AASHTO LRFD Bridge Design Specifications: Updates to AASHTO's *Manual for Bridge Evaluation*

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This article provides an overview of one agenda item that updates the American Association of State Highway and Transportation Officials' *Manual for Bridge Evaluation* (AASHTO MBE)[1] for load rating of segmental bridges. The "Perspectives on Structural Behavior and Redundancy" series of articles, published in *ASPIRE*® in 2021, serves as a primer to the changes in this working agenda item that was approved in June 2025. The approved changes to the AASHTO MBE explain the logic behind load ratings of segmental bridges, help ensure the safety of such bridges, and give due consideration to the cost-effectiveness of the engineering solutions. With that stated, let's focus on the newly approved 15-part working agenda item for the upcoming changes in the AASHTO MBE.

Part 1

The consideration of the temperature gradient *TG* when evaluating segmental bridges is clearly explained by replacing the second paragraph in Article C6A.2.3.6, as follows:

For segmental concrete box girder bridges, TU *and* TG *shall be considered at the service limit state at the design-load inventory level. The corresponding load factors,* γ_{TU} *and* γ_{TG}*, shall be taken as 1.0 and 0.5, respectively, when live load is considered.*

Part 2

Commentary language is added to Article C6A.2.3.6 to clarify intent, as follows:

> *Corven Engineering (2004)*[2] *and Popok et al. (2024)*[3] *have established that when assessing bridge designs for Strength Limit State, the impact of thermal gradients need not be considered. Inventory Ratings, which are as reliable as new designs, should account for thermal gradients at the Service Limit State as per LRFD guidelines.*[4] *However, the likelihood of the maximum live load and the peak thermal gradient occurring at the same time is minimal. Therefore, during Inventory Ratings, only half the thermal gradient value (0.5*TG*) is considered alongside the design live load at service. Operating Ratings exclude thermal gradient effects at both Service and Strength Limit States due to their negligible influence and the minor consequences of limit exceedance.*
>
> *Typically, the longitudinal expansion and contraction in concrete bridges are managed by sliding or flexible bearings, which minimally affect the superstructure. However, when superstructures are fixed to substructures, the forces from thermal expansion and contraction* (TU) *must be taken into account. These forces are only factored in at the Service Limit State for Inventory Ratings, with a factor of* γ_{TU} *= 1.0, as including them at the Strength Limit State does not substantially alter the core reliability of the structure and could lead to unnecessary work. Similarly, for Operating Ratings, the effects of thermal expansion and contraction are considered negligible, and the repercussions of surpassing the limit are not significant.*

Part 3

To provide clarity, the following language is added to Article 6A.4.2.2:

> *In Table 6A.4.2.2-2, the application of the live load factor of 1.0 does not extend to segmental concrete box girder bridges, designed using gross sections and utilizing time-step methods in the time-dependent prestress loss calculations rather than the refined estimates as specified in LRFD Design Article 5.9.3.4. The live load factor of 0.8 should be applied for Service III load combination for segmental concrete box girder bridges at the design-load inventory level.*

Part 4

Commentary to Article 6A.4.2.2 (C6A.4.2.2) is expanded to include the following explanation:

> *For segmental concrete box girder bridges, a live load factor of 0.8 at the design-load inventory level is calibrated for the Service III limit state to achieve a target reliability index,* β_T*, of 1.0, corresponding to a one-year return period (Popok et al., 2024).*[3]

Part 5

The second row of Table 6A.4.2.2-2, "Load Factors for Live Load for the Service III Load Combination, γ_{LL}, at the Design-Load Inventory Level," is revised to read:

Prestressed concrete components rated using the refined estimates of time-dependent losses as specified in LRFD Design Article 5.9.3.4 in conjunction with taking advantage of the elastic gain	1.0

Part 6

The third row of Table 6A.4.2.2-2 is revised to read:

All other prestressed concrete components, including segmental concrete box girders	0.8

Part 7

Article 6A.5.11.3 is revised to read:

For the Multiple Presence Factors [MPFs] in segmental concrete bridges, the following table is recommended for spans up to 400 ft.

Table 6A.5.11.3—Multiple Presence Factors for Segmental Bridges

Number of Loaded Lanes	MPF
1	1.20
2	1.00
3	0.75
4 or more	0.60

For the transverse operating load ratings of the top slab of segmental concrete box girders, the factor of 1.20 specified in Table 6A.5.11. for one loaded lane shall be limited to a maximum of 1.00.

Part 8

Article C6A.5.11.3 is expanded by adding the explanation below:

The multiple presence factors derived for segmental concrete bridges are based on actual WIM [weigh-in-motion] *data which has at least hundredth of a second timestamp. Truck traffic simulations and probabilistic modeling are used to capture the load effects created by the cross-lane multiple presence events. It is found that the MPFs are affected by two major factors: truck loads and probability of multiple presence events. Truck loads can be further defined as the mean and standard deviation of loads on each lane, and the covariance of the load effects across the lanes. The probability of multiple presence events is related to average daily truck traffic, and the distribution of the traffic (Lou et al., 2023).*[5] *For spans up to 400 ft, the current AASHTO LRFD is reasonably conservative for the single-lane and two-lane factor, while three-lane and four-or-more-lane factors are decreased to account for the low probability of side-by-side events based on WIM data.*

Part 9

Article 6A.5.11.4 is revised as follows:

The Strength I and both the Service I and the Service III limit states shall be checked for the design-load rating of segmental concrete bridges. For the Service III limit state, the number of live load lanes shall be taken as the number of design lanes for both inventory and operating ratings. The live load factor of 0.65 shall be used for operating rating. For segmental concrete bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.9.2.3.3.

To offer clear guidance, Article 6A.5.11.5.1 is revised to read:

Both the Service I and Service III limit states are mandatory for legal load rating of segmental concrete box girder bridges. For the Service III limit state, the number of live load lanes shall be taken as the number of design lanes, and the live load factor of 0.65 shall be used. For segmental concrete box girder bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.9.2.3.3.

Article 6A.5.11.5.2 is revised as follows:

Both the Service I and Service III limit states are mandatory for permit load rating of segmental concrete box girder bridges. For the Service III limit state, the number of live load lanes shall be taken as the number of design lanes, and the live load factor of 0.65 shall be used. For segmental concrete box girder bridges, the Service III limit state specifically includes the principal tensile stress check of LRFD Design Article 5.9.2.3.3.

Part 10

The Commentary Article C6A.5.11.4 is revised to read:

The principal tensile stress check is necessary to verify the adequacy of webs of segmental box girder bridges for longitudinal shear and torsion.

The use of a live load factor of 0.65 for operating rating derived from the calibration of the service limit states conducted by Popok et al. (2024)[3] *and distinguishes the operating rating from the inventory rating, where a live load factor of 0.8 is appropriately used. The lesser load effects resulting from the reduced live load factor for the operating rating acknowledges a lower target reliability index for operating rating as opposed to inventory.*

The strength limit states are calibrated to achieve target reliabilities, β_T, of 3.5 and 2.5 for inventory and operating evaluation levels, respectively. For the Service III limit state, the live load factors of 0.8 for inventory and 0.65 for operating rating evaluation levels are calibrated to achieve target reliabilities, β_T, of 1.0 and 0, respectively, for the return period of one year (Popok et al., 2024).[3]

Part 11

A new article, 6A.5.11.5.3—Stress Limits for Concrete, is added as follows:

The limits in Table 6A.5.11.5.3 shall apply for compressive and tensile stresses at the Inventory and Operating Ratings in segmentally constructed bridges.

Table 6A.5.11.5.3—Stress Limits in Concrete at the Inventory and Operating Ratings for Segmental Bridges			
At the Service Limit State after losses	**Stress Limit INVENTORY Rating**	**Stress Limit OPERATING Rating**	**Source of Criteria**
Compression (Longitudinal or Transverse): Compressive stress under effective prestress, permanent load, and transient loads	$0.60\phi w f'_c$	$0.60\phi w f'_c$	LRFD Table 5.9.2.3.2a-1 LRFD Article 5.6.4.7.2c
Longitudinal Tensile Stress in Precompressed Tensile Zone: (Intended for Segmental and similar construction) • For components with bonded prestressing tendons or reinforcement that are subject to not worse than: For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment • For components with unbonded prestressing tendons only	$0.0948\lambda\sqrt{f'_c} \le 0.3$ ksi tension No tension	$0.24\lambda\sqrt{f'_c}$ ksi tension No tension	LRFD Table 5.9.2.3.2b-1 and FDOT[6] no distinction for Environ't LRFD Table 5.9.2.3.2b-1
Longitudinal Tensile Stress through Joints in Precompressed Tensile Zone: (Intended for Segmental and similar construction) • Type A joints with minimum bonded auxiliary longitudinal reinforcement sufficient to carry the calculated longitudinal tensile force at a stress of $0.5f_y$; for internal and/or external PT (e.g., cast-in-place construction) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment • Type A joints without the minimum bonded auxiliary reinforcement through the joints; internal and/or external PT (e.g., match-cast epoxy joints or unreinforced cast-in-place closures between precast segments or between spliced girders or similar components.) • Type B joints (Dry-joints without epoxy. These bridges use external tendons only.)	$0.0948\lambda\sqrt{f'_c} \le 0.3$ ksi tension No tension 0.1 ksi min comp.	$0.24\lambda\sqrt{f'_c}$ ksi tension No tension No tension	LRFD Table 5.9.2.3.2b-1 Seg. Guide Spec. 9.2.2.2[7] FDOT no distinction for Environ't Ditto and FDOT Seg. Rating Criteria[8] Seg. Guide Spec. 9.2.2.2 FDOT Seg. Rating Criteria
Transverse Tension, Bonded PT: • Tension in the transverse direction in precompressed tensile zone calculated on basis of uncracked section (i.e., top prestressed slab) For (a) an aggressive corrosion environment and (b) moderately aggressive corrosion environment	$0.0948\lambda\sqrt{f'_c} \le 0.3$ ksi tension	$0.19\lambda\sqrt{f'_c}$ ksi tension	Seg. Guide Spec. 9.2.2.3 LRFD Table 5.9.2.3.2b-1 FDOT no distinction for Environ't FDOT Seg. Rating Criteria
Tensile Stress in Other Areas: • Areas without bonded reinforcement • Areas with bonded reinforcement sufficient to carry the tensile force in the concrete calculated on the assumption of an uncracked section is provided at a stress of $0.5f_y$ (<30 ksi)	No tension $0.19\lambda\sqrt{f'_c}$ ksi tension	No tension $0.19\lambda\sqrt{f'_c}$ ksi tension	Seg. Guide Spec. 9.2.2.4 LRFD Table 5.9.2.3.2b-1 Seg. Guide Spec. 9.2.2.4 LRFD Table 5.9.2.3.2b-1
Principal Tensile Stress at Centroidal Axis in Webs (Service III): • All types of segmental or beam construction with internal and/or external tendons.*	$0.110\lambda\sqrt{f'_c}$ ksi tension	$0.110\lambda\sqrt{f'_c}$ ksi tension	LRFD 5.9.2.3.3 FDOT LRFR Rating Criteria[9]

**Principal tensile stress is calculated for longitudinal stress and maximum shear stress due to shear or combination of shear and torsion, whichever is the greater. For segmental box, check centroidal axis. For composite beam, check at centroidal axis of beam only and at centroidal axis of composite section and take the minimum value. Web width is measured perpendicular to the place of web. For segmental box, it is not necessary to consider coexistent web flexure. Account should be taken of vertical compressive stress from vertical PT bars provided in the web, if any, but not including vertical component of longitudinal draped post-tensioning—the latter should be deducted from shear force due to applied loads.*

Check section at H/2 *from edge of bearing or face of diaphragm, or at end of anchor block transition, whichever is more critical.*

For the design of a new bridge, a temporary principal tensile stress of $0.142\lambda\sqrt{f'_c}$ *may be allowed during construction—per AASHTO Seg. Guide Spec.*

Initial load ratings for new design should be based upon specified concrete strength.

Load rating of an existing bridge should be based upon actual concrete strength from construction or subsequent test data.

Part 12

A new commentary Article C6A.5.11.5.3 is added as follows:

The initial stress limits for concrete, as outlined in Table 6A.5.11.5.3, were predominantly established by Corven Engineering (2004).[2] Furthermore, these limits were also specified in the work of Popok et al. (2024),[3] which used the 2004 study by Corven Engineering as a reference point.

Part 13

Table 6A.5.11.6-1 is revised to read:

Table 6A.5.11.6-1—System Factors for Post-Tensioned Segmental Concrete Box Girder Bridges

Bridge Type	Span Type	# of Hinges to Failure	System Factors (ϕ_s)[a]		
			No. of Tendons per Web		
			1/web[b]	2/web	3+/web
Balanced Cantilever, Type A Joints or Cast-in-Place	Interior Span	3	1.00 (0.95)	1.20 (1.05)	1.20 (1.05)
	End or Hinge Span	2	0.95 (0.90)	1.10 (0.95)	1.10 (0.95)
	Statically Determinate	1	n/a	0.95	0.95
Precast Span-by-Span, Type A or B Joints	Interior Span	3	n/a	1.20 (1.05)	1.20 (1.05)
	End or Hinge Span	2	n/a	1.10 (0.95)	1.10 (0.95)
	Statically Determinate	1	n/a	n/a	0.95

[a]*When two values are presented, the first entry refers to the case where one span is loaded (i.e., side-by-side vehicles); the second, in parentheses, refers to two adjacent spans loaded (i.e., multiple vehicles in the same lane).*

[b]*For sections with 1 tendon per web, if three or more webs are present, increase by 0.10 (0.05). This increase applies only for the case of 1 tendon per web.*

Part 14

Commentary Article C6A.5.11.6 is revised to read:

In the context of post-tensioned segmental box girders, the system factor must account for a few significant and important aspects different than other types of bridges. In particular, for a post-tensioned segmental bridge, the system factor, ϕ_s, must properly and appropriately account for:

- *longitudinally continuous versus simply supported spans,*
- *inherent integrity afforded by the closed continuum of the box section,*
- *multiple-tendon load paths,*
- *number of webs per box, and*
- *types of details and their post-tensioning.*

System Factor may vary significantly from one post-tensioned segmental box structure to the next and depends on a variety of factors including bridge geometry, number of spans, span continuity, boundary conditions, number of webs, number of tendons per web, prestress steel area, effective level of prestress, how live load is applied on the structure, as well as other parameters. Thus ideally, System Factors are calculated specifically for the structure under consideration, following the procedure outlined in NCHRP 406.[10] This procedure generally requires a 3-dimensional nonlinear finite element analysis. Such an analysis should account for the change in behavior of the structural system as a component fails and the accompanying redistribution of external load as well as internal forces. In lieu of such an analysis, System Factors for longitudinal flexure at the Strength Limit State may be taken from Table 6A.5.11.6-1. As system factors account for behavior at load levels much beyond those considered for service evaluation (i.e., from component to complete system failure), they are not to be used for service limit states.

Span type and No. of Hinges to Failure refers to the number of plastic hinges needed to form a collapse mechanism. That is to say, 1 hinge for a simple span or statically determinate structure; 2 hinges for the end span of a continuous unit; and 3 for an interior span or monolithic portal frame. The same reasoning applies whether a bridge is built using span-by-span or balanced cantilever construction. Hence a "statically determinate" cantilever bridge (i.e., two cantilevers with a suspended "drop-in" span) is to be treated as a simple span bridge.

The Table lists two values for continuous multi-span structures: a value for one span loaded, such as the case when a single very heavy truck or two heavy trucks in adjacent lanes load the bridge; and a value for two adjacent spans loaded, such as when two heavy vehicles in the same lane load the structure, where the vehicles are separated such that each is on an adjacent span. The two values given are not to be interpreted as use for rating positive or negative moment regions separately; the same System Factor applies to the entire structural system and should be used for both cases. Rather, which System Factor is most appropriate will depend on the local traffic pattern, which will determine the governing load case. Here the concern is what load configuration is most likely to cause failure of the structural system. If this traffic information is unavailable, the following guidance may be used: in general, the two adjacent spans loaded case tends to govern only on longer-span bridges, such as those with each span on the order of 300 ft and longer. For shorter spans, the single span loaded scenario is most likely to control, and in this case the corresponding System Factor may be used. The values in Table 6A.5.11.6-1 were developed from bridges with typical segmental box sections. For other structural types, such as those with significant curvature (approaching a radius of curvature of 800 ft or less), suspension bridges, or other unique characteristics, a System Factor analysis is recommended.

System Factors were developed from the conceptual approach described in NCHRP 406,[10] where factors are a function of the redundancy ratios of system to component capacity. For multi-girder bridges, the focus of NCHRP 406,[10] a component is defined as an individual girder. However, the definition of a component must be reconsidered when segmental bridges are analyzed. For longitudinal flexure, System Factors are meant to account for three types of behavior: longitudinal continuity, the integrity of the box section, and multiple tendon paths. As such, components must be defined to recognize these effects. Thus, in terms of longitudinal continuity, a component is *a potential plastic hinge necessary for bridge failure; for section integrity, a component is an individual web; and when considering multiple tendon load paths, a component is a tendon.*

For consideration of bridge type, the live load on the bridge used for System Factor evaluation ranges from first tendon yield to ultimate capacity and is significantly higher than service loads. As such, distinction between different types of construction (balanced cantilever vs cast-in-place) or joint type (A or B), which generally becomes important for consideration of service rather than strength limit states, was not made for System Factor determination. As such, the System Factors for all bridge types are identical when values are presented. A distinction is made between two broad categories of construction, however: a) balanced cantilever, Type A joints or cast-in-place; and b) precast span-by-span, Type A or B Joints. This distinction is made to separate recommended and/or existing from non-recommended/non-existing designs, depending on the number of tendons per web. For example, there is no known case of span-by-span construction with only one external tendon per web. This consideration led to the insertion of "n/a" in the Table (meaning "not applicable" or "not allowed"). A System Factor analysis is recommended if such a case is found to exist.

In general, the analysis indicated that structures with greater longitudinal continuity tended to have greater System Factors. For these cases, System Factor was nearly always governed by ultimate capacity analysis (rather than functionality or damage), where first component failure was almost always tendon yield.

It was further found that one of the most significant effects of live load placement in System Factor determination was how many spans of a continuous structure were loaded, where it was generally observed that loading two adjacent spans rather than a single span could result in significantly lower System Factors.

System factors are based on evaluations of ultimate system capacity, functionality, and component damage. For the structures considered, the analysis indicated that the component damage evaluation did not govern for any of the cases where System Factors are provided. Thus, values for 2 or 3 tendons per web are identical. An exception is the case of 2 tendons per web for statically determinate span-by-span construction, which is given a system factor of "n/a" due to the impracticality of this case.

Due to the inherent lack of redundancy with only 1 tendon per web, however, System Factors were lowered for the 1 tendon/web case to 1.0 for interior spans (and to 0.95 for adjacent spans loaded), even though the analysis suggests no change in System Factor from the 2 tendon per web case is required. This reduction is also to account for wider structures than those considered in the analysis, where loads with greater eccentricity to the damaged side may lower System Factors further than those found in the analysis. Similarly, system factors were lowered to 0.95 (0.90 for adjacent spans loaded) for the 1 tendon/web case for end or hinge spans. Because 3 web cases were found to have higher System Factors than 2 web cases, an increase of 0.10 (0.05 for 2 spans loaded) was allowed for the case of 1 tendon/web only. This increase of 0.10 applies only to sections that have three or more webs; it does not increase further for sections with more than three webs.

For longitudinal shear and shear torsion, the system factor is taken as 1.00 for the strength limit state for all circumstances.

With transverse post-tensioning of the deck slab, a segmental box is simply a prestressed concrete structure. Therefore, the system factor for transverse flexure of 1.00 is appropriate, regardless of the spacing of tendons; likewise for the local detail of a transverse beam support to an expansion joint device, although the possibility of having only one tendon in the effective section is recognized by reducing the system factor to 0.90.

For local details involving local shear and/or strut-and-tie action or analysis where the resistance is provided by local post-tensioning tendons or bars, a system factor of 1.00 is considered appropriate for two or more tendons. A reduced factor of 0.90 should be used where only one tendon or bar provides the resistance.

Part 15

The references from Corven Engineering[2] and Popok et al.[3] are added to the references section of the AASHTO MBE.

Conclusion

Collectively, the changes detailed herein establish a more definitive guideline for the inclusion of *TU* and *TG* effects in segmental bridges, specifying the limit states, rating levels, and associated load factors. The revised provisions introduce a clear direction to use a live load factor of γ_{LL} = 0.8 for segmental concrete box girders at the design-load inventory level, calibrated as an outcome of the NCHRP 12-123 project (with a final deliverable of NCHRP Report 1128),[3] as opposed to γ_{LL} = 1.0 applied for prestressed concrete components rated using the refined estimates of time-dependent losses as specified in Article 5.9.3.4 of the AASHTO LRFD specifications. The revised multiple presence factors decrease the factor for three loaded lanes from 0.85 to 0.75, and the factor for more than three loaded lanes from 0.65 to 0.6, as compared to AASHTO LRFD Table 3.6.1.1.2-1. The factors for one loaded lane and two loaded lanes remain the same as LRFD Table 3.6.1.1.2-1. With the newly adopted revisions, the use of the number of striped lanes has been replaced with the number of design lanes, but incorporating a live load factor of 0.65. It is important to recognize that the current system factors in AASHTO MBE are based on engineering judgment and experience. The newly adopted factors were developed from analyses based on the conceptual protocol outlined in NCHRP Report 406,[10] with appropriate modifications for the segmental bridges.

References

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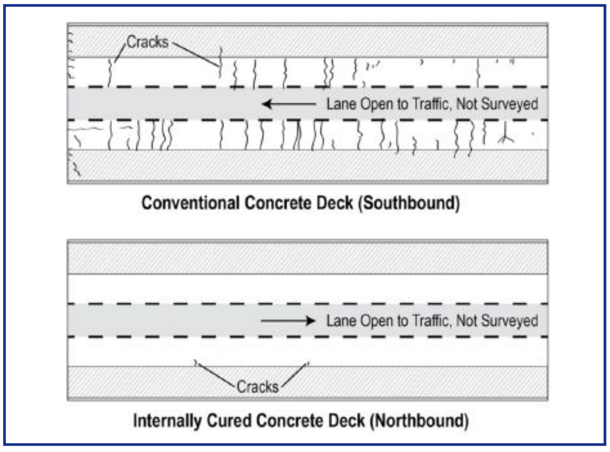
Improving Service Life of Concrete Bridge Decks using Prewetted Lightweight Aggregate

An article in the Summer 2024 issue of ASPIRE by Dr. Barrett, who works for the Federal Highway Administration (FHWA), describes the "Enhancing Performance with Internally Cured Concrete (EPIC2)" initiative in FHWA's current Every Day Counts (EDC) program. This initiative highlights the relatively simple approach of replacing a portion of the conventional fine aggregate with prewetted lightweight fine aggregate to provide internal curing. The higher absorption of manufactured structural lightweight aggregate is used to carry curing water into concrete so the entire body of concrete can more fully hydrate and have the improved characteristics of well-cured concrete. The absorbed water does not contribute to the mixing water (that is, it does not affect the *w/cm*) because it remains within the lightweight aggregate until after the concrete has set and pore sizes in the partially cured cement paste become smaller than the pores within the lightweight aggregate particles. As mentioned in Dr. Barrett's article, projects in Ohio and New York have demonstrated that internal curing can significantly reduce cracking in bridge decks.

The concept of internal curing from absorbed water in lightweight aggregate is not new. It has been known to some concrete technologists since at least 1957 when the beneficial curing effects were reported for lightweight concrete in papers by Klieger and by Jones and Stephenson that were presented during the World Conference on Prestressed Concrete held in San Francisco, CA.



"Enhancing Performance with Internally Cured Concrete (EPIC^2)" by Timothy J. Barrett, *ASPIRE*, Summer 2024

Replacing a portion of the conventional fine aggregate with prewetted lightweight fine aggregate to provide internal curing is a more recent approach that provides internal curing but without significantly reducing the concrete density.

More information is available on the EPIC^2 webpage (see ref. 3 in FHWA article), as well as on the ESCSI webpage: www.escsi.org/internal-curing/

Information on other uses of lightweight aggregate can be found at www.escsi.org