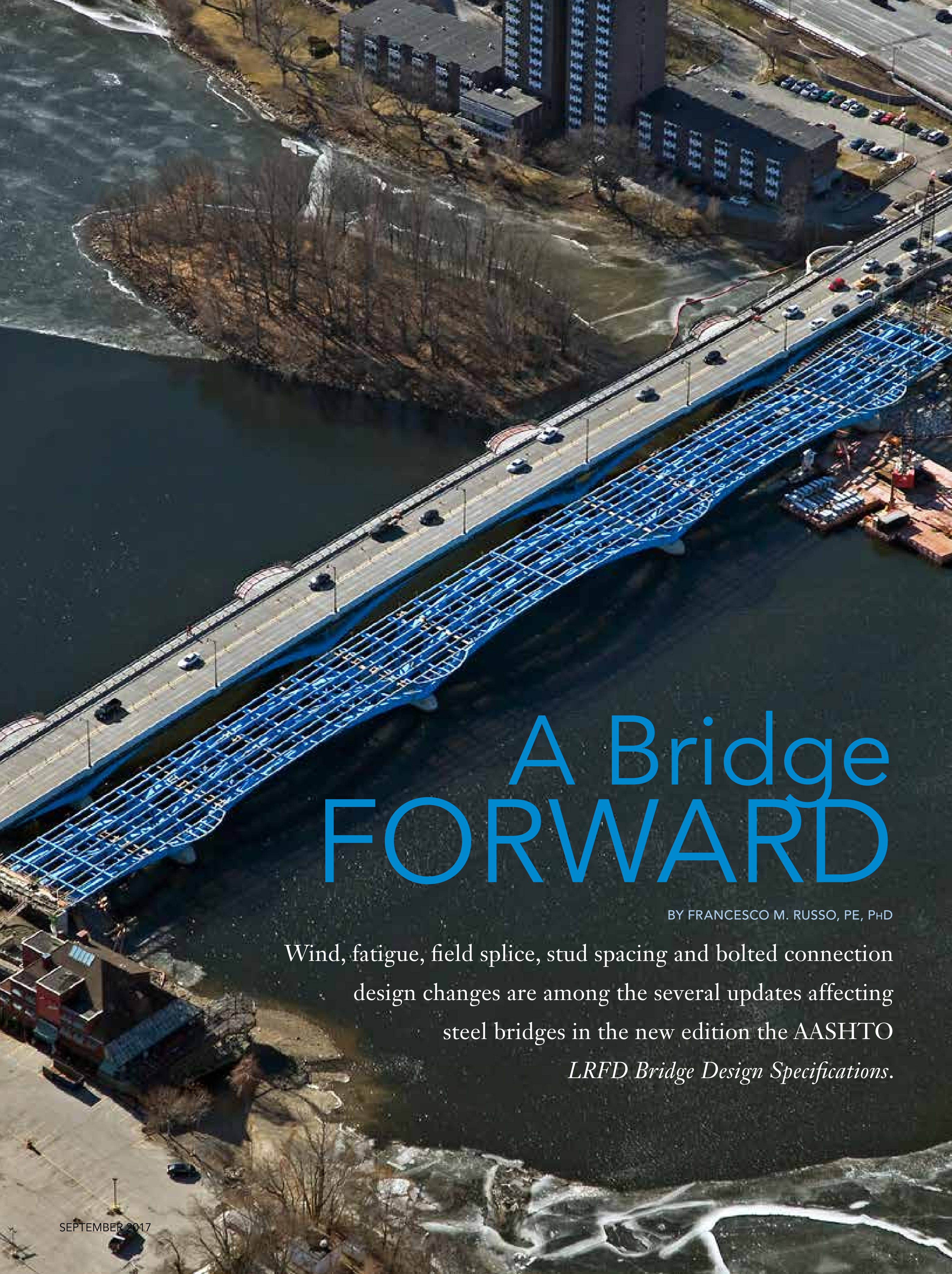


A Bridge FORWARD

BY FRANCESCO M. RUSSO, PE, PHD

Wind, fatigue, field splice, stud spacing and bolted connection design changes are among the several updates affecting steel bridges in the new edition the AASHTO *LRFD Bridge Design Specifications*.

THE 8TH EDITION of the AASHTO *LRFD Bridge Design Specifications* introduces a number of changes affecting steel bridges.

The majority of these changes appear in Chapter 3 – Loads and Load Factors and Chapter 6 – Steel Structures. In addition, a new AASHTO guide specification, *Guide Specifications For Wind Loads On Bridges During Construction*, introduces tools to evaluate the effects of wind loads on bridges of all types under construction. Here, we'll cover some important changes in the new AASHTO *LRFD Specifications* as well as the new *Guide Specifications* and how they apply to steel bridge design.

Chapter 3

Let's begin with Chapter 3 of the *LRFD Specifications*. A significant change in this chapter affecting steel structures is the introduction of new Fatigue I and II Limit State load factors. The load factors that have been commonly used through the 7th Edition *Specifications*—1.5 for Fatigue I and 0.75 for Fatigue II—are based on prior research on effective truck weights and experimental testing of steel structures. Historically, it has been assumed that the 1.5 and 0.75 load factors were sufficient to represent the effects of maximum and effective fatigue loading. It was also believed that only a single truck in a single lane contributed to the stress range. There were also assumptions of how many cycles of stress were produced by the passage of a truck for simple spans, continuous spans, cantilever structures, floor beams, etc. These rules had not been examined in several decades. As a result, the Transportation Research Board sponsored Project R19B as part of the SHRP2 program and one of the goals of the project was to assess and calibrate the fatigue limit state.

The R19B team, led by Modjeski and Masters, collected weigh-in-motion (WIM) data from around the country in order to quantify actual truck axle weights and spacing. Using approximately 8.7 million records, they were able to simulate the ranges of bending moments in a family of simple- and two-span continuous bridges, and they were able to compare those to the moments produced by the AASHTO fatigue design loading: a three-axle vehicle with a gross weight of 72 kips. (Note that this work specifically focused on moments, a value relatable to stress range, and not simply truck weight.) Prior fatigue studies have generally been based on vehicle weight, but it is obvious that weight is only one factor that, along with axle spacing and relative axle loading, produces the stress range.

Using the statistics of the WIM data, the R19B team was able to determine the effective truck moments using Miner's rule, the probability-based maximum moments and the appropriate load factors for each limit state. Although the R91B project initially recommended load factors of 2.0 for the Fatigue I Limit State and 0.8 for the Fatigue II Limit State, further examination of the data resulted in AASHTO adopting new load factors as follows: a Fatigue I Load Factor of 1.75 and a Fatigue II Load Factor of 0.8. Both are clearly larger than the current practice. Also, note the historic relationship of 2:1 between the Fatigue I and II load factors is no longer valid. This is due to a growing number of vehicles that produce large bending moments in relationship to the effective value. The relationship between the Fatigue I and II load factors is now approximately 2.2—i.e., 1.75/0.8. These changes only affect the loading aspects of fatigue design; the resistances of the various details have not changed as a result of this work.

Other aspects of the calibration of the Fatigue Limit State included determining if a single truck in a single lane is still a valid design approach, as well as determining if the cycles-per-passage table in AASHTO is still applicable. The R19B project confirmed that it is still valid, based on the WIM data, to assume that a single truck in a single lane is the proper loading to produce the design stress range. Although there are occasional passages of trucks in adjacent lanes, it is rare that they are fully correlated in terms of passing time and force effects such that a multi-lane effect needs to be considered. The study also evaluated the AASHTO cycles-per-passage approach and recommended some simplifications. For longitudinal members such as rolled beams or plate girders in a multi-beam cross section, the new recommendations for cycles per passage are as follows:

Table 1: Cycles per Passage for Longitudinal Members

Longitudinal Members		*N*
Simple-span girders		1.0
Continuous girders	Near interior support	1.5
	Elsewhere	1.0

This approach removes the distinction of bridges with spans under and over 40 ft. Recommendations for cantilever spans and floor beams are also found in AASHTO in the revised table.

Chapter 6

Numerous changes to Chapter 6 were also introduced in the new *Specifications*. Some of these are major changes in practice, such as new bolted field splice provisions, new design approaches for compression members and changes in shear stud spacing that will facilitate the use of precast deck panels. Other changes in detailing skewed bridges, longitudinal stiffeners and connection plates and editorial changes to various bolt design provisions (to reflect changes in ASTM designations) are also discussed.

Bolted Field Splices. A major change in the design procedure for bolted field splices was adopted in the new edition, greatly simplifying the design approach. The approach in the 7th Edition, stemming from work to rationally address bolted splices in composite members, has been around for nearly

Frank Russo (**frusso@mbakerintl.com**) is associate vice president and senior technical manager – bridges with Michael Baker International. The author would like to acknowledge **Michael Grubb, PE**, of M.A. Grubb and Associates, LLC, who is responsible for creating the many ballot items for AASHTO T-14 each year and is also an AISC consultant.

twenty years. Though deemed safe, it was also perceived by some as complex and lacking in clarity. The new approach described below results in similar or slightly larger flange splices with a general lowering of the number of web design bolts and is a substantially simpler process.

To determine whether a new method of splice design could be advanced, a task force was formed, working on behalf of AASHTO T-14, to develop a new design approach for flexural splices. This task force consisted primarily of Michael A. Grubb of M.A. Grubb and Associates, Karl Frank of Hirschfeld Industries, Justin Ocel of the FHWA and the author. The work resulted in a simple approach that requires the engineer to design the splice as follows:

- Provide a web splice to develop the factored shear resistance of the web
- Provide a flange splice that develops the factored strength of the smaller of two abutting flanges at a splice

In following these two simple rules, the capacity of the web in shear is fully developed across the splice as is the capacity of the smaller of each of the abutting top and bottom flanges. If a model that includes only the axial capacity of the flanges is sufficient to resist the factored moments at the point of splice, the design is deemed sufficient. This is demonstrated in Figure 1. This model determines if the capacity of the flanges alone is sufficient to carry the design moments—i.e., there is no need for the web to carry any moment.

Note that there is no longer a requirement for the flexural capacity of the splice to be a function of the strength of the section. The splice must be capable of resisting the factored moments at the point of the splice after proportioning the web and flange as described above. This is a significant change in philosophy in the *Specifications*. The new premise is that if the web is fully spliced for the shear strength of the section and the flange is fully spliced for the capacity of the flanges, those two requirements bound the possible limits for each component. If the moment resistance provided by the flange couple shown in Figure 1 is insufficient to resist the factored moments at the point of splice, an additional horizontal force, H_w, is added to the web as illustrated in Figure 2.

The additional horizontal force added to the web is that required for the design moments to be resisted. The horizontal force is vectorially added to the vertical force on the web splice for purposes of checking the web bolts.

Many splice designs were performed using the 7th Edition and proposed 8th Edition provisions. These splice calculations covered girder spacing from 7.5 ft to 12 ft. and three-span bridges with center spans ranging from 150 ft to 300 ft. There were some instances in which the 8th Edition provisions produced a substantial decrease in the number of web bolts due to the omission of a required moment to be carried by the web. In order to assess if this was a concern with regard to overall performance, a series of nonlinear finite element analyses including nonlinear bolt shear force distribution models were performed. The analyses were conducted on a bolted splice in an approximately 109-in.-deep plate girder to assess the expected safety of these new splices with fewer bolts. The results of the modeling indicated that the forces were easily accommodated in these smaller bolt patterns.

Coinciding with the introduction of this new design approach, AISC has published an annotated design example and an accompanying design spreadsheet (visit **www.steelbridges.org/nsbasplice** to access these resources).

Axial Strength of Compression Members. The provisions for compression member strength have been simplified and reorganized in the 8th Edition. They are similar to the approaches used by AISI and AISC for members with and without slender compression elements. The 7th Edition approach implements the "Q factor" reductions for slender elements and combines slender and non-slender compression members in Article 6.9.4.1.1. Specifically, Table 6.9.4.1.1-1 includes two parallel columns, one in which only "column buckling modes" are applicable—i.e., Q=1—and one for which a blended effect of column buckling and local buckling interact—i.e., Q<1. The 8th Edition does away with the Q factor blending of local and column buckling and instead relies on the unified effective width concept for the treatment of local buckling of slender sections in a revised Article 6.9.4.2 and accompanying sub-articles.

Compression member strength is now treated with a simpler two-step process for members with and without slender compression elements. In the first step, the axial compression strength of the gross section is defined as $P_{cr} = F_{cr}A_g$ where F_{cr} is related to the limit states of flexural, torsional and flexural-

Figure 1. Positive moment flexural resistance based on flange capacity alone.

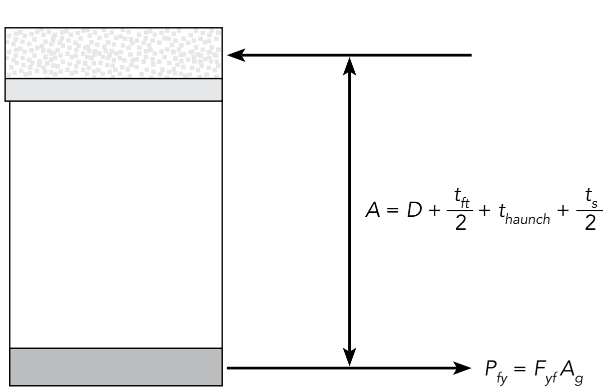
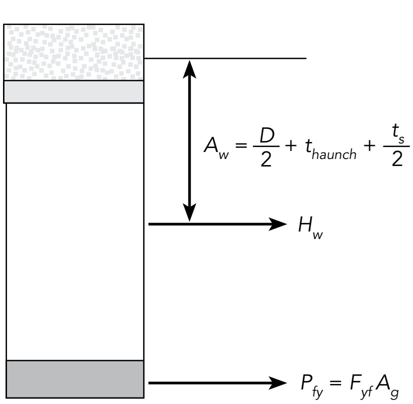


Figure 2. Positive moment flexural resistance relying on a web contribution.



The 8th Edition of the *LRFD Specifications* includes changes in shear stud spacing that will facilitate the use of precast deck panels.

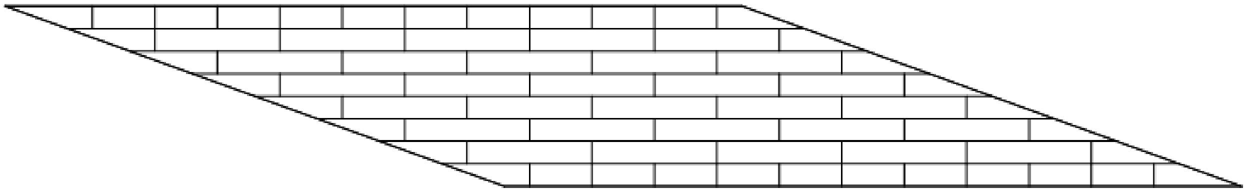
torsional buckling of the gross section, assuming local buckling is precluded. For a member with non-slender elements—i.e., *b/t* and *D/t* limits that satisfy non-slender limits of AASHTO 6.9.4.2.1—only the member stability limits apply. Nevertheless, nearly all compression members have their capacity limited by overall member slenderness to some stress, F_{cr}, less than F_y. Thus compression members with and without slender elements are likely to have their capacities limited to less than F_y regardless of the local slenderness.

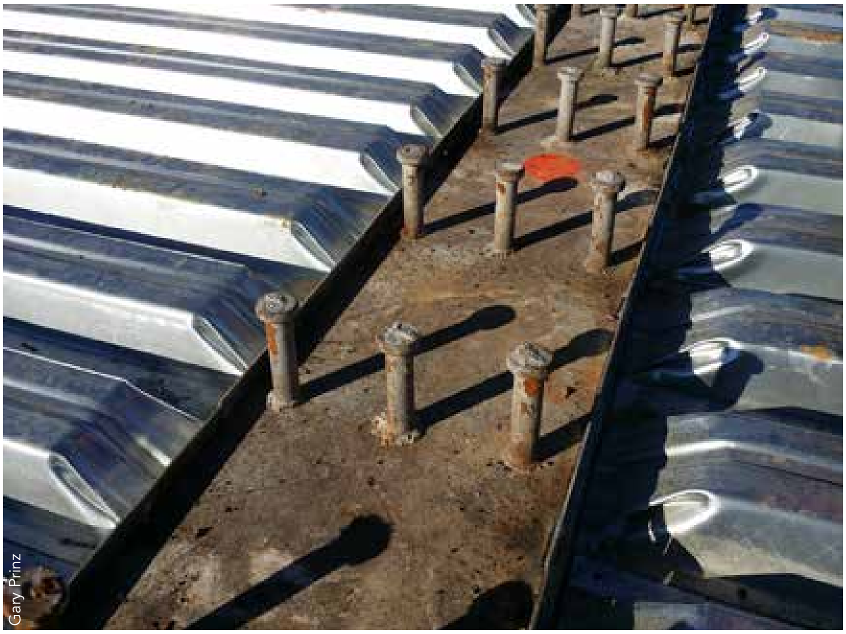
For a member containing slender elements, the capacity of the section is defined in Section 6.9.4.2.2, but the element slenderness need not be checked against a limit based on F_y; rather its slenderness need only be sufficient to be stable to a level of stress, F_{cr}, that corresponds to the member stability limits. This is a change in prior practice and a substantial benefit in the computed strength for slender elements. Implementation of these unified effective width provisions is an essential part of ongoing work that will replace the current LRFD non-composite box member provisions in the next few years.

Maximum Shear Stud Spacing. Over the course of several research projects, researchers at the University of Texas, George Washington University, the University of Arkansas and the FHWA Turner Fairbanks Laboratory have investigated the maximum shear stud spacing used for composite construction. The 24-in. limit in LRFD is historically linked to work completed by Newmark in the 1940s, which concluded that a 24 in. limit seemed reasonable. With a greater interest in precast concrete deck panels as a means of accelerated bridge construction (ABC), the 24-in. limit has become a constraint. The results of FHWA's tests on steel beams made composite with precast deck panels with pockets spaced 12 in., 24 in., 36 in. and 48 in. on center showed no discernable difference in the moment vs. deflection response of the specimens. All tests were carried out on 24-in.-deep beams.

The George Washington University tests yielded similar results. As a result, the spacing limit has been relaxed. The new provisions of Article 6.10.10.1.2 allow for shear studs to be placed up to 48 in. on center for beam depths of 24 in. or greater. For beams shallower than 24 in., the current 24-in. spacing limit is retained since that limit is consistent with test results from prior researchers.

Steel Detailing for Fit. Continuing with the incremental introduction of fit and detailing considerations into LRFD, various definitions have been added describing terms, such as no load fit (NLF), steel dead load fit (SDLF) and total dead load fit (TDLF) and other terms related to fit, girder, diaphragm and cross-frame detailing. The designer's attention is drawn to the impact of staged construction on girder deflection and fit via changes to Article 6.7.2. One of the more important changes is that Article 6.7.2 now defines a series of conditions for which the contract documents are required to stipulate the anticipated fit condition. Combinations of skew, span length and girder radius are provided for which the fit condition must be provided on the plans. A detailed commentary is provided as is a method





Gary Prinz

Josh Rodems

Figure 3. AASHTO Figure C6.7.4.2-1: Beneficial staggered diaphragm or cross-frame arrangement for a straight bridge with parallel skew.

Maximum shear stud spacing has been the subject of several recent research projects.

to reduce the cross-frame design forces for structures in which a total dead load fit is chosen.

A brief summary and a more comprehensive document addressing the various aspects of girder fit in straight, straight-skewed and curved steel girder bridges can be found at **www.steelbridges.org**.

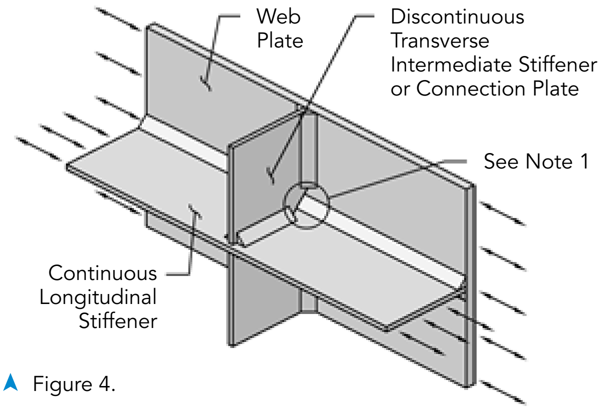
Cross-Frame Forces in Skewed Bridges. In the new commentary to Article 6.7.4.2, the effects of skew are further explored with respect to the placement of cross frames in highly skewed structures. The commentary builds on recent research conducted at Georgia Tech on the forces in skewed steel bridges. The commentary describes a practice of omitting cross frames near highly skewed corners, staggering cross frames in straight bridges so as to minimize the stiffness of the bridge along transverse lines and providing a recommended offset of the first cross frame from a skewed support in highly skewed structures (Figure 3 provides an example). Note that every other cross frame in the figure is also intentionally omitted within the bays between the interior girders. This is done to reduce the total number of cross frames required within the bridge as well as to reduce the overall transverse stiffness effects.

Constraint-Induced Fracture: Updates on Detailing. Article 6.6.1.2.4 addresses the detailing of structures to minimize the possibility of constraint induced fracture in steel structures. The guidance has been updated to clarify a minimum ½-in. gap between adjacent weld toes and to provide enhanced graphics illustrating

Josh Rodems

the preferred detailing at the intersection of longitudinal stiffeners and lateral connection plates with transverse intermediate stiffeners and bearing stiffeners. Two examples from the updated figures are provided (see Figure 4). The first example demonstrates that in areas of tension or reversal, when a longitudinal and a transverse stiffener intersect, the longitudinal stiffener should be kept continuous to improve the fracture and fatigue performance. The second demonstrates the preferred detailing at the intersection of a bearing stiffener and a lateral connection plate in a region subject to compression only. In this case, since the web is in compression at the connection plate, fracture is precluded and it is acceptable to cope the connection plate to fit around the continuous bearing stiffener.

Global Stability of Narrow I-Girder Bridge Units. The 8th Edition includes an equation that serves as an indicator as to when global stability of the spans of two- and three-girder systems may be critical as a failure mode when in their non-composite condition during the deck place-



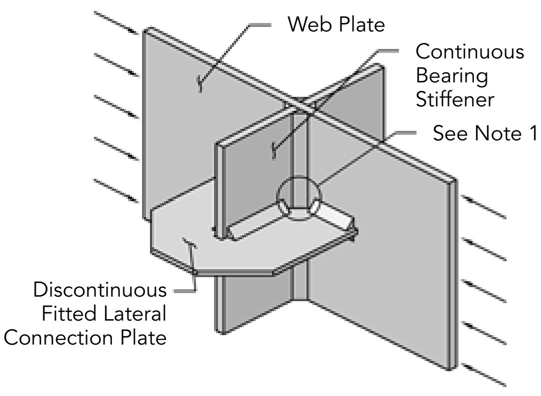


Figure 4.

ment operation. This is found in Article 6.10.3.4.2, which has been renamed "Global Displacement Amplification in Narrow I-Girder Bridge Units." The recommendations in this article, resulting from research at the University of Texas, are intended to avoid excessive amplification of the lateral and vertical displacements of narrow, straight, I-girder bridge units, with no external bracing or flange-level lateral bracing during the deck placement operation or at any other time before the concrete deck has hardened. The global buckling mode in this case refers to buckling of the bridge unit as a structural unit generally between permanent supports, and not buckling of the girders between intermediate braces. The provisions are not intended for application to I-girder bridge spans in their full or partially composite condition, or to I-girder bridge units with more than three girders. The current equation for the elastic global lateral-torsional buckling resistance of the span acting as a system, M_{gs}, is shown below, with the introduction of a C_{bs} factor in the 8th Edition that reflects the moment gradient conditions of the structure:

$$M_{gs} = C_{bs} \frac{(\pi^2 w_g E)}{L^2} \sqrt{(I_{eff} I_x)}$$

The value of C_{bs} is 1.1 for simple-span units and 2.0 for fully erected continuous-span units. For continuous units in the partly erected condition, the 1.1 value for simple spans is conservatively used. In addition to the introduction of the C_{bs} term, the 8th Edition also increases the percentage of this moment that can be applied to the system prior to needing to introduce measures such as lateral bracing systems or resizing the beams to provide a higher degree of stiffness. The new provisions allow the applied factored moment to reach 70% of M_{gs} as a limiting value. Cautionary guidance is given that the behavior of narrow straight girder systems should not assume to apply to narrow curved girder systems; these systems require a more careful examination of displacement and stress amplification when external bracing or flange level lateral bracing is not provided.

Updates to Bolted Connection Provisions. The shear strength of bolts with threads included and excluded from the shear plane has been increased to reflect a slight increase in the stated value of the ratio of the yield to tensile strength of high-strength bolts (raised from 0.6 to 0.625), as

▲ The *Guide Specifications For Wind Loads On Bridges During Construction* introduces tools to evaluate the effects of wind loads on bridges of all types under construction.

▲ ▼ The flow of wind around a completed structure is fundamentally different than it is around an open frame during construction.

The new specification provides long-needed guidance for contractors and their engineers who need to evaluate strength and stability during critical stages of erection.

well as to reflect newer information on the non-uniform load sharing in lap splice tension connections (correction raised from 0.8 to 0.9). This results in the common shear strength of a bolt being raised from a traditional value of $0.6 \times 0.8 = 0.48A_bF_u$ to a new value of $0.625 \times 0.9 = 0.56A_bF_u$—an increase of 16.7% for a typical high-strength bolt with the treads excluded from the shear plane. A similar increase is provided for threads included in the shear plane. However, due to the increase in the non-uniformity factor from 0.8 to 0.9, a revision in the long-connection correction factor was needed. The existing provision that requires an additional 0.8 factor to be applied for lap-splice tension connections longer than 50 in. has been revised to a correction factor of 0.83 for connections longer than 38 in.

Additional changes to the bolted connection provisions include slight changes to the slip coefficient table and the introduction of a new Class D surface condition having a slip coefficient value of 0.45, slightly below the 0.5 Class B value. Some coating systems are not able to meet the 0.5 Class B slip coefficient and as a result, were then required to have the bolts designed using the much lower 0.33 coefficient. Introduction of the new Class D surface condition provides a slight reduction in capacity, but reflects the actual performance of these coating systems and their influence on bolt capacity.

A new article on high-strength structural fasteners, 6.4.3.1, is now included to introduce the new ASTM F3125 standard for high-strength bolts, which combines ASTM A325, A325M, A490, A490M, F1852 and F2280. ASTM will no longer maintain the many specifications related to high-strength bolts, nuts, washers, indicators, etc. All bolting components are now included in the new F3125 standard. In terms of specification, what was once called an A325 bolt will now be referred to as ASTM F3125, Grade A325 bolt.

Guide Specification for Wind Loads

In 2015, interim revisions to the 7th Edition of the *Specifications* introduced new wind load provisions based on a "three-second-gust" procedure for determining the design wind speeds. This replaced the prior definitions based on the "fastest mile" approach. In parallel, new wind load provisions for temporary loading of bridges during construction were also being prepared. In 2016, these provisions were successfully balloted and have been published as a new *Guide Specification for Wind Loads on Bridges During Construction*. They reflect that the flow of wind around a completed structure is fundamentally different than on an open frame during construction. The exposure period for construction also differs greatly from that for completed bridges. Completed bridges need to be designed for maximum wind loads that they might experience over their lifetime, while the critical construction period for a typical girder bridge might be as short as a few weeks. This correlates to a much different probability of exceedance for short exposure wind loads. All of these factors have been considered in the new *Guide Specification*.

The publication is based on factors that relate to the elevation of the structure, gust factors and drag on open framing systems. Concerning drag, unique loadings are specified for windward, interior and leeward beams in a cross section. The gust factors also reflect the type of girder; for steel bridges, both I-girder and tub girder cross sections are addressed along with a correction for characteristics of girder spacing versus girder depth. This new specification provides long-needed guidance for engineers who design bridges, as well as for contractors and their engineers who need to evaluate strength and stability during critical stages of erection.

These are just a few of the important changes in the 8th Edition of the AASHTO *LRFD Specifications* that will influence the design, detailing and construction of steel bridges. The intent of all of these changes is to integrate the latest research, clarify the provisions for steel structures when needed and provide engineers the most current state of the practice for safe and efficient steel bridges.