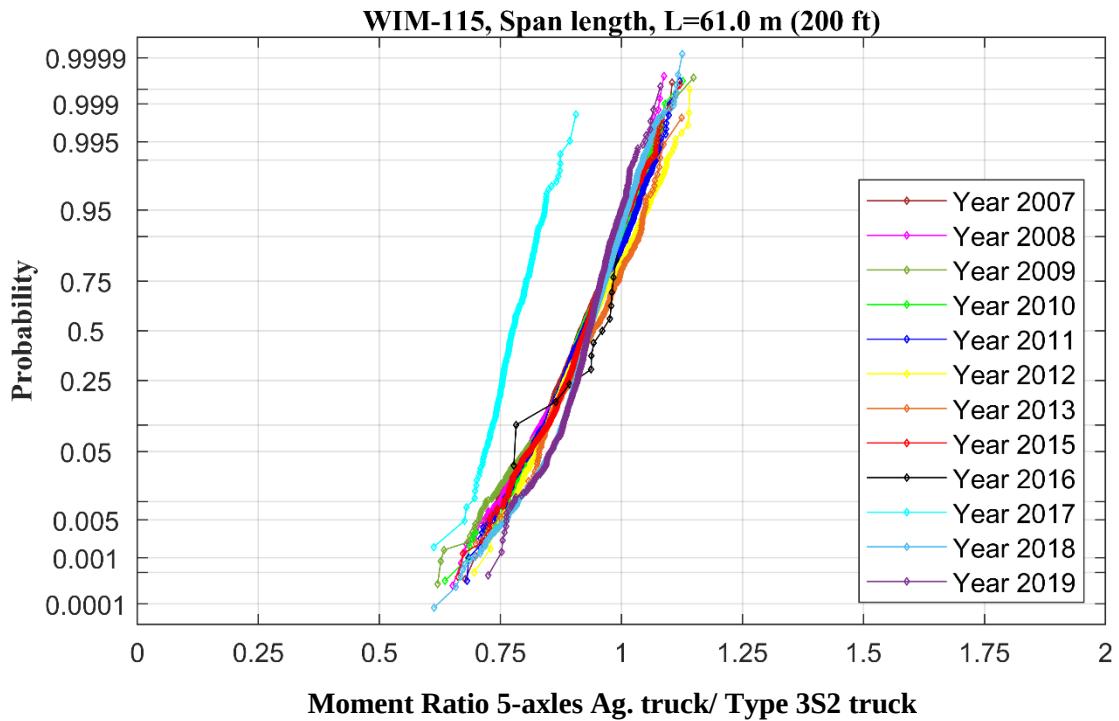
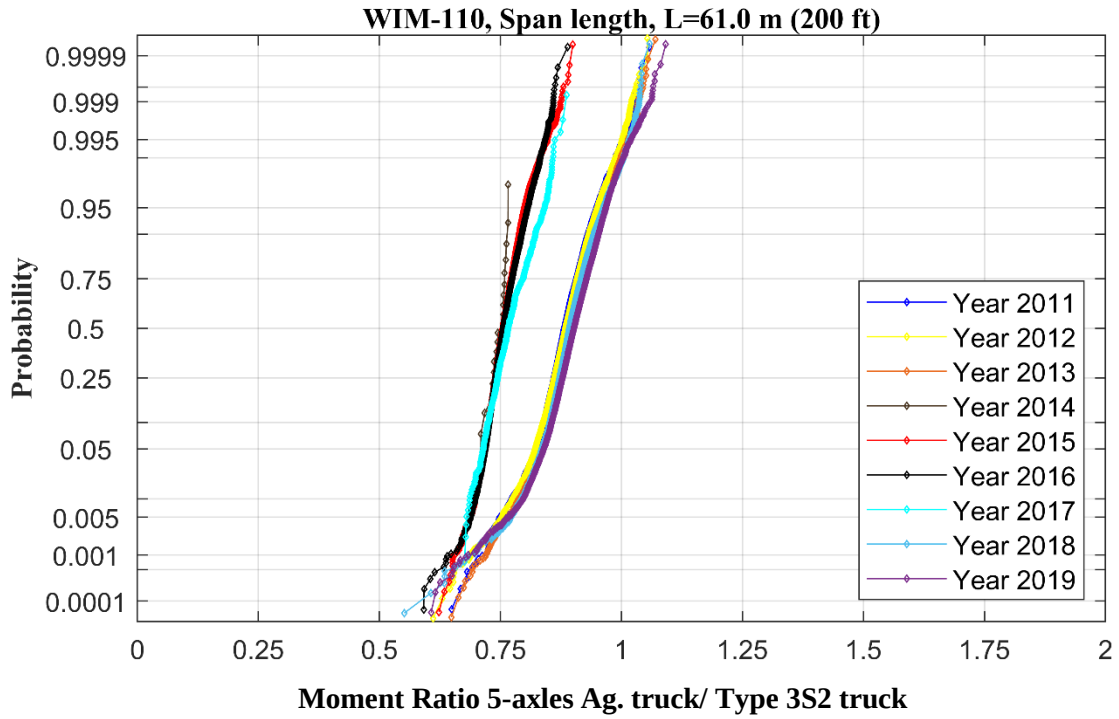
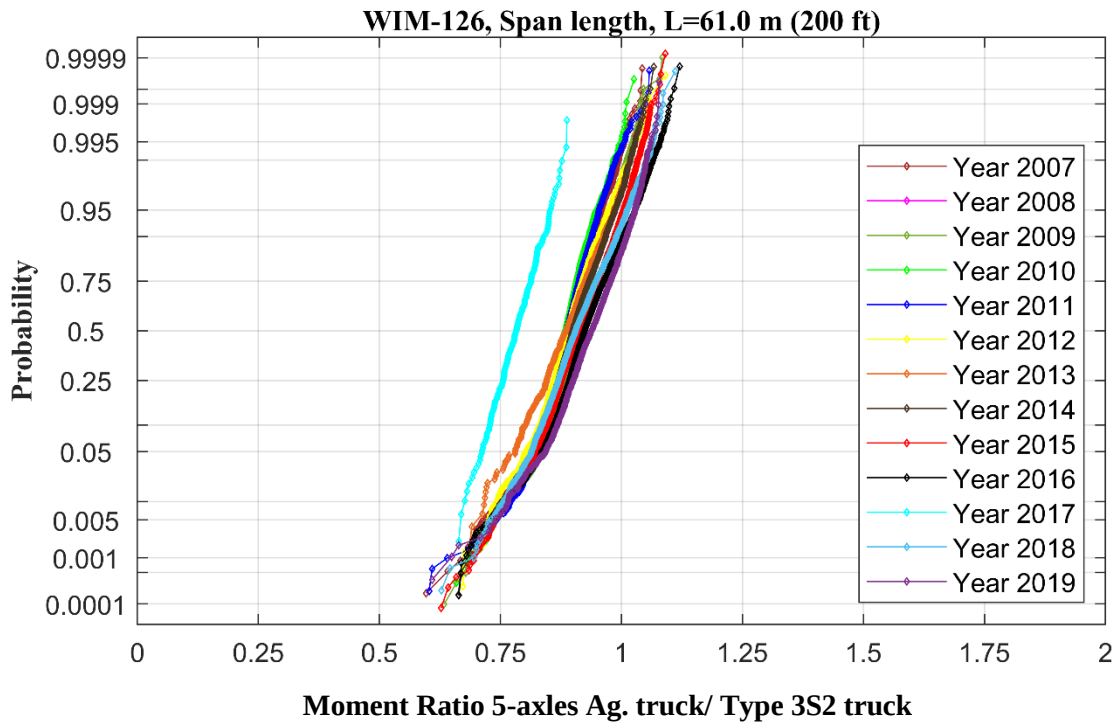
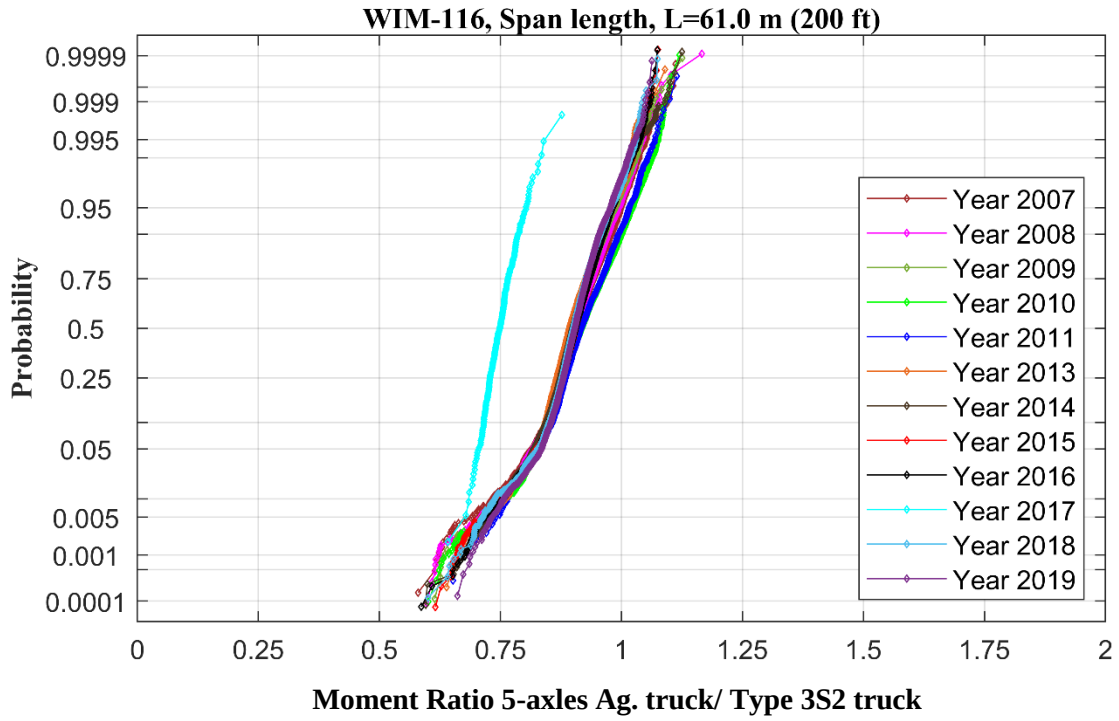


Figure 4-G.4. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 36.6 m (120 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





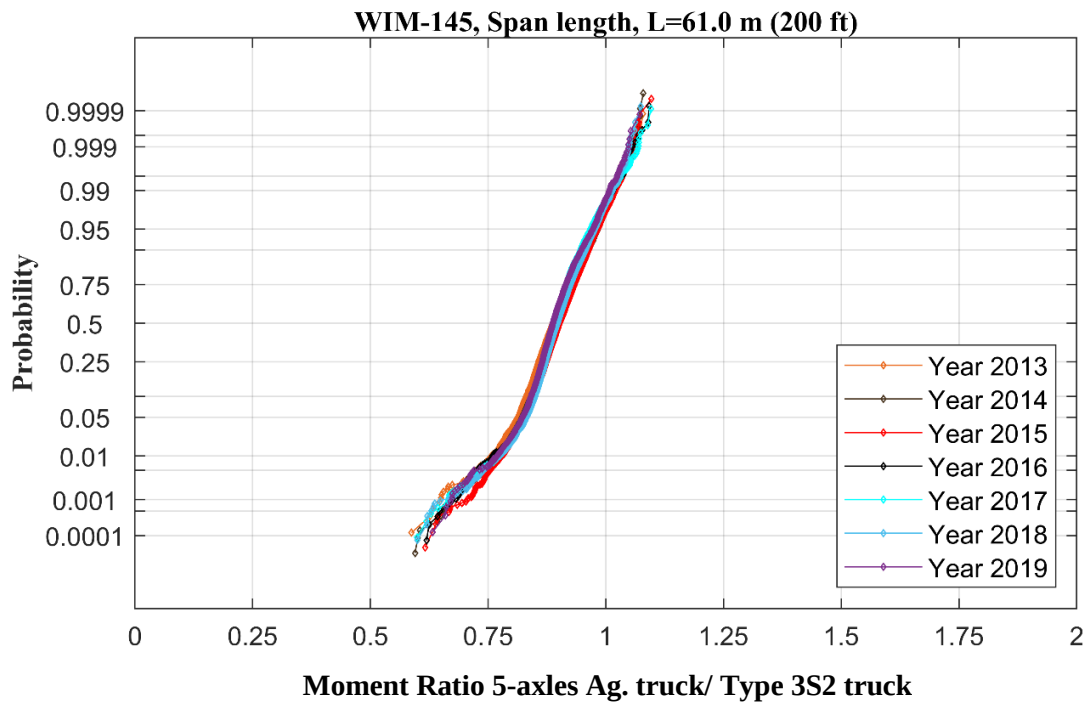
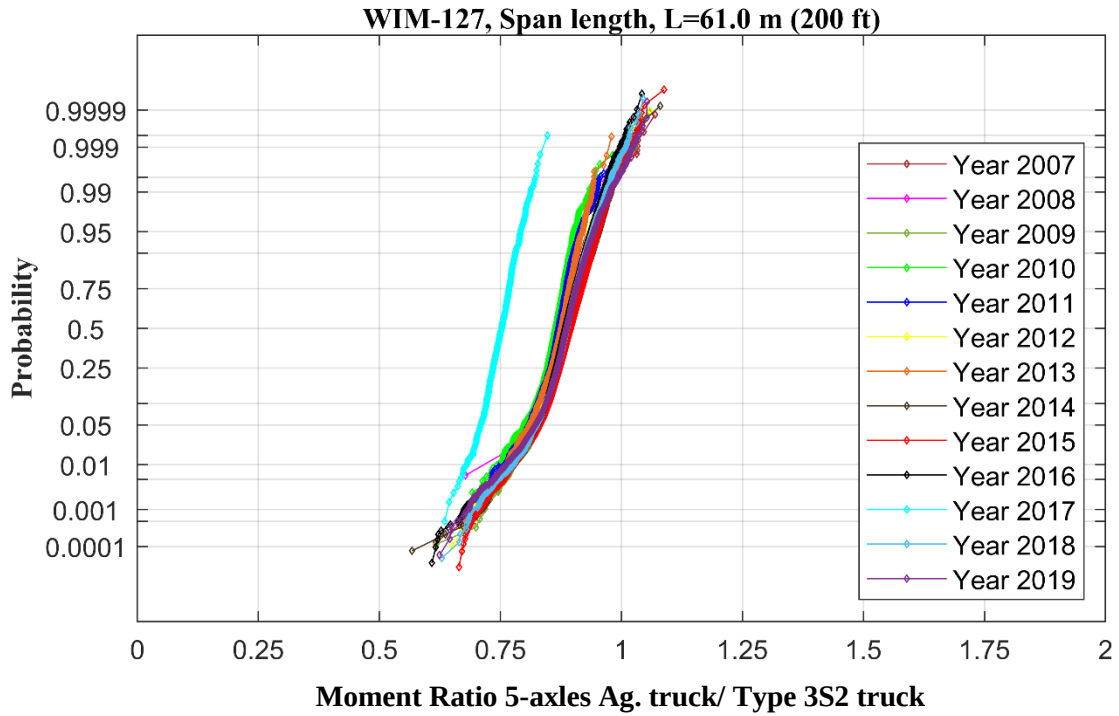
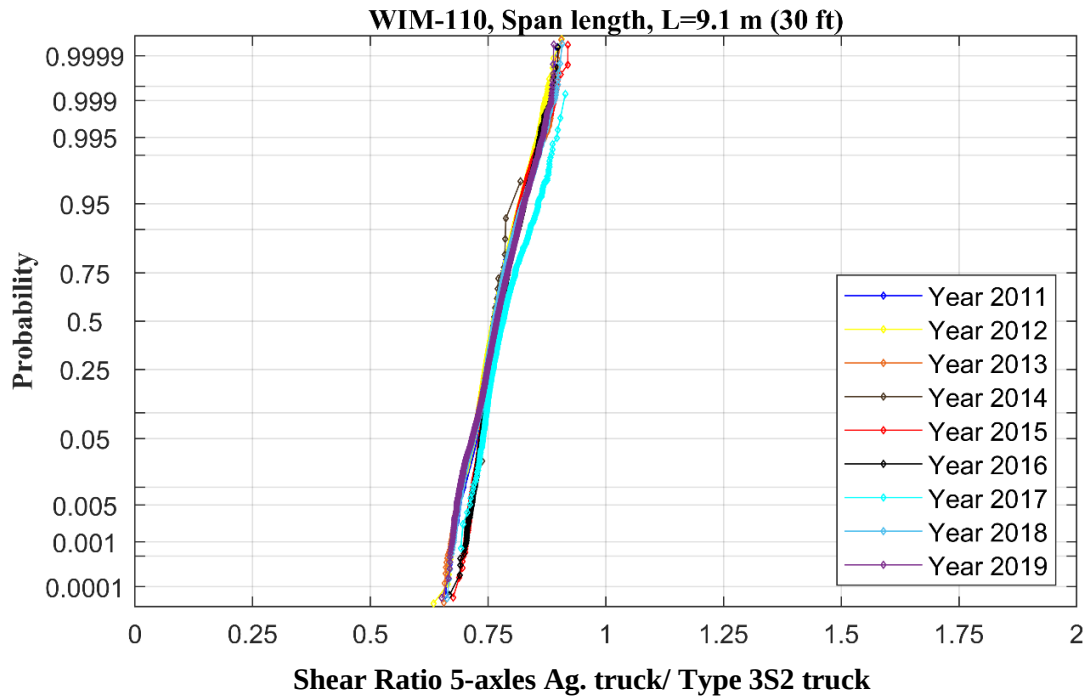
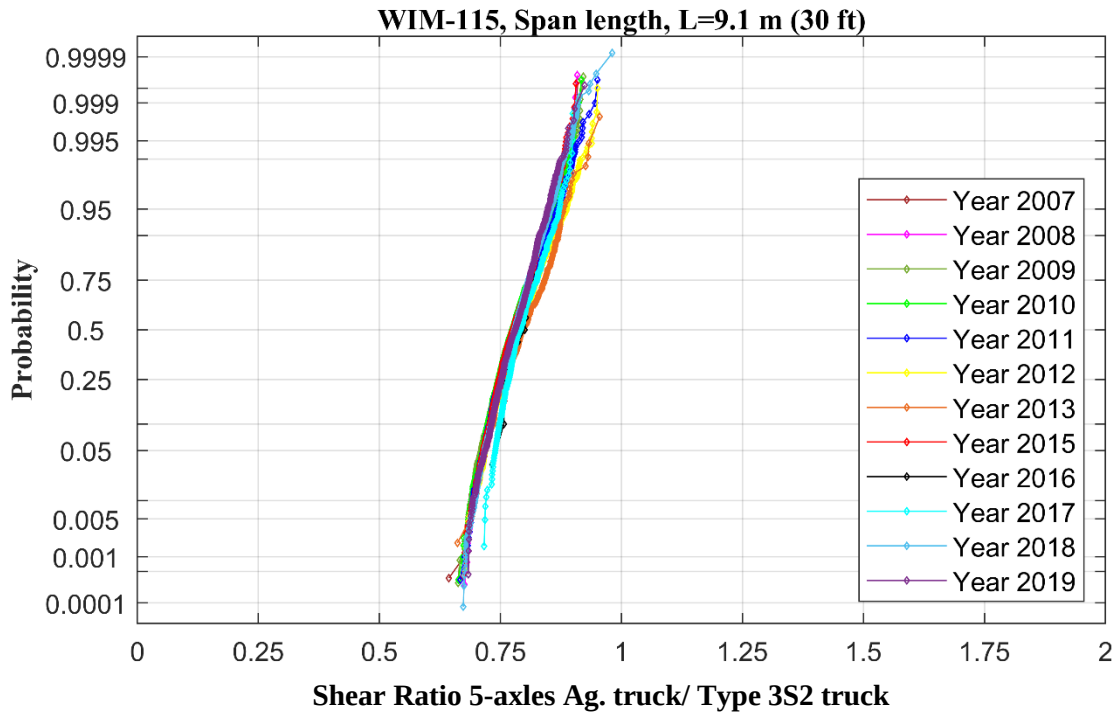


Figure 4-G.5. CDF of moment ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 61 m (200 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.

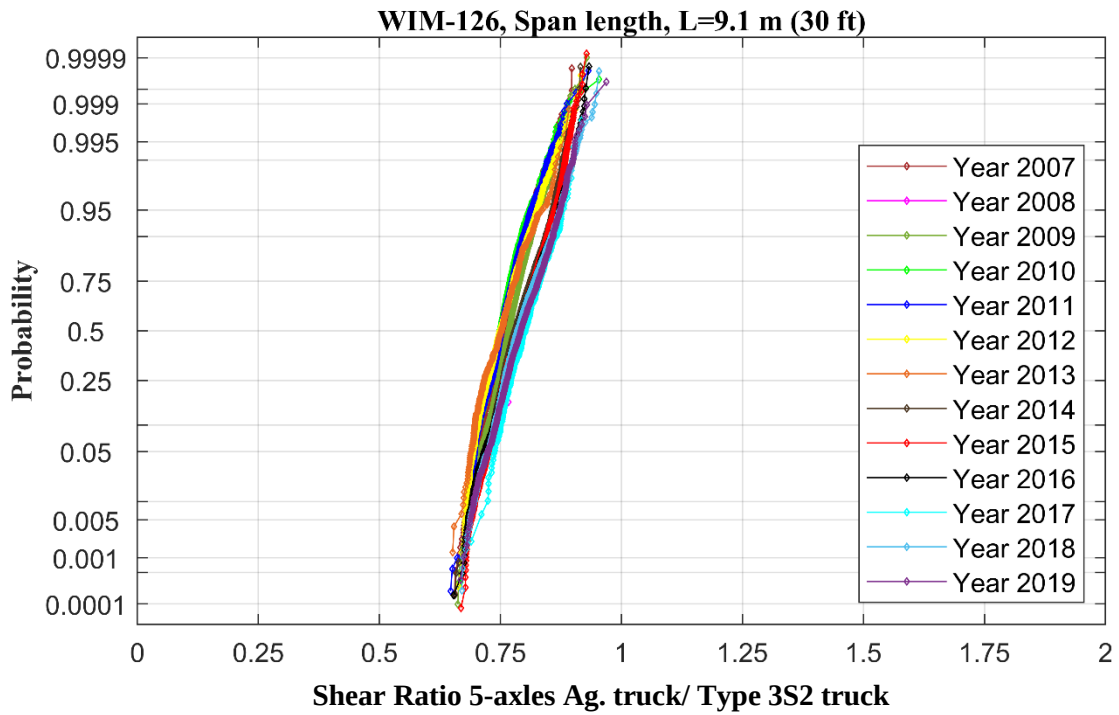
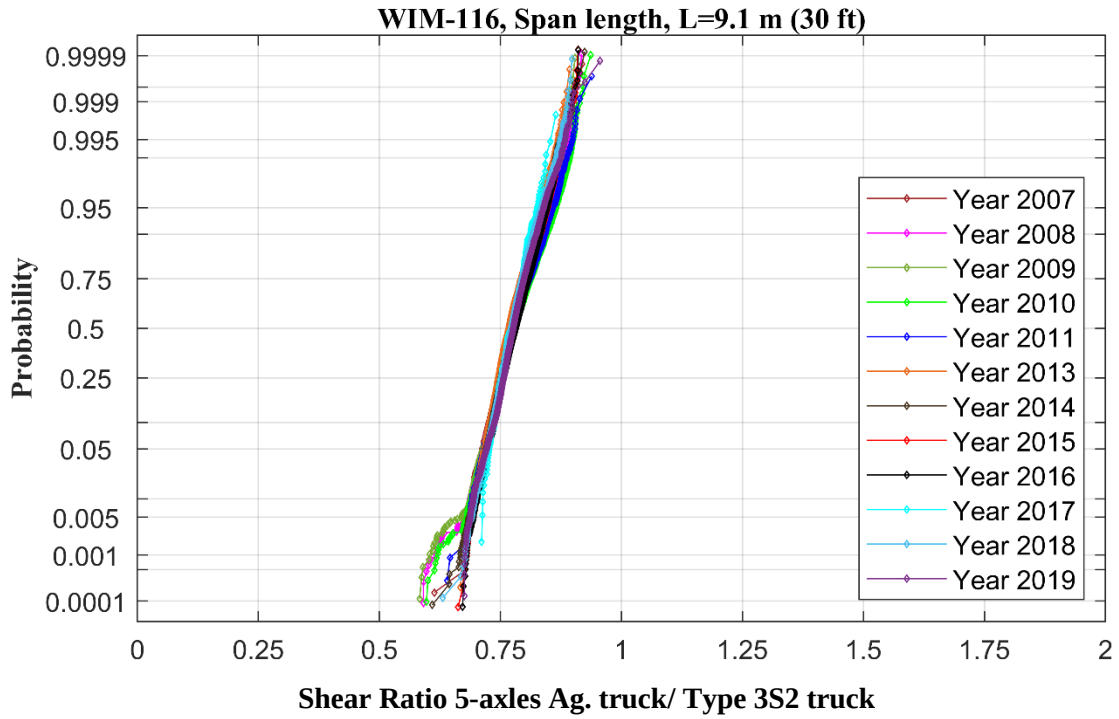
Appendix 4-H. CDF of shear ratio for 5-axle agricultural to Type 3S2 truck



(a)



(b)



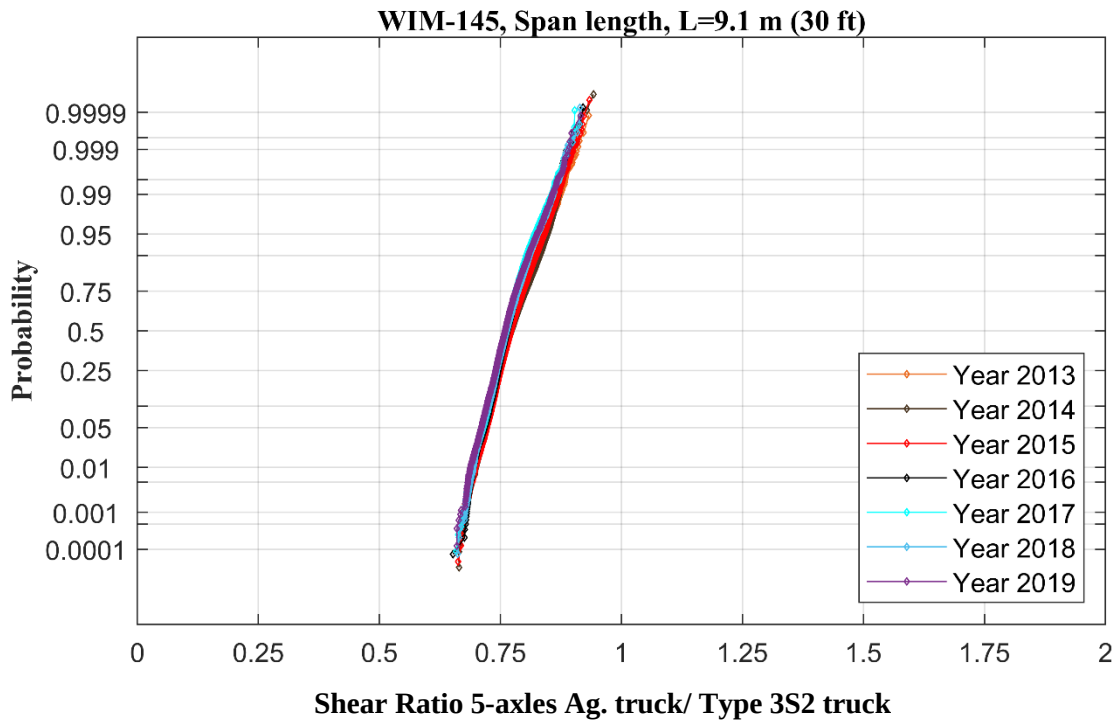
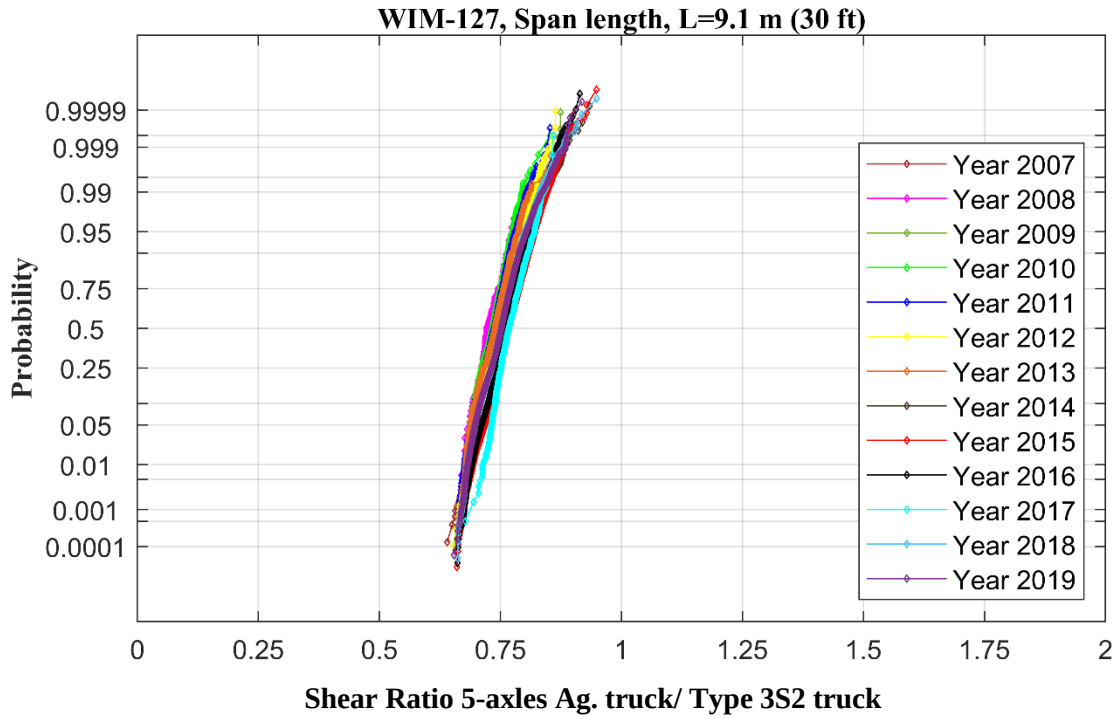
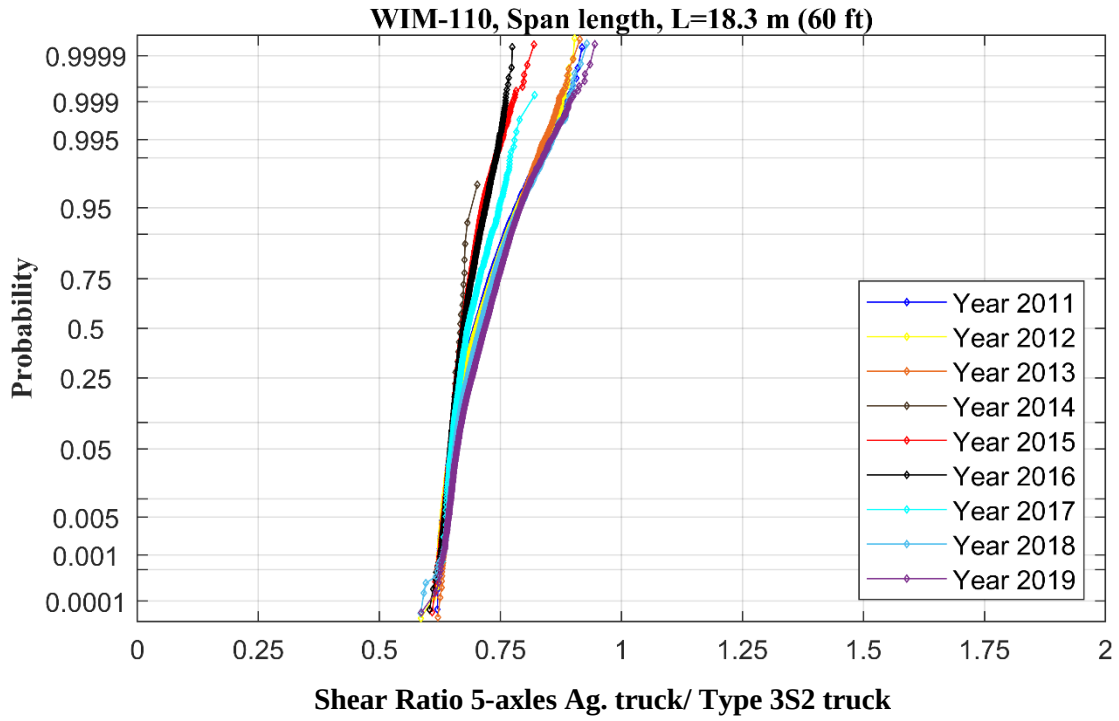
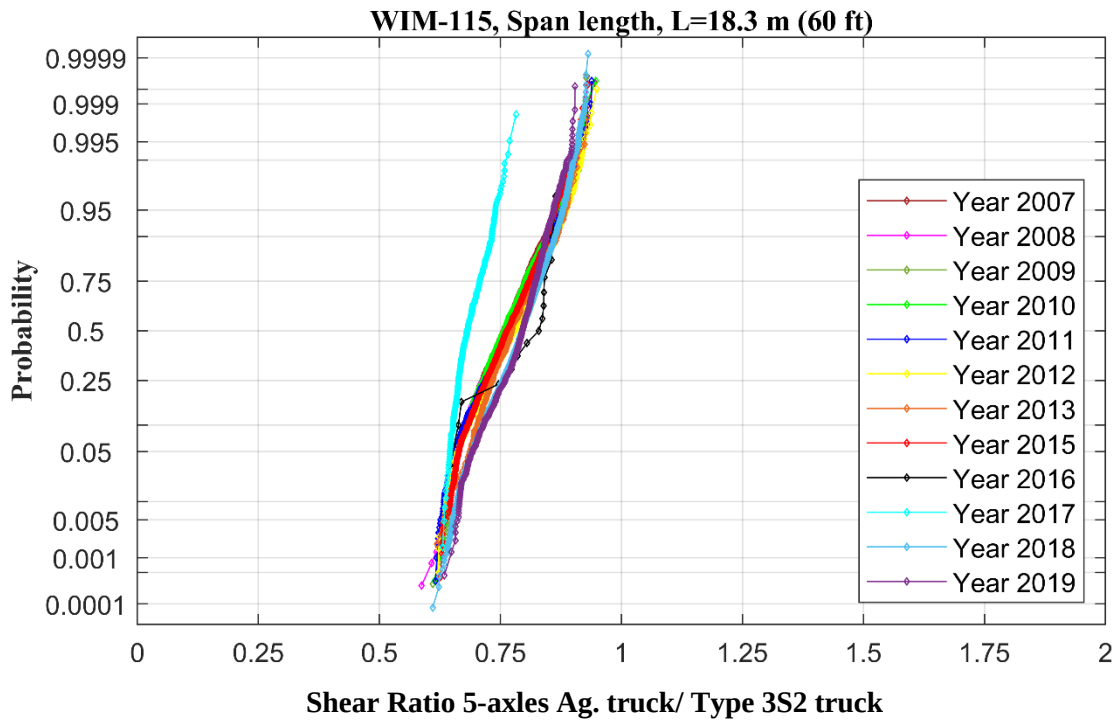


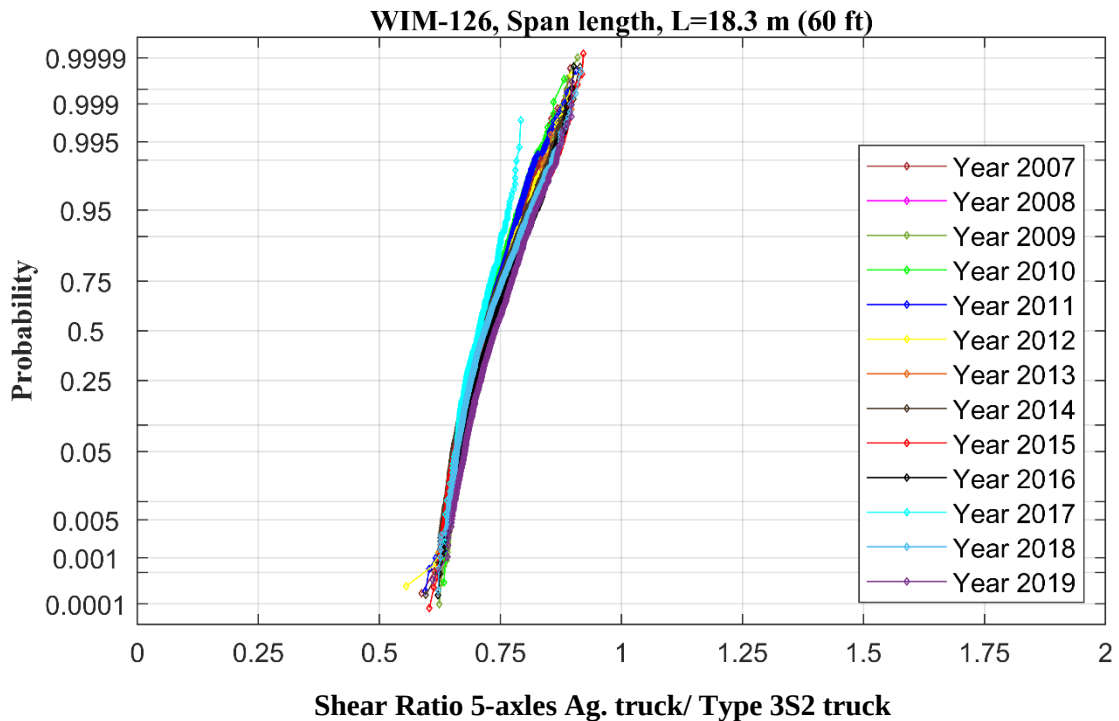
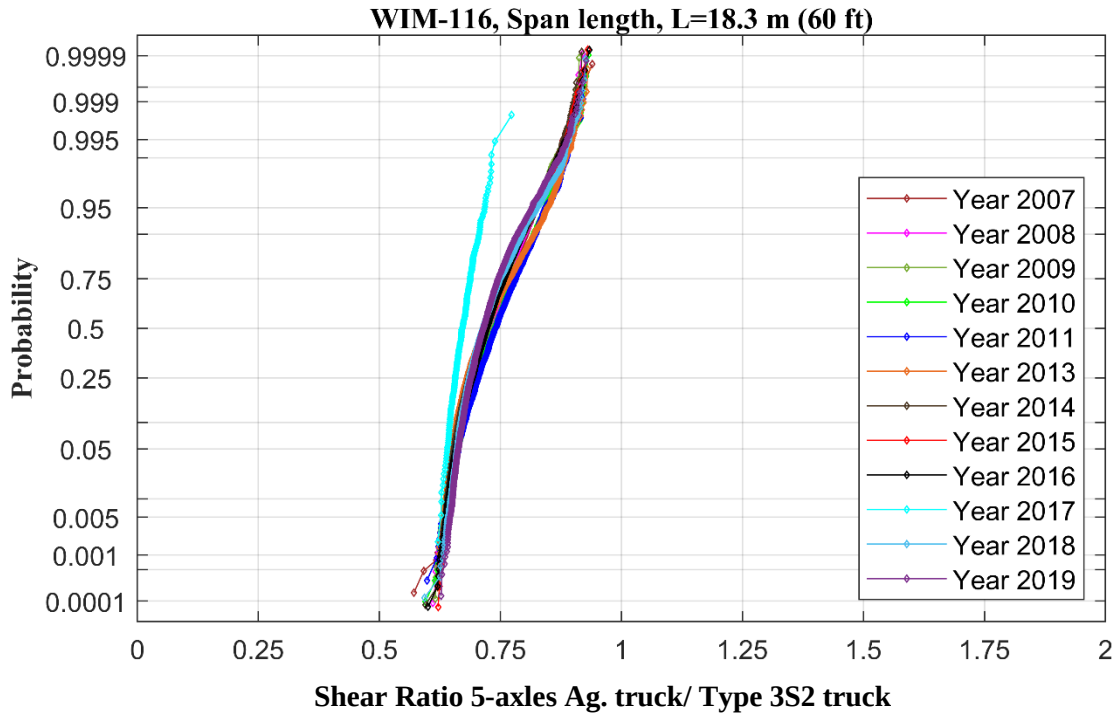
Figure 4-H.1. CDF of shear ratio for 5-axles agricultural vehicles to Type 3S2 vehicle for span length of 9.1 m (30 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



(a)



(b)



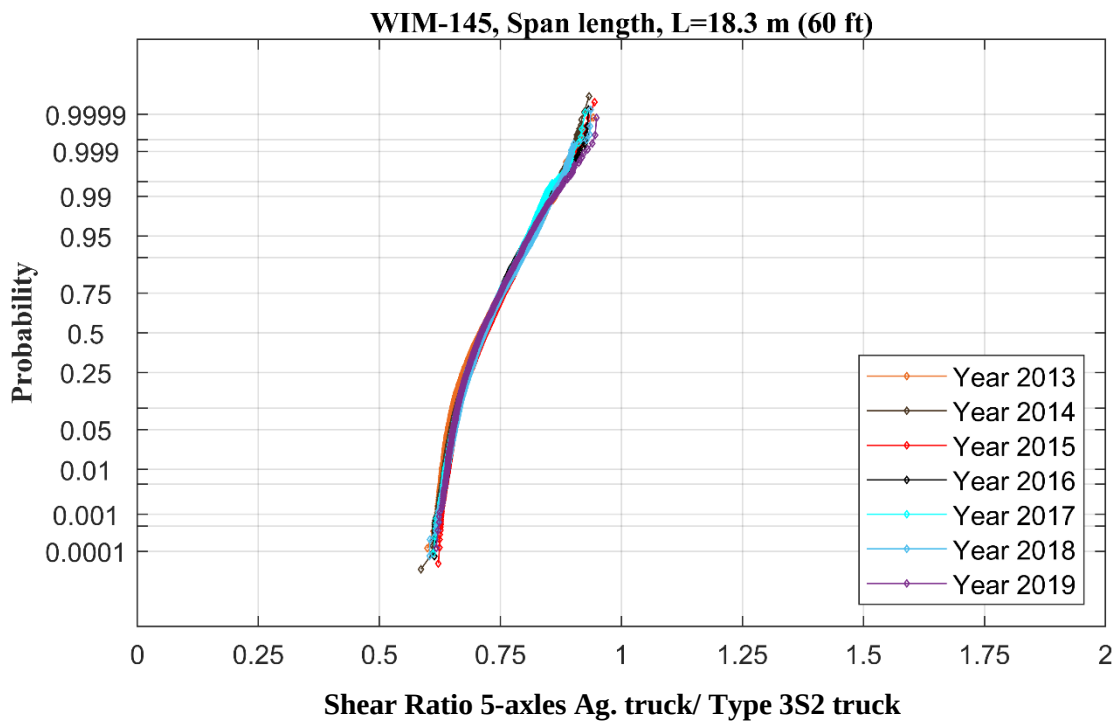
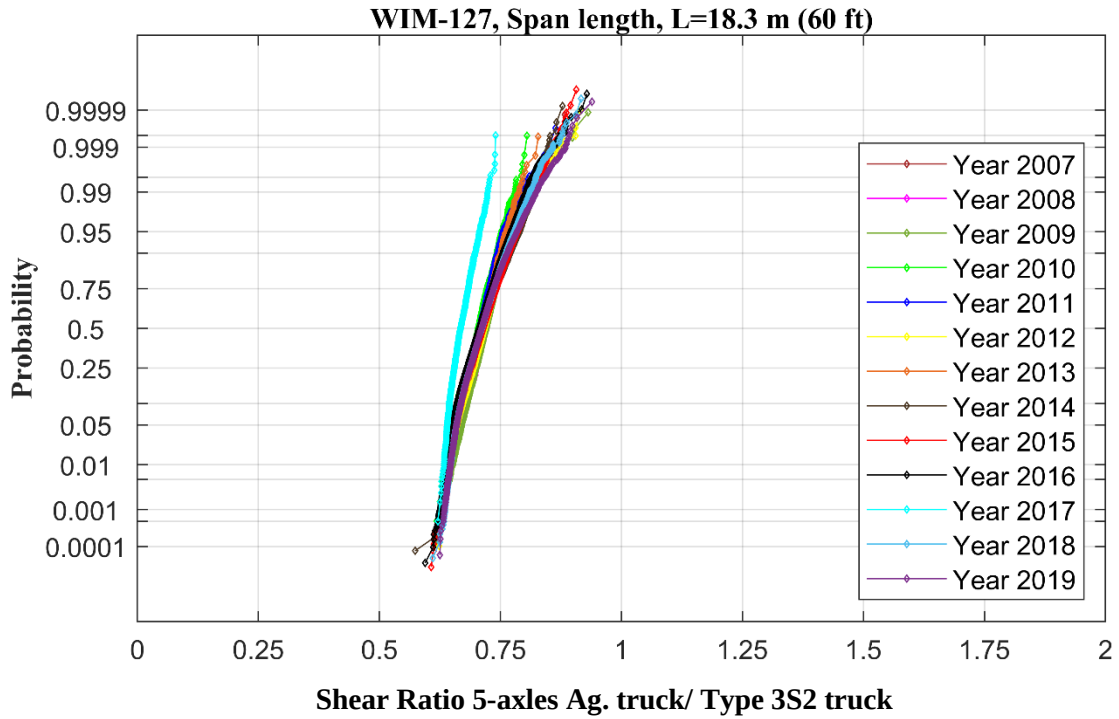
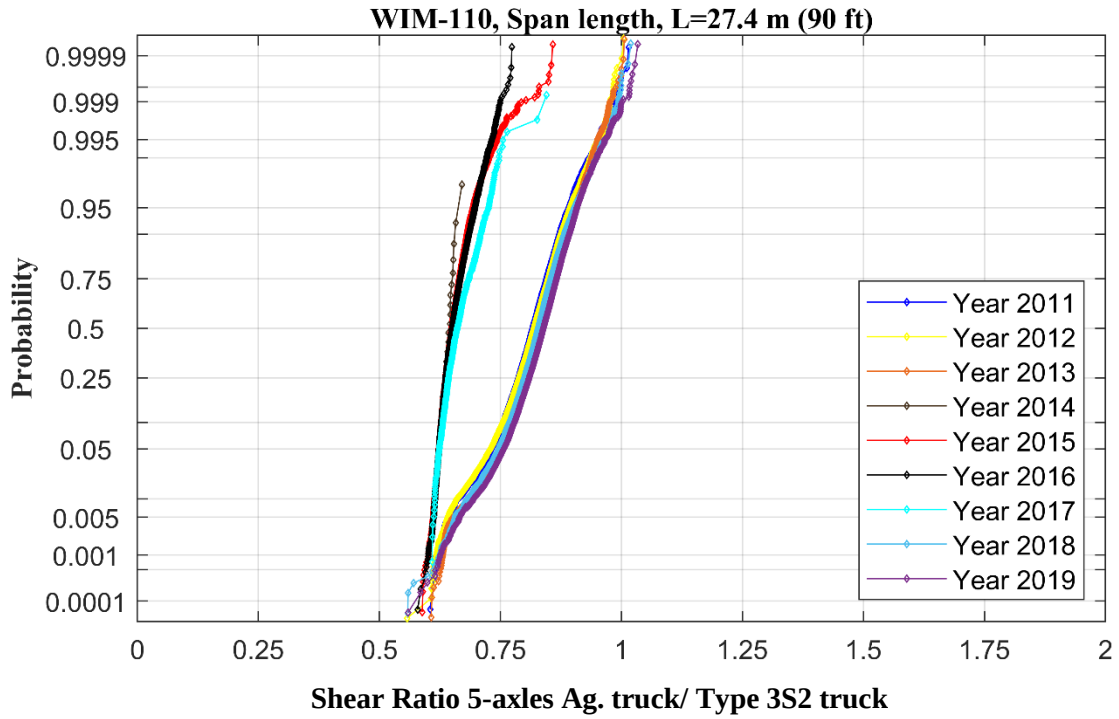
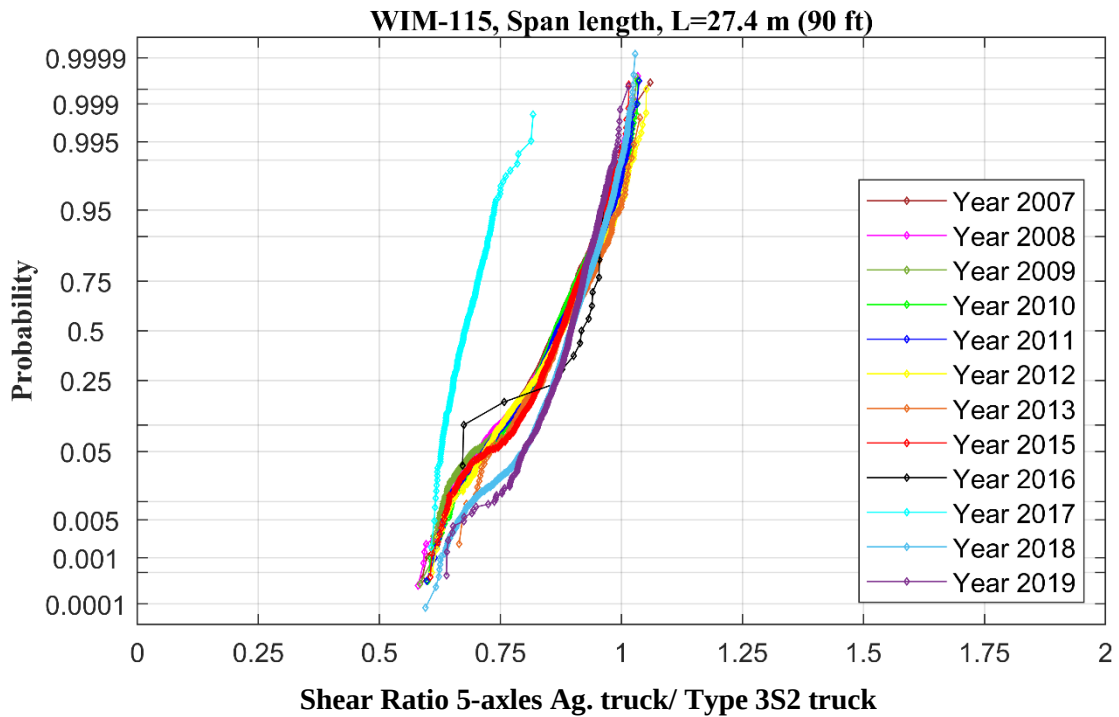


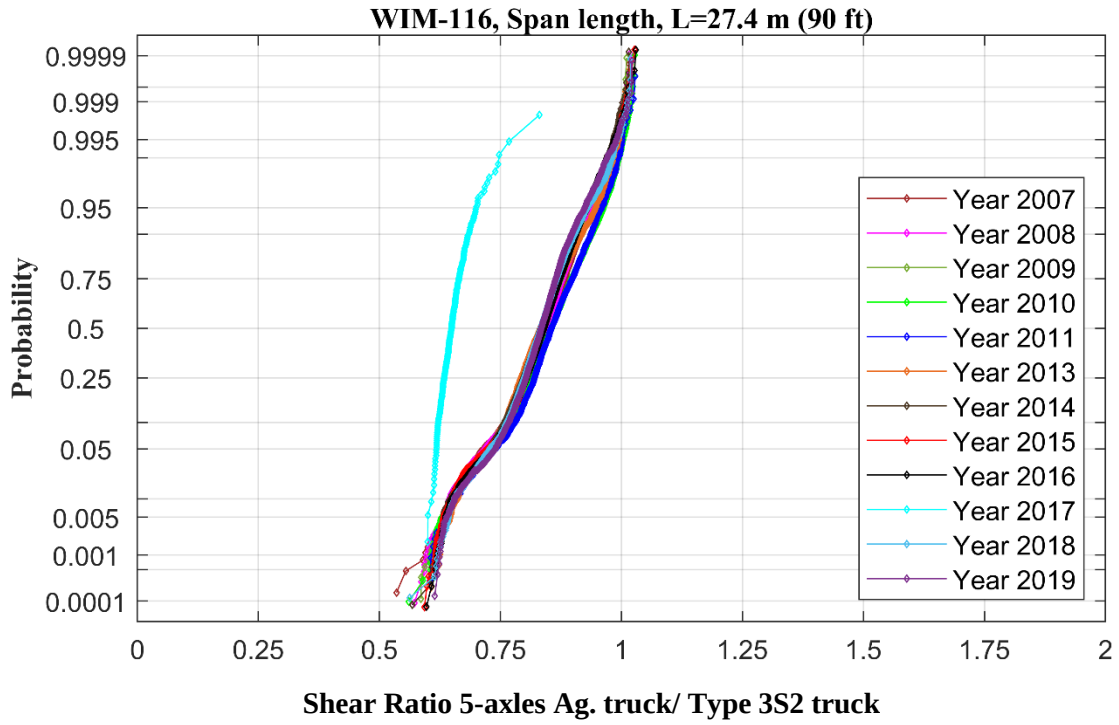
Figure 4-H.2. CDF of shear ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 18.3 m (60 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.



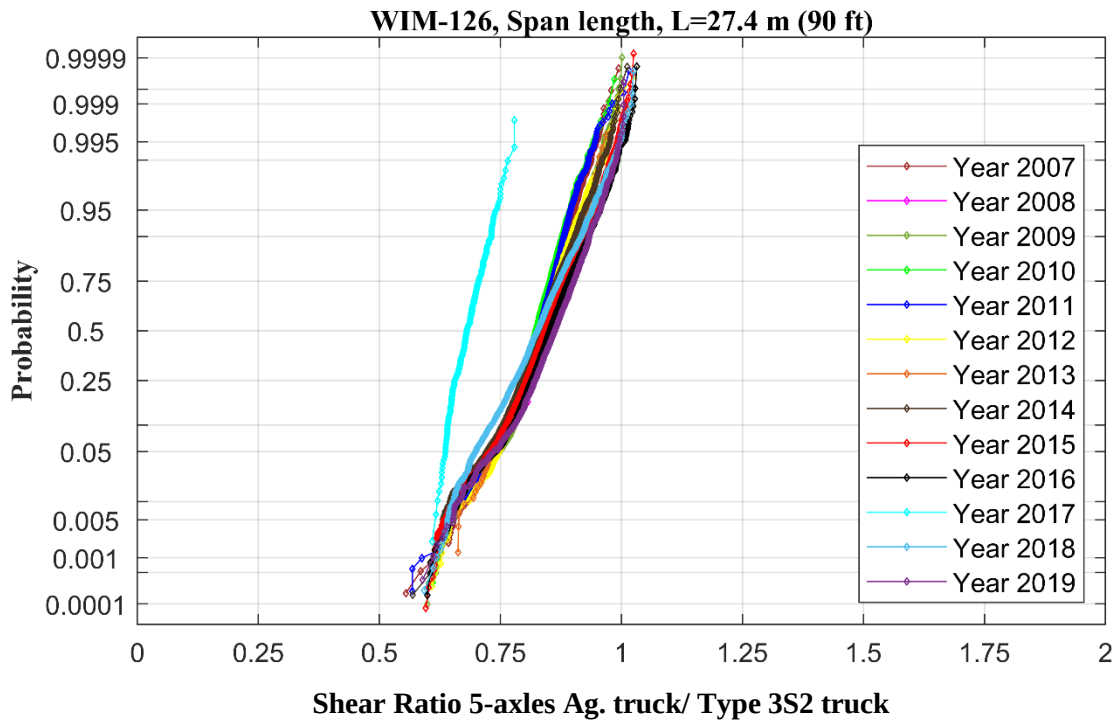
(a)



(b)



(c)



(d)

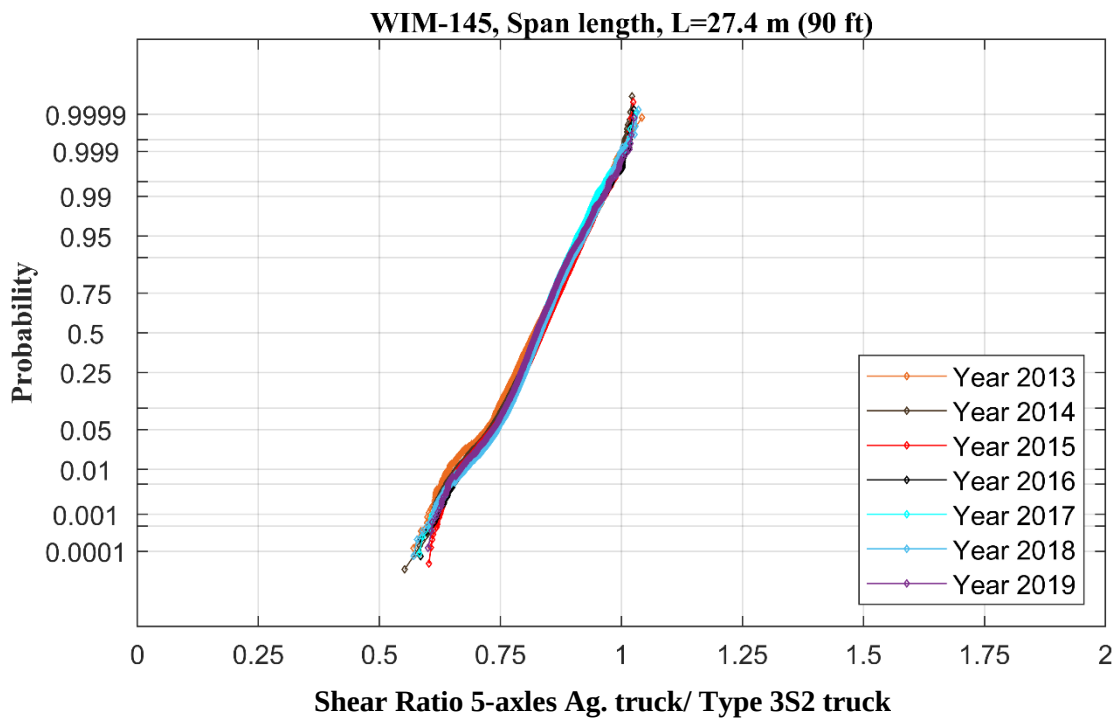
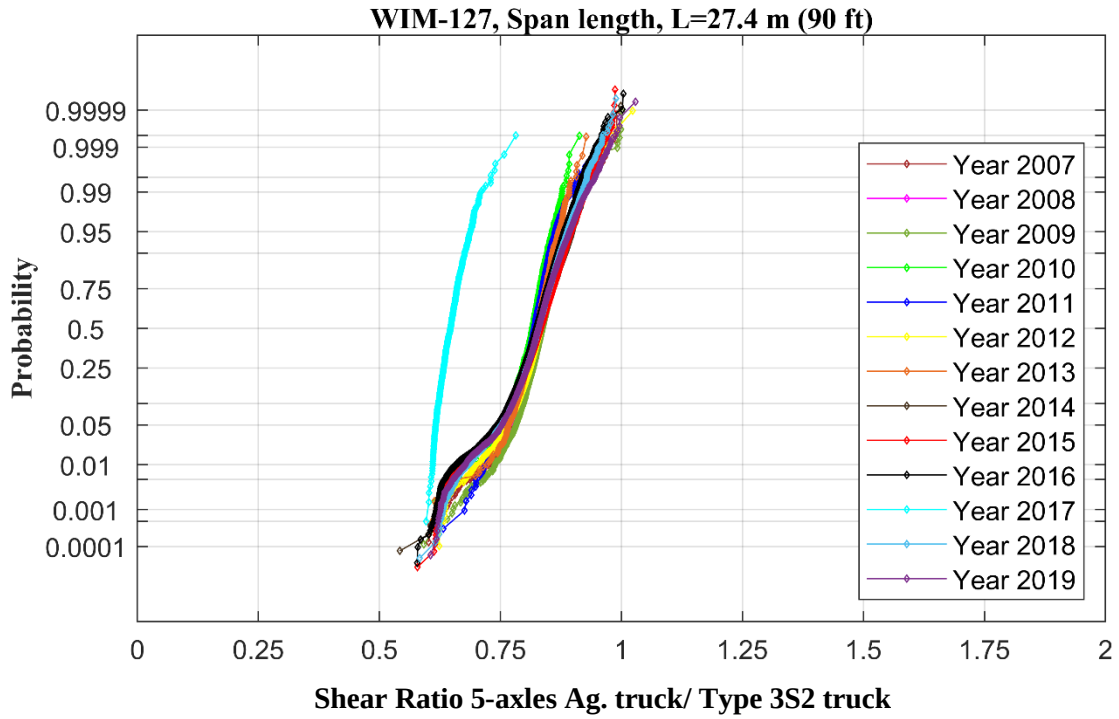
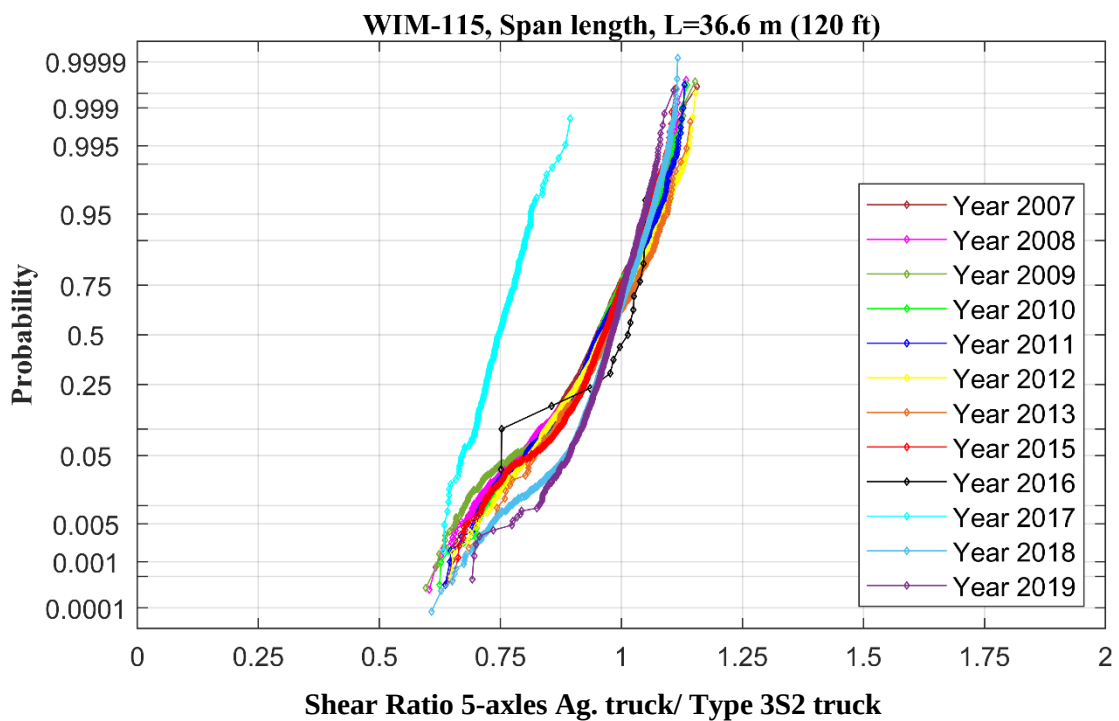
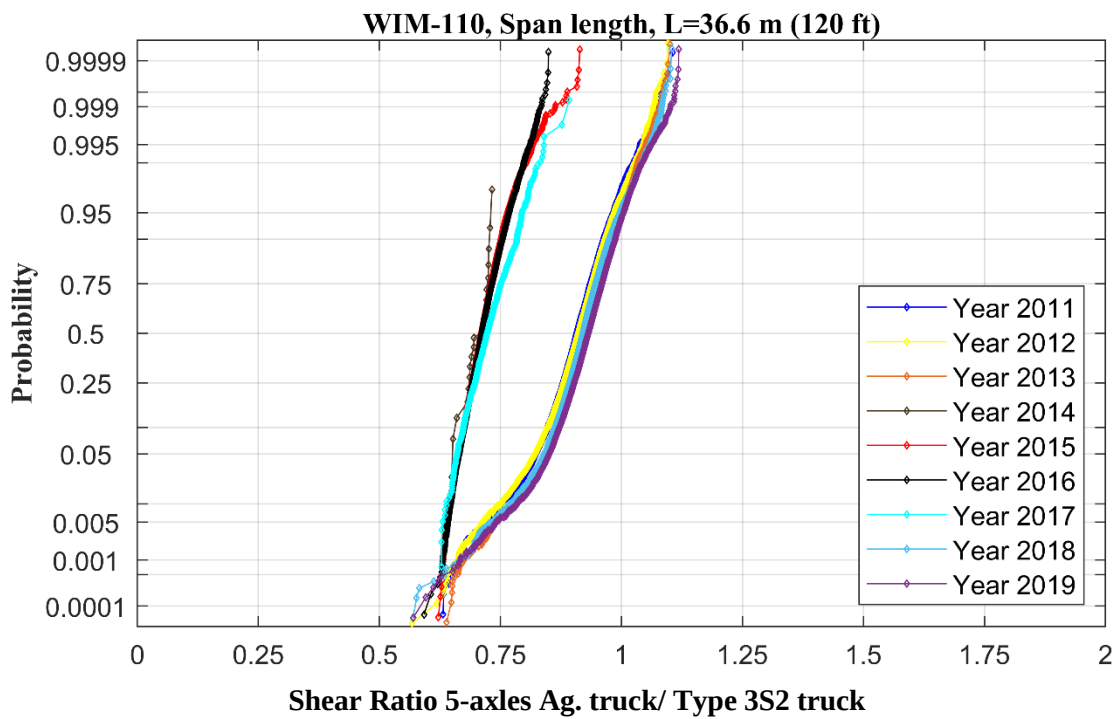
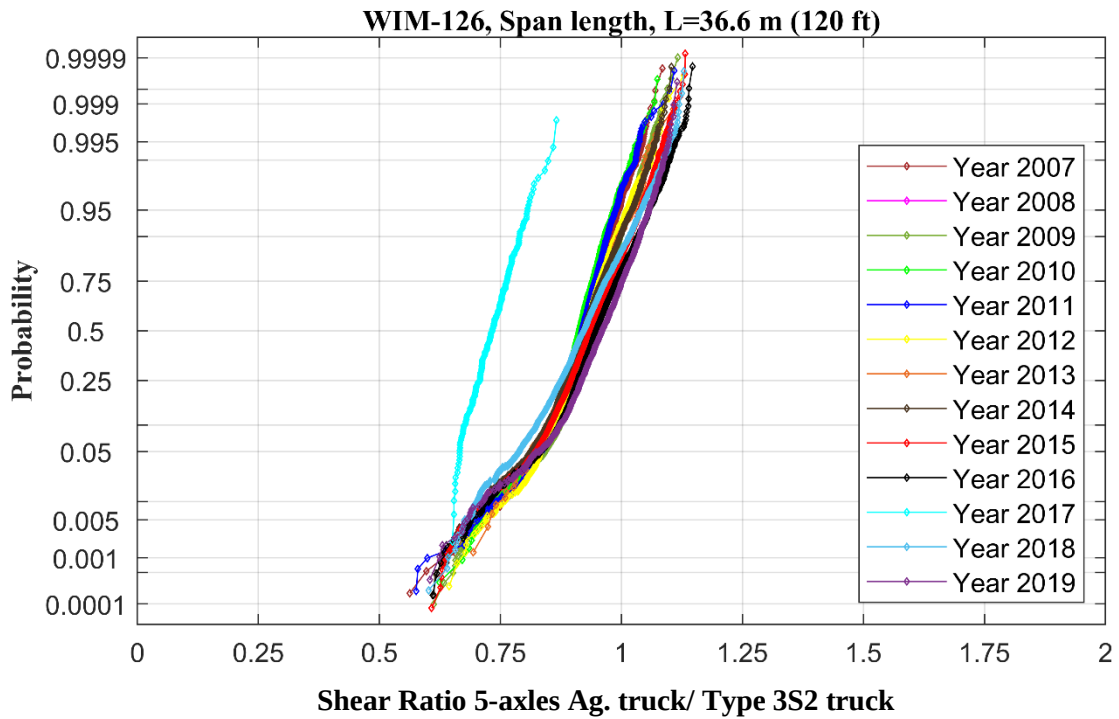
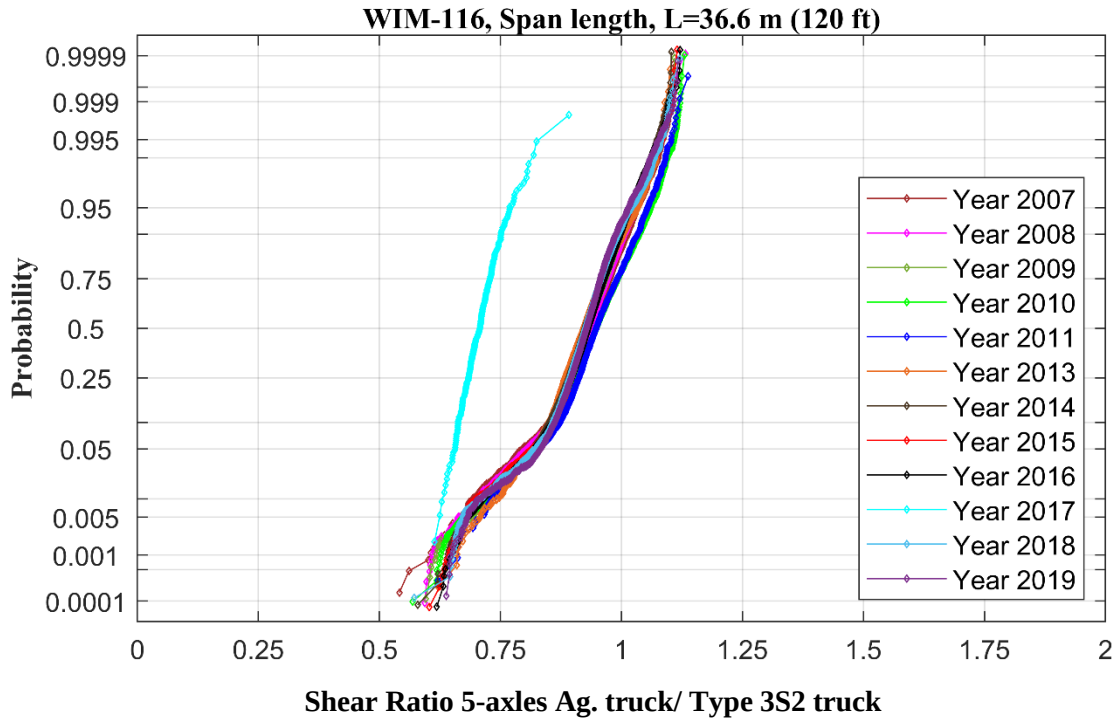


Figure 4-H.3. CDF of shear ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 27.4 m (90 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





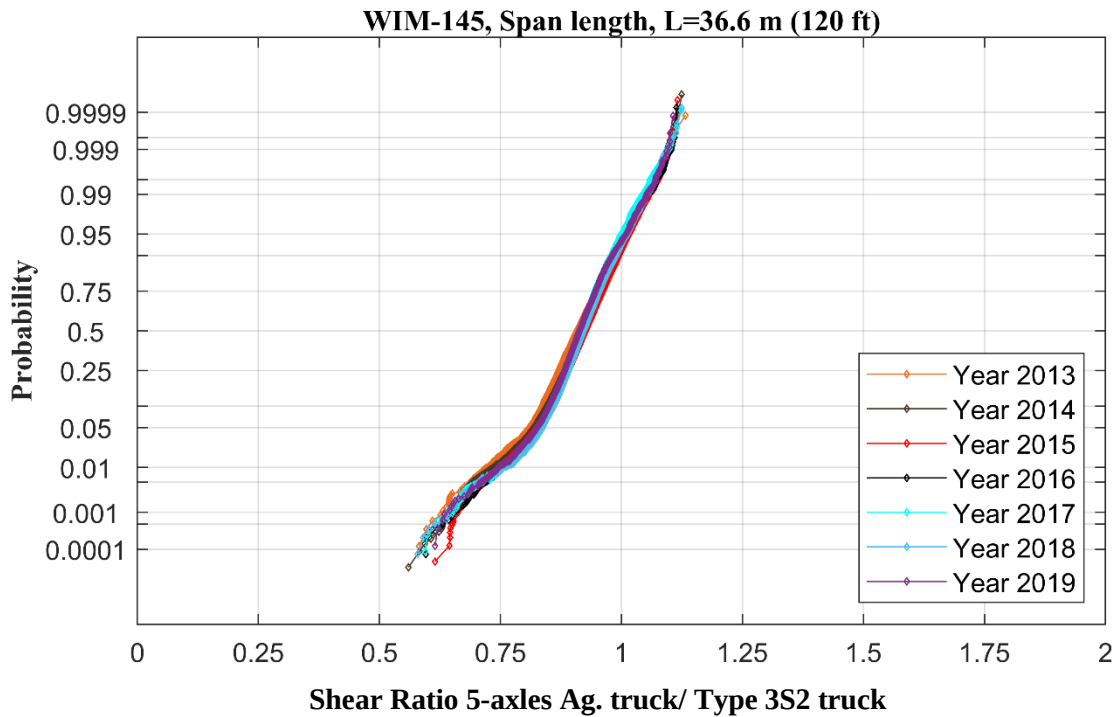
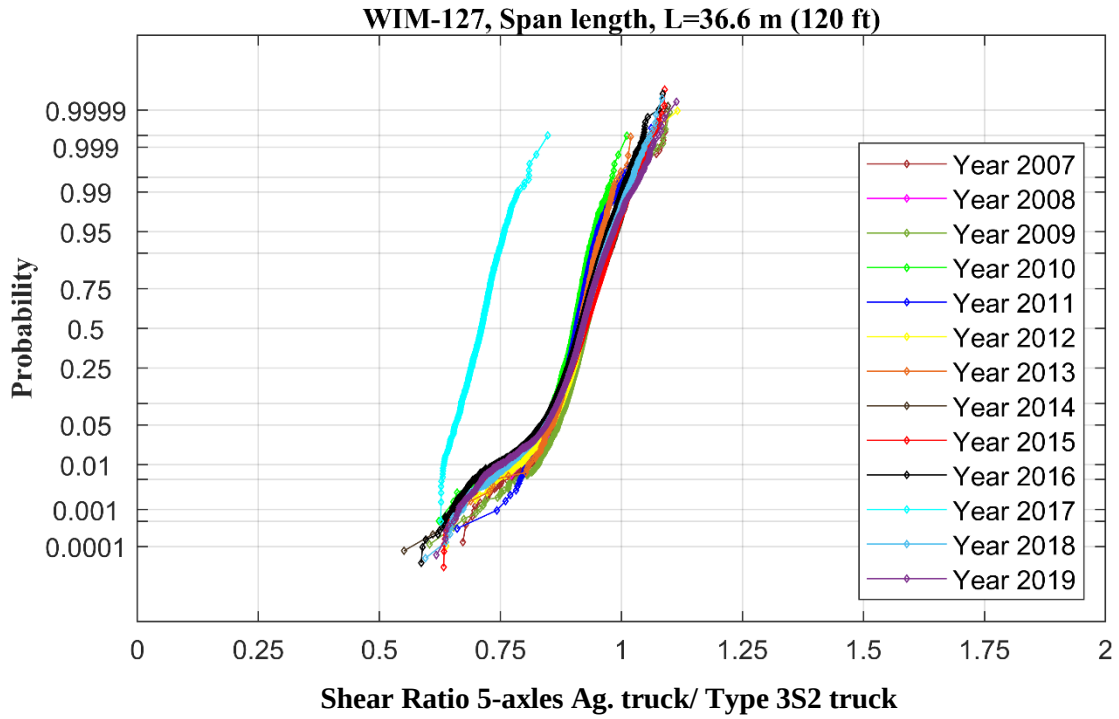
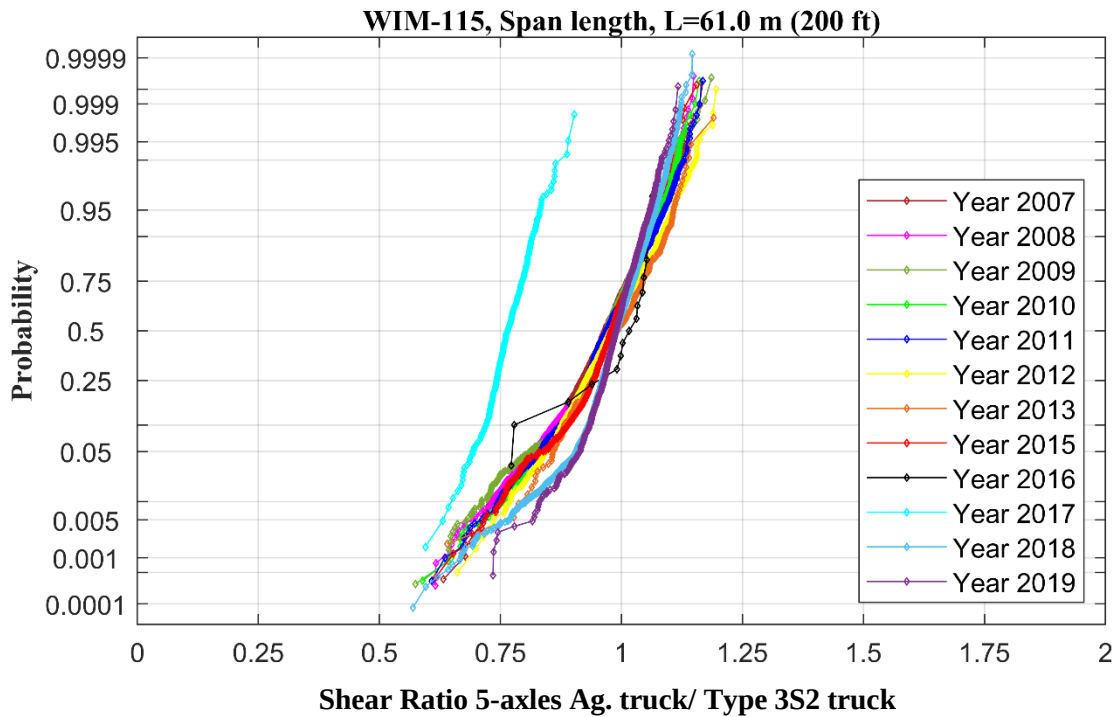
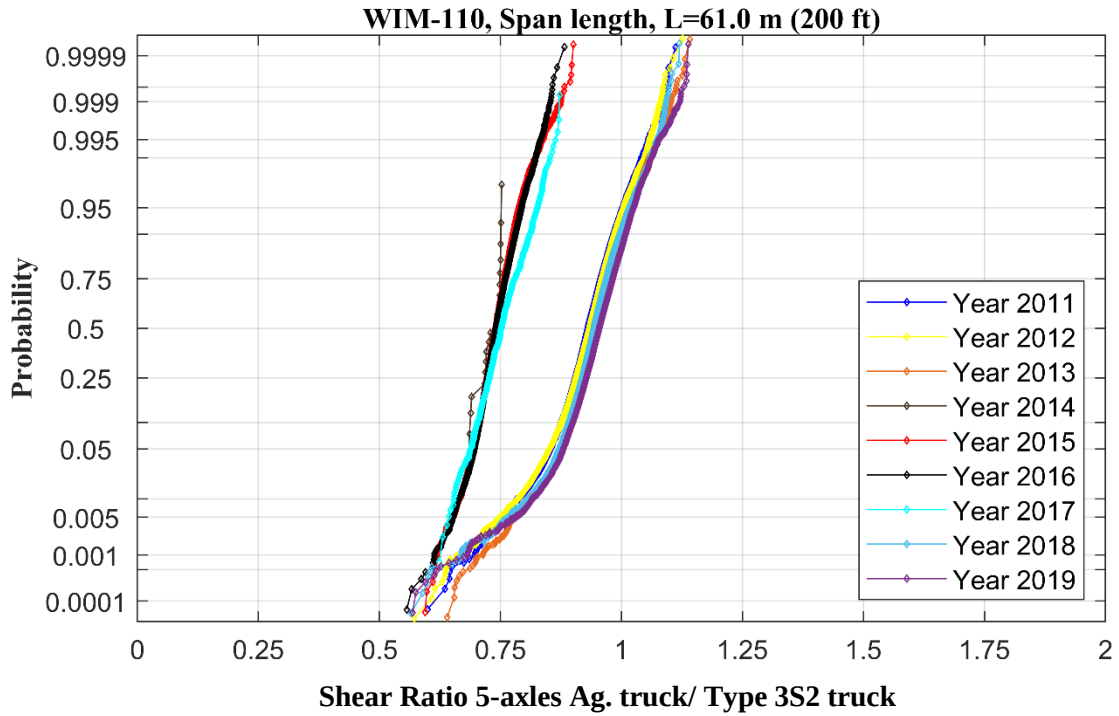
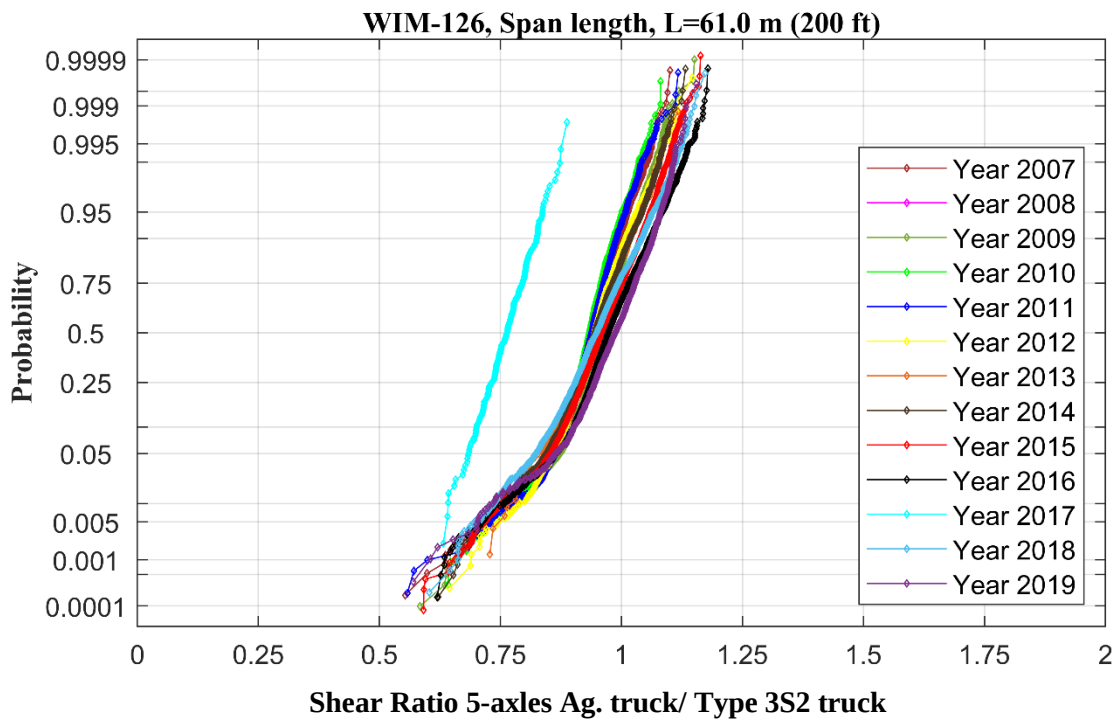
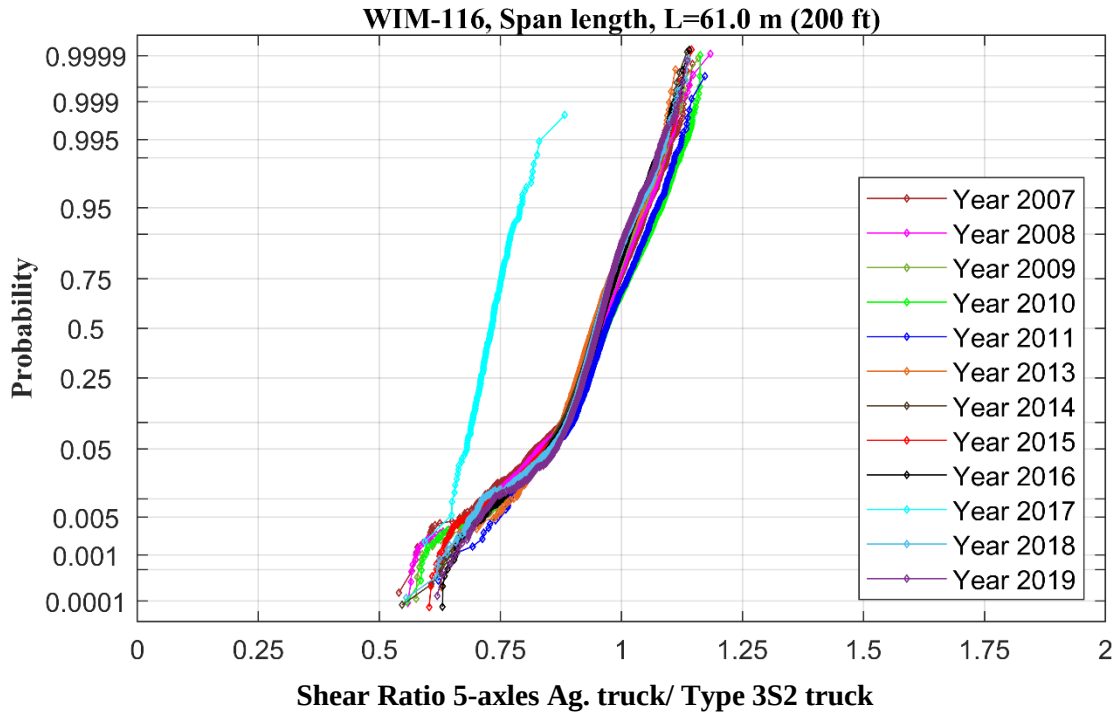


Figure 4-H.4. CDF of shear ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 36.6 m (120 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.





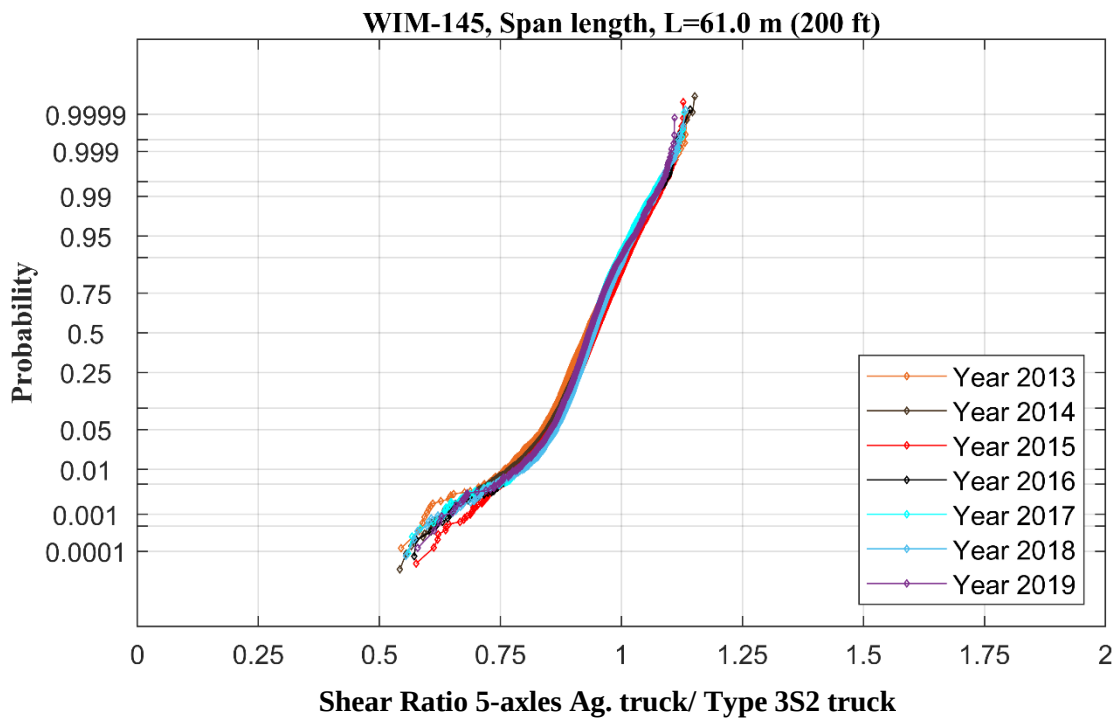
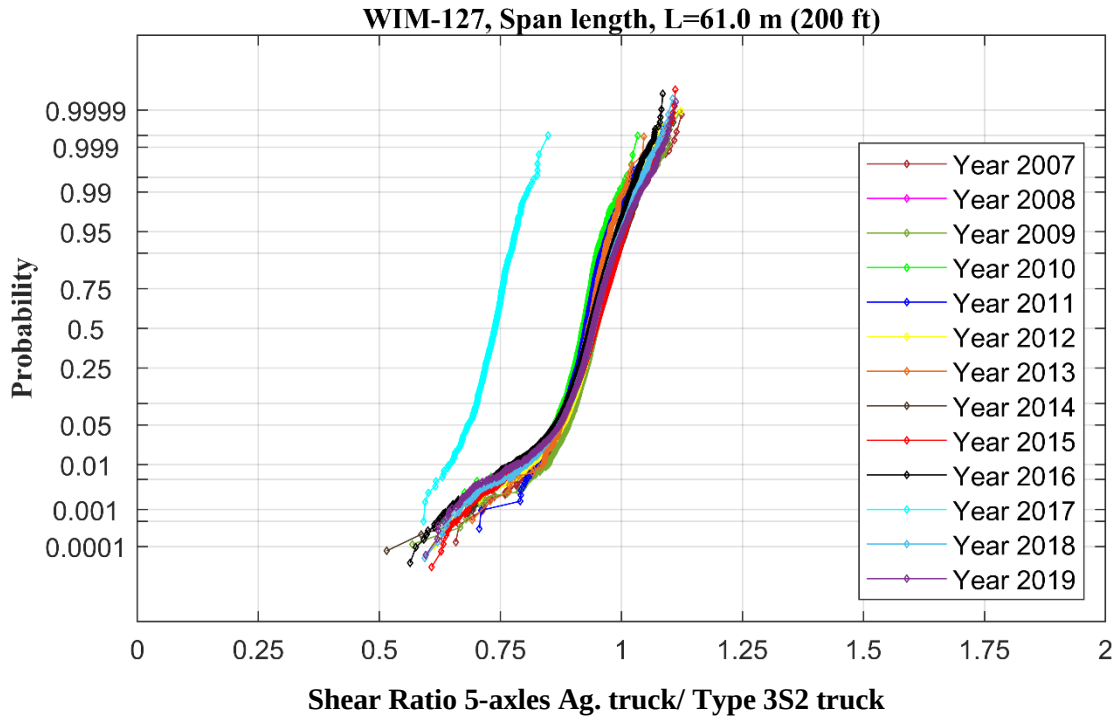


Figure 4-H.5. CDF of shear ratio for 5-axes agricultural vehicles to Type 3S2 vehicle for span length of 61 m (200 ft) for (a) WIM-110 (b) WIM-115, (c) WIM-116, (d) WIM-126, and (e) WIM-127, and (f) WIM-145.

Appendix 4-I. Notional vehicle selection for grandfather agricultural vehicles

Load effects for 3-axle and 5-axle agricultural vehicles were compared with commonly used vehicles for operating legal rating. For considered span lengths, the percentage of vehicles that produce a force effect above rating truck is presented for evaluation truck and modified truck.

Type 3 truck

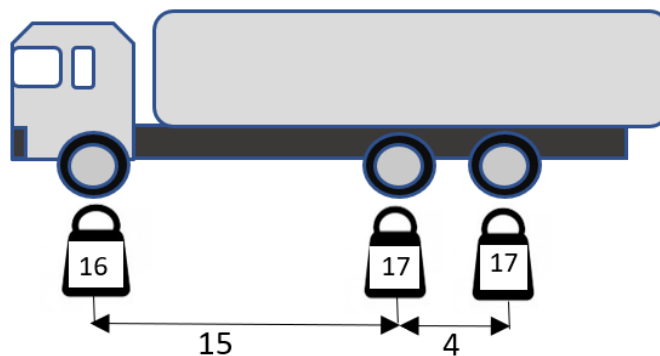


Figure 4-I.1. Type 3 legal vehicle from AASHTO legal loads.

Table 4-I.1. Number of possible agricultural vehicles above Type 3 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	226	78.50	37.60	36.00
60	598	26.40	43.80	40.70
90	973	31.20	45.87	42.00
150	1722	36.20	47.52	42.90
200	2347	38.30	48.14	43.20
250	2972	39.50	48.51	43.50
300	3596	40.40	48.76	43.60

The modified Type 3

results from multiplying each axle weight by the same scale factor of 1.175.

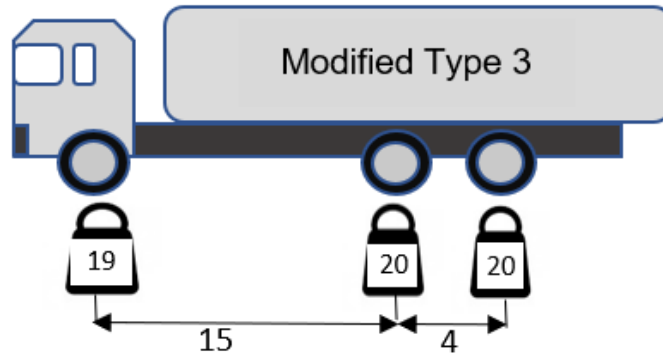


Figure 4-I.2. Modified Type 3 legal vehicle from AASHTO Legal loads.

Table 4-I.2. Number of possible agricultural vehicles above modified Type 3 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	266	1.00	44.3	0.00
60	703	0.00	51.48	0.20
90	1143	0.00	53.90	0.40
150	2024	0.20	55.84	0.70
200	2758	0.30	56.57	0.80
250	3492	0.40	57.00	0.90
300	4226	0.50	57.30	1.00

Type 3S2 truck

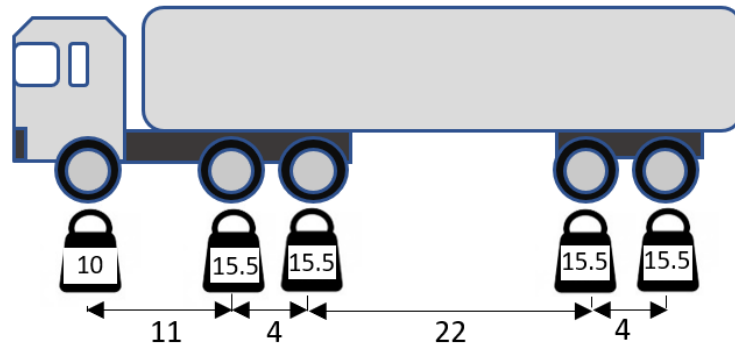


Figure 4-I.3. Type 3S2 legal vehicle from AASHTO Legal loads.

Table 4-I.3. Number of possible agricultural vehicles above Type 3S2 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	222	87.3	33.93	73.8
60	618	3.1	49.67	4.2
90	1153	9.6	57.11	26.0
150	2229	52.0	63.07	68.9
200	3127	73.3	65.3	79.4
250	4026	81.1	66.64	83.3
300	4925	84.3	67.53	85.3

The modified Type 3

results from multiplying each axle weight by the same scale factor 1.225.

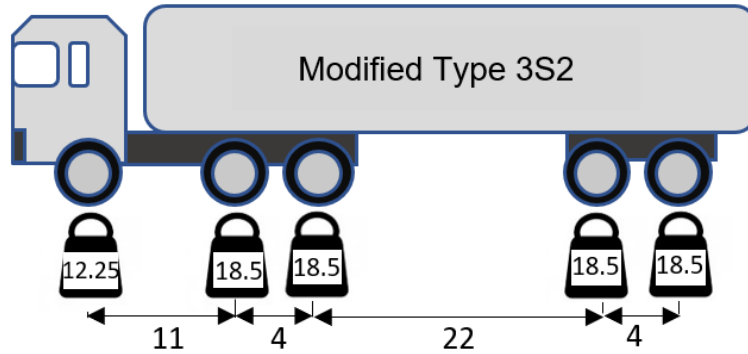


Figure 4-I.4. Modified Type 3S2 legal vehicle from AASHTO Legal loads.

Table 4-I.4. Number of possible agricultural vehicles above modified Type 3S2 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	272	0.00	41.59	0.00
60	758	0.00	60.88	0.00
90	1413	0.00	70.00	0.00
150	2731	0.00	77.30	0.10
200	3833	0.10	80.04	0.30
250	4935	0.20	81.68	0.50
300	6037	0.40	82.78	0.60

Type 3-3 truck

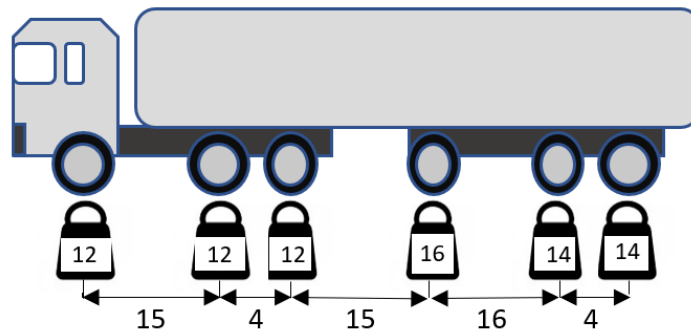


Figure 4-I.5. Type 3-3 legal vehicle from AASHTO Legal loads.

Table 4-I.5. Number of possible agricultural vehicles above Type 3-3 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	183	100.00	31.47	98.70
60	564	41.90	48.13	8.40
90	1143	11.60	58.76	13.40
150	2342	17.80	67.25	21.90
200	3342	22.00	70.44	25.90
250	4341	25.00	72.35	28.50
300	5341	27.20	73.63	30.40

The modified Type 3-3

results from multiplying each axle weight by the same scale factor of 1.1.

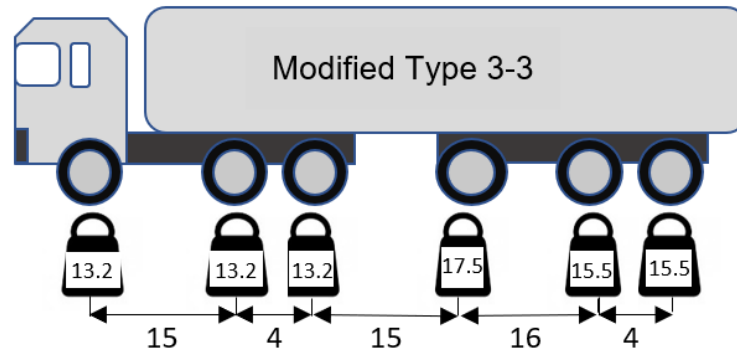


Figure 4-I.6. Modified 3-3 legal vehicle from AASHTO Legal loads.

Table 4-I.6. Number of possible Ag. Vehicles above modified Type 3-3 legal vehicle.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	203	99.9	34.77	51.4
60	621	2.60	53.12	0.60
90	1260	0.60	64.83	0.60
150	2582	0.70	74.20	1.00
200	3685	0.90	77.71	1.20
250	4787	1.10	79.82	1.40
300	5890	1.20	81.22	1.50

The results for 30 ft span bridge indicate large percentage of vehicles above modified legal Type 3-3 vehicles, due to the different configurations, and hence number of axles and axle weight for compared vehicles.

HS-20 truck

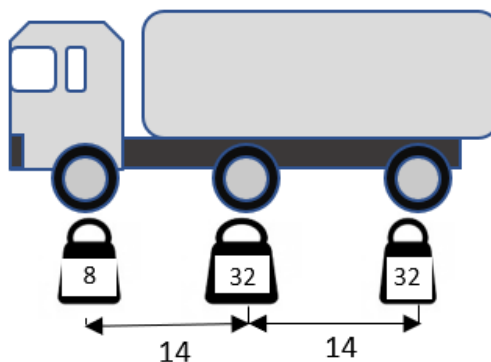


Figure 4-I.7. AASHTO HS-20 design truck.

Table 4-I.7. Number of possible Ag. Vehicles above HS-20

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	282	0.00	49.6	0.00
60	806	0.00	60.80	0.00
90	1344	0.00	64.53	0.80
150	2423	7.20	67.52	19.80
200	3322	25.50	68.64	45.70
250	4222	46.50	69.31	63.80
300	5121	62.30	69.76	73.20

The modified Type 3-3

results from multiplying each axle weight by the same scale factor of 1.175.

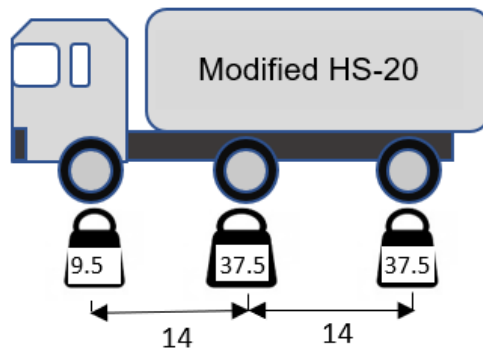


Figure 4-I.8. Modified HS-20.

Table 4-I.8. Number of possible Ag. Vehicles above modified HS-20.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	326	0.00	57.4	0.00
60	938	0.00	70.70	0.00
90	1566	0.00	75.13	0.00
150	2824	0.00	78.68	0.00
200	3873	0.10	80.01	0.30
250	4923	0.30	80.81	0.80
300	5972	0.70	81.34	1.40

SU5 truck

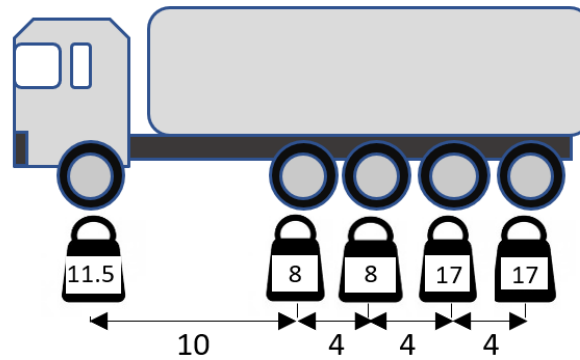


Figure 4-I.9. Bridge Posting vehicle SU5.

Table 4-I.9. Number of possible Ag. Vehicles above SU5.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	293	0.00	45.47	0.00
60	741	0.00	53.48	0.50
90	1201	3.30	56.16	36.10
150	2122	83.60	58.29	89.80
200	2891	92.00	59.09	92.10
250	3659	93.50	59.58	92.90
300	4428	94.30	59.90	93.50

The modified SU5 truck

results from multiplying each axle weight by the same scale factor of 1.350.

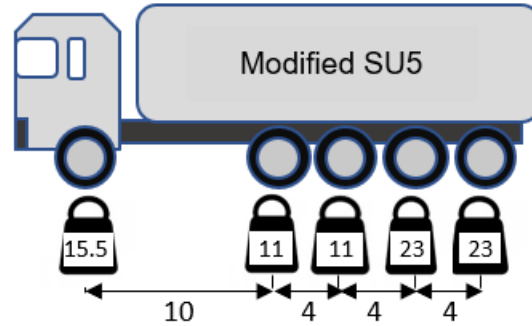


Figure 4-I.10. Modified Bridge Posting vehicle SU5.

Table 4-I.10. Number of possible Ag. Vehicles above modified SU5.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	398	0.0	61.73	0.0
60	1006	0.00	72.62	0.00
90	1631	0.00	76.24	0.00
150	2882	0.00	79.15	0.00
200	3925	0.00	80.23	0.20
250	4968	0.20	80.89	0.70
300	6012	0.50	81.32	1.40

SU7 truck

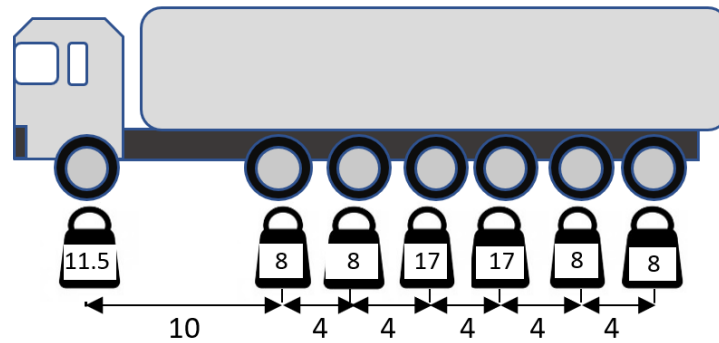


Figure 4-I.11. Bridge Posting vehicle SU7.

Table 4-I.11. Number of possible Ag. Vehicles above SU7.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	335	0.0	45.47	0.0
60	897	0.00	60.75	0.00
90	1478	0.00	66.33	0.20
150	2641	0.20	70.80	5.20
200	3610	2.10	72.47	12.20
250	4578	5.50	73.48	19.30
300	5547	9.60	74.15	25.60

The modified SU7 truck

results from multiplying each axle weight by the same scale factor of 1.120.

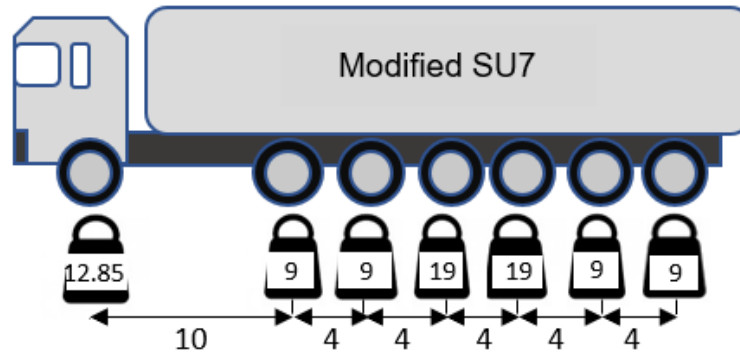


Figure 4-I.12. Modified Bridge Posting vehicle SU7.

Table 4-I.12. Number of possible Ag. Vehicles above modified SU7.

Bridge Span length [ft]	Maximum Moment [kip-ft]	% of agricultural vehicles above maximum moment	Maximum Shear [kips]	% of agricultural vehicles above maximum shear
30	375	0.0	50.89	0.0
60	1005	0.00	68.09	0.00
90	1656	0.00	74.34	0.00
150	2959	0.00	79.35	0.00
200	4045	0.00	81.22	0.10
250	5130	0.00	82.35	0.30
300	6216	0.10	83.10	0.50

It was concluded that for 3-axle possible agricultural vehicles Type 3 truck is the best representation, and for 5-axle possible Ag. Vehicles is Type 3S2 legal vehicle. Along with appropriate scale factor selected vehicles envelop the loading of agricultural vehicles with 20% overload provisions in Montana.