Operational Strategies for Truck Platoon Permits—Effective Use of Prestressed Concrete Girders

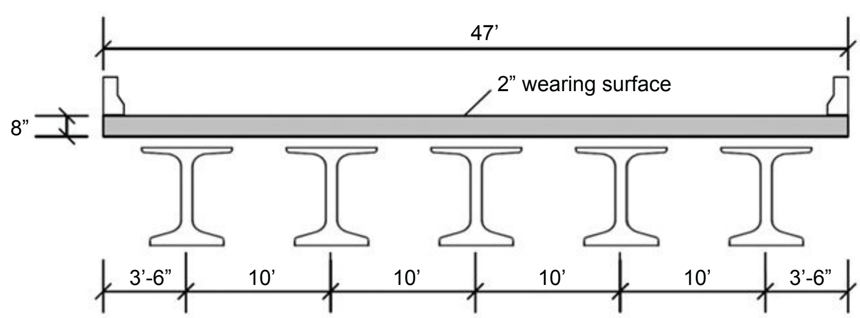
by Dr. Jay Puckett and Dr. Joshua Steelman, University of Nebraska

We recently studied effective strategies for truck platoon operations for strength, service, and fatigue limit states. (For more information about truck platoons and their effect on bridges, see the FHWA article in the Summer 2019 issue of *ASPIRE*®.) The premise of this work is that some trucks are—or will become—"smart," with the ability to drive long distances autonomously. With such intelligence, these trucks can likely report their axle weights and spacings as well as control their relative headway distances. Given such controls, the traditional statistics used to calibrate live-load factors become better known. For example, the truck live-load variability is much smaller than the variabilities used for typical permitting or design. Dynamic headway controls can also position trucks to maintain safety and service performance thresholds as platoons cross bridges of varying spans along a route. Our recent studies are one of the first efforts to study truck platooning from a reliability perspective.[1,2]

If and when a new permit process that allows larger live loads than current legal loads is created, new calibration using reliability indices β that are reasonably based on current practice and bridge performance will be required. For example, the strength limit for design is typically targeted to be $\beta = 3.5$ (0.023% probability of exceeding the limit). For load rating, $\beta = 2.5$ (0.62% probability of exceeding the limit) is often used for strength. To date, limited studies have been performed on calibrating load factors for service and fatigue limit states.

To develop a platooning permit strategy, the service and fatigue limit states must be addressed to enable larger loads and minimize damage to concrete components. For prestressed concrete girder bridge design, prestressing strands are selected to limit concrete tensile stresses, with stress limits depending on the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[3] the state transportation agency's standards, and/or the owner's practice. For permit ratings, some owners might limit tensile stresses to zero (bottom fibers always in compression). In contrast, others might allow $0.19\sqrt{f'_c}$, where f'_c is the design concrete compressive strength in ksi, similar to the Service III limit state in the AASHTO LRFD specifications. Performance assessment computations are complicated by various assumptions either adopted by agencies or left to the discretion of individual engineers, such as prestress loss calculation methods, use of gross or transformed section analysis, consideration or neglect of elastic gains, and live-load factor selection. Furthermore, computations for the same girders designed with different assumptions would yield different numbers of strands, which provide various resistances to cracking and potential damage due to repetitive loads from heavy platoons.

The aforementioned assumptions complicate the live-load factor calibration process for platoon permitting by affecting resistance computations. For example, should permit computations use zero or $0.19\sqrt{f'_c}$ for the tensile-stress limit?



Design Parameters
- 120-ft simple-span bridge
- Optimally design for tensile stress limit = $0.0948\sqrt{f'_c}$
- Optimal design = exact number of strands for Service III limit state
- Refined losses with elastic gain
- Design live load factor $\gamma_{LL} = 1.0$
- NU 1600 girder
- Haunch = 1 in.

Prestress Parameters
- 270 ksi low-relaxation strand
- $f_{py} = 243$ ksi

Concrete Parameters
- $f'_{c\_girder} = 8$ ksi
- $f'_{c\_deck} = 4$ ksi

Figure 1. Example prestressed concrete bridge. All Figures: University of Nebraska–Lincoln.

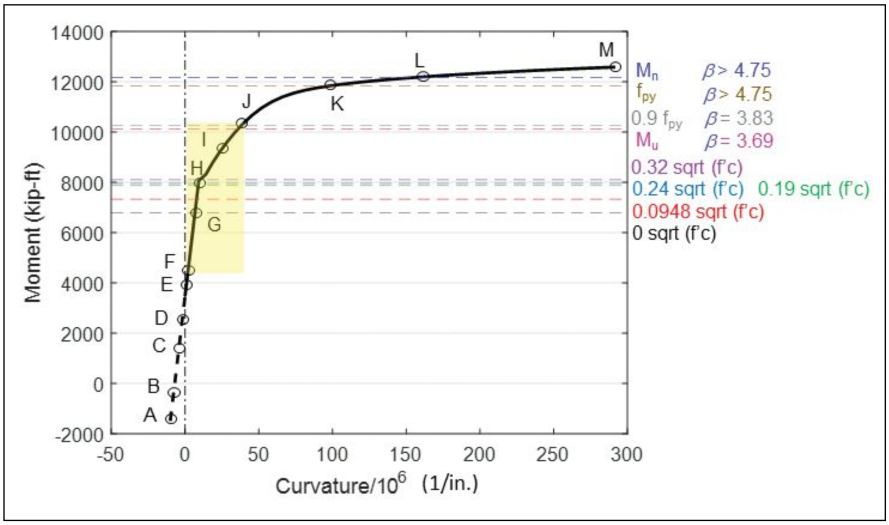


Figure 2. Moment-curvature diagram of the example prestressed concrete girder.

Should permit computations allow tensile stress up to the full modulus of rupture ($0.24\sqrt{f'_c}$) cracking strength? If so, should the average statistical value or the higher value be used for estimating cracking moment? The Service II limit state for steel girders uses an allowable stress $f_{allowable}$ of $0.95F_y$ (where F_y is the yield stress of steel) for composite sections, which anticipates permanent deformations due to partial plastification with residual stresses. Should concrete girders use similar limits, considering that recompression can still be achieved even after some degree of prestressing strand yielding?

So, the question arises: Are the traditional methods for evaluating prestressed concrete girder service performance suitable for effectively using the bridge inventory for innovative strategies such as platoons?

Basic Mechanics

When considering difficult engineering problems, it is often best to return to mechanics for understanding, or at least as a framework. Let's start with an example.

Consider the prestressed concrete girder bridge shown in **Fig. 1**. The girders were optimally designed for HL-93 loading using tensile stress limits from the AASHTO LRFD specifications' Service III limit state. **Figure 2** shows the girder moment-curvature (M-ϕ) diagram.

The M-ϕ diagram best represents the behavior of flexural components (or beam-columns) from the transfer of prestress to the strength limit. Items that influence flexural behavior, such as gross cross-section geometry, elastic gains, cracking, yielding, and tension stiffening, are all included in Fig. 2 as the diagram tracks the mechanistic behavior. This method is used in many research areas and advanced analysis in performance-based seismic design. Numerical and experimental data comparisons have demonstrated excellent agreement over various load levels. Example reliability indices are shown for the 75-year design life. The reliability index at the strength level can be confidently determined from common methods and assumptions used in bridge reliability assessment and calibration. At the Service III limit state considered for this design (noted in Fig. 1), the reliability index is an evolving topic that will be the focus of future *ASPIRE* perspectives.

In Fig. 2, the M-ϕ diagram "starts" at point A, which is at transfer of prestress, moving next to include girder self-weight. Then after the composite deck has been placed and cured, the superimposed dead load and any potential wearing surface loads are applied, and finally superimposed live load is applied (at point F).

Information presented in the yellow-shaded box in Fig. 2 includes limiting tensile-stress formulations from relevant specifications that have been used in the related research activities.[3,4] They consider various levels of permitted tension stresses such as zero tension, $0.0948\sqrt{f'_c}$, or $0.19\sqrt{f'_c}$ (which are traditional limits for various levels of environmental exposure and potential corrosion), and $0.24\sqrt{f'_c}$, which is associated with the modulus of rupture (that is, the theoretical cracking limit). The yellow box also contains information demonstrating that under increased live load (at point I), the section moves toward a high fraction of yield such as $0.9f_{py}$, which is used

Table 1. Moment-curvature diagram operational points of interest

Operational Point	
A	Transfer of prestress excluding girder self-weight (A and B occur simultaneously)
B	Transfer of prestress including girder self-weight
C	Deck is placed, not hardened, slight camber
C	Deck is hardened and slope changes to composite section
D	Composite dead load of components (DC stage II) is placed; perhaps dead load of utilities and wearing surfaces (DW) are as well; all small load effects
E	Zero curvature, only P_e/A_g stress
F	Live load is applied
G	Zero tension in bottom fiber
H	Section cracks
I	Tension increases, neutral axis rises, curvature increases, concrete behavior is nonlinear
J	Strand stress at $0.9f_{py}$, the AASHTO *Manual for Bridge Evaluation*[4] limit state per Article 6A.5.4.2.2b
K	Yield of prestressing steel in bottom layer
L	Code-based nominal moment M_n, at concrete strain ε_c of 0.003
M	Actual plastic moment M_p

Source: University of Nebraska–Lincoln.
Note: A_g = gross area of the concrete section; e = strand eccentricity; f_{py} = yield strength of prestressing strands; P = prestressing force after prestress losses.

in Article 6A.5.4.2.2b of AASHTO's *Manual for Bridge Evaluation*.[4] There is a significant range of operational resistances that might be rationally adopted to effectively use bridge inventories for platoon loading.

Conclusion

The history of taking a stress limitation approach is long, but that approach may impose unnecessary limitations on platooning operations. Thus, platoons are a prominent motivation for better understanding heavy loads on prestressed concrete girders. Perhaps a more rigorous computation approach, based on mechanics, could be used to better understand the performance of girders operating with permits that allow repeated platoon travels. This future activity should involve deterioration and fatigue modeling coupled with economic modeling that can be linked to various operational strategies associated with bridge service life.

In future articles, we hope to expand to reliability indices associated with different load levels and the target reliability indices associated with various design assumptions mentioned in this article. Platooning was the impetus for our research, but the findings will also help guide design more generally.

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 (NDOT). https://dot.nebraska.gov/media/vvalloir/spr-1-20-m030-truck-platooning-effects-on-girder-bridges-final-report.pdf.
2. Steelman, J. S., J. A. Puckett, D. G. Linzell, and B. Yang. 2023. *Truck Platooning Effects on Girder Bridges: Phase II—Service*. Final Report no. SPR-FY22(011). Lincoln, Neb.: NDOT https://dot.nebraska.gov/media/0ese55i1/spr-fy22-011-final-report-ver2.pdf.
3. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
4. AASHTO. 2018. *Manual for Bridge Evaluation*. 3rd ed. Washington, DC: AASHTO.

Dr. Jay Puckett is a professor in the Durham School for Architectural Engineering and Construction at the University of Nebraska–Lincoln. He has conducted bridge engineering research and software development for four decades and is the author of several NCHRP reports.

Dr. Joshua Steelman is an associate professor in the department of Civil and Environmental Engineering at the University of Nebraska–Lincoln. His current work focuses on bridge design, evaluation, and management, as well as in crash safety for roadside hardware.