Approved Changes to the Ninth Edition *AASHTO LRFD Bridge Design Specifications:* Design of Segmental Bridges

by Dr. Oguzhan Bayrak, University of Texas at Austin

Article 5.12.5—Segmental Concrete Bridges of the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] was originally based on the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*.[2] AASHTO's Committee on Bridges and Structures Agenda Item 27, Working Agenda Item 218, will update many parts of this article and other articles related to segmental concrete bridges in the forthcoming 10th edition of the AASHTO LRFD specifications.[3] Selected approved updates are as follows:

- The 10th bullet item in Article 3.4.1—Load Factors and Load Combinations will be revised to read as follows:
  - *Service III – Load combination for longitudinal analysis relating to flexural tension–and principal tension in the webs of prestressed concrete superstructures with the objective of crack control.*
- The 18th paragraph of Article C3.4.1 will be deleted to streamline the commentary.
- A new definition will be added to Article 5.2, as follows:
  Diabolo—A formed void in a concrete deviation saddle or diaphragm in a shape to align and direct an external tendon through the horizontal and vertical tendon profile required for the design.
- Article 5.4.6.3 will be revised to read as follows:
  5.4.6.3—External Tendons Passing through Deviation Saddles
  External tendons passing through deviation saddles shall utilize either of the following details:
  - *Galvanized rigid steel pipe ducts*
  - *Diabolos*

  The minimum tendon radius at deviation saddles shall be as large as permitted by the geometry of the tendon and deviation saddle but, unless verified by testing, shall not be less than:

  $$R_{min,d} = 0.306\sqrt{f_{pu}}\,A_{ps} \geq 6.6\,ft \qquad (5.4.6.3\text{-}1)$$

  where:

  $R_{min,d}$ = *minimum tendon radius at deviation saddle (ft)*

  f_{pu} = *specified tensile strength of prestressing steel (ksi)*

  A_{ps} = *area of prestressing steel (in.²)*

  To utilize a smaller radius than specified by this article, the testing specified by Article 10.3.2.2 of the AASHTO LRFD Bridge Construction Specifications *for rigid pipe duct and plastic ducts through diabolos and Article 10.8.3 of the* AASHTO LRFD Bridge Construction Specifications *for plastic ducts through diabolos shall be performed to verify the adequacy of the proposed details.*

  The galvanized rigid steel pipe ducts in the deviation saddles shall meet the requirements set forth in Article 10.8.2, and the external plastic tendon ducts passing through the deviation saddles shall meet the requirements set forth in Article 10.8.3 of the AASHTO LRFD Bridge Construction Specifications.
- Commentary to Article 5.4.6.3 (C5.4.6.3) will be added, as follows:
  C5.4.6.3
  The minimum tendon radius specified by Article 5.4.6.3 is similar to the minimum radii recommended for external tendons in [Table 6-2 of] *the U.S. Department of Transportation Publication No. FHWA-HIF-19-067,* Replaceable Grouted External Post-Tensioned Tendons, *October 2019:*

  [Table 6-1, at bottom, from the same publication] *also recommends the following minimum radii and tangent lengths at anchorages:*

Tendon Size	Minimum Radius at Deviators (feet)
7-0.6″	6.6
12-0.6″	8.2
15-0.6″	9.0
19-0.6″	9.8
22-0.6″	10.7
27-0.6″	11.5
31-0.6″	12.3

Tendon Size	Minimum Radius at Anchorages (feet)	Minimum Tangent Length at Anchorages (feet)
7-0.6″	9.8	2.5
12-0.6″	11.5	3.3
15-0.6″	12.3	3.3
19-0.6″	13.1	3.9
22-0.6″	13.9	3.9
27-0.6″	14.8	4.3
31-0.6″	15.6	4.8

- The third paragraph in Article 5.8.4.4.2—Bearing Resistance will be revised to read as follows:
The full bearing plate may be used for A_g *and the calculation of* A_b *if the plate material does not yield at the factored tendon force, taken as 1.2 times the maximum jacking force in accordance with Article 3.4.3.2, and the slenderness of the bearing plate,* n/t, *shall satisfy:*
- The associated (Eq. 5.8.4.4.2-4) remains unchanged. The list of parameters after the third paragraph in Article 5.8.4.4.2 will be revised to read as follows:
n = *projection of bearing plate beyond the wedge hole or wedge plate, as appropriate (in.)*
f_b = *the factored tendon force, divided by the effective net area of the bearing plate,* A_b *(ksi)*
- The following paragraph will be added to the end of Article C5.9.2.3.3—Principal Tensile Stresses in Webs:
The principal tension calculation methods in Article 5.9.2.3.3 reflect the standard practice of including only the shear and normal stresses in the computation of the principal stresses. While these methods do not consider the effect of flexural stresses due to transverse moments on the calculation of the principal stresses, they have been shown to be adequate for the design of well-proportioned box girder bridges in conjunction with using the principal stress limits in this specification. The designer should be aware that flexural stresses due to transverse moments do affect principal stresses and may want to consider their inclusion in the principal stress calculation for atypically proportioned box girder bridges. The principal tensile stress limit in this specification does not strictly apply if flexural stresses are included in the principal stress computation.
- The second paragraph of Article 5.12.5.1—General (under Article 5.12.5—Segmental Concrete Bridges) will be revised to read as follows:
The method and schedule of construction assumed for the design shall be shown in the contract documents. Temporary supports required prior to the time the structure, or component thereof, is capable of supporting itself including loads and sequence in construction, shall also be shown in the contract documents.
- The first paragraph of Article C5.12.5.1 will be revised to read as follows:
For segmental construction, superstructures of single or multiple-cell box sections are generally used. Segmental construction includes construction by free cantilever, span-by-span, incremental launching, or other methods using either precast or cast-in-place concrete segments which are connected together to produce either continuous or simple spans.
- The first paragraph of Article 5.12.5.2.3—Analysis of the Final Structural System will be revised to read as follows:
The final structural system shall be analyzed for redistribution of construction-stage force effects due to internal deformations from creep and shrinkage and changes in support and restraint conditions, including accumulated locked-in force effects resulting from the construction process.
- Article 5.12.5.3.2—Construction Loads will be revised, as follows:
WS = *horizontal wind load on structures in accordance with the provisions of Section 3. The wind speed associated with load combinations e and f in Table 5.12.5.3.3-1 and the wind speed associated with load combinations c and d in Table 5.12.5.3.3-1 shall be as determined by the Owner (ksf)*
A = *static weight of precast segment being handled for precast cantilever construction or the empty weight of the form-traveler being utilized for cast-in-place cantilever construction (kip)*
AI = *dynamic response due to accidental release or application of a precast segment load, empty form-traveler load or other sudden application of an otherwise static load to be added to the dead load; in lieu of a dynamic analysis, may be taken as 100 percent of load* A *(kip)*
- The eighth paragraph of Article C5.12.5.3.2 will be revised and additional explanation will be provided, as follows:
The following information is based on some past experience and could be considered for preliminary design. Form-travelers for cast-in-place segmental construction for a typical two-lane bridge with 15.0 to 16.0 ft segments may be estimated to weigh 160 to 180 kips. The weight of form-travelers for wider double-celled box sections may range up to approximately 280 kips. Consultation with contractors or subcontractors experienced in free cantilever construction, with respect to the specific bridge geometry under consideration, is recommended to obtain a design value for form-traveler weight.
Using wind speeds of 0.75 of the speed used for the in-service Strength III limit state for load combinations c and d and 70 mph for load combinations e and f is a reasonable approximation of wind loads that would have resulted from past practice using the AASHTO LRFD Bridge Design Specifications *prior to the change from fastest mile wind speed to 3-second gust wind speed. ASCE 37 can also help give guidance for wind loads during construction.*
Cast-in-place segments are typically supported by form-travelers. Accidental release may involve the release of the empty form-traveler. Accidental release may also involve failure of the supports for the bottom soffit form during segment casting, thereby releasing the bottom slab form and working platform, as well as the bottom slab and web concrete. In the worst case, accidental release may involve failure of the entire form-traveler and all newly cast segment concrete. The weight of the empty form-traveler is the minimum load that should be included in AI. *Note that using 100 percent of the weight of the empty form-traveler for* AI *is a practice that has been successfully utilized in France without collapse of a cantilever. Owners seeking a further reduction of risk could consider including the weight of some segment concrete in* A *and* AI.
For precast segments being lifted by a beam and winch or deck-mounted crane, and CIP [cast-in-place] segments being supported by form-travelers, the dynamic effect of an accidental release is an upward rebound. For precast segments being lifted by a ground-based crane where the segment could be above the cantilever, the dynamic effect is a downward impact force from the segment being suddenly released during lifting.
- The third paragraph of Article 5.12.5.3.3—Construction Load Combinations at the Service Limit State will be revised to read as follows:
The distribution and application of the individual erection loads appropriate to a construction phase shall be selected to produce the most unfavorable effects. The compressive stress in

concrete during construction shall not exceed $0.6\phi_w\sqrt{f'_c}$*, where* f'_{ci} *is the design concrete compressive strength at the time of load application or transfer of prestress. The value for* ϕ_w *shall be calculated as described in Article 5.9.2.3.2a.*

- New commentary Article C5.12.5.3.3, will be added, as follows:
  C5.12.5.3.3
  Table 5.12.5.3.3-1 was developed primarily for balanced cantilever construction but provides a reasonable basis for load combinations for other segmental construction methods. When applied to construction methods other than balanced cantilever construction, loads specific to balanced cantilever, such as DIFF, U *and* WUP*, may be disregarded.*
  The compressive stress limit of $0.6\phi_w\sqrt{f'_{ci}}$ *is specified during construction of segmental bridges, as opposed to the limit of* $0.65f'_{ci}$ *contained in Article 5.9.2.3.1a. This is due to the uncertainty associated with construction loads.*

- Article 5.12.5.3.4 will be revised, as follows:
  5.12.5.3.4—Construction Load Combinations at Strength and Extreme Event Limit States
  5.12.5.3.4a—Construction Load Combinations at the Strength Limit State
  The factored resistance of a component shall be determined using resistance factors specified in Article 5.5.4.2. The following components shall be evaluated for construction loads at the Strength I, III and V limit states:
  - o *Prestressed or conventionally reinforced substructures*
  - o *Prestressed or conventionally reinforced segmental superstructures*

  The Strength I, III and V load combinations from Table 3.4.1-1 shall apply with the load factors in the table and as modified by this article. The loads DIFF, CEQ *and* IE *shall be included and factored with* γ_p *for* DC. *The load* WUP *shall be included with* WS. *The load* CLL *shall be included and used in place of* LL. *The load* WE *shall be included with load factors of 0.9 and 0.4 for the Strength III and Strength V load combinations, respectively. The wind speeds for* WS *associated with the Strength III and Strength V load combinations shall be as determined by the Owner.*

- Article C5.12.5.3.4a will be revised to read as follows:
  C5.12.5.3.4a
  Using a load factor of 1.25 for construction equipment loads for segmental construction, such as erection gantries, cranes supported by the structure and segment transporters is reasonable as opposed to the 1.5 load factor specified in Article 3.4.2 for construction loads. This is because construction equipment loads are well known or included in the plans for the construction method assumed for design. Smaller miscellaneous construction loads are included in the CLL *allowance.*
  Using load factors of 1.0 with wind speeds of 0.95 of the speed used for the in-service Strength III limit state for the construction Strength III load combination, and 75 mph for the construction Strength V load combination are reasonable approximations of wind loads that would have resulted from past practice using the AASHTO LRFD Bridge Design Specifications *prior to the change from fastest mile wind speed to 3-second gust wind speed. ASCE 37 can also help give guidance for wind loads during construction.*
  5.12.5.3.4b—Construction Load Combinations at the Extreme Event Limit State
  In accordance with Article 1.3.2.1, the resistance factor for concrete design shall be 1.0. The loads DIFF *and* CLL *shall be placed to maximize the force effects.*
  - o *For maximum force effects:*
    $$\sum \Upsilon Q = 1.1\,(DC + DIFF) + 1.3\,(CEQ + CLL) + A + AI$$
    (5.12.5.3.4b-1)
  - o *For minimum force effects:*
    $$\sum \Upsilon Q = DC + DIFF + CEQ + CLL + A + AI$$
    (5.12.5.3.4b-2)

  In addition to the superstructure, the following substructure elements shall consider the dynamic response (AI)*:*
  - o *Temporary supports and their foundations.*
  - o *Piers, including the pier to footing or pier to monoshaft connection.*
  - o *Above grade or buried footings. The design shall be based on pile or shaft force effects resulting from an analysis of the dynamic response, regardless of whether the static geotechnical resistance of the piles or shafts is exceeded.*
  - o *Piles and shafts supporting footings where the ground or mudline is less than one diameter of a foundation element above the bottom of footing, regardless of whether the static geotechnical resistance of the piles or shafts is exceeded. The structural resistance of the piles or shafts shall be checked from the bottom of the footing to the first points of maximum moment and shear below the ground or mudline.*
  - o *The structural resistance of monoshafts, even if below the ground or mudline, shall be checked to the first points of maximum moment and shear below the ground or mudline.*

- Article C5.12.5.3.4b will be revised, as follows:
  C5.12.5.3.4b
  The construction load combinations evaluated at the extreme event limit state are intended to address a low probability event. This extreme event limit state evaluation does not replace the evaluation of construction loads under the service limit state or strength limit state.

- The first paragraph of Article 5.12.5.3.6—Creep and Shrinkage will be revised, as follows:
  Creep and shrinkage shall be determined in accordance with Article 5.4.2.3. Forces and stresses shall be determined for redistribution of restraint stresses developed by creep and shrinkage deformations that are based on the assumed construction schedule as stated in the contract documents.

- The fourth paragraph will be revised and new discussion added to Article C5.12.5.3.8d (Torsional Reinforcement), as follows:
  Unlike solid sections, when designing the webs of segmental bridges, the shear and torsion reinforcement should be directly added together. Reinforcement for transverse bending in the webs and other box girder elements should be accounted for in the total reinforcement demand. Standard practice for typical segmental highway bridges is to combine the shear and torsion reinforcement, and transverse bending reinforcement in each box girder element so that the total reinforcement on each face of each web or flange exceeds the greater of:
  - o $1.0(A_{sv}+A_{st}) + 0.5A_{sb}$ *(C5.12.5.3.8d-1)*
  - o $0.5(A_{sv}+A_{st}) + 1.0A_{sb}$ *(C5.12.5.3.8d-2)*

  where:
  A_{sv} = *area of required shear reinforcement for one face of a*

web within a distance s *(in.²)*

A_{st} = *area of required torsion reinforcement for one face of an exterior web or flange within a distance* s *(in.²)*

A_{sb} = *area of required reinforcement due to transverse bending for one face of a web or flange within a distance* s *(in.²)*

The equations above acknowledge that the maximum shear and torsion effect is unlikely to occur concurrently with maximum transverse bending. However, the engineer must consider if this condition is applicable to the structure in design. Examples where direct combination of the reinforcement are applicable include single lane ramp structures and transit structures.

- Articles 5.12.5.3.9a and 5.12.5.3.9b (within Article 5.12.5.3.9—Provisional Post-Tensioning Ducts and Anchorages) will be revised to read as follows:
  5.12.5.3.9a—General
  Provisions for adjustments of prestressing force to compensate for unexpected losses during construction specified in the contract documents.
  5.12.5.3.9b—Bridges with Internal Ducts
  Provisional duct or anchorage capacity shall accommodate increases to both positive moment and negative moment post-tensioning forces. The increases shall be taken as not less than five percent of the total post-tensioning forces specified in the contract documents.
  For continuous bridges, positive moment force adjustment need only be provided for the middle 50 percent of each span.
  Provisional capacity shall be uniformly distributed to each web and located symmetrically about the bridge centerline. Anchorages shall be distributed uniformly at three-segment intervals along the length of the bridge.
  Utilized and unutilized provisional ducts shall be grouted at the same time as other ducts in the span.

- Article C5.12.5.3.9b will be revised, as follows:
  Provisional post-tensioning duct and/or anchorage capacity permit the introduction of additional prestressing force to compensate for installation or stressing problems that might arise during construction.
  Excess capacity may be provided by use of oversize ducts and oversize anchorage hardware at selected anchorage locations well distributed along the length of the bridge.

- Article 5.12.5.3.9c will be revised to read as follows:
  5.12.5.3.9c—Provision for Future Load
  Provision shall be made for installation and stressing access and for anchorage attachments, pass-through openings, and deviation saddle attachments to permit future addition of corrosion-protected unbonded external tendons located inside the box section symmetrically about the bridge centerline. Diaphragm and deviation saddles shall be designed to accommodate the forces from the future tendons. At a minimum one future tendon for exterior webs and two future tendons for interior webs shall be provided. These tendons shall have a minimum size equivalent to 12 0.6-in. diameter strand tendons. The future tendons shall satisfy one of the following requirements:
  - *Future tendons shall provide a post-tensioning force of not less than ten percent of the primary positive moment and negative moment post-tensioning forces.*
  - *Future tendons shall be designed to increase the superstructure resistance for flexure, shear, and torsion for the in-service Strength I, Service I and Service III load combinations to accommodate the combined effect of the following load increases:*
    –Ten percent DW
    –Ten percent bridge railing dead load
    –Ten percent LL + I
    –Ten percent CR *and* SH

  Any extra design resistance in the original design can be utilized to resist the specified load increases, such that the initial and future post-tensioning together meet the increased load requirements.

- Article C5.12.5.3.9c will be revised, as follows:
  This provides for future addition of external unbonded post-tensioning tendons.
  The first requirement specifies an addition of future tendons to provide for a force of ten percent of the primary positive moment and negative moment post-tensioning forces and does not require any additional analysis.
  The second requirement provides for a refinement of the future post-tensioning system by designing for a ten percent increase in the specified loads. Relative to the first option, this refinement allows for a reduction in future post-tensioning forces in regions where there is already extra capacity. An example is a balanced cantilever bridge constructed with form-travelers or beam and winches. The post-tensioning required to support the free cantilevers along with the equipment loads can be more than what is required for the in-service structure. Therefore, the future post-tensioning tendons for negative moment at the piers required to resist specified load increases can possibly be reduced from a ten percent increase in the negative post-tensioning force.
  Typically, future post-tensioning tendons are draped external tendons that are close to the bottom slab near midspan and anchor in the pier diaphragms as high as possible. The tendons typically lap through the pier diaphragms with anchorages located on opposite sides of the diaphragm. Provision for larger amounts of post-tensioning might be developed, as necessary, to carry specific amounts of additional load as considered appropriate for the structure.

- The third paragraph will be revised and a new fifth paragraph will be added to the end of Article C5.12.5.3.11a—Minimum Flange Thickness (within Article 5.12.5.3.11—Box Girder Cross Section Dimensions), as follows:
  Where the clear span between the faces of webs is 15.0 ft or larger, transverse prestressing of the top deck is typically utilized. However, prestressing may also be utilized to improve deck durability (regardless of span length) at the Owner's discretion. To ensure that these benefits are realized, Owners need to ensure that project specifications explicitly state when transverse prestressing of the deck is required for other than structural purposes.
  For most existing segmental bridges, the cantilever length of the top flange is less than 0.60 of the interior clear span of the top flange. When the cantilever exceeds 0.45 of the interior span, designers should consider investigating top flange deflections during casting and erection and the need for partial post-tensioning of the top flange for geometry control.

- Article 5.12.5.3.11c—Length of Top Flange Cantilever will be deleted and the current Article 5.12.5.3.11d—Overall Cross Section Dimensions will be renumbered.

- The second paragraph of Article C5.12.5.4.1 (the General portion of Article 5.12.5.4—Types of Segmental Bridges) will be revised to read as follows:
Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons. Bridges built with erection trusses may utilize internal tendons, external tendons, or combinations thereof. Due to considerations of segment weight, span lengths for precast segmental box girder bridges, except for cable-stayed bridges, generally do not exceed 400 ft.

- The second paragraph of Article 5.12.5.4.2—Details for Precast Construction will be revised, as follows:
Multiple small-amplitude shear keys at match-cast joints in webs of precast segmental bridges shall extend over as much of the web as is compatible with other details. Details of shear keys in webs should be similar to those shown in Figure 5.12.5.4.2-1. Alignment keys shall also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single-element keys.

- The fourth paragraph of Article 5.12.5.4.2 will be deleted.

- The fifth paragraph of Article 5.12.5.4.2 will be revised, as follows:
Where an epoxy joint is specified, a temporary or permanent prestressing system shall provide a minimum compressive stress of 0.030 ksi and an average stress of not less than 0.040 ksi across the joint until the epoxy has cured. If the segment or segments being erected are supported by the prestressing system, the stresses due to the prestressing and the weight of the segment being erected shall be combined and the same stress limits apply.

- A new second paragraph will be added and the third paragraph will be revised in Article C5.12.5.4.2, as follows:
To aid in geometry control, the stress across the joint should be as uniform as practical until the epoxy has cured. Having a difference between maximum and minimum compressive stress of no more than 0.060 ksi is recommended.
Small-amplitude shear keys in the webs are less susceptible to stress concentrations and construction damage, which will result in loss of geometry control, than larger single-element keys. Alignment keys in the top and bottom flanges are less susceptible to such damage.

- The third paragraph of Article 5.12.5.4.3—Details for Cast-in-Place Construction will be revised, as follows:
Diaphragms shall be provided at abutments, piers, hinge joints, and bottom flange angle points in structures with straight haunches. Diaphragms shall be substantially solid at piers and abutments, except for access openings and utility holes. Diaphragms shall be sufficiently wide as required by design.

- A new first paragraph to Article C5.12.5.4.4—Cantilever Construction will be added, and the fourth paragraph will be revised, as follows:
Permanent or temporary longitudinal strand or bar tendons satisfy the requirement to anchor a minimum of two tendons in each segment.
Lengths of segments for free cantilever construction typically range between 8.0 and 18.0 ft. Lengths vary with the construction method, the span length, and location within the span.

- The second paragraph of Article 5.12.5.4.5—Span-by-Span Construction will be revised, as follows:
Forces and stresses due to the changes in the structural system, in particular the effects of the application of a load to one system and its removal from a different system, shall be accounted for. Redistribution of such forces and stresses by creep shall be taken into account and allowance made for possible variations in the creep rate and magnitude.

- The first and second paragraphs of Article 5.12.5.4.6a (the General portion of Article 5.12.5.4.6—Incrementally Launched Construction) will be revised to read as follows:
  - *Stresses under all stages of launching shall not exceed the limits specified in Article 5.12.5.3.3.*
  - *Provision shall be made to resist the frictional forces on the substructure during launching and to restrain the superstructure if the structure is launched down a gradient.*

- The second paragraph of Article C5.12.5.4.6a will be revised, as follows:
For determining the critical frictional forces, the friction on launching bearings should be assumed to vary between 0 and 4 percent, whichever is critical. The upper value may be reduced to 3.5 percent if pier deflections and launching jack forces are monitored during construction. These friction coefficients are only applicable to bearings employing a combination of virgin PTFE and stainless steel with a roughness of less than 1.0×10^{-4} *in.*

- Article 5.12.5.4.6b—Force Effects Due to Construction Tolerances will be revised to read as follows:
Force effects due to permissible construction tolerances shall be considered, both during construction and for the in-service structure. Unless otherwise specified in the contract documents, the tolerances shall be taken as:
  - *In the longitudinal direction between two adjacent bearings .... 0.2 in.*
  - *In the transverse direction between two adjacent bearings .... 0.1 in.*
  - *Between the fabrication area and the launching equipment in the longitudinal and transverse direction .... 0.1 in.*
  - *Lateral deviation at the outside of the webs .... 0.1 in.*

  The horizontal force acting on the lateral guides of the launching bearings shall be taken as less than one percent of the vertical support reaction.
  For design of the in-service structure, locked-in force effects from construction tolerances, EL, *shall be included in load combinations according to Table 3.4.1-1. For forces and stresses during construction, one-half the effects of construction tolerances and one-half the effects of temperature gradient shall be added together and applied as load* TG *in Table 5.12.5.3.3-1. Concrete stresses due to the load combinations in Table 5.12.5.3.3-1 shall not exceed those specified in Article 5.12.5.3.3.*

- A new article, C5.12.5.4.6b, will be added, as follows:
C5.12.5.4.6b
The designer can reduce tolerances through construction details, in which case the force effects may be reduced accordingly. Permanent bearings grouted in place after launching has been completed is an example of a construction detail to alleviate tolerance force effects.

- The third and fourth paragraphs of Article 5.12.5.4.6c—Design Details will be revised, as follows:
The straight tendons required for launching shall be sufficient to meet the service and strength limit state requirements of this specification. Not more than 50 percent of the tendons shall be

coupled at one construction joint. Anchorages and locations for the straight tendons shall be designed for the concrete strength at the time of tensioning.

The faces of construction joints shall be provided with keys or a roughened surface with a minimum roughness amplitude of 0.25 in. Bonded nonprestressed reinforcement shall be provided longitudinally and transversely at all exterior concrete surfaces. The longitudinal reinforcement shall run continuously through the segments and joints. Longitudinal reinforcement shall not be spliced at joints.

- A new paragraph will be added to the Commentary Article C5.12.5.4.6c, as follows:
  Minimum reinforcement of the equivalent of No. 4 bars spaced at 5.0 in. both longitudinally and transversely on all exterior concrete surfaces of the girder, with the longitudinal bars running across the joints has been recommended in past editions of this specification. Splices of the longitudinal bars may be located just to one side of joints such that no length of the splice is running through the joints.
- The fourth paragraph of Article 5.12.5.5—Use of Alternative Construction Methods will be revised to read as follows:
  For the value engineering, the Contractor shall provide a complete set of design computations and revised contract documents. The value engineering redesign shall be prepared by a Professional Civil Engineer or Structural Engineer experienced in segmental bridge design. Upon acceptance of a value engineering redesign, the Professional Civil Engineer or Structural Engineer responsible for the redesign shall become the Engineer in Responsible Charge. Based upon the jurisdiction in which the project resides, the Owner shall specify whether a Professional Civil Engineer or Structural Engineer is required for the redesign.
- A new second paragraph to Article C5.12.5.5 will be added, as follows:
  It is recommended that the Engineer in Responsible Charge for the value engineering redesign be licensed and that the working drawings and calculations are signed and sealed accordingly.

Conclusion

The segmental bridge design provisions of the ninth edition AASHTO LRFD specifications will be revised to be more consistent with the remainder of Section 5, be more current with the remaining provisions, and reflect more current practice. The cost implications of the revisions summarized in this article are anticipated to be minimal, if any.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
2. AASHTO. 1999. *Guide Specifications for Design and Construction of Segmental Concrete Bridges*. 2nd ed., with 2003 Interim Revisions. Washington, DC: AASHTO.
3. AASHTO. Forthcoming. *AASHTO LRFD Bridge Design Specifications*. 10th ed. Washington, DC: AASHTO.

References in Quoted Material

- AASHTO. 2017. *AASHTO LRFD Bridge Construction Specifications*. 4th ed., with 2020 Interim Revisions. Washington, DC: AASHTO.
- American Society of Civil Engineers (ASCE). 2015. *Design Loads on Structures during Construction*. ASCE/SEI 37-14R (2019). Reston, VA: ASCE.
- Ledesma, T. 2019. *Replaceable Grouted External Post-Tensioned Tendons*. FHWA-HIF-19-067. Washington, DC: Federal Highway Administration. https://www.fhwa.dot.gov/bridge/concrete/hif19067.pdf.