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Truck Platooning Impacts on Bridges: Phase II—Structural Serviceability

January 2025

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Phase II—Structural Serviceability

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Truck platooning is a new Connected and Automated Vehicle (CAV) application poised to rapidly affect the nation's trucking operations, which could have several profound impacts on the national inventory of bridges. The current study goal is to investigate potential impacts to bridge serviceability, as well as evaluate the effect on the strength limit state from a probabilistic perspective. This is being done by quantifying the numbers and degree to which bridges might have their service life reduced from more widespread use of platooning operations for heavy trucking in the nation's highway system. The focus is on determining bridge characteristics that may put a bridge at risk of not meeting current service limit state provisions found in the AASHTO LRFD Spec. (23 CFR 625.4(d)(1)(v)) and MBE (23 CFR 650.317). The calibration approach used in the development of the AASHTO LRFD Spec. (23 CFR 625.4(d)(1)(v)), as well as that used in recent NCHRP and other research projects, is applied to truck platooning to determine whether current rating methods and load factors are adequate for a truck platooning environment. Items considered include span length, continuity, material type, age, design standard, and design loading. The study investigated platooning parameters such as numbers of trucks, headway distance, and their level of penetration into trucking operations.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or “metric ton”)	Mg (or “t”)
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or “t”)	megagrams (or “metric ton”)	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Disclaimer

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Executive Summary

Introduction

Truck platooning is a new Connected and Automated Vehicle (CAV) application poised to rapidly affect the nation's trucking operations, which could have several profound impacts on the national inventory of bridges such as increased live load, long-term deterioration, deflection effects, changes to the fatigue loads, increased braking force, altered multiple presence effects, and more severe barrier impacts. Expanded evaluation of load carrying capacity or even modifications to the current design and load rating standards might be justified due to the change in traffic load pattern and the vehicle characteristics and increased road use by truck platoons in the near future. Several recent research studies have been conducted to quantify the potential impacts to bridges. However, many studies have been based on direct comparisons of platoon scenario loading as compared to existing design and or legal load levels. In the prior phase (https://www.fhwa.dot.gov/bridge/loadrating/pubs/hif21043.pdf) of the current program funded by Federal Highway Administration (FHWA), the research team (RT) focused on the impacts to bridges from the strength limit state, based primarily on load rating factors provided in the National Bridge Inventory.

Objectives of this Study

The current study goal is to investigate potential impacts to bridge serviceability, as well as evaluate the effect on the strength limit state from a probabilistic perspective. This is being done by quantifying the numbers and degree to which bridges might have their service life reduced from more widespread use of platooning operations for heavy trucking in the nation's highway system. The focus is on determining bridge characteristics that may put a bridge at risk of not meeting current service limit state provisions found in the AASHTO (American Association of State Highway and Transportation Officials) LRFD (Load and Resistance Factor Design) Bridge Design Specifications (LRFD Spec.) (23 CFR 625.4(d)(1)(v)) and the Manual for Bridge Evaluation (MBE) (23 CFR 650.317). The calibration approach used in the development of the LRFD Spec., as well as that used in recent NCHRP (National Cooperative Highway Research Program) and other research projects, is applied to truck platooning to determine whether current rating methods and load factors are adequate for a truck platooning environment. Items considered include span length, continuity, material type, age, design standard, and design loading. The study investigated platooning parameters such as numbers of trucks, headway distance, and their level of penetration into trucking operations.

This study has three distinct parts: 1) Tier 1 - Deterministic analysis of a representative sampling of 2000+ bridges with detailed load rating calculations and estimation of impacts to the national inventory, 2) Tier 2 - Probabilistic analysis of truck load effects on bridges and associated impacts using actual Weigh-in-Motion (WIM) data synthesized to represent future states with possible platoon operations and penetration, and 3) Calibration of load factors for load rating that will provide for uniform reliability across bridge types when subjected to platoon loading.

Key Factors and Assumptions

Key aspects to this study are that it includes longer span lengths, up to 500ft, which are more likely to have impacts from platoon loading. Platoon scenarios consider 2, 3, and 4 trucks with headway of 30, 50, and 70 ft and penetration rates of zero to 100%. Second, it is based on load rating of real bridges with representative variability in design type, size, and location. Finally, this study is based on actual WIM data, loading simulation, and reliability-based calibration.

Tier 1 Deterministic Analysis

The Deterministic Analysis begins with detailed calculations of span lengths and ranges for which platoon scenarios indicate load effects in excess of design loads HL-93, HS20, and the Fatigue Design Truck. Next, a database of 2000+ bridges is analyzed with AASHTOWare BrR to determine load rating factors for the service limit states and how they are affected by platoon loading. From this, seven (7) cases are identified that describe bridge types that are likely to be negatively affected by platoon loading. Negatively affected bridges are those for which a service limit state rating factor is lower than 1.0 and lower than the governing service limit state rating factor for HL-93 at the operating level, i.e., $RF_{Platoon.Service} < 1.0$ and $RF_{Platoon.Service} < RF_{HL93.Service}$. These cases are then used to calculate the numbers of potentially affected bridges in every state on either Interstate, National Highway System, or National Network.

Tier 2 Probabilistic Analysis

The Probabilistic Analysis is based on loading simulations using representative WIM data from 32 sites across 6 states to develop the statistical characteristics of the live load effects on bridges, to more accurately estimate impacts to bridges from platoons. This analysis performs simulations of platoon operations by creating “synthetic WIM data” which is derived from collected WIM data, modified with existing random heavy Class 9 trucks being joined together into platoons. Statistical parameters such as bias factors and coefficients of variation (COVs) are calculated for the different platoon scenarios and the resulting load effects of shear and moment on simple and continuous span girder bridges.

Calibration

The calibration of Strength Limit State I for MBE at the operating level for platoon loading using HL-93 as a notional live load is conducted to determine live load factors that provide a minimum target safety in terms of reliability indices. The synthetized WIM platoons along with the other individual WIM trucks were run on an influence line analysis to determine the live load envelope for simple and continuous span bridges. The calibration procedure was carried out using two approaches: 1) Calculation of live load factor for target reliability index of 2.50 per MBE operating level, and 2) Calculation of reliability index for the current HL-93 operating rating with live load factor of 1.35.

Key Findings

According to the Tier 1 analysis truck platoons may cause increased maximum load effects in terms of common bridge girder shears and moments, which may reduce load rating factors below 1.0 and impact service limit states for certain bridges. There are seven (7) cases of bridges differentiated by type, span length, age, and design which are more likely to have serviceability negatively affected by platoons. Fatigue in medium to longer span steel bridges is the limit state most likely to be affected. According to the Tier 2 analysis truck platoons may cause higher load effects in bridges as compared to existing baseline loading for longer span lengths, and the delta factor for maximum loading provides the degree of increase for load effects for positive and negative moments and shears for simple and continuous spans. However, it is observed that the HL-93 load model appears to adequately envelop these increased effects due to the uniform load portion of the model. The calibration effort suggests that no adjustments are necessary to the MBE at the operating level for HL-93 to account for the platoon loading for Strength Limit State I.

Bridges that fall within the cases established in our study have an increased potential for impacts to their serviceability. This may result in increased deterioration and maintenance. The numbers of these bridges found in each state are provided in the appendix to this report. If greater confidence in expected performance is desired for these bridges, it is suggested to verify the design level operating rating from Load and Resistance Factor Rating (LRFR) method is greater than 1.0, or perform optional load rating to determine if they pass the service limit state(s) of concern.

As CAV technology advances in the future and major shipping companies make investments for platooning operations, it will become clearer about the types, sizes, weights and variability of trucks that most frequently run in platoons on our highway network. At that time, the simulation assumptions and procedures developed in this work can be updated to provide a more accurate characterization of loading, reliability calibration, and estimation of potential impacts to bridges.

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Chapter 0: Introduction

Truck platooning is a new Connected and Automated Vehicle (CAV) application poised to rapidly affect the nation's trucking operations, which could have potentially profound impacts on the national inventory of bridges such as increased live load, long-term deterioration, deflection effects, changes to the fatigue loads, increased braking force, altered multiple presence effects, and more severe barrier impacts. Expanded evaluation of load carrying capacity or even modifications to the current design and load rating standards might be justified due to the change in traffic load patterns, vehicle characteristics, and increased road use by truck platoons in the near future. Several recent research studies have been conducted to quantify the potential impacts to bridges. However, many studies have been based on direct comparisons of platoon scenario loading as compared to existing design and or legal load levels. In the prior phase of the current program funded by Federal Highway Administration (FHWA), the research team (RT) focused on the impacts to bridge strength limit states, based primarily on load rating factors provided in the National Bridge Inventory.

The current study goal is to investigate potential impacts to bridge serviceability, as well as evaluate the effect on the strength limit state from a probabilistic perspective. This is being done by quantifying the numbers and degree to which bridges might have their service life reduced from more widespread use of platooning operations for heavy trucking in the nation's highway system. The focus is on determining bridge characteristics that may put a bridge at risk of not meeting current service limit state provisions found in the AASHTO (American Association of State Highway and Transportation Officials) LRFD (Load and Resistance Factor Design) Bridge Design Specifications (LRFD Spec.) (23 CFR 625.4(d)(1)(v)) and the Manual for Bridge Evaluation (MBE) (23 CFR 650.317). The calibration approach used in the development of the LRFD Spec. (23 CFR 625.4(d)(1)(v)), as well as that used in recent NCHRP (National Cooperative Highway Research Program) and other research projects, is applied to truck platooning to determine whether current rating methods and load factors are adequate for a truck platooning environment. Items considered include span length, continuity, material type, age, design standard, and design loading. The study investigated platooning parameters such as numbers of trucks, headway distance, and their level of penetration into trucking operations.

Assessing the impacts to serviceability of bridges from truck platoons is particularly challenging since the CAV technology is rapidly changing and highly speculative for application into highway freight operations. The technical approach to complete this research is based on current and relevant Weigh-in-Motion (WIM) data on truck weights on the road network, coupled with sound predictions and assumptions of how truck platooning will operate and penetrate the fleet of the future. The primary goal of this research is to determine the impacts to bridge serviceability, which in many ways is more challenging than strength. For instance, serviceability is based on the "average" or "effective" stresses created by the full spectrum of truck size, axle spacing and weight, and the limit states are more based in the acceptability of damage and such as cracking and/or deflection which can be more subjective. Strength impacts are simpler because it is mostly based on the possibility of one heavy truck scenario exceeding the safe load capacity of bridge(s). The technical approach for this work is based on the following foundations:

- Consider span lengths up to 500ft, which are more likely to have impacts from platoon loading.
- Based on load rating of real bridges with representative variability in design type, size, and location.
- Based on actual WIM data, loading simulation, and reliability-based calibration.
- Platoon scenarios consider 2,3, and 4 trucks with headway of 30, 50, and 70 ft and penetration rates (portion of truck traffic assembled into platoons) of zero to 100%. Platooning is known to be more feasible in long-haul trucking operations.
- Simulation of platoon scenarios by synthesizing the WIM data with the shift of 3S2 Class 9 trucks into platoons.

The research follows a tiered analysis approach, where Tier 1 is based on a deterministic analysis and Tier 2 is a probabilistic analysis that accounts for variability of random variables of load and resistance. The deterministic analysis counts the numbers of bridges that "fail" the applicable specification limit state verification for a given platoon configuration and produce a rating factor lower than for current design loads. The probabilistic analysis allows us to account for the statistical distribution of truck loading when it is put into a platoon and perform new calibration and load factor development. Platoons create a more controlled (less variable) loading scenario and thus offset any increases in nominal load effect, in general.

The Tier 1 approach provides an estimate of the numbers of bridges that will be affected using the load effects from assumed truck platoon scenarios for specific weights, wheelbase, headway, penetration and checking the service limit states from the LRFD Spec. (23 CFR 625.4(d)(1)(v)) and the MBE (23 CFR 650.317). Also, it provides a meaningful approach to conduct "comparative" analysis and see how sensitive the nation's bridge inventory is to the platoons. The primary concerns of the research are bridges that fail the service limit state under the platooning scenario with rating factors lower than those computed for current design loads, and also below 1.0 for the same service limit state.

The Tier 2 approach applies WIM data to simulations with "synthetic" truck platoons formed from trucks found in the WIM data sets. These utilize likely truck platoon configurations with prescribed penetration rates into the truck fleet. Three criteria are developed to characterize truck platoon impacts based on the following: statistical live load models for the traffic fleet with and without truck platooning, the number of platoons that contribute to the greatest load effects, and load effects at 1 in 10,000 exceedances for the traffic fleet with and without truck platooning.

Following the tiered analysis, a comprehensive reliability-based calibration utilizing synthesized WIM data and live load statistical parameters to capture characteristics of expected load from the platooned trucks is performed. The calibration of Strength Limit State I for MBE (23 CFR 650.317) at the operating level for platoon loading uses the HL-93 as a notional live load to determine live load factors that achieve the target reliability index for the full range of spans and bridge types. The calibration approach includes the formulation of a limit state function, the selection of representative bridges, the development of load and resistance statistical parameters, the selection of the analysis procedure, and target reliability index, and finally the calculation of load factors.

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Chapter 1: Literature Review

1.1 State-of-the-Art Review of Truck Platooning

Automated driving of passenger cars has been studied since 1950s (Tsugawa et al., 2013). A few vehicle platooning projects were conducted in 2000s such as SARTRE (Safe Road Trains for the Environment), PATH (California Partners for Advanced Transit and Highways), GCDC (Grand Cooperative Driving Challenge), Energy-ITS (International Transportation Systems), and Scania (Bergenhem et al., 2010). These studies considered different vehicle types such as mixed or heavy vehicles, direction of automated control such as latitude and/or longitude, or sensors to achieve various goals such as comfort, safety, energy, etc. It is known that the efficiency of platooning is in correlation with proper planning and optimization (Bhoopalam, 2018). Many researchers have been studying on platoon planning but focusing mainly on minimizing energy consumption and fuel efficiency (Sokolov et al. (2017); Zhang et al. (2017); Adler et al.,2016; Larson et al.,2016; Liang et al.,2016; Van de Hoef, 2016; Nourmohammadzadeh and Hartmann, 2016; Larsson et al, 2015; Liang et al.,2014; Liang et al.,2013; Larson et al, 2013; Meisen et al., 2008). Additionally, a number of challenges for platooning such as the interaction with other drivers on the public motorways were also pointed out by other researchers (Bergenhem (2010).

Tsugawa et al. (2016) stated that various studies on heavy truck platooning started in the mid-1990s with 'heavy truck" described in one reference (Tsugawa (2013)) as having a gross vehicle weight (GVW) of 25 tons (50 kips). Today, with the help of advanced technologies, it is achievable to electronically couple vehicles; however, there are still many questions about the effects of truck platooning and its configurations on highway bridges due to its uncertain structural effects and safety issues including comfort of other drivers on the roadways. For highway bridges, the number, type, weight, headway spacing in a platoon, and the speed of trucks might be the main parameters that are needed to describe a truck platoon. Truck platooning effects of concern include the following: 1) fatigue, 2) dynamic amplification and impact factor, 3) cracking in concrete decks, 4) deflection, 5) the effect of braking and collision, etc. According to the Competitive Enterprise Institute (CEI), Following-too-Closely (FTC) statutes in state motor vehicle codes is the main obstruction to conduct platoon testing on public highways (Scribner, 2019). It was also reported that up to 2019, commercial automated vehicle platooning has been authorized by 20 U.S. jurisdictions. Figure 1 shows a map with the number of states authorized, not authorized, and those who took action to authorize truck platooning. Since 2015, various jurisdictions are approving exemptions in FTC statute for platoons since (Scribner, 2019) which resulted in heightened activities in research studies on truck platooning in the last years. However, most of the available studies are about platooning technology and optimization. There are very limited studies which evaluate the effects of truck platoons on bridges (Wassef, 2021).

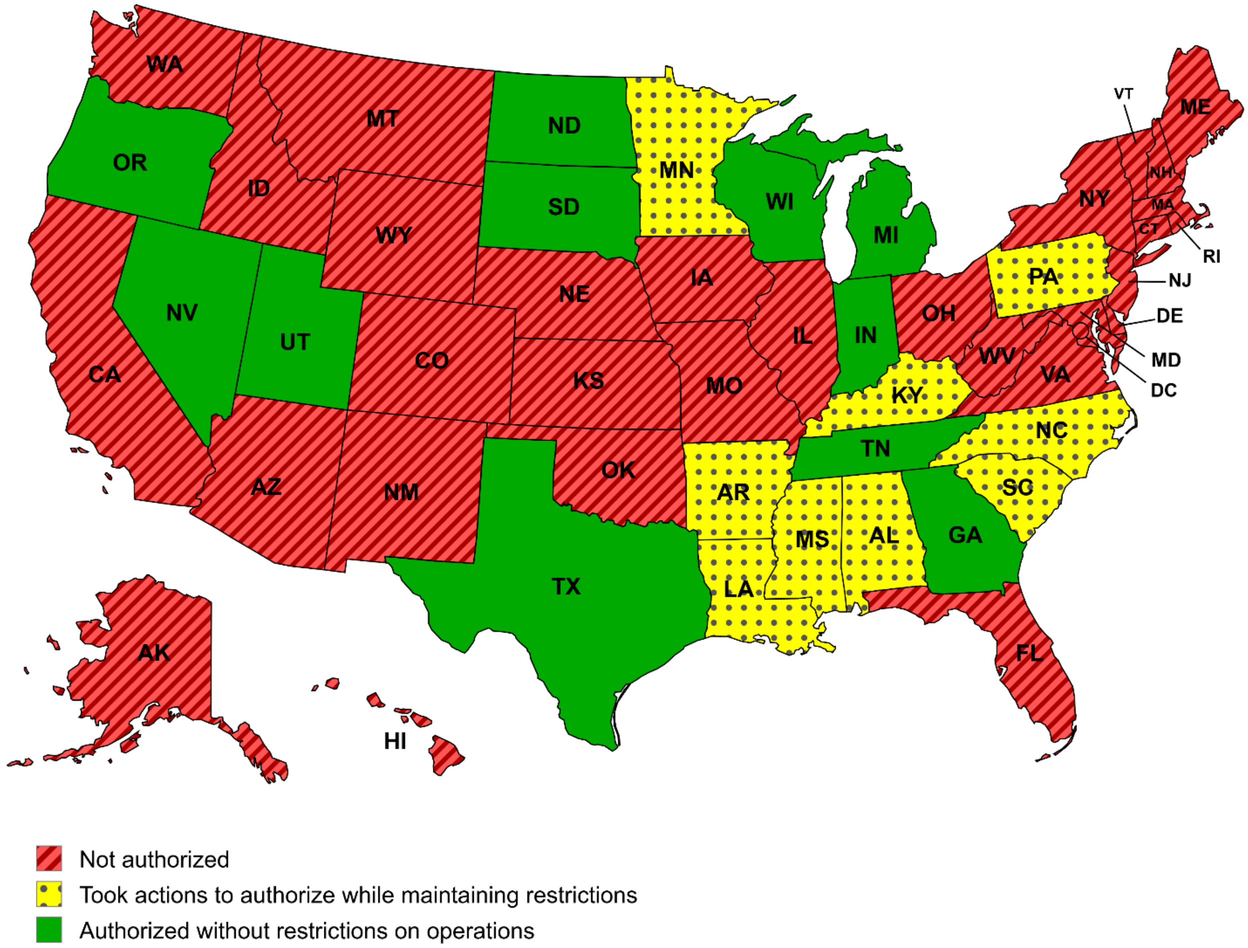

Figure 1. Classification of U.S. jurisdictions based on authorization of platoon (Scribner, 2019).

1.1.1 Recent Studies on Truck Platooning for Highway Bridges

Hartmann (2019) discussed truck platoons for highway bridges and stressed the need for a clear understanding of the possible load models in order to determine if the design and/or rating loads are adequate. Creating an adequate live load model addresses various aspects, such as the number of trucks, truck configuration, speed, weight limits and the effect of braking and collision. Furthermore, the speed of a truck is one of the primary factors to contribute the impact (dynamic load), adding to the apparent vertical loads. Additionally, for horizontally curved bridges, centrifugal forces may be generated at a level higher than expected. Also, if braking in a platoon is coordinated, it might cause larger horizontal loads which may result in an increased rate of failure of bridge bearings as well as expansion joints. For the load models, it is suggested to treat the platoon as one long truck with individual and tandem axle weights and spacings. The weight limits will need to be determined to evaluate bridges and establish their load ratings. According to Hartmann (2019), in the future, some bridges may benefit from identifying the locations where headway distances should be increased.

1.1.1.1 Load Effects of Truck Platoons (Fatigue and Strength I)

According to Lipari et al. (2017) congestion events can lead to critical load effects on medium and long-span bridges. For instance, automated vehicle platooning may deteriorate the aging road infrastructure when the inter-vehicle gaps are small. Thus, ITS could be used to avoid such events. The authors proposed an infrastructure-to-vehicle (I2V) system that could mitigate bridge loading based on the minimum control gap among heavy vehicles. A sensor in the truck computes the distance between the front vehicle and the driver is warned if the gap falls below the indicated threshold, as shown in Figure 2.

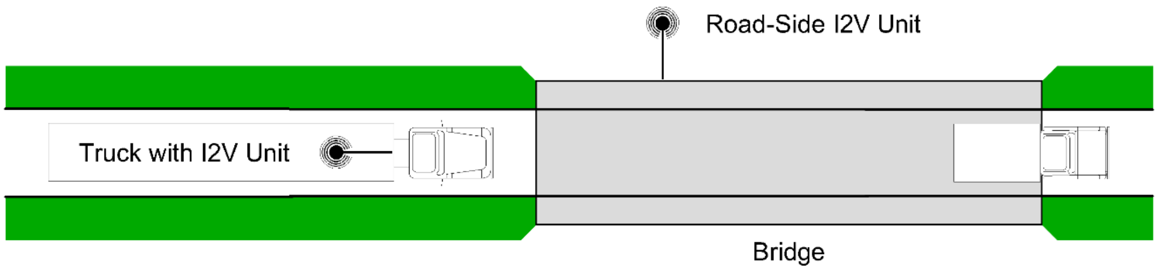

Figure 2. Illustration of the proposed gap control system (Lipari et al., 2017).

The results showed that the application of the proposed technology reduces the loading by 10% on a 200m span bridge when 10% of the trucks comply with the suggested gaps. When 90% of the trucks respond to the gap control technology, the loading during congestion in the 200 m span bridge is reduced by 47%. Additionally, Kamranian (2018) studied the performance of the Hay River bridge under the effect of different truck platoons including two-, three- and four-truck platoons of Alberta non-permit and Alberta permit trucks. It was found that the bridge had adequate capacity for two-truck platoons. However, for three- and four-truck platoons, the load ratings were insufficient.

Recently, Couto Braguim et al. (2021) investigated the effects of truck platooning on the fatigue performance of steel girder bridges. Various types of platoons for simple spans and two-equal continuous spans bridges were simulated using line girder analysis. The rainflow counting method and Miner's rule were used to obtain the stress ranges and cycles and quantify the fatigue damage, respectively. For comparison purposes, the fatigue damage of different platoons is normalized by the fatigue damage caused by the AASHTO LRFD Fatigue Truck. It was concluded that, in some cases, truck platooning decreases the fatigue damage and the number of cycles. For specific ratios of span-to-wheelbase in a platoon, fatigue damage decreases as gap distance increases. Furthermore, for specific ranges of span lengths and depending on the platoon configurations, it may be less damaging to travel in truck platoons rather than traveling as a single truck in relation to fatigue damage.

Puckett et al. (2020) and Yang et al. (2021) presented a framework to determine a platoon permit load by evaluating strength limit states for steel and prestressed concrete I-girder bridges designed using both, LRFD and LFD (Load Factor Design). A parametric study was performed using different girder spacings, span lengths, numbers of spans, types of structures, truck configurations, numbers of trucks, and adjacent lane loading scenarios. They used the Monte Carlo simulation to calculate the reliability indices β. It was concluded that loads significantly higher than legal loads are acceptable for truck platoons with lower uncertainties while maintaining a traditional operating target β = 2.5, consistent with permit loading in the MBE (23 CFR 650.317). Live load factors were developed and presented for a potential new permit load, i.e., a platoon permit.

Barker and Puckett (2016) evaluated bridges on the I-80 corridor in Wyoming where a tight convoy of trucks might appear after temporary closure of the bridges due to environmental conditions. They considered their effect to be similar to that of a platoon of trucks due to 1) higher demands of truck traffic than those considered in LRFD Spec. (23 CFR 625.4(d)(1)(v)) and 2) appearance of a tight convoy of trucks. A reliability-based analysis was conducted showing that the reliability indices are significantly less than 3.50. This was shown to be true for shorter multi-span bridges under the impact of multiple closely spaced heavy axles that are not well represented by the HL-93 design load. It should be noted that the live load bias for the controlling vehicle groupings were high, between 1.36 and 1.92. Recommendations to increase the reliability indices for design of bridges along the I-80 corridor in Wyoming are as follows: 1) incorporate the LRFD Spec. (23 CFR 625.4(d)(1)(v)) commentary dual tandem load into the standard HL-93 loading, and 2) increase live load factor from 1.75 to 1.90 to meet the target reliability index of 3.5.

Wassef (2021) investigated the effects of truck platoons with various configurations, numbers and spacings of trucks in a platoon on highway bridges at the strength limit state. In the evaluation of the existing bridges, truck platoons are treated as legal loads. It was concluded that the load effects of truck platoons might be higher than the standard design loads, which depends on the number of trucks, the spacing between trucks, the configuration of trucks in a platoon, and the bridge span length. Higher load effect was observed for the smaller spacing between trucks in a platoon, except the negative moment region for continuous spans. Also, the ratio of the load effects of truck platooning and design loads increases with the increase in the number of trucks in a platoon for long-span bridges whereas design loads control the design for short spans. It was also concluded that existing bridges with an inventory rating factor of higher than one for HL-93 and HS-20 design load will be able to carry all truck platoons considered in that study. Bridges with an operating rating factor of higher than one for HL-93 will safely support the truck platoons except for spans longer than 200 ft under the spacing of 30 ft between three- and four-truck platoons. Bridges with an operating rating factor of higher than one for HS-20 might be affected by two-, three- and four-truck platoons with the spacing of 30, 50 and 70 ft between trucks.

1.1.1.2 Design Implications of Truck Platoons

Yarnold (2019) conducted a study to identify potential conditions to which these fundamental assumptions that underlie our past and current design codes may not be adequate. A series of analytical studies were conducted on steel multi-girder bridges. It was concluded that the current design specification performs well, except for longer span structures carrying closely spaced truck platoons. In these cases, live-load positive bending moment and shear force demands might be substantially greater. The same conclusion was also given in the other related study by Tohme (2019). Tohme and Yarnold (2020) performed a parametric study for a variety of bridge span configurations and span lengths as single-span, two-span and three-span continuous bridges with span lengths of 6m (19.7 ft), 20m (65.6 ft), 37m (121.4 ft) and 74m (242.8 ft). Load ratings were calculated using three different methodologies (Allowable Stress Design, Load Factor Design, and Load and Resistance Factor Design) for each structure for a range of truck platoons with various number of trucks and spacing between trucks in a platoon. For comparison, AASHTO design and legal load ratings were also calculated for each bridge and were used to quantify the adequacy of current bridges to carry truck platoons. It was concluded that existing load rating factors computed per LRFR (Load and Resistance Factor Rating) may be reduced for truck platoons in the positive moment region for long spans; however, there is likely no issue for negative moment region. The number of trucks and the spacing between trucks in a platoon are the most influential parameters in the load rating, which should be quantified.

1.1.1.3 General Assessment of Truck Platoons: Impact, Advantages, Obstacles and Solutions

Birgisson et al. (2020) studied the assessment of the potential impacts, benefits, and impediments of the introduction of automated trucks and truck platooning on Texas highway infrastructure. The authors analyzed bridges for two- and three-truck platoons with constant spacing of either 30 or 40 ft and including the following vehicles: the Alabama 3S2_AL (18-wheeler), Delaware T540 (DE five-axle semi), Florida C5, Kentucky Type 4, and the Mississippi HS-Short. The truck properties are given in Figure 3 to Figure 7. The overall methodology is given in Table 1. It should be noted that in Table 1 “Prioritization Metric (PM)” was used to prioritize/rank the bridges for carrying future truck platooning. The PM was calculated as the product of the converted LRFR load rating factors and the National Bridge Inventory (NBI) structural evaluation appraisal ratings.

Similarly, Thulaseedharan and Yarnold (2020) focused on evaluation of the Texas concrete bridge inventory when subjected to potential truck platoon loading. NBI database combined with a literature review was utilized to make assumptions allowing truck platoon load ratings to be back calculated for the prestressed concrete bridges likely to foresee platoons (nearly 3,000 bridges). It was concluded that the original design load of the bridge has a significant impact on its ability to support future truck platoon loading; for instance, bridges designed for HL-93 loading are likely low priority for further evaluation. The spacing between trucks and the truck type (axle weights and spacing) within a platoon can have a moderate and significant influence on the bridge load demands, respectively. The number of trucks within a platoon was found to have little impact in shorter spans. The same conclusion can be also found in Thulaseedharan (2020)'s study which is on the impact of truck platooning on Texas bridges by using the same approach.

Figure 3. Axle weights and spacings of Alabama 3S2_AL (18-wheeler) (Birgisson et al., 2020).

Figure 4. Axle weights and spacings of Delaware T540 (DE 5-axle semi) (Birgisson et al., 2020).

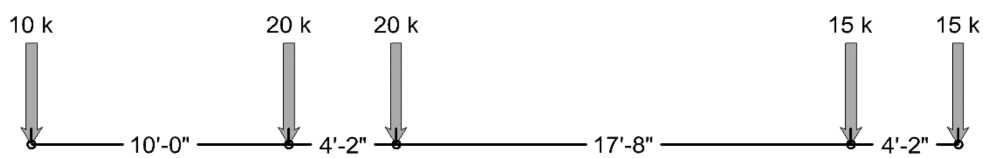

Figure 5. Axle weights and spacings of Florida C5 (Birgisson et al., 2020).

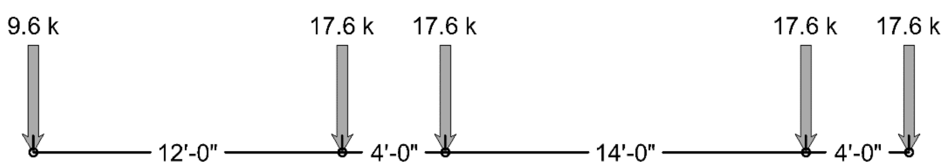

Figure 6. Axle weights and spacings of Kentucky Type 4 (Birgisson et al., 2020).

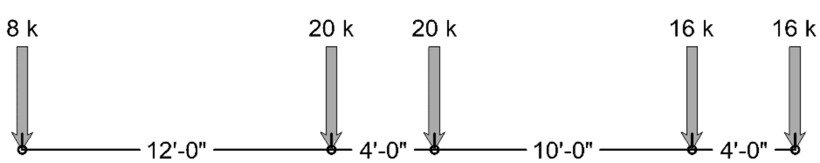

Figure 7. Axle weights and spacings of Mississippi HS-Short (Birgisson et al., 2020).

Table 1. Prioritization of bridges for future truck platoon loading (Birgisson et al.,2020).

Step	Description
1	Extract NBI data
2	Identify design methodology (the year built (or rehabilitated) was utilized to identify the design methodology)
3	Calculate design live load demands (from the original design methodology)
4	Determine bridge capacity and dead load demands (by the original design methodology)
5	Calculate truck platoon live load demands (the maximum truck platoon moments)
6	Calculate truck platoon load ratings (by the original design methodology) • The impact factor for truck platoons was the same as that prescribed by AASHTO • Deterioration was not considered at this stage. However, this factor has been included in the prioritization, see Step #7.
7	Determine Bridge Prioritization: • The rankings were made using a Prioritization Metric (PM) • The bridges were categorized for simplicity: o Prioritization Metric (PM) < 0.7: Category 5 (Highest) o PM 0.7-0.8: Category 4 o PM 0.8-0.9: Category 3 o PM 0.9-1.0: Category 2 o PM > 1.0: Category 1 (Lowest)

Hassan et al. (2020) investigated the impacts of truck platooning on transportation infrastructure in the south-central region accounting for three objectives: 1) the operational, environmental (fuel savings, and emissions), and safety impacts of various truck platooning, 2) impacts to the structural pavement resulting from these truck platooning, and 3) a feasibility study for implementation of truck platooning. The result of corridor-level analysis showed that the truck platooning adversely affects traffic operation, and safety during peak-hours period while it performed very well in the off-peak hour period. Thus, minimizing the size of platoon to two trucks only during peak hours is suggested. For the addressing of impact of truck platooning on pavement, elastic and dynamic-viscoelastic FEA (Finite Element Analysis) models were used. Some of the conclusions are that lateral tire wandering can have an influential effect on the fatigue life and permanent deformation damage of pavement. Also, fatigue life of pavement can be decreased by between 14% and 35%, in terms of numbers of cycles over 20-year design life. Sayed et al. (2020) illustrated the impact of truck platooning on the performance and integrity of existing bridges. A case study representing a typical scenario for truck platooning is provided to show the effect of platooning on the foundation of an existing bridge. It showed that a reduced integrated bridge load rating should be anticipated for truck platoons compared with the conventional single or two truck patterns commonly used in practice. Based on examples for continuous and simple-beam (stringer or girder) bridges, it was concluded that the components of both the superstructure and the foundation would be subjected to straining actions (reactions/foundation loads, moments and shears) due to truck platoon loads that are potentially higher than those used in current load ratings.

1.1.1.4 Dynamic Impact of Truck Platoons

Braley (2019) studied characterization of vehicle-bridge interaction including platooning cases. Three-dimensional (3D) FEA models are created for a single span and 2-span bridges with span lengths of 100 and 140 ft. The midspan displacement amplification and the maximum midspan deflections were plotted for various truck headway distances. It was concluded that Rolling straightedge criteria with common limits (i.e., 1/8 to 1/4 inch deviation over 10 to 16 feet) are not effective at reducing dynamic amplification. More vehicles with short spacings between vehicles increase the static load effect, but the dynamic amplification will likely be less than what would occur for a single vehicle. Therefore, in the design of a truck platoon, the dynamic amplification factor for a single vehicle can be used to conservatively account for the total dynamic response. Additionally, Bergenudd (2020) calculated the dynamic amplification factor (DAF) based on bending moment for various numbers of vehicles in a platoon and headway distances which are both random and equal distances. It was seen that a platoon with equal distances had higher DAF than the one with random distances. Also, resonance might be seen for a platoon with equal distances. The relation of DAF and number of vehicles for various bridge lengths from 5 m (16.4 ft) to 40 m (131.2 ft) was also presented. In addition, Bergenudd (2020) considered single and multiple trucks events. In multiple trucks events, gross simplifications were made when estimating the traffic from multiple heavy vehicles traversing the bridges. WIM measurements were used to estimate the percentages of each vehicle type traveling on the bridge. Several statistical assumptions related to vehicle velocity, model, headway distance and flow rate (i.e., vehicles per hour with an estimated value of 200 vehicle/h) were also made to simulate traffic for the multi truck events based on the published literature (see Table 2) . Side-by-side truck events were not considered since the DAF would be lower for trucks traveling simultaneously in multi-lane bridges.

Table 2. Statistical assumptions for multiple truck events based on literature.

Parameters	Statistics	Source
Velocity	Log-normal distribution	Caprani et al. (2012)
Vehicle model	Percentage from measurements	Vägverket (2004)
Headway	Poisson probability distribution	Caprani (2005)
Flow rate	Maximum vehicle/h from measurements	Vägverket (2004)

1.1.2 Braking Control Technology for Truck Platoons

Truck platoons have potentially multiple redundant ways of communicating that the lead vehicle is braking. Vehicle-to-vehicle communication (V2V) and radar are both used. The understanding is that the wireless link between the vehicles creates a slower lag than active braking systems or traditional braking. A comparison of the braking process during typical operation, radar control, and V2V is provided in Figure 8.

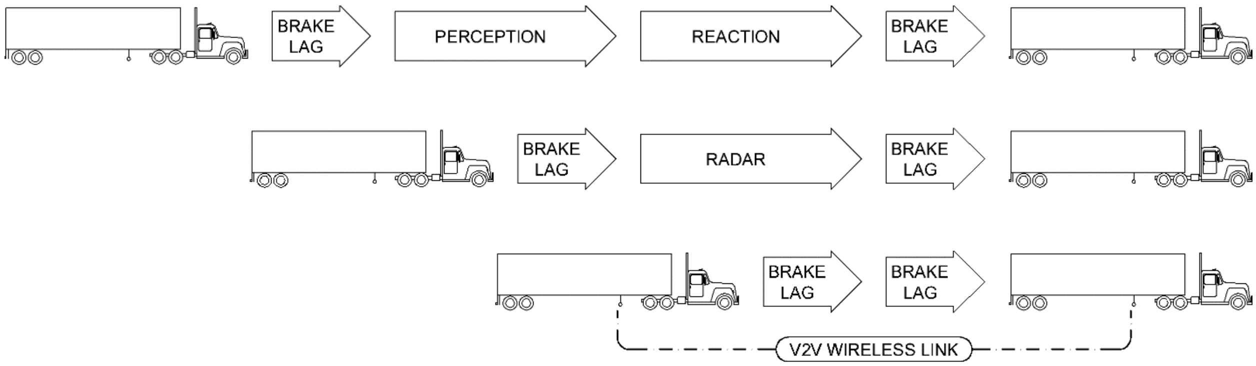

Figure 8. Braking process comparison among typical operation (top), braking control using radar (middle), and braking controls with truck-to-truck (V2V) communication (Windover et al., 2018).

Information from brake test studies is limited. The Federal Motor Carrier Safety Administration (FMCSA) conducted a study (Lascurain, 2021) finding that, while platooning capitalizes on the reduced drag associated with close following distances, these following distances should account for variability in stopping distance performance, particularly the effect of brake type. In this study, the stopping distance variability shows that for 60 mph full effectiveness stops for a tractor and unbraked control trailer loaded to the tractor GVWR (gross vehicle weight rating, i.e., maximum weight a vehicle can safely carry when fully loaded), drum/drum brakes have a 95 percent probability that the vehicle will have a stopping distance between 208.8 feet and 255.5 feet. Given the same test conditions, disc/drum brakes have a 95 percent probability that the vehicle will have a stopping distance between 196.3 feet and 250.7 feet. Disc/disc brakes have a 95 % probability that the vehicle will have a stopping distance between 192.8 feet and 249.1 feet.

Platooning may also be limited by the lowest braking capability of the vehicle in the platoon. Murthy et al. (2016) showed how the lowest braking capability may decide the braking power. Fahad et al. (2023) investigated the effects of lateral tire wander. In the case of zero wander mode, fatigue life of the pavement decreases by 1.2 years; while the use of uniform wander mode delays the rutting by 1.6 years, thereby increasing lifetime of the pavement.

1.2 Stakeholder Engagement

Truck platooning models can be classified into two major types per the ENSEMBLE (Enabling Safe Multi-Brand Platooning for Europe) (2024) project from EU which focused on evaluating the potential of multi-brand solutions.

- Platooning as a Support Function (PSF) is the first technology model that should be quickly deployable. It is a driver support function like Adaptive Cruise Control (ACC), as it regulates the distance to the vehicle in front. By using V2V communication, the PSF is safer as strong braking events are detected faster than by using conventional, on-board sensors alone like radar or camera. Additionally, braking waves (string instability) can be damped to optimize traffic throughput. The gaps between vehicles are around 1.5 seconds for PSF type applications. The gap, combined with the real-world stability of platoon formation and speed, resulted in minimal fuel consumption and emissions compared to adaptive cruise control. However, such systems may improve the overall safety of truck operations.

- Platooning as an Autonomous Function (PAF) is the second technology model. The PAF provides the vision of the ENSEMBLE Partners for the future of Platooning based on theoretical considerations and assumed advancement of the state of the art of automated driving technologies. It foresees a driver in the first truck followed by a maximum of two trucks with the driver out of the loop, travelling from hub to hub. In this case, the V2V connectivity between the trucks acts as an enabler allowing the automation of the following vehicles. The PAF places itself between a support function and a fully autonomous truck.

In terms of policy, generally, the rules are that the individual trucks conforming a platoon need to follow Truck Size and Weight (TS&W) provisions (e.g., Minnesota Statutes 2023 169.881). Other laws and/or policies have focused on Following Too Closely (FTC) laws, or hands on the wheel. Regardless of the truck platooning model, currently, the maturity of truck platooning business models and adoption is unclear as it is shaped by broader trends and market factors in the industry.

- There is significant interest in Level 4 automation for trucks. There are more market possibilities for a truck that drives itself from hub-to-hub so naturally attention and resources appear to be focused in this area by companies like TuSimple, and Torc Robotics. While Level 4 automation is conceptually a plus for platooning, companies today are focused on proving the technology works for a single vehicle in real-world settings before focusing on PAF type applications in terms of their product.

- Truck platooning, at least in the U.S, is dependent on a particular type of hauling between specific points (hub to hub). Those are a subset of the types of loads and routes fleet operators typically operate. For platooning to be scaled, there are both technology aspects (interoperability of systems between different makes and models of trucks) and business models (identification of platooning opportunities, dispatch coordination) that still have not been resolved.

- As technology matures, the workforce-related issues, especially around training, roles and responsibilities of operators, and acceptance of new technology remain to be fleshed out.

Early-mover companies that were advancing PAF type functionality (like Locomation) have shut down or shifted focus.

While companies like Walmart and FedEx have participated in successful demonstrations of truck platooning, partnering with original equipment manufacturers (OEMs), their long-term plan for using platooning technology is unclear. A study by New York State Energy Research and Development Authority on Truck Platooning Policy Barriers interviewed several trucking companies who expressed interest but had specific and differing operational processes, vehicle types that create barriers in platooning technology adoption. Some OEMs like Daimler have expressed skepticism about truck platooning while others still view it as a promising technological advancement.

Another factor complicating the industry outlook is the translation of field test results to real-world operations. A recent Canadian cooperative truck platooning trial on Alberta's public highways using trucks involved in commercial operations, highlighted the challenges in realizing the desired fuel and emissions savings in real-life due to weather, hilly terrain, frequent disengagement of platoons (and their subsequent reengagement).

1.3 Summary of Literature Review

A review of the state-of-the-art research for truck platooning effects on highway bridges is presented. The following conclusions can be drawn from this review:

- There is still uncertainty in defining a truck platoon for highway bridges in order to quantify its effect on bridges.
- Studies on the load effects of truck platoons, as well as fatigue and impact factor, were initiated in recent years. There are also studies that investigated the impacts, benefits, and obstacles of truck platoons and proposed solutions. However, the existing studies are still very limited.
- There are not any studies that considered other effects of truck platooning on highway bridges at the service limit state such as deflection or cracking.

Truck platooning remains an important topic calling for further research to determine both the make-up and effects of these groups of closely spaced trucks on highway bridges. The technology to link trucks travelling in a platoon exists and industry is looking to capitalize on the benefits of this mode of transport; however, the effects of truck platoons on highway bridges have not yet been fully established. The performance of highway bridges should be evaluated not only at the ultimate limit state but also at the service limit state. Current bridge design and evaluation manuals also need to be revisited. The effects of fatigue, impact, cracking, deformation, braking and collision, etc., also need to be investigated. Phase II of this research will go a long way in addressing these needs and will act to inform the bridge design community of the potential effects of truck platooning on bridges. At the same time, the findings of this research may aid in developing measures for safe operation of truck platoons.

Chapter 2: Tier 1 – Deterministic Analysis

2.1 Possible Platoon Configurations

In Phase I of the FHWA platooning research program, an initial WIM analysis was conducted to find the most probable truck configurations that could create a platoon (Wassef, 2021). From that WIM data analysis, it was found that the most common vehicles are Class 9, 5-axle trucks. The WIM traffic data evaluated as part of the project also contains a great number of Class 5 and Class 6 trucks. However, these vehicles were not found to produce the controlling load effects in platoon configurations. Therefore, a 5-axle tractor-trailer truck with a steering axle, dual-axle at the rear of the tractor, and dual-axle at the rear of the trailer was identified as the controlling loading for platoon configurations. Phase II will build upon Phase I results by using synthetic WIM platoons to evaluate the impact of platoon loading on US bridges.

In order to determine the effects of the selected platoon configuration on highway bridges, the research investigated the effects on bridges in terms of moments and shears of 5-axle single trucks and platoons on representative US bridges.

2.2 Platoon Configurations

The current research considered three configurations of 5-axle and 6-axle vehicles to investigate the effects of platoon loadings on bridges. The platoon trucks were created from AASHTO legal trucks including Type 3S2 (Figure 10) and Type 3-3, and a typical Class 9 WIM truck developed based on WIM data analysis (Figure 9).

The platoon configurations were formed from 2, 3, and 4 trucks. Headway spacing between these trucks is the distance from the rear axle of the leading truck to the lead axle of the following truck. Based on Phase I assumptions, Chapter 2 analysis considered the same headway spacings of 30, 50, and 70 ft.

Platoon truck configurations were created from three different 5-axle and 6-axle trucks, formed into 2, 3, and 4-truck platoons, spaced 30, 50, and 70 ft apart. The nomenclature used in the analysis for every platoon case is described by the type of 5-axle truck, the number of trucks in a platoon, and the headway spacing. For instance, a platoon truck created from a Type 3S2 truck, with two trucks spaced 30 ft apart is denoted by 3S2-2-30 (Figure 11). The list of all truck platoon cases used in the analysis is presented in Table 3.

Chapter 2 deterministic analysis developed live load ratios for the identified platoon configurations, comparing the load effects from the platoons to HS-20 and HL-93 loading. Influence line analysis was used to determine maximum load effects for 30 different platoon cases across a wide range of span lengths and compared these maximum loads to the AASHTO design load effects. The analysis investigated both simple span and two-span continuous bridges with span lengths ranging from 80 to 500 ft in 20 ft increments.

The span lengths considered in the Chapter 2 analysis extend beyond what was done in Phase I. Live load factors and dynamic impact allowance are taken from AASHTO for the calculation of the platoon live load ratios to be consistent with typical rating practice. The live load ratio results will be used to identify span lengths and platoon configurations for the synthetic Weigh-in-Motion analysis in Chapter 3. Results from the Chapter 3 analysis provided statistical parameters of platoon live load for calibration of the platoon live load factor in Chapter 4. A calibrated live load factor is used to assess the effect of platoon loading on bridge structures.

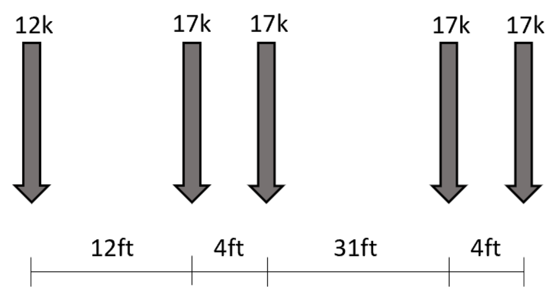

Figure 9. Typical 5-axle WIM truck, Phase I representative 5-axle truck configuration.

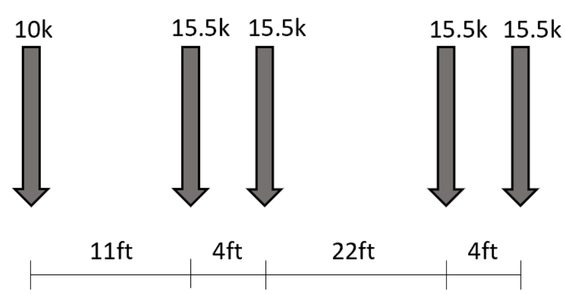

Figure 10. Type 3S2 truck, Phase I representative 5-axle truck configuration.

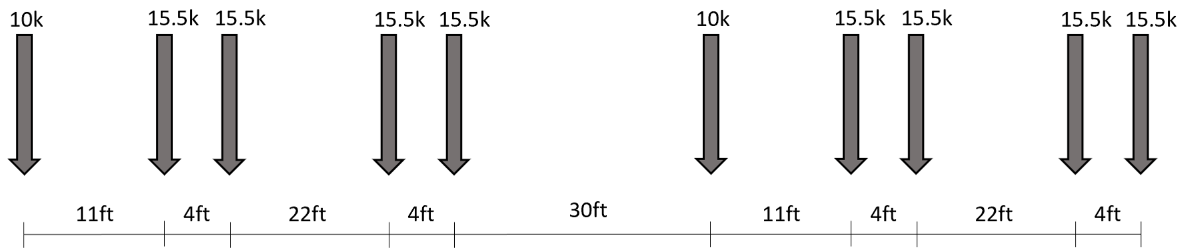

Figure 11. Truck platoon configuration for 3S2-2-30.

Table 3. Platoon cases used from Phase I project.

WIM truck	Type 3S2 truck	Type 3-3 truck
WIM-1-00	3S2-1-00	33-1-00
WIM-2-30	3S2-2-30	33-2-30
WIM-2-50	3S2-2-50	33-2-50
WIM-2-70	3S2-2-70	33-2-70
WIM-3-30	3S2-3-30	33-3-30
WIM-3-50	3S2-3-50	33-3-50
WIM-3-70	3S2-3-70	33-3-70
WIM-4-30	3S2-4-30	33-4-30
WIM-4-50	3S2-4-50	33-4-50
WIM-4-70	3S2-4-70	33-4-70

2.3 Truck Platooning Effects on Different Span Lengths

2.3.1 Representative Bridge Spans Used for Platoon Loading

Influence line analysis conducted in Chapter 2 considers bridge spans ranging from 80 ft to 500 ft. The minimum span length of 80 ft. was selected as smaller span lengths cannot generally accommodate more than one truck at a time, and thus are not influenced by platoon loading. Selected 5-axle trucks have lengths varying from 41 to 54 ft and with a minimum headway spacing of platoon taken as 30 ft, the minimum bridge span length converges to 80 ft. For the maximum spans, the upper limit of 500 ft was selected, as about 0.1% of the current US bridge inventory (NBI,2020) has a maximum span length greater than 500 ft.

2.3.2 Influence Line Analysis for Design Loads

Influence line analysis is conducted to find maximum shear and moments of AASHTO design loads, including HL-93, HS-20, and AASHTO LRFD Fatigue Truck loadings.

The governing load case for HS-20 loading is taken as the maximum of:

- The standard HS-20 truck, with axle weights of 8 kips, 32 kips, and 32 kips with a fixed first spacing of 14 ft and variable second spacing between 14 and 30ft ,
- The military loading, tandem load with axle weights of 24 kips spaced 4 ft apart,
- The lane load of 0.64 kip/ft with a point load of 18 kips for a moment, and lane load with a point load of 26 kips for shear.
- The lane load of 0.64 kip/ft with two point loads of 18 kips, placed in two adjacent spans, for a negative moment case to maximize the load effect.

The governing load case for HL-93 loading is taken as the maximum of:

- The truck and lane load, HS-20 truck with axle weights of 8 kips, 32 kips, 32 kips with a fixed first spacing of 14 ft, and variable second spacing between 14 and 30 ft , and lane load of 0.64 kip/ft
- The tandem and lane load, tandem load with axle weights of 25 kips spaced 4 ft apart, and lane load of 0.64 kip/ft
- The alternate load for negative moment - 90% of two HS-20 trucks with fixed second spacing of 14 ft spaced 50 ft apart combined with 90% of a design lane load of 0.64 kip/ft

There is a single load case for the AASHTO LRFD Fatigue Truck loading comprised of the design truck HS-20 with a fixed spacing of 30 ft between the 32-kip axles.

The dynamic impact for HS-20 loading is taken as 50/(125+span length) with an upper limit of 0.30. In the case of HL-93 loading, the dynamic load allowance is 0.33 of the axle loads. For AASHTO LRFD Fatigue Truck the dynamic load allowance is 0.15. Maximum live load effects for HS-20 and HL-93 with dynamic impact are presented in Table 4. The maximum load cases considered in the analysis are referred as:

- M – moment at midspan for simply supported beam model
- S – end shear for simply supported beam model
- M+ – positive moment for two-equal continuous beam model
- M- – negative moment at the interior support for two-equal continuous beam model
- Sc – shear at the interior support for two-equal continuous beam model

The maximum load effects were computed at the critical location mentioned above. The load cases with the lane load were maximized based on the equations provided in the AISC (American Institute of Steel Construction) Moment, Shear, and Reactions for Continuous Highway Bridges. Patch lane load patterns, where lane load is not applied to the entire span so that a particular load effect is maximized, were not considered in the calculations, as they generally yield results within a 1% difference from those presented. The only notable difference was observed for the negative moment effect, which differed by up to 5%.

In the assessment of rating factors, the MBE (23 CFR 650.317) uses HL-93 loading for LRFR method, and HS-20 loading for LFR and ASR (Allowable Stress Rating) methods. In order to determine the impact of platoon loading on existing bridges, the platoon loading is compared to both HS-20 and HL-93 loading. The live load effects for these two live load models differ, as shown in Table 4, so the ratios between these two load models are shown in Table 5. Because the ratios vary significantly from case to case, it is necessary to keep this effect in mind when analyzing platoon live load ratios. Additionally, for the evaluation of the impact of the fatigue resistance the live load effects of the AASHTO LRFD Fatigue Truck are presented in Table 6.

Table 4. Maximum HS-20 and HL-93 load effects for selected load case scenarios (impact or dynamic load allowance included, per lane).

Span [ft]	HS-20 M [kip-ft]	HS-20 S [kip]	HS-20 M+ [kip-ft]	HS-20 M- [kip-ft]	HS-20 Sc [kip]	HL-93 M [kip-ft]	HL-93 S [kip]	HL-93 M+ [kip-ft]	HL-93 M- [kip-ft]	HL-93 Sc [kip]
80	1,443	79	1,167	-981	83	2,055	110	1,637	-1,701	120
100	1,858	80	1,506	-1,400	82	2,822	119	2,247	-2,311	130
120	2,264	80	1,840	-1,886	89	3,652	127	2,908	-2,970	139
140	2,663	84	2,168	-2,439	97	4,547	134	3,617	-3,686	148
160	3,254	91	2,528	-3,057	106	5,506	141	4,377	-4,422	156
180	3,960	97	3,071	-3,741	114	6,529	148	5,185	-5,150	164
200	4,731	104	3,664	-4,490	122	7,616	155	6,041	-5,890	173
220	5,567	110	4,305	-5,304	131	8,766	162	6,947	-6,648	181
240	6,467	117	4,996	-6,182	139	9,981	169	7,901	-7,633	189
260	7,432	123	5,735	-7,125	147	11,260	176	8,903	-8,723	196
280	8,462	130	6,524	-8,133	155	12,603	182	9,954	-9,859	206
300	9,556	136	7,361	-9,206	163	14,010	189	11,055	-11,056	214
320	10,714	143	8,248	-10,342	171	15,480	195	12,204	-12,305	222
340	11,937	149	9,183	-11,544	179	17,015	202	13,402	-13,605	230
360	13,224	156	10,167	-12,809	188	18,614	208	14,649	-14,962	238
380	14,575	162	11,200	-14,139	196	20,277	215	15,944	-16,378	246
400	15,990	169	12,282	-15,533	204	22,004	222	17,288	-17,847	254
420	17,470	175	13,413	-16,991	212	23,794	228	18,680	-19,373	262
440	19,014	182	14,592	-18,514	220	25,649	235	20,121	-20,954	270
460	20,622	188	15,820	-20,100	228	27,568	241	21,611	-22,591	278
480	22,294	194	17,097	-21,751	236	29,551	247	23,149	-24,286	286
500	24,030	201	18,422	-23,466	244	31,598	254	24,736	-26,039	294

Table 5. Ratio of maximum load effects of HL-93 to HS-20 loading (impact or dynamic load allowance included).

Span [ft]	M [-]	S [-]	M+ [-]	M- [-]	Sc [-]
80	1.42	1.39	1.40	1.73	1.46
100	1.52	1.49	1.49	1.65	1.57
120	1.61	1.58	1.58	1.57	1.56
140	1.71	1.59	1.67	1.51	1.52
160	1.69	1.56	1.73	1.45	1.48
180	1.65	1.53	1.69	1.38	1.44
200	1.61	1.50	1.65	1.31	1.41
220	1.57	1.47	1.61	1.25	1.38
240	1.54	1.44	1.58	1.23	1.36
260	1.52	1.42	1.55	1.22	1.34
280	1.49	1.40	1.53	1.21	1.33
300	1.47	1.38	1.50	1.20	1.31
320	1.44	1.37	1.48	1.19	1.30
340	1.43	1.35	1.46	1.18	1.28
360	1.41	1.34	1.44	1.17	1.27
380	1.39	1.33	1.42	1.16	1.26
400	1.38	1.31	1.41	1.15	1.25
420	1.36	1.30	1.39	1.14	1.24
440	1.35	1.29	1.38	1.13	1.23
460	1.34	1.28	1.37	1.12	1.22
480	1.33	1.27	1.35	1.12	1.21
500	1.31	1.26	1.34	1.11	1.21

Table 6. Maximum AASHTO LRFD Fatigue Truck load effects for selected load case scenarios (dynamic load allowance included, per lane).

Span [ft]	M [kip-ft]	S [kip]	M+ [kip-ft]	M- [kip-ft]	Sc [kip]
80	1,040	56	866	514	68
100	1,454	59	1,191	696	72
120	1,868	61	1,523	872	74
140	2,282	63	1,858	1,043	74
160	2,696	65	2,195	1,210	76
180	3,110	67	2,534	1,376	77
200	3,524	68	2,872	1,542	78
220	3,938	69	3,212	1,705	79
240	4,352	70	3,552	1,864	78
260	4,766	70	3,892	2,027	78
280	5,180	71	4,232	2,191	77
300	5,594	72	4,572	2,354	80
320	6,008	72	4,914	2,515	80
340	6,422	72	5,255	2,675	80
360	6,836	73	5,596	2,836	80
380	7,250	73	5,938	2,996	80
400	7,664	73	6,279	3,157	79
420	8,078	74	6,620	3,317	79
440	8,492	74	6,962	3,478	81
460	8,906	74	7,303	3,638	81
480	9,320	74	7,644	3,799	81
500	9,734	74	7,985	3,959	81

2.3.3 Influence Line Analysis for Platoon Live Load

A sample of maximum shear and moments for Type 3S2 platoons are shown in Table 7 and Table 8. Load effects for all platoon cases are not shown for brevity.

Table 7. Maximum simple span moment effects of Type 3S2 platoon trucks, per lane, no dynamic load allowance. Values provided in kip-ft.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	962	962	962	962	962	962	962	962	962	962
100	1,322	1,389	1,322	1,322	1,389	1,322	1,322	1,389	1,322	1,322
120	1,682	1,904	1,682	1,682	1,904	1,682	1,682	1,904	1,682	1,682
140	2,042	2,492	2,109	2,042	2,492	2,109	2,042	2,492	2,109	2,042
160	2,402	3,204	2,624	2,402	3,313	2,624	2,402	3,313	2,624	2,402
180	2,762	3,924	3,212	2,829	4,228	3,212	2,829	4,228	3,212	2,829
200	3,122	4,644	3,924	3,344	5,210	4,033	3,344	5,210	4,033	3,344
220	3,482	5,364	4,644	3,932	6,290	4,948	3,932	6,290	4,948	3,932
240	3,842	6,084	5,364	4,644	7,370	5,930	4,753	7,421	5,930	4,753
260	4,202	6,804	6,084	5,364	8,450	7,010	5,668	8,656	7,010	5,668
280	4,562	7,524	6,804	6,084	9,530	8,090	6,650	9,952	8,090	6,650
300	4,922	8,244	7,524	6,804	10,610	9,170	7,730	11,376	9,170	7,730
320	5,282	8,964	8,244	7,524	11,690	10,250	8,810	12,816	10,301	8,810
340	5,642	9,684	8,964	8,244	12,770	11,330	9,890	14,256	11,536	9,890
360	6,002	10,404	9,684	8,964	13,850	12,410	10,970	15,696	12,832	10,970
380	6,362	11,124	10,404	9,684	14,930	13,490	12,050	17,136	14,256	12,050
400	6,722	11,844	11,124	10,404	16,010	14,570	13,130	18,576	15,696	13,181
420	7,082	12,564	11,844	11,124	17,090	15,650	14,210	20,016	17,136	14,416
440	7,442	13,284	12,564	11,844	18,170	16,730	15,290	21,456	18,576	15,712
460	7,802	14,004	13,284	12,564	19,250	17,810	16,370	22,896	20,016	17,136
480	8,162	14,724	14,004	13,284	20,330	18,890	17,450	24,336	21,456	18,576
500	8,522	15,444	14,724	14,004	21,410	19,970	18,530	25,776	22,896	20,016

Table 8. Maximum simple span shear effects of Type 3S2 platoon trucks, per lane, no dynamic load allowance. Values provided in kip.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	55	58	55	55	58	55	55	58	55	55
100	59	67	61	59	67	61	59	67	61	59
120	61	79	68	63	79	68	63	79	68	63
140	62	88	78	69	88	78	69	88	78	69
160	64	95	86	77	98	86	77	98	86	77
180	65	101	93	85	109	93	85	109	93	85
200	65	105	98	91	119	100	91	119	100	91
220	66	109	102	95	128	109	95	129	109	95
240	66	112	106	100	135	117	102	139	117	102
260	67	114	108	103	142	125	108	149	125	108
280	67	116	111	106	147	131	116	159	132	116
300	68	118	113	108	151	137	123	168	140	123
320	68	120	115	111	156	142	129	175	148	129
340	68	121	117	113	159	146	134	182	157	134
360	68	122	118	114	162	150	138	188	164	140
380	68	123	120	116	165	154	142	193	170	148
400	69	125	121	117	168	157	146	198	176	155
420	69	125	122	119	170	160	149	202	182	161
440	69	126	123	120	172	162	152	206	186	167
460	69	127	124	121	174	165	155	210	191	172
480	69	128	125	122	176	167	158	213	195	177
500	69	128	126	123	177	169	160	216	199	181

2.3.4 Live Load Ratios

Based on the influence line analysis for platoon and design loadings described previously, the live load ratios are computed. These ratios are analyzed to determine critical platoon configurations, and corresponding bridge span lengths, considered to be sensitive to platoon loading. The calculated live load ratios consist of the maximum platoon load effect in the numerator, and AASHTO design load in the denominator.

Equation 1 presents the ratio of the maximum platoon loading to HL-93 loading with the dynamic load allowance.

$$Ratio_{HL-93} = \frac{(1+IM)\,LL_{platoon}}{(1+IM)\,LL_{HL-93-truck} + LL_{HL-93-lane}} \qquad \text{Eq.1}$$

where:

- IM — dynamic allowance of 33%
- $LL_{platoon}$ — maximum load effect due to platoon
- $LL_{HL-93-truck}$ — maximum load effect due to HL-93 design truck
- $LL_{HL-93-lane}$ — maximum load effect due to HL-93 design lane load.

In Equation 2, the ratio of the maximum platoon loading to HS-20 loading is presented, and since the impact factor is the same for both the platoon and HS-20 loading, it does not appear in this ratio.

$$Ratio_{HS-20} = \frac{LL_{platoon}}{LL_{HS-20}} \qquad \text{Eq.2}$$

where:

- LL_{HS-20} — maximum load effect due to HS-20 loading (truck, lane with point load, or military load).

Similarly, in Equation 3, the ratio of the maximum platoon loading to AASHTO LRFD Fatigue Truck loading is presented, and since the impact factor is the same for both the platoon and AASHTO LRFD Fatigue Truck loading, it does not appear in this ratio.

$$Ratio_{Fatigue\ Truck} = \frac{LL_{platoon}}{LL_{Fatigue\ Truck}} \qquad \text{Eq.3}$$

where:

- $LL_{Fatigue\ Truck}$ – maximum load effect due to AASHTO LRFD Fatigue Truck loading

The ratios calculated from Eq. 1 and Eq.2 can be used along with bridge rating data to determine the number of bridges impacted by platoon loading, by capturing the bridges with a rating factor below 1.0. The bridge rating factor for platoon loading can be calculated from Equation 4, assuming the live load factor for platoons and HL-93 is the same. This assumption is valid for service limit state.

$$RF_{platoon} = \frac{RF_{operating\ (HL-93)}}{Ratio_{HL-93}} \qquad \text{Eq.4}$$

where:

- $RF_{platoon}$ – bridge rating factor for platoon loading
- $RF_{operating(HL-93)}$ – MBE (23 CFR 650.317) bridge rating factor for operating level and HL-93 loading.
- $Ratio_{HL-93}$ – ratio of platoon to HL-93 loading presented in Eq. 1

The live load ratios were computed for all considered cases and assumptions described above.

A subset of live load ratios for Type 3S2 truck platoons, compared to HS-20 loading for a simple span moment and shear, are presented in Table 9 and Table 10. Similarly, platoon ratios compared to HL-93 are presented in Table 11 and Table 12. Additionally, platoon ratios compared to AASHTO LRFD Fatigue Truck loading are presented in Table 13 and Table 14. All live load ratios obtained from this analysis are not provided for brevity.

Table 9. Load ratios of Type 3S2 platoon trucks to HS-20 for simple span moments.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
100	0.87	0.91	0.87	0.87	0.91	0.87	0.87	0.91	0.87	0.87
120	0.89	1.01	0.89	0.89	1.01	0.89	0.89	1.01	0.89	0.89
140	0.91	1.11	0.94	0.91	1.11	0.94	0.91	1.11	0.94	0.91
160	0.87	1.16	0.95	0.87	1.20	0.95	0.87	1.20	0.95	0.87
180	0.81	1.15	0.94	0.83	1.24	0.94	0.83	1.24	0.94	0.83
200	0.76	1.13	0.96	0.82	1.27	0.98	0.82	1.27	0.98	0.82
220	0.72	1.10	0.96	0.81	1.29	1.02	0.81	1.29	1.02	0.81
240	0.68	1.07	0.94	0.82	1.30	1.04	0.84	1.30	1.04	0.84
260	0.64	1.03	0.92	0.82	1.28	1.07	0.86	1.32	1.07	0.86
280	0.61	1.00	0.90	0.81	1.27	1.07	0.88	1.32	1.07	0.88
300	0.58	0.96	0.88	0.80	1.24	1.07	0.90	1.33	1.07	0.90
320	0.55	0.93	0.86	0.78	1.21	1.06	0.91	1.33	1.07	0.91
340	0.52	0.90	0.83	0.76	1.18	1.05	0.92	1.32	1.07	0.92
360	0.50	0.87	0.81	0.75	1.16	1.04	0.92	1.31	1.07	0.92
380	0.48	0.84	0.78	0.73	1.13	1.02	0.91	1.29	1.07	0.91
400	0.46	0.81	0.76	0.71	1.10	1.00	0.90	1.27	1.08	0.90
420	0.44	0.79	0.74	0.70	1.07	0.98	0.89	1.25	1.07	0.90
440	0.43	0.76	0.72	0.68	1.04	0.96	0.88	1.23	1.06	0.90
460	0.41	0.74	0.70	0.66	1.01	0.94	0.86	1.21	1.05	0.90
480	0.40	0.72	0.68	0.65	0.99	0.92	0.85	1.18	1.04	0.90
500	0.38	0.69	0.66	0.63	0.96	0.90	0.83	1.16	1.03	0.90

Table 10. Load ratios of Type 3S2 platoon trucks to HS-20 for simple span shear.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	0.87	0.91	0.87	0.87	0.91	0.87	0.87	0.91	0.87	0.87
100	0.90	1.03	0.93	0.90	1.03	0.93	0.90	1.03	0.93	0.90
120	0.92	1.19	1.03	0.94	1.19	1.03	0.94	1.19	1.03	0.94
140	0.88	1.25	1.10	0.97	1.25	1.10	0.97	1.25	1.10	0.97
160	0.82	1.23	1.12	1.00	1.27	1.12	1.00	1.27	1.12	1.00
180	0.77	1.20	1.11	1.01	1.30	1.11	1.01	1.30	1.11	1.01
200	0.73	1.17	1.09	1.01	1.32	1.11	1.01	1.32	1.11	1.01
220	0.68	1.13	1.06	0.99	1.33	1.13	0.99	1.34	1.13	0.99
240	0.65	1.08	1.03	0.97	1.32	1.14	0.99	1.35	1.14	0.99
260	0.61	1.04	0.99	0.94	1.30	1.14	0.99	1.37	1.14	0.99
280	0.58	1.00	0.96	0.92	1.27	1.14	1.00	1.38	1.14	1.00
300	0.55	0.97	0.93	0.89	1.24	1.12	1.01	1.38	1.15	1.01
320	0.53	0.93	0.90	0.86	1.21	1.11	1.00	1.37	1.16	1.00
340	0.50	0.90	0.87	0.84	1.18	1.09	0.99	1.35	1.16	0.99
360	0.48	0.87	0.84	0.81	1.15	1.06	0.98	1.33	1.16	0.99
380	0.46	0.84	0.81	0.79	1.12	1.04	0.96	1.31	1.15	1.00
400	0.45	0.81	0.79	0.76	1.09	1.02	0.95	1.29	1.14	1.00
420	0.43	0.78	0.76	0.74	1.06	1.00	0.93	1.26	1.13	1.00
440	0.41	0.76	0.74	0.72	1.03	0.97	0.91	1.24	1.12	1.00
460	0.40	0.73	0.72	0.70	1.00	0.95	0.90	1.21	1.10	0.99
480	0.39	0.71	0.69	0.68	0.98	0.93	0.88	1.19	1.09	0.99
500	0.37	0.69	0.67	0.66	0.95	0.91	0.86	1.16	1.07	0.98

Table 11. Load ratios of Type 3S2 platoon trucks to HL-93 for simple span moments.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62
100	0.62	0.65	0.62	0.62	0.65	0.62	0.62	0.65	0.62	0.62
120	0.61	0.69	0.61	0.61	0.69	0.61	0.61	0.69	0.61	0.61
140	0.60	0.73	0.62	0.60	0.73	0.62	0.60	0.73	0.62	0.60
160	0.58	0.77	0.63	0.58	0.80	0.63	0.58	0.80	0.63	0.58
180	0.56	0.80	0.65	0.58	0.86	0.65	0.58	0.86	0.65	0.58
200	0.55	0.81	0.69	0.58	0.91	0.70	0.58	0.91	0.70	0.58
220	0.53	0.81	0.70	0.60	0.95	0.75	0.60	0.95	0.75	0.60
240	0.51	0.81	0.71	0.62	0.98	0.79	0.63	0.99	0.79	0.63
260	0.50	0.80	0.72	0.63	1.00	0.83	0.67	1.02	0.83	0.67
280	0.48	0.79	0.72	0.64	1.01	0.85	0.70	1.05	0.85	0.70
300	0.47	0.78	0.71	0.65	1.01	0.87	0.73	1.08	0.87	0.73
320	0.45	0.77	0.71	0.65	1.00	0.88	0.76	1.10	0.89	0.76
340	0.44	0.76	0.70	0.64	1.00	0.89	0.77	1.11	0.90	0.77
360	0.43	0.74	0.69	0.64	0.99	0.89	0.78	1.12	0.92	0.78
380	0.42	0.73	0.68	0.64	0.98	0.88	0.79	1.12	0.94	0.79
400	0.41	0.72	0.67	0.63	0.97	0.88	0.79	1.12	0.95	0.80
420	0.40	0.70	0.66	0.62	0.96	0.87	0.79	1.12	0.96	0.81
440	0.39	0.69	0.65	0.61	0.94	0.87	0.79	1.11	0.96	0.81
460	0.38	0.68	0.64	0.61	0.93	0.86	0.79	1.10	0.97	0.83
480	0.37	0.66	0.63	0.60	0.91	0.85	0.79	1.10	0.97	0.84
500	0.36	0.65	0.62	0.59	0.90	0.84	0.78	1.08	0.96	0.84

Table 12. Load ratios of Type 3S2 platoon trucks to HL-93 for simple span shear.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	0.67	0.70	0.67	0.67	0.70	0.67	0.67	0.70	0.67	0.67
100	0.66	0.75	0.68	0.66	0.75	0.68	0.66	0.75	0.68	0.66
120	0.64	0.83	0.72	0.66	0.83	0.72	0.66	0.83	0.72	0.66
140	0.62	0.88	0.77	0.68	0.88	0.77	0.68	0.88	0.77	0.68
160	0.60	0.90	0.81	0.73	0.93	0.81	0.73	0.93	0.81	0.73
180	0.58	0.90	0.83	0.76	0.97	0.83	0.76	0.97	0.83	0.76
200	0.56	0.90	0.84	0.78	1.02	0.86	0.78	1.02	0.86	0.78
220	0.54	0.89	0.84	0.78	1.05	0.89	0.78	1.06	0.89	0.78
240	0.52	0.88	0.83	0.78	1.07	0.92	0.80	1.09	0.92	0.80
260	0.51	0.86	0.82	0.78	1.07	0.95	0.82	1.13	0.95	0.82
280	0.49	0.85	0.81	0.77	1.07	0.96	0.85	1.16	0.96	0.85
300	0.48	0.83	0.80	0.76	1.07	0.97	0.86	1.18	0.98	0.86
320	0.46	0.81	0.78	0.75	1.06	0.97	0.87	1.19	1.01	0.87
340	0.45	0.80	0.77	0.74	1.05	0.96	0.88	1.20	1.03	0.88
360	0.44	0.78	0.76	0.73	1.03	0.96	0.88	1.20	1.05	0.90
380	0.42	0.76	0.74	0.72	1.02	0.95	0.88	1.19	1.05	0.91
400	0.41	0.75	0.73	0.70	1.01	0.94	0.88	1.19	1.06	0.93
420	0.40	0.73	0.71	0.69	0.99	0.93	0.87	1.18	1.06	0.94
440	0.39	0.72	0.70	0.68	0.98	0.92	0.86	1.17	1.06	0.95
460	0.38	0.70	0.68	0.67	0.96	0.91	0.86	1.16	1.05	0.95
480	0.37	0.69	0.67	0.65	0.94	0.90	0.85	1.14	1.05	0.95
500	0.36	0.67	0.66	0.64	0.93	0.88	0.84	1.13	1.04	0.95

Table 13. Load ratios of Type 3S2 platoon trucks to AASHTO LRFD Fatigue Truck for simple span moments.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06
100	1.05	1.10	1.05	1.05	1.10	1.05	1.05	1.10	1.05	1.05
120	1.04	1.17	1.04	1.04	1.17	1.04	1.04	1.17	1.04	1.04
140	1.03	1.26	1.06	1.03	1.26	1.06	1.03	1.26	1.06	1.03
160	1.02	1.37	1.12	1.02	1.41	1.12	1.02	1.41	1.12	1.02
180	1.02	1.45	1.19	1.05	1.56	1.19	1.05	1.56	1.19	1.05
200	1.02	1.52	1.28	1.09	1.70	1.32	1.09	1.70	1.32	1.09
220	1.02	1.57	1.36	1.15	1.84	1.44	1.15	1.84	1.44	1.15
240	1.02	1.61	1.42	1.23	1.95	1.57	1.26	1.96	1.57	1.26
260	1.01	1.64	1.47	1.29	2.04	1.69	1.37	2.09	1.69	1.37
280	1.01	1.67	1.51	1.35	2.12	1.80	1.48	2.21	1.80	1.48
300	1.01	1.69	1.55	1.40	2.18	1.89	1.59	2.34	1.89	1.59
320	1.01	1.72	1.58	1.44	2.24	1.96	1.69	2.45	1.97	1.69
340	1.01	1.73	1.61	1.48	2.29	2.03	1.77	2.55	2.07	1.77
360	1.01	1.75	1.63	1.51	2.33	2.09	1.85	2.64	2.16	1.85
380	1.01	1.76	1.65	1.54	2.37	2.14	1.91	2.72	2.26	1.91
400	1.01	1.78	1.67	1.56	2.40	2.19	1.97	2.79	2.36	1.98
420	1.01	1.79	1.69	1.58	2.43	2.23	2.02	2.85	2.44	2.05
440	1.01	1.80	1.70	1.60	2.46	2.27	2.07	2.91	2.52	2.13
460	1.01	1.81	1.72	1.62	2.49	2.30	2.11	2.96	2.58	2.21
480	1.01	1.82	1.73	1.64	2.51	2.33	2.15	3.00	2.65	2.29
500	1.01	1.82	1.74	1.65	2.53	2.36	2.19	3.05	2.71	2.36

Table 14. Load ratios of Type 3S2 platoon trucks to AASHTO LRFD Fatigue Truck for simple span shears.

Span Length [ft]	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
80	1.13	1.19	1.13	1.13	1.19	1.13	1.13	1.19	1.13	1.13
100	1.14	1.32	1.19	1.14	1.32	1.19	1.14	1.32	1.19	1.14
120	1.15	1.49	1.29	1.18	1.49	1.29	1.18	1.49	1.29	1.18
140	1.13	1.60	1.42	1.25	1.60	1.42	1.25	1.60	1.42	1.25
160	1.12	1.68	1.52	1.36	1.73	1.52	1.36	1.73	1.52	1.36
180	1.11	1.73	1.60	1.46	1.87	1.60	1.46	1.87	1.60	1.46
200	1.10	1.78	1.65	1.53	2.02	1.70	1.53	2.02	1.70	1.53
220	1.10	1.81	1.70	1.59	2.13	1.81	1.59	2.15	1.81	1.59
240	1.09	1.84	1.74	1.64	2.23	1.93	1.67	2.29	1.93	1.67
260	1.09	1.86	1.77	1.68	2.31	2.04	1.77	2.44	2.04	1.77
280	1.09	1.88	1.80	1.71	2.38	2.13	1.88	2.58	2.14	1.88
300	1.09	1.90	1.82	1.74	2.43	2.20	1.97	2.70	2.25	1.97
320	1.08	1.91	1.84	1.77	2.48	2.27	2.05	2.80	2.37	2.05
340	1.08	1.92	1.86	1.79	2.53	2.33	2.12	2.89	2.49	2.13
360	1.08	1.93	1.87	1.81	2.57	2.38	2.19	2.97	2.59	2.22
380	1.08	1.94	1.88	1.82	2.60	2.42	2.24	3.04	2.68	2.33
400	1.08	1.95	1.90	1.84	2.63	2.46	2.29	3.10	2.77	2.43
420	1.08	1.96	1.91	1.85	2.66	2.49	2.33	3.16	2.84	2.52
440	1.07	1.97	1.92	1.87	2.68	2.53	2.37	3.21	2.91	2.60
460	1.07	1.97	1.93	1.88	2.70	2.56	2.41	3.26	2.97	2.67
480	1.07	1.98	1.93	1.89	2.72	2.58	2.44	3.30	3.02	2.74
500	1.07	1.99	1.94	1.90	2.74	2.61	2.47	3.34	3.07	2.8

2.3.5 Bridge Span Lengths Impacted by Platoons

The live load ratios developed using the methodology described in the previous section were used to determine which bridge span lengths are impacted by possible truck platoon loading scenarios. Table 15, Table 16, and Table 17 present bridge spans for which live load ratios were found to be greater than 1.0 for HS-20 and HL-93 loading, and 1.5 for AASHTO LRFD Fatigue Truck loading. The range of spans impacted by all platoon cases is shown in Table 18.

A live load ratio larger than 1.0 does not mean that the bridge requires posting. It indicates that the maximum live load effect of the platoon exceeds the AASHTO design loading at the operating or service stress level. So, the computed live load ratios need to be compared with the representative bridge rating data to find the number of bridges that may be affected by the platoon loading.

A different live load ratio was used in the comparison against AASHTO LRFD Fatigue Truck loading because it is estimated that most of the bridges designed for fatigue had 1.5 times the load effect produced by the AASHTO LRFD Fatigue Truck as the design target. This is because the load demands utilized in fatigue design throughout the different provisions issued by AASHTO are similar until 2016 when the load factors increased to 1.75 for Fatigue I and 0.8 for Fatigue II. This is discussed further in Section 2.5.7.

Platoon cases with 5-axle and 6-axle trucks created from Type 3S2, Type 3-3, and WIM vehicles provide consistent results in terms of impacted bridge span lengths. The live load ratio analysis did not find a governing truck configuration.

The computed live load ratios can be used to identify critical platoon cases that need to be considered in synthetic WIM data analysis. Platoon configurations with 3 and 4 trucks spaced 30 ft apart, and 4 truck platoons spaced 50 ft apart are impacting a large portion of the bridge span lengths selected in this study and are further investigated in Chapter 3 and 4 analyses.

Moreover, it was found that platoon loading compared to HS-20 loading affects a broad spectrum of spans as opposed to platoons compared to HL-93 loading. Thus, the HS-20 loading provides an inconsistent live load envelope in comparison to HL-93 loading, and for that reason, platoon live load is only represented by HL-93 loading for the calibration conducted in Chapter 4.

Table 15. Bridge span lengths impacted by Type 3S2 platoon trucks. Span lengths provided in ft.

Scenario	Load Cases	3S2-1-00	3S2-2-30	3S2-2-50	3S2-2-70	3S2-3-30	3S2-3-50	3S2-3-70	3S2-4-30	3S2-4-50	3S2-4-70
Platoon/HS-20 Load Effect is Greater Than 1.0	M	0-0	120-260	0-0	0-0	120-460	220-380	0-0	120-500	220-500	0-0
	S	0-0	100-280	120-240	160-200	100-460	120-400	160-320	100-500	120-500	160-440
	M+	0-0	140-300	180-200	0-0	140-480	180-400	0-0	140-500	180-500	0-0
	M-	0-0	80-100	80-100	100-100	80-140	80-100	100-100	80-220	80-220	100-100
	Sc	0-0	0-0	0-0	0-0	200-220	0-0	0-0	200-340	0-0	0-0
Platoon/HL-93 Load Effect is Greater Than 1.0	M	0-0	0-0	0-0	0-0	280-320	0-0	0-0	260-500	0-0	0-0
	S	0-0	0-0	0-0	0-0	200-400	0-0	0-0	200-500	320-500	0-0
	M+	0-0	0-0	0-0	0-0	0-0	0-0	0-0	280-500	0-0	0-0
	M-	0-0	0-0	0-0	0-0	0-0	0-0	0-0	180-180	0-0	0-0
	Sc	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0
Platoon/Fatigue Truck Load Effect is Greater Than 1.5	M	0-0	200-500	280-500	360-500	180-500	240-500	300-500	180-500	240-500	300-500
	S	0-0	140-500	160-500	200-500	140-500	160-500	200-500	140-500	160-500	200-500
	M+	0-0	220-500	300-500	380-500	200-500	260-500	320-500	200-500	260-500	320-500
	M-	0-0	80-500	80-500	80-500	80-500	80-500	80-500	80-500	80-500	80-500
	Sc	0-0	240-500	280-500	340-500	200-500	240-500	280-500	200-500	240-500	280-500

Table 16. Bridge span lengths impacted by WIM platoon trucks. Span lengths provided in ft.

Scenario	Load Cases	WIM-1-00	WIM-2-30	WIM-2-50	WIM-2-70	WIM-3-30	WIM-3-50	WIM-3-70	WIM-4-30	WIM-4-50	WIM-4-70
Platoon/HS-20 Load Effect is Greater Than 1.0	M	0-0	120-300	0-0	0-0	120-500	220-440	0-0	120-500	220-500	0-0
	S	0-0	100-300	120-280	180-240	100-500	120-460	180-380	100-500	120-500	180-500
	M+	0-0	140-320	160-260	0-0	140-500	160-460	0-0	140-500	160-500	0-0
	M-	0-0	80-100	80-120	100-120	80-180	80-120	100-120	80-240	80-240	100-120
	Sc	0-0	0-0	0-0	0-0	0-0	0-0	0-0	280-340	0-0	0-0
Platoon/HL-93 Load Effect is Greater Than 1.0	M	0-0	0-0	0-0	0-0	280-420	0-0	0-0	260-500	480-500	0-0
	S	0-0	0-0	0-0	0-0	220-480	0-0	0-0	220-500	320-500	0-0
	M+	0-0	0-0	0-0	0-0	300-420	0-0	0-0	280-500	0-0	0-0
	M-	0-0	0-0	0-0	0-0	0-0	0-0	0-0	180-240	0-0	0-0
	Sc	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0
Platoon/Fatigue Truck Load Effect is Greater Than 1.5	M	0-0	200-500	260-500	320-500	180-500	240-500	300-500	180-500	240-500	300-500
	S	0-0	140-500	160-500	200-500	140-500	160-500	200-500	140-500	160-500	200-500
	M+	0-0	200-500	260-500	340-500	200-500	260-500	320-500	200-500	260-500	320-500
	M-	0-0	80-500	80-500	100-500	80-500	80-500	100-500	80-500	80-500	100-500
	Sc	0-0	220-500	260-500	280-500	200-500	240-500	280-500	200-500	240-500	280-500

Table 17. Bridge span lengths impacted by Type 3-3 platoon trucks. Span lengths provided in ft.

Scenario	Load Cases	33-1-00	33-2-30	33-2-50	33-2-70	33-3-30	33-3-50	33-3-70	33-4-30	33-4-50	33-4-70
Platoon/HS-20 Load Effect is Greater Than 1.0	M	0-0	140-300	0-0	0-0	140-500	240-440	0-0	140-500	240-500	0-0
	S	0-0	120-300	120-280	180-240	120-500	120-460	180-360	120-500	120-500	180-500
	M+	0-0	140-320	200-240	0-0	140-500	200-440	0-0	140-500	200-500	0-0
	M-	0-0	80-100	80-120	100-120	80-160	80-120	100-120	80-240	80-240	100-120
	Sc	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0
Platoon/HL-93 Load Effect is Greater Than 1.0	M	0-0	0-0	0-0	0-0	280-420	0-0	0-0	280-500	0-0	0-0
	S	0-0	0-0	0-0	0-0	220-480	0-0	0-0	220-500	340-500	0-0
	M+	0-0	0-0	0-0	0-0	340-380	0-0	0-0	300-500	0-0	0-0
	M-	0-0	0-0	0-0	0-0	0-0	0-0	0-0	180-240	0-0	0-0
	Sc	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0	0-0
Platoon/Fatigue Truck Load Effect is Greater Than 1.5	M	0-0	200-500	260-500	320-500	200-500	240-500	300-500	200-500	240-500	300-500
	S	0-0	140-500	180-500	200-500	140-500	180-500	200-500	140-500	180-500	200-500
	M+	0-0	220-500	280-500	340-500	220-500	260-500	320-500	220-500	260-500	320-500
	M-	0-0	80-500	80-500	100-500	80-500	80-500	100-500	80-500	80-500	100-500
	Sc	0-0	220-500	260-500	280-500	220-500	240-500	280-500	220-500	240-500	280-500

Table 18. Bridge span lengths impacted by Phase I platoon truck configurations. Span lengths provide in ft.

Scenario	Trucks	2-30	2-50	2-70	3-30	3-50	3-70	4-30	4-50	4-70
Platoon/HS-20 Load Effect is Greater than 1.0	Type 3S2	80-300	80-240	100-200	80-480	80-400	100-320	80-500	80-500	100-440
	WIM truck	80-320	80-280	100-240	80-500	80-460	100-380	80-500	80-500	100-500
	Type 3-3	80-320	80-280	100-240	80-500	80-460	100-360	80-500	80-500	100-500
Platoon/HL-93 Load Effect is Greater than 1.0	Type 3S2	0-0	0-0	0-0	200-400	0-0	0-0	180-500	320-500	0-0
	WIM truck	0-0	0-0	0-0	220-480	0-0	0-0	180-500	320-500	0-0
	Type 3-3	0-0	0-0	0-0	220-480	0-0	0-0	180-500	340-500	0-0
Platoon/Fatigue Truck Load Effect is Greater Than 1.5	Type 3S2	80-500	80-500	80-500	80-500	80-500	80-500	80-500	80-500	80-500
	WIM truck	80-500	80-500	100-500	80-500	80-500	100-500	80-500	80-500	100-500
	Type 3-3	80-500	80-500	100-500	80-500	80-500	100-500	80-500	80-500	100-500
Platoon/Fatigue Truck Load Effect is Greater Than 1.5 M & M+ Only	Type 3S2	200-500	280-500	360-500	180-500	240-500	300-500	180-500	240-500	300-500
	WIM truck	200-500	260-500	320-500	180-500	240-500	300-500	180-500	240-500	300-500
	Type 3-3	200-500	260-500	320-500	200-500	240-500	300-500	200-500	240-500	300-500

2.3.6 Summary of Span Length Study

Computed live load ratios provide a basis to determine the impact of the platoon loading on US bridges, and help select critical platoon cases for the calibration. The following observations regarding platoon live load ratios can be made:

- The key parameters that relate to how sensitive a bridge is to platoon loading include bridge span length, the number of trucks in the platoon, and headway spacing. Various platoon configurations based on these parameters were developed.
- Longer spans are affected by platoon loading, which creates load effects exceeding those produced by the HL-93 and HS 20 design loads. This analysis demonstrates that spans over 180 ft produce higher platoon live load ratios than current AASHTO design loads, for the critical platoon cases considered in this study.
- In general, shorter span lengths can only accommodate one truck on the bridge, and thus platoons will not affect the computed live load ratios.
- Platoon live load ratios for Type 3S2 (72 kips), Type 3-3 (80 kips), and WIM (80 kips) trucks provide consistent results in terms of impacted bridge span lengths. The live load ratio analysis did not distinguish between governing trucks. Therefore, the most common WIM 5-axle trucks are used to create the synthetic platoons in WIM analysis.
- Platoon loading compared to HS-20 loading affects a broad spectrum of bridges. The HS-20 loading provides lower load effects in comparison to HL-93 (see Table 5). It is known that HS-20 does not perform as well as HL-93 as a nominal live load in terms of uniformly encompassing highway traffic loads for differing span lengths, and therefore HL-93 loading is selected and utilized to represent possible platoon loading in the subsequent analysis.
- For the assumed conditions in which platoon loading has the same live load factor and dynamic impact as HL-93 loading, the bridge's span affected by platoons varies from 180 to 500 ft.
- Platoons of 2 trucks compared to HL-93 loading do not produce live load ratios greater than one. It shows that a 2-truck platoon load effect is enveloped by the HL-93 loading. Although, a 2-truck platoon will be still considered in the synthetic WIM analysis and calibration efforts for completeness of the analysis.
- Based on the influence line analysis the critical platoon configuration includes 3 truck platoons spaced 30 ft apart, and 4 truck platoons spaced 30 and 50 ft apart. These cases will be investigated in the calibration.

- The evaluation of bridge span lengths affected by platoon loads compared to AASHTO LRFD Fatigue Truck considered ratios exceeding 1.50. The 1.50 factor is chosen as it was the load factor prescribed by the LRFD Specs. for the evaluation of infinite fatigue life prior to 2017.

- While the single truck scenario does not exceed 1.50·fatigue loading for any case, all platoons exceed 1.50·fatigue loading among longer span lengths.

- Platoon configurations generate positive flexure live load ratios larger than 1.50 in span lengths measuring 200-500 ft.

- Platoons with 2, 3, and 4 trucks, and headway spacing of 30 ft. and 50 ft. will be considered in WIM data analysis (Chapter 3) to develop statistical parameters of potential platoon live load and for calibration (Chapter 4). Platoons spaced at 70 ft do not exceed the HL-93 design loading. Therefore, headway spacing of 70 ft was not considered in the analyses conducted in Chapter 3 and Chapter 4.

2.4 Truck Platooning Effects on Finite Fatigue Life

To assess the effect of truck platooning on the finite fatigue life of steel bridge girders, positive flexure generated by truck passage were calculated for simple spans and continuous spans. In this assessment, the selected base truck is the AASHTO LRFD Fatigue Truck as described in LRFD Spec. (23 CFR 625.4(d)(1)(v)) 3.6.1.4. The platoon configurations are composed of two, three, or four AASHTO LRFD Fatigue Trucks. Two platoon headway distances were considered, 30 feet and 70 ft. These headway distances were chosen as they are the minimum and maximum distances initially considered in the current study. The assessment was conducted for simple spans, and end spans and center spans for three equal continuous spans. The span lengths considered were 100 ft, 140 ft, 200 ft, 300 ft, 360 ft, and 500 ft. Since the dominant behavior affecting fatigue in beam-type bridges is positive flexure, the evaluated load effects were moments at the locations with the largest influence line ordinate for positive moment. As an example, Figure 12 shows a comparison between the moment due to the passage of a single truck and a platoon of 4 AASHTO LRFD Fatigue Trucks with a headway distance of 70 ft, at the center span of a continuous three span measuring 140 ft.

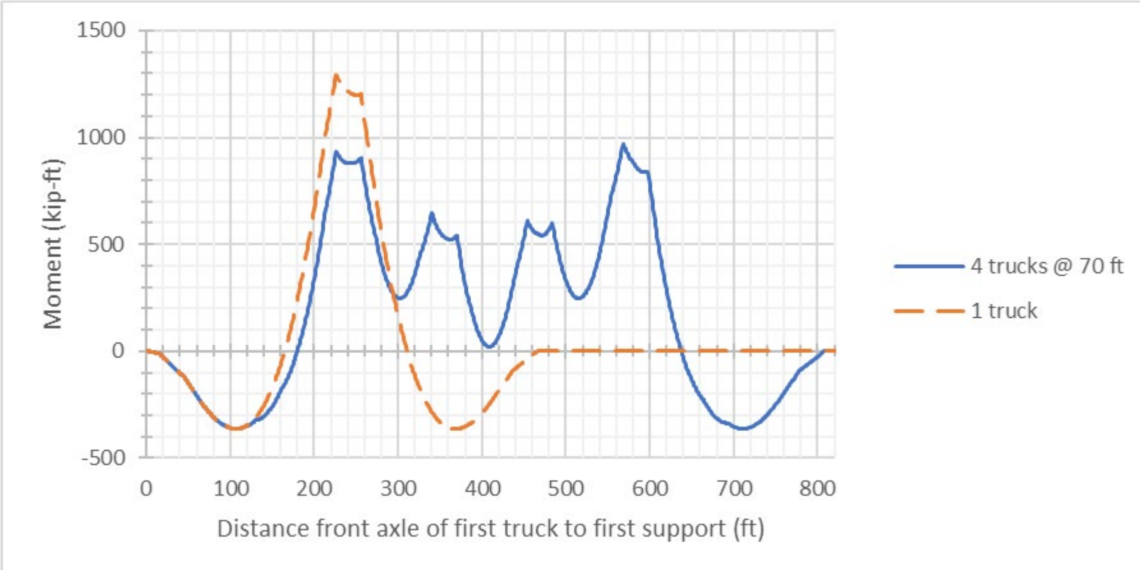

Figure 12. Positive moment at the midspan of center span, three equal continuous 140 ft spans.

These resulting moments were subjected to a rainflow-counting algorithm to calculate positive moment half-cycle ranges. The ratio between these half-cycle ranges and the maximum stress range were then combined using Miner's rule to calculate the number of effective stress cycles for each truck or truck platoon passage. These number of effective stress cycles per truck or truck platoon passage is equivalent to the variable n in LRFD Spec 6.6.1.2.5 and MBE (23 CFR 650.317) 7.2.5 and is computed as follows:

$$n = \sum_{i=1}^{k} \left(\frac{\Delta M_i}{\Delta M_{max}} \right)^3 \quad \text{Eq. 5}$$

where k is the number of tensile half-cycles during a single truck or truck platoon passage, ΔM_i is the moment range of the i-th tensile half-cycle, and ΔM_{max} is the maximum moment range, i.e. the difference between the maximum and minimum moments generated by the single truck or truck platoon.

After calculating the number of effective stress cycles per truck passage, the damage ratio of a platoon with respect to the base truck, $DR_{platoon}$, was computed as:

$$DR_{platoon} = \frac{\left(\Delta M_{platoon}\right)^3 n_{platoon}}{\left(\Delta M_{single\ truck}\right)^3 n_{single\ truck}} \quad \text{Eq. 6}$$

where $\Delta M_{platoon}$ and $\Delta M_{single\ truck}$ are the maximum moment ranges for the AASHTO LRFD Fatigue Truck platoon and single AASHTO LRFD Fatigue Truck, respectively, and $n_{platoon}$ and $n_{single\ truck}$ are the number of effective stress cycles per truck platoon passage and single truck passage, respectively.

To account for the assumption that platooning should not result in increased ADTT (Average Daily Truck Traffic), i.e., platoons are grouping of vehicles that all already part of the transportation network, a replacement damage ratio, $RDR_{platoon}$, was computed by factoring $DR_{platoon}$ by the number of trucks in a platoon, $N_{trucks\ per\ platoon}$, as follows:

$$RDR_{platoon} = \frac{DR_{platoon}}{N_{trucks\ per\ platoon}} \qquad \text{Eq. 7}$$

The first conclusion that can be obtained from the computation of the number of effective stress cycles per truck passage is that, when the maximum moment range of the truck platoon is larger than for a single truck, the number of effective stress cycles per truck platoon passage is between 1.00 and 1.03. For comparison, a single AASHTO LRFD Fatigue Truck results in 1.00 to 1.01 effective stress cycles per truck passage. Therefore, it does not appear that an adjustment to the number of effective cycles per truck is granted for truck platoons. Figure 13, Figure 14, and Figure 15 plot the computed number of effective stress cycle per truck platoon passage, $n_{platoon}$, against the maximum moment range ratio of the truck platoon with respect to a single truck, $\Delta M_{platoon}/\Delta M_{single\ truck}$, for simple spans, continuous interior spans, and continuous end spans, respectively. As previously discussed, once the moment range generated by the passage of the truck platoon exceeds that of the single truck, it appears reasonable to assume that the number of effective stress cycles is approximately equal to 1.0.

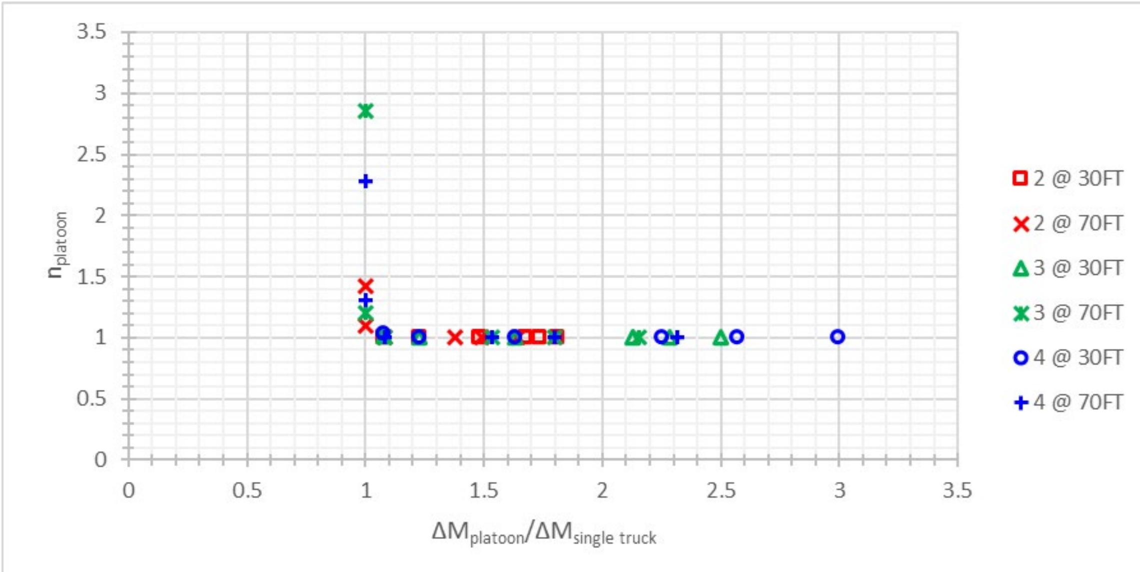

Figure 13. Number of effective stress cycles for platoon passage against maximum moment range ratio of the truck platoon with respect to a single truck, simple spans.

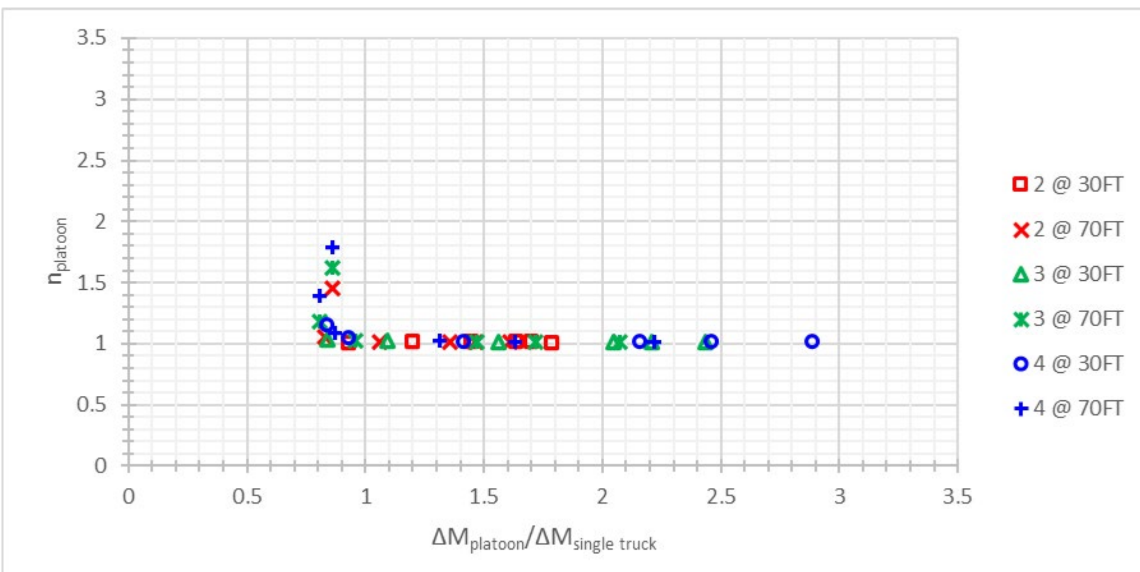

Figure 14. Number of effective stress cycles for platoon passage against maximum moment range ratio of the truck platoon with respect to a single truck, continuous interior spans.

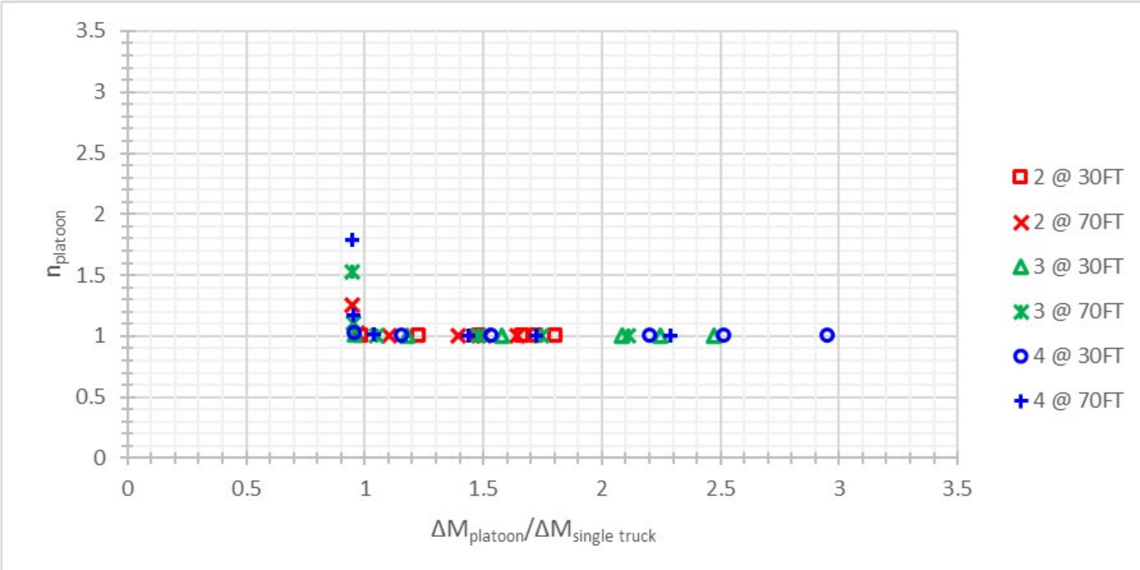

Figure 15. Number of effective stress cycles for platoon passage against maximum moment range ratio of the truck platoon with respect to a single truck, continuous end spans.

The second conclusion is that, in general, truck platooning is detrimental to the finite fatigue life of bridges, except for shorter spans. The following figures, Figure 16, Figure 17, and Figure 18, show the computed replacement damage ratio as a function of span length for simple spans, continuous interior spans, and continuous end spans, respectively. Different curves are displayed for the different platoon configurations considered. A replacement damage ratio equal or lesser than one means that platooning is not detrimental to the finite life fatigue performance of the bridge. For each of the truck platoon and span types considered, the minimum span length at which truck platooning may have a detrimental effect is summarized in Table 19.

For the truck platoon configurations considered, all bridges spans measuring 350’ or above will have detrimental effect from platooning; however, headway distance and the number of trucks per platoon have an important influence in degree of effect:

- When the headway distance is 30’, platooning is detrimental to the finite fatigue life of bridge with spans above 150’. Conversely, for a headway distance of 70’, this negative impact in fatigue life occurs for simple spans and continuous end spans above 250’, and continuous interior spans above 260’. Platooning is not shown to be detrimental to the finite fatigue life of spans shorter than these.
- Lesser number of trucks per platoon typically results in larger fatigue damage for shorter spans and lesser fatigue damage at longer spans. This effect is more evident for continuous spans.

- These curves are horizontally asymptotic. For a theoretical infinite span length, the maximum replacement damage ratio is 4, 9, and 16 for 2-truck, 3-truck, and 4-truck platoons, respectively.
- At very short spans, not depicted in the figures below, individual axles tend to dominate flexural behavior. Therefore, for very short spans the replacement damage ratio tends to 1.

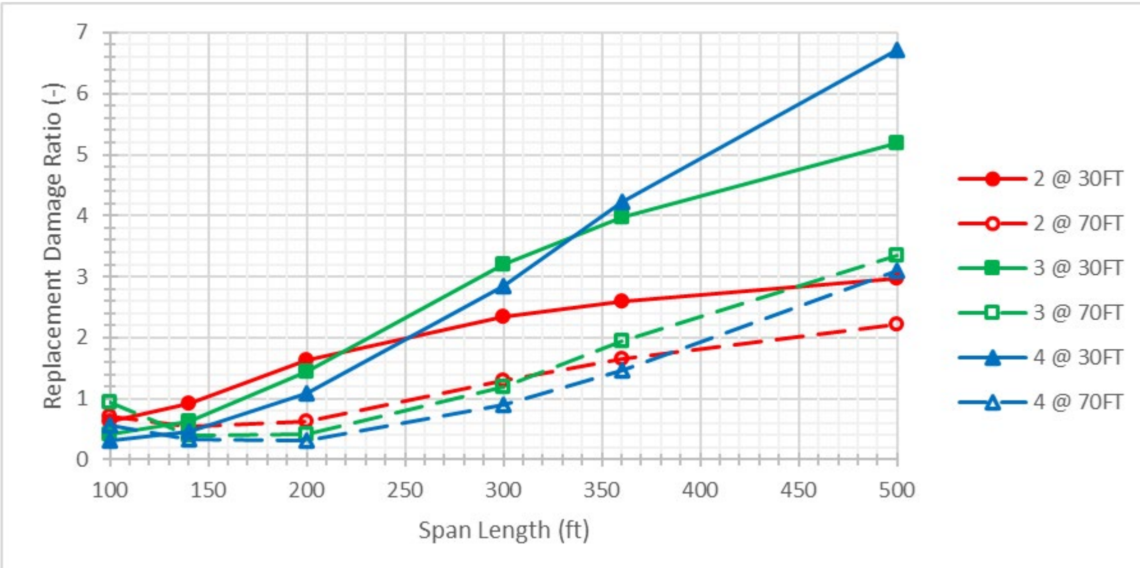

Figure 16. Replacement damage ratio for simple spans. Based on fatigue moment range at location of maximum positive moment.

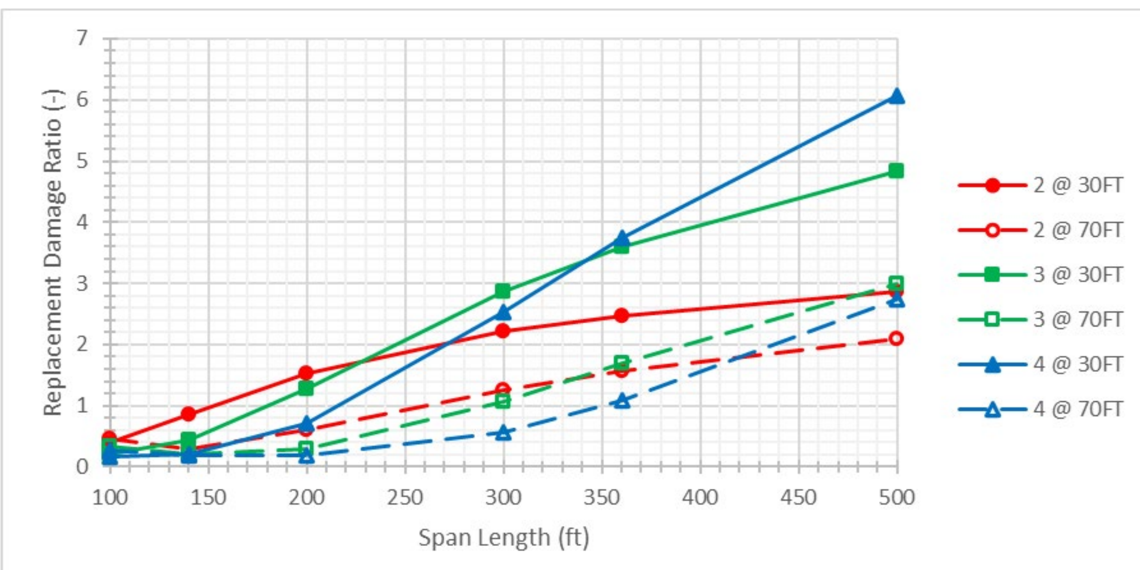

Figure 17. Replacement damage ratio for continuous interior spans. Based on fatigue moment range at location of maximum positive moment.

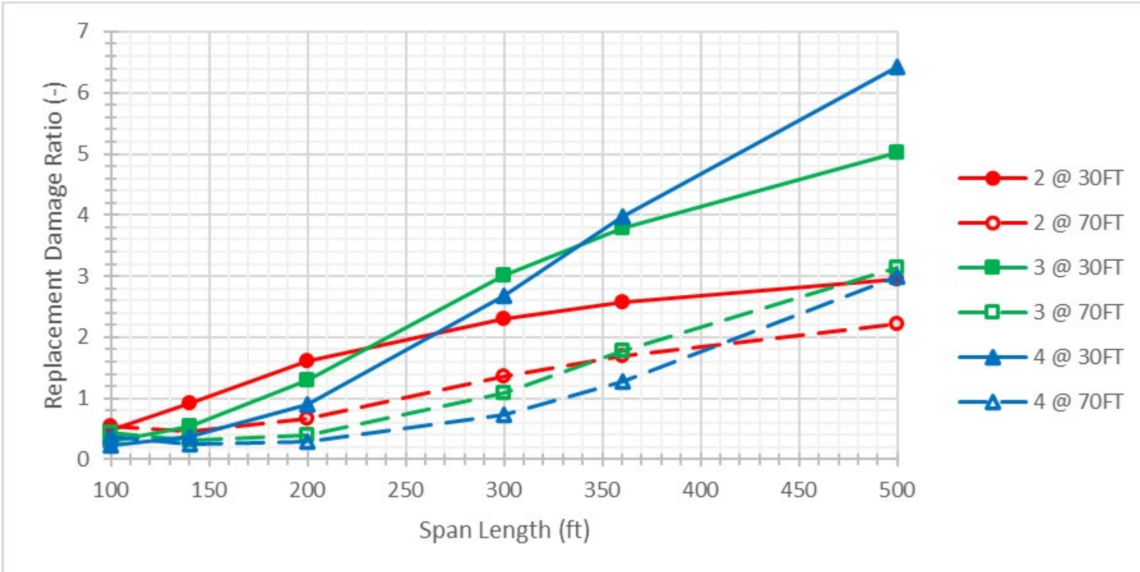

Figure 18. Replacement damage ratio for continuous end spans. Based on fatigue moment range at location of maximum positive moment.

Table 19. Minimum span lengths at which truck platooning may have detrimental effects on fatigue finite life.

Platoon Type	Simple Span	Continuous Interior Span	Continuous End Span
2 Trucks @ 30 ft	150 ft	150 ft	150 ft
2 Trucks @ 70 ft	250 ft	260 ft	250 ft
3 Trucks @ 30 ft	170 ft	180 ft	180 ft
3 Trucks @ 70 ft	270 ft	290 ft	290 ft
4 Trucks @ 30 ft	190 ft	220 ft	210 ft
4 Trucks @ 70 ft	310 ft	350 ft	330 ft

Finally, the replacement damage ratios previously discussed can be used to compute fatigue finite life reductions for different levels of truck platoon penetration rates. For a given replacement damage ratio, $RDR_{platoon}$, and a platoon penetration ratio, PR, the change in fatigue life, Δ_{FL}, is computed as follows:

$$\Delta_{FL} = \frac{1}{\left[(1 - PR) + PR \cdot RDR_{platoon}\right]} - 1 \qquad \text{Eq. 8}$$

Table 20 shows the ranges of Δ_{FL} computed for the span lengths considered. It should be noted that it is limited to truck platoons of 2, 3, or 4 trucks and headway distances of 30' and 70'. Examination of Table 20 reveals that platooning effects are detrimental for spans measuring more than 200'. While the penetration rates of truck platoons into the ADTT are not known yet, it could be inferred that in the short-term and for most bridges in the US, th longer spans are much more susceptible to fatigue finite life reductions due to truck platooning, particularly if truck platoons are expected to be a much larger part of the ADTT.

Table 21 shows the finite fatigue life reduction ranges computed for the headway distances considered, and in agreement with the information regarding replacement damage ratios in Figure 16, Figure 17 and Figure 18, increasing the headway distance results in better fatigue finite life performance.

Table 20. Change in fatigue finite life ranges for various span lengths and penetration rates. Positive values indicate an increase in finite fatigue life. Negative values indicate a decrease in finite fatigue life.

Span Length (ft)	100% Penetration	50% Penetration	20% Penetration	10% Penetration
100	5.1% to 500.9%	2.5% to 71.5%	1.0% to 20.0%	0.5% to 9.1%
140	7.5% to 455.8%	3.6% to 69.5%	1.4% to 19.6%	0.7% to 8.9%
200	-38.4% to 453.9%	-23.7% to 69.4%	-11.1% to 19.6%	-5.9% to 8.9%
300	-68.7% to 74.1%	-52.4% to 27.0%	-30.5% to 9.3%	-18.0% to 4.4%
360	-76.4% to -8.9%	-61.8% to -4.6%	-39.3% to -1.9%	-24.4% to -1.0%
500	-85.1% to -52.3%	-74.1% to -35.4%	-53.3% to -18.0%	-36.4% to -9.9%

Table 21. Change in fatigue finite life ranges for various headway distances and penetration rates. Positive values indicate an increase in finite fatigue life. Negative values indicate a decrease in finite fatigue life.

Platoon Type	100% Penetration	50% Penetration	20% Penetration	10% Penetration
2 Trucks @ 30ft	-66.4% to 149.4%	-49.7% to 42.8%	-28.3% to 13.6%	-16.5% to 6.4%
2 Trucks @ 70ft	-54.9% to 236.1%	-37.8% to 54.1%	-19.6% to 16.3%	-10.8% to 7.6%
3 Trucks @ 30ft.	-80.7% to 398.9%	-67.7% to 66.6%	-45.6% to 19.0%	-29.5% to 8.7%
3 Trucks @ 70ft.	-70.1% to 388.8%	-54.0% to 66.0%	-31.9% to 18.9%	-19.0% to 8.6%
4 Trucks @ 30ft.	-85.1% to 500.9%	-74.1% to 71.5%	-53.3% to 20.0%	-36.4% to 9.1%
4 Trucks @ 70ft.	-67.7% to 455.8%	-51.2% to 69.5%	-29.5% to 19.6%	-17.3% to 8.9%

The main finding of the study of platooning effects on finite fatigue life is that truck platooning is expected to be detrimental to the finite fatigue performance of bridges with spans above 200 ft, although it could affect spans down to 150 ft. Truck platooning might not be detrimental to shorter spans due to the tradeoff between higher magnitude of fatigue stress cycle and lower total number of effective stress cycles. In general, except for very short continuous spans, the magnitude of the fatigue moment range due to the passage of a platoon is greater than for a single truck. However, as discussed before, platoons do not generate a significantly higher number of stress cycles; in fact, it can be assumed that both, a platoon and a single truck generate approximately 1.0 effective stress cycles. Therefore, for platooning to result in detrimental effects to finite life fatigue performance, the stress ranges due to a platoon should be large enough to compensate for fewer total effective fatigue cycles. This compensation begins to occur at span lengths of 150’, although it is more notable for span 200’ or longer.

2.5 Truck Platooning Effects on a Representative Sample of Bridges

The main objective of this study in Section 2.5 to 2.7 is to estimate the number of bridges which service or fatigue limit states would be negatively affected by truck platooning. To accomplish this objective, a database of AASHTOWare Bridge Rating (BrR) bridge models representative of the NBI was gathered. The collected bridge models were built using AASHTOWare BrR. For each of the models in the database the results from BrR analysis runs are used to establish which service or fatigue limit states are negatively affected. All bridges were analyzed according to LRFR method. A bridge is negatively affected when the rating factor for a truck platoon configuration is below 1.0 and lower than the rating factor computed for HL-93 at the operating level. I. e., the bridge does not have sufficient capacity against the demands due to the platoon loading, and these demands are larger than those due to HL-93. After analyzing the entire bridge database, the characteristics common among affected bridges are grouped. These groups are used to develop queries applicable to the NBI to estimate the total number of bridges negatively affected by truck platooning at the service and fatigue limit states.

The RT was able to utilize a developer license for AASHTOWare BrR, which allowed batch running of the models in the database. The use of BrR allows for the input of any number of custom truck definitions characterizing platoon truck configurations. For each bridge span the points of interest are located over supports (i.e., 0.0L and 1.0L) and at 0.4L for a head span, 0.5L for an interior span, and 0.6L for a tail span. Based on the results from the batch analysis and post-processing, the RT identified bridges that are negatively affected by the truck platooning for service and fatigue limit states. The results for service limit states are summarized in Table 22. It should be noted that negatively affected bridges are those which rating factor is lower than 1.0 and lower than the rating factor for HL-93 at the operating level. It should also be noted that Service I rating factors are computed in accordance with MBE (23 CFR 650.317) 6A.4.2.2 and 6A.5.4.2.2 (see Section 2.5.2 for further discussion).

Table 22. Summary of controlling rating factors.

Rating Factors		Min. HL-93	Min. Platoon	Max. HL-93	Max. Platoon	Avg. HL-93	Avg. Platoon	Std. Dev. HL-93	Std. Dev. Platoon	No. of bridges analyzed	No. of bridges negatively affected
Service II	Steel Plate	0.506	0.118	8.155	15.507	2.111	2.638	0.694	1.156	843	6
	Steel Built-up	1.352	1.684	3.239	6.176	1.966	2.741	0.525	1.380	9	0
	Steel Rolled	0.696	0.725	3.660	7.083	1.476	1.975	0.649	1.392	17	0
Service I	PS I	2.082	0.744	42.078	39.946	21.549	15.050	7.321	9.615	164	2
	PS Box	13.485	7.691	59.468	50.677	29.180	22.576	14.368	14.485	16	0
	PT MCB	11.608	5.163	81.784	64.459	45.193	18.796	23.134	15.049	35	0
	RC MCB	3.084	2.649	5.622	4.252	4.338	3.222	1.269	0.894	2	0
Service III	PS I	0.373	0.085	2.467	14.926	0.933	1.296	0.322	1.203	165	20
	PS Box	0.874	0.793	2.518	5.027	1.665	2.653	0.631	1.547	16	2
	PT MCB	0.807	0.714	2.522	3.562	1.753	2.243	0.400	0.744	35	2

2.5.1 BrR Bridge Model Database

The RT gathered 1519 BrR bridge models used by the developers of BrR to assess updates to the software plus 837 additional BrR bridge models from six state DOTs: Alabama, California, Illinois, Michigan, New York, and South Carolina. The following figures show how these bridges are distributed according to age, Figure 19, and structure type, Figure 20. The states were selected so that the resulting database was representative of the NBI, intending to avoid a database that is skewed towards a particular environment or inventory make-up. For the resulting database, over 70% of the bridges were built between 1960 and 2010; and over 80% of the bridges with span lengths longer than 100 ft were built between 1970 and 2020. For over 80% of the bridges in the database, the primary superstructure members are prestressed concrete I-beams, steel plate girders, or steel rolled beams; and when looking only at bridge with span lengths longer than 100 ft, over 70% are steel plate girders. It should be noted that an additional model of a reinforced concrete girder bridge has been analyzed; this model is not included in the figures below, nor in Table 22.

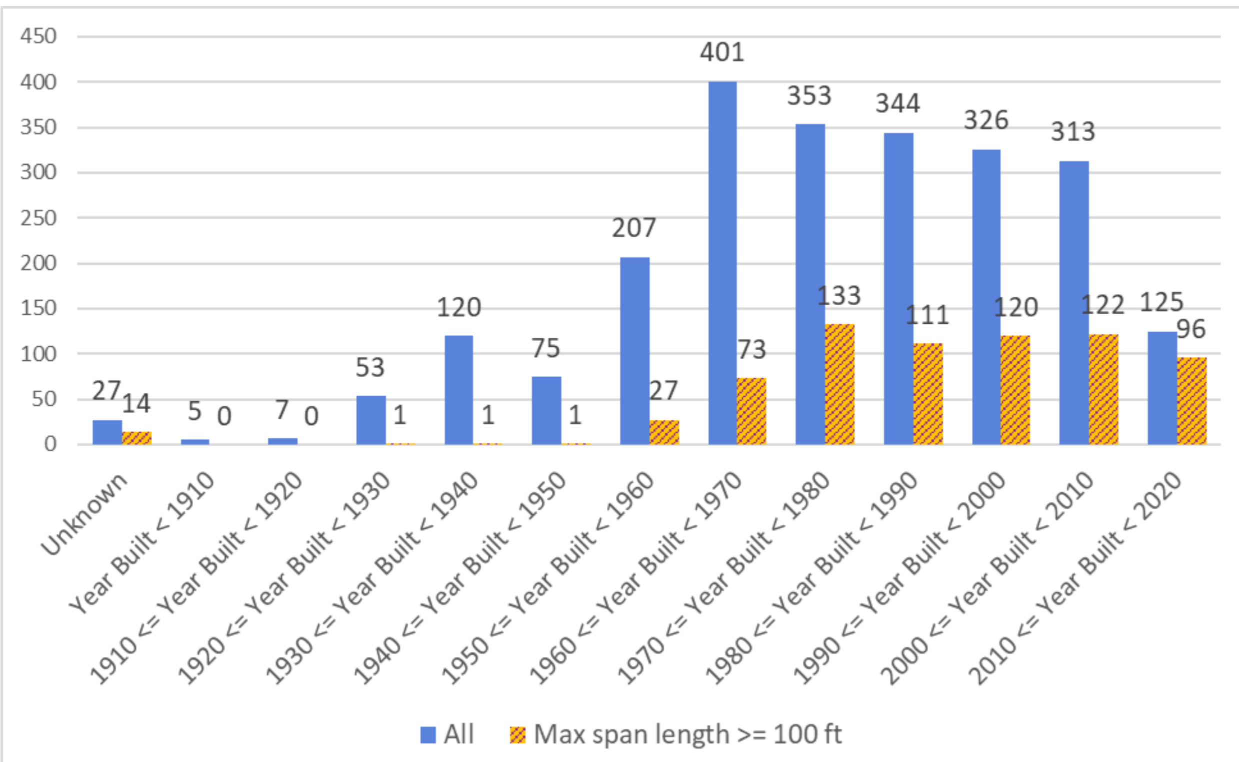

Figure 19. Breakdown of bridges in the database according to age.

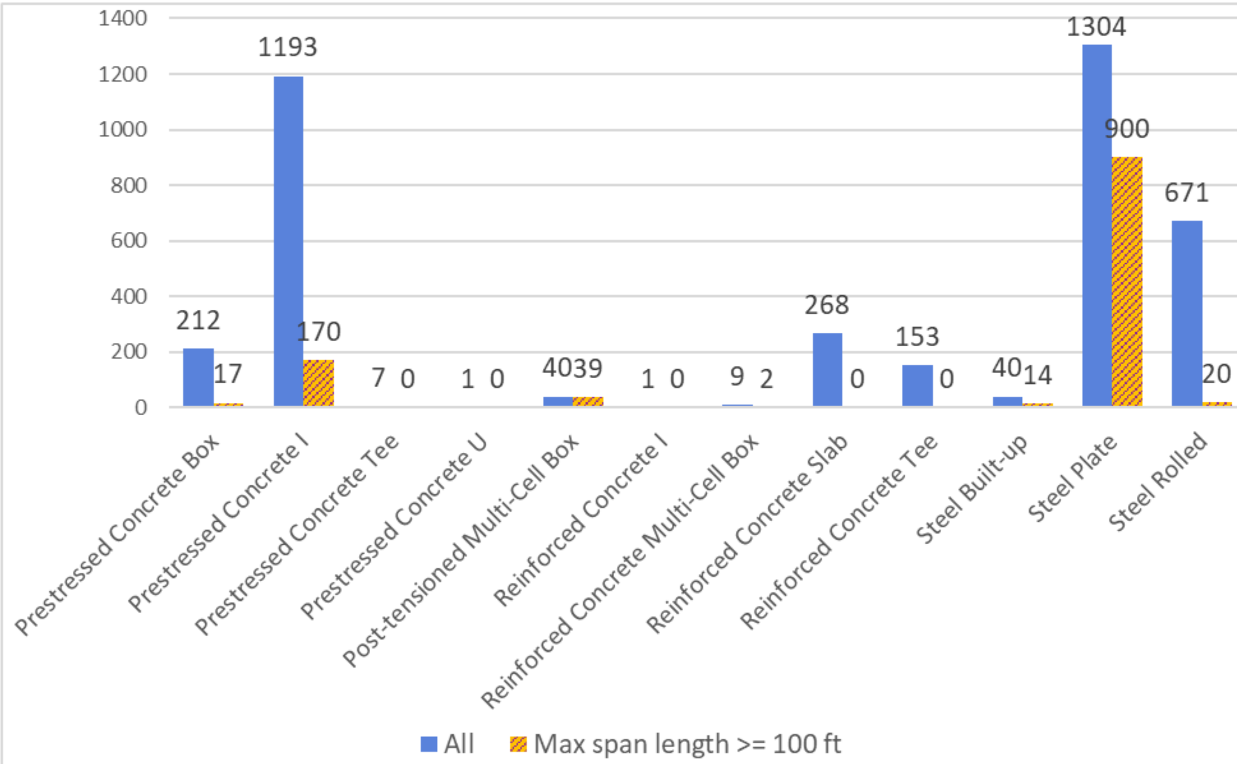

Figure 20. Breakdown of bridge superstructures in the database according to primary superstructure member.

2.5.2 Effect on Service I Limit States

As BrR is a tool primarily conceived for load rating, the Service I limit state for which rating factors are computed is tensile steel stresses in concrete components for permit load rating, as in MBE (23 CFR 650.317) 6A.4.2.2 and 6A.5.4.2.2. The rating factor is calculated for the Service I load combination using the distribution factors specified in LRFD Spec. (23 CFR 625.4(d)(1)(v)) 4.6.2. The results of the analysis are summarized in Figure 21, Figure 22, Figure 23, and Figure 24 for PS (prestressed) I-beams, PS box beams, PT (post-tensioned) muti-cell box beams, and RC (reinforced concrete) multi-cell box girders beams, respectively. A summary of results can be found in Table 22 in 2.5.

The calculated rating factors indicate that for Service I in concrete bridges the average rating factors are decreased for platoons in all cases. However, as the rating factor computation for Service I in BrR is intended to comply with MBE (23 CFR 650.317) 6A.5.4.2.2b, it should be noted that:

- The truck platoon must be run as a single lane or as multi-lane vehicles. I. e., it is not run with an adjacent design vehicle.

- The full HL-93 model is not run. Only the single truck or tandem are considered. The single truck or tandem in combination with lane load are considered for negative flexure only. The application of the HL-93 as described in the third bullet of LRFD Spec. (23 CFR 625.4(d)(1)(v)) 3.6.1.3.1 is not implemented.

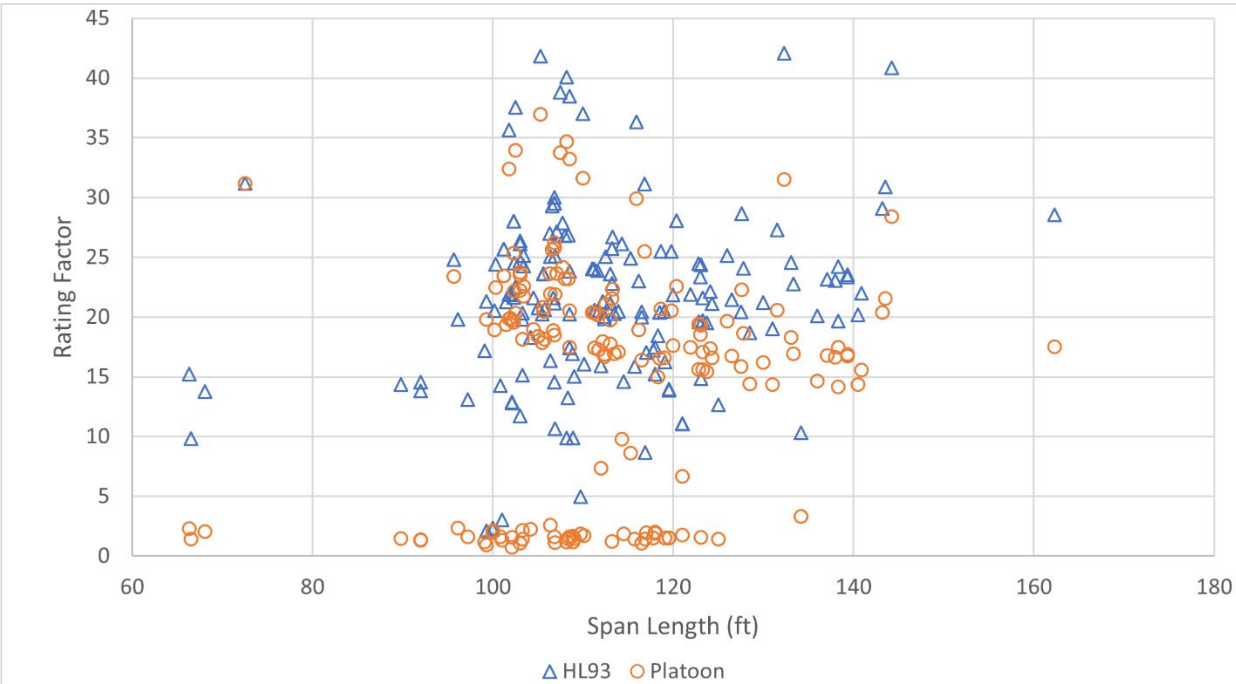

Figure 21. Rating factors for Service I for PS I-beam.

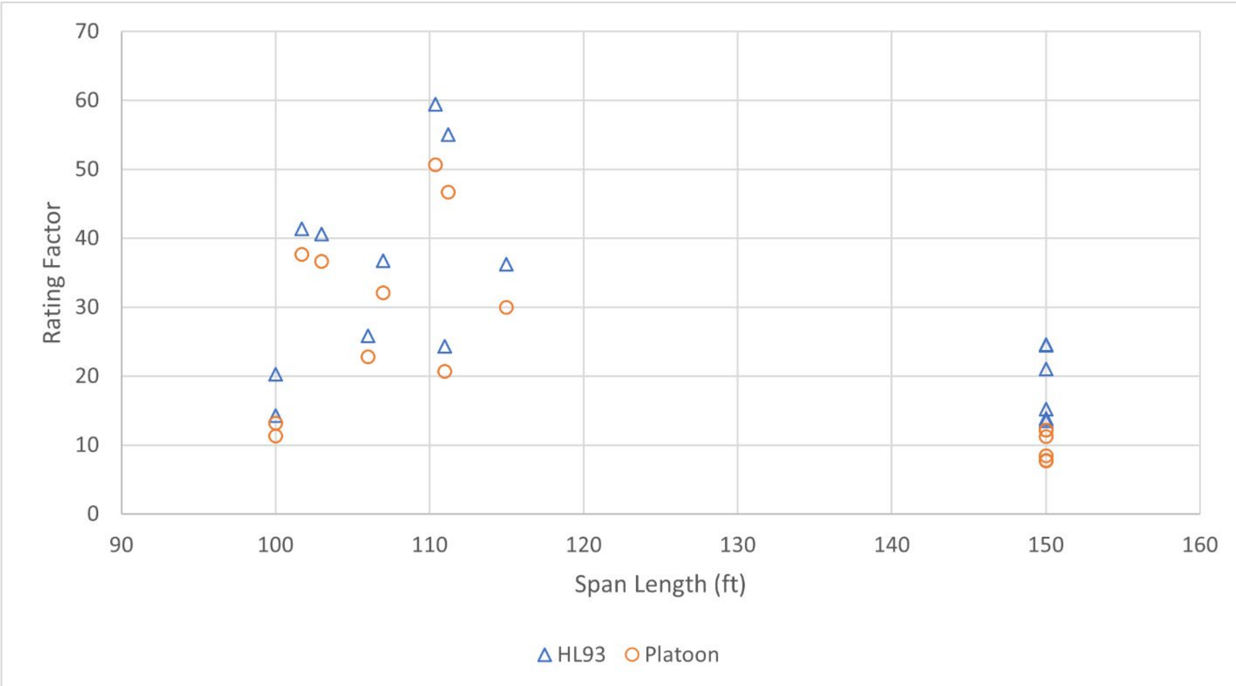

Figure 22. Rating factors for Service I for PS box-beam.

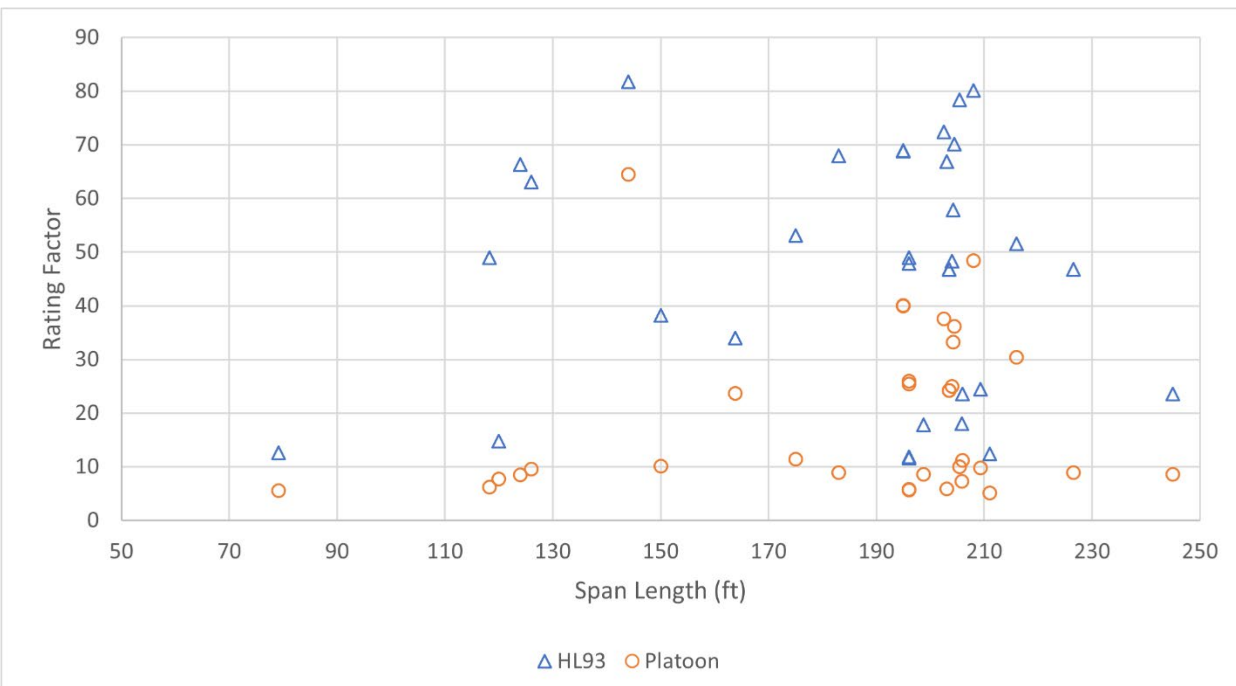

Figure 23. Rating factors for Service I for PT multi-cell box.

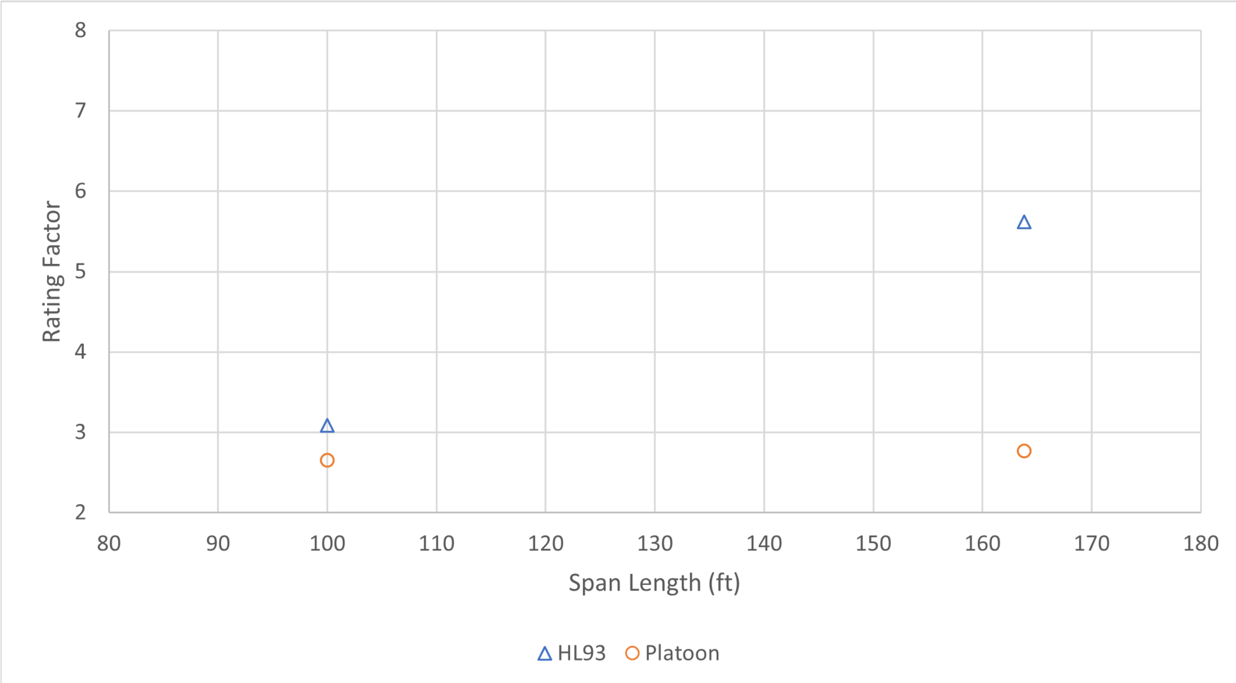

Figure 24. Rating factors for Service I for RC multi-cell box.

2.5.3 Effect on Service II Limit States

As BrR is a tool primarily conceived for load rating, the Service II limit state for which rating factors are computed is permanent deformations in steel components due to flexure. This limit state is described in MBE (23 CFR 650.317) 6A.4.2.2 and 6A.6.4.2. The rating factor is calculated for the Service II load combination. The results of the analysis are summarized in Figure 25, Figure 26, and Figure 27 for steel plate girders, steel rolled beams, and steel built-up girders, respectively. A summary of results can be found in Table 22 in 2.5.

The results indicate that for Service II in steel bridges, the average rating factors are increased for platoons and fewer numbers of bridges are affected. Only 35 out of 843 steel plate girder bridges have reduced load rating factors.

It should be noted that, in the computation of rating factors for the platoon vehicle, BrR applies the load effect due to the adjacent vehicle in the numerator rather than the denominator as follows:

$$RF_{platoon} = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW - \gamma_{LL} (LL + IM)_{adj}}{\gamma_{LL} (LL + IM)_{platoon}} \quad \text{Eq. 9}$$

where:

- $RF_{platoon}$ – Rating factor for the platoon vehicle

- C – Capacity against service limit state
- $\gamma_{DC}DC$ – Factored demand due to dead load of components and attachments.
- $\gamma_{DW}DW$ – Factored demand due to the dead load of wearing surfaces and utilities.
- $\gamma_{LL}(LL+IM)_{adj}$- Factored demand due to the live load of adjacent vehicle. In the context of the analyses discussed in the current report, the adjacent vehicle is the HL-93 load model.
- $\gamma_{LL}(LL+IM)_{platoon}$ – Factored demand due to the live load of the platoon vehicle.

For a given set of inputs that produce a rating factor equal to 1.0 using the typical rating factor equation that places the total factored live load demand in the denominator, the same set of inputs will produce a rating factor equal to 1.0 using Eq. 9. However, Eq. 9 will result in higher rating factors when they are above 1.0, and lower rating factor when they are below 1.0 with respect to the typical rating factor equation that places the total factored live load demand in the denominator. Nonetheless, the equation used by BrR is not erroneous; it is intended to be a direct factor of the permit truck load effect so that an Agency can control permit truck weight.

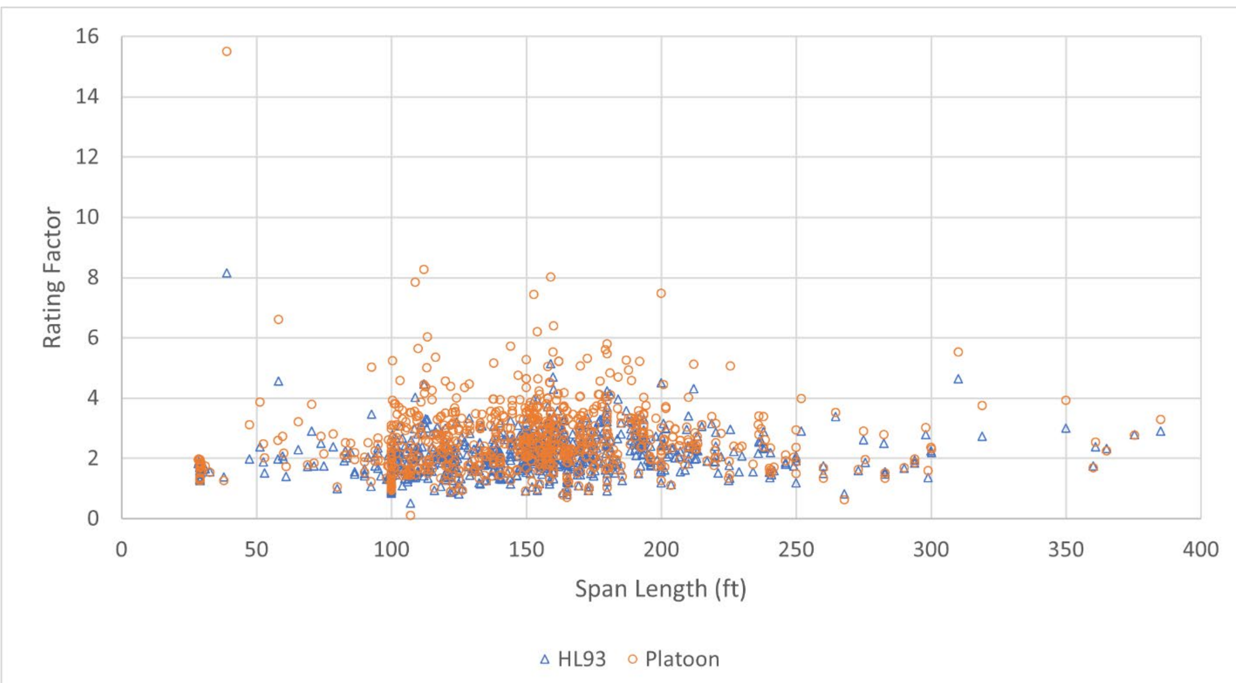

Figure 25. Rating factors for Service II for steel plate girder.

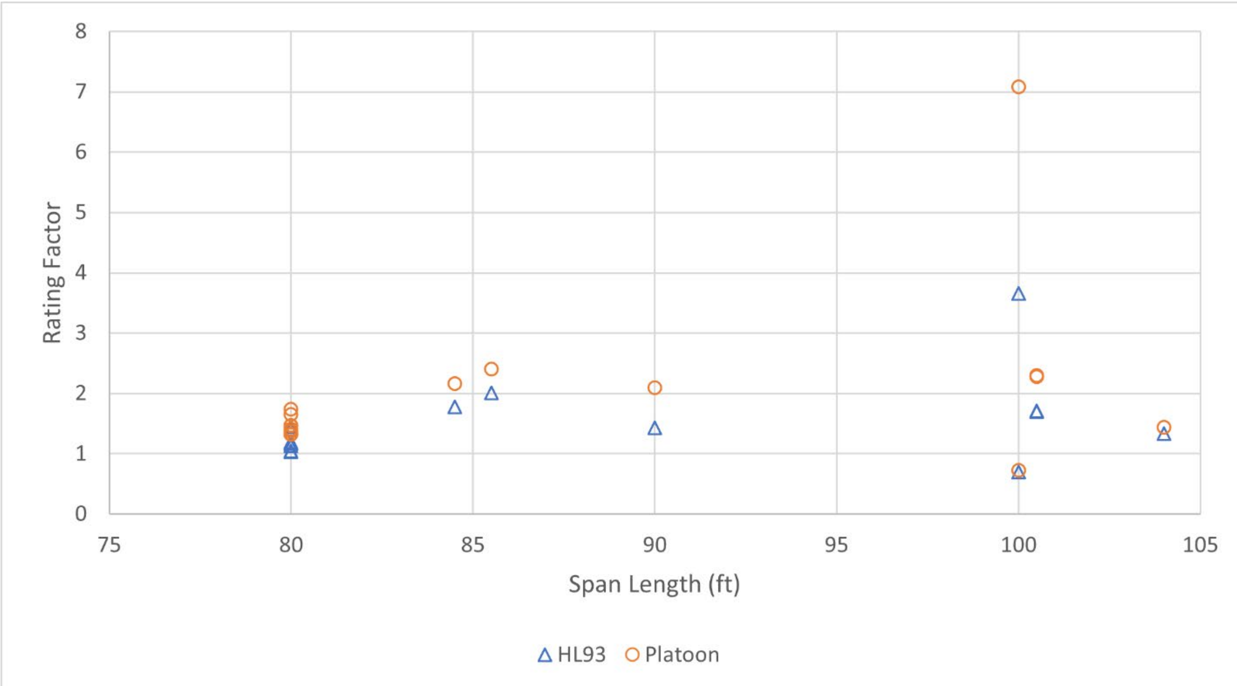

Figure 26. Rating factors for Service II for steel rolled beam.

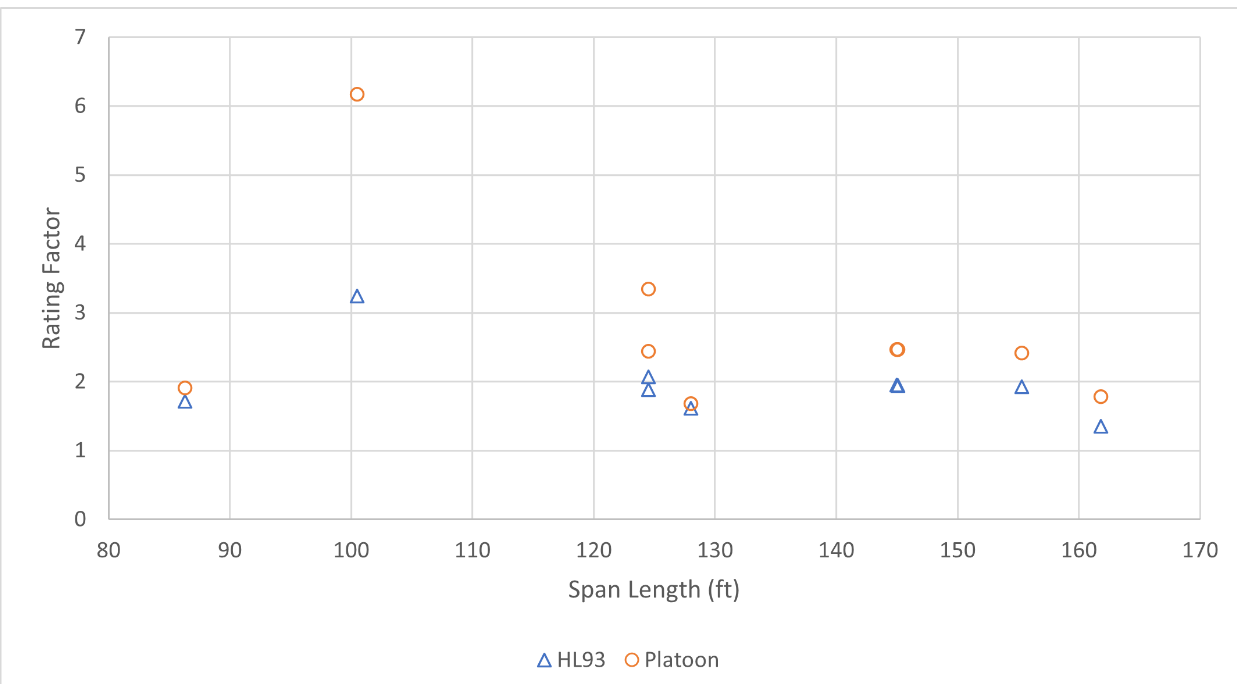

Figure 27. Rating factors for Service II for steel built-up girder.

2.5.4 Effect on Service III Limit States

As BrR is a tool primarily conceived for load rating, the Service III limit state for which rating factors are computed is concrete tensile stresses after losses in prestressed concrete components as in MBE (23 CFR 650.317) 6A.4.2.2 and 6A.5.4.2.2. The rating factor is calculated for the Service III load combination. The results of the analysis are summarized in Figure 28, and Figure 29 for PS I-beams, and PT multi-cell box beams, respectively. A summary of results can be found in Table 22 in 2.5.

The results indicate that for Service III in concrete bridges, the average rating factors are increased for platoons and few bridges are negatively affected. Only 20 out of 165 concrete I girder bridges have reduced load rating factors.

As previously discussed in 2.5.3, it should be noted that, in the computation of rating factors for the platoon vehicle, BrR applies the load effect due to the adjacent vehicle in the numerator rather than the denominator as shown in Eq. 9. In the context of the analyses discussed in the current report, the adjacent vehicle is the HL-93 load model.

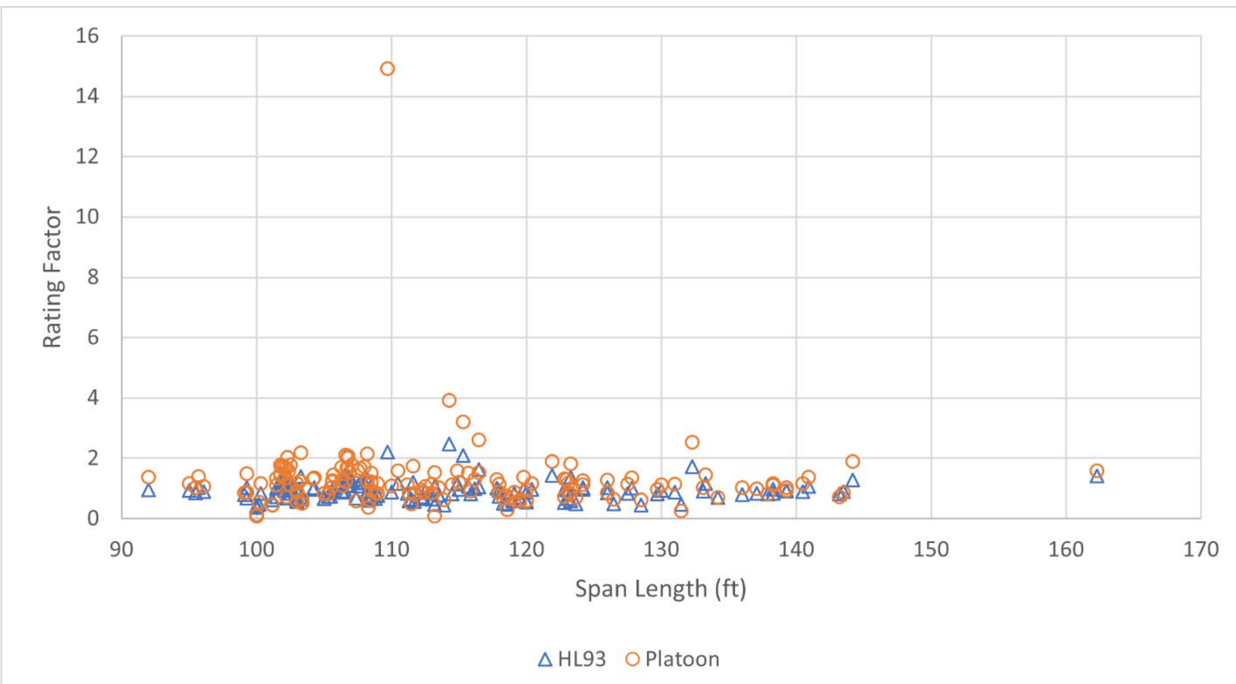

Figure 28. Rating factors for Service III for PS I-beam.

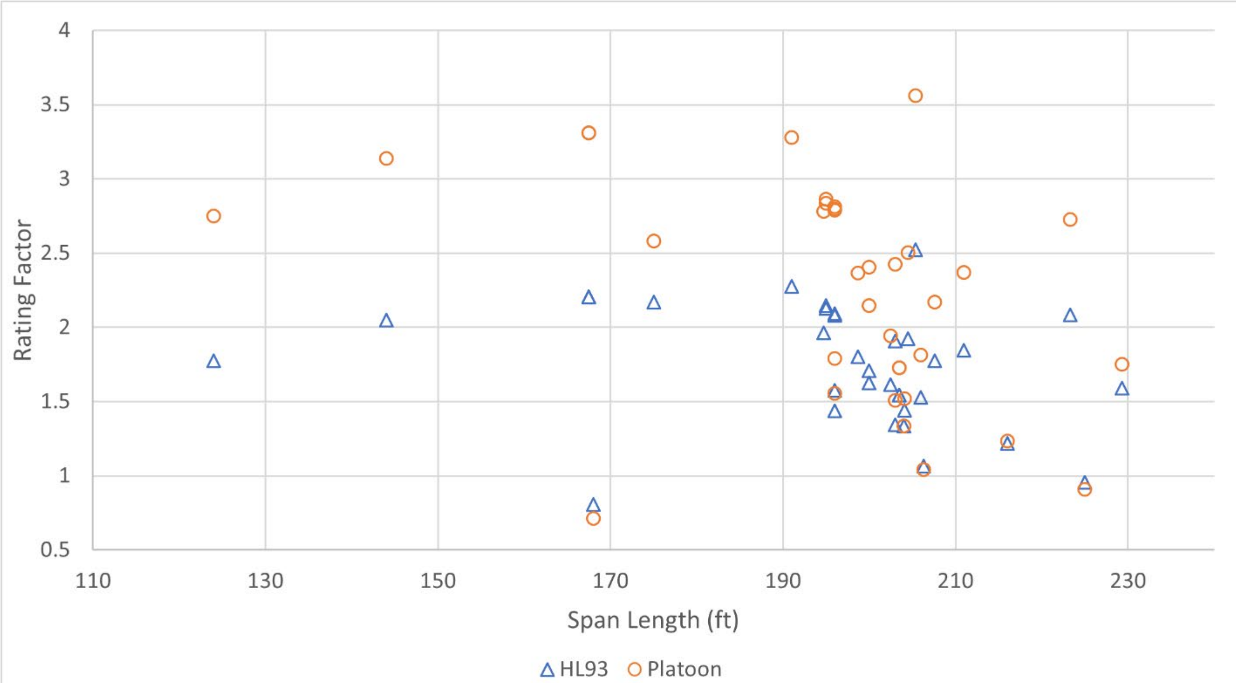

Figure 29. Rating factors for Service III for PT multi-cell box.

2.5.5 Effect on Fatigue Limit State

Fatigue life reduction ratios were computed for the platoon trucks relative to the AASHTO LRFD Fatigue Truck; these are shown in Table 23. The results show that truck platoons will produce stress cycles that significantly impact the fatigue performance of steel bridges. This is reflected in the fatigue life reduction factor which is the decrease in fatigue life due to the application of truck platoons with respect to the fatigue life computed for the AASHTO LRFD Fatigue Truck. The fatigue life reduction ratio is computed as follows:

$$\text{Fatigue Life Reduction Ratio} = 1 - \frac{[(\Delta \mathrm{f})_{LRFD}]^{\frac{-1}{3}} - [(\Delta \mathrm{f})_{Platoon}]^{\frac{-1}{3}}}{[(\Delta \mathrm{f})_{LRFD}]^{\frac{-1}{3}}} \quad \text{Eq. 10}$$

where:

- $(\Delta \mathrm{f})_{LRFD}$ is the live load stress range due to the AASHTO LRFD Fatigue Truck.
- $(\Delta \mathrm{f})_{Platoon}$ is the live load stress range due to the platoon vehicle.

In the computation of fatigue life reduction ratios, full penetration of the platoon truck is assumed. Other parameters used in the calculation of fatigue life reduction ratios such as partial load factors, or number of cycles per truck passage are assumed to be the same for the platoon truck and the AASHTO LRFD Fatigue Truck. Therefore, the fatigue life reduction factors are not directly equivalent to a reduction of the expected fatigue life of the bridge, rather, they indicate the relative impact for the passage of a platoon relative to a single truck. Table 23 shows that, platoons typically generate fatigue stress cycles of larger magnitude than the AASHTO LRFD Fatigue Truck; consequently, the passage of a platoon is more damaging to the fatigue performance of a bridge than the AASHTO LRFD Fatigue Truck. However, this does not fully account for the lower number of effective fatigue cycles that result from grouping single trucks into platoons while ADTT is remains equal. Section 2.4 does account for this effect and provides more detail discussion.

Table 23. Summary of fatigue life reduction ratios.

Structure Type	Min	Max	Average	Std. Deviation
Steel Plate	0.665	1.000	0.890	0.084
Steel Built-up	0.670	0.988	0.915	0.085
Steel Rolled	0.665	0.973	0.898	0.065

2.5.6 Bridges in the BrR Bridge Model Database Negatively Affected by Truck Platoons

After conducting load ratings on the structures contained in the database described in Section 2.5.1, thirty-three structures were found to be negatively impacted by the application of platoon loading on the service limit states. Bridges negatively affected have rating factors less than 1.0 for the platoon vehicle configurations considered, and less than the rating factor calculated for the HL-93 load model at the operating level. Table 24 shows the controlling limit state and information obtained from BrR models of bridges negatively affected by platoon loading, and Table 25 shows the controlling limit state and information obtained from the NBI of bridges negatively affected by platoon. It should be noted that the number of bridges affected (33) is very small compared to the overall size of the model database, 3899.

Comparison between the values reported in Table 24 and Table 25 reveals that inconsistent reporting is common in the NBI. This is particularly evident when comparing maximum span lengths input in BrR models against the values reported in the NBI. Only three structures (7, 8, and 32) have the same maximum span lengths in the BrR model and in the NBI. There are also disagreements regarding structure type, as six bridges (1, 2, 18, 19, 29, 30) had their main span material (e.g., concrete, concrete continuous, steel, etc.) erroneously identified.

Table 24. Controlling limit state and BrR model information of bridges negatively affected by platoon configurations considered.

Bridge Number	Controlling Limit State	State	Bridge ID	Bridge Name	Year Built	Type	Max. Span Lengths (ft)	Span Lengths (ft)
1	Service I	AL	180	015940	1991	PS I beam	100.02	99.27-100.01-99.27
2	Service I	SC	645	000000000008525	1992	PS I beam	103.17	102.08-103.17-102.17
3	Service II	Unknown	1709	STMS-1137-MGSP	1961	Steel plate girder	141	107-141-107
4	Service II	Unknown	1713	STMS-1141-MGSP	1966	Steel plate girder	267.83	107.08-267.83-112
5	Service II	AL	133	011201	1974	Steel plate girder	126.17	126.17-126.16
6	Service II	CA	292	24 0004L	1966	Steel plate girder	165	164.7-190.44-163.62-165-165-165
7	Service II	CA	292	24 0004L	1966	Steel plate girder	275	165-165-167.4-205-275-275-205-163.6
8	Service II	CA	293	24 0004R	1966	Steel plate girder	275	165-165-175.3-205-275-275-205-163.3
9	Service III	Unknown	978	PSSS-0510-PSIB	1969	PS I beam	100	100
10	Service III	Unknown	1020	PSSS-0553-PSIB	2008	PS I beam	143.15	143.15
11	Service III	Unknown	1072	PSSS-0605-PSIB	1969	PS I beam	100	100
12	Service III	Unknown	1074	PSSS-0607-PSIB	1976	PS I beam	103	103
13	Service III	Unknown	1075	PSSS-0608-PSIB	1997	PS I beam	108.42	108.42
14	Service III	Unknown	1080	PSSS-0613-PSIB	2008	PS I beam	143.15	143.15
15	Service III	Unknown	713	PSMS-1387-PSIB	2007	PS I beam	134.21	134.21-123.21
16	Service III	Unknown	842	PSMS-1516-PSIB	2004	PS I beam	110	108.34-110-108.34
17	Service III	AL	238	021151	2018	PS I beam	137.98	137.98

Bridge Number	Controlling Limit State	State	Bridge ID	Bridge Name	Year Built	Type	Max. Span Lengths (ft)	Span Lengths (ft)
18	Service III	SC	638	000000000002668	2004	PS I beam	120.86	119.48-120.86-120.49-119.75-118.37
19	Service III	SC	638	000000000002668	2004	PS I beam	120.86	119.48-120.86-118.65
20	Service III	SC	644	000000000008519	1992	PS I beam	101.24	101.24
21	Service III	SC	644	000000000008519	1992	PS I beam	112.71	112.71
22	Service III	SC	644	000000000008519	1992	PS I beam	103.42	103.42
23	Service III	SC	645	000000000008525	1992	PS I beam	101.24	101.24
24	Service III	SC	645	000000000008525	1992	PS I beam	118.55	118.55
25	Service III	SC	645	000000000008525	1992	PS I beam	113.16	113.16
26	Service III	SC	645	000000000008525	1992	PS I beam	108.62	108.62
27	Service III	SC	645	000000000008525	1992	PS I beam	103.42	103.42
28	Service III	SC	655	000000000009826	2005	PS I beam	131.5	131.5
29	Service III	CA	302	35 0038	1981	PS Box beam	150	150-150-150
30	Service III	CA	302	35 0038	1981	PS Box beam	150	150-150-150
31	Service III	Unknown	2169	M2003-PSMS-PTMCB	Unknown	PT Multi-Cell Box	168	126-168-118
32	Service III	CA	306	04 0228	1971	PT Multi-Cell Box	225	120-225-120
33	Service I	IN	N/A	NBI=005165 (CRCG)	1958	RC T girder	101.5	72.5-101.5-101.5-101.5-72.5

Table 25. Controlling limit state and information obtained from NBI of bridges negatively affected by platoon configurations considered.

Bridge Number	Controlling Limit State	Year Built	Design Load	Structure Kind	Max. Span Length (ft)	Inventory Rating Method
1	Service I	1991	HS 20	Steel	190.9	Load Factor
2	Service I	1992	HS 20 + Mod	Steel Continuous	202.1	Load and Resistance Factor
3	Service II	Unknown	Unknown	Unknown	Unknown	Unknown
4	Service II	Unknown	Unknown	Unknown	Unknown	Unknown
5	Service II	1974	HS 20	Steel Continuous	274.9	Load Factor
6	Service II	1966	HS 20	Steel	274.9	Load and Resistance Factor
7	Service II	1966	HS 20	Steel	274.9	Load and Resistance Factor
8	Service II	1966	HS 20	Steel	274.9	Load and Resistance Factor
9	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
10	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
11	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
12	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
13	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
14	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
15	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
16	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
17	Service III	2018	HL 93	PS Concrete	160.1	Load and Resistance Factor
18	Service III	2004	HS 25 or greater	Steel Continuous	299.9	Load and Resistance Factor
19	Service III	2004	HS 25 or greater	Steel Continuous	299.9	Load and Resistance Factor
20	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor

Bridge Number	Controlling Limit State	Year Built	Design Load	Structure Kind	Max. Span Length (ft)	Inventory Rating Method
21	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
22	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
23	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
24	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
25	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
26	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
27	Service III	1992	HS 20 + Mod	PS Concrete	202.1	Load and Resistance Factor
28	Service III	2005	HS 25 or greater	PS Concrete	212.9	Load and Resistance Factor
29	Service III	1981	HS 20	Steel Continuous	339.9	Load and Resistance Factor
30	Service III	1981	HS 20	Steel Continuous	339.9	Load and Resistance Factor
31	Service III	Unknown	Unknown	Unknown	Unknown	Unknown
32	Service III	1971	HS 20	PS Concrete Continuous	225.1	Load and Resistance Factor
33	Service I	1958	HS 20	Concrete Continuous	101.5	Load Factor

2.5.7 Truck Platooning Effects on Service Limit States Not Analyzed by BrR

As AASHTOWare BrR is primarily a load rating tool, it is developed to evaluate service limit states that are considered in the MBE (23 CFR 650.317). For LRFR, the service limit states evaluated by BrR are:

- Service I – Tensile stresses in reinforcing and/or prestressing steel (MBE (23 CFR 650.317) 6A.5.4.2b);

- Service II – Permanent deformation in steel flexural members (MBE (23 CFR 650.317) 6A.6.4.2.2, analogous to LRFD Spec. 6.10.4.2 and 6.11.4);

- Service III - Tensile stresses in concrete at service limit state after losses (MBE (23 CFR 650.317) 6A.5.4.2a, analogous to LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.3.2b).

It should be noted that Service I – Tensile stresses in reinforcing and/or prestressing steel (MBE (23 CFR 650.317) 6A.5.4.2b) does not have a directly analogous section in the LRFD Spec. (23 CFR 625.4(d)(1)(v)). However, since it is a stress check in tensile reinforcement and or prestressing to ensure concrete crack closure and reserve ductility (MBE (23 CFR 650.317) C6A.5.4.2.2b), it indirectly provides a check for the following limit states in the LRFD Spec. (23 CFR 625.4(d)(1)(v)):

- Service I - Cracking Control by Reinforcement Distribution (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.6.7);

- Service I - Stress Limitations for Prestressing Steel (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.2).

Although there are some differences between the stress limits and live load distribution analysis described in MBE (23 CFR 650.317) 6A.5.4.2b and the provisions for LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.6.7 and 5.9.2.2, truck platooning is expected to have a similar effect on these three service limit states.

LRFD Spec. (23 CFR 625.4(d)(1)(v)) includes other service limit states for which BrR does not compute a load rating factor:

- Service I - Elastic Deformations (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 2.5.2.6): This limit state is the optional criteria for elastic deformations, which are primarily intended to result in adequate minimum stiffness so that undesirable structural or psychological effects are avoided. As the presence of truck platoons is not expected to affect the dynamic behavior of the bridge itself, truck platooning is not deemed to affect this limit state.

- Service I - Compressive Stresses at Service Limit State after Losses (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.3.2a): This limit state only applies to prestressed members. As the load effects are evaluated for flexure, the critical locations of live loads would typically coincide with those for MBE (23 CFR 650.317) 6A.5.4.2b (tensile stresses in reinforcing and/or prestressing steel). Consequently, the impact of truck platooning on MBE (23 CFR 650.317) 6A.5.4.2b partially captures the impact on LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.3.2a.

- Service II - Slip (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 6.13.2.8). In primary steel members the main concern is slip at splice locations. Since the demands affecting slip are primarily flexural, the estimated impact of truck platooning on MBE (23 CFR 650.317) 6A.6.4.2.2 is an indirect measure of the impact on LRFD Spec. (23 CFR 625.4(d)(1)(v)) 6.13.2.8. In secondary members, such as bracing, slip is a concern at their connections. The live load demands are principally dictated by the maximum single axle weight rather than the total live load and can be less critical than those generated by construction loads, wind loads, or other transient loading.

- Service III - Principal Tensile Stresses in Webs (LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.3.3). This limit state only applies to post-tensioned members and pre-tensioned members which concrete compression strength (f_c') is greater or equal to 10 ksi. There is not a similar service limit state that can be used for comparison; however, given the limited number of bridges in the database this service limit states are applicable to, the RT has performed checks outside of BrR. To that end, the post-tensioned bridges in the BrR database, total of 35, were first rated for flexure and shear strength limit states over interior supports only. The three most susceptible bridges were selected based on the computed rating factors for truck platoon versus HL-93, and they were load rated for the Service III – Principal Tensile Stresses in Web limit states outside of BrR. For this particular limit state, none of those three bridges were found to have a rating factor lesser than 1.0 for the truck platoons considered while having rating factor over 1.0 for HL-93. It should be noted that, as the calculation was performed for Service III limit state, the rating factor corresponds to the inventory level.

2.6 Bridge Inventory Selection Tool (BIST)

To determine the total numbers of bridges affected by platooning, a NBI query tool that provides the following capability is needed.

- The tool should be able to filter the NBI data for any year and any state/district.

- The tool should be able to filter the NBI data with various combinations of criteria to cover various bridge properties and breakdown to each state and district.

- The tool should be able to show the filtering results in a tabular format.

- The tool should be automated for filtering the NBI data and populating results.

Because there is no off-the-shelf application that satisfies all the capabilities, the RT has developed the Bridge Inventory Selection Tool (BIST) and uses it to estimate the total number of bridges negatively affected by truck platooning. The BIST reads the NBI ASCII delimited text file, parses it with the given filtering criteria, and outputs the filtering results in tables.

To ensure the correctness of the BIST, the results from the BIST were verified with two references.

- Report on Truck Platooning Impacts on Bridges Phase I – Structural Safety: Table 84 in the Phase I report shows the number of bridges in the Interstate, National Highway System, and National Network in South Dakota and the US, respectively.

- FHWA LTBP (Long-Term Bridge Performance) InfoBridge Bridge Selection and Data Representation: The LTBP InfoBridge provides a web application which can filter the current year or 2020 NBI data and show the results in a table or on a map.

To verify the BIST, the same filtering criteria were applied to it and the filtering results were compared with the numbers of bridges from Table 84 in the Phase I – Structural Safety report and the LTBP InfoBridge. The numbers match perfectly.

2.7 Development of Criteria for the National Bridge Inventory Query

A set of filtering cases were developed and implemented in the BIST. The cases were developed based on the results of the line girder study of truck platooning effects on different span lengths (Section 2.3), the load rating results obtained from the analyses performed on the BrR model database (Section 2.5), and a review of the specifications used in bridge design over the year, as presented hereafter.

It should be noted that the design methodology of the bridge is not specifically stated in the NBI; however, it can approximately be inferred from the year built and design load data. In the cases described hereafter, the following information was used:

- The first AASHTO (AASHO at the time until 1973) specifications for bridge design were issued in 1931, previously a 1928 specification was available for the design of highway steel bridges.

- From 1931 until 1941 the vehicular loads used in design where:

    - H 20 for “frequent heavy” traffic with “regular higher” loads.

    - H 15 for “normal heavy” traffic with “occasional higher” loads.

    - H 10 for temporary or semi-temporary structures with light traffic.

  During that time the lane loads used in designed were referred to as equivalent load.

- In 1941 AASHTO introduced two new vehicles: HS 20, and HS 15.
- In 1944 AASHTO introduced the variable axle spacing requirement for HS 20, and HS 15.
- In 1971 AASHTO introduced the first LFD provisions, until then all design methodologies were based on the application of Allowable Stress Design (ASD).
- From 1978 onwards the HS 25 vehicle was proposed as a new design load on several occasions; however, it was never fully adopted in the design specifications. Nevertheless, a significant number of structures have been designed for HS 25.
- In 1994 AASHTO introduced the first Load and Resistance Factor design provisions. These are tied to the application of the HL-93 vehicular live load model.

For clarification purposes, it is needed to distinguish design method from design load. Particularly for the HS 20 and HS 15 design loads as they may be designed using Allowable Stress or Load Factor design methods depending on the design year. On the other hand, it is reasonable to expect that structures designed for H 10, H 15, and H 20 are designed using Allowable Stress. Similarly, structures designed for HL-93 can be assumed to be designed using Load and Resistance Factor design, and those designed for HS 25 using Load Factor design method.

Many other changes and updates to the bridge design specifications issued by AASHTO have occurred over the years. While ideally the changes to the different articles would be tracked to assess their impact on design for the different service limit states, such as the introduction of the overload provisions or the live load distribution factor equations, there are very many and these changes do not occur individually, so their impact is not uniform through time. Therefore, in general, individual design provisions or articles, other than design live load or methodology, are not used to develop the cases presented hereafter.

While rating factors or tonnage are provided in the NBI; it is very common to report rating factors that are not based on a formal load rating analysis. It is common that bridge owners may input a rating factor equal to 1.00 (or 99.99) as a placeholder for newly constructed or rehabbed bridges. Therefore, reported rating factor or rating tonnage are not used in the cases presented hereafter.

Lastly, an important note to be taken into consideration regarding the development of the cases presented hereafter. These cases are established through a deterministic analysis that neglects the probabilistic aspects of platoon loading. I. e., the platoon truck configurations applied use load factors that do not consider their reduced recurrence as they were developed for design or legal vehicles. Therefore, this is a conservative approach that should only be used as a baseline subject to further refinement. Once the relative recurrence of platoons with respect to measured truck loading is taken into consideration, it is expected that some of the criteria for the following case be alleviated, even erased. The evaluation of the probabilistic aspects of platoon truck loading is detailed in Chapter 3.

2.7.1 Case 1: Steel Bridges Designed before Load Factor Design Provisions

Case 1 applies to steel bridges. Based on the results described in Section 2.5.6, there are five steel bridges in the database built before the issuance of Load Factor Design provisions by AASHTO in 1971 that were negatively affected by the application of platoon loads. These five structures produced rating factors of less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service II – Steel Flexural Stresses. These structures are summarized in Table 26.

Table 26. Bridges in BrR database used to establish Case 1.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
3	Unknown	STMS-1137-MGSP	Steel Plate Girder	1961	141	Unknown
4	Unknown	STMS-1141-MGSP	Steel Plate Girder	1966	267.83	Unknown
6	CA	24 0004L	Steel Plate Girder	1966	165	HS 20
7	CA	24 0004L	Steel Plate Girder	1966	275	HS 20
8	CA	24 0004R	Steel Plate Girder	1966	275	HS 20

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HS 20 loading for spans 80 ft or longer, as shown in Table 18. According to the year built and the design load of the affected structures described in Table 26, these would have been designed for loads equal or lesser than HS 20. Therefore, the criteria shown in Table 27 were established for Case 1 bridges:

Table 27. Filtering criteria for Case 1 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≤ 1973	1973 or older
31	Design Load	0, 1, 2, 3, 4, 5, 6	Unknown, H 10, H 15, HS 15, H 20, HS 20, HS 20 + Mod
43A	Structure Type, Kind of Material and or Design	3, 4	Steel, Steel Continuous
48	Length of Maximum Span in Meters	≥ 00243	80 ft or longer

2.7.2 Case 2: Steel Bridges Designed after Load Factor Design Provisions

Case 2 applies to steel bridges. Based on the results described in Section 2.5.6, there is one steel bridge in the database built after the issuance of Load Factor design provisions by AASHTO in 1971 that was negatively affected by the application of platoon loads. This structure produced rating factors less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service II – Steel Flexural Stresses. The structure is summarized in Table 28.

The sole bridge in Case 2 was built in 1974, which theoretically should mean that it was designed following Load Factor Design methods. As previously discussed, the first issuance of Load Factor Design provisions by AASHTO took place in 1971. However, it is possible that this bridge might have been designed per earlier specifications that only contained Allowable Stress Design provisions, as the built year is only 3 years after the issuance of the 1971 AASHTO Standard Specifications for Highway Bridges. In the context of the current study, and until additional information becomes available, it is assumed that the bridge was designed in accordance with the 1971 AASHTO Standard Specifications for Highway Bridges using Load Factor Design provisions.

Table 28. Bridges in BrR database used to establish Case 2.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
5	AL	011201	Steel Plate Girder	1974	126.17	HS 20

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HS 20 loading for spans 80 ft or longer, as shown in Table 18. According to the year built and the design load of the affected structure described in Table 28, it would have been designed for at least HS 20 loading, but not up to HL-93 loading. Therefore, the criteria shown in Table 29 were established for Case 2 bridges.

Table 29. Filtering criteria for Case 2 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≥ 1974	1974 or newer
31	Design Load	0, 3, 5, 6, 9	Unknown, HS 15, HS 20, HS 20 + Mod, HS 25 or greater.
43A	Structure Type, Kind of Material and or Design	3, 4	Steel, Steel Continuous
48	Length of Maximum Span in Meters	≥ 00243	80 ft or longer

2.7.3 Case 3: Prestressed and Post-tensioned Concrete Bridges Designed per Allowable Stress Design Provisions

Case 3 applies to PS and PT concrete bridges. Based on the results described in Section 2.5.6, there are two PS concrete bridges in the database built before the issuance of Load Factor Design provisions by AASHTO in 1971 that were negatively affected by the application of platoon loads. These structures produced rating factors of less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service III – Concrete Tensile Stress. These structures are summarized in Table 30.

Table 30. Bridges in BrR database used to establish Case 3.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
9	Unknown	PSSS-0510-PSIB	PS I Beam	1969	100	Unknown
11	Unknown	PSSS-0605-PSIB	PS I Beam	1969	100	Unknown

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HS 20 loading for spans 80 ft or longer, as shown in Table 18. According to the year built and the design load of the affected structure described in Table 30, these would have been designed for loads equal or lesser than HS 20. Therefore, the criteria shown in Table 31 were established for Case 3 bridges.

Table 31. Filtering criteria for Case 3 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≤ 1973	1973 or older
31	Design Load	0, 1, 2, 3, 4, 5, 6	Unknown, H 10, H 15, HS 15, H 20, HS 20, HS 20 + Mod
43A	Structure Type, Kind of Material and or Design	5, 6	PS/PT Concrete, PS/PT Concrete Continuous
48	Length of Maximum Span in Meters	≥ 00243	80 ft or longer

2.7.4 Case 4: Prestressed and Post-tensioned Concrete Bridges Designed per Load Factor Design Provisions

Case 4 applies to prestressed and post-tensioned concrete bridges. Based on the results described in Section 2.5.6, there are twenty PS and PT concrete bridges in the database built after the issuance of Load Factor Design provisions by AASHTO in 1971 and before the issuance of Load and Resistance Factor Design provisions by AASHTO in 1994 that were negatively affected by the application of platoon loads. These structures produced rating factors of less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service I – Reinforcement Yielding for two of the structures (1, and 2), while the remaining eighteen were controlled by Service III – Concrete Tensile Stress. These structures are summarized in Table 32.

Table 32. Bridges in BrR database used to establish Case 4.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
1	AL	015940	PS I beam	1991	100.02	HS 20
2	SC	000000000008525	PS I beam	1992	103.17	HS 20 + Mod
12	Unknown	PSSS-0607-PSIB	PS I beam	1976	103	Unknown
13	Unknown	PSSS-0608-PSIB	PS I beam	1997	108.42	Unknown
16	Unknown	PSMS-1516-PSIB	PS I beam	2004	110	Unknown
18	SC	000000000002668	PS I beam	2004	120.86	HS 25
19	SC	000000000002668	PS I beam	2004	120.86	HS 25
20	SC	000000000008519	PS I beam	1992	101.24	HS 20 + Mod
21	SC	000000000008519	PS I beam	1992	112.71	HS 20 + Mod
22	SC	000000000008519	PS I beam	1992	103.42	HS 20 + Mod
23	SC	000000000008525	PS I beam	1992	101.24	HS 20 + Mod
24	SC	000000000008525	PS I beam	1992	118.55	HS 20 + Mod
25	SC	000000000008525	PS I beam	1992	113.16	HS 20 + Mod
26	SC	000000000008525	PS I beam	1992	108.62	HS 20 + Mod
27	SC	000000000008525	PS I beam	1992	103.42	HS 20 + Mod
28	SC	000000000009826	PS I beam	2005	131.5	HS 25
29	CA	35 0038	PS Box beam	1981	150	HS 20
30	CA	35 0038	PS Box beam	1981	150	HS 20
31	Unknown	M2003-PSMS-PTMCB	PT Multi-Cell Box	Unknown	168	Unknown
32	CA	04 0228	PT Multi-Cell Box	1971	225	HS 20

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HS 20 loading for spans 80 ft or longer, as shown in Table 18. According to the year built and the design load of the affected structure described in Table 32, these would have been designed for at least HS 20 loading, but not up to HL-93 loading. Therefore, the criteria shown in Table 33 were established for Case 4 bridges.

Table 33. Filtering criteria for Case 4 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≥ 1974	1974 or newer
31	Design Load	0, 3, 5, 6, 9	Unknown, HS 15, HS 20, HS 20 + Mod, HS 25 or greater.
43A	Structure Type, Kind of Material and or Design	5, 6	PS/PT Concrete, PS/PT Concrete Continuous
48	Length of Maximum Span in Meters	≥ 00243	80 ft or longer

2.7.5 Case 5: Prestressed and Post-tensioned Concrete Bridges Designed per Load and Resistance Factor Design Provisions

Case 5 applies to prestressed and post-tensioned concrete bridges. Based on the results described in Section 2.5.6, there are four PS and PT concrete bridges in the database built after the issuance of Load and Resistance Factor Design provisions by AASHTO in 1994 that were negatively affected by the application of platoon loads. These structures produced rating factors of less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service III – Concrete Tensile Stress. These structures are summarized in Table 34.

Table 34. Bridges in BrR database used to establish Case 5.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
10	Unknown	PSSS-0553-PSIB	PS I Beam	2008	143.15	Unknown
14	Unknown	PSSS-0613-PSIB	PS I Beam	2008	143.15	Unknown
15	Unknown	PSMS-1387-PSIB	PS I Beam	2007	134.21	Unknown
17	AL	021151	PS I Beam	2018	137.98	HL-93

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HL-93 loading for spans 180 ft or longer, as shown in Table 18. However, Table 34 contains bridges with spans between 137.98 ft and 143.15 ft. According to the year built and the design load of the affected structures described in Table 34, these would have been designed for at least HL-93 loading. Therefore, the criteria shown in Table 35 were established for Case 5 bridges.

Table 35. Filtering criteria for Case 5 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≥ 1998	1998 or newer
31	Design Load	0, A, B	Unknown, HL 93, Greater than HL 93
43A	Structure Type, Kind of Material and or Design	5, 6	PS/PT Concrete, PS/PT Concrete Continuous
48	Length of Maximum Span in Meters	≥ 00365	120 ft or longer

2.7.6 Case 6: Reinforced Concrete Bridges Designed before Modern Crack Control Provisions

Case 6 applies to reinforced concrete (i.e., non-tensioned) bridges. Based on the results described in Section 2.5.6, there is one reinforced concrete bridge in the database built before the issuance of Load and Resistance Factor Design provisions by AASHTO in 1994 that was negatively affected by the application of platoon loads. Crack control provisions appeared in AASHTO provisions between 1977 and 1992; However, as the particular implementation year has not been found yet, it is assumed that structures designed after 1992 are designed for crack control. The structure produced rating factors of less than 1.0 for the platoon loading that were lesser than the rating factors produced by the HL-93. The controlling limit state was Service I – Reinforcement Yielding. The structure is summarized in Table 36.

Table 36. Bridges in BrR database used to establish Case 6.

Bridge Number	State	Bridge Name	Type	Year	Max. Span Length (ft)	Design Load
33	IN	NBI=005165 (CRCG)	RC T Girder	1958	101.5	HS 20

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate larger demands than HS 20 loading for spans 80 ft or longer, as shown in Table 18. According to the year built and the design load of the affected structure described in Table 36, this would have been designed for at least HS 20 loading. Therefore, the criteria shown in Table 37 were established for Case 6 bridges.

Table 37. Filtering criteria for Case 6 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≤ 1992	1992 or older
31	Design Load	0, 1, 2, 3, 4, 5, 6, 9	Unknown, H 10, H 15, HS 15, H 20, HS 20, HS 20 + Mod, HS 25 or greater
43A	Structure Type, Kind of Material and or Design	1, 2	Concrete, Concrete Continuous
48	Length of Maximum Span in Meters	≥ 00243	80 ft or longer

2.7.7 Case 7: Steel Bridges with Welded Primary Steel Tension or Flexural Members

The primary purpose of the filtering criteria is to establish steel bridges that have potential for their fatigue life to be reduced because of truck platooning. In the selection of filtering criteria, first the different provisions regarding fatigue evaluation and fatigue-related phenomena in design specifications and other policy were reviewed:

- In 1957 AASHTO (AASHO at the time until 1973) started allowing welding in primary members. Until then, primary members had to be built-up using fasteners, typically rivets. Welding was allowed in secondary members prior to that date.
- AASHTO's first fatigue design provisions were introduced in 1965. These were based on stress ratios and Goodman diagrams, which are not used in current fatigue evaluation procedures for bridges.
- From 1967 to 1978 AASHTO, AWS (American Welding Society) and FHWA developed and established the first Fracture Control Plan. The provisions are currently housed in the NBIS (National Bridge Inspection Standards, 23 CFR 650) and throughout the AASHTO specifications.
- AASHTO introduced the basis of the current design methodology for fatigue in 1974. The methodology was, and still is, based on the computation of effective fatigue stress cycles to different fatigue-prone detail categories.
- AASHTO introduced in 1994 the first LRFD specifications, in which the live load model for fatigue evaluation was modified.
- AASHTO introduced in 2009 the two load combinations currently used for fatigue evaluation. It effectively separated infinite-life and finite-life evaluations, although both segments of fatigue resistance were considered previously.
- AASHTO increased the load factors used in fatigue evaluation in 2016.

Further review of the effectiveness of these changes on the design for load-induced fatigue revealed that:

- Although the fatigue design methodology of 1965 is considered obsolete, the results of an evaluation by the provision of 1965 would typically be similar or more conservative than the results of an evaluation by the provisions of 1974.
- From 1974 until now, the methodology has not changed significantly. Different adjustments to the live load model used, load and resistance factors, live load distribution, and other variables

specific to the distinct fatigue-prone detail categories have been implemented. However, the resulting evaluations do not differ significantly from one version of the specifications to another.

- In 1986 results from the investigation by Keating and Fisher corroborated the AASHTO approach albeit small changes to the design variables used particular to each fatigue-prone detail category.

- The most significant changes in the specifications are related to fatigue development due to distortion and constraint. These damage mechanisms are to be avoided and are handled through detailing.

- While the load factor used in fatigue evaluation has increased in 2016, the number of existing bridges in the NHS (National Highway System) designed by the most recent provisions would not exceed 2,000, which is less than 1.5% of the NHS bridge inventory. Prior to 2016 infinite fatigue life was checked against 1.5 times the effective stress cycle generated by the AASHTO LRFD Fatigue Truck.

Consequently, it can be concluded that (1) prior to 1957 fatigue susceptibility was lesser due to the prohibition of weldments in primary members; and (2) the resulting designs from the different provisions for the evaluation of load-induced fatigue are similar. Therefore, after including a time allowance to allow implementation of the specifications, steel bridges built after 1960 with a maximum moment ratio for a truck platoon configuration with respect to the AASHTO LRFD Fatigue Truck larger than 1.5 are expected to experience fatigue life reduction due to truck platooning.

Based on the results of the line girder study summarized in 2.3.5, platoon loading will generate demands that are 1.5 times larger than the AASHTO LRFD Fatigue Truck for spans 80 ft or longer, as shown in Table 18. The 1.5 factor is chosen as it is the load factor prescribed by the LRFD Spec. for the evaluation of infinite fatigue life until 2016. However, the load effect that typically controls fatigue design is positive flexure, as the largest fatigue stresses in longitudinal steel members regularly occur in bottom flanges near mid spans. Consequently, when focusing on positive flexure, only spans 200 ft or longer are expected to undergo positive flexural demands due to platoon truck loading that are 1.5 times larger than the AASHTO LFRD Fatigue Truck. Similarly, the assessment of the effects of truck platooning on finite fatigue life in 2.4 showed that for spans measuring 200 ft or longer it is more likely that truck platooning results in a finite fatigue life reduction. Therefore, the criteria shown in Table 38 were established for Case 7 bridges.

Table 38. Filtering criteria for Case 7 bridges.

Item	Item Description	Code	Code Description
27	Year Built	≥ 1957	1957 or newer
43A	Structure Type, Kind of Material and or Design	3, 4	Steel, Steel Continuous
48	Length of Maximum Span in Meters	≥ 00609	200 ft or longer

2.8 Summary of Tier 1 – Deterministic Analysis

The Tier 1 – Deterministic analysis comprised the development and completion of the following subtask:

- Determination of platoon configurations to be considered.
- Computation of shears and moments generated by truck platoons relative to design loads using line girder analysis.
- Evaluation of truck platooning effects on finite fatigue life.
- Estimation of truck platooning effects on service limit states using a database of BrR models.
- Development of a Bridge Inventory Selection Tool (BIST) to perform affected bridge counts using NBI data.
- Development of filtering criteria to be applied to NBI data to estimate the number of bridges which service or fatigue limit states may be affected by truck platooning.

After completion of the tasks describe above the following conclusions were drawn:

- The platoon configurations used for the deterministic analysis are composed of 2, 3, or 4 identical trucks with headways distances of 30 ft, 50 ft, or 70 ft. The trucks forming the platoons are Type 3S2, Type 3-3, or typical WIM- 5-axle truck.
- Span length study found that:
    - The platoon configurations studied would generate shear and/or moments exceeding those generated by HL-93 loading in spans equal to or longer than 140 ft.
    - In terms of HS-20 loading, truck platoons generate larger shear and/or moments in spans equal or longer than 80 ft. Spans shorter than 80 ft were not considered because they would not accommodate more than one truck;

  - Relative to 1.5 times the positive flexural demand generated by the AASHTO LRFD Fatigue Truck, the truck platoons considered generate larger demands in spans equal or longer than 200 ft.

  - Increasing headway distances results in lesser demands, particularly for longer spans. Platoons with headway distances of 70 ft did not generate demands larger than HL-93 loading and, therefore, are not studied in the Tier 2 – Probabilistic Analysis.

  - The critical platoon configuration includes 3 truck platoons spaced 30 ft apart, and 4 truck platoons spaced 30 and 50 ft apart. Therefore, platoons with 2, 3, and 4 trucks, and headway of 30 ft. and 50 ft. will be considered in WIM data analysis Chapter 3 to determine statistical parameters of platoon live load.

- The evaluation of the effects of truck platooning on finite fatigue life found that:

  - Platooning is detrimental to finite fatigue life of bridge with longer spans. Fatigue life reductions are expected for spans measuring 200 ft or longer, although finite life reductions could occur down to 150 ft. Spans longer than 350 ft showed finite fatigue life reductions for all cases considered.

  - Platooning is not expected to be detrimental to the overall fatigue performance in spans measuring 200 ft or shorter. No finite life reductions were computed for spans measuring 140' or shorter.

  - While truck platoons increase the number of effective stress cycles per passage, this effect is only significant when the effects of span continuity are greater, and the overall fatigue damage is less. Therefore, the current AASHTO provisions regarding number of effective stress cycles per passage are adequate when evaluating truck platooning.

- Batch load rating analysis of the BrR database revealed that:

  - 235 out of 247 bridges in the database for which "Service I – Tensile stresses in reinforcing and/or prestressing steel" (MBE (23 CFR 650.317) 6A.5.4.2b) is applicable resulted in lower rating factors for truck platoons relative to HL-93.

  - 35 out of 869 bridges for which "Service II – Permanent deformation in steel flexural members" (MBE (23 CFR 650.317) 6A.6.4.2.2, analogous to LRFD Spec. (23 CFR 625.4(d)(1)(v)) 6.10.4.2 and 6.11.4) is applicable resulted in lower rating factors for truck platoons relative to HL-93.

- 28 out of 218 bridges for which “Service III - Tensile stresses in concrete at service limit state after losses (MBE (23 CFR 650.317) 6A.5.4.2a, analogous to LRFD Spec. (23 CFR 625.4(d)(1)(v)) 5.9.2.3.2b)” is applicable resulted in lower rating factors for truck platoons relative to HL-93.

- Truck platoons generally generate larger flexural demands than the AASHTO LRFD Fatigue Truck that result in relatively larger fatigue life reduction factors. However, these are not directly equivalent to a reduction of the expected fatigue life of the bridge, rather, they indicate the relative impact for a single truck passage.

- 33 out of 1519 bridges resulted in service limit states rating factors for platoons lower than 1.0 and lower than those computed for HL-93. This is a relatively low portion of the BrR database and indicates that truck platooning would have limited effect on service limit states relative to HL-93 loading.

- Based on findings from span length study, evaluation of finite fatigue life and batch load rating of the BrR model database, seven distinct cases were developed and implemented in the Bridge Inventory Selection Tool (BIST):

  - Case 1: Steel Bridges Designed before Load Factor Design Provisions. See Table 27 for Case 1 criteria and Table 49 for counts of bridges meeting Case 1 criteria.

  - Case 2: Steel Bridges Designed after Load Factor Design Provisions. See Table 29 for Case 2 criteria and Table 50 for counts of bridges meeting Case 2 criteria.

  - Case 3: Prestressed and Post-tensioned Concrete Bridges Designed per Allowable Stress Design Provisions. See Table 31 for Case 3 criteria and Table 51 for counts of bridges meeting Case 3 criteria.

  - Case 4: Prestressed and Post-tensioned Concrete Bridges Designed per Load Factor Design Provisions. See Table 33 for Case 4 criteria and Table 52 for counts of bridges meeting Case 4 criteria.

  - Case 5: Prestressed and Post-tensioned Concrete Bridges Designed per Load and Resistance Factor Design Provisions. See Table 35 for Case 5 criteria and Table 53 for counts of bridges meeting Case 5 criteria.

  - Case 6: Reinforced Concrete Bridges Designed before Modern Crack Control Provisions. See Table 37 for Case 6 criteria and Table 54 for counts of bridges meeting Case 6 criteria.

- Case 7: Steel Bridges with Welded Primary Steel Tension or Flexural Members. See Table 38 for Case 7 criteria and Table 55 for counts of bridges meeting Case 7 criteria.

- Results from filtering the NBI data with the Bridge Inventory Selection Tool (BIST) are included in Appendix A.

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Chapter 3: Tier 2 – Probabilistic Analysis

3.1 Methodology

This portion of the study intends to investigate the impact of truck platoons for bridge strength and serviceability by using a fully probabilistic methodology and based on actual recorded WIM data. Specifically, the impact of platoons will be reflected by three criteria: 1) statistical live load models for the traffic fleet with and without truck platooning, 2) the number of platoons that contribute to the greatest load effects, and 3) load effects at 1 in 10,000 exceedances for the traffic fleet with and without truck platooning for fatigue evaluation. The research work to achieve this objective included the following steps:

1. Select representative WIM sites across the nation.

2. Filter WIM data to exclude the erroneous records and permit-like overweight trucks.

3. Develop the algorithm to simulate the WIM data including truck platooning. Due to the limited real-world deployment of the truck platoon on highways, the analysis of the traffic fleet with the truck platoon primarily relies on simulations, namely "synthetic WIM data."

4. Develop line-girder analysis algorithms to compute load effects caused by single trucks and platoons. By doing so, the upper tail load effects on the corresponding truck type (single truck vs. platoons) can be identified.

5. Develop the statistical live load model for the traffic with and without truck platooning. The statistical live load model provides essential information to calculate the bridge reliability indices. To perform the probabilistic calibration, the statistical live load model includes:

    a. Mean-to-nominal ratio (i.e., bias factor) that describes how large the future maximum load effect is.

    b. Coefficient of variation (COV) that describes the level of dispersion of the future maximum load effect.

6. Obtain the load effects at 1 in 10,000 exceedances (NASEM 2014). This step aims to evaluate the impact of truck platoons on fatigue design.

In the later sections, each one of the above-mentioned steps will be illustrated in more detail.

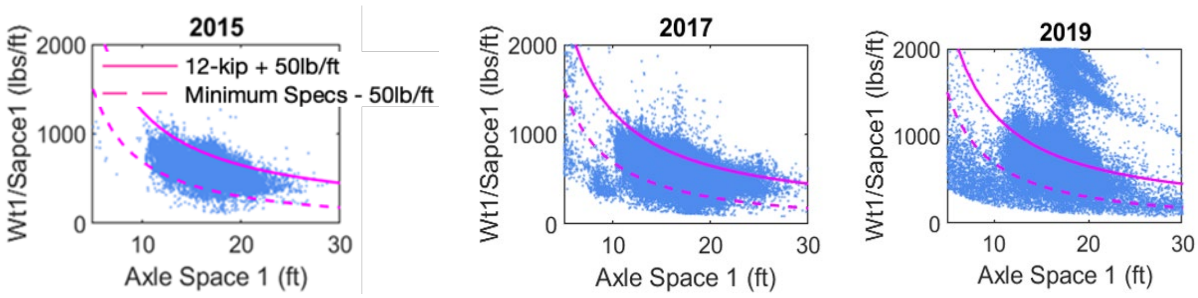
3.2 WIM Site Selection

The live load statistical model depends greatly on the actual traffic scenarios. The WIM technique that captures the traffic data is used to develop the load model in this study. The selected WIM sites are expected to meet certain criteria to provide desired results, as will explained in this section.

- **Comprise geographical variation.** This study aims at developing a nationally representative live load model for the traffic fleet with and without truck platooning. Therefore, it is necessary to consider the traffic variation in different geographical locations. Six states were selected across the nation, including California, New Jersey, Indiana, Texas, Florida, and Alabama.

- **Provide good-quality data**. WIM data from over a hundred of the sites were received from the six states, whereas only parts of the sites are to be used for the analysis after the validation and quality control. The selected WIM sites should provide good data quality to ensure the reliability of the truckload analysis, so quality assurance was performed to avoid using the WIM data that are out of calibration. The WIM data validating methodology developed by Southgate (2000) was applied, in which the front axle weights (FAW) of FHWA Class 9 truck (3S2, semi-tractor trailer) are regressed to first axle spacing. Southgate (2000) used these Class 9 3S2 type trucks not only because they are common commercial trucks but also because they have a stable relationship between the FAW and the steering axle spacing. This technique established a logarithmic relation between the first axle spacing (S12) and the ratio between steering axle weight and the first axle spacing (A1/S12). The upper bound is formed by the 12-kip practical weight limit, and the lower bound is sourced from truck manufacturers' minimum specifications. If the regression curve of all Class 9 falls within the upper and the lower boundary and is close to the reference equation, then the quality of WIM data is acceptable.

  The Southgate algorithm has been applied to all the WIM sites, and the data quality check was performed for the annual data rather than all-year data together. WIM systems, regardless of different sensor techniques, require periodic calibration. Therefore, it is possible to observe fluctuation of the WIM data quality from the same site in different years. Figure 30 shows typical plots for a set of good-quality, median-quality, and poor-quality data based on Southgate (2000). In this example, the WIM data from the Alabama Site 961 in different years were shown. It can be observed that as the WIM system has served for years, the collected data have gradually become out of calibration. In this case, only the data in the year 2015 which exhibited good data quality is selected for the analysis.



Good Data Quality **Medium Data Quality** **Poor Data Quality**

Figure 30. Typical data quality demonstration (Alabama site 961).

- **Cover full-year data.** To capture seasonal variation, it is desired to have a full year of data from each site.

Taking all three criteria into consideration, 32 WIM sites were selected to perform the live load analysis. The WIM sites information is tabulated in Table 39.

Table 39. Selected WIM sites.

State	Site	Year	Average Daily Filtered Truck Traffic
NJ	78A	2016	13694
	295	2016	4718
	040	2015	882
	540	2016	213
Indiana	041000	2021	936
	272300	2021	7881
	303600	2019	13153
	695200	2021	566
	746200	2021	1667
	826500	2019	1538
Texas	0WT506	2020	4136
	0WT526	2020	7517
	0WT531	2020	7246
	0WT541	2020	263
	0WT545	2020	1641
	0WT546	2020	723
Florida	9907	2015	898
	9909	2015	422
	9914	2015	7631
	9918	2015	2364
	9947	2015	3666
	9950	2015	5057
Alabama	915	2016	912
	934	2015	397
	942	2015	2063
	961	2017	3540
	963	2018	7589
	965	2016	4911
California	073	2015	7775
	077	2017	9110
	108-1	2015	3036
	108-2	2015	3000

3.3 WIM Data Processing

3.3.1 Synthetic WIM Data

Given the truck platoon's limited deployment on actual highways, the analysis of the traffic fleet relies on simulations utilizing "synthetic WIM data". These synthetic WIM data are derived from actual truck statistics obtained from collected WIM data, wherein the trucks' order is rearranged and 3S2 trucks (5-axle Class 9 trucks) are manipulated to travel as platoons with specific headway distances.

3S2 trucks can be identified from WIM data based on the truck configuration. Per FHWA's definition, 3S2 trucks are the trucks that conform to the following criteria (FHWA, 2014):

1. FHWA Class 9 vehicle
2. Spacing between axle 1 and 2: 6.00 ft – 30.00 ft
3. Spacing between axle 2 and 3: 2.50 ft – 6.29 ft
4. Spacing between axle 3 and 4: 6.30 ft – 65.00 ft
5. Spacing between axle 4 and 5: 2.50 ft – 11.99 ft

Figure 31 presents the typical axle spacing distributions for 3S2 trucks, using California Site 073 as an example. Axle S12 represents the spacing between the 1st and 2nd axles, with similar definitions for Axle S23, Axle S34, and Axle S45. The distribution is illustrated by plotting the axle spacing against the Normal Inverse value, which corresponds to the z-score derived from the cumulative distribution probability. Based on the figure, 3S2 trucks exhibit two major configurations: one with rear tandems and another with two rear single axles. Similar trends are observed at other sites. Additionally, approximately 90% of 3S2 trucks feature a rear tandem. As a result, the majority of truck platoons are formed using 3S2 trucks with rear tandem configurations.

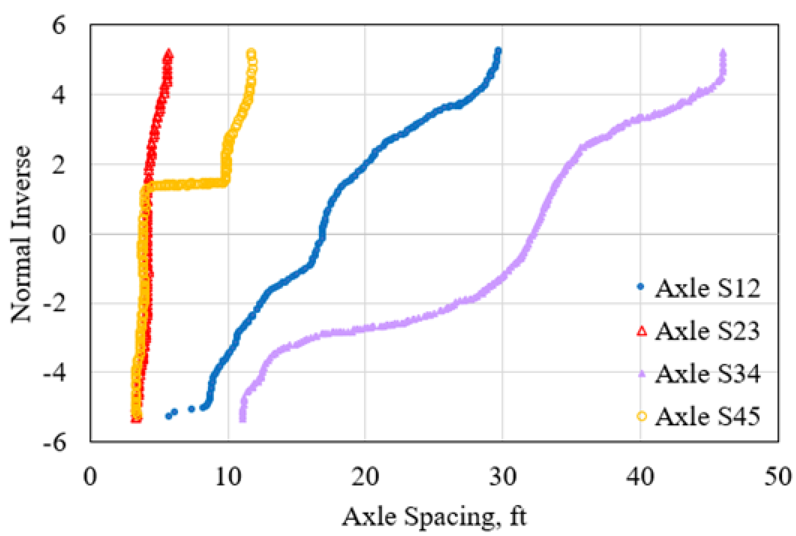


Figure 31. 3S2 Truck axle spacing distributions (California site 073).

Being identified from WIM data, 3S2 trucks were treated as platoon candidates. For the certain penetration rate, the platoon candidates were subdivided into two groups. Trucks in the first group travel as platoons, and trucks in the other group travel individually as usual. Eventually, the synthetic WIM data contains three groups of trucks: 1) truck platoons, 2) rest of the platoon candidates, and 3) non-candidate regular truck traffic. Both heavy and light trucks are deemed candidates for forming the truck platoons; however, based on more realistic considerations, an assumption is made to cap individual truck GVW in a platoon to a maximum of 100-kip.

The statistical live load models were derived for 19 scenarios, including 1 actual traffic scenario without platoon and 18 synthetic traffic scenarios with platoons. The 18 platoon scenarios are composed by the following pre-determined configurations:

1. Penetration rate (p): 25%, 50%, and 100%
2. Number of trucks in a platoon (n): 2, 3, and 4
3. Headway distance (x): 30 ft and 50 ft

In summary, the procedure to produce the synthetic WIM data is visualized in Figure 32.

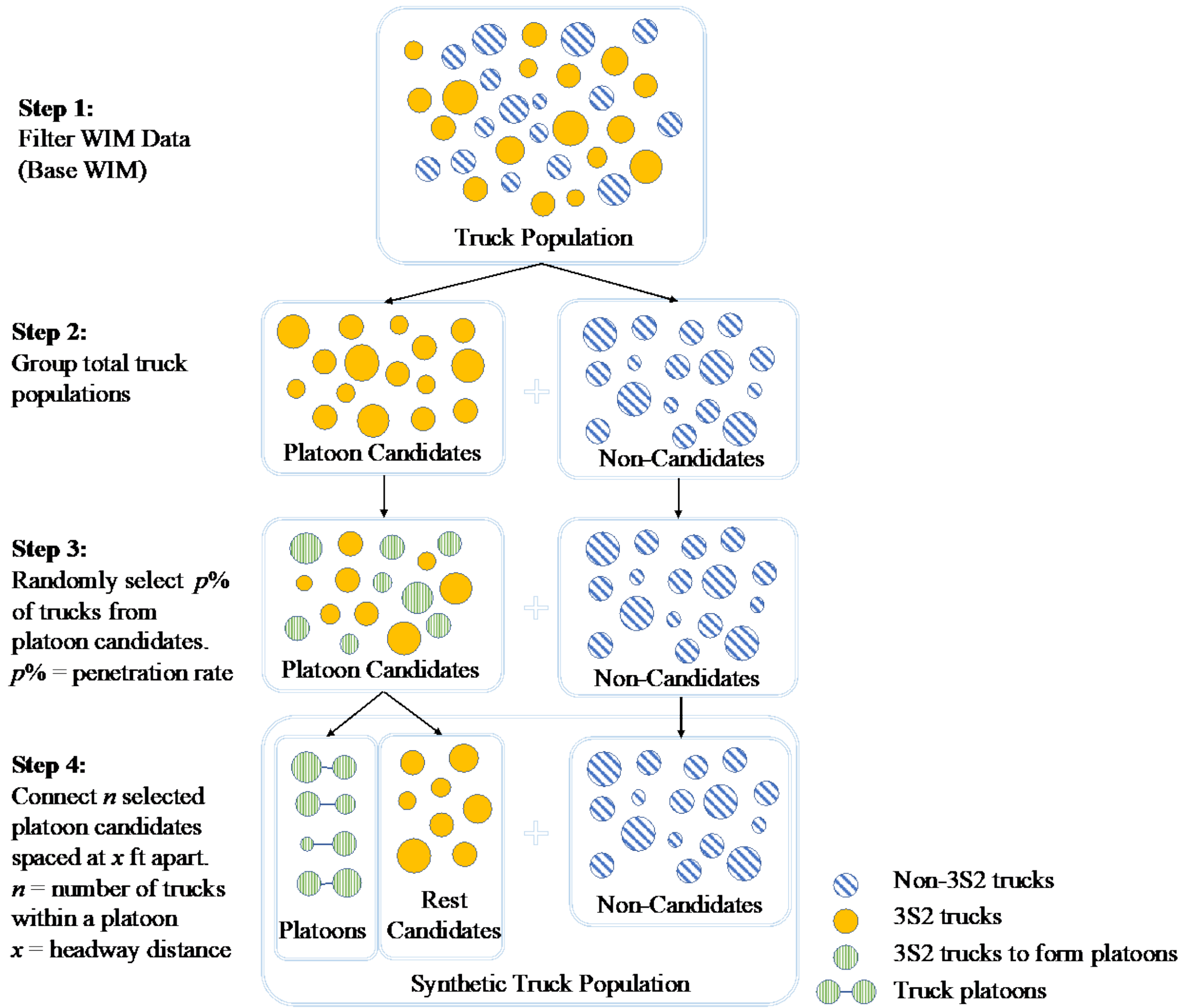


Figure 32. Schematic of synthetic WIM data.

In Chapter 3 and 4, the truck platoon scenarios are abbreviated by "number of trucks – headway – penetration rate". For example, "2-30-25" represents the synthetic WIM scenario where 2 trucks travel as platoons with a headway distance of 30 ft, and the penetration rate is 25%.

3.3.2 WIM Data Filtering

The WIM data includes possible measurement errors that need to be recognized in the data review process, such as extremely low GVW, unrealistic configuration, unreasonably fast speed, etc. Also, passenger cars whose weights are too light to impact the bridge are not of interest and should be filtered out. The filtering criteria developed by NCHRP Project 12-83 study are used to eliminate the erroneous and passenger car records, as summarized below (Wassef et al., 2014):

- Individual axle weight greater than 70 kips or less than 2 kips
- GVW less than 12 kips

- Total length greater than 120 ft or less than 7 ft
- First axle spacing less than 5 ft
- Individual axle spacing less than 3.4 ft
- Speed less than 10 mph or greater than 100 mph
- GVW+/- the sum of the axle weights by greater than 10%

In addition, to do the live load analysis for regular truck traffic, the permit or illegal overweight vehicles should be excluded from the traffic fleet. NCHRP Project 12-83 developed 3 filters to screen out these vehicles:

- Total number of axles less than 3 and GVW is more than 50 kips
- Steering axle weight more than 35 kips
- Individual axle weight more than 45 kips

However, it was found that these three filters alone often failed to eliminate the permit-like heavy vehicles, and therefore, the upper tail load effects were often governed by the heavy permit-like single trucks, nullifying the impact of truck platoons. Figure 33 shows the 5-year aggregated bias factor of the Base WIM (i.e., single trucks) and 3 truck platoon scenarios (i.e., 2-30-25, 3-30-25, and 4-30-25) to HL-93 load. It is observed that for the bridges spanning from 140 ft to 260 ft, the Base WIM exhibits higher bias factor than the truck platoon scenarios.

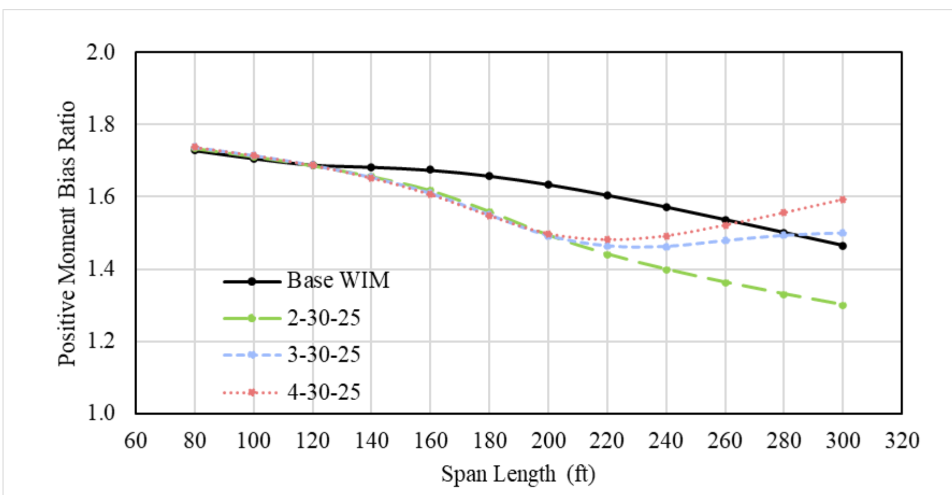


Figure 33. Aggregated bias factor with and without platoons by using NCHRP Project 12-83 Filter.

To illustrate this phenomenon in a microscopic view, the normalized load effect to HL-93 load of the Base WIM and one of the platoon cases (i.e., 4-50-100) of Florida site 9950 is presented in the normal probability paper, as shown in Figure 34. It is observed that the top tail of the platoon load effects overlaps with that of the single trucks, indicating that the heaviest load effects are caused by the single trucks. The configurations of these heavy single trucks are shown in Figure 35. Figure 35(a) shows that these trucks are extremely heavy with GVW of more than 135 kips, while Figure 35(b) illustrates that these trucks are all multi-axle trucks with 7 or more axles.

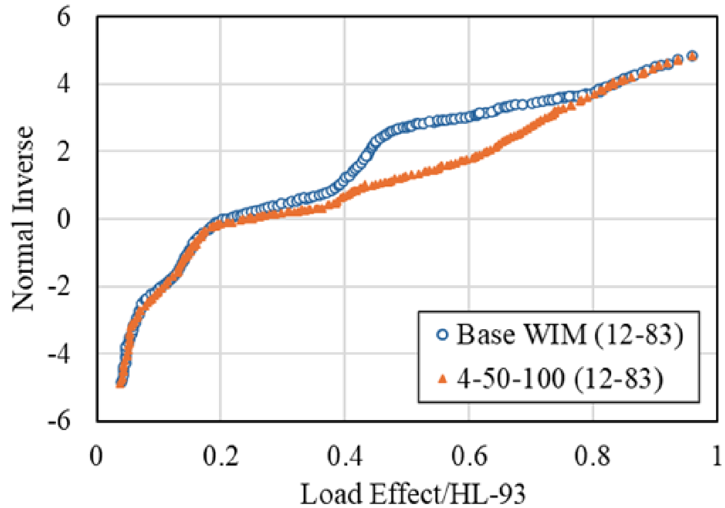


Figure 34. Normalized load effects of Base WIM and platoon (Florida site 9950, span length = 300 ft).

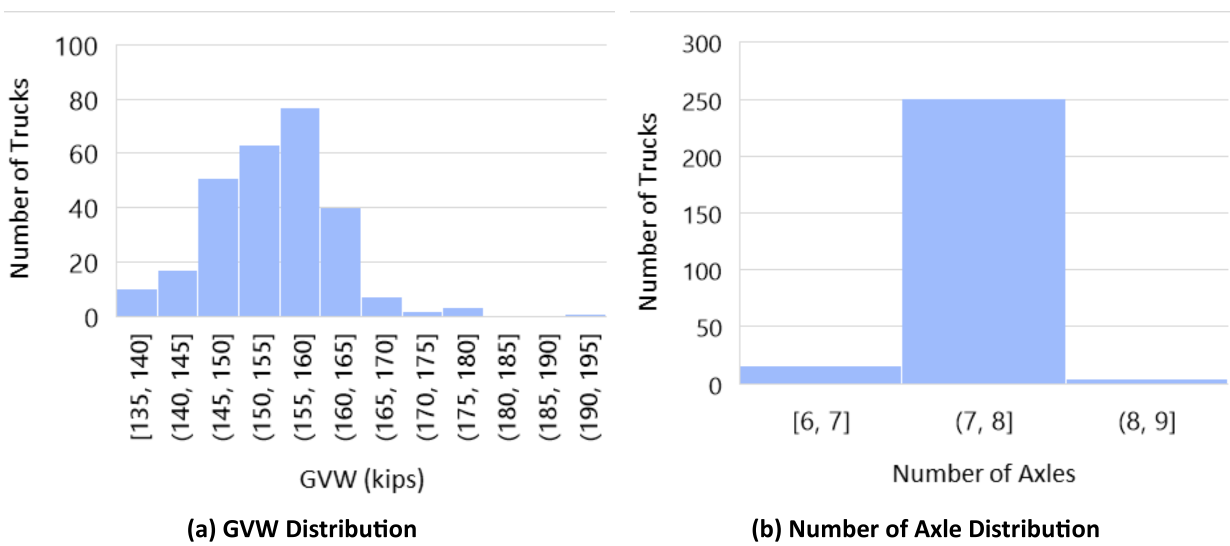


Figure 35. Configuration of heaviest single trucks (Florida site 9950).

Based on these findings, the RT has identified that permit-like heavy vehicles are often characterized as 7- or more-axle trucks. This conclusion is supported by NCHRP Project 12-76, where all 7- or more-axle trucks are considered as permit vehicles (Sivakumar et al., 2011). However, it is important to note that the number of axles alone does not provide a comprehensive indication of permit trucks. For instance, 7-axle specialized hauling vehicles (SHVs) are legally permissible trucks despite having 7 axles. To ensure that the filtering criteria do not penalize legal trucks, the RT combines the number of axles with a limitation on the GVW to capture overweight vehicles that resemble permit-like trucks.

The federal legal weight limit is 80 kips, but certain allowances need to be considered due to driver tendencies and measurement errors in WIM sensors. To determine a rational GVW limitation for overweight vehicles, a sensitivity analysis is conducted. Figure 36 shows the results of the sensitivity analysis where the x-axis represents the weight threshold beyond which 7- or more-axle trucks are excluded from the live load analysis, while the y-axis represents the extrapolated normalized load effects. Since 7-axle trucks tend to have longer wheelbases and can affect bridges with longer spans, the study presents the load effect ratios for spans of 160 ft, 200 ft, and 300 ft. The extrapolation results indicate stability until 7-axle trucks with GVW greater than 100 kips are excluded from the analysis. In other words, 7-axle trucks weighing over 100 kips significantly influence the upper-tail load effects, resulting in biased extrapolation results.

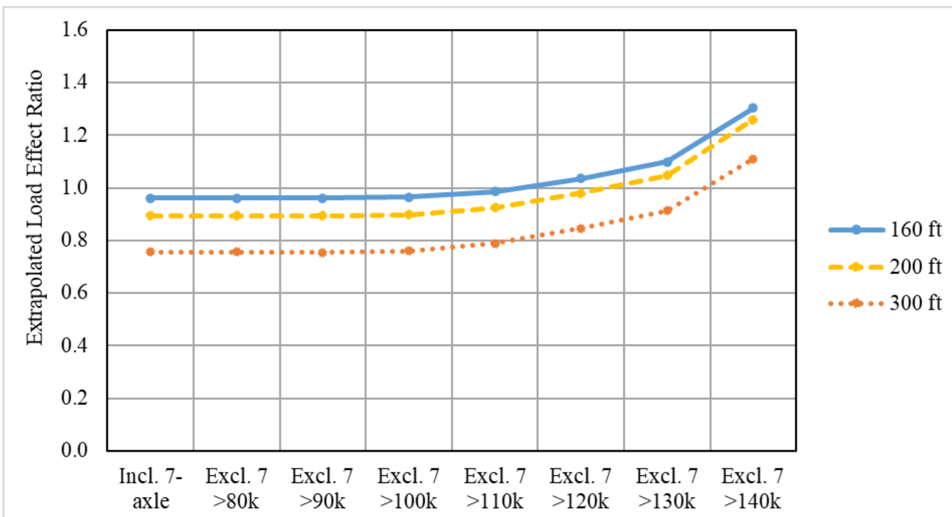


Figure 36. Sensitivity analysis of weight limitation for permit-like overweight vehicle.

Therefore, as an add-on to the NCHRP Project 12-83 filtering criteria, the study proposes an additional filtering criterion to identify permit-like overweight vehicles as follows:

- Total number of axles greater than 6 and GVW exceeding 100 kips.

In summary, Figure 37 shows the WIM data filters used in this study.

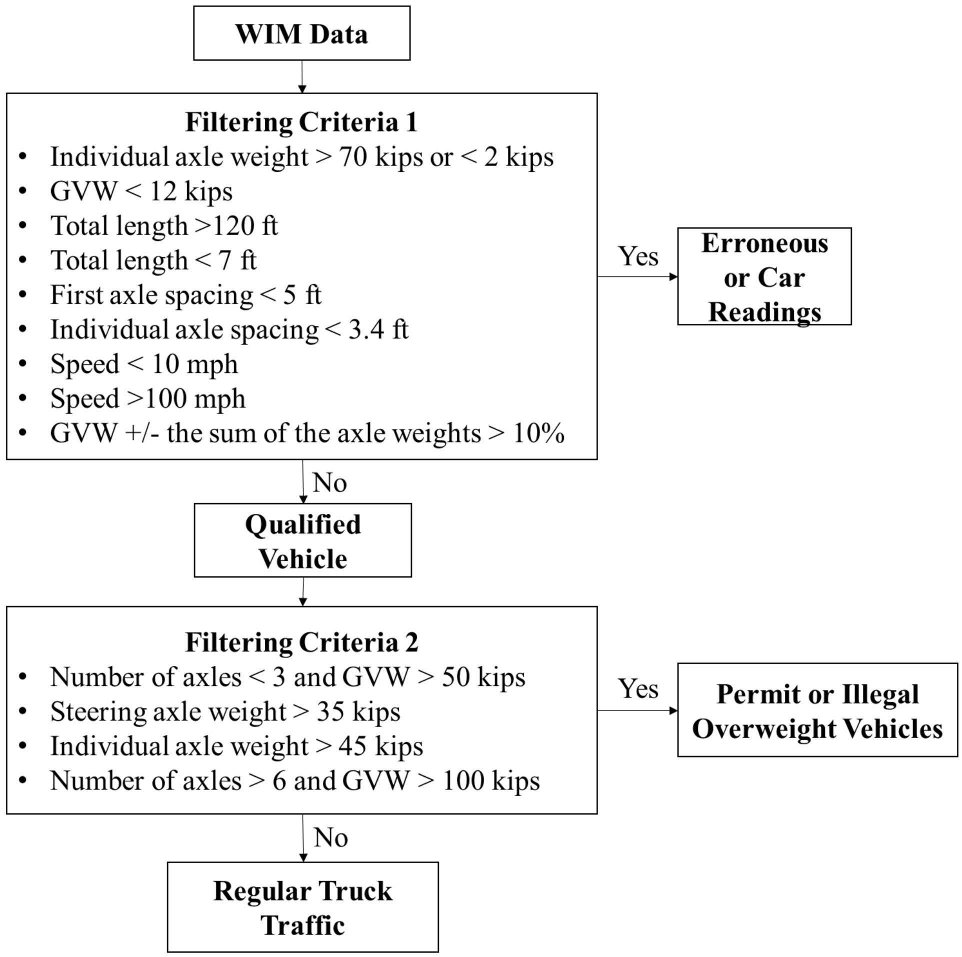


Figure 37. Revised WIM data filters based on NCHRP Project 12-83.

3.4 Development of Statistical Live Load Model

The load effects were calculated for a series of simple-span and two-equal-continuous-span bridges. The span lengths range from 80 ft up to 300 ft with an increment of every 20 ft. The load effects considered in this study include the following:1) simple span moment, 2) simple span shear, 3) continuous span positive moment, 4) continuous span negative moment, and 5) continuous span shear at interior support. Influence lines were developed to compute the maximum load effects caused by single trucks and truck platoons, as shown in Figure 38 through Figure 42.

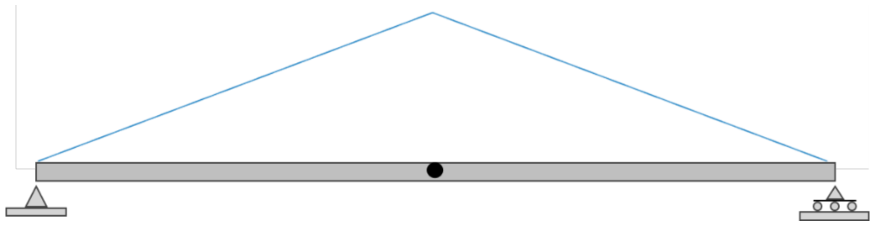


Figure 38. Influence line for simple span moment (M).

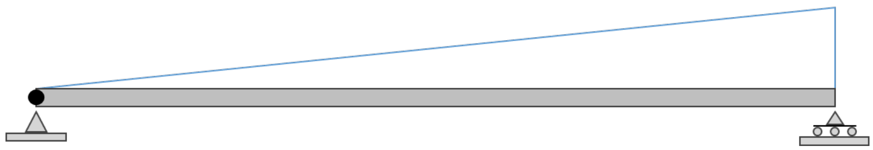


Figure 39. Influence line for simple span shear (S).

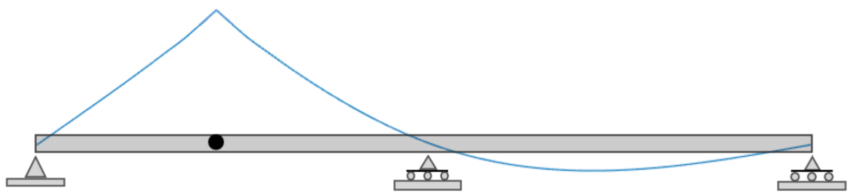


Figure 40. Influence line for continuous span positive moment (M+).

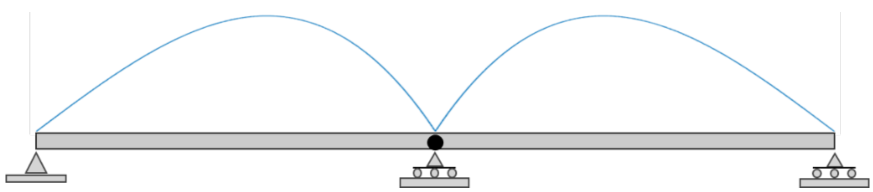


Figure 41. Influence line for continuous span negative moment (M-).

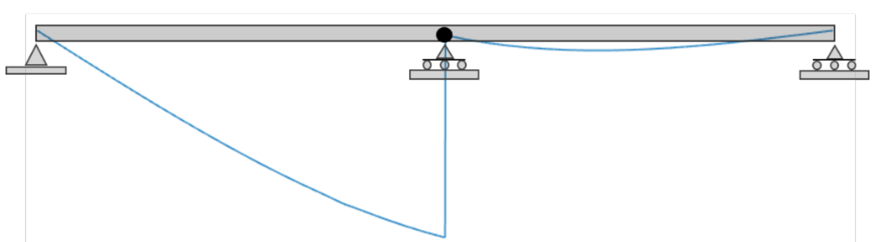


Figure 42. Influence line for continuous span shear (Sc).

3.4.1 Load Effect Extrapolation

As aforementioned, the selected WIM data covers a full year of records. For longer period, the maximum load effects are obtained by extrapolating the upper tail of the CDF (Cumulative Distribution Function) plotted on a normal probability paper (NPP).

The extrapolation method used in this study followed the approach from NCHRP Report 368 (Nowak, 1999). This method assumes that the upper tail of the maximum load effect over a given return period approaches a normal distribution, exhibited as a straight line in the NPP. The extrapolation is then performed by extending the straight line towards the desired return period. To be specific, the maximum live load ratio can be estimated based on the following procedure:

- Obtain the live load effects for a suite of simple and continuous spans by using influence line and normalize them by the load effects of HL-93 loading.
- Arrange the live load effect ratio, x, in an increasing order (x_1 is the smallest and x_n is the largest value)
- The probability, p, of the CDF, is calculated by Eq.11:

$$p_i = \frac{i}{X+1} \qquad \text{Eq. 11}$$

where X is the population size.

- The standard normal inverse value (i.e., z-score) can be transformed from the probability by using Eq. 12:

$$z = \Phi^{-1}[p] \qquad \text{Eq. 12}$$

where Φ^{-1} is the inverse of the standard normal distribution function.

- The total number of trucks in a 5-year return period, is:

$$N_{5-year} = ADTT \times 365 \times 5 \qquad \text{Eq. 13}$$

In this study, ADTT of 250, 1000, 2500, 5000, 10000, and 15000 were evaluated, yet ADTT of 5,000 was emphasized to be consistent with MBE (23 CFR 650.317) calibration.

- The corresponding cumulative probability to have the maximum load effects in a 5-year return period, p_{5-year}, is:

$$p_{5-year} = \frac{N_{5-year}}{N_{5-year} + 1} \quad \text{Eq. 14}$$

- The corresponding z-score for is:

$$z_{5-year} = \Phi^{-1}[p_{5-year}] \quad \text{Eq. 15}$$

- The NPP plot is made of the standard normal inverse (z) versus the load effect ratio (x). Following a normal distribution, the upper tail of the load effects exhibits as a straight line in the NPP so that the best-fit linear regression line can be made.

- The extrapolated maximum load effect ratio in a 5-year return period is represented by the x-axis of the intercept of the best-fit linear regression line and the z_{5-year} value.

The above-mentioned extrapolation method is applied for both single truck scenario and platoon scenarios. California site 073 is taken as an example to illustrate the method, as shown in Figure 43. The positive moment effects are computed for a span length of 300 ft. The single truck from Base WIM, “2-30-25”and “2-30-100” scenarios are included in Figure 43.

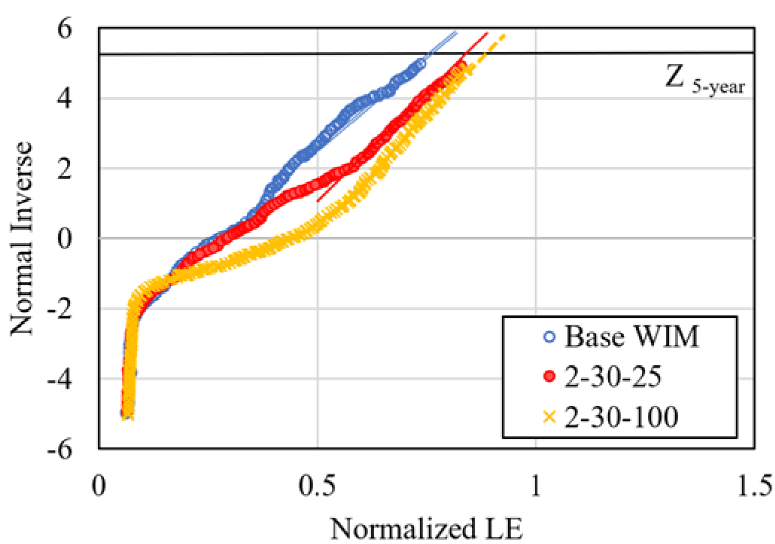


Figure 43. Example of load effect extrapolation (California site 073, span length = 300 ft).

3.4.2 Statistical Live Load Model

A statistical live load model consists of three components: the distribution type, bias factor (λ), and COV. The bias factor represents the ratio between the mean max load effect and the nominal load effect. It is determined by averaging the 5-year maximum load effect ratios across all selected sites for each span length. To clarify further, the bias factor for a specific scenario, such as the single truck scenario illustrated in Figure 44, is calculated by first plotting the 5-year maximum load effect ratios against the span lengths from all selected sites. Then, for each span length, the bias factor is obtained as the average of load effect ratios across all sites.

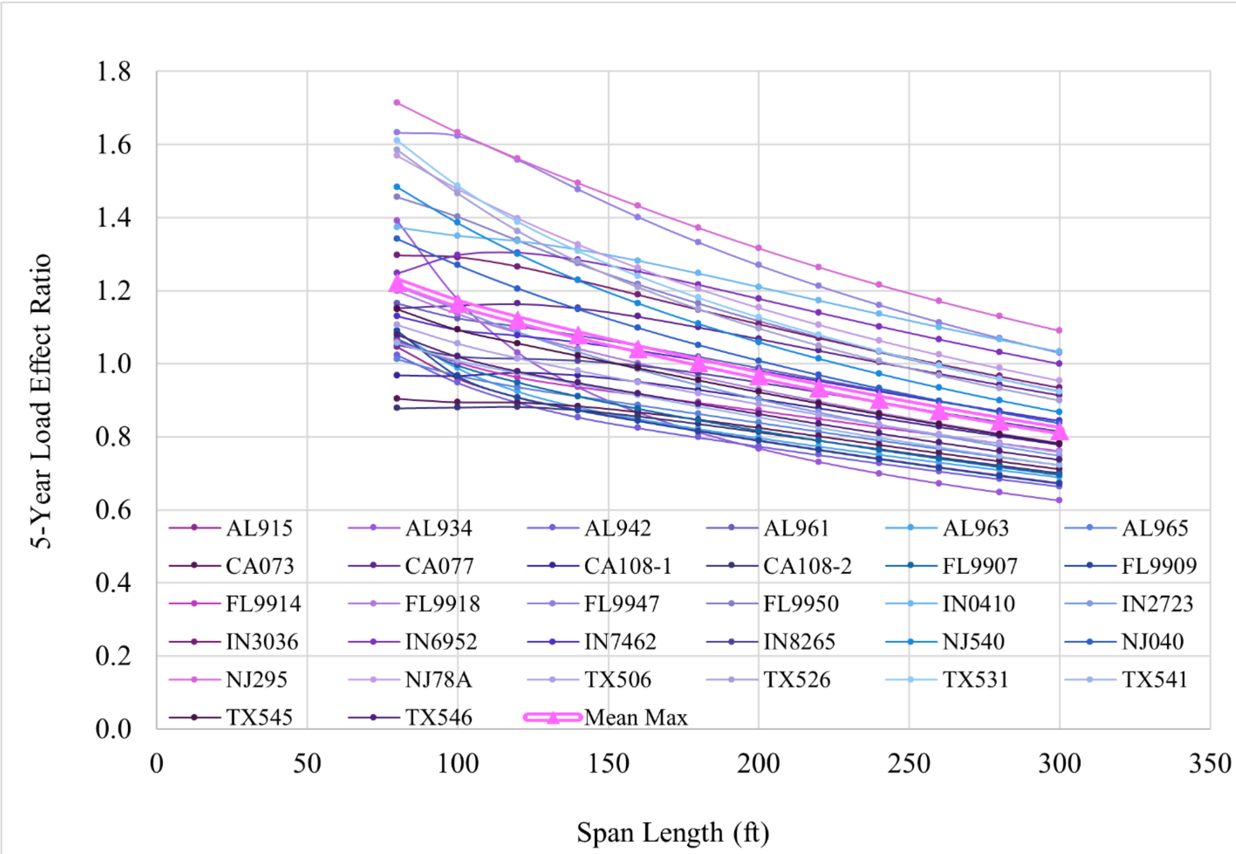


Figure 44. Illustration of bias factor (Base WIM scenario, ADTT = 5,000).

Figure 45 through Figure 49 show the variation of the bias factor with span length for simple span moment, simple span shear, continuous span positive moment, continuous span negative moment, and continuous span shear at interior support, respectively.

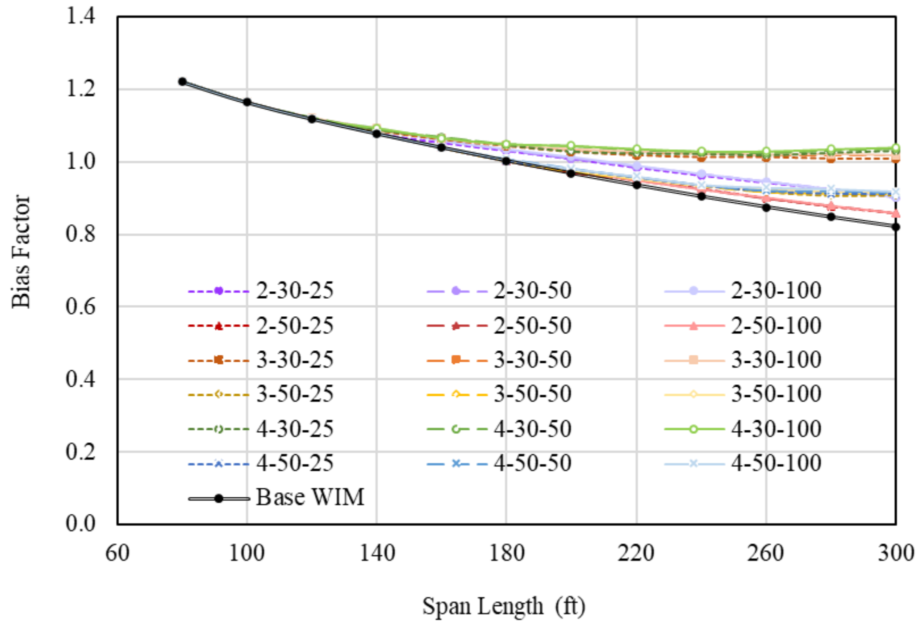


Figure 45. Bias factor for simple span moment (M).

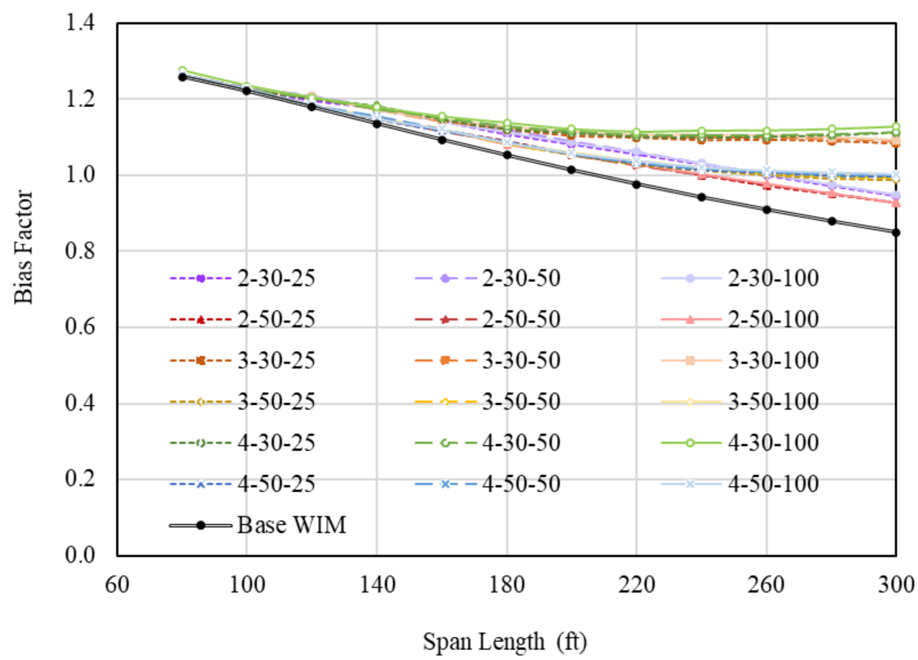


Figure 46. Bias factor for simple span shear (S).

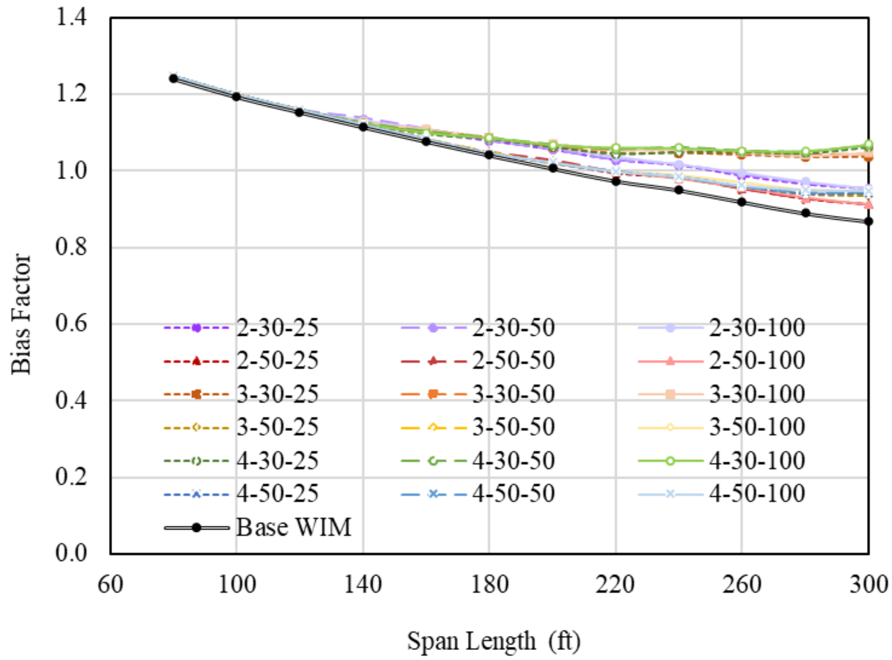


Figure 47. Bias factor for continuous span positive moment (M+).

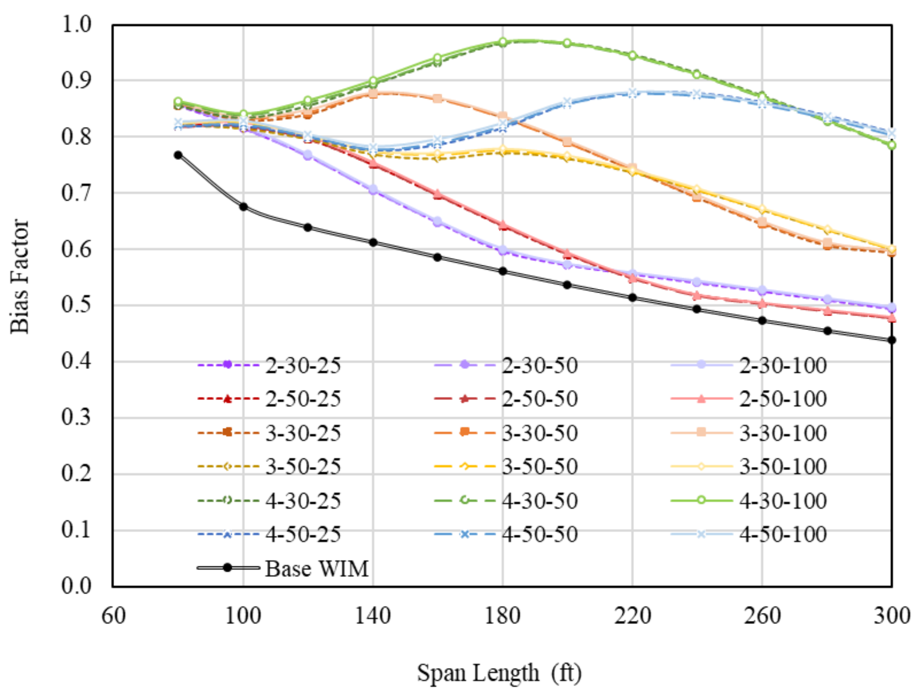


Figure 48. Bias factor for continuous span negative moment (M-).

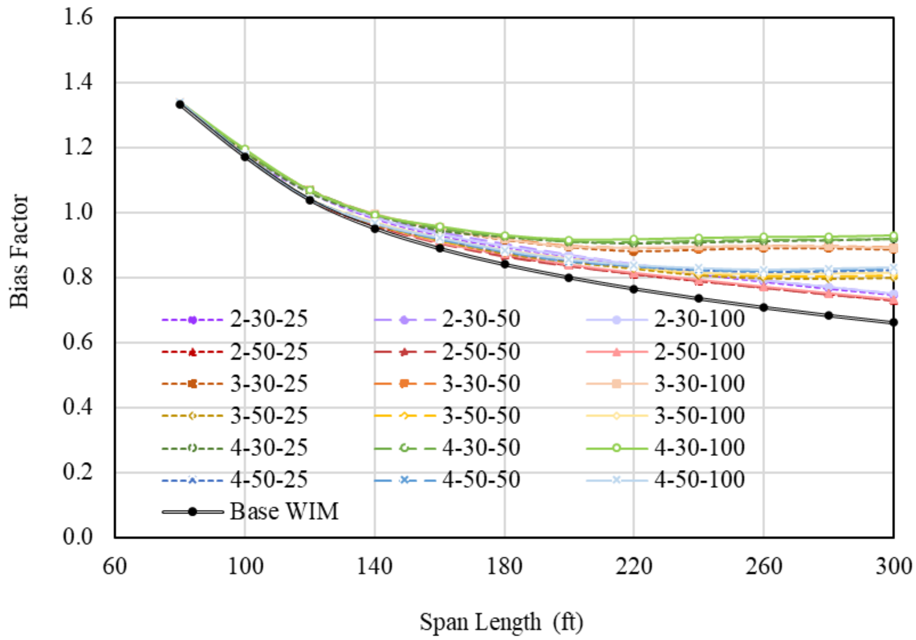
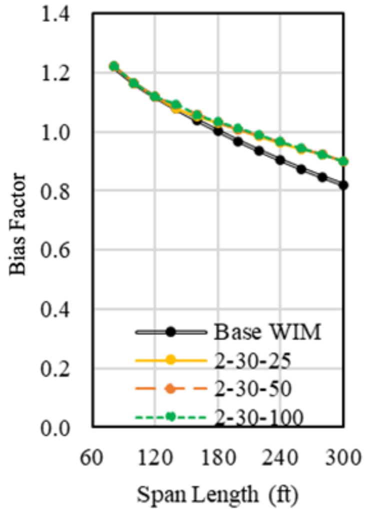


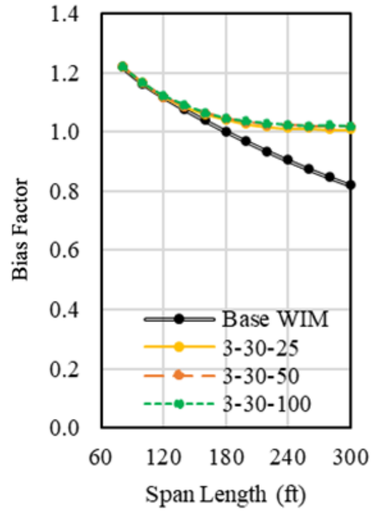
Figure 49. Bias factor for continuous span shear at interior support (Sc).

As outlined previously, the synthetic WIM data incorporates varying parameters such as penetration rates, headway distances, and the number of trucks within the platoons, thereby encompassing 18 distinct platooning scenarios. To comprehend the effect of each individual configuration and visualize the bias factors, a comparative study was conducted. The bias factors of different traffic scenarios were plotted on a singular graph to offer a unified view of the outcomes.

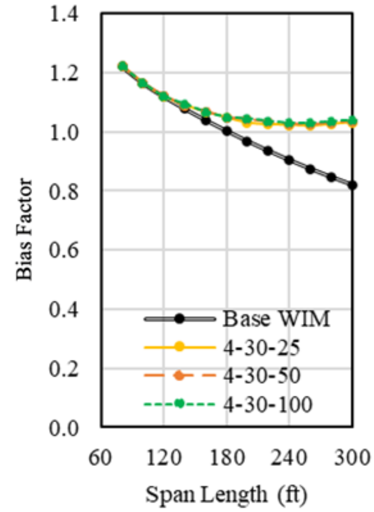
To scrutinize the influence of the penetration rate, Figure 50 presents six sets of figures. Each set maintains the same headway distance and number of trucks in a platoon. The closely spaced lines representing different platoon scenarios in each figure suggest that variations in the penetration rate appear to exert minimal impact on the bias factor. As a result, the inclusion of multiple penetration rates was deemed unnecessary for the next phase of the study. A penetration rate of 100% was suggested for use in the reliability calibration. This suggestion was based on the premise that it represents the most critical scenario, despite the marginal effects of the penetration rate.



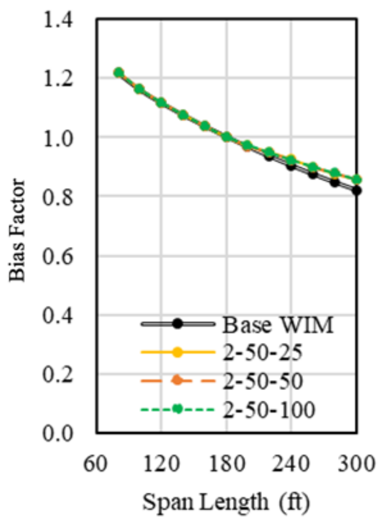
(a) 2-truck platoon with 30 ft headway



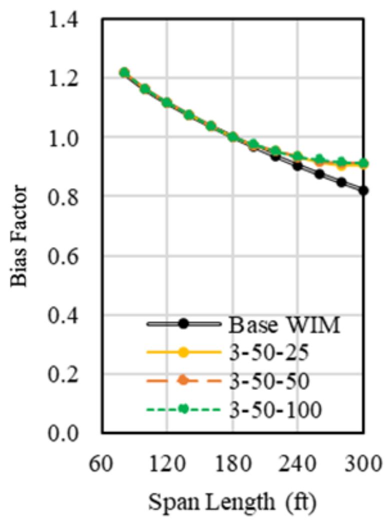
(b) 3-truck platoon with 30 ft headway



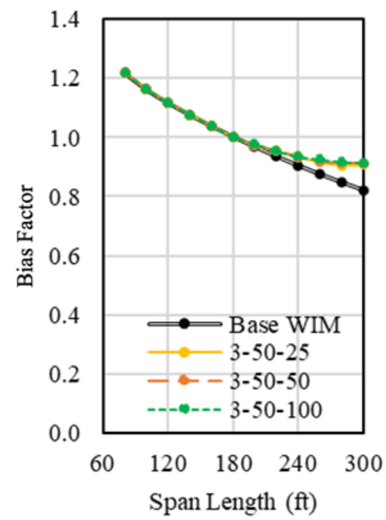
(c) 4-truck platoon with 30 ft headway



(d) 2-truck platoon with 50 ft headway



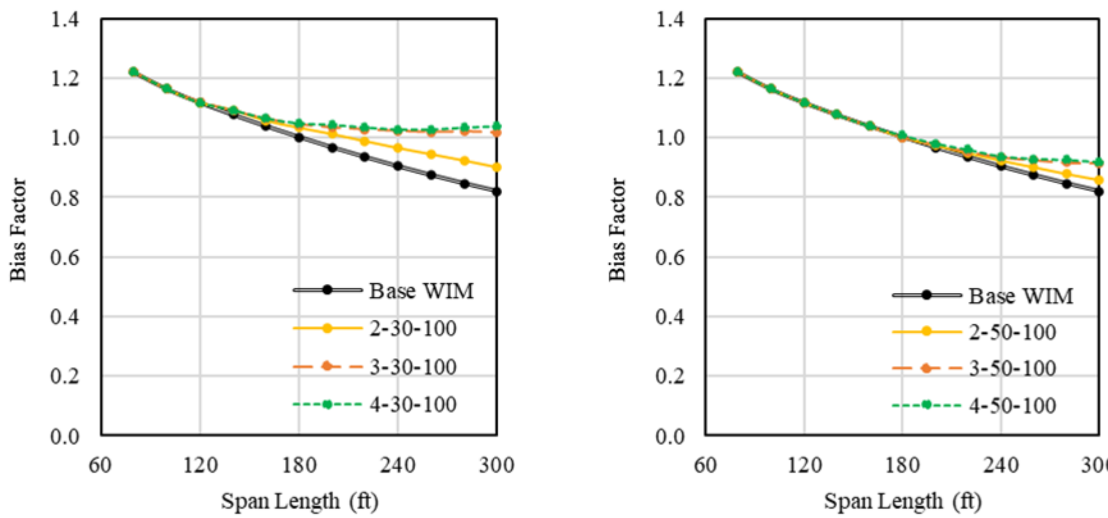
(e) 3-truck platoon with 50 ft headway



(f) 4-truck platoon with 50 ft headway

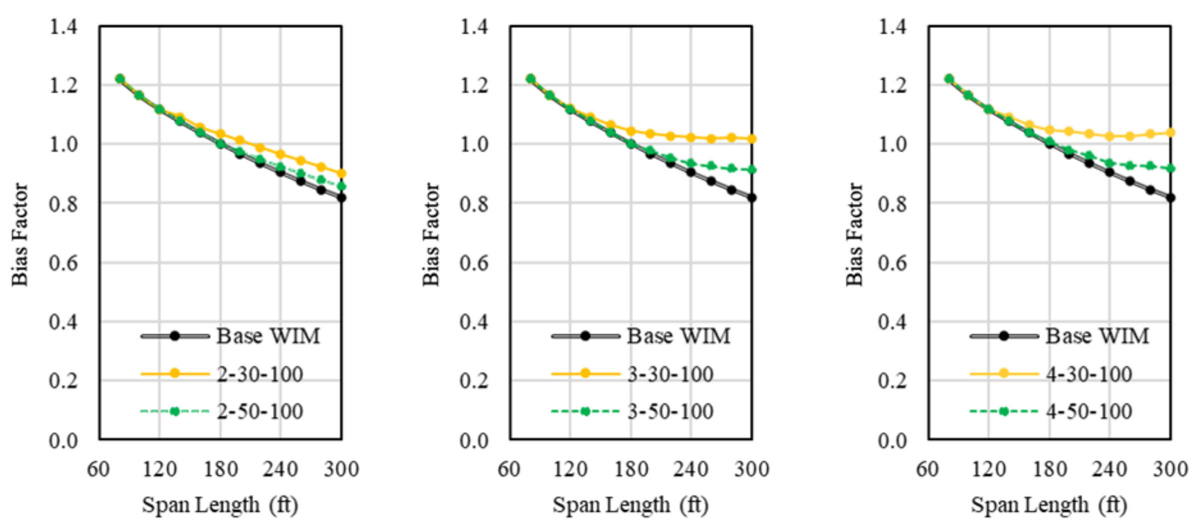
Figure 50. Effect of penetration rate for simple span moment.

The number of trucks in a platoon emerges as a key determinant of the bias factor, as illustrated in Figure 51. To clarify, the bias factor does not exhibit any significant discrepancy for span lengths shorter than 140 ft and 200 ft for headway distance of 30 ft and 50 ft, respectively. However, for spans exceeding these lengths, a positive correlation is identified between the number of trucks in the platoons and the bias factor. Additionally, Figure 52 shows that the headway distance is an influential parameter that affects the bias factor. A shorter headway distance is seen to yield a larger bias factor.



(a) Headway = 30 ft (b) Headway = 50 ft

Figure 51. Effect of number of trucks in a platoon for simple span moment.



(a) 2-truck platoons (b) 3-truck platoons (c) 4-truck platoons

Figure 52. Effect of headway distance for simple span moment.

As illustrated above, the bias factor calculated from the simple span moment was presented for illustration purpose, yet the same trends were found for the other 4 types of load effects. In summary, the influential parameters that affect the bias factor are found to be the number of trucks within a platoon and the headway distance. Based on these two parameters, 6 distinct platooning scenarios are suggested for consideration in the reliability calibration as follows:

1. 2-30-100: Two trucks in a platoon with a 30-ft headway and 100% penetration rate

2. 2-50-100: Two trucks in a platoon with a 50-ft headway and 100% penetration rate

3. 3-30-100: Three trucks in a platoon with a 30-ft headway and 100% penetration rate
4. 3-50-100: Three trucks in a platoon with a 50-ft headway and 100% penetration rate
5. 4-30-100: Four trucks in a platoon with a 30-ft headway and 100% penetration rate
6. 4-50-100: Four trucks in a platoon with a 50-ft headway and 100% penetration rate

In addition to the bias factor, the COV plays a vital role in the formulation of the statistical live load model. Figure 44 also reveals variations in the maximum load effects among different sites. Preserving these variations is essential to maintain a realistic representation of the inherent randomness in the data, and the COV is hereby employed to capture this variability. The COV is calculated as the standard deviation divided by the mean value of the maximum load effects across different sites for a given span length.

The statistical live load models correspond to the above-listed traffic scenario for the purpose of reliability calibration are tabulated in Table 40.

Table 40. Statistical parameters for Base WIM scenario (ADTT = 5,000).

Scenario	Span (ft)	M λ	M COV	S λ	S COV	M+ λ	M+ COV	M- λ	M- COV	Sc λ	Sc COV
Base WIM	80	1.22	0.19	1.26	0.16	1.24	0.19	0.77	0.13	1.33	0.17
	100	1.16	0.18	1.22	0.15	1.19	0.19	0.68	0.17	1.17	0.15
	120	1.12	0.18	1.18	0.15	1.15	0.18	0.64	0.18	1.04	0.14
	140	1.08	0.17	1.14	0.15	1.11	0.18	0.61	0.18	0.95	0.14
	160	1.04	0.17	1.09	0.14	1.08	0.17	0.59	0.18	0.89	0.14
	180	1.00	0.16	1.05	0.14	1.04	0.17	0.56	0.19	0.84	0.14
	200	0.97	0.16	1.01	0.14	1.01	0.16	0.54	0.20	0.80	0.14
	220	0.94	0.16	0.98	0.14	0.97	0.16	0.51	0.21	0.77	0.14
	240	0.90	0.15	0.94	0.14	0.95	0.16	0.49	0.22	0.74	0.14
	260	0.87	0.15	0.91	0.14	0.92	0.16	0.47	0.23	0.71	0.15
	280	0.85	0.15	0.88	0.14	0.89	0.15	0.46	0.24	0.68	0.15
	300	0.82	0.15	0.85	0.14	0.87	0.15	0.44	0.25	0.66	0.15
2-30-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.86	0.12	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.82	0.10	1.18	0.15
	120	1.12	0.18	1.21	0.13	1.16	0.18	0.77	0.09	1.07	0.13
	140	1.09	0.16	1.17	0.12	1.13	0.17	0.71	0.10	0.99	0.11
	160	1.06	0.15	1.14	0.11	1.11	0.15	0.65	0.11	0.93	0.11
	180	1.03	0.13	1.11	0.10	1.08	0.14	0.60	0.13	0.90	0.09
	200	1.01	0.12	1.09	0.09	1.06	0.13	0.57	0.15	0.87	0.09
	220	0.99	0.11	1.06	0.08	1.04	0.12	0.56	0.15	0.84	0.08
	240	0.97	0.10	1.03	0.07	1.02	0.11	0.54	0.15	0.82	0.08
	260	0.95	0.09	1.00	0.07	0.99	0.10	0.53	0.16	0.79	0.08
	280	0.92	0.08	0.98	0.06	0.97	0.09	0.51	0.17	0.77	0.08
	300	0.90	0.07	0.95	0.06	0.95	0.08	0.50	0.18	0.75	0.08

Scenario	Span (ft)	M λ	M COV	S λ	S COV	M+ λ	M+ COV	M- λ	M- COV	Sc λ	Sc COV
2-50-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.82	0.13	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.83	0.10	1.18	0.15
	120	1.12	0.18	1.19	0.15	1.16	0.18	0.80	0.09	1.04	0.14
	140	1.08	0.17	1.15	0.14	1.12	0.18	0.76	0.09	0.96	0.13
	160	1.04	0.17	1.12	0.12	1.08	0.17	0.70	0.09	0.91	0.12
	180	1.00	0.16	1.08	0.12	1.05	0.17	0.64	0.11	0.87	0.11
	200	0.97	0.15	1.06	0.11	1.02	0.15	0.59	0.13	0.84	0.10
	220	0.95	0.14	1.03	0.10	1.00	0.14	0.55	0.15	0.81	0.09
	240	0.92	0.13	1.00	0.09	0.98	0.13	0.52	0.17	0.79	0.09
	260	0.90	0.12	0.98	0.08	0.96	0.12	0.50	0.18	0.77	0.09
	280	0.88	0.11	0.95	0.07	0.93	0.12	0.49	0.18	0.75	0.09
	300	0.86	0.11	0.93	0.07	0.91	0.11	0.48	0.19	0.73	0.09
3-30-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.86	0.11	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.83	0.10	1.19	0.14
	120	1.12	0.18	1.21	0.14	1.16	0.18	0.85	0.10	1.07	0.12
	140	1.09	0.16	1.17	0.12	1.13	0.17	0.88	0.09	1.00	0.11
	160	1.06	0.15	1.15	0.11	1.11	0.15	0.87	0.08	0.95	0.10
	180	1.05	0.13	1.13	0.10	1.09	0.13	0.84	0.07	0.92	0.09
	200	1.03	0.11	1.11	0.08	1.07	0.12	0.79	0.07	0.90	0.08
	220	1.03	0.09	1.11	0.08	1.06	0.11	0.75	0.08	0.89	0.09
	240	1.02	0.08	1.11	0.07	1.06	0.09	0.70	0.09	0.90	0.09
	260	1.02	0.07	1.10	0.07	1.05	0.08	0.65	0.10	0.90	0.09
	280	1.02	0.07	1.10	0.07	1.04	0.08	0.61	0.12	0.90	0.09
	300	1.02	0.06	1.09	0.07	1.05	0.08	0.60	0.13	0.89	0.08
3-50-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.83	0.13	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.83	0.10	1.18	0.15
	120	1.12	0.18	1.19	0.15	1.16	0.18	0.80	0.09	1.04	0.14
	140	1.08	0.17	1.15	0.14	1.12	0.18	0.78	0.09	0.97	0.12
	160	1.04	0.17	1.12	0.12	1.08	0.17	0.77	0.09	0.92	0.11
	180	1.00	0.16	1.09	0.11	1.05	0.17	0.78	0.09	0.88	0.10
	200	0.98	0.15	1.06	0.11	1.02	0.16	0.77	0.08	0.86	0.09
	220	0.95	0.14	1.04	0.09	1.00	0.14	0.74	0.08	0.83	0.09
	240	0.93	0.12	1.02	0.08	0.99	0.13	0.71	0.09	0.81	0.09
	260	0.93	0.11	1.01	0.07	0.97	0.12	0.67	0.09	0.81	0.09
	280	0.92	0.09	1.01	0.06	0.95	0.11	0.64	0.11	0.81	0.09
	300	0.92	0.08	1.00	0.06	0.95	0.09	0.60	0.12	0.81	0.08

Scenario	Span (ft)	M λ	M COV	S λ	S COV	M+ λ	M+ COV	M- λ	M- COV	Sc λ	Sc COV
4-30-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.86	0.11	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.84	0.10	1.20	0.14
	120	1.12	0.18	1.20	0.14	1.16	0.18	0.87	0.10	1.07	0.12
	140	1.09	0.16	1.18	0.12	1.12	0.17	0.90	0.10	0.99	0.11
	160	1.07	0.15	1.15	0.11	1.10	0.16	0.94	0.10	0.96	0.10
	180	1.05	0.12	1.14	0.09	1.09	0.14	0.97	0.09	0.93	0.09
	200	1.04	0.11	1.12	0.08	1.07	0.12	0.97	0.08	0.92	0.08
	220	1.04	0.08	1.11	0.08	1.06	0.11	0.95	0.08	0.92	0.09
	240	1.03	0.07	1.12	0.07	1.06	0.09	0.91	0.07	0.92	0.09
	260	1.03	0.07	1.12	0.07	1.05	0.08	0.87	0.07	0.92	0.09
	280	1.03	0.07	1.12	0.07	1.05	0.08	0.83	0.07	0.93	0.09
	300	1.04	0.07	1.13	0.08	1.07	0.08	0.79	0.07	0.93	0.09
4-50-100	80	1.22	0.18	1.27	0.16	1.25	0.19	0.83	0.13	1.34	0.17
	100	1.16	0.18	1.23	0.15	1.20	0.19	0.83	0.10	1.18	0.15
	120	1.12	0.18	1.19	0.15	1.16	0.18	0.80	0.08	1.04	0.14
	140	1.08	0.17	1.15	0.14	1.12	0.18	0.78	0.09	0.97	0.13
	160	1.04	0.17	1.12	0.12	1.08	0.17	0.80	0.09	0.92	0.11
	180	1.01	0.16	1.09	0.11	1.05	0.17	0.83	0.10	0.88	0.10
	200	0.98	0.15	1.06	0.11	1.02	0.15	0.86	0.09	0.86	0.09
	220	0.96	0.13	1.04	0.09	1.00	0.14	0.88	0.09	0.84	0.08
	240	0.94	0.12	1.02	0.08	0.99	0.13	0.88	0.08	0.83	0.08
	260	0.93	0.10	1.01	0.07	0.96	0.12	0.86	0.08	0.83	0.08
	280	0.93	0.08	1.01	0.06	0.95	0.11	0.84	0.08	0.83	0.08
	300	0.92	0.07	1.00	0.06	0.95	0.09	0.81	0.08	0.83	0.08

3.5 Impact of Platoons

3.5.1 Delta Factors (Δ) of Bias Factor

To better comprehend the impact of truck platoons, the RT introduced the delta factor (i.e., Δ-factor) which represent the differences between the bias factor of truck platoons and Base WIM, as shown in Eq.16.

$$\Delta = \frac{\text{Platoon Bias Factor} - \text{Base WIM Bias Factor}}{\text{Base WIM Bias Factor}} \quad \text{Eq. 16}$$

Figure 53 to Figure 57 illustrates the Δ-factor for five different load effect cases: M, S, M+, M-, and Sc. Among these cases, the continuous span negative moment stands out with the highest Δ-factor. This is attributed to the symmetrical nature of its influence line, which allows trucks located on both spans to significantly contribute to the overall load effects. As a result, truck platoons with individual trucks spaced apart result in much heavier load effects compared to single trucks. In the most critical platoon cases, such as 4-30-100, the Δ-factor for the negative moment reaches 85%. Moreover, the continuous span negative moment is significantly influenced by vehicles with long wheelbases. Consequently, as the span lengths increase, the Δ-factor shows a fluctuating trend of increase and decrease. However, it is worth noting that despite exhibiting the highest Δ-factor, the negative moment does not greatly impact the overall bridge reliability. This is because the bias factor for the negative moment is always less than 1.0, indicating that the HL-93 loading can encompass the effects of the truck platoon.

Except the continuous span negative moment, the other load effects follow a similar pattern, where the Δ-factor increases as the span lengths increase. As aforementioned, with the span length increases, the bias factor for Base WIM case decreases due to the rapid increase of the uniform load effects, whereas the bias factor for the platoon cases approaches a stable level. As a result, the delta factor is amplified for a longer span.

The shear effects at the continuous span interior support exhibit the second highest delta, exceeding 40% for a span length of 300ft. In contrast, the continuous span positive moment, which has the greatest bias factors, shows the smallest Δ-factor.

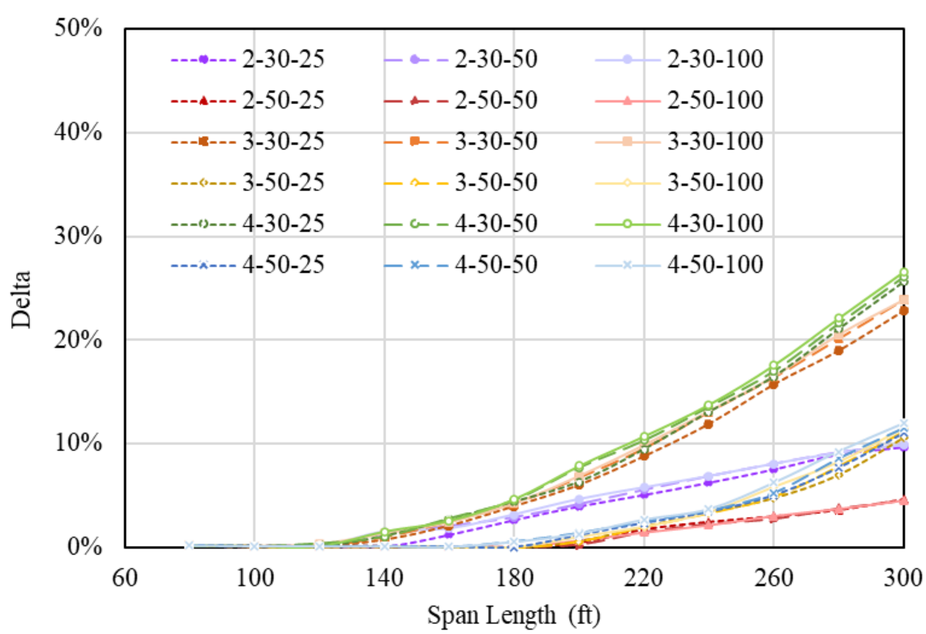


Figure 53. Delta factors for simple span moment.

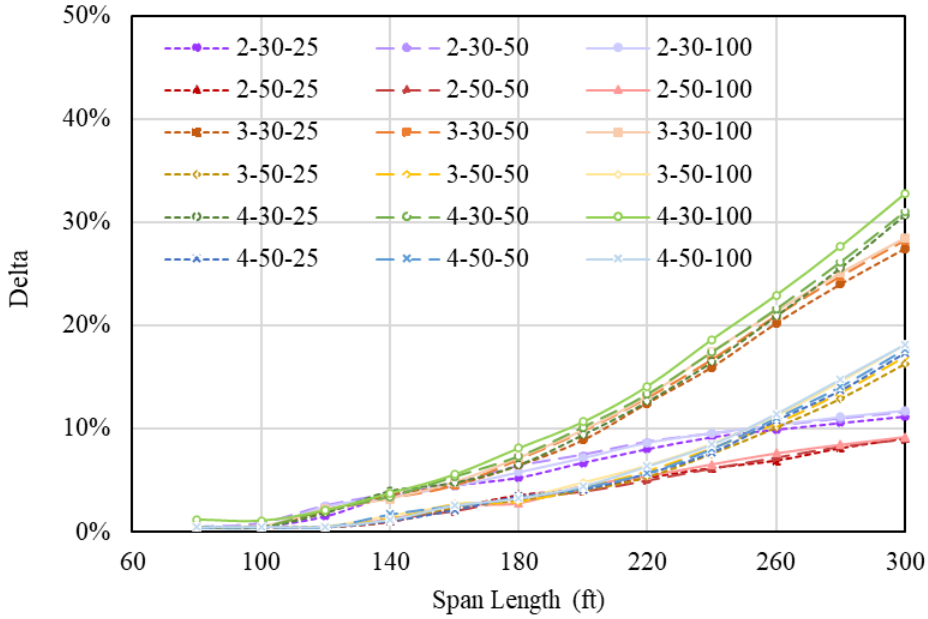


Figure 54. Delta factors for simple span shear.

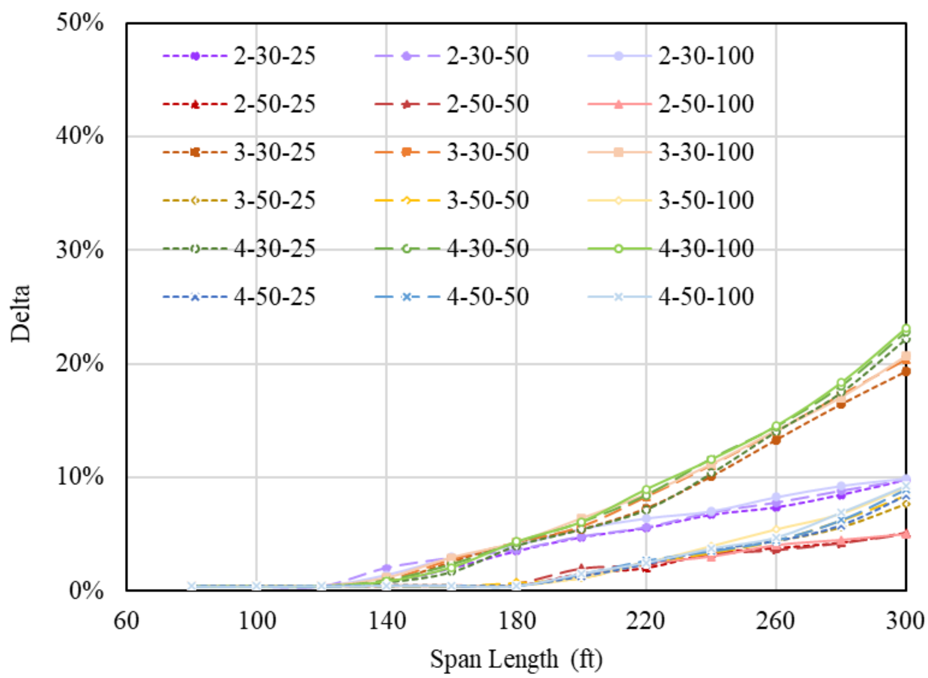


Figure 55. Delta factors for continuous span positive moment.

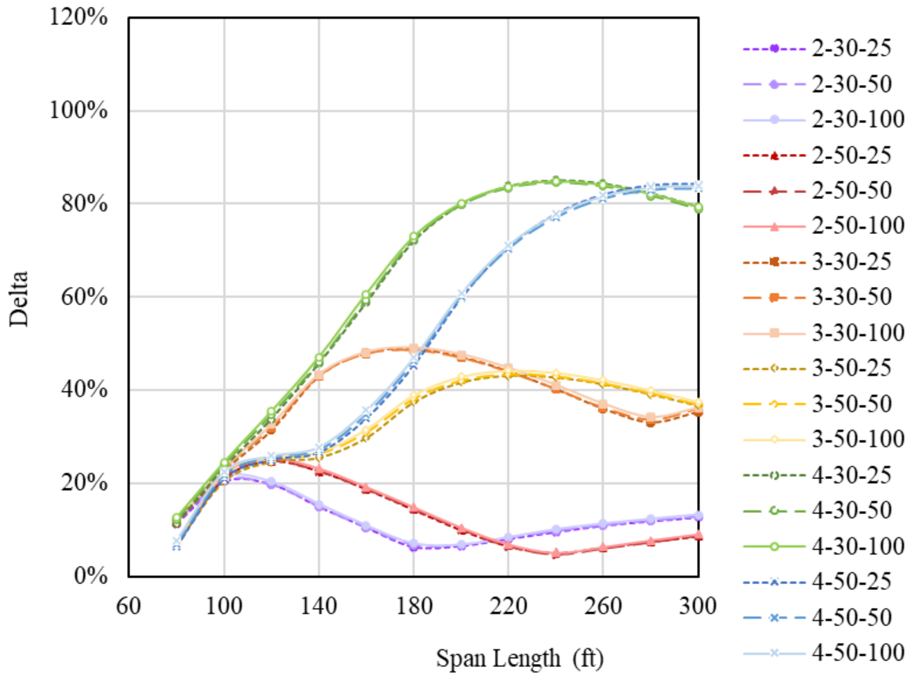


Figure 56. Delta factors for continuous span negative moment.

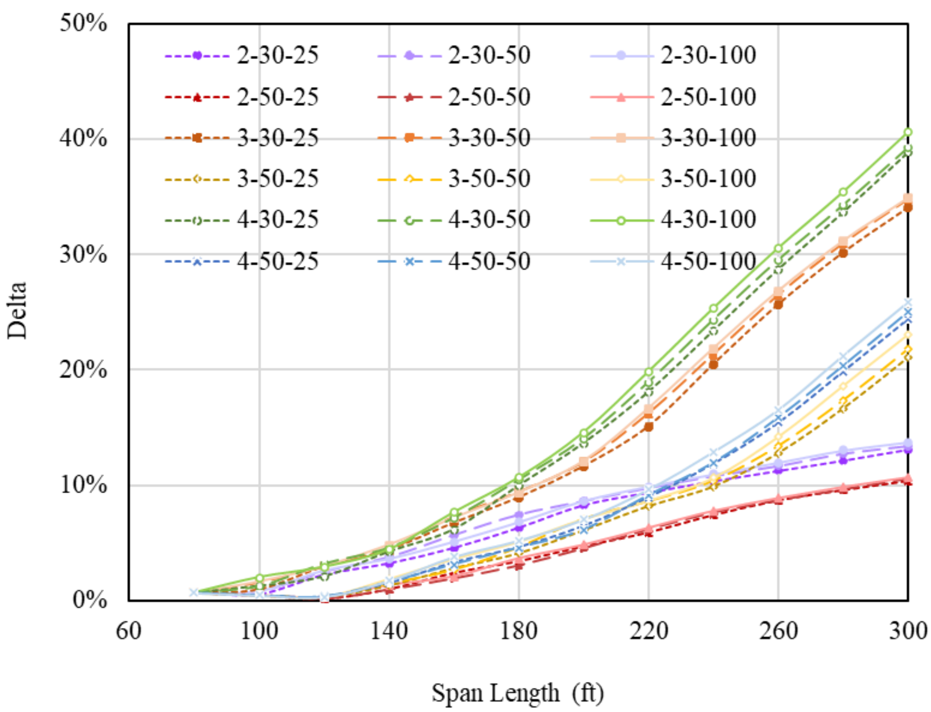
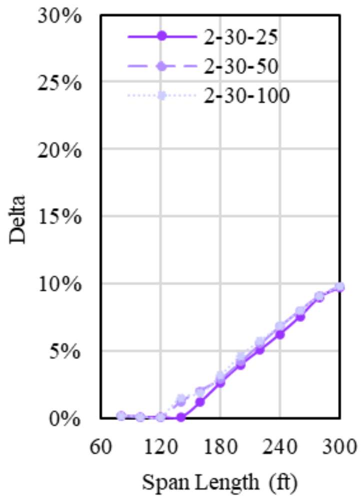
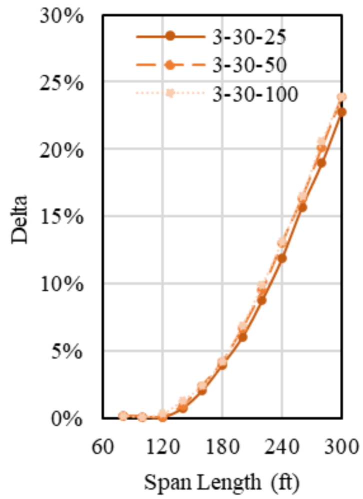


Figure 57. Delta factors for continuous span interior support shear.

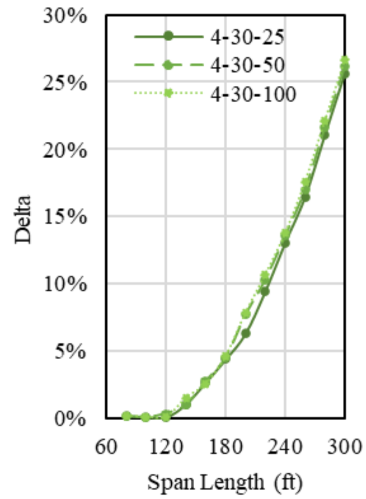
The impact of each individual configuration is investigated as well. In Figure 58, the number of trucks within a platoon and headway distances are held constant, while the effects of different penetration rates are compared. It is observed that the Δ-factor, which is closely correlated with the bias factor, exhibits minimal sensitivity to the penetration rate. Consequently, similar conclusions can be drawn regarding the penetration rate as for the bias factor. As a result, in the subsequent illustrations presented in Figure 59 and Figure 61, the penetration rate is not considered as a variable. Instead, only the critical penetration rate (i.e., 100%) is used to conduct the comparative study.



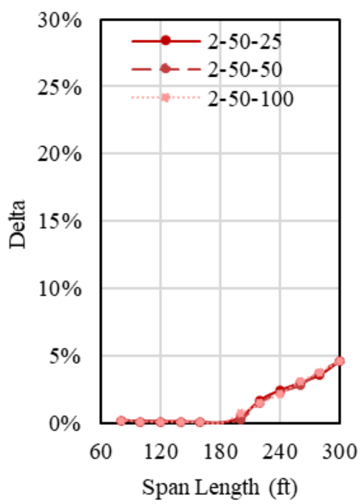
(a) 2-truck platoon with 30 ft headway



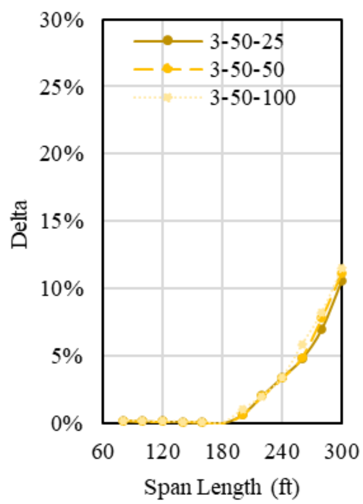
(b) 3-truck platoon with 30 ft headway



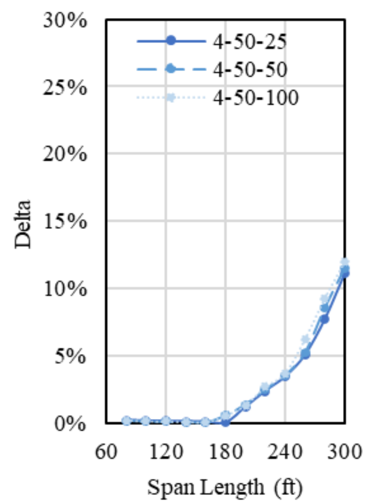
(c) 4-truck platoon with 30 ft headway



(d) 2-truck platoon with 50 ft headway



(e) 3-truck platoon with 50 ft headway

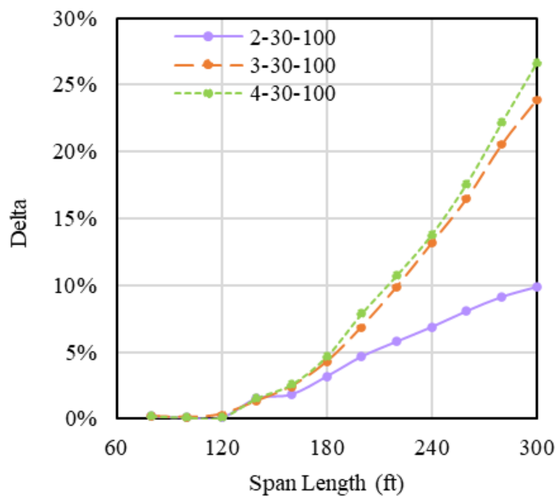


(f) 4-truck platoon with 50 ft headway

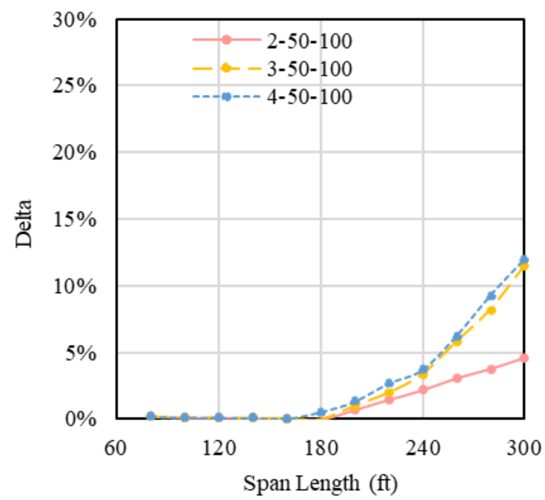
Figure 58. Effect of penetration rate for simple span moment.

Figure 59 and Figure 60 provide insights into the influence of the number of trucks in a platoon on the Δ-factor for simple span moment and continuous span negative moment, respectively. Specifically, it shows that the headway distances of 30 ft and 50 ft for the truck platoons begin to affect the bridges longer than 120 ft and 180 ft, respectively. The Δ-factor increases as the number of trucks within the platoon increases. For simple span moment, the increase in the Δ-factor is more significant when going from 2 to 3 trucks, whereas further increasing the number of trucks to 4 does not have a substantial impact. While for continuous span negative moment, significant changes in Δ-factor are observed when number of trucks changes from both 2 to 3 and 3 to 4.

This phenomenon can be attributed to the limitations of the span lengths considered in the investigation, which are mostly less than 300 ft. These span lengths can accommodate three fully contained Class 9 trucks. As a result, the fourth truck either cannot be on the bridge simultaneously with the other three trucks or can only be partially on the bridge, which does not significantly increase the load effects. Hence, the Δ-factor shows a relatively small increase when the number of trucks exceeds three in these scenarios.

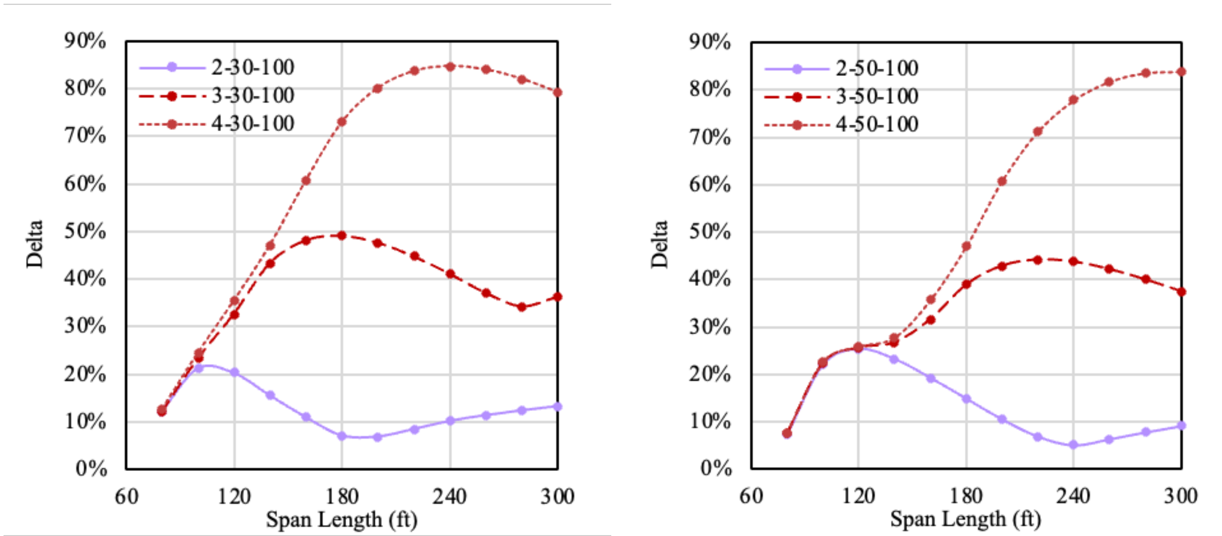


(a) Headway = 30 ft



(b) Headway = 50 ft

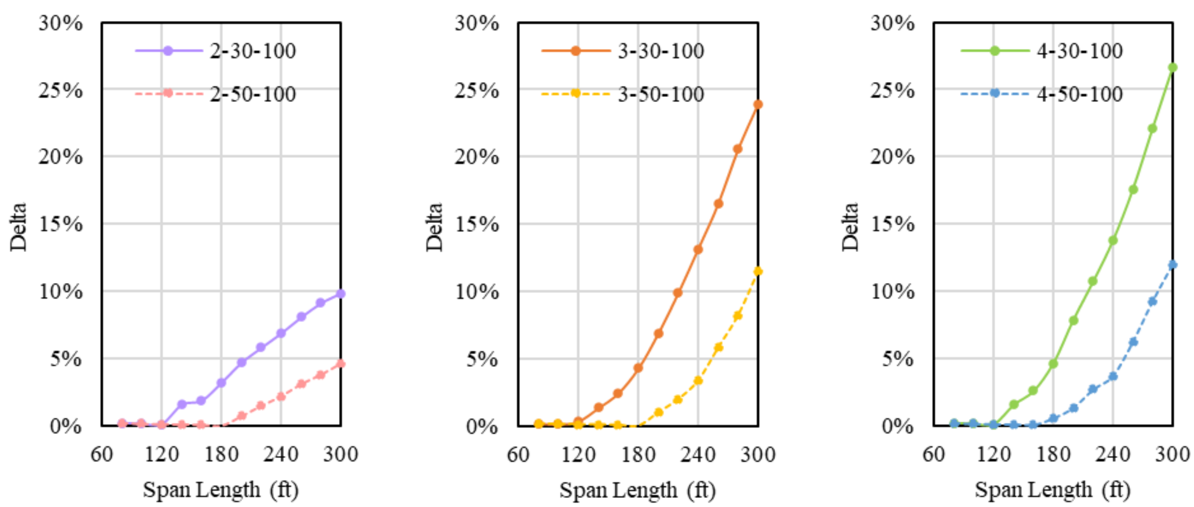
Figure 59. Effect of number of trucks in a platoon for simple span moment.



(a) Headway = 30 ft **(b) Headway = 50 ft**

Figure 60. Effect of number of trucks in a platoon for continuous span negative moment.

In addition to the number of trucks within a platoon, the headway distance also led to a great difference in bias factor. Smaller headway corresponds to higher Δ-factor despite of different number of trucks within a platoon, as shown in Figure 61.



(a) 2-truck platoons **(b) 3-truck platoons** **(c) 4-truck platoons**

Figure 61. Effect of headway distance for simple span moment.

The Δ-factors for the impactable cases, excluding different penetration rate, are tabulated in Table 41.

Table 41. Delta factors (ADTT =5,000).

Scenario	Span (ft)	M	S	M+	M-	Sc
2-30-100	80	0%	0%	0%	12%	1%
	100	0%	1%	0%	21%	1%
	120	0%	2%	0%	20%	3%
	140	2%	3%	1%	16%	4%
	160	2%	4%	3%	11%	5%
	180	3%	6%	4%	7%	7%
	200	5%	7%	5%	7%	9%
	220	6%	9%	6%	8%	10%
	240	7%	10%	7%	10%	11%
	260	8%	10%	8%	11%	12%
	280	9%	11%	9%	12%	13%
	300	10%	12%	10%	13%	14%
2-50-100	80	0%	0%	0%	7%	1%
	100	0%	1%	0%	22%	0%
	120	0%	0%	0%	25%	0%
	140	0%	1%	0%	23%	1%
	160	0%	3%	0%	19%	2%
	180	0%	3%	0%	15%	4%
	200	1%	4%	1%	11%	5%
	220	1%	5%	2%	7%	6%
	240	2%	7%	3%	5%	8%
	260	3%	8%	4%	6%	9%
	280	4%	8%	5%	8%	10%
	300	5%	9%	5%	9%	11%
3-30-100	80	0%	0%	0%	12%	1%
	100	0%	1%	0%	23%	2%
	120	0%	2%	0%	33%	3%
	140	1%	3%	1%	43%	5%
	160	2%	5%	3%	48%	7%
	180	4%	7%	4%	49%	9%
	200	7%	10%	6%	48%	12%
	220	10%	13%	9%	45%	17%
	240	13%	17%	11%	41%	22%
	260	17%	21%	14%	37%	27%
	280	21%	25%	17%	34%	31%
	300	24%	28%	21%	36%	35%

Scenario	Span (ft)	M	S	M+	M-	Sc
3-50-100	80	0%	0%	0%	7%	1%
	100	0%	1%	0%	22%	0%
	120	0%	1%	0%	26%	0%
	140	0%	2%	0%	27%	2%
	160	0%	3%	0%	32%	4%
	180	0%	3%	0%	39%	5%
	200	1%	5%	1%	43%	7%
	220	2%	6%	3%	44%	9%
	240	3%	9%	4%	44%	11%
	260	6%	11%	5%	42%	14%
	280	8%	15%	7%	40%	19%
	300	11%	18%	9%	37%	23%
4-30-100	80	0%	1%	0%	13%	1%
	100	0%	1%	0%	24%	2%
	120	0%	2%	0%	35%	3%
	140	2%	4%	1%	47%	4%
	160	3%	6%	2%	61%	8%
	180	5%	8%	4%	73%	11%
	200	8%	11%	6%	80%	15%
	220	11%	14%	9%	84%	20%
	240	14%	19%	12%	85%	25%
	260	18%	23%	15%	84%	31%
	280	22%	28%	18%	82%	35%
	300	27%	33%	23%	79%	41%
4-50-100	80	0%	0%	0%	8%	1%
	100	0%	1%	0%	23%	0%
	120	0%	1%	0%	26%	0%
	140	0%	1%	0%	28%	2%
	160	0%	3%	0%	36%	4%
	180	1%	3%	0%	47%	5%
	200	1%	4%	2%	61%	7%
	220	3%	6%	3%	71%	10%
	240	4%	8%	4%	78%	13%
	260	6%	11%	5%	82%	17%
	280	9%	15%	7%	84%	21%
	300	12%	18%	9%	84%	26%

3.5.2 Delta Factors of COV

The other critical parameter that would affect the statistics of maximum load is the COV of the bias factor. To evaluate the impact of truck platoons on the COV, Δ_{COV} is computed as shown in Eq. 17.

$$\Delta_{COV} = \frac{\text{Platoon COV of Bias Factor}}{\text{Base WIM COV of Bias Factor}} \qquad \text{Eq. 17}$$

To illustrate the impact of platoon, Figure 62 and Figure 63 show the delta factor of COV, Δ_{COV}, for simple span moment and continuous span negative moment, respectively. The other load effects, such as simple span shear, continuous span positive moment, and continuous span interior support shear, follow the same trend as simple span moment. For simple span moment shown in Figure 62, the delta factor of COVs of bias factor start to decrease as the span length increase over 120 ft. This indicates that COVs would decrease when the platoons govern over single trucks. The COV of bias factor for platoons can be as low as 43% (4-30-100) of COV of bias factor for Base WIM. For continuous span negative moment shown in Figure 63, the delta factor of COVs of bias factor decrease significantly as the span length increase. As span length increases over 160 ft, some scenarios have increased COVs while some scenarios has decreased COVs. The COVs of bias factor for platoons can be as low as 29% (4-30-100) of COV of bias factor for Base WIM. In general, the platoons have lower COVs of bias factor compared to the COVs of Base WIM. In addition, the COV of the bias factor decreases as penetration rate increases. However, the overall impact is minimal.

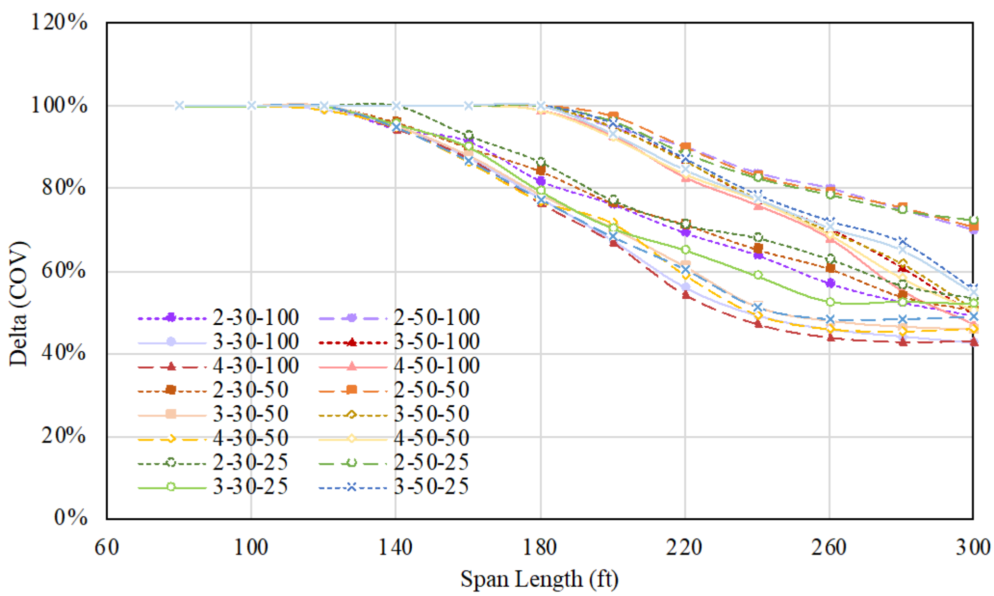


Figure 62. Delta factors of COV, Δ_{COV},for simple span moment.

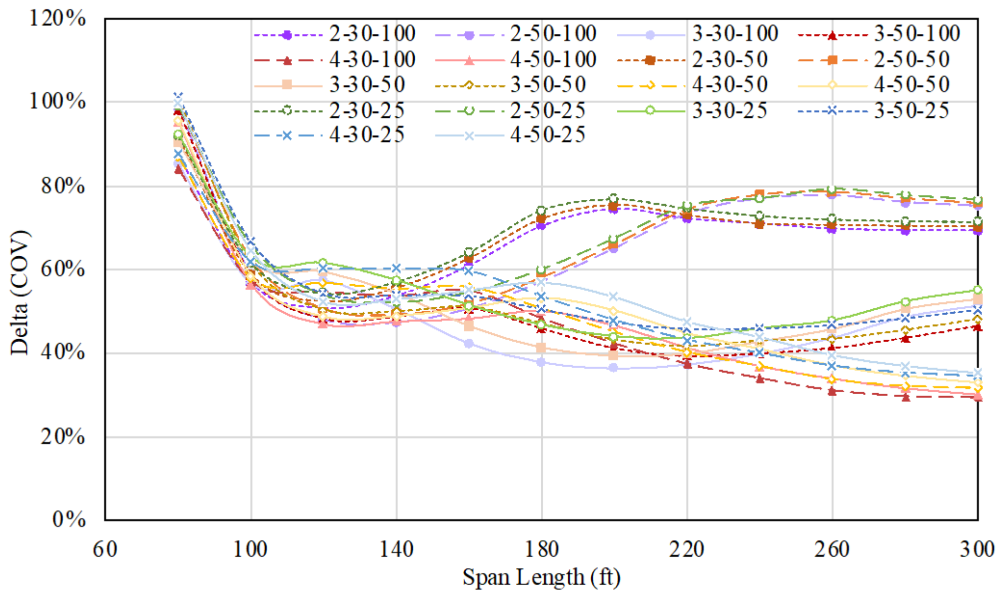


Figure 63. Delta factors of COV, Δ_{COV},for continuous span negative moment.

3.5.3 Breakdown of Truck Type in Upper Tail

In an effort to investigate the impact of the truck platoons, the RT distinguished the type of trucks, either single trucks or platoons, that produce the most critical load effects on bridges.

Representing instances where bridges experience the greatest stresses, the 3,600 trucks with the largest load effects are regarded as the "upper-tail" trucks in this study. The number of 3,600 trucks in the upper tail was optimized to provide accurate and consistent results for all WIM sites. Next, the upper-tail trucks were categorized as either resulting from a single truck or a truck platoon. The categorization was interpreted as the "percentage of platoons in the upper tail", which equals the number of platoons in the heaviest 3,600 loading events divided by 3,600. The higher percentage corresponds to a bigger impact of the platoons. This indicator helps facilitate the identification of the trends in the impact of truck platoon on the load effects.

The percentage of platoons in the upper tail are depicted from Figure 64 to Figure 68 for different load effect types. In general, percentage of truck platoons increases as span length increases, except for continuous span negative moment. For negative moments, the percentage is mostly above 90% regardless of span lengths.

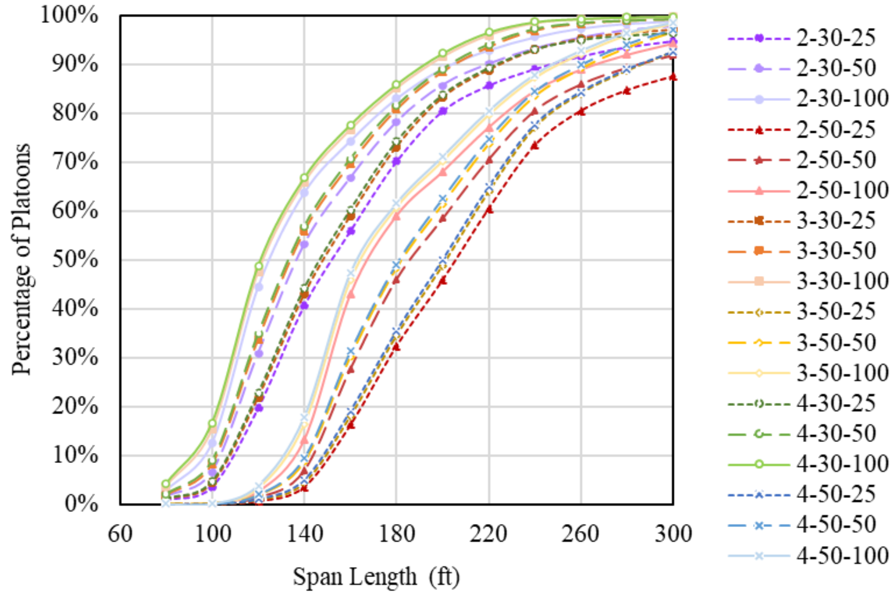


Figure 64. Percentage of truck platoons in the upper tail of simple span moment.

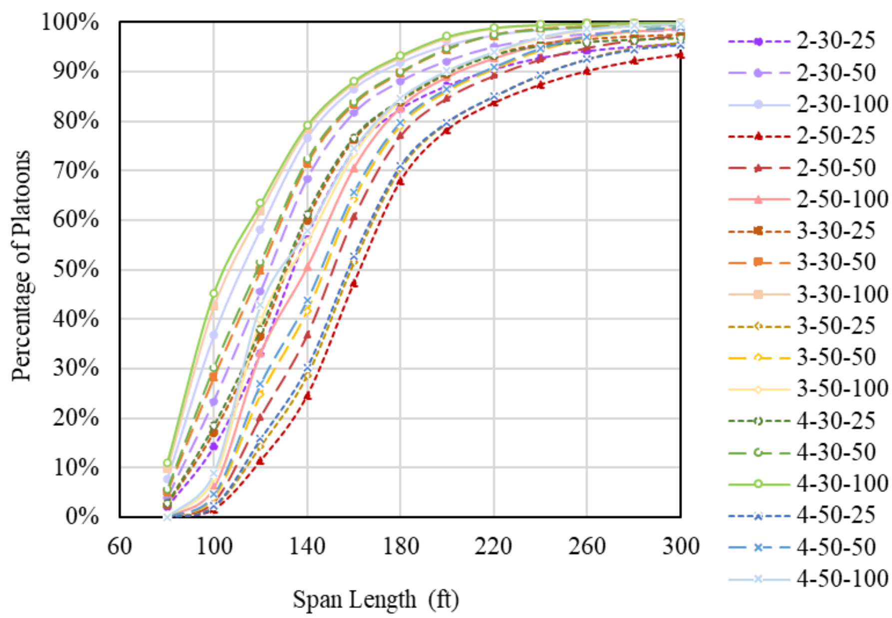


Figure 65. Percentage of truck platoons in the upper tail of simple span shear.

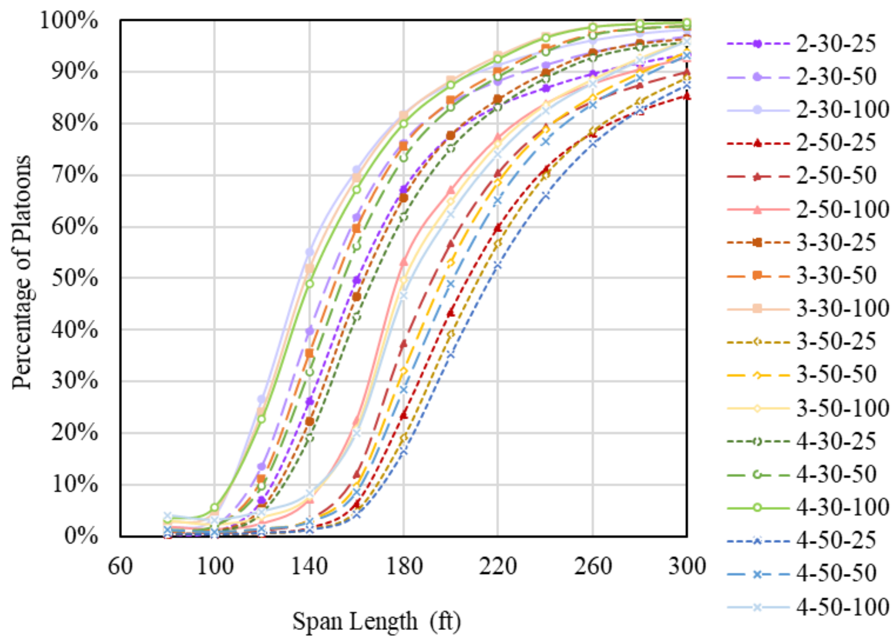


Figure 66. Percentage of truck platoons in the upper tail of continuous span positive moment.

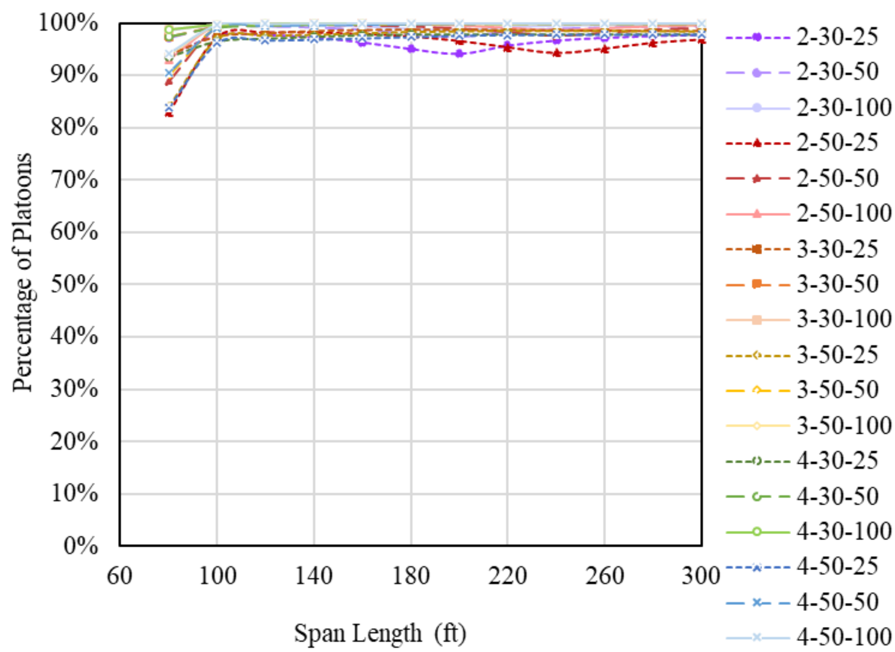


Figure 67. Percentage of truck platoons in the upper tail of continuous span negative moment.

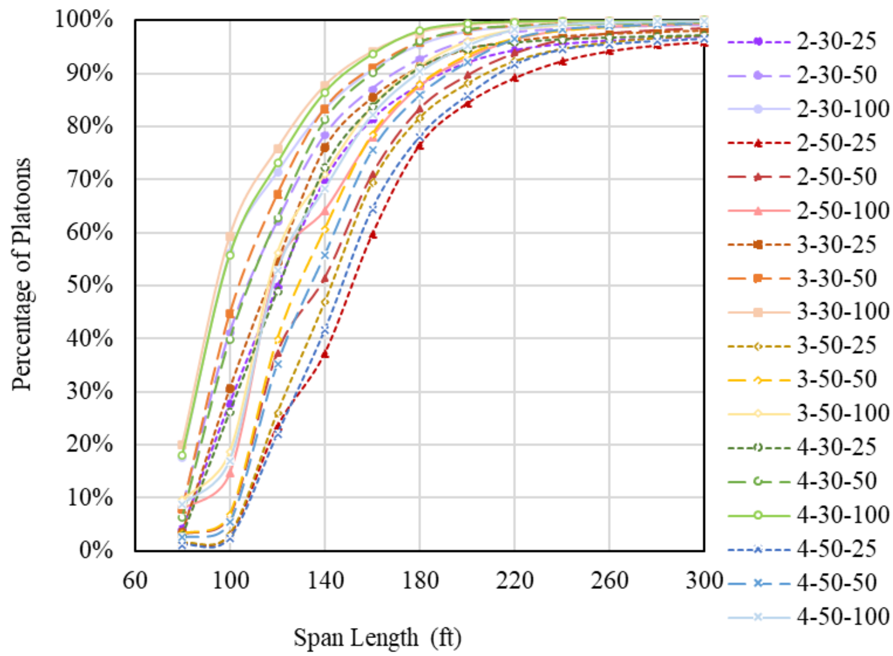
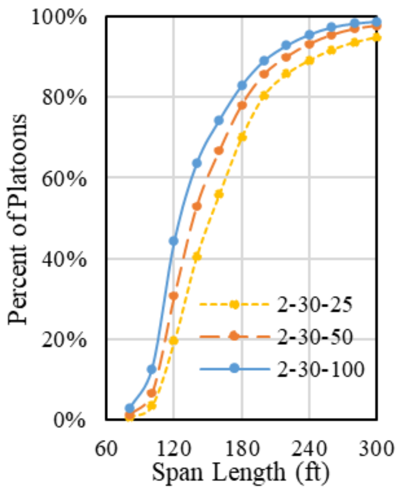


Figure 68. Percentage of truck platoons in the upper tail of continuous span interior support shear.

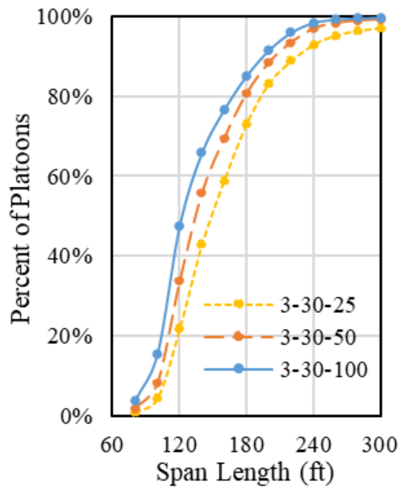
The impacts of the penetration rate, number of trucks within platoons, and headway distances are plotted from Figure 69 to Figure 71, respectively. In contrast to the bias factor and Δ-factor, the penetration rate does exhibit an impact on the percentage of platoons in the upper tail load effects, as demonstrated in Figure 69. A higher penetration rate leads to a larger number of platoons in the upper tail.

The number of trucks within platoons does not have a clear impact, as depicted in Figure 70. This can be attributed to the presence of heavy single trucks. Although efforts have been made to filter out illegal overweight trucks and permit-like heavy vehicles, there are still heavy single trucks that consistently occupy spaces in the upper tail. Therefore, increasing the number of trucks within the platoon does not necessarily increase the percentage of platoons in the upper tail, despite raising the overall load effect levels in that range.

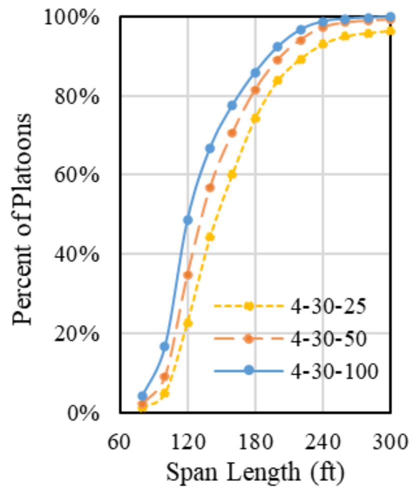
Lastly, the headway distance has the most significant impact on the percentage. Increasing the headway distance from 30 ft to 50 ft results in a distinct decrease in the number of platoons in the upper tail, as illustrated in Figure 71.



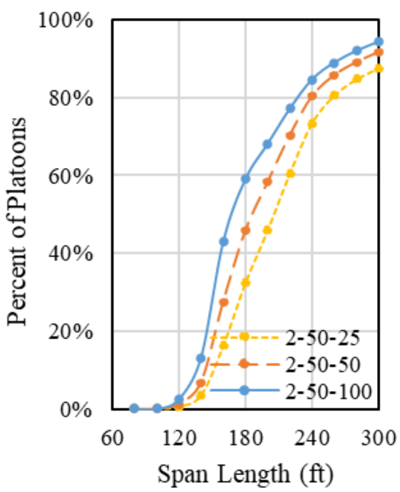
(a) 2-truck platoon with 30 ft headway



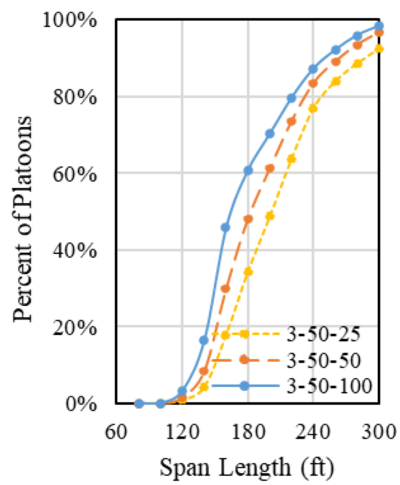
(b) 3-truck platoon with 30 ft headway



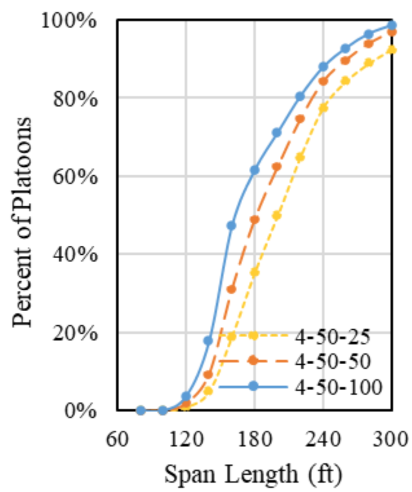
(c) 4-truck platoon with 30 ft headway



(d) 2-truck platoon with 50 ft headway

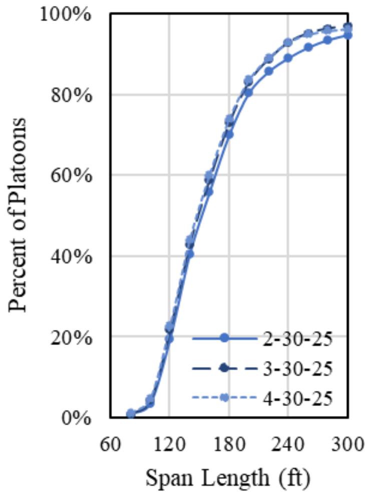


(e) 3-truck platoon with 50 ft headway

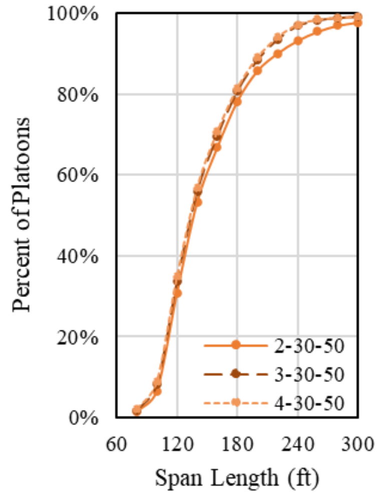


(f) 4-truck platoon with 50 ft headway

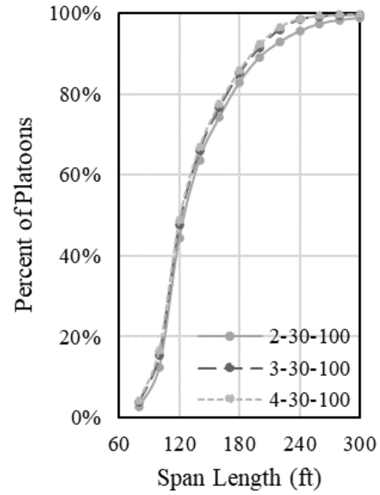
Figure 69. Effect of penetration rate for simple span moment.



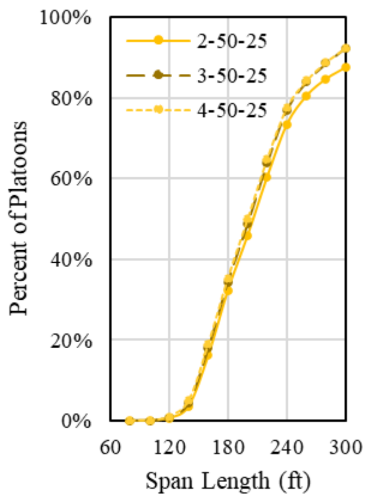
(a) 25% penetration rate with 30-ft headway



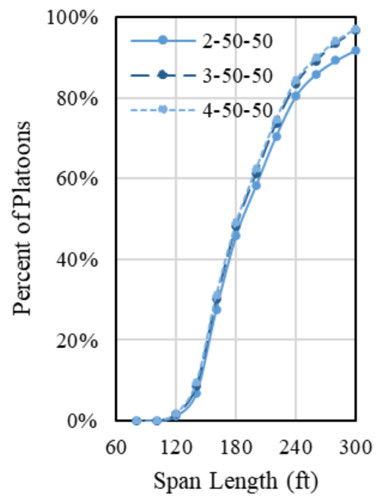
(b) 50% penetration rate with 30-ft headway



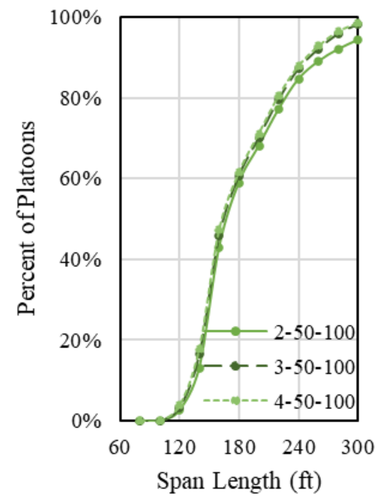
(c) 100% penetration rate with 30-ft headway



(d) 25% penetration rate with 50-ft headway

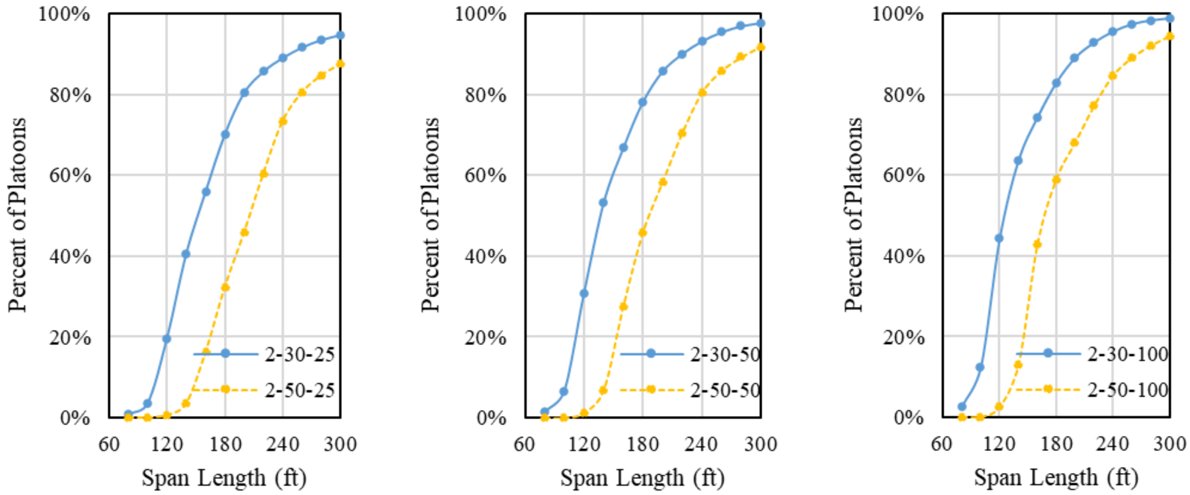


(e) 50% penetration rate with 50-ft headway



(f) 100% penetration rate with 50-ft headway

Figure 70. Effect of number of trucks in a platoon for simple span moment.



2-truck platoons **3-truck platoons** **4-truck platoons**

Figure 71. Effect of headway distance for simple span moment.

Table 42 presents the percentages of platoons in the upper tail, focusing on the most critical penetration rate of 100%. It is noteworthy that, apart from the negative moment for continuous spans, all the percentages are less than 100%. The occurrence of 100% values in the other truck platoon scenarios is mainly attributed to rounding issues rather than the actual data. Only in the case of the negative moment, at long spans with high penetration rates, do we observe percentages of 100%.

Table 42. Percentage of platoons in upper-tail load effects (ADTT =5,000).

Scenario	Span (ft)	M	S	M+	M-	Sc
2-30-100	80	1.1%	2.9%	0.5%	93.5%	2.8%
	100	4.7%	18.5%	0.9%	96.4%	26.1%
	120	22.8%	38.0%	4.6%	96.9%	49.0%
	140	44.2%	61.2%	19.2%	97.3%	72.2%
	160	60.2%	76.7%	42.6%	97.6%	83.7%
	180	74.1%	84.2%	62.0%	97.7%	91.0%
	200	83.7%	89.6%	75.2%	97.7%	94.6%
	220	89.2%	93.2%	83.1%	97.7%	95.8%
	240	93.0%	95.1%	88.7%	97.7%	96.4%
	260	94.9%	95.9%	92.7%	97.7%	96.7%
	280	95.8%	96.3%	94.7%	97.7%	97.0%
	300	96.3%	96.7%	95.7%	97.7%	97.1%
2-50-100	80	2.1%	5.6%	1.1%	97.4%	6.3%
	100	9.0%	30.3%	1.9%	99.2%	39.8%
	120	34.9%	51.5%	9.6%	99.6%	62.8%
	140	56.9%	72.3%	31.9%	99.8%	81.2%
	160	70.7%	83.9%	56.1%	99.9%	90.0%
	180	81.6%	90.0%	73.3%	99.9%	96.0%
	200	89.1%	94.8%	83.1%	100.0%	98.4%
	220	94.1%	97.5%	89.0%	100.0%	99.0%
	240	97.2%	98.6%	93.9%	100.0%	99.3%
	260	98.5%	99.0%	97.2%	100.0%	99.5%
	280	98.9%	99.3%	98.4%	100.0%	99.6%
	300	99.2%	99.4%	98.9%	100.0%	99.7%
3-30-100	80	4.1%	11.1%	3.3%	98.8%	18.1%
	100	16.6%	45.3%	5.6%	99.8%	55.8%
	120	48.7%	63.4%	22.7%	99.9%	73.0%
	140	66.8%	79.2%	48.9%	100.0%	86.4%
	160	77.6%	88.1%	67.2%	100.0%	93.6%
	180	85.9%	93.3%	80.1%	100.0%	98.0%
	200	92.3%	97.2%	87.4%	100.0%	99.4%
	220	96.6%	98.9%	92.5%	100.0%	99.7%
	240	98.7%	99.5%	96.7%	100.0%	99.9%
	260	99.4%	99.7%	98.8%	100.0%	99.9%
	280	99.7%	99.9%	99.4%	100.0%	100.0%
	300	99.8%	99.9%	99.7%	100.0%	100.0%

Scenario	Span (ft)	M	S	M+	M-	Sc
3-50-100	80	0.0%	0.0%	0.6%	83.9%	1.1%
	100	0.1%	2.3%	0.4%	96.2%	2.4%
	120	1.0%	15.9%	0.7%	96.6%	21.9%
	140	5.0%	30.3%	1.3%	96.8%	41.5%
	160	19.0%	52.8%	4.2%	97.1%	64.3%
	180	35.4%	71.1%	16.5%	97.3%	78.0%
	200	49.9%	79.8%	35.2%	97.6%	85.7%
	220	64.9%	85.0%	52.6%	97.7%	91.6%
	240	77.6%	89.3%	66.1%	97.7%	94.4%
	260	84.3%	92.4%	76.0%	97.7%	95.4%
	280	88.9%	94.4%	82.6%	97.7%	96.1%
	300	92.5%	95.3%	87.5%	97.7%	96.4%
4-30-100	80	0.0%	0.1%	1.2%	90.4%	2.5%
	100	0.1%	4.6%	0.9%	99.1%	5.4%
	120	1.9%	26.9%	1.5%	99.4%	35.2%
	140	9.5%	43.8%	2.8%	99.5%	55.8%
	160	31.2%	65.6%	8.6%	99.7%	75.5%
	180	49.1%	79.8%	28.5%	99.8%	85.9%
	200	62.5%	86.5%	49.0%	99.9%	92.1%
	220	74.7%	90.9%	65.3%	99.9%	96.5%
	240	84.4%	94.6%	76.6%	100.0%	98.3%
	260	89.9%	97.1%	83.7%	100.0%	98.9%
	280	94.1%	98.3%	88.8%	100.0%	99.2%
	300	97.1%	98.8%	93.2%	100.0%	99.4%
4-50-100	80	0.0%	0.1%	4.0%	94.2%	8.7%
	100	0.1%	9.0%	3.1%	99.7%	17.0%
	120	3.8%	42.8%	4.8%	99.9%	52.8%
	140	17.8%	57.8%	8.3%	99.9%	68.2%
	160	47.2%	74.4%	20.0%	100.0%	82.2%
	180	61.5%	84.7%	46.8%	100.0%	90.3%
	200	71.1%	90.2%	62.5%	100.0%	95.3%
	220	80.5%	94.0%	74.1%	100.0%	98.3%
	240	87.9%	97.0%	82.4%	100.0%	99.3%
	260	92.8%	98.6%	87.8%	100.0%	99.6%
	280	96.5%	99.3%	92.2%	100.0%	99.8%
	300	98.6%	99.6%	95.9%	100.0%	99.9%

3.5.4 Fatigue Limit State

To assess the impact of truck platooning on the fatigue limit state, fatigue load effects from the platoon scenarios are compared to the baseline load effects as well as the fatigue design load effects from the LRFD Spec. (23 CFR 625.4(d)(1)(v)). Based on the Tier 1 Analysis, it is expected that adding platoons to the load spectrum will shift upward the "maximum" loading and the "effective" loading which relate to LRFD

Spec. (23 CFR 625.4(d)(1)(v)) load combinations Fatigue I and Fatigue II, respectively. However, extrapolating a change in loading and its effect on the national bridge inventory is complicated and call for simplifying assumptions.

Our approach aims to identify bridges most likely to be affected by a shift in maximum fatigue loading and causing change from infinite life to finite life performance in existing bridges. The majority of steel bridges in the inventory have been designed to some level of fatigue stress limit. Despite many changes to the fatigue design provisions going back to the 1970s, it can be said that designing to a maximum range of stress to achieve infinite life is most often the approach implemented in design. Attempting some type of damage accumulation (or remaining life) analysis using presumed ADTT and variability of age is deemed overly precise considering the sensitivity to existing fatigue damage and additional unknowns about platoons. Therefore, we are assuming that steel bridges with span lengths where the maximum fatigue load effect with platoons exceeds the Fatigue I load effect are those that are most likely to be negatively affected.

To determine the affected span ranges, the ratio between the maximum moment produced by the platoon spectra and the AASHTO LRFD Fatigue Truck was computed. Specifically, the focus was on the 1 in 10,000 probabilistic maximum platoon truck as the criterion for infinite fatigue life check is based on this recurrence. Recent calibration of the Fatigue I limit state demonstrated that this limit generally corresponds to an allowable frequency of exceedance of 1/10,000 to ensure infinite life performance. This load factor is currently 1.75 in the LRFD Spec.; therefore, if the load effect ratio is above 1.75, truck platoon would result in stress cycles larger than the factored demand used to conduct the infinite fatigue life check; potentially affecting the remaining fatigue life of the bridge. This negative effect would be applicable to any steel primary flexural members independently of whether its fatigue life is established based on infinite or finite life evaluation.

Figure 72 shows the computed ratio for the platoon configurations considered, as well as the Base WIM; showing that at spans equal or greater to 180 feet, the moment ratio begins to be larger than 1.75 for certain truck platoon configurations. For this analysis single trucks that are lighter than 20 kips are filtered out from the population to be consistent with the calibration work done by NCHRP Project 12-83. This suggests that truck platoons are expected to produce a negative effect on fatigue life for steel bridges with spans 180 ft or longer. At span shorter than 100 ft, truck platooning would have a small negative effect; however, a similar shift in the Base WIM data suggests that this may likely be a spurious effect. It should be noted that the ratios from the baseline WIM support the load factor of 1.75 as currently specified in the LRFD Spec. (23 CFR 625.4(d)(1)(v)).

The span length study results summarized in Section 2.3 showed that platoon trucks are expected to produce larger stress cycles than the AASHTO LRFD Fatigue Trucks for spans 180 ft and longer as well. The apparently slight shift at shorter span is not corroborated by the results from line girder analysis.

Consequently, the filter for Case 7 described in Section 2.7.7 still applies, and steel bridges built on 1957 or after with spans larger than 180 ft are likely to experience remaining fatigue life reduction as a result of truck platooning.

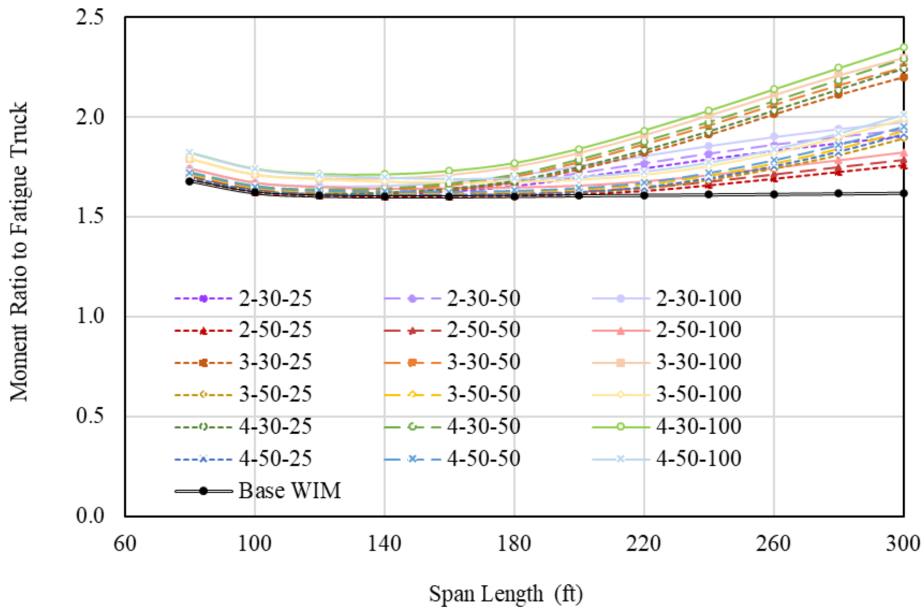


Figure 72. 1 in 10,000 moment ratio with respect to AASHTO LRFD Fatigue Truck.

3.6 Summary of the Tier 2 Analysis

Covering the regional and seasonal variations, the full year data of 32 WIM sites from six states (i.e., California, New Jersey, Indiana, Texas, Florida, and Alabama) were selected for the study to develop the statistical live load model for reliability calibration as well as to investigate the impact of truck platoons. The major conclusions drawn from the Tier 2 Analysis are listed as follows:

- A data validation algorithm was applied to assure the quality of WIM data. In addition to using the reliable dataset, filters were adopted from NCHRP Project 12-83 project to clean the data for the use of developing live load statistics.

- Permit-like heavy vehicles, often characterized as 7 or more-axle trucks, significantly influenced the upper-tail load effects, leading to biased extrapolation results. A sensitivity analysis indicated that including 7-axle trucks weighing over 100 kips significantly influenced the extrapolated normalized load effects, implying the need for careful filtering in live load analysis. These unusual heavy trucks were further filtered out from live load analysis.

- Due to the limited deployment of truck platoons on highways, synthetic WIM data was used to simulate truck platoon scenarios. This process consisted of rearranging the actual Class 9 trucks with specific headway distances, penetration rates, and number of trucks within a platoon. A statistical live load model was derived for 19 scenarios, including one actual traffic scenario without platoons and 18 synthetic scenarios with platoons.

- Truck platoons may cause higher bridge load effects as compared to existing baseline loading for longer span lengths (i.e., in excess of 120ft). However, the Tier 2 simulation shows that the platoon loading bias factors and COVs decrease as span length increases. This suggests that HL-93 will envelope the load effects of platoons for design and evaluation, but it will be confirmed by the full calibration in Chapter 4. Among the various factors, the headway distance has the most significant impact on the load effects, followed by the number of trucks within the platoon. The penetration rate seems has little influence.

- The delta factor (Δ-factor) was introduced as a new parameter to understand the difference between the impact of single trucks and truck platoons on bridge loads. The continuous span negative moment has the highest Δ-factor for bias factors, indicating significant impact of truck platoons on this specific load effect. However, the bias factor for the continuous span negative moment is always less than 1.0, suggesting that the standard HL-93 loading can envelope the effects of truck platoons. This means the practical impact on bridge reliability is lessened.

- In the upper tail (most stressful) loading events, there is an increasing trend of truck platoons as span length increases, indicating that platoons are a significant source of the most critical load effects on bridges.

- Overall, the Tier 2 analysis suggests that truck platoons may impact the Fatigue I limit state. This is based on increased frequency of load effects above 1.75 times the AASHTO LRFD Fatigue Truck moment for longer spans. This may have implications for bridge design, maintenance, and long-term serviceability of longer span steel bridges.

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Chapter 4: Calibration and Other Effects

4.1 Reliability-based Calibration

In the LRFD Spec. (23 CFR 625.4(d)(1)(v)) safety is provided in terms of load and resistance factors determined in the reliability-based calibration. The acceptance criterion for safety factors is to keep the probability of failure at an acceptably low level to meet the target reliability index. Code calibration is conducted for a limit state function, which is the mathematical representation of the acceptable and unacceptable performance. In a simple case, when R represents the resistance, and L represents the load, the limit state function can be formulated as:

$$g = R - L \geq 0 \qquad \text{Eq. 18}$$

where:

g —limit state function

R — resistance

L — load effect

The value of g is the safety margin, so if the value of the limit state function is greater than or equal to zero the structure is safe, but if it is less than zero it fails. Thus, in the LRFD Spec. (23 CFR 625.4(d)(1)(v)) safety is provided by load and resistance factors, where the limit state function can be expressed as below.

$$\gamma \cdot L_n \leq \phi \cdot R_n \qquad \text{Eq. 19}$$

where:

γ —load factor

ϕ —resistance factor

L_n — nominal load effect

R_n — nominal resistance

In probabilistic calibration, both load and resistance are treated as random variables, depending on material properties, quality of workmanship, accuracy of the analytical model, etc. To address the variability, statistical models are developed to describe these random variables. Statistical parameters such as the bias factor, the COV, and the distribution type make up the statistical characteristics of load and resistance. The bias factor is the ratio of the actual to nominal/predicted value. The coefficient of variation is a measure of the variability of the data and is equal to the standard deviation divided by the mean. Additionally, the distribution type is used to determine the probability of failure, critical for the reliability assessment. Statistical parameters are used in probabilistic calibration of the design codes to provide the minimum safety margin considering variability of random variables. By incorporating probabilistic methods and considering statistical models, the calibration process results in appropriate safety factors, effectively accounting for uncertainties and mitigating the risk of structural failures.

The probability of failure is equal to the probability of a limit state function being below zero, and it can be calculated using one of the available reliability analysis procedures (Nowak and Collins 2012). Techniques like Monte Carlo simulation enable the consideration of diverse distributions in assessing safety level. Rather than probability of failure, safety is characterized by a reliability index, β. This is the parameter that is used to keep the probability of failure at an acceptable level.

The calculation of the reliability index is a key step in the probabilistic calibration. It entails the definition of the limit state function and the development of statistical parameters for all of the random variables. The simulation technique needs to be selected, followed by the calculation of load and resistance factors to achieve a minimum acceptable reliability index β, not less than the target reliability index, β_T. The target reliability index in general depends on the consequences of failure and incremental cost. The general procedure for reliability-based calibration is illustrated in Figure 73.

Specific input parameters for each calibration step are described in detail below.

- **Formulation of Limit State Function.** Strength Limit State I for the MBE (23 CFR 650.317) at the operating level for design load (HL-93).

- **Selection of Representative Bridges.** The RT reviewed US bridge inventory data to determine representative structural types, and lengths to account for the variability of bridge inventory. Bridge span lengths from 80 feet to 300 feet in 20 feet increments are considered, using steel, prestressed concrete, or reinforced concrete. Simple span and two span continuous bridges are considered.

- **Determine Load and Resistance Parameters.** For Strength Limit State calibration, the load and resistance parameters include dead load, live load, and resistance.

- **Develop Statistical Models for Load and Resistance.** The dead load statistical model is developed for prefabricated, and cast-in-place elements as well as wearing surfaces. These are consistent with previous AASHTO code calibrations. The dead load model appropriate for the material and the span length is developed based on the available US bridge inventory data. The platoon live load model is based on the synthetic WIM data analysis developed in Chapter 3. It considered 2, 3, and 4 trucks platoons and headway spacing of 30 and 50 ft. Resistance model for steel, prestressed concrete, and reinforced concrete for flexure and shear are considered based on previous AASHTO code calibrations.

- **Development of Reliability Analysis Procedure.** The calibration procedure uses a Monte Carlo simulation. Random variables including dead load, live load, and resistance are simulated using the distributions and statistical parameters described above to evaluate the structural safety in terms of reliability index.

- **Selection of Target Reliability Index.** Target reliability, β = 2.50 is selected for Strength Limit State I consistent with the MBE (23 CFR 650.317) at the operating level.

- **Calculation and Selection of Live Load Factor.** The calibration results in a live load factor to envelope platoon loading that provides safety at the target level.

- **Calculation of Reliability Indices.** The calibration procedure can also be used to examine the reliability index that results from specific load factors. The calculated live load factor, and MBE (23 CFR 650.317) live load factor (γ_{LL} = 1.35) will be checked for all calibration cases to determine the level of reliability and provide recommendations for bridge rating under platoon loading.

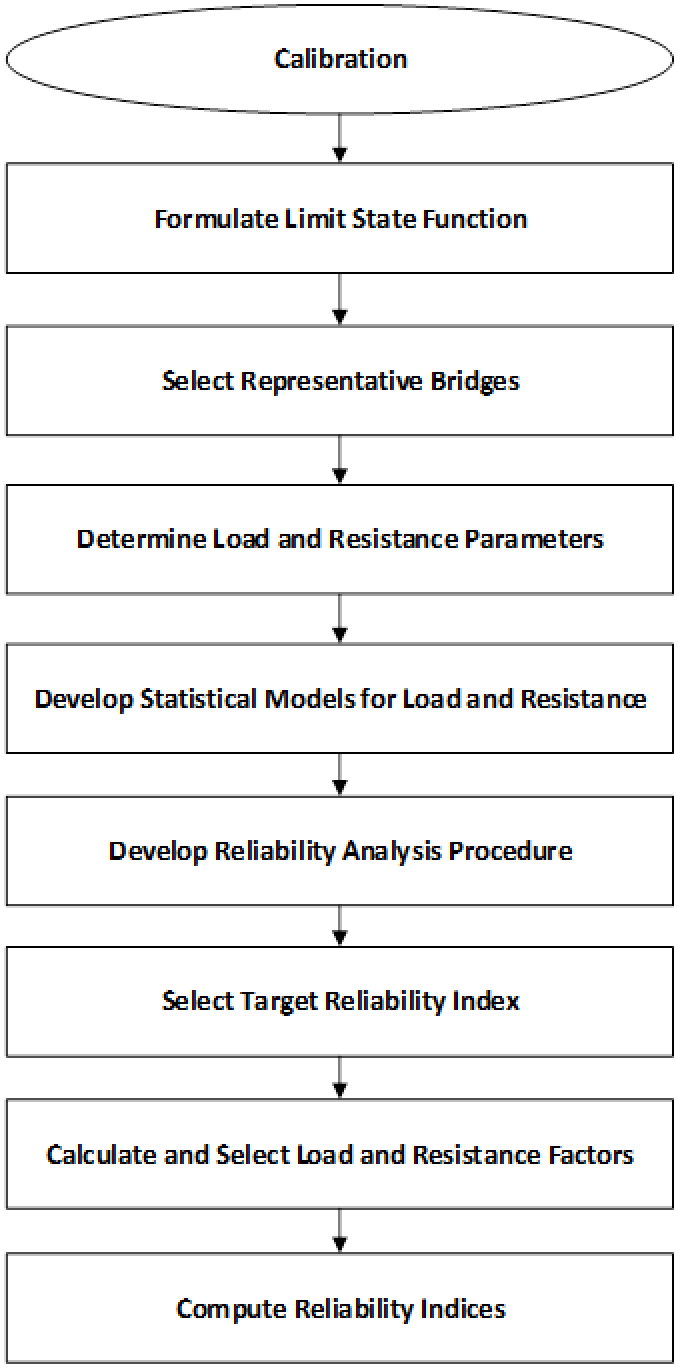

Figure 73. General procedure for reliability-based calibration.

4.2 Platoon Live Load Model

The platoon live load model was developed using representative US WIM data focusing on 5-axle trucks with a Gross Vehicle Weight less than 100,000 lbs. to create 2, 3, and 4 truck platoons with a headway spacing of 30 and 50 ft. The WIM data analysis considered variable penetration rate, which is a percentage of 5-axle trucks grouped into platoons. The penetration rates of 0%, 25%, 50%, and 100% were studied.

The synthetized platoons along with the other individual WIM trucks were run on an influence line analysis to determine the live load envelope for simple and continuous span bridges with lengths from 80 to 300 ft in 20 ft increments. The WIM live load was compared with the HL-93 nominal loading to determine platoon live load statistical parameters. The platoon live load model is determined by the bias factor which is a ratio of the maximum expected load to nominal load, and the COV, computed using statistical data from selected WIM sites from different locations. Calculated bias factors were determined for various ADTT using 250, 1000, 2500, 5000, 10000, and 15000. Developed statistical parameters are reviewed to determine critical platoon cases accounting for penetration rate, number of trucks, headway spacing, ADTT, load case, and span length.

In the development of the live load model, the ADTT is used in the extrapolation process. ADTT plays an important role in determining the maximum expected load effects. The extrapolation process considers the ADTT and return period of the synthetic WIM to determine maximum expected live load ratio related to the set probability of failure. The return period is established as 5-years for MBE (23 CFR 650.317) (inspection period) and 75 years for LRFD Spec. (23 CFR 625.4(d)(1)(v)) (service life of a bridge). The bias factor increases with ADTT; thus, this parameter is significant in determining live load statistical parameters. The developed WIM-based platoon load model accounts for six different ADTTs. The RT recommends using ADTT of 5,000 to maintain consistency with the original MBE (23 CFR 650.317) calibration, and this is what was used in the present calibration.

Penetration rates of 0%, 25%, 50%, and 100% were used in the development of the live load statistical parameters. The 0% penetration rate (i.e., Base WIM) means there are no synthetic platoons, and a 100% penetration rate converts all 5-axle trucks with a Gross Vehicle Weight of less than 100,000 lbs. to form platoons. To understand the impact of the penetration rate, the developed live load statistical parameters were compared for variable penetration rates. This comparison concludes that bias factors for different penetration rates yield comparable values. It means that an increase in penetration rate does not contribute to the additional increase in loading. Since the maximum platoon loading effects are captured at the 25% penetration rate, as well as 100%, a single penetration rate of 100% was used in the calibration.

Representative span lengths ranging from 80 ft to 300 ft were selected for the analysis. The minimum span length of 80 ft. was chosen as smaller span lengths cannot generally accommodate more than one truck at a time, and thus are not influenced by platoon loading. For the maximum spans, the upper limit is 300 ft as 99.5% of all US bridges have equal or lesser maximum span lengths.

Bias factors were developed for both Base WIM, and six distinct platoon configurations including 2, 3, and 4 truck platoon configurations spaced 30, and 50 feet apart. These factors are presented versus span length in Figure 74 to Figure 78 for all considered load cases. It can be noted that the bias factor for Base WIM and platoon remains constant for span lengths from 80 feet to 120 feet. However, the difference in the Base WIM and platoon cases becomes noticeable for spans equal to or longer than 140 feet. Therefore, only span lengths from 140 to 300 feet are considered.

The calibration considers five different load cases. Figure 79 shows the developed bias factor for every platoon and load case. The legend describes load cases where M indicates simple span moment, S simple span shear, M+ continuous span positive moment, M- continuous span negative moment, and Sc continuous span shear. Shorter spans yield a larger bias factor. For longer spans, where maximum live load is dictated by platoons, the 3 and 4 truck platoons spaced 30 ft apart show the largest bias factors. In terms of load case, the continuous span positive moment, and shear load cases have the largest bias. The lowest bias factor is seen for the continuous span negative moment case, where the governing load case considers 90% of two HL-93 design trucks spaced at least 50 feet apart. The bias factor for the negative moment falls below one, indicating that all platoon cases are encompassed by the HL-93 truck train live load model.

The Base WIM and selected platoon load cases were investigated in the full calibration process to determine the live load factor that provides the minimum target safety. Therefore, platoon cases for 2, 3, and 4-truck platoons with headway spacing of 30 and 50 ft, span lengths from 140 to 300 ft, and five load cases were considered in the probabilistic calibration.

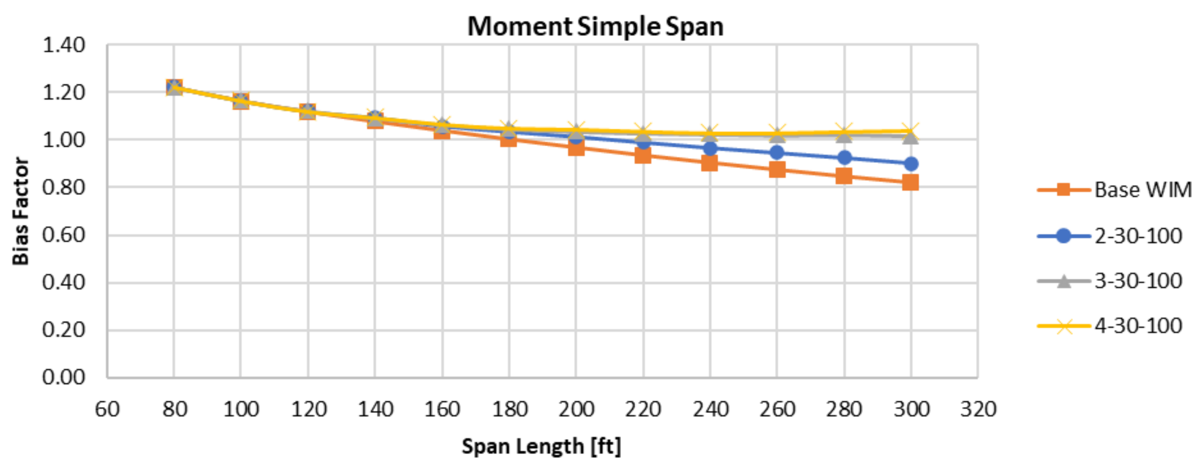

Figure 74. Bias factor vs span length for simple span moment for Base WIM and platoons with headway 30ft and 100% penetration.

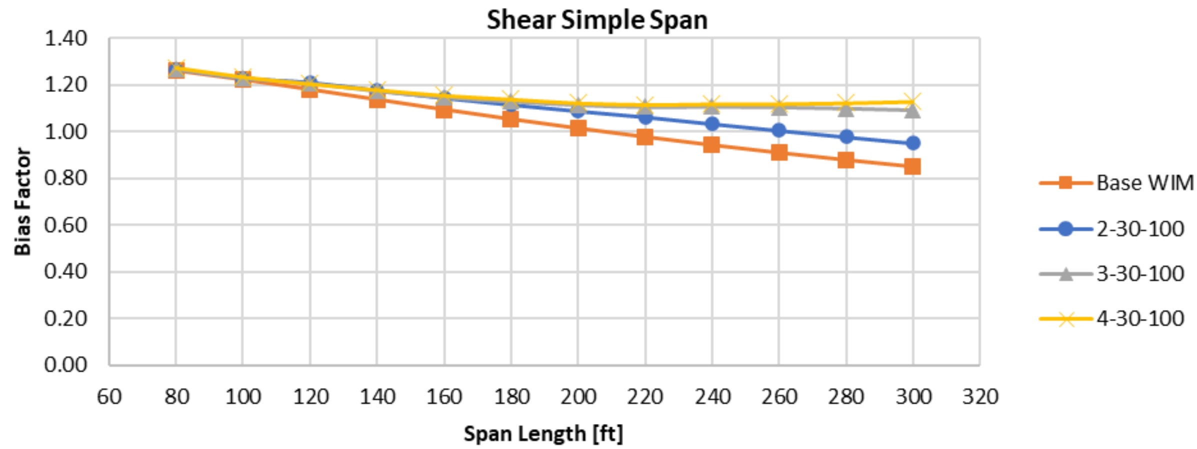

Figure 75. Bias factor vs span length for simple span shear for Base WIM and platoons with headway 30ft, and 100% penetration.

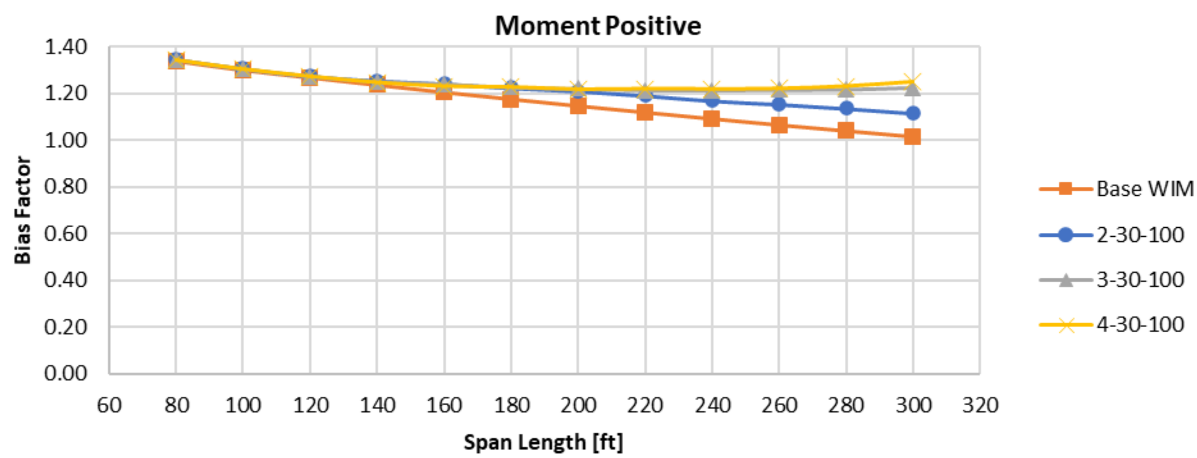

Figure 76. Bias factor vs span length for continuous span positive moment for Base WIM and platoons with headway 30ft and 100% penetration.

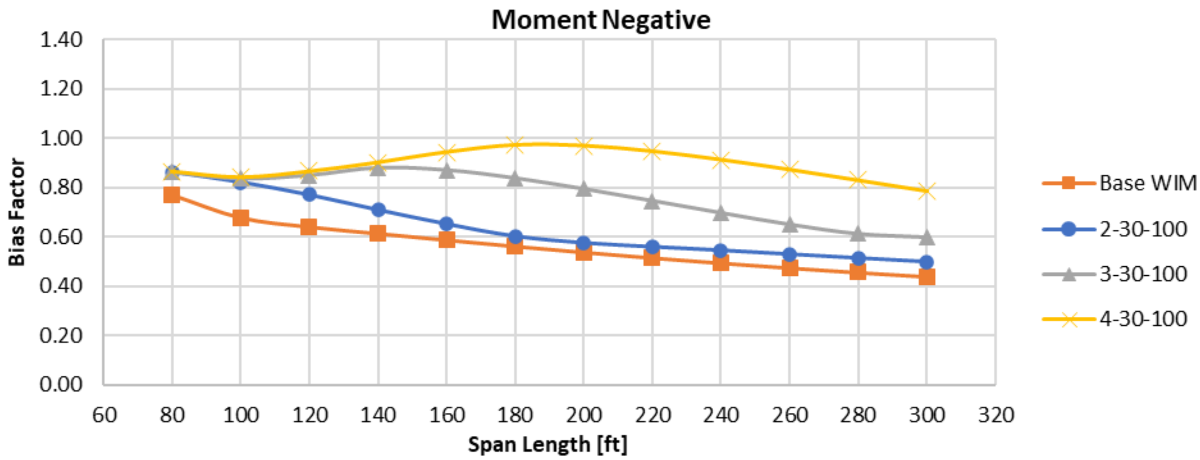

Figure 77. Bias factor vs span length for continuous span negative moment for Base WIM and platoons with headway 30ft and 100% penetration.

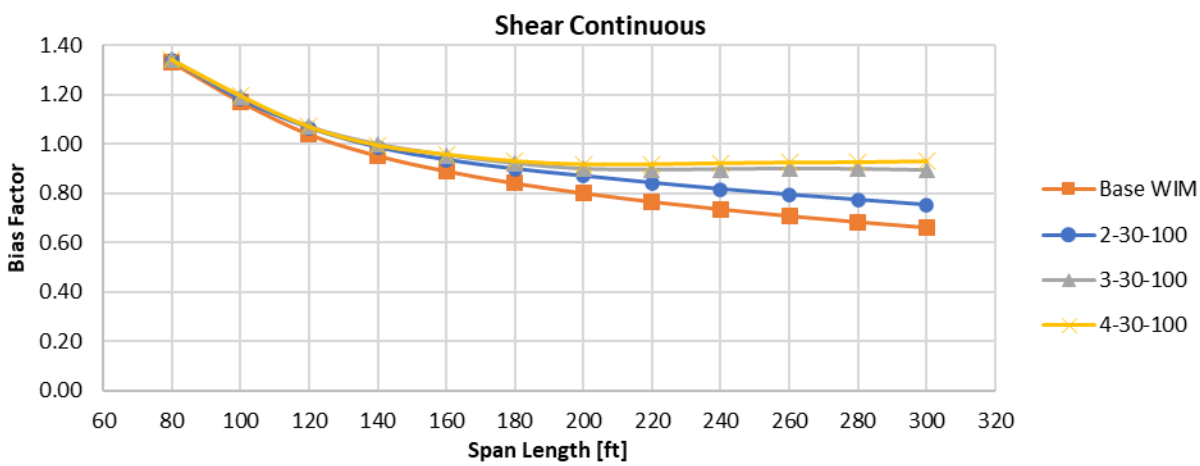

Figure 78. Bias factor vs span length for continuous span shear for Base WIM and platoons with headway 30ft, and 100% penetration.

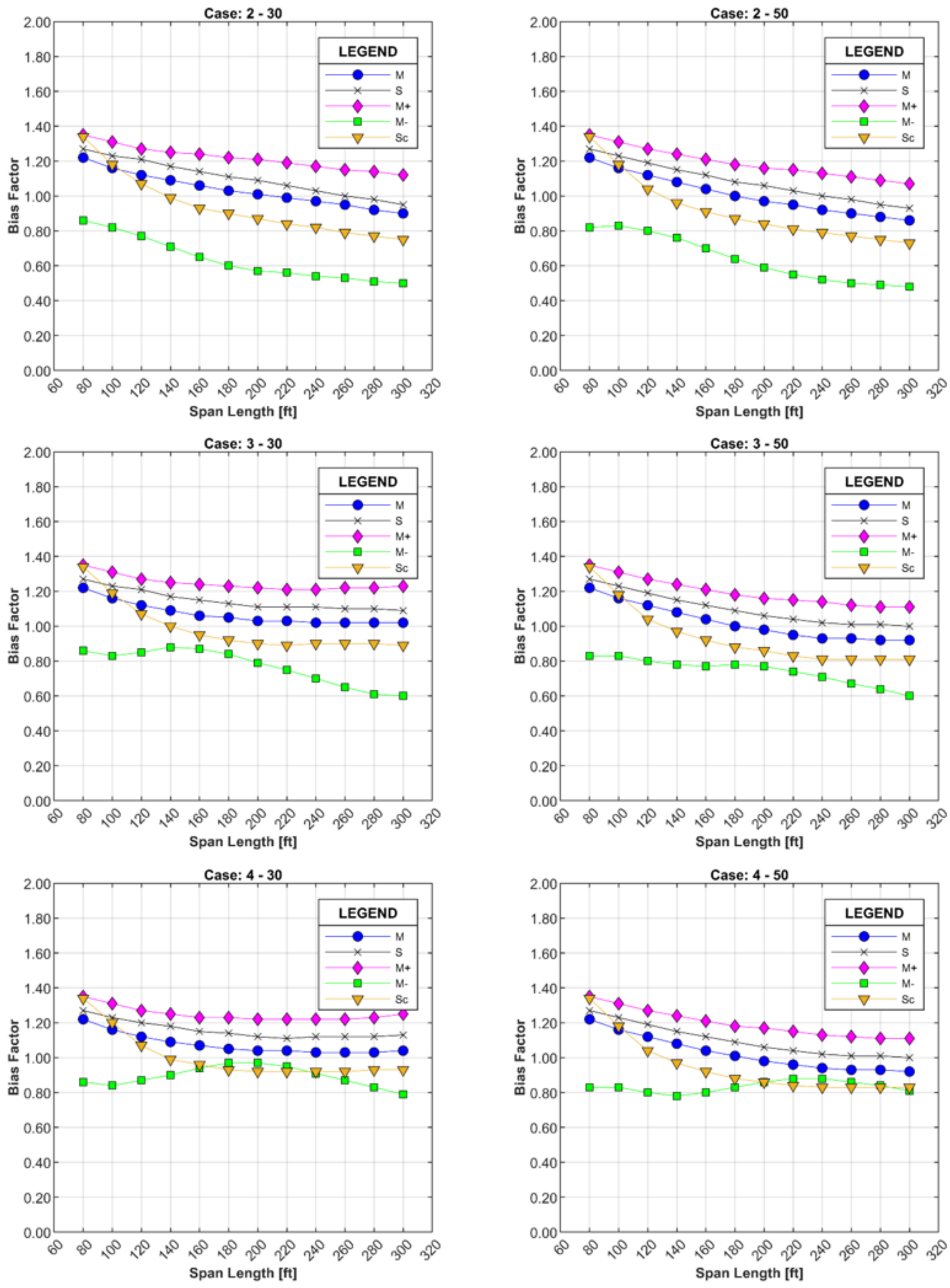

Figure 79. Bias factor vs span length for all platoon cases considered in the calibration.

4.3 Dead Load Model

The dead load model aims to account for the variations between predicted and actual self-weight. In the original AASHTO code calibration, the dead load model distinguished between prefabricated and cast-in-place elements, as well as wearing surface. The MBE (23 CFR 650.317) dead load factors are separate for structural components and wearing surface. Table 43 presents the dead load statistical parameters and load factors for the probabilistic calibration. Dead load is assumed to be normally distributed. These parameters are consistent with previous calibration efforts (NCHRP Report 368).

Dead load varies not only with material type but also with span length. While shorter bridges are primarily influenced by live load, the significance of dead load becomes more pronounced in longer span bridges. This correlation between dead load and live load is investigated to refine the accuracy of the dead load model. To achieve this, extensive bridge inventory data is utilized. It includes dead load to live load ratios for a broad range of bridges. This database serves as a basis for the development of a dead load model for this study. The development of the material and span-based dead load model is described in the next section.

Table 43. Dead load statistical parameters and load factors.

Dead Load Component	Bias Factor	COV	Load Factor
D1: Prefabricated	1.03	8%	1.25
D2: Cast in place	1.05	10%	1.25
D3: Wearing surface	1.00	25%	1.50

4.3.1 Representative Bridges Database

Using a representative set of bridges is important to account for the variability of US bridge inventory when conducting the probabilistic calibration. The dead load model used for this study is determined based on the available bridge data. Over three thousand data points of dead load to live load ratios for approximately a thousand bridge structures were used. The data includes information about maximum bridge span length, the number of bridge spans, material type, structural type, as well as moment and shear dead load to live load ratios. The dead load to live load database will be further referred to as "D/L data". The RT reviewed the bridges and compared them to the 2021 NBI database with approximately five hundred thousand bridges across the US.

Figure 80 presents histograms of maximum bridge span length for both NBI and D/L data. Available D/L data contains a representative set of the span lengths that cover representative span lengths for the US bridge inventory. The RT also checked the bridge structural types included in the D/L data and compared them with NBI. Figure 81 shows the representative bridge structural types, and the types included in D/L data. The nomenclature of bridge structural type differs between these two databases, but this comparison shows that D/L data include representative bridge types to develop a comprehensive dead load model for the calibration procedure.

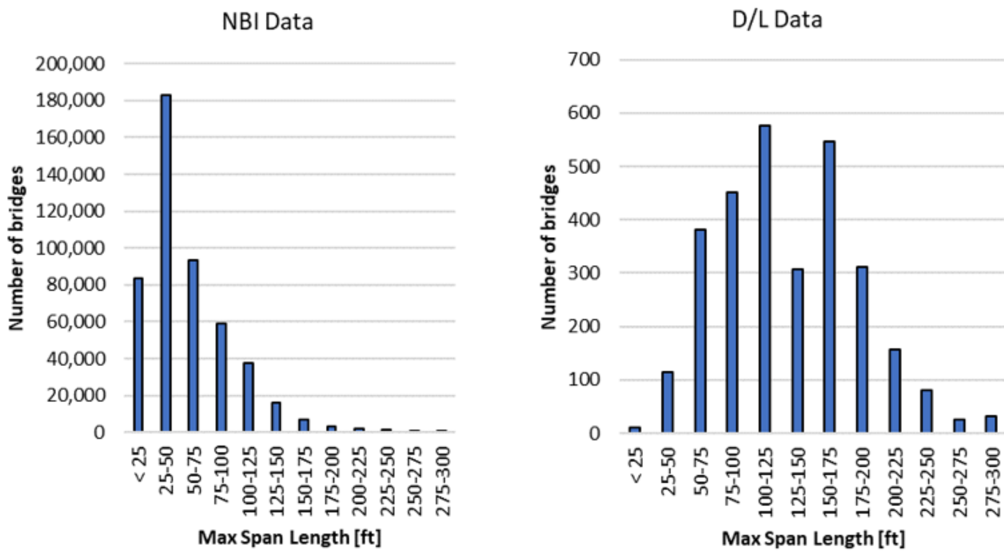

Figure 80. Histogram of the maximum bridge span length per National Bridge Inventory data for the US, and available D/L data.

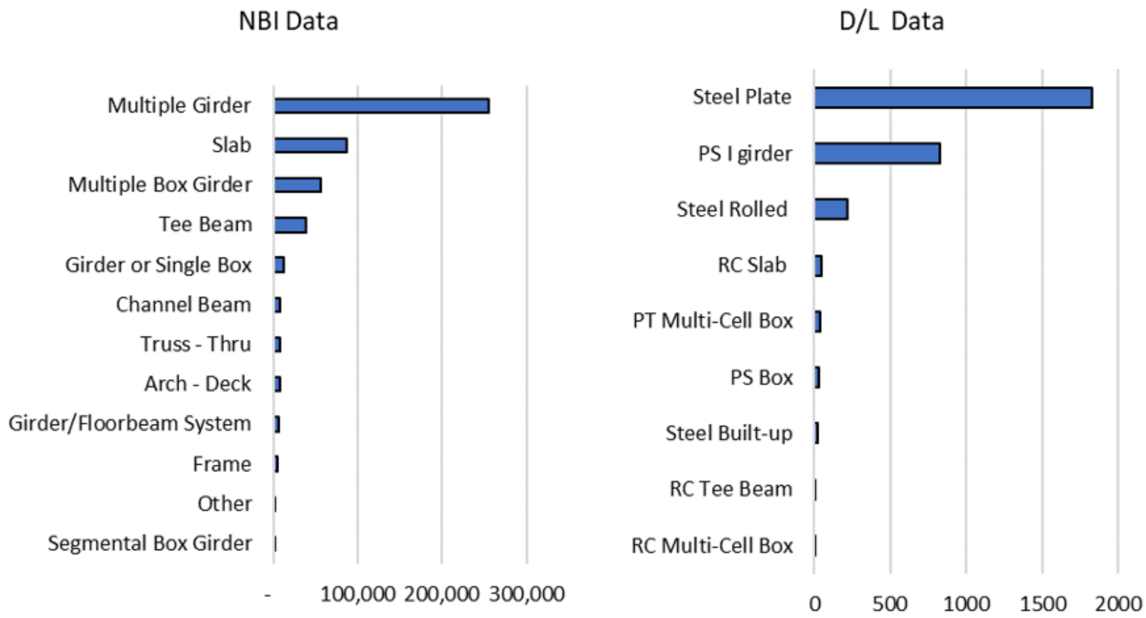

Figure 81. Bridge structural types per National Bridge Inventory data for the US, and available D/L data.

4.3.2 Dead Load to Live Load Ratio

The representative D/L data include the dead load to live load ratio for over three thousand data points. The dead load and live load ratios for different bridge span lengths, and structural types are evaluated to determine the correlation between these parameters. Figure 82 and Figure 83 present scatter plots of the dead load to live load ratios and span length for moment and shear forces for dead load structural components (DC), and dead load wearing surface (DW), respectively. The correlation is not apparent therefore, in the next step of the analysis, D/L ratios were separately investigated for steel and concrete bridges, as the dead load can vary significantly for these two groups.

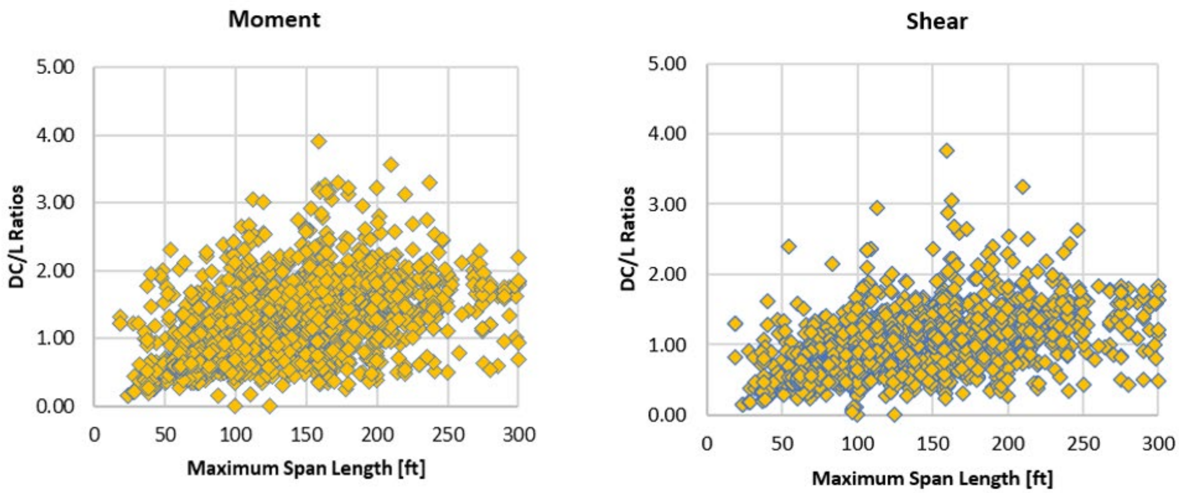

Figure 82. Dead load to live load ratio (DC/L) for structural components vs bridge maximum span length for moments, and shear effects.

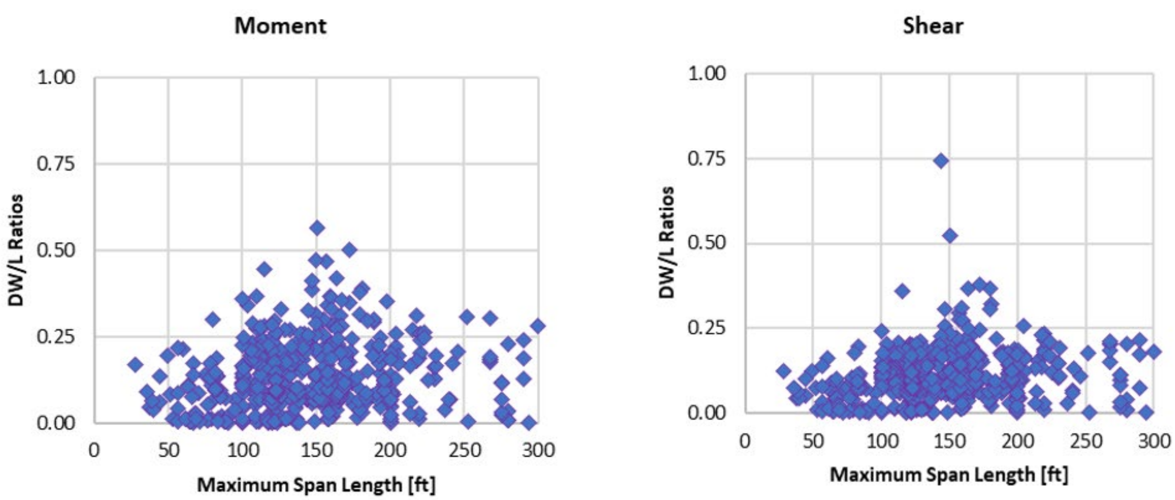

Figure 83. Dead load to live load ratio (DW/L) for wearing surface vs bridge maximum span length for moments, and shear effects.

To better understand the correlation between dead load, bridge material type, and span length, the dead load to live load ratios for steel bridges were grouped for span ranges from 80-300 ft in 20-ft increments. Figure 84 and Figure 85 present box plots of D/L ratios for different span length ranges in flexure, and shear for steel bridges. The plots show that there is no linear correlation between dead load and span length, but there is a specific range of dead load to live load ratios for every span range. There is a natural variability associated with each bridge span length, and that can be included in dead load model. Thus, it is proposed to use a variable D/L ratio for selected span ranges to better represent US bridges.

In this case, D/L ratio distributions for span length groups were developed separately for steel and concrete bridges. The representative D/L range for each span length group is characterized by the minimum and maximum value of the D/L ratio. The minimum is taken as 5th, and the maximum 95th percentile from dead load to live load ratio distributions. Due to the variable sample size of D/L data for span length groups minor adjustments were introduced to capture the trend between D/L and span lengths. For the small sample size groups, the interpolation method was used to determine representative D/L ranges.

The available D/L data was analyzed for 1) Material Type: separate models for steel, and concrete bridges, 2) Dead Load: DC - structural components, and DW - wearing surface, and 3) Load effects: flexure and shear. Table 44 and Table 45 present the ranges of representative dead load to live load ratios with minimum and maximum values for DC/L for steel and concrete bridges. Similarly, DW/L ratios were considered for steel and concrete bridges, but the results were comparable, hence DW/L ratios are considered the same for steel and concrete bridges. Table 46 shows the DW/L ranges for dead load wearing surface in flexure and shear. Figure 86 presents the range of the DC/L for steel and concrete bridges in flexure.

The developed ranges of dead load to live load ratios allow for the simulation of the variability of dead load associated with selected groups of span ranges. The dead load model for the full calibration accounts for the statistical parameters and load factors shown in Table 43 as well as the developed range of the dead load to live load ratios presented in Table 44, Table 45, and Table 46. The variability associated with the component type (prefabricated, cast-in-place, wearing surface), material type (steel, concrete, and wearing surface), as well as the span range will be accounted in the developed dead load model. The ranges of D/L ratios were applied to the simple span and continuous bridges as the base data set included variable bridge structural types. The explicit use of the developed dead load model is presented in the calibration section.

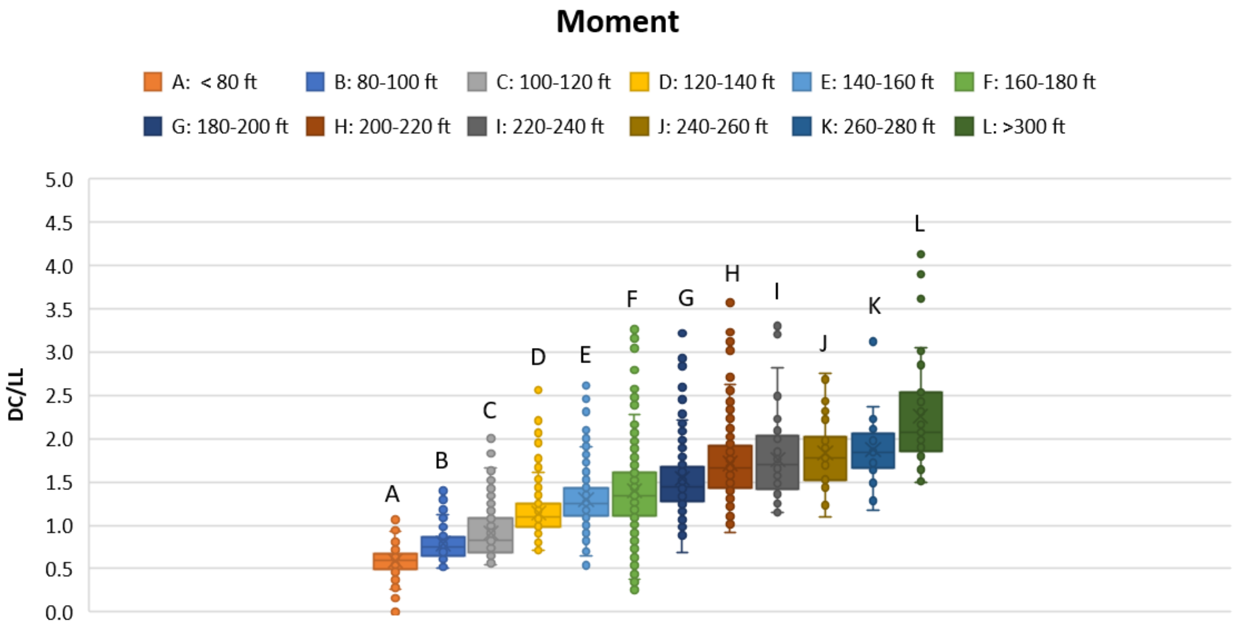

Figure 84. Dead load to live load (DC/L) ratio ranges for flexure in steel bridges.

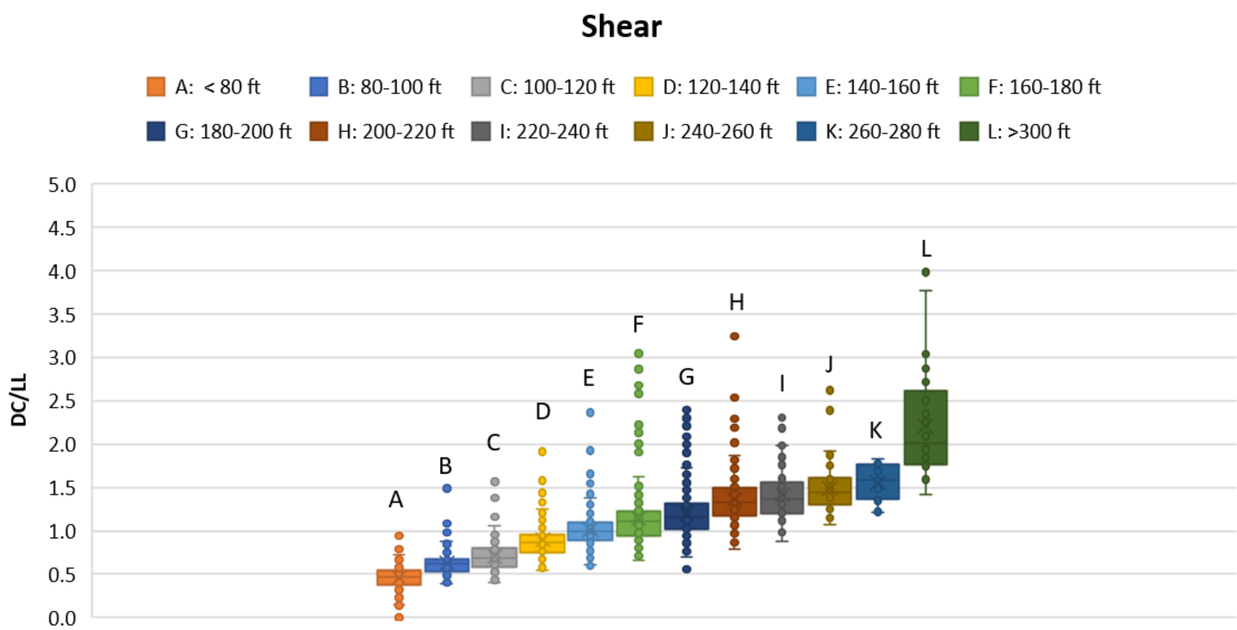

Figure 85. Dead load to live load (DC/L) ratio ranges for shear in steel bridges.

Table 44. DC/L ranges for flexure and shear for steel bridges.

Span [ft]	DC/L Moment Min Ratio	DC/L Moment Max Ratio	DC/L Shear Min Ratio	DC/L Shear Max Ratio
80	0.40	0.80	0.30	0.70
100	0.60	1.20	0.50	0.90
120	0.70	1.40	0.55	0.95
140	0.90	1.60	0.70	1.20
160	1.00	1.80	0.80	1.30
180	1.10	2.10	0.90	1.50
200	1.20	2.20	1.00	1.55
220	1.30	2.30	1.10	1.70
240	1.35	2.35	1.15	1.80
260	1.50	2.40	1.20	1.90
280	1.55	2.45	1.30	1.95
300	1.60	2.65	1.70	2.60

Table 45. DC/L ranges for flexure and shear for concrete bridges.

Span [ft]	DC/L Moment Min Ratio	DC/L Moment Max Ratio	DC/L Shear Min Ratio	DC/L Shear Max Ratio
80	0.50	0.90	0.50	0.80
100	0.60	1.30	0.80	1.10
120	0.80	1.85	0.90	1.40
140	0.90	1.90	1.10	1.50
160	1.00	2.10	1.20	1.65
180	1.15	2.35	1.50	1.70
200	1.25	2.65	1.60	2.30
220	1.40	2.90	1.70	2.50
240	1.52	3.00	1.75	2.70
260	1.60	3.30	2.20	2.90
280	1.75	3.65	2.35	3.15
300	1.90	3.90	2.50	3.30

Table 46. DW/L ranges for flexure and shear for bridges.

Span [ft]	DW/L Moment Min Ratio	DW/L Moment Max Ratio	DW/L Shear Min Ratio	DW/L Shear Max Ratio
80	0.010	0.200	0.010	0.140
100	0.015	0.250	0.015	0.160
120	0.018	0.275	0.018	0.230
140	0.020	0.300	0.020	0.240
160	0.040	0.350	0.040	0.260
180	0.050	0.400	0.050	0.330
200	0.060	0.400	0.060	0.350
220	0.060	0.400	0.060	0.400
240	0.060	0.400	0.060	0.400
260	0.060	0.400	0.060	0.400
280	0.060	0.400	0.060	0.400
300	0.060	0.400	0.060	0.400

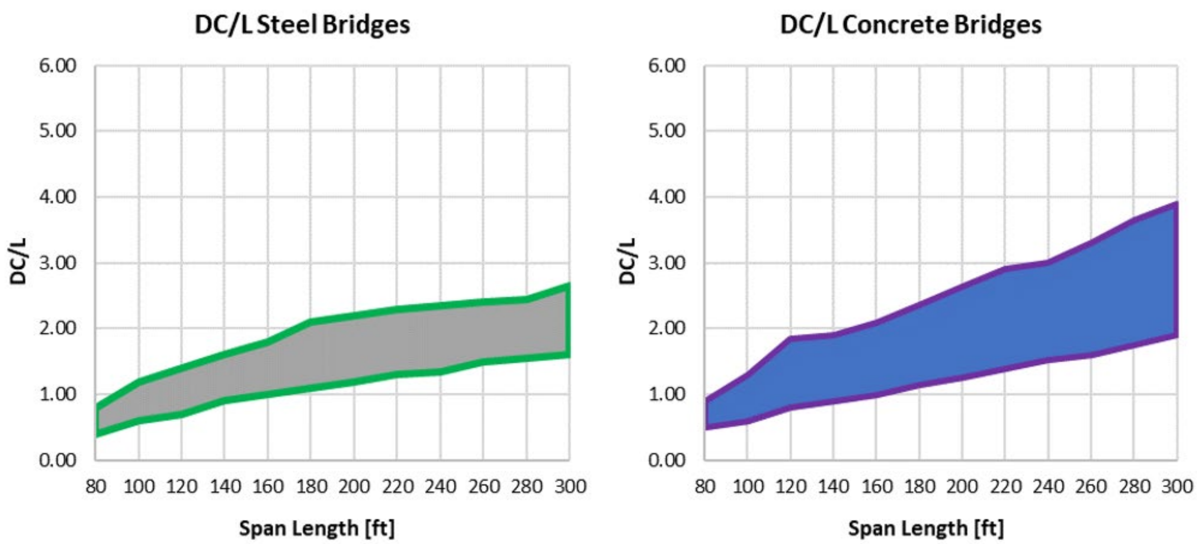

Figure 86. Range of dead load to live load (DC/L) ratios for representative span lengths in flexure.

4.4 Resistance Model

The resistance model can be considered as a function of three factors, reflecting uncertainty in material properties (strength, modulus of elasticity), dimensions (length, moment of inertia), and the accuracy of the analytical model to represent actual behavior. To treat these uncertainties as random variables, the statistical parameters of resistance are determined by lab tests, field measurements, and simulations Resistance parameters are considered for steel, reinforced concrete, and prestressed concrete bridges for various load effects. Resistance follows lognormal distribution. Table 47 presents the resistance statistical parameters that are consistent with the original AASHTO code calibration (NCHRP 368).

Table 47. Statistical parameters of resistance

Bridge Material	Flexure Bias Factor	Flexure COV	Shear Bias Factor	Shear COV
Steel	1.12	10%	1.14	10.5%
Reinforced Concrete	1.14	13%	1.20	15.5%
Prestressed Concrete	1.05	7.5%	1.15	14%

4.5 Calibration Approach

With all input parameters established, Figure 87 presents the flowchart with the iterative reliability-based analysis procedure that was followed. The calibration procedure begins with the calculation of nominal HL-93 live load effects for representative bridge span lengths from 140 to 300 ft, and five load cases. The live load, dead load, and resistance were simulated a million times using the distribution type and developed statistical parameters. Then, dead load is computed based on the nominal live load and representative range of dead load to live load ratios shown in Table 44 to Table 46**.** The uniform values in between minimum and maximum ratios are used to determine nominal dead load. For each ratio, the mean dead load is simulated using the dead load statistical parameters. One million dead load simulations are conducted to account for the whole range of D/L ratios determined by the minimum and maximum values. With the live load and dead load simulation, the iterative procedure begins where the iteration accounts for the variable live load factor ranging from 1 to 2 in 0.05 increments. The nominal resistance is then calculated based on the demand. The nominal values of live load (HL-93) and dead load (computed based on D/L ratios) are used along with the dead load factors, as well as the resistance factor depending on the load case. The required nominal resistance is determined and then simulated using resistance statistical parameters. The live load, dead load, and resistance are simulated a million times using the distribution type, and coefficient of variation described in the statistical models. The simulated load and resistance values are used to calculate the limit state function and determine the probability of failure based on a number of simulated cases with limit state function below zero. For each iteration (different live load factor) the reliability index beta is calculated. The calibration is set up to determine the live load factor that provides a reliability index close to the target reliability index of 2.5.

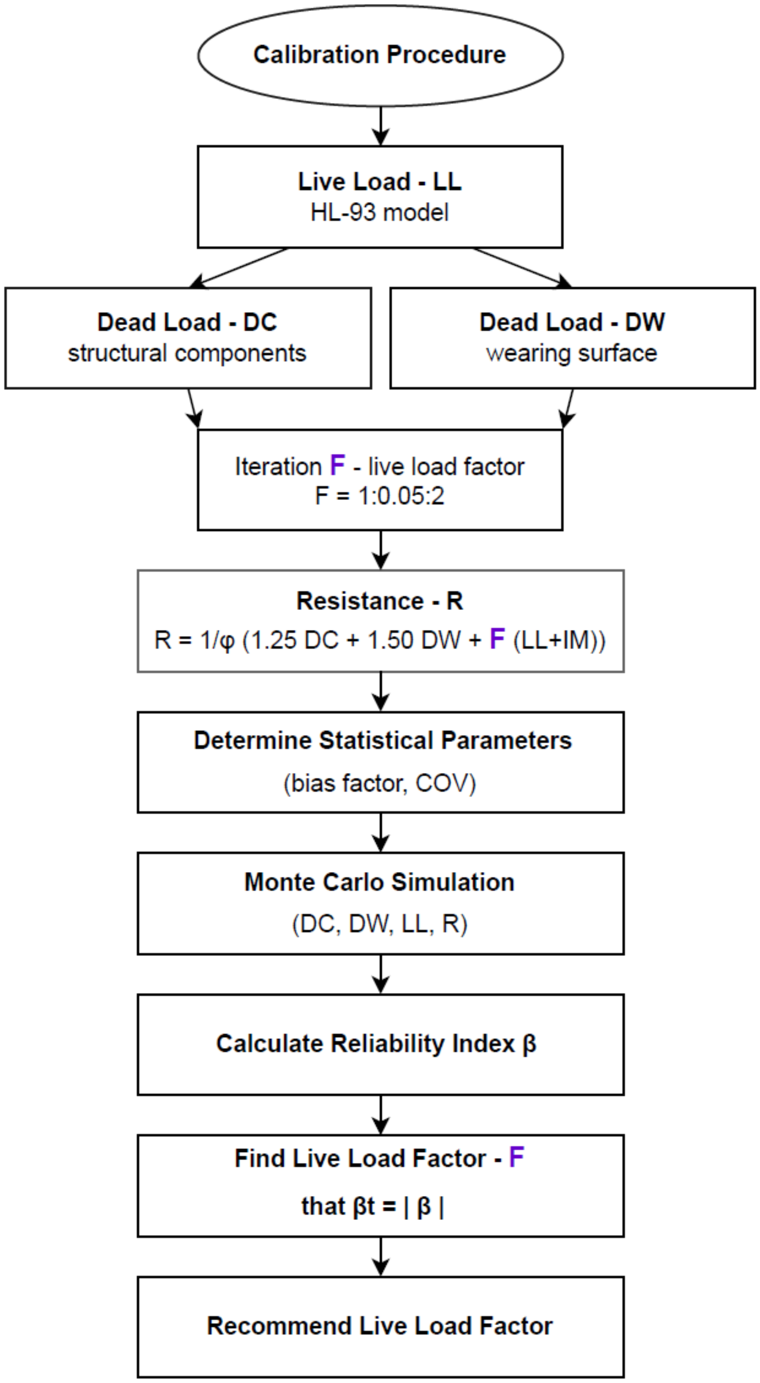

Figure 87. Reliability-based calibration procedure.

4.6 Calibration Results

The developed calibration procedure was carried out with two different approaches: 1) Calculate the live load factor to achieve a target reliability index of 2.50 per MBE (23 CFR 650.317) Operating Level, and 2) Calculate the reliability index with a live load factor of 1.35.

This section presents the results of the calibration for Base WIM (i.e., no platoons) and all considered platoon cases, including 2, 3, and 4 truck platoons spaced 30 ft and 50 ft apart. The platoon configurations are denoted as ‘a number of trucks – headway spacing’. For example, a 2-30 platoon represents a platoon configuration of two trucks spaced 30 ft apart, while the Base WIM is denoted as case 1-1. The results are presented for span lengths ranging from 140 to 300 ft. Live load factors are calibrated for five load cases, including simple span and two span continuous beams..

4.6.1 Calibration of Live Load Factor

In the first approach, the live load factor for HL-93 is computed to encompass platoon loading and provide the minimum level of safety at the target reliability index. Figure 88 to Figure 93 illustrates the calibrated live load factors for considered span lengths, and load cases. The majority of the calibrated live load factors fall below the MBE (23 CFR 650.317) live load factor of 1.35. The largest live load factors are observed for span lengths of 140 ft and 160 ft. It is important to note that these live load factors for platoon cases are practically the same as Base WIM. In other words, the platooning of trucks had little effect in this span range. Longer span bridges, where platoon load effects dominate over single trucks, show live load factors less than 1.35; the HL-93 envelopes the simulated platoon load effects . Also, among all 2-truck platoons, and any platoon with 50 ft headway spacing, live load factors decrease as span lengths increase. For 3 and 4 trucks platoons with headway spacing of 30 ft, the live load factor does not decrease with span lengths, and live load remains constant providing consistent safety in terms of reliability index for platoon loading using HL-93 notional load. Figure 88 to Figure 90 presents calibrated live load factor for Base WIM cases, and Figure 91 to Figure 93 for platoon cases for steel, prestressed concrete, and reinforced concrete bridges respectively.

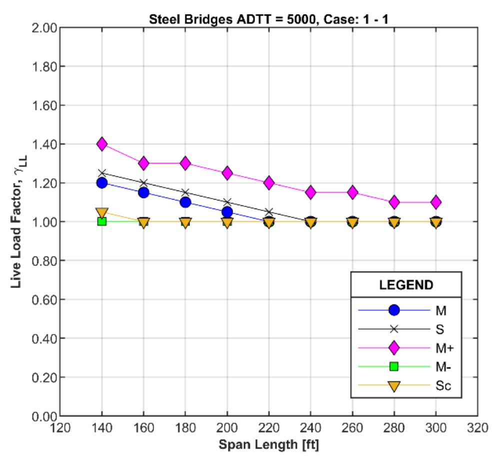

Figure 88. Calibrated live load factor for Base WIM and steel bridges.

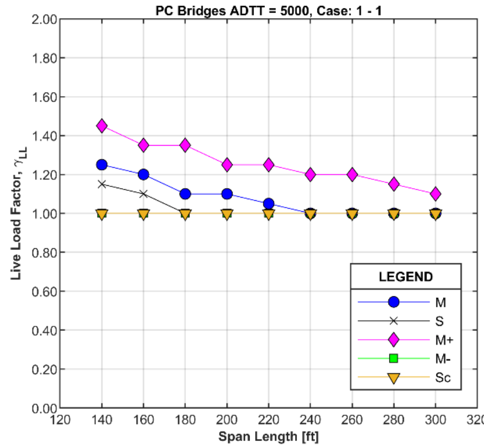

Figure 89. Calibrated live load factor for Base WIM and prestressed concrete bridges.

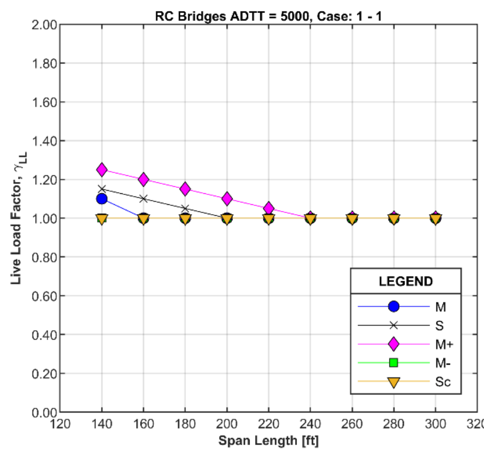

Figure 90. Calibrated live load factor for Base WIM and reinforced concrete bridges.

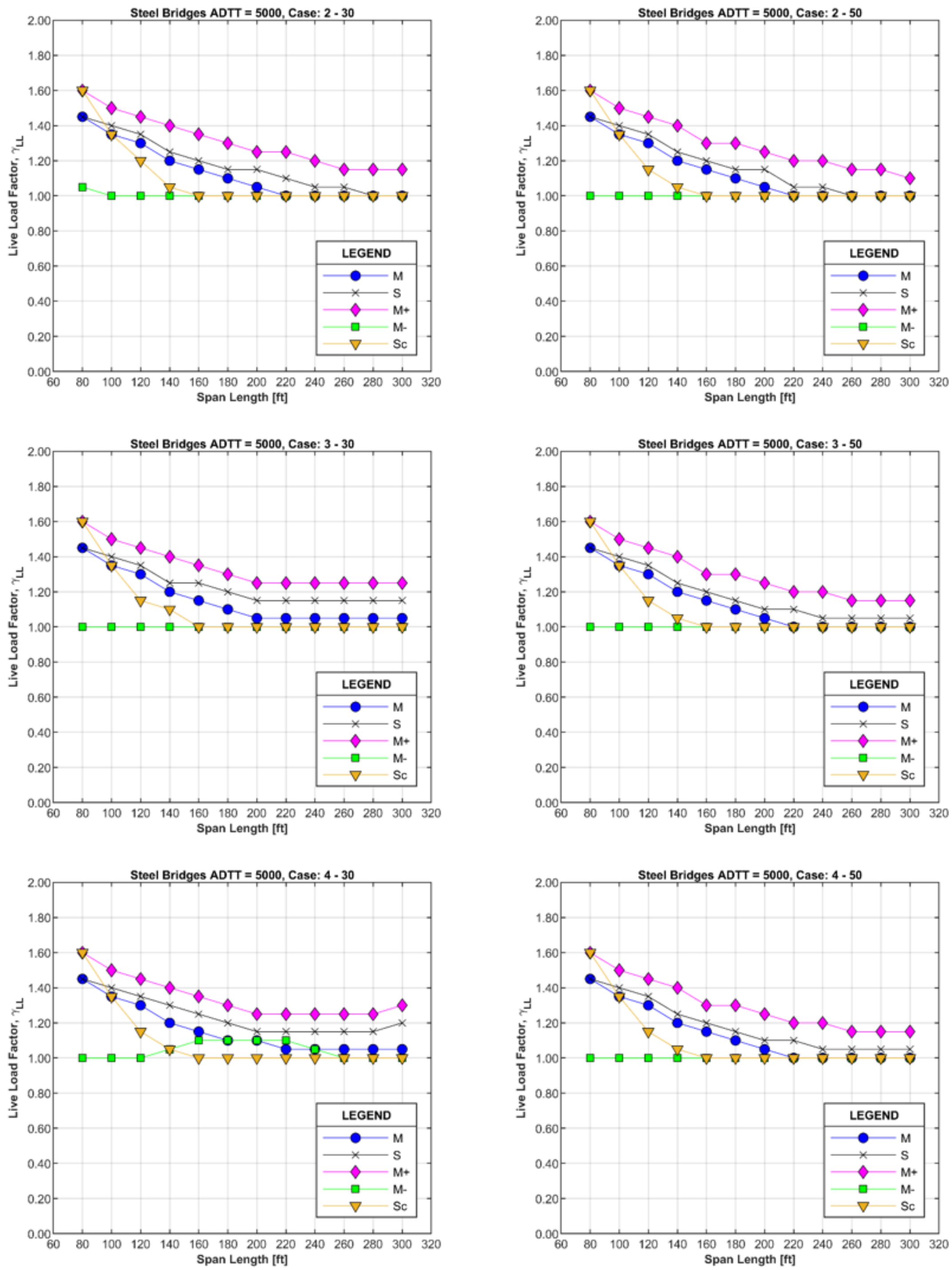

Figure 91. Calibrated live load factor for platoons and steel bridges.

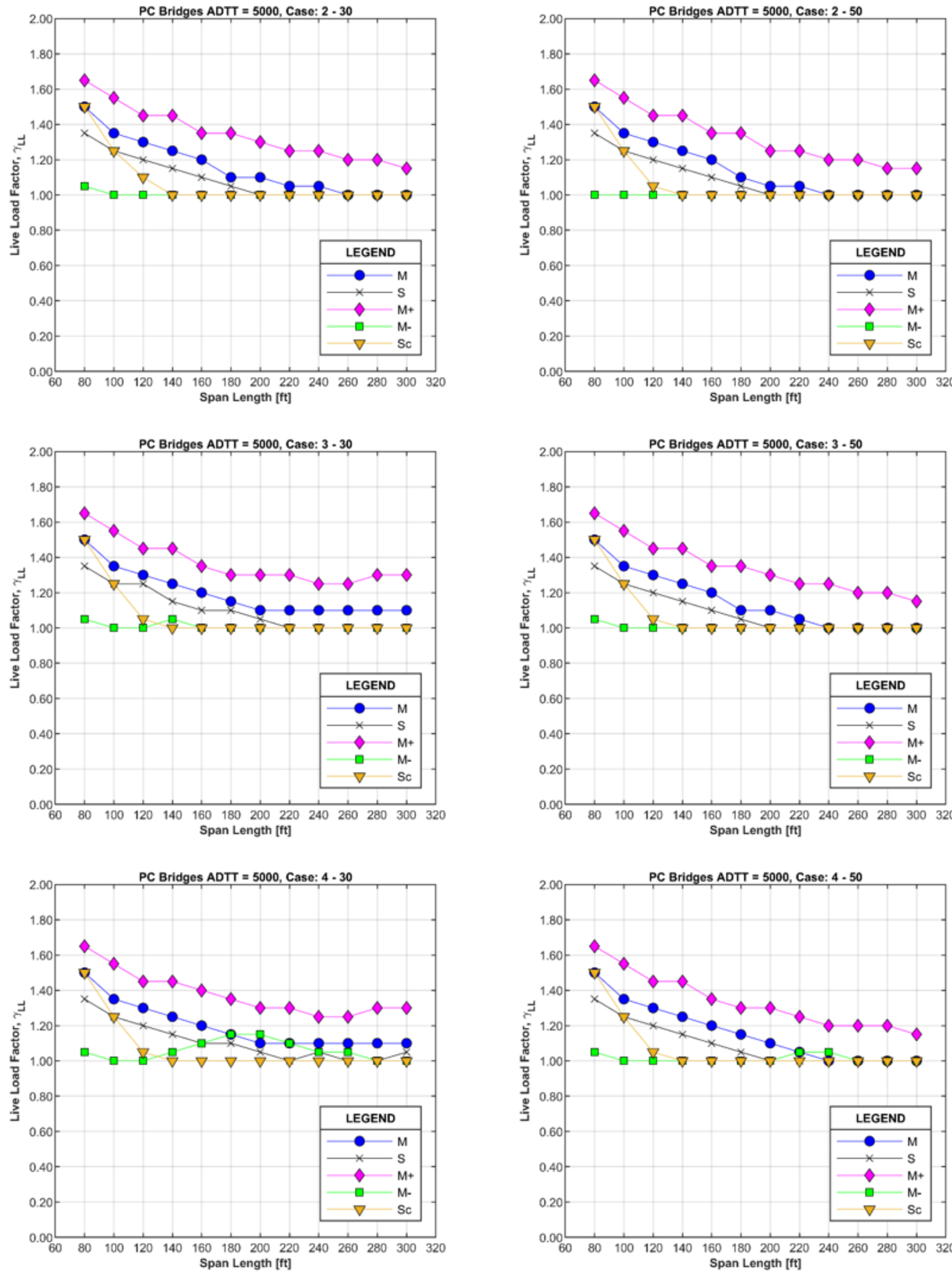

Figure 92. Calibrated live load factor for platoons and prestressed concrete bridges.

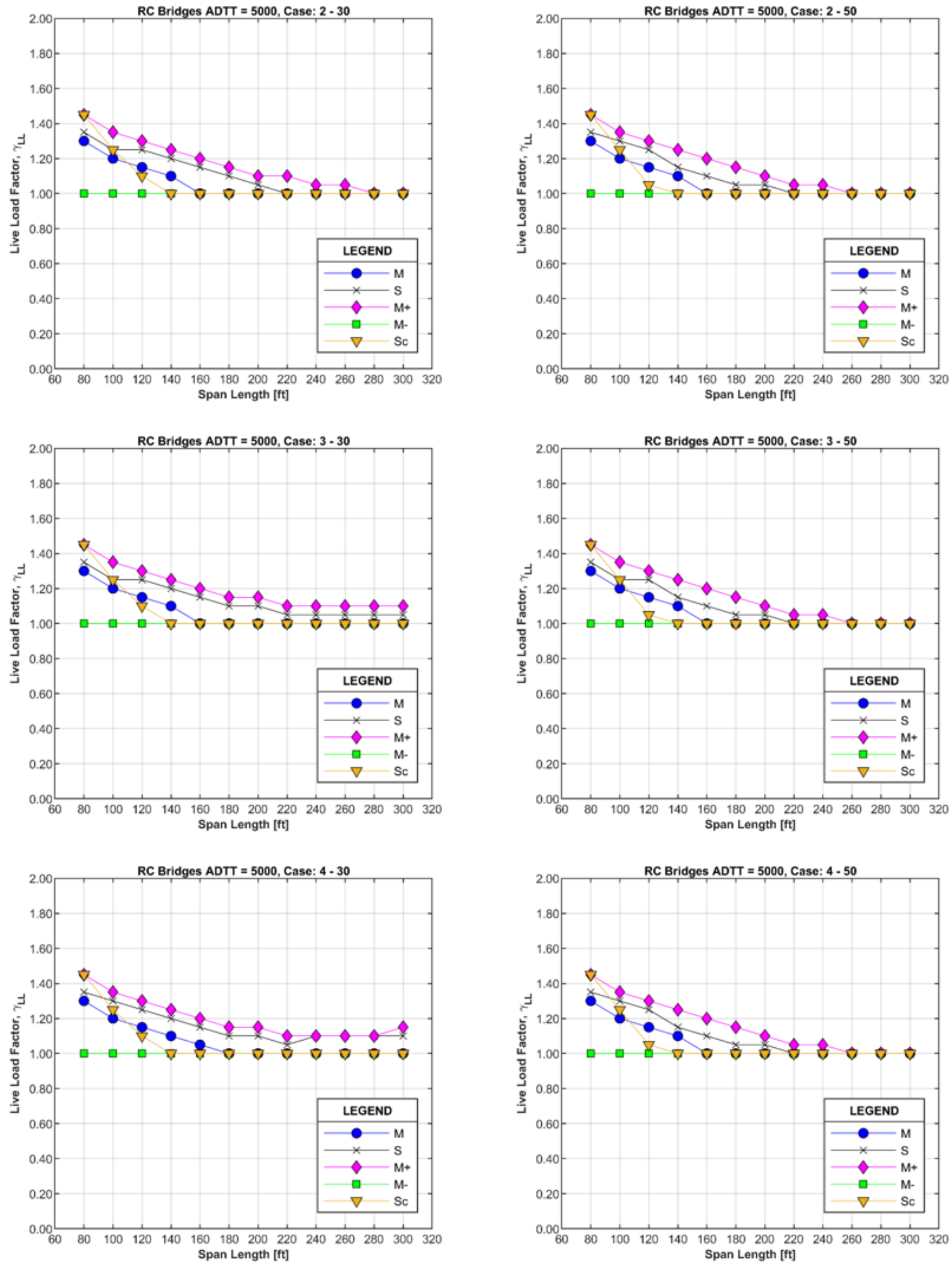

Figure 93. Calibrated live load factor for platoons and reinforced concrete bridges.

4.6.2 Reliability Index with a Live Load Factor of 1.35

The second approach involves selecting an appropriate live load factor and assessing the resulting reliability index. Since most of the calibrated live load factors fall below the MBE (23 CFR 650.317) load factor of 1.35, the reliability analysis for the MBE (23 CFR 650.317) provisions with the platoon loading is conducted to determine the safety in terms of the reliability index. Figure 94 to Figure 96 shows the reliability index for Base WIM, and Figure 97 to Figure 99 presents the calculated reliability index for platoon cases. The MBE (23 CFR 650.317) uses a target reliability index of 2.50 with a minimum threshold of 1.50. Analysis of Base WIM data shows that shorter span bridges exhibit the lowest safety in terms of reliability index. None of the cases show a reliability index less than 1.50. The platoon cases yield more uniform reliability for long span bridges. All platoon cases for long span bridges yield a reliability index larger than 2.50.

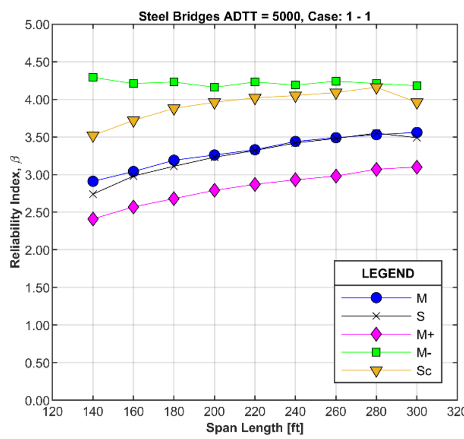

Figure 94. Reliability index for Base WIM and steel bridges for MBE (23 CFR 650.317) live load factor of 1.35.

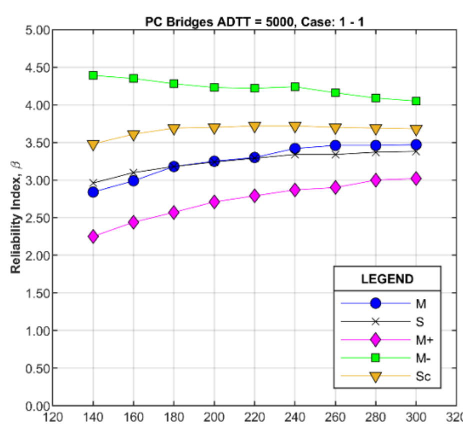

Figure 95. Reliability index for Base WIM and prestressed concrete bridges for MBE (23 CFR 650.317) live load factor of 1.35.

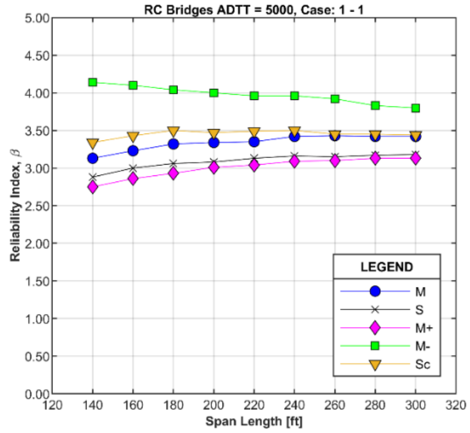

Figure 96. Reliability index for Base WIM and reinforced concrete bridges for MBE (23 CFR 650.317) live load factor of 1.35.

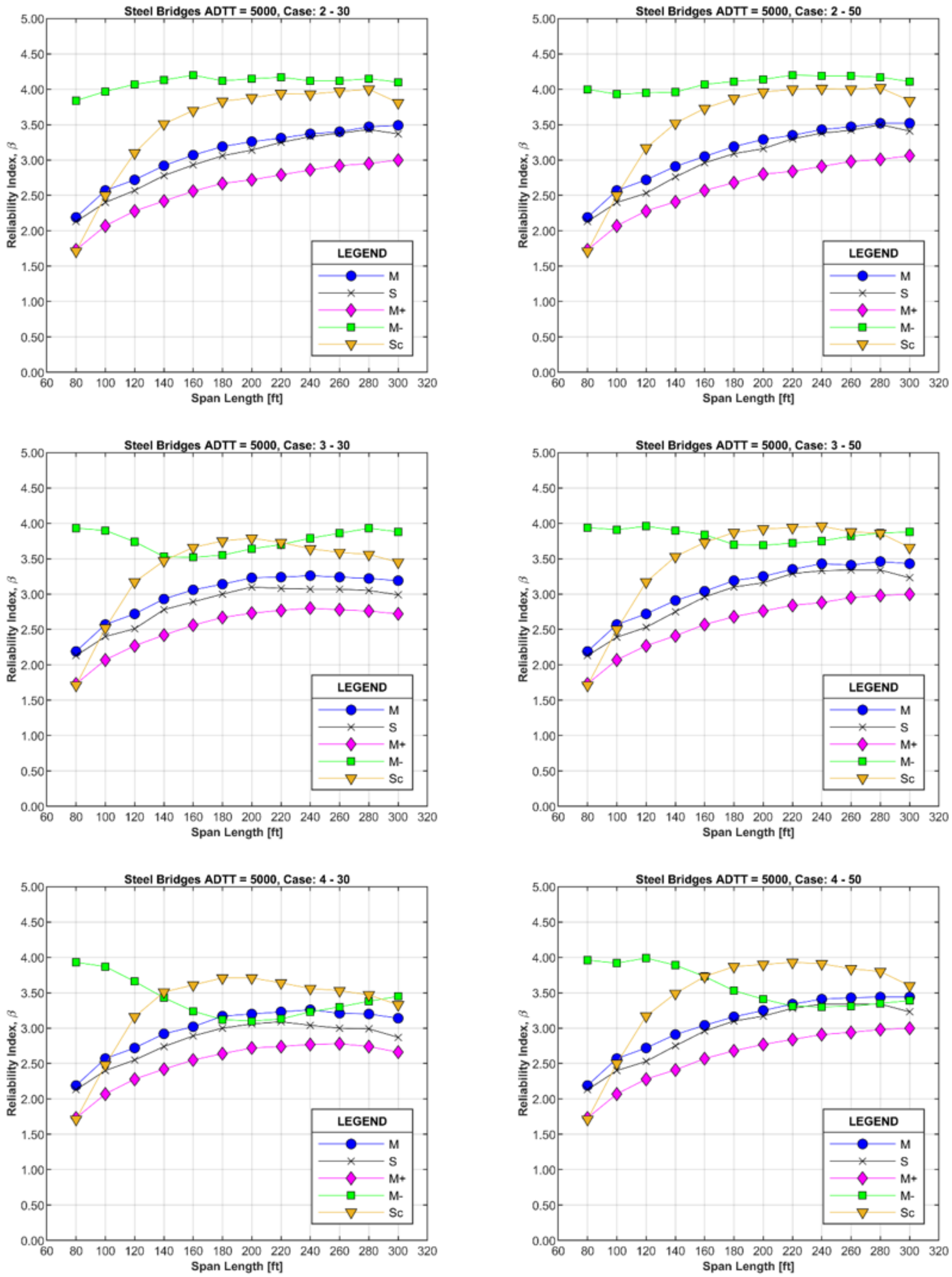

Figure 97. Reliability index for platoons and steel bridges for MBE (23 CFR 650.317) live load factor of 1.35.

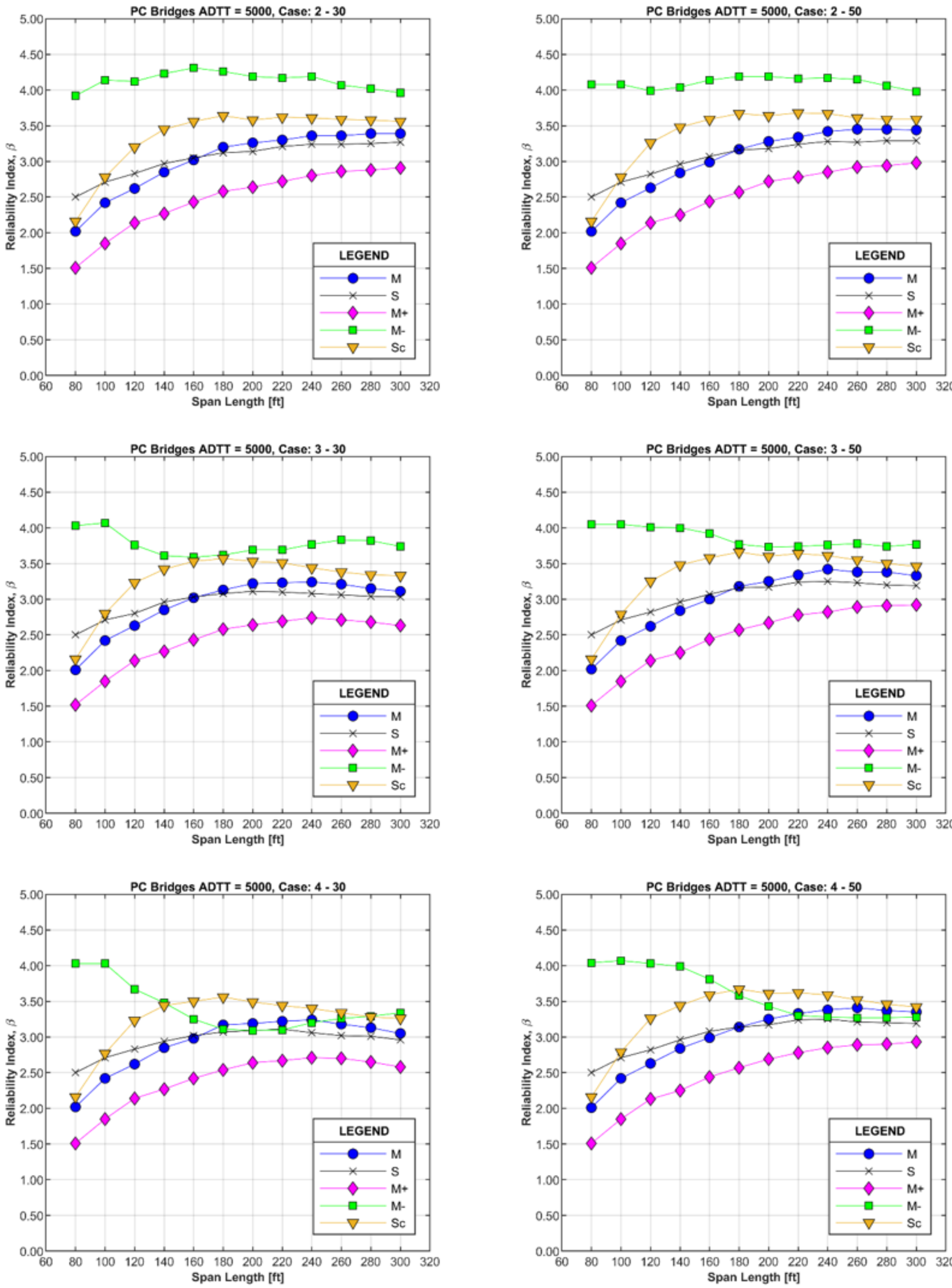

Figure 98. Reliability index for platoons and prestressed concrete bridges for MBE (23 CFR 650.317) live load factor of 1.35.

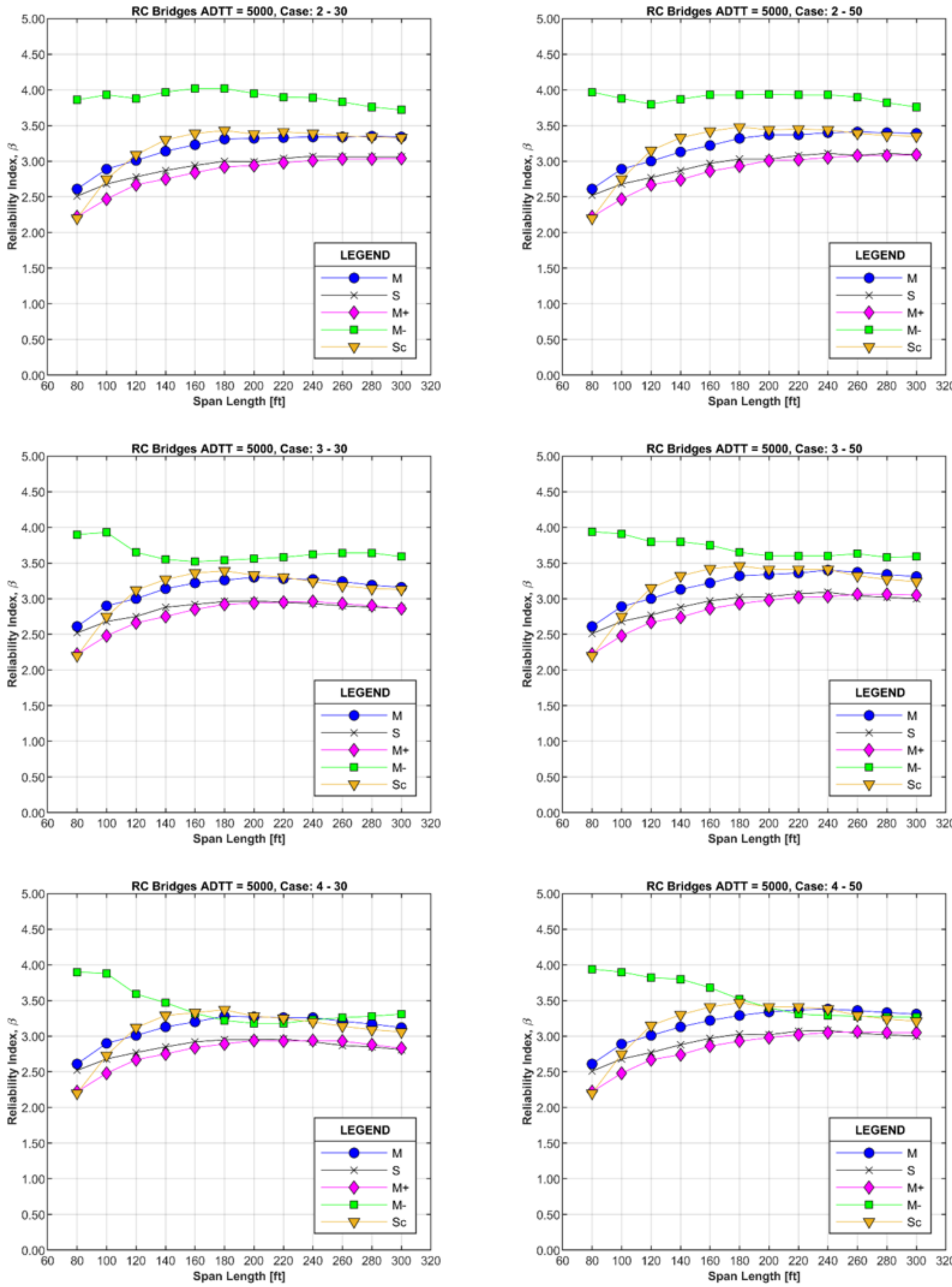

Figure 99. Reliability index for platoons and reinforced concrete bridges for MBE (23 CFR 650.317) live load factor of 1.35.

4.7 Effect of Platoons

The calibration results demonstrate that HL-93 live load model at the operating rating level encompasses all evaluated platoon loadings. It confirms that no adjustments are necessary to the MBE (23 CFR 650.317) provisions for the operating level for the platoon configurations considered in the current study. To better understand the impact of various platoon configurations on bridges, further analysis was performed to determine their effect using the delta factor concept, which is outlined in the next section.

Delta Factors

To evaluate the effect of truck platoons, a delta factor (Δ) is used. This factor represents an increase in the calibrated live load factor for platoon cases compared to the Base WIM scenario. Delta factor is expressed by the following equation:

$$\Delta = \frac{\text{Platoon Live Load Factor} - \text{Base WIM Live Load Factor}}{\text{Base WIM Live Load Factor}} \qquad \text{Eq. 20}$$

It should be noted that the delta factor is a measure of the increase in load factor to achieve a target reliability index, compared to the Base WIM. It does not relate to the actual live load factor required by the specifications. The Base WIM scenario considered only individual trucks, excluding any impact from truck platoons.

The delta factor analysis was performed for all platoon cases and span lengths across steel, prestressed concrete, and reinforced concrete bridges. Table 48 presents the maximum delta factor observed for each material type, load effect (M, S, M+, M-, and Sc), and span lengths ranging from 80 to 300 feet. A delta factor exceeding 5% indicates an impact of the platoons over single truck traffic. The delta factor below 5% is not reported as it falls within the limits of precision of the calibration process. Thus, the iteration step and proximity to the target reliability themselves may result in a small difference in the calibrated reliability index. For example, if the calibrated live load factor for Base WIM is 1.10, and for platoon case is 1.15 with resulting reliability indices of 2.48, and 2.52 respectively, the delta factor for this case would be approximately 5%. Therefore, the impact of platoons is considered only when the delta factor exceeds 5%, to account for considered analytical approximations. Delta factors exceeding 5% are indicative of an increased impact of platoon load effects compared to the Base WIM.

Among these cases, simple span shear and continuous span positive moment exhibit the highest delta factors, reaching a maximum of 20%. The span lengths affected by platoons begin at 180 feet. Notably, only platoon configurations involving 3 or 4 trucks spaced 30 feet apart demonstrate an effect, while those with 2 trucks or platoons spaced 50 feet apart do not lead to an increase in the live load factor based on conducted reliability analysis.

In the case of a negative moment, delta factor increase occurs for spans ranging from 160 to 220 feet, specifically with 4-truck platoons spaced 30 feet apart. This is attributed to the use of a different HL-93 live load model to assess negative moments and behavior of the developed bias factor.

Despite the increased delta factor for some cases the MBE (23 CFR 650.317) provisions for HL-93 loading encompass the effects of truck platoons. In conclusion, the delta factor analysis reveals that 2-truck platoons and platoons spaced 50 feet apart do not significantly increase the required live load factor compared to single trucks. Conversely, 3- and 4-truck platoons spaced 30 feet apart demonstrate a 10-20% increase in live load factor for spans of 180 feet and beyond.

Table 48. Delta factor for all calibrated platoon scenarios.

Load Case	Span [ft]	2-30	2-50	3-30	3-50	4-30	4-50
Simple Span Moment - M	80	0%	0%	0%	0%	0%	0%
	100	0%	0%	0%	0%	0%	0%
	120	0%	0%	0%	0%	0%	0%
	140	0%	0%	0%	0%	0%	0%
	160	0%	0%	0%	0%	5%	0%
	180	0%	0%	5%	0%	5%	5%
	200	0%	0%	0%	0%	5%	0%
	220	0%	0%	5%	0%	5%	0%
	240	5%	0%	10%	0%	10%	0%
	260	0%	0%	10%	0%	10%	0%
	280	0%	0%	10%	0%	10%	0%
	300	0%	0%	10%	0%	10%	0%
Simple Span Shear - S	80	0%	0%	0%	0%	0%	0%
	100	4%	4%	4%	4%	4%	4%
	120	4%	4%	4%	4%	4%	4%
	140	4%	0%	4%	0%	4%	0%
	160	5%	0%	5%	0%	5%	0%
	180	5%	5%	10%	5%	10%	5%
	200	5%	5%	10%	5%	10%	5%
	220	5%	0%	10%	5%	10%	5%
	240	5%	5%	15%	5%	15%	5%
	260	5%	0%	15%	5%	15%	5%
	280	0%	0%	15%	5%	15%	5%
	300	0%	0%	15%	5%	20%	5%

Load Case	Span [ft]	2-30	2-50	3-30	3-50	4-30	4-50
2-Span Pos. Moment - M+	80	0%	0%	0%	0%	0%	0%
	100	0%	0%	0%	0%	0%	0%
	120	0%	0%	0%	0%	0%	0%
	140	0%	0%	0%	0%	0%	0%
	160	4%	0%	4%	0%	4%	0%
	180	0%	0%	0%	0%	0%	0%
	200	4%	0%	5%	4%	5%	4%
	220	5%	0%	5%	0%	5%	0%
	240	5%	5%	10%	5%	10%	5%
	260	5%	0%	10%	0%	10%	0%
	280	5%	5%	14%	5%	14%	5%
	300	5%	5%	18%	5%	18%	5%
2-Span Neg. Moment - M-	80	5%	0%	5%	5%	5%	5%
	100	0%	0%	0%	0%	0%	0%
	120	0%	0%	0%	0%	0%	0%
	140	0%	0%	5%	0%	5%	0%
	160	0%	0%	0%	0%	10%	0%
	180	0%	0%	0%	0%	15%	0%
	200	0%	0%	0%	0%	15%	0%
	220	0%	0%	0%	0%	10%	5%
	240	0%	0%	0%	0%	5%	5%
	260	0%	0%	0%	0%	5%	0%
	280	0%	0%	0%	0%	0%	0%
	300	0%	0%	0%	0%	0%	0%
2-Span Span Shear - Sc	80	3%	3%	3%	3%	3%	3%
	100	0%	0%	0%	0%	0%	0%
	120	5%	0%	5%	0%	5%	0%
	140	0%	0%	5%	0%	0%	0%
	160	0%	0%	0%	0%	0%	0%
	180	0%	0%	0%	0%	0%	0%
	200	0%	0%	0%	0%	0%	0%
	220	0%	0%	0%	0%	0%	0%
	240	0%	0%	0%	0%	0%	0%
	260	0%	0%	0%	0%	0%	0%
	280	0%	0%	0%	0%	0%	0%
	300	0%	0%	0%	0%	0%	0%

4.8 Summary of Calibration Efforts

The calibration of Strength Limit State I for the MBE (23 CFR 650.317) at the operating level for platoon loading using HL-93 as a notional live load was conducted to determine live load factors that provide a minimum target safety in terms of reliability indices.

The synthetized WIM with platoons along with the other individual WIM trucks were run on an influence line analysis to determine the live load envelope for simple and continuous span bridges with lengths from 80 to 300 ft in 20 ft increments. The WIM load effects were compared with the HL-93 to determine platoon live load statistical parameters. The platoon live load model was developed for various ADTT using 250, 1000, 2500, 5000, 10000, and 15000. The WIM live load model provided a basis for a full probabilistic calibration of the various platoon configurations.

The summary of calibration efforts for synthetic WIM with platoons are as follows:

- ADTT has an impact on developed statistical parameters, however, for consistency with previous AASHTO derivations, an ADTT of 5,000 was adopted for calibration purposes.
- The calibration was conducted for both Base WIM, and 100% platoon penetration with six platoon scenarios with 2, 3, and 4-truck platoon configurations, headway spacing of 30 and 50 feet, five load cases, span lengths from 140 to 300 feet, and separate for steel, reinforced concrete, and prestressed concrete bridges.
- The reliability-based calibration employed the developed statistical models to account for the load and resistance uncertainties. Monte Carlo was used as a simulation technique.
- The dead load model developed for this study accounts for the variability associated with the component type (prefabricated, cast-in-place, wearing surface), material type (steel, concrete, and wearing surface), as well as the span length. Based on the bridge inventory database with available dead load to live load ratios, a refined dead load model was developed and utilized in the calibration process.
- The calibration procedure was carried out using two approaches: 1) Calculation of live load factor for target reliability index of 2.50 consistent with MBE (23 CFR 650.317) operating level, and 2) Calculation of reliability index for the platooning load considered with live load factor of 1.35.
- The live load factor calibration showed that the majority of the calibrated live load factors fall below the MBE (23 CFR 650.317) live load factor of 1.35, indicating that HL-93 already encompasses the effects of truck platoons considered in this study.
- The largest values of live load factors were observed for span lengths of 140 feet and 160 feet. These live load factors are practically the same as Base WIM, meaning platoons have little or no effect on these spans. For longer bridges, where platoon load effects dominate over single trucks, live load factors were found to be lower than 1.35.

- The calibrated live load factors for platoon cases with 2 trucks, and all platoon cases with 50 feet headway spacing decrease with the increase in span length. This indicates there is no load amplification in comparison to the HL-93 load model.

- Platoons with a headway spacing of 30 feet yield larger live load factors than those with 50 feet headway. Thus, the 30 feet headway is more critical.

- The difference between the calibrated live load factors for 3 and 4 platoon scenarios is relatively small, so there is a minimal increase in maximum expected load effects for the span range considered.

- Evaluation of the reliability index for the platooning loading considered with the live load factor of 1.35 shows that no platoon case has a reliability index less than the minimum of 1.50 and that all platoon cases for long span bridges yield a reliability index larger than the target of 2.50.

- The platoon loading considered in this study yields more uniform reliability for long span bridges, which means that HL-93 effectively encompasses platoons.

- The calibration results showed that no adjustments are necessary to the MBE (23 CFR 650.317) at the operating level for HL-93 to account for the platoon loading for Strength Limit State I.

- To better understand the impact of various platoon configurations on bridges, further analysis was performed to determine their impact using the delta factor. This factor measures an increase in the calibrated platoon live load factor compared to the Base WIM scenario. The analysis showed that 2-truck platoons and platoons spaced 50 feet apart do not significantly increase the required live load factor to achieve the minimum reliability index compared to single truck traffic. On the contrary, 3- and 4-truck platoons spaced 30 feet apart demonstrate a 10-20% increase in live load factor for spans over 180 feet, however, these cases are still enveloped by the current load factor on HL-93 loading.

4.9 Truck Platooning Effects on Braking Loads, Centrifugal Loads, and Wind on Live Load

Although the primary focus of the current research project is on gravity vehicular live loads (LL), it is anticipated that other load effects related to vehicular use of bridge could be affected. These are braking loads, centrifugal loads and wind on live load.

Current provisions in the LRFD Spec. (23 CFR 625.4(d)(1)(v)) for braking loads (BR) are based on the application of a longitudinal force offset from the roadway surface by 6 feet. The magnitude of the longitudinal force is typically 25% of the HL-93 axle weight (for longer span bridges the force due to 5% of the HL-93 truck and lane load may control), and is based on energy principles, and assuming uniform deceleration.

A conservative interpretation of current specifications suggests that 25% of the total gross vehicle weight of the trucks conforming platoon would typically need to be applied longitudinally to capture the effect of braking by a truck platoon. However, platoon braking is a much more nuanced subject due to:

- Performance of the different communication systems among the trucks in a platoon and possible lag in brake system actuation;
- Truck platoon braking is limited by the truck that brakes least efficiently;
- Headway distance effects on drag and stopping distance;
- Velocity of truck platoon relative to free-flowing traffic.

Limited information is available regarding the issues described above; therefore, it is difficult to establish recommendations regarding braking forces until further information becomes available. Until further refinements can be performed, braking force for truck platoons should be computed based on a conservative application of the LRFD Spec. (23 CFR 625.4(d)(1)(v)) provisions.

Centrifugal forces (CE) are applied similarly to braking forces (BR) in current the LRFD Spec. (23 CFR 625.4(d)(1)(v)), except that the force is applied transversely and instead of a fixed percentage of the axle load, the percentage depends on the highway design speed and curve radius. As with braking forces, the effects of truck platooning of centrifugal forces are difficult to assess until more information regarding their operation patterns and connectivity systems is available. Therefore, until this information becomes available, centrifugal forces for truck platoons should be computed based on a conservative application of the language in the LRFD Spec.

LRFD Spec. (23 CFR 625.4(d)(1)(v)) provisions for wind on live load (WL) are based on a uniform transverse load of 0.1 klf applied 6 ft above the roadway surface. The value is based on a long row of randomly sequenced passenger cars, commercial vans, and trucks exposed to the maximum speed that vehicle can safely travel. The assumptions and methods used to develop these provisions are considered applicable to truck platoon; and therefore, current provisions for wind on live load do not need modifications related to truck platooning.

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Chapter 5: Summary of Research Findings

This study has three distinct parts: 1) Tier 1 - Deterministic analysis of a representative sampling of 2000+ bridges with detailed load rating calculation and estimation of impacts to the national inventory, 2) Tier 2 - Probabilistic analysis of truck load effects on bridges and associated impacts using actual WIM data synthesized to represent future state with possible platoon operations and penetration, and 3) Calibration of load factors for load rating that will provide for uniform reliability across bridge types when subjected to platoon loading.

According to the Tier 1 analysis truck platoons may cause increased maximum load effects in terms of shears and moments in common bridge girders, which may reduce load rating factors below 1.0 and impact service limit states for certain bridges. To estimate the potential impacts to the national inventory of bridges, a number of cases of bridges categorized by type, span length, age, and design which are more likely to have serviceability negatively affected by platoons have been developed. The cases include the following:

- Case 1: Steel Bridges Designed before Load Factor Design Provisions, built prior to 1974 and with span lengths greater than 80 ft.
- Case 2: Steel Bridges Designed after Load Factor Design Provisions, built after 1974 and with span lengths greater than 80 ft.
- Case 3: Prestressed and Post-tensioned Concrete Bridges Designed per Allowable Stress Design Provisions, built prior to 1974 and with span lengths greater than 80 ft.
- Case 4: Prestressed and Post-tensioned Concrete Bridges Designed per Load Factor Design Provisions, built after 1974 and with span lengths greater than 80 ft.
- Case 5: Prestressed and Post-tensioned Concrete Bridges Designed per Load and Resistance Factor Design Provisions, built after 1998 and with span lengths greater than 120 ft.
- Case 6: Reinforced Concrete Bridges Designed before Modern Crack Control Provisions, built before 1992 and with span lengths greater than 80 ft.
- Case 7: Steel Bridges with Welded Primary Steel Tension or Flexural Members, built after 1957 and with span lengths greater than 200 ft.

Bridges that fall within the cases determined from our study have increased potential for impacts to their serviceability. This may result in increased deterioration and maintenance consistent with the limit state exceedance observed. Fatigue in medium to longer span steel bridges is the limit state most likely to be affected. The numbers of these bridges found in each state are provided in the appendix to this report.

According to the Tier 2 analysis truck platoons may cause higher load effects in bridges as compared to existing baseline WIM loading for longer span lengths. However, it is observed that the HL-93 load model with current AASHTO load factors appears to adequately envelop these increased effects due to the uniform load portion of the notional load model. The calibration effort suggests that no adjustments are necessary to the MBE (23 CFR 650.317) to account for the platoon loading for Strength Limit State I. This means that bridges that provide sufficient capacity at the operating level for HL-93 are expected to have adequate strength for the truck platoon configurations considered.

If greater confidence in expected performance is desired for these bridges, it is suggested to verify the design level operating rating from LRFR method is greater than 1.0, or perform load rating to determine if they pass the strength and service limit state(s) of concern.

As CAV technology advances in the future and major shipping companies make investments for platooning operations, it will become clearer about the types, sizes, weights and variability of trucks that most frequently run in platoon on our highway network. At that time, the simulation assumptions and procedures developed in this work can be updated to provide a more accurate characterization of loading, reliability calibration, and estimation of potential impacts to bridges.

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Appendix A: Bridges projected to have service limit states negatively affected by truck platooning

Table 49. Case 1: Steel bridges designed before Load Factor Design provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	109	214	152	215
Alaska	23	30	19	30
Arizona	48	60	58	62
Arkansas	59	102	108	119
California	132	318	188	319
Colorado	39	81	77	92
Connecticut	231	445	345	447
Delaware	24	50	27	51
District of Columbia	24	49	2	49
Florida	25	96	34	96
Georgia	94	226	126	231
Guam	0	0	0	0
Hawaii	0	4	1	4
Idaho	7	24	14	27
Illinois	382	681	392	681
Indiana	191	275	339	340
Iowa	69	191	217	238
Kansas	90	139	163	171
Kentucky	105	178	185	190
Louisiana	116	184	196	215
Maine	4	95	66	95
Maryland	170	304	198	305
Massachusetts	297	493	305	514
Michigan	276	439	388	467
Minnesota	74	98	95	120
Mississippi	34	64	105	115

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Missouri	162	317	271	325
Montana	49	70	95	97
Nebraska	40	84	113	118
Nevada	25	25	25	25
New Hampshire	63	96	67	96
New Jersey	328	627	219	629
New Mexico	32	38	35	39
New York	405	940	441	940
North Carolina	72	154	150	173
North Dakota	19	33	34	39
Ohio	378	755	634	817
Oklahoma	114	206	115	212
Oregon	97	171	147	195
Pennsylvania	240	652	312	654
Puerto Rico	8	17	11	17
Rhode Island	22	80	41	80
South Carolina	53	98	74	98
South Dakota	31	47	103	106
Tennessee	116	196	137	197
Texas	300	505	322	545
Utah	27	51	34	53
Vermont	119	166	119	166
Virgin Islands	0	0	0	0
Virginia	192	347	237	350
Washington	49	172	178	205
West Virginia	130	182	155	182
Wisconsin	100	194	133	199
Wyoming	62	94	93	97
Sum	5856	11157	8095	11847

Table 50. Case 2: Steel bridges designed after Load Factor Design provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	216	336	283	345
Alaska	3	26	3	26
Arizona	18	21	20	23
Arkansas	181	349	356	384
California	8	20	11	20
Colorado	72	130	97	131
Connecticut	119	255	191	255
Delaware	14	72	21	72
District of Columbia	1	2	0	2
Florida	133	356	154	357
Georgia	108	269	140	272
Guam	0	0	0	0
Hawaii	0	0	0	0
Idaho	12	24	16	27
Illinois	289	753	341	754
Indiana	97	199	262	266
Iowa	66	205	159	207
Kansas	91	244	257	272
Kentucky	112	181	176	201
Louisiana	122	190	197	227
Maine	1	56	24	56
Maryland	299	547	326	550
Massachusetts	114	222	112	237
Michigan	103	242	197	256
Minnesota	82	88	117	153
Mississippi	23	93	84	102
Missouri	131	527	328	534
Montana	30	44	50	51

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Nebraska	36	144	168	184
Nevada	37	55	48	56
New Hampshire	59	164	86	166
New Jersey	148	341	136	343
New Mexico	18	28	22	33
New York	453	945	431	946
North Carolina	384	810	693	869
North Dakota	11	34	27	40
Ohio	245	592	441	646
Oklahoma	45	80	46	85
Oregon	6	9	8	11
Pennsylvania	190	572	294	575
Puerto Rico	5	10	6	10
Rhode Island	15	49	18	49
South Carolina	140	246	176	246
South Dakota	33	55	72	79
Tennessee	127	306	148	307
Texas	224	532	320	611
Utah	115	180	143	181
Vermont	37	80	38	80
Virgin Islands	0	0	0	0
Virginia	381	917	439	926
Washington	14	56	77	83
West Virginia	141	305	190	305
Wisconsin	131	281	129	287
Wyoming	39	87	99	103
Sum	5479	12329	8177	13001

Table 51. Case 3: Prestressed and post-tensioned concrete bridges designed per Allowable Stress Design provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	7	7	5	7
Alaska	0	3	0	3
Arizona	7	8	9	10
Arkansas	0	0	0	0
California	330	850	557	850
Colorado	22	45	39	45
Connecticut	17	71	51	72
Delaware	0	0	0	0
District of Columbia	1	1	0	1
Florida	138	294	186	297
Georgia	5	7	5	7
Guam	0	0	0	0
Hawaii	15	20	0	20
Idaho	18	22	19	24
Illinois	43	55	43	55
Indiana	7	15	25	25
Iowa	12	56	52	60
Kansas	0	1	1	1
Kentucky	4	11	9	12
Louisiana	34	42	40	45
Maine	0	0	0	0
Maryland	1	5	1	5
Massachusetts	4	7	4	7
Michigan	4	8	4	9
Minnesota	36	56	50	60
Mississippi	37	53	56	58
Missouri	2	3	2	3

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Montana	27	41	53	53
Nebraska	8	12	17	17
Nevada	4	6	4	6
New Hampshire	4	4	4	4
New Jersey	24	46	23	46
New Mexico	49	52	48	54
New York	10	27	10	27
North Carolina	0	3	0	3
North Dakota	0	0	0	0
Ohio	0	6	5	7
Oklahoma	1	49	1	49
Oregon	44	95	76	110
Pennsylvania	110	265	164	268
Puerto Rico	89	100	94	100
Rhode Island	3	4	4	4
South Carolina	1	1	1	1
South Dakota	0	2	3	3
Tennessee	52	66	57	68
Texas	135	443	155	474
Utah	58	62	57	64
Vermont	0	0	0	0
Virgin Islands	0	0	0	0
Virginia	8	14	8	14
Washington	142	292	268	309
West Virginia	0	0	0	0
Wisconsin	15	74	32	74
Wyoming	0	0	0	0
Sum	1528	3304	2242	3431

Table 52. Case 4: Prestressed and post-tensioned concrete bridges designed per Load Factor Design provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	132	340	239	352
Alaska	54	103	44	108
Arizona	201	641	292	649
Arkansas	6	11	9	12
California	672	2183	1051	2188
Colorado	206	573	423	600
Connecticut	29	62	46	62
Delaware	0	16	4	16
District of Columbia	0	0	0	0
Florida	353	871	487	877
Georgia	217	794	292	813
Guam	0	0	0	0
Hawaii	50	90	1	90
Idaho	33	68	46	75
Illinois	131	316	149	316
Indiana	72	170	216	227
Iowa	53	481	443	533
Kansas	20	131	175	177
Kentucky	112	390	373	436
Louisiana	179	305	302	327
Maine	0	10	2	10
Maryland	15	69	19	70
Massachusetts	77	99	77	131
Michigan	79	221	162	237
Minnesota	78	297	243	329
Mississippi	126	592	663	704
Missouri	114	435	298	452

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Montana	61	125	147	160
Nebraska	18	111	149	160
Nevada	128	232	203	234
New Hampshire	4	12	9	12
New Jersey	27	71	29	73
New Mexico	77	147	100	171
New York	44	130	45	130
North Carolina	106	284	267	339
North Dakota	8	39	27	41
Ohio	47	174	115	195
Oklahoma	56	378	61	381
Oregon	72	198	118	214
Pennsylvania	157	477	193	479
Puerto Rico	162	340	268	365
Rhode Island	3	11	3	12
South Carolina	50	159	63	159
South Dakota	18	35	75	80
Tennessee	133	434	185	445
Texas	963	3377	1352	3728
Utah	189	262	223	262
Vermont	2	2	2	2
Virgin Islands	0	0	0	0
Virginia	45	110	49	110
Washington	162	433	412	473
West Virginia	7	59	17	59
Wisconsin	151	693	167	694
Wyoming	0	3	7	8
Sum	5699	17564	10342	18777

Table 53. Case 5: Prestressed and post-tensioned concrete bridges designed per Load and Resistance Factor Design provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	10	27	15	27
Alaska	22	42	15	43
Arizona	15	54	22	61
Arkansas	0	0	0	0
California	27	153	62	153
Colorado	70	109	89	114
Connecticut	0	0	0	0
Delaware	0	10	9	10
District of Columbia	0	0	0	0
Florida	142	366	200	382
Georgia	11	33	19	33
Guam	0	0	0	0
Hawaii	2	4	0	4
Idaho	9	29	20	32
Illinois	0	0	0	0
Indiana	63	114	120	129
Iowa	34	76	67	85
Kansas	4	11	12	12
Kentucky	19	36	29	40
Louisiana	8	10	10	10
Maine	0	5	1	5
Maryland	0	0	0	0
Massachusetts	0	1	0	1
Michigan	27	33	43	45
Minnesota	38	82	71	98
Mississippi	15	22	31	35
Missouri	16	51	32	54

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Montana	2	15	15	18
Nebraska	14	64	78	84
Nevada	17	43	53	54
New Hampshire	0	0	0	0
New Jersey	9	22	4	22
New Mexico	11	14	12	15
New York	3	13	3	13
North Carolina	48	84	67	96
North Dakota	2	4	3	4
Ohio	29	36	33	38
Oklahoma	25	49	20	53
Oregon	44	104	69	106
Pennsylvania	59	156	73	157
Puerto Rico	2	3	3	5
Rhode Island	0	0	0	0
South Carolina	10	35	19	36
South Dakota	0	1	7	7
Tennessee	49	83	22	84
Texas	350	1010	487	1142
Utah	37	67	43	69
Vermont	4	4	4	4
Virgin Islands	0	0	0	0
Virginia	6	20	11	20
Washington	51	153	149	173
West Virginia	9	29	18	29
Wisconsin	67	112	16	112
Wyoming	0	0	0	0
Sum	1380	3389	2076	3714

Table 54. Case 6: Reinforced concrete bridges designed before modern crack control provisions.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	27	56	47	61
Alaska	0	0	0	0
Arizona	16	22	19	23
Arkansas	0	5	8	10
California	745	1650	1112	1652
Colorado	18	29	21	29
Connecticut	0	1	0	1
Delaware	0	1	1	1
District of Columbia	2	12	1	12
Florida	0	3	1	3
Georgia	1	4	2	4
Guam	0	0	0	0
Hawaii	27	65	5	70
Idaho	12	20	14	21
Illinois	4	9	4	9
Indiana	4	13	34	35
Iowa	0	7	4	7
Kansas	27	62	76	77
Kentucky	54	104	109	111
Louisiana	19	39	19	39
Maine	0	1	1	1
Maryland	6	16	4	16
Massachusetts	1	6	0	6
Michigan	2	8	11	13
Minnesota	6	9	12	15
Mississippi	72	102	102	110
Missouri	20	55	38	55

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Montana	4	4	5	5
Nebraska	0	2	3	4
Nevada	22	25	22	25
New Hampshire	0	1	0	1
New Jersey	1	17	1	17
New Mexico	6	10	7	10
New York	10	53	13	53
North Carolina	0	0	1	1
North Dakota	0	0	0	0
Ohio	1	10	9	13
Oklahoma	9	11	9	14
Oregon	38	110	72	123
Pennsylvania	6	40	9	40
Puerto Rico	5	9	7	9
Rhode Island	0	7	0	7
South Carolina	3	5	3	5
South Dakota	2	1	4	5
Tennessee	72	151	84	152
Texas	6	26	9	28
Utah	3	7	4	7
Vermont	0	0	0	0
Virgin Islands	0	0	0	0
Virginia	0	14	0	14
Washington	140	284	254	305
West Virginia	4	4	4	4
Wisconsin	1	1	0	1
Wyoming	6	9	8	9
Sum	1402	3100	2173	3233

Table 55. Case 7: Steel bridges with welded primary steel tension or flexural members.

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Alabama	31	73	45	74
Alaska	12	17	12	17
Arizona	12	16	15	17
Arkansas	28	54	56	59
California	26	40	28	40
Colorado	18	28	21	28
Connecticut	32	53	39	53
Delaware	3	9	6	9
District of Columbia	9	11	4	12
Florida	95	242	109	242
Georgia	7	30	12	30
Guam	0	0	0	0
Hawaii	0	1	1	1
Idaho	9	23	17	25
Illinois	51	120	55	120
Indiana	12	20	23	25
Iowa	41	67	44	67
Kansas	16	43	41	45
Kentucky	44	84	83	95
Louisiana	38	84	88	102
Maine	0	8	4	8
Maryland	49	74	54	75
Massachusetts	18	27	19	28
Michigan	21	32	29	35
Minnesota	23	41	30	48
Mississippi	14	54	59	64
Missouri	28	70	51	71
Montana	10	12	17	17

State, District or Territory	Interstate (IS)	National Highway System (NHS)	National Network (NN)	Either IS, NHS, NN
Nebraska	8	23	22	24
Nevada	7	7	7	7
New Hampshire	9	20	11	20
New Jersey	22	43	18	43
New Mexico	5	6	6	7
New York	85	168	100	168
North Carolina	33	50	50	59
North Dakota	2	9	4	9
Ohio	40	85	60	90
Oklahoma	12	28	12	28
Oregon	27	45	39	49
Pennsylvania	76	183	90	184
Puerto Rico	1	2	1	2
Rhode Island	8	15	9	15
South Carolina	11	34	21	34
South Dakota	10	19	18	19
Tennessee	54	126	65	128
Texas	141	377	223	419
Utah	51	85	69	85
Vermont	15	22	16	22
Virgin Islands	0	0	0	0
Virginia	49	82	43	83
Washington	31	82	93	95
West Virginia	34	109	50	109
Wisconsin	45	60	23	60
Wyoming	4	13	12	14
Sum	1427	3026	2024	3180