University Transportation Research Center - Region 2

Final Report

LOAD AND RESISTANCE FACTOR RATING (LRFR) IN New York State - Volume I

Performing Organization: The City College of New York/CUNY

September 2011

Sponsors:
New York State Department of Transportation
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University Transportation Research Center - Region 2

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PROJECT NO. C-06-13

LOAD AND RESISTANCE FACTOR RATING (LRFR) IN NYS

Volume I

Final Report

by

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16. Abstract

This report develops a Load and Resistance Factor Rating (NYS-LRFR) methodology for New York bridges. The methodology is applicable for the rating of existing bridges, the posting of under-strength bridges, and checking Permit trucks. The proposed LRFR methodology is calibrated to provide uniform reliability index values for all applications. The reliability calibration of live load factors is based on live load models developed using Truck Weigh-In-Motion (WIM) data collected from several representative New York sites. The live load models provide statistical projections of the maximum live load effects expected on New York bridges.

A new set of NYS Legal Trucks along with appropriate live load factors are proposed for performing Operating Level Ratings of existing bridges. Permit load factors are calibrated for unlimited crossings of divisible loads as well as single crossings and unlimited crossings of non-divisible loads. An equation is proposed for determining Posting weight limits for bridges with low Rating Factors as a function of the effective span length. It is proposed that different posting weights be imposed for single unit trucks and semi-trailer trucks.

17. Key Word	18. Distribution Statement
Bridges, Load and Resistance Factor Rating, LRFR, Load Posting, Permit Truck Checking, Reliability Index, Code Calibration, Weigh-In-Motion, Load Model, Live Load Factor, Rating Factor, Operating Rating, Inventory Rating.	No Restrictions

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PROJECT NO. C-06-13

LOAD AND RESISTANCE FACTOR RATING (LRFR) IN NYS

TABLE OF CONTENTS

4.2 Future Research...158

EXECUTIVE SUMMARY

This report proposes a Load and Resistance Factor Rating (LRFR) methodology for New York State bridges. The methodology is applicable for the rating of existing bridges, the posting of under-strength bridges, and checking Permit trucks. The proposed LRFR methodology is calibrated based on a target reliability index β_{target}=2.0 which has been set to provide somewhat more conservative ratings than current NYSDOT procedures. The calibration process also aims at producing a tight range of reliability index values such that the minimum reliability index does not fall below β_{min}=1.50 for all applications. The reliability calibration of live load factors is based on live load models developed using Truck Weigh-In-Motion (WIM) data collected from several representative New York sites. The live load models provide statistical projections of the maximum live load effects expected on New York bridges.

The safety evaluation of existing New York bridges can be executed using a LRFR equation that takes the form:

$$\boldsymbol{R.F.} = \frac{\phi_c \phi_s \phi R_n - \gamma_{DC}\left(D_{C1} + D_{c2}\right) - \gamma_{DW} D_W}{\gamma_L L_n}$$

where R.F. is the rating factor, R_n is the nominal resistance, D_{C1} is the dead load effect of pre-fabricated components, D_{C2} is the dead load effect of cast-in-place components and attachments, D_W is the nominal dead load effect for the wearing surface, L_n is the live load effect of the nominal load used to calculate the Rating Factor including dynamic allowance and load distribution factor, ϕ is the resistance factor, ϕ_c is the condition factor, ϕ_s is the system factor, and γ are the load factors. A Rating Factor R.F. $\geq$ 1.0 indicates that the trucks whose live load effects can be modeled by L_n can safely cross the bridge.

This project calibrated appropriate condition factors, ϕ_c, and live load factors, γ_L, based on NYSDOT current practices and collected WIM data. The other factors remain as provided in the AASHTO LRFR.

Table 1 provides the recommended NYS-LRFR condition factors. These factors have been calibrated to produce similar changes in the reliability levels as those obtained when using the different rating criteria of NYSDOT which change based on primary member condition rating.

Table 1. NYS-LRFR Condition Factor: ϕ_c.

Structural Condition of Member	Condition Rating	ϕ_c
Fair, satisfactory or good	$\geq$ 4 on NYS scale of 1. to 7.	1.0
Poor	$\leq$ 3 on NYS scale of 1. to 7.	0.95

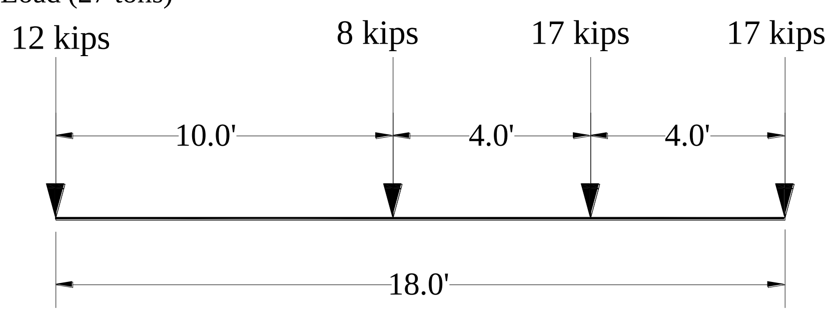
The analysis of the WIM data showed that New York State live loads are significantly higher than the live loads assumed during the calibration of the AASHTO LRFD and LRFR specifications particularly for single lane bridges. This required the adoption of a new set of NYS Legal Trucks along with appropriate live load factors for use in performing Operating Level Ratings of existing bridges. Figure 1 provides the proposed NYS Legal Trucks while Table 2 lists the proposed NYS Live Load factors for Legal Truck Load Operating Rating.

Table 2. NYSDOT Live-Load Factors, γL for Legal Loads

Traffic Volume (one direction)[1]	Load Factor for Multi-lane bridges (use LRFD load distribution factor for multi-lanes)	Load Factor for Single-lane bridges (use LRFD load distribution factor for a single lane without removing the multiple presence factor)[2]
ADTT $\geq$ 5000	1.95	2.65
ADTT=1000	1.85	2.50
ADTT $\leq$ 100	1.65	2.20

[1] Linear interpolation is permitted for other ADTT
[2] The AASHTO LRFD load distribution factor tables for single loaded lanes already includes a multiple presence factor MP=1.2. This factor must be included when the analysis employs other methods for determining the load on a bridge member.

a) SU4 Legal Load (27 tons)



b) Type 3S2 Legal Load (36 tons)

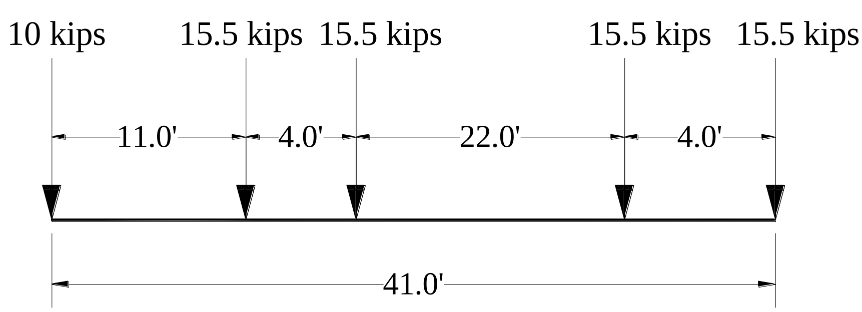


Figure 1. Proposed New York State Legal Trucks for bridge rating.

Permit load factors are calibrated for divisible loads and non-divisible loads for single crossings as well as unlimited crossings of bridges. The calibration of the permit load factors was based on the analysis of multiple presence probabilities and on the uncertainties associated with estimating the load effects of the permit trucks and those of the random trucks that may cross simultaneously with the permit. Accordingly, lower permit load factors are recommended than those in the AASHTO LRFR. The permit load factors are listed in Table 3.

Table 3. NYSDOT Permit Load Factors, γ_L

Permit Type	Frequency	Loading Condition	DF	ADTT (one direction)[1]	Permit Load Factor, γ_L
Annual Divisible Load	Unlimited trips	Multi-lane bridges Mix with traffic	Multi-lane	ADTT $\geq$ 5000	1.20
				ADTT=1000	1.15
				ADTT $\leq$ 100	1.10
Annual Divisible load	Unlimited trips	Single lane bridges	Single Lane DF after dividing out MP=1.2	ADTT $\geq$ 5000	1.20
				ADTT=1000	1.15
				ADTT $\leq$ 100	1.10
Non-divisible loads	Unlimited trips	Multi-lane bridges Mix with traffic	Multi-lane	All ADTT	1.10
Non-Divisible loads	Unlimited trips	Single lane bridges	Single Lane DF after dividing out MP=1.2	All ADTT	1.10
Special Hauling and Superloads	Single Crossing	Multi-lane bridges Mix with traffic	Single Lane DF after dividing out MP=1.2	All ADTT	1.10
Special Hauling and Superloads	Single Crossing	Single lane bridges	Single Lane DF after dividing out MP=1.2	All ADTT	1.10

[1] Linear interpolation is permitted for other ADTT

An equation is proposed for determining Posting weight limits for bridges with low Rating Factors as a function of the effective span length. The posting equation was calibrated so that posted bridges will meet the same target reliability β_{target}=2.0 used for the Legal Load Ratings and the Permit weight checks. The posting weight calibration however was based on several assumptions regarding the probability of having overweight trucks cross posted bridges. It is proposed that two different posting weights should be provided one for single unit trucks and the other for semi-trailer trucks. The proposed equation is given as:

$$Safe\ Posting\ Load = W\left[RF + 0.00375(L-110)(1-RF)\right]$$

where W = Weight of Rating Vehicle (27 Tons for Single Trucks,
or 36 Tons for semi-trailers)
RF= Legal Load Rating Factor for the governing NYS-Legal Truck
L = Effective span length in feet

CHAPTER ONE

INTRODUCTION

1.1 Background

The American Association of Highway and Transportation Officials (AASHTO) has recently adopted the Load and Resistance Factor Rating (LRFR) Specifications (2003) and included it into the Manual for Bridge Evaluation, MBE (2008). All the states are expected to follow this newly developed approach during the safety assessment of existing bridges or the development of policies for permit load issuance and for load posting substandard bridges. The LRFR are currently allowed by the Federal Highway Administration (FHWA) for load rating new and existing bridges and a growing number of states are already making progress towards implementing the LRFR into their bridge safety assessment practices. Over the next few years, all bridges designed using the Load and Resistance Factor Design Specifications, LRFD (2007), will be required to be load rated by the LRFR Specifications. The LRFR developed by Lichtenstein Engineering Associates (Lichtenstein) was calibrated following a reliability-based procedure compatible with that adopted during the development of the AASHTO Load and Resistance Factor Design (LRFD) specifications (Moses, 2001).

The main purpose of the LRFR and LRFD codes is to account in a rational manner for the uncertainties associated with determining the load carrying capacity of new and existing bridges as well as the uncertainties associated with estimating the loads to be applied. The LRFR specifications were calibrated to provide uniform reliability levels represented by a target reliability index β=3.5 for inventory rating and β=2.5 for operating rating. The former target value was selected in order for the LRFR to remain consistent with the LRFD specifications, while the latter value is equal to the upper range of reliability indices for a sample of bridges that satisfy the Allowable Stress Operating Ratings.

Although the calibration of the LRFR methodology followed rational and technically sound methods, some bridge agencies have voiced concerns that certain LRFR procedures and load factors calibrated for national use may not be entirely compatible with their particular procedures. Research studies have shown that some of the differences between the load ratings obtained from LRFR and those from traditional procedures are due to the fact that the LRFR design load rating is based on the heavier AASHTO LRFD HL-93 design loading rather than the AASHTO standard HS-20 or H trucks which are the most widely used criteria for current traditional ratings. However, the research studies have also found that in some instances and even after accounting for the effects of the different truck weights and configurations, the standard LRFR procedures still leads to more conservative ratings than the traditional Allowable Stress Rating (ASR) and Load Factor Rating (LFR) procedures. This is attributed to the fact that the reliability index value of β=2.5 used for calibrating the operating level load rating in LRFR is higher than the reliability index value implicit in traditional procedures.

While the LRFR target reliability index may have been conservatively selected, recent observations made on truck weight data collected from Weigh-In-Motion stations (WIM) at representative NY State sites (Sivakumar et al, 2008) have shown that trucks travelling over the State's highway system can be significantly heavier than the generic truck weight data used during the calibration of the AASHTO LRFD and LRFR specifications. In fact, the generic live load models used during the calibration of the LRFR were based on data collected in the mid and late 1970's in Canada and may not represent current US or New York State loads (Nowak, 1999).

Recognizing the limitations of the generic truck weight data and the conservative reliability targets selected during the calibration process, the LRFR specifications provide sufficient flexibility and allow state agencies to adjust the LRFR load factors based on their individual conditions and site-specific or state-specific information. The goal would be to develop an LRFR process that is compatible with current state procedures while considering their particular loading conditions.

The LRFR provides specific instructions as to how the recalibration of the load factors can be executed based on state-specific or site-specific Weigh-In-Motion (WIM) data. These instructions assume that WIM data of sufficient quality and quantity are available particularly in the upper tail region of the truck weight histogram. The live load factors provided in the LRFR manual can thus be recalibrated by the same statistical method used originally in the development of the LRFR code. By using the same statistical method, the reliability index values used in the original development of the LRFR are maintained and the new factor will only reflect the differences in the truck weight data. In maintaining the same reliability levels, it is assumed that these reliability index values are satisfactory and are compatible with current procedures, which may not be always true given that the reliability index target value used during the LRFR calibration was equal to the upper range of reliability indices for a sample of bridges that satisfy the Allowable Stress Operating Ratings. Thus, a more consistent approach would require the recalibration process to first include a review of the reliability levels implicit in a State's current practice, determine the adequacy of current procedures based on state truck weight data and the experience of the state bridge engineers, and finally perform a new calibration of the live load factor based on the evaluation of the current reliability levels.

1.2 Research Objectives

The New York State Department of Transportation (NYSDOT) intends to revise its current policies to adopt LRFR as the analytical method for load rating and load posting of bridges and for evaluating overweight permit vehicles. The objectives of this project are to calibrate a NYS-LRFR methodology and develop load capacity evaluation and bridge posting and permit issuance guidelines that are consistent with current NYSDOT procedures. Current NYSDOT procedures, which are based on traditional AASHTO Load Factor Rating methods, have had a proven track record in terms of providing safe and reliable bridges in New York State. There is however some concern regarding the increasing number of overweight trucks that are travelling over the New York highway network. Therefore, any changes to the current methods should take into consideration the current levels of structural reliability, determine an appropriate reliability target based on the experience of the bridge engineers and ensure that this target level is uniformly maintained for the whole range of applications.

The objectives of this study are to review current NYSDOT load rating, load posting and overweight permit policies to ascertain the level of reliability implicit in current procedures. The project then aims to calibrate new NYS-LRFR live load factors that will lead to ratings that are similar to current NYSDOT practice and yet provide uniform and consistent levels of bridge safety and reliability over all pertinent bridge classes and configurations. The proposed load factors must reflect current bridge loading conditions in New York State as measured through the array of WIM sites established by NYSDOT. In order to be consistent with the LRFR philosophy, the proposed NYS-LRFR must be calibrated using sound structural reliability procedures based on statistical load and resistance models that actually represent the typical loading conditions observed throughout the state of New York.

1.3 Research Approach

The discussion presented above highlights the following important points:

1. Any rational procedure for bridge load rating and safety assessment must account for the uncertainties associated with estimating the strength of bridge members and the applied loads. Thus, any changes in existing safety evaluation or load rating procedures must be consistent with the principles of structural reliability.
2. The purpose of bridge rating specifications is to provide a simple to use procedure that will produce uniform and consistent levels of reliability for all bridge types and classifications.
3. The existing LRFR load factors were developed based on outdated generic truck load data from a single site in Ontario Canada that may not be necessarily representative of the loads currently observed on New York state bridges. New load factors must be re-calibrated to reflect unbiased load data collected from the array of WIM systems spread throughout New York State.
4. LRFR load factors calibrated to a somewhat conservative reliability target may lead to the posting and closure of bridges which, based on past experiences, have been known to provide adequate levels of safety. On the other hand, increases in the frequency of overweight trucks may imply increased risks to current bridge networks.

5. Therefore, the reliability level to be used during the bridge code calibration must be compatible with the experience of the bridge owners and expert engineers gathered over years of practice in bridge evaluations and safety assessment in order to ensure the safety of the traveling public without being overly conservative and unnecessarily overburdening existing budget constraints.

To resolve the above mentioned issues, particularly items 3, 4 and 5, this research report will first review the reliability levels implied in existing NYSDOT load rating and posting procedures. A recalibration of the live load factors of the current LRFR is undertaken to provide reliability levels consistent with those implied in NYSDOT evaluation practices that have been known to provide adequate levels of safety. The report also proposes new bridge load rating, load posting and overweight permitting guidelines which are consistent with the experience, current practice, and policies of the NYSDOT, that reflect the current loading conditions, and are also consistent with the AASHTO LRFR methodology and uniform reliability goals.

The recalibration approach followed to achieve the objectives of this study is consistent with reliability-based code calibration procedures as described by Moses (2001) and Nowak (1999) applied for the calibration of load factors for the LRFR bridge evaluation and LRFD bridge design specifications. The approach consists of the following steps:

1. Select a representative sample of bridges covering a whole range of load rating levels and span lengths, configurations and material types.
2. Collect Weigh-In-Motion data that provide a good representation of the truck weights and headways at typical New York bridge sites. The data should include truck weights and configurations as well as multiple presence statistics.
3. Obtain Load Factor ratings and load posting levels for the representative set of bridges based on current procedures.
4. Obtain permit load ratings for these bridges for typical configurations of Divisible Load permits and Special Hauling permits.
5. Perform reliability analyses and deduce the reliability index values implicit in current NYSDOT methods for all the bridges.
6. Select an appropriate target reliability index. The target reliability should provide adequate levels of safety for typical bridges under current NY State loading conditions without penalizing bridges that have demonstrable safe performance based on NYSDOT experience.
7. Use the selected target reliability and New York State WIM loads to re-calibrate LRFR load factors so that bridges of various types, configurations and span lengths that give a rating factor R.F.=1.0 will meet the target reliability as closely as possible. These proposed load factors should be adjusted for nominal live loads consisting of NY state legal loads and for representative permit loads.
8. Rate sample bridges using proposed NYS-LRFR with new load factors.

9. Compare proposed NYS-LRFR ratings to current NYS-LFR ratings and explain differences and fine tune calibration based on results of comparisons.
10. Develop New York State Load Rating/Posting Guidelines for State-Owned Highway Bridges

1.4 Report Outline

The research study described in this report consisted of the following main tasks:

1. Review of current NYSDOT procedures
2. Review of national practice
3. Calibrate NYS-LRFR load rating factors
4. Develop NYS-LRFR load posting methodology
5. Develop NYS-LRFR live load factor for overweight permits
6. Compare proposed NYS-LRFR to current NYSDOT procedures
7. Present the results in a NYS Engineering Instructions document.

This report describes the calibration procedures followed during the research study and presents the final recommendations for a new NYS-LRFR Load Rating, Posting and Permit Issuance procedure. The Report consists of the following four Chapters and four Appendices assembled in two volumes.

Volume I:

1. Chapter 1 gave a brief review of this research project's background and motivation and outlined the research objectives and approach.
2. Chapter 2 describes in detail the research methodology including a review of the reliability-based calibration method and the data base used in this study to calibrate the proposed NYS-LRFR methodology. The validity of the methodology is verified by comparing the results to those previously obtained during the calibration of the AASHTO LRFD and LRFR.
3. Chapter 3 describes the implementation of the research methodology to develop the NYS-LRFR live load factors for legal trucks and permit trucks and proposes a load posting procedure for under-strength bridges.
4. Chapter 4 gives the conclusions of this study including possible future research.

Volume II:

1. Appendix I which provides the proposed NYSDOT Engineering Instruction Manual.
2. Appendix II presents a comprehensive review of current NYSDOT load rating, load posting and permit issuance procedures.
3. Appendix III presents a review of National Practice.
4. Appendix IV compares the ratings obtained from the proposed NYS-LRFR procedures to those obtained from the current method.

CHAPTER TWO

RESEARCH APPROACH

The calibration of the AASHTO LRFR live load factors was performed using a structural reliability framework based on the models and a generic database described in NCHRP report 454 (Moses, 2001). The database was primarily extracted to provide results that match those used during the calibration of the AASHTO LRFD specifications as reported by Nowak (1999). Recognizing that the current truck loads in different states may be different than the generic data that was originally used, the AASHTO LRFR provides enough flexibility for each state to modify the LRFR live load factors to reflect its loading conditions based on truck Weigh-In-Motion data. The simplified approach proposed in the AASHTO LRFR for modifying the live load factors preserves several assumptions about the target reliability level and the shape of the truck weight spectra that may not be consistent with the truck weights. An alternative approach for modifying the live load factors would require the development of state-specific live load models and performing a reliability-based calibration following the same methods applied during the calibration of the AASHTO LRFR and LRFD.

This Chapter presents a methodology for developing live load models based on Truck Weigh-In-Motion data collected from several representative sites. The approach is applied to obtain statistical projections of the maximum live load effects expected on New York bridges. In addition, a review of the AASHTO LRFR and LRFD calibration process is undertaken to verify the validity of the procedures followed in this study to calibrate a New York State Load and Resistance Factor Rating methodology (NYSDOT-LRFR). The live load models obtained in this Chapter are subsequently used in Chapter 3 to calibrate New York state live load factors for legal load rating, permit load rating and the load posting of deficient bridges.

2.1 Background

In this Chapter, a review of the reliability-based calibration methodology performed in NCRP Report 454 is presented to evaluate the reliability index values associated with the AASHTO LRFR procedures for load rating. The live load model provides an essential part of the reliability calibration process. The AASHTO LRFR calibration made several assumptions on the truck weight spectra in order to produce a live load model that matches the one developed by Nowak (1999) during the calibration of the AASHTO LRFD specifications. The database used during the calibration of the AASHTO LRFD specifications was adopted from a truck survey conducted in the 1970’s in Ontario Canada. Kulicki et al (2007) explain that the Ontario database was biased in the sense that only the trucks that appeared to be heavy were flagged for weighing. In his review of the Nowak (1999) results, Moses (2001) concluded that the Ontario data approximately represented the heaviest 20% of the trucks that crossed the highway at the survey site. By assuming that the average weight of these heavy trucks is 68 kips and the standard deviation is 18 kips and that the weight spectrum follows a Normal probability distribution, a good match between the AASHTO LRFR and LRFD load models was obtained. In this Chapter, the live load model used by Moses (2001) is compared to the live load model used by Nowak (1999). Also, a comparison between these two models and the data collected from five New York Weigh-In-Motion (WIM) sites is performed.

In addition to the live load model, the reliability analysis requires as input the statistics of the resistance and the dead loads. This information also includes the type of the probability distribution of the random variables. Moses (2001) assumed that both the resistance and the total load follow Lognormal distributions and used the Lognormal model for the calculation of the reliability index, β. On the other hand, Nowak (1999) assumed that the load follows a Normal distribution while the resistance follows a Lognormal distribution. A comparison between the results of the two models is performed in this report to study the implication of the assumptions made on the final results.

The initial assumption by Nowak (1999) that on the average 1000 Heavy Trucks cross a typical highway bridge in given day was subsequently modified as reported by Kulicki et al (2007) to further increase this number to 5000 Trucks per day. In both cases, the same biased Ontario database was used to develop the live load model. Furthermore, both Nowak (1999) and Moses (2001) conservatively assume that for bridges with 5000 ADTT the probability that two heavy trucks may cross a bridge side-by-side is 6.67% (1 in 15 trucks are side-by-side). These assumptions on the percentage of side-by-side trucks are compared to recently collected New York Weigh-In-Motion data on truck headways and modified as necessary for implementation in the development the New York live load model.

In this Chapter, a review of the approach, data and models used during the calibration of the AASHTO LRFD and LRFR live load factors is performed. Also, the Weigh-In-Motion data collected at five representative New York sites is analyzed. A procedure to use the data to obtain state-specific live load models is presented and implemented. These results and methods are then applied in Chapter to calibrate New York state live load factors for legal load rating, permit load rating and the load posting of deficient bridges. Specifically, Section 2 of this Chapter gives a review of structural reliability methods. Section 3 describes the models and data base for the dead weights and resistance used by Nowak (1999) and Moses (2001) during the calibration of the AASHTO LRFD and LRFR codes. Section 4 performs the analysis of the New York WIM data and presents a procedure to develop state-specific live load models. Section 5 implements the proposed procedure and presents the New York live load model. Section 6 reviews the reliability index calculations for a number of cases to verify that the reliability analysis method followed in this study is consistent with the methods used during the calibration of the AASHTO LRFD and LRFR.

2.2 Basic Concepts of Structural Reliability

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

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

$$V_R = \frac{\sigma_R}{\overline{R}} \tag{2.1}$$

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

$$\overline{R} = b_r\, R_n \tag{2.2}$$

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

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

Let the reserve margin of safety of a bridge component be defined as, Z, such that:

$$Z = R - S \tag{2.3}$$

Where R is the resistance or member capacity, S is the total load effect. Probability of failure, $P_{f,}$ is the probability that the resistance R is less than or equal to the total applied load effect S or the probability that Z is less or equal to zero. This is symbolized by the equation:

$$P_f = \Pr [R \leq S] \tag{2.4}$$

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

$$P_f = \Phi\left(\frac{0-\bar{Z}}{\sigma_Z}\right) = \Phi\left(-\frac{\bar{R}-\bar{S}}{\sqrt{\sigma_R^2+\sigma_S^2}}\right) \tag{2.5}$$

where Φ is the normal probability function that gives the probability that the normalized random variable is below a given value. $\bar{Z}$ is the mean safety margin and σ_Z is the standard deviation of the safety margin. Thus, Equation 5 gives the probability that Z is less than 0 (or R less than S). The reliability index, β, is defined such that:

$$P_f = \Phi(-\beta) \tag{2.6}$$

For example, if the reliability index β=3.5, then the implied probability of failure is obtained from the Normal Distribution tables given in most books on statistics as $P_f=2.326\text{x}10^{-4}$. If β=2.5 then $P_f=6.21\text{x}10^{-3}$. A β=2.0 implies that $P_f=2.23\text{x}10^{-2}$. One should note that these P_f values are only notional measures of risk that are used to compare different structural design and load capacity evaluation methodologies but are not actuarial values.

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

$$\beta = \frac{\bar{Z}}{\sigma_Z} = \frac{\bar{R}-\bar{S}}{\sqrt{\sigma_R^2+\sigma_S^2}} \tag{2.7}$$

Thus, the reliability index, β, which is often used as a measure of structural safety, gives in this instance the number of standard deviations that the mean margin of safety falls on the safe side as represented in Figure 2.1.

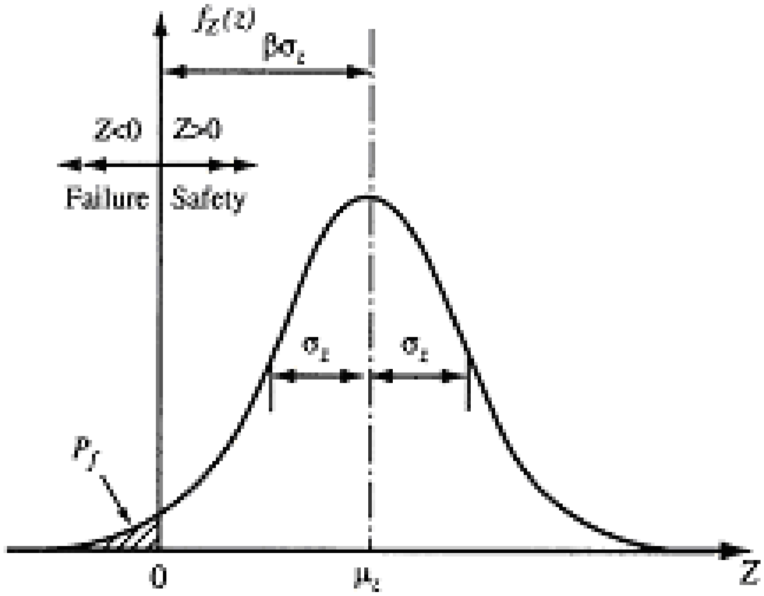


Figure 2.1. Graphical Representation of Reliability Index

The reliability index, β, defined in Equations 6 and 7 provides an exact evaluation of risk (failure probability) if R and S follow normal distributions. Although β was originally developed for normal distributions, similar calculations can be made if R and S are Lognormally distributed (i.e. when the logarithms of the basic variables follow normal distributions). In this case, the reliability index can be calculated as:

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\frac{\sqrt{1+V_S^2}}{\sqrt{1+V_R^2}}\right)}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_S^2\right)\right]}} \tag{2.8}$$

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

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} \tag{2.9}$$

"Level II" methods have also been developed to obtain the reliability index for the cases when the basic variables are neither normal nor lognormal. Level II methods, often referred to as FORM (First Order Reliability Methods) or FOSM (First Order Second

Moment) involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure equation (i.e. when Z=0) by a tangent multi-dimensional plane at the point on the failure surface closest to the mean value. For example, during the calibration of the AASHTO LRFD code, Nowak (1999) used the FORM algorithm developed by Rackwitz and Fiessler (1978) to calculate the reliability index values when R is assumed to follow a lognormal distribution and S is a normal random variable. More advanced techniques including SORM (Second Order Methods) have also been developed.

On the other hand, Monte Carlo simulations can be used to provide estimates of the probability of failure. Monte Carlo simulations are suitable for any random variable distribution type and failure equation. In essence, a Monte Carlo simulation creates a large number of "experiments" through the random generation of sets of resistance and load variables. Estimates of the probability of failure are obtained by comparing the number of experiments that produce failure to the total number of generated experiments. Given values of the probability of failure, P_f, the reliability index, β, is calculated from Equation 6 and used as a measure of structural safety even for non-normal distributions. Kulicki et al (2007) used the Monte Carlo simulation while reviewing the code calibration effort reported by Nowak (1999) and verified that the results of the FORM method with the Rackwitz-Fiessler algorithm and those of the Monte Carlo simulation are essentially similar. More detailed explanations of the principles discussed in this section can be found in published texts on structural reliability (e.g. Thoft-Christensen & Baker; 1982, Nowak & Collins; 2000, Melchers, 1999).

The reliability index has been used by many code writing groups throughout the world to express structural risk. Reliability index values, β, in the range of 2 to 4 are usually specified for different structural applications. For example, the calibration of the Strength I limit state in AASHTO LRFD Specifications aimed to achieve a uniform target reliability index β_{target}=3.5 for a range of typical bridge span lengths, beam spacing and materials (Nowak, 1999). A reliability index β_{target}=2.5 was used by Moses (2001) for the calibration of the Operating Rating in the AASHTO LRFR. These values usually correspond to the failure of a single component. If there is adequate redundancy, overall system reliability indices will be higher.

Reliability-based Code Calibration Approach

The reliability index β is seldom used in practice for making decisions regarding the safety of a new bridge design or an existing structure but it is rather used by code writing groups for recommending appropriate load and resistance safety factors for new structural design or evaluation specifications. One commonly used calibration approach is based on the principle that each type of structure should have uniform or consistent reliability levels over the full range of applications. For example, load and resistance factors should be chosen to produce similar β values for steel and concrete bridges of different span lengths, number of lanes, number of beams and beam spacing, simple or continuous spans, and roadway categories. Thus, a single target β must be achieved for all applications. Some engineers and researchers on the other hand are suggesting that higher values of β should be used for more important structures such as bridges with longer spans, bridges that carry more traffic, or bridges that, according to AASHTO, are classified as critical for “social/survival or security/defense requirements”. Since higher β levels would require higher construction costs, the justification should be based on a cost-benefit analysis whereby target β values are chosen to provide a balance between cost and risk (Aktas, Moses and Ghosn, 2001). However, there is currently no consensus on how this can be achieved. Therefore, traditional and recent code calibration efforts have been based on the principle of maintaining uniform reliability levels.

In most code calibration efforts, appropriate target β values are deduced based on the performance of a sample population of satisfactorily performing existing designs. That is, if the safety performance of bridges designed according to current standards has generally been found satisfactory, then the average reliability index obtained from current designs is used as the target that the new code should satisfy. The aim of the calibration procedure is to minimize designs that deviate from the target reliability index.

The calibration based on past performance have been found to be robust in the sense that they minimize the effects of any inadequacies in the database as reported by Ghosn & Moses (1985). Ghosn and Moses (1985) found that the load and resistance factors obtained following a calibration based on "safe existing designs" are relatively insensitive to errors in the statistical data base as long as the same statistical data and criteria used to find the target reliability index are also used to calculate the load and resistance factors for the new code. In fact, a change in the load and resistance statistical properties (e.g. in the coefficients of variation) would affect the computed β values for all the bridges in the selected sample population of existing bridges and consequently their average β value. Assuming that the performance history of these bridges is satisfactory, then the target reliability index would be changed to the new "average" and the calibrated load and resistance factors that would be used for new designs would remain approximately the same.

The calibration of resistance and live load factors for a new bridge code is usually executed by code writing groups as follows:

- A representative sample of bridges that have been designed to efficiently satisfy existing codes and that have shown good safety record is assembled.
- Reliability indices are calculated for each bridge of the representative sample. The calculation is based on statistical information about the randomness of the strength of members, the statistics of load intensities, and their effects on the structures.
- In general, there will be considerable scatter in such computed reliability indices. A target β is selected to correspond to the average reliability index of the representative bridge sample.

- For the development of the new code, load and resistance factors as well as nominal loads are selected by trial and error to satisfy the target β as closely as possible for the whole range of applications.

Resistance Modeling

To execute the calculation of the reliability index, one needs to obtain the statistical data for all the random variables that affect the safety margin Z of Equation 3 including all the uncertainties in estimating the variables that describe the member resistances and the load effects. Experimental and simulation studies have developed statistical estimates of member resistances for different types of bridge structural members. These models have accounted for the variability and uncertainties in estimating the material properties; modeling errors; differences between predicted member capacities and measured capacities; human error and construction control. For example, Nowak (1999) followed the approach of Ellingwood et al. (1980) and represented a bridge member resistance capacity by a variable R that is the product of several variables, such that:

$$R= M\ F\ P\ R_n \qquad (2.10)$$

Where M= material factor representing the variability in properties such as strength, modulus of elasticity, capacity to resist cracking, and chemical composition; F= fabrication factor representing the variability in geometric properties including dimensions, moments of inertia, and section modulus; P= analysis factor representing the variations of the actual strength when compared to the approximate code specified models for estimating member capacity; R_n= predicted member capacity using code specified methods. Equation (10) can be used to find the mean of R using Equation (2) if the total resistance bias, b_r, is set to be equal to the product of the mean values of M F and P.

Note that the resistance model of Equation (10) does not account for member deterioration or other changes with time. Thus, all the resistance variables are usually assumed to be time-independent random variables. Research studies on modeling member deterioration have been ongoing for a number of years. However, these have not been implemented in code calibration practice pending more studies to verify the validity of these models when compared to actual behavior of bridge members over time.

Live Load Modeling

For a bridge member (or structural system) to be safe, the resistance should be large enough to withstand the maximum load effect that could occur within the structure's service life. Estimating the effects of the maximum loads involves a number of random variables, which are often associated with large levels of modeling uncertainties. The permanent loads of a bridge are constant over time and studies have been conducted to compare field verified as-built permanent loads to the nominal dead loads estimated during the bridge design or load capacity evaluation processes. Accordingly, the dead load is usually represented as a random variable that follows a normal distribution having a mean value and a standard deviation that can be expressed using a bias and COV that can be expressed in a similar manner as that used for the resistance as shown in Eq. (2.1) and (2.2).

On the other hand, the intensities of the maximum live loads are time-dependent random variables in the sense that longer service lives imply higher chances that the structure will be subjected to a given extreme load level. However, it is generally not possible to collect live load for extended periods of time corresponding to the service life of the structure. Therefore, statistical projections of data collected in the past over a limited period of time must be performed. This assumes that past data will also be valid in the future and that data collected at some representative locations are valid for the location of the bridge to be analyzed. It should be noted, that the projection of limited load intensity data, collected from previous measurements over short periods of time, to future return periods is associated with various levels of statistical modeling uncertainties. In addition, modeling the structure's response to the applied loads and estimating the variables that control the effects of the loads on the structure are also associated with high levels of structural modeling uncertainty. These load projection and structural modeling uncertainties are independent of the return period.

To find the probability distribution for the maximum loading event in a period of time t we have to start by assuming that N loading events occur during this period of time t. These events are designated as S_1, S_2, … S_N. The maximum of these N events, call it $S_{max,N}$, is defined as:

$$S_{max,N} = \max (S_1, S_2, \ldots S_N) \tag{2.11}$$

We are interested in finding the probability distribution of the maximum live load event that will control whether the structure will be safe or unsafe. The probability distribution of the load can be represented by the cumulative probability distribution of $S_{max,N}$. This cumulative probability distribution, $F_{s\ max\ N}(S)$, gives the probability that $S_{max,N}$ is less than or equal to a value S. If the maximum of N events, i.e. $S_{max,N}$ is less than S, this implies that each one of these N events is less than S. Therefore, S_1 is less than S, S_2 is less than S, … and S_N is less than S. Hence, assuming that the loading events are independent, using the basics concepts of the theory of probability, the probability that $S_{max,N} \leq S$ can be calculated from:

$$F_{s_{\max N}}\left(S\right) = F_{s_1}(S).F_{s_2}(S)...F_{s_N}(S) \tag{2.12}$$

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

If S_1, S_2 ... S_N are independent random loading events that are drawn from the same probability distribution, then:

$$F_{s_1}(S) = F_{s_2}(S) = ... = F_{s_N}(S) = F_s(S) \quad (2.13)$$

Therefore, Equation (2.13) reduces to

$$F_{s_{maxN}}(S) = \left[F_s\ (S)\right]^N \quad (2.14)$$

The cumulative distributions of the load effects, F(S), can be assembled by sending the truck weight and axle configuration data assembled at a WIM site through appropriate influence lines and the data from all the trucks assembled into cumulative distribution histograms. This could be done for individual trucks and for multi-truck loading events whether these multi-truck events consist of a series of trucks in a single lane (trucks following each other) or trucks in multi-lanes (side-by-side or staggered).

The number of events expected in a return period can be obtained based on information from WIM data on the Average Daily Truck Traffic (ADTT) as well as WIM headway data. Generally speaking, WIM data has shown that Interstate highways can be subjected to up to 5000 trucks per day, more than 85% of which travel in the main driving lane and about 1% to 2% of which can be close to each other in two contiguous lanes to be considered side-by-side. Note that equation 14 assumes that the number of events, N, is a known deterministic value. The sensitivity analysis performed by Sivakumar, Ghosn & Moses (2008) has however demonstrated that the results of equation 2.14 are not highly sensitive to variations in N as the number of events, N, becomes large.

The probability distribution of the maximum live load intensity using Eq. (2.14) can be used to find the mean and the standard deviation of the maximum intensity, Smax, expected in a return period.

Besides the uncertainties associated with estimating the maximum laod effect, Smax, obtained from Eq. (14) given the histogram for the WIM data, F(S), collected at a particular WIM site, the total load effect must also account for several types of modeling uncertainties. One type of modeling uncertainties is related to the variability in the data collection process and the sufficiency of the quantity of WIM data in adequately describing the true distribution of the load F(S). This modeling uncertainty will be referred to as data variability. Another modeling uncertainty is related to how well does the WIM data collected at one site represent the load at the bridge site. This type of modeling uncertainty is referred to as site-to-site variability. In addition, the structural analysis process involves a level of uncertainty that should be included when assessing the safety of bridges. Particularly important is the analysis of lateral distribution of the load in multi-girder bridges and the dynamic impact analysis.

2.3 Bridge Configurations, Resistance and Dead Load Model

Bridge Configurations

The reliability calibration of load rating specifications requires that the bridge load rating process leads to uniform reliability levels for the applicable bridge configurations. Hence, the calibration has to be performed on a sample set of bridges that are most representative of the bridges to which the specifications will apply. An analysis of the National Bridge Inventory (NBI) files for New York state bridges has shown that 85% of New York state bridges are single span steel, reinforced concrete or prestressed concrete with simple span steel bridges forming 59% of the total (Pan, 2007). Furthermore, multi-girder bridges form 58% of the total New York state bridge inventory. Therefore, the calibration process that is undertaken in this study will focus on simple span multi-girder steel bridges. To verify that the results will also be applicable to other material types, a sensitivity analysis will be performed to verify the validity of the results for application to the rating of reinforced and prestressed concrete bridges. The focus of the analysis will also be on studying the moment effects while a sensitivity analysis will verify the compatibility of the results with those of shear loads. The bridges used for the purposes of this study have span lengths varying between 40-ft to 200-ft and beam spacings varying between 4-ft and 12-ft. These lengths and spacings were selected because they correspond to the range of applicability of the AASHTO LRFD as calibrated by Nowak (1999). The selected bridges are assumed to have the same dead loads as those used by Nowak (1999) during the calibration of the AASHTO LRFD. Table 1 gives an example of the steel bridge configurations used during the calibration process.

In addition to the basic bridge configuration database, the reliability analysis requires as input information on the statistics of all the random variables that are used in the safety assessment of a bridge member. Specifically, for each bridge configuration, the probability distributions as well as the means and coefficients of variation of the member strength, dead loads and live load are required. In order to be consistent with the current bridge design process, the same database for member resistance and dead load statistics used by Nowak (1999) during the calibration of the AASHTO LRFD is used in this study to represent the resistance and the dead load on typical bridge configurations.

On the other hand, recent observations made on truck weight data collected from Weigh-In-Motion stations (WIM) at representative NY State sites (Sivakumar et al, 2008) have shown that trucks travelling over the New York State's highway system can be significantly heavier than the generic truck weight data used during the calibration of the AASHTO LRFD specifications. In fact, the generic live load models used during the calibration of the LRFD were based on data collected in the mid and late 1970's in Canada and may not represent current US or NY state loads (Nowak, 1999). In this study, the truck weight, configuration and headway data collected in each direction of five different New York WIM stations will be used to model the live load effects on typical bridge as will be described further below.

Dead Load Model

Following Nowak (1999)'s approach, the total dead load, DL is divided into the dead load of pre-fabricated members, D_{C1}, the dead load of cast-in-place members, D_{C2}, and the dead load of the wearing surface, D_w, such that the mean total dead load is given by:

$$\overline{DL} = \overline{D_{c1}} + \overline{D_{c2}} + \overline{D_w} \,, \tag{2.15}$$

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

$$\sigma_{DL} = \sqrt{\sigma_{DC1}^2 + \sigma_{DC2}^2 + \sigma_{DW}^2} \tag{2.16}$$

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

$$V_{DL} = \frac{\sigma_{DL}}{\overline{DL}} \tag{2.17}$$

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

$$\begin{aligned} \overline{D_{C1}} &= 1.03\ D_{C1} \qquad & V_{DC1} &= 8\% \\ \overline{D_{C2}} &= 1.05\ D_{C2} \qquad & V_{DC2} &= 10\% \\ \overline{D_W} &= 1.0\ D_W \qquad & V_{DW} &= 25\% \end{aligned} \tag{2.18}$$

Where D_{c1}, D_{c2} and D_w are respectively the nominal values of the dead load of pre-fabricated members, cast-in-place members, and wearing surface. Table 2.1 provides typical nominal values for the moment effect of each dead load component for a typical set of simple span composite multi-girder steel bridges. These data are obtained from Nowak (1999) for the 60-ft, 120-ft and 200-ft spans. For the 40-ft and 100-ft spans, the moment effects of the dead weights are obtained from estimates of the dead weights per unit length. These estimates are obtained by interpolation from the weight per unit length values obtained from the report by Nowak (1999).

Table 2.1. Nominal dead weight moment effects for typical composite steel girders.

Span (ft)	Spacing (ft)	D_{C1} (kip-ft)	D_{C2} kip-ft)	D_W kip-ft)
40 – ft	4	14	109	21
	6	15	149	32
	8	20	185	43
	10	23	231	54
	12	27	284	64
60 –ft	4	39	245	49
	6	48	335	73
	8	70	414	97
	10	84	521	122
	12	103	639	146
100 –ft	4	329	681	135
	6	361	931	203
	8	386	1150	270
	10	407	1447	337
	12	458	1775	405
120 – ft	4	502	981	194
	6	607	1341	292
	8	650	1656	389
	10	681	2083	486
	12	773	2556	583
200 – ft	4	2780	2725	540
	6	3303	3725	810
	8	3790	4600	1080
	10	4190	5788	1350
	12	4875	7100	1620

Resistance Model

The nominal resistance of an existing bridge member depends on the dead load and the rating of the member. Specifically, the relationship between the Rating Factor, R.F., and the nominal member resistance, R_n, can be expressed by:

$$R.F.=\frac{\phi R_n-\gamma_{DC}\left(D_{C1}+D_{c2}\right)-\gamma_{DW}D_W}{\gamma_L L_n} \qquad (2.19)$$

where R_n is the nominal resistance, D_{C1} is the dead load effect of pre-fabricated components, D_{C2} is the dead load effect of cast-in-place components and attachments, D_W is the nominal dead load effect for the wearing surface, L_n is the live load effect of the nominal load used to calculate the Rating Factor including dynamic allowance and load distribution factor, ϕ is the resistance factor, and γ are the load factors.

The resistance factor, ϕ and load factors γ depend on the specifications used and the type of load effect being considered. For example, according to the AASHTO LRFR specifications ϕ =1.0 for the bending moment capacities of steel and prestressed concrete members, γ_{DW}=1.50, γ_{DC}=1.25. The Inventory Rating live load factor is given as γ_L=1.75 and γ_L=1.35 for the Operating Rating using the HL-93 live load model. The AASHTO LRFR Operating Rating for the AASHTO legal loads is given as γ_L=1.80 for ADTT $\geq$ 5000, γ_L=1.65 for 1000<ADTT <5000, and γ_L=1.40 for ADTT $\leq$ 100. In the AASHTO LRFR, the dynamic allowance factor is 1.33 times the static truck moment effect and the load distribution factor is calculated as a function of span length and beam spacing for different numbers of loaded lanes as provided in the AASHTO LRFD load distribution tables. The format of Eq. (2.19) is applicable for traditional load rating methods although different nominal loads, dynamic impact and load distribution factors as well as resistance and load factors are used for AASHTO Load Factor Ratings (LFR) or Allowable Stress Ratings (ASR).

Equation (2.19) can be used to find the nominal resistance of a bridge member for different values of the rating factor. Nowak (1999) assumed that the member resistances can be modeled by lognormal probability distributions where the mean and COV of the moment resistance of bridge girders are related to the nominal values by:

$$\overline{R}=1.12\ R_n \qquad V_R=10\% \qquad \text{For steel beams}$$

$$\overline{R}=1.05\ R_n \qquad V_R=7.5\% \qquad \text{For prestressed concrete beams} \qquad (2.20)$$

$$\overline{R}=1.14\ R_n \qquad V_R=13\% \qquad \text{For reinforced concrete beams}$$

For the shear resistance, the mean and COV are given by Nowak (1999) as:

$$\overline{R}=1.14\ R_n \qquad V_R=10.5\% \qquad \text{For steel beams}$$

$$\overline{R} = 1.15\ R_n \qquad V_R = 14\% \qquad \text{For prestressed concrete beams} \qquad (2.21)$$
$$\overline{R} = 1.20\ R_n \qquad V_R = 15.5\% \qquad \text{For concrete beams with steel}$$
$$\overline{R} = 1.40\ R_n \qquad V_R = 17\% \qquad \text{For concrete beams without steel}$$

2.4 Analysis of New York WIM Data

The object of this study is to develop a set of NYSDOT-LRFR specifications that will produce acceptable levels of reliability for New York bridges under current loading conditions. To achieve this goal, it is critical to use recent New York statistical information on truck weights, truck configurations, and multiple presence data. For the purposes of this study, the New York Weigh-In-Motion (WIM) data collected by Sivakumar et al (2008) as part of NCHRP project 12-76 is analyzed to obtain projections for the maximum load effect expected on New York bridge members.

New York State Weigh-In-Motion Data

Figure 2.2 shows the NYSDOT Weigh-In-Motion sites. Five representative sites highlighted by arrows in Figure 2.2 have been selected for the purposes of this study as listed in Table 2.2. The sites selected include two rural interstate principal arterial highways, two other principal arterials and one urban interstate principal arterial WIM data on the truck weights was assembled for each direction of traffic, thus providing a total 10 sets of data.

As shown in Table 2.3, WIM data was collected in 2005 in each lane of each site over a period close to one year except for Lane 4 of the I-95 site for which no data was available. The statistical data on the heaviest 20% of the trucks for each site and each direction are provided in Table 2.4. The data shows differences between the average truck weights from site to site and also for the two traffic directions within a site. The average weight for the 20% heaviest trucks is found to be 91 kips with a standard deviation of 15 kips. It is noted that the 91 kip average value is significantly higher than the 68 kips used by Moses (2001) during the calibration of the AASHTO LRFR although the 15 kip standard deviation is lower than the 18 kips that was used by Moses (2001).

Table 2.2. New York WIM Sites for LRFR Calibration

No	State	WIM Site ID	Route	FHWA Functional Class	Description
1	New York	9121	I-81	1	Rural Interstate Principal Arterial
2	New York	2680	Route 12	12	Other Principal Arterial
3	New York	8280	I-84	1	Rural Interstate Principal Arterial
4	New York	9631	Route 17	12	Other Principal Arterial
5	New York	0199	I-95	11	Urban Interstate Principal Arterial

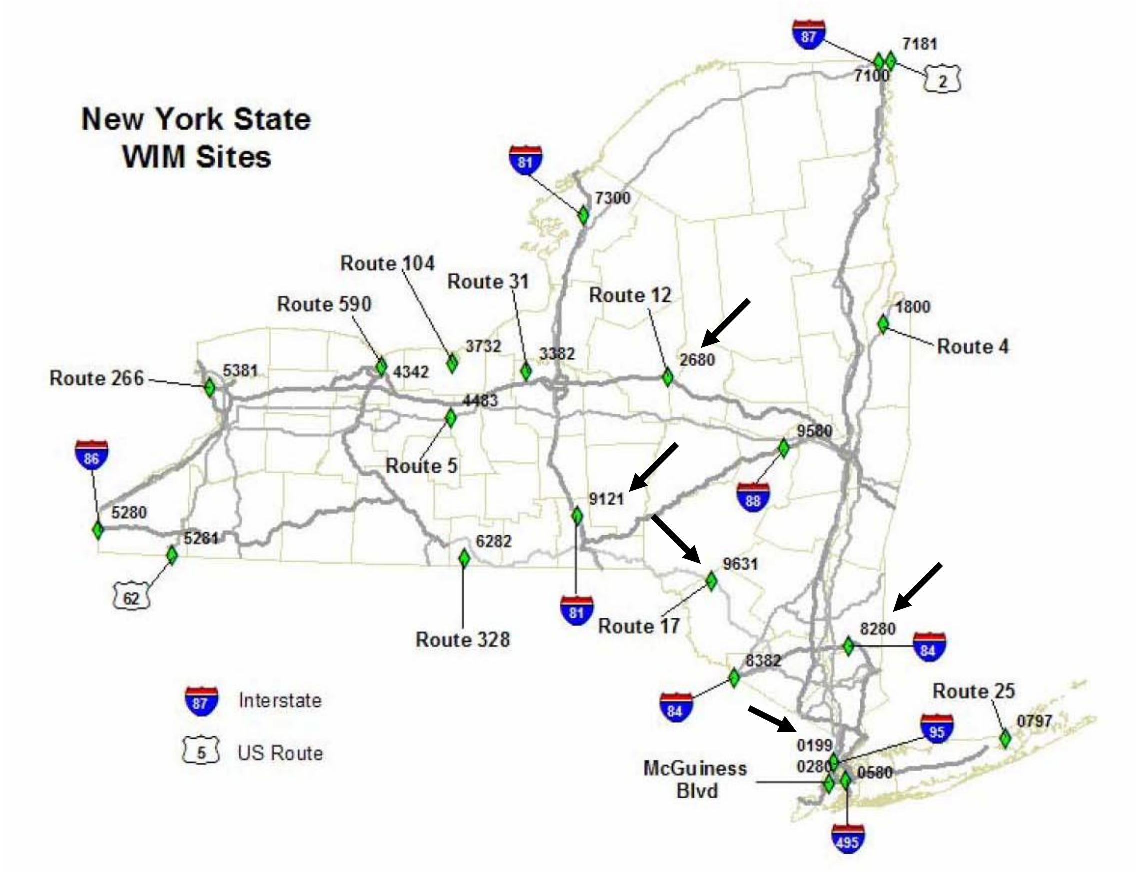


Figure 2.2. Location of Weigh –In-Motion sites in New York State

Table 2.3. Days WIM Data was Recorded in 2005 at NY WIM sites

Days Recorded per Lane at each NY WIM Site				
WIM Site (Route)	**Lane 1**	**Lane 2**	**Lane 3**	**Lane 4**
0199 (I-95)	227	227	227	0
2680 (Rt. 12)	274	252	267	276
8280 (I-84)	273	273	273	265
9121 (I-81)	329	319	305	286
9631 (Rt. 17)	254	254	251	254

Table 2.4. WIM truck weight (kips) statistics for the heaviest 20% of the measured truck data.

		Lane 1 & 2	Lane 3 & 4
WIM Site 9631 (Rt 17) Rural	Truck count	110329	112661
	Mean Top 20% Trucks	89	107
	Std Dev Top 20% Trucks	12	16
	COV Top 20% Trucks	14%	15%
WIM Site 9121 (I-81) Principal Arterial	Truck count	531041	525732
	Mean Top 20% Trucks	90	87
	Std Dev Top 20% Trucks	19	10
	COV Top 20% Trucks	21%	12%
WIM Site 8280 (I-84) Principal Arterial	Truck Count	759214	732009
	Mean Top 20% Trucks	95	78
	Std Dev Top 20% Trucks	21	9
	COV Top 20% Trucks	22%	11%
WIM Site 2680 (Rt 12) Rural	Truck Count	29676	56464
	Mean Top 20% Trucks	91	99
	Std Dev Top 20% Trucks	15	17
	COV Top 20% Trucks	17%	17%
WIM Site 0199 (I-95 Bronx)Urban	Truck Count	1518000	347807
	Mean Top 20% Trucks	86	89
	Std Dev Top 20% Trucks	13	16
	COV Top 20% Trucks	16%	18%
Average value from New York WIM data	Mean Top 20% Trucks	91 kips	
	Std Dev Top 20% Trucks	15 kips	
Value used in AASHTO LRFR by Moses (2001)	Mean Top 20% Trucks	68 kips	
	Std Dev Top 20% Trucks	18 kips	

Multiple Presence Probability

The truck arrival data collected at the New York WIM sites were also analyzed by Sivakumar et al (2008). The daily truck traffic volume was classified into three categories: 1) Light with less than 1000 trucks per day; 2) Average with more than 1000 trucks but less than 2500 trucks per day; and 3) Heavy with more than 2500 trucks but less than 5000 trucks per day. Although a very heavy volume category was defined for cases where more than 5000 trucks crossed a WIM site on a given day, there were not enough cases of such situations to obtain any useful information.

When considering multiple trucks on a given span, a multiple presence event occurs if the gap between two trucks, that is the distance between the last axle of the leading truck and the first axle of the trailing truck, is less than the span length. Multiple presence probabilities were compiled for two trucks in adjacent lanes side-by-side, two trucks in adjacent lanes staggered, and two trucks in the same lane. Multiple presence probabilities for trucks in two adjacent lanes were compiled for headway separations up to 300 feet, in 20-foot increments as shown in Table 2.5.

The number of multiple presence events that occurred in a given day is recorded as a percentage of the total truck count for that day. The average multiple presence percentage is then calculated for days with light truck volume, average truck volume, heavy truck volume. Each direction of traffic was considered separately. The maximum multiple side-by-side presence cumulative percentages for each truck volume category are summarized in Table 2.5. Following the assumption made in NCHRP 12-76, it will be conservatively assumed that trucks in adjacent lanes within 60-ft head-to-head are actually side-by-side. Accordingly, for light volume sites, the percentage of side-by-side trucks is obtained as P_{sxs}=0.54%. This indicates that 0.54% of the trucks were found to be in adjacent lanes where the headways were 60-ft head to head. On average volume sites, 1.25% of the trucks were found to be in adjacent lanes within 60-ft headways. For heavy truck traffic sites 1.95% of the trucks were found to be in adjacent lanes within 60-ft head-to-head.

The probability of side-by-side events as calculated in this study are compared to the values used by Moses (2001) during the calibration of the AASHTO LRFR. Specifically, Moses (2001) used a side-by-side probability P_{sxs}=0.5% for light volume sites with ADTT=100, P_{sxs}=1% for average sites with ADTT=1,000 and P_{sxs}=6.67% for heavy traffic sites with ADTT=5000. It is observed that the values used by Moses (2001) are reasonably similar to the values obtained from the New York WIM data for the low volume and average volume sites. However, the 6.67% side-by-side probability used by Moses (2001) that was adopted from Nowak (1999) is larger than observed in New York. Furthermore, in order to match the load projection of Nowak (1999), Moses (2001) applied the side-by-side probabilities on only the heaviest 20% of the trucks while the data in Table 2.5 is for all the trucks. Thus, the Moses (2001) and Nowak (1999) assumptions add a significant level of conservatism to the maximum values of side-by-side percentages observed on New York sites. These comparisons are highlighted in

Table 2.6. The last row of Table 2.6 provides the rounded values that are proposed for use in this study.

Table 2.5. Upper envelope for percentage of side-by-side trucks on New York bridges

Maximum Side-by-Side Truck Multiple Presence Cumulative Probabilities				
	Site Truck Traffic			
Headway H (ft)	Light: ADTT < 1k	Average: 1k < ADTT < 2.5k	Heavy: 2.5k < ADTT < 5k	Very Heavy: ADTT > 5k
H ≤ 20	0.19	0.41	0.61	0.00
H ≤ 40	0.33	0.84	1.27	0.00
H ≤ 60	0.54	1.25	1.95	0.00
H ≤ 80	0.80	1.60	2.57	0.00
H ≤ 100	1.00	2.13	3.33	0.00
H ≤ 120	1.21	2.54	4.14	0.00
H ≤ 140	1.45	2.88	4.80	0.00
H ≤ 160	1.62	3.18	5.41	0.00
H ≤ 180	1.80	3.47	5.97	0.00
H ≤ 200	1.99	3.73	6.49	0.00
H ≤ 220	2.09	3.97	6.97	0.00
H ≤ 240	2.23	4.21	7.42	0.00
H ≤ 260	2.35	4.43	7.85	0.00
H ≤ 280	2.49	4.64	8.26	0.00
H ≤ 300	2.60	4.84	8.66	0.00

Table 2.6. Comparison of ASSHTO LRFR side-by-side data and maximum measured New York data.

Traffic Volume	Light	Average	Heavy
AASHTO LRFR classification	ADTT ≤ 100	ADTT=1000	ADTT ≥ 5000
AASHTO LRFR Percent side-by-side P_{sxs}	0.5% of heaviest 20% of trucks	1% of heaviest 20% of trucks	6.67% of heaviest 20% of trucks
NCHRP 12-76 classification	ADTT<1000	1000<ADTT<2500	2500<ADTT<5000
NCHRP 12-76 Percent side-by-side P_{sxs}	0.54% of all trucks	1.25% of all trucks	1.95% of all trucks
Proposed ADTT classification for calibration study	ADTT=100	ADTT=1000	ADTT=5000
Proposed for NYSDOT LRFR Load Simulation	0.5% of all trucks	1.25% of all trucks	2% of all trucks

The analysis of the reliability of bridges requires statistical information on the maximum live load effects expected to be witnessed by the bridge within its service period. For the design of new bridges, AASHTO LRFD assigns a design life of 75 years. For the evaluation of existing bridges that are inspected on a regular basis, a rating period of 5 years has been recommended by Moses (2001).

The evaluation of the safety of a single lane bridge requires the evaluation of the maximum load effect in the lane. For multi-lane bridges, the bridge must be able to sustain the maximum load effect expected when a single lane is loaded as well as the maximum load effect from multi-lanes. The next two sections describe the process used in this study to obtain the statistics for the maximum load for single lane and multi-lane loading.

Single lane loading

The analysis of the safety of existing bridges is based on ensuring that bridges maintain an acceptable level of reliability to sustain the maximum load effect expected within a pre-set return period. The return period for the load rating of an existing bridge is taken to be 5 years following the recommendation of Moses (2001) for the calibration of the AASHTO LRFR code. It should be clearly stated that it is not possible to obtain exact values for the maximum expected 5-year load due to the limitations in the available database. In fact, to obtain accurate results, one would need several cycles of WIM data collected over 5 years for each cycle which is not possible at this stage due to the relative recent adoption of WIM technology in the U.S. even if one assumes that the load spectra are stationary and do not change over time. Hence, some form of statistical projection will be needed for any practical load modeling effort as will be described in this section.

A single lane truck loading event is defined as the occurrence of a single truck or following trucks in a single lane of the bridge. NYSDOT WIM systems are capable of providing axle weights and axle spacings for each truck crossing the site of an installation and are capable of taking continuous uninterrupted data at normal highway traffic speeds with accurate time stamps which are able to identify single lane and multi-lane loading events involving several trucks. The New York WIM systems can thus provide the axle weights, axle spacings and relative positions of all the trucks involved in each single-lane or multilane loading event.

Lacking WIM data from single lane bridges, in this analysis we will assume that single lane bridges will be exposed to the same truck loads as those in the main driving lane of multi-lane bridges. The process of obtaining the maximum effect in a return period T for a single lane of truck traffic is based on the implementation of Eq. (2.11) through (2.14) and consists of the following steps:

- Given the time of arrival of trucks and their axle weight and axle spacing, obtain the load effect of each loading event by sending the sequence of trucks through the appropriate influence line.
- Assemble the single lane load effects into a histogram such as the one shown in Figure 2.3.
- Assemble the load effects into a cumulative distribution function, $F_s(S)$ such as the one plotted in black in Figure 2.4.

Ideally, the next step would require applying $F_s(S)$ into Equation 2.14 to obtain the cumulative distribution function for $F_{s_{\max N}}(S)$ which is the maximum effect in a given return period, T, where within the return period we will have N bridge loading events. For example, given an ADTT=2000 in one lane and a return period of 5 years, N=2000x365x5=3,650,000 events. Eq. 2.14 assumes the truck weight and other truck properties for each loading event are independent but they are sampled from the same probability distribution that does not change over time.

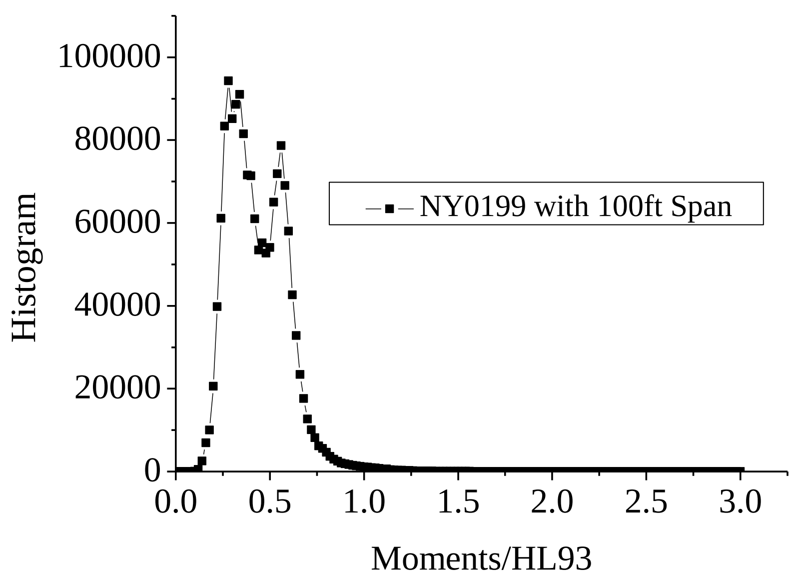


Figure 2.3 - Histogram of normalized moment effect for a single lane events

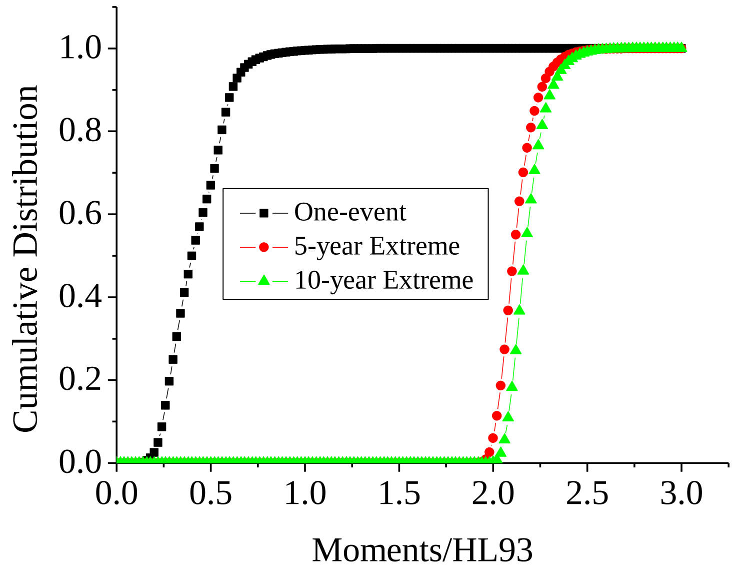


Figure 2.4 - Cumulative distribution for single lane for a single event, 5-year maximum and 10-year maximum load effects

In most instances, we do not have data for a whole period, T, the available WIM data in the tail end of $F_s(S)$ is usually not sufficient to provide accurate values of $F_{s_{\max N}}(S)$ when Eq. (2.14) is applied. Therefore, some sort of statistical projection is required to extend the range of the cumulative distribution $F_s(S)$. It would be possible to apply Eq. (2.14) if the probability distribution function of a single loading event, $F_s(S)$, is known. However, the probability distribution of one loading event as shown in Figure 2.3 does not follow any known probability distribution type. On the other hand, careful observations of the tail ends of the WIM data histograms for single lane events assembled from several New York and National sites have indicated that the tail end of the histograms of single loading events may approach the tail ends of Normal probability distributions. For example, Figure 2.5 shows the plot of the data collected at WIM site 0199 in upstate New York on a Normal probability scale. A Normal probability plot is executed by taking the standard deviate which is the Normal inverse of $F_s(S)$ represented by $\Phi^{-1}[F_s(S)]$ and plotting it versus the magnitude of the load effect, S. The plot would produce a straight line if S follows a Normal distribution. In this case, the mean of S would correspond to the abscissa for which $\Phi^{-1}[F_s(S)]$ is zero. The mean plus on standard deviation would correspond to the abscissa for which $\Phi^{-1}[F_s(S)]$ is equal to 1.0.

The plots of the WIM data in Figure 2.3 and Figure 2.5 show that the data as a whole (plotted in black) does not follow a Normal distribution as the curve in Figure 2.5 does not follow a straight line. However, Figure 2.5 shows that the upper 5% of the data does approach a straight line indicating that the tail end of the data resembles the tail end of a hypothetical Normal distribution. A linear fit (shown in red) of the Normal probability plot of the upper 5% of the data collected at this 0199 WIM site will produce a slope, m, and an intercept, n, which will give the mean of the equivalent Normal distribution that best fits the tail end as μ_{event}= -n/m. The standard deviation of the best-fit Normal is σ_{event}=1/m. As an example, the analysis of the WIM data of site 0199 for a 100-ft simply supported bridge shows that the upper 5% gives matches that of a normal distribution with an equivalent mean value μ_{event}= -0.18522, and a standard deviation σ_{event}=0.4363. Figure 2.6 shows the match of the tail between the original histogram and the normal histogram. Figure 2.7 zooms in on the tail end to better show the match.

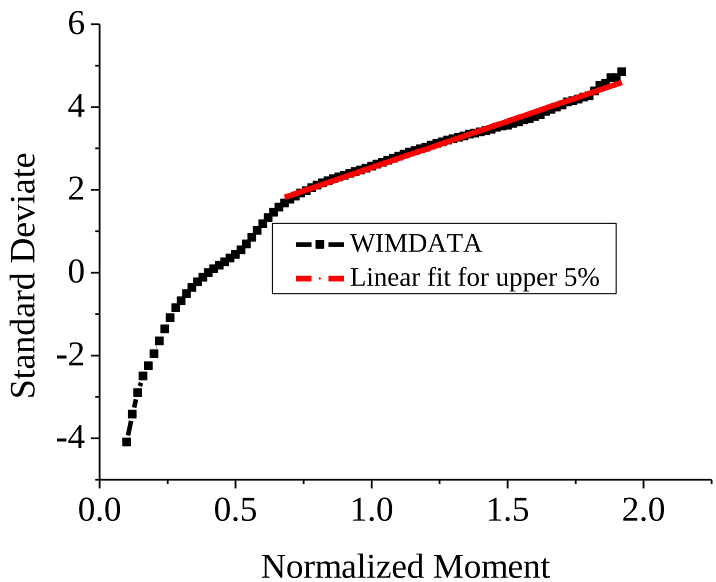


Figure 2.5 - Normal Probability Plot for Moment Effect of Single Lane Events

The application of Eq. 2.14 can be executed numerically for any parent probability distribution including a hybrid distribution where the lower 95% of the data is taken from the actual histogram of $F_s(S)$ and the upper 5% taken from the Normal probability distribution that fits the tail end. However, the fact that the tail end of the WIM data matches that of a Normal distribution allows for the application of extreme value theory to obtain the statistics of the maximum load effect in closed form. The approach is based on the following known concept as provided in Ang and Tang (2007): "If the parent distribution of the initial variable, S, has a general Normal distribution with mean μ_{event} and standard deviation σ_{event}, then the maximum value after N repetitions approaches asymptotically an Extreme Value Type I (Gumbel) distribution" with an inverse measure of dispersion α_N given by:

$$\alpha_N = \frac{\sqrt{2\ln(N)}}{\sigma_{event}} \tag{2.22}$$

and a most probable value u_N given by:

$$u_N = \mu_{event} + \sigma_{event}\left(\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}}\right) \tag{2.23}$$

α_N and u_N can be used to find the mean of the maximum load effect, L_{max}, its standard deviation, σ_{Lmax}, and its Coefficient of Variation (COV) V_{Lmax} for any return period having N repetitions as:

$$\bar{L}_{\max} = \mu_{\max} = u_N + \frac{0.577216}{\alpha_N} \tag{2.24}$$

$$\sigma_{L\max} = \frac{\pi}{\sqrt{6}\alpha_N} \tag{2.25}$$

$$V_{L\max} = \frac{\sigma_{L\max}}{\bar{L}_{\max}} \tag{2.26}$$

The most probable value u_N and the inverse dispersion coefficient α_N, can also be used to describe the probability distribution function, $f_{smax}(S)$ and the cumulative distribution function, $F_{smax}(S)$, of the maximum load effect S_{max} by:

$$f_{s\max}(S) = \alpha_N e^{-\alpha_N(S-u_N)} e^{-e^{-\alpha_N(S-u_N)}} \tag{2.27}$$

$$F_{s\max}(S) = e^{-e^{-\alpha_N(S-u_N)}} \tag{2.28}$$

As an example, to find the cumulative distribution of the maximum effect in 5 years, we take the cumulative distribution of one event obtained from the WIM data as shown in Figure 2.4 and then adjust its tail end by using the cumulative Normal distribution with mean μ_{event}= -0.18522, and a standard deviation σ_{event}=0.4363 for the cases when Fs(S) is greater than 95%. The new extended composite distribution of $F_s(S)$ is raised to the N's power. For ADTT=5000, in a 5-year return period, the number of truck loading events becomes N=5year*365days/year*5000 trucks/day =9125000. The distribution of $F_{smax}(S)$ is obtained as shown in Figure 2.8. The mean for L_{max} is found to be 2.13 and the standard deviation is 0.010.

Since the tail end of the WIM data matches that of a Normal distribution, this allows for the application of extreme value theory to obtain the statistics of the maximum load effect in closed form. By applying Eq. (2.25) and (2.26) the extreme value distribution of the maximum five-year load effect is found to have a mean equal to 2.125, and a standard deviation σ_{Lmax}= 0.0988. Figure 2.8 shows a comparison between the results of the projection using Eq. (2.14) and those obtained by plotting the Extreme distribution of Eq. (2.27). The implementation of Eq. (2.28) for the moment effects on a 100-ft span for the data of WIM site 0199 for 5-year and 10-year projections are also plotted in Figure 2.4.

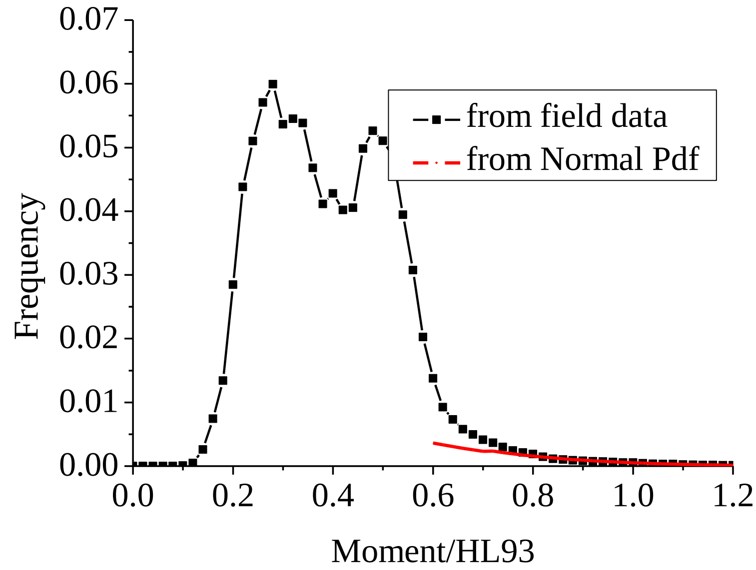


Figure 2.6 Comparison of WIM data and Normal distribution that best match the tail end of the Normalized Moment Histogram for Site NY0199

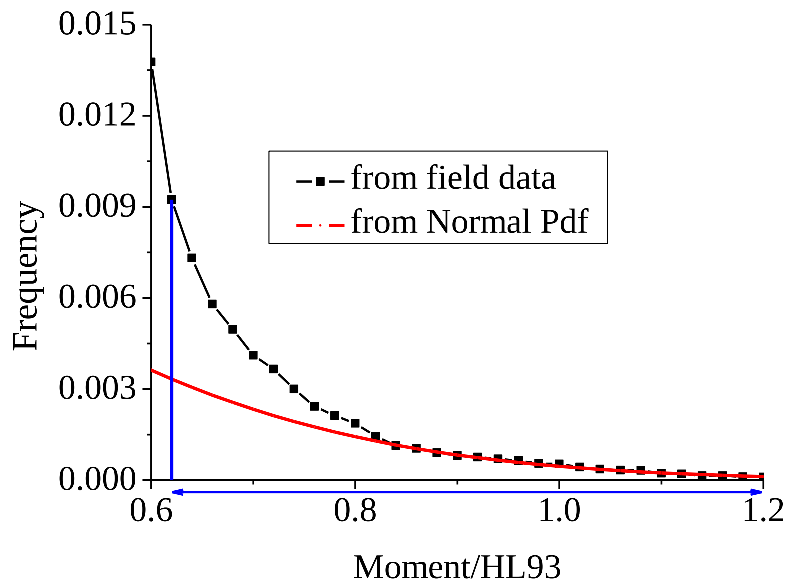


Figure 2.7 Zoom on the tail end of the WIM data and the corresponding Normal probability distribution.

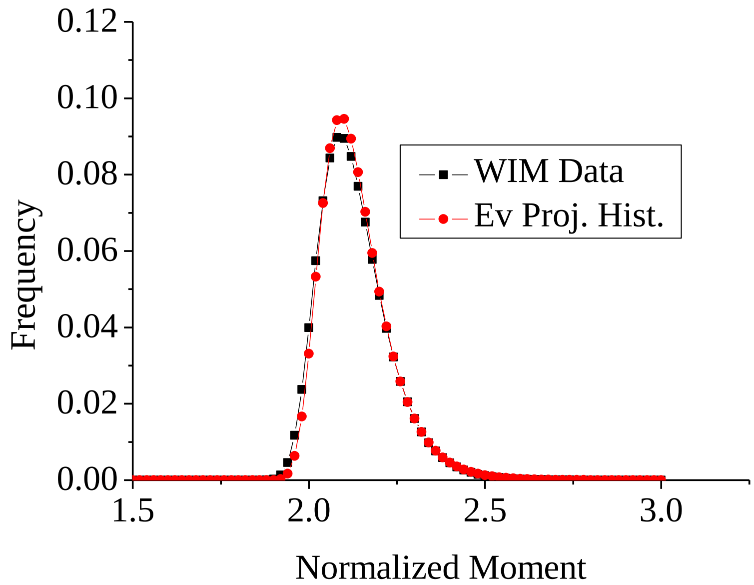


Figure 2.8 Comparison of Histograms for 5-year Maximum for Simulated WIM data and Extreme Value Model

Two-lane loading

The maximum load on a multi-lane bridge could occur due to a single lane is loaded or when several lanes are loaded. Due to the lack of data and the low probability of having several trucks simultaneously in three or more lanes, this study following the approach taken by Nowak (1999) and Moses (2001) will focus on two-lane loads. Although the New York WIM data provides sufficient information to obtain the location of multi-lane truck events, it is herein decided to use a convolution approach to obtain the load effect when two adjacent bridge lanes are simultaneously loaded by heavy trucks. The convolution approach will provide the flexibility of addressing multi-lane loading events for low volume sites with low ADTT as well as high volume and regular sites and will be consistent with the approach that will be followed when studying the load effects when permit trucks could be simultaneously on a bridge with random truck traffic. In this process, it is conservatively assumed that the truck weights and statistics on the number of following trucks in each of the lanes belong to the same truck weight and headway populations.

In a first step, using the WIM data files, the shear force or bending moment effect of each truck loading event in the WIM record is calculated for a given bridge span length by passing the sequence of trucks through the proper influence line. The shear or moment for each truck load event is then normalized by dividing the calculated value by the shear or moment of the HL-93 load model. The shear and moment data for the single lane loading are collected into separate percent frequency histograms. Each histogram provides a discretized form of the probability density function PDF) of the shear or moment effects for the site. The histogram is designated as $H_x(X)$ while the PDF is designated as $f_x(X)$. The relation between $H_x(X)$ and $f_x(X)$ is given by:

$$H_x(X) = \int_{X_l}^{X_u} f_x(x)dx \qquad (2.29)$$

where X_l and X_u give the upper and lower bounds of the bin within which X lies. If the bin size is small, then $f_x(X)$ can be assumed to be constant within the range of X_l to X_u. For example, Figure 2.9 shows in red the moment load effect histogram for a single lane obtained from the WIM data collected at site 9121 for a 100-ft simple span. The bin size in this figure and the data assembled in this study is $\Delta X=X_u-X_l=0.02$.

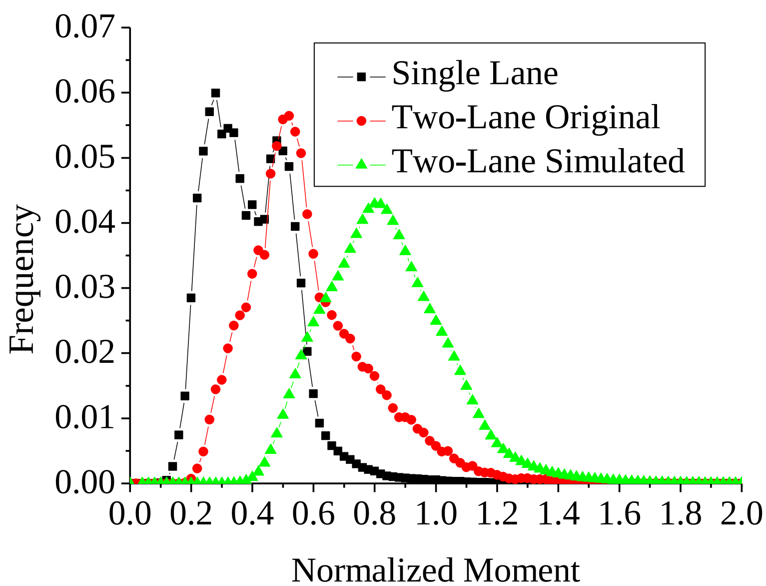


Figure 2.9. Normalized 100-ft moment histogram for trucks in a single lane and both lanes of WIM site 9121.

The total moment effect when two lanes are loaded is obtained from the single lane load effect as $S=x_1+x_2$ where x_1 is the effect of the trucks in the drive lane and x_2 is the effect of the trucks in the passing lane. Assuming independence between truck moments, the probability density function of the effect of side-by-side trucks $f_s(S)$ can be calculated using a convolution approach. The convolution equation is presented as:

$$f_s(S) = \int_{-\infty}^{+\infty} f_{x2}(S - x_1) f_{x1}(x_1) dx_1 \qquad (2.30)$$

where $f_s(S)$ is the probability distribution of the multi-lane effects, $f_{x1}(\ldots)$ is the probability distribution of the effects of trucks in lane 1, $f_{x2}(\ldots)$ is the probability distribution of the effects of trucks in lane 2. In these calculations we assume that $f_{x2}(\ldots)$ and $f_{x1}(\ldots)$ are actually the same distribution function.

Equation 30 can be interpreted as follows: Given that the effect of the truck in lane 1 is equal to x_1, then the probability that the effects of two side-by-side trucks will take a value S, is equal to the probability that the effect of the truck in lane 1 is x_1, times the probability that the effect of the truck in lane 2 is equal to X_2=S-x_1. This will lead to the following expression: $f_{x2}(S-x_1)f_{x1}(x_1)$. The integration is executed to cover all possible values of x_1. Equation 28 gives the probability density function (PDF) for one particular value of S. Thus, equation 28 must be repeated for each possible value of S. Equation 27 is then used to convert the PDF's into equivalent histograms.

Figure 2.9 shows the histogram obtained from the WIM data for the single lane events in red and the histogram for the two-lane loading events obtained from applying Eq. (2.28) in blue. The figure compares the latter to the two-lane loading events obtained directly from the WIM data show in black. The results confirm that the convolution approach yields more conservative values. This due to the assumption that trucks in two lanes that are within $\pm$60-ft head to head are compressed so that they are placed side-by-side, also some additional conservatism is due to the assumption that the percentage of trucks closely following each other is the same in both lanes.

To find the cumulative distribution of the maximum effect for multi-lane loading in 5 years, we take the cumulative distribution of one event, $F_s(S)$, and raise it to the N's power as shown in Eq. 14. For a site with ADTT=5000, the percentage of side-by-side loading events has been estimated as 2% as per Table 2.6. Accordingly, in a 5-year return period, N=5000$\times$2%$\times$365 days/year x 5 years=182,500. For ADTT = 1000, the number of events becomes N=1000$\times$1.25%$\times$365x 5=22813 and for ADTT =100, N=913.

Since the distribution of multi-lane loading events have been obtained from the convolution and because of the relative small values of N for the number of multi-lane events in the 5-year return period, it would be possible to find $F_{s_{\max N}}(S)$ directly from Equation (2.14). Alternatively, the same approach followed for a single lane event which consists of fitting the tail end of the data into an equivalent normal distribution can also be used to obtain the mean and COV of the Gumbel distribution that describes the maximum load effect in a 5-year return period. Figure 2.10 shows a comparison between the results of the projection using Eq. (2.14) and those obtained by plotting the Gumbel distribution using the statistics obtained from Eq. (2.27). A good match is observed between the two procedures.

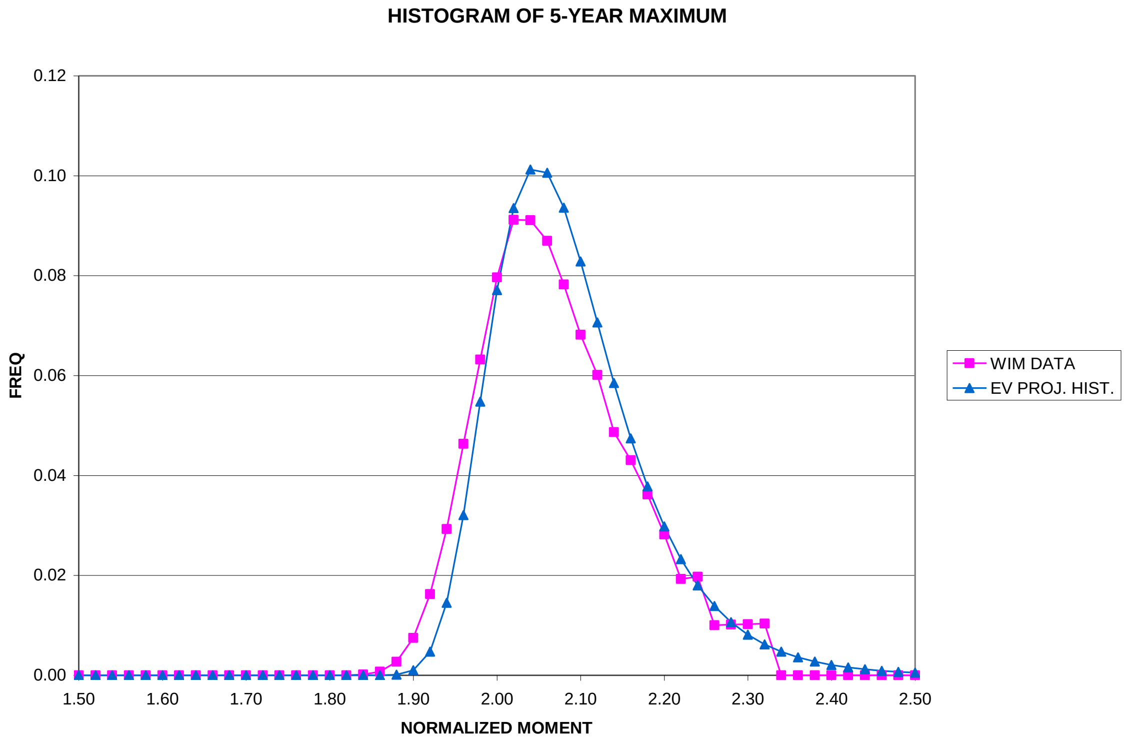


Figure 2.10. Comparison of Histogram for 5-year Maximum from Eq. (2.14) and Extreme Value Model

2.5 New York State Live Load Model

Total Static Load on a Bridge Span

The statistical analysis of the New York State WIM data from each direction of the five sites identified in Figure 2.2 and Table 2.3 has been performed to project for the maximum live load effects expected on simple span bridges for five-year and 10-year projection periods. The load effects studied are for the moment at midspan of the bridges and for the shear near the support.

As an example, Tables 7, 8 and 9 give the results for $\overline{L}_{\max}$ and V_{Lmax} obtained for two-lane loading by applying Eq. (2.24) and Eq. (2.26) for the moment effects on simple span beams for the data collected at all the New York State WIM sites studied and for each of the three ADTT categories. The Tables show large variability between the results of each site. This variability indicates that the use of data from a single site, no matter how extensive is the data collection effort, is not sufficient to model the loads for all the bridges in a given state or region. The statistical variability between the loads at different sites must be taken into consideration when developing specifications at the state or national levels. In this study, this site-to-site variability will be represented by comparing the mean values for each site and taking the COV of these mean values which will be represented by the parameter $V_{site-to-site}$. The final value of $\overline{L}_{\max}$ that will be used is the average of the values obtained from all the sites and the final $V_{L\max}$ is the average value from all the sites. Table 2.7 through 9 show that V_{Lmax} may be on the order of 5% to 10% depending on the site and the ADTT, while $V_{site-tos-ite}$ is on the order of 10% to 12% for New York State sites.

Tables 2.10, 2.11, 2.12, 2.13 and 2.14 give summaries of the result for the data collected at all ten New York State WIM sites studied and for each of the three ADTT categories. The results for each ADTT case are presented in terms of the average of $\overline{L}_{\max}$, the average of V_{Lmax} and the variability of $\overline{L}_{\max}$ expressed in terms of $V_{site-to-site}$. Tables 10 and 11 give respectively the moments for two lanes for a 5-year return period and a 10-year return period. The results indicate that the difference between 5 years and 10 years is minimal causing an increase in the $\overline{L}_{\max}$ values by a range of 2.5% to 5%. Tables 2.12 and 2.13 give the results for the one-lane moment for the 5-year and 10-year return periods. The differences in the results vary between 2% to 3.5%. Table 2.14 shows the results for the projected maximum two-lane shear for the 5-year period. All the results are normalized by the corresponding effect of the HL-93 load.

Table 2.7. Results of Extreme Value Projections for Two lane loads for ADTT=5000

P_{sxs}=2%		Direction 1		Direction 2	
WIM site	Span	$\bar{L}_{max}$	V_{Lmax}	$\bar{L}_{max}$	V_{Lmax}
NY 0199	40-ft	2.89	0.05	2.99	0.05
	60-ft	2.67	0.06	2.71	0.05
	100-ft	2.49	0.05	2.55	0.05
	120-ft	2.41	0.05	2.48	0.05
	200-ft	2.11	0.05	2.21	0.05
NY 8280	40-ft	3.03	0.06	2.31	0.06
	60-ft	2.81	0.05	2.02	0.06
	100-ft	2.72	0.05	1.93	0.05
	120-ft	2.66	0.05	1.92	0.05
	200-ft	2.33	0.05	1.73	0.05
NY 2680	40-ft	2.70	0.04	2.97	0.04
	60-ft			2.68	0.04
	100-ft	2.31	0.04	2.58	0.05
	120-ft	2.23	0.04	2.51	0.04
	200-ft	1.96	0.04	2.20	0.04
NY 9121	40-ft	2.65	0.05	2.30	0.05
	60-ft	2.33	0.05	2.06	0.05
	100-ft	2.35	0.05	2.08	0.04
	120-ft	2.40	0.05	2.05	0.04
	200-ft	2.18	0.05	1.85	0.04
NY 9631	40-ft	2.50	0.05	2.83	0.05
	60-ft	2.19	0.05	2.60	0.05
	100-ft	2.26	0.05	2.60	0.05
	120-ft	2.25	0.05	2.63	0.05
	200-ft	1.98	0.05	2.36	0.05

Table 2.8. Results of Extreme Value Projections for Two lane loads for s for ADTT=1000

P_{sxs}=1.25%		Direction 1		Direction 2	
site	Span	$\overline{L}_{max}$	V_{Lmax}	span	$\overline{L}_{max}$
NY 0199	40-ft	2.62	0.07	2.06	0.07
	60-ft	2.41	0.07	1.83	0.07
	100-ft	2.26	0.06	1.75	0.06
	120-ft	2.19	0.06	1.74	0.06
	200-ft	1.93	0.06	1.57	0.06
NY 8280	40-ft	2.74	0.07	2.06	0.07
	60-ft	2.54	0.07	1.83	0.07
	100-ft	2.47	0.06	1.75	0.06
	120-ft	2.42	0.06	1.74	0.06
	200-ft	2.13	0.06	1.57	0.06
NY 2680	40-ft	2.50	0.05	2.74	0.05
	60-ft	2.25	0.05	2.48	0.05
	100-ft	2.14	0.05	2.39	0.05
	120-ft	2.07	0.05	2.32	0.05
	200-ft	1.82	0.05	2.03	0.05
NY 9121	40-ft	2.42	0.06	2.11	0.05
	60-ft	2.14	0.06	1.90	0.05
	100-ft	2.15	0.06	1.92	0.05
	120-ft	2.19	0.06	1.90	0.05
	200-ft	2.00	0.06	1.71	0.05
NY 9631	40-ft	2.26	0.06	2.59	0.06
	60-ft	2.00	0.06	2.37	0.06
	100-ft	2.06	0.06	2.38	0.06
	120-ft	2.06	0.06	2.40	0.06
	200-ft	1.82	0.06	2.17	0.06

Table 2.9. Results of Extreme Value Projections for Two lane loads for r ADTT=100

P_{sxs}=0.5%		Direction 1		Direction 2	
site	Span	$\bar{L}_{max}$	V_{Lmax}	span	$\bar{L}_{max}$
NY 0199	40-ft	2.12	0.10	2.22	0.09
	60-ft	1.95	0.10	2.00	0.10
	100-ft	1.85	0.09	1.91	0.09
	120-ft	1.81	0.09	1.87	0.09
	200-ft	1.60	0.09	1.67	0.09
NY 8280	40-ft	2.21	0.10	1.63	0.11
	60-ft	2.06	0.10	1.47	0.10
	100-ft	2.03	0.09	1.43	0.10
	120-ft	2.01	0.09	1.42	0.09
	200-ft	1.79	0.08	1.30	0.09
NY 2680	40-ft	2.13	0.07	2.32	0.08
	60-ft	1.92	0.07	2.11	0.07
	100-ft	1.83	0.07	2.03	0.07
	120-ft	1.78	0.07	1.98	0.07
	200-ft	1.57	0.07	1.74	0.07
NY 9121	40-ft	2.02	0.09	1.77	0.08
	60-ft	1.77	0.09	1.60	0.08
	100-ft	1.80	0.08	1.62	0.08
	120-ft	1.83	0.08	1.62	0.07
	200-ft	1.67	0.08	1.46	0.07
NY 9631	40-ft	1.84	0.10	2.18	0.08
	60-ft	1.65	0.09	1.97	0.09
	100-ft	1.71	0.09	1.99	0.08
	120-ft	1.70	0.09	2.00	0.08
	200-ft	1.52	0.08	1.83	0.08

Table 2.10. Summary of L_{max} for two-lane Moments for 5 years

	ADTT=5000 - P_{sxs}=2%			ADTT=1000- P_{sxs}=1.25%			ADTT=100- P_{sxs}=0.5%		
span	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$
40-ft	2.72	0.05	0.10	2.48	0.06	0.10	2.05	0.09	0.11
60-ft	2.45	0.05	0.12	2.23	0.06	0.12	1.85	0.09	0.12
100-ft	2.38	0.05	0.10	2.18	0.06	0.10	1.82	0.08	0.11
120-ft	2.35	0.05	0.10	2.15	0.06	0.10	1.80	0.08	0.10
200-ft	2.09	0.05	0.10	1.92	0.06	0.10	1.61	0.08	0.10
average		0.05	0.10		0.06	0.10		0.08	0.11

Table 2.11. Summary of L_{max} for two-lane Moments for 10 years

	ADTT=5000 - P_{sxs}=2%			ADTT=1000- P_{sxs}=1.25%			ADTT=100- P_{sxs}=0.5%		
span	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$
40-ft	2.79	0.05	0.10	2.56	0.06	0.10	2.15	0.08	0.11
60-ft	2.52	0.05	0.12	2.31	0.06	0.12	1.94	0.08	0.12
100-ft	2.45	0.05	0.10	2.25	0.05	0.10	1.91	0.08	0.10
120-ft	2.41	0.04	0.10	2.22	0.05	0.10	1.88	0.08	0.10
200-ft	2.14	0.04	0.10	1.98	0.05	0.10	1.69	0.07	0.10
average		0.05	0.10		0.05	0.10		0.08	0.11

Table 2.12. Summary of L_{max} for one-lane Moments for 5 years

	ADTT=5000			ADTT=1000			ADTT=100		
span	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$
40-ft	2.22	0.04	0.13	2.09	0.05	0.12	1.89	0.06	0.12
60-ft	2.02	0.04	0.14	1.90	0.05	0.14	1.72	0.06	0.13
100-ft	1.94	0.04	0.12	1.83	0.05	0.12	1.66	0.06	0.11
120-ft	1.90	0.04	0.11	1.79	0.05	0.11	1.62	0.06	0.11
200-ft	1.67	0.04	0.10	1.57	0.05	0.10	1.43	0.06	0.10
average		0.04	0.12		0.05	0.12		0.06	0.12

Table 2.13. Summary of L_{max} for one-lane Moments for 10 years

	ADTT=5000			ADTT=1000			ADTT=100		
Span	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$
40-ft	2.27	0.04	0.13	2.15	0.05	0.13	1.95	0.06	0.12
60-ft	2.07	0.04	0.14	1.95	0.05	0.14	1.77	0.06	0.14
100-ft	1.99	0.04	0.12	1.88	0.05	0.12	1.71	0.06	0.11
120-ft	1.94	0.04	0.11	1.84	0.05	0.11	1.67	0.05	0.11
200-ft	1.71	0.04	0.10	1.61	0.04	0.10	1.47	0.05	0.10
Average		0.04	0.12		0.05	0.12		0.06	0.12

Table 2.14. Summary of L_{max} for two-lane Shear for 5 years

	ADTT=5000 - P_{sxs}=2%			ADTT=1000- P_{sxs}=1.25%			ADTT=100- P_{sxs}=0.5%		
span	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$	$\overline{L}_{\max}$	$V_{L\max}$	$V_{site-to-site}$
40-ft	2.66	0.05	0.11	2.42	0.06	0.11	2.01	0.09	0.11
60-ft	2.67	0.05	0.10	2.44	0.06	0.10	2.02	0.09	0.11
100-ft	2.62	0.05	0.10	2.40	0.06	0.10	2.01	0.08	0.10
120-ft	2.54	0.05	0.10	2.33	0.06	0.10	1.95	0.08	0.10
200-ft	2.22	0.05	0.10	2.03	0.06	0.10	1.71	0.08	0.10
average		0.05	0.10		0.06	0.10		0.08	0.10

Load Model on a Bridge Member

The statistical analysis of the WIM data as described in the previous section leads to the calculating the mean value of the maximum normalized total static load effect, $\overline{L}_{max}$, applied on the bridge. To analyze the safety of a single member under actual traffic conditions, the distribution of the total load to the most heavily loaded member must be considered along with the dynamic amplification due to the moving vehicles. Accordingly, the mean live load effect on the most critical girder of two-lane and one-lane multi-girder bridges can be respectively obtained from:

For two loaded lanes $$\overline{LL} = \overline{L}_{max} \times HL_{93} \times \overline{IM} \times \overline{D.F.}/2 \quad (2.31)$$

For a single loaded lane $$\overline{LL} = \overline{L}_{max} \times HL_{93} \times \overline{IM} \times \overline{D.F.}/1.2 \quad (2.32)$$

Where HL_{93} is the effect of the HL-93 vehicle, $\overline{IM}$ is the mean dynamic amplification factor and $\overline{D.F.}$ is the mean of the load distribution factor. Dividing the D.F. of two lanes by 2 is done to account for the fact that the $\overline{L}_{max}$ values for two lanes while the D.F. values are applied on a single lane of load. The one-lane distribution factor is divided by 1.2 to remove the multiple presence factor already included in the LRFD equations.

To obtain the mean values of the load distribution factors in Eq. (2.31) and (2.32), Nowak (1999) and Moses (2001) assume that the D.F. values given by the AASHTO LRFD Specifications are the actual mean values of the distribution factors. This is not strictly speaking correct because the LRFD equations for the distribution factors include some level of conservatism as explained by Zokaie et al (1988). However, in this report because no other data is available, we are following the assumption made by Nowak (1999) that the AASHTO LRFD load distribution equations give the mean values of the live load distribution. This assumption is on the conservative side as suggested by Zokaie et al (1988).

For bending of typical reinforced and prestressed concrete and steel girder bridges loaded by one lane of traffic, the load distribution factor equation is given as (AASHTO, LRFD, 2007):

$$D.F. = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \quad (2.33)$$

Where S is the beam spacing, L is the span length, t_s is the deck thickness, and K_g is a beam stiffness parameter. Not having enough information to calculate the term within the last parenthesis, it is taken as 1.0 as recommended in the AASHTO LRFD specifications. Note that Equation 33 already includes a multiple presence factor MP=1.2

which accounts for the higher probability of having one heavy truck in one lane as compared to the probability of having two side-by-side heavy trucks in two adjacent lanes. Since in this study we calculated L_{max} directly, the multiple presence factor will have to be removed when calculating the maximum applied live load as shown in Equation (2.32).

For two lanes loaded, the load distribution factor equation for bending becomes (AASHTO, LRFD, 2007):

$$D.F. = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} \tag{2.34}$$

According to the AASHTO LRFD, Equation (2.34) should be applied on the effect of a single lane. Since, the calculations for two lanes of loading, we need to divide the load effect by 2 as shown in Eq, 31.

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

$$D.F. = 0.36 + \frac{S}{25} \tag{2.35}$$

For two lanes loaded, the shear distribution factor becomes:

$$D.F. = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \tag{2.36}$$

The COV of the maximum live load effect on a single girder should account for the site-to-site variability represented by $V_{site\text{-}to\text{-}site}$, the variability within a site represented by V_{Lmax}, and also must account for the uncertainty associated with the limited WIM data sample size. In NCHRP 12-76, Sivakumar et al suggested that the effect of data limitation be represented by an additional Coefficient of Variation, V_{data}. The variability in the dynamic amplification factor, V_{IM} and the variability in the load distribution factor V_{DF} should also be included. Thus, the final COV for the applied live load effect on a single beam can be obtained from:

$$V_{LL} = \sqrt{V_{L\max}^2 + V_{site-to-site}^2 + V_{data}^2 + V_{IM}^2 + V_{DF}^2} \tag{2.37}$$

$V_{site\text{-}to\text{-}site}$ is obtained by comparing the $\overline{L}_{max}$ values from different WIM sites within the state. For the data shown above, $V_{site\text{-}tos\text{-}ite}$ is observed to be on the order of 10% to 12% for New York State sites. In these calculations we will use an average value of 10% for two-lane loadings and 12% for one-lane loads. For the two-lane moments and shear cases, the uncertainties within a site are associated with a COV on the order of V_{Lmax}=5% for the two side-by-side load effect when the ADTT=5000, 6% for ADTT=1000 and around 8% when the ADTT=100. For one-lane loadings, these values are slightly reduced to 4%, 5% and 6% respectively. Additional uncertainties are associated with L_{max} due to the limited number of data points used in the projections and the confidence levels associated with the number of sample points. Using the +/-95% confidence limits, Sivakumar, Ghosn & Moses (2008) estimated that the COV associated with the use of one year worth of WIM data is on the order of V_{data}=3% for the two-lane case and 2% for the one-lane case.

Nowak (1999) observed that the dynamic amplification factors augmented the static load effect, L_{max}, by an average of 10% for side-by-side trucks. This indicates that the mean dynamic amplification factor in Eq. (2.31) is $\overline{IM} = 1.10$. The dynamic amplification also resulted in a COV of V_{IM}=5.5% on the two-lane effect. For single lanes of traffic, the dynamic mean and COV are $\overline{IM} = 1.13$ and V_{IM}=9%.

In previous studies on live load modeling, Ghosn & Moses (1985) included the uncertainties in estimating the lane distribution factor which was associated with a COV equal to V_{DF}=8% based on field measurements on typical steel and prestressed concrete bridges.

The use of Equations (2.31) and (2.32) along with the approach followed in this report to find $\overline{L}_{max}$ based on the implementation of Eq. (2.14), assume that the truck traffic within the bridge service life will not experience any growth in volume or changes in the truck types, configurations or weight spectra during the service life period. This is the same assumption made by Nowak (1999) and Moses (2001). The projection of changes in truck volume and type require a detailed forecasting analysis of economic growth and economic changes in New York State and its effect on the transportation of goods and the modes of transportation throughout the State that is beyond the scope of this study.

Review of the AASHTO LRFR and LRFD Live Load Models

As stated by Kulicki, Mertz and Nowak (2007), “at the time of calibration of AASHTO LRFD in the 1980’s, there was no reliable truck data available for the USA. All available WIM data was found to be inadequate or flawed for this purpose. Therefore, the statistical parameters of live load used for the AASHTO LRFD calibration were based on the Ontario truck survey. The ADTT usually associated with that survey is 1000. In the survey of 9,250 trucks, an attempt had been made to subjectively select “heavy” trucks to weigh so that data was probably skewed to the high side, i.e., ADTT>1000, but there is no way to quantify the effect. Since some effort had been made to select heavier trucks, the 9,250 measured trucks were considered representative of two weeks traffic at an ADTT of 1,000.”

Also, at the time of the AASHTO LRFD calibration, there was very limited data on truck headways and the probability of side-by-side truck occurrences. Nowak (1999) used data obtained from some sites in Michigan to estimate that 1 out 15 trucks (P_{sxs}=6.67%) will travel over a bridge alongside another truck and he assumed that this percentage applies to all the trucks in the Ontario survey and assumed that the trucks in Ontario survey although biased to the heavy side do actually represent the whole truck population. Nowak (1999) subsequently used these assumptions along with some additional assumptions on a limited level of truck weight correlation in a simulation program to obtain the live load statistics for different projection periods up to a return period of 75 years which had been selected as the design for U.S. bridges. Nowak (1999) also assumed that the tail end of the truck weight histogram followed a normal probability distribution.

To avoid the need for executing a full-fledged simulation to obtain the maximum live load effect, L_{max}, Moses (2001) used a simplified approach which provides consistent results with those of Nowak (1999). In fact, Moses (2001) studied the data and tables and figures of Nowak (1999) and observed that the truck weight spectra used for the AASHTO LRFD calibration produced average load effects which are equivalent to the effect of a 3S-2 truck with a gross weight of 68 kips. Also, by studying the Normal probability plots in Nowak's (1999) report and the projections for maximum effect for return periods varying between one day until 75 years, Moses (2001) observed that these plots and projections imply that the average truck weight of 68 kips is associated with a standard deviation of about 18 kips. By looking at raw Weigh-In-Motion data from sites throughout the U.S., Moses (2001) observed that the average weight of trucks on typical sites is in the range of 45 to 55 kips which is much lower than the 68-kips implied by the AASHTO LRFD data. He observed that the 68 kips is more in the range of the weights of the upper 20% of the data. Moses (2001) also noted that typical truck weight histograms are bi-modal and are not close to a Normal distribution. But, that the second mode, which could very roughly approach a Normal distribution, would approximately represent the upper 20% of the data. These observations, combined with the fact that the Ontario survey did not sample all the trucks on the highway and the fact that Nowak (1999) used a typical ADTT of 1000 trucks per day, as subsequently confirmed by Kulicki et al (2007), while bridge sites with heavy truck traffic may average close to 5000 trucks per day, have led Moses (2001) to conclude that the AASHTO LRFD calibration data can be matched by conservatively using the upper 20% of the trucks that cross a bridge site. These assumptions would provide a simple approach to project for the maximum load effect expected in the design life or the rating period of a bridge structure.

By following the above-stated assumptions regarding the shape of the truck weight histogram and the number of multiple lane side-by-side loading events, Moses (2001) was able to closely match the results of the AASHTO LRFD live load modeling effort. Accordingly, Moses (2001) proposed to model the live load effect on bridges based on the following assumptions:

- The histogram representing the heaviest 20% of the truck weights follows a Normal distribution.

- The mean value of the heaviest 20% of the truck weights is 68 kips with a standard deviation of 18 kips. These heavy trucks have the configurations of the AASHTO 3S-2 legal truck.
- A typical site will be subject to 1000 heavy truck loading events in a typical day and 1/15 of these events will consist of multilane events which are essentially side-by-side trucks.

Therefore, the weight of the maximum truck expected in a 75-year design life in one lane can be approximately obtained from:

$$W_{75year} = \overline{W}_H + t_{75\,\mathrm{single}} \sigma_H = 68 + 5.39 \times 18 = 165\,kips \tag{2.38}$$

where $\overline{W}_H$ is the mean weight of the heavy trucks, σ_H is the standard deviation of the heavy truck weights, and $t_{75\ single}$ is the standard deviate corresponding to the probability of exceeding the number of single lane loading events, N, in return period T=75 years. Thus, $t_{75\ single}$ is the standard deviate of the probability of exccedance in 75 years, P_{ex}, which can be calculated as a function of N by:

$$P_{ex} = \frac{1}{N} = \frac{1}{75\ years \times 365\ days/year \times 1000\ heavy\ trucks/day} = 36.5 \times 10^{-9} \tag{2.39}$$

which when entered in a Normal probability table produces $t_{75\ single}$=5.39.

The weight of the maximum total weight of two side-by-side trucks expected in a 75-year design life can be obtained from:

$$W_{75year} = 2\overline{W}_H + t_{75\,side-by-side}\sqrt{2}\sigma_H = 2 \times 68 + 4.87 \times \sqrt{2} \times 18 = 260\ kips \tag{2.40}$$

where $t_{75\ side\text{-}by\text{-}side}$ is the standard deviate corresponding to the probability of exceeding the number of side-by-side loading events in 75 years. The mean of two side-by-side heavy trucks is $2\overline{W}_H$ and the standard deviation of the combined weights of two independent heavy side-by-side trucks is $\sqrt{2}\sigma_H$. Thus, $t_{75\ side\text{-}by\text{-}side}$ is the standard deviate of P_{ex} calculated as a function of the number of side-by-side as:

$$P_{ex} = \frac{1}{75\ years \times 365\ days/year \times \dfrac{1000}{15}\ side-by-side\ heavy\ trucks/day} = 54.8 \times 10^{-8} \tag{2.41}$$

which when entered in a Normal probability table produces $t_{75\ side\text{-}by\text{-}side}$=4.87.

By using the configuration of the AASHTO legal 3S-2 truck with the weight of the maximum 75-year truck W_{75year}, the moment effect of the maximum expected live load can be obtained for different span lengths. These values are compared to those provided by Nowak (1999) as shown in Table 2.15. The results show acceptable agreement for single lane loading with differences between 3% for the 60-ft span up to 11% for the 200-ft span. The two-lane loading results show larger differences which would reach 17% for the 200-ft span. These differences may be due to the fact that the Moses (2001) model assumes that the dominant trucks have the configuration of 3S-2 vehicles while the Nowak (1999) simulations account for all the truck configurations.

The database used by Nowak (1999) for modeling the live loads for the AASHTO LRFD and subsequently adopted by Moses (2001) for calibrating the AASHTO LRFR is over thirty years old. It is reasonable to expect that current truck loads on New York State bridges will have significantly different truck weight configurations and statics.

Table 2.15. Comparison of maximum 75-year load effect using Moses's model and Nowak's results.

Simple span	One-lane moment (kip-ft)		Two-lane moment (kip-ft)	
	AASHTO LRFR	AASHTO LRFD	AASHTO LRFR	AASHTO LRFD
30 ft	508	537	800	913
60 ft	1403	1444	2210	2455
90 ft	2617	2608	4124	4434
120 ft	3855	3917	6074	6659
200 ft	7154	8036	11274	13661

Kulicki et al (2007) explain that the results presented by Nowak (1999) were subsequently updated to use 5000 heavy trucks per day rather than the original ADTT of 1000. Accordingly, they found that the live load moment effects will increase by about 2.5% and the shear load effect will increase by 3.5% (Kulicki et al, 2007). By using 5000 trucks per day, in the Moses (2001) model, the standard deviate $t_{75\ single}$ becomes 5.67 and $t_{75\ side\text{-}by\text{-}side}$=5.18. In this case, the results of Moses (2001) and Kulicki et al (2007) are compared as shown in Table 2.16 revealing similar percent differences as those of Table 2.15.

Table 2.16. Comparison of maximum 75-year load effect using Moses' model and updated Nowak's results.

Simple span	One-lane moment (kip-ft)		Two-lane moment (kip-ft)	
	AASHTO LRFR	AASHTO LRFD	AASHTO LRFR	AASHTO LRFD
30 ft	523	550	824	936
60 ft	1445	1480	2277	2516
90 ft	2696	2673	4250	4544
120 ft	3971	4015	6259	6825
200 ft	7371	8237	11617	14003

Comparison of Proposed Live Load Model to AASHTO LRFR

Using Equation (2.37) with the statistical data listed above, namely V_{Lmax}=4%, $V_{site\text{-}to\text{-}site}$=12%, V_{data}=2%, V_{IM}=9% and V_{DF}=8%, we observe that the live load COV, for one lane will be V_{LL}=18% which is equal to the value used by Nowak (1999) for the 75-year maximum total load effect for the combination of live load and dynamic allowance. This same V_{LL}=18% was subsequently used by Moses (2001) for the 5-year maximum two-lane load effect. For the two-lane case Eq. (2.37) yields a slightly lower value of V_{LL}=15%. Neither did Nowak (1999) nor Moses (2001) identify the contributions of each random parameter to V_{LL}. The separation of the COV's for each of the random variables that underline the live load effect, however, is important for the reliability analysis of Permit loads as will be explained in Chapter 3 of this report.

Although as implied in Eq. 2.22 through 2.28, L_{max}, approaches a Gumbel probability distribution, Moses (2001) assumed that the overall live load effect LL of Eq. (2.31) and (2.32) follows a Lognormal probability distribution. Given that LL is the product of three random variables, and following the Central Limit Theorem that states that a random variable which is the product of several underlying random variables will approach a lognormal distribution as the number of the underlying variables increases, Moses' (2001) modeling of the live load as lognormal random variable may be justified. On the other hand, Nowak (1999) assumed that the sum of all the applied loads including the dead and live loads follows a normal distribution. This also may be justified based on the Central Limit Theorem that states that the sum of several random variables will approach a Normal distribution. In this report, we will use a FORM algorithm where the dead load effect is assumed to be normal, L_{max}, is Gumbel and the remaining variables including the impact and load distribution factors are normal. We also include two normal modeling variables $\lambda_{site\text{-}to\text{-}site}$ and λ_{data} which have mean values of 1.0 and COV's $V_{site\text{-}to\text{-}site}$ and V_{data} to represent the effects of the site-to-site variability and the variations in the results due to data sampling size.

As shown in Table 2.4, the average weight of the heavy trucks implied in the Ontario database which had been taken during the AASHTO LRFR calibration to be equal to 68 kips is significantly lower than the average value of 91 kips observed from the New York WIM sites. On the other hand, the standard deviation of the heaviest trucks is on other order of 15 kips. To account for the higher weight spectra, the AASHTO LRFR allows the adjustment of the live load factors based on state-wide WIM data. The approach would follow the same steps proposed by Moses (2001) as given in Eq. (2.38) through (2.40) but by changing the mean value and the standard deviation of the truck weight $\overline{W}_H$ and σ_H to those of the heaviest 20% of the trucks as measured using state-specific WIM data. For the cases of the data collected at the sites analyzed in this chapter this would mean using $\overline{W}_H$ =91 kips and σ_H=15 kips. When $\overline{W}_H$ 91 kips and σ_H=15 kips are entered into Eq. (2.38) and (2.40), but using a 5-year rating period, the expected maximum live loads will appear as shown in Tables 2.17 and Table 2.18 in the group of columns labeled Moses (2001) model using the weights of upper 20% of NY WIM trucks.

Tables 2.17 and 2.18 compare the results of $\bar{L}_{\max} \times HL_{93}$ obtained from the New York State WIM data as obtained from the load modeling approach proposed in this report to those used by Moses (2001) during the calibration of the AASHTO LRFR and what would have been obtained if the truck weight average of 91 kips and the standard deviation of 15 kips were used with the Moses (2001) modeling method keeping the Moses (2001) side-by-side probabilities.

Table 2.17 shows an increase on the order of 30% in the expected maximum 5-year two-lane load effect as calculated in this report when compared the AASHTO LRFR live load data. The table also shows that the mean load effect obtained using the approach described in this report still yields higher values although closer when compared to those obtained using the Moses modeling approach with the New York WIM heavy truck weight data. It is noticed that despite the many simplifying assumptions made by Moses regarding the shape of the truck weight spectra and the percent of trucks that will be side-by-side, the differences in the load effects when the truck weights collected from the New York WIM data are used, are in the range of 15% to 5% depending on the ADTT with the higher difference being for ADTT=5000 and the lower difference is for sites with ADTT=100.

The differences are however significantly higher for the one-lane cases. Where the WIM data for New York show an increase on the order 55% to 64% in the one-lane effect when compared to the generic data used in the calibration of the AASHTO LRFR. It is noted that the ratio between the two-lane effect to the one-lane effect as calculated in this report is on the order of 1.25 to 1.21 for sites with ADTT=5000 or 1.13 to 1.08 for ADT=100. These ratios are significantly lower than the 1.70 (2x0.85) observed during the AASHTO LRFD calibration. The decrease in the multi-lane to one-lane ratio is primarily due to the high number of overweight trucks allowed on New York bridges but the lower probability of having two of these overweight trucks side-by-side. This lower ratio may be an indication that in some cases the one-lane load may govern the loading of multi-lane bridges.

It is clear that the results projected in this report from the New York WIM data are significantly higher than those used for the calibration of the AASHTO LRFR. A major source for the difference is the higher truck weights observed in New York as compared to the truck weights in the generic LRFR database. The differences between the results of the Moses (2001) model applied with the New York average truck statistics and the results from this report are on the same order as the differences between the ASSHTO LRFR and AASHTO LRFD results. These smaller differences may be primarily due to the assumptions made by Moses (2001) on the shape of the heavy truck weight histogram and due to the variations in the truck configurations from the 3S-2 configuration that Moses (2001) assumed as being dominant.

Tables 2.18, 2.19, and 2.20 compare the two-lane and one-lane moment and two-lane shear load effects of the projected 5-year maximum load obtained in this study to the AASHTO Legal and design trucks. The tables show that the two-lane moment may be on the order of 2.7 to 2.1 times the HL-93 load effect with the higher value corresponding to the shorter spans. These results will be used in Chapter 3 to calibrate New York State live load factors for Legal Load ratings and Permit Load rating and to propose a load posting methodology. The rest of this Chapter presents a review of the AASHTO LRFD and AASHTO LRFR calibration efforts to examine the target reliability levels implied in the previous calibrations and to validate the reliability-based calibration approach that will be followed in this report.

Table 2.17. Comparison of 5-year Mean Maximum Moment Effects

Two-lane Moment Effects									
	AASHTO LRFR data			Moses (2001) model using the weights of the upper 20% of NY WIM trucks			This report		
span	ADTT=5000	ADTT=1000	ADTT=100	ADTT=5000	ADTT=1000	ADTT=100	ADTT=5000	ADTT=1000	ADTT=100
60-ft	2088	1904	1572	2324	2170	1893	2665	2426	2012
120-ft	5739	5232	4319	6387	5964	5203	7123	6517	5456
200-ft	10653	9712	8017	11855	11071	9658	13622	12514	10494
One-lane Moment Effects									
	AASHTO LRFR data			Moses (2001) model using the weights of the upper 20% of NY WIM trucks			This report		
span	ADTT=5000	ADTT=1000	ADTT=100	ADTT=5000	ADTT=1000	ADTT=100	ADTT=5000	ADTT=1000	ADTT=100
60-ft	1342	1280	1202	1413	1361	1297	2197	2067	1871
120-ft	3667	3498	3287	3862	3721	3545	5759	5426	4911
200-ft	6784	6471	6080	7144	6884	6558	10885	10233	9320

Table 2.18. Comparison of 5-year Mean Maximum Two-Lane Moment AASHTO Truck Load Moment Effects

Span	$\overline{L}_{max} \times HL_{93}$ ADTT=5000	$\overline{L}_{max} \times HL_{93}$ ADTT=1000	$\overline{L}_{max} \times HL_{93}$ ADTT=100	SU-4	AASHTO legal 3	AASHTO legal 3S2	Max. Legal	HL-93	HS20
40-ft	1558	1420	1174	406	349.6	324.4	349.6	577.8	449.8
60-ft	2665	2426	2012	676	598.4	618.4	618.4	1094.6	806.6
100-ft	5520	5057	4221	1216	1097.4	1331.8	1331.8	2324.0	1524.0
120-ft	7123	6517	5456	1486	1347.4	1690.2	1690.2	3032.0	1880.0
200-ft	13622	12514	10494	2566	2346.8	3127.0	3127.0	6503.3	3303.3

Table 2.19. Comparison of 5-year Mean Maximum One-Lane Moment AASHTO Truck Load Moment Effects

span	$\bar{L}_{max} \times HL_{93}$ ADTT=5000	$\bar{L}_{max} \times HL_{93}$ ADTT=1000	$\bar{L}_{max} \times HL_{93}$ ADTT=100	SU-4	AASHTO legal 3	AASHTO legal 3S2	Max. Legal	HL-93	HS20
40-ft	1272	1197	1082	406	349.6	324.4	349.6	577.8	449.8
60-ft	2197	2067	1871	676	598.4	618.4	618.4	1094.6	806.6
100-ft	4500	4245	3850	1216	1097.4	1331.8	1331.8	2324.0	1524.0
120-ft	5759	5426	4911	1486	1347.4	1690.2	1690.2	3032.0	1880.0
200-ft	10885	10233	9320	2566	2346.8	3127.0	3127.0	6503.3	3303.3

Table 2.20. Comparison of 5-year Mean Maximum Two-Lane Shear AASHTO Truck Load Moment Effects

span	$\bar{L}_{max} \times HL_{93}$ ADTT=5000	$\bar{L}_{max} \times HL_{93}$ ADTT=1000	$\bar{L}_{max} \times HL_{93}$ ADTT=100	SU-4	AASHTO legal 3	AASHTO legal 3S2	Max. Legal	HL-93	HS20
40-ft	181	165	137	45	41	39	41	68	55
60-ft	214	195	162	48	44	50	50	80	61
100-ft	254	233	195	51	46	59	59	97	65
120-ft	267	245	205	51	47	61	61	105	66
200-ft	295	270	227	52	48	65	65	133	90

2.6 Review of AASHTO LRFD and LRFR Calibration

The calibration of the AASHTO LRFD and LRFR specifications as executed by Nowak (1999) and Moses (2001) followed the basic reliability procedure and the modeling assumptions outlined in the previous sections of this Chapter. Specifically, during the calibration of these specifications, safety is measured using the reliability index, β which can be either calculated from a Monte Carlo simulation or using the Rackwitz-Fiessler (1978) algorithm which has been programmed into a First Order Reliability Method (FORM) program. These two methods can accommodate all probability distribution types. Alternatively, and for the simple cases when the Resistance and the Loads can be represented by either Normal or Lognormal probability distributions, the closed form expressions of Eq. (2.7) and (2.9) may be used.

Nowak (1999) divided the random variables that control the safety of bridge members into three categories: The member resistance R, the permanent dead load effects, DL, and the live load effect, LL such that the total load S in Eq. (2.3) is given as S=DL+LL. He assumed that the combined load effect (S=DL+LL) follows a Normal distribution while the resistance R follows a Lognormal distribution. He used the Rackwitz-Fiessler (1978) algorithm to calculate the reliability index β although a Monte Carlo simulation procedure can also be used as explained in the report NCHRP 20-07 Task 186 (Kulicki et al; 2007).

To describe the reliability calculation process, the case when the resistance, the dead load and the live load follow Gaussian (Normal) probability distributions is considered in the following formulation. In this case, Eq. (2.7) would become:

$$\beta = \frac{\overline{R} - \overline{DL} - \overline{LL}}{\sqrt{\sigma_R^2 + \sigma_{DL}^2 + \sigma_{LL}^2}} \tag{2.42}$$

Note that the live load effect of one member, LL, is a function of several parameters, including the total load applied on the bridge, how the total load produces moment and shear effects, how the total effects are distributed to each member and the dynamic response of the member due to the moving load. In this Section, we will use the same live load data previously used by Nowak (1999) and Moses (2001) during the calibration of the AASHTO LRFD and LRFR codes. Chapter 3 will use the live load statistics obtained from the New York WIM data. The objective of this section is to illustrate how the reliability index calculations are performed and to confirm that the approach followed in this study is consistent with the approaches used during the calibration of the AASHTO LRFD and LRFR specifications.

To evaluate the reliability index implied in current load rating procedures, the resistance implied for different rating factors is first calculated. The load rating equation takes the form:

$$R.F. = \frac{\phi R_n - \gamma_{DW} D_W - \gamma_{DC} D_C}{\gamma_L L_n} \tag{2.43}$$

where ϕ and γ are the resistance and load factors, R_n is the nominal resistance, D_W is the dead load effect for the wearing surface, D_C is the dead load effect for the components and attachments and L_n is the live load effect of the appropriate rating load including dynamic allowance and load distribution factor.

The resistance and live load factors to be used in Eq. (2.43) depend on the specifications being followed. For example, according to the LRFR and LRFD specifications ϕ =1.0 for the bending moment capacities of steel and prestressed concrete members, γ_{DW}=1.50, γ_{DC}=1.25. The Inventory Rating live load factor is given as γ_L=1.75 and γ_L=1.35 for the Operating Rating using the HL-93 live load model. The AASHTO LRFR Operating Rating for the legal loads is given as γ_L=1.80 for ADTT $\geq$ 5000, γ_L=1.65 for $100 \leq$ ADTT ≤ 5000 and γ_L=1.40 for ADTT $\leq$ 100. The dynamic allowance factor is 1.33 times the truck moment effect and the load distribution factor is calculated as a function of span length and beam spacing for different numbers of loaded lanes as given in Eq (2.33) through (2.36) for interior beams.

The AASHTO LRFD was calibrated so that all bridge members designed using the specified load and resistance factors produce a uniform level of risk expressed in terms of a reliability index β equal to a target value β_{target}=3.5. The target β_{target}=3.5 was extracted based on the average reliability index obtained for a set of representative bridge members designed using the LFD approach. The representative bridge sample used by Nowak (1999) included simple span bridges of span lengths =30ft, 60-ft, 90-ft, 120-ft and 200-ft. The reliability analysis of a set of composite steel, non-composite steel, prestressed concrete and reinforced concrete bridges with beams at 4-ft, 6-ft, 8-ft, 10-ft and 12-ft spacing for both shear and maximum moment was executed using the models described in the previous section and a FORM algorithm. In this Chapter, the same sample of composite steel bridges is used under maximum moment to illustrate the methodology.

For the AASHTO LRFR Inventory Rating when a bridge member provided a rating factor exactly equal to R.F.=1.0, the rating equation (2.43) becomes the same as the design equation for LRFD. Thus, the reliability index for the LRFR Inventory Rating would be the same as that obtained by Nowak (1999) and should produce the same β_{target}=3.5 for a 75-year design period. For the Operating Rating level, the AASHTO LRFR was calibrated to produce a reliability index β_{target}=2.5 for a 5-year rating period.

During the calibration of the AASHTO LRFD, Nowak (1999) assumed that the resistance is Lognormal while the combined effect of the dead and live loads is Normal. The reliability index calculations were then executed using a First Order Reliability Method (FORM) Ractwitz Fiessler algorithm and not Eq. (2.42). On the other hand, Moses (2001) used the LogNormal model of Eq. (2.9) which assumes that both the resistance and the loads are lognormal. In order to illustrate the procedure and to compare the results from the different models, this report will compare the reliability calculations using the Normal model of Eq. (2.42), the Lognormal model of Eq. (2.9) and the Rackwitz-Fiessler FORM algorithm.

In this Chapter several cases are considered for the nominal design loads and the live load models. These include:

1. Evaluation of HS-20 Nominal load Inventory Rating.
2. Evaluation of HL-93 Nominal load Inventory Rating.
3. Evaluation of HS-20 Nominal load and AASHTO Legal Load LFD Operating Rating
4. Evaluation of HL-93 and Legal Load LRFR Operating Rating.

Cases 1 and 2 are selected to verify that the calculations performed in this report are consistent with the results of Nowak (1999). Cases 3 and 4 are used to verify the calibration procedure used during the calibration of the AASHTO LRFR code.

Case 1. Evaluation of HS-20 Nominal Load Inventory Rating.

A simple example is provided in the first part of this section to illustrate the reliability analysis procedure. The example assumes a 60-ft simple span bridge with composite steel beams at 8-ft spacing. Following the data provided by Nowak (1999), the dead loads are given as Dc_1=70kip-ft, Dc_2=414 kip-ft, D_W=97 kip-ft. The HS-20 vehicular load produce a moment HS_{20}= 805 kip-ft. The impact factor is IM=1.27 and the distribution factor is D.F.=1.455 of the wheel load. For the LFD Inventory rating for steel beams in bending a resistance factor ϕ=1.0 is used along with a dead load factor γ_D=1.30 and live load factor γ_L=2.17. When the Inventory Rating Factor is exactly R.F.=1.0, the application of Eq. (2.43) with ϕ=1.0, γ_D=1.30 and γ_L=2.17 implies that the nominal resistance R_n can be calculated as R_n=2369 kip-ft:

$$\begin{aligned}\phi R_n &= \gamma_D\left(D_{c1}+D_{c2}+D_w\right)+\gamma_L\left(HS_{20}\times IM\times D.F.\right)\\ &=1.3(70+414+97)+2.17(805\times 1.27\times 1.455/2)=2369\ kip-ft\end{aligned}$$

Using Eq. (2.18) and (2.20), the mean dead loads and resistance as well their standard deviations are given as:

$$\begin{aligned}&\overline{D}_{C1}=1.03\times 70=72.1\,kip-ft && \sigma_{DC1}=72.1\times 8\%=5.8\ kip-ft\\ &\overline{D}_{C2}=1.05\times 414=434.7\ kip-ft && \sigma_{DC2}=434.7\times 10\%=43.5\ kip-ft\\ &\overline{D}_{W}=1.0\times 97\ kip-ft && \sigma_{DC1}=97\times 25\%=24.3\ kip-ft\\ &\overline{R}=1.12\times 2369=2653.3\ kip-ft && \sigma_{R}=2653.3\times 10\%=265.3\ kip-ft\end{aligned}$$

The load distribution factor for two lanes is obtained from Eq. (2.34)

$$D.F. = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1} = 0.075 + \left(\frac{8}{9.5}\right)^{0.6}\left(\frac{8}{60}\right)^{0.2}(1)^{0.1} = 0.678$$

Nowak (1999) states that for the 60ft simple span beam, the mean maximum 75-year moment for one lane is given as L_{max} xHL_{93}=1444 kip-ft. To obtain the 2-lane moment he suggested multiplying this value by 2x0.85 leading to L_{max}xHL_{93}=2454.8kip-ft. The mean live load for side-by-side trucks is given as $\overline{IM} = 1.10$ and the mean distribution factor is that obtained from the AASHTO LRFD equations. Accordingly, the mean live load moment on one member is obtained as:

$$\overline{LL} = L_{\max} \times HL_{93} \times \overline{IM} \times D.F./2 = 2454.8 \times 1.10 \times 0.678/2 = 915\ kip-ft$$

With a live load COV, V_{LL}=18%, the standard deviation is given as:

$$\sigma_L = 915 \times 18\% = 164.7 kip-ft\ .$$

The total mean load and its standard deviation become:

$$\overline{S} = \overline{D} + \overline{L} = (72.1 + 434.7 + 97) + 915 = 1518.8\ kip-ft$$
$$\sigma_S = \sqrt{5.8^2 + 43.5^2 + 24.3^2 + 164.7^2} = 172\ kip-ft$$

Assuming that both the resistance and the total load are normal, the application of Eq. (2.7) will produce the following reliability index value:

For the Normal Model $\beta = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \frac{2653.3 - 1518.8}{\sqrt{265^2 + 172^2}} = 3.59$

Alternatively, if one assumes that the resistance and the total load are lognormal, the reliability index obtained from Eq. (2.9) is:

For the LogNormal Model: $\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{2653.3}{1518.8}\right)}{\sqrt{0.10^2 + \left(\frac{172}{1518.8}\right)^2}} = 3.69$

For the case when R is Lognormal and S is Normal as used by Nowak (1999) the Rackwitz-Fiessler FORM algorithm is used. In this case, we find β=3.94 which is very similar to the value β=3.96 reported by Nowak (1999) for this 60-ft composite steel beam in bending.

Subsequent to the publication of the calibration report, Kulicki et al (2007) observed that the original calibration was executed for ADTT of 1000 trucks per day. They found that a change of ADTT from 1000 trucks per day to 5000 should lead to an increase in the estimated maximum live load moment effect by about 2.5% and in the maximum shear effect by 3.5%. A 2.5% increase in the mean of the maximum live load moment of a 60-ft example bridge would lead to an adjusted mean live load moment on a member equal to $\overline{LL}$=938 kip-ft and the standard deviation becomes σ_L=169kip-ft. In this case, the reliability index for the Normal model is reduced to β=3.49, for the LogNormal model the reliability index becomes β=3.53 and for LogNormal-Normal case β=3.82.

Figure 2.11 provides a plot of the reliability index obtained using the Rackwitz-Fiessler also known as the FORM algorithm for simple span composite steel bridges varying in length between 30-ft to 200-ft having beam spacing between 4-ft and 12-ft. The dead load data for these bridges, which are needed to execute the calculations, are obtained from Nowak (1999). In this case, the modified live load data given by Kulicki et al (2007) is used in the reliability calculations. The data shows that the reliability indexes produce an average value of β=3.38 with a minimum value of 1.95 and a maximum value of 4.38. The results show a large difference between the values obtained for different beam spacing. By using a β_{target}=3.5, which is slightly higher than that obtained for LFD bridges, during the calibration of the AASHTO LRFD (LRFR Inventory Ratings), Nowak (1999) was intentionally introducing a small level of conservatism.

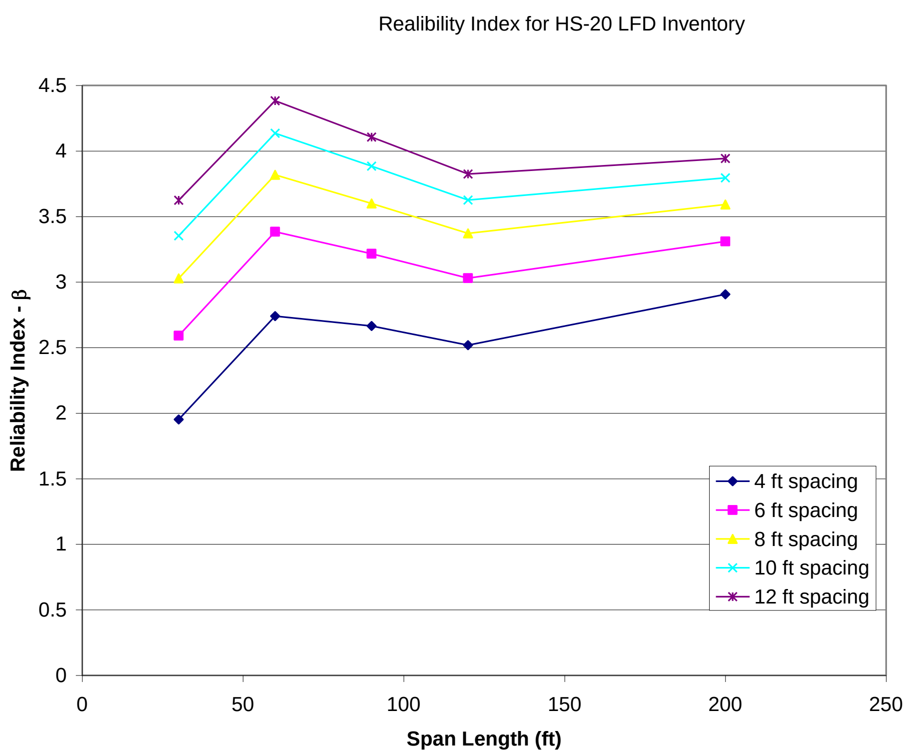


Figure 2.11 Reliability indexes for composite steel bridges in bending for LFD HS-20.

Case 2. Evaluation of HL-93 Nominal Load Inventory Rating.

In this case, we are evaluating the reliability index implied in the AASHTO LRFR Inventory Ratings. As a simple example, we analyze the same configuration for a 60-ft simple span bridge with composite steel beams at 8-ft spacing which was used to illustrate the Case 1 procedure. The same dead loads given by Nowak as Dc_1=70kip-ft, Dc_2=414 kip-ft, D_W=97 kip-ft are used. The HL-93 vehicular load produces a moment = 805 kip-ft while the distributed load gives a moment of 288 kip-ft. The impact factor is 1.33 applied on the truck load effect and the distribution factor for two lanes loaded is 0.678 of the total one lane live load effect. For an Inventory Rating Factor R.F.=1.0, Eq. (2.43) is applied with ϕ=1.0, γ_D=1.25 for component weights and γ_D=1.50 for the wearing surface. For the live load the load factor is γ_L=1.75. The nominal resistance R_n can be calculated as R_n= 2362 kip-ft from:

$$\begin{aligned}\phi R_n &= \gamma_D\left(D_{c1}+D_{c2}\right)+\gamma_D D_W+\gamma_L\left(\left(HL_{93truck}\times IM+HL_{93lane}\right)D.F.\right)\\ &=1.25(70+414)+1.5\times 97+1.75(805\times 1.33+288)\times 0.678=2362\ kip-ft\end{aligned}$$

The mean and standard deviation of the resistance are obtained as:

$$\overline{R}=1.12\times 2362=2645\ kip-ft \qquad \sigma_R=2645\times 10\%=264.5kip-ft$$

With the updated mean live load $\overline{LL}$ =915x1.025 =938 kip-ft, the total mean load and its standard deviation are given as:

$$\overline{S}=\overline{D}+\overline{L}=(72.1+434.7+97)+938=1542\ kip-ft$$

$$\sigma_S=\sqrt{5.8^2+43.5^2+24.3^2+169^2}=176\ kip-ft$$

The application of Eq. (2.7) and (2.9) will produce the following reliability index values:

For Normal Model $\beta=\dfrac{\overline{R}-\overline{S}}{\sqrt{\sigma_R^2+\sigma_S^2}}=\dfrac{2645-1542}{\sqrt{265^2+176^2}}=3.47$

For the LogNormal Model: $\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{2645}{1542}\right)}{\sqrt{0.10^2 + \left(\frac{176}{1542}\right)^2}} = 3.56$

For the case when R is Lognormal and S is Normal and using the Rackwitz-Fiessler algorithm we find β=3.80.

Figure 2.12 provides a plot of the reliability index obtained using the Rackwitz-Fiessler FORM algorithm for simple span composite steel bridges varying in length between 30-ft to 200-ft having beam spacing between 4-ft and 12-ft. The data shows that the reliability indexes produce an average value of 3.71 with a minimum value of 3.40 and a maximum value of 4.28. There is an overall downward trend with increasing span length. However, the differences between the results for beam spacing observed with the LFD have been essentially eliminated based on the assumption that the LRFD load distribution factors are significantly more accurate. The trends observed in Figure 2.12 are similar to those reported by Kulicki et al (2007) and shown in Figure 2.13. It is noted that for these spans and beam spacings, the average reliability index is slightly higher than the target β_{target}=3.5 and even higher than the average β=3.38 obtained from LFD HS-20 bridges. This confirms observations made by other researchers that the LRFD code is slightly more conservative than the LFD code.

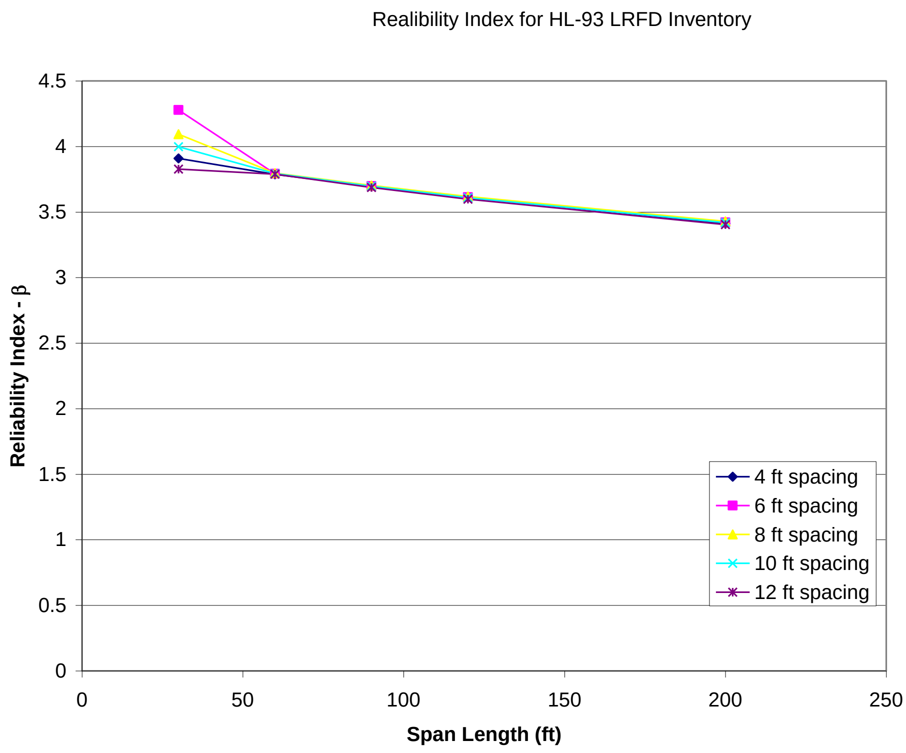


Figure 2.12 Reliability indexes for composite steel bridges in bending for LRFD HL-93.

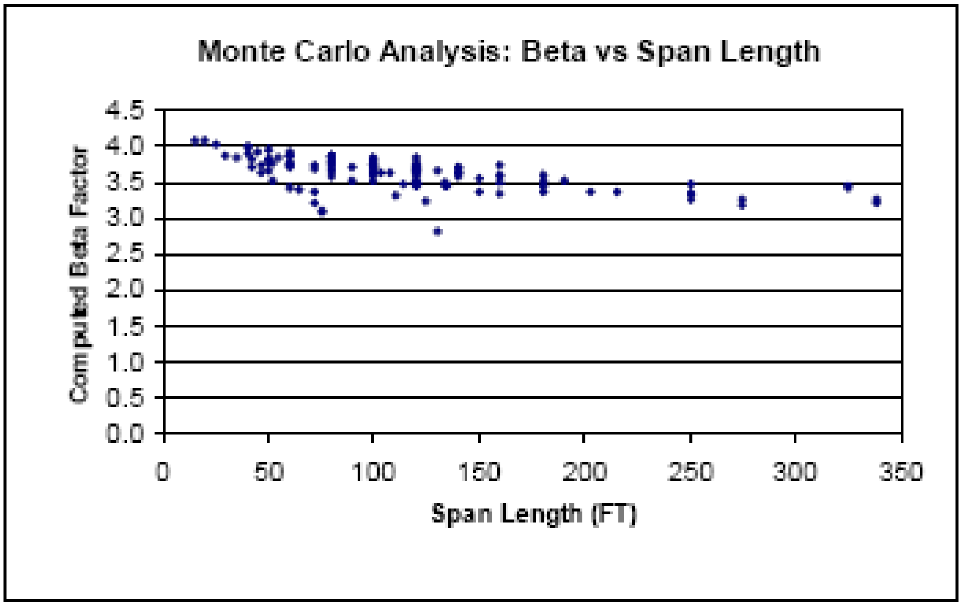


Figure 2.13 Reliability indexes for a representative sample of bridges as reported by Kulicki et al (2007).

Case 3. Evaluation of HS-20 Nominal load and AASHTO Legal Load LFD Operating Rating

During the calibration of the AASHTO LRFR, the target reliability index for bridges with Operating R.F.=1.0 was set at β_{target}=2.5. A 5-year service life period was selected for Operating Rating (Moses, 2001). The Nowak (1999) tables do not provide an approach for finding the maximum live load effects for multi-lanes for the 5-year period. Therefore, the Moses (2001) model will be used in this set of calculations. Note that Tables 2.15 and 2.16 show that the Moses (2001) modeling assumptions generally lead to lower maximum moment effects than those of the AASHTO LRFD. This means that the reliability index values that are calculated in this section will be higher than those that would have been obtained using Nowak's (1999) load data and simulation.

HS-20 Operating Rating

For an LFR Operating Rating Factor R.F.=1.0, the application of Eq. (2.43) for the 60-ft simple span composite steel bridge studied earlier, with ϕ=1.0, γ_D=1.30 and γ_L=1.30 implies that the nominal resistance R_n can be calculated as R_n=1722 kip-ft:

$$\phi R_n = \gamma_D \left(D_{c1} + D_{c2} + D_W\right) + \gamma_L \left(HS_{20} \times IM \times D.F.\right)$$
$$= 1.3(70 + 414 + 97) + 1.30(805 \times 1.27 \times 1.455 / 2) = 1722 \ kip - ft$$

The mean and standard deviation of the resistance are obtained as:

$$\overline{R} = 1.12 \times 1722 = 1929 \ kip - ft \qquad \sigma_R = 1929 \times 10\% = 192.9 kip - ft$$

The Moses (2001) model will produce a maximum mean weight for two side-by-side heavy trucks in the 5-year return period which can be calculated as:

$$W_{5year} = 2\overline{W}_H + t_{5\,side-by-sdie}\sqrt{2}\sigma_H = 2 \times 68 + 4.31 \times \sqrt{2} \times 18 = 246 \ kips$$

Based on $t_{5side\text{-}by\text{-}side}$ =4.31 which is the standard deviate of the probability of exceedance:

$$P_{ex} = \frac{1}{N} = \frac{1}{5 \ years \times 365 \ days / year \times \frac{1000}{15} \ side - by - side \ heavy \ trucks / day} = 82.2 \times 10^{-7}$$

The 246 kip side-by-side vehicles with the 3-S2 configuration will produce a total moment on the bridge =2088 kip-ft. The mean live load on a member is obtained as:

$$\overline{LL} = 2088 \times \overline{IM} \times D.F./2 = 2088 \times 1.10 \times 0.678 / 2 = 779 \ kip - ft$$

The standard deviation of the live load is $\sigma_L = 779 \times 18\% = 140 \ kip - ft$.

The mean and standard deviation of the total loads become:

$$\overline{S} = \overline{D} + \overline{L} = (72.1 + 434.7 + 97) + 779 = 1383 kip - ft$$

$$\sigma_S = \sqrt{5.8^2 + 43.5^2 + 24.3^2 + 140^2} = 148.7 \ kip - ft$$

The application of Eq. (2.7) and (2.9) will produce the following reliability index values:

For Normal Model $\beta = \frac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \frac{1929 - 1383}{\sqrt{192.9^2 + 148.7^2}} = 2.24$

For the LogNormal Model: $\beta = \frac{\ln\left(\frac{\bar{R}}{\bar{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \frac{\ln\left(\frac{1929}{1383}\right)}{\sqrt{0.10^2 + \left(\frac{148.7}{1383}\right)^2}} = 2.27$

For the case when R is Lognormal and S is Normal and using the Rackwitz-Fiessler algorithm we find β=2.34.

When using the FORM algorithm for the whole set of composite steel bridges, the reliability indexes are obtained as shown in Figure 2.14. The average reliability index is obtained as β=2.22 with a minimum value of 0.63 and a maximum of 3.34. This average value is similar to that reported by Moses (2001) and slightly lower than the β_{target}=2.5 that he used for calibrating the AASHTO LRFR Operating rating. This indicates that the AASHTO LRFR was intentionally calibrated to provide a slight level of conservatism when compared to the LFR Operating Ratings with HS-20 nominal loads.

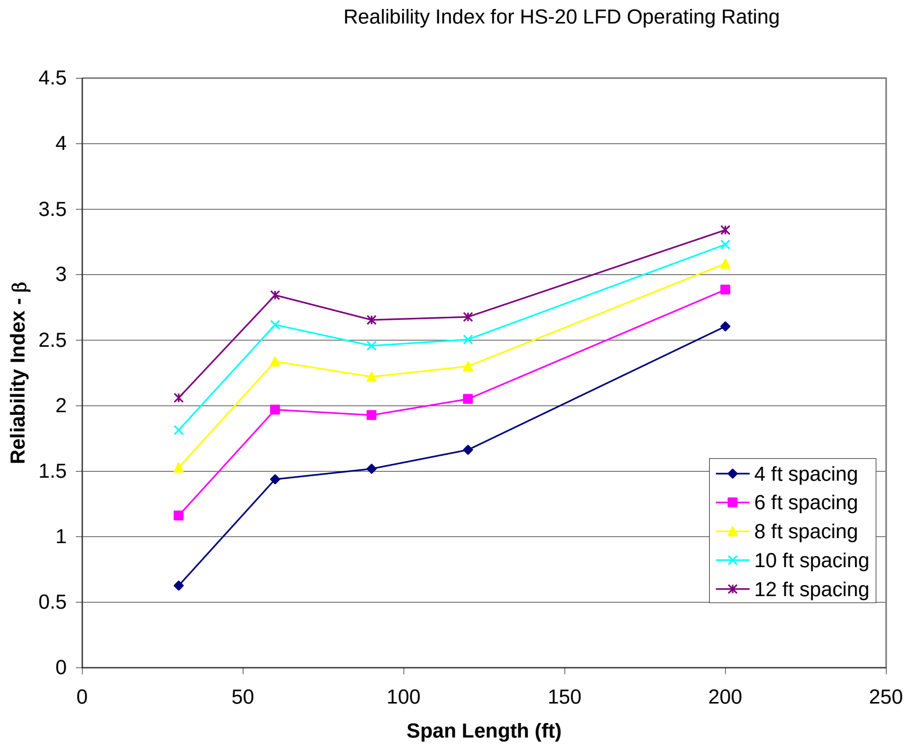


Figure 2.14. Reliability indexes for composite steel bridges in bending for LFD HS-20 Operating Ratings.

H-20 Operating Rating

If instead of using the HS-20 vehicle, the H-20 AASHTO vehicle is used, the moment effect of the H-20 is 544 kip-ft, while the lane load is 558 kip-ft then R_n is changed to 1426 kip-ft obtained from:

$$\phi R_n = \gamma_D \left(D_{c1} + D_{c2} + D_W\right) + \gamma_L \left(H_{20} \times IM \times D.F.\right)$$
$$= 1.3(70 + 414 + 97) + 1.30(558 \times 1.27 \times 1.455 / 2) = 1426\ kip - ft$$

The mean and standard deviation of the resistance are obtained as:

$$\overline{R} = 1.12 \times 1426 = 1597\ kip - ft \qquad \sigma_R = 1597 \times 10\% = 159.7 kip - ft$$

The live loads are not changed and the application of Eq. (2.7) and (2.9) will produce the following reliability index values:

For Normal Model $\beta = \dfrac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \dfrac{1597 - 1383}{\sqrt{159.7^2 + 148.7^2}} = 0.98$

For the LogNormal Model: $\beta = \dfrac{\ln\left(\dfrac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \dfrac{\ln\left(\dfrac{1597}{1383}\right)}{\sqrt{0.10^2 + \left(\dfrac{148.7}{1383}\right)^2}} = 0.98$

A reliability index value β=0.82 is obtained for the LogNormal-Normal model.

For the whole set of composite steel bridges, the average reliability index is 1.62 with a minimum value of 0.07 and a maximum of 3.34 as shown in Figure 2.15. Notice that for the longer spans, the H-20 and HS-20 ratings produce the same results because in both cases the HS lane loading governs.

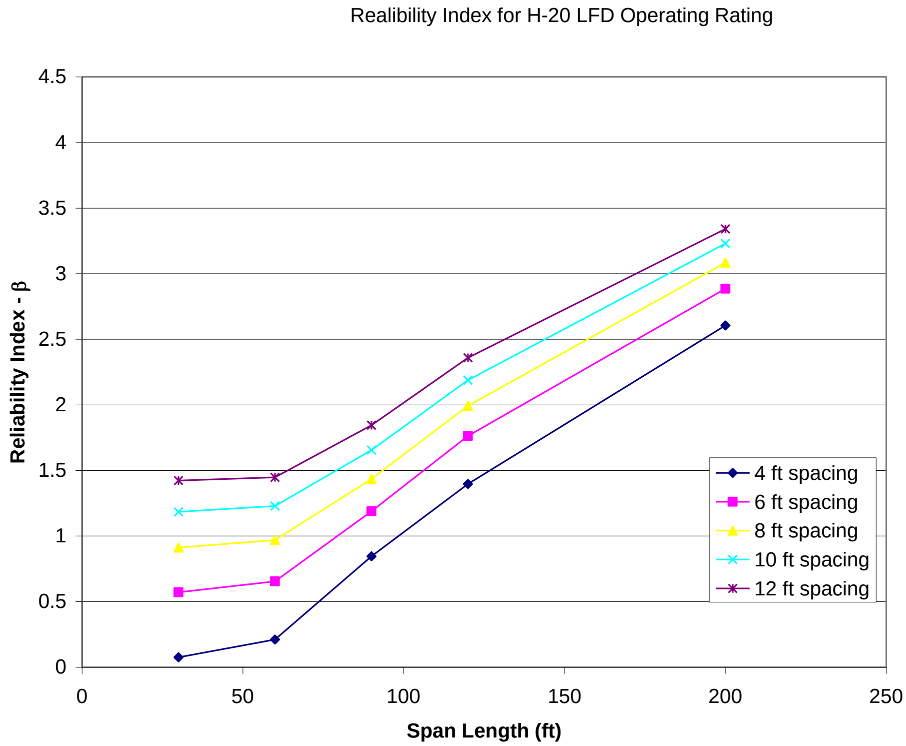


Figure 2.15. Reliability indexes for composite steel bridges in bending for LFD H-20 Operating Ratings.

Legal Load Rating

If the nominal resistance, R_n, is calculated based on the maximum effect of the three AASHTO Legal trucks, the average reliability index becomes 1.59 with a minimum value of -0.24 and a maximum value of 2.89. These values which are shown in Figure 2.16 are lower than those obtained for the HS-20 loads since the load effects of the HS-20 trucks and lane loads are higher than those of the AASHTO Legal trucks which indicates that the HS-20 Operating Ratings are more conservative than the Legal Load Ratings and thus the reliability index values for the HS-20 ratings are higher than those of the Legal Trucks.

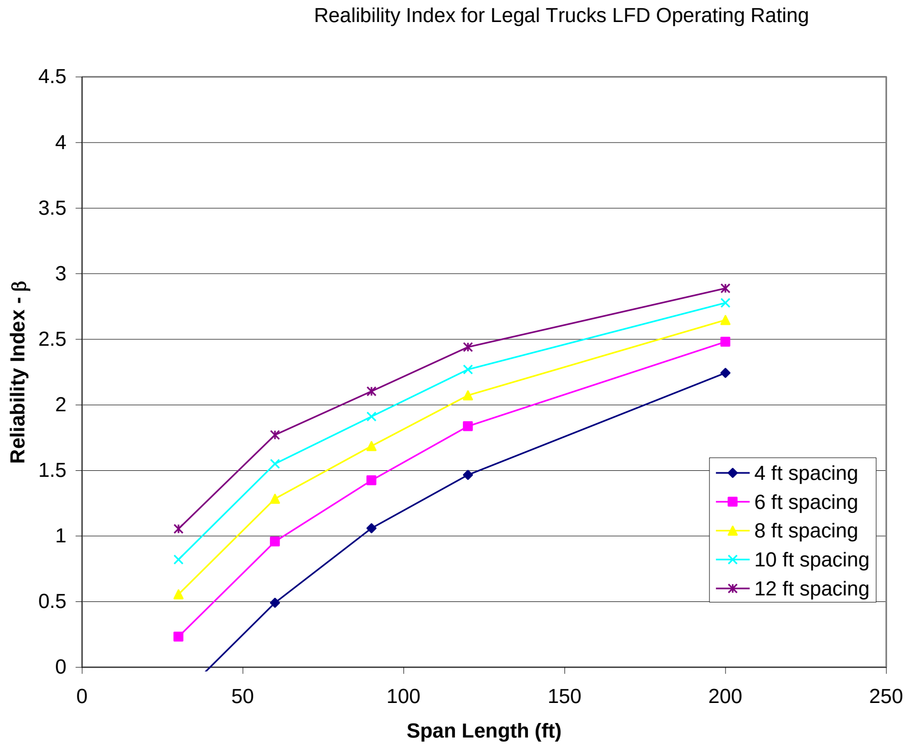


Figure 2.16 Reliability indexes for composite steel bridges in bending for LFD Legal Truck Operating Rating.

Case 4. Evaluation of HL-93 and Legal Load AASHTO LRFR Operating Rating.

The objective of the analysis performed in this case is to determine how closely did the AASHTO LRFR live load factors lead to reliability levels that meet the target reliability index β_{target}=2.5 as set by Moses (2001). In the first step the AASHTO LRFR Operating Rating for HL-93 design load with a live load factor γ_L=1.35 is used.

The simple analysis example is performed for the 60-ft simple span composite steel bridge with girders at 8-ft spacing. In this case, the nominal resistance associated with a R.F.=1 will be equal to R_n=1994 kip-ft calculated as shown below.

$$\phi R_n = \gamma_D (D_{c1} + D_{c2}) + \gamma_D D_W + \gamma_L ((HL_{93truck} \times IM + HL_{93lane}) D.F.)$$
$$= 1.25(70 + 414) + 1.5 \times 97 + 1.35(805 \times 1.33 + 288) \times 0.678 = 1994\ kip - ft$$

The mean dead and live loads as well as their standard deviations remain the same as calculated above. The mean and standard deviation of the resistance are obtained as:

$$\overline{R} = 1.12 \times 1994 = 2233 \; kip - ft \qquad \sigma_R = 2233 \times 10\% = 222.3 \; kip - ft$$

The reliability indexes for the three models become

For Normal Model $\beta = \dfrac{\overline{R} - \overline{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \dfrac{2233 - 1383}{\sqrt{222.3^2 + 148.7^2}} = 3.18$

For the LogNormal Model: $\beta = \dfrac{\ln\left(\dfrac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2 + V_S^2}} = \dfrac{\ln\left(\dfrac{2233}{1383}\right)}{\sqrt{0.10^2 + \left(\dfrac{148.7}{1383}\right)^2}} = 3.26$

For the case when R is Lognormal and S is Normal and using the Rackwitz-Fiessler algorithm we find β= 3.43.

As shown in Figure 2.17, the average reliability index for the whole set of composite steel bridges is β=3.29 with a minimum value of 2.88 and a maximum of 3.56. This indicates that the AASHTO LRFR Operating Rating with γ_L=1.35 and the HL-93 loading did overshoot the pre-set target β_{target}=2.5. This is why the AASHTO LRFR suggests using the Operating Ratings with the HL-93 design load only for screening purposes and that final decisions on postings should be made based on the Legal Truck Ratings.

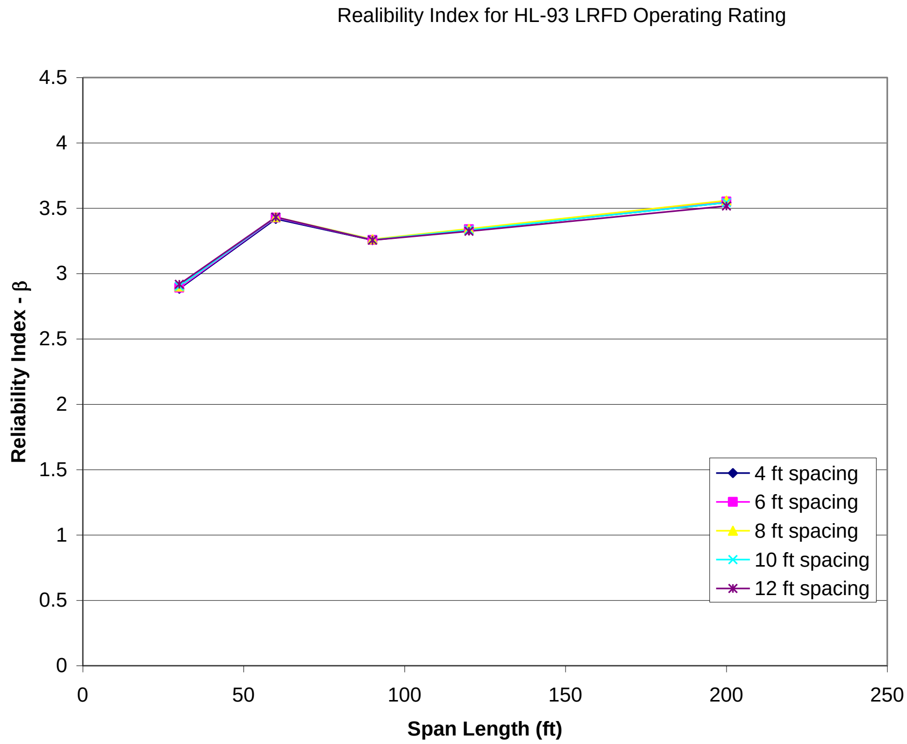


Figure 2.17. Reliability indexes for composite steel bridges in bending for LRFR HL-93 Operating Rating.

AASHTO LRFR Legal Load Rating

For the AASHTO LRFR Operating Rating with the AASHTO Legal Loads, the live load factors are functions of the bridge site's ADTT. Table 2.21 gives the Live load mean moments for different ADTT values and different span lengths. The live load factors are γ_L=1.80 for ADTT$\geq$ 5000, γ_L=1.65 for ADTT=1000, and γ_L=1.40 for ADTT$\leq$100. As explained above, in the case of ADTT=5000 Moses (2001) assumes that 1000 of these trucks are heavy with a mean weight of 68 kips and a standard deviation of 18 kips and that 1/15 of these heavy trucks will be side-by-side. In the case of sites with ADTT=1000, Moses (2001) again assumes that 1/5 of these trucks i.e. 2000 will be heavy with only 1% of the heavy trucks being side-by-side. The lower percentage of side-by-side events is consistent with the data collected by Sivakumar et al (2008) although the actual value of 1% is still conservative. Following the same logic, sites with ADTT=100 are assumed to have 20 heavy trucks per day with 0.1% of the trucks being side-by-side. Using these assumptions and calculating the probability of exceedances and the corresponding standard deviates for a return period of 5 years, Moses (2001) obtains the maximum expected live load moments on the simple span shown in Table 2.21.

Table 2.21 AASHTO LRFR maximum moment effects in kip-ft for different ADTT assuming a five-year rating period.

Moses (2001) model		Single Lane			Two Lanes		
ADTT		5000	1000	100	5000	1000	100
Prob. of exceedance		5.48E-07	2.74E-06	2.74E-05	8.23E-06	0.00027	0.0274
		t_{5years}=4.87	t_{5years}=4.55	t_{5years}=4.03	t_{5years}=4.31	t_{5years}=3.46	t_{5years}=1.92
Span	Legal 3-S2 moment (kip-ft)	Max. moment (kip-ft)	Max. moment (kip-ft)	Max. moment (kip-ft)	Max. moment (kip-ft)	Max. moment (kip-ft)	Max. moment (kip-ft)
30 ft	222	479	461	433	756	689	569
60 ft	612	1324	1273	1195	2088	1904	1572
90 ft	1142	2470	2376	2230	3897	3553	2933
120 ft	1682	3638	3500	3285	5739	5232	4319
200 ft	3122	6752	6496	6097	10653	9712	8017

To illustrate the reliability calculation process we consider the same 60-ft span bridge configuration studied above. In this case, the maximum legal load moment for the 60-ft simple span is 612 kip-ft due to the 3S-2 truck, leading to R_n=1744 kip-ft

$$\begin{aligned}\phi R_n &= \gamma_D(D_1 + D_2) + \gamma_D D_3 + \gamma_L(L_{3S2} \times IM \times D.F.) \\ &= 1.25(70+414) + 1.5\times 97 + 1.80(612\times 1.33)\times 0.678 = 1744\ kip-ft\end{aligned}$$

The mean dead and live loads as well as their standard deviations remain the same as those calculated above. The mean and standard deviation of the resistance are obtained as:

$$\overline{R} = 1.12 \times 1774 = 1953\ kip-ft \qquad \sigma_R = 1953 \times 10\% = 195.3\ kip-ft$$

The reliability indexes for the three models become

For Normal Model $\beta = \dfrac{\overline{R}-\overline{S}}{\sqrt{\sigma_R^2+\sigma_S^2}} = \dfrac{1953-1383}{\sqrt{195.3^2+148.7^2}} = 2.32$

For the LogNormal Model: $\beta = \dfrac{\ln\left(\dfrac{\overline{R}}{\overline{S}}\right)}{\sqrt{V_R^2+V_S^2}} = \dfrac{\ln\left(\dfrac{1953}{1383}\right)}{\sqrt{0.10^2+\left(\dfrac{148.7}{1383}\right)^2}} = 2.35$

For the case when R is Lognormal and S is Normal and using the Rackwitz-Fiessler algorithm we find β=2.43 .

The average reliability index for the whole set of simple span composite steel bridges is obtained as 2.59 with a minimum of 2.27 and a maximum of 2.93. Figure 2.18 shows a plot of the reliability index versus span length. The results obtained herein indicate that the AASHTO LRFR Operating Rating with γ_L=1.80 and the legal truck loading did meet the pre-set target β_{target}=2.5 with a reasonably small range between the minimum and maximum values.

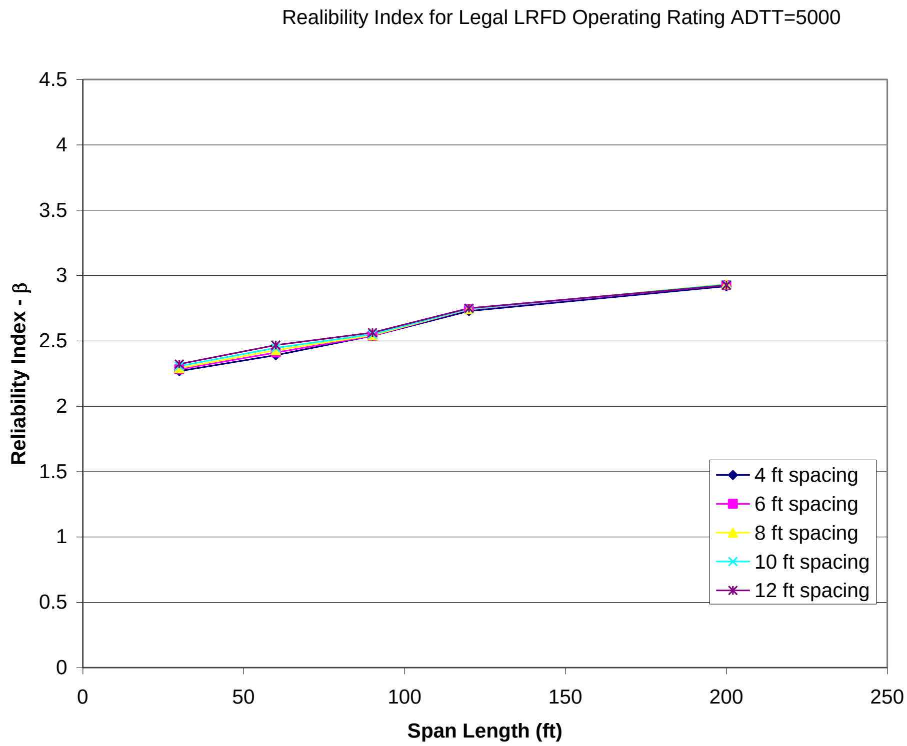


Figure 2.18 Reliability indexes for composite steel bridges in bending for LRFR Legal Truck Operating Rating for ADTT=5000.

Using γ_L=1.65 with the maximum load for sites with ADTT=1000 yields reliability index values of 2.37 for the Normal case, 2.41 for the LogNormal and 2.48 for LogNormal-Normal model of the 60-ft bridge with beams at 8 ft. In this case, when the whole set of composite steel bridges is analyzed for the Lognormal-Normal model, the average β is 2.63 with a minimum of 2.33 and a maximum of 2.95. This indicates that for the ADTT=1000, the AASHTO LRFR is producing an average reliability index for the composite steel bridges slightly higher than the target β of 2.5. Figure 2.19 shows how the reliability index varies with span length.

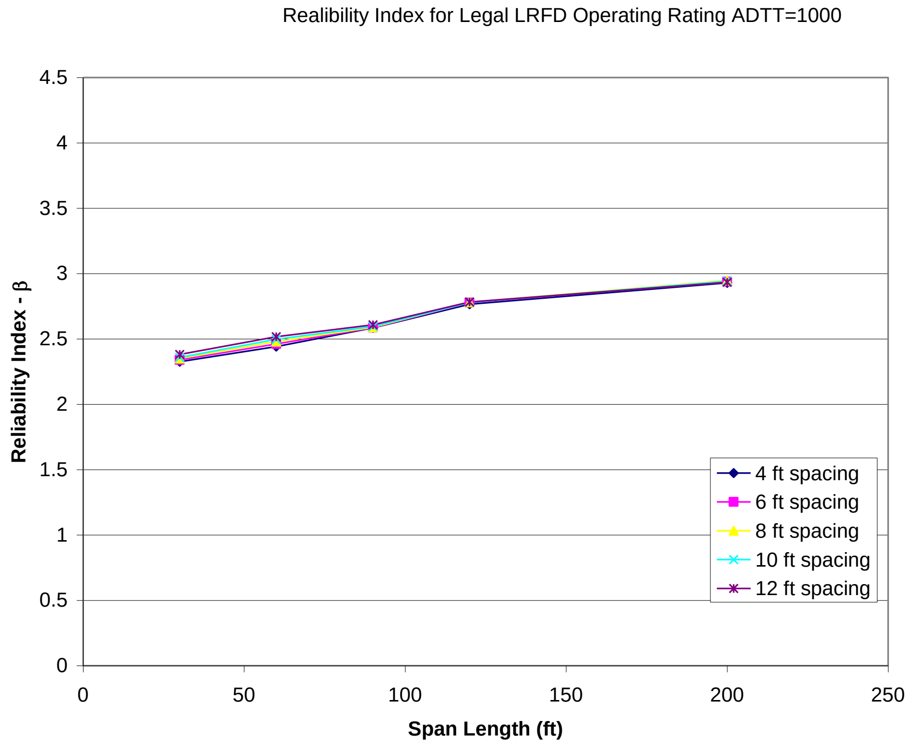


Figure 2.19. Reliability indexes for composite steel bridges in bending for LRFR Legal Truck Operating Rating for ADTT=1000.

Using γ_L=1.40 with the maximum load for sites with ADTT=100, the 60-ft steel bridge with beams at 8-ft yields reliability index values of 2.48 for the Normal case, 2.56 for the LogNormal and 2.62 for LogNormal-Normal for two lanes loaded. In this case, the average of all the simple span composite steel bridges is β=2.75 with a minimum of 2.50 and a maximum of 2.99. This case which is illustrated in Figure 2.20 shows that the target beta of 2.5 was largely exceeded.

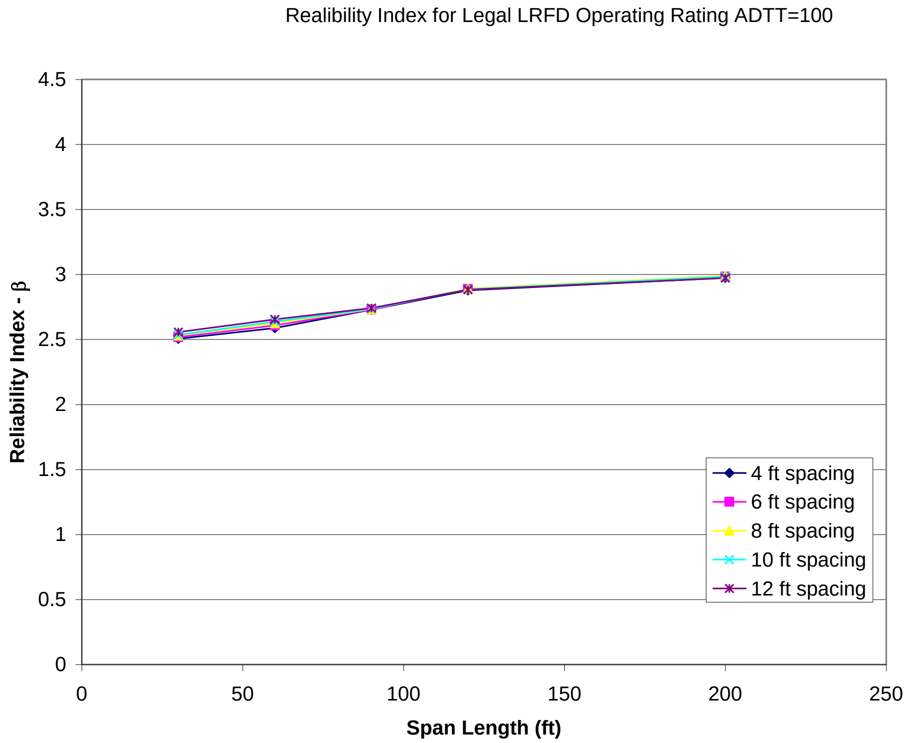


Figure 2.20. Reliability indexes for composite steel bridges in bending for LRFR Legal Truck Operating Rating for ADTT=100.

Summary

In summary, the reliability analysis performed in this section demonstrates that in all the cases considered, the AASHTO LRFD and the AASHTO LRFR have conservatively achieved reliability index values higher than the pre-set target levels. In addition, the pre-set target reliability index values were also conservatively selected to be higher than those obtained from the AASHTO LFD and LFR ratings with HS-20 loadings. This demonstrates the general conservativeness of the AASHTO LRFD and LRFR compared to the AASHTO LFD and LFR although for individual cases, it is possible to find that the AASHTO LRFD and LRFR may produce lower reliability index values than the LFD and LFR.

It is noted that these calculations are based on generic models of the live load which is not representative of the loading conditions on New York State bridges. In the next Chapter, the live load models obtained from the New York WIM data will be used to study the reliability of New York State bridges.

2.7 Conclusions

This Chapter reviewed the calibration procedures of the AASHTO LRFD and the AASHTO LRFR codes. The review was performed to ensure that the data base and the methodology being followed in this study are consistent with those followed by the AASHTO calibration teams. The review showed that the AASHTO LRFD did slightly overshoot the pre-set target value β=3.5 even when using the more conservative live load models described by Kulicki et al (2007) as compared to the original live load models provided by Nowak (1999). The review also showed that the AASHTO LRFR live load factor γ_L=1.35 for Operating Rating associated with the HL-93 design load would provide highly conservative results and higher reliability levels than those observed with LFD HS-20 Operating Ratings. When using the Moses (2001) data base, the LFD HS-20 Operating Rating yielded a reliability index beta close to β=2.20 as predicted by Moses (2001) who chose to conservatively use β=2.5 as a target for his AASHTO LRFR calibration effort. However, when using the Legal Trucks with the AASHTO LFD and the Moses (2001) data base, the average reliability index becomes β=1.59. Finally, when reviewing the AASHTO LRFR live load factor for the Legal trucks, it was observed that when using the Moses (2001) data base, the γ_L==1.80 associated with bridges with 5000 ADTT did meet the target beta of 2.5. Similarly, the γ_L=1.65 for 1000 ADTT bridges was slightly higher than the 2.5 target beta. The case for bridges with ADTT=100 produced an average reliability β=2.75 overshooting the target index.

The live load data bases used by Moses (2001) and Nowak (1999) were primarily obtained from a set of truck weights collected in the 1970's in Ontario Canada along with side-by-side truck occurrence statistics collected on one severe site in Michigan. These data bases were found to be inconsistent with Weigh-In-Motion data collected at several representative sites in New York State. In particular, the WIM data showed that: a) the upper 20% of the truck weights do not follow a Normal Probability distribution, b) the weights of the upper 20% the truck population has a mean value of 91 kips rather than the 68 kips assumed by Moses (2001) and the standard deviation of the upper 20% of the trucks is closer to 15 kips rather than the 18 kips used by Moses (2001). Also, the New York WIM data show that the percentage of side-by-side truck occurrences is close to 2% of the total number of truck passages rather than the 6.67% of the passages of the heavy trucks as used by Nowak (1999) and Moses (2001) for the sites with ADTT$\geq$5000.

To correct for such differences between state-specific data and the generic database, the AASHTO LRFR provides a method to adjust the live load factors. However, the AASHTO LRFR live load factor adjustment method presumes that the target reliability index β=2.5 is to be maintained and that the upper 20% of the weights will still follow a Normal distribution. To overcome these limitations, the Chapter proposed a live load modeling approach that uses the full set of WIM data collected at representative sites in the State of New York. The models generated from the procedure show that New York bridges may be subjected to much higher live loads than assumed in the AASHTO LRFR. Therefore, it is recommended that a reliability-based calibration procedure, which follows the same steps of the AASHTO LRFR and LRFD calibration efforts, be applied to extract a target reliability that is consistent with current New York State Department of Transportation Procedures and to propose a NY-LRFR methodology for bridge load rating, permit load evaluation and load posting of deficient bridges. The calibration of the proposed procedures will be presented in Chapter 3.

CHAPTER THREE

RESEARCH IMPLEMENTATION AND FINDINGS

3.1 BACKGROUND

The live load models used during the calibration of the AASHTO LRFD and subsequently during the calibration of the AASHTO LRFR were based on a generic data set whereby the truck weight spectra were assembled from a weighing operation in Ontario Canada in the 1970's that weighed the heaviest trucks observed on one site. The AASHTO LRFD and LRFR calibrations were also based on very limited multiple presence data which was shown to be more severe than what was observed at typical sites throughout the U.S. (Sivakumar, Ghosn Moses, 2008). Recognizing the limitations in the generic data, the commentaries of the AASHTO LRFR allow the adjustment of the specified live load factors based on state-specific and site-specific truck weight data using a methodology developed by Moses (2001). The adjustment process preserves several main assumptions used during the calibration of the AASHTO LRFR which had been specifically made to match the results of the AASHTO LRFD load model. These assumptions are:

- Although the truck weight histogram may follow a bimodal distribution, the heaviest 20% of the truck weights approximately follow a Normal Probability distribution.
- The heaviest 20% of the trucks are used to estimate the maximum live load effects expected in 75 years for the design of new bridges, or 5 years for the load rating of existing bridges.
- The probability of having side-by-side heavy truck loading events is assumed to be P_{sxs}=6.67% (=1./15) for sites with ADTT=5000, P_{sxs}=1% for sites with ADTT=1000, and P_{sxs}= 0.5% for sites with ADTT=100.
- The Coefficient of Variation (COV) for live load effects on a bridge member remains constant at V_{LL}=18% for all loading conditions including random truck traffic as well as loading events involving permit trucks.
- A target reliability index β_{target}=2.5 is used to calibrate the live load factors for legal load rating. This target is close to but slightly higher than the average value obtained from the ASSHTO LFR and AS Operating ratings when using the HS-20 loads.
- The AASHTO LRFR live load factor adjustment procedure for legal load ratings and Permit trucks are meant to preserve the same target β_{target}=2.5

while maintaining the same assumptions on the shape of the truck weight spectrum and truck multiple presence.

- The posting procedure was calibrated for more conservative reliability levels ranging from β=2.5 to β=3.5 for bridges with the lowest ratings.

The Weigh-In-Motion (WIM) truck data collected from each direction of traffic at five different New York sites as part of NCHRP 12-76 were used in Chapter 2 to develop New York State-specific live load models that are more representative of the loading conditions of New York bridges and do not require the simplifying modeling assumptions listed above. In this Chapter, the live load models developed in Chapter 2 are used to calibrate a new set of LRFR live load factors for load rating, load posting and load permitting that reflect the current bridge loads observed on New York State bridges and provide uniform levels of reliability that are consistent with the safety levels that have traditionally been considered adequate by NYSDOT.

3.2 Calibration of Live Load Factors for Legal Load Rating

The purpose of this section is to calibrate NYSDOT LRFR live load factors for the load rating of bridges. The section first reviews the reliability index implied in current NYSDOT procedures and truck loading conditions to extract a target reliability index. New live load factors to be applied with a proposed set of NYSDOT Legal trucks are then calibrated to provide uniform reliability levels. Sensitivity analyses are performed to study how the reliability index values would change under different scenarios.

Review of AASHTO LFR HS-20 Operating Rating

A First Order Reliability Method (FORM) algorithm is used with the resistance, dead load and live load models provided in Chapter 2 to calculate the reliability index of bridges that satisfy the current NYSDOT Operating Rating criteria. For that end, the failure function, Z, is defined as:

$$Z = R - DL - LL \qquad (3.1)$$

Where R is the resistance of a cross section of the most critically loaded bridge member, DL is the dead load effect on that section and LL is the live load effect on the section. R, DL and LL are random variables that can be described using the models presented in Chapter 2. The dead load DL is the combination of the dead load effects of pre-fabricated beams, cast in place dead load and the wearing surface. The live load LL accounts for the effect of the maximum load in a 5-year return period including: a) variability within a site; b) site-to-site variability, and c) the effect of the sample size, also

LL accounts for: d) the dynamic amplification, and e) the load distribution factor. Table 3.1 gives a summary of the statistical data used in this report for the reliability analysis.

Table 3.1 – Summary of statistical data for random variables

Variable		Bias	COV	Distribution type	Source
Bending Resistance	Steel beams	1.12	10%	Lognormal	Nowak (1999)
	Prestressed	1.05	7.5%	Lognormal	Nowak (1999)
	R/Concrete	1.14	13%	Lognormal	Nowak(1999)
Shearing Resistance	Steel beams	1.14	10.5%	Lognormal	Nowak (1999)
	Prestressed	1.15	14%	Lognormal	Nowak (1999)
	R/ Concrete	1.20	15.5%	Lognormal	Nowak (1999)
Dead Loads	Prefabricated	1.03	8%	Normal	Nowak (1999)
	Cast in place	1.05	10%	Normal	Nowak (1999)
	Wearing surface	1.00	25%	Normal	Nowak (1999)
L_{max} single lane moment	ADTT=5000	Table 2.19	4%	Gumbel	This report
	ADTT=1000	Table 2.19	5%	Gumbel	This report
	ADTT=100	Table 2.19	6%	Gumbel	This report
L_{max} two lane moment	ADTT=5000	Table 2.18	5%	Gumbel	This report
	ADTT=1000	Table 2.18	6%	Gumbel	This report
	ADTT=100	Table 2.18	8%	Gumbel	This report
L_{max} two lane shear	ADTT=5000	Table 2.20	5%	Gumbel	This report
	ADTT=1000	Table 2.20	6%	Gumbel	This report
	ADTT=100	Table 2.20	8%	Gumbel	This report
Site to site variability	One lane	1.0	12%	Normal	This report
	Two lanes	1.0	10%	Normal	This report
Data sample size	one lane	1.0	2%	Normal	NCHRP 12-76
	two lanes	1.0	3%	Normal	NCHRP 12-76
Dynamic amplification	One lane	Mean=1.13	9%	Normal	Nowak (1999)
	Two lanes	Mean=1.10	5.5%	Normal	Nowak (1999)
Distribution Factor	Relative to LRFD D.F.	1.0	8%	Normal	Nowak (1999); Ghosn & Moses (1986)

When rating a bridge using the AASHTO LFR Operating Rating, the nominal resistance is obtained from an equation of the form:

$$R.F. = \frac{\phi R_n - 1.30\left(D_{C1} + D_{C2} + D_W\right)}{1.30 L_n} \tag{3.2}$$

Where the rating factor R.F. is set exactly equal to 1.0 for bridges that meet the AASHTO LFR Operating Rating requirements. D_{c1}, D_{c2} and D_W are respectively the pre-fabricated, cast in place and wearing surface dead load effects. L_n is the nominal live load effect of the AASHTO HS-20 load including the LFR load distribution factor and the LFR impact factor. ϕ is the resistance factor which is ϕ=1.0 for steel beams in flexure.

Given the bridge database provided in Table 2.1 and the HS-20 live load effect, Eq. (3.2) can be used to find the nominal resistance R_n when R.F.=1.0. The application of Equations (2.20) and (2.21) will provide the statistical information on the resistance R which as explained in Chapter 2 can be reasonably well represented by a lognormal probability distribution.

Using Eq. (2.18), the statistical properties of the dead load components can be obtained. Following Nowak (1999), the dead load components are assumed to be well represented by normal probability distributions.

For two-lane bridges, the live load effect is represented by Eq. (2.31) and the appropriate COV's for each of the variables in that expression is obtained as described in Section 2.4. In this set of calculations and since the AASHTO LFR uses the same criteria independent of the site's ADTT, and following the recommendation of Moses (2001), we use the mean maximum live load effect for ADTT=5000 based on the New York WIM data as presented in Table 2.10. The summary of all the random variable components of the live load effects on one member are listed in Table 3.1.

Equation (2.3) is used to define the margin of safety and the FORM algorithm provides the reliability index β for each bridge having the dead load effects listed in Table 2.1. The reliability index gives a measure of the distance between the mean value of Z and the failure surface. The reliability index β is a function of the mean of Z and the standard deviation of Z, σ_Z. For normal probability distributions the relation between the mean and standard deviation of Z and the reliability index is described as shown in Figure 2.1. The FORM algorithm can accommodate normal as well as non-normal probability distributions.

The results of the reliability analysis show that if the rating of typical composite steel multi-girder bridges was executed using the LFD HS-20 Operating Rating and the bridge members are subjected to the expected 5-year maximum live load for two lanes as projected from the NYSDOT WIM data, the average reliability index is obtained as β=1.47 with a minimum value of -0.35 and a maximum of 2.79 as plotted in Figure 3.1.

These results show that generally speaking the higher live loads observed on New York state bridges lead to an average reliability index considerably lower than expected from the generic live load data that was assumed during the calibration of the AASHTO LRFD and LRFR specifications. Also, the range in the reliability index values is considerable showing acceptable levels of reliability for the long span bridges, while the reliability levels for short span bridges particularly those with narrow girder spacing are very low.

To verify that the results obtained are reasonable, a simplified example for one case is analyzed assuming that the resistance is lognormal and that the total load composed of dead plus live load is also lognormal. By making this assumption, the reliability index can be obtained from the closed-form expression provided in Eq. (2.9). We take as an example the case of a 40-ft span with beams at 6-ft center to center. Given D_{c1}=15 kip-ft, D_{c2}=149 kip-ft and D_W=32 kip-ft and given an HS-20 load effect of 450 kip-ft and impact factor of 1.3 and a distribution factor of 1.09 of the wheel load, the nominal resistance that corresponds to a Rating Factor R.F.=1.0 is R_n=670 kip-ft. The corresponding mean resistance is $\overline{R}$=750 kip-ft with a COV=10%. The mean dead load is obtained as 204 kip-ft and the dead load standard deviation is 17.6 kip-ft. The mean static load effect $L_{max}xHL_{93}$ value is obtained from Table 2.18 as 1558 kip-ft for two lanes. Given an LRFD distribution factor D.F.=0.59 and a mean impact factor $\overline{IM} = 1.10$, the mean live load effect on one member is obtained as $\overline{LL}$=506 kip-ft with a standard deviation σ_L=76.5 kip-ft. Combining the live load with the dead load, the mean total load becomes $\overline{S}$=506 kip-ft with a standard deviation σ_S=78.5 kip-ft or a COV, V_S=11%. Applying these values into Eq. (2.9), the reliability index obtained for this simplified example is β=0.37. The analysis using the FORM algorithm where the resistance and the load random variables are separated with the statistical data shown in Table 3.1 yields a reliability index β=0.25.

This example and the results for all the cases plotted in Figure 3.1 demonstrate that the AASHTO LFR produces low reliability levels for New York state short spans with closely spaced beams. One goal of the reliability calibration that will be performed in this Report will be to provide an NYS-LRFR methodology that provides uniform levels of reliability for the most common bridge span lengths and beam spacings.

In traditional reliability calibration procedures for new codes, the average reliability index of existing codes may be used as a target that the new code should meet. This assumes that there is general satisfaction with the safety levels implied in the current codes. However, in this case, the relatively low average reliability index which is close to 1.5 may be providing a warning sign that the increased intensity of live loads that has been occurring over the years may be increasing the risks to New York state bridges. This may justify the use of a higher target reliability level. Accordingly, and following several discussions with this project's New York State Research Task Group, it is herein recommended to use a reliability β_{target}=2.0 as the target that the new NYSDOT LRFR should achieve. The goal of the calibration process will be that New York state bridges provide reliability index values as close to the target index of 2.0 as possible while remaining above a minimum value of β=1.50. As will be seen in the next sections, achieving these reliability goals will require the implementation of the LRFR equations with a new set of live load factors and a new set of Legal Trucks that will uniformly provide the required safety levels for all the spans and beam spacings of the database.

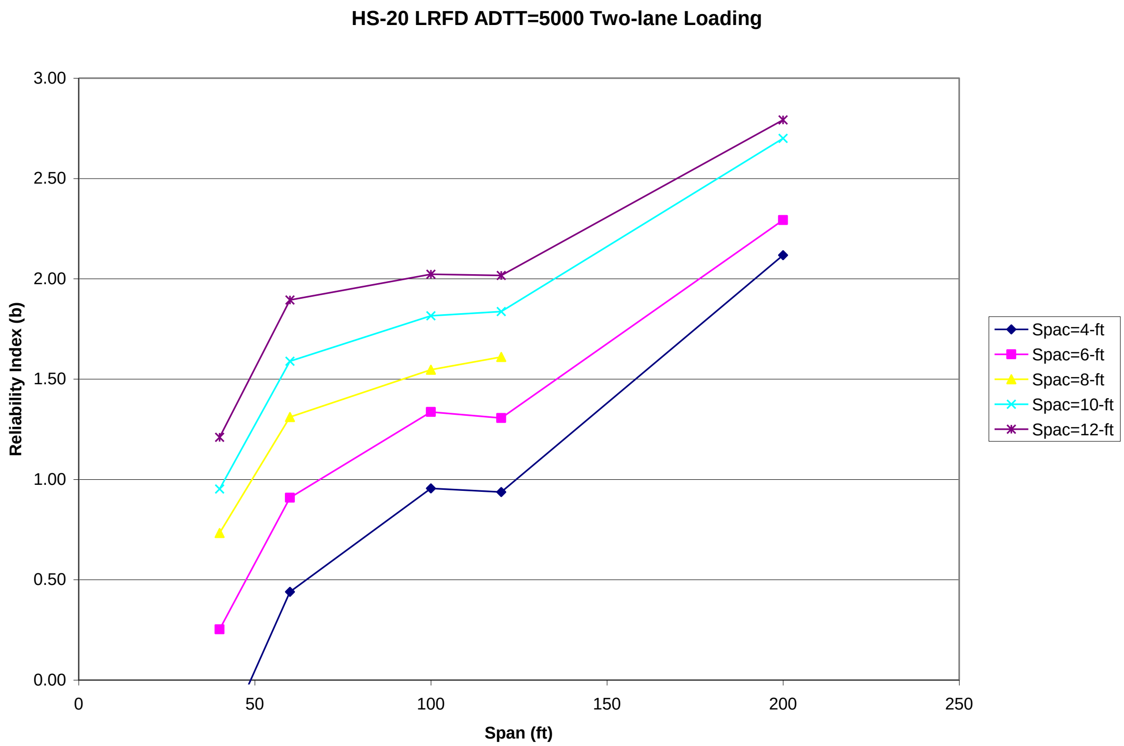


Figure 3.1. Reliability indexes for composite steel bridges in bending for LFD HS-20 Operating Rating for 5-year maximum with ADTT=5000 using NYSDOT WIM data.

Review of AASHTO LRFR Legal Truck Operating Rating

If the AASHTO LRFR legal load operating rating procedure is applied for the moment effects of simple span composite steel bridges, Eq. (3.2) is changed so that the nominal resistance R_n is found from:

$$R.F. = \frac{\phi R_n - 1.25(D_{C1} + D_{C2}) - 1.5(D_W)}{\gamma_L L_n} \qquad (3.3)$$

where the live load factor γ_L=1.80 is applied for sites with ADTT=5000 as specified in the AASHTO LRFR. However, in this case, and following the suggestion of the NYSDOT project panel, the NYSDOT rating will not use the AASHTO Type 3-3 Legal Trucks as this truck type is not usually allowed on New York Interstate bridges. Therefore, L_n, is obtained as the largest of the effects of the AASHTO Type 3 or Type

3S-2 Legal Trucks along with an impact factor of 1.33 and load distribution factors of the AASHTO LRFD.

The mean and the COV of the resistance are then related to the nominal value using Eq. (2.20) and (2.21) and the dead load and live load models are obtained using the data summarized in Table 3.1.

The application of the FORM algorithm leads to the reliability index values for simple span steel bridges plotted in Figure 3.2. The figure shows the reliability index values for the legal truck ratings with 5000 ADTT and the NY state WIM data along with a live load factor γ_L=1.80. In this case, the average reliability index is obtained as 1.72 with a minimum beta of 0.95 and a maximum of 2.28. It is noted that although the γ_L=1.80 produces an average reliability index less than 2.5, it would still provide more conservative ratings than the current LFR with the HS-20 truck, which as illustrated in Figure 3.1, produces an average reliability index $\beta_{average}$=1.47. Also, it is noted that the plots in Figure 3.2 are more uniform than observed in Figure 3.1. However, the reliability index for short spans is significantly below that obtained for the longer spans and falls to a low value of β_{min}=0.95. This lack of uniformity in the reliability index, particularly the low values observed for short spans, reflects an inconsistent level of risk across all the possible span ranges that would require rectification. The rectification can be achieved by using a heavy truck to rate the short span bridges.

Calibration of NYSDOT LRFR Legal Truck Operating Rating

Sites with ADTT=5000

As mentioned above, one goal of this calibration process is to provide an average reliability index of 2.0. A trial and error procedure showed that a live load factor γ_L=1.95 applied on the AASHTO Type 3 and 3-S2 trucks would lead to a reliability index $\beta_{average}$=2.00, but the range of reliability values will vary between β_{min}=1.31 and β_{max}=2.46. This constitutes a large range in the reliability index and also shows that for some cases, the reliability index would fall to a level lower than a β_{min}=1.50.

In order to keep an average reliability index close to 2.0 for all the span lengths considered, it is herein recommended to replace the Type 3 AASHTO Legal Truck by the SU-4 truck that has been found by Sivakumar et al (2007) in NCHRP 12-63 to provide a good envelope for the effect of many short haul trucks while still satisfying the weight limits imposed by the Federal Bridge Formula (FBF). The configuration and axle weights of the SU-4 truck are provided in Figure 3.3. By using the SU-4 in conjunction with the 3S-2 Legal truck, a reliability index average $\beta_{average}$=2.05 is achieved with a lower live load factor γ_L=1.85 while maintaining a more uniform range of reliability such that β_{max}=2.34 and β_{min}=1.81 as shown in Figure 3.4. Table 3.2 gives the live load factors that would be required to achieve target reliability levels β_{target}=2.5, 2.25, 2.0, 1.75, and 1.5.

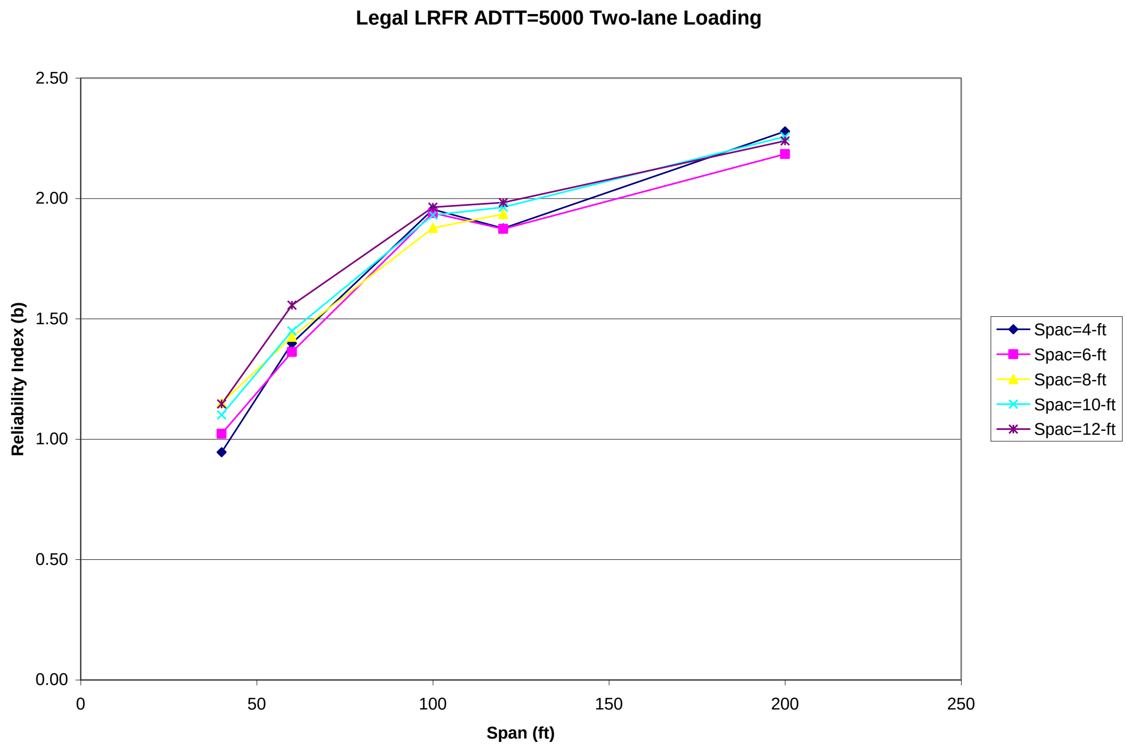
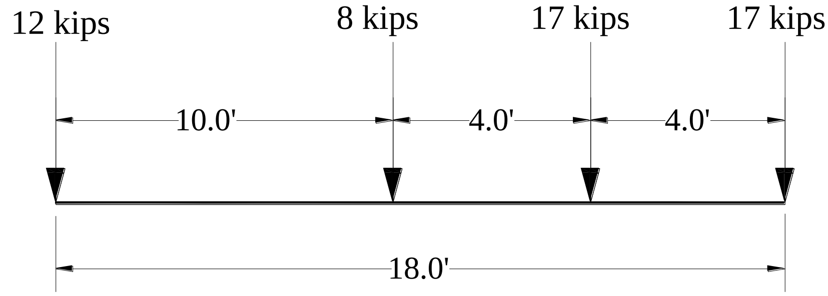


Figure 3.2. Reliability indexes for two-lane composite steel bridges in bending for LRFR AASHTO Legal Truck Operating Rating with for 5-year maximum with γ_L=1.80 and ADTT=5000 using NYSDOT WIM data.

c) SU4 Legal Load (27 tons)



d) Type 3S2 Legal Load (36 tons)

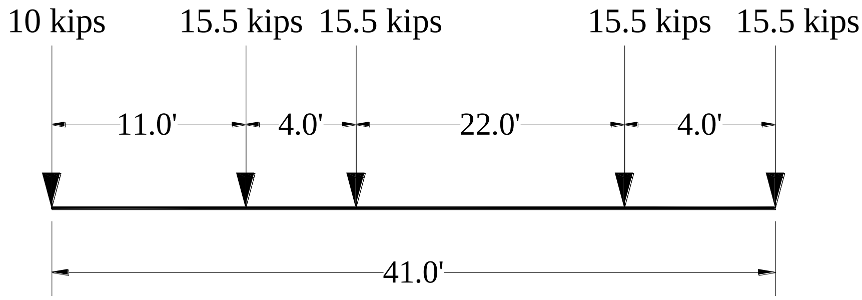


Figure 3.3 Proposed New York State Legal Trucks for bridge rating.

Table 3.2. LRFR Live load factors required to meet different target reliability indexes for two lane bridges where both lanes are loaded.

	AASHTO LRFR	β_{target}=2.5	β_{target}=2.25	β_{target}=2.0	β_{target}=1.75	β_{target}=1.5
	Legal Load Rating Two-lane Bridges ADTT=5000					
Live load factor	1.80	2.10	1.95	1.85	1.70	1.60
Average beta	1.95	2.52	2.24	2.05	1.75	1.55
Minimum beta	1.68	2.39	2.06	1.81	1.42	1.14
Maximum beta	2.28	2.64	2.46	2.34	2.16	2.04
	Legal Load Rating Two-lane Bridges ADTT=1000					
Live load factor	1.65	1.90	1.80	1.65	1.55	1.45
Average beta	1.99	2.49	2.30	1.99	1.79	1.57
Minimum beta	1.73	2.37	2.13	1.73	1.44	1.15
Maximum beta	2.31	2.62	2.50	2.31	2.19	2.06
	Legal Load Rating Two-lane Bridges ADTT=100					
Live load factor	1.40	1.60	1.45	1.35	1.25	1.15
Average beta	2.18	2.53	2.22	2.00	1.77	1.54
Minimum beta	1.66	2.43	2.01	1.71	1.39	1.05
Maximum beta	2.55	2.65	2.46	2.33	2.20	2.07

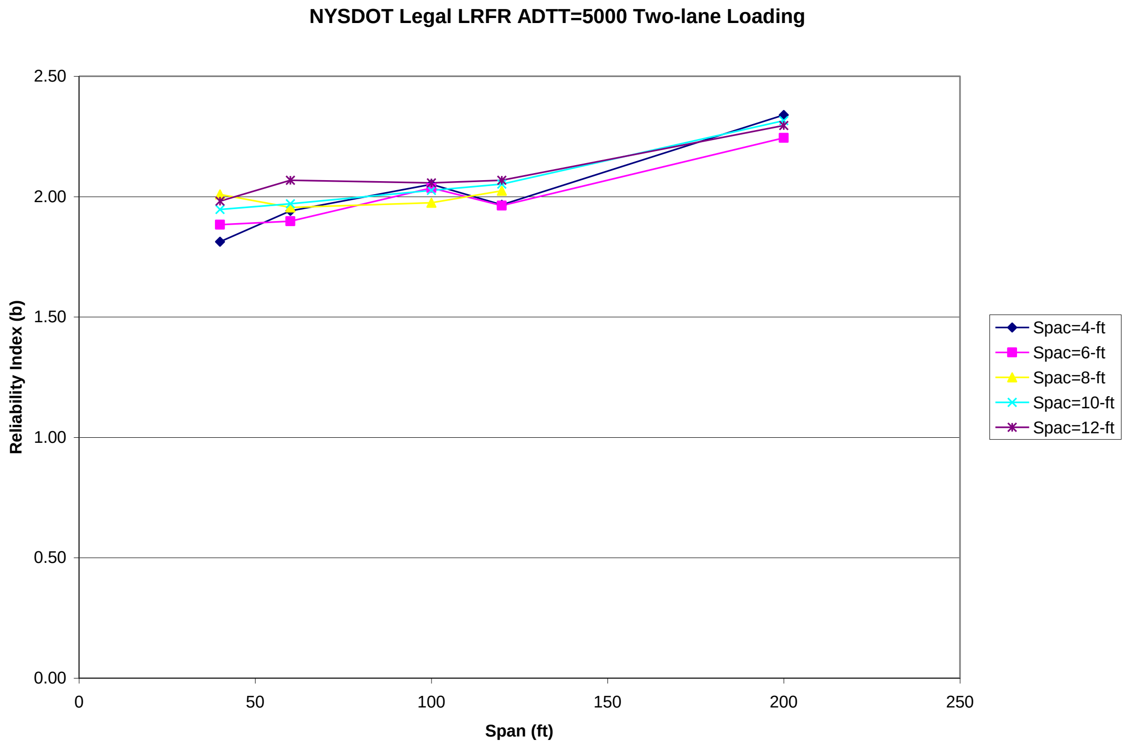


Figure 3.4. Reliability indexes for two-lane composite steel bridges in bending for LRFR NYSDOT Legal Truck Operating Rating with for 5-year maximum with γ_L=1.85 and ADTT=5000 using NYSDOT WIM data.

Reliability Index for Concrete Bridges

According to Nowak (1999), prestressed concrete bridges will have heavier beam weights and will be associated with a resistance bias b_R=1.05 and a COV V_R=7.5% as compared to a bias of 1.12 and a COV of 10% for steel members. If the live load factor γ_L=1.85 is used for sites with ADTT=5000, it is observed that the effect of the differences in the statistical data between prestressed concrete and steel members on the reliability of two-lane simple span bridges is minimal reducing the average reliability index slightly from $\beta_{average}$=2.05 for steel bridges to $\beta_{average}$=1.94 for prestressed girder bridges when using the same live load factor γ_L=1.85. For the prestressed concrete configurations considered, the range of the reliability indexes lies between β_{min}=1.65 and β_{max}=2.32 as shown in Figure 3.5.

For reinforced concrete bridges, Nowak (1999) applies a resistance bias b_R=1.14 and a COV=13%. Also, the AASHTO LRFD uses a strength reduction factor ϕ=0.90 when checking the safety of reinforced concrete bridges in bending. In these calculations, we study the reliability index obtained when using the live load factor γ_L=1.85 with a strength reduction factor ϕ=0.90 when rating two-lane reinforced concrete bridges at 8-ft spacing loaded by the maximum two-lane load. In this case, the average reliability index increases to $\beta_{average}$=2.47 with a minimum value of 2.44 and a maximum of 2.52 as shown in Figure 3.6.

These calculations indicate that the live load factor γ_L=1.85 applied on the SU-4 and Type 3-S2 truck effects will lead to reliability levels that meet the target β_{target}=2.0 while keeping a narrow range for the reliability index values between β_{min}=1.65 and β_{max}=2.52 when considering all the material types.

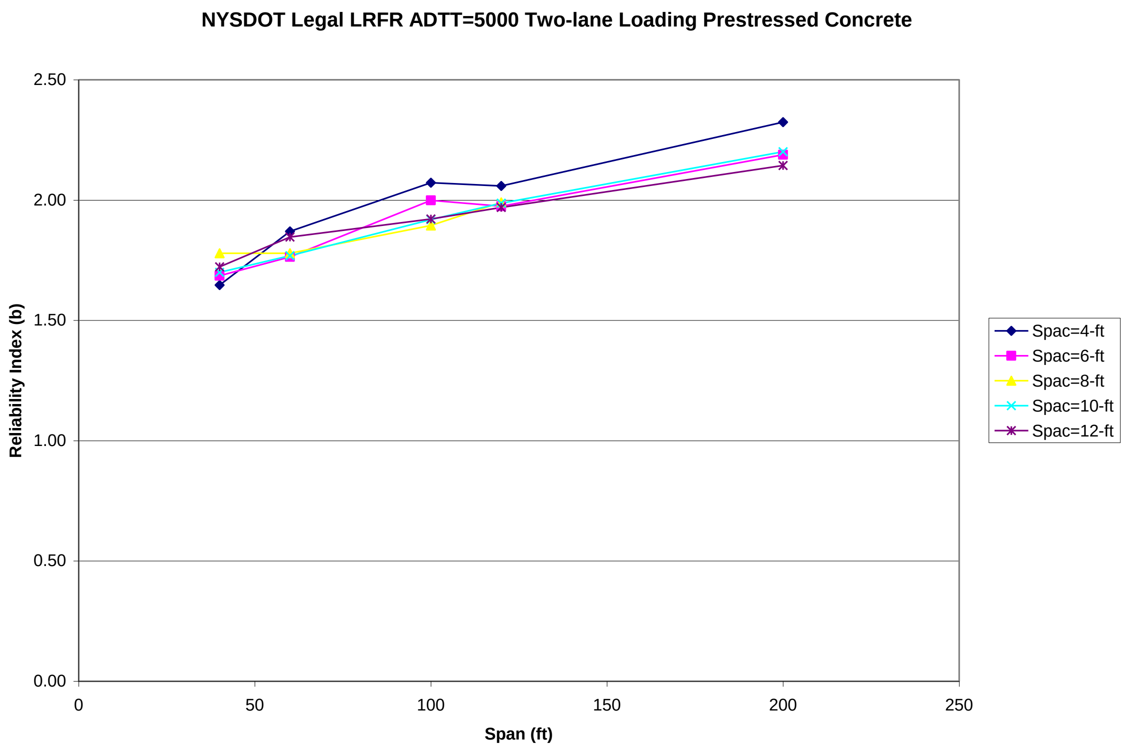


Figure 3.5. Reliability indexes for two-lane prestressed bridges with γ_L=1.85

To verify the results, a simple example is again used assuming a lognormal model. For this case, we consider the 100-ft steel bridge with beams at 8-ft spacing. The dead load effects are given as D_{c1}=386 kip-ft, D_{c2}=1150 kip-ft and D_w=270 kip-ft. For this span length, the 3S-2 truck governs with truck moment effect 1332 kip-ft. The AASHTO LRFD load distribution factor is calculated from Eq. (2.34) to be D.F.=0.62. Using an impact factor =1.33 and the live load factor γ_L=1.85, the nominal resistance is calculated as R_n=4354 kip-ft. The mean resistance becomes $\overline{R}$=4876 kip-ft with a COV=10%. The mean dead load is $\overline{DL}$=1875 kip-ft with a standard deviation σ_{DL}=142 kip-ft. The mean static live load L_{max}xHL_{93} is obtained from Table 2.18 and is equal to 5520 kip-ft. Applying a mean impact factor $\overline{IM}$=1.10 and a mean distribution factor $\overline{D.F.}$=0.62/2, the mean live load on one member is $\overline{LL}$=1882 kip-ft with a live load COV V_{LL}=15% leading to a standard deviation σ_{LL}=282 kip-ft. The total mean load is then $\overline{S}$=3757 kip-ft and the standard deviation is σ_S=316 kip-ft for a COV V_S=8.4%. Using Eq. (2.9), the reliability index for the lognormal model is obtained as β=1.99. The FORM algorithm which uses as input the values as well as the probability distributions in Table 3.1 yields a reliability index β=1.97 for this case.

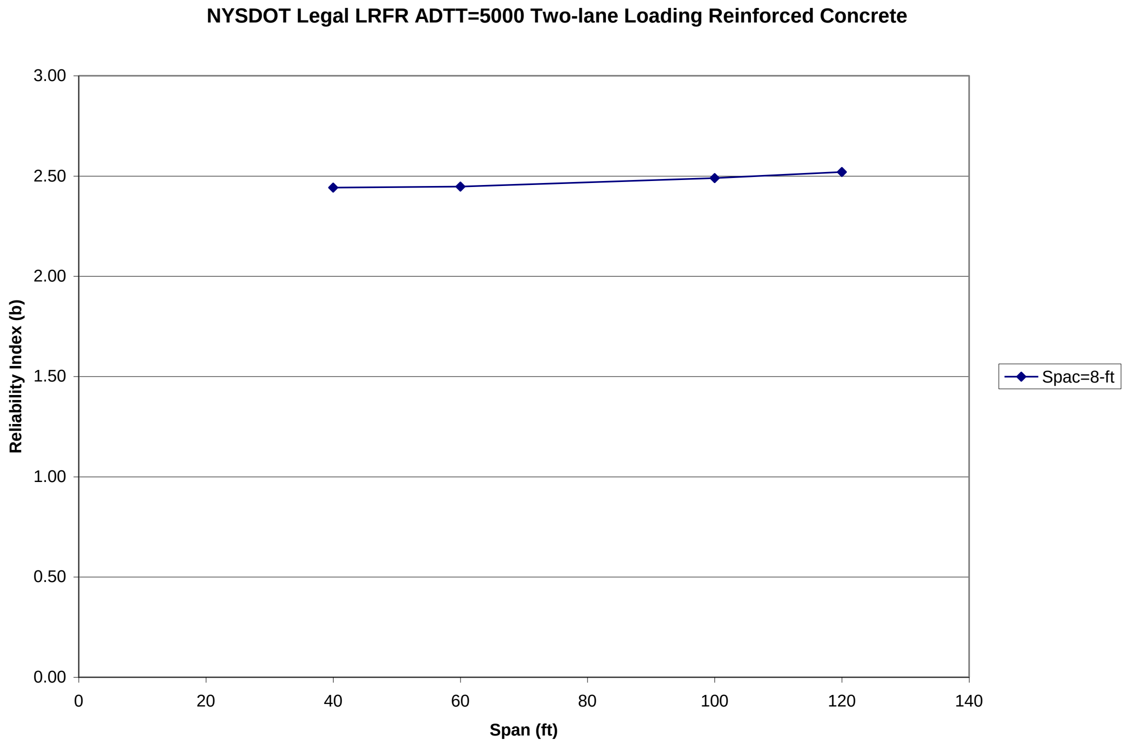


Figure 3.6. Reliability indexes for two-lane reinforced concrete bridges with γ_L=1.85

Effect of 10-year Rating Period

To study how the reliability index changes if the rating period is changed from 5 years to 10 years, the L_{max} values used to model the live load are those taken from Table 2.11. The reliability calculations for the simple span composite steel bridges are repeated with the same live load factor γ_L=1.85 applied on the SU-4 and 3S-2 trucks used as the rating trucks. In this case, the average reliability decreases slightly to $\beta_{average}$=1.95 while maintaining a narrow range of reliability that remains above 1.50 such that β_{max}=2.28 and β_{min}=1.71. The small effect observed when we increase the return period from 5 years to 10 years indicates that using the 10-year return period is not necessary and will not be used in the rest of the report.

Reliability Index for Shear

The previous calculations considered flexural bending as the primary mode of failure. To investigate how the reliability index would change for shear, the same analysis is repeated for the steel bridges with beams at 8-ft spacing assuming a two-lane loading with a 5-year return period. When using the live load factor γ_L=1.85, the reliability index drops to an average value $\beta_{average}$=1.72 with a minimum value of β_{min}=1.48 and a maximum of β_{max}=2.01. The average can be raised up to $\beta_{average}$ 2.08 with a minimum of β_{min}=1.81 and maximum value of β_{max}=2.42 as shown in Figure 3.7 if we apply a strength reduction factor ϕ=0.95 in combination with the live load factor γ_L=1.85. It is emphasized that this recommendation is based on the bias and COV values provided in the calibration report by Nowak (1999) based on earlier studies. Since that time, the AASHTO LRFD has implemented advanced shear analysis and design models for steel and concrete girders and it is not clear whether the biases and COV provided by Nowak (1999) apply to these new models.

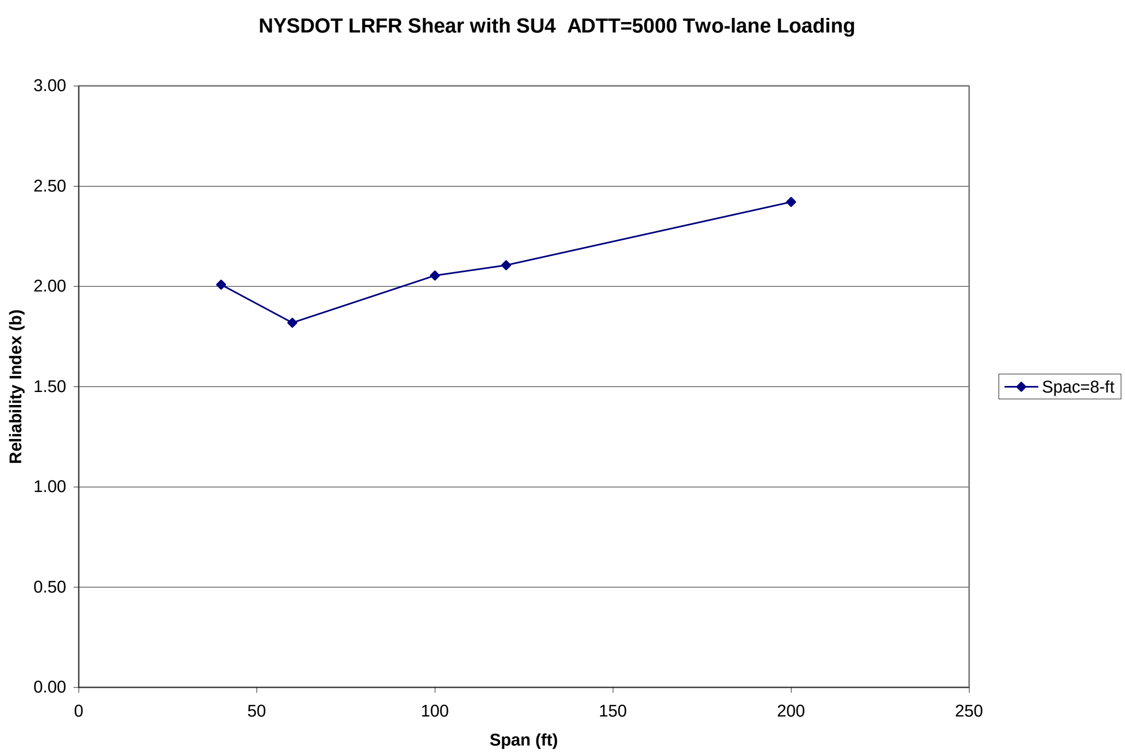


Figure 3.7. Reliability indexes for two-lane composite steel bridges in shear for LRFR NYSDOT Legal Truck Operating Rating with for 5-year maximum with γ_L=1.85 and ϕ=0.95.

Reliability Index for One-lane Bridges

The rating of one-lane bridges is first checked using Eq. (3.3) with the same live load factor γ_L=1.85 but where the nominal live load L_n is associated with the one-lane Load Distribution Factor provided in the AAHTO LRFD while keeping the multiple presence factor MP=1.2 already embedded in the LFRD formula.

The reliability index calculations are applied with the 5-year one-lane L_{max} values of Table 2.12 applied in conjunction with the mean impact factor and the AASHTO LRFD and the other modeling and site to site variables as shown in Eq. (3.32) and in Table 3.1. In this case, with γ_L=1.85, the reliability index drops to an average $\beta_{average}$=0.91 and a minimum value β_{min}=0.29 and a maximum value β_{max}=1.80. Such a large drop in the reliability index value for the one-lane case as compared to the two-lane case is due to the large number of overweight vehicles observed on New York state bridges as compared to what was observed from the generic data used in the AASHTO LRFR and LRFD calibrations coupled with the lower side-by-side probabilities.

Therefore, in order to maintain the target beta β_{target}=2.0, the live load factor for one-lane bridges will have to be raised to γ_L=2.65. This will lead to an average reliability index $\beta_{average}$=2.03 with a minimum value β_{min}=1.81 and a maximum value β_{max}=2.41 as shown in Figure 3.8. Table 3.3 gives the live load factors that would be required to achieve target reliability levels β_{target}=2.5, 2.25, 2.0, 1.75, and 1.5 for one lane bridges.

To verify the results, a simple example is used assuming a lognormal model. For this case, we consider the 100-ft bridge with beams at 8-ft spacing. The dead load effects are given as D_{c1}=386 kip-ft, D_{c2}=1150 kip-ft and D_w=270 kip-ft. For this span length, the 3S-2 truck governs with truck moment effect 1332 kip-ft. The one-lane AASAHTO LRFD load distribution factor is calculated from Eq. (2.33) to be D.F.=0.435. Using an impact factor =1.33 and the live load factor γ_L=2.65, the nominal resistance is calculated as R_n=4367 kip-ft. The mean resistance becomes $\overline{R}$=4891 kip-ft with a COV=10%. The mean dead load is $\overline{DL}$=1875 kip-ft with a standard deviation σ_{DL}=142 kip-ft. The mean live load is obtained from $L_{max}xHL_{93}$ from Table 2.19 which is equal to 4500 kip-ft multiplied by a mean impact factor $\overline{IM}$=1.13 and mean distribution factor $\overline{D.F.}$=0.36 which give a mean live load $\overline{LL}$=1831 kip-ft with a live load COV V_{LL}=18% leading to a standard deviation σ_{LL}=330 kip-ft. The total mean load is then $\overline{S}$=3706 kip-ft and the standard deviation is σ_S=359 kip-ft for a COV V_S=9.7%. Using Eq. (2.9), the reliability index for the lognormal model is obtained as β=2.0. The FORM algorithm which uses as input the statistical values in Table 3.1 along with the probability distributions shown in that Table yields a reliability index β=1.99 for this case.

The assumption made here is that the data collected on two-lane WIM sites also describes the truck loads that will cross single lane bridges. The higher live load factor is necessary to compensate for the low two-lane to one-lane maximum load ratio. In fact, Tables 2.10

and 2.12 show that the maximum two lane load is on the average 1.23 times the maximum one-lane load. This value is close to the multiple presence factor implied in the LRFD load distribution factors. However, it is assumed that the two-lane load is equally spread to the two lanes of a bridge. Thus, the maximum applied two-lane load would produce about the same load effect on the girder as the maximum one-lane load. However, the mean two-lane distribution factor used during the rating process is on the average 1.68 times the distribution factor for one lane after removing the multiple presence factor or 1.40 times the one lane distribution factor if the multiple presence factor is kept as is. Thus, it would be necessary to use a live load factor for one lane bridges on the order of 1.36 (1.36=1.68/1.23) times that used for the two-lane bridges to keep the same level of reliability. The ratio of the live load factors obtained in this calibration is 1.43 (1.43=2.65/1.85) which is close to the 1.36 value. The 5% difference between the 1.43 and 1.36 is due to the slight differences in the one-lane versus two-lane live load biases and COV's as shown in Table 3.1. Note that the data analyzed by Nowak (1999) showed that the two-lane to one-lane maximum load ratio is on the order of 2 times 0.85=1.70. The analysis of the New York WIM data shows a ratio of 1.23. Thus, the increase in the live load factor by a factor of 1.4 is approximately equal to the ratio between the New York WIM data and the generic data of the AASHTO LRFD (1.38=1.70/12.3).

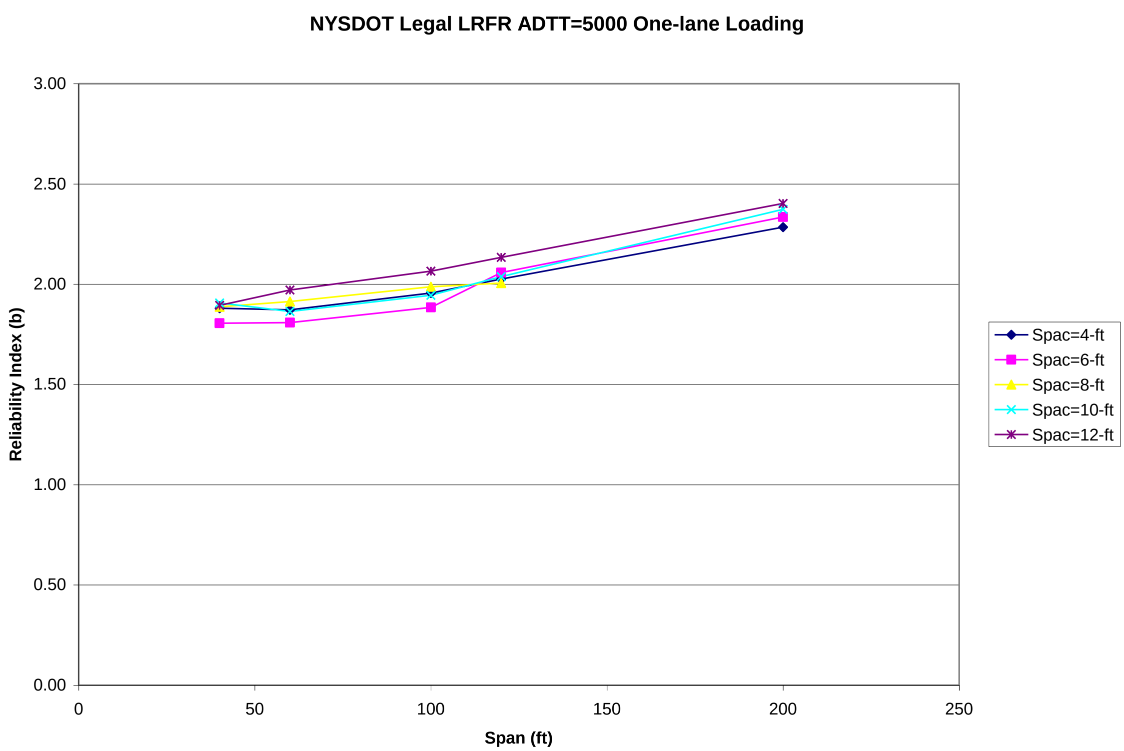


Figure 3.8. Reliability indexes for one-lane composite steel bridges with γ_L=2.65

Reliability Index for Two-lane Bridges under Maximum Single-Lane Load

A smaller increase in the live load factor than that observed for one-lane bridges may be necessary for multi-lane bridges loaded by a single lane of trucks when the calculation of the Rating Factor is performed with the two-lane distribution factor. In fact, when a two-lane bridge is subjected to a single lane loading, the target reliability index of 2.0 along with a minimum value that is equal to 1.50 can be achieved if the live load factor is increased from γ_L=1.85 to γ_L=1.95. This change will produce an average reliability $\beta_{average}$=2.10 and a minimum value β_{min}=1.47 and a maximum value β_{max}=2.70 as shown in Figure 3.9. The increase from 1.85 to 1.95 is necessary for the following reasons: a) to account for variations in the biases and COV's of the single lane loads as compared to the two-lane loads; b) to compensate for the range of differences between the two-lane distribution factor used in the rating equation as compared to the one-lane distribution factor associated with the one-lane loading; and c) to ensure that no reliability levels for any of the cases analyzed fall below the minimum set target of 1.50. The results in Figure 3.9 show a larger spread in the reliability index for different beam spacings due to the inconsistent differences between the one-lane and two-lane AASHTO LRFD distribution factors.

To illustrate why the increase is necessary, we follow the same simple example for the 100-ft steel bridge with beams at 8-ft spacing. The dead load effects are given as D_{c1}=386 kip-ft, D_{c2}=1150 kip-ft and D_w=270 kip-ft. For this span length, the 3S-2 truck governs with truck moment effect 1332 kip-ft. The two lane AASHTO LRFD load distribution factor is calculated from Eq. (2.34) to be D.F.=0.62. Using an impact factor =1.33 and the live load factor γ_L=1.95, the nominal resistance is calculated as R_n=4467 kip-ft. The mean resistance becomes $\overline{R}$=5003 kip-ft with a COV=10%. The mean dead load is $\overline{DL}$=1875 kip-ft with a standard deviation σ_{DL}=142 kip-ft. The mean live load is obtained from $L_{max}xHL_{93}$ from Table 2.19 which is equal to 4500 kip-ft multiplied by a mean impact factor $\overline{IM}$=1.13 and mean distribution factor $\overline{D.F.}$=0.36 which give a mean live load $\overline{LL}$=1831 kip-ft with a live load COV V_{LL}=18% leading to a standard deviation σ_{LL}=330 kip-ft. The total mean load is then $\overline{S}$=3706 kip-ft and the standard deviation is σ_S=359 kip-ft for a COV V_S=9.7%. Using Eq. (2.9), the reliability index for the lognormal model is obtained as β=2.16. The FORM algorithm which uses as input the values and probability distributions in Table 3.1 yields a reliability index β=2.15 for this case. Essentially, the 1.95 live load factor used here with the two lane distribution factor compensates for using the 2.65 live load factor in conjunction with the one-lane distribution factor.

Table 3.4 shows the relationship between the live load factors and the average reliability index obtained for the bridges analyzed. A slightly higher live load factor is used leading to a slightly higher average reliability index than the target β_{target}=2.0 to ensure that the minimum reliability index value remains at or above 1.50.

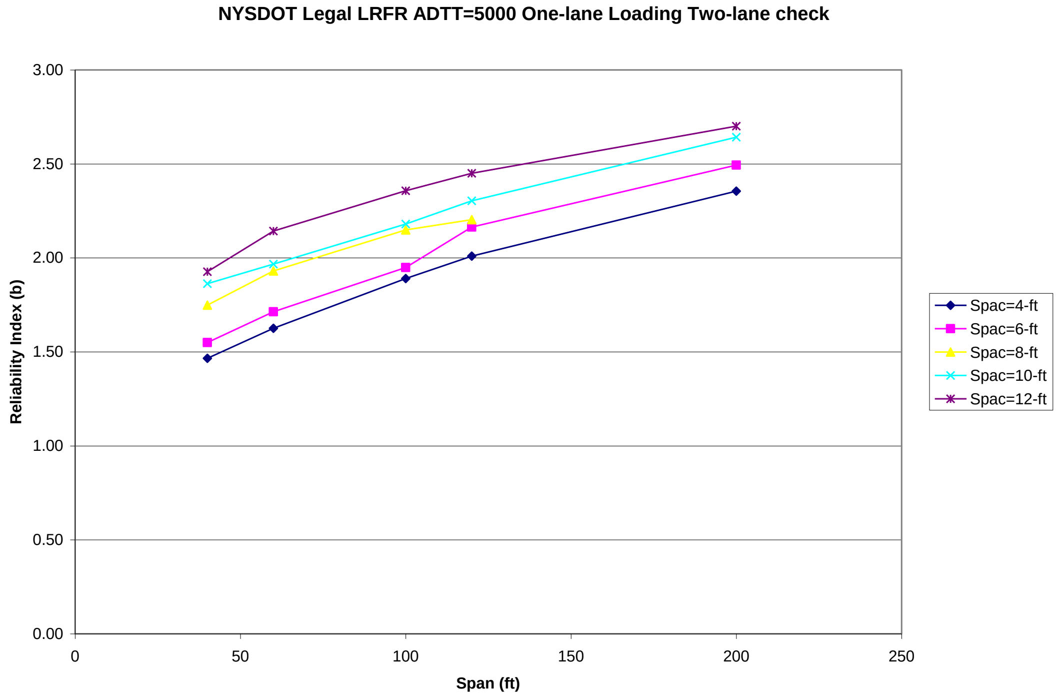


Figure 3.9. Reliability indexes for two-lane composite steel bridges rated using the two-lane distribution factor with γ_L=1.95 when loaded by the maximum single lane load.

Table 3.3. LRFR Live load factors required to meet different target reliability indexes for one lane bridges.

	β_{target}=2.5	β_{target}=2.25	β_{target}=2.0	β_{target}=1.75	β_{target}=1.5
	Legal Load Rating One-lane Bridges ADTT=5000				
γ_L	3.05	2.85	2.65	2.45	2.25
β_{avarge}	2.52	2.28	2.03	1.77	1.49
β_{min}	2.38	2.11	1.81	1.46	1.09
β_{max}	2.72	2.57	2.41	2.26	2.11
	Legal Load Rating One-lane Bridges ADTT=1000				
γ_L	2.85	2.65	2.50	2.30	2.10
β_{avarge}	2.50	2.25	2.05	1.78	1.50
β_{min}	2.36	2.07	1.82	1.46	1.08
β_{max}	2.72	2.56	2.44	2.28	2.13
	Legal Load Rating One-lane Bridges ADTT=100				
γ_L	2.60	2.40	2.20	2.05	1.85
β_{avarge}	2.54	2.27	2.00	1.78	1.48
β_{min}	2.40	2.10	1.73	1.44	1.02
β_{max}	2.73	2.57	2.41	2.29	2.13

Table 3.4. LRFR Live load factors required to meet different target reliability indexes for two lane bridges where a single lane is loaded.

	AASHTO LRFR	NYS-LRFR	β_{target}=2.5	β_{target}=2.25	β_{target}=2.0	β_{target}=1.75	β_{target}=1.5
		Legal Load Rating Two-lane Bridges ADTT=5000					
γ_L	1.80	1.95	2.20	2.05	1.90	1.75	1.65
β_{avarge}	1.82	2.10	2.53	2.27	2.00	1.73	1.54
β_{min}	1.12	1.47	2.00	1.69	1.35	1.00	0.75
β_{max}	2.53	2.70	2.97	2.81	2.65	2.48	2.36
		Legal Load Rating Two-lane Bridges ADTT=1000					
γ_L	1.65	1.85	2.05	1.90	1.80	1.65	1.50
β_{avarge}	1.76	2.14	2.49	2.23	2.05	1.76	1.47
β_{min}	1.03	1.52	1.97	1.63	1.40	1.03	0.64
β_{max}	2.50	2.73	2.95	2.78	2.67	2.50	2.33
		Legal Load Rating Two-lane Bridges ADTT=100					
γ_L	1.40	1.65	1.85	1.70	1.60	1.45	1.35
β_{avarge}	1.61	2.11	2.49	2.22	2.02	1.72	1.51
β_{min}	0.82	1.49	1.97	1.61	1.36	0.96	0.67
β_{max}	2.41	2.70	2.93	2.76	2.64	2.46	2.35

Sites with ADTT=1000

The following three loading/rating cases were analyzed for bridges with ADTT=1000 trucks per day: a) Two-lane bridges subjected to the maximum two-lane load of Table 2.10, b) Two-lane bridges rated with the AASHTO two-lane distribution factor but subjected to the maximum single lane load of Table 2.12 and 3) One-lane bridges rated using the AASHTO one-lane distribution factor and subjected to the maximum single lane load of Table 2.12. To achieve the required target β_{target}=2.0 while maintaining a minimum reliability index above 1.50 for the bending of the simple span composite steel bridge configurations considered in this section, the following LRFR live load factors will need to be applied: γ_L=1.65, γ_L =1.85 and γ_L =2.50 respectively. Two-lane bridges subjected to the maximum two-lane load rated with γ_L=1.65 will yield reliability indexes ranging between 1.73 and 2.31 with $\beta_{average}$=2.0 as shown in Figure 3.10. The application of a live load factor γ_L=1.85 when rating two-lane bridges using the AASHTO two-lane distribution factor but when the bridge is subjected to the maximum single lane load will yield an average reliability index $\beta_{average}$=2.14 in a range between β_{min}=1.52 and β_{max}=2.73 as shown in Figure 3.11. A live load factor γ_L=2.50 is required when rating one-lane bridges to yield an average reliability index $\beta_{average}$=2.05 in a range between β_{min}=1.82 and β_{max}=2.44 (see Figure 3.12). A summary showing how the reliability index averages and ranges change with the live load factors for each of the bridge lane loading cases is provided in Tables 3.2, 3.3, and 3.4.

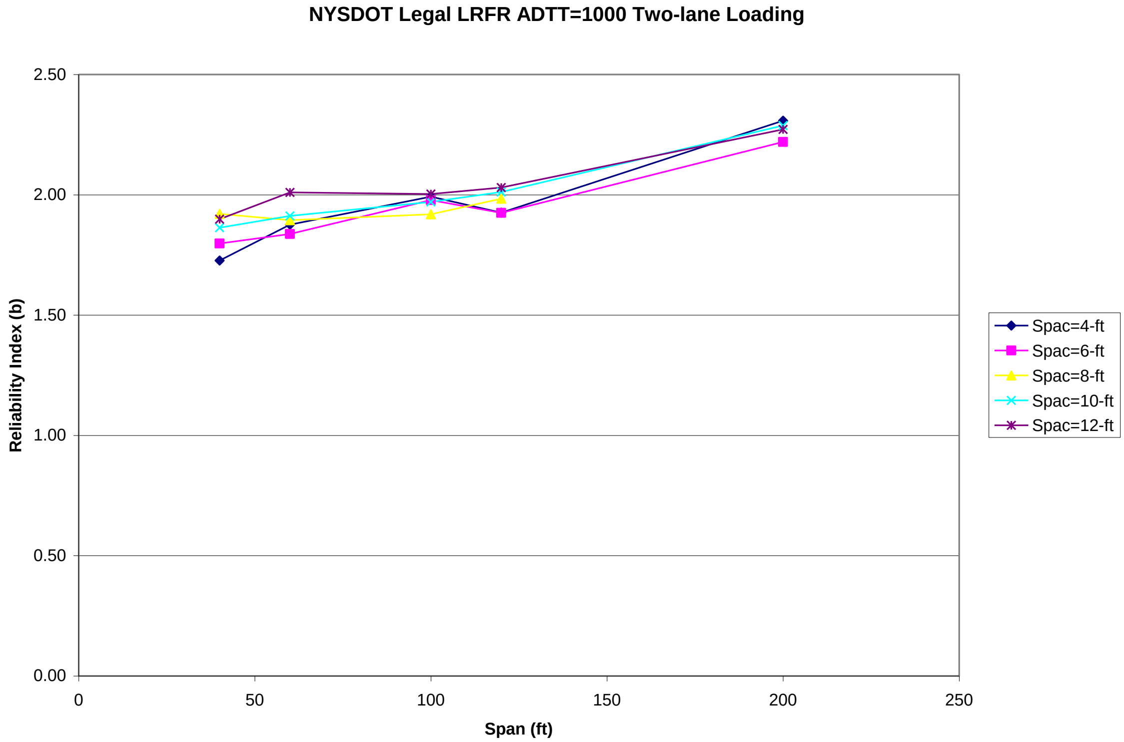


Figure 3.10. Reliability indexes for two-lane composite steel bridges with ADTT=100 using γ_L=1.65

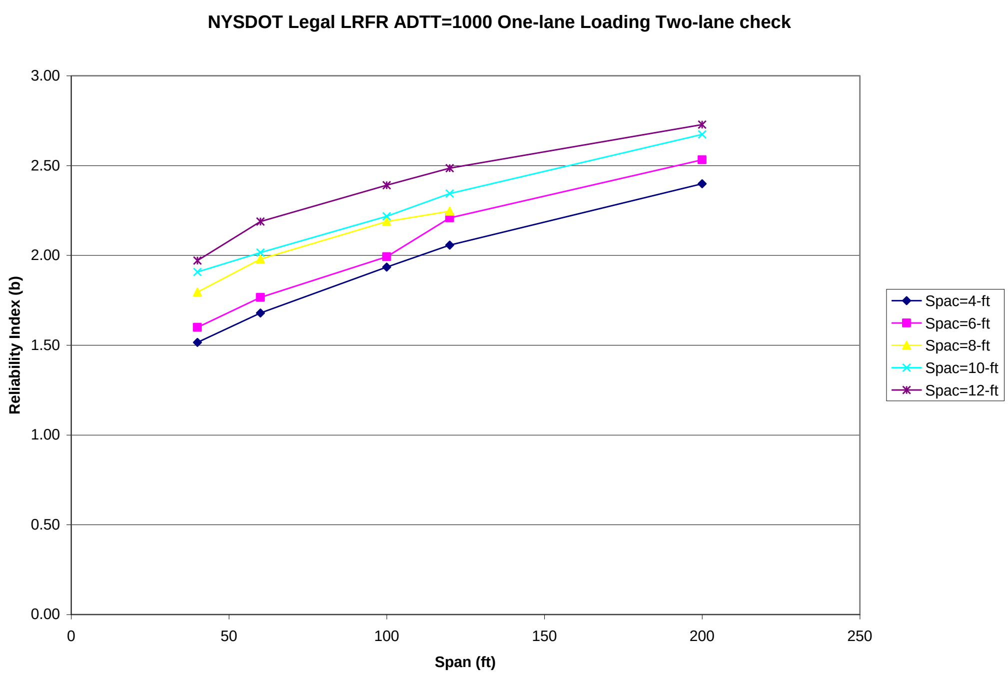


Figure 3.11. Reliability indexes for two-lane composite steel bridges with ADTT=1000 rated using the two-lane distribution factor with γ_L=1.85 when loaded by the maximum single lane load.

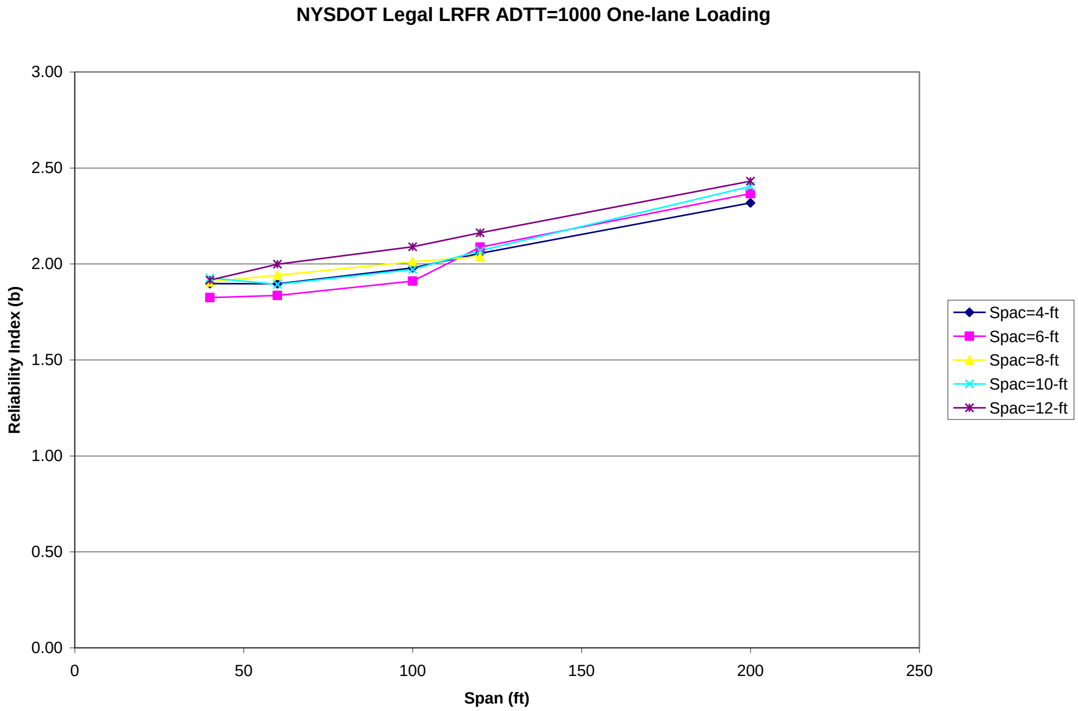


Figure 3.12. Reliability indexes for one-lane composite steel bridges with ADTT=1000 using γ_L=2.50

Sites with ADTT=100

The same three loading/rating cases were analyzed for bridges with ADTT=100 trucks per day. In the case of sites with ADTT=100, the required target β_{target}=2.0 while maintaining a minimum reliability index above 1.50 was achieved for bridges loaded with the maximum two-lane load when these bridges are rated using a live load factor γ_L=1.35 which yielded reliability indexes ranging between 1.71 and 2.33 with $\beta_{average}$=2.0 as shown in Figure 3.13. The application of a live load factor γ_L=1.65 when rating two-lane bridges with the AASHTO two-lane distribution factor subjected to the maximum single lane load will yield an average reliability index $\beta_{average}$=2.12 in a range between β_{min}=1.49 and β_{max}=2.70 as shown in Figure 3.14. A live load factor γ_L=2.20 is required when rating one-lane bridges to yield an average reliability index $\beta_{average}$=2.0 in a range between β_{min}=1.73 and β_{max}=2.41 as plotted in Figure 3.15. A summary showing how the reliability index averages and ranges change with the live load factors for each of the bridge lane loading cases is shown is provided in Tables 3.2, 3.3, and 3.4.

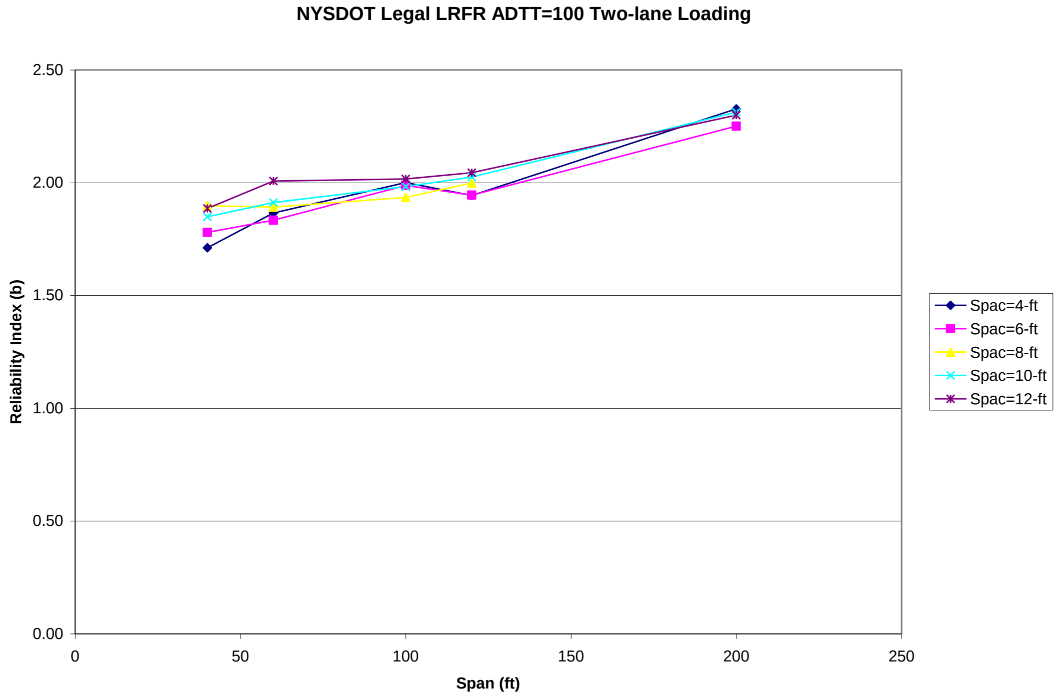


Figure 3.13. Reliability indexes for two-lane composite steel bridges with ADTT=100 using γ_L=1.35

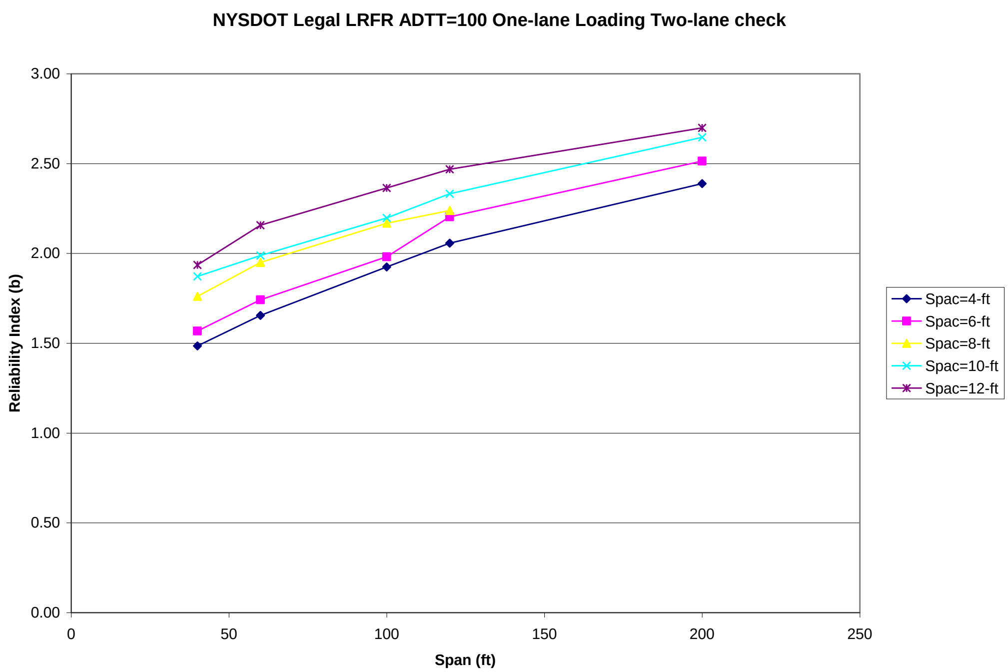


Figure 3.14. Reliability indexes for two-lane composite steel bridges with ADTT=100 rated using the two-lane distribution factor with γ_L=1.65 when loaded by the maximum single lane load.

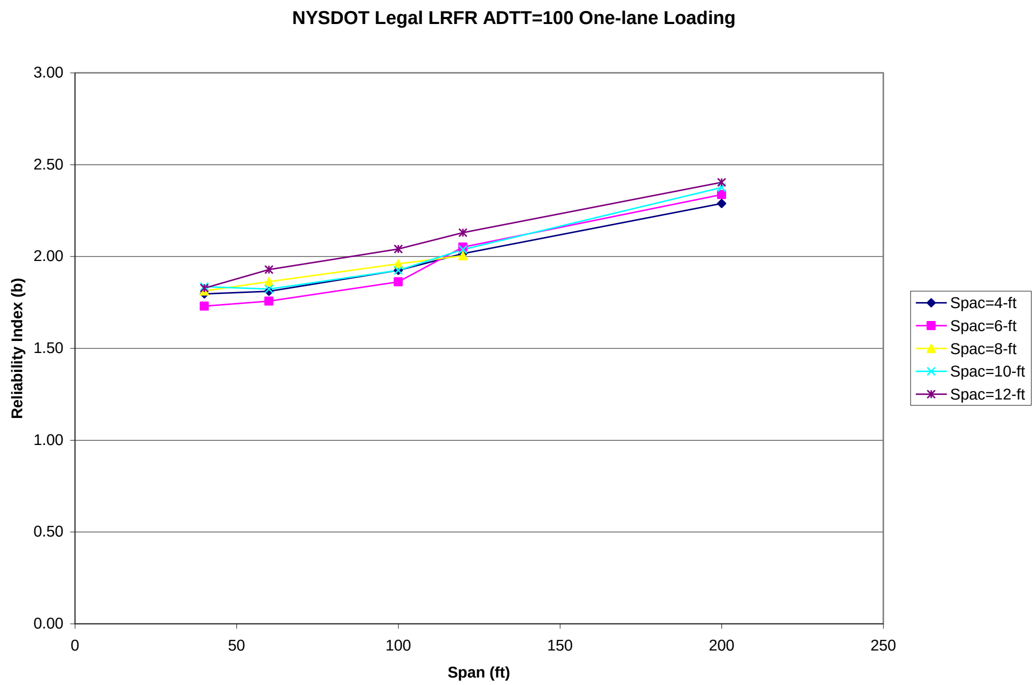


Figure 3.15 Reliability indexes for one-lane composite steel bridges with ADTT=100 using γ_L=2.20

Summary and Recommendations

This section of Chapter 3 reviewed and revised the previous calibrations performed for the AASHTO LRFR for legal load ratings. Based on the analysis performed in this report the following recommendations are made:

- Use the SU-4 vehicle and the AASHTO legal 3S-2 trucks for the NYSDOT LRFR legal load rating as these two trucks provide a better envelope of the live load effects by reducing the spread in the reliability index values for the range of spans considered.
- For multi-lane bridges use the live load factors γ_L=1.95, γ_L=1.85 and γ_L=1.65 for bridges with ADTT=5000, 1000 and 100 respectively. These live load factors are higher than those in the AASHTO LRFR and are justified based on the fact that the New York State WIM data shows higher loads and load effects than those used during the calibration of the AASHTO LRFR. These live load factors associated with the recommended SU-4 and 3S-2 Rating trucks will ensure that the reliability index is more uniform for all span lengths and remains higher than a minimum value of β=1.50. Also, these live load factors will envelope the effects of multi-lane bridges where the single lane load may be the governing load as observed from the analysis of the New York WIM data.

- For single lane bridges, a higher set of live load factors will be necessary to meet the target reliability level set in this study. Accordingly, the live load factors γ_L=2.65, γ_L=2.50 and γ_L==2.20 should be used for bridges with ADTT=5000, 1000 and 100 respectively when applying the AASHTO LRFD load distribution factors that implicitly use a multiple presence factor MP=1.20. Neither the AASHTO LRFD nor the AASHTO LRFR calibrated live load factors for single lane bridges, although a recommendation was made in NCHRP Report 545 to calibrate such factors based on state-specific WIM data. In such a case Moses (2001) recommends that even when the state DOT may lower the live load factors, the minimum live load factor for single lane bridges maintains a ratio of 1.40 (1.38=1.80/1.30) as compared to multi-lane bridges. The ratio in the live load factors recommended by Moses (2001) is very similar to the ratios of the live load factors recommended in this study ($1.36=2.65/1.95 \approx 2.50/1.85 \approx 2.20/1.65$).
- For the rating of bridges for shear, a strength reduction factor of ϕ=0.95 for steel and prestressed concrete bridges would maintain the same average reliability levels as those observed for bridges under flexural bending. This is based on the assumption that the bias factors provided by Nowak (1999) still apply for the recently developed AASHTO LRFD methods for checking the safety of bridges subjected to shear loads. However, based on the changes in the AASHTO LRFD shear provisions, which are believed to produce lower levels of uncertainties compared to the AASHTO LFD provisions. It is recommended to maintain the same resistance factors as those provided in the current AASHTO LRFD.

3.3 Reliability Calibration of LRFR Load Posting Methodology

The current New York State Posting Procedure can be summarized as follows:

1. Based on the effective span length, find the H equivalent of Legal Load from Table 1 of EI 05-034.
2. Calculate the Operating Rating for the AASHTO H 20 load (HOR) using Load Factor Rating (LFR) or Allowable Stress Rating (ASR).
3. Convert the rating factor RF to tonnage.
4. Adjust the HOR to get the Safe Load capacity (SLC). SLC can vary from 0.60HOR for non-redundant members in poor condition (rating of 3 or less) to 0.85HOR for redundant members in good condition (rating of 4 or more). Only members with excess capacity of 125% can go up to HOR. Note that in the AASHTO LRFR, the adjustment to the rating capacity for bridge redundancy and member condition is effected directly within the load rating equation by using resistance modifiers and system factors.
5. If SLC < H equivalent load, posting is necessary.
6. For bridges needing posting, adjust the SLC computed in Step 5, using Table 3 of EI 05-034 to account for axle spacings of the legal trucks. The adjustment is based on the value of SLC and the bridge's span length. The final posting value is the adjusted SLC.

A reliability index β_{target}=2.0 was chosen for target during the calibration of the live load factors for the recommended New York State Legal Load Operating rating. This target is slightly more conservative than the average reliability value implicit in current New York State DOT practice. The target reliability index reflects current loading conditions on New York bridges as projected from New York WIM data. The load posting of bridges is often done based on more conservative criteria than the rating criteria. For example, Moses (2001) calibrated the posting equation of the AASHTO LRFR based on meeting a target reliability level β=3.5 for bridges with low ratings even though his calibration of the live load factor for legal load rating and permits was based on meeting a target reliability β=2.5. Moses (2001) then used an interpolation procedure so that bridges evaluated using the AASHTO LRFR Legal Load Ratings with rating factors R.F.=1.0 will not need to be posted. The target reliability level used by Moses (2001) is applicable for bridges that are subjected to loadings consistent with those described in the AASHTO LRFR and LRFD calibration reports. Since the New York state bridge loads are considerably different, a target reliability level applicable for New York bridges may need to be extracted from currently posted NY bridges that have shown acceptable performance.

This Section of Chapter 3 presents a reliability analysis of the current NYSDOT Posting procedure and recommends a new method based on LRFR principles. To that end, an assessment of the reliability levels implicit in the current NYSDOT procedure is first executed. The results are used to extract an appropriate target reliability levels for the calibration of the NYSDOT LRFR Posting procedure. A reliability-based calibration method is then performed to propose a NYSDOT LRFR posting methodology. Since no data is available on posted bridge loadings, several assumptions are made to obtain load models following the approach used by Moses (2001) but adjusted to reflect NYSDOT WIM data. Because load posting is normally imposed on secondary bridges, the calculations performed in this section are primarily based on bridges with ADTT=100. A sensitivity analysis is used to show that the results are insensitive to the site ADTT.

Reliability Analysis of Current NYSDOT Posting Procedure

Load Model

To evaluate the reliability levels implicit in current NYSDOT posting procedures, a reliability analysis of the current NYSDOT posting method is executed for a set of simulated posted bridges with the posting loads provided in Table 3 of the New York State DOT Engineering Instruction Document No. EI05-034. The analysis is performed for simple span bridges with lengths of 40, 60, 100, 120 and 200-ft which have SLC values of 6, 10, 14, 18, 21 and 24 tons. The corresponding posting levels as extracted from Table 3 of EI05-034 are provided in Table 3.5 below.

Table 3.5. NYSDOT posting values for different SLC and span lengths

SLC	6 tons	10 tons	14 tons	18 tons	21 tons	24 tons
Span	Posting weight (tons)					
40 ft	6	10	14	18	22	
60 ft	6	10	14	18	22	25
100 ft	6	10	16	22	25	
120 ft	6	12	18	25	28	
200 ft	6	12	20	28		

According to the current NYSDOT method, for each SLC and span length, the same posting weight applies to all truck types. In order to get an evaluation of the maximum load effect that will govern bridges posted as per the values of Table 3.5, the maximum moment effects that would be obtained if the weight limits are imposed on the three AASHTO Legal vehicles are calculated for each of the three AASHTO Legal vehicles and the maximum moment is provided in Table 3.6. In other words, Table 3.6 gives the maximum allowed moment effect in kip-ft for each posted weight of Table 3.5.

Table 3.6. NYSDOT maximum allowed moment for different SLC and span lengths

SLC	6 tons	10 tons	14 tons	18 tons	21 tons	24 tons
Span	Maximum allowed moment per truck (kip-ft)					
40 ft	84	140	196	252	308	
60 ft	144	239	335	431	527	598
100 ft	263	439	702	966	1097	
120 ft	323	647	970	1347	1509	
200 ft	563	1126	1877	2628		

Moses (2001) conjectures that the actual maximum weight that crosses a posted bridge will be significantly higher than the posted weight due to the possibility of driver error as well as illegal trucks. Specifically, Moses (2001) assumes that bridges posted for 6,000 lbs (3 tons) will actually be subjected to maximum loads that may average around 16,000 lbs producing a bias of 2.67.

For bridges that do not need posting where the rating factor is exactly equal to R.F.=1.0, the Weigh-In-Motion data analyzed in Chapter 2 show an actual bias between the expected maximum load effects on two-lane bridges that ranges between 1.57 to 1.68 with an average value of 1.60 when compared to the effect of two side-by-side AASHTO legal trucks on sites with ADTT=100. The New York state legal load limits allow higher loads than the federal limits for trucks with gross weights below 71 kips. If the weight of the AASHTO Type 3 truck is then raised to the 54 kips allowed in New York, the biases would change to those shown in Table 3.7 which the ratios for different span lengths and ADTT for bridges with Rating Factor R.F.=1.0.

Because posting is normally used for bridges with low ADTT levels the calculations that will be performed in this section will be based on sites with ADTT=100. Following the approach by Moses (2001), a linear interpolation between the bias of 2.67 that he proposed for bridges posted at 3 Tons and the biases shown in Table 3.7 for ADTT=100 are used to estimate the maximum load effect on bridges posted for weights greater than 3.0. The biases and moments for different posting loads are provided in Table 3.8 for different span lengths to illustrate how the maximum 5-year live load moment would change for different posting weights.

Table 3.7. Comparison of maximum legal moments and L_{max} for two-lane bridges with different ADTT

Moment effects (kip-ft)		$L_{max}xHL_{93}$ (kip-ft)			Ratio L_{max}/Legal		
Span	maximum legal	ADTT=5000	ADTT=1000	ADTT=100	ADTT=5000	ADTT=1000	ADTT=100
40 ft	350	1558	1420	1174	2.10	1.92	1.58
60 ft	618	2665	2426	2012	2.10	1.91	1.59
100 ft	1343	5520	5057	4221	2.06	1.88	1.57
120 ft	1743	7123	6517	5456	2.04	1.87	1.57
200 ft	3342	13622	12514	10494	2.04	1.87	1.57
Average					2.07	1.89	1.58

Table 3.8. Load bias and corresponding maximum moment effect for different SLC values.

	SLC=6	SLC=10	SLC=14	SLC=18	SLC=21	SLC=24
	bias					
40 ft	2.53	2.34	2.16	1.98	1.79	
60 ft	2.53	2.34	2.16	1.98	1.79	1.65
100 ft	2.55	2.39	2.15	1.91	1.79	
120 ft	2.55	2.33	2.10	1.84	1.73	
200 ft	2.57	2.36	2.09	1.83		
	Moment (kip-ft)					
40 ft	212	328	423	497	551	
60 ft	363	561	724	852	944	990
100 ft	671	1049	1511	1847	1968	
120 ft	826	1506	2040	2480	2608	
200 ft	1445	2663	3933	4799		

In addition to the effect of overloads, Moses (2001) accounted for the higher dynamic amplification factor that is associated with the lower truck weights that travel over posted bridges as compared to the heavy weights over regular bridges. In this report, we will assume that the dynamic effect remains on the average the same independent of the static weight based on the observation made by Nassif et al (2005). Therefore, if one defines the dynamic amplification factor IM as the ratio of the sum of the Static effect, S, plus the dynamic effect, D, divided by the static effect, such that:

$$IM = \frac{S+D}{S} \tag{3.4}$$

Then, the dynamic effect alone becomes:

$$D = (IM - 1)S \tag{3.5}$$

If D remains independent of the magnitude of S, then for lower values of static weights, the impact factor will increase such that:

$$IM^* = 1 + (IM - 1)\frac{S}{S^*} \tag{3.6}$$

Where the variables with the * are those associated with posted bridges. The standard deviation of the IM* will thus also vary as a function of S/S* such that:

$$\sigma_{IM^*} = \sigma_{IM}\frac{S}{S^*} \tag{3.7}$$

Table 3.9 provides the mean values of IM* and the corresponding COV's of the dynamic amplification factor for each span length and posting level. The estimation of the mean impact and COV are based on Eq. (3.6) and (3.7) where S is taken as the load effect of Table 3.8. When the bridge does not need posting, the maximum load effect is $L_{max}xHL_{93}$ and the mean impact factor is IM=1.10 and the COV=5.5% for two side-by-side heavy trucks on regular bridges. The values shown in Table 3.9 for the maximum mean Impact vary between 1.25 to 1.33 for SLC=6. These represent an increase by a factor 1.15 to 1.22 times the impact value used for non-posted bridges with a significantly higher COV which may reach as high as 16% when compared to the V_{IM}=5.5% used for non-posted bridges. Moses (2001) used an increase in impact mean of 1.1, but implicitly kept the COV at its initial value. The increase in the standard deviation and the corresponding COV proposed herein is justified based on the data presented by Nassif et al (2005) as depicted in Figure 3.16.

Table 3.9. Mean and COV of dynamic amplification factor corresponding to different SLC

	SLC=6	SLC=10	SLC=14	SLC=18	SLC=21	SLC=24
Span	Mean Impact					
40 ft	1.25	1.16	1.12	1.11	1.10	
60 ft	1.25	1.16	1.13	1.11	1.10	1.10
100 ft	1.28	1.18	1.13	1.10	1.10	
120 ft	1.30	1.16	1.12	1.10	1.10	
200 ft	1.33	1.18	1.12	1.10		
	COV					
40 ft	0.13	0.09	0.07	0.06	0.06	
60 ft	0.13	0.09	0.07	0.06	0.06	0.06
100 ft	0.15	0.10	0.07	0.06	0.06	
120 ft	0.15	0.09	0.07	0.06	0.06	
200 ft	0.16	0.10	0.07	0.06		

In addition to the changes in the impact and load bias, Moses (2001) proposed an additional bias of 1.1 to account for changes in the axle weight distributions of the trucks crossing a posted bridge as compared to the effect of the AASHTO 3S-2 truck configuration which he had used as the basis for calculating the moment effects. In this report, this factor is not explicitly included since variations in axle weight distributions are considered through the direct use of the load effects in the reliability analysis of the WIM data rather than the gross weights as done by Moses (2001).

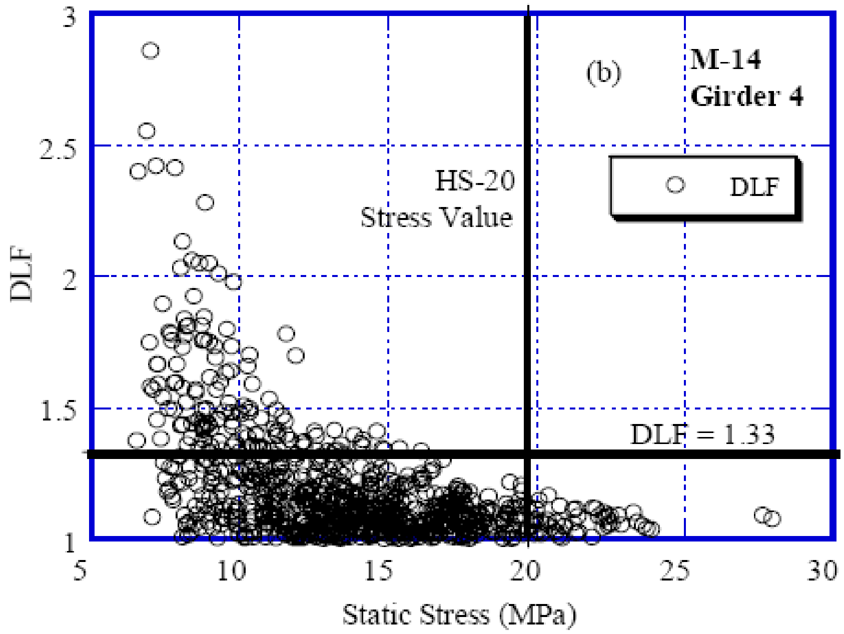


Figure 3.16. Changes in Dynamic Amplification Factor with maximum static stress based on Nassif et al (2005).

Current NYSDOT Resistance Model

The SLC values used as input in Table 3 of NYSDOT EI 05-034 are obtained based on the H truck's Operating Ratings (HOR) with additional reduction factors, herein referred to by the symbol RED, with RED=0.60, 070, 0.80, 0.85 or 1.0 depending on the member condition and bridge redundancy. The nominal resistance is related to the SLC by:

$$R_n = 1.3D_n + 1.3L_n(SLC) \tag{3.8}$$

Where D_n is the nominal dead load effect and L_n (SLC) is the live load effect due to a truck having the configuration and axle weight distribution of the AASHTO H truck but a weight equal to the Safe Load Capacity (SLC). SLC and the H-20 Operating Rating, HOR, are related by:

$$HOR = \frac{SLC}{RED} \tag{3.9}$$

The nominal resistance, R_n, can be directly related to the HOR using the equation:

$$R_n = 1.3D_n + 1.3L_n(HOR) \tag{3.10}$$

For the calculation of the nominal live load, L_n, the AASHTO ASD load distribution factor, D.F., and dynamic amplification factor are used. These are given as:

$$IM = 1 + \frac{50}{125 + Span} \leq 1.30 \tag{3.11}$$

$$D.F. = \frac{spacing}{5.5} \tag{3.12}$$

Where D.F. is applied on the wheel load for two-lane bridges.

The base line check for the NYSDOT current procedure uses a Reduction factor RED=0.85. This value is used for bridges that are redundant and having member inspection rating ≥ 4 on the New York scale which assigns condition factors between 1 and 7. This case will be used as the base case to extract the target reliability index that will be used for calibrating a proposed NYSDOT LRFR posting methodology.

Reliability Analysis of Current Posting Method

The reliability analysis is performed using the First Order Reliability Method (FORM) with the same biases, COV's and probability distributions for the resistance, the load distribution factor, and the load modeling provided in Chapter 2 and summarized in Table 3.1. The impact mean and COV are as shown in Table 3.5 and the original L_{max}xHL93 values are replaced by those shown as moment effect in Table 3.8.

The results of the reliability analysis that are plotted in Figure 3.17 show that the reliability index for the base case is on the average equal to $\beta_{average}$=1.33 with a range varying between β_{min}=0.0 to β_{max}=2.5. Note that this average reliability level is somewhat lower than $\beta_{average}$=1.47 observed for the LFD HS-20 Load rating. The difference is due to the use of the lower H-20 nominal load and also due to the higher load bias associated with the bridges with low posting weights. It is noted that the ratio of HS-20 load effect to the effect of the H-20 truck varies from about 1.15 for short spans to over 2.1 for the longer spans thus vastly offsetting the reduction factor RED=0.85 used with the current NYSDOT H-20 load posting base case. The plot of the reliability index values in Figure 3.17 shows a large spread in the reliability indexes over the span lengths and the SLC values.

As the reduction factor RED is reduced from 0.85 to 0.60 as stipulated by the NYSDOT current procedures for nonredundant and deteriorated bridges, the average reliability index increases from $\beta_{average}$=1.33 to $\beta_{average}$=2.10. Such an increase is consistent with the approach taken by Ghosn and Moses (1998) which specifies a higher member reliability index for nonredundant bridges because the consequences of a member failure in such cases would lead to system collapse. On the other hand, bridges with redundant members will have high system reserve such that the bridge will continue to carry loads after the failure of one member. The increase in the reliability index associated with low member inspection rating is also consistent with the approach followed by Moses (2001) during the calibration of the AASHTO LRFR. The AASHTO LRFR requires the application of lower structural condition factors for members in poor conditions leading to higher overall safety factors. These higher safety factors are used to offset the higher level of uncertainty associated with the determination of the strength capacity of members in poor condition.

When no reduction factor is applied (RED=1.0) for bridges in good condition that are known to have overstrength capacity, the member reliability index implied in the current NYSDOT procedures has an average reliability index $\beta_{average}$=1.03. A summary of these average reliability values is provided in Table 3.10.

Table 3.10. Variation of average reliability index with SLC calculation basis.

SLC=0.60 HOR	SLC=0.70 HOR	SLC=0.80 HOR	SLC=0.85 HOR	SLC=1.0 HOR
$\beta_{average}$=2.10	$\beta_{average}$=1.74	$\beta_{average}$=1.45	$\beta_{average}$=1.33	$\beta_{average}$=1.03

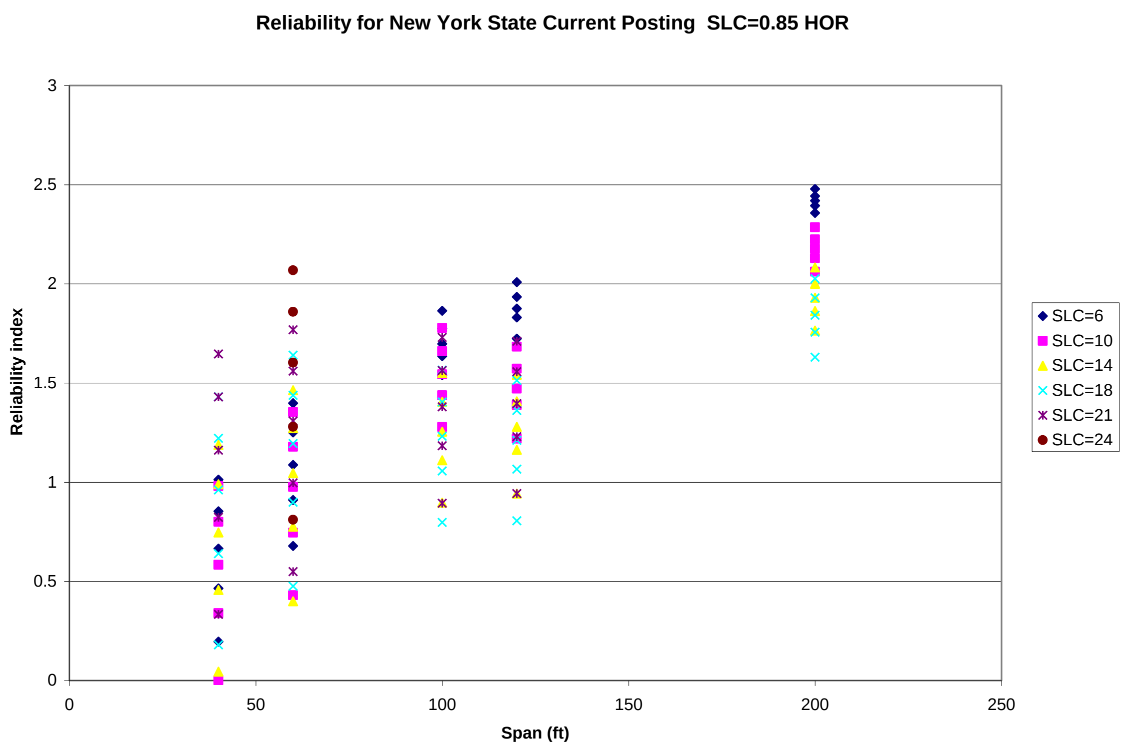


Figure 3.17. Plot of reliability index values for current NYSDOT posting methodology for SLC=0.85 HOR.

Summary

Several assumptions have to be made in order to estimate the reliability of posted bridges. This was necessary because little information is available on the loads that cross posted bridges and how these loads change with the posting levels. Also, the relationship between the estimated strength and the actual strength of deteriorated members may have different levels of biases and uncertainties than those of new or existing members in good condition. In this report, we also estimated the actual dynamic amplification factor for vehicles crossing posted bridges based on a qualitative evaluation of such factors available in published reports. The assumptions made in this report are generally similar to those made by Moses (2001) during the calibration of the AASHTO LRFR.

The following observations can be drawn from the analysis performed on the current NYSDOT Load Posting procedures:

- For the base case of posted bridges with configurations that provide sufficient levels of redundancy and having members in relatively good conditions, the reliability index on the average is found to be lower than that observed for bridges with rating factor R.F.=1 when evaluated using the HS-20 load model.
- A large spread in the current reliability indexes is observed depending on the posting weight and span length where posted short span bridges are associated with significantly lower reliability index values than longer span bridges.
- Higher reliability index values are obtained for nonredundant bridges with heavily deteriorated members when these bridges are associated with the additional safety factors specified in current NYSDOT procedures. The reliability analysis performed assumes the same bias and COV for such members as those previously used for members in relatively good conditions.
- Since the NYSOT load posting methodology is found to be less conservative but reasonably close to that implied in current NYSDOT load rating methodology, it is herein recommended that the same target reliability index β_{target}=2.0, which was used in Section 3.2 for calibrating live load factors for bridge rating, be also used for calibrating the NYSDOT LRFR posting methodology.
- Since, generally speaking, the current average is significantly below the proposed target β_{target}=2.0, the proposed load posting method will be expected to be on the average more conservative than the existing NYSDOT method. Due to this additional conservatism, it is not deemed necessary to use different target reliability levels for bridges with low posting weights as compared to the target used for bridges with higher posting weights. Therefore, it is herein proposed to use the same β_{target}=2.0 for all posting levels. This recommendation differs from the approach taken by Moses (2001) who used a higher target reliability of β_{target}=3.5 for posting bridges with lower posted weights as compared to using a target β_{target} =2.5 for the higher posted levels. The Moses (2001) different target level approach was adopted to improve the spread in the reliability index values and insure that the reliability index does not fall to unacceptably low levels. An

optimization algorithm is used in this study to propose a load posting procedure that provides uniform reliability for all posting levels and span lengths as will be explained in the next section.

Reliability Calibration of NYSDOT LRFR Load Posting Procedure

The calibration of the LRFR load posting procedure consists of determining the appropriate posting load that should be imposed on bridges that rate below R.F.=1.0. The determination of the appropriate load must be based on ensuring that posted understrength bridges will still provide an acceptable level of reliability. The same assumptions made above with respect to the relation between the posted weight and the actual live loads including dynamic amplification factors will be maintained during the calibration of the NYSDOT LRFR load posting method. By maintaining the same assumptions made earlier and by setting a target reliability index $\beta_{target}=2.0$, we will ensure that not only will the proposed NYSDOT LRFR posting method lead to higher reliability index values than currently observed, but that the reliability levels will be more uniform for all posting levels and span lengths.

A simple optimization algorithm is developed for the purpose of calibrating the required posting weight. The objective of the optimization is to determine the posting weights WPOST that will produce a reliability index equal to $\beta_{target}=2.0$ for all span lengths and for all bridges that have rating factors R.F.<1.0. The reliability index will depend on the live load model which as explained earlier is itself a function of the posted weight WPOST.

It is assumed that the basis of the rating factor calculation will be either the SU-4 truck or the ASSHTO 3-S2 which have been proposed for use as the NYSDOT Legal trucks. The relationship between the mean maximum load effect and the posted weight WPOST will be based on an interpolation between a ratio of 2.67 (=16000/6000) as proposed by Moses (2001) for bridges posted for 6000 lbs and a ratio of L_{max}/rating truck effect that corresponds to the case when the Rating Factor R.F.=1.0. So that when the posted weight is exactly equal to the weight of the rating truck, the L_{max} values obtained from the WIM data as described in Chapter 2 are met. On the other hand when the posting weight is 6,000 lbs, the maximum load effect is that of 16,000 lb truck having the same configuration as the rating truck.

During the calibration process, Equations 3.6 and 3.7 are used to find the dynamic amplification factor. For lack of better information, the other random variables affecting the live load, member resistance and dead loads are assumed to have the same biases and COV's as those listed in Table 3.1.

The calibration is performed for bridges with ADTT=100 as bridges with high truck volumes are not normally posted. However, a sensitivity analysis will be performed to investigate whether the posted weights will be affected by a change in the ADTT.

The different conditions addressed for the ADTT=100 include:

- Two lane bridges loaded by two side-by-side trucks rated using the live load factor γ_L=1.35 which had been found in Section 3.2 to produce a uniform reliability index β=2.0 when the rating factor is R.F.=1.0 for bridges loaded by two lanes and checked with the two-lane AASHTO LRFD distribution factor.
- Two lane bridges loaded by two side-by-side trucks rated using the live load factor γ_L=1.65 which had been found in Section 3.2 to produce a uniform reliability index β=2.0 when the rating factor is R.F.=1.0 for bridges loaded by a single lane and checked with the two-lane AASHTO LRFD distribution factor.
- Two lane bridges loaded by a single lane of trucks rated using the live load factor γ_L=1.65 which had been found in Section 3.2 to produce a uniform reliability index β=2.0 when the rating factor is R.F.=1.0 for bridges loaded by a single lane and checked with the two-lane AASHTO LRFD distribution factor.
- Single lane bridges loaded by a single lane of trucks rated using the live load factor γ_L=2.20 which had been found in Section 3.2 to produce a uniform reliability index β=2.0 when the rating factor is R.F.=1.0 for bridges loaded by a single lane and checked with the one-lane AASHTO LRFD distribution factor.

In each of the above 4 situations, we find the posting weight WPOST for each of the rating trucks SU4 and 3S-2 when the rating is executed with the governing load. The analysis is performed for the same set of composite simple span steel bridge configurations having span lengths varying between 40-ft and 200-ft and beam spacing ranging from 4-ft to 12-ft which have the dead loads summarized in Table 2.1.

The results for all the 8 cases (4 bridge/loading conditions times 2 different legal trucks) studied are summarized in Table 3.11. The similarities between the results for Cases 1, 5 and 7 on the one hand and for the Cases, 2, 6 and 8 on the other reinforce the fact that the calibration of the live load factors performed in Section 3.2 do indeed lead to uniform reliability levels for bridges with single lane loading and two-lane loading. However, as indicated in Section 3.2, two-lane bridges on New York state highways are more likely to be controlled by a single lane of heavy trucks. Therefore, it is recommended to use the higher live load factor γ_L=1.65 when rating two-lane bridges with ADTT=100 in combination with using the two-lane AASHTO LRFD load distribution factor. For single lane bridges of ADTT=100, a live load factor γ_L=2.2 is recommended in combination with the AASHTO LRFD single lane load distribution factor while maintaining the implicit multiple presence factor MP=1.2.

The results of Table 3.11 show significant differences in the posting weights of semi-trailer trucks as compared to those of single-unit trucks. Accordingly, it is herein recommended that the New York weight posting signs distinguish between the two types of vehicles if such a change is possible to implement.

Furthermore, the differences in the required posted weights for different spans are significantly different. This justifies maintaining the current NYSDOT approach of selecting the posting weights as a function of span length.

The recommended posting weights that will provide consistent levels of reliability for single unit trucks and semi-trailer trucks can then be summarized as shown in Table 3.13 which compares the posting weights obtained to those in the AASHTO LRFR.

When compared to the AASHTO LRFR load posting, the proposed values in Table 3.13 are higher for the low rated bridges with $\text{R.F.} \leq 0.6$ but very similar to the average values represented by the 100-ft span lengths for $\text{R.F.} \geq 0.7$. This is due to the approach followed by Moses (2001) who used a much higher reliability target for low rated bridges which is the same as that used for the Inventory level as compared to bridges with higher ratings for which he used the operating rating reliability levels. In our calibration, we use the same target reliability β_{target}=2.0 that was used for the operating rating. This target is more conservative than the average reliability level implied in the current NYSDOT load posting procedures. A qualitative comparison between the results of this table and the current NYSDOT load posting value shows a reasonable level of similarities on the average between the weights in Table 3.13.a and those in Table 3 of the EI-05-034 document. Table 3.13.a however shows a larger spread for the different span lengths. This was due to the attempt in this report to produce a uniform level of reliability across all the span lengths. It is noted however, that the recommended NYSDOT LRFR rating factor calculations uses different rating trucks and safety factors than those of the current NYSDOT procedure.

The authors of this report believe that using one set of posting weights for all types of trucks will be unduly conservative for semi-trailer type trucks. Thus, for the semi-trailers, the posting weights shown in Table 3.13.b should be used. However, if it will not prove feasible to use two different posting weights, then following the current NYSDOT procedure it is recommended to use the weight in Table 3.13.a which will provide conservative envelopes for both truck types.

Table 3.11. Summary of Posting Weights Required to Produce a Reliability Index β=2.0 for Different Rating Factors and Assumed Loading Conditions and Live Load Factors.

SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9	R.F.=1.0
Case 1. Posting Weight of Single Unit Trucks on Two Lane Bridges Controlled by Two-Lane Loading with γ_L=1.35								
40	8	14	20	27	33	39	45	51
60	10	16	22	28	34	40	46	52
100	14	20	26	31	37	42	48	53
120	16	21	27	32	38	43	48	53
200	25	31	36	41	47	52	57	62
Case 2..Posting Weight of Single Unit Trucks on Two Lane Bridges Controlled by Two-Lane Loading, γ_L=1.65								
40	12	20	27	35	42	50	57	64
60	14	21	29	36	43	50	58	65
100	18	25	32	39	45	52	59	65
120	19	26	33	39	46	52	59	65
200	29	35	42	48	55	61	67	73
Case 3. Posting Weight of Semi-TrailerTrucks on Two Lane Bridges Controlled by Two-Lane Loading with γ_L=1.35								
40	15	23	30	38	46	53	61	68
60	16	24	31	39	47	54	62	69
100	20	27	35	42	49	57	64	71
120	21	28	36	43	50	57	64	71
200	33	41	48	55	62	69	76	80
Case 4. Posting Weight of Semi-Trailer Trucks on Two Lane Bridges Controlled by Two-Lane Loading, γ_L=1.65								
40	20	29	39	48	57	67	76	80
60	21	31	40	49	58	68	77	80
100	25	34	43	52	61	69	78	80
120	26	35	44	53	61	70	78	80
200	38	47	56	64	73	80	80	80
Case 5. Posting Weight of Single Unit Trucks on Two Lane Bridges Controlled by Single Lane Loading,γ_L=1.65								
40	7	13	19	25	30	36	41	47
60	9	15	20	26	31	37	42	47
100	14	19	25	30	35	40	46	51
120	16	22	27	33	38	44	49	55
200	25	31	37	42	48	53	58	64
Case 6. Posting Weight of Semi-Trailer Trucks on Two Lane Bridges Controlled by Single Lane Loading, γ_L=1.65								
40	11	18	26	34	41	48	56	63
60	12	20	27	35	42	49	56	63
100	18	25	32	40	47	54	61	68
120	21	28	36	43	51	58	66	73
200	33	41	48	56	63	71	78	80
Case 7. Posting Weight of Single-Unit Trucks on Single Lane Bridges with γ_L=2.2								
40	8	14	20	26	32	38	44	50
60	10	16	21	27	33	39	44	50
100	14	20	25	31	36	41	47	52
120	16	21	27	32	37	43	48	53
200	25	30	36	41	46	52	57	62
Case 8. Posting Weight of Semi-trailer Trucks on Single Lane Bridges with γ_L=2.2								
40	12	20	28	36	44	51	59	67
60	13	21	29	37	44	52	59	66
100	18	26	33	40	48	55	62	69
120	20	28	35	42	50	57	64	71
200	33	40	47	55	62	69	76	80

In order to compare the effect of the ADTT on the results, the same calculations are performed for the case when the rating is executed for the multi-lane bridges having ADTT=5000. The live load factor to be used for this case is γ_L=1.95 associated with the distribution factor for two loaded lanes. Assuming that the one-lane load is controlling the maximum load effect, the posted weights required to meet the target reliability index β=2.0 are obtained for each Rating Factor and span length as shown in Table 3.12. These results are compared to those of a site with ADTT=100 analyzed using γ_L=1.65. Table 3.12 shows that generally speaking the posted weights are similar but slightly higher than those for ADTT=100 providing more conservative postings.

Table 3.12 Comparison of required Posted weights for sites with ADTT=5000 to sites with ADTT=100

SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9	R.F.=1.0
Posting Weight of Semitrailer Trucks on Two Lane Bridges ADTT=5000Controlled by one-Lane Loading with γ_L=1.95								
40	14	22	30	37	45	52	60	68
60	14	22	30	38	45	53	60	68
100	18	26	34	42	49	57	64	72
120	21	29	37	45	53	61	69	77
200	32	40	48	56	64	72	80	80
Posting Weight of Semitrailer Trucks on Two Lane Bridges ADTT=100Controlled by one-Lane Loading with γ_L=1.65								
40	11	18	26	34	41	48	56	63
60	12	20	27	35	42	49	56	63
100	18	25	32	40	47	54	61	68
120	21	28	36	43	51	58	66	73
200	33	41	48	56	63	71	78	80

Posting Weight of Semitrailer Trucks on Two Lane Bridges ADTT=5000Controlled by one-Lane Loading with γ_L=2.65								
40	16	24	32	41	49	57	65	73
60	16	24	33	41	49	57	65	73
100	19	28	36	44	51	59	67	75
120	21	29	37	45	53	61	68	76
200	32	40	48	56	64	71	79	80
Posting Weight of Semitrailer Trucks on Two Lane Bridges ADTT=100Controlled by one-Lane Loading with γ_L=2.20								
40	12	20	28	36	44	51	59	67
60	13	21	29	37	44	52	59	66
100	18	26	33	40	48	55	62	69
120	20	28	35	42	50	57	64	71
200	33	40	47	55	62	69	76	80

Table 3.13. Recommended posting weights in kips (1000 lbs)

a) Posting weights for single unit trucks							
SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9
40 ft	7	13	19	25	30	36	41
100 ft	14	19	25	30	35	40	46
200 ft	25	30	36	41	46	52	57
AASHTO LRFR	0	8	15	23	31	39	46
b) Posting weights for semi-trailer trucks							
SPAN	R.F.=0.3	R.F.=0.4	R.F.=0.5	R.F.=0.6	R.F.=0.7	R.F.=0.8	R.F.=0.9
40 ft	11	18	26	34	41	48	56
100 ft	18	25	32	40	47	54	61
200 ft	33	40	47	55	62	69	76
AASHTO LRFR	0	11	23	34	46	57	69

To simplify the process of obtaining the load posting weights, the results of Table 3.13 are fitted through an equation of the form:

$$Safe\ Posting\ \ Load = W\left[RF + 0.00375(L - 110)(1 - RF)\right] \qquad (3.13)$$

where W = Weight of Rating Vehicle
RF= Legal Load Rating Factor
L = Span length in feet

The application of Equation (3.13) was found to produce posted weight values similar to those obtained directly from the reliability calibration as listed in Table 3.13. The average ratio between the posted loads from Equation (3.12) as compared to those obtained from the reliability calibration is 1.05 with a COV of 7.5%. The range of the difference is generally between $\pm 15\%$.

The use of Equation (3.13) instead of Table (3.13) would naturally break the uniformity of the reliability index. In fact, the reliability index that would be obtained by using Eq. (3.13) will vary between a minimum value of β=1.52 and a maximum β=2.74 with an average value $\beta_{average}$=2.15 as shown in Figure 3.18. Nevertheless, the variation in the reliability index obtained from the proposed equation is significantly narrower and more uniform than that observed in the current NYSDOT procedure as depicted in Figure 3.17.

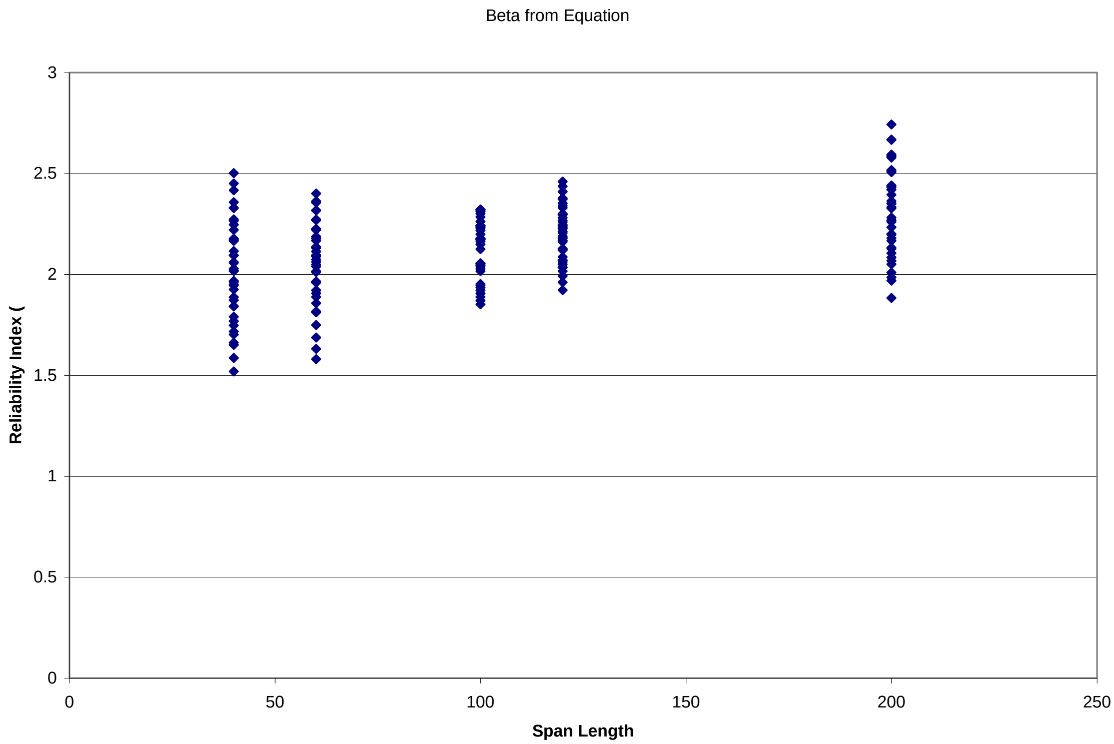


Figure 3.18. Plot of reliability index implied by using proposed posting equation.

Calibration of Member Structural Condition Factor

The current NYSDOT load posting procedure specifies different criteria based on member condition as well as system redundancy. The AASHTO LRFR has recommended a set of system factors to account for bridge redundancy. These recommended factors were somewhat based on the results of a reliability-based calibration previously performed by Ghosn & Moses (1998) and presented using a conservative and practical format. It is herein recommended to maintain these factors as it had been demonstrated that the system factor calibration process followed by Ghosn and Moses (2001) is robust and relatively insensitive to the member design and rating criteria.

On the other hand, the member condition factors in the AASHTO LRFR were recommended based on engineering judgment. The results of Table 3.10 show that to account for highly deteriorated member conditions of redundant bridges, the current NYSDOT imply an increase in the reliability index by an average $\Delta\beta=0.12$. On the other hand, nonredundant bridges with highly deteriorated members were associated with an increase in the reliability index of $\Delta\beta=0.77$.

A trial and error procedure is used in this study to find the appropriate component factor ϕ_c that would be required to raise the reliability index from a target β_{target}=2.00 to a higher target β_{target}=2.12 without any major changes in the posting weights. An increase by $\Delta\beta$=0.12 in the target reliability will be consistent with the approach followed in the current NYSDOT procedures. The reliability calculations indicate that a component factor ϕ_c=0.95 is more than sufficient to achieve that goal. In fact, the application a factor ϕ_c=0.95 will produce an average reliability index of β=2.12 with posting weights higher than those observed in Table 3.11 by an average factor of 1.12.

Similarly, for bridges that are nonredundant and also of poor condition rating, it would be desirable to obtain a target β_{target}=2.77 so that these bridges will have an increase in the reliability of $\Delta\beta$=0.77. The reliability analysis performed shows that if the component factor ϕ_c=0.95 is applied simultaneously with the most conservative system factor ϕ_s=0.85, then the target reliability index of β_{target}=2.77 would be easily exceeded when using the posting weights of Table 3.11. Actually, β_{target}=2.77 can still be achieved with posting weights higher than those shown in Table 3.11 by an average factor of 1.39.

In summary, in order to remain consistent with the increased conservativeness implied in the current NYSDOT procedures, it is herein recommended to use the component factor ϕ_c=0.95 for bridges with members that have member inspection rating less or equal to 3 based on the New York state scale that varies between 1 and 7, while using ϕ_c=1.0 when the member inspection rating is greater or equal to 4.

Summary and Recommendations

A reliability calibration procedure has been performed to calibrate posting loads for bridges with rating factor R.F.<1.0. The calibration is performed to ensure that posted bridges will still meet the target reliability level β_{target}=2.0 set during the calibration of the live load rating factors executed in Section 3.2. According to the calibration results, the following load posting process is recommended based on the available New York WIM data and other assumptions made on the loading of posted bridges.

- Two-lane bridges with low truck volumes should be posted if the rating analysis performed using a live load factor γ_L=1.65 and the maximum effect of the NYSDOT Legal Trucks consisting of the SU-4 single unit truck and the 3S-2 semi-trailer truck lead to Rating Factors R.F.<1.0. The rating equation should also include the system factor ϕ_s tabulated in the AASHTO LRFR manual and a component factor ϕ_c.

$$R.F. = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_P P}{\gamma_L \left(LL + IM^* \right)} \qquad (3.14)$$

Where the factored component capacity is: $C = \phi_c \phi_s \phi R_n$
The dynamic component is IM* =0.33 x LL
DC is the dead load effect of bridge components
DW is the dead load effect of the wearing surface
P is the effect of other permanent loads
LL is the live load effect
γ_{DC} is the dead load factor for DC
γ_{DW} is the dead load factor for DW
γ_P is the dead load factor for P

- A component condition factor ϕ_c=1.0 should be used for bridges with member condition rating ≥ 4. A component factor ϕ_c=0.95 should be used for bridges with member condition rating ≤ 3 on the New York scale that assigns condition ratings between 1 and 7.
- The system factor table of the AASHTO LRFR should be used for ϕ_s
- The posting weights can be obtained using the Equation:

$$Safe\ Posting\ Load = W\left[RF + 0.00375(L - 110)(1 - RF)\right] \quad (3.15)$$

Where W = Weight of Rating Vehicle
RF= Legal Load Rating Factor
L = Effective span length in feet

3.4 Reliability Calibration of NYSDOT LRFR Live Load Factors for Overweight Permits

The current NYSDOT load permit checking procedure uses the traditional AASHTO LFR Operating Rating live load factor γ_L=1.3 along with the LFR load distribution and impact factors to check whether to allow an overweight vehicle to cross a particular bridge. The concept is based on assuring that truck overweight crossings do not reduce bridge safety levels below the minimum level set for legal trucks. However, this concept which relies on using the same live load factors for both legal truck ratings and permit checking does not necessarily assure the same level of safety. In fact, the traditional LFR safety level criteria do not properly account for the levels of uncertainty encountered when evaluating the safety of bridges. In particular, the level of uncertainties associated with estimating the load effects of permit trucks are considerably different than those of random trucks. For this reason, the safety of bridges under overweight permit loads can only be assessed using reliability based methods that account for the differences in the levels of uncertainties when a bridge is crossed by a known load as compared to when the same bridge is crossed by random trucks. To avoid the need to perform a reliability analysis for every overweight permit request, a reliability-based code calibration process can be performed to propose Permit load factors that can be used in the safety check equation so that bridges that satisfy the equation will meet a target reliability level.

Retaining the same concept of having permit crossings maintain a safety level comparable to that implied for the crossing of random trucks, it is proposed that the permit load factors be calibrated to produce the same reliability index target β_{target}=2.0 that was used in Sections 3.2 and 3.3 for the calibration of the load rating factors and the posting loads. The calibration of the permit load factors must account for the uncertainties associated with estimating the load effects of the permit truck as well as the random trucks that may cross the bridge simultaneously with the permit truck. The latter will depend on the actual truck weight histograms as well as the multiple presence statistics for typical New York bridges as collected from New York WIM sites.

The LRFR permit checking equation will still take the form of Eq. (.3) where in this case the nominal live load L_n is the effect of the Permit truck including the Dynamic Amplification and load distribution. The object of this section is then to calibrate an appropriate set of Permit load factors, γ_L, for application in the NYSDOT LRFR equation as presented in Eq. (3.3).

Permit Load Classification

New York State has several different Permit classifications depending on the permit loading type and number of trips allowed. For the purposes of this reliability analysis, the permit loading types are classified into two categories, those carrying divisible loads and those carrying non-divisible loads. Non-divisible load permits are assumed to be controlled so that the truck weights are known to be equal to the permit weight. Divisible load permits, which are issued for a year's period, are less easily controlled and some of these have been observed to be overloaded by exceeding the permit weight limits. In terms of trip categories, the permits in this report will be divided into single-crossing (single-trip) and unlimited crossing (multi-trip) permits.

The calibration of the live load factors for the New York State LRFR for short to intermediate span bridges will consider the following four cases:

I. Permit vehicle alone on a bridge which can occur whether the permit has been issued for a single trip or multiple trips.
II. Unlimited crossings of multiple trip permits where two Permit trucks could cross a bridge simultaneously side-by-side.
III. Unlimited crossings where a Permit truck mixes with other random vehicles.
IV. Single Permit trips where the Permit truck could mix with other random vehicles.

Case I is not affected by the WIM data. Case II depends on the probability of having two Permit trucks side-by-side and this in turn will depend on the number of permits that may cross a bridge within the five year rating period and the probability of side-by-side events for that number of permit crossings. Following the recommendation of the NYSDOT Technical Working Group, we will assume up to 100 Permits per day as an upper limit for New York State bridge sites. The probability of having two side-by-side permits will then be equal to 0.5% based on the WIM data collected on New York state sites on low truck traffic volume days. For cases III and IV, the reliability analysis should account for the number of random vehicles. Following the AASHTO LRFR classifications we will consider sites with ADTT=5000, 1000 or 100. The percentage of side-by-side vehicles is taken as P_{sxs}=2% for sites with ADTT=5000, P_{sxs}=1.25% for sites with ADTT=1000 and P_{sxs}=0.5% for sites with ADTT=100. These P_{sxs} values are upper bound values obtained from the headway data collected at ten New York State WIM sites as reported by Sivakumar et al (2008).

Variability in Divisible Permit Weights

Permit trucks carrying divisible loads have been observed to be often overloaded. For example, the data collected by Sivakumar et al (2008) as part of NCHRP 12-76 have shown that New York State Type 6-A trucks which are normally limited to 120 kips often carry higher loads than the permit weight limits. Figure 3.19 shows the gross weight histogram of the Type 6-A trucks collected from the east bound lane of New York WIM site.8280. As illustrated in Figure 3.20, an exponential fit of the data for the trucks with gross weights exceeding 120 kips shows that the data can be reasonably well represented by a shifted Exponential distribution with a parameter λ=0.151 for the eastbound data and λ=0.145 for the westbound data. A typical value of λ=0.15 will be used in this report. This would result in a mean value for the overloaded permit trucks equal to:

$$\overline{P} = P + \frac{1}{\lambda} \tag{3.16}$$

Where in this case, $\overline{P}$ is the mean weight of the overloaded permits, P is the permit weight and λ is the exponential distribution parameter. The standard deviation of the overloads is given as:

$$\sigma_P = \frac{1}{\lambda} \tag{3.17}$$

Since no other data is available on overloaded divisible load permits, in this report we will assume that all divisible permit trucks may be overloaded in such a way that the trucks follow an exponential probability distribution function with the same parameter λ=0.15. Non-divisible permits are assumed to have weights exactly equal to the permitted load.

As an example, the results indicate that when Type 6-A Permit trucks with a Permit gross weight limit of 120 kips are provided with multiple crossing permits, the actual average weight of the trucks would be $\overline{P}$ =127 kips (120+1./0.15) with a standard deviation σ_P=6.67 kips.

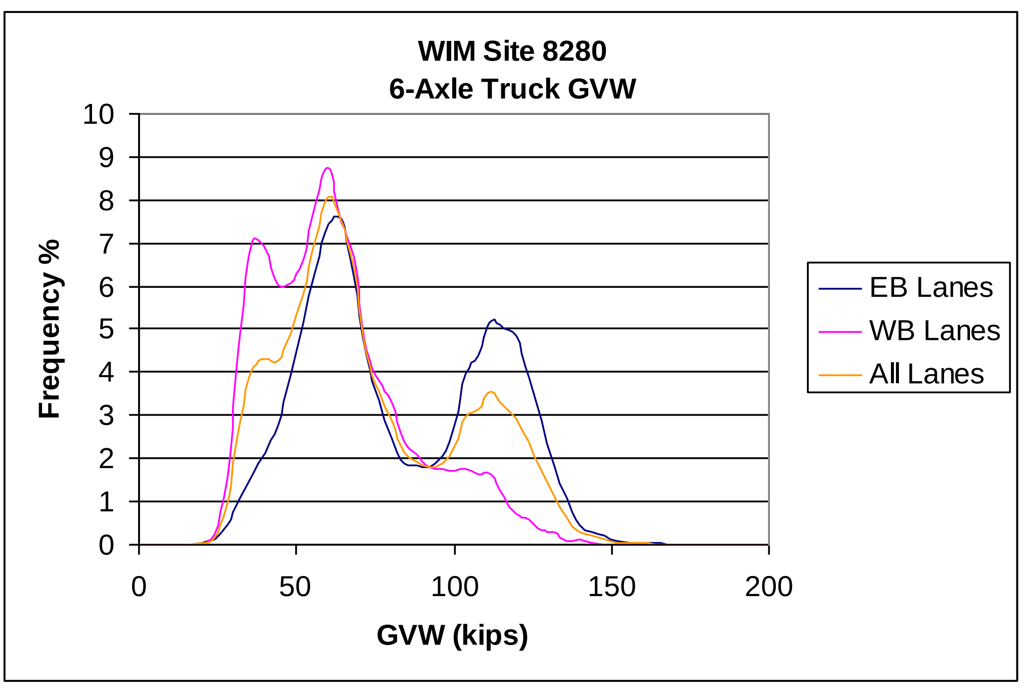


Figure 3.19. Gross weight histogram for Type 6-A trucks.

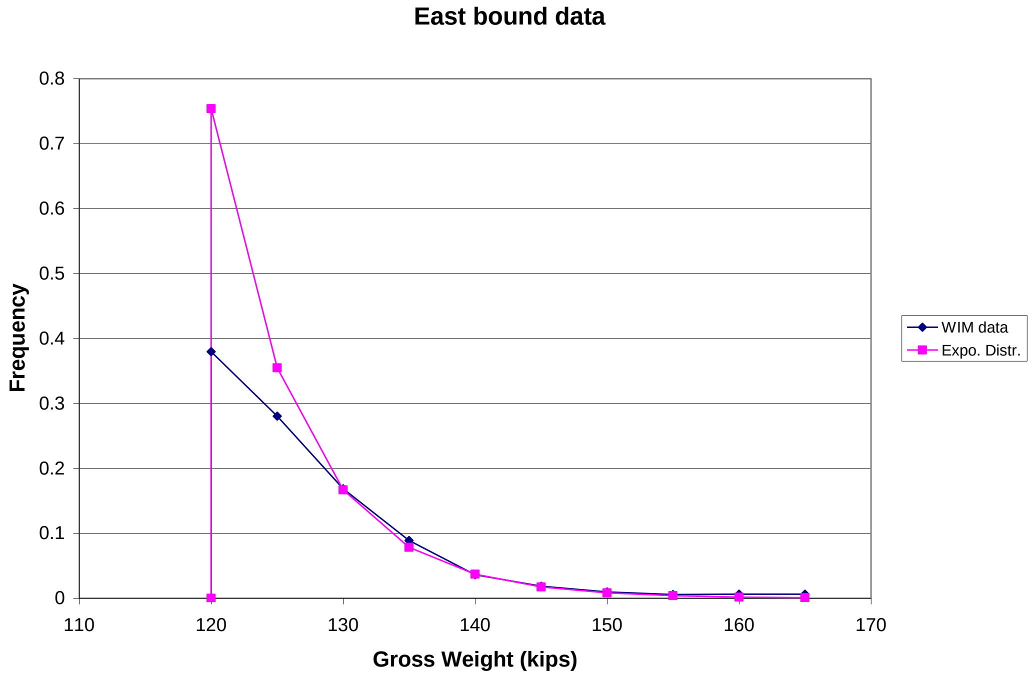


Figure 3.20. Exponential Distribution fit to WIM data for site 8280

Reliability Analysis for Case I – Permit Vehicle Alone

Deterministic Non-divisible loads

In the case where a non-divisible permit truck is alone on the bridge, we can assume that the axle configuration and axle weights of the permit truck are perfectly known so that the total maximum static live load effect on the bridge P is a deterministic value and $\overline{P} = P$. However, this does not imply that the total live load effect on a bridge member is deterministic due to the uncertainties in estimating the dynamic effect represented by the dynamic amplification factor, IM, and the uncertainties in the structural analysis process that allocates the fraction of the total load to the most critical member. For multi-girder bridges, the structural analysis is represented by the load distribution factor, D.F. The equations for the D.F. of multi-girder bridges loaded by a single lane given in the AASHTO LRFD specifications already includes a multiple presence factor MP=1.2. Following Nowak (1999), the equations provided in the AASHTO LRFD for calculating D.F. are assumed to give on the average good approximations to the actual distribution factor and therefore $\overline{DF} = DF$. Therefore, the expression for estimating the mean value of the maximum load effect on the most critical beam when a single vehicle is on the bridge can be calculated from:

$$\overline{LL} = \overline{P} \times \overline{IM} \times \overline{D.F.}/1.2 \tag{3.18}$$

Where $\overline{\boldsymbol{P}} = \boldsymbol{P}$ is the mean effect of the permit truck, $\overline{\boldsymbol{IM}}$ is the mean dynamic amplification factor and $\overline{\boldsymbol{D.F.}} = \boldsymbol{D.F.}$ is the mean load distribution factor. Dividing of $\overline{\boldsymbol{D.F.}}$ one lane by 1.2 is done to remove the AASHTO LRFD multiple presence factor.

Assuming that the weight and axle configuration of the permit vehicle are exactly known, the Coefficient of Variation (COV) of the maximum beam live load effect is obtained from the COV of IM, V_{IM}, and the COV of DF, V_{DF}:

$$V_{LL} = \sqrt{V_{IM}^2 + V_{DF}^2} \tag{3.19}$$

For the statistics of the random variables considered in Eq. (3.18) we will use the data of Nowak (1999) who observed that the actual dynamic amplification factor augmented the load effect of a single truck by an average of $\overline{IM} = 1.13$ or an additional 13% for one lane of traffic and was associated with a COV V_{IM}=9%.

Nowak (1999) also assumes that the AASHTO LRFD distribution factors give values that are very close to the mean of the actual distribution factors but he does not give an explicit value for the Coefficient of Variation V_{DF}. In previous studies on live load modeling, Ghosn & Moses (1985) found that the lane distribution factor produced variations with a COV equal to V_{DF}=8% based on field measurements on typical steel and prestressed concrete bridges.

Based on the estimates for V_{DF} and V_{IM}, the load effect of a single permit vehicle, will be associated with a COV: $V_{LL} = \sqrt{(9\%)^2 + (8\%)^2} = 12\%$.

The reliability analysis is executed using a FORM algorithm for the set of typical New York State permit vehicles having the configurations shown in Figure 3.21. In Figure 3.21, the vehicles labeled NYP-1 through NYP-5 are examples of non-divisible loads. Those labeled NYP-6 though NYP-10 for typical configurations for divisible loads. This first set of calculations will be performed for all the vehicles in Figure 3.21 assuming that they have issued non-divisible load permits and that their weights are deterministic.

The maximum moment effects of these vehicles for simple spans of 40 ft, 60 ft, 100 ft, 120 ft and 200-ft are provided in Table 3.14.

The analysis of the permit loads is performed for typical bridges having the dead load data given in Table 2.1 for simple span composite steel bridges.

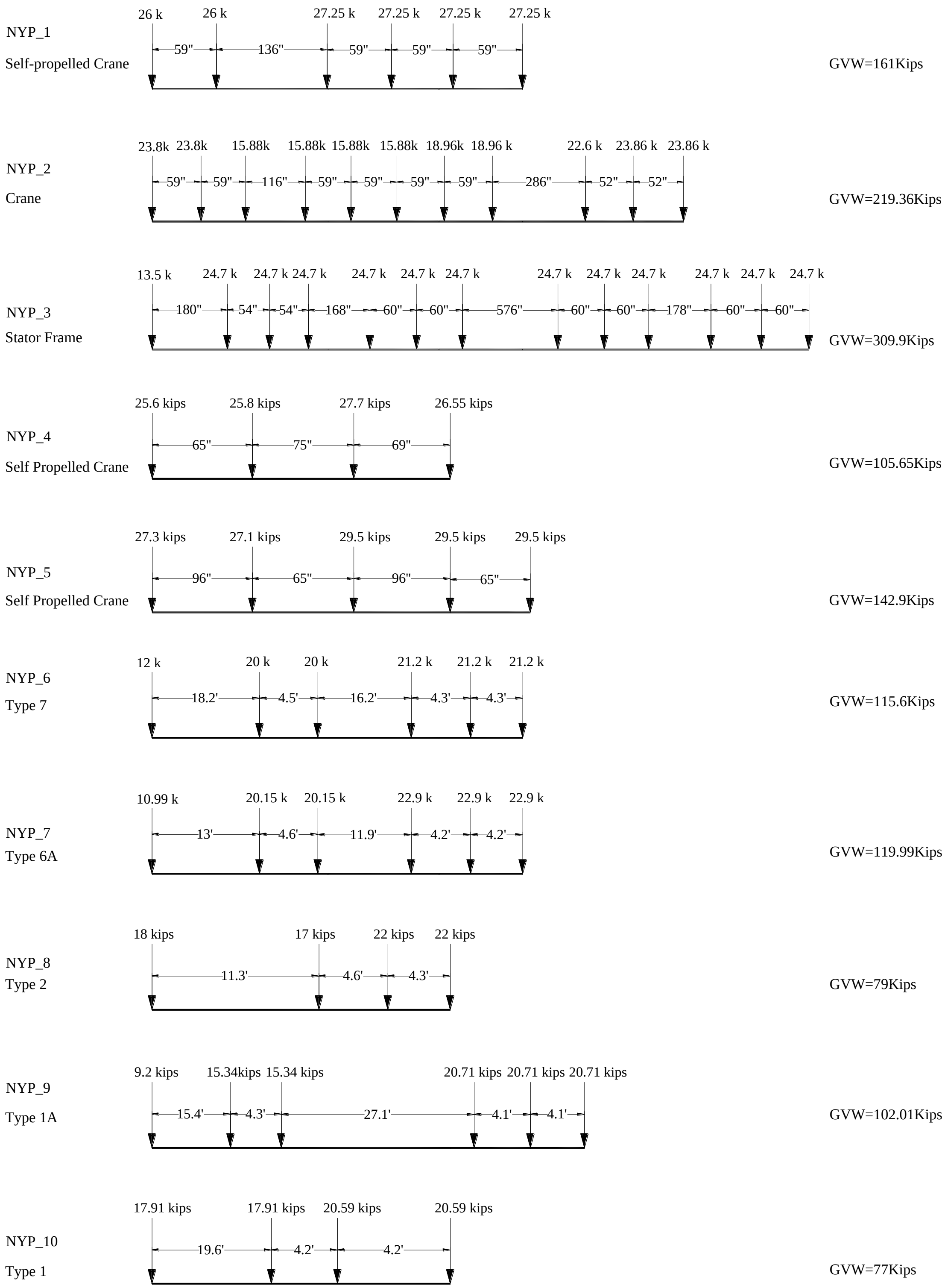


Figure 3.21 Examples of New York State Permit Truck Configuration

Table 3.14. Moment Live Load Effect for Set of New York Permit Trucks on Simple Span bridges

Span Length	NYP-1	NYP-2	NYP-3	NYP-4	NYP-5	NYP-6	NYP-7	NYP-8	NYP-9	NYP-10
	Moment in kip-ft									
40-ft	871	603	661	750	857	547	660	539	536	510
60-ft	1661	1310	1393	1279	1571	1048	1207	934	847	861
100-ft	3271	3094	3010	2335	3000	2151	2407	1724	1767	1631
120-ft	4076	4191	3818	2863	3715	2729	3007	2119	2273	2016
200-ft	7296	8578	9255	4976	6573	5041	5407	3699	4312	3556

In this report, the calculation of the reliability index implicit in using different values of Permit load factor γ_L is executed using the FORM algorithm and the failure function of Eq. (3.1).

Equation 3.3 is used to find the nominal resistance, R_n, required to carry each of the Permit trucks of Figure 3.21 for different permit load factors, such that the Rating Factor is always equal to R.F.=1.0. R_n, is found by assuming that the nominal resistance is obtained using a live load factor γ_L applied on the permit truck effect with the distribution factor for one lane after removing the multiple presence factor MP=1.2.

Given the nominal resistance, the statistical data for the resistance and dead loads are obtained using the models described in Section 2.2 and summarized in Table 3.1.

The live load model is obtained using Eq. (3.18) where P is deterministic but DF and IM are Normal random variables having the statistical data associated with a single truck as shown in Table 3.1.

When the AASHTO LRFR γ_L=1.15 is used to find, R_n, the calculations lead to the reliability index values plotted in Figure 3.22 for all the span lengths, beam spacings and permit trucks considered. The results show that there is no significant change in β for the different Permit Truck types and that the effect of the truck weight and axle spacing is not significant. Also, there is no significant difference due to beam spacing. This is due to the assumption that the AASHTO LRFD lane distribution factors on the average give accurate estimates of the lateral distribution of the load. There is a small decrease in β with span length but overall the range in the reliability index values is small with an average reliability index $\beta_{average}$=2.98 and a minimum value of 2.84 and a maximum value of 3.04.

Hence, the use of a live load factor γ_L=1.15 with Permit trucks that have fixed weights would lead to higher reliability indexes than the target value of β_{target}=2.0 that was set in Section 3.2 and higher than the β_{target}=2.5 used during the AASHTO LRFR calibration. The higher average reliability index is primarily due to the fact that the AASHTO LRFR γ_L=1.15 is a conservative upper limit for a range of values calculated for different permit weights and due to the assumption made by Moses (2001) that the COV of the live load for a single permit crossing remains essentially similar to that for random truck crossings at about V_{LL}=20%. However, in this set of calculations as obtained from Equation 3.19, the COV reduces to 12% for single crossings of a Permit Truck of known weights.

If the live load factor is reduced to γ_L=1.0 for the deterministic single Permit truck, it would lead to an average reliability index of $\beta_{average}$=2.55 with a minimum of 2.22 and a maximum of 2.73 as shown in Figure 3.23. Thus, in this case γ_L=1.0 would still lead to higher reliability index values than the target set in Section 3.2 at β_{target}=2.0. The higher reliability index is due to the safety margins produced by the dead load factors and the high impact value IM=1.33 used when evaluating the safety of the bridge member as compared to the mean value $\overline{IM}$ =1.13.

If a live load factor γ_L=1.10 is used, then the average reliability index becomes $\beta_{average}$ =2.84 with a minimum value of 2.72 and maximum value of 2.90 as illustrated in Table 3.15. The results show the very narrow range in the reliability index value that is obtained for all span lengths, beam spacings and Permit truck configurations and weights.

If the live load factor is kept at γ_L=1.15 but the Permit truck is to cross the bridge at crawl speed so that there is no dynamic amplification for the live load effects, then the COV for the live load effects of Eq. (3.19) reduces to V_{LL}=8% which reflects the uncertainties in the load Distribution Factor D.F. only. In this case, the average reliability index becomes $\beta_{average}$=2.49 with a minimum of 2.10 and a maximum of 2.71 as shown in Figure 3.24. Lower reliability values are obtained than for the case which includes impact because the nominal dynamic amplification factor IM=1.33 recommended by the AASHTO LRFD and LRFR is significantly higher than the average value of 1.13 observed in the field for heavy trucks. The ratio of 1.33/1.13 provides an additional live load safety factor which is removed when a Permit truck travels at crawl speeds.

It is also noted that lowering the live load factor from γ_L=1.15 to γ_L=1.0 or removing the dynamic allowance will reduce the reliability index of the short span bridges by a more significant amount than the longer spans as observed when comparing Figures 3.23 and 3.24 to Figure 3.22. This is because the effect of the dead load is more significant for the longer spans and that the dead loads are not affected by changing the live load factors or the dynamic allowance.

If the analysis is performed for the case where no Dynamic Amplification is assumed along with a Live Load Factor γ_L=1.0, then the average reliability index becomes β=2.11 with a minimum value of 1.38 and a maximum value of 2.60. These results are illustrated Figure 3.25.

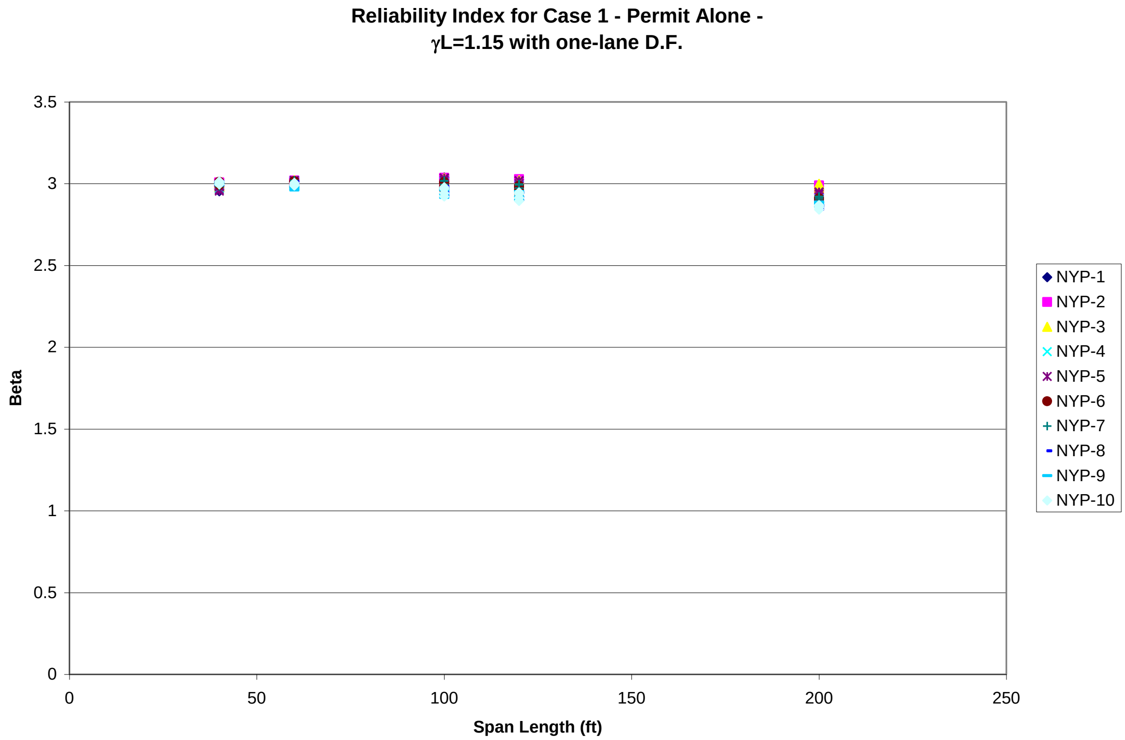


Figure 3.22. Variation of reliability index for AASHTO LRFR escorted Permit crossing with live load factor γ_L=1.15

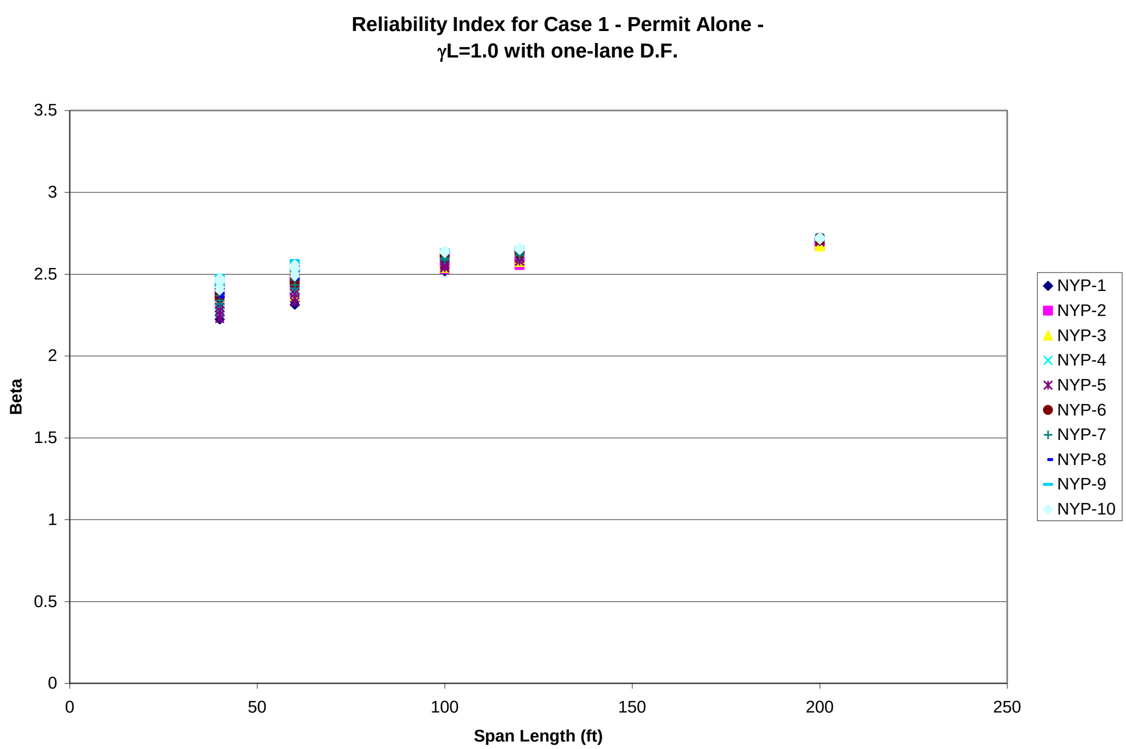


Figure 3.23. Reliability index for AASHTO LRFR escorted Permit crossing with live load factor γ_L=1.00

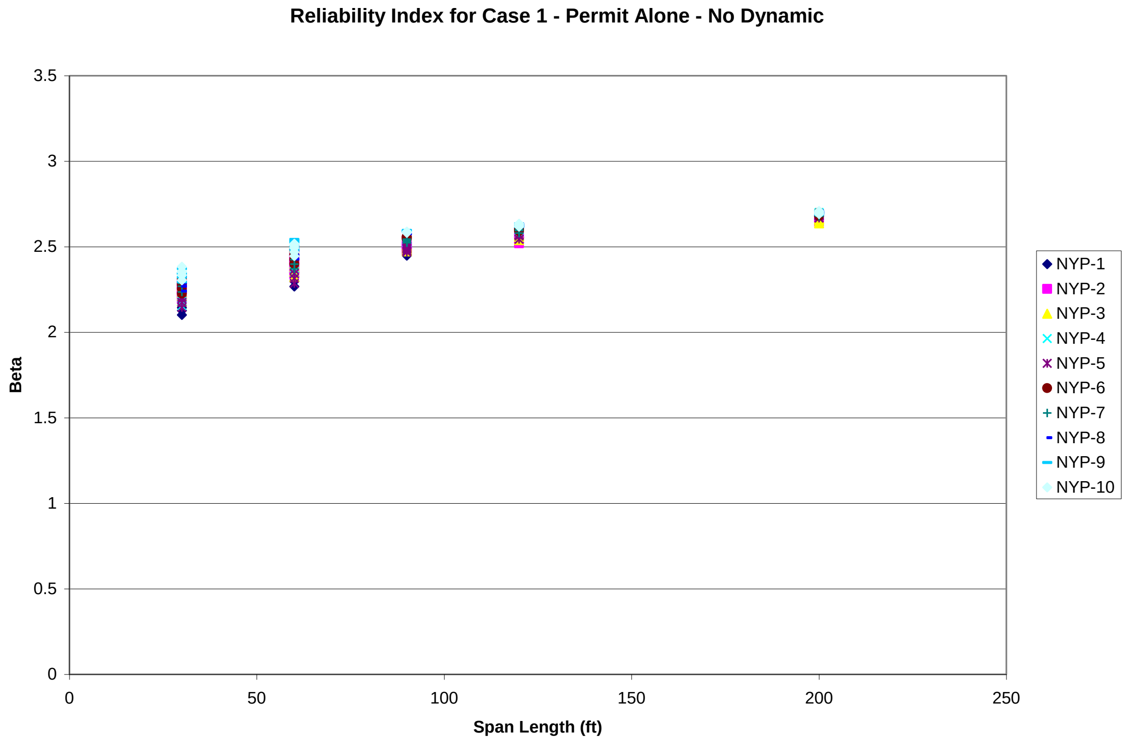


Figure 3.24. Reliability index for AASHTO LRFR escorted Permit crossing at crawl speed with live load factor γ_L=1.15

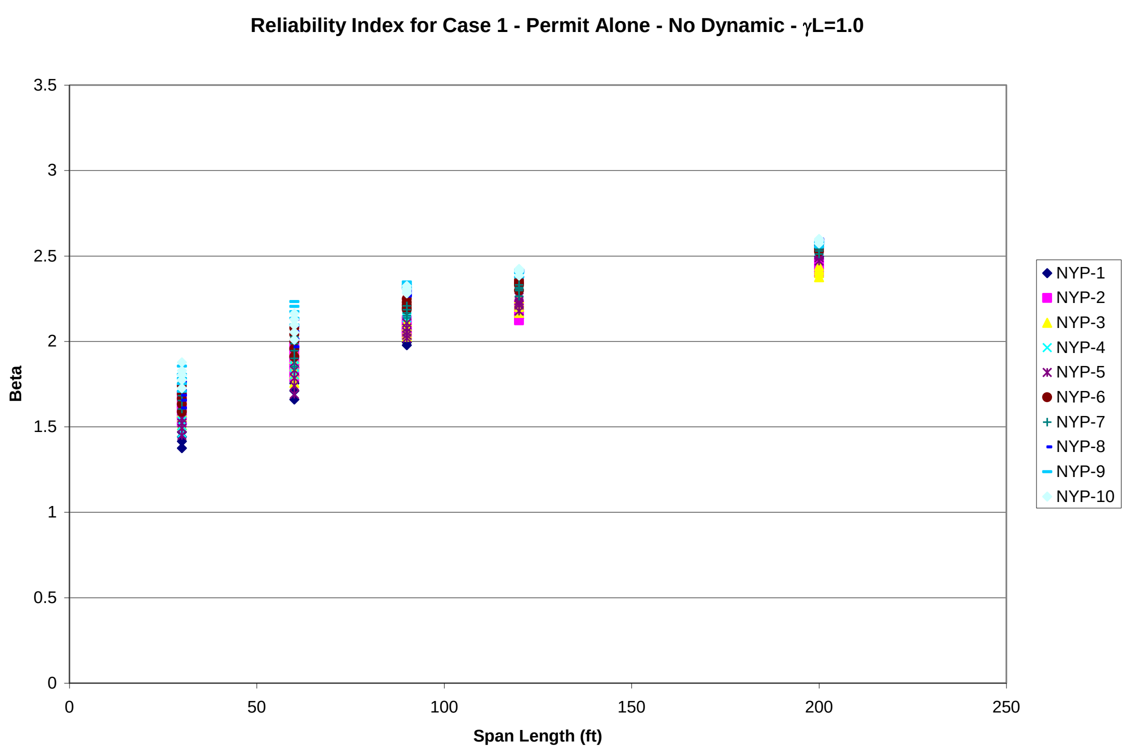


Figure 3.25. Reliability index for AASHTO LRFR escorted Permit crossing at crawl speed with live load factor γ_L=1.00

Table 3.15 Reliability Index for a Non-divisible Load Single Permit with γ_L=1.10

		Reliability Index results from FORM										
Span (ft)	Spacing (ft)	NYP-1	NYP-2	NYP-3	NYP-4	NYP-5	NYP-6	NYP-7	NYP-8	NYP-9	NYP-10	max-min
40	4	2.72	2.77	2.76	2.74	2.72	2.79	2.77	2.79	2.81	2.80	0.09
	6	2.73	2.78	2.77	2.75	2.73	2.79	2.78	2.80	2.81	2.81	0.08
	6	2.75	2.80	2.79	2.77	2.75	2.82	2.80	2.82	2.84	2.83	0.09
	10	2.76	2.81	2.80	2.78	2.76	2.82	2.81	2.82	2.84	2.83	0.08
	12	2.77	2.81	2.80	2.79	2.77	2.82	2.81	2.82	2.83	2.83	0.06
60	4	2.77	2.80	2.79	2.80	2.78	2.82	2.81	2.83	2.84	2.83	0.07
	6	2.78	2.81	2.80	2.81	2.79	2.83	2.82	2.83	2.83	2.84	0.06
	6	2.80	2.83	2.82	2.83	2.81	2.85	2.84	2.85	2.85	2.85	0.05
	10	2.81	2.83	2.83	2.83	2.81	2.84	2.84	2.85	2.84	2.84	0.04
	12	2.82	2.84	2.84	2.84	2.83	2.85	2.85	2.85	2.84	2.85	0.03
100	4	2.87	2.87	2.87	2.88	2.87	2.88	2.88	2.87	2.86	2.87	0.01
	6	2.87	2.87	2.88	2.88	2.88	2.88	2.88	2.86	2.86	2.86	0.02
	6	2.88	2.88	2.88	2.88	2.88	2.88	2.88	2.86	2.86	2.86	0.02
	10	2.87	2.87	2.87	2.87	2.87	2.86	2.87	2.85	2.84	2.84	0.03
	12	2.86	2.86	2.86	2.85	2.86	2.85	2.85	2.83	2.82	2.82	0.04
120	4	2.87	2.87	2.88	2.87	2.88	2.87	2.87	2.85	2.85	2.85	0.02
	6	2.88	2.88	2.88	2.87	2.88	2.87	2.87	2.85	2.85	2.85	0.03
	6	2.88	2.88	2.88	2.87	2.88	2.87	2.88	2.85	2.85	2.85	0.03
	10	2.87	2.87	2.87	2.86	2.87	2.85	2.86	2.83	2.83	2.83	0.04
	12	2.87	2.87	2.86	2.84	2.86	2.84	2.85	2.82	2.82	2.81	0.05
200	4	2.87	2.88	2.89	2.84	2.86	2.84	2.85	2.81	2.83	2.81	0.07
	6	2.88	2.89	2.90	2.85	2.87	2.85	2.85	2.82	2.83	2.82	0.08
	6	2.88	2.89	2.90	2.85	2.87	2.85	2.86	2.82	2.83	2.82	0.07
	10	2.87	2.88	2.89	2.84	2.86	2.84	2.84	2.81	2.82	2.81	0.08
	12	2.86	2.87	2.88	2.83	2.85	2.83	2.83	2.80	2.81	2.80	0.08
average		2.83	2.85	2.85	2.83	2.83	2.84	2.84	2.83	2.84	2.83	

Summary and Recommendation for Non-divisible Loads

The calculations executed in this section, demonstrate that the reliability index remains on the average higher than the target β_{target}=2.0 even when a live load factor γ_L=1.0 is used for the case when a single permit truck is alone on the bridge assuming that the permit truck weights are exactly known. However, it is not deemed reasonable to not use any live load factor as that may be perceived as implying that no uncertainties are involved in estimating the load effect of a permit truck. For this reason, it is herein recommended that the single crossing of a non-divisible load Permit should be associated with a minimum live load factor γ_L=1.10. This live load factor would be used for single lane bridges when the truck has acquired a non-divisible load permit and the weight is controlled to avoid any changes from the permitted weights. Also, this γ_L=1.10 should be used as a minimum value for permits on multi-lane bridges. This minimum live load factor may have to be exceeded if the case of a permit truck alongside random trucks may govern as will be examined in the remainder of this report. For the case where the Permit truck travels at crawl speed, it is recommended to apply a live load factor γ_L=1.05 in order to ensure that the reliability index does not fall below a minimum value β_{min}=1.5.

Random Weight Divisible Load Permits

As mentioned earlier, in the case where we have multi-crossings of divisible loads, it is possible to find that some of the Permitted trucks may have gross weights exceeding the permit limit. In these cases, the Permit truck load effect, P, in Equation 3.18 cannot be assumed to be deterministic. As observed from the data collected at WIM site 8280, the permit truck gross weight may be assumed to follow an exponential distribution where P is the effect of a shifted gross vehicle weight distribution with parameter λ=0.15. The shift corresponds to the weight of the permit truck. The mean value of P can be represented as given in Eq. (3.16) and the standard deviation as given in Eq. (3.17).

In this case, the reliability index for Case I for a permit alone on the bridge is repeated when the nominal resistance, R_n, is calculated using a live load factor γ_L=1.10 applied on the permit load effect with the one lane distribution factor after removing the multiple presence factor MP=1.2. The reliability index would decrease so that the average reliability index becomes $\beta_{average}$=2.68 with a minimum of 2.32 and a maximum value of 2.86 as shown in Figure 3.26. These values are compared to $\beta_{average}$ =2.84 with a minimum value of 2.72 and maximum value of 2.90 for the case when the Permit load effect is assumed to be deterministic. Looking only at the divisible load Permit trucks labeled NYP-6 through NYP-10 in Figure 3.21, the average reliability index for these cases is $\beta_{average}$=2.66 with a minimum of 2.32 and a maximum value of 2.80. This set of calculations demonstrates that a live load factor γ_L=1.10 would still provide reliability levels above β_{target}=2.0 for the case when a permit truck may cross a bridge alone even when accounting for the possibility that some Permit trucks may exceed their weight limits.

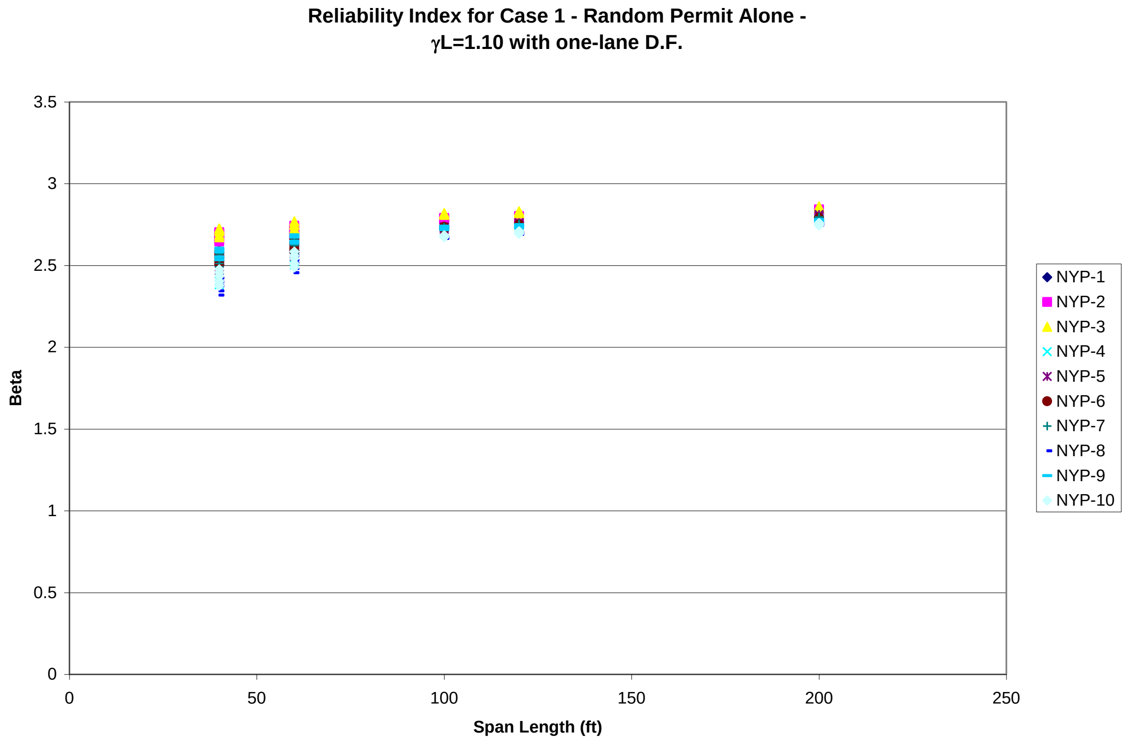


Figure 3.26. Reliability index for Random Permits crossing with live load factor γ_L=1.10

Summary and Recommendation for Divisible Loads

The calculations executed in this section, demonstrate that the reliability index remains above the target β_{target}=2.0 for the cases of divisible loads with random weights when a live load factor γ_L=1.10 is used for checking the safety of one lane bridges analyzed using the AASHTO LRFD distribution factors after removing the multiple presence factor of MP=1.2.

Reliability Analysis for Case II – Two Permits side-by-side

Deterministic Non-divisible Loads

In this case, we start by assuming that the axle weights and axle configurations of the two permit trucks are the same and are perfectly known so that the total maximum static live load effect on the bridge 2P is a deterministic value. However, as was the situation with Case I, even having deterministic weights for the permits does not imply that the total live load effect on a bridge member is deterministic due to the uncertainties in estimating the dynamic effect represented by the dynamic amplification factor, IM, and the uncertainties in the structural analysis process that allocates the fraction of the total load to the most critical member as represented by the load distribution factor, D.F.

The AASHTO LRFR considers two Permit Categories which may possibly involve two permits trucks side-by-side. These are the Routine/Annual permit or the Special/Limited Multiple Permit trips with less than 100 Permit Crossings within the evaluation period. As per the suggestion of the NYSDOT Technical Working Group, in this project we will only consider the case of unlimited crossings. For this purpose, we will assume that there will be up to 100 crossings of a particular type of Permit trucks in one day over each analyzed bridge. According to the New York WIM data, this will mean that the probability of having two side-by-side Permits is 0.50%.

For the case of unlimited crossings, we will calculate the required nominal resistance, R_n, using the two-lane distribution factor as recommended in the AASHTO LRFR. The D.F. of multi-girder bridges loaded in two lanes given in the AASHTO LRFD specifications assume that the two lanes are loaded by the same vehicle and give the load on the most critical beam as a function of the load in one of the lanes which in this case is represented by the load effect of the permit vehicle, P. The mean and COV of the resistance which is assumed to follow a lognormal probability distribution are obtained as explained in Section 2.2 and summarized in Table 3.1. The statistical data for the dead loads are also provided in Section 2.2 and summarized in Table 3.1.

For two lanes loaded by the same permit truck, the mean value of the live load effect on one member can be given as:

$$\overline{LL} = \overline{P} \times \overline{IM} \times \overline{D.F.} \tag{3.20}$$

In the case of non-divisible loads, P is assumed to be deterministic such that the mean permit load effect is equal to the effect of the permit truck with the assigned weight $\overline{P} = P$. According to Nowak (1999), the dynamic amplification augments the truck load effect by an average of 9% for heavy side-by-side trucks or $\overline{IM}$ =1.10. The dynamic amplification also produces a Coefficient of Variation COV equal to V_{IM}=5.5%.

As per the single lane case, Nowak (1999) also assumes that the equations for obtaining the load distribution factors for multi-lanes given in the LRFD specifications produce values that are on the average equal to the actual values. We will also assume that the same COV for the lane distribution factor V_{DF}=8% obtained by Moses & Ghosn (1986) from field measurements on typical steel and prestressed concrete bridges is still valid. Therefore, for the loading of a single permit vehicle, the live load COV becomes $V_{LL} = \sqrt{(5.5\%)^2 + (8\%)^2} = 9.71\%$. It is noted that this V_{LL} value is lower than that obtained for a single lane and also significantly lower than the COV value used in the AASHTO LRFD for random truck crossings which was also implicitly adopted during the calibration of the AASHTO LRFR.

The reliability index conditional on the arrival of two side-by-side permits on the bridge can then be calculated using the FORM algorithm and the failure function given in Eq. (3.1). The reliability index calculated from the FORM algorithm where the live load is described by Eq. (3.20) is designated as β_c which is defined as the reliability index conditional on having two side-by-side trucks.

The probability that a bridge member would fail given that two permit vehicles are side-by-side can be calculated from:

$$P_{f|side-by-side} = \Phi(-\beta_C) \tag{3.21}$$

where $\Phi(...)$ is the cumulative standard Normal distribution function. The final unconditional probability of failure will depend on the conditional probability of failure given two side-by-side permit events, $P_{f|side-by-side}$ and the probability of having a situation with side-by-side permits, P_{SxS}. Thus:

$$P_f = P_{f|side-by-side} \times P_{S\times S} \tag{3.22}$$

where the probability of having two-side-by-side permits P_{SxS} is related to the total number of permit crossings. The probability of two permits side-by-side is not affected by the ADTT of the site but it rather depends on the number of permits per day. In these calculations we will assume P_{SxS}=0.5% for up to 100 independent crossings of Permits in one day at one bridge site as obtained as an upper limit from Sivakumar et al (2008).

The final unconditional reliability index, β, is obtained from:

$$\beta = -\Phi^{-1}(P_f) \tag{3.23}$$

The reliability calculations are performed using the FORM algorithm assuming that a Permit load factor γ_L =1.10 is used for finding the nominal resistance R_n. The reliability analysis indicates that, as observed in Case I, the weights of the Permits do not affect the reliability index. The conditional reliability index values are shown in Table 3.16. The final unconditional reliability index values for all the span lengths and beam spacing of the composite steel bridges are provided in Table 3.17 for the bridge configurations analyzed in this example.

Table 3.16 shows that the average conditional reliability index is $\beta_{average}$=3.09 with a minimum value of 2.93 and a maximum value of 3.14. These results are plotted in Figure 3.27 showing little difference in the reliability index for the different permit trucks with a slight decrease in β as the span length increases.

The average unconditional reliability index is 4.41 with a minimum value of 4.30 and a maximum value of 4.45 as shown in Figure 3.28 and Table 3.17. The higher reliability levels observed for the unconditional case are clearly due to the low probability of having two Permits side-by-side when 100 Permits are independently crossing the same bridge each day.

The results of the calculations performed in this section illustrate the following points:

- The difference in the average reliability index for the most unconservative AASHTO LRFR live load factor of γ_L=1.10 between the unconditional and conditional situation is significant with $\beta_{average}$ increasing from about 3.10 to 4.4. This increase is due to the very low probability of having two side-side permits in situations where up to 100 permits independently cross the same bridge in one day. The assumptions made are that the crossings of the permit trucks over a given bridge are totally random and no clumping of permit trucks is expected but that the weights of the permits are deterministic.
- Even if two permits happen to cross the bridge side-by-side, the average conditional reliability index of $\beta_{average}$=3.09 is significantly higher than the target β_{target}=2.0 set for the load rating of New York State bridges under random traffic loads. This indicates that the live load factor γ_L=1.10 of the AASHTO LRFR is conservative for side-by-side permits.
- According to the observations made in the previous bullets, the case of two side-by-side permits of the same type is not expected to control the safety of a bridge system. However, it is possible that the case of a permit alongside a heavy random truck may control as will be discussed in the next section.
- For a given span length and beam spacing, the different vehicle configurations produce little change in the reliability index. The largest difference in the unconditional β being on the order of 0.11 for the 200-ft span bridge at 12-ft spacing.
- Changing the beam spacing for a given span length leads to insignificant changes in the reliability index values obtained for a given truck configuration. This is because it is assumed that the distribution factors of the LRFD specifications lead to values which are equal to the average actual distribution factors.
- Increasing the span length leads to a slight decrease in the conditional reliability index values from an average of 3.12 for the 40-ft span to 3.01 for the 200-ft span.

- The average unconditional reliability index for the span lengths and beam spacings considered is on the order of β_{ave}=4.41 with a minimum value of β=4.30 and a maximum value β=4.45
- The higher reliability index obtained for the two side-by-side permits with β_{ave}=4.41 as compared to the single permit with β_{ave}=3.09 is primarily due to the low probability of having side-by-side permit trucks. If one looks at the conditional reliability index, then the average $\beta_{conditional}$=3.09 is closer to but still higher than the β_{ave}=2.84 obtained for a single permit truck analyzed under Case I with γ_L=1.10. In Case II, a higher conditional reliability index value is obtained due to the lower mean impact factor ($\overline{IM}$ =1.09 versus 1.13) and the lower corresponding COV (V_{IM}=5.5% versus 9%) for side-by-side events which are justified by the low likelihood of having the peaks of the dynamic oscillations of the two side-by-side vehicles occur simultaneously.

Random Permits Side-by-Side

In the case where some of the divisible load permits are overloaded such that the gross weights of the overloaded permits follow a shifted exponential probability distribution with parameter λ=0.15, the conditional reliability index obtained with γ_L=1.10 will be lower than observed for the deterministic loads. If all the trucks in Figure 3.21 are assumed to be carrying random divisible weights, the average conditional reliability index is obtained as $\beta_{average}$=2.91 with a minimum value of 2.68 and maximum value of 3.06. If only the trucks labeled NYP-6 to NYP-10 are considered, then the average drops to $\beta_{average}$=2.86 with a minimum value of 2.68 and maximum value of 2.94 as plotted in Figure 3.29. These values are compared to the values observed for the deterministic permits where the conditional $\beta_{average}$=3.09 with a minimum value of 2.93 and maximum value of 3.14.

The unconditional reliability index values for NYP-6 to NYP-10 remain high at an average value $\beta_{average}$=4.25 with a minimum value of 4.12 and maximum value of 4.31. In these calculations we assume that the weights of the two permit trucks are uncorrelated although the side-by-side Permit trucks are always taken to be of the same type. The possibility of have permit trucks of different types cross a bridge side-by-side is considered using the Permit alongside a random truck as will be discussed next.

Summary and Recommendation

In summary the use of live load factor of γ_L=1.10 would lead to relatively high reliability levels that exceed the target β_{target}=2.0 for the cases when side-by-side permits could occur even if the two permit weights are assumed to be random to account for the possibility of having overloaded permit trucks.

Table 3.16. Conditional Reliability Index for Case II

SPAN (ft)	Spacing (ft)	NYP-1	NYP-2	NYP-3	NYP-4	NYP-5	NYP-6	NYP-7	NYP-8	NYP-9	NYP-10	max-min
40	4	3.103	3.117	3.115	3.11	3.103	3.118	3.116	3.119	3.115	3.117	0.02
	6	3.103	3.116	3.114	3.11	3.104	3.117	3.115	3.117	3.112	3.115	0.01
	8	3.107	3.123	3.121	3.115	3.108	3.125	3.122	3.125	3.121	3.124	0.02
	10	3.11	3.124	3.122	3.118	3.111	3.124	3.123	3.124	3.118	3.122	0.01
	12	3.113	3.122	3.121	3.119	3.114	3.121	3.122	3.12	3.11	3.114	0.01
60	4	3.116	3.122	3.122	3.123	3.118	3.12	3.122	3.116	3.094	3.111	0.03
	6	3.117	3.123	3.122	3.123	3.119	3.12	3.122	3.115	3.091	3.109	0.03
	8	3.123	3.129	3.128	3.129	3.125	3.125	3.128	3.12	3.096	3.114	0.03
	10	3.123	3.126	3.127	3.126	3.125	3.12	3.124	3.113	3.085	3.106	0.04
	12	3.126	3.126	3.127	3.125	3.127	3.117	3.122	3.108	3.075	3.1	0.05
100	4	3.126	3.125	3.124	3.107	3.123	3.099	3.109	3.072	3.061	3.066	0.06
	6	3.137	3.135	3.134	3.116	3.134	3.107	3.117	3.077	3.066	3.071	0.07
	8	3.141	3.139	3.137	3.119	3.137	3.11	3.121	3.081	3.069	3.074	0.07
	10	3.129	3.127	3.125	3.103	3.125	3.093	3.105	3.061	3.049	3.055	0.08
	12	3.124	3.12	3.118	3.092	3.118	3.081	3.095	3.046	3.033	3.039	0.09
120	4	3.128	3.13	3.123	3.094	3.121	3.087	3.099	3.048	3.048	3.043	0.09
	6	3.127	3.128	3.122	3.092	3.12	3.085	3.097	3.046	3.046	3.041	0.09
	8	3.131	3.132	3.126	3.095	3.124	3.089	3.101	3.049	3.049	3.044	0.09
	10	3.125	3.127	3.119	3.084	3.117	3.077	3.09	3.034	3.034	3.028	0.10
	12	3.113	3.116	3.107	3.068	3.103	3.06	3.075	3.015	3.015	3.009	0.11
200	4	3.051	3.076	3.086	2.987	3.034	2.989	3.002	2.936	2.959	2.934	0.15
	6	3.061	3.086	3.096	2.996	3.044	2.998	3.011	2.945	2.968	2.944	0.15
	8	3.065	3.089	3.099	3.001	3.048	3.003	3.016	2.951	2.973	2.949	0.15
	10	3.059	3.084	3.095	2.993	3.042	2.995	3.008	2.942	2.965	2.94	0.16
	12	3.045	3.072	3.083	2.978	3.027	2.98	2.993	2.927	2.95	2.925	0.16
Average		3.11	3.12	3.12	3.08	3.10	3.08	3.09	3.06	3.05	3.05	

Table 3.17. Unconditional Reliability Index for Case II

Unconditional beta results from FORM												
SPAN (ft)	Spacing (ft)	NYP-1	NYP-2	NYP-3	NYP-4	NYP-5	NYP-6	NYP-7	NYP-8	NYP-9	NYP-10	max-min
40	4	4.43	4.44	4.44	4.43	4.43	4.44	4.44	4.44	4.44	4.44	0.01
	6	4.43	4.44	4.43	4.43	4.43	4.44	4.44	4.44	4.43	4.44	0.01
	8	4.43	4.44	4.44	4.44	4.43	4.44	4.44	4.44	4.44	4.44	0.01
	10	4.43	4.44	4.44	4.44	4.43	4.44	4.44	4.44	4.44	4.44	0.01
	12	4.43	4.44	4.44	4.44	4.43	4.44	4.44	4.44	4.43	4.43	0.01
60	4	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.42	4.43	0.02
	6	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.42	4.43	0.02
	8	4.44	4.45	4.44	4.45	4.44	4.44	4.44	4.44	4.42	4.43	0.02
	10	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.43	4.41	4.43	0.03
	12	4.44	4.44	4.44	4.44	4.44	4.44	4.44	4.43	4.41	4.42	0.04
100	4	4.44	4.44	4.44	4.43	4.44	4.42	4.43	4.40	4.40	4.40	0.05
	6	4.45	4.45	4.45	4.44	4.45	4.43	4.44	4.41	4.40	4.40	0.05
	8	4.45	4.45	4.45	4.44	4.45	4.43	4.44	4.41	4.40	4.41	0.05
	10	4.45	4.44	4.44	4.43	4.44	4.42	4.43	4.40	4.39	4.39	0.06
	12	4.44	4.44	4.44	4.42	4.44	4.41	4.42	4.39	4.38	4.38	0.07
120	4	4.44	4.45	4.44	4.42	4.44	4.41	4.42	4.39	4.39	4.38	0.06
	6	4.44	4.44	4.44	4.42	4.44	4.41	4.42	4.39	4.39	4.38	0.06
	8	4.45	4.45	4.44	4.42	4.44	4.42	4.43	4.39	4.39	4.38	0.06
	10	4.44	4.44	4.44	4.41	4.44	4.41	4.42	4.38	4.38	4.37	0.07
	12	4.43	4.44	4.43	4.40	4.43	4.40	4.41	4.36	4.36	4.36	0.08
200	4	4.39	4.41	4.41	4.34	4.38	4.34	4.35	4.31	4.32	4.30	0.11
	6	4.40	4.41	4.42	4.35	4.38	4.35	4.36	4.31	4.33	4.31	0.11
	8	4.40	4.42	4.42	4.35	4.39	4.35	4.36	4.32	4.33	4.32	0.11
	10	4.39	4.41	4.42	4.35	4.38	4.35	4.36	4.31	4.33	4.31	0.11
	12	4.38	4.40	4.41	4.34	4.37	4.34	4.35	4.30	4.32	4.30	0.11
Average		4.43	4.44	4.44	4.41	4.43	4.41	4.42	4.39	4.39	4.39	

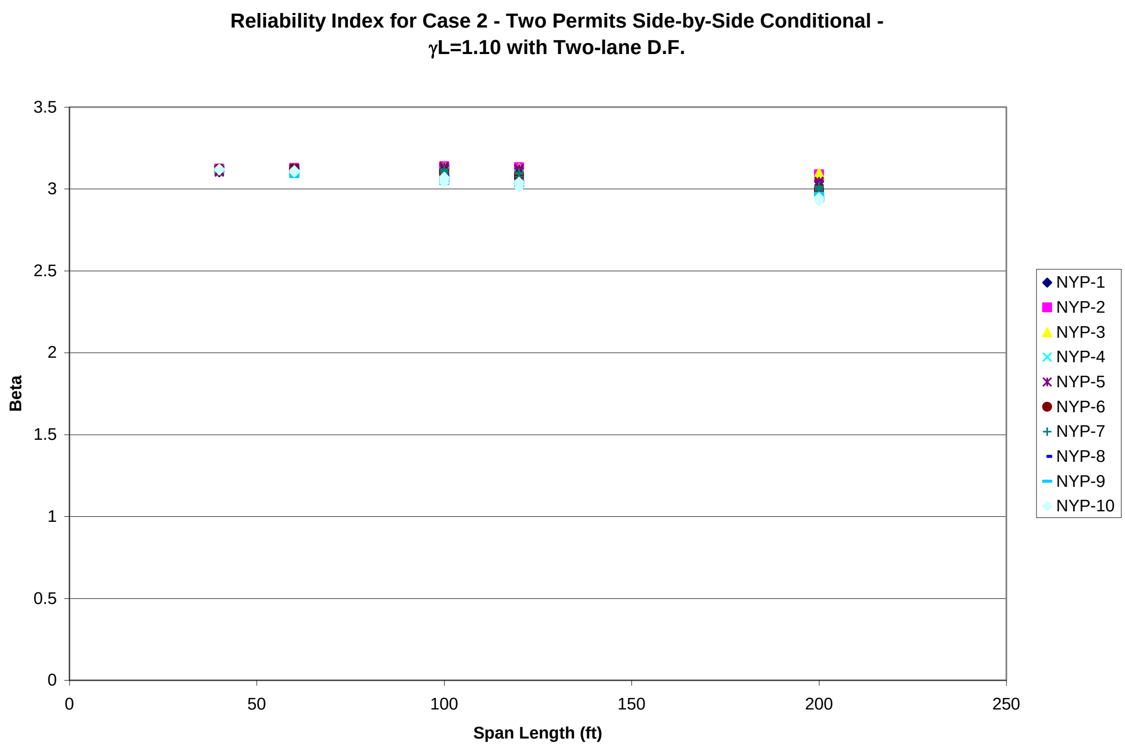


Figure 3.27 Conditional Reliability Index for Case II for Routine Permits

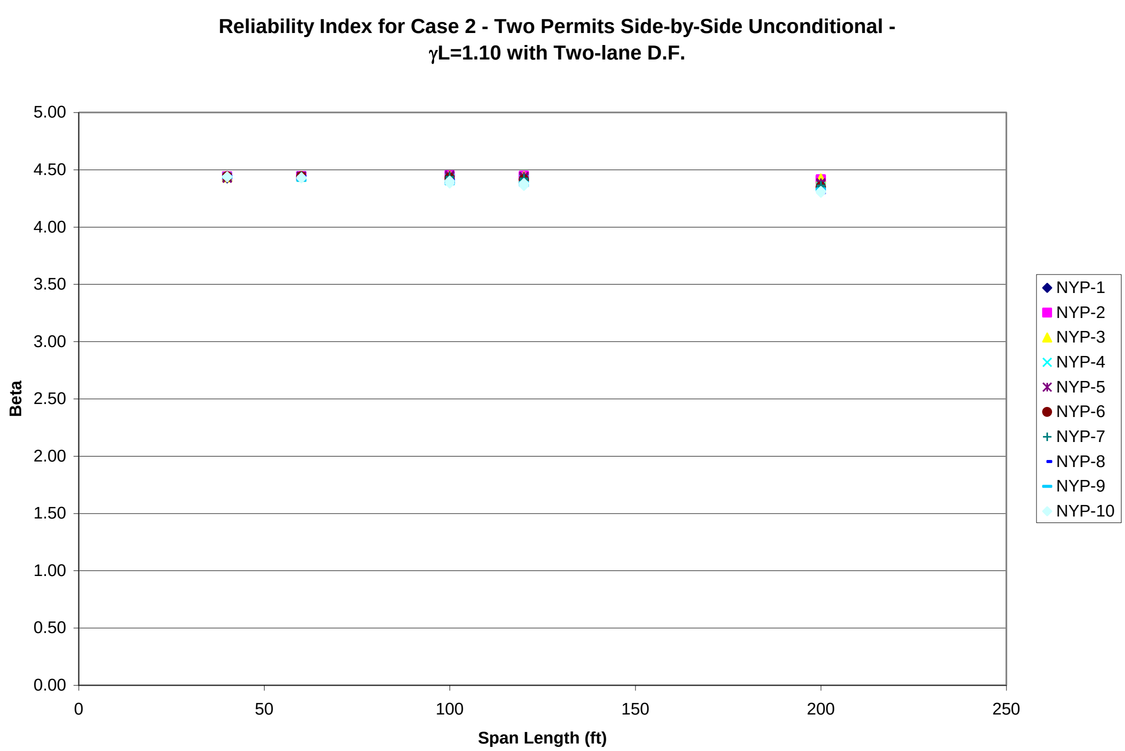


Figure3.28. Unconditional Reliability Index for Case II for Routine Permits

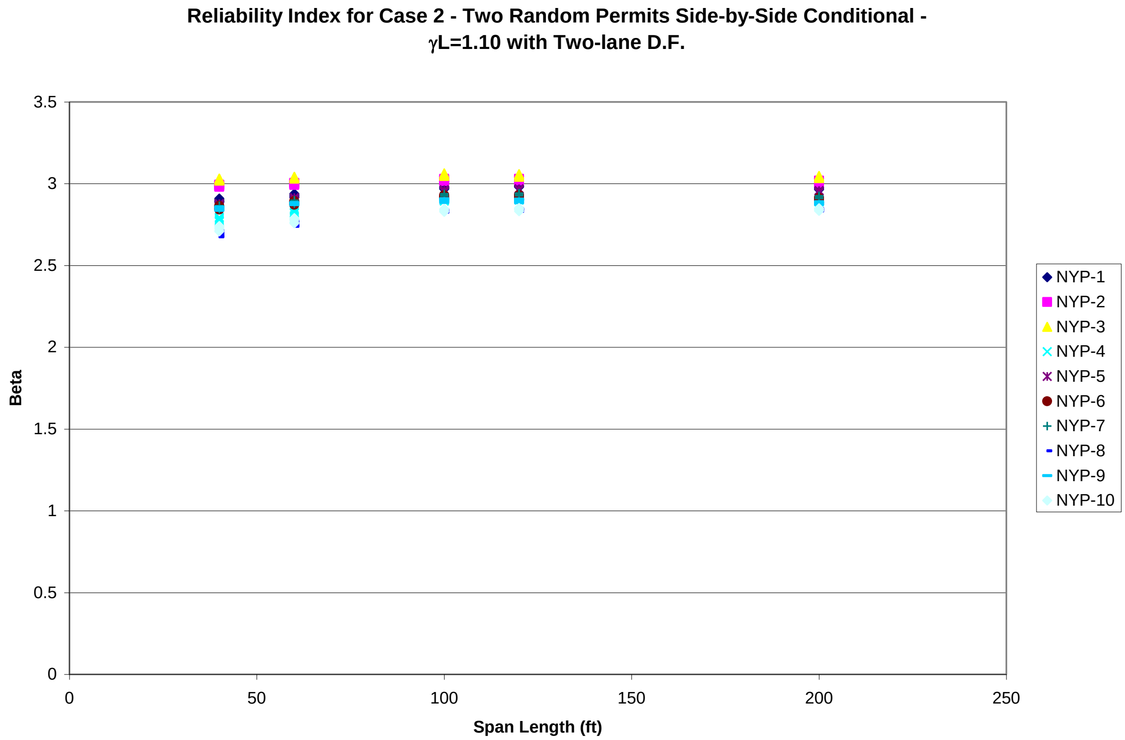


Figure 3.29. Reliability index for conditional two random permits side-by-side.

Reliability Analysis for Case III – Permit Truck alongside Random Truck for Unlimited Permit Crossings.

Load Modeling

For this case, we need to analyze the maximum live load effect that is due to the permit truck alongside the maximum truck expected to occur simultaneously in the other lane. The maximum total load effect is random and it depends on the number of side-by-side events expected within the return period. As mentioned earlier, a five-year return period is used in this study following the recommendation of Moses (2001).

To determine the number of side-by-side permit-random truck events that would occur within the five-year rating period, we will assume that the number of side-by-side events involving one random truck will depend on the ADTT. Based on the upper envelopes of the New York State WIM data, sites with ADTT=100 will have 0.5% of the loading events formed by side-by-side trucks. Sites with ADTT=1000 and 5000 will have 1.25% and 2% probability of side-by-side trucks respectively.

In this report, the estimation of the load effect of the random trucks that will cross a bridge alongside a Permit truck are calculated using the actual live load effects of the trucks recorded by WIM. An example histogram of the moment load effects of trucks moving in one lane of a 100-ft simple span bridge was provided in Figure 2.3 based on the data collected at WIM site 0199. The load effects are normalized by dividing the actual moments by the moment of the HL-93 live load.

As explained in Chapter 2, the histogram of the load effect in a single lane and its cumulative distribution can be used to find the cumulative distribution of the maximum load effect in any given rating period. To find the cumulative distribution for the maximum loading event in a return period of time T, we have to start by estimating the number of loading events, N, that occur during this period of time T. The number of events N is obtained from the ADTT and the WIM headway data at a site.

We define N_p as the number of events where a Permit will cross the bridge alongside a random truck within the five-year rating period, T. The percentage of side-by-side events involving a random truck is P_{sxs}. Thus, within a return period, T, there will N_R crossings of Permits alongside a random truck:

$$N_R = P_{S\times S} \times N_P \tag{3.24}$$

The percentage of side-by-side events, P_{SxS}, depends on the ADTT. As an example, for a bridge site with ADTT=5000 and 100 permits per day the number of permit crossings will be N_p= 182,500 (100 truck per day x 365 days per year x 5 years). Given a probability of side-by-side events P_{sxs}=2% for sites with ADTT=5000, the number of events when the permit truck will be alongside a random truck within a 5-year rating period is N_R=3650 (182,500 x 2%).

For low values of N_R, the cumulative distribution of the maximum load effect for the random truck that may cross alongside a Permit may be obtained directly from Eq. (2.14). As N_R increases, Eq. (2.28) needs to be used along with the most probable value u_N and the inverse dispersion coefficient α_N given in Eq. (2.22) and (2.23). Note that it is recommended to use Eq. (2.14) for N_R values less than 10 and use Eq. (2.22) to (2.28) for higher N_R values. Figure 3.30 shows examples of cumulative probability distributions for different values of N_R. The plot shows how the cumulative distribution shifts to the right as N_R increases resulting in higher mean values $\bar{L}_{max}$ for the random truck that will cross alongside the Permit.

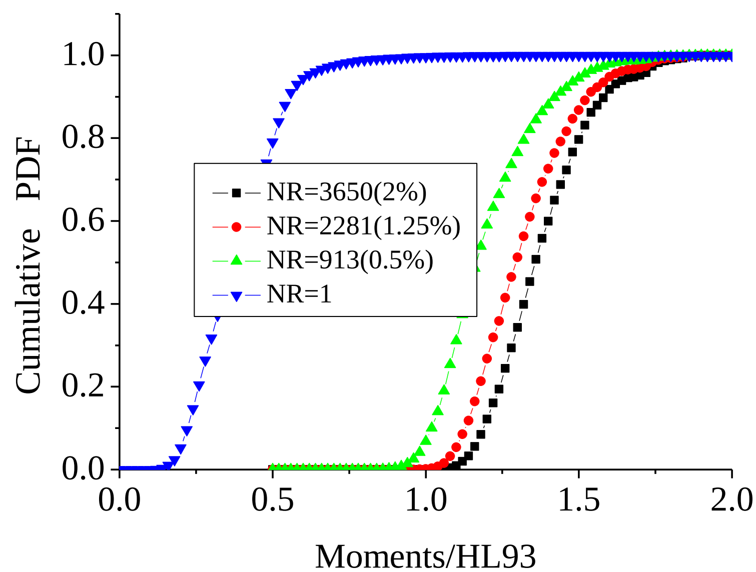


Figure 3.31. Plot of cumulative distribution for different number of events.

The application of Eq. (22) through (28) is executed for the WIM data from all ten New York WIM sites. Table 3.18 shows the results for the mean of the maximum load effect, $\bar{L}_{max}$, the standard deviation, σ_{Lmax}, and the COV, V_{Lmax}, for the case when the permits are crossing a bridge with ADTT=5000 where the probability of side-by-side events is P_{sxs}=2%. For example, the Table shows that WIM site 0199 would produce an average value for the maximum normalized moment effect for the 100-ft span equal to $\bar{L}_{max}$=1.40.

Table 3.19 gives a summary of the results for each of the ADTT considered. The summary gives the overall average of all the values represented by $\bar{L}_{max}$, the average of the COV's of each L_{max} given as V_{Lmax}, the variability of $\bar{L}_{max}$ from site to site expressed in terms of $V_{site\text{-}to\text{-}site}$ which is the COV of all the $\bar{L}_{max}$ values obtained from the ten WIM sites. These are provided for the cases where the random truck traffic volume is considered to be ADTT=5000 with 2% probability of side-by-side events as well as sites with ADTT=1000 with P_{sxs}=1.25% and ADTT=100 with P_{sxs}=100. For example, $\bar{L}_{max}$ varies between 1.49 to 1.15 depending on the span length for the sites with ADTT=5000. The COV for variations in L_{max} within a site is 9% while the site-to-site variability in L_{max} is 11%.

It is further noted that the analysis of the data performed in NCHRP 12-76 demonstrates that the WIM data sample size will result in a variability in the estimated value of L_{max} for each site that may be expressed by a coefficient of variation $V_{sample\ size}$=2%. Table 3.20 gives the moment load effect L_{max} HL93 which provide the expected maximum moment of the random truck that will run along the Permit truck.

The COV data provided in Table 3.19 is used to find the level of uncertainty associated with estimating the effect of the random truck that may cross the bridge alongside the Permit. Given that the analysis of the static load effect for one beam is associated with a COV V_{DF}=8%, the overall coefficient of variation for the static random truck load effect $L^*_{max} = L_{max\ N_R} \times HL93 \times DF_R$ without the dynamic effect can then be estimated from:

$$\boldsymbol{V_{L\max^*} = \sqrt{V^2_{L\max} + V^2_{site\ to\ site} + V^2_{samplesize} + V^2_{DF}}} \tag{3.25}$$

Based on the data in Table 3.19, V_{Lmax*} is obtained as $V_{L\max^*} = \sqrt{V^2_{L\max} + V^2_{site\ to\ site} + V^2_{samplesize} + V^2_{DF}} = \sqrt{(9\%)^2 + (11\%)^2 + (2\%)^2 + (8\%)^2} = 16.4\%$. If the dynamic effects are included, the $\boldsymbol{V_{L\max}}$ will be close to 19%. However, this COV is for the effect of the random truck. Given that the Permit load effect is better known, the overall COV for the total load effect will be lower.

Table 3.18. L_{max} for sites with ADTT=5000 where P_{sxs}=2%

ADTT=5000 P_{sxs}=2%		direction 1			direction 2		
site	Span	mean	Standard deviation	COV	mean	Standard deviation	COV
NY0199	40 ft	1.64	0.17	0.10	1.71	0.17	0.10
	60 ft	1.50	0.16	0.10	1.53	0.15	0.10
	100 ft	1.40	0.14	0.10	1.41	0.13	0.09
	120 ft	1.35	0.13	0.10	1.38	0.12	0.09
	200 ft	1.17	0.11	0.09	1.22	0.11	0.09
NY2680	40 ft	1.41	0.09	0.06	1.60	0.12	0.07
	60 ft	1.32	0.09	0.07	1.46	0.10	0.07
	100 ft	1.25	0.09	0.07	1.40	0.10	0.07
	120 ft	1.35	0.13	0.10	1.36	0.10	0.07
	200 ft	1.06	0.07	0.07	1.20	0.09	0.07
NY8280	40 ft	1.70	0.17	0.10	1.22	0.14	0.11
	60 ft	1.61	0.16	0.10	1.11	0.13	0.11
	100 ft	1.55	0.15	0.09	1.06	0.12	0.11
	120 ft	1.35	0.13	0.10	1.03	0.11	0.10
	200 ft	1.29	0.11	0.08	0.93	0.10	0.10
NY9121	40 ft	1.47	0.13	0.09	1.26	0.11	0.09
	60 ft	1.27	0.11	0.08	1.13	0.10	0.08
	100 ft	1.30	0.11	0.08	1.14	0.10	0.08
	120 ft	1.30	0.11	0.08	1.13	0.10	0.09
	200 ft	1.20	0.10	0.08	1.01	0.09	0.09
NY9631	40 ft	1.34	0.15	0.11	1.59	0.14	0.09
	60 ft	1.20	0.12	0.10	1.45	0.13	0.09
	100 ft	1.23	0.13	0.10	1.45	0.13	0.09
	120 ft	1.22	0.13	0.11	1.13	0.10	0.09
	200 ft	1.08	0.11	0.10	1.30	0.11	0.08

Table 3.19 Average L_{max} values for different ADTT

	summary of L_{max} values								
	ADTT =5000 P_{sxs}=2%			ADTT= 1000 P_{sxs}=1.25%			ADTT=100 P_{sxs}=0.5%		
	mean	within site COV	site to site COV	mean	within site COV	site to site COV	mean	within site COV	site to site COV
40 ft	1.49	0.09	0.12	1.44	0.10	0.12	1.33	0.11	0.12
60 ft	1.36	0.09	0.13	1.31	0.10	0.13	1.21	0.11	0.13
100 ft	1.32	0.09	0.12	1.27	0.09	0.12	1.18	0.11	0.12
120 ft	1.26	0.09	0.10	1.25	0.09	0.11	1.16	0.11	0.11
200 ft	1.15	0.09	0.11	1.11	0.09	0.11	1.03	0.11	0.11
Average		0.09	0.11		0.09	0.12		0.11	0.12

Table 3.20 Average L_{max}xHL93 values for different ADTT

	summary of L_{max} x HL93 values								
	ADTT =5000 P_{sxs}=2%			ADTT= 1000 P_{sxs}=1.25%			ADTT=100 P_{sxs}=0.5%		
	Mean (kip-ft)	within site COV	site to site COV	Mean (kip-ft)	within site COV	site to site COV	Mean (kip-ft)	within site COV	site to site COV
40 ft	861.22	0.09	0.12	832.32	0.1	0.12	768.74	0.11	0.12
60 ft	1486.48	0.09	0.13	1431.83	0.1	0.13	1322.53	0.11	0.13
100 ft	3067.68	0.09	0.12	2951.48	0.09	0.12	2742.32	0.11	0.12
120 ft	3819.06	0.09	0.1	3788.75	0.09	0.11	3515.96	0.11	0.11
200 ft	7476.15	0.09	0.11	7216.11	0.09	0.11	6696.03	0.11	0.11
Average		0.09	0.11		0.09	0.12		0.11	0.12

Reliability Analysis for Deterministic Permit alongside a Random Truck

Given N_R events where the permit truck will be alongside a random truck, the maximum load will occur when the permit truck will be alongside the one truck out of the N_R which produces the maximum load effect. The mean normalized live load effect of this truck is labeled $\overline{L}_{\max}$ and its actual effect is $\overline{L}_{\max} \times HL93$. The maximum total live load effect on a member is obtained from:

$$LL = \left(P \times DF_P + \lambda_{site\,to\,site} \times \lambda_{samplesize} \times L_{\max\ N_R} \times HL93 \times DF_R\right) IM \qquad (3.26)$$

where P is the load effect of the permit truck, DF_P is the load distribution factor for the load P, $\boldsymbol{L}_{\max}$ is the maximum load effect of N_R random trucks, DF_R is the distribution factor for the random load, and IM is the impact factor for side-by-side events. $\lambda_{site\ to\ site}$ is introduced to reflect the variability in L_{max} from site to site with a mean value of 1.0 and a COV $V_{site\ to\ site}$=12% as per Table 3.19. $\lambda_{sample\ size}$ is introduced to reflect the variability in L_{max} due to the WIM data sample size, it is associated with a mean value of 1.0 and a COV $V_{sample\ size}$=2% as per NCHRP 12-76.

The tables for the load distribution factors, D.F., provided in the AASHTO LRFD for two lanes assume that the two side-by-side trucks are of equal weight, which is clearly not the case. Therefore, following the procedure provided in Eq. 4.6.2.2..4-1 of the LRFD, DF_p will be obtained based on the one-lane distribution factor (Eq. 2.33 of this Report after removing the multiple lane factor of 1.2), while DF_R is obtained from the difference between the DF of two lanes and that of a single lane (Eq. 2.34 minus Eq. 2.33 of this Report after removing the multiple lane factor MP=1.2). DF_R and DF_p are assumed to be uncorrelated and their coefficients of variation is estimated as V_{DF}=8%. For the side-by-side trucks, the mean value of IM is given as $\overline{IM} = 1.10$ and the COV is V_{IM}=5.5%.

The calculation of the reliability index values is executed using the FORM algorithm with the failure function expressed as shown in Eq. (3.1) with LL of Eq. (3.26). R is Lognormal, DL is Normal, and L_{max} is Gumbel. The rest of the random variables are assumed to follow Normal distributions. In a first step, we assume that the permit load effect P is deterministic.

For the case when a single live load factor γ_L=1.10 is used for all the Permit types and weights shown in Figure 3.21, the average reliability index is obtained as $\beta_{average}$=2.59 with a minimum value of 1.45 and a maximum value of 3.40 as plotted in Figure 3.31. Table 3.21 gives the range of reliability index values obtained for each of the Permit trucks. The lowest values of β are for the Permit truck NYP-9. For the NYP-9, the average reliability index is 1.89 with a minimum value of 1.45. This observation leads us to conclude that a γ_L=1.10 is sufficient to meet the target beta set β_{target}=2.0 in Section 3.2 for each of the permit trucks. In particular, the reliability indexes for NYP-1 through NYP-5 which represent the non-divisible loads or cranes which can be assumed to be deterministic produce an average value for this group $\beta_{average}$=2.99 with a minimum value of 2.25 and a maximum value of 3.40. This confirms the observation made by Moses (2001) that the heavier Permit trucks generally correspond to lower reliability index values. This is due to the fact that the higher the Permit weight is, the lower is the probability of having a random truck of equal or higher weight alongside of it. However, significantly higher reliability index values are obtained in our calculations than by Moses (2001) because Moses (2001) used the same COV for the Permit load effects as those of random trucks. In our analysis we assume that the weights of the permit trucks are much better known and the overall COV of their load effects is significantly lower than the COV of random truck load effects. For this reason, a live load factor γ_L=1.10 is found to be sufficient to meet the target reliability index β=2.0 for non-divisible Permit trucks of known Permit weights for all weight categories and the separation of the live load factors by Permit weight as done in the AASHTO LRFR is not necessary.

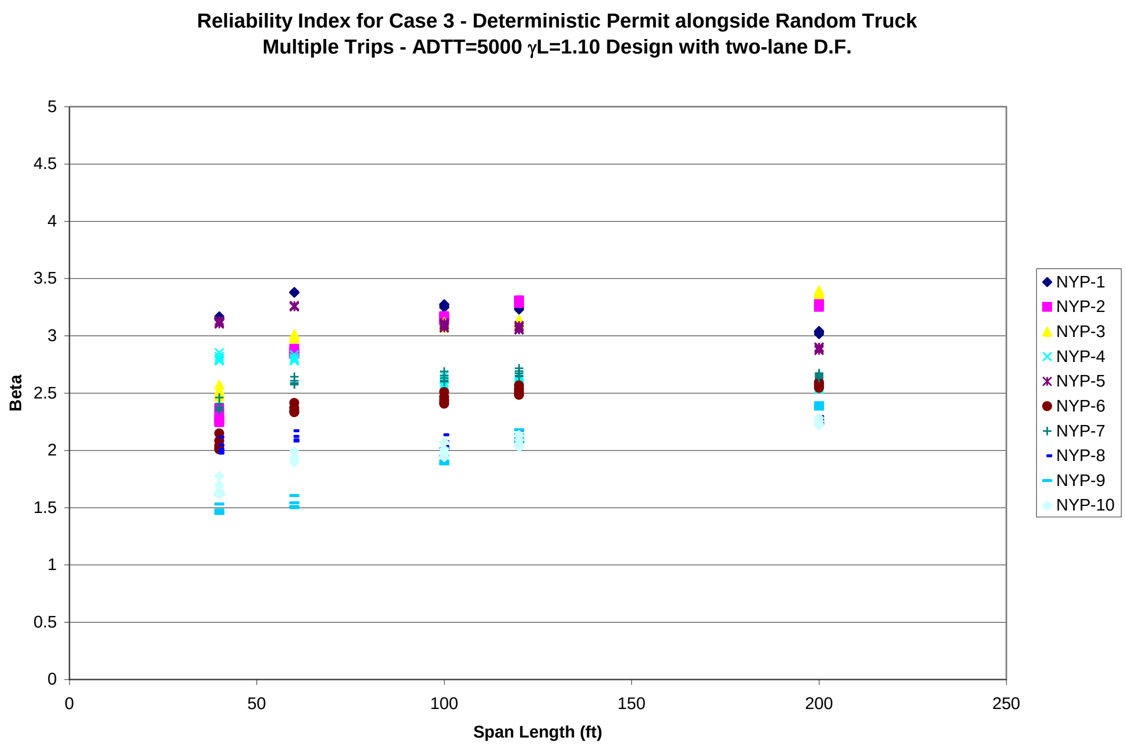


Figure 3.31 Reliability index values for routine permits mixed with random truck using a γ_L=1.10 for permit checking

Table 3.21 Reliability index values for routine deterministic permits mixed with random trucks for sites with ADTT=5000 using a γ_L=1.10 for permit checking.

Permit Truck	NYP-1	NYP-2	NYP-3	NYP-4	NYP-5	NYP-6	NYP-7	NYP-8	NYP-9	NYP-10
γ_L	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
$\beta_{average}$	3.21	2.97	3.02	2.67	3.08	2.39	2.59	2.11	1.89	1.98
max	3.38	3.31	3.40	2.85	3.26	2.59	2.72	2.29	2.42	2.28
Min	3.01	2.25	2.47	2.53	2.87	2.01	2.35	1.98	1.45	1.61
max-min	0.37	1.06	0.93	0.32	0.39	0.58	0.37	0.32	0.97	0.67
Overall average	2.59									
Overall min	1.45									
Overall max	3.40									

Random Permits alongside Random Trucks

The previous calculations in Table 3.21 assume that all the Permit vehicles are deterministic. This situation is not likely to be true for the cases when annual permits are issued for divisible type loads. If one assumes that some of the Permit trucks may be overloaded, then the load effect P of Eq. (3.26) must be considered to be a random variable. In this set of calculations we assume that the Gross Vehicular weight associated with an overloaded permit follows an Exponential probability distribution with a parameter λ=0.15. The mean and standard deviation will be as shown in Eq. (3.16) and (3.17). In this case, the reliability index will be lower than observed for the deterministic permit loads. If γ_L=1.10 is used, then the overall average reliability index reduces to $\beta_{average}$=2.50 with a minimum value of 1.35 and maximum value of 3.37. For the trucks labeled NYP-6 to NYP-10 which are likely to be the trucks carrying divisible loads, the average drops to $\beta_{average}$=2.09 with a minimum value of 1.35 and a maximum value of 2.62. The minimum value remains above 2.19 for the trucks labeled NYP-1 to NYP-5 which are unlikely to be the types that carry divisible loads anyway.

If the live load factor is increased to γ_L=1.15, then the average reliability index for the trucks labeled NYP-6 to NYP-7 becomes $\beta_{average}$=2.25 with a minimum value of 1.55 and maximum value of 2.79. If γ_L=1.20 is used, then for the NYP-6 to NYP-10 types, the average reliability index is $\beta_{average}$=2.41 with a minimum value of 1.74 and maximum value of 2.96 as shown in Figure 3.32. These calculations are executed for sites with ADTT=5000.

If the live load factor γ_L=1.15 is used for sites with ADTT=1000, then the average reliability index for the trucks labeled NYP-6 to NYP-7 becomes $\beta_{average}$=2.31 with a minimum value of 1.63 and maximum value of 2.83 as shown in Figure 3.33. Note that for the case where the Permit trucks are assumed to be deterministic, there was no need t execute the calculations for ADTT less than 5000 since the reliability index for the higher ADTT was already satisfied with the lowest possible live load factor γ_L=1.10.

When a live load factor γ_L=1.10 is applied for sites with ADTT=100, then the average reliability index for the trucks labeled NYP-6 to NYP-7 becomes $\beta_{average}$=2.30 with a minimum value of 1.65 and maximum value of 2.78 as shown in Figure 3.34.

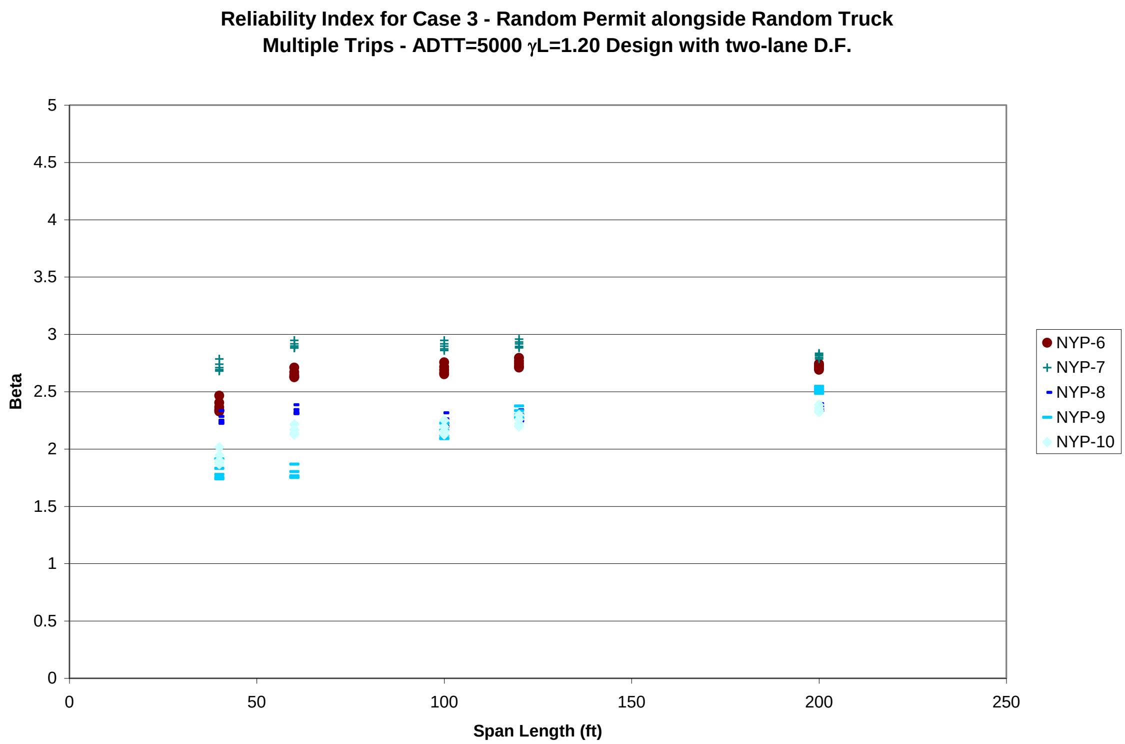


Figure 3.32. Reliability index values for divisible permits mixed with random truck using a γ_L=1.20 for permit checking sites with ADTT=5000.

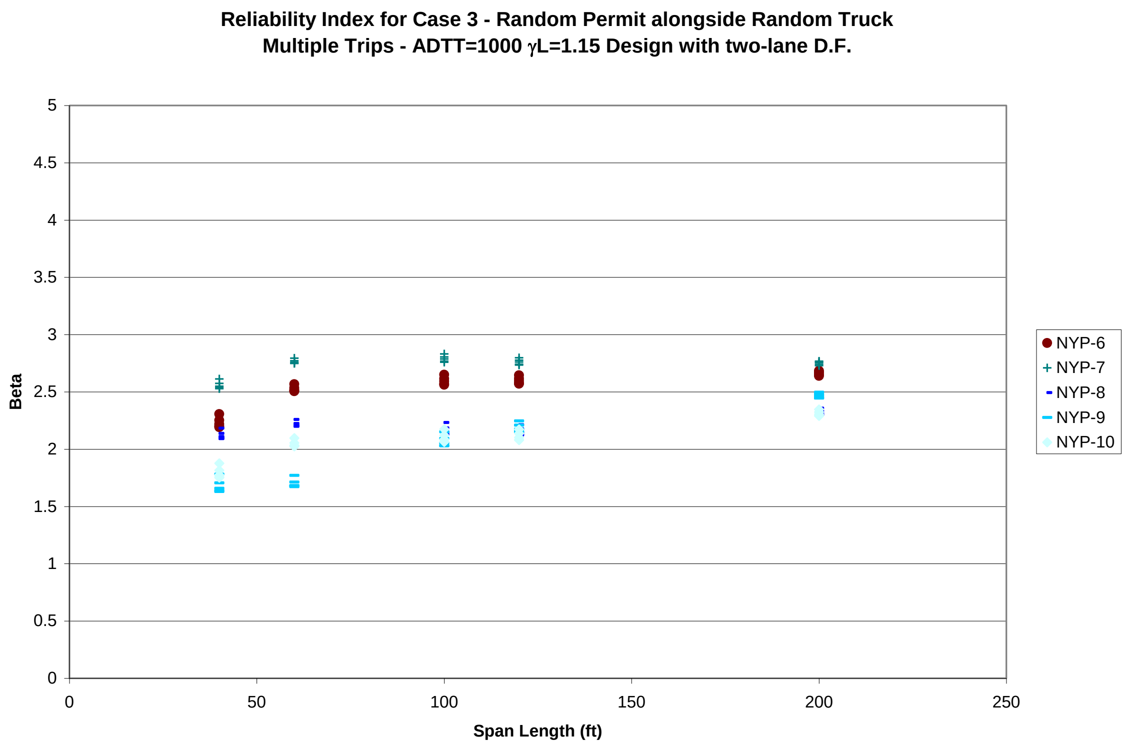


Figure 3.33 Reliability index values for divisible permits mixed with random truck using a γ_L=1.15 for permit checking sites with ADTT=1000.

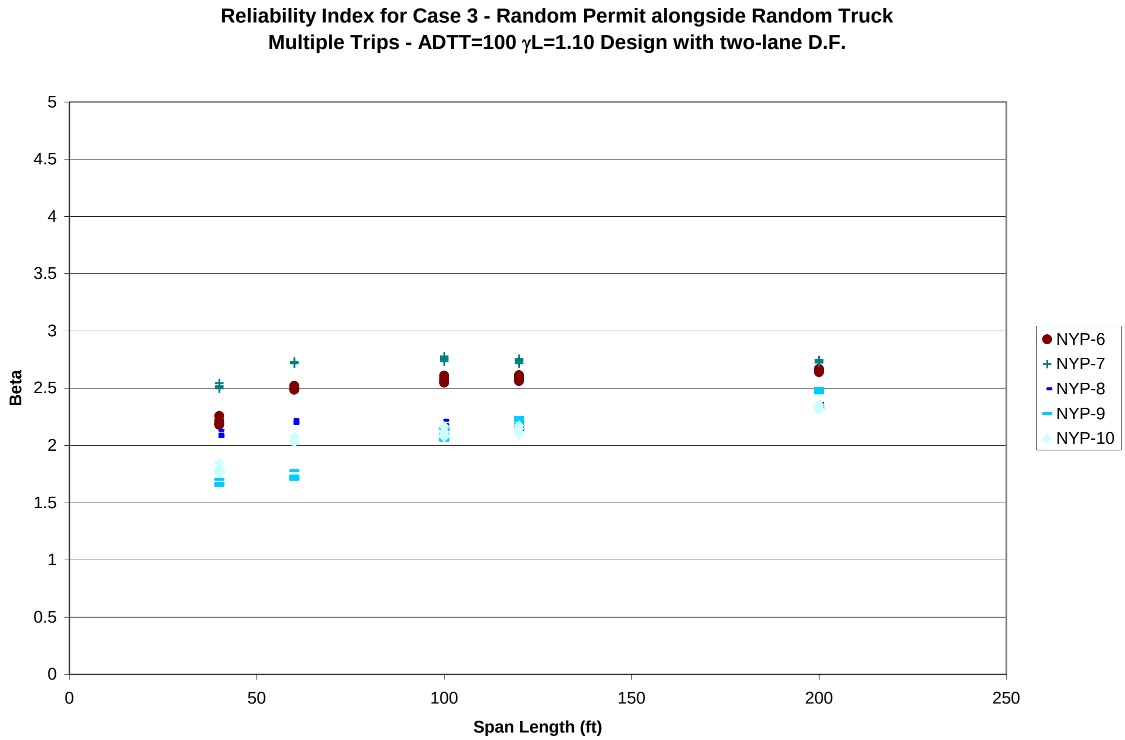


Figure 3.34 Reliability index values for divisible permits mixed with random truck using a γ_L=1.10 for permit checking sites with ADTT=100.

Summary and Recommendation

In summary, the use of live load factor of γ_L=1.10 would lead to acceptable reliability levels for the cases when a permit with exactly known weights would cross a bridge alongside a random truck. This situation may represent cases when non-divisible load permits or permit for cranes with known weights are issued. If the permit is issued for a truck carrying divisible loads that could possibly exceed the weight limit, then the live load factors can be set at γ_L =1.10 for sites with ADTT=100, γ_L=1.15 for sites with ADTT=1000 and γ_L =1.20 for sites with ADTT=5000. Using these proposed live load factors result in average reliability index values above β_{target}=2.0 while the minimum value remains above β=1.50. The calculations use the statistical data collected on Type 6-A permit truck overloads collected at WIM site 8280 as being representative of all permit overloads.

Reliability Analysis for Case IV – Permit Truck alongside a Random Truck for Single Crossings of Permits.

When the Permit truck is limited to a single trip mixed with regular traffic, the AASHTO LRFR specifications recommend using the distribution factor for one lane after removing the multiple presence factor MP=1.2 and provide a range of live load factors varying from 1.50 to 1.35 based on the ADTT of the site.

The reliability analysis for this case, when we have a single crossing of a Permit, uses the same models described earlier where the permit's weight is deterministic and the moment effect of the expected maximum weight of the random truck that could be alongside the Permit is obtained from the histogram of the single lane load effect with $N_R=1$ as depicted in Figure 3.35. Table 3.22 gives the mean values for the single lane truck load effects obtained for each of the ten New York State WIM sites. Table 3.23 gives the average of the mean values and the standard deviation and the COV of the overall average. This COV will be used to express the site-to-site variability in the single lane truck load effects which is on the order of $V_{site\ to\ site}=7\%$. For the reliability calculations, an "average histogram" representing the data from all ten New York WIM sites is used for each span length. The corresponding cumulative distributions of these representative histograms are depicted in Figure 3.35. Only non-divisible loads with deterministic weights are analyzed because single crossing Permits are issued for non-divisible loads.

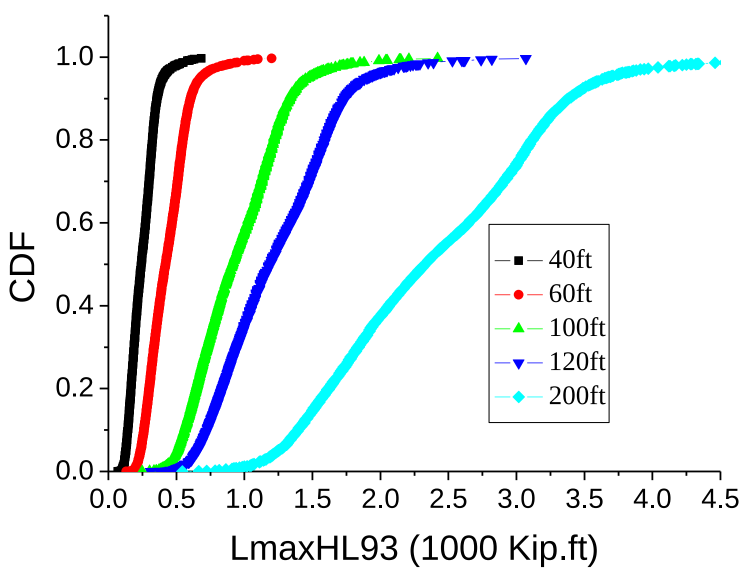


Figure 3.35. Representative Cumulative distributions of load effects for a single random truck crossing.

Table 3.22. Mean L_{max} values of single lane moment effects for each WIM site

		DIRECTION 1	DIRECTION 2
Site	Span	mean	mean
NY0199	40 ft	0.43	0.42
	60 ft	0.39	0.38
	100 ft	0.40	0.39
	120 ft	0.40	0.38
	200 ft	0.36	0.35
NY2680	40 ft	0.43	0.46
	60 ft	0.39	0.42
	100 ft	0.39	0.42
	120 ft	0.38	0.41
	200 ft	0.33	0.36
NY8280	40 ft	0.42	0.34
	60 ft	0.39	0.31
	100 ft	0.40	0.32
	120 ft	0.41	0.32
	200 ft	0.37	0.30
NY9121	40 ft	0.42	0.42
	60 ft	0.38	0.38
	100 ft	0.39	0.39
	120 ft	0.39	0.39
	200 ft	0.35	0.36
NY9631	40 ft	0.38	0.42
	60 ft	0.35	0.38
	100 ft	0.36	0.39
	120 ft	0.36	0.39
	200 ft	0.32	0.36

Table 3.23. Average of mean L_{max} values and L_{max} HL-93 values.

	Mean L_{max}	standard deviation	$V_{site\ to\ site}$	L_{max} HL93 (kip-ft)
40 ft	0.41	0.03	0.08	237.92
60 ft	0.38	0.03	0.08	412.81
100 ft	0.38	0.03	0.07	890.96
120 ft	0.38	0.03	0.07	1161.56
200 ft	0.35	0.02	0.06	2247.77
average			0.07	

Because the cumulative histograms in Figure 3.35 do not follow any known probability distribution type, in this set of reliability calculations we use the Monte Carlo simulation to extract possible values for the random truck that could cross a bridge alongside a Permit truck. The Monte Carlo simulation program which extracts the random truck weight directly from the cumulative histogram of random truck weights is also directly used to find the probability of failure and the corresponding reliability index

The Monte-Carlo simulation requires the performance of an analysis a large number of times and then assembling the results of the analysis into a histogram that will describe the scatter in the final results. The process can be executed for the situation where we have permit truck side-by-side with a random truck as depicted in Figure 3.36. Figure 3.36 gives a schematic representation of the Monte Carlo Simulation, which follows the procedure described in the following steps:

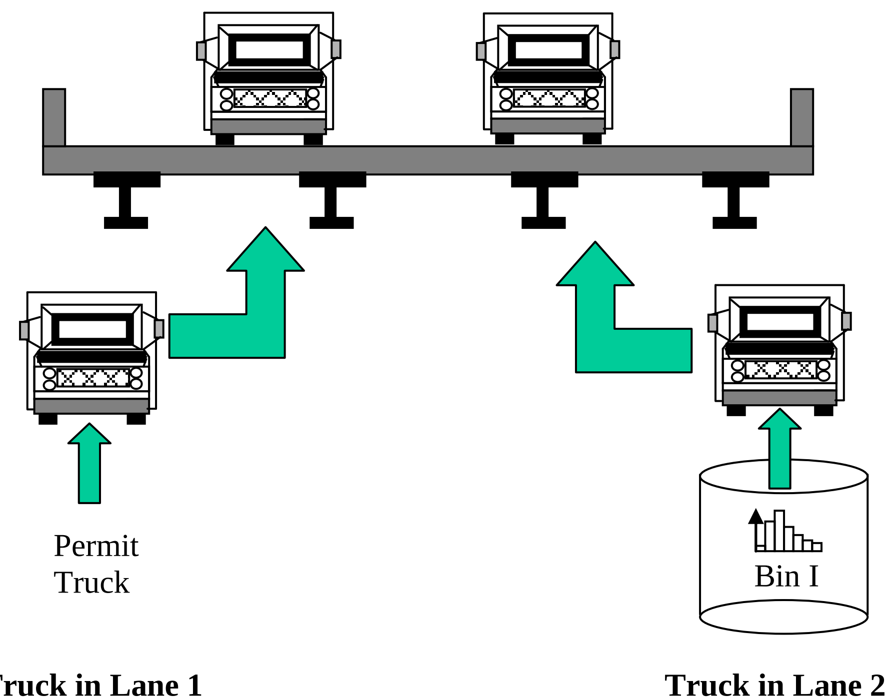


Figure 3.36 Schematic illustration of Monte Carlo simulation procedure for smapling truck weights

1. For a bridge of certain span length and beam spacing, use Eq. 3.3 to find the nominal resistance R_n for a given Permit load for the case when the Rating factor R.F. is exactly equal to 1.0.
2. Assemble the data representing the load effects for the random trucks in single lane into a histogram labeled Bin I as shown in Figure 3.36.
3. Assemble the corresponding cumulative frequency curves for the random truck effects in a single lane as shown in Figure 3.35.
4. Use a uniform distribution random generator to produce a pseudo random number varying between 0 and 1. Such random generator routines are provided in all general-purpose computer software and programming tools (such as EXCEL or MATLAB).
5. The pseudo-random number of step 4 will serve to select a single value from Bin I representing the load effect of a random truck, i. The selection of the moment effect is executed by assuming that the pseudo-random number generated (call it ran_i) represents the cumulative frequency of the moment for this truck. Thus, to find the value of the moment effect of this truck i, L_i, the cumulative distribution function needs to be inverted so that $L_i = F_{x1}^{-1}(ran_{1i})$ where $F_{x1}^{-1}(...)$ is the inverse of the cumulative function for the effect of the trucks in the drive lane. The process is illustrated in Figure 3.37.
6. Use the random number generators to generate a sample bridge member resistance, R_i, a dead load value DL_i, a distribution factor for the Permit truck DF_{Pi}, a site to site modeling random bias $\lambda_{site\ to\ site\ i}$, a sample site random bias $\lambda_{sample\ size\ i}$, a distribution factor value for the random truck DF_{Ri}, and an impact value IM_i. Where R is a Lognormal random variable related to R_n of step 1 with a bias of 1.12 and COV V_R=10%. The dead loads are normally distributed with a mean total dead load, $\overline{DL}$, obtained as the sum of the means of the load effects of the prefabricated components, D_{C1}, the cast in place components, D_{C2}, and the wearing surface, D_w, with the biases and COV given in Table 3.1. The effect of the permit truck P is assumed to be deterministic, while the distribution factor applied on the permit, DF_P is assumed to be a Normally distributed variable with a mean value equal to the value given in the AASHTO LRFD tables for a single lane after removing the multiple presence factor MP=1.2. The site to site bias is assumed to be a random variable with a Normal distribution and a mean value equal to 1.0 and a COV=7% as obtained from Table 3.23. The data sampling bias is assumed to be a Normally distributed variable with mean 1.0 and a COV=2% based on the analysis performed in NCHRP 12-76. The live load effect of the random truck is obtained from the number random generation using the approach depicted in Figure 3.37 and explained in step 5. The distribution factor for the random trucks DF_R is also assumed to be a Normally distributed random variable with a COV=8% and a mean value obtained as the difference between the distribution factor of the two lanes provided in the AASHTO LRFD and DF_P. The impact factor IM follows a Normal distribution with a mean value of 1.10 for two trucks side-by-side and a COV=5.5%.

7. For each set of variables generated in iteration i, find whether the safety margin, Z_i, of Eq. (3.27) yields a value less than zero. The value of the total live load effect, LL_i, in Eq. (3.27) is obtained from Eq. (3.28).

$$Z_i = R_i - DL_i - LL_i \tag{3.27}$$

$$LL_i = \left(P \times DF_{Pi} + \lambda_{site\ to\ site\ i} \times \lambda_{sample size\ i} \times L_i \times HL93 \times DF_{Ri}\right) IM_i \tag{3.28}$$

8. After repeating the process described in steps 4 through 7 a 100,000 times, the number of cases where Z_i is less than 0.0 is divided by the total number of iterations to yield an estimate of the probability of failure $P_{fc} = P_{f|side-by-side}$. This probability of failure is conditional on having a random truck alongside the permit truck.
9. The probability of having a permit alongside a random truck depends on the probability of side-by-side events P_{sxs}. For sites with ADTT =5000 we use P_{SxS}=2%. For sites with ADTT=1000 P_{sxs}=1.25% and for ADTT=100 P_{sxs}=0.5%. The unconditional probability of failure is obtained from

$$P_f = P_{f|side-by-side} \times P_{S\times S} \tag{3.29}$$

10. The final unconditional reliability index, β, is obtained from

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{3.30}$$

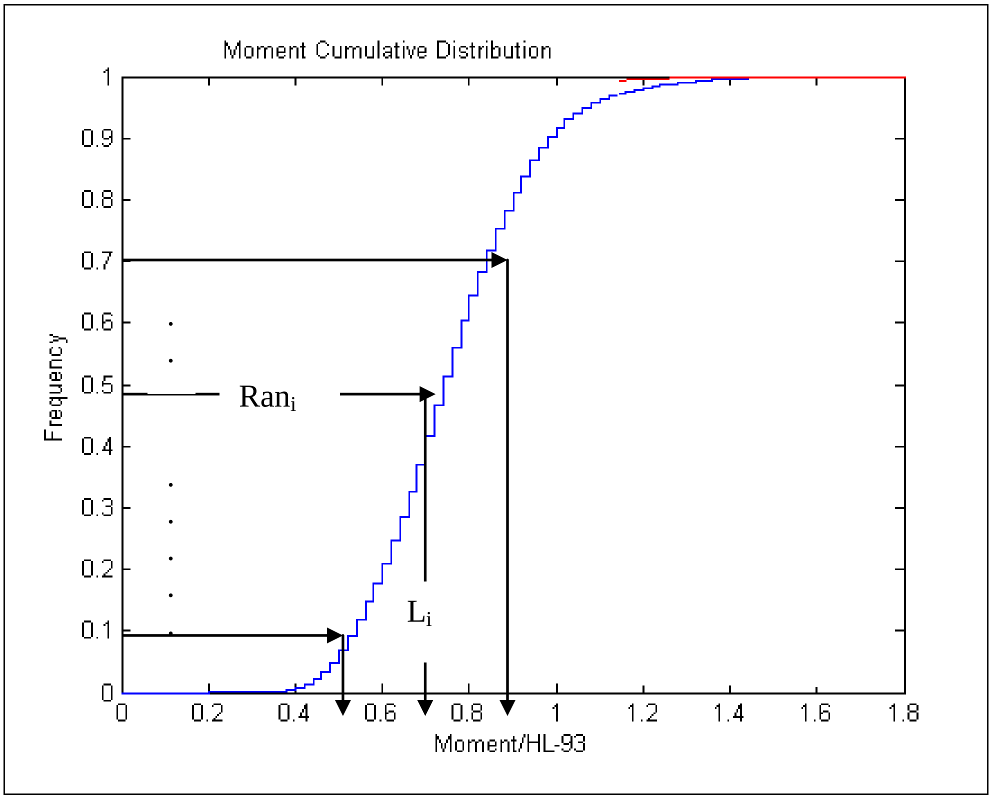


Figure 3.37 Schematic illustration of the random generation of a sample L_i.

The results of the simulation for the conditional reliability index β_c = $\beta_c = -\Phi^{-1}(P_{fc})$ are given in Table 3.24 for each of the permit vehicles, span lengths and beam spacing when the checking of the safety of the permit is executed using a live load factor γ_L=1.10 applied with the distribution factor for one lane after removing the multiple presence factor MP=1.2. The results which are also shown in Figure 3.38 indicate that the average conditional reliability index is equal to 2.20 with a maximum value of 2.57 and a minimum value of 1.67. The table shows that the average for each permit truck remains above 2.0 with the lowest average being for Permit truck NYP-9 which is associated with an average reliability index β=2.03.

These conditional reliability values are clearly lower than values obtained above when a single permit truck is alone on the bridge and when the checking of the safety was performed with γ_L=1.10. In that Case I analysis, the average reliability index was $\beta_{average}$ =2.84 with a minimum value of 2.72 and maximum value of 2.90. The lower reliability index values are due to the presence of the random truck alongside the permit. However, these results are for the conditional reliability index when we assume that there always will be a random truck along side the Permit truck. Because of the low probabilities of having side-by-side trucks, the unconditional reliability indexes for Case IV will be significantly higher.

The Case IV conditional reliability index values are also lower than the conditional reliability index values obtained for two permits side-by-side which were analyzed under Case II above. Under case II, the average conditional reliability index was $\beta_{average}$=3.09 with a minimum value of 2.93 and a maximum value of 3.14. The lower conditional reliability index values observed in case IV are due to the use of the single lane distribution factor applied when checking the safety of Case IV as compared to the two-lane distribution factor used for Case II and also due to the fact that the weights of the random trucks are associated with higher COV values as compared to the weight of the permits which are assumed to be deterministic for single crossings scenarios.

The unconditional reliability index values for the case when a single permit is issued but the truck is allowed to mix with random truck traffic are shown in Table 2.25 for the different probabilities of side-by-side events. The results show that the average unconditional reliability indexes increase to $\beta_{average}$=3.46, 3.58 and 3.81 for P_{sxs}=2%, 1.25% and 0.5% respectively. These average reliability index values are significantly higher than the target reliability index β_{target}=2.0 even when a low live load factor γ_L=1.10 is used.

Table 3.24 Conditional reliability index values for Permit alongside a random truck.

Span	Spacing	NYP1	NYP2	NYP3	NYP4	NYP5	NYP6	NYP7	NYP8	NYP9	NYP10
40 ft	4 ft	2.34	2.11	2.17	2.24	2.33	2.03	2.13	2.02	1.86	1.92
40 ft	6 ft	2.20	1.96	2.03	2.11	2.19	1.89	1.99	1.89	1.73	1.78
40 ft	8 ft	2.20	1.94	2.01	2.10	2.19	1.88	1.98	1.87	1.70	1.75
40 ft	10 ft	2.16	1.90	1.97	2.06	2.15	1.84	1.93	1.83	1.68	1.72
40 ft	12 ft	2.14	1.90	1.96	2.05	2.13	1.83	1.93	1.83	1.67	1.72
60 ft	4 ft	2.44	2.30	2.34	2.29	2.41	2.17	2.24	2.10	1.93	2.05
60 ft	6 ft	2.31	2.18	2.21	2.16	2.28	2.05	2.11	1.99	1.83	1.94
60 ft	8 ft	2.37	2.23	2.26	2.21	2.34	2.10	2.16	2.03	1.87	1.98
60 ft	10 ft	2.26	2.14	2.17	2.12	2.23	2.02	2.08	1.95	1.81	1.91
60 ft	12 ft	2.28	2.15	2.18	2.13	2.25	2.02	2.08	1.96	1.81	1.91
100 ft	4 ft	2.44	2.42	2.40	2.29	2.40	2.26	2.30	2.15	2.13	2.14
100 ft	6 ft	2.35	2.32	2.31	2.21	2.31	2.18	2.21	2.09	2.06	2.07
100 ft	8 ft	2.35	2.33	2.32	2.20	2.32	2.16	2.21	2.07	2.04	2.05
100 ft	10 ft	2.28	2.25	2.24	2.13	2.24	2.10	2.14	2.02	1.99	2.01
100 ft	12 ft	2.33	2.31	2.29	2.18	2.29	2.14	2.19	2.06	2.03	2.04
120 ft	4 ft	2.49	2.49	2.46	2.34	2.45	2.32	2.36	2.24	2.24	2.23
120 ft	6 ft	2.42	2.43	2.39	2.27	2.38	2.25	2.29	2.16	2.16	2.15
120 ft	8 ft	2.34	2.35	2.32	2.21	2.31	2.19	2.23	2.11	2.11	2.10
120 ft	10 ft	2.30	2.31	2.27	2.16	2.26	2.15	2.18	2.06	2.07	2.06
120 ft	12 ft	2.33	2.34	2.30	2.19	2.29	2.17	2.21	2.09	2.09	2.08
200 ft	4 ft	2.51	2.55	2.57	2.43	2.49	2.43	2.45	2.38	2.41	2.38
200 ft	6 ft	2.50	2.54	2.56	2.42	2.48	2.43	2.44	2.37	2.40	2.37
200 ft	8 ft	2.50	2.54	2.57	2.41	2.47	2.41	2.43	2.35	2.38	2.35
200 ft	10 ft	2.46	2.51	2.52	2.39	2.45	2.39	2.41	2.34	2.36	2.34
200 ft	12 ft	2.47	2.51	2.53	2.40	2.45	2.40	2.41	2.35	2.37	2.34
Average		2.35	2.28	2.29	2.23	2.32	2.15	2.20	2.09	2.03	2.06
Max		2.51	2.55	2.57	2.43	2.49	2.43	2.45	2.38	2.41	2.38
Min		2.14	1.90	1.96	2.05	2.13	1.83	1.93	1.83	1.67	1.72
overall average	2.20										
Max	2.57										
Min	1.67										

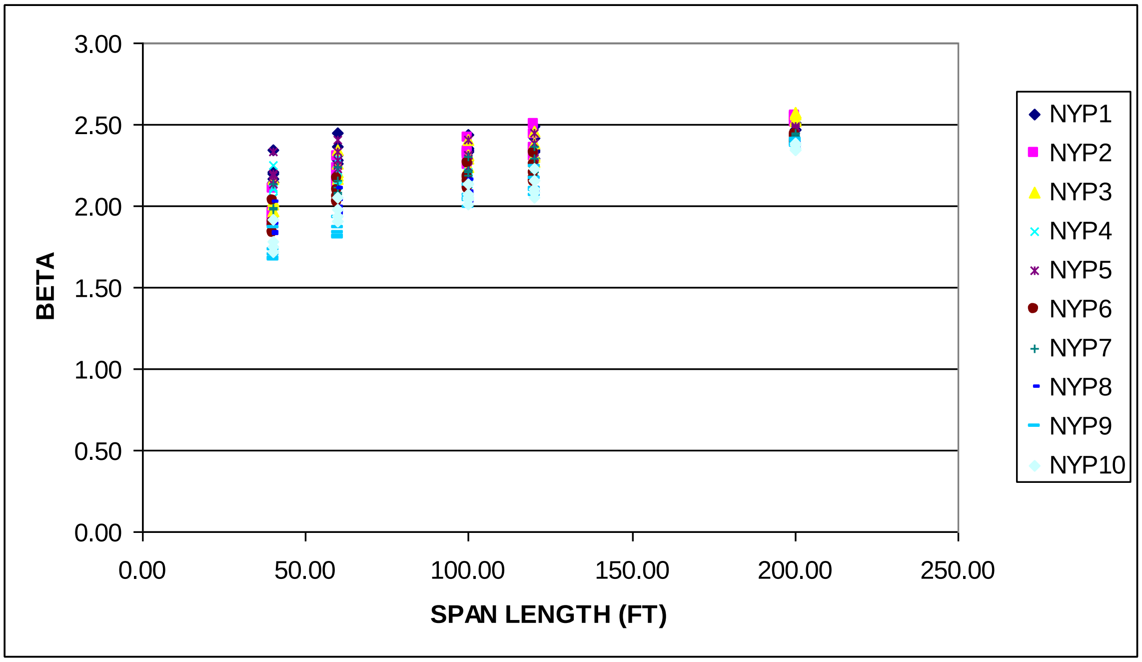


Figure 3.38 Conditional reliability index values for a single permit crossing alongside a random truck.

Table 3.25 Reliability index values for different P_{sxs}

	beta	conditional	2%	1.25%	0.50%
NYP1	average	2.35	3.56	3.68	3.91
	max	2.51	3.67	3.79	4.01
	min	2.14	3.41	3.54	3.77
NYP2	average	2.28	3.51	3.63	3.86
	max	2.55	3.70	3.82	4.04
	min	1.90	3.25	3.38	3.63
NYP3	average	2.29	3.52	3.64	3.87
	max	2.57	3.71	3.83	4.05
	min	1.96	3.29	3.42	3.66
NYP4	average	2.23	3.47	3.60	3.83
	max	2.43	3.62	3.74	3.96
	min	2.05	3.35	3.48	3.71
NYP5	average	2.32	3.54	3.66	3.89
	max	2.49	3.66	3.78	4.00
	min	2.13	3.41	3.53	3.77
NYP6	average	2.15	3.42	3.55	3.78
	max	2.43	3.62	3.74	3.96
	min	1.83	3.21	3.34	3.59
NYP7	average	2.20	3.46	3.58	3.81
	max	2.45	3.63	3.75	3.97
	min	1.93	3.27	3.40	3.65
NYP8	average	2.09	3.38	3.51	3.74
	max	2.38	3.58	3.70	3.93
	min	1.83	3.20	3.34	3.58
NYP9	average	2.03	3.34	3.47	3.71
	max	2.41	3.60	3.72	3.94
	min	1.67	3.10	3.24	3.49
NYP10	average	2.06	3.36	3.48	3.72
	max	2.38	3.58	3.70	3.93
	min	1.72	3.14	3.27	3.52
ALLTRUCKS	AVERAGE	2.20	3.46	3.58	3.81
	MAX	2.57	3.71	3.83	4.05
	MIN	1.67	3.10	3.24	3.49

3.5 Conclusions

This Chapter presents the results of the reliability calibration of live load factors for use in proposed NYSDOR LRFR procedure for rating New York bridges, checking the safety of bridges for Permit issuance and for posting bridges with Rating Factors less than 1.0. The target reliability index set for the calibration is β_{target}=2.0 with the goal of achieving reliability index values for all conditions that remain above a minimum β_{min}=1.50. The calculations performed in this report demonstrate that using live load factors γ_L=1.95, 1.85 and 1.65 for sites with ADTT=5000, 1000 and 100 will lead to uniform reliability levels that meet the target for multi-lane bridges checked with the AASHTO LRFD multi-lane distribution factor. Significantly higher live load factors γ_L=2.65, 2.50 and 2.20 will be required for single lane bridges. The rating should be executed for the AASHTO SU-4 and type 3S-2 trucks from now on labeled as the NYSDOT Legal trucks. The final rating factor R.F. is the lowest obtained after checking the two NYSDOT legal trucks.

A bridge that produces a Rating Factors less 1.0 may need to be posted. Two posting weights should be calculated one for single unit trucks and the other for semi-trailers. The posting weights for each truck type must be calculated from the following equation:

$$Safe\ Posting\ \ Load = W\left[RF + 0.00375(L - 110)(1 - RF)\right]$$

Where RF is the lowest Rating Factor, W is the weight of the posting truck and L is the effective span length.

This Chapter also demonstrates that a live load factor γ_L=1.10 for non-divisible permit loads will provide average reliability index values greater than the target β_{target}=2.0. For the cases of divisible loads where some data shows that Permit loads may exceed the Permit weight limits, having live load factors varying from γ_L=1.20 for sites with ADTT=5000, γ_L=1.15 for sites with ADTT=1000, and γ_L=1.10 for sites with ADTT=100 will increase the reliability index values so that the minimum value remains above β=1.50. Non-divisible permits trucks travelling over bridges at crawl speed should still be checked with a dynamic allowance factor of 1.05 to satisfy the minimum value of β_{min}=1.50.

CHAPTER FOUR

CONCLUSIONS AND FUTURE RESEARCH

4.1 Conclusions

This report developed a Load and Resistance Factor Rating (LRFR) methodology for New York State bridges. The methodology is applicable for the rating of existing bridges, the posting of under-strength bridges, and checking Permit trucks. The proposed LRFR methodology was calibrated based on a target reliability index β_{target}=2.0 which has been set to provide on the average slightly more conservative ratings than current NYSDOT procedures. The calibration process also aimed at producing a tight range of reliability index values such that the minimum reliability index does not fall below β_{min}=1.50 for all applications. The reliability calibration of live load factors is based on live load models developed using Truck Weigh-In-Motion (WIM) data collected from several representative New York sites. The live load models provide statistical projections of the maximum live load effects expected on New York bridges.

The analysis of the WIM data showed that New York State live loads are significantly higher than the live loads assumed during the calibration of the AASHTO LRFD and LRFR specifications particularly for single lane bridges. This required the adoption of a new set of NYS Legal Trucks along with appropriate live load factors for use in performing Operating Level Ratings of existing bridges.

Permit load factors are calibrated for divisible loads and non-divisible loads for single crossings as well as unlimited crossings of bridges. The calibration of the permit load factors was based on the analysis of multiple presence probabilities and on the uncertainties associated with estimating the load effects of the permit trucks and those of the random trucks that may cross simultaneously with the permit. Accordingly, lower permit load factors are recommended than those in the AASHTO LRFR.

An equation is proposed for determining Posting weight limits for bridges with low Rating Factors as a function of the effective span length. It is proposed that different posting weights be imposed for single unit trucks and semi-trailer trucks. The posting equation was calibrated so that posted bridges will meet the same target reliability β_{target}=2.0 used for the Legal Load Ratings and the Permit weight checks. The posting weight calibration however was based on several assumptions regarding the risks of having overweight trucks cross posted bridges.

4.2 Future Research

Reliability-based methods for calibrating bridge design and structural evaluation codes have long been established and have been used to develop the AASHTO LRFD and LRFR specifications. However, the implementation of these calibration methods require extensive and complete sets of statistical data and accurate models on bridge member strengths, dead loads and live loads. Although several models have been proposed over the years to provide such statistical information, the research team has found that additional work in that direction is still necessary to improve the available database and ensure that it accurately represents current bridge member strengths and loading conditions.

Specifically, future research can be targeted to assemble larger amounts of Weigh-In-Motion data and performing more thorough analyses of the data. WIM data are currently available at certain sites for continuous periods of several years. Such data including weights, truck types and multiple presence probabilities can be very useful in verifying the accuracy of the statistical projection techniques used to estimate the maximum live load that a bridge is expected to carry. The data should also be used to forecast future local and regional increases in truck volume and weights. The expected loads should also be correlated to legal load limits, permit issuance policies, and the frequency of illegally loaded trucks. For setting up load posting limits, statistical information on the crossing of posted bridges by heavy vehicles are also needed.

Following the methods applied during the AASHTO LRFD and LRFR calibration studies, this project assumed that the load distribution factors in the AASHTO LRFD tables provide on the average a good representation of the actual load distribution. Recent studies have however confirmed that these tables are quite conservative providing an additional margin of safety which has not been taken into consideration during any of the previous code calibration studies.

Statistical data on the dynamic amplification of bridge live loads have been assembled in the past and used for calibrating the AASHTO LRFD and LRFR. However, more thorough statistical analyses of such data are needed to correlate the dynamic factors with truck weights and surface roughness. The AASHTO LRFR proposes different dynamic amplification factors based on riding surface conditions. However, the proposed range of factors is based on a qualitative estimation of the factor and did not include a complete statistical analysis.

In addition, the resistance models developed during the calibration of the AASHTO LRFD which have also been used during the development of the AASHTO LRFR and in this project have been based on the methods of member designs that were used in the AASHTO LFD specifications. The fact that the AASHTO LRFD has adopted new design methods for shear as well as the bending of non-compact steel sections may indicate that the existing statistical models for resistance do not accurately reflect the variations of the actual member strengths from those estimated when using the LRFD code equations. The same situation applies for the actual strength of reinforced and prestressed concrete bridge components. It is especially important to relate the statistical models of bridge member strengths to the material properties of existing and deteriorated bridge members. For concrete members it is also important to correlate these models to the local and regional environmental conditions as well to the construction practices in the State.

To account for the increased uncertainties associated with estimating the strengths of deteriorated members, the AASHTO LRFR includes a condition factor in the rating equations. The condition factors proposed in the AASHTO LRFR specifications were simply based on the judgment of the code writers. Similarly, the condition factors were calibrated in this project to match the reliability levels implied in current NYSDOT procedures. No statistical models are currently available to verify if these proposed factors do reflect the variability in the actual deteriorated member strengths compared to the estimation made by the bridge inspectors and rating engineers.

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