Steel Bridge Design Handbook

CHAPTER 14

Splice Design

February 2022

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Foreword

The Steel Bridge Design Handbook covers a full range of topics and design examples to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. The Handbook has a long history, dating back to the 1970s in various forms and publications. The more recent editions of the Handbook were developed and maintained by the Federal Highway Administration (FHWA) Office of Bridges and Structures as FHWA Report No. FHWA-IF-12-052 published in November 2012, and FHWA Report No. FHWA-HIF-16-002 published in December 2015. The previous development and maintenance of the Handbook by the FHWA, their consultants, and their technical reviewers is gratefully appreciated and acknowledged.

This current edition of the Handbook is maintained by the National Steel Bridge Alliance (NSBA), a division of the American Institute of Steel Construction (AISC). This Handbook, published in 2021, has been updated and revised to be consistent with the 9th edition of the AASHTO LRFD Bridge Design Specifications which was released in 2020. The updates and revisions to various chapters and design examples have been performed, as noted, by HDR, M.A. Grubb & Associates, Don White, Ph.D., and NSBA. Furthermore, the updates and revisions have been reviewed independently by Francesco Russo, Ph.D., P.E., Brandon Chavel, Ph.D., P.E., and NSBA.

The Handbook consists of 19 chapters and 6 design examples. The chapters and design examples of the Handbook are published separately for ease of use, and available for free download at the NSBA website, www.aisc.org/nsba.

The users of the Steel Bridge Design Handbook are encouraged to submit ideas and suggestions for enhancements that can be implemented in future editions to the NSBA and AISC at solutions@aisc.org.

TECHNICAL REPORT DOCUMENTATION PAGE

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7. Supplementary Notes The previous edition of this Handbook was published as FHWA-HIF-16-002 and was developed to be current with the 7th edition of the AASHTO LRFD Bridge Design Specifications. This edition of the Handbook was updated to be current with the 9th edition of the AASHTO LRFD Bridge Design Specifications, released in 2020.	
8. Abstract Typically, it is not possible to fabricate, handle, ship or erect the entire length of a girder in one piece. In these cases, provisions must be made to splice multiple pieces of the girder together in the field to provide the required length. These splices must be capable of transmitting the shear and moment in the girder at the point of the splice. This volume focuses on the factors which influence and the principles of the design of bolted field splices. Factors that influence field splice design and layout are presented, including span layout, curvature, and girder properties. General design provisions are also addressed in this module, including flexural resistance provided by a bolted field splice at the Strength and Service limit states, as well as detailing considerations.	
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Steel Bridge Design Handbook: Splice Design

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1.0 INTRODUCTION

Often, it is not possible to fabricate, handle, ship or erect the entire length of a girder in one piece. In these cases, provisions must be made to join, or splice, multiple pieces of the girder together to provide the required length. These splices must be capable of transmitting the shear and moment in the girder at the point of the splice.

When done in the fabrication shop, the splice is defined as a shop splice as opposed to a field splice which is performed at the project site. Field splices can be performed either on the ground prior to erection or in the erected position.

After a brief description of welded shop splices, this volume focuses on the factors which influence and the principles of the design of bolted field splices. Following this discussion, a design example is provided which demonstrates the current bolted splice design provisions of the AASHTO *LRFD Bridge Design Specifications, 9th Edition, (2020)* (referred to herein as AASHTO LRFD BDS), (1). Throughout this volume, specific provisions of these Specifications are referred to by Article number.

Figure 1 Photograph of a Steel Plate I-Girder Field Splice

2.0 SHOP SPLICES

The steel plates which are used for the webs and flanges of girders are available in limited lengths and widths. Availability of plate thicknesses, widths and lengths may vary by plate manufacturer, and are typically listed in "plate availability tables" available from the manufacturers.

Figure 2 Photograph of a Steel Plate I-Girder Bolted Field Splice at a Flange Width Transition

In order to join pieces of a particular girder component (web or flange) together in the fabrication shop, shop splices are made. These shop splices are generally complete joint penetration (CJP) butt-welded splices. As shown in Figure 3, groove welds are utilized to join the pieces together.

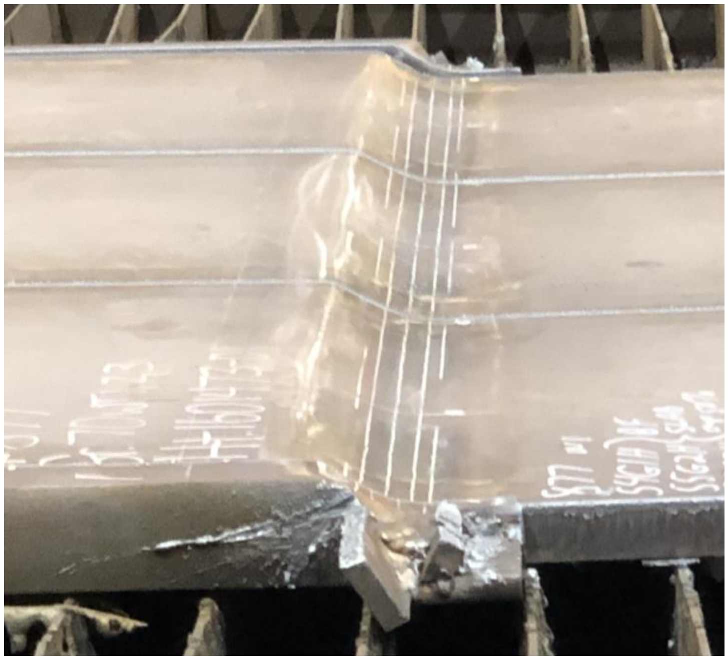

Figure 3 Photograph of a butt-welded slab shop splice prior to cutting flanges

Where the thickness of a component remains constant, the fabricator will only provide shop splices where necessary in order to minimize the cost of fabrication. This is generally not a consideration for the bridge designer, beyond provision of the appropriate details in the bridge plans.

However, a designer's decision of whether or not to provide a plate thickness transition (for a flange), which necessitates a shop splice, can influence the economy of the design. The AASHTO/NSBA Steel Bridge Collaboration G12.1 *Guidelines to Design for Constructability and Fabrication* provides guidance on whether a potential savings in material cost (by reducing the flange plate thickness) offsets the cost of edge preparation, fit-up, welding and testing associated with providing a welded shop splice (2). The weight savings required to produce an economical welded shop splice is dependent on the width and thickness of the smaller of the plates being joined. If the weight savings is not sufficient, it is more economical to extend the heavier flange plate.

Some other design considerations for welded shop splices are listed below.

- Since the CJP weld develops the full-strength of the connected plates, actual welded shop splice design by the bridge designer is not required.

- Changing the width of a flange plate within a field section (at a shop splice) is not recommended because it can preclude the "slabbing and stripping" method of girder fabrication often employed for economy. Fabricators generally purchase plate material of the required thicknesses as wide slabs directly from the mill or from suppliers. After preparing the edges, the slabs are welded together as shown in Figure 4. This reduces welding time and eliminates many of the runout tabs and the excess weld metal which are wasted by welding the flanges as individual pieces. This topic is discussed in greater depth in the volume titled *Stringer Bridges and Making the Right Choices* of the Steel Bridge Design Handbook.

- It is not advisable to reduce the area of a flange by more than 50% at any flange transition location. A 40% to 50% reduction in area is typical. For longer spans (> 180'), the reduction may be as small as 30% and still be economical.

- Repeating the same plate arrangements for multiple flange locations can help reduce the cost of fabrication since the fabricator can reduce the amount of fit-up and testing which must be performed.

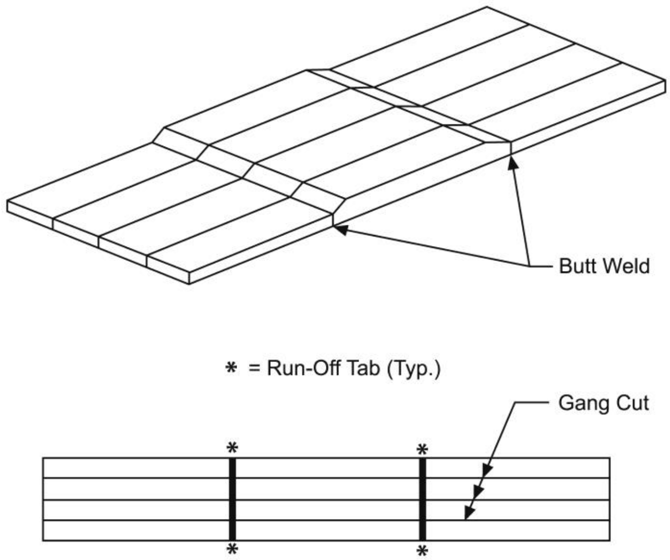

Figure 4 Sketch Illustrating Slabbing and Stripping of Flanges

- Flange thicknesses should be changed using the following increments:

  - 3/4" < t < 2" – use 1/8" increments

  - 2" ≤ t – use 1/4" increments

- The minimum length for any flange plate should be limited to 10'.

- Top and bottom flange transitions do not have to be placed at the same locations along the girder. Placing them at different locations will result in additional design effort (more sections to check), but may result in more efficient plates since the top and bottom flanges are governed by different requirements.

- Flange transitions in positive moment sections are typically uneconomical except for very long spans. The top flange is usually governed by the b/t ratios and lateral buckling stresses in the flange prior to the hardening of the concrete slab. Therefore, the stresses in the top flange only reach 35 to 50 percent of the flange capacity under full factored loads. A bottom flange transition may be economical in the end spans near the end support due to the small live load and fatigue load force effects that must be carried by the section, but bottom flange transitions between the points of maximum positive moment and points of dead load contraflexure are generally not economical due to the large fatigue stresses and possible compressive stresses in the bottom flange.

- Flange transitions in negative moment sections are common. Typically, one top and one bottom flange plate transition between the centerline of the interior pier and the adjacent bolted field splice is generally economical.

- Longitudinal web splices are costly due to the welding associated with joining the plates together. Some points to remember when designing deep girders are:

- Web depths in excess of 130" (assuming 6" for camber cuts and 136" available plate width) may necessitate a longitudinal web splice, which can significantly increase the cost of the girder.

- Girders that are haunched to depths greater than 130" may also require a longitudinal web splice (typically welded, but sometimes bolted).

The AASHTO/NSBA Steel Bridge Collaboration *G1.4, Guidelines for Design Details* provides recommended details for welded shop splices in plate girders, as shown in Figure 5 (3).

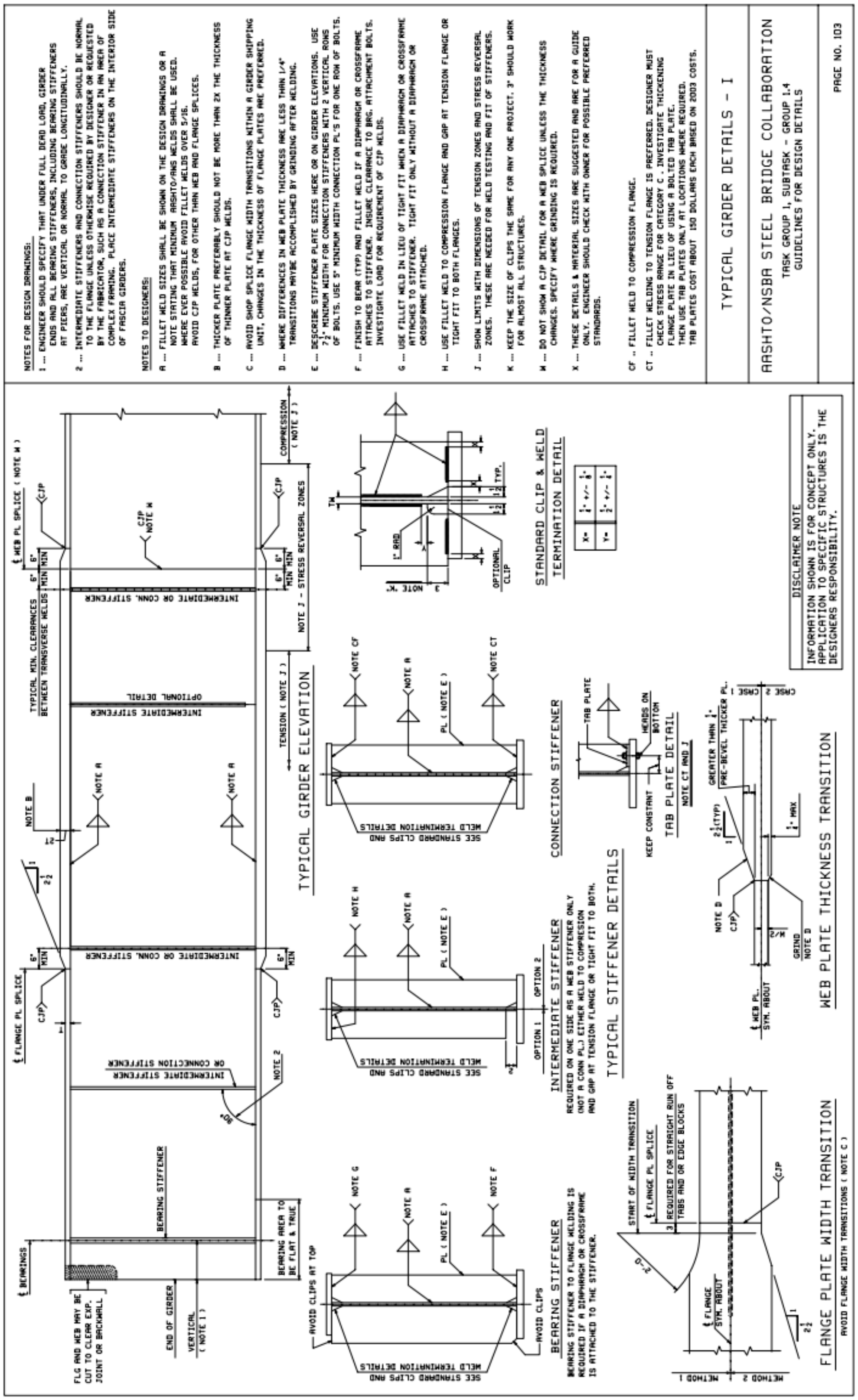

Figure 5 NSBA Recommended Shop Welded Splice Details (3)

3.0 FIELD SPLICES

Field splices are the connection of separate sections of the girders (field sections) which occur in the field. Generally these splices are bolted connections as field welding is usually discouraged due to the high cost of field welding and problems associated with welding in less than ideal conditions, such as inclement weather. It should be noted that some agencies do permit welded field splices, although their use is becoming less frequent.

Due to piece shipping constraints and job site constraints, the majority of bridges that exceed 120 to 140 feet in length will require at least one field splice somewhere in the girder.

Introducing a bolted field splice in the girder also introduces a region where stresses may be significantly higher than in the areas which are immediately adjacent due to the reduced area of the flange plates. The area of the web and girder flange plates is reduced by the holes for the connection bolts; these areas also produce stress concentrations around the holes themselves which are not easily quantified.

Field splices can be made between girder sections (or components) which are either similar in size or of different width and/or thickness.

The remainder of this volume will focus on the factors which influence the design of bolted field splices.

4.0 FACTORS INFLUENCING FIELD SPLICE DESIGN

Many factors influence the design of steel beam and girder field splices. Some of the more important factors are described below. While this list may not be all-encompassing (there are often unforeseen or project-specific issues that could also influence the design), the factors discussed generally play an important role in the field splice design process.

4.1 Span/Structure Length

Perhaps the most important factor influencing the locations and design of field splices is the length and the span arrangement of a bridge structure. If the overall structure length is less than the length of girder that can reasonably be fabricated, shipped, handled and erected, then field splices are not necessary, and should not be included in the design. Many agencies have limitations on shipping dimensions and weights. Designers should investigate local regulations and contact regional fabricators prior to making decisions on girder lengths.

If the overall girder length must be longer than can reasonably be fabricated, shipped, handled and erected, then at least one field splice will be required. Selecting the appropriate locations of field splices is influenced by other factors, as described in the following sections.

Often, based on perceived shipping, handling and erection considerations, the designer may provide for field splices in the design. However, it is possible that a contractor may be able to utilize some other mode of transportation to ship the girder pieces or employ an unanticipated method to erect them, which may eliminate the need for certain field splices. In recognition of this fact, some agencies stipulate that either all or certain field splices should be designated as "optional". Such requirements should be verified with the particular agency during the design process.

4.2 Span Layout

Continuous Spans: On continuous structures, it is normal for the designer to attempt to locate field splices near the points of dead-load contraflexure (where dead load bending effects change from positive to negative and are therefore zero). As specified in AASHTO LRFD Article 6.13.6.1.3a, bolted splices in continuous spans should be made at or near points of permanent load contraflexure if possible. Splices located in areas of stress reversal near points of permanent load contraflexure are to be investigated for both positive and negative flexure to determine the governing condition.

Simple Span: Since there are no dead load inflection points on a simple-span bridge, as the entire girder is in positive bending, it is not possible to locate field splices near points where dead load bending effects are nearly zero. The field splices for simple span structures are therefore sometimes larger than those for continuous bridges, as shown in Figure 6.

Figure 6 Photograph of a Field Splice for a Long Simple Span Steel I-Girder

Continuous for Live Load: Occasionally it is possible to provide field sections for a multi-span structure which can span from support to support. This option can minimize the need for erection devices (such as falsework or pier brackets) and can reduce the impact on traffic beneath the structure. In these cases a girder splice is often made at the supports to provide continuity of the girder (and thereby eliminate the need for a deck expansion joint, which is desirable). These splices are often made after the majority of the deck has been cast and therefore the splice need only be designed for live load effects. These types of splices are somewhat unique, and are not discussed in further detail in this volume.

4.3 Curvature

If a girder is curved in plan, it can influence both the location chosen for (frequency of) and design of the field splices. The radius of the curvature determines the amount of sweep of a particular length (arc length) of girder. If the sweep is too large then the section cannot be reasonably shipped and an additional field splice may be required. Typically this will only control for relatively tight radii. This is demonstrated in Figure 7. For example, a 120 ft long piece with a radius of 400 ft has a sweep, S = 4.5 ft.

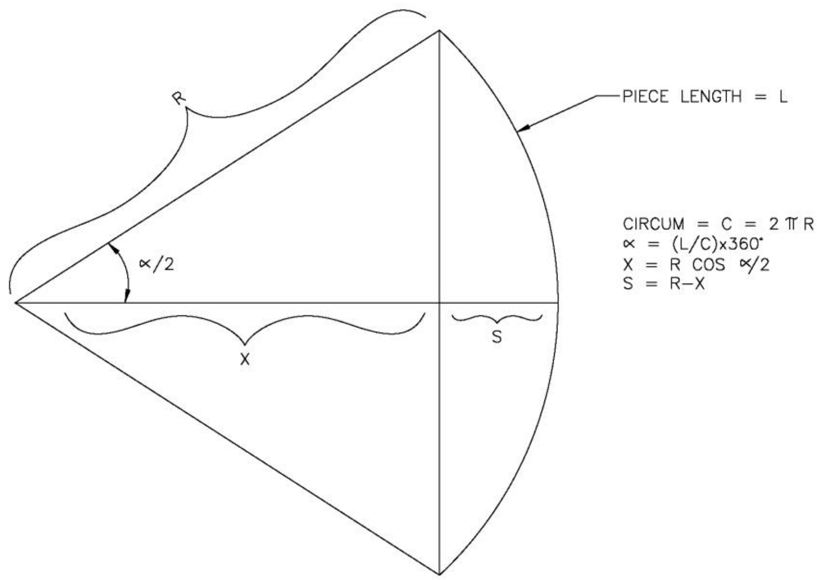

Figure 7 Sketch of a Curved Girder Sweep

The flanges of a horizontally curved girder are subjected to flange lateral bending. However, for flanges with one web in straight bridges and in horizontally curved girders, the effects of flange lateral bending do not need to be considered in the design of bolted flange splices since the combined area of the flange splice plates will typically equal or exceed the area of the smaller flanges to which they are attached. In the current AASHTO LRFD BDS (1) bolted field splice design provisions, the effects of flange lateral bending due to curvature do not need to be considered in the design of the flange splices since the flange splices are designed to develop the full yield capacity of the flanges, which cannot be exceeded in the design of the flanges for combined major-axis and lateral bending for constructability and at the strength limit state.

4.4 Girder Properties

Steel Grade – The field splice design provisions of the AASHTO LRFD BDS are applicable for all grades of steel commonly used for bridge construction (F_y = 36, 50, 70, 100 ksi).

Hybrid Plate-Girders – Hybrid plate-girders are those which have flange and web plates with different yield strengths. The current bolted field splice design provisions in the AASHTO LRFD BDS do not consider the hybrid factor, R_h. The hybrid factor, R_h, as well as the load-shedding factor, R_b, are not included since they are considered in the calculation to determine the flange sizes. The bolted splice connections are designed to carry the full design yield resistance of the flanges.

Flange Width – As a general rule-of-thumb, it is advisable to use a minimum flange width of L/85, where L is the length of the field piece. Doing so helps to prevent difficulties during handling and erection. Some agencies have even more restrictive minimum flange width ratios.

4.5 Coordination of Details

Cross-frame Connection Plates/Transverse Stiffeners – Field splices should be located far enough away from cross-frame connection plates/transverse stiffeners that there will be no interference between the connection plates/stiffeners and the splice plates. In preparing preliminary layouts for the framing, it is advisable to leave a minimum of 3' (and preferably 4') between proposed cross-frame or stiffener locations and the centerlines of proposed field splices to provide adequate length for the flange splice plates. On rare occasions, girders are kinked at field splice locations to accommodate complex (curved or flaring) framing; in these cases, providing a cross-frame as close as possible (but not interfering) with the splice is advisable to help resist lateral loads on the girder due to the kink.

Longitudinal Stiffeners – Two options are possible where longitudinal stiffeners intersect field splices. Termination of the longitudinal stiffener at the splice location (if acceptable by design) can be accomplished by providing a radiused end treatment at the end of the longitudinal stiffener, or the longitudinal stiffener can be spliced at the field splice location. The termination point of longitudinal stiffeners is known to be a fatigue sensitive detail. Designers should verify the fatigue resistance of the details chosen.

Inspection Handrails – When inspection handrails are attached to the girder web, they will need to be continued over the web splice, but not interfere with the splice. Figure 8 provides a method to maintaining the handrail continuity across the web splices.

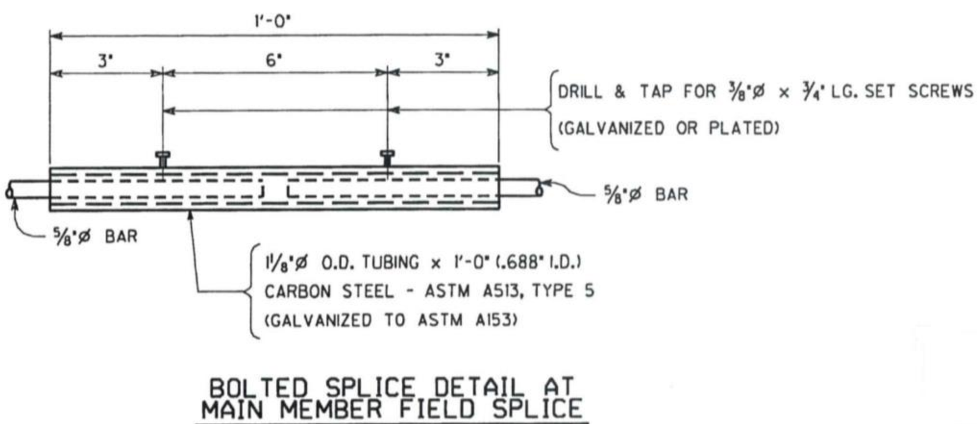

Figure 8 Sketch of a Handrail Detail at Field Splice

4.6 Other Factors

Shipping Requirements – Most states require hauling permits for loads over a certain length and/or certain weight. Beyond length and weight, special permits and/or restricted routes (i.e. multi-lane highways) may be required. Designers should review agency hauling restrictions and requirements, but can also check with regional fabricators about shipping lengths and weights from their shop to the particular job site. These restrictions and requirements can have a impact on where field splices should be located. While a contractor may be able to obtain the required special permits to allow shipping of longer pieces and thereby eliminate a field splice, it may not be prudent to make this assumption during the design phase. Some agencies may require that shipping requirements for a particular design be included in the contract documents.

Project Site Accessibility – This factor may drive or go hand-in-hand with the shipping requirements. Remote locations which are accessible by minor roadways (which may have several tight turns) may prevent field piece lengths that would normally be acceptable. In addition, a site which requires girder picks which are very tall or at a long reach may limit the weight of individual girder pieces unless a prohibitively large crane is used.

Steel Erection Limitations – Girder pick limitations discussed previously are a consideration. Also, longer or imbalanced field sections can force the erector to utilize additional falsework, pier brackets, stiffening trusses and other types of erection devices. These factors can influence the decision on limiting or establishing an appropriate field piece length and layout.

5.0 FIELD SPLICE DESIGN – GENERAL CONSIDERATIONS

5.1 Previous Bolted Field Splice Design Provisions

The bolted field splice provisions prior to the 8th edition of the AASHTO LRFD Bridge Design Specifications were to be designed at the strength limit state for not less than the larger of:

- The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of the splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or

- 75 percent of the factored flexural, shear, or axial resistance of the member or element.

Where the girder section changes at a splice, which is frequently the case, the "smaller" section was to be used for determining the above requirements.

The above requirements were relatively straightforward when applied to a splice or connection for a truss member subject only to axial tension or compression since the stress is equal in the various components of the member. Application of this rule to the design of a bolted splice for a composite steel flexural member becomes more complex however since the stresses in the flanges are typically not equal, and the distribution of the stress in the web is a function of the loads applied to the composite and non-composite sections. In most designs, the factored resistance of the member controls the design of the bolted splice since the Engineer typically places the splice in a low-moment region near the point of dead-load contraflexure.

These requirements were initially replaced in the 8th edition of the AASHTO LRFD Bridge Design Specifications.

5.2 Current Bolted Field Splice Design Provisions

Beginning with the 8th edition AASHTO LRFD BDS, the bolted field splice provisions for flexural members were significantly revised to simplify the design process. Experimental research at the University of Texas showed that a simpler method of design, on which the design procedure provided in the 8th Edition AASHTO LRFD BDS is based in principle, produced a connection with adequate design resistance (7, 8). The results showed that the web did not carry significant moment until the flange connection slipped. After the flange connection slipped, the web connection slipped and the force in the web did not increase until the flange bolts went into bearing and the flange yielded.

The design procedure for the design of bolted splices for flexural members introduced in the 8th edition AASHTO LRFD BDS, and subsequently provided in the 9th Edition LRFD Bridge Design Specifications (1), is based upon designing the bolted flange and web splice connections for 100 percent of the individual design resistances of the flange and web; that is, the individual flange splices are designed for the smaller design yield resistance of the corresponding flanges on either side of the splice, and the web splice is designed for the smaller factored shear resistance of the web on either side of the splice. Therefore, the method satisfies the AASHTO design criteria since the web and flange splices have design resistances equal to the design resistances of their respective components. However, additional forces in the web connection may need to be considered if the flanges are not adequate to develop the factored design moment at the point of splice. No additional checks of the web connection shear resistance are required.

Furthermore, in accordance with the new design provisions bolted splice connections for flexural members are to be designed as slip-critical connections. Slip-critical connections are proportioned to prevent slip under Load Combination Service II and to provide bearing and shear resistance under the applicable strength limit state load combinations. In addition, bolted connections for flange and web splices in flexural members are proportioned to prevent slip during the casting of the concrete deck to provide geometry control.

5.3 Current Design Procedures and Examples

The NSBA publication *Bolted Field Splices for Steel Bridge Flexural Members: Overview & Design Examples* () guides an engineer through how to design a bolted splice connection from start to finish using the current AASHTO LRFD BDS (1). The publication is available for free download at the NSBA website (www.aisc.org/nsba), and it provides a comprehensive overview of the current design procedures, in addition to three design examples. Design Example 1 is an example design of a bolted field splice for the interior girder of an I-section flexural member. Design Example 2 is an example design of a bolted field splice for the exterior girder of an I-section flexural member. Design Example 3 is an example design of a bolted field splice for a horizontally curved continuous twin trapezoidal tub girder on radial supports.

Furthermore, NBSA has developed NSBA Splice, which is presented in an easy-to-understand Microsoft Excel spreadsheet form and can be used by designers to for the design and checking of a bolted field splice connections used in steel bridge flexural members. NSBA Splice is based on the current AASHTO LRFD BDS (1) and allows users to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design. NSBA Splice is available for free download at the NSBA website (www.aisc.org/nsba).

6.0 ROLLED BEAM SPLICE DESIGN

The procedure for the design of field splices for rolled beams is similar to the procedure for the design example of plate girders. Webs are to be spliced symmetrically by plates on each side. The splice plates are to extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances or fillet areas on rolled beams.

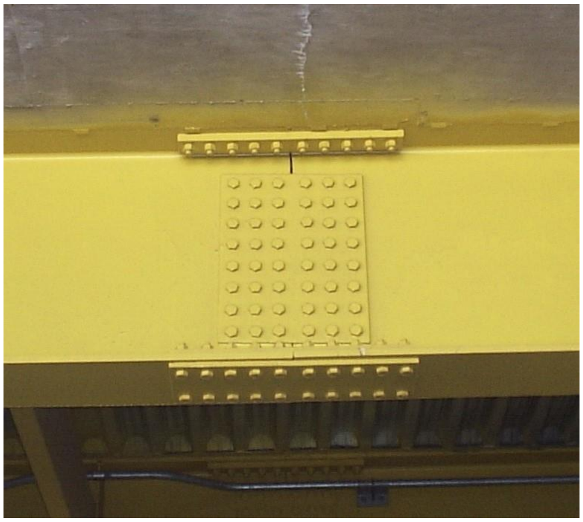

Figure 9 Photograph of a Rolled Beam Field Splice

7.0 BOX GIRDER SPLICE DESIGN

The primary differences between the design of bolted field splices for plate-girders and box girders are provided below.

7.1 Flange Splice Design

The vector sum of the St. Venant torsional shear and the design yield resistance is to be considered in the design of the bottom flange splice at the strength limit state (AASHTO LRFD Article 6.13.6.1.3b) for the following box sections:

- single box section in straight bridges
- multiple box section ins straight bridges which do not satisfy the requirements (including geometric restrictions and no skew) of AASHTO LRFD BDS Article 6.11.2.3
- single or multiple box section in horizontally curved bridges
- single or multiple box sections with box flanges not fully effective according to the provisions of AASHTO LRFD BDS Article 6.11.1.1

Longitudinal warping stresses due to cross-section distortion may be ignored at the strength limit state since the bottom flange splices are designed to develop the full design yield capacity of the flanges. These longitudinal warping stresses are typically relatively small in the bottom flange at the service limit state and for constructability and may be neglected when checking the bottom flange splices for slip. Furthermore, flange lateral bending due to curvature is not a consideration for bottom flanges of box girders.

The effects of flange lateral bending need not be considered in the design of the top-flange splices since the flange splices are designed to develop the full yield capacity of the flanges, which cannot be exceeded in the design of the flanges for combined major-axis and lateral bending for constructability and at the strength limit state, and because the top flanges are continuously braced at the strength limit state. Longitudinal warping stresses due to cross-section distortion also do not need to be considered because the top flanges are continuously braced at the strength limit state.

7.2 Web Splice Design

For box sections specified in AASHTO LRFD BDS Article 6.13.6.1.3b, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state since the web splice is to be designed as a minimum for the full factored shear resistance of the web.

Furthermore, for box sections specified in AASHTO LRFD BDS Article 6.13.6.1.3b, the web shear for checking slip is to be taken as the sum of the factored flexural and St. Venant torsional shears in the web subjected to additive shears. For box girders with inclined webs, like tub girders, the factored shear is to be taken as the component of the factored vertical shear in the plane of the web.

Figure 10 Photograph of a Box Girder Field Splice

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