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

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This article is the fourth 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 of this work is that trucks will become more intelligent and will be capable of driving long distances autonomously. With such intelligence, trucks in a platoon can likely 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 concrete T-beam bridge. Then, in the Winter 2025 issue, we presented the designs (optimal number of strands) for one- and two-span bridges of four span lengths, using six different scenarios involving loss models and Service III limit state live-load factors (0.8, 1.0), with gross or transformed sections. Limited data were presented in that article; however, complete results are provided elsewhere.[1–3] These designs provided the basis for our reliability analysis.

In the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[4] Service III is not formally calibrated, and load and resistance factors are based on judgment and experience. A goal of our research was to determine the implied index $\beta_{implicit}$ for current Service III designs, which was a significant task. We would then use $\beta_{implicit}$ as a benchmark for calibrating platoon permit load factors. Once estimated, our calibration task was relatively straightforward, and we can now determine the load factors for permitting these loads.

In this article, we build on the previous discussion and address the determination of $\beta_{implicit}$ and the probability of cracking during a bridge's service life. The analysis assumptions are the same as would be used for design (that is, 75-year design life, HL-93 loading, and AASHTO load factors). Refer to our article in the Winter 2025 issue of *ASPIRE* for definitions of variables and discussion of the reliability of different design options.

## Probabilities of Exceeding Tension Limits

Monte Carlo simulation (MCS) was implemented to determine dead load, live load, and resistance according to distributions based on nominal values and statistical parameters.[1–3] As one example of optimal design using the AASHTO LRFD specifications, **Fig. 1** presents probability density functions (PDFs) for Service III with tensile stress limited to $f_t = 0.0948\sqrt{f'_c}$ ($\kappa_{eva}$ = 0.0948) for a 120-ft simple-span bridge designed using AASHTO LRFD specifications published after 2005 (*Post*), $\gamma_L$ = 1.0 with elastic gains from live load included in the precompression stress (*Post-1.0-Gains*).

The mean resistance is close to the nominal resistance for evaluation in Fig. 1. However, the mean load is greater than the mean resistance.

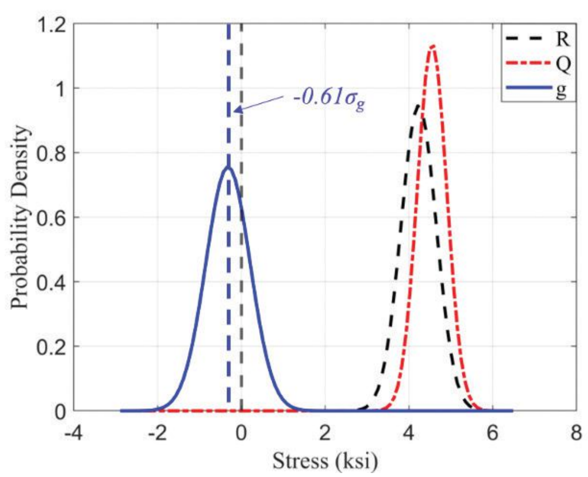



Figure 1. Probability density function for evaluating the Service III limit state at $f_t$ ($\kappa$ = 0.0948) for 120-ft simple-span bridges designed by using *Post-1.0-Gains*. All Figures: University of Nebraska.[1,2]

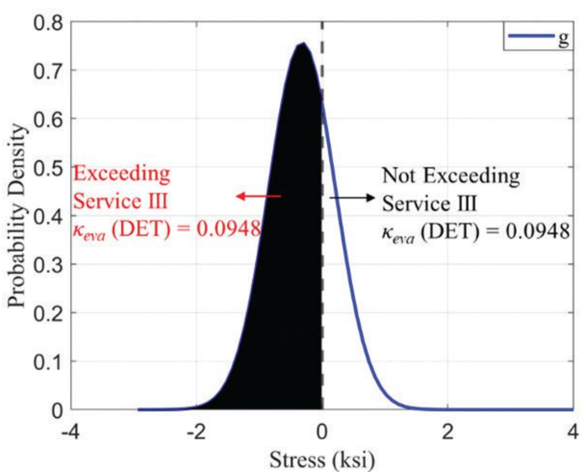



Figure 2. Probability density function showing the probability that Service III $f_t$ ($\kappa_{eva}$ = 0.0948) will be exceeded during the service life for 120-ft simple-span bridges designed by using *Post-1.0-Gains*.

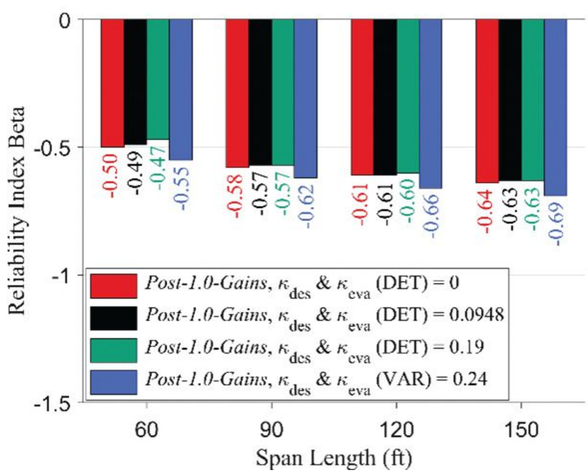



Figure 3. Reliability index β of exceeding tension limits for simple-span bridges designed using the *Post-1.0-Gains* method.

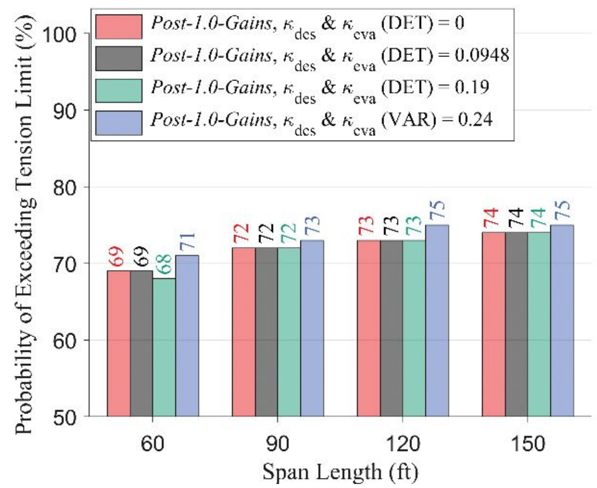



Figure 4. Probabilities of exceeding the tension limit for simple-span bridges designed using the *Post-1.0-Gains* method.

The $\beta_{implicit}$, representing the number of standard deviations between the mean for the limit state function, $g$ = *resistance – load effect* = $R - Q$, and the failure threshold, is $\beta_{implicit} = -0.61$. A negative value indicates that the probability of exceeding the Service III limit state is more than 50%. **Figure 2** further examines the PDF for $g$. Figure 2 depicts the probability that Service III $f_t$ ($\kappa_{eva}$ = 0.0948) will be exceeded during the service life. The figure shows about 72% of the 120-ft simple-span bridges designed with $f_t$ ($\kappa_{eva}$ = 0.0948) are expected to exceed the Service III limit state during their service lives, even though they are optimally proportioned to satisfy Service III design criteria. Note that the computed negative values are a "head-spinner" for reliability experts and were not accepted without careful verification.

**Figures 3** and **4** expand the data to include all span lengths and reasonably represent much more extensive data.[3] Bar colors indicate design and reliability analysis assumptions. "DET" indicates the use of a deterministic value for $f_r$, and "VAR" indicates the use of statistical properties. Observations drawn from Fig. 3 and Fig. 4 include the following:

- $\beta_{implicit}$ was consistently about –0.60 for bridges designed using *Post-1.0-Gains*, was not prominently sensitive to the tension stress limit $f_t$ (κ), and was slightly sensitive to span length. This consistency is expected, as the permitted tensile stresses are the same for both the optimal design and the reliability analysis.
- Figure 4 presents corresponding probabilities of exceeding tension limits. Probabilities are greater than 50% for all cases. The average probability is approximately 73%.
- Considering $f_t$ (κ = 0.0948), the $f_{eval}$ of *Post-1.0-Gains* in Fig. 4 can be used as a reference case because it provided consistent reliability indices regarding span lengths and $f_t$.

## Cracking Probability During Service Life

A negative $\beta_{implicit}$ implies that optimally designed bridges for the Service III limit state are more than 50% likely to violate tensile stress limits under current design live loads at some point during their 75-year service lives. However, exceeding the service limit does not necessarily mean they experience flexural cracking in the precompressed tensile zone. **Figure 5** presents graphs for the likelihood of cracking at modulus of rupture $f_r$ ($\kappa_{eva}$ = 0.24) for a 120-ft simple-span bridge designed using *Post-1.0-Gains*. This 120-ft simple-span bridge is assumed to be designed with $f_t$ (κ = 0.0948).

Figure 5 shows that the mean resistance is slightly larger than the mean load—the corresponding cracking reliability index is +0.08. The total area under the blue "g" line in **Fig. 6** represents all possible outcomes, and the shaded area represents the proportion of cracking cases. The shaded area in Fig. 6 is smaller than that in Fig. 2, reflecting that cracking is less likely than exceeding a limit state set to a tension stress limit less than the theoretical concrete cracking strength. Figure 6 indicates that about 47% of the 120-ft simple-span bridges are expected to crack during their 75-year service lives.

To further evaluate $\beta_{cracking}$ in bridges, bridges designed using a typical $f_t$ (κ = 0.0948) were evaluated over a range of $f_r$ (κ). The moduli of rupture were assumed equal to $0.24\sqrt{f'_c}$, $0.30\sqrt{f'_c}$, and $0.37\sqrt{f'_c}$ to investigate how cracking probability varied. The maximum value considered here is approximately the same as the upper-bound concrete cracking stress specified for minimum reinforcement in Article 5.6.3.3 of the AASHTO LRFD specifications.

**Figure 7** presents the cracking reliability index and corresponding cracking probability for the *Post-1.0-Gains*, *Post-0.8-No-gains*, and *Approx.-0.8-No-gains* methods. These results are typical of present practice. The figure provides results for the typical modulus of rupture $0.24\sqrt{f'_c}$ and the additional cases for moduli of rupture of $0.30\sqrt{f'_c}$ and $0.37\sqrt{f'_c}$. The cracking probability graphs show that the higher, and perhaps more likely, $f_t$ decreases the probability of cracking significantly, perhaps to a level more closely matching in-service observations and inspections. However, it is important to keep in mind that the higher level of cracking stress in Article 5.6.3.3 of the AASHTO LRFD specifications reflects the upper tail of a concrete cracking strength probability distribution, so using this value as the mean concrete cracking strength is not strictly consistent with test data. On the other hand, using the higher value can be considered to approximately represent increasing concrete strength over time.

## Summary of Key Points

We observed the following key points from our reliability study. Note that only limited data are provided here. Please see Steelman et al.[1,2] for expanded results.

- $\beta_{implicit}$ values of −0.60 and −1.20 imply a 73% to 88% probability of exceeding the Service III limit during service life.
- $\beta_{implicit}$ was approximately −1.20 when averaged across all considered span lengths for bridges designed using the following loss methods:
  - Post-2005 loss method with elastic gains and using $\gamma_L = 0.8$
  - Approximate loss method with elastic gains and using $\gamma_L = 0.8$
- A target of $\beta_{implicit} = -0.60$ is recommended to evaluate girder bridges for platoon loading at Service III. A more liberal value of −1.20 may be acceptable if it is supported by observations of adequate performance in service for girders designed considering elastic gains and using $\gamma_L = 0.8$.
- Bridge cracking probabilities range between 10% and 51% when bridges are designed using the code-specified service load, indicating that optimally designed bridges may crack during their service life, despite the nominal expectation that cracking will be avoided by using tensile stress limits less than the modulus of rupture. A nominal change of $f_r$ ($\kappa = 0.24$) to $f_r$ ($\kappa = 0.37$) produces an average increase of $\beta$ equal to 0.64 and an average decrease in cracking probability of 22%. $\beta_{Cracking}$ and cracking probability change approximately linearly as a function of nominal moduli of rupture and can be preliminarily estimated using an assumed value.
- Bridges designed using the Pre-2005 loss method (Article 9.16.2.1 of the *AASHTO Standard Specifications for Highway Bridges*[5]) without elastic gains and using $\gamma_L = 1.0$ possessed the highest cracking reliabilities, which indicates that heavy platoon loads pose lower risks to these bridges than to other bridges designed using more-recent prestress loss estimation methods.

## Negative Reliability?

The Service III limit state has not been formally calibrated and is based on decades of experience. Strength limit states have much higher reliability indices and lower exceedance probabilities. However, an open question remains about the likelihood of exceeding the service limits using the same rules as the governing design, which we determined is likely. We also determined that the probability that the girders would crack during the service life is lower but also likely.

Although it may seem odd at first, negative reliability is just a number that predicts a likelihood of occurrence. Over 75 years, concrete may crack, pretension closes the cracks, and reasonable performance is likely observed. So, how often can this cycle occur without adverse effects if we permit platoons (or any other heavier loads)? This is an open question. Another item for discussion is whether evaluation load levels should be based on shorter return periods, such as two years, or another period based on the inspection frequency, actual weigh-in-motion data, long-term strength increases (bias), and other factors. Using such assumptions will lower the probability of exceeding the limit state or modulus of rupture.

## Takeaway

The probability that a prestressed concrete girder will crack during its service life seems to be relatively high; however, little work that explores the repeated load effect between cracking and yielding of reinforcement has been done. A better understanding of the effect of heavier loading due to possible platoon permits brings us back to the moment-curvature relationship and repeated loading presented in the first article in this series (*ASPIRE* Winter 2024).

Service III typically controls the required number of strands in a design. The findings explored in this series raise questions about whether it is time to rethink the Service III design process related to concrete cracking, and/or a new limit state related to a percentage of strand yielding.

**Figure 5. Probability density functions for evaluating cracking at modulus of rupture $f_r$ ($\kappa = 0.24$) for 120-ft simple-span bridges designed using the *Post-1.0-Gains* method.**

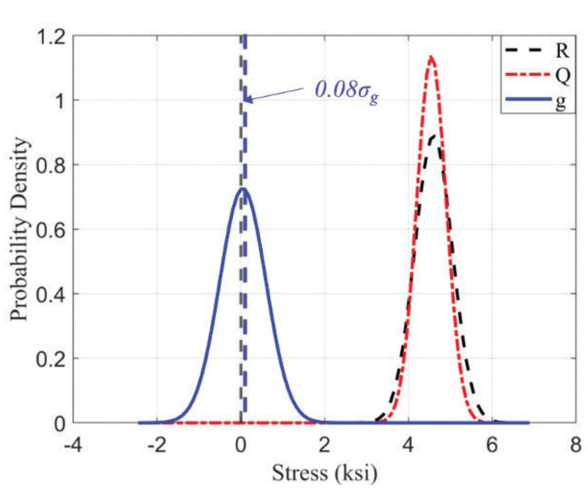
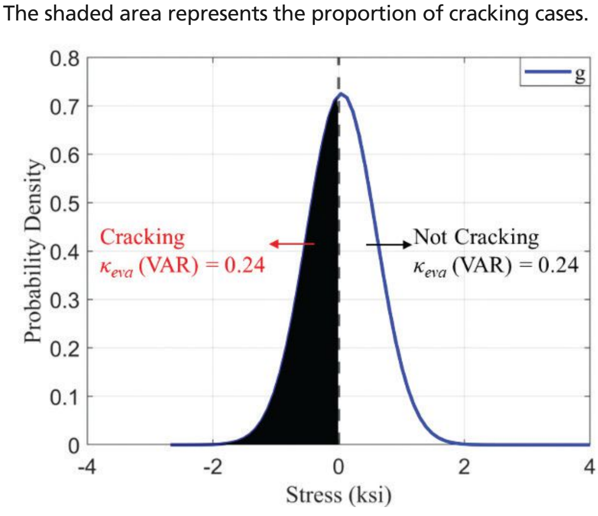



**Figure 6. Probability density function for the likelihood of cracking at modulus of rupture $f_r$ ($\kappa = 0.24$) for a 120-ft simple-span bridge designed using *Post-1.0-Gains*, representing all possible outcomes. The shaded area represents the proportion of cracking cases.**



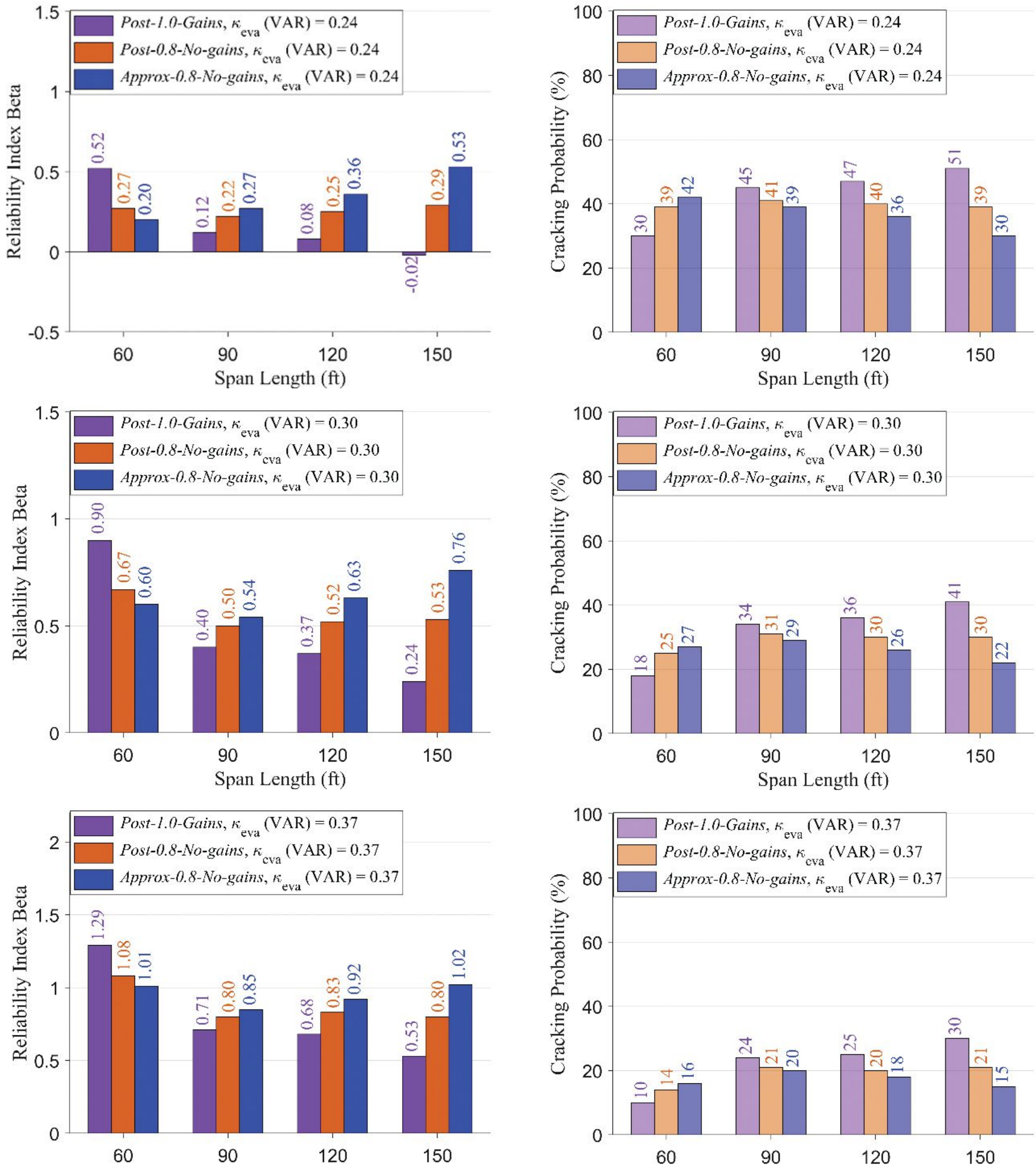



Figure 7. Reliability index $\beta_{cracking}$ and cracking probabilities for simple-span bridges designed using *Post-1.0-Gains*, *Post-0.8-No-gains*, and *Approx.-0.8-No-gains* methods and different values for $f_r$ (κ = 0.24, 0.30, and 0.37).

## References


1. Steelman, J. S., J. A. Puckett, D. G. Linzell, and B. Yang. 2021. *Truck Platooning Effects on Girder Bridges*. Final report no. SPR-1(20) M030. Lincoln: Nebraska Department of Transportation.
2. Steelman, J. S., J. A. Puckett, D. G. Linzell, and B. Yang. 2023. *Truck Platooning Effects on Girder Bridges*: Phase II. Final report no. SPR-FY22(011). Lincoln: Nebraska Department of Transportation.
3. Yang, B., J. S. Steelman, J. A. Puckett, and D. G. Linzell. 2024. "Reliability-Based Service III Evaluation for Prestressed Girder Bridges Under Platoon Loads" *Transportation Research Record*. 2678 (7): 515–536. https://doi.org/10.1177/03611981231208200.
4. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
5. AASHTO. 2002. *AASHTO Standard Specifications for Highway Bridges*. 17th ed. Washington, DC: AASHTO.