NCHRP Project 20-07/Task 410

Load Rating for the Fast Act Emergency Vehicles Ev-2 and Ev-3

REVISED FINAL REPORT

Prepared for:

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Prepared by:

HNTB Corporation
350 Fifth Avenue, 57th Floor
New York, NY 10118

In association with:
Dr. Michel Ghosn
The City College of New York

March 2019

Acknowledgements

This study was conducted for the *AASHTO Technical Committee T18*, with funding provided through the National Cooperative Highway Research Program (NCHRP) Project 20-07 Task 410, *Load Rating for the Fast Act Emergency Vehicles Ev-2 and Ev-3*. The NCHRP is supported by annual voluntary contributions from the state Departments of Transportation. Project 20-07 is intended to fund quick response studies on behalf of *AASHTO Technical Committee*. The report was prepared by *HNTB Corp. in association with Dr. Michel Ghosn*. The work was guided by a technical working group. The project was managed by *Dr. Waseem Dekelbab P.E.*, NCHRP Senior Program Officer.

Table of Contents

1. INTRODUCTION

Emergency vehicles in general, and fire apparatus in particular, are often heavier than typical commercial vehicles. Size and weight regulations applicable to emergency vehicles vary widely from one State to the next (IAFC-FAMA, 2011). Many states exempt emergency vehicles from size and weight regulations. On December 4, 2015, the President signed into law the Fixing America's Surface Transportation Act (FAST Act) (P.L. 114-94), which includes new truck size and weight provisions that affect bridge load rating and posting requirements. Among those provisions is the exemption of emergency vehicles from meeting the nationwide Interstate truck weight limits set forth in 23 U.S.C. 127(a). An emergency vehicle as defined in the FAST Act is designed to be used under emergency conditions, including equipment for firefighting and equipment used to mitigate other hazardous situations in an emergency. The emergency vehicles exempted from these weight limits by the FAST Act can create greater load effects in certain bridges than the previous legal loads.

There are two main types of fire apparatus: Fire engines (pumpers) and Fire trucks (or aerial ladder trucks). Both types are usually heavier than typical commercial vehicles. Fire engines carry hose, tools, and pump water and may also carry ladders, but these are of the type that can be set up by the fire fighters and can be carried around. Key components of a fire engine include: water tank; pump; and complements of various types of hoses (for both attack and supply). Fire trucks are equipped with very large hydraulically operated (aerial) ladders, elevating platforms and a full complement of ground ladders of various types and lengths to support fire-fighting and rescue operations, as shown in Fig. 1.1. Fire suppression necessitates lots of water (also industrial foam), requiring tankers with sufficient capacity. Aerial platform trucks need a large mass to counterbalance the aerial device. Pumps and aerial platform trucks typically have tandem rear axles with high capacity axles. The National Fire Protection Association (NFPA) establishes operational and safety criteria for all aspects of fire apparatus including the minimum water, equipment and hose capacity for each type of apparatus. These criteria necessitate the use of high capacity axles.

Fire Apparatus: National Fire Protection Association (NFPA) estimates that there are 29,727 fire departments and 58,750 fire stations across the US (Haynes and Stein, 2017). The NFPA report also notes that there are 71,800 pumpers, 7,300 aerial apparatus, and 79,050 other suppression vehicles. In 2016 there were 1,342,000 fires reported in the US, that is 45 fires per fire department: 475,500 were structure fires, 173,000 were vehicle fires and 662,500 were outside and other fires.

Figure 1.1 – Aerial Ladder Truck (Photo Credit: Joe Mabel)

The average number of miles traveled by any configuration of fire apparatus is less than 5,000 miles per year. This means that the effect of total miles traveled on state and federal roads by these heavy vehicles is minimal (IAFC-FAMA, 2011). Most of their miles will be logged within few miles of the fire station. Some vehicles may occasionally travel long distances to support natural disasters or wild fires. A State may treat emergency response vehicles as non-divisible vehicles or loads and issue special permits. Uniformity in emergency vehicle regulations among states is particularly important given the relatively small number of emergency vehicles and the increasing use of mutual aid agreements that make equipment and personnel available to other states in the event of major emergencies. It should be noted that majority of States already provide size and weight exemptions for emergency vehicles.

The FAST Act Emergency Vehicles (EVs) are designed for use under emergency conditions to transport personnel and equipment to suppress fires and mitigate other hazardous situations. (FHWA, 2016) According to the FAST Act, no State may enact or enforce any law denying reasonable access to motor vehicles subject to this title to and from the Interstate Highway System to terminals and facilities for food, fuel, repairs, and rest. Under this provision, the gross vehicle weight (GVW) limit for EVs is 86,000 pounds. The statute also authorizes the following additional weight limits, depending upon vehicle configuration:

- 24,000 pounds on a single steering axle;
- 33,500 pounds on a single drive axle;
- 62,000 pounds on a tandem axle; or
- 52,000 pounds on a tandem rear drive steer axle.

FAST Act EVs are permitted to have much higher axle weight and higher gross vehicle weight than normal legal vehicles, and the axle group weight of these EVs may not comply with Federal Bridge Formula B as described by the Federal Highway Administration (FHWA, 2018). The moments and shear forces in the girder created by an EV with a front single axle, a tandem rear axle, and a gross vehicle weight of 86,000 lbs may be 82% greater than those caused by an American Association of State Highway and Transportation Officials (AASHTO) Type 3 Legal Truck (AASHTO, 2018) which represents the most common single unit legal load. Therefore, allowing these heavy emergency vehicles to operate freely across highway bridges might compromise bridge safety, serviceability, and durability. A State may also allow these EVs to operate as legal loads on non-Interstate highways without a permit. The FAST Act made FAST Act EVs legal on the Interstate System and the routes within reasonable access to the Interstate. Therefore, States cannot require special permits for FAST Act EVs to cross bridges on the Interstate and within reasonable access to the Interstate. For bridges not on the Interstate System and beyond reasonable access to the Interstate, each State may choose how to revise its law to either allow posting for FAST Act EVs or require permits or restrictions. If a bridge is rated inadequate to carry those EVs, the bridge must be posted for weight in accordance with the National Bridge Inspection Standards (NBIS).

The FAST Act only establishes maximum single axle weight, tandem axle weight, and gross vehicle weight for EVs. It does not explicitly prescribe axle spacing or axle configuration. The FAST Act's EV provisions generally concern fire trucks that do not exceed the weight limits specified above. The Guideline developed by International Association of Fire Chiefs (IAFC) and Fire Apparatus Manufacturers' Association (FAMA) includes a summary of typical fire apparatus configurations (IAFC-FAMA 2011). For any particular type, the wheelbase is not clearly identified in the Guideline. However, the range of wheelbase can be reasonably estimated from the charts and information from fire apparatus manufacturers.

FHWA Guidance: The FHWA Office of Bridges and Structures performed a comparative study to investigate how the wheelbase impacts the maximum moments and shears and develop envelope EV load models as given below. Based on this comparative study, it is concluded that a 15 ft wheelbase, single rear axle configuration and a 17 ft wheelbase, tandem rear axle configuration encompass all the typical fire apparatus, single axle or tandem axle respectively, for the purpose of bridge load rating and posting as shown in Fig 1.2.

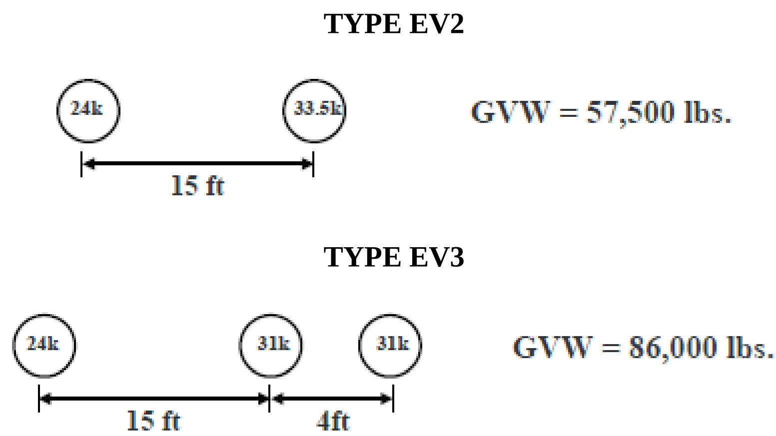

Figure 1.2 – FHWA Emergency Vehicles Live Load Models EV2 and EV3

On November 2016, the Federal Highway Administration Office of Bridges and Structures issued a Memorandum titled "Load Rating for the FAST Act's Emergency Vehicles" https://www.fhwa.dot.gov/bridge/loadrating/161103.cfm to remind States of the load rating and posting requirements and to provide further guidance on appropriate consideration of EVs in bridge rating and posting. The Memorandum introduced two live load configurations defined as EV2 and EV3, provided a screening criterion to help with the prioritization of load rating efforts, and provided guidance on multiple presence and live load factor applications. The two live load model definitions are shown in Fig. 2.a. It should be noted that EV2 and EV3 live load models do not meet the Federal Bridge Formula-B which sets weight limits on groups of axles and in many instances, they produce higher load effects than the three AASHTO Legal Trucks. For example, Fig. 1.3 compares the maximum moments obtained from EV2 and EV3 on simple span bridges to the most critical effect of the three AASHTO Legal Trucks. The figure shows a large difference between the moments produced by EV3 for spans varying between 30 and 300-ft in length, while EV2 produces higher moments than the AASHTO Legal Trucks for spans shorter than 100-ft.

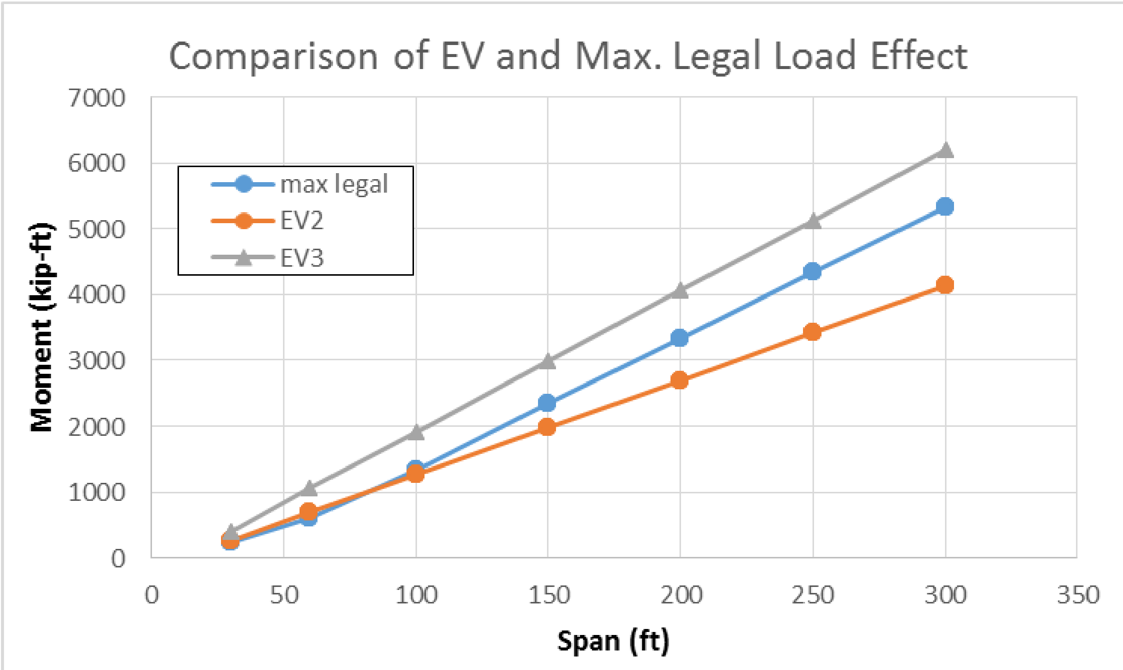

Figure 1.3 – Comparison of Maximum Bending Moments for EV2, EV3 and AASHTO Legal Trucks

The Federal Highway Administration (FHWA) has determined that, for the purpose of load rating, the two emergency vehicle configurations (EV2, EV3) produce load effects in typical bridges that envelop the effects resulting from the family of typical emergency vehicles that is covered by the FAST Act. According to the FHWA Memorandum, when combined with other unrestricted legal loads for rating purposes, the emergency vehicle needs only to be considered in a single lane of one direction of a bridge, which results in the loading condition illustrated in Fig. 1.4.

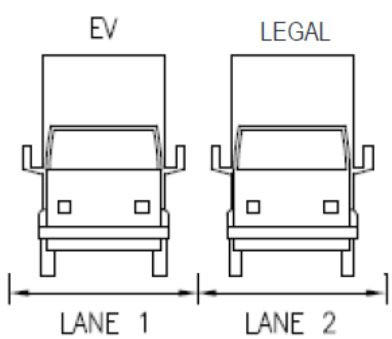

Figure 1.4 – Multiple Presence in Emergency Vehicle Ratings

To account for the low probability of side by side presence of two heavy EVs on a bridge, only one EV is placed in one lane with other unrestricted legal vehicles in other lanes when performing the analysis of a bridge. This exception will reduce the computed load effects on the bridge and yield higher load ratings than placing two EV´s side-side. A 200 lb/ft lane load may also need to be considered for continuous spans in addition to the legal trucks to account for the possibility of having other trucks ahead or behind the EV plus Legal truck combination.

FHWA Memorandum recommends that a live load factor of 1.3 may be utilized in the Load and Resistance Factor Rating (LRFR) or Load Factor Rating (LFR) methods. Note that this single live load factor of 1.3 does not apply to buried structures (i.e., culverts). For buried structures, the appropriate live load factor of 2.0 per AASHTO MBE (2018) Article 6A.5.12.10.3 should be used.

The simplified application of an arbitrarily chosen live load factor of 1.3, as outlined in the FHWA Memorandum, may not lead to a uniform level of safety for all bridge configurations and traffic conditions. Thus, appropriate live load factors for EV should be calibrated for implementation in the AASHTO Manual for Bridge Evaluation (AASHTO, 2018). The calibration of live load factors for use with emergency vehicles should be based on a reliability analysis consistent with the LRFR methodology which has been the basis for the evaluation of existing bridges as stipulated in AASHTO MBE (2018). Adjacent vehicle live load effects should be considered by establishing multiple presence factors appropriate for the FAST Act emergency vehicles using recent traffic data that provide information on the characteristics of the trucks that may cross bridges simultaneously with emergency vehicles. These characteristics include truck axle and gross weights, axle configurations, and truck volumes and headways. In addition to determining appropriate Multiple Presence (mp) factors, the inclusion of lane loads for longer spans, the utilization of striped lanes, positioning of the emergency vehicle for maximum effect, and the appropriate use of dynamic load impact should be investigated to assess the safety of bridges under the effects of Emergency Vehicles. Modifications to the load factors, impact factors, lane loads in the AASHTO Manual for Bridge Evaluation for both LFR and LRFR methodologies are considered in this research and final recommendations are provided and supported by examples.

2. RESEARCH OBJECTIVES

The objective of this research is to propose modifications to the load factors for emergency vehicles in the AASHTO Manual for Bridge Evaluation, for Load Factor Rating “LFR” and Load and Resistance Factor rating “LRFR” methodologies. For the LRFR, the load factors shall be calibrated based on the reliability analysis methodology which is the basis for the current criteria established in AASHTO Manual for Bridge Evaluation (AASHTO, 2018) with appropriate modifications (Moses, 2001; Sivakumar and Ghosn, 2011). The LRFR accounts for the probability of having an emergency vehicle in combination with random trucks on a bridge using multiple presence statistics derived from traffic Weigh-In- Motion (WIM) data. The likelihood of side-by-side presence of trucks and their likely weights will drive the critical combination of trucks (EV + Random) for maximum loading for the load rating period. Research is undertaken in this report to establish whether existing load factors, multiple presence factors based on likely traffic situations, and exposure intervals as specified in the AASHTO MBE (2018) are appropriate for emergency vehicles; further modifications to these load factors are calibrated to be aligned with the LRFR reliability analysis in the AASHTO MBE (2018).

3. RESEARCH APPROACH

The object of this study is to calibrate a load model and live load factors for implementation during the rating of bridges for the crossing of Emergency Vehicles (EV). According to FHWA, the two live load configurations defined as EV2 and EV3 having the axle spacings and weights shown in Fig. 2 produce load effects in typical bridges that envelop the effects resulting from the family of typical emergency vehicles that is covered by the FAST Act. Thus, the calibrated live load factors will be associated with these two truck configurations.

Specifically, the goal of the calibration is to develop a load rating method that can be used to verify that the crossing of EV2 and EV3 when combined with other unrestricted loads will not lead to reduced safety levels for bridges of interest. This could be achieved by selecting appropriate nominal live load model associated with live load rating factors for use in the LRFR rating equation. To be consistent with the AASHTO LRFR principles, appropriate safety levels are achieved when bridges that produce Rating Factors R.F.=1.0 will meet a target reliability index β_{target}=2.5. The application of the same load factors for use in the LFR methodology will also be investigated.

The following steps are followed to complete the calibration process:

1. Assemble a population of typical bridge configurations. The population consists of simple span and two-span bridges up to 300-ft in span lengths.
2. Assemble a representative set of Weigh-In-Motion (WIM) truck data. The data should include the axle configurations of the trucks, their axle weights, speeds, and arrival times.
3. Analyze the WIM data to study the probability of having an EV cross a bridge of a certain length and configuration simultaneously with other random trucks. The random trucks could cross the bridge in the same lane or in a lane parallel to that occupied by EV.
4. Assume a load rating format that includes a live load model and live load factor. For example, a typical LRFR equation may take the following format:

$$RF=\frac{\phi R_n-\gamma_{DW}D_w-\gamma_{DC}D_c}{\gamma_{EV}\left(truck+lane+IM\right)\times mp\times DF} \qquad (1)$$

where RF is the rating factor, ϕ is the resistance factor, R_n is the nominal member resistance, γ_{DW} is the dead weight factor for wearing surface, D_w is the dead load effect of the wearing surface, γ_{DC} is the permanent load factor for bridge components, D_c is the permanent load effect, γ_{EV} is the EV load factor, EV is the permit load effect, *lane* is the lane load, IM is the dynamic amplification, *mp* is the multiple presence factor, DF is the load distribution factor. It is observed that *lane* represents the effects of random trucks that may travel in the same lane as EV, while *mp* represents the effects of random trucks that may travel in parallel lanes simultaneously with EV. It is noted that in current AAHTO LRFR formats, the multiple presence factor is embedded in the load distribution factor or in the live load factor. Also, in short simple span bridges lane is set at zero except when performing HL-93 Inventory and Operating ratings, or when analyzing continuous spans or simple spans longer than 200-ft. Following current practice, bridges whose members produce a rating RF=1.0 or higher are considered structurally safe.

5. Apply the load rating equation with appropriate nominal live load models and load and resistance factors on a population of typical bridge configurations and determine the required resistance R_n that will ensure that each bridge would produce a Rating Factor R.F.=1.0.
6. Develop a load simulation algorithm to estimate the expected maximum truck loads that may cross the bridge of interest simultaneously with EV.
7. Develop a reliability analysis procedure applicable for evaluating the reliability index of bridges being crossed by EV.
8. Assemble probabilistic models from the literature and the simulation of step 6 for all the random variables that affect the rating of bridges.
9. Apply the probabilistic models of step 8 and the load simulation of step 6 into the procedure of Step 7 to find the reliability index β for the population of bridges identified in Step 1 that have achieved a rating factor RF=1.0 for the given set of nominal live load model and live load factors established in Step 5.
10. Using the reliability index values obtained in Step 9, verify that the bridges that produced a R.F.=1.0 in Step 5 will meet a reliability index β_{target}=2.5. If this condition is not satisfied, then adjust the load factor, γ_{EV}, lane load, the multipresence factor, *mp*, and other parameters and repeat steps 5 through 8 until the target reliability index is achieved. The target β_{target}=2.5 for a 5-year rating cycle is selected as the main calibration criterion for consistency with the current AASHTO LRFR criteria (Moses, 2011, Sivakumar and Ghosn, 2011).

This report gives details on the above listed analysis steps that were implemented to calibrate a proposed EV Load Rating procedure for possible implementation as AASHTO EV LRFR specifications. The recommendations are presented for review by the NCHRP Project Panel and the AASHTO T-18 Committee.

4. BRIDGE POPULATION

The reliability calibration of load rating specifications requires that the bridge load rating process leads to uniform reliability levels for the applicable bridge configurations. Hence, the calibration has to be performed on a sample set of bridges that are most representative of the bridges to which the specifications will apply. During the calibration of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2017), Nowak (1999) used a representative sample of multi-girder concrete T-beam, prestressed concrete and composite steel bridges having span lengths varying between 30-ft to 200-ft and beam spacings varying between 4-ft and 12-ft. The analysis was performed for shear and bending moment effects. These same parameters were also used for the calibration of the AASHTO LRFR (Sivakumar and Ghosn, 2011). Subsequently, the AASHTO LRFR extended the range of applicability to bridges up to 300-ft in span length which is the recommended span length for the applicability of the proposed EV Load Rating Methodology.

Besides the span length and beam spacing, the load rating analysis requires typical values for the dead weights of the bridge population. In this study, the dead weight values proposed by Nowak (1999) during the calibration of the AASHTO LRFD were extended to bridges up to 300-ft in span length using a multi-variable linear regression model. The resulting dead weights are provided in Tables 4.1, for simple span prestressed concrete, composite steel and reinforced concrete bridges and Tables 4.2, for the continuous bridges. In Tables 4.1 and 4.2, following the notation of Nowak (1999) DC1 represents the weight per unit length (kip/ft) of the precast members, DC2 represents the weight per unit length in kip/ft of cast-in-place members, and DW represents the weight of the wearing surface applied on one beam per unit length in kip/ft.

Table 4.1 – Dead Weight per Unit length of Simple Span Bridges (kip/ft)

Span (ft)	Spacing (ft)	Simple Span Prestressed Concrete Bridges			Simple Span Composite Steel Bridges			Simple Span Reinforced Concrete Bridges		
		DC1	**DC2**	**DW**	**DC1**	**DC2**	**DW**	**DC1**	**DC2**	**DW**
30	4	0.42	0.53	0.11	0.01	0.52	0.11	0.00	0.70	0.11
	6	0.43	0.74	0.16	0.02	0.74	0.16	0.00	0.99	0.16
	8	0.43	0.96	0.22	0.05	0.96	0.21	0.00	1.29	0.21
	10	0.43	1.17	0.27	0.09	1.17	0.27	0.00	1.58	0.27
	12	0.44	1.39	0.32	0.12	1.39	0.32	0.00	1.87	0.32
60	4	0.58	0.53	0.11	0.10	0.52	0.11	0.00	1.04	0.11
	6	0.58	0.74	0.16	0.13	0.74	0.16	0.00	1.33	0.16
	8	0.59	0.96	0.22	0.17	0.96	0.22	0.00	1.62	0.22
	10	0.59	1.17	0.27	0.20	1.17	0.27	0.00	1.91	0.27
	12	0.60	1.39	0.32	0.24	1.39	0.32	0.00	2.21	0.32
100	4	0.79	0.53	0.11	0.25	0.52	0.11	0.00	1.48	0.11
	6	0.79	0.74	0.16	0.29	0.74	0.16	0.00	1.78	0.16
	8	0.80	0.96	0.22	0.33	0.96	0.22	0.00	2.07	0.22
	10	0.80	1.17	0.27	0.36	1.17	0.27	0.00	2.36	0.27
	12	0.81	1.39	0.32	0.40	1.39	0.32	0.00	2.65	0.32
150	4	1.05	0.52	0.11	0.45	0.52	0.11	0.00	2.04	0.11
	6	1.06	0.74	0.16	0.49	0.74	0.16	0.00	2.34	0.16
	8	1.06	0.96	0.22	0.52	0.96	0.22	0.00	2.63	0.22
	10	1.06	1.17	0.27	0.56	1.17	0.27	0.00	2.92	0.27
	12	1.07	1.39	0.32	0.59	1.39	0.32	0.00	3.21	0.32
200	4	1.32	0.52	0.11	0.65	0.52	0.11			
	6	1.32	0.74	0.16	0.68	0.74	0.16			
	8	1.32	0.95	0.22	0.72	0.96	0.22			
	10	1.33	1.17	0.27	0.75	1.17	0.27			
	12	1.33	1.39	0.32	0.79	1.39	0.32			
250	4	1.58	0.52	0.11	0.84	0.52	0.11			
	6	1.58	0.74	0.16	0.88	0.74	0.16			
	8	1.59	0.95	0.22	0.91	0.96	0.22			
	10	1.59	1.17	0.27	0.95	1.18	0.27			
	12	1.59	1.38	0.32	0.99	1.39	0.33			
300	4	1.84	0.52	0.11	1.04	0.52	0.11			
	6	1.85	0.74	0.16	1.08	0.74	0.16			
	8	1.85	0.95	0.22	1.11	0.96	0.22			
	10	1.85	1.17	0.27	1.15	1.18	0.27			
	12	1.86	1.38	0.33	1.18	1.39	0.33			

Table 4.2 - Dead Weight per Unit length of Continuous Bridges (kip/ft)

Span (ft)	Spacing (ft)	Continuous Prestressed Concrete Bridges			Continuous Composite Steel Bridges			Continuous Reinforced Concrete Bridges		
		DC1	**DC2**	**DW**	**DC1**	**DC2**	**DW**	**DC1**	**DC2**	**DW**
30	4	0.45	0.52	0.11	0.01	0.52	0.11	0.00	0.70	0.11
	6	0.44	0.74	0.16	0.01	0.74	0.16	0.00	0.99	0.16
	8	0.44	0.96	0.22	0.04	0.96	0.21	0.00	1.29	0.21
	10	0.44	1.17	0.27	0.08	1.17	0.27	0.00	1.58	0.27
	12	0.44	1.39	0.32	0.12	1.39	0.32	0.00	1.87	0.32
60	4	0.60	0.52	0.11	0.09	0.52	0.11	0.00	1.04	0.11
	6	0.60	0.74	0.16	0.12	0.74	0.16	0.00	1.33	0.16
	8	0.59	0.96	0.22	0.16	0.96	0.22	0.00	1.62	0.22
	10	0.59	1.17	0.27	0.19	1.17	0.27	0.00	1.91	0.27
	12	0.59	1.39	0.32	0.23	1.39	0.32	0.00	2.21	0.32
100	4	0.80	0.52	0.11	0.24	0.52	0.11	0.00	1.48	0.11
	6	0.80	0.74	0.16	0.27	0.74	0.16	0.00	1.78	0.16
	8	0.79	0.96	0.22	0.31	0.96	0.22	0.00	2.07	0.22
	10	0.79	1.17	0.27	0.34	1.17	0.27	0.00	2.36	0.27
	12	0.79	1.39	0.32	0.38	1.39	0.32	0.00	2.65	0.32
150	4	1.05	0.52	0.11	0.43	0.52	0.11	0.00	2.04	0.11
	6	1.05	0.74	0.16	0.46	0.74	0.16	0.00	2.34	0.16
	8	1.05	0.96	0.22	0.50	0.96	0.22	0.00	2.63	0.22
	10	1.04	1.17	0.27	0.53	1.17	0.27	0.00	2.92	0.27
	12	1.04	1.39	0.32	0.57	1.39	0.32	0.00	3.21	0.32
200	4	1.30	0.52	0.11	0.62	0.52	0.11			
	6	1.30	0.74	0.16	0.65	0.74	0.16			
	8	1.30	0.96	0.22	0.69	0.96	0.22			
	10	1.30	1.17	0.27	0.72	1.17	0.27			
	12	1.29	1.39	0.32	0.76	1.39	0.32			
250	4	1.56	0.52	0.11	0.81	0.52	0.11			
	6	1.55	0.74	0.16	0.84	0.74	0.16			
	8	1.55	0.96	0.22	0.88	0.96	0.22			
	10	1.55	1.18	0.27	0.91	1.18	0.27			
	12	1.55	1.39	0.32	0.95	1.39	0.33			
300	4	1.81	0.53	0.11	0.99	0.52	0.11			
	6	1.81	0.74	0.16	1.03	0.74	0.16			
	8	1.80	0.96	0.22	1.07	0.96	0.22			
	10	1.80	1.18	0.27	1.10	1.18	0.27			
	12	1.80	1.39	0.33	1.14	1.39	0.33			

5. WEIGH-IN-MOTION (WIM) TRUCK DATA

As an Emergency Vehicle (EV) travels on a highway, there is a possibility that other heavy trucks may be crossing a bridge simultaneously with EV. These other heavy vehicles may be crossing in the same or in adjacent lanes. The total load effect that the bridge is subjected to depends on the relative locations of the heavy trucks and emergency vehicles, their axle configurations and the distribution of their weights to the individual axles. As has been the norm for over three decades, information on truck axle configurations, truck weights and their positions relative to each other as they cross a particular bridge can be assembled from existing Weigh-In-Motion (WIM) systems (Sivakumar, Ghosn & Moses, 2011). This information can be used to estimate the expected maximum load effect that a bridge may be subject to due to the combination of EV and random truck crossings.

For this study, quality WIM data recently collected from three WIM sites are used for the analysis of the effects of truck multiple truck presence on highway bridges. Ninety days of data was extracted from a site located on I-95 in New York City, 90 days of data from I-95 New Jersey Turnpike in New Jersey and 120 days of data from a site located on I-90 in Idaho. The New York site was selected because it represents truck traffic in a heavily congested urban area with a significant number of heavily loaded trucks of mixed single truck and semi-trailer configurations. The New Jersey site represents truck traffic in a suburban area with a significant number of single unit trucks. Both the New York and New Jersey sites are exposed to high Average Daily Truck Traffic (ADTT) on the order of 5700 and 7000, respectively. The Idaho site on the other hand represents traffic in a rural area with a low ADTT close to 600 consisting largely of semi-trailer trucks. All three sites show a significant number of overloaded trucks. Figures 5.1, 5.2 and 5.3 give the truck weight histograms for the three sites. The three figures have distinct features where the NY data shows a typical bi-modal distribution of gross weights with peaks near 35 kip and 80 kip. The first mode would correspond to loaded single trucks and empty semi-trailers. The second mode represents the heavy semi-trailers. The New Jersey data on the other hand shows a very sharp peak at GVW=35 kip with a much smaller peak near 80 kips. The Idaho data shows a sharp peak near GVW=80 kip with hardly any peak at the lower end of the histogram.

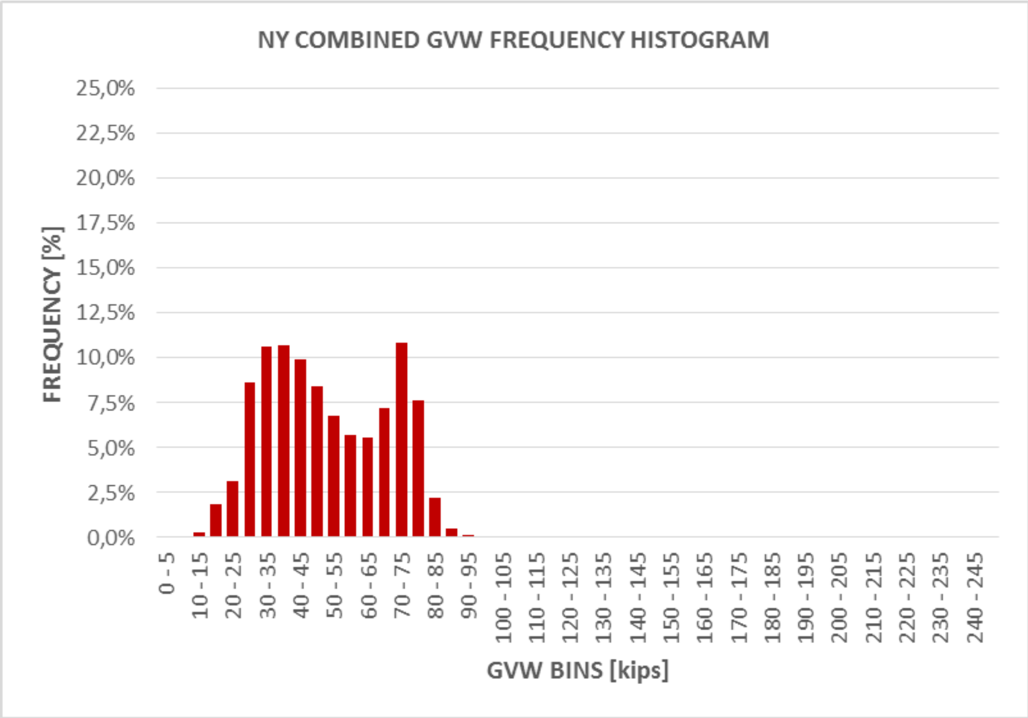

Figure 5.1 – Truck weight histogram for NY I-95 site.

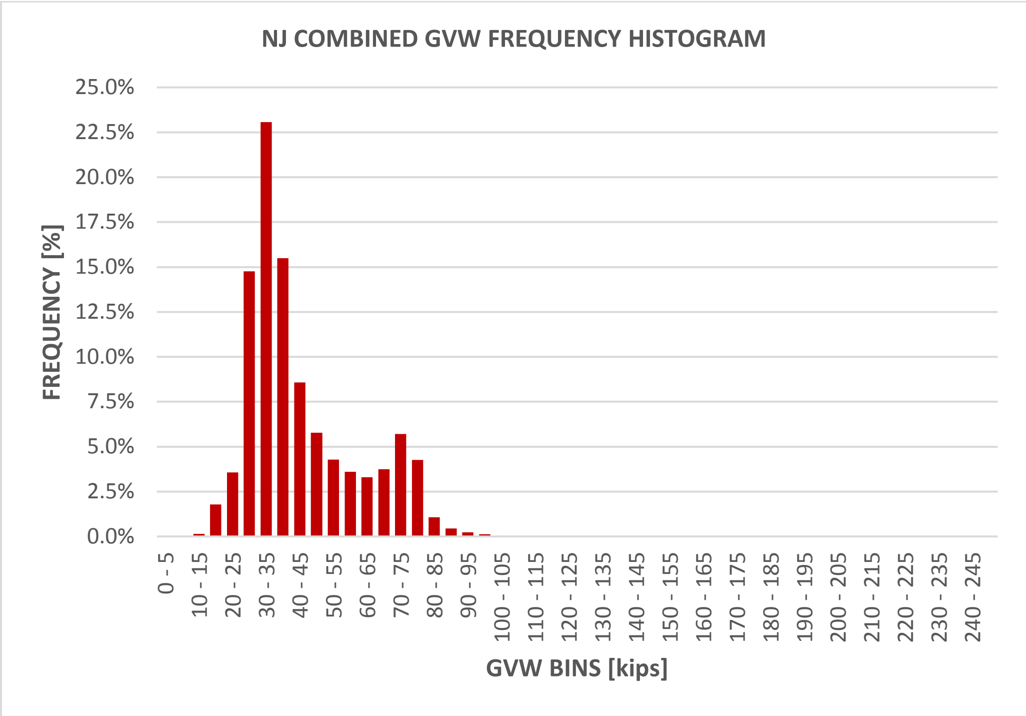

Figure 5.2 – Truck weight histogram for NJ I-95 site.

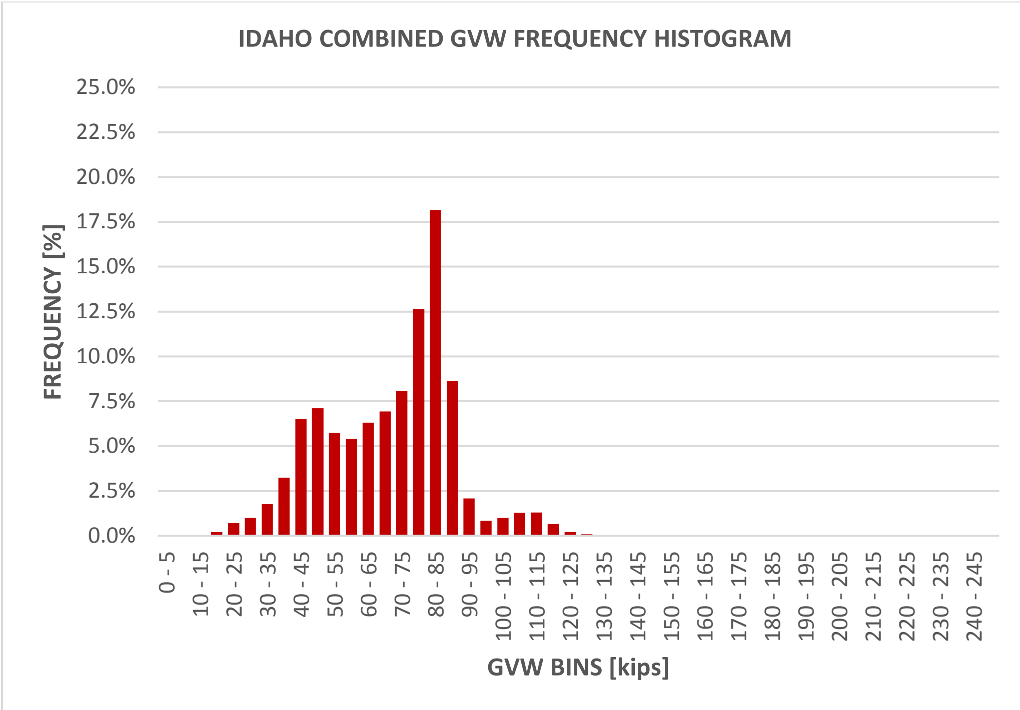

Figure 5.3 – Truck weight histogram for ID I-90 site.

In addition to each truck weight and configuration, the WIM data includes timestamps that can be used to find headways between the trucks. The headway data gives information on the position of the trucks relative to each other. A program was developed to compute headways and extract "loading events" where trucks are simultaneously present on a bridge of a particular length. Headways can be computed based on the timestamp and the vehicle speed data which are included for every truck WIM record. The loading events include "single lane following" and "multi-lane side by side" cases.

Single Lane Following Events

The algorithm presented here describes the steps for identifying the position of trucks relative to each other in one predetermined bridge length (L). In this study, total lengths varying between 30 ft to 600 ft (two spans of 300-ft each) were investigated individually.

Assumptions:

- Trucks do not accelerate or decelerate during bridge crossings
- Headways remain constant, during bridge crossings based on the initial calculation
- Each event includes the maximum number of trucks possible within the event, and subsets of truck combinations are discarded to avoid double counting.

Analysis Steps:

1. Filter the WIM data to include trucks only for the lane of interest (e.g. Lane 1).
2. Sort the data based on each truck´s timestamps, which corresponds to the arrival of the first axle of the truck over the WIM sensor.
3. Determine each truck´s arrival time ($t_{ARRIVAL}$) which is equal to the timestamp in the record.
4. Compute the departure time ($t_{DEPARTURE} = t_{ARRIVAL} + \Delta t$) for each truck in the data file where Δt is calculated as follows:

$$\Delta t = \frac{L + L_{TRUCK}}{V_{TRUCK}} \tag{2}$$

where Δt is the crossing time, L is the total bridge length of interest in ft, L_{TRUCK} is the distance between the first and last axles of the truck in ft, and V_{TRUCK} is the speed of the truck in ft/seconds as recorded in the WIM data file. Then, $t_{DEPARTURE}$ can be computed as:

$$t_{DEPARTURE} = t_{ARRIVAL} + \Delta t \tag{3}$$

5. For each truck record:
   a. Select one truck from the designated lane (e.g. Lane 1) to be the leading truck
   b. Identify all the trucks in the same lane in the WIM file whose arrival times are less than the departure time of the leading truck of interest and mark this collection of trucks collection as a "single lane following" event. The total number of trucks in the loading event is n.
   c. Compute n-1 headways between the consecutive trucks in the event as follows:

$$H_i = \left(t_{ARRIVAL_{i+1}} - t_{ARRIVAL_i}\right) \times V_{TRUCK_i} - L_{TRUCK_i} \tag{4}$$

   where H_i is i^{th} headway within the event, which varies from H_1 to H_{n-1}, n corresponding to the number of trucks within the event.

   d. Construct an "axle train" model with axle weights and spacings where the axle train represents all the axles from all the trucks constituting this loading event. Headways computed in the previous step are injected as spacings between the last and first axles of consecutive trucks, as shown in Fig. 5.4 for a 2-truck event.

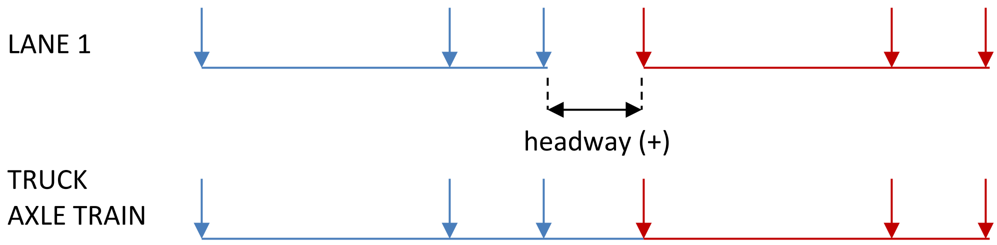

Figure 5.4 – Establishing a Train of truck Axles for Sending through Influence Line

6. Move to the next truck record in the same lane of the WIM data file, designate this new truck as a leading truck and repeat the process.

Multi Lane Side by Side Events

Algorithm steps for the ¨multi lane side by side events¨ follow the same logic described earlier for one lane but brings trucks and their axle configurations from the entire WIM data set (not just a single lane). The algorithm presented here describes the required steps for one predetermined span length (L). In this study, total lengths from 30 ft to 600 ft (two spans of 300-ft each) were investigated individually.

Assumptions:

- Trucks do not accelerate or decelerate during bridge crossings
- Headways remain constant during bridge crossings, based on the initial calculation
- Each event includes the maximum number of trucks possible within the event, and subsets of the maximum truck combination are discarded to avoid double counting.

Analysis Steps:

1. Filter the data for two side by side lanes (e.g. Lane 1 and Lane 2), which corresponds to the entire data set when WIM data is collected from all the lanes of the site.
2. Sort the data based on the timestamps, which corresponds to the first axle of each truck traveling over the sensor.
3. Determine each truck´s arrival time ($t_{ARRIVAL}$) which is equal to the timestamp in the record.
4. Compute the departure time ($t_{DEPARTURE} = t_{ARRIVAL} + \Delta t$) for each truck in the data file using Eq. (2). Then, compute $t_{DEPARTURE}$ using Eq. (3).
5. For each truck record:

a. Designate one truck to be the leading truck
b. Collect all subsequent trucks in the record file whose arrival times are less than the departure time of the leading truck of interest. If both Lane 1 and Lane 2 trucks are present in the collection, mark this truck collection as a "multi lane side by side" event.
c. Compute headways between consecutive trucks using Eq. (4).
Note that in this case, it is possible to compute headways with negative (-) signs in a side by side event. This means that the first axle of a following truck that happens to be in a different lane than the lead truck is ahead of the last axle of the lead truck.
d. Construct an "axle train" live load model with axle weights and spacings from the trucks constituting this event and place each set of axles in the correct position relative to the axles of the leading truck. Figure 5.5 below shows an example of how the positioning of the axles takes place for a 2-truck event.

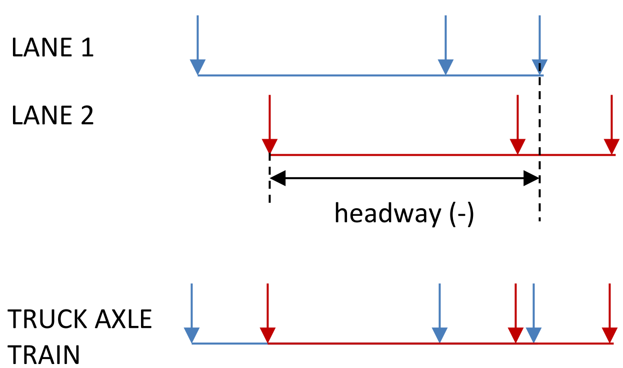

Figure 5.5 – Establish a Multilane Train of Truck Axles for Sending through Influence Line

6. Move to the next truck in the WIM data file, designate this new truck as a leading truck and repeat the process.

The analysis of the single lane and multilane trains of axle trucks was performed for the three WIM data files collected from New York, New Jersey and Idaho. Tables 5.1, 5.2 and 5.3 give the percentage of the total truck crossing events that were found to have multiple trucks fitting within different bridge lengths. The values in the tables were obtained by taking the total number of cases where one truck was found to be simultaneously on a bridge with other following trucks divided by the total number of trucks.

Three cases are considered for each WIM data set: single lane crossings in lane 1, single lane crossings in lane 2, and multilane crossings. The analysis assumes simple span bridges and continuous bridges with two spans of equal lengths. The percentage of multi presence events is found to be quite high for New York and New Jersey sites. This might be due to the heavy traffic congestions observed in those two regions. On the other hand, the Idaho site shows a low percentage of multiple presence events. The larger percentage of multi-lane multiple presence events observed in Tables 5.1, 5.2 and 5.3 compared to those given by Sivakumar, Ghosn, & Moses (2011) is due to the different way of counting where in this report multiple events include cases where the leading truck is just about to leave the far end of a bridge when the following truck is just arriving at the other end. Although these cases are counted as multiple presence loading events, the maximum load effect of the event will be dominated by the load effect of one of the trucks as will be seen further below.

Table 5.4, 5.5 and 5.6 give the average number of trucks per multiple presence event for each span length considered for single lane and multilane loadings. It is observed that for the Idaho site, the average number of multiple trucks simultaneously on the bridge remains very close to 2 even when studying two span bridges with a total length of 600-ft (or 300-ft per span). On the other hand, for the New York site which is travelled by a high volume of trucks per day in congested traffic conditions, the average number is close to 4 (actually = 3.85) trucks for the 600-ft total length and it is less than 3.0 (actually = 2.92) for the New Jersey site even though the daily volume of trucks is higher. The difference is obviously due to higher concentration of trucks in the New York site due to the congested traffic condition. This shows that not only the number of multiple presence events is higher for New York but that also every event will see a large number of trucks in tight formations which would lead to heavy expected load effects.

Table 5.1 - Percentage of NY I-95 truck crossings that include multiple presence.

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	0.14%	0.19%	8.84%	2.44%	3.93%	11.94%
60	2.44%	3.93%	11.94%	12.36%	14.88%	15.92%
100	8.90%	11.60%	14.76%	23.37%	24.42%	19.67%
150	17.12%	19.16%	17.50%	31.26%	30.75%	22.85%
200	23.37%	24.42%	19.67%	36.06%	34.45%	25.28%
250	27.86%	28.08%	21.40%	39.29%	36.77%	27.05%
300	31.26%	30.75%	22.85%	41.49%	38.42%	28.47%

Table 5.2 - Percentage of NJ I-95 truck crossings that include multiple presence.

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	0.14%	0.06%	6.92%	0.99%	0.50%	9.28%
60	0.99%	0.50%	9.28%	7.46%	3.63%	13.30%
100	5.03%	2.40%	12.06%	15.85%	8.61%	17.33%
150	10.96%	5.58%	14.94%	22.91%	14.02%	21.14%
200	15.85%	8.61%	17.33%	27.80%	18.61%	24.03%
250	19.77%	11.39%	19.37%	31.36%	22.49%	26.39%
300	22.91%	14.02%	21.14%	34.13%	25.93%	28.33%

Table 5.3 - Percentage of ID I-90 truck crossings that include multiple presence.

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	0.02%	0.00%	0.44%	0.07%	0.22%	0.79%
60	0.07%	0.22%	0.79%	0.27%	0.40%	0.93%
100	0.08%	0.29%	0.84%	0.98%	0.90%	1.23%
150	0.76%	0.65%	1.17%	2.52%	1.52%	1.39%
200	0.95%	0.90%	1.23%	4.20%	1.73%	1.51%
250	2.11%	1.37%	1.38%	5.83%	1.84%	1.61%
300	2.52%	1.52%	1.39%	7.17%	2.24%	1.67%

Table 5.4 - New York site average number of trucks per multiple presence event

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	2.00	2.00	2.00	2.00	2.00	2.06
60	2.00	2.00	2.06	2.00	2.01	2.27
100	2.00	2.00	2.20	2.04	2.08	2.55
150	2.01	2.02	2.37	2.16	2.24	2.90
200	2.04	2.08	2.55	2.29	2.41	3.22
250	2.10	2.16	2.72	2.43	2.57	3.54
300	2.16	2.24	2.90	2.56	2.73	3.85

Table 5.5 - New Jersey site average number of trucks per multiple presence event

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	2.00	2.00	2.00	2.00	2.00	2.01
60	2.00	2.00	2.01	2.00	2.00	2.10
100	2.00	2.00	2.07	2.02	2.01	2.24
150	2.00	2.00	2.16	2.08	2.04	2.42
200	2.02	2.01	2.24	2.16	2.07	2.59
250	2.04	2.02	2.33	2.25	2.12	2.76
300	2.08	2.04	2.42	2.33	2.16	2.92

Table 5.6 – Idaho site average number of trucks per multiple presence event

Span Length [ft]	SIMPLE SPAN			CONTINUOUS		
	LANE 1	LANE 2	MULTILANE	LANE 1	LANE 2	MULTILANE
30	2.00	0.00	2.00	2.00	2.00	2.00
60	2.00	2.00	2.00	2.00	2.00	2.01
100	2.00	2.00	2.01	1.94	2.00	2.02
150	2.00	2.00	2.01	2.01	2.02	2.06
200	2.00	2.00	2.02	2.01	2.06	2.09
250	2.00	2.03	2.04	2.02	2.08	2.13
300	2.01	2.02	2.06	2.03	2.08	2.17

6. EV LOAD AND RESISTANCE FACTOR RATING (LRFR) FORMATS

The Load and Resistance Factor Rating (LRFR) format assigns resistance and load factors to each component of the rating equation to account for the uncertainties associated with the estimation of the parameters that control the safety of existing bridges. The goal of these factors is to ensure that the implementation of the rating equation will lead to safe bridges that meet consistent target reliability levels. The EV rating equation should take into consideration the load effect (bending moments and shear forces) that the crossing of EV imparts on a bridge in addition to the effects of the permanent loads. The live load effects should account for the possible simultaneous crossing of the bridge by random trucks that may travel within the same lane as EV and in adjacent lanes. To keep the bridge evaluation process simple, the actual live load effects are presented using nominal live load models that are easy to implement in everyday bridge engineering practice.

For consistency with current rating methods, an EV LRFR equation can take two alternate formats. The first format uses a nominal load in the EV's travel lane by calculating the effect of the EV itself and applying a lane load to represent the effect of trucks ahead and behind the EV in the same lane, multiple presence of other vehicles in adjacent lanes is implicitly accounted for through a properly calibrated live load factor. This format is consistent with the format currently implemented in the AASHTO LRFR MBE (2018). Specifically, the first proposed load rating equation may take the form:

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left(EV + lane + IM\right)\times DF} \tag{5}$$

where RF is the rating factor, ϕ is the resistance factor, R_n is the nominal member resistance, γ_{DW} is the dead weight factor for wearing surface, D_w is the dead load effect of the wearing surface, γ_{DC} is the permanent load factor for bridge components, D_c is the permanent load effect, EV is the emergency vehicle load effect, lane is the lane load, IM is the dynamic amplification, DF is the load distribution factor, γ_{EV} is the live load factor which implicitly includes the multiple presence factor that accounts for the effects of random trucks that may travel in parallel lanes.

An alternative format will also represent the effect of EV and a lane load in the same lane but accounts for multiple presence in adjacent lanes by placing a legal truck in a position parallel to that of the EV. In the latter case, the live load factor will only account for the variability between the effects of the nominal live load model and the actual load effects. This format is consistent with that proposed by FHWA memo on Emergency Vehicle load rating. The load rating equation would then take the form:

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left[(EV + lane) DF_{EV} + Legal \times DF_{Lgl}\right] IM} \tag{6}$$

where RF is the rating factor, ϕ is the resistance factor, R_n is the nominal member resistance, γ_{DW} is the dead weight factor for wearing surface, D_w is the dead load effect of the wearing surface, γ_{DC} is the permanent load factor for bridge components, D_c is the permanent load effect, *EV* is the permit load effect, *lane* is the lane load, *Legal* is the effect of the AASHTO Legal trucks, *IM* is the dynamic amplification, DF_{EV} is the load distribution factor for the main lane and DF_{Lgl} is the load distribution factor for vehicles in the adjacent lanes, lane represents the effects of random trucks that may travel in the same lane as EV, γ_{EV} is the live load factor for the case where the effects of loads in adjacent lanes is explicitly accounted for through the effects of the legal vehicles.

In Eq. (6), DF_{EV} and DF_{Lgl} can be obtained using two procedures. The preferred method would obtain the load effects directly using a refined structural analysis whereby the bridge superstructure is modeled using a finite element mess or a grillage of beam elements and implemented into a structural analysis program and the EV truck and the AASHTO Legal load are placed on the deck to perform the analysis. An alternate simplified method for determining the load distribution factors DF_{EV} and DF_{Lgl} is given in LRFD A4.6.2.2.5—Special Loads with Other Traffic. This simplified approach relies on the Load distribution tables in the AASHTO LRFD. The simplified approach sets DF_{EV} as the one lane AASHTO LRFD load distribution factor after removing the 1.2 multiple presence factor while DF_{Lgl} is obtained as the difference between load distribution factor for multiple lanes minus DF_{EV}. Because of the conservative nature of the AASHTO LRFD Load distribution tables and the large variability in the results, different live load factors would be applied if a refined analysis is performed as compared to the case when the adjusted AASHTO LRFD load distribution tables are used.

7. IMPLEMENTATION OF EV LRFR EQUATIONS

Implementations of the EV load rating equations, whether that of the format in Eq. 5 or the one in Eq. 6, follow traditional load rating methods. The goal of load rating is to verify that all bridges achieve rating factors $RF \geq 1.0$. Two examples, one that uses Eq. 5 and the other based on Eq. 6, are presented to illustrate the rating analysis process.

Example 1. Rating of Two-Span Continuous Concrete Bridge Using Eq. (5)

EV rating of a multi-lane two-span continuous concrete bridge is illustrated using Eq. (5). The bridge is assumed to have six beams at 6-ft spacing loaded by two lanes of traffic which mixes random trucks with EV2. The rating analysis is performed for negative bending over the middle support with a load factor $\gamma_{EV} = 1.40$ for EV2.

Nominal dead load intensity from Table 1:

$$DC1 = 0\ kip/ft$$
$$DC2 = 1.78\ kip/ft$$
$$DW = 0.16\ kip/ft$$

Dead Load Moments:

$$DC1 = 0\ kip-ft$$
$$DC2 = 2220 kip-ft$$
$$DW = 203 kip-ft$$

Nominal live load effect of EV2:

$$EV = 540\ kip-ft$$

Lane load:

$q = 0.2\ kip/ft$ or $lane = 250\ kip-ft$

Design Dynamic Amplification:

$$IM = 1.33$$

Resistance factor for concrete in bending:

$$\phi = 0.90$$

Multi-lane LRFD Load Distribution Factor:

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$DF = 0.075 + \left(\frac{6}{9.5}\right)^{0.6}\left(\frac{6}{100}\right)^{0.2}(1)^{0.1} = 0.51$$

Nominal Resistance Rn:

$$R_n = 4255 kip - ft$$

Rating Factor:

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}(EV + lane + IM) \times DF}$$

$$RF = \frac{0.9 \times 4255 - 1.50 \times 203 - 1.25 \times 2220}{1.4(540 + 250) \times 1.33 \times 0.51} = 1.0$$

Example 2. Rating of Simple-Span Bridge Using Eq. (6)

EV rating of a multi-lane 200-ft simple span composite steel bridge is illustrated using Eq. (6). The bridge is assumed to have six beams at 8-ft spacing loaded by two lanes of traffic which mixes random trucks with EV3. The rating analysis is performed for positive bending with a load factor $\gamma_{EV} = 1.32$ for EV3.

Nominal dead load intensity from Table 1:

$$DC1 = 0.72\ kip/ft$$
$$DC2 = 0.96\ kip/ft$$
$$DW = 0.22\ kip/ft$$

Dead Load Moments:

$$DC1 = 3593\ kip - ft$$
$$DC2 = 4790 kip - ft$$
$$DW = 1083 kip - ft$$

Nominal live load effect of EV3:

$$EV = 4058\ kip - ft$$

Nominal live load for type 3-3 legal vehicle:

$$Legal = 3340 kip - ft$$

Lane load for simple span:

$q = 0\ kip/ft$ or $lane = 0\ kip - ft$

Design Dynamic Amplification:

$IM = 1.33$

Resistance factor for steel beam in bending:

$\phi = 1.0$

Multi-lane LRFD Load Distribution Factor:

$$DF^* = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$DF^* = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{200}\right)^{0.2}(1)^{0.1} = 0.55$$

Single-lane LRFD Load Distribution Factor:

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$DF = 0.06 + \left(\frac{8}{14}\right)^{0.4}\left(\frac{8}{100}\right)^{0.3}(1)^{0.1} = 0.36$$

Remove multiple presence factor for single lane

DF_1 = 0.36/1.2 = 0.30

Obtain load distribution factor for adjacent lane

DF^*-DF_1= 0.55-0.30 = 0.25

Nominal Resistance Rn:

$R_n = 15695 kip - ft$

Rating Factor:

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left[(EV + lane)DF_{EV} + Legal \times DF_{Lgl}\right]IM}$$

$$RF = \frac{1.0 \times 15695 - 1.50 \times 1083 - 1.25 \times (3593 + 4790)}{1.25[(4058 + 0) \times 0.30 + 3340 \times 0.25] \times 1.33} = 1.0$$

8. EXPECTED MAXIMUM LIVE LOAD

Live Load Simulation

The nominal live load models adopted in the denominators of Eq. (5) or Eq. (6) provide simplified approaches for evaluating the safety of existing bridges. A bridge whose members produce a rating factor RF=1.0 must be able to safely carry the expected maximum total live load effect on the most critical bridge member when one emergency crosses the bridge. The load rating process must consider the load effect of the emergency vehicle in addition to the effects of the random trucks that may be crossing the bridge behind or ahead of the EV when analyzing single lane loadings in addition to the trucks in the other lanes of the bridge when analyzing multi-lane bridges. Due to the nature of bending moment and shear influence lines, and depending on the headway separation between trucks, the effects of trucks behind and ahead of EV may be significant for long simple span bridges and when analyzing continuous span bridges. In this study, we are analyzing bridges with up to 300-ft in span length. It is therefore anticipated that multiple presence in one lane will contribute to amplifying the effects of the emergency vehicle especially for continuous bridges subjected to high Average Daily Truck Traffic and congestions. This effect will be represented by the parameter *lane* in Eq. (5) and (6). For multi-lane bridges, the probability of having trucks in lanes adjacent to those travelled by EV would also significantly contribute to amplifying the effects of the emergency vehicle. The multi-lane effect is represented by the multiple presence parameter, *mp*.

Simulations and extreme value projections must be performed to determine appropriate values for the parameters *lane* and *mp*. It is noted that these values would change depending on span length as well as traffic characteristics. However, in order to simplify the rating process, the current AASHTO specifications smooth out the variability of *lane* and *mp* with span length so as to present only one distributed lane load intensity that would be valid for all span lengths and adjust the associated live load factor accordingly. Furthermore, it is noted that in previous AASHTO LRFD and LRFR calibrations the multiple presence factor *mp* was implicitly included in the distribution factor DF or in the live load factor. The use of a single lane load intensity for all lengths and the inclusion of the multiple presence factor in the load distribution factor, or live load factors, are done to simplify the bridge rating process (Ghosn, Sivakumar and Miao, 2013; Sivakumar and Ghosn; 2011; Moses, 2001; Nowak, 1999).

The live load simulations must consider the probability of having heavy trucks simultaneously on the bridge with the emergency vehicle. They must account for the probable number of vehicles simultaneously on the bridge, their probable configurations, their probable weights and their probable location relative to each other. All these important factors are random and depend on the bridge site conditions. Adopting worst case scenarios for all these factors, such as taking the heaviest encountered trucks and jam them all together in a tight formation would produce unreasonable highly conservative loads as it is unlikely that all the trucks simultaneously on the bridge will be carrying the highest permitted (or recorded) weights or that they will be tightly packed on the bridge. The simulations must use realistic expected load combination data as collected by available WIM stations.

As observed in Section 5 of this report, current WIM systems are capable of providing a complete set of information to execute the required simulations. This information includes: a) truck timestamps and speeds that help locate the position of each truck at a given point in time; and b) axle spacings and axle weights. This information combined with structural analysis influence lines will help analyze the response of the bridge during EV truck crossings.

Because available WIM data does not necessarily include emergency vehicles, or it is difficult to recognize which of the truck records is actually that of an emergency vehicle, the approach adopted in this study to execute the simulation based on a given WIM data set follows these steps:

1. Choose a bridge span length and calculate the influence lines for the bending moment and shear forces at the most critical locations.
2. Choose a leading vehicle from the WIM data file.
3. Extract the timestamp of the leading vehicle and treat it as the truck's arrival time to the bridge.
4. Given the vehicle speed as recorded in the WIM file and the arrival time, determine the time at which the leading vehicle exits the bridge.
5. From the recorded timestamps and speeds of the vehicles that arrive after the leading truck, determine the number of trucks and their locations within the bridge span length range when the first truck exits the bridge. The process is explained in detail in Section 5 above.
6. Assign to each truck in the group identified in step 5 its axle spacings and axle weights as recorded in the WIM data file.
7. Replace one of the trucks in the group with the emergency vehicle. A top view snapshot of the group may appear as shown in Fig. 8.1 for a single lane bridge or Figure 8.2 for a two-lane bridge.
8. Run the truck group after swaping one of the vehicles by EV through an influence line of the bridge being analyzed for evaluating the load effect of the group on critical bridge members and record the maximum load effects. The load effects considered in this study include the maximum shear force and positive bending moment for simple span bridges, and the shear force as well as positive and negative bending moments for two-span continuous bridges.
9. Store the maximum load effects of each group in data files.

10. Repeat steps 7 through 9 as necessary to replace each random truck by EV one at a time and collect all the maximum responses for each swapping scenario.
11. Extract the next truck group from the WIM data file and repeat steps 3 through 10.
12. Collect all the relevant responses into frequency and cumulative distribution histograms. An example of two frequency histograms for the moment on a single lane of a bridge with span length equal to 200 ft is provided in Fig. 8.3. The histograms give the maximum load effects for the cases where EV2 or EV3 are parts of the group of trucks being analyzed. An example of a cumulative histogram for the moment on a single lane of a bridge with span length equal to 200-ft is provided in Fig. 8.4. The high peaks in the first bin of each histogram in Fig. 8.3 correspond to the cases where EV is on the bridge by itself. These bins correspond to the vertical straight lines in the cumulative distributions of Fig. 8.4. These cases are dominant because even if one can fit more than one truck on the bridge, the spacing between these trucks and the shape of the influence line entail that the maximum load effect is often obtained when the EV is located at the peak of the influence line which means that the other trucks could be far enough to have no contributions to the EV´s load effect or to contribute very little. Alternatively, there may be situations where trucks other than the EV´s would dominate, especially when analyzing combinations involving EV2, whose load effect as seen in Fig. 1.3, if lower than that of legal trucks for most span lengths. Fig. 8.5 plots the tail ends of the cumulative distributions on normal probability scale. In addition, the cases involving EV2 and EV3, Fig. 8.5 plots the maximum load effect of the group as collected from the WIM data before swaping any of the trucks with an EV which is labeled ¨WIM Trucks¨. The results show how for this 200-ft simple span the cases with EV2 produce load effects very similar bit slightly lower than those when a random group of trucks cross the bridge with no emergency vehicle. This is expected because as shown in Fig. 1.3, EV2 produces lower load effect than the legal trucks and certainly lower than those overloaded trucks that have been detected in the WIM data file. On the other hand, when the group of trucks includes an EV3 type vehicle, the load effects are significantly higher than those of the random WIM truck groups.

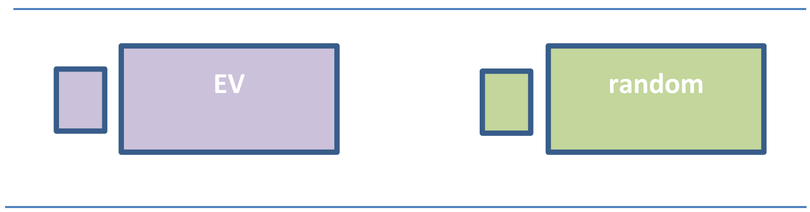

Figure 8.1 – Multiple presence in one lane

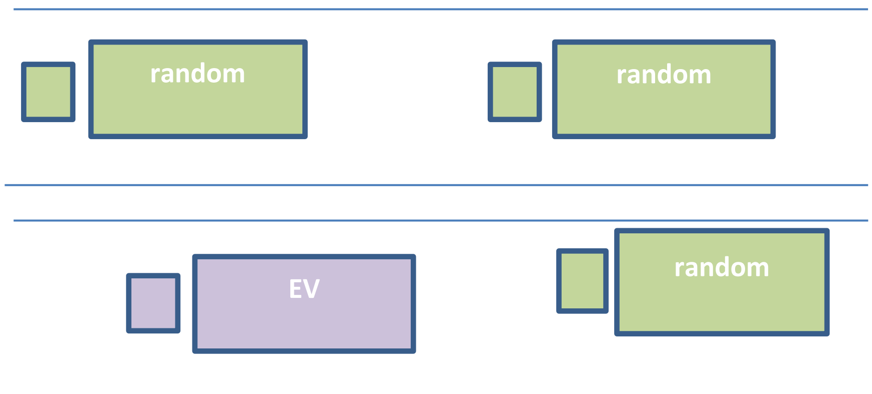

Figure 8.2 – Multi-lane loading for EV load rating

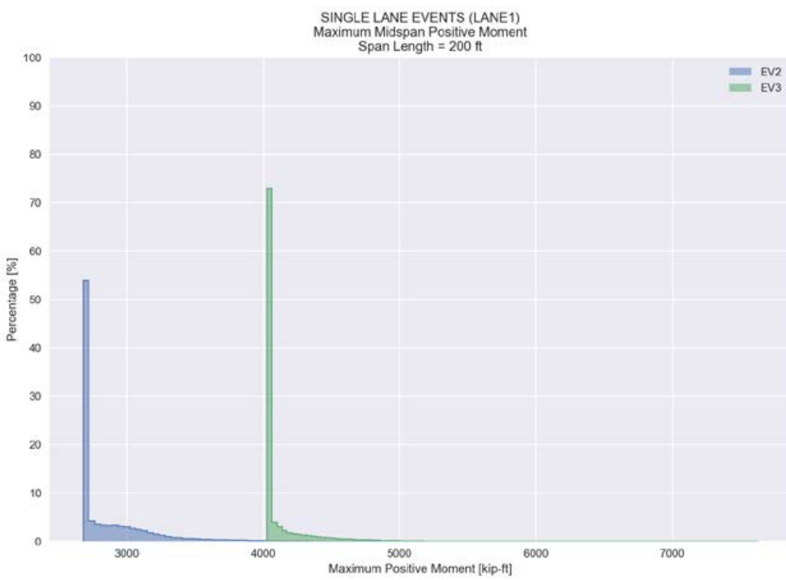

Figure 8.3 - Frequency Histogram for Maximum Midspan Moment for Single Lane Events (Span Length = 200 ft)

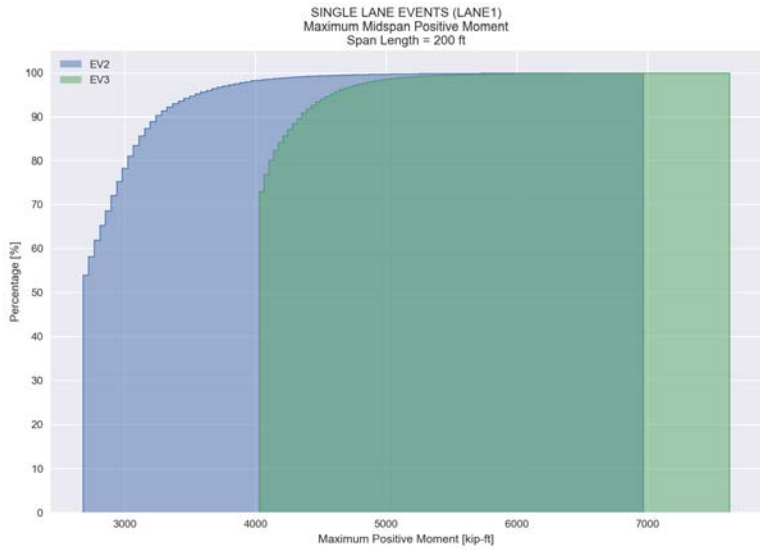

Figure 8.4 - Cumulative Frequency Histogram for Maximum Midspan Moment for Single Lane Events (Span Length = 200 ft)

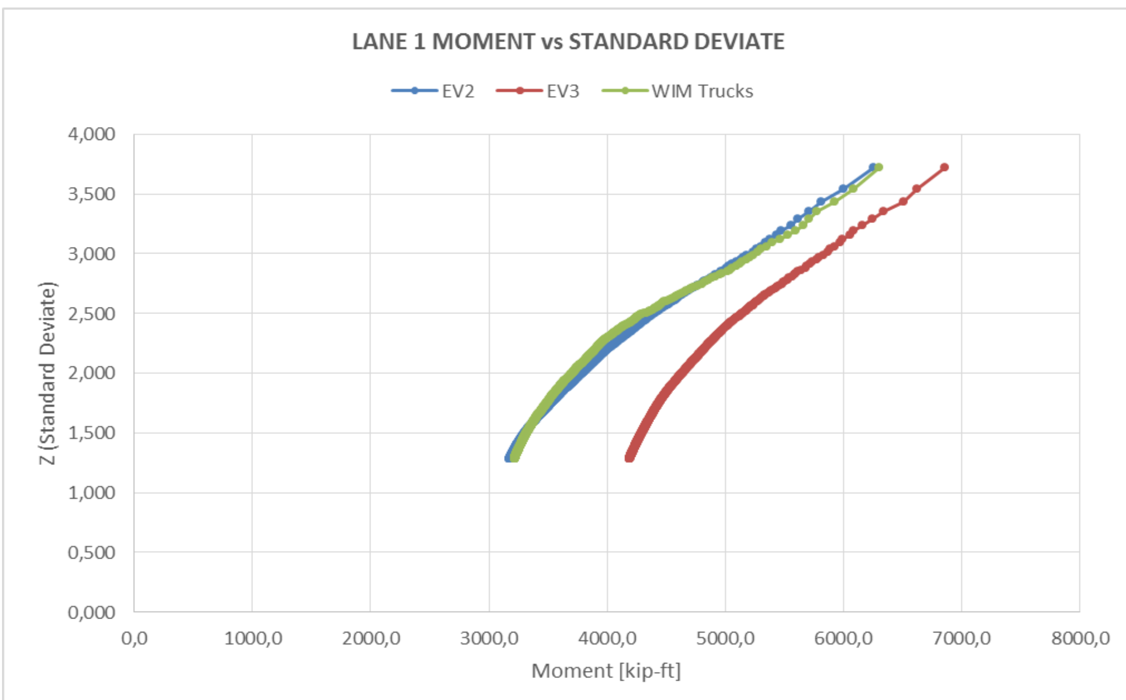

Figure 8.5 - Plot of tail end of cumulative distribution on normal probability scale

The approach adopted for the simulation in this study has two features that may be important under certain conditions:

1. The positioning of the trucks relative to the emergency vehicle represents realistic multiple presence scenarios that properly describe the flow of trucks within the traffic stream as recorded by the WIM data. This is believed to provide more realistic multiple presence representations than those that could be artificially created using micro-simulation programs that attempt to model driver behavior.
2. Taking snapshots of actual truck groups in their relative positions and assigning to them their actual weights and configurations would directly take into consideration possible correlations between trucks that may be carrying similar cargos and traveling together in platoon formations. It is true that swapping one of the trucks with the EV may break this possible formation. But, that would be a realistic scenario when platoons would make room for a passing emergency vehicle.

Extreme Value Projection

The simulations performed as described above would give statistics of the load effects for the commonly observed simultaneous occurrences of trucks on a bridge. However, bridge safety evaluation would require extrapolating these commonly observed observations to extract the expected maximum value that could take place within the bridge rating period. The original AASHTO LRFR was calibrated in the MBE for a five-year rating period (Moses, 2001). This is lower than the design life of new bridges which was assumed to be 75-years during the calibration of the AASHTO LRFD specifications (Nowak, 1999). The lower rating period reflects the fact that bridges are currently inspected on a two-year cycle to monitor deterioration and their condition thus a shorter than a 75-year evaluation period may be justified.

Because the WIM data collected over a relatively short period of time (usually a few months) compared to the five-year rating period and because of the random nature of the load effects, it is unlikely that the histograms assembled from the simulations would reproduce the maximum five-year load effect. Therefore, a statistical projection should be used to extrapolate the histograms assembled from the simulations and extract an estimate of the likely maximum load. The extrapolation can be performed using extreme value statistics depending on the number of EV bridge-crossing events that take place simultaneously with other trucks on the bridge within the rating period. The proposed projection approach assumes that past load data collected at the WIM site that generated the histograms will also be valid in the future and that data collected at the WIM site location are also valid for the location of the particular bridge to be analyzed. It should be noted, that the projection of limited load intensity data, collected from previous measurements over short periods of time, to future service periods is associated with various levels of statistical modeling uncertainties. In addition, modeling the structure's response to the applied loads and estimating the variables that control the effects of the loads on the structure are also associated with high levels of structural modeling uncertainty. These load projection and structural modeling uncertainties are independent of the service period.

To find the probability distribution for the maximum loading event in a service or rating period of time duration, *T*, we have to start by assuming that *N* loading events occur during this period of time *T*. The load effects due to these events are designated as $S_1, S_2 \ldots S_N$. The maximum load effect due to these N events, call it $S_{max,N}$, is defined as:

$$S_{max,N} = max\ (S_1, S_2 \ldots S_N) \tag{7}$$

The cumulative distribution of the effect of one loading event, *i*, S_i is obtained from the simulation would take the shape shown in Figure 7 is defined as $F_{s_i}(S)$. The cumulative distribution $F_{s_i}(S)$ gives the probability that S_i is less than or equal to a value *S*. Given $F_{s_i}(S)$, we are interested in finding the probability distribution of the maximum live load event that will control whether the structure will be safe or unsafe. The probability distribution of the maximum live load effect can be represented by the cumulative probability distribution of $S_{max,N}$ which is defined as $F_{s_{\max N}}(S)$. $F_{s_{\max N}}(S)$ gives the probability that $S_{max,N}$ is less than or equal to a value *S*.

Extreme value projections are based on the realization that if the maximum of *N* events, i.e. $S_{max,N}$ is less than *S*, this implies that each one of these *N* events is less than *S*. Therefore, S_1 is less than *S*, S_2 is less than *S*, … and S_N is less than *S*. Hence, assuming that the loading events are independent random variables, using the basics concepts of the theory of probability, the probability that $S_{max,N} \leq S$ can be calculated from:

$$F_{s_{\max N}}(S) = F_{s_1}(S).F_{s_2}(S)...F_{s_N}(S) \tag{8}$$

where $F_{s_i}(S)$ is the cumulative distribution of event S_i.

If $S_1, S_2 \ldots S_N$ are independent random loading events that are drawn from the same probability distribution, then all $F_{s_i}(S)$ functions are exactly the same independent of which event, *i*, we are specifically targeting:

$$F_{s_1}(S) = F_{s_2}(S) = ... = F_{s_N}(S) = F_s(S) \tag{9}$$

Therefore, Equation (8) reduces to

$$F_{s_{\max N}}(S) = [F_s(S)]^N \tag{10}$$

The statistics of the maximum live load expected during the rating period of bridges, can be obtained from Eq. (10) as will be described further below after raising the cumulative distribution of the load effects, $F_s(S)$ to the pertinent value of N. The cumulative distribution $F_s(S)$ is assembled as explained earlier in Section 5 by sending the group of trucks extracted from a Weigh-In-Motion (WIM) data set through appropriate influence lines and gathering the data from all the groups into cumulative distribution histograms. This could be done for individual trucks and for multi-truck loading events whether these multi-truck events consist of a series of trucks in a single lane (trucks following each other) or trucks in multi-lanes (side-by-side or staggered). It is noted that although we are assuming that the loading events are independent and identically distributed the simulations performed to find $F_{s_i}(S)$ account for possible correlations in the characteristics of the trucks within the group if any with the exception of the EV.

The number of events expected in a bridge's rating period, which is the exponent N in Eq. (10), can be obtained based on information on the number of crossings of EV in simultaneous presence of other trucks. The value for N is obtained by taking the number of EV crossings on a given bridge times the multiple presence percentages of Tables 5.1 through 5.3 times the number of days in the 5-year rating period. This information is obtained from the number of EV sorties per day and the percentage of these sorties that will take place simultaneously with other trucks. A discussion regarding the number of EV sorties is provided in Section 12 further below.

The determination of the number of loading events N is obtained from WIM data in combination with information on the number of times an EV is expected to cross a bridge. For example, the analysis of the WIM data collected at a WIM site on I-95 in New York City has shown that over a three-month period, a total of 508710 trucks crossed the bridge. This represents an Average Daily Truck Traffic ADTT of over 5650 trucks per day at this two-lane highway site. Out of these 320871 crossed in lane 1, 187839 crossed in lane 2. The analysis of simultaneous presence showed 75002 cases of simultaneous crossings by two or more trucks over a 200-ft span of lane 1. There were 45884 cases of simultaneous crossings by two or more trucks for lane 2 over a 200-ft span. Also, 100086 simultaneous truck crossings were recorded in multi-lanes. This indicates that 23.4% (=75002/320871) of lane 1 truck crossings occurred simultaneously with other trucks; 24.4% (=45884/187839); and 19.7% (=100086/508710) involved simultaneous crossings in both lanes. Thus, if on average ten emergency vehicles crossed a particular bridge every day, over the five-year rating period there will be N=10x23.4%x5x365=4271 instances where another truck will simultaneous cross the bridge in lane 1 with EV, or N=10x19.7%x5x365=3595 simultaneous crossings in two lanes. It is noted that the recorded percentages of multiple crossings are extremely high and they are due to the very heavy congestion that takes place on this WIM site which is located on an entrance route to New York City near the George Washington Bridge.

Note that Eq. (10) assumes that the number of events, N, is a known deterministic value. However, a sensitivity analysis performed by Sivakumar, Ghosn & Moses (2011) in NCHRP 683 demonstrated that the results of Eq. (10) are not highly sensitive to small variations in N when the number of events, N, becomes large.

The probability distribution of the maximum live load intensity using Eq. (10) can be used to find the mean and the standard deviation of the maximum intensity, S_{max}, expected in a service period. Figure 8 shows the cumulative distribution of the maximum moment effect, S_{max}, for Lane 1 calculated using Eq. (10) with N=4217 for the 200-ft simple span bridge. The first plot gives the load effects for the cases where EV2 crosses Lane 1 with other vehicles in the same lane. The second plot is for the combination of EV3 plus other trucks in Lane 1. The maximum moment due to EV2 on a 200-ft bridge is calculated to be 2679 kip-ft while the moment due to EV3 is 4029 kip-ft. The plots in Figure 8 are presented using a standard normal scale on the ordinate versus the moment in kip-ft on the abscissa. A straight line on the normal probability scale indicates that the distribution of the maximum load effect is close to a normal distribution. The regression coefficients obtained for a linear fit to the plotted data with R^2=0.99 shows a very good linear fit. Accordingly, the mean value of the maximum expected load corresponds to a standard deviate equal to zero which is calculated to be 6158 kip-ft for combinations of trucks with EV2 and 6742 kip-ft for combinations with EV3. This indicates that for EV2 crossings an additional moment equal to 3479 kip-ft is expected while for EV3 crossings an additional moment equal to 2713 kip-ft is expected. The higher expected additional moment for EV2 compared to that for EV3 is due to the fact that the load effect of EV3 is quite high (50% higher than that of EV2) which means that the probability that the trucks crossing the bridge simultaneously with EV3 have higher moments than that of EV3 is significantly smaller than the probability that the weights of the random trucks are higher than that of EV2. Therefore, EV2 is more likely to be surrounded by trucks heavier than it can carry as compared to the weights of those trucks surrounding EV3.

The standard deviation of the expected maximum load effects can also be extracted from the plots in Fig. 8.6 because the value of the mean moment plus one standard deviation corresponds to a standard deviate on the ordinate equal to 1.0. This calculation would yield a mean plus one standard deviation equal to 6516 kip-ft or standard deviation equal to 358 kip-ft (=6% of the mean) for EV2 and a mean plus one standard deviation equal to 7038 kip-ft or a standard deviation equal to 296 kip-ft (=4% of the mean) for EV3.

Statistical analyses and projections similar to the ones described in this section on live load simulations will provide the input required to perform the reliability-based calibration of the AASHTO EV LRFR as will be described in the next sections of this report.

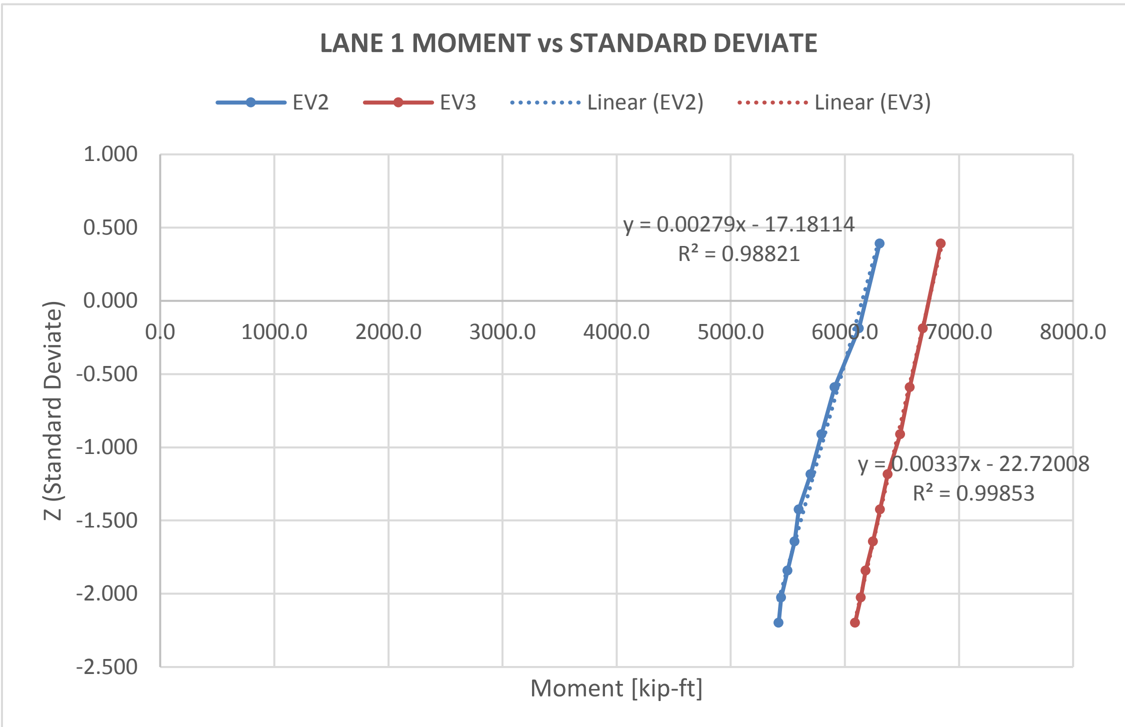

Figure 8.6 - Plots of Maximum 5-yr Moments for Simultaneous Presence of Trucks in Lane 1

9. RELIABILITY ANALYSIS METHODOLOGY

The aim of structural reliability theory is to account for the uncertainties encountered while evaluating the safety of structural systems or during the calibration of load and resistance factors for structural design and evaluation codes. To account for the uncertainties associated with predicting the load carrying capacity of a structure, the intensities of the loads expected to be applied, and the effects of these loads as well as the capacity of structural members may be represented by random variables.

The value that a random variable can take is described by a probability distribution function. That is, a random variable may take a specific value with a certain probability and the ensemble of these values and their probabilities are described by the distribution function. The most important statistical characteristics of a random variable are its mean value or average, and the standard deviation that gives a measure of dispersion or a measure of the uncertainty in estimating the variable. For example, assuming that R represents the resistance capacity of a member, the standard deviation of the random variable R which may have a mean value equal to $\overline{R}$ is represented by σ_R. A dimensionless measure of the uncertainty is the coefficient of variation (COV) which is the ratio of the standard deviation divided by the mean value. For example, the COV of the random variable R is represented by V_R such that:

$$V_R = \frac{\sigma_R}{\overline{R}} \tag{11}$$

Structural design codes and standards often specify nominal or characteristic values for the variables used in design equations. These nominal values are related to the means through bias values. The bias is defined as the ratio of the mean to the nominal value used during the design or evaluation process. For example, if R is the member resistance, the mean of R, namely, $\overline{R}$ can be related to the nominal or design value, R_n, using a bias factor such that:

$$\overline{R} = b_r\, R_n \tag{12}$$

where: b_r is the resistance bias, and R_n is the nominal value as specified by the design code. For example, A50 steel has a nominal design yield stress of 50 ksi but coupon tests show an actual average value close to 56 ksi. Hence, the bias of the yield stress is 56/50 or 1.12.

In structural analysis, safety may be described as the situation where capacity (member strength or resistance) exceeds demand (applied load, moment, or stress). Probability of failure, i.e., probability that capacity is less than applied load effects, may be formally calculated; however, its accuracy depends upon detailed data on the probability distributions of load and resistance variables. Since such data are often not available, approximate models are usually used for calculation.

The reserve margin of safety of a bridge component, also known as failure function or limit state equation can be defined as, Z, such that:

$$Z = R - S \tag{13}$$

where R is the resistance or member capacity, S is the total load effect. Probability of failure, P_f, is the probability that the resistance R is less than or equal to the total applied load effect S or the probability that Z is less or equal to zero. This is symbolized by the equation:

$$P_f = Pr\,[\,R \leq S\,] \tag{14}$$

Where Pr is used to symbolize the term probability. If R and S follow independent Normal (Gaussian) distributions, then the probability of failure can be obtained based on the mean of Z and its standard deviation which can be calculated from the mean of R and S and their standard deviations:

$$P_f = \Phi\left(\frac{0-\overline{Z}}{\sigma_Z}\right) = \Phi\left(-\frac{\overline{R}-\overline{S}}{\sqrt{\sigma_R^2+\sigma_S^2}}\right) \tag{15}$$

where $\Phi(...)$ is the normal cumulative probability function that gives the probability that the normalized random variable is below a given value. $\overline{Z}$ is the mean safety margin and σ_Z is the standard deviation of the safety margin. Thus, Equation 15 gives the probability that Z is less than 0 (or R less than S).

The reliability index, β, is defined such that:

$$P_f = \Phi(-\beta) \tag{16}$$

For example, if the reliability index is β=3.5, then the implied probability of failure is obtained from the Normal probability distribution tables given in most books on statistics as P_f=2.326x10^{-4}. A reliability index β=2.5 leads to P_f=6.21x10^{-3}. A β=2.0 implies that P_f=2.23x10^{-2}. Because it is often difficult to ascertain the type of probability distribution that each variable follows and because it is difficult to collect the real failure data to verify the numbers of failures, calculated values of probability of failure are often considered to be notional measures of likelihood of failure that are used to compare different structural design options and compare various load capacity evaluation methodologies rather than being considered as actuarial values.

For the Normal distribution case, the reliability index is obtained from:

$$\beta = \frac{\overline{Z}}{\sigma_Z} = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{17}$$

Thus, the reliability index, β, which is often used as a measure of structural safety, gives in this instance the number of standard deviations that the mean margin of safety falls on the safe side as represented in Figure 9.

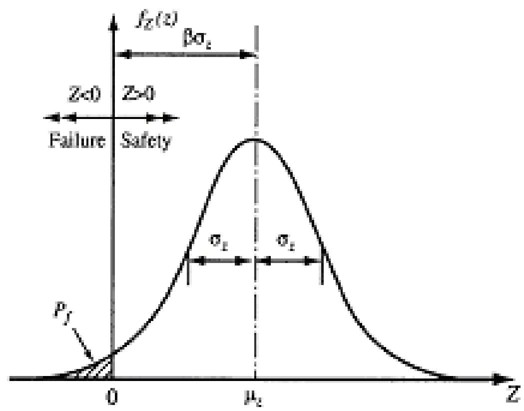

Figure 9.1 - Graphical Representation of Reliability Index.

The reliability index, β, defined in Eq. (16) and (17) provides an exact evaluation of the probability of failure if R and S follow normal distributions. Although β was originally developed for normal distributions, similar calculations can be made if R and S are lognormally distributed (i.e. when the logarithms of the basic variables follow normal distributions). In this case, the reliability index can be calculated as:

$$\beta = \frac{\ln\left(\dfrac{\overline{R}}{\overline{S}} \dfrac{\sqrt{1+V_S^2}}{\sqrt{1+V_R^2}}\right)}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_S^2\right)\right]}} \tag{18}$$

Which, for small values of V_R and V_S on the order of 20% or less can be approximated as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} \tag{19}$$

When *R* and *S* are functions of several underlying random variables or when the safety margin equation is not linear, the evaluation of the reliability index β becomes more complicated. In such cases, one can resort to using simulation techniques or “Level II methods”. "Level II" methods have also been used to obtain the reliability index for the cases when the basic variables are neither normal nor lognormal. These methods often referred to as FORM (First Order Reliability Methods) or FOSM (First Order Second Moment) involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure equation (i.e. when $Z=0$) by a tangent multi-dimensional plane at the point on the failure surface closest to the mean value. For example, during the calibration of the AASHTO LRFD code, Nowak (1999) used the FORM algorithm to calculate the reliability index values when *R* is assumed to follow a lognormal distribution and *S* is a normal random variable. More advanced techniques including SORM (Second Order Methods) have also been developed. In recent years with advancement in computer capacities, Monte Carlo Simulations have also been often used.

The approximate nature of Eq. (19) notwithstanding, the authors of this report have found in several previous studies that the use of Eq. (19) is sufficiently accurate to produce reliability index values very close to those obtained using FORM algorithms and Monte Carlo Simulation (Ghosn, Sivakumar, & Miao, 2013). Therefore, Eq. (19) will be used in this study to perform the reliability calibration of the EV live load factors.

10. PROBABILISTIC MODELS FOR RATING VARIABLES

As observed from Section 9, reliability-based calibrations of rating equations will require probabilistic models for all the random variables that control the safety of bridge structural members. These can be assembled into three groups: Live loads, Permanent Loads and Member Resistance. Below is the description of the models adopted in this study based on the work of Nowak (1999) and Sivakumar and Ghosn (2011.a and 2011.b) in NCHRP Reports # 368, 683 and 20-07 Task 285.

Live Load Reliability Model

The factored nominal live load models used in the calibrated rating equations shown in Eq. 5 and Eq. 6 take deterministic simplified formats that do not explicitly include all the parameters that control the effects of live loading on the bridge nor do the models explicitly reflect the random nature of the live load. A more realistic representation of live load effects on bridge members would take the form:

$$LL_T = \left(\left(EV + lane\right)\times DF_1 + LL_2 \times DF_2\right)IM \times \lambda_{L\max} \times \lambda_{site-to-site} \times \lambda_{data} \times \lambda_{DF} \qquad (20)$$

where LL_T = total live load effect on the member, EV=emergency vehicle load effect on a bridge, $lane$= load due to the trucks following and ahead of the EV, DF_1=distribution factor that gives the fraction of the load effect caused by the trucks in the lane loaded by EV applied to the bridge member being analyzed, LL_2= maximum load due to the random trucks on the bridge in the other lane, DF_2=distribution factor that gives the fractions of the effects of the random trucks in the other lane that are applied to the member being analyzed, IM= dynamic amplification of the total load effect, λ_{Lmax} is a variable that reflects the uncertainties in estimating the maximum load effect calculated using the live load projection methodology described in Section 6 of this report, $\lambda_{site-to-site}$ is a variable representing the variation in the projected live load between data collected at different WIM sites, λ_{data} is a variable representing the effect of limitations in the approach taken to perform the live load simulation and the extreme value projection technique utilized, λ_{DF} is a variable that represents the variation between the load distribution factors used in the analysis and actual field-measured load distributions.

The live load model presented in Eq. (20) can be used to perform reliability calculations of bridge members using methods consistent with those applied when developing the AASHTO LRFD and LRFR specifications (Nowak, 1999; Moses, 2001; Sivakumar and Ghosn, 2011). Statistical data related to the random variables of Eq. (20) are obtained from three sources: a) the data used in NCHRP 368 during the calibration of the AASHTO LRFD as described in NCHRP 368 by Nowak (1999), b) The analysis of large numbers of WIM data sets assembled from various parts of the US as described by Sivakumar & Ghosn

(2011) in the web based report NCHRP 20-07 Task 285, and c) the live load simulations performed in this study as described in Section 8 of this Report.

Maximum Load Effects due to Multiple Presence

The simulations described in Section 8 provide the expected maximum load effect on a bridge for two loading scenarios: a) loading in a single lane; and b) loading in two parallel lanes. The simulations have shown that, generally speaking, the maximum loading approaches a normal distribution as evidenced by the linear trend on a normal probability paper as for example shown in Fig.8.6.

The normal probability plots obtained in Section 8 for each span length and load effect (bending moment or shear) serve to obtain the mean value of the maximum load effect and its standard deviation. Thus, for single lane loading, the results of the simulation will directly lead to the statistical characterization of the term *EV+lane* in Eq. (20).

For example, Fig. 10.1 shows a plot of the expected maximum positive moment of simple span bridges crossed by EV2 or EV3 in combination with other trucks from the New York WIM site. It also shows the expected maximum moment with the crossing of combinations of only random trucks without EV over simple span single lane bridges. The figure also plots the maximum moments if only an EV by itself crosses the bridge. The results in the figure show how the loading when EV2 crosses the bridge is dominated by the random trucks. This is because, given that EV2 load effects are close to those of legal loads, the probability of having at least one truck heavier than EV2 in combination with other heavy trucks is almost as high if not higher than the probability of having one EV2 plus another heavy truck in the same lane. This is obviously not true for EV3 plus other heavy trucks because the effects of EV3 are considerably higher than legal trucks.

Fig. 10.1 also highlights the large increase in the expected maximum load effect due to the multiple presence of trucks simultaneously with EV as the span length increases. This is clearly caused by the higher probability of having multiple heavy trucks in combination with EV as the span length increases.

Fig. 10.2 shows the results for expected maximum shear in simple spans for combination of EV with truck data collected at NY WIM site in a single lane. The trends are similar to those observed in Fig. 10.1.

Figs. 10.3 and 10.4 show the results for two-lane bridges. Unlike the results for single lanes, the results for two-lanes show that EV2 plus random trucks dominate the cases where only random trucks cross the bridge. This is because for multilane bridges on high ADTT congested NY WIM site there is very high probability that EV2 (as well as EV3) will cross the bridge simultaneously with several other heavy trucks and the probability of having several random trucks heavier than EV2 simultaneously on the bridge is lower than the probability of having EV2 plus other random heavy trucks.

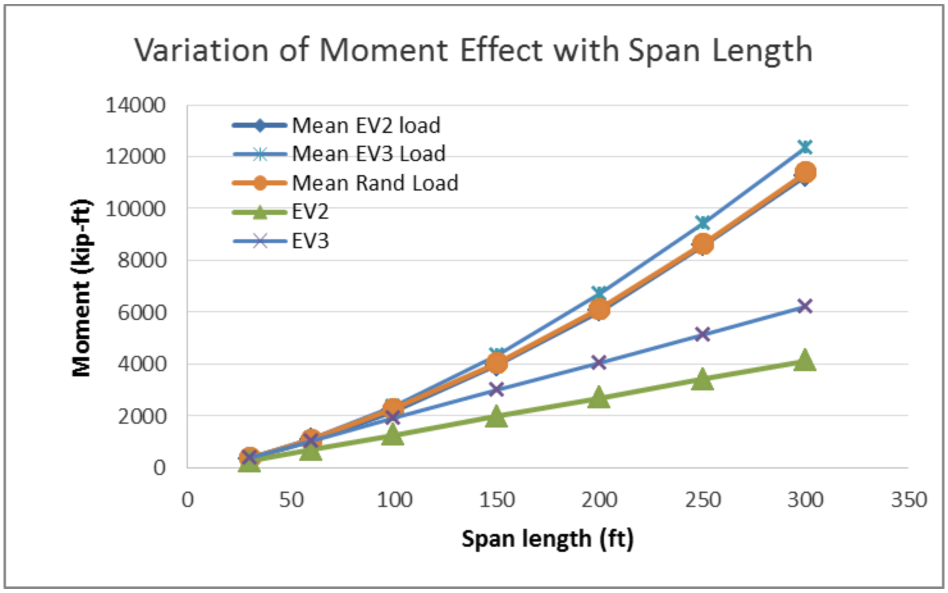

Figure 10.1 – Maximum positive moment in simple span single lane bridges for EV and combinations of EV with NY WIM truck data set

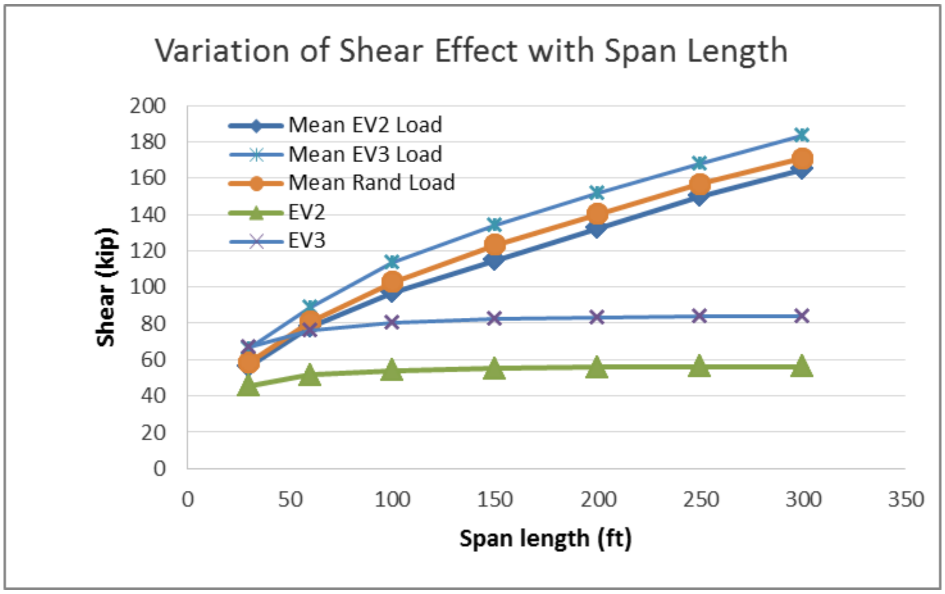

Figure 10.2 – Maximum shear in simple span single lane bridges for EV and combinations of EV with NY WIM truck data set

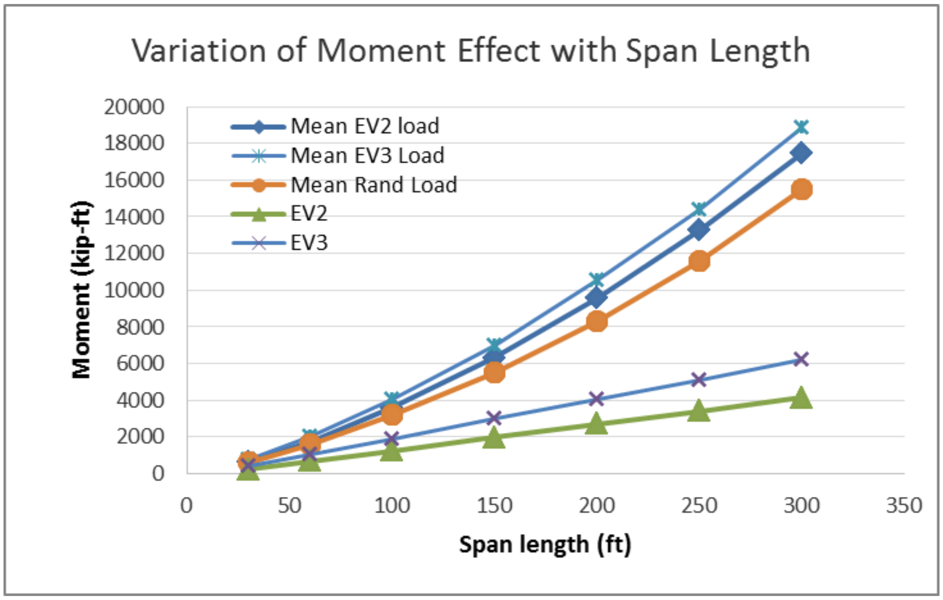

Figure 10.3 – Maximum positive moment in simple span two-lane bridges for EV and combinations of EV with NY WIM truck data set

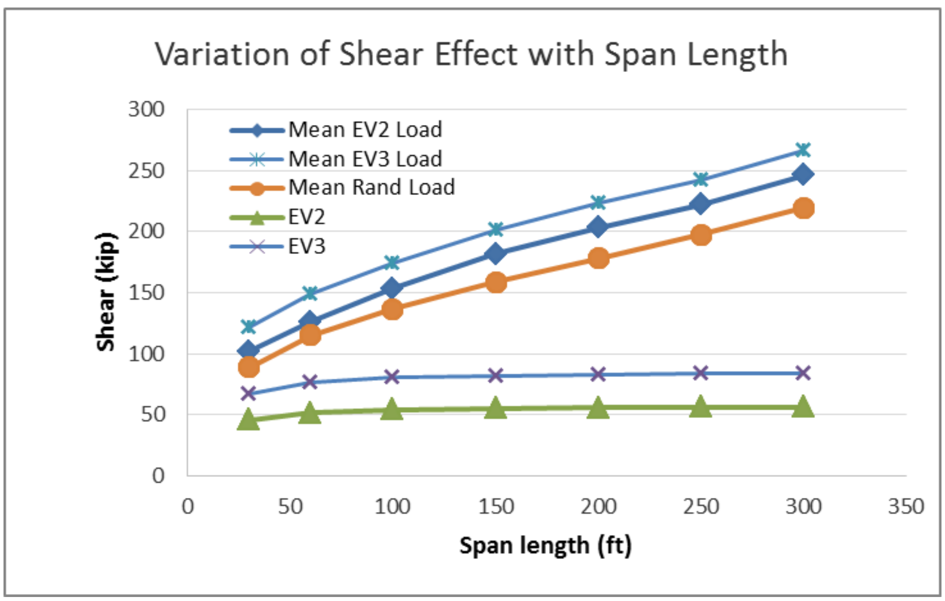

Figure 10.4 – Maximum shear in simple span two-lane bridges for EV and combinations of EV with NY WIM truck data set

For multilane loadings, the normal probability plots will give the total load on the bridge which consists of the sum of *EV+lane+LL2*. This sum is not directly applicable for the analysis of multi-girder bridges because it does not take into consideration the way the total load is distributed to each girder. Thus, a de-aggregation process is necessary to separate the load in the main lane represented by *EV+lane* from the load in the adjacent lane represented by *LL2*. In this study, the de-aggregation is executed by the simple subtraction of the mean maximum total load from the maximum one lane load both of which are evaluated from the same number of multiple presence events N which is the number of events for multi-lane loading obtained by the multiplication of the number of EV crossings within the 5-year rating period times the percentage of multiple loading events as listed in Tables 5.1 through 5.3 for the three WIM sites analyzed in this study.

In addition to the expected maximum load effect from the simulation, the reliability analysis requires the consideration of the uncertainties in the maximum load estimate. These uncertainties are represented by the three variables λ_{Lmax}, λ_{data}, and $\lambda_{site\text{-}to\text{-}site}$. If the mean and standard deviation of the maximum load are obtained from the normal probability plots as explained in Section 8 earlier, then the COV of λ_{Lmax} is directly calculated to reflect the uncertainties in the estimation of the maximum load effect. For example, Fig. 10.5 gives the COV for λ_{Lmax} as calculated using the statistical extrapolations of maximum moment effects on single span bridges for the combination of EV with NY WIM data. The figure shows that the COV for λ_{Lmax} varies within the range of 3 to 6%. Previous sensitivity analyses such as those performed in NCHRP 368 have indicated that λ_{data} can approximately have a COV on the order of 2% to 5%. A value of 5% is adopted in this study. Finally, the variability in the results between sites having the same ADTT but accounting for different truck weight histograms is represented by $\lambda_{site\text{-}to\text{-}site}$, which in NCHRP Report 20-07 Task 285 have shown variations in COV with a conservative estimate on the order of 20% which is the value adopted in this study (Sivakumar and Ghosn, 2011).

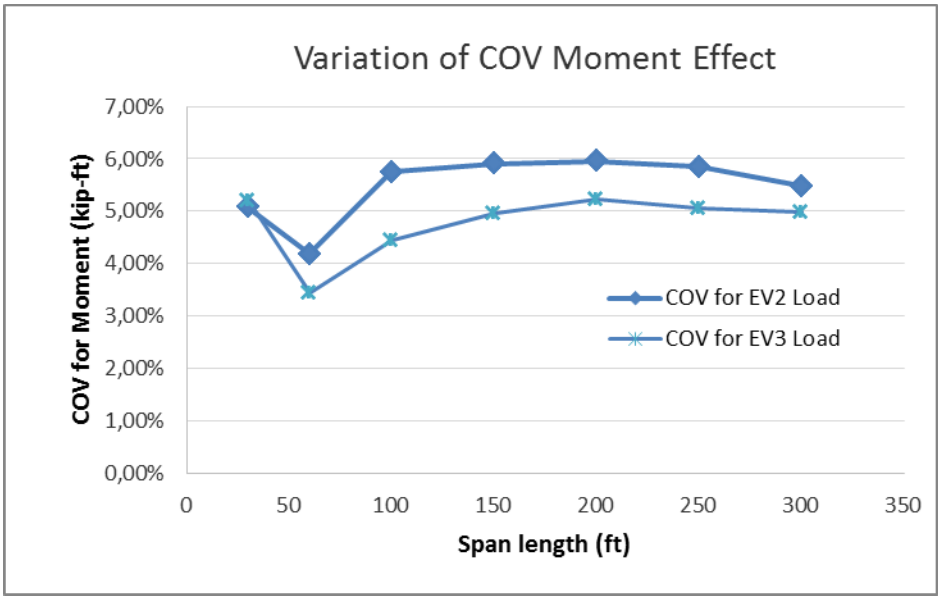

Figure 10.5 – COV for λ_{Lmax} in simple span two-lane bridges for combinations of EV with NY WIM truck data set

Variability in Dynamic Amplification Factors

The AASHTO LRFD specifies that a nominal dynamic amplification IM=1.33 be used on the truck load effect to account for the increased stresses due to the vibrations of the bridge under moving loads. However, it has been well established that this value is a conservative upper bound. On the average, Nowak (1999) indicates that heavily loaded trucks usually produce lower values than the nominal value with a mean value IM = 1.13 and a COV of VIM=9% for individual truck crossings. For simultaneous crossings in multi-lanes the average is IM=1.09 and the COV is VIM=5.5%.

Variability in AASHTO Load Distribution Factors

To simplify the analysis process, the load distribution factors DF_1 and DF_2 in Eq. (20) can be calculated using the AASHTO LRFD load distribution tables after taking out the multiple presence factor mp=1.2 already included in the tabulated AASHTO load distribution factors for single lanes. For bending of typical reinforced and prestressed concrete and steel girder bridges loaded by one lane of traffic, the LRFD load distribution factor equation for single lane after removing the multiple presence factor is given as (AASHTO, LRFD, 2017):

$$DF_1 = \left(0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \right) \Big/ 1.2 \tag{21}$$

Where S is the beam spacing, L is the span length, t_s is the deck thickness, and K_g is a beam stiffness parameter. Not having enough information to calculate the term within the last parenthesis, it is taken as 1.0 as recommended in the AASHTO LRFD specifications (2017). Note that the original equation in the AASHTO Specifications already includes a multiple presence factor mp=1.2 which accounts for the higher probability of having one heavy truck in one lane as compared to the probability of having two side-by-side heavy trucks in two adjacent lanes. The two side-by-side trucks are used in the AASHTO as the base line for the design and rating of bridges. Because in this study we calculated L_{max} directly, the multiple presence factor will have to be removed when calculating the maximum applied live load as shown in Equation 21.

For two lanes loaded, the load distribution factor equation for bending becomes (AASHTO, LRFD, 2017):

$$DF^* = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{22}$$

According to the AASHTO LRFD, Equation 22 reflects the load on one beam under the effects of two-lane loadings. If Eq. 21 reflects the effect of the main loading lane, and DF^* reflects the effects of two lanes, then the AASHTO MBE recommends using the following approximation to evaluate the effect of the second lane load only:

$$DF_2 = DF^* - DF_1 \tag{23}$$

The approximation of Eq. 23, adopted from the AASHTO LRFD specifications (LRFD A4.6.2.2.5—Special Loads with Other Traffic), is used in this report to evaluate the effects of two lane loadings on the most critical bridge members for lack of better models.

The distribution factor for shear in typical reinforced and prestressed concrete and steel girder bridges for one lane loaded after removing the multiple presence factor is given by the AASHTO LRFD as:

$$DF_1 = \left(0.36 + \frac{S}{25} \right) \Big/ 1.2 \tag{24}$$

For two lanes loaded, the LRFD shear distribution factor becomes:

$$DF^* = 0.20 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \tag{25}$$

The load distribution factors adopted in the AASHTO Bridge Design Specifications (2017) have been derived with a conservative bias factor on the order of 10% as explained by Zokaie et al. (1991). It has also long been recognized that the AASHTO load distribution factors are conservative producing higher load effects than those obtained from refined structural analysis and from those measured in the field. In this case, the mean distribution factor can be obtained by dividing the AASHTO values with the ratios calculated by Puckett et al (2007) leading to the set of biases and COV´s in Table 4 for multi-girder steel, prestressed concrete and reinforced concrete girder bridges. Table 10.1 shows that the AASHTO LRFD load distribution tables can over-predict the actual *DF* by as much as 29% for single lanes and 11% for two loaded lanes of multi-girder steel and prestressed concrete bridges. The variability of the AASHTO LRFD values is also high averaging around 14% with a range of 11% to 18%.

Table 10.1 - Comparison of field measured load distribution to AASHTO DF

		λ_{DF} = Field results/ASSHTO DF			
		MOMENT		SHEAR	
BRIDGE TYPE	STATISTIC	1 LANE	2 LANES	1 LANE	2 LANES
A	AVERAGE	0.78	0.90	0.72	0.82
	COV	11%	14%	14%	18%
E	AVERAGE	0.79	0.93	0.76	0.88
	COV	16%	15%	12%	18%
K	AVERAGE	0.78	0.90	0.77	0.88
	COV	12%	13%	11%	16%

Model for Refined Analysis

For the purposes of this study and based on the observations made by Shwarz & Laman (2001), Barr et al (2001) as well as Jamboktar (2006), it will be assumed that if the EV rating is executed using a refined finite element or grillage analysis that has been carefully calibrated and "tuned" so that the distribution factor obtained from the analysis is on the average similar to the one actually experienced by the bridge, then the mean value of λ_{DF} will be taken as 1.0 with a COV equal to Van=8% (Sivakumar and Ghosn, 2011). It will also be assumed that the load distribution factor found from the refined analysis will follow a Normal probability distribution.

Permanent Load Model

Following the approach adopted by Nowak (1999), the total permanent load effect, DL is divided into the dead load of pre-fabricated members, DC_1, the dead load of cast-in-place members, DC_2, and the dead load of the wearing surface, DW, such that the mean total dead load is given by:

$$\overline{DL} = \overline{DC}_1 + \overline{DC}_2 + \overline{DW} \tag{26}$$

The standard deviation of the total dead load, σ_{DL}, is expressed as a function of the standard deviations of each dead load component:

$$\sigma_{DL} = \sqrt{\sigma^2_{DC_1} + \sigma^2_{DC_1} + \sigma^2_{DW}} \tag{27}$$

The relationship between the standard deviation, σ_{DL}, mean value, $\overline{DL}$, and the coefficient of variation (COV) of the dead load, V_{DL}, is obtained as:

$$V_{DL} = \frac{\sigma_{DL}}{\overline{DL}} \tag{28}$$

Following Nowak (1999), the dead load effects are assumed to follow Normal probability distributions where the mean values and the COV's of each dead load component are given as:

$$\begin{aligned} &\overline{DC}_1 = 1.03DC_1 && V_{DC_1} = 8\% \\ &\overline{DC}_2 = 1.05DC_2 && V_{DC_2} = 10\% \\ &\overline{DW} = DW && V_{DW} = 25\% \end{aligned} \tag{29}$$

Where DC_1, DC_2 and DW are respectively the nominal values of the dead load of pre-fabricated members, cast-in-place members, and wearing surface. Tables 4.1 and 4.2 provide typical nominal values for the moment effect of each dead load component for members of a typical set of simple span and continuous two-span prestressed concrete, composite and reinforced concrete multi-girder bridges. The base lines of these data are obtained from Nowak (1999) and extended via linear regression to cover a range of bridge spans varying between 30-ft to 300-ft.

Resistance Model

The methods used to calculate the moment and shear capacities of bridge members have been found to lead to conservative estimates of the actual capacities. The data used by Nowak (1999) to account for the biases and the variability in the existing current member analysis methods are summarized as follows:

$$
\begin{array}{lll}
\text{Bending capacity of composite steel beams;} & \overline{R} = 1.12 R_n & V_R = 10\% \\
\text{Bending of prestressed concrete beams;} & \overline{R} = 1.05 R_n & V_R = 7.5\% \\
\text{Bending of reinforced concrete beams;} & \overline{R} = 1.14 R_n & V_R = 13\% \\
 & & \\
\text{Shear capacity of composite steel beams;} & \overline{R} = 1.14 R_n & V_R = 10.5\% \\
\text{Shear capacity of prestressed concrete beams;} & \overline{R} = 1.15 R_n & V_R = 14\% \\
\text{Shear capacity of reinforced concrete beams;} & \overline{R} = 1.20 R_n & V_R = 15.5\%
\end{array}
\qquad (30)
$$

11. RELIABILITY ANALYSIS EXAMPLE

In this example, we illustrate the application of Eq. (19) in combination with Eq. (20) for the reliability analysis of the 200-ft simple span bridge loaded with EV3 in a multi-lane loading scenario. The reliability analysis is executed for the same Example 2 of Section 7 which is analyzed a composite steel bridge that achieved a rating factor RF=1.0 with a live load factor γ_{EV}=1.25 for EV3 LRFR rating when using the format of Eq. (6). The two-lane 200-ft simple span composite steel girder bridge has 6 beams at 8-ft spacing.

When studying the loads on a two-lane bridge member, the entire live load is obtained using the live load modeling procedure outlined in Section 8 to give the entire expected maximum load which can be represented by the sum $EV+lane+LL_2$. The simulations performed in this study were developed for the load on the entire bridge. In order to include the effects of the load distribution which are different for the trucks in lane 1 than those in lane 2, a de-aggregation process is needed to separate $EV+lane$ of lane 1 from LL_2 of lane 2. In this study, this effected by performing the simulation for the entire load on two lanes and then subtracting the results of the simulation from the load in lane 1 to obtain an estimate of the load LL_2. This approach is conservative as it considers that the main contributions of the load are from lane 1 which is associated with the highest load distribution factor.

For the two-lane 200-ft simple span bridge, the expected maximum moment obtained using the simulations described in Section 8 using the New Jersey WIM data was found to be 8777 kip-ft. The maximum load effect in one lane was found to be 6329 kip-ft. This implies that the multiple presence factor mp=1.40 (8777/6329). This mp reflects the number of side-by-side loading events that may occur when EV is crossing a 200-ft bridge. It is noted that the effect of EV3 on a 200-ft simple span bridge is 4058 kip-ft. This indicates that the lane effect is 2271 kip-ft (6329-4058) or 56% of the effect of EV3. The corresponding distributed load becomes 0.45 kip/ft.

The calculations in Section 8 show that the expected maximum live load will be associated with a COV for λ_{Lmax}=4.5%. A variation in the estimate with a COV λ_{data}=5% is used to reflect variations on the expected maximum load associated with the method and the number of data points used to perform the regression fit applied as shown in Fig. 8.6. Finally, based on previous work that studied a large number of WIM data sites, it is estimated that the variations due to changes in the WIM data can be associated with $\lambda_{site\text{-}to\text{-}site}$ on the order of 20%.

DF_1 in Eq. (20) is equal to the one-lane distribution factor in the AASHTO LRFD tables after removing the 1.2 multiple presence factor embedded in the single lane AASHTO distribution factors. Using Eq. (21)

we obtain DF_1=0.30. Using Eq. (22) the load distribution factor for two-lane loading is $DF^*=0.55$. DF_2 in Eq. (20) is obtained by subtracting DF_1 from DF^* leading to DF_2=0.25.

To obtain the mean values of the load distribution factors in Eq. (8), Nowak (1999) and Moses (2001) assume that the *DF* values given by the AASHTO LRFD Specifications are the actual mean values of the distribution factors. This is not strictly speaking correct because the LRFD equations for the distribution factors include some level of conservatism as explained by Zokaie et al. (1991). Therefore, the random variable λ_{DF} will have the mean value and COV shown in Table 10.1.

In this study, we assume a lognormal model for the resistance, *R*, as well as a lognormal model for the combined effects of all the applied loads represented by the variable, *S* in Eq. (19). Previous studies by Sivakumar and Ghosn (2011) during the re-calibration of the AASHTO MBE showed that Eq. (19) yields results very close to those obtained using a First Order Reliability Method (FORM).

The live load data obtained using the results obtained using the method illustrated in Figure 8 are used to analyze the reliability of a 200-ft steel continuous bridge member in bending. The bridge is assumed to have six beams at 8-ft spacing loaded by two lanes of traffic which mixes random trucks with EV3. The reliability analysis is performed for the emergency vehicle rating of a beam that gives a R.F. exactly equal to 1.0 when evaluated for different live load factors γ_{EV} in Eq. (1). The bending moment calculated above for EV3 plus random trucks in two lanes is found to be equal to 1054 kip-ft and for one lane the expected maximum load is 6634 kip-ft. Note that a slightly lower mean value is obtained here for the one lane loading as compared to the one calculated from the data in Figure 8 because the value of N=3595 used in these calculations corresponds to the number of crossings in multiple lanes because this example analyzes crossings in two lanes. The one lane loading is used in this example to deaggregate the total load into two parts, the one that crosses in Lane 1 and the portion from Lane 2.

Design Input Data:

Nominal dead load effect:

$$DC_1 = 3593 kip - ft$$
$$DC_2 = 4790 kip - ft \quad \text{(Table 1)}$$
$$DW = 1083 kip - ft$$

Nominal live load effect of EV3: $EV = 4058\ kip - ft$

Design Dynamic Amplification: $IM = 1.33$

LRFD Load Distribution Factor for one lane after removing multiple presence:

$$DF_1 = 0.36/1.2 = 0.30$$
$$DF_2 = 0.55 - 0.30 = 0.25$$

Nominal Resistance: $R_n = 15695 kip - ft$

Rating Factor: $RF = 1.0$

In this example, the reliability of the bridge member for the case when the rating factor RF=1.0 is calculated when the live load factor is set at γ_{EV}=1.32 for EV3 + legal load as detailed in Example 2 of Section 7. If the reliability index, β, is equal to the target β_{target}=2.50, then the live load factor is considered to be adequate.

Reliability Analysis Input Data:

Mean resistance: $\overline{R} = b_R R_n = 1.12 \times 15695 = 17578\ kip - ft$

COV of resistance: $V_R = 10\%$

Mean dead load:

$$\overline{DC}_1 = b_D DC_1 = 1.03 \times 3593 = 3700\ kip - ft$$
$$\overline{DC}_2 = b_D DC_1 = 1.05 \times 4790 = 5030\ kip - ft$$
$$\overline{DW} = b_D DW = 1.00 \times 1083 = 1083\ kip - ft$$

Standard deviation of dead load :

$$\sigma_{DC_1} = V_D \times \overline{DC}_1 = 8\% \times 3700 = 296\ kip - ft$$
$$\sigma_{DC_2} = V_D \times \overline{DC}_2 = 10\% \times 5030 = 503\ kip - ft$$
$$\sigma_{DW} = V_D \times \overline{DW} = 28\% \times 1083 = 271\ kip - ft$$

Mean of total live load L_{max} in lane 1: $\overline{L}_{\max lane1} = \overline{EV + lane} = 6634 \times 0.30 = 1990\ kip - ft$

Mean of total live load L_{max} in lane 2: $\overline{L}_{\max lane2} = \overline{LL}_2 = (10544 - 6634) \times 0.25 = 978\ kip - ft$

COV of L_{max}: $V_{L_{Max}} = 5.24\%$

COV site-to-site: $V_{site-to-site} = 20\%$

COV data limitation: $V_{data} = 5\%$

Mean Dynamic Amplification: $\overline{IM} = 1.10$

COV Dynamic Amplification: $V_{IM} = 5.5\%$

Mean of Random Live Load Effect:

$$LL_T = \left(\left(\overline{EV + lane}\right) \times DF_1 + \overline{LL}_2 \times DF_2\right) \times \overline{IM} \times \lambda_{L\max} \times \lambda_{site-to-site} \times \lambda_{data} \times \lambda_{DF}$$

$$\overline{LL}_T = \left(6634 \times 0.30 + 3910 \times 0.25\right) \times 1.10 \times 1.0 \times 1.0 \times 0.90 = 2938\, kip - ft$$

COV of Applied Random Live Load:

$$V_{LT} = \sqrt{V_{L_{\max}}^2 + V_{site-to-site}^2 + V_{data}^2 + V_{IM}^2 + V_{DF}^2} = \sqrt{(4.24\%)^2 + (20\%)^2 + (5\%)^2 + (5.5\%)^2 + (14\%)^2} = 26\%$$

Standard deviation for live load: $\sigma_L = \overline{LL}_T \times V_{LT} = 2938 \times 26\% = 764\, kip - ft$

Mean of total load $\overline{S} = \overline{D} + \overline{LL}_T = 3700 + 5030 + 1083 + 2938 = 12751\, kip - ft$

Standard deviation for total load: $\sigma_S = \sqrt{\sigma_D^2 + \sigma_L^2} = \sqrt{296^2 + 503^2 + 271^2 + 764^2} = 999\, kip - ft$

COV of total load: $V_S = \dfrac{\sigma_S}{\overline{S}} = \dfrac{999}{12751} = 7.8\%$

Find reliability index assuming Lognormal Model:

$$\beta = \frac{\ln\left(\dfrac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\dfrac{17578}{12751}\right)}{\sqrt{(10\%)^2 + (7.8\%)^2}} = \frac{0.321}{0.127} = 2.53$$

The calculated reliability index β=2.53 is higher than the target reliability index β_{target} which was set at 2.50. This indicates that for two lane simple steel bridges of 200 ft in length, the target reliability index is met (slightly exceeded) when using the LRFR format of Eq. (6) the bridge member achieves a rating factor *RF*=1.0 where the live load factor used with EV3 is set at γ_{EV}=1.32.

Similar calculations to those described in this section were performed for all the bridge configurations listed in Section 4. For example, Fig. 11.1 shows the reliability index values obtained for the analysis of the bending moment for all two-lane simple span bridges loaded by EV3 in combination with the truck data extracted from the NY site. The analysis is for the bridges that produce a rating factor *RF=1.0* when using equation 6 with the live load factor γ_{EV}=1.32. Fig. 11.2 shows the reliability indexes for shear. Both figures show the tight spread around the target reliability index β_{target}=2.50, although slightly lower values are obtained for bending in short spans on the order of 30 ft in length and shear in long spans on the order of 150 ft or more.

For the analysis of simple span bridges, when γ_{EV}=1.32 the target reliability index is achieved when the distributed load in Eq. (6) *lane* is equal to zero.

Fig. 11.3 and 11.4 show the reliability index values obtained for continuous bridges loaded with EV3 in combination with trucks extracted from NY WIM data set. In this case, achieving the target reliability β_{target}=2.50 for bridges that produce a rating factor *RF*=1.0 when using Eq. (6) required the application of a live load factor γ_{EV}=1.25 when the distributed load in Eq. (6) *lane* is equal to 0.20 kip/ft. Applying a lane load is needed to account for the possibility of having other trucks ahead or behind EV plus the truck adjacent to it. Fig. 11.3 and 11.4 illustrate the tight spread of the reliability index values around the target β_{target}=2.50 with a small reduction for bending moments for continuous bridges with individual spans on the order of 50-ft.

Similar calculations to those described in this section were performed to calibrate appropriate live load factors and lane load intensities for a wide range of cases as will be explained in Section 12.

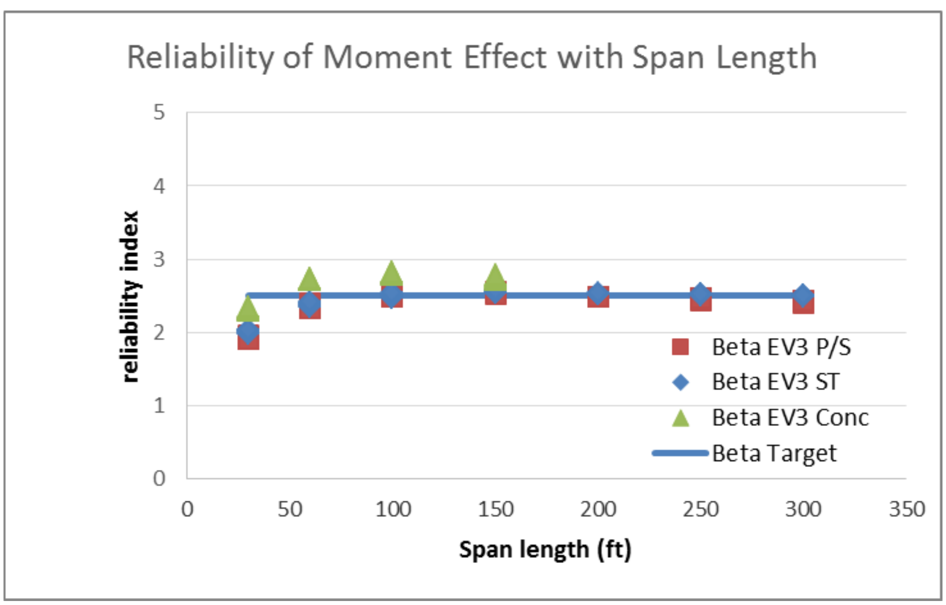

Figure 11.1 – Reliability index values for bending in simple span multi lane bridges loaded by EV3 in combination with trucks from NY WIM data set when RF=1.0 using Eq. (6) with γ_{EV}=1.32.

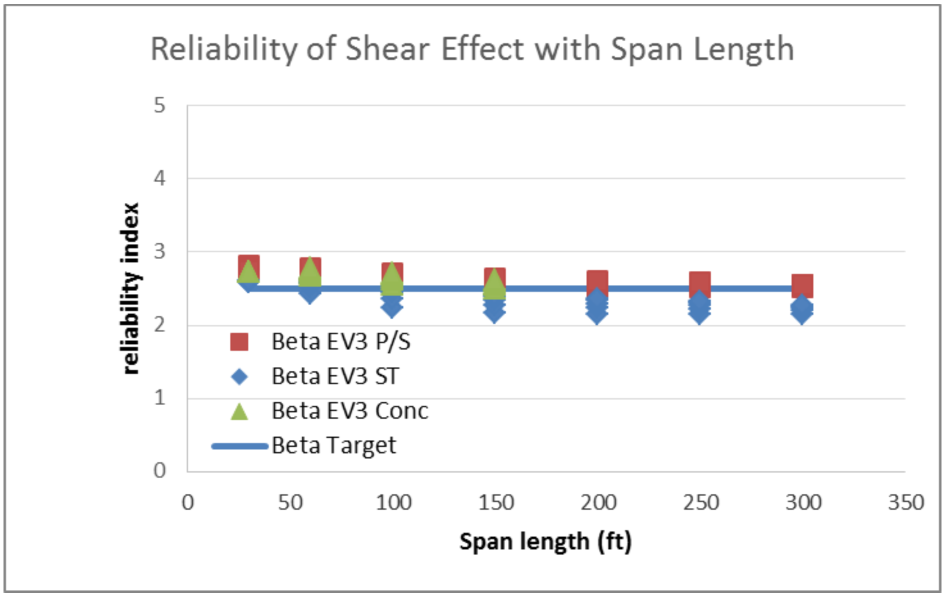

Figure 11.2 – Reliability index values for shear in simple span multi lane bridges loaded by EV3 in combination with trucks from NY WIM data set when RF=1.0 using Eq. (6) with γ_{EV}=1.32.

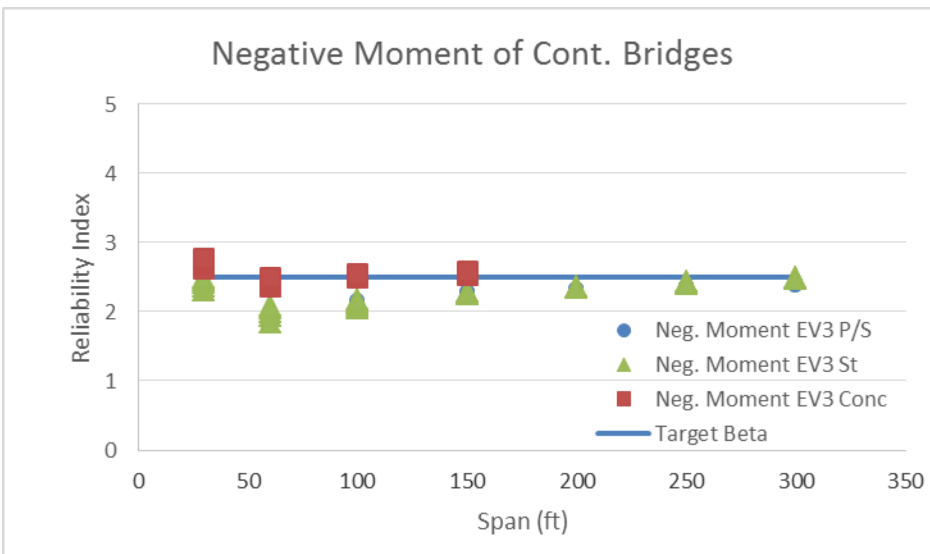

Figure 11.3 – Reliability index values for negative bending in continuous span multi lane bridges loaded by EV3 in combination with trucks from NY WIM data set when RF=1.0 using Eq. (6) with γ_{EV}=1.25 and *lane*=0.20 kip/ft.

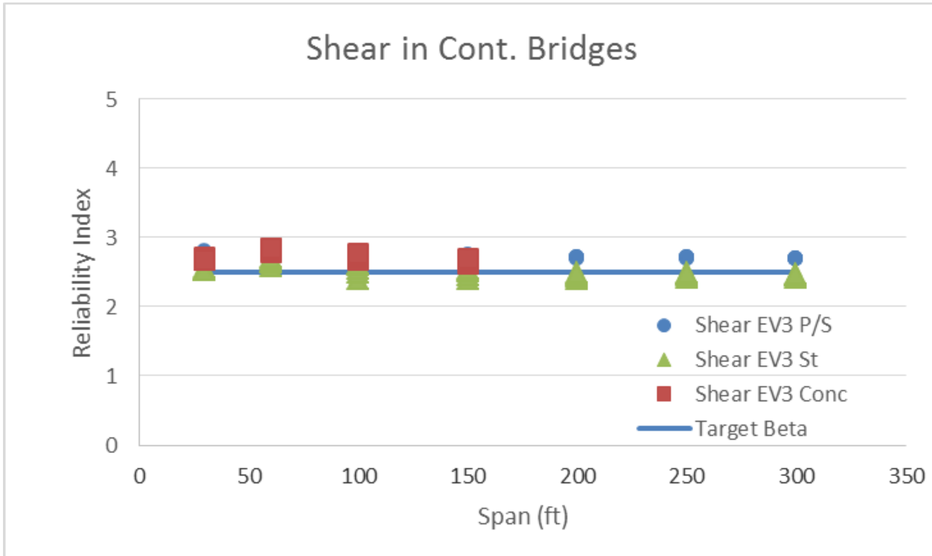

Figure 11.4 – Reliability index values for shear in continuous span multi lane bridges loaded by EV3 in combination with trucks from NY WIM data set when RF=1.0 using Eq. (6) with γ_{EV}=1.25 and *lane*=0.20 kip/ft.

12. RELIABILITY CALIBRATION OF EV LRFR EQUATIONS

Cases Considered

The calibration of EV LRFR Equations has been performed in this study using 18 alternative analyses to cover the following cases:

a. Three WIM sites:
   i. New York City I-95 representing high ADTT traffic in congested urban conditions
   ii. New Jersey I-95 Turnpike representing high ADTT traffic in free flowing conditions
   iii. Idaho I-90 site representing low ADTT traffic in rural area
b. Average Number of EV crossings:
   i. 1 crossing per day for low emergency regions as described below
   ii. 10 crossings per day to cover regions subject to high emergency calls as described below
c. Two alternative EV LRFR formats
   i. Format of Eq. (5) with nominal loading using the LRFD load distribution tables. In this case, multiple truck presence is implicitly accounted for in the live load factor and the load distribution factor.
   ii. Format of Eq. (6) where traffic in adjacent lanes is accounted by applying the EV in one lane and the Legal Truck in the adjacent lane. Two sub-cases are considered:
      1. Using adjusted AASHTO distribution factors in Eq. 6
      2. Using a refined structural analysis of the bridge where the load effect is obtained by placing EV in one lane and a legal vehicle is placed in the second lane.

Number of EV Crossings per Bridge

In this study, two scenarios for EV crossings are considered. On the lower end, it is assumed that a bridge will on the average be crossed by an EV once per day. The other scenario considers that a bridge can be crossed by an EV ten times per day. These scenarios are based on several sources.

A document published by the International Association of Fire Chiefs Association entitled: *Emergency Vehicle Size and Weight Regulation Guideline* indicates that on average, an EV such as a fire apparatus travels about 5,000 miles per year. This comes down to roughly 14 miles per day or seven miles each way. Also, according to the American Transportation and Road Builders Association´s web https://www.artba.org/about/faq/ the USA has 4.12 million center-line miles of roads and the NBI files indicate over 600,000 US bridges. This comes down to roughly 7 miles of roadway between bridges or each individual EV may on the average cross one bridge per day on the way out and another on the way back. Of course, different emergency trips will not necessarily cross the same bridge, and the number of bridges crossed will be higher in urban areas. However, if this bridge is located next to the emergency station, the likelihood that every trip will cross the same bridge becomes very high. Also, the more bridges EV's will cross, the higher are the chances that at least one of these bridges will be exposed to the simultaneous presence of EV with other very heavy vehicles that may undermine its safety.

Depending on the type of emergency and the intensity of the fire, one call may require dispatching several EV´s. For example, Table 5 depicts the number of vehicles that the New York Fire Department (NYFD) dispatches for various emergency calls. The New York Post (December 10, 2015 issue) reports that of the 1,167,974 calls to the FDNY in 2014, 61,952 (about 5 percent) were for structural fires and another 226,724 runs were for other calls. Given that NYFD operates about 340 ladder and engine companies and assuming a structural fire leads to five EV runs and the other calls require an average of 1.3 EV runs, then each EV in New York City would undergo an average of 5 runs per day. A national run survey for the year 2016, http://media.cygnus.com/files/base/FHC/document/2017/06/NRS2016-Part2.pdf shows that the busiest 100 US fire engine companies easily exceed 3373 runs per year or over 9 runs per day with the busiest engine in San Francisco reaching up to 30 runs per day. Thus, bridges near busy fire stations could possibly be exposed to ten or more crossings of EV's per day.

Table 12.1 – Number of apparatuses per New York City emergency run

http://www.fdnewyork.com/aa.asp	
FDNY Dispatch Policy	
1 Engine	brush fires
	downed wires
	CFR runs
	medical alert central station alarms
	other non-structural outside fires
1 Ladder	water leaks
	downed trees/limbs blocking the street
	loose or hanging cornice
	stuck occupied elevators
1 Engine + 1 Ladder	car fires
	struck pedestrian
	vehicle collisions
	automatic alarms (CO, smoke, fire) in a private residence
	gasoline leaks on the street
	stuck elevator with injured passenger
Box Transmission (1st alarm)	
2 engines + 2 ladders + battalion chief	natural gas leaks
3 engines + 2 ladders + battalion chief	report of a structural fire
Second alarm (signal 2-2)	
8 engines + 5 ladders + 5 battalion chiefs + other vehicles	Depends on the severity of the fire and need to support and relieve first responders

Calibration of Live Load Factors for Eq. (A) applying only EV load

The calibration of live load factor for implementation in Eq. (A) which is the same as Eq. (5) are obtained as shown in Table 12.2. Two sets of factors are calculated. The first set is for bridges expected to be exposed to 10 EV crossings per day and the second is for bridges in rural regions where a bridge is expected to be exposed to 1 EV crossing per day. These results are valid for multilane girder bridges having spans up to 300-ft in length. Multi-lane simple span bridges should be analyzed placing one EV in the most critical position on the bridge combination with the multi-lane AASHTO LRFD load distribution factor. Continuous bridges should be analyzed for EV plus a lane load equal to 0.20kip/ft. This value of 0.20 kip/ft is chosen for consistency with the lane load recommended in the current AASHTO MBE for continuous spans. Simple span bridges will be analyzed without lane load. Single lane bridges will be analyzed using the AASHTO single lane load distribution factor after removing the embedded multiple presence factor *mp*=1.20.

The results in Table 6 are consistent with the load factors in the AASHTO LRFR MBE where the heavier EV3 (similar to heavy permit loads) will be associated with lower live load factors. The lower live load factor reflects the lower probability that the heavier EV3 will be crossing the bridge alongside an even heavier random truck. The chances that the lighter EV2 will be crossing the bridge alongside a truck that produces a load effect higher than that of the EV2 is higher. Table 6 gives the values calibrated to meet the average the target reliability index β_{target}=2.5 with the condition that none of the cases analyzed yields a reliability β lower than 1.50. Table 6 shows that in some cases there is very low probability that random trucks as heavy as or heavier than the emergency vehicle will travel simultaneously on the bridge with the emergency vehicle leading to live load factors less than 1.0. The recommended live load factors are presented in Table A in a format consistent with that of the AASHTO MBE, LRFR. The recommended final values are adjusted such that no live load factor is set at a value lower than 1.10 as a conservative lower limit.

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left(EV + lane + IM\right) \times DF} \tag{A}$$

Table 12.2 – Calibrated live load factors, γ_{EV}, for implementation in Eq. (A) in combination with 2-lane Distribution Factor

10 crossings per day		EV2	EV3
Idaho site (ADTT=600)	Simple span without lane load	1.09	0.95
	continuous span with 0.2kip/ft lane	0.91	0.83
New Jersey site (ADTT=6000)	Simple span without lane load	1.42	1.11
	continuous span with 0.2kip/ft lane	1.21	1.00
New York site (ADTT=5500)	Simple span without lane load	1.53	1.18
	continuous span with 0.2kip/ft lane	1.33	1.08

only 1 crossing per day		EV2	EV3
Idaho site (ADTT=600)	Simple span		
	continuous span with 0.2kip/ft lane		
New Jersey site (ADTT=6000)	Simple span	1.23	0.98
	continuous span with 0.2kip/ft lane	1.07	0.86
New York site (ADTT=5500)	Simple span	1.32	1.03
	continuous span with 0.2kip/ft lane	1.14	0.93

Calibration of LRFR Live Load Factors for Eq. (B) --- Approximate Analysis applying EV plus Legal Truck load

Using the reliability analysis steps described in the Introduction, the calibration of live load factor for implementation in Eq. (B) which is the same as Eq. (6) are obtained as shown in Table 7. Two sets of factors are calculated. The first set is for bridges expected to be exposed to 10 EV crossings per day and the second is for bridges in rural regions where a bridge is expected to be exposed to 1 EV crossing per day. These results are valid for multilane girder bridges having spans up to 300-ft in length.

Multi-lane simple span bridges should be analyzed by placing one EV in the most critical position on one lane and in only one span of the bridge in combination with the adjusted single lane AASHTO LRFD load distribution factor and one AASHTO legal truck load in the other lane with the adjusted second lane AASHTO distribution factor as described in Example 2 of Section 7. Continuous bridges should be analyzed for EV plus a lane load equal to 0.20kip/ft in the lane occupied by EV. This value of 0.20 kip/ft is chosen for consistency with the lane load recommended in the current AASHTO MBE for continuous spans. Simple span bridges will be analyzed without the lane load. Single lane bridges will be analyzed using the AASHTO single lane load distribution factor after removing the embedded multiple presence factor *mp*=1.20.

The results in Table 12.3 are consistent with the load factors in the AASHTO MBE LRFR where heavier EV3 will be associated with lower live load factors. The lower live load factor reflects the lower probability that the heavier EV3 will be crossing the bridge alongside an even heavier random truck. The chances that the lighter EV2 will be crossing the bridge alongside a truck that produces a load effect higher than that of the EV2 is high. Table 6 gives the values calibrated to meet on the average the target reliability index β_{target}=2.5 with the condition that none of the cases analyzed yields a reliability β lower than 1.50. The results in Table 12.3 show that in a few continuous span cases the target reliability is achieved with a live load factor less than 1.0. This is because the probability of having a heavy truck equal to the legal truck in addition to other trucks simultaneously on the bridge along with the emergency vehicle is rather low. The recommended live load factors are presented in Table B in a format consistent with that of the AASHTO LRFR MBE. The recommended final values are adjusted such that no live load factor is set at a value lower than 1.10 a conservative lower value.

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV} \left[(EV + lane) DF_{EV} + Legal \times DF_{Lgl} \right] IM} \qquad \text{(B)}$$

Table 12.3 – Calibration of live load factors for placing EV in combination with adjacent Legal Truck using AASHTO DF in Eq. (B)

10 crossings per day		EV2	EV3
Idaho site (ADTT=600)	Simple span	1.10	1.06
	continuous span with 0.2kip/ft lane	0.96	0.96
New Jersey site (ADTT=6000)	Simple span	1.36	1.22
	continuous span with 0.2kip/ft lane	1.29	1.13
New York site (ADTT=5500)	Simple span	1.50	1.32
	continuous span with 0.2kip/ft lane	1.40	1.25

only 1 crossing per day		EV2	EV3
Idaho site (ADTT=600)	Simple span		
	continuous span with 0.2kip/ft lane		
New Jersey site (ADTT=6000)	Simple span	1.19	1.10
	continuous span with 0.2kip/ft lane	1.13	0.99
New York site (ADTT=5500)	Simple span	1.30	1.15
	continuous span with 0.2kip/ft lane	1.20	1.08

A comparison between the member moment capacities required to obtain RF=1.0 when using Eq. A with the load factors of Table 12.2 and the moment capacities required to obtain RF=1.0 when using Eq. B with the load factors of Table 12.4 is executed as shown in Figure 13. The plots conform that the use either of the two recommended formats on the same bridge will yield the same rating factor RF=1.0 for the cases where the bridges load carrying capacities are found to be satisfactory.

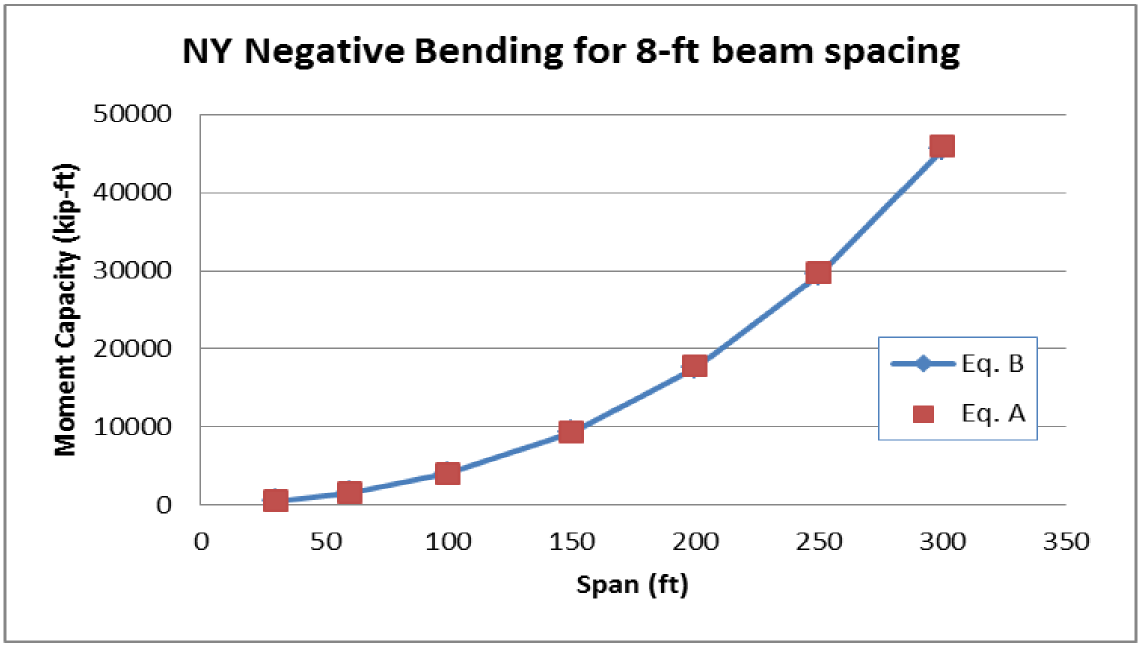

Figure 12.1 – Comparison of Bending Moment Capacities for Simple Span Bridge Members that Satisfy Eq. (A) and Eq. (B) with RF=1.0.

Calibration of LRFR Live Load Factors for Eq. (B) with Refined Structural Analysis

Using the reliability analysis steps described in the Introduction, the calibration of live load factor for implementation in Eq. (6) are obtained as shown in Table 12.4. Two sets of factors are calculated. The first set is for bridges expected to be exposed to 10 EV crossings per day and the second is for bridges in rural regions where a bridge is expected to be exposed to 1 EV crossing per day. These results are valid for multilane girder bridges having spans up to 300-ft in length.

Multi-lane simple span bridges should be analyzed by placing one EV in the most critical position on one lane and only one span of the bridge and only one AASHTO legal truck load in the other lane. Continuous bridges should be analyzed for EV plus a lane load equal to 0.20kip/ft in the lane occupied by EV. This value of 0.20 kip/ft is chosen for consistency with the lane load recommended in the current AASHTO MBE for continuous spans. Simple span bridges will be analyzed without lane load. Single lane bridges will be analyzed with only EV and no legal truck is necessary.

The results in Table 12.4 are consistent with the load factors in the AASHTO MBE LRFR where heavier EV3 will be associated with lower live load factors. The lower live load factor reflects the lower probability that the heavier EV3 will be crossing the bridge alongside an even heavier random truck. The chances that the lighter EV2 will be crossing the bridge alongside a truck that produces a load effect higher than that of the EV2 is high. Table 12.2 gives the values calibrated to meet on the average the target reliability index β_{target}=2.5 with the condition that none of the cases analyzed yields a reliability β lower than 1.50. The recommended live load factors are presented in Table 12.4 in a format consistent with that of the AASHTO LRFR MBE. The recommended final values are adjusted such that no live load factor is set at a value lower than 1.10 as a conservative lower limit.

The live load factors in Table 12.4 are about 0.10 to 0.15 higher than those in Table 12.3. This is because the AASHTO load distribution factors used in combination with the results of Table 12.3 are conservative while the refined analysis should be based on well calibrated structural models that should give results compatible with field observed bridge responses under traffic loads.

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left[(EV + lane)DF_{EV} + Legal \times DF_{Lgl}\right]IM} \quad \text{(B)}$$

Table 12.4 – Calibration of live load factors for placing EV in combination with adjacent Legal Truck using refined analysis in Eq. (B)

10 crossings per day		EV2	EV3
Idaho site (ADTT=600)	Simple span	1.20	1.16
	continuous span with 0.2kip/ft lane	1.05	1.05
New Jersey site (ADTT=6000)	Simple span	1.49	1.33
	continuous span with 0.2kip/ft lane	1.40	1.23
New York site (ADTT=5500)	Simple span	1.65	1.44
	continuous span with 0.2kip/ft lane	1.53	1.36

only 1 crossing per day		EV2	EV3
Idaho site (ADTT=600)	Simple span		
	continuous span with 0.2kip/ft lane		
New Jersey site (ADTT=6000)	Simple span	1.31	1.19
	continuous span with 0.2kip/ft lane	1.24	1.08
New York site (ADTT=5500)	Simple span	1.43	1.27
	continuous span with 0.2kip/ft lane	1.32	1.19

13. RECOMMENDATIONS FOR EV RATINGS USING LRFR

LRFR Recommendations

Table A gives the live load factors for use in combination with EV LRFR Eq. (A) which have been calibrated to meet on the average the target reliability index β_{target}=2.5 with the condition that none of the cases analyzed yields a reliability β lower than 1.50. The recommended live load factors are presented in Table A in a format consistent with that of the AASHTO MBE LRFR. The recommended final values are adjusted such that no live load factor is set at a value lower than 1.10.

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left(EV + lane + IM\right) \times DF} \tag{A}$$

Eq. (A) is applicable for simple span and continuous bridges with each span up to 300-ft in length. Lane load is not required for simple spans up to 300 ft (with only one EV on the span). A lane load equal to 0.20kip/ft is applied for all continuous spans in combination with only one EV on one span of the entire bridge. The AASHTO LRFD Multi-lane distribution factor is applied for multilane bridges.

Table A gives the recommended live load factors for two cases, the crossing of one EV per day or the crossing of 10 EV per day. The table also is calibrated for crossings of EV2 type emergency vehicles or EV3. It is expected that the use of the live load factors for 10 daily crossings would be appropriate in densely populated urban regions, while for rural areas using the load factors associated with one crossing per day would be reasonable. It is also noted that most emergency calls would require sending a smaller emergency vehicle which can be represented by EV2, while EV3 type vehicles would be required to respond to structural fires. Therefore, it would be reasonable in many jurisdictions to use the load factors for 10 crossings of EV2 while also checking bridges using one EV3 crossing per day. However, given the minimum recommended live load factor of 1.10, that would only affect sites with very high ADTT>6000 on highway with congested traffic.

Table A – EV load factors, γ_{EV}, for implementation in Eq. (A)

EV Frequency	Truck traffic condition	DF	EV2	EV3
10 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes	1.10	1.10
	ADTT > 6000 free flowing		1.40	1.10
	ADTT > 6000 congested		1.50	1.20
1 EV crossing per day	ADTT < 1000 free flowing	Two or more lanes	1.10	1.10
	ADTT > 6000 free flowing		1.20	1.10
	ADTT > 6000 congested		1.30	1.10

Table B gives the live load factors for to be used for EV LRFR with Eq. (B) which have been calibrated to meet on the average the target reliability index β_{target}=2.5 with the condition that none of the cases analyzed yields a reliability β lower than 1.50. The recommended live load factors are presented in Table B in a format consistent with that of the AASHTO MBE LRFR. The recommended final values are adjusted such that no live load factor is set at a value lower than 1.10.

Load factors for other ADTT values may be obtained by using a linear interpolation. The congested conditions pertain to bridges that experience traffic backups on a regular basis (say daily or more frequently), unrelated to the emergency situation. Congested traffic conditions will increase the multiple presence probabilities for trucks, particularly on high ADTT routes, and the likely maximum traffic loading, thus requiring an increased live load factor.

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_c}{\gamma_{EV}\left[(EV + lane)\times DF_1 + Legal \times DF_2 + IM\right]} \tag{B}$$

Where DF_1 is the load distribution factor for the Emergency Vehicle and DF_2 is the load distribution factor for the adjacent legal truck. Eq. (B) is applicable for the analysis of multilane simple span and continuous bridges with each span up to 300-ft in length. Lane load is not required for simple spans up to 300 ft (with only one EV and legal truck on the span). A lane load equal to 0.20 kip/ft is applied for all continuous spans in combination with only one EV on one span of the entire bridge in lane 1 and a governing legal truck in lane 2. No lane load is applied in the second lane.

The live load factors in Table B are obtained directly from a structural analysis of the superstructure using refined a grillage model or a finite element model. Alternatively, the load distribution factors in Eq. B can be obtained using the method provided in the AASHTO LRFD specifications (LRFD A4.6.2.2.5—Special Loads with Other Traffic). These adjusted load distribution factors can be represented as DF_1 for the Emergency Vehicle and $DF_2 = DF^* - DF_1$ for the adjacent legal truck, where DF^* is the tabulated AASHTO LRFD load distribution factor for two lanes and DF_1 is the tabulated AASHTO LRFD load distribution factor for a single lane after removing the multiple presence factor mp=1.2 which is already included in the AASHTO LRFD single lane tables.

When rating the bridge using the adjusted AASHTO LRFD load distribution tables, the live load factors in Table B should be decreased by 0.10. The reduction of 0.10 in the live load factors of Table B when using the adjusted AASHTO LRFD load distribution tables is recommended to account for the conservative bias already introduced in the AASHTO LRFD equations. A minimum live load factor value of 1.10 should be used as a conservative lower limit.

Table B gives the recommended live load factors for two cases, the crossing of one EV per day or the crossing of 10 EV per day. The table also is calibrated for crossings of EV2 type emergency vehicles or EV3. It is expected that the use of the live load factors for 10 daily crossings would be appropriate in densely populated urban regions, while for rural areas using the load factors associated with one crossing per day would be reasonable. It is also noted that most emergency calls would require sending a smaller emergency vehicle which can be represented by EV2 while EV3 type vehicles would be required to respond to structural fires. Therefore, it would be reasonable in many jurisdictions to use the load factors for 10 crossings of EV2 while also checking bridges using one EV3 crossing per day.

Table B – EV load factors, γ_{EV}, for implementation in Eq. (B)

EV Frequency	Truck traffic condition	DF	EV2	EV3
10 EV crossings per day	ADTT < 1000 free flowing	From Refined Analysis	1.20	1.15
	ADTT > 6000 free flowing		1.50	1.35
	ADTT > 6000 congested		1.65	1.45
1 EV crossing per day	ADTT < 1000 free flowing	From Refined Analysis	1.20	1.10
	ADTT > 6000 free flowing		1.30	1.20
	ADTT > 6000 congested		1.45	1.30

* Table B load factors are calibrated for use with refined analysis of the bridge structures. Refined analyses can be executed using well calibrated grillage analyses or finite element models of the bridge superstructure.

* Subtract 0.10 from the live load factors of Table B, but do not use a value lower than 1.10, if a refined analysis is replaced by the adjusted AASHTO load distribution factors proposed in the AASHTO LRFD specifications (LRFD A4.6.2.2.5—Special Loads with Other Traffic. These adjusted load distribution factors can be represented as DF_1 for the Emergency Vehicle and $DF_2 = DF^* - DF_1$ for the adjacent legal truck, where DF^* is the tabulated AASHTO LRFD load distribution factor for two lanes and DF_1 is the tabulated AASHTO LRFD load distribution factor for a single lane after removing the multiple presence factor mp=1.2 which is already included in the AASHTO LRFD single lane tables. The reduction of 0.10 in the live load factors of Table B when using the adjusted AASHTO LRFD load distribution tables is recommended to account for the conservative bias already introduced in the AASHTO LRFD equations.

14. APPLICATION TO ANALYSIS OF FLOORBEAMS AND TRANSVERSE MEMBERS

The analysis of floor-beams and transverse members involves the application of factored nominal live loads on tributary areas of relatively short lengths in the traffic direction. Thus, the forces that truck traffic imposes on such members would be similar to the reactions imposed by traffic load on the supports of longitudinal beams. While the analysis performed in this report did not explicitly address floor beams, transverse beams nor did it study the reactions of simple or continuous span beams, the effect of reactions on the supports of longitudinal beams is related to the shear forces in those beams. Hence, it is suggested that the live load tables calibrated in this study are also applicable for analyzing the effect of traffic load on floor beams and transverse members. Accordingly, the live load factors listed in Tables A and B which are applicable for bending and shear in short span and long span bridges is considered to be also applicable for floor beams and transverse members.

15. IMPLEMENTATION IN LFR

The various AASHTO LRFR (Load and Resistance Factor Rating) equations and associated live load factors provided in the AASHTO MBE are based on clear reliability criteria. Specifically, the LRFR equations were calibrated to provide consistent levels of reliability index values (see Moses, 2001 and Sivakumar and Ghosn, 2011). On the other hand, the AASHTO LFR (Load Factor Rating) equations and associated live load factors were arbitrarily chosen many decades ago based on intuition and judgment of the code writers. To the knowledge of the authors, there is no engineering basis or rational criteria that explain how the LFR factors were determined. This is especially true for the LFR operating legal load rating equations which apply the same load factor equal to 1.30 for both the dead load effects and the live load effects. Furthermore, it is noted that the same LFR live load factor = 1.30 is also used for permit load ratings. The use of the same live load factor for both legal and permit load ratings is not rational. This is because the probability that heavily loaded random trucks will cross simultaneously a bridge is not similar to the probability of having an overloaded permit truck cross the bridge simultaneously with equally loaded trucks. While it is quite possible that two heavy random trucks may cross a bridge simultaneously, the likelihood of having two super heavy permit trucks on the bridge is an unlikely scenario when the number of permit crossings is limited and controlled by the bridge authorities. Hence, it is clear that the AASHTO LFR equations do not provide similar levels of safety for the various possible loading conditions and the use of the same live load factors for all possible situations is not rational.

Conversely, the AASHTO LRFD and LRFR equations (Nowak, 1991, Moses, 2001, Sivakumar and Ghosn, 2011) were calibrated to achieve consistent reliability levels for bridges of various material types, beam spacing and span lengths for legal loads and also for various types of permit load ratings. Specifically, the AASHTO LRFR was designed to achieve a target reliability index β_{target}=2.5. Different live load factors were calibrated so that the same target reliability is achieved for various loading conditions including legal load rating and several conditions of permit load rating. In fact, previous calculations performed by Moses (2001), Sivakumar and Ghosn (2011) and Ghosn, Sivakumar, and Miao (2013) showed that the LFR equations for legal load operating ratings, which form the basic LFR scenario, produce low reliability levels for short span bridges and higher reliabilities for long span bridges.

Because of the inconsistencies inherent in the LFR equations, this study could not perform a specific calibration for the LFR live load factors for emergency vehicles because there is no clear target that one can match. Instead, this section presents a few plots to illustrate how the reliability index would vary if the same live load factors proposed in Table A for the LRFR method are also used in combination with the

LFR equations. The LFR calculations are executed using the live load factors in Table A for EV2 and EV3, dead load factors γ_{DW}=γ_{DC}= 1.30 and the following resistance factors:

ϕ = 1.00 for Steel and prestressed concrete in flexure

ϕ = 0.90 for Steel and prestressed concrete in shear

ϕ = 0.90 for Concrete T-beams in flexure

ϕ = 0.85 for Concrete T-Beams in shear

The LFR impact and S/… load distribution factors are used to calculate the load effect of EV2 and EV3. The dead loads are extracted from Table 4.1. The plots shown in Figures 15.1 and 15.2 for simple span bridges demonstrate how the LFR equation with γ_L=1.40 would lead to higher reliabilities for the long spans and lower reliabilities for the short spans with an average reliability index close to the target value β_{target}=2.50 set in the calibration of the LRFR live load factors. Such trends are consistent with those observed in previous calculations that studied the reliability levels implied in the LFR legal load operating rating method. The figures show that the average reliability index for 10 EV2 crossings with a live load factor γ_{LL}=1.40 as set in Table A would lead to an average reliability index β=2.47 which is close to the target index set at 2.50. However, the reliability index has a minimum value equal to β=0.79 which is significantly lower than the target minimum value set at 1.50. The maximum value of the reliability index is β=3.28 which is significantly higher than the target β=2.50. The range in betas is equal to 2.49. Using the lower live load factor=1.30 set in the traditional LFR would drop the minimum beta even further to β=0.57.

Similar calculations are executed for 10 crossings of EV3 with a live load factor γ_L=1.10 as set in Table A. The reliability analysis is performed using the New Jersey WIM data and the results are plotted in Figs. 15.3 and 15.4. In this case, the average reliability index is β=2.41 which is slightly lower than the target value with a minimum index β=0.08 and a maximum value equal to 3.26. Using a live load factor equal to 1.30 as implied in the LFR procedure would have increased the average reliability index to β=2.78 and the minimum value would have increased to 1.41.

The contrast between the spread in the reliability indexes observed in Figs 15.1 through 15.4 with those in Figs. 11.1 through 11.4 is very clear.

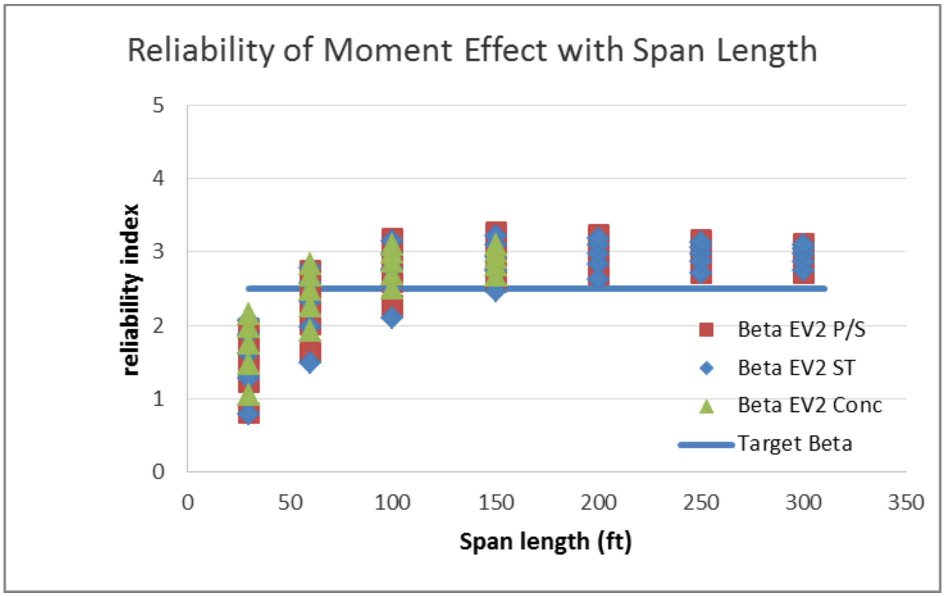

Figure 15.1 – Reliability index β for bending of simple span bridges under the effect of 10 EV2 crossings per day in combination with New Jersey truck WIM data (ADTT=6000) when using live load factors of Table A with LFR equation

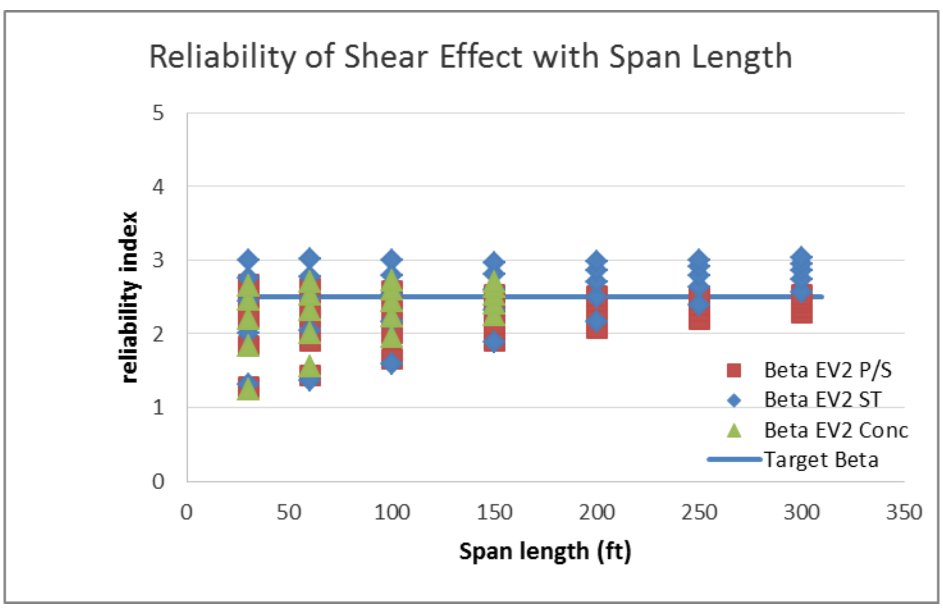

Figure 15.2 – Reliability index β for shear of simple span bridges under the effect of 10 EV2 crossings per day in combination with New Jersey truck WIM data (ADTT=6000) when using live load factors of Table A with LFR equation

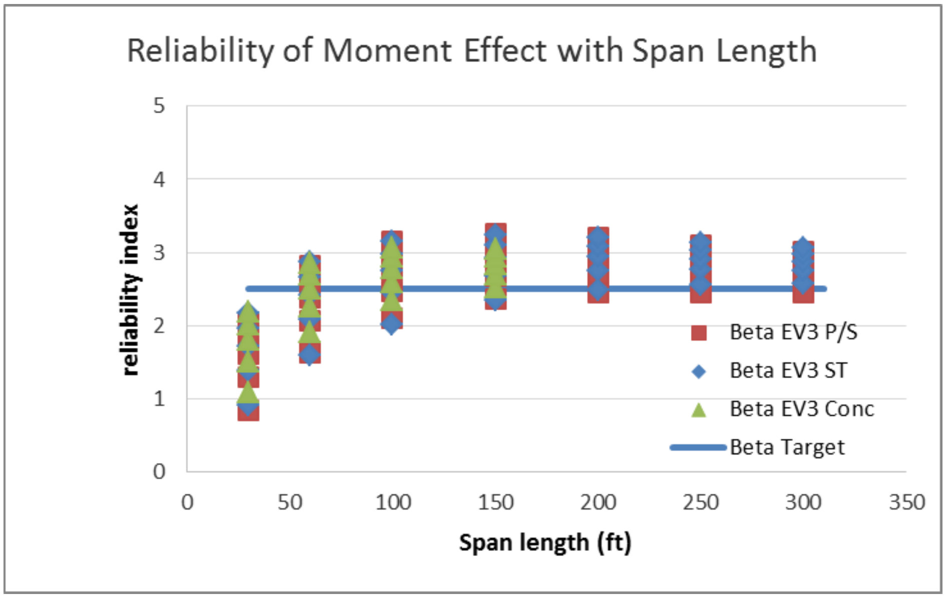

Figure 15.3 – Reliability index β for bending of simple span bridges under the effect of 10 EV3 crossings per day in combination with New Jersey truck WIM data (ADTT=6000) when using live load factors of Table A with LFR equation

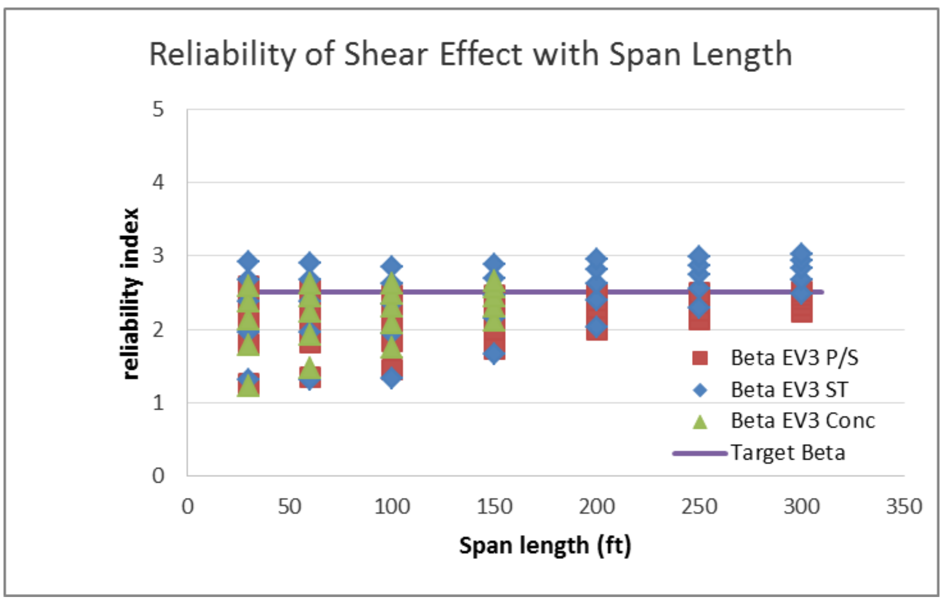

Figure 15.4 – Reliability index β for shear of simple span bridges under the effect of 10 EV3 crossings per day in combination with New Jersey truck WIM data (ADTT=6000) when using live load factors of Table A with LFR equation.

16. RECOMMENDED AASHTO MBE MODIFICATIONS FOR EV RATINGS

Item 1- Revise the text of Article 6A.1.5.2—Legal Load Rating, as follows:

This second level rating provides a single safe load capacity (for a given truck configuration) applicable to AASHTO and state legal loads, and FAST Act emergency vehicles (see Article 6A.4.4.2.1c). Live load factors are selected based on the truck traffic conditions at the site. Strength is the primary limit state for load rating; service limit states are selectively applied. The results of the load rating for legal loads could be used as a basis for decision making related to load posting or bridge strengthening.

Item 2- Revise the text of Article 6A.2.3—Transient Loads, as follows:

6A.2.3.1—Vehicular Live Loads (Gravity Loads): *LL*

The nominal live loads to be used in the evaluation of bridges are selected based on the purpose and intended use of the evaluation results. Live load models for load rating include:

Design Load: HL-93 Design Load per LRFD Design Specifications

Legal Loads:

1. AASHTO Legal loads, as specified in Article 6A.4.4.2.1a.
2. The Notional Rating Load as specified in Article 6A.4.4.2.1b or state legal loads.
3. FAST Act Emergency Vehicles as specified in Article 6A.4.4.2.1c.

Permit Load: Actual Permit Truck

Load factors for vehicular live loads appropriate for use in load rating are as specified in Articles 6A.4.3.2.2, 6A.4.4.2.3, and 6A.4.5.4.2.

State legal loads having only minor variations from the AASHTO legal loads should be evaluated using the same procedures and factors specified for AASHTO trucks in this Manual.

State legal loads significantly heavier than the AASHTO legal loads should be load rated using load factors specified for routine permits in this Manual if the span has sufficient capacity for AASHTO legal loads.

Item 3 - Add the following after the last paragraph of Article C6A.2.3.1

The FAST Act Emergency Vehicles are designed for use under emergency conditions to transport personnel and equipment to suppress fires and mitigate other hazardous situations. The FAST Act signed into law on December 4, 2015 made these emergency vehicles legal on the Interstate System and the routes within reasonable access to the Interstate

Item 4 – Revise Table 6A.4.2.2-1 as follows:

Table 6A.4.2.2-1—Limit States and Load Factors for Load Rating

Bridge Type	Limit State*	Dead Load	Dead Load	Design Load		Legal Load	Permit Load
				Inventory	Operating		
		γ_{DC}	γ_{DW}	γ_{LL}	γ_{LL}	γ_{LL}	γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	Table 6A.4.2.2-2	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*

Notes:

- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F_y stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Item 5- Add a new Article 6A.4.4.2.1c—FAST Act Emergency Vehicles:

6A.4.4.2.1c—FAST Act Emergency Vehicles

Two emergency vehicle configurations EV2 and EV3 produce load effects in typical bridges that envelop the effects resulting from the family of typical emergency vehicles that are covered by the FAST Act. The vehicle configurations for EV2 and EV3 shown in Figure D6A-8 should be used for legal load ratings.

The EV loading is applicable for simple span and continuous bridges with each span up to 300-ft in length. Lane load is not required for simple spans up to 300 ft (with only one EV on the span). A lane load equal to 0.20kip/ft is applied for all continuous spans in combination with only one EV on one span of the entire bridge. The AASHTO LRFD Multi-lane distribution factor is applied for multilane bridges.

The FAST Act Emergency Vehicles EV2 and EV3 are also applicable for the load rating of floor beams and transverse members.

Item 6- Add commentary to new Article 6A.4.4.2.1c—FAST Act Emergency Vehicles:

C6A.4.4.2.1c

The FAST Act Emergency Vehicles (EVs) are designed for use under emergency conditions to transport personnel and equipment to suppress fires and mitigate other hazardous situations. It should be noted that EV2 and EV3 live load models do not meet the Federal Bridge Formula-B which sets weight limits on groups of axles, and in many instances the EVs produce higher load effects than the three AASHTO Legal Trucks. Gross vehicle weight (GVW) limit for EVs is 86,000 pounds. The FAST Act signed into law on December 4, 2015 made FAST Act EVs legal on the Interstate System and the routes within reasonable access to the Interstate. Among those provisions is the exemption of emergency vehicles from meeting the nationwide Interstate truck weight limits set forth in 23 U.S.C. 127(a). Therefore, States cannot require special permits for FAST Act EVs to cross bridges on the Interstate and within reasonable access to the Interstate. Fire trucks exempted under the FAST Act can create significantly greater load effects in certain bridges than the previous legal loads. Therefore, load rating provisions are needed to incorporate these heavy emergency vehicles so that highway bridge safety, serviceability, and durability are not compromised. The EV2 and EV3 load models referenced in Figure D6A-8 provide analysis efficiency in load ratings by serving as envelope vehicles and are not meant as examples of real fire apparatus.

Item 7- Add a new Article 6A.4.4.2.3c—Generalized Live Load Factors for Fast Act Emergency Vehicles

6A.4.4.2.3c—Generalized Live Load Factors for Fast Act Emergency Vehicles

Generalized live load factors for the Strength I limit state are given in Table 6A.4.4.2.3c-1 for use with LRFD multi-lane distribution factors or with refined methods of analysis for the load rating for emergency vehicles specified in Article 6A.4.4.2.1c on structures other than buried structures. The load factors have been calibrated for EV2 and EV3 based upon site traffic conditions and the estimated number of EV crossings per day.

If in the Engineer's judgment, an increase in the live load factor is warranted due to conditions or situations not accounted for in this Manual when determining the safe legal load, the Engineer may increase the factors in Table 6A.4.4.2.3c-1, not to exceed the value of the factor multiplied by 1.3.

Table 6A.4.4.2.3c-1—Generalized Live Load Factors γ_L for Fast Act Emergency Vehicles

EV Frequency	Traffic Volume (One Direction)	Live Load Distribution	EV2	EV3
10 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.40	1.10
	ADTT > 6000 congested		1.50	1.20
10 EV crossings per day	ADTT < 1000 free flowing	From Refined Analysis	1.20	1.15
	ADTT > 6000 free flowing		1.50	1.35
	ADTT > 6000 congested		1.65	1.45
1 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.20	1.10
	ADTT > 6000 congested		1.30	1.10
1 EV crossing per day	ADTT < 1000 free flowing	From Refined analysis	1.20	1.10
	ADTT > 6000 free flowing		1.30	1.20
	ADTT > 6000 congested		1.45	1.30

Notes:
[a] *DF* = LRFD-distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

When bridges crossed by Emergency Vehicles are evaluated using a refined analysis, the same live load factor given in Table 6A.4.4.2.3c-1 shall be applied on the Emergency Vehicle and on the governing AASHTO or state legal truck placed in the adjacent lane (with only one EV and legal truck on the span). Lane load is not required for simple spans up to 300 ft. A lane load equal to 0.20kip/ft is applied for all continuous spans in combination with only one EV on one span of the entire bridge in one lane and only one governing legal truck in the second lane. No lane load is applied in the second lane with the legal truck. The dynamic amplification factor is applied on the total live load effect.

Load factors given in Table 6A.4.4.2.3c-1 shall also be used for the load rating of floor beams and transverse members.

Item 8- Add commentary to new Article 6A.4.4.2.3c—Generalized Live Load Factors for Fast Act Emergency Vehicles:

C6A.4.4.2.3c

Table 6A.4.4.2.3c-1 gives the LRFR live load factors for use with EVs and when applicable an adjacent legal truck and lane load. These load factors have been calibrated to meet on the average the target reliability index of 2.5 with the condition that none of the cases analyzed yields a reliability index lower than 1.50. The load factor calibration accounts for the probability of having an emergency vehicle in combination with random trucks on a bridge using multiple-presence statistics derived from Weigh-In- Motion (WIM) traffic data. For members where the one-lane DF may govern over multi-lane DF, such as fascia girders, the load factors given in Table 6A.4.4.2.3c-1 should be used with the built-in 1.2 multiple-presence factor divided out.

Table 6A.4.4.2.3c-1 gives the recommended live load factors for two crossing frequencies: the crossing of one EV per day or the crossing of 10 EVs per day. The use of the live load factors for 10 daily crossings would be appropriate in densely populated urban regions, while for rural areas using the load factors associated with one crossing per day would be reasonable. It is also noted that most emergency calls would require sending a smaller emergency vehicles, such as a pumper truck, which can be represented by EV2, while the heavier EV3 type vehicles, such as aerial ladder trucks, would be required to respond to structural fires. US fire departments have far more pumper trucks in operation than ladder trucks. The average number of miles traveled by any configuration of fire apparatus is less than 5,000 miles per year. Therefore, it would be reasonable in many jurisdictions to use the load factors for 10 crossings of EV2 while also checking bridges using one EV3 crossing per day. However, given the minimum recommended live load factor of 1.10, that would only affect sites with very high ADTT>6000 on highway with congested traffic.

Load factors for other ADTT values may be obtained by using a linear interpolation. The congested conditions pertain to bridges that experience traffic backups on a regular basis (say daily or more frequently), unrelated to the emergency situation. Congested traffic conditions will increase the multiple presence probabilities for trucks, particularly on high ADTT routes, and consequently increase the likely maximum traffic loading, thus requiring an increased live load factor.

Table 6A.4.4.2.3c-1 also gives the LRFR live load factors, for use with refined analysis methods of live load distribution, such as a grillage model or a finite element model. Alternatively, the load distribution analysis can be obtained using the method provided in the *AASHTO LRFD 4.6.2.2.5—Special Loads with Other Traffic.*

$$RF = \frac{\phi R_n - \gamma_{DW} D_w - \gamma_{DC} D_C}{\gamma_L[(EV + lane) \times DF_1 + Legal \times DF_2 + IM]}$$

(C6A.4.4.2.3c-1)

Where (EV+ lane) x DF_1 represents the static load effect of EV and lane load (when appropriate) placed in the main traffic lane of the bridge; (Legal) x DF_2 represents the static load effect of the legal load placed in an adjacent lane. IM is the dynamic amplification = 1.33 x total static live load including lane load. The LRFR live load factor of Table 6A.4.4.2.3c-1 is applied to the total live load effect. When using *AASHTO LRFD 4.6.2.2.5* the adjusted load distribution factors can be represented as DF_1 for the Emergency Vehicle and $DF_2 = DF^* - DF_1$ for the adjacent legal truck, where DF^* is the tabulated AASHTO LRFD load distribution factor for two lanes and DF_1 is the tabulated AASHTO LRFD load distribution factor for a single lane after removing the multiple presence factor mp =1.2 When rating the bridge using the *LRFD 4.6.2.2.5* load distribution method, the live load factors in Table 6A.4.4.2.3c-1 for refined analysis should be decreased by 0.10. The reduction of 0.10 in the live load factors when using the AASHTO LRFD 4.6.2.2.5 load distribution method is recommended to account for the conservative bias already introduced in the AASHTO LRFD distribution factor equations. A minimum live load factor value of 1.10 should be used as a conservative lower limit.

Item 9 – Revise Appendix B6A—Limit States and Load Factors for Load Rating, as follows:

Table B6A-1—Limit States and Load Factors for Load Rating (6A.4.2.2-1)

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	Table 6A.4.2.2-2	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1, ~~and~~ 6A.4.4.2.3b-1 and 6A.4.4.2.3c-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

* Defined in the *AASHTO LRFD Bridge Design Specifications*

Notes:

- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F_y stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Revise table number for Table B6A-5 as follows:

Table B6A- ~~5~~ 6—Permit Load Factors: γ_L (6A.4.5.4.2a-1)

Insert new Table B6A-5

Table B6A-5—Generalized Live Load Factors, γ_L for Fast Act Emergency Vehicles (6A.4.4.2.3c-1)

EV Frequency	Traffic Volume (One Direction)	Live Load Distribution	EV2	EV3
10 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.40	1.10
	ADTT > 6000 congested		1.50	1.20
10 EV crossings per day	ADTT < 1000 free flowing	From Refined Analysis	1.20	1.15
	ADTT > 6000 free flowing		1.50	1.35
	ADTT > 6000 congested		1.65	1.45
1 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.20	1.10
	ADTT > 6000 congested		1.30	1.10
1 EV crossing per day	ADTT < 1000 free flowing	From Refined analysis	1.20	1.10
	ADTT > 6000 free flowing		1.30	1.20
	ADTT > 6000 congested		1.45	1.30

Notes:

[a] *DF* = LRFD-distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out.

Item 10 -- Add a new figure to Appendix D6A—AASHTO Legal Loads

f. FAST Act Emergency Vehicles

EV2

EV3

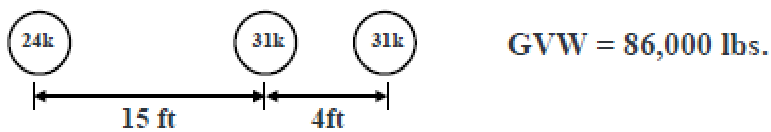

Figure D6A-8—Bridge Posting Loads for FAST Act Emergency Vehicles

Item 11- Revise the text of Article 6B.4.3 —Load Factor, as follows:

6B.4.3—Load Factor

For the load factor method, A_1 = 1.3 and A_2 varies depending on the rating level desired. For inventory level, A_2 = 2.17 and for operating level, A_2 = 1.3.

The nominal capacity, *C*, is the same regardless of the rating level desired (see Article 6B.5.3).

Operating level load factors to be used for the load factor method for load rating FAST Act Emergency Vehicles (see Article 6B7.2) shall be as given in Table 6B.4.3-1.

Table 6B.4.3-1 Operating Level Live Load Factors for FAST Act Emergency Vehicles

EV Frequency	Traffic Volume (One Direction)	Live Load Distribution	EV2	EV3
10 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.40	1.10
	ADTT > 6000 congested		1.50	1.20
1 EV crossings per day	ADTT < 1000 free flowing	Two or more lanes DF[a]	1.10	1.10
	ADTT > 6000 free flowing		1.20	1.10
	ADTT > 6000 congested		1.30	1.10

[a] = AASHTO STD Specs.

Item 12- Add new commentary to Article 6B.4.3 – Load Factor:

C6B.4.3—Load Factor

The FAST Act Emergency Vehicles are designed for use under emergency conditions to transport personnel and equipment to suppress fires and mitigate other hazardous situations. The FAST Act signed into law on December 4, 2015 made these emergency vehicles legal on the Interstate System and the routes within reasonable access to the Interstate.

There is a low probability of side by side occurrence of two heavy emergency vehicles. The EV load factors account for the probability of having an emergency vehicle in combination with random trucks on a multi-lane bridge. Table 6B.4.3-1 gives the recommended live load factors for two crossing frequencies: the crossing of one EV per day or the crossing of 10 EVs per day. The use of the live load factors for 10 daily crossings would be appropriate in densely populated urban regions, while for rural areas using the load factors associated with one crossing per day would be reasonable. It is also noted that most emergency calls would require sending a smaller emergency vehicle, such as a pumper truck, which can be represented by EV2, while the heavier EV3 type vehicles, such as aerial ladder trucks, would be required to respond to structural fires. US fire departments have far more pumper trucks in operation than ladder trucks. The average number of miles traveled by any configuration of fire apparatus is less than 5,000 miles per year. Therefore, it would be reasonable in many jurisdictions to use the load factors for 10 crossings of EV2 while also checking bridges using one EV3 crossing per day.

Load factors for other ADTT values may be obtained by using a linear interpolation. The congested conditions pertain to bridges that experience traffic backups on a regular basis (say daily or more frequently), unrelated to the emergency situation. Congested traffic conditions will increase the multiple presence probabilities for trucks, particularly on high ADTT routes, and consequently increase the likely maximum traffic loading, thus requiring an increased live load factor.

Item 13- Revise the text of Article 6B.7.2 —Posting loads, as follows:

6B.7.2—Posting Loads

The live load to be used in the rating Eq. 6B.4.1-1 for posting considerations should be any of the three typical legal loads shown in Figure 6B7.2-1, any of the four single-unit legal loads shown in Figure 6B7.2-2 or State legal loads, and the two FAST Act Emergency Vehicles shown in Figure 6B7.2-4. For spans over 200 feet in length, the selected legal load should be spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane and a single vehicle load should be applied in the adjacent lanes(s). When the maximum legal load under state law exceeds the safe load capacity of a bridge, restrictive posting shall be required.

Item 14- Insert the following paragraph at the end of commentary C6B.7.2 :

The FAST Act Emergency Vehicles (EVs) are designed for use under emergency conditions to transport personnel and equipment to suppress fires and mitigate other hazardous situations. It should be noted that EV2 and EV3 live load models do not meet the Federal Bridge Formula-B which sets weight limits on groups of axles, and in many instances the EVs produce higher load effects than the three AASHTO Legal Trucks. Gross vehicle weight (GVW) limit for EVs is 86,000 pounds. The FAST Act signed into law on December 4, 2015 made FAST Act EVs legal on the Interstate System and the routes within reasonable access to the Interstate. Among those provisions is the exemption of emergency vehicles from meeting the nationwide Interstate truck weight limits set forth in 23 U.S.C. 127(a). Therefore, States cannot require special permits for FAST Act EVs to cross bridges on the Interstate and within reasonable access to the Interstate. Fire trucks exempted under the FAST Act can create significantly greater load effects in certain bridges than the previous legal loads. Therefore, load rating provisions need to incorporate these heavy emergency vehicles so that highway bridge safety, serviceability, and durability are not compromised. The EV2 and EV3 load models referenced in Figure 6B7.2-4 provide analysis efficiency in load ratings by serving as envelope vehicles and are not meant as examples of real fire apparatus.

Item 15 -- Add a new figure to Article 6B.7.2—Posting Loads

EV2

EV3

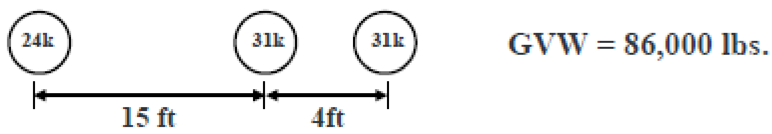

Figure 6B.7.2-4—Bridge Posting Loads for FAST Act Emergency Vehicles

.

Item 16 -- Revise the Table of Contents – Section 6 Load Rating, as follows

REFERENCES

1. International Association of Fire Chiefs (IAFC) and Fire Apparatus Manufacturers Association (FAMA), “Emergency Vehicle Size and Weight Regulation Guideline”, November 2011.
2. Federal Highway Administration (FHWA), “Fixing America’s Surface Transportation Act or “FAST Act” Truck Size and Weight Provisions”, February 2016.
3. Haynes, H. and Stein G., “U.S. Fire Department Profile”, National Fire Protection Association, 2015.
4. Federal Highway Administration (FHWA) “Bridge Formula Weights.” Traffic Congestion and Reliability: Trends and Advanced Strategies for Congestion Mitigation: Chapter 2, 20 June 2018, ops.fhwa.dot.gov/freight/publications/brdg_frm_wghts/index.htm.
5. AASHTO Bridge Design Specifications (2017) 8th Edition by American Association of State and Highway Transportation Officials, Washington DC
6. AASHTO MBE-3 (2018), Manual for Bridge Evaluation, 3rd Edition standard by American Association of State and Highway Transportation Officials, Washington DC
7. Barr, P.J., Eberhard, M.O. and Stanton, J.F., (2001), Live-load distribution factors for prestressed concrete girder bridges.” ASCE Journal of Bridge Engineering Vol. 6 No.5 pp. 298-306.
8. Ghosn, M., Sivakumar, B., and Miao, F. (2013) Development of State-Specific Load and Resistance Factor Rating Method, Journal of Bridge Eng., 18(5), 351–361.
9. Jambotkar, O. (2006), INVESTIGATION OF DISTRIBUTION FACTORS FOR LOAD RATING OF BRIDGES, M.S. Thesis, Department of Civil and Environmental Engineering, The University of Cincinnati, Cincinnati, Ohio
10. Moses (2001) Calibration of Load Factors for LRFR Bridge Evaluation, National Cooperative Highway Research Program NCHRP Report 454, Transportation Research Board, National Research Council, Washington DC.
11. Nowak, A.S. (1999) Calibration of LRFD Bridge Design Code, National Cooperative Highway Research Program, NCHRP Report, 368 Transportation Research Board, National Research Council, Washington DC.
12. Puckett, J., et al. (2007) Simplified Live Load Distribution Factor Equations, NCHRP Report 592, Transportation Research Board, The National Academies, Washington DC.
13. Schwarz, M. and Laman, J.A. (2001), “Dynamic Impact Allowance and Distribution Factors for Prestressed Concrete I-Girder Bridges”, ASCE, Journal of Bridge Engineering, Vol. 6, No. 1, February pp. 1-8.

14. Sivakumar, B, Ghosn, M. and Moses, F. (2011) Protocols for Collecting and Using Traffic Data in Bridge Design, NCHRP Report Issue Number: 683, National Cooperative Highway Research Program, Transportation Research Board, The National Academies, Washington DC
15. Sivakumar, B., and Ghosn M. (2011) "Recalibration of LRFR Live Load Factors in the AASHTO Manual for Bridge Evaluation", Final Report NCHRP Project 20-07/Task 285, Web Report, Transportation Research Board, The National Academies, Washington DC, March 2011.
16. Zokaie, T., Osterkamp, T.A. and Imbsen, R.A., (1991) "Distribution of Wheel Loads on Highway Bridges." NCHRP Proj. Rep. 12-26, Transportation Research Board, Washington, D.C.