

Guidelines to Design for Constructability and Fabrication

G12.1-2020



AMERICAN ASSOCIATION
OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS
AASHTO



American Association of State Highway and
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National Steel Bridge Alliance
AASHTO/NSBA Steel Bridge Collaboration
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Preface

This document presents guidelines developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each document represents the consensus of a diverse group of professionals.

It is desired that Owners adopt and support Collaboration guidelines in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the guidelines recommended herein. In such cases, Owners may adopt these guidelines with the exceptions they feel are necessary.

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AASHTO Publication No: NSBAGDC-4

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FOREWORD

The original G12.1 document was published in 2003. Review comments on the original document were requested from Owners, steel suppliers, Fabricators, Erectors, university professors, and Designers. Based on the review comments and subsequent task group discussions, the document was updated to reflect then current practice and published in 2016. Subsequently, the document was reviewed by another group of independent reviewers, along with task group discussions, comments, and revisions. This current updated document reflects the latest practice for steel mills, fabrication, detailing, erection, and design.

This document has been prepared as a guide and thus much of the information is general in nature, representing the latest steel industry positions. Recommendations should not be considered as strict rules to be followed by any Contracting Engineer, Authority, Fabricator, or Contractor. Also, this document should be used in conjunction with the other Collaboration documents for further clarification on specific issues.

FURTHER READING

AASHTO LRFD Bridge Design Specifications (2020)

AASHTO LRFD Bridge Construction Specifications (2017)

G1.2, Design Drawings Presentation Guidelines (2003)

G9.1, Steel Bridge Bearing Design and Detailing Guidelines (2004)

G13.1, Guidelines for Steel Girder Bridge Analysis (2019)

S8.1, Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges (2014)

S10.1, Steel Bridge Erection Guide Specification (2019)

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SECTION 1

DESIGN

1.1—ROLLED BEAMS VERSUS WELDED PLATE GIRDERS

Where choice is possible between rolled beams and welded plate girders for short span applications, typically rolled beams are considered first (for spans up to 100 ft in length). However, allow the Fabricator to substitute welded plate girders that satisfy design requirements.

C1.1

Generally, rolled beams are more economical than welded plate girders for short spans. Market conditions, girder depth restrictions, and product availability may also affect the decision. Standard designs, including details for rolled beam and welded plate girder solutions (developed by the Short Span Steel Bridge Alliance) are available at <http://www.ShortSpanSteelBridges.org>, or search engine key words: “Short Span Steel Bridge Alliance Standard Designs.”

For increased flexibility, Designers may want to include a plate girder option in the design plans. A welded plate girder option should especially be developed if one of the following criteria is met:

- For horizontally curved members with a radius less than 1,200 ft
- For members requiring camber greater than $\frac{1}{4}$ of the depth of the member (e.g., 6-in. camber for a 24-in.-deep member)
- For members requiring camber greater than $2\frac{1}{2}$ in. on a 50-ft length
- If cover plates are required for the rolled beam option
- The span length is over 60 ft
- Rolled section is not available on short notice

1.2—GIRDER SPACING

When choosing the number of girders in the bridge cross-section, consider the following:

- Owner preferences and limitations
- Cost of steel fabrication, transportation, erection, and coating
- Deck thickness and forming methods
- Provisions for future widening
- Stability and redundancy of the structure during future staged/partial re-decking
- Vertical clearances

C1.2

Many Owners have their own preferences and guidelines for girder spacing. The Designer should refer to these Owner preferences and guidelines, as applicable, when choosing a girder spacing.

For simple spans up to 140 ft, the standards published by the Short Span Steel Bridge Alliance (SSSBA) should be considered.

In general, the use of fewer girders in the bridge cross-section will result in a more economical design because less steel is used.

- Overhang width

The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common historical practice is to make the overhang approximately 0.25 to 0.35 times the girder spacing.

The NSBA website has Steel Span to Weight Curves on the Design Resources page, which show trendlines of data for various ranges of girder spacing as a function of average span length. Wider girder spacings often result in a lower structural steel unit weight (unit weight taken as pounds of structural steel per square foot of deck area).

Generally, for a bridge with an average span length less than 175 ft, there is not an appreciable difference in the structural steel unit weight for the various girder spacings. For a bridge with an average span length more than 175 ft, the Designer may want to consider a wider girder spacing, perhaps between 11 ft and 13 ft, as this wider girder spacing trends to a lighter steel superstructure. However, as noted later in this commentary, there are tradeoffs associated with a wider girder spacing—such as a thicker concrete deck or deeper girders—that may infringe on vertical clearances. The decision on whether or not to choose a wider girder spacing must be made with consideration of other factors and not structural steel unit weight alone.

Typically, the following benefits are derived from the use of wider girder spacing:

- Lower total structural steel weight
- Fewer girders to fabricate, inspect, handle, coat, transport, and erect
- Fewer cross frames to fabricate, inspect, handle, coat, transport, and erect
- Fewer bolts and connections
- Reduced time of fabrication and erection
- Fewer bearings to purchase, install, and maintain

The following issues need to be evaluated during the decision-making process when wider girder spacing is being considered:

- Potential for a thicker concrete deck resulting in more weight, concrete, and reinforcing steel
- Methods for forming the deck
- Optimal (larger) fascia overhang versus an Owner-specified limit

1.3—MINIMUM THICKNESS FOR STIFFENERS, WEBS, AND FLANGES

For welded girder construction, certain minimum requirements for material thicknesses are normally recommended to reduce deformation and the potential for weld defects, as well as increased corrosion resistance/durability.

Use a $\frac{1}{2}$ in. minimum thickness for stiffeners, connection plates, and webs. Use a $\frac{3}{4}$ in. minimum thickness for flanges.

It is preferable to detail stiffeners and connector plates, and smaller gusset plates, so that they can be fabricated from bar stock or cut from larger plate at the Fabricator's discretion.

At girder field splices with plate thickness transitions across the splice, fill plate thicknesses of $\frac{1}{16}$ in. or less should be avoided.

For a web thickness change of $\frac{1}{8}$ in., use a $\frac{1}{8}$ -in. fill on one side of the web rather than $\frac{1}{16}$ -in. fills on each side.

- Girder depth and infringement on vertical clearance

C1.3

Preferred minimum thicknesses depend on the welding equipment used.

To allow the use of flat bar for stiffeners and connection plates, dimensions of these members should follow the parameters below:

- Widths less than 8 in.: increase width in increments of $\frac{1}{2}$ in.
- Widths between 8 in. and 12 in.: increase width in increments of 1 in.
- Thickness: $\frac{1}{2}$ in., $\frac{5}{8}$ in., $\frac{3}{4}$ in., $\frac{7}{8}$ in., 1 in.

Fill plates with thicknesses of $\frac{1}{16}$ in. or less pose difficulties in fabrication and handling. If the fill plates are blast-cleaned, distortion from the blasting operations can become problematic. Handling of large $\frac{1}{16}$ in.-thick fill plates is also difficult due to the plate flexibility. Note that no filler plates are required for web splices with thickness differences of $\frac{1}{16}$ in. or less, per the *AASHTO LRFD Bridge Design Specifications*, Article 6.13.6.1.3c.

If using CNC to drill flanges without assembly, the $\frac{1}{16}$ -in. offset in the web centerline can be programmed, or the $\frac{1}{16}$ -in. hole oversize may be sufficient to accommodate flange assembly.

1.4—MATERIAL SIZE AVAILABILITY

1.4.1 Plate Material Size Availability

When sizing girder flanges, maximum lengths available for the various plate widths and thicknesses should be considered.

For the design, select material that is readily available. Tables 1.4.1-1 and 1.4.1-2 show dimensions of typically-available plates. Material in the shaded area is available from three U.S. rolling mills as of press time. Contact a mill or Fabricator for the latest plate availability information.

C1.4.1

The availability of material sizes varies from mill to mill. The minimum width available from one mill is 48 in. and from two others is 60 in. Plate is also available in widths up to 190 in. from one mill.

Designers are encouraged to be aware of this example maximum plate length availability data, but generally should not engage in over-analysis of their designs in an effort to maximize plate usage. The layout of flanges and webs within a given plate to maximize plate usage is best left to steel detailers, who develop the exact shape and dimensions of each plate, understand the intricacies of flange and web nesting within a plate (including issues related to runout, camber, heat curving versus cut curving, cutting loss, etc.), and can best address the specific capacities and preferences of a given mill and Fabricator.

Instead, this data is useful to Designers for identifying the likely need for welded shop splices in flanges and webs. In general, it is prudent to allow the use of optional flange and web shop splices at the discretion of the Fabricator, and to include the preferred details for such splices in the contract plans. Also, when evaluating the economics of introducing flange thickness changes (and their associated mandatory flange shop splices) as discussed in Article 1.5.4, it is useful to have some idea of maximum available plate lengths. If the length of a particular flange exceeds the limits in Tables 1.4.1-1 or 1.4.1-2, that may provide more reason to introduce a thickness change.

In all circumstances, though, keep in mind the guidance provided in Articles 1.5.1 and C1.5.1 regarding limiting the number of different plate thicknesses used in a design.

Table 1.4.1-1—Example Maximum Plate Length Availability
ASTM A709 Grades 36, 50, 50W (all dimensions in inches)

Plate Thickness	Plate Width				
	72	84	96	108	120
½	972	972	972	972	972
¾	1,035	1,035	1,035	1,035	1,035
1	1,035	1,035	1,035	980	808
1½	1,035	1,035	1,035	720	680
2	1,035	1,035	1,035	720	680
2½	1,035	1,006	880	720	680
3	970	838	734	652	587
3½	830	920	800	635	600
4	720	800	685	600	600

Notes: Widths and thicknesses are grouped for convenience. Other widths and thicknesses are available in similar lengths. Interpolate between adjacent values for other size plates. Material in the shaded area is available from three U.S. rolling mills as of press time.

Table 1.4.1-2—Plate Length Availability
ASTM A709 Grade HPS 70W (all dimensions in inches)

Plate Thickness	Plate Width				
	72	84	96	108	120
½	580	580	580	580	580
¾	580	580	580	580	580
1	580	580	580	580	580
1½	580	580	580	580	580
2	580	580	580	580	580
2½	600	600	600	600	600
3	600	600	600	600	600
3½	600	600	600	600	600
4	600	600	600	600	600

Notes: Widths and thicknesses are grouped for convenience. Other widths and thicknesses are available in similar lengths. Interpolate between adjacent values for other size plates. Material in the shaded area is available from three U.S. rolling mills as of press time.

1.4.2 Wide Flange Beam Length Availability

Structural shape sections of various sizes are produced domestically.

Refer to the American Institute of Steel Construction (AISC) website for specific section availability at <https://www.aisc.org/steelavailability/>.

C1.4.2

Examples of common rolled beam bridge sections are shown in Table 1.4.2-1. Longer lengths may be available, depending on the Producer.

Table 1.4.2-1: Example Maximum Wide Flange Beam Length Availability
ASTM A709 Grades 50 and 50W

Section Size & Foot-Weight Range	Maximum Length (ft)
W44 × 230–335	100
W40 × 431	80
W40 × 149–397	100
W36 (all)	100
W33 (all)	100
W30 (all)	100
W27 × 368	80
W27 × 336	90
W27 × 301–307	100
W27 × 94–281	100
W24 (all)	65
W21 (all)	50

Note: Information provided by Nucor-Yamato Steel (April 2014)

1.5—PLATE SIZING

1.5.1 Flange Plate Thickness

Limit the number of different plate thicknesses for a project. Select flange thicknesses in at least $\frac{1}{8}$ -in. increments up to $2\frac{1}{2}$ -in. and $\frac{1}{4}$ -in. increments over $2\frac{1}{2}$ in. See Article 1.3 for minimum flange plate thickness criteria.

When locating flange thickness transitions (shop flange splices), include no more than two butt splices or three different flange thicknesses for an individual flange between field splices, except for unusual cases such as very long or heavy girders or mill length availability limits.

At welded flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate as a rule of thumb.

C1.5.1

An economical individual girder shipping piece has from one to three thicknesses per flange, with each flange having zero to two shop-welded splices. More flange thickness changes are usually not economical and should be avoided unless the girders are unusually heavy or limits on available plate lengths necessitate additional shop flange splicing with or without a thickness change. Availability of material sizes varies from mill to mill; see Article 1.4 for more information. Minimizing the number of flange plate thicknesses for a project reduces mill quantity extras and simplifies fabrication and inspection operations. See Table 1.5.4-1 for information on when thickness transitions are economically justified.

1.5.2 Flange Plate Width

Size flange material so that flanges can be economically cut from plate between 60 in. (preferably 72 in.) and 96 in. wide, even where girder flanges vary from girder to girder.

Keep individual flange widths constant within an individual shipping piece. When changing flange widths is unavoidable, avoid changing flange width at welded shop splices.

For straight plate girders, group flanges to make efficient use of material. For straight plate girders comprised of three flange plates, keep the center plate width, thickness, and length constant between girder lines so that shops can order material as wide plate. Keep the end plates the same width as the center plate, and use a common thickness so that shops can order material as wide plate, and then splice it as shown in Figure 1.5.2-1.

Larger order quantities of plate cost less. Similar sizes of flanges obtained during preliminary design should be grouped to minimize the number of thicknesses of plate that must be ordered. For example, if preliminary design results in eight thicknesses of $1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{3}{4}$, $1\frac{7}{8}$, 2, $2\frac{1}{8}$, and $2\frac{1}{2}$ in., consider optimizing the design to four plate thicknesses of $1\frac{1}{4}$, $1\frac{1}{2}$, 2, and $2\frac{1}{2}$ in., or a similar grouping.

C1.5.2

The most economical size plate to buy from a mill is between 72 in. and 96 in. wide. For size availability, see Article 1.4. Fabricators order plate with additional width and length to account for cutting ($\frac{1}{8}$ in. per cut between plates and along sides), plate sweep tolerance, and waste (about $\frac{1}{2}$ in. on each outside edge). For example, a Fabricator might order a plate 74 in. wide to cut five 14 in. wide plates (4 in. of wasted material).

For straight girder bridges, Fabricators order girder flange material from wide plate and splice it either as wide plate or as individual flanges after cutting to width. For constant-width flanges, advantages to welding wide plate rather than stripping and then splicing include having one set of run-on tabs and run-off tabs, fewer crane moves, and considerably fewer weld starts and stops. Changes in thickness rather than width in a field section save as much as 35 percent of the labor required to join the flanges. However, shops frequently decide whether to weld first or strip first depending on crane capacity, hook height, and other individual preferences.

Because flange material with butt splices must be ordered as wide plate and then spliced and stripped or stripped and spliced, a Designer should size the flanges so that plates can be ordered with minimal waste. For bridges with non-parallel supports where the geometry of the flanges could vary from girder to girder, a Designer should consider how material might be ordered and spliced. See Figure 1.5.2-1 as an example.

For curved plate girders:

- Size flanges to get as many pieces as possible from a wide plate.
- Keep flanges in each area the same thickness and approximate length to allow splicing as shown in Figure 1.5.2-2.
- Maintain constant flange widths full length within a field section and consider nesting during sizing of plates for curved girders. (In fabrication, nesting is the technique of laying out component cutting patterns on a plate to optimize material use.) See Figure 1.5.2-2.

When in doubt, consult a Fabricator.

For curved-girder bridges, if the Fabricator chooses to heat-curve the members, the approach will be the same as for bridges with non-parallel supports, and the shop will curve the members after completing most of the fabrication. If the Fabricator chooses to cut-curve the members, the amount of material that will be wasted in cutting the curve is an additional consideration.

As an example of the material wasted, if the radius for the flanges in Figure 1.5.2-2 was 700 ft and the center plate was 2 in. thick by 60 ft long, the amount of waste for the center plate (the shaded area) would be about 3,100 pounds whether the plate cuts four flanges or one flange. Depending on whether adjacent girders use common flange thicknesses and transition points, some Fabricators may choose to splice the flanges as wide plate similar to straight girders and some will cut curve the plates to width prior to splicing. In either case, the amount of waste material may be significant. In the interest of economy, the Designer should consider how material might be ordered and spliced. See Figure 1.5.2-2 as an example.

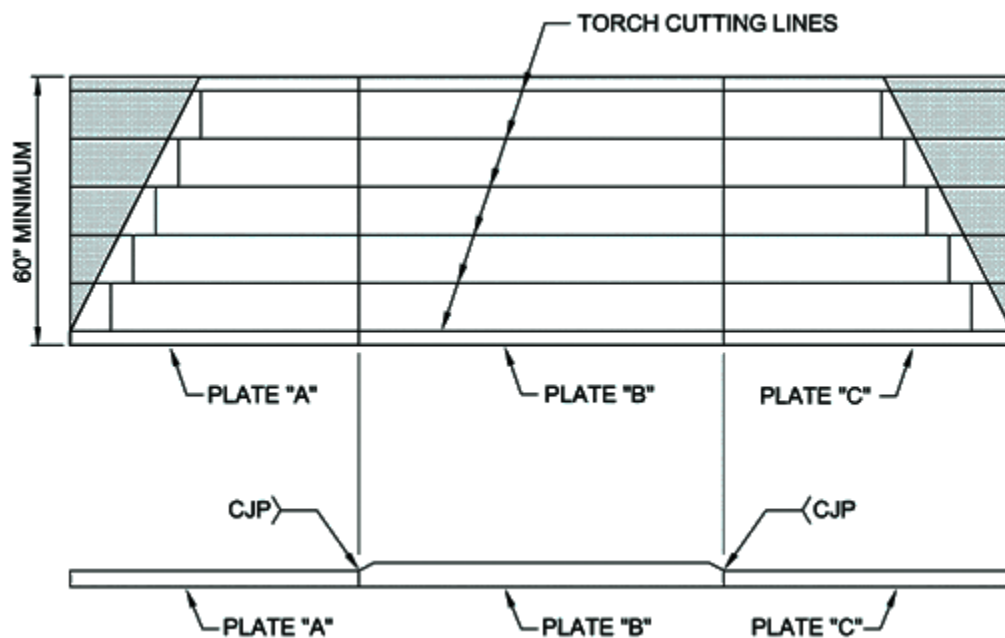


Figure 1.5.2-1

Straight Girder Example of Welding Wide Plates, then Stripping out the Flanges

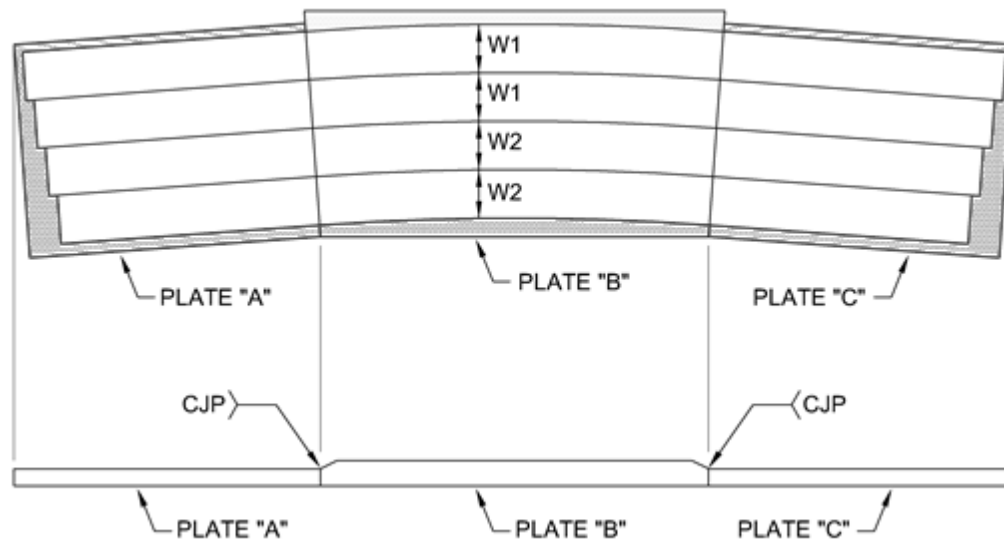


Figure 1.5.2-2

Curved Girder Example of Welding Wide Plates, then Stripping out the Flanges

1.5.3 Web Sizing

See Article 1.3 for minimum web plate thickness.

Minimize the use of transverse intermediate stiffeners.

The selection of web depth should consider geometric issues (such as minimum vertical clearances), fabrication and transportation issues, and overall economy of the design.

C1.5.3

A web thickness 1/16" less than that required to achieve an unstiffened web typically leads to a "partially stiffened" web design in which a few stiffeners may be needed near the supports. Partially stiffened web designs are generally considered the most economical choice for routine steel girder highway bridges. Excessive use of intermediate stiffeners is usually uneconomical due to the increased cost of fabrication.

When establishing girder web depths, first verify that minimum vertical clearance requirements under the bridge can be met. Other considerations include fabrication issues (maximum plate widths versus the need to introduce longitudinal shop splices in the web), transportation issues (maximum girder dimensions for transportation), and overall economy.

Economy is achieved using girders that can be shipped web vertical by truck, which is limited by overhead clearances on the shipping route. Girders that are under 9' deep can generally be shipped vertically on a truck.

Above this height it will depend on the route from the Fabricator to the site. Girders that are too deep to ship vertically can be shipped with the web horizontal, but supporting the full length with the girder's weak axis in bending tends to be much more challenging and costly. Horizontal limits also depend upon constraints along the route, such as toll booths, bringing the girder vertical at the jobsite, and other issues of practicality. It may be possible to ship girders 18' deep horizontally, but for bridge members at this depth it is prudent to check with potentially affected state Departments of Transportation (DOTs) regarding permits, a major Fabricator, and a hauling contractor about actual constraints.

Deeper girders are generally more economical, but only up to a point. To assess overall economy, it may be valuable to perform a web depth study where the web depth is incrementally increased, the girder is redesigned (targeting a partially stiffened web design), and the resulting girder weight versus web depth is recorded. These data points (girder weight versus web depth) can then be plotted to determine the optimum (minimum girder weight) web depth. Some steel girder design software packages (e.g., LRFD SIMON by NSBA) offer automated web depth study features; otherwise the study can be performed by simply iterating the design with different web depths.

Avoid the use of longitudinal web stiffeners in routine steel girder bridges.

Typically, the use of longitudinal web stiffeners is only economical in long-span steel girders where using very slender webs can lead to significant reductions in girder weight.

See Articles 2.1.2.3 and 2.1.2.4 for further discussion.

1.5.4 Shop-Welded Splices

Introduce a shop flange splice and a flange thickness transition when the weight savings will justify the work associated with the welded splice. Table 1.5.4-1 provides a method to make the evaluation.

Specify a shop-welded splice when the savings in flange material and when plate length limitation or special circumstances dictate. Table 1.5.4-1 provides a method to make the evaluation.

C1.5.4

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. The parameters affecting the cost of shop-welded flange splices vary from shop to shop. For both straight and curved girder bridges, Fabricators often request to eliminate a shop splice by

In the design or specifications, provide criteria the Fabricator may follow to eliminate shop-welded flange splices by extending thicker plate.

extending a thicker flange plate. Design and specifications should consider allowing this practice, subject to the approval of the Engineer. When evaluating the request, Designers should review the percent change in deflections and stresses.

Many Owners have guides for economical flange thickness transitions. Some have graphs based on thickness change, length of change, and the thicker plate, but others use experience-based guidelines (e.g., the Texas Department of Transportation, a.k.a. TxDOT, estimates saving 800 to 1,000 pounds may justify a butt splice). Table 1.5.4-1 shows weight savings per inch of flange width that may be used to evaluate placement of shop splices. The criteria vary, especially for large curved girders, so Fabricators should be consulted whenever possible.

The following example demonstrates the use of Table 1.5.4-1:

Evaluate splicing a plate 16 in. $\times$ 1 in. $\times$ 35 ft to a plate 16 in. $\times$ 1½ in. $\times$ 35 ft versus using a plate 16 in. $\times$ 1½ in. $\times$ 70 ft. The weight saved by adding the splice is equivalent to the weight of a plate 16 in. $\times$ ½ in. $\times$ 35 ft (16 in. $\times$ 0.5 in. $\times$ 3.4 lbs/in.²/ft $\times$ 35 ft = 952 lbs), about 950 pounds. The weight savings needed to justify adding the splice is determined by using a factor of 70 pounds per inch from Table 1.5.4-1, times the plate width of 16 inches, resulting in a value of 1,120 pounds. Because the actual saving is 950 pounds, Table 1.5.4-1 indicates that it is more economical to extend the 1½ in. plate for the full 70 ft than to add the shop splice.

**Table 1.5.4-1: Estimated Weight Saving Factor per Inch of Plate Width
For ASTM A709 Grade 50 Non-Fracture-Critical Flanges Requiring Zone 1 CVN Testing**

Multiply weight savings/inch × flange width (length of butt weld)							
Thinner Plate at Splice (inches)	Thicker Plate at Splice (inches)						
	1.0	1.5	2.0	2.5	3.0	3.5	4.0
1.0	70	70	70	—	—	—	—
1.5	—	80	80	80	80	—	—
2.0	—	—	90	90	90	70	70
2.5	—	—	—	100	100	80	80
3.0	—	—	—	—	110	90	90
3.5	—	—	—	—	—	110	110
4.0	—	—	—	—	—	—	130

Notes:

- Source: compiled from various Fabricators, November 2001.
- Weight factors for non-fracture-critical Zone 2 material are the same as for Zone 1, as shown, except that in the shaded areas the factors should be reduced by 20 percent.
- For compression flanges where Charpy V-Notch (CVN) testing is not required, the factors should be increased by about 10 percent, except the bottom two rows should increase by about 30 percent.
- For fracture-critical material, the factors should be reduced by values between 10 percent and 25 percent, depending upon the thickness.
- Materials other than A709 Gr. 50 will have values that will vary from those shown in the table.
- For intermediate thicknesses, interpolate between closest values.
- Where equal plate thicknesses are joined, table values indicate welded splice cost in terms of steel weight. Steel cost per pound is based on unfabricated steel plate, not the bid price of fabricated, delivered steel.

1.6—DIFFERENTIAL DEFLECTIONS

1.6.1 Fit and Differential Deflections

When choosing a fit condition for a steel I-girder bridge, consider:

- Differential deflections at each cross frame
- Span length
- Radius (for curved bridges)
- The need for lateral rotational capacity of the bearings

For recommendations on what fit condition is appropriate for a given bridge, see the document “Skewed and Curved Steel I-Girder Bridge Fit,” published by NSBA in 2016.

In practice historically, contractors or Fabricators have made the fit choice when not specified by the Designer in the contract plans, but considering the influence the fit condition can have on member loads (as well as constructability), it is prudent for the Designer to make this decision, as is currently

C1.6.1

In an I-girder bridge, “fit” refers to how the cross frames are detailed and fabricated to fit to the girders. They may be:

- Detailed to fit when all dead loads are applied (“total dead load fit,” TDLF, or “final fit”);
- Detailed to fit at erection (“steel dead load fit,” SDLF, or “erected fit”);
- Detailed to fit in the no-load condition (“no-load fit,” NLF, or “fully-cambered fit”); or
- Detailed to fit at some other condition in between.

The girders and cross frames may actually fit at more than one or all of these conditions. The distinction here is not whether or not the bridge components actually fit in these

required by the *AASHTO LRFD Bridge Design Specifications*.

conditions, rather it is how the bridge is detailed to fit.

The detailed fit condition can influence:

- The ability to construct the bridge. For example, choosing TDLF for a sharply curved bridge can make the bridge unconstructable; and
- Internal loads associated with the fit condition.

“Differential deflections” refer to the difference in girder deflection at either end of each cross frame. When differential deflections exist (as they must on skewed and curved bridges), cross frames tend to deflect a different amount on either end. Since the cross frames are very stiff, they cannot easily distort to accommodate these differential deflections, so the result for most bridges is that the girders twist.

For bridges detailed to TDLF, there will be layover (twist) at erection, but generally the girders will come back to plumb under total dead load. Conversely, on a skewed bridge detailed to steel dead load fit, girders will be plumb at erection but will experience some final layover under total dead load. For this reason, the fit condition is sometimes referred to as the “plumb” condition. However, it is not recommended to refer to fit in this way because it confuses the issue—particularly when the “plumb” discussion is extended to curved girders in which layover and plumb do not work the same way.

The NSBA document referenced in the recommendation provides more explanation about these choices and phenomena.

1.6.2 Deflection due to Phased Construction

Consider using single member top and bottom struts; omission of the cross frames or diaphragms between construction phases; or use field-drilled holes, slotted holes, or field-welded connections where phased construction would cause significant differential deflection in the bay between previous and new construction (phase 1 versus phase 2, existing versus widening, etc.). Note that for curved girders, the *AASHTO LRFD Bridge Design Specifications* do not allow slotted or oversize holes.

Provide a deck placement sequence diagram in the contract plans.

C1.6.2

If phased construction is required, the differential deflection between phases due to the application of dead loads at different times can be significant. There are many ways to address this in the design and detailing of cross frames between the adjacent construction phases.

The use of independent single-member top and bottom struts without diagonals (also known as “lean-on” bracing) with a single bolt in each end is one simple and effective possible way to address this situation when the differential deflections are not too significant.