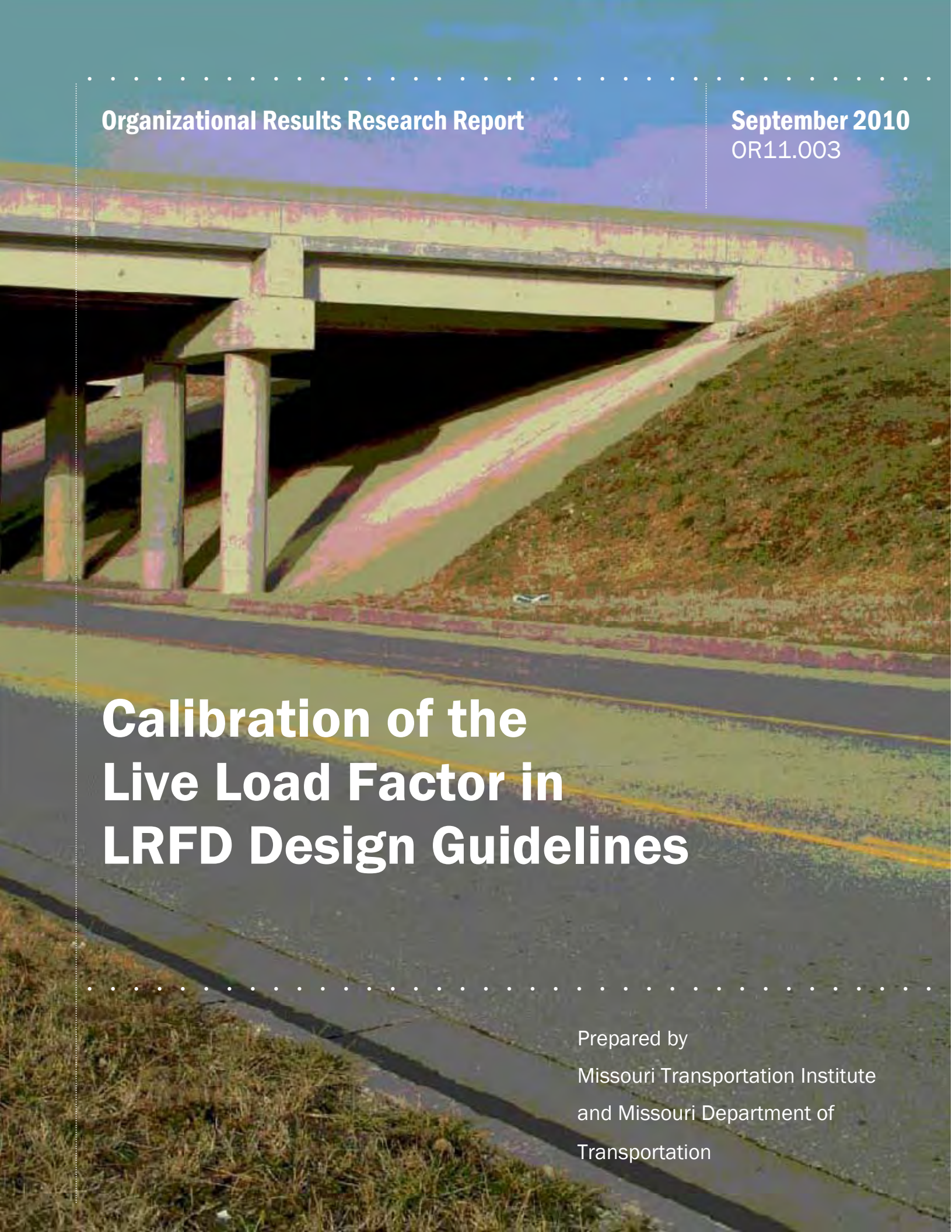


Organizational Results Research Report

September 2010
OR11.003

Calibration of the Live Load Factor in LRFD Design Guidelines

Prepared by
Missouri Transportation Institute
and Missouri Department of
Transportation

Final Report
TRyy0913

Calibration of the Live Load Factor in LRFD Design Guidelines

Prepared for
Missouri Department of Transportation
Organizational Results

by

Oh-Sung Kwon and Eungsoo Kim
Missouri University of Science and Technology

Sarah Orton, Hani Salim, and Tim Hazlett
University of Missouri, Columbia

September 2010

The opinions, findings, and conclusions expressed in this publication are those of the principal investigators and the Missouri Department of Transportation. They are not necessarily those of the U.S. Department of Transportation, Federal Highway Administration. This report does not constitute a standard or regulation.

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. OR11-003	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Calibration of the Live Load Factor in LRFD Design Guidelines		5. Report Date September 2010
		6. Performing Organization Code
7. Author(s) Oh-Sung Kwon, Sarah Orton, Hani Salim, Eungsoo Kim, and Tim Hazlett		8. Performing Organization Report No.
9. Performing Organization Name and Address		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Missouri Department of Transportation Research, Development and Technology P. O. Box 270-Jefferson City, MO 65102		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The Load and Resistant Factor Design (LRFD) approach is based on the concept of structural reliability. The approach is more rational than the former design approaches such as Load Factor Design or Allowable Stress Design. The LRFD Specification for Bridge Design has been developed through the 1990s and 2000s. In the development process, many factors were carefully calibrated such that a structure designed with LRFD can achieve a reliability index of 3.5 for a single bridge girder (probability of failure of about 2 in 10,000). As the initial development of the factors in the LRFD Specification was intended to be applied to the entire nation, state-specific traffic conditions or bridge configuration were not considered in the development process. In addition, due to lack of reliable truck weigh data in the early 1990s in the U.S., the truck weights from Ontario, Canada measured in the 1970s were used for the calibration. Hence, the reliability of bridges designed with the current LRFD specification needs to be evaluated based on the Missouri-specific data and the load factor needs to be re-calibrated for optimal design of bridges.

The objective of the study presented in this report is to calibrate the live load factor in the Strength I Limit State in the AASHTO LRFD Bridge Design Specification. The calibration is based on the Missouri-specific data such as typical bridge configurations, traffic volume, and truck weights. The typical bridge configurations and the average daily truck traffic of the bridges in Missouri are identified from statistical analyses of 2007 National Bridge Inventory. The Weigh-In-Motion (WIM) data from 24 WIM stations in Missouri are used to simulate realistic truck loads. Updated material and geometric parameters are also used to update the resistance distributions.

From this study, it was found that most representative bridges in Missouri have reliability indices higher than 3.5. For many bridges in rural areas with Average Daily Truck Traffic (ADTT) of 1,000 or less, the average reliability indices are higher than 5.0. This study proposes a table of calibration factors which can be applied to the current live load factor of 1.75. The calibration factor is developed as a function of ADTT such that bridge design practitioners can select a calibration factor considering the expected ADTTs of a bridge throughout its life span. Impact of the calibration factor on the up-front bridge construction cost is also presented.

17. Key Words LRFD calibration, live load factor, bridge design, WIM Process, NBI database		18. Distribution Statement No restrictions. This document is available to the public through National Technical Information Center, Springfield, Virginia 22161	
19. Security Classification (of this report) Unclassified	20. Security Classification (of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 (06/98)

EXECUTIVE SUMMARY

The Load and Resistant Factor Design (LRFD) approach is based on the concept of structural reliability. The approach is more rational than the former design approaches such as Load Factor Design or Allowable Stress Design. The LRFD Specification for Bridge Design has been developed through the 1990s and 2000s. In the development process, many factors were carefully calibrated such that a structure designed with LRFD can achieve a reliability index of 3.5 for a single bridge girder (probability of failure of about 2 in 10,000). As the initial development of the factors in the LRFD Specification was intended to be applied to the entire nation, state-specific traffic conditions or bridge configuration were not considered in the development process. In addition, due to lack of reliable truck weigh data in the early 1990s in the U.S., the truck weights from Ontario, Canada measured in the 1970s were used for the calibration. Hence, the reliability of bridges designed with the current LRFD specification needs to be evaluated based on the Missouri-specific data and the load factor needs to be re-calibrated for optimal design of bridges.

The objective of the study presented in this report is to calibrate the live load factor in the Strength I Limit State in the AASHTO LRFD Bridge Design Specification. The calibration is based on the Missouri-specific data such as typical bridge configurations, traffic volume, and truck weights. The typical bridge configurations and the average daily truck traffic of the bridges in Missouri are identified from statistical analyses of 2007 National Bridge Inventory. The Weigh-In-Motion (WIM) data from 24 WIM stations in Missouri are used to simulate realistic truck loads. Updated material and geometric parameters are also used to update the resistance distributions.

From this study, it was found that most representative bridges in Missouri have reliability indices higher than 3.5. For many bridges in rural areas with Average Daily Truck Traffic (ADTT) of 1,000 or less, the average reliability index could be higher than 5.0. This study proposes a table of calibration factors which can be applied to the current live load factor of 1.75. The calibration factor is developed as a function of ADTT such that bridge design practitioners can select a calibration factor considering the expected ADTTs of a bridge throughout its life span. The proposed calibration factors are suggested to be applied to bridges which do not carry US or Inter-State (IS) Highways considering the importance of the US and IS highways in the transportation network. The bridges carrying non-US or IS highways comprise approximately 70% of the bridge inventory of the state. On average, these bridges have ADTTs of less than 500. The application of calibration factor of 0.8 to these bridges is expected to have up to 11% of upfront cost saving for RC slab bridges and 7% of for steel girder bridges. The cost impact to prestressed concrete bridges is expected to be negligible due to standardized section dimension of prestressed concrete bridges.

TABLE OF CONTENTS

LIST OF FIGURES **III**

LIST OF TABLES **IV**

1 INTRODUCTION **1**

2 RESEARCH OBJECTIVE **2**

3 OVERVIEW OF THE CALIBRATION PROCEDURE **2**

4 TECHNICAL APPROACH **6**

4.1 Selection of Representative Bridges in Missouri **6**

4.1.1 Statistical Analysis of Bridges in Missouri in 2007 NBI Database 6
4.1.2 Bridges used in Initial LRFD Calibration in NCHRP 368 Project 15
4.1.3 Average Daily Truck Traffics of Bridges in Missouri 16

4.2 Screening of Measured Truck Weights from WIM Station **18**

4.2.1 Measured WIM Data 18
4.2.2 Screening Criteria for WIM Data 19
4.2.3 Statistical Characteristics of the Screened WIM Data 21

4.3 Location of Maximum Positive Moment **23**

4.4 Maximum Load Effects from Moving Loads **25**

4.4.1 Selection of Heavy Single Truck Cases 25
4.4.2 Simulation of Multiple Truck Events 27
4.4.3 Projection of Daily Maximum Load Effects to 75 Year Load Effects 35
4.4.4 Projection Method using Normal Probability Plot 36
4.4.5 Projection Method Using Extreme Value Theory 37

4.5 Statistical Parameters for Dead Load **40**

4.6 Statistical Distribution of Resistance **41**

4.6.1 Comparison of Required Strengths and Strengths in As-Designed Condition 41
4.6.2 Material and Geometric Parameters 44
4.6.3 Statistical Parameters of Resistance 47

4.7 Reliability Indices of Representative Bridges with Current LRFD Specification **53**

5 RESULTS AND DISCUSSION **56**

5.1 Proposed Live Load Calibration Factor **56**

5.2 Impact of the Live Load Calibration on Bridge Construction Cost **59**

6 CONCLUSIONS **61**

7 RECOMMENDATIONS **62**
BIBLIOGRAPHY **66**
APPENDIX A: SELECTED BRIDGES **68**
APPENDIX B: FHWA VEHICLE CLASSIFICATION **72**
APPENDIX C: BRIDGE RESISTANCE **73**
APPENDIX D: RELIABILITY INDICES OF SELECTED BRIDGES **99**

LIST OF FIGURES

Figure 3.1 Overview of the live load factor calibration procedure 4
Figure 4.1 Construction years of bridges in Missouri 8
Figure 4.2 Distribution of maximum span lengths 10
Figure 4.3 Distribution of number of lanes 12
Figure 4.4 Distribution of number of spans 13
Figure 4.5 Sampling points in CCC method for three random variables 14
Figure 4.6 Distribution of girder spacing of representative bridges 15
Figure 4.7 Distribution of span length for representative bridges 16
Figure 4.8 ADTTs of bridges in Missouri 17
Figure 4.9 Distribution of ADTTs for Missouri IS highway and US highway 17
Figure 4.10 Location of WIM stations in Missouri 19
Figure 4.11 Daily average GVW of selected WIM stations 20
Figure 4.12 Distribution of GVW 22
Figure 4.13 Hourly distribution of truck traffic (hours from midnight) 22
Figure 4.14 Distribution of maximum moment location 23
Figure 4.15 Comparison of maximum moment distributions 24
Figure 4.16 Correlation of load effects and GVW 26
Figure 4.17 Traffic monitoring 29
Figure 4.18 Simulation of multiple truck cases 30
Figure 4.19 Distribution of load effects to girders 32
Figure 4.20 The number of days on which multiple-truck events govern 34
Figure 4.21 Daily maximum moments fitted to Gumbel Type I distribution 34
Figure 4.22 Projection to 75-year load effect using Normal Probability Paper (Nowak 1999) 35
Figure 4.23 Daily maximum moment of 60 ft span simply supported bridge 37
Figure 4.24 Distribution of daily maximum moment (NBI bridge No.11877) 39
Figure 4.25 Distribution of 75-year maximum moment (NBI bridge No.11877) 39
Figure 4.26 Comparison of required/actual strength based on bridge year for RC bridges 42
Figure 4.27 Ratio of required/actual strength based on bridge year for PS I girder bridges 43
Figure 4.28 Ratio of required/actual strengths based on bridge year for PS double tee bridges 43
Figure 4.29 Comparison of required/actual strength based on bridge year for steel bridges 44
Figure 4.30 Comparison of required/actual strength based on span length for steel bridges 44
Figure 4.31 Bias and COV for moment in reinforced concrete bridges 49
Figure 4.32 Previous and updated distribution of moment resistance for RC bridges 49
Figure 4.33 Bias and COV for moment in prestressed bridges 50
Figure 4.34 Previous and updated distribution of moment resistance for PS concrete bridges 51
Figure 4.35 Bias and COV for moment in steel bridges 52
Figure 4.36 Previous and updated distribution of moment resistance for steel bridges 52
Figure 4.37 Effect of bias factors and COVs for resistance (positive moment) 53
Figure 4.38 Reliability indices in first span of representative bridges 54
Figure 4.39 Average reliability index for different ADTT values 55
Figure 5.1 Cost ratios for live load calibration factors (60ft, RC Slab bridge) 60
Figure 5.2 Cost impacts for live load calibration factors (60ft, RC Slab bridge) 60

LIST OF TABLES

Table 4.1 Construction materials of bridges in Missouri 6
Table 4.2 Material and construction type of bridges constructed after 1980 8
Table 4.3 WIM stations in Missouri 18
Table 4.4 Screening criteria in NCHRP Web-Only Document 135 21
Table 4.5 Most frequent locations of maximum positive moment 24
Table 4.6 Correlation coefficient between load effects and GVW 26
Table 4.7 Mean maximum load effect when subset of trucks were used (single trucks only) 27
Table 4.8 Bias factors and coefficient of variations for dead load (Nowak 1999) 40
Table 4.9 Previous material and geometric parameters for RC and prestressed bridges 45
Table 4.10 Updated material properties for reinforced concrete and prestressed bridges 46
Table 4.11 Geometric parameters 46
Table 4.12 Professional parameters 46
Table 4.13 Updated material parameters for plate steel 46
Table 4.14 Comparison of test case using previous material parameters 47
Table 4.15 Statistical parameters for moment resistance of reinforced concrete bridges 49
Table 4.16 Statistical parameters for moment resistance of prestressed bridges 50
Table 4.17 Statistical parameters for moment resistance of steel bridges 52
Table 5.1 Average reliability index for positive moment as a function of calibration factor and projected ADTT 57
Table 5.2 Average reliability index for shear force as a function of calibration factor and projected ADTT 58
Table 5.3 Proposed live load calibration factor for moment 59
Table 5.4 Proposed live load calibration factor for shear force 59
Table 5.5 Cost impact for different ADTT values 60
Table 7.1 Average reliability index for positive moment as a function of calibration factor and projected ADTT 63
Table 7.2 Average reliability index for shear force as a function of calibration factor and projected ADTT 64
Table 7.3 Proposed live load calibration factor for moment 65
Table 7.4 Proposed live load calibration factor for shear force 65

1 INTRODUCTION

Bridge design philosophies and specifications have been developed over the years to design bridges with a desired level of reliability. When AASHTO began publishing the standard specification for highway bridges in the 1930s, a factor of safety was used to ensure that the resistance of a bridge is greater than the combination of service loads. The design method is referred to as Allowable Stress Design (ASD). In the 1970s, AASHTO began applying different factors for each load in relation to the engineer's ability to predict that load. Hence, uncertainties in load prediction are considered through the load factors. This bridge design method is referred to as Load Factor Design (LFD). Recently, the bridge engineering profession has been moving toward the Load and Resistance Factor Design (LRFD). Load factors and resistance factors in the LRFD specifications were developed based on the reliability analysis of bridges. FHWA mandated that the AASHTO LRFD Bridge Design Specification should be used for new bridges on which states initiate preliminary engineering from October 2007.

The load factors in the AASHTO LRFD Specification have been calibrated through several NCHRP research projects. The NCHRP Project 12-33 produced NCHRP Report 368 (Nowak 1999), which is the basis of the current LRFD Specification. When the research was carried out, there was no reliable truck data available in the United States. Hence, the truck survey results from the Ontario Ministry of Transportation were used to calibrate the live load factor. The survey was carried out in the mid 1970's and collected weights of 9,250 heavy trucks. In the Project 12-33, it was assumed that the surveyed truck data from Ontario represented two-weeks of heavy traffic on a two lane bridge with Average Daily Truck Traffic (ADTT) of 1,000 in one direction. The development of live load factor was based on around 200 representative bridges from various geographical regions in the United States. These bridges were selected to cover a wide range of materials, types, and span of bridges. Using the reliability analysis results, the live load factor of 1.7 and load combination equation for strength limit state was proposed. In addition, the Project 12-33 proposed a new truck live load model, HL-93, which is currently being used in the LRFD Specification. In NCHRP Report 20-7/186 (Kulicki et al. 2007), the live load factor was increased from 1.7 to 1.75 due to the increase in the design ADTT from 1,000 to 5,000.

The development process of LRFD Specification in the 1980s and 1990s was thorough at the time of the research. Since then, more reliable truck weight data have been collected through the Weigh-In-Motion (WIM) system. Furthermore, the bridge data of the entire nation was organized in the National Bridge Inventory from which a typical bridge configuration of a specific state can be statistically identified. Thus, the load factors in the LRFD Specification can be refined for each state based on state-specific truck weights, traffic volumes, and bridge configurations. So far, the Michigan DOT has proposed an adjustment factor to increase the live load to account for heavy truck traffic in metropolitan areas (Van de Lindt et al. 2005). In addition, Oregon, Florida, and New York DOTs calibrated live load factors for Load and Resistance Factor Rating (LRFR) using Weigh-in-Motion (WIM) data collected in each state. The state-specific refinement of load factors for bridge design could result in uniform reliability and optimal design.

2 RESEARCH OBJECTIVE

The objectives of the research are twofold: 1) to calibrate the live load factor for Strength I Limit State in AASHTO LRFD Bridge Design Specification considering the typical bridge configurations and traffic environments in Missouri, and 2) to evaluate the effect of the load factor calibration on the up-front bridge construction cost.

3 OVERVIEW OF THE CALIBRATION PROCEDURE

The design equations in the LRFD Specification were developed based on the reliability analysis of bridges considering the uncertainties in the applied loads and in the resistance of bridge components. For instance, a typical design equation has the form of the following equation:

$$\phi R_n \geq \Sigma \gamma_i Q_i \tag{3.1}$$

where ϕ, R_n, γ_i, and Q_i correspond to resistance factor, nominal resistance, load factors, and service loads. For most structures, the nominal resistance, R_n, and the service loads, Q_i, are random in nature. The randomness of the parameters depends on the type of resistance or loads. Through the quantification of statistical parameters of the resistance and the loads, the reliability of a structure designed with the above equation can be evaluated. Based on the calculated reliability, the factors in Eq. (3.1) can be calibrated such that a structure can achieve a target reliability.

The uncertainties in loads can be evaluated based on the measurement of the applied loads. As dead load is mainly a function of density of materials and geometry of structural elements, it can be estimated with small margin of error. The statistical characterization of vehicular live loads, however, should be based on the effects of vehicle weights that a bridge may experience throughout the 75 years of design life of the bridge. Hence, the statistical evaluation of vehicular live loads requires a large number of weight data and statistical methods to project short-term observations to 75-years of bridge design life.

For the uncertainties in resistance, it is assumed that the *minimum required strength*, which is the factored loads on the right-hand side of Eq. (3.1), is the resistance of a bridge component. As the actual strengths of bridge components are higher than the minimum required strength if designed properly, the reliability analysis based on the minimum required strength results in more conservative reliability indices than those of actual bridges.

If both resistance and loads follow normal distribution, the reliability index can be calculated as

$$\beta = \frac{\mu_R - \Sigma \mu_{Qi}}{\sqrt{\sigma_R^2 + \Sigma \sigma_{Qi}^2}} \tag{3.2}$$

where μ and σ correspond to the mean and standard deviation of resistance R or loads Q. If the random numbers do not follow the same distribution, β can still be derived but approximate methods such as First Order Reliability Method (FORM), or numerical methods such as Monte-Carlo Simulation, can also be used to calculate the reliability index.

In this research, the focus of the load factor calibration is on Strength I Limit State in AASHTO LRFD Bridge Design Specification (2007). The load combination for the Strength I Limit State is

$$\phi R_n \geq 1.25\text{DC}+1.5\text{DW}+1.75(\text{LL}+\text{IM}) \tag{3.3}$$

where DC, DW, LL, and IM correspond to the dead load of structural components and nonstructural attachments, dead load of wearing surface, vehicular live load, and vehicular dynamic load allowance, respectively. The load combination in Eq. (3.3) provides a *minimum required strength,* R_n.

The calibration of live load factor is based on the typical bridge configurations and truck traffic environments in Missouri. A subset of National Bridge Inventory (NBI) database is used to identify the typical bridge configurations of the state. Truck weights measured in 24 Weigh-In-Motion (WIM) stations in the state are processed to evaluate the effects of trucks on bridge girders. A projection method based on the extreme value theory is used to project the daily maximum load effects to a long-term (75 year) maximum load effects. Figure 3.1 shows the overview of the calibration procedure and the corresponding sections in this report.

In Section 4.1, representative bridges in Missouri are selected based on the NBI database. Several parameters defining the structural characteristics of bridges, such as construction materials, superstructure types, span lengths, number of spans, and number of lanes, are used as main parameters when selecting the representative bridges. The analysis of Average Daily Truck Traffic (ADTT) of the bridges in Missouri is also presented.

In Section 4.2, a procedure for pre-processing of WIM data is introduced. Approximately 41 million WIM data from 22 permanent WIM stations and 2 temporary WIM stations are processed. Outlier data points are screened using the criteria suggested in NCHRP Report 135 and inspection of average daily gross vehicle weight.

In Section 4.3, the location of maximum load effects along the length of a span is estimated. Excessive computational time would be required to analyze numerous moving loads at many points on a bridge span to find the maximum load effect along the length of the span. Hence, based on analyses of a few representative bridges, the location of maximum load effect along the length of a bridge span is found to reduce the computational demand.

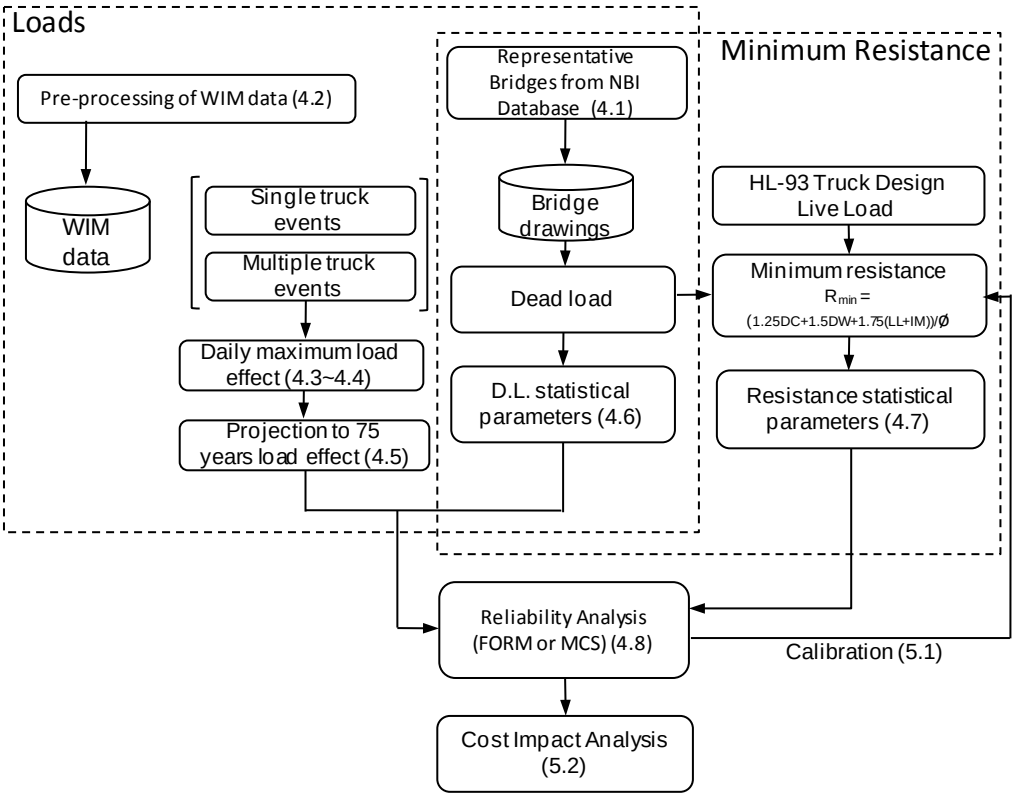


Figure 3.1 Overview of the live load factor calibration procedure

In Section 4.4, simulation methods for single truck events and multiple truck events are proposed. As the maximum load is used for design of bridge girders, the top 5% of heavy trucks are used to analyze bridges for single truck events. For multiple truck events, a statistical method is used to simulate the distance between trucks running in series or running in parallel. The maximum load effects to an internal girder are simulated considering both the single and multiple truck events.

The maximum daily load effects for 100 days are simulated using the method in Section 4.4. These simulated daily maximum values are used to define the statistical distribution of daily maximum load effects. The daily maximum load distribution is projected to evaluate the distribution of 75 year maximum load effects. Two projection methods are compared, and the one which results in more consistent and conservative values is adopted in Section 4.5.

In Section 4.6, the statistical parameters of dead loads are presented. The bias factor and COV for the dead loads are primarily based on previous studies.

In Section 4.7, the statistical distribution of minimum required strength is presented. The dead load effect from Section 4.6 and the live load effect using the HL-93 design truck are used to calculate the minimum required strength. Bias factor and coefficient of variation of the resistance are evaluated based on uncertainties in material, fabrication, and professional parameters.

Based on the statistical parameters for loads (Section 4.5 and 4.6) and resistance (Section 4.7), the reliability indices of the representative bridges are calculated in Section 4.8. The First-Order Reliability Method is used for the reliability analysis.

Calibration factors are proposed in Section 5.1 as a function of the Average Daily Truck Traffic (ADTT). A target reliability index of 3.5 is used for the calibration. The impact of the calibrated live load factor on the bridge construction cost is presented in Section 5.2. The cost impact analysis is based on the relationship between construction cost and load effects of several design trucks presented in a previous study. Section 6 summarizes findings from the study.

4 TECHNICAL APPROACH

4.1 Selection of Representative Bridges in Missouri

The Federal Highway Administration (FHWA)'s National Bridge Inventory (NBI) database includes detailed information on structural characteristics of bridges such as construction materials, superstructure types, number of spans, span lengths, construction year, average daily truck traffic, etc. Several regions in the nation have different bridge design practices as design loads, especially earthquake and wind loads, and geotechnical environment, such as depth to bedrock and soil conditions, vary from one region to another. Thus, the statistical analysis of a subset of the NBI database for a state can provide information on typical bridge configurations of a state. As the calibrated live load factor in this study will be applied to the state of Missouri, the subset of NBI database for Missouri is analyzed and representative bridges are selected in this section.

4.1.1 Statistical Analysis of Bridges in Missouri in 2007 NBI Database

The 2007 NBI database is analyzed to identify the typical bridge configurations in Missouri. In the database, there are 24,120 bridges in the state including culverts and bridges maintained by counties. As the focus of this study is on the bridges that are maintained by the state, all culverts are excluded from the database in addition to the bridges maintained by counties. After the screening, it is found that 7,094 bridges are maintained by the state.

The 7,094 bridges are analyzed in terms of construction material, superstructure type, construction year, number of spans, and span lengths. Almost all bridges are constructed with either reinforced concrete, steel, or prestressed concrete as summarized in Table 4.1. In the following section, the statistical analysis of bridges for each material type is presented.

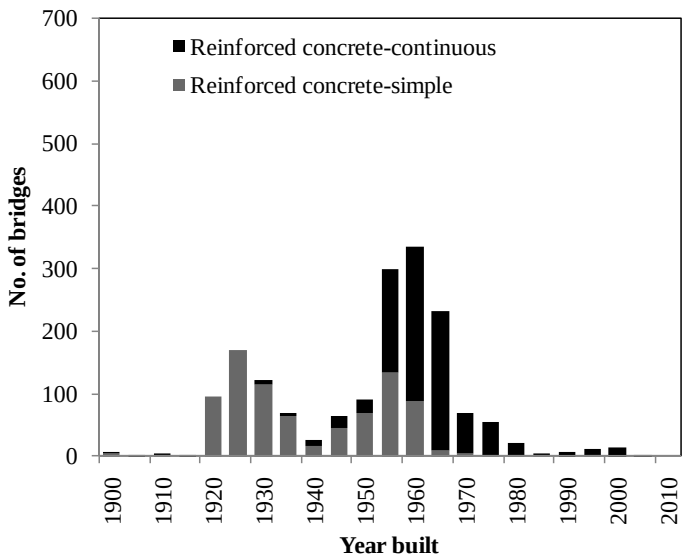
Table 4.1 Construction materials of bridges in Missouri

Material	Type of material and/or design	Number of bridges	Relative frequency (%)
1	Reinforced concrete – simply supported	821	11.57
2	Reinforced concrete – continuous	881	12.42
3	Steel – Simply supported	1457	20.54
4	Steel – continuous	2505	35.31
5	Prestressed concrete – Simply supported	107	1.51
6	Prestressed concrete – continuous	1309	18.45
7	Wood or Timber	13	0.81
8	Masonry	1	0.01
	Total	7904	100%

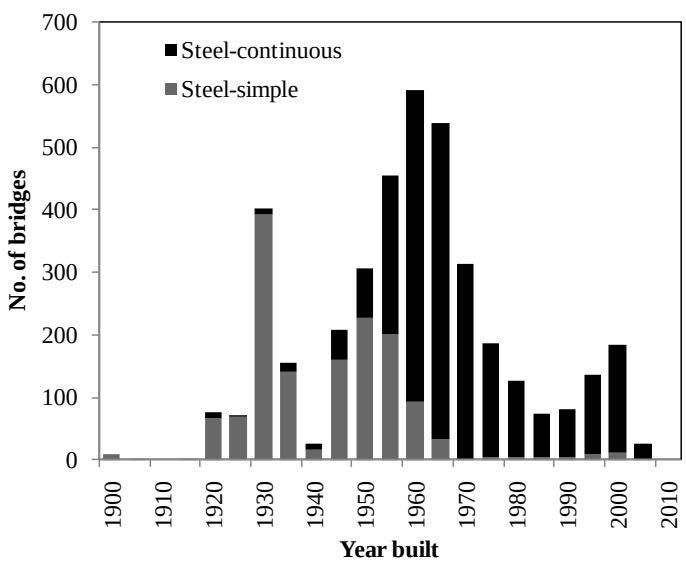
Construction Year

Figure 4.1 presents the construction years of bridges maintained by Missouri Department of Transportation (MoDOT). Many reinforced concrete and steel bridges were constructed in the 1930s. The number of bridges constructed in the 1940s is relatively small due to the involvement of the U.S. in the 2nd World War. By the late 1950s, the highway construction boom had begun and interstate highways began to appear. A large number of reinforced concrete and steel bridges were constructed between the 1950s and 1970s. Total 1,932 bridges have been constructed after

the 1980s, among which only a few bridges were constructed with reinforced concrete. Most of the bridges constructed after the 1980s are either steel or prestressed concrete bridges. As it can be observed from the Figure 4.1, the trend of bridge construction material changed from reinforced concrete and steel bridges to prestressed concrete and steel continuous bridges. The selection between steel and prestressed concrete bridges largely depends on the relative cost of steels and concrete when a bridge is constructed. As the bridge construction trends vary with time, the statistical analysis of bridge configuration is carried out with bridges constructed after the 1980s.

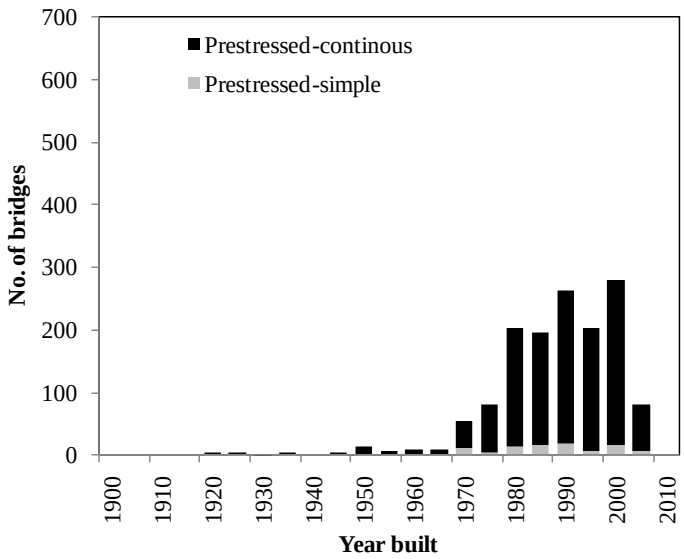


(a) Construction years of reinforced concrete bridges



(b) Construction years of steel bridges

Figure 4.1 Construction years of bridges in Missouri (*contd.*)



(c) Construction years of prestressed concrete bridges

Figure 4.1 Construction years of bridges in Missouri

Table 4.2 Material and construction type of bridges constructed after 1980

Type	Material	Superstructure type	Number of bridges constructed after 1980
1	Reinforced concrete – Simply Supported	Slab	4
2	Reinforced concrete – continuous	Slab	55
3		Other	2
4	Steel – Simply supported	Girder	31
5		Other	8
6	Steel – continuous	Girder	590
7		Other	7
8	Prestressed concrete – Simply Supported	I Girder	59
9		Double-tee	15
10		Other	5
11	Prestressed concrete – continuous	I-Girder	958
12		Double tee	190
13		Other	8

Predominant Superstructure Type

A superstructure can be designed in different load carrying systems even with a same material. For instance, steel can be used for truss bridges as well as girder bridges. In the NBI database, the superstructure type is indicated in Item 43B with 23 types of design and/or construction. Among the 1,932 bridges constructed after 1980, most continuous steel bridges are girder type bridges. Only a few simply supported steel bridges are truss type bridges. Approximately 80% of prestressed concrete bridges are I girder type bridges and most of the rest are double tee beam bridges. Most continuously supported reinforced concrete bridges are slab type bridges. Only

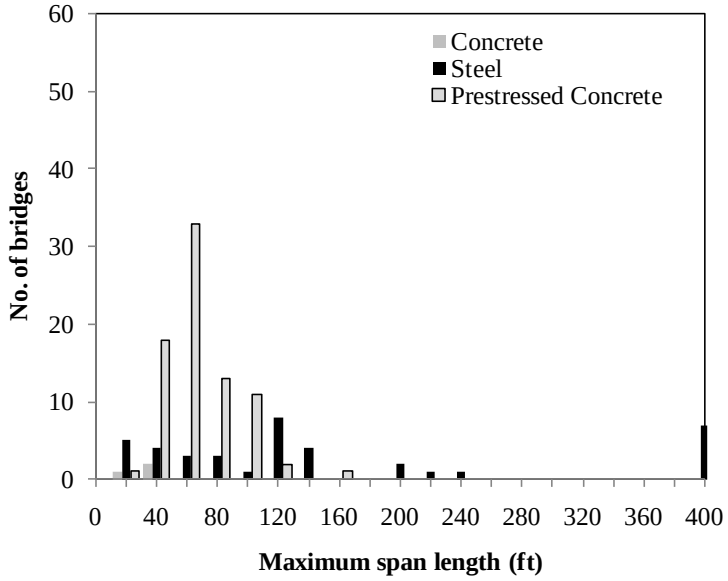
four simply supported reinforced concrete bridges were constructed after 1980. Based on these statistics, slab and girder type bridges are selected as representative types of reinforced concrete and steel bridges, respectively. Also, I girder shape and double-tee shape type bridges are selected as representative types of prestressed concrete bridges. The simply supported reinforced concrete bridges are not included in the analysis as only a few bridges of the type were constructed after 1980. Based on the above, a total of seven bridge types, 2, 4, 6, 8, 9, 11, and 12 in Table 4.2 are used for reliability analysis.

Bridge Configurations

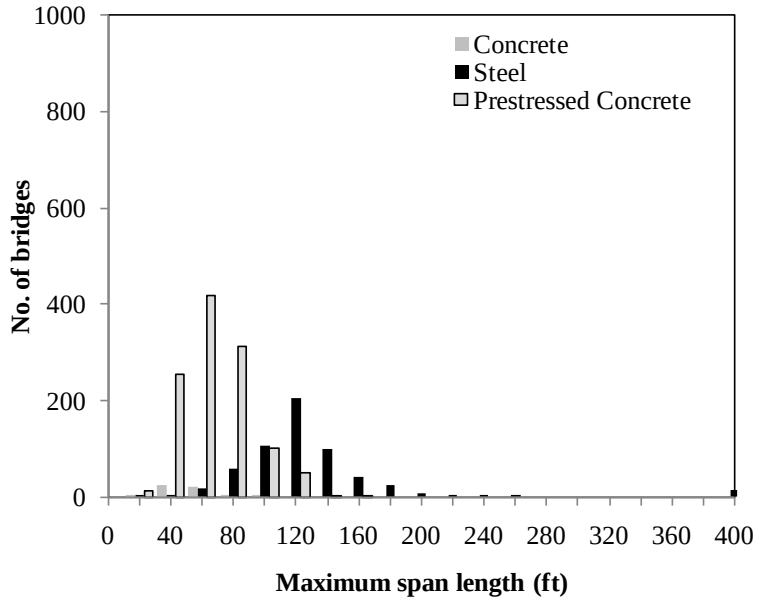
The statistics of maximum span length, number of lanes, and number of spans for the bridges constructed after 1980 are analyzed.

The distribution of span length for simply supported bridges (bridges with no negative moment resistance designed at bents) is presented in Figure 4.2(a), and for continuous bridges Figure 4.2(b). The average maximum span lengths for simply supported bridges are 22.3ft, 122.3ft, and 66.1ft for reinforced concrete, steel, and prestressed concrete bridges, respectively. The average maximum span lengths for continuously supported bridges are 47.9ft, 120.0ft, and 64.9ft for reinforced concrete, steel, and prestressed concrete bridges. The distribution of the maximum span length is close to log-normal distribution.

Figure 4.3 shows the distribution of the number of lanes for the three material types. Two-lane bridges are most prevalent for all of materials. About 79% of bridges have one or two lanes; only 21% have three or more lanes. In terms of number of spans, three-span bridges are the most frequently used as shown in Figure 4.4. Most single-span bridges are prestressed concrete (41%) or steel (54%) bridges. About 81% of two-span bridges are continuously supported steel bridges.

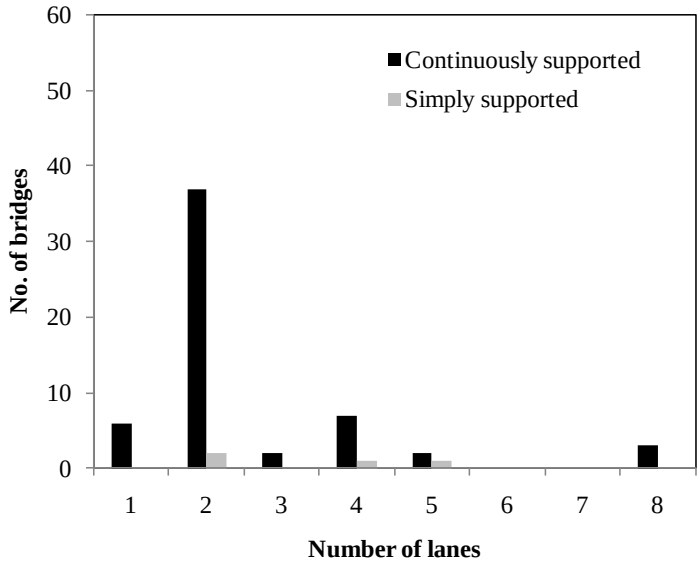


(a) Simply supported bridges

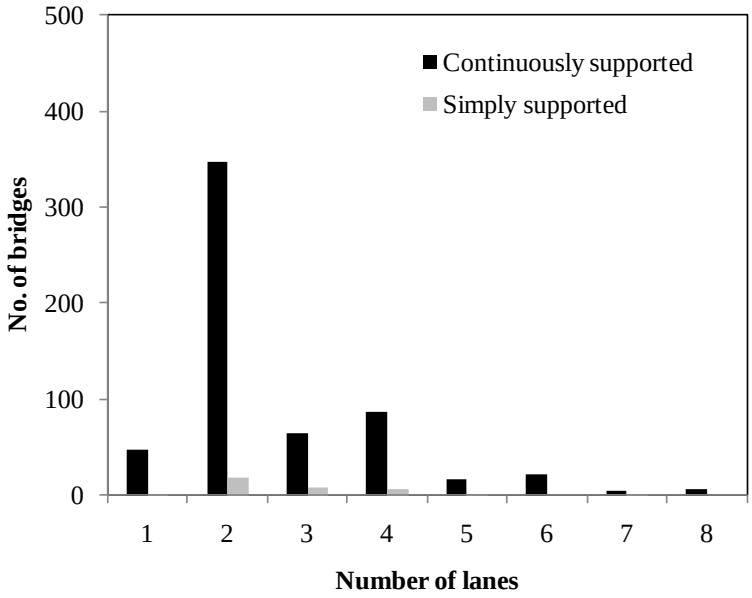


(b) Continuous bridges

Figure 4.2 Distribution of maximum span lengths

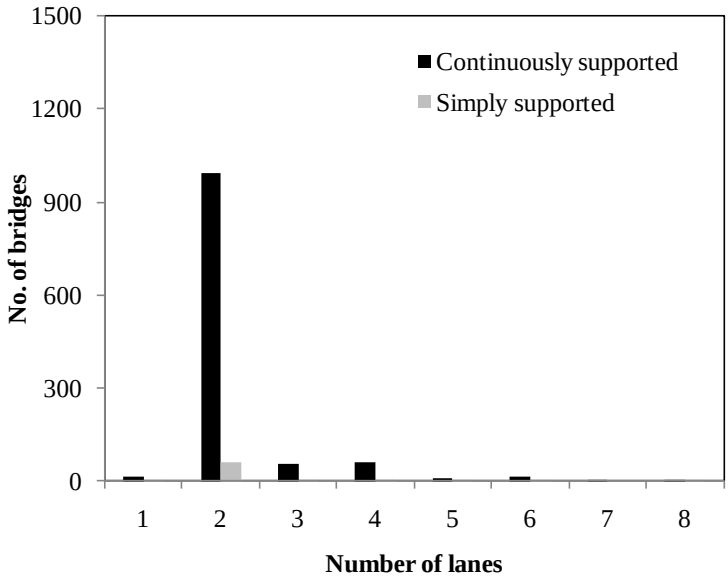


(a) Reinforced concrete bridges



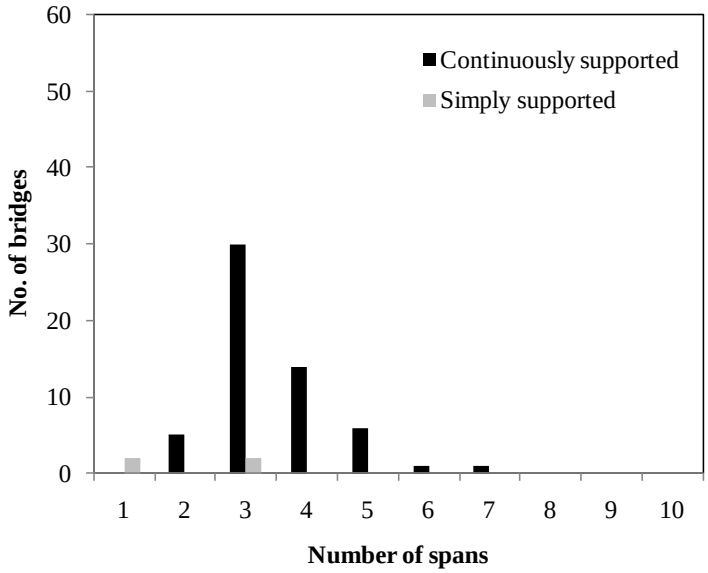
(b) Steel bridges

Figure 4.3 Distribution of number of lanes (*contd.*)



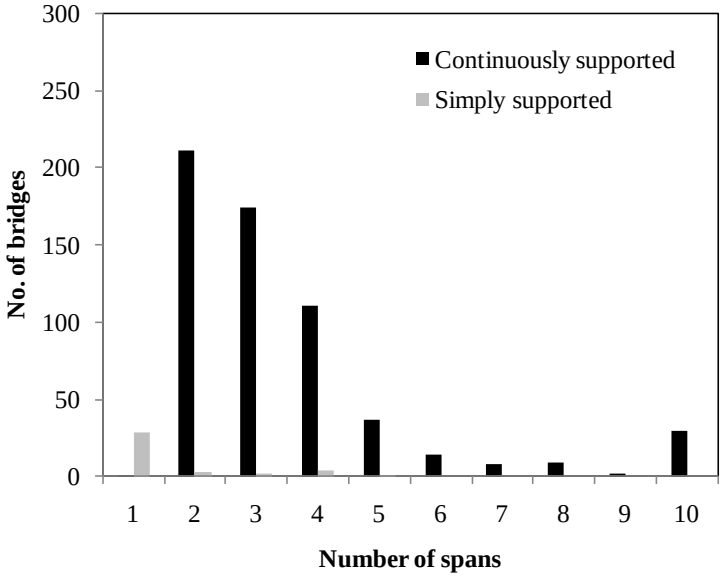
(c) Prestressed concrete bridges

Figure 4.3 Distribution of number of lanes

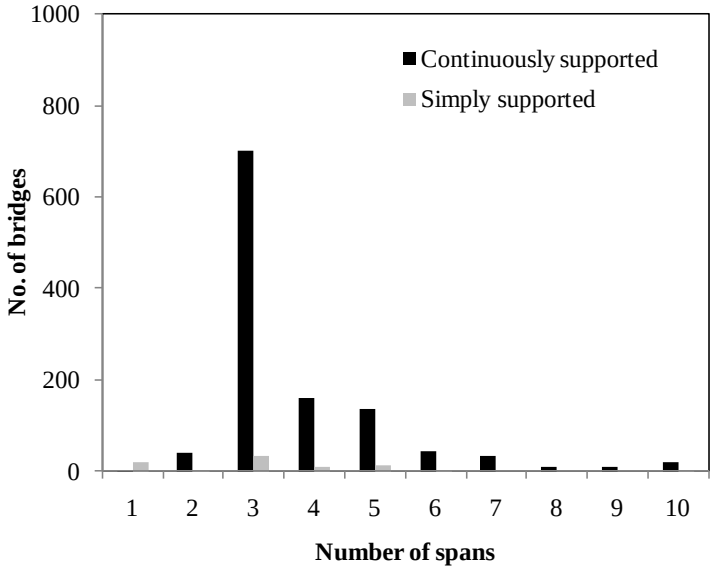


(a) Reinforced concrete bridges

Figure 4.4 Distribution of number of spans (*contd*.)



(b) Steel bridges



(c) Prestressed concrete bridges

Figure 4.4 Distribution of number of spans

Selection of Representative Bridges

Central Composite Design (CCD) method (Box and Wilson, 1951) is used to select representative bridges for Bridge Types 2, 4, 6, 8, 9, 11, and 12 in Table 4.2. There are several variations of CCD method in literature. In this research, the modified Central Composite Circumscribed (CCC) method is used as it has been commonly adopted in reliability studies in Civil Engineering. Representative bridges are selected from continuously supported reinforced concrete bridges (slab type, Type 2 in Table 4.2), simply and continuously supported steel bridges (girder type, Types 4 and 6), and simply and continuously supported prestressed concrete bridges (I girder type and double-tee type, Types, 8, 9, 11, and 12) constructed after 1980. Figure 4.5 illustrates the sampling points in the CCC method with three random variables.

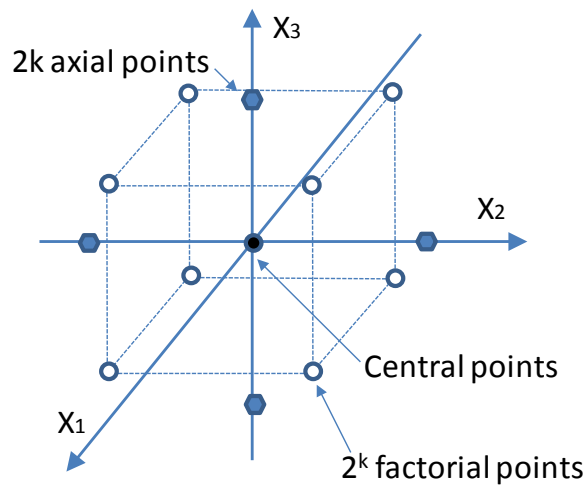


Figure 4.5 Sampling points in CCC method for three random variables

As depicted in the figure, for k number of random variables, the target sampling points in the CCC consist of 2^k factorial points, a center point, and 2k axial points on the axis of each random variable. The factorial points are located at an equal distance of standard deviation, σ, from the center point, and the axial points are located on the axis of the random variable and at a distance of $\sqrt{2}\sigma$ from the center point. The total number of sampling points in the CCC method is $N=2^k+2k+1$. In this study, the number of spans, the number of lanes, and the maximum span length are selected as random variables. As three random variables are considered, fifteen bridges ($N=2^3 + 2 \times 3 + 1=15$) are selected for each bridge type. A total of 105 bridges (7 bridge types x 15 bridges per each type) are selected as representative bridges.

The selected random variables related to the bridge configurations cannot have negative values. For these random variables, it is more appropriate to choose lognormal distribution rather than normal distribution. For lognormal distribution, the normalized random variable, X_i, is defined as

$$X_i = \frac{\ln(x_i) - \lambda}{\zeta} \tag{4.2}$$

where λ and ζ are the location parameter and scale parameter of lognormal distribution and x_i is a random variable. For X_i random variable space, the sampling points can be defined as a matrix, $[X]$,

$$[X]=\begin{array}{c} \begin{array}{ccccccccccccccc} C_1 & F_1 & F_2 & F_3 & F_4 & F_5 & F_6 & F_7 & F_8 & A_1 & A_2 & A_3 & A_4 & A_5 & A_6 \end{array} \\ \begin{bmatrix} 0 & +1 & +1 & +1 & +1 & -1 & -1 & -1 & -1 & \sqrt{2} & -\sqrt{2} & 0 & 0 & 0 & 0 \\ 0 & +1 & +1 & -1 & -1 & +1 & +1 & -1 & -1 & 0 & 0 & \sqrt{2} & -\sqrt{2} & 0 & 0 \\ 0 & +1 & -1 & +1 & -1 & +1 & -1 & +1 & -1 & 0 & 0 & 0 & 0 & \sqrt{2} & -\sqrt{2} \end{bmatrix} \end{array} \tag{4.3}$$

where C_1 is central point, F_1 through F_8 are factorial points, and A_1 through A_6 are axial points. After the three random variables are normalized, bridges located at the closest distance from each sampling point are selected. Appendix A summarizes the list of selected bridges using the CCC method. Bridge drawings of 105 bridges were acquired from MoDOT. Seven bridges could not be used for reliability analysis because the drawings of the bridges did not give enough information to calculate dead load.

4.1.2 Bridges used in Initial LRFD Calibration in NCHRP 368 Project

In the NCHRP Report 368, the hypothetical bridges were used for calibration. The analysis in NCHRP Report 368 was focused on girder type bridges, including steel non-composite and composite beams, reinforced concrete T-beams, and prestressed concrete AASHTO type girders. The girders have spans from 30 to 200ft with girder spacing from 4 to 12ft. In Figure 4.6 the girder spacing used in NCHRP Report 368 (4 to 12 ft) is compared with the girder spacing of representative bridges in Missouri. As it can be observed from the figure, most of representative bridges in Missouri have narrower band of girder spacing between 6 and 10 ft. Figure 4.7 compares the span length used in the initial calibration study with those of representative bridges in Missouri. The figure clearly shows that the bridges in Missouri have relatively short spans in comparison with the bridges used in NCHRP Report 368.

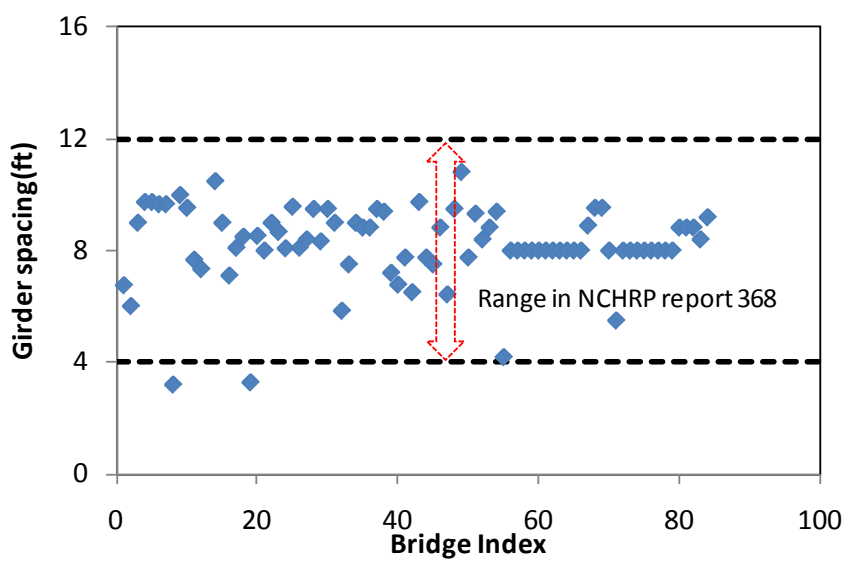


Figure 4.6 Distribution of girder spacing of representative bridges

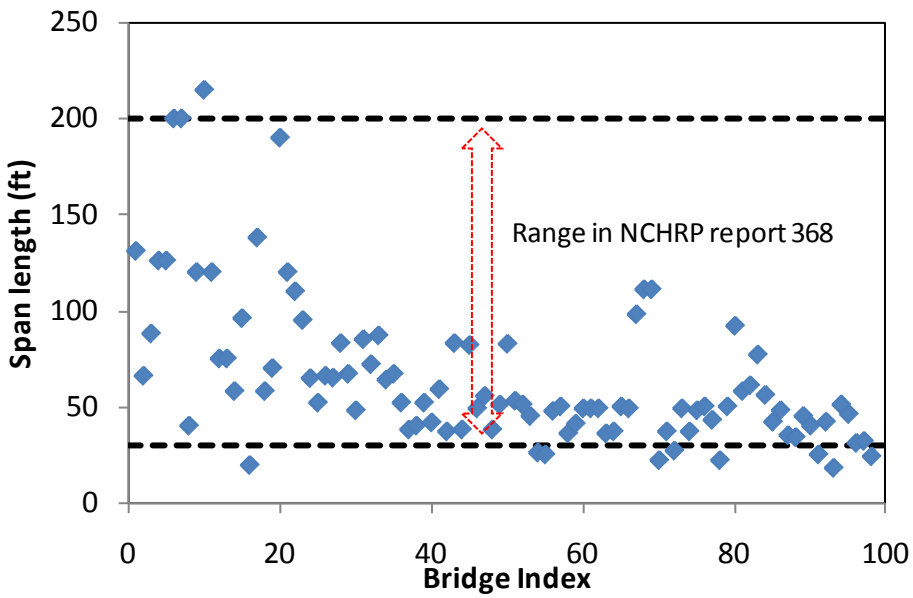


Figure 4.7 Distribution of span length for representative bridges

4.1.3 Average Daily Truck Traffics of Bridges in Missouri

The NBI database includes data fields for Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT). As the heavy trucks mainly govern the design of bridges, the ADTT is the main parameter that needs to be considered when the reliability of a bridge is evaluated. The live load factor in the current AASHTO LRFD specification was initially calibrated with ADTT of 1,000 in NCHRP Report 368 (Nowak 1999). In the recent calibration study, NCHRP Report 20-7/186(Kulicki et al. 2007), ADTT of 5,000 was used for load factor calibration.

The geographical distribution of ADTTs in Missouri is depicted in Figure 4.8. Each mark in the figure indicates the location of a bridge. The height of the mark shows the relative ADTT. As it can be observed from the figure, a high volume of truck traffic is concentrated in the metropolitan areas of Kansas City and St. Louis. For many bridges in rural areas, the ADTT is relatively very small. Figure 4.9 compares the distribution of ADTT for bridges in the Interstate (IS) highway system and for bridges in the US highway system. The average ADTT for IS highway bridges is close to 5,000; which was used for the calibration of the live load factor in current AASHTO LRFD. On the other hand, the average ADTT of the US highway bridges is only around 1,100 and bridges on other roads have an average ADTT of 415 only. Considering that the majority of bridges in Missouri are US highway or local roads, 1257 (18%) and 4960 (70%) bridges respectively, a different live load factor may need to be applied for the bridges. The discussion on the effect of ADTT on the reliability of bridges is provided in more detail in Section 4.7.

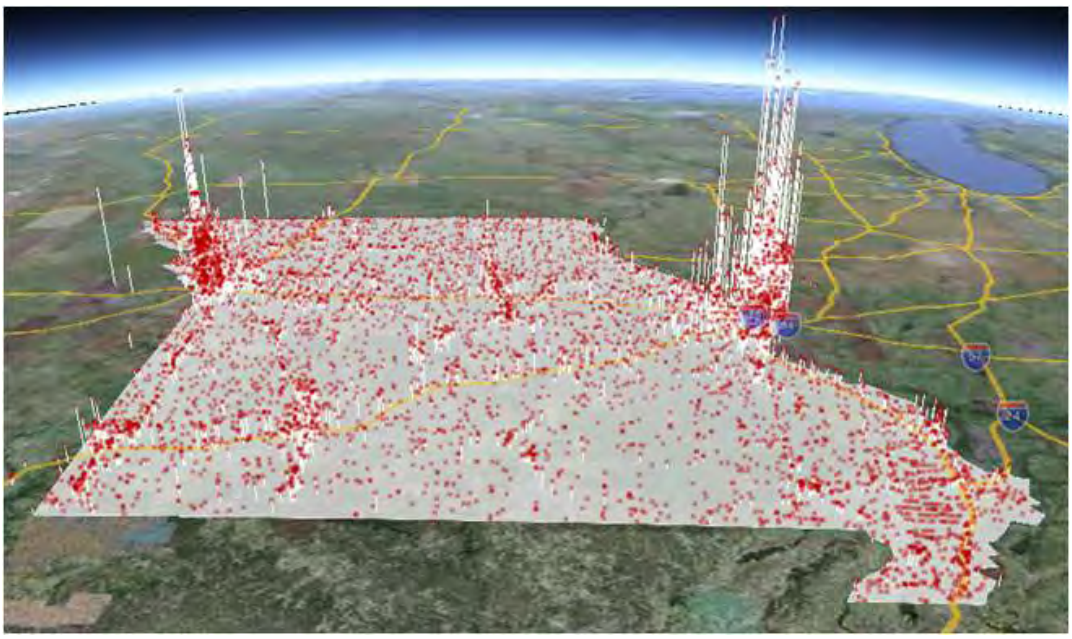


Figure 4.8 ADTTs of bridges in Missouri

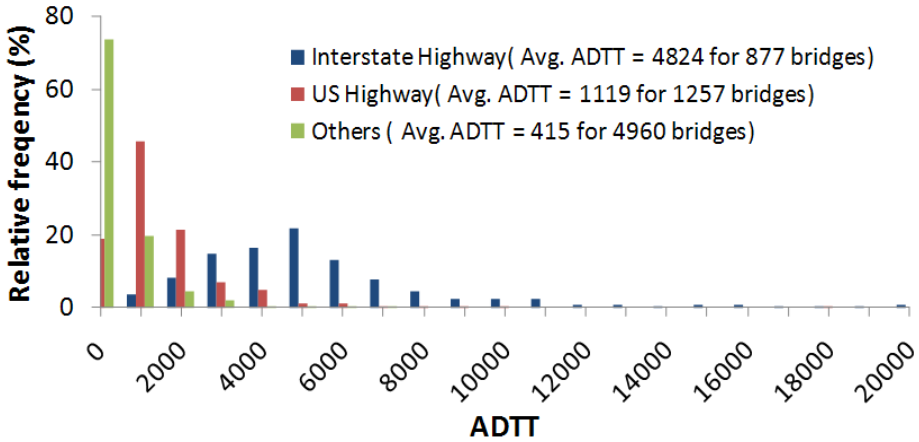


Figure 4.9 Distribution of ADTTs for Missouri IS highway and US highway

4.2 Screening of Measured Truck Weights from WIM Station

Traffic data collected at WIM stations in Missouri are used to evaluate effects of live loads on bridge girders. WIM stations record the axle weights, axle spacing, and types of vehicles while the vehicles pass the stations running at highway speed. As the high-speed measurement in WIM stations is prone to various errors, the data used in this study are inspected and screened based on an engineering judgment and the criteria suggested in NCHRP Web-Only Document 135 (Sivakumar et al. 2008).

4.2.1 Measured WIM Data

The WIM data used in this research were collected for five years between 2004 and 2008 at 22 permanent and 2 temporary WIM stations in Missouri. The locations of the WIM stations are presented in Figure 4.10. Table 4.3 presents the WIM station's identification number, the route on which each WIM station is located, and the travel direction of the route.

Approximately 41 million WIM data were collected at the 24 WIM stations. Except four WIM stations (4201, 4413, 7403, and 7602), two WIM stations are located at a same site but in different directions to collect data on vehicles traveling in both directions. Approximately 84% of this data was recorded on IS highways.

Table 4.3 WIM stations in Missouri

Station ID	Route	Direction	Station ID	Route	Direction
1821	IS 29	North	4413	US 65	South
1823	IS 29	South	5002	IS 70	East
1881	IS 35	North	5004	IS 70	West
1883	IS 35	South	6101	IS 55	North
2001	US 63	North	6103	IS 55	South
2003	US 63	South	7403	US 71	South
2021	US 63	North	7602	IS 44	East
2023	US 63	South	9202	US 60	East
3021	US 61	North	9204	US 60	West
3023	US 61	South	9302	IS 44	East
4201	IS 435	North	9304	IS 44	West
5212	Temporary		5214	Temporary	

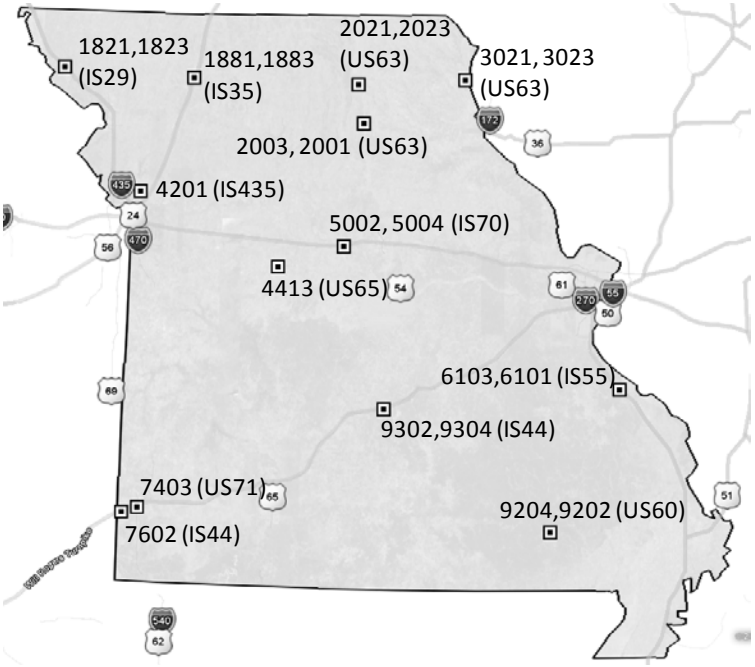
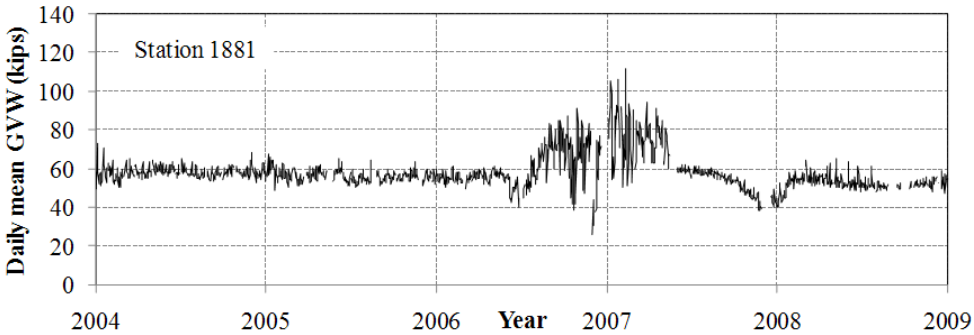


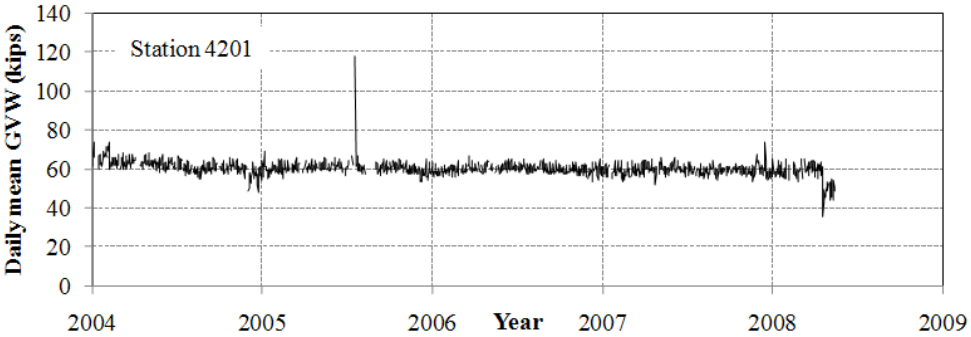
Figure 4.10 Location of WIM stations in Missouri

4.2.2 Screening Criteria for WIM Data

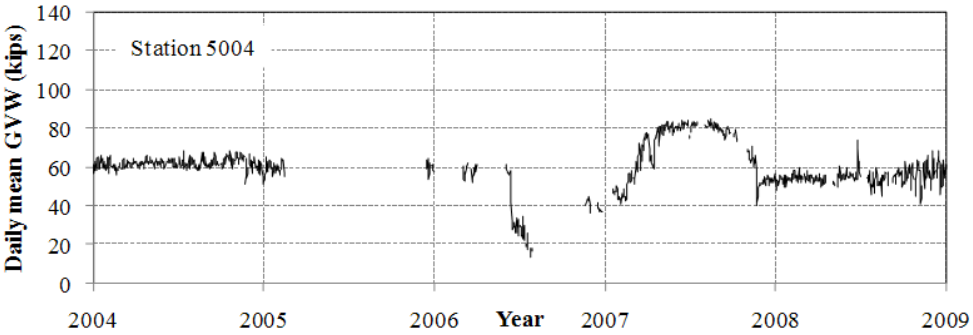
The WIM stations record the weight of each axle and axle spacing of trucks. As the weights are measured while trucks run at highway speed, the recorded data is prone to error. Careful inspection of recorded data revealed that a non-negligible portion of data has unrealistically large axle weight, gross vehicle weight, or axle spacing. In addition, several stations showed that for a certain period of time, the daily average gross vehicle weights (GVW) consistently exceeded the legal limit of 80 kip. For instance, Figure 4.11 presents average daily GVW of three WIM stations. It can be observed from Figure 4.11(a) that for one year beginning in the middle of 2006, the daily average GVW fluctuates often exceeding the legal limit of 80 kip. This period of measurement (one year beginning the middle of 2006) can be considered as erroneous presumably due to an error in the measurement system. For the Station 4201 in Figure 4.11(b), average daily GVW exceeded 110 kip for a short period. Truck weights can occasionally exceed legal limit due to negligence of trucking companies or when overweight trucks are permitted by the Department of Transportation. But having average daily truck weights exceeding 80 kip appears to be an error considering not many overweight trucks are permitted. For the period of the erroneous weight measurement, the number of truck traffics was also reviewed to confirm that the large daily average GVW was not from a limited number of measurements. Station 5004 in Figure 4.11(c) has a large gap between measurement as well as large fluctuations in the daily average GVW. After inspection of daily average GVW from all WIM stations, the erroneous data segments are removed from the database.



(a) Station 1881 showing erroneous measurement between 2006 and 2008



(b) Station 4201 showing large peak



(c) Station 5004 showing fluctuation and gaps in avearge daily GVW

Figure 4.11 Daily average GVW of selected WIM stations

After screening WIM data based on the daily average GVW as discussed above, the screening criteria suggested in NCHRP Web-Only Document 135 (Sivakumar et al. 2008) is applied to filter outlier data points. Any vehicle exceeding parameters presented in Table 4.4 were removed from the data set. Most criteria in Table 4.4 screened vehicles with unusual axle weight or axle spacing. For instance, vehicles with wide axle spacing may actually represent two vehicles separated with the measured spacing. In addition, all light-weight vehicles in the FHWA's Vehicle Class 1 through 7 (Appendix B), such as cars and pickup trucks, are also removed from the database. Approximately 61% of WIM data are filtered and not used for statistical analysis. About 34% among these filtered WIM data were screened by the criteria in Table 4.4 and remaining were screened based on the daily average GVW.

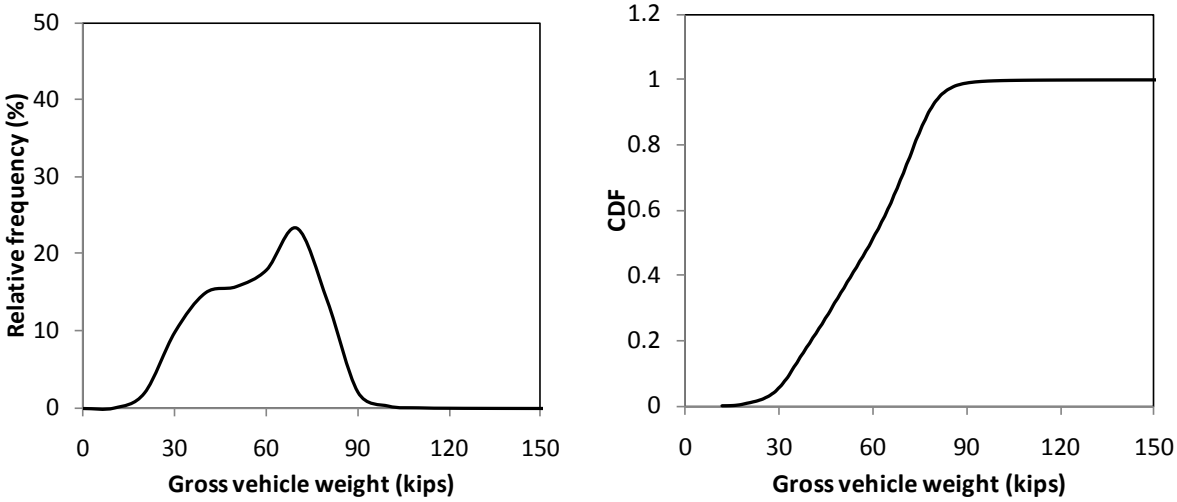
Table 4.4 Screening criteria in NCHRP Web-Only Document 135

	Item	Criteria
1	Vehicle Length	< 120ft
2	Total axle count	> 2
3	Gross vehicle weight	>12kips
4	Any axle weight	<70kips
5	Any axle weight	>2kips
6	Steer axle weight	< 25kips
7	Steer axle weight	> 6kips
8	First axle spacing	>5ft
9	Any axle spacing	>3.4ft
10	Ratio of GVW to the sum of axle weight	< 110%

4.2.3 Statistical Characteristics of the Screened WIM Data

Figure 4.12 presents the probability density and cumulative distribution function (CDF) of GVW of screened WIM data. The mean GVW of the screened WIM data is around 60 kips, which is approximately 75% of the legal limit. The maximum recorded GVW is 276 kips, which is from a truck with 9 axles and total length of 105.9 ft. Due to the spacing of the axles, the largest GVW may not have a significant effect on the force demand on bridge girders. It is assumed that overweight trucks in the processed database are either permitted by Department of Transportation or represent the possible overweight due to negligence of trucking companies. Hence, it is assumed that the screened WIM data represent the actual truck weights that a bridge in Missouri would experience.

The WIM data includes the time when the weight measurement was recorded. Through the analysis of the hourly distribution of trucks, the time of the busiest truck traffic can be identfied. Figure 4.13 presents hourly distribution of truck traffic. It can be observed from the figure that around 52.9% of trucks travel between 9a.m. and 6p.m. In Section 4.4, field monitoring of traffic flow is presented. The field monitoring was carried out to identify the distribution of trucks in the bypass lane and in the travel lane. The times for the field observation was determined based on the busiest time of truck traffic in a day in Figure 4.13.



(a) Probability density function (b) Cummulative distribution function

Figure 4.12 Distribution of GVW

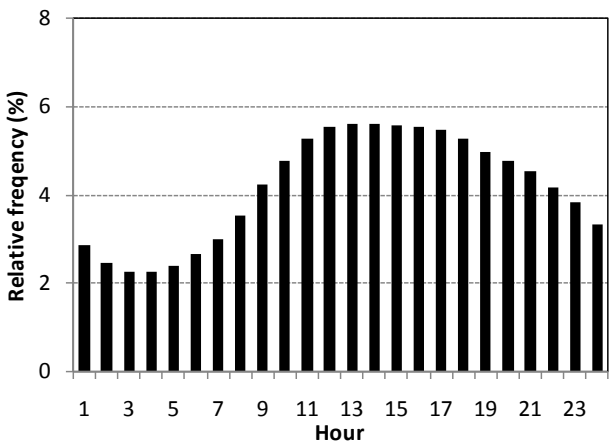


Figure 4.13 Hourly distribution of truck traffic (hours from midnight)

4.3 Location of Maximum Positive Moment

When a bridge is subjected to a moving vehicular load, the location of maximum moment varies depending on the axle configurations and distribution of weight to each axle. To identify the maximum moment and its location on a bridge span, moving load analyses is carried out and the moment along the length of a span compared. For the reliability analysis of 98 bridges a large number of simulations need to be carried out. In this research, daily maximum load effects of 100 days are calculated for each bridge. For each day, several hundreds of analyses need to be carried out to find maximum load effects. In total, several millions of analyses are required. To minimize the computational demand without sacrificing the accuracy in the analysis, it is necessary to identify the location of maximum moment on a bridge and use smaller degree of freedom for the moving load analyses.

Several hypothetical bridges are used to identify the location of maximum moment. Considered bridges have 20ft, 40ft, 60ft, 80ft, 100ft, and 150ft spans. One to three spans of equal length are used for the analysis. 10,000 Class 9 trucks measured at WIM station 1821 are used for moving load analysis. Figure 4.14 shows the distribution of location of maximum moment when a 40ft span bridge is analyzed with the randomly selected trucks. For a single span bridge, the maximum positive moment is developed most frequently at 0.46L of a span where L is the length of the span. The maximum moment does not always develop in the middle of the span as the configurations of trucks are not symmetric. For two and three span bridges, the maximum positive moment mostly occur at 0.4L from the end of the spans. For the middle span of the three-span bridges, the positive maximum moment is developed most frequently around the middle of the span. Similar analyses are carried out for different span lengths. The most frequent locations of the maximum positive moments are summarized in Table 4.5. The location of maximum negative moment is at the intermediate supports for all bridges.

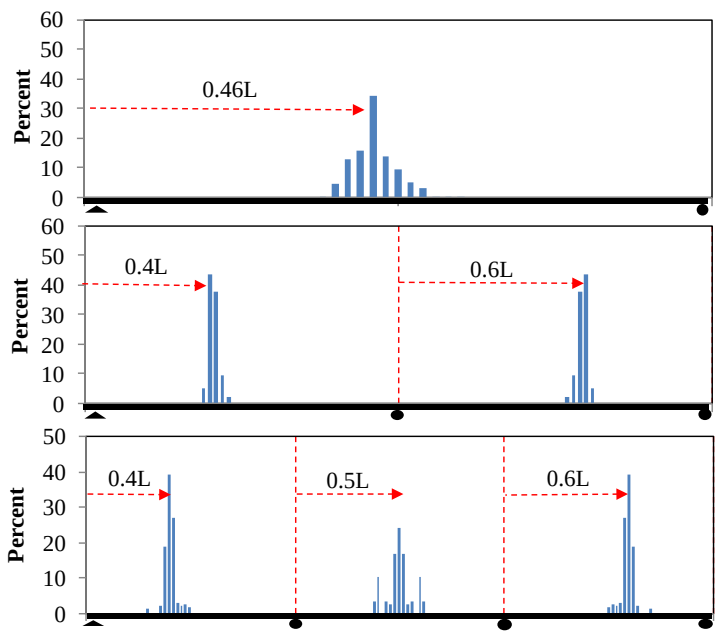


Figure 4.14 Distribution of maximum moment location

Table 4.5 Most frequent locations of maximum positive moment

Number of spans	Span No.	Most frequent location (L = span length)						Proposed location to check max. moment
		L=20ft	L=40ft	L=60ft	L=80ft	L=100ft	L=150ft	
1- span	1	0.44L	0.46L	0.46L	0.46L	0.46L	0.48L	0.46L
2-span	1	0.40L	0.40L	0.42L	0.40L	0.40L	0.40L	0.40L
	2	0.60L	0.60L	0.58L	0.60L	0.60L	0.60L	0.60L
3-span	1	0.40L	0.40L	0.42L	0.40L	0.40L	0.46L	0.40L
	2	0.50L	0.50L	0.44L& 0.56L	0.44L& 0.56L	0.44L& 0.56L	0.48L& 0.52L	0.50L
	3	0.60L	0.60L	0.58L	0.60L	0.60L	0.54L	0.60L

To confirm the effect of using a fixed point to find maximum positive moment, the maximum positive moment at the fixed location proposed in Table 4.5 is compared with the maximum positive moment along the span of a beam in Figure 4.15. The analyses are carried out with 40 ft span bridges. It can be observed from the figure that the statistical distribution of moment at the fixed point in Table 4.5 is very close to that of the actual maximum moment along the length of the bridge. This approach is still approximate in a sense that many continuous bridges do not have equal spans. In addition, if the maximum load effects are developed by multiple trucks, the location of maximum load effects may vary. Due to limited computational resources, however, the locations on spans identified in the above are used to identify maximum load effects.

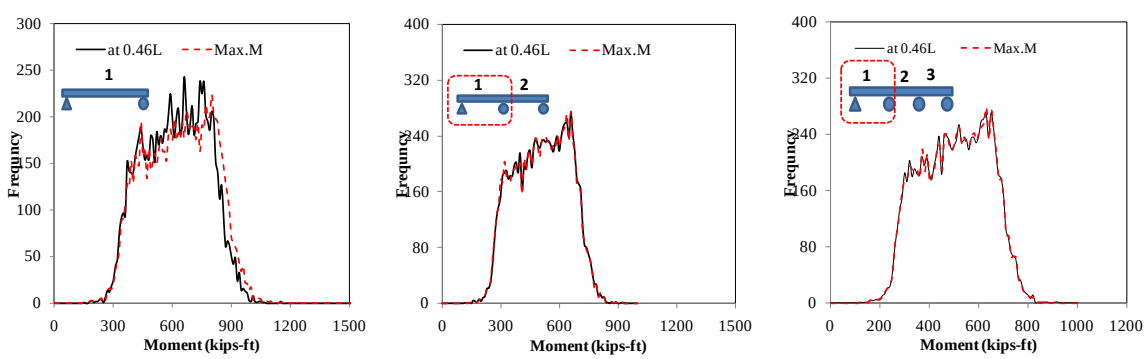


Figure 4.15 Comparison of maximum moment distributions

4.4 Maximum Load Effects from Moving Loads

Girders supporting a bridge are subjected to various configurations of live loads. For instance, on a single day, a single heavy truck may cause the maximum moment or shear force on the girder. If trucks run close to each other, two or more trucks may cause maximum moment or shear force. If a bridge supports more than one lane of load, trucks on a second lane can influence the loads on girders supporting the first lane. Thus, to realistically simulate the moment or shear force on girders due to various configurations of live loads, it is necessary to simulate possible layout of trucks on a bridge. In the previous calibration studies (NCHRP 368 and NCHRP 20-7), several assumptions were used based on limited observation of truck traffic.

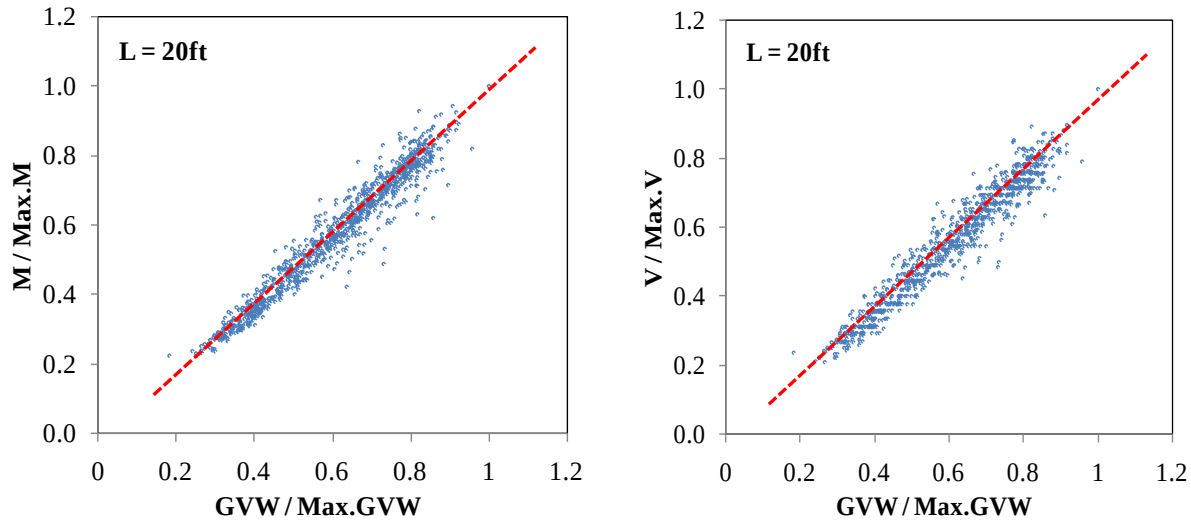
The data from WIM stations introduced in Section 4.2 are comprehensive and can realistically represent traffic volume and weight of a highway system. For this reason, rather than adopting the assumptions based on limited observations as in the previous studies, for this study the distribution of trucks on a highway is simulated considering the Average Daily Traffic (ADT), Average Daily Truck Traffic (ADTT), and minimum headway distance between vehicles. Shifted-exponential distribution, which is often used in traffic flow simulation, is used to simulate time (or distance) between trucks running sequentially. As maximum moment on a girder may be caused by a single heavy truck or multiple trucks on a bridge; the two cases (single truck case and multiple truck case) are analyzed independently. The maximum load effect from the two cases is considered as the maximum load on the girder. The following describes how the single truck cases and multiple truck cases are simulated.

4.4.1 Selection of Heavy Single Truck Cases

As trucks with light GVW are not likely to develop the daily maximum load effects, bridges do not need to be analyzed using the light-weight trucks. In addition, considering the amount of computational demand for simulations, the number of analyses can be reduced by analyzing bridges with only heavy trucks.

In general the maximum load effects are most sensitive to the gross vehicle weight (GVW) of heavy trucks. As trucks have infinite number of axle configurations, such as axle distances and weight on each axle, and as the load effects also depend on the configuration of a bridge, such as number of spans and span lengths, it is not straightforward to define the ‘heavy’ trucks which will most likely develop the maximum load effects. Hence in this section, a parametric study is carried out to find the threshold weight of a heavy truck for single truck case simulations.

Figure 4.16 presents correlations between maximum load effects and GVWs. It can be observed from the figure that the maximum load effects are closely related to the GVW. It is not perfectly linear due to the influence of other parameters, such as axle spacing, distribution of GVW to each axle, and bridge span. To evaluate how the GVW is related to the maximum load effects, 10,000 trucks in Vehicle Class 9 (Appendix B) are randomly selected from the WIM database processed in Section 4.2.2, and the moving load analyses are carried out with bridges having 20ft to 150ft spans. In this research, it was assumed that 10,000 vehicles are enough to check a relationship between GVW and load effect.



(a) Moment-GVW relationship (b) Shear force-GVW relationship

Figure 4.16 Correlation of load effects and GVW

Table 4.6 Correlation coefficient between load effects and GVW

	Span length					
	L=20ft	L=40ft	L=60ft	L=80ft	L=100 ft	L=150 ft
Moment (M)	0.9746	0.9717	0.9723	0.9797	0.9893	0.9965
Shear (V)	0.9715	0.9631	0.9686	0.9812	0.9753	0.9789

Table 4.6 presents correlation coefficients between GVW and load effects. As it can be observed from the table, as span length increases, the correlation coefficients for moment increases. With shorter spans, the other parameters, such as axle spacing and axle weight, more sensitively affect the maximum moment. For shear force, the correlation coefficients are generally lower than the moment and are not noticeably affected by the span length of bridges. For all span lengths, however, the correlation coefficients for moment and shear force are greater than .96, which can be interpreted that parameters other than GVW can be practically ignored when selecting the heavy trucks for single truck analyses.

Based on the above observation, several thresholds weights are tested as criteria to select heavy trucks. For instance, in one set of simulations, trucks within the top 5% of the GVW are used for simulation. In the other set, all 5,000 trucks are used to find daily maximum value. Simulations for 100 days are carried out and maximum daily values (100 data points) are retrieved from the simulation. Using the extreme value theory introduced in Section 4.5, the mean maximum 75-year load effects are calculated. Table 4.7 compares the mean maximum values when the top 5, 10, 20, and 50% of heavy trucks are used. The maximum load effects are compared with the case when all trucks are used (100%) for the simulation. As it can be observed from the table, the moment and shear force when only top 5% of trucks are used for simulation are very close to those when all trucks are used. The difference is within 1.5%, which is practically negligible. Hence, trucks within top 5% of GVW are used to simulate single truck events.

Table 4.7 Mean maximum load effect when subset of trucks were used (single trucks only)

	Selection percentage (% heaviest trucks)				
	5%	**10%**	**20%**	**50%**	**100%**
Moment (kips-ft)	422.68	425.44	431.65	428.39	427.08
Shear (kips)	87.87	89.15	89.49	89.93	88.61

4.4.2 Simulation of Multiple Truck Events

Headway Distance Distribution

A bridge may be subjected to loads from multiple trucks. The chance that multiple trucks exist on a bridge highly depends on the span length, number of spans, number of lanes, average daily traffic, and average daily truck traffic. In the previous calibration studies (NCHRP 368 and NCHRP 20-7), the following assumptions were used based on limited observation of truck traffic; every 50th truck is followed by another truck with the headway distance < 100 ft, every 150th truck is followed by a truck with partially correlated GVW, every 500th truck is followed by a fully correlated truck. Similar assumptions were also used for trucks running on bridges with two or more lanes. As comprehensive and state-specific data on truck weight and bridge configuration are available for this study, more realistic analyses are carried out to simulate existence of multiple trucks on a bridge.

The multiple presences of trucks can be characterized by the headway distance between two trucks and the bridge length. If the headway distance is shorter than the length of a bridge, then some or all axles of two trucks could be on the bridge at the same time. The headway distance is largely a function of 1) minimum headway distance that drivers tend to keep for safety and 2) traffic volume. The minimum headway distance is related to a response time that a driver considers safe. For instance, if trucks run at a slow speed, the headway distance between the trucks is relatively short as drivers think that they can stop the truck in an emergency before his/her truck hits the vehicle running in front of the truck. Based on a field survey, He and Rong (2002) suggested a minimum headway distance corresponding to 0.55 sec of travel time for vehicle velocities of 50 mph or above. Other studies, Chang and Kim (2000) used minimum allowable headway of 0.95 sec to simulate traffic flow in the Korea highway and Pratim Dey and Chandra (2009) used the minimum headway of 0.257 sec to 0.893 sec to simulate traffic flow on two-lane highways. In this study, the minimum headway distance of 0.5 sec is used following the suggestion in He and Rong (2002). For trucks running on highways with average speed of 70 mph, the minimum headway distance corresponding to 0.5 sec is 51 ft.

Traffic flow is often modeled as a Poisson process (Schuhl 1955). When an event follows a Poisson process, the time between events, or the time between vehicle's arrival to a bridge in this case, is described by exponential distributions (Cowan 1975). Assuming that vehicles keep minimum headway distance, the exponential distribution can be shifted considering the minimum headway distance, which is known as the *shifted exponential distribution*. Shifted lognormal or shifted gamma distribution (Law and Kelton 2000) may be used to avoid

inordinately high probability near allowed minimum headway distance when shifted exponential distribution is applied. However, shifted lognormal or shifted gamma distribution requires scale parameters (or standard deviations) and location parameters (or mean values) of minimum headway distances. As those values are not available, and as having high probability around minimum headway distance leads to more multiple truck events, which is conservative for reliability assessment, the shifted exponential distribution is adopted in this study.

Shifted exponential distribution can be expressed with Eq (4.4) and Eq. (4.5) for probability density function and cumulative distribution function, respectively.

$$\begin{aligned} f(x) &= \lambda e^{-\lambda(x-\tau)} \quad && x \geq \tau \\ f(x) &= 0 && x < \tau \end{aligned} \tag{4.4}$$

$$\begin{aligned} F(x) &= 1 - e^{-\lambda(x-\tau)} \quad && x \geq \tau \\ F(x) &= 0 && x < \tau \end{aligned} \tag{4.5}$$

where x, λ and τ are the random variable representing the difference between two vehicle's arrival time to a bridge, traffic flow rate in terms of number of truck traffic per unit time, and time to travel a minimum headway distance. For instance, if it is assumed that a bridge has a daily traffic volume of 5,000 trucks, 0.05787 (=5,000/24*60*60) trucks pass the bridge per second which is the λ in the above equations. The time to travel minimum headway distance, τ, is assumed as 0.5 sec. Based on the above equations, the differences in trucks arrival time, x, can be numerically simulated. The expected value of exponential distribution is $1/\lambda$. Hence, if we generate travel time for headway distances of 5,000 trucks, x_1 through x_{5000}, we can get total one-day of travel time of trucks (1/0.05787 * 5000 = 86400 sec = 1 day). Once the travel time between trucks is generated, the distance between trucks can be found by multiplying the average speed, v_{avg}, of trucks to the travel time, x_i. If the distance between two trucks is less than a bridge's length, it can be assumed that the trucks can impose loads on the bridge at the same time, and moving load analysis needs to be carried out considering the multiple trucks.

The shifted exponential distribution is based on an assumption that the traffic flow follows a Poisson process and the time between arrivals of two trucks are used as a random variable. To take account the lengths of vehicles, which is also a random variable, it is more straightforward to simulate the traffic flow in terms of position of a vehicle, length of a vehicle, and distance between vehicles, rather than using the arrival time as a random variable. For instance, if an average speed of trucks is v_{avg}, the total headway distance, $\Sigma v_{avg} x_i$, of all vehicles in a day can be calculated as

$$Y = v_{avg} * 24 \text{ hours} - \text{ADT} * \tau \tag{4.6}$$

where Y is the total headway distances less minimum headway distances between all vehicles in a day. Then, the ADT number of vehicles can be randomly positioned on the distance of Y. The

above procedure using Eq.(4.6) results in identical headway distribution with the one using the shifted exponential distribution and is used in this research.
When simulating the traffic flow of real vehicles, however, the length of the vehicle also needs to be considered. For instance, trucks in Vehicle Class 9 or higher have lengths in the range of 30 to 40 ft. Because the minimum headway distance of 51 ft is the distance from the front of one truck to the rear of another, the simulation needs to also consider the truck length. Thus, the total headway distances in Eq. (4.6) can be modified as

$$Y = v_{avg} * 24 \text{ hours} - \text{ADT} * \tau - \Sigma L_i \quad (4.7)$$

where L_i is the length of a vehicle.

The above procedure is followed to randomly simulate single lane traffic flow. When applying the above procedure, all vehicles including those in Vehicle Class 1 through 7 are also included in the simulation as those also affect traffic flow and distance between trucks.

ADTT/ADT Ratio and Traffic Distribution over Two Lanes

Based on the 2007 NBI database, the bridges in Missouri have ADTT/ADT ratios of 21%, 18%, and 11% for bridges carrying IS highway, US highway, and other routes. To confirm the ADTT/ADT ratio and to confirm the traffic ratio between the first and second lane on two-lane roads, field monitoring was carried out on IS-44 by the research team. To reduce bias in the field monitoring, three different locations on IS-44 were monitored. At each monitoring point, traffic flow was recorded for one hour. The monitoring locations are located between exits 189 and 195, between exits 186 and 189, and near exit 186 on IS-44. Figure 4.17 provides images of traffic flow recorded at two different locations.

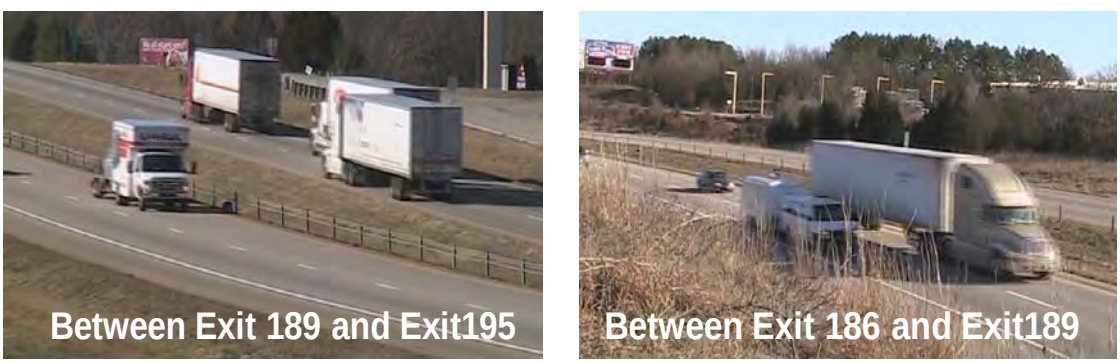


Figure 4.17 Traffic monitoring

It is observed from the field traffic monitoring that approximately 25% of vehicles are heavy trucks in Vehicle Classes 8 to 13. This ADTT/ADT ratio is not far from the ratio for IS highway (21%) based on NBI Database. Based on actual observations, the ADTT/ADT ratio of 25% is used in this study. Also, the traffic monitoring shows approximately 70% of vehicles use the travel lane and the others use the bypass lane. Even though it is limited observation at IS-44, due to lack of further data, this information is used to simulate multiple truck events. For bridges

with three or more lanes, it was assumed that vehicles only use the right two lanes following the distribution of vehicles on the travel and bypass lane.

Simulation of Traffic Flow in Multiple Lanes

Traffic distribution in multiple lanes is controlled by many factors. For instance, on a highway with steep uphill grade, trucks tend to stay on the travel lane (slow lane) and light weight vehicles use the fast lane. Average travel speed could be very low due to traffic jam, which may be caused by accidents, construction, or large traffic volume. The simulation of traffic distribution and traffic flow rates considering the chance of accident, construction, or future traffic volume are extremely difficult and not many studies are available on this topic. Due to lack of the previous studies, the locations of trucks on multiple lanes are randomly simulated based on the *Headway Distance Distribution* in Section 4.4.2. The procedure for multiple truck event simulation is summarized in the following.

1) Determine ADTT of a bridge.
2) With ADTT/ADT ratio of 0.25, randomly sample ADT number of vehicles from the WIM database introduced in Section 4.2.
3) Determine average daily traffic of each lane. For two-lane bridges, it is assumed based on field observation that 70% of traffic runs in the travel lane.
4) Assume average travel speed of the road carried by the bridge.
5) Randomly generate the location of vehicles using the method introduced in the previous section.
6) Compare distances between trucks with length of a bridge. If the distance between two or more trucks is less than the length of a bridge, as depicted in Figure 4.18, use those trucks for moving load analysis.

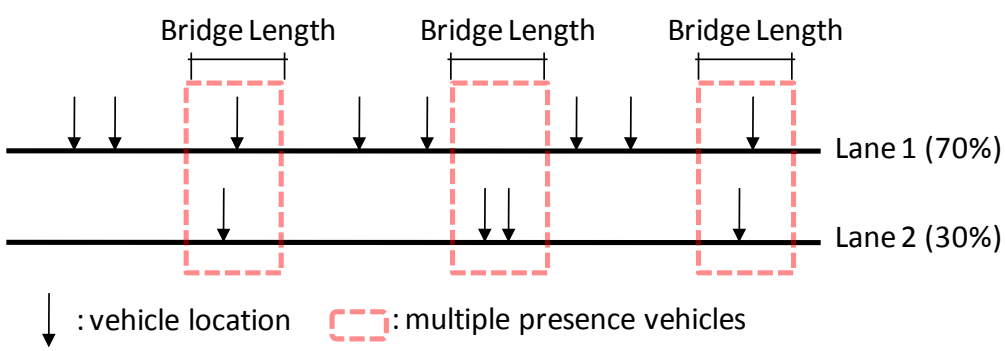


Figure 4.18 Simulation of multiple truck cases

The above procedure can simulate the location of trucks on a bridge including trucks running in series or running in multiple lanes. To understand how many multiple presence effects should be considered, the above analysis is carried out with hypothetical bridges with lengths of 30, 60, and 90 ft and ADTT of 5,000. For these bridges, on average 42, 93, and 188 combinations of trucks are selected for multiple presence effects, which shows that as the bridges' length gets longer, the chance for multiple presence cases increases.

Girder Distribution Factor

In the current AASHTO LRFD Specification, the load effects on a bridge span is distributed to girders using girder distribution factors. For instance, if there is a truck on a one-lane bridge as shown in Figure 4.19(a), the interior and exterior girders are subjected to different moments depending on the stiffness of bridge deck and supporting girders.

AASHTO Standard Specifications use a 'S/D' formula to calculate the girder distribution factor, which was developed for the interior girder of simply supported bridges. In this formula, S is the spacing of girders and D is a constant based on bridge type. The girder distribution factors calculated using this formula tends to be conservative for widely spaced interior girders and un-conservative for an exterior girder (Imbsen et al. 1987). Also, the formula presented in the AASHTO standard specifications is applicable only to straight girders. The NCHRP Project 12-26 proposed a new simplified formula that is more accurate than the S/D formula in AASHTO Standard Specifications. The study also considers the effects of parameters such as bridge length and slab thickness on the girder distribution factor. To identify the parameters significant to live load distribution, a sensitivity study was performed using a finite element model loaded with a HS-20 truck. The formulas proposed by NCHRP Project 12-26 were adopted for the AASHTO LRFD bridge specifications for distribution of live load on highway bridges.

When a bridge supports a one-lane road, the load effect to an internal girder is

$$LL_{SL} = LL_1 \times mg_1 \tag{4.8}$$

where LL_{SL}, LL_1, and mg_1 are the load effect on a girder, live load effect due to a truck on a single lane, and girder distribution factor, respectively. If two or more lanes are supported by a bridge as shown in Figure 4.19(b), a different distribution factor needs to be applied to take into account the multiple lane effects.

$$LL_{ML} = LL_1 \times mg_2 \tag{4.9}$$

where LL_1 and mg_2 are live load effects due to one-lane loads and girder distribution factor. The girder distribution factor, mg_2, takes into account the increase in load effect due to the multi-lane loads. The mg_1 and mg_2 in the above equations are defined in Section 4.6 in the AASHTO LRFD Specification and depend on the cross-section of the bridge span. For most typical bridges in Missouri, the girder distribution factors are

$$mg_1 = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \tag{4.10}$$

$$mg_2 = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \tag{4.11}$$

where S is girder spacing in ft, L is span length in ft, and t_s is deck thickness in inches. The term K_g is the longitudinal stiffness parameter, calculated as

$$K_g = n\left(I + Ae_g^2\right) \tag{4.12}$$

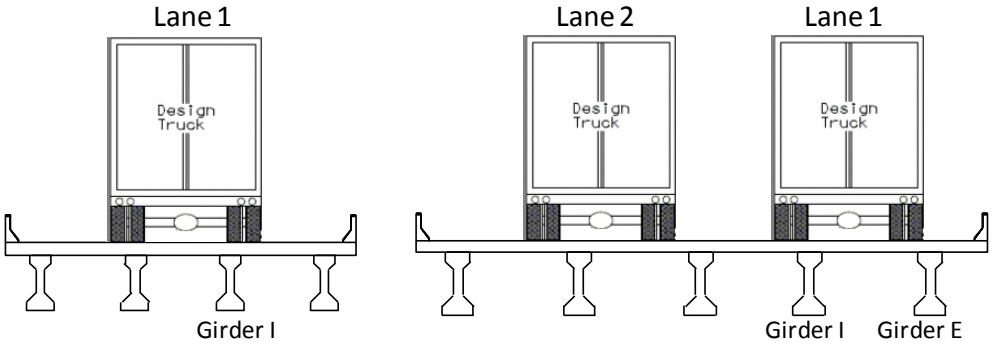
where n is the ratio of the elastic modulus of the girder material to the elastic modulus of deck material, I and A are moment of inertia in in^4 and section area of girder in in^2, respectively, and e_g is the distance between the center of gravity of the non-composite girder and the deck in inches.

As the girder distribution factor in Eq. (4.9) is developed when the design loads coexist in multiple lanes, the net effect of the design load in one lane (Lane 2) to the Girder I on Lane 1 can be approximated as mg_2 - mg_1. If the load effects for each lane can be calculated independently, then Eq. (4.9) can be rewritten as

$$LL_{ML} = LL_2 \times (mg_2 - mg_1) + LL_1 \times mg_1 \tag{4.13}$$

where LL_2 is live load effect due to trucks in Lane 2. In the design process, as identical design loads are applied on all lanes, LL_1 and LL_2 are the same. Hence, Eq. (4.13) is equivalent to Eq. (4.9).

In this study, the effects of multiple lane loads on a girder are approximated using the Eq. (4.13). For instance, for the load effects due to trucks in Lane 1, girder distribution factor of mg_1 is applied. The effects of trucks in other lanes are superimposed after multiplying girder distribution factor of mg_2-mg_1. To consider uncertainties in live load distribution on girders, a coefficient of variation of 0.12 and bias factor of 1.0 are used based on Nowak (1999).



(a) Single-lane load (b) Multiple-lane loads

Figure 4.19 Distribution of load effects to girders

Dynamic Impact Factor

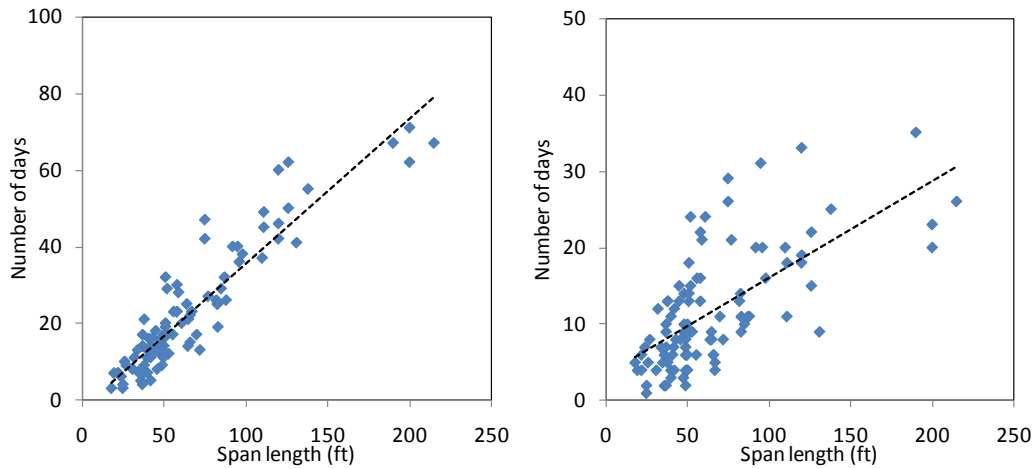
Due to the roughness of bridge surfaces and the dynamic interaction between bridge superstructures and vehicles, a dynamic amplification factor is applied to the load effects from vehicular live loads. For instance, the load effects on girders in Eq. (4.9) or Eq. (4.13) are

multiplied with (1+IM), in which IM represents the ratio of the live load added due to the dynamic interaction to the original live load. The dynamic impact factor is a random variable, with relatively high COV. For instance, several researchers have shown that the dynamic impact factor for actual bridges varies significantly depending on bridge type and truck configuration. Page (1973) measured dynamic loads on 30 highway bridges to study the variation of wheel loads on the bridges. The dynamic impact factors ranged from 0.09 to 0.75, and seventeen of these exceeded the maximum value indicated in the AASHTO standard specifications. Shepherd and Aves (1973) measured dynamic impacts on highway bridges subjected to a standard two-axle truck. They measured impacts of 0.10 to 0.70 for 14 bridges, 9 of which yielded dynamic impact values higher than 0.30. Hwang and Nowak (1991) proposed the mean values and coefficient of variation of the dynamic impact factor based on numerical modeling of bridges and vehicles to reflect the high uncertainty of dynamic impact in the reliability analysis of bridges. The mean dynamic impact factors of 0.10 and 0.15 were taken for two parallel trucks and single truck, respectively. The coefficient of variation for the dynamic impact factor was taken as 0.8. In this study, to be consistent with the procedure adopted for the development of the current AASHTO LRFD Specification, the mean values and the coefficient of variation used by Hwang and Nowak (1991) are adopted to reflect the uncertainties in the dynamic impact.

Distribution of Daily Maximum Load Effects

Following the above procedure, the effect of a single heavy truck and multiple trucks are simulated. With an assumed ADTT value, maximum moments of interior girders are calculated for each load case. For instance, when an ADTT of 5,000 is assumed, a total of 250 of the heaviest trucks (5% of ADTT) are used to simulate heavy single truck cases. In addition, multiple truck cases are simulated based on the procedure in the previous sections. As mentioned in the previous section, on average 42, 92, and 188 sets of trucks are selected for multiple presence effects when bridge lengths are 30, 60, and 90 ft, ADTT is 5,000, and bridges support a two-lane road. The uncertainties in the dynamic impact factor and girder distribution factor are also considered by randomly generating those factors when maximum load effects are calculated.

The maximum load effects here imply the maximum moment (or shear force) along the length of a bridge when the bridge is subjected to the ADTT number of trucks per day. Daily maximum load effects may be caused by single-truck events or multiple-truck events. Figure 4.20 shows the relationship between span length of selected bridges and percentage of daily maximum values governed by multiple truck events. As it can be observed from the Figure 4.20(a), for long span bridges, a large percentage of maximum moments are caused by multiple truck events. For shear forces, however, the correlation between span length and governing multiple truck events is weak.



(a) Maximum positive moment (b) Maximum shear force

Figure 4.20 The number of days on which multiple-truck events govern

As each daily maximum value is an extreme value of the day, the distribution of the daily maximum values can be assumed to follow extreme value distribution. In this study, it is assumed that the daily maximum value follows Gumbel Type I distribution. For example, Figure 4.21 presents the Cumulative Distribution Function (CDF) of raw maximum values of positive moment of one of the selected bridges (NBI bridge No.11877). As it can be noticed from the figure, Gumbel Type I distribution fits the maximum values very well.

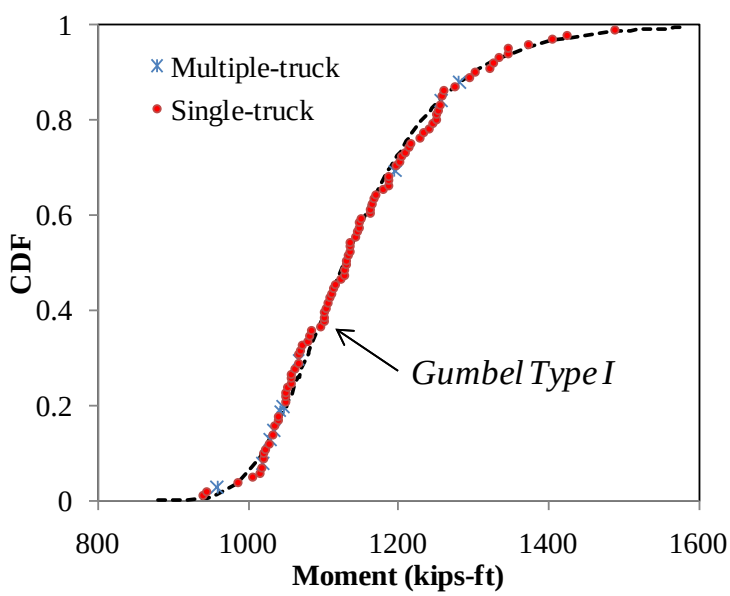


Figure 4.21 Daily maximum moments fitted to Gumbel Type I distribution

4.4.3 Projection of Daily Maximum Load Effects to 75 Year Load Effects

Bridges are designed to resist maximum load effects for a 75-year bridge design life. As the monitoring of truck weights have been carried out for a relatively short time period in comparison with the design life of bridges, it is necessary to rely on statistical procedures to project the short-term observation for the long-term prediction.

There have been several studies on the projection of short-term observation to long-term period. For instance, Nowak (1999) used the normal probability plot to project 75-year load effects. The Cumulative Distribution Functions (CDFs) for moment or shear force were plotted on the normal probability paper, and the long-term return period value was estimated by extrapolating the tail portion of the CDF. The extrapolation of the tail-portion of the plot is based on assumption that the maximum moments or shear forces from many simulations of 75-year period follows a normal distribution. As the simulation data do not follow a normal distribution, the extrapolation should be somewhat subjective as shown in Figure 4.22.

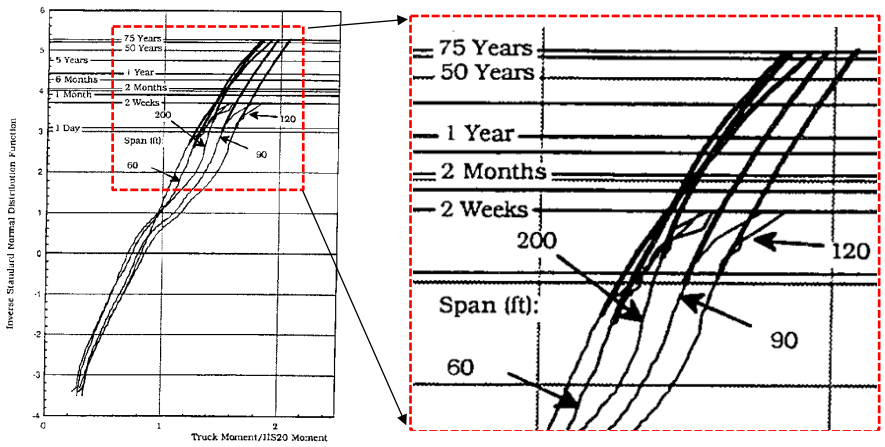


Figure 4.22 Projection to 75-year load effect using Normal Probability Paper (Nowak 1999)

Van de Lindt et al. (2005) tried to estimate 75-year load effects by taking the power of CDF of load effects for short-term WIM data. Kozikowski (2009) projected the 75-year load effect using non-parametric distribution based on the Kernel density functions. The Kernel density estimation is a method to estimate the probability density function of a random variable. After converting the distribution of load effects for short-term WIM data to a non-parametric distribution function using the Kernel density function, the non-parametric distribution function was plotted on a normal probability paper. O'Brien et al. (1995) used an 8-minute measurement period for each 4 hour period to determine the characteristic deflection of a bridge. Thus, each day's deflection measurements were represented by a 48-minute sample. O'Brien et al. then considered the daily maximum deflection as an extreme value population. They used the Gumbel distribution and extrapolated the maximum deflection for the 1000-year return period. Cooper (1995) used a traffic model of about 81,000 measured truck events representing one year of traffic to determine the distribution of load effects due to a single-truck event. The study also used a Gumbel distribution to extrapolate load effects to 2400-year return period.

In this research two methods are evaluated to project short-term observation for long-term prediction. The first method is based on a Normal Probability plot used in Nowak (1999). The method is evaluated as it was used for the initial development of the LRFD Specification. The second method evaluated in this study is the Gumbel Type I distribution and extreme value theory.

4.4.4 Projection Method using Normal Probability Plot

The initial study for LRFD Bridge Specification development (Nowak 1999) adopted a projection method using a normal probability plot. If a random variable X follows a normal distribution, CDF for the normal distribution can be expressed as below.

$$F\left(X\right) = \Phi\left(\frac{X-\mu}{\sigma}\right) \tag{4.14}$$

where μ and σ are mean and standard deviation of random variable X, respectively. The random variable X can be expressed in terms of standard normal variable U as below.

$$X = \mu + \sigma \times U \tag{4.15}$$

Hence, the realization of X, X_i, should have a linear relationship with the corresponding realization of standard normal variable U, U_i. The corresponding standard variable can be found from the following equation.

$$U_i = \Phi^{-1}\left(F(X_i)\right) \tag{4.16}$$

where Φ^{-1} and F are inverse normal cumulative distribution function and empirical distribution function of X, respectively. If X is perfectly normally distributed, the empirical distribution function $F(X_i)$ is identical to $\Phi((X_i - \mu)/\sigma)$.

In the normal probability plot, the horizontal and vertical axes are a realization (experiments or observation) of X, X_i, and corresponding standard normal variable, U_i. If the random variable X represents daily maximum load effects, the experiments of maximum values for n number of days will result in n points in the normal probability plot. If the number of experiments, n, is infinitely large and the observations follow a normal distribution, then the expected maximum value in a certain time period, for example 75 years, can be found from the normal probability plot using following relationship.

$$X_{75} = \mu + \sigma N_{75} = \mu + \sigma \Phi^{-1}\left(1 - \frac{1}{75 \times 365}\right) \tag{4.17}$$

where μ and σ are parameters identified from linear curve fitting of (X_i, U_i) data points and these correspond to mean and standard deviation. Due to lack of data or lack of computational power, however, the number of experiments is generally not large enough to find maximum

value, X_{75}, in 75 years of bridge life span. For this case, extrapolation of the normal probability plot is inevitable.

The normal probability plot of daily maximum values is not generally linear as it can be observed from Figure 4.22. Linear extrapolation of the tail portion of the plot can result in inconsistent projection depending on how many data points are used at the tail portion of the plot.

To evaluate this method, daily maximum positive moments of a simply supported hypothetical bridge with 60ft span length is simulated. A total of 100-days of daily maximum values are simulated. Based on the experimental data points, a 75-year maximum value for the five sets of simulations, in which one hundred days' of truck traffic is simulated, is obtained using the normal probability plot method. As the maximum values do not perfectly follow the normal distribution, only the tail portion of the plot is extrapolated as depicted in Figure 4.23. This process is repeated five times to evaluate the projection consistency. From the five sets of simulations, it is found that the maximum 75-year moments are 1618, 1629, 1630, 1674, and 1792 kip.ft. The difference between the largest and the smallest projected value is found to be 10%.

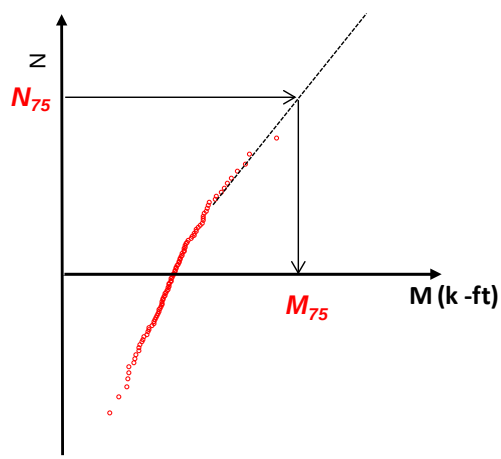


Figure 4.23 Daily maximum moment of 60 ft span simply supported bridge

4.4.5 Projection Method Using Extreme Value Theory

Extreme value theory is used for the assessment of highly unusual events, such as the 75-year maximum load effect. If random variable X represents daily maximum load effects, and if n days of experiment are carried out, the maximum values in n days can be found from

$$Y_n = \max\left(X_1, X_2, ..., X_n\right) \tag{4.18}$$

where $X_1, X_2, ..., X_n$ are independent random variables having the identical distribution function of $F_X(x)$ as follows:

$$F_{X_1}(x) = F_{X_2}(x) = \cdots = F_{X_n}(x) = F_X(x) \tag{4.19}$$

Then, the cumulative distribution of Y_n is

$$F_{Y_n}(y) = P(Y_n \le y) = P(X_1 \le y, X_2 \le y, ..., X_n \le y) = [F_X(y)]^n \tag{4.20}$$

If n becomes large, $F_{Y_n}(y)$, follows one of several asymptotic distributions such as a Gumbel or Weibull distribution (Gumbel 1958).

Assuming that the load effect of a single truck follows a certain probabilistic distribution, the daily maximum load effect is the maximum values caused by several thousands of trucks. Thus the distribution of daily maximum load effect is assumed to follow Gumbel Type I distribution as in Eq. (4.21) and (4.22).

$$F_{X-1day}(x) = \exp\left(-\exp\left(-\frac{x-u}{\alpha}\right)\right) \tag{4.21}$$

$$f_{X-1day}(x) = \frac{1}{\alpha}\exp\left(-\frac{x-u}{\alpha}\right)F_{X-1day}(x) \tag{4.22}$$

in which the scale parameters (α) and location parameters (u) can be estimated by maximum likelihood estimation (MLE) using the likelihood function. The distribution of maximum load effect for 75 years, $F_{X-75years}$ can be projected using distribution of daily maximum load effects:

$$F_{X-75years}(x) = \left[F_{X-1day}(x)\right]^N = \left[\exp\left(-\exp\left(-\frac{x-u}{\alpha}\right)\right)\right]^N = \exp\left\{-\exp\left(-\frac{x-u_n}{\alpha_n}\right)\right\} \tag{4.23}$$

where N is the number of days in 75 years. Based on the stability postulate property of the Gumbel distribution, the maximum load effect for 75 years also follows the Gumbel distribution. The scale parameter of $F_{X-75years}(x)$, α_n is the same as $F_{X-1day}(x)$, and the location parameter of $F_{X-75years}(x)$, u_n is $u + \alpha \ln N$ (Ang and Tang 1984).

Figure 4.24 shows the distribution of daily maximum moment for the interior girder of NBI Bridge No.11877. The dashed line in the figure represents the cumulative distribution of the daily maximum moment modeled using the Gumbel distribution; the circle dots in this figure are the 100 daily maximum moments. The scale parameters and location parameters of the fitted Gumbel distribution estimated using MLE are 65 kips-ft and 1028 kips-ft, respectively.

Figure 4.25 presents the distribution of 75-year maximum moment for the same bridge in Figure 4.24. The dashed line in this figure indicates the distribution of daily maximum moment and the solid line indicates the distribution of 75-year maximum moment projected using Eq. (4.23).

To understand the consistency of the projected maximum load effects, the five sets of 100-day simulation results introduced in Section 4.4.1 are used to estimate 75-year load effects. When the extreme value theory is used, the 75-year maximum load effects are found to be 2125, 2162, 2167, 2164, and 2190 kip.ft with the difference between the maximum and the minimum values less than 3%. The consistent results of this method mainly results from the fact that in this method, all data points are used to define daily maximum distribution, which is used to estimate the 75-year load effect. On the other hand, in the normal probability plot method introduced in Section 4.5.1, only the tail portion of the probability plot is used for extrapolation, which tends to be unstable. The extreme value theory resulted in around 30% higher projected moment. This large difference mainly results from the different characteristics in the projection methods. In this study, the extreme value theory is used for projection as the method is more systematic than the normal probability plot and provides consistent and conservative projected values.

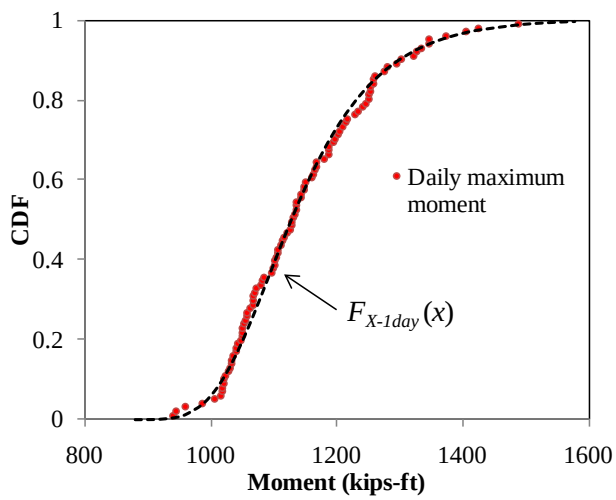


Figure 4.24 Distribution of daily maximum moment (NBI bridge No.11877)

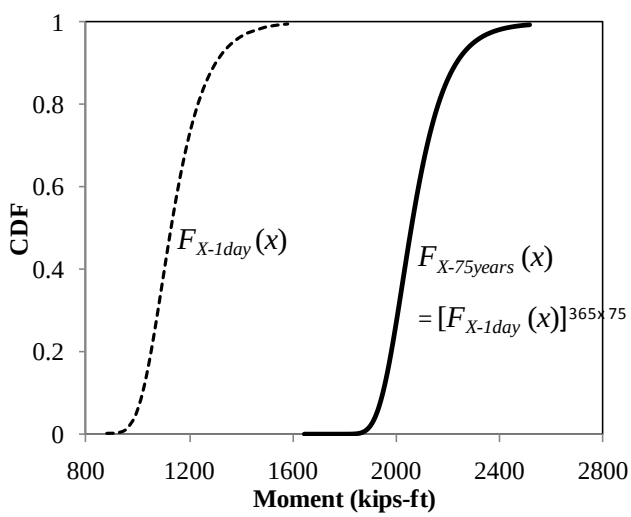


Figure 4.25 Distribution of 75-year maximum moment (NBI bridge No.11877)

4.5 Statistical Parameters for Dead Load

Dead load is the gravity load due to the self weight of the structural and nonstructural elements permanently connected to the bridges. The dead load was assumed to act as a uniformly distributed load. Because of different degrees of variation, it is convenient to consider three components of dead load: weight of factory made elements (steel, precast concrete), weight of cast-in-place concrete members, and weight of the wearing surface (Nowak 1999). The nominal dead load effects for factory made elements and cast-in-place concrete members were calculated based on bridge drawings provided by MoDOT. A 3-inch (35 psf) future wearing surface is considered to calculate the dead load effect by wearing surface according to the MoDOT Engineering Policy Guide article 751.10.1 (LRFD Bridge Design Guidelines). In case of prestressed girder bridges, dead load effects by self weight of girder and deck are calculated assuming simple support. The dead load effects by the barrier and future wearing surface constructed after installing of girder and deck are calculated assuming continuous support. The mean (μ_D) and standard deviations (σ_D) for the dead load effect were estimated using the bias factor (λ_D) and COV presented in Table 4.8

Mean value: $\mu_D = D_n \times \lambda_D$ (4.24)

Standard deviation: $\sigma_D = \mu_D \times COV$ (4.25)

where D_n is nominal dead load.

Table 4.8 Bias factors and coefficient of variations for dead load (Nowak 1999)

Component	**Bias factor**	**COV**
Factory-made members	1.03	0.08
Cast-in-place members	1.05	0.10
Wearing surface	1.00	0.25

4.6 Statistical Distribution of Resistance

The statistical distribution of resistance is based on the uncertainties in the materials (strength, modulus of elasticity, etc), fabrication (geometry), and analysis (accuracy of analysis equations). Therefore the resistance R is considered as the product of the nominal resistance R_n times three factors: materials/properties M, fabrication F, and professional/analysis P (Nowak et al. 1994).

$$R = R_n\, MFP \tag{4.26}$$

For small coefficients of variation (COVs), the COV can also be found as the square root sum of the squares.

$$COV_R = (COV_M^2 + COV_F^2 + COV_P^2)^{1/2} \tag{4.27}$$

The statistical distribution can be described in terms of a bias factor λ and the coefficient of variation COV. The bias factor is the ratio of the mean to the nominal design value. For example, the design value of a concrete mix may be specified to be 4,000 psi, however the concrete that is actually delivered is generally a bit stronger, say 4,500 psi. Therefore the concrete strength would have a bias of 1.125. The coefficient of variation is the ratio of the standard deviation to the mean. It gives an indication of the uncertainty of the parameter.

In order to determine the statistical distribution of the resistance function, plans from the 100 sample bridges (14 reinforced concrete, 58 prestressed, and 28 steel) were analyzed and the strength of the bridges determined using the 2008 version of the AASHTO code. Then, variations in the material and geometric parameters are included by a Monte Carlo analysis to determine the overall statistical distribution of resistance for each bridge. The effect of professional/analysis uncertainty is included by using Eq.(4.26) and Eq.(4.27) after the Monte Carlo analysis.

4.6.1 Comparison of Required Strengths and Strengths in As-Designed Condition

The actual strength of each sample bridge is determined from provided design drawings using the 2008 version of the AASHTO Code considering the nominal material properties reported in the design drawings. The required strength of the bridge is also determined using the 2008 version of the AASHTO Code using the loads and truck weights specified in the code. Although most of these bridges were not designed to the 2008 AASHTO, or even a LRFD code, the comparison between actual and required resistance needs a consistent code usage in order to make a compatible comparison, and therefore the latest 2008 version is chosen. For the analysis, resistance factors are not used in calculating the actual strength of the bridge. Load factors are used in calculating the required resistance.

Figure 4.26 gives the required/actual moment resistance ratios for the reinforced concrete bridges. Detailed information about the required and actual moments can be found in Appendix C. In most cases the required/actual resistance is less than one as expected, however for some bridges the value is slightly above 1, with the worst case being 1.18. This is likely due to the fact that the 2008 version of the AASHTO code was used rather than the actual version the bridge was designed under, leading to some discrepancies in the results. On average the reinforced concrete

bridges have a required/actual resistance value of 0.87, indicating that their strength is relatively close to the required factored load.

Figure 4.27 gives the required/actual moment resistance values for the prestressed I girder bridges. The negative moment capacity was calculated over the center (or near center support). The positive moment capacity was calculated for each span. As seen in the concrete bridges, some required/actual values are slightly greater than 1 (especially in earlier years). Again, this is likely due to the fact that the 2008 version of the AASHTO code was used rather than the actual version the bridge was designed under, leading to some discrepancies in the results. The average required/actual resistance for negative moment is 0.70 and for positive moment is 0.76. Figure 4.28 gives the required/actual moment resistance values for the prestressed double tee bridges. The average required/actual resistance for negative moment is 0.76 and for positive moment is 0.78. All of the ratios are under (or close to) 1 and generally above 0.6. This indicates a fairly consistent prestressed girder and tee beam design.

For steel bridges, the ratio of required to actual resistance of the bridge is presented in Figure 4.29. The average ratio 0.67 and 0.45 is for negative and positive moments, respectively. These ratios are a bit lower than those for the reinforced concrete or prestressed bridges, likely due to other factors such as fatigue or deflection controlling the design of the bridge. Figure 4.30 presents the relationship between span length and the required/actual moment capacity. As can be seen from the figure, the bridges with the lowest ratios have the shortest spans. This is because those spans are often the short spans in a multiple span bridge. For example, bridge A4999 has 3 spans with lengths of 59.5 ft, 119 ft, and 55.5 ft and ratios of 0.2, 0.66, and 0.18 respectively. Therefore, it is likely that the design of the shorter spans was influenced by the longer span (web depth generally remains constant throughout the bridge).

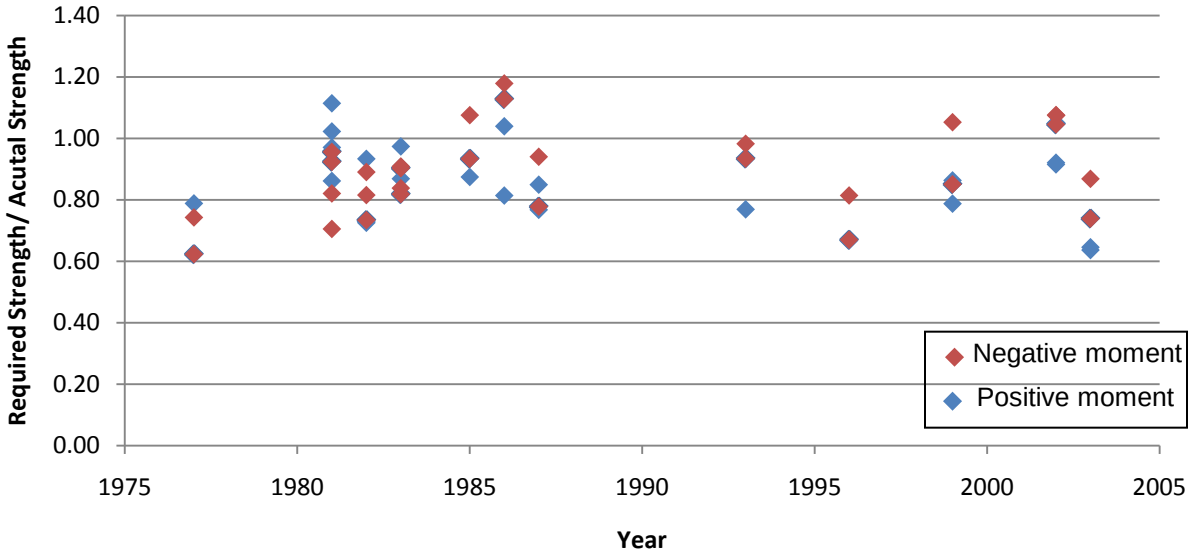


Figure 4.26 Comparison of required/actual strength based on bridge year for RC bridges

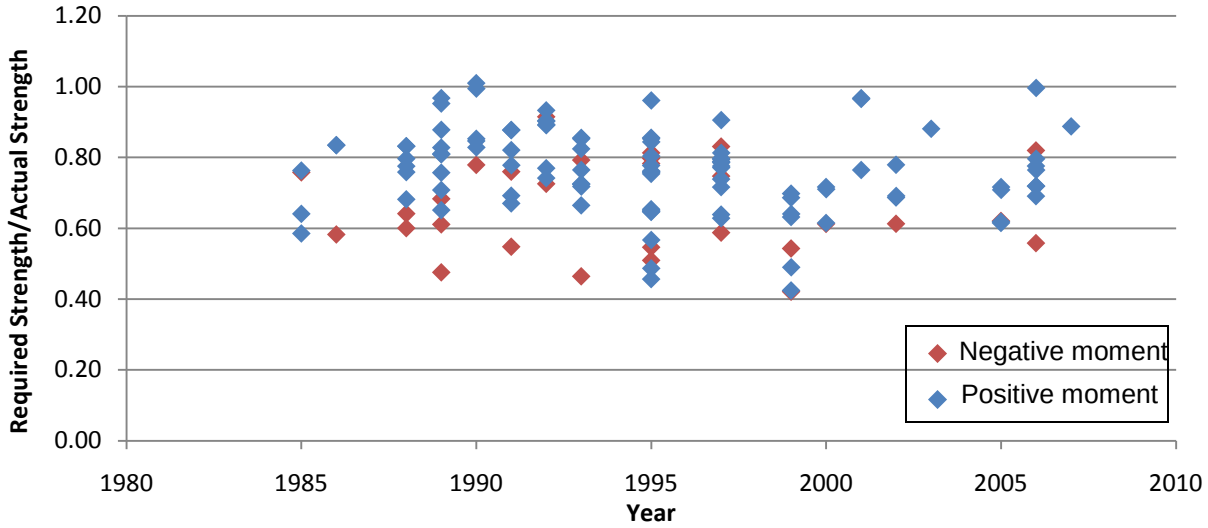


Figure 4.27 Ratio of required/actual strength based on bridge year for PS I girder bridges

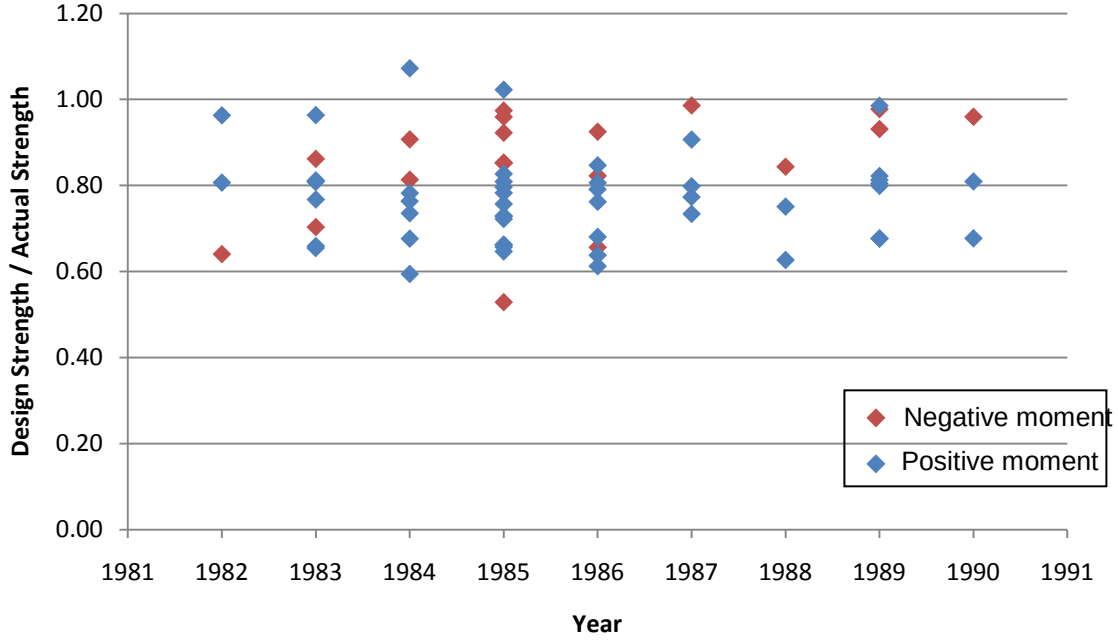


Figure 4.28 Ratio of required/actual strengths based on bridge year for PS double tee bridges

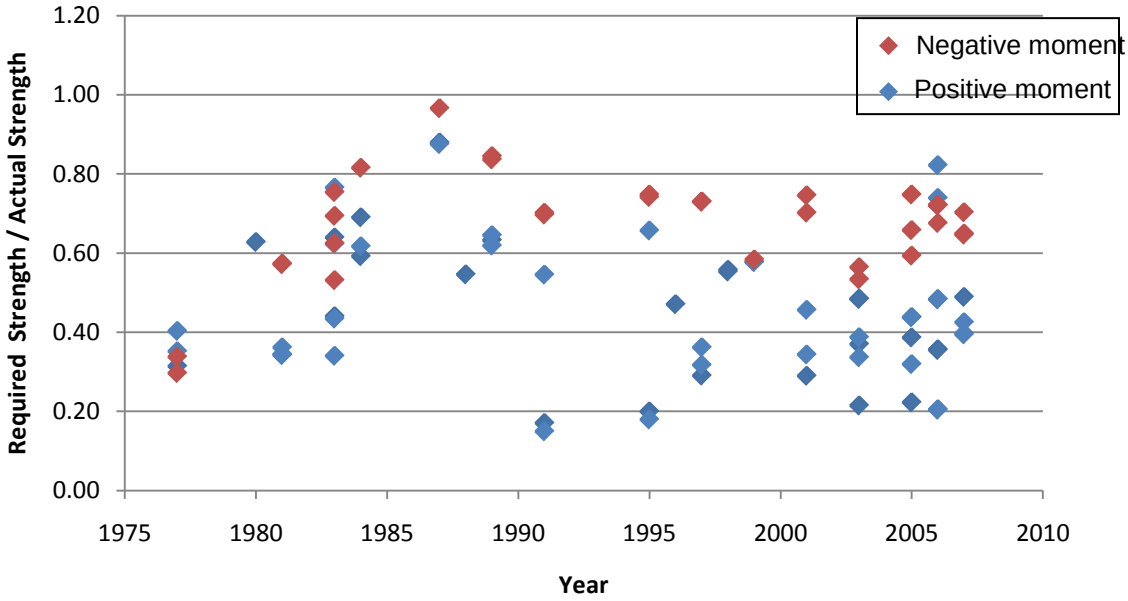


Figure 4.29 Comparison of required/actual strength based on bridge year for steel bridges

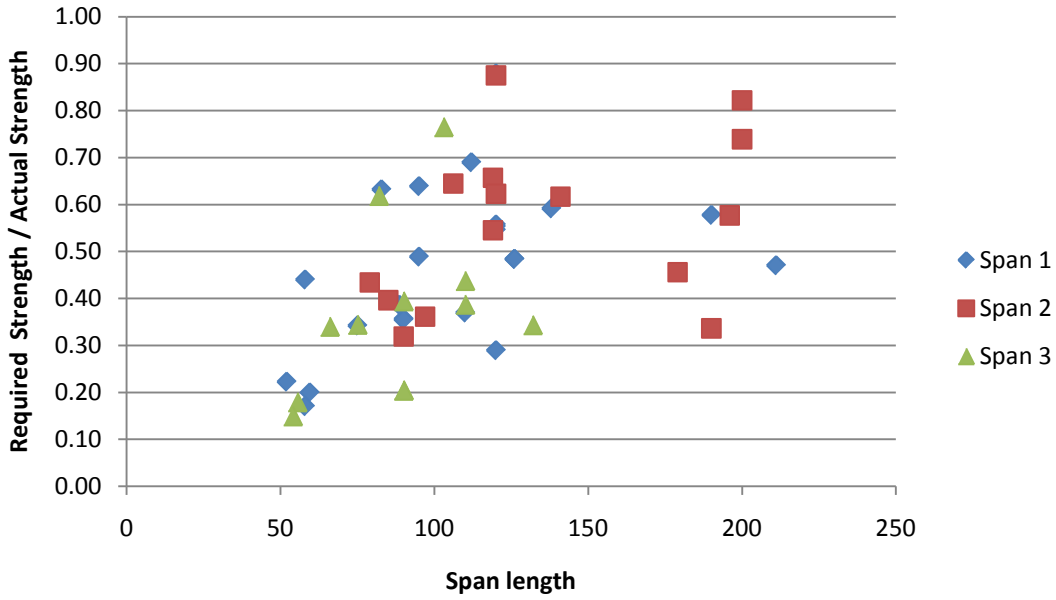


Figure 4.30 Comparison of required/actual strength based on span length for steel bridges

4.6.2 Material and Geometric Parameters

It is important to have the most accurate material and geometric parameters when trying to perform the calibration. Parameters used for the NCHRP 368 calibration for bridges are from data from 1980. Nowak et al. (1994) calculated statistical parameters for sample reinforced and

prestressed concrete girders. They used statistical parameters of material properties and dimensions found by Ellingwood (1980), shown in Table 4.9, and combined them using a Monte Carlo simulation. All distributions were treated as normal variables, with the exception of the yield strength of the steel which was treated as lognormal.

Table 4.9 Previous material and geometric parameters for RC and prestressed bridges

Variable	Notation	Nominal	Bias	COV
Concrete Strength	f'_c	3000 psi	0.92	0.18
	f'_c	5000 psi	0.805	0.15
Yield Stress of Reinforcing Steel	f_y	40000 psi	1.125	0.12
		60000 psi	1.12	0.1
Strength of Prestressing Steel	f_{pu}	270 ksi	1.04	0.025
Modulus of Elasticity	E_s	29000 ksi	1	0.06
Area of Prestressing Steel	A_{sp}		1	0.0125
Area of Reinforcing Bars			1	0.015
Effective Depth			1	.7/d
Section Height			1	.4/h
Slab Thickness			1	.4/t

For this research the material and geometric parameters were updated using the latest published research. A 2003 report by Nowak and Szerszen found statistical parameters for reinforced and prestressed concrete. This study was meant for concrete in buildings but the material properties should be the same for bridges. The material parameters used for concrete and prestressing is shown in Table 4.10. The most significant changes were that the bias for concrete and steel increased, and the COV decreased. This is as expected as improvements have been made in the batching of concrete and steel. Furthermore, the bias and COV for prestressing tendons has remained relatively unchanged.

For concrete, Nowak and Szerszen (2003) give the bias based on concrete compressive cylinder strength as

$$\lambda = -0.0081 f_c^{'3} + 0.1509 f_c^{'2} - 0.9338 f_c^{'} + 3.0649 \tag{4.28}$$

where $f_c^{'}$ is in ksi. Bartlett and MacGregor (1996) give the ratio of concrete compressive strength measured in-place to that measured from cylinder test as 0.95 for elements less than 17 in. thick (generally slabs) and 1.03 for elements greater than 17 in. thick (generally beams). Bartlett and MacGregor further say that the concrete compressive strength increased by about 25% from 28 days to 1 year. For this study, the 28 day compressive strength is used.

For professional factors and factors related to geometry (sizes) no updated information was found. Thus, in this study the same factors as in the NCHRP 368 are used as summarized in Table 4.11 and Table 4.12.

A study done on steel by Schmidt and Bartlett (2002) updated the material parameters for steel. In general the size of the plate (thickness and height) is a deterministic variable, with a bias close to 1 and a low COV. The yield strength of the steel depends on the thickness of the plate. Although it would be assumed that thinner plates should have a higher bias that is not the case. Schmidt and Bartlett found out that each plate thickness range uses a slightly different composition of steel, the differences in the composition leads to higher bias values in the thickest plates. The previous parameters used in Kulicki et al. (2007) for the yield strength of the steel had a bias factor of 1.05 and a COV of 0.10. Therefore, the yield strength of the steel has increased in bias and lowered in COV. The professional factor for steel bridges will remain the same as in Kulicki et al. (2007) with a bias of 1.05 and a COV of 0.06.

Table 4.10 Updated material properties for reinforced concrete and prestressed bridges

Variable			**Bias**	**COV**
Concrete Strength	f'c	3000 psi	1.4029	0.1
		4000 psi	1.2257	0.1
		5000 psi	1.1559	0.1
		6000 psi	1.1449	0.1
		7000 psi	1.1441	0.1
Ratio of in place strength to cylinder strength	slabs	<17 in	0.95	
	beam	> 17 in	1.03	
Yield Stress of Steel	fy	60 ksi	1.145	0.05
Strength of Prestressing Steel	fpu	270 ksi	1.045	0.025

Table 4.11 Geometric parameters

Variable	**Bias**	**COV**
Area of Prestressing Steel	1	0.0125
Area of Reinforcing Bars	1	0.015
Effective Depth	1	.7/d
Section Height	1	.4/h
Slab Thickness	1	.4/t

Table 4.12 Professional parameters

Item	**λ**	**V**
Reinforced concrete moment	1.02	0.06
Prestressed moment	1.01	0.06
Steel moment	1.05	0.06

Table 4.13 Updated material parameters for plate steel

Variables		**Bias**	**COV**
Plate thickness		1.04	0.025
Plate height		0.999	0.002
Yield stress	0 to 1.18 in.	1.11	0.053
	1.181 to 1.57 in.	1.16	0.063
	larger than 1.57 in.	1.2	0.055
Young's Modulus		1	0.06

4.6.3 Statistical Parameters of Resistance

Once the material parameters have been updated, new bias and COV values can be computed for each type of bridge. For this task, the program Crystal Ball (Oracle, 2008) was used in conjunction with Excel to perform a Monte Carlo analysis. Oracle Crystal Ball is a leading spreadsheet-based application suite for predictive modeling, forecasting, simulation, and optimization. Crystal Ball performs a Monte Carlo analysis by randomly assigning numerical values to selected cells following a defined statistical distribution. Using Crystal Ball, the material and geometric properties in the Excel spreadsheets used to calculate the as-designed strengths of the bridges were given statistical distributions according to Table 4.10 through Table 4.13. The program then performed a Monte Carlo analysis while keeping track of the forecast value, the moment capacity of the bridge. The forecast values were then plotted and fitted to a log-normal distribution to obtain the bias and COV. In order to check the accuracy of the procedure the results of the analysis are compared to those presented in Nowak et al. (1994) and show a fairly close agreement on the bias and COV, see Table 4.14.

Table 4.14 Comparison of test case using previous material parameters

Beam	Section	Nominal Strength	Nowak et al. 1994		Crystal Ball	
			Bias	COV	Bias	COV
Reinforced Concrete	A	2688	1.115	0.120	1.100	0.098
	B	3812	1.117	0.119	1.099	0.096
	C	4584	1.118	0.119	1.101	0.096
Prestressed	II	1699	1.036	0.039	1.026	0.046
	III	3304	1.037	0.035	1.022	0.047
	IV	5592	1.037	0.035	1.018	0.047

After the Monte Carlo simulations, the bias factors and coefficients of variation for the materials and fabrication of reinforced concrete bridges are reported in Figure 4.31. A complete list of the bias and COV for each bridge can be found in Table C-32. The average bias and COV for positive moment is 1.15 and 0.066. For negative moment the average bias and COV is 1.15 and 0.069. The materials and fabrication factors can then be combined with the professional factors (Table 4.12) to give the distribution of resistance for the bridge. The combined resistance (Table 4.15) bias and COV for reinforced concrete bridges is 1.17 and 0.09, which is a slightly higher bias, and lower COV than the previous NCHRP report. This is reasonable because the material parameters, particularly the steel strength, have improved (reduced variability). A graphical representation of the change in the bias and COV factors is presented in Figure 4.32. This shows that the distribution resistance has narrowed and moved to the right.

The bias factors and coefficients of variation for the materials and fabrication of prestressed girder and double tee bridges are reported in Figure 4.33 and a complete list can be found in Table C-33 and C-34. For I girder bridges, the average bias and COV for positive moment is 1.044 and 0.031. For negative moment the average bias and COV is 1.157 and 0.054. For the double tee bridges, the average bias and COV for positive moment is 1.045 and 0.038. For negative moment the average bias and COV is 1.145 and 0.058. The materials and fabrication factors can then be combined with the professional factors to give the distribution of resistance for the bridge. The combined resistance bias and COV for reinforced concrete bridges is 1.055

and 0.068, which is nearly the same as the previous NCHRP report. The similarity between the previous and the updated distributions is also evident in Figure 4.34. This is reasonable because the material parameters, particularly the prestressing steel strength, did not change significantly, and therefore the bias and COV remained similar.

The bias factors and coefficients of variation for the materials and fabrication of steel are reported in Figure 4.35. The average bias and COV for positive moment is 1.17 and 0.057. For negative moment the average bias and COV is 1.18 and 0.053. The materials and fabrication factors can then be combined with the professional factors to give the distribution of resistance for the bridge. The combined resistance bias and COV for steel bridges is 1.23 and 0.081, which is a much higher bias than the previous NCHRP report. This difference between the two distributions is illustrated in Figure 4.36. The difference is because the material parameters particularly the for the yield strength of the steel strength, changed from 1.05 to 1.11 or 1.16 (depending on plate thickness). Because the overall resistance parameter is very sensitive to the material property of yield strength, the overall resistance parameter also increased significantly.

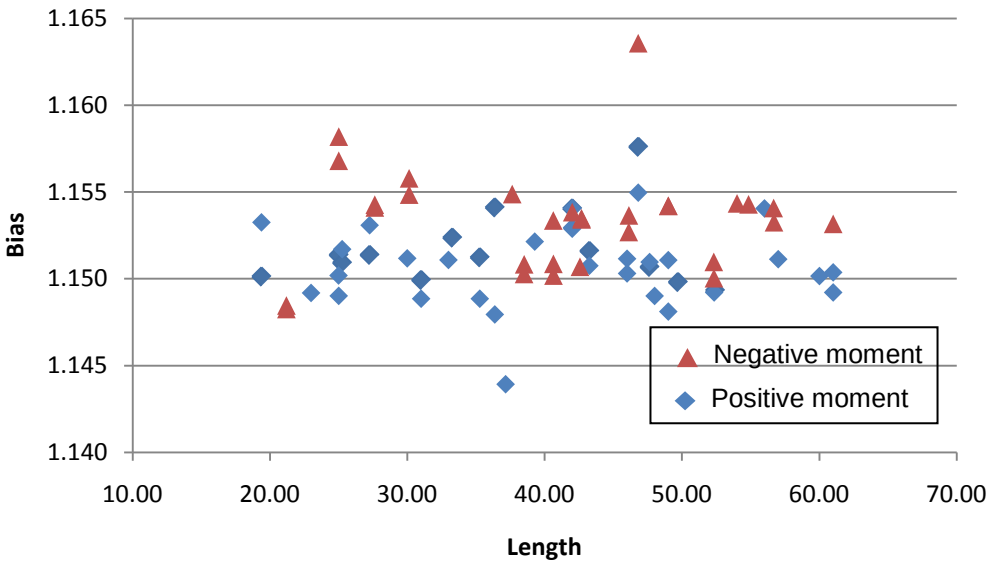


Figure 4.31 Bias and COV for moment in reinforced concrete bridges (contd.)

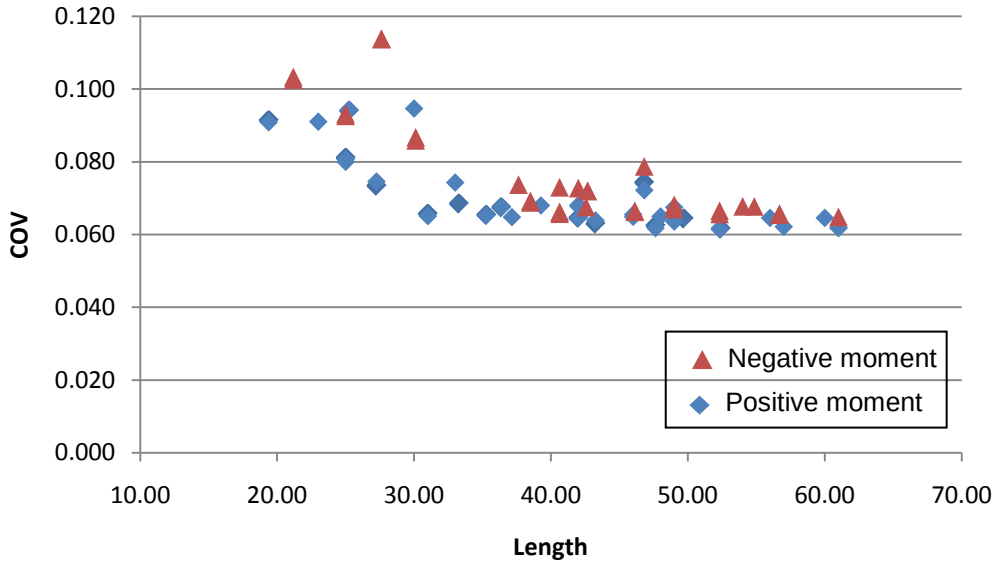


Figure 4.31 Bias and COV for moment in reinforced concrete bridges

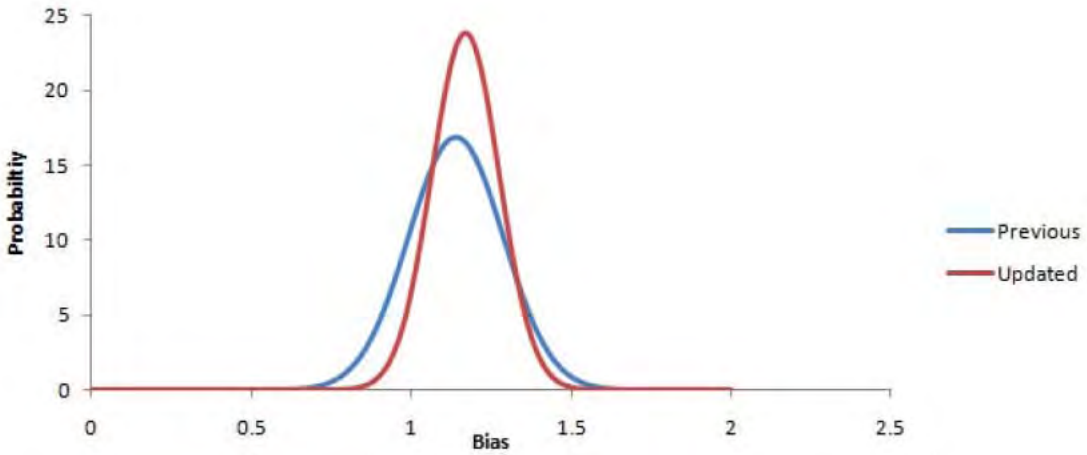


Figure 4.32 Previous and updated distribution of moment resistance for RC bridges

Table 4.15 Statistical parameters for moment resistance of reinforced concrete bridges

	Material/Fabrication		**Professional**		**Resistance**	
	Bias	**COV**	**Bias**	**COV**	**Bias**	**COV**
Previous	1.12	0.12	1.02	0.06	1.14	0.13
Updated	1.15	0.074	1.02	0.06	1.17	0.09

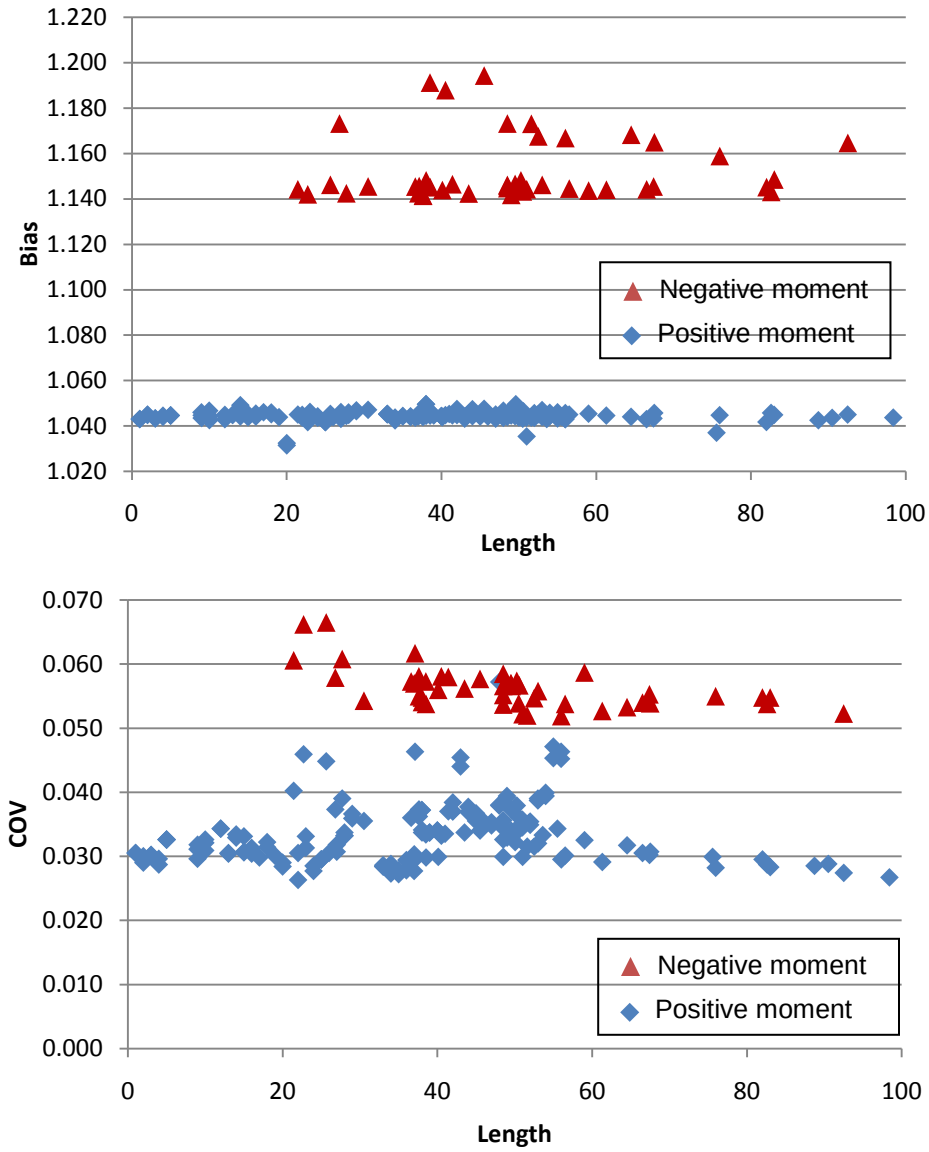


Figure 4.33 Bias and COV for moment in prestressed bridges

Table 4.16 Statistical parameters for moment resistance of prestressed bridges

	Material/Fabrication		**Professional**		**Resistance**	
	Bias	**COV**	**Bias**	**COV**	**Bias**	**COV**
Previous	1.04	0.045	1.01	0.06	1.05	0.075
Current	1.045	0.033	1.01	0.06	1.055	0.069

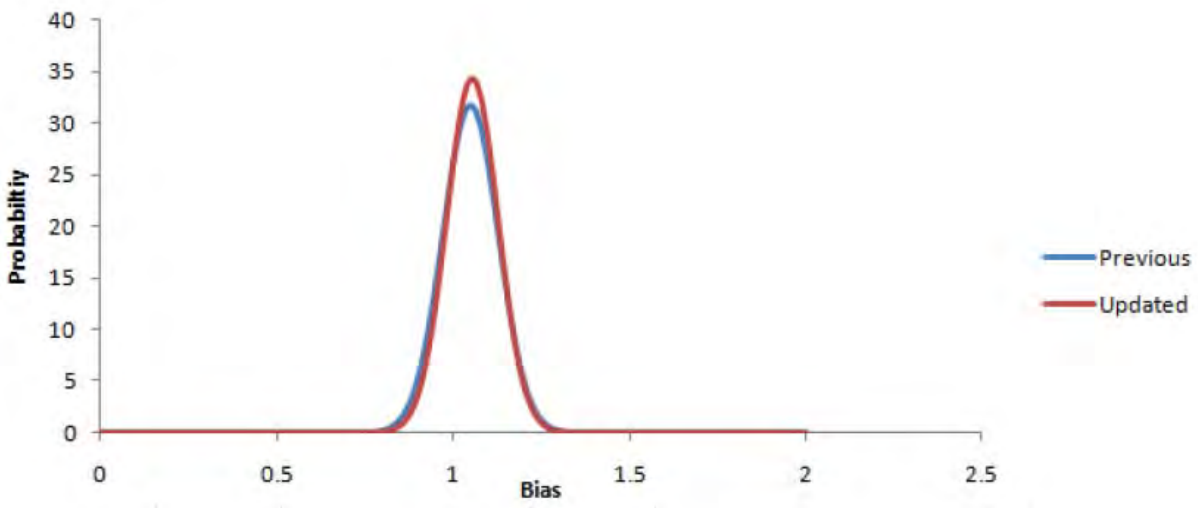


Figure 4.34 Previous and updated distribution of moment resistance for PS concrete bridges

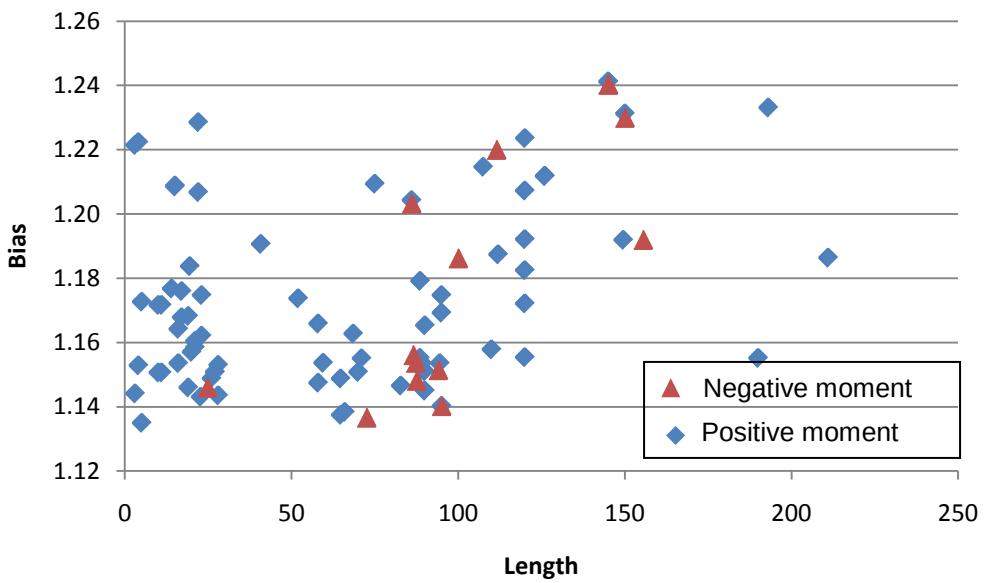


Figure 4.35 Bias and COV for moment in steel bridges (contd.)

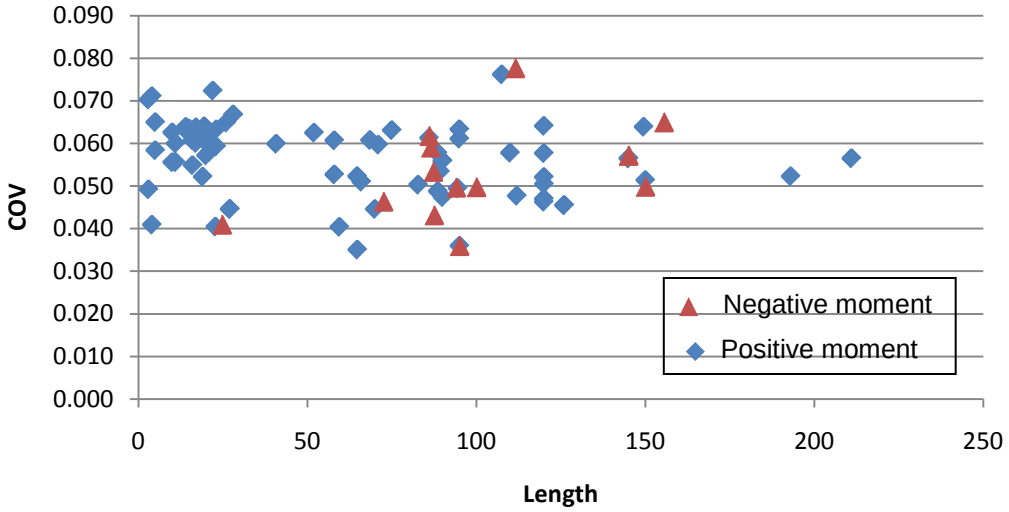


Figure 4.35 Bias and COV for moment in steel bridges

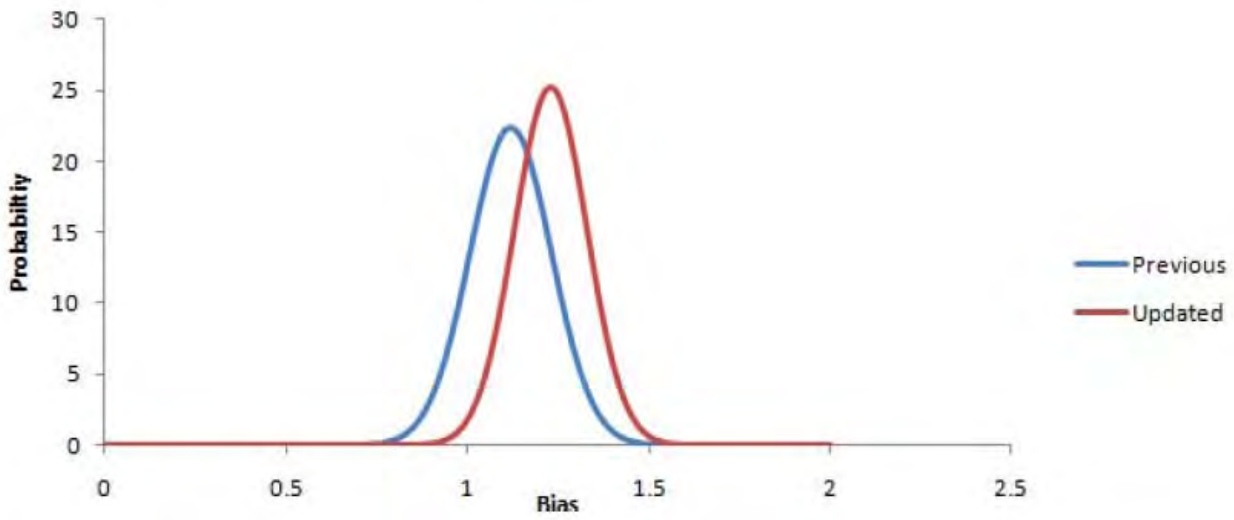


Figure 4.36 Previous and updated distribution of moment resistance for steel bridges

Table 4.17 Statistical parameters for moment resistance of steel bridges

	Material/Fabrication		**Professional**		**Resistance**	
	Bias	**COV**	**Bias**	**COV**	**Bias**	**COV**
Previous	1.07	0.08	1.05	0.06	1.12	0.10
Current	1.18	0.055	1.05	0.06	1.23	0.081

4.7 Reliability Indices of Representative Bridges with Current LRFD Specification

The reliability indices of the selected bridges in Section 4.1 are calculated based on the uncertainties in live load (Section 4.4), dead load (Section 4.5), and minimum required resistance (Section 4.6). The minimum required resistance (or design resistance) is used in order to ignore any unintended contribution from overdesign. As presented in Section 4.6, the as-built strengths will likely be stronger and thus give an even greater reliability. Uncertainties for moment resistance are considered by applying new bias factors and COVs for resistance proposed in this project. The uncertainties in girder distribution factor and impact factor are embedded in the live loads by randomly generating the factors when the load effect to girder is calculated. In this research, the reliability indices are calculated using First Order Reliability Method (FORM) (Der Kiureghian, 2005) because not all the random variables follow the same distribution, and values from FORM are verified using with Monte-Carlo Simulation (MCS) for ADTT of 5000. MCS method needs a sample size larger than 10^{10} to get a reliability index more than 6.0 which requires tremendous computational resources. The limit-state function, *g* used for the FORM is

$$g = R-(DL + LL) \tag{5.1}$$

where *R*, *DL*, and *LL* are resistance, dead load effect, and live load effect, respectively. As presented in Section 4.6, newly calculated bias factors for moment resistance are larger than the bias factor in NCHRP report 368 (Nowak 1999) and newly calculated COVs for moment resistance are less than COVs in NCHRP report 368. It causes the increase of reliability index for moment resistance as shown in Figure 4.37. For steel girder bridges and RC bridges, the newly calculated bias factors and COVs for resistance increase the reliability index up to about 38%. For prestressed concrete bridges, the newly calculated bias factors and COVs for resistance increase the reliability index only about 7%. The small change in the reliability index in prestressed bridges is due to the small change in the bias and COV factors.

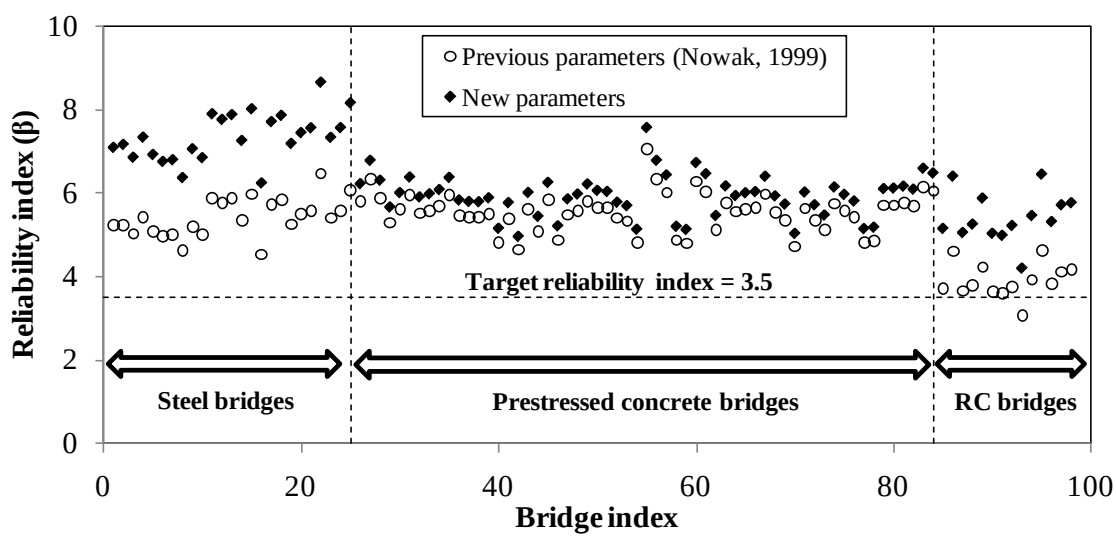


Figure 4.37 Effect of bias factors and COVs for resistance (positive moment)

The reliability indices for positive moments and shear forces of selected bridges are presented in Appendix D and summarized in Figure 4.38. The presented results are based on an ADTT of 5,000. From Figure 4.38, it can be found that the reliability indices for positive moments are higher than the target reliability index of 3.5. The average reliability index for positive moment is around 6.2. Most bridges have reliability indices larger than 4.0. The average reliability index for shear for all bridge types is around 4.3.

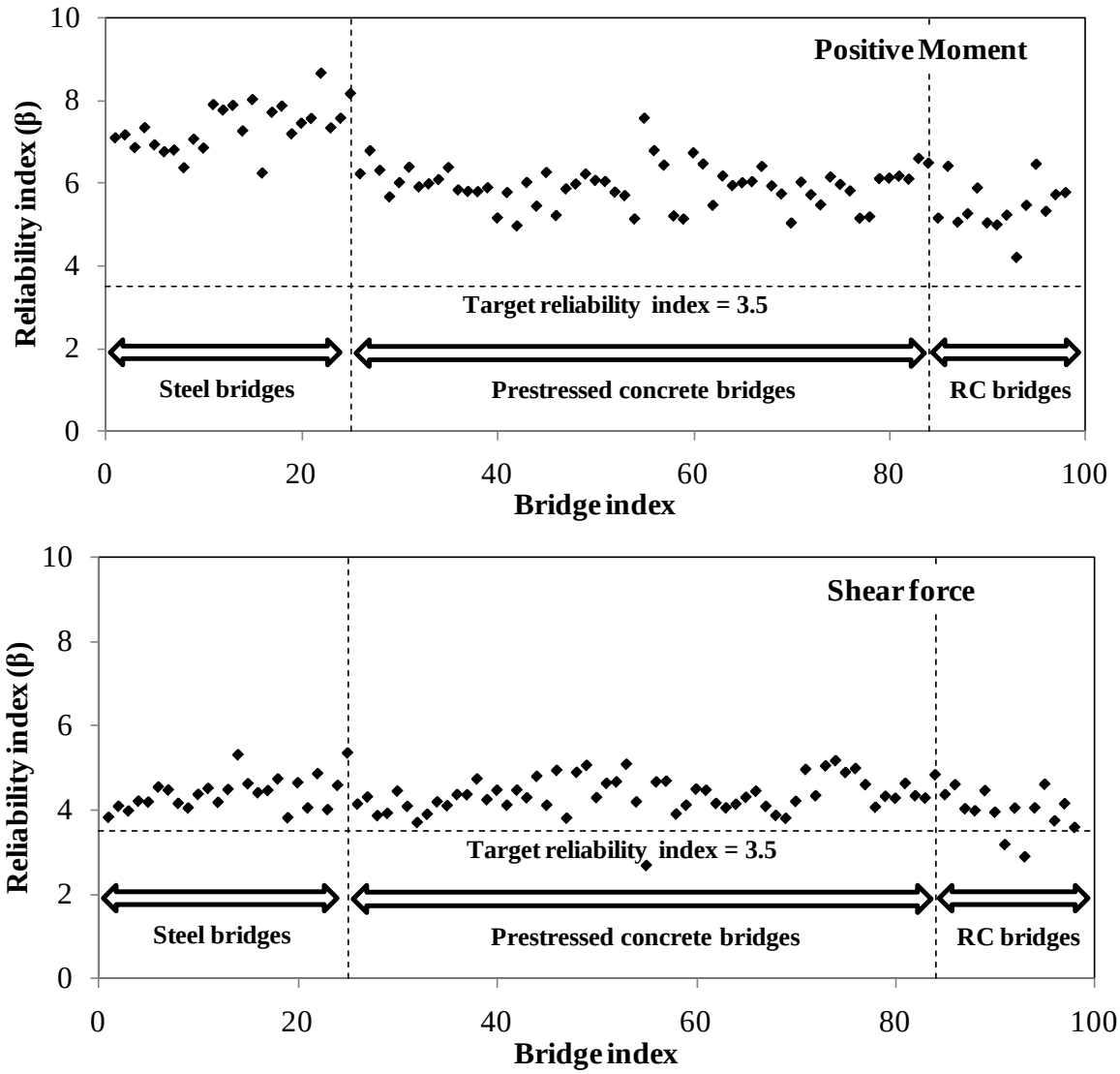


Figure 4.38 Reliability indices in first span of representative bridges

The analysis results in Appendix D and Figure 4.38 are based on an ADTT of 5,000. If a bridge is subjected to a higher volume of truck traffic than other bridges, the bridge will have a higher probability of reaching a limit state. Thus, the ADTT needs to be considered when evaluating the reliability of bridges. The projection of ADTT that a bridge may experience throughout its lifespan is a challenging task and is not a scope of this research. Instead, the reliability index of

bridges conditioned on ADTT is presented. Then, once the ADTT of a bridge is estimated, the reliability of the bridge can be easily calculated.

Figure 4.39 presents variation of reliability indices for selected bridges as a function of ADTT. The average differences between maximum reliability index and minimum reliability index are about 4.5 and 2.6 for positive moment and shear force, respectively. As ADTT decreases from 5,000, the average reliability indices of bridges increases. The decrease in reliability with the increase of ADTT is relatively small when ADTT is greater than 5,000.

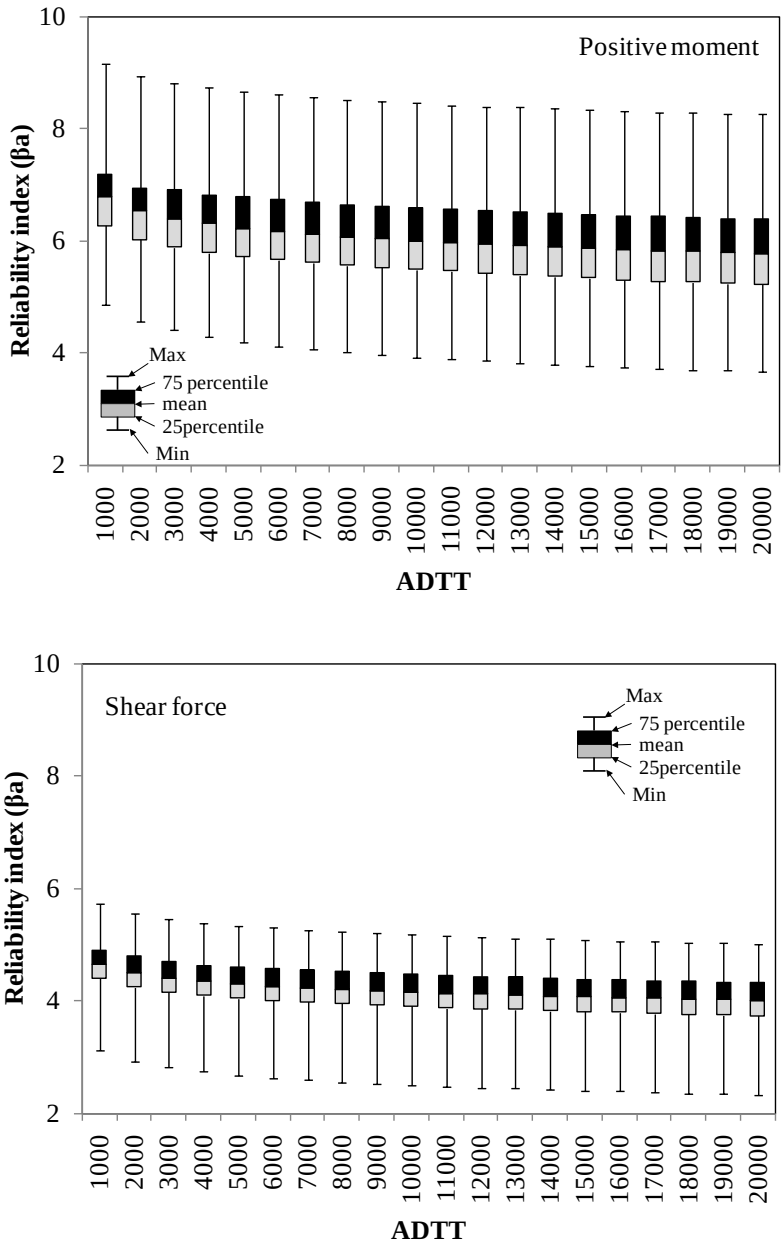


Figure 4.39 Average reliability index for different ADTT values

5 RESULTS AND DISCUSSION

5.1 Proposed Live Load Calibration Factor

In the previous section, it was found that the average reliability indices of selected bridges are much greater than the target reliability index when the ADTT is 5,000. As ADTT decreases from 5,000, the reliability indices rapidly increase. In Section 4.1.3, the ADTTs of bridges in Missouri were analyzed. It was found that only bridges carrying IS highways in Missouri, which takes account around 12% of bridge inventory of the state, have average ADTT of close to 5,000. Bridges carrying US highways (18% of bridge inventory) have an average ADTT of 1,100 and a majority of bridges (70% of bridge inventory) have an average ADTT of less than 500. Hence a large number of bridges in the state have much less truck traffic than the ADTT used for load factor calibration in AASHTO LRFD Design Specification and consequently have a larger reliability.

To optimize the design of bridges considering low ADTT, a live load factor of 1.75 in the current LRFD specification is adjusted with a calibration factor, α, which is based on minimum required strength values. The new live load factor is

$$\gamma_{LL} = 1.75\alpha \tag{5.1}$$

where α depends on ADTT. The relationship between the live load calibration factor, α, on the average reliability indices of bridges is tabulated in Table 5.1 and Table 5.2 for positive moment and shear force, respectively. The gray highlighted portions of the table indicate ADTT and α values that result in a reliability index less than 3.5. The current ADTT may not be used for the design of a bridge as the traffic volume may change throughout the lifespan of the bridge. Once a projected ADTT is estimated, an appropriate live load calibration factor can be identified from the tables.

For simpler application to design practice, live load calibration factors are proposed considering both moment and shear force in Table 5.3. For instance, if a bridge has ADTT of 1,000 or less, the live load calibration factor of 0.8 can be applied still maintaining the average reliability index of 3.5 or greater. Most of bridges in the state belong to this ADTT range. For bridges with ADTT of 5,000 or less, proposed calibration factor is 0.85. The proposed factors are primarily controlled by the reliability index for shear.

When the live load calibration factors in Table 5.3 are applied, reliability indices of positive moment for 53%, 45%, and 61% of representative bridges are greater than the target reliability index of 3.5 for ADTT = 1000, 5000, and 10000, respectively. In terms of shear forces, around 63%, 58%, and 66% of representative bridges have reliability indices greater than the target reliability index for live load calibration factor corresponding to ADTT = 1000, 5000, and 10000 in Table 5.5, respectively. However, the result does not necessarily mean that bridges with reliability indices lower than 3.5 are not safe for several reasons. Firstly, these reliability indices are calculated based on minimum required strength not actual strength and second, on average

close to 60% of bridges have reliability indices greater than 3.5. With these reasons and based on engineering judgment, it is concluded that the proposed reliability indices are acceptable for bridges carrying non-US or non-IS highways.

Table 5.1 Average reliability index for positive moment as a function of calibration factor and projected ADTT

Calibration Factor	**Projected ADTT**									
	1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
1	6.79	6.55	6.41	6.31	6.23	6.17	6.12	6.08	6.04	6.00
0.98	6.63	6.38	6.24	6.15	6.07	6.01	5.96	5.92	5.88	5.84
0.96	6.46	6.22	6.08	5.98	5.91	5.85	5.80	5.75	5.72	5.68
0.94	6.29	6.05	5.91	5.82	5.74	5.68	5.63	5.59	5.55	5.52
0.92	6.12	5.88	5.75	5.65	5.58	5.52	5.47	5.42	5.39	5.35
0.9	5.95	5.71	5.58	5.48	5.41	5.35	5.30	5.26	5.22	5.18
0.88	5.78	5.54	5.41	5.31	5.24	5.18	5.13	5.09	5.05	5.01
0.86	5.61	5.37	5.23	5.14	5.07	5.01	4.96	4.91	4.88	4.84
0.84	5.43	5.19	5.06	4.96	4.89	4.83	4.78	4.74	4.70	4.67
0.82	5.25	5.01	4.88	4.79	4.71	4.66	4.61	4.57	4.53	4.50
0.8	5.07	4.83	4.70	4.61	4.54	4.48	4.43	4.39	4.35	4.32
0.78	4.88	4.65	4.52	4.43	4.36	4.30	4.25	4.21	4.17	4.14
0.76	4.70	4.47	4.34	4.25	4.18	4.12	4.07	4.03	3.99	3.96
0.74	4.51	4.28	4.15	4.06	3.99	3.94	3.89	3.85	3.81	3.78
0.72	4.32	4.10	3.97	3.88	3.81	3.75	3.70	3.66	3.63	3.60
0.7	4.13	3.91	3.78	3.69	3.62	3.57	3.52	3.48	3.44	3.41
0.68	3.94	3.71	3.59	3.50	3.43	3.38	3.33	3.29	3.26	3.23
0.66	3.74	3.52	3.40	3.31	3.24	3.19	3.14	3.10	3.07	3.04
0.64	3.54	3.33	3.20	3.12	3.05	3.00	2.95	2.92	2.88	2.85
0.62	3.35	3.13	3.01	2.92	2.86	2.81	2.76	2.73	2.69	2.66

Table 5.2 Average reliability index for shear force as a function of calibration factor and projected ADTT

Calibration Factor	**Projected ADTT**									
	1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
1	4.66	4.51	4.42	4.36	4.31	4.27	4.24	4.21	4.19	4.16
0.98	4.56	4.41	4.32	4.26	4.21	4.17	4.14	4.11	4.09	4.07
0.96	4.46	4.31	4.22	4.16	4.11	4.07	4.04	4.01	3.99	3.97
0.94	4.36	4.21	4.12	4.06	4.01	3.97	3.94	3.91	3.89	3.87
0.92	4.26	4.11	4.02	3.96	3.91	3.87	3.84	3.81	3.79	3.77
0.9	4.16	4.00	3.92	3.86	3.81	3.77	3.74	3.71	3.69	3.67
0.88	4.05	3.90	3.81	3.75	3.70	3.67	3.63	3.61	3.58	3.56
0.86	3.94	3.79	3.71	3.65	3.60	3.56	3.53	3.50	3.48	3.46
0.84	3.84	3.69	3.60	3.54	3.50	3.45	3.42	3.39	3.37	3.35
0.82	3.73	3.58	3.49	3.43	3.38	3.35	3.31	3.29	3.26	3.24
0.8	3.62	3.47	3.38	3.32	3.27	3.24	3.20	3.18	3.15	3.13
0.78	3.50	3.35	3.27	3.21	3.16	3.12	3.09	3.07	3.04	3.02
0.76	3.39	3.24	3.15	3.09	3.05	3.01	2.98	2.95	2.93	2.91
0.74	3.27	3.12	3.04	2.98	2.93	2.90	2.86	2.84	2.81	2.79
0.72	3.16	3.01	2.92	2.86	2.82	2.78	2.75	2.72	2.70	2.68
0.7	3.04	2.89	2.80	2.74	2.70	2.66	2.63	2.60	2.58	2.56
0.68	2.91	2.77	2.68	2.62	2.58	2.54	2.51	2.48	2.46	2.44
0.66	2.79	2.64	2.56	2.50	2.46	2.42	2.39	2.36	2.34	2.32
0.64	2.67	2.52	2.44	2.38	2.33	2.30	2.27	2.24	2.22	2.20
0.62	2.54	2.39	2.31	2.25	2.21	2.17	2.14	2.11	2.09	2.07

Table 5.3 Proposed live load calibration factor for moment

	Live load calibration factor			
	ADTT ≤ 1000	**1000 < ADTT ≤ 5000**	**5000 < ADTT ≤ 10000**	**10,000<ADTT**
Moment	0.64	0.70	0.72	-
Proposed factors	0.65	0.70	0.75	1.00

Table 5.4 Proposed live load calibration factor for shear force

	Live load calibration factor			
	ADTT ≤ 1000	**1000 < ADTT ≤ 5000**	**5000 < ADTT ≤ 10000**	**10,000<ADTT**
Shear	0.78	0.84	0.88	-
Proposed factors	0.80	0.85	0.90	1.00

5.2 Impact of the Live Load Calibration on Bridge Construction Cost

The cost impact analysis of the proposed calibration factor is based on the relationship between truck weight and bridge construction cost in NCHRP Report 495 (Fu et al. 2003). In the report, the relative cost of bridge superstructure construction is presented with respect to the bridge construction cost due to HS20 truck loading. As the objective of this research is to estimate the cost impact when the HL-93 truck load is reduced using the live load calibration factor proposed in Section 5.1, the cost ratio - truck weight relationship in NCHRP Report 495 is converted to cost ratio - maximum moment relationship. For example, Figure 5.1 presents the cost ratios for RC slab bridges with span lengths of 60 ft. The marks indicated with triangles relate cost ratios with the maximum moment developed by HS-20, HS-22.5, and HS-25 design loads. The figure clearly shows that bridge construction cost increases as moments increase. The moment due to HL-93 design loads with different calibration factors (0.7, 0.8, 0.9, and 1.0) are plotted on top of the cost ratio-moment relationship. Then, the cost ratios are re-plotted in Figure 5.2 with respect to the cost of bridges designed with HL-93 and the calibration factors applied to the load. Similar analyses are carried out for different types of bridges and the range of cost impacts are summarized in Table 5.5. To achieve a uniform reliability level, bridges with lower ADTTs can be designed with a lower calibration factor as presented in Table 5.3, which leads to savings in construction costs. It can be found from Table 5.5 that for bridges with ADTT less than 1,000, the construction cost can be reduced by about 17% depending on the bridge construction materials. For bridges with ADTT less than 5,000, the cost saving is up to about 12%. Steel bridges or reinforced concrete slab bridges have larger cost savings than prestressed concrete bridges. For prestressed concrete bridges, the cost saving is negligible as most prestressed concrete girders use standard section shapes with different numbers of tendons depending on the applied loads. The actual monetary value of cost savings is not evaluated at the time of this report preparation.

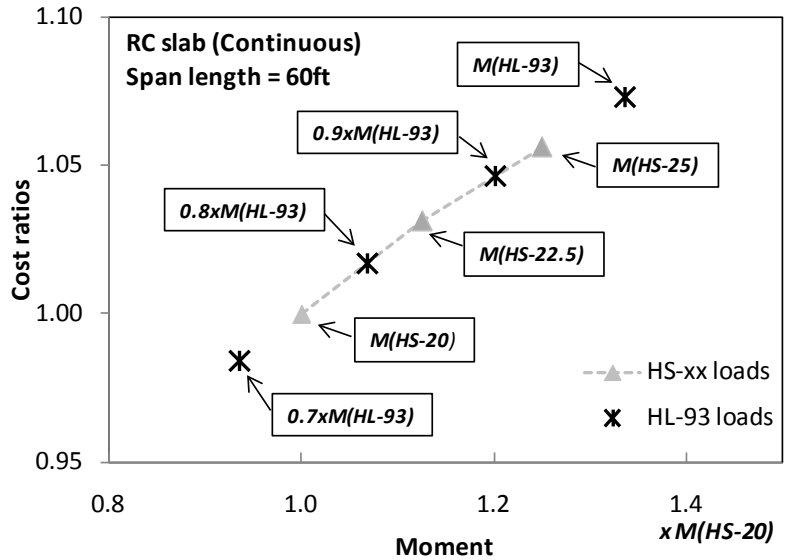


Figure 5.1 Cost ratios for live load calibration factors (60ft, RC Slab bridge)

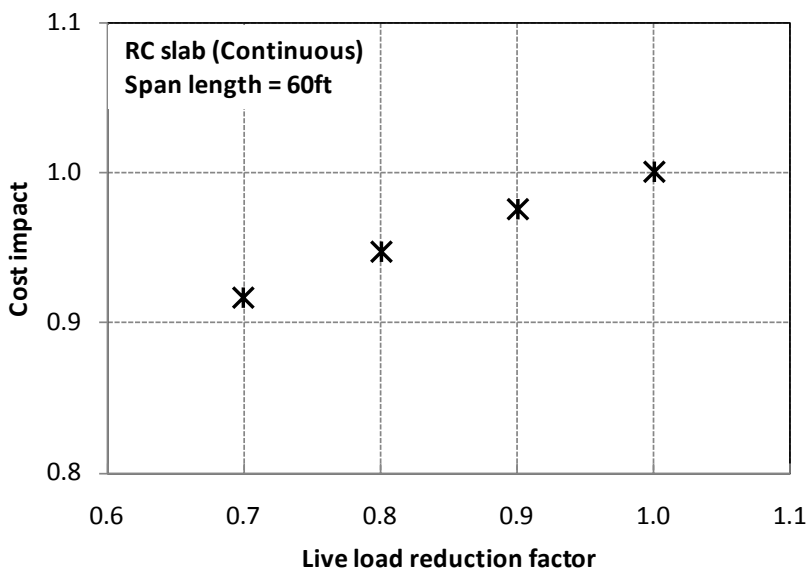


Figure 5.2 Cost impacts for live load calibration factors (60ft, RC Slab bridge)

Table 5.5 Cost impact for different ADTT values

Super structure type	**Cost impacts**					
	ADTT = 1000		**ADTT = 5000**		**ADTT = 10000**	
	α = 0.65	**α = 0.80**	**α = 0.70**	**α = 0.85**	**α = 0.75**	**α = 0.90**
Steel girder	0.89~0.94	0.93~0.97	0.90~0.95	0.95~0.98	0.92~0.96	0.96~0.98
Reinforced concrete slab	0.83~0.93	0.89~0.98	0.88~0.96	0.92~1.00	0.91~0.97	0.95~1.00
PS concrete girder	1.00	1.00	1.00	1.00	1.00	1.00

6 CONCLUSIONS

The reliability indices of bridges in Missouri are evaluated based on the bridge configurations and traffic environments in the state. Typical bridge configurations of the state are identified based on statistical analysis of bridge configurations in the NBI database. WIM data are screened through inspection of daily GVW and using the criteria in NCHRP Web-Only Document 135. It is assumed that the distribution of daily maximum load effects follow Gumbel Type I distribution and extreme value theory is applied to project daily maximum values to 75-year maximum values. FORM is used to run reliability analysis. Based on the reliability analysis, the following are found from the research:

- The ADTT of bridges in the state are much lower than the ADTT used for the development of the AASHTO LRFD Specification. Around 70% of bridges have an ADTT less than 500 and 18% of bridges have an ADTT less 1,100. Bridges carrying interstate highways have an ADTT close to 5,000.
- When an ADTT of 5,000 is assumed, the average reliability indices of bridges based on the minimum required (or design) resistance in the state are 6.2 and 4.3 for positive moment and shear force, respectively, which are much higher than the target reliability index of 3.5.
- The ratio of minimum required (design) strength to the actual strength is generally less than 1, indicating that bridges would have an even greater reliability.
- Updated material parameters improved the reliability of steel girder bridge moments by 38%, but only improved the reliability of prestressed bridges by 7%. The material parameters for prestressed bridges have not changed greatly.
- Live load calibration factors are proposed in this study. The proposed live load calibration factor is a function of ADTT.
- Monetary value of the cost savings has not been evaluated yet due to lack of data. The calibration factors may provide cost savings in the superstructure up to 10% depending on bridge material type.
- Steel bridges and reinforced concrete slab bridges have relatively large cost savings. Prestressed concrete bridges, however, do not have noticeable cost savings as standard sections are used for the bridge girders.

The calibration study conducted in this project utilizes up-to-date data on bridge inventory, truck weights, and material variability. Adopted methodologies are carefully reviewed and best efforts were made to fully justify the approaches. Still, there are many opportunities to refine and improve the reliability estimate. For instance, the traffic flow simulation can be improved through collaboration with experts in traffic engineering. The reliability indices in this study are based on the assumption that the bridge strength does not deteriorate throughout its lifetime. In future studies, it would provide more reliable reliability indices by incorporating a time-dependent deterioration model of bridge girders. In addition, other limit states, such as Service Limit State or Extreme Events limit state, also need further refinement.

7 RECOMMENDATIONS

Reliability of bridges is sensitive to many parameters such as bridge configuration, truck weight, traffic volume, resistance, girder distribution factor, etc. Among these parameters, the bridge configuration, truck weight, and traffic volume varies depending on the region. The proposed calibration factors are developed based on state-wide bridge configuration and truck weight. From the research, it was found that bridges in the state have wide range of ADTTs based on NBI database. As the reliability of a bridge highly depends on ADTT, the calibration factors are proposed as a function of ADTT.

The ADTT in this research is an average daily truck traffic that a bridge may experience throughout its life span. Hence, to design a bridge using the factors proposed in this study, the expected ADTT of the bridge should be known in advance. The prediction of ADTT, however, is a very challenging task even for the engineers in traffic planning. The followings are recommendations for practicing designers.

- If a sound justification for the ADTT is not available, it is suggested to use a factor of 0.8 for bridges which do not carry US highways or IS highways. Based on the NBI database, majority of bridges in this category have ADTT much less than 5,000, with an average of 415. Considering importance and consequence of failure in transportation network, it is not recommended to apply the factors to the design of bridges carrying US or IS highways.
- Alternatively, the simplified tables, Table 7.3 and Table 7.4, which are based on Table 7.1 and Table 7.2, can be used for a simpler and more conservative design. Again, considering the importance of bridges in the transportation network and a large consequence of a failure, the ADTT of US or IS highways should be fully justified before the calibration factors are applied for the design of bridges carrying US or IS highways.
- If the ADTT of a bridge can be predicted and justified, Table 7.1 and Table 7.2 can be used to find the optimal live load calibration factor. If the design of a bridge is governed by moment even after applying the factor in Table 7.1, then the factor in Table 7.1 may be used. If the design is controlled by shear, then the factor in Table 7.2 should be applied. For the design of bridges carrying US or IS highway system, the engineering judgment should be exercised when applying the factors.

It is expected that the third method in the above will result in the largest cost saving as smaller factors can be used. On the other hand, sound justification should be provided and engineering judgment should be exercised to use the method.

Table 7.1 Average reliability index for positive moment as a function of calibration factor and projected ADTT

Calibration Factor	**Projected ADTT**									
	1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
1	6.79	6.55	6.41	6.31	6.23	6.17	6.12	6.08	6.04	6.00
0.98	6.63	6.38	6.24	6.15	6.07	6.01	5.96	5.92	5.88	5.84
0.96	6.46	6.22	6.08	5.98	5.91	5.85	5.80	5.75	5.72	5.68
0.94	6.29	6.05	5.91	5.82	5.74	5.68	5.63	5.59	5.55	5.52
0.92	6.12	5.88	5.75	5.65	5.58	5.52	5.47	5.42	5.39	5.35
0.9	5.95	5.71	5.58	5.48	5.41	5.35	5.30	5.26	5.22	5.18
0.88	5.78	5.54	5.41	5.31	5.24	5.18	5.13	5.09	5.05	5.01
0.86	5.61	5.37	5.23	5.14	5.07	5.01	4.96	4.91	4.88	4.84
0.84	5.43	5.19	5.06	4.96	4.89	4.83	4.78	4.74	4.70	4.67
0.82	5.25	5.01	4.88	4.79	4.71	4.66	4.61	4.57	4.53	4.50
0.8	5.07	4.83	4.70	4.61	4.54	4.48	4.43	4.39	4.35	4.32
0.78	4.88	4.65	4.52	4.43	4.36	4.30	4.25	4.21	4.17	4.14
0.76	4.70	4.47	4.34	4.25	4.18	4.12	4.07	4.03	3.99	3.96
0.74	4.51	4.28	4.15	4.06	3.99	3.94	3.89	3.85	3.81	3.78
0.72	4.32	4.10	3.97	3.88	3.81	3.75	3.70	3.66	3.63	3.60
0.7	4.13	3.91	3.78	3.69	3.62	3.57	3.52	3.48	3.44	3.41
0.68	3.94	3.71	3.59	3.50	3.43	3.38	3.33	3.29	3.26	3.23
0.66	3.74	3.52	3.40	3.31	3.24	3.19	3.14	3.10	3.07	3.04
0.64	3.54	3.33	3.20	3.12	3.05	3.00	2.95	2.92	2.88	2.85
0.62	3.35	3.13	3.01	2.92	2.86	2.81	2.76	2.73	2.69	2.66

Note: This table is identical to Table 5.1. It is repeated here such that it can be easily referred from the Recommendation section.

Table 7.2 Average reliability index for shear force as a function of calibration factor and projected ADTT

Calibration Factor	**Projected ADTT**									
	1000	**2000**	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
1	4.66	4.51	4.42	4.36	4.31	4.27	4.24	4.21	4.19	4.16
0.98	4.56	4.41	4.32	4.26	4.21	4.17	4.14	4.11	4.09	4.07
0.96	4.46	4.31	4.22	4.16	4.11	4.07	4.04	4.01	3.99	3.97
0.94	4.36	4.21	4.12	4.06	4.01	3.97	3.94	3.91	3.89	3.87
0.92	4.26	4.11	4.02	3.96	3.91	3.87	3.84	3.81	3.79	3.77
0.9	4.16	4.00	3.92	3.86	3.81	3.77	3.74	3.71	3.69	3.67
0.88	4.05	3.90	3.81	3.75	3.70	3.67	3.63	3.61	3.58	3.56
0.86	3.94	3.79	3.71	3.65	3.60	3.56	3.53	3.50	3.48	3.46
0.84	3.84	3.69	3.60	3.54	3.50	3.45	3.42	3.39	3.37	3.35
0.82	3.73	3.58	3.49	3.43	3.38	3.35	3.31	3.29	3.26	3.24
0.8	3.62	3.47	3.38	3.32	3.27	3.24	3.20	3.18	3.15	3.13
0.78	3.50	3.35	3.27	3.21	3.16	3.12	3.09	3.07	3.04	3.02
0.76	3.39	3.24	3.15	3.09	3.05	3.01	2.98	2.95	2.93	2.91
0.74	3.27	3.12	3.04	2.98	2.93	2.90	2.86	2.84	2.81	2.79
0.72	3.16	3.01	2.92	2.86	2.82	2.78	2.75	2.72	2.70	2.68
0.7	3.04	2.89	2.80	2.74	2.70	2.66	2.63	2.60	2.58	2.56
0.68	2.91	2.77	2.68	2.62	2.58	2.54	2.51	2.48	2.46	2.44
0.66	2.79	2.64	2.56	2.50	2.46	2.42	2.39	2.36	2.34	2.32
0.64	2.67	2.52	2.44	2.38	2.33	2.30	2.27	2.24	2.22	2.20
0.62	2.54	2.39	2.31	2.25	2.21	2.17	2.14	2.11	2.09	2.07

Note: This table is identical to Table 5.2. It is repeated here such that it can easily referred from the Recommendation section.

Table 7.3 Proposed live load calibration factor for moment

	Live load calibration factor			
	ADTT ≤ 1000	**1000 < ADTT ≤ 5000**	**5000 < ADTT ≤ 10000**	**10,000<ADTT**
Moment	0.64	0.70	0.72	-
Proposed factors	0.65	0.70	0.75	1.00

Note: This table is identical to Table 5.3. It is repeated here such that it can easily referred from the Recommendation section.

Table 7.4 Proposed live load calibration factor for shear force

	Live load calibration factor			
	ADTT ≤ 1000	**1000 < ADTT ≤ 5000**	**5000 < ADTT ≤ 10000**	**10,000<ADTT**
Shear	0.78	0.84	0.88	-
Proposed factors	0.80	0.85	0.90	1.00

Note: This table is identical to Table 5.4. It is repeated here such that it can easily referred from the Recommendation section.

BIBLIOGRAPHY

AASHTO (2007), "LRFD bridge design specifications." 4th Edition, Washington, DC

AASHTO (1996), "Standard specifications for Highway bridge design." 16th Edition, Washington, DC

Ang, A.H.-S. and Tang, W.H. (1975), "Probability concepts in engineering planning and design," Volume I – basic principles, John Wiley & Sons, Inc., New York.

Ang, A.H.-S. and Tang, W.H. (1984), "Probability concepts in engineering planning and design," Volume II – decision, risk and reliability, John Wiley & Sons, Inc., New York.

Bartlett, M. and MacGregor, J.S. (1996), "Statistical analysis of the compressive strength of concrete in structures," ACI Materials Journal, Vol. 93, No. 2, pp. 158-168.

Box, G.E.P. and Wilson, K.B. (1951), "On the experimental attainment of optimum conditions", Journal of the Royal Statistical Society, B13, pp.1-38.

Chang, M.S. and Kim, Y.K. (2000), "Development of Capacity Estimation Method from Statistical Distribution of Observed Traffic Flow", Proceedings: Fourth International Symposium on Highway Capacity, pp.299-309.

Cooper, D.I. (1995), "The determination of highway bridge design loading in the United Kingdom from traffic measurements", Pre-Proceedings of the First European Conference on Weigh-in-Motion of Road Vehicles, Zürich, pp.413-421.

Cowan, R.J. (1975), "Useful headway models", Transportation Research, 9, pp.371-375.

Der Kiureghian, A. (2005), "First- and Second-Order Reliability Methods," Engineering Design Reliability Handbook, CRC, Boca Raton, FL, Chap. 14.

Gumbel, E.J. (1958), Statistics of Extremes, Columbia University Press.

He, M., Liu, X., and Rong, J. (2002), "Driver's Preferred Time Headway Selection: Experimental Findings", Proceedings of the Third International Conference on Traffic and Transportation Studies.

Hwang, E.S. and Nowak, A.S. (1991), "Simulation of dynamic load for bridges." Journal of Structural Engineering, ASCE, Vol.117, No.5, pp.1413–1434.

Imbsen, R.A., Liu, W.D., and Schamber, R.A. (1987), "Strength Evaluation of Existing Reinforced Concrete Bridges," NCHRP Report 292, Transportation Research Board, Washington, D.C.

Kozikowski, M. (2009), "WIM Based Live Load Model for Bridge Reliability", Ph.D Thesis, University of Nebraska – Lincoln.

Kulicki, J. M., Prucz, A., Clancy, C. M., Mertz, D. R., and Nowak, A. S. (2007). "Updating the calibration report for AASHTO LRFD code," Final Report, Project No. NCHRP 20-7/186, Transportation Research Board of the National Academies, Washington, D.C.

Law, A. M. and Kelton, W. D. (2000), "Simulation modeling & analysis" 3rd Edition, New York, McGraw-Hill, Inc.

Nowak, A.S., Yamani, A.S. and Tabsh, S.W. (1994), "Probabilistic Models for Resistance of Concrete Bridge Girders", ACI Structural Journal, Vol. 91, No. 3, pp. 269-276.

Nowak, A.S. (1999), "NCHRP Report 368: Calibration of LRFD Bridge Design Code." Transportation Research Board, National Research Council, Washington, DC.

OBrien, E.J., Sloan, T.D., Butler, K.M. and Kirkpatrick, J. (1995), "Traffic load fingerprinting of bridges for assessment purposes", The Structural Engineer, 73(19), pp. 320-324.

Oracle (2008) Oracle Crystal Ball Fusion Edition Release 11.1.1.1.00, October 2008.

Page, J. (1973), “Dynamic Wheel Load Measurement on Motorway Bridges”, Report LR722, Transport and Road Research Laboratory, Crowthorne, England.

Pelphrey, J., and Higgins, C. (2006), “Calibration of LRFR live load factors for Oregon state-owned bridges using weigh-in-motion data.” Kiewit Center for Infrastructure and Transportation Rep., Dept. of Civil Engineering, Oregon State Univ., Corvallis, Ore.

Pratim Dey, P. and Chandra, S. (2009), “Desired Time Gap and Time Headway in Steady-State Car-Following on Two-Lane Roads”, Journal of transportation engineering, ASCE, 135-10, pp.687-693.

Schmidt, B.J. and Bartlett, M. (2002), “Review of resistance factor for steel: Resistance distributions and resistance factor calibration,” Canadian Journal of Civil Engineering, Vol. 29, No. 1, pp. 109-118.

Schuhl, A. (1955), “The probability theory applied to the distribution of vehicles on two lane highways,” Physical Infrastructure Centre Report, 92-Control, The Eno Foundation for Highway Traffic Arterial Roads. Sangatuck, CT.

Shepherd, R. and Aves, R.J. (1973), “Impact factor for simple concrete bridges.” Proceedings, Institute of civil engineering, Part II, Vol.55, pp.191-210.

Sivakumar, B., Ghosn, M., and Moses, F. (2008), “Protocols for Collecting and Using Traffic Data in Bridge Design.”, NCHRP web-only document 135, Transportation Research Board, National Research Council, Washington, D.C. (2008).

Van de Lindt, J.W., Fu, G., Pablo Jr., R., and Pei S. (2005), “LRFD Load Calibration for State of Michigan Trunkline Bridges”, Report RC-1466, Michigan Department of Transportation, MI.

APPENDIX A: SELECTED BRIDGES

Table A-1 Representative bridges for continuously supported concrete bridge (slab)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Concrete -continuous	C1	2908	3	2	14.9	0.061	1983
	F1	29086	4	3	18.6	0.836	2002
	F2	28993	3	2	7.0	1.419	2003
	F3	2856	4	1	18.3	0.897	1981
	F4	3686	3	2	7.6	1.422	1986
	F5	29534	2	2	14.6	1.433	2003
	F6	29321	2	2	4.9	1.027	2002
	F7	3376	3	1	17.4	1.106	1985
	F8	3113	2	1	14.1	0.990	1981
	A1	3309	4	2	12.8	0.523	1983
	A2	2983	3	2	14.0	1.419	1983
	A3	3947	3	4	14.9	0.603	1993
	A4	2984	3	2	14.0	1.419	1983
	A5	2746	3	2	17.1	0.831	1987
	A6	28990	3	2	9.1	0.667	2003

*: Target point
§ : Distance from target point to selected bridge data in normalized space

Table A-2 Representative bridges for simply supported steel bridges (girder)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Steel -simple	C1	30894	1	2	27.1	0.290	2005
	F1	31528	1	3	63.4	1.231	2006
	F2	31581	1	2	7.1	1.424	2005
	F3	11877	2	2	40.8	1.636	1995
	F4	9398	2	1	9.7	0.979	1982
	F5	31529	1	3	63.4	1.231	2006
	F6	29748	1	2	7.4	1.425	2003
	F7	29592	1	2	21.0	2.033	2003
	F8	31577	1	2	9.0	1.431	2005
	A1	29665	1	2	12.2	1.577	2003
	A2	30159	1	2	11.3	1.588	2002
	A3	28962	1	3	38.4	0.267	2003
	A4	28968	1	3	38.4	3.094	2003
	A5	11761	1	4	66.8	4.426	1998
	A6	11993	1	4	37.3	4.614	1998

*: Target point
§ : Distance from target point to selected bridge data in normalized space

Table A-3 Representative bridges for continuously supported steel bridges (girder)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Steel -continuous	C1	3204	3	2	36.6	0.000	1983
	F1	12126	6	3	54.6	0.337	2001
	F2	31756	6	3	33.5	0.919	2005
	F3	2692	6	1	46.7	0.526	1984
	F4	3190	6	1	22.3	0.486	1983
	F5	3945	2	3	51.6	0.775	1989
	F6	31500	2	3	20.3	0.835	2006
	F7	12073	2	1	61.1	0.861	2001
	F8	2852	3	1	29.6	1.236	1981
	A1	31812	8	2	38.1	0.129	2007
	A2	2664	2	2	36.6	1.097	1987
	A3	4043	3	3	36.3	0.089	1991
	A4	4412	3	1	36.6	0.650	1997
	A5	29559	3	2	57.9	0.169	2003
	A6	3475	3	2	7.9	0.481	1981

*: Target point
§ : Distance from target point to selected bridge data in normalized space

Table A-4 Representative bridges for simply supported prestressed bridges (girder)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Prestressed concrete -simple	C1	3612	3	2	20.3	0.016	1989
	F1	3393	5	3	19.7	1.282	1993
	F2	4417	4	2	14.8	1.097	1997
	F3	3922	3	1	20.5	1.414	1989
	F4	3851	5	2	16.1	1.313	1990
	F5	31798	1	2	25.9	1.025	2007
	F6	29626	1	2	15.8	1.107	2003
	F7	31310	1	2	27.4	1.180	2006
	F8	3973	3	2	11.8	1.426	1992
	A1	3706	5	2	19.8	0.289	1987
	A2	31534	1	2	21.9	0.607	2006
	A3	29736	3	3	20.4	0.402	2003
	A4	3756	3	2	20.6	1.416	1988
	A5	3732	3	2	26.8	0.024	1988
	A6	4315	3	2	12.4	0.629	1995

*: Target point
§ : Distance from target point to selected bridge data in normalized space

Table A-5 Representative bridges for simply supported prestressed bridges (Double tee)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Prestressed concrete -simple	C1	3550	3	2	15.1	0.600	1986
	F1	3574	4	2	15.1	1.904	1985
	F2	3774	4	2	15.1	1.106	1989
	F3	10077	4	2	15.1	1.904	1985
	F4	3326	5	2	12.6	1.239	1983
	F5	3964	1	2	15.8	1.839	1989
	F6	3499	1	2	15.2	1.087	1984
	F7	3148	3	2	11.3	2.320	1983
	F8	3366	3	2	7.6	1.415	1982
	A1	3685	3	2	11.2	1.647	1985
	A2	12121	1	2	32.3	3.433	2001
	A3	3707	6	2	11.5	3.281	1987
	A4	29057	1	6	34.7	8.435	2001
	A5	29060	1	6	34.7	6.511	2001
	A6	3759	11	2	15.2	11.496	1989

*: Target point
§ : Distance from target point to selected bridge data in normalized space

Table A-6 Representative bridges for continuously supported prestressed bridges (Girder)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Prestressed concrete -continuous	C1	3919	3	2	19.8	0.000	1989
	F1	4322	4	2	25.0	1.003	1997
	F2	4034	4	2	13.9	1.065	1992
	F3	31246	4	2	25.0	1.003	2005
	F4	3276	4	1	18.2	0.801	1991
	F5	3583	2	2	25.5	1.077	1986
	F6	4045	2	2	15.1	1.168	1991
	F7	11933	3	1	25.6	1.016	2000
	F8	31613	2	1	16.2	0.701	2006
	A1	3753	4	2	19.8	0.477	1997
	A2	10086	1	2	17.1	0.603	1995
	A3	3332	3	3	19.8	0.626	1985
	A4	11893	3	1	19.8	0.564	1999
	A5	11821	3	2	26.9	0.005	1999
	A6	4052	3	2	8.2	0.493	1993

*: Target point
§ : Distance from target point to selected bridge data in normalized space

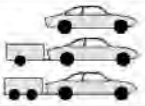
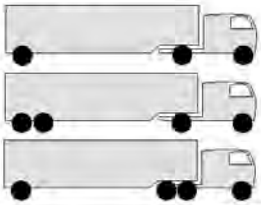
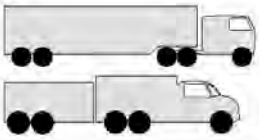
Table A-7 Representative bridges for continuously supported prestressed bridges (Double tee)

Bridge types	TP*	Bridge No.(NBI)	No. of span	No. of lane	Max. span length(m)	Distance from TP§	Year Built
Prestressed concrete -continuous	C1	3531	3	2	15.4	0.515	1985
	F1	10164	4	2	29.3	1.485	1995
	F2	3548	4	2	11.4	1.022	1990
	F3	4206	3	2	28.2	1.611	1995
	F4	3508	4	2	11.5	1.024	1984
	F5	4207	3	2	28.2	1.611	1995
	F6	3501	3	2	6.9	1.415	1985
	F7	29023	3	2	30.5	2.044	2002
	F8	3621	3	2	6.9	1.415	1986
	A1	3509	4	2	14.8	0.742	1986
	A2	3731	3	2	15.4	1.505	1990
	A3	3568	3	3	14.9	0.839	1985
	A4	3329	3	2	15.3	1.508	1984
	A5	31413	3	2	31.0	1.228	2006
	A6	2478	3	2	8.4	0.499	1988

*: Target point

§ : Distance from target point to selected bridge data in normalized space

APPENDIX B: FHWA VEHICLE CLASSIFICATION

CLASS GROUP		DESCRIPTION	NO. OF AXLES
1		MOTORCYCLES	2
2		ALL CARS CARS W/ 1-AXLE TRAILER CARS W/ 2-AXLE TRAILER	2 3 4
3		PICK-UPS & VANS 1 & 2 AXLE TRAILERS	2, 3, & 4
4		BUSES	2 & 3
5		2-AXLE, SINGLE UNIT	2
6		3-AXLE, SINGLE UNIT	3
7		4-AXLE, SINGLE UNIT	4
8		2-AXLE, TRACTOR, 1-AXLE TRAILER (2&1) 2-AXLE, TRACTOR, 2-AXLE TRAILER (2&2) 3-AXLE, TRACTOR, 1-AXLE TRAILER (3&1)	3 4 4
9		3-AXLE, TRACTOR, 2-AXLE TRAILER (3&2) 3-AXLE, TRUCK W/ 2-AXLE TRAILER	5 5
10		TRACTOR W/ SINGLE TRAILER	6 & 7
11		5-AXLE MULTI-TRAILER	5
12		6-AXLE MULTI-TRAILER	6
13	ANY 7 OR MORE AXLE		7 or more
14	NOT USED		
15	UNKNOWN VEHICLE TYPE		

HEAVY TRUCKS

APPENDIX C: BRIDGE RESISTANCE

Table C-1 Representative bridges for continuously supported reinforced concrete bridge (slab)

Type	Bridge #		Year	Spans	Length (ft)						
	State	NBI			Span 1	Span 1-2	Span 2	Span 2-3	Span 3	Span 3-4	Span 4
Voided Slab	A3246	2746	1987	3	42.00	49.00	56.00	49.00	42.00	-	-
Voided Slab	A3390	2856	1981	4	49.70	54.85	60.00	54.00	48.00	42.58	37.15
Voided Slab	A3462	2908	1983	3	36.36	42.68	49.00	42.68	36.36	-	-
Voided Slab	A3562	2983	1983	3	35.27	40.64	46.00	40.64	35.27	-	-
Voided Slab	A3563	2984	1977	3	31.00	38.50	46.00	38.50	31.00	-	-
Slab	A3741	3113	1981	2	46.81	46.81	46.81	-	-	-	-
Slab	A3772	3138	1996	3	27.25	30.13	33.00	30.13	27.25	-	-
Slab	A4027	3309	1982	4	33.27	37.64	42.00	42.00	42.00	40.64	39.27
Voided Slab	A4113	3376	1985	3	47.64	52.32	57.00	52.32	47.64	-	-
Slab	A4507	3686	1986	3	25.00	25.00	25.00	25.00	25.00	-	-
Voided Slab	A4844	3947	1993	3	43.25	46.13	49.00	46.13	43.25	-	-
Slab	A6447	28990	1999	3	25.25	27.63	30.00	27.63	25.25	-	-
Slab	A6450	28993	2003	3	19.38	21.19	23.00	21.19	19.38	-	-
Voided Slab	A6460	29086	2002	4	52.41	56.70	61.00	61.00	61.00	56.67	52.33

Table C-2 Actual and required shear resistance (kip per ft width of bridge)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI	Shear (k/ft)	Shear (k/ft)	Kip/ft	Kip/ft	Kip	
A3246	2746	36.40	22.79	5.96	1.05	93.75	11.92
A3390	2856	40.93	22.88	6.03	0.47	101.13	12.09
A3462	2908	38.07	19.71	4.65	0.37	88.50	11.60
A3562	2983	41.63	21.22	5.43	0.35	91.10	11.46
A3563	2984	40.72	19.75	4.22	0.35	91.40	11.46
A3741	3113	37.92	24.08	6.57	1.02	94.20	11.50
A3772	3138	56.27	18.15	3.40	0.60	80.10	10.78
A4027	3309	30.34	22.63	6.75	0.41	87.40	11.26
A4113	3376	28.45	24.56	6.54	1.04	97.10	11.46
A4507	3686	22.75	16.28	2.81	0.53	70.40	10.29
A4844	3947	72.76	21.88	5.26	0.93	92.20	11.60
A6447	28990	30.34	16.36	2.55	0.55	74.80	10.60
A6450	28993	19.72	14.18	1.95	0.42	64.00	10.08
A6460	29086	46.50	24.81	6.71	1.14	101.70	12.09

Table C-3 Actual and required moment resistance for Span 1
(positive moment per ft width of bridge)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip	
A3246	2746	0.78	171.50	133.37	22.57	3.99	638.07	11.26
A3390	2856	0.92	177.93	164.37	33.01	2.56	792.80	11.63
A3462	2908	0.90	116.80	105.43	16.08	1.27	522.68	10.96
A3562	2983	0.82	130.13	106.51	19.51	1.24	500.10	10.90
A3563	2984	0.62	130.13	81.05	9.86	0.81	411.20	10.66
A3741	3113	0.96	166.43	159.12	34.67	5.39	707.50	11.50
A3772	3138	0.67	105.37	70.57	10.54	1.84	325.60	10.43
A4027	3309	0.73	136.97	100.51	18.93	1.14	463.40	10.79
A4113	3376	0.93	176.47	164.73	35.84	5.68	734.40	11.54
A4507	3686	1.13	59.39	66.98	9.38	1.75	309.40	10.29
A4844	3947	0.93	152.77	142.69	27.69	4.90	651.70	11.32
A6447	28990	0.85	76.73	65.26	7.51	1.62	314.70	10.30
A6450	28993	0.74	64.08	47.34	4.42	0.95	228.30	9.89
A6460	29086	1.05	182.62	190.95	41.09	6.97	867.70	11.76

Table C-4 Actual and required moment resistance for Span 1-2
(negative moment over support)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3246	2746	0.94	170.29	160.05	49.02	8.67	584.17	11.92
A3390	2856	0.92	200.07	184.32	61.15	4.75	696.20	12.09
A3462	2908	0.89	135.90	121.55	35.80	2.83	481.00	11.60
A3562	2983	0.89	146.09	130.01	39.98	2.54	499.08	11.46
A3563	2984	0.74	145.46	107.99	29.11	2.38	445.38	11.46
A3741	3113	0.96	186.51	178.67	61.54	9.57	574.12	11.50
A3772	3138	0.81	95.51	77.52	18.28	3.20	307.10	10.78
A4027	3309	0.80	143.20	114.05	36.15	2.17	422.10	11.26
A4113	3376	1.07	168.70	179.99	60.75	9.63	612.50	11.96
A4507	3686	1.16	50.38	58.66	11.70	2.18	239.60	10.29
A4844	3947	0.98	147.15	144.46	42.04	7.44	535.30	11.60
A6447	28990	1.07	60.93	65.32	12.46	2.68	276.90	10.60
A6450	28993	0.87	52.07	45.33	7.33	1.58	196.10	10.15
A6460	29086	1.07	198.48	212.96	67.64	11.47	768.40	12.09

Table C-5 Actual and required moment resistance for Span 2 (positive moment)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3246	2746	0.85	171.50	145.63	28.59	5.05	696.77	11.92
A3390	2856	1.02	152.77	156.17	31.93	2.48	777.58	12.09
A3462	2908	0.97	116.80	113.70	21.16	1.67	561.75	11.60
A3562	2983	0.87	130.13	113.01	22.48	1.43	541.90	11.46
A3563	2984	0.79	130.13	102.57	19.38	1.59	497.40	11.46
A3741	3113	0.97	166.43	161.50	34.67	5.39	723.10	11.50
A3772	3138	0.67	105.37	70.31	8.94	1.56	349.70	10.78
A4027	3309	0.73	136.97	99.33	19.33	1.16	472.50	11.26
A4113	3376	0.87	176.47	154.27	28.93	4.59	760.30	11.96
A4507	3686	0.81	59.39	48.33	2.94	0.55	257.60	10.29
A4844	3947	0.77	152.77	117.46	17.27	3.06	605.10	11.60
A6447	28990	0.79	76.73	60.41	5.82	1.25	310.50	10.60
A6450	28993	0.64	64.08	40.78	3.43	0.74	205.30	10.15
A6460	29086	0.92	182.62	167.11	31.11	5.27	831.20	12.09

Table C-6 Actual and required moment resistance for Span 2-3 (negative moment over support)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3246	2746	0.94	170.29	160.05	49.02	8.67	584.2	11.9
A3390	2856	0.82	200.07	164.09	49.13	3.81	669.9	12.1
A3462	2908	0.91	135.90	123.44	35.80	2.83	493.5	11.6
A3562	2983	0.84	146.09	122.37	39.98	2.54	449.1	11.5
A3563	2984	0.74	145.46	107.94	29.11	2.38	445.1	11.5
A3772	3138	0.81	95.51	77.72	18.28	3.20	308.3	10.8
A4027	3309	0.82	143.20	116.71	35.43	2.13	445.5	11.3
A4113	3376	1.08	168.70	181.44	60.75	9.63	622.4	12.0
A4507	3686	1.18	50.38	59.40	11.70	2.18	243.9	10.3
A4844	3947	0.98	147.15	144.54	42.04	7.44	535.8	11.6
A6447	28990	1.05	60.93	64.13	12.46	2.68	269.7	10.6
A6450	28993	0.87	52.07	45.19	7.33	1.58	195.3	10.2
A6460	29086	1.08	198.48	213.57	62.21	10.55	829.1	12.1

Table C-7 Actual and required moment resistance for Span 3 (positive moment)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3246	2746	0.77	171.50	131.60	22.57	3.99	626.7	11.3
A3390	2856	1.11	117.71	131.09	31.21	2.42	583.9	11.6
A3462	2908	0.90	116.80	105.65	16.08	1.27	524.0	11.0
A3562	2983	0.82	130.13	106.36	19.51	1.24	499.2	10.9
A3563	2984	0.62	130.13	81.08	9.86	0.81	411.4	10.7
A3772	3138	0.67	105.37	70.57	10.54	1.84	325.6	10.4
A4027	3309	0.73	136.97	99.87	16.15	0.97	503.3	11.3
A4113	3376	0.93	176.47	164.73	35.84	5.68	734.4	11.5
A4507	3686	1.04	59.39	61.71	9.38	1.75	278.4	10.3
A4844	3947	0.93	152.77	142.49	27.69	4.90	650.4	11.3
A6447	28990	0.86	76.73	66.22	7.51	1.62	320.3	10.3
A6450	28993	0.65	64.08	41.38	4.42	0.95	194.6	9.9
A6460	29086	0.92	182.62	168.12	31.11	5.27	838.2	12.1

Table C-8 Actual and required moment resistance for Span 3-4 (negative moment over support)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3390	2856	0.70	137.44	96.86	17.72	1.38	501.9	12.1
A4027	3309	0.89	143.20	127.43	42.63	2.56	452.4	11.3
A6460	29086	1.07	198.48	213.19	67.64	11.47	770.0	12.1

Table C-9 Actual and required moment resistance for Span 3-4 (negative moment over support)

Bridge #		Required/ Actual	Actual Strength	Required Strength	DC	DW	LL	mg
State	NBI		Neg Moment (k-ft)	Neg Moment (k-ft)	Kip-ft	Kip-ft	Kip	
A3390	2856	0.86	129.51	111.53	16.67	1.29	558.14	11.01
A4027	3309	0.93	136.97	127.81	29.30	1.76	562.60	11.12
A6460	29086	1.05	182.62	191.83	41.09	6.97	873.40	11.75

Table C-10 Representative bridges for concrete prestressed girder bridge

Type	Bridge #		Year	Spans	Length (ft)				
	State	NBI			Span 1	Span 2	Span 3	Span 4	Span 5
Girder	A4422	3612	1989	3	66.50	66.00	66.50	-	-
Girder	A4560	3732	1988	3	56.50	88.00	56.50	-	-
Girder	A4809	3922	1989	3	67.40	67.00	67.40	-	-
Girder	A5362	4241	1995	3	38.52	83.00	36.52	-	-
Girder	A5861	4417	1997	4	48.50	48.00	48.00	48.50	-
Girder	A7175	31310	2006	1	90.50	-	-	-	-
Girder	A6954	31534	2006	1	75.55	-	-	-	-
Girder	A6784	31798	2007	1	88.72	-	-	-	-
Girder	A4147	3393	1993	5	64.50	64.00	64.00	64.00	64.50
Girder	A4713	3851	1990	5	52.50	52.00	46.00	51.00	51.60
Girder	A6676	29626	2003	1	53.60	-	-	-	-
Girder	A4879	3973	1992	3	38.50	38.00	38.50	-	-
Girder	A4587	3756	1988	3	67.50	67.00	67.50	-	-
Girder	A5519	4315	1995	3	40.50	40.00	40.50	-	-
Girder	A3973	3276	1991	4	59.00	59.00	43.00	43.00	-
Girder	A4058	3332	1985	3	37.00	65.00	42.00	-	-
Girder	A4383	3583	1986	2	83.00	83.00	-	-	-
Girder	A4582	3753	1997	4	38.00	38.00	65.00	38.00	-
Girder	A5001	4045	1991	2	49.00	49.00	-	-	-
Girder	A5529	4322	1997	4	51.00	82.00	82.00	66.00	-
Girder	A5134	10086	1995	1	55.50	-	-	-	-
Girder	A3190	11821	1999	3	40.10	88.26	36.32	-	-
Girder	A5161	11893	1999	3	38.00	65.00	40.00	-	-
Girder	A5554	11933	2000	3	82.60	82.60	82.60	-	-
Girder	A4824	31246	2005	4	82.00	82.00	82.00	82.00	-
Girder	A6910	31613	2006	2	53.00	53.00	-	-	-
Girder	A4805	3919	1989	3	51.60	65.00	51.60		-
Girder	A4990	4034	1992	4	45.50	45.00	45.00	45.50	-
Girder	A5018	4052	1993	3	26.80	26.00	26.80	-	-
Girder	A6219	12121	2001	1	98.40	-	-	-	-
Girder	A6342	29057	2001	1	114.10	-	-	-	-
Girder	A6343	29060	2001	1	114.10	-	-	-	-
Girder	A5342	10164	1995	4	61.30	96.00	96.00	95.00	-
Girder	A5314	4206	1995	3	92.50	92.00	92.50	-	-
Tee	A5315	4207	1995	3	48.50	90.50	90.50	-	-
Girder	A6569	29023	2002	3	56.00	100.00	74.00	-	-
Girder	A6148	31413	2006	3	75.95	103.17	75.95	-	-

Table C-11 Prestressed girder actual and required moment resistance for negative moment

Bridge #		Bents		DW (kip-in)	LL (kip-in)	mg Factor	Required/Actual
		Actual	Required				
State	NBI	Neg Mom (kip-in)	Neg Mom (kip-in)				
A4422	3612	31742	19396	135.71	1130.20	0.71	0.61
A4560	3732	41248	24766	190.28	1311.20	0.78	0.60
A4809	3922	29793	18932	131.93	1153.50	0.68	0.64
A5362	4241	38659	21129	149.06	1114.60	0.79	0.55
A5861	4417	14883	11119	83.38	574.83	0.80	0.75
A4147	3393	23329	18485	137.21	1047.70	0.73	0.79
A4713	3851	13416	10448	91.31	562.60	0.75	0.78
A4879	3973	11505	8346	48.98	427.80	0.83	0.73
A4587	3756	31007	19886	139.82	1161.00	0.71	0.64
A5519	4315	11303	8844	53.01	462.30	0.81	0.78
A3973	3276	14886	13855	104.36	858.00	0.66	0.93
A4058	3332	11938	10815	31.06	766.20	0.64	0.91
A4383	3583	48620	32936	293.60	1787.00	0.74	0.68
A4582	3753	18555	13358	84.25	784.30	0.72	0.72
A5001	4045	16753	11255	92.70	605.00	0.75	0.67
A5529	4322	36190	35189	223.16	1466.60	1.01	0.97
A3190	11821	30204	19092	140.04	1219.90	0.65	0.63
A5161	11893	29808	15053	103.06	754.60	0.83	0.50
A5554	11933	35449	25197	184.88	1662.70	0.63	0.71
A4824	31246	39764	24681	189.44	1652.80	0.61	0.62
A6910	31613	20152	13777	114.78	720.40	0.77	0.68
A4805	3919	22302	15234	158.15	822.80	0.72	0.68
A4990	4034	10745	9826	68.07	535.60	0.76	0.91
A5018	4052	11303	5251	22.86	263.00	0.88	0.46
A5342	10164	49068	34760	277.87	2038.40	0.70	0.71
A5314	4206	63539	32377	263.08	1954.50	0.67	0.51
A5315	4207	45461	36239	72.28	2200.80	0.76	0.80
A6569	29023	52739	32318	261.17	1738.30	0.76	0.61
A6148	31413	38039	31175	269.03	1853.20	0.68	0.82

Table C-12 Prestressed girder actual and required moment resistance for Span 1

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Required/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	21678.70	39850.07	32238.02	772.79	109.66	1244.80	0.81
A4560	3732	26782.62	34352.03	27369.73	677.95	54.78	996.20	0.80
A4809	3922	20770.35	35913.22	31536.97	763.75	106.40	1265.50	0.88
A5362	4241	23869.80	29810.10	14513.65	303.81	7.01	594.00	0.49
A5861	4417	15115.10	23506.80	18135.86	313.14	60.67	737.90	0.77
A7175	31310	53769.00	73833.60	58800.69	1534.65	269.25	2445.60	0.80
A6954	31534	23683.90	34470.90	34347.74	737.02	145.55	1885.30	1.00
A6784	31798	59627.23	70104.45	62241.84	1625.41	200.72	2371.90	0.89
A4147	3393	21285.60	35919.90	30729.65	735.32	102.38	1168.20	0.86
A4713	3851	15532.80	24965.80	21292.99	464.08	65.69	840.20	0.85
A6676	29626	15197.60	26320.80	23191.49	420.17	90.14	1136.50	0.88
A4879	3973	9991.90	15661.30	13957.91	249.94	39.63	544.20	0.89
A4587	3756	22233.90	39581.90	32928.01	795.07	113.00	1267.90	0.83
A5519	4315	11037.60	18680.00	15018.78	272.90	43.16	594.60	0.80
A3973	3276	16139.60	28325.80	23251.47	536.94	71.52	996.50	0.82
A4058	3332	12960.90	16335.70	10468.64	195.94	5.36	555.30	0.64
A4383	3583	44654.10	61480.50	51332.30	1488.89	165.21	1681.90	0.83
A4582	3753	9787.50	15469.60	12293.07	223.10	35.12	550.70	0.79
A5001	4045	14075.30	21683.00	19030.90	404.27	52.22	759.00	0.88
A5529	4322	27690.80	35951.10	29228.00	622.29	51.32	892.50	0.81
A5134	10086	14883.30	25112.70	21195.57	326.12	86.63	1193.50	0.84
A3190	11821	20583.10	26423.40	12944.58	288.44	5.37	627.30	0.49
A5161	11893	12909.90	22347.10	14314.56	263.17	20.35	571.80	0.64
A5554	11933	51615.30	62315.00	44272.20	1306.56	147.91	1673.50	0.71
A4824	31246	29411.50	55244.80	39145.29	1031.29	136.47	1648.80	0.71
A6910	31613	18435.70	31762.40	22845.28	492.98	64.62	878.90	0.72
A4805	3919	15677.20	21626.70	20930.95	431.00	83.59	860.90	0.97
A4990	4034	10858.80	18462.70	16660.29	331.24	49.56	672.50	0.90
A5018	4052	6390.20	10329.50	7900.76	119.05	19.03	313.70	0.76
A6219	12121	95255.40	108465.00	82943.28	2200.36	410.30	3011.80	0.76
A6342	29057	109416.20	109416.20	105640.34	3075.69	589.10	3395.40	0.97
A6343	29060	109559.00	109273.40	105737.98	3082.20	589.10	3395.40	0.97
A5342	10164	33313.00	39143.60	29819.66	800.39	57.99	1148.70	0.76
A5314	4206	76706.20	79018.90	60002.99	1853.50	212.00	2007.00	0.76
A5315	4207	34531.00	42084.80	23860.80	523.38	58.31	942.40	0.57
A6569	29023	33193.20	39123.70	27051.43	682.08	40.25	1013.10	0.69
A6148	31413	49313.30	57251.10	43773.43	1306.54	122.11	1546.70	0.76

Table C-13 Prestressed girder actual and required moment resistance for Span 2

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Required/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	21678.70	39850.07	25962.11	761.21	4.30	964.43	0.65
A4560	3732	44654.10	61283.59	46504.38	1644.63	4.63	1336.40	0.76
A4809	3922	20770.35	35913.22	25433.73	754.71	4.06	978.10	0.71
A5362	4241	50163.00	61251.90	40056.71	1410.52	4.30	1137.30	0.65
A5861	4417	15115.10	23506.80	14776.14	306.72	27.64	578.40	0.63
A4147	3393	21285.60	35919.90	26068.88	723.97	42.37	945.10	0.73
A4713	3851	15849.00	24965.81	17736.81	455.29	31.18	661.10	0.71
A4879	3973	8644.70	12522.50	11688.45	243.49	11.69	448.50	0.93
A4587	3756	22233.90	39581.90	26992.52	783.33	33.56	978.50	0.68
A5519	4315	11037.60	18680.00	12161.03	266.20	12.39	465.50	0.65
A3973	3276	16139.60	28325.80	19608.12	536.94	40.03	776.10	0.69
A4058	3332	20770.30	35259.40	20641.29	604.70	21.65	835.30	0.59
A4383	3583	44654.10	61480.50	51332.30	1488.89	165.21	1681.90	0.83
A4582	3753	9787.50	15469.60	11083.73	223.10	35.12	470.60	0.72
A5001	4045	14075.30	21683.00	19030.90	404.27	52.22	759.00	0.88
A5529	4322	70138.20	67181.10	52983.14	1608.72	120.03	1255.80	0.79
A3190	11821	53346.50	63065.90	39852.30	1397.30	110.33	1244.60	0.63
A5161	11893	21678.70	39175.60	27346.81	770.01	74.03	827.00	0.70
A5554	11933	51615.30	62315.00	38329.06	1306.56	46.23	1360.80	0.62
A4824	31246	29411.50	55244.80	33982.35	1031.29	64.28	1348.60	0.62
A6910	31613	18435.70	31762.40	22845.28	492.98	64.62	878.90	0.72
A4805	3919	19009.00	31294.30	25364.61	683.92	83.59	903.40	0.81
A4990	4034	10858.80	18462.70	14213.43	324.00	22.53	557.20	0.77
A5018	4052	6390.20	10329.50	6867.47	112.05	4.77	277.20	0.66
A5342	10164	54641.60	64957.30	55417.66	1963.01	131.11	1617.40	0.85
A5314	4206	76877.80	79018.90	51182.59	1833.51	63.84	1593.10	0.65
A5315	4207	79610.70	81500.80	52676.65	1822.33	17.40	1576.60	0.65
A6569	29023	72844.00	89847.60	61731.19	2175.00	161.62	1648.90	0.69
A6148	31413	86255.00	92602.00	63973.77	2410.88	158.06	1757.00	0.69

Table C-14 Prestressed girder actual and required moment resistance for Span 3

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Required/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	21678.70	39850.07	30167.90	772.79	16.41	1218.70	0.76
A4560	3732	26782.62	34352.03	26636.89	677.95	17.68	992.20	0.78
A4809	3922	20770.35	35913.22	29734.26	763.75	15.51	1253.90	0.83
A5362	4241	23869.80	29810.10	13597.90	273.08	16.41	556.30	0.46
A5861	4417	15115.10	23506.80	15012.06	306.72	27.64	592.50	0.64
A4147	3393	21285.60	35919.90	26001.63	723.97	59.69	920.30	0.72
A4713	3851	10820.10	18215.02	15118.47	356.28	25.86	594.80	0.83
A4879	3973	9991.90	15661.30	14156.83	249.94	38.56	556.70	0.90
A4587	3756	22233.90	39581.90	32928.01	795.07	113.00	1267.90	0.83
A5519	4315	11037.60	18680.00	14520.41	272.90	43.16	565.40	0.78
A3973	3276	12665.35	18199.70	12196.40	285.21	11.06	553.00	0.67
A4058	3332	12960.90	16335.70	12465.39	252.47	8.87	636.40	0.76
A4582	3753	18435.70	31696.80	23415.62	652.76	62.90	827.40	0.74
A5529	4322	70138.20	67181.10	53576.88	1608.72	103.66	1297.60	0.80
A3190	11821	20583.10	26423.40	11206.86	236.62	5.14	556.90	0.42
A5161	11893	12909.90	22347.10	15336.25	291.60	25.04	601.00	0.69
A5554	11933	51615.30	62315.00	44656.26	1306.56	147.91	1702.70	0.72
A4824	31246	29411.50	55244.80	34156.09	1031.29	64.28	1362.10	0.62
A4805	3919	15677.20	21626.70	20598.22	431.00	83.59	838.80	0.95
A4990	4034	10858.80	18462.70	13694.73	324.00	22.53	524.90	0.74
A5018	4052	6390.20	10329.50	8515.37	119.05	19.03	347.10	0.82
A5342	10164	54641.60	64957.30	55591.64	1963.01	88.62	1681.70	0.86
A5314	4206	76706.20	79018.90	59556.08	1853.50	212.00	1975.40	0.75
A5315	4207	79610.70	81500.80	61795.04	1822.33	57.07	2106.00	0.76
A6569	29023	56062.10	54866.00	42757.42	1191.03	108.53	1443.80	0.78
A6148	31413	49313.30	57251.10	44419.95	1306.54	122.11	1592.20	0.78
A5315	4207	79610.70	81500.80	52676.65	1822.33	17.40	1576.60	0.65
A6569	29023	72844.00	89847.60	61731.19	2175.00	161.62	1648.90	0.69
A6148	31413	86255.00	92602.00	63973.77	2410.88	158.06	1757.00	0.69

Table C-15 Prestressed girder actual and required moment resistance for Span 4

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Required/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A5861	4417	15115.10	23506.80	18229.56	313.14	60.67	743.50	0.78
A4147	3393	21285.60	35919.93	25781.50	723.97	42.37	926.30	0.72
A4713	3851	15849.00	24965.81	17344.11	437.94	29.73	654.30	0.69
A3973	3276	12665.35	18199.70	14157.29	285.21	52.68	639.80	0.78
A4582	3753	9787.50	15469.60	12150.07	223.10	15.85	564.20	0.79
A5529	4322	38276.90	49091.60	44435.37	1042.17	115.97	1256.90	0.91
A4824	31246	29411.50	55244.80	39576.43	1031.29	136.47	1682.30	0.72
A4990	4034	10858.80	18462.70	16475.61	331.24	49.56	661.00	0.89
A5342	10164	54641.60	64957.30	62392.61	1922.33	205.13	2045.70	0.96

Table C-16 Prestressed girder actual and required moment resistance for Span 5

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Required/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4147	3393	21285.60	35919.90	30729.65	735.32	102.38	1168.20	0.86
A4713	3851	15532.80	24965.80	21292.99	464.08	65.69	840.20	0.85

Table C-17 Representative bridges for concrete prestressed double tee bridge

Type	Bridge #		Year	Spans	Length (ft)				
	State	NBI			Span 1	Span 2	Span 3	Span 4	Span 5
Tee	A4340	3550	1986	3	49.6	49	49.6	-	-
Tee	A4373	3574	1985	4	49.5	49	49	49.5	-
Tee	A4612	3774	1989	4	49.5	49	49	49.5	-
Tee	A4403	10077	1985	2	48.5	48	-	-	-
Tee	A4051	3326	1983	5	41.4	41	41	41	41.4
Tee	A3786	3148	1983	3	37.1	36	37.1	-	-
Tee	A4504	3685	1985	3	36.6	36	36.6	-	-
Tee	A4529	3707	1987	6	37.6	37	37	37	37
Tee	A4593	3759	1989	11	30.5	30	50	33	33
Tee	A4316	3531	1985	3	50.5	50	50.5	-	-
Tee	A4290	3508	1984	4	37.61	37	37	37.61	-
Tee	A4433	3621	1986	3	21.42	21.42	21.42	-	-
Tee	A4291	3509	1986	4	48.5	48	35	35.5	-
Tee	A4559	3731	1990	3	50.58	50	50.58	-	-
Tee	A4365	3568	1985	3	43.5	49	43.5	-	-
Tee	A4055	3329	1984	3	50.25	49	50.25	-	-
Tee	A2880	2478	1988	3	27.69	27	27.69	-	-
Tee	A4283	3501	1985	3	22.7	22	22.7	-	-
Tee	A4099	3366	1982	3	25.63	25	25.63	-	-
Tee	A4528	3706	1984	1	50.04	-	-	-	-
Tee	A4867	3964	1989	1	51.83	-	-	-	-

Table C-18 Prestressed double tee actual and required moment resistance

Bridge #		Bents		DW (kip-in)	LL (kip-in)	Required/Actual
		Actual	Required			
State	NBI	Neg Mom (kip-in)	Neg Mom (kip-in)			
A4340	3550	11600	9669.9	64.47	600.3	0.83
A4373	3574	10006	8652.23	59.75	593.1	0.86
A4612	3774	10587	8815.46	61.31	593.1	0.83
A4403	10077	11361.05	9316.69	70.71	602	0.82
A4051	3326	10431.7	7337.84	20.51	468.7	0.70
A3786	3148	7320.8	6309.93	15.01	408.6	0.86
A4504	3685	9797.89	6671.99	34.95	403.8	0.68
A4529	3707	8452.03	6662.8	72.57	399.5	0.79
A4593	3759	11770.46	9226.7	79.86	523.5	0.78
A4316	3531	10642.18	8254.47	24.76	613.4	0.78
A4290	3508	9635.77	6324.22	16.19	420.9	0.66
A4433	3621	8004.43	3934.99	63.39	166.9	0.49
A4291	3509	12691.33	8786.92	29.97	570.6	0.69
A4559	3731	11770.46	9509.68	63.83	614.8	0.81
A4365	3568	11872.81	8527.45	54.25	537.3	0.72
A4055	3329	9487.9	8608.63	25.68	613.5	0.91
A2880	2478	6756.43	4269.64	17.18	279.8	0.63
A4283	3501	4737.7	3294.84	11.72	207	0.70
A4099	3366	4849.78	4202.84	16.82	246.9	0.87

Table C-19 Prestressed double tee actual and required moment resistance for Span 1

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Require/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	7417.4	18458.6	15630.69	247.77	52.44	773.9	0.85
A4560	3732	10461.4	20263	15861.7	363.25	43.44	753.9	0.78
A4809	3922	10461.4	17498	16191.68	372.44	44.58	753.9	0.93
A5362	4241	10490	22337.8	16267.36	369.6	40.48	748	0.73
A5861	4417	7094.6	14628.5	11874.93	176.32	15.2	602.4	0.81
A7175	31310	3745.5	10400.3	10019.79	144.7	12.5	515.8	0.96
A6954	31534	7140.6	14284.5	11554.9	230.07	28.62	507.1	0.81
A6784	31798	6992.9	12472.2	11306.91	209.59	12.53	592	0.91
A4147	3393	7197.12	11393.12	9365.25	151.51	13.78	460	0.82
A4713	3851	9821.8	21439	16229.46	378.07	20.08	801	0.76
A6676	29626	7176.4	14290.2	11178.21	230.39	11.85	523.9	0.78
A4879	3973	4947.7	8794.2	7091.12	80.58	51.14	296.4	0.81
A4587	3756	10490.8	22468.3	17121.79	413.11	20.76	730	0.76
A5519	4315	10940.8	22381	18113.87	416.69	51.89	803.7	0.81
A3973	3276	9686.3	17555.2	14515.29	310.8	36.02	655	0.83
A4058	3332	6953.3	19584.4	14404.19	232.62	21.24	793.2	0.74
A4383	3583	4947.7	8741.4	6564.12	113.67	14.22	325.2	0.75
A4582	3753	3754.45	5220.4	5336.92	83.35	9.78	262.5	1.02
A5001	4045	3146.45	6800.69	6548.5	93	13.91	310.4	0.96
A5529	4322	9027.36	20578.82	22061.08	476.39	76.06	1019.4	1.07
A5134	10086	18663	22853.2	22513.88	421.76	84.62	1083	0.99

Table C-20 Prestressed double tee actual moment resistance for Span 2

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Require/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	7417.40	18458.60	12563.39	241.81	15.05	611.30	0.68
A4560	3732	10461.40	20263.00	13427.16	355.95	19.81	605.20	0.66
A4809	3922	10461.40	17498.00	13709.36	364.95	20.33	605.20	0.78
A5362	4241	10490.00	22337.80	16129.15	362.02	39.12	748.00	0.72
A5861	4417	7094.60	14628.50	9645.20	172.93	6.48	466.40	0.66
A7175	31310	3745.50	10400.30	7980.22	136.24	3.14	397.80	0.77
A6954	31534	7140.60	14284.50	9386.05	222.59	7.98	394.50	0.66
A6784	31798	6992.90	12472.20	9153.76	202.95	6.89	446.40	0.73
A4147	3393	7408.00	11393.12	7712.67	146.59	7.59	361.40	0.68
A4713	3851	9821.80	21439.00	13868.21	370.63	5.86	644.40	0.65
A6676	29626	7176.40	14290.20	9667.83	222.98	5.30	434.50	0.68
A4879	3973	4947.70	8794.20	5388.93	80.58	15.26	233.30	0.61
A4587	3756	10490.80	22468.30	14337.61	404.64	11.26	558.00	0.64
A5519	4315	10940.80	22381.00	15157.27	407.19	14.92	645.70	0.68
A3973	3276	10490.80	22391.30	14822.12	394.36	22.00	605.60	0.66
A4058	3332	6953.30	19584.40	11646.23	221.19	5.53	619.70	0.59
A4383	3583	4947.70	8741.40	5481.36	108.07	3.71	268.00	0.63
A4582	3753	3754.45	5220.40	3807.32	78.29	2.43	173.80	0.73
A5001	4045	3146.45	6800.69	5484.49	88.52	3.64	259.00	0.81

Table C-21 Prestressed double tee actual and required moment resistance for Span 3

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Require/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Pos Mom (kip-in)				
A4422	3612	7417.40	18458.60	15559.82	247.77	52.44	768.90	0.84
A4560	3732	10461.40	20263.00	13324.96	355.95	19.81	597.20	0.66
A4809	3922	10461.40	17498.00	13605.34	364.95	20.33	597.20	0.78
A5861	4417	7094.60	14628.50	9654.40	172.93	8.98	464.00	0.66
A7175	31310	3745.50	10400.30	9744.85	144.70	12.50	497.20	0.94
A6954	31534	7140.60	14284.50	11067.04	230.07	28.62	474.50	0.77
A6784	31798	6992.90	12472.20	9862.36	202.95	24.73	475.30	0.79
A4147	3393	11099.26	22380.98	16371.82	407.19	27.22	656.90	0.73
A4713	3851	9821.80	21439.00	16229.46	378.07	20.08	801.00	0.76
A6676	29626	7176.40	14290.20	9242.14	222.98	5.30	404.80	0.65
A4879	3973	4947.70	8794.20	7346.15	80.58	50.05	312.80	0.84
A4587	3756	9128.80	13418.50	8913.46	215.14	3.06	389.60	0.66
A5519	4315	10940.80	22381.00	18168.27	416.69	51.89	807.70	0.81
A3973	3276	9686.30	17555.20	14515.29	310.80	36.02	655.00	0.83
A4058	3332	6953.30	19584.40	14365.68	232.62	21.24	790.30	0.73
A4383	3583	4947.70	8741.40	6981.69	113.67	14.22	354.70	0.80
A4582	3753	3754.45	5220.40	5336.92	83.35	9.78	262.50	1.02
A5001	4045	3146.45	6800.69	6659.07	93.00	13.91	317.40	0.98

Table C-22 Prestressed double tee actual and required moment resistance for Span 4

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Require/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Mom (kip-in)				
A4560	3732	10461.40	20263.00	16141.47	363.25	43.44	775.80	0.80
A4809	3922	10461.40	17498.00	16476.43	372.44	44.58	775.80	0.94
A5861	4417	7094.60	14628.50	9579.78	172.93	6.48	462.00	0.65
A6784	31798	6992.90	12472.20	9647.60	202.95	12.80	475.30	0.77
A4147	3393	7768.86	11393.12	9175.28	177.37	14.09	420.80	0.81
A6676	29626	7176.40	14290.20	10915.91	230.39	11.85	505.60	0.76
A4587	3756	9128.80	13418.50	10619.89	221.33	11.84	490.30	0.79

Table C-23 Prestressed double tee actual and required moment resistance for Span 5

Bridge #		Actual	Actual	Required	DC (kip-in)	DW (kip-in)	LL (kip-in)	Require/ Actual
State	NBI	Pos Mom non-com (k-in)	Pos Mom com (k-in)	Mom (kip-in)				
A5861	4417	7094.60	14628.50	11822.89	176.32	15.20	598.90	0.81
A6784	31798	6992.90	12472.20	9957.16	202.95	30.00	475.30	0.80
A4147	3393	7768.86	11393.12	22095.57	1038.72	14.09	420.80	

Table C-24 Representative bridges for steel

Span	Type	Bridge #		Year	Length (ft)				
		State	NBI		Spans	Span 1	Span 2	Span 3	Span 4
Simple	Plate	A3961	3267	1984	1	112	-	-	-
Simple	Plate	A4376	3576	1988	1	120	-	-	-
Continuous	Plate	A4802	3917	1989	3	82.8	106.1	81.93	-
Continuous	Plate	A4999	4038	1995	3	59.5	119	55.46	-
Continuous	Plate	A6477	29559	2003	3	110	190	110	-
Simple	Plate	A5170	11994	1998	1	120	-	-	-
Simple	Plate	A6682	28962	2003	1	126	-	-	-
Simple	Plate	A6683	28968	2003	1	126	-	-	-
Simple	Plate	A6570	30894	2005	1	88.6	-	-	-
Continuous	Plate	A6723	31528	2006	3	90	200	90	-
Continuous	Plate	A6724	31529	2006	3	90	200	90	-
Continuous	Plate	A5731	11761	1996	1	211			
Continuous	Plate	A5169	11993	1998	1	120			
Continuous	Plate	A3101	2664	1987	2	120	120	-	-
Continuous	Plate	A3386	2852	1981	3	75	97	75	-
Continuous	Plate	A3848	3190	1983	3	58	79	66	-
Continuous	Plate	A3869	3204	1983	3	95	120	103	-
Continuous	Plate	A4840	3945	1984	2	138	141	-	-
Continuous	Plate	A4999	4043	1991	3	58	119	54	-
Continuous	Plate	A6168	12073	1999	2	190	196		
Continuous	Plate	A6248	12126	2001	3	120	179	132	-
Continuous	Plate	A7566	31756	2005	6	52	90	110	110
Continuous	Plate	A6480	31812	2007	4	95	85	90	85
Simple	W30x173	A6485	29592	2003	1	66	-	-	-
Has steel deck	W24x68	A7014	29665	1980	1	40.8	-	-	-
Continuous	W30x173	A7300	31500	2006	2	64.75	64.75	-	-
Continuous	W24x250	A5822	4412	1997	3	70	120	70	-

Table C-25 Steel actual and required moment resistance for Span 1 (positive moment per girder)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A3961	3267	9385	6476.67	1707.55	462.56	3312.70	0.63	0.69
A4376	3576	13366	7297.38	2070.00	525.60	3631.50	0.62	0.55
A4802	3917	4891	3092.93	556.45	152.26	1723.40	0.72	0.63
A4994	4038	7619	1519.79	94.92	25.12	1099.10	0.71	0.20
A6477	29559	11750	4344.78	765.01	167.81	2708.50	0.66	0.37
A5170	11994	14504	8021.25	2419.20	599.40	3631.50	0.68	0.55
A6682	28962	19167	9277.32	2877.53	676.71	3898.50	0.68	0.48
A6683	28968	19167	9277.32	2877.53	676.71	3898.50	0.68	0.48
A6570	30894	12278	4744.39	1205.95	258.07	2367.90	0.69	0.39
A6723	31528	13856	4941.74	1494.96	285.53	2093.40	0.72	0.36
A6724	31529	13929	4941.74	1494.96	285.53	2093.40	0.72	0.35
A5169	11993	49101	23102.90	9516.36	1563.80	8238.00	0.61	0.47
A3101	2664	15107	8421.29	2538.00	523.80	3631.00	0.70	0.56
A3386	2852	4429	4472.01	952.69	226.43	2886.00	0.58	1.01
A3848	3190	9284	3176.40	690.75	120.60	1487.90	0.82	0.34
A3869	3204	4388	1931.39	273.11	66.76	1026.40	0.83	0.44
A4840	3945	5175	3650.29	692.52	164.61	2134.90	0.68	0.71
A4999	4043	10683	6321.67	1546.60	339.90	3532.30	0.63	0.59
A6168	12073	8455	1446.53	78.90	17.38	1060.70	0.71	0.17
A7566	31756	18648	11137.77	3560.38	658.94	5625.00	0.58	0.60
A6480	31812	15447	4633.59	926.80	190.10	3039.00	0.60	0.30
A6485	29592	6423	1613.47	166.80	38.78	972.00	0.79	0.25
A7014	29665	5751	3178.41	516.31	116.73	2037.00	0.66	0.55
A7300	31500	4451	955.32	451.16	114.35	1675.00	0.08	0.21
A5822	4412	1070	641.45	58.68	23.06	790.80	0.39	0.60
A4256	3475	4005	2046.19	320.23	71.90	1305.00	0.67	0.51
A3961	3267	4011	1163.54	126.80	20.20	1550.00	0.36	0.29
A4376	3576	1219	392.28	23.70	5.80	266.30	0.76	0.32

Table C-26 Steel actual and required moment resistance for Span 1-2

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Neg Mom (k-ft)	Neg Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A4802	3917	5104.00	4312.73	1100.60	301.16	1974.70	0.72	0.84
A4994	4038	5668.00	4239.34	1123.41	297.31	1925.90	0.71	0.75
A6477	29559	18280.00	10318.08	3531.27	774.62	4094.60	0.66	0.56
A6723	31528	16785.00	12104.46	4131.25	789.04	4556.70	0.72	0.72
A6724	31529	16785.00	12104.46	4131.25	789.04	4556.70	0.72	0.72
A3386	2852	6113	5904.96	1900.31	398.03	2877.2	0.58	0.97
A3848	3190	8139	4654.97	1459.30	242.70	1721.5	0.82	0.57
A3869	3204	4306	2690.50	604.37	144.18	1184.1	0.83	0.62
A4840	3945	7145	4958.80	1372.60	301.35	2348	0.68	0.69
A4999	4043	10991	8964.05	3205.73	626.86	3658	0.63	0.82
A6168	12073	5907	4145.85	1120.28	246.75	1906.1	0.71	0.70
A7566	31756	26809	15653.34	6378.40	1180.48	5833	0.58	0.58
A6480	31812	12013	8433.02	2889.80	549.80	3807	0.60	0.70
A6485	29592	3843	2874.56	591.60	137.55	1392	0.79	0.75
A7014	29665	5497	3865.01	1021.20	230.89	1937	0.66	0.70
A4256	3475	3258	2384.96	521.80	117.20	1321	0.67	0.73
A3961	3267	3336	2434.87	596.30	95.00	2460	0.36	0.73
A4376	3576	1034	306.97	37.00	9.00	186	0.76	0.30

Table C-27 Steel actual and required moment resistance for Span 2 (positive moment per girder)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A4802	3917	5197.00	3350.96	603.00	165.00	1867.00	0.72	0.64
A4994	4038	5918.00	3886.56	874.56	231.46	1971.90	0.71	0.66
A6477	29559	25892.00	8707.69	2546.03	558.50	4047.40	0.66	0.34
A6723	31528	12424.00	10211.13	3251.25	620.96	4128.30	0.72	0.82
A6724	31529	12424.00	10211.13	3251.25	620.96	4128.30	0.72	0.82
A3386	2852	4429	4450.81	952.69	226.43	2865.20	0.58	1.00
A3848	3190	9666	3493.14	773.50	134.70	1622.00	0.82	0.36
A3869	3204	5328	2312.19	344.98	83.55	1209.50	0.83	0.43
A4840	3945	5481	3723.93	653.54	154.98	2250.00	0.68	0.68
A4999	4043	10683	6588.89	1661.61	365.18	3610.20	0.63	0.62
A6168	12073	6312	3790.34	835.20	192.90	1971.60	0.71	0.60
A7566	31756	18648	11137.77	3560.38	658.94	5625	0.58	0.60
A6480	31812	14041	6816.19	1784.10	362.04	3851.70	0.60	0.49
A6485	29592	7791	2763.08	409.20	95.14	1522	0.79	0.35
A7014	29665	5751	2576.83	559.50	126.50	1458	0.66	0.45
A4256	3475	4005	2046.19	320.23	71.90	1305.00	0.67	0.51
A3961	3267	4011	1450.18	196.40	31.30	1841.00	0.36	0.36
A4376	3576	1209	439.07	30.40	7.40	293.40	0.76	0.36

Table C-28 Steel actual and required moment resistance for Span 2-3

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Neg Mom (k-ft)	Neg Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A4802	3917	5110.00	4273.89	1087.81	297.66	1960.70	0.72	0.84
A4994	4038	5671.00	4204.23	1113.84	294.78	1910.30	0.71	0.74
A6477	29559	18343.00	9785.96	3199.41	701.82	4087.60	0.66	0.53
A6723	31528	16785.00	12100.41	4131.25	789.04	4553.50	0.72	0.72
A6724	31529	16785.00	12100.41	4131.25	789.04	4553.50	0.72	0.72
A3848	3190	8139	4665.29	1459.30	242.70	1728.7	0.82	0.57
A3869	3204	5707	3032.73	721.71	166.75	1295.5	0.83	0.53
A4840	3945	7058	5320.35	1520.99	333.93	2455	0.68	0.75
A6168	12073	5911	4120.38	1113.08	245.16	1894.8	0.71	0.70
A6480	31812	12000	8949.20	3129.03	595.33	3948.8	0.60	0.75
A6485	29592	6926	4556.03	1095.60	254.73	2024	0.79	0.66
A7014	29665	5926	3846.15	1009.34	228.21	1937	0.66	0.65
A3961	3267	3336	2434.87	596.30	95.00	2460	0.36	0.73
A4376	3576	1027	346.95	45.00	11.00	206.3	0.76	0.34

Table C-29 Steel actual and required moment resistance for Span 3 (positive moment per girder)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A4802	3917	4904.00	3035.33	541.01	148.04	1698.00	0.72	0.62
A4994	4038	7619.00	1367.30	55.51	14.69	1028.50	0.71	0.18
A6477	29559	11679.00	4523.76	866.63	190.10	2724.50	0.66	0.39
A6723	31528	13856.00	2842.881	142.93	27.30	2076.40	0.72	0.21
A6724	31529	13856.00	2842.88	142.93	27.30	2076.40	0.72	0.21
A3848	3190	9284	3198.39	690.80	120.60	1503.20	0.82	0.34
A3869	3204	7149	2432.30	395.33	93.80	1238.30	0.83	0.34
A4840	3945	4767	4157.17	844.14	201.13	2355.80	0.68	0.87
A6168	12073	8477	1270.06	42.43	9.35	965.34	0.71	0.15
A6480	31812	15229	5488.35	1239.04	254.12	3390.00	0.60	0.36
A6485	29592	8405	3836.17	669.60	155.68	1996	0.79	0.46
A7014	29665	6223	2589.61	501.98	113.50	1548	0.66	0.42
A3961	3267	4011	1272.48	124.70	95.00	1549.00	0.36	0.32
A4376	3576	1198	503.13	38.00	9.30	332.30	0.76	0.42

Table C-30 Steel actual and required moment resistance for Span 3-4 (negative moment over support)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A6485	29592	8703	5409.98	1274.40	296.30	2434	0.79	0.62
A7014	29665	5926	3702.62	1009.34	228.21	1813	0.66	0.62

Table C-31 Steel actual and required moment resistance for Span 4 (positive moment per girder)

Bridge #		Actual Strength	Required Strength	DC	DW	LL	mg	Required/ Actual
State	NBI	Pos Mom (k-ft)	Pos Mom (k-ft)	Kip-ft	Kip-ft	Kip-ft		
A6485	29592	8726	3822.73	661.20	153.73	1996	0.79	0.44
A7014	29665	6233	2813.11	501.31	113.34	1742	0.66	0.45

Table C-32 Bias and COV for reinforced concrete bridges

Bridge #		Spans	Span 1		Span 2		Span 3		Span 4	
State	NBI		Bias	COV	Bias	COV	Bias	COV	Bias	COV
A3246	2746	3	1.154	0.065	1.154	0.065	1.154	0.065		
A3390	2856	4	1.15	0.064	1.15	0.065	1.149	0.065	1.14	0.06
A3462	2908	3	1.154	0.068	1.148	0.068	1.148	0.068		
A3562	2983	3	1.151	0.065	1.151	0.066	1.149	0.065		
A3563	2984	3	1.15	0.066	1.15	0.065	1.149	0.065		
A3741	3113	2	1.158	0.074	1.155	0.072				
A3772	3138	3	1.151	0.073	1.151	0.074	1.153	0.075		
A4027	3309	4	1.152	0.069	1.153	0.068	1.153	0.068	1.15	0.07
A4113	3376	3	1.151	0.063	1.151	0.062	1.151	0.062		
A4507	3686	3	1.151	0.081	1.15	0.081	1.149	0.08		
A4844	3947	3	1.152	0.063	1.151	0.064	1.151	0.064		
A6447	28990	3	1.151	0.094	1.151	0.095	1.152	0.094		
A6450	28993	3	1.15	0.091	1.149	0.091	1.153	0.091		
A6460	29086	4	1.149	0.062	1.149	0.062	1.15	0.062	1.15	0.06

Span 1-2		Span 2-3		Span 3-4	
Bias	COV	Bias	COV	Bias	COV
1.154	0.067	1.154	0.068		
1.154	0.068	1.154	0.068	1.2	0.07
1.153	0.072	1.153	0.072		
1.151	0.066	1.15	0.066		
1.15	0.069	1.151	0.069		
1.164	0.079				
1.156	0.087	1.155	0.086		
1.155	0.074	1.154	0.073	1.2	0.07
1.151	0.067	1.15	0.066		
1.157	0.093	1.158	0.093		
1.153	0.067	1.154	0.066		
1.154	0.114	1.154	0.114		
1.148	0.103	1.148	0.103		
1.153	0.066	1.153	0.065	1.2	0.07

Table C-33 Bias and COV for Prestressed Girder Bridges

Bridge #		Negative moment		Span 1		Span 2		Span 3		Span 4	
State	NBI	Bias	COV	Bias	COV	Bias	COV	Bias	COV	Bias	COV
A4422	3612	1.144	0.054	1.043	0.031	1.043	0.031	1.043	0.031		
A4560	3732	1.145	0.054	1.045	0.03	1.045	0.029	1.045	0.03		
A4809	3922	1.145	0.055	1.043	0.03	1.043	0.03	1.043	0.03		
A5362	4241	1.145	0.054	1.045	0.03	1.044	0.029	1.045	0.03		
A5861	4417	1.146	0.057	1.045	0.033	1.045	0.033	1.045	0.033	1.045	0.033
A7175	31310			1.044	0.029						
A6954	31534			1.037	0.03						
A6784	31798			1.043	0.029						
A4147	3393	1.168	0.053	1.044	0.032	1.043	0.03	1.045	0.031	1.046	0.032
A4713	3851	1.168	0.055	1.045	0.032	1.044	0.031	1.043	0.033	1.047	0.032
A6676	29626			1.043	0.033						
A4879	3973	1.191	0.057	1.047	0.033	1.045	0.034	1.043	0.034		
A4587	3756	1.165	0.054	1.046	0.031	1.045	0.031	1.045	0.03		
A5519	4315	1.188	0.058	1.045	0.033	1.049	0.033	1.044	0.033		
A3973	3276	1.144	0.059	1.045	0.033	1.045	0.031	1.044	0.033	1.045	0.031
A4058	3332	1.142	0.057	1.045	0.03	1.044	0.03	1.045	0.031		
A4383	3583	1.149	0.055	1.045	0.028	1.046	0.03				
A4582	3753	1.148	0.055	1.046	0.034	1.045	0.032	1.046	0.031		
A5001	4045	1.142	0.057	1.045	0.033	1.044	0.03				
A5529	4322	1.144	0.052	1.035	0.03	1.032	0.029	1.031	0.028		
A5134	10086			1.044	0.034						
A3190	11821	1.144	0.056	1.044	0.03	1.045	0.026	1.044	0.031		
A5161	11893	1.145	0.054	1.044	0.034	1.046	0.031	1.046	0.033		
A5554	11933	1.143	0.054	1.046	0.029	1.044	0.028	1.044	0.029		
A4824	31246	1.145	0.055	1.042	0.03	1.042	0.03	1.042	0.03	1.042	0.03
A6910	31613	1.146	0.056	1.044	0.032	1.044	0.031				
A4805	3919	1.173	0.052	1.044	0.032	1.043	0.031	1.046	0.032		
A4990	4034	1.194	0.058	1.048	0.034	1.045	0.033	1.045	0.034	1.046	0.033
A5018	4052	1.173	0.058	1.045	0.037	1.047	0.036	1.047	0.037		
A6219	12121			1.044	0.027						
A6342	29057			1.042	0.028						
A6343	29060			1.042	0.027						
A5342	10164	1.144	0.053	1.045	0.029	1.045	0.029	1.045	0.028	1.045	0.028
A5314	4206	1.165	0.052	1.045	0.027	1.044	0.029	1.042	0.027		
A5315	4207	1.173	0.054	1.044	0.03	1.044	0.028	1.044	0.027		
A6569	29023	1.167	0.052	1.046	0.03	1.044	0.028	1.044	0.029		
A6148	31413	1.159	0.055	1.045	0.028	1.044	0.028	1.044	0.029		

Table C-34 Bias and COV for Prestressed Double Tee Bridges

Bridge #		Negative moment		Span 1		Span 2		Span 3		Span 4		Span 5	
State	NBI	Bias	COV	Bias	COV	Bias	COV	Bias	COV	Bias	COV	Bias	COV
A4340	3550	1.145	0.057	1.05	0.037	1.05	0.037	1.05	0.037				
A4373	3574	1.145	0.057	1.045	0.034	1.045	0.034	1.045	0.034				
A4612	3774	1.147	0.057	1.044	0.034	1.044	0.034	1.044	0.034	1.044	0.034		
A4403	10077	1.146	0.059	1.045	0.036	1.045	0.034						
A4051	3326	1.146	0.058	1.045	0.037	1.045	0.037	1.045	0.037	1.047	0.038	1.045	0.038
A3786	3148	1.146	0.062	1.046	0.046	1.043	0.045	1.044	0.044				
A4504	3685	1.145	0.057	1.044	0.036	1.047	0.037	1.044	0.038				
A4529	3707	1.141	0.058	1.044	0.036	1.045	0.036	1.045	0.036	1.044	0.037	1.046	0.038
A4593	3759	1.145	0.054	1.047	0.036	1.044	0.035	1.045	0.035	1.044	0.035	1.044	0.05
A4316	3531	1.143	0.054	1.043	0.036	1.043	0.035	1.045	0.035				
A4290	3508	1.141	0.055	1.044	0.037	1.047	0.038	1.045	0.038	1.044	0.057		
A4433	3621	1.144	0.061	1.045	0.04	1.047	0.039	1.046	0.039				
A4291	3509	1.145	0.055	1.045	0.034	1.045	0.034	1.045	0.034	1.044	0.034		
A4559	3731	1.145	0.057	1.046	0.034	1.044	0.035	1.045	0.036				
A4365	3568	1.142	0.056	1.045	0.034	1.044	0.035	1.046	0.035				
A4055	3329	1.148	0.057	1.046	0.038	1.047	0.039	1.047	0.039				
A2880	2478	1.142	0.061	1.044	0.039	1.046	0.039	1.044	0.04				
A4283	3501	1.142	0.066	1.042	0.046	1.043	0.045	1.046	0.047				
A4099	3366	1.146	0.067	1.045	0.045	1.043	0.046	1.043	0.045				
A4528	3706			1.044	0.032								
A4867	3964			1.044	0.035								

Table C-35 Bias and COV for Steel Bridges

Bridge #		Span 1		Span 2		Span 3		Span 1-2		Span 2-3	
State	NBI	COV	Bias	Bias	COV	Bias	COV	Bias	COV	Bias	COV
A3961	3267	1.19	0.048								
A4376	3576	1.17	0.046								
A4802	3917	1.15	0.05	1.22	0.07	1.14	0.049	1.15	0.05	1.15	0.05
A4999	4038	1.15	0.04	1.22	0.071	1.15	0.041	1.15	0.053	1.15	0.053
A6477	29559	1.16	0.058	1.17	0.065	1.14	0.058	1.23	0.051	1.23	0.05
A5170	11994	1.18	0.052								
A6682	28962	1.21	0.046								
A6683	28968	1.21	0.046								
A6570	30894	1.18	0.049								
A6723	31528	1.15	0.056	1.17	0.063	1.15	0.056	1.24	0.057	1.24	0.057
A6724	31529	1.15	0.056	1.17	0.06	1.15	0.056	1.24	0.057	1.24	0.057
A5731	11761	1.19	0.057								
A5169	11993	1.21	0.047								
A3101	2664	1.19	0.064	1.18	0.064			1.22	0.051		
A3386	2852	1.21	0.063	1.21	0.062	1.21	0.064	1.2	0.061	1.2	0.062
A3848	3190	1.17	0.061	1.16	0.061	1.15	0.055	1.16	0.061	1.14	0.046
A3869	3204	1.17	0.061	1.17	0.06	1.18	0.064	1.21	0.076	1.22	0.078
A4840	3945	1.18	0.052	1.18	0.051			1.24	0.059		
A4999	4043	1.15	0.053	1.17	0.062	1.15	0.052	1.16	0.058	1.16	0.059
A6168	12073	1.16		1.16	0.057			1.23	0.052		
A6248	12126	1.16	0.058	1.16	0.06	1.16	0.058	1.19	0.064	1.19	0.065
A7566	31756	1.17	0.063	1.23	0.072	1.21	0.063	1.16	0.06	1.19	0.05
A6480	31812	1.17	0.063	1.17	0.063	1.16	0.059	1.17	0.047	1.15	0.043
A6485	29592	1.14	0.05								
A7014	29665	1.19	0.06								
A7300	31500	1.15	0.05	1.15	0.065			1.14	0.035		
A5822	4412	1.15	0.04	1.15	0.045	1.15	0.045	1.14	0.036	1.14	0.036
A4256	3475	1.18	0.06	1.15	0.067	1.14	0.067	1.14	0.041	1.15	0.041

APPENDIX D: RELIABILITY INDICES OF SELECTED BRIDGES

Table D-1 Reliability index for positive moment of steel girder bridges (ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
1	11877	7.10							
2	29592	7.17							
3	30894	6.86							
4	28962	7.35							
5	28968	6.93							
6	31528	6.76							
7	31529	6.81							
8	29665	6.38							
9	11993	7.07							
10	11761	6.85							
11	2664	7.91	7.70						
12	2692	7.77	8.45	7.65	7.54	7.41			
13	2852	7.89	7.88	7.86					
14	3190	7.27	7.77	7.65	7.65	7.56	7.52		
15	3204	8.03	8.19	7.81					
16	3475	6.25	6.72	6.67					
17	3945	7.72	7.53						
18	4043	7.87	7.26	7.68					
19	4412	7.20	6.73	7.25					
20	12073	7.46	7.32						
21	12126	7.57	7.05	7.76	8.64	7.45	7.51		
22	29559	8.67	7.42	8.63					
23	31812	7.34	8.51	8.11	8.75	7.84	7.74	8.03	7.16
24	31500	7.57	7.79						
25	31756	8.17	8.54	8.35	8.44	8.78	7.87		

Table D-2 Reliability index for positive moment of prestressed concrete bridges (Girder type, ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
26	3612	6.23							
27	3706	6.79							
28	3732	6.31							
29	3922	5.67							
30	4417	6.02							
31	31798	6.39							
32	31534	5.91							
33	31310	5.99							
34	3393	6.09							
35	3756	6.38							
36	3851	5.84							
37	3973	5.80							
38	4315	5.80							
39	29626	5.89							
40	11821	5.16	5.23	5.31					
41	3276	5.78	5.20	4.85	5.74				
42	3332	4.97	5.10	5.69					
43	3583	6.02	6.21						
44	3753	5.45	4.04	5.23	5.40				
45	31246	6.26	6.03	5.91	6.35				
46	4045	5.22	5.43						
47	10086	5.87							
48	11893	5.98	5.46	6.10					
49	4322	6.22	5.70	5.66	6.10				
50	11933	6.07	5.67	6.31					
51	31613	6.04	5.74						
52	3919	5.79	5.33	5.58					
53	4034	5.70	4.61	4.67	5.80				
54	4052	5.14	5.13	5.07					
55	3366	7.58	7.58						

Table D-3 Reliability index for positive moment of prestressed concrete bridges (Double tee beam type, ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
56	3499	6.79							
57	3964	6.44							
58	3148	5.21							
59	3326	5.13							
60	3550	6.74							
61	3574	6.47							
62	10077	5.47							
63	3685	6.18							
64	3707	5.95							
65	3759	6.01							
66	3774	6.04							
67	12121	6.41							
68	29057	5.94							
69	29060	5.74							
70	3501	5.04	5.17	5.22					
71	3548	6.03	5.60	5.43	6.02				
72	2478	5.72	6.06	5.48					
73	3329	5.48	4.08	6.08					
74	3508	6.15	5.44	5.22	6.19				
75	3509	5.97	5.50	5.37	6.41				
76	3531	5.81	4.19	6.25					
77	3568	5.15	4.67	5.84					
78	3621	5.19	5.32	5.07					
79	3731	6.11	4.73	6.45					
80	4206	6.12	5.53	5.77					
81	4207	6.17	5.52	5.96					
82	10164	6.10	5.45	5.38	5.60				
83	31413	6.60	5.48	6.18					
84	29023	6.49	5.32	6.23					

Table D-4 Reliability index for positive moment of concrete slab bridges (ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
85	2746	5.16	5.91	6.03					
86	2856	6.41	6.76	6.01	6.33				
87	2908	5.06	5.39	5.42					
88	2983	5.26	5.41	5.62					
89	3113	5.89	6.32						
90	3376	5.04	5.73	5.37					
91	3686	4.99	4.95	4.68					
92	3947	5.23	4.74	5.51					
93	28993	4.20	4.33	4.45					
94	29086	5.47	5.73	5.82	5.83				
95	29534	6.46	6.39						
96	2984	5.32	5.70	5.54					
97	3309	5.72	5.60	5.47	5.67				
98	28990	5.77	6.11	5.33					

Table D-5 Reliability index for shear force of steel girder bridges (ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
1	11877	3.83							
2	29592	4.09							
3	30894	3.98							
4	28962	4.21							
5	28968	4.19							
6	31528	4.54							
7	31529	4.47							
8	29665	4.16							
9	11993	4.05							
10	11761	4.37							
11	2664	4.51	3.97						
12	2692	4.18	4.12	3.83	3.93	3.74			
13	2852	4.48	4.26	4.43					
14	3190	5.31	4.85	5.26	5.06	5.11	5.04		
15	3204	4.62	4.50	4.50					
16	3475	4.41	3.98	4.34					
17	3945	4.46	4.19						
18	4043	4.73	3.92	4.50					
19	4412	3.82	3.20	3.68					
20	12073	4.64	4.38						
21	12126	4.05	4.00	4.04	4.28	4.02	3.88		
22	29559	4.86	4.48	4.65					
23	31812	4.01	4.10	4.04	4.34	3.86	4.00	3.95	3.73
24	31500	4.58	4.60						
25	31756	5.35	4.40	4.20	4.42	4.73	5.08		

Table D-6 Reliability index for shear force of prestressed concrete bridges (Girder type, ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
26	3612	4.14							
27	3706	4.31							
28	3732	3.87							
29	3922	3.92							
30	4417	4.45							
31	31798	4.08							
32	31534	3.70							
33	31310	3.90							
34	3393	4.19							
35	3756	4.10							
36	3851	4.36							
37	3973	4.36							
38	4315	4.73							
39	29626	4.24							
40	11821	4.47	3.74	4.15					
41	3276	4.11	4.04	4.12	4.25				
42	3332	4.47	3.84	4.00					
43	3583	4.29	4.15						
44	3753	4.79	4.18	4.12	4.07				
45	31246	4.11	4.23	4.16	3.95				
46	4045	4.94							
47	10086	3.80	3.80						
48	11893	4.89	4.37	4.68					
49	4322	5.05	4.27	4.31	4.45				
50	11933	4.29	4.34	4.10					
51	31613	4.63	4.41						
52	3919	4.66	4.38	4.62					
53	4034	5.08	4.73	4.67	4.49				
54	4052	4.19	3.85	3.98					
55	3366	2.69	2.69						

Table D-7 Reliability index for shear force of prestressed concrete bridges (Double tee beam type, ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
56	3499	4.66							
57	3964	4.68							
58	3148	3.90							
59	3326	4.11							
60	3550	4.49							
61	3574	4.47							
62	10077	4.15							
63	3685	4.05							
64	3707	4.13							
65	3759	4.30							
66	3774	4.44							
67	12121	4.08							
68	29057	3.87							
69	29060	3.80							
70	3501	4.20	4.03	3.81					
71	3548	4.96	4.24	4.23	4.33				
72	2478	4.34	4.02	4.16					
73	3329	5.04	4.63	4.48					
74	3508	5.16	4.38	4.39	4.52				
75	3509	4.89	4.69	4.22	4.17				
76	3531	4.98	4.46	4.57					
77	3568	4.60	4.23	4.20					
78	3621	4.06	3.97	4.17					
79	3731	4.32	4.18	4.03					
80	4206	4.27	4.07	4.01					
81	4207	4.63	4.05	4.10					
82	10164	4.34	3.71	3.80	3.73				
83	31413	4.28	4.04	4.17					
84	29023	4.83	3.96	4.44					

Table D-8 Reliability index for shear force of concrete slab bridges (ADTT = 5000)

Bridge Index	Bridge No. (NBI)	Span No.							
		1	2	3	4	5	6	7	8
85	2746	4.36	4.18	3.95					
86	2856	4.60	4.51	4.34	3.94				
87	2908	4.03	4.03	3.56					
88	2983	3.98	4.07	3.52					
89	3113	4.46	4.22						
90	3376	3.95	3.96	3.66					
91	3686	3.18	2.94	3.09					
92	3947	4.04	3.91	3.85					
93	28993	2.89	2.90	2.86					
94	29086	4.05	4.02	3.97	3.74				
95	29534	4.61	4.25						
96	2984	3.74	3.94	3.33					
97	3309	4.15	4.23	4.15	4.07				
98	28990	3.59	3.83	3.60					

Missouri Department of Transportation
Organizational Results
P. O. Box 270
Jefferson City, MO 65102

573.526.4335
1.888.ASK.MODOT
innovation@modot.mo.gov