3-28-2019

Reliability-Based Shear Rating of Prestressed Concrete Bridge Girders Considering Capacity Adjustment Factor

Alaa I. Chehab
Nesai/Nasr Consulting Engineers, Inc., alaa.chehab@wayne.edu

Christopher D. Eamon
Wayne State University, eamon@eng.wayne.edu

Recommended Citation
Chehab, A. I. and Eamon, C. D. 2019. "Reliability-based shear rating of prestressed concrete bridge girders considering capacity adjustment factor." *J. Risk Uncertainty Eng. Syst., Part A: Civ. Eng.* 5 (2): 04019006. https://doi.org/10.1061/AJRUA6.0001009.

Reliability-Based Shear Rating of Prestressed Concrete Bridge Girders Considering Capacity Adjustment Factor

Alaa I. Chehab, M.ASCE[1] and Christopher D. Eamon, M.ASCE[2]

[1]Structural Engineer, Desai/Nasr Consulting Engineers, Inc., West Bloomfield Township, MI 48322 (corresponding author). Email: alaa.chehab@wayne.edu.

[2]Associate Professor of Civil and Environmental Engineering, Wayne State Univ., Detroit, MI 48202. Email: eamon@eng.wayne.edu.

Abstract

In this study, the reliability of prestressed concrete (PC) bridge girders in the shear rating process is quantified, where the use of a simple procedure to enhance the accuracy of shear capacity evaluation for rating is evaluated. It was found that the approximate methods used in code-based procedures to develop shear resistance used for reliability calibration of the AASHTO Bridge Design Specifications and the Manual for Bridge Evaluation result in larger discrepancies in reliability than previously assumed, as well as substantially conservative shear capacity evaluation in most cases. To examine the effects of using a more accurate shear rating process, a selection of typical PC bridge girders designed and rated for Michigan legal loads is considered as an example. For the example girders, use of existing methods resulted in some reliability indices below the required levels. However, use of the more accurate procedure resulted in shear reliability indices that met the target reliability index, thus preventing unnecessary traffic restrictions as well as avoiding potentially rating structures unconservatively.

Author keywords: prestressed concrete; shear; bridge structures.

Introduction

Bridge load rating is required by the US Department of Transportation (DOT) to assure that bridge structures within each state inventory are sufficiently safe for vehicular traffic. Specific bridge rating procedures are specified in the Manual for Bridge Evaluation (MBE) (AASHTO 2018), where rating for design, legal, and permit loads is discussed. Generally, it is desired by the DOTs to limit bridge posting as much as possible, as restrictions prevent the general public, as well as commercial vehicles, from fully utilizing the transportation network. Typically, the design load rating evaluates the ability of the bridge to carry the HL-93 design load specified in the American Association of State Highway and Transportation Officials Load and Resistance Factor Design Specifications (AASHTO LRFD 2017), and is used to complete the Federal inventory rating. The design load is also used to evaluate the bridge at the Federal operating level, where capacity associated with a lower level of reliability is assessed. Structures found to be inadequate for design-based load rating must be further evaluated considering legal load rating, to determine if traffic restriction is required. In the rating process, to ensure target levels of structural safety, it is necessary to use accurate procedures for evaluating bridge capacity. Although various failure modes may be of interest, flexural and shear strength-based limit states are often of greatest concern for rating typical highway bridge girders. The specific concern of this study is girder shear capacity. While the analysis procedures given in AASHTO LRFD may be expected to generally well-predict flexural capacity, it has been shown that significant inaccuracies exist using current methods to model shear behavior (Llanos et al. 2009; Ross et al. 2011; Wilder et al. 2015, Chehab and Eamon 2018, Chehab et al. 2018). Although generally conservative in most cases, inaccuracies in code-based analysis methods create the undesirable situation where shear capacity is significantly under-reported, potentially resulting in unnecessary traffic restrictions. A more troublesome result is that, in a smaller number of cases, bridges may be over-rated in capacity (Chehab and Eamon 2018). The current study addresses this concern and examines the effect of using a simple shear capacity adjustment factor on the structural reliability of prestressed concrete (PC) bridge girders. The purpose of the adjustment factor is to enhance the accuracy of shear assessment with minimal additional effort, potentially enabling bridge girders to meet target reliability levels in the rating process.

Since 2003, with the publication of the Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, rating bridge girders for vehicular traffic has been implicitly based on an assessment of structural reliability. The Manual for Bridge Evaluation (MBE) was later published by AASHTO in 2008 (AASHTO 2008), replacing the initial LRFR specifications as well as the alternative 1998 Manual for Condition Evaluation of Bridges (based on Load Factor Rating, which was not reliability-based). The live load factors specified in the MBE were later revised in 2011 (Sivakumar and Ghosn 2011) using weigh-in-motion (WIM) data from truck traffic collected from six states. Based on a 5-year return period for load rating, the recalibrated MBE rating process was formulated to result in an average target reliability index of 2.5, with a minimum level of 1.5 for any particular structure. For bridge girders rated according to the MBE, shear capacity is assumed to be calculated based on procedures specified in AASHTO LRFD, although more refined procedures are allowed (AASTHO 2017).

Prior to the release of the 1st Edition of AASHTO LRFD in 1994 (AASHTO 1994), bridge girders in the United States were primarily designed to at least meet the minimum standards given by the AASHTO Standard Specifications for Highway Bridges, which was last published in 2002 (AASHTO 2002). The shear design provisions in the AASHTO Standard Specifications were very similar to those currently presented by the American Concrete Institute Building Code

Requirements for Structural Concrete (ACI 318, 2014), where the concrete contribution to shear strength is taken as an empirical function of the square root of concrete compressive strength. However, a major change in shear design provisions was presented in AASHTO LRFD, where shear capacity is based on the Modified Compression Field Theory (MCFT) (Vecchio and Collins, 1986), resulting in a significantly more complex, and often more accurate method than that used in the Standard Specifications. The theoretically-developed MCFT provided significant changes in calculation of the concrete shear strength contribution, diagonal crack angle, and maximum allowable shear stress.

Since the 1994 AASHTO LRFD code was released, revisions to the shear design procedure were published in subsequent editions of the specifications. These revisions were primarily made to simplify the procedure, although the resulting nominal shear capacity was not substantially changed. During this time, additional research has been conducted on the shear behavior of PC beams. In some research, good agreement was found between the code model and experimental results. However, as noted above, in other cases, significant discrepancies have been demonstrated (Hawkins and Kuchma, 2007; Hawkins et al. 2005; Laskar et al. 2010; Pei et al. 2008; Chehab et al. 2018). These discrepancies motivated the development of the shear capacity adjustment factor suggested by Chehab and Eamon (2018) which is considered in this study.

AASHTO LRFD Shear Design Procedure

Two procedures are given in AASHTO LRFD (AASHTO 2017) to determine nominal shear resistance; the General Procedure and the Simplified Procedure. The MCFT-based General Procedure, generally thought to be most accurate, is the concern of this study. In this method, the nominal shear resistance, V_n, of PC girders is taken as the lesser of Equations (1) or (2):

$$V_n = V_c + V_s + V_p = 0.083\beta\sqrt{f_c^{'}}b_v d_v + \frac{A_v f_y d_v(\cot\theta)}{s} + V_p \qquad (1)$$

$$V_n = 0.25 f_c^{'} b_v d_v + V_p \qquad (2)$$

where V_c is the shear capacity attributed to the concrete; V_s is the shear capacity attributed to the web reinforcement; V_p is the vertical component of the prestressing force; β is a factor indicating the ability of diagonally cracked concrete to transmit tension and shear; $f_c^{'}$ is the compressive strength of concrete; b_v is the web width; d_v is the effective shear depth, taken as the distance between the resultants of the tensile and compressive forces due to flexure; A_v is the area of the transverse reinforcement (stirrups) at a spacing s; f_y is the yield stress of the transverse steel; and θ is the principle compression angle. Note that Eq. 2 represents the upper limit of V_n, which is intended to prevent a web crushing failure prior to yielding of the transverse reinforcement. Typically, this limit does not govern the design shear capacity (Eamon et al., 2014).

For sections that contain at least the minimum specified amount of transverse reinforcement ($A_{v\text{-}\min} = 0.083\sqrt{f_c^{'}}\frac{b_v s}{f_y}$), β is taken as:

$$\beta = \frac{4.8}{(1+750\varepsilon_s)} \qquad (3)$$

where ε_s is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement, given by:

$$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps} f_{po}\right)}{(E_s A_s + E_p A_{ps})} \qquad (4)$$

In this expression, M_u is the factored moment; N_u is the factored axial force*;* V_u is the factored shear load; f_{po} is a parameter usually taken as $0.7 f_{pu}$; E_s and A_s are the Young's Modulus and area of the nonprestressed steel on the flexural tension side; and E_p and A_{ps} are the Young's Modulus and area of the prestressed steel on the flexural tension side, respectively. When the calculated ε_s is negative, it may be recomputed by replacing the dominator of Eq. 4 by $(E_s A_s + E_p A_{ps} + E_c A_c)$, where E_c and A_c are the elasticity modulus and cross sectional girder area of concrete on the flexural tension side, respectively.

For sections that do not contain the minimum amount of shear reinforcement, β becomes:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \frac{51}{(39 + s_{ex})} \tag{5}$$

The crack spacing parameter s_{xe} is calculated as:

$$s_{xe} = s_x \frac{1.38}{a_g + 0.63} \tag{6}$$

where s_x is the lesser of d_v or the maximum distance between layers of longitudinal crack control reinforcement, and a_g is the maximum aggregate size.

The value of θ, regardless of the amount of transverse reinforcement, is given by:

$$\theta = 29 + 3500\varepsilon_s \tag{7}$$

Shear Adjustment Factor

The shear adjustment factor considered in this study is given by Chehab and Eamon (2018), and is taken as:

$$r_{LRFD} = 0.0061 f_c^{'} + 0.028\sigma + 0.0012 s(\frac{142}{A_v}) + 0.00036h - 0.15 \tag{8}$$

where $f_c^{'}$ = concrete compressive strength (MPa); σ = average stress in the section due to prestress force (MPa); s = stirrup spacing (mm); and h = girder height (mm). Using this procedure, shear capacity is evaluated by multiplying the nominal shear capacity found from the LRFD General Procedure (i.e. the result of Eq. 1) by the result of Eq. 8, where r_{LRFD} is not to exceed the range: $0.88 \leq r_{LRFD} \leq 2.62$. To further enhance accuracy, Chehab et al. (2018) recommended to modify the computation of V_n in Eq. 1 by iterating until V_n equals V_u (from Eq. 4), by changing the value of V_u until balance occurs. As discussed in Chehab and Eamon (2018), Eq. 8 was developed from a selection of experimental results and a database of validated finite element analyses. Its applicability was to be limited to Type II, III, and IV AASHTO PC girders with $f_c^{'}$ from 38-55 MPa, σ from 3.4-17 MPa, and s from 75-610 mm, which represents the range of parameters for which it was developed. Practically, Eq. 8 represents an estimation of the ratio of the “exact” shear capacity to the nominal capacity found from the LRFD method.

Use of the above procedure was found to significantly improve accuracy as well as decrease variability, while producing no unconservative estimations of shear capacity. A selection of the results reported by Chehab and Eamon (2018) is shown in Fig. 1, where the ratio of an "exact" assessment of shear capacity to the LRFD assessment of nominal shear capacity "Exact/LRFD" as well as the ratio of exact shear capacity to that assessed by the use of Eq. 8 "Exact/(LRFD x r_{LRFD})" is shown for approximately 200 typical PC bridge girders. Here, the "exact" assessment is taken as the shear capacity determined from an experimentally-validated finite element model (Chehab et al., 2017; Chehab and Eamon 2018; Chehab et al., 2018). As shown in Fig. 1, the majority of the unmodified LRFD results are substantially inconsistent, where the Exact/LRFD ratio ranged from 0.88 to 2.62, with an overall mean ratio of 1.67 and coefficient of variation (V) of 0.25. Particularly troublesome are the several cases with Exact/LRFD ratios below 1.0, indicating unconservative estimates of shear capacity. Use of Eq. 8 still results in substantial conservatism overall, where the mean ratio of Exact/(LRFD x r_{LRFD}) is 1.38. However, the upper range of ratios is substantially decreased from 2.62 to 1.72, with no cases where capacity is over-predicted. Correspondingly, the consistency in shear capacity estimation is likewise greatly improved, with V reduced by approximately 50%, to 0.12. Note that the use of a refined finite element analysis procedure (i.e. the "exact" approach) to evaluate shear capacity would reduce discrepancies even further to near zero, but such an approach is not practical for routine bridge rating due to the modeling skill and computational effort involved.

The normalized sensitivity of Eq. 8 to its input parameters is shown in Fig. 2. As indicated in the figure, average prestress force has most influence on results, followed by transverse steel spacing, whereas concrete strength and especially girder height have least influence. Due to the general conservatism of the LRFD method, as shown in the Figure, all parameters positively influence Eq. 8, and hence increase the (Exact/LRFD) V_n ratio. As noted above, in some cases, the LRFD approach was found to be unconservative, and in such cases it is possible for Eq. 8 to produce a value less than 1.0 (due to the presence of the negative constant 0.15 in the expression). Note that these results indicate the sensitivity of the V_n ratio (Exact/LRFD) to these parameters, not the direct sensitivity of V_n.

Bridge Girders Considered for Rating Evaluation

To examine the effect of using a more accurate shear capacity assessment method in rating such as that given by Eq. 8, consideration should be given to location, where legal loads as well as rating procedures vary from state to state. In this study, the State of Michigan is considered as an example, which has relatively high legal loads (Eamon et al., 2016), and where use of the (generally) significantly conservative existing procedure to evaluate shear capacity may be particularly detrimental. Once the location of consideration was selected, 20 hypothetical prestressed concrete AASHTO bridge girders (including Types II, III and IV) of four span lengths (15.2, 24.4, 30.5 and 61 m), and five girder spacings (1.2, 1.8, 2.4, 3.1 and 3.7 m), were considered for rating in accordance to the current procedure specified in the MBE (AASHTO 2018).

According to the MBE, the rating factor for legal loads is determined by:

$$RF = \frac{\phi R_n - 1.25DC - 1.5DW}{\gamma_{LL}(LL + IM)} \tag{9}$$

In the expression above, R_n is the nominal resistance of the component; DW and DC are the dead loads of the wearing surface and the remaining structural components, respectively; IM is the vehicular dynamic load allowance, specified as 33% of the vehicular static live load; and the resistance factor (ϕ) is given as 0.9 for a shear limit state for PC beams.

The remaining parameters, LL and γ_{LL}, represent the maximum shear load effect generated from a legal vehicle configuration and the associated live load factor used for rating, respectively. Here, the total legal vehicle shear load on the bridge is proportioned to an individual girder by a distribution factor, as a function of bridge geometry, as specified in AASHTO LRFD (AASHTO 2017). For calculation of dead loads as well as the live load distribution factor, it is assumed that bridges have a width of 14.6 m (two traffic lanes with shoulders) with a 230 mm thick concrete deck (f'_{cs} = 28 MPa), and 65 mm wearing surface. A summary of design parameters used, which are typical for many existing AASHTO-type PC bridge girders designed in Michigan as well as other states, is given in the last column of Table 1. Note that many of these values vary, depending on the specific bridge geometry considered.

The Michigan Department of Transportation (MDOT) procedure for load rating is more complex than that of many other states, where 28 different legal truck configurations are considered, each with different load factors. For the bridge spans considered in this study, two of these configurations govern, depending on span, and are shown in Fig. 3. Generally, rating factors resulting from Eq. 9 that are greater than 1.0 are acceptable while those less than 1.0 require traffic restriction across the structure.

Limit State Function

The limit state function considered for shear capacity evaluation can be simply written as:

$$g = R(R_i) - Q(Q_i) \tag{10}$$

where R is girder shear resistance, as a function of resistance random variables R_i, and Q is the maximum shear load that the girder experiences, as the sum of the dead and live load shear effects, as a function of load random variables Q_i. These models and the corresponding random variables are described below.

Initial Resistance Model

To provide an accurate assessment of the reliability level inherent in a given standard, it is important that the reliability of a component is evaluated for the minimum requirements set by that standard. In the general LRFD approach, this condition is expressed as: $\phi R_n = \sum \gamma_i Q_i$, (where γ_i are load factors and Q_i are load effects) which is the code-specified limit between acceptability and non-acceptability. In the case of rating, acceptability is expressed in terms of the rating factor, for which the limit is taken as 1.0. Setting Eq. 9 equal to 1.0 and solving for the required R_n results in:

$$R_n = (1/\phi)(1.25DC + 1.5DW + \gamma_{LL}(LL + IM)) \tag{11}$$

which, in this study, becomes the nominal shear resistance for consideration in reliability rating. Here it should be noted that R_n from Eq. 11 represents a notional, or theoretical resistance, used for evaluation of the reliability level associated with the rating process, and does not necessarily represent the resistance of an actual girder. This is analogous to the evaluation of components with resistance set just equal to the design limit for reliability assessment of design code specifications, whereas overdesign of an actual girder, particularly for shear, is rarely unavoidable.

Statistical parameters of shear resistance uncertainties for typical PC bridge girders are available, and have been conveniently expressed as a single random variable, *R* (Nowak, 1999). However, the development of these statistics have assumed that the existing code procedure is sufficiently accurate to model capacity. In this study, however, it is suggested that simplified code methods are not necessarily adequate for accurate assessment of rating reliability, and hence the use of Eq. 8. This requires that the statistical parameters of girder shear resistance are recalculated. Since Eq. 8 is a function of girder properties, the uncertainties associated with specific girder designs must be established in order for Eq. 8 to be evaluated. To facilitate this, the set of girders described above, corresponding to 20 hypothetical bridges with spans from 15.2-61 m and girder spacing 1.2-3.7 m, are designed according to MDOT standards. These standards are identical to AASHTO LRFD specifications, with the exception of a higher live load than the HL-93 design load specified in the AASHTO code. For shear design, this is equivalent to satisfying Eq. 11, but in this case, *LL* is determined not from legal loads but from the MDOT-specified HL-93-mod design load (taken from 1.2 - 2.7 times higher than AASHTO's HL-93 load, depending on the spans considered here), and the live load factor (γ_{LL}) is specified as 1.75.

Resistance random variables used for reliability analysis are given in Table 1, where statistical parameters, in terms of coefficient of variation (V) and bias factor (λ, the ratio of mean to nominal value) are taken as those used to calibrate the AASHTO LRFD Specifications for consistency with previously established reliability levels (Ellingwood et al. 1980, Siriaksorn 1980; Nowak 1999; Nowak and Szerszen 2003; Yamani 1992).

For comparison purposes, it is useful to first evaluate reliability based on resistance established with the current, or initial LRFD model, without application of Eq. 8. In this initial resistance model, Eq. 11 is first used to determine the required nominal shear capacity for a girder, R_n, for a given bridge span and girder spacing. A hypothetical girder design is then developed to exactly match this required value using the exiting LRFD shear procedure (Eqs. 1-7). Resistance (R) in Eq. 10 thus becomes a function of Eqs. 1-7, above, with all equation parameters no longer deterministic but replaced with the appropriate random variables shown in Table 1; either directly, such as random variables R_1, R_2, R_4, R_5, etc., which have a corresponding parameter in Eqs. 1-7, or replaced with functions of these random variables such as the parameter d_v, which can be expressed as an algebraic function of random variables R_3 (d_e), R_7 (f_{pu}), R_9 (A_{ps}), R_{10} (b_e), and R_{13} (f'_{cs}) applicable for the design cases considered. Exceptions are parameters E_s and A_s, which appear in Eq. 4, parameters for longitudinal non-prestressed (mild) steel, which is not used in the prestressed girder designs considered, and parameter a_g, in Eq. 6, which is taken as a deterministic value.

Updated Resistance Model

The updated resistance model considers the effect of Eq. 8 in the shear resistance evaluation. This model is identical to the initial model described above, except now the resistance function is multiplied by the result of Eq. 8. Note that the girder design itself is unchanged, only the evaluation of its capacity R for reliability analysis within Eq. 11. When reliability is evaluated considering Eq. 8, its result is no longer a deterministic value, but its input parameters (f'_c, σ, s, A_v, and h) also become functions of the random variables given in Table 1. Of these, R_1, R_4, R_6, and R_{11} directly replace parameters $f_c^{'}$, A_v, s, and h, respectively, while σ is expressed as a function of random variables R_2 (b_v), R_7 (f_{pu}), R_9 (A_{ps}), and R_{11} (h).

Simplified Resistance Model

This model is used only as part of the model verification process, as discussed further below. In this case, rather than forming a resistance function with Eqs. 1-7 and using the fundamental random variables given in Table 1, girder shear resistance is taken as a single random variable R, using the final shear resistance parameters reported for the LRFD calibration (where R is lognormal with λ_R = 1.15 and V_R = 0.14). This model represents the approach used in the LRFD and MBE calibrations to compute reliability indices. As with the other resistance models above, the nominal value for R is determined from Eq. 11.

Professional Factor

For this study, one resistance random variable that requires further consideration is the professional factor (P), which is used to account for uncertainty in the analysis model used to establish member strength. In the initial AASHTO LRFD calibration, P_{LRFD} was taken to have a bias factor of λ=1.075 with V=0.10 (Nowak 1999). P_{LRFD} is used in this study when the Initial Resistance Model is considered. Although the origin of these values are not clearly documented, these statistics appear to greatly underestimate the actual level of uncertainty and conservatism in the LRFD shear capacity model. For example, if the results of Chehab and Eamon (2018) are considered, the resulting statistics for P for the LRFD model are λ=1.67 and V=0.25, whereas the adjustment of Eq. 8 produces an estimated P factor of λ=1.38 and V=0.12. As expected, Eq. 8 results in less conservatism as well as less variation than the unmodified LRFD procedure.

In general, for reliability analysis using Eq. 8, when the Updated Resistance Model is considered, it is desirable to use the most accurate statistics for P that are available. However, because the target reliability levels within the MBE were set with what appears to be non-representative *P* factor statistics, using the correct bias factor for Eq. 8 would produce nonsensical results in the context of the MBE. This difficulty can be illustrated as follows. As discussed above, Eq. 8 has shown to produce both lower conservatism as well as lower variation from the existing model; i.e. λ as well as V have decreased, as would be expected from any model improvement. However, the values of λ=1.075, V=0.10 were used for the existing model in the LRFD and MBE calibrations to set the target reliability indices rather than the reportedly more accurate values of λ=1.67 and V=0.25. Because of this, if the correct *P* factor associated with Eq. 8 is used for reliability assessment, an improvement in model accuracy would not be indicated, but rather accuracy will have nominally worsened, as both *P* factor values λ and V associated with Eq. 8 (1.38 and 0.12, respectively) are greater than the values used for the existing model used in the MBE calibration (1.075 and 0.10, respectively).

Ideally, the MBE shear results would be recalculated, and new shear reliability targets set with the existing LRFD model while using the more accurate *P* factor values of 1.67, 0.25. This would allow direct use of the correct *P* factor values associated with the updated shear model in reference to the existing MBE reliability targets. However, recalibration of the MBE is not a practical solution, at least in the context of this study. As an alternative, rather than recalibrating the MBE, the *P* factor for Eq. 8 can be calibrated to fit within the context of the MBE. The calibrated *P* factor should represent an equivalent degree of model improvement from the LRFD model, but yet remain compatible with original MBE assumptions.

Although alternate ways of calibrating the *P* factor may by possible, the method suggested here is to scale the correct *P* factor to the assumed baseline of the MBE while maintaining the correct proportional differences in the models. The calibrated coefficient of variation for *P* is then simply: $V_{EQ8} = V_{LRFD} \cdot (V_{cEQ8}/V_{cLRFD})$, where V_{LRFD} is the value originally used in the MBE for the existing LRFD model (i.e. V_{LRFD} = 0.10), and V_{cEQ8}, V_{cLRFD} are the 'correct' values for the updated shear model and LRFD models, respectively (V_{cEQ8} = 0.12 and V_{cLRFD} = 0.25). Using this expression produces a calibrated coefficient of variation (V_{EQ8}) of 0.048.

A similar process can be used to calibrate bias factor, but the calibration scale must be adjusted to a baseline of 1.0 (which indicates no bias) rather than 0 as used for V (which indicates no variation), resulting in: $\lambda_{EQ8} = 1.0 + (\lambda_{LRFD} - 1.0) \cdot (\lambda_{cEQ8}/\lambda_{cLRFD})$. This expression produces a calibrated bias factor of λ_{EQ8} = 1.062. Thus, the resulting scaled P factor (P_{EQ8}) values of λ=1.062 and V=0.048 are used when Eq. 8 is considered to compute shear resistance; i.e. when the Updated Resistance Model is considered.

It must be emphasized that an adjustment of this nature is essential, as if it was not done and the uncalibrated λ_{cEQ8} and λ_{cLRFD} bias factors were used, mean capacity and hence reliability index of the Updated and Initial Models would be significantly over-predicted relative to the MBE reference reliability levels. In the reliability analysis for both models, P is introduced as an additional random variable multiplied with the resistance function (i.e. as P_{LRFD} for the Initial Model and P_{EQ8} for the Updated Model). Note for the Simplified Resistance Model, the P factor given for the Initial Resistance Model (P_{LRFD}) was already included in the final resistance statistics reported in Table 1 for R (Nowak 1999).

Load Models

The dead load model is composed of random variables describing variation in the weight of prefabricated components (Q_P) such as the girders; site-cast components (Q_S) such as the deck and barriers, and the wearing surface (Q_W). Statistical parameters are taken as those used to calibrate the AASHTO MBE and LRFD (Nowak, 1999), and are given in Table 2. Dead load RVs are normally distributed. The live load model is taken from Eamon et al. (2016), and was developed from 66 million truck records collected over two years of weigh-in-motion data representing legal and permit vehicles across twenty sites on Michigan roadways. Actual vehicle load effects were calculated by incrementing the recorded vehicle configurations and spatial relationships across a beam model of the considered span length. It was found that the extreme (high) values of load effect well-fit a normal distribution. Correspondingly, load effects were then statistically projected with extreme value theory to determine maximum expected load statistics for a 5-year return period, similar to the process used to set legal and routine permit reliability levels in the MBE. Live load uncertainties include those from the data projection (V_{proj}), geographic location (V_{site}), the data collection at a particular site (V_{data}), and vehicle dynamic load (V_{IM}). Values were taken as 0.02 for V_{data}, and 0.09 and 0.055 for V_{IM} for single lane and two-lane load effects, respectively (Eamon et al. 2016). Values for the mean maximum shear effect, V_{proj}, V_{site}, and the final resulting coefficient of variation of shear effect, $V_{L\max}$, are span-dependent and given in Table 3.

Live load effects are distributed to the girder using AASHTO LRFD distribution factors, which were similarly used in the AASHTO LRFD and MBE reliability calibration efforts. For the spans considered in this study, the one-lane shear live load effect governs for the Michigan-specific data, as discussed below (Eamon et al. 2016). The corresponding live load distribution factor for shear is taken as: $0.36 + 0.131S$, where S (m) is girder spacing (AASHTO 2017). Note that an additional source of uncertainty that may be considered is that due to vehicle live load distribution to the girders, which generally results in a net increase in calculated girder reliability due to the conservativeness of the AASHTO LRFD load distribution expressions. However, a concern with this adjustment is that it is primarily due to the edge the stiffening effect of barriers (Eamon and

Nowak 2002; 2005), which, although are effective in reducing girder live load under service loads, are not designed nor intended to act as primary structural elements and may not be reliable in aiding load distribution during an overload. Therefore, this adjustment may not be desirable in rating and was not included in this study. Rather, load distribution was conservatively taken as deterministic, as it was for the AASHTO LRFD calibration (Nowak 1999).

Verification of Reliability Model

Prior to evaluating the effects of the Updated Resistance Model, the girder reliability indices computed from this study using the Initial Resistance Model should be similar to those that were found during the initial calibration of the AASHTO Specifications. A verification of the model used here is necessary because the basic shear resistance random variables that were used to calibrate the AASHTO specifications, as well as the exact version of the method used to evaluate shear capacity, were not clearly documented (Nowak, 1999; Kulicki et al., 2007). It is important that the Initial Resistance Model considered in this study produces similar results as that used to develop the MBE, if the target reliability indices specified for the MBE are to have meaningful comparison value to the results of the Updated Resistance Model. Here note that AASHTO LRFD calibration information is referenced, in which girder shear resistance statistics were developed and later used for the MBE calibration as well.

For the validation, using the load models considered in the AASHTO LRFD code calibration, (the same dead load model discussed above, but the live load model is not Michigan-specific) (Nowak 1999), reliability index was computed using the First Order Reliability Method (FORM) (Rackwitz and Fiessler 1978) for a selection of typical PC girders, where girder shear resistance is computed with the Initial Resistance Model. The results of this calculation are given in Fig. 4, and are compared to the values reported for the AASHTO LRFD Calibration ("LRFD"). As shown in the figure, results are very similar, and nearly exact in most cases. As a second means of verification, the Simplified Resistance Model was also considered. In this case, it was found that reliability results were also very close to the LRFD calibration results, as shown in Fig. 4. Therefore, the resistance model used in this study was taken as validated. Note that the target reliability indices shown in Fig. 4, specified for design, are substantially higher (from 3.5 to 4) than those which are specified for rating (from about 1.5 to 2.5; see Results discussion next).

Results

The reliability index for Eq. 10 is computed using FORM, and results are given in Figs. 5-7. In the figures, reliability indices are given in order for 4 span lengths with 5 girder spacings each (from 1.2 to 3.7 m), for different methods of computing girder resistance. For various cases, FORM results were verified with Monte Carlo Simulation using $1x10^6$ simulations, and were found to be nearly identical (differences in reliability index less than 1-2%).

Fig. 5 presents results using the Simplified Resistance Model (i.e. that used in the MBE and LRFD calibrations), where R is taken as a single random variable with constant statistical parameters ($\lambda_R = 1.15$ and $V_R = 0.14$) for all girders. As shown, the values in Fig. 5 are substantially lower than those given in Fig. 4, where the only difference between the results shown in Figs. 4 and 5 is the live load model used. This difference is expected, as the Michigan live load data have larger load effects than those used for the LRFD code calibration (Eamon et al. 2016). Although not as severe as those in Michigan, the load effects of some states considered in the MBE calibration were found to have relatively high load effects as well, resulting in a target reliability index for rating of 2.5, with a minimum index allowed for any girder of 1.5 (Sivakumar and Ghosn 2011) (compared to a target as well as minimum allowed reliability index for design with AASTHO LRFD of 3.5). Also note that the values given in Fig. 5 have a larger variation than those given in Fig. 4. This is also due to the different live load model used. In the LRFD calibration live load model, two-lane load effects always governed; thus, the distribution factor used to distribute live load shear to a girder for design is the same factor as that used in the reliability analysis (Nowak 1999). However, for the Michigan traffic data, it was found that multiple very heavy vehicles in a single lane dominated the shear load effect (Eamon et al. 2016). In this case, although a two-lane distribution factor still must be used for design for a two-lane bridge (as well as for rating, for legal and routine permit loads), a single lane distribution factor is used in the reliability analysis to proportion the dominant single lane load effect to the girder. For the spans considered, it can be shown that the ratio of the single lane to the two-lane distribution factor increases as girder spacing decreases. Thus, the proportion of actual shear load (based on the 1-lane factor) to the shear load used for rating (based on the two-lane factor) similarly increases (Eamon et al. 2016). This causes a drop in reliability level for more closely-spaced girders. As shown in the figure, reliability indices ranged from 1.37 to 2.36, with a mean value of 1.87. As the target reliability index of MBE-rated girders is a minimum of 1.5 in any case and 2.5 overall, two considered girders currently do not meet the individual minimum, and the group overall does not meet the average required.

Fig. 6 results are computed based on the Initial Resistance Model, where resistance is evaluated as a function of the basic random variables given in Table 1 and Eqs. 1-7, rather than the single resistance random variable used in the original LRFD calibration (the Simplified Model). The average reliability results of Figs. 5 and 6 are similar, as expected, since the resistance models are the same as those used to compute the design reliability indices shown in Fig. 4, which show very close results for these two resistance models when the nominal girder resistances used for the LRFD calibration are considered. However, although the average reliability shown in Fig.

6 for the Initial Model (1.93) is very close to that using the Simplified Model shown in Fig. 5 (1.87), there is substantially more variation of reliability shown in Fig. 6, where reliability indices range from 1.1 to 2.69. This is because the constant, single variable (*R*), resistance statistics used for all girders in the MBE and LRFD calibrations as per the Simplified Model, which represents a typical value, actually varies somewhat depending on the specific girder geometry considered. This variation becomes apparent when a different set of girders (i.e. those appropriate for MI traffic loads) are considered rather than the set corresponding to the hypothetical girder resistance values used in the LRFD calibration.

Here again, the average reliability index of 1.93 is shown to be below that required (2.5), and 4 girders have reliability below the minimum of 1.5 required. This represents a significant problem, as these reliability indices are based on non-deteriorated resistance (see Eq. 11); i.e. some existing structures, as-designed, will not meet rating reliability requirements. This finding would require recalibration of Michigan's live load model used for rating, resulting in an increase of the required live load factors used for rating and a decrease in the rating factor for bridges with PC girders. Practically, this may result in additional traffic restrictions on numerous structures.

Fig. 7 provides girder reliability indices when the adjustment suggested by Eq. 8 is used to evaluate shear capacity and thus the Updated Model is evaluated. To illustrate the effect of Eq 8. on results, three different girder designs were developed for each bridge case. The middle line represents typical girder designs used in Michigan, those commonly considered for the span and girder spacing considered. The upper and lower lines represent designs using different concrete strengths and corresponding stirrup spacings, but such that the nominal shear capacity is unchanged (f'_c and stirrup spacing vary, but are within the range shown in Table 1, across the spans and girder spacings considered, where higher f'_c and wider stirrup spacing (*s*) result in the higher reliability cases when Eq. 8 is applied; see Fig. 2). Comparing Fig. 6 and Fig. 7 results, it can be seen that only minor differences occur for typical designs within the smallest span (15.2 m) when Eq. 8 is applied, whereas much larger differences in reliability assessment occur for the longer spans. This is because the existing LRFD procedure was found to be relatively accurate for the lower prestress levels needed for the shorter spans; as the spans increase, the section prestress level increases, causing a greater discrepancy between the capacity provided by the LRFD method and the adjustment from Eq. 8 (see Fig. 2). Using Eq. 8 to evaluate capacity results in reliability indices for typical designs ranging from 1.81 to 4.81, with an average of 3.77. Thus, in the case of Michigan bridge girders, this more accurate shear assessment allows all typical girder designs to meet minimum required reliability targets, avoiding unnecessary traffic restrictions as well as over-rating shear resistance in some cases.

Note that the reliability targets in the MBE were set considering girders in an undeteriorated state, as are the reliability results shown in Figs. 5-7. However, clearly, as girders deteriorate, shear capacity, and hence reliability, may drop. If the reliability index of a girder drops below the minimum acceptable level, either the girder must be repaired, or traffic must be restricted to restore reliability to that required target level. In the rating process, the rating factor is used as a surrogate metric for assessment of adequate reliability. Note that Eq. 8 does not have specific input parameters for material deterioration or other types of environmental damage. However, such damage can be accounted for in the rating procedure using Eq. 8 in the same way that existing code procedures can be used to do so. That is, V_n using the adjustment of Eq. 8 can be calculated with reduced $f_c^{'}$, web thickness, area of steel, prestress force, etc., as appropriate, based on semi-annual inspection and evaluations of the structure, to account for concrete deterioration and steel corrosion. Thus, whether undamaged or deteriorated girders are considered, a more accurate assessment of actual shear capacity and rating factor can be obtained, and hence a more accurate assessment of shear reliability relative to the target levels. To show the practical effect of the use of Eq. 8 in the reliability-based rating of existing structures, Fig. 8 provides the fractional reduction in nominal shear capacity (V_n) allowed that would provide a reliability index equal to 2.5. In particular, if V_n is reduced to the fraction shown in Fig 8 due to, say, a loss of transverse steel area from corrosion or concrete web area from spalling, a reliability index of 2.5 could still be maintained. As shown in the figure, most cases allow V_n to be reduced to about 80-90% (i.e. a loss of 10-20%) of its nominal value while still maintaining an acceptable level of reliability for rating.

Summary and Conclusion

In this study, the effect of using a simple procedure to enhance the accuracy of shear capacity prediction on PC bridge girder reliability in the rating process was examined. Although more accurate methods for shear capacity evaluation are available, such as a detailed finite element approach, such methods are associated with substantial effort and knowledge to implement and are generally impractical for routine bridge rating. Although conservative, a simple method such as that considered in this study is easy to implement, and can provide moderate increases in accuracy.

As a result of the study, it was found that significantly more variation in shear reliability in PC girders exists than assumed in the LRFD and MBE calibrations even when using the existing shear capacity and load models without adjustment. This was due to the use of a constant bias factor for shear resistance development, where use of a more detailed reliability model revealed that resistance statistical parameters vary with beam geometry. For girders on the edge of acceptability, such modeling, such as that used in this study, may be useful to prevent bridge posting.

It was also found that using the simple shear adjustment factor considered may provide significant advantages in bridge rating. Using Michigan PC bridge girders as an example, prior to application of the shear adjustment factor, several cases were found to have reliability indices below the minimum and average required rating reliability levels. Conversely, reliability indices were found to have met the required levels when the shear adjustment factor was applied, thus avoiding potentially rating structures unconservatively as well as reducing unnecessary traffic restrictions.

Because most code-based procedures are conservative, the use of many refined techniques, whether for capacity analysis such as that presented here, or, for example, the use of finite element analysis for load distribution in lieu of the AASHTO expressions, frequently result in more accurate, but less-conservative results. Although the AASHTO specifications allow such refined methods, the code reliability targets, whether for rating or design, were set based on less accurate, but more conservative methods. Thus, applying refined methods, such as Eq. 8, will practically result in a lower safety margin overall than if the less accurate methods are used, even if the required code reliability indices are met. If the overall reduction in conservatism is of concern, a possible solution that retains the accuracy of the refined method, as well as the prior level of conservatism, is to introduce an additional resistance factor with the refined procedure. This factor would be set such that the average level of reliability using the refined method matches that for the same structures analyzed using the code-based methods. As such, an adjustment is not required, however, most agencies may be reluctant to implement this approach, particularly in the context of rating. Regardless, although this is a broad issue and is beyond the scope of this study to fully address, it should be considered in the use of any refined technique such as that presented in the present paper.

References

AASHTO. (1994). *LRFD bridge design specifications*, 1st Ed. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2002). *Standard specifications for highway bridges*, 17th Ed. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2008). *Manual for bridge evaluation*, 1st Ed. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2017). *LRFD bridge design specifications*, 8th Ed. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. (2018). *Manual for bridge evaluation*, 3rd Ed. American Association of State Highway and Transportation Officials, Washington, D.C.

ACI (American Concrete Institute). (2014). "Building code requirements for structural concrete and commentary." *ACI 318-14*, Farmington Hills, MI.

Chehab, A. I., & Eamon, C. D. (2018). "Regression-based adjustment factor to better estimate shear capacity for load-rating simple span PC girders." *J. Bridge Eng.*, 10.1061/(ASCE)BE.1943-5592.0001224, 04018017.

Chehab, A. I., Eamon, C. D., Parra-Montesinos, G. J., & Dam, T. X. (2018). "Shear testing and modeling of AASHTO type II prestressed concrete bridge girders." *ACI Struct. J,* 15(3), 801-8011.

Chehab, A., Dam, T., Eamon, C. D., and Parra-Montesinos, G. (2017). "Finite element modeling of shear failure in prestressed concrete AASHTO beams." *Proc., PCI Convention and National Bridge Conf.*, Precast/Prestressed Concrete Institute, Chicago.

Eamon, C. D., Parra-Montesinos, G., and Chehab, A. (2014). "Evaluation of prestressed concrete beams in shear." *MDOT Rep. RC-1615*, Michigan DOT, Lansing, MI.

Eamon, C., Kamjoo, V., and Shinki, K. (2016). "Design live-load factor calibration for Michigan highway bridges." *J. Bridge Eng.*, 10.1061/(ASCE)BE.1943-5592.0000897, 04016014.

Eamon, C., and Nowak, A. S. (2002). "Effects of Edge-Stiffening Elements and Diaphragms on Bridge Resistance and Load Distribution," *J. Bridge Eng.*, Vol. 7, No. 5, pp. 258-266.

Eamon, C. and Nowak, A.S. (2005). "Effect of Edge-Stiffening and Diaphragms on the Reliability of Bridge Girders," *J. Bridge Eng.*, Vol. 10, No. 2, pp 206-214.

Ellingwood, B., Galambos, T.V., MacGregor, J.G. and Cornell, C.A., "Development of a Probability Based Load Criterion for American National Standard A58", NBS Special Report 577, U.S. Department of Commerce, National Bureau of Standards, 1980.

Hawkins, N. M., and Kuchma, D. A. (2007). “Application of LRFD bridge design specifications to high-strength structural concrete: Shear provisions.” *NCHRP Rep. 579*, Transportation Research Board, Washington, DC.

Hawkins, N. M., Kuchma, D. A., Mast, R. F., Marsh, M. L., and Reineck, K.-H. (2005). “Simplified shear design of structural concrete.” *NCHRP Rep. 549*, Transportation Research Board, Washington, DC.

Kulicki, J. M., Prucz, Z., Clancy, C. M., Mertz, D. R., & Nowak, A. S. (2007). “Updating the calibration report for AASHTO LRFD code.” *NCHRP Rep. 20-7(186)*, Transportation Research Board, Washington, D.C.

Laskar, A., Hsu, T. C., and Mo, Y. L. (2010). “Shear strengths of prestressed concrete beams, Part 1: Experiments and shear design equations.” *ACI Struct. J.*, 107(3), 330–339.

Llanos, G., Ross, B. E., and Hamilton, H. R. (2009). “Shear performance of existing prestressed concrete bridge girders.” *BD545-56*, Dept. of Civil and Coastal Engineering, Univ. of Florida, Gainesville, FL, 160.

MDOT (Michigan DOT). (2009). *Bridge analysis guide: 2005 ed., with 2009 interim update, parts 1 and 2*. MDOT Construction and Technology Support Area, Lansing, MI.

Nowak, A.S. (1999). “Calibration of LRFD bridge design code.” *NCHRP Rep. 368*. Transportation Research Board, Washington, D.C.

Nowak, A.S. and Szerszen, M.M., 2003, “Calibration of Design Code for Buildings (ACI 318): Part 1-Statistical Models for Resistance.” *ACI Structural Journal*, V. 100, No. 3, pp. 377-382.

Pei, J. S., Martin, R. D., Sandburg, C. J., and Kang, T. H. (2008). “Rating precast prestressed concrete bridges for shear.” *FHWA-OK-08-08*, Univ. of Oklahoma, Norman, OK.

Rackwitz, R., and Fiessler, B. (1978). “Structural reliability under combined random load sequences.” *Computers & Structures*, 9(5), 489-494.

Ross, B. E., Ansley, M., and Hamilton, H. R. (2011). “Load testing of 30-year old AASHTO Type III highway bridge girders.” *PCI J.*, 56(4), 152–163.

Siriaksorn, A., (1980), "Serviceability and Reliability Analysis of Partially Prestressed Concrete Beams", Ph. D. Thesis, The University of Illinois at Chicago Circle.

Sivakumar, B., & Ghosn, M. (2011). “Recalibration of LRFR live load factors in the AASHTO manual for bridge evaluation.” *NCHRP Project, 20-07*. Transportation Research Board, Washington, DC.

Vecchio, F. J., and Collins, M. P. (1986). “The modified compression-field theory for reinforced concrete elements subjected to shear.” ACI J., 832(2), 219–231.

Wilder, K., Lava, P., Debruyne, D., Wang, Y., De Roeck, G., and Vandewalle, L. (2015). “Stress field based truss model for shear-critical prestressed concrete girders.” *Structures*, 3(Aug), 28–42.

Yamani, A. S. (1992). “Reliability evaluation of shear strength in highway girder bridges (Doctoral dissertation).” Univ. of Michigan, Ann Arbor, MI.

List of Tables

Table 1. Resistance Random Variables

RV		Parameter Name	Bias Factor	V	Distribution*	Nominal Value**
R_1^a	$f_c^{'}$ (MPa)	Concrete Strength	0.95	0.150	LN	28-62
R_2^a	b_v (mm)	Web Width	1.00	0.05-0.067	N	150-200
R_3^b	d_e (mm)	Strand Effective Depth	1.00	0.017	N	1065-1500
R_4^b	A_v (mm^2)	Stirrup Area	1.00	0.015	N	140-250
R_5^b	f_y (MPa)	Stirrup Yield Strength	1.12	0.10	LN	414
R_6^c	s (mm)	Stirrup Spacing	1.00	0.040	N	75-610
R_7^a	f_{pu} (MPa)	Strand Ultimate Strength	1.04	0.025	LN	1,861
R_8^a	E_{ps} (MPa)	Strand Young's Modulus	1.00	0.060	LN	196,500
R_9^b	A_{ps} (mm^2)	Strand Area	1.00	0.0125	LN	1,615-
R_{10}^a	b_e (mm)	Effective Width	1.00	0.003-0.008	N	1,220-
R_{11}^b	h (mm)	Girder Height	1.00	0.011	N	915- 1,370

R_{12}[b]	t_s (mm)	Slab Thickness	1.01	0.044	N	230
R_{13}[a]	f'_{cs} (MPa)	Slab Concrete Strength	0.99	0.15	LN	28
	P_{LRFD}[d]	LRFD Professional Factor	1.075	0.10	N	1.0
	P_{EQ8}	Eq. 8 Professional Factor	1.062	0.048	N	1.0

*N=normal; LN=log-normal.
**Value depends on the specific girder design considered.
[a]Ellingwood et al. 1980; [b]Siriaksorn 1980; [c]Nowak 1999; [c,d]Nowak and Szerszen 2003

Table 2. Dead Load Random Variables

RV	Bias Factor	V
Q_P	1.03	0.08
Q_S	1.05	0.10
Q_W	*	0.25

*Wearing surface is taken as a mean thickness of 89 mm.

Table 3. Shear Live Load Statistics for Michigan-Specific Traffic

Span (m)	Mean Maximum Shear Load (kN)	Coefficient of Variation		
		V_{proj}	V_{site}	$V_{L\max}$
15.2	712	0.035	0.12	0.14
24.4	938	0.035	0.13	0.15
30.5	1040	0.035	0.13	0.16
61	1380	0.037	0.13	0.16

List of Figures

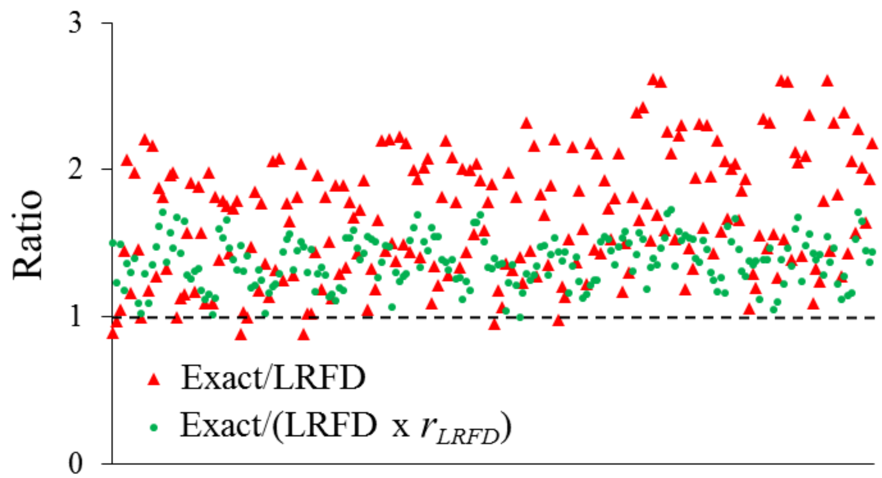

Fig. 1. Comparison of shear capacity ratios

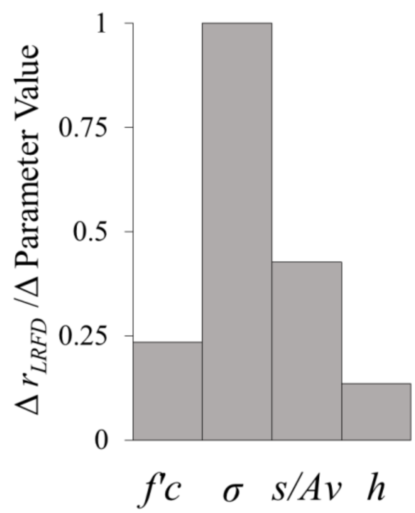

Fig. 2. Normalized sensitivity results of shear resistance variables

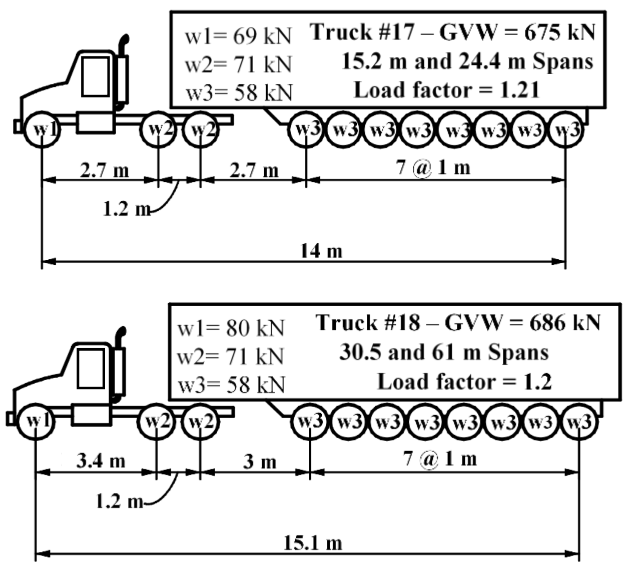

Fig. 3. MDOT governing legal trucks considered for reliability analysis (data from MDOT 2009)

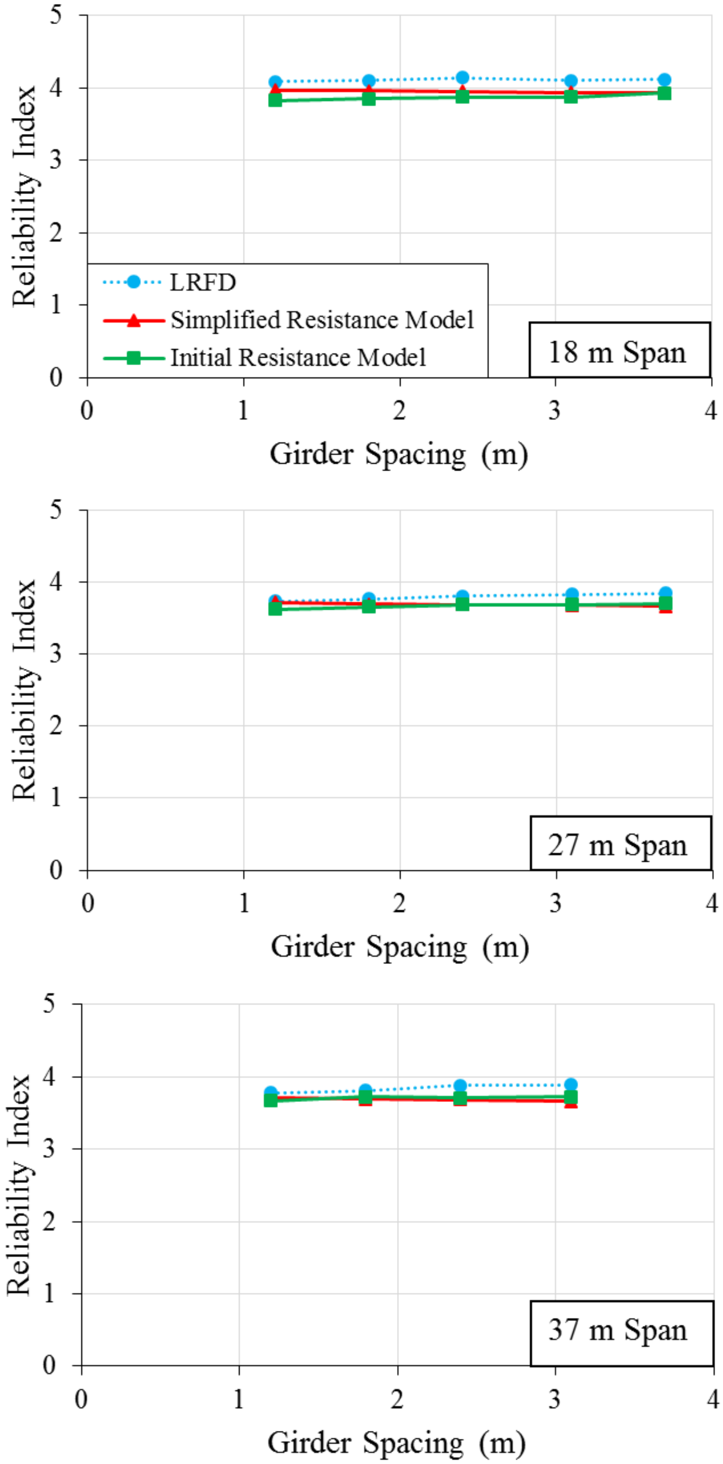

Fig. 4. Reliability indices for typical PC girders using AASHTO LRFD load models

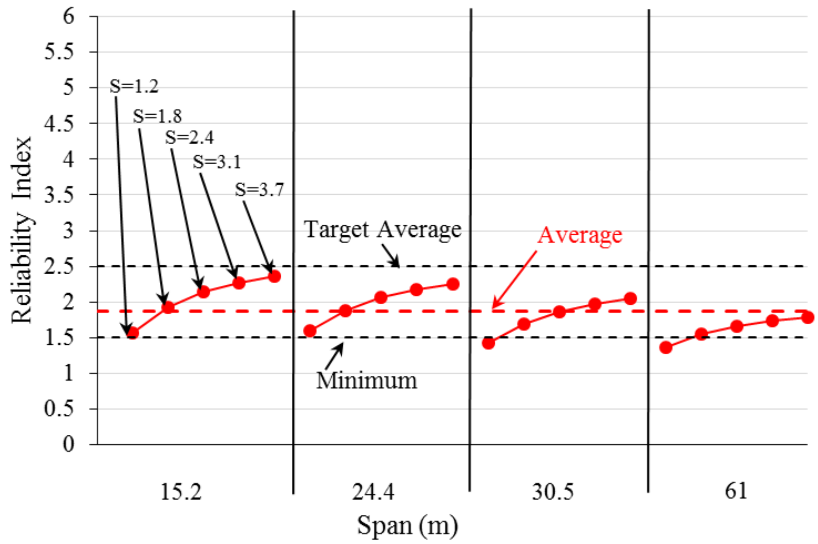

Fig. 5. Reliability indices for shear based on Michigan traffic loads and Simplified Resistance Model

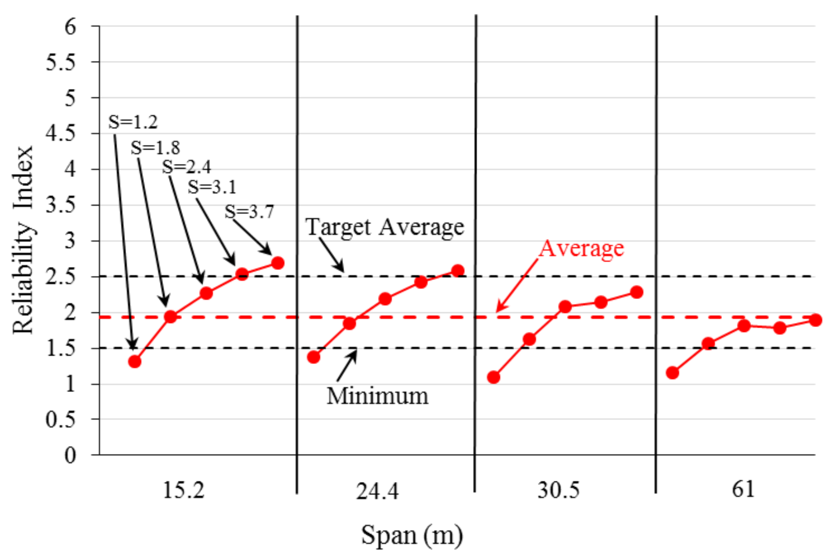

Fig. 6. Reliability indices for shear based on Michigan traffic loads and Initial Resistance Model

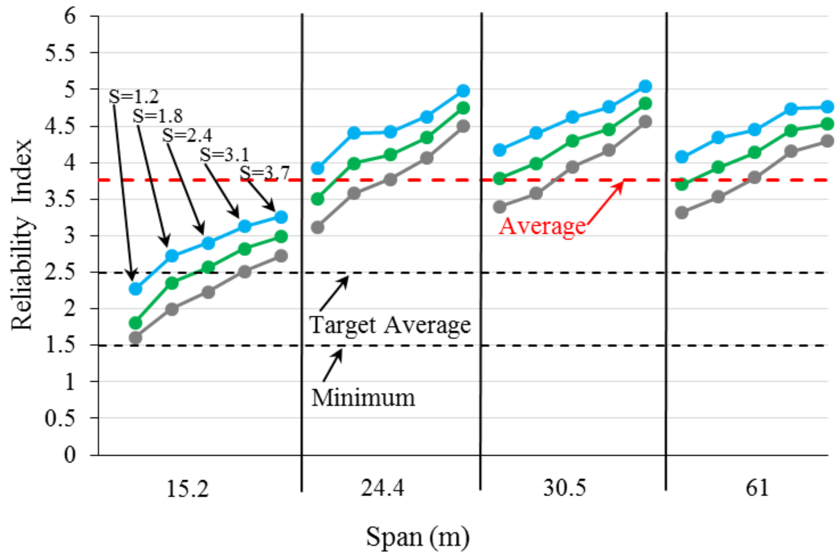

Fig. 7. Reliability indices for shear based on Michigan traffic loads and Updated Resistance Model

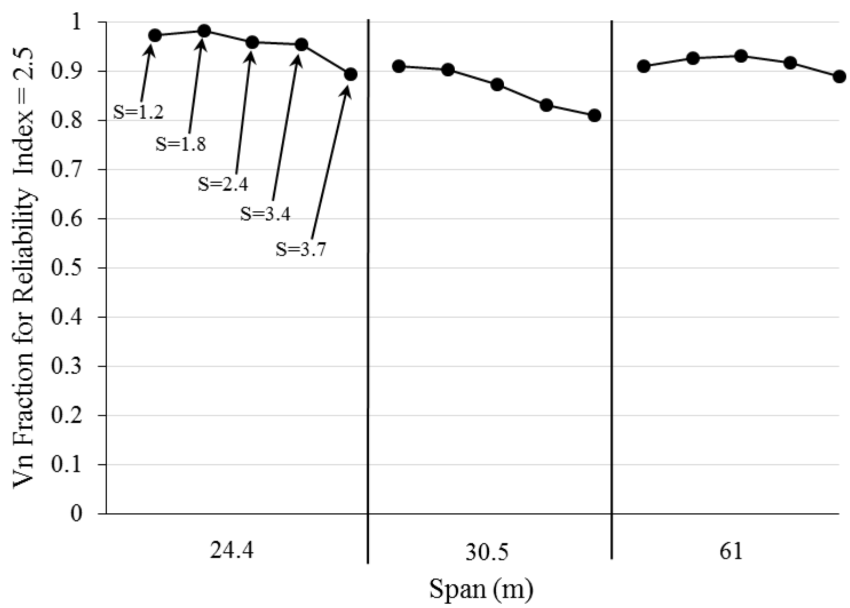

Fig. 8. Typical acceptable reduction in V_n for a reliability index of 2.5