Example Factor Calibration Calculations for Platoon-Permit Loads

by Dr. Jay Puckett and Dr. Joshua Steelman, University of Nebraska

In the Winter 2024 issue of *ASPIRE®*, we introduced a potential strategy based on live-load factor calibration and reliability principles to allow truck platoons to increase truck weights safely. The premise of this work is that trucks are and will become smarter and will achieve the ability to drive long distances autonomously. With such intelligence, these trucks will likely be able to report their axle weights and spacings and control their relative headways.

This article uses an example of a simple-span, concrete T-beam bridge with conventional reinforcment to review how computations might be used to support platoon-permit loads being larger than legal loads. A forthcoming article will expand these concepts to the more complex case of pretensioned concrete girder bridges, and the details related to structural reliability.

Example Bridge

Figure 1 shows a typical six-girder, conventionally reinforced concrete T-beam bridge and gives the cross-section properties for a single girder with the composite concrete deck. This bridge was selected to provide computations that are easy to follow. For more details, refer to Barker and Puckett's[1] discussion of a similar bridge.

The bridge is statically determinate. The load factors and live-load distribution factors are in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications;*[2] self-weight and future wearing-surface loads are considered, but the barrier weight was neglected for simplicity. For the sake of brevity, fatigue and shear are not considered in this example.

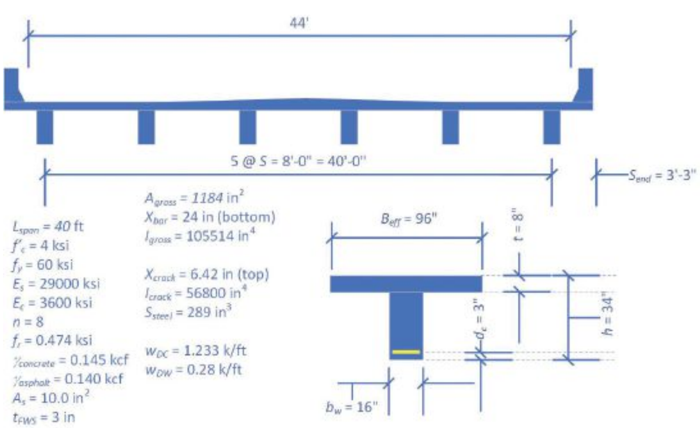

Figure 1. Example bridge. All Figures: University of Nebraska.

Table 1 shows the calculated flexural moments used in the Service I and Strength I limit states listed in **Table 2**, as well as the calculated cracking-moment check. The usual stress-block approach is used to determine the moment capacity ϕM_n. The example design used an area of reinforcing steel to exactly satisfy the Strength I limit state; that is, the performance ratio *PR* = 1 for Strength I (*PR* ≤ 1 indicates that the requirements of the limit state are met). The Service I limit state has a *PR* of approximately 1.0 using a resistance limit of 0.6 f_y. However, the analysis illustrates that the section is expected to crack under the design service load.

The flexural strength M_n is 1504 ft-kip. A quick sensitivity analysis demonstrates the effects of f'_c and f_y on this computation. If f'_c increases from 4 ksi to 6 ksi (that is, 1.5 times), M_n increases by 1.01 times to 1519 ft-kips. However, if f_y increases by 1.5 times to 90 ksi, M_n increases by 1.48 times to 2222 ft-kips. Although this behavior is well known, it is important to note that M_n varies almost linearly with A_s or f_y. From a practitioner's perspective, both A_s and f_y are perceived as deterministic because uncertainties have been integrated into load and resistance factors. In reality, f_y is significantly more influential than A_s on the reliability index β, because fabrication

Table 1. Flexural moments

Parameter	Example	Comment
M_u, ft-kip	1360	$1.25M_{DC} + 1.5\ M_{DW} + 1.75M_{distIM}$
$M_{service}$, ft-kip	856	$1.0M_{DC} + 1.0M_{DW} + 1.0M_{distIM}$

Table 2. Limit-state and cracking checks

Parameter	Example	Comment
Service I		Very close to optimal design
f_s, ksi	35.6	Sum of steel stresses due to all loadings
f_{allow}, ksi	36	$0.6f_y$
Performance ratio, *PR*	0.99	*PR* = 35.6/36 ≤ 1; OK
Strength I		Optimally designed
M_n, ft-kip	1504	*T* × (lever arm)
ϕM_n, ft-kip	1354	$\phi = 0.9$
PR	1.0	*PR* = 1360/1354 = 1; OK
Cracking		
M_{cr}, ft-kip	174	$f_r \times I_{gross}/X_{bar}$
$1.2M_{cr}$, ft-kip	209	$\phi M_n \geq 1.2M_{cr}$; OK
PR	4.1 > 1	*PR* = 856/209 = 4.1; definitely cracks

tolerances result in reinforcing steel bar areas with high accuracy (bias λ practically equal to 1) and high consistency (coefficient of variation *V* close to 0). On the other hand, actual steel yield strength is routinely higher than the nominal value used to calculate strength, and the variability of yield strength from one bar to the next can be greater than the variability of the area because bars may be produced from different heat numbers or mills.

Reliability Analysis

In terms of statistics, all design parameters have bias λ and coefficient of variation *V*. Bias factors are ratios of nominal values to expected mean values. For example, if samples of reinforcing bar with a nominal f_y of 60 ksi are taken from the field to a lab and subjected to tension testing, the samples would likely have a mean of approximately $\lambda(f_y) = 1.14(60) = 68.4$ ksi. The standard deviation σ_R is determined from the product of the mean and V_R, as $V_R(68.4) = 0.08(68.4) = 5.47$ ksi. **Table 3** provides the statistical data used in this example. Nowak and Collins[3] provide a general discussion of structural reliability.

The following equations calculate the mean values for Strength I resistance and load effects. Coefficients of variation scale these mean values to determine standard deviations.

$$\bar{R} = \lambda_R R_n = 1.14(1504) = 1715 \qquad \sigma_R = V_R \bar{R} = 0.08(1715) = 137.2$$

$$\bar{Q}_{DC} = \lambda_{DC} Q_{DCn} = 1.05(247) = 259 \qquad \sigma_{DC} = V_{DC} \bar{R} = 0.1(259) = 25.9$$

$$\bar{Q}_{DW} = \lambda_{DW} Q_{DWn} = 1.0(56) = 56 \qquad \sigma_{DW} = V_{DW} \bar{R} = 0.25(56) = 14.0$$

$$\bar{Q}_{LL} = \lambda_{LL} Q_{LLn} = 1.2(553) = 664 \qquad \sigma_{LL} = V_{LL} \bar{R} = 0.2(664) = 132.8$$

The limit-state function is defined as:

$$g(R,Q) = \bar{R} - \bar{Q}$$

If $g(R,Q)$ is positive, the limit state is met. For simplicity, normal probability distributions are assumed, and β values are computed as follows.

$$\beta_{Strength\,I} = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{1715 - 259 - 56 - 664}{\sqrt{137.2^2 + 25.9^2 + 14.0^2 + 132.8^2}} = 3.81$$

Figure 2 illustrates the normal distributions for the load, resistance, and limit state function. β is the number of standard deviations the average of the limit state function is away from "failure," in this case, 3.81 times. The shaded area is the probability of not meeting the limit state, in this case, about 0.06×10^{-3}.

The Service I limit state addresses crack control in the AASHTO LRFD specifications Article 5.6.7, where steel stress is limited to $0.6f_y$, which is used here to represent Service I.

$$\beta_{Service\,I} = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{0.6(60)(1.14) - 10.3(1.05) - 2.33(1.0) - 23.0(1.2)}{\sqrt{3.28^2 + 1.08^2 + 0.58^2 + 5.52^2}} \approx 0$$

Another possible limit state of interest is the yield limit, or a fraction of yield to be used for rating. Using similar computations as shown for Strength I and Service I, and assuming f_y is the resistance with service load effects, β is 3.56 for the yield limit. Note that β_{Yield} is similar to $\beta_{Strength\,I}$, as

Table 3. Statistical properties of primary variables used in this example

	Bias λ	Coefficient of variation *V*	Resistance factor or load factor
Flexural resistance *R**	1.14	0.08	$\phi = 0.9$ (Strength I)
Component self-weight *DC*†	1.05	0.10	$\gamma_{DC} = 1.25$ (Strength I) $\gamma_{DC} = 1.00$ (Service I)
Wearing surface dead load *DW*‡	1.0	0.25	$\gamma_{DW} = 1.25$ (Strength I) $\gamma_{Dw} = 1.25$ (Service I)
Live load *LL*§	1.2	0.1 to 0.20	$\gamma_{LL} = 1.75$ (Strength I) $\gamma_{LL} = 1.00$ (Service I)

*Mostly dependent on f_y statistics.
†Cast-in-place concrete self-weight.
‡Wearing surface (asphalt).
§Dynamic load effect, live-load distribution, and weights/spacings are combined into one parameter here.

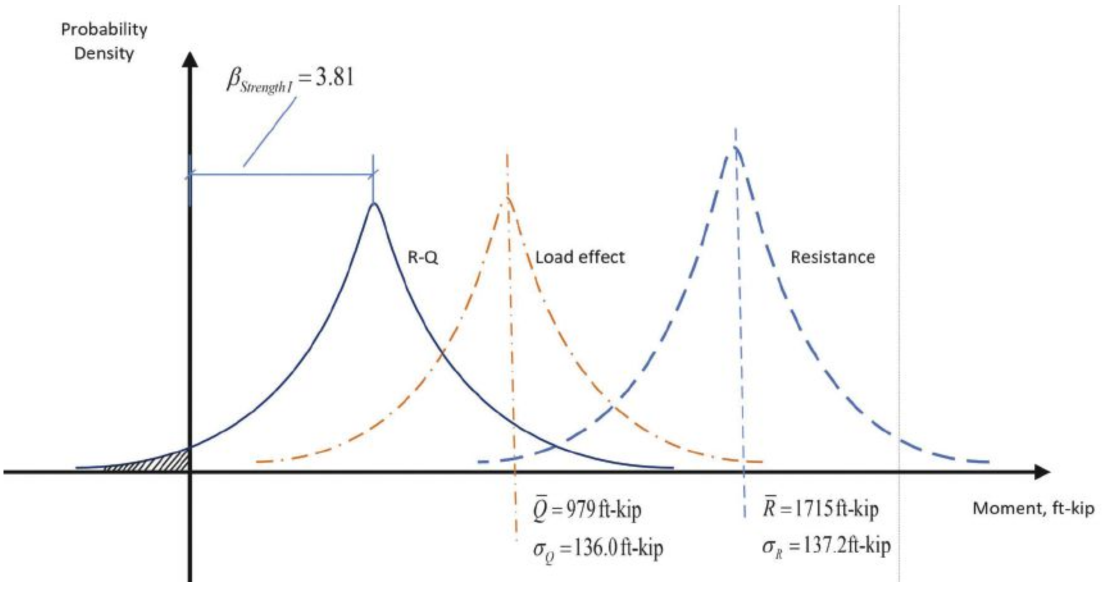

Figure 2. Normal distributions of load, resistance, and limit state functions.

expected, because the moment capacities are approximately the same.

Cracking is not a formal limit state and is provided for insight into the bridge's expected behavior during the 75-year service life. Using the same method again and assuming $\lambda = 1.1$ and $V_R = 0.1$ for M_{cr}, $\beta_{Cracking}$ is –5.6; that is, cracking is expected, which is an obvious result because even self-weight loads exceed any estimate of the cracking moment.

Parametric Study

Designs were performed for T-beam bridges with varied spacings S = 6, 8, 10, and 12 ft; span lengths L_{span} = 35, 40, 45, and 50 ft; and girder depths $h = 0.07 \times L_{span}$ (rounded up) = 30, 34, 38, and, for a deeper case, 50 in. The design was optimized for each case; that is, the performance ratio for Strength I was set to unity. The β values were almost uniform, with a minimum of 3.74 and a maximum of 3.96. This indicates that the calibration is meeting its aim (load and resistance factors are providing uniform reliability) for this class of bridge type for typical geometry and material properties. Further study regarding live-load (platoons) statistical parameters can be readily conducted with a typical structure, as β computations are insensitive to L_{span} and S.

Platoon Example

Assume that a platoon operator can measure and report axle weights and spacings while operating. Consequently, the platoon live-load bias and variance can be lower than assumed for routine operations. **Table 4** provides β values for a range of assumed bias and variance values. Note that because the live load is a function of transverse load distribution, dynamic load allowance, and weight characteristics, λ and V_{LL} cannot be driven to 1 and 0, respectively. The case where $\lambda = 1.1$ with V_{LL} between 0 and 0.1 is a realistic range with β values of 5.2 and 5.6, respectively. Other values provide bounds.

Table 4. Reliability index β with various live-load statistics

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	3.8	4.3	4.8
	0.1	4.8	**5.2**	5.6
	0	5.3	**5.6**	6.0

Table 5. Potential factor to apply to live load to maintain β = 3.5 (inventory)

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	1.00	1.10	1.20
	0.1	1.20	**1.30**	1.40
	0	1.30	**1.45**	1.60

Table 6. Potential factor to apply to live load to maintain β = 2.5 (operating)

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	1.30	1.40	1.55
	0.1	1.45	**1.60**	1.75
	0	1.60	**1.70**	1.90

Because β values increase with lower bias and variance, the opportunity exists to increase platoon live loads to move to a typical design target of β = 3.5, or to a typical operating target of β = 2.5.[4]

Table 5 provides the available increase in live load to maintain β = 3.5. Again, the practical ranges of bias and variance correspond to live-load increases of 30% to 45%.

In AASHTO load rating, the operating level for load rating targets β = 2.5. **Table 6** provides factors for increasing live load to maintain β = 2.5. Again, note the highlighted values.

Discussion

The example and associated computations for a single bridge type and geometry demonstrate the potential to offer a new permit-use case, a platoon permit. The load increases would depend on the operators' ability to invest in technologies to drive the λ and V_{LL} downward in a consistent and likely, reportable manner.

To summarize, when the $\overline{Q}$ and σ_Q are driven downward with better technology to $\overline{Q}_{platoon}$ and $\sigma_{Q\_platoon}$, these changes increase β. Therefore, the nominal platoon live load could be increased to maintain a constant β = 3.85, or to target a design β = 3.5 or an operating β = 2.5, as desired for various operational strategies.

$$\beta_{StrengthI} = \frac{\overline{R} - \overline{Q}_{platoon}}{\sqrt{\sigma_R^2 + \sigma_{Q\_platoon}^2}}$$

The β values for the yield limit computed for the various geometries varies a little around β = 3.5. This assumes the full yield capacity of the reinforcing steel. If the steel stress is limited to $0.6f_y$, for the example bridge, β is approximately zero for the 75-year design life, indicating a 50% chance of exceedance.

Next Article

This example and discussion provide a relatively simple bridge system to demonstrate reliability computations for a conventionally reinforced concrete bridge. A forthcoming article aims to unpack some of the complexities associated with pretensioned concrete bridge girders and evaluation for platoon operations. For a pretensioned concrete girder, the design (number of strands, prestress, and eccentricity) is more complex, involving different loss methods, different live-load factors, gross or transformed section properties, allowable tension, and design specifications that have changed over time (and are still changing).

References

1. Barker, R. M., and J. A. Puckett. 2021. *Design of Highway Bridges: An LRFD Approach*, 4th ed. New York, NY: Wiley.
2. American Association of State Highway Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications.* 9th ed. Washington, DC: AASHTO.
3. Nowak, A. S., and K. R. Collins. 2012. *Reliability of Structures*. New York, NY: CRC Press.
4. AASHTO. 2018. *The Manual for Bridge Evaluation.* 3rd ed. Washington, DC: AASHTO.