


U.S. Department of Transportation
Federal Highway Administration

Steel Bridge Design Handbook

Splice Design

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16. 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 module 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. Lastly, a thorough design example of a bolted field splice for a steel I-girder is provided, illustrating calculations for flange and web stress, splice plate design, and bolt design. Strength, Service, and Fatigue limit states are considered, and design checks are provide for tension, compression, and shear resistance of splice plates, fracture and bearing resistance of splice plates, and strength and slip resistances of the bolted connections.

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FOREWORD

It took an act of Congress to provide funding for the development of this comprehensive handbook in steel bridge design. This 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 is based on the Fifth Edition, including the 2010 Interims, of the AASHTO LRFD Bridge Design Specifications. The hard work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR Engineering and their sub-consultants in producing this handbook is gratefully acknowledged. This is the culmination of seven years of effort beginning in 2005.

The new *Steel Bridge Design Handbook* is divided into several topics and design examples as follows:

- Bridge Steels and Their Properties
- Bridge Fabrication
- Steel Bridge Shop Drawings
- Structural Behavior
- Selecting the Right Bridge Type
- Stringer Bridges
- Loads and Combinations
- Structural Analysis
- Redundancy
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- Load Rating
- Corrosion Protection of Bridges
- Design Example: Three-span Continuous Straight I-Girder Bridge
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- Design Example: Two-span Continuous Straight Wide-Flange Beam Bridge
- Design Example: Three-span Continuous Straight Tub-Girder Bridge
- Design Example: Three-span Continuous Curved I-Girder Beam Bridge
- Design Example: Three-span Continuous Curved Tub-Girder Bridge

These topics and design examples are published separately for ease of use, and available for free download at the NSBA and FHWA websites: http://www.steelbridges.org, and http://www.fhwa.dot.gov/bridge, respectively.

The contributions and constructive review comments during the preparation of the handbook from many engineering processionals are very much appreciated. The readers are encouraged to submit ideas and suggestions for enhancements of future edition of the handbook to Myint Lwin at the following address: Federal Highway Administration, 1200 New Jersey Avenue, S.E., Washington, DC 20590.

Myint Lwin

M. Myint Lwin, Director
Office of Bridge Technology

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 module 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, 5th Edition, (2010)* (referred to herein as AASHTO LRFD (5th Edition, 2010)), (1). Throughout this module, 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 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. Typically, groove welds are utilized to join the pieces together. It should be noted that once the individual components (web or flange) are complete, they are typically welded together with fillet welds (flange-to-web welds).

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 NSBA 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. For additional information, see Reference (2):

- Per AASHTO LRFD Article 6.6.1.2.4, “Transversely loaded partial penetration groove welds shall not be used,” except for certain orthotropic steel deck applications, therefore only complete joint penetration (CJP) welds are employed, and designed in accordance with Article 6.13.3.

- 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 3. 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 module titled *Stringer Bridges* 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.

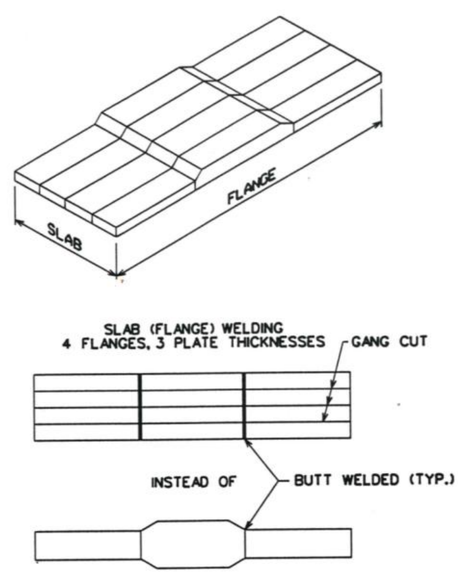

Figure 3 Sketch Illustrating Slabbing and Stripping of Flanges

- Flange thicknesses should be changed using the following increments:

- t ≤ 7/8" – use 1/16" increments

- 7/8" < 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, and the following guidelines can be used to provide economical transitions:

- For spans up to approximately 150', one top flange plate transition between an interior pier and the adjacent field splice is generally economical.

- For spans over 150', two top flange plate transitions between an interior pier and the adjacent field splice may be economical.

- Except for very long spans, one bottom flange plate transition between an interior pier and the adjacent field splice is usually 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 144" (assuming 6" for camber cuts and 150" maximum commonly 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 144" may also require a longitudinal web splice (typically welded, but sometimes bolted).

The NSBA provides recommended details for welded (shop) splices, as shown in Figure 4 on the following page. In addition, the AASHTO LRFD (5th Edition, 2010) provides recommended details as shown in Figure 5.

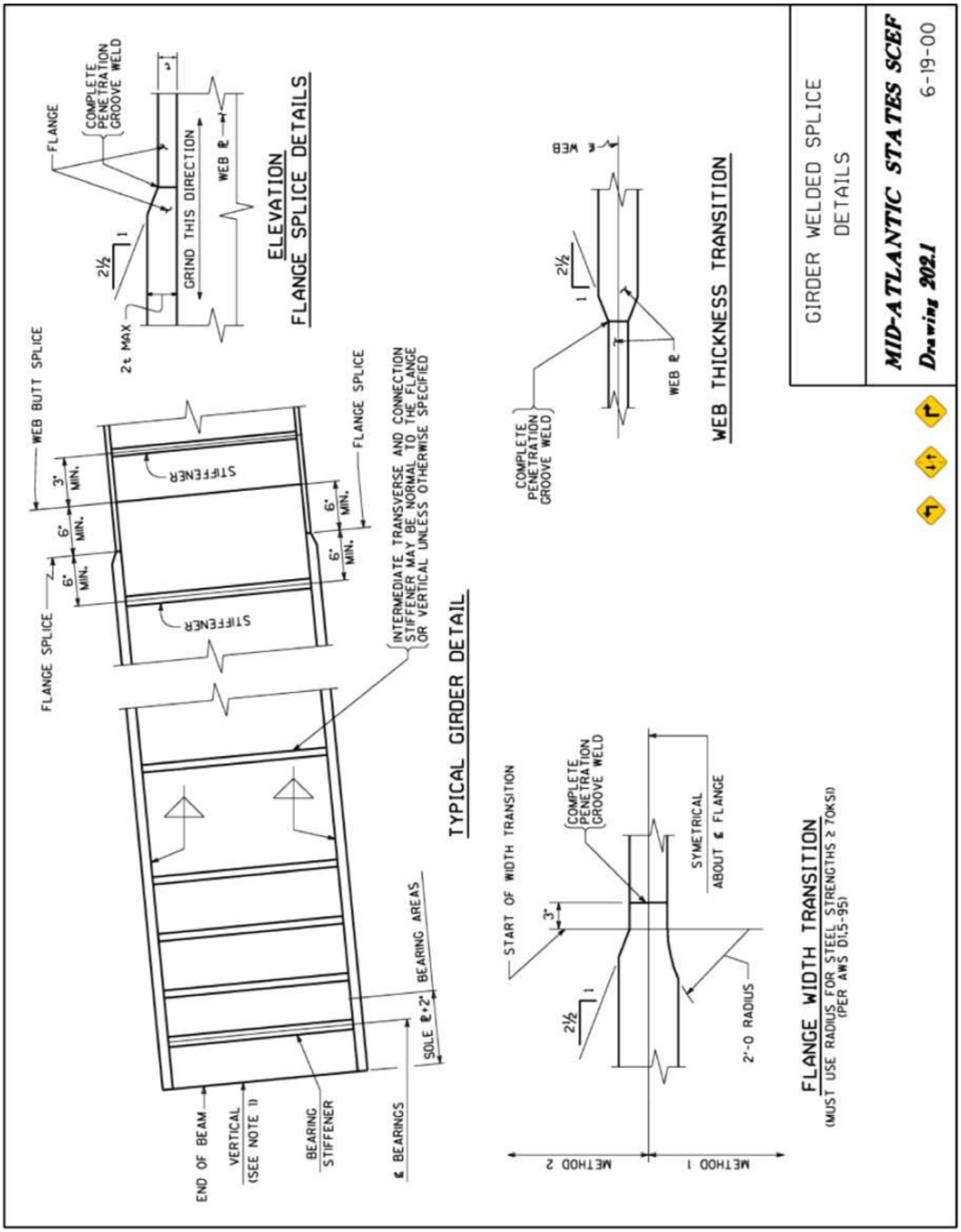

Figure 4 NSBA Recommended Welded Splice Details (3)

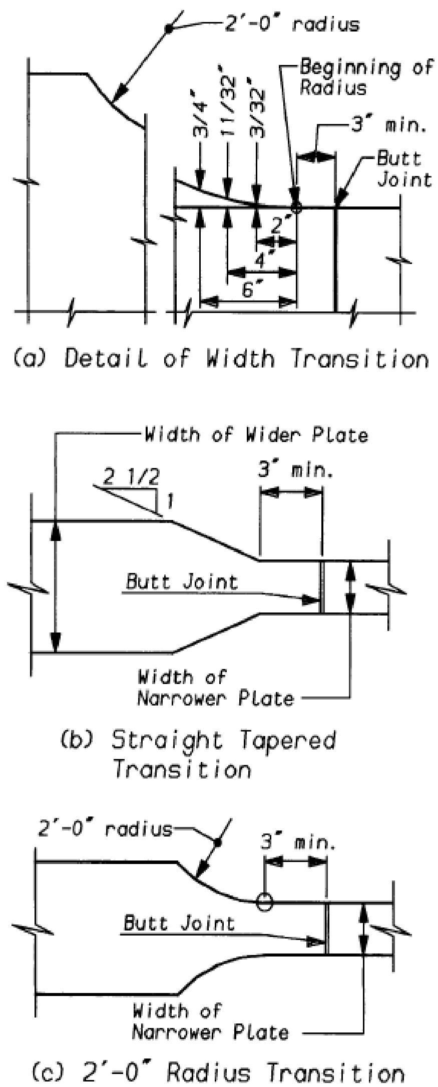

Figure 6.13.6.2-1—Splice Details

Figure 5 AASHTO LRFD (5th Edition, 2010) Recommended Welded Splice Details

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.

The majority of bridges that exceed 120 to 140 feet in length will require at least one field splice somewhere in the girder. Numerous design constraints must be satisfied during the design of the splices. In addition, the intent of the AASHTO LRFD (5th Edition, 2010) must be kept in mind during the design process.

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. To account for the uncertainty in the behavior of bolted girder splices, the underlying design philosophy behind splice design ensures that the capacity of the splice is well in excess of the maximum force in the web or flanges.

The behavior of the girder must be kept in mind when designing the splice. It must be remembered that the girder properties are different depending on when the girder is examined. The basic girder section starts out as a non-composite section prior to hardening of the deck slab, and then becomes a composite section. In the positive moment regions, the composite section properties are different for long-term composite dead loads and short-term composite live loads. Thus, the splice design must be based on the stresses in the flanges and the web and not on the moments for the non-composite and composite girder sections.

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 module will focus on the factors which influence and the principles of 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 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 often 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). By doing so, the design forces for the splice are minimized. Web and flange splices in areas of stress reversal (which is likely to be the case near the points of dead load contraflexure) should be investigated for both positive and negative flexure (Article 6.13.6.1.4a). Sometimes, if the span lengths are large enough, field splices may be required at additional locations. The design of the splices away from the point of dead load contraflexure will typically be heavier due to the higher forces in the girder.

- 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 often larger than 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 module.

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.

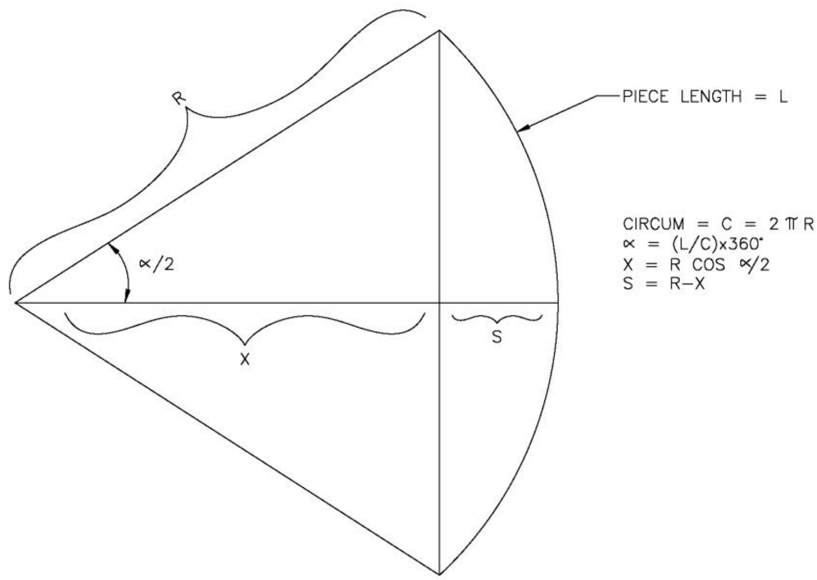

Figure 7 Sketch of a Curved Girder Sweep

A curved girder is also subject to lateral flange bending, the force effects of which should be included in the design of the flange splices, where the flange is partially braced (Article 6.13.6.1.4c). This applies to the top flange prior to curing of the deck slab and to the bottom flange. To account for the effects of lateral flange bending, the bolts should be designed for the combined effects of moment and shear from both primary (vertical) and lateral effects using the traditional elastic vector method. Generally, this effect is relatively minor as compared to the primary bending effects, but must be accounted for. An example of field splice design for a curved girder bridge is provided in the 2003 AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (4).

4.4 Girder Properties

- Steel Grade – The field splice design provisions of the AASHTO LRFD (5th Edition, 2010) 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 splice design provisions in the AASHTO LRFD (5th Edition, 2010) address the use of such hybrid sections through the application of a Hybrid Factor, R_h, which is defined in Article 6.10.1.10.1, and which is included in the field splice design equations.

- Homogeneous Plate-Girders – In cases where the yield strength of the flanges and web are the same, the Hybrid Factor in the field splice design equations is set equal to 1.0.

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

- Crossframe Connection Plates/Transverse Stiffeners – Field splices should be located far enough away from crossframe 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 crossframe 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 crossframe 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 – see detail below

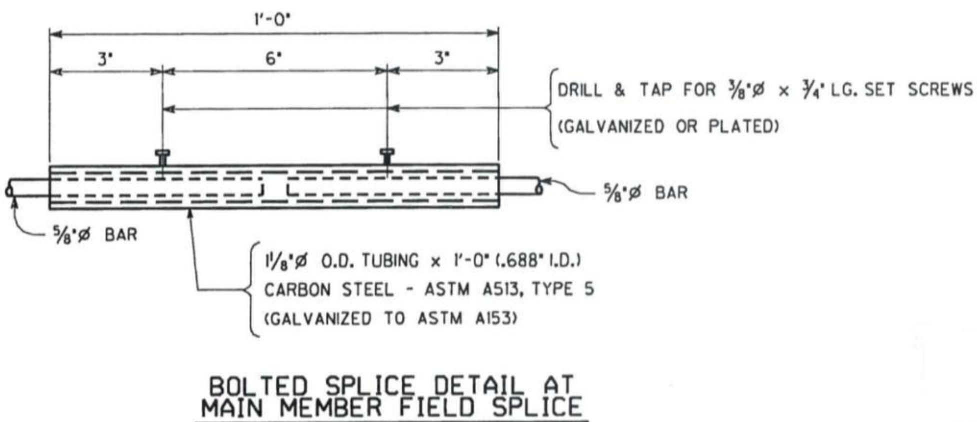

Figure 8 Sketch of a Handrail Detail at Field Splice

4.6 Other Factors

- Shipping Requirements – Most states require hauling permits for loads over a certain length. Beyond this length, special permits and/or restricted routes (i.e. multi-lane highways) may be required. Even longer lengths may be completely prohibited. It is important that the designer is aware of the hauling restrictions and requirements within the project state and from viable fabrication shops to the project site. These restrictions and requirements can have a significant 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. Some agencies 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 be winding) 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.

Erection Requirements – 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 Current Splice Design Provisions

The splice design provisions in the AASHTO LRFD (5th Edition, 2010) specification were developed in an attempt to eliminate confusion regarding the application of the previous provisions, to extend the previous provisions to cover the design of splices in all regions of composite girders, and to provide for a more consistent application of the provisions at all limit states.

These current provisions are discussed in this and the following sections and implemented in the example field splice design which follows.

According to Article 6.13.1, splices for primary members shall 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.

Note, as stated in Article C6.13.1, where the girder section changes at a splice, which is frequently the case, the "smaller" section is to be used for these requirements.

These statements can be expressed in an equation of generic form:

$$\tfrac{1}{2}(Q + R_r) \geq 0.75R_r \qquad (1)$$

where:

Q = The factored force effect

R_r = The factored resistance

This requirement has existed in a similar form in the *AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002* (referred to herein as Standard Specification) (5) since the days of riveted girders. Thus, in essence, it was originally developed to handle the case of symmetrical non-composite, non-compact girders.

In the case of flexure, the splice is designed to provide this minimum flexural resistance to ensure that a stiffness consistent with the flexural stiffness of the member at the splice is provided and to allow for possible shifts in the girder moment at the splice. The minimum flexural resistance is used to check the flexural resistance of the splice plates and the bearing resistance at bolt holes, and to check the shear resistance of high-strength bolts assuming the bolts have slipped and gone into bearing at the strength limit state.

Following similar reasoning, splices should also be designed to provide a minimum flexural resistance at the permanent deflection service limit state (i.e., when checking for slip of the bolts and localized yielding of the splice plates). A minimum flexural resistance at the permanent deflection service limit state (a lower value than the strength limit state) is defined in order to be consistent with the philosophy used to check the splice at the strength limit state and to again allow for possible shifts in the girder moment at the splice.

The proposed minimum flexural resistances that must be provided by the splice at the strength limit state and at the permanent deflection service limit state are reviewed below.

5.2 "Smaller" Girder Section

Often, where the girder section changes at a splice, and the "smaller" section must be used, it is obvious by inspection which side of the splice is smaller because all of the section components are of lesser dimension (width and/or thickness) and have the same yield stress as the "larger" section. When a hybrid girder with smaller flanges is connected to a girder with a larger homogeneous section (all plates of the same yield strength), it is not obvious which section is the "smaller" section. When this is the case (as illustrated in the numerical splice design example which follows) it is appropriate to determine the "smaller" section as the side of the splice which has lower flange forces.

5.3 Controlling Flange

In the design of flange splices, determination of the controlling flange is critical. The definition of controlling flange is provided in Article C6.13.6.1.4c, but is included here for completeness.

> "The controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its midthickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the noncontrolling flange."

Article C6.13.6.1.4c also indicates that in areas of stress reversal (which is likely if the splice is located near the point of dead load contraflexure), the splice must be checked independently for both positive and negative flexure.

5.4 Effective Flange Area

In the design of flange splices, the required minimum design resistance (the design force) is determined by finding the product of the minimum flexural design stress (as described in the following sections) and the smaller effective flange area, A_e, on either side of the splice. For compression flanges, A_e shall be taken as the gross area of the flange. For tension flanges, A_e shall be taken as defined in Equation (6.13.6.1.4c-2).

It is possible that the smaller girder section at the splice (as designated by having the smaller moment of inertia) may have a larger effective flange area for the controlling flange than the section on the opposite side of the splice. In such case, the effective flange area on the opposite side of the splice should be used.

5.5 Minimum Flexural Resistance: Strength Limit State

Typically, splices have been designed by treating the flanges and the web of the girder as individual components and then proportioning the total moment in the girder at the splice to each component. The determination of the proportion of the total moment carried by the web is not necessarily straightforward for an unsymmetrical composite girder. Many different approaches have been used, which have not always led to consistent results.

For a composite girder, dead- and live-load moments due to the factored loads are applied to different sections and should not be directly summed when at elastic stress levels (up to and including F_{yf}). In addition, the factored flexural resistance of a composite section is different in positive and negative flexure. In areas of stress reversal, it is not always clear which flexural resistance should be used in Equation (1). For these reasons, it becomes more convenient to express the minimum flexural resistance of each member component (flanges and web) that must be provided by the splice in terms of stress rather than moment, which is the approach taken in the current provisions.

5.5.1 Flange Splices (Article 6.13.6.1.4c)

Once the smaller girder section at the point of splice and the controlling and non-controlling flanges have been determined, the flange splice plates and their connections can be proportioned to provide a flexural resistance to satisfy the strength limit state. The required flexural resistance is obtained by providing splice plates and connections that produce a factored resistance greater than the design force described in the following paragraphs.

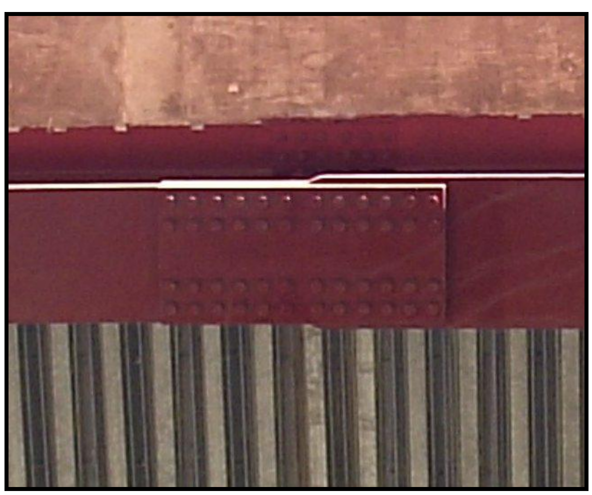

Figure 9 Photograph of a Typical Flange Splice

For the controlling flange, the design force is calculated as the product of the smaller effective flange area, A_e, on either side of the splice and the minimum flexural design stress for the controlling flange, F_{cf}, which follows the form of Equation (1) as follows:

$$F_{cf} = \frac{1}{2}\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}\right) \geq 0.75\alpha\phi_f F_{yf} \qquad (2)$$

where:

f_{cf} = maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at the point of splice (ksi)

R_h = hybrid factor

α = 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}

F_n = nominal flexural resistance of the flange (ksi)

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

F_{yf} = specified minimum yield strength of the flange (ksi)

For the non-controlling flange, the required design force is calculated as the product of the smaller effective flange area, A_e, on either side of the splice and the minimum flexural design stress for the non-controlling flange, F_{ncf}, as follows:

$$F_{ncf} = R_{cf}\left(\left|\frac{f_{ncf}}{R_h}\right|\right) \geq 0.75\alpha\phi_y F_{yf} \qquad (3)$$

where:

R_{cf} = the absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange

f_{ncf} = flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf} (ksi)

Use of the R_{cf} factor results in the flexural stress in the noncontrolling flange being factored up in the same proportion as the flexural stress in the controlling flange producing a consistent design. However, as a minimum, the factored-up stress must be equal to or greater than $0.75\alpha\varphi_f F_{yf}$.

Once the appropriate design forces have been determined, the splice plates and connections are designed as follows:

- For splice plates subject to tension, the design force shall not exceed the factored resistance in tension specified in Article 6.13.5.2

- For splice plates subject to compression, the design force shall not exceed the factored resistance, R_r, in compression taken as, $R_r = \varphi_c F_y A_s$

- At the strength limit state, the design force shall not exceed the factored resistance (bearing, shear and tensile), R_r or T_r of the bolted connections of the flange splice plates (Article 6.13.2.2). Note that the bolted connections must be designed as slip-critical, proportioned to prevent slip under Load Combination SERVICE II, as discussed later.

5.5.2 Web Splices (Article 6.13.6.1.4b)

Web splices and their connections are to be designed for the portion of the flexural moment to be resisted by the web and for the moment due to the eccentricity of the shear at the point of splice. The eccentricity is defined as the distance from the centerline of the splice to the centroid of the connection on the side of the joint under consideration.

Figure 10 Photograph of a Girder Web Splice

The web moment is assumed to be applied at the mid-depth of the web. For unsymmetrical sections, this means that a horizontal force resultant must also be applied at the mid-depth of the web in order to maintain equilibrium. This horizontal force resultant may be assumed distributed equally to all web bolts. The following equations are suggested to determine a design moment, M_{uw}, and a design horizontal force resultant, H_{uw}, to be applied at the mid-depth of the web for designing the web splice plates and their connections at the strength limit state:

$$M_{uw} = \frac{t_w D^2}{12} \left| R_h F_{cf} - R_{cf} f_{ncf} \right| \qquad (4)$$

$$H_{uw} = \frac{t_w D}{2} \left| R_h F_{cf} + R_{cf} f_{ncf} \right| \qquad (5)$$

where:

t_w = web thickness (in)

D = web depth (in)

R_h, F_{cf}, R_{cf} and f_{ncf} are as previously defined.

For sections where the neutral axis is located at the mid-depth of the web, H_{uw} will equal zero. For all other sections, M_{uw} and H_{uw} applied together will yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web.

5.6 Minimum Flexural Resistance: Permanent Deflection Service Limit State

The proposed minimum flexural resistance of the flanges and web that must be provided by the flange and web splices to satisfy the permanent deflection service limit state is given below. The minimum flexural resistance at the permanent deflection service limit state is used to check for slip of high-strength bolts and to check for localized yielding of the splice plates.

5.6.1 Flange Splices

In the Load Factor Design method given in the Standard Specifications (5), the ratio of the maximum design loads (used to check strength) to the Overload (used to control permanent deformations) is 1.3. In the AASHTO LRFD (5th Edition, 2010) (1), the ratio of the live load used in the STRENGTH I load combination to the live load used in the SERVICE II load combination is approximately equal to 1.3. Therefore, the proposed minimum flexural resistance of the flange at the splice, F_{fs}, that must be provided by the flange splice plates and their connections to satisfy the permanent deflection service limit state (i.e. to prevent slip of the bolts, etc.) is taken as the minimum flexural resistance of the flange at the splice at the strength limit state, F_{cf} or F_{ncf}, divided by 1.3:

$$F_{fs} = (F_{cf} \text{ or } F_{ncf})/1.3 \approx 0.80(F_{cf} \text{ or } F_{ncf}) \qquad (6)$$

The resistance factor is not applied since this is a serviceability check.

5.6.2 Web Splices

In order to determine a design moment and a design horizontal force resultant to be applied at the mid-depth of the web for designing the web splice plates and their connections at the permanent deflection service limit state, Equations (4) and (5) can be utilized with the following substitutions. This design moment and horizontal force is then utilized to check the web bolts for slip:

- Replace F_{cf} with the maximum flexural stress, f_s, due to load combination SERVICE II at the midthickness of the flange under consideration for the smaller section at the point of splice

- Replace f_{ncf} with the flexural stress, f_{os}, due to load combination SERVICE II at the midthickness of the other flange at the point of splice concurrent with f_s in the flange under consideration

- Set the factors R_h and R_{cf} equal to 1.0

This results in the following equations:

$$M = \frac{t_w D^2}{12}\left|f_s - f_{os}\right| \tag{7}$$

$$H = \frac{t_w D}{2}\left|f_s + f_{os}\right| \tag{8}$$

5.7 Shear

5.7.1 Strength Limit State

Since the maximum shear is not actually concurrent with the maximum moment at the splice and the shear at the splice is unlikely to shift to the degree that the moment at the splice may shift, it is felt that the splice need not be designed to resist the larger of 75 percent of the factored shear resistance of the member at the splice or the average of the applied shear and the factored shear resistance. Instead, the proposed minimum shear resistance of the web at the splice, V_{uw}, that must be provided by the web splice plates and their connections to satisfy the strength limit state is taken as follows:

If $V_u < 0.50\phi_v V_n$, then:

$$V_{uw} = 1.50V_u \tag{9}$$

If $V_u \geq 0.50\phi_v V_n$, then:

$$V_{uw} = 1/2\ (V_u+\phi_v V_n) \tag{10}$$

where:

V_u = sum of the maximum calculated shears at the splice due to the factored loads at the strength limit state

V_n = nominal shear resistance of the web of the splice

ϕ_v = resistance factor for shear (Article 6.5.4.2)

5.7.2 Permanent Deflection Service Limit State

The proposed minimum shear resistance of the web at the splice, V_{ws}, that must be provided by the web splice plates and their connections to satisfy the permanent deflection service limit state is taken as:

$$V_{ws} = V_{wr}/1.3 \approx 0.80V_{wr} \qquad (11)$$

5.8 Detailing Considerations

The following detailing requirements are stipulated in the AASHTO LRFD (5th Edition, 2010):

- In both web and flange splices, there shall not be less than two rows of bolts on each side of the joint (Article 6.13.6.1.4a).

- Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices (Article 6.13.6.1.4a).

- Web splice plates are to be symmetrical on each side of the web and are to extend as near as practical to the full depth of the web between flanges without impinging on bolt assembly clearances. The required bolt assembly clearances are given in the AISC Steel Construction Manual (Tables 7-16 and 7-17) (6).

- The minimum mandatory edge distances used for bolted connections are provided in the table below (from Article 6.13.2.6.6)

- The minimum spacing between centers of bolts in standard holes shall be no less than three times the diameter of the bolt (Article 6.13.2.6.1)

Table 1 AASHTO LRFD Table 6.13.2.6.6-1 Specifying Minimum Edge Distances

Bolt Diameter	Sheared Edges	Rolled Edges of Plates or Shapes, or Gas Cut Edges
in.	in.	in.
5/8	1-1/8	7/8
3/4	1-1/4	1
7/8	1-1/2	1-1/8
1	1-3/4	1-1/4
1-1/8	2	1-1/2
1-1/4	2-1/4	1-5/8
1-3/8	2-3/8	1-3/4

- The maximum spacing between bolts in a single line adjacent to a free edge of an outside plate or shape shall satisfy (Article 6.13.2.6.2):

$$s \leq (4.0 + 4.0t) \leq 7.0 \text{ inches}$$

- If there is a second line of fasteners uniformly staggered with those in the line adjacent to the free edge, at a gage less than 1.5 + 4.0t, the staggered spacing, s, in two such lines, considered together, shall satisfy (Article 6.13.2.6.2):

$$s \leq 4.0 + 4.0t - [(3.0g) / 4.0] \leq 7.0 \text{ inches}$$

- For bolted web splices with thickness differences of 1/16" or less, no filler plates are required (Article 6.13.6.1.5).

- Fillers 1/4" or more in thickness shall consist of not more than two plates (Article 6.13.6.1.5)

- The specified minimum yield strength of fillers 1/4" or greater in thickness should not be less than the larger of 70% of the specified minimum yield strength of the connected plate and 36.0 ksi (Article 6.13.6.1.5)

- For fillers less than 1/4" in thickness, no such provisions are stipulated.

- For applications involving the use of weathering steels, a weathering grade product should be specified for the filler plate material.

- Hole sizes for design should be assumed to be 1/8" larger than the nominal diameter of the bolt (Article 6.8.3).

In addition, the following detailing recommendations should be considered:

- The orientation of the bolt heads for field splices should be designated as shown in Figure 11.

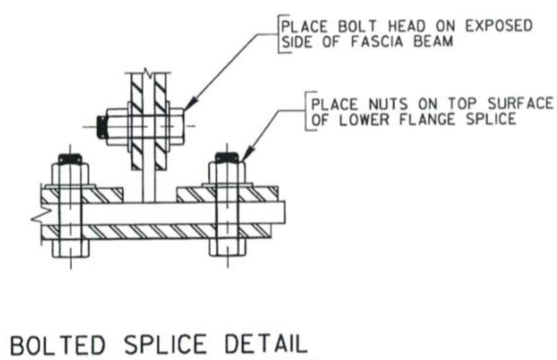

Figure 11 Sketch of a Bottom Flange Bolt Head Orientation

- In order to provide a tolerance for punching, drilling and reaming of bolt holes, additional edge distance (beyond that required by AASHTO LRFD (5th Edition, 2010)) should be used as shown in Figure 12.

- Minimum bolt spacings in excess of 3 times the diameter are preferable:
  - For 7/8-inch fasteners, 3 inches
  - For 1-inch fasteners, 3-1/2 inches
  - For 1-1/8-inch fasteners, 4 inches

- Using the preceding recommended criteria, a minimum flange width of approximately 15" is required to accommodate 4 lines of 7/8" diameter high-strength bolts spaced across the width of the flange (not staggered).

- A non-varying (repeating) bolt pattern should be used for economy during fabrication.

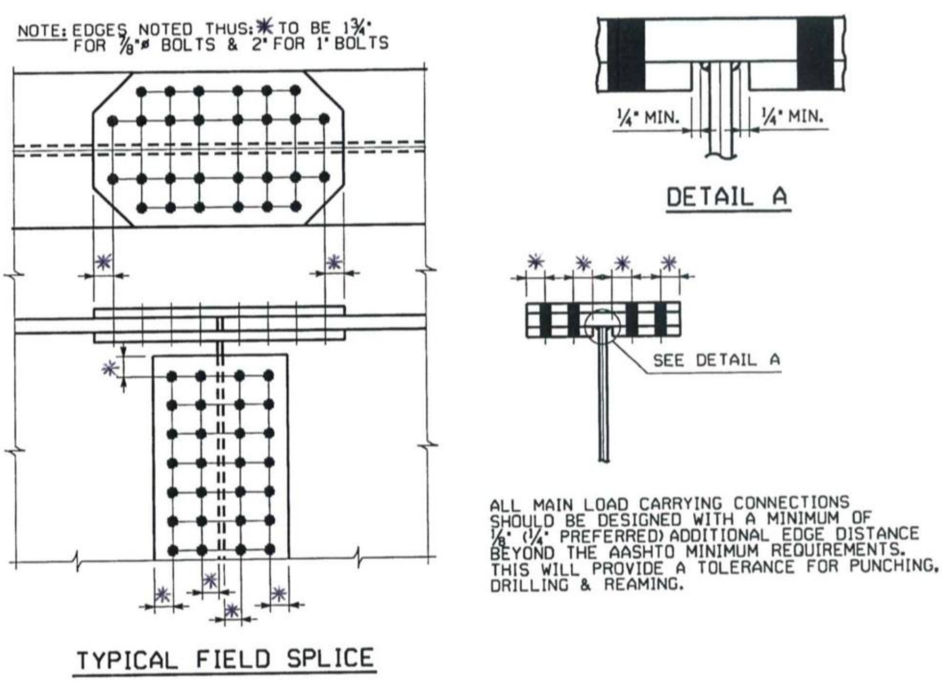

Figure 12 Sketch of the Preferable Bolt Edge Distance

6.0 HOMOGENEOUS PLATE-GIRDER EXAMPLE INTRODUCTION

The following discussion and example calculations have been prepared for the design of a field splice (located in the end spans) for the exterior girder of the three-span continuous, straight, composite I-girder (plate-girder) structure discussed in detail in EXAMPLE 1 of the Steel Bridge Design Handbook. The span lengths for this bridge are 140'-0" - 175'-0" - 140'-0", and the field splice of interest is located 100'-0" from the end support (see Figure 13 for the location of the splice which occurs near the point of dead-load contraflexure). The girder elevation for this example is provided in Figure 14.

Additional Design Criteria:

- Girder Webs, Positive Moment Section Flanges and Splice Plate Yield Strength: $F_y = 50$ ksi
- Negative Moment Section Flanges: $F_y = 70$ ksi
- Therefore, Flange Yield Strength: $F_{yf} = 50$ ksi or $F_{yf} = 70$ ksi
- Girder and Splice Plate Tensile Strength: $F_u = 70$ ksi or $F_u = 85$ ksi
- Girder Spacing: 12'-0"
- Deck Slab Overhang Width: 3'-6"
- Deck Slab Thickness: 9-1/2" (includes 1/2" integral wearing surface)
- Deck Slab Haunch: 3-1/2" (bottom of deck to top of web)
- Controlling Effective Slab Width: 100.0"
- Area of Deck Slab Long. Reinf. and Location: 9.30 in^2 at 4.63" above bottom of slab
- Deck Slab Strength and Modular Ratio: $f'_c = 4.0$ ksi and $n = 8$
- Average Daily Truck Traffic (One Direction): $ADTT_{SL} = 1600$ trucks/day
- $\eta_i = \eta_D * \eta_R * \eta_I = 1.0$

The splice will be designed as a slip-critical connection to prevent slip at the permanent deflection service limit state and during erection of the steel and casting of the concrete deck slab (constructability check). The bolted connections will also be proportioned to provide bearing and shear resistance under the governing strength limit state load combination (Article 6.13.2.1.1).

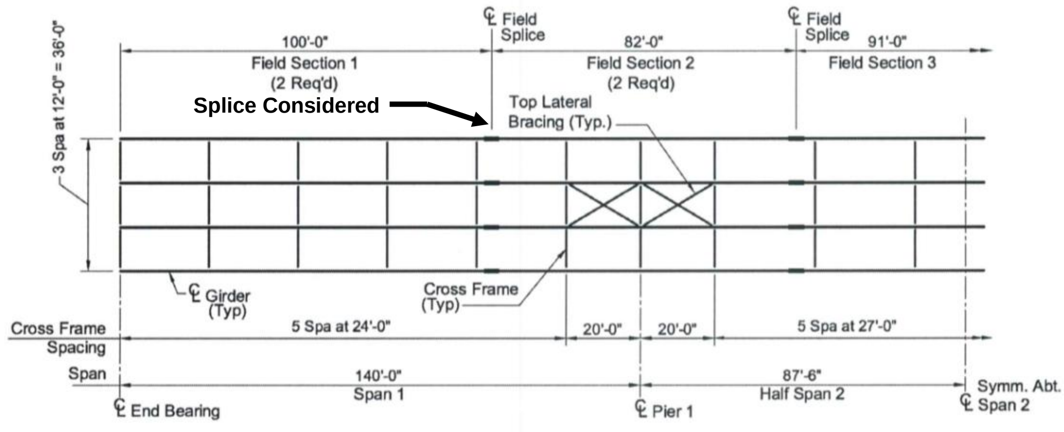

Figure 13 Sketch of the Superstructure Framing Plan

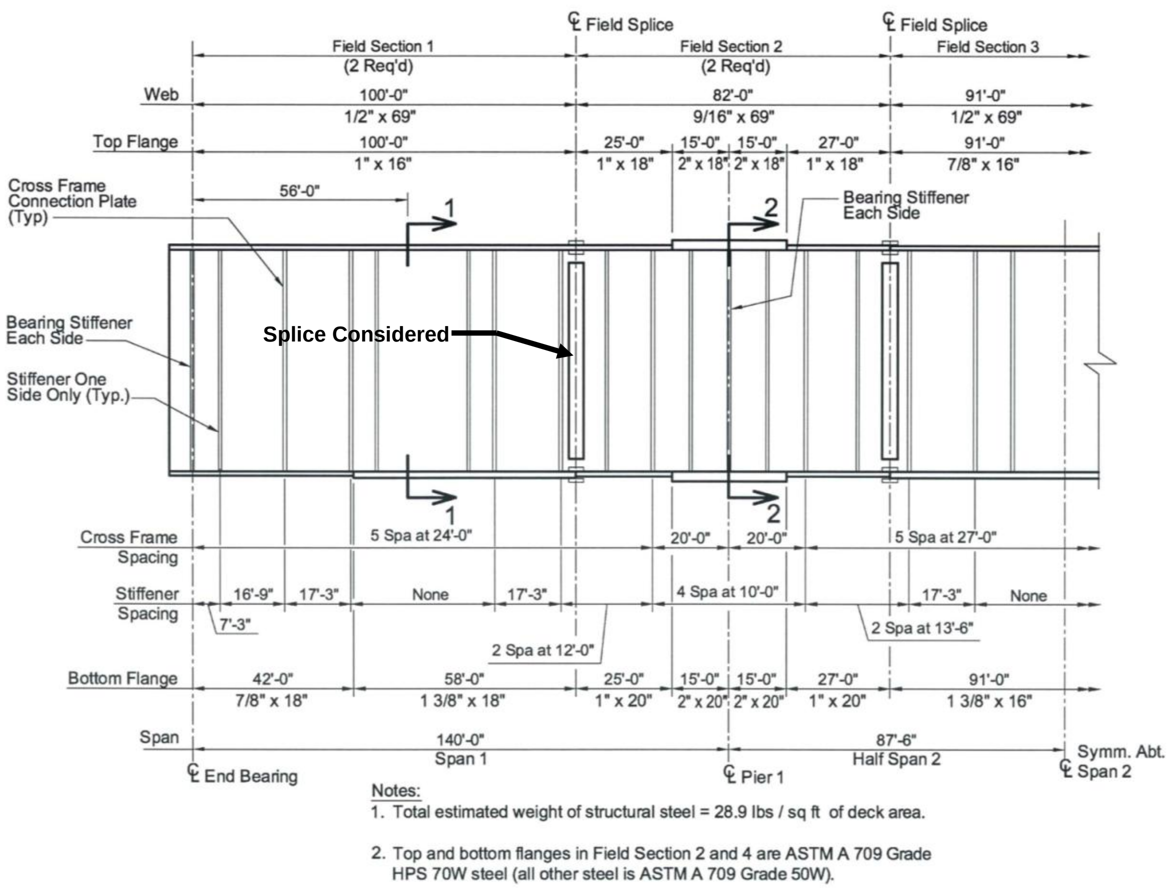

Figure 14 Sketch of the Girder Elevation

Table 2 Summary of Girder Plate Dimensions

	Left Section	Right Section
Web Thickness, t_w (in.)	1/2	9/16
Web Depth, D (in.)	69	69
Top Flange Width, b_{tf} (in.)	16	18
Top Flange Thickness, t_{tf} (in.)	1	1
Bottom Flange Width, b_{bf} (in.)	18	20
Bottom Flange Thickness, t_{bf} (in.)	1-3/8	1

7.0 LOADS FOR SPLICE DESIGN

The moments and shears due to the unfactored loads at the splice in the end spans as calculated in EXAMPLE 1 of the Steel Bridge Design Handbook are:

M_{DC1}	= 74 kip-ft	V_{DC1}	= -85 kips
$M_{deck\ casting}$	= 1,279 kip-ft	$V_{deck\ casting}$	= -77 kips
M_{DC2}	= 26 kip-ft	V_{DC2}	= -12 kips
M_{DW}	= 25 kip-ft	V_{DW}	= -12 kips
M_{+LL+IM}	= 2,339 kip-ft	V_{+LL+IM}	= 18 kips
M_{-LL+IM}	= -1,832 kip-ft	V_{-LL+IM}	= -114 kips

8.0 CONTROLLING SPLICE DESIGN SECTION

Per AASHTO LRFD (5th Edition, 2010) Article 6.13.6.1.4c, the splice is designed for the smaller section at the splice location. Due to the hybrid girder section on the right side of the splice having smaller flange plates than the homogeneous girder section on the left side of the splice, it is not obvious which section will control. The design flange force must be calculated for the controlling flange on both sides of the splice, and the section with the minimum design flange force will be the controlling section. Per the commentary of AASHTO LRFD (5th Edition, 2010) Article 6.13.6.1.4c, for composite sections in positive flexure, the controlling flange is typically the bottom flange, and therefore the design flange force will be calculated for the bottom flange of both sections. Calculations and commentary showing that the bottom flange plate is the controlling flange are provided later in this module. To further simplify the calculation, the field splice is located at approximately the point of dead-load contraflexure and the design loads are minimal. Therefore the minimum requirements of AASHTO LRFD (5th Edition, 2010) Equation 6.13.6.1.4c-3 will be used as the design stress.

Left side of splice:

$$F_{cf} = \frac{1}{2}\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}\right) \geq 0.75\alpha\phi_f F_{yf} \qquad \text{(Eq. 6.13.6.1.4c-3)}$$

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.00)(50) = 37.50$ ksi

Therefore, $F_{cf} = 37.50$ ksi

Right side of splice:

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.00)(70) = 52.50$ ksi

Therefore, $F_{cf} = 52.50$ ksi

The design flange stress is then multiplied by the effective flange area for tension or compression to determine the design flange force. In order to determine the effective flange area, $A_{effective}$, to be used, both the gross area, A_{gross}, and net area, A_{net}, of the flange must first be calculated. It should be noted that the value of $A_{effective}$ used cannot be taken as larger than A_{gross}. The section considered to determine the net section of the flange (left side of splice) is shown in Figure 15:

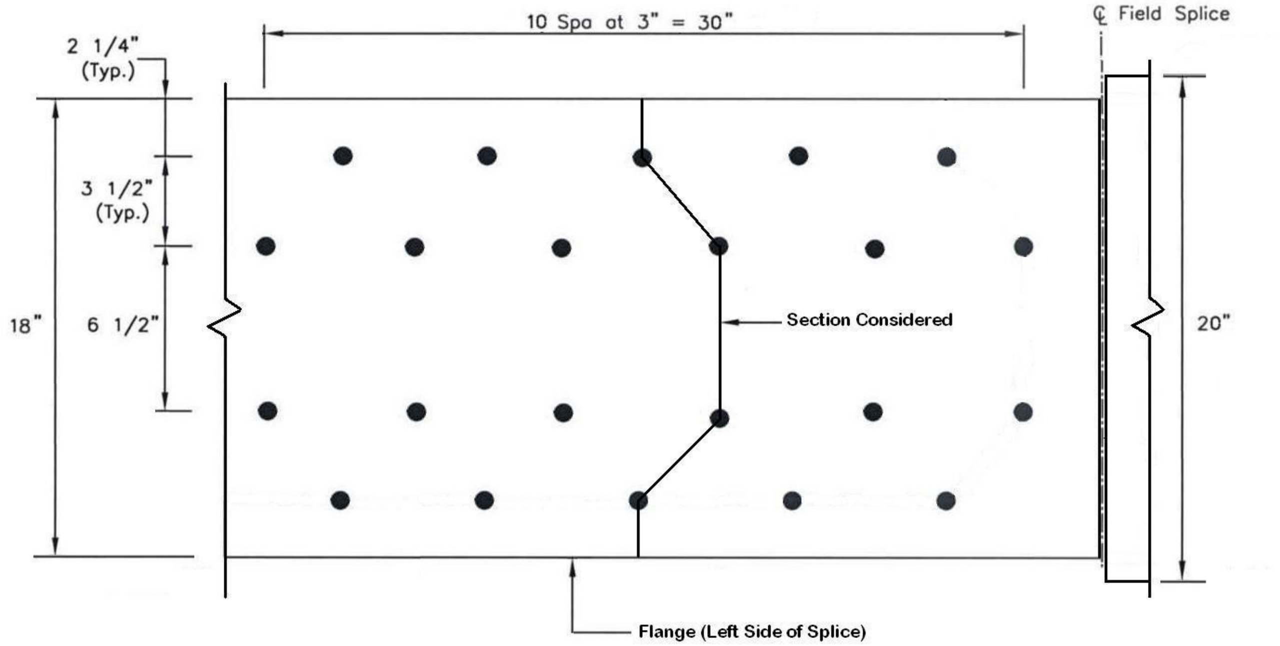

Figure 15 Sketch of the Girder Flange – Left Side of Splice

Let side of the splice:

$$A_{gross} = (1.375)(18) = 24.75 \text{ in.}^2$$

$$A_{net} = \left[18 - 4(1) + 2\left(\frac{3.0^2}{4.0(3.5)}\right)\right](1.375) = 21.02 \text{ in.}^2$$

$$A_{effective} = \left(\frac{0.80(70)}{0.95(50)}\right)(21.02) = 24.78 \text{ in.}^2$$

Since $A_{effective}$ is greater than A_{gross}, therefore use A_{gross}.

$P_{TEN} = (37.5)(24.75) = 928$ kips

$P_{COMP} = (37.5)(24.75) = 928$ kips (controls)

Right side of the splice:

$$A_{gross} = (1.000)(20) = 20.00 \text{ in.}^2$$

$$A_{net} = \left[20 - 4(1) + 2\left(\frac{3.0^2}{4.0(3.5)}\right)\right](1.000) = 17.29 \text{ in.}^2$$

$$A_{\text{effective}} = \left(\frac{0.80(85)}{0.95(70)}\right)(17.29) = 17.68\ \text{in.}^2$$

P_{TEN} = (52.5)(17.68) = 928 kips

P_{COMP} = (52.5)(20.00) = 1050 kips (controls)

9.0 SECTION PROPERTIES FOR SPLICE DESIGN

The applicable gross section properties for the section on the left side of the splice are summarized below. While the gross section properties are appropriate to use for the SERVICE II and FATIGUE limit states, more refined section properties (which include a reduction in flange area to account for bolt holes in the tension flange) should be utilized for the different loading (positive and negative flexure) conditions for the STRENGTH I limit state. However, this calculation is iterative as the flange splices are proportioned (the number of bolt holes are determined). For this example, the gross section properties will be utilized for all limit states, as the differences are minimal and do not significantly affect the design for this example:

Steel section:

I_{NC} = 62,658 in.^{4}

$S_{NC\ TOP\ OF\ STEEL}$ = 1,581 in.^{3}

$S_{NC\ BOT\ OF\ STEEL}$ = 1,973 in.^{3}

N.A. at 4.13 inches below mid-depth of the web

Steel section plus longitudinal reinforcement:

I_{STEEL} = 80,367 in.^{4}

$S_{TOP\ OF\ STEEL}$ = 2,327 in.^{3}

$S_{BOT\ OF\ STEEL}$ = 2,182 in.^{3}

N.A. at 0.96 inches above mid-depth of the web

For sections subject to positive flexure, the composite section consists of the steel section and the transformed area of the effective width of the concrete deck. As described in Article 6.10.1.1.1b, for transient loads, a short-term modular ratio, $n = E / E_c$, should be used to transform the concrete deck area. For permanent loads, a long-term modular ratio of 3n should be used for the transformation. For this example, n = 8 and 3n = 24.

3n Composite section:

I_{LT} = 117,341 in.^{4}

$S_{LT\ TOP\ OF\ STEEL}$ = 4,863 in.^{3}

$S_{LT\ BOT\ OF\ STEEL}$ = 2,483 in.^{3}

N.A. at 11.37 inches above mid-depth of the web n Composite section:

I_{ST} = 161,518 in.^{4}

$S_{ST\ TOP\ OF\ STEEL}$ = 13,805 in.^{3}

$S_{ST\ BOT\ OF\ STEEL}$ = 2,706 in.^{3}

N.A. at 23.80 inches above mid-depth of the web

10.0 FLANGE SPLICE DESIGN

10.1 Calculation of Stresses

For the section on the left side of the splice in Figure 14, determine the average unfactored flexural stress on the gross section of the bottom and top flange.

10.1.1 Bottom-Flange Stresses

In the following, negative live-load moments are assumed to be applied to the section consisting of the steel girder plus the longitudinal reinforcement.

In this example, the DC_2 and DW moments are positive at the splice location. However, the DC_2 and DW moments are relatively small since the splice is located near the point of dead-load contraflexure. From inspection, it is apparent that the DC_2 and DW flexural compressive stresses in the concrete slab are overcome by the negative live-load flexural tensile stress due to the factored loads. When this is the case, the flexural stresses due to DC_2 and DW moments are combined with the live-load stress at the splice by assuming that these dead-load moments are applied to the appropriate section for the live-load moment under consideration. For this example, the flexural compressive stresses due to DC_2 and DW at the splice are computed assuming that these moments act on the 3n composite section for combination with the positive live-load flexural stress or the steel girder plus longitudinal reinforcement for combination with the negative live-load flexural stress.

The unfactored flexural stresses at the bottom of the bottom flange are computed for the various moments acting on the appropriate sections. These equations are in the general form of f = Moment/$S_{BOT\ OF\ STEEL}$, with Moment with units of kip-ft (converted to kip-in by a factor of 12) and S with units of cubic inches:

f_{DC1} = $74(12) \div 1973$ = 0.45 ksi (tension)

$f_{deck\ casting}$ = $1279(12) \div 1973$ = 7.78 ksi (tension)

f_{DC2} = $26(12) \div 2483$ = 0.13 ksi (tension)

or f_{DC2} = $26(12) \div 2182$ = 0.14 ksi (tension)

f_{DW} = $25(12) \div 2483$ = 0.12 ksi (tension)

or f_{DW} = $25(12) \div 2182$ = 0.14 ksi (tension)

$f_{+(LL+I)}$ = $2339(12) \div 2706$ = 10.37 ksi (tension)

$|f_{-(LL+I)}|$ = $|-1832(12) \div 2182|$ = 10.08 ksi (compression)

The unfactored flexural stresses at the bottom of the web are then computed. These equations are in the general form of f = (Moment x C)/I, with Moment in terms of kip-in, and C, the distance from the neutral axis to the bottom of the web:

f_{DC1}	=	74(12)(30.37) ÷ 62658	= 0.43 ksi (tension)
$f_{deck\ casting}$	=	1279(12)(30.37) ÷ 62658	= 7.44 ksi (tension)
f_{DC2}	=	26(12)(45.87) ÷ 117341	= 0.12 ksi (tension)
or f_{DC2}	=	26(12)(35.46) ÷ 80367	= 0.14 ksi (tension)
f_{DW}	=	25(12)(45.87) ÷ 117341	= 0.12 ksi (tension)
or f_{DW}	=	25(12)(35.46) ÷ 80367	= 0.13 ksi (tension)
$f_{+(LL+I)}$	=	2339(12)(58.30) ÷ 161518	= 10.13 ksi (tension)
$\|f_{-(LL+I)}\|$	=	|-1832(12)(35.46) ÷ 80367|	= 9.70 ksi (compression)

Therefore, the average flexural stresses in the bottom flange (to be used in the design of the splice) are taken as the average of the calculated stresses at the bottom and top of the bottom flange:

$(f_{DC1})_{avg}$	=	(0.45 + 0.43) ÷ 2	= 0.44 ksi (tension)
$(f_{deck\ casting})_{avg}$	=	(7.78 + 7.44) ÷ 2	= 7.61 ksi (tension)
$(f_{DC2})_{avg}$	=	(0.13 + 0.12) ÷ 2	= 0.13 ksi (tension)
or $(f_{DC2})_{avg}$	=	(0.14 + 0.14) ÷ 2	= 0.14 ksi (tension)
$(f_{DW})_{avg}$	=	(0.12 + 0.12) ÷ 2	= 0.12 ksi (tension)
or $(f_{DW})_{avg}$	=	(0.14 + 0.13) ÷ 2	= 0.14 ksi (tension)
$(f_{+(LL+I)})_{avg}$	=	(10.37 + 10.13) ÷ 2	= 10.25 ksi (tension)
$(f_{-(LL+I)})_{avg}$	=	(10.08 + 9.70) ÷ 2	= 9.89 ksi (compression)

For the controlling strength limit state load combination (STRENGTH I), determine the maximum average bending stress in the bottom flange at the splice due to the factored loads. The maximum permanent-load load factor γ_p (Table 3.4.1-2) is applied to permanent-load effects having the same sign as the live-load effects. For permanent-load effects with a sign opposite to the live-load effects, the minimum permanent-load load factor (Table 3.4.1-2) is applied, and the effect of the future wearing surface is conservatively ignored. In both cases, η is taken equal to

1.0. If it is known with certainty that the future wearing surface will eventually be applied, consideration should be given to including the effect of the wearing surface and applying the minimum permanent-load load factor of 0.65 specified in Table 3.4.1-2 to the wearing surface stress.

$$f_{fu} = 1.0\ [1.25(0.44+0.13) + 1.50(0.12) + 1.75(10.25)] = 18.83 \text{ ksi (governs)}$$

$$f_{fu} = 1.0\ [0.90(0.44+0.14) + 1.75(-9.89)] = -16.79 \text{ ksi}$$

10.1.2 Top-Flange Stresses

For the section on the left side of the splice in Figure 14, determine the average unfactored flexural stresses on the gross section of the top flange.

The unfactored flexural stresses at the top of the top flange are (refer to the preceding bottom-flange splice design for a discussion on what sections are used to compute the various stresses):

f_{DC1}	=	$74(12) \div 1581$	= 0.56 ksi (compression)
$f_{deck\ casting}$	=	$1279(12) \div 1581$	= 9.71 ksi (compression)
f_{DC2}	=	$26(12) \div 4863$	= 0.06 ksi (compression)
or f_{DC2}	=	$26(12) \div 2327$	= 0.13 ksi (compression)
f_{DW}	=	$25(12) \div 4863$	= 0.06 ksi (compression)
or f_{DW}	=	$25(12) \div 2327$	= 0.13 ksi (compression)
$f_{+(LL+I)}$	=	$2339(12) \div 13805$	= 2.03 ksi (compression)
$\lvert f_{-(LL+I)} \rvert$	=	$\lvert -1832(12) \div 2327 \rvert$	= 9.45 ksi (tension)

The unfactored flexural stresses at the top of the web are computed as:

$f_{DC1} = 74(12)(38.63) \div 62658 = 0.55$ ksi (compression)

$f_{deck\ casting} = 1279(12)(38.63) \div 62658 = 9.46$ ksi (compression)

$f_{DC2} = 26(12)(23.13) \div 117341 = 0.06$ ksi (compression)

or $f_{DC2} = 26(12)(33.54) \div 80367 = 0.13$ ksi (compression)

$f_{DW} = 25(12)(23.13) \div 117341 = 0.06$ ksi (compression)

or $f_{DW} = 25(12)(33.54) \div 80367 = 0.13$ ksi (compression)

$f_{+(LL+I)} = 2339(12)(10.70) \div 161518 = 1.86$ ksi (compression)

$|f_{-(LL+I)}| = |-1832(12)(33.54) \div 80367| = 9.17$ ksi (tension)

Therefore, the average flexural stresses in the top flange (used in the design of the splice) are:

$(f_{DC1})_{avg} = (0.56 + 0.55) \div 2 = 0.56$ ksi (compression)

$(f_{deck\ casting})_{avg} = (9.71 + 9.46) \div 2 = 9.59$ ksi (compression)

$(f_{DC2})_{avg} = (0.06 + 0.06) \div 2 = 0.06$ ksi (compression)

or $(f_{DC2})_{avg} = (0.13 + 0.13) \div 2 = 0.13$ ksi (compression)

$(f_{DW})_{avg} = (0.06 + 0.06) \div 2 = 0.06$ ksi (compression)

or $(f_{DW})_{avg} = (0.13 + 0.13) \div 2 = 0.13$ ksi (compression)

$(f_{+(LL+I)})_{avg} = (2.03 + 1.86) \div 2 = 1.95$ ksi (compression)

$(f_{-(LL+I)})_{avg} = (9.45 + 9.17) \div 2 = 9.31$ ksi (tension)

For the controlling strength limit state load combination (STRENGTH I), determine the maximum average bending stress in the top flange at the splice due to the factored loads. Apply the load factors in the same manner as illustrated for the bottom-flange splice:

$f_{fu} = 1.0\ [1.25(-0.56 + -0.06) + 1.50(-0.06) + 1.75(-1.95)] = -4.28$ ksi

$f_{fu} = 1.0\ [0.90(-0.56 + -0.13) + 1.75)(9.31)] = 15.67$ ksi (governs)

10.2 Determination of Controlling Flange

Based on the preceding stress calculations, the results shown in Table 3 have been obtained:

Table 3 Summary of STRENGTH I Flange Stresses

LIMIT STATE	LOCATION	STRESS (KSI)	
		DEAD + POS. LL	DEAD + NEG. LL
STRENGTH I	Top Flange	-4.28	15.67
	Bottom Flange	18.83	-16.79

A review of the stresses in itself is not necessarily adequate to determine the controlling flange for the different loading conditions, as the largest stress is not the determining factor. As stated previously, the controlling flange is the flange which has the maximum ratio of the elastic flexural stress at its midthickness due to the factored flexural loads to its factored flexural resistance. Generally, the factored flexural resistance (in terms of stress) of a compression flange will be equal to or lower by some degree than the factored flexural resistance (in terms of stress) of a tension flange.

If one flange is in tension and one flange is in compression and the absolute value of the compressive stress is greater than the tensile stress, then the compression flange will be the controlling flange. If the tensile stress is significantly larger than the absolute value of the compressive stress, then the tension flange will be the controlling flange. If the tensile stress is only slightly larger than the absolute value of the compressive stress, then the actual flexural resistances should be determined and the ratios of the elastic flexural stresses to the factored flexural resistances evaluated to determine the controlling flange.

If both flanges are in tension or in compression, then the flange with the maximum absolute value of the stress should be the controlling flange, although if the values are close then calculation of the factored flexural resistances and evaluation of the ratios of the elastic flexural stresses to the factored flexural resistances is advisable.

By inspection of the stresses in Table 3, it is apparent that the Bottom Flange is the controlling flange for both positive and negative live load for the STRENGTH I limit state.

10.3 Bottom-Flange Splice Design

For this example, the bottom flange has been determined to be the controlling flange.

10.3.1 Minimum Flexural Resistance: Strength Limit State

Determine the minimum flexural resistance of the flange, F_{cf}, that must be provided by the splice to satisfy the strength limit state. From the previous calculation, the largest flexural stress in the bottom flange at the splice due to the factored loads is tensile (positive flexure controls).

From Equation (3) given in the introduction to this splice-design example:

$$F_{cf} = \frac{1}{2}\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}\right) \geq 0.75\alpha\phi_f F_{yf}$$ (Eq. 6.13.6.1.4c-3)

$f_{cf} = f_{fu} = 18.83$ ksi

$\alpha = 1.0$ (Article C6.13.6.1.4c)

$R_h = 1.0$

$$\frac{1}{2}\left(\left|\frac{18.83}{1.0}\right| + 1.0(1.00)(50)\right) = 34.42 \text{ ksi}$$

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.00)(50) = 37.50$ ksi

Therefore, $F_{cf} = 37.50$ ksi (for both positive and negative bending)

The minimum design force, P_{cf}, for checking the resistance of the bottom-flange splice and for determining the number of bolts required to satisfy the strength limit state is therefore equal to F_{cf} times the effective net area for positive bending (tension in the bottom flange) and F_{cf} times the gross area for negative bending (compression in the bottom flange). The flange area used for the calculation of F_{cf} is that of the controlling flange, and P_{cf} of the smaller flange at the splice is as follows:

$P_{cf} = F_{cf}A_g = 37.50(24.75) = 928$ kips (negative bending)

$P_{cf} = F_{cf}A_{effective} = 37.50(24.75) = 928$ kips (positive bending)

The total combined area of the inside and outside splice plates should approximately equal or exceed the area of the smaller flange at the splice. Therefore, try a 5/8" x 18" outside splice plate with two - 3/4" x 8" inside splice plates. Include a 3/8" x 18" fill plate on the outside (Figure 16).

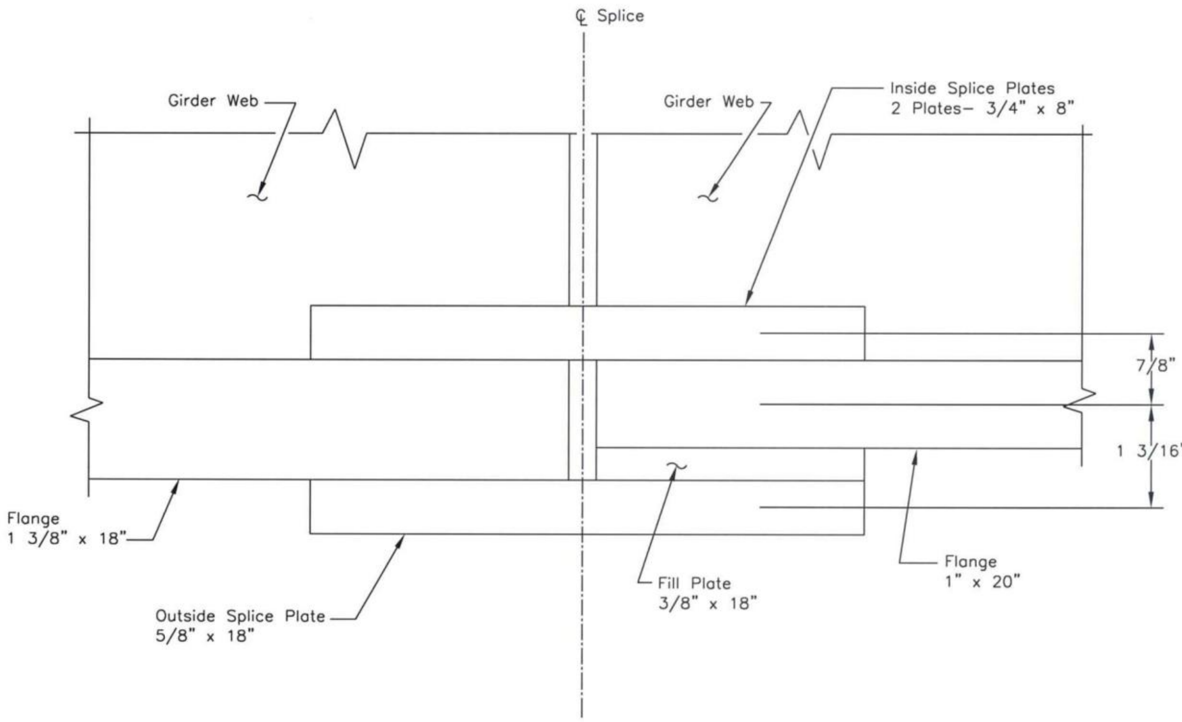

Figure 16 Sketch of the Bottom Flange Splice

The combined area of the inside splice plates is within 10 percent of the area of the outside splice plate. Therefore, design both the inside and outside splice plates for tension for 1/2 of P_{cf} equal to 464.0 kips. The total number of bolts required to provide adequate shear and slip resistance will then be determined by assuming two shear/slip planes ($N_s = 2$). If the areas of the inside and outside splice plates were to differ by more than 10 percent, P_{cf} would instead be proportioned to the inside and outside splice plates by the ratio of the area of the inside or outside splice plate(s) to the total area of the splice plates. In this case, the shear and slip resistance of the bolts would be checked at each individual shear/slip plane ($N_s = 1$) using the force in the adjacent splice plate(s).

10.3.2 Flange Bolts

10.3.2.1 Shear Resistance: Strength Limit State

Determine the number of bolts for the bottom flange splice plates that are required to satisfy the strength limit state assuming the bolts in the connection have slipped and gone into bearing. A minimum of two rows of bolts should be provided. First, compute the factored shear resistance of an ASTM A325 7/8-in. diameter high-strength bolt, assuming the threads are excluded from the shear planes. According to Article 6.13.2.2, the factored resistance R_r of a bolt in shear at the strength limit state shall be taken as:

$$R_r = \phi_s R_n \qquad \text{(Eq. 6.13.2.2-2)}$$

where:

R_n = the nominal resistance of the bolt in shear as specified in Article 6.13.2.7

ϕ_s = the resistance factor for an A325 bolt in shear (Article 6.5.4.2)

Since the length between extreme fasteners measured parallel to the line of action of the force is assumed to be less than 50 in. (measured between the extreme bolts on only one side of the splice per Article C6.13.2.7), the factored shear resistance in double shear (N_s = 2) is computed as follows:

$R_n = 0.48A_bF_{ub}N_s$ (Eq. 6.13.2.7-1)

F_{ub} = tensile strength of the bolt = 120 ksi (Article 6.4.3.1)

$$R_n = 0.48\left(\frac{(0.875)^2\pi}{4}\right)(120)(2) = 69.27\text{ kips}$$

$R_r = \phi_s R_n$

ϕ_s = 0.80 for bolts in shear (Article 6.5.4.2)

$R_r = 0.80(69.27) = 55.42$ kips/bolt

Once the bolts slip and go into bearing, the fill plate must be developed to distribute the total stress uniformly over the combined section of the member and the fill plate. Rather than extending the fill plate beyond the splice and providing additional bolts to develop the fill plate, Article 6.13.6.1.5 provides a reduction factor to be applied to the bolt shear strength for fill plates 1/4 in. or more in thickness. The factor is given as:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma}\right]$$

Where:

$\gamma = A_f / A_p$

A_f = sum of the area of the fillers on the top and bottom of the connected plate

A_p = sum of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate

$A_f = (0.375)(18) = 6.75$ in.^{2}

$A_p = (1.375)(18) = 24.75$ in.^{2}

$A_p = (0.625)(18) + 2(0.75)(8) = 23.25 \text{ in.}^2$ (Controls)

$$R = \left[\frac{\left(1+\frac{6.75}{23.25}\right)}{\left(1+2\frac{6.75}{23.25}\right)}\right] = 0.82$$

$R_r = R(55.42) = 0.82(55.42) = 45.44$ kips/bolt

This reduced value of R_r is only to be applied to the bolts on the side of the splice where the filler plate is located. The number of bolts required on this side of the splice is therefore computed as:

$$N = \frac{P_{cf}}{R_r} = \frac{928}{(45.44)} = 20.4 \text{ bolts}$$

The number of bolts required on the other side of the splice without the filler plate is:

$$N = \frac{P_{cf}}{R_r} = \frac{928}{(55.42)} = 16.7 \text{ bolts}$$

While a different number of bolts could theoretically be used on opposite sides of the splice, it is common practice to provide the same (larger) number of bolts on each side of the splice.

Therefore, the minimum number of bolts required to provide the necessary shear resistance at the strength limit state is twenty-two (22).

10.3.2.2 Slip Resistance: Permanent Deflection Service Limit State and for Constructability

Determine the minimum number of bolts required to provide adequate slip resistance at the permanent deflection service limit state and also during casting of the concrete deck (whichever governs).

First, determine the minimum flexural resistance of the flange F_{fs} that must be provided by the splice to satisfy the permanent deflection service limit state. F_{fs} is determined from Equation (6) given in the introduction to this splice-design example as follows:

$F_{fs} = 0.80F_{cf}$

$F_{fs} = 0.80(37.50) = 30.00$ ksi

The minimum design force P_{fs} for the bottom-flange splice at the permanent deflection service limit state is therefore equal to:

$P_{fs} = F_{fs}A_g = 30.00(18.0)(1.375) = 742.5$ kips

Separate calculations show that P_{fs} is larger than the factored flange force on the gross section of the flange due to the deck-casting sequence.

According to Article 6.13.2.2, the factored resistance R_r of a bolt at the permanent deflection service limit state and for checking slip during the deck-casting sequence shall be taken as:

$R_r = R_n$

where R_n is the nominal slip resistance of the bolt specified in Article 6.13.2.8. The resistance factor is taken as 1.00 since this is a serviceability criterion. Determine the slip resistance assuming a Class B surface condition for the faying surface, standard holes and two slip planes, N_s, per bolt.

$R_n = K_h K_s N_s P_t$

$K_h = 1.0$ (Table 6.13.2.8-2)

$K_s = 0.50$ (Table 6.13.2.8-3)

$P_t = 39$ kips (Table 6.13.2.8-1)

$R_r = R_n = 1.0(0.50)(2)(39) = 39.00$ kips/bolt

A Class A surface condition refers to a faying surface consisting of unpainted clean mill scale or to blast cleaned surfaces painted with a Class A coating. A Class B surface condition refers to an unpainted faying surface that has been blast cleaned, or else, to a surface that has been blast cleaned and painted with a Class B coating. Most commercially available primers will qualify as Class B coatings. Unpainted faying surfaces on weathering steel that have been blast cleaned qualify as Class B surfaces. Since faying surfaces are typically blast cleaned as a minimum, a Class A surface condition should only be used to compute the slip resistance when Class A coatings are applied or when unpainted clean mill scale is left on the faying surface.

The minimum number of bolts required to prevent slip is:

$$N = \frac{P_{fs}}{R_r} = \frac{742.5}{39.00} \cong 19 \text{ bolts}$$

which is less than 22 bolts, which was determined previously to satisfy the strength limit state and detailing.

Therefore, the number of bolts required for the bottom-flange splice is controlled by the shear resistance at the strength limit state. Use twenty-two (22) 7/8-in. diameter high-strength bolts on each side of the splice.

10.3.3 Tensile Resistance of Splice Plates: Strength Limit State

Check the tensile resistance of the bottom flange splice plates at the strength limit state:

As specified in Article 6.13.6.1.4c, the tensile resistance of the splice plate is determined according to the provisions of Article 6.13.5.2. The factored resistance R_r in tension is taken as the lesser of the values given by either Equation 6.8.2.1-1 for yielding, Equation 6.8.2.1-2 for fracture, or the block shear rupture resistance specified in Article 6.13.4. Block shear rupture and fracture on the net section of the smaller girder flange plate at the splice are also checked.

- For yielding on the gross section of the inside and outside splice plates (Article 6.8.2.1-1):

$$R_r = P_r = \phi_y P_{ny} = \phi_y F_y A_g$$

$$\phi_y = 0.95 \text{ (Article 6.5.4.2)}$$

Outside: $P_r = 0.95(50)(0.625)(18.0) = 534.4 \text{ kips} > P_{cf}/2 = 464.0 \text{ kips}$ ok

Inside: $P_r = 0.95(50)(2)(0.75)(8.0) = 570.0 \text{ kips} > P_{cf}/2 = 464.0 \text{ kips}$ ok

- For fracture on the net section of the inside and outside splice plates (Article 6.8.2.1-2):

$$R_r = P_r = \phi_u P_{nu} = \phi_u F_u A_n U$$

$F_u = 70$ ksi (Table 6.4.1-1)

$\phi_u = 0.80$ (Article 6.5.4.2)

U = reduction factor to account for shear lag = 1.0 for splice plates (Article 6.13.5.2)

To compute the net area A_n of the splice plates, assume four 7/8-in. bolts across the width of the splice plate. The bolt pattern is staggered in this case because of the need to increase the resistance of the splice plates and the smaller girder flange at the splice to fracture on the net section. A separate check is made to ensure that the splice plates do not interfere with any stiffeners or connection plates. First, check the indicated spacing of the bolts on the outside splice plate according to the requirements of Article 6.13.2.6:

10.3.3.1 Minimum Spacing (Article 6.13.2.6)

The minimum spacing between centers of bolts in standard holes shall not be less than three times the diameter of the bolt. For this example:

$$s_{min} = 3d = 3(0.875) = 2.63 \text{ in.} < \sqrt{(3.0)^2 + (3.5)^2} = 4.61 \text{ in.} \quad \text{ok}$$

10.3.3.2 Maximum Spacing for Sealing (Article 6.13.2.6.2)

The maximum spacing of the bolts along the free edges of the splice plates is limited to prevent the penetration of moisture in the joint.

First, check for sealing along the edges of the plate parallel to the applied force. The gage, g, or distance between adjacent uniformly staggered lines of bolts adjacent to a free edge, is limited as follows:

$$g < 1.5 + 4.0t$$

$$3.5 \text{ in.} < 1.5 + 4.0(0.625) = 4.0 \text{ in.} \qquad \text{ok}$$

The staggered spacing *s* between two adjacent lines of staggered bolt holes is limited as follows:

$$s \leq 4.0 + 4.0t - (3.0g / 4.0) \qquad \text{(Eq. 6.13.2.6.2-2)}$$

but not less than one-half the requirement for a single line of bolts = 4.0 + 4.0t.

$$3.0 \text{ in.} < 4.0 + 4.0(0.625) - \left(\frac{3.0(3.5)}{4.0}\right) = 3.875 \text{ in.} < 7.0 \text{ in.} \qquad \text{ok}$$

$$3.875 \text{ in.} > \frac{1}{2}\left[4.0 + 4.0(0.625)\right] = 3.25 \text{ in.} \qquad \text{ok}$$

Next, check for sealing along the free edge at the end of the outside splice plate:

$$s \leq 4.0 + 4.0t \leq 7.0$$

$$6.5 \text{ in.} = 4.0 + 4.0(0.625) = 6.5 \text{ in.} < 7.0 \text{ in. ok}$$

Note that the maximum pitch requirements for stitch bolts in the current Articles 6.13.2.6.3 and 6.13.2.6.4 apply only to the connection of plates in mechanically fastened built-up members and would not apply here.

10.3.3.3 Edge Distance (Article 6.13.2.6.6)

The edge distance is defined as the distance perpendicular to the line of force between the center of the outermost line of holes and the edge of the plate. The specified edge distance of 2-1/4 in. satisfies the minimum edge distance of 1-1/2 in. specified for 7/8-in. diameter bolts (Table 6.13.2.6.6-1), and also satisfies the maximum edge distance requirement of 8.0t (not to exceed 5.0 in.) = 8.0(0.625) = 5.0 in.

10.3.3.4 End Distance (Article 6.13.2.6.5)

The end distance is defined as the distance along the line of force between the center of a hole and the end of the plate. The specified end distance of 1-1/2 in. for the two outermost bolts at the end of the plate is equal to the minimum end distance of 1-1/2 in. specified for 7/8-in. diameter bolts (Table 6.13.2.6.6-1). The end distance of 4.5 in. for the two corner bolts satisfies the maximum end distance requirement of 8.0t = 5.0 in. The distance from the corner bolts to the corner of the plate, equal to:

$$\sqrt{(4.5)^2 + (2.25)^2} = 5.03 \text{ in.}$$

slightly exceeds the maximum end distance requirement. Although not shown here, the corners of the plate can be clipped, if desired, to satisfy the end distance requirement.

10.3.3.5 Fracture Check

As specified in Article 6.8.3, the net width, W_n, shall be determined for each chain of holes extending across the member along any transverse, diagonal or zigzag line. W_n is determined by subtracting from the width of the element the sum of the width of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain, where s is the pitch between two consecutive holes and g is the gage of the same two holes. The design width of each hole is taken as 1/8-in. greater than the nominal diameter of the bolt (Article 6.8.3), or 1 in. for a 7/8-in. diameter bolt. Also, as specified in Article 6.13.5.2, A_n of the splice plates *to be used in calculating the fracture strength of the splice plates* shall not exceed 85 percent of the gross area of the plates. Appropriate chains checked for the outside splice plate are shown in Figure 17:

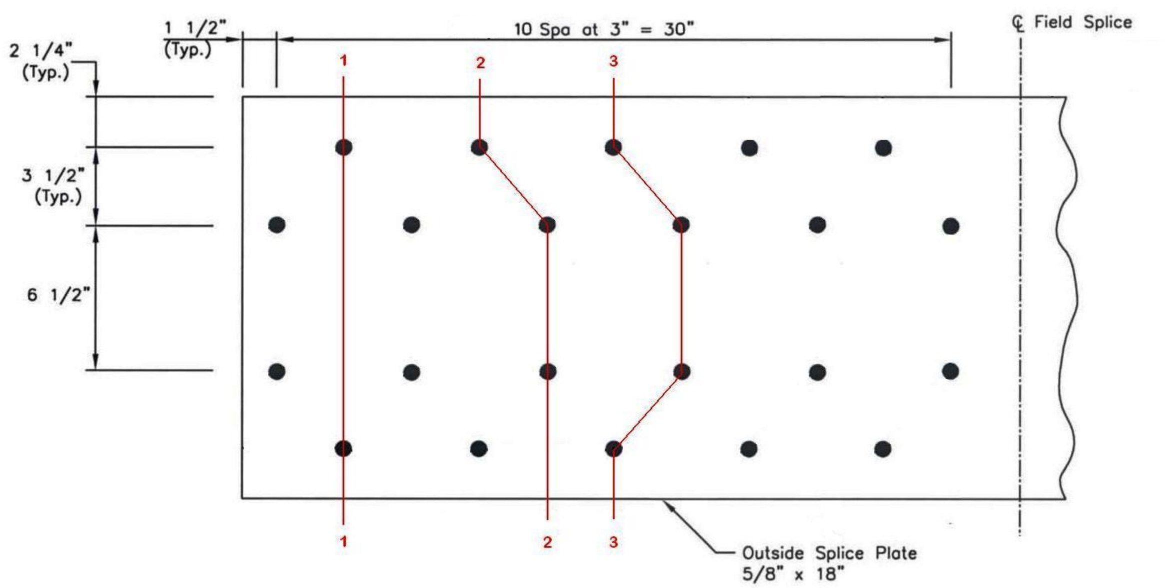

Figure 17 Sketch of the Outside Splice Plate Fracture Chains

Outside: $W_n = 18.0 - (2)(1.0) = 16.00$ in.

$W_n = 18.0 - 3(1.0) + (3.0)^2 \div [(4)(3.5)] = 15.64$ in.

$W_n = 18.0 - 4(1.0) + (2)(3.0)^2 \div [(4)(3.5)] = 15.29$ in. (governs)

Note: Other chains do not govern by inspection.

$A_n = 15.29(0.625) = 9.56$ in.^{2}

$0.85A_g = 0.85(0.625)(18.0) = 9.56$ in.^{2} (ok)

Therefore, $A_n = 9.56$ in.^{2}

$P_r = 0.80(70)(9.56)(1.0) = 535.4$ kips $> P_{cf}/2 = 464.0$ kips (ok)

Inside: $W_n = 8.0 - (1.0) = 7.00$ in.

$W_n = 8.0 - 2(1.0) + (3.0)^2 \div [(4)(3.5)] = 6.64$ in. (governs)

$A_n = (2)(6.64)(0.75) = 9.96$ in.^{2}

$0.85A_g = 0.85[(2)(0.75)(8.0)] = 10.20$ in.^{2} > 9.96 in.^{2}

Therefore, $A_n = 9.96$ in.^{2}

$P_r = 0.80(70)(9.96)(1.0) = 557.84$ kips $> P_{cf}/2 = 464.0$ kips (ok)

- For block shear rupture on the inside and outside splice plates (Article 6.13.4): Block shear rupture will typically not govern except for short compact connections. Therefore, the block shear checks will not be illustrated in this example.

- For fracture on the net section of the girder flange at the splice (Article 6.8.2.1), both sides were checked and the right side controls:

  $A_n = \{20.0 - 4(1.0) + (2)(3.0)^2 \div [(4)(3.5)]\}(1.00) = 17.29$ in.^{2}

  $P_r = 0.80(85)(17.29)(1.0) = 1176$ kips $> P_{fc} = 928$ kips ok

10.3.4 Compressive Resistance of Splice Plates: Strength Limit State

Since the splice plates are subject to stress reversal, check the compressive resistance of the bottom flange splice plates at the strength limit state.

The maximum calculated compressive stress in the bottom flange at the splice due to the factored loads was computed earlier to be -16.79 ksi, and the minimum compressive design flange force (P_{cf}) was computed previously as 928 kips.

According to the Commentary to Article 6.13.6.1.4c, for compression, an unbraced length of zero may be assumed for the splice plates. Therefore, the factored compressive resistance of the splice plates can be computed as follows:

$$P_r = \phi_c P_n = \phi_c F_y A_s \qquad \text{(Articles 6.9.2.1-1 and 6.9.4.1-1)}$$

$$\phi_c = 0.90 \qquad \text{(Article 6.5.4.2)}$$

Outside: $P_r = 0.90(50)(0.625)(18.0) = 506.3 \text{ kips} > (P_{fcr})_{bf}/2 = 928/2 = 464 \text{ kips}$ ok

Inside: $P_r = 0.90(50)(2)(0.75)(8.0) = 540.0 \text{ kips} > (P_{fcr})_{bf}/2 = 928/2 = 464 \text{ kips}$ ok

10.3.5 Bearing Resistance At Bolt Holes: Strength Limit State

Check bearing of the bolts on the flange material at the strength limit state (Article 6.13.2.9). The nominal bearing resistance of the flange is calculated as the sum of the resistance of the individual bolt holes parallel to the line of the applied force.

The nominal bearing resistance for standard holes is taken as (Article 6.13.2.9):

For $L_c > 2.0d$: $R_n = 2.4dtF_u$

Otherwise: $R_n = 1.2L_ctF_u$

where:

L_c = the clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

F_u = tensile strength of the connected material = 70 ksi for the left side of the splice and 85 ksi for the right side of the splice (Table 6.4.1-1)

d = nominal diameter of the bolt (in.)

t = thickness of the connected material (in.)

As specified in Article 6.8.3, the design width for standard bolt holes to be used in design shall be taken as 1/8-in. greater than the nominal diameter of the bolt.

For the outer lines of bolts, the end distance is 4-1/2 in. Therefore, the clear end distance between the edge of the first hole and the end of the splice plate is:

$$L_{c1} = 4.5 - 1.0/2 = 4.0 \text{ in.} > 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

For the inner two lines of bolts, the end distance is 1-1/2 in. Therefore, the clear end distance is:

L_{c1} = 1.5 - 1.0/2 = 1.0 in. > 2.0d = 1.75 in.

The center-to-center distance between bolts in the direction of the force in both the outer and inner lines of bolts is 6 in. Therefore, the clear distance between edges of adjacent holes is computed as:

L_{c2} = 6.0 - 1.0 = 5.0 in. > 2.0d = 1.75 in.

Therefore, the nominal bearing resistance of an outer line of bolt holes in the bottom flange at the right side of the splice is taken as:

$R_{n\_outer} = 5(2.4dtF_u) = 5[2.4(0.875)(1.00)(85)] = 892.5$ kips

The nominal bearing resistance of an inner line of bolt holes is taken as:

$R_{n\_inner} = 1(1.2L_ctF_u) + 5(2.4dtF_u) = 1[1.2(1.0)(1.00)(85)] + 892.5 = 994.5$ kips

$R_{n\_TOTAL} = 2(892.5) + 2(994.5) = 3774$ kips

$R_r = \phi_{bb}R_n = 0.80(3774) = 3019$ kips

$P_{fr} = 928$ kips $< R_r = 3019$ kips ok

10.3.6 Fatigue Resistance of Splice Plates: Fatigue and Fracture Limit State

Check the fatigue stresses in the base metal of the bottom flange splice plates adjacent to the slip-critical connections.

By inspection, the bottom flange is subject to a net tensile stress. The moments at the splice due to the fatigue load plus the 15 percent dynamic load allowance specified in Table 3.6.2.1-1 and factored by the 0.75 load factor specified for the FATIGUE load combination are:

$M_{+LL+IM} = 0.75(905) = 679$ kip-ft

$M_{-LL+IM} = 0.75(-687) = -515$ kip-ft

The maximum flange stresses rather than the average flange stresses will be used in the fatigue check. The maximum bottom-flange stresses computed on the gross section are (negative fatigue-load moments are assumed to be applied to the section consisting of the steel girder plus the longitudinal reinforcement since the maximum composite dead-load compressive stresses in the slab at the splice are overcome by the maximum tensile stress in the slab at the splice due to the factored negative fatigue-load moment):

$f_{+(LL+IM)} = 679(12) \div 2706 = 3.01$ ksi (tension)

$f_{-(LL+IM)} = -515(12) \div 2182 = -2.83$ ksi (compression)

$\gamma(\Delta f) = f_{+(LL+IM)} + |f_{-(LL+IM)}| = 3.01 + |-2.83| = 5.84$ ksi

From Equation (6.6.1.2.5-1), the nominal fatigue resistance is specified as:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{TH}$$

Earlier computations showed that the 75-year $(ADTT)_{SL}$ is 1600 trucks per day. Using the table presented in Example 1, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category B detail (with n = 1.0) is 865 trucks per day < 1600 trucks per day.

Thus:

$$(\Delta F)_n = \frac{1}{2}(\Delta F)_{TH}$$

For a Category B detail, $(\Delta F)_{TH}$ = 16.0 ksi (Table 6.6.1.2.5-3). Therefore:

$(\Delta F)_n = 16.0 / 2 = 8.00$ ksi

Calculate the range of flange force from the computed stress range in the flange:

$\Delta P_{FLG} = 5.84(1.375)(18.0) = 144.5$ kips

Since the combined area of the inside splice plates is within 10 percent of the area of the outside splice plate, check both the inside and outside splice plates for 1/2 of the total range of flange force. The range of fatigue force and stress in the outside splice plate is:

$\Delta P = 144.5 / 2 = 72.3$ kips

$$\Delta f = \frac{72.3}{(0.625)(18.0)} = 6.42 \text{ ksi} < 8.00 \text{ ksi} \qquad \text{ok}$$

The range of fatigue force and stress in the inside splice plates is:

$\Delta P = 72.3$ kips

$$\Delta f = \frac{72.3}{2(0.75)(8.0)} = 6.03 \text{ ksi} < 8.00 \text{ ksi} \qquad \text{ok}$$

10.3.7 Permanent Deflection Service Limit State: Splice Plates

To satisfy the permanent deflection service limit state, check the stress in the bottom flange splice plates to control local yielding of the plates.

The following criterion will be used to make this check (refer to Article 6.10.3.2):

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad \text{(Eq. 6.10.3.2.3-1)}$$

The resistance factor is equal to 1.0.

The minimum design force on the gross section of the bottom flange, P_{fs}, at the permanent deflection service limit state was computed earlier to be 742.5 kips.

For reasons discussed previously, the force on the inside and outside splice plates is taken as (742.5/2) = 371.3 kips. The resulting stress in the outside splice plate is then computed as:

$$f_{bu} = \frac{371.3}{(0.625)(18.0)} = 33.00 \text{ ksi} < 1.0(1.0)(50) = 50.0 \text{ ksi} \qquad \text{ok}$$

The stress in the inside splice plates is computed as:

$$f_{bu} = \frac{371.3}{2(0.75)(8.0)} = 30.94 \text{ ksi} < 1.0(1.00)(50) = 50.0 \text{ ksi} \qquad \text{ok}$$

10.4 Top-Flange Splice Design

For this example, the top flange has been determined to be the non-controlling flange.

10.4.1 Minimum Flexural Resistance: Strength Limit State

Determine the minimum flexural resistance of the non-controlling flange F_{ncf} that must be provided by the splice to satisfy the strength limit state. From the previous calculation, the largest flexural stress in the top flange at the splice due to the factored loads is tensile (negative flexure controls). However, this is not concurrent with the largest flexural stress in the controlling (bottom) flange (f_{cf} = 18.83 ksi), that was used in the determination of F_{cf}. Therefore, two separate checks will be made to determine which loading condition controls for the top flange (assuming f_{cf} = 18.83 ksi concurrent with f_{ncf} = -4.28 ksi and assuming f_{cf} = -16.79 ksi concurrent with f_{ncf} = 15.67 ksi).

Equation (3) presented in the introduction to this splice-design example will be used to compute F_{ncf}:

$$F_{ncf} = R_{cf}\left(\left|\frac{f_{ncf}}{R_h}\right|\right) \geq 0.75\alpha\phi_y F_{yf}$$

<u>Check #1:</u>

for f_{cf} = 18.83 ksi and F_{cf} = 37.50 ksi, R_{cf} = (37.50/18.83) = 1.99

$f_{ncf} = f_{fu} = -4.28$ ksi

$\alpha = 1.0$ (Article C6.13.6.1.4c)

$R_h = 1.0$

$$1.99\left(\left|\frac{-4.28}{1.0}\right|\right) = 8.52 \text{ ksi}$$

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.00)(50) = 37.50$ ksi (governs)

Therefore, $F_{ncf} = 37.50$ ksi

Check #2:

for $f_{cf} = -16.79$ ksi and $F_{cf} = 37.50$ ksi, $R_{cf} = (37.50/16.79) = 2.23$

$f_{ncf} = f_{fu} = 15.67$ ksi

$\alpha = 1.0$ (Article C6.13.6.1.4c)

$R_h = 1.0$

$$2.25\left(\left|\frac{15.67}{1.0}\right|\right) = 39.94 \text{ ksi}$$

$0.75\alpha\phi_f F_{yf} = 0.75(1.0)(1.00)(50) = 37.50$ ksi (governs)

Therefore, $F_{ncf} = 37.50$ ksi

For either case, the design stress for the non-controlling flange, F_{ncf}, is taken as 37.50 ksi. The minimum design force, P_{ncf}, for checking the resistance of the top-flange splice and for determining the number of bolts required to satisfy the strength limit state is therefore equal to F_{ncf} times the effective net area for negative bending (tension in the top flange) and F_{cf} times the gross area for positive bending (compression in the top flange). The maximum force for the non-controlling flange will be the product of the design stress and the gross area of the smaller plate. The design flange force is as follows:

$P_{ncf} = F_{ncf}A_g = 37.50(16.0)(1.00) = 600.0$ kips

The total combined area of the inside and outside splice plates should approximately equal or exceed the area of the smaller flange at the splice. The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. Therefore, try a 9/16" x 16" outside plate with two - 5/8" x 7" inside plates. The combined area of the inside plates is within

10 percent of the area of the outside splice plate. Therefore, design both the inside and outside splice plates for tension for 1/2 of P_{ncf} equal to 300.0 kips.

10.4.2 Flange Bolts

Following the procedures demonstrated earlier for the design of the bolts for the bottom-flange splice:

10.4.2.1 Shear Resistance: Strength Limit State

Since no filler plate is required, R_r is taken as 55.42 kips/bolt on both sides of the splice. Thus, the number of bolts required for the top-flange splice to satisfy the factored shear resistance at the strength limit state is:

$$N = P_{ncf} / R_r = 600.0 / 55.42 = 10.8 \text{ bolts}$$

Therefore, the minimum number of bolts required to provide the necessary shear resistance at the strength limit state is twelve (12).

10.4.2.2 Slip Resistance: Permanent Deflection Service Limit State & for Constructibility

Determine the minimum number of bolts required to provide adequate slip resistance at the permanent deflection service limit state and also during casting of the concrete deck (whichever governs).

First, determine the minimum flexural resistance of the flange, F_{fs}, that must be provided by the splice to satisfy the permanent deflection service limit state. F_{fs} is determined from Equation (6) given in the introduction to this splice-design example as follows:

$$F_{fs} = 0.80F_{ncf}$$

$$F_{fs} = 0.80(37.50) = 30.00 \text{ ksi}$$

The minimum design force, P_{fs}, for the top-flange splice at the permanent deflection service limit state is therefore equal to:

$$P_{fs} = F_{fs}A_g = 30.00(16.0)(1.0) = 480.0 \text{ kips}$$

Separate calculations show that P_{fs} is larger than the factored flange force on the gross section of the flange due to the deck-casting sequence.

The factored slip resistance of a bolt in double shear, R_r, was computed earlier to be 39.00 kips. Thus, the minimum number of bolts required to provide adequate slip resistance is computed as:

$$N = P_{fs} / R_r = 12.3 \text{ bolts} > N = 12 \text{ bolts determined previously for STRENGTH limit state}$$

Therefore, the number of bolts required for the top-flange splice is controlled by the slip resistance at the permanent deflection service limit state. Use fourteen (14) 7/8-in. diameter high-strength bolts (7 rows of 2 bolts at 3-in. spacing – staggered) on each side of the splice.

10.4.3 Tensile Resistance of Splice Plates: Strength Limit State

Check the tensile resistance of the top-flange splice plates at the strength limit state. Block shear rupture and fracture on the net section of the girder flange plate at the splice are also checked.

- For yielding on the gross section of the inside and outside splice plates (Article 6.8.2.1-1)

  Outside: $P_r = 0.95(50)(0.5625)(16.0) = 427.5$ kips $> P_{ncf}/2 = 300.0$ kips ok

  Inside: $P_r = 0.95(50)(2)(0.625)(7.0) = 415.6$ kips $> P_{ncf}/2 = 300.0$ kips ok

- For fracture of the net section of the inside and outside splice plates (Article 6.8.2.1):

  $R_r = P_r = \phi_u F_u A_n U$ (6.8.2.1-2)

10.4.3.1 Minimum spacing (Article 6.13.2.6)

$$s_{min} = 3d = 3(0.875) = 2.63\text{ in.} < \sqrt{(3.0)^2 + (3.5)^2} = 4.61\text{ in.} \qquad \text{ok}$$

10.4.3.2 Maximum spacing for sealing (Article 6.13.2.6.2)

$g < 1.5 + 4.0t$

3.5 in. $< 1.5 + 4.0(0.5625) = 3.75$ in.

Therefore, $s \leq 4.0 + 4.0t - [3.0g / 4.0] \leq 7.0$, but *s* shall not be less than one-half the requirement for single line of bolts (6.13.2.6.2-2).

3.0 in. $< 4.0 + 4.0(0.5625) - [(3.0)(3.5)/4.0] = 3.625$ in. < 7.0 in. ok

3.625 in. $> (1/2)[4.0 + 4.0(0.5625)] = 3.125$ in. ok

Check for sealing along the free edge at the end of the splice plate:

$s \leq 4.0 + 4.0t \leq 7.0$

4.5 in. $= 4.0 + 4.0(0.5625) = 6.25$ in. < 7.0 in. ok

10.4.3.3 Edge distance (Article 6.13.2.6.6)

Edge distance $\geq$ 1-1/2 in. (Table 6.13.2.6.5)

1-3/4 in. $\geq$ 1-1/2 in. ok

Edge distance $\leq$ 8.0t $\leq$ 5.0

$4.5 \leq 8.0(0.5625) \leq 5.0$

4.5 in. $\leq$ 4.5 in. $\leq$ 5.0 in. ok

Distance from the corner bolts to the corner of the plate $\leq$ 5.0

$$\sqrt{(4.5)^2 + (2.25)^2} = 5.03\,\text{in.} > 5.00\,\text{in.}$$

Therefore, clip the corners of the plate.

10.4.3.4 Fracture check:

Outside: $W_n = 16.0 - (2)(1.0) = 14.00$ in.

$W_n = 16.0 - 3(1.0) + (3.0)^2 \div [(4)(3.5)] = 13.64$ in.

$W_n = 16.0 - 4(1.0) + (2)(3.0)^2 \div [(4)(3.5)] = 13.29$ in. (governs)

Note: Other chains do not govern by inspection.

$A_n = 13.29(0.5625) = 7.48$ in.^{2}

$0.85A_g = 0.85(0.5625)(16.0) = 7.65$ in.^{2} (ok)

Therefore, $A_n = 7.48$ in.^{2}

$P_r = 0.80(70)(7.48)(1.0) = 418.9$ kips $> P_{ncf}/2 = 300.0$ kips (ok)

Inside: $W_n = 7.0 - (1.0) = 6.00$ in.

$W_n = 7.0 - 2(1.0) + (3.0)^2 \div [(4)(3.5)] = 5.64$ in. (governs)

$A_n = (2)(5.64)(0.625) = 7.05$ in.^{2}

$0.85A_g = 0.85[(2)(0.625)(7.0)] = 7.44$ in.^{2} (ok)

Therefore, $A_n = 7.05$ in.^{2}

$P_r = 0.80(70)(7.05)(1.0) = 394.8$ kips $> P_{cf}/2 = 300.0$ kips (ok)

- Block shear rupture on the inside and outside splice plates (Article 6.13.4):

Block shear rupture will typically not govern except for short compact connections. Therefore, the block shear checks will not be illustrated in this example.

- For fracture on the net section of the smaller girder flange at the splice (Article 6.8.2.1):

$A_n = \{16.0 - 4(1.0) + (2)(3.0)^2 \div [(4)(3.5)]\}(1.00) = 13.29 \text{ in.}^2$

$P_r = 0.80(85)(13.29)(1.0) = 744 \text{ kips} > P_{fc} = 600 \text{ kips}$ ok

10.4.4 Compressive Resistance of Splice Plates: Strength Limit State

Since the splice plates are subject to stress reversal, check the compressive resistance of the top flange splice plates at the strength limit state.

The maximum calculated compressive stress in the top flange at the splice due to the factored loads was computed earlier to be -4.28 ksi, and the minimum compressive design flange force, P_{ncf}, was computed previously as 600 kips.

According to the Commentary to Article 6.13.6.1.4c, for compression, an unbraced length of zero may be assumed for the splice plates. Therefore, the factored compressive resistance of the splice plates can be computed as follows:

$P_r = \phi_c P_n = \phi_c F_y A_s$ (Articles 6.9.2.1-1 and 6.9.4.1-1)

$\phi_c = 0.90$ (Article 6.5.4.2)

Outside: $P_r = 0.90(50)(0.5625)(16.0) = 405.0 \text{ kips} > (P_{fcr})_{tf}/2 = 600.0/2 = 300.0 \text{ kips}$ ok

Inside: $P_r = 0.90(50)(2)(0.625)(7.0) = 393.8 \text{ kips} > (P_{fcr})_{tf}/2 = 600.0/2 = 300.0 \text{ kips}$ ok

For composite sections, the compressive resistance of the top-flange splice plates should not govern in most cases.

10.4.5 Bearing Resistance at Bolt Holes: Strength Limit State

Check bearing of the bolts on the flange material at the strength limit state:
For bolts with a 1-1/2 in. end distance, the clear distance between the edge of the hole and the left edge of the splice plate is:

$L_{c1} = 1.5 - 1.0 / 2 = 1.0 \text{ in.} < 2.0d = 2.0(0.875) = 1.75 \text{ in.}$

Therefore, $R_{n\_HOLE1} = 1.2L_c tF_u = 1.2(1.0)(1.00)(70) = 84.0 \text{ kips}$ (6.13.2.9-2)

For bolts with a 4-1/2 in. end distance, the clear distance between the edge of the hole and the edge of the splice plate is:

$L_{c2} = 4.5 - 1.0 / 2 = 4.0$ in. $> 2.0d = 2.0(0.875) = 1.75$ in.

Therefore, $R_{n\_HOLE2} = 2.4dtF_u = 2.4(0.875)(1.00)(70) = 147.0$ kips (6.13.2.9-1)

The center-to-center distance between bolts in the direction of force is equal to 6 in. Therefore, the clear distance between edges of adjacent holes is computed as:

$L_{c3} = 6.0 - 1.0 = 5.0$ in. $> 2.0d = 1.75$ in.

Therefore, $R_{n\_HOLE3} = R_{n\_HOLE2} = 147.0$ kips

$R_{n\_TOTAL} = 2[84.00 + 6(147.0)] = 1932$ kips

$R_r = \phi_{bb}R_n = 0.80(1932) = 1546$ kips (6.13.2.2-2)

$P_{ncf} = 600.0$ kips $< R_r = 1546$ kips ok

10.4.6 Fatigue Resistance of Splice Plates: Fatigue and Fracture Limit State

Check fatigue stresses in the base metal of the top-flange splice plates adjacent to the slip-critical connections.

The maximum flange stresses rather than the average flange stresses will be used in the fatigue check. The maximum top-flange stresses computed on the gross section due to the factored fatigue load plus the 15 percent dynamic load allowance (Table 3.6.2.1-1) are (as discussed previously, negative fatigue-load moments are assumed to be applied to the section consisting of the steel girder plus the longitudinal reinforcement):

$f_{+(LL+IM)} = 679(12) \div 13805 = 0.59$ ksi (tension)

$f_{-(LL+IM)} = -515(12) \div 2327 = -2.66$ ksi (compression)

By inspection, the compressive force in the top flange due to the unfactored permanent loads is less than twice the tensile flange force due to the negative factored fatigue-load moment. Thus, fatigue must be checked.

$\gamma(\Delta f) = f_{+(LL+IM)} + |f_{-(LL+IM)}| = 0.59 + |-2.66| = 3.25$ ksi

From the previous calculations: $(\Delta F)n = 8.00$ ksi for a fatigue Category B detail.

Calculate the range of flange force from the computed stress range in the flange:

$\Delta P_{FLG} = 3.25(1.00)(16.0) = 52.00$ kips

Since the combined area of the inside splice plates is within 10 percent of the area of the outside splice plate, check both the inside and outside splice plates for 1/2 of the total range of flange force. The range of fatigue force and stress in the outside splice plate is:

$\Delta P = 52.00 / 2 = 26.00$ kips

$\Delta f = 26.00 / [(0.5625)(16.0)] = 2.89$ ksi < 8.00 ksi ok

The range of fatigue force and stress in the inside splice plates is:

$\Delta P = 26.00$ kips

$\Delta f = 26.00 / [2(0.5625)(7.0)] = 2.97$ ksi < 8.00 ksi ok

10.4.7 Permanent Deflection Service Limit State: Splice Plates

To satisfy the permanent deflection service limit state, check the stress in the top-flange splice plates to control local yielding of the plates.

The following criterion will be used to make this check (refer to Article 6.10.3.2):

$f_{bu} \leq \phi_f R_h F_{yf}$ (Eq. 6.10.3.2.3-1)

The resistance factor is equal to 1.0.

The minimum design force on the gross section of the top flange, P_{fs}, at the permanent deflection service limit state was computed earlier to be 480.0 kips.

For reasons discussed previously, the force on the inside and outside splice plates is taken as (480.0/2) = 240.0 kips. The resulting stress in the outside splice plate is then computed as:

$f = 240.00 / [(0.5625)(16.0)] = 26.67$ ksi $< 1.0(1.00)(50) = 50.0$ ksi ok

The stress in the inside splice plates is computed as:

$f = 240.00 / [2(0.\ 625)(7.0)] = 27.43$ ksi $< 1.0(1.00)(50) = 50.0$ ksi ok

NOTE: In many instances, it may be possible to deduce that the fatigue and permanent deflection service limit state checks illustrated above for the top- and bottom-flange splice plates do not govern, and thus, need not be made. However, the checks have been included here for completeness and to illustrate a suggested methodology for application to those situations where the checks are deemed to be necessary.

11.0 WEB SPLICE DESIGN

In this example, the web splice is designed under the conservative assumption that the maximum moment and shear at the splice will occur under the same loading condition.

Determine the minimum shear resistance of the web at the splice, V_{wr}, that must be provided by the web splice plates and their connections to satisfy the strength limit state. For the controlling strength limit state load combination (STRENGTH I), first determine the maximum shear at the splice due to the factored loads. The maximum permanent-load load factor, γ_p, (Table 3.4.1-2) is applied to permanent-load effects having the same sign as the live-load effects. For permanent-load effects with a sign opposite to the live-load effects, the minimum permanent-load load factor (Table 3.4.1-2) is applied and the effect of the future wearing surface is conservatively ignored. In both cases, η is taken equal to 1.0.

$V_u = 1.0[1.25(-85-12) + 1.50(-12) + 1.75(-114)] = -339$ kips governs

$V_u = 1.0[0.9(-85-12) + 1.75(18)] = -56$ kips

The nominal shear resistance of the web at the splice for a 12.00 ft (144.0 in) transverse-stiffener spacing (see Figure 14) is computed as follows (Article 6.10.9.3.2):

$$\frac{2Dt_w}{\left(b_{fc}t_{fc} + b_{ft}t_{ft}\right)} \le 2.5$$

$$\frac{2(69)(0.50)}{(16)(1.0) + (18)(1.375)} = 1.69 \le 2.5$$

Therefore:

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \qquad \text{(Eq. 6.10.9.3.2-2)}$$

$d_o / D = 144 / 69.0 = 2.1$

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{(2.1)^2} = 6.13 \qquad \text{(Eq. 6.10.9.3.2-7)}$$

$D / t_w = 69.0 / 0.5 = 138.0$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(6.13)}{50}} = 83.5$$

$$\frac{D}{t_w} = 138.0 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 83.5$$

Therefore:

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \qquad \text{(Eq. 6.10.9.3.2-6)}$$

$$C = \frac{1.57}{(138.0)^2}\left(\frac{(29000)(6.13)}{50}\right) = 0.29$$

$V_p = 0.58F_{yw}Dt_w = 0.58(50)(69.0)(0.500) = 1001$ kips (Eq. 6.10.9.3.2-3)

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right]$$

$$V_n = 1{,}001\left[0.29 + \frac{0.87(1-0.29)}{\sqrt{1+(2.1)^2}}\right] = 556 \text{ kips}$$

The factored shear resistance V_r at the strength limit state is therefore equal to:

$V_r = \phi_v V_n = 1.00(556) = 556$ kips

The proposed minimum shear resistance of the web V_{wr} that must be provided by the web splice to satisfy the strength limit state is taken from either Equation (9) or Equation (10) given earlier in the introduction to this splice-design example. The equation to use depends on the value of V_u with respect to V_r as follows:

$0.5\phi_v V_n = 0.5V_r = 0.5(556) = 278$ kips

$|Vu| = |-339 \text{ kips}| = 339 \text{ kips} > 278$ kips

Therefore: $V_{wr} = (1/2)\,(V_u + \phi_v V_n)$ (Eq. 6.13.6.1.4b-2)

V_{wr} = (1/2)(339 + 556) = 448 kips

Two vertical rows of bolts with 22 bolts per row will be investigated (Figure 18). The bolts are spaced horizontally and vertically as indicated in Figure 18. The outermost rows of bolts are located 4 5/16 in. from the flanges to provide clearance for assembly (see the AISC *Manual of Steel Construction* for required bolt assembly clearances). The web is spliced symmetrically by plates on each side with a thickness not less than one-half the thickness of the web. Assume 3/8" x 64" splice plates on each side of the web. A fill plate is not included since the difference in thickness of the web plates on either side of the splice is only 1/16".

Using the equations given for web splices in the introduction to this splice design example, determine the portion of the flexural moment to be resisted by the web, M_{uw}, and the horizontal force resultant, H_{uw}, in the web.

For these calculations (for the strength checks), M_{uw} and H_{uw} will be computed by conservatively using the stresses at the mid-thickness of the flanges per Article C6.13.6.1.4b. By doing so, the same stress values that were used for the flange splice design can be utilized, thereby simplifying the calculations. Alternatively, the stresses at the inner fibers of the flanges can be used (which will be illustrated in the SERVICE II checks for slip of the web bolts, later in this example).

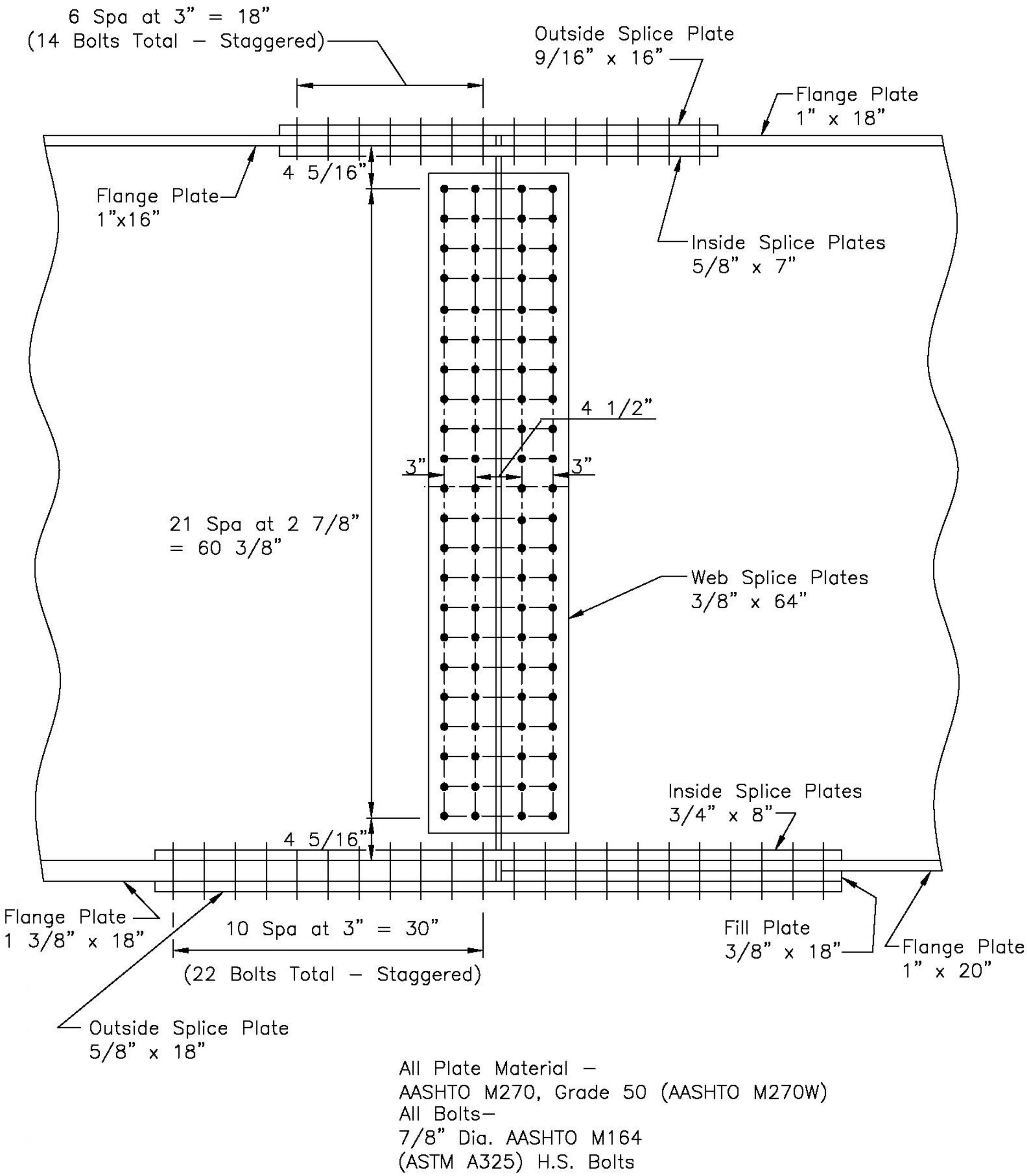

Figure 18 Sketch of the End Span Field Splice

For the loading condition (referred to subsequently as Case 1) causing the maximum flexural stress in the bottom (controlling) flange, the flexural moment to be resisted by the web is computed per Equation (4) as:

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| \qquad \text{(Eq. C6.13.6.1.4b-1)}$$

$R_hF_{cf} = 1.0(37.50) = 37.50$ ksi

$R_{cf}f_{ncf} = |F_{cf} / f_{cf}| (f_{ncf}) = |37.50 / 18.83|(-4.28) = -8.52$ ksi

$$M_{uw} = \frac{(0.50)(69.0)^2}{12}\left|37.50 - (-8.52)\right| = 9129 \text{ kip-in} = 761 \text{ kip-ft}$$

Because M_{uw} is to be applied at the centroid of the web, a horizontal force resultant H_{uw} must also be applied at the centroid of the web to maintain equilibrium. For this case, H_{uw} can be computed from the average of the maximum flexural stresses in the web from Equation (5) as follows:

$$H_{uw} = \frac{t_w D}{2}\left|R_h F_{cf} + R_{cf} f_{ncf}\right|$$

$$H_{uw} = \frac{(0.500)(69.0)}{2}\left|37.50 + (-8.52)\right| = 500 \text{ kips}$$

Similarly, for the loading condition (referred to subsequently as Case 2) causing the maximum flexural stress in the top flange, M_{uw} is computed as:

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| \qquad \text{(Eq. C6.13.6.1.4b-1)}$$

$R_hF_{cf} = 1.0(37.50) = 37.50$ ksi

$R_{cf}f_{ncf} = |F_{cf} / f_{cf}| (f_{ncf}) = |37.50 / -16.79|(15.67) = 35.00$ ksi

$$M_{uw} = \frac{(0.50)(69.0)^2}{12}\left|37.50 - 35.00\right| = 496 \text{ kip-in} = 41 \text{ kip-ft}$$

The horizontal force resultant H_{uw} for this case is computed as:

$$H_{uw} = \frac{0.500(69.0)}{2}\left|37.50 + 35.00\right| = 1251 \text{ kips}$$

The moment due to the eccentricity of V_{wr} from the centerline of the splice to the center of gravity of the web-splice bolt group is:

$M_e = V_{wr}(e) = (448)(2.25 + 3.0/2)(1/12) = 140$ kip-ft

Therefore:

$M_{TOTAL\_CASE1} = M_{uw} + M_e = 761 + 140 = 901$ kip-ft (Case 1)

or, $M_{TOTAL\_CASE2} = M_{uw} + M_e = 41 + 159 = 200$ kip-ft (Case 2)

11.1 Web Bolts

Web-splice bolts are to be designed for the effects of the moment due to the eccentric shear in addition to the effects of the flexural moment in the web and the horizontal force resultant, which are all applied at the centroid of the web.

Calculate the polar moment of inertia, I_p, of the bolts with respect to the centroid of the web:

The polar moment of inertia of the web bolts about the centroid of the web can be computed from the following equation (for a completely filled rectangular bolt group):

$$I_p = \frac{nm}{12}\left[s^2(n^2 - 1) + g^2(m^2 - 1)\right] \qquad \text{(Eq. C6.13.6.4.4b-3)}$$

where: m = number of vertical rows of bolts

n = number of bolts in one vertical row

s = the vertical pitch

g = the horizontal pitch

Therefore:

$$I_p = \frac{22(2)}{12}\left[(2.875)^2(22^2 - 1) + (3.0)^2(2^2 - 1)\right] = 14{,}737 \text{ in.}^2$$

11.1.1 Shear Resistance: Strength Limit State

For the strength limit state checks:

Determine the vertical bolt force due to the design shear assuming two vertical rows with 22 bolts per row (Figure 18) for a total number of bolts, N_b, equal to 44:

$P_s = V_{wr} / N_b = 448 / 44 = 10.18$ kips/bolt

For Case 1:

Determine the bolt force due to the horizontal force resultant:

$P_H = H_{uw} / N_b = 500 / 44 = 11.36$ kips/bolt

Determine the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice:

$P_{My} = (M_{TOTAL})(x) / I_p = (901)(12)(3.0 / 2) / 14737 = 1.10$ kips

$P_{Mx} = (M_{TOTAL})(y) / I_p = (901)(12)(30.19) / 14737 = 22.15$ kips

The resultant bolt force is:

$$P_r = \sqrt{(P_s + P_{Mv})^2 + (P_H + P_{Mh})^2}$$

$$P_r = \sqrt{(10.18 + 1.10)^2 + (11.36 + 22.15)^2} = 35.36 \text{ kips} \quad \text{(governs)}$$

For Case 2:

Determine the bolt force due to the horizontal force resultant:

$P_H = H_{uw} / N_b = 1251 / 44 = 28.43$ kips/bolt

Determine the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice:

$P_{My} = (M_{TOTAL})(x) / I_p = (200)(12)(3.0 / 2) / 14737 = 0.24$ kips

$P_{Mx} = (M_{TOTAL})(y) / I_p = (200)(12)(30.19) / 14737 = 4.92$ kips

The resultant bolt force is:

$$P_r = \sqrt{(P_s + P_{Mv})^2 + (P_H + P_{Mh})^2}$$

$$P_r = \sqrt{(10.18 + 0.24)^2 + (28.43 + 4.92)^2} = 34.94 \text{ kips}$$

At the strength limit state, it is assumed that the bolts in the connection have slipped and gone into bearing. The factored shear resistance of an ASTM A325 7/8-in. diameter high-strength bolt in double shear, assuming the threads are excluded from the shear planes, was computed earlier to be:

$R_r = 55.42$ kips

$P_r = 35.36 \text{ kips} < R_r = 55.42$ kips ok

Since the bolt shear strength for both the flange and web splices is based on the assumption that the threads are excluded from the shear planes, an appropriate note should be placed on the drawings to ensure that the splice is detailed to exclude the bolt threads from the shear planes.

11.1.2 Slip Resistance: Permanent Deflection Service Limit State and for Constructability:

Determine the minimum number of bolts required to provide adequate slip resistance at the permanent deflection service limit state and also during the casting of the concrete deck (constructability check), whichever governs.

For the constructability check, the maximum shear force at the splice factored by γ_p equal to 1.25, with η taken equal to 1.0, is computed as:

$$Vu = (1.00)(1.25)|-77| = 96 \text{ kips}$$

The minimum shear resistance of the web at the splice, V_{ws}, that must be provided by the web splice plates and their connections to satisfy the permanent deflection service limit state is taken from Equation (11) given in the introduction to this splice design example as follows:

$$V_{ws} = 0.80V_{wr} = 0.80(448) = 358 \text{ kips} \quad \text{(governs)}$$

For the SERVICE II load combination, the flexural stresses at the top of the web are combined as follows:

$$(f_{ws})_{tw} = 1.00(-0.55) + 1.00(-0.06) + 1.00(-0.06) + 1.30(-1.86) = -3.09 \text{ ksi} \quad \text{(Case 1)}$$

$$(f_{ws})_{tw} = 1.00(-0.55) + 1.00(-0.13) + 1.00(-0.13) + 1.30(9.17) = 11.11 \text{ ksi} \quad \text{(Case 2)}$$

For the SERVICE II load combination at the bottom of the web:

$$(f_{ws})_{bw} = 1.00(0.43) + 1.00(0.12) + 1.00(0.12) + 1.30(10.13) = 13.84 \text{ ksi} \quad \text{(Case 1)}$$

$$(f_{ws})_{bw} = 1.00(0.43) + 1.00(0.14) + 1.00(0.13) + 1.30(-9.70) = -11.91 \text{ ksi} \quad \text{(Case 2)}$$

For the constructability check, the deck-casting flexural stress at the top of the web factored by γ_p equal to 1.25, with η taken equal to 1.0, is computed as:

$$f_{tw} = (1.0)(1.25)(-9.46) = -11.83 \text{ ksi} \quad \text{(Case 3)}$$

For the constructability check at the bottom of the web:

$$f_{bw} = (1.0)(1.25)(7.44) = 9.30 \text{ ksi} \quad \text{(Case 3)}$$

For these 3 cases, the values for f_{tw} and f_{bw} will be substituted for f_s and f_{os} in Equations (7) and (8). As discussed previously, the stresses at the inner fiber of the flanges are used in lieu of the stresses at the midthickness of the flanges for illustrative purposes, which is permitted by the Specifications:

Case 1:

$$M = \frac{0.500(69.0)^2}{12}\left|13.84-(-3.09)\right| = 3{,}358 \text{ kip}-\text{in.} = 280 \text{ kip}-\text{ft.}$$

$$H = \frac{0.5000(69)}{2}\left|13.84+(-3.09)\right| = 185 \text{ kips}$$

Case 2:

$$M = \frac{0.500(69.0)^2}{12}\left|11.11-(-11.91)\right| = 4{,}567 \text{ kip}-\text{in.} = 381 \text{ kip}-\text{ft}$$

$$H = \frac{0.500(69)}{2}\left|11.11+(-11.91)\right| = 14 \text{ kips}$$

Case 3:

$$M = \frac{0.500(69.0)^2}{12}\left|9.30-(-11.83)\right| = 4{,}192 \text{ kip}-\text{in.} = 349 \text{ kip}-\text{ft}$$

$$H = \frac{0.500(69)}{2}\left|9.30+(-11.83)\right| = 44 \text{ kips}$$

The moment due to the eccentricity of V_{ws} from the centerline of the splice to the center of gravity of the web-splice bolt group is:

$M_e = V_{wr}(e) = (358)(2.25 + 3.0/2)(1/12) = 112$ kip-ft

Therefore:

$M_{TOTAL\_CASE1} = M_{uw} + M_e = 280 + 112 = 392$ kip-ft (Case 1)

or, $M_{TOTAL\_CASE2} = M_{uw} + M_e = 381 + 112 = 493$ kip-ft (Case 2)

or, $M_{TOTAL\_CASE3} = M_{uw} + M_e = 349 + 112 = 461$ kip-ft (Case 3)

Determine the vertical bolt force due to V_{ws}:

$Ps = V_{ws} / N_b = 358 / 44 = 8.13$ kips/bolt

For Case 1:

Determine the bolt force due to the horizontal force resultant:

$P_H = H / N_b = 185 / 44 = 4.20$ kips/bolt

Determine the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice:

$P_{My} = (M_{TOTAL})(x) / I_p = (392)(12)(3.0 / 2) / 14737 = 0.48$ kips

$P_{Mx} = (M_{TOTAL})(y) / I_p = (392)(12)(30.19) / 14737 = 9.64$ kips

The resultant bolt force is:

$$P_r = \sqrt{(P_s + P_{Mv})^2 + (P_H + P_{Mh})^2}$$

$$P_r = \sqrt{(8.13 + 0.48)^2 + (4.20 + 9.64)^2} = 16.30 \text{ kips}$$

For Case 2:

Determine the bolt force due to the horizontal force resultant:

$P_H = H / N_b = 14 / 44 = 0.32$ kips/bolt

Determine the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice:

$P_{My} = (M_{TOTAL})(x) / I_p = (493)(12)(3.0 / 2) / 14737 = 0.60$ kips

$P_{Mx} = (M_{TOTAL})(y) / I_p = (493)(12)(30.19) / 14737 = 12.12$ kips

The resultant bolt force is:

$$P_r = \sqrt{(P_s + P_{Mv})^2 + (P_H + P_{Mh})^2}$$

$$P_r = \sqrt{(8.13 + 0.60)^2 + (0.32 + 12.12)^2} = 15.20 \text{ kips}$$

For Case 3:

Determine the bolt force due to the horizontal force resultant:

$P_H = H / N_b = 44 / 44 = 1.00$ kips/bolt

Determine the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice:

$P_{My} = (M_{TOTAL})(x) / I_p = (461)(12)(3.0 / 2) / 14737 = 0.56$ kips

$$P_{Mx} = (M_{TOTAL})(y) / I_p = (461)(12)(30.19) / 14737 = 11.33 \text{ kips}$$

The resultant bolt force is:

$$P_r = \sqrt{(P_s + P_{Mv})^2 + (P_H + P_{Mh})^2}$$

$$P_r = \sqrt{(8.13 + 0.56)^2 + (1.00 + 11.33)^2} = 15.08 \text{ kips}$$

The nominal slip resistance for a Class B surface and standard holes was determined previously for two shear planes to be:

R_r = 39.00 kips/bolt

16.30 kips < 39.00 kips ok

Therefore, the number of bolts detailed for the web splice (44) is adequate for slip resistance at the permanent deflection service limit state. Given that the slip resistance is significantly greater than the calculated design force in the bolts and the factored shear resistance is significantly greater than the calculated design force at the STRENGTH I load combination, it is likely that the number of bolts in the web splice could be reduced somewhat.

11.2 Shear Resistance of Splice Plates: Strength Limit State

Check the factored shear resistance of the plates at the strength limit state:

- For yielding on the gross section:

  According to Article 6.13.5.3, the factored shear resistance of the web splice plates is taken as the following for checking yielding on the gross section:

  $R_r = \phi_v R_n$

  $R_n = 0.58 A_g F_y$

  ϕ_v = 1.00 for shear (Article 6.5.4.2)

  R_r = 1.00(0.58)(0.375)(64.0)(50)(2) = 1,392 kips

  V_{wr} = 448 kips < R_r = 1,392 kips ok

- For fracture on the net section:

  For checking fracture on the net section of the web splice plates, the following relationship is used for the factored shear resistance (assume 1 in. diameter bolt holes):

$R_r = \phi_u R_n$

$R_n = 0.58F_uA_nU$

F_u = 70 ksi (Table 6.4.1-1)

ϕ_u = 0.80 (Article 6.5.4.2)

U = reduction factor to account for shear lag = 1.0 for splice plates (Article 6.13.5.2)

$A_n = 2[64.0 – 22(1.0)](0.375) = 31.50 \text{ in.}^2$

As specified in Article 6.13.5.2, A_n of the splice plates to be used in calculating the fracture strength of the splice plates cannot exceed 85 percent of the gross area of the plates = $0.85(2)(0.375)(64.0) = 40.80 \text{ in.}^2 > 31.50 \text{ in.}^2$ ok.

$R_r = 0.80(0.58)(70)(31.50)(1.0) = 1{,}023$ kips

$V_{wr} = 448 \text{ kips} < R_r = 1{,}023$ kips ok

- For block shear rupture of the web splice plates (Article 6.13.4):
-

Block shear is unlikely to govern for the web splice plates. Therefore, the block shear checks will not be illustrated in this example.

11.3 Flexural Resistance of Splice Plates: Strength Limit State

Check the maximum normal stress on the gross section of the web splice plates at the strength limit state due to the total web moment and the horizontal force resultant applied at the centroid of the web:

$A_{PL} = 2(0.375)(64.0) = 48.00 \text{ in.}^2$

$S_{PL} = (1/6)(2)(0.375)(64.0)^2 = 512.0 \text{ in.}^3$

$f = (M_{TOTAL} / S_{PL}) + (H / A_{PL})$

For Case 1: $f = (901)(12) / (512) + (500)/(48.00) = 31.53$ ksi

$31.53 \text{ ksi} < \phi_f F_y = 1.00(50) = 50.0$ ksi ok

For Case 2: $f = (200)(12) / (512) + (1251)/(48.00) = 30.75$ ksi

$30.75 \text{ ksi} < \phi_f F_y = 1.00(50) = 50.0$ ksi ok

11.4 Bearing Resistance at Bolt Holes: Strength Limit State

Check the bearing of the bolts on the connected material at the strength limit state assuming the bolts have slipped and gone into bearing. The bearing resistance of the thinner 1/2 in. web at the location of the extreme bolt in the splice is computed as the minimum resistance along the two orthogonal shear failure planes shown in Figure 19. The maximum force acting in the extreme bolt is compared to this calculated resistance, which is conservative since the components of this force parallel to the failure surfaces are smaller than the maximum force.

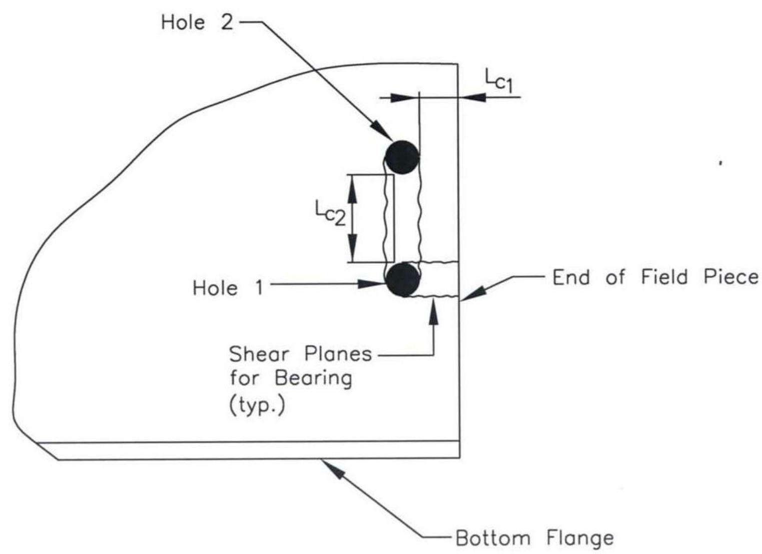

Figure 19 Sketch of the Shear Planes for Bearing Resistance

The edge distance from the center of the hole to the edge of the field piece is taken as 2-1/8 in., which exceeds the minimum edge distance of 1-1/2 in. specified for sheared or thermal cut edges (Table 6.13.2.6.3-1). Therefore, the clear distance between the edge of the hole and the edge of the field piece is computed as follows:

$L_{c1} = 2.125 - 1.0 / 2 = 1.625$ in. $< 2.0d = 2.0(0.875) = 1.75$ in.

Therefore, $R_{n\_HOLE1} = 1.2L_ctF_u = 1.2(1.625)(0.50)(70) = 68.25$ kips (governs)

Check the bearing on the web material in the vertical direction between horizontal bolt rows:

$L_{c2} = 2.875 - 1.0 = 1.875$ in. $> 2.0d = 2.0(0.875) = 1.75$ in.

Therefore, $R_{n\_HOLE2} = 2.4dtF_u = 2.4(0.875)(0.50)(70) = 73.5$ kips

From Article 6.13.2.2, the factored resistance, R_r, of the connected material in bearing shall be taken as:

$R_r = \phi_{bb} R_n$

where ϕ_{bb} is the resistance factor for bolts bearing on material (Article 6.5.4.2) equal to:

$\phi_{bb} = 0.80$

Therefore: $R_r = 0.80(68.25) = 54.60$ kips

The maximum force on the extreme bolt at the strength limit state, P_r, was computed earlier to be 35.36 kips.

$P_r = 35.36 \text{ kips} < R_r = 54.60 \text{ kips}$ ok

11.5 Fatigue Resistance of Splice Plates: Fatigue and Fracture Limit State

Check the fatigue stress in the base metal of the web splice plates (Article 6.13.6.1.4a). The nominal fatigue resistance of uncoated weathering steel base metal is determined for fatigue detail Category B (Table 6.6.1.2.3-1). The same category is used to check the base metal adjacent to slip-critical bolted connections. Since the edges of the splice plates are further from the neutral axis than the outermost bolt lines, fatigue of the base metal at the edges of the plates will be checked. The splice plates will be checked for the portion of the fatigue load moment to be resisted by the web, the horizontal force resultant of the elastic flexural fatigue-load stresses in the web, and the moment due to the eccentricity of the fatigue-load shear at the splice.

The factored moments at the splice due to the fatigue load (factored by the 0.75 load factor specified for the FATIGUE load combination) plus the dynamic load allowance of 15 percent are:

$M_{+(LL+IM)} = 679$ kip-ft

$M_{-(LL+IM)} = -515$ kip-ft

The flexural stresses at the top and bottom of the web, f_{tw} and f_{bw}, due to the fatigue-load moments are first computed. In computing the stresses, positive fatigue-load moments are assumed to be applied to the short-term composite section and negative fatigue-load moments are assumed to be applied to the section consisting of the steel girder plus the longitudinal reinforcement (since the maximum composite dead-load compressive stresses in the slab are overcome by the maximum tensile stress in the slab due to the factored negative fatigue-load moment).

The fatigue stresses at the top of the web are:

$f_{tw\_+(LL+IM)} = (679)(12)(10.7) / 161518 = 0.54$ ksi (compression)

$|f_{tw\_-(LL+IM)}| = |(-515)(12)(10.7) / 161518| = 2.58$ ksi (tension)

The fatigue stresses at the bottom of the web are:

$f_{tw\_+(LL+IM)}$ = (679)(12)(58.30) / 161518 = 2.94 ksi (tension)

$|f_{tw\_-(LL+IM)}|$ = |(-515)(12)(35.46) / 161518| = 2.73 ksi(compression)

Under the positive fatigue-load moment, the portion of the moment to be resisted by the web and the horizontal force resultant are computed from Equations (7) and (8), respectively, as (Note: absolute value signs are removed from the equations to keep track of the signs):

$$+M_w = \frac{0.500(69.0)^2}{12}[2.94-(-0.54)] = 690.3\,\text{kip}-\text{in.} = 57.53\,\text{kip}-\text{ft}$$

$$+H_w = \frac{0.500(69.0)}{2}[2.94+(-0.54)] = 41.40\text{ kips}$$

Under the negative fatigue-load moment, the portion of the moment to be resisted by the web and the horizontal force resultant are computed as:

$$-M_w = \frac{0.500(69.0)^2}{12}(-2.73-2.58) = -1{,}053\,\text{kip}-\text{in.} = -87.78\,\text{kip}-\text{ft}$$

$$-H_w = \frac{0.500(69.0)}{2}(-2.73+2.58) = -2.59\text{ kips}$$

The factored shears at the splice due to the fatigue load (factored by the specified 0.75 load factor) plus the dynamic load allowance of 15 percent are:

$V_{+(LL+IM)}$ = 5 kips

$V_{-(LL+IM)}$ = -31 kips

The moments due to the eccentricity of the factored fatigue-load shear forces from the centerline of the splice to the center of gravity of the web-splice bolt group are:

$$+M_e = +5\left(2.25+\frac{3.0}{2}\right)(1/12) = 1.56\text{ kip}-\text{ft}$$

$$-M_e = -31\left(2.25+\frac{3.0}{2}\right)(1/12) = -9.69\text{ kip}-\text{ft}$$

$+M_{TOTAL} = +M_w + M_e$ = 7.53 +1.56 = 59.09 kip-ft

$-M_{TOTAL} = -M_w + (-M_e)$ = -87.78 +(-9.69) = -97.47 kip-ft

From separate calculations, it is determined that the stress range at the bottom edge of the splice plate controls. As a reminder, fatigue at the top edge of the splice plates would only need to be checked if the compressive stress at the top of the plates due to the unfactored permanent-load moments (conservatively ignoring the future wearing surface) is less than twice the maximum tensile stress at the top of plates due to the negative factored fatigue-load moment (Article 6.6.1.2.1), which happens to be the case in this example.

The normal stresses at the bottom edge of the splice plates due to the total positive and negative fatigue-load web moments and the corresponding horizontal force resultants are as follows:

$$f = M_{TOTAL} / S_{PL} + H_w / A_{PL}$$

$$f_{+(LL+IM)} = (59.09)(12) / (512.0) + 41.40 / 48.00 = 2.25 \text{ ksi} \quad \text{(tension)}$$

$$f_{-(LL+IM)} = (-97.47)(12) / (512.0) + -2.59 / 48.00 = -2.34 \text{ ksi} \quad \text{(compression)}$$

The total fatigue-load stress range at the bottom edge of the web splice plates is therefore:

$$\gamma(\Delta f) = f_{+(LL+IM)} + |f_{-(LL+IM)}|$$

$$\gamma(\Delta f) = 2.25 + |-2.34| = 4.59 \text{ ksi}$$

From previous calculations: $(\Delta F)_n$ = 8.0 ksi for a Category B detail.

From Equation (6.6.1.2.2-1):

$$\gamma(\Delta f) \leq (\Delta F)_n$$

$$4.59 \text{ ksi} < 8.00 \text{ ksi} \qquad \text{ok}$$

11.6 Permanent Deflection Service Limit State: Splice Plates

To satisfy the permanent deflection service limit state, check the maximum normal stress on the web splice plates to control local yielding of the plates.

Calculate the maximum normal stress on the gross section of the web splice plates at the permanent deflection service limit state due to the total web moment and the corresponding horizontal force resultant applied at the centroid of the web as follows:

$$f = (M_{TOTAL} / S_{PL}) + (H / A_{PL})$$

For Case 1: $f = (392)(12) / (512) + (185)/(48.00) = 13.04$ ksi

$$13.04 \text{ ksi} < 0.95F_{yf} = 0.95(50) = 47.5 \text{ ksi} \qquad \text{ok}$$

For Case 2: $f = (493)(12) / (512) + (14)/(48.00) = 11.85$ ksi

11.85 ksi $< 0.95F_{yf} = 0.95(50) = 47.5$ ksi ok

For Case 3: $f = (461)(12) / (512) + (44)/(48.00) = 11.72$ ksi

11.72 ksi $< 0.95F_{yf} = 0.95(50) = 47.5$ ksi ok

12.0 HYBRID PLATE-GIRDER SPLICE DESIGN DIFFERENCES

In general, the basic procedures provided in the previous example are the same when designing a splice for a hybrid plate-girder. The AASHTO LRFD (5th Edition, 2010) have addressed the differences in design through the use of the hybrid factor, R_h, discussed in detail in AASHTO Articles 6.10.1.10.1 and C6.10.1.10.1.

This hybrid factor appears in the splice design equations previously presented. For rolled shapes, homogeneous built-up sections and built-up sections with a higher strength steel in the web than in both flanges, the hybrid factor is set equal to 1.0.

13.0 ROLLED BEAM SPLICE DESIGN

The procedure for the design of field splices for rolled beams is similar to the procedure illustrated in the design example previously presented for plate girders.

In the past, based on outdated code provisions which required that web splices be designed for a minimum of 75 percent of the shear capacity of the web, rolled beam web splices tended to have significantly more bolts in the web splice, a function of the stocky webs and subsequently significantly higher design shear strength.

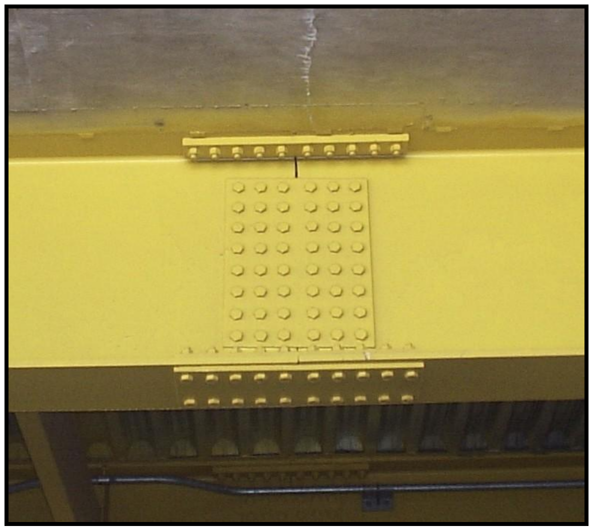

Figure 20 Photograph of a Rolled Beam Field Splice

Current design provisions, which require that the minimum shear resistance is determined based on Equations (8) and (9), should not result in overly conservative designs. However, given the stocky nature of rolled beam webs, the webs will tend to carry a larger portion of the flexural moment than plate girders, which may result in somewhat larger (relatively) web splices.

14.0 BOX GIRDER SPLICE DESIGN

The primary differences between the design of bolted field splices for plate-girders (as illustrated in the previous example) and box girders are listed below:

14.1 Web Splice Design

In the following cases, the shear used in the design of the web splices should be taken as the sum of the flexural and St. Venant torsional shears in the web which is subjected to additive shears, in accordance with Article 6.13.6.1.4b:

- All single box sections
- Multiple box sections in bridges which do not satisfy the requirements (including geometric restrictions and no skew) of Article 6.11.2.3
- Multiple box sections whose flanges are not fully effective according to the provisions of Article 6.11.1.1
- Curved box sections

It is also noted, that for box sections with inclined webs, the web splice should be designed for the component of the vertical shear in the plane of the web.

14.2 Flange Splice Design

For box girders, it is likely that for the calculation of the minimum flexural design stress for the controlling flange F_{cf} (which was previously provided in Equation (2)), taking the term $\alpha = 1.0$ may be overly conservative. Article C6.13.6.1.4c indicates that a lower value equal to (F_n/F_{yf}) may be appropriate for bottom box flanges in compression or tension, where the the calculated F_n of the flange may be significantly below F_{yf}.

In accordance with Article 6.13.6.1.4c, for the following cases, the longitudinal warping stresses due to cross-section distortion must be considered when checking bolted flange splices for slip and for fatigue. As described in Article C6.13.6.1.4c, for box sections, these longitudinal warping stresses can be significant under construction and service conditions. Once the deck has cured, and the top flange can be considered continuously braced, these warping stresses can typically be ignored. In addition, the warping stresses can be ignored in the flanges at the strength limit state. For these cases, the St. Venant torsional shear must also be considered in the design of the box girder flange bolted splices at all limit states:

- All single box sections
- Multiple box sections in bridges which do not satisfy the requirements (including geometric restrictions and no skew) of Article 6.11.2.3

- Multiple box sections whose flanges are not fully effective according to the provisions of Article 6.11.1.1

- Curved box sections

It is also noted that the lateral bending effects in discretely braced top flanges of tub sections (box girders) must be considered in the design of the bolted flange splices. These effects can be ignored for top flanges once the flange is continuously braced by the cured deck.

Figure 21 Photograph of a Box Girder Field Splice

15.0 REFERENCES

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