Reliability-Based Service III Evaluation for Prestressed Concrete Girder Bridges under Platoon Loads

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This article is the third in a series. In the Winter 2024 issue of *ASPIRE*®, we introduced a potential strategy to safely allow truck platoons to increase truck weights based on live-load factor calibration and reliability principles. The premise underlying this strategy is that some trucks are—or will become—"smart," with the ability to drive long distances autonomously. With such intelligence, trucks will likely be able to report their axle weights and spacings and control their relative headways. In the Summer 2024 issue of *ASPIRE*, we outlined how reliability indices are used for a simple-span reinforced concrete T-beam bridge. The goal of that article was to provide an easy-to-follow example. We encourage readers to review the previous two articles for background relevant to this third article.

We need target reliability indices β to establish safe load limits that will not damage prestressed concrete girders. For example, for the strength limit state, β = 3.5 is often used for design, and β = 2.5 is used for load rating. However, there has not been much work on target reliability indices for Service Limit III. Wassef et al.[1,2] examined this issue in some detail with assumptions different from those for the 75-year design life. In the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[3] Service III was not formally calibrated, and load and resistance factors were based on judgment and experience.

In recent research,[4] we examined the estimated reliability index using the current AASHTO LRFD specifications and the 75-year design loads. Our goal was to determine whether the currently used index is acceptable for design as a benchmark for calibrating platoon permit load factors for service limits to be termed $\beta_{implicit}$. Once estimated, our calibration task was relatively straightforward. However, this task was also a significant one, and it produced information that could interest the design and rating communities. During the time that we were doing this work, there was also a robust debate—which continues to this day—about the computation of losses, the use of gross or transformed section, and live-load factors for design. Interim updates to the 9th edition of the AASHTO LRFD specifications (for example, Table 3.4.1-4) reflect this debate. The concepts here are also closely aligned with those issues. This article will focus on the reliability of different design options. The next article will address the determination of $\beta_{implicit}$ and the probability of cracking during a bridge's service life.

Background

Studies have investigated potential target β values for Service III, Wassef et al.[1,2] assumed that bridges designed based on the prestress loss method in the *AASHTO Standard Specifications for Highway Bridges*[5] performed well in service. To characterize live load means and coefficients of variation (CoVs), service evaluations were performed using weigh-in-motion (WIM) data for a site with an annual average daily truck traffic (ADTT) of 5000. Target $\beta_{implicit}$ indices were recommended for bridges according to various performance limit criteria (for example, decompression and maximum tensile stress). Wassef et al.[1] recommended a live load factor of 1.0 when the refined time-dependent loss method is used with elastic gains. Table 3.4.1-4 in the AASHTO LRFD specifications gives the Service III live load factor as either 0.8 or 1.0 depending on loss method and whether elastic gains are included.

Barker et al.[6] evaluated Service III for Wyoming bridges under large traffic volumes due to detours created by extended roadway closures that resulted in long strings of trucks. It had been assumed that bridges designed and evaluated under criteria from the current AASHTO LRFD specifications would perform satisfactorily. However, using Interstate 80 WIM vehicle load characteristics, roadway closures created load effects that resulted in negative Service III reliability indices. This research indicates that some reliability concerns may exist for heavy-load truck-train situations.

Reliability-based evaluation for Service III and its association with performance objectives, such as limiting cracking probability, are currently not clearly established in the AASHTO LRFD specifications, and that gap in the specifications hampers optimal truck platoon deployment. Furthermore, the flexibility afforded to bridge designers by language in the AASHTO LRFD specifications—such as selecting a prestress loss calculation method, using gross or transformed section properties, and determining whether elastic gains should be included—creates ambiguity concerning the level of performance intended by the specifications.

Our study focused on the following three objectives:

- Analyze the reliability of different design options.
- Determine $\beta_{implicit}$ for Service III.
- Investigate cracking probability under current design live load conditions during bridge service lives.

Sample results for the first objective are presented in this article.

General Reliability Analysis Procedure

Structural reliability analysis can determine whether the probability of exceeding a limiting criterion is acceptable. Equation (1) represents a general limit state function:

$$g(R,Q) = R - Q \quad (1)$$

where R and Q are random variables representing resistance and load effects, respectively. Monte Carlo simulation (MCS) is commonly used to conduct reliability analyses.[7] The index is routinely used in structural reliability frameworks to characterize structural safety (that is, strength limit states). **Figure 1** presents typical probability density functions (PDFs) for loads, resistance, and the strength limit state function. β represents the number of standard deviations that the mean value of $g(R,Q)$ is from zero (Fig. 1). At the strength level, design and inventory load ratings target $\beta = 3.5$, whereas operating load ratings relax the target β to 2.5. (See our article in the Summer 2024 issue of *ASPIRE* for details on the computation of β.)

Similar to Wassef et al.[1,2] and Barker et al.,[6] we assumed in this study that the $\beta_{implicit}$ based on current and past design criteria was adequate to provide satisfactory in-service performance. Nominal demands, live loads, and resistances were mapped onto probabilistic distributions with characteristic means and CoVs, and the probability of failure for each parametric combination was calculated using MCS with $N = 1$ million samples to determine β.

This process was repeated to conduct reliability analysis for different design scenarios, identify a $\beta_{implicit}$, and investigate $\beta_{cracking}$ for optimally designed prestressed concrete bridges for service. For this article, $\beta_{cracking}$ is defined based on a bridge designed using a tensile stress limit $0.948\sqrt{f'_c}$ (where f'_c is the design concrete compressive strength in ksi) and evaluated using a range of rupture moduli between $0.24\sqrt{f'_c}$ and $0.37\sqrt{f'_c}$.

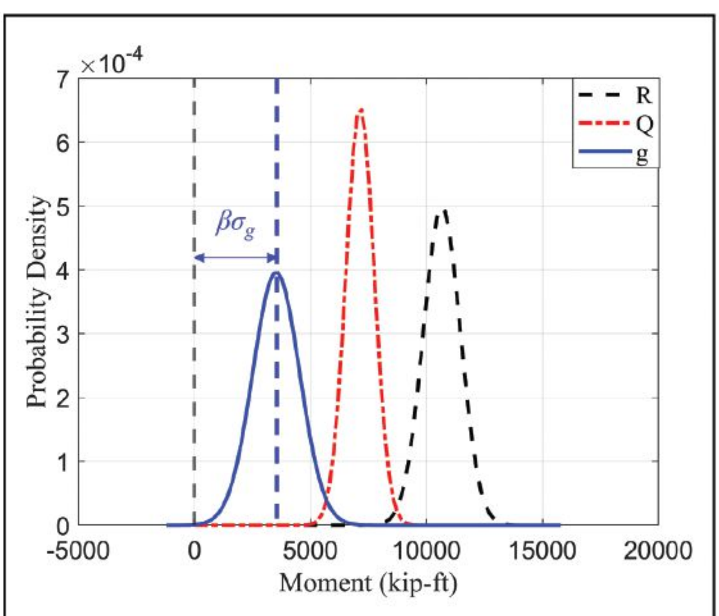

Figure 1. Example of probability density functions of load and resistance illustrating the reliability index for the strength limit state. All Figures: University of Nebraska.

Bridge Parameters

The AASHTO LRFD specifications, AASHTO standard specifications, and *Manual for Bridge Evaluation*[8] address tensile stress limits in positive moment regions. Accordingly, this study considered only positive moments at the middle of simple-span prestressed concrete girder composite bridges, and at 40% of the span length from supporting abutments for two equal-span, simple-made-continuous, prestressed concrete girder composite bridges. Bridges were assumed to carry two traffic lanes, and interior girders were designed using Nebraska University (NU) I-girders. The details are as follows:

- 60, 90, 120, and 150 ft spans
- NU 900 (60 ft), 1100 (90 ft), 1600 (120 ft), and 2000 (150 ft)
- 5 girders
- Girder spacings of 6, 8, 10, and 12 ft
- Composite deck thickness of 8.5 in. with 0.5 in. wear-off and 2 in. asphalt wearing surface
- 1 in. deck haunch
- 3.5 ft deck overhang
- 1-ft 5-in.-wide barrier with a weight of 0.124 kip/ft per girder
- Simple and continuous spans

Four bottom-fiber tension stress limit values f_t were considered for service designs. For simplicity, the coefficient before $\sqrt{f'_c}$ in the design f_t is referred to herein as κ. Preliminary analyses indicated that service reliability results for optimally designed bridges were not sensitive to final and initial concrete strength parameters. Initial concrete strength was used to calculate losses based on the refined time-dependent loss method in the AASHTO LRFD specifications. Parameters considered for κ, f_t, f'_c, and f'_{ci} included the following:

$f_t = \kappa\sqrt{f'_c}$ ksi, where $\kappa = 0$, 0.0948, 0.19, and 0.24

$f'_{c\_girder} = 8$ ksi

$f'_{ci\_girder} = 5$ ksi

$f'_{c\_deck} = 4$ ksi

Table 1 presents the design scenarios considered. Six different methods were used to design the girders optimally. Then, optimal designs were evaluated using MCSs for reliability analysis. In summary, hundreds of designs were evaluated with thousands of MCSs to yield the statistical data to compute the implicit reliability index $\beta_{implicit}$.

The design permutations totaled 96. However, the reliability index computations were observed to be insensitive to girder spacings; therefore, data presented here are limited to the 10 ft girder spacing.

The primary case considered was Post-1.0-Gains. Researchers also considered the Post-0.8-Gains scenario, representing how some bridges might have been designed in the period between the time when the post-2005 losses were included in the AASHTO LRFD specifications and the time when the required design γ_L was increased to 1.0. The Post-0.8-No-gains scenario is consistent with Table 3.4.1-4 in the AASHTO LRFD specifications. The use of approximate loss methods with or without elastic gains was interpreted by the investigators as an instance of ambiguity or subtlety in the AASHTO LRFD specifications, as Commentary C5.9.3.3 (Approximate Estimate

Table 1. Design scenarios for bridges

Scenario name	Loss computation	Design live load factor γ_L	Elastic gains considered?	Comment
Post-1.0-Gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	1.0	Yes	AASHTO LRFD specifications Table 3.4.1-4: Prestressed concrete components designed using refined estimates for time-dependent losses in Article 5.9.3.4 in conjunction with elastic gains
Post-0.8-Gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	0.8	Yes	Same as Scenario Post-1.0-Gains with a lower live-load factor
Post-0.8-No-gains	Post-2005 loss method (AASHTO LRFD specifications Article 5.9.3.4)	0.8	No	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Approx-0.8-Gains	Approximate loss method (AASHTO LRFD specifications Article 5.9.3.3)	0.8	Yes	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Approx-0.8-No-gains	Approximate loss method (AASHTO LRFD specifications Article 5.9.3.3)	0.8	No	AASHTO LRFD specifications Table 3.4.1-4: All other prestressed concrete components
Pre-1.0-No-gains	Pre-2005 loss method	1.0	No	AASHTO standard specifications Article 9.16.2.1

of Time-Dependent Losses) notes that "the losses or gains due to elastic deformations at the time of transfer or load application should be added to the time-dependent losses to determine total losses. However, these elastic losses (or gains) must be taken equal to zero if transformed section properties are used in stress analysis." After this work,[4] the *PCI Bridge Design Manual* clarified γ_L in Table 8.2.2.6-1.[9] Due to higher prestress loss predictions and a design live-load factor of 1.0, bridges designed using the Pre-1.0-No-gains scenario typically required the most strands.

Limit State Function for Service III

Reliability analyses were implemented by using nominal values and statistical parameters. HL-93 loading, girder distribution factors GDF_m, and dynamic load allowance IM from the AASHTO LRFD specifications were applied.

Equation (1) represents the general Service III limit state function. The present study expanded the equation as follows:

$$g = R - Q = \left(f_t \frac{P_{eval}}{A_g} + \frac{P_{eval} e_{nc}}{S_{ncb}} \right) - \left(\frac{\left(D_{gw} + D_{nc}\right)}{S_{ch}} + \frac{\left(D_c + D_w\right)}{S_{ch}} + \frac{LL_{HL\text{-}93}(1+IM)GDF_m}{S_{ch}} \right) \quad (2)$$

$$P_{eval} = A_{ps}\left(f_{pi} - \Delta f_{S_{eval}}\right) = A_{ps}\left(f_{pi} - \left(\Delta f_{pES} + \Delta f_{pLT} - \Delta f_{GainDL} - \Delta f_{GainLL}\right)\right) \quad (3)$$

$$f_{Reval} = f_t + \frac{P_{eval}}{A_g} + \frac{P_{eval} e_{nc}}{S_{ncb}} \quad (4)$$

where

P_{eval} = effective prestressing force for the evaluation of reliability index β

Δf_{Seval} = prestress loss for the evaluation of reliability index β

Δf_{pES} = elastic shortening loss

Δf_{pLT} = long-term prestress loss

Δf_{GainDL} = elastic gain from dead loads

Δf_{GainLL} = elastic gain from live loads

f_{Reval} = available resistance to tension stress for evaluation of reliability index β, which can be determined using Eq. (4).

Optimal Designs

The AASHTO LRFD specifications were used to determine the optimal numbers of strands for each prestress loss method, increased with span, with the design tensile stress parameter increased from $\kappa = 0$ to $\kappa = 0.24$. Fractional strands were used to avoid rounding issues. This article presents example summaries of these designs. The designs were then evaluated to determine the reliability indices for the Service III and cracking. Here, we focus on the design comparisons using the different assumptions. Different design assumptions lead to different reliabilities, which will be presented in the next article.

As one example, f_t with $\kappa = 0.0948$ is used to show intermediate calculation results for design and evaluation.

Figure 2 presents the optimal area of prestressing steel A_{ps} for f_t with $\kappa = 0.0948$, using an initial tensioning stress of $0.75f_u$. For 60 ft bridges, A_{ps} was close for different loss methods, but A_{ps} varied more across design cases when span lengths increased. A_{ps} for the Approx-0.8-Gains and Post-0.8-Gains scenarios were generally close and smaller than A_{ps} for the

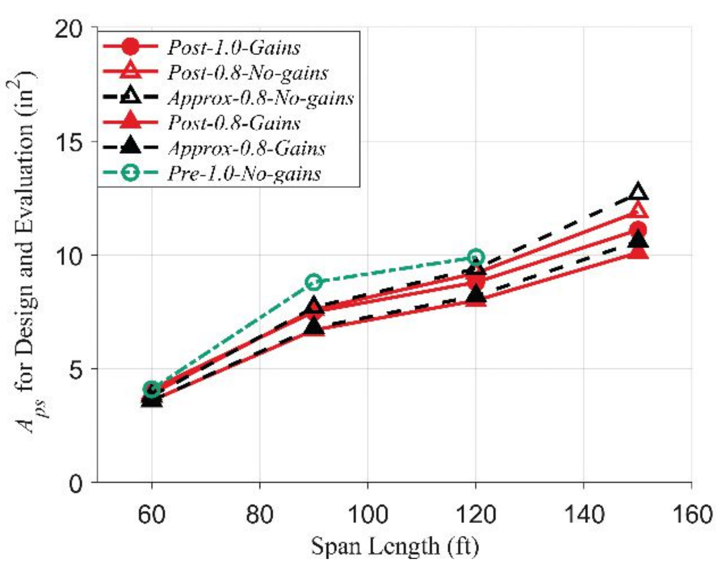

Figure 2. Optimal area of prestressing steel A_{ps} for various design scenarios.

Post-1.0-Gains option. Without considering elastic gains, A_{ps} for 150-ft-span bridges designed with approximate and post-2005 loss methods (γ_L = 0.8) was slightly larger than that for the Post-1.0-Gains method. Generally, the Pre-1.0-No-gains scenario produced larger A_{ps} than other methods. The Post-1.0-Gains scenario predicted the lowest losses, and the Pre-1.0-No-gains scenario gave the highest. The approximate and post-2005 methods predicted similar losses for cases using the same design live load factor and the same consideration of elastic gains.

Other design cases for f_t with κ ranging from 0.0 to 0.24 and as provided in Table 1 are found in Steelman et al.[4]

Summary

In this article, we presented the optimal designs used to evaluate platoon permits for Service III. The various design options with the AASHTO standard specifications and AASHTO LRFD specifications were used. This article summarizes the effects of a variety of typical design assumptions on the required A_{ps} for one span length; other spans are presented in Steelman et al. and Yang et al.[4,10] The next article will extend this work to evaluate the reliability indices for Service III and cracking limits.

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