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UPDATING THE CALIBRATION REPORT FOR AASHTO LRFD CODE

FINAL REPORT

Requested by:

American Association of State Highway
and Transportation Officials (AASHTO)

Highway Subcommittee on Bridge and Structures

Prepared by:

John M. Kulicki, Zolan Prucz, and Chad M. Clancy
Modjeski and Masters, Inc.
Harrisburg, Pennsylvania

Dennis R. Mertz
University of Delaware

Andrzej S. Nowak
University of Nebraska

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1.0 INTRODUCTION AND RESEARCH APPROACH

1.1 Background

The published calibration report from NCHRP 12-33 (NCHRP Report 368) was completed prior to the final selection of load and resistance factors used in the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD). While the published report bears a date on 1999, the report was actually written in 1991 and the 4^{th} draft of AASHTO LRFD was approved in 1993. The reliability analysis that serves as a basis for load and resistance factors included in the AASHTO LRFD was performed for the AASHTO Standard Specifications (1989 edition). As a result, NCHRP Report 368 provides the background information and the calibration procedure for the LRFD Specifications, but it does not correspond to the actual code provisions. Therefore, there is a need for updating the calibration report so that it can serve as a reference for the specifications.

1.2 Objective

The objective of this project is to fully document the strength limit state calibration of the AASHTO LRFD Bridge Design Specifications (LRFD). Reliability analyses for representative beam-slab bridges, including steel (noncomposite and composite girders), reinforced concrete T-beams, and prestressed concrete AASHTO-type girders shall be performed using the LRFD load and resistance factors.

Accomplishment of the project objective will require at least the following tasks:

1.3 Scope

Task 1:

Review the original calibration procedure and input data. Revise the statistical database so that it is consistent with the LRFD Specifications. For each random variable used (live load model and strength models) in the calibration provide:

- source of data;
- number of data points, normal mean unfiltered bias;
- normal COV of unfiltered bias; type of data filtering to remove outliers;
- number of data points, filtered; filtered normal mean bias;
- filtered normal COV of bias;
- type of statistical distribution;
- normal mean of bias used for calibration; and normal COV of bias used for calibration.

This information shall be assembled in a single table for convenient reference.

Develop an outline and criteria for a guide on data collection and documentation for data set analysis. This data set analysis approach (laboratory and field) should separate “Materials and Strengths” from “Workmanship and Construction Tolerance” issues for each bridge system.

***Task 2*:**

Select a representative suite of bridges (ASD, LFD, and LRFD designs) for reliability analysis. Bridge types shall include steel (noncomposite and composite girders), reinforced concrete T-beams, and prestressed concrete AASHTO type girders. Redundant and nonredundant bridge systems should be included.

***Task 3*:**

Prepare a detailed description of a “universal” reliability analysis procedure to be used in this and future studies. NOTE: The TRB Circular prepared by Allen, et. al. is an appropriate starting point for this description.

***Task 4*:**

Prepare and submit a letter report providing the results of Tasks 1 through 3 for NCHRP review. A conference call with the panel may be needed to discuss the letter report. Work on remaining tasks is not to proceed without NCHRP approval of the letter report.

***Task 5*:**

Perform the reliability analysis of the suite of bridges. Perform a sensitivity analysis illustrating the interdependency of the reliability index and load and resistance factors.

***Task 6*:**

Prepare a final report documenting the entire research effort.

This Letter Report and the attached Summary Report in Appendix I are submitted in satisfaction of Task 4.

2.0 REVIEW OF CALIBRATION DATABASE

2.1 Introduction

The original calibration of AASHTO LRFD was based on very condensed information available from the sources identified herein. The detailed information sought in this task is simply not available. It is possible to document what was used and is largely described in the original calibration report by Dr. Nowak (NCHRP 368), (referred to as the Calibration Report herein) and in his attached "Summary Report". Several final updates to parameters were made after the Calibration Report was written, and they are also documented herein. Much of the material in this section of this report has been excerpted from the Summary Report, with additional explanation provided.

Two very important constraints were placed on the original NCHRP 12-33 project that have implications for calibration. They were:

- No change in resistance over time was to be considered. It was to be assumed that maintenance would be adequate to preserve the original strength, and
- No growth in traffic loads (not density as in ADTT) such as may be associated with increases in legal loads were to be included.

Finally, the original calibration was for the strength limit state, and calculations were carried out for beam- and slab-type bridges.

2.2 Load Models

2.2.1 General

The statistical parameters of the total load, Q, are determined as a function of statistical parameters of load components. The mean of Q is a sum of the mean values of components,

$$\mu_Q = \mu_{DL} + \mu_{LL} + \mu_{IM} \tag{1}$$

where:

μ_{DL} = mean dead load
μ_{LL} = mean live load
μm_{IM} = mean dynamic load

The mean values of load components are calculated using bias factors, λ, and nominal (design) value of the considered load component. For example, for dead load,

$$\mu_{DL} = (\lambda_{DL}) \text{ (nominal value of DL)} \tag{2}$$

The variance of Q, σ^2_Q, is a sum of variances of the individual load components,

$$\sigma^2_Q = \sigma^2_{DL} + \sigma^2_{LL} + \sigma^2_{IM} \tag{3}$$

The standard deviation of Q, σ_Q, is equal to the square root of σ^2_Q. The coefficient of variation of Q, V_Q, is

$$V_Q = \sigma_Q/\mu_Q \tag{4}$$

The statistical parameters of components are summarized as follows.

2.2.2 Dead Load

Dead load effect is considered as a sum of four subcomponents, each of which is considered to be normally distributed, and hence the total is also considered to be normally distributed.

- DL_1 – weight of factory made elements (steel and precast concrete girders)
- DL_2 – weight of cast-in-place concrete
- DL_3 – weight of wearing surface (asphalt)
- DL_4 – weight of miscellaneous items (e.g. railing, luminaries)

The statistical parameters for dead load components, i.e., the bias factor and coefficient of variation, are summarized in Table 1.

Table 1 - Statistical Parameters of Dead Load

Dead Load Component	Bias Factor	Coefficient of Variation
Factory made members, DL_1	1.03	0.08
Cast-in-place, DL_2	1.05	0.10
Wearing surface, DL_3	1.0	0.25
Miscellaneous, DL_4	1.03 ~ 1.05	0.08 ~ 0.10

The bias factors for DL_1 and DL_2 were provided by the Ontario Ministry of Transportation based on surveys of actual bridges in conjunction with calibration of the Ontario Highway Bridge Design Code (OHBDC 1979; Lind and Nowak 1978). The coefficients of variation provided by the Ministry of Transportation for dead load were 0.04 and 0.08 for DL_1 and DL_2, respectively (Lind and Nowak 1978). However, there is no report available to support this data. The coefficients of variation used in calibration were taken from the NBS Report 577 (Ellingwood et al. 1980), and they include other uncertainties including human error.

The parameters of DL_3 are calculated using the survey data provided by the Ontario Ministry of Transportation in conjunction with calibration of the OHBDC (1979). The average of a number of field measurements of asphalt wearing surface, DL3, taken in Ontario was 3.5 in. The bias factor is assumed = 1.0. From the results of measurements, the coefficient of variation is taken = 0.25. The only direct use of this piece of information was to inform the choice of the load factor for DW which was taken as 1.5 due to the perceived higher variability of this component of dead load compared to the other components which were combined as DC in the AASHTO LRFD.

2.2.3 Live Load

2.2.3.1 The Ontario Truck Survey

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

Each vehicle from the survey was "run" over the influence lines to determine the calculated maximum bending moment, shear force and negative moment at the interior support of two span units of equal span. The calculations were carried out for span lengths from 10 ft through 200 ft. The resulting cumulative distribution functions (CDF) were plotted on the normal probability paper for an easier interpretation and extrapolation, as shown in Figure 1 for simple span moment. Other examples are documented in Appendix I. Similar results were obtained for shear and negative moment. The CDF's are presented for the surveyed truck moments divided by the HS20 moment. The results indicate that there is a considerable difference between the moment ratios depending on the span length. The shape of the CDF curves, i.e. not straight lines, indicates that the live load moments are not normal random variables. However, the distribution of the sum of the loads approaches a normal distribution. The data up to the "2 weeks" line represent the effect of the measured trucks; the darker extension to the "75 years" line illustrates the extrapolation for longer periods of time.

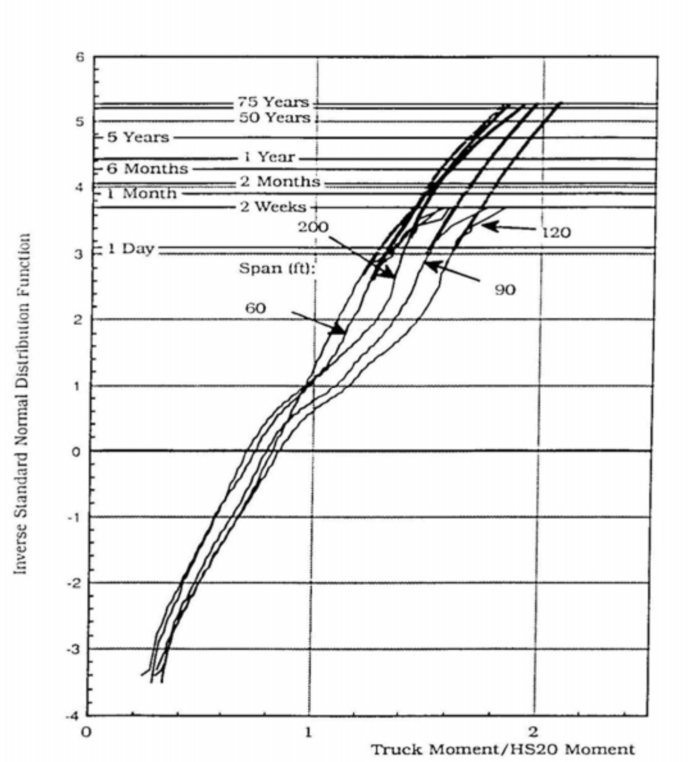


Figure 1 - Original and Extrapolated CDF's of Simple Span Moment due to Surveyed Trucks

Note that all available data was used in developing Figure1, but that some data at the upper end before the extensions were not used in arriving at the extrapolations from 2 weeks to 75 years. Two of the 9,250 trucks were extremely heavy and were classified as "outliers" for the purpose of projecting the truck weights.

2.2.3.2 Consideration of Multiple Vehicles on Bridge

Multiple presence can be considered in the same lane and/or in the parallel traffic lanes. The multiple presence factors are derived using the basic truck survey data and results of limited observations on Interstate Highways in Michigan. The assumptions involve a considerable degree of subjective judgment.

The multiple presence factors depend on the degree of correlation between the trucks. No correlation means that one truck has nothing to do with another one that happens to be on the bridge in the same time. Full correlation, practically means that both trucks are identical, they have the same axle configuration, carry the same load, belong to the same owner. There is almost no data on correlations, therefore, the assumptions are entirely based on judgment.

The live load effect per lane (moment or shear) can be caused by a single truck or by two or more trucks traveling in the same lane. Based on the results of observations, it is assumed that about every 50th truck is followed behind by another truck with the headway distance < 100 ft, about every 150th truck is followed behind by a partially correlated truck, and about every 500th truck is followed behind by a fully correlated truck. Therefore, the following combinations were considered:

(a) Max. 75 year truck
(b) Max. 1 year truck and average truck from the survey
(c) Max 6 month truck and max 1 day truck
(d) Max 1 month truck and max 1 month truck

If the headway distance is 50 ft (which is conservative), then the maximum 75 year truck governs for spans up to 140 ft for moment and about 120 ft for shear. For the span of 200 ft, the bias factor due to two trucks compared to a single truck bias, is increased by about 5-7%.

For two traffic lanes (in the same direction), based on limited observations it is assumed that about every 15th truck is on the bridge simultaneously with another truck in adjacent lane. It is assumed that every 150th truck is simultaneously with another truck that is partially correlated, and about every 450th truck is fully correlated. Therefore, the following combinations are considered:

(a) Max 75 year truck
(b) Max 5 year truck and average truck from the survey
(c) Max 6 month truck and max 1 day truck
(d) Max 2 month truck and max 2 month truck

The analysis was performed using the influence surfaces, to determine the governing truck combination. Assuming the trucks are 4 ft apart (which is conservative), the case with two fully correlated trucks governs. The maximum 2 month truck effect is about 0.85 of the maximum 75 year truck effect for moment and about 0.88 for shear. These two values were used in calibration calculations to treat the controlling live load case as two lanes loaded. In the discussion to follow in this article, the multiple presence factor for two loaded traffic lanes is conservatively taken as 0.85.

In AASHTO LRFD, it was decided to assume that the multilane reduction factor for two traffic lanes is 1.00. Therefore, for a single loaded lane the multiple presence factor is 1/0.85 = 1.20.

For three traffic lanes, it is assumed that the third truck occurs simultaneously with the other two trucks following the occurrence rates for the second truck (but applied to events with two trucks). Therefore, every 225th truck (15*15) occurs with two uncorrelated trucks, and every 202500th truck (450*450) occurs with two other fully correlated trucks. Therefore, the following combinations are considered:

(a) Max 75 year truck
(b) Max 4 month truck and average two trucks from the survey
(c) Max 2 week truck and max 1 day truck and average truck from the survey

(d) Max 1 day truck and max 1 day truck and max 1 day truck

The maximum 1 day truck effect is about 0.70-0.77 of the maximum 75 year truck effect, and the corresponding multiple presence factor is 0.75. However, because multiple presence factor for two lanes is assumed = 1.00, therefore, multiple presence factor for three lanes is 0.70-0.77 divided by 0.85 = 0.82-0.89, and recommended value is 0.85.

For more than three lanes, the multiple presence factor is about 0.70-0.75 = 0.85*(0.82-0.89).

The statistical parameters of live load (bias factor in terms of the HL-93 notional design live load and coefficient of variation) for ADTT of 1000 are summarized in Table 2 for spans from 20 ft through 200 ft for a single lane loaded and two lanes loaded. The bias factor represents the ratio of the mean value and nominal value. The coefficient of variation for the maximum 75-year lane moment was determined from the truck survey data, and it is 0.12. It was assumed that it is also 0.12 for the maximum 75-year load effect resulting from two-lane traffic.

The biases in Table 2 were determined without consideration of dynamic amplification, A.K.A. impact. Dynamic amplification is discussed in Article 2.2.3.4 both for the 75 year mean maximum live load and the HL-93 notional design live load.

2.2.3.3 Change from ADTT of 1,000 to 5,000

In the Calibration Report, reliability analysis was performed using ADTT of 1000. This value of ADTT led to an initial live load factor of 1.7 based on the considered values of 1.6 and 1.7 as reported in the Calibration Report. In the final pass through the calibration after the Calibration Report had been written, it was decided to base the specification on an ADTT of 5,000. At the time, it was estimated that this would increase the live load force effects by about 2.5% for moment and about 3.5% for shear. These estimates were determined by increasing the number of vehicles accounted for in figures such as Figure 1 by a factor of 5. As can be seen in Figure 1, the values of "Inverse Standard Normal Distribution Function" shown on the vertical axis get very compressed as the number of vehicles increases. The extensions in Figure 1 (and other figures) were further extended to the value on the vertical axis corresponding to the increased number of vehicles and the resulting change in moment or shear ratio on the horizontal axis was noted. This has the effect of increasing the 75 year mean maximum live load effects which would have been reflected in an increase in the live load biases reported in table 2 for ADTT of 1,000. The range of biases determined for an ADTT of 5,000 are also shown in Table 2. These two percentages, used as multipliers of 1.025 and 1.035, were used throughout the calibration process to upgrade the projected live load for ADTT of 5,000. (Table 2 and Figure1 are based on the span specific two lane and ADTT adjustments as opposed to the representative values quoted above and are therefore slightly more accurate.) As can be seen by the range of values for bias reported in Table 2 there is some span dependency. This is further illustrated in Figures 2-4 for both the HL-93 and HS20 live loads for ADTT of 5,000. These figures also show that the range of biases is significantly smaller for the HL-93 design loading compared to the HS20 design loading. The bias factor for HL-93 at a span of 10 ft is relatively high in Figure 2. This is a result of two closely spaced axles each weighing 44 kips which were recorded in the Ontario truck survey.

From the design demand point of view, the corresponding change for the higher value of ADTT was an increase in the live load factor to 1.75. As with many aspects of engineering knowledge, these ADTT adjustments continue to be reviewed by the research community.

Table 2 - Summary of the HL-93 Statistical Parameters of the Maximum 75 Year Live Load, ADTT = 1000 and ADTT = 5000

Moment or Shear	Number of Loaded Lanes	Bias Factor ADTT = 1000	Bias Factor ADTT = 5000	Coefficient of Variation
+ moment	1	1.23-1.36	1.26-1.38	0.12
	2	1.08-1.15	1.10-1.20	0.12
shear	1	1.17-1.28	1.21-1.32	0.12
	2	1.04-1.14	1.08-1.18	0.12
- moment	1	1.20-1.33	1.23-1.36	0.12
	2	1.10-1.22	1.14-1.26	0.12

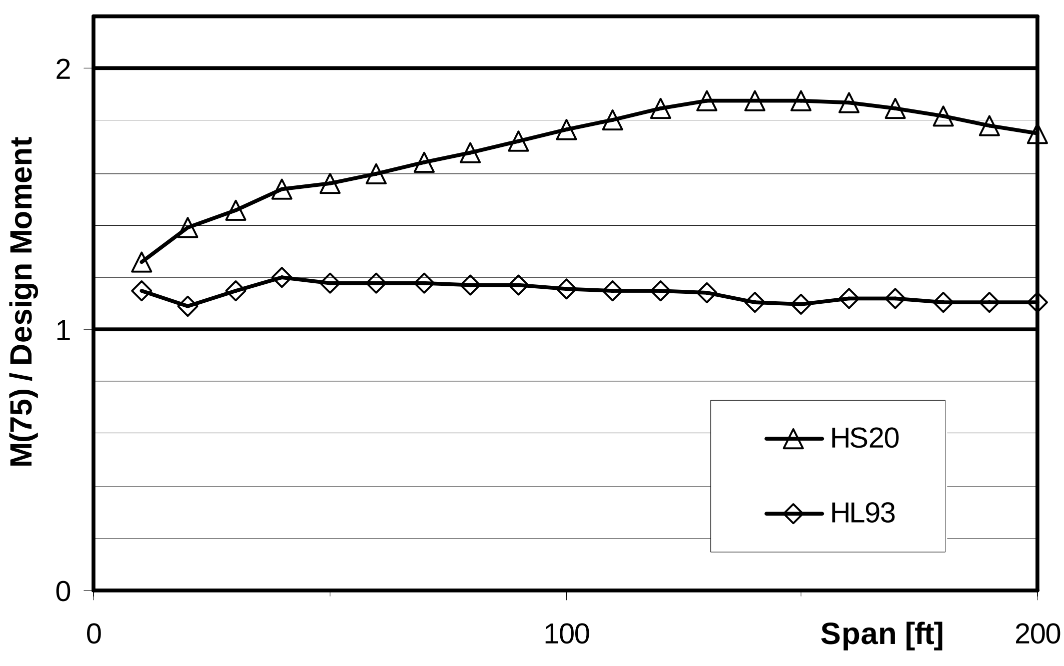


Figure 2 - Bias Factor for Two Lanes Loaded, Simple Span Moment; Ratio M(75)/M(HL-93) and M(75)/M(HS20), ADTT = 5000

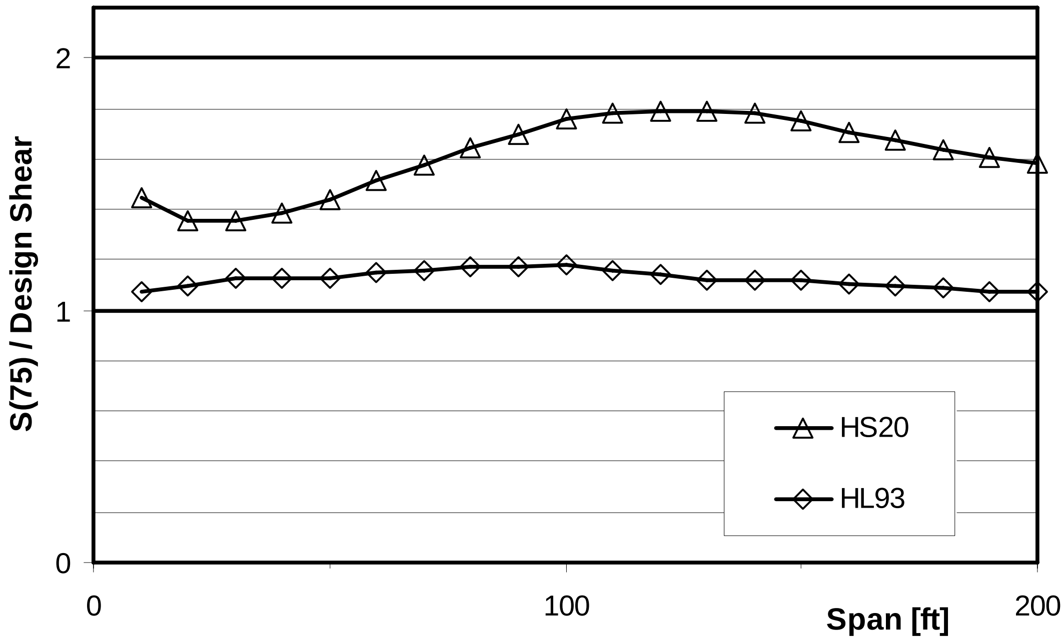


Figure 3 - Bias Factors for Two Lanes Loaded, Shear; Ratio S(75)/S(HL-93) and S(75)/S(HS20), ADTT = 5000

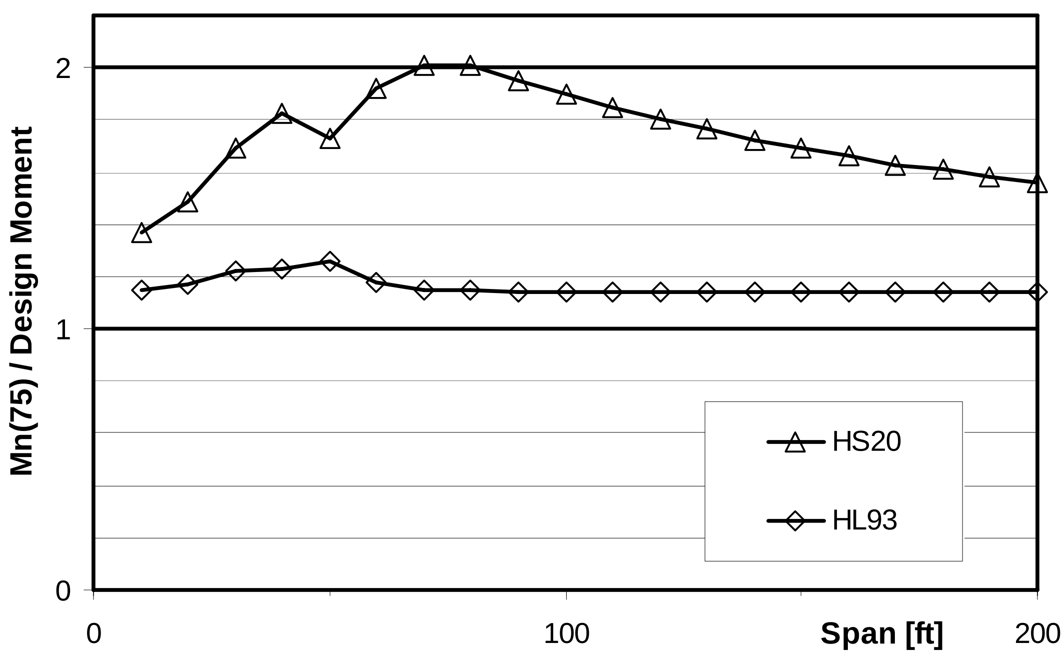


Figure 4 - Bias Factors for Two Lanes Loaded, Negative Moment; Ratio Mn(75)/Mn(HL-93) and Mn(75) / Mn(HS20), ADTT = 5000

2.2.3.4 Dynamic Load Allowance

Based on the reports cited in the original Calibration Report and the attached Summary Report, the dynamic load allowance associated with the maximum 75 year two-lane live load was 10%, i.e. a multiplier of 1.10 was used to convert the 75 year two-lane live load to 75 year two-lane live load plus impact. The COV for the combination of live load and dynamic allowance is 0.18.

In the calibration of AASHTO LRFD it was initially assumed that the design dynamic load allowance was 0.25 of the total HL-93 live load (design truck and uniformly distributed load), and the reliability indices are calculated for this value of the nominal dynamic load. After the initial calibration was completed, it was decided to specify dynamic load allowance as 0.33 of the design truck only, with no dynamic load factor applied to the uniformly distributed load. The practical effect of the change is relatively small because there is an offsetting decrease in omitting the dynamic load effect on the uniform load and increasing it on the design truck. This change was made because evidence indicated that the dynamic effect decreases when additional trucks were on the bridge, (the Calibration Report is one source showing this multi-truck effect) so it seemed illogical and less realistic to apply a dynamic load allowance to the uniformly distributed load associated with the HL-93 notional live load model. This reasoning was consistent with the original charge to make AASHTO LRFD more maintainable in the future, in part, by making the provisions more representative of bridge behavior.

2.2.3.5 Summary of Live Load Changes

As can be seen in the discussion above, there were several final adjustments made to the live load model after the work reported in the published Calibration Report was finished in 1991. Revised calibration calculations were made after the Calibration Report was written to confirm that the target reliability indices were still being met with the revised design dynamic load allowances and the live load moment and shear increased by 2.5% and 3.5%, respectively based on the change of ADTT.

2.3 Resistance Models

2.3.1 General

As in the case of loads, the requested detail is not available. The actual data used in the original calibration is summarized below from the two sources cited under the discussion of loads above.

The resistance was considered as a product of a nominal resistance, R_n, and three factors: M = material factor (strength of material, modulus of elasticity), F = fabrication factor (geometry, dimensions), and P = professional factor (use of approximate resistance models, e.g. the Whitney stress block, idealized stress and strain distribution model).

$$R = R_n \cdot M \cdot F \cdot P \tag{5}$$

The mean value, μ_R, and the coefficient of variation, V_R, of resistance, R, may be approximated by the following accepted equations for the range of values that were considered:

$$\mu_R = R_n \cdot \mu_M \cdot \mu_F \cdot \mu_P \tag{6}$$

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{7}$$

Eq. 6 is the same for correlated and uncorrelated parameters M, F and P. However, Eq. 7 is for uncorrelated M, F and P. In the case of the LRFD calibration, all data was considered uncorrelated. M, F and P were taken as uncorrelated. There is no known reason to assume that these distinct items, M, F and P are correlated. Eq. 7 is accurate if M, F and P are lognormal random variable. However, in practice it produces accurate results also for nonlognormal variables.

2.3.2 Sources of Data

The statistical parameters of resistance were determined using the test results available prior to 1990, special simulations, and engineering judgment. They were developed for noncomposite and composite steel girders, reinforced concrete T-beams, and prestressed concrete AASHTO-type girders. Bias factors and coefficients of variation were determined for material factor, *M*, fabrication factor, *F*, and analysis factor, *P*. Factors *M* and *F* were combined.

Material parameters for steel girders were taken from Ellingwood et al. (1980). Only statistical parameters such as the mean and standard deviation or coefficient of variation were available, but no actual test data (no raw data). For structural steel, the statistical parameters were based on the papers by Ravindra and Galambos 1978; Yura, Galambos and Ravindra 1978; Cooper, Galambos and Ravindra 1978; Hansell, Galambos, Ravindra and Viest 1978; Galambos and Ravindra 1978. The information included the mean values and coefficient of variation for the yield strength of steel, tensile strength of steel and modulus of elasticity, for hot-rolled beams and plate girders. In addition, they provided the parameters (mean value and coefficient of variation) for fabrication factor and professional factor. In the very last phase of calibration, the American Iron and Steel Institute provided the upgraded bias factors and coefficients of variation for yield strength of structural steel. These values were then used in Monte Carlo simulations to determine the parameters of resistance for noncomposite and composite girders, for the moment carrying capacity and shear.

For concrete components, the material parameters were taken from Ellingwood et al. (1980). As in the case of structural steel, only the statistical parameters were obtained but no raw test data. The basis for these parameters was research by Mirza and MacGregor (1979) and Mirza et. al. (1979). The data included mean value and coefficient of variation for the compressive strength of concrete, yield strength of reinforcing bars, and prestressing strands. In addition, the data included the statistical parameters of fabrication factor and professional factor.

The material data, combined with statistical parameters of the fabrication factor and professional factor, were used in Monte Carlo simulations that resulted in the statistical parameters of resistance for steel girders (noncomposite and composite), reinforced concrete T-beams and prestressed girders, for moment and shear, as shown in Table 3.

It was assumed that resistance is a lognormal random variable. Regardless of the source documents, resistance is considered as a product of several parameters, including M, F and P. The product of any types of random variables takes the form of a lognormal random variable (Nowak and Collins, 2000). This can be derived from the observation that the sum of any random variables takes the form of a normal variable. A logarithm of a product is the sum of logarithms, hence the logarithm of a product can be treated as a normal variable and from definition, the product itself can be treated as a lognormal variable.

Table 3 - Statistical Parameters of Component Resistance

Type of Structure	Material and Fabrication Factors, F M		Professional Factor, P		Resistance, R	
	λ	V	λ	V	λ	V
Noncomposite steel girders						
Moment (compact)	1.095	0.075	1.02	0.06	1.12	0.10
Moment (noncom.)	1.085	0.075	1.03	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Composite steel girders						
Moment	1.07	0.08	1.05	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Reinforced concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.13
Shear w/steel	1.13	0.12	1.075	0.10	1.20	0.155
Shear no steel	1.165	0.135	1.20	0.10	1.40	0.17
Prestressed concrete						
Moment	1.04	0.045	1.01	0.06	1.05	0.075
Shear w/steel	1.07	0.10	1.075	0.10	1.15	0.14

The values for bias and COV for yield strength of steel, tensile strength of steel and modulus of elasticity, for hot-rolled beams were taken from the NBS Report 577 (Ellingwood, MacGregor, Galambos and Cornell, 1980). However, the statistical parameters of resistance were obtained by combining the material data, dimensions and professional factor, using Monte Carlo simulations and Rosenblueth 2n+1 method. Some statistics:
Yield stress mean, flanges = 1.05 F_y, V = 0.10

Yield stress mean, webs = 1.1 F_y, V = 0.11
Tensile strength mean = 1.1 F_u, V = 0.11
Modulus of elasticity bias = 1.0, V = 0.06

Professional factor was determined solely based on engineering judgment. The NCHRP 12-33 Calibration Task Group reviewed the available literature and it was agreed to use the values listed in the NBS Report 577 (Ellingwood, MacGregor, Galambos and Cornell, 1980). So far these values were not challenged. Fabrication factors were also taken from NBS 577 report. Material factors were taken from the NBS Report 577; however, for structural steel they were later revised using the statistical parameters provided by steel industry (John Barsom). Material and fabrication factors were used as input data in the analysis to derive the statistical parameters. Rosenblueth 2n+1 method was applied (Nowak and Collins, 2000). Therefore, the parameters for FM shown in Table 3 are for components and they include the statistical parameters for materials and dimensions.

3.0 GUIDE TO DATA COLLECTION AND DOCUMENTATION

3.1 Introduction

3.1.1 General

This report proposes criteria for a guide on data collection and documentation for data set analysis used for calibrations for the LRFD Specifications. The criteria proposed address field and laboratory data collected for the development of load and strength models, and survey or other type of data collected for determining the effects of workmanship and construction tolerances on strength reliability. The field data may include traffic related measurements and field tests of dynamic loads, the laboratory data may include material properties tests, and the workmanship related data my include measurement data or surveys of engineers and contractors. The goal of the analysis used for calibrating load and resistance factors is to provide a design with an acceptable and consistent level of safety expressed in terms of a reliability index. As data on performance of LRFD bridges comes in over time, it will demonstrate the efficacy of choices made. The minimum parameters required for this level of analysis include mean, standard deviation, coefficient of variation and the type of distribution that best fits the data. The accuracy of the results is directly dependent on the adequacy, in terms of quantity and quality, of the input data used.

The main objective of the data collection criteria presented in this report is to ensure that the data collected is of sufficient quality and quantity to calibrate load and resistance factors or conduct reliability analyses. The objective of the proposed data documentation criteria is to ensure that the data collected and the results of its analysis will be readily available for future calibrations and reliability analyses when more data becomes available or when improvements in design methods are incorporated.

Some of the specific issues involved in the development of a guide on data collection and documentation that are addressed in this report include:

- What assurance measures are needed to collect high quality data?
- What are the amount and the quality of the data that is needed?
- How is a candidate data set evaluated to determine that it is suitable for use in developing information for the LRFD Specifications?
- What level of data is to be preserved?
- What type of information should be preserved for a data set?

The data collection and documentation guidelines proposed are separated by the phases involved in the data collection process and presented in several sections. Most of the sections apply to both the originators and the users of the data, however, Section 8 is mainly intended for the users of the databases, summaries and other information that is generated by others.

3.1.2 Sources of Information

3.1.2.1 Literature Review and Contact with Various Agencies

A limited number of email or telephone contacts were made with agencies that process large amounts of data requesting information on their data protocols. These included NASA, EPRI, USCOE and NEES. No response was received from the internet inquiry sites for NASA and EPRI. Personnel communication from a representative of the USCOE's ERDC at Vicksburg (often referred to as WES) indicated that a brief review revealed no standing written policies on data vetting or retention.

Information was obtained from a literature review and internet sites dealing with the Federal Data Quality Act, Climatic Data Recording sites, USGS and NEESinc, all of which have detailed policies and/or procedures. Since NEESinc has a structural engineering focus, it is most germane to the study.

3.1.2.2 Review of ASTM Standards

Several ASTM Standards (mainly from Committee E11.10 on Sampling) were reviewed. They include the standards listed below:

- E105-58 (Reapproved 1996) Standard Practice for Probability Sampling of Materials
- E141-91(2003) Standard Practice for Acceptance of Evidence Based on the Results of Probability Sampling
- E1402-99 Standard Terminology Relating to Sampling
- E1994-98 (2003) Standard Practice for Use of Process Oriented AOQL and LTPD Sampling Plans
- E178-94 Standard Practice for Dealing with Outlying Observations

- D5457-93 Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design

Most of these Standards deal primarily with evaluating production runs of a product and are not necessarily directly applicable to bridge engineering. However, some of the Standards include information that may be used or referred to. For example, ASTM E178-94 contains some good general information on outlying observations, and ASTM D5457 and E105 contain detailed information on sampling of wood products for LRFD design.

3.1.2.3 TRB Circular: Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design (Allen et al. 2005)

This Circular describes methodologies that can be used to determine load and resistance factors for geotechnical and structural design. It includes detailed procedures used to characterize data to develop the statistics and functions needed for reliability analysis and information on the quality and quantity of the data needed. The Circular also includes a listing of supporting documentation required for load and resistance factors calibrations (Appendix C of the Circular), which is included in this report.

3.2 General Data Considerations

3.2.1 General

To calibrate load and resistance factors and conduct reliability analyses the data collected needs to have sufficient quality and quantity to statistically define the variations and distribution of the data using mathematical relationships, in a reliable manner.

3.2.2 Characteristics of Data Collection

Certain characteristics are common to all types of data collection and may be separated into three main tasks:

- Defining the population and the sampling units to be measured
- Defining the specific information about the population that is to be measured
- Determining the methods of performing the measurements

A clear and precise definition of the population to be measured and the selection of unique, easily identifiable and representative sampling units is the first step in obtaining high quality data sets. The specific information that is to be measured must be clearly defined, easily recognizable and have clearly defined measurement parameters. In addition, the characteristics of the information measured including sources of uncertainty and variability must be well understood. Evaluating and selecting the most appropriate method of performing the measurements, which is consistent with the objectives of the data collection, is the third important task of the data collection process.

3.2.3 Attributes of High Quality Data

The general qualitative attributes of high quality data sets that are suitable for analysis and that can yield meaningful and reliable results are listed below. These attributes form the basis for the data collection criteria presented in this report. While it is not always possible to quantify these attributes, they can be used as ranking scales for the data sets.

- <u>Sensitivity.</u> The sensitivity criterion ensures that the measurement results can reflect small variations in the data being measured. It ensures that the proper techniques, instruments and calibrations are used.
- <u>Reliability.</u> The reliability criterion affects the precision of the data. It minimizes the random errors in measurements, and is usually indicated by consistency or stability in measurements over distance or time. It addresses errors such as those due to inconsistency, or lack of repeatability in testing or procedures used to measure the values in the data set. It also addresses the consistency in the criterion used to establish the measured values and the compatibility of the data if it is obtained from multiple sources.
- <u>Validity.</u> The validity criterion ensures that the data is accurate and that it measures what is intended to be measured. It addresses how well the measurements obtained reflect the actual situation being modeled (e.g., are the measurements based on small scale model studies or full-scale structures, does the laboratory test used to get the data accurately reflect how the parameter affects performance).
- <u>Standardization.</u> The standardization criterion ensures that the data collection procedures and techniques are uniformly followed and that the data differences are not due to technique, procedural or personnel differences.
- <u>Completeness.</u> The completeness criterion ensures that all the intended data, records and observations are present. For example, for reliability analysis it is important that a sufficient amount of data corresponding to the upper tail of the load distribution and the lower tail of the resistance distributions be collected.

Among the statistics used for calibrations purposes, data quality issues primarily affect the coefficient of variation of the random variable.

The quantity of the data can also have a strong effect on the estimation of statistical parameters such as mean value and coefficient of variation, depending on the required confidence level. The higher the confidence level desired, the larger the number of samples required. For a given confidence level, the required number of samples can be determined using the formulas and tables provided in textbooks on statistics or the appropriate testing standards when applicable. The quantity of data also affects the amount of extrapolation required when performing reliability analyses.

3.3. Initial Planning for Data Collection

3.3.1 General Recommendations

The initial planning of the data collection is an important first step in ensuring the quality of the data collected. This step usually involves decisions on the scope, the technical procedures, the system and instrumentation used, and the extent, analysis and documentation of the data, among others. It should also include a well defined quality assurance and quality control plan to assure the highest level of confidence in the data collected. Recommendations for the initial planning of the data collection process to improve data quality include the following:

- Select automated systems for collection, storage and processing of data to the maximum extent possible.
- Understand and document the capabilities and limitations of each data collection process.
- Use existing standards, specifications and procedures developed by others (for example, ASTM, AASHTO, FHWA) to the fullest extent possible.
- Ensure adequate longer term planning for the collection of bi-variate and/or temporal data such as fatigue and wind pressure related data that depend the frequency and duration of the data collection process.

3.3.2 Data Quality Objectives

Data quality objectives are those qualitative and quantitative statements developed by data users to specify the quality of data needed from a particular data-collection activity. As a minimum the data quality objectives to be established before the data collection begins shall include the following:

- Establish the data quality appropriate for the objectives of the work.
- Assess the attributes of quality data.
- Define data collection procedures to ensure the appropriate quality.
- Establish a process that will produce estimates of the variability of the data to be collected.
- Ensure that the work will be done by or supervised by qualified personnel.
- Verify that there is no potential for intentional or unintentional bias in the reported results or interpretation.
- Ensure that the equipment used in acquiring data or loading specimens will be calibrated and maintained properly.
- Establish quality control procedures.

3.4. Data Collection

3.4.1 Technical Procedure Documentation

The methods used to collect a specific data set shall be documented and the documentation shall be maintained with the data. Written records of exactly how data are collected are critical to establishing the consistency, comparability, repeatability, and traceability of the data.

Field Activities:

For field activities detailed documentation shall be generated prior to data collection. The documentation shall include detailed description of the technical procedure and the sequence of actions to be used to collect data to ensure repeatability of the work and comparability of results. If during the data collection there are deviations from the technical procedure and the planned sequence of actions, then these deviations shall be clearly described and documented with the data reporting. As a minimum the documentation made prior to the data collection shall contain the following:

- Purpose and description of the technical procedure that will be used.
- Identification of materials and instruments used to collect the data.
- Quantitative statement of the expected accuracy of the data to be collected and limitations on the use of the data.
- Step-by-step instructions to collect the data that would enable an independent, qualified person to repeat the work and produce comparable results.
- A description of how the data collected by using the procedure will be recorded and preserved.
- A listing of technical references used to compile the technical procedure.
- In the case of weigh-in-motion surveys, the time of year and even the time of day can significantly affect the data collected. These variables need to be documented.

Laboratory Work:

For laboratory work the documentation made prior to the actual tests shall include a detailed description of the specimens and the testing set up, including information such as:

- Detailed dimensional data (measured and nominal) that will allow for independent calculation of engineering properties.
- The position and magnitude of the loading devices.
- Boundary conditions/supports.

3.4.2 Instrumentation

Instrumentation shall be identified, calibrated, maintained and operated in an approved manner to ensure consistency, compatibility and repeatability of the data collected. Quality assurance activities for the instrumentation include:

- Identification of the instruments by type, manufacturer, and model.
- Calibration procedures and schedules for each instrument based on the stability characteristics of the instrument, required accuracy, intended use, manufacturer's recommendation, and other conditions that may affect the quality of the data. Calibration shall be performed whenever the accuracy of the instrument is suspect, regardless of the calibration schedule.
- Documentation of the calibrations including:
    - Name of organization and individual performing the calibration.
    - Identification of the instrument by type, manufacturer, model, serial number, or other unique and permanent identifier.
    - Identification of calibration standard, including the range and accuracy.
    - Identification of the document that describes the calibration process.
    - Date of current calibration and date or milestone for next scheduled calibration.
    - Records of instrument readings before and after any calibration.
- Data collected with instruments found to be out-of-calibration shall be evaluated to determine the effect on the intended use of the data. Affected data shall be discarded or their limitations documented in the data base and in any application of the data.
- All instruments used to collect data shall be operated in accordance with the manufacturer's manual, unless otherwise documented. Modifications to the manufacturer's operating procedure shall be appended to the manufacturer's manual, which shall be kept with the equipment at all times.
- Instrument maintenance shall be performed in accordance with the manufacturer's recommendations. A log shall be used to record maintenance performed.

Instrumentation shall be placed in appropriate places to measure the information required and used to support conclusions either in the individual work or to support conclusions from ongoing work using this data by others.

- Examples of laboratory auditing standards that address these quality assurance activities include the following standards:

- ASTM: E329 Standard Specification for Agencies Engaged in the Testing and/or Inspection of Materials Used in Construction

- ASTM: C1077 Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation

- ASTM: D3666 Standard Specification for Minimum Requirements for Agencies Testing and Inspecting Road and Paving Materials

- ASTM: D3740 Standard Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soils and Rock as Used in Engineering Design and Construction

- ASTM: E543 Standard Practice for Agencies Performing Nondestructive Testing

3.4.3 Identification and Control of Samples

The sample size directly influences the quality of the data set and its ability to provide data parameters and answer specific statistical questions with confidence. A certain minimum sample size is required for the data to be representative, and an increase in the sample size will result in more reliable estimates. However, the increase in the quality of the estimate does not remain proportional to the increase in the sample size. Although a larger sample size can compensate for some flaws in data collection and improve accuracy, there is a limit on the ability of a large sample size to compensate for poor quality measurements.

Field Activities:

Samples are often taken during the data collection and used to supplement field observations or to allow laboratory tests, analyses, and measurements. Types of samples may include soil samples, rock cores and structural material samples. Because these samples can be critical scientific evidence to support interpretations, they must be easily identifiable, handled and stored in a controlled manner and be traceable, and the relation between samples and the data set they represent must be maintained. Quality assurance activities for samples include:

- Development of unique identifiers which shall be placed on each individual sample using materials and methods that are clearly visible, legible, and durable and as well as other information that is critical for the intended use of a sample (such as orientation).
- Documentation of the sample including:
  - Sample identifier, type of sample and general description.
  - Location, date and time of collection.
  - Reference to technical procedure used description of sample collection, handling, preservation, transportation, and storage.
  - Storage location.
- Acquiring of companion material samples for laboratory investigations.

Laboratory Work:

To the extent possible, sample types and sizes for laboratory testing shall conform to the appropriate ASTM or AASHTO specifications. For example, ASTM D5457, which covers procedures for computing the reference resistance of wood-based materials for use in load and resistance factor design, includes a section (Section 4) on sampling.

ASTM D5457 Section 4.1 states the following: "Samples selected for analysis and implementation with this specification shall be representative of the population about which inferences are to be made. Both manufacturing and material source variability shall be considered. The principals of Practice E 105 [*Practice for Probability Sampling of Materials*] shall be maintained. Method D2915 [*Practice for evaluating Allowable Properties for Grades of Structural Lumber*] provides methods for establishing a sampling plan. Special attention is directed to sampling procedures in which the variability is low and results can be influenced significantly by manufacturing variables. It is essential that the sampling plan address the relative magnitude of the sources of variability." The warning provided in this section is intended to ensure that either the sampling plan or the daily quality control procedures are sensitive enough to reflect population shifts caused by factors such as manufacturing variations or shifts in material sources.

Section 4.2 of ASTM D5457, which addresses sample sizes, notes the following: "The confidence with which population properties can be estimated decreases with decreasing sample size. For sample sizes less than 60, extreme care must be taken during sampling to ensure a representative sample". The Specification requires that "For lower tail data sets, a minimum of 60 failed observations is required for sample sizes of n = 600 or less (This represents at least the lower 10% of the distribution.) For sample sizes greater than 600, a minimum of the lowest 10% of the distribution is required."

3.5 Data Analysis and Interpretation

3.5.1 General

The scope of the analysis of the data collected may be separated into two phases. The first phase is mainly concerned with the evaluation of the basic data to determine the sufficiency and adequacy of the data collected. It includes activities ranging from simple statistical applications to the development and application of complex numerical models. Issues involved include precision of the data, bias values in the data set, representativeness of the data, and completeness of the data.

The second phase of the analysis of the data collected is related to the application of the data to calibrations for AASHTO LRFD. The needed statistics for performing load and resistance factors calibrations using reliability analysis include as a minimum the mean, standard deviation, coefficient of variation and the type of distribution that best fits the data collected.

For calibration purposes the bias, which is defined as the ratio of the measured values and the design model predicted values, is the main variable used in analysis. The bias is usually characterized in terms of a cumulative distribution function, that represents the probability that a bias value is less than or equal to a given value. When a statistical distribution is developed to represent the data collected special attention needs to be paid to the distribution of the data in the tails of the cumulative distribution, since in most cases the data in the tails control the magnitude of the estimated load and resistance factors.

Because data analysis is not part of the scope of this task, only general guidelines are included herein. Specifics on data analysis can be found in standard text books in statistics such as, Nowak and Collins; Allen, et al. 2005.

3.5.2 General Information on Outliers

Outliers are notable data points that deviate a significant amount from the norm. Data points that are outside the expected range are generally evaluated to see if they can be considered to be outliers and, if justified, be removed from the data set. Leaving a few data points that are not part of the data set can skew the statistical parameters.

Typical reasons to consider a given data point to be an outlier include (Allen, et al. 2005):

- The data near a structure physical boundary are not specifically accounted for in the design model being used.
- A different criterion is used to establish the value of a given point or set of points (i.e., different failure criterion).
- A different measurement technique is used.
- Data from a source that may be suspect.
- Data that are affected by regional factors (e.g., regional geology effects on soil or rock properties.
- Any other issues that would cause the data within a given data set to not be completely random in nature.

Special consideration should be given before removing data at the upper ends of the load related data and the lower ends of the resistance related data, since they can have a significant effect on the results.

Additional information on dealing with outlying observations is included in ASTM 178-94. In this Standard an outlier is defined as a data point "that appears to deviate markedly from other members of the sample in which it occurs". Several methods of assessing data for potential classification as outliers are described. There are several caveats including "The procedures covered herein apply primarily to the simplest kind of experimental data, that is, replicate measurements of some property of a given material or observations in a supposedly single random sample". Thus, the applicability to data derived by varying one or more parameters in an experiment is not clear unless there are several replicate experiments in which case the Standard may apply to a subset of the data.

In terms of a general approach to identify possible outliers, ASTM 178-94 Section 2.1 states "When the experimenter is clearly aware that a gross deviation of the described experimental procedure has taken place, the result and observation should be discarded, whether or not it agrees with the rest of the data and without recourse to statistical test for outliers". This may be a result of merely being an extreme case of the random variable under consideration, in which case, the Standard recommends that it be retained and processed along with other data. If the outlier observation is a result of gross deviation from prescribed experimental procedure or an error in calculation, the Standard recommends an investigation take place to determine the reason for the potential outlier value. These observations deal with a situation where an investigator is determining these values as opposed to trying to use them from the literature. In the latter case, it may be extremely difficult to ascertain where there is a procedural recording or calculation error. Nonetheless, some of the methods outlined in this Standard could be used to determine the potential for an outlier or outliers in data used for bridge calibrations.

Another, perhaps simpler, way of approaching the information is to plot the data on probability format as described in Allen 2005. Aberrant data should appear as unusual in the context of the other data in a visual presentation, e.g., they should stand out.

Declaring information to be an outlier and not including it in the data has certain judgmental risks involved. Therefore, unless there is an obvious reason for an aberration, the data should be processed with all points considered prior to any filtering of presumed outliers. They can be ignored when extrapolating the results. This way the situation is visible and if needed, a revised interpretation can be considered.

3.5.3 Grouping of Data from Multiple Sources

When attempting to group data from several sources together to create a data set used to characterize a random variable special attention should be paid to ensure that the data is compatible. For example, the data from the different sources should be based on the same failure criteria and the same materials if the material type influences the results.

3.5.4 Sources of Uncertainty and Errors

Possible sources of uncertainties should be identified, evaluated and documented, and the data collected should properly reflect all the uncertainties involved. For example, common sources of uncertainty involved in resistance models include variability in material characteristics, tolerances and errors in fabrication and construction, and inaccuracies and errors in data interpretation and analysis. The type of data collected to reflect these uncertainties may include laboratory or field tests, and various measurements and surveys.

The errors associated with the data collection, interpretation and analysis process may be separated into systematic and nonsystematic errors. Systematic errors are those that follow a pattern or are controlled by a function that may bias the observations in a specific direction. They are usually not affected by the sample size and can not be corrected by repeated sampling. Nonsystematic errors are those that occur at random. They usually fall into the broad categories of data definition, procedure, measurement, and data handling.

Examples of errors associated with the procedures used to collect, process, and analyze the data include:

- Data definition errors, such as overlapping or inconsistent measurements caused by ambiguities in defining and describing the data to be collected.
- Data collection errors, such as inaccurate measurements or improper recording of the data.
- Spatial variability errors, such as the use of samples from locations that may be outside of representative areas.
- Data handling errors, such as excluding relevant data.
- Errors due to lack of standardization among techniques, equipment and personnel used.
- Data collection procedures and techniques that skew the data in one direction.
- Problems with testing or data collection equipment.
- Errors in the computational procedures used for sorting, reporting and analyzing the data.

3.5.5 Quality Assurance Aspects

Quality assurance aspects related to the analysis of the data sets shall include the following:

- A detailed plan that describes methods and approaches of data analyses shall be developed and documented
- A review of the quality and quantity of the data sets to be used in analysis shall be made to address the following questions:
    - Do the data sets represent the variables being modeled, including all sources of uncertainty that can affect the variable?
    - Is enough known and documented about how the data was developed and the conditions the data represent to provide sufficient confidence in the use of the data?
    - Are enough data available to ensure the mean, standard deviation and cumulative distribution function adequately characterize the data for the particular application? Guidelines may be found in sources such as the ASTM Standards and textbooks on statistical methods or testing of engineering materials (e.g., Bethea, 1984 or Davis, 1982).
    - Has the data been collected or measured using the same techniques?
    - Have outliers been properly identified and removed from the dataset?
- Interpretations made shall be appropriately qualified, including descriptions of model limitations and data uncertainty.
- The type of distribution selected to represent the data should fit the full data set and the upper and lower tails, as needed for the particular application.
- Independent reviews of the data analysis procedures and interpretations shall be included at different stages of the work. Reviews of data analysis procedures ensure that the selected analysis techniques are appropriate for meeting project objectives.
- The original data, data analyses, and data that were collected but not used in analysis, along with reasons for the exclusion, shall be documented and preserved.

- Interpretations of data analysis shall be appropriately qualified, including descriptions of limitations and data uncertainty.
- The results of the data analysis shall be independently reviewed to ensure that the analysis is valid. The reviewer shall be provided with any necessary background information to adequately perform the review.

3.6 Data Collection Documentation

It is important that the basic data collected and the data used for load and resistance factor calibration be adequately documented so that the data can be reused, as appropriate, to update load and resistance factors as design specification improvements are made or as new statistical data become available.

The documentation guidelines for the data used for load and resistance factor calibration listed in the TRB Circular (Allen, et al. 2005) include the following general requirements:

- The documentation is detailed enough for users of the design specifications to understand the bias of the calibrations used to develop the load and resistance factors, to make it possible for designers to rationally apply engineering judgment and adapt those factors for project– or site-specific considerations when necessary.
- Rationale for omitting outlier data is explained.
- The input data and assumptions used are documented in adequate detail so that future researchers can reproduce the nominal predictions of each measured resistance or load contained in the database.
- The methods and criteria used to measure resistances, loads, or input parameters (i.e., those parameters defined as random variables in the calibration) are adequately documented to allow rational combination of that data with future data gathered for the purpose of expanding the database.
- The documentation is developed in a way that it can be stored in an accessible repository to ensure that the investment made in gathering the extensive data necessary to conduct reliability calibrations is not lost.
- A description of the theoretical basis and computational procedure in sufficient detail to perform the analysis.
- All data requirements or options for the data-analysis procedure.
- Comparisons of the technique with known or accepted solutions.

Guidelines for documenting the technical procedures used in the data collection are described in Section 3.4.1 of this report, and additional information on the documentation of data may be found in Article 3.9.

3.7 Analysis Data Documentation

The type and amount of documentation for the data used and the data developed as part of the calibrations for the LRFD Specifications will vary to some extent with the specific design procedures considered. When developing databases to support reliability based calibrations, the following summary data shall be documented as a minimum (Allen, et al. 2005):

- Description of random variable for which data are available
- If a random variable is a design method result (e.g., shaft bearing resistance, beam bending, etc.), identify the design method used for the nominal prediction, including source and date of publication of the method (an example calculation could be useful here)
- Source of data
- Number of data points, including outliers
- Normal mean of measured/predicted values (bias)
- Normal COV of measured/predicted values
- Type of distribution used to characterize the data (e.g., normal, lognormal, etc.)
- Final normal mean of measured/predicted values used for calibration (bias) – best fit to tail
- Final normal COV of measured/predicted values used for calibration– best fit to tail
- Type of distribution used to characterize the tail (e.g., normal, lognormal, etc.)
- Number of data points when outliers are removed
- If outliers are removed, provide reason for removal
- For each random variable reported in the summary table, provide a standard normal variable versus measured/predicted value plot, and show on the plot:
    - each individual data point
    - the distribution calculated from the mean and COV of the entire dataset (after outliers are removed), and
    - distribution used for the best fit to tail, and its mean and standard deviation (used for the final calibration)

For the documentation of the reliability analyses conducted, the following guidelines should be followed (Allen, et al. 2005):

- Provide detailed design and limit state equations (g) used, including their derivation and source, and include the load and resistance factors in the design equation that are to be determined or considered in the calibration
- Identify any assumptions used to develop the limit state equation
- Provide the target β value to be used and the reason for the selection of that target value
- For resistance factor calibration, identify loads considered and load factors used, and describe the calibration approach
- For load factor calibration, describe equations, process and logic used to determine load factors

3.8 Data Quality and Quantity Assessment

3.8.1 General

This section deals mostly with quality assurance steps that a user of existing data, typically found during a literature review, should consider when determining that that data is suitable for code calibration purposes. At this writing there is increasing interest is systematic methods of reviewing published technical reports, often coming from medical researchers. The assessment of data quality is sometimes referred to as "Metadata" which might be thought of as data about data. This field of study is becoming as discipline unto itself, although it has equally passionate proponents and skeptics. A recent paper (Egan, Petticrew and Ogilvie, TRR 1998) contains the following summary:

> The most striking difference between systematic reviews and more traditional literature reviews is the highly structured methodologies that systematic reviews use to reduce bias. Typically they contain the following stages. They begin with the writing of a protocol in which a research question is framed (frequently, but by no means exclusively, focusing on the effects or effectiveness of a specific intervention). The protocol also prescribes the criteria by which relevant primary studies will be identified, included, appraised, and synthesized, in sufficient detail to enable replication. The next stage is a comprehensive literature search of electronic databases, citations, and contact with experts in the field. Reviewers will then critically appraise each included study (often two or more researchers do this independently and compare appraisals) and then synthesize their results with appropriate weighting given to the most methodologically robust studies. The intention is to bring to research synthesis similarly high standards of methodological rigor and transparency that one would expect from primary research, by encouraging reviewers to select and appraise studies using explicit and methodologically sound principals *(7-9)*.

The following three references are those cited in the excerpt above:

7. Cooper, H. *Guidelines for Preparing C2 Protocols for Systematic Reviews*. campbellcollaboration.org/Fraguidelines.org. Accessed July 1, 2004.
8. Centre for Reviews and Dissemination. *Undertaking Systematic Reviews of Research on Effectiveness: CRD's Guidance for Those Carrying Out or Commissioning Reviews.* CRD Report 4, 2d ed. University of York. 2001. www.york.ac.uk/inst/crd. Accessed March 23, 2005.
9. Alderson, P., S. Green, and J. Higgins, eds. *Cochrane Reviewers' Handbook 4.2.2: The Cochrane Library,* Issue 1. John Wiley and Sons, Chichester, England, 2004.

Elvik (TRR 1998) suggest the following regarding a systematic approach to data evaluation using the following three key elements:

- A specification of what a sensitivity analysis should include: publication bias, choice of estimator of effect, outlier bias, choice of statistical weights, and assessment of study quality;
- A specification of the order in which the items considered should be analyzed: start by testing for publication bias, continue by testing for choice of estimator of effect, outlier bias and statistical weighting, and end by testing for study quality assessment;
- A proposal for how to describe the results of a sensitivity analysis, by stating how consistent summary estimates are with respect to direction and magnitude of effect, and by estimating a robustness score, ranging from 0 to 1.

  There surely are sources of error in meta-analysis other than those considered here. The items given here should be seen as mandatory: any sensitivity analysis should consider these items but may consider other items as well."

3.8.2 Recommended Guidelines

For the purpose of this study, it is suggested that an assessment of the value and quality of the data for specification development be based on the following questions as a minimum:

- Was the work done by or actually supervised by qualified personnel?
- Does a review of the sponsor of the work, author(s) of the research needs statement, etc indicated any potential for intentional or unintentional bias in the reported results or interpretation?
- Was equipment used in acquiring data or loading specimens calibrated and maintained?
- Were companion material sample tests taken for laboratory investigations?
- Were resistance and/or load prediction calculations based on the measured material properties or assumed specification properties?
- Was the method of acquiring data sufficiently well explained to determine that the state of the art was utilized?
- Was this instrumentation placed in appropriate places to measure the information used to support conclusions either in the individual work or to support conclusions from ongoing work using this data by others?
- Do the measurements support conclusions in the original work or is it sufficient to support conclusions for the intended use as data by others.
- Were test specimens constructed in a manner generally consisted with U.S. construction methods and means?
- Were detailed measurements of the actual dimensions provided to allow independent calculation of engineering properties?
- Was the position and magnitude of loading devices documented and consistent with the state of results?
- Were boundary conditions and supports well documented?
- Were the actions of supports considered in evaluating the results?

- Is sufficient raw data provided/available to justify any filtering that might have been done or any summarizing through statistical treatment?
- Are outliers identified, vetted, and explained? Is statistical data developed or is sufficient information presented for independent development without consideration of the outliners? (Note that this is consistent with the approach discussed in reference to Figure 1)
- Can the user make any independent checks or other quality assurance checks? This could include something as simple as equilibrium checks or a general scan of results looking for perturbations or other points of curiosity.

This list of information should be stored with the archive as a form of metadata.

3.9 Data Archiving

3.9.1 General

Archiving is the systematic process of storing data and information to protect it from change or loss, by providing the necessary security and storage medium stability. Data archiving ensures that the data and its interpretation could be available for further analysis, if needed. Having access to the data used for load and resistance factor calibrations will be important for new calibrations, which are expected to occur as design specifications evolve, and as more statistical data that allows for improved calibrations becomes available.

Recommended general guidelines for data archiving include the following:

- Records should be maintained to reflect the archived level of quality for completed work.
- The data should be stored in such a manner and location as to protect and preserve the information.
- Data identification and storage procedures should ensure that the data can be traced to the original source and are easily retrievable; this calls for a systematic approach for data storage.
- Maintenance procedures for the data should include provisions for retention, protection, preservation, traceability, and retrieval.

3.9.2 Extent of Data to be Archived

Regarding the extent of the data that should be saved, there are philosophical and practical issues to be deal with. The Federal Data Quality Act and the NSF NEESinc data repository are at one end of the spectrum and a “do nothing” attitude is at the other.

NEESinc’s data policy speaks to storing all of the data collected by data acquisition systems monitoring an experiment in multiple laboratories, as well as “metadata” about the instrumentation right down to serial numbers on transducers, calibration data on load cells etc. This speaks to a data infrastructure, data security, specialized staff at individual NEES sites and at the repository, continuing funding to maintain the infrastructure and the institutional will to do so long term. In the context of data for code calibration, retention of real time data streams from experiments is not warranted. The shear volume of information would be unworkable.

“Do nothing” ignores the fact that generation of data represents a large portion of the national investment in research.

3.9.3 Type of Data to be Archived

The following detailed data should be archived for future use, and should be adequate for future researchers to be able to reproduce the measured and predicted nominal values (Allen, et al. 2005):

- For each measured/predicted value in the data set, for each random variable, tabulate:
    - Each measured value, including
        - the test methods and criteria used to determine those values, including test sample size, if applicable, or measurement method used, and any information on the accuracy of the method used,
        - the geometry of structural element on which measurements were taken, and
        - the source of the data.
    - Each predicted value, including all input data used to calculate the predicted value, including
        - structure and surrounding structure geometry that affects the nominal value
        - input parameters that were assumed to be deterministic
        - any simplification or modification of the design method used to calculate the nominal values,
        - any design assumptions, test data, or correlations used to generate the input design parameters for the nominal value design equation, including the test methods used to obtain measured parameters, specimen size, and any modifications to the test methods used that are not reflected in the test standard, and
        - the source of the data.
    - Include this in a report appendix and an electronic format suitable to the sponsor.

Consideration should also be given to archiving all the documented analysis data described in Section 3.7 of this report.

4.0 BRIDGE DATABASE

4.1 NCHRP 12-33 Bridge Database

A sample bridge database was established during the development of the LRFD Specifications under NCHRP 12-33. Approximately 200 representative bridges were selected from various regions of the United States by requesting sample bridge plans from various states. The selection was based on structural-type, material and geographic location to represent a full-range of materials and design practices as they vary around the Country. Anticipated future trends were also considered by questionnaires sent to selected states. One-hundred and seven sets of plans were received from which the 200 representative bridges were selected. Obviously, some plan sets contained more than one bridge or the bridge contained several separable units. The list of structures provided by the State Departments of Transportation is fully described in the Calibration Report. The bridges represented mid-1980's designs or earlier and clearly do not contain bridges designed by the LRFD Specifications.

For bridges selected from within this database, moments and shears were calculated for the dead load components, the live load and the dynamic load allowance. Nominal or design values were calculated using the 1989 edition of the AASHTO Standard Specifications. The statistically projected live load and the notional values of live load force effects were calculated. Resistance was calculated in terms of moment and shear capacity. For each structure, both the nominal design resistance, indicated by the cross-section shown on the plans, and the minimum actual required resistance according to the 1989 AASHTO were developed.

While the data tabulated for these bridges still exists in the Calibration Report, the plans themselves were in an office at the University of Michigan that was flooded and only six of the plan sets are still available. Some information relative to the date on the plan set or whether they were designed by ASD or LFD can now only be determined for those six plan sets. Therefore, a new, updated bridge database is desired.

4.2 New Bridge Database

With respect to the use of the information in this new database, it is important to consider how the reliability index was computed for bridges during the development of AASHTO LRFD. It was assumed that all designs exactly satisfied the requirements of the factored loads in the specifications and were not over-strength for reasons of either designer bias or the possibility that another limit state controlled. For example, in the design of precast, prestressed beams, it is relatively common for the service limit state to control and, therefore, these beams would be oversized or over-strength for the strength limit state. This excess capacity was ignored. The sum of the specification required factored loads was substituted for the resistance in the calibration equation. Alternatively, when evaluating an existing bridge for its reliability index, it is possible to substitute the total actual resistance provided into the same reliability analysis.

Two databases were developed. The first utilizes "real" bridges taken from the following sources:

- A series of bridges used by Dr. Dennis Mertz in NCHRP 20-7/122 in relationship to the investigations on the load and resistance rating (LRFR) methodology identified as "DRM".
- Several LRFD designs completed by the State of West Virginia identified as "WVDOT".
- Plan sets available from Modjeski and Masters' files from LFD and LRFD designs done in Oklahoma. Identified as "OK".
- A series of ASD designs from multi-cell box girders from the State of Tennessee. Identified as "Tenn".

These bridges were augmented with bridges retained from the original NCHRP 12-33 bridge database identified as "Nowak". The whole set of possibilities is summarized in Table 4.

- The use of box girder designs from California was discussed with a representative of that Department of Transportation. Seismic provisions control many of those designs and, therefore, they have strength characteristics which would not necessarily be typical nationwide. The difference is in longitudinal mild steel at each bent, and in cap stirrups around each column-to-cap connection. Perhaps more germane to differences between bridges in California's and those in other parts of the country, is that:
    - Caltrans uses whole-width design, and
    - its design permit vehicle generally controls girder shear.

Several other states in the west use those same provisions. Therefore, with respect to the multi-cell box girder construction, some California designs from the original 12-33 database, which probably precede those requirements, were used along with designs provided by the Tennessee DOT. Due to the high redundancy of these multi-web box beams and their high torsional rigidity, it is reasonable to expect that their system reliability is quite high.

In the original 12-33 calibration, there were a series of hypothetical bridges developed based on real bridges. These hypothetical bridges had the benefit of having a uniformly varying set of span lengths and girder spacing. This made the results of the effect of various parameters easier to see. Therefore, a second database comprised of a set of hypothetical bridges from the following sources was also compiled:

- The results of preliminary designs by Modjeski and Masters, inc. for prestressed concrete bridges using adjacent box beam bridges, spread box beams, and I-beams. These preliminary designs were developed using the "QUICKBEAM" spreadsheet developed by Dr. Alex Aswad, PE. This spreadsheet is based on the results of a multitude of designs complying with the PaDOT Design Manual and the types of beams used in the Atlantic States. The spreadsheet uses either the HL-93 loading and LRFD or HS-25 loading and LFD. Of course, in setting up the database it is easy to convert HS-

25 to HS-20. It is also possible to adjust the dead load of the beam itself to approximate the effect of the reduction in live load on the beam weight.

- A series of slab bridges designed using software developed by Dr. Jay Puckett for CRSI. These are all LRFD design using the HL-93 Live load.
- A series of preliminary designs for steel plate girder bridges available in *Composite Steel Plate Girder Bridge Superstructures*, formerly available from the United States Steel Company. Design from this source are part of the bridge database developed under NCHRP 12-50.

The characteristics of these bridges are shown in Table 5.

Reliability analysis was performed for the bridges in Tables 4 and 5 and the results are presented in Section 5.3.

Table 4 - Bridge Set 1 – Actual Bridges

Prestressed/Pretensioned Concrete Boxes

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
PS Box #1	Nowak #54 (PA)	N54-PA-G1	Prestressed Concrete Boxes	b	1966	HS20	1964 AASHTO	43	42	No	9.4	3.75
PS Box #2	Nowak #54 (PA)	N54-PA-G2	Prestressed Concrete Boxes	b	1966	HS20	1964 AASHTO	43	42	No	9.4	3.75
PS Box #3	Nowak #45 (PA)	N45-PA-G1	Prestressed Concrete Boxes	b	1969	HS20	1964 AASHTO	81.5 Avg	47	No	9.3	2
PS Box #4	Nowak #45 (PA)	N45-PA-G2	Prestressed Concrete Boxes	b	1970	HS20	1964 AASHTO	81.5 Avg	47	No	9.3	2
PS Box #5	Nowak #50 (PA)	N50-PA-G1	Prestressed Concrete Boxes	b	1966	HS20	1964 AASHTO	37.5	52	No	10.2	2.75
PS Box #6	Nowak #50 (PA)	N50-PA-G2	Prestressed Concrete Boxes	b	1966	HS20	1964 AASHTO	37.5	52	No	10.2	2.75
PS Box #7	Nowak #51 (PA)	N51-PA-G1	Prestressed Concrete Boxes	b	1964	HS20	1961 AASHTO	35.5	72	No	7.3	2.75
PS Box #8	Nowak #51 (PA)	N51-PA-G2	Prestressed Concrete Boxes	b	1964	HS20	1961 AASHTO	35.5	72	No	7.3	2.75
PS Box #9	Nowak #47 (PA)	N47-PA-G1	Prestressed Concrete Boxes	b	1965	HS20	1964 AASHTO	45.5	72	No	9.5	2.75
PS Box #10	Nowak #47 (PA)	N47-PA-G2	Prestressed Concrete Boxes	b	1965	HS20	1964 AASHTO	45.5	72	No	9.5	2.75

“California” Prestressed Concrete Boxes

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
CA Box #1	Nowak #60 (CA)	N60-CA-G1	"California" Prestressed Concrete Boxes	d	1979	HS20	LFD	53.25	125	No	7	3
CA Box #2	Tenn Ex #1	BR No 5 Airport Rd	"California" Prestressed Concrete Boxes	d	1970	HS20	LFD	44	128-134	Yes	9.25	3.5
CA Box #3	Tenn Ex #2A	7A: S-2492	"California" Prestressed Concrete Boxes	d	1971	HS20	LFD	46	41-102-52	Yes	7.67	4.33
CA Box #4	Tenn Ex#2B	7B: S-2492	"California" Prestressed Concrete Boxes	d	1971	HS20	LFD	46	47.5-102-47.5	Yes	7.67	4.33
CA Box #5	Tenn Ex #3	Br. No 4 Nauvoo Rd	"California" Prestressed Concrete Boxes	d	1975	HS20	LFD	32	110-110	yes	8.75	2.5

Prestressed Concrete I-Beams

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
P/S I #1	OK	US 59, Spans 1,2 (Int)	Prestressed Concrete I-Beams	k	1999	HL93	LRFD	81	134	No	6.083333	4
P/S I #2	OK	US 59, Spans 1,2 (Ext)	Prestressed Concrete I-Beams	k	1999	HL93	LRFD	81	134	No	6.083333	4

Steel Welded Plate Girders

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
Pl Girder #1	WVDOT	10044	Steel Welded Plate Girders	a	2003	HL-93	LRFD	44.49	103	N/A	9.5	3.25
Pl Girder #2	WVDOT	4200	Steel Welded Plate Girders	a	2003	HL-93	LRFD	44.79	108	N/A	9.5	3.42
Pl Girder #3	WVDOT	4201	Steel Welded Plate Girders	a	2003	HL-93	LRFD	44.46	2 @ 98 1 @ 146 Avg. = 114 Max. = 146	Yes	9.5	3.25
Pl Girder #4	WVDOT	10045	Steel Welded Plate Girders	a	2003	HL-93	LRFD	30.5	1 @ 162 1 @ 155 Avg. = 158.5 Max. = 155	Yes	11.67	3.58
Pl Girder #5	OK	US-59, Span 3, Int+	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	250	Yes	12.17	4
Pl Girder #6	OK	US-59, Span 5, Int+	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	325	Yes	12.17	4
Pl Girder #7	OK	US-59, Span 3, Int-	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	275	Yes	12.17	4
Pl Girder #8	OK	US-59, Span 5, Int-	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	338	Yes	12.17	4
Pl Girder #9	OK	US-59, Span 3, Ext+	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	250	Yes	12.17	4
Pl Girder #10	OK	US-59, Span 5, Ext+	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	325	Yes	12.17	4
Pl Girder #11	OK	US-59, Span 3, Ext-	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	275	Yes	12.17	4
Pl Girder #12	OK	US-59, Span 5, Ext-	Steel Welded Plate Girders	a	1999	HL93	LRFD	81	338	Yes	12.17	4

Table 5 - Bridge Set 2 – Generated Bridges

Prestressed Concrete Boxes

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
PS Box #11	M&M	MM-AB-L-01	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	31	50	No	3	0.5
PS Box #12	M&M	MM-AB-L-16	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	37	50	No	4	0.5
PS Box #13	M&M	MM-AB-L-02	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	31	60	No	3	0.5
PS Box #14	M&M	MM-AB-L-17	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	37	60	No	4	0.5
PS Box #15	M&M	MM-AB-L-23	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	47	80	No	3	1
PS Box #16	M&M	MM-AB-L-18	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	37	80	No	4	0.5
PS Box #17	M&M	MM-AB-L-14	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	37	100	No	3	0.5
PS Box #18	M&M	MM-AB-L-19	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	37	100	No	4	0.5
PS Box #19	M&M	MM-AB-L-05	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	31	120	No	3	0.5
PS Box #20	M&M	MM-AB-L-10	Prestressed Concrete Boxes	f	2006 Design	HL-93	LRFD	31	120	No	4	1.5
PS Box #21	M&M	MM-AB-A-1	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	31	50	No	3	0.5
PS Box #22	M&M	MM-AB-A-16	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	37	50	No	4	0.5
PS Box #23	M&M	MM-AB-A-2	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	31	60	No	3	0.5
PS Box #24	M&M	MM-AB-A-17	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	37	60	No	4	0.5
PS Box #25	M&M	MM-AB-A-23	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	47	80	No	3	1
PS Box #26	M&M	MM-AB-A-18	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	37	80	No	4	0.5
PS Box #27	M&M	MM-AB-A-14	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	37	100	No	3	0.5
PS Box #28	M&M	MM-AB-A-19	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	37	100	No	4	0.5
PS Box #29	M&M	MM-AB-A-5	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	31	120	No	3	0.5
PS Box #30	M&M	MM-AB-A-10	Prestressed Concrete Boxes	f	2006 Design	HS25	LFD	31	120	No	4	1.5
PS Box #31	M&M	MM-SB-L-07	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	31	60	No	7.75	3.88
PS Box #32	M&M	MM-SB-L-08	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	31	80	No	7.75	3.88
PS Box #33	M&M	MM-SB-L-09	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	31	100	No	7.75	3.88
PS Box #34	M&M	MM-SB-L-10	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	31	120	No	7.75	3.88
PS Box #35	M&M	MM-SB-L-26	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	47	40	No	11.75	5.88
PS Box #36	M&M	MM-SB-L-27	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	47	60	No	11.75	5.88
PS Box #37	M&M	MM-SB-L-28	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	47	80	No	11.75	5.88
PS Box #38	M&M	MM-SB-L-29	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	47	100	No	11.75	5.88
PS Box #39	M&M	MM-SB-L-30	Prestressed Concrete Boxes	b	2006 Design	HL-93	LRFD	47	120	No	11.75	5.88

Prestressed Concrete Boxes (Continued)

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
PS Box #40	M&M	MM-SB-A-18	Prestressed Concrete Boxes	b	2006 Design	HS25	LFD	37	80	No	9.25	4.63
PS Box #41	M&M	MM-SB-A-19	Prestressed Concrete Boxes	b	2006 Design	HS25	LFD	37	100	No	9.25	4.63
PS Box #42	M&M	MM-SB-A-20	Prestressed Concrete Boxes	b	2006 Design	HS25	LFD	37	120	No	5.29	2.64
PS Box #43	M&M	MM-SB-A-21	Prestressed Concrete Boxes	b	2006 Design	HS25	LFD	47	40	No	11.75	5.88
PS Box #44	M&M	MM-SB-A-22	Prestressed Concrete Boxes	b	2006 Design	HS25	LFD	47	60	No	9.4	4.7

Prestressed Concrete I-Beams

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
P/S I #3	M&M	MM-IB-L-01	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	60	No	7.75	3.875
P/S I #4	M&M	MM-IB-L-02	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	80	No	7.75	3.875
P/S I #5	M&M	MM-IB-L-03	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	100	No	7.75	3.875
P/S I #6	M&M	MM-IB-L-04	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	120	No	7.75	3.875
P/S I #7	M&M	MM-IB-L-05	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	140	No	7.75	3.875
P/S I #8	M&M	MM-IB-L-12	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	60	No	11.75	5.875
P/S I #9	M&M	MM-IB-L-13	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	80	No	11.75	5.875
P/S I #10	M&M	MM-IB-L-14	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	100	No	11.75	5.875
P/S I #11	M&M	MM-IB-L-15	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	120	No	11.75	5.875
P/S I #12	M&M	MM-IB-L-16	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	140	No	11.75	5.875
P/S I #13	M&M	MM-IB-L-17	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	160	No	9.4	4.7
P/S I #14	M&M	MM-IB-L-18	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	80	No	7.75	3.875
P/S I #15	M&M	MM-IB-L-19	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	100	No	7.75	3.875
P/S I #16	M&M	MM-IB-L-20	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	120	No	7.75	3.875
P/S I #17	M&M	MM-IB-L-21	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	140	No	7.75	3.875
P/S I #18	M&M	MM-IB-L-22	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	31	160	No	7.75	3.875
P/S I #19	M&M	MM-IB-L-29	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	80	No	11.75	5.875
P/S I #20	M&M	MM-IB-L-30	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	100	No	11.75	5.875
P/S I #21	M&M	MM-IB-L-31	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	120	No	11.75	5.875
P/S I #22	M&M	MM-IB-L-32	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	140	No	11.75	5.875

Prestressed Concrete I-Beams (Continued)

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
P/S I #23	M&M	MM-IB-L-33	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	160	No	9.4	4.7
P/S I #24	M&M	MM-IB-L-34	Prestressed Concrete I-Beams	k	2006 Design	HL-93	LRFD	47	180	No	7.833	3.9175
P/S I #25	M&M	MM-IB-A-1	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	31	80	No	7.75	3.875
P/S I #26	M&M	MM-IB-A-2	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	31	100	No	7.75	3.875
P/S I #27	M&M	MM-IB-A-3	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	31	120	No	7.75	3.875
P/S I #28	M&M	MM-IB-A-4	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	31	140	No	7.75	3.875
P/S I #29	M&M	MM-IB-A-5	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	31	160	No	6.2	3.1
P/S I #30	M&M	MM-IB-A-12	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	47	80	No	11.75	5.875
P/S I #31	M&M	MM-IB-A-13	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	47	100	No	11.75	5.875
P/S I #32	M&M	MM-IB-A-14	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	47	120	No	11.75	5.875
P/S I #33	M&M	MM-IB-A-15	Prestressed Concrete I-Beams	k	2006 Design	HS25	LFD	47	130	No	9.4	4.7

Reinforced Concrete Slabs

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
Slab #1	M&M	MM-Slab-L-01	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	15	No	N/A	N/A
Slab #2	M&M	MM-Slab-L-02	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	20	No	N/A	N/A
Slab #3	M&M	MM-Slab-L-03	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	25	No	N/A	N/A
Slab #4	M&M	MM-Slab-L-04	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	30	No	N/A	N/A
Slab #5	M&M	MM-Slab-L-05	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	35	No	N/A	N/A
Slab #6	M&M	MM-Slab-L-06	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	40	No	N/A	N/A
Slab #7	M&M	MM-Slab-L-07	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	45	No	N/A	N/A
Slab #8	M&M	MM-Slab-L-08	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	50	No	N/A	N/A
Slab #9	M&M	MM-Slab-L-09	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	55	No	N/A	N/A
Slab #10	M&M	MM-Slab-L-10	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	60	No	N/A	N/A
Slab #11	M&M	MM-Slab-L-11	Reinforced Concrete Slabs	N/A	2006 Design	HL-93	LRFD	31	65	No	N/A	N/A

Steel Welded Plate Girders

Number	Source	Identifier	Structure Type	LRFD Key	Year Built	Design Load	Design Method	Bridge Width	Span Lengths (ft)	Continuous	Girder Spacing (ft)	Overhang Width (ft)
Pl Girder #13	USS	III/3 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	80	Yes	9.25	5
Pl Girder #14	USS	III/3 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	80	Yes	9.25	5
Pl Girder #15	USS	III/5 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	90	Yes	9.25	5
Pl Girder #16	USS	III/5 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	90	Yes	9.25	5
Pl Girder #17	USS	III/7 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	100	Yes	9.25	5
Pl Girder #18	USS	III/7 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	100	Yes	9.25	5
Pl Girder #19	USS	III/11 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	120	Yes	9.25	5
Pl Girder #20	USS	III/11 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	120	Yes	9.25	5
Pl Girder #21	USS	IV/40 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	150	Yes	9.25	5
Pl Girder #22	USS	IV/40 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	150	Yes	9.25	5
Pl Girder #23	USS	III/182 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	135	Yes	9.25	5
Pl Girder #24	USS	IV/15 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	180	Yes	9.25	5
Pl Girder #25	USS	IV/15 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	180	Yes	9.25	5
Pl Girder #26	USS	IV/29 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	180	Yes	9.25	5
Pl Girder #27	USS	IV/29 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	202.5	Yes	9.25	5
Pl Girder #28	USS	IV/37 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	190	Yes	9.25	5
Pl Girder #29	USS	IV/37 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	215	Yes	9.25	5
Pl Girder #30	USS	V/15 Pos	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	250	Yes	9.25	5
Pl Girder #31	USS	V/15 Neg	Steel Welded Plate Girders	a	n/a	HS20	LFD	47	250	Yes	9.25	5

5.0 UNIVERSAL CALIBRATION PROCEDURE

The reliability analysis procedure applied in the previous calibration was based on the iterative procedure (Rackwitz and Fiessler). However, at this time the most efficient and accurate method is based on Monte Carlo simulations. In the past, the application of Monte Carlo technique was limited by the availability of fast computers. This is not a problem anymore and, therefore, Monte Carlo is a leading contender for consideration under this Task.

5.1 INTRODUCTION

The reliability analysis procedure applied in the previous calibration was based on a simplified calibration procedure based on Rackwitz and Fiessler procedure (NCHRP Report 368) (Rackwitz; Fiessler, 1977). The Rackwitz and Fiessler method is based on approximation of non-normal random variables by normal variables that satisfy special requirements. The approximating normal variables must have the same value of the cumulative distribution function (CDF) and probability density function (PDF) as the original (non-normal) variable at the so called "design point". Using the Rackwitz and Fiessler procedure, the reliability index and "design point" are determined by iterations. As indicated in Section 2, loads have been assumed to be normal and resistance lognormal. The lognormal distribution for resistance has to be approximated using a normal distribution at the "design point". Based on experience from extensive reliability analysis, it was found that for structural components the "design point" is equal to about 2 standard deviations from the mean. This observation allowed for derivation of a closed-form formula for the reliability index that has been used in bridge calibration with success (NCHRP Report 368). However, other methods are now widely used which eliminate the need to approximate the lognormal distribution.

At this time the most efficient and accurate method is based on Monte Carlo simulations. In the past, the application of Monte Carlo technique was limited by the availability of fast computers. This is not a problem anymore and, therefore, Monte Carlo is a leading contender for consideration under this Task.

5.2 SOLUTION METHODS FOR THE PROBABILITY OF FAILURE

5.2.1 Closed-form Solutions

The reliability index, β, is defined as

$$\beta = \phi^{-1}(P_f)$$

where ϕ^{-1} is the inverse of the standard normal distribution, and P_f is probability of failure.

If the limit state function can be expressed in terms of two random variables, R representing the resistance and Q representing the load effect,

g = R - Q

then the probability of failure is

P_f = Prob (g < 0)

Then, the reliability index, β, can be calculated using a closed-form formula in two cases: when both R and Q are normal random variables, or when both R and Q are lognormal random variables. In all other cases, the available procedures produce approximate results.

In case of R and Q both being normal random variables, the reliability index, β, can be calculated using the following formula,

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R^{\ 2} + \sigma_Q^{\ 2}}} \tag{8}$$

where:

$\overline{R}$ = mean or expected value of the distribution of resistance
$\overline{Q}$ = mean or expected value of the distribution of load
σ_R = standard deviation of the distribution of resistance
σ_Q = standard deviation of the distribution of load

Sometimes, $R - Q$ is termed M, the margin of safety. Using this terminology the equation becomes,

$$\beta = \frac{\overline{M}}{\sigma_M} \tag{9}$$

For the case when both distributions are lognormally distributed, a more complete derivation of the closed form solutions and how they can be applied to LRFD calibration is shown by Allen, et al. (2005).

In practice, either load or resistance or both are not normally distributed. Using distributions to model load and resistance that are convenient only for computational purposes should be avoided. Practicing engineers have sometimes used these closed-form solutions, which were intended only for illustrative purposes, for code calibration. Computer-based numerical integration methods such as Monte Carlo simulation are available and more useful for a variety or combination of distributions.

5.2.2 Monte Carlo Simulation: *The Preferred Calibration Tool*

5.2.2.1 General

Rather than utilizing a more limited closed-form solution (one that assumes like distributions for load and resistance), Monte Carlo simulation provides a powerful method to solve the problem of determining the failure rate numerically. Many times, MS Excel® is used to perform Monte Carlo simulation due to the software's universality.

Code calibration is accomplished in an iterative process by assuming agreeable load and resistance factors, γ's and ϕ's, and determining the resultant reliability index, β. When the desired target reliability index, β_T, is achieved, an acceptable set of load and resistance factors has been determined. One unique set of load and resistance factors does not exist; different sets of factors can achieve the same target reliability index.

The typical application of Monte Carlo simulation for bridge-structures reliability as reported in the literature (Allen, et al. 2005; Nowak and Collins 2000) is quite simple, as follows:

- Load is usually assumed to be normally distributed. The load side of the LRFD equation is a summation of force effects and thus has been assumed to be normally distributed.
- Resistance is usually assumed to be lognormally distributed. The resistance side of the LRFD equation is a product of terms and thus has been assumed to be lognormally distributed.
- Using the reported statistics of load and resistance and computer-generated random numbers, the distributions of load and resistance are basically reconstituted and values chosen randomly from these reconstituted distributions. For example for the simple load combination of dead load plus live load, random values of dead load and live load are chosen from the reconstituted normal distributions of dead and live load respectively. A random value of resistance is chosen from the reconstituted lognormal distribution of resistance.
- The limit-state function, $R_i - (D_i + L_i)$ is calculated. If the value is equal to or greater than zero, the function is satisfied and the individual case is safe. If the value is negative, the criterion is not satisfied and it fails.
- After a number of iterations, the failures are counted and the failure rate determined. For the sampling to be significant at least ten failures should be observed, otherwise, more iteration is necessary.
- Using the failure rate, the reliability index is determined as the inverse of the standard normal cumulative distribution.

Several observations can be made regarding this process:

The solution is only as good as the modeling of the distribution of load and resistance. If load is not correctly modeled as normally distributed or resistance is not correctly modeled as lognormally distributed, the solution is not accurate.

Further, if the statistical parameters (in other words, mean, standard deviation and bias) are not well defined, the solution is equally inaccurate.

If both the distribution of load and resistance are normally or lognormally distributed then the closed-form formulas provide accurate results.

If this is not the case, then Monte Carlo simulation can produce accurate solutions. The accuracy depends on the number of simulations.

The power of the Monte Carlo simulation is its ability to use a variety of distributions for load and resistance.

In the researcher's opinion, refinement in the calibration should be pursued in the determination of the more accurate statistical parameters of the various components of force effect and resistance rather than development of reliability analysis methods. The Monte Carlo simulation as discussed above is quite adequate and understandable to the practicing engineer.

5.2.2.2 Example

An example Monte Carlo simulation is included in the form of an adaptation of the Excel® spreadsheet developed and validated during NCHRP 20-7/122. In this example, it has been assumed that the loads are normally distributed, and the resistance is lognormally distributed. Determination of the failure rate and the associated reliability index, β, through Monte Carlo simulation using MS Excel started with the following 15-step computational procedure adapted from Nowak and Collins, 2000:

1. determine the nominal dead load, D_n, the nominal live load plus impact, L_n, and the nominal resistance, R_n, for the subject bridge according to the *LRFD Bridge Design Specifications*.
2. assume i=1.
3. generate a uniformly distributed random number $0 \leq u_{Di} \leq 1$ using the command RAND.
4. calculate the corresponding value of D_i (a normal random variable)

   $$D_i = \mu_D + \sigma_D \, \phi^{1}(u_{Di})$$

   where ϕ^{-1} = the inverse standard normal distribution function calculated using the command NORMSINV

   $\mu_D = \lambda_D D_n$

   $\sigma_D = V_D \mu_D$.
5. generate a uniformly distributed random number $0 \leq u_{Li} \leq 1$ using the command RAND.
6. calculate the corresponding value of L_i (a normal random variable)

   $$L_i = \mu_L + \sigma_L \, \phi^{-1}(u_{Li})$$

   where ϕ^{-1} = the inverse standard normal distribution function calculated using the command NORMSINV

   $\mu_L = \lambda_L L_n$

   $\sigma_L = V_L \mu_L$.
7. generate a uniformly distributed random number $0 \leq u_{Ri} \leq 1$ using the command RAND.

8. calculate the corresponding value of R_i (a lognormal random variable)

   $R_i = \exp(\mu_{lnR} + \sigma_{lnR}\, \phi^{-1}(u_{Ri}))$

   where ϕ^{-1} = the inverse standard normal distribution function calculated using the command NORMSINV

   μ_{lnR} = $\ln(\mu_R)$- ½ σ_{lnR}^2

   σ_{lnR} = $(\ln(V_R^2+1))^{1/2}$.
9. calculate the limit state function, $Y_i = R_i - (D_i + L_i)$, and save the value.
10. assume i=i+1, go back to step 3 and iterate until the desired number of simulations, N, is obtained.
11. rearrange the values of Y_i in ascending order using the command RANK and re-assign the values of i in ascending order also.
12. calculate the probabilities

    $p_i = i/(1+N)$.
13. calculate the corresponding values of the inverse standard normal distribution function, $\phi^{-1}(p_i)$ using the command NORMSINV.
14. plot $\phi^{-1}(p_i)$ versus Y_i , the resulting curve is the cumulative distribution function of Y.
15. the reliability index, β, is equal to the negative value of the plotted cumulative distribution function for Y = 0.

In this case, the spreadsheet was expanded to include two components of dead load, DC and DW as defined in the *LRFD Bridge Design Specifications*, as opposed to a single component of dead load. The first page of the spreadsheet is illustrated in Table 6.

Table 6 - Partial Output from Monte Carlo Spreadsheet

DCn	DWn	Ln	Rn	bias DC	COV DC	bias DW	COV DW	bias L	COV L	bias R	COV R
8496	1493	7120	26585	1.05	0.1	1	0.25	1.18	0.18	1.12	0.1

uDCi	mu DC	DCi	uDWi	mu DW	DWi	uLi	mu L	Li	uRi	mu lnR	sigma lnR	Ri	Yi	rate	
0.419878	8920.8	8740.416	0.443836	1493	1471.911	0.216984	8401.6	7218.358	0.358719	10.29646	0.099751	28577	11146.31	0.00026	-3.47024
0.111041	8920.8	7831.559	0.118536	1493	1316.477	0.015878	8401.6	5154.003	0.582538	10.29646	0.099751	30249.75	15947.71		
0.100961	8920.8	7782.42	0.722376	1493	1581.074	0.428567	8401.6	8129.352	0.280095	10.29646	0.099751	27954.82	10461.97		
0.528656	8920.8	8984.934	0.637979	1493	1545.712	0.354282	8401.6	7836.33	0.728889	10.29646	0.099751	31484.48	13117.5		
0.934025	8920.8	10264.68	0.693445	1493	1568.492	0.550473	8401.6	8593.443	0.751907	10.29646	0.099751	31708.4	11281.79		
0.434034	8920.8	8772.615	0.757603	1493	1597.303	0.435846	8401.6	8157.35	0.277691	10.29646	0.099751	27934.87	9407.605		
0.852775	8920.8	9856.065	0.534933	1493	1506.09	0.681061	8401.6	9113.383	0.910046	10.29646	0.099751	33868.01	13392.47		
0.51019	8920.8	8943.588	0.745512	1493	1591.603	0.533723	8401.6	8529.588	0.69696	10.29646	0.099751	31191.33	12126.55		
0.117619	8920.8	7861.926	0.041384	1493	1233.987	0.589223	8401.6	8742.69	0.01176	10.29646	0.099751	23636.12	5797.513		
0.927962	8920.8	10223.93	0.462024	1493	1478.766	0.239514	8401.6	7331.104	0.773615	10.29646	0.099751	31931.55	12897.74		
0.265925	8920.8	8363.084	0.991551	1493	1849.667	0.221657	8401.6	7242.266	0.751173	10.29646	0.099751	31701.08	14246.06		
0.251008	8920.8	8321.928	0.228195	1493	1381.801	0.867372	8401.6	10086.37	0.821511	10.29646	0.099751	32478.73	12688.63		
0.418034	8920.8	8736.206	0.20623	1493	1370.638	0.157379	8401.6	6881.315	0.4644	10.29646	0.099751	29364.52	12376.37		
0.388982	8920.8	8669.258	0.862609	1493	1656.053	0.119696	8401.6	6622.384	0.102098	10.29646	0.099751	26102.93	9155.24		
0.804138	8920.8	9684.861	0.075045	1493	1278.126	0.248983	8401.6	7376.734	0.069405	10.29646	0.099751	25560.43	7220.704		
0.363862	8920.8	8610.217	0.974877	1493	1785.308	0.280806	8401.6	7523.792	0.836821	10.29646	0.099751	32674.8	14755.49		
0.07622	8920.8	7644.262	0.891541	1493	1677.351	0.126225	8401.6	6670.909	0.397258	10.29646	0.099751	28867.61	12875.09		
0.459669	8920.8	8830.461	0.207193	1493	1371.141	0.858107	8401.6	10022.55	0.074785	10.29646	0.099751	25660.53	5436.374		
0.847954	8920.8	9837.587	0.298837	1493	1414.207	0.683577	8401.6	9124.058	0.909925	10.29646	0.099751	33865.5	13489.65		
0.502928	8920.8	8927.348	0.578845	1493	1522.702	0.122269	8401.6	6641.726	0.315039	10.29646	0.099751	28237.72	11145.95		
0.820307	8920.8	9738.425	0.285523	1493	1408.42	0.678002	8401.6	9100.458	0.496364	10.29646	0.099751	29600.51	9353.209		
0.131983	8920.8	7924.289	0.328578	1493	1426.734	0.981031	8401.6	11540.38	0.807284	10.29646	0.099751	32306.81	11415.4		
0.167675	8920.8	8061.377	0.895839	1493	1680.848	0.9249	8401.6	10577.51	0.039432	10.29646	0.099751	24863.62	4543.882		
0.247371	8920.8	8311.701	0.035391	1493	1223.234	0.647096	8401.6	8972.476	0.592051	10.29646	0.099751	30323.56	11816.15		
0.746738	8920.8	9513.373	0.862869	1493	1656.23	0.039703	8401.6	5748.836	0.101126	10.29646	0.099751	26088.68	9170.238		
0.674224	8920.8	9323.67	0.06162	1493	1262.882	0.524304	8401.6	8493.788	0.88772	10.29646	0.099751	33443.19	14362.85		
0.902065	8920.8	10074.62	0.54536	1493	1510.012	0.19903	8401.6	7123.579	0.461989	10.29646	0.099751	29346.75	10638.54		
0.418746	8920.8	8737.831	0.062696	1493	1264.194	0.857793	8401.6	10020.44	0.92424	10.29646	0.099751	34184.16	14161.7		
0.028162	8920.8	7218.244	0.105431	1493	1306.196	0.146125	8401.6	6808.861	0.819404	10.29646	0.099751	32452.69	17119.38		
0.265726	8920.8	8362.545	0.227722	1493	1381.567	0.037669	8401.6	5712.144	0.583535	10.29646	0.099751	30257.46	14801.2		
0.177351	8920.8	8095.172	0.269812	1493	1401.422	0.202592	8401.6	7142.773	0.308196	10.29646	0.099751	28183.26	11543.89		

Three composite steel plate girders from the proposed bridge database were analyzed to determine the reliability index, β. The results are tabulated in Table 7. the target reliability index of the *LRFD Bridge Design Specifications* is 3.5.

Table 7 - Results of Sample Monte Carlo Simulation Analysis

Number	Identifier	DC	DW	LL	RN	Failures	Simulations	Beta
Pl Girder #10	US-59 Span 3 Interior +	9071	1247	5332	23667	30	100,000	3.43
Pl Girder #13	US-59 Span 5 Interior -	27017	3529	11521	62188	33	100,000	3.41
Pl Girder #14	US-59 Span 3 Exterior +	8496	1493	7120	26585	26	100,000	3.47

The statistics used in the presented Monte Carlo simulations are identical to those of NCHRP 20-7/122 except the value of the COV for LL+I was chosen as 0.18. For simplicity, the example is based on a bias factor of 1.18 applied when LL+I are entered into the simulation. The bias factor value of 1.3 as reported in the literature is believed by the research team to be appropriate when only the value of HL93 LL without impact is entered into the spreadsheet. Actually, the biases for HL93 are somewhat span dependent and quite span dependent for HS loading as shown in Table 2 and Figure 2.

5.3 MONTE CARLO ANALYSIS OF RELIABILITY INDICES FOR THE NEW BRIDGE DATABASE

The Monte Carlo method of simulation as described above was applied to the 124 bridges in the bridge database documented in Section 4.2. Figure 5 and Table 8 show the beta factors for the entire set of bridges in the database. In general, the range of results is similar to that obtained in the original Calibration Report, some additional trends were noted as will be discussed in the text that accompanies the figures that follow.

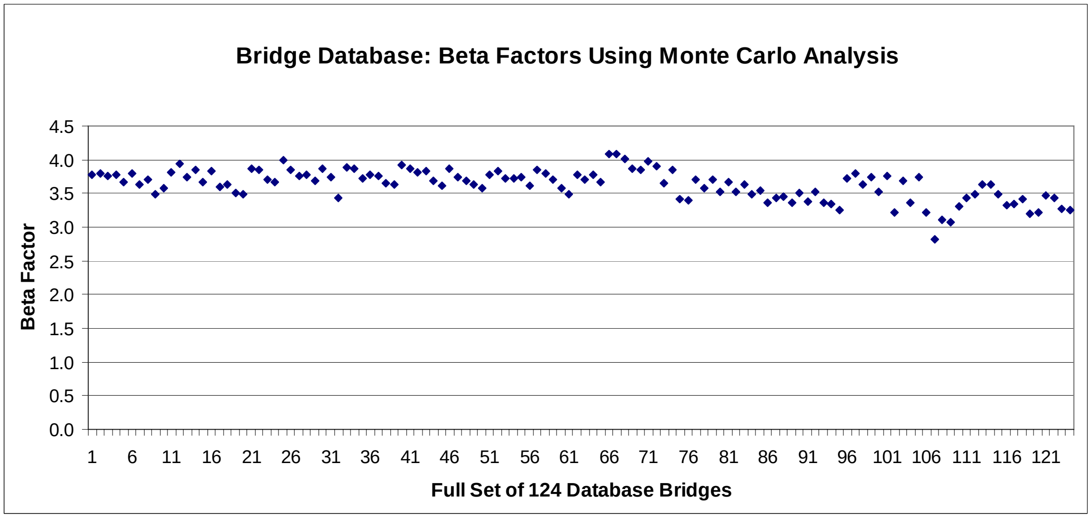


Figure 5 – Monte Carlo Results for All Bridges in the Database

Table 8 - Database Bridges: Beta Factors using Monte-Carlo Simulation

Bridge	Identifier	Type	Span Length (FT)	Beta
PS Box #1	N54-PA-G1	Prestressed Concrete Boxes	42	3.72
PS Box #2	N54-PA-G2	Prestressed Concrete Boxes	42	3.80
PS Box #3	N45-PA-G1	Prestressed Concrete Boxes	47	3.63
PS Box #4	N45-PA-G2	Prestressed Concrete Boxes	47	3.74
PS Box #5	N50-PA-G1	Prestressed Concrete Boxes	52	3.53
PS Box #6	N50-PA-G2	Prestressed Concrete Boxes	52	3.77
PS Box #7	N51-PA-G1	Prestressed Concrete Boxes	72	3.22
PS Box #8	N51-PA-G2	Prestressed Concrete Boxes	72	3.69
PS Box #9	N47-PA-G1	Prestressed Concrete Boxes	72	3.37
PS Box #10	N47-PA-G2	Prestressed Concrete Boxes	72	3.75
PS Box #12	MM-AB-L-01	Prestressed Concrete Boxes	50	3.77
PS Box #13	MM-AB-L-16	Prestressed Concrete Boxes	50	3.80
PS Box #14	MM-AB-L-02	Prestressed Concrete Boxes	60	3.77
PS Box #15	MM-AB-L-17	Prestressed Concrete Boxes	60	3.77
PS Box #16	MM-AB-L-23	Prestressed Concrete Boxes	80	3.67
PS Box #17	MM-AB-L-18	Prestressed Concrete Boxes	80	3.79
PS Box #18	MM-AB-L-14	Prestressed Concrete Boxes	100	3.63
PS Box #19	MM-AB-L-19	Prestressed Concrete Boxes	100	3.70
PS Box #20	MM-AB-L-05	Prestressed Concrete Boxes	120	3.48
PS Box #21	MM-AB-L-10	Prestressed Concrete Boxes	120	3.58
PS Box #22	MM-AB-A-1	Prestressed Concrete Boxes	50	3.82

Bridge	Identifier	Type	Span Length (FT)	Beta
PS Box #23	MM-AB-A-16	Prestressed Concrete Boxes	50	3.95
PS Box #24	MM-AB-A-2	Prestressed Concrete Boxes	60	3.74
PS Box #25	MM-AB-A-17	Prestressed Concrete Boxes	60	3.86
PS Box #26	MM-AB-A-23	Prestressed Concrete Boxes	80	3.66
PS Box #27	MM-AB-A-18	Prestressed Concrete Boxes	80	3.83
PS Box #28	MM-AB-A-14	Prestressed Concrete Boxes	100	3.60
PS Box #29	MM-AB-A-19	Prestressed Concrete Boxes	100	3.64
PS Box #30	MM-AB-A-5	Prestressed Concrete Boxes	120	3.50
PS Box #31	MM-AB-A-10	Prestressed Concrete Boxes	120	3.49
PS Box #32	MM-SB-L-07	Prestressed Concrete Boxes	60	3.87
PS Box #33	MM-SB-L-08	Prestressed Concrete Boxes	80	3.86
PS Box #34	MM-SB-L-09	Prestressed Concrete Boxes	100	3.70
PS Box #35	MM-SB-L-10	Prestressed Concrete Boxes	120	3.67
PS Box #36	MM-SB-L-26	Prestressed Concrete Boxes	40	3.99
PS Box #37	MM-SB-L-27	Prestressed Concrete Boxes	60	3.86
PS Box #38	MM-SB-L-28	Prestressed Concrete Boxes	80	3.76
PS Box #39	MM-SB-L-29	Prestressed Concrete Boxes	100	3.78
PS Box #40	MM-SB-L-30	Prestressed Concrete Boxes	120	3.69
PS Box #41	MM-SB-A-18	Prestressed Concrete Boxes	80	3.87
PS Box #42	MM-SB-A-19	Prestressed Concrete Boxes	100	3.75
PS Box #43	MM-SB-A-20	Prestressed Concrete Boxes	120	3.43
PS Box #44	MM-SB-A-21	Prestressed Concrete Boxes	40	3.89
PS Box #45	MM-SB-A-22	Prestressed Concrete Boxes	60	3.87
CA Box #1	N60-CA-G1	"California" Prestressed Concrete Boxes	125	3.23
CA Box #2	BR No 5 Airport Rd	"California" Prestressed Concrete Boxes	130	2.82
CA Box #3	7A: S-2492	"California" Prestressed Concrete Boxes	75	3.10
CA Box #4	7B: S-2492	"California" Prestressed Concrete Boxes	75	3.07
CA Box #5	Br. No 4 Nauvoo Rd	"California" Prestressed Concrete Boxes	110	3.31
P/S I #2	US 59, Spans 1,2 (Int)	Prestressed Concrete I-Beams	134	3.44
P/S I #3	US 59, Spans 1,2 (Ext)	Prestressed Concrete I-Beams	134	3.49
P/S I #4	MM-IB-L-01	Prestressed Concrete I-Beams	60	3.72

Bridge	Identifier	Type	Span Length (FT)	Beta
P/S I #5	MM-IB-L-02	Prestressed Concrete I-Beams	80	3.77
P/S I #6	MM-IB-L-03	Prestressed Concrete I-Beams	100	3.76
P/S I #7	MM-IB-L-04	Prestressed Concrete I-Beams	120	3.65
P/S I #8	MM-IB-L-05	Prestressed Concrete I-Beams	140	3.64
P/S I #9	MM-IB-L-12	Prestressed Concrete I-Beams	60	3.93
P/S I #10	MM-IB-L-13	Prestressed Concrete I-Beams	80	3.87
P/S I #11	MM-IB-L-14	Prestressed Concrete I-Beams	100	3.82
P/S I #12	MM-IB-L-15	Prestressed Concrete I-Beams	120	3.83
P/S I #13	MM-IB-L-16	Prestressed Concrete I-Beams	140	3.69
P/S I #14	MM-IB-L-17	Prestressed Concrete I-Beams	160	3.61
P/S I #15	MM-IB-L-18	Prestressed Concrete I-Beams	80	3.87
P/S I #16	MM-IB-L-19	Prestressed Concrete I-Beams	100	3.74
P/S I #17	MM-IB-L-20	Prestressed Concrete I-Beams	120	3.69
P/S I #18	MM-IB-L-21	Prestressed Concrete I-Beams	140	3.63
P/S I #19	MM-IB-L-22	Prestressed Concrete I-Beams	160	3.58
P/S I #20	MM-IB-L-29	Prestressed Concrete I-Beams	80	3.77
P/S I #21	MM-IB-L-30	Prestressed Concrete I-Beams	100	3.83
P/S I #22	MM-IB-L-31	Prestressed Concrete I-Beams	120	3.73
P/S I #23	MM-IB-L-32	Prestressed Concrete I-Beams	140	3.72
P/S I #24	MM-IB-L-33	Prestressed Concrete I-Beams	160	3.75
P/S I #25	MM-IB-L-34	Prestressed Concrete I-Beams	180	3.61
P/S I #26	MM-IB-A-1	Prestressed Concrete I-Beams	80	3.84
P/S I #27	MM-IB-A-2	Prestressed Concrete I-Beams	100	3.80
P/S I #28	MM-IB-A-3	Prestressed Concrete I-Beams	120	3.71
P/S I #29	MM-IB-A-4	Prestressed Concrete I-Beams	140	3.59
P/S I #30	MM-IB-A-5	Prestressed Concrete I-Beams	160	3.49
P/S I #31	MM-IB-A-12	Prestressed Concrete I-Beams	80	3.78
P/S I #32	MM-IB-A-13	Prestressed Concrete I-Beams	100	3.70
P/S I #33	MM-IB-A-14	Prestressed Concrete I-Beams	120	3.77
P/S I #34	MM-IB-A-15	Prestressed Concrete I-Beams	130	3.67
Slab #10	MM-Slab-L-01	Reinforced Concrete Slabs	15	4.08
Slab #11	MM-Slab-L-02	Reinforced Concrete Slabs	20	4.09
Slab #12	MM-Slab-L-03	Reinforced Concrete Slabs	25	4.02
Slab #13	MM-Slab-L-04	Reinforced Concrete Slabs	30	3.87
Slab #14	MM-Slab-L-05	Reinforced Concrete Slabs	35	3.85
Slab #15	MM-Slab-L-06	Reinforced Concrete Slabs	40	3.97
Slab #16	MM-Slab-L-07	Reinforced Concrete Slabs	45	3.91
Slab #17	MM-Slab-L-08	Reinforced Concrete Slabs	50	3.65

Bridge	Identifier	Type	Span Length (FT)	Beta
Slab #18	MM-Slab-L-09	Reinforced Concrete Slabs	55	3.85
Slab #19	MM-Slab-L-10	Reinforced Concrete Slabs	60	3.41
Slab #20	MM-Slab-L-11	Reinforced Concrete Slabs	65	3.40
Pl Girder #1	10044	Steel Welded Plate Girders	103	3.63
Pl Girder #2	4200	Steel Welded Plate Girders	108	3.64
Pl Girder #6	4201	Steel Welded Plate Girders	114	3.49
Pl Girder #7	10045	Steel Welded Plate Girders	160	3.33
Pl Girder #10	US-59, Span 3, Int+	Steel Welded Plate Girders	250	3.34
Pl Girder #11	US-59, Span 5, Int+	Steel Welded Plate Girders	325	3.42
Pl Girder #12	US-59, Span 3, Int-	Steel Welded Plate Girders	275	3.20
Pl Girder #13	US-59, Span 5, Int-	Steel Welded Plate Girders	338	3.22
Pl Girder #14	US-59, Span 3, Ext+	Steel Welded Plate Girders	250	3.48
Pl Girder #15	US-59, Span 5, Ext+	Steel Welded Plate Girders	325	3.44
Pl Girder #16	US-59, Span 3, Ext-	Steel Welded Plate Girders	275	3.28
Pl Girder #17	US-59, Span 5, Ext-	Steel Welded Plate Girders	338	3.26
Pl Girder #26	III/3 Pos	Steel Welded Plate Girders	80	3.70
Pl Girder #27	III/3 Neg	Steel Welded Plate Girders	80	3.58
Pl Girder #28	III/5 Pos	Steel Welded Plate Girders	90	3.71
Pl Girder #29	III/5 Neg	Steel Welded Plate Girders	90	3.53
Pl Girder #30	III/7 Pos	Steel Welded Plate Girders	100	3.66
Pl Girder #31	III/7 Neg	Steel Welded Plate Girders	100	3.52
Pl Girder #32	III/11 Pos	Steel Welded Plate Girders	120	3.63
Pl Girder #33	III/11 Neg	Steel Welded Plate Girders	120	3.49
Pl Girder #34	IV/40 Pos	Steel Welded Plate Girders	150	3.55
Pl Girder #35	IV/40 Neg	Steel Welded Plate Girders	150	3.36
Pl Girder #36	III/182 Neg	Steel Welded Plate Girders	135	3.44
Pl Girder #37	IV/15 Pos	Steel Welded Plate Girders	180	3.46
Pl Girder #38	IV/15 Neg	Steel Welded Plate Girders	180	3.37
Pl Girder #39	IV/29 Pos	Steel Welded Plate Girders	180	3.51
Pl Girder #40	IV/29 Neg	Steel Welded Plate Girders	203	3.38
Pl Girder #41	IV/37 Pos	Steel Welded Plate Girders	190	3.53

Bridge	Identifier	Type	Span Length (FT)	Beta
Pl Girder #42	IV/37 Neg	Steel Welded Plate Girders	215	3.36
Pl Girder #43	V/15 Pos	Steel Welded Plate Girders	250	3.35
Pl Girder #44	V/15 Neg	Steel Welded Plate Girders	250	3.25

Figure 6 below shows the relationship between the beta factors and the span length. While the values are clustered around the target beta factor of 3.5, there is a general decrease in the beta factors as the span increases which suggests that there may be a correlation between the dead to live load ratio and the beta factor. Figures 7 and 8 further illustrate that correlation.

The beta factors shown in Figure 8 reflect that for concrete bridges, the dead to live load ratio is higher than for steel which results in somewhat higher beta factors for the steel bridges than for concrete bridges with a similar span length. However, in making such comparisons, the following should be noted. In reviewing the beta factors for the bridge database, the bridges having beta factors that were somewhat low (i.e., less than 3.25) consist of 1970s era California-style box beams with D/L ratios ranging from 2.9 to 3.9.

If the construction tolerances for the California-style boxes were considered to be comparable to that for a factory-made component, this would justify a slight decrease in the statistical parameters for bias and coefficient of variation. An analysis was performed using such an assumption which resulted in an increase in the beta factors for these bridges of approximately 0.3. This brings the range of beta factors for these bridges from 2.8 to 3.3 up to 3.2 to 3.6..

Figure 9 shows the beta factors for various types of prestressed concrete girders. Again, the trends exhibited are primarily related to the increase in the D/L ratio with increasing span length. The beta factors for this set of bridges were determined using the appropriate bias factors and coefficients of variation for the factory-made and field-made bridge components. It should be noted that analyses were also performed with “weighted average” values for these statistical factors and the resulting beta values were considerably lower for the bridges having high dead to live load ratios. This highlights the importance of differentiating these statistical parameters in performing the calibration because slight changes to these parameters do have a significant impact on the results.

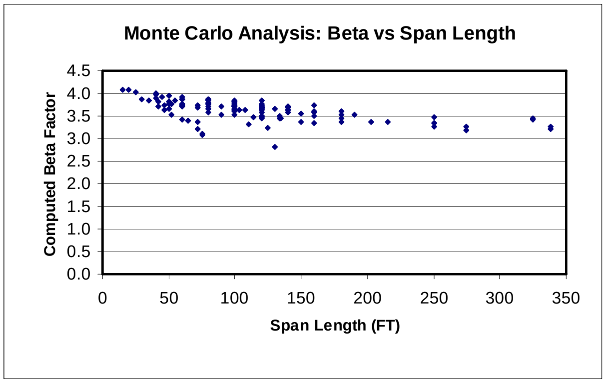


Figure 6 – Beta Factors as a Function of Span Length

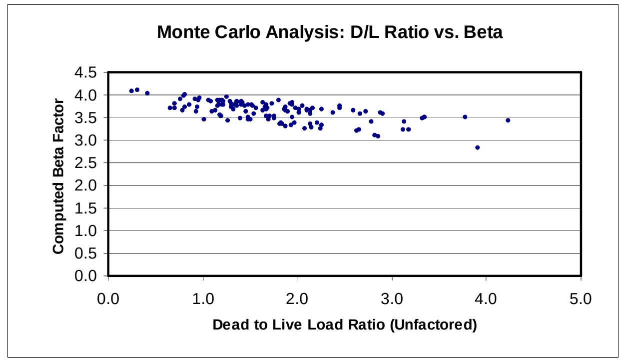


Figure 7 – Beta Factor vs. D/L Ratio

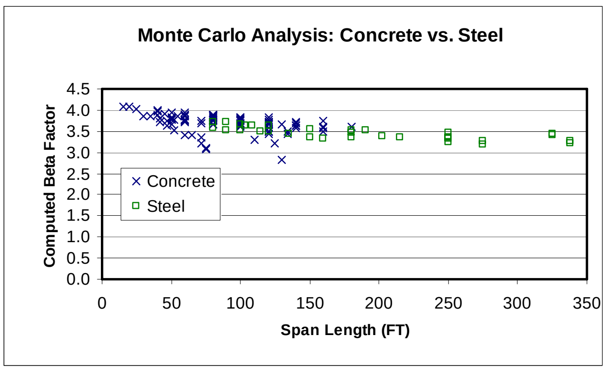


Figure 8 – Beta Factors: Concrete vs. Steel

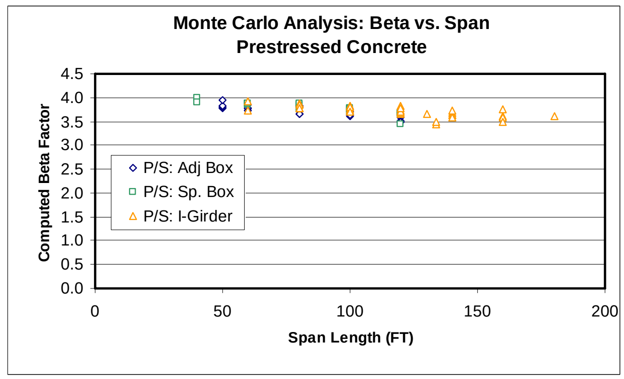


Figure 9 – Beta Factors for Various Types of P/S Concrete Girders

6.0 ANALYSIS OF THE SENSITIVITY OF RELIABILITY INDEX TO CHANGES IN LOAD AND RESISTANCE FACTORS

6.1 INTRODUCTION

During NCHRP 12-33, a series of simple span hypothetical bridges were developed which had a uniform change in span and spacing as follows:

- Prestressed concrete I-beam bridges with spans of 30, 60, 90, 120, 200 ft.
- Reinforced concrete T-beams with spans of 30, 60, 90, 120 ft.
- Noncomposite steel girders with spans of 30, 60, 90, 120, 200 ft.
- Composite steel girders with spans of 30, 60, 90, 120, 200 ft.

For each of the materials and spans indicated above, the spacings of girders was varied as follows: 4, 6, 8, 10, 12 ft. Reliability indices were calculated for each of these combinations for both shear and moment, with the exception of the composite steel girders for which only reliability indices for moment were calculated as the shear capacity is the same as for the noncomposite girders.

In the process, it was observed that the variation of reliability index for spacing change was quite small.

6.2 ADEQUACY OF RACKWITZ AND FIESSLER PROCEDURE FOR THIS STUDY

A study was made comparing the results of a million iterations of the Monte Carlo simulation to the results of the Rackwitz and Fiessler procedure embodied in spreadsheets used in the original calibration of the LRFD Specification. The sole purpose of this analysis was to verify that the Monte Carlo and Rackwitz and Fiessler methods would give sufficiently smaller results for the cases studied to enable an efficient demonstration of the sensitivity of computed values of the reliability index, beta, for variations in load and resistance factors which is reported in the next section. Figure 10 shows the result of that analysis for each of the structures identified above for bending moment. A perfect comparison between the two methods would be indicated by a 100 percent value on Figure 10. Each of the dots are actually values for each of the spacings identified above. Thus in the top panel of Figure 10, the left most dot is actually five data points that are so close together that the difference cannot be discerned in the plot. The small differences that do exist can be attributed in part to the slight difference in the statistical parameters used between the two methods. In the implementation of the Rackwitz-Fiessler procedure, Nowak differentiated between the dead load due to factory-built and field-built components with slightly different values of bias and coefficients of variation. The Monte Carlo simulations presented herein combined these two load components and used “average” values for the bias and coefficient of variation. For the DW-type dead load, both methods used the same values for the bias and coefficient of variation for this load component. Figure 10 indicates that the results are in sufficient agreement to proceed with the Rackwitz and Fiessler method for the sensitivity studies that follow.

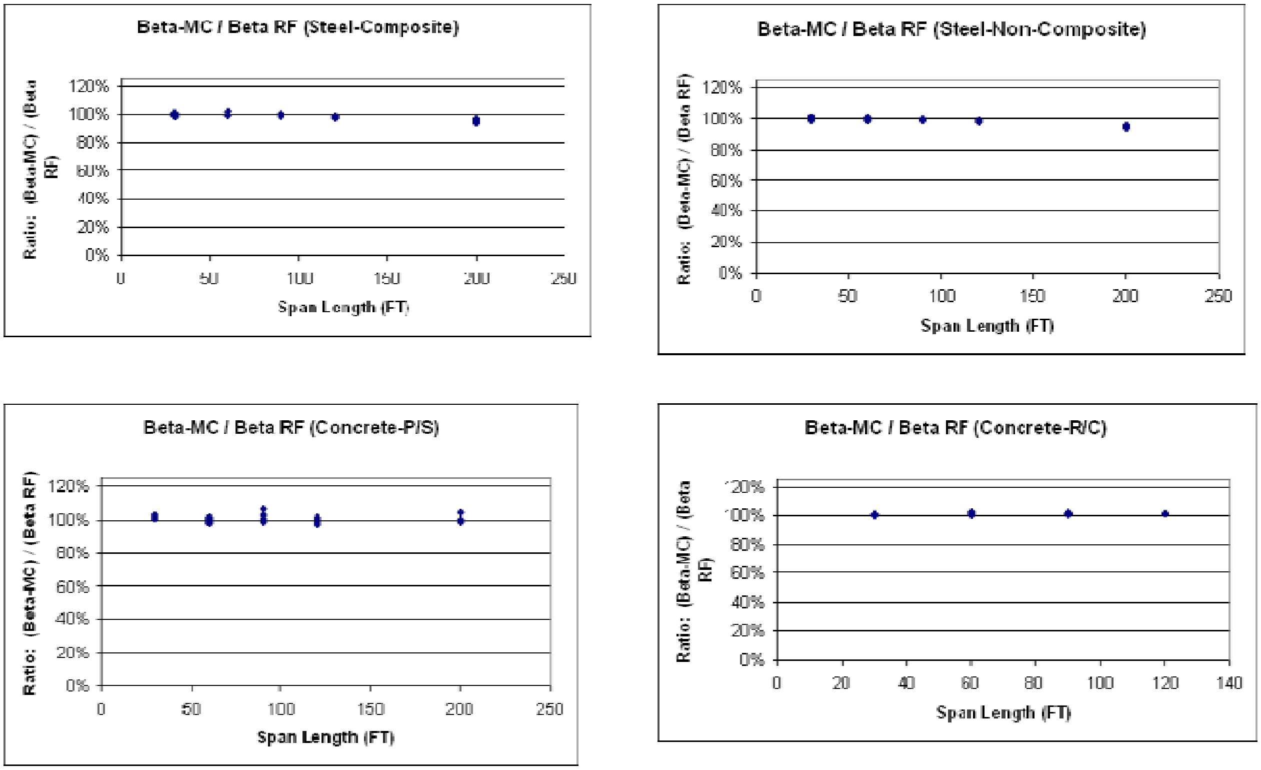


Figure 10 – Comparison of the Results of the Monte Carlo Simulation and Rackwitz and Fiessler Method

6.3 SENSITIVITY STUDY

In the reported study, a systematic variation of load and resistance factors was made for each of the structures indicated in the bullet list above and for each of the spaces indicated. For the purpose of summarizing the results, reliability index for each of the span lengths and materials indicated above was calculated by averaging the results for the girder spacings considered. The parameters varied are indicated in the following equation which will be recognized as a variation of Strength Limit State Load Combination I from Table 3.4.1-1 of AASHTO LRFD:

$$\eta\ [(D_1\ D_2\ \text{Scalar}) * 1.25\ (D_1 + D_2) + (D_A\ \text{Scalar}) * 1.5\ D_A + (L\ \text{Scalar}) * 1.75\ (L + I)] = \phi\ R \quad (10)$$

where:

(D_1 D_2 Scalar) = a factor applied to the load factor for site and factory made dead load components

(D_A Scalar) = a factor applied to the load factor for the weight of asphaltic wearing sufaces and utilities

(L Scalar) = a factor applied to the load factor for live load plus impact

η	=	an outside load factor used in Eq. 1-1 of AASHTO LRFD to represent the effects of importance, ductility and redundancy, used herein as a scalar on all load factors
ϕ	=	resistance factor

Using these various scalars, it is possible to investigate the change in reliability index (β) caused by a change of load factors, resistance factors, and partial load factors on dead load and live load. The reliability indices were calculated using a variation of the original NCHRP 12-33 spread sheets. These spread sheets were varied in two cycles. In 1996, a variation was made to include a variable factor η. In 2006, all of the other scalar factors identified above were added.

Tabular data and graphs in Figures 11 through 16 show the variation of reliability index for a systematic variation of the scalar parameters as indicated on the figures. Each curve represents the average of the spacings considered, but as indicated previously, the variation in β was extremely small with the change in spacing, and, thus, an average value is adequate to show the variation in β. Figures 11 through 16 contain the following information:

- Figure 11 shows the effect of varying the scalar η on reliability index for bending moment.
- Figure 12 shows the effect of varying the scalar η on reliability index for shear.
- Figure 13 shows the effect of varying the dead load scalars on the reliability index for moment.
- Figure 14 shows the effect of varying the dead load scalar on reliability indices for shear.
- Figure 15 shows the effect of varying the live load scalar on the reliability indices for moment.
- Figure 16 shows the effect of varying the live load scalar on the reliability indices for shear.

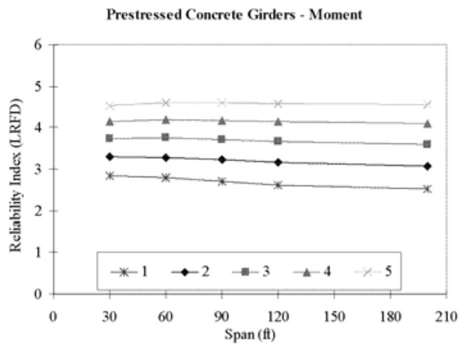
The following observations can be made:

- Varying the effect of all of the load factors by using the η scalar produced a relatively uniform parallel offset in each of the curves in Figures 11 and 12. It is noted that varying the resistance factor ϕ by the same percentage as the η factors were varied produces identical results, i.e., multiplying the η factor by 1.10% is equivalent to dividing the resistance factor by 1.10%.
- Figures 13 and 14 indicate that varying the dead load scalars produce an increasing effect on the reliability index as the span length increases. This is entirely logical because as the span increases the dead load moment and shear get proportionately larger compared to the live load.
- The opposite effect is observed in Figures 15 and 16 wherein as the span length increases the reliability indices tend to converge showing that as the span length increases the affect of live load becomes increasingly less compared to the dead load.

Prestressed Concrete Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

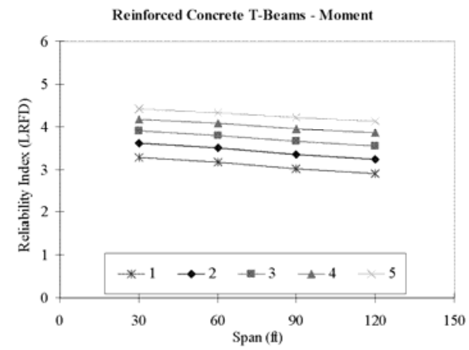
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	2.85	3.30	3.73	4.13	4.52
60	2.79	3.28	3.74	4.18	4.60
90	2.70	3.22	3.71	4.17	4.61
120	2.62	3.16	3.67	4.14	4.59
200	2.51	3.07	3.60	4.09	4.55



Reinforced Concrete T-Beams – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

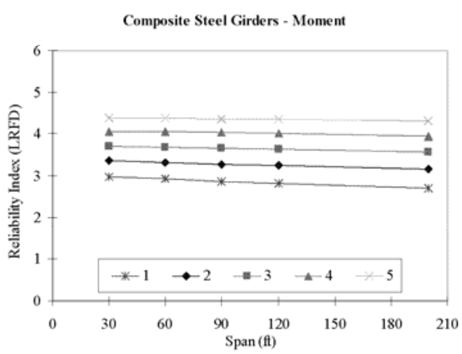
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.29	3.60	3.90	4.17	4.42
60	3.17	3.50	3.80	4.08	4.34
90	3.01	3.35	3.66	3.95	4.22
120	2.89	3.24	3.56	3.85	4.12



Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	2.98	3.36	3.72	4.05	4.37
60	2.93	3.33	3.70	4.05	4.37
90	2.86	3.28	3.66	4.02	4.36
120	2.82	3.24	3.64	4.00	4.34
200	2.70	3.15	3.56	3.94	4.30



Non-Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	2.98	3.36	3.72	4.05	4.37
60	2.93	3.33	3.70	4.05	4.37
90	2.86	3.28	3.66	4.02	4.36
120	2.82	3.24	3.64	4.00	4.34
200	2.69	3.14	3.56	3.94	4.29

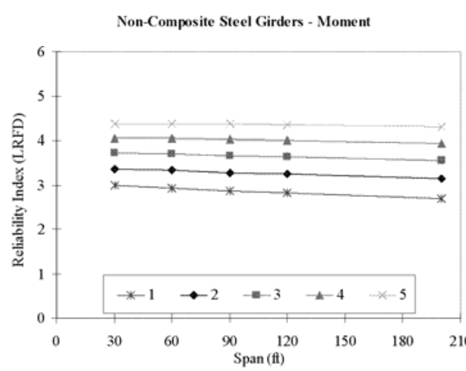
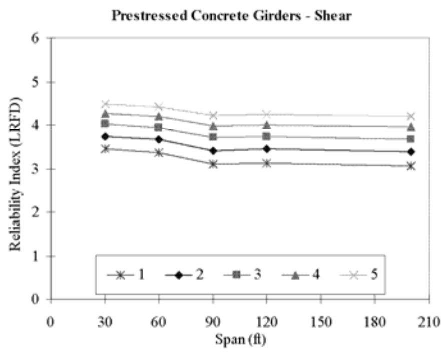


Figure 11 – Effect of η on β for Moment

Prestressed Concrete Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

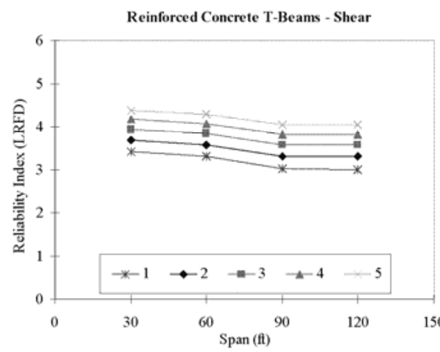
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.46	3.75	4.02	4.26	4.49
60	3.36	3.66	3.94	4.19	4.43
90	3.10	3.42	3.70	3.97	4.21
120	3.13	3.44	3.73	4.00	4.24
200	3.06	3.38	3.68	3.95	4.20



Reinforced Concrete T-Beams – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.43	3.70	3.94	4.17	4.38
60	3.31	3.59	3.84	4.07	4.28
90	3.03	3.32	3.58	3.82	4.05
120	3.01	3.30	3.57	3.81	4.04



Steel Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	0.90	0.95	1.00	1.05	1.10
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.34	3.70	4.04	4.35	4.65
60	3.27	3.64	3.99	4.32	4.62
90	2.96	3.35	3.71	4.05	4.36
120	3.06	3.45	3.82	4.16	4.48
200	2.98	3.39	3.78	4.13	4.46

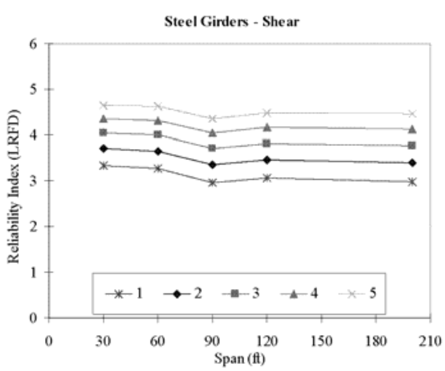
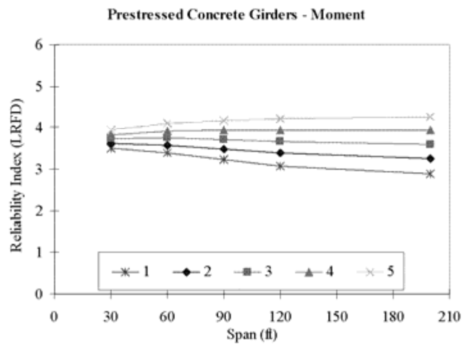


Figure 12 – Effect of η on β for Shear

Prestressed Concrete Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

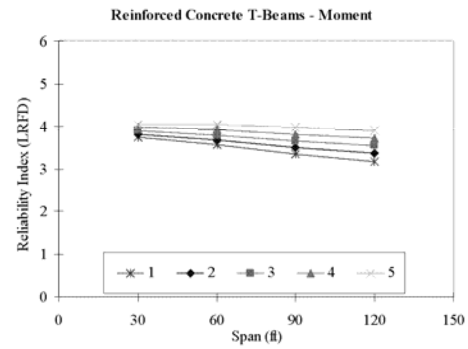
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.51	3.62	3.73	3.83	3.93
60	3.38	3.57	3.74	3.92	4.09
90	3.23	3.47	3.71	3.94	4.17
120	3.08	3.38	3.67	3.95	4.22
200	2.88	3.25	3.60	3.94	4.26



Reinforced Concrete T-Beams – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

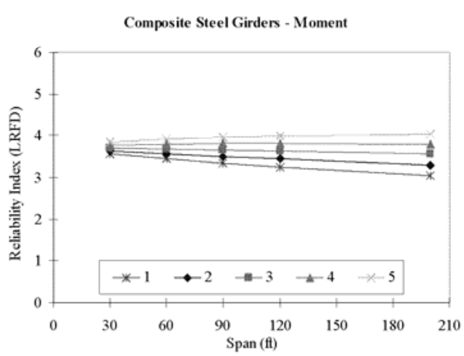
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.75	3.82	3.90	3.97	4.04
60	3.56	3.68	3.80	3.92	4.03
90	3.34	3.51	3.66	3.82	3.96
120	3.17	3.37	3.56	3.73	3.90



Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.57	3.64	3.72	3.79	3.86
60	3.46	3.58	3.70	3.81	3.92
90	3.34	3.50	3.66	3.81	3.96
120	3.26	3.45	3.64	3.82	3.99
200	3.04	3.31	3.56	3.81	4.04



Non-Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.57	3.64	3.72	3.79	3.86
60	3.46	3.58	3.70	3.81	3.93
90	3.34	3.50	3.66	3.81	3.96
120	3.25	3.45	3.64	3.82	3.99
200	3.03	3.30	3.56	3.80	4.04

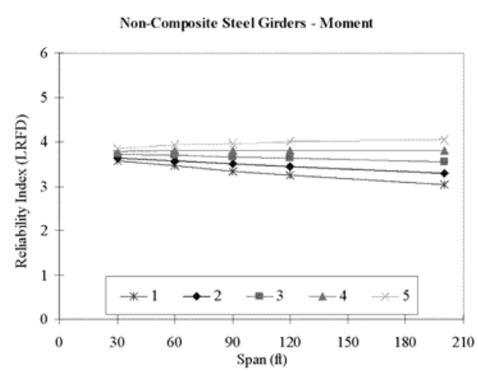
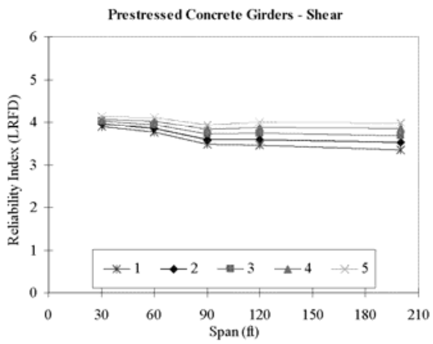


Figure 13 – Effect of Dead Load Scalars on β for Moment

Prestressed Concrete Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

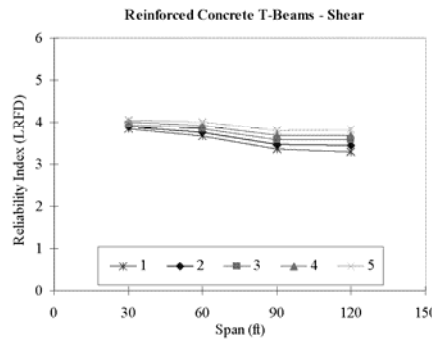
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.90	3.96	4.02	4.07	4.13
60	3.76	3.85	3.94	4.02	4.11
90	3.47	3.59	3.70	3.82	3.93
120	3.45	3.59	3.73	3.87	3.99
200	3.34	3.51	3.68	3.84	3.99



Reinforced Concrete T-Beams – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.84	3.89	3.94	4.00	4.05
60	3.67	3.76	3.84	3.92	4.00
90	3.35	3.47	3.58	3.69	3.79
120	3.30	3.44	3.57	3.69	3.82



Steel Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	0.90	0.95	1.00	1.05	1.10
DA Scalar =	0.90	0.95	1.00	1.05	1.10
L Scalar =	1.00	1.00	1.00	1.00	1.00

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.92	3.98	4.04	4.10	4.16
60	3.81	3.90	3.99	4.08	4.16
90	3.47	3.59	3.71	3.83	3.95
120	3.53	3.68	3.82	3.96	4.09
200	3.37	3.58	3.78	3.97	4.15

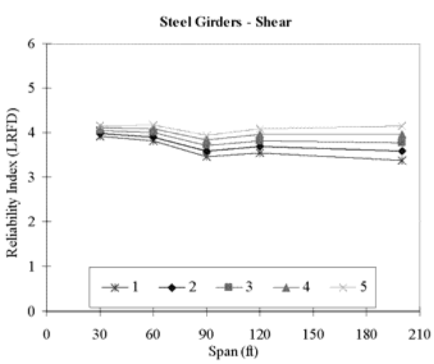
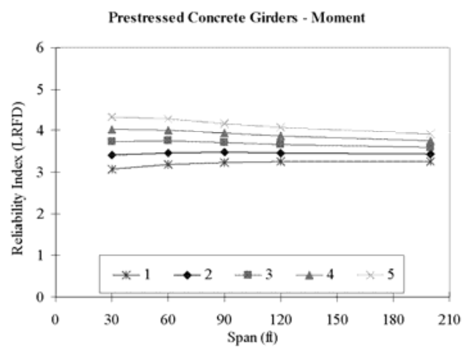


Figure 14 – Effect of Dead Load Scalars on β for Shear

Prestressed Concrete Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

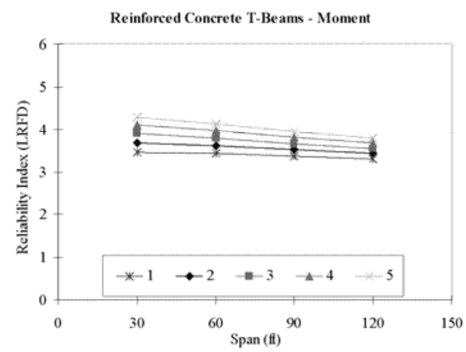
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.08	3.41	3.73	4.03	4.33
60	3.18	3.47	3.74	4.01	4.27
90	3.22	3.47	3.71	3.95	4.18
120	3.24	3.46	3.67	3.87	4.07
200	3.26	3.43	3.60	3.77	3.93



Reinforced Concrete T-Beams – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

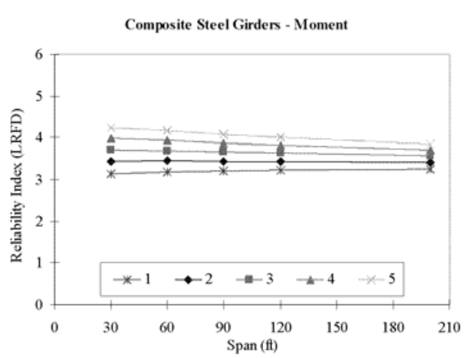
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.45	3.68	3.90	4.10	4.29
60	3.44	3.62	3.80	3.97	4.13
90	3.36	3.52	3.66	3.81	3.94
120	3.30	3.43	3.56	3.68	3.79



Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.14	3.44	3.72	3.98	4.24
60	3.19	3.45	3.70	3.94	4.16
90	3.21	3.44	3.66	3.87	4.08
120	3.23	3.44	3.64	3.83	4.02
200	3.26	3.41	3.56	3.71	3.85



Non-Composite Steel Girders – Moment

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.14	3.44	3.72	3.98	4.24
60	3.19	3.45	3.70	3.94	4.16
90	3.21	3.44	3.66	3.87	4.08
120	3.23	3.44	3.64	3.83	4.01
200	3.26	3.41	3.56	3.70	3.84

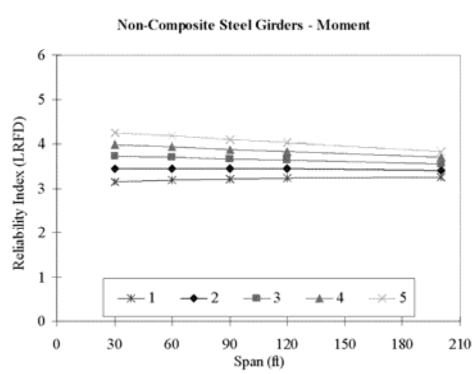
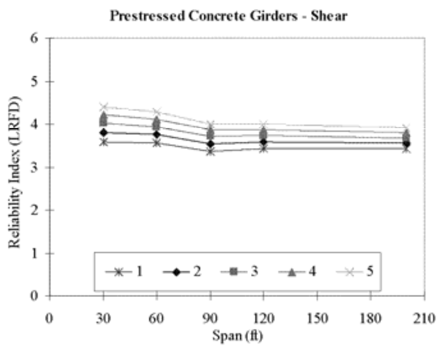


Figure 15 – Effect of Live Load Scalars on β for Moment

Prestressed Concrete Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

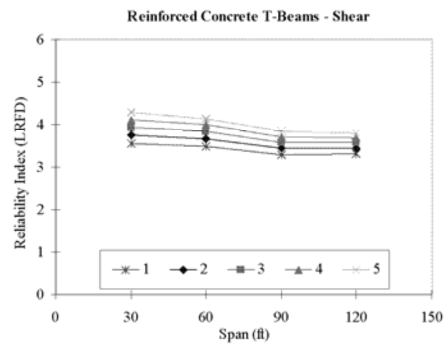
Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.59	3.81	4.02	4.21	4.40
60	3.56	3.76	3.94	4.11	4.28
90	3.37	3.54	3.70	3.86	4.01
120	3.44	3.59	3.73	3.87	4.00
200	3.43	3.56	3.68	3.80	3.91



Reinforced Concrete T-Beams – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	0.90	0.90	0.90	0.90	0.90
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.55	3.76	3.94	4.12	4.29
60	3.50	3.67	3.84	4.00	4.14
90	3.28	3.43	3.58	3.72	3.85
120	3.31	3.44	3.57	3.69	3.81



Steel Girders – Shear

Input Parameters					
Variable	Series 1	Series 2	Series 3	Series 4	Series 5
η=	1.00	1.00	1.00	1.00	1.00
ϕ=	1.00	1.00	1.00	1.00	1.00
D1 D2 Scalar =	1.00	1.00	1.00	1.00	1.00
DA Scalar =	1.00	1.00	1.00	1.00	1.00
L Scalar =	0.90	0.95	1.00	1.05	1.10

Beta Value					
Span (ft)	Series 1	Series 2	Series 3	Series 4	Series 5
30	3.47	3.76	4.04	4.30	4.55
60	3.47	3.74	3.99	4.23	4.46
90	3.23	3.48	3.71	3.94	4.15
120	3.38	3.60	3.82	4.03	4.23
200	3.42	3.60	3.78	3.94	4.11

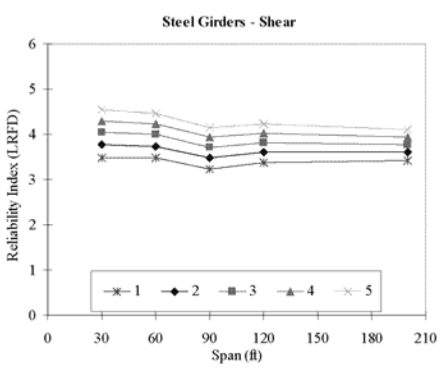


Figure 16 – Effect of Live Load Scalars on β for Shear

Figures 11 and 12 show the affect that varying η has on the reliability index. From these figures it is seen that changing η causes a parallel shift in the plot of reliability index. A natural question to ask is if the magnitude of this shift can be predicted. To investigate this possibility, a ratio of $\beta_{\text{series } i} / \beta_{\text{series } 3}$ was taken. Figure 17 shows this ratio for different values of η for moment in prestressed concrete girders.

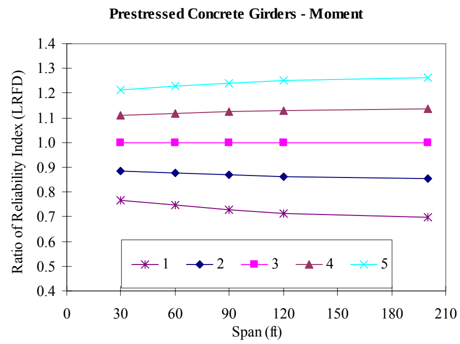
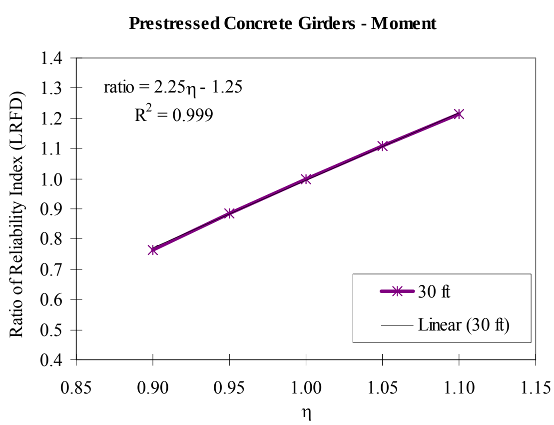


Figure 17 – Typical Plot of $\beta_{\text{series } i} / \beta_{\text{series } 3}$ vs. Span Length for Different Values of η

As shown in Figure 17, which is a typical result, the ratio of reliability index is span dependent. However, for a given span length, the ratio of reliability index appears to vary linearly with η. The variation of reliability index as a function of η is shown in Figure 18 for prestressed girders with 30 ft and 200 ft spans.



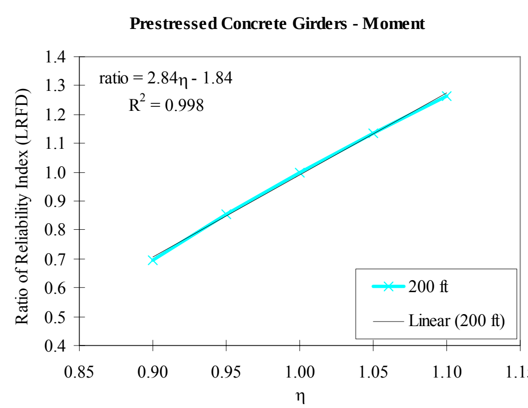


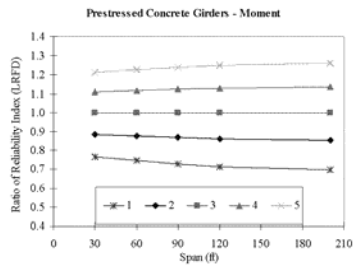
Figure 18 – Typical Plots of β vs. η for Given Span Lengths

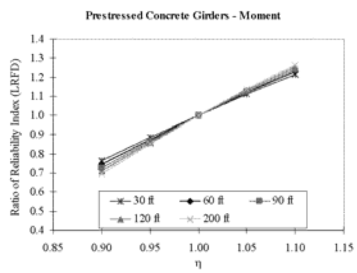
Figure 18 contains best fit equations for the data. The R^2 values of 0.999 and 0.998 indicate that the linear approximation fits the data well. The equations of the best fit lines reveal that the slopes are 2.25 for 30 ft spans and 2.84 for 200 ft spans. These slopes indicate that a change in η will be amplified by 2.25 and 2.84 in the reliability index (ie a 10% increase in η will cause a 22.5% increase in β for 30 ft spans and a 28.4% increase in β for 200 ft spans).

Graphs similar to Figure 17 and Figure 18 are shown in Figure 19 for moment and Figure 20 for shear for the various girder types investigated.

Prestressed Concrete Girders – Moment

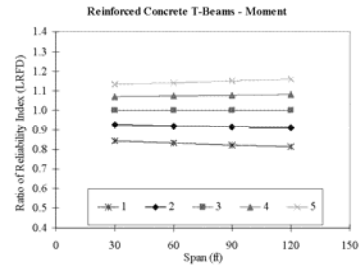
Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.77	0.89	1.00	1.11	1.21
60	0.75	0.88	1.00	1.12	1.23
90	0.73	0.87	1.00	1.12	1.24
120	0.71	0.86	1.00	1.13	1.25
200	0.70	0.85	1.00	1.14	1.26

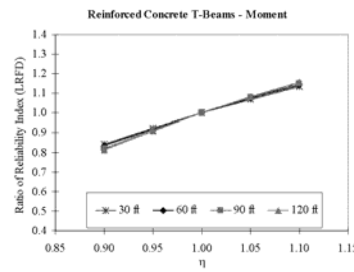




Reinforced Concrete T-Beams – Moment

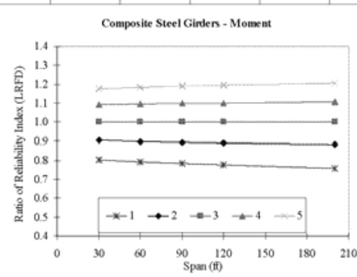
Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.84	0.92	1.00	1.07	1.13
60	0.83	0.92	1.00	1.07	1.14
90	0.82	0.91	1.00	1.08	1.15
120	0.81	0.91	1.00	1.08	1.16

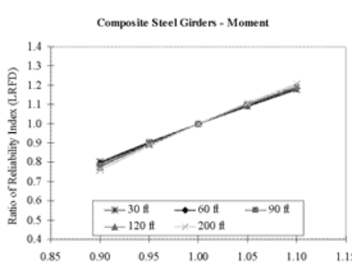




Composite Steel Girders – Moment

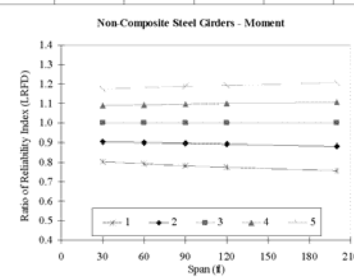
Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.80	0.90	1.00	1.09	1.18
60	0.79	0.90	1.00	1.09	1.18
90	0.78	0.89	1.00	1.10	1.19
120	0.78	0.89	1.00	1.10	1.19
200	0.76	0.88	1.00	1.11	1.21





Non-Composite Steel Girders – Moment

Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.80	0.90	1.00	1.09	1.18
60	0.79	0.90	1.00	1.09	1.18
90	0.78	0.89	1.00	1.10	1.19
120	0.78	0.89	1.00	1.10	1.19
200	0.76	0.88	1.00	1.11	1.21



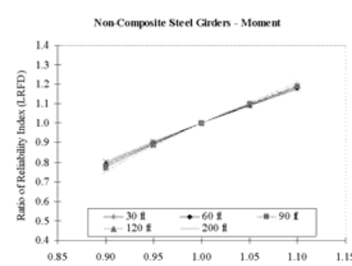
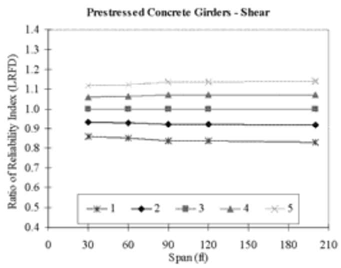
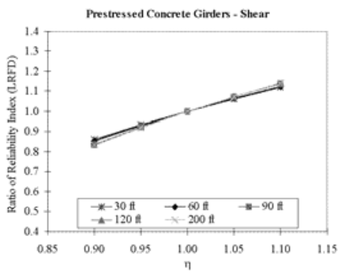


Figure 19 – Effect of η on β Ratio for Moment

Prestressed Concrete Girders – Shear

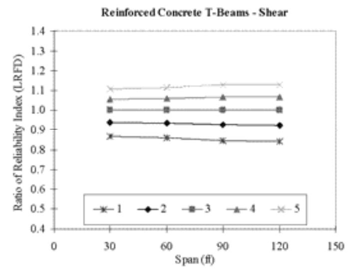
Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.86	0.93	1.00	1.06	1.12
60	0.85	0.93	1.00	1.06	1.12
90	0.84	0.92	1.00	1.07	1.14
120	0.84	0.92	1.00	1.07	1.14
200	0.83	0.92	1.00	1.07	1.14

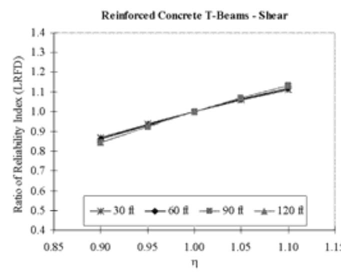




Reinforced Concrete T-Beams – Shear

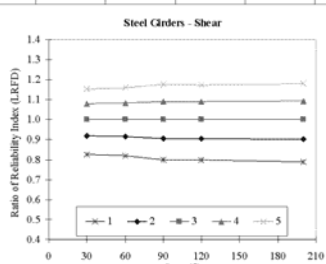
Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.87	0.94	1.00	1.06	1.11
60	0.86	0.93	1.00	1.06	1.12
90	0.85	0.93	1.00	1.07	1.13
120	0.84	0.93	1.00	1.07	1.13





Steel Girders – Shear

Beta Series i / Beta Series 3					
Span (ft)	Series 1 η=0.90	Series 2 η=0.95	Series 3 η=1.00	Series 4 η=1.05	Series 5 η=1.10
30	0.83	0.92	1.00	1.08	1.15
60	0.82	0.91	1.00	1.08	1.16
90	0.80	0.90	1.00	1.09	1.18
120	0.80	0.90	1.00	1.09	1.17
200	0.79	0.90	1.00	1.09	1.18



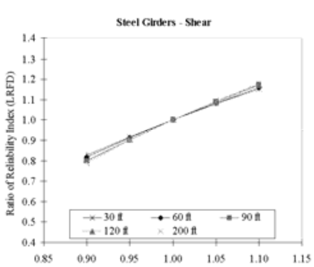


Figure 20 – Effect of η on β Ratio for Shear

The results for all girders investigated are generally similar in that the affect of η on β is span dependant, but is roughly linear for a given span length. Table 9 shows a summary of the results. From this table, it may be seen that, for the spans lengths and beam types examined, the change in β for moment will be 1.45 to 2.84 times larger than the change in η that caused it. For shear the change in β will be 1.20 to 1.95 times larger than the change in η that caused it. Although the general effect that η has on β is similar for the variety of girders examined, to accurately obtain β values for a given change of η, rigorous calculation that consider the girder type and length must be performed.

Table 9 - Summary of Slopes from Plots in Figures 13 and 14

Range of Slope for η vs. Ratio of Reliability Index Plots		
Girder Type	**Moment**	**Shear**
Prestressed Concrete	2.25 to 2.84	1.29 to 1.54
Reinforced Concrete T-Beam	1.45 to 1.72	1.20 to 1.44
Steel – Composite	1.86 to 2.24	1.63 to 1.95
Steel – Non-Composite	1.86 to 2.25	

SUMMARY

This project had a series of very discrete tasks for which deliverables had been prepared and included in this report. These included:

- A reconstruction of the calibration work done during the original NCHRP 12-33 to develop AASHTO LRFD. Various parameters used in that original work have been clarified and thoroughly documented. This applied to both the dead load and live load, and to resistance models.

- A Guide to Data Collection and Documentation has been prepared and has in fact received early implementation in some NCHRP projects.

- A revised, expanded and updated bridge database was developed and data for these bridges are included herein. Reliability indices for these bridges have been produced using Monte Carlo simulation.

- Various calibration procedures were reviewed, and it was determined that for future work, the Monte Carlo simulation is the most robust flexible method. An example of its application is provided.

- An analysis of the sensitivity of reliability indices to changes in load factors, resistance factors, and both factors simultaneously has been reported. In the process of doing this, a demonstration that the Rackwitz and Fiessler procedure used in the original NCHRP 12-33 work yields almost exactly the same results for the class of bridges studies as the Monte Carlo simulation was made.

ACKNOWLEDGEMENTS

Researchers wish to acknowledge the Project Panel and Senior Program Officer David B. Beal. Their guidance has been much appreciated. Mr. Timothy J. Stuffle, EIT, assisted with the study of parameter variation. Finally, we wish to acknowledge the assistance of Ms. Diane M. Long for her work in producing the manuscript.

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APPENDIX I – SUMMARY REPORT

UNLCE 05-08

May 14, 2006
Revised August 7, 2006

Summary of all statistical parameters used in calibration of the AASHTO LRFD Specifications (based on NCHRP Report 368)

Progress Report
NCHRP 20-7/186

Andrzej S. Nowak,
Artur A. Czarnecki and Piotr J. Podhorecki
University of Nebraska
Lincoln, NE 68588-0531

List of Contents

1. Objective

The objective of this report is to summarize the calibration procedure used for the development of load and resistance factors in AASHTO LRFD (1994, 1998, 2004), including:

- Design formula and limit state function
- Load model
- Resistance model
- Reliability analysis procedure
- Load and resistance parameters

This procedure was unanimously accepted by the Calibration Task Group (Andy Nowak, Alin Cornell, Ted Galambos, Fred Moses, Dan Frangopol, Roger Green and Kamal Rojiani).

2. Design Formula

The format of basic design formula in AASHTO LRFD (1994, 1998, 2004) is

$$\gamma_D \, DL + \gamma_L \, (LL + IL) \leq \phi \, R \tag{1}$$

where γ_D = dead load factor, γ_L = live load factor, ϕ = resistance factor, DL, LL and IL are nominal values of load components, and R = nominal value of resistance.

Load and resistance are random variables due to natural and man-made uncertainties. To provide an adequate safety level, the design values of load and resistance have to be conservative: loads are overestimated (Figure 1) and load carrying capacity (resistance) is underestimated (Figure 2). Therefore, load and resistance factors represent partial safety margins.

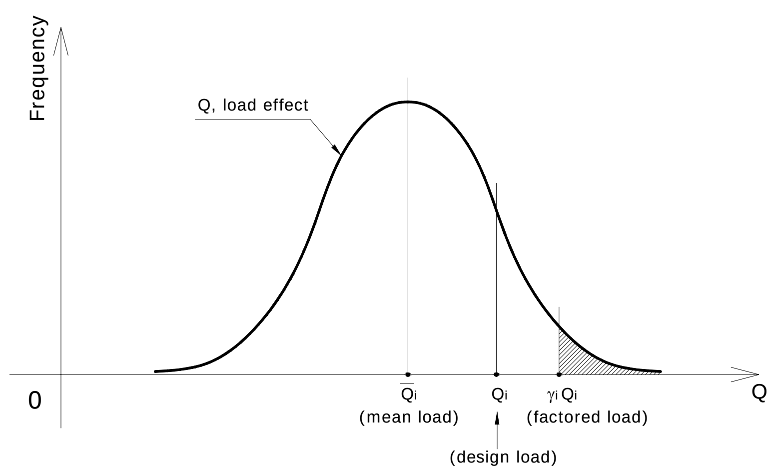

Figure 1 - Mean Load, Design (Nominal) Load and Factored Load

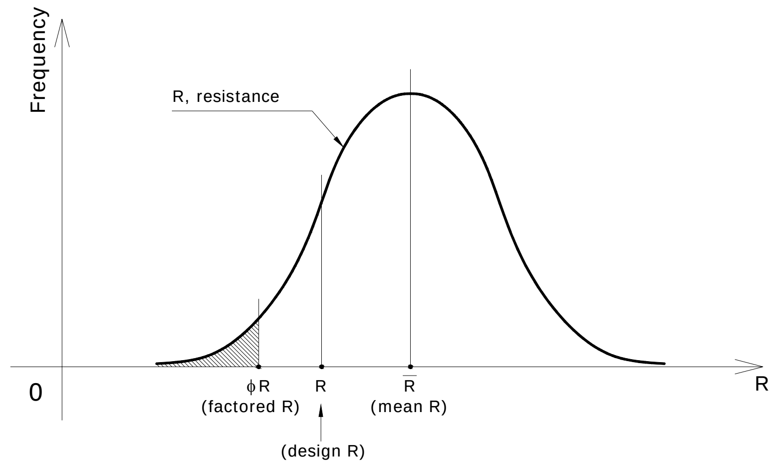

Figure 2 - Mean Resistance, Design (Nominal) Resistance and Factored Resistance

Objective of the code calibration is to determine load and resistance factors so that the reliability of structural components is at the acceptable level (target reliability). This involves the development of reliability analysis procedure, selection of the target reliability level, and implementation (i.e. selection of the load and resistance factors).

3. Limit State Function

The limit state function is a mathematical representation of the acceptability criterion. In the basic design case (Eq. 1), the limit state function is,

$$g = R - (DL + LL + IL) = 0 \tag{2}$$

where R, DL, LL and IL are random variables representing resistance and load components. This is very different than Eq. 1, where these symbols represent nominal values. If $g \geq 0$, the structural component is safe, and if $g < 0$, the component fails. The boundary between safe and unsafe domain is represented by $g = 0$. This corresponds to the case of total load being equal to resistance.

Probability of failure for the considered component, P_F, is equal to

$$P_F = P(g < 0) \tag{3}$$

In this calibration, the probability of failure is calculated indirectly, as a function of the reliability index, β,

$$P_F = \phi(-\beta) \tag{4}$$

where ϕ = standard normal cumulative distribution function.

Reliability index is determined using Rackwitz and Fiessler procedure. This was the most efficient reliability analysis method in the 1990's. The input data are statistical parameters for each load component and resistance:

- Mean value, and bias factor, λ = ratio of the mean-to-nominal
- Coefficient of variation, V = ratio of standard deviation and the mean value
- Type of cumulative distribution function

The load and resistance models are based on the literature available in the 1980's and additional studies performed specially for this calibration. The total load effect is represented by a single random variable, Q, so that the limit state function is

$$g = R - Q = 0 \tag{5}$$

where Q = DL + LL + IM.

4. Load Model

The statistical parameters of the total load, Q, are determined as a function of statistical parameters of load components. The mean of Q is a sum of the mean values of components,

$$m_Q = m_{DL} + m_{LL} + m_{IM} \tag{6}$$

where m_{DL} = mean dead load; m_{LL} = mean live load, and m_{IM} = mean dynamic load. The mean values of load components are calculated using bias factors, λ, and nominal (design) value of the considered load component, for example, for dead load,

$$m_{DL} = (\lambda_{DL})\text{ (nominal value of DL)} \tag{7}$$

The variance of Q, σ^2_Q, is a sum of variances of load components,

$$\sigma^2_Q = \sigma^2_{DL} + \sigma^2_{LL} + \sigma^2_{IM} \tag{8}$$

Then the standard deviation of Q, σ_Q, is equal to the square root of σ^2_Q. The coefficient of variation of Q, V_Q, is

$$V_Q = \sigma_Q/m_Q \tag{9}$$

The total load is a sum of components such as dead load, live load and dynamic load. For these load components only summary statistics were available. However, based on the Central Limit Theorem, in many practical cases, a sum of any random variables can be approximated by a normal variable. Therefore, it is assumed that the total load effect is normally distributed (i.e. Q is treated as a normal random variable).

The statistical parameters of components are summarized as follows.

4.1. Dead load

Dead load effect is considered as a sum of four sub-components.

- DL_1 – weight of factory made elements (steel and precast concrete girders),
- DL_2 – weight of cast-in-place concrete,
- DL_3 – weight of wearing surface (asphalt),
- DL_4 – weight of miscellaneous items (e.g. railing, luminaires)

The statistical parameters for dead load components are summarized in Table 1, including bias factor and coefficient of variation.

Table 1. Statistical Parameters of Dead Load

Dead Load Component	Bias Factor	Coefficient of Variation
Factory made members, DL_1	1.03	0.08
Cast-in-place, DL_2	1.05	0.10
Wearing surface, DL_3	1.0	0.25
Miscellaneous, DL_4	1.03 ~ 1.05	0.08 ~ 0.10

The bias factors for DL_1 and DL_2 were provided by the Ontario Ministry of Transportation based on surveys of actual bridges in conjunction with calibration of the Ontario Highway Bridge Design Code (OHBDC 1979; Lind and Nowak 1978). The coefficients of variation provided by the Ministry of Transportation for dead load were 0.04 and 0.08 for DL_1 and DL_2, respectively (Lind and Nowak 1978). However, there is no report available to support this data. The coefficients of variation used in calibration were taken from the NBS Report 577 (Ellingwood et al. 1980), and they include other uncertainties (also human error).

The parameters of DL_3 are calculated using the survey data provided by the Ontario Ministry of Transportation in conjunction with calibration of the OHBDC (1979). The average of a number of field measurements of asphalt wearing surface, DL3, taken in Ontario was 3.5 in. The bias factor is assumed = 1.0. From the results of measurements, the coefficient of variation is taken = 0.25. The only direct use of this piece of information was to inform the choice of the load factor for DW which was taken as 1.5 due to the perceived higher variability of this component of dead load compared to the other components which were combined as DC in the AASHTO LRFD.

The construction and use of the normal probability paper is presented in textbooks (e.g Nowak and Collins 2000) and TRB Circular E-C079 (Allen, et al. 2005). The CDF of any normal random variable is represented by a straight line on the normal probability paper, and any straight line on the normal probability paper represents a normal random variable. The mean value corresponds to 0 on the vertical axis. The slope of CDF is an indication of the standard deviation and coefficient of variation. The curves representing asphalt thickness in Figure 3 indicate that the thickness can be treated as a normal random variable with the mean equal to 3.5 in, and coefficient of variation of about 0.25.

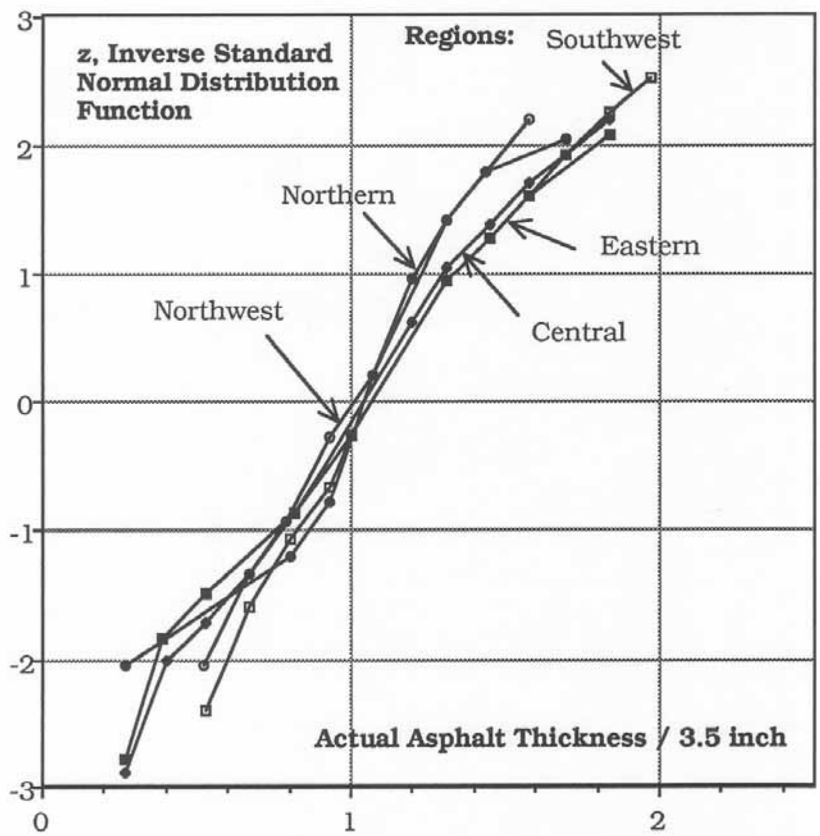

Figure 3 - Cumulative Distribution Functions (CDF) of the Asphalt Thickness on Bridges in 5 Regions of Ontario

4.2. Live load

The analysis for derivation of the statistical parameters for bridge live load was performed for ADTT = 1000. In addition the statistical parameters are considered for ADTT = 100 and 5000. The parameters are determined for one traffic lane and for two and more traffic lanes.

4.2.1. Basic Assumptions

The analysis is based on the truck survey results from Ontario and field observations from Michigan, used in the Calibration Report (NCHRP Report 368). The following assumptions are made:

(a) The truck survey from Ontario is representative for live load on the considered bridges.
(b) The surveyed trucks represent two weeks of heavy traffic on a road with ADTT = 1000.
(c) The truck population will not change in the future (the legal loads will not be changed).

4.2.2. Ontario Truck Survey

At the time of calibration of AASHTO LRFD in the 1980's, there was no reliable truck data available for the USA. All available WIM data was found to be inadequate or flawed for this purpose. Therefore, the live load model is based on the truck survey results provided by the Ontario Ministry of Transportation. The survey was carried out in conjunction with calibration of the Ontario Highway Bridge Design Code (OHBDC 1979). However, multiple presence and extrapolations for longer time periods were considered using analytical simulations.

The survey was carried out in mid 1970's and included 9,250 vehicles, measured at various locations in the Province of Ontario, Canada. At this time the legal loads in US and Canada were the same. For each measured vehicle, the record include: number of axles, axle spacing, axle loads and gross vehicle weight. Only the vehicles that appeared to be heavily loaded were stopped and weighed. It was assumed that the surveyed trucks represent a two week heavy traffic on a two lane bridge with ADTT = 1000 (in one direction).

Each vehicle from the survey was "run" over the influence lines to determine the maximum bending moment, shear force and negative moment for two span bridges. The calculations were carried out for span lengths from 10 ft through 200 ft. The resulting cumulative distribution functions (CDF) were plotted on the normal probability paper for an easier interpretation and extrapolation, as shown in Figures 4, 5 and 6 for positive moment, shear and negative moment, respectively. The CDF's are presented for the surveyed truck moments divided by the HS20 moment. The results indicate that there is a considerable difference between the moment ratios depending on the span length. The shape of the CDF curves indicates that the moments are not normal random variables.

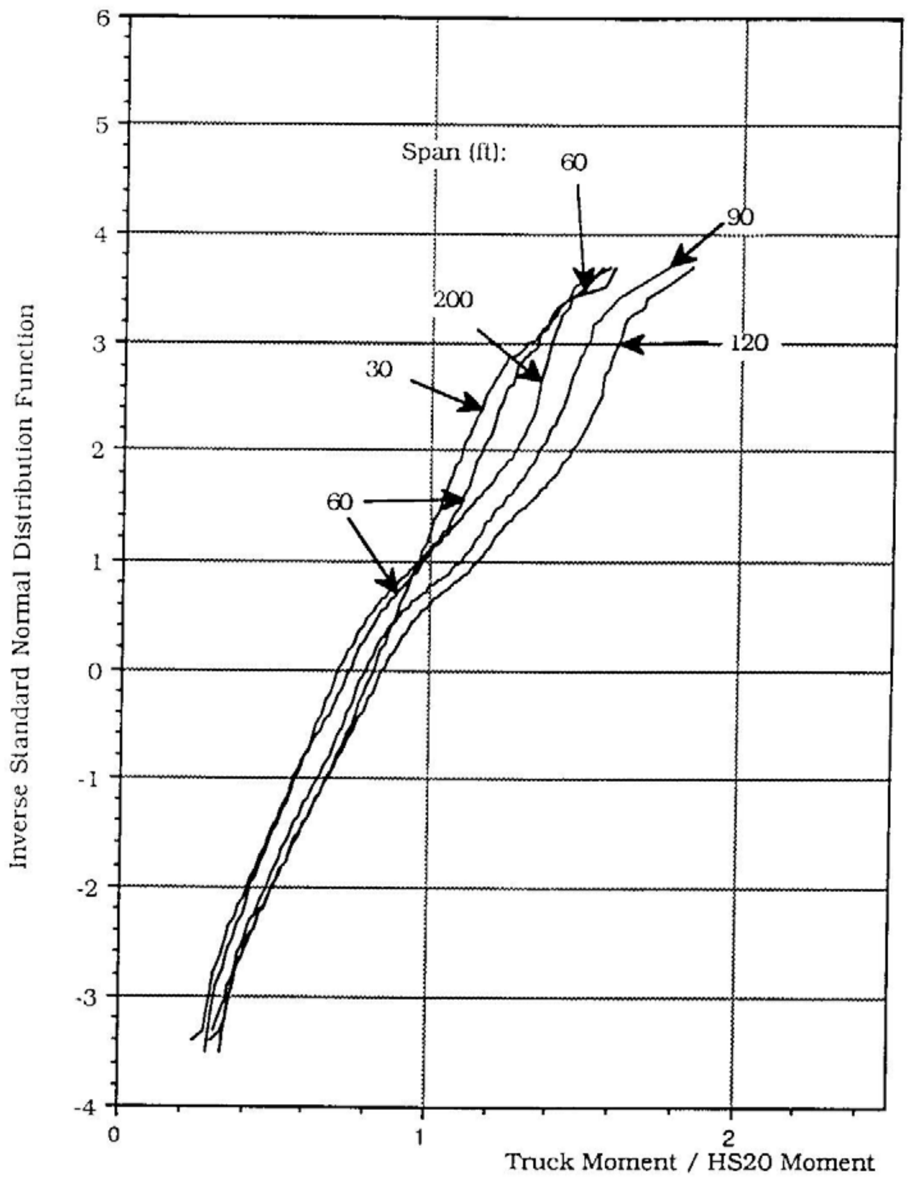

Figure 4 - Cumulative Distribution Functions of Moments due to Surveyed Trucks

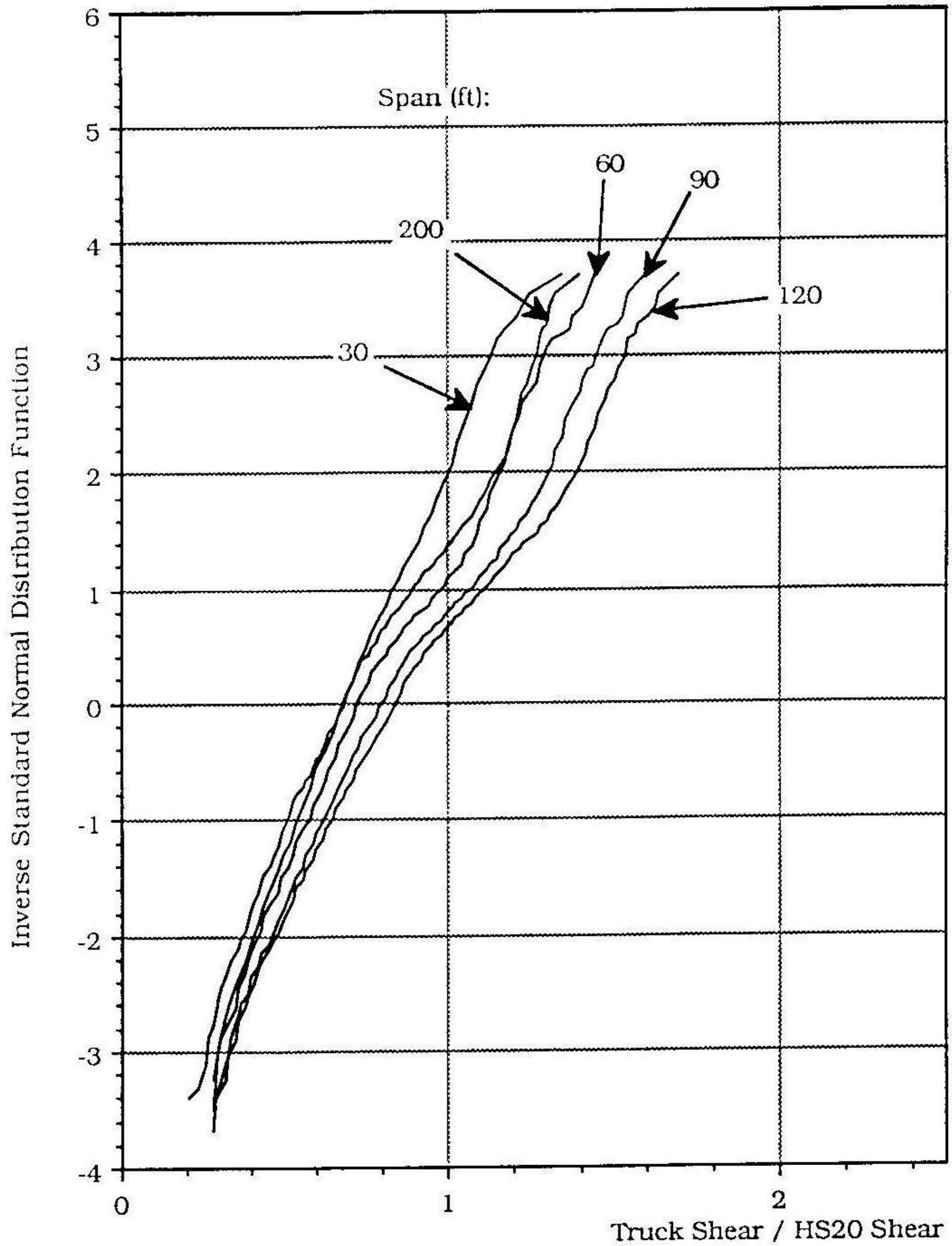

Figure 5 - Cumulative Distribution Functions of Shear due to Surveyed Trucks

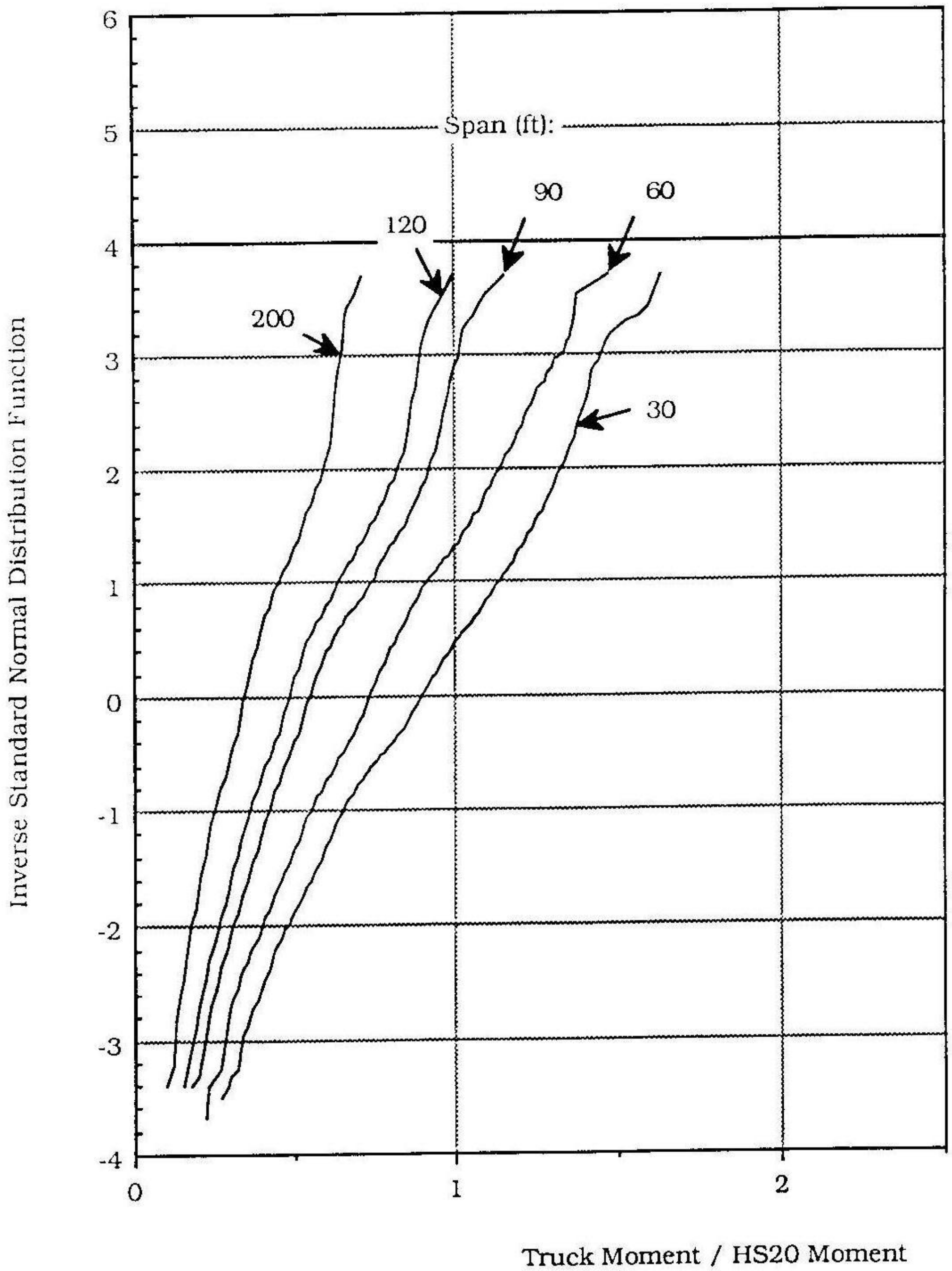

Figure 6 - Cumulative Distribution Functions of Negative Moments due to Surveyed Trucks

4.2.3. Interpolations and Extrapolations of Live Load Effects

It is assumed that the survey data represents two weeks of heavy traffic on a bridge with ADTT = 1000. Different time periods correspond to different values on the vertical axis. The total number of trucks in the survey is 9250. This corresponds to the probability of 1/9250 = 0.00011, the inverse normal standard distribution function corresponding to this probability is 3.71.

The important question is what would be the expected curve representing the CDF, if the survey was continued for longer periods of time, e.g. 2 months, a year, 10 years, 50 years or 75 years? The CDF's corresponding to longer time periods will be similar to the curves in Figures 4-6, but the upper tails will have to be extended. The longer is the time period, the larger is the number of trucks. Therefore, the CDF's would be longer, they would extend up and down. How far should these CDF's be extended depends on the considered time period? For example, for 75 years, the total number of trucks will be 1950 x 9250 = 18 million, because there are 1950 two week periods in 75 years. The probability of occurrence of the heaviest truck in this population is 1/18,000,000 = 5.33 $E10^{-8}$, this corresponds to 5.33 on the vertical axis in Figures 4-6, because $\phi^{-1}(5E10^{-8}) = 5.33$, where ϕ^{-1} that is inverse standard normal distribution. However, extension of the upper tails is subjective as it involves a considerable dose of engineering judgment.

Therefore, in case of loads, the most important is the upper tail of the CDF. This is particularly important in extrapolation of the results to predict the most likely CDF for longer time periods, up to 75 years. In some cases, the upper tail of the CDF clearly shows a trend, and the extrapolation is easy. However, in other cases there can be a need to make a decision about the "outliers" (see TRB Circular E-C079). In the Ontario survey data, there are 2-3 very heavy vehicles that seem not to follow the trend. In the extrapolations, the heaviest two trucks were treated as "outliers" and the CDF's were extrapolated as shown in Figures 7, 8 and 9, for the positive moment, shear and negative moment, respectively.

The bias factor is defined as the ratio of the mean value and nominal (design) value. The mean live load varies depending on the reference time period. The bias factors were determined by reading the values directly from the graphs in Figures 7-6. The nominal values of live load in Figures 7-9 are determined using HS-20 loading (AASHTO Standard 2002). In addition, the bias factors were calculated for nominal values determined using HL-93 loading (AASHTO LRFD 2004). For ADTT = 1000, for HS-20, the results were tabulated and shown in Table 2 for simple span moments, Table 3 for shears, and Table 4 for negative moments, and for HL-93 in Tables 5, 6 and 7, respectively.

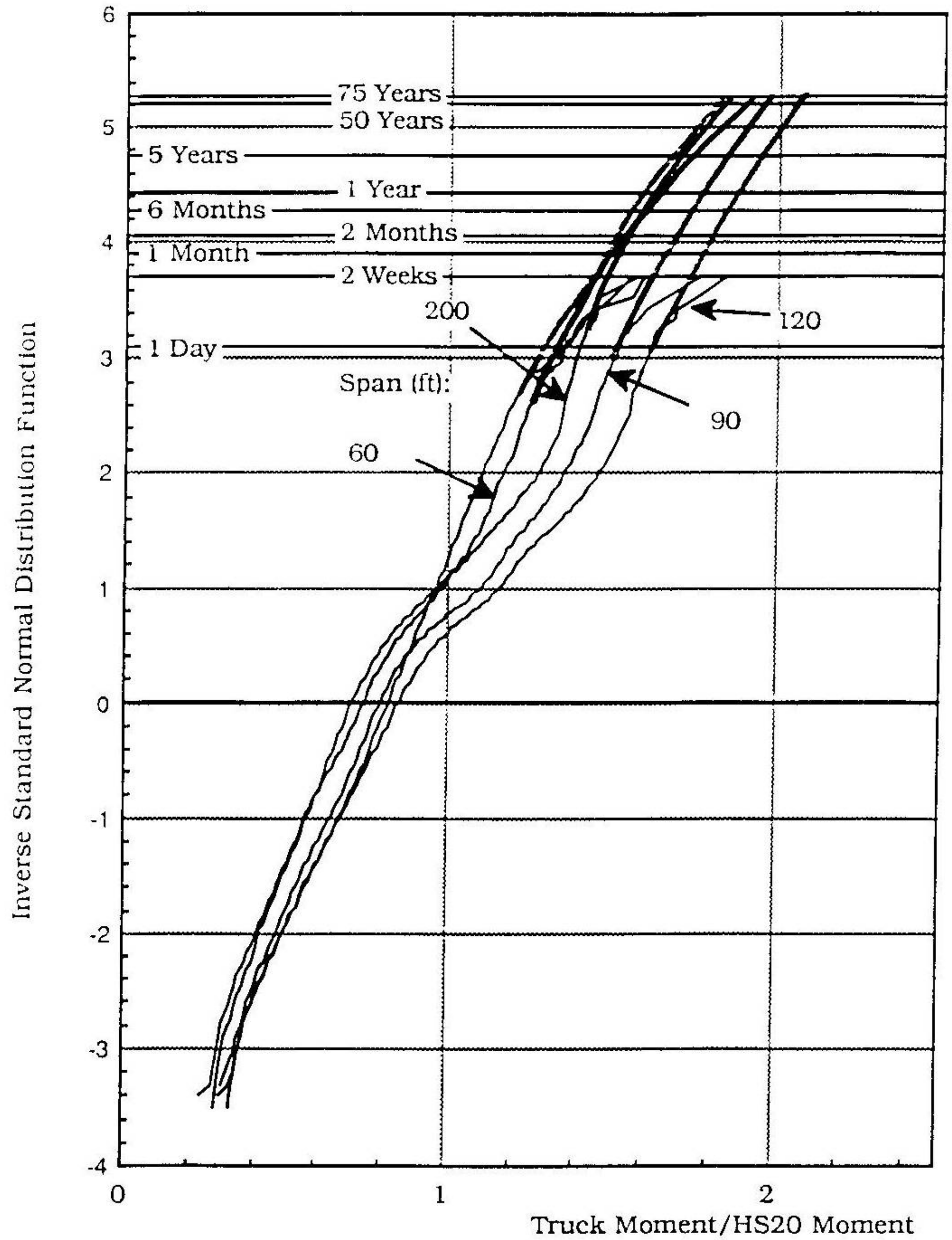

Figure 7 - Extrapolated CDF's of Moment due to Surveyed Trucks

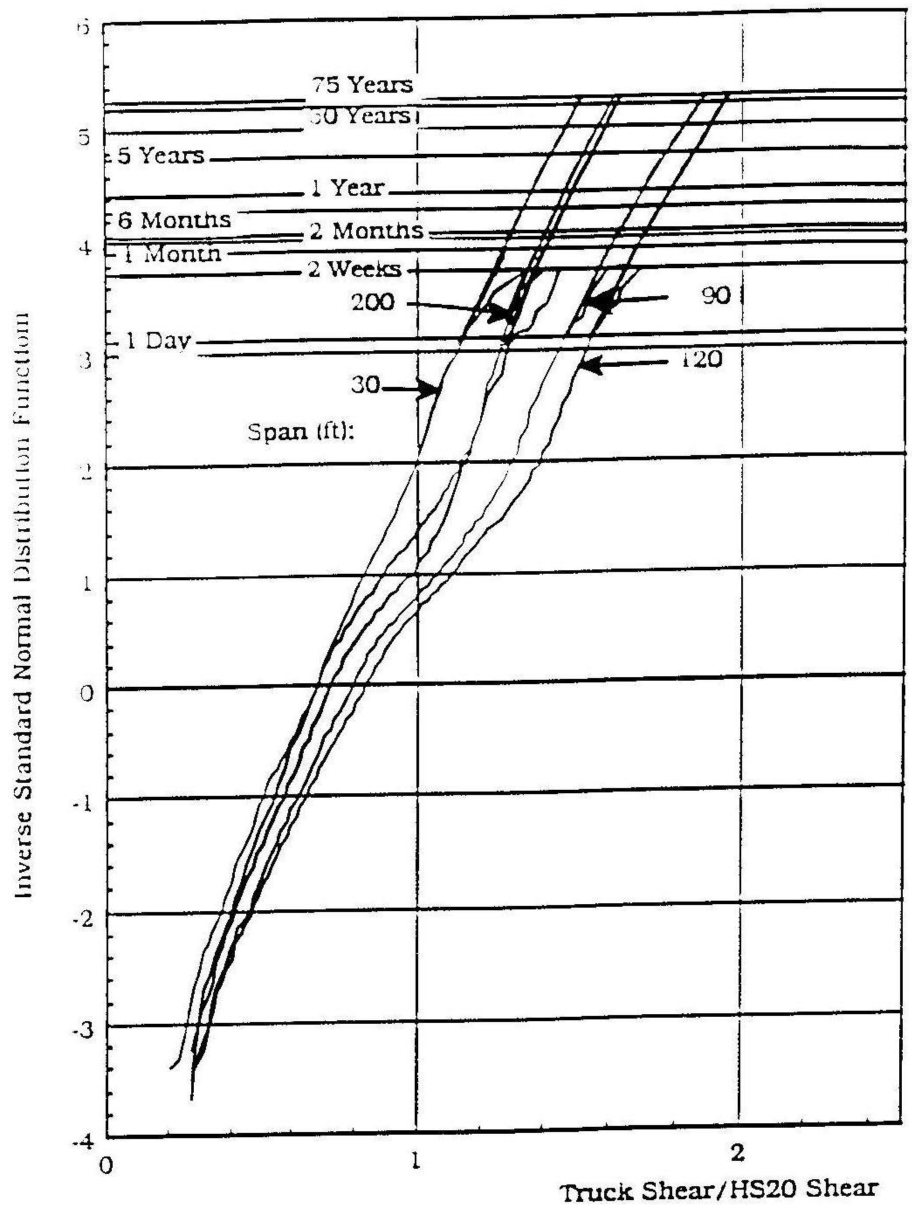

Figure 8 - Extrapolated CDF's of Shear due to Surveyed Trucks

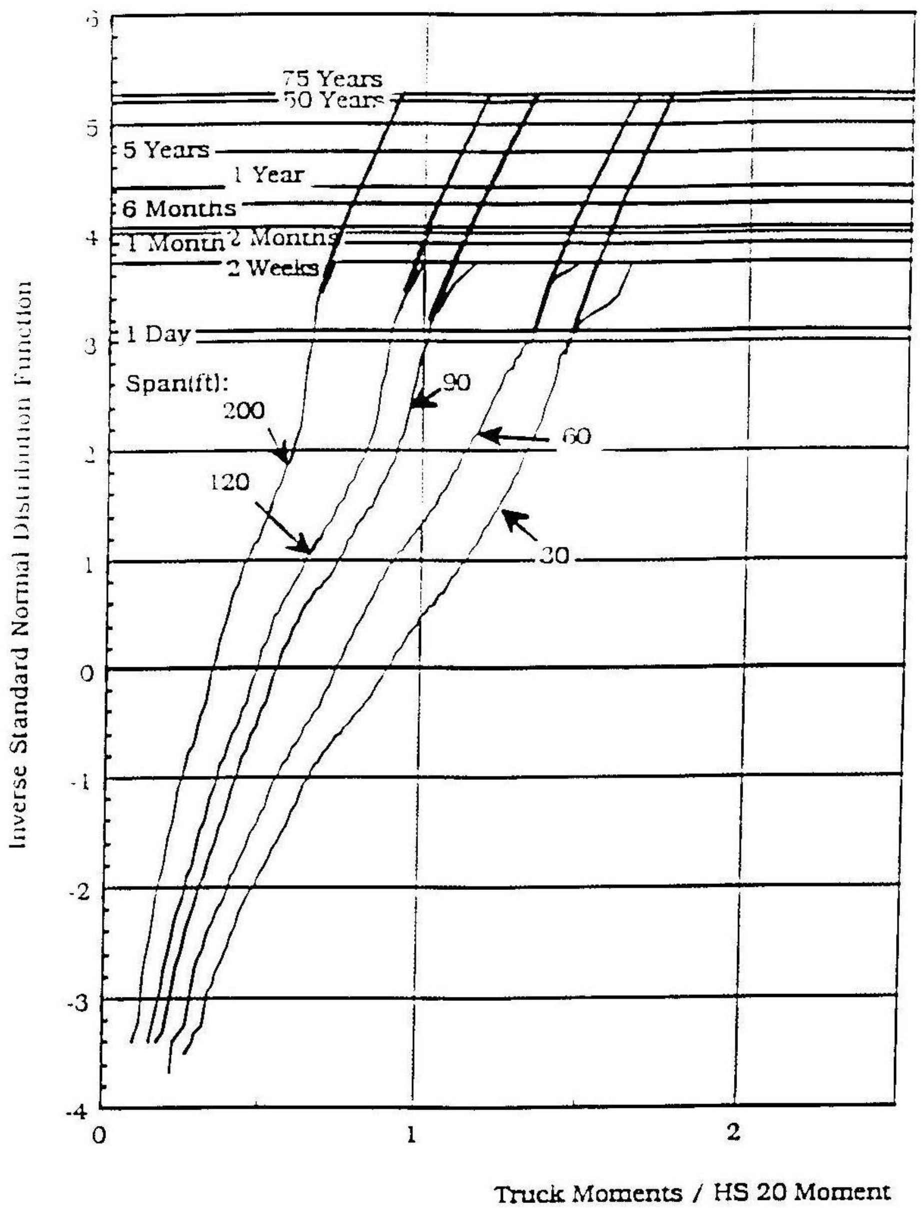

Figure 9 - Extrapolated CDF’s of Negative Moment due to Surveyed Trucks

Table 2. Mean Maximum Moments for Simple Spans Due to a Single Truck Divided by HS20 Moment, ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.62	0.97	1.12	1.18	1.23	1.30	1.37	1.46	1.63	1.65
20	0.71	1.15	1.25	1.31	1.36	1.41	1.47	1.56	1.66	1.68
30	0.74	1.20	1.32	1.37	1.42	1.47	1.52	1.61	1.70	1.72
40	0.75	1.31	1.42	1.46	1.50	1.55	1.58	1.64	1.72	1.74
50	0.72	1.32	1.43	1.47	1.52	1.56	1.60	1.65	1.73	1.75
60	0.72	1.37	1.47	1.52	1.56	1.60	1.64	1.69	1.77	1.79
70	0.74	1.42	1.51	1.56	1.60	1.64	1.68	1.74	1.81	1.83
80	0.77	1.47	1.55	1.60	1.64	1.68	1.73	1.79	1.86	1.89
90	0.79	1.51	1.60	1.64	1.68	1.72	1.78	1.84	1.92	1.94
100	0.82	1.55	1.64	1.68	1.72	1.76	1.82	1.89	1.98	2.00
110	0.84	1.60	1.68	1.72	1.76	1.81	1.86	1.94	2.03	2.05
120	0.85	1.63	1.72	1.76	1.80	1.85	1.90	1.97	2.06	2.08
130	0.86	1.66	1.75	1.80	1.83	1.87	1.92	1.99	2.08	2.10
140	0.86	1.67	1.76	1.80	1.83	1.87	1.92	1.99	2.08	2.10
150	0.85	1.64	1.73	1.78	1.81	1.84	1.88	1.96	2.05	2.07
160	0.84	1.60	1.68	1.73	1.76	1.80	1.84	1.91	2.01	2.03
170	0.81	1.56	1.63	1.69	1.72	1.76	1.80	1.87	1.96	1.98
180	0.78	1.50	1.58	1.64	1.67	1.71	1.75	1.82	1.91	1.94
190	0.75	1.45	1.53	1.58	1.62	1.66	1.70	1.77	1.86	1.88
200	0.70	1.38	1.48	1.54	1.57	1.60	1.64	1.71	1.80	1.82

Table 3. Mean Maximum Shears for Simple Spans Due to a Single Truck (Divided by HS20 Shear), ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.78	1.20	1.31	1.38	1.40	1.44	1.48	1.52	1.61	1.62
20	0.72	1.14	1.25	1.30	1.31	1.36	1.38	1.43	1.51	1.52
30	0.68	1.14	1.24	1.29	1.31	1.35	1.38	1.42	1.48	1.49
40	0.66	1.18	1.28	1.32	1.34	1.37	1.40	1.43	1.50	1.51
50	0.69	1.24	1.33	1.37	1.39	1.43	1.45	1.48	1.55	1.56
60	0.73	1.30	1.40	1.44	1.46	1.49	1.52	1.56	1.61	1.62
70	0.74	1.37	1.47	1.50	1.52	1.55	1.58	1.62	1.69	1.70
80	0.77	1.43	1.53	1.57	1.59	1.63	1.66	1.70	1.77	1.78
90	0.80	1.48	1.58	1.62	1.64	1.69	1.72	1.76	1.84	1.85
100	0.81	1.53	1.63	1.67	1.70	1.73	1.77	1.82	1.89	1.90
110	0.82	1.58	1.67	1.70	1.72	1.76	1.80	1.85	1.92	1.93
120	0.83	1.58	1.67	1.71	1.73	1.77	1.80	1.86	1.92	1.93
130	0.83	1.57	1.66	1.70	1.72	1.75	1.78	1.83	1.90	1.91
140	0.82	1.53	1.63	1.66	1.68	1.72	1.74	1.79	1.86	1.87
150	0.79	1.48	1.58	1.62	1.64	1.67	1.70	1.74	1.82	1.83
160	0.76	1.44	1.53	1.57	1.59	1.62	1.65	1.70	1.79	1.80
170	0.74	1.40	1.48	1.52	1.54	1.57	1.60	1.66	1.74	1.75
180	0.72	1.35	1.44	1.47	1.49	1.52	1.56	1.62	1.69	1.70
190	0.70	1.31	1.40	1.43	1.45	1.48	1.51	1.57	1.64	1.65
200	0.68	1.27	1.36	1.39	1.41	1.43	1.47	1.52	1.59	1.60

Table 4. Mean Maximum Negative Moments Due to a Single Truck (Divided by HS20 Moment), ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.63	1.12	1.25	1.30	1.33	1.37	1.40	1.46	1.54	1.55
20	0.67	1.30	1.40	1.43	1.44	1.47	1.50	1.54	1.59	1.60
30	0.89	1.50	1.59	1.62	1.64	1.66	1.68	1.72	1.76	1.77
40	0.93	1.63	1.73	1.75	1.77	1.81	1.83	1.86	1.92	1.93
50	0.83	1.51	1.63	1.67	1.68	1.72	1.74	1.78	1.84	1.85
60	0.73	1.34	1.44	1.49	1.51	1.54	1.56	1.61	1.66	1.67
70	0.63	1.24	1.33	1.37	1.39	1.42	1.43	1.47	1.51	1.53
80	0.59	1.16	1.24	1.27	1.29	1.31	1.33	1.35	1.39	1.40
90	0.55	1.11	1.18	1.21	1.22	1.25	1.26	1.29	1.32	1.33
100	0.53	1.07	1.13	1.16	1.17	1.19	1.20	1.22	1.26	1.27
110	0.50	1.03	1.09	1.11	1.12	1.15	1.16	1.18	1.22	1.22
120	0.48	1.00	1.06	1.08	1.09	1.11	1.12	1.15	1.17	1.18
130	0.46	0.97	1.02	1.04	1.05	1.07	1.09	1.10	1.14	1.14
140	0.44	0.94	1.00	1.01	1.02	1.03	1.05	1.07	1.09	1.10
150	0.42	0.90	0.96	0.97	0.99	1.00	1.01	1.03	1.06	1.07
160	0.40	0.86	0.92	0.94	0.95	0.96	0.97	1.00	1.02	1.03
170	0.38	0.84	0.90	0.92	0.93	0.94	0.95	0.97	0.99	1.00
180	0.37	0.82	0.87	0.89	0.90	0.92	0.92	0.94	0.96	0.97
190	0.35	0.80	0.84	0.86	0.87	0.88	0.89	0.91	0.93	0.94
200	0.33	0.78	0.83	0.84	0.85	0.87	0.88	0.89	0.91	0.92

Table 5. Mean Maximum Moments for Simple Spans Due to a Single Truck (Divided by HL-93 Moment), ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.56	0.88	1.02	1.07	1.12	1.18	1.25	1.33	1.48	1.50
20	0.59	0.96	1.04	1.09	1.13	1.18	1.23	1.30	1.38	1.40
30	0.58	0.95	1.04	1.08	1.12	1.16	1.20	1.27	1.34	1.36
40	0.57	1.00	1.09	1.12	1.15	1.19	1.21	1.26	1.32	1.33
50	0.54	1.00	1.08	1.11	1.15	1.18	1.21	1.25	1.31	1.32
60	0.53	1.01	1.09	1.12	1.15	1.18	1.21	1.25	1.31	1.32
70	0.53	1.02	1.08	1.12	1.15	1.18	1.20	1.25	1.30	1.31
80	0.54	1.02	1.08	1.11	1.14	1.17	1.20	1.24	1.29	1.31
90	0.53	1.02	1.08	1.11	1.14	1.16	1.20	1.24	1.30	1.31
100	0.54	1.02	1.08	1.10	1.13	1.15	1.19	1.24	1.30	1.31
110	0.54	1.02	1.07	1.10	1.12	1.16	1.19	1.24	1.30	1.31
120	0.53	1.01	1.07	1.09	1.12	1.15	1.18	1.22	1.28	1.29
130	0.52	1.00	1.06	1.09	1.11	1.13	1.16	1.20	1.26	1.27
140	0.51	0.98	1.04	1.06	1.08	1.10	1.13	1.17	1.23	1.24
150	0.50	0.96	1.01	1.04	1.06	1.08	1.10	1.15	1.20	1.21
160	0.50	0.95	1.00	1.03	1.05	1.07	1.10	1.14	1.20	1.21
170	0.49	0.94	0.98	1.02	1.04	1.06	1.09	1.13	1.18	1.20
180	0.48	0.92	0.97	1.00	1.02	1.05	1.07	1.12	1.17	1.19
190	0.47	0.90	0.95	0.98	1.01	1.03	1.06	1.10	1.15	1.17
200	0.44	0.87	0.93	0.97	0.99	1.01	1.03	1.08	1.13	1.14

Table 6. Mean Maximum Shears for Simple Spans Due to a Single Truck (Divided by HL-93 Shear), ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.58	0.89	0.97	1.02	1.04	1.07	1.10	1.13	1.19	1.20
20	0.58	0.92	1.01	1.05	1.06	1.10	1.12	1.16	1.22	1.23
30	0.57	0.96	1.04	1.08	1.10	1.13	1.16	1.19	1.24	1.25
40	0.54	0.96	1.04	1.07	1.09	1.11	1.14	1.16	1.22	1.23
50	0.54	0.97	1.04	1.07	1.09	1.12	1.14	1.16	1.22	1.22
60	0.55	0.99	1.06	1.09	1.11	1.13	1.16	1.19	1.22	1.23
70	0.54	1.01	1.08	1.10	1.12	1.14	1.16	1.19	1.24	1.25
80	0.55	1.02	1.09	1.12	1.13	1.16	1.18	1.21	1.26	1.27
90	0.55	1.02	1.09	1.12	1.13	1.17	1.19	1.22	1.27	1.28
100	0.54	1.03	1.09	1.12	1.14	1.16	1.19	1.22	1.27	1.28
110	0.53	1.03	1.09	1.11	1.12	1.15	1.17	1.21	1.25	1.26
120	0.53	1.00	1.06	1.08	1.10	1.12	1.14	1.18	1.22	1.22
130	0.52	0.98	1.04	1.06	1.07	1.09	1.11	1.14	1.18	1.19
140	0.52	0.97	1.03	1.05	1.06	1.09	1.10	1.13	1.18	1.18
150	0.51	0.95	1.01	1.04	1.05	1.07	1.09	1.11	1.17	1.17
160	0.49	0.93	0.99	1.02	1.03	1.05	1.07	1.10	1.16	1.17
170	0.49	0.92	0.97	1.00	1.01	1.03	1.05	1.09	1.14	1.15
180	0.48	0.90	0.96	0.98	0.99	1.01	1.04	1.08	1.12	1.13
190	0.47	0.88	0.94	0.96	0.97	0.99	1.01	1.05	1.10	1.11
200	0.46	0.86	0.92	0.94	0.96	0.97	1.00	1.03	1.08	1.09

Table 7. Mean Maximum Negative Moments Due to a Single Truck (Divided by HL-93 Moment), ADTT = 1000

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.53	0.95	1.06	1.10	1.13	1.16	1.18	1.24	1.30	1.31
20	0.53	1.03	1.11	1.13	1.14	1.17	1.19	1.22	1.26	1.27
30	0.65	1.09	1.16	1.18	1.19	1.21	1.22	1.25	1.28	1.29
40	0.63	1.11	1.18	1.19	1.20	1.23	1.24	1.26	1.30	1.31
50	0.59	1.08	1.17	1.20	1.20	1.23	1.25	1.27	1.32	1.32
60	0.45	0.82	0.89	0.92	0.93	0.95	0.96	0.99	1.02	1.03
70	0.36	0.71	0.76	0.78	0.80	0.81	0.82	0.84	0.86	0.88
80	0.34	0.66	0.71	0.72	0.73	0.75	0.76	0.77	0.79	0.80
90	0.32	0.65	0.69	0.70	0.71	0.73	0.73	0.75	0.77	0.77
100	0.32	0.64	0.68	0.69	0.70	0.71	0.72	0.73	0.75	0.76
110	0.31	0.63	0.67	0.68	0.69	0.71	0.71	0.72	0.75	0.75
120	0.30	0.63	0.67	0.68	0.69	0.70	0.70	0.72	0.74	0.74
130	0.30	0.62	0.66	0.67	0.68	0.69	0.70	0.71	0.73	0.73
140	0.29	0.62	0.66	0.66	0.67	0.68	0.69	0.70	0.72	0.72
150	0.28	0.60	0.64	0.65	0.66	0.67	0.68	0.69	0.71	0.72
160	0.27	0.59	0.63	0.64	0.65	0.66	0.66	0.68	0.70	0.70
170	0.26	0.58	0.63	0.64	0.65	0.65	0.66	0.67	0.69	0.70
180	0.26	0.58	0.61	0.63	0.64	0.65	0.65	0.66	0.68	0.69
190	0.25	0.57	0.60	0.62	0.62	0.63	0.64	0.65	0.67	0.67
200	0.24	0.57	0.60	0.61	0.62	0.63	0.64	0.65	0.66	0.67

4.2.4. Field Observations and Engineering Judgment

The field observations were carried out by the research team at the University of Michigan, in conjunction with the development of live load model for bridges (NCHRP Report 368). The occurrence of multiple presence was recorded in several different locations on Interstate highways in Michigan. The considered roadways had two traffic lanes, with traffic moving in the same direction.

It was observed that about every 50th truck is followed by another truck with the headway distance less than 100 ft. It was also observed that about every 15th truck is on the bridge simultaneously with another truck side-by-side in a parallel lane.

It is important to consider the correlation between truck weights. The trucks can belong to the same company, they can carry the same load, and so on. Therefore, an important question is are the trucks in parallel lanes or in the same lane correlated? However, there is no field data available that can be used for developing a statistical model. Therefore, the statistical parameters for multiple presence of correlated trucks is based on engineering judgment.

For the case of a single traffic lane, it is assumed that about every 150th truck is followed by a partially correlated truck, and about every 500th truck is followed by a fully correlated truck with the headway distance less than 100 ft.

For two parallel traffic lanes, it is assumed that every 150th truck is on the bridge simultaneously with another partially correlated truck side-by-side in a parallel lane, and every 500th truck is with another fully correlated truck.

There is no field data for three or more than three parallel lanes. Therefore, the statistical parameters are based on engineering judgment. For three lanes, it is assumed that in addition to the first two lanes loaded, the third lane is loaded as follows: every 200th truck is on the bridge simultaneously with two other uncorrelated trucks side-by-side in parallel lanes, every 1500th truck is with two other partially correlated trucks, and every 5000th truck is with two other fully correlated trucks.

4.2.5. Moments and Shear Forces for Trucks in One Lane

For trucks in one lane, based on observations and engineering judgment, two cases were considered:

(a) traffic with a normal speed and headway distance of 50ft and dynamic load,
(b) bumper-to-bumper traffic with crawling speed and headway distance of 15ft and no dynamic load.

The statistical parameters of the lane moment and shear were obtained as follows. The total of the number of trucks is the survey is about 10,000, and corresponds to a two week traffic. Therefore, the number of trucks in one year is about 25 times larger, and in 75 years it is about 2000 times larger, i.e. 20,000,000. Therefore, the mean maximum 75 year moment (or shear) due to a single truck is obtained by extrapolation. For two trucks in one lane, the maximum total moment (or shear) is obtained as the largest combination of

(a) maximum truck out of 400,000 (every 50^{th} truck out of 20,000,000, so 20,000,000/50 = 400,000) vehicles and an average truck from the survey.
(b) maximum truck out of 150,000 (every 150^{th} truck, 20,000,000/150 = 150,000) vehicles and the maximum 1 day truck
(c) two trucks, each being the maximum out of 40,000 (every 500^{th} truck, 20,000,000/500 = 40,000)

An infinite number of possibilities exist. Given the approximate nature of the traffic observation, it seemed reasonable to pick three cases – a lower case, an upper case, and one in between as a check on process.

The results normalized using HS-20 live load are summarized for ADTT = 1000 in Table 8 for simple span moments, Table 9 for shears, and Table 10 for negative moments, and using HL-93 live load in Tables 11, 12 and 13, respectively. For ADTT = 5000, the results are summarized in Tables 14-19.

The bias factors for the maximum 75 year live load and ADTT = 5000 are plotted in Figures 10, 11 and 12, for a single truck and multiple trucks in one traffic lane.

The bias factors for live load, for ADTT = 5000 and for different time periods, are plotted in Figure 13 to 18 for HS-20 and HL-93.

Further explanation on the development of the data in Tables 8 through 13 can be found in Nowak and Hong (1991) and Nowak (1993).

Table 8. Mean Max. Moment for Simple Spans Due to Multiple Trucks in One Lane (Divided by HS20 Moment), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.97	1.12	1.18	1.23	1.30	1.37	1.46	1.65	1.65
20	1.08	1.18	1.23	1.28	1.33	1.38	1.47	1.58	1.58
30	1.20	1.32	1.37	1.42	1.46	1.52	1.78	1.72	1.72
40	1.31	1.42	1.46	1.50	1.55	1.58	1.64	1.74	1.74
50	1.32	1.43	1.47	1.52	1.56	1.60	1.65	1.75	1.75
60	1.37	1.47	1.52	1.56	1.60	1.64	1.69	1.79	1.79
70	1.42	1.51	1.56	1.60	1.64	1.68	1.74	1.83	1.83
80	1.47	1.55	1.60	1.64	1.68	1.73	1.79	1.89	1.89
90	1.51	1.60	1.64	1.68	1.72	1.78	1.84	1.94	1.94
100	1.55	1.64	1.68	1.72	1.76	1.82	1.89	2.00	2.00
110	1.60	1.68	1.72	1.76	1.81	1.86	1.94	2.05	2.05
120	1.63	1.72	1.76	1.80	1.85	1.90	1.97	2.08	2.08
130	1.66	1.75	1.80	1.83	1.87	1.92	1.99	2.10	2.10
140	1.67	1.76	1.81	1.84	1.87	1.92	1.99	2.10	2.10
150	1.67	1.76	1.80	1.83	1.87	1.92	1.99	2.10	2.10
160	1.65	1.74	1.79	1.82	1.85	1.90	1.97	2.08	2.08
170	1.63	1.71	1.77	1.80	1.84	1.88	1.95	2.06	2.06
180	1.60	1.68	1.73	1.77	1.81	1.85	1.92	2.03	2.03
190	1.56	1.65	1.70	1.74	1.78	1.82	1.89	2.00	2.00
200	1.52	1.62	1.67	1.71	1.74	1.79	1.85	1.96	1.96

Table 9. Mean Max. Shear for Simple Spans Due to Multiple Trucks in One Lane (Divided by HS20 Shear), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	1.20	1.31	1.38	1.40	1.44	1.48	1.52	1.61	1.62
20	1.14	1.25	1.30	1.31	1.36	1.38	1.43	1.51	1.52
30	1.14	1.24	1.29	1.31	1.35	1.38	1.42	1.48	1.49
40	1.18	1.28	1.32	1.34	1.37	1.40	1.43	1.50	1.51
50	1.24	1.33	1.37	1.39	1.43	1.45	1.48	1.55	1.56
60	1.30	1.40	1.44	1.46	1.49	1.52	1.56	1.61	1.62
70	1.37	1.47	1.50	1.52	1.55	1.58	1.62	1.69	1.70
80	1.43	1.53	1.57	1.59	1.63	1.66	1.70	1.77	1.78
90	1.48	1.58	1.62	1.64	1.69	1.72	1.76	1.84	1.85
100	1.53	1.63	1.67	1.70	1.73	1.77	1.82	1.89	1.90
110	1.57	1.66	1.70	1.72	1.76	1.80	1.85	1.92	1.93
120	1.59	1.67	1.71	1.73	1.77	1.80	1.86	1.92	1.93
130	1.59	1.68	1.71	1.73	1.77	1.79	1.85	1.91	1.92
140	1.57	1.67	1.70	1.72	1.75	1.77	1.82	1.90	1.91
150	1.53	1.63	1.67	1.69	1.72	1.75	1.79	1.87	1.88
160	1.50	1.59	1.63	1.65	1.68	1.71	1.76	1.84	1.85
170	1.47	1.56	1.60	1.62	1.65	1.68	1.74	1.81	1.82
180	1.44	1.53	1.56	1.58	1.61	1.65	1.71	1.78	1.79
190	1.41	1.50	1.53	1.55	1.58	1.61	1.67	1.74	1.75
200	1.39	1.48	1.51	1.53	1.55	1.59	1.64	1.71	1.72

Table 10. Mean Maximum Negative Moments for Continuous Spans Due to Multiple Trucks in One Lane (Divided by HS20 Neg. Moment), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	1.12	1.25	1.30	1.33	1.37	1.40	1.46	1.54	1.55
20	1.30	1.40	1.43	1.44	1.47	1.50	1.54	1.59	1.60
30	1.50	1.59	1.62	1.64	1.66	1.68	1.72	1.75	1.76
40	1.63	1.73	1.75	1.77	1.81	1.83	1.86	1.91	1.92
50	1.58	1.67	1.69	1.71	1.75	1.77	1.80	1.85	1.86
60	1.72	1.83	1.85	1.87	1.92	1.94	1.97	2.02	2.03
70	1.80	1.92	1.94	1.96	2.01	2.03	2.06	2.12	2.13
80	1.80	1.91	1.94	1.96	2.00	2.03	2.06	2.12	2.13
90	1.75	1.86	1.88	1.90	1.95	1.97	2.00	2.06	2.07
100	1.70	1.81	1.83	1.85	1.89	1.91	1.94	2.00	2.01
110	1.66	1.76	1.78	1.80	1.84	1.86	1.89	1.95	1.96
120	1.62	1.72	1.74	1.76	1.80	1.82	1.85	1.90	1.91
130	1.58	1.68	1.70	1.72	1.76	1.78	1.81	1.85	1.86
140	1.55	1.64	1.66	1.68	1.72	1.74	1.77	1.81	1.82
150	1.52	1.61	1.63	1.65	1.69	1.70	1.73	1.78	1.79
160	1.49	1.58	1.60	1.62	1.66	1.67	1.70	1.75	1.76
170	1.46	1.55	1.57	1.59	1.63	1.64	1.67	1.72	1.73
180	1.44	1.53	1.55	1.57	1.60	1.62	1.64	1.69	1.70
190	1.42	1.51	1.52	1.54	1.58	1.59	1.62	1.66	1.67
200	1.42	1.48	1.50	1.52	1.55	1.57	1.60	1.64	1.65

Table 11. Mean Maximum Moments for Simple Spans Due to Multiple Trucks in One Lane (Divided by HL-93 Moment), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.88	1.02	1.07	1.12	1.18	1.25	1.33	1.50	1.50
20	0.90	0.98	1.03	1.07	1.11	1.15	1.23	1.32	1.32
30	0.95	1.04	1.08	1.12	1.15	1.20	1.41	1.36	1.36
40	1.00	1.09	1.12	1.15	1.19	1.21	1.26	1.33	1.33
50	1.00	1.08	1.11	1.15	1.18	1.21	1.25	1.32	1.32
60	1.01	1.09	1.12	1.15	1.18	1.21	1.25	1.32	1.32
70	1.02	1.08	1.12	1.15	1.18	1.20	1.25	1.31	1.31
80	1.02	1.08	1.11	1.14	1.17	1.20	1.24	1.31	1.31
90	1.02	1.08	1.11	1.14	1.16	1.20	1.24	1.31	1.31
100	1.02	1.08	1.10	1.13	1.15	1.19	1.24	1.31	1.31
110	1.02	1.07	1.10	1.12	1.16	1.19	1.24	1.31	1.31
120	1.01	1.07	1.09	1.12	1.15	1.18	1.22	1.29	1.29
130	1.00	1.06	1.09	1.11	1.13	1.16	1.20	1.27	1.27
140	0.98	1.04	1.07	1.08	1.10	1.13	1.17	1.24	1.24
150	0.98	1.03	1.06	1.07	1.10	1.13	1.17	1.23	1.23
160	0.98	1.04	1.07	1.08	1.10	1.13	1.17	1.24	1.24
170	0.98	1.03	1.07	1.09	1.11	1.14	1.18	1.24	1.24
180	0.98	1.03	1.06	1.08	1.11	1.13	1.18	1.24	1.24
190	0.97	1.02	1.06	1.08	1.11	1.13	1.17	1.24	1.24
200	0.96	1.02	1.05	1.08	1.09	1.13	1.16	1.23	1.23

Table 12. Mean Maximum Shears for Simple Spans Due to Multiple Trucks in One Lane (Divided by HL-93 Shear), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.89	0.97	1.02	1.04	1.07	1.10	1.13	1.19	1.20
20	0.92	1.01	1.05	1.06	1.10	1.12	1.16	1.22	1.23
30	0.96	1.04	1.08	1.10	1.13	1.16	1.19	1.24	1.25
40	0.96	1.04	1.07	1.09	1.11	1.14	1.16	1.22	1.23
50	0.97	1.04	1.07	1.09	1.12	1.14	1.16	1.22	1.22
60	0.99	1.06	1.09	1.11	1.13	1.16	1.19	1.22	1.23
70	1.01	1.08	1.10	1.12	1.14	1.16	1.19	1.24	1.25
80	1.02	1.09	1.12	1.13	1.16	1.18	1.21	1.26	1.27
90	1.02	1.09	1.12	1.13	1.17	1.19	1.22	1.27	1.28
100	1.03	1.09	1.12	1.14	1.16	1.19	1.22	1.27	1.28
110	1.02	1.08	1.11	1.12	1.15	1.17	1.21	1.25	1.26
120	1.01	1.06	1.08	1.10	1.12	1.14	1.18	1.22	1.22
130	0.99	1.05	1.07	1.08	1.10	1.12	1.15	1.19	1.20
140	0.99	1.06	1.07	1.09	1.11	1.12	1.15	1.20	1.21
150	0.98	1.04	1.07	1.08	1.10	1.12	1.15	1.20	1.20
160	0.97	1.03	1.06	1.07	1.09	1.11	1.14	1.19	1.20
170	0.96	1.02	1.05	1.06	1.08	1.10	1.14	1.19	1.19
180	0.96	1.02	1.04	1.05	1.07	1.10	1.14	1.18	1.19
190	0.95	1.01	1.03	1.04	1.06	1.08	1.12	1.17	1.17
200	0.94	1.00	1.02	1.04	1.05	1.08	1.11	1.16	1.17

Table 13. Mean Maximum Negative Moments for Continuous Spans Due to Multiple Trucks in One Lane (Divided by HL-93 Neg. Moment), ADTT = 1000

Span (ft)	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.95	1.06	1.10	1.13	1.16	1.18	1.24	1.30	1.31
20	1.03	1.11	1.13	1.14	1.17	1.19	1.22	1.26	1.27
30	1.09	1.16	1.18	1.19	1.21	1.22	1.25	1.27	1.28
40	1.11	1.18	1.19	1.20	1.23	1.24	1.26	1.30	1.30
50	1.13	1.20	1.21	1.22	1.25	1.27	1.29	1.32	1.33
60	1.06	1.13	1.14	1.15	1.18	1.19	1.21	1.24	1.25
70	1.03	1.10	1.11	1.12	1.15	1.16	1.18	1.21	1.22
80	1.02	1.09	1.10	1.12	1.14	1.16	1.17	1.21	1.21
90	1.02	1.08	1.09	1.10	1.13	1.14	1.16	1.20	1.20
100	1.02	1.08	1.09	1.11	1.13	1.14	1.16	1.19	1.20
110	1.02	1.08	1.09	1.10	1.13	1.14	1.16	1.20	1.20
120	1.02	1.08	1.09	1.11	1.13	1.14	1.16	1.20	1.20
130	1.02	1.08	1.09	1.11	1.13	1.15	1.16	1.19	1.20
140	1.02	1.08	1.09	1.10	1.13	1.14	1.16	1.19	1.20
150	1.02	1.08	1.09	1.11	1.13	1.14	1.16	1.19	1.20
160	1.02	1.08	1.09	1.11	1.13	1.14	1.16	1.20	1.20
170	1.02	1.08	1.09	1.11	1.13	1.14	1.16	1.20	1.20
180	1.02	1.08	1.10	1.11	1.13	1.15	1.16	1.19	1.20
190	1.02	1.08	1.09	1.10	1.13	1.14	1.16	1.19	1.20
200	1.03	1.08	1.09	1.11	1.13	1.14	1.16	1.19	1.20

Table 14. Mean Maximum Moment for Simple Spans due to Multiple Trucks in One Lane (Divided by Corresponding HS20 Moment, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.99	1.15	1.21	1.26	1.33	1.40	1.50	1.67	1.69
20	1.18	1.28	1.34	1.39	1.45	1.51	1.60	1.70	1.72
30	1.23	1.35	1.40	1.46	1.51	1.56	1.65	1.74	1.76
40	1.34	1.46	1.50	1.54	1.59	1.62	1.68	1.76	1.78
50	1.35	1.47	1.51	1.56	1.60	1.64	1.69	1.77	1.79
60	1.40	1.51	1.56	1.60	1.64	1.68	1.73	1.81	1.83
70	1.46	1.55	1.60	1.64	1.68	1.72	1.78	1.86	1.88
80	1.51	1.59	1.64	1.68	1.72	1.77	1.83	1.91	1.94
90	1.55	1.64	1.68	1.72	1.76	1.82	1.89	1.97	1.99
100	1.59	1.68	1.72	1.76	1.80	1.87	1.94	2.03	2.05
110	1.64	1.72	1.76	1.80	1.86	1.91	1.99	2.08	2.10
120	1.67	1.76	1.80	1.85	1.90	1.95	2.02	2.11	2.13
130	1.70	1.79	1.85	1.88	1.92	1.97	2.04	2.13	2.15
140	1.71	1.80	1.85	1.88	1.92	1.97	2.04	2.13	2.15
150	1.71	1.80	1.85	1.88	1.92	1.97	2.04	2.15	2.15
160	1.69	1.78	1.83	1.87	1.90	1.95	2.02	2.13	2.13
170	1.67	1.75	1.81	1.85	1.89	1.93	2.00	2.11	2.11
180	1.64	1.72	1.77	1.81	1.86	1.90	1.97	2.08	2.08
190	1.60	1.69	1.74	1.78	1.82	1.87	1.94	2.05	2.05
200	1.56	1.66	1.71	1.75	1.78	1.83	1.90	2.01	2.01

Table 15. Mean Maximum Moment for Simple Spans due to Multiple Trucks in One Lane (Divided by Corresponding HL93 Moment, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.90	1.05	1.10	1.15	1.21	1.28	1.36	1.54	1.54
20	0.94	1.06	1.10	1.15	1.19	1.25	1.33	1.46	1.46
30	0.97	1.07	1.11	1.15	1.18	1.22	1.29	1.38	1.38
40	1.05	1.14	1.17	1.20	1.24	1.26	1.31	1.38	1.38
50	1.03	1.11	1.15	1.18	1.21	1.25	1.28	1.36	1.36
60	1.05	1.12	1.15	1.18	1.21	1.24	1.27	1.35	1.35
70	1.05	1.11	1.15	1.18	1.21	1.24	1.28	1.34	1.34
80	1.05	1.11	1.14	1.17	1.20	1.24	1.28	1.35	1.35
90	1.05	1.11	1.14	1.17	1.19	1.23	1.27	1.34	1.34
100	1.05	1.11	1.13	1.16	1.19	1.23	1.27	1.34	1.34
110	1.05	1.10	1.13	1.15	1.18	1.22	1.27	1.34	1.34
120	1.04	1.10	1.12	1.15	1.18	1.21	1.25	1.32	1.32
130	1.03	1.09	1.12	1.14	1.16	1.19	1.23	1.30	1.30
140	1.00	1.06	1.09	1.11	1.13	1.16	1.20	1.27	1.27
150	1.00	1.06	1.09	1.10	1.12	1.15	1.19	1.26	1.26
160	1.00	1.06	1.10	1.12	1.13	1.16	1.20	1.27	1.27
170	1.01	1.06	1.10	1.12	1.14	1.16	1.21	1.27	1.27
180	1.00	1.06	1.09	1.11	1.14	1.16	1.20	1.27	1.27
190	0.99	1.05	1.09	1.11	1.13	1.16	1.20	1.27	1.27
200	0.98	1.05	1.08	1.11	1.12	1.15	1.19	1.26	1.26

Table 16. Mean Maximum Shear for Single Spans due to Multiple Trucks in One Lane (Divided by Corresponding HS20 Shears, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	1.24	1.36	1.43	1.45	1.49	1.53	1.57	1.67	1.68
20	1.18	1.29	1.35	1.36	1.41	1.43	1.48	1.56	1.57
30	1.18	1.28	1.34	1.36	1.40	1.43	1.47	1.53	1.54
40	1.22	1.32	1.37	1.39	1.42	1.45	1.48	1.55	1.56
50	1.28	1.38	1.42	1.44	1.48	1.50	1.53	1.60	1.61
60	1.35	1.45	1.49	1.51	1.54	1.57	1.61	1.67	1.68
70	1.42	1.52	1.55	1.57	1.60	1.64	1.68	1.75	1.76
80	1.48	1.58	1.62	1.65	1.69	1.72	1.76	1.83	1.84
90	1.53	1.64	1.68	1.70	1.75	1.78	1.82	1.90	1.91
100	1.58	1.69	1.73	1.76	1.79	1.83	1.88	1.96	1.97
110	1.64	1.73	1.76	1.78	1.82	1.86	1.91	1.99	2.00
120	1.64	1.73	1.77	1.79	1.83	1.86	1.93	1.99	2.00
130	1.65	1.74	1.77	1.79	1.83	1.85	1.91	1.98	1.99
140	1.62	1.73	1.76	1.78	1.81	1.83	1.88	1.97	1.98
150	1.58	1.69	1.73	1.75	1.78	1.81	1.85	1.94	1.95
160	1.55	1.65	1.69	1.71	1.74	1.77	1.82	1.90	1.91
170	1.52	1.61	1.66	1.68	1.71	1.74	1.80	1.87	1.88
180	1.49	1.58	1.61	1.64	1.67	1.71	1.77	1.84	1.85
190	1.46	1.55	1.58	1.60	1.64	1.67	1.73	1.80	1.81
200	1.44	1.53	1.56	1.58	1.60	1.65	1.70	1.77	1.78

Table 17. Mean Maximum Shear for Single Spans due to Multiple Trucks in One Lane (Divided by Corresponding HL93 Shear, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.92	1.00	1.06	1.08	1.11	1.13	1.17	1.23	1.24
20	0.95	1.05	1.09	1.10	1.14	1.16	1.20	1.26	1.27
30	0.98	1.07	1.12	1.13	1.17	1.19	1.23	1.28	1.29
40	0.99	1.08	1.11	1.13	1.15	1.18	1.20	1.26	1.27
50	1.00	1.08	1.11	1.13	1.16	1.18	1.20	1.26	1.26
60	1.02	1.10	1.13	1.15	1.17	1.19	1.23	1.26	1.27
70	1.04	1.12	1.14	1.16	1.18	1.20	1.23	1.28	1.29
80	1.06	1.13	1.16	1.17	1.20	1.22	1.25	1.30	1.31
90	1.06	1.13	1.16	1.17	1.21	1.23	1.26	1.31	1.32
100	1.07	1.13	1.16	1.18	1.20	1.22	1.26	1.31	1.32
110	1.06	1.12	1.15	1.16	1.19	1.21	1.25	1.29	1.30
120	1.04	1.10	1.12	1.14	1.16	1.18	1.22	1.26	1.26
130	1.02	1.08	1.11	1.12	1.14	1.16	1.19	1.23	1.24
140	1.02	1.09	1.11	1.12	1.15	1.16	1.19	1.24	1.24
150	1.01	1.08	1.11	1.12	1.14	1.16	1.19	1.24	1.24
160	1.00	1.07	1.09	1.11	1.13	1.15	1.18	1.23	1.24
170	0.99	1.06	1.09	1.10	1.12	1.14	1.18	1.23	1.23
180	0.99	1.06	1.08	1.09	1.11	1.14	1.17	1.22	1.23
190	0.98	1.05	1.07	1.08	1.10	1.12	1.16	1.21	1.21
200	0.97	1.04	1.06	1.08	1.09	1.12	1.15	1.20	1.21

Table 18. Mean Maximum Negative Moment for Continuous Spans due to Multiple Trucks in One Lane (Divided by Corresponding HS20 Negative Moment, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	1.15	1.29	1.34	1.37	1.41	1.44	1.50	1.59	1.60
20	1.34	1.44	1.47	1.48	1.51	1.55	1.59	1.64	1.65
30	1.55	1.64	1.67	1.69	1.71	1.73	1.77	1.81	1.82
40	1.68	1.78	1.80	1.82	1.86	1.88	1.92	1.98	1.99
50	1.56	1.68	1.72	1.73	1.77	1.79	1.83	1.90	1.91
60	1.76	1.88	1.90	1.92	1.97	1.99	2.02	2.07	2.08
70	1.85	1.97	1.99	2.01	2.06	2.08	2.11	2.17	2.18
80	1.85	1.96	1.99	2.01	2.05	2.08	2.11	2.17	2.18
90	1.79	1.91	1.93	1.95	2.00	2.02	2.05	2.11	2.12
100	1.74	1.86	1.88	1.90	1.94	1.96	1.99	2.05	2.06
110	1.70	1.80	1.82	1.85	1.89	1.91	1.94	2.00	2.01
120	1.66	1.76	1.78	1.80	1.85	1.87	1.90	1.95	1.96
130	1.62	1.72	1.74	1.76	1.80	1.82	1.86	1.90	1.91
140	1.59	1.68	1.70	1.72	1.76	1.78	1.81	1.86	1.87
150	1.56	1.65	1.67	1.69	1.73	1.74	1.77	1.82	1.83
160	1.53	1.62	1.64	1.66	1.70	1.71	1.74	1.79	1.80
170	1.50	1.59	1.61	1.63	1.67	1.68	1.71	1.76	1.77
180	1.48	1.57	1.59	1.61	1.64	1.66	1.68	1.73	1.74
190	1.46	1.55	1.56	1.58	1.62	1.63	1.66	1.70	1.71
200	1.46	1.52	1.54	1.56	1.59	1.61	1.64	1.68	1.69

Table 19. Mean Maximum Negative Moment for Continuous Spans due to Multiple Trucks in One Lane (Divided by Corresponding HL93 Negative Moment, ADTT = 5000)

Span	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.96	1.09	1.13	1.15	1.18	1.21	1.26	1.33	1.34
20	1.06	1.14	1.17	1.17	1.20	1.22	1.25	1.29	1.30
30	1.12	1.18	1.21	1.22	1.24	1.25	1.28	1.30	1.31
40	1.14	1.20	1.22	1.23	1.25	1.27	1.29	1.33	1.33
50	1.16	1.23	1.24	1.26	1.29	1.30	1.32	1.36	1.36
60	1.09	1.16	1.17	1.18	1.21	1.22	1.24	1.27	1.28
70	1.06	1.13	1.14	1.15	1.18	1.19	1.21	1.25	1.25
80	1.06	1.12	1.13	1.15	1.17	1.18	1.20	1.24	1.24
90	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.23	1.23
100	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.23	1.23
110	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.23	1.23
120	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
130	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
140	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
150	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
160	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.23	1.23
170	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.23	1.23
180	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
190	1.05	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23
200	1.06	1.11	1.12	1.14	1.16	1.17	1.19	1.22	1.23

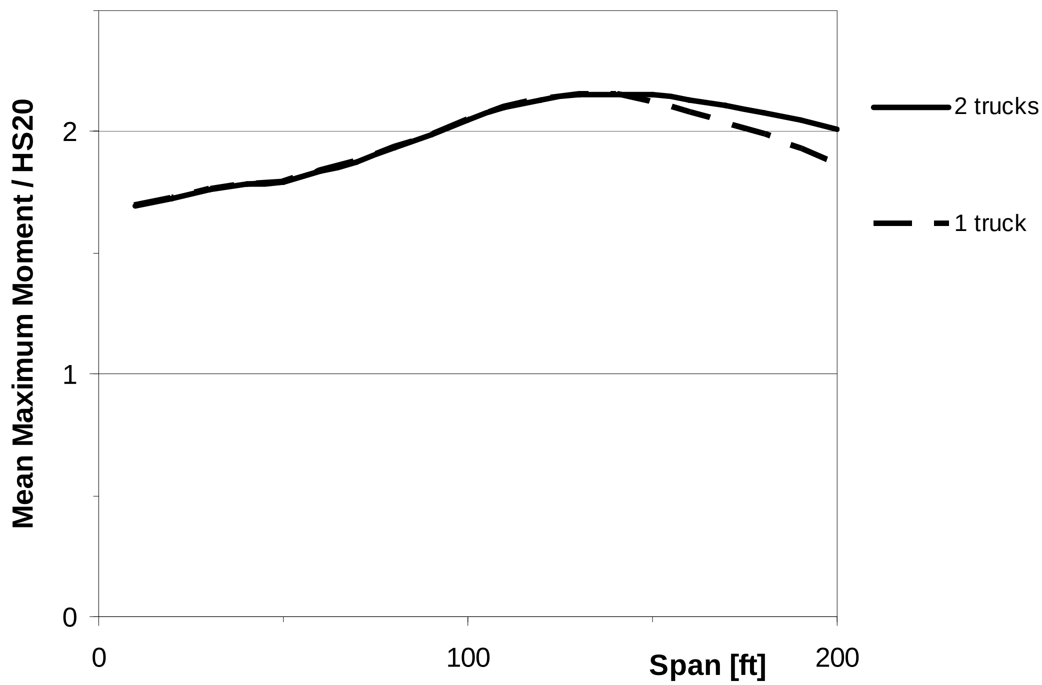

Figure 10 - Mean Maximum 75 Year Simple Span Moment for One Lane Loaded (Divided by Corresponding HS20 Moment, ADTT = 5000)

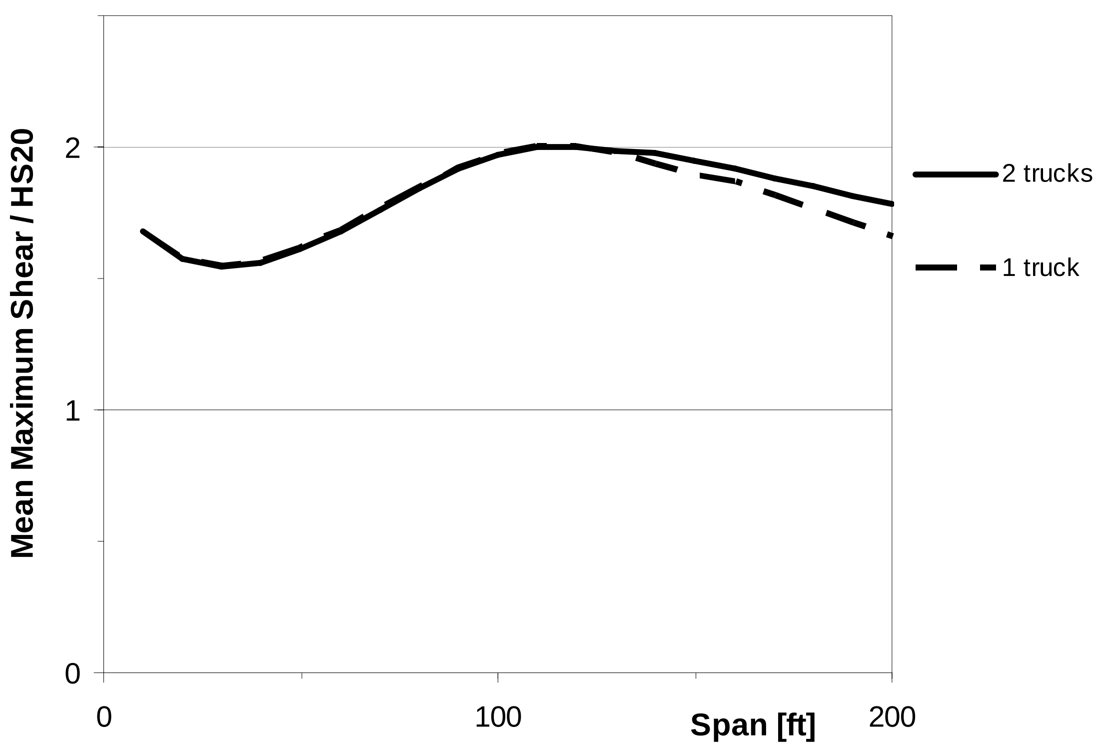

Figure 11 - Mean Maximum 75 Year Shear for One Lane Loaded (Divided by Corresponding HS20 Shear, ADTT = 5000)

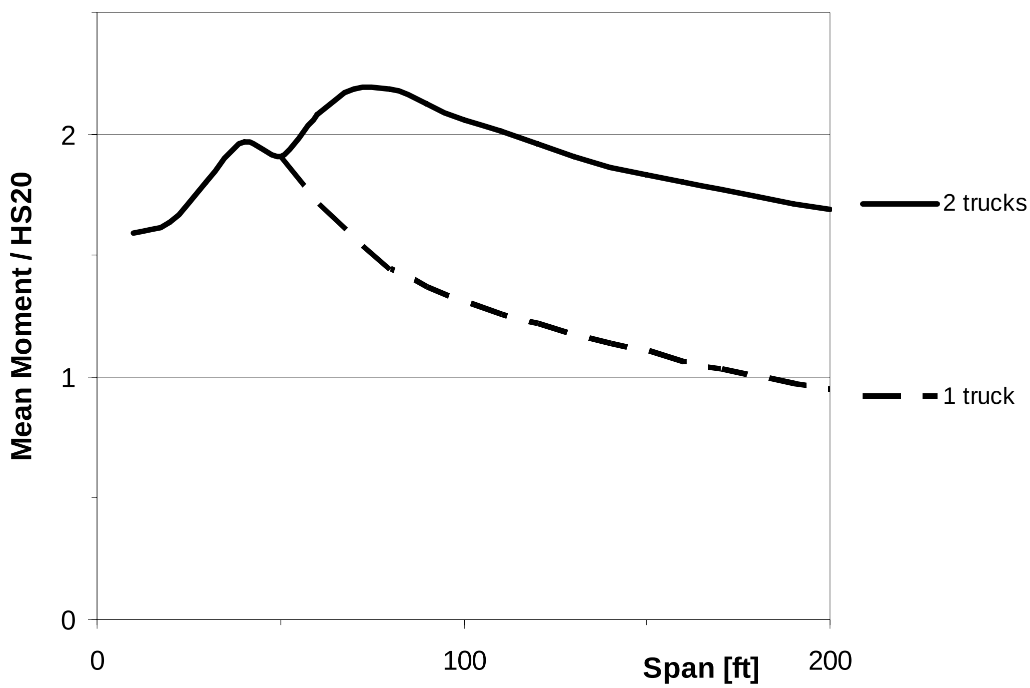

Figure 12 - Mean Maximum 75 Year Negative Moment for One Lane Loaded (Divided by Corresponding HS20 Moment, ADTT = 5000)

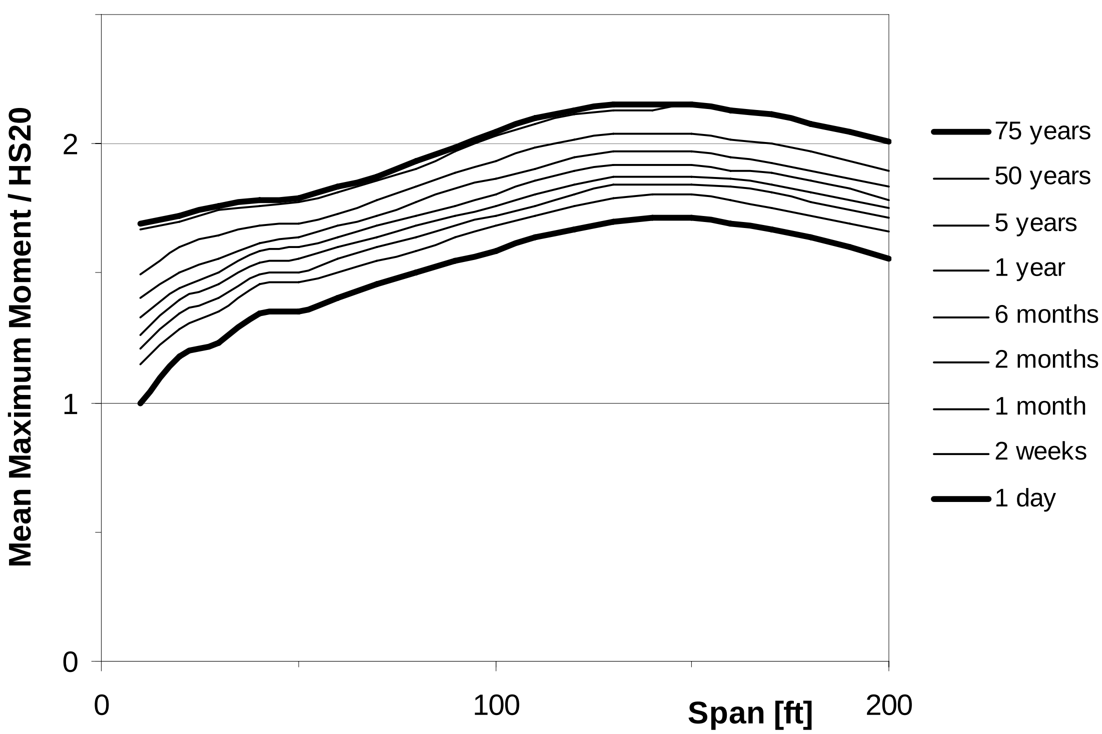

Figure 13 - Mean Maximum Moment for Simple Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HS20 Moment, ADTT = 5000)

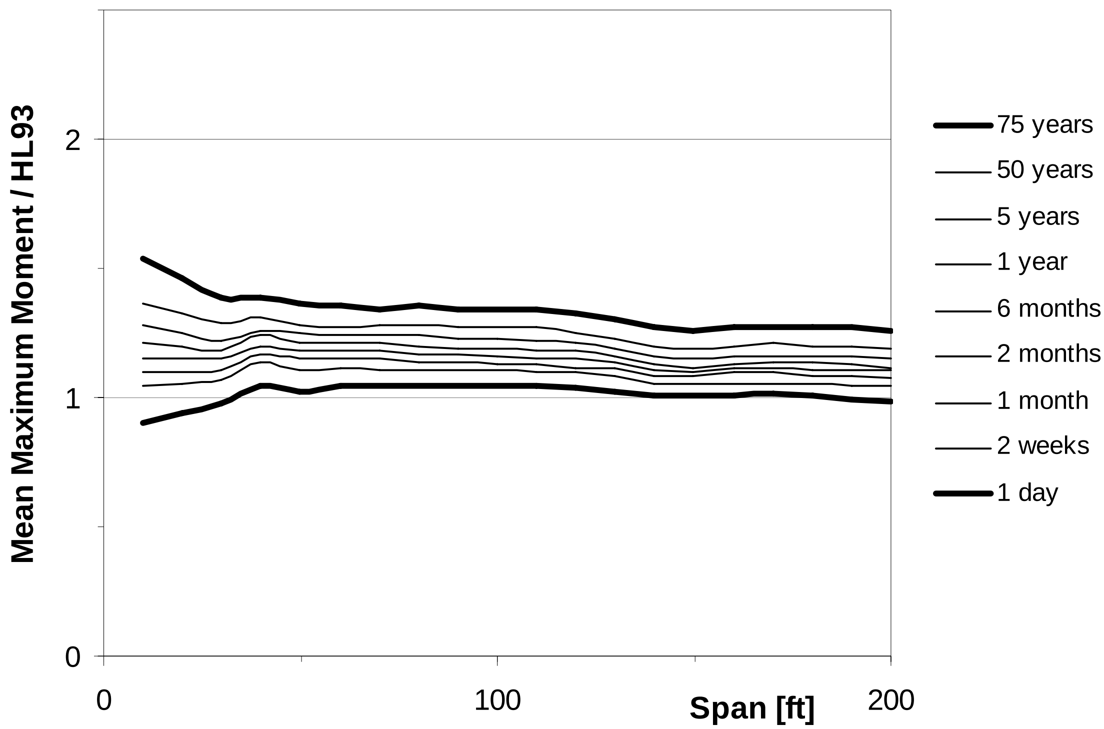

Figure 14 - Mean Maximum Moment for Simple Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HL93 Moment, ADTT = 5000)

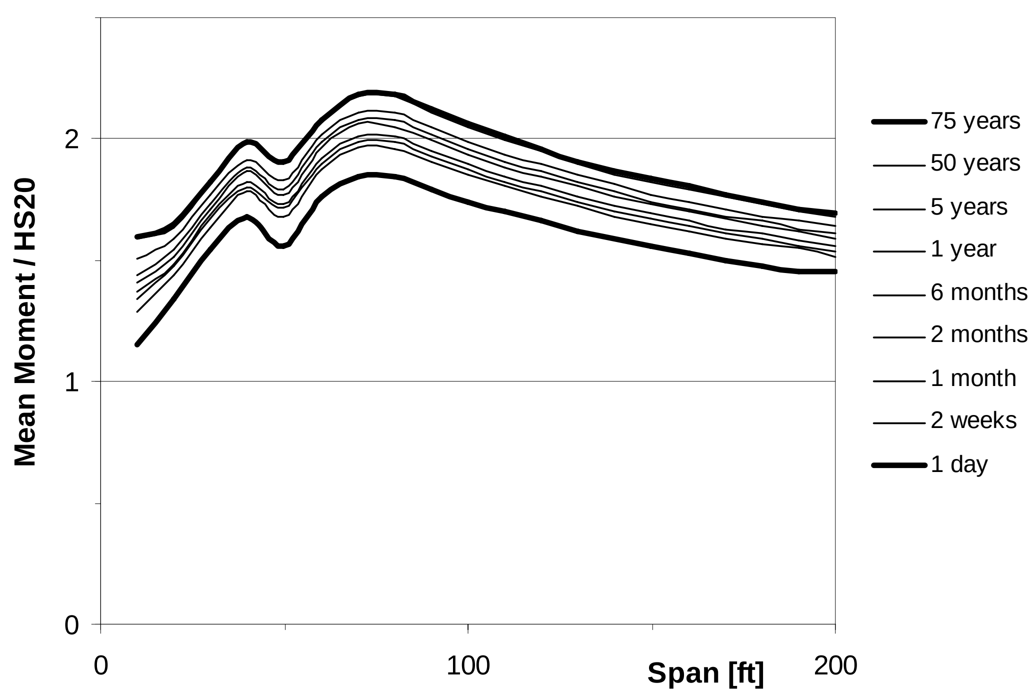

Figure 15 - Mean Maximum Negative Moment for Continuous Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HS20 Moment, ADTT = 5000)

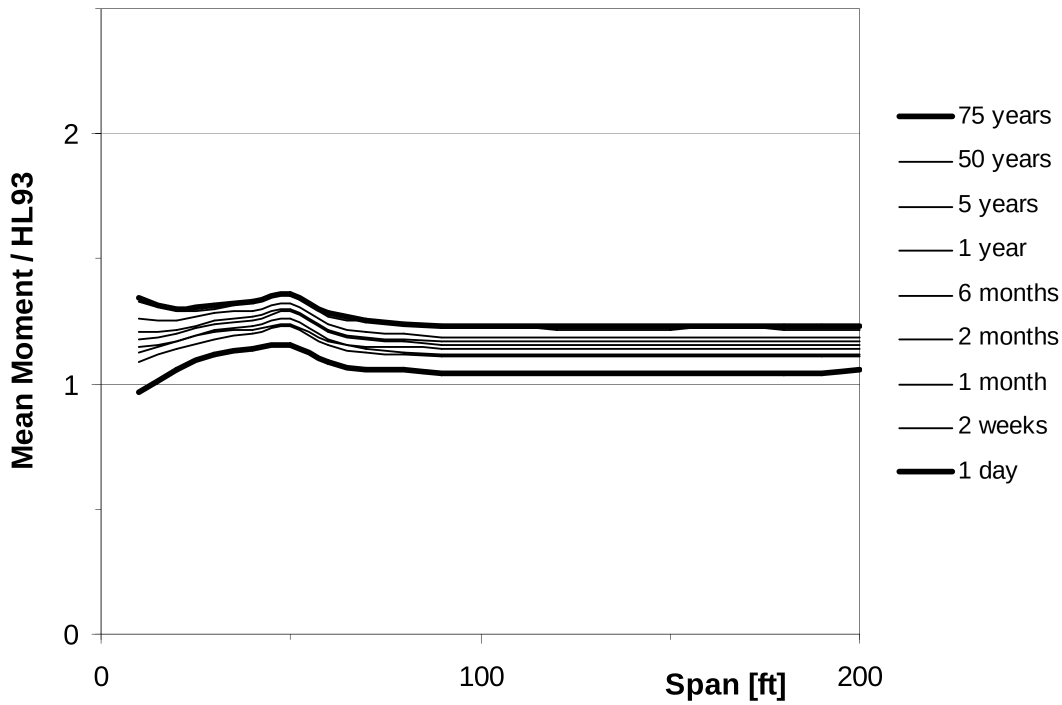

Figure 16 - Mean Maximum Negative Moments for Continuous Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HL93 Moment, ADTT = 5000)

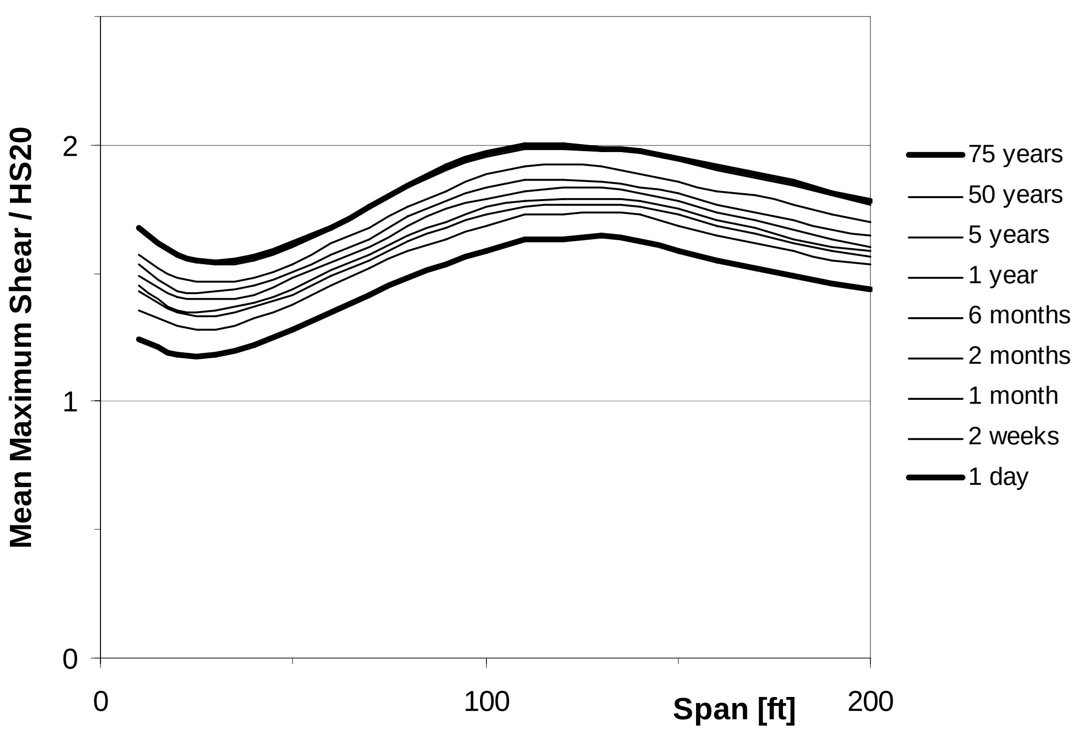

Figure 17 - Mean Maximum Shear for Single Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HS20 Shear, ADTT = 5000)

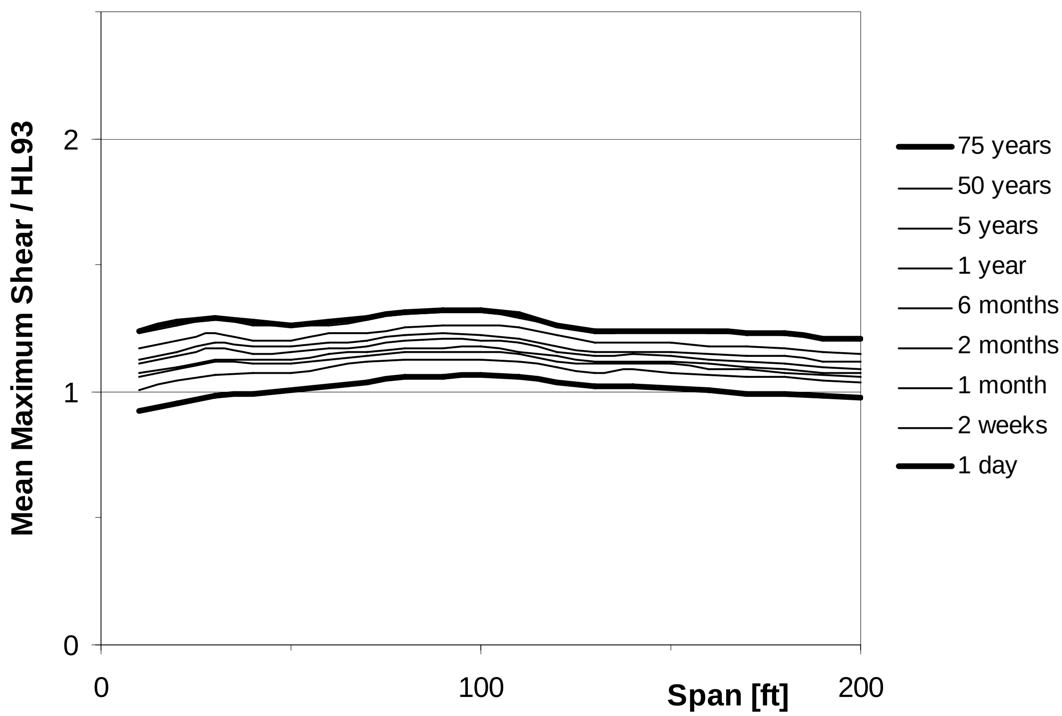

Figure 18 - Mean Maximum Shears for Single Spans for One Lane Loaded and Different Time Periods (Divided by Corresponding HL93 Shear, ADTT = 5000)

4.2.6. Multiple Presence in Parallel Lanes

For two parallel lanes, the statistical parameters of the moment and shear were obtained as the largest combination of

(a) maximum truck out of 1,500,000 (every 15th truck out of 20,000,000, so 20,000,000/15 = about 1,500,000) vehicles and an average truck from the survey.

(b) maximum truck out of 150,000 (every 150th truck, 20,000,000/150 = about 150,000) vehicles and the maximum truck out of 1000.

(c) two trucks, each being the maximum out of 40,000 (every 500th truck, 20,000,000/500 = 40,000). Note that 40,000 trucks correspond to two months traffic based on about 10,000 trucks in the survey of two weeks traffic.

For three parallel traffic lanes, the statistical parameters of the moment and shear were obtained as the largest combination of two lanes (a) or (b) or (c) and the third lane:

(d) maximum truck out of 100,000 (every 200th truck out of 20,000,000, so 20,000,000/200 = about 100,000) vehicles and two average trucks from the survey.

(e) maximum truck out of 15,000 (every 1500th truck, 20,000,000/1500 = 15,000) vehicles, the maximum truck out 500, and an average truck

(f) three fully correlated trucks, each being the maximum out of 100 (every 250,000th truck, 20,000,000/250,000 = about 100)

For four parallel traffic lanes, the statistical parameters of the moment and shear were obtained as the largest combination of

(g) maximum truck out of 5000 (every 3000th truck out of 20,000,000, so 20,000,000/3000 = about 5000) vehicles and three average trucks from the survey.

(h) maximum truck out of 500 (every 30,000th truck, 20,000,000/30,000 = about 500) vehicles, the maximum truck out of 100, and two average trucks

(i) four trucks, each being the maximum out of 50 (every 500,000th truck, 20,000,000/500,000 = about 50)

4.2.7. Calculation of Multiple Presence Factors

The multiple presence factor for two lanes can be obtained as a ratio of the maximum 40,000th truck (corresponding to 1-2 month truck) moment (or shear) and the maximum 75 year truck moment (or shear) in Tables 2.7-2.12. For the simple span moment this ratio is about 0.85-0.87, and for shear it is 0.88-0.89.

For three lanes, the multiple presence factor is equal to the ratio of the maximum 100th truck (corresponding to less than the maximum 1 day truck) and the maximum 75 year truck effect in Tables 2.7-2.12. For 1 day truck, the simple span moment this ratio is 0.77-0.79, and for shear it is 0.80-0.82.

For four lanes, the multiple presence factors are equal to the ratio of the average truck and the maximum 75 year truck effect.

In AASHTO LRFD (2004), it is assumed that the multiple presence factor for two lane bridge is 1.00. Therefore, according to this study, the multiple presence factor for a one lane loaded is the inverse of 0.85-0.87 for moment and inverse of 0.88-0.89 for shear, i.e. 1.16-1.19 for moment and 1.14-1.16 for shear.

For three lanes loaded, the multiple presence factor can be taken as about 0.85 (as 0.9 corresponds to the maximum 1 day truck).

For four lanes, the multiple presence factor is further reduced, and can be taken as 0.65

4.2.8. ADTT and Multiple Presence Factors

The analysis presented in this report is carried out for ADTT = 1000. For ADTT = 5000, the number of vehicles throughout the calculations will have to be multiplied by 5. The multiple presence factors will practically not be affected by the change in ADTT. However, the mean maximum values of live load will be increased by up to 5%.

4.2.9. Recommended Multiple Presence Factors

The recommended multiple presence factor are as follows:

(a) for a single lane loaded: 1.20 for moment and 1.15 for shear
(b) for two lanes loaded: 1.00 for moment and shear
(c) for three lanes loaded: 0.85 for moment and shear
(d) for four lanes loaded: 0.65 for moment and shear

4.2.10. Statistical Parameters of Live Load

The statistical parameters include bias factors and coefficients of variation. The bias factors for the 75 year maximum live load are shown in Figure 19 for one loaded lane, and Figure 20 for two loaded lanes, for simple span moment, Figure 21 for one loaded lane, and Figure 22 for two loaded lanes, for shear, and Figure 23 for one loaded lane, and Figure 24 for two loaded lanes, for negative moment, all for ADTT = 5000. The resulting values are presented for HS-20 and HL-93.

For two loaded lanes, the maximum 75 year live load is caused by the maximum two month live load effect in each lane. Therefore, the bias factor for two lanes can be determined from the tabulated values corresponding to two month period of time.

The bias factors are summarized in Table 20 for one and two lanes loaded, and for ADTT = 1000 and ADTT = 5000. The corresponding coefficient of variation for the 75 year maximum live load effect is 0.12 for all considered cases. It was obtained as the tangent of the slope of the CDF of the maximum 75 live load.

Table 20. Bias Factors for Live Load

	Bias Factor (HL-93)	
	ADTT = 1000	**ADTT = 5000**
One Lane Loaded:		
- simple span moment	1.23 - 1.36	1.26 - 1.38
- shear	1.17 - 1.28	1.21 - 1.32
- negative moment	1.20 - 1.33	1.23 - 1.36
Two Lanes Loaded:		
- simple span moment	1.08 - 1.15	1.10 - 1.20
- shear	1.04 - 1.14	1.08 - 1.18
- negative moment	1.10 - 1.22	1.14 - 1.26

Coefficient of variation = 0.12 for all cases

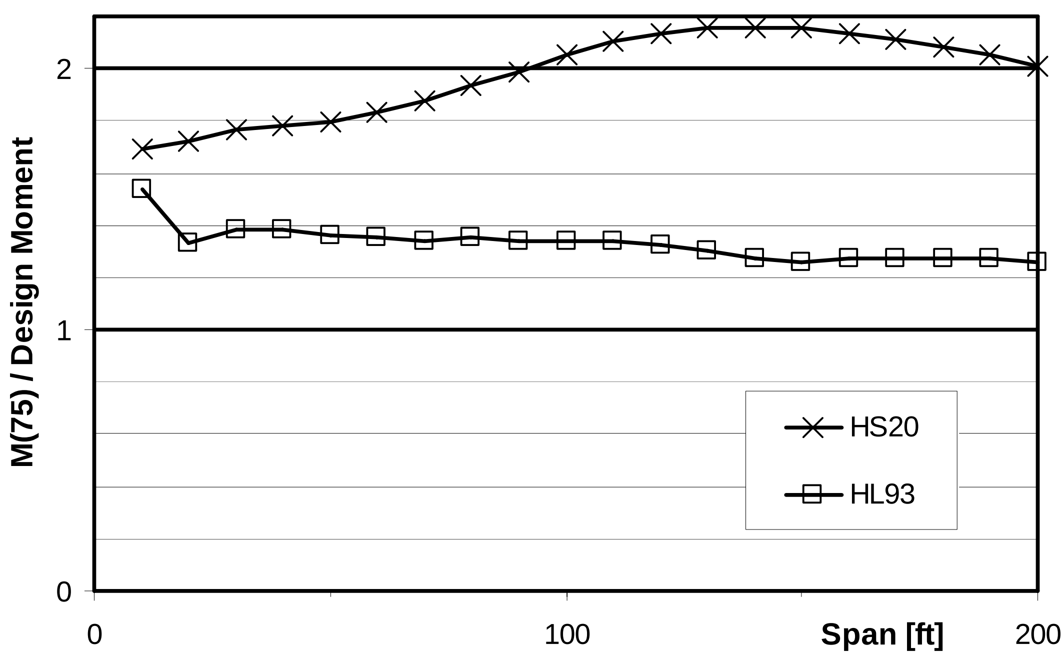

Figure 19 - Bias Factor for One Lane Loaded, Simple Span Moment; Ratio M(75)/M(HL93) and M(75)/M(HS20), ADTT = 5000

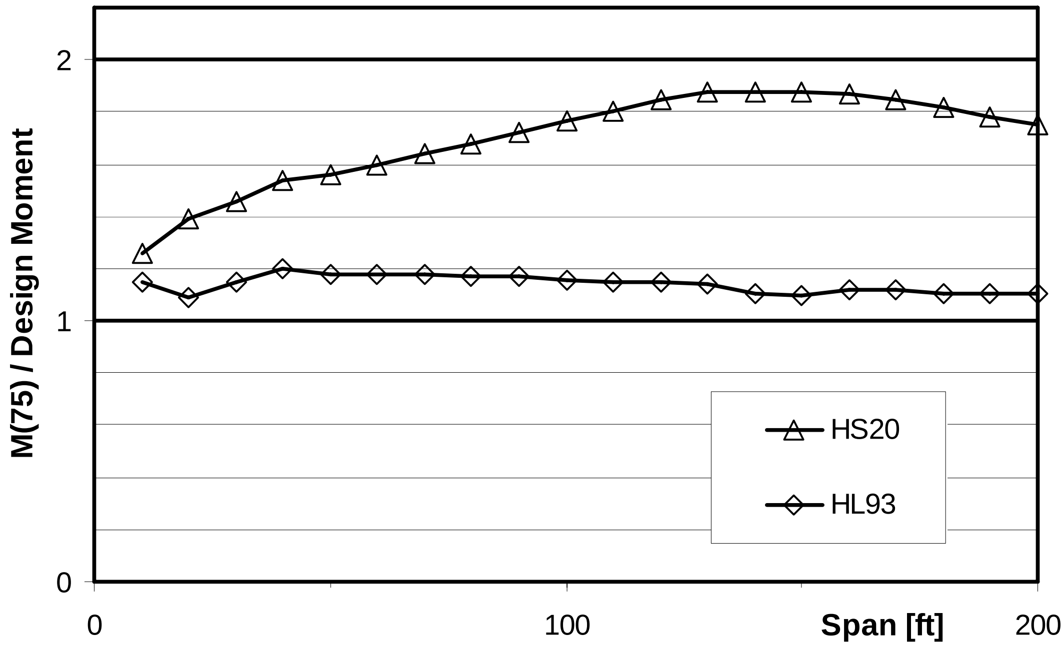

Figure 20 - Bias Factor for Two Lanes Loaded, Simple Span Moment; Ratio M(75)/M(HL93) and M(75)/M(HS20), ADTT = 5000

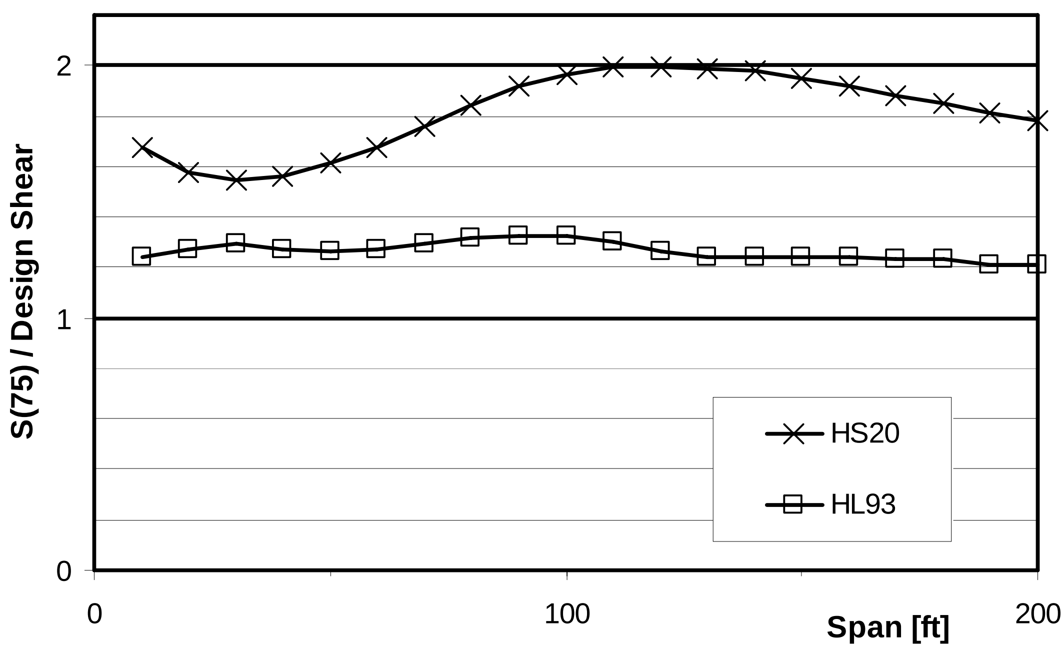

Figure 21 - Bias Factors for One Lane Loaded, Shear; Ratio S(75)/S(HL93) and S(75)/S(HS20), ADTT = 5000

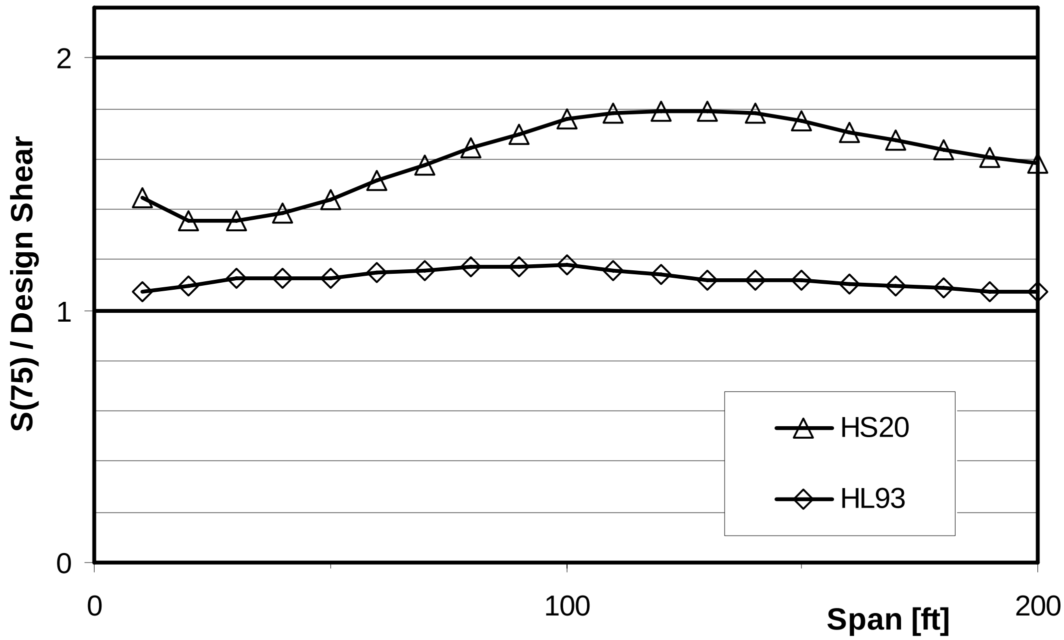

Figure 22 - Bias Factors for Two Lanes Loaded, Shear; Ratio S(75)/S(HL93) and S(75)/S(HS20), ADTT = 5000

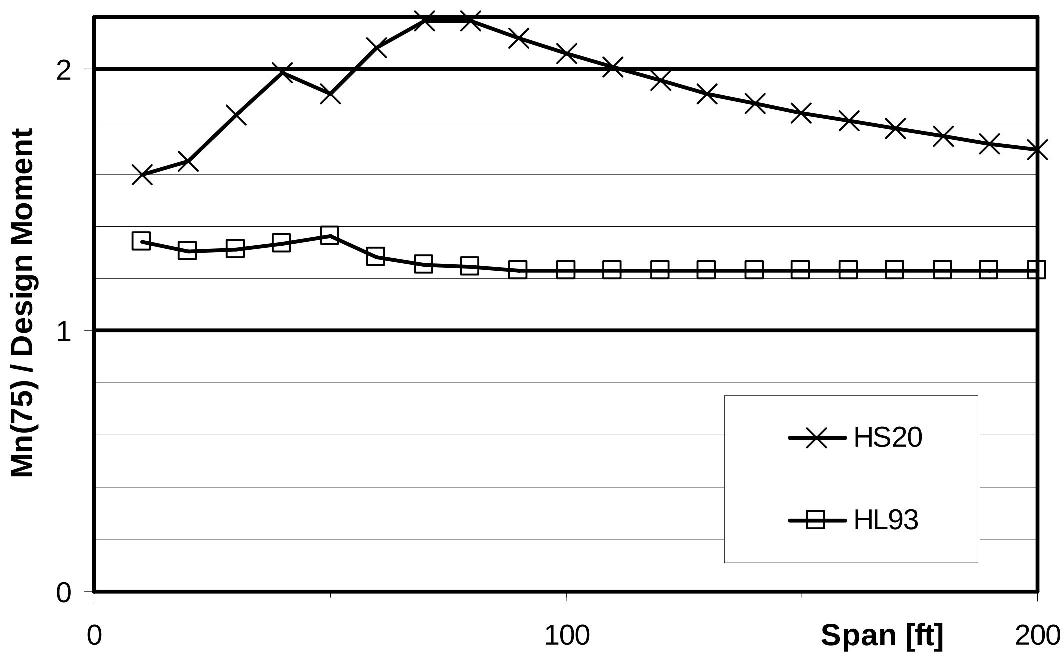

Figure 23 - Bias Factors for One Lane Loaded, Negative Moment; Ratio Mn(75)/Mn(HL93) and Mn(75) / Mn(HS20), ADTT = 5000

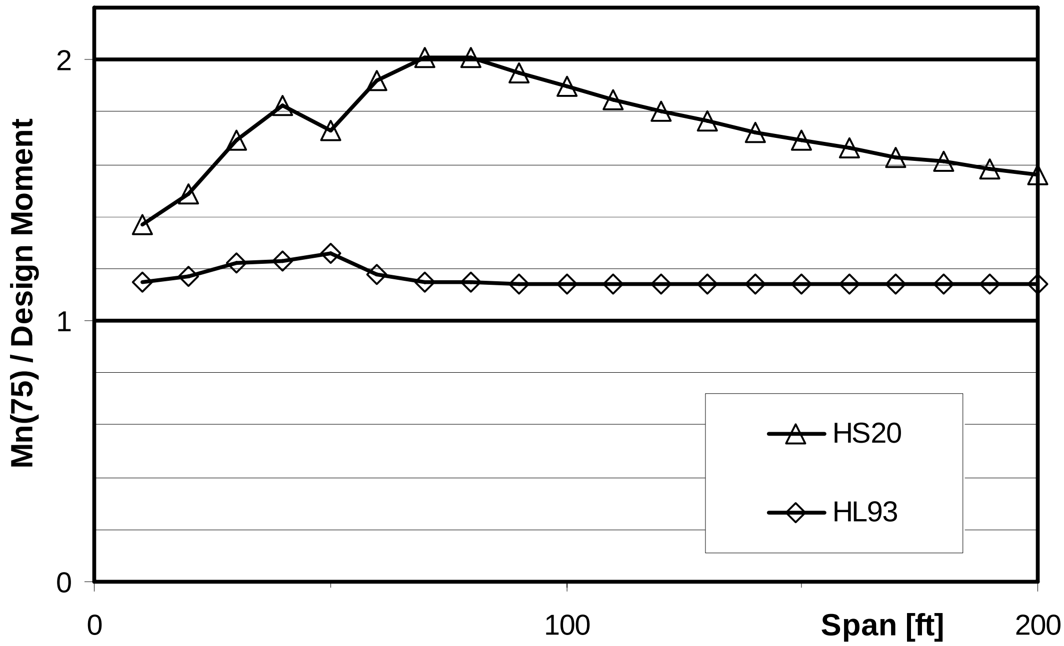

Figure 24 - Bias Factors for Two Lanes Loaded, Negative Moment; Ratio Mn(75)/Mn(HL93) and Mn(75) / Mn(HS20), ADTT = 5000

4.3. Dynamic load

Dynamic load is considered as an additional static load added to live load. Therefore, it is specified as a dynamic load amplification (DLA). The DLA specified in AASHTO LRFD is 0.33 of the design truck effect, and 0 for the uniformly distributed load. The mean DLA for very heavy trucks was first based on analytical modeling (Hwang and Nowak 1991),

DLA < 0.17 for a single heavy truck
DLA < 0.10 for two heavy trucks side-by-side (10)

These analytical values were confirmed by extensive field measurements (Kim and Nowak 1997; Eom and Nowak 2001). The corresponding coefficient of variation is 0.80.

4.4. Combination of Live Load and Dynamic Load

The mean value of the combination of live load, LL, and dynamic load, IM, per girder is calculated using the following formula,

$$\mu_{LL+IM} = (1.1)\,(\lambda_{LL})\,(1.0)(LL) \tag{11}$$

where LL = HL-93 live load, 1.1 represents dynamic load (10%) and the bias factor for GDF = 1.0. But the coefficient of variation of GDF is 0.12, therefore, the coefficient of variation of live load (only the static portion), V_{LL_P}, and standard deviation, σ_{LL_P}, were given as:

$$V_{LL\_P} = \sqrt{V_{LL}^2 + V_P^2} \tag{12}$$

$$\sigma_{LL\_P} = V_{LL\_P} \cdot \mu_{LL\_P} \tag{13}$$

where $V_P = 0.12$.

Consequently, the standard deviation, σ_{LL_P+IL}, and coefficient of variation, V_{LL_P+IL}, of the mean maximum combination of live load, *LL*, and dynamic load, *IL*, were given as:

$$\sigma_{LL\_P+IL} = \sqrt{\sigma_{LL\_P}^2 + \sigma_{IL}^2} \tag{14}$$

$$V_{LL\_P+IL} = \frac{\sigma_{LL\_P+IL}}{\mu_{LL\_P+IL}} \tag{15}$$

For a single lane, $V_{LL\_P+IL} = 0.19$ for most spans, and 0.205 for short spans. For two lane bridges, $V_{LL\_P+IL} = 0.18$ and 0.19 for very short spans.

5. Resistance Models

The resistance was considered as a product of a nominal resistance, R_n, and three factors: *M* = material factor (strength of material, modulus of elasticity), *F* = fabrication factor (geometry, dimensions), and *P* = professional factor (use of approximate resistance models, e.g. the Whitney stress block, idealized stress and strain distribution model).

$$R = R_n \cdot M \cdot F \cdot P \tag{16}$$

The mean value, μ_R, and the coefficient of variation, V_R, of resistance, R, may be approximated by the following accepted equations for the range of values that were considered:

$$\mu_R = R_n \cdot \mu_M \cdot \mu_F \cdot \mu_P \tag{17}$$

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{18}$$

The statistical parameters of resistance were determined using the test results available prior to 1990, special simulations, and engineering judgment. They were developed for non-composite and composite steel girders, reinforced concrete T-beams, and prestressed concrete AASHTO-type girders. Bias factors and coefficients of variation were determined for material factor, *M*, fabrication factor, *F*, and analysis factor, *P*. Factors *M* and *F* were combined.

Material parameters for steel girders were taken from Ellingwood et al. (1980). Only statistical parameters such as the mean and standard deviation or coefficient of variation were available, but no actual test data (no raw data). For structural steel, the statistical parameters were based on the papers by Ravindra and Galambos 1978; Yura, Galambos and Ravindra 1978; Cooper, Galambos and Ravindra 1978; Hansell, Galambos, Ravindra and Viest 1978; Galambos and Ravindra 1978. The information included the mean values and coefficient of variation for the yield strength of steel, tensile strength of steel and modulus of elasticity, for hot-rolled beams and plate girders. In addition, they provided the parameters (mean value and coefficient of variation) for fabrication factor and professional factor. In the very last phase of calibration, the steel industry (American Iron and Steel Institute) provided the upgraded bias factors and coefficients of variation for yield strength of structural steel. These values were then used in Monte Carlo simulations to determine the parameters of resistance for non-composite and composite girders, for the moment carrying capacity and shear.

For concrete components, the material parameters were taken from Ellingwood et al. (1980). As in the case of structural steel, only the statistical parameters were obtained but no raw test data. The basis for these parameters was research by Mirza and MacGregor (1979). The data included mean value and coefficient of variation for the compressive strength of concrete, yield strength of reinforcing bars, and prestressing strands. In addition, the data included the statistical parameters of fabrication factor and professional factor.

The material data, combined with statistical parameters of the fabrication factor and professional factor, were used in Monte Carlo simulations that resulted in the statistical parameters of resistance for steel girders (non-composite and composite), reinforced concrete T-beams and prestressed girders, for moment and shear, as shown in Table 21.

It was assumed that resistance is a lognormal random variable.

Table 21. Statistical Parameters of Component Resistance

Type of Structure	Material and Fabrication factors, F M		Professional factor, P		Resistance, R	
	λ	V	λ	V	λ	V
Non- Composite steel girders						
Moment (compact)	1.095	0.075	1.02	0.06	1.12	0.10
Moment (non-com.)	1.085	0.075	1.03	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Composite steel girders						
Moment	1.07	0.08	1.05	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Reinforced concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.13
Shear w/steel	1.13	0.12	1.075	0.10	1.20	0.155
Shear no steel	1.165	0.135	1.20	0.10	1.40	0.17
Prestressed concrete						
Moment	1.04	0.045	1.01	0.06	1.05	0.075
Shear w/steel	1.07	0.10	1.075	0.10	1.15	0.14

6. Reliability Analysis

The reliability is calculated in terms of the reliability index, β. The loads are represented by a normal random variable, Q, and resistance is represented by a lognormal random variable, R. The limit state function is given by Eq. 5. The reliability analysis is performed using the Rackwitz ad Fiessler procedure (Nowak and Collins 2000), based on approximation of non-normal distributions with normal distributions at the so called design point with coordinates (R^*, Q^*). The design point satisfies the following limit state equation.

$$R^* - Q^* = 0 \tag{19}$$

so $R^* = Q^*$, and it is located between the mean values of R and Q. The design point is found by iterations.

The procedure is shown graphically in Figure 1.5. The CDF's of resistance, R, and load, Q, are plotted on the normal probability paper. Because Q is a normal random variable, its CDF is represented by a straight line. The horizontal line corresponding to 0 on the vertical scale intersects with CDF at the mean value of Q. R is a lognormal random variable, therefore, its CDF is not a straight line. However, because of relatively low value of the coefficient of variation ($V << 0.2$), the curvature of CDF is also low.

The analytical procedure involves iterations, starting with guessing the initial value of the design point, shown in Figure 25. Then, a lognormal CDF of R is approximated by a normal CDF at the design point. The approximating normal CDF is a straight line that is tangent to lognormal CDF, shown in Figure 25. The mean and standard deviation of the approximating normal CDF can be read directly from the graph. The intermediate value of the reliability index can be calculated using the following formula,

$$\beta = \frac{m'_R - m_Q}{\sqrt{\sigma'^{\,2}_R + \sigma_Q^{\,2}}} \qquad (20)$$

where m'_R = the mean of the approximating normal CDF of R, and σ'_R = the standard deviation of the approximating normal CDF of R, The new design point is found,

$$R^* = Q^* = m'_R - \frac{\sigma'^{\,2}_R \beta}{\sqrt{\sigma'^{\,2}_R + \sigma_Q^{\,2}}} \qquad (21)$$

and iterations are continued until β does not change any more. The design point can be defined as

$$R^* = m_R - k\, \sigma_R \qquad (22)$$

where k = a parameter (to be determined by iterations). The mean and standard deviation can be replaced using the nominal value, R_n, bias factor, λ_R, and coefficient of variation, V_R,

$$R^* = R_n\, \lambda_R\, (1 - k\, V_R) \qquad (23)$$

and the intermediate value of the reliability index is

$$\beta = \frac{R_n \cdot \lambda_R \cdot (1 - k \cdot V_R) \cdot [1 - \ln(1 - k \cdot V_R)] - m_Q}{\sqrt{[R_n \cdot V_R \cdot \lambda_R \cdot (1 - k \cdot V_R)]^2 + \sigma_Q^{\,2}}} \qquad (24)$$

For the design cases considered in calibration of AASHTO LRFD, parameter k is about 1.8-2.1. Furthermore, the reliability index is not sensitive with regard to the exact position of design point, represented by parameter k. Therefore, in this calibration, the reliability indices are calculated for k = 2.

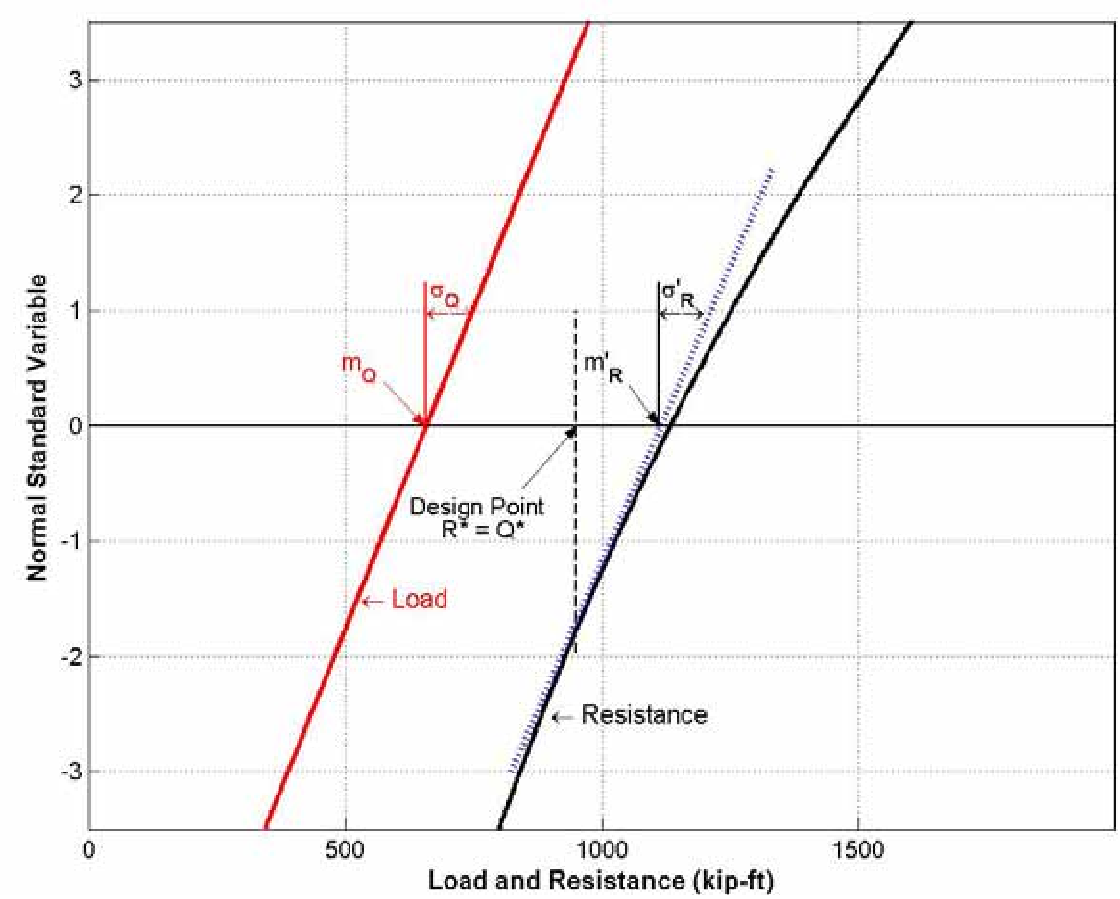

Figure 25 - Graphical Procedure for Calculation of the Reliability Index

7. Reliability Indices for Bridge Components Designed According to AASHTO LRFD Code

The value of β calculated for bridge components designed according to the AASHTO LRFD Code (2004) are summarized in Table 22 for moment and Table 23 for shear.

Table 22. Final Values of β Corresponding to AASHTO LRFD Code (2004), Moment

Type of structure	Span range [ft]	Spacing range [ft]	Resistance factor	Reliability index range		
Prestressed concrete	30-200	4-12	1.00	3.55	-	3.84
Reinforced concrete	30-120	4-12	0.90	3.54	-	3.97
Steel non-composite	30-200	4-12	1.00	3.46	-	3.71
Steel composite	30-200	4-12	1.00	3.58	-	3.80

Table 23. Final Values of β Corresponding to AASHTO LRFD Code (2004), Shear

Type of structure	Span range [ft]	Spacing range [ft]	Resistance factor	Reliability index range		
Prestressed concrete	30-200	4-12	0.90	3.62	-	4.02
Reinforced concrete	30-120	4-12	0.90	3.53	-	3.95
Steel	30-200	4-12	1.00	3.70	-	4.03

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