Study of the Impacts of Implements of Husbandry on Bridges

Volume I: Live Load Distribution Factors and Dynamic Load Allowances

August 2017

BRIDGE ENGINEERING CENTER

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IOWA STATE UNIVERSITY
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Brent Phares (orcid.org/0000-0001-5894-4774), Chandra Teja Kilaru (orcid.org/0000-0002-1399-7132), Lowell Greimann (orcid.org/0000-0003-2488-6865), Junwon Seo (orcid.org/0000-0001-6046-9319), and Katelyn Freeseman (orcid.org/0000-0003-0546-3760)	InTrans Projects 9-364 and 11-399
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16. Abstract

The objectives of this study were to develop guidance for engineers on how implements of husbandry loads are resisted by traditional bridges, with a specific focus on bridges commonly found on the secondary road system; provide recommendations for accurately analyzing bridges for these loading effects; and make suggestions for the rating and posting of these bridges.

To achieve the objectives, the distribution of live load and dynamic impact effects for different types of farm vehicles on three general bridge types—steel-concrete, steel-timber, and timber-timber—were investigated through load testing and analytical modeling. The types of vehicles studied included, but were not limited to, grain wagons/grain carts, manure tank wagons, agriculture fertilizer applicators, and tractors.

Once the effects of these vehicles had been determined, a parametric study was carried out to develop live load distribution factor (LLDF) equations that account for the effect of husbandry vehicle loads. Similarly, recommendations for dynamic effects were also developed. The live load distribution factors and dynamic load allowances are the primary subjects of this first of three report volumes.

Finally, suggestions on the analysis, rating, and posting of bridges for husbandry implements were developed. Those suggestions are covered in the second volume of the report.

The third volume of the report contains six appendices that include the 19 mini-reports for field tested and analytically modeled steel-concrete, steel-timber, and timber-timber bridges, the farm implement and bridge inventories for the project, and survey responses.

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STUDY OF THE IMPACTS OF IMPLEMENTS OF HUSBANDRY ON BRIDGES

Volume I: Live Load Distribution Factors and Dynamic Load Allowances
August 2017

Principal Investigator
Brent Phares, Director
Bridge Engineering Center, Iowa State University

Co-Principal Investigator
Terry Wipf, Professor and Chair
Civil, Construction, and Environmental Engineering, Iowa State University

Authors
Brent Phares, Chandra Kilaru, Lowell Greimann, Junwon Seo, and Katelyn Freeseman

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A report from
Bridge Engineering Center and
Institute for Transportation
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-8103 / Fax: 515-294-0467
www.intrans.iastate.edu

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THREE-VOLUME EXECUTIVE SUMMARY

The deterioration of bridges is a prevalent issue in the US. A portion of that deterioration comes from the frequent subjection of bridges to oversized loads. Of those oversized loads, implements of husbandry are of particular interest. Although states differ in their definition, an implement of husbandry can generally be thought of as a vehicle used to carry out agricultural activities. These vehicles often carry heavy loads, and little is known on how husbandry implements affect today's bridges.

The behavior of bridges with these vehicles, particularly regarding live load distribution and impact, is not explicitly enveloped within the design, rating, and posting vehicles presented in current American Association of State Highway and Transportation Officials (AASHTO) specifications. Because of the large axle loads and varying axle spacings, the current AASHTO vehicles, such as the HL-93 design truck and the HS20 rating truck, may not accurately represent husbandry implements.

The objectives of this research, presented in a three-volume report series, were to develop guidance for engineers on how implements of husbandry loads are resisted by traditional bridges, with a specific focus on bridges commonly found on the secondary road system; provide recommendations for accurately analyzing bridges for these loading effects; and make suggestions for the rating and posting of these bridges

Volume I focuses on the impacts of husbandry implements on actual bridges by way of field testing as well as analytical finite element models. With these data, the objective was to develop equations and limits for dynamic load allowances and live load distribution factors that apply directly to husbandry vehicles.

Included in the testing were bridges with steel girders with both concrete and timber decks as well as bridges with timber girders and timber decks. Field testing was conducted on 19 of the bridges in this collection. Brief reports for each of the 19 bridges are in Volume III: Appendices.

The data collected from field tests were used to determine a reasonable bound for impact factors for husbandry implements as well as to get a base understanding of how live load moments created by husbandry vehicles are distributed among girders. In addition to the field tests, finite element models were created for the 19 bridges and calibrated with the field test results. Using these models as guidelines, finite element modes were created for 151 bridges included in the inventory (also included in Volume III: Appendices). The finite element models were subjected to the loads of 121 typical husbandry vehicles inventoried (also included in Volume III: Appendices) and modeled using finite element analysis.

Results show that the impact factors currently presented in the AASHTO specifications are too low for husbandry vehicles. Similarly, provisions provided by AASHTO for live load distribution are, in some cases, drastically different from live load distribution factors determined from loading the 151 bridges with the 121 husbandry vehicles. Volume I provides recommendations on upper limits for dynamic load allowances as well as several equations for determining live load distribution specifically for husbandry implements.

The purpose of the work covered in Volume II was to determine whether current AASHTO rating and posting vehicles can be used to accurately represent husbandry implements. Using software generated by the Bridge Engineering Center at Iowa State University's Institute for Transportation, AASHTO vehicles and the same 121 husbandry vehicles inventoried and used in the Volume I work were theoretically driven across 174 bridges (151 of which were also included in the parametric study in Volume I).

With the moments produced by both the AASHTO and husbandry vehicles on these bridges, comparisons were made between moment envelopes for both vehicle types as well as for theoretical operating ratings for both vehicle types. Results showed that the vehicles provided in AASHTO specifications do not accurately represent the effects caused by husbandry vehicles. In addition, on shorter span bridges, husbandry vehicles tend to produce lower operating ratings than the AASHTO vehicles. On longer span bridges, husbandry vehicles seem to lead to higher operating ratings than AASHTO vehicles.

Volume II presents the development of an overarching husbandry vehicle, recommendations on signage and posting for husbandry vehicles, as well as bridge rating examples, for both short and long span bridges, using updated distribution and impact factors as presented in Volume I.

Finally, Volume III is a collection of appendices referenced in Volumes I and II. Appendices A, B, and C are a series of mini reports for the 19 field tested bridges from Volume I. Appendix D includes detailed information of the 121 farm vehicles used for the study. Appendix E is a detailed inventory of the 151 bridges from Volume I and 174 bridges used in Volume II. Appendix F includes the survey sent to the state departments of transportation and responses to questions about their rules and regulations for husbandry implements on bridges.

1 INTRODUCTION

In the US, bridges are typically designed and load rated based on the specifications provided by the American Association of State Highway and Transportation Officials (AASHTO). These specifications were developed to ensure the safety of bridges for traditional highway vehicles. As a part of both the design and rating process, live loads in the form of a typical highway truck are distributed across the various structural elements to determine the shear and moments in those elements. Although the process to determine these shear and moments can be quite intensive, the process has been simplified to a degree through the use of the live load distribution factors (LLDFs) and the dynamic load allowance (IM) specified by the AASHTO standards and LRFD specifications (AASHTO 1996, AASHTO 2010).

LLDFs can be broadly defined as the ratio of the maximum live-load effect in a component to the maximum live-load effect in a system when using beam-line model techniques (Barker and Puckett 2013). LLDFs were developed to examine the bridge’s capability to resist traditional highway-type vehicles (e.g., trucks, which tend to have relatively consistent widths and other characteristics) (AASHTO 1996, AASHTO 2010). AASHTO defines the dynamic load allowance, IM, as an increase in the applied static force effects to account for the dynamic interaction between the bridge and moving loads.

While the AASHTO specifications are generally thought to be conservative when used to predict the response of bridges to highway-type vehicles, concerns have been raised about their applicability to non-highway vehicles such as husbandry implements, which often have large axle loads and varying axle spacings.

1.1 Problem Statement

As of 2013, there were 607,380 bridges in the US (ASCE 2013), with the majority of these bridges found on secondary roadways and generally thought of as “rural” bridges. Statistics show that 13 percent of the rural bridges are structurally deficient and 10 percent are functionally obsolete (Orr 2012). Combining these statistics indicates that there are a large number of bridges in rural settings that do not meet current design standards, although this does not necessarily mean they are unsafe.

At the same time, changing technology in farming has led to heavier farm vehicles in a variety of configurations. While these vehicles are developed for use on a farm, they commonly travel on the roadway system as well. These vehicles tend to have different wheel spacing, gauge widths, wheel footprints, and dynamic coupling characteristics than traditional highway vehicles, which means they are likely resisted differently than the vehicles addressed by AASHTO specifications (Wood and Wipf 1999, Phares et al. 2005, Seo et al. 2013).

Currently, an engineer who wants to assess a bridge’s ability to resist implements of husbandry must make many assumptions and use best judgement. Therefore, there is a need to provide engineers with the tools to accurately assess how highway bridges resist these atypical vehicles.

1.2 Research Objective and Scope

The objectives of this study were to develop guidance for engineers on how implements of husbandry loads are resisted by traditional bridges, with a specific focus on bridges commonly found on the secondary road system; provide recommendations for accurately analyzing bridges for these loading effects; and make suggestions for the rating and posting of these bridges.

1.3 Research Methodology

To achieve the objectives, the distribution of live load and dynamic impact effects for different types of farm vehicles on three general bridge types—steel-concrete, steel-timber, and timber-timber—were investigated by load testing and analytical modeling. The types of vehicles studied included, but were not limited to, grain wagons/grain carts, manure tank wagons, agriculture fertilizer applicators, and tractors.

Once the effects of these vehicles had been determined, a parametric study was carried out to develop live load distribution factor (LLDF) equations that account for the effect of husbandry vehicle loads. Similarly, recommendations for dynamic effects were also developed. Finally, suggestions on the analysis, rating, and posting of bridges for husbandry implements were developed.

1.4 Three-Volume Report Organization

This final report is presented in three volumes and summarizes the results of this project as follows.

Volume I: Live Load Distribution Factors and Dynamic Load Allowances

Volume II: Rating and Posting Recommendations

Volume III: Appendices

The appendices in Volume III are referenced in Volumes I and II. Volume III includes the following for this project:

- Appendix A. Field Tested Steel-Concrete Bridges
- Appendix B. Field Tested Steel-Timber Bridges
- Appendix C. Field Tested Timber-Timber Bridges
- Appendix D. Farm Implement Inventory
- Appendix E. Bridge Inventory
- Appendix F. Survey Responses

1.5 Methodology for Live Load Distribution Factors and Dynamic Load Allowances (Volume I)

The approach to develop the LLDF equations and IM for this study was multi-pronged, as shown by the flowchart in Figure 1 and outlined below.

1. Review LLDF provisions in the AASHTO LRFD Bridge Design Specifications for the selected three bridge types, including (a) steel girder bridges with concrete deck, (b) steel girder bridges with timber deck, and (c) timber girder bridges with timber deck

2. Determine LLDFs from AASHTO specifications for traditional vehicles

3. Select in-service representative bridges covering the selected three bridge types for field tests with actual husbandry vehicles and a conventional highway truck

4. Determine field LLDFs and IM for the field-tested bridges

5. Develop analytical models for the field-tested bridges using commercially available finite element analysis (FEA) software and calibrate the models using the field data

6. Determine analytical LLDFs for the field bridges for different husbandry vehicles

7. Determine the statistical limits of the LLDFs

8. Compare analytical and statistical LLDFs to those obtained from the AASHTO specifications and the field tests

9. Obtain a bridge inventory of bridges common to those states participating in the study

10. Develop analytical FEA models for all the inventory bridges

11. Determine analytical LLDFs for all the bridges, considering the following:

12. A single-axle vehicle with varying gauge width

13. Husbandry vehicle inventory consisting of 121 vehicles

14. Develop parametric equations for each bridge type similar to the AASHTO LRFD specifications

15. Develop a generic vehicle configuration that replicates all the husbandry vehicles

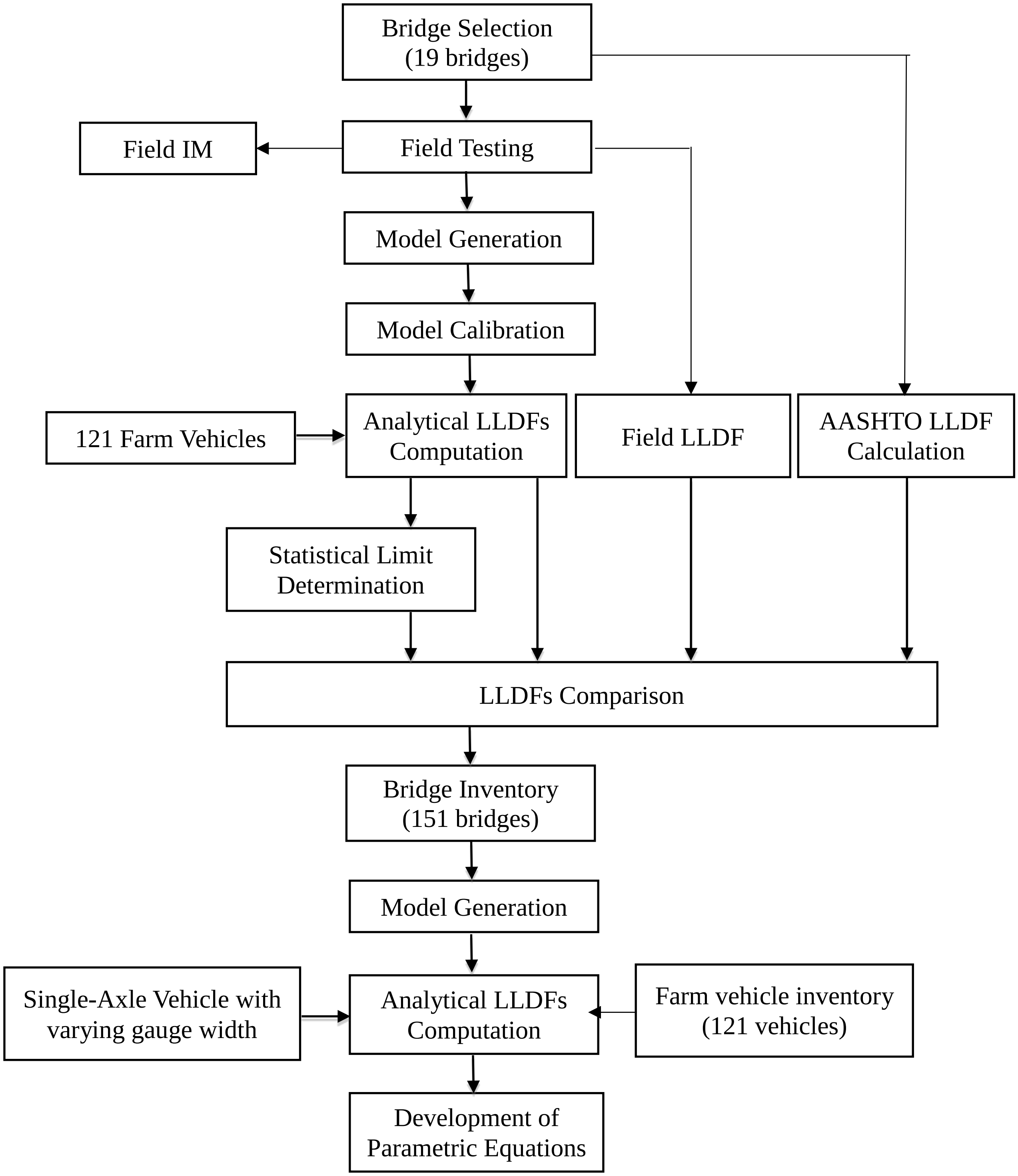


Figure 1. Flowchart of methodology

1.6 Volume I Organization

This study explores LLDFs and IM of three bridge types under the effect of husbandry vehicles and develops recommendations for the analysis, rating, and posting of such bridges. This report volume is divided into five chapters.

Chapter 2 summarizes previous work related to implements of husbandry vehicles and their impact on bridges and pavements. Chapter 2 also includes a brief literature review related to LLDFs and IM. Chapter 3 details the entire procedure used to determine LLDFs for implements of husbandry and is structured into four sections. The first section provides a brief overview of the procedure. The second section includes descriptions of the bridges and testing vehicles utilized during field testing along with a description of the testing methodology. The third section describes the analytical modeling procedure including model generation, model calibration, and analytical vehicle combinations. The method to determine the analytical LLDFs and the statistical analyses is also presented. The final section of Chapter 3 introduces the parametric study of the three bridge types. The effect of gauge width on LLDFs is discussed, and the efforts to incorporate this effect into the live load distribution formulae are outlined in this section. Also, the extrapolation concept of analytical modeling and the general form of the empirical equations are presented.

Chapter 4 summarizes the results in three sections. The first and second sections include LLDFs and IM of 19 field-tested bridges, and these are compared with current AASHTO specifications. In the third section, the results of the parametric study and the newly developed live load distribution formulae specific to each bridge type are presented, followed by a comparison to the AASHTO specified formulas. Also, the effect of skew on LLDFs is addressed.

Finally, the insights from this study are summarized and recommendations for future work are provided in Chapter 5.

2 LITERATURE REVIEW

A brief literature review of pertinent technical publications was completed and is summarized in this section. The focus of the literature review was on previous work describing the effect of implements of husbandry on bridges and pavements and means and methods for the determination of LLDFs and IM.

2.1 Previous Work Related to Implements of Husbandry

The Wisconsin Department of Transportation (WisDOT) formed a task force group to study farm vehicles and their effects on the transportation system in two phases. In the first phase, the group recommended creating or amending statutory definitions and categories of farm vehicles to assist in determining whether a vehicle, piece of equipment or machinery, or trailer is designed for agricultural purposes and used exclusively in the conduct of agricultural operations (WisDOT 2013a). The second phase of the work included an evaluation of weight limits for farm vehicles and recommended implementing new safety standards to accommodate the unique needs of the agriculture industry. It was recognized that farm vehicle weight impacts are very different from the impacts of traditional highway vehicles, particularly on bridges and culverts. Some of those differences include axle spacing, weight distribution, length, and tire design (WisDOT 2013b).

2.1.1 Bridges

The WisDOT Synthesis Report summarizes some quantifiable information relating the impact of farm vehicles on the structural performance on bridges (Phares et al. 2005). In the work completed by Wood and Wipf (1999), the authors describe the procedures and results from testing four timber bridges in the Iowa State University Structural Engineering Laboratory (Wood and Wipf 1999). The testing was completed by constructing four bridges from nominal 4 in. by 12 in. timber stringers removed from an existing bridge. Other bridge components, including nominal 3 in. by 12 in. deck planks, sill plates, and blocking, were fabricated from new timber. Loading was applied through a 30 in. by 20 in. footprint (simulating a tire from a grain cart) on the 16 ft span bridges. The test results indicated good load sharing between the stringers, and the bridge failures were characterized as flexure failure of the bridge stringers. The maximum bending forces induced from a farm vehicle were computed for single-span bridges with span lengths ranging from 20 ft to 140 ft and compared to that of standard design vehicles such as HS20 (representative of a loaded semi-tractor trailer), H20 and NAFTA truck (similar to 98,000 lb timber hauler’s truck). It was observed that the farm vehicles typically had higher bending moments than the design vehicles, indicative of an increased likelihood of damage/failure (Wood and Wipf 1999).

Rholl (2004) recounts one incident where a loaded farm vehicle had punched through the bridge deck (Rholl 2004). A further investigation revealed that the vehicle was legal under Minnesota’s Implements of Husbandry Law. Further, structural collapses of rural bridges have often been observed in association with agricultural loads (Stachura 2007, Nixon 2012).

2.1.2 Pavements

County engineers in Iowa widely believe that farm vehicles play a significant role in the degradation of roads (Oman and Deusen 2001). Past studies also report that farm vehicles have a critical influence on rural roads and pavements (Oman 2001, Fanous et al. 2000). The referenced pavement degradation was specifically linked by numerous researchers to three common attributes of farm vehicles: exceeding the 20,000 lb single-axle weight limit; having wide transverse tire spacing(s), which places heavy loads on pavement edges (this phenomenon can decrease the design life of rigid pavements by up to twenty times); and moving slowly, which increases the load duration and exacerbates rutting (permanent deformations) in flexible pavements (Oman et al. 2001).

A study completed by Iowa State University (Fanous et al. 2000) that included both an analytical investigation and the collection of experimental field data showed that a single-axle, single-tire grain cart or liquid manure tank with an axle load of 24,000 lb has the same effect on 8 in. thick rigid pavement as that caused by a 20,000 lb, single-axle, dual-tire semi-trailer (Fanous et al. 2000). The comparison was based on the amount of bending stresses caused by each vehicle. Tests on flexible pavements indicated that stresses in the spring time are much higher than stresses induced in the fall and summer due to thawing of the subgrade soils in the spring. The final report from that study (Fanous et al. 2000) also indicated that tracked vehicles induced lower stresses in both rigid and flexible pavements (Fanous et al. 2000). This reduced impact was attributed to the larger track-pavement contact area.

A study conducted in South Dakota (Sebaaly 2002) aiming to document the impact of various agricultural equipment types on pavements examined the impact of heavy loadings as compared to the 18,000 lb single-axle truck by instrumenting and monitoring both thick and thin pavement sections (Sebaaly 2002). The study collected data from pressure cells, surface deflection gauges, and strain sensors. The specific vehicles investigated were the TerraGator 8013 (a single-tire on the steering axle and dual-tire on the drive axle), the TerraGator 8144 (two tires on both the steering and drive axles), a grain cart (a single two-tire axle pulled by a tractor), and a tracked tractor (tracks on both the steering and the drive axle). The results for flexible pavements with thin asphalt layers (1.5 in. or less) over a 6 to 12 in. thick coarse aggregate base were quite interesting. Sebaaly (2002) reported that one trip of an empty TerraGator was equivalent to 51 to 150 trips of the 18,000 lb single-axle truck (Sebaaly 2002). This means that if a pavement section is designed for 20 years of service with certain ride quality (serviceability), at the end of the design life, one trip of an empty TerraGator consumes the planned design life 51 to 150 times faster than a standard 18,000 lb single-axle truck. Similarly, one trip of a loaded TerraGator was 230 to 605 times, a legally loaded grain cart was 77 to 240 times, and an overloaded grain cart was 264 to 799 times more damaging than the 18,000 lb single-axle truck.

2.2 Live Load Distribution Factor

LLDFs, commonly known as distribution factors, are important quantities when designing new bridges and evaluating the structural capacity of existing bridges. The codified LLDFs are used to estimate how much live load individual beams/girders must resist. In practice, a designer computes the design moment of each girder by multiplying the LLDF with the resulting maximum girder moment (Mgirder) determined using a beam-line model technique. This approach allows the designer to analyze bridge response by considering longitudinal and transverse effects of wheel loads as an uncoupled phenomenon. LLDFs can generally be defined as the ratio of the maximum live-load effect in a single component to the maximum live-load effect in a system when using beam-line model techniques (Barker and Puckett 2013). Overestimation of LLDFs can lead to bridges that are unnecessarily overdesigned and underestimation may result in a structure that is unable to carry the required load (Eom and Nowak 2006). The AASHTO bridge design specifications provide LLDFs in many different forms (AASHTO 1996, AASHTO 2010). The AASHTO Standard code specifies LLDF equations based upon the simple S-over rule for all types of bridges. The S-over rule, a function of girder spacing S and bridge type, has been in use for designing new bridges since the 1930s (BridgeTech, Inc., et al. 2007). These traditional factors are easy to apply but are often overly conservative and sometimes non-conservative in some parameter ranges (Eom and Nowak 2001, Bridge Tech, Inc., et al. 2007). The concept, assumptions, and drawbacks when using the S-over equations were presented by Bakht and Moses (1987). The AASHTO Standard Specifications for interior girders of single and multiple lanes of select bridge types are given below.

For steel-concrete bridges:

$$LLDF_{\mathrm{single-lane}} = \frac{S}{7.0} \tag{1-a}$$

$$LLDF_{multiple-lane} = \frac{S}{5.5} \tag{1-b}$$

where S = girder spacing (ft)

For steel-timber bridges:

$$LLDF_{single-lane} = \frac{S}{4.5} \tag{2-a}$$

$$LLDF_{multiple-lane} = \frac{S}{4.0} \tag{2-b}$$

where S = girder spacing (ft)

For timber-timber bridges:

$$LLDF_{single-lane} = \frac{S}{4.0} \tag{3-a}$$

$$LLDF_{multiple-lane} = \frac{S}{3.75} \tag{3-b}$$

where S = girder spacing (ft)

For exterior girders, the AASHTO Standard Specification calls for applying the live load bending moment based upon the reaction of the wheel load obtained by assuming the flooring to act as a simple span between stringers or beams.

The distribution factors from AASHTO Standard equations are based on wheel load. Hence, the LLDFs in the above equations were multiplied by a factor of 0.5 in order to compare them directly with LRFD specifications, and analytical LLDFs (Eom and Nowak 2006), which are based on axle loads in the subsequent chapters.

The AASHTO LRFD specification has more sophisticated formulas (for some bridge types), which take into account specific bridge geometric conditions and other factors. The AASHTO LRFD equations were developed through parametric studies involving finite element analysis simulations of highway bridges loaded by conventional highway-type vehicles (Zokaie et al. 1992). These parametric studies are a part of the extensive research work of the National Cooperative Highway Research Program (NCHRP) 12-26 report. In general, the AASHTO LRFD code (AASHTO 1998) specified LLDFs are thought to be more consistent than the AASHTO Standard code (AASHTO 1996), particularly for bridges with long span lengths (Eom and Nowak 2001). However, an iterative design procedure is necessary since the equations require parameters that are not known until girder selection. The LLDF equations from AASHTO LRFD for selected bridge types are given below.

For steel-concrete bridges:

(a) Interior girders

$$LLDF_{single-lane} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{4-a}$$

$$LLDF_{multiple-lane} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{4-b}$$

where S = girder spacing (ft)
L = span length (ft)
K_g = $n(I+Ae^2)$, longitudinal stiffness (in^4)
t_s = deck thickness (in.)
n = modular ratio between steel and concrete

I = girder stiffness (in^4)
A = area of cross-section of girder (in^2)
e = eccentricity between centroids of girder and slab (in.)

(b) Exterior girders

$$LLDF_{multiple-lane} = \left(0.77 + \frac{d_e}{9.1}\right) LLDF_{\text{interior}} \tag{4-c}$$

where d_e = distance from the centerline of the web of the exterior girder to interior edge of the curb (ft)

$LLDF_{interior}$ = distribution factor specified for interior girders in equation (4-a) and (4-b).

Note that for single-lane bridges, exterior LLDFs can be determined based upon the lever rule as specified in the AASHTO LRFD Code (AASHTO 2010).

The AASHTO LRFD specifications provide skew correction factors (SCF) for skewed steel-concrete bridges. These factors are then multiplied with LLDFs of non-skewed bridges to apply for skewed bridges. The equation to determine SCFs for steel-concrete bridges is given below.

$$SCF = 1 - 0.25 \left(\frac{K_g}{12Lt_s^3}\right)^{0.25} \left(\frac{S}{L}\right)^{0.5} (tan\theta)^{1.5} \tag{5}$$

where S = girder spacing (ft)
L = span length (ft)
K_g = longitudinal stiffness (in^4)
t_s = deck thickness (in.)
θ = skew angle (degrees)

The AASHTO LRFD specifications for steel-timber and timber-timber bridges are based on the simple S-over rule for the interior girders.

For steel-timber bridges:

$$LLDF_{\text{single}-lane} = \frac{S}{8.8} \tag{6-a}$$

$$LLDF_{multiple-lane} = \frac{S}{9.0} \tag{6-b}$$

where S = girder spacing (ft)

For timber-timber bridges:

$$LLDF_{single-lane} = \frac{S}{6.7} \tag{7-a}$$

$$LLDF_{multiple-lane} = \frac{S}{7.5} \tag{7-b}$$

where S = girder spacing (ft)

Note that the lever rule recommended by the AASHTO specifications is used to determine the LLDFs of exterior girders for the steel-timber and timber-timber bridges. The lever rule is a method of computing the LLDF by summing moments about the first interior girder, assuming a notional hinge to get the reaction at the exterior girder (BridgeSight 1999). More details on the lever rule can be found in the AASHTO LRFD specifications (AASHTO 2010).

The AASHTO code-specified LLDFs have been compared with those obtained from field tests for different types of bridges loaded with highway trucks. Kim and Nowak (1997) determined field-measured LLDFs for highway steel I-girder bridges located in Michigan and showed that these LLDFs were lower than the AASHTO specifications-compliant LLDFs (Kim and Nowak 1997). The same trend was observed in the studies carried out by Nowak et al. (1999), with Eom (2001) and Nowak et al. (1999) showing that experimental LLDFs were consistently below the AASHTO values (Eom 2001, Nowak et al. 1999). This tendency is also found in studies on prestressed concrete bridges (Sotelino and Liu 2004, Cai et al. 2002). To more accurately determine lateral live load distribution characteristics, modifications to the codified formulas have been made for various bridge types based upon computational parametric studies (Fanous et al. 2010, Cai 2005). These studies improved the accuracy of LLDFs by accounting for a wide range of bridge geometries. Puckett et al. (2011) also developed a simplified computation protocol considering bridge geometric factors to statistically predict LLDFs of highway bridges.

Most studies related to LLDFs have focused on the determination of experimental and analytical LLDFs for normal highway truck-loaded bridges and the comparison of those LLDFs with AASHTO values. A search for literature related to LLDFs for non-highway trucks did not yield any results.

2.3 Dynamic Load Allowance

AASHTO LRFD specifications (2010) define the dynamic load allowance, IM, as an increase in the applied static force effects to account for the dynamic interaction between the bridge and moving loads. The factor to be applied to the static loads should be taken as follows:

$(1 + IM/100)$

The percentage increase, IM, is specified as 33 percent in Table 3.6.2.1.1 of the AASHTO LRFD specification (AASHTO 2010). In the AASHTO Standard Specifications (1996) the Dynamic Load Allowance is prescribed to be:

$50/(L+125) \leq 0.3$

in which L is the span length in feet.

Recognizing that the LLDF and IM of implements of husbandry vehicles can be different from AASHTO codified results, more research is carried out in this project.

3 APPROACH

A detailed description of the entire procedure adopted for the study is presented in this section, which includes a brief summary of the overall project methodology, the field testing procedure, the analytical modeling description, and the parametric study.

3.1 General Description

To study the impact of farm vehicle loadings on live load distribution, different bridge types were identified as common to those states participating in this study. The most common bridge types used for secondary roadways in the Midwest, and which therefore were the focus of this study, include the following:

1. Steel girder bridges with concrete deck (steel-concrete)
2. Steel girder bridges with timber deck (steel-timber)
3. Timber girder bridges with timber deck (timber-timber)

Field testing was carried out on 19 in-service bridges which included 5 steel-concrete bridges, 11 steel-timber bridges and 3 timber-timber bridges. The load tests were completed using four farm vehicles and one five-axle semi-truck. The strain data were employed to determine field LLDFs and field IM.

A computational/analytical model was created of each of the field tested bridges using commercially available finite element analysis software. Each model was calibrated using the collected field data resulting from each vehicular load. The calibrated model then served as a tool to obtain behavioral information when applying 121 actual farm vehicles from a vehicle inventory created through an extensive search of farm vehicles available at the beginning of the project. Each vehicle was made to cross each of the bridge models covering various transverse locations. The analytical strain response was recorded at the same locations as the field testing. The analytical LLDFs were computed for each group of interior and exterior girders, and a statistical analysis was done to determine a representative value for each girder group.

The LLDFs evaluated from each process, which included AASHTO specifications, field testing, analytical simulations, and statistical analysis, are presented as graphical envelopes in Chapter 4. The variability of farm vehicles on LLDFs was evaluated via a comparison of results from different processes.

A parametric study was conducted to develop a new set of equations for LLDFs for farm vehicles similar to AASHTO specifications. A bridge inventory consisting of 151 in-service bridges (Appendix E) was compiled covering the three bridge types from the participating states. Analytical FEA models were generated for each of the bridges by utilizing the experience gained from the calibration of the 19 field tested bridges. The 121 farm vehicles (detailed in Appendix D) were applied to each of the 151 bridges and the response was captured. It was found that farm vehicles have widely varying axle gauge widths that result in different LLDFs compared to typical vehicles. As a result, eight single-axle vehicles with gauge widths varying from 5 ft to 12 ft were used as input in a similar way and were applied to each of the 151 bridges. The FEA models were analyzed considering 121 farm vehicles and eight single-axles to compute analytical LLDFs for each bridge.

Based upon the results of the analytical analysis, two sets of empirical equations were developed for each bridge type using a regression analysis technique. The two sets of equations include an AASHTO LRFD form of equation for (1) generic farm vehicles representing the previously mentioned 121 farm vehicles and (2) eight single-axles with varying gauge widths, respectively. The following sections in this chapter include details of the approach outlined above.

3.1.1 Bridge Description

Nineteen in-service bridges, including five steel-concrete bridges, eleven steel-timber bridges and three timber-timber bridges, were selected for field testing. Representative photographs and cross-section details for each bridge is a part of Volume III - Appendices A, B, and C. The overview of bridges specific to each bridge type is presented in the following sub-sections.

3.1.2 Steel-Concrete Bridges

Five simply supported short-span, steel, I-girder bridges with zero skew were selected for this study. They are located on rural roadways in Boone and Greene counties in Iowa. Table 1 summarizes the significant parameters of the bridges.

Table 1. Steel-concrete bridge characteristics selected for field testing

Bridge	Span Length (ft)	Exterior Girder Spacing (ft)	Interior Girder Spacing (ft)	Number of Girders	Width (ft)	Deck Thickness (in.)	Skew (deg)	Number of Lanes
1	29.9	3.0	2.3	9	18.0	7.5	0	Single
2	39.7	3.3	2.3	12	26.9	7.5	0	Multiple
3	36.1	2.3	2.3	9	18.0	7.5	0	Single
4	37.0	4.9	4.9	5	19.4	7.5	0	Single
5	42.0	3.0	3.3	9	24.3	7.5	0	Multiple

The geometric information for each bridge was obtained from Iowa Department of Transportation (DOT) inspection records and field measurements. Figure 2 shows photographs of a representative steel-concrete bridge. Bridges 1, 3, and 4 are classified as one-lane bridges; whereas, Bridges 2 and 5 have two lanes, as defined by AASHTO specifications (AASHTO 1996) (AASHTO 2010). Bridges 1, 2 and 3 are composed of a concrete deck, all steel interior girders, and two concrete exterior girders; whereas, Bridges 4 and 5 have a concrete deck supported by all steel girders. For all five bridges, the 7.5 in. thick concrete decks were in good condition based upon county inspection data found via the Iowa DOT inspection software.

Figure 2. A representative steel-concrete bridge

3.1.2.1 Steel-Timber Bridges

Eleven continuous single and multi-span steel girder bridges with timber decks (steel-timber) were considered for this study. They are located in Crawford, Boone, and Greene counties in Iowa. Table 2 summarizes the significant parameters of the bridges.

Table 2. Steel-timber bridges characteristics selected for field testing

Bridge	Number of spans	Span Lengths (ft)	Girder Spacing (ft)	Number of Girders	Deck Width (ft)	Deck Thickness (in.)	Skew (deg)	Number of Lanes
1	1	31.0	2.6	10	24.7	4.0	0	multiple
2	1	33.5	2.8	9	24.5	3.0	30	multiple
3	3	34.0, 34.0, 34.0	3.5	7	24.0	4.0	0	multiple
4	2	33.7, 42.0	3.2	8	23.7	4.0	0	multiple
5	1	38.1	2.7	9	22.0	4.0	0	multiple
6	2	24.0, 42.0	3.2	7	21.0	4.0	0	multiple
7	2	19.7, 19.7	1.7	15	23.6	4.0	0	multiple
8	1	28.9	1.7	13	20.4	4.0	7.3	multiple
9	1	29.5	1.7	13	20.3	3.0	0	multiple
10	1	29.9	2.5	8	18.0	3.0	0	single
11	3	24.5,24.5,24.5	1.12*, 3.1	8	18.0	4.0	0	single

* Girder spacing between exterior and interior girders

The geometric information for each bridge was obtained from Iowa DOT inspection records and field measurements. Figure 3 shows photographs of a representative steel-timber bridge. Bridges 10 and 11 are classified as single-lane bridges; whereas, the rest all provide two lanes, based on the AASHTO LRFD specifications (2010). All the bridges have a timber deck and all have steel interior and exterior I-section girders.

Figure 3. A representative steel-timber bridge

3.1.2.2 Timber-Timber Bridges

Three multi-span timber bridges located on a rural roadway in Audubon County in Iowa were selected for this study. Each of the bridges has multiple timber girders with plank decking. The bridge characteristics are summarized in Table 3.

Table 3. Timber-timber bridge characteristics selected for field testing

Bridge	Number of Spans	Span Length (ft)	Average Girder Spacing (ft)	Number of Girders	Width (ft)	Deck Thickness (in)	Skew (deg)	Number of Lanes
1	2	15.1, 15.1	0.98	17	18.0	3.0	0	Single
2	3	19.0, 24.0, 19.0	1.00	27	20.0	6.0	25	Multiple
3	2	32.1, 29.0	1.00	18	17.7	3.0	30	Single

The geometric information for each bridge was obtained from Iowa DOT inspection records and field measurements. Figure 4 shows photographs of a representative timber-timber bridge. Bridge 1 is classified as two traffic lanes and has two equal spans of 15.1 ft and zero skew supports. Bridge 2 carries two-way traffic and is a three-span timber girder bridge. It has a total span length of 62 ft from center to center of abutments. The first, second, and third span lengths are 19.0 ft, 24.0 ft, and 19.0 ft, respectively. Bridge 3 carries two-way traffic and has a total span length of 61.1 ft. This bridge has two unequal spans of 32.1 ft and 29.0 ft For Bridges 1 and 3, the 3 in. thick timber deck and for Bridge 2, the 6 in. thick timber deck was in satisfactory condition.

Figure 4. A representative timber-timber bridge

3.1.3 Testing Vehicles Description

The heaviest vehicles that could safely cross the structure without overloading, overstressing, or causing damage were obtained for all the test bridges. The state rating engineer approved final selection of the load testing vehicles. The farm vehicles used in field testing included a tractor with one honey wagon tank, a tractor with two honey wagon tanks, a TerraGator (with either a dual-wheel front axle or single-wheel front axle), and a tractor with a grain wagon. In addition to farm vehicles, a five-axle semi-truck was also used in field testing as the only conventional highway truck in the inventory. The configurations and photographs of the vehicle inventory are shown in Table 4 and Figure 5 respectively.

Table 4. Characteristics of test vehicles

Vehicle	Axle Number	Axle Weight (kips)						Axle Spacing (ft)				
		W1	W2	W3	W4	W5	W6	S1	S2	S3	S4	S5
Tractor with one tank	5	11.80	15.92	16.28	16.28	16.28	NA	10.8	18.4	5.9	5.9	NA
Tractor with two tanks	6	20.26	16.07	7.15	7.15	9.15	9.15	12.8	21.0	6.2	17.1	6.2
TerraGator	3	11.06	16.21	16.21	NA	NA	NA	19.4	6.2	NA	NA	NA
TerraGator with single-wheel front axle	2	11.06	16.21	16.21	NA	NA	NA	19.4	6.2	NA	NA	NA
Tractor grain wagon	3	18.84	18.66	15.67	NA	NA	NA	11.2	24.0	NA	NA	NA
Five-axle semi truck	5	11.04	17.38	17.38	17.02	17.02	NA	12.1	4.3	31.8	3.9	NA

Axle weights and spacings are defined as the total weight of all wheels on each axle and the spacing between wheel axles, respectively
NA = Data are not applicable

(a) Tractor with one honey wagon tank

(b) Tractor with two honey wagon tanks

(c1) TerraGator with single-wheel front axle

(c2) TerraGator with dual-wheel front axle

(d) Tractor with a grain wagon

(e) Semi-truck

Figure 5. Test vehicles

3.1.4 Testing Methodology

The field testing followed a step by step procedure that included inspecting the bridge visually, implementing the instrumentation plan, assembling load testing equipment, conducting parallel load tests, and dissembling the instrumentation installation.

Visual inspection of the bridge was completed to ensure that the overall structure condition was acceptable. Load posting information for each bridge was obtained and reviewed prior to any load testing. As-built plans and previous inspection/rehabilitation reports for each bridge were reviewed and a site visit was made to confirm any modification or member replacements made to the bridge. The primary goal of the testing was to measure the live load response of selected structural components. A network of multiple strain gauges attached to the bottom flanges at the mid-span of all girders was used to measure strain quantities. The field data acquisition system for strain gauge measurements was acquired from Bridge Diagnostics Inc. The system was tested and adjusted for the initial condition before conducting the live load testing. The bridge was open to traffic with special guidance and awareness of safety during the instrumentation and testing process.

Once the setup was ready for testing, the deck of the bridge was marked at intervals along the vehicle path. The test vehicles were run along a predesignated path with a manual clicker marking the location so that the strain values could be analyzed as a function of vehicle location. Each test vehicle was driven approximately along the centerline of the bridge as shown in Figure 6.

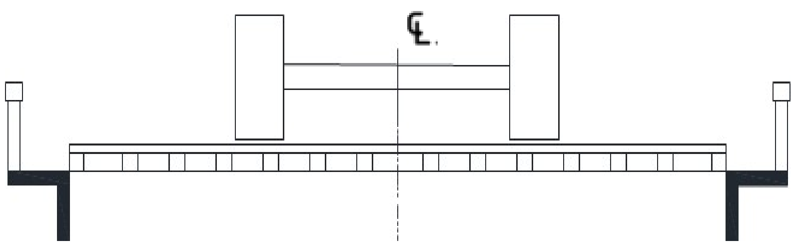
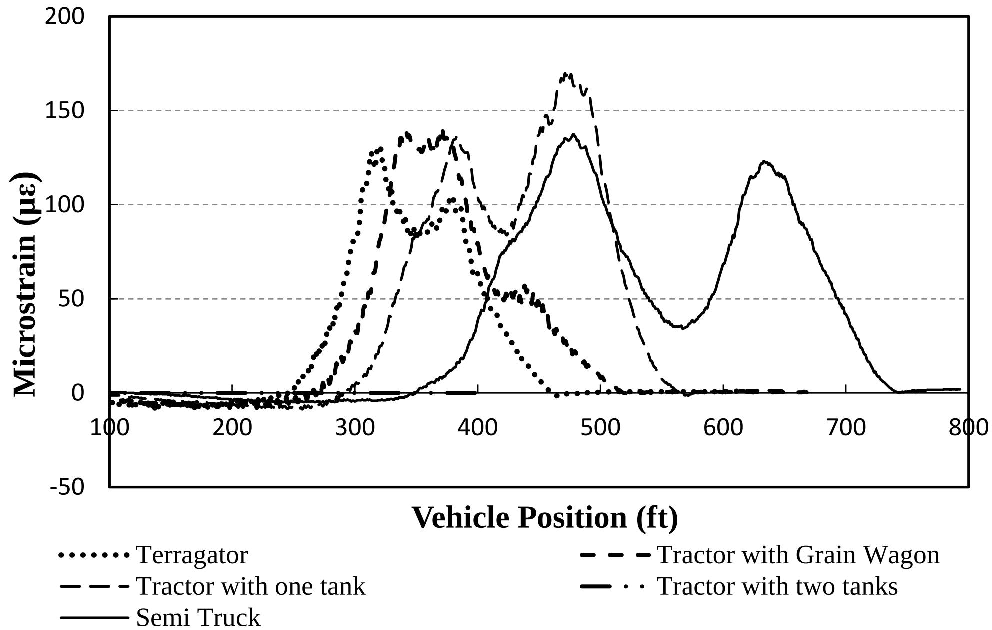


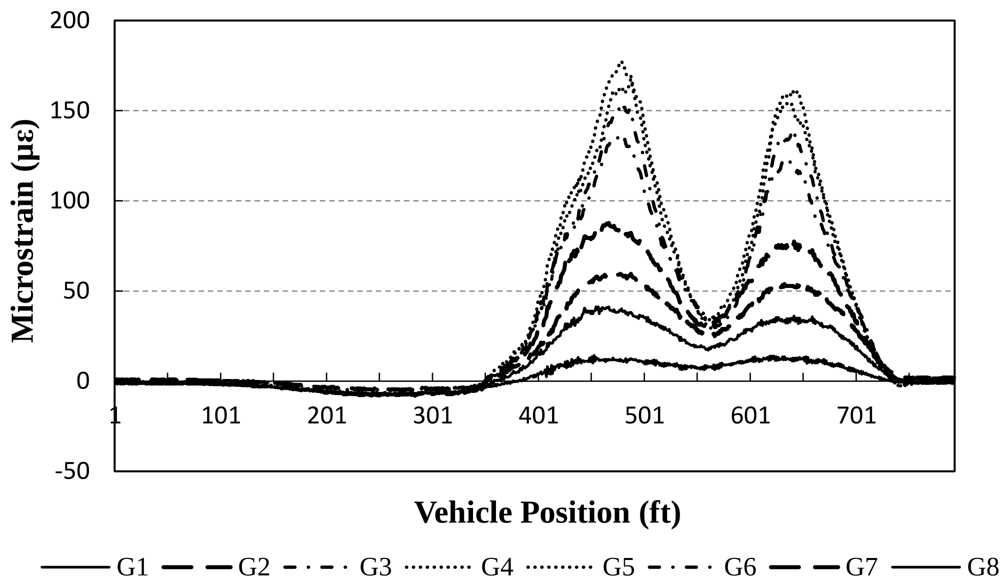
Figure 6. Location of vehicle during field testing

3.1.4.1 Static Load Testing

During the static load testing process, the test vehicles were driven across the bridge at a crawl speed of approximately 3 mph. Figure 7(a) shows a sample plot of a girder strain for one of the representative bridges. Maximum strain was observed in the central girders for all the bridges as shown in sample strain plot Figure 7(b) for all girders in one of the representative bridges.



(a) Strain plot for representative bridge girder



(b) Strain plot for all girders of a representative bridge under semi-truck loading

Figure 7(a-b). Sample static strain plots

The strain data acquired was employed to calculate field LLDFs for each girder using the following equation (Hosteng 2004).

$$LLDF = \frac{\varepsilon^{m} \max i,t}{\sum_{i=1}^{n} \varepsilon^{m} \max i,t} \; or \; \frac{M^{m} \max i,t}{\sum_{i=1}^{n} M^{m} \max i,t} \tag{8}$$

where: LLDF = field live load distribution factor
ϵm = measured maximum strain (με)
Mm = measured maximum moment (kip-in)

In the above equation (8), the field LLDFs for steel-concrete bridges were computed using moment values because the steel girder sections behave composite with the concrete deck. Whereas, strain values were used to calculate the field LLDFs for steel-timber and timber-timber bridges.

3.1.4.2 Dynamic Load Testing

During the dynamic load testing process, the test vehicles were driven across the bridge at a speed of 10 to 25 mph (maximum safe speed at the site). Generally, the girders experienced more strain under dynamic loading than static loading. Figure 8 shows a comparison between the dynamic and static strains for a representative girder. The strain values from the dynamic load tests were utilized to calculate the IM for each girder.

IM = (Maximum dynamic strain – Maximum static strain) ÷ Maximum static strain × 100

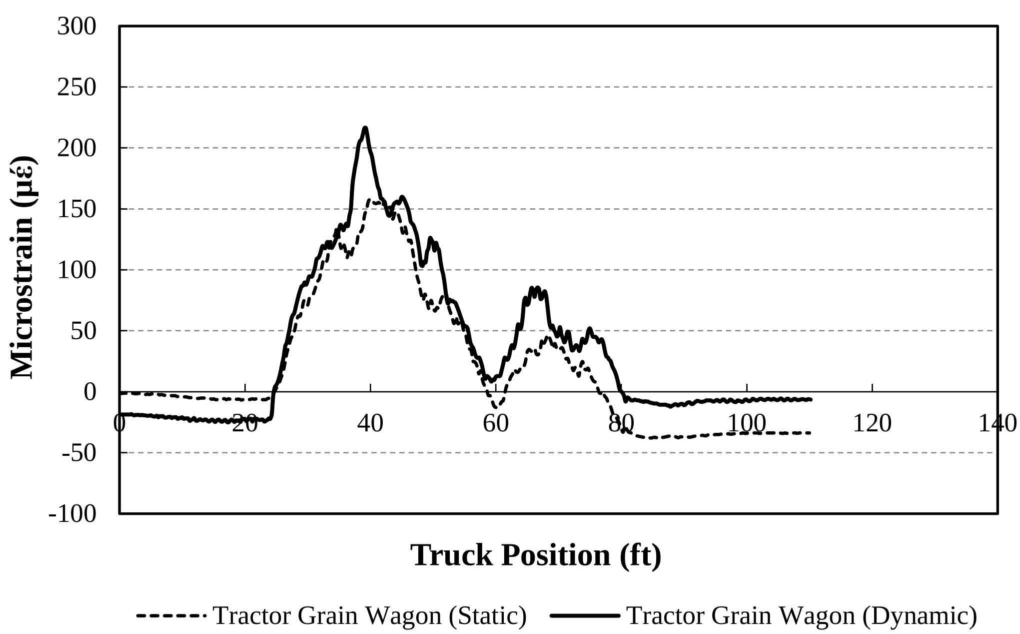


Figure 8. Comparison plot between dynamic and static strain for a representative girder

3.2 Analytical Modeling

Field testing is not feasible when the LLDF characteristics for bridges loaded with a large number of different agricultural vehicles are needed. Also, it is practically impossible to conduct field testing on every bridge of interest. Therefore, an analytical study was conducted (Sotelino and Liu 2004, Puckett et al. 2011). Many researchers like Bishara et al. (1993), Kim and Nowak (1997) and Fanous et al. (2011) considered FEA to be the most efficient way to determine analytical LLDFs. FEA simulations have been considered efficient for reasonably determining LLDFs for typical steel or timber girder bridges (Brockenbrough 1986, Tarhini and Frederick 1992, Phuvoravan 2006).

3.2.1 Methodology

The Methodology described below includes a description of model generation and calibration processes, as well as analytical vehicle combinations. It also includes a method to evaluate analytical LLDFs and statistical analyses to interpret the analytical results.

3.2.1.1 Model Generation

Each of the bridges was modeled with appropriate geometric and material properties using commercially available finite element software (BDI 2010). The geometric information, such as girder spacing, was obtained from the bridge plans and/or field inspections. Each FEA model consists of beam elements for girders, shell elements for the deck, and rotational springs necessary for simulating actual behavior of supports such as abutments and bearings at piers. The modulus of elasticity for steel and timber girders were assumed to be 29000 and 1600 Ksi, respectively. The moduli of elasticity for concrete and timber decks were assumed to be 3200 and 1600 Ksi, respectively. The assumptions were based upon the materials used during construction taken from bridge plans and considering the AASHTO LRFD Specifications (AASHTO 2010). Figure 9 shows a representative bridge model.

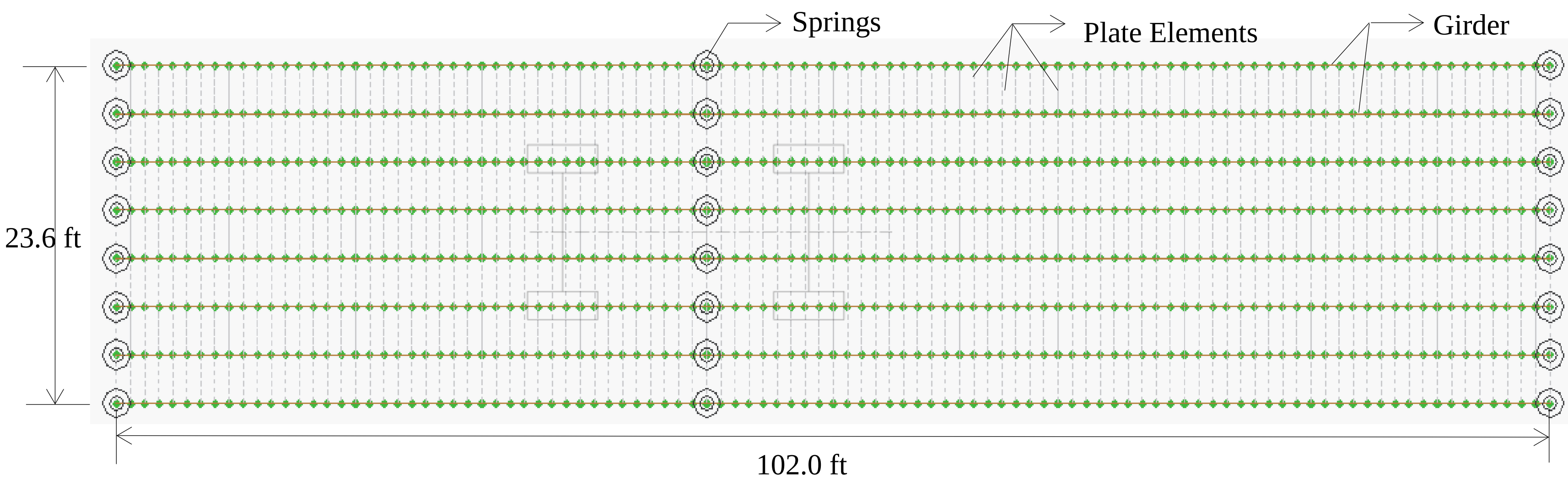


Figure 9. Finite element bridge model

3.2.1.2 Model Calibration

After initial model generation, each model was calibrated with the field-collected data. Model calibration is an iterative process to obtain the highest correlation and the lowest errors between the analytical and field strain responses. This was accomplished by altering sectional and/or material properties for each model within reasonable limits that were established by previous work, field inspection, and bridge plans (Seo et al. 2013); thus, this made the model as accurate as possible so that it could reasonably predict the actual behavior of each bridge. Calibration parameters included the modulus of elasticity and moment of inertia for timber girders and decks, and rotational stiffness at the supports. For each of the iteration processes, a graphical user interface tool in the BDI software was utilized to graphically and statistically make comparisons between the field and analytical results. The same procedure was repeated with each of the test vehicles, and model parameters for each of the 19 bridges were modified.

The model accuracy is measured using the following statistical measures (Seo et al. 2013).

$$\delta_p = \left[\frac{\sum \left(\varepsilon_f - \varepsilon_a \right)^2}{\sum \left(\varepsilon_f \right)^2} \right] \times 100 \tag{9}$$

$$\delta_s = \left(\frac{\sum \max \left| \varepsilon_f - \varepsilon_a \right|}{\sum \max \left| \varepsilon_f \right|} \right) \times 100 \tag{10}$$

$$\rho_{f,a} = \left[\frac{\sum \left(\varepsilon_f - \overline{\varepsilon_f} \right) \left(\varepsilon_a - \overline{\varepsilon_a} \right)}{\sum \sqrt{\left(\varepsilon_f - \overline{\varepsilon_f} \right)^2 \left(\varepsilon_a - \overline{\varepsilon_a} \right)^2}} \right] \tag{11}$$

where: δp = Percent error
δs = Percent scale error
$\rho f,a$ = Correlation coefficient
ϵf and ϵa = Field and analytical strain quantities
$\bar{\epsilon}_f$ and $\bar{\epsilon}_a$ = sample means of ϵf and ϵa, respectively

The aim was to calibrate the model with the lowest possible percent error (δp) and the highest possible correlation coefficient ($\rho f,a$).

3.2.1.3 Analytical Vehicle Combinations

To extend the study to a more complete range of vehicles, a variety of farm vehicles and implements with differing vehicular characteristics was used. Through internet searches and manufacturer inquiries, information regarding axle weights and configurations was gathered for 121 farm vehicles and implements. These combinations encompassed most of the combinations seen on US secondary roadway bridges at the time the study was initiated. The distribution of length and weight of the 121 vehicles and implement combinations are described in Figure 10 through histograms. Vehicular characteristics of farm vehicles vary widely, with an average weight and length of approximately 100 kips and 40 ft, respectively. The details of each farm vehicle, including the number of axles, axle width, axle weight, and axle spacing, are provided in Volume III – Appendix D.

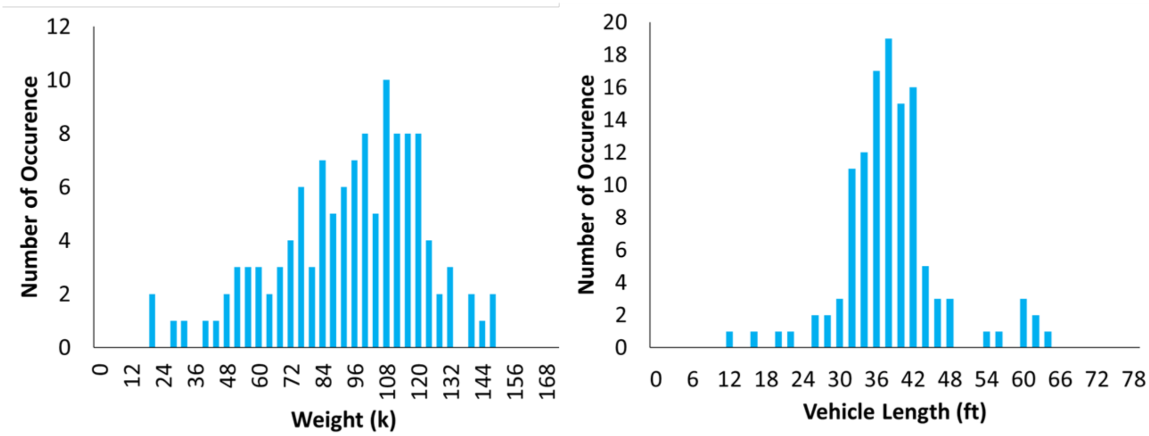


Figure 10. Histograms of vehicle inventory with reference to weight and length

Each of the 19 calibrated bridge models was loaded by each of the 121 farm vehicles through an automation process developed specially for this study to explore the effects of variability in the farm vehicle characteristics on LLDFs. The vehicles were made to cross each model covering various transverse locations. Note that the number of transverse locations for each vehicle depends upon its axle width and the bridge width measured from curb to curb. For example, the first transverse location is taken at a 2 ft distance from one edge of the curb of the bridge and the rest are taken at 1 ft intervals from the previous location until the vehicle is at a 2 ft distance from the curb on the other side. Figure 9 shows a sample transverse location of a test vehicle on bridge model travelling in the longitudinal direction.

3.2.1.4 Analytical Live Load Distribution Factor

The analytical strain values were used to determine analytical LLDFs for each girder in each of the 19 bridges for each of the 121 farm vehicles using the following equation (Hosteng 2004).

$$LLDF = \frac{\varepsilon^{m} \max i,t}{\sum_{i=1}^{n} \varepsilon^{m} \max i,t} \, or \, \frac{M^{m} \max i,t}{\sum_{i=1}^{n} M^{m} \max i,t} \tag{12}$$

where: $LLDF$ = Analytical Live Load Distribution Factor
ϵ^m = Recorded maximum strain (με)
M^m = Recorded maximum moment (kip-in)

where $LLDF$ is the Analytical Live Load Distribution Factor; ϵ^m and M^m are the calculated maximum analytical strains and moments for individual girders, respectively.

3.2.1.5 Statistical Analysis

As stated previously, the AASHTO specifications provide LLDF equations for all interior girders and all exterior girders for all bridge types. Hence, the analytical LLDFs of all the girders of each bridge were grouped into interior and exterior girder LLDFs (Cai 2005, Fanous et al. 2010, Barker and Puckett 2013). Statistical analysis was completed on the computed analytical LLDFs for each girder group of all bridge types based upon a basic probabilistic theory, resulting in their discrete cumulative distribution functions (CDFs) as shown in equation (13).

$$F_X(x) = P(X \leq x) = \sum_{all \quad x_i \leq x} P(X = x_i) \tag{13}$$

where F_X = CDF
X = simulated data
P = discrete probability corresponding to x

CDF plots show the probabilistic variation trend of analytical LLDFs and help us to determine any statistical limit of interest. Statistical interior and exterior girder LLDF limits for the bridges were defined to be the 95% confidence thresholds, showing the probability that computed LLDFs are beyond the thresholds of 5%. Figure 11 shows a CDF plot of a sample bridge showing the LLDF distribution of exterior and interior group of girders.

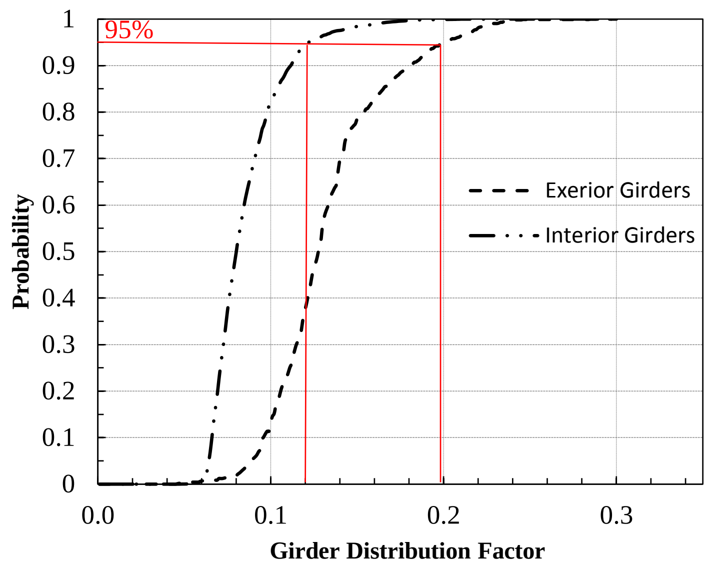


Figure 11. CDF plot showing LLDF distribution

3.3 Parametric Study

The parametric study involves analytical analyses of a large number of in-service bridges to develop a new set of empirical equations to determine LLDFs for farm vehicles. A detailed account of the procedure selecting the 151 in-service bridges, the effect of gauge width on LLDFs, the extrapolation of modeling concepts from the 19 field tested bridges to the bridge inventory, and the empirical equation development are presented in this section.

3.3.1 Bridge Inventory

Two primary sources of data were used to develop the bridge inventory of existing, in-service bridges of all three bridge types necessary for the parametric study and regression analysis. The two sources were the Structure Inventory and Inspection Management System (SIIMS) from the Iowa DOT website and bridge plans received from Wisconsin and Oklahoma departments of transportation. From these sources, a total of 174 bridges were identified. Of these 174 bridges, 151 were identified as having sufficient data for the parametric study.

Overall, the 151 in-service bridges included 45 steel-concrete bridges, 54 steel-timber bridges and 52 timber-timber bridges. The gathered information included both structural and management information for each specific bridge.

Structural information included data that are used for analytical modeling and evaluating code-specified LLDFs, such as bridge type, material, number of spans, individual span lengths, transverse bridge width, skew angle, and number of traffic lanes. It also included information regarding girder geometry and material properties.

Management information included data on the cost, year of construction, and inspection information.

For some bridges, information that was missing on two or three bridge geometric parameters was assumed based on the AASHTO specifications and engineering judgment. Table 5 provides an overview of the bridge inventory with respect to structural parameters. The detailed information specific to each bridge is presented in Volume III – Appendix E. Bridge Inventory.

Table 5. Bridge inventory summary

Bridge Type		No. of bridges	No. of spans	Span length (ft)		Girder Spacing (ft)		No. of Girders		Bridge Width (ft)		Deck thickness (in.)		Skew angle (deg)	
				Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Steel-Concrete	One-Way	5	1	26	50	2.0	6.4	4	10	16	20	7.5	7.5	0°	0°
	Multiple	24	1-4	30	132	1.7	10.0	3	16	20	40	4.6	8.8	0°	6°
	Skewed	16	1-4	28	104	1.8	9.7	4	14	20	36	4.6	10.5	20°	55°
Steel-Timber	One-Way	23	1-3	19	61	1.5	5.0	4	11	15	20	3.0	6.0	0°	0°
	Multiple	21	1-2	20	59	1.5	4.5	6	17	20	31	2.8	6.0	0°	0°
	Skewed	10	1	21	49	2.3	2.8	7	11	16	25	3.0	4.0	15°	45°
Timber-Timber	One-Way	33	1-4	16	58	0.8	2.2	8	23	15	20	2.8	4.5	0°	0°
	Multiple	9	1-7	15	24	0.8	2.1	12	28	21	24	3.0	3.4	0°	0°
	Skewed	10	1-4	17	26	0.9	2.5	10	22	15	24	2.8	4.0	10°	47°

3.3.2 Gauge Width

The AASHTO equations were developed using typical highway-type vehicles H20 and HS20 trucks with an axle gauge width of 6 ft. The axle gauge width of a vehicle is defined as the center to center distance between the wheels of the axle, as shown in Figure 12.

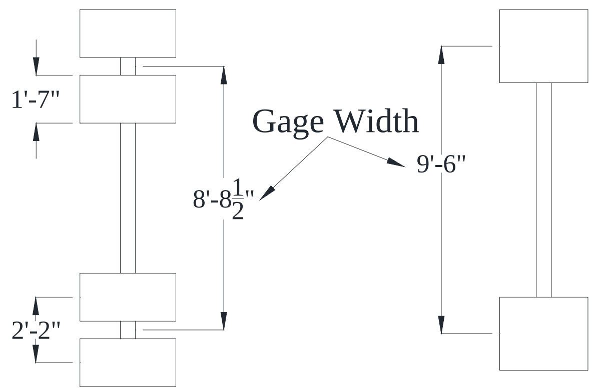


Figure 12. Axle gauge width measurement for two types of axles

A study of the 121 farm vehicles determined the variation of axle gauge widths in farm vehicles. Figure 13 shows the population distribution of the number of axles with respect to axle gauge width for the 121 farm vehicles. The histogram shows that most of the farm vehicles have axle gauge width between 7 ft to 11 ft; quite different from typical highway vehicles.

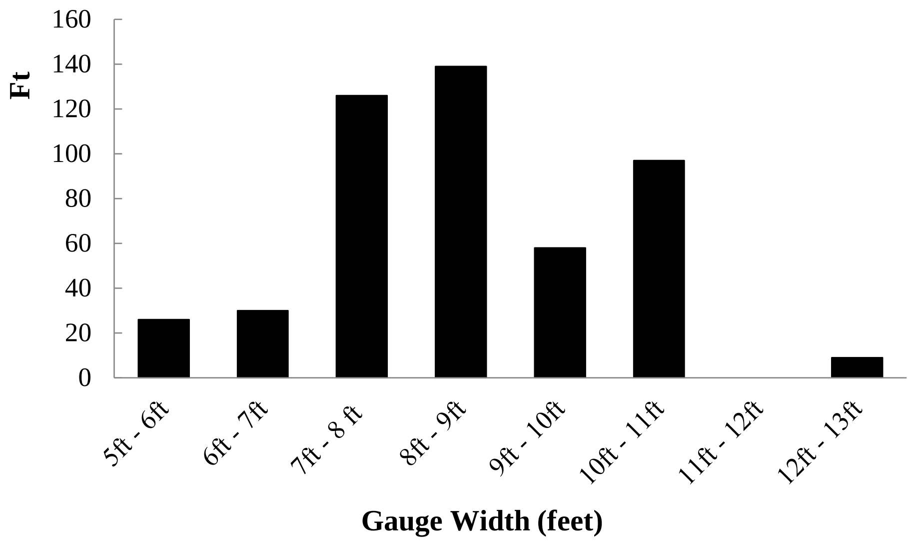


Figure 13. Histogram showing variation of gauge width of farm vehicle inventory

In some cases, a farm vehicle consists of a 5.1 ft front axle and a 12.2 ft rear axle. Therefore, eight single-axles with gauge width varying from 5 ft to 12 ft in 1 ft increments were selected as shown in Figure 14. For analysis purposes only, an axle weight of 10 kips was considered on each of the eight axles.

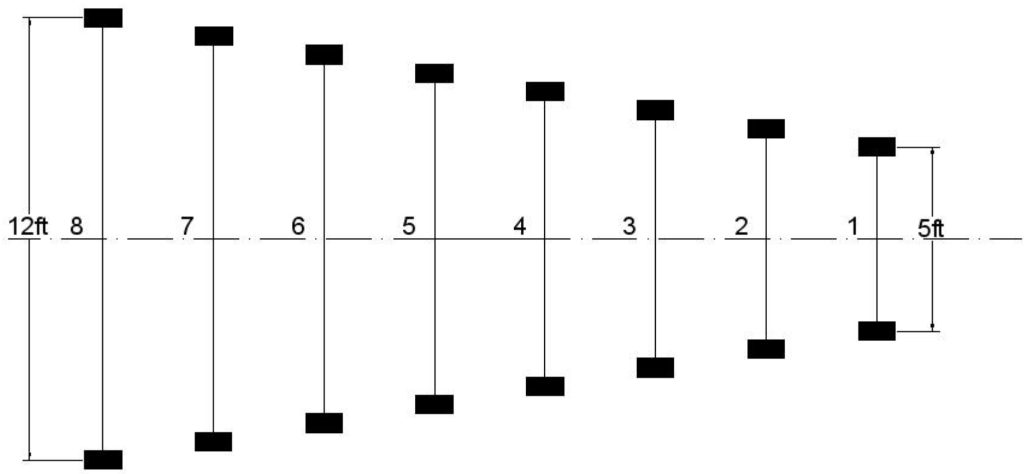


Figure 14. Footprints of eight axles with varying gauge widths of 5 ft to 12 ft

3.3.3 Extrapolation

FEA models of 151 bridges were generated, similar to the 19 field-tested bridges, using bridge geometric parameters following a common linear-elastic approach as explained in Section 3.3.1.1. After model generation, each of the eight axles was analytically modeled as point loads with 5 kips per wheel.

The 121 farm vehicles and eight axles were driven across each of the 151 models at 1 ft increments in the longitudinal direction covering various transverse locations similar to the 19 field bridges as explained in Section 3.2.1.3. Analytical LLDFs were determined from the calculated strain for 121 farm vehicles and eight axles as explained in Section 3.2.1.4 using equation (12). Each girder has a large number of analytical LLDFs for every transverse position of each axle in the lateral direction. Then the girders are grouped into two groups, exterior and interior, and statistical analysis is carried out as explained in Section 3.2.1.5.

Skewed bridges were modeled twice: considering the bridge with skew and without skew. The skew correction factors were determined as a ratio of the maximum analytical LLDFs found in two models, skewed and non-skewed, respectively.

3.3.4 Empirical Equation Development

The maximum exterior and interior analytical LLDFs were calculated from the analysis of 121 farm vehicles and eight axles for each bridge. Therefore, each bridge analysis resulted in two sets of maximum exterior and interior analytical LLDF values: one for the 121 farm vehicles and one for the eight axles. The above procedure was implemented for all the bridges in the inventory with the help of an automation program developed using Visual Basic and MATLAB. These values were used as data points to develop empirical equations for the 121 farm vehicles and eight axles.

3.3.4.1 Modified AASHTO Code Equation – Effect of 121 farm vehicles

The empirical equations were developed considering the same bridge geometric parameters considered by AASHTO LRFD specifications. As stated previously, AASHTO LRFD equations are based on S-over rule for steel-timber and timber-timber bridges, and more sophisticated equations for steel-concrete bridges.

The general form of the S-over rule for steel-timber and timber-timber bridges can be written as equation (14).

$$LLDF = \frac{S}{D} \tag{14}$$

where S = girder spacing (ft)
D = Numerical factor

The generic form of the LLDF equations, according to AASHTO LRFD specifications for steel-concrete bridges, can be written as equation (15).

$$LLDF = \left(\frac{S}{D}\right)^{\alpha}\left(\frac{S}{L}\right)^{\beta}\left(\frac{K_g}{12Lt_s^3}\right)^{\gamma} \tag{15}$$

where S = girder spacing (ft)
L = maximum span length (ft)
K_g = longitudinal stiffness (in^4)
t_s = deck thickness (in.)
D = numerical factor
α, β and γ = exponential coefficients

The above equation (15) was developed for each bridge type for exterior and interior girders considering the effect of 121 farm vehicles. The logarithmic function was applied to equation (15) to make it linear, and it can be written as equation (16).

$$\log(LLDF) = \alpha \log(S) - \alpha \log(D) + \beta \log\left(\frac{S}{L}\right) + \gamma \log\left(\frac{K_g}{12Lt_s^3}\right) \quad (16)$$

The maximum analytical LLDFs recorded from analytical analysis and bridge geometric parameters are used as data points to determine the numerical factor D and exponential coefficients α, β and γ, using a linear regression analysis tool. The linear regression tool outputs the coefficients as well as an intercept. The coefficients are parallel to α, β and γ, while the intercept represents the –αlog (D) term in equation 16. The fit of the empirical equation developed is measured using R square and Standard error values. The R square is a number that indicates how well the equation fits the data that is measured from analytical LLDFs and predicted LLDFs from the equation. The Standard error measures the deviation of predicted LLDFs from equation to analytical LLDFs. High R square and low standard error values indicate good agreement between the equation and the data.

3.3.4.2 Modified AASHTO Code Equation – Effect of Gauge Width (varying axle spacing)

The effect of gauge width is considered in developing empirical equations for the eight axles. The general form of the S-over equation that includes the effect of the gauge width factor can be written as equation (17).

$$LLDF = \left(\frac{S}{D_1}\right)\left(\frac{6}{G}\right)^{\phi} \quad (17)$$

where S = girder spacing (ft)
D1 = Numerical factor
ϕ = Exponential coefficient

The general form of the AASHTO LRFD equation for incorporating the gauge width factor can be written as equation (18).

$$LLDF = \left(\frac{S}{D_1}\right)^{\alpha_1}\left(\frac{S}{L}\right)^{\beta_1}\left(\frac{K_g}{12Lt_s^3}\right)^{\gamma_1}\left(\frac{6}{G}\right)^{\phi} \quad (18)$$

where S = girder spacing (ft)
L = maximum span length (ft)
K_g = longitudinal stiffness (in^4)

t_s = deck thickness (in.)
D_1 = numerical factor
α_1, β_1, γ_1 and ϕ = exponential coefficients

The above specified generic forms of equations 17 and 18 were developed for exterior and interior girders for each bridge type. The numerical factor, D_1 and exponential coefficients α_1, β_1, and γ_1 and ϕ are determined using the same procedure explained above.

4 RESULTS

The approach presented in Chapter 3 was applied to the three bridge types as had been directed by the project advisory committee: (1) Steel girder bridges with concrete deck (steel-concrete), (2) Steel girder bridges with timber deck (steel-timber), and (3) Timber girder bridges with timber deck (timber-timber).

Results of the field tested bridges described in Section 3.1 are presented, which include LLDFs and IM following the approach outlined in Section 3.2. Additionally, the analytical LLDFs from the parametric study and the empirical equations subsequently developed are presented following the approach outlined in Section 3.3.

4.1 Live Load Distribution Factors in Peak Strain for Bridges Field Tested

The results of the field testing of 19 bridges are presented, including LLDFs from: (1) field testing, (2) analytical simulations, (3) statistical analysis, (4) AASHTO specifications (standard and LRFD). For each bridge, both field and analytical values are calculated for each girder following the approach outlined in Chapter 3. The statistical, AASHTO specifications -compliant values were determined for each group of interior and exterior girders. The LLDFs evaluated are presented graphically in the form of envelopes for each bridge.

The bridge location and its description, an explanation of the field testing results, model calibration details and CDF plots for each of the 19 bridges is included in Volume III -Appendix A, B, and C. Mini Reports.

4.1.1 Steel Girder Bridges with Concrete Deck

Table 6 shows the maximum strain experienced by each steel-concrete bridge when the testing vehicles were crossing the bridge. It was observed that the semi-truck caused the maximum strain in the girders compared to all testing vehicles considered.

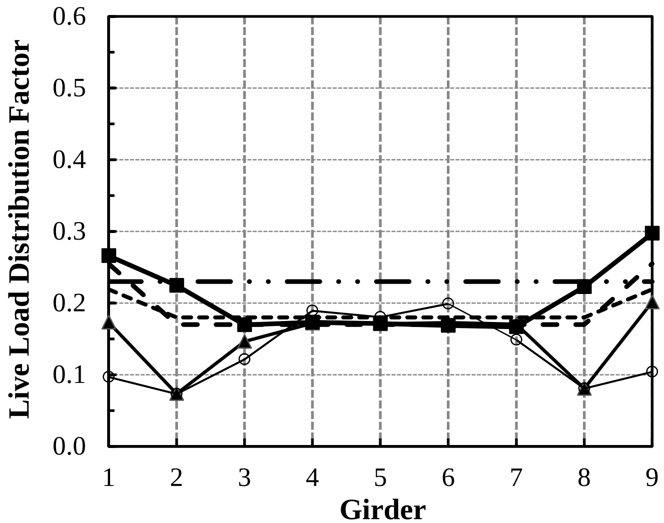
Table 6. Maximum static strain experienced by field-tested steel-concrete bridges

	Testing Vehicles				
Bridge	**Tractor with one tank**	**Tractor with two tanks**	**TerraGator**	**Tractor Grain Wagon**	**Semi-Truck**
1	76	57	61	54	84
2	101	79	85	73	127
3	73	50	57	48	85
4	74	51	59	52	89
5	60	38	44	39	68

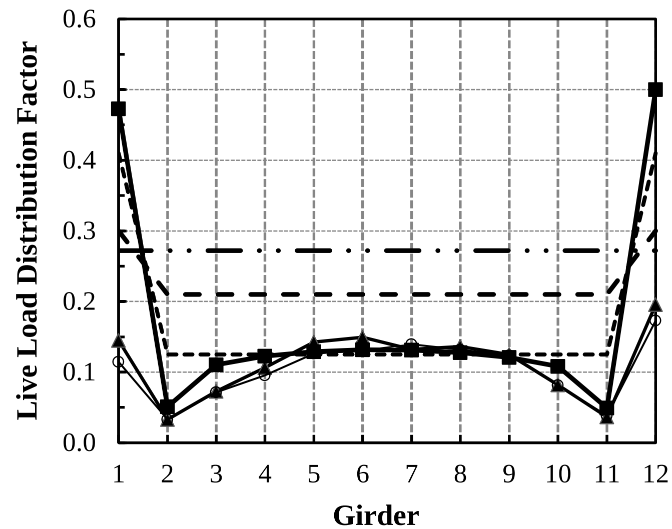
Note: The units of the strain values shown above are microstrain (με)

Figure 15 shows LLDFs for the implements of husbandry and, separately, for the semi-truck of each of the five steel-concrete bridges using the methodology outlined in Chapter 3. Figure 15(a) for Bridge 1 shows that the interior analytical LLDFs for the implements of husbandry are, in all cases, larger than that of the semi-truck, and the exterior envelope is also, in all cases, larger than that of the semi-truck. Figure 15(a) also indicates that the analytical envelope for implements of husbandry for all interior girders is less than the AASHTO LRFD distribution factors, although the envelope for the central girders, such as G3, G4, G5, G6, and G7, is close to the AASHTO standard values. Similarly, the semi-truck LLDFs for the central girders, including G4, G5, and G6, are slightly above the AASHTO standard values and less than the AASHTO LRFD values for all girders. In addition, the analytical LLDFs for implements of husbandry for exterior concrete girders is greater than the AASHTO code values, probably as a result of the larger stiffness of the exterior girders and curbs. The AASHTO standard and LRFD values are 14% and 5% greater, respectively, than the statistical exterior girder limit and 6% smaller and 22% greater, respectively, than the interior girders.

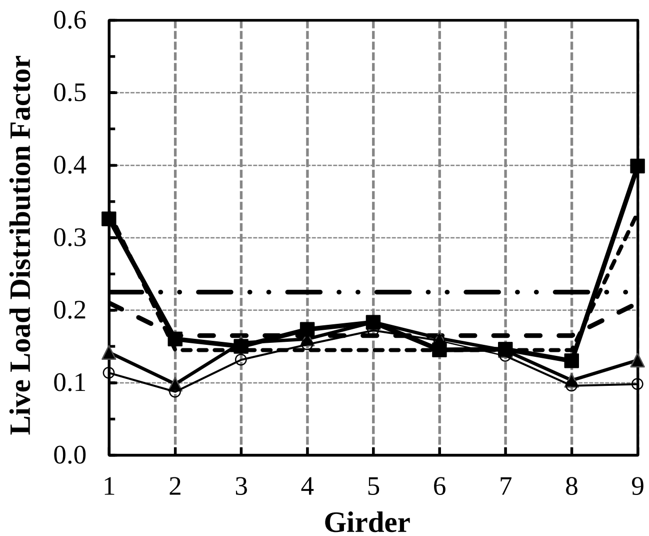
Figure 15(b–d) for Bridges 2 through 4 indicate that the field LLDFs for the implements of husbandry for all interior steel girders are below the AASHTO standard and LRFD values. The analytical LLDFs for the girders of the five steel-concrete bridges are summarized in Table 7, along with both AASHTO values. Analytical LLDFs for the implements of husbandry for the concrete exterior girders are larger than both the AASHTO standard and LRFD values. The analytical LLDFs for the interior girders, however, are less than the AASHTO values, probably due to the increased stiffness of the exterior girders. The statistical exterior girder limits for Bridges 2, 3, and 4 exceed the AASHTO standard values by up to 37%, 60%, and 6%, respectively, and exceed the AASHTO LRFD values by up to 51%, 50%, and 29%, respectively. The statistical interior girder limits for Bridges 2, 3, and 4 are 41%, 12%, and 29% lower, respectively, than the AASHTO standard values and 54%, 36%, and 31% lower, respectively, than the LRFD values. The percent difference between AASHTO values and statistical limits was calculated for all bridges and summarized in Table 8. In contrast, Figure 15(e) for Bridge 5, which consists of all steel girders, shows that the field and analytical envelopes for both exterior and interior girders for the implements of husbandry are smaller than both the AASHTO standard and LRFD values. The statistical limit for interior girders was 29% and 36% smaller respectively, than the AASHTO standard and LRFD values, and the exterior girder limit was 59% and 55% smaller than the AASHTO standard and LRFD values, respectively. By comparison, both AASHTO codes for Bridges 1, 2, 3, and 4, which have exterior girders with significant extra stiffness, when subjected to various normal farm vehicle types and their axle configurations, are, in most cases, acceptable for the interior girders but unsatisfactory for the exterior girders. For Bridge 5, the AASHTO codes are suitable, yet conservative, for both interior and exterior girders.



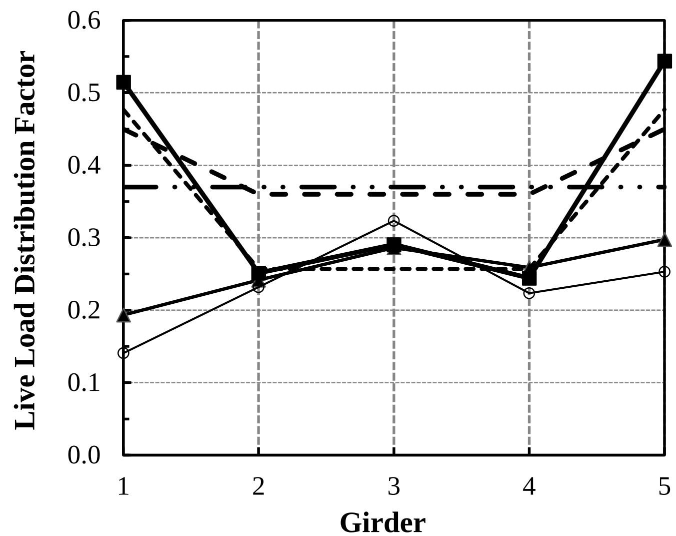
(a) Bridge 1



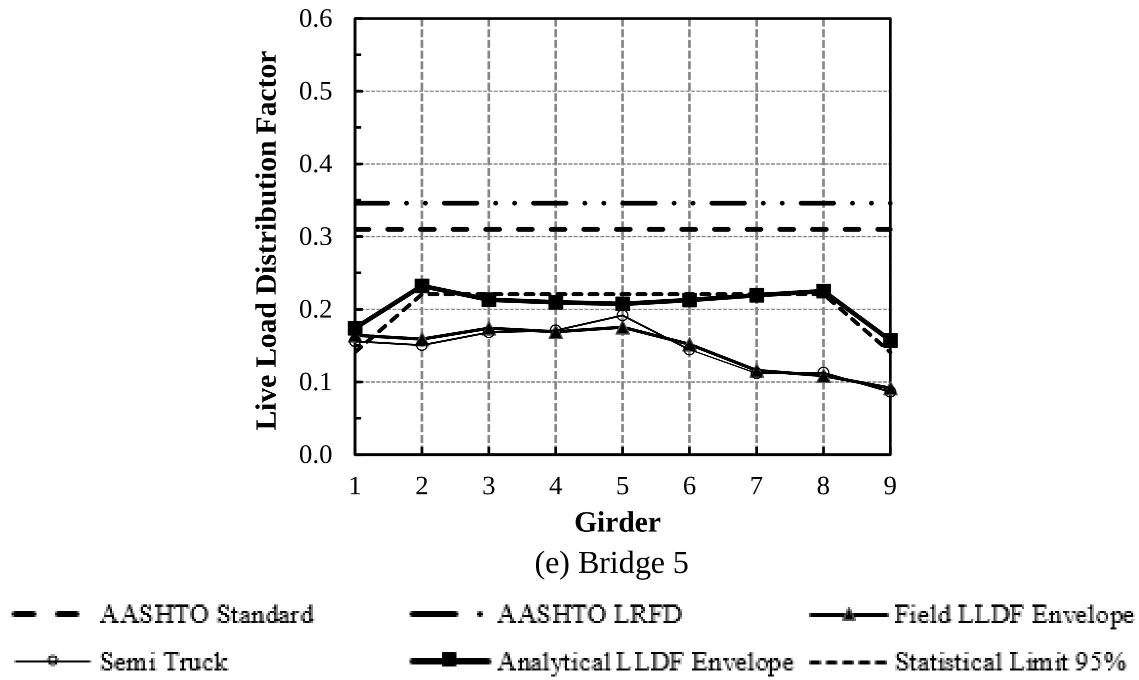
(b) Bridge 2



(c) Bridge 3



(d) Bridge 4



(e) Bridge 5

Figure 15(a-e). LLDFs for field-tested steel-concrete bridges

Table 7. Comparison of analytical and AASHTO-specified LLDFs for field-tested steel-concrete bridges

	Analytical LLDFs												Statistical Limit		AASHTO Codes	
Bridge	**G1**	**G2**	**G3**	**G4**	**G5**	**G6**	**G7**	**G8**	**G9**	**G10**	**G11**	**G12**	**Interior Girders**	**Exterior Girders**	**LRFD**	**Standard**
1	0.27	0.22	0.17	0.17	0.17	0.17	0.17	0.22	**0.30**				0.18	0.22	0.23	0.17
2	**0.47**	0.05	0.11	0.12	0.13	0.13	0.13	0.13	0.12	0.11	0.05	**0.50**	0.13	0.41	0.27	0.21
3	**0.33**	0.16	0.15	0.17	**0.18**	0.15	0.15	0.13	**0.40**				0.15	0.34	0.23	0.17
4	**0.51**	0.25	0.29	0.24	**0.54**								0.26	0.48	0.37	0.36
5	0.17	0.23	0.21	0.21	0.21	0.21	0.22	0.23	0.16				0.22	0.14	0.35	0.31

Note: Highlighted values in the table indicate that analytical LLDFs were greater than AASHTO-specified LLDFs in that case

Table 8. Percent difference between AASHTO-specified LLDFs and statistical limits for field-tested steel-concrete bridges

	Exterior Girder LLDF		Interior Girder LLDF	
Bridge	**AASHTO Standard**	**AASHTO LRFD**	**AASHTO Standard**	**AASHTO LRFD**
1	29%	-4%	6%	-22%
2	95%	52%	-38%	-52%
3	100%	48%	-12%	-35%
4	33%	30%	-28%	-30%
5	-55%	-60%	-29%	-37%

Note: Negative sign indicates that the analytical LLDF was higher than the AASHTO LLDF

4.1.2 Steel Girder Bridges with Timber Deck

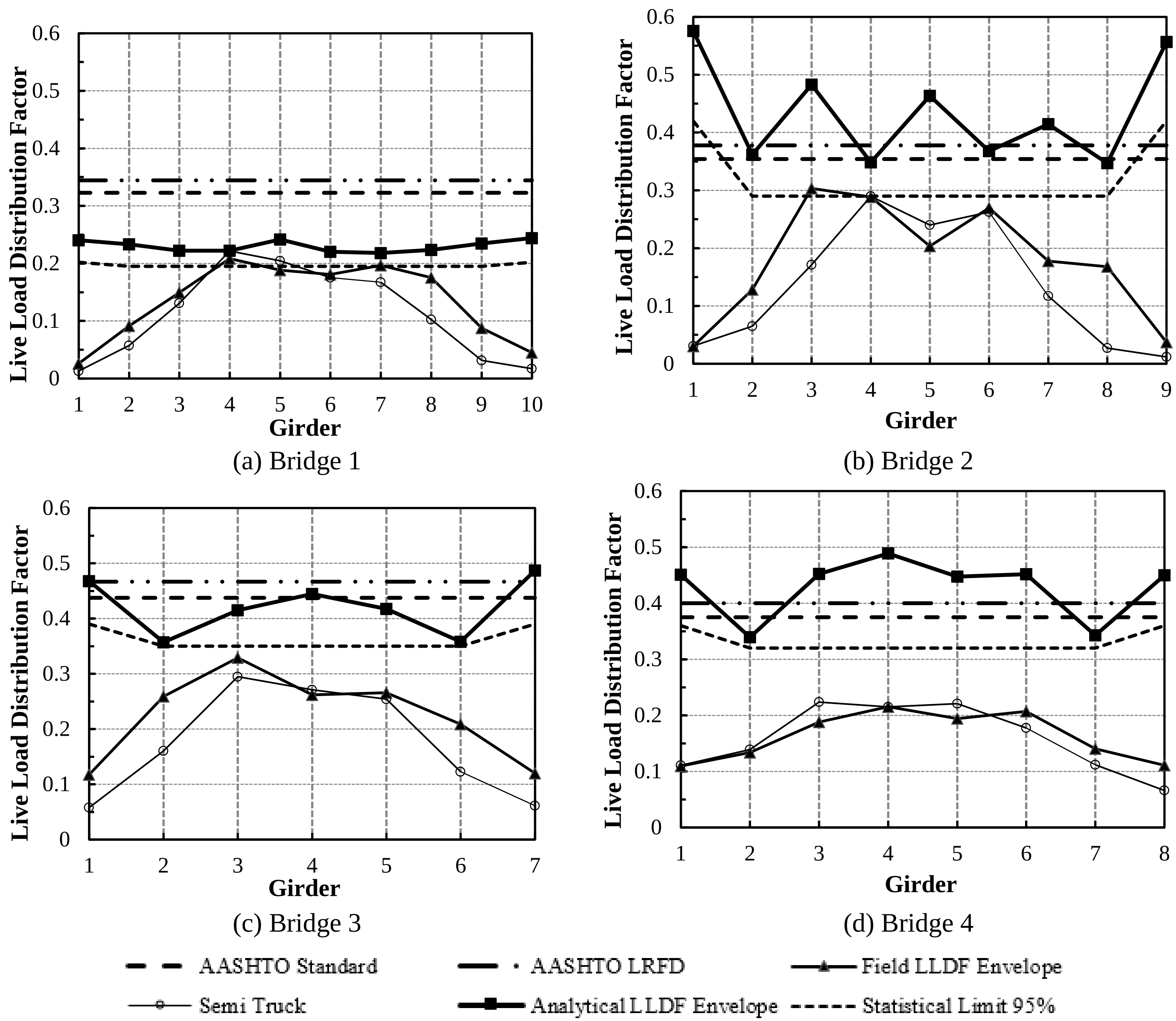
Table 9 shows the maximum strain experienced by each steel-timber bridge when each of the testing vehicles is passed over it. It was observed that the semi-truck caused the maximum girder strain compared to all other testing vehicles for most of the bridges, with the exception of Bridge 8. The TerraGator with a single front axle and the tractor with one tank (half-filled) had strain values closer to that of the semi-truck for Bridges 7, 8, 9, 10, and 11.

Table 9. Maximum static strain experienced by field-tested steel-timber bridges

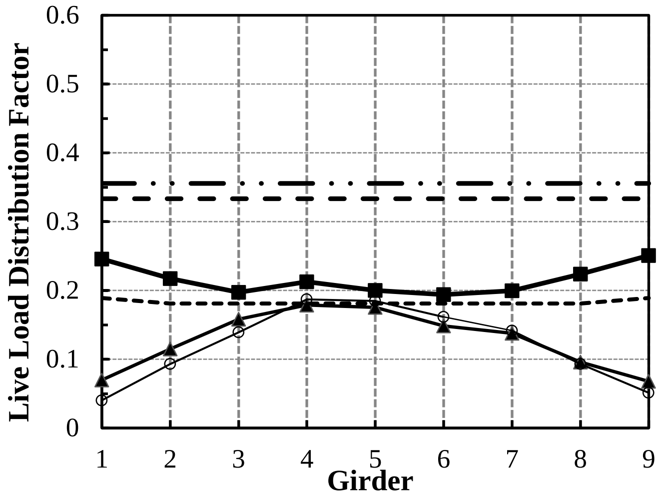
	Testing Vehicles				
Bridge	**Tractor with one tank**	**Tractor with two tanks**	**TerraGator**	**Tractor Grain Wagon**	**Semi-Truck**
1	249	166	234	209	296
2	150	100	133	122	194
3	130	89	124	112	148
4	211	138	182	146	212
5	294	190	267	216	322
6	119	82	98	102	141
Bridge	**Tractor with one tank (half-filled)**	**Tractor with one tank (empty)**	**TerraGator with single front axle**	**Tractor Grain Wagon**	**Semi-Truck**
7	162	108	155	105	181
8	219	153	267	185	264
9	220	135	179	143	240
10	174	119	137	119	197
11	208	177	172	149	229

Note: The units of the strain values shown above are microstrain (με)

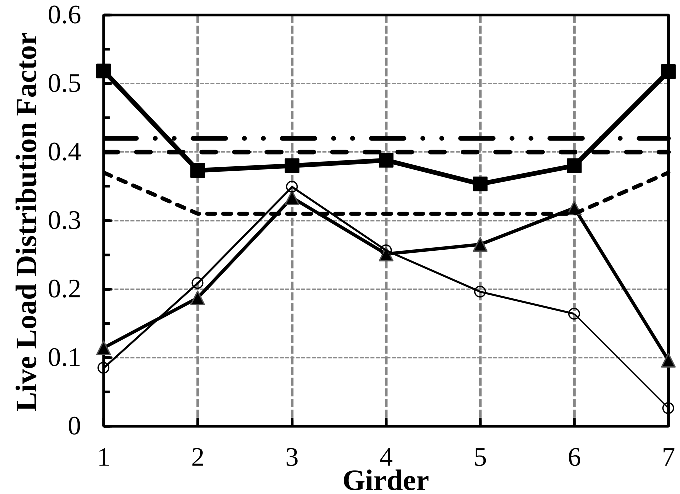
Figures 16(a-k) show the LLDFs for the eleven steel-timber bridges.



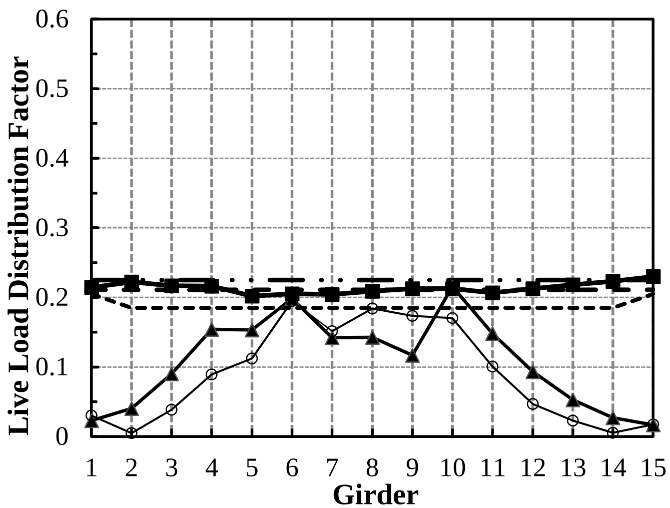
Live Load Distribution Factor
Girder
(a) Bridge 1
(b) Bridge 2
(c) Bridge 3
(d) Bridge 4
AASHTO Standard
AASHTO LRFD
Field LLDF Envelope
Semi Truck
Analytical LLDF Envelope
Statistical Limit 95%



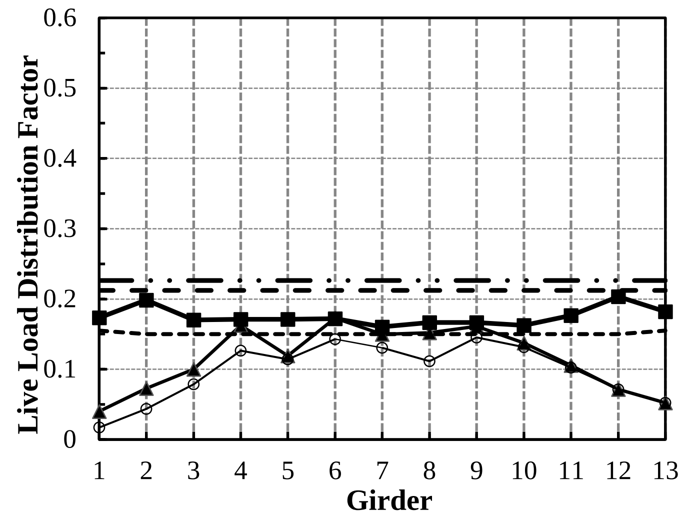
(e) Bridge 5



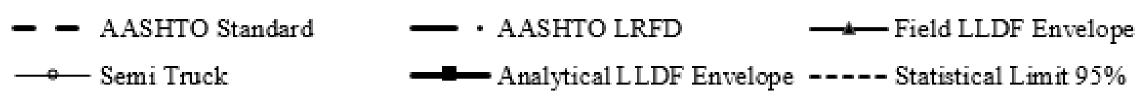
(f) Bridge 6

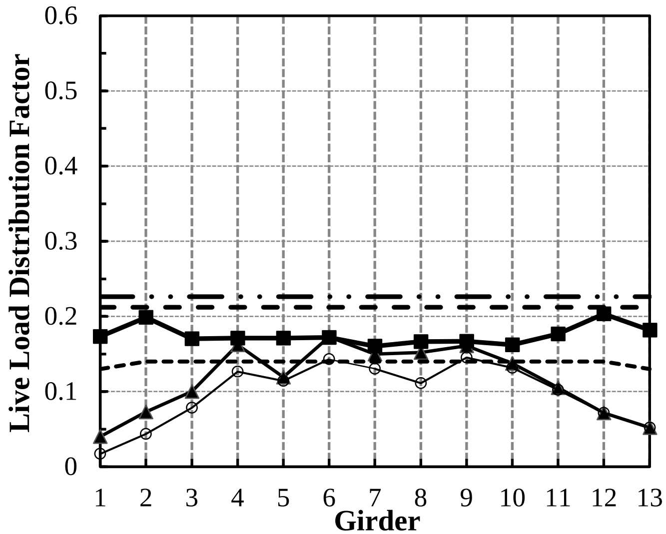


(g) Bridge 7

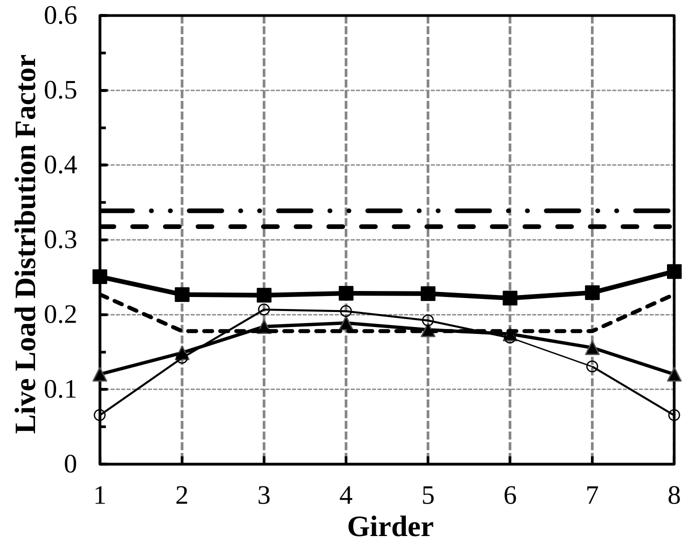


(h) Bridge 8

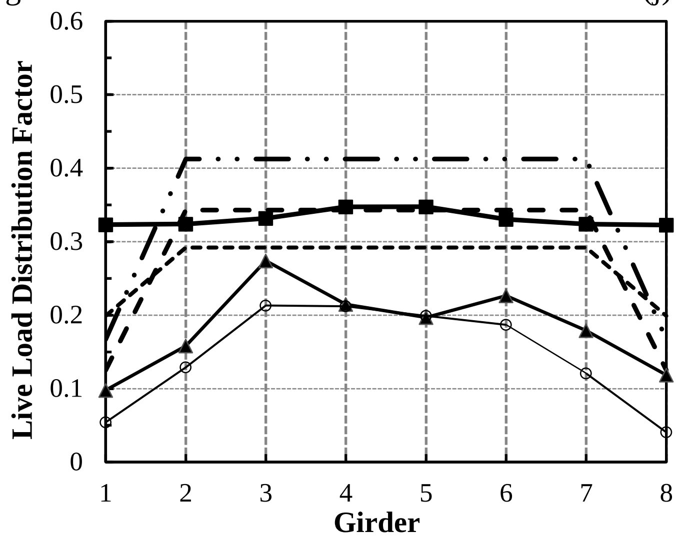




(i) Bridge 9



(j) Bridge 10



(k) Bridge 11

Figure 16(a-k). LLDFs for field-tested steel-timber bridges

Figure 16(a) for Bridge 1 shows that the analytical LLDFs for all exterior girders and interior girders are smaller than those from the AASHTO specifications. The bridge showed a consistent behavior for all the steel girders. Similarly, the field LLDF envelope and semi-truck LLDFs for the interior and exterior girders are less than the AASHTO standard and LRFD values. The statistical limit for interior girders was 40% and 43% smaller than the AASHTO standard and LRFD values, respectively, and the exterior girder limit was 47% and 50% smaller, respectively. For Bridge 1, the analytical LLDF and field LLDF envelopes are, in most cases, larger than the semi-truck plot.

The analytical LLDFs for all the girders of the eleven steel-timber bridges are summarized in Table 10 along with both AASHTO values.

As AASHTO codes specify single LLDF values for exterior and interior girders, the statistical limits for exterior and interior girders are also included. The analytical LLDFs that are higher than AASHTO values are shown in bold. For almost all the bridges, the AASHTO specifications proved to be conservative. The analytical LLDFs exceeded AASHTO values for Bridges 2, 3, 4, and 6 for the exterior girders and Bridges 2 and 4 for the interior girders. The statistical limits were lower than the AASHTO values for all the bridges, except for exterior girders for Bridge 2. The variability of LLDFs in Bridge 2 can be attributed to the skew angle of the bridge. When a farm vehicle with an axle width of 10 ft is made to run across Bridge 2 with a width of 24.5 ft and 30 degrees skew angle, it is possible for one wheel to be on the bridge while the other is completely off the bridge, causing unexpected moment on the girders which results in different LLDFs.

The field LLDFs were greater than the LLDFs from the semi-truck in most girders for all the bridges. Also, the field LLDFs for farm vehicles and a five-axle semi-truck were, in most cases, less than both the AASHTO standard and LRFD values for all the eleven bridges.

Table 10. Comparison of analytical and AASHTO-specified LLDFs for field-tested steel-timber bridges

	Analytical LLDFs for Girders															Statistical Limit		AASHTO Codes	
Bridge	**G1**	**G2**	**G3**	**G4**	**G5**	**G6**	**G7**	**G8**	**G9**	**G10**	**G11**	**G12**	**G13**	**G14**	**G15**	**Interior Girders**	**Exterior Girders**	**LRFD**	**Standard**
1	0.24	0.23	0.22	0.22	0.24	0.22	0.22	0.22	0.23	0.24						0.19	0.20	0.34	0.32
2	**0.58**	0.36	**0.48**	0.35	**0.46**	0.37	**0.41**	0.35	**0.56**							0.29	**0.42**	0.38	0.35
3	**0.47**	0.36	0.41	0.44	0.42	0.36	**0.49**									0.35	0.39	0.47	0.44
4	**0.45**	0.34	**0.45**	**0.49**	**0.45**	**0.45**	0.34	**0.45**								0.32	0.36	0.42	0.40
5	0.25	0.22	0.20	0.21	0.20	0.19	0.20	0.22	0.25							0.18	0.19	0.36	0.33
6	**0.52**	0.37	0.38	0.39	0.35	0.38	**0.52**									0.31	0.37	0.42	0.40
7	0.21	0.22	0.22	0.22	0.20	0.21	0.20	0.21	0.21	0.21	0.21	0.21	0.22	0.22	0.23	0.19	0.21	0.23	0.21
8	0.17	0.20	0.17	0.17	0.17	0.17	0.16	0.17	0.17	0.16	0.18	0.20	0.18			0.15	0.16	0.23	0.21
9	0.16	0.20	0.17	0.17	0.17	0.17	0.16	0.17	0.17	0.17	0.17	0.20	0.16			0.14	0.12	0.23	0.21
10	0.25	0.23	0.23	0.23	0.23	0.22	0.23	0.26								0.19	0.23	0.38	0.28
11	0.32	0.32	0.33	0.35	0.35	0.33	0.32	0.32								0.20	0.29	0.34	0.41

Note: Highlighted values in the table indicate that analytical LLDFs were greater than the AASHTO-specified LLDFs in that case

The percent difference between AASHTO values and statistical limits were calculated for all bridges and summarized in Table 11. For Bridge 5, the AASHTO standard and LRFD LLDFs were the most conservative compared to analytical LLDFs among all the eleven bridges; greater than the exterior girder statistical limit by 43% and 47% respectively, and 46% and 49% greater than the interior girder statistical limit, respectively. Bridges 4 and 6 have the same girder spacing and AASHTO codes provide the same LLDFs. It was observed that Bridges 4 and 6 have different analytical LLDFs, indicating that other bridge characteristics are important in determining LLDFs. Bridge 2 has exterior girder statistical limits greater than AASHTO values by 19% and 11%, respectively.

Table 11. Percent difference between AASHTO-specified LLDFs and statistical limits for field-tested steel-timber bridges

	Exterior Girder Statistical Limit		Interior Girder Statistical Limit	
Bridge	**AASHTO Standard**	**AASHTO LRFD**	**AASHTO Standard**	**AASHTO LRFD**
1	37%	41%	40%	44%
2	-19%	-11%	18%	23%
3	11%	16%	20%	25%
4	9%	15%	19%	24%
5	43%	47%	46%	49%
6	7%	12%	22%	27%
7	0%	7%	10%	16%
8	24%	29%	29%	33%
9	44%	47%	34%	39%
10	19%	39%	33%	50%
11	30%	15%	52%	42%

Note: Negative sign indicates that the analytical LLDF was higher than the AASHTO LLDF

4.1.3 Timber Girder Bridges with Timber Deck

Table 12 shows the maximum strain experienced by each timber-timber bridge when each of the testing vehicles is passed over it.

Table 12. Maximum static strain experienced by field-tested timber-timber bridges

	Test Vehicles				
Bridge	**Tractor with one tank (half-filled)**	**Tractor with one tank (empty)**	**TerraGator with single front wheel**	**Tractor Grain Wagon**	**Semi-Truck**
1	321	197	326	242	331
2	347	213	364	246	389
3	507	311	471	371	519

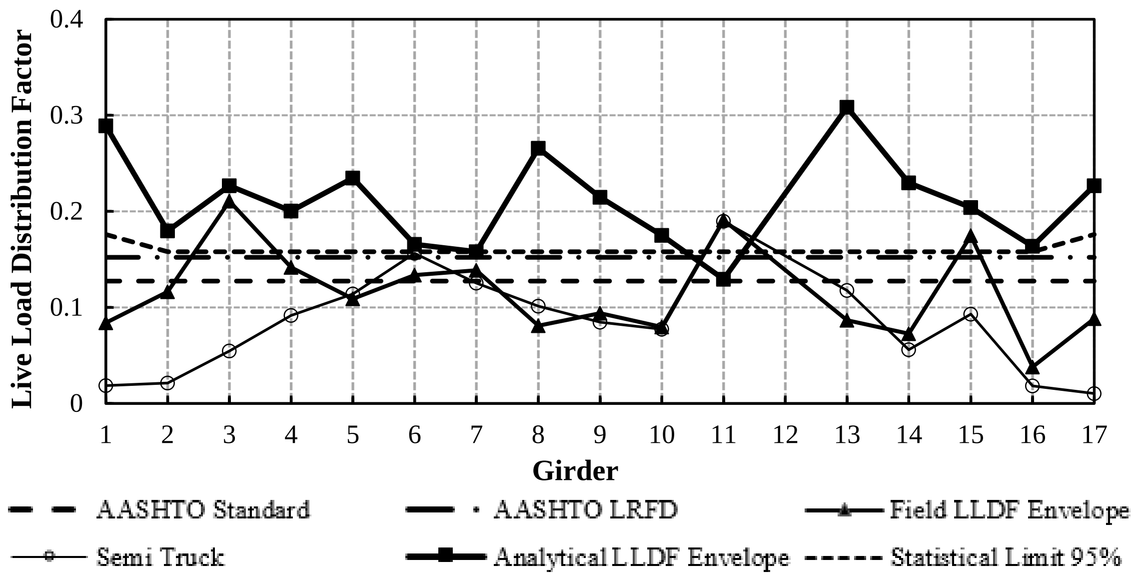
Note: The units of the strain values shown above are microstrain (με)

It was observed that the semi-truck caused the maximum strain in girders compared to all testing vehicles in Bridges 1 and 3. For Bridge 2, the TerraGator with a single front axle caused the maximum strain in the girders.

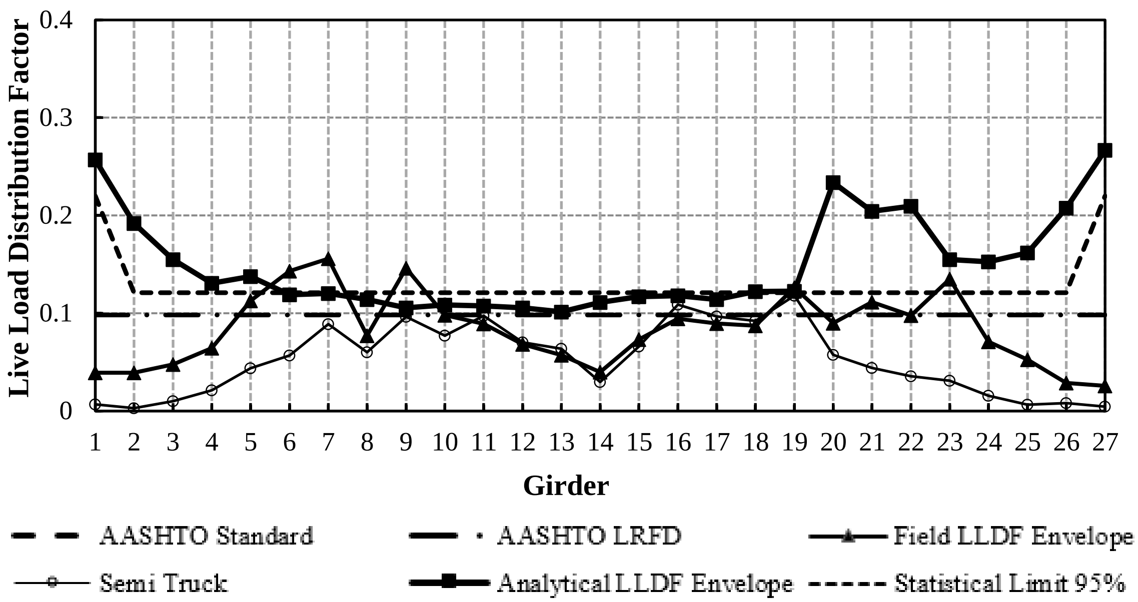
Figure 17 shows the LLDFs of three timber-timber bridges. Figure 17(a) for Bridge 1 shows that the analytical LLDFs for most girders are larger than those from the AASHTO specifications, except for G11. During the model calibration process, girder G12 was acting more like a stiffener to girder G11 than an individual girder due to the very narrow girder spacing between girders G11 and G12. The same was observed from the field testing data of girder G12. Therefore, a separate cross-section was considered for girder G11, combining the effect of girder G12. This resulted in a wider girder spacing between adjacent girders G11 and G13, explaining why the LLDF in G11 is lower and G13 is higher than the other girders. In general, the analytical values are much higher when compared to the AASHTO standard and LRFD limits. The field LLDF envelope has values larger than those of the semi-truck for most of the girders. The statistical limit for the exterior girders of Bridge 1 was 38% and 20% greater compared to the AASHTO standard and LRFD Specifications-compliant LLDFs, respectively. For interior girders, the statistical limit was greater by 23% and 7% than AASHTO specifications.

Figure 17(b) shows Bridge 2, which carries two way traffic (bridge width > 20ft), and has analytical LLDF envelope values larger than AASHTO specifications for all the girders, although the envelope for the central girders G9-G14 is close to AASHTO values. The AASHTO standard and LRFD provided a 120% smaller value relative to the statistical exterior girder limit and 20% smaller value than that of the statistical interior girder limit. Field LLDF values were larger than those of the semi-truck for most of the girders. Figure 17(c) for Bridge 3 shows that the analytical LLDFs for most of the interior girders G5 to G14 were lower than the AASHTO values.

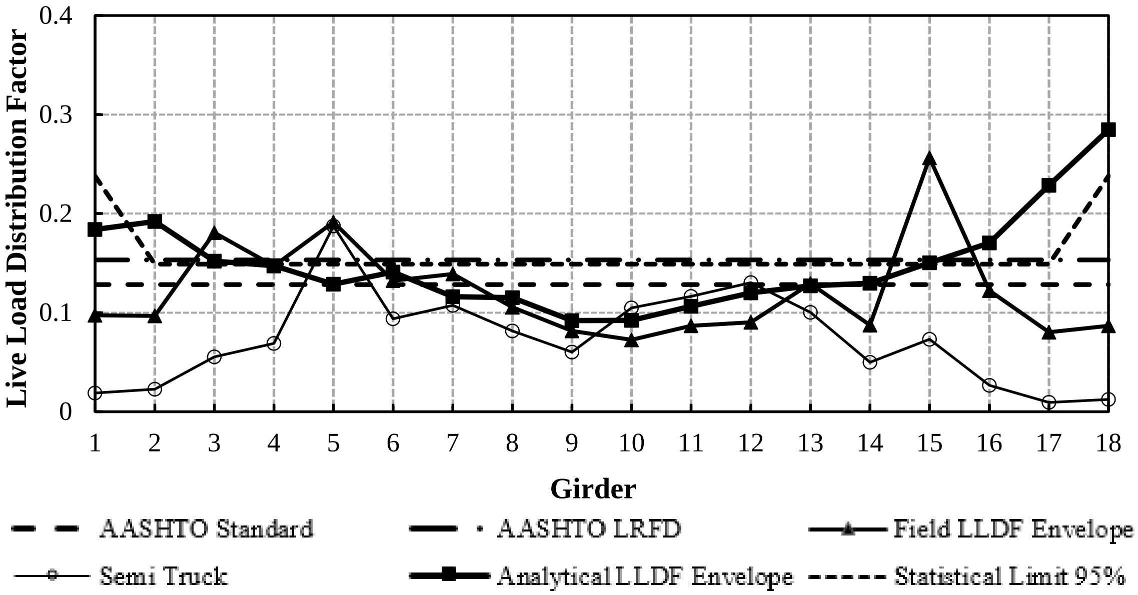
The analytical LLDFs for all the girders of the three timber-timber bridges are summarized in Table 13, along with both AASHTO values.



(a) Bridge 1



(b) Bridge 2



(c) Bridge 3

Figure 17(a-c). LLDFs for field-tested timber-timber bridges

Table 13. Comparison of analytical and AASHTO-specified LLDFs for field-tested timber-timber bridges

	Analytical LLDFs for Girders																											Statistical Limit		AASHTO Codes	
Bridge	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12	G13	G14	G15	G16	G17	G18	G19	G20	G21	G22	G23	G24	G25	G26	G27	Interior Girders	Exterior Girders	LRFD	Standard
1	0.29	0.18	0.23	0.20	0.23	0.17	0.16	0.27	0.21	0.17	0.13	0.21	0.31	0.23	0.20	0.16	0.23											0.16	0.18	0.15	0.13
2	0.26	0.19	0.15	0.13	0.14	0.12	0.12	0.11	0.11	0.11	0.11	0.11	0.10	0.11	0.12	0.12	0.11	0.12	0.12	0.23	0.20	0.21	0.15	0.15	0.16	0.21	0.27	0.12	0.22	0.10	0.10
3	0.18	0.19	0.15	0.15	0.13	0.14	0.12	0.12	0.09	0.09	0.11	0.12	0.13	0.13	0.15	0.17	0.23	0.28										0.16	0.24	0.15	0.13

	Analytical LLDFs for Girders																	
Bridge	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12	G13	G14	G15	G16	G17	G18
1	**0.29**	**0.18**	**0.23**	**0.2**	**0.23**	**0.17**	**0.16**	**0.27**	**0.21**	**0.17**	0.13	**0.21**	**0.31**	**0.23**	**0.2**	**0.16**	**0.23**	
2	**0.26**	**0.19**	**0.15**	**0.13**	**0.14**	**0.12**	**0.12**	**0.11**	**0.11**	**0.11**	**0.11**	**0.11**	**0.1**	**0.11**	**0.12**	**0.12**	**0.11**	**0.12**
3	**0.18**	**0.19**	**0.15**	**0.15**	0.13	**0.14**	0.12	0.12	0.09	0.09	0.11	0.12	0.13	0.13	**0.15**	**0.17**	**0.23**	**0.28**

Analytical LLDFs for Girders									Statistical Limit		AASHTO Codes	
G19	G20	G21	G22	G23	G24	G25	G26	G27	Interior Girders	Exterior Girders	LRFD	Standard
									0.16	**0.18**	0.15	0.13
0.12	**0.23**	**0.2**	**0.21**	**0.15**	**0.15**	**0.16**	**0.21**	**0.27**	**0.12**	**0.22**	0.1	0.1
									0.16	**0.24**	0.15	0.13

Note: Highlighted values in the table indicate that analytical LLDFs were greater than the AASHTO-specified LLDFs in that case

The probability distribution for these interior girders shows that AASHTO standard and LRFD provided 85% and 57% lower values respectively, and 21% and 2% lower values relative to the statistical exterior girder limit. Similar to Bridges 1 and 2, the field LLDFs of the five-axle semi-truck were lower than the field LLDFs resulting from farm vehicles for most of the girders. The percent differences, AASHTO values, and statistical limits are summarized in Table 14 for the three bridges.

Table 14. Percent difference between AASHTO-specified LLDFs and statistical limits for field-tested timber-timber bridges

	Exterior Girder LLDF		Interior Girder LLDF	
Bridge	AASHTO Standard	AASHTO LRFD	AASHTO Standard	AASHTO LRFD
1	-38%	-20%	-23%	-7%
2	-120%	-120%	-20%	-20%
3	-85%	-57%	-21%	-2%

Note: Negative sign indicates that the analytical LLDF was higher than the AASHTO LLDF

4.2 IM Factors for Field Tested Bridges

A set of IM values for each girder in a typical steel-concrete bridge (Bridge 2) are listed in Table 15 for all of the vehicles driven on that bridge at two different speeds.

Table 15. IM for each girder at different speeds (steel-concrete Bridge 2)

#76891		IM (%)											
Vehicle Type	Speed	G1	G2	G3	G4	G5	G6	G7	G8	G9	G10	G11	G12
TerraGator	10 mph	17	14	13	11	5	9	12	6	9	10	11	13
	20 mph	29	23	18	21	9	12	16	18	10	12	13	17
Tractor with grain wagon	10 mph	0	8	6	0	0	2	11	0	0	9	9	11
	20 mph	6	18	23	18	57	33	27	29	33	22	23	24
Five-axle semi	10 mph	0	0	1	0	0	2	2	0	4	6	6	5
	20 mph	0	0	0	0	0	0	0	0	0	2	3	13
Tractor with one tank	10 mph	0	5	5	4	0	7	9	3	4	4	16	19
	20 mph	0	0	0	0	1	0	1	0	3	3	4	11
Tractor with two tanks	10 mph	13	16	17	0	0	17	16	1	0	19	15	15
	20 mph	52	55	52	29	24	50	56	28	13	49	47	46

The maximum IM for all the girders in the steel-concrete bridges are listed in Table 16 and Table 17 for interior and exterior girders, respectively.

Table 16. Maximum IM for interior girders for all steel-concrete bridges

	Maximum IM for Interior Girders				
Vehicle Type	**Bridge 1**	**Bridge 2**	**Bridge 3**	**Bridge 4**	**Maximum**
TerraGator	4	23	6	23	23
Tractor with grain wagon	4	57	3	6	57
Five-axle semi	2	6	6	29	29
Tractor with one tank	2	16	16	4	16
Tractor with two tanks	N/A	55	N/A	N/A	55
All	4	57	16	29	57

Table 17: Maximum IM for exterior girders for steel-concrete bridges

	Maximum IM for Exterior Girders				
Vehicle Type	**Bridge 1**	**Bridge 2**	**Bridge 3**	**Bridge 4**	**Maximum**
TerraGator	0	29	7	31	31
Tractor with grain wagon	5	24	3	6	24
Five-axle semi	2	13	21	57	57
Tractor with one tank	7	19	20	10	20
Tractor with two tanks	N/A	52	N/A	N/A	52
All	7	52	21	57	57

The IM for all steel-concrete bridges is summarized in Figure 18 by grouping interior and exterior girders for all the husbandry vehicles into one category.

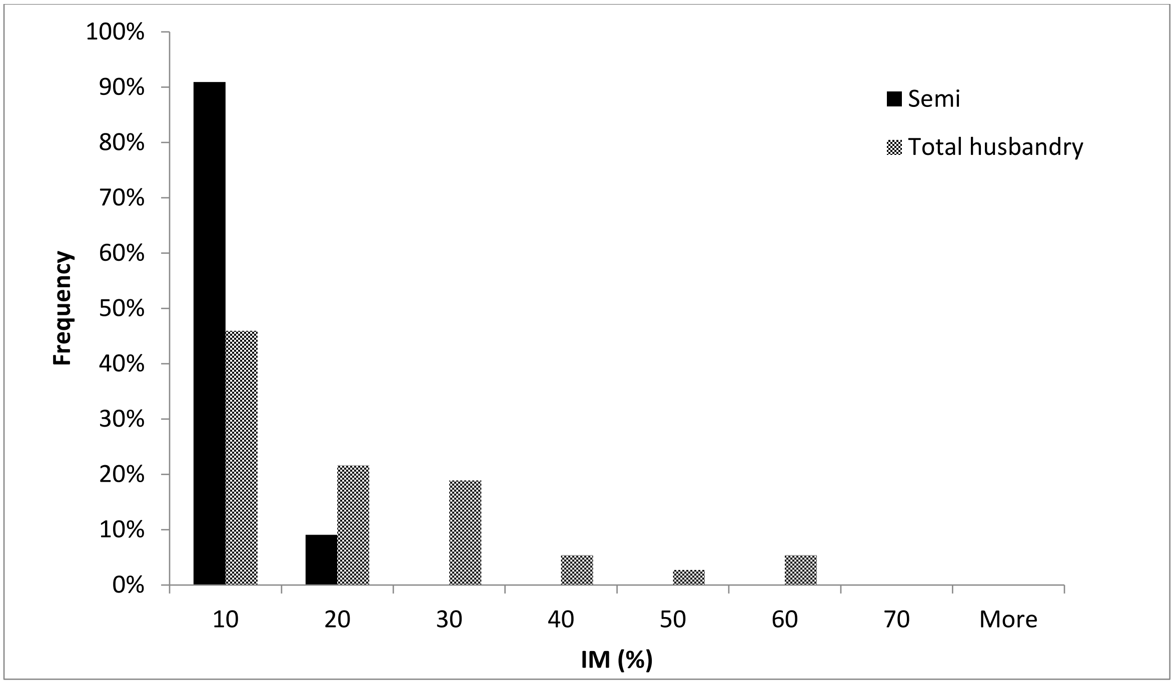


Figure 18. IM frequency for steel/concrete bridges for all vehicles

Figure 19 and Figure 20 are similar histograms for the other two bridge types, steel-timber and timber-timber, respectively.

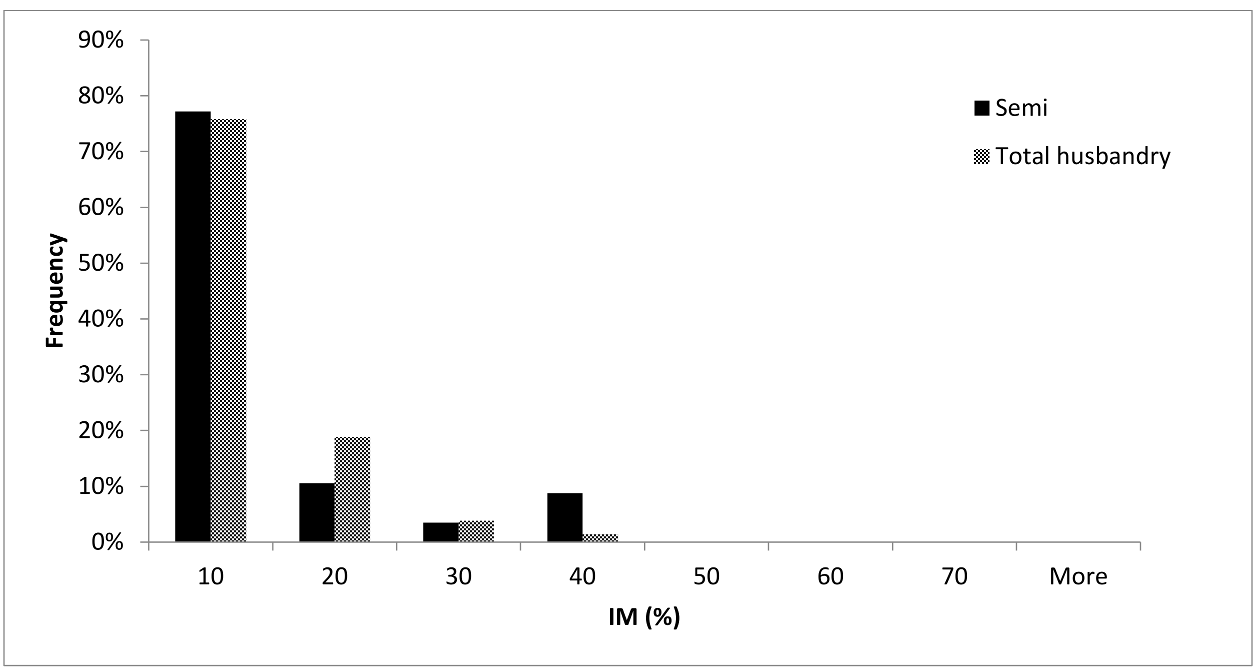


Figure 19. IM frequency for all girders and all vehicles for steel-timber bridges

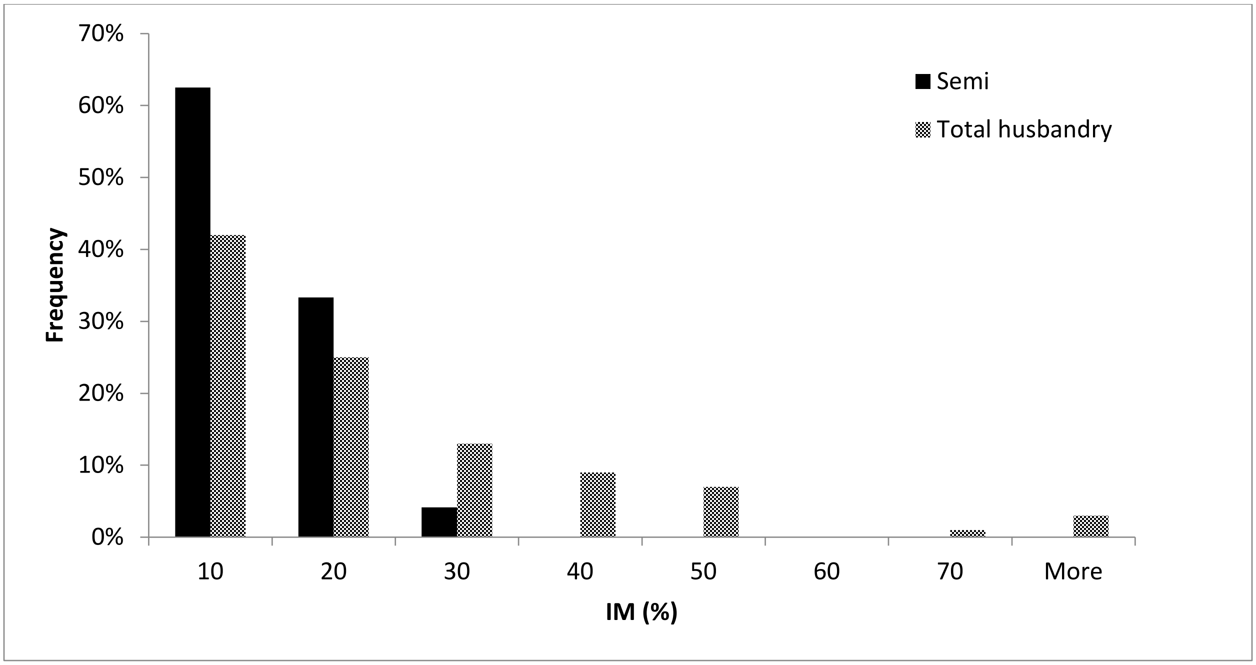


Figure 20. IM frequency for all girders and all vehicles for timber-timber bridges

The IM frequency for all bridge types are grouped into Figure 21.

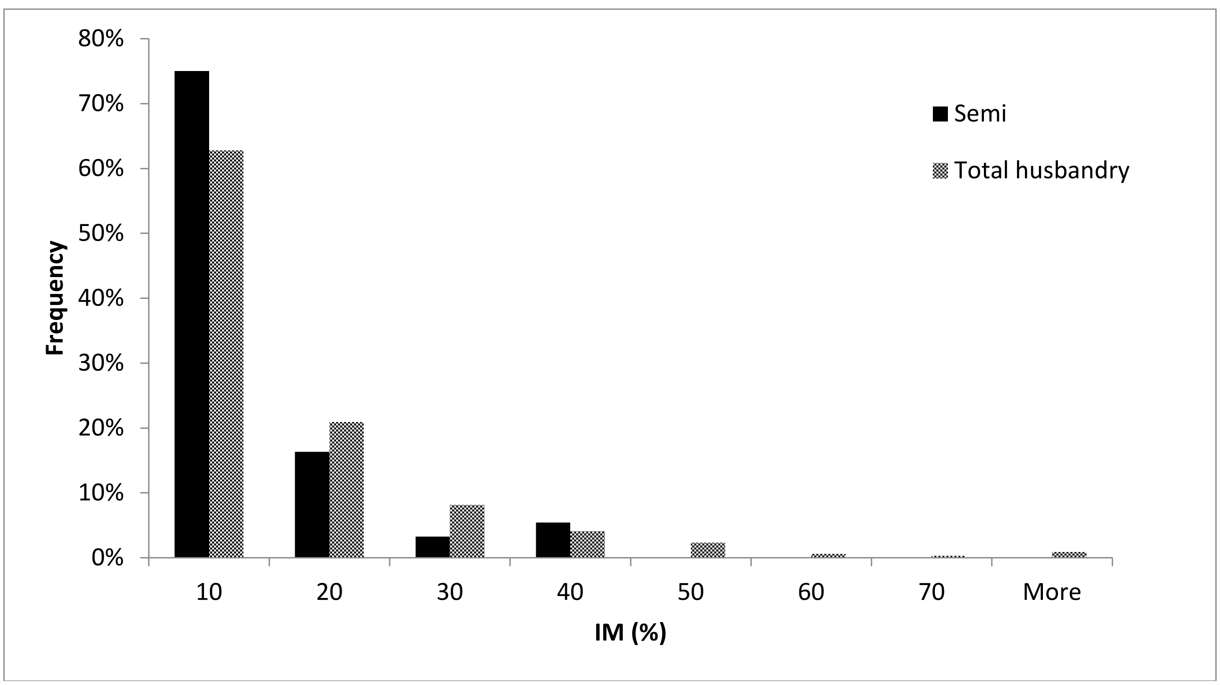


Figure 21. IM frequency for all bridge types and all vehicles

Recognizing that the above results are for a limited number of tests with a limited number of vehicles and a limited number of each bridge type, it is probably premature to select an IM for all husbandry vehicles for all bridge types. However, given this limitation, one could reasonably select IM = 66% for AASHTO LRFD specifications and IM = 100/(L+125) ≤ 0.6 for AASHTO standard specifications, which would be appear to be conservative until more data are obtained.

4.3 Empirical Equations and Skew Correction Factors from the Parametric Study

Following the approach outlined in Chapter 3, the results from the parametric study of 151 in-service bridges are presented in this section. Results for each bridge type include:

Two sets of modified AASHTO code type equations for non-skewed bridges considering the effect of the following:

1. 121 farm vehicles with actual gauge widths
2. Single-axles with varying gauge widths from 5 ft to 12 ft
3. Skew correction factors for skewed bridges

The Bridge Number on the horizontal axis in each of the graphs presented in the following sections refers to the tables in Volume III – Appendix E. Bridge Inventory. All the non-skewed bridges and skewed bridges are tabulated in ascending order of their girder spacing and skew angle, respectively. For non-skewed bridges, the figures presented in this section include graphs showing AASHTO LLDFs (standard and LRFD), analytical LLDFs obtained from the parametric study and predicted LLDFs from the empirical equations. The maximum and minimum limits for bridge parameters with the use of the empirical equations presented in this section are given in Table 18. For skewed bridges, the figures include graphs showing the variation of skew correction factor against skew angle.

Table 18. Limits of husbandry LLDF equations

		Parameter Ranges					
		Span length (ft)		Girder Spacing (ft)		Skew angle (deg)	
Bridge Type		**Min**	**Max**	**Min**	**Max**	**Min**	**Max**
Steel-Concrete Bridges	One Way	26	50	2.0	6.4	0°	0°
	Multiple	30	132	1.7	10.0	0°	6°
	Skewed	28	104	1.8	9.7	20°	55°
Steel-Timber Bridges	One Way	19	61	1.5	5.0	0°	0°
	Multiple	20	59	1.5	4.5	0°	0°
	Skewed	21	49	2.3	2.8	15°	45°
Timber-Timber Bridges	One Way	16	58	0.8	2.2	0°	0°
	Multiple	15	24	0.8	2.1	0°	0°
	Skewed	17	26	0.9	2.5	10°	47°

4.3.1 Steel Girder Bridges with Concrete Deck

A total of 45 in-service steel-concrete bridges including: 5 one-way traffic lane, 24 multiple way traffic lane, and 16 skewed bridges were considered for the parametric study. The graphs and equations are presented below.

4.3.1.1 Modified AASHTO Code Equation – Effect of 121 vehicles

4.3.1.1.1 One-Way Traffic Lane Bridges

Figure 22 and Figure 23 present the analytical LLDFs of exterior and interior girders respectively, for single traffic lane steel-concrete bridges for all 121 farm vehicles. The bridge numbers in the graphs refer to Table 1(a) in Volume III – Appendix E. Bridge Inventory.

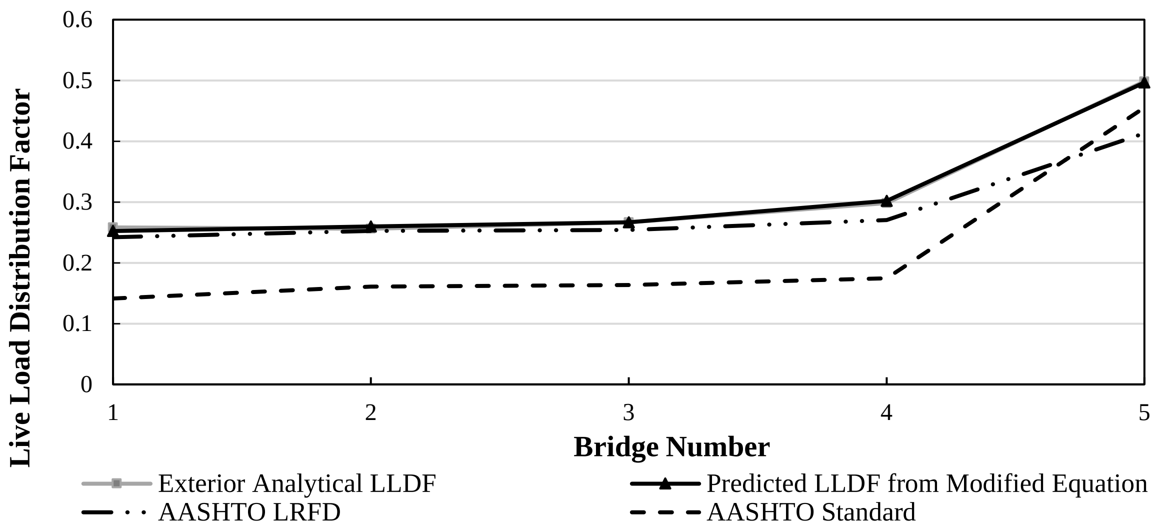


Figure 22. Exterior analytical LLDFs with 121 farm vehicles - one-way traffic lane steel-concrete bridges

Figure 22 shows the exterior LLDFs of the 5 bridges with single traffic lanes. For all bridges, the analytical LLDFs were greater than both the AASHTO LRFD and standard values.

The AASHTO LRFD equation was modified to predict the exterior LLDFs for the 121 farm vehicles and is given as

$$LLDF_{exterior} = \left(\frac{S}{2.5}\right)^{0.35} \left(\frac{S}{L}\right)^{0.41} \left(\frac{K_g}{12Lt_s^3}\right)^{0.16} \tag{18}$$

Figure 22 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.997 and 0.03, respectively.

At this time, this equation has been included for completeness. It is not the intention of the authors that this equation be used for analysis, as it was developed from a linear regression of only 5 data points. As such, this equation is not an accurate representation of the distribution of live load for single-lane steel-concrete bridges. Further testing should be conducted to determine a better fit for this type of bridge.

Figure 23 shows interior LLDFs of the 5 bridges. The analytical LLDFs were greater than AASHTO standard values for Bridges 1 through 4; whereas, they were larger than AASHTO LRFD values for Bridges 2, 3, and 4.

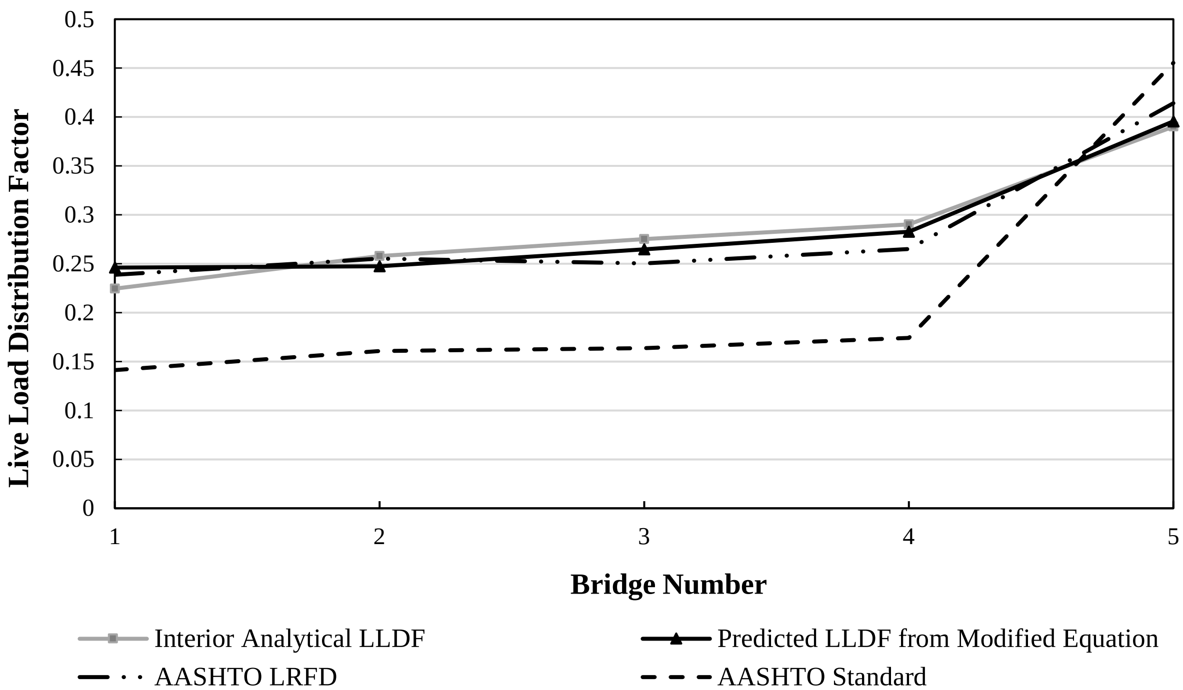


Figure 23. Interior analytical LLDFs with 121 farm vehicles - one traffic lane steel-concrete bridges

The AASHTO LRFD equation was modified to predict the interior LLDFs under 121 farm vehicles and given as follows:

$$LLDF_{interior} = \left(\frac{S}{102.5}\right)^{0.59} \left(\frac{S}{L}\right)^{-0.33} \left(\frac{K_g}{12Lt_s^3}\right)^{-0.02} \tag{19}$$

Figure 23 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.93 and 0.11, respectively.

At this time, this equation has been included for completeness. It is not the intention of the authors that this equation be used for analysis, as it was developed from a linear regression of only 5 data points. As such, this equation is not an accurate representation of the distribution of live load for single-lane steel-concrete bridges. Further testing should be conducted to determine a better fit for these type of bridges.

For both exterior and interior girders, almost all have analytical LLDFs exceeding AASHTO standard values.

4.3.1.1.2 Multiple Traffic Lane Bridges

Figure 24 and Figure 25 present the analytical LLDFs of exterior and interior girders, respectively, for multiple traffic lane steel-concrete bridges for all 121 farm vehicles. The bridge numbers in the graphs refer to Table 1(b) in Volume III – Appendix E. Bridge Inventory.

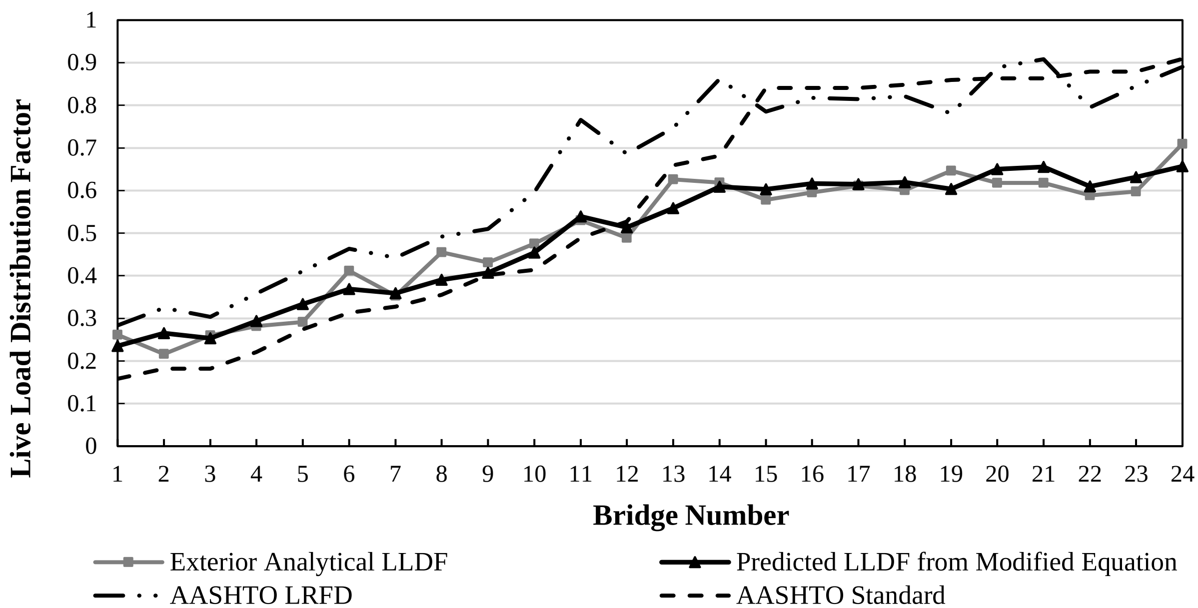


Figure 24. Exterior analytical LLDFs with 121 farm vehicles – multiple traffic lane steel-concrete bridges

Figure 24 shows the exterior LLDFs of the 24 multiple traffic lane bridges. For Bridges 1-12, the analytical LLDFs were greater than AASHTO standard values; whereas, analytical LLDFs in all cases were less than AASHTO LRFD values.

The AASHTO LRFD equation was modified to predict the exterior LLDFs for the 121 farm vehicles and is given as

$$LLDF_{exterior} = \left(\frac{S}{16.2}\right)^{0.49} \left(\frac{S}{L}\right)^{0.10} \left(\frac{K_g}{12Lt_s^3}\right)^{0.05} \quad (20)$$

Figure 24 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.94 and 0.09, respectively.

Figure 25 shows interior LLDFs of 24 bridges. The analytical LLDFs were greater than AASHTO standard values for Bridges 1 through 7; whereas, they were smaller than AASHTO LRFD values in all cases.

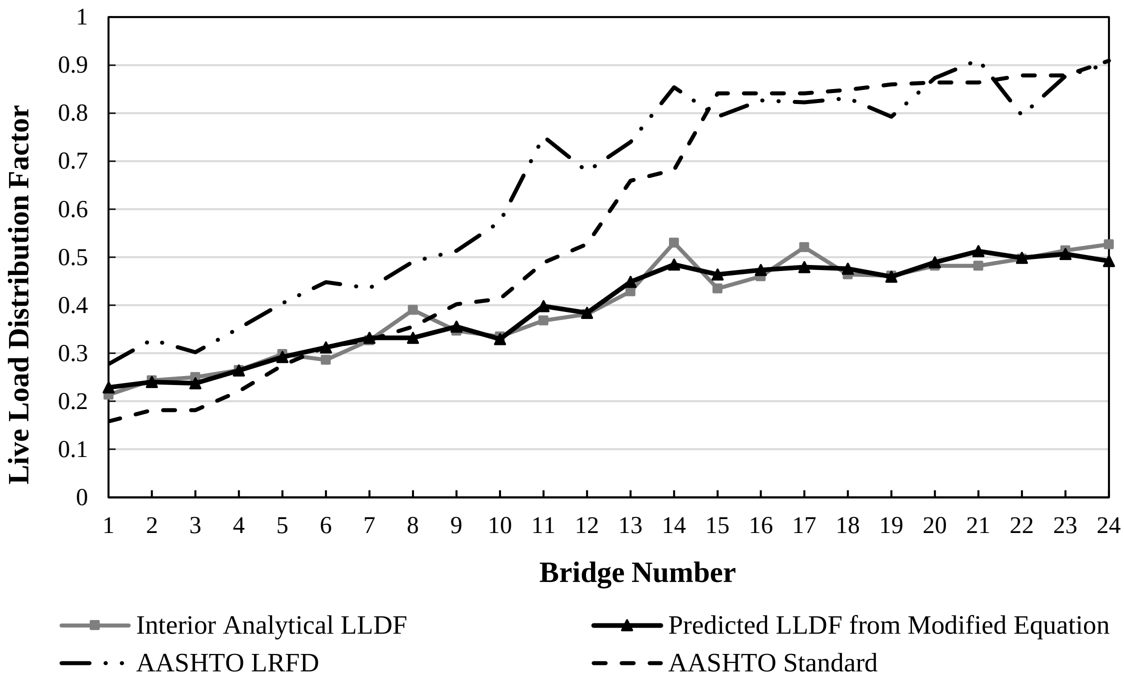


Figure 25. Interior analytical LLDFs with 121 farm vehicles – multiple traffic lane steel-concrete bridges

The AASHTO LRFD equation was modified to predict the interior LLDFs under 121 farm vehicles and given as:

$$LLDF_{interior} = \left(\frac{S}{29.2}\right)^{0.41} \left(\frac{S}{L}\right)^{0.12} \left(\frac{K_g}{12Lt_s^3}\right)^{-0.01} \tag{21}$$

Figure 25 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.95 and 0.06 respectively.

For both exterior and interior girders for bridges with narrow girder spacing, the analytical LLDFs exceed AASHTO standard values; whereas, bridges with wider girder spacing have conservative AASHTO values (standard and LRFD).

4.3.1.2 Modified AASHTO Code Equation – Effect of Gauge Width (varying axle spacing)

4.3.1.2.1 One-Way Traffic Lane Bridges

Figure 26 and Figure 27 present the analytical LLDFs of exterior and interior girders, respectively, for single traffic lane steel-concrete bridges including the effect of gauge width. The bridge numbers in the graphs refer to Table 1(a) in Volume III – Appendix E. Bridge Inventory.

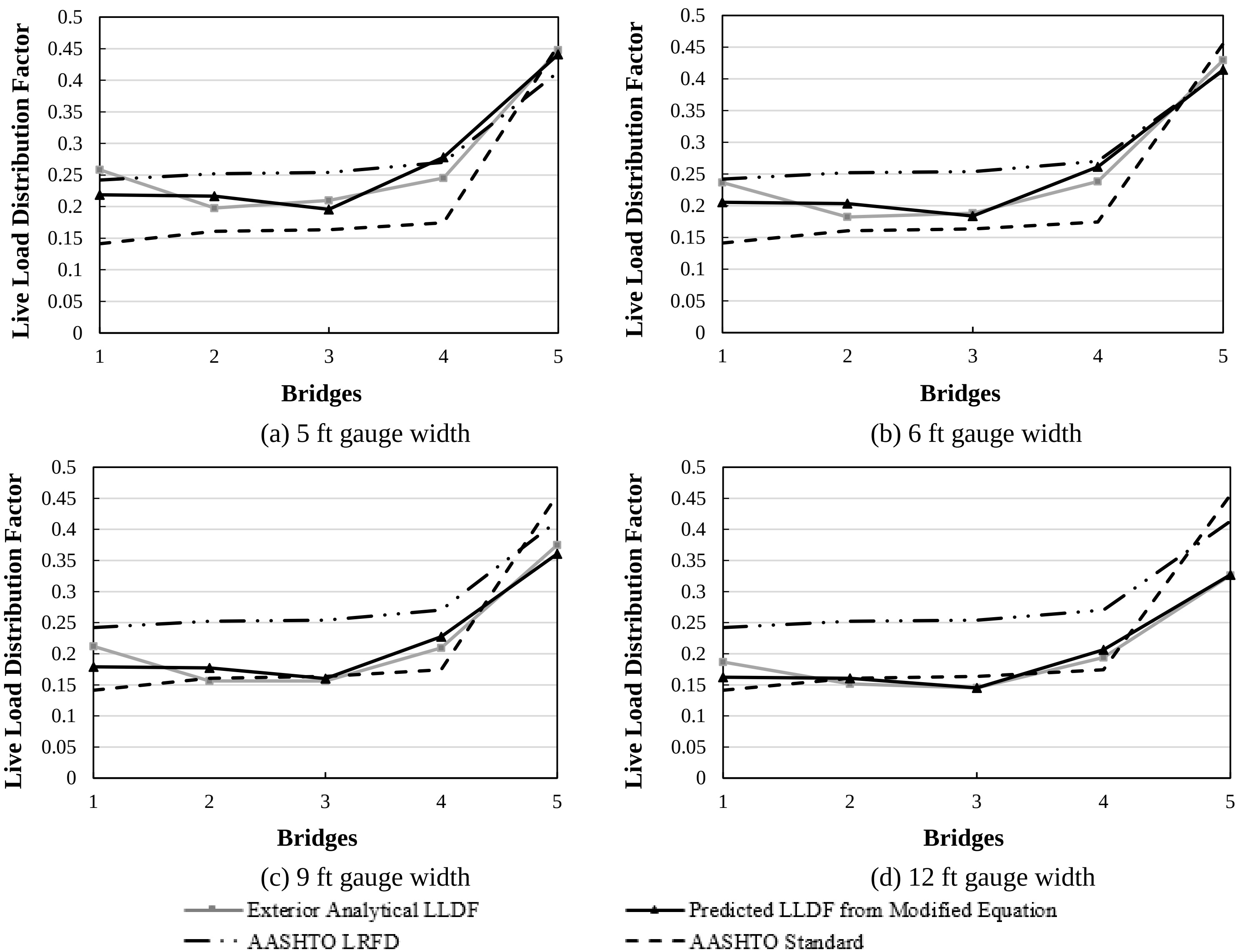
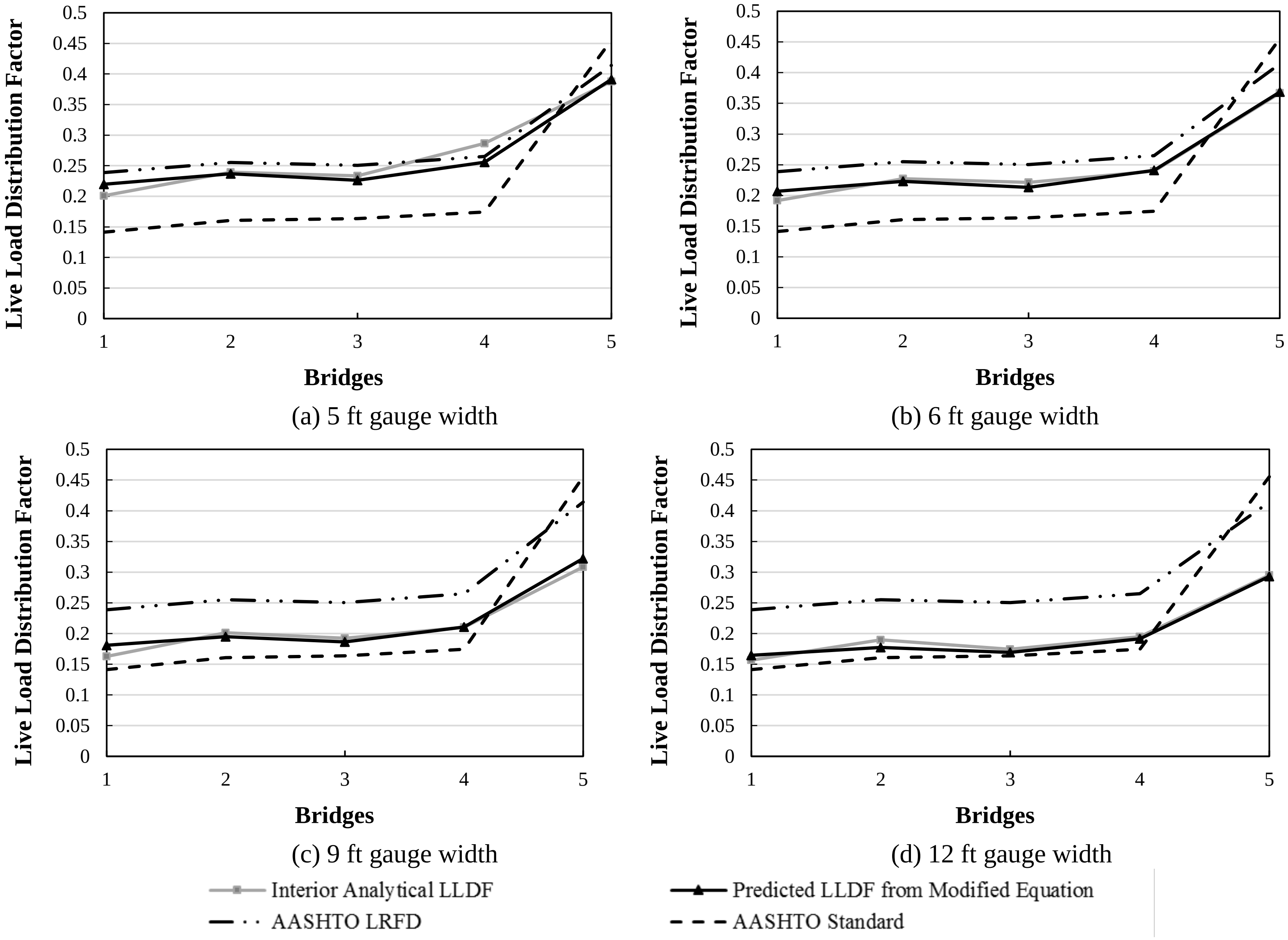


Figure 26(a-d). Exterior analytical LLDFs with variation in gauge width – one-way traffic lane steel-concrete bridges



(a) 5 ft gauge width

(b) 6 ft gauge width

(c) 9 ft gauge width

(d) 12 ft gauge width

Figure 27(a-d). Interior analytical LLDFs with variation in gauge width – one-way traffic lane steel-concrete bridges

Figure 26(a-d) shows the exterior LLDFs of 5 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for most of the bridges are larger than the AASHTO standard LLDFs. For Bridges 2, 3, and 5, the analytical LLDFs with 9 ft and 12 ft gauge width vehicles are smaller than AASHTO standard values.

$$LLDF_{exterior} = \left(\frac{S}{1886}\right)^{-1.22} \left(\frac{S}{L}\right)^{3.30} \left(\frac{K_g}{12Lt_s^3}\right)^{0.93} \left(\frac{6}{G.W.}\right)^{0.34} \quad (22)$$

Figure 26 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.91 and 0.10, respectively.

At this time, this equation has been included for completeness. It is not the intention of the authors that this equation be used for analysis, as it was developed from a linear regression of only 5 data points. As such, this equation is not an accurate representation of the distribution of live load for single-lane steel-concrete bridges. Further testing should be conducted to determine a better fit for this type of bridge.

Figure 27(a-d) shows interior LLDFs of the 5 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The interior analytical LLDFs for most of the bridges are larger than the AASHTO standard LLDFs. For Bridge 5 the analytical LLDFs for all gauge width vehicles are smaller than AASHTO standard values.

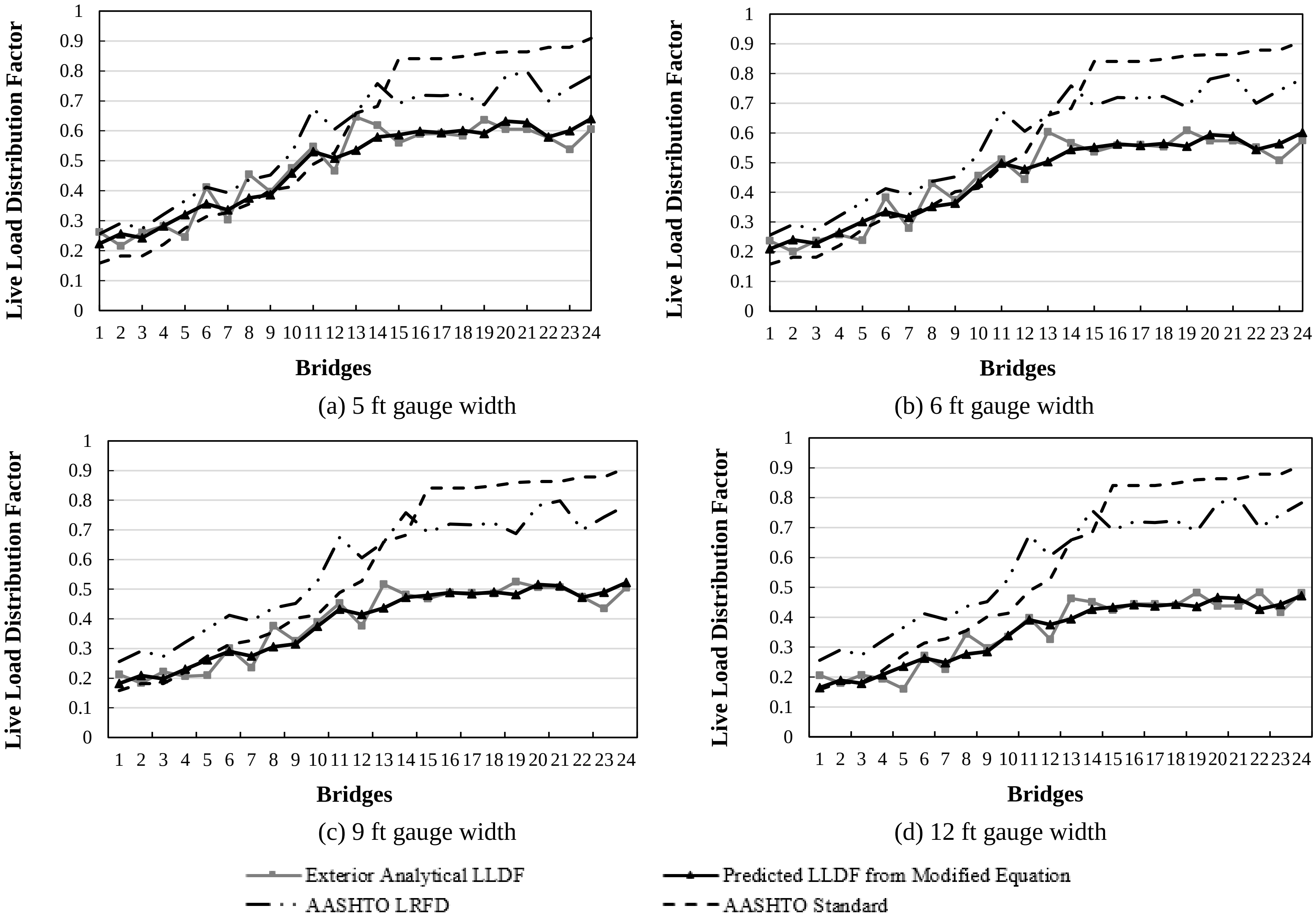
$$LLDF_{interior} = \left(\frac{S}{1.05}\right)^{0.16} \left(\frac{S}{L}\right)^{0.52} \left(\frac{K_g}{12Lt_s^3}\right)^{0.20} \left(\frac{6}{G.W.}\right)^{0.33} \quad (23)$$

Figure 27 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.96 and 0.05, respectively.

At this time, this equation has been included for completeness. It is not the intention of the authors that this equation be used for analysis, as it was developed from a linear regression of only 5 data points. As such, this equation is not an accurate representation of the distribution of live load for single-lane steel-concrete bridges. Further testing should be conducted to determine a better fit for this type of bridge.

4.3.1.2.2 Multiple Traffic Lane Bridges

Figure 28 and **Error! Reference source not found.** present the analytical LLDFs of exterior and interior girders, respectively, for multiple traffic lane steel-concrete bridges including the effect of gauge width. The bridge numbers in the graphs refer to Table 1(b) in Volume III – Appendix E. Bridge Inventory.



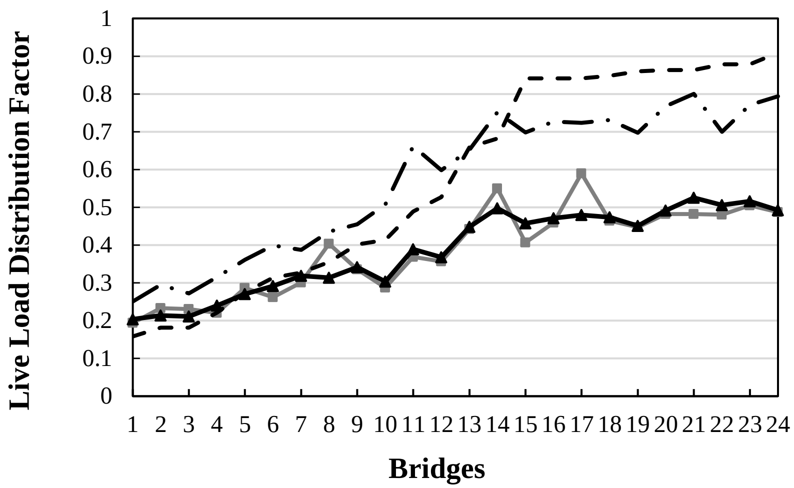
(a) 5 ft gauge width

(b) 6 ft gauge width

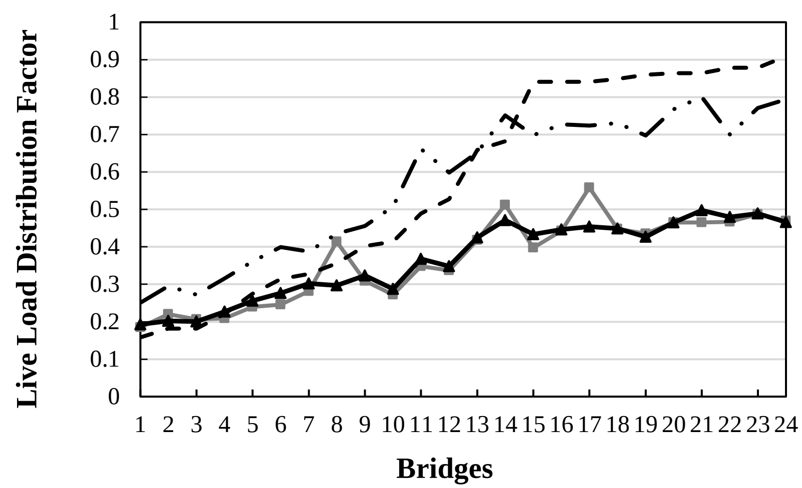
(c) 9 ft gauge width

(d) 12 ft gauge width

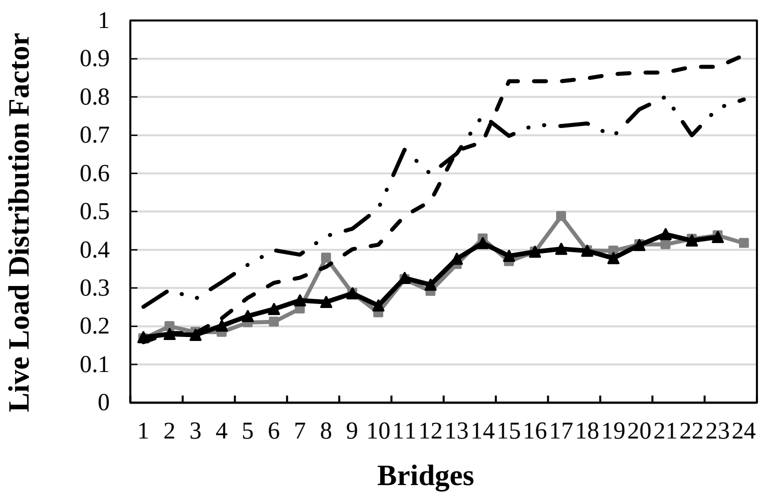
Figure 28(a-d). Exterior analytical LLDFs with variation in gauge width – multiple traffic lane steel-concrete bridge



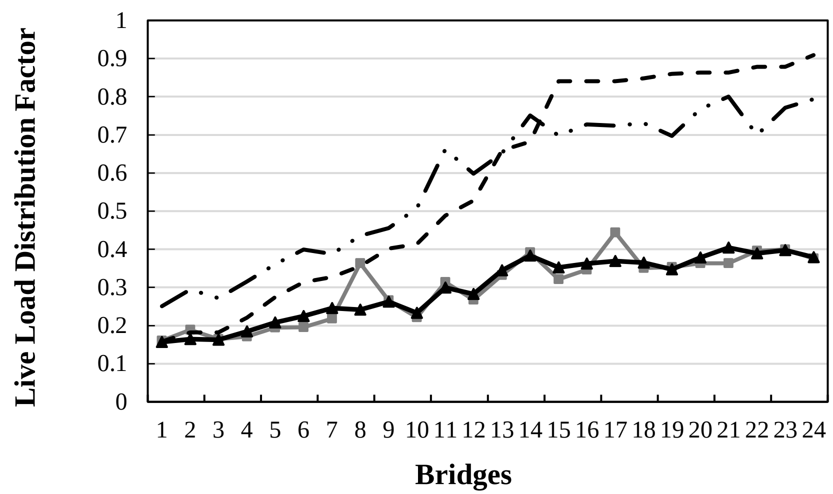
(a) 5 ft gauge width



(b) 6 ft gauge width



(c) 9 ft gauge width



(d) 12 ft gauge width

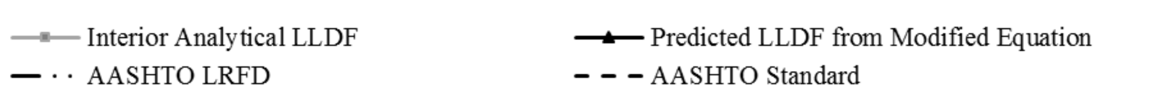


Figure 29(a-d). Interior analytical LLDFs with variation in gauge width – multiple traffic lane steel-concrete bridges

Figure 28(a-d) shows the exterior LLDFs of 24 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for most of the bridges are smaller than both AASHTO Code LLDFs. For Bridges 1, 2, 3, 4, 6, 8, 9, 10, and 11, the analytical LLDFs with 5 ft and 6 ft gauge width vehicles exceeded AASHTO standard values.

The AASHTO LRFD equation was modified to predict the exterior LLDFs under varying gauge width vehicles and given as:

$$LLDF_{exterior} = \left(\frac{S}{22.0}\right)^{0.50} \left(\frac{S}{L}\right)^{0.08} \left(\frac{K_g}{12Lt_s^3}\right)^{0.07} \left(\frac{6}{G.W.}\right)^{0.35} \tag{24}$$

Figure 28 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.91 and 0.11, respectively, and are considered to be acceptable based on previous research.

Error! Reference source not found.(a-d) shows the interior LLDFs of 24 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The interior analytical LLDFs for most of the bridges are smaller than both AASHTO Code LLDFs. For Bridges 1, 2, 3, 5, and 8, the analytical LLDFs with 5 ft, 6 ft, and 7 ft gauge widths vehicles exceeded AASHTO standard values.

The AASHTO LRFD equation was modified to predict the interior LLDFs under varying gauge width vehicles and given as:

$$LLDF_{interior} = \left(\frac{S}{22.5}\right)^{0.46} \left(\frac{S}{L}\right)^{0.17} \left(\frac{K_g}{12Lt_s^3}\right)^{-0.01} \left(\frac{6}{G.W.}\right)^{0.30} \tag{25}$$

The predicted LLDFs from the above equation compared to analytical LLDFs is shown in Figure 25. An accurate prediction of the analytical LLDFs was observed using the above equation. The R square and standard error values for the above equation were 0.91 and 0.10, respectively.

For both exterior and interior girders, it was observed that for bridges with narrow girder spacing, the analytical LLDFs were close to AASHTO standard values; whereas, bridges with wider girder spacing have overly conservative AASHTO values (standard and LRFD). The same trend was observed in the analysis of the 121 farm vehicles (Figure 24 and Figure 25). AASHTO LRFD values were conservative in all cases.

4.3.1.3 Skew Correction Factor

Figure 30 presents the Analytical skew correction factors for skewed bridges for all 121 farm vehicles.

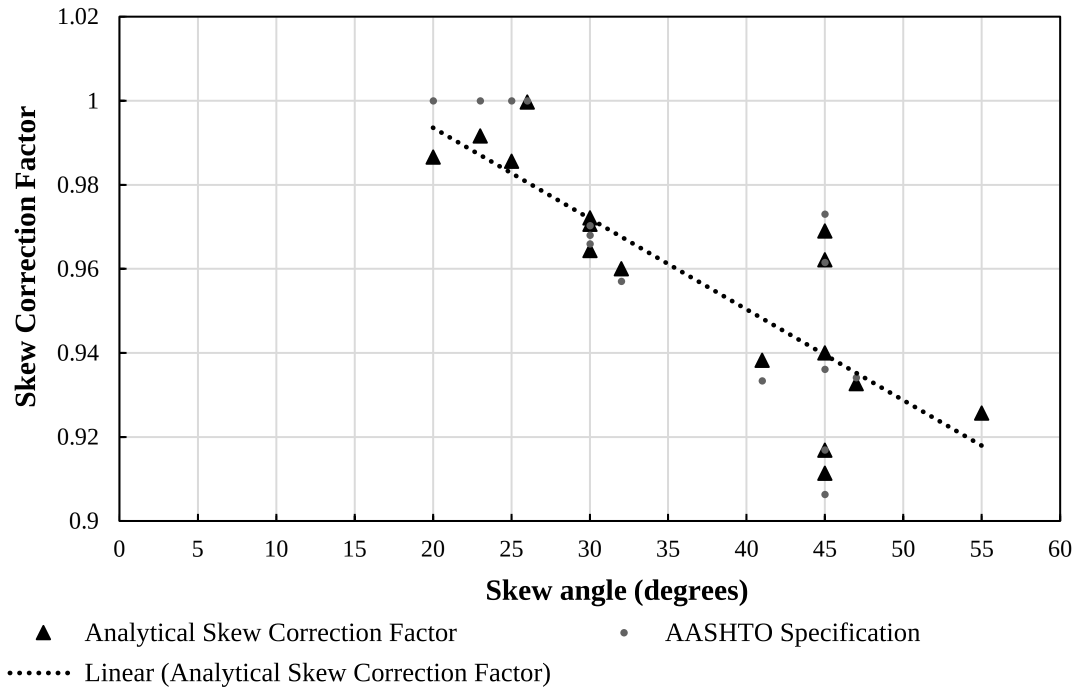


Figure 30. Skew Correction Factors - Steel-Concrete Bridges

The graph refers to bridges summarized in Table 1(c) in Volume III – Appendix E. Bridge Inventory. The skew correction factors determined are close to AASHTO LRFD values provided for steel-concrete bridges. The skew correction factors show a decreasing trend as the skew angle in the bridge increases, thus decreasing the LLDF values for skewed bridges.

4.3.2 Steel Girder Bridges with Timber Decks

A total of 54 in-service steel-timber bridges, including 23 one-way traffic lane, 21 multiple traffic lane and 10 skewed bridges, were considered for the parametric study. The graphs and equations are presented below.

4.3.2.1 Modified AASHTO Code Equation – Effects of 121 Vehicles

4.3.2.1.1 One-Way Traffic Lane Bridges

Figure 31 and Figure 32 present analytical LLDFs of exterior and interior girders, respectively, for 23 one-way traffic lane steel-timber bridges for all 121 farm vehicles. The bridge numbers in the graphs refer to Table 2(a) in Volume III – Appendix E. Bridge Inventory.

Figure 31 shows the exterior LLDFs of the 23 bridges. The analytical LLDFs are similar to AASHTO standard values in most cases; whereas, analytical LLDFs were less than AASHTO LRFD values in most cases.

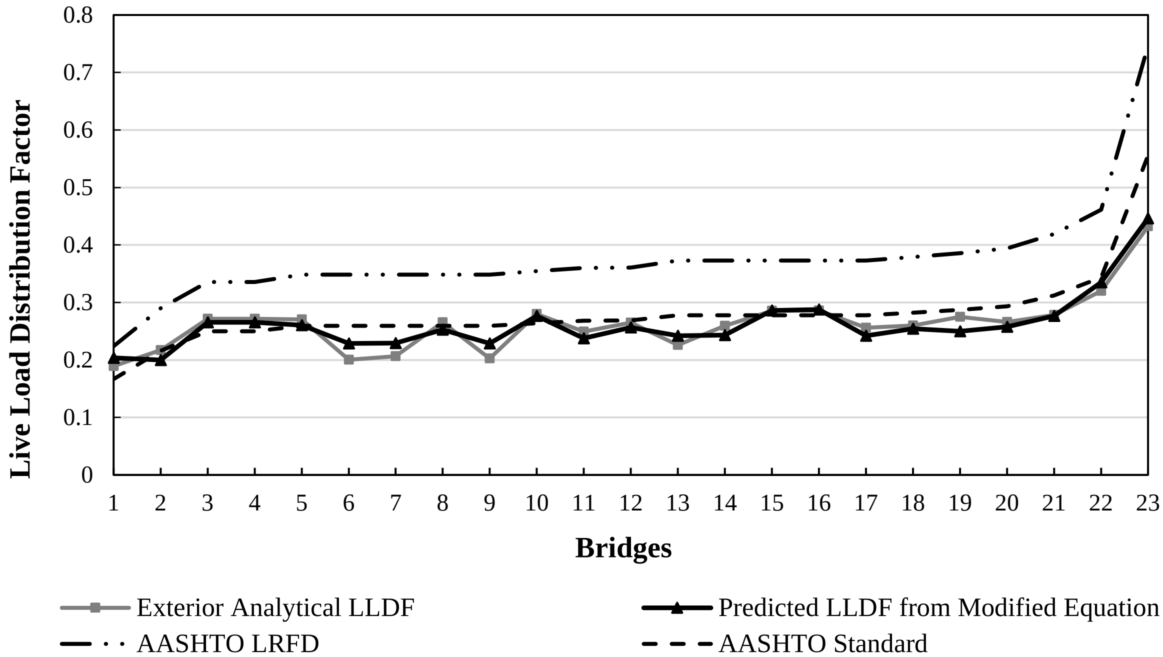


Figure 31. Exterior analytical LLDFs with 121 farm vehicles – one-way traffic lane steel-timber bridges

A modified AASHTO LRFD type equation similar to the one developed for steel-concrete bridges to predict the exterior LLDFs for the 121 farm vehicles is given below:

$$LLDF_{exterior} = \left(\frac{S}{10.1}\right)^{0.60}\left(\frac{S}{L}\right)^{0.18}\left(\frac{K_g}{12Lt_s^3}\right)^{-0.01} \qquad (26)$$

Figure 31 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.87 and 0.07 respectively.

Figure 32 shows interior LLDFs of the 23 bridges. The analytical LLDFs were similar or less than AASHTO values (standard and LRFD) in most cases, except for Bridges 8, 10, and 13 where analytical LLDFs exceeded AASHTO standard values.

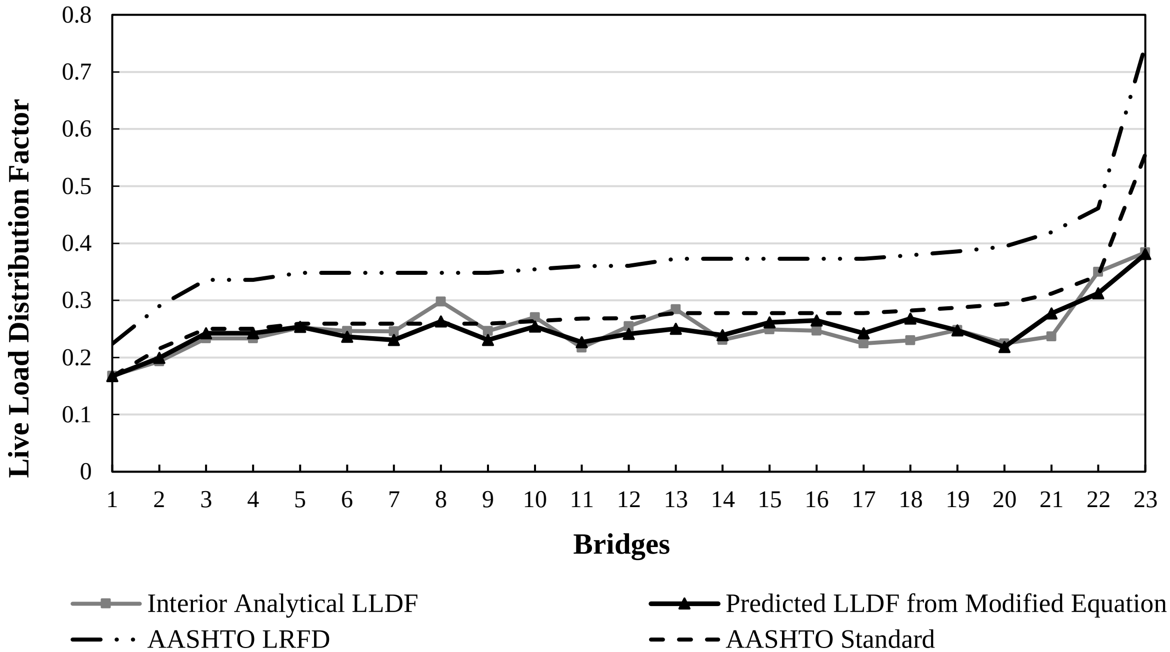


Figure 32. Interior analytical LLDFs with 121 farm vehicles – one-way traffic lane steel-timber bridges

A modified AASHTO LRFD type equation to predict the interior LLDFs for the 121 farm vehicles is given below.

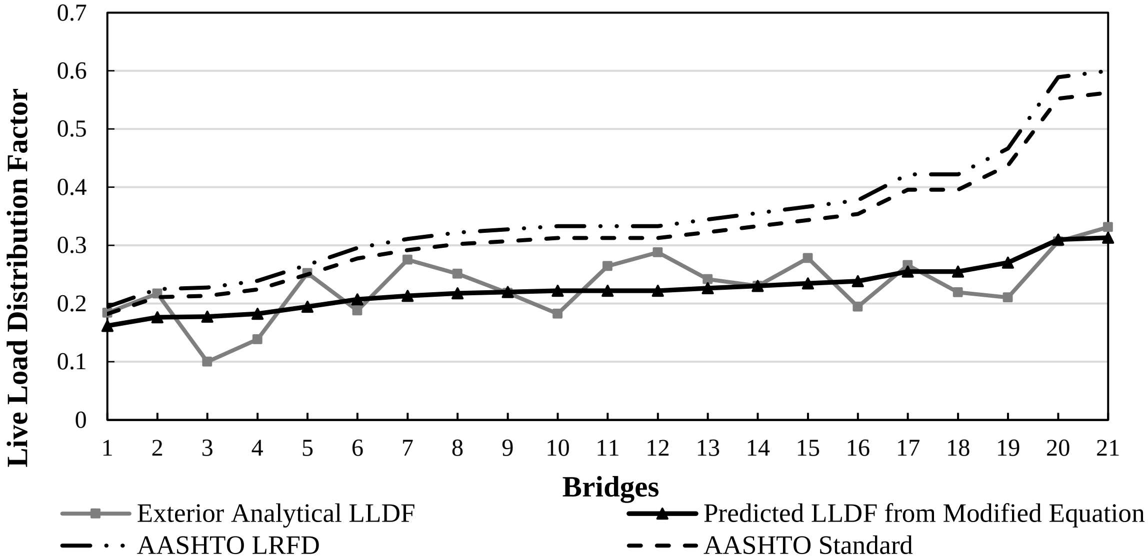
$$LLDF_{interior} = \left(\frac{S}{17.4}\right)^{0.59} \left(\frac{S}{L}\right)^{0.11} \left(\frac{K_g}{12Lt_s^3}\right)^{0.08} \tag{27}$$

Figure 32 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.80 and 0.08 respectively.

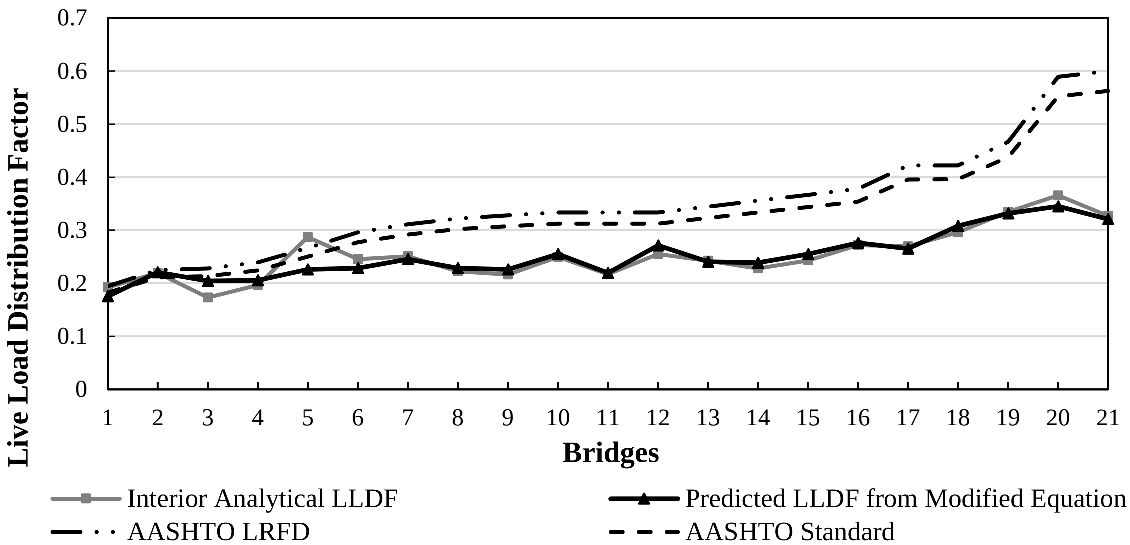
For both exterior and interior girders, it was observed that the analytical LLDFs were similar to AASHTO standard values except in a few cases. ASHTO LRFD values were conservative in all cases.

4.3.2.1.2 Multiple Traffic Lane Bridges

Figure 33 (a-b) presents the analytical LLDFs of exterior and interior girders for 21 multiple traffic lane steel-timber bridges for all 121 farm vehicles. The bridge numbers in the graphs refer to Table 2(b) in Volume III – Appendix E. Bridge Inventory.



(a) Exterior Analytical LLDFs



(b) Interior analytical LLDFs

Figure 33(a-b). Exterior and interior analytical LLDFs with 121 farm vehicles – multiple traffic lane steel-timber bridges

Figure 33(a) shows the exterior LLDFs of the 21 bridges. The analytical LLDFs were less than the AASHTO specifications (standard and LRFD) in all cases. A modified AASHTO equation to predict the exterior LLDFs for the 121 farm vehicles is given below. In this case, the analytical LLDFs depend only on girder spacing among all the bridge geometric parameters.

$$LLDF_{exterior} = \left(\frac{S}{32.6}\right)^{0.59} \tag{28}$$

Figure 33 (a) also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.38 and 0.22 respectively.

Figure 33 (b) shows the interior LLDFs of the 21 bridges. The analytical LLDFs were less than the AASHTO specifications (standard and LRFD) in all cases, except for Bridges 2 and 5. A modified AASHTO LRFD type equation similar to the one developed for steel-concrete bridges to predict the exterior LLDFs for the 121 farm vehicles is given below.

$$LLDF_{interior} = \left(\frac{S}{29.8}\right)^{0.43} \left(\frac{S}{L}\right)^{0.15} \left(\frac{K_g}{12Lt_s^3}\right)^{0.07} \qquad (29)$$

Figure 33 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.83 and 0.08 respectively.

Though only 21 bridges were considered for this study, AASHTO values proved to be conservative for both exterior and interior girders in all cases.

4.3.2.2 Modified AASHTO Code Equation – Effect of Gauge Width (Varying Axle Spacing)

4.3.2.2.1 One-Way Traffic Lane Bridges

Figure 34 and Figure 35 present analytical LLDFs of exterior and interior girders, respectively, for one-way traffic lane steel-timber bridges, including the effect of gauge width. The bridge numbers in the graphs refer to Table 2(a) in Volume III – Appendix E. Bridge Inventory.

Figure 34(a-d) shows the exterior LLDFs of 23 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for all the bridges are smaller than both AASHTO code LLDFs in all the cases. As stated previously, AASHTO specifications provide LLDFs for steel-timber bridges based on the S-over rule. The modified S-over AASHTO equation to predict the exterior LLDFs under the varying gauge width vehicles is given as:

$$LLDF_{exterior} = \left(\frac{S}{16.2}\right)^{0.88} \left(\frac{6}{G.W.}\right)^{0.23} \qquad (30)$$

Figure 34 also shows the predicted LLDFs from the above equation and the comparison with the analytical LLDFs. The R square and standard error values for the above equation were 0.69 and 0.12, respectively.

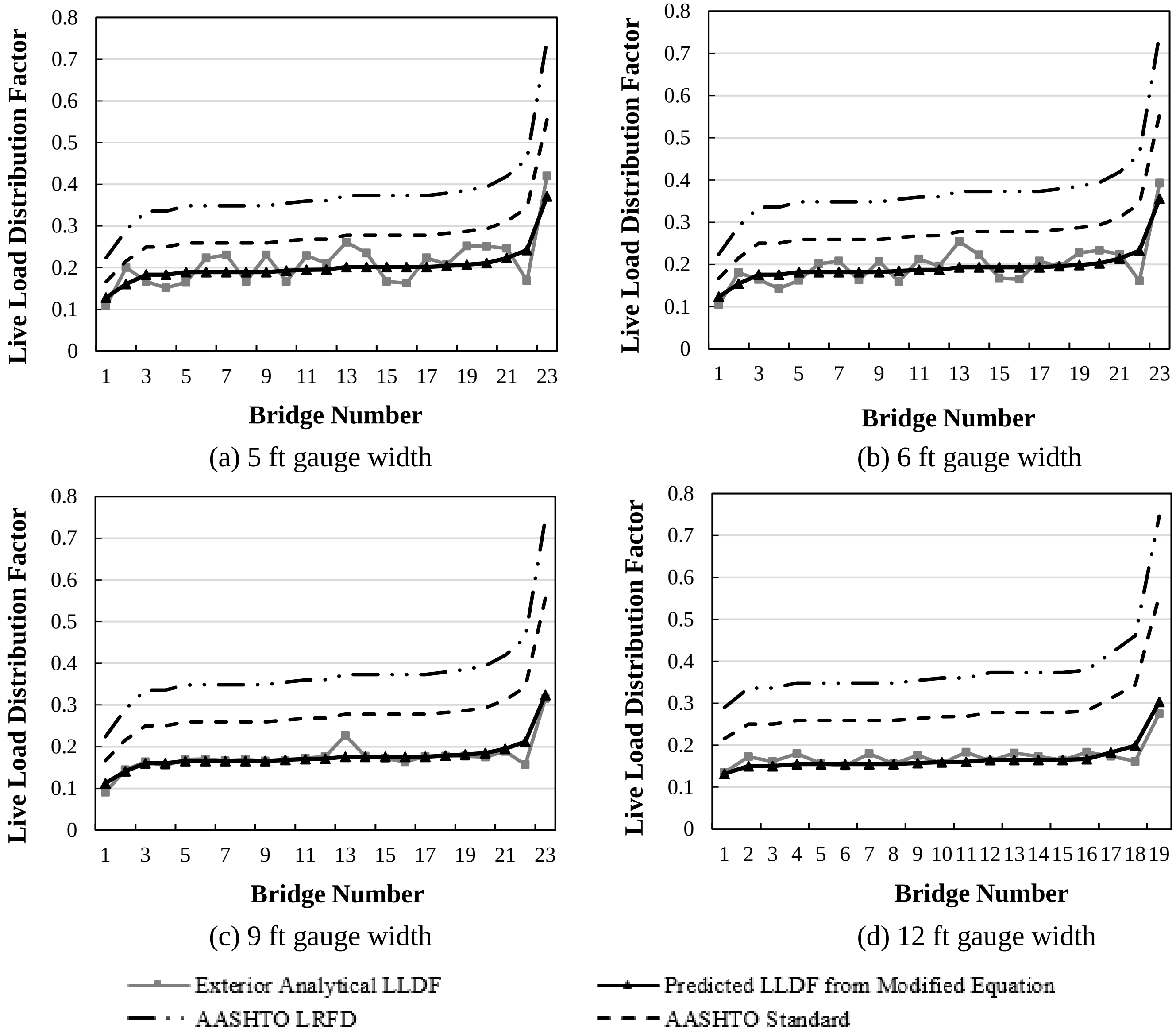


Figure 34(a-d). Exterior analytical LLDFs with variation in gauge width – one-way traffic lane steel-timber bridges

Figure 35(a-d) shows the interior LLDFs of 23 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The interior analytical LLDFs for most of the bridges are smaller than both AASHTO code LLDFs. For Bridges 3, 4, 5, 10, 15, and 16, the analytical LLDFs exceeded AASHTO standard values with a 5 ft gauge width vehicle.

The AASHTO LRFD equation was modified to predict the interior LLDFs under varying gauge width vehicles and is given as:

$$LLDF_{interior} = \left(\frac{S}{25.4}\right)^{0.62} \left(\frac{6}{G.W.}\right)^{0.27} \tag{31}$$

The predicted LLDFs from the above equation compared to the analytical LLDFs is shown in Figure 35. The R square and standard error values for the above equation were 0.73 and 0.09 respectively.

For both exterior and interior girders, it was observed that analytical LLDFs for most of the bridges were less than AASHTO specifications. The same trend was observed in the analysis of 121 farm vehicles (Figure 31 and Figure 32).

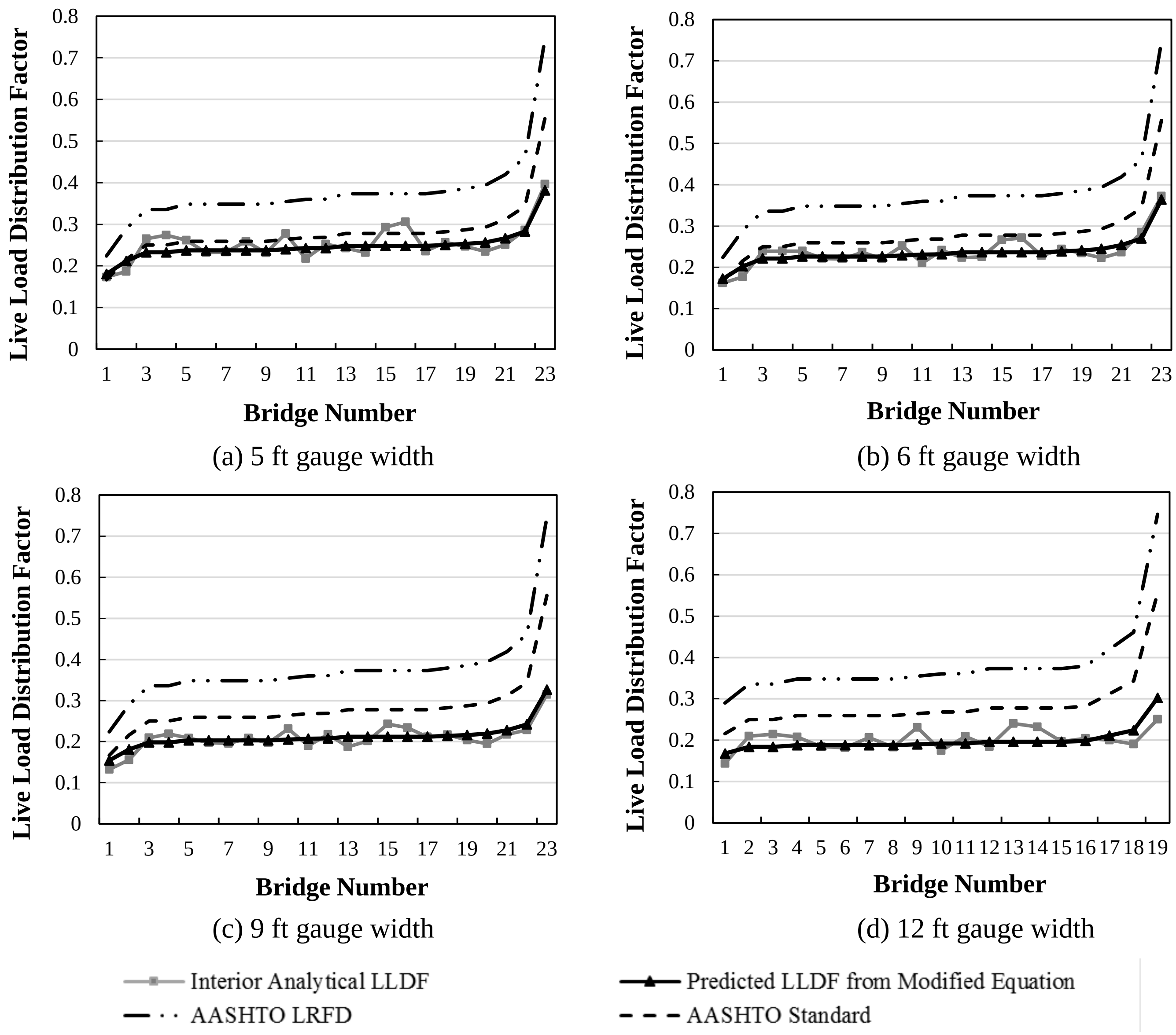


Figure 35(a-d). Interior analytical LLDF with variation in gauge width – one-way traffic lane steel-timber bridges

4.3.2.2.2 Multiple Traffic Lane Bridges

Figure 36 and Figure 37 present analytical LLDFs of exterior and interior girders, respectively, for multiple traffic lane steel-timber bridges including the effect of gauge width. The bridge numbers in the graphs refer to Table 2(b) in Volume III – Appendix E. Bridge Inventory.

Figure 36(a-d) shows the exterior LLDFs of 21 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for all the bridges are smaller than both AASHTO standard and LRFD values in most cases, except for Bridge 1. As stated previously, AASHTO specifications provide LLDFs for steel-timber bridges based on the S-over rule. The modified S-over AASHTO equation to predict the exterior LLDFs under the varying gauge width vehicles is given as:

$$LLDF_{exterior} = \left(\frac{S}{71.9}\right)^{0.49}\left(\frac{6}{G.W.}\right)^{0.10} \tag{32}$$

Figure 36 also shows the predicted LLDFs from the above equation and the comparison with the analytical LLDFs. The R square and standard error values for the above equation were 0.36 and 0.19, respectively.

Figure 37(a-d) shows interior LLDFs of 21 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft. The interior analytical LLDFs for most of the bridges are smaller than both AASHTO code LLDFs. For Bridges 1 and 2, the analytical LLDFs with 5 ft and 6 ft gauge width vehicles exceeded AASHTO standard values. AASHTO LRFD values were conservative in all cases.

The AASHTO LRFD equation was modified to predict the interior LLDFs under varying gauge width vehicles and is given as:

$$LLDF_{interior} = \left(\frac{S}{57.4}\right)^{0.45}\left(\frac{6}{G.W.}\right)^{0.16} \tag{33}$$

The predicted LLDFs from the above equation compared to analytical LLDFs is shown in Figure 37. The R square and standard error values for the above equation were 0.53 and 0.13, respectively.

For both exterior and interior girders, it was observed that analytical LLDFs for most of the bridges were less than AASHTO specifications. The same trend was observed in the analysis of 121 farm vehicles (Figure 33).

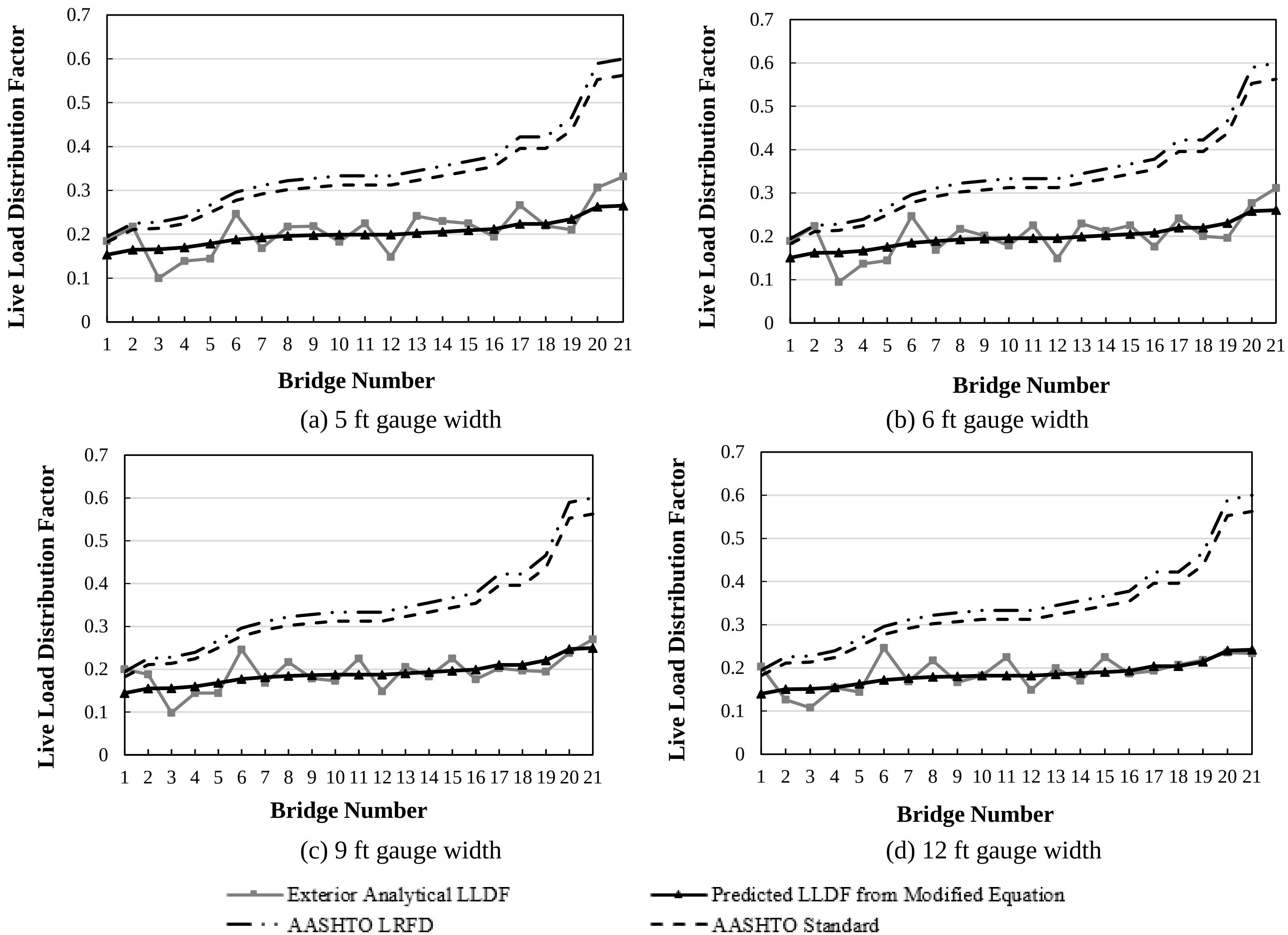
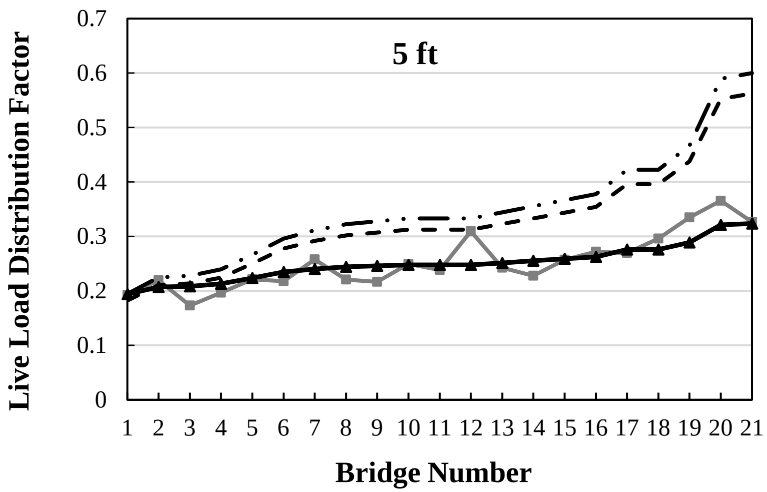
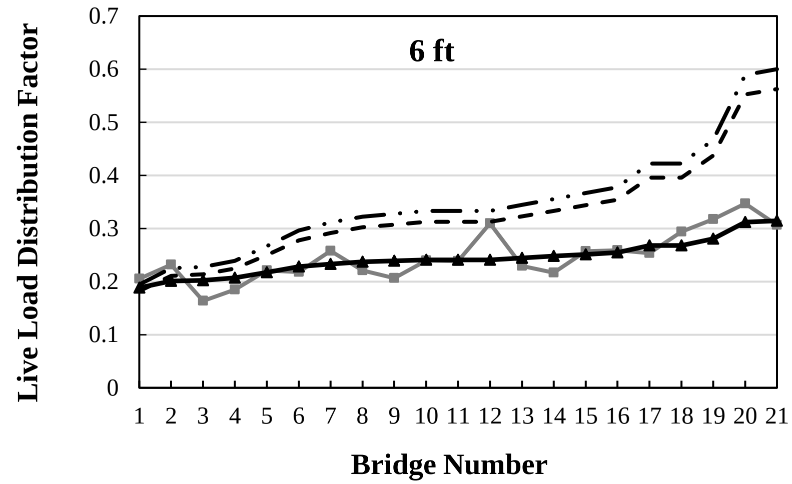


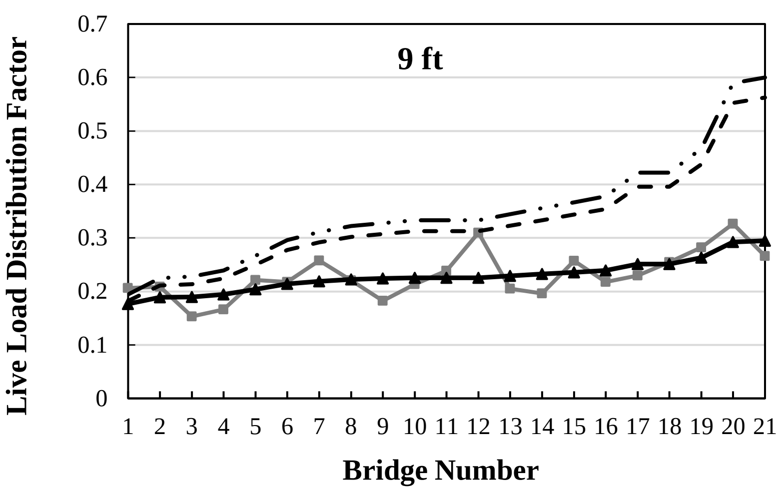
Figure 36(a-d). Exterior analytical LLDF with variation in gauge width – multiple traffic lane steel-timber bridges



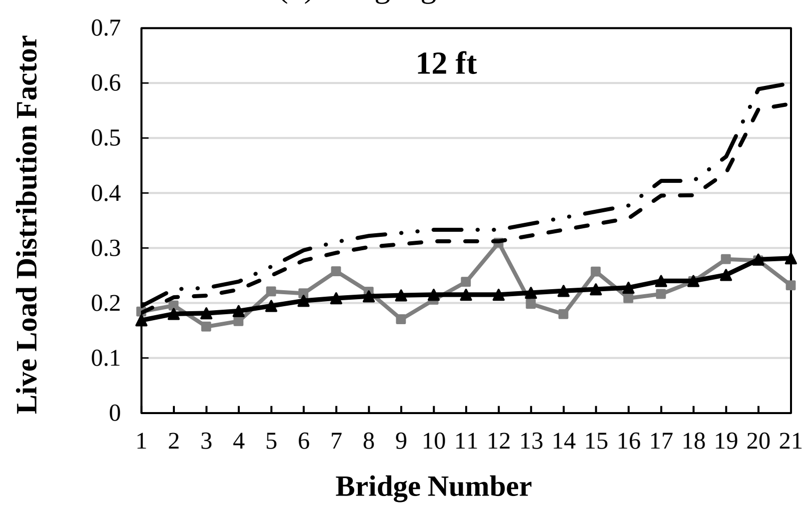
(a) 5 ft gauge width



(b) 6 ft gauge width



(c) 9 ft gauge width



(d) 12 ft gauge width

Interior Analytical LLDF
Predicted LLDF from Modified Equation
AASHTO LRFD
AASHTO Standard

Figure 37(a-d). Interior analytical LLDF with variation in gauge width – multiple traffic lane steel-timber bridges

4.3.2.3 Skew Correction Factor

Figure 38 presents the analytical skew correction factors for skewed steel-timber bridges for all 121 farm vehicles.

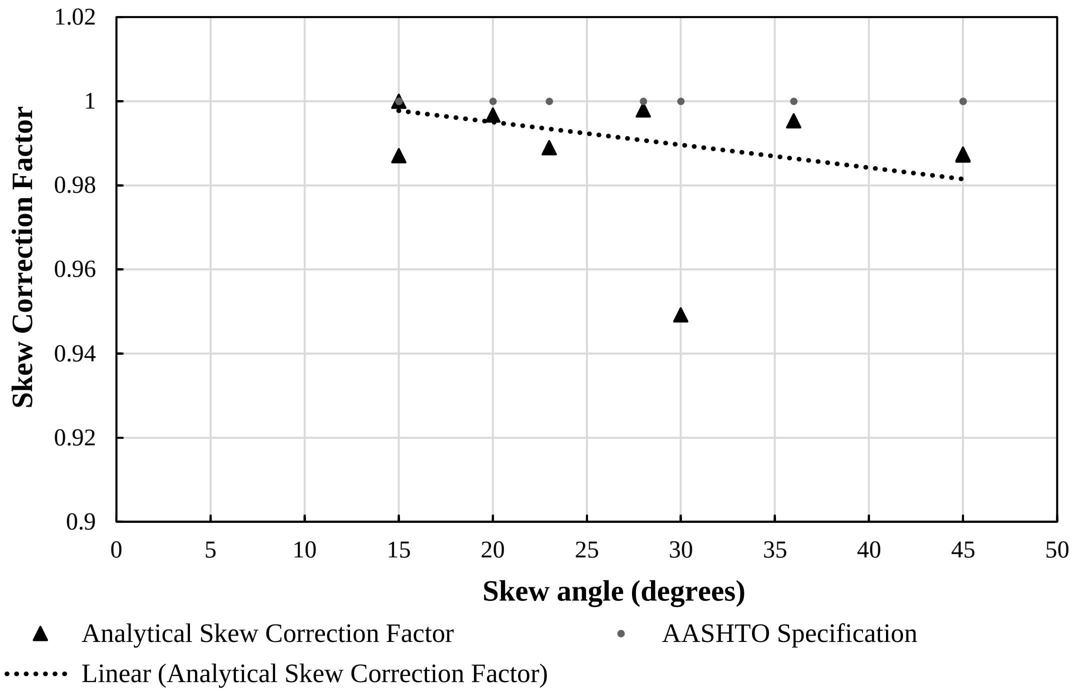


Figure 38. Skew correction factors – steel-timber bridges

The graph refers to bridges summarized in Table 2(c) in Volume III – Appendix E. Bridge Inventory. As stated previously, AASHTO codes do not specify skew correction factors for steel-timber bridges and so were taken as 1.0 as shown in Figure 38. The analytical skew correction factors are constant with variation in skew angle (the skew correction factor at 30 ° skew differs only slightly from the trend).

4.3.3 Timber Girder Bridges with Timber Deck

A total of 52 in-service timber-timber bridges, including 33 one-way traffic lane, 9 multiple traffic lane, and 10 skewed bridges, were considered for the parametric study. The graphs and equations are presented below.

4.3.3.1 Modified AASHTO Code Equation – Effect of 121 Farm Vehicles

4.3.3.1.1 One-Way Traffic Lane Bridges

Figure 39 and Figure 40 present analytical LLDFs of exterior and interior girders, respectively, for 33 one-way traffic lane timber-timber bridges for all 121 farm vehicles. The bridge numbers shown in the graphs refer to Table 3(a) in Volume III – Appendix E. Bridge Inventory.

Figure 39 shows the exterior LLDFs of the 33 bridges. The analytical LLDFs are similar or less than AASHTO standard values in most cases, except for Bridges 15, 16, 17, and 18. AASHTO LRFD values are conservative in all cases, except for Bridge 15.

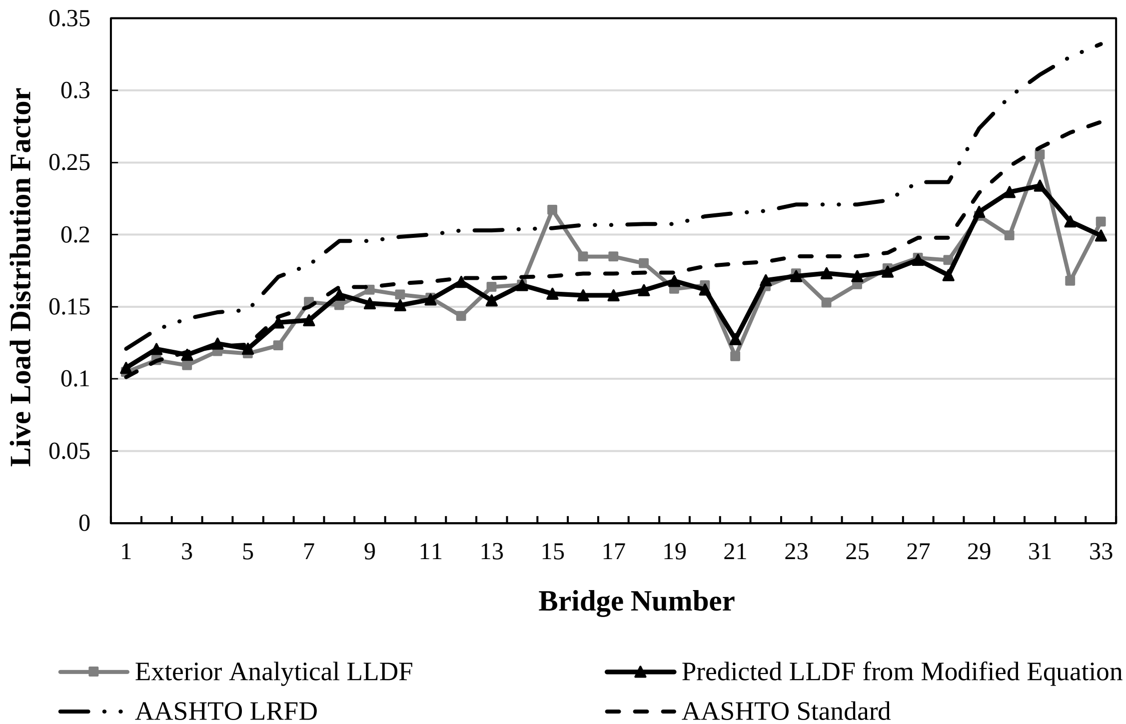


Figure 39. Exterior analytical LLDFs with 121 farm vehicles – one-way traffic lane timber-timber bridges

A modified AASHTO LRFD type equation similar to the one developed for steel-concrete bridges to predict the exterior LLDFs for the 121 farm vehicles is given below.

$$LLDF_{exterior} = \left(\frac{S}{12.2}\right)^{0.56}\left(\frac{S}{L}\right)^{0.22}\left(\frac{K_g}{12Lt_s^3}\right)^{0.09} \quad (34)$$

Figure 39 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.77 and 0.11, respectively.

Figure 40 shows interior LLDFs of the 33 bridges. The analytical LLDFs are less than the AASHTO standard values in most cases, except for Bridges 1, 7, 12, 13, 15, 29, 31 and 32; whereas, AASHTO LRFD values are conservative in all cases, except for Bridge 15.

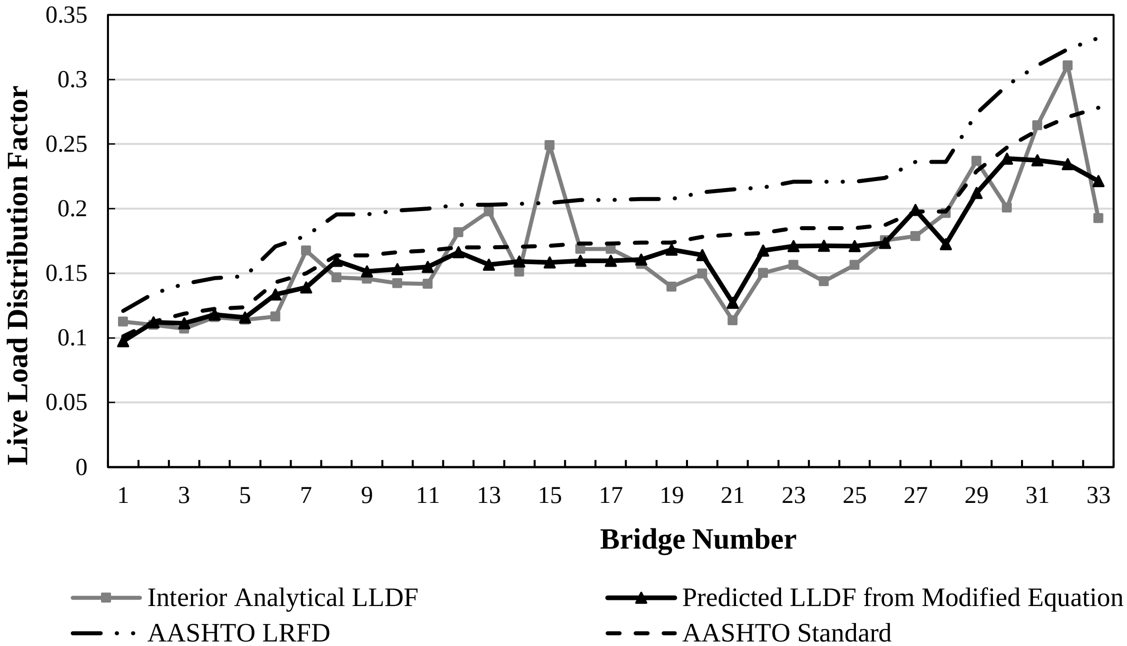


Figure 40. Interior analytical LLDFs with 121 farm vehicles – one-way traffic lane timber-timber bridges

A modified AASHTO LRFD type equation to predict the interior LLDFs for the 121 farm vehicles is given below.

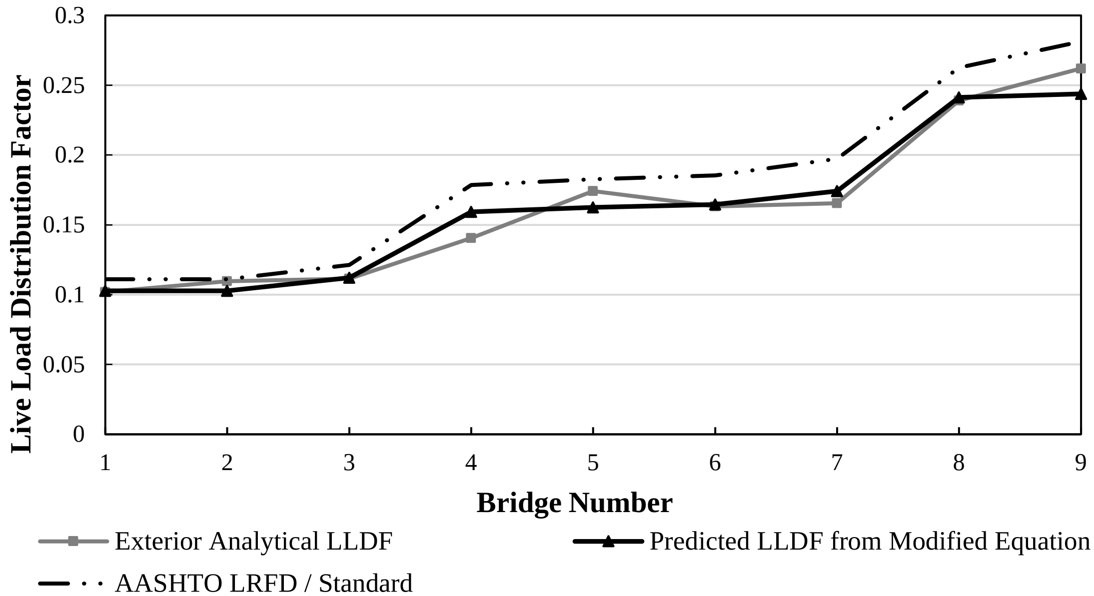
$$LLDF_{interior} = \left(\frac{S}{6.6}\right)^{0.68}\left(\frac{S}{L}\right)^{0.28}\left(\frac{K_g}{12Lt_s^3}\right)^{0.02} \quad (35)$$

Figure 40 also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.70 and 0.15 respectively.

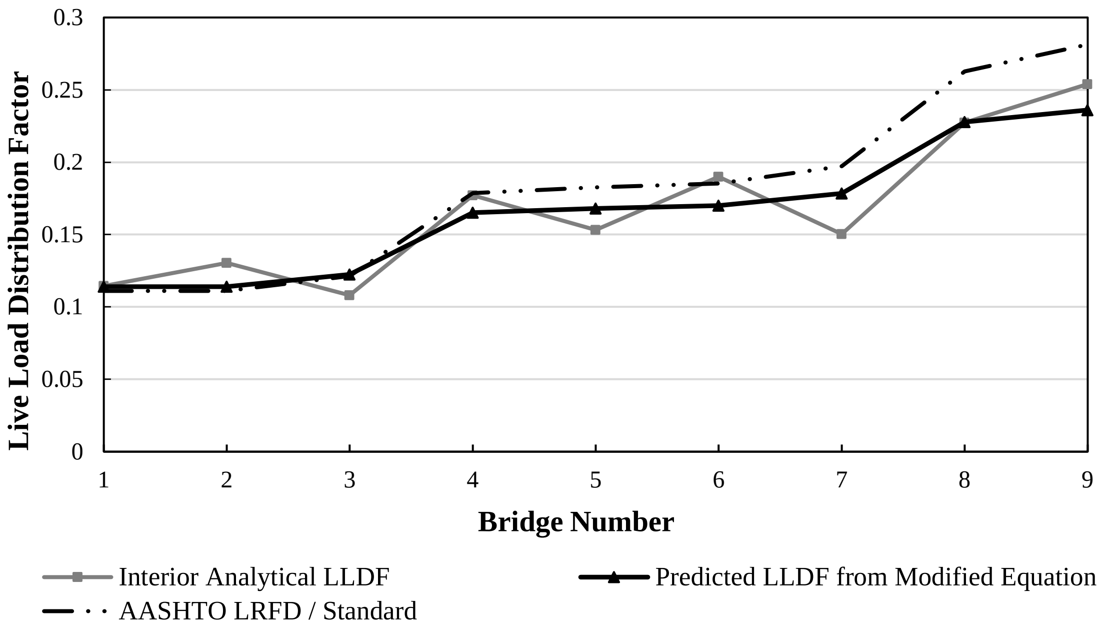
For both exterior and interior girders, it was observed that the analytical LLDFs were less than AASHTO values (standard and LRFD) in most cases. ASHTO LRFD values were conservative in all cases, except Bridge 15.

4.3.3.1.2 Multiple Traffic Lane Bridges

Figure 41(a-b) presents analytical LLDFs of exterior and interior girders for 9 multiple traffic lane timber-timber bridges for all 121 farm vehicles. The bridge numbers shown in the graphs refer to Table 3(b) in Volume III – Appendix E. Bridge Inventory.



(a) Exterior Analytical LLDFs



(b) Interior analytical LLDFs

Figure 41(a-b). Exterior and interior analytical LLDFs with 121 farm vehicles – multiple traffic lane timber-timber bridges

Figure 41(a) shows the exterior LLDFs of the 9 bridges. The analytical LLDFs were less than the AASHTO specifications (standard and LRFD) in all cases. A modified AASHTO equation to predict the exterior LLDFs for the 121 farm vehicles is given below. In this case, the analytical LLDFs depend only on girder spacing and span length among all the bridge geometric parameters.

$$LLDF_{exterior} = \left(\frac{S}{8.6}\right)^{0.75} \left(\frac{S}{L}\right)^{0.15} \quad (36)$$

Figure 41(a) also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.96 and 0.07, respectively.

Figure 41(b) shows interior LLDFs of the 9 bridges. The analytical LLDFs were less than the AASHTO specifications (standard and LRFD) in all cases, except for Bridges 2 and 6. A modified AASHTO LRFD type equation similar to the one developed for steel-concrete bridges to predict the interior LLDFs for the 121 farm vehicles is given below.

$$LLDF_{interior} = \left(\frac{S}{13.1}\right)^{0.72} \left(\frac{S}{L}\right)^{0.05} \quad (37)$$

Figure 41(b) also shows the predicted LLDFs from the above equation compared to the analytical LLDFs. The R square and standard error values for the above equation were 0.86 and 0.13, respectively.

Though only 9 bridges were considered for this study, AASHTO values proved to be conservative for both exterior and interior girders. However, at this time, these equations for multi-lane timber-timber bridges have been included for completeness. It is not the intention of the authors that these equations be used for analysis, as they were developed from a linear regression of only 9 data points. As such, these equations are not an accurate representation of the distribution of live load for multi-lane timber-timber bridges. Further testing should be conducted to determine a better fit for this type of bridge.

4.3.3.2 Modified AASHTO Code Equation – Effect of Gauge Width (varying axle spacing)

4.3.3.2.1 One-Way Traffic Lane Bridges

Figure 42 and Figure 43 present analytical LLDFs of exterior and interior girders, respectively, for one-way traffic lane timber-timber bridges including the effect of gauge width. The bridge numbers in the graphs refer to Table 3(a) in Volume III – Appendix E. Bridge Inventory.

Figure 42(a-d) shows the exterior LLDFs of the 33 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for all the bridges are smaller than both AASHTO Code LLDFs in all cases. As stated previously, AASHTO specifications provide LLDFs for steel-timber bridges based on the S-over rule. The modified S-over AASHTO equation to predict the exterior LLDFs under the varying gauge width vehicles is given as:

$$LLDF_{exterior} = \left(\frac{S}{11.9}\right)^{0.96} \left(\frac{6}{G.W.}\right)^{0.18} \quad (38)$$

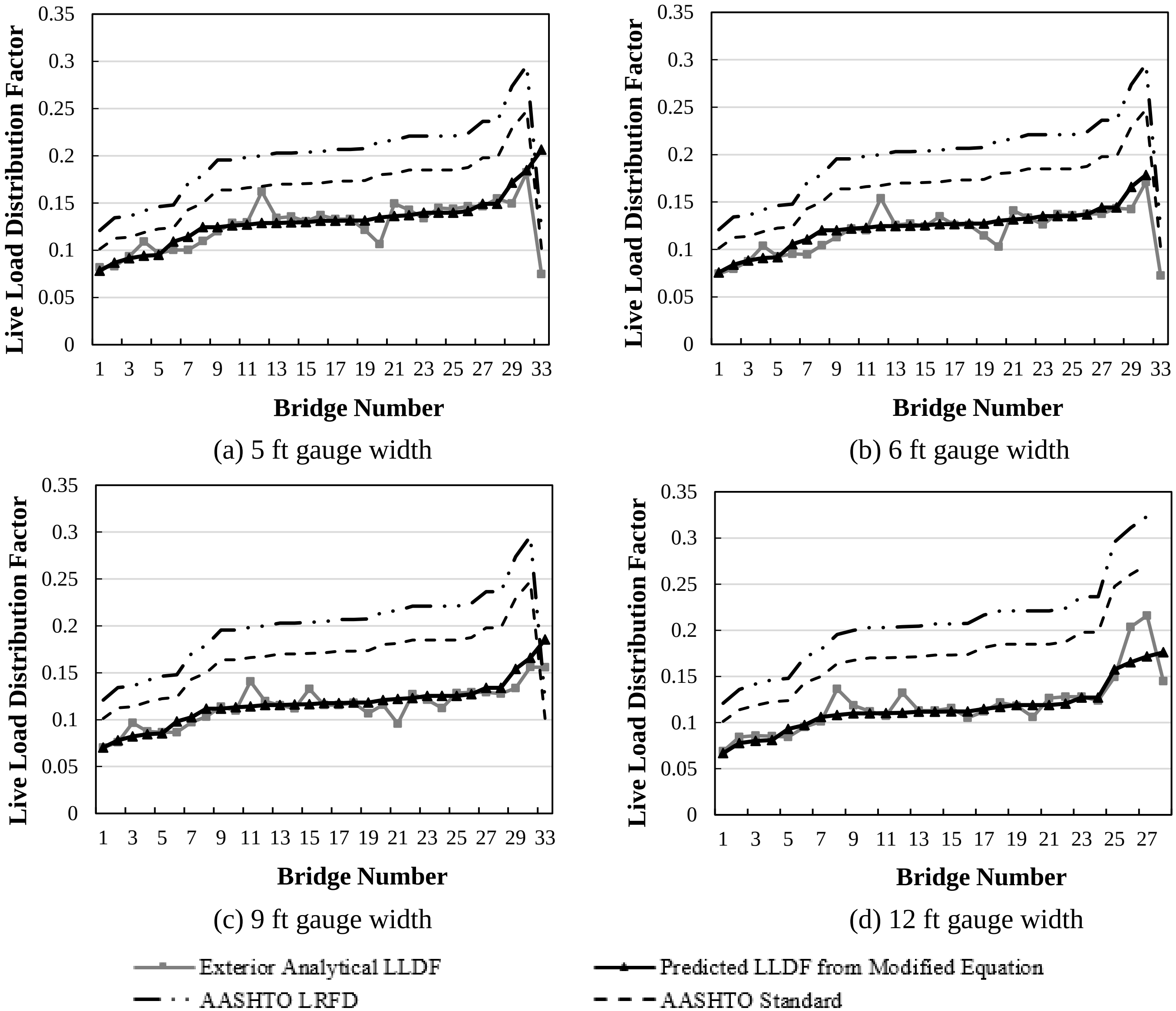


Figure 42(a-d). Exterior analytical LLDF with variation in gauge width – one-way traffic lane timber-timber bridges

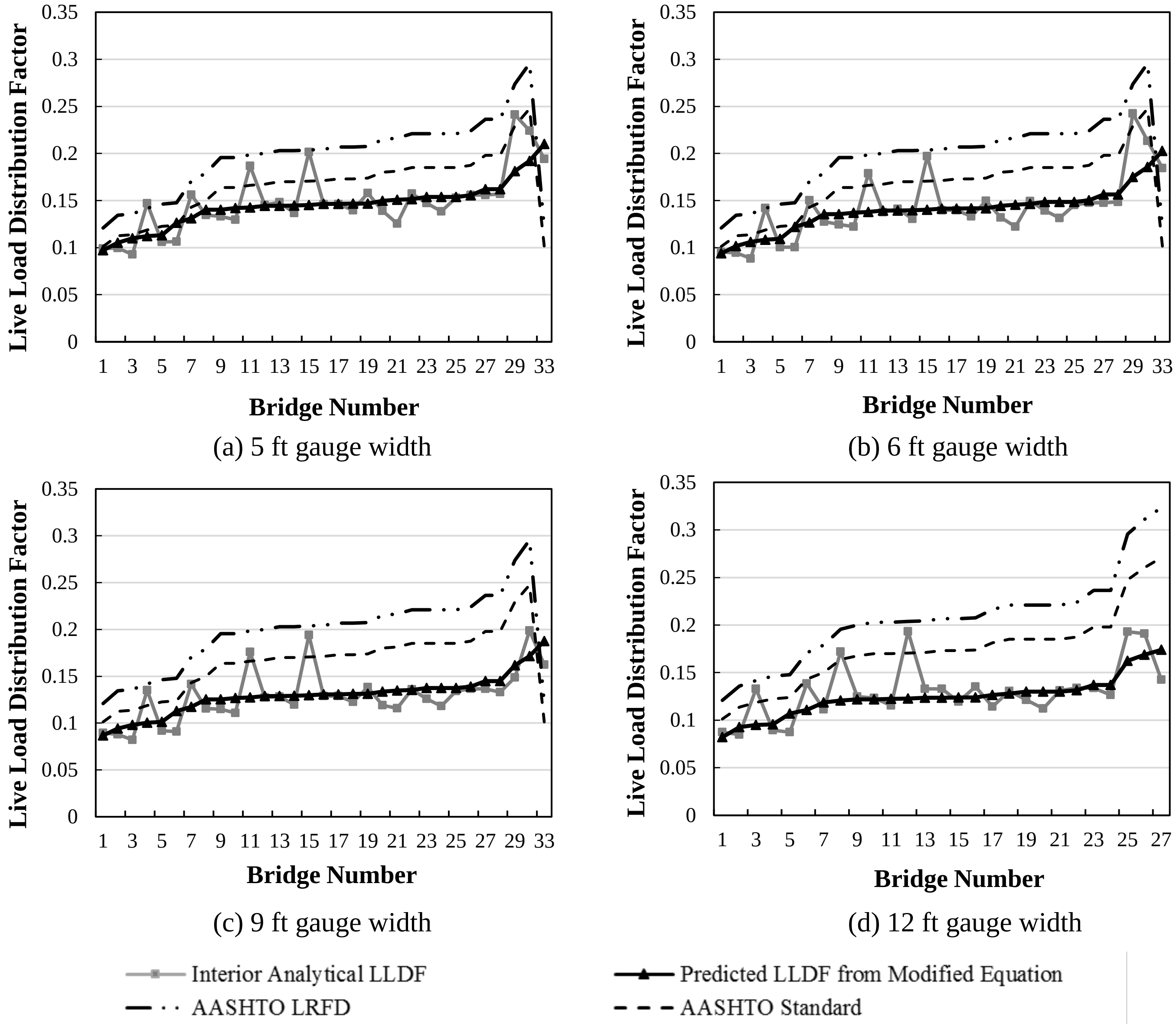


Figure 43(a-d). Interior analytical LLDF with variation in gauge width – one-way traffic lane timber-timber bridges

Figure 42 also shows the predicted LLDFs from the above equation and the comparison with the analytical LLDFs. The R square and standard error values for the above equation were 0.84 and 0.10, respectively.

Figure 43(a-d) shows interior LLDFs of 33 one-way traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The interior analytical LLDFs for most of the bridges are smaller than both AASHTO code LLDFs. For Bridges 4, 11 and 15, the analytical LLDFs for all gauge width vehicles exceeded AASHTO standard values; whereas, for Bridges 7 and 15, the analytical LLDFs with 5 ft and 6 ft gauge width vehicles exceeded AASHTO standard values. AASHTO LRFD values were conservative in all the cases.

The AASHTO LRFD equation was modified to predict the interior LLDFs under varying gauge width vehicles and is given as:

$$LLDF_{interior} = \left(\frac{S}{18.0}\right)^{0.76}\left(\frac{6}{G.W.}\right)^{0.19} \tag{39}$$

The predicted LLDFs from the above equation compared to analytical LLDFs is shown in Figure 43. The R square and standard error values for the above equation were 0.63 and 0.14, respectively.

For both exterior and interior girders, it was observed that analytical LLDFs for most of the bridges were less than AASHTO specifications. The same trend was observed in the analysis including 121 farm vehicles (Figure 42 and Figure 43).

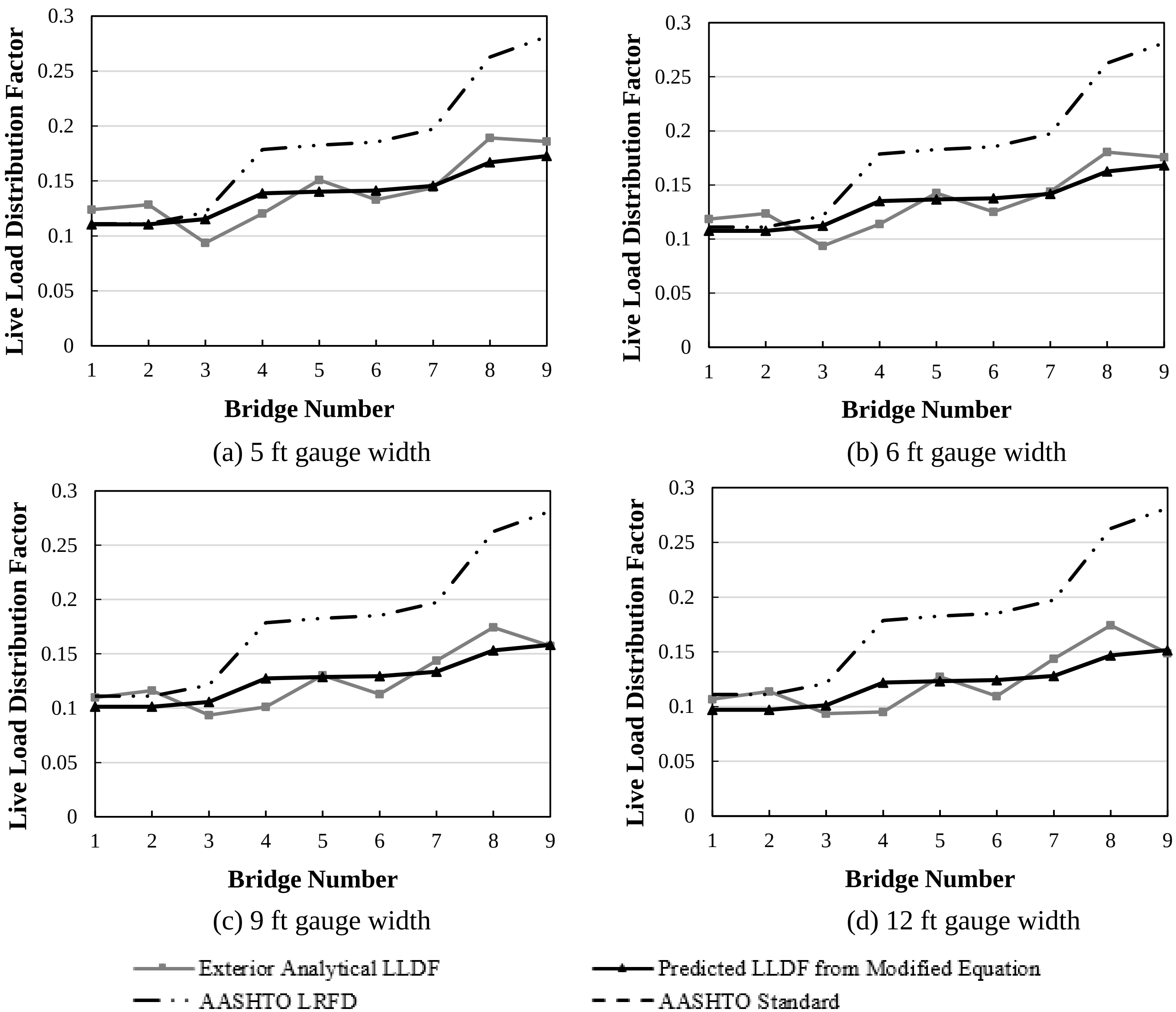
4.3.3.2.2 Multiple Traffic Lane Bridges

Figure 44 and Figure 45 present analytical LLDFs of exterior and interior girders, respectively, for multiple traffic lane timber-timber bridges including the effect of gauge width. The bridge numbers in the graphs refer to Table 3(b) in Volume III – Appendix E. Bridge Inventory.

Figure 44(a-d) shows the exterior LLDFs of the 9 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The exterior analytical LLDFs for all the bridges are smaller than both AASHTO Code LLDFs in all the cases, except for Bridges 1 and 2. As stated previously, AASHTO standard and LRFD Codes provide the same LLDF for multiple lane timber-timber bridges based on the S-over rule. The modified S-over AASHTO equation to predict the exterior LLDFs under the varying gauge width vehicles is given as:

$$LLDF_{exterior} = \left(\frac{S}{86.8}\right)^{0.48}\left(\frac{6}{G.W.}\right)^{0.15} \tag{40}$$

Figure 44 also shows the predicted LLDFs from the above equation and the comparison with the analytical LLDFs. The R square and standard error values for the above equation were 0.64 and 0.12 respectively.



(a) 5 ft gauge width

(b) 6 ft gauge width

(c) 9 ft gauge width

(d) 12 ft gauge width

Figure 44(a-d). Exterior analytical LLDF with variation in gauge width – multiple traffic lane timber-timber bridges

Figure 45(a-d) shows interior LLDFs of the 9 multiple traffic lane bridges for gauge widths of 5 ft, 6 ft, 9 ft, and 12 ft The interior analytical LLDFs for most of the bridges are smaller than both AASHTO Code LLDFs, except for Bridges 1 and 2.

The AASHTO LRFD equation was modified to predict the interior LLDFs under varying gauge width vehicles and is given as:

$$LLDF_{interior} = \left(\frac{S}{142.8}\right)^{0.40} \left(\frac{6}{G.W.}\right)^{0.12} \quad (41)$$

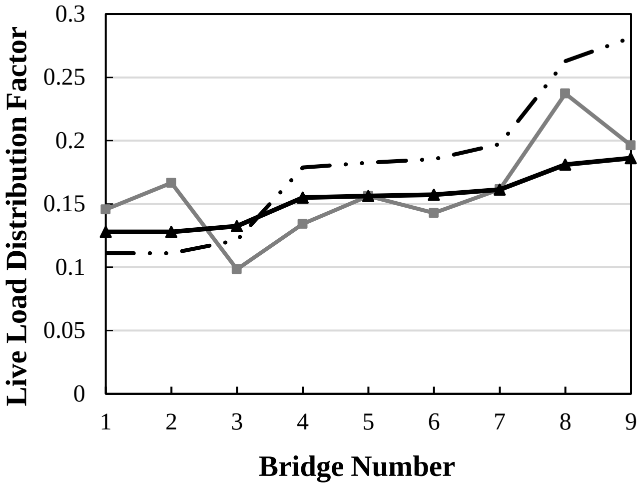
The predicted LLDFs from the above equation compared to the analytical LLDFs are shown in Figure 45(a-d). The R square and standard error values for the above equation were 0.38 and 0.18 respectively.

At this time, these equations for multi-lane timber-timber bridges have been included for completeness. It is not the intention of the authors that these equations be used for analysis, as they were developed from a linear regression of only 9 data points. As such, these equations are not an accurate representation of the distribution of live load for multi-lane timber-timber bridges. Further testing should be conducted to determine a better fit for these type of bridges.

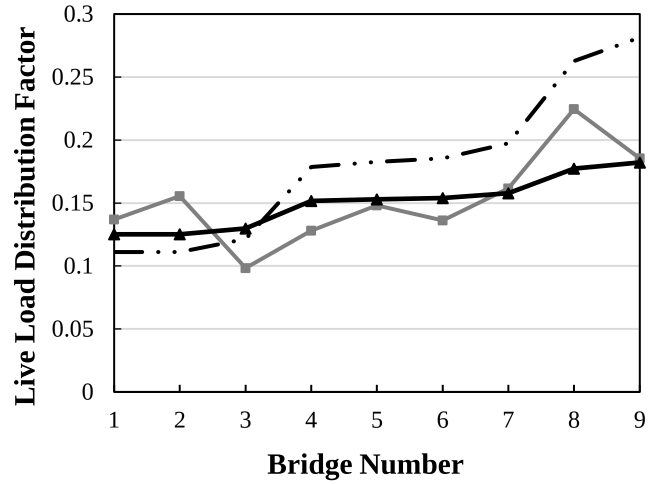
For both exterior and interior girders, it was observed that analytical LLDFs for most of the bridges were less than AASHTO specifications. The same trend was observed in the analysis including farm vehicles (Figure 41).

4.3.3.3 Skew Correction Factor

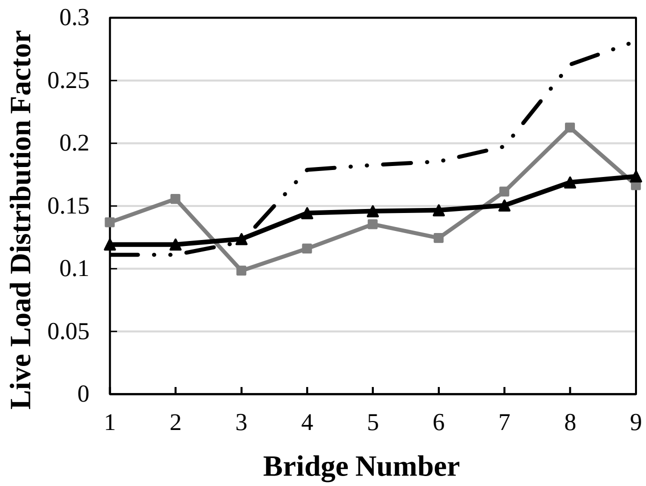
The analytical skew correction factors for the bridges, summarized in Table 3(C) in Volume III Appendix E. Bridge Inventory, were computed. The values of the factors are constant and very close to one. As stated previously, AASHTO codes do not specify skew correction factors for timber-timber bridges and is taken as 1.0.



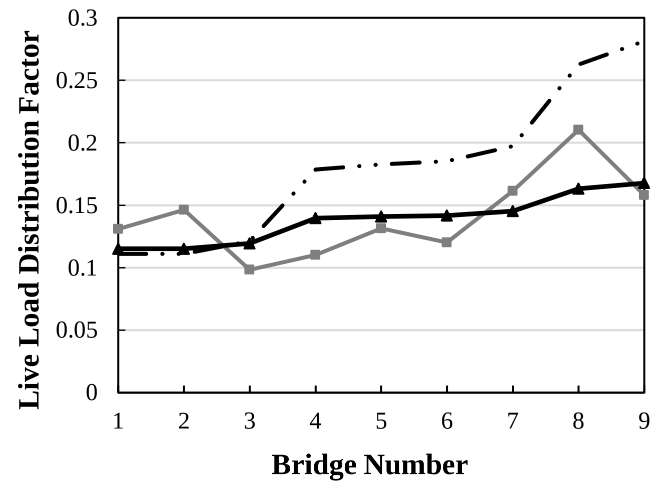
(a) 5 ft gauge width



(b) 6 ft gauge width



(c) 9 ft gauge width



(d) 12 ft gauge width

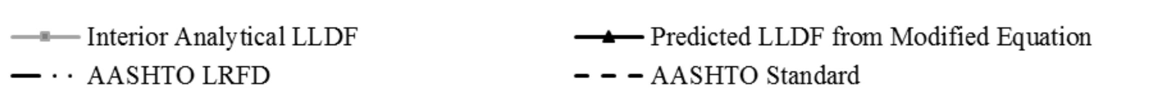


Figure 45(a-d). Interior analytical LLDF with variation in gauge width – multiple traffic lane steel-concrete bridge

5 SUMMARY AND CONCLUSIONS FOR LIVE LOAD DISTRIBUTION FACTORS AND DYNAMIC LOAD ALLOWANCES (VOLUME I)

This section includes the summary, conclusions, and recommendations for Volume 1 of the report.

5.1 Summary

The objective of this study was to develop guidance for engineers on how loading induced by husbandry vehicles is resisted by traditional highway bridges, with a specific focus on bridges commonly found on the secondary road system. To achieve this objective, the distribution of live load and dynamic impact effects for different types of husbandry vehicles on three general bridge types were investigated by load testing and analytical modeling. The three bridge types common to those states participating in this study include: Steel girder bridges with concrete decks (steel-concrete), steel girder bridges with timber decks (steel-timber) and timber girder bridges with timber decks (timber-timber). These slab over girder bridges are most common on secondary roadways in the Midwest and are frequently used by farmers for agricultural purposes. The AASHTO specifications (standard and LRFD) for the three bridge types are documented along with previous research on the LLDFs. AASHTO codes provide LLDFs for steel-timber and timber-timber bridges based on the S-over rule; whereas, the AASHTO LRFD specification has more sophisticated formulas for steel-concrete bridges.

Field testing was carried out on 19 in-service bridges which included five steel-concrete bridges, eleven steel-timber bridges and three timber-timber bridges. As-built plans and previous inspection/rehabilitation reports of each bridge were reviewed, along with a site visit to confirm any modification or member replacements made to the bridge. A network of multiple strain gauges attached to the bottom flanges at the mid-span of all girders was used to measure strain quantities. The load tests were completed using four husbandry vehicles and one five-axle semi-truck. The test vehicles were made to run approximately along the center line of the bridge with a manual clicker marking the location so that the strain values are a function of vehicle location. Initial static load testing was completed with the vehicles traveling at approximately 3 mph such that the pseudo-static bridge response could be captured. Later, dynamic load testing was completed with the vehicles traveling at approximately 10 to 15 mph (maximum safe speed at the site). The strain data was employed to determine field LLDFs and field IM.

A computational/analytical model was created for each of the field-tested bridges using commercially available FEA software. The FEA model for each bridge was based on the geometric information obtained from the Iowa Department of Transportation inspection records and field measurements. Each model was calibrated using the collected field data resulting from each vehicular load. This was accomplished by altering sectional and/or material properties for each model within reasonable limits that were established by previous work, field inspection, and bridge plans; thus making the model as accurate as possible to reasonably predict the actual behavior of each bridge. The calibrated models were then loaded with 121 actual husbandry vehicles from a vehicle inventory with different axle weights and axle configurations gathered through internet searches and manufacturer inquiries. Each vehicle crossed the bridge model at various transverse locations. The analytical strain response was calculated at the same locations as the data from the field testing. The analytical LLDFs were computed for each group of interior and exterior girders. Then, a statistical analysis was completed on the computed analytical LLDFs based upon basic probabilistic theory, resulting in a 95% confidence threshold for each girder group. The above procedure was repeated for all the field tested bridges. The field IMs were used as a basis for the recommended IM for husbandry vehicles.

Following the results of the field tests, including determination of AASHTO LLDFs, field LLDFs, and analytical LLDFs for the 19 bridges, comparison of the LLDFs was done. However, the size of the sample group for each field tested bridge type was not always large enough to generalize the LLDFs trend for the entire population of slab over girder bridges on rural roadways. Therefore, a parametric study was conducted.

The parametric study was conducted to develop a new set of equations for LLDFs similar to AASHTO specifications for the three bridge types, considering the effect of husbandry vehicles. Two primary sources of data were used to collect bridge inventory, which included Structure Inventory and Inspection Management System of the Iowa Department of Transportation website and bridge plans received from the Wisconsin and Oklahoma Departments of Transportation. Overall, 151 in-service bridges were selected from the participating states and covering the three bridge types. The sample included 45 steel-concrete bridges, 54 steel-timber bridges and 52 timber-timber bridges. Analytical FEA models were generated for each of the bridges by utilizing the experience gained from the calibration of the 19 bridges. The 121 husbandry vehicles were applied to each of the 151 bridges and the response captured. Husbandry vehicles have widely varying axle gauge widths that result in different LLDFs compared to typical vehicles. As a result, eight single-axles with gauge widths varying from 5 ft to 12 ft were similarly used as an input and applied to each of the 151 bridges. The FEA models were analyzed considering both the 121 husbandry vehicles and the eight single-axles to compute analytical LLDFs. The maximum exterior and interior analytical LLDFs were taken from the analysis of 121 husbandry vehicles and eight axles for each bridge. Therefore, each bridge resulted in two sets of maximum exterior and interior analytical LLDF values: one for the 121 husbandry vehicles and one for the eight axles.

After gathering the results of the analytical analysis, the maximum exterior and interior analytical LLDFs for each bridge were used as data points to develop empirical equations using a linear regression analysis technique. The empirical equations were developed for each bridge type for 121 husbandry vehicles and eight axles. Skewed bridges were modeled twice: being considered with skew and without skew. The skew correction factors were determined as the ratio of maximum analytical LLDFs determined in the two models; skewed and non-skewed, respectively, for each bridge type.

5.2 Conclusions

The conclusions from the field tested bridges and parametric study are presented in this section.

5.2.1 Field Tested Bridges

5.2.1.1 Steel-Concrete Bridges

1. The interior analytical LLDFs for farm vehicles were smaller than the AASHTO design values (standard and LRFD) for all five bridges. The exterior analytical LLDFs for concrete girders for Bridges 1, 2, 3, and 4 were greater than AASHTO design values, probably because of the increase in stiffness of the exterior girders. The exterior analytical LLDFs for steel girders for Bridge 5 were smaller than AASHTO design values.

2. Comparisons between the statistical limits and AASHTO design values revealed that AASHTO codes for the five bridges are conservative for steel interior and exterior girders, but are not conservative when the exterior girders are concrete.

3. The measured field LLDFs for farm vehicles and a five-axle semi-truck were, in most cases, smaller than AASHTO design values for the five bridges.

5.2.1.2 Steel-Timber Bridges

1. The interior and exterior analytical LLDFs for farm vehicles were smaller than the AASHTO design values (standard and LRFD) in most cases for the eleven bridges. Bridges with identical girder spacing have different analytical LLDFs for both exterior and interior girders, which is not covered by AASHTO specifications based on the S-over rule.

2. Comparisons between the statistical limits and AASHTO design values revealed that AASHTO code values for all eleven bridges are conservative for steel interior and exterior girders.

3. The measured field LLDFs for farm vehicles and a five-axle semi-truck were, in most cases, smaller than AASHTO design values for the eleven bridges.

5.2.1.3 Timber-Timber Bridges

1. The interior and exterior analytical LLDFs for farm vehicles were greater than the AASHTO design values (standard and LRFD) in most cases for the three bridges, probably due to non-uniform girder spacing.

2. Comparisons between the statistical limits and AASHTO design values revealed that AASHTO code values for all eleven bridges are unsatisfactory for timber interior and exterior girders.

3. The measured field LLDFs for farm vehicles and a five-axle semi-truck were, in some cases, greater than AASHTO design values for the three bridges. However, the size of the sample group of studied bridges may be not enough to generalize the trends in LLDFs for the entire population of slab over girder bridges on rural roadways.

5.2.1.4 Dynamic Load Allowance

Recognizing that the field test results are for a limited number of tests with a limited number of vehicles and a limited number of each bridge type, it is probably premature to select an IM for all husbandry vehicles for all bridge types. For the bridges and vehicles field-tested in this study, an upper bound to the Dynamic Load Allowance IM is 60 percent.

5.2.2 Parametric Study

The conclusions for one-way traffic lane bridges, multiple traffic lane bridges and skewed bridges are presented.

For the analysis which included loading from 121 husbandry vehicles, the analytical LLDFs were generally greater than the AASHTO standard, except for multi-lane steel-timber bridges. AASHTO LRFD design values were conservative in all cases for analytical LLDFs for all bridges.

For the analysis with eight axles, the analytical LLDFs were greater than AASHTO standard design values for some cases, e.g., steel-concrete bridges, and smaller for others, e.g., steel-timber bridges. AASHTO LRFD design values were conservative in all cases for exterior and interior analytical LLDFs for the bridges.

The empirical equations developed provide a good estimation of LLDFs, with the following exception: single-lane steel-concrete and multilane timber-timber bridges, which had only a few bridges in this study.

The skew correction factors were close to one and showed a small decreasing trend with increase in skew angle, similar to AASHTO specifications.

5.3 Recommendations

1. In general, AASHTO LRFD specifications were conservative for LLDFs in designing and rating slab over girder bridges for husbandry vehicles.

2. The empirical equations provide a good estimation of the LLDFs and are recommended to be considered in designing and rating slab over girder bridges for husbandry vehicles, but do have limitations, primarily because of the small number of bridges analyzed for some bridge types.

3. This study can be extended to other bridge types that are built on secondary roadways and subjected to husbandry vehicle loadings. Additionally, more steel-concrete, steel-timber, and timber-timber bridges should be added to the above study to increase the confidence in the empirical equations.

4. Because limited dynamic data was available, further investigation of the IM of husbandry vehicles would be appropriate.

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