Steel Bridge Resources

2019

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Stronger.
Steel.

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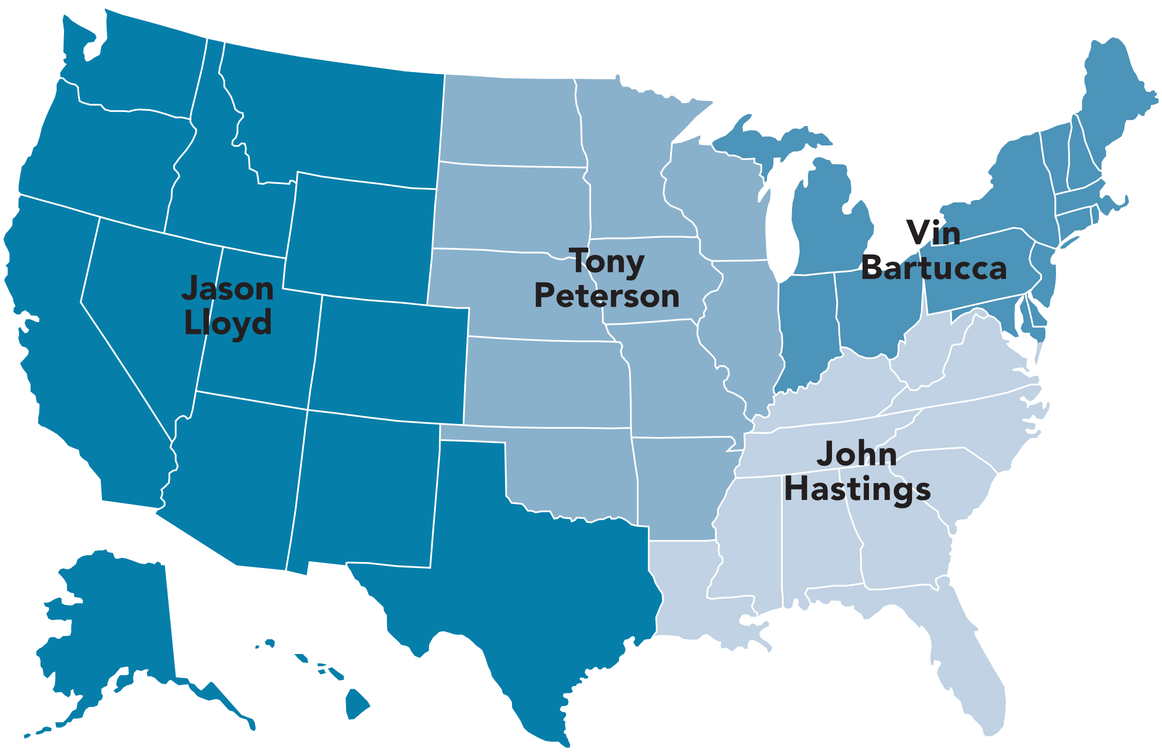
- Provide free design and technical resources
- Assist in connecting designers with fabricators
- Leverage steel industry expertise
- Meet in-person to discuss steel solutions
- Provide steel marketplace updates
- Make technical presentations (providing CEUs or PDHs)
- Develop free conceptual steel bridge solutions
- **Deliver efficient and economical steel bridge designs!**



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Average Mill Price of A709-50W

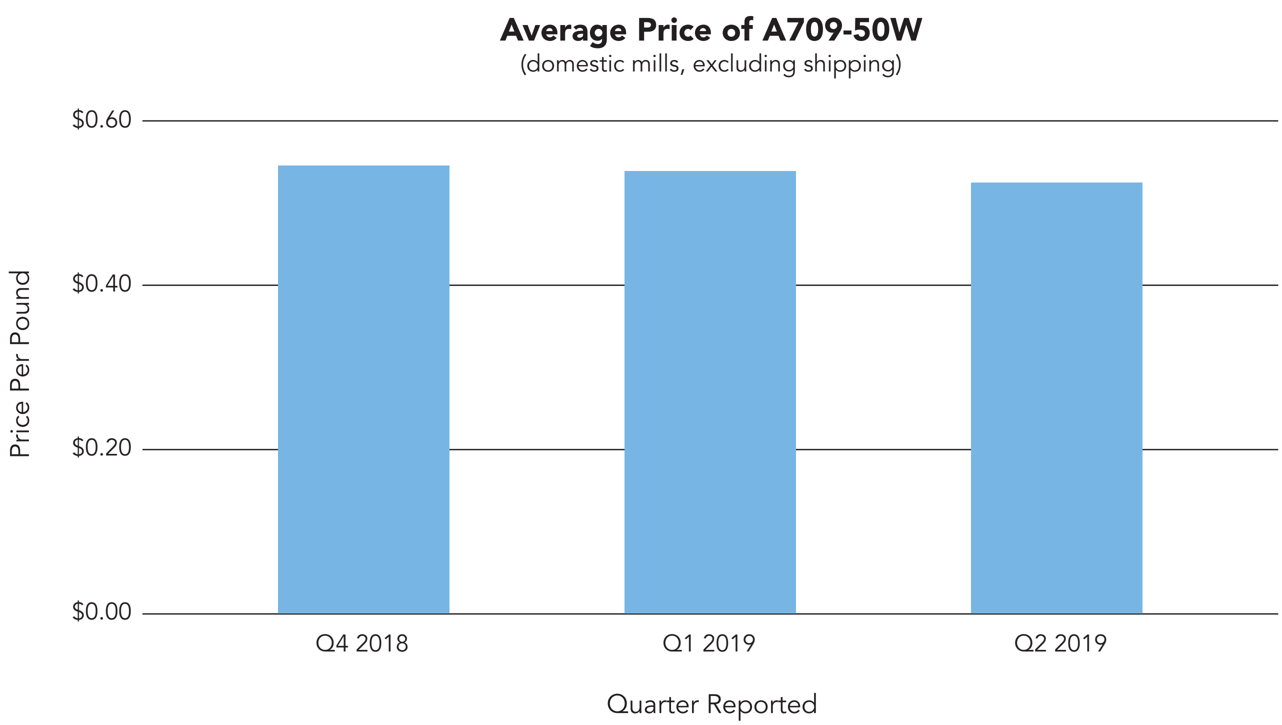
(1½ in. thick × 96 in. wide × 636 in. long)

SURVEY QUESTION

Please provide the average price that you are currently charging a customer (excluding shipping costs) for the following plate size and grade:

Grade: A 709–50W
Size: 1½ in. thick × 96 in. wide × 636 in. long

Quarter Reported	Cost CWT	Cost Per Ton	Cost Per Pound
Q4 2018	$54.53	$1,090.60	$0.55
Q1 2019	$54.23	$1,084.60	$0.54
Q2 2019	$51.73	$1,034.60	$0.52



Raw material pricing presented in this chart is a small snapshot of a limited time and is not representative of long term historical trends and future trends.

B002-19

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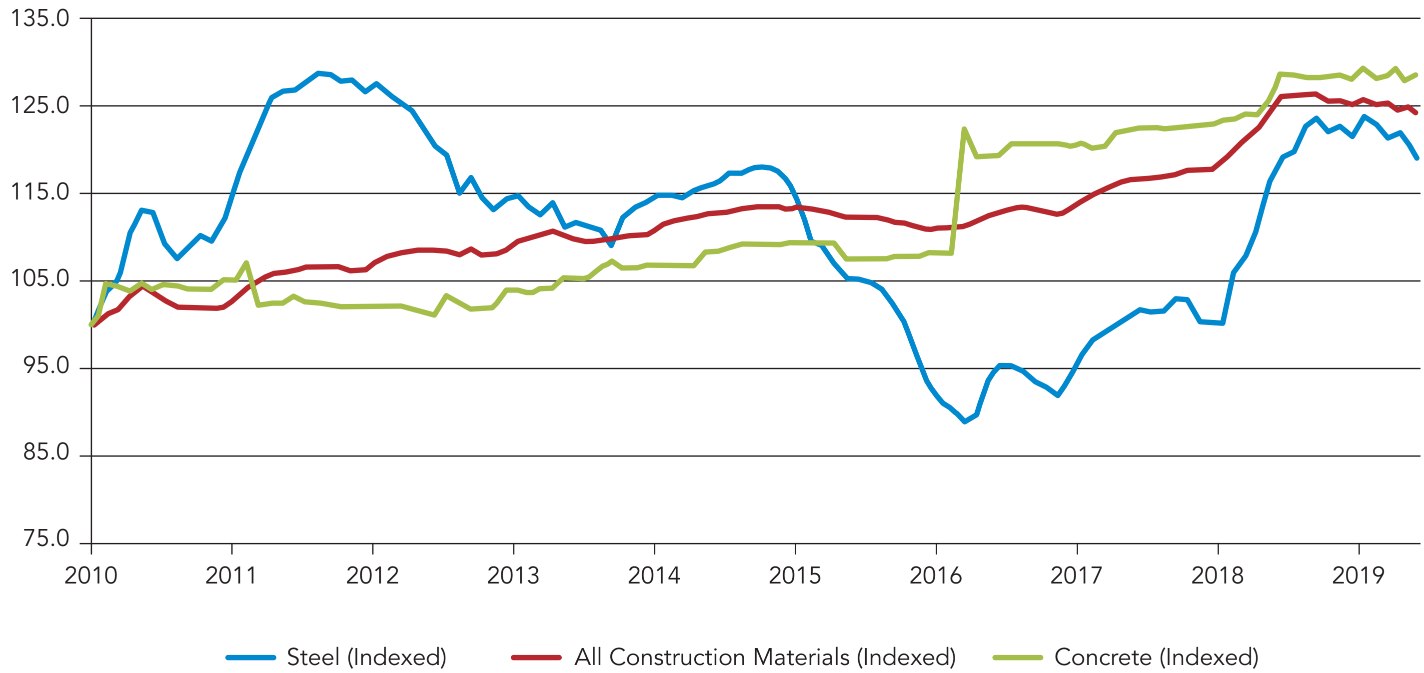
Historic Material Costs

The following chart is based solely on publicly available data provided by the U.S. Bureau of Labor Statistics and is a summary of relative pricing indices of various construction materials.

Relative Construction Producer Price Index



Key Takeaways

- While the price index for steel has increased over the last year or two, it is important to note that the long term inflation rate of steel has been less than other materials.
- The recent increase in steel prices not only has an affect on the cost of steel bridges, but it also has an affect on the cost of concrete bridges since internal steel reinforcing is critical to the performance of concrete bridges.

Average Annual Inflation Rate Since 2010

All Construction Materials = 2.4%
Steel = 2.1%
Concrete = 2.8%

Source: U.S. Bureau of Labor Statistics (St. Louis Federal Reserve) – through April 1, 2019
updated on May 17, 2019.

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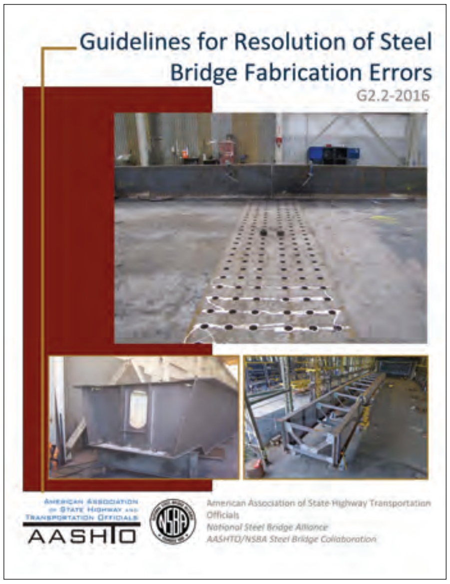
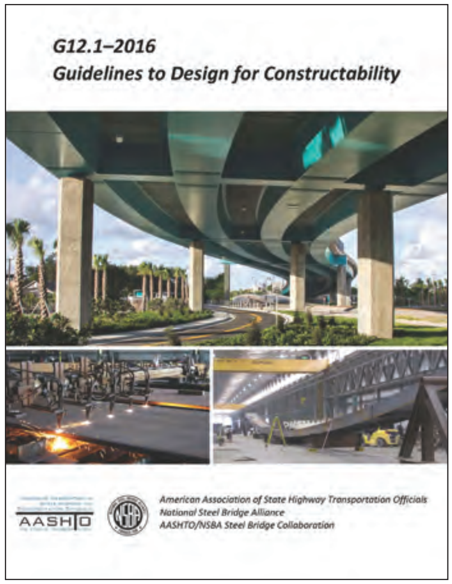
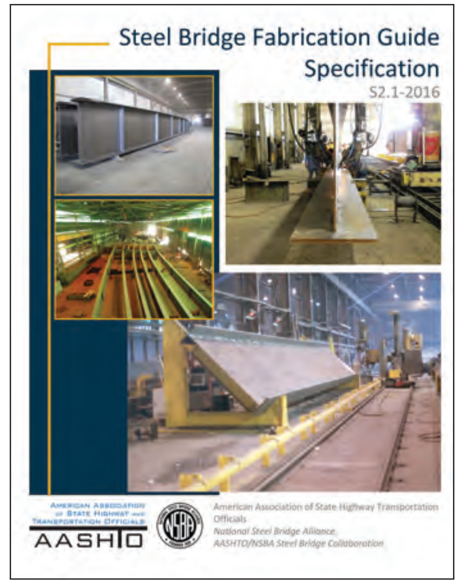






AASHTO/NSBA COLLABORATION

CELEBRATING 20 YEARS

Achieving the highest quality and value in steel bridge design and construction through the standardization of design, fabrication, and erection processes.

GET INVOLVED!

The collaboration is comprised of volunteer task groups. Contact NSBA to see how you can participate and contribute. Visit **www.steelbridges.org/collaborationstandards** for more information.

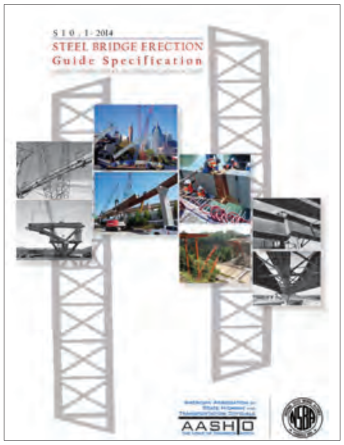
STANDARDS

The collaboration writes documents in two forms: Specification "S" documents and Guide "G" Specification documents. "S" documents are written in spec language and are intended to be adopted in whole as part of the contract documents. "G" documents are written as references to be used during the design, fabrication and construction processes.

All documents represent a consensus of best practices developed by industry. Referencing the document allows for a common language across all stakeholders.

Available Standards

- Steel Bridge Fabrication
- Application of Coating Systems
- QA/QC
- Steel Bridge Erection
- Design Detailing
- Steel Girder Analysis
- Resolving Fabrication Errors
- Bearing Design and Details
- Designing for Constructability

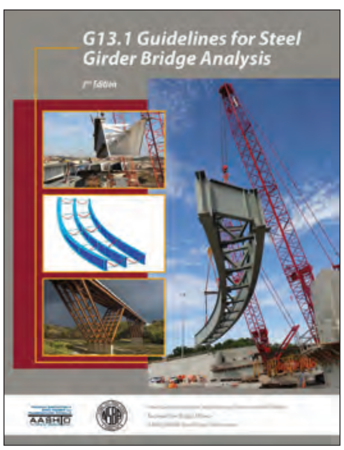


S10.1-2014

Steel Bridge Erection Guide Specification

Category:
Guide Specification
Applicable Groups:
Contractors, Owners, Engineers

S10.1 provides guidance for the safe and economical erection of steel girders. The document covers all aspects of erection, from transportation and jobsite storage of girders, to field bolted connections, to inspection and repairs.



G13.1-2014

Guidelines for Steel Girder Bridge Analysis

Category:
Guide
Applicable Groups:
Engineers

G13.1 is an update to the 2011 document of the same name. The document provides a comprehensive treatment of issues in steel girder analysis. The document includes guidance on the appropriate level of analysis based on geometric complexity.

G2.2-2016

Guidelines for Resolution of Steel Bridge Fabrication Errors

Category:
Guide
Applicable Groups:
Fabricator, Contractor, Owner, Engineer

G2.2 addresses common fabrication issues. From a misaligned bolt hole to a miscut member, this document discusses common issues and how they should be resolved. Many times, the fix can be more costly and detrimental than the original error. This document provides the necessary guidance to ensure an economical fix that preserves the long term resilience of steel girders.

S2.1-2016

Steel Bridge Fabrication Guide Specification

Category:
Guide Specification
Applicable Groups:
Fabricator, Contractor, Owner, Engineer

S2.1 was written to ensure high quality and value during the fabrication process. S2.1 is written in specification langauge such that it can be referenced in the contract documents.

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Steel Span Weight Curves

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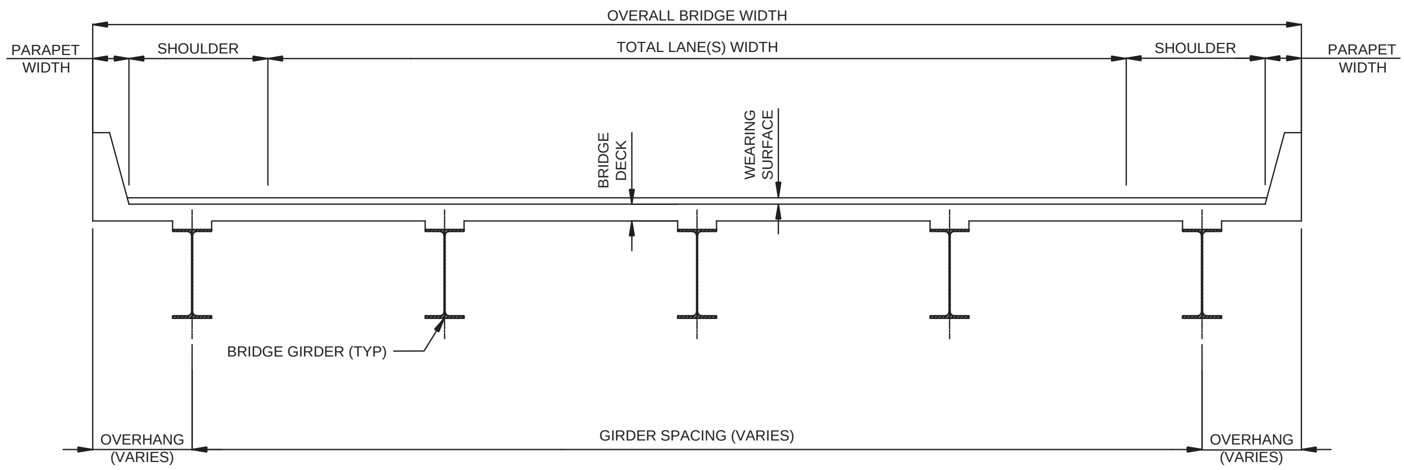
Printed in the United States of America

These graphical design aids are intended to be used during the preliminary phases of design for evaluation of alternative structures to quickly determine the relative costs of various girder spacings and number of girder spans. The curves have been constructed from cost-effective conceptual solutions that NSBA has prepared. They represent the predicted pounds of steel per square foot for various span lengths and girder spacings for single spans, two spans, and three or more spans.



TYPICAL BRIDGE SECTION
N.T.S.

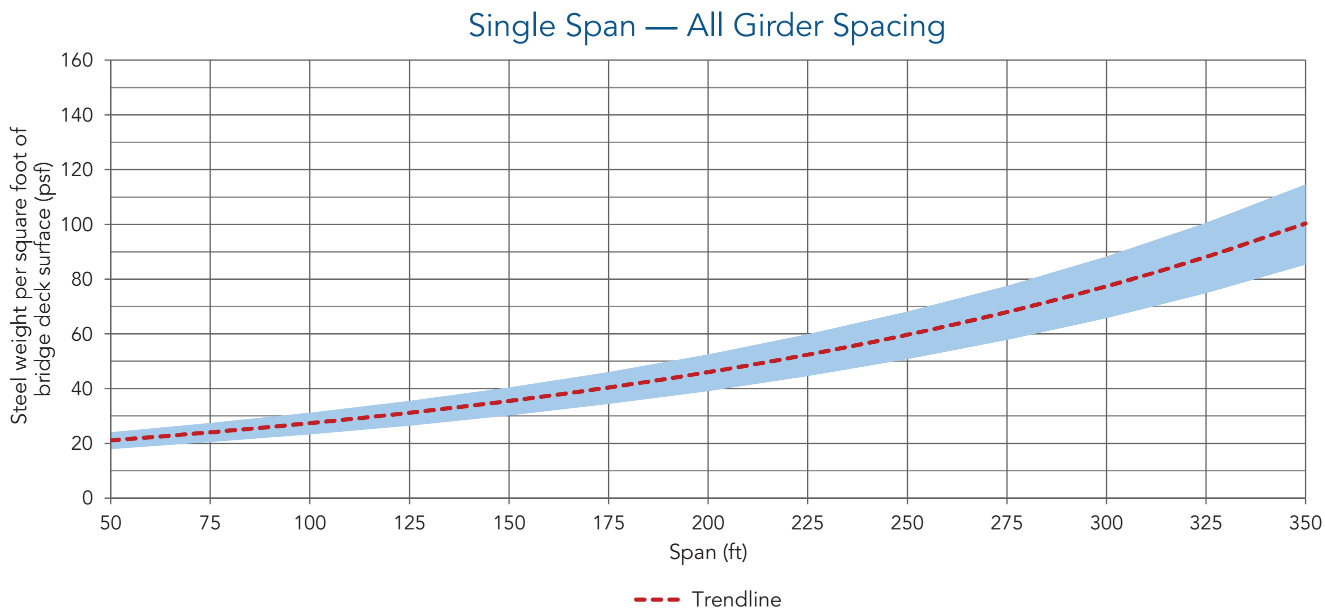
Design Parameters

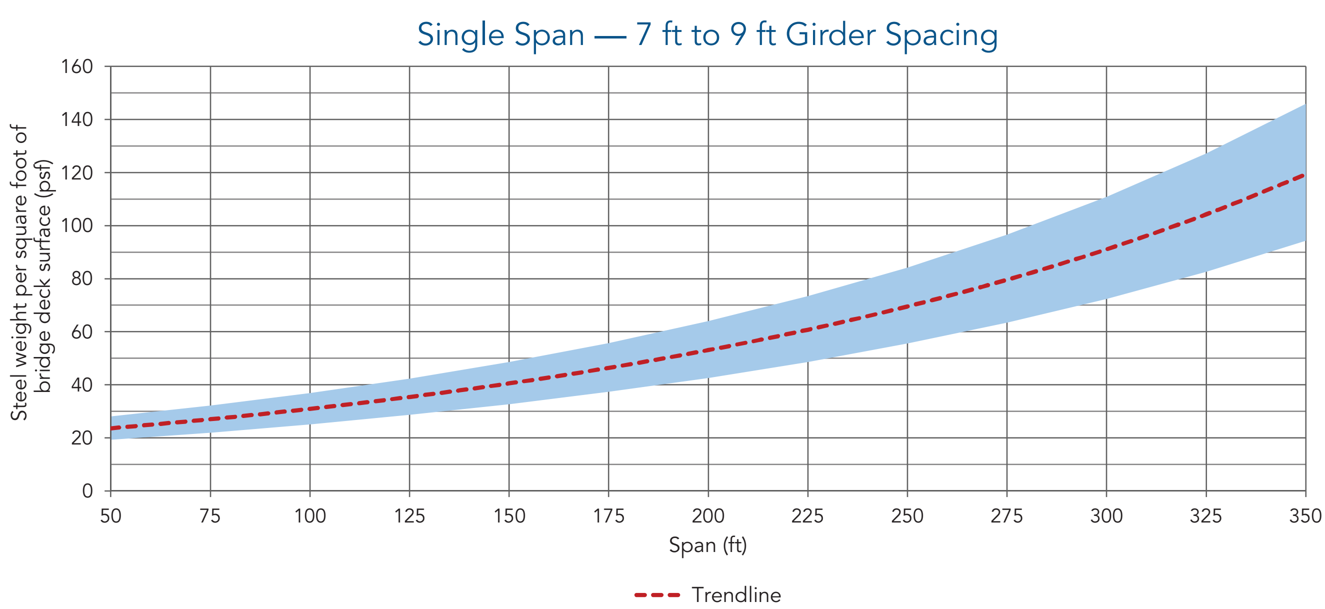
These curves represent predicted pounds of steel per square foot derived from data from more than 800 NSBA conceptual solutions optimized for economical bridge designs. Every bridge is unique and other factors can influence the design, resulting in values outside the ranges shown in these curves. Care should be taken to ensure that an appropriate analysis is conducted. The figure below represents a typical bridge section view.

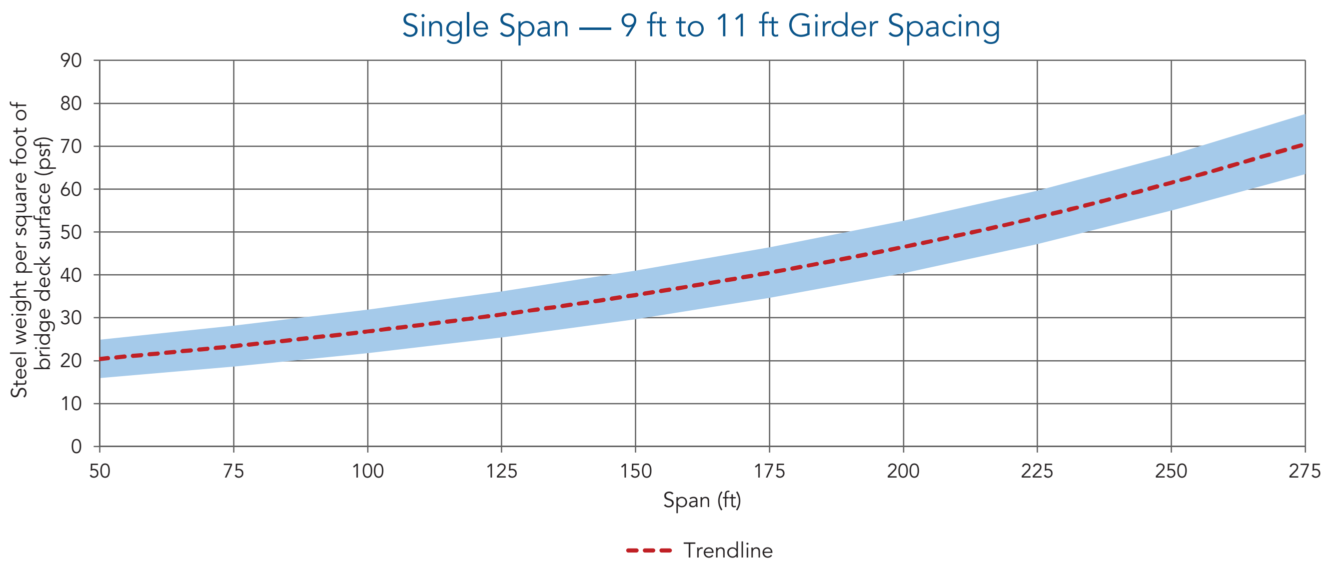
Assumptions

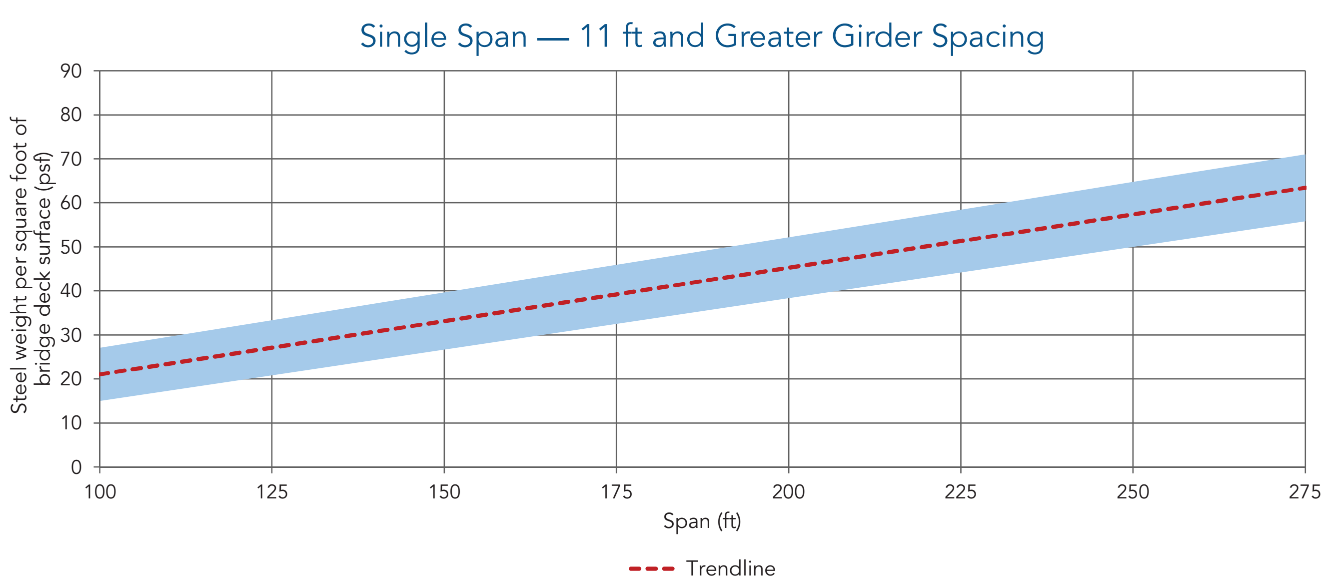
- Section is designed as composite.
- Girders are assumed continuous.
- Design considers fatigue loading.
- Span lengths are based upon the maximum span distance. Where more than one span exists, use the maximum span to determine span weight.
- Trend line value represents the line of best fit based upon the discrete values.
- Shaded area represents deck areas in which 68% of the sample bridges are located.
- Both curved and straight girders are included in the curves.

single-span bridges



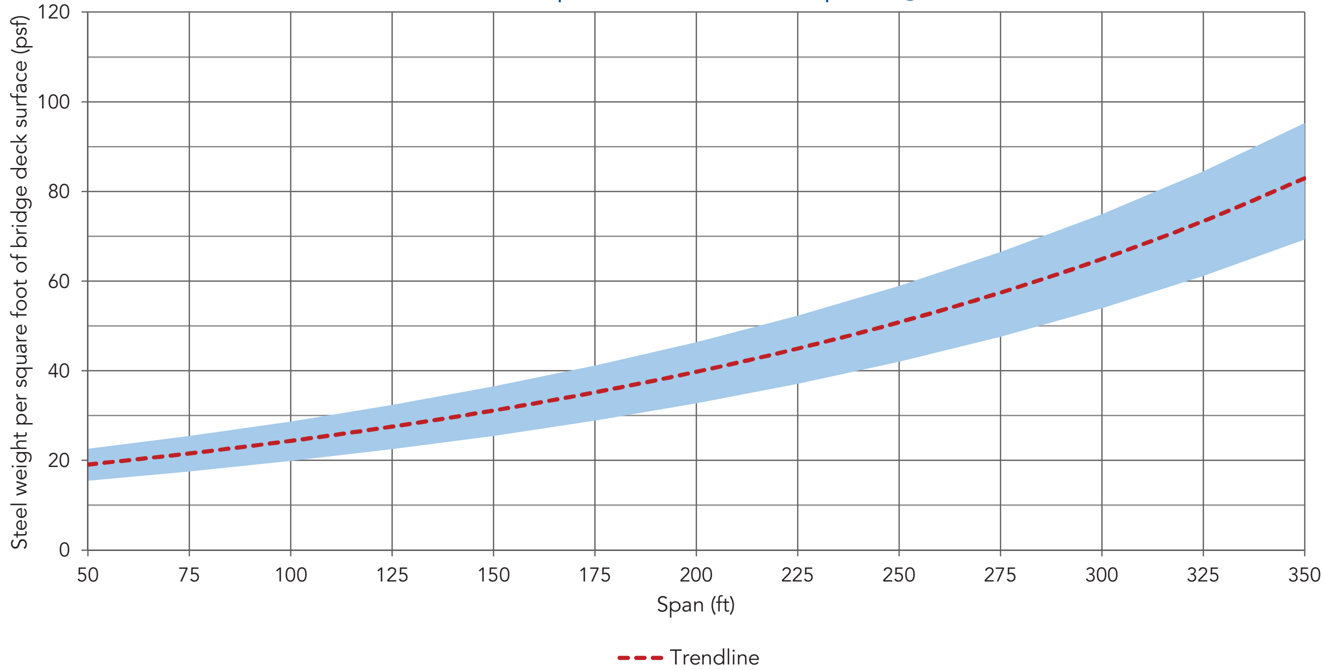



Single Span — 9 ft to 11 ft Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
0
10
20
30
40
50
60
70
80
90
50
75
100
125
150
175
200
225
250
275
Span (ft)
Trendline

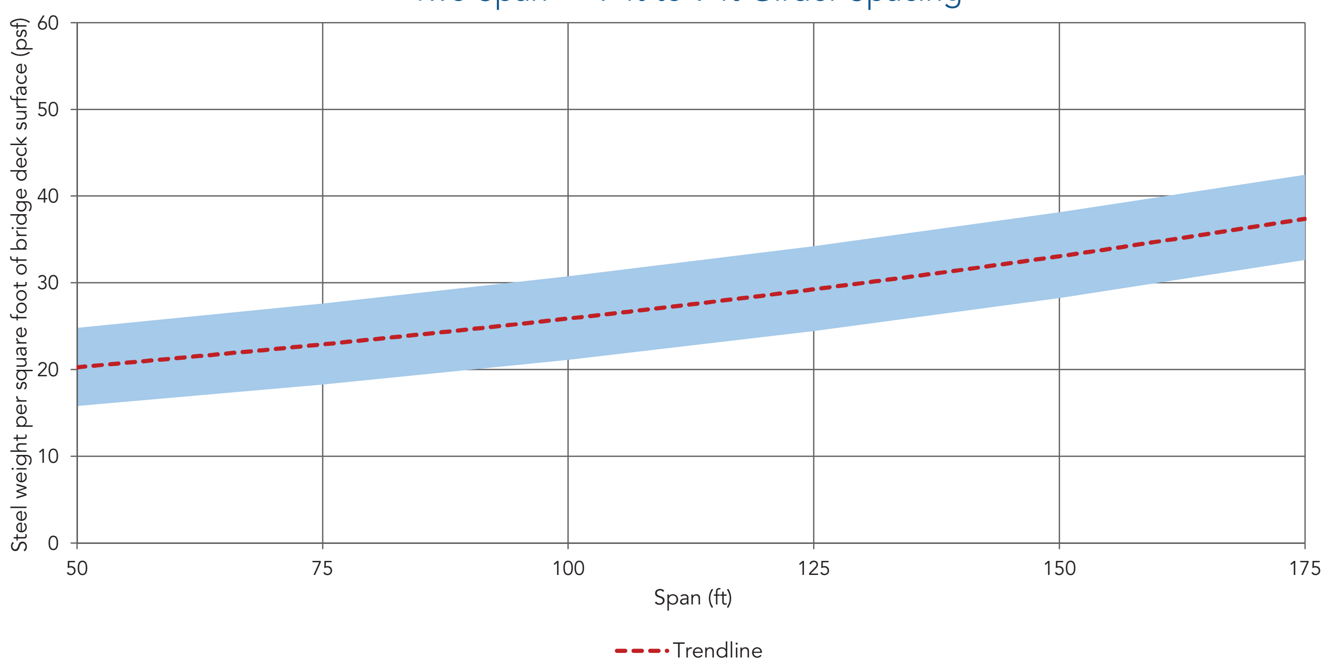

Single Span — 11 ft and Greater Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
0
10
20
30
40
50
60
70
80
90
100
125
150
175
200
225
250
275
Span (ft)
Trendline

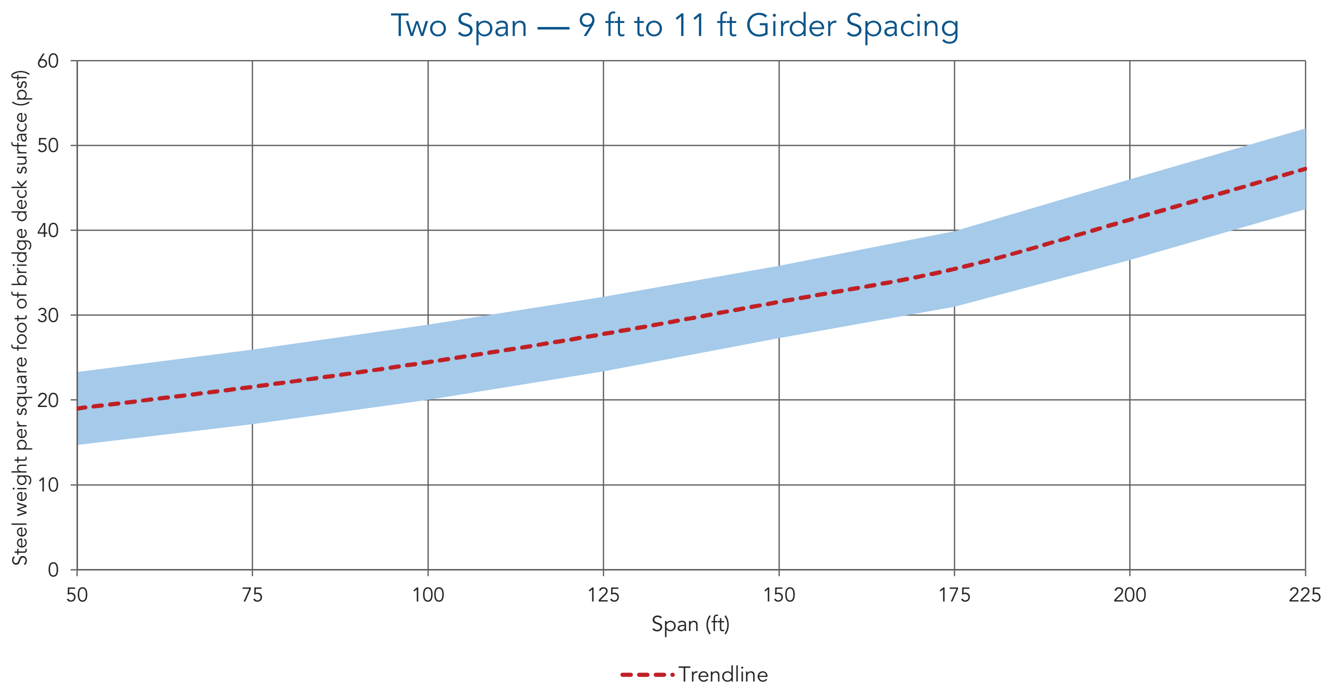
two-span bridges

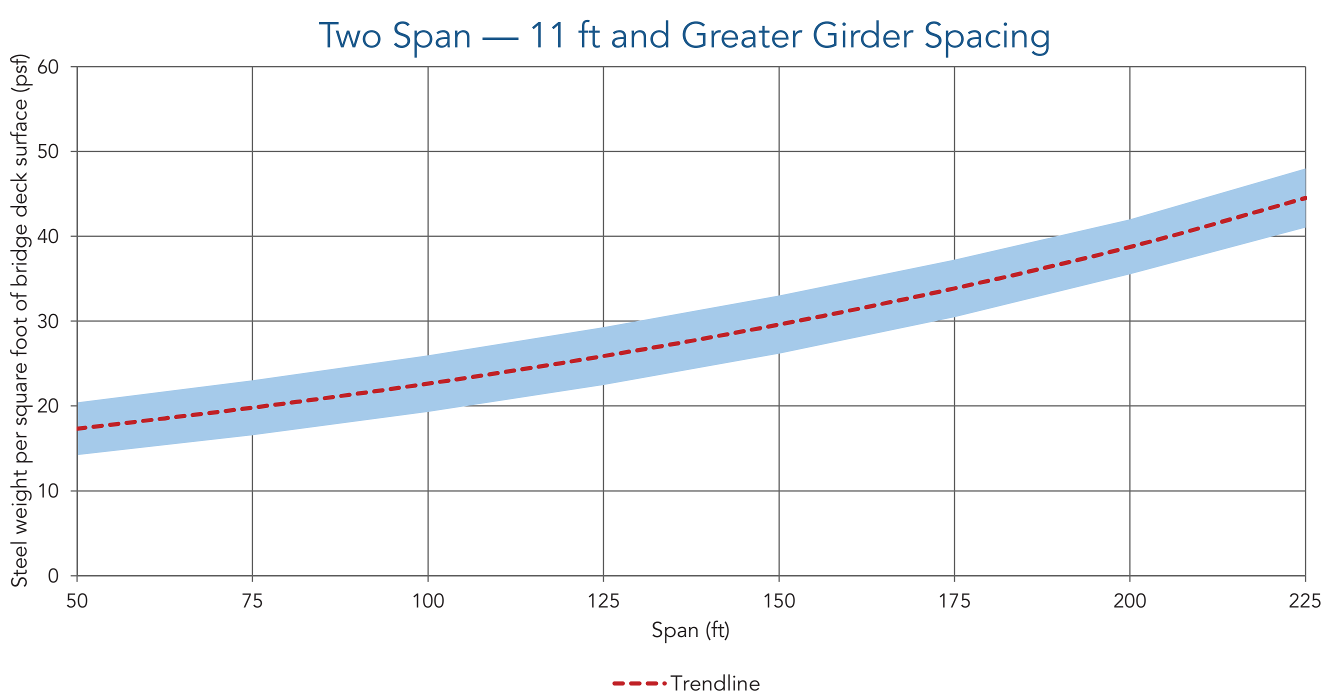
Two Span — All Girder Spacing



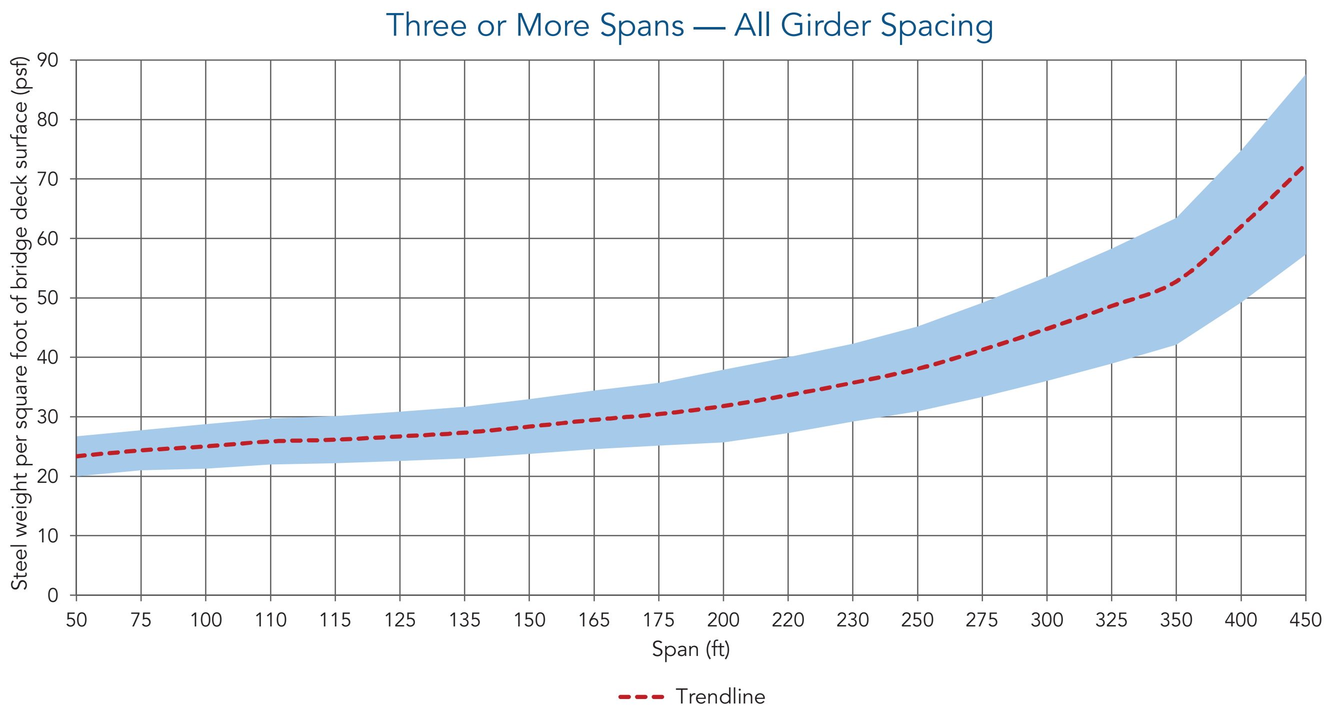
Two Span — 7 ft to 9 ft Girder Spacing

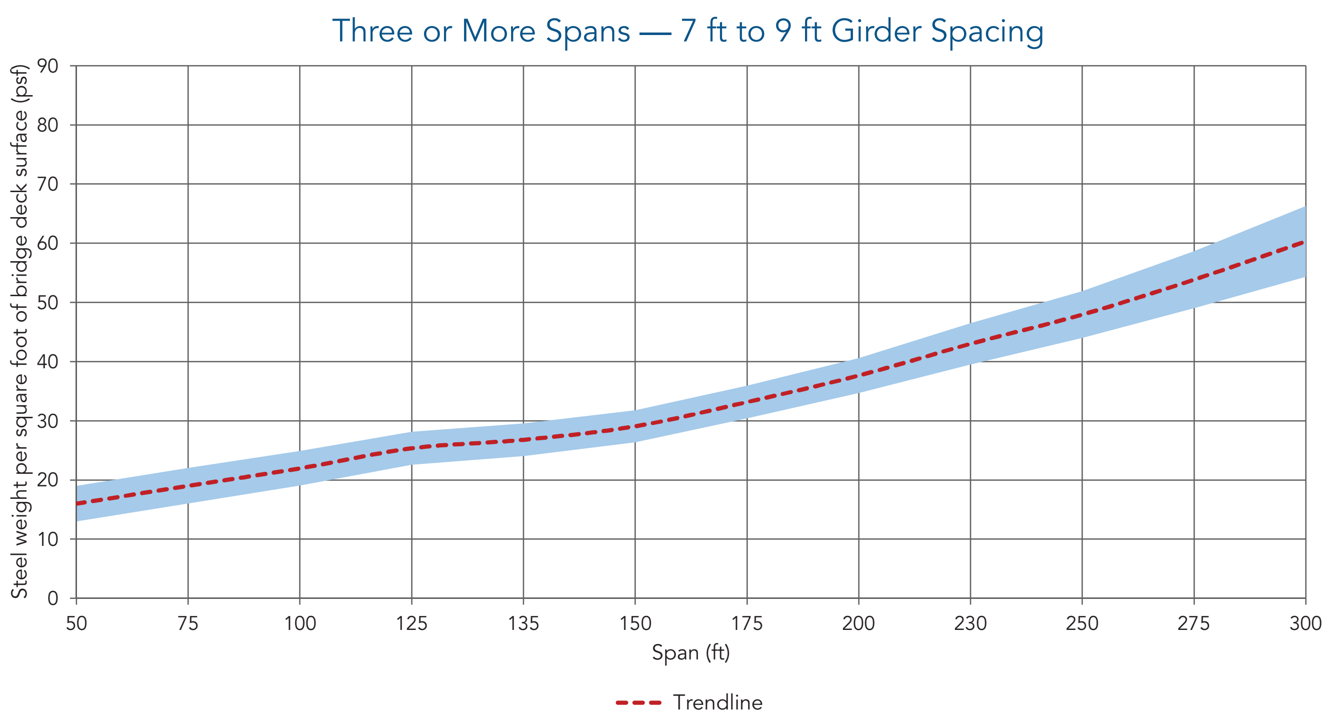


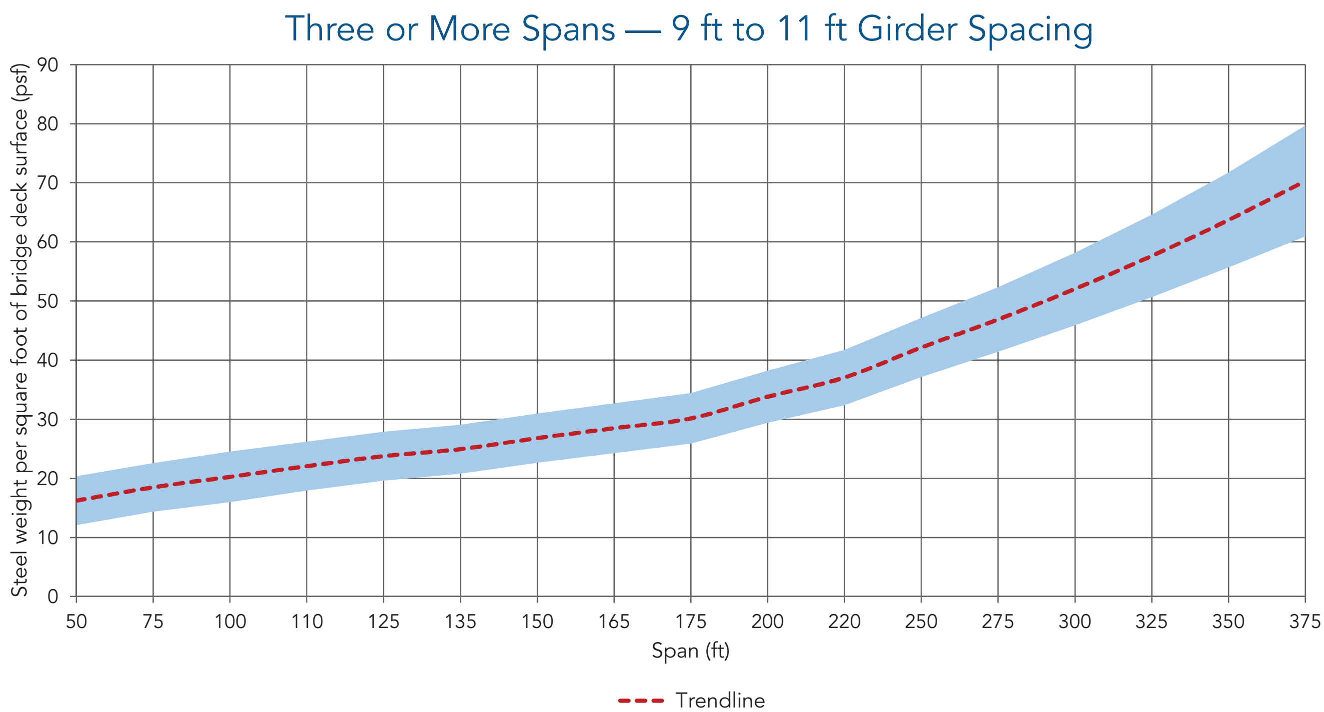

Two Span — 9 ft to 11 ft Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
60
50
40
30
20
10
0
50
75
100
125
150
175
200
225
Span (ft)
Trendline

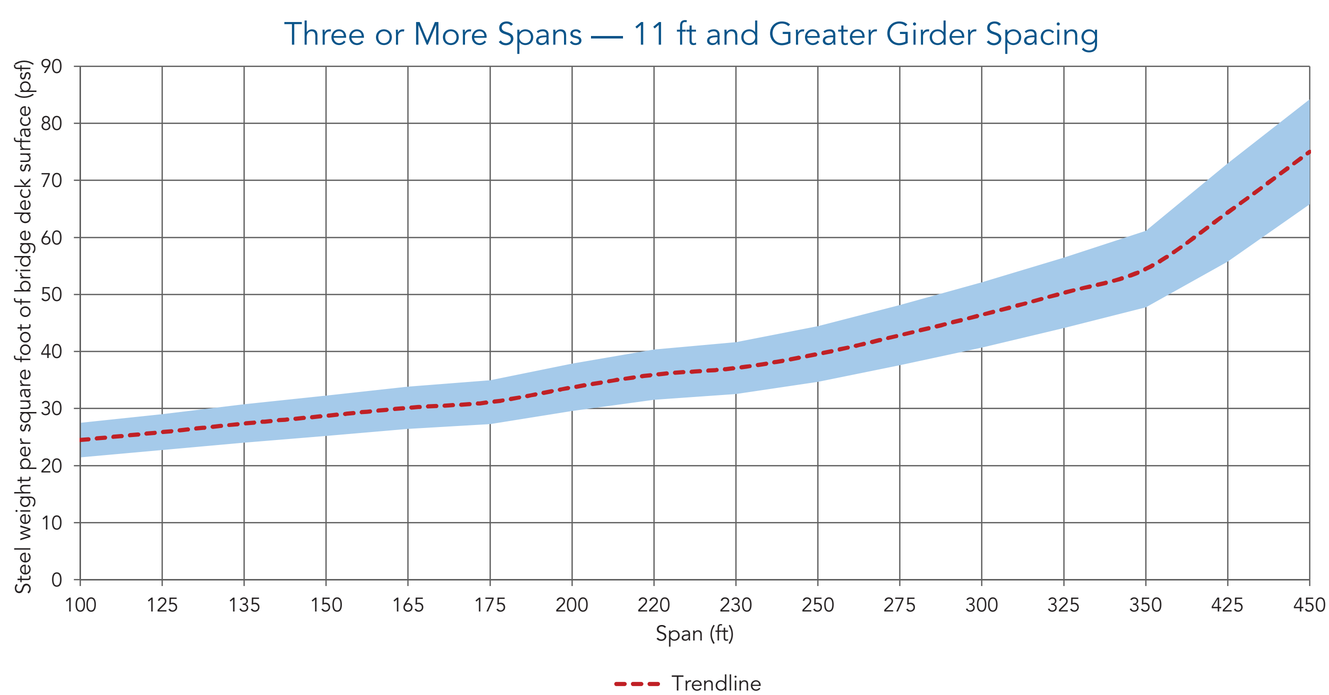

Two Span — 11 ft and Greater Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
60
50
40
30
20
10
0
50
75
100
125
150
175
200
225
Span (ft)
Trendline

three-or-more-span bridges






Three or More Spans — 9 ft to 11 ft Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
90
80
70
60
50
40
30
20
10
0
50
75
100
110
125
135
150
165
175
200
220
250
275
300
325
350
375
Span (ft)
Trendline


Three or More Spans — 11 ft and Greater Girder Spacing
Steel weight per square foot of bridge deck surface (psf)
90
80
70
60
50
40
30
20
10
0
100
125
135
150
165
175
200
220
230
250
275
300
325
350
425
450
Span (ft)
Trendline

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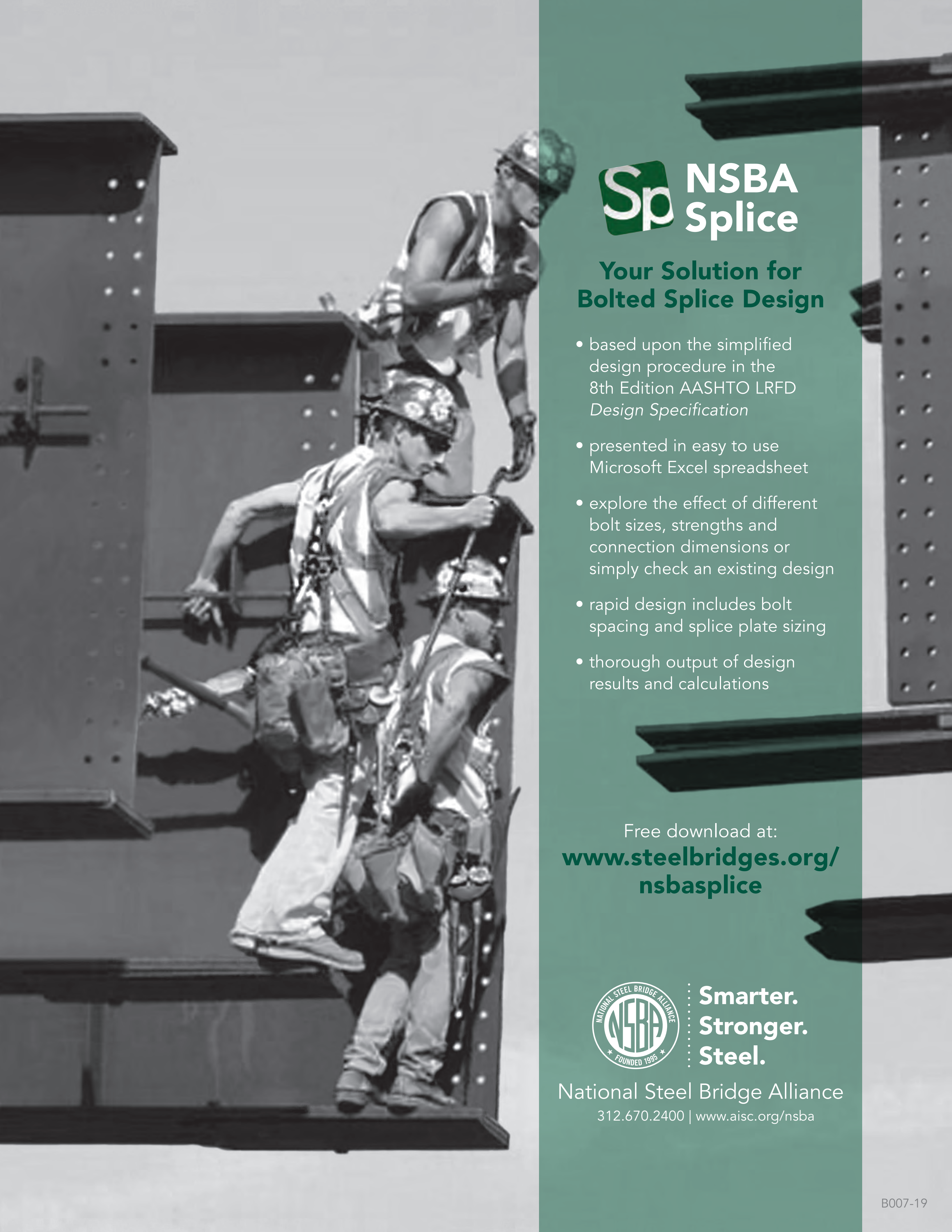
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B007-19

Steel Plate Availability for Highway Bridges

BY CHRISTOPHER GARRELL, PE, LEED AP

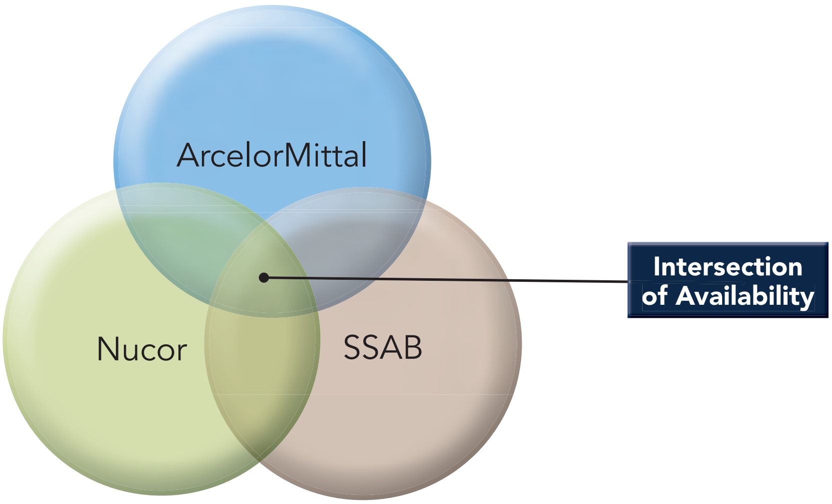

Fig. 1. The rationalization of plate availability.

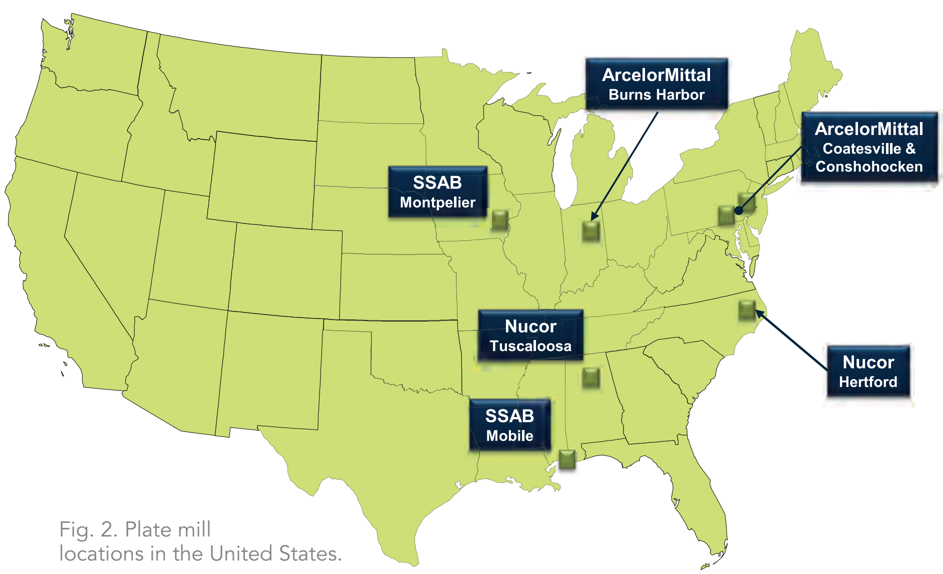

Fig. 2. Plate mill locations in the United States.

THE LENGTH AVAILABILITY for the various plate widths and thicknesses is a very common question engineers have when designing highway structures. Understanding availability of plate material while performing design iterations will ensure that the material used can be sourced from all steel mills and result in better economy for the overall bridge superstructure.

The information listed below is not intended to be an all-encompassing summary of available plates that a mill may be able to produce. It is instead intended to provide a look at plate availability across the steel mills within the United States by width, thickness, and length, as shown in Figure 1. Other widths, thicknesses, and lengths may be available from one or more of these producers. In cases where a dimension is not shown, one should consult the steel mill or a local steel bridge fabricator. For specific contact information, please contact your NSBA bridge steel specialist. Alternatively, the AISC Steel Solutions Center can assist you by phone at 866.ASK.AISC and online at **www.aisc.org/askaisc**.

The tables that follow outline availability of A709-50 and A709-50W for non-fracture critical applications only. All units are in inches unless otherwise specified.

Availability and Relative Cost

Steel plate producers in the United States are ArcelorMittal, Nucor, and SSAB. Geographically, most steel plate mills are located within the eastern third of the United States as shown in Figure 2. Despite their location, many plate providers will choose to equalize on freight or meet a competitive price depending on their target markets.

An overview of plate sizes commonly produced by domestic mills.

reprinted from Modern Steel Construction SEPTEMBER 2011
(REVISED 6/2014)

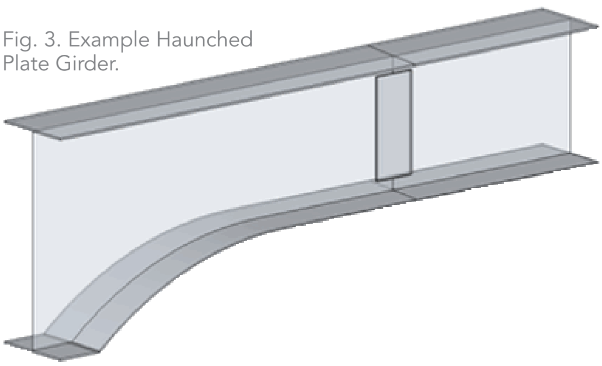

Fig. 3. Example Haunched Plate Girder.

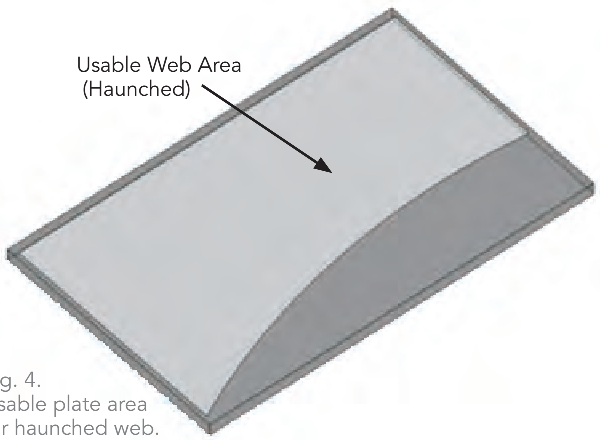

Fig. 4.
Usable plate area for haunched web.

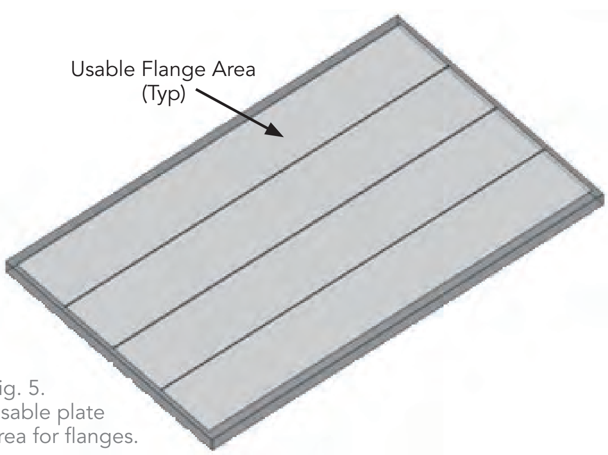

Fig. 5.
Usable plate area for flanges.

Usable Area

The source plate from which each component of a steel plate girder is cut and fabricated is referred to as the “mother” plate. Given the variability of plate squareness and the thickness of each cut, the net usable area of a mother plate is reduced. For example, consider the haunched girder section shown in Figure 3.

The depth of the haunched web is controlled by the width availability of steel plate and also the material loss due to the cutting and squaring process (Figure 4). With respect to the flanges, a fabricator will optimize the layout of the flanges in order to maximize the number that can be obtained from a single width of plate (Figure 5). However, similar to the web, the net available area is reduced by the material lost to squaring the plate and n-1 cuts (where n represents the number of flange plates that can be cut from a single mother plate). Similar to a haunch, the amount of camber a girder has also affects the net usable area of a plate.

While it is not entirely necessary for engineers to include optimization of plate usage into their design process, it is important to understand how design decisions may affect the size and number of plates purchased by a fabricator to accommodate the design. At a minimum, an engineer should be conscious of how chosen sizes compare to the length and width boundaries of available steel plate, as an inch may force a fabricator to the next larger available plate size. In turn, this may increase material waste and also limit availability. For reference, Table 1 summarizes approximate material loss due to the fabrication process. Note that this can vary from fabricator to fabricator, and can be dependent on their capabilities and equipment.

Table 1 **Approximate material loss due to squaring and cutting during fabrication.**

	Width	Length	Notes
Web Plate	1 in. – 4 in.	1 in. – 6 in.	Material loss will increase if web is haunched or cambered.
Flange Plate	1 in. – 4 in. total plus an additional ¼ in. per burn	1 in. – 6 in.	A fabricator may choose to increase flange widths specified by the engineer from ¼ in. – ⅜ in.

A709-50 and A709-50W (Non-FC) Availability

The plate availability for ArcelorMittal, Nucor and SSAB was compiled so that the common widths and thicknesses could be tabularized. The goal of this process is to obtain steel plate thicknesses, widths and lengths that are available from all three steel plate mills. The following sections summarize the availability of A709-50 and A709-50W non-fracture critical materials, which are appropriate for the majority of the steel highway bridges being designed. As stated previously, while the capability of some steel mills exceeds what is shown, the purpose is to only summarize sizes that are available from three mills.

Table 2 **Plate thickness availability by steel mill (in inches).**

	ArcelorMittal	Nucor	SSAB
3⁄16	×		×
1⁄4	×		×
5⁄16	×	×	×
3⁄8	×	×	×
7⁄16	×	×	×
1⁄2	×	×	×
9⁄16	×	×	×
5⁄8	×	×	×
11⁄16	×	×	×
3⁄4	×	×	×
13⁄16	×	×	×
7⁄8	×	×	×
1	×	×	×
1 1⁄8	×	×	×
1 1⁄4	×	×	×
1 1⁄2	×	×	×
1 3⁄4	×	×	×
2	×	×	×
2 1⁄4	×	×	×
2 1⁄2	×	×	×
2 3⁄4	×	×	×
3	×	×	×
3 1⁄4	×		
3 1⁄2	×		
3 3⁄4	×		
4	×		

Table 3 **Plate width availability by steel mill (in inches).**

	ArcelorMittal	Nucor	SSAB
48	×		
54	×		
60	×		
66	×		
72	×	×	×
75	×	×	×
78	×	×	×
81	×	×	×
84	×	×	×
87	×		×
90	×	×	×
93	×	×	×
94	×	×	×
95	×	×	×
96	×	×	×
99	×	×	×
102	×	×	×
108	×	×	×
111	×	×	×
114	×	×	×
117	×	×	×
120	×	×	×
123	×	×	
126	×		
132	×		
138	×		

Thickness Availability

For the steel mills with information available at the time of printing, thicknesses range from 3⁄16 in. through 4 in.; note that the AASHTO LRFD *Bridge Design Specification* limits the thickness of material used for structural applications to 4 in. Available thicknesses are indicated by an "×" in a cell in Table 2, above.

Width Availability

Similarly, widths from all of the surveyed steel mills were tabularized to compare availability. A range from 48 in. through 138 in. is shown in Table 3. While wider plate is available, the number of steel mills that can produce it decreases to a single provider. Available widths are indicated by an "×" in a cell in Table 3 above.

Standard industry widths are 72 in., 96 in. and 120 in. Outside these standard widths, the ability for a mill to supply the plate may become a consideration. When possible, consolidation will be performed to minimize the number of non-standard widths, which will make steel more economical. Otherwise, a special heat sequence, which can equate to a minimum order size, may be necessary to provide plate outside the standard industry widths.

Christopher Garrell (**garrell@aisc.org**) is chief bridge engineer with NSBA.

Table 4 **Composite plate chart: Maximum length (in inches) for given plate thickness and width.**

	Plate Width								
	72	78	84	90	96	102	108	114	120
⅜	972	972	972	972	972	972	972	972	750
½	972	972	972	972	972	972	972	972	750
9⁄16	972	972	972	972	972	972	972	972	972
⅝	972	972	972	972	972	972	972	972	972
¾	1,030	1,030	1,030	1,030	1,030	1,030	1,030	1,030	1,030
⅞	1,030	1,030	1,030	1,030	1,030	1,030	1,007	954	907
1	1,030	1,030	1,030	1,030	992	933	882	835	793
1¼	1,030	1,030	907	846	793	747	705	668	635
1½	1,030	1,030	756	705	661	622	588	557	529
1¾	1,030	1,030	648	604	567	533	504	477	453
2	937	937	567	529	496	467	441	418	397
2¼	833	833	504	470	441	415	392	371	353
2½	749	749	453	423	397	373	353	334	317
2¾	681	681	412	385	361	339	321	304	288
3	624	624	378	353	331	311	294	278	264

Thickness, Width and Length Charts

The availability of different steel plate thicknesses and widths is important when making choices for plate girder cross sections; however the piece lengths and locations of splices will be affected by the length of plate that steel mills can provide. Maximum plate length from a steel mill is a function of both plate width and thickness (Table 4).

To ensure the maximum availability, the table below was developed around cases where the thicknesses and widths are available from all four steel mills. The associated lengths for each mill at each common thickness and width were then reviewed. The minimum length for the group was then used to create Table 4. While in some instances, mills can produce longer pieces, the length values shown below ensure that if one chooses from this table, a fabricator can obtain the plate from ArcelorMittal, Nucor and SSAB.

Closing

This distillation of steel plate availability may help ease part of the process of designing steel plate girder highway bridges. Further information regarding best practices can be found in the AASHTO/NSBA Steel Bridge Collaboration document "Guidelines for Design for Constructability;" this and other similar documents can be found on the NSBA website, **www.aisc.org/steelbridges**, under AASHTO/NSBA Steel Bridge Collaboration Standards.

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technical resource

Skewed and Curved Steel I-Girder Bridge Fit

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WHAT IS FIT AND WHY IS IT IMPORTANT?

The "fit" or "fit condition" of an I-girder bridge refers to the deflected girder geometry associated with a specific load condition in which the cross-frames or diaphragms are detailed to connect to the girders. Consideration of the fit condition is important because the appropriate fit decision can provide a significant benefit to the constructability and the overall performance of the bridge system.

In all bridge systems (trusses, arches, etc.) the steel components change shape between the fabricated condition, the erected condition, and the final condition. Therefore the associated relationship, or fitting, of the members also changes. When the changes are small, the fit choice can be inconsequential, but when the changes are large, the proper fit choice is essential for achieving a successful project.

Article 6.7.2 of the AASHTO LRFD *Bridge Design Specifications* (8th Edition, 2017) specifies that the contract documents should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges:

- Straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- Horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03; and
- Horizontally curved bridges with or without skewed supports and with a maximum L/R greater than 0.03.

where L is the span length bearing to bearing along the centerline of the bridge and R is the radius of the centerline of the bridge cross-section.

A fit decision always must be made so that the fabricator/detailer can complete the shop drawings and fabricate the bridge components in a way that allows the erector/contractor to assemble the steel and achieve a desired geometry in the field. The fit decision also affects design decisions regarding the rotation demands on the bearings as well as the internal forces for which the cross-frames and girders must be designed. The fit condition generally should be selected to accomplish the following objectives, in order of priority: 1. facilitate the construction of the bridge; 2. offset large girder dead load twist rotations and corresponding lateral movements at the deck joints and barrier rails, which occur predominantly at sharply skewed abutment lines; 3. in straight skewed bridges, reduce the dead load forces in the cross-frames or diaphragms and the flange lateral bending stresses in the girders, and in horizontally curved bridges, limit the magnitude of additive locked-in dead load force effects.

The question, then, is in what condition should an I-girder bridge be detailed to fit? Certainly, the final condition is of great interest: to perform effectively in service, girders and cross-frames need to be in place, properly connected and properly supporting the roadway and traffic. Therefore, one might infer that bridges should be detailed simply to fit in their final constructed condition. For some bridges fitting the cross-frames to the final condition is fine and indeed may be the best choice; however, for others, fitting to the final condition significantly increases the internal cross-frame forces and can potentially make the bridge unconstructable. For every bridge, the fit condition must be selected to effectively manage the structure's constructed geometry and internal forces, and to facilitate the construction of the bridge.

It should be noted that, in practice, I-girder bridge fit is accomplished by the choice the detailer makes in setting the "drops" for the cross-frame and connection plate fabrication. The drop is defined as the difference in elevation on either side of a cross-frame. Since the fit decision directly influences the cross-frame fabricated geometry, as well as the bridge constructability and subsequent internal forces, the fit condition ideally should be selected by the engineer, who best knows the loads and capacities of the structural members. To facilitate an informed decision regarding detailing and constructability, the engineer can consult with experienced fabricators, and/or erectors prior to completing the contract documents.

Common Fit Conditions

The fit of an I-girder bridge is influenced by the difference in deflection between the sides of the cross-frames: the greater the skew, the sharper the curve, the greater the variation in the girder lengths, and the greater the span lengths, the greater this differential deflection will be. In fact, a quick way to evaluate potential constructability issues is to note the differences in the deflections across the width of the bridge at each stage of loading.

Given that dead loads cause deflections, and differences in girder deflections affect fit, it follows that the common fit conditions are associated with different bridge dead load conditions. These are shown in Table 1. Engineers tend to be more familiar with names associated with loading conditions; fabricators tend to be more familiar with terms associated with stages of construction. The setting of drops discussed in the "Practice" column of the table refers to the detailer establishing the relative position of each cross-frame to each girder.

This is a stand-alone summary that is complimentary to a larger guide document on fit published by the NSBA.

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Table 1 **Common Fit Conditions**

Loading Condition Fit	Construction Stage Fit	Description	Practice
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.	The fabricator (detailer) sets the drops using the no-load elevations of the girders (i.e., the fully cambered girder profiles).
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge steel dead load at the completion of the erection.	The fabricator (detailer) sets the drops using the girder vertical elevations at steel dead load, calculated as the fully cambered girder profiles minus the steel dead load deflections.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.	The fabricator (detailer) sets the drops using the girder vertical elevations at total dead load, which are equal to the fully cambered girder profiles minus the total dead load deflections.

Since differential deflections cause twisting of the girders when they are connected by cross-frames, the girders can be plumb only in the load condition in which the bridge is fit. That is, if a bridge is detailed for Steel Dead Load Fit (SDLF), the girders will be approximately plumb at the completion of the steel erection, but not when the remaining dead loads are ap-plied. Furthermore, if a bridge is detailed for Total Dead Load Fit (TDLF), the girders will not be plumb at erection but will theoretically be plumb after all the dead loads are applied. Hence, another way to refer to the fit condition is to speak of when the girders are approximately plumb: plumb at no-load, plumb at erection, or plumb in the final condition.

Although the above terminology is commonly used, it is not the best way to refer to fit conditions for two reasons. First, the natural answer to the question "when should the girders be plumb?" is "at the end." However, choosing plumb "at the end" is not always best and can lead to significant problems in some bridges. Second, the question of "how plumb is plumb enough?" cannot be answered effectively. Due to tolerances and constraints, the girders will not be truly plumb in the associated fit condition. For example, if TDLF is used, the deck casting sequence and hardening of the deck during casting may cause the girders to be somewhat out of plumb after the total dead load is applied unless the associated changes in stiffness are estimated sufficiently in the camber calculations. Likewise, the sequence of erection, cross-frame connection tolerances, and shoring conditions can influence the actual plumb condition at the erection of the girders in a bridge detailed for SDLF.

Customary Practice

Fabricators use the fit condition prescribed in the plans (AASHTO 6.7.2 specifies that this direction should be provided for significantly curved and/or skewed I-girder bridges, although the actual provision of such direction is not universal). A key element to consider when choosing the fit condition is the girder differential deflections between the locations where they are connected by cross-frames (i.e., what are the drops?). If the deflection of the girders at each side of the cross-frames is about the same, then the structure is not sensitive to the fit choice. For example on straight, non-skewed bridges with uniformly spaced girders and typical overhangs, the girders will deflect essentially equally at all dead load stages without producing any differential deflections. Conversely, the larger the difference in girder deflection between the locations where they are connected by the cross-frames, the more the fit choice matters.

For straight bridges (skewed and non-skewed), both SDLF and TDLF are common and effective. SDLF gives approximately plumb girder webs once the erection of the steel is completed and is favored for ease of construction. Since the steel dead load corresponds to the condition when all the girders are erected and all the cross-frames are connected, skewed bridges detailed for this condition require little force to fit the cross-frames to the girders. However, on skewed bridges, the application of subsequent dead loads (due to the weight of the deck, barriers, etc.) will introduce a final and permanent twist into the girders. Conversely, TDLF gives approximately plumb girder webs once the bridge is subjected to its total dead load; but for skewed bridges, the cross-frames do not match the geometry of the girders during erection, so the cross-frames must be forced into position and the girders will be tilted after the steel is erected until the final dead loads are applied.

Note that although Table 1 refers to girder elevations, major-axis girder rotations also affect fit. Although intuition might suggest that fit issues associated with differential deflections can be avoided by framing the cross-frames along the skew, doing so results in similar fit responses because the axis of the skewed cross-frames (which have high in-plane shear stiffness, or high racking stiffness) is not normal to the girder webs. As the girders undergo major-axis rotations, the cross-frames roll about their own axis, and since they have high in-place shear stiffness, they resist racking deformations and cause the girders to twist (or lay over).

For curved bridges, the use of SDLF is most common. Furthermore, practice has demonstrated that the use of TDLF on curved bridges can potentially render the bridge unconstructable. This is because curved girders cannot be twisted as readily as girders in straight bridges to facilitate erection. Therefore, as specified in AASHTO 6.7.2, the use of TDLF detailing should not be specified for horizontally curved bridges with or without skew and with a maximum *L/R* greater than 0.03. TDLF detailing may be specified but is not recommended for horizontally curved bridges when the supports are skewed more than 20 degrees from normal, spans are less than or equal to about 200 feet in length, and *L/R* in all spans is less than or equal to 0.03.

Article 6.7.2 further specifies that horizontally curved bridges with or without skew and with a maximum *L/R* greater than 0.03 may be detailed for a NLF or a SDLF, unless the maximum *L/R* is greater than or equal to 0.2. In this case, either the bridge should be detailed for a NLF, or the additive locked-in force effects associated with the SDLF detailing should be considered (refer to the section on Design and Analysis below). The additive locked-in force effects tend to be particularly significant for bridges with a maximum *L/R* greater than or equal to 0.2 that are detailed for a SDLF (NCHRP 2015). Detailing these bridges for a NLF avoids the introduction of these additional locked-in force effects. Furthermore, such bridges are likely to require temporary shoring and support during the erection as a matter of course—as such, the bridge is erected in a "quasi" NLF condition as a general practice and the cross-frames can be easily installed in this shored condition. For curved bridges with smaller L/R that are detailed for a SDLF, the horizontal curvature effects are smaller, and hence the additive locked-in force effects are smaller and may be neglected.

Recommended Fit Conditions

I-girder bridges have been detailed for fit for as long as steel stringers, including rolled beams, have been used in bridges. However the challenge of making a good fit choice has increased as bridge geometries have become more complex, and as greater skews, longer span lengths, and sharper curves have resulted in greater differential deflections. Tables 2 and 3 provide general fit recommendations which reflect historic experience blended with improved understanding of fit-up forces from recent research:

1. To facilitate fit-up (i.e., assembly of the steel) during erection;
2. To limit bearing rotation demands and to facilitate deck joint alignment and barrier rail alignment at skewed bearing lines; and
3. In straight skewed bridges, to reduce the dead load forces in the cross-frames and diaphragms and the flange lateral bending stresses in the girders, and in horizontally curved bridges, to limit the magnitude of additive locked-in dead load force effects.

The generalized terms used in the preceding tables are described as follows:

- L = span length, bearing to bearing along the centerline of the bridge
- R = radius of the centerline of the bridge cross-section
- The skew index, I_s, in Table 2 is defined as follows (AASHTO Eq. 4.6.3.3.2-2):

$$I_s = \frac{w_g \tan\theta}{L} \tag{1}$$

where:

- w_g is the bridge width perpendicular to the centerline, fascia girder to fascia girder, and
- θ is the maximum skew angle of the bearing lines at the end of a given span measured from a line perpendicular to the span centerline (equal to zero for no skew).

For continuous-span bridges, I_s is defined as the largest value for any of the spans. Equation 1 has been found to be a useful indicator of the influence of skew on the potential development of transverse load paths in the bridge system in straight skewed bridges (NCHRP, 2012). A strong correlation was found between the skew index and the general magnitude of the cross-frame forces caused by skew. For highly curved bridges, there is a complex interrelationship between the direction of the skew and the direction of the horizontal curvature when considering the fit behavior, and the associated effects are more involved than just the consideration of I_s. For the various recommended fit conditions presented in Tables 2 and 3, the span length and skew index limits should be considered as approximate guidelines and should be evaluated in the full context of the geometric and structural complexity of a given bridge.

Both SDLF and TDLF are customary long-used industry practices for straight bridges, but they are not used universally for all situations. That is, there are trade-offs between the two approaches. TDLF results in a bridge whose webs are nominally plumb after construction and produces smaller rotation demands at the bearings. However, at the end of the steel erection there will be an initial girder layover (until final dead loads are applied), and the girders and cross-frames must be forced together during erection. The use of such force is common, but may not be workable in some cases for longer span highly-skewed bridges. Conversely, SDLF makes straight skewed bridges easier to erect and results in webs that are plumb after erection; however, after

Table 2 **Recommended Fit Conditions for Straight I-Girder Bridges (including Curved I-Girder Bridges with L/R in all spans ≤ 0.03)**

Square Bridges and Skewed Bridges up to 20 deg Skew			
	Recommended	Acceptable	Avoid
Any span length	Any		None
Skewed Bridges with Skew > 20 deg and $I_s \le 0.30$ +/-			
	Recommended	Acceptable	Avoid
Any span length	TDLF or SDLF		NLF
Skewed Bridges with Skew > 20 deg and $I_s > 0.30$ +/-			
	Recommended	Acceptable	Avoid
Span lengths up to 200 ft +/-	SDLF	TDLF	NLF
Span lengths greater than 200 ft +/-	SDLF		TDLF & NLF

Table 3 **Recommended Fit Conditions for Horizontally Curved I-Girder Bridges ($(L/R)_{MAX}$ > 0.03)**

Radial or Skewed Supports			
	Recommended	Acceptable	Avoid
$(L/R)_{MAX} \ge 0.2$	NLF[1]	SDLF[2]	TDLF
All other cases	SDLF	NLF	TDLF

Note 1: The recommendation transitions to NLF at or above a maximum L/R of 0.2 because research on these types of bridges (NCHRP 2015) shows that the increase in the cross-frame forces from SDLF detailing can become more significant as the degree of curvature increases.

Note 2: SLDF detailing is considered acceptable in these cases if the additive locked-in force effects are considered (see Design and Analysis section below).

the final dead loads are applied, some girder layover will be present. This final layover is not known to cause any particular girder behavior problems, but the bearings must be able to accommodate the associated girder rotations. Generally NLF is not recommended for straight skewed bridges because NLF would lead to a need to accommodate girder twist rotations at the abutment bearings that can otherwise be avoided, and it does not facilitate fit-up or improve the final plumb condition. In the limiting condition of a bridge that is straight with no skew in any of the supports, (i.e., a "square" bridge), the fit-up effects become small and essentially inconsequential and the results of the different cross-frame detailing methods are all the same.

The emphasis of the above discussion is on straight skewed bridges. Additional considerations regarding horizontally curved bridges, with or without skew, are addressed in the following discussions.

Special Considerations

The following are key points to consider regarding fit. Although there are many fit considerations, these are highlighted here because they reduce the chances for construction problems:

- To facilitate construction at skewed abutments and piers, keep the first intermediate normal cross-frames a minimum of the larger of $4b_f$ and $0.4L_{b.adj}$ away from the support where practicable when laying out the cross-frames in design as noted in AASHTO C6.7.4.2, where b_f is the largest girder flange width within the unbraced lengths on either side of the intermediate cross-frame, and $L_{b.adj}$ is the adjacent unbraced length to the offset under consideration (NCHRP, 2015).
- Be cautious using oversize or slotted holes in the cross-frame to girder connections in straight skewed bridges; oversize holes (or

slots) can be used to facilitate assembly of discrete pieces that are difficult to frame in, but use of oversize or slotted holes throughout the system can compromise the bridge geometry. AASHTO 6.13.1 states that "Unless otherwise permitted by the contract documents, standard-size bolt holes shall be used in connections in horizontally curved bridges."

- Be sure to tighten fasteners in girder-to-cross-frame connections before casting the deck.
- As reflected in the tables above, avoid TDLF in curved bridges unless the supports are skewed and the degree of curvature is small. Given the stiffness and coupled vertical and torsional deflections of curved girders under load, there is no practical way to assemble some TDLF curved bridges (since substantial extra loads would need to be applied to account for the missing dead loads during erection).
- When TDLF is used on straight skewed bridges, note the expected initial layover (under steel dead load) in the design plans or shop drawings. This practice is recommended so that the layover does not cause alarm and delays when it is noticed during the steel erection.

Design and Analysis

Two different types of forces are influenced by the selected fit condition: 1. the bridge internal dead load forces and 2. the "fit-up" forces, which are external forces the erector may need to apply to assemble the structural steel during erection.

For SDLF/TDLF on a straight skewed bridge, the cross-frame internal forces due to the SDLF/TDLF detailing are opposite in sign to and a significant fraction of the internal steel dead load/total dead load (SDL/TDL) forces calculated by building an accurate grid (as defined in NCHRP, 2012) or 3D FEA model, and simply turning the corresponding gravity loads on (or which are nominally present in the cross-frames if the bridge were built with NLF detailing). Since the locked-in forces due to the SDLF/TDLF detailing are opposite in sign to and a significant fraction of the above SDL/TDL internal forces, the total internal dead load forces in the cross-frames of a straight skewed bridge detailed for SDLF are relatively small under the SDL (at the completion of the steel erection), and the total internal dead load forces in the cross-frames of a straight skewed bridge detailed for TDLF are relatively small under the TDL (at the completion of the bridge construction).

It is conservative to design the cross-frames in a straight-skewed bridge using the results from an accurate grid or 3D FEA model and neglecting the SLDF or TDLF effects. This is the current common practice when the engineer chooses to utilize more than a line girder analysis for the design. In I-girder bridges having a particularly large skew index, I_s, the cross-frame forces estimated in this way can be overly conservative. In some cases, this can lead to excessively large cross-frame member designs. Due to the eccentricity of the cross frame connection plates to the centroid of the members, the axial stiffness of the angles and tee sections typically used as cross frame members is reduced. Stiffness reduction coefficients are contained in Basttistini et al (2016). The reduced axial stiffness should be used when modeling the cross frame members in accurate grid or 3D FEA analysis. In lieu of requiring a refined analysis that directly determines the locked-in force effects due to the DLF detailing, the larger guide document on fit provides simple reduction factors that may be applied to the cross-frame forces (for TDLF only) and the girder flange lateral bending stresses obtained via a refined analysis that does not otherwise account for these effects.

For straight skewed bridges detailed for SDLF, little to no forcing is needed to fit the cross-frames and girders during the steel erection. That is, the required external "fit-up" forces are small. In straight skewed bridges detailed for TDLF, the cross-frames must be forced to fit to the girders during the erection of the steel, but the associated internal forces largely come back out when the final dead loads are applied and the system deflects to the TDL condition. As the skew approaches zero in a straight I-girder bridge, both the internal forces due to SDLF or TDLF detailing, as well as the fit-up forces required to erect the steel, become small and inconsequential.

The girders in curved bridges have radial forces introduced by the cross-frames to satisfy equilibrium with their major-axis bending moments, and to restrain their tendency to twist. SDLF and TDLF detailing tends to increase these internal cross-frame forces and girder flange lateral bending stresses, since the cross-frames are used to twist the girders back in the direction opposite to the direction they naturally roll under the dead loads. Further, curved girders can be much stiffer than straight girders and the girder vertical and torsional deflections are generally coupled. The additional forces associated with TDLF detailing tend to be prohibitive for highly-curved I-girder bridges, and thus TDLF detailing of these types of structures is strongly discouraged.

The additional internal cross-frame forces and flange lateral bending stresses due to SDLF effects tend to be relatively small in horizontally curved bridges, unless the maximum L/R is greater than or equal to approximately 0.2 (NCHRP, 2015).

For these curved bridges with more significant horizontal curvature, the local twisting of I-girders to make the connections may become more difficult. In these cases, NLF is recommended, unless the additive locked-in force effects associated with SDLF detailing are considered. It is possible to directly calculate the internal "locked-in forces" associated with SDLF detailing in such cases by performing a refined analysis that includes the lack-of-fit due to the SDLF detailing (NCHRP, 2015). In lieu of such an analysis, the larger guide document on fit provides an approximate approach for estimating the additional locked-in force effects.

Conclusion

In I-girder bridges, the relationship between the girders changes as the girders deflect under the dead load. These changes introduce internal forces and affect fit-up; when the changes are significant, it is important that the appropriate fit decision be made to facilitate the construction of the bridge and to achieve benefits in limiting girder dead load twist rotations, cross-frame dead load internal forces and girder flange lateral bending stresses. Making the right fit choice is a key consideration that can impact engineers, fabricators and erectors, and the best fit choice is one made by the engineer informed by all of the stakeholders.

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Are you Sure that's Fracture Critical?

BY ROBERT J. CONNOR, PhD, KARL FRANK, PE, PhD, BILL McELENEY AND JOHN YADLOSKY, PE

ONE OF THE MOST NOTEWORTHY bridge failures in the United States occurred in 1967, when the Point Pleasant Bridge over the Ohio River (also known as the Silver Bridge) collapsed, resulting in 46 deaths.

The collapse was due to brittle fracture of one of the eyebars that formed the suspension system of the bridge. The subsequent failure investigation revealed that the fracture was due to brittle propagation of a tiny crack in the eyebar. Because the fracture toughness of the eyebar was extremely low, a relatively small crack led to a brittle fracture of the eyebar, which in turn led to the collapse of the bridge.

This collapse was the catalyst for many changes in material specifications, design, fabrication and shop inspection of steel bridges. These requirements are codified in the AASHTO *Bridge Design Specifications* and the AASHTO/AWS D1.5 *Bridge Welding Code* (AWS) and are applied to tension members whose fracture could lead to bridge collapse. (Another bridge incident—the failure of a pin-and-hanger assembly, which triggered the collapse of one span of the Mianus River Bridge in 1983—served as the impetus for enhanced field inspection requirements for these same members.)

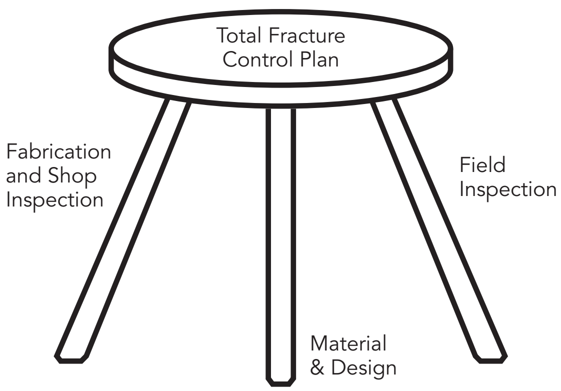

Fig. 1.
The three "legs" of a total fracture control plan for bridges.

The Three-Legged Stool

Today, a total fracture control plan (FCP) is often illustrated as a three-legged stool, where each leg is made up of a part of the plan, as illustrated in Figure 1. (Since the introduction of the FCP, the authors are not aware of any failures in fracture critical members fabricated to the FCP. Hence, the FCP concept appears to be serving its intended purpose.)

It is essential to understand that the FCP was specifically developed in response to failures (i.e., brittle fractures) in non-redundant tension members that occurred in the 1970s. Such members, which may be either entirely (e.g., a truss member) or partially (e.g., a flexural member) in tension became known as fracture critical members (FCMs). An FCM is defined by the *Code of Federal Regulations* (23CFR650 – Bridges, Structures and Hydraulics) as "a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse."

Prior to the FCP, the design of tension members was based solely upon prevention of yielding; there were minimal requirements on steel toughness (i.e., no Charpy V Notch toughness requirements) and less stringent fabrication and shop inspection requirements. In fact, there was no AWS bridge welding code in existence. Researchers and engineers alike recognized that control of brittle fracture in non-redundant tension members, or portions of members in tension, was important.

In short, the primary objective of the FCP is to prevent brittle fractures of non-redundant tension members and tension components. The material, fabrication and shop inspection portions of the FCP are intended to minimize the frequency and size of discontinuities that might initiate a crack and also to ensure that materials with greater flaw tolerance are used for these members. Arms-length in-service field inspection is intended to discover fatigue cracks before they become a critical size.

Classifying a Member as an FCM

To be classified as an FCM, two basic yet specific criteria must be met:

1. An FCM must be subjected to net tensile stresses from either axial or bending forces. For example, a member that carries 100 kips in dead load compression but 200 kips in live load tension would satisfy this portion of the definition since the *net* force is tension. It is recognized that for brittle fracture to occur and propagate, tensile forces that exceed any compressive forces *must* be present in the member. As another example, in a simple-span beam only the components of the beam in tension (i.e., bottom flange and portion of the web in tension) would meet this requirement.

2. An FCM must be determined to be non-redundant. While definitions vary slightly, the concern is for members whose fracture would result in collapse of the bridge or a portion of the bridge. A member with an alternate load path—i.e., a redundant member—should not be considered fracture critical. Members such as the lower tension chord of a truss, single or double eyebars or pin and link hangers

Understanding which steel bridge elements are fracture critical members will provide the required protection while saving on in-service inspection.

are typically considered as non-redundant members and identified as FCMs because it is presumed that if the member were to fail in brittle fracture, it could trigger the collapse of the bridge. In the absence of a more rigorous system analysis, this is of course a reasonable assumption. It is these types of members that were on the minds of the individuals who developed the FCP. In contrast, however, the tension flanges of multi-girder bridges are not considered FCMs because the adjacent girders provide a redundant load path and load capacity in the event of a fracture of any given girder.

If either of the above criteria is not met, the member shall not be considered an FCM. That is true of every specification in the United States governing steel bridge design, fabrication and in-service inspection that includes the concept of an FCM.

The responsibility to designate a member or member component as an FCM is incumbent on the design engineer. Once it is determined that the element meets both of the above criteria, the member must be clearly labeled as FCM on the design plans. This is essential as it alerts the fabricator to obtain the proper material and fabricate the member to the FCP. However, in addition to the more stringent material and fabrication requirements, the member will also be subject to more rigorous and costly arms-length in-service inspection every two years for a highway bridge.

Applying an FCP

Interestingly, during the development of the FCP, those who crafted the provisions recognized that engineers, given the choice, will often specify the most conservative option provided in a specification and in this case, potentially require the FCP regardless of member loading, type, etc. simply because it would be perceived to be "safer." To avoid this, the commentary to the FCP in AWS explicitly states that it is not intended to be used for members the engineer simply deems "important." In fact, the commentary goes so far as to state that the FCP is not intended to be used for anything but bridges. For example, see this wording from the commentary:

"The fracture control plan should not be used indiscriminately by the designers as a crutch 'to be safe' and to circumvent good engineering practice. Fracture critical classification is not intended for 'important' welds on non-bridge members or ancillary products; rather it is only intended to be for those members whose failure would be expected to result in a catastrophic collapse of the bridge."

Thus, although a member may be deemed "important," if it does not meet the two criteria cited above the member shall not be classified as an FCM. For example, failure of an end-post of a simple span truss will most likely cause collapse of the span. However, since it is never subjected to tension, it would be incorrect to label it as an FCM simply because it is a critical or "important" member in the bridge. This commentary leaves little to interpretation.

Despite the guidance in the specifications, it has become apparent that some design engineers occasionally incorrectly classify steel members as FCMs. This is likely due to inexperience and lack of familiarity with the spirit and objective of the AASHTO/AWS FCP. Nevertheless, in order to properly identify when a member should be classified as an FCM, it is best to first examine the definitions contained in various specifications (underlines are for emphasis):

From AWS:

- AASHTO/AWS D1.5 *Bridge Welding Code*, Article 12.2.2– Definitions

"Fracture critical member (FCM). Fracture critical members or member components are tension members <u>or tension components of bending members</u> (including those subject to reversal of stress), <u>the failure of which would be expected to result in collapse of the bridge</u>. The designation 'FCM' shall mean fracture critical member or member component. <u>Members and components that are not subject to tensile stress under any condition of live load shall not be defined as fracture critical.</u>"

- AASHTO/AWS D1.5 *Bridge Welding Code*, Article C12.2.2– Commentary on Definitions

"Tension members or member components whose failure would not cause collapse of the bridge are not fracture critical. Compression members and <u>portions of bending members in compression</u> may be important to the structural integrity of the bridge, but <u>do not come under the provisions of this plan</u>. Compression components do not fail by fatigue crack initiation and extension, but rather by yielding or buckling."

From the American Railway Engineering and Maintenance-of-Way Association (AREMA):

- AREMA *Manual for Railway Engineering*, Chapter 15, Article 9.1.14.2a

"Fracture critical members (FCM) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function. The identification of such components must, of necessity, be the responsibility of the bridge designer since virtually all bridges are inherently complex and the categorization of every bridge and every bridge member is impossible. <u>However, to fall within the fracture critical category, the component must be in tension</u>. Further, a fracture critical member may be either a complete bridge member or it may be a part of a bridge member."

- AREMA *Manual for Railway Engineering*, Chapter 15, Article 9.1.14.2b

"Members or member components whose failure would not cause the bridge to be unserviceable are not considered fracture critical. <u>Compression members and member components in compression may, in themselves, be critical but do not come under the provisions of this Plan</u>. "

As clearly stated in these specifications, compression members or components of members in compression are not to be considered FCM. Both AREMA and AWS use essentially the same definitions and state that compression members "do not" come under the provisions of the FCP. Further, redundant members do not come under the provisions of the FCP. The use of the phrase "do not" also leaves no interpretation and differs from other typical specification type verbiage, such as "should" or "may."

FCM or not?

In the interest of providing guidance, a few typical members found in steel bridges are listed along with basic rationale for either classifying or not classifying the member as an FCM.

Multi-girder bridges and stringers. Bridges with multiple longitudinal members, such as girder bridges with three or more girders or stringer beams of long-span bridges, are examples of members with alternate load paths in the event of a fracture. Their criticality is similar to the bridge deck, where fracture would result in local failure of the deck but not collapse of the bridge. As an example, fatigue cracks were found in late 1970 at cover plate terminations on the Yellow Mill Pond bridge, which carried I-95 in Connecticut. The girders had numerous small cracks and although one girder almost completely fractured, the bridge continued to carry traffic.

While a portion of these members is subjected to tension due to bending, failure of a single stringer or girder would not result in collapse of the bridge or even a part of the roadway. Multiple stringers supported by transverse floor beams are also inherently redundant.

Floor beams. Some engineers have chosen to classify floor beams fracture critical, perhaps in consideration of the support of the roadway. Floor beams should be assessed for FCM status in the same manner as any other bridge member—i.e., is fracturing of a floor beam likely to result in the collapse of the bridge? Regarding roadway support, consider the following:

1. Is the bridge deck composite with the stringers and floor beams? If so, in order for the riding surface to collapse, the entire floor system must suffer a fracture.
2. Are there continuous stringers over the floor beams? Continuous stringers offer an alternate load path for the vehicle load.
3. How are the floor beams framed into the main longitudinal elements? Can a failed floor beam in conjunction with the bridge deck carry load via an arching action spanning across the fracture?
4. Assuming the tension side of the floor beam fails, is it reasonable to assume the entire floor beam would suffer a full-depth fracture?

In most cases, floor beams in conjunction with continuous stringers and the continuity of the deck will provide a redundant system capable of carrying the vehicle load without a collapse.

The authors have observed cases where engineers have classified floor beams as FCMs on bridges where the floor beams are spaced very closely, such as three feet or less. It is difficult to imagine that failure of a floor beam spanning from main girder to main girder spaced so closely could result in collapse of the bridge or roadway. If one were to idealize the main girders as supports between which the floor beams span, the cross section that carries the load would be comprised of multiple girders (i.e., floor beams). Hence, by definition, the floor beams could not be classified as FCMs at such close spacing.

If a floor beam is judged to be fracture critical, only the portion subjected to tensile stresses should be subjected to the FCP. If the floor beam is a rolled beam, while the entire beam would be required to meet the more stringent CVN material requirements, only the portion in tension is subjected to the FCP fabrication and inspection requirements. Hence, welds made to the compression flange would not be subjected to the FCP even though the rolled beam is a single piece of steel. If the floor beam is a fabricated plate girder, the tension flange and the web must meet the more stringent CVN material requirements of the FCP. However, only the portion of the web that is in tension needs to meet the FCP fabrication requirements. The top flange, which is only in compression, would not be considered fracture critical. Also, if the floor beam is designed as a simply supported member, small negative moments that may be produced due to a shear connection at the ends would not justify classifying the top flange as FC material.

Primary longitudinal girders. While the FCP applies to various elements, it was failure in elements such as primary longitudinal girders that led to the development of the plan. The classic main girders of a "two-girder" bridge can reasonably be classified as FCMs since failure of one of the beams may be expected to lead to collapse of the bridge. In the absence of any rigorous system analysis, the portions of the girders subjected to tension (flange and web) would be classified as FCMs and be required to meet the FCP, while the portion of the girder that is only subjected to compression does not, as illustrated in Figure 2 (on the following page).

Tension chords or diagonals in trusses. Generally speaking, most tension diagonals and chords in trusses would be classified as FCMs.

Tie girders. Generally speaking, tension ties would be classified as FCMs.

Miscellaneous attachments to FCMs. In addition to primary members, certain attachments must also be classified as FCMs and be fabricated to the requirements of the FCP. The reason for this is to ensure that components such as longitudinal stiffeners meet the same requirements as the base metal of the primary member. Further, the welds used to attach these components to the primary member must also meet the provisions of the FCP. For example, see this excerpt from AWS Article 12.2.2.2 Attachments:

"Any attachment welded to a tension zone of an FCM member shall be considered an FCM when any dimension of the attachment exceeds 100 mm [4 in.] in the direction parallel to the calculated tensile stress in the FCM. Attachments designated FCM shall meet all requirements of this FCP."

The FCP clearly states the attachment must be located on the portion of the member subjected to *tensile* stresses. Hence, a longitudinal stiffener that is welded to a girder in the tension zone of the web plate must meet the FCP, while a longitudinal stiffener in the compression zone of a web plate does not need to meet the FCP, as shown in Figure 2 (found on page 4). Note that even though the attachment is welded to a web plate—which is designated as FCM in terms of the *material* selection (see AWS C12.2.2.2)—due to the fact that a portion of the web is in tension (since the welding of the longitudinal stiffener is on the compression portion of the web) there is no need to invoke the FCP. Note also that short attachments, such as a transverse stiffener, which is always less than 4 in. long in the direction of primary stress, need not be classified as FCM.

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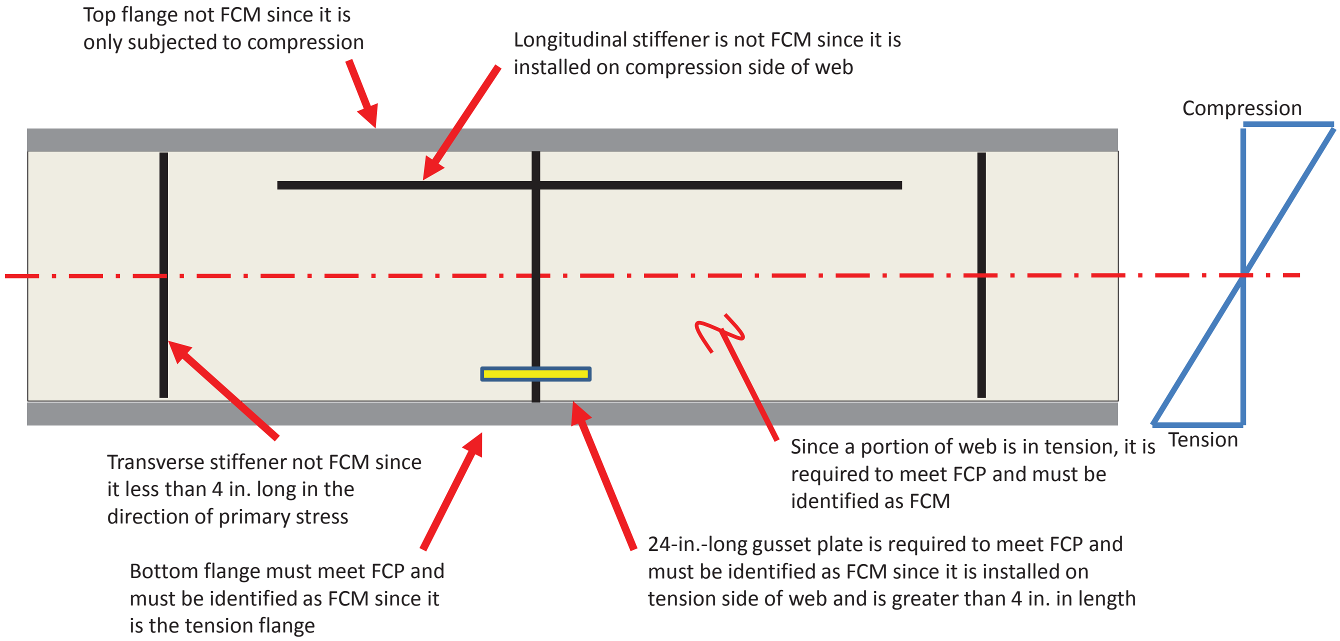

Fig. 2. Example of classification of FCM components on a plate girder (created by Robert Connor).

Ongoing Research

There are currently several research projects under way focusing on bridges and bridge members traditionally classified as fracture critical. Individual projects are studying the following areas:

Member-level redundancy. This research effort is examining the strength and fatigue performance of both riveted and bolted built-up members. While it is accepted that built-up members possess some level of internal redundancy, it has not been fully quantified through large-scale experimental or analytical research. Pooled fund study TPF-5(253) is characterizing this behavior and will result in evaluation and design guidelines for such members to ensure sufficient redundancy exists.

System redundancy. Several studies are under way, such as NCHRP Project 12-87a (research funded by AISC/NSBA focusing on twin-tub girders) as well as research sponsored by other agencies that are working to develop modeling, evaluation and design guidance related to analyzing bridges traditionally classified with FCMs. While it is generally presumed that failure of an FCM will cause collapse of the structure, field experiences where such failures have occurred suggest otherwise in all but extreme cases, such as in the Silver Bridge. These projects will result in rational criteria to characterize the benefits of load redistribution provided by the structural contributions of the deck slab, secondary members, parapets and other components not traditionally used. Further, the minimum live load capacity that is to be maintained in the faulted state will also be defined.

Exploitation of superior-toughness steel. It is well known that modern steels, in particular the HPS grades, offer far superior fracture toughness than "older" steels. However, the current A709 toughness requirements for HPS grade, while good, do not fully exploit the potential benefits of the HPS grades in terms of fracture resistance. These grades are consistently produced with toughness levels that far exceed minimum requirements. The research being conducted through pooled fund study TPF-5(238) explores the benefits of increasing the toughness requirements of some steel grades so that brittle fracture is no more likely than any other limit state, thereby effectively "taking fracture off the table" so to speak. In the extremely unlikely event a fatigue crack were to develop, tolerable crack sizes will be large enough to be reliably detected during normal inspections. By treating brittle fracture like any other limit state (e.g., buckling), it can be effectively mitigated eliminating the need for the term "FCM" in terms of long-term inspection.

Safer Bridges

The AASHTO/AWS D1.5 FCP has been in place for nearly 35 years and appears to have eliminated brittle fractures in steel bridges through improved material toughness, fabrication practices and shop inspection. Additionally, the modern steels, in particular the HPS grades, possess far superior toughness than those used before the introduction of the FCP. The combination of these factors provides much greater safety than our legacy bridges built before the FCP.

While the additional first cost associated with the FCP have been estimated to be 5% to 10% of the total steel fabrication cost, the FCP should not be invoked based on the false assumption that this will somehow make the bridge "better." Designers and owners must appreciate that once a member is classified as an FCM, it is subjected to arms-length biennial inspections for the life of the bridge. As a result, the *long-term* costs associated with inspection greatly increase the life-cycle cost of the structure. When invoked arbitrarily, this simply increases costs, with little or no increase in actual performance of the structures.

In summary, engineers are encouraged to become familiar with the existing AASHTO/AWS D1.5 *Bridge Welding Code* provisions to ensure they are specified only when necessary and appropriate. Doing so will result in the most economical steel structure and is in the best interest of the owner, fabricator and public. Further, as current research progresses and is moved into practice, the meaning of the term fracture critical will certainly evolve. In fact, with modern steels, modern fatigue design approaches and advanced analytical tools, we may see a time when the term fracture critical will no longer be relevant.

Piece by Piece

BY MICHAEL P. CULMO, PE

Span-by-span bridge construction, using modular steel bridge elements, can serve as a viable and economical bridge-building alternative.

CME Associates, Inc.

ACCELERATED BRIDGE CONSTRUCTION (ABC) has come a long way in the last 10 years.

And prefabricated, modular elements made with steel beams have been a big factor in making this happen, as they can be used to reduce the weight of the assemblies, thereby making crane installations more cost effective and viable.

Modular steel beam/deck elements generally consist of two or three steel beams with a composite concrete deck cast in the fabrication plant. They are erected quickly and joined with reinforced concrete closure pours made with high-early-strength concrete; a bridge superstructure can be built in as little as two days using this technique.

One of the more successful examples of this method was the 93Fast14 project in Medford, Mass. (a 2012 NSBA Prize Bridge Awards winner), which involved replacing 41 spans on 14 bridges along Interstate 93. The 14 bridge superstructures were replaced during ten 55-hour weekend work periods. The use of structural steel for the beam elements made the project possible since crane capacities controlled many of the sites.

Span by Span

Let's take a look at the two common ABC methods to design and construct a multi-span bridge. The first is to detail multiple simple spans between supports, sometimes referred to as "span-by-span" construction. Conventional simple-span bridges require expansion joints at each pier—historically a problematic feature of many bridges—as leaking joints, considered by many to be the most common cause of premature bridge deterioration, lead to the corrosion of beam ends and deterioration of the substructures under the joints.

above photo: The 93Fast14 Project in Medford, Mass., demonstrated the viability of modular steel bridge construction by replacing 41 spans in ten 55-hour weekend work periods.

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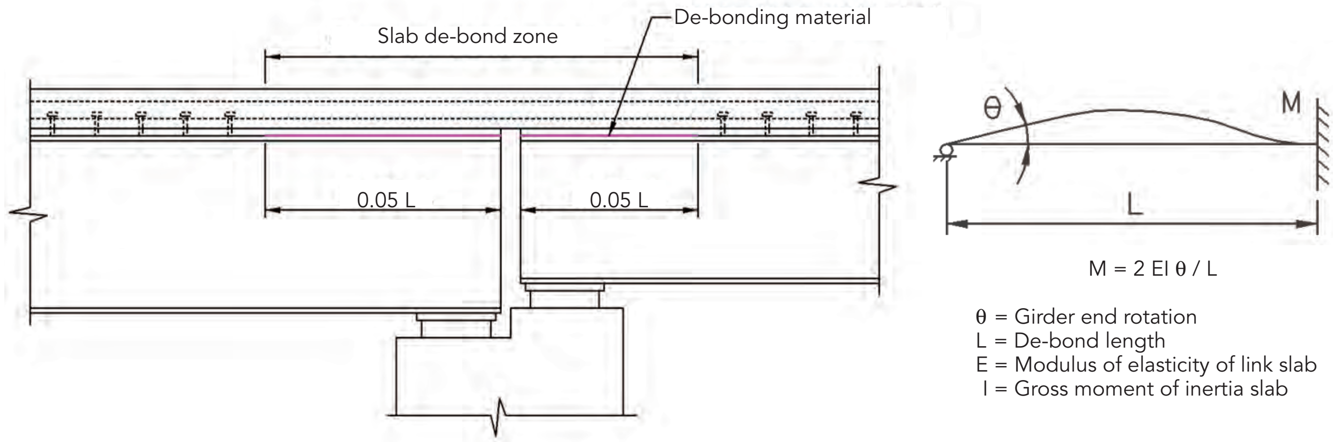


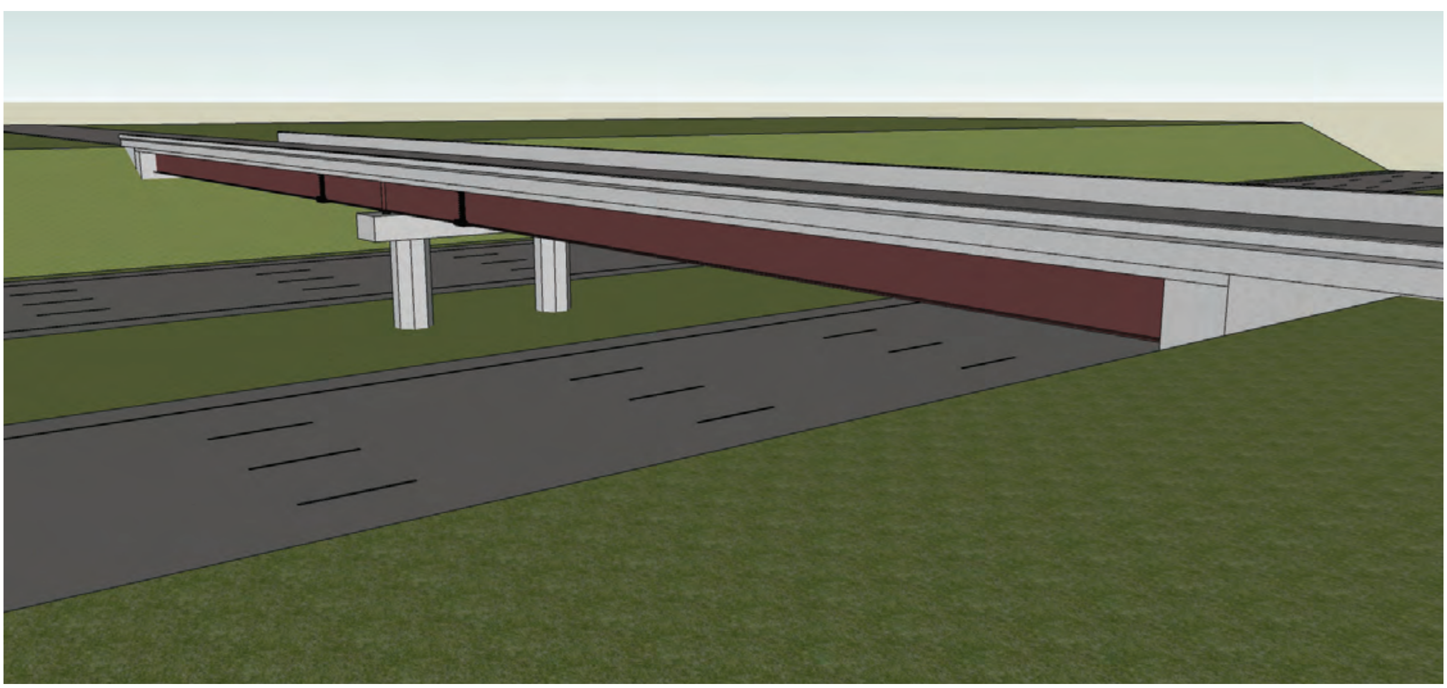
Bridge deck joints can be eliminated at piers through the use of "link slabs."

The second method for designing multi-span bridges is to use continuous-span beams, which do not require deck expansion joints at the interior supports, and require less structural steel for a given span arrangement.

Span-by-span beams are simply erected on the substructures without the need for splicing and shoring towers. The problem with leaking deck joints has been addressed by designing these bridges to be either joint-less or continuous for live load by using simple concrete pours at interior supports to eliminate the need for deck expansion joints. Using span-by-span techniques for the superstructure can accelerate the process by eliminating the need for welded or bolted field splices in continuous girders. Beam erection can progress very rapidly as the modular units are inherently stable. Once set, the crane can release the beam without the need for any external bracing.

One method that has been developed to eliminate deck joints on simple-span bridges is "link slab" technology. A link slab is built by simply casting the slab continuously across the pier linking the two spans. The link slab is designed to accommodate the live load rotation of the girders without significant cracking. This is accomplished by de-bonding a portion of the deck near the support to form the link slab, which acts as a flexible beam. The recommended length of de-bonding is 5% of the adjacent span on each side of the pier. Keep in mind that link slabs are not a form of continuity. The bending moments in the link slab are much less than typical negative bending moments in continuous girder bridges; therefore, the design of the girders is based on simple-span supports.

The bending moment in the link slab can be calculated using a simple equation. Reinforcing can then be designed to resist the bending and control cracking. The bending stresses in link slabs are often less than the tension stresses that develop in continuous-span bridges. The same principals of crack control reinforcing design are applied to both.



Typical two-span overpass bridge.

Greater Efficiency

We are taught in engineering courses that continuous steel girders are more efficient than simple-span girders and that "least weight equals least cost." In principle, these lessons are true. But in order understand the true efficiency of steel bridge construction, the engineer needs to look at the total cost of the bridge, including the cost of connections, construction methods and deck reinforcement. In order to study the efficiency of span-by-span construction, we investigated the preliminary design of a hypothetical two-span bridge. The bridge selected is a typical expressway overpass with equal spans of 122 ft and five girder lines.

Two bridge types were studied for this structure: continuous girders and simple-supported girders. The NSBA computer program Simon was used to complete a preliminary design of the girders. (Simon is available for free at **www.aisc.org/steelbridges** and can be used to design efficient steel girders for simple- and multiple-span bridges based on the AASHTO LRFD *Bridge Design Specifications*.)

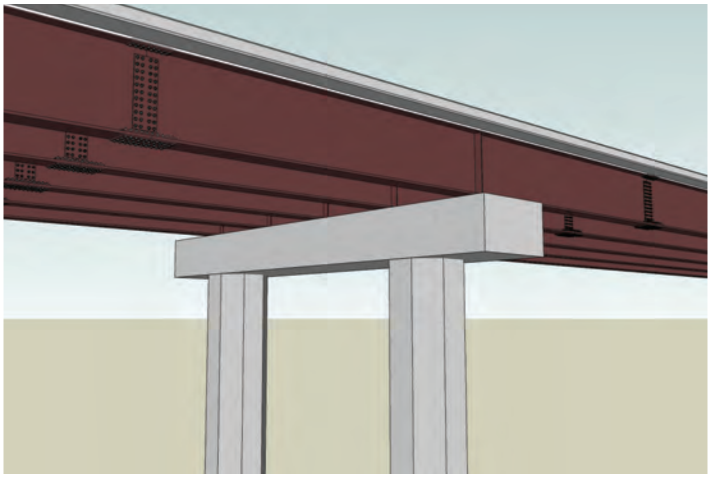
The results of the preliminary design showed that the simple-span bridge required 30 more tons of steel at a cost of $70,000 more than the continuous-span option (based on construction costs in the Northeast). The remainder of the study was dedicated to investigating the total cost of the bridge in order to determine if other factors would offset the increased cost for the structural steel.

On such factor was splicing. The 122-ft-long simple-span girders can be shipped in one piece (without field splices), where the continuous girders would need at least one field splice. The study assumed that two field splices would be required for the bridge. It may be possible to build this bridge with one splice, but the length of the pieces would be more than what some permitting agencies would allow.

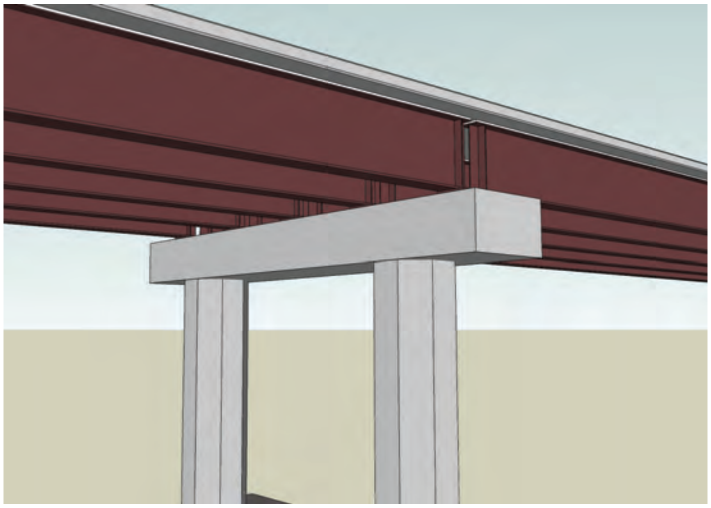
Another NSBA computer program, Splice, was used to design the bolted splice for the continuous girder study bridge. This program can efficiently design a bolted field splice according to the requirements of the AASHTO LRFD *Bridge Design Specifications*. The final design of the splice included 116 high-strength bolts, and the cost for fabrication and installation of the splice was estimated to be $5,800 per splice (again, based on typical regional construction costs). By eliminating the need for bolted field splices in the span-by-span bridge, an estimated cost savings of $58,000 could potentially be realized.

The *Bridge Design Specifications* require the use of longitudinal reinforcing steel in the negative moment region of continuous girder bridges in order to control cracking due to composite dead load and live load moments. In general, the design of link slabs results in longitudinal reinforcing that is much less than that used in continuous girder bridges. In addition, the link slab reinforcing steel need only be applied over the link slab zone, which is typically smaller than the negative moment region of a continuous girder. For the study bridges, the link slab design saved considerable reinforcing steel when compared to the continuous-span bridge, which equated to an approximate savings of $22,000.

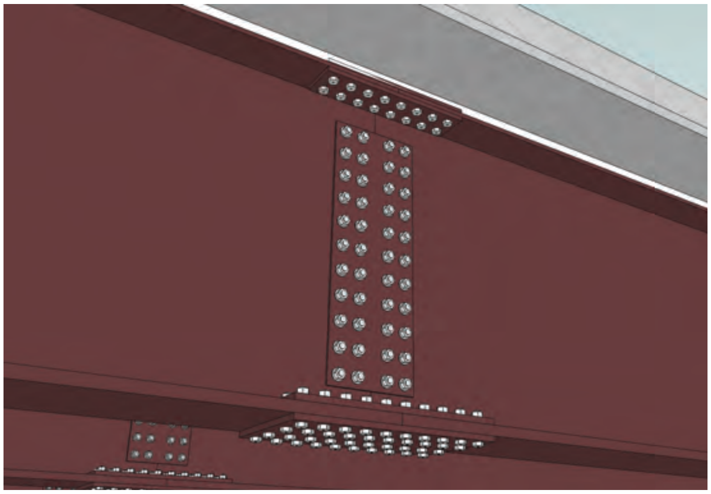
Another avenue of potential cost savings with simple-span construction is erection. Many agencies require the use of shoring towers under bolted splices. Even if shoring towers are not used, the cranes are required to hold the girders until sufficient bolts are installed in the field splices, which is a less efficient process. The potential erection cost savings for the simple-span bridge was estimated to be approximately $30,000.



Continuous girder with bolted splices.



Simple-span bridge with joint-less deck.



Bolted field splice designed using NSBA's Splice program.

When it comes to bearings, simple-span construction requires two lines of bearings at the center pier, compared to one line of bearings in the continuous girder bridge. The simple-span bearings are small but there are more to fabricate and install, and the cost of the extra bearings was estimated to be approximately $1,500.

When the above items are accounted for, an estimated net cost savings of $38,500 could be realized for the span-by-span bridge.

Item	Net Cost Savings
Structural Steel	–$70,000
Bolted Splices	$58,000
Additional Deck Reinforcing	$22,000
Steel Erection Cost	$30,000
Bearings	–$1,500
Net Savings	$38,500

Net cost savings for simple-span construction as compared to continuous bridge construction.

To recap:

1. Continuous-girder spans require less structural steel and fewer bearings.
2. The simple-span construction method may not need bolted field splices, uses less additional deck reinforcement and may be less expensive to erect when compared to a continuous girder bridge.
3. Least weight of structural steel does not always equate to least overall bridge cost.
4. By using link slab technology, simple-span construction can be accomplished with a joint-less deck that is durable.
5. Simply put, simple-span construction is a valuable tool for accelerated bridge construction projects.

This study was limited in that only one bridge was investigated. Other bridge configurations will yield different results. In some cases, a continuous-girder bridge may have a lower overall bridge cost. The conclusion of the study is that simple-span construction should not be ignored due to concerns over the structural efficiency of the girders alone. When total bridge costs are applied, this method can be competitive or even less expensive than conventional continuous-girder designs. ■

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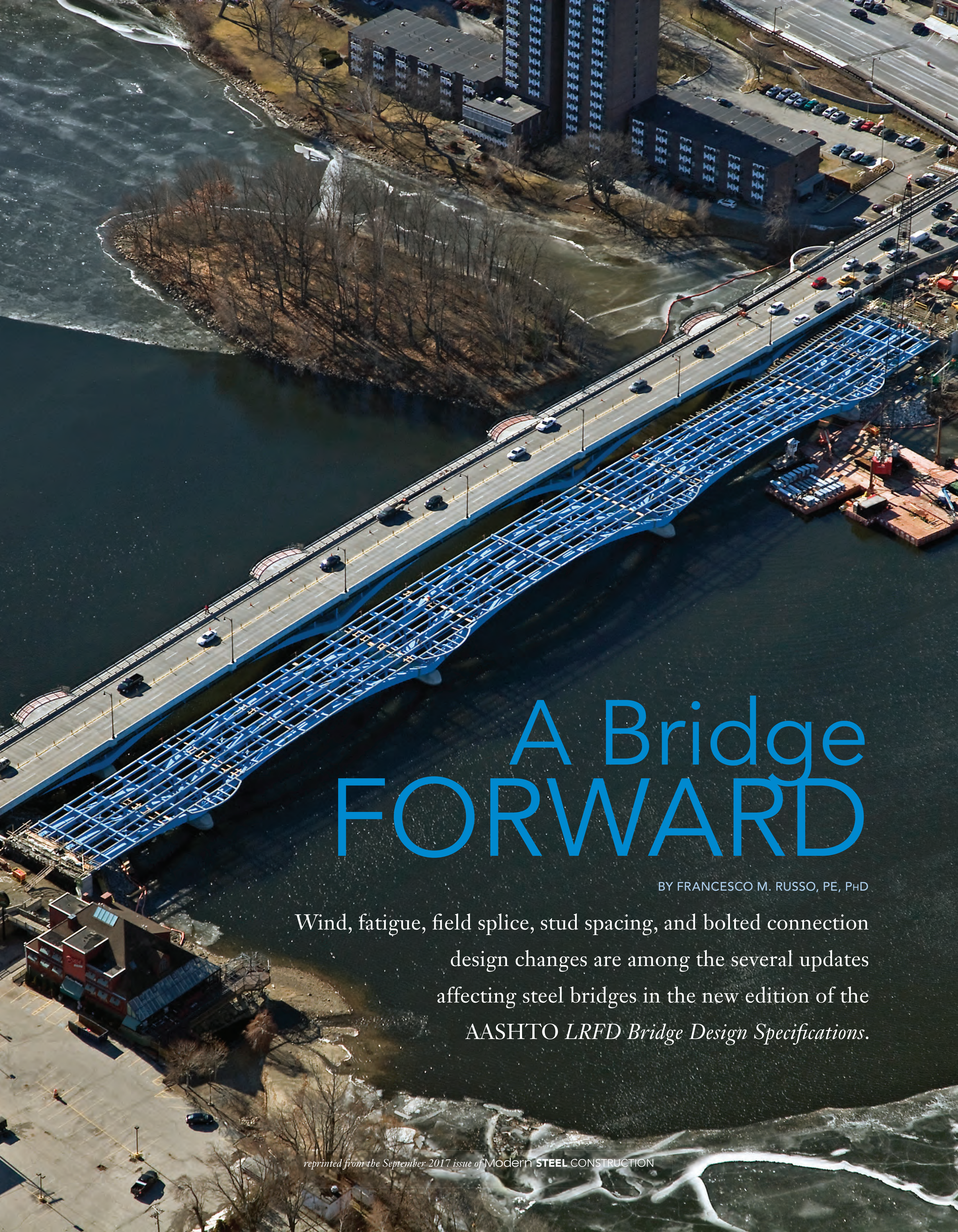

A Bridge FORWARD

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 of the AASHTO *LRFD Bridge Design Specifications*.

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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.

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.

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 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 nonslender 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.

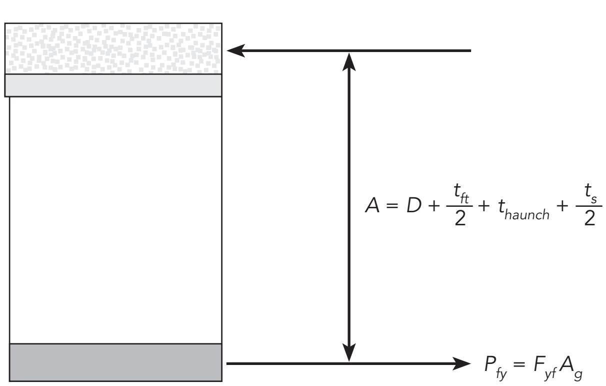
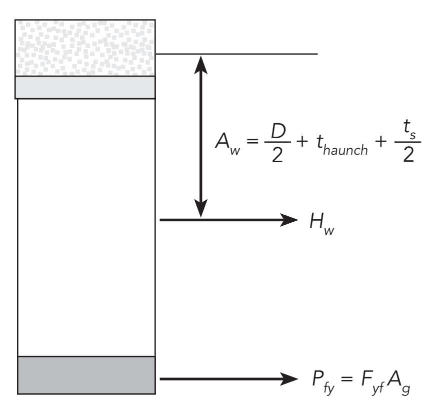
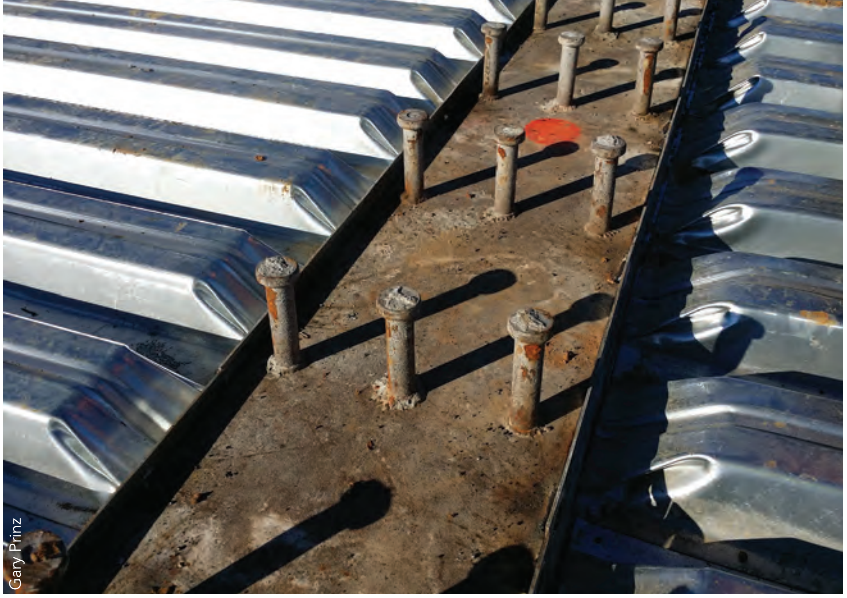


Figure 2. Positive moment flexural resistance relying on a web contribution.





Gary Prinz

John Rodems

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

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.

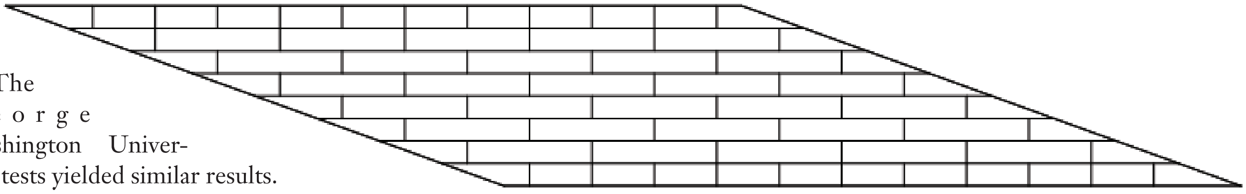
For a member containing slender elements, the capacity of the section is defined in Section 6.9.4.2.2, but the element slenderness need not be checked against a limit based on F_y; rather its slenderness need only be sufficient to be stable to a level of stress, F_{cr}, that corresponds to the member 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.

Commentary in the new specification builds on recent research conducted at Georgia Tech on forces in skewed steel bridges.

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

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



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 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.

The specification includes changes regarding detailing skewed bridges, longitudinal stiffeners and connection plates.

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 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.

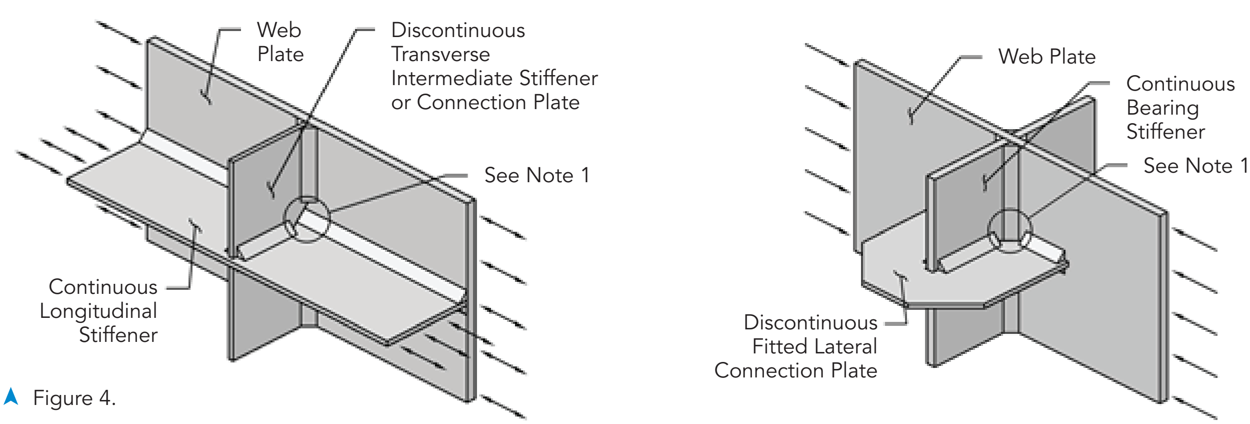


Figure 4.

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 placement 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.


John Rodems

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.


Steve Percassi, Bergmann Associates

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


Steve Percassi, Bergmann Associates

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

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 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 $0.6 \times 0.8 = 0.48A_b F_u$ to a new value of $0.625 \times 0.9 = 0.56A_b F_u$—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.

The Basics of Steel Bridge Design

Workshop at the 2020 World Steel Bridge Symposium

ATLANTA

Tuesday, April 21, 2020 | 1:00 p.m. – 6:00 p.m.



The National Steel Bridge Alliance (NSBA) is pleased to announce that we will be hosting a workshop on the basics of steel bridge design and fabrication April 21 at the 2020 World Steel Bridge Symposium (WSBS) in Atlanta. From calculating loads to best practices for detailing, this workshop aims to take attendees through the complete design of a 3-span continuous plate girder bridge.

Whether you are a novice just starting out or an expert in need of a refresher, this workshop will offer something for all levels of experience. Additionally, attendees from federal and state transportation departments will be eligible for complimentary registration and travel expense reimbursement.

Topic	Presenter
Conceptual Layout and Framing Plan	Frank Russo Michael Baker International
Loads	TBD
Basics of Steel Bridge Design	Domenic Coletti, HDR Brandon Chavel, NSBA
Bolted Field Splices	Chris Garrell, NSBA
Steel Bridge Detailing and Fabrication	Gary Wisch, DeLong's
Basics of Bridge Welding	Ronnie Medlock High Steel
Modern Corrosion Protection Systems	Justin Ocel, FHWA

In the meantime, please visit **aisc.org/wsbs** for updates on this workshop and to access our library of steel bridge design resources.

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