RELIABILITY-BASED ASSESSMENT FOR LOAD AND RESISTANCE FACTORED RATING UNDER NJDOT PERMIT LOAD FOR STEEL BRIDGES

By

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And approved by

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ABSTRACT OF THE THESIS

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Nowadays, the aging bridges are exposed to much higher truck load than they were originally designed for, since trucks have the capacity to carry more weight. This becomes a major concern for engineers as they try to maintain the bridges and ensure that they remain safe. The engineers have come up with several load rating methods, such as Allowable Stress Rating (ASR), Load Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR), that can evaluate and inspect the conditions and safety of the bridge structure. Load rating is the determination of the live-load carrying capacity and load limits of an existing bridge. Based on the AASHTO LRFR, a rating factor of equal to or greater than 1 means that the structure has sufficient capacity to carry the vehicle that weighs less than or equal to the loading vehicle. In New Jersey alone, there are 3,142 steel girder bridges and according to 2017 National Bridge Inventory (NBI), majority of those bridges were designed according to the Allowable Strength Design (ASD), whereas most of the modern bridges (after the year 2000) were designed according to the AASHTO Load and

Resistance Factor Design (LRFD) Bridge Design Specifications. The LRFR is consistent with the LRFD philosophy and is the main focus of this thesis.

The objective of this study is to perform a reliability-based assessment of the current AASHTO LRFR in terms of the reliability indices β of steel girder bridges versus the LRFR rating factors. In particular, attention is placed on the Strength II and Service II limit states using New Jersey Department of Transportation (NJDOT) design permit truck. The bridge database consists of two sets. The first dataset includes the existing bridges obtained from NJDOT bridge inventory and the previous research. These bridges are designed in accordance with the AASHTO Standard Specifications for Highway Bridges. The second dataset includes the new bridges that are designed in accordance with the AASHTO LRFD and they are invented just for this study. The current AASHTO Manual of Bridge Evaluation (MBE) indicates the live load factors for permit loads have been calibrated to achieve a target reliability of 2.5 with a minimum of 1.5 for the Strength II limit state when rating factor is equal to 1. However, the live load factors in Service II limit state have not been calibrated through similar procedures, particularly for a simply-supported steel girder bridge. Therefore, to ensure the same level of uniformity, there is a need to assess the serviceability levels for the current permit loads and their corresponding live load factors for the Service II limit state. This thesis presents the results of bridge load rating using the LRFR procedures and the corresponding reliability indices, specifically for Strength II and Service II limit state. The results of this study show that the current LRFR live load factors at the Service II limit state do not truly reflect the current level of serviceability of the steel girder bridges. Based on the review of the analysis results, the study suggested a recommendation on the load and the resistance factors for the Service II limit state.

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Chapter I
INTRODUCTION

1.1. Background

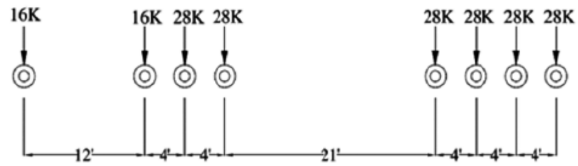
Nowadays, the aging bridges are exposed to much higher truck load than they were originally designed for, since trucks have the capacity to carry more weight. This becomes a major concern for engineers as they try to maintain the bridges and ensure that they remain safe. In some cases, upgrading an in-service bridge is the least costly option because it is more expensive than incorporating extra capacity at the design stage. Load rating a bridge at inventory level is currently performed to evaluate its safety. The evaluation can be done at lower cost by using standardize analysis and load rating procedures. However, in cases where load posting, or bridge strengthening is considered warranted, a more refined approach to evaluating the load capacity of an existing bridge can be economically justified.

In general, load and resistance factored rating is calibrated through reliability-based analysis. Deterministic approaches usually do not reveal the actual uniformity of safety reserve of the structure [*1*]. The current design specifications and evaluation manual, AASHTO Load and Resistance Factor Design (LRFD) [*2*] and AASHTO The Manual for Bridge Evaluation (MBE) [*3*], are introduced as the results of that. The AASHTO LRFD Bridge Design Specifications introduces the Strength I limit state calibrated to achieve a uniform level of safety in bridge design, beta of 3.5. This beta value is corresponding to probability of exceedance of 2.0E-04 during the 75-year design life of the bridge [*2*, *3*, *4*, *5*]. Similarly, the AASHTO MBE provides a methodology for load rating a bridge consistent with the load and resistance factor design philosophy of the AASHTO LRFD.

However, the LRFR procedures adopt a reduced target reliability index of approximately 2.5 for load rating at operating level, during 5-year evaluation period [*3*].

This paper aims at presenting a reliability-based assessment of steel bridges with composite concrete deck under the New Jersey design permit vehicle. The resistance statistical parameters of steel bridges are adopted from Nowak and Szerszen [*6*]. The statistical parameters were based on the available statistical data, material tests, load surveys, lab tests, field tests, and simulations. The live load and dead load statistical parameters were studied by various authors [*7, 8, 9, 10, 11*]. On the other hand, the live load parameters are obtained based on Lou et al [*12*]. The statistical live load model is developed based 208,487 permit vehicle records in a period of 5 years provided by NJDOT. The paper provided mean and standard deviation of 1 year, 5 years, 10 years and 75 years maximum loads for different length ranging from 20 ft to 200 ft.

The impact of overweight vehicles on highway infrastructure, especially in bridges and pavements has caused lots of concern throughout the country. One of the main problems of operating an overweight vehicle is the deterioration of bridge structures. It was found that overweight vehicles greatly affected the deterioration of concrete decks and prestressed concrete girders, as concluded by Lou et al [*19*]. This conclusion was mainly based on their study on New Jersey data. At the same time, AASHTO LRFD defines Strength II limit state for agencies to consider the load combination relating to the use of bridges by owner-specified special design vehicles, evaluation permit vehicles, or both. As the result, different transportation agencies have come up with different configuration and characteristics of permit vehicles. Figure 1 presents examples of axle weight and configuration of permit vehicles for different transportation agencies. The permit vehicle is usually based on the envelope of load effects created by issued routine permit vehicles, which does not share the same philosophy of reliability-based.



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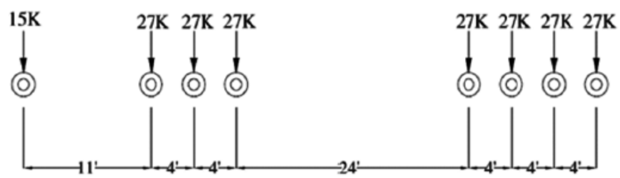

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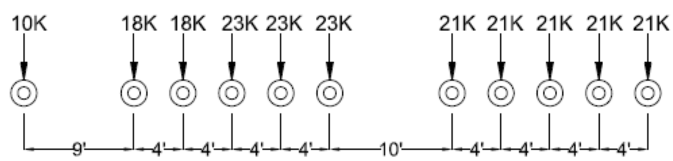

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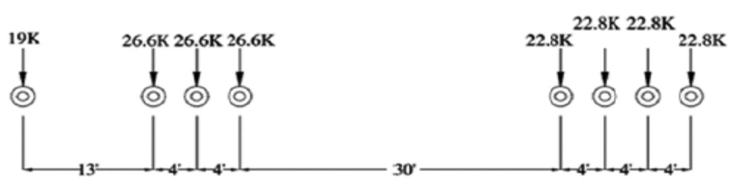

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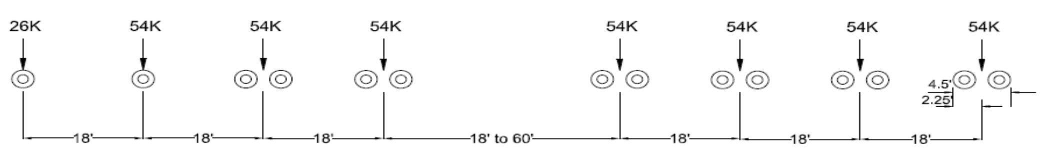

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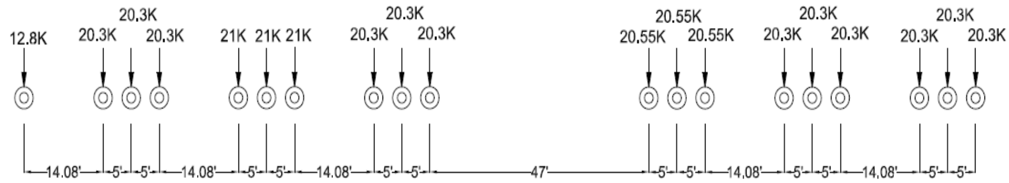

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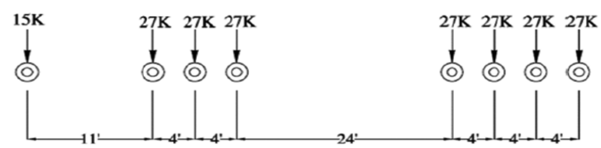

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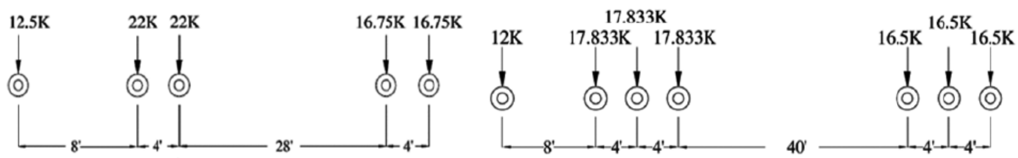

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Figure 1. Permit vehicles from various agencies. (a) NJDOT design permit vehicle. (b) Pennsylvania design permit vehicle. (c) NYSDOT design permit vehicle. (d) Wisconsin permit vehicle. (e) California design permit vehicle. (f) Connecticut department operating vehicle (design). (g) Connecticut department operating vehicle (design). (h) Virginia BP-90 and BP-115. [Development of Live Load Model for Strength II Limit State in AASHTO LRFD Design Specifications]

As load and resistance parameters are random variables, reliability analysis methods contain the necessary ingredients to provide a more rational evaluation strategy for existing bridges. With these methods, the structural performance can be measured in terms of the reliability index. The reliability index is a function of statistical parameters of load and resistance. The evaluation in a reliability-based approach provides a more uniform target level of safety by reducing uncertainties. For these reasons, the new generation of bridge design codes, the LRFD, is based on probability and statistic philosophy.

1.2. AASHTO Load and Resistance Factor Rating (LRFR)

The LRFR is consistent with the LRFD Specifications in using a reliability-based limit states philosophy and extends the provisions of the LRFD Specifications to the areas of inspection, load rating, posting and permit rules, fatigue evaluation, and load testing of existing bridges. While the strength limit state in AASHTO LRFD has been calibrated for a target reliability index of 3.5 which is corresponding to the probability of failure of 2.0E-04. The LRFR, on the other hand, has adopted a reduced target reliability index of approximately 2.5 for design (operating level) load rating of strength limit state. However, it is stated in the MBE that the live load factors for service limit state were not established through reliability-based calibration. These factors in service limit state were only selected based on engineering judgment and expert opinion. The level of reliability represented by this serviceability check is unknown.

It is important for the owner and the designer to maintain the bridge's rating factor greater than one for the safety of public. The LRFR is a deterministic analysis to determine whether the bridge is sufficient or not to carry a certain vehicle load. The reliability aspects are embedded in the evaluation process through the use of load factors. The methodology for the load and resistance factor rating of bridges consists of three distinct procedures: First, design load rating, second, legal load rating, and third, permit load rating. The results of each procedure serve specific uses and guide the need for further evaluations to verify bridge safety or serviceability. Rating factor greater than or equal to one is accepted as the criteria of the structural sufficient for every limit state.

Design load rating is a first-level assessment of bridges based on the HL-93 loading. It is a measure of the performance of existing bridges to current LRFD bridge design standards. The HL-93 designation consists of a design truck plus design lane load or design tandem plus design lane load, whichever produces the highest load effect. Design truck consists of three axles with the front axle weighing 8kips and the two rear axles weighing 32kips. The spacing between first axle and second axle is 14 feet, while the spacing between second axle and third axle is varied between 14 feet to 30 feet. Design tandem consists of twin axles, each weighing 25 kips. The spacing between two axles is 4 feet. Lastly, the design lane load consists of a uniform distributed load of 0.64 kip/ft. The combination of design live load and lane load is shown in Figure 2 and Figure *3*. The dynamic allowance, *IM,* is specified as 33% of the truck load only, with no dynamic allowance applied to the lane load. The code also specifies the girder distribution factor, *GDF,* to be used in structural analysis. The *GDF* equations will be presented later in this chapter.

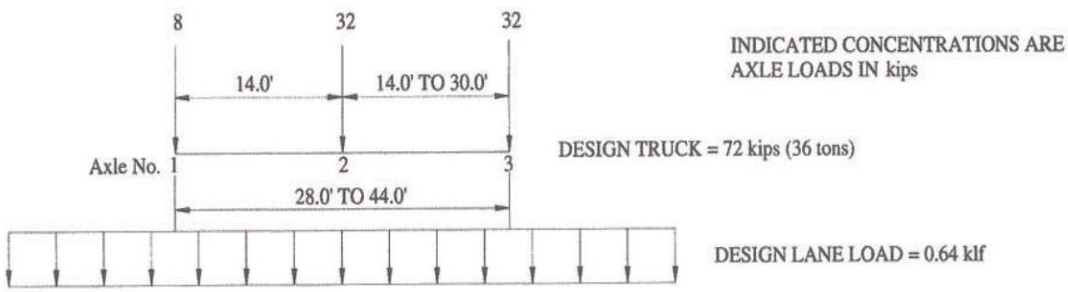


Figure 2. The Configuration of Design Truck and Design Lane Load [2018 AASHTO-MBE]

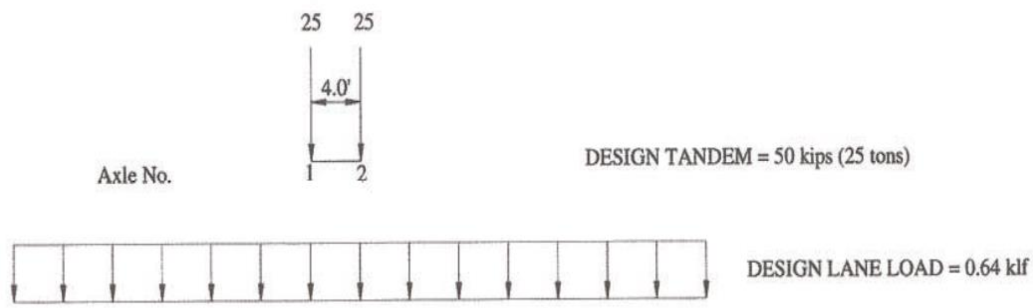


Figure 3. The Configuration of Design Tandem and Design Lane Load [2018 AASHTO-MBE]

Design load rating can serve as a screening process to identify bridges that should be load rated for legal loads. There are two sublevels under the design load rating; Inventory and Operating Rating level. As defined by the current AASHTO MBE, Inventory Rating is load that can safely utilize the bridge for an indefinite period of time. Operating Rating, on the other hand, is the maximum permissible live load that can be placed on the bridge. Allowing unlimited usage at the Operating Rating level will reduce the life of the bridge [*3*]. The bridges that pass the design load check (RF $\geq$ 1) at Inventory level will have a satisfactory load rating for all legal loads that fall within the LRFD exclusion limits. Secondly, legal load rating is a second level rating which provides a single safe load capacity applicable to AASHTO and State legal loads. 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. Finally, permit load rating which is a third level rating which checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legal loads.

1.3. Reliability Analysis

Reliability analysis is based on making a distinction between the "success" and the "failure" of a structure's ability to accomplish its intended purpose. The limit state function is used in developing this boundary. It is defined as a boundary between the desired and undesired performance of a structure, and it is mathematically represented. Generally, limit states functions can be grouped into three main categories: ultimate, serviceability, and fatigue.

Strength limit state functions mainly deal with the ultimate capacity of the structure, in terms of flexure, shear, torsion, or buckling. For this limit state, modes of failure may include: exceeding the moment carrying capacity of the structure, crushing of concrete in compression, or buckling of the web. Serviceability limit state (SLS) functions deal with the lifetime performance of the structure in terms of deterioration and the user's comfort. For this limit state, modes of failure include excessive deflection or vibration, concrete deck cracking, and permanent deformations. Finally, fatigue limit states (FLS) are related to the deterioration and damage of the structure under repeated loading. For this limit state, modes of failure include formation of fatigue cracks, high stresses in secondary members, and damage to welded connections.

The structural reliability analysis begins by establishing the limit state function, which can be expressed as follows:

$$g(R,Q) = R - Q \qquad \text{Eq. 1}$$

where, the R is the resistance or capacity and Q is the load effects.

For each limit state, the probability of failure can be expressed as:

$$P_f = P(R - Q < 0) = P(g < 0) \qquad \text{Eq. 2}$$

Figure 4 presents the probability density function (PDF) of load and resistance and probability of failure is corresponding to the shaded area in Figure 5.

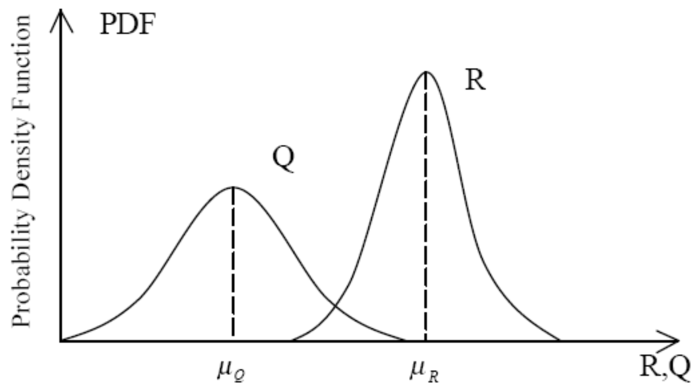


Figure 4. PDF of Load and Resistance

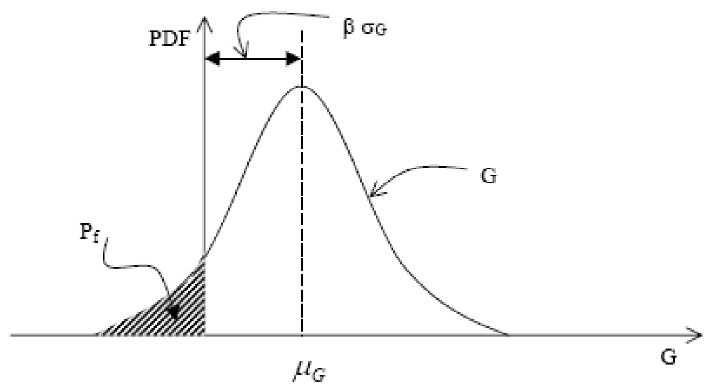


Figure 5. Illustration of Probability of Failure

Once the statistical information such as distribution type, mean and standard deviation of R and Q has been obtained from the experimental or simulation techniques, the safety level of the structural member can be evaluated using reliability index β:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad \text{Eq. 3}$$

where, β is reliability Index, μ_R is mean value of resistance moment, μ_Q is mean value of the moment due to applied loads, σ_R is standard deviation of the resistance moment, and σ_Q is standard deviation of the moment due to applied loads.

The reliability index β is correlated with the probability of failure and can be converted to it by looking up the tabulated values for normal probability distribution.

1.3.1. Target Reliability Index

The reliability index considers only two parameters of the random variable (mean and standard deviation) when describing its statistical variation, hence the term "second moment". This information is used to specify a boundary between safe and unsafe conditions. The limit state function also has its own statistical variability, with a mean and standard deviation based on the input random variables. Since this boundary is specified when the limit state function is equal to zero, the reliability index measures how far the mean of the limit state function is from failure in terms of number of standard deviations. Thus, the further the central tendency of the limit state function is away from zero or the failure boundary, the lower the probability of failure and the greater the reliability of the structure. This relation is shown in Figure 5. Cornell (1969) defines the reliability index in the following way:

$$\beta = \frac{\mu_g}{\sigma_g} \qquad \text{Eq. 4}$$

The probability of failure, P_f, is given by

$$P_f = \Phi(-\beta) \qquad \text{Eq. 5}$$

where, $\Phi(\,)$ is the cumulative standard normal distribution function. It follows that the reliability index can be also given by

$$P_f = \Phi(-\beta) \qquad \text{Eq. 6}$$

where, $\Phi^{-1}(\,)$ is the inverse cumulative standard normal distribution function.

Table 1 provides the relation between the probability of failure and the reliability index. For example, the probability that one out of a million units will fail, 10^{-6}, corresponds to a reliability index β of 4.75.

Table 1 Relationship between Probability of Failure and Reliability Index

Probability of failure P_f	Reliability Index β
10^{-1}	1.28
10^{-2}	2.33
$6.31*10^{-3}$	2.5
10^{-3}	3.09
$2.51*10^{-4}$	3.5
10^{-4}	3.71
10^{-5}	4.26
10^{-6}	4.75
10^{-7}	5.19
10^{-8}	5.62
10^{-10}	5.99

1.3.2. Random Variables

In order to perform the reliability analysis for the different limit states, the probabilistic distribution and statistical parameters (μ and σ) of various random variables are needed. More information and a summary of random variables are discussed later in Chapter III.

1.3.3. Monte Carlo Simulation

Estimation techniques are sometimes necessary and useful in determining the reliability index of the more complicated, nonlinear limit state functions. The methods include Monte Carlo simulation, Rosenblueth's *2n+1* method, Latin Hypercube Sampling, and integration methods.

The Monte Carlo simulation is one of the most fundamental estimation techniques. It involves using a random number generator to generate values for each random variable, based on their corresponding probability distribution, and calculating the limit state function. Values of the limit state function are thus statistically variable with a mean and standard deviation. To obtain a certain level of accuracy, a high number of simulations must be carried out. The number of simulations will depend on the complexity of the structure and the reliabilities of the various components. Elements with a high reliability will require a greater number of simulations as opposed to those with a lower reliability. With the availability of high-speed computers, however, this becomes of little concern.

In complex engineering problems, it becomes necessary to simulate distributions based on a fewer number of tests, as it is often impractical or not economical to run tests multiple times in order to establish probability distributions for each random variable. Once these probability distributions are constructed, parameters such as the mean and the standard deviation may be obtained for each random variable. The limit state function is then evaluated at each set of random values for the random variables. A CDF for the limit state function is then constructed and the probability of failure can be determined.

If N is the total number of simulations and k is the number of simulations in which $g<0$, then the probability of failure is

$$P_f = \Pr\left(g < 0\right) = \lim_{N \to \infty} \frac{k}{N} \qquad \text{Eq. 7}$$

The simulated values are plotted on normal probability paper (NPP), for which the vertical axis is the cumulative probability associated with the corresponding value of the random variable on the horizontal axis. If for *N* simulations, the limit state function does not fall below zero, that is $k=0$, then the cumulative distribution function of the simulated *g* values can be extended until it intersects the vertical axis. The probability of failure is then the ordinate of the intersection point.

The basis of the Monte Carlo simulation is the generation of random numbers, *u*, that are uniformly distributed between *0* and *1*. This is often contained in computer software as a built-in function or subroutine. Values of each random variable are then simulated based on their respective probability distribution.

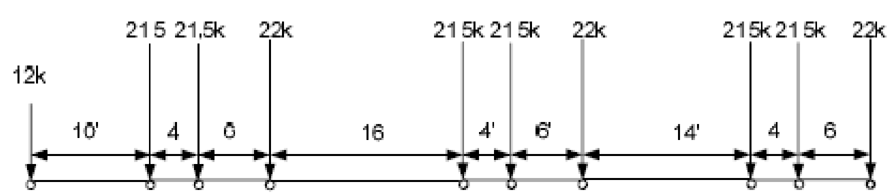
1.4. Literature Review

2001-NCHRP Report 454 Calibration of Load Factors for LRFR Bridge Evaluation (Moses et al.)

The study presents the methodology and data used to calibrate the LRFR criteria for the Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges. The philosophy in this report is consistent with the existing approaches used in the calibrating the load and resistance factors for the AASHTO LRFD Bridge Design Specifications. The same site truck weight statistics as the Nowak data was used in this study. This report provides the calibration of live load factors for legal load rating for routine traffic, as well as permit analysis, including routine, special and escorted vehicles. These load factors of permit vehicles presented in this report are still in use in the current Evaluation Manual. On the other hand, it was mentioned in this report that the New York study proposed a live load factor of 1.36 for routine permits under 130 kips and 1.05 for routine permit above 130 kips. However, there is no mention regarding the calibration of a live load factor for service limit state.

2011-NCHRP Report 700 A Comparison of AASHTO Bridge Load Rating Methods (Mlynarski, et al.)

This report presents the analysis of 1,500 bridges representing various material types and configurations to compare the load factor rating (LFR) to load and resistance factor rating (LRFR) for both shear and moment. There are a total of 3,043 girders from 1,500 bridges being analyzed. The live load model used in the analysis includes design vehicles, AASHTO legal loads and eight additional permit/legal vehicles. The permit vehicles being used in this report are based on various regions: Northwest (WA-02, and OR-06), Southwest (NM-04, and TX-04), Northeast (IL-01, and DE-07), and Southeast (FL-04, and NC-21), Figure 6. However, only five vehicles (DE-07, FL-04, NC-21, NM-04, and TX-04) are classified as routine permit vehicles used in the analysis. The results in this report for routine permit vehicle were based on the assumption of ADTT is equal to 1000 and using multiple lane load distribution factor.



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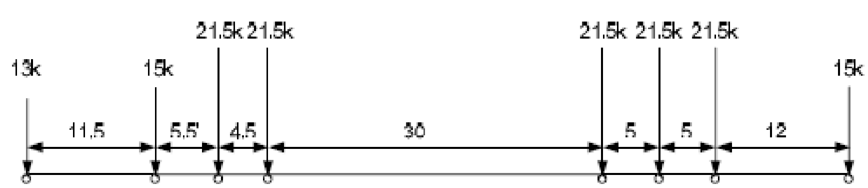

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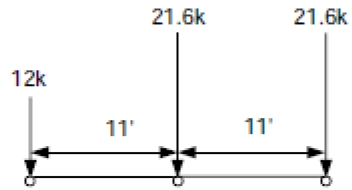
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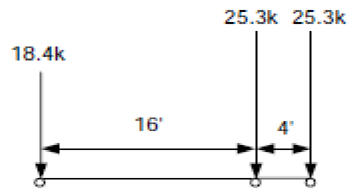
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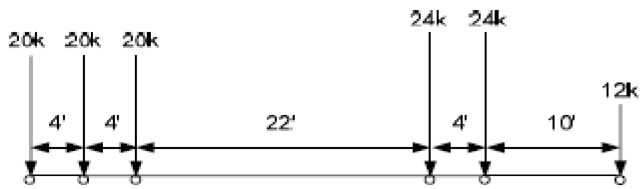
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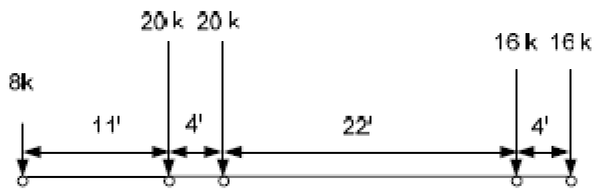
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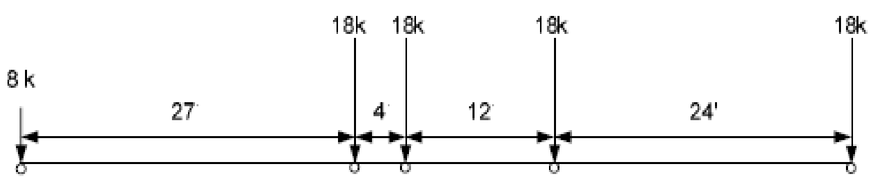
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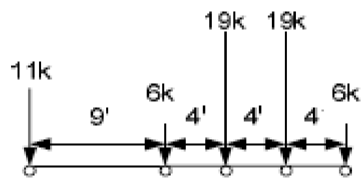


(h)

Figure 6. Permit configuration; (a) WA-02, (b) OR-06, (c) NM-04, (d) TX-04, (e) LI-01, (f) DE-07, (g) FL-04, (h) NC-21.

In addition, the reliability index was calculated to determine if it meets the assumed reliability index in the development of the MBE. For routine permits, the assumed target reliability index β is 2.5. Based on the analysis conducted in this report, the authors propose a decrease in live load factor for routine permits from 1.60 to 1.25 for an ADTT is equal to 1,000. In addition, the authors proposed to eliminate the distinction between vehicles up to 100 kips and vehicles above 150 kips and remove the rows corresponding to ADTT less than 100. Table 2 summarizes statistical parameters for the resistance of the cross-section used to calculate the reliability index.

Table 2 Statistical parameters for resistance

Bridge type	Moment		Shear	
	Bias factor	COV	Bias factor	COV
Multi-girder steel rolled shapes	1.12	0.10	1.14	0.105
Multi-girder steel plate girders	1.08 M+, 1.05 M-	0.11 M+, 0.10 M-	1.13	0.16
Multi-girder steel built-up shapes	1.11 M+, 1.12 M-	0.123	X	x
Prestressed I Beam	1.05	0.075	1.15	0.14
Prestressed Box Beam	1.05	0.075	1.16	0.14
Reinforced concrete T beam	1.14	0.13	1.20	0.155
Reinforced concrete slab	1.13	0.13	-	-

1.5. Research Objective

The objective of this paper is to perform a reliability-based assessment of the current AASHTO LRFR in terms of the reliability indices β of steel girder bridges versus the LRFR rating factors. In particular, attention is placed on the Strength II and Service II limit states using NJDOT design permit truck. The bridge database considered in this study includes existing bridges and new bridge designed based on the AASHTO LRFD. A total of 229 simply-supported steel girder bridges will be load rated for Strength II and Service II limit state for the design and permit vehicle, per AASHTO LRFR and NJDOT Design Manual. For strength limit state, this study only focuses on the flexure resistance of the structure. Shear resistance is not investigated in this study. As it is previously mentioned, the load factors in service limit states in the MBE provisions are not calibrated based on reliability theory. It is suspected that the current live load factor does not reflect the actual level of reliability of the structure.

A total of 229 simply-supported steel girder bridges will be evaluated using a deterministic approach, as well as reliability approach. For a deterministic approach, each bridge will be load rated according to AASHTO LRFR procedures with NJDOT design permit vehicle. The rating factor will be calculated for Strength II and Service II limit state. For reliability approach, statistical parameters are obtained from the studies that have been done in the past. Monte Carlo simulation is utilized to calculate the reliability index for each bridge. The results of rating factors are compared with the corresponding reliability index and the level of reliability is observed.

From the literature review, no one has worked on calibrating permit live load factor for service limit state for steel bridges. The originality of this project is to evaluate strength and service limit state using NJDOT permit vehicle by relating the rating factor with the corresponding reliability index.

Chapter II
BRIDGE DATABASE

2.1. Steel Girder Bridges Inventory

For this study, the bridge database consists of two sets. The first dataset includes the existing bridges obtained from NJDOT bridge inventory and the previous research project. The second dataset includes new re-designed bridges that are optimally designed in accordance with the AASHTO LRFD Specifications. The bridges in the first set are designed in accordance with the AASHTO Standard.

2.1.1. Existing bridges

For the first dataset, thirteen (13) existing simply-supported steel girder bridges from the NJDOT inventory are used to perform the analysis at the service limit state. A total of twenty-six (26) steel girders are obtained from this inventory. In addition, one hundred nineteen (119) simply-supported steel girder bridges from the previous research projects [*13*] are also selected to perform the analysis for service limit state. The data is taken from the database described in the NCHRP Project 12-26 report for real bridges. As mentioned above, the bridges in this first dataset were designed according to the AASHTO Standard Specifications for Highway Bridges. The characteristics of the steel girder bridges used in this study are summarized in Table 3. To be consistent with the design code, the unit weights of materials are taken from the AASHTO LRFD. Typical components of steel bridge, mainly concrete deck, steel girder, wearing surface, parapet, railing, stiffeners, diaphragms, are considered when performed the analysis. From this point forward, the bridges from this dataset will be referred as existing bridges.

Table 3 Summary of Steel Girder Bridge Properties

	Span Length (ft)	Skew Angle	No. of Girders	Girder Spacing (ft)	Girder Depth (ft)	Slab thickness (in)	Over-hang (ft)	Width (C-C) (ft)	Year built	Eccentricity (in)	Girder Inertia (in^4)	Girder Area (in^2)
1	75.88	13	9	7.08	3.12	8	3.17	52.00	1966	26.13	14885	42.86
2	77.58	13	9	7.08	3.15	8	3.17	52.00	1966	25.64	15750	69.5
3	59.83	13	9	7.08	3.01	8	3.17	52.00	1966	22.09	10500	50
4	59.83	13	9	7.08	3.01	8	3.17	52.00	1966	22.09	10500	50
5	87.00	9	6	7.88	36.26	7.5	2.75	39.00	1967	25.88	23052	97.13
6	87.00	9	6	7.88	36.26	7.5	2.75	39.00	1967	25.88	23052	97.13
7	90.00	9	6	7.88	38.02	8	2.75	39.00	1967	26.34	25287	104.9
8	90.00	9	6	7.88	38.02	8	2.75	39.00	1967	26.34	25287	104.9
9	85.77	2	6	8.21	45.38	7.5	2.81	42.25	1970	32.05	23423	64.5
10	85.77	2	6	8.21	45.38	7.5	2.81	42.25	1970	32.05	23423	64.5
11	62.00	11	7	8.75	36.17	9.5	1.75	52.50	2009	22.84	10500	50
12	62.00	11	7	8.75	36.17	9.5	2.67	52.50	2009	22.84	10500	50
13	85.00	2	6	7.84	37.28	7.5	2.67	39.01	1996	26.06	20489	98.23
14	85.00	2	6	7.84	36.90	7.5	2.67	39.01	1996	25.05	19178	82.6

15	86.89	8	6	7.88	3.14	7.5	2.31	39.00	1968	25.88	23052	97.13
16	86.89	8	6	7.88	3.14	7.5	2.31	39.00	1968	25.88	23052	97.13
17	89.73	8	6	7.88	3.15	7.5	2.31	39.00	1968	26.17	23497	99
18	89.73	8	6	7.88	3.15	7.5	2.31	39.00	1968	26.17	23497	99
19	86.50	20	8	7.25	54.00	8	3.13	48.80	1974	37.84	240431	51
20	86.50	20	8	7.25	54.00	8	3.13	48.80	1974	37.84	240431	51
21	92.00	19	7	7.42	4.25	8	3.25	42.25	1974	35.67	29393	64.5
22	92.00	19	7	7.42	4.25	8	3.25	42.25	1974	35.67	29393	64.5
23	107.00	32	8	7.42	57.50	8	3.25	42.50	1974	37.81	40868	69.25
24	107.00	32	8	7.42	57.50	8	3.25	42.50	1974	37.81	40868	69.25
25	92.00	33	9	8.25	4.00	7.75	5.25	67.00	1964	34.13	27889	72.5
26	92.00	33	9	8.25	4.27	7.75	5.25	67.00	1964	34.89	34956	77
27	20	0	5	5	1.50	6.5	1.08	22	1940	11.5	801	14.7
28	20.5	0	12	2.17	1.00	7.5	0.5	22	1957	9.8	234	9.1
29	27	0	5	5.75	1.75	7.25	2.75	24	1953	14.13	1327	18.2
30	28	0	9	2.58	1.25	6.5	0.25	19	1926	11.25	516	11.2
31	30	0	6	4.5	1.75	6	2	24	1935	14.38	1327	18.2

32	30	0	5	7.5	2.00	7.75	2	32	1937	15.92	2364	24.7
33	30.07	8.42	5	7.25	2.00	7.5	2.83	32.5	1953	15.25	2987	29.4
34	31.25	0	5	5.67	2.00	8	1.29	24	1932	15.19	2096	22.4
35	31.67	30	6	4.92	2.00	7.5	1.21	26	1947	15.8	2364	24.7
36	31.92	28	5	7.61	2.75	7	2.42	27	1960	20.05	6699	38.3
37	34.5	29	10	5	1.98	7	1.33	45	-	15.36	1802	19.85
38	34.75	0	5	5.25	2.17	8.75	0.5	20	1928	17.38	3000	27
39	36	0	5	5.25	2.17	8.75	0.5	20	1928	17.38	3000	27
40	37.17	0	5	5.25	2.17	8.75	0.33	20	1927	16.43	2364	24.7
41	38.75	0	5	6.58	2.50	7.5	1.83	28	1956	23.66	4461	31.8
42	38.92	8.42	5	7.25	2.50	7.5	2.83	32.5	1953	18.83	5347	36.5
43	39	0	5	5.75	2.25	6.75	2.5	24	1941	16.83	3267	27.7
44	39.08	2.06	8	6.83	2.50	7	2.25	36	1955	18.41	4461	31.8
45	40	30	13	2.29	1.50	10	0	26.33	1938	11.27	890	16.2
46	40	30	6	4.92	2.25	7.5	1.21	26	1947	17.21	3267	27.7
47	40	0	5	7.5	2.00	7.75	2	32	1937	15.92	2364	24.7
48	41.25	0	5	6.58	2.50	7.5	1.83	28	1956	20.3	6699	38.3

49	41.25	7.16	6	8.79	2.75	7.25	2	36	1957	21.77	7721	42.3
50	43	0	7	4.83	2.25	7	0.33	27	1935	17.04	3267	30
51	43	0	4	7.5	2.50	8	2.75	28	1985	18.82	3989	29.1
52	44	30	6	4.92	2.25	7.5	1.21	24	1935	17.29	3604	30
53	44	13.5	6	7.97	1.25	7.5	1.42	39	1965	17.29	3604	30
54	44	25	5	8.87	2.75	7.5	2	40	1955	20.41	7442	41.5
55	44.52	8.29	13	6.58	2.00	7	0	50	1945	15.88	5110	47
56	45	20	5	5.25	2.25	6.5	1.5	22	1940	16.79	3604	30.1
57	45	24.59	4	9	3.00	7.75	3.17	24	1964	21.8	9012	44.2
58	45.71	30	6	4.92	2.25	7.5	1.21	24	1935	17.21	3267	27.7
59	46	9.77	4	8.83	3.00	7.5	4.25	30	1959	21.75	9739	47.1
60	48	10.26	7	8.25	3.00	9	2.83	49.83	1959	22.5	9739	47.1
61	48	10.26	7	8.25	3.00	9	2.83	37.83	1959	22.42	9012	44.2
62	48.75	16.35	8	9.5	3.00	7.5	2.67	60	1955	26.38	10967	52.9
63	50	0	5	5.17	2.75	6.5	1.58	22	1938	20.55	6699	38.3
64	50	0	5	7	2.75	6	2.58	30	1950	19.55	6699	38.3
65	50	30	5	7.5	3.75	6.5	3.38	28	1955	21.49	12103	57.1

66	50	0	5	7.92	2.75	7.5	1.17	30	1950	22.61	8641	42.8
67	51	0	7	5.25	2.50	6.5	0.33	30	1940	18.16	4461	31.3
68	51.25	0	5	6.5	3.25	6.5	3.83	28	1957	20.29	5357	36.3
69	51.25	0	9	6.5	3.25	7	2.83	52	1957	20.29	5367	36.3
70	51.67	33	9	5.8	2.99	7.5	1.33	45	-	21.42	8902	43.7
71	51.9	6.43	12	5.55	3.00	7.5	0.67	50	-	21.83	10470	50
72	52	20.06	9	8.56	3.00	8	1.25	68.5	1968	21.92	9012	44.2
73	52.5	25	8	6	2.48	8	1.25	40	-	20.82	4930	34.9
74	53	30	4	9.33	2.75	7	2.25	30	1958	20.05	6699	38.3
75	53.5	24.59	4	9	3.00	7.75	3.17	24	1964	21.8	9012	44.2
76	55	12	6	7.87	2.75	8.25	1.89	36	1985	20.58	6699	33.3
77	55	25	5	8.87	2.75	7.5	2	40	1955	20.41	7442	41.5
78	55	15.52	4	10	3.65	7.13	3.96	28	1958	21.51	14988	67.7
79	56	30	6	7.87	3.00	7.75	1.79	44	1963	21.66	7796	39.7
80	56	0	7	8.33	3.00	8.5	3.5	50	1965	22.25	9739	47.1
81	56.25	0	7	5.33	2.50	7.25	0.17	30	1920	18.78	5753	38.8
82	58.58	17	5	7.61	2.75	7	2.42	27	1960	23.87	10949	56.5

83	59.83	0	5	6.58	3.17	7.5	1.83	28	1956	21.83	10470	50
84	60	0	5	5	3.00	7	2	22	1935	21.42	9012	44.2
85	60	2.17	4	9.5	3.58	7.13	5.1	26	1958	21.69	17234	76.6
86	61	0	6	4.92	2.75	7.5	1.21	24	1936	20.41	7442	41.5
87	62.56	2.5	8	8.83	3.00	8.75	1.53	55.8	1972	22.32	14988	57.7
88	63	8.31	6	7.95	3.00	8.75	2	39.7	1964	22.41	16092	72
89	64	2.5	4	8.83	3.00	8.75	1.58	26.5	-	22.32	14988	67.7
90	64	30	4	11	4.50	7.5	0.3	32	1969	35.24	39977	58
91	65	0	5	7	3.00	6	2.58	30	1950	20.92	9012	44.2
92	65.54	0	5	5.75	1.75	7.25	2.75	24	1953	20.18	6699	38.3
93	65.56	2.5	8	8.83	3.00	8.75	1.53	55.8	1972	22.49	12103	57.1
94	66.25	0	7	8.33	3.00	8.5	3.5	50	1965	22.25	9739	47.1
95	67	20	4	8.83	2.92	7	3.75	30	1961	21.69	11282	53.5
96	67	20	4	8.83	2.92	7	3.75	30	1951	21.59	11282	53.5
97	67	30	4	9.33	2.75	7	2.25	30	1958	20.05	6699	38.3
98	68	0	4	6.66	3.75	6.75	3.33	21	1954	21.32	14988	67.7
99	68	0	5	7	3.00	6	2.54	30	1955	20.92	9012	44.2

100	70	0	6	7.5	3.00	8.5	1.92	39	1977	22.03	7796	39.7
101	70	30	6	7.87	3.00	7.75	1.79	44	1963	21.66	7796	39.7
102	71.72	10.64	7	8.57	3.00	7	2	50	1962	26.22	17002	74.6
103	72	20.06	9	8.56	3.00	8	1.25	68.5	1968	22.08	10470	50
104	73	20	4	8.83	2.92	7	3.75	30	1961	21.69	11282	53.5
105	74.5	0	7	8.33	3.00	8.5	3.5	50	1965	22.19	14988	67.7
106	75	0	5	6	2.77	5.75	2	24	1963	19.38	11048	58.3
107	75	0	5	8.25	3.00	6	2	28	1956	25.6	16856	74
108	75.25	0	6	7.75	5.00	6.63	2.17	37	1955	35.34	17101	41.5
109	75.67	7.16	6	8.79	3.00	7.25	2	36	1957	25.71	20252	87
110	77	20	4	8.83	2.92	7	3.75	30	1961	21.69	11282	53.5
111	78.5	14	6	6.25	3.12	7.5	2.33	31.167	-	25.05	15482	68.63
112	79	9.77	4	8.83	3.00	7.5	4.25	30	1959	21.75	9739	47.1
113	80	10.26	7	8.25	3.00	9	2.83	49.83	1959	22.53	16092	72
114	80	10.26	7	8.25	3.00	9	2.83	37.83	1959	22.44	14988	67.7
115	80	0	7	8.33	3.00	8.5	3.5	50	1965	22.49	12103	57.1
116	80.25	16.35	8	9.5	3.00	7.5	2.67	60	1955	26.72	21353	92.7

117	80.66	0	5	5.25	3.51	6	3.5	24	1949	21	9739	47.1
118	83.6	13	8	8.23	4.00	7.5	2.83	40	1950	34.25	33754	52.5
119	84.58	2.06	8	6.83	3.00	7	2.25	36	1955	23.44	14988	67.7
120	86	20	4	8.83	2.92	7	3.75	30	1961	21.69	11282	53.5
121	87.3	0	5	7	3.00	8.5	1.21	28	1970	24.53	17871	77.7
122	89	0	4	9.33	3.00	6.75	3	30	1962	25.13	10629	42.8
123	90	0	5	8	3.00	9	1.42	32	1979	27.33	18554	80.5
124	91.25	0	6	7.75	5.00	6.63	2.17	37	1955	36.13	24195	52.8
125	93.12	0	7	8.33	3.00	8.5	3.5	50	1965	22.28	16092	73
126	95	30	6	7.33	3.50	7.5	2.5	38.83	1970	28.38	20700	57.4
127	96.52	10	5	5.75	3.00	6.5	2	24	1956	25.65	17780	78.1
128	98	0	5	9.83	4.83	8.25	2.57	36	1973	35.81	29122	53.5
129	100	0	5	7	3.00	8	1.92	29.83	1973	23.71	10460	49
130	105	0	6	8.67	3.00	12.01	1.69	36	1955	29.56	15587	69.1
131	110	0	4	8.5	4.00	8.5	2.33	28	1971	32.94	29835	69.2
132	113	0	6	9	4.00	7	6	70	1962	34.64	27429	61.2
133	113.17	0	4	8.5	5.21	7.13	4.25	28	1957	39.78	27833	61.5

134	116	0	4	9	6.50	7	3.33	28	1960	42.25	68862	82.5
135	121.5	0	8	8.08	3.67	9	3.33	46.83	1978	26.5	41824	109.3
136	125	0	4	11	5.00	10	3.25	38	1983	40.99	51463	32.6
137	130	0	6	8.67	3.00	12.01	1.69	36	1955	32.8	19181	86.6
138	130	0	3	15.5	7.92	9.63	5	39	1971	51.87	188585	130
139	140	0	6	13.5	8.81	6.5	2	58	1951	55.75	203546	109.3
140	142	0	10	9.25	5.50	7.5	2	79.25	1975	37.55	59869	72.9
141	147.65	0	10	8.04	4.20	8.66	2.46	63	2004	38.91	32816	80.31
142	151.13	0	3	12	8.33	7.75	4.55	28	1958	51.38	287125	155
143	152.5	30	4	11	4.50	7.5	0.9	32	1969	37.79	45716	64.5
144	155	0	3	15.5	7.92	9.63	5	39	1971	51.87	188585	130
145	180	22.92	6	8.2	4.67	7	2.11	44	1980	41.25	43570	73.5

2.1.1. Design of new bridges

The broad range of realistic designs is required to investigate the reliability indices for composite steel girder bridges. For the second dataset, 12 span lengths are considered ranging from 20 to 240 ft, with the girder spacing *S* ranging from 4 to 16 ft, resulting in a total of 84 steel bridges with composite concrete deck according to the current AASHTO LRFD Specifications. It is important to note that these bridges are invented just for this study. The details of the different parameters are shown in Table 4. From this point forward, the bridges from this dataset will be referred as new bridges.

Table 4 Design summary of the bridges

Parameters / number	Span Length (ft)	Girder Spacing, *S* (ft)	Total numbers
	20, 40, 60, 80, 100, 120, 140, 160, 180, 200, 220, 240.	4, 6, 8, 10, 12, 14, 16.	N=12*7=84
number	12	7	

Figure 7 shows the cross-section used for all bridge spans. The bridge cross-section is comprised of an 8.5 in slab with 5 girders spaced evenly as specified in Table 4. The thickness of future wearing surface is 2.5 in. The overhang is 3 ft and 2.25 in. The width of parapet is 1 ft and 5.25 in. The concrete compressive strength of slab is 5 ksi, the steel yield strength is 50 ksi.

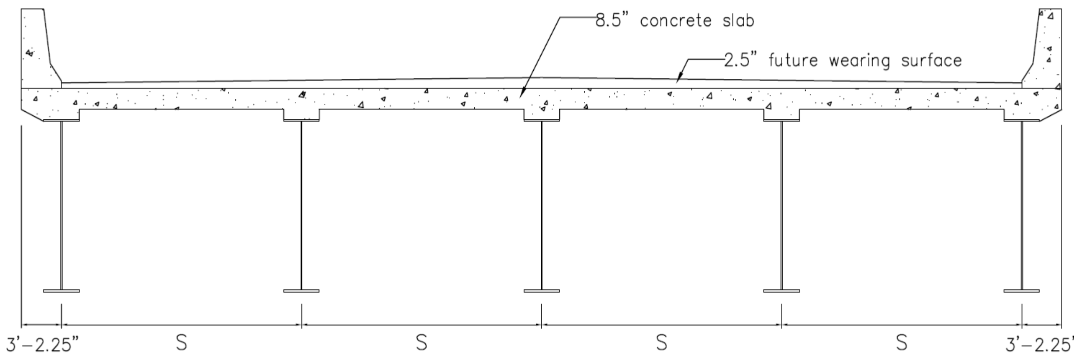


Figure 7. Cross-section of 5 girder bridge

Chapter III
LRFR RATING FACTOR

3.1. General Load Rating Equation

The load rating is generally expressed as a rating factor for a particular live load model, using the general load rating Eq. 8. In MBE, the rating procedure is carried out at each limit state and load effect with the lowest value determining the controlling rating factor [3]. In this thesis, both strength and service limit state are considered. Strength limit states are limit states relating to strength and stability during the design life, while service limit states are limit states relating to stress, deformation, and cracking under regular operating conditions. Eq. 8 is used to determine the load rating of a component (2018 MBE Eq. 6A.4.2.1-1).

$$RF = \frac{C - \gamma_{DC}\mathrm{DC} - \gamma_{DW}\mathrm{DW} \pm \gamma_P \mathrm{P}}{\gamma_L \mathrm{LL}(1 + \mathrm{IM})} \qquad \text{Eq. 8}$$

For the strength limit states:

$$C = \phi_c \phi_s \phi R_n$$

where, the following lower limit shall apply:

$$\phi_c \phi_s \geq 0.85$$

For the service limit states:

$$C = f_R$$

where, *RF* is rating factor, *C* is capacity, f_R is allowable stress specified in LRFD Specifications, R_n is nominal member resistance, DC is dead load effect due to structural components and attachments, DW is dead load effect due to wearing surface and utilities, P is permanent loads other than dead loads, LL is live load effect, *IM* is dynamic load allowance, γ_{DC} is LRFD load factor for structural components and attachments, γ_{DW} is LRFD load factor for wearing surfaces and utilities, γ_P is LRFD load factor for permanent loads other than dead loads, γ_L is evaluation live load factor, φ_C is condition factor, φ_S is system factor, and φ is LRFD resistance factor.

Resistance factor is a statistically-based multiplier applied to nominal resistance accounting primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance, but also related to the statistics of the loads through the calibration process.

Resistance Factor and Resistance Modifiers for Strength Limit States

Resistance factor (φ) has the same value for new design and for load rating. Resistance factors are taken as specified in the LRFD Specifications for new construction. A reduction factor based on member condition, condition factor (φ_c), is applied to the resistance of degraded members. An increased reliability index is maintained for deteriorated and non-redundant bridges by using condition and system factors in the load rating equation.

The condition factor (φ_c) provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

The system factors (φ_s) are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factor member capacities reduced, and therefore will have lower load ratings. The purpose of the system factor is to provide reserve capacity for safety. For simply-supported steel girder bridges, the system factor is taken as 1.0.

For this thesis, the effect of condition factor, system factor and LRFD resistance factor is not investigated. The results are based on the assumption that all the members are in good condition with no section loss and therefore all the calculations in this study are based on the factors are equal to 1.0.

Resistance Factors and Resistance Modifiers for the Service Limit States

For all non-strength limit states, resistance factor, condition factor and system factor are also taken as 1.0. Maximum steel stress is limited to 95% and 80% of the yield stress F_y for composite and non-composite compact girders, respectively. Since all steel girders in this study are composite, the maximum steel stress is equal to $.95F_y$.

Dead Load and Live Load Factors

Dead load consists of two components, DC and DW. DC is dead load of structural components and nonstructural attachments. DW, on the other hand, is dead load of wearing surfaces and utilities. Furthermore, DC consists of two components:

1) noncomposite dead load (DC1) which includes deck, stringer, cover plate, diaphragm, and stiffeners;

2) composite dead load (DC2) which includes curb, parapet, and railing.

The impacts due to dead loads, which are shown later in this chapter, are calculated based on the assumption of all permanent loads on the deck are uniformly distributed among the girders. Dead load factors are assigned to appropriate dead load components to increase the load effects. The purpose of increasing the load is to account for uncertainties in estimating the load effects. For this thesis, the dead load factors of 1.25 and 1.50 are used for DC and DW, respectively, for Strength II limit state. AASHTO MBE recommends a reduction factor of DW if the wearing surface is filed measured. For Service II limit state, the dead load factor of 1.00 is used for both DC and DW.

As stated in the 2018 AASHTO MBE, the permit load factors for Strength II limit state shown in Table 6 are calibrated to provide a uniform level of reliability index β of 2.5, while the permit load factors for Service II limit state shown in Table 5 are not calibrated based on reliability theory to achieve a target reliability but are based on past practice. The MBE also leaves the evaluation under Service II limit state for permit to be optional check.

It is important to note that the permit vehicle configuration may vary from state to state. For New Jersey, the NJDOT permit vehicle presents a gross vehicle weight (GVW) of 200 kips. Using the equation "GVW/AL" provided by the 2018 MBE, the appropriate permit load factor is selected. With the NJDOT permit truck having a gross vehicle weight GVW of 200 kips and the front axle to rear axle length AL of 53 ft, GVW/AL is equal to 3.77 kip/ft. For this study, the value of average daily truck traffic is assumed to be greater than 5000 (ADTT>5000) to produce the highest load effect. Therefore, the load factor of 1.30 is selected and used to perform LRFR rating factor calculation for Strength II limit state. For Service II limit state, the live load factor of 1.3 and 1.0 is used when evaluating the design load at inventory and operating level, respectively, while the live load factor of 1.0 is used for NJDOT permit load.

In general, the factor for live load is usually higher than that used for dead loads, because the dead loads can be estimated more accurately than live loads. The summary of dead load and live load factors are summarized in Table 5 and Table 6. These two tables,

Table 5 and Table 6, are partially taken from Table 6A.4.2.2-1 and Table 6A.4.5.4.2a-1 in the 2018 MBE, respectively.

Table 5 Limit states and load factors for steel bridge (2018 MBE Table 6A.4.2.2-1)

Limit State	Dead Load γ_{DC}	Dead Load γ_{DW}	Permit Load
			γ_{LL}
Strength II	1.25	1.50	Table 6
Service II	1.00	1.00	1.00

Table 6 Permit load factor: γ_{LL} (2018 MBE - Table 6A.4.5.4.2a-1)

Permit Type	Frequency	Loading Condition	DF[a]	ADTT (one direction)	Load Factor by Permit Weight Ratio		
					GVW / AL < 2.0 (kip/ft)	2.0 < GVW / AL < 3.0 (kip/ft)	GVW / AL > 3.0 (kip/ft)
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.40	1.35	1.30
				=1000	1.35	1.25	1.20
				<100	1.30	1.20	1.15
	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	All ADTTs	1.40		

Note:
DF = LRFD-distribution factor. When one-lane distribution factor is used, the built-in multiple presence factor should be divided out
Permit Weight Ratio = GVW/AL; GVW = Gross Vehicle Weight; AL = Front axle to rear axle length; Use only axles on the bridge.

3.2. Estimate the Resistance

Nominal resistance is resistance of a component based on the dimensions and strength of materials. It is important to correctly calculate the resistance of the components. The resistance is one of the factors that can influence the result of rating factor. Overestimating the resistance could result in a higher rating factor, and vice versa. Condition of the structure is another factor that could influence the rating factor. This factor is also a part of the LRFR rating factor equation, Eq. 8. A reduction factor is applied to the resistance to account for deterioration identified on the structural component. As previously mentioned, the condition factor is not considered in this study. All results presented in this thesis are based on the assumption of the structures are in good condition, in other words, the condition factor is equal to 1.0.

This study mainly focuses on the girder resistance in strength and service limit state. In general, strength limit state is a limit state relating to strength and stability of the structure, while service limit state is a limit state relating to stress, deformation, and cracking. Both interior and exterior girders are considered in this study. The nominal resistance of steel girder in both limit states is calculated per the AASHTO LRFD [*2*]. All steel sections are composite with the concrete slab in their final condition. The contribution of longitudinal reinforcement in concrete slab is not accounted in nominal flexural resistance calculation.

In Service II limit state, maximum steel stress is limited to 95% and 80% of the yield stress for composite and non-composite compact girders, respectively. Since all the girders in this study are composite, 95% of the specified yield stress is used to calculate the maximum capacity. The maximum capacity to resist the flexural stress of bottom steel flange of a composite section is calculated using Eq. 9

$$f_R = 0.95 R_h F_{yf} \qquad \text{Eq. 9}$$

where, F_{yf} is flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi), and R_h is a hybrid factor. For rolled shapes and homogenous built-up sections, R_h shall be taken as 1.0.

The summary of nominal resistance of the 13 bridges from NJDOT inventory for Strength II and Service II limit state is present in Table 7. The same calculation procedures are performed for the rest of the bridges.

Table 7 Summary of nominal resistance

Bridge Number	Girder	Resistance	
		Strength II Limit State (k-ft)	Service II Limit State (ksi)
1	Interior	4452.04	34.20
	Exterior	4662.79	34.20
2	Interior	3078.54	34.20
	Exterior	3043.47	34.20
3	Interior	6405.18	34.20
	Exterior	5749.05	34.20
4	Interior	6952.66	34.20
	Exterior	5989.04	34.20
5	Interior	5815.85	34.20
	Exterior	5693.27	34.20
6	Interior	4798.82	47.50
	Exterior	4471.59	47.50
7	Interior	5949.17	34.20
	Exterior	5259.38	34.20

8	Interior	6405.18	34.20
	Exterior	5644.63	34.20
9	Interior	6571.00	34.20
	Exterior	5676.75	34.20
10	Interior	5647.37	34.20
	Exterior	5596.86	34.20
11	Interior	6507.45	34.20
	Exterior	6454.87	34.20
12	Interior	7437.12	34.20
	Exterior	7389.04	34.20
13	Interior	6925.90	34.20
	Exterior	7508.17	34.20

3.3. Estimate Dead Load and Live Load Effects

Highway bridges are subjected to different types of loads during their service lives. This includes dead load, live load (including impact), environment loads (wind, earthquake, temperature), and other loads (collision, braking, etc.). For short to medium span bridges, environmental loads do not govern, therefore only dead load and live load are considered in this study. Dead load and live load effects must be determined in order to calculate the rating factor. Dead load effects are calculated per 2018 AASHTO LRFD. According to the code, dead load is divided into three components; DC1, DC2, and DW. DC1 is a noncomposite dead load namely deck, stinger, cover plate, diaphragm, and stiffeners. DC2 is a composite dead load namely curb, parapet, and railing. Both DC1 and DC2 are dead load of structural components and nonstructural attachments. Lastly, DW is dead load of wearing surface and utilities. The moment effects due to dead loads are calculated based on the assumption of all permanent loads on the deck are uniformly distributed among the girders. The unfactored bending moment per girder due to dead load is calculated and the results are presented in Table 8 and Table 9 for interior and exterior girder, respectively.

For service limit state, the bending stress effects due to dead loads are calculated by dividing bending moment by the section modulus S_x. Since the structure is a simply-supported, the maximum flexural stress generally occurs at the bottom flange. The maximum stress on the composite section due to the applied loads is the sum of the stresses caused by the loads applied separately to the steel section, short-term composite section, and long-term composite section.

To calculate flexural stresses within sections subjected to positive flexure, the composite section consists of the steel section and the transformed area of the effective width of the concrete deck. For transient loads assumed applied to the short-term composite section, the concrete deck area is transformed by using the short-term modular ratio, n. For permanent loads assumed applied to the long-term composite section, the concrete deck area is transformed by using the long-term modular ratio, 3n. n is the modular ratio, calculated using Eq. 10.

$$n = \frac{E}{E_c} \qquad \text{Eq. 10}$$

where, E is modulus of elasticity of the steel (ksi), and E_c = Modulus of elasticity of the concrete (ksi).

The dead load components consist of some dead loads that are resisted by the noncomposite section, as well as other dead loads that are resisted by the composite section. DC1 dead load is applied to the non-composite section (bare steel). DC2 and DW dead loads are applied to the long-term composite section (3n). The unfactored bending stress per girder due to dead load is calculated and the results are presented in Table 10 and Table 11 for interior and exterior girder, respectively.

Table 8 Unfactored bending moment due to dead loads on interior girder

Bridge Number	DC (k-ft)	DW (k-ft)
1	812.53	76.99
2	515.70	47.87
3	1188.48	45.01
4	1343.20	48.17
5	1165.91	45.60
6	689.53	101.61
7	1170.50	42.78
8	1200.87	44.90
9	1282.48	47.89
10	1276.66	40.97
11	1416.66	47.43
12	1753.13	64.16
13	1260.97	131.84

Table 9 Unfactored bending moment due to dead loads on exterior girder

Bridge Number	DC (k-ft)	DW (k-ft)
1	881.52	76.23
2	493.86	45.34
3	1055.34	36.80
4	1152.90	39.38
5	1017.74	38.43
6	658.93	90.47
7	1058.12	35.94
8	1052.17	35.64

9	1124.22	38.01
10	1170.97	38.14
11	1336.25	44.49
12	1707.33	60.18
13	1403.18	149.81

Table 10 Unfactored bending stress due to dead loads on interior girder

Bridge Number	DC (ksi)	DW (ksi)
1	9.82	0.85
2	10.14	0.78
3	9.33	0.31
4	9.73	0.30
5	9.91	0.34
6	13.63	1.53
7	9.92	0.31
8	9.41	0.31
9	9.75	0.32
10	12.32	0.33
11	10.85	0.32
12	11.88	0.38
13	9.45	0.84

Table 11 Unfactored bending stress due to dead loads on exterior girder

Bridge Number	DC (ksi)	DW (ksi)
1	10.34	0.75
2	9.70	0.74
3	8.28	0.26
4	8.36	0.25
5	8.63	0.29

6	12.99	1.42
7	9.98	0.30
8	8.25	0.25
9	8.55	0.26
10	11.26	0.31
11	10.18	0.30
12	11.57	0.36
13	9.60	0.88

The maximum unfactored live load moments per girder is calculated based on the HL-93 design load and the NJDOT permit vehicle. The NJDOT permit vehicle load is heavier and therefore creates a greater impact on the structure compare to HL-93. The vehicle consists of eight axles with a GVW of 200 kips as shown in Figure 8. This permit configuration is only valid for NJDOT as it may vary from state to state. Influence line is used to calculate the maximum bending moment due to live load. To account for dynamic effects, impact factor *IM* of 25% is applied to the permit truck, per NJDOT Design Manual recommendation. Similar to dead load, live load effect is applied to the short-term composite section (n) to calculate live load effect for service limit state.

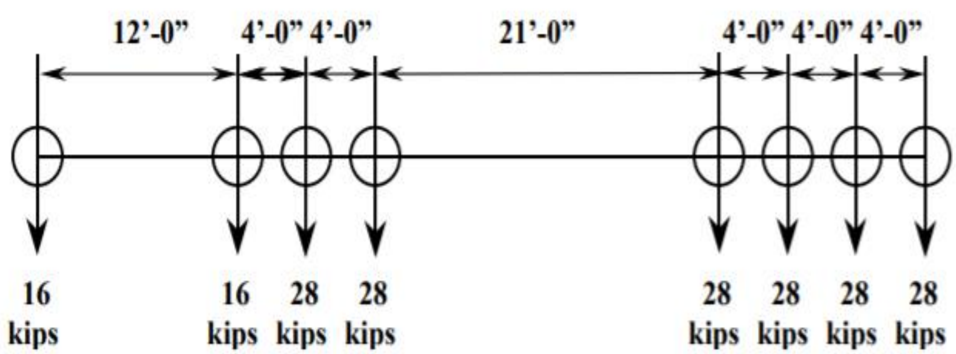


Figure 8. NJDOT Permit Vehicle NJDOT [2016 NJDOT Design Manual]

The LRFD introduced "exponential distribution" formulas which were derived to represent the girder distribution from the refined analysis. These formulas are used to replace the traditional "S-over" formulas in the Standard Specifications, which can be overly conservative in some parameter ranges while unconservative in others. For this study, the moment girder distribution factor *GDF* is used to calculate the live load effect on the interior and exterior girder. As specified by AASHTO LRFD, the live load moment *GDF* for each structure is calculated using the following equations. Eq. 11 and Eq. 12 are used to calculate live load moment *GDF* for interior beams. The LRFD specifies the ranges of applicability for each equation. If one or more of the parameters exceed the ranges of applicability, engineering judgment needs to be exercised before using these formulas. For this study, all the parameters fall in the ranges of applicability. In general, the live load distribution for exterior beams is taken as the larger of the value obtained from three methods specified, which include:

1) The lever rule;

2) Distribution formulas;

3) Special Analysis.

The lever rule and Eq. 13 are used to calculate one lane loaded and two or more lanes loaded moment *GDF*, respectively, for exterior beams. In addition, the special analysis is also performed using Eq. 14.

One Lane Loaded:

$$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.1} \qquad \text{Eq. 11}$$

Two or More Lanes Loaded:

$$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^{\,3}}\right)^{0.1} \qquad \text{Eq. 12}$$

where, S is girder spacing (ft), L is span length (ft), t_s is thickness of slab (in.), K_g is stiffness parameter (in^4).

One Lane Loaded: Using Lever Rule

Two or More Lanes Loaded:

$$\begin{aligned} g &= eg_{\text{interior}} \\ e &= 0.77+\frac{d_e}{9.1} \end{aligned} \qquad \text{Eq. 13}$$

where, e is correction factor, d_e is horizontal distance from the centerline of the exterior web of exterior beam at deck level to the interior edge of curb or traffic barrier (ft)

Special Analysis:

$$R=\frac{N_L}{N_b}+\frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2} \qquad \text{Eq. 14}$$

where, R is reaction on exterior beam in terms of lanes, N_L is number of loaded lanes under consideration, e is eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders (ft), x is horizontal distance from the center of gravity of the pattern of girders to each girder (ft), X_{ext} is horizontal distance from the center of gravity of the pattern of girders to the exterior girder (ft), N_b is number of beams or girders

Since the majority of the existing bridges in this study are skew, the moment *GDF*s are adjusted accordingly using the Eq. 15 and Eq. 16.

$$e=1-c_1(\tan\theta)^{1.5} \qquad \text{Eq. 15}$$

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^{\ 3}}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \qquad \text{Eq. 16}$$

If $\theta < 30°$ then $c_1 = 0.0$

If $\theta > 60°$ use $\theta = 60°$

Since the study focuses on the evaluation of NJDOT permit vehicle, the selection of *GDF* is based on the MBE recommendation in conjunction with NJDOT regulation on the analysis of the permit vehicle. The summary of live load effect (including dynamic impact and *GDF*) is shown in Table 12.

Table 12 The summary of live load effect

Bridge Number	Girder	Live Load Effects	
		GDF(M_{LL+IM}) (k-ft)	*GDF*(f_{LL+IM}) (ksi)
1	Interior	1695.98	18.00
	Exterior	1701.21	15.12
2	Interior	1204.66	17.73
	Exterior	1126.68	16.65
3	Interior	2347.29	14.53
	Exterior	2296.47	14.44
4	Interior	2429.39	13.75
	Exterior	2416.72	14.08
5	Interior	2391.55	16.53
	Exterior	2437.54	16.98
6	Interior	1395.61	19.16
	Exterior	1372.99	19.37
7	Interior	2264.32	15.09
	Exterior	2223.12	16.77
8	Interior	2344.78	14.51

	Exterior	2272.01	14.32
9	Interior	2449.35	14.69
	Exterior	2386.14	14.58
10	Interior	2151.53	16.20
	Exterior	1765.16	13.34
11	Interior	2391.88	14.91
	Exterior	2490.78	15.58
12	Interior	2884.15	16.10
	Exterior	2846.20	15.94
13	Interior	2522.67	14.82
	Exterior	2400.04	13.12

3.4. Load Rating Summary

After resistance and load effects have been determined, the LRFR rating equations as per AASHTO MBE [*3*] are used to determine level of safety of the steel girder bridges, in terms of rating factor. For each structure, the bridges are load rated for Strength II and Service II limit state for the design load and NJDOT permit vehicle. The purpose of Strength II limit state is to check the strength and stability, while the Service II limit state is to check the stress due to dead and live load effect.

Rating factor is a function of resistance, dead load, and live load (including dynamic impact). Once the dead load and live load effects are defined, the rating factor can be calculated. The procedures to calculate dead load and live load effect are already described in the previous section. To simplify the LRFR rating factor equation for Strength II limit state, Eq. 8 can be rewritten in terms of bending moment M and bending stress f, as shown in Eq. 17 and Eq. 18 below.

$$RF = \frac{R - \gamma_{DC} M_{DC} - \gamma_{DW} M_{DW}}{\gamma_L (GDF)(M_{LL+IM})} \qquad \text{Eq. 17}$$

where, γ_{DC} is equal to 1.50, γ_{DW} is equal to 1.25, and γ_L is equal to 1.3. R is flexural resistance of a composite section, M_{DC} is moment due to dead load, M_{DW} is moment due to wearing surface, *GDF* is moment girder distribution factor, and M_{LL+IM} is moment due to live load plus dynamic allowance.

$$RF = \frac{f_R - \gamma_{DC} f_{DC} - \gamma_{DW} f_{DW}}{\gamma_L (GDF)(f_{LL+IM})} \qquad \text{Eq. 18}$$

where, $\gamma_{DC,}$ and γ_{DW} is equal to 1.0. γ_L is equal to 1.30 and 1.0 for design load at inventory and operating level, respectively, while γ_L is equal to 1.0 for permit. f_R is flexural stress resistance of a composite section, f_{DC} is flexural stress due to dead load, f_{DW} is flexural stress due to wearing surface, and f_{LL+IM} is bending stress due to live load plus dynamic allowance.

3.4.1. Strength II limit state

For this section, the rating factor of 84 bridges from the new bridges is calculated using above Eq. 17 with live load factor of 1.3. Figure 9 shows the calculated LRFR rating factor versus span length for different girder spacings. There is zero bridge with an LRFR rating factor of less than 1.0. The minimum rating factor is 1.06 with the average of 1.32.

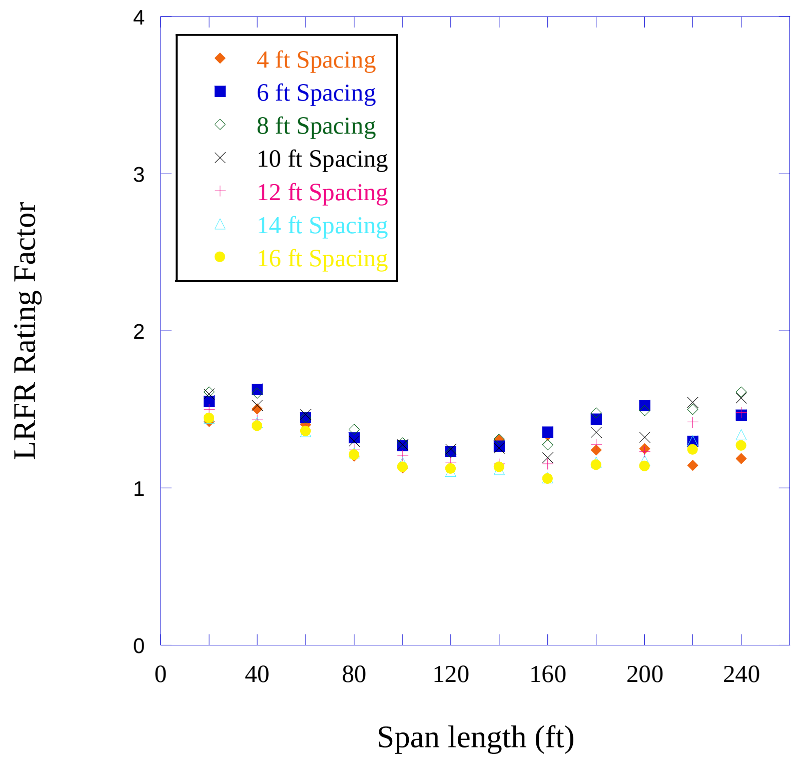


Figure 9. LRFR rating factor Strength II limit state versus span length (new bridges)

3.4.2. Service II limit state

For this section, the rating factors for Service II limit state are calculated for both existing and new bridges. The rating factor is calculated for both HL-93 design load and NJDOT routine permit using above Eq. 18. Live load factor of 1.3, and 1.0 is used for the HL-93 at inventory and operating level, respectively, while a live load factor of 1.0 is used for the NJDOT permit load.

3.4.2.1. Existing bridges

Figure 10 to Figure 12 show the calculated LRFR rating factor of the existing 145 bridges. There are 40 and 21 bridges with an LRFR rating factor of less than 1.0 for inventory and operating level, respectively, while there are 49 bridges with an LRFR rating factor of less than 1.0 for the permit. Based on the results, the bridges have the lowest rating factors when calculated based on permit load followed by the design load at inventory level and at operating level, respectively.

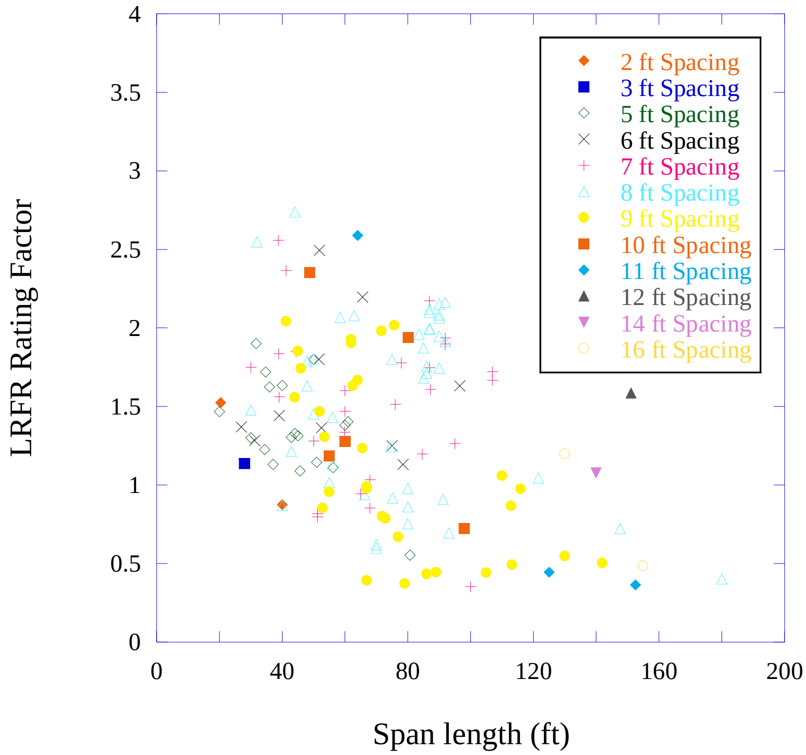


Figure 10. LRFR rating factor (design load at inventory level) versus span length (existing bridges)

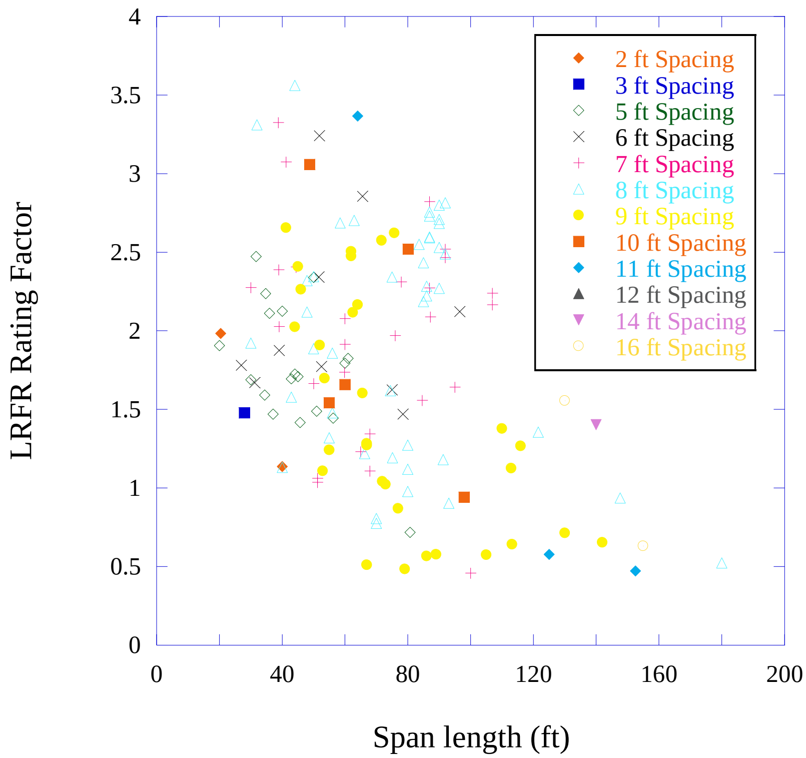


Figure 11. LRFR rating factor (design load at operating level) versus span length (existing bridges)

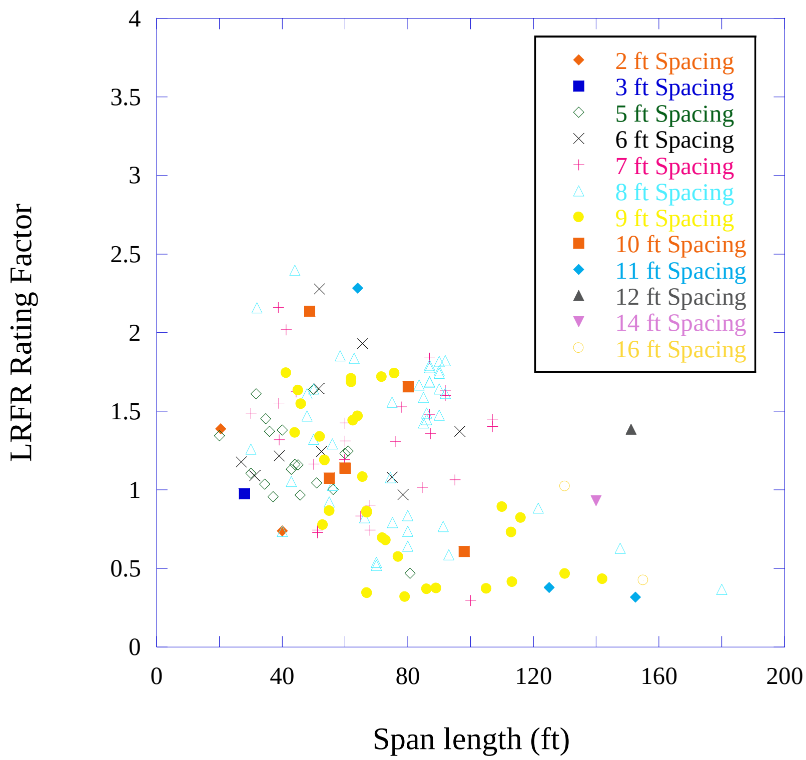


Figure 12. LRFR rating factor (permit) versus span length (existing bridges)

3.4.2.2. New bridges

Figure 13 to Figure 15 show the calculated LRFR rating factor of 84 new bridges. There are 9 and zero bridges with an LRFR rating factor of less than 1.0 for inventory and operating level, respectively, while there are 41 bridges with an LRFR rating factor of less than 1.0 for the routine permit.

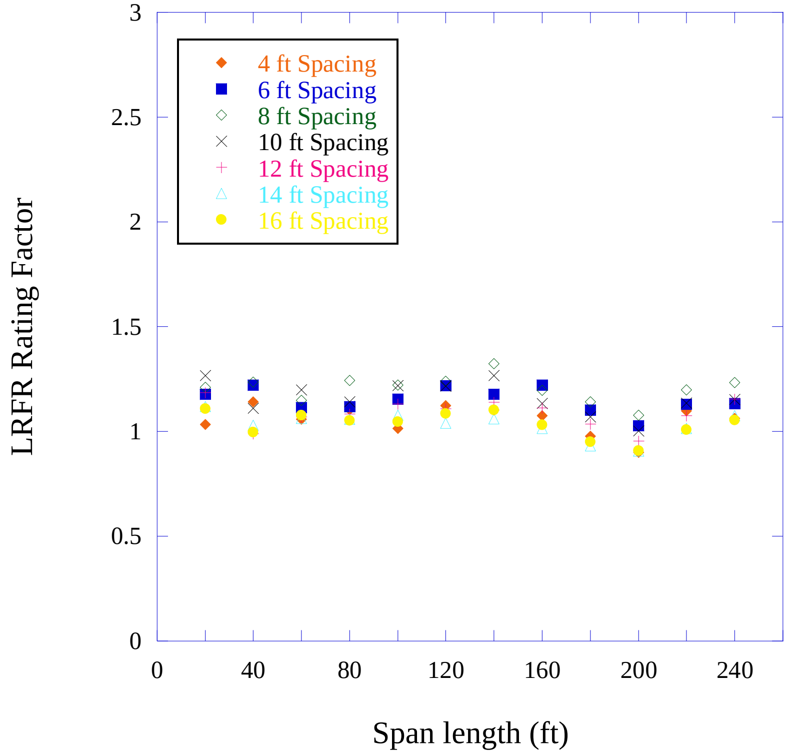


Figure 13. LRFR rating factor (design load at inventory level) versus span length (new bridges)

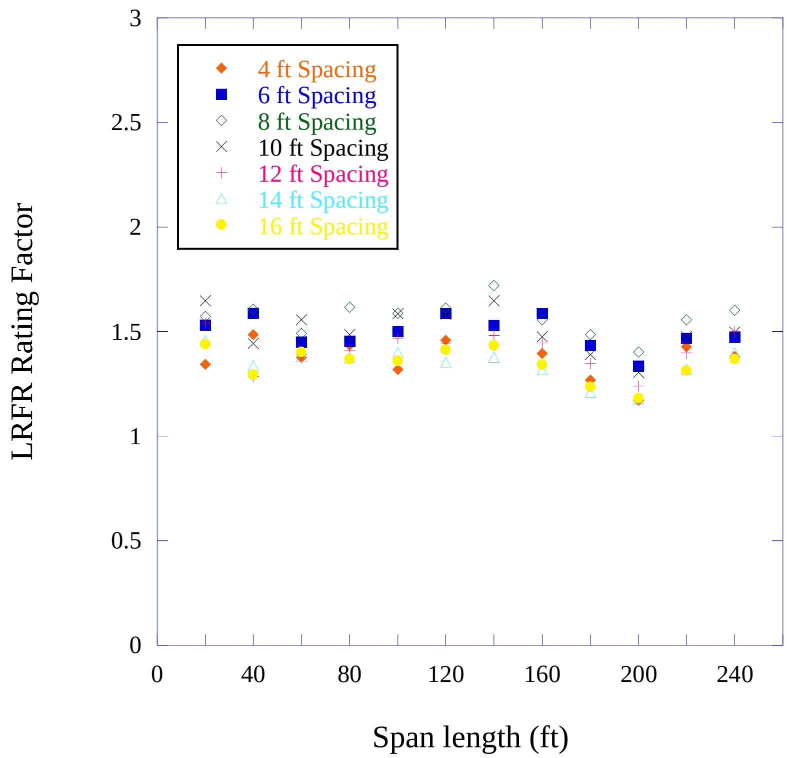


Figure 14. LRFR rating factor (design load at operating level) versus span length (new bridges)

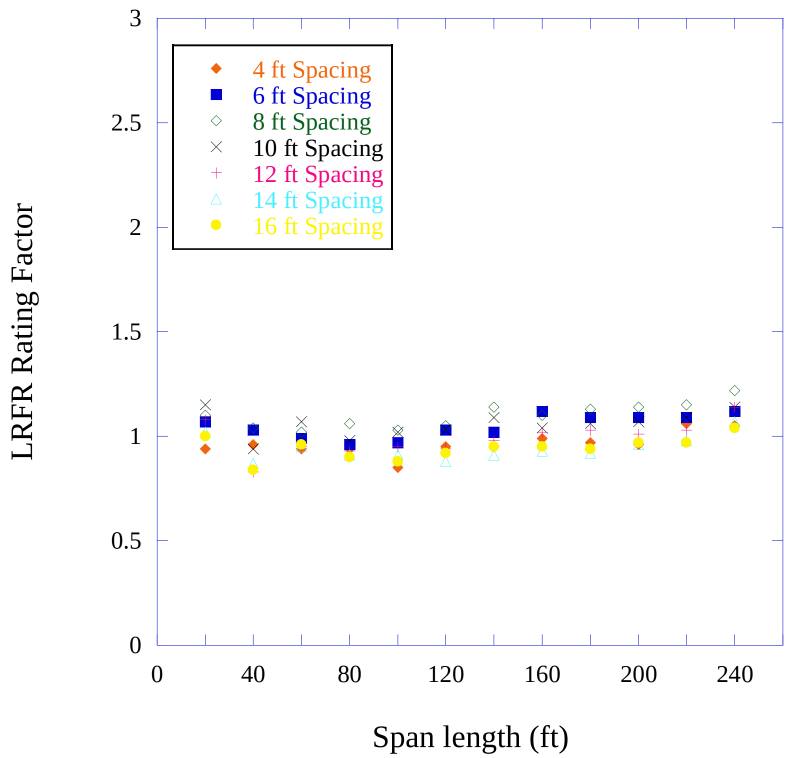


Figure 15. LRFR rating factor (permit) versus span length (new bridges)

Chapter IV
ASSESSMENT OF RELIABILITY INDICES

4.1. Unfactored Bending Moments Due to Dead Load And Live Load

In this section, the nominal dead load and live load effects are computed. These values serve as the inputs to perform reliability analysis. Computation of dead effects in this chapter is slightly different from those calculated in Chapter III because dead load components are separated differently. In Chapter III, dead load is divided into three components; weight of noncomposite dead loads such as deck, stinger (DC1), weight of composite dead loads such as curb, parapet, and railing (DC2), and weight of wearing surface and utilities (DW). In this chapter, the computation of dead load components is divided as follows: the self-weight of factory-made elements such as steel, precast concrete members (DC1), the weight of the cast-in-place concrete including the parapets (DC2), and the weight of wearing surface or asphalt overlay (DC3). Similar to dead load effect calculations in Chapter III, the bending moment and bending stress due to dead load are calculated based on the unit weights specified in AASHTO code and assuming that the dead loads are evenly distributed among the girder. Table 13 to

Table *16* illustrates the unfactored bending moments and bending stress per girder due to dead load for both interior and exterior girders of the existing bridges from NJDOT inventory. The same calculations are performed for the rest of the bridges.

Table 13 Unfactored bending moment due to dead loads on interior girder

Bridge Number	M_{DC1} (k-ft)	M_{DC2} (k-ft)	M_{DC3}(k-ft)
1	131.00	681.53	76.99
2	91.93	423.76	47.87

3	305.11	883.37	45.01
4	349.68	993.52	48.17
5	267.64	898.27	45.60
6	95.90	593.62	101.61
7	289.92	880.58	42.78
8	304.08	896.79	44.90
9	326.02	956.45	47.89
10	189.86	826.11	40.97
11	265.53	976.44	47.43
12	380.02	1287.59	64.16
13	304.63	956.34	131.84

Table 14 Unfactored bending moment due to dead loads on exterior girder

Bridge Number	M_{DC1} (k-ft)	M_{DC2} (k-ft)	M_{DC3}(k-ft)
1	196.29	685.23	76.23
2	86.31	407.54	45.34
3	295.22	760.12	36.80
4	339.46	813.44	39.38
5	227.09	790.65	38.43
6	103.44	555.49	90.47
7	280.11	778.00	35.94
8	294.36	757.81	35.64
9	315.99	808.23	38.01
10	180.96	780.90	38.14
11	255.84	929.39	44.49
12	368.74	1223.95	60.18
13	304.62	1067.80	149.81

Table 15 Unfactored bending stress due to dead loads on interior girder

Bridge Number	f_{DC1} (ksi)	f_{DC2} (ksi)	f_{DC3}(ksi)
1	1.62	8.20	0.85
2	1.90	8.24	0.78
3	2.46	6.87	0.31
4	2.60	7.13	0.30
5	2.34	7.57	0.34
6	1.98	11.65	1.53
7	2.54	7.37	0.31
8	2.45	6.96	0.31
9	2.55	7.20	0.32
10	1.91	7.98	0.33
11	2.10	7.44	0.32
12	2.62	8.62	0.38
13	2.33	7.12	0.84

Table 16 Unfactored bending stress due to dead loads on exterior girder

Bridge Number	f_{DC1} (ksi)	f_{DC2} (ksi)	f_{DC3}(ksi)
1	2.41	7.93	0.75
2	1.78	7.92	0.74
3	2.38	5.90	0.26
4	2.53	5.83	0.25
5	1.99	6.65	0.29
6	2.14	10.85	1.42
7	2.73	7.24	0.30
8	2.38	5.87	0.25
9	2.48	6.07	0.26
10	1.82	7.53	0.31

11	2.02	7.08	0.30
12	2.54	8.19	0.36
13	2.12	7.27	0.88

For live load side, the load effects are calculated based on the effects of the design load HL-93 and the NJDOT design permit vehicle for different span lengths. The configuration of NJDOT design permit vehicle is shown in Figure 8. Similar to the previous chapter, dynamic load of the NJDOT permit is specified as 25% of the truck load with the same moment *GDF* is used. The nominal resistance of each girder R_u for strength limit state remains the same as previously calculated in Chapter III while the resistance R_s for service limit state is F_y without the 95% limitation. Table 17 to Table 29 summarizes loads and resistance of each girder calculated based on the NJDOT permit load. The live load values as shown in these tables do not include *IM* and *GDF*. The calculation procedures are performed for the rest of the bridges. These values serve as nominal values in reliability analysis.

Table 17 Summary of bridge number 1

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	4452.04	131.00	681.53	76.99	2304.24
	Service (ksi)	36	1.62	8.20	0.85	24.46
Exterior	Strength (k-ft)	4662.79	196.29	685.23	76.23	2304.24
	Service (ksi)	36	2.41	7.93	0.75	24.46

Table 18 Summary of bridge number 2

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}

Interior	Strength (k-ft)	3078.54	91.93	423.76	47.87	1582.15
	Service (ksi)	36	1.90	8.24	0.78	23.29
Exterior	Strength (k-ft)	3043.47	86.31	407.54	45.34	1582.15
	Service (ksi)	36	1.78	7.92	0.74	23.29

Table 19 Summary of bridge number 3

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6405.18	305.11	883.37	45.01	2850.88
	Service (ksi)	36	2.46	6.87	0.31	17.64
Exterior	Strength (k-ft)	5749.05	295.22	760.12	36.80	2850.88
	Service (ksi)	36	2.38	5.90	0.26	17.64

Table 20 Summary of bridge number 4

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6952.66	349.68	993.52	48.17	2999.40
	Service (ksi)	36	2.60	7.13	0.30	16.97
Exterior	Strength (k-ft)	5989.04	339.46	813.44	39.38	2999.40
	Service (ksi)	36	2.53	5.83	0.25	16.97

Table 21 Summary of bridge number 5

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	5815.85	267.64	898.27	45.60	2791.40

	Service (ksi)	36	2.34	7.57	0.34	19.29
Exterior	Strength (k-ft)	5693.27	227.09	790.65	38.43	2791.40
	Service (ksi)	36	1.99	6.65	0.29	19.45

Table 22 Summary of bridge number 6

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	4798.82	95.90	593.62	101.61	1677.85
	Service (ksi)	50	1.98	11.65	1.53	23.04
Exterior	Strength (k-ft)	4437.88	103.44	555.49	90.47	1677.85
	Service (ksi)	50	2.14	10.85	1.42	23.67

Table 23 Summary of bridge number 7

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	5949.17	289.92	880.58	42.78	2750.40
	Service (ksi)	36	2.54	7.37	0.31	18.32
Exterior	Strength (k-ft)	5259.38	280.11	778.00	35.94	2750.40
	Service (ksi)	36	2.73	7.24	0.30	20.75

Table 24 Summary of bridge number 8

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6405.18	304.08	896.79	44.90	2844.30
	Service (ksi)	36	2.45	6.96	0.31	17.60

Exterior	Strength (k-ft)	5644.63	294.36	757.81	35.64	2844.30
	Service (ksi)	36	2.38	5.87	0.25	17.93

Table 25 Summary of bridge number 9

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6571.00	326.02	956.45	47.89	2986.97
	Service (ksi)	36	2.55	7.20	0.32	17.91
Exterior	Strength (k-ft)	5676.75	315.99	808.23	38.01	2986.97
	Service (ksi)	36	2.48	6.07	0.26	18.25

Table 26 Summary of bridge number 10

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	5647.37	189.86	826.11	40.97	2825.43
	Service (ksi)	36	1.91	7.98	0.33	21.28
Exterior	Strength (k-ft)	5596.86	180.96	780.90	38.14	2825.43
	Service (ksi)	36	1.82	7.53	0.31	21.35

Table 27 Summary of bridge number 11

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6507.45	265.53	976.44	47.43	3098.40
	Service (ksi)	36	2.10	7.44	0.32	19.32
Exterior	Strength (k-ft)	6454.87	255.84	929.39	44.49	3098.40

	Service (ksi)	36	2.02	7.08	0.30	19.38

Table 28 Summary of bridge number 12

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	7437.12	380.02	1287.59	64.16	3839.36
	Service (ksi)	36	2.62	8.62	0.38	21.43
Exterior	Strength (k-ft)	7389.04	368.74	1223.95	60.18	3839.36
	Service (ksi)	36	2.54	8.19	0.36	21.50

Table 29 Summary of bridge number 13

Girder	Limit State	R	DC1	DC2	DC3	M_{LL}
Interior	Strength (k-ft)	6925.90	304.63	956.34	131.84	3098.40
	Service (ksi)	36	2.33	7.12	0.84	18.20
Exterior	Strength (k-ft)	7508.17	304.62	1067.80	149.81	3098.40
	Service (ksi)	36	2.12	7.27	0.88	16.94

4.2. Statistical Parameters of Resistance and Load

The statistical parameters of the resistance of composite steel girder bridges have been studied by Tabsh and Nowak [*8*] for the AASHTO LRFD Code. Their resistance models were developed by using simulation based on available test results for materials and components. It is found that moment resistance of composite steel girder is distributed in a lognormal manner with a bias factor λ_R of 1.11 and a coefficient of variation V_R of 0.12. These statistical parameters are used in the calculation of the resistance model for Strength II limit state. To calculate the resistance model for Service II limit state, statistical parameters of steel tensile strength is used. It is found that the tensile strength of steel is distributed in a lognormal manner with a bias factor λ_R of 1.055 and a coefficient of variation V_R of 0.10. The nominal dead load and live load effects are computed similarly as in load rating. For dead load, all components are treated as normal distribution random variable. The bias ratio (ratio of mean to nominal value) of DC1, DC2 and DC3 are taken to be 1.03, 1.05 and 1.0, respectively, while their coefficients of variation are taken to be 0.08, 0.10 and 0.25, respectively, as concluded by Nowak [*4*]. Table 30 summarizes the types of probability distribution and statistical parameters of the dead load and resistance for reliability analysis.

For live load model, the statistical parameters for design load are taken from NCHRP Project 12-83 for 1 year, 5 years, and 75 years maximum load and are treated as normal distribution random variable, as shown in Table 31 [*14*]. The statistical parameters, which are mean to nominal ratio, λ, and Coefficient of Variation, CoV, for NJDOT permit load is based on the study conducted by Lou et al [*12*] as shown in Table 32. The λ and

CoV are determined for each bridge by interpolating between span lengths. Spans greater than 200 feet long use the bias and coefficient of variation for spans that are 200 feet long.

Table 30 Statistical parameters for load and resistance in reliability analysis

	Dead load			Moment resistance	Tensile strength
	DC1	DC2	DC3		
Distribution type	Normal			Lognormal	Lognormal
λ	1.03	1.05	1.0	1.11	1.055
CoV	0.08	0.10	0.25	0.12	0.10

Table 31 Statistical parameters of live load moments for ADTT 5,000

		30 ft	60 ft	90 ft	120 ft	200 ft	300 ft
1 Year	λ	1.35	1.38	1.38	1.36	1.31	1.25
	CoV	0.12	0.14	0.09	0.09	0.15	0.15
5 Years	λ	1.39	1.40	1.40	1.41	1.34	1.28
	CoV	0.13	0.12	0.08	0.11	0.15	0.15
75 Years	λ	1.42	1.45	1.45	1.46	1.40	1.31
	CoV	0.11	0.10	0.08	0.11	0.15	0.15

Table 32 Statistical parameters of live load for simple span positive moment

Span length (ft)	75 year		10 year		5 year		1 year	
	λ	CoV	λ	CoV	λ	CoV	λ	CoV
20	1.39	18%	1.29	18%	1.25	18%	1.17	18%

40	1.43	20%	1.32	20%	1.28	20%	1.19	20%
60	1.32	20%	1.22	20%	1.18	20%	1.10	20%
80	1.19	19%	1.10	19%	1.07	19%	0.99	19%
100	1.19	19%	1.10	19%	1.07	19%	1.00	19%
120	1.25	19%	1.15	19%	1.12	19%	1.05	19%
140	1.30	19%	1.20	19%	1.17	19%	1.09	19%
160	1.33	19%	1.23	19%	1.20	19%	1.12	19%
180	1.36	19%	1.26	19%	1.22	19%	1.14	19%
200	1.38	19%	1.28	19%	1.24	19%	1.16	19%
Max	1.43	20%	1.32	20%	1.28	20%	1.19	20%
Min	1.19	18%	1.10	18%	1.07	18%	0.99	18%
Average	1.31	19%	1.21	19%	1.18	19%	1.10	19%

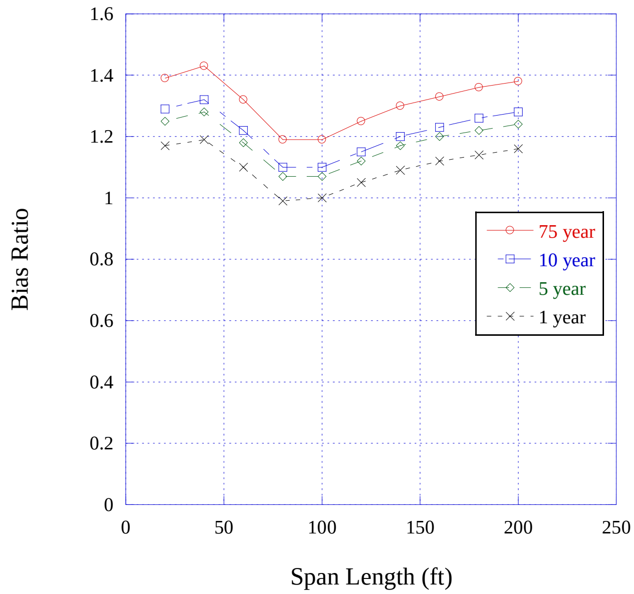


Figure 16. New Jersey permit data for simple span positive moment

4.3. Procedure for reliability analysis

A limit state function, *g*, is set up to perform the reliability analysis. This is a function of *R* and *Q*, as shown in Eq. 19. The resistance, *R*, is the bending moment carrying capacity and bending stress carrying capacity in strength limit state and service limit state, respectively. The load, *Q*, is the applied bending moment and bending stress due to dead load (DC1, DC2, and DC3) and live load. A limit state function can then be rewritten in terms of *R*, *D*, and *L*, as shown in Eq. 20.

$$g = R - Q \quad \text{Eq. 19}$$

$$g = R - Q = R - DC1 - DC2 - DC3 - L \quad \text{Eq. 20}$$

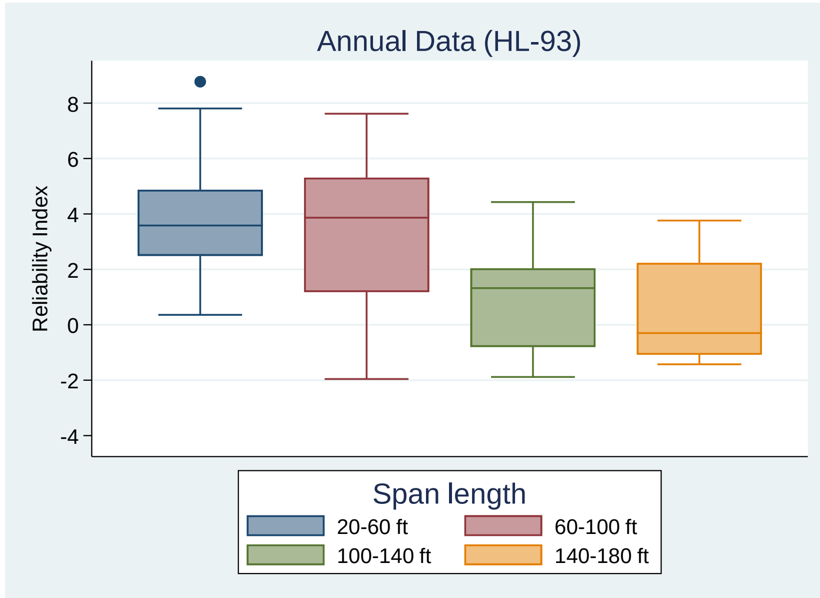
where, *R* is the resistance or load-carrying capacity, *D* is the dead load effect, and *L* is the live load effect per girder. In the study, failure refers to the case when the girder section capacity is reached. Failure occurs if g is less than or equal to zero. Structural safety can be conveniently measured in terms of the reliability index β as defined in Eq. 21.

$$\beta = \frac{\mu_g}{\sigma_g} \qquad \text{Eq. 21}$$

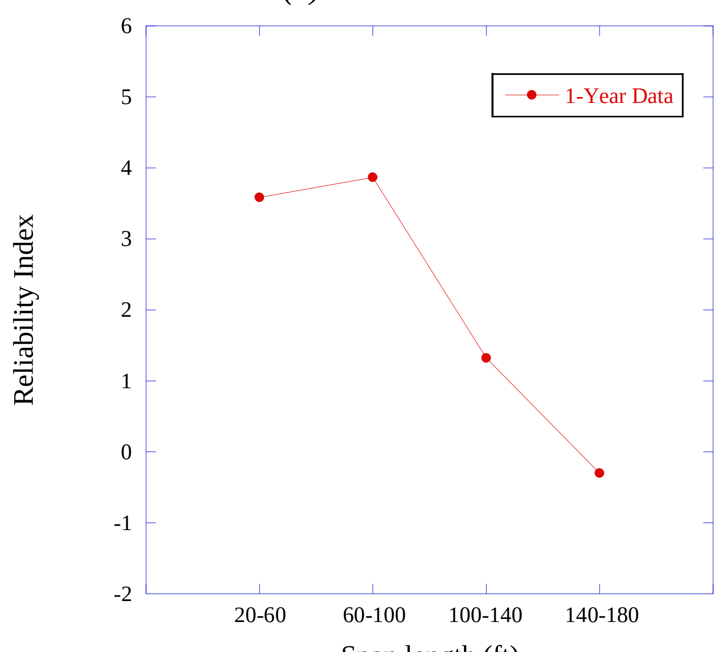
where, μ_g and σ_g denote the mean and standard deviation of the performance function g, respectively. The reliability index of each steel girder is calculated using Monte-Carlo with 100k simulation.

4.4. Reliability Indices

Once load and resistance parameters have been defined, the calculations of reliability index are then performed. With the above assumptions and statistical parameters, reliability indices of 229 steel girder bridges are calculated using Monte Carlo simulation. Based on the NCHRP report 12-78 [*15*], target flexural reliability index for routine permit vehicles was assumed to be 2.5. The NCHRP report 20-07 Task 285 presents the work on recalibration of live load factor for the permit load. It is found that although the target reliability index is set to be 2.5, the minimum value of reliability index values for all conditions is 1.50. However, the reliability index for routine permit vehicles at service limit state has not been calculated and discussed in the previous study. In this study, the reliability index of the existing steel girder bridges is investigated using available statistical data. Figure 17 shows the reliability index for different span length range (20-60 ft, 60-100 ft, 100-140 ft, and 140-180 ft) using annual data of the design load HL-93 obtained from the NCHRP project 12-83 and SHRP 2 Report S2-R19B-RW-1 using 1 lane distribution factor. Similarly, Figure 18 shows the reliability index for different span length range using annual data of the NJDOT design permit vehicle obtained from Lou et al.

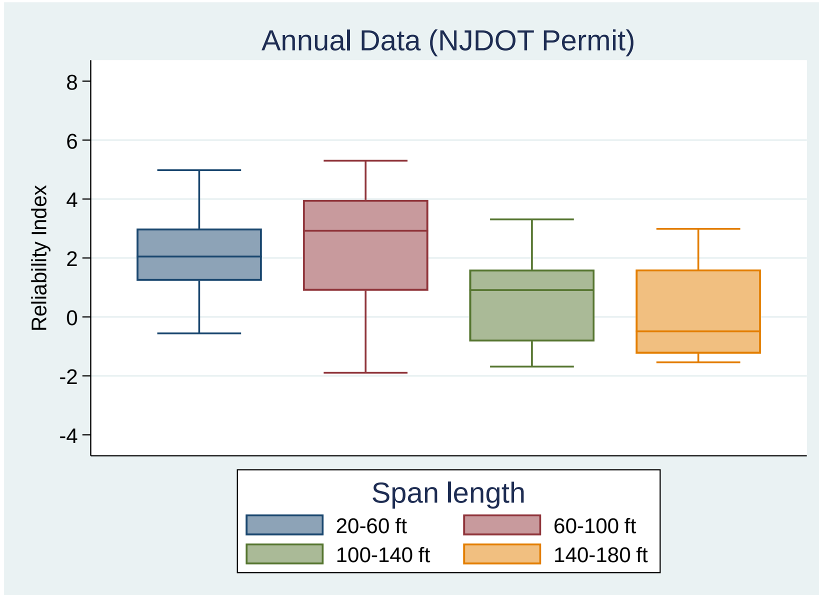


(a) annual data

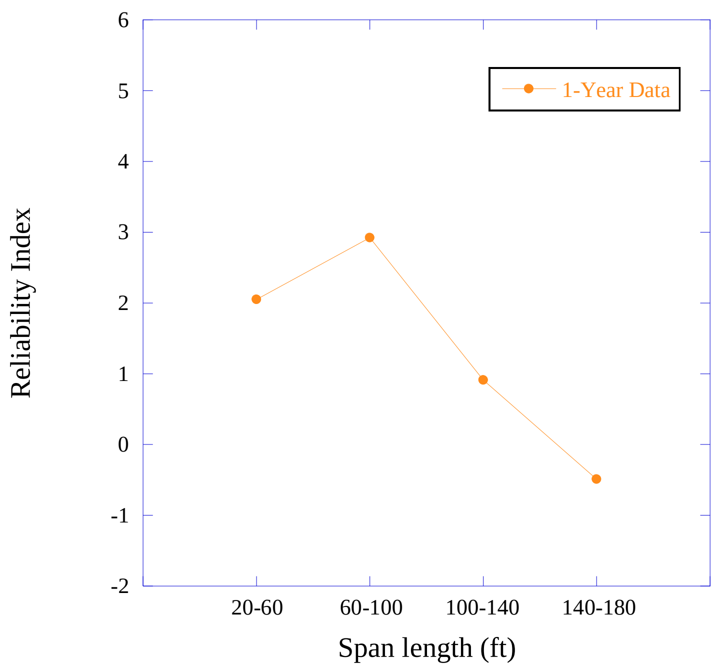


(b) Median reliability index

Figure 17. Reliability index of existing steel bridges using HL-93



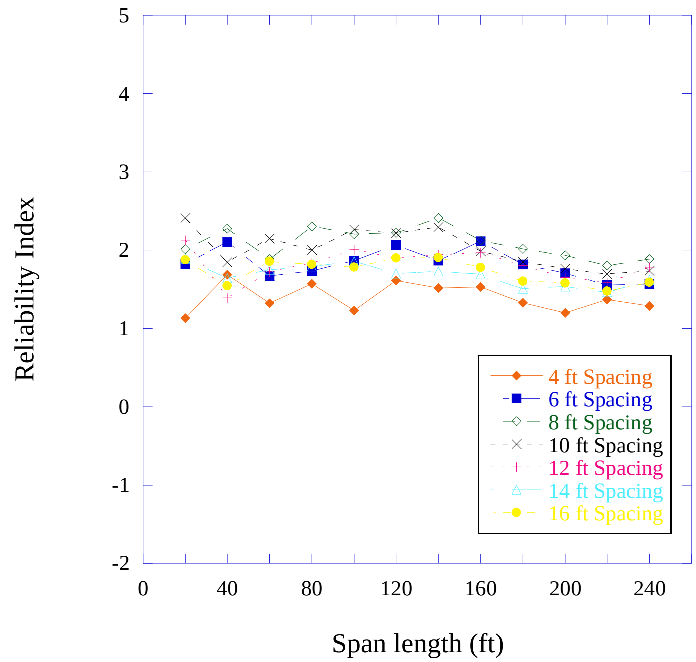
(a) annual data



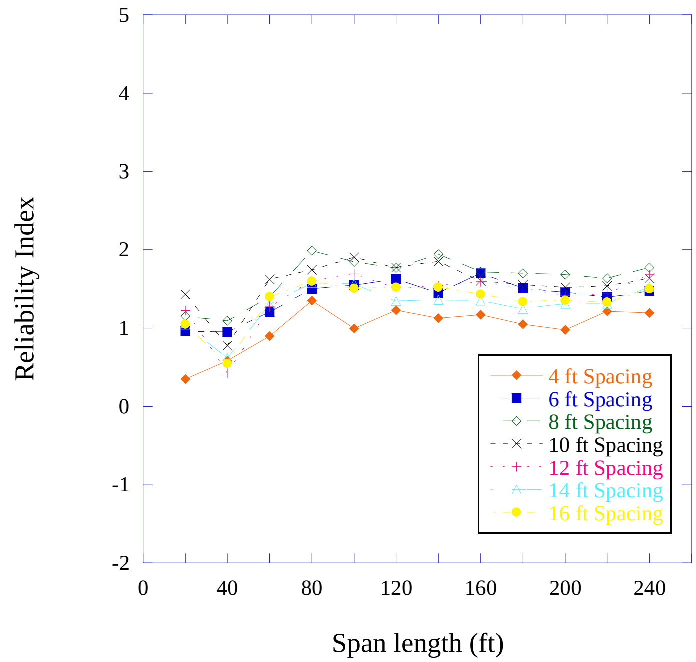
(b) Median reliability index

Figure 18. Reliability index of existing steel bridges using NJDOT permit

Figure 19 shows the reliability index of the new bridges designed according to LRFD with the minimum design. To be consistent with the work presented in SHRP 2 Report S2-R19B-RW-1, an annual data is used for design load at inventory level and permit load.



(a) Design load at Inventory level



(b) Permit load

Figure 19. Reliability index of the new steel bridges

Based on the results presented in Figure 17 and Figure 18, the average median reliability index is about 1.8 to 2 for HL-93 using annual data Similarly, the average median reliability index is about 1.35 for NJDOT permit using annual data, respectively. Based on the assessment of reliability index of the existing steel bridges, the target beta of 2.0, and 1.35 is selected and will be used to calibrate the LRFR live load factors for Service II design load at inventory level, and permit load, respectively. The target beta of about 2.0 for design load found in this study is consistent with the value presented in the SHRP 2 Report S2-R19B-RW-1.

4.4.1. Strength II limit state

The reliability indices of 84 new bridges are calculated based upon the strength of the section and also based upon the NJDOT permit load. Figure 20 shows the reliability indices versus the LRFR rating factor in Strength II limit state. Based on the results, the LRFR rating factor of 1 has a corresponding reliability index of 1.8. The result satisfies the minimum requirement of 1.5 for Strength II limit state.

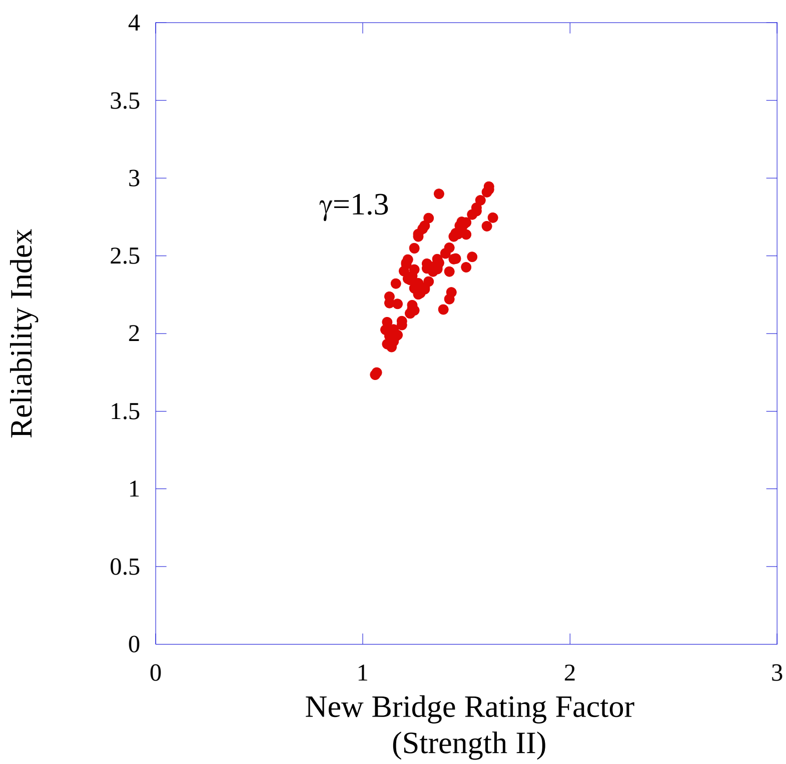


Figure 20. Reliability index versus LRFR rating factor Strength II limit state of new bridges

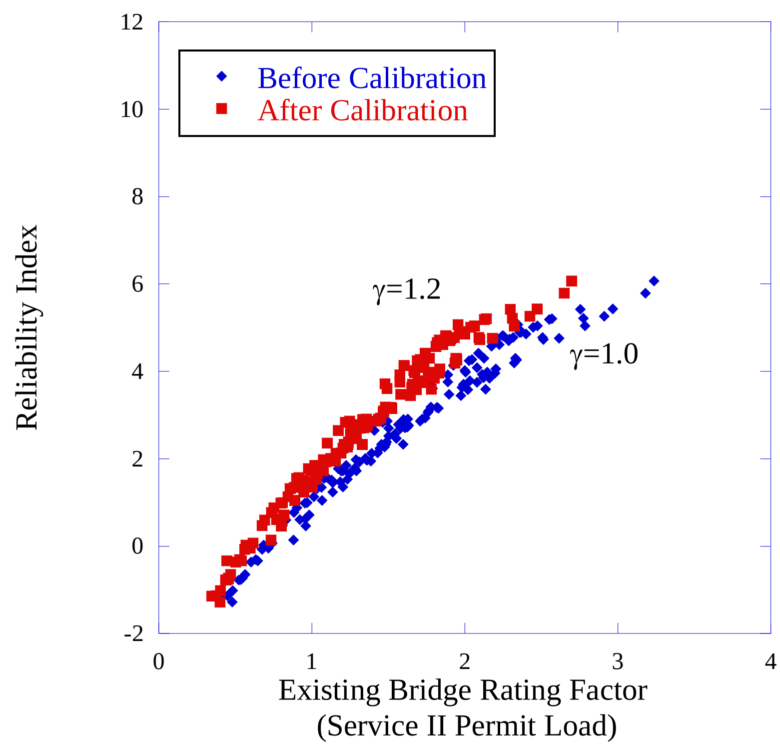
4.4.2. Service II limit state

For this section, the reliability index is calculated for both existing and new bridges. Two types of load, design load, and NJDOT permit load, are considered.

4.4.2.1. Existing bridges

The reliability indices of 145 existing steel girder bridges are calculated based upon the design load at the operating level and the NJDOT permit load. If a target reliability index has not been achieved, a new live load factor is proposed assuming dead load factors and the resistance factor remain the same. Figure 21 shows the reliability indices versus the LRFR rating factor in Service II limit state of the existing bridges before and after the live load factor calibration. Based on the results, the LRFR rating factor of 1 has a corresponding reliability index of about 1.9 for design load at inventory, which is very close to the target beta of 2. However, the rating factor of 1 for permit load has a corresponding reliability index of about 0.9. The result suggests that the current live load for permit (γ_L = 1.0) has not been calibrated based on structural reliability theory. An increased live load factor γ_L of 1.2 is found to achieve target β of 1.35 for permit load rating.

(a) Design load at inventory level



(b) Permit load

Figure 21. Reliability index versus LRFR rating factor Service II limit state of existing bridges

Alternatively, Figure 22 shows the reliability indices versus the LRFR rating factor in Service II limit state of the existing bridges before and after the calibration of resistance factor, Φ, while using the current live load factor γ_L of 1.0. A decreased resistance factor of 0.9 is found to achieve target β of 1.35 for permit load rating.

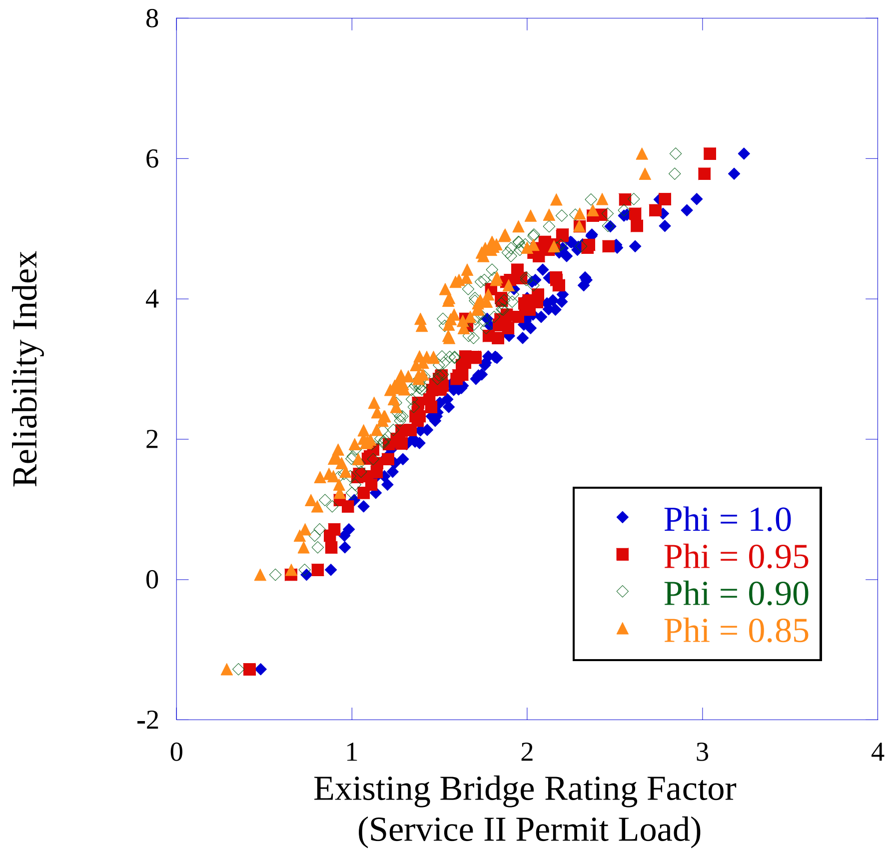
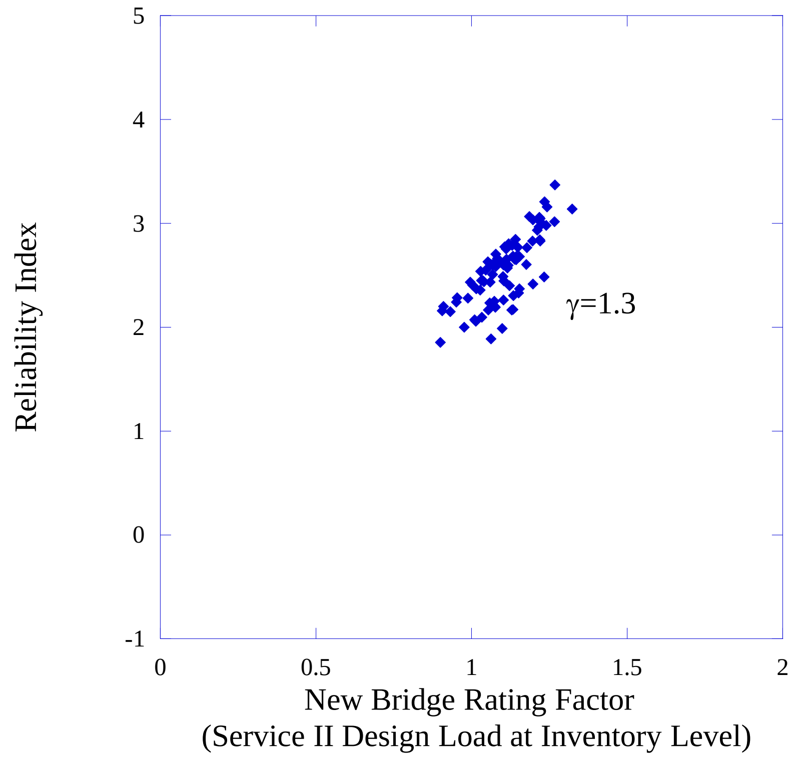


Figure 22. Reliability index versus LRFR rating factor Service II limit state of existing bridges (using different resistance factors)

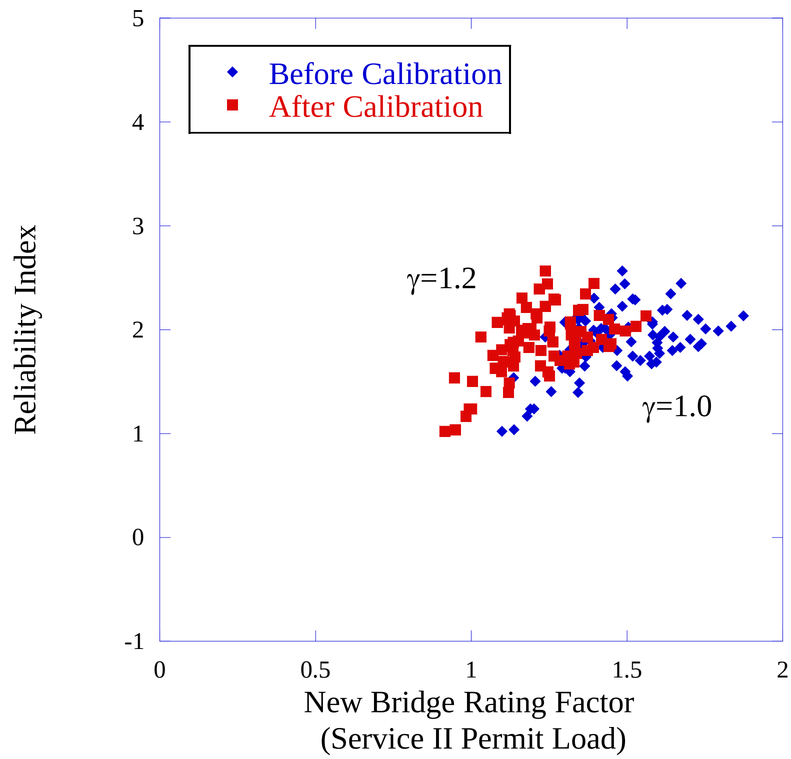
4.4.2.2. New bridges

The reliability indices of 84 new steel girder bridges are calculated based upon the design load, and the NJDOT permit load. Figure 23 shows the reliability indices versus the

LRFR rating factor in Service II limit state of the new bridges before and after calibration. Similar to the results of existing bridges, the LRFR rating factor of 1 has a corresponding reliability index of about 2 for design load at inventory, whereas the rating factor of 1 has a corresponding reliability index of about 1.0 for permit load. Again, the result suggests that the current live load for permit (γ_L = 1.0) has not been calibrated based on structural reliability theory. The same live load factor γ_L of 1.2 is found to achieve target β of 1.35 for permit load rating.



(a) Design load at inventory level



(b) Permit load

Figure 23. Reliability index versus LRFR rating factor Service II limit state of new bridges

Figure 24 shows the reliability indices versus the LRFR rating factor in Service II limit state of the new bridges before and after the calibration of resistance factor, Φ. A decreased resistance factor of 0.9 is also found to achieve target β of 1.35 for permit load rating.

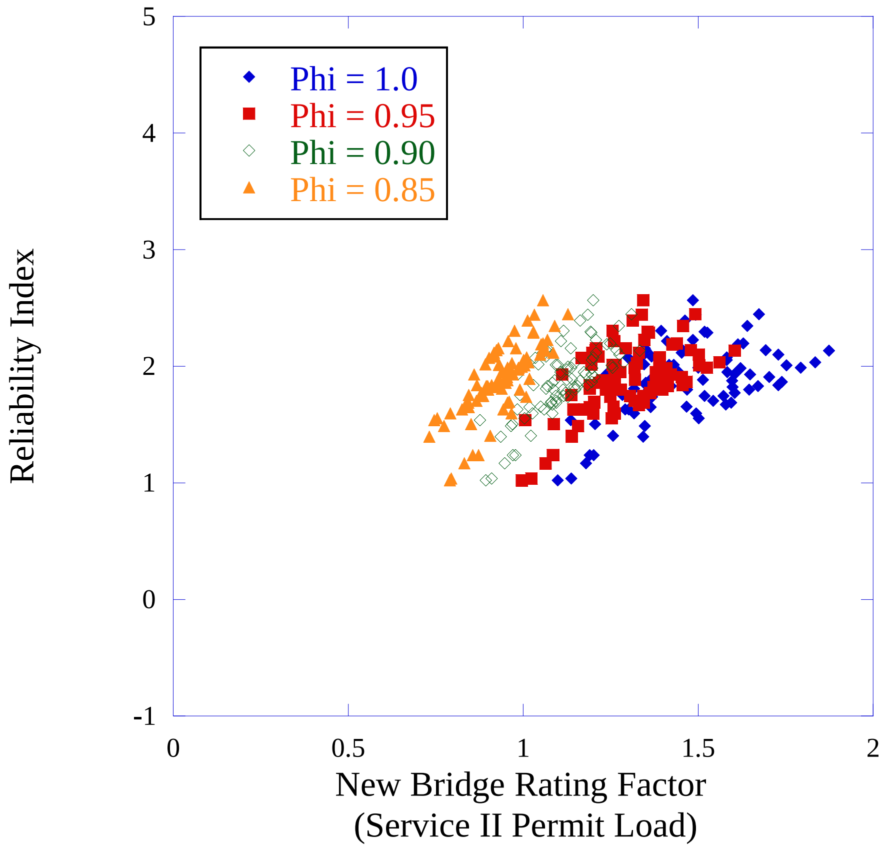


Figure 24. Reliability index versus LRFR rating factor Service II limit state of new bridges (using different resistance factors)

Chapter V
CONCLUSIONS AND RECOMMENDATIONS

This study presents the correlation between the reliability indices β and the LRFR rating factors for simply-supported composite steel girder bridges. This study mainly focuses particularly on Strength II and Service II limit state for NJDOT permit vehicle. The LRFR rating factor is calculated per the 2018 AASHTO MBE and the 2016 NJDOT Design Manual for Bridges and Structures. The current MBE live load factor for a routine permit of 1.3 and 1.0 is used to calculate LRFR rating factors for Strength II and Service II limit state, respectively. The live load factor for design load of 1.3 is used to calculate the rating factors for Service II limit state at inventory level. The reliability index of each bridge is calculated based on the available dead load and live load statistical parameters and reliability analysis using Monte Carlo with 100k simulation. Based on the results in this study, the comparison between LRFR rating factor and corresponding reliability index is made and the conclusions could be summarized as follows:

1. All steel girder bridges being evaluated in Strength II limit state (routine permit) have LRFR rating factor greater than 1.
2. From the LRFR perspective, all bridges are sufficient to carry the NJDOT permit, based on the results.
3. Based on the results in this study, it is found that the LRFR rating factor of 1 corresponds to the reliability index of 1.8 for Strength II limit state, as compared to the target value of 2.5 from the NCHRP project 12-78. However, the steel girder bridges still achieve the minimum reliability index of 1.5 with the live load factor γ_L = 1.3.

4. For Service II limit state, it is found that not all the bridges have LRFR rating factor greater than or equal to 1. The failure in service limit state implies to yielding of steel at the bottom flange and does not mean the bridge would collapse.
5. Based on the results in this study, it is found that the LRFR rating factor of 1 for existing steel bridges corresponds to the reliability index of about 1.9 for design load at inventory, and about 0.9 for permit load. While for the new bridges, it is found that the LRFR rating factor of 1 corresponds to the reliability index of about 2 for the design load at inventory, and about 1 for permit load.
6. Based on the results in this study, it shows that the current LRFR live load factor for permit load in Service II limit state does not truly reflect the current level of serviceability of the steel girder bridges.
7. Based on the results in this study, it has been proven that the current live load factors in Service II limit state for permit vehicle is not calibrated based on structural reliability theory. Therefore, the proposed live load factor of 1.2 for Service II limit state is recommended for the permit load to achieve the reliability index of 1.35 which was found to be the target beta in this study.
8. Based on the results in this study, a decreased resistance factor, φ, of 0.9 is recommended for Service II limit state for permit load to achieve the same reliability index of 1.35, while keeping the current LRFR live load factor of 1.0.
9. This recommendation may be valid to be used for New Jersey, since the permit configuration may vary from state to state.

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