Revised load and resistance factors for the *AASHTO LRFD Bridge Design Specifications*

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- The load and resistance factors in the 2014 edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* were determined using statistical parameters from the 1970s and early 1980s.
- This paper revisits the original calibration and recalculates the load and resistance factors as coordinates of the design point.
- The recommended new load and resistance factors provide consistent reliability and a rational safety margin.

The basis for the current edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] was developed in the 1980s. The major conceptual change with respect to the AASHTO *Standard Specifications for Highway Bridges*[2] was the introduction of four types of limit states and corresponding load and resistance factors.

Equation (1) is the basic design formula for structural components given in the 2002 AASHTO standard specifications.[2]

$$1.3D + 2.17(L + I) < \phi R \qquad (1)$$

where

D = dead load

LL = live load (HS-20)

IM = dynamic load

R = resistance (load-carrying capacity)

ϕ = resistance factor = 1 (by default)

Equation (2) is the equivalent design formula in the current AASHTO LRFD specifications.[1]

$$1.25D + 1.50DW + 1.75(LL + IM) < \phi R \qquad (2)$$

where

DW = dead load due to wearing surface

LL = live load (HL-93)

ϕ = 1 for steel girders and pretensioned concrete girders and 0.9 for reinforced concrete T beams

The differences between Eq. (1) and (2) are on the load side only. The role of the load and resistance factors is to provide safety margins; that is, the load factors increase the design loads so that there is an acceptably low probability of their being exceeded. The role of the resistance factor is to decrease the design-load-carrying capacity, resulting in an acceptably low probability of exceeding the critical level. However, if ϕ equals 1, then resistance is not reduced and most of the safety reserve is on the load side of Eq. (1) and (2).

Therefore, there is a need to determine values of the load and resistance factors that represent rational and optimum safety margins. The derivation procedure involves the reliability analysis procedure and calculation of the design point.[3] The product of the load and load factor is the factored load, and the product of the resistance and resistance factor is the factored resistance. The coordinates of the design point are values of the factored load and factored resistance corresponding to the minimum reliability index. The objective of this paper is to calculate the optimum load and resistance factors for selected representative bridge components and to propose a modified design formula to replace Eq. (2).

Limit state function and reliability index

For each limit state, a structural component can be in two states: safe when the resistance R exceeds the load Q and unsafe (failure) when the load exceeds the resistance. The boundary between the safe and unsafe states g can be represented in a simple form by the limit state function (Eq. [3]).

$$g = R - Q = 0 \tag{3}$$

Because R and Q can be considered to be random variables, the probability of failure P_F is equal to the probability P of g being negative (Eq. [4]).

$$P_F = P(g < 0) \tag{4}$$

In general, R and Q can be functions of several variables, such as dead load, live load, dynamic load, strength of material, dimensions, girder distribution factors, and so on. Therefore, the limit state function can be a complex function (Eq. [5]).

$$g(X_1, \dots, X_n) = 0 \tag{5}$$

where

$X_1, \dots, X_n$ = input random variables (load and resistance components)

A direct calculation of the probability of failure can be difficult, in particular when g is nonlinear. Instead, the reliability index β can be calculated. Equations (6) and (7) show the relationship between β and the probability of failure P_F.

$$P_F = \Phi(-\beta) \tag{6}$$

$$\beta = -\Phi^{-1}(P_F) \tag{7}$$

where

Φ = cumulative distribution function of the standardized normal random variable

Φ^{-1} = the inverse of Φ (Nowak and Collins[3])

There are several formulas and analytical procedures available to calculate β. If the limit state function is linear and all the variables are normal (Gaussian), they are Eq. (8) to (11).

$$g\left(X_1, \dots, X_n\right) = a_0 + \sum_{i=1}^{n} a_i X_i \tag{8}$$

where

$a_0 \dots a_i$ = constants of the limit state function

then

$$\beta = \frac{\mu_g}{\sigma_g} \tag{9}$$

where

$$\mu_g = g(\mu_1, \dots, \mu_n) \tag{10}$$

μ_g = mean value of g

$(\mu_1, \dots, \mu_n)$ = mean values of $X_1, \dots, X_n$

σ_g = standard deviation of g

$$\sigma_g = \sqrt{\sum \left(a_i \sigma_i\right)^2} \tag{11}$$

where

σ_i = standard deviation of X_i

If the variables are nonnormal, then Eq. (9) can be used as an approximation. Otherwise, a more accurate value of β

can be calculated using an iterative procedure developed by Rackwitz and Fiessler.[4] However, in practical cases the results obtained using Eq. (9) can be considered to be accurate.

If the limit state function is nonlinear, then accurate results can be obtained using Monte Carlo simulations.[3]

Design point

The result of the reliability analysis is the reliability index β. In addition, the reliability analysis can be used to determine the coordinates of the design point—that is, the corresponding value of the factored load for each load component and the value of the factored resistance. For the limit state function in Eq. (5), the design point is a point in n-dimensional space (denoted by $X_1^*, \ldots, X_n^*$, where $X_1^* \ldots X_n^*$ are coordinates of the design point) that satisfies Eq. (5), and if failure is to occur, it is the most likely combination of $X_1^*, \ldots, X_n^*$.[3]

For example, if the limit state function is given by Eq. (3) and R and Q are normal random variables, then the coordinates of the design point are determined by Eq. (12) and (13).[3]

$$R^* = \mu_R - \frac{\beta\sigma_R^2}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (12)$$

where

R^* = coordinate of the design point for R

μ_R = mean value of R

σ_Q = standard deviation of Q

σ_R = standard deviation of R

$$Q^* = \mu_Q + \frac{\beta\sigma_Q^2}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (13)$$

where

Q^* = coordinate for the design point for Q

μ_Q = mean value of Q

If R and Q are not both normally distributed, then R^* and Q^* can be calculated by iterations using the Rackwitz and Fiessler procedure.[4] However, a wider range of design point coordinates corresponds to the same value of the reliability index, so in practice, Eq. (12) and (13) can be used even for nonnormal distributions.

Statistical parameters of load components

The basic load combination for bridge components includes the dead load D, dead load due to the wearing surface DW, live load LL, and dynamic load IM. Each random variable is described by its cumulative distribution function, including the mean and standard deviation. It is also convenient to use the bias factor λ, which is the ratio of mean value divided by nominal (design) value, and the coefficient of variation V, which is equal to the ratio of the standard deviation and the mean. Both λ and V are nondimensional.

The total load is the sum $D + DW + LL + IM$. Dead load is time invariant, so the only time-varying load components are LL and IM. In the original calibration,[5] the maximum expected 75-year live load was considered for Strength I limit state for the economic lifetime of the bridge; therefore, the same time period is considered in this paper.

The statistical parameters of the dead load that were used in the original calibration[5] have not been challenged so far. Therefore, for factory-made components (structural steel and precast, prestressed concrete), λ equals 1.03 and V equals 0.08. For the cast-in-place concrete, λ equals 1.05 and V equals 0.10. For the wearing surface, it is assumed that the mean thickness is 3.5 in. (90 mm) with λ equal to 1.00 and V equal to 0.25.

The live load parameters used in the original calibration were based on the Canadian Ministry of Transportation truck survey data from Agarwal and Wolkowicz[6] with fewer than 10,000 vehicles because no other reliable data were available at that time. In the meantime, a considerable weigh-in-motion database was collected by the Federal Highway Administration. Therefore, the statistical parameters for live load are taken from the recent Strategic Highway Research Program 2 (SHRP2) R19B report.[7] The processed data included 34 million vehicles from 37 locations in 18 states. For each location, the annual number of vehicles was one to two million.

The live load is the effect of trucks; therefore, the vehicles in the weigh-in-motion database were run over influence lines to determine the moments and shears. Cumulative distribution functions of the maximum simple-span moments were calculated for 30, 60, 90, 120, and 200 ft (9, 18, 27, 37, and 61 m). To facilitate the interpretation of the results, the moments were divided by the corresponding HL-93 moments.[1] For the locations considered, the maximum ratios were about 1.35 to 1.40 of HL-93.

The cumulative distribution functions were extrapolated to predict the mean maximum 75-year moment. **Figure 1** plots the span length versus the ratio of mean to nominal value, or bias factor for the live load moment, for average daily truck traffic (ADTT) values from 250 to 10,000. The average coefficient of variation of the static live load effect is 0.12 for the span length from 30 to 200 ft (9 to 61 m).

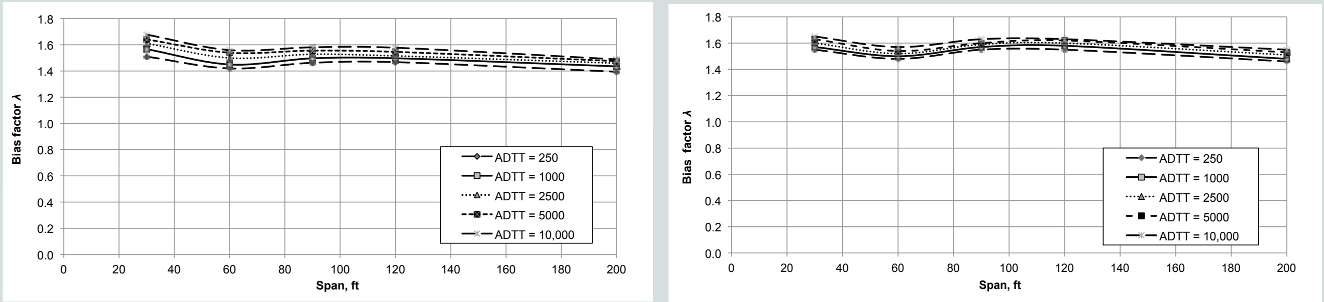

Figure 1. Span length versus bias factor for the maximum 75-year moment and shear. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

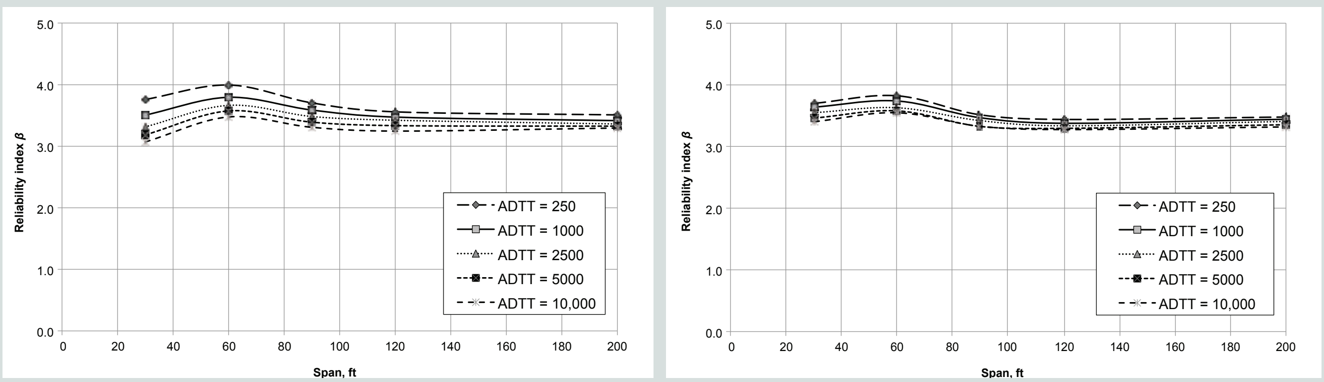

Figure 2. Span length versus reliability index for moment and shear for prestressed concrete girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

Field tests showed that the dynamic load does not depend on the truck weight.[8] Therefore, the dynamic load factor decreases for heavier trucks. It is further reduced when multiple trucks are present, in particular when they are side by side. In the reliability analysis, the mean value of the dynamic load factor is taken as 0.10 and the coefficient of variation is 0.8.[5] Therefore, the resultant coefficient of variation for combined static and dynamic live load is as 0.14.

The total load as a sum of several components can be considered to be a normal random variable.

Statistical parameters of resistance

The load carrying capacity is the product of three factors representing the uncertainties related to material properties, dimensions/geometry, and the analytical model. **Table 1** lists the statistical parameters, bias factor λ, and coefficient of variation V that were used in the original calibration.

Since the original calibration, a considerable amount of research has been conducted in conjunction with the revision of the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*.[9–12] The database included the compressive strength of concrete, yield strength of reinforcing bars, and tensile strength of prestressing strands. The results showed that the material properties are more predictable than they were 30 years ago. There has been a reduction in the coefficient of variation because of more efficient quality control procedures. The compressive strength of concrete has a bias factor of 1.3 for a concrete compressive strength f_c' of 3000 psi (21 MPa) and 1.1 for f_c' of 12,000 psi (83 MPa), and the corresponding coefficient of variation varies from 0.17 for f_c' of 3000 psi to 0.10 for f_c' of 12,000 psi. For reinforcing steel, λ equals 1.13 and V equals 0.03. For prestressing strands, λ equals 1.04 and V equals 0.015. These material parameters can serve as a basis for revising the resistance models for bridge components. It is estimated that the mean load-carrying capacity of bridge girders is 5% to 10% higher than the original calibration. However, because additional analysis is required to develop updated statistical parameters for the resistance of bridge components, the reliability analysis in this paper was conducted using the parameters from Table 1.

Representative design cases

The reliability indices were calculated for the design cases considered in the original calibration using Eq. (9).[5] **Figure 2** shows the results for prestressed concrete girders, **Fig. 3** shows the results for reinforced concrete T beams,

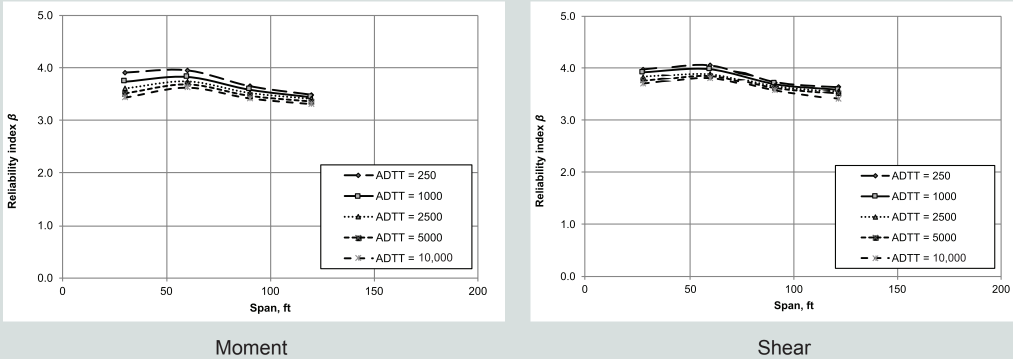

Figure 3. Span length versus reliability index for moment and shear for reinforced concrete T beams. Note: ADDT = average daily truck traffic. 1 ft = 0.305 m.

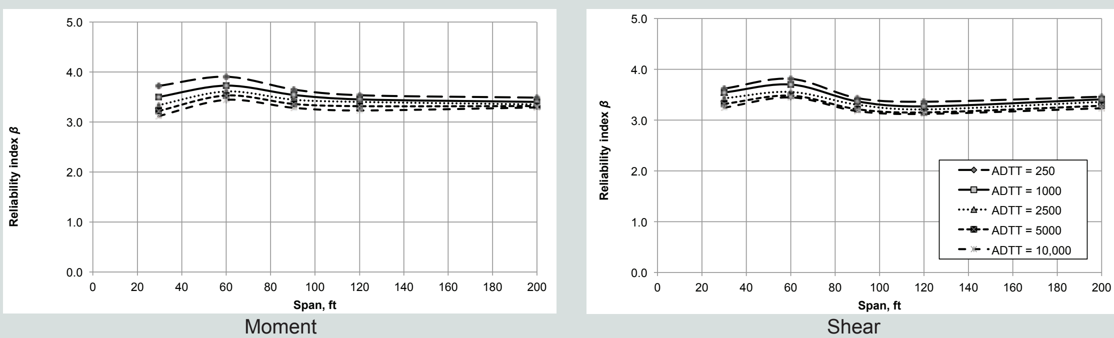

Figure 4. Span length versus reliability index for moment and shear for steel girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

and **Fig. 4** shows the results for steel girders. For each material, the analysis was performed for spans of 30, 60, 90, 120, and 200 ft (9, 18, 27, 37, and 61 m), and girder spacings of 4, 6, 8, 10, and 12 ft (1.2, 1.8, 2.4, 3.0, and 3.6 m). For reinforced concrete T beams, the span length was limited to 120 ft (37 m). The analysis was performed for ADTT values from 250 to 10,000.

The resulting reliability indices are about 3.5, with a small degree of variation. This is an indication that the specifications are consistent.

Table 1. Statistical parameters of resistance from 1999 NCHRP report 368 *Calibration of LRFD Bridge Design Code*

Material	Moment		Shear	
	λ	V	λ	V
Noncomposite steel	1.12	0.1	1.14	0.105
Composite steel	1.12	0.1	1.14	0.105
Reinforced concrete	1.14	0.13	1.2	0.155
Prestressed concrete	1.05	0.075	1.15	0.14

Note: V = coefficient of variation; λ = bias factor, the ratio of mean to nominal value.

Optimum load and resistance factors

Reliability indices were calculated for the design cases considered in the original calibration. For these design cases, the parameters of the design point were also calculated using Eq. (12) and (13).

For each load component X, the optimum load factor γ_X is determined by Eq. (14).

$$\gamma_X = \frac{\lambda_X X^*}{\mu_X} \tag{14}$$

where

λ_X = bias factor of X

X^* = coordinate of the design point

μ_X = mean value of X

Equation (15) calculates resistance.

$$\phi = \frac{\lambda_R R^*}{\mu_R} \tag{15}$$

where

λ_R = bias factor of R

Therefore, for the dead load of factory-made elements DC_1, the load factor λ_{DC_1} is calculated in Eq. (16).

$$\gamma_{DC_1} = \frac{\lambda_{DC_1} DC_1^*}{\mu_{DC_1}} \tag{16}$$

where

λ_{DC_1} = bias factor of DC_1

DC_1^* = coordinate of the design point for DC_1

μ_{DC_1} = mean value of DC_1

For the dead load of cast-in-place concrete DC_2, the load factor λ_{DC_2} is calculated in Eq. (17).

$$\gamma_{DC_2} = \frac{\lambda_{DC_2} DC_2^*}{\mu_{DC_2}} \tag{17}$$

where

λ_{DC_2} = bias factor of DC_2

DC_2^* = coordinate of the design point for DC_2

μ_{DC_2} = mean value of DC_2

For DW (weight of the wearing surface), the load factor λ_{DW} is calculated in Eq. (18).

$$\gamma_{DW} = \frac{\lambda_{DW} DW^*}{\mu_{DW}} \tag{18}$$

where

λ_{DW} = bias factor of DW

DW^* = coordinate of the design point for DW

μ_{DW} = mean value of DW

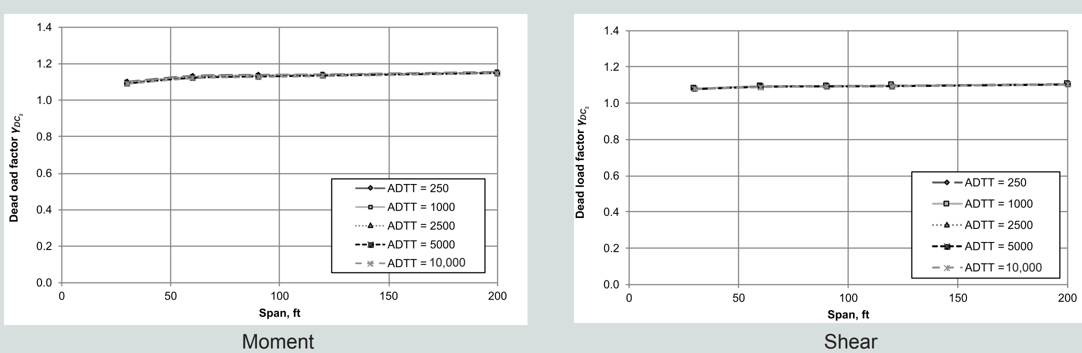

Figure 5. Span length versus dead load factors for moment and shear for prestressed concrete girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

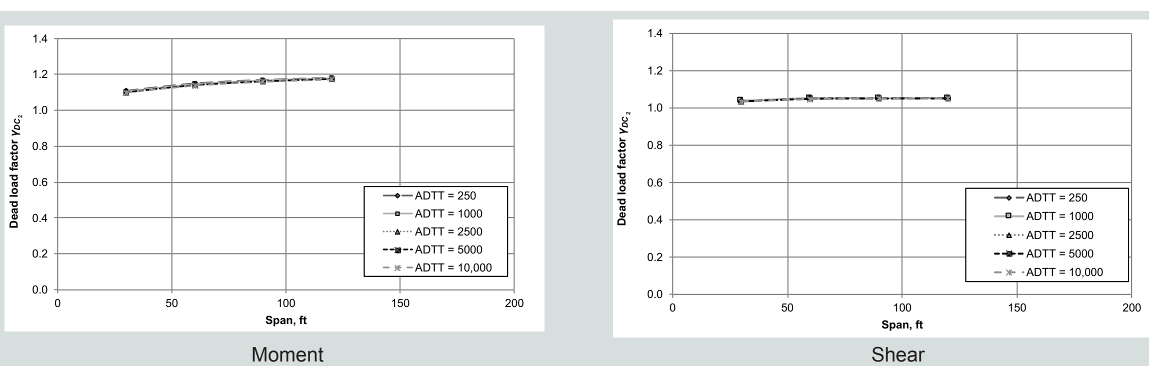

Figure 6. Span length versus dead load factors for moment and shear for reinforced concrete T-beam girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

For the live load LL, the load factor γ_{LL} is calculated in Eq. (19).

$$\gamma_{LL} = \frac{\lambda_{LL} LL^*}{\mu_{LL}} \tag{19}$$

where

λ_{LL} = bias factor of LL

LL^* = coordinate of the design point for LL

μ_{LL} = mean value of LL

The dead load factors calculated using Eq. (16) to (18) are as follows:

for DC_1, γ_{DC_1} = 1.05 – 1.1

for DC_2, γ_{DC_2} = 1.10 – 1.17

for DW, γ_{DW} = 1.03 – 1.1

As an example, **Fig. 5** shows the values of thedead load factor γ_{DC_2} for prestressed concrete girders, **Fig. 6** shows the values for reinforced concrete T beams, and **Fig. 7** shows the values for steel girders.

Figure 8 shows the calculated values for the live load factor for prestressed concrete girders, **Fig. 9** shows the values for reinforced concrete T beams, and **Fig. 10** shows the values for steel girders. In most cases, the optimum live load factor γ_{LL} is between 1.4 and 1.55 for ADTT equal to 10,000 and the range is 1.3 to 1.5 for ADTT equal to 250. Therefore, 1.55 can be considered to be a conservative live load value, even for ADTT equal to 10,000.

The resistance factors were calculated using Eq. (15). **Figure 11** shows the results for prestressed concrete girders, **Fig. 12** shows the results for reinforced concrete T beams, and **Fig. 13** for steel girders. **Table 2** provides a summary of the results.

Recommended load and resistance factors

The load and resistance factors corresponding to the coordinates of the design point are about 10% to 15% lower than those given in the current AASHTO LRFD specifications.[1] The reliability indices calculated for design according to the AASHTO LRFD specifications are consistent at about the 3.5 level (Fig. 2 to 4). However, the bias factor for the live load (Fig. 1) is higher for short spans than it is for other span lengths, which is an indication that the design live load for short spans has to be increased.

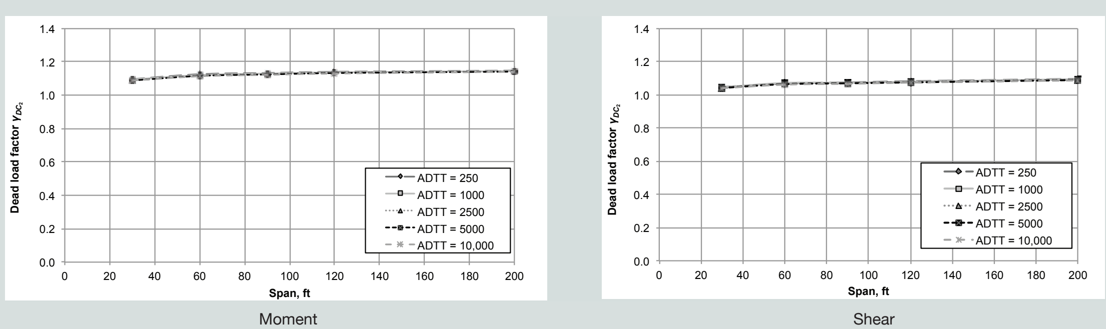

Figure 7. Span length versus dead load factors for moment and shear for steel girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

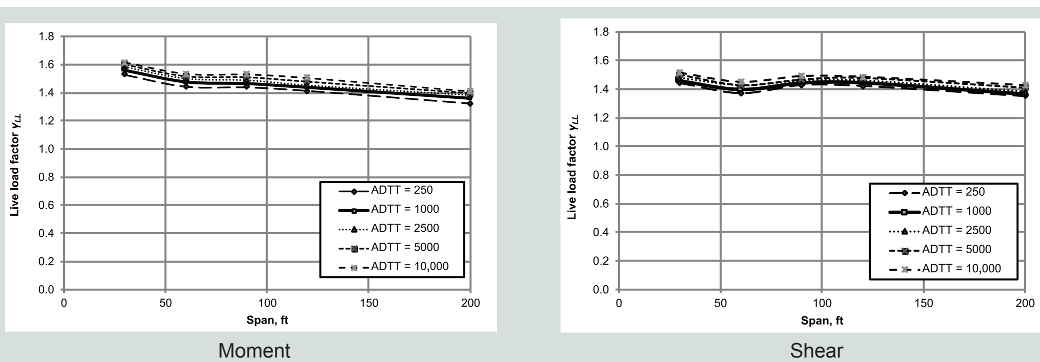

Figure 8. Span length versus live load factor for moment and shear for prestressed concrete girders. ADTT = average daily truck traffic. Note: 1 ft = 0.305 m.

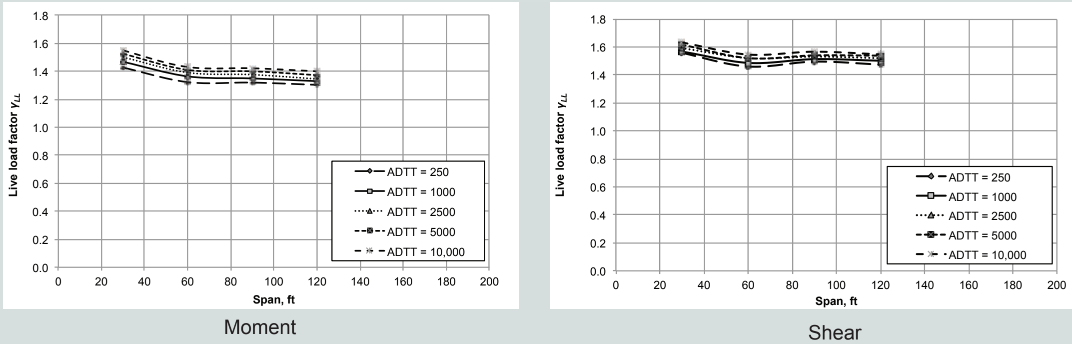

Figure 9. Span length versus live load factor for moment and shear for reinforced concrete T beams. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

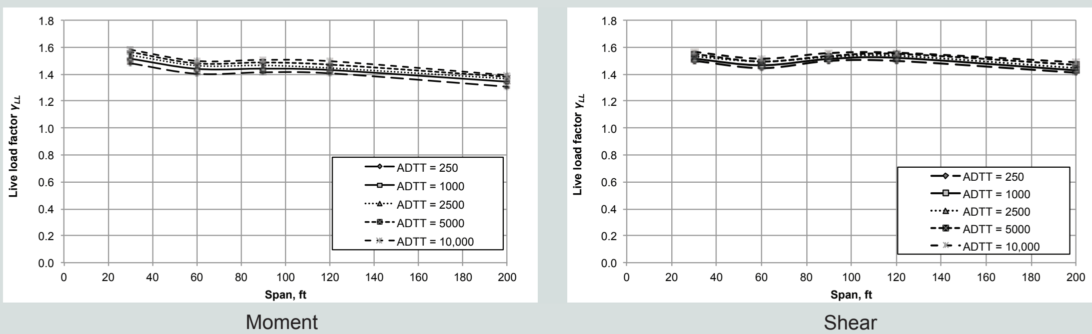

Figure 10. Span length versus live load factor for moment and shear for steel girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

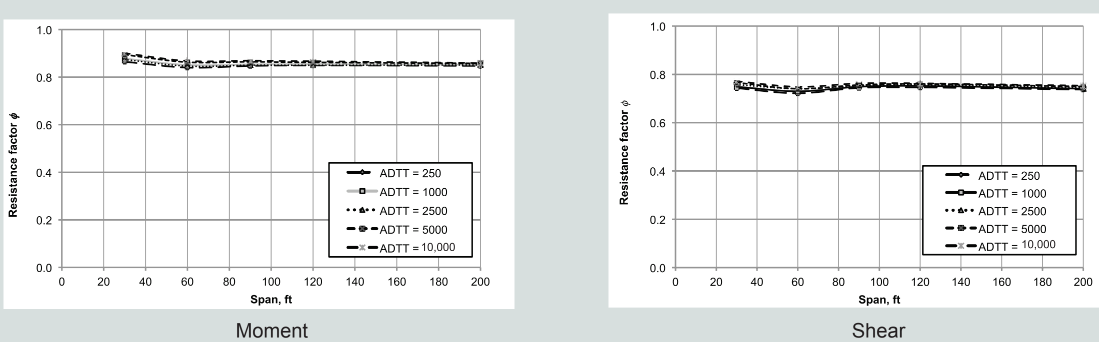

Figure 11. Span length versus resistance factor for moment and shear for prestressed concrete girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

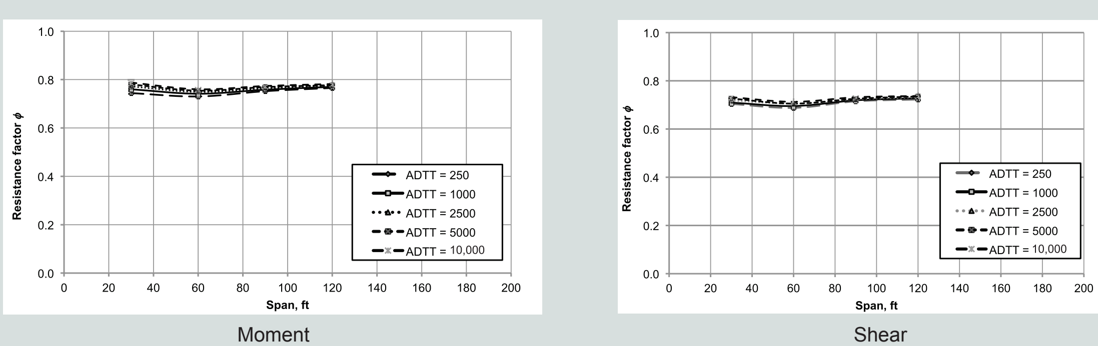

Figure 12. Span length versus resistance factor for moment and shear for reinforced concrete T beams. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

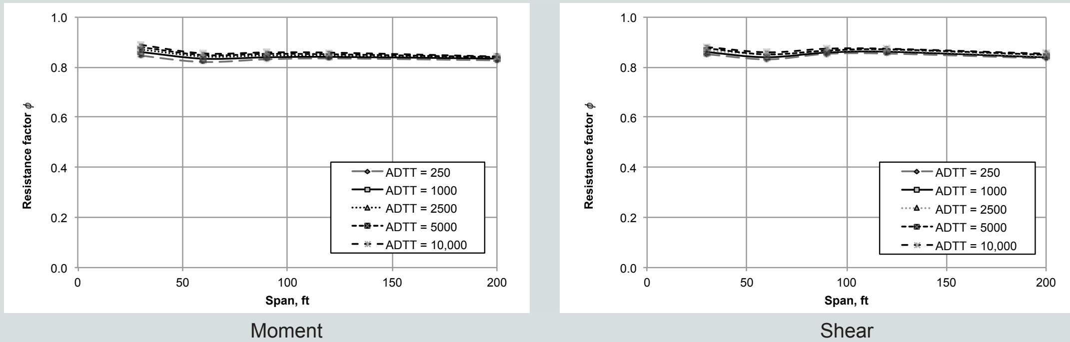

Figure 13. Span length versus resistance factor for moment and shear for steel girders. Note: ADTT = average daily truck traffic. 1 ft = 0.305 m.

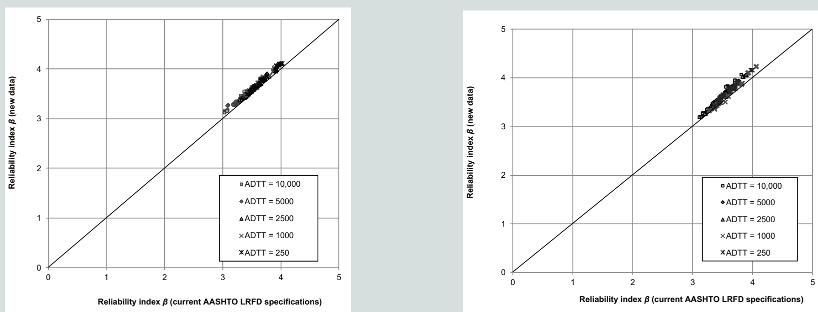

Figure 14. Reliability indices for the 2014 *AASHTO LRFD Bridge Design Specifications* versus new recommended reliability indices for moment and shear. Note: ADTT = average daily truck traffic.

The calculated values of the dead load factor for DC_1, DC_2, and DW are 1.05 to 1.17. For dead load due to wearing surface, the statistical parameters are based on an assumption about future overlays, and for simplicity of the code, one dead load factor of 1.20 is recommended for all dead load components.

The calculated values of the live load factor γ_{LL} are between 1.40 and 1.50. A higher value was found only for short spans due to the design load being too low. Therefore, the live load factor can be 1.50, but a conservative value of 1.60 is recommended.

Table 2 shows the calculated values of the resistance factor for flexure corresponding to the design point. However, it is recommended that the listed values be increased by 0.05 because of conservatism in the dead load factor and live load factor. **Table 3** shows the recommended ϕ factors for shear.

Therefore, Eq. (20) is the recommended new design formula.

$$1.20(D + DW) + 1.6(LL + IM) < \phi R \tag{20}$$

The reliability indices are calculated for the recommended load and resistance factors and compared with the

Table 2. Resistance factors according to 2014 *AASHTO LRFD Bridge Design Specifications*, calculated, and recommended for flexure

Material	Resistance factor in current AASHTO LRFD specifications ϕ	Calculated resistance factor ϕ	Recommended resistance factor ϕ
Steel (composite and noncomposite)	1.00	0.85	0.9
Prestressed concrete	1.00	0.85	0.9
Reinforced concrete	0.90	0.75	0.8

Table 3. Resistance factors according to 2014 *AASHTO LRFD Bridge Design Specifications*, calculated, and recommended for shear

Material	Resistance factor in current AASHTO LRFD specifications ϕ	Calculated resistance factor ϕ	Recommended resistance factor ϕ
Steel (composite and noncomposite)	1.00	0.85	0.9
Prestressed concrete	0.9	0.75	0.8
Reinforced concrete	0.85	0.70	0.75

reliability indices corresponding to the current AASHTO LRFD specifications and Eq. (2). **Figure 14** shows the results as scatter plots for moment and shear. The required moment carrying capacity corresponding to the recommended load and resistance factors is about 3% to 5% higher than that given in the current AASHTO LRFD specifications,[1] and for shear capacity it is about 5% higher.

The recommended loads are 1.20 for dead load and 1.60 for live load. The recommended resistance factors are 0.90 for steel and prestressed concrete girders. Incidentally, these load and resistance factors are the same as those given in ASCE/SEI (Structural Engineering Institute) 7-10,[13] ACI 318-14,[9] the American Institute of Steel Construction's *Steel Construction Manual*,[14] and *National Design Specification for Wood Construction*.[15]

Conclusion

Load factors in the AASHTO LRFD specifications[1] were selected so that the factored load corresponds to two standard deviations from the mean value. In this study, the optimum load factors were determined as corresponding to the design point and were about 10% lower than those specified in the code. The corresponding resistance factors were calculated as corresponding to the target reliability index. The resulting ϕ factors were also about 10% lower than those given in the AASHTO LRFD specifications. The acceptability criterion was, as in the original calibration, closeness to the target reliability index. The selection of load and resistance factors was checked on a set of representative bridges, the same as used in National Cooperative Highway Research Program (NCHRP) report 368.[5] In general, the recommended load and resistance factors were about 10% lower than those given in the current AASHTO LRFD specifications.[1] The reliability indices calculated for design cases using the current and recommended new load and resistance factors showed good agreement.

Acknowledgments

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Notation

$a_0 \dots a_i$ = constants of the limit state function

D = dead load

DC_1 = dead load of factory-made elements

DC_1^* = coordinate of the design point for DC_1

DC_2 = dead load of cast-in-place concrete

DC_2^* = coordinate of the design point for DC_2

DW = dead load for wearing surface

DW^* = coordinate of the design point for DW

f_c' = compressive strength of concrete

g = boundary between the safe and unsafe states

IM = dynamic load

LL = live load

LL^* = coordinate of the design point for LL

P = probability

P_F = probability of failure

Q = load (combination of loads)

Q^* = coordinate of the design point for Q

R = resistance (load-carrying capacity)

R^* = coordinate of the design point for R

V = coefficient of variation

X = load component

$X_1, \dots, X_n$ = input random variables (load and resistance components)

X^* = coordinate of the design point

$X_1^*, \dots, X_n^*$ = coordinates of the design point for $X_1, \dots, X_n$

β = reliability index

γ_{DC_1} = load factor for load DC_1

γ_{DC_2} = load factor for load DC_2

γ_{DW} = load factor for load DW

γ_{LL} = live load factor

γ_X = optimum load factor for load component X

λ = bias factor, the ratio of mean-to-nominal value

λ_{DC_1} = bias factor of DC_1

λ_{DC_2} = bias factor of DC_2

λ_{DW} = bias factor of DW

λ_{LL} = bias factor of LL

λ_R = bias factor of R

λ_X = bias factor of X

$\mu_1, \ldots, \mu_n$ = mean values of the input random variables

μ_{DC1} = mean value of DC_1

μ_{DC2} = mean value of DC_2

μ_{DW} = mean value of DW

μ_g = mean value of g

μ_{LL} = mean value of LL

μ_Q = mean value of Q

μ_R = mean value of R

μ_X = mean value of X

σ_g = standard deviation of g

σ_i = standard deviation of X_i

σ_Q = standard deviation of Q

σ_R = standard deviation of R

ϕ = resistance factor

Φ = cumulative distribution function of the standard normal random variable

Φ^{-1} = the inverse of Φ

About the authors

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Abstract

There has been considerable progress in reliability-based code development procedures. The load and resistance factors in the 2014 edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* were determined using statistical parameters from the 1970s and early 1980s. Load and resistance factors were determined by first fixing the load factors and then calculating resistance factors. Load factors were selected so that the factored load corresponded to two standard deviations from the mean value, and the resistance factors were calculated so that the reliability index was close to the target value. However, from a theoretical point of view, the load and resistance factors should be determined as coordinates of the design point that corresponds to less than two standard deviations from the mean. Therefore, the optimum load and resistance factors are about 10% lower than those specified in the AASHTO LRFD specifications. The objective of this paper is to revisit the original calibration and recalculate the load and resistance factors as coordinates of the design point for the Strength I limit state. The analysis was performed for the same types of girder bridges—reinforced concrete T beams, prestressed concrete girders, and steel girders—as in the original calibration presented in the 1999 National Cooperative Highway Research report 368, *Calibration of LRFD Bridge Design Code*. The recommended new load and resistance factors provide consistent reliability and a rational safety margin.

Keywords

Bridge, bridge live load, design formula, design point, reliability index, resistance factor, safety margin.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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