
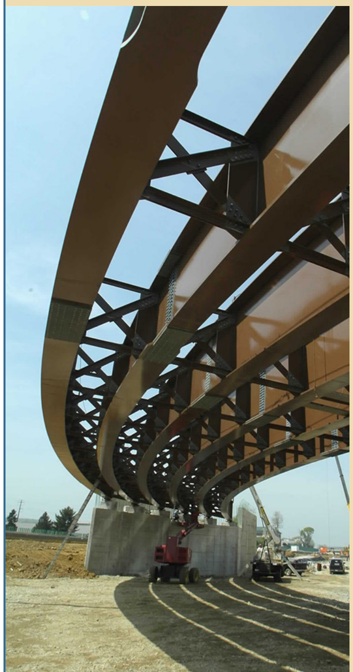

U.S. Department of Transportation
Federal Highway Administration

Steel Bridge Design Handbook

Design Example 3: Three-Span Continuous Horizontally Curved Composite Steel I-Girder Bridge

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FOREWORD

This handbook covers a full range of topics and design examples intended to provide bridge engineers with the information needed to make knowledgeable decisions regarding the selection, design, fabrication, and construction of steel bridges. Upon completion of the latest update, the handbook is based on the Seventh Edition of the AASHTO LRFD Bridge Design Specifications. The hard and competent work of the National Steel Bridge Alliance (NSBA) and prime consultant, HDR, Inc., and their sub-consultants, in producing and maintaining this handbook is gratefully acknowledged.

The topics and design examples of the handbook 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 received during the preparation of the handbook from many bridge engineering processionals across the country are very much appreciated. In particular, I would like to recognize the contributions of Bryan Kulesza with ArcelorMittal, Jeff Carlson with NSBA, Shane Beabes with AECOM, Rob Connor with Purdue University, Ryan Wisch with DeLong’s, Inc., Bob Cisneros with High Steel Structures, Inc., Mike Culmo with CME Associates, Inc., Mike Grubb with M.A. Grubb & Associates, LLC, Don White with Georgia Institute of Technology, Jamie Farris with Texas Department of Transportation, and Bill McEleney with NSBA.

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Horizontally curved steel bridges present many unique challenges. Despite their challenges, curved girder bridges have become widespread and are commonly used at locations that require complex geometries and have limited right-of-way, such as urban interchanges. Some of the important issues that differentiate curved steel girders from their straight counterparts include the effects of torsion, flange lateral bending, their inherent lack of stability, and special constructibility concerns. Also, the complex behavior of horizontally curved bridges necessitates the consideration of system behavior in the analysis.

This design example illustrates the design calculations for a curved steel I-girder bridge, considering the Strength, Service, fatigue and Constructibility Limits States in accordance with the AASHTO LRFD Bridge Designs specifications. Calculations are provided for design checks at particular girder locations, a bolted field splice design, a cross frame member design, shear connector design, and a bearing stiffener design.</td></tr>
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1.0 INTRODUCTION

Horizontally curved steel bridges present many unique challenges. Despite their challenges, curved girder bridges have become widespread and are commonly used at locations that require complex geometries and have limited right-of-way, such as urban interchanges. Some of the important issues that differentiate curved steel girders from their straight counterparts include the effects of torsion, flange lateral bending, their inherent lack of stability, and special constructibility concerns. Also, the complex behavior of horizontally curved bridges necessitates the consideration of system behavior in the analysis.

Curved steel girder bridges have been built in the United States since the 1950s. Cured-girder bridges represent a significant percentage of the total steel bridge market.

Horizontally curved girders typically offer certain advantages over kinked or chorded girders. Some of these advantages include:

- Overall simplification of the structure by allowing curved girders to follow the roadway alignment
- Use of longer spans and reduced number of intermediate permanent supports
- Continuity over several spans permitting simplified framing, efficient use of material, increased vertical clearance, and fewer joints
- Simplified forming of the deck with a constant deck overhang
- Simpler reinforcing bar schedule
- Improved aesthetics

However, horizontally curved girder bridges require special attention during design and construction. Fabrication can require additional labor or material, and shipping costs may be greater than for a straight girder. Due to torsional behavior during lifting of the girders during erection, additional lifting points and temporary supports may be required, leading to increased costs. Nevertheless, curved girder bridges are typically more economical than kinked or chorded girder bridges that are on a horizontally curved alignment.

Another unique concern of curved girder bridges is the classification of its cross frames as primary load-carrying members according to the governing design specifications. Also, flange level lateral bracing may need to be considered as primary members. As such, these elements require greater attention during bridge inspections.

Starting with the 3rd Edition, the AASHTO *LRFD Bridge Design Specifications* [1] have provided a unified design approach for both straight and horizontally curved girders within a single design specification. It should be noted that kinked (chorded) girders exhibit the same behavior as curved girders and should be treated as horizontally curved girders with respect to the AASHTO specifications.

The example calculations provided herein comply with the current AASHTO *LRFD Bridge Design Specifications* (7th Edition, 2014), but the analysis described herein was not performed as part of this design example. The analysis results and general superstructure details contained within this design example were taken from the design example published as part of the National Cooperative Highway Research Program (NCHRP) Project 12-52 published in 2005, titled "AASHTO-LRFD Design Example: Horizontally Curved Steel I-Girder Bridge, Final Report"[2].

2.0 OVERVIEW OF LRFD ARTICLE 6.10

The design of I-section flexural members is covered within Article 6.10 of the AASHTO Seventh Edition of the *LRFD Bridge Design Specifications* [1], referred to herein as *AASHTO LRFD (7^{th} Edition, 2014).* The provisions of Article 6.10 are organized to correspond to the general flow of the calculations necessary for the design of I-section flexural members. Each of the sub-articles are written such that they are largely self-contained, thus minimizing the need for reference to multiple sub-articles to address any of the essential design considerations. Many of the individual calculations and equations are streamlined, and selected resistance equations are presented in a more general format as compared to earlier LRFD Specifications (prior to the 3^{rd} Edition). The provisions of Article 6.10 are organized as follows:

6.10.1 General
6.10.2 Cross-Section Proportion Limits
6.10.3 Constructibility
6.10.4 Service Limit State
6.10.5 Fatigue and Fracture Limit State
6.10.6 Strength Limit State
6.10.7 Flexural Resistance - Composite Sections in Positive Flexure
6.10.8 Flexural Resistance - Composite Sections in Negative Flexure and Noncomposite Sections
6.10.9 Shear Resistance
6.10.10 Shear Connectors
6.10.11 Stiffeners

Section 6 also contains four appendices relevant to the design of flexural members. It should be noted that Appendices A6 and B6 are not applicable to horizontally curved I-girder bridges since they relate to straight I-sections only. The other two appendices are applicable and are as follows:

Appendix C6 - Basic Steps for Steel Bridge Superstructures
Appendix D6 - Fundamental Calculations for Flexural Members

Flow charts for flexural design of steel girders according to the provisions, along with an outline giving the basic steps for steel-bridge superstructure design, are provided in Appendix C6. Appendix C6 can be a useful reference for horizontally curved I-girder design. Fundamental calculations for flexural members are contained within Appendix D6.

General discussion of Article 6.10 is provided in Example 1 of the *Steel Bridge Design Handbook* for a straight I-girder bridge. This section will highlight several of the provisions of the *AASHTO LRFD (7^{th} Edition, 2014)* as they relate to horizontally curved I-girder design.

In the *AASHTO LRFD (7^{th} Edition, 2014),* flange lateral bending stress is included in the design checks. The provisions of Articles 6.10 provide a unified approach for consideration of major-axis bending and flange lateral bending for both straight and curved bridges. Flange lateral bending is caused by the torsional behavior of a curved bridge, resulting in cross frame forces which impart a lateral load on the flanges. Other sources of flange lateral bending are wind loads, temporary support brackets for deck overhangs, and flange level lateral bracing systems.

In addition to providing adequate strength, the constructibility provisions of Article 6.10.3 ensure that nominal yielding does not occur and that there is no reliance on post-buckling resistance for main load-carrying members during critical stages of construction. The *AASHTO LRFD (7th Edition, 2014)* specifies that for critical stages of construction, both compression and tension flanges must be investigated, and the effects of flange lateral bending should be considered when deemed necessary by the Engineer. For noncomposite flanges in compression, constructibility design checks ensure that the maximum combined stress in the flange will not exceed the minimum yield strength, that the member has sufficient strength to resist lateral torsional and flange local buckling, and that web bend-buckling will not occur. For noncomposite flanges in tension, constructibility design checks make certain that the maximum combined stress will not exceed the minimum yield strength of the flanges during construction.

3.0 DESIGN PARAMETERS

The following data apply to this design example:

Specifications:	2014 AASHTO *LRFD Bridge Design Specifications* [1], Customary U.S. Units, Seventh Edition
Structural Steel:	AASHTO M270, Grade 50 (ASTM A709, Grade 50) steel with F_y = 50 ksi, F_u = 65 ksi
Concrete:	f_c' = 4.0 ksi, γ = 150 pcf
Slab Reinforcing Steel:	AASHTO M31, Grade 60 (ASTM A615, Grade 60) with F_y = 60 ksi

The bridge has spans of 160.0 feet – 210.0 feet – 160.0 feet measured along the centerline of the bridge. Span lengths are arranged to give similar positive dead load moments in the end and center spans. The radius of the bridge is 700 feet at the centerline of the bridge. The out-to-out deck width is 40.5 feet, and there are three 12-foot traffic lanes. All supports are radial with respect to the bridge centerline. There are four I-girders in the cross section.

The total deck thickness is 9.5 inches, with a 0.5-inch integral wearing surface assumed. Therefore, the structural thickness of the concrete deck is taken as 9.0 inches. The deck haunch thickness is taken as 4.0 inches and is measured from the top of the web to the bottom of the deck. That is, the top flange thickness is included in the haunch. The width of the haunch is assumed to be 20 inches for load computation purposes. The haunch thickness is considered in section property computations, but the haunch concrete area is not considered.

Concrete railings are each assumed to weigh 495 plf. Permanent steel stay-in-place deck forms are used between the girders; the forms are assumed to weigh 15.0 psf since it is assumed concrete will be in the flutes of the deck forms. An allowance for a future wearing surface of 30.0 psf is incorporated in this design example.

The bridge is designed for HL-93 live load in accordance with Article 3.6.1.2. Live load for fatigue is taken as defined in Article 3.6.1.4. The bridge is designed for a 75-year fatigue life, and single-lane average daily truck traffic ($ADTT_{SL}$) in one direction is assumed to be 1,000 trucks per day.

The bridge site is assumed to be located in Seismic Zone 1, so seismic effects are not considered in this design example. Steel erection is not explicitly examined in this example, but sequential placement of the concrete deck is considered.

Bridge underclearance is limited such that the total bridge depth may not exceed 120 inches at the low point on the cross section. The roadway is superelevated 5 percent.

The girders in this example are composite throughout the entire span, including regions of negative flexure, since shear connectors are provided along the entire length of each girder. Shear connectors are required throughout the entire length of a curved continuous composite bridge according to the provisions of Article 6.10.10.1.

4.0 GENERAL STEEL FRAMING CONSIDERATIONS

Detailing guidelines can be found on the website for the AASHTO/NSBA Steel Bridge Collaboration, with particular attention given to the Collaboration standard entitled *Guidelines for Design Details* [3]. Three other detailing references offering guidance are the Texas Steel Quality Council's *Preferred Practices for Steel Bridge Design, Fabrication, and Erection* [4], the Mid-Atlantic States Structural Committee for Economic Fabrication (SCEF) Standards, and the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructibility* [5].

4.1 Span Arrangement

Careful consideration of the layout of the steel framing is an important part of the design process and involves evaluating alternative span arrangements and their corresponding superstructure and substructure costs in order to determine the most economical solution. Often, site-specific features will influence the span arrangement required. However, in the absence of these issues, choosing a balanced span arrangement for continuous steel bridges (end spans approximately 80% of the length of the center spans) will provide an efficient design. The span arrangement for this design example has spans of 160-210-160 feet, which is a reasonably balanced span arrangement.

4.2 Girder Spacing

When developing the bridge cross-section, the designer typically evaluates the number of girder lines required relative to the overall cost. Specifically, the total cost of the superstructure is a function of steel quantity, details, and erection costs. Developing an efficient bridge cross-section should also give consideration to providing an efficient deck design, which is generally influenced by girder spacing and overhang dimensions. Specifically, with the exception of an empirical deck design, girder spacing significantly effects the design moments in the deck slab. Larger deck overhangs result in a greater load on the exterior girder. Larger overhangs will increase the bending moment in the deck, caused by the cantilever action of the overhang, resulting in additional deck slab reinforcing for the overhang region of the deck.

In addition, wider deck spans between top flanges can become problematic for several reasons. Some owners have very economical deck details standards that may not be suited, or even permitted, for wider decks spans. At the same time, wider deck spans are progressively more difficult to form and construct.

The bridge cross-section in this design example consists of four I-girders spaced at 11 feet on center with 3.75-foot deck overhangs. The deck overhangs are 34 percent of the adjacent girder spacing. Reducing the girder spacing below 11 feet would lead to an increase in the size of the deck overhangs which would, in turn, lead to larger loading on the exterior girders, particularly the girder on the outside of the curve. Wider girder spacing would increase the deck thickness with a corresponding increase in dead load. The bridge cross-section is shown in Figure 1.

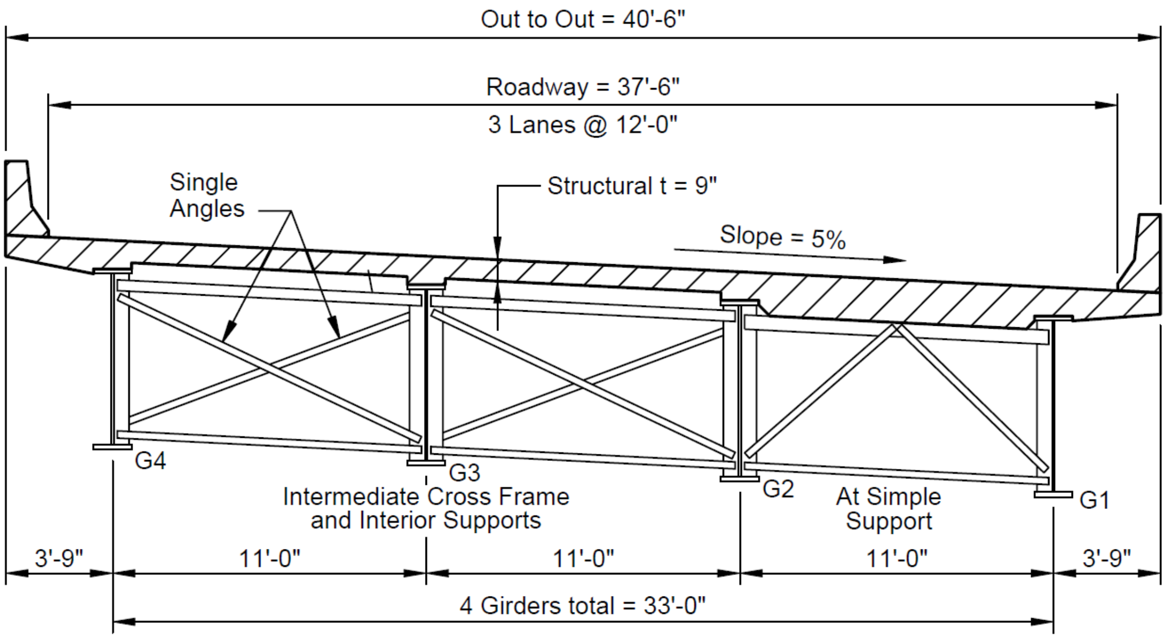


Figure 1: Typical Bridge Cross-Section

4.3 Girder Depth

Article 2.5.2.6.3 sets the maximum span-to-depth ratio, L_{as}/D, to 25 where the specified minimum yield stress is not greater than 50 ksi. In checking this requirement, the arc girder length, L_{as}, for spans continuous on both ends is defined as eighty percent of the longest girder in the span (girder length is taken as the arc length between bearings). The arc girder length of spans continuous on only one end is defined as ninety percent of the longest girder in the span. The longest arc span length (either end or interior span) controls. The maximum arc length occurs at the center span of the outside girder, G4, and is 214.95 feet. Therefore, the recommended girder depth is computed as follows:

$$0.8(214.95)/25 = 6.88 \text{ ft} = 82.5 \text{ in.}$$

Therefore, a web depth of 84 inches is selected.

4.4 Cross-Section Proportions

Proportion limits for webs of I-girders are specified in Article 6.10.2.1. Provisions for webs with and without longitudinal stiffeners are presented. For this design example, a longitudinally stiffened web is not anticipated. Therefore, the web plate must be proportioned such that the web plate thickness (t_w) meets the following requirement:

$$\frac{D}{t_w} \leq 150 \qquad \text{Eq. (6.10.2.1.1-1)}$$

Rearranging:

$$\left(t_w\right)_{min.} = \frac{D}{150} = \frac{84}{150} = 0.56\,\text{in.}$$

Based on preliminary designs, a web thickness of 0.625 inches is found to be sufficient for a transversely stiffened web and is used in the field sections over the interior piers. A 0.5625-inch thick web is used in positive-moment regions.

For illustration purposes, the proportions of girder G4 in Span 1 at the maximum positive moment location are checked. These plate sizes are applicable to the section defined later in this example as Section G4-1. The flanges are selected as follows:

Top flange (compression flange): 1.0 in. x 20 in.
Bottom flange (tension flange): 1.5 in. x 21 in.

The flanges must satisfy the provisions of Article 6.10.2.2:

$$\frac{b_f}{2t_f} \le 12.0 \qquad \text{Eq. (6.10.2.2-1)}$$

Top flange: $\frac{20}{2(1)} = 10 \le 12.0$ Bottom flange: $\frac{21}{2(1.5)} = 7 \le 12.0$ Both flanges OK

$$b_f \ge \frac{D}{6} \qquad \text{Eq. (6.10.2.2-2)}$$

$\frac{84}{6} = 14\,\text{in.}$ Both flanges OK

$$t_f \ge 1.1 t_w \qquad \text{Eq. (6.10.2.2-3)}$$

1.0 in. ≥ 1.1(0.5625) = 0.619 in. Both flanges OK

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10 \qquad \text{Eq. (6.10.2.2-4)}$$

$I_{yc} = \frac{1(20)^3}{12} = 667\,\text{in.}^4$ $I_{yt} = \frac{1.5(21)^3}{12} = 1{,}158\,\text{in.}^4$

$0.1 \le \frac{667}{1{,}158} = 0.576 \le 10$ OK

In addition to the flange proportions required by Article 6.10.2.2, Article C6.10.3.4 provides a guideline for minimizing problems during construction that arise from the fact that economical composite girders normally have smaller top flanges than bottom flanges. Such girders typically result in more than half the web depth being in compression in regions of positive flexure during deck placement. These conditions can lead to, for example, out-of-plane distortions of the compression flanges and web during construction. The relation given by Eq. (C6.10.3.4-1) should be satisfied to minimize such problems during construction, and should be used in conjunction with Eq. (6.10.2.2-2). L is taken as the length of the shipping piece, say 123 ft, which is the length of Field Section 1 of G4 as shown in Figure 3.

$$b_{fc} \geq \frac{L}{85} \qquad \text{Eq. (C6.10.3.4-1)}$$

$$20\,\text{in.} > \frac{123(12)}{85} = 17.4\,\text{in.} \quad \text{OK}$$

Therefore, all section properties for this location are satisfied. Section proportion checks for the other design locations are not shown. All subsequent sections satisfy these limits.

4.5 Cross Frames

The chosen cross frame spacing of 20 feet is within the maximum spacing allowed by Eq. (6.7.4.2-1) for horizontally curved I-girder bridges, and also is less than the prescribed maximum limit of 30 feet. Reduction of the cross frame spacing reduces cross frame forces since the load transferred between girders is a function of the curvature. Reduction of cross frame spacing also reduces flange lateral bending moments and transverse deck stresses. By reducing flange lateral bending, flange sizes can be reduced, but at the expense of requiring more cross frames. The design herein uses a spacing of approximately 20 feet measured along the centerline of the bridge.

In the analytical model used to analyze the bridge, cross frames are composed of single angles with an area of 5.0 square inches. Cross frames with an "X" configuration with top and bottom chords are used for intermediate cross frames and at interior supports. A “K” configuration is assumed at the simple end supports with the “K” pointing up (see Figure 1). The “K” configuration is advantageous at end supports because the top member, typically a channel or W shape, can support the deck edge beam. Also, as support members to the top beam at the midpoint, the diagonals help to distribute the deck load to the bearings.

Figure 2 shows the selected framing plan for this design example. Cross frames are spaced at approximately 20 feet measured along the centerline of the bridge, which results in 8 panels in the end spans and 11 panels in the center span. Critical girder sections are identified in Figure 2. These sections will be referred to frequently in the following narratives, tables, and calculations. Although not shown in Figure 2, transverse stiffeners are provided at three equal spaces between cross frame locations.

4.6 Field Section Sizes

The lengths of field sections are generally dictated by shipping weight and length restrictions. Generally, the weight of a single shipping piece is restricted to 200,000 lbs. The piece length is typically limited to a maximum of 140 feet, with an ideal piece length of 120 feet. However, shipping requirements are often dictated by state or local authorities, in which additional restrictions may be placed on piece weight and length. Handling issues during erection and in the fabrication shop also need to be considered as they may govern the length of field sections. Therefore, the Engineer should consult with contractors and fabricators regarding any specific restrictions that might influence the field section lengths.

Field section lengths should also be determined with consideration given to the number of field splices required as well as the locations of field splices. It is desirable to locate field splices as close as possible to dead load inflection points so as to reduce the forces that must be carried by the field splice. Field splices located in higher moment regions can become quite large, with cost increasing proportionally to their size. The Engineer must determine what the most cost competitive solution is for the particular span arrangement. For complex and longer span bridges, the fabricator's input can be helpful in reaching an economical solution.

The final girder field section lengths for this example are shown in the girder elevation in Figure 3. There is one field splice in each end span and two field splices in the center span, resulting in five field sections in each girder line or 20 field sections for the entire bridge. For this layout, the field sections weigh approximately 30,000 to 45,000 pounds. The longest field section, the center field section of G4, is approximately 137 feet in length. Field sections in this length and weight range can generally be fabricated, shipped, and erected without significant issues.

To verify that the shipping width is practical, the out-to-out width of the flanges taking into account the sweep should be computed. In this example, the shipping width for Field Section 3 (the center field section) of G4 taking into account the sweep is approximately 6 feet, which is reasonable for shipping.

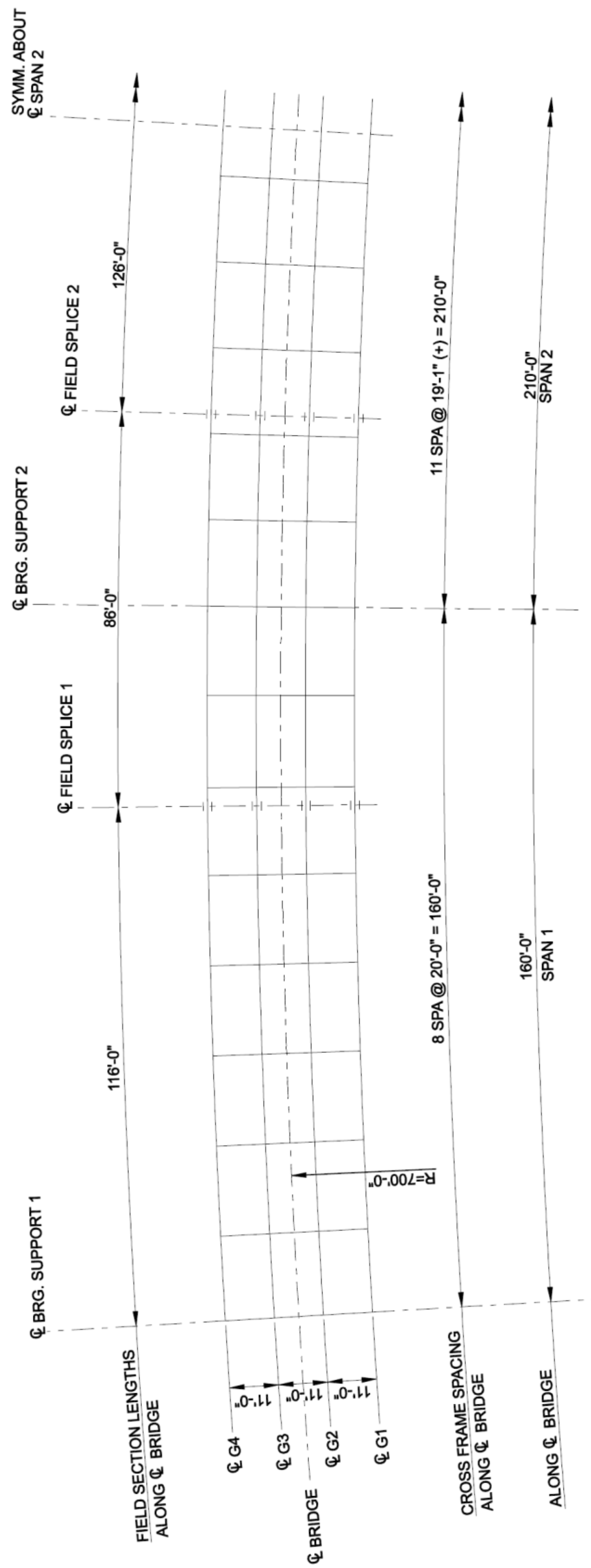
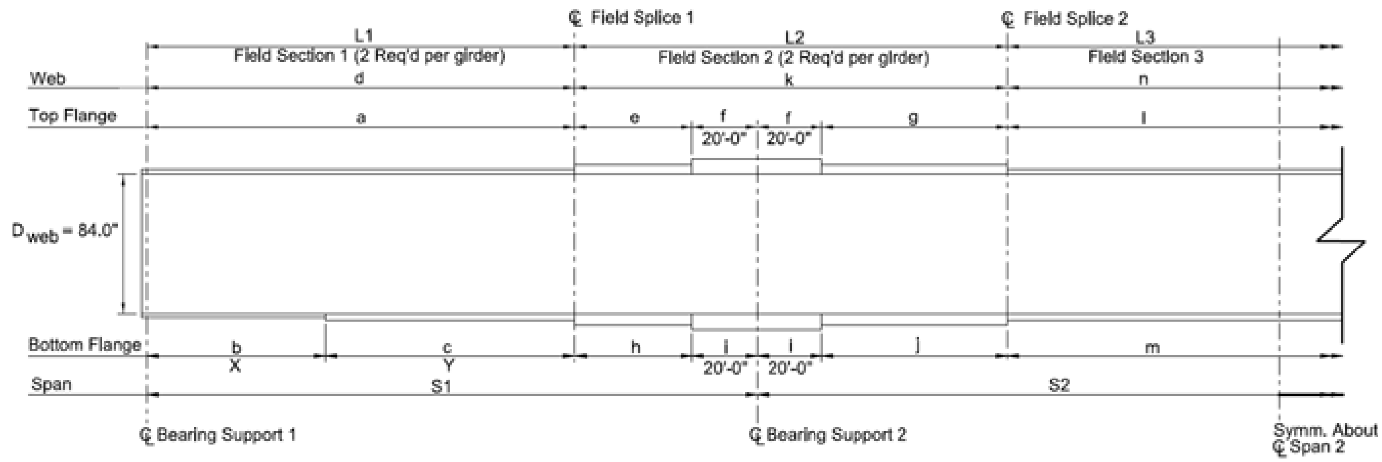


Figure 2: Framing Plan



NOTE:
Transverse stiffeners not shown for clarity.

Dimensions (Shown in feet)

	L1	L2	L3	S1	S2	X	Y
G1	113.3	84.0	123.0	156.2	205.1	0	113.3
G2	115.1	85.3	125.0	158.7	208.4	0	115.1
G3	116.9	86.7	127.0	161.3	211.7	0	116.9
G4	118.7	88.0	129.0	163.8	215.0	33.0	85.7

Member Sizes (Shown in inches)

	a	b	c	d	e	f	g	h	i	j	k	l	m	n
G1	15X1	n/a	16X1	84x9/16	21x1.25	21x2.5	21x1.25	21x1.50	21x3	21x1.50	84x5/8	15X1	18X1	84x9/16
G2	15X1	n/a	16X1	84x9/16	18x1.25	18x2.5	18x1.25	19x1.50	19x3	19x1.50	84x5/8	15X1	17X1	84x9/16
G3	16X1	n/a	18X1	84x9/16	20x1.25	20x2.5	20x1.25	21x1.50	21x3	21x1.50	84x5/8	15X1	20X1	84x9/16
G4	20X1	21X1	21X1.5	84x9/16	28x1.25	28x2.5	28x1.25	27x1.50	27x3	27x1.50	84x5/8	17X1	21X1.5	84x9/16

Figure 3: Girder Elevation

5.0 FINAL DESIGN

5.1 AASHTO LRFD Limit States

5.1.1 Service Limit State (Articles 1.3.2.2 and 6.5.2)

To satisfy the service limit state, restrictions on stress and deformation under regular conditions are specified to ensure satisfactory performance of the bridge over its service life. As specified in Article 6.10.4.1, optional live load deflection criteria and span-to-depth ratios (Article 2.5.2.6) may be invoked to control deformations.

Steel structures must also satisfy the requirements of Article 6.10.4.2 under the Service II load combination. The intent of the design checks specified in Article 6.10.4.2 is to prevent objectionable permanent deformations caused by localized yielding and potential web bend-buckling under expected severe traffic loadings, which might impair rideability. The live-load portion of the Service II load combination is intended to be the design live load specified in Article 3.6.1.1. For a permit load situation, a reduction in the specified load factor for live load under the Service II load combination should be considered for this limit state check.

5.1.2 Fatigue and Fracture Limit State (Articles 1.3.2.3 and 6.5.3)

To satisfy the fatigue and fracture limit state, restrictions on stress range under regular service conditions are specified to control crack growth under repetitive loads and to prevent fracture during the design life on the bridge (Article 6.6.1). Material toughness requirements are also addressed (Article 6.6.2).

For checking fatigue in steel structures, the fatigue load specified in Article 3.6.1.4 applies, and the Fatigue I or Fatigue II load combination is used, as applicable. Fatigue resistance of details is discussed in Article 6.6. A special fatigue requirement for webs (Article 6.10.3) is also specified to control out-of-plane flexing of the web that might potentially lead to fatigue cracking under repeated live loading.

5.1.3 Strength Limit State (Articles 1.3.2.4 and 6.5.4)

At the strength limit state, it must be ensured that adequate strength and stability are provided to resist the statistically significant load combinations the bridge is expected to experience over its design life. The applicable Strength load combinations (discussed later) are used to check the strength limit state.

Although not specified as a separate limit state, constructibility is one of the basic design objectives of LRFD. The bridge must be safely erected and have adequate strength and stability during all phases of construction. Specific design provisions are given in Article 6.10.3 of the *AASHTO LRFD (7th Edition, 2014)* to help ensure constructibility of steel I-girder bridges, particularly when subject to the specified deck-casting sequence and deck overhang force effects. The constructibility checks are typically made on the steel section only under the factored noncomposite dead loads using the appropriate strength load combinations.

5.1.4 Extreme Event Limit State (Articles 1.3.2.5 and 6.5.5)

At the extreme event limit state, structural survival of the bridge must be ensured during a major earthquake or flood, or when struck by a vessel, vehicle, or ice flow. Extreme event limit states are not covered in this design example.

5.2 Loads

5.2.1 Noncomposite Dead Load

The steel weight is applied as body forces to the fully erected noncomposite structure in the analysis. A steel density of 490 pounds per cubic foot is assumed. The entire concrete deck is assumed to be placed at one time for the strength limit state design checks.

5.2.2 Deck Placement Sequence

The deck is considered to be placed in the following sequence for the constructibility limit state design checks, which is also illustrated in Figure 4. The concrete is first cast from the left abutment to the dead load inflection point in Span 1. The concrete between dead load inflection points in Span 2 is cast second. The concrete beyond the dead load inflection point to the abutment in Span 3 is cast third. Finally, the concrete between the points of dead load contraflexure over the two piers is cast. In the analysis, earlier concrete casts are assumed composite for each subsequent cast.

For the constructibility design checks, the noncomposite section is checked for the moments resulting from the deck placement sequence or the moments computed assuming the entire deck is cast at one time, whichever is larger.

The deck load is assumed to be applied through the shear center of the interior girders in the analysis. However, the weight of the fresh concrete on the overhang brackets produces significant lateral force on the flanges of the exterior girders. This eccentric loading and subsequent lateral force on the flanges must be considered in the constructibility design checks.

5.2.3 Superimposed Dead Load

The concrete railing loads are applied along the edges of the deck elements in the three-dimensional analysis. These superimposed dead loads are applied to the composite structure in the analysis.

The superimposed dead load is considered a permanent load applied to the long-term composite section to account in an approximate fashion for the effects of concrete creep. For computing flexural stresses from permanent loading, the long-term composite section in regions of positive flexure is determined by transforming the concrete deck using a modular ratio of $3n$ (Article 6.10.1.1.1b). In regions of negative flexure, the long-term composite section is assumed to consist of the steel section plus the longitudinal reinforcement within the effective width of the concrete deck (Article 6.10.1.1.1c), except as permitted otherwise for the service and fatigue limit states (see Articles 6.6.1.2.1 and 6.10.4.2.1).

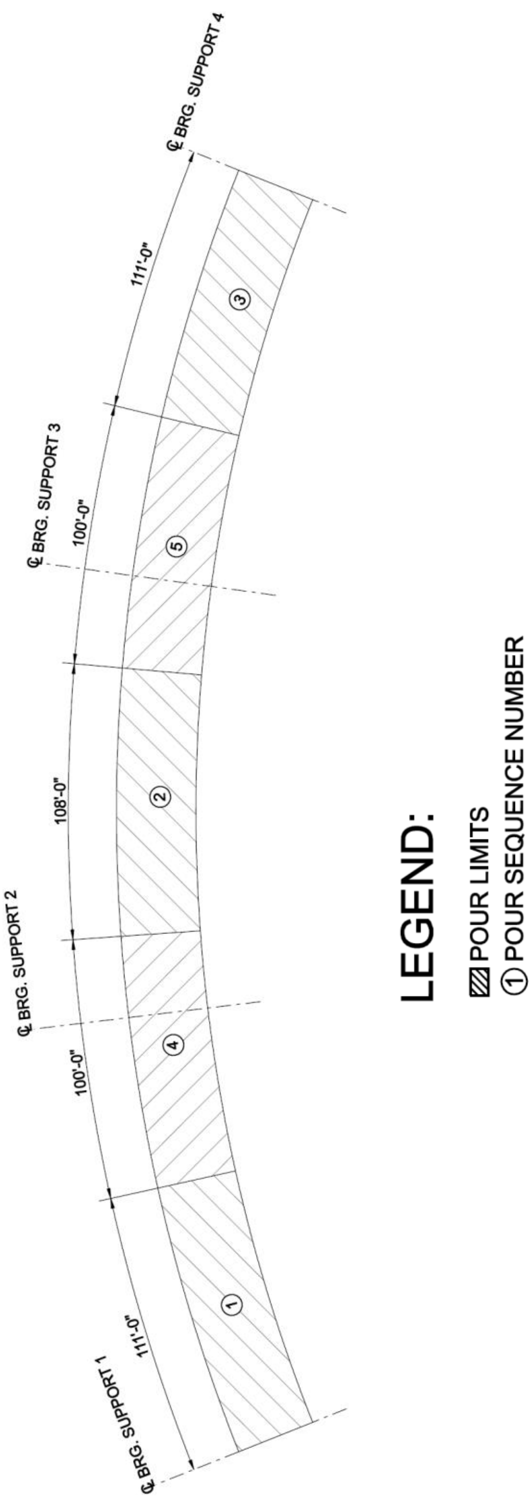


Figure 4: Deck Placement Sequence

5.2.4 Future Wearing Surface

The future wearing surface is applied uniformly over the deck area and is applied to the composite structure.

The future wearing surface is considered a permanent load applied to the long-term composite section. Flexural stresses are computed in the same manner as described previously for the superimposed dead load.

5.2.5 Live Load

Live loads are assumed to consist of gravity loads (vehicular live loads, rail transit loads and pedestrian loads), the dynamic load allowance, centrifugal forces, braking forces and vehicular collision forces. Live loads illustrated in this example include the HL-93 vehicular live load and a fatigue load, which include the appropriate dynamic load allowance and centrifugal force (see Section 5.3) effects.

Influence surfaces are utilized to determine the live load force effects in this design example. More details regarding influence surfaces and the live load analysis associated with the 3D analysis model are provided in Section 6.1.2 of this example.

Live loads are considered to be transient loads applied to the short-term composite section. For computing flexural stresses from transient loading, the short-term composite section in regions of positive flexure is determined by transforming the concrete deck using a modular ratio of n (Article 6.10.1.1.1b). In regions of negative flexure, the short-term composite section is assumed to consist of the steel section plus the longitudinal reinforcement within the effective width of the concrete deck (Article 6.10.1.1.1c), except as permitted otherwise for the fatigue and service limit states (see Articles 6.6.1.2.1 and 6.10.4.2.1).

When computing longitudinal flexural stresses in the concrete deck (see Article 6.10.1.1.1d), due to permanent and transient loads, the short-term composite section should be used.

<u>Design Vehicular Live Load (Article 3.6.1.2)</u>
The design vehicular live load is designated as the HL-93 and consists of a combination of the following placed within each design lane:

- a design truck *or* design tandem.
- a design lane load.

The design vehicular live load is discussed in detail in Example 1 of the *Steel Bridge Design Handbook.*

<u>Fatigue Load (Article 3.6.1.4)</u>
The vehicular live load for checking fatigue consists of a single design truck (without the lane load) with a constant rear-axle spacing of 30 feet (Article 3.6.1.4.1). The fatigue live load is discussed in detail in Example 1 of the *Steel Bridge Design Handbook*.

5.3 Centrifugal Force Computation

The centrifugal force is determined according to Article 3.6.3. The centrifugal force has two components, the radial force and the overturning force. The radial component of the centrifugal force is assumed to be transmitted from the deck through the support cross frames or diaphragms to the bearings and the substructure.

The overturning component of centrifugal force occurs because the radial force is applied at a distance above the top of the deck. The center of gravity of the design truck is assumed to be 6 feet above the roadway surface according to the provisions of Article 3.6.3. The transverse spacing of the wheels is 6 feet per Figure 3.6.1.2.2-1. The overturning component causes the exterior (with respect to curvature) wheel line to apply more than half the weight of the truck and the interior wheel line to apply less than half the weight of the truck by the same amount. Thus, the outside of the bridge is more heavily loaded with live load. The effect of superelevation, which reduces the overturning effect of centrifugal force, can be considered as permitted by Article 3.6.3. Figure 5 shows the geometric relationship between the centrifugal force and the superelevation. The dimensions denoted by *s* and *h* in Figure 5 are both equal to 6 feet.

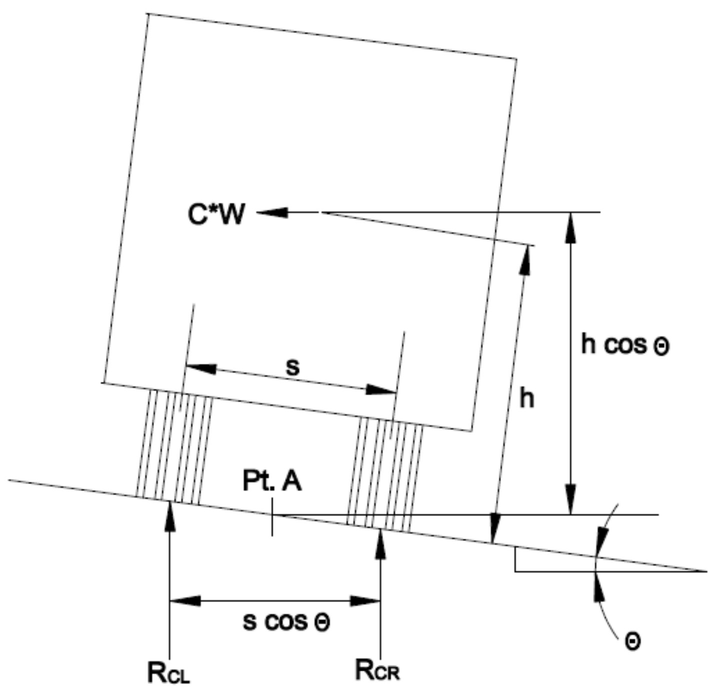


Figure 5: Vehicular Centrifugal Force Wheel-Load Reactions

Article 3.6.3 states that the centrifugal force shall be taken as the product of the axle weights of the design truck or tandem and the factor C, taken as:

$$C = f\,\frac{v^2}{gR} \qquad \text{Eq. (3.6.3-1)}$$

where: f = 4/3 for load combinations other than fatigue and 1.0 for fatigue
v = highway design speed (ft/sec)
g = gravitational acceleration: 32.2 ft/sec^2
R = radius of curvature of traffic lane (ft)

Use the average bridge radius, R = 700 ft in this case. For the purpose of this design example, the design speed is assumed to be 35 mph = 51.3 ft/sec.

$$C = \frac{4}{3}\left[\frac{51.3^2}{(32.2)(700)}\right] = 0.156$$

The factor C is applied to the axle weights. Per Figure 3.6.1.2.2-1, the total weight of the design truck axles is 72 kips.

The radial force is computed as follows:

Truck in one lane = 1.2(0.156)(72) = 13.48 kips
Truck in two lanes = 1.0(0.156)(72)(2) = 22.46 kips
Truck in three lanes = 0.85(0.156)(72)(3) = 28.64 kips

All three cases have been adjusted by the appropriate multiple presence factor given in Table 3.6.1.1.2-1. The centrifugal force due to trucks in two lanes is used since the two lanes loaded case controls for major-axis bending. The force will be applied to the deck in the radial direction. The force is resisted by the shear strength of the deck and is transferred to the bearings through the cross frames at the bearings.

The overturning force is computed by taking the sum of the moments about the inside wheel and setting the sum equal to zero. First, the location of the vehicle center of gravity is determined taking into account the 5% cross slope of the deck (see Figure 1 and Figure 5). For 5% cross slope, the angle θ is equal to:

$$\theta = \arctan\left(\frac{5}{100}\right) = 2.862^\circ$$

Referring to Figure 5 and measuring from the inside wheel, vehicle gravity acts at a horizontal distance equal to:

$$\frac{s}{2}\cos\theta - h\sin\theta = \frac{6}{2}\cos\left(2.862^\circ\right) - 6\sin\left(2.862^\circ\right) = 2.70 \text{ ft}$$

In Figure 5, the right wheel is on the inside of the curve, and its reaction is denoted as R_{CR}. The left wheel is on the outside of the curve, and its reaction is denoted as R_{CL}. Take the sum of the moments about the inside wheel:

$$W \times [2.70\text{ ft} + 0.156(6\text{ ft})] - R_{CL}(6\text{ ft}) = 0$$

where: W = axle load
R_{CL} = reaction of the outside wheel

Solve for R_{CL}:

$$R_{CL} = 0.61W$$

Compute R_{CR}, which is the force on the inside wheel:

$$R_{CR} = W(1.0 - 0.61) = 0.39W$$

The R_{CL} and R_{CR} terms were computed with respect to the axle load. Therefore, the wheel loads in each lane that are applied to the influence surfaces are adjusted by two times these factors (since there are two wheels per axle), or 1.22 applied to the outside wheel and 0.78 applied to the inside wheel of each axle. The result is that the outermost girder will receive slightly higher load and the innermost girder will receive slightly lower load. Thus, it is also necessary to compute the condition with no centrifugal force, i.e., a stationary vehicle, and select the worst case. The inside of the bridge will be more heavily loaded for the stationary vehicle case. The designer may wish to consider the effect of superelevation, particularly if the superelevation is significant, since superelevation causes an increase in the vertical wheel loads toward the inside of the bridge and an unloading of the vertical wheel loads toward the outside of the bridge.

Article 3.6.3 specifies that lane load is neglected in computing the centrifugal force since the spacing of vehicles at high speeds is assumed to be large, resulting in a low density of vehicles following and/or preceding the design truck.

5.4 Load Combinations

Table 3.4.1-1 is used to determine load combinations for strength according to Article 3.4. Strength I loading is used for design of most members for the strength limit state. However, Load Combinations Strength III and V and Service I and II from Table 3.4.1-1 are also checked for temperature and wind loadings in combination with vertical loading.

The following load combinations and load factors are typically checked in a girder design similar to this design example. In some design instances, other load cases may be critical, but for this example, these other load cases are assumed not to apply.

From Table 3.4.1-1 (minimum load factors of Table 3.4.1-2 are not considered here):

Strength I	η x [1.25(DC) + 1.5(DW) + 1.75((LL + IM) + CE + BR) + 1.2(TU)]
Strength III	η x [1.25(DC) + 1.5(DW) + 1.4(WS) + 1.2(TU)]
Strength V	η x [1.25(DC) + 1.5(DW) + 1.35((LL + IM) + CE + BR) + 0.4(WS) + 1.0(WL) + 1.2(TU)]
Service I	η x [DC + DW + (LL + IM) + CE + BR + 0.3(WS) + WL + 1.2(TU)]
Service II	η x [DC + DW + 1.3((LL + IM) + CE + BR) + 1.2(TU)]

Fatigue I η x [1.5((LL + IM) + CE)]
Fatigue II η x [0.75((LL + IM) + CE)]

where:

- η = Load modifier specified in Article 1.3.2
- DC = Dead load: components and attachments
- DW = Dead load: wearing surface and utilities
- LL = Vehicular live load
- IM = Vehicular dynamic load allowance
- CE = Vehicular centrifugal force
- WS = Wind load on structure
- WL = Wind on live load
- TU = Uniform temperature
- BR = Vehicular braking force

In addition to the above load combinations, two additional load combinations are included for the constructibility checks that are defined in Article 3.4.2 as follows:

Construction: η x [1.25(D) + 1.5(C) + 1.25(WC)]
η x [1.4(D + C)]

where:

- D = Dead load
- C = Construction loads
- WC = Wind load for construction conditions from an assumed critical direction. Magnitude of wind may be less than that used for final bridge design.

In this design example, it has been assumed that there is no equipment on the bridge during construction and the wind load on the girders is negligible. Thermal loads and vehicular braking forces are also not considered.

For the purpose of this example, it has been assumed that the Strength I load combination governs for the strength limit state, so only Strength I loads are checked in the sample calculations for the strength limit state included herein. Also, the load modifier, η, is assumed to be 1.0 throughout this example unless noted otherwise.

6.0 ANALYSIS

Article 4.4 of the *AASHTO LRFD (7^{th} Edition, 2014)* requires that the analysis be performed using a method that satisfies the requirements of equilibrium and compatibility and utilizes stress-strain relationships for the proposed materials. Article 4.6.1.2 provides additional guidelines for structures that are curved in plan. The moments, shears, and other force effects required to proportion the superstructure components are to be based on a rational analysis of the entire superstructure. Equilibrium of horizontally curved I-girders is developed by the transfer of load between the girders, thus the analysis must recognize the integrated behavior of structural components.

Furthermore, in accordance with Article 4.6.1.2, the entire superstructure, including bearings, is to be considered as an integral structural unit in the analysis. Boundary conditions should represent the articulations provided by the bearings and/or integral connections used in the design.

In most cases, small deflection elastic theory is acceptable for the analysis of horizontally curved steel girder bridges. However, curved girders, especially I-girders, are prone to deflect laterally when the girders are insufficiently braced during erection, and this behavior may not be appropriately recognized in some cases by small deflection theory.

In general, three levels of analysis exist for horizontally curved girder bridges: approximate methods of analysis, 2D (two-dimensional) methods of analysis, and 3D (three-dimensional) methods of analysis. The V-load method is an approximate analysis method that is typically used to analyze curved I-girder bridges. This method was developed based on the understanding of the distribution of forces through the curved bridge system. The two primary types of 2D analysis models are the traditional grid (or grillage) model and the plate and eccentric beam model. In a 2D grid model, the girders and cross frames are modeled using beam elements, with nodes in a single horizontal plane. In a plate and eccentric beam model, the girders and cross frames are modeled using beam elements, with nodes in a single horizontal plane, and the deck is modeled with plate elements offset a vertical distance from the steel superstructure elements. A 3D model recognizes the depth of the superstructure. In a 3D model, the girders are typically modeled using beam elements for the flanges, plate elements for the webs, and all cross frame members are modeled using truss-type elements. Two planes of nodes are typically used on each girder, one in the plane of the top flange and the second in the plane of the bottom flange. Further details regarding these methods of analysis can be found in the *Steel Bridge Design Handbook* topic on Structural Analysis.

It should be noted that when an I-girder bridge meets the requirements of Article 4.6.1.2.4b, the effects of curvature may be ignored in the analysis for determining the major-axis bending moments and shears. If the requirements of Article 4.6.1.4b are satisfied, the I-girders may be analyzed as individual straight girders with a span length equal to the arc length, but flange lateral bending effects should be considered via approximate methods, and cross frame member forces shall be determined via rational methods.

6.1 Three-Dimensional Finite Element Analysis

A three-dimensional finite element analysis is used to analyze the superstructure in this design example. The girder webs are modeled using plate elements. The top and bottom flanges are modeled with beam elements. The girder elements connect to nodes that are placed in two horizontal planes, one plane at the top flange and one plane at the bottom flange. The horizontal curvature of the girders is represented by straight elements that have small kinks at the nodes, rather than by curved elements. Nodes are placed at the top and bottom flanges along the girders at each cross frame location and typically at the third points of each cross frame bay.

The composite deck is modeled using a series of eight-node solid elements attached to the girder top flanges with beam elements, which represent the shear studs.

Bearings are modeled with dimensionless elements called "foundation elements." These dimensionless elements can provide six different stiffnesses, with three for translation and three for rotation. If a guided bearing is orientated along the tangential axis of a girder, a stiffness of zero would be assigned to the stiffness in the tangential direction. The stiffness of the bearing, and supporting structure if not explicitly modeled, would be assigned to the direction orthogonal to the tangential axis.

Cross frame members are modeled with individual truss elements connected to the nodes at the top and bottom flange of the girders.

6.1.1 Bearing Orientation

The orientation and lateral restraint of bearings affect the behavior of most girder bridges for most load conditions. This is particularly true for curved and skewed girder bridges.

In this example, the bearings at the piers are assumed fixed against translation in both the radial and tangential directions. The bearings at the abutments are assumed fixed against radial movement but free in the tangential direction. The pier stiffness in the tangential direction is considered and is simulated in the analysis by using a spring with a spring constant based on the stiffness of the pier in the tangential direction. In the radial directions, the piers and abutments are assumed perfectly rigid.

The lateral restraints resist the elastic lengthening of the girders due to bending. The result is large lateral bearing forces, which in turn cause an arching effect on the girders that reduces the apparent bending moments due to gravity loads. If the reduced moments were used in the girder design, the bearings would have to function as assumed for the life of the bridge to prevent possible overstress in the girders. To avoid this situation, the lateral bearing restraints are assumed free for the gravity load analyses used to design the girders. However, the proper bearing restraints are assumed in the analyses to determine cross frame forces and lateral bearing forces for the design of these elements.

6.1.2 Live Load Analysis

The use of live load distribution factors is typically not appropriate for horizontally curved steel I-girder bridges because these structures are best analyzed as a system. Therefore, influence surfaces are most often utilized to more accurately determine the live load force effects in curved girder bridges. Influence surfaces are an extension of influence lines, such that an influence surface not only considers the longitudinal position of the live loads but also the transverse position.

Influence surfaces provide influence ordinates over the entire deck. The influence ordinates are determined by applying a series of unit vertical loads, one at a time, at selected longitudinal and transverse positions on the bridge deck surface. The magnitude of the response for the unit vertical load is the magnitude of the ordinate of the influence surface for the particular response at the point on the deck where the load is applied. The entire influence surface is created by curve fitting between calculated ordinates. Specified live loads are then placed on the surface, mathematically, at the critical locations (maximum and minimum effects), as allowed by the governing specification. The actual live load effect is determined by multiplying the live load by the corresponding ordinate. In the case of an HL-93 truck load, a different ordinate will probably exist for each wheel load. The total HL-93 truck live load effect is the summation of all the wheel loads times their respective ordinates.

The fatigue load, which consists of a single design truck without a lane load, is analyzed in a similar manner as the HL-93 truck load.

In curved girder bridges, influence surfaces are generally needed for all force results, such as major-axis bending moments, flange lateral bending moments, girder shears, reactions, torques, deflections, cross frame forces, lateral bracing forces, etc.

Unless noted otherwise, all live load force effects in this example are computed using influence surfaces developed using the three-dimensional analysis. The dynamic load allowance (impact) is applied to the force effects in accordance with Article 3.6.2 for strength, service, and fatigue as required. Multiple presence factors are also appropriately applied to the force effects from the analysis. Also, as appropriate, centrifugal force effects are considered in the analysis by applying adjustment factors to the wheel loads as shown in Section 6.3 of this design example.

6.2 Analysis Results

This section shows the results from the three-dimensional analysis of the superstructure. Analysis results are provided for the moments and shears for all four girders. All analysis results are unfactored. The reported live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 1 Girder G1 Unfactored Shears by Tenth Point

Girder G1 Unfactored Shears									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
0	0.00	14	66	17	13	109	-31	45	-11
1	15.62	9	45	6	9	87	-21	33	-5
2	31.25	5	26	2	5	69	-27	27	-8
3	46.87	1	9	2	2	55	-36	23	-12
4	62.49	-2	-9	0	-1	43	-46	19	-16
5	78.11	-5	-29	-4	-5	34	-58	13	-20
6	93.74	-9	-49	-8	-9	27	-73	9	-27
7	109.36	-14	-70	-12	-13	25	-89	8	-33
8	124.98	-20	-98	-14	-18	22	-106	8	-37
9	140.61	-28	-127	-23	-24	20	-125	7	-41
10	156.23	-40	-159	-35	-30	12	-146	4	-48
10	0.00	41	159	35	31	148	-12	49	-4
11	20.50	25	116	22	23	124	-24	39	-7
12	41.01	17	83	11	15	104	-31	36	-9
13	61.51	10	50	8	9	83	-33	29	-9
14	82.02	4	24	4	4	66	-37	24	-12
15	102.52	0	0	0	0	51	-52	19	-19
16	123.03	-5	-25	-4	-4	41	-66	15	-24
17	143.53	-10	-51	-7	-10	33	-81	11	-29
18	164.04	-16	-80	-12	-15	29	-102	9	-36
19	184.54	-26	-119	-21	-23	25	-121	7	-40
20	205.05	-41	-160	-36	-31	12	-152	4	-51
20	0.00	40	158	35	31	154	-11	52	-4
21	15.62	28	126	24	23	121	-18	43	-5
22	31.25	20	96	16	17	107	-21	39	-5
23	46.87	14	72	10	14	91	-25	33	-8
24	62.49	9	50	7	9	75	-30	28	-11
25	78.11	6	30	4	6	62	-34	24	-15
26	93.74	1	9	1	2	48	-44	17	-19
27	109.36	-1	-8	-1	-1	38	-55	13	-23
28	124.98	-5	-26	-3	-6	31	-69	9	-27
29	140.61	-9	-45	-7	-9	24	-86	8	-33
30	156.23	-14	-66	-17	-13	29	-108	9	-45

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 2 Girder G2 Unfactored Shears by Tenth Point

Girder G2 Unfactored Shears									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
0	0.00	16	71	7	15	109	-12	41	-3
1	15.87	10	47	8	9	73	-13	23	-3
2	31.75	6	26	7	5	59	-24	19	-7
3	47.62	1	9	0	2	49	-33	15	-9
4	63.50	-2	-11	-2	-2	39	-42	12	-12
5	79.37	-6	-30	-4	-5	32	-52	12	-15
6	95.25	-10	-51	-7	-9	25	-63	9	-19
7	111.12	-15	-71	-10	-13	17	-75	5	-21
8	126.99	-21	-92	-15	-18	8	-89	1	-25
9	142.87	-28	-116	-16	-24	1	-108	0	-31
10	158.74	-37	-139	-16	-29	4	-138	1	-44
10	0.00	37	139	16	30	138	-4	44	-1
11	20.83	24	109	15	22	101	-9	28	-3
12	41.67	17	78	14	16	84	-22	20	-5
13	62.50	11	52	8	9	70	-27	20	-8
14	83.34	5	26	3	5	58	-33	16	-9
15	104.17	0	0	0	0	45	-46	12	-13
16	125.01	-6	-26	-3	-5	34	-56	11	-16
17	145.84	-11	-51	-8	-10	28	-68	8	-20
18	166.68	-17	-79	-12	-15	19	-84	5	-23
19	187.51	-26	-109	-17	-22	12	-97	4	-25
20	208.35	-37	-139	-15	-30	4	-148	1	-47
20	0.00	37	139	15	30	148	-4	47	-1
21	15.87	28	117	16	23	101	-7	31	-1
22	31.75	21	93	13	19	89	-14	27	-4
23	47.62	15	71	11	13	77	-21	23	-7
24	63.50	10	50	8	9	66	-27	20	-9
25	79.37	7	31	5	5	56	-34	17	-12
26	95.25	2	11	1	2	47	-42	13	-13
27	111.12	-1	-7	-2	-1	38	-51	11	-16
28	126.99	-6	-27	-5	-6	29	-60	8	-20
29	142.87	-10	-48	-7	-9	20	-76	5	-24
30	158.74	-16	-71	-7	-15	12	-111	3	-43

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 3 Girder G3 Unfactored Shears by Tenth Point

Girder G3 Unfactored Shears									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
0	0.00	18	78	8	16	113	-17	40	-4
1	16.13	12	53	9	10	84	-18	23	-3
2	32.25	7	29	6	6	64	-28	19	-7
3	48.38	1	8	0	1	51	-37	15	-9
4	64.50	-3	-12	-2	-2	41	-45	12	-12
5	80.63	-7	-34	-5	-6	32	-54	11	-15
6	96.75	-12	-56	-8	-9	26	-67	8	-19
7	112.88	-17	-77	-10	-15	19	-81	5	-21
8	129.01	-23	-98	-17	-18	11	-95	3	-25
9	145.13	-31	-123	-17	-25	3	-114	0	-31
10	161.26	-42	-151	-17	-31	6	-143	1	-44
10	0.00	42	150	17	32	143	-6	44	-1
11	21.16	28	114	16	24	109	-14	28	-4
12	42.33	19	84	16	15	90	-24	20	-7
13	63.49	13	56	8	11	75	-27	20	-7
14	84.66	6	28	4	4	60	-34	16	-9
15	105.82	0	0	0	0	46	-47	12	-13
16	126.99	-6	-28	-4	-5	36	-60	11	-16
17	148.15	-13	-56	-9	-11	28	-73	8	-19
18	169.32	-19	-84	-13	-17	20	-90	5	-23
19	190.48	-29	-115	-17	-25	16	-103	4	-27
20	211.65	-42	-150	-17	-31	6	-153	1	-47
20	0.00	42	151	17	31	153	-6	47	-1
21	16.13	31	124	17	25	108	-6	31	-3
22	32.25	23	99	15	20	95	-15	27	-5
23	48.38	17	77	12	15	83	-22	23	-7
24	64.50	12	55	9	10	69	-28	20	-9
25	80.63	8	35	5	7	57	-35	17	-12
26	96.75	3	13	1	3	48	-42	13	-13
27	112.88	-1	-7	-2	-1	39	-52	11	-16
28	129.01	-6	-29	-5	-5	30	-65	8	-19
29	145.13	-12	-53	-8	-11	23	-84	5	-24
30	161.26	-18	-77	-8	-16	17	-112	4	-41

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 4 Girder G4 Unfactored Shears by Tenth Point

Girder G4 Unfactored Shears									
10th Point	Span	Dead Load				LL+I		Fatigue LL+I	
	Length	$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
0	0.00	23	92	23	18	143	-37	53	-11
1	16.38	16	69	11	13	119	-33	41	-9
2	32.75	11	44	5	10	99	-33	36	-8
3	49.13	3	10	3	2	79	-42	29	-11
4	65.51	-4	-19	-2	-3	58	-58	21	-19
5	81.89	-10	-47	-7	-9	40	-77	16	-25
6	98.26	-18	-74	-13	-14	25	-96	9	-33
7	114.64	-24	-101	-18	-18	17	-114	4	-40
8	131.02	-30	-121	-20	-23	14	-132	3	-45
9	147.39	-36	-134	-26	-27	13	-148	3	-49
10	163.77	-45	-144	-36	-28	9	-159	3	-55
10	0.00	44	142	36	29	159	-9	55	-3
11	21.49	33	131	27	27	150	-24	47	-5
12	42.99	25	107	17	21	137	-26	45	-7
13	64.48	18	77	12	15	114	-30	37	-8
14	85.98	9	38	7	7	90	-41	31	-13
15	107.47	0	-1	0	0	65	-65	23	-23
16	128.97	-9	-38	-7	-7	45	-88	15	-31
17	150.46	-17	-76	-12	-15	35	-110	9	-36
18	171.96	-26	-109	-18	-21	27	-132	7	-44
19	193.45	-33	-127	-26	-25	24	-146	5	-48
20	214.95	-44	-141	-36	-29	7	-159	3	-56
20*	0.00	45	144	36	28	169	-7	60	-3
21*	16.38	36	134	28	25	140	-15	49	-3
22*	32.75	30	121	22	21	130	-15	47	-3
23*	49.13	24	101	17	19	116	-17	41	-5
24*	65.51	18	74	12	15	98	-26	35	-9
25*	81.89	10	47	8	8	81	-40	29	-16
26*	98.26	4	19	3	2	59	-57	21	-21
27*	114.64	-3	-10	-1	-4	45	-78	13	-29
28*	131.02	-11	-44	-7	-8	36	-98	8	-36
29*	147.39	-16	-69	-12	-12	30	-117	8	-43
30*	163.77	-23	-92	-23	-18	36	-142	9	-53

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

* Exact analysis results for DC1 shears in Span 3 of Girder 4 are not provided in the NCHRP example referenced by this design example. For this design example, DC1 shears in Span 3 of Girder 4 are based on Span 1 Girder 4 shears, as the bridge is symmetrical.

Table 5 Girder G1 Unfactored Major-Axis Bending Moments by Tenth Point

Girder G1 Unfactored Major-Axis Bending Moments									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
0	0.00	0	0	0	0	0	0	0	0
1	15.62	178	889	184	188	1415	-381	529	-116
2	31.25	295	1478	288	311	2409	-718	873	-200
3	46.87	351	1767	327	375	3003	-1006	1049	-252
4	62.49	348	1754	316	373	3249	-1245	1103	-291
5	78.11	284	1438	260	313	3192	-1448	1067	-327
6	93.74	156	804	161	189	2875	-1605	955	-412
7	109.36	-42	-184	6	-6	2201	-2003	741	-512
8	124.98	-322	-1553	-229	-274	1465	-2569	463	-621
9	140.61	-716	-3348	-564	-619	770	-3305	181	-764
10	156.23	-1333	-5897	-1169	-1167	883	-5274	185	-991
10	0.00	-1333	-5897	-1169	-1167	883	-5274	185	-991
11	20.50	-569	-2719	-447	-505	842	-2755	232	-624
12	41.01	-123	-648	-78	-94	1694	-1796	588	-484
13	61.51	157	709	141	176	2655	-1485	917	-369
14	82.02	331	1554	293	347	3273	-1481	1085	-329
15	102.52	384	1812	335	400	3498	-1462	1144	-360
16	123.03	323	1513	272	338	3297	-1488	1089	-327
17	143.53	159	717	150	182	2678	-1528	924	-371
18	164.04	-131	-688	-87	-103	1705	-1871	597	-497
19	184.54	-575	-2733	-433	-489	906	-2700	261	-620
20	205.05	-1302	-5781	-1124	-1130	885	-5113	180	-956
20	0.00	-1302	-5781	-1124	-1130	885	-5113	180	-956
21	15.62	-726	-3371	-560	-617	776	-3236	191	-744
22	31.25	-323	-1555	-237	-277	1464	-2544	468	-612
23	46.87	-42	-187	0	-5	2196	-1980	744	-505
24	62.49	154	797	160	187	2866	-1567	956	-405
25	78.11	283	1433	262	313	3186	-1420	1068	-323
26	93.74	347	1750	315	373	3247	-1222	1107	-284
27	109.36	350	1761	323	372	3003	-988	1052	-251
28	124.98	294	1473	282	309	2420	-706	880	-204
29	140.61	177	881	183	184	1436	-376	543	-112
30	156.23	0	0	0	0	0	0	0	0

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 6 Girder G2 Unfactored Major-Axis Bending Moments by Tenth Point

Girder G2 Unfactored Major-Axis Bending Moments									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
0	0.00	0	0	0	0	0	0	0	0
1	15.87	206	962	139	201	1210	-185	373	-43
2	31.75	340	1585	247	330	1996	-376	581	-87
3	47.62	404	1875	312	392	2444	-570	681	-132
4	63.50	397	1840	322	389	2632	-772	715	-179
5	79.37	322	1488	271	321	2582	-986	695	-228
6	95.25	177	820	149	189	2325	-1196	631	-280
7	111.12	-38	-182	-23	-17	1813	-1635	507	-335
8	126.99	-334	-1533	-247	-291	1203	-2146	331	-391
9	142.87	-733	-3262	-494	-644	605	-2683	148	-455
10	158.74	-1324	-5605	-817	-1186	556	-4053	112	-560
10	0.00	-1324	-5605	-817	-1186	556	-4053	112	-560
11	20.83	-597	-2681	-419	-526	652	-2177	167	-369
12	41.67	-143	-676	-95	-109	1351	-1347	400	-301
13	62.50	159	700	145	173	2070	-931	591	-241
14	83.34	355	1600	284	355	2505	-760	703	-184
15	104.17	416	1879	333	410	2668	-664	739	-143
16	125.01	347	1550	293	344	2521	-764	703	-185
17	145.84	162	714	139	178	2060	-927	585	-243
18	166.68	-150	-708	-106	-120	1355	-1375	396	-308
19	187.51	-602	-2690	-412	-513	688	-2142	179	-364
20	208.35	-1297	-5504	-811	-1151	552	-3942	109	-549
20	0.00	-1297	-5504	-811	-1151	552	-3942	109	-549
21	15.87	-742	-3274	-495	-640	649	-2644	164	-447
22	31.75	-336	-1539	-248	-295	1236	-2139	339	-387
23	47.62	-39	-185	-25	-14	1835	-1640	509	-332
24	63.50	176	816	148	187	2344	-1214	633	-279
25	79.37	321	1485	264	320	2600	-992	699	-228
26	95.25	395	1833	318	388	2650	-775	719	-177
27	111.12	403	1865	314	389	2458	-572	685	-131
28	126.99	338	1575	248	328	2017	-379	588	-87
29	142.87	203	950	135	196	1240	-189	383	-43
30	158.74	0	0	0	0	0	0	0	0

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 7 Girder G3 Unfactored Major-Axis Bending Moments by Tenth Point

Girder G3 Unfactored Major-Axis Bending Moments									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
0	0.00	0	0	0	0	0	0	0	0
1	16.13	248	1090	163	226	1388	-301	389	-71
2	32.25	406	1775	281	366	2296	-581	600	-133
3	48.38	478	2080	349	429	2814	-845	700	-195
4	64.50	468	2024	355	422	3038	-1105	733	-256
5	80.63	379	1622	294	345	2993	-1365	708	-316
6	96.75	206	873	156	196	2703	-1628	639	-381
7	112.88	-48	-237	-44	-20	2143	-2126	508	-452
8	129.01	-388	-1708	-292	-326	1435	-2711	339	-525
9	145.13	-842	-3570	-568	-702	727	-3254	169	-608
10	161.26	-1517	-6112	-931	-1283	750	-4594	209	-732
10	0.00	-1517	-6112	-931	-1283	750	-4594	209	-732
11	21.16	-694	-2960	-485	-578	699	-2517	173	-421
12	42.33	-183	-803	-122	-129	1454	-1560	371	-344
13	63.49	164	708	149	179	2255	-1160	541	-272
14	84.66	390	1696	307	377	2837	-1015	659	-207
15	105.82	461	2006	362	439	3026	-914	696	-160
16	126.99	380	1646	317	367	2851	-1020	657	-209
17	148.15	167	721	145	183	2259	-1165	535	-276
18	169.32	-191	-832	-134	-140	1459	-1591	368	-352
19	190.48	-700	-2965	-476	-562	727	-2461	184	-419
20	211.65	-1486	-5999	-923	-1244	733	-4458	203	-711
20	0.00	-1486	-5999	-923	-1244	733	-4458	203	-711
21	16.13	-852	-3586	-569	-698	747	-3200	183	-595
22	32.25	-389	-1714	-293	-330	1450	-2685	345	-519
23	48.38	-47	-240	-40	-27	2153	-2120	511	-448
24	64.50	206	870	155	195	2711	-1623	641	-377
25	80.63	378	1619	287	344	3002	-1360	711	-313
26	96.75	468	2017	352	420	3044	-1100	735	-253
27	112.88	476	2065	350	426	2811	-837	697	-192
28	129.01	403	1759	281	362	2299	-572	599	-132
29	145.13	244	1071	156	220	1408	-298	395	-68
30	161.26	0	0	0	0	0	0	0	0

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

Table 8 Girder G4 Unfactored Major-Axis Bending Moments by Tenth Point

Girder G4 Unfactored Major-Axis Bending Moments									
10th Point	Span Length	Dead Load				LL+I		Fatigue LL+I	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.
	(ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
0	0.00	0	0	0	0	0	0	0	0
1	16.38	328	1364	287	288	2009	-529	695	-143
2	32.75	558	2305	463	483	3570	-1059	1192	-289
3	49.13	678	2775	542	586	4636	-1582	1497	-436
4	65.51	675	2744	527	586	5134	-2076	1611	-580
5	81.89	546	2192	425	479	5084	-2546	1560	-715
6	98.26	293	1136	241	269	4575	-2966	1396	-843
7	114.64	-69	-374	-24	-32	3498	-3745	1072	-957
8	131.02	-532	-2263	-375	-411	2286	-4502	657	-1060
9	147.39	-1108	-4482	-814	-846	1135	-5092	249	-1161
10	163.77	-1917	-7272	-1537	-1478	1368	-6726	351	-1315
10	0.00	-1917	-7272	-1537	-1478	1368	-6726	351	-1315
11	21.49	-940	-3811	-675	-713	1078	-3926	280	-852
12	42.99	-277	-1151	-155	-165	2307	-2610	749	-737
13	64.48	208	881	214	257	3687	-2110	1207	-620
14	85.98	531	2224	474	537	4842	-1924	1484	-495
15	107.47	635	2658	554	629	5192	-1768	1579	-395
16	128.97	518	2173	452	526	4832	-1940	1487	-500
17	150.46	210	888	225	260	3765	-2147	1225	-631
18	171.96	-284	-1174	-163	-177	2337	-2377	767	-759
19	193.45	-945	-3805	-648	-689	1130	-3812	317	-844
20	214.95	-1871	-7126	-1474	-1432	1309	-6519	336	-1259
20*	0.00	-1871	-7126	-1474	-1432	1309	-6519	336	-1259
21*	16.38	-1108	-4482	-806	-854	1140	-4897	271	-1124
22*	32.75	-532	-2263	-381	-405	2272	-4379	665	-1032
23*	49.13	-69	-374	-24	-32	3470	-3676	1069	-937
24*	65.51	293	1136	243	267	4553	-2915	1393	-827
25*	81.89	546	2192	429	475	5070	-2505	1560	-703
26*	98.26	675	2744	529	584	5127	-2044	1612	-569
27*	114.64	678	2755	540	588	4643	-1557	1503	-428
28*	131.02	558	2305	460	486	3607	-1051	1209	-285
29*	147.39	328	1364	286	289	2054	-531	716	-144
30*	163.77	0	0	0	0	0	0	0	0

Note: Live load results include multiple presence factors, dynamic load allowance (impact), and centrifugal force effects.

*Exact analysis results for DC1 moments in Span 3 of Girder 4 are not provided in the NCHRP example referenced by this design example. For this design example, DC1 moments in Span 3 of Girder 4 are based on Span 1 Girder 4 moments, as the bridge is symmetrical.

Table 9 Selected Girder G4 Unfactored Major-Axis Bending Moments

Girder G4 Unfactored Moments Used in Example Calculations*											
Location	10th Point	Dead Load				LL+I		Fatigue LL+I		Concrete Casts	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.	#1	#2
		(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)	(kip-ft)
Section G4-1	4.2	661	2682	510	583	5125	-	1603	-603	3932	-3035
Section G4-2	10	-1917	-7272	-1537	-1478	-	-6726	351	-1315	-	-
Field Splice 2	11.8**	-382	-1585	-250	-237	2054	-2772	664	-759	-1910	-169

* Values not shown are not critical and/or are not used in the example calculations.

** Actual Field Splice 2 location is at 10th Point 12, but the values at 10th Point 11.8 are conservatively used for design.

Table 10 Selected Girder G4 Unfactored Shears by Tenth Point

Girder G4 Unfactored Shears Used in Example Calculations(1)											
Location	10th Point	Dead Load				LL+I		Fatigue LL+I		Concrete Casts	
		$DC1_{STEEL}$	$DC1_{CONC}$	DC2	DW	Pos.	Neg.	Pos.	Neg.	#1	#2
		(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)	(kip)
Section G4-1	4.2	-5	-23.8	-4	-2.9	-	-61.3	20	-20	-	-
Section G4-2	10	-45	-144	-36	-28	-	-159	3	-55	-	-
Section G4-3	0	23	92	23	19	143	-	-	-	-	-
Field Splice 2	11.8(2)	27	112	19	22	139	-	-	-	7	92

(1) Values not shown are not critical and/or are not used in the example calculations.

(2) Actual Field Splice 2 location is at 10th Point 12, but the values at 10th Point 11.8 are conservatively used for design.

7.0 DESIGN

7.1 General Design Considerations

7.1.1 Flanges

The size of curved I-girder flanges is a function of girder depth, girder radius, cross frame spacing, and minimum specified yield stress of the flange.

According to Article 6.10.6.2.3, sections in negative flexure in kinked (chorded) continuous or horizontally curved steel girder bridges are to be proportioned at the strength limit state according to the provisions specified in Article 6.10.8. That is, the sections must always be treated as slender-web sections regardless of the web slenderness meaning that the provisions of Appendix A6 may not be used. In regions of negative flexure, the bottom (compression) flange is a discretely braced compression flange and must be checked for local and lateral torsional buckling under the combined major-axis bending and flange lateral bending stress (Article 6.10.8.1.1). The top (tension) flange in regions of negative flexure is considered to be continuously braced by the composite concrete deck at the strength limit state. Continuously braced flanges in tension must be checked for yielding under only the major-axis bending stress at the strength limit state (Article 6.10.8.1.3). Any flange lateral bending stresses need not be considered once the flange is continuously braced (Article C6.10.1.6).

The smaller flange plate should be used to compute the lateral torsional buckling resistance of a discretely braced compression flange between brace points when the flange size changes within a panel, unless the transition to the smaller section is located at a distance less than or equal to 20 percent of the unbraced length from the brace point with the smaller moment in which case the flange transition may be ignored (Article 6.10.8.2.3).Otherwise, the largest major-axis bending stress within the unbraced length should be used in conjunction with the largest flange lateral bending stress and the smallest flange size within the panel to compute the nominal flange stress for checking the lateral torsional buckling resistance (Article 6.10.1.6). For checking the local buckling resistance, the major-axis bending and flange lateral bending stress at the section under consideration may be used.

According to Article 6.10.6.2.2, at the strength limit state, composite sections in positive flexure in kinked (chorded) continuous or horizontally curved steel girder bridges are to be considered as noncompact sections designed according to the requirements of Article 6.10.7.2. For noncompact sections, the nominal flexural resistance is not permitted to exceed the moment at first yield. The nominal flexural resistance in these cases is therefore more appropriately expressed in terms of the elastically computed flange stress. The major-axis bending stress in compression flanges of noncompact composite sections in positive flexure is not permitted to exceed the flange yield stress at the strength limit state. For composite sections in positive flexure, lateral bending does not need to be considered in the compression flange at the strength limit state because the flange is continuously supported by the concrete deck. The combined major-axis bending and flange lateral bending stress in tension flanges of noncompact composite sections in positive flexure is also not permitted to exceed the flange yield stress (Article 6.10.7.2.1). *AASHTO LRFD* Article 6.10.1.6 specifies that for design checks where the flexural resistance is based on yielding, the major-axis bending and flange lateral bending stresses may be determined as the stresses at the section under consideration.

For constructibility, Article 6.10.3 requires that noncomposite top flanges in regions of positive flexure be designed as discretely braced compression flanges prior to hardening of the concrete to ensure that no local or lateral torsional buckling occurs under the combined major-axis bending and flange lateral bending stresses during the deck placement, which tends to lead to the use of wider top flanges in these regions.

7.1.2 Webs

According to the *AASHTO LRFD (7th Edition, 2014)*, webs are investigated for elastic bend-buckling at the service limit state and for constructibility without consideration of post-buckling shear or bending strength. Bend-buckling must be considered for both the noncomposite and composite cases since the effective slenderness changes when the neutral axis shifts.

7.1.3 Shear Connectors

Shear connectors are to be provided throughout the entire length of the bridge in cases of curved continuous structures according to Article 6.10.10.1. The required pitch of the shear connectors is determined for fatigue and checked for strength. Three 7/8-inch diameter by 6-inch shear studs per row are assumed in the design. The fatigue strength specified in Article 6.10.10.2 is used for the design of the shear connectors.

The design longitudinal shear range in each stud is computed for a single passage of the factored fatigue truck. The analysis is made assuming that the heavy wheel of the truck is applied to both the positive and negative shear sides of the influence surfaces. This computation implicitly assumes that the truck direction is reversed. In addition to major-axis bending shear, Article 6.10.10.1.2 requires that the radial shear due to curvature or radial shear due to causes other than curvature (whichever is larger) be added vectorially to the bending shear for the fatigue check. The deck in the regions between points of dead load contraflexure is considered fully effective in computing the first moment for determining the required pitch for fatigue. This assumption requires tighter shear connector spacing in these regions than if only the longitudinal reinforcing is assumed effective, as is often done. There are several reasons the concrete is assumed effective. First, known field measurements indicate that it is effective at service loads. Second, the horizontal shear force in the deck is considered effective in the analysis and the deck must be sufficiently connected to the steel girders to be consistent with this assumption. Third, maximum shear range occurs when the truck is placed on each side of the point under consideration. Most often this produces positive bending so that the deck is in compression, even when the location is between the point of dead load contraflexure and the pier. The point of dead load contraflexure is obviously a poor indicator of positive or negative bending when moving loads are considered.

The strength check for shear connectors requires that a radial shear force due to curvature be considered. The tension force in the concrete deck in the negative-moment region is given as $0.45f_c'$ in Article 6.10.10.4.2. This value is a conservative approximation to account for the combined contribution of both the longitudinal reinforcing steel and the concrete that remains effective in tension based on its modulus of rupture. For both fatigue and strength checks, the parameters used in the equations are determined using the deck within the effective flange width.

7.1.4 Details (Stiffeners, Cross Frames, Fatigue Categories)

In this example, there are intermediate transverse web stiffeners at three even spaces between cross frame locations. Intermediate stiffeners are typically fillet welded to the web and to the compression flange. Article 6.10.11.1.1 states that single-sided stiffeners on horizontally curved girders should be attached to both flanges. In this example, the intermediate stiffeners are assumed fillet welded to the tension flange. The termination of the stiffener-to-web weld adjacent to the tension flange is typically stopped a distance of $4t_w$ from the flange-to-web weld. The base metal adjacent to the stiffener weld to the tension flange is checked for fatigue. Condition 4.1 from Table 6.6.1.2.3-1 applies, which corresponds to the base metal at the toe of transverse stiffener-to-flange fillet welds, and Category C′ is the indicated fatigue category. Where the stiffener is fillet welded to the compression flange and the flange undergoes a net tension, the flange must also be checked for Category C′. When the girder is curved, the flange lateral bending creates an additional stress at the tip of the stiffener-to-flange weld away from the web. Thus, the total stress range is computed from the sum of the lateral and major-axis bending stress ranges.

Transverse web stiffeners used as connection plates at cross frames are fillet welded to the top and bottom flange. When flanges are subjected to a net tensile stress, fatigue must be checked at these points. This detail is also Condition 4.1 from Table 6.6.1.2.3-1, so the applicable fatigue category is Category C′.

Base metal at the shear stud connector welds to the top flange must be checked for fatigue whenever the flange is subjected to a net tensile stress. Condition 9.1 from Table 6.6.1.2.3-1 relates to the base metal at stud-type shear connectors that are attached by fillet or automatic stud welding, and Category C is the indicated fatigue category.

In this design example, cross-frame angles are fillet welded to gusset plates. Condition 7.1 from Table 6.6.1.2.3-1 applies.

7.1.5 Wind Loading

7.1.5.1 Loading

Article 3.8 provides the wind loading to be used for design. Article 3.8.1 requires that various wind directions be examined in order to determine the extreme force effects in the various elements of the structure. The governing wind force on the curved bridge in this example equals the wind intensity times the projected area of the bridge; in other words, the wind is applied along the chord length. It should be noted that the total force along the chord length is less than that computed if the wind were assumed to be applied perpendicular to the bridge along the arc length. Depending on how the analysis model is set up, the wind force at each node may need to be separated into a transverse and longitudinal component. For simplicity, many designers choose to apply the wind force perpendicular to the girder at each node, which is a conservative approach.

Since there are nodes at the top and bottom of the girder, it is possible to divide the wind force between the top and bottom flange. The tributary area for the top of the windward girder equals half of the girder depth plus the height of the exposed deck and railing concrete times the average spacing to each adjacent node. The tributary area for the bottom of the girder is simply half of the girder depth times the average spacing to each adjacent node.

Since the bridge is superelevated, the girders on the inside of the curve extend below the bottom of girder G4. Each successive girder extends approximately 6 inches lower. This exposed area is included in the load computation if the wind is applied from the G4 side of the bridge. If wind is applied from the G1 side of the bridge, an additional upward projection due to superelevation is manifest in the railing on the opposite side near G4 and is used in computing the wind loading.

When the girders are being erected, wind load may be applied across the ends of the girders, which are temporarily exposed. An erection analysis is not included in this example. A wind load analysis is also not included

7.1.5.2 Construction

The need for wind bracing during each critical phase of construction must also be examined as specified by Article 4.6.2.7.3. When investigating wind loads during construction, a load factor of 1.25 is to be used for the wind load in the Strength III load combination, as specified in Article 3.4.2.1.

7.1.6 Steel Erection

Erection is one of the most significant issues pertaining to curved girder bridges. Curved I-girder bridges often require more temporary supports than a straight I-girder bridge of the same span. The temporary supports are needed to provide stability and deflection control. Erection of girders in this design example is assumed to be performed by assembling and lifting pairs of girders with the cross frames between the girders bolted into place.

The first lift is composed of two pairs of girders, G1-G2 and G3-G4, in Span 1. The positive moment sections of each pair are spliced to the corresponding pier sections before lifting. Prior to erection, each pair of girders is fit up with cross frames and the bolts are tightened. These assemblies are assumed to be accomplished while the girders are fully supported, which simulates the no-load condition that was used in the shop, so that strain due to self-weight is negligible. Each girder pair is then erected. Cross frames between girders G2 and G3 are then erected and their bolts are tightened. This procedure is repeated in Span 3. The sections in Span 2 are similarly fit up in pairs and erected. Finally, the bolts in the splices in Span 2 are installed and tightened and the cross frames between girders G2 and G3 in Span 2 are installed.

According to the provisions of Article 2.5.3, one feasible erection sequence should be defined in the contract documents when the designer has assumed a particular sequence that induces certain stresses under dead load or when the bridge is of unusual complexity. A curved girder bridge is a good candidate for including an erection sequence in the contract documents. Although it is not the responsibility of the designer to consider all potential conditions during the construction of the bridge, sufficient conditions should be considered during a study of the erection scheme to ensure that it is feasible. A detailed steel erection analysis is not included in this example.

7.1.7 Deck Placement Sequence

The deck is assumed to be placed in four casts. The first cast is in Span 1 commencing at the abutment and ending at the point of dead load contraflexure. The second cast is in Span 2 between points of dead load contraflexure. The third cast is in Span 3 from the point of dead load contraflexure to the abutment. The fourth cast is over both piers. The deck placement sequence is illustrated in Figure 4.

The unfactored moments from the deck staging analysis are presented in Table 9. $DC1_{STEEL}$ moments are due to the steel weight based on the assumption that it was placed at one time. $DC1_{CONC}$ moments are due to the deck weight assumed to be placed on the bridge at one time. The concrete cast moments are due to the particular deck cast. DC2 and DW are superimposed dead loads placed on the fully composite bridge. Included in the DC2 and concrete cast moments are the moments due to the deck haunch and the stay-in-place forms. Reactions are accumulated sequentially in the analysis so that uplift can be checked at each stage. Accumulated deflections by stage are also computed.

In each analysis stage of the deck placement, prior casts are assumed to be composite. The modular ratio for the deck is assumed to be *3n* to account for creep. A somewhat smaller modular ratio may be desirable for the staging analyses since full creep usually takes approximately three years to occur. A modular ratio of *n* should be used to check the deck stresses.

7.2 Section Properties

The calculation of the section properties for Sections G4-1 and G4-2 is illustrated in this section. In computing the composite section properties, the structural slab thickness, or total thickness minus the thickness of the integral wearing surface, should be used. In this example, the total slab thickness is 9.5 inches with a 0.5-inch integral wearing surface; therefore, the structural thickness of the deck slab is 9.0 inches.

For all section property calculations, the haunch depth of 4.0 inches is considered in computing the section properties, but the area of the haunch concrete is not included. Since the actual depth of the haunch concrete may vary from its theoretical value to account for construction tolerances, many designers ignore the haunch concrete depth in all calculations. For composite section properties including only longitudinal reinforcement, a haunch depth is considered when determining the vertical position of the reinforcement relative to the steel girder. For this example, the longitudinal reinforcement steel area is assumed to be equal to 8.0 in.^{2} per girder and is assumed to be placed 4.0 inches from the bottom of the deck.

The composite section must consist of the steel section and the transformed area of the effective width of the concrete deck. Therefore, compute the modular ratio *n* (Article 6.10.1.1.1b):

$$n = \frac{E}{E_c} \qquad \text{Eq. (6.10.1.1.1b-1)}$$

where E_c is the modulus of elasticity of the concrete determined as specified in Article 5.4.2.4. A unit weight of 0.150 kcf is used for the concrete in the calculation of the modular ratio, which is more conservative than the value given in Table 3.5.1-1 since it includes an additional 0.005 kcf to account for the weight of the reinforcement.

$$E_c = 33{,}000\, K_1\, w_c^{1.5}\sqrt{f'_c} \qquad \text{Eq. (5.4.2.4-1)}$$

$$E_c = 33{,}000\,(1.0)\,(0.150)^{1.5}\sqrt{4.0} = 3{,}834\ \text{ksi}$$

$$n = \frac{29{,}000}{3{,}834} = 7.56$$

Even though Article C6.10.1.1.1b permits *n* to be taken as 8 for concrete with f_c' equal to 4.0 ksi, *n* = 7.56 will be used in all subsequent computations in this design example.

7.2.1 Section G4-1 Properties – Span 1 Positive Moment

Section G4-1 is located near the mid-span of Span 1 and is as shown in Figure 6. For this section, the longitudinal reinforcement is conservatively neglected in computing the composite section properties as is typically assumed in design.

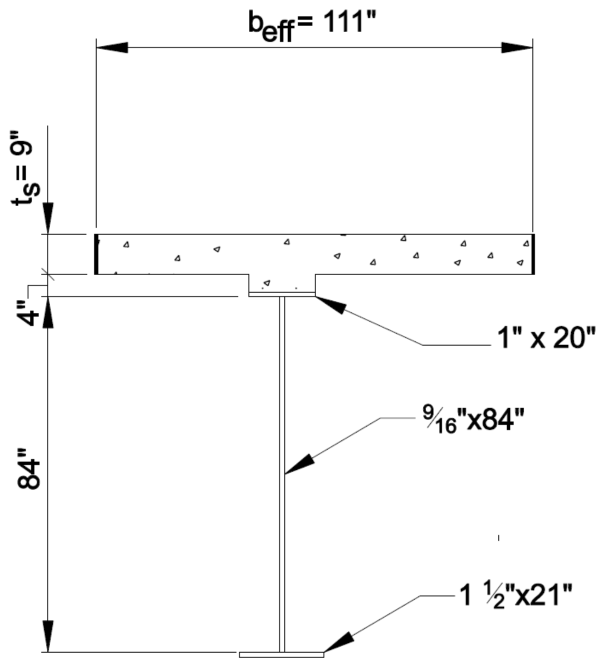


Figure 6: Sketch of I-girder Cross-Section at Section G4-1

7.2.1.1 Effective Width of Concrete Deck

As specified in Article 6.10.1.1.1e, the effective flange width is to be determined as specified in Article 4.6.2.6. According to Article 4.6.2.6, the deck slab effective width for an interior composite girder may be taken as one-half the distance to the adjacent girder on each side of the component; and for an exterior girder it may be taken as one-half the distance to the adjacent girder plus the full overhang width. Therefore, the deck slab effective width, b_{eff}, for girder G4 is:

$$b_{eff} = \frac{11.0}{2} + 3.75 = 9.25\,\text{ft} = 111\,\text{in.}$$

7.2.1.2 Elastic Section Properties: Section G4-1

In the calculation of the section properties that follow in Table 11 to Table 13, *d* is measured vertically from a horizontal axis through the mid-depth of the web to the centroid of each element of the I-girder. Section properties are calculated for the noncomposite (steel only) section, composite section using *3n*, and composite section using *n*.

Table 11 Section G4-1: Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange (1" x 20")	20.00	42.50	850	36,125	1.67	36,127
Web (9/16" x 84")	47.25	0.00			27,783	27,783
Bottom Flange (1.5" x 21")	31.50	-42.75	-1,347	57,568	5.91	57,574
	98.75		-497			121,484

$$(d_s)\Sigma(Ad) = -(-5.03)(-497) = -2{,}500$$

$$I_{NA} = 118{,}984\ \text{in.}^4$$

$$d_s = \frac{-497}{98.75} = -5.03\ \text{in.}$$

$$d_{TOP\,OF\,STEEL} = 1.00 + \frac{84}{2} - (-5.03) = 48.03\ \text{in.} \qquad d_{BOT\,OF\,STEEL} = 1.50 + \frac{84}{2} + (-5.03) = 38.47\ \text{in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{118{,}984}{48.03} = 2{,}477\ \text{in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{118{,}984}{38.47} = 3{,}093\ \text{in.}^3$$

Table 12 Section G4-1: 3*n*=22.68 Long-term Composite Section Properties

Component	A	d	Ad	Ad²	Io	I
Steel Section	98.75		-497			121,484
Concrete Slab (9" x 111")/22.68	44.05	50.50	2,225	112,339	297	112,636
	142.80		1,728			234,120

$$(d_{3n})\Sigma(Ad) = -(12.10)(1{,}728) = -20{,}909$$

$$I_{NA} = 213{,}211 \text{ in.}^4$$

$$d_{3n} = \frac{1{,}728}{142.80} = 12.10 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 1.00 + \frac{84}{2} - 12.10 = 30.90 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 1.50 + \frac{84}{2} + 12.10 = 55.60 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{213{,}211}{30.90} = 6{,}900 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{213{,}211}{55.60} = 3{,}835 \text{ in.}^3$$

Table 13 Section G4-1: *n*=7.56 Short-term Composite Section Properties

Component	A	d	Ad	Ad²	Io	I
Steel Section	98.75		-497			121,484
Concrete Slab (9" x 111")/7.56	132.14	50.50	6,673	336,990	892	337,882
	230.89		6,176			459,366

$$(d_n)\Sigma(Ad) = -(26.75)(6{,}176) = -165{,}208$$

$$I_{NA} = 294{,}158 \text{ in.}^4$$

$$d_n = \frac{6{,}176}{230.89} = 26.75 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 1.00 + \frac{84}{2} - 26.75 = 16.25 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 1.50 + \frac{84}{2} + 26.75 = 70.25 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{294{,}158}{16.25} = 18{,}102 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{294{,}158}{70.25} = 4{,}187 \text{ in.}^3$$

7.2.1.3 Plastic Moment Neutral Axis: Section G4-1

Per Article 6.10.6.2.2 for sections in positive flexure, the ductility requirements of Article 6.10.7.3 must be satisfied for compact and noncompact sections, to protect the concrete deck from premature crushing. This requires the computation of the plastic neutral axis, in accordance with Article D6.1. The longitudinal deck reinforcement is conservatively neglected. The location of the plastic neutral axis for the I-girder is computed as follows:

$P_t = F_{yt}\, b_t\, t_t$ = (50)(21.0)(1.5) = 1,575 kips
$P_w = F_{yw}\, D\, t_w$ = (50)(84.0)(0.5625) = 2,363 kips
$P_c = F_{yc}\, b_c\, t_c$ = (50)(20.0)(1.0) = 1,000 kips
$P_s = 0.85\, f_c'\, b_{eff}\, t_s$ = (0.85)(4.0)(111)(9.0) = 3,397 kips
$P_{rb} = P_{rt} = 0$ kips

$P_t + P_w + P_c > P_s + P_{rb} + P_{rt}$
1,575 + 2,363 + 1,000 = 4,938 kips > 3,397 kips

Therefore, the plastic neutral axis (PNA) is in the top flange, per Case II of Table D6-1. Compute the PNA in accordance with Case II:

$$\bar{Y} = \frac{t_c}{2}\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$$

$$\bar{Y} = \frac{1.0}{2}\left[\frac{2{,}363 + 1{,}575 - 3{,}397 - 0 - 0}{1{,}000} + 1\right]$$

$$\bar{Y} = 0.77 \text{ in. downward from the top of the top flange (PNA location)}$$

7.2.2 Section G4-2 Properties – Support 2 Negative Moment

Section G4-2 is located at Support 2 and is as shown in Figure 7.

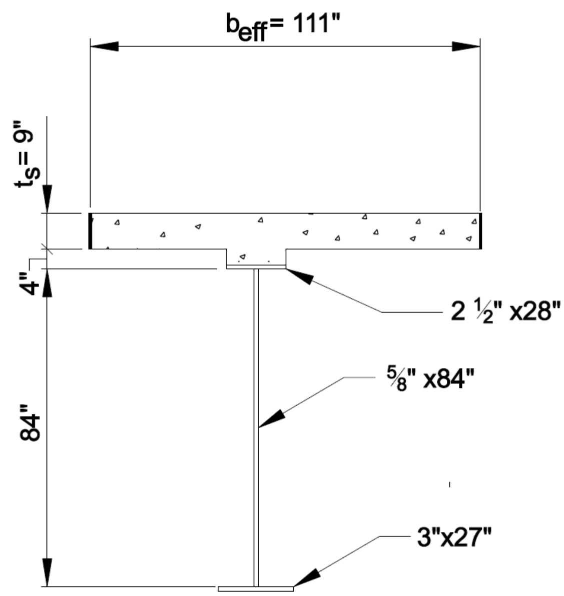


Figure 7: Sketch of I-girder cross-section at Section G4-2

The effective width of concrete deck is the same for Section G4-2 as calculated for Section G4-1, b_{eff} = 111 in.

7.2.2.1 Elastic Section Properties: Section G4-2

For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, Articles 6.6.1.2.1 and 6.10.4.2.1 permit the concrete deck to be considered effective for negative flexure when computing stress ranges and flexural stresses acting on the composite section at the fatigue and service limit states, respectively. Therefore, section properties for the long-term (3*n*) and short-term (*n*) composite section, including the concrete deck, are determined in Table 15 and Table 16, respectively, for later use in the calculations for Section G4-2 at these limits states. Longitudinal reinforcement could have been included in these section property calculations but was ignored due to its minimal effect on the moment of inertia. The concrete deck should not be considered effective for negative flexure at the strength limit state. For this scenario, longitudinal reinforcement but not the concrete is used to compute the section properties as shown in Table 17 and Table 18.

Although not required by the *AASHTO LRFD (7th Edition, 2014)*, for stress calculations involving the application of long-term loads to the composite section in regions of negative flexure, the area of the longitudinal reinforcement is conservatively adjusted in this example for the effects of concrete creep. Creep effects are accounted for by dividing the area of longitudinal reinforcement by 3 (i.e. 8.00 in.^{2}/3 = 2.67 in.^{2}) as shown in Table 17 for the long-term (3*n*) composite section properties of the steel section with longitudinal reinforcement. The concrete is assumed to transfer the force from the longitudinal deck reinforcement to the rest of the cross-section, and concrete creep acts to reduce that force over time. However, the short-term (*n*) composite section properties, as shown in Table 18, consider the full area of longitudinal reinforcement. The concrete is assumed to be cracked in both Table 17 and Table 18 and therefore is not included. The centroid of the longitudinal steel reinforcement is assumed to be located 4.0 inches from the bottom of the deck slab.

In the calculation of the section properties that follow in Table 14 to Table 18, *d* is measured vertically from a horizontal axis through the mid-depth of the web to the centroid of each element of the I-girder.

Table 14 Section G4-2: Steel Only Section Properties

Component	A	d	Ad	Ad^2	I_o	I
Top Flange (2.5" x 28")	70.00	43.25	3,028	130,939	36.46	130,975
Web (5/8" x 84")	52.50	0.00			30,870	30,870
Bottom Flange (3.0" x 27")	81.00	-43.50	-3,524	153,272	60.75	153,333
	203.50		-496			315,178

$$-(-2.44)(-496) = -1{,}210$$

$$I_{NA} = 313{,}968 \text{ in.}^4$$

$$d_s = \frac{-496}{203.50} = -2.44 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 2.50 + \frac{84}{2} - (-2.44) = 46.94 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 3.0 + \frac{84}{2} + (-2.44) = 42.56 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{313{,}968}{46.94} = 6{,}689 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{313{,}968}{42.56} = 7{,}377 \text{ in.}^3$$

Table 15 Section G4-2: 3*n*=22.68 Composite Section Properties with Transformed Deck

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	203.50		-496			315,178
Concrete Slab (9" x 111")/22.68	44.05	50.50	2,225	112,339	297	112,636
	247.55		1,729			427,814
					$-6.98(1{,}729) =$	-12,068
					$I_{NA} =$	415,746 in.^{4}

$$d_{3n} = \frac{1{,}729}{247.55} = 6.98 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 2.50 + \frac{84}{2} - 6.98 = 37.52 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 3.0 + \frac{84}{2} + 6.98 = 51.98 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{415{,}746}{37.52} = 11{,}081 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{415{,}746}{51.98} = 7{,}998 \text{ in.}^3$$

Table 16 Section G4-2: *n*=7.56 Composite Section Properties with Transformed Deck

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	203.50		-496			315,178
Concrete Slab (9" x 111")/7.56	132.14	50.50	6,673	336,990	892	337,882
	335.64		6,177			653,060
					$-18.40(6{,}177) =$	-113,657
					$I_{NA} =$	539,403 in.^{4}

$$d_n = \frac{6{,}177}{335.64} = 18.40 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 2.50 + \frac{84}{2} - 18.40 = 26.10 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 3.0 + \frac{84}{2} + 18.40 = 63.40 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{539{,}403}{26.10} = 20{,}667 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{539{,}403}{63.40} = 8{,}508 \text{ in.}^3$$

Table 17 Section G4-2: Long-term (3*n*) Composite Section Properties with Longitudinal Steel Reinforcement

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	203.50		-496			315,178
Longitudinal Reinforcement	2.67	50.00	134	6,675		6,675
	206.17		-362			321,853
				$-(-1.76)(-362) =$		-637
					$I_{NA} =$	321,216 in.^{4}

$$d_{3n} = \frac{-362}{206.17} = -1.76 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 2.50 + \frac{84}{2} - (-1.76) = 46.26 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 3.0 + \frac{84}{2} + (-1.76) = 43.24 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{321{,}216}{46.26} = 6{,}944 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{321{,}216}{43.24} = 7{,}429 \text{ in.}^3$$

Table 18 Section G4-2: Short-term (*n*) Composite Section Properties with Longitudinal Steel Reinforcement

Component	A	d	Ad	Ad^2	I_o	I
Steel Section	203.50		-496			315,178
Longitudinal Reinforcement	8.00	50.00	400	20,000		20,000
	211.50		-96			335,178
				$-(-0.45)(-96) =$		-43
					$I_{NA} =$	335,135 in.^{4}

$$d_n = \frac{-96}{211.50} = -0.45 \text{ in.}$$

$$d_{TOP\,OF\,STEEL} = 2.50 + \frac{84}{2} - (-0.45) = 44.95 \text{ in.} \qquad d_{BOT\,OF\,STEEL} = 3.0 + \frac{84}{2} + (-0.45) = 44.55 \text{ in.}$$

$$S_{TOP\,OF\,STEEL} = \frac{335{,}135}{44.95} = 7{,}146 \text{ in.}^3 \qquad S_{BOT\,OF\,STEEL} = \frac{335{,}135}{44.55} = 7{,}523 \text{ in.}^3$$

7.2.3 Check of Minimum Negative Flexure Concrete Deck Reinforcement

To control concrete deck cracking in regions of negative flexure, Article 6.10.1.7 specifies that the total cross-sectional area of the longitudinal reinforcement must not be less than 1 percent of the total cross-sectional area of the deck. The minimum longitudinal reinforcement must be provided wherever the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service II exceeds ϕf_r. ϕ is to be taken as 0.9 and f_r is to be taken as the modulus of rupture of the concrete determined as follows:

- For normal weight concrete: $f_r = 0.24\sqrt{f'_c}$
- For lightweight concrete: f_r is calculated as specified in Article 5.4.2.6.

It is further specified that the reinforcement is to have a specified minimum yield strength not less than 60 ksi and a size that should not exceed No. 6 bars. The reinforcement should be placed in two layers uniformly distributed across the deck width, and two-thirds should be placed in the top layer. The individual bars should be spaced at intervals not exceeding 12 inches.

Article 6.10.1.1.1c states that for calculating stresses in composite sections subjected to negative flexure at the strength limit state, the composite section for both short-term and long-term moments is to consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck. Referring to the cross-section shown in Figure 1:

A_{deck} = (entire width of 9" thick deck) + (triangular portion of overhang)

$$A_{deck} = \frac{9.0}{12}(40.5) + 2\left[\frac{1}{2}\left(\frac{4.0}{12}\right)\left(3.75 - \frac{28/2}{12}\right)\right] = 31.24\text{ ft}^2 = 4{,}498\text{ in.}^2$$

$$0.01(4{,}498) = 44.98\text{ in.}^2$$

$$\frac{44.98}{40.5} = 1.11\text{ in.}^2/\text{ft} = 0.093\text{ in.}^2/\text{in.}$$

$$0.093(111) = 10.32\text{ in.}^2 \text{ per exterior girder}$$

Therefore, the assumption of 8.00 in.^{2} of longitudinal deck reinforcement is conservative for the purpose of section property calculations and is left as shown in Table 17 and Table 18. When the reinforcement is detailed, #6 bars at 6 inches placed in the top layer and #4 bars spaced at 6" in the bottom layer could be specified. Therefore, the total area of deck reinforcement steel in the given effective width of concrete deck would be:

$$A_S = (0.44 + 0.44 + 0.20 + 0.20)\left(\frac{111}{12}\right) = 11.84\text{ in.}^2 > 10.32\text{ in.}^2$$

Also, approximately two-thirds of the reinforcement is in the top layer: $\frac{0.44 + 0.44}{1.28} = 0.69 \approx \frac{2}{3}$.

The use of the longitudinal reinforcement computed above is also addressed within the deck constructibility checks shown later in this design example. It should be noted that the area of longitudinal reinforcement shown above is required in the "positive moment region" and even at the location of maximum positive moment in the case of this example because of the presence of negative moment at these locations during the placement of the deck.

7.3 Girder Check: Section G4-3, Shear at End Support (Article 6.10.9)

According to the provisions of Article 6.10.9.1, at the strength limit state, straight and curved web panels are to satisfy:

$$V_u \le \phi_v V_n$$ Eq. (6.10.9.1-1)

where:

ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively
V_u = factored shear in the web at the section under consideration

Since the web at Support 1 is an end panel, Article 6.10.9.3.3 applies, and the nominal shear resistance is to be taken as:

$$V_n = V_{cr} = CV_p$$ Eq. (6.10.9.3.3-1)

where: C = ratio of the shear-buckling resistance to the shear yield strength
V_{cr} = shear-buckling resistance
V_p = plastic shear force

7.3.1 Applied Shear

The unfactored shears for G4 at Support 1 are shown below. These results are directly from the three-dimensional analysis as reported in Table 10.

Steel Dead Load:	$V_{DC1\text{-}STEEL}$	= 23 kips
Concrete Deck Dead Load:	$V_{DC1\text{-}CONC}$	= 92 kips
Composite Dead Load:	V_{DC2}	= 23 kips
Future Wearing Surface Dead Load:	V_{DW}	= 19 kips
Live Load (including IM + CF):	V_{LL+IM}	= 143 kips

The maximum Strength I factored shear is computed as:

$$V_u = 1.25(23 + 92 + 23) + 1.50(19) + 1.75(143) = 451\,\text{kips}$$

7.3.2 Shear Resistance

Compute the plastic shear force:

$$V_p = 0.58F_{yw}Dt_w$$ Eq. (6.10.9.3.3-2)

$$= 0.58(50)(84)(0.5625) = 1{,}370\ \text{kips}$$

To determine the ratio C, the shear-buckling coefficient must first be computed as follows:

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

At this particular location, the transverse stiffener spacing is assumed to be 82 inches. Therefore, d_o = 82 in.

$$k = 5 + \frac{5}{\left[\frac{82}{84}\right]^2} = 10.2$$

Check the following relation in order to select the appropriate equation for computing C:

$$\frac{D}{t_w} = \frac{84}{0.5625} = 149.3 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(10.2)}{50}} = 108$$

Since the above relation is true, the ratio C is computed using Eq. (6.10.9.3.2-6) as follows:

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

$$C = \frac{1.57}{\left(\frac{84}{0.5625}\right)^2}\left(\frac{29{,}000(10.2)}{50}\right) = 0.416$$

The nominal shear resistance is then computed in accordance with Eq. (6.10.9.3.3-1):

$$V_n = V_{cr} = (0.416)(1{,}370) = 570 \text{ kips}$$

Using the above results, check the requirement of Article 6.10.9.1, $V_u \le \phi_v V_n$:

$$V_u = 451 \text{ kips} \le \phi_v V_n = (1.0)(570) = 570 \text{ kips} \quad \text{OK (Ratio = 0.791)}$$

Therefore, the web is satisfactory for shear at Support 1. It should be noted that the sample calculation shown above is for a web end panel, but for interior web panels, the provisions of Article 6.10.9.3.2 apply.

7.4 Girder Check: Section G4-1, Constructibility (Article 6.10.3)

For critical stages of construction, the provisions of Articles 6.10.3.2.1 through 6.10.3.2.3 are to be applied to the flanges of the girder. However, in many cases, such as in this design example, 6.10.3.2.3 does not apply since neither flange is continuously braced during construction. Web shear is to be checked in accordance with Article 6.10.3.3.

As specified in Article 6.10.3.4, sections in positive flexure that are composite in the final condition, but noncomposite during construction, are to be investigated during the various stages of deck placement. The effects of forces from deck overhang brackets acting on the fascia girders are also to be considered. Wind load effects on the noncomposite structure prior to casting are also an important consideration during construction. The presence of construction equipment may also need to be considered. Lastly, potential uplift at bearings should be investigated at each critical construction stage. For this design example, the effects of wind load on the structure and the presence of construction equipment are not considered.

Calculate the maximum flexural stresses in the flanges of the steel section due to the factored loads resulting from the application of steel self-weight and Cast #1 of the deck placement sequence. Cast #1 yields the maximum positive moment for the noncomposite Section G4-1. As specified in Article 6.10.1.6, for design checks where the flexural resistance is based on lateral torsional buckling, f_{bu} is to be determined as the largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending. For design checks where the flexural resistance is based on yielding, flange local buckling or web bend-buckling, f_{bu} may be determined as the corresponding stress values at the section under consideration. From Figure 2, brace points adjacent to Section G4-1 are located at intervals of approximately 20 feet, and the largest stress occurs within this unbraced length.

In accordance with Article 3.4.2.1, when investigating Strength I, III, and V during construction, load factors for the weight of the structure and appurtenances, DC and DW, are not to be taken to be less than 1.25. Also, as discussed previously, the η factor is taken equal to 1.0 in this example. As shown in Table 9, the unfactored moments due to steel self-weight and Cast #1 are 661 kip-ft and 3,932 kip-ft, respectively, for a total of 4,593 kip-ft. Therefore, for

For the Strength I Load Combination:

General: $$f_{bu} = \frac{\eta \gamma M_{DC}}{S_{nc}}$$

Top Flange: $$f_{bu} = \frac{1.0(1.25)(4{,}593)(12)}{2{,}477} = -27.81 \text{ ksi}$$

Bot. Flange: $$f_{bu} = \frac{1.0(1.25)(4{,}593)(12)}{3{,}093} = 22.27 \text{ ksi}$$

For the Special Load Combination specified in Article 3.4.2.1:

$$\text{Top Flange:} \quad f_{bu} = \frac{1.0(1.4)(4{,}593)(12)}{2{,}477} = -31.15 \text{ ksi}$$

$$\text{Bot. Flange:} \quad f_{bu} = \frac{1.0(1.4)(4{,}593)(12)}{3{,}093} = 24.95 \text{ ksi}$$

The Special Load Combination controls in this case.

Section G4-1 must be checked for steel weight and for Cast #1 of the concrete deck on the noncomposite section as discussed above. The factored steel stresses during the sequential placement of the concrete are not to exceed the nominal resistances specified in Article 6.10.3.2.1 for compression and Article 6.10.3.2.2 for tension flanges. The effect of the overhang brackets on the flanges must also be considered according to Article C6.10.3.4 since G4 is an exterior girder.

7.4.1 Constructibility of Top Flange

7.4.1.1 Deck Overhang Bracket Load

During construction, the weight of the deck overhang wet concrete is resisted by the deck overhang brackets. Other loads supported by the overhang bracket during construction include the formwork, screed rail, railing, worker walkway, and the deck finishing machine.

The deck overhang construction loads are typically applied to the noncomposite section and removed once the concrete deck has become composite with the steel girders. The deck overhang bracket imparts a lateral force on the top and bottom flanges, resulting in lateral bending of the flanges. The lateral bending of both flanges must be considered as part of the constructibility check.

Since G4 is an exterior girder, half of the overhang weight is assumed placed on the girder and the other half is placed on the overhang brackets. The overhang bracket loading is shown in Figure 8.

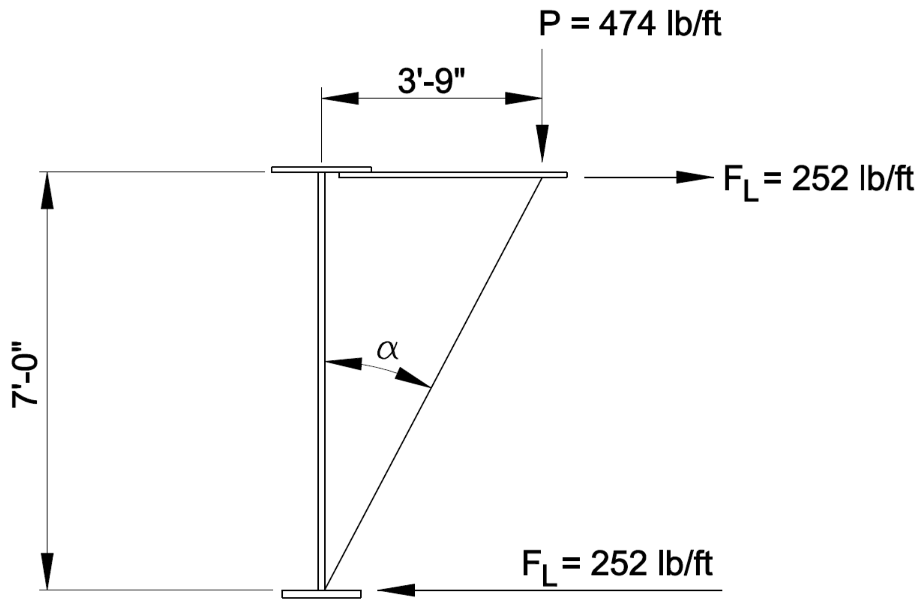


Figure 8 Deck Overhang Bracket Loading

The bracket loads are assumed to be applied uniformly although the brackets are actually spaced at about 3 feet along the girder.

The unbraced length, L_b, of the top flange is 20 feet. Assume that the average deck thickness in the overhang is 10 inches. The weight of the deck finishing machine is not considered.

Compute the vertical load on the overhang brackets.

$$\text{Deck} = \frac{1}{2}(3.75)\left(\frac{10}{12}\right)(150) = 234 \text{ lb/ft}$$

Deck forms + screed rail = 240 lb/ft (assumed)

Uniform load on brackets = 234 + 240 = 474 lb/ft

Compute the lateral force on the flange due to the overhang brackets.

α = arctan(3.75 ft/7.00 ft) = 28°

$$F_\ell = \frac{474\tan(28°)}{1000} = 0.252 \text{ kips/ft}$$

The lateral force, F_ℓ, is used to compute the flange lateral bending moment on top flange due to the deck overhang bracket. The flange lateral moment at the brace points due to the overhang forces is negative in the top flange of girder G4 on the outside of the curve because the stress due to the lateral moment is compressive on the convex side of the flange at the brace points. The opposite would be true on the convex side of the girder G1 top flange on the inside of the curve at the brace points. In the absence of a more refined analysis, the equations given in Article C6.10.3.4 may be used to estimate the maximum flange lateral bending moments in the discretely braced compression flange due to the lateral forces from the brackets. Assuming the flange is continuous with the adjacent unbraced lengths that are approximately equal, the flange lateral bending moment due to a statically equivalent uniformly distributed lateral bracket force may be estimated as:

$$M_{\ell} = \frac{F_{\ell} {L_b}^2}{12} \qquad \text{Eq. (C6.10.3.4-2)}$$

$$= -\left[\frac{0.252(20)^2}{12}\right] = -8.4 \text{ kip - ft (unfactored)}$$

7.4.1.2 Curvature Effects

In addition to the lateral bending moment due to the overhang brackets, lateral bending due to curvature must also be considered, which can either be taken from the analysis results or estimated by the approximate V-load equation given in Article C4.6.1.2.4b. The V-load equation assumes the presence of a cross frame at the point under investigation and a constant major-axis moment over the distance between the brace points. Although the use of the V-load equation is not theoretically pure for locations between brace points, it may conservatively be used. Note that throughout this example, the web depth, D, is conservatively used in this equation. Referring to Table 9, the moment due to the steel weight plus Cast #1 is used for M: 661 + 3,932 = 4,593 kip-ft.

$$M_{lat} = \frac{M\ell^2}{NRD} \qquad \text{Eq. (C4.6.1.2.4b-1)}$$

where: M = major-axis bending moment (kip-ft)

ℓ = unbraced length (ft)

N = a constant taken as 10 or 12 in past practice (the constant of 12 is generally recommended for use and will be used in this example)

R = girder radius (ft)

D = web depth (ft)

Therefore,

$$M_{lat} = \frac{(4{,}593)(20)^2}{12(716.5)(7)} = -30.5 \text{ kip} - \text{ft}$$

Although the flange lateral bending stresses are always additive to the major-axis bending stresses, it is helpful to understand the correct flange lateral moment sign when checking analysis results. The flange lateral moment at the brace points due to curvature is negative in the top flange of all four girders whenever the top flange is subjected to compression because the stress due to the lateral moment is compressive on the convex side of the flange at the brace points. The opposite is true whenever the top flange is subjected to tension.

The total factored lateral bending moment due to the combination of overhang brackets and curvature is therefore (the Special Load Combination specified in Article 3.4.2.1 controls by inspection):

$$M_{tot\_lat} = -[-8.4 + (-30.5)](1.4) = -54.5 \text{ kip} - \text{ft} \quad \text{(factored)}$$

7.4.1.3 Top Flange Lateral Bending Amplification

According to Article 6.10.1.6, lateral bending stresses determined from a first-order analysis may be used in discretely braced compression flanges for which:

$$L_b \leq 1.2L_p \sqrt{\frac{C_b R_b}{f_{bu}/F_{yc}}} \qquad \text{Eq. (6.10.1.6-2)}$$

L_p is the limiting unbraced length specified in Article 6.10.8.2.3 determined as:

$$L_p = 1.0r_t \sqrt{\frac{E}{F_{yc}}} = \frac{1.0(4.81)\sqrt{\frac{29{,}000}{50}}}{12} = 9.65 \quad \text{ft} \qquad \text{Eq. (6.10.8.2.3-4)}$$

where r_t is the effective radius of gyration for lateral torsional buckling specified in Article 6.10.8.2.3 determined as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{20}{\sqrt{12\left[1 + \frac{1}{3}\frac{47.03(0.5625)}{20(1)}\right]}} = 4.81 \text{ in.} \qquad \text{Eq. (6.10.8.2.3-9)}$$

Since the stresses remain reasonably constant over the section, the moment gradient factor, C_b, is taken as 1.0. Article C6.10.1.10.2 indicates that the web load-shedding factor, R_b, is taken as 1.0 for constructibility.

Check the relation given in Eq. (6.10.1.6-2):

$$L_b = 20 \text{ ft} > 1.2(9.65)\sqrt{\frac{1.0(1.0)}{\frac{31.15}{50}}} = 14.6 \text{ ft}$$

Because Eq. (6.10.1.6-2) is not satisfied, Article 6.10.1.6 requires that second-order elastic compression-flange lateral bending stresses be determined. The second-order compression-flange lateral bending stresses may be determined by amplifying the first-order values. First compute the first-order compression-flange lateral bending stress acting at the tip of the flange:

$$S_{top\_flange} = \frac{1.0(20)^2}{6} = 66.7 \text{ in.}^3$$

$$f_{\ell 1} = \frac{M_{tot\_lat}}{S_{top\ flange}} = \frac{-54.5(12)}{66.7} = -9.80 \text{ ksi (factored)}$$

The first-order values are amplified as follows:

$$f_\ell = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}} \right) f_{\ell 1} \geq f_{\ell 1} \quad \text{(second-order analysis)} \qquad \text{Eq. (6.10.1.6-4)}$$

where: f_{bu} = top flange stress calculated without consideration of flange lateral bending
F_{cr} = elastic lateral torsional buckling stress for the flange under consideration determined using Eq. (6.10.8.2.3-8)

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} = \frac{1.0(1.0)(\pi^2)(29{,}000)}{\left[\frac{20(12)}{4.81} \right]^2} = 115 \text{ ksi} \qquad \text{Eq. (6.10.8.2.3-8)}$$

The amplification factor (AF) is then determined as follows:

$$AF = \left(\frac{0.85}{1 - \frac{31.15}{115}} \right) = 1.17 > 1.0 \text{ OK}$$

Therefore, the total flange stress due to lateral bending, including the amplification factor is:

$$f_\ell = (AF)(f_{\ell 1}) = (1.17)(-9.80) = -11.47 \text{ ksi}$$

7.4.1.4 Flexure in Top Flange (Article 6.10.3.2.1)

During construction, the top flange at Section G4-1 is a discretely based compression flange, so the provisions of Article 6.10.3.2.1 apply. The article indicates that if the section has a slender web, Eq. (6.10.3.2.1-1) is not checked when f_ℓ is zero, and for sections with compact or noncompact webs, Eq. (6.10.3.2.1-3) is not checked. In this case, the web is slender (as demonstrated later) and f_ℓ is not zero, so all three equations must be checked.

$$f_{bu} + f_\ell \le \phi_f R_h F_{yc} \qquad \text{Eq. (6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc} \qquad \text{Eq. (6.10.3.2.1-2)}$$

$$f_{bu} \le \phi_f F_{crw} \qquad \text{Eq. (6.10.3.2.1-3)}$$

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)

R_h = hybrid factor specified in Article 6.10.1.10.1 (1.0 at homogeneous Section G4-1)

F_{crw} = nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9

F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2 (i.e. local or lateral torsional buckling resistance, whichever controls). The provisions of Article A6.3.3 are not to be used to determine the lateral torsional buckling resistance of top flanges of curved I-girder bridges, per Article 6.10.3.2.1.

First, check Eq. (6.10.3.2.1-1), using the previously calculated values of flange stresses:

$$f_{bu} + f_\ell = 31.15 + |-11.47| = 42.62 \text{ ksi} < \phi_f R_h F_{yc} = 1.0(1.0)(50) = 50 \text{ ksi OK}$$

(Ratio = 0.852)

Secondly, check Eq. (6.10.3.2.1-2). The equation must be satisfied for both local buckling and lateral torsional buckling using the the appropriate value of the nominal flexural resistance, F_{nc}, for local buckling (Article 6.10.8.2.2) or for lateral torsional buckling (Article 6.10.8.2.3), as applicable.

Determine the local buckling resistance of the compression flange. First, check the flange slenderness.

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{20}{2(1)} = 10 \qquad \text{Eq. (6.10.8.2.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15 \qquad \text{Eq. (6.10.8.2.2-4)}$$

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_{yr}}} = 0.56\sqrt{\frac{29{,}000}{0.7(50)}} = 16.12 \qquad \text{Eq. (6.10.8.2.2-5)}$$

Since $\lambda_{pf} < \lambda_f < \lambda_{rf}$, the flange is noncompact and the nominal flexural resistance is determined using Eq. (6.10.8.2.2-2).

R_b is taken as 1.0 for constructibility checks per Article 6.10.3.2.1, and R_h is taken as 1.0 per Article 6.10.1.10.1. Therefore, F_{nc} for the local buckling resistance is calculated as:

$$F_{nc} = \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.2-2)}$$

$$= \left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{10 - 9.15}{16.12 - 9.15}\right)\right](1.0)(1.0)(50) = 48.17 \text{ ksi}$$

Determine the lateral torsional buckling resistance of the compression flange. First, compare the unbraced length, L_b, to the limiting unbraced lengths L_p and L_r.

L_b = 20 ft = unbraced length
L_p = 9.65 ft (calculated previously in top flange lateral bending amplification calculation)

L_r is the limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression-flange residual stress effects and is determined as follows:

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = \frac{\pi(4.81)\sqrt{\frac{29{,}000}{0.7(50)}}}{12} = 36.2 \text{ ft.} \qquad \text{Eq. (6.10.8.2.3-5)}$$

Since $L_p < L_b < L_r$, use Eq. (6.10.8.2.3-2) to calculate the lateral torsional buckling resistance.

$$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.3-2)}$$

$$= 1.0\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{20 - 9.65}{36.2 - 9.65}\right)\right](1.0)(1.0)(50) = 44.15 \text{ ksi}$$

Therefore, check Eq. (6.10.3.2.1-2) for local buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell = 31.15 + \frac{1}{3}(11.47) = 34.97 \text{ ksi} < \phi_f F_{nc} = 1.0(48.17) = 48.17 \text{ ksi} \quad \text{OK}$$

(Ratio= 0.726)

Check Eq. (6.10.3.2.1-2) for lateral torsional buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell = 31.15 + \frac{1}{3}(11.47) = 34.97 \text{ ksi} < \phi_f F_{nc} = 1.0(44.15) = 44.15 \text{ ksi} \quad \text{OK}$$

(Ratio= 0.792)

Thirdly, check Eq. (6.10.3.2.1-3) since the web is slender, as shown below. The slenderness is checked according to Article 6.10.6.2.3 for noncomposite sections:

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.6.2.3-1)}$$

$$\frac{2(47.03)}{0.5625} = 167.2 > 5.7\sqrt{\frac{29{,}000}{50}} = 137.3 \qquad \text{slender web, noncompact section}$$

Because the web is slender, Eq. (6.10.3.2.1-3) is checked to control the out-of-plane web distortions that may occur during construction.

$$f_{bu} \le \phi_f F_{crw} \qquad \text{Eq. (6.10.3.2.1-3)}$$

where the nominal web bend-buckling resistance, F_{crw}, is taken as:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad \text{Eq. (6.10.1.9.1-1)}$$

but F_{crw} cannot exceed R_hF_{yc} and $F_{yw}/0.7$ per Article 6.10.1.9.1 for webs without longitudinal stiffeners.

First, compute the bend-buckling coefficient, k, in which D_c is the depth of web in compression. Since the girder is noncomposite for this check, D_c is the distance from the inner edge of the compression flange to the neutral axis.

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} = \frac{9}{\left(\frac{47.03}{84}\right)^2} = 28.7 = \text{bend-buckling coefficient} \qquad \text{Eq. (6.10.1.9.1-2)}$$

$$F_{crw} = \frac{0.9(29{,}000)(28.7)}{\left(\frac{84}{0.5625}\right)^2} = 33.6 \text{ ksi} < 1.0(50.0) = 50.0 \text{ ksi} < \frac{50}{0.7} = 71.4 \text{ ksi}$$

Therefore, use F_{crw} = 33.6 ksi to check Eq. (6.10.3.2.1-3):

$$f_{bu} = 31.15 \text{ ksi} < \phi_f F_{crw} = 1.0(33.6) = 33.6 \text{ ksi} \quad \text{OK (Ratio = 0.927)}$$

The compression flange proportions satisfy the criteria given in Article 6.10.3.2.1.

It should be noted that the web bend-buckling resistance (F_{crw}) is generally checked against the maximum compression flange stress due to factored loads without consideration of flange lateral bending, as shown in the previous calculation. Since web bend-buckling is a check of the web, the maximum flexural compression stress in the web could be calculated and used for comparison against the bend-buckling resistance. However, the precision associated with making the distinction between the stress in the compression flange and the maximum compressive stress in the web is typically not warranted.

7.4.2 Constructibility of Bottom Flange

For critical stages of construction, the following requirement must be satisfied for discretely braced tension flanges according to Article 6.10.3.2.2.

$$f_{bu} + f_{\ell} \leq \phi_f R_h F_{yt} \qquad \text{Eq. (6.10.3.2.2-1)}$$

The factored tensile flange stress due to steel self-weight and Cast #1, calculated without consideration of the lateral bending, f_{bu}, in the bottom flange due to the Special Load Combination specified in Article 3.4.2.1 was calculated previously as:

$$f_{bu} = 24.95 \text{ ksi}$$

The total lateral bending moment due to overhang brackets and curvature effects, factored for constructibility, is 54.5 kip-ft as previously calculated. Therefore, the lateral bending stress in the bottom flange is as follows:

$$f_{\ell} = \frac{M_{tot\_lat}}{S_{bot\ flange}} = \frac{54.5(12)}{(1.5)(21)^2/6} = 5.93 \text{ ksi}$$

Therefore,

$$f_{bu} + f_{\ell} = 24.95 + 5.93 = 30.88 \text{ ksi} < \phi_f R_h F_{yc} = 1.0(1.0)(50) = 50 \text{ ksi} \ \text{OK}$$

(Ratio = 0.618)

7.4.3 Constructibility Shear Strength, Web

Panels of webs with transverse stiffeners are investigated for constructibility, with or without longitudinal stiffeners, and must satisfy the requirement specified in Article 6.10.3.3 during critical stages of construction. This calculation is similar to the shear strength check at the strength limit state (shown previously for end panels) and therefore is not shown.

7.4.4 Constructibility of Deck

The concrete deck is checked for constructibility according to Article 6.10.3.2.4, which states that the longitudinal tensile stress in the composite concrete deck due to factored loads shall not exceed ϕf_r during critical stages of construction unless longitudinal reinforcement is provided according to Article 6.10.1.7. Article 6.10.1.7 states that whenever the tensile stress in the deck exceeds ϕf_r, longitudinal reinforcement equal to at least one percent of the total cross-sectional area of the deck must be placed in the deck.

By inspection, it is observed that Cast #2 will cause negative moment near mid-span of Span 1. In practice, multiple locations would be checked to determine where the one percent longitudinal reinforcement is no longer required. For the purpose of this example, the deck tensile stress will be checked only at the location of G4-1 due to Cast #2. The major-axis moment at G4-1 due to Cast #2 is -3,035 kip-ft, as shown in Table 9. This location is appropriate to check since it lies within the Cast #1 composite section, which is 100 feet long and assumed to be hardened for Cast #2. See Figure 4 for the placement sequence diagram.

According to Article 6.10.1.1.1d, the short-term modular ratio, *n*, is used to calculate longitudinal flexural stresses in the concrete deck due to all permanent and transient loads.

Assume no creep: n = 7.56.

The Special Load Combination specified in Article 3.4.2.1 controls by inspection. Calculate the factored tensile stress at the top of the structural slab:

$$f_{deck} = (1.4)\frac{(-3{,}035)(12)(-28.25)}{294{,}158}\left(\frac{1}{7.56}\right) = 0.65 \text{ ksi}$$

Assume the compressive strength of the hardened concrete from Cast #1 is 3,000 psi at the time Cast #2 is made. The modulus of rupture is:

$$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{3} = 0.42 \text{ ksi}$$

Therefore,

$$\phi f_r = 0.9(0.42) = 0.38 \text{ ksi} < 0.65 \text{ ksi}$$

where ϕ = 0.9 from Article 5.5.4.2.1. Since $f_{deck} > \phi f_r$, one percent longitudinal reinforcement is required at this section. The reinforcement is to be 60.0 ksi or higher strength, and should be a #6 bar or smaller spaced at not more than 12 inches according to Article 6.10.1.7. The required reinforcement should be placed in two layers uniformly distributed across the deck width, and two-thirds should be placed in the top layer. As discussed under Section Properties earlier in this example, #6 bars spaced at 6 inches in the top layer and #4 bars spaced at 6 inches in the bottom layer satisfy these requirements.

The longitudinal reinforcement selected above would be continued into the "negative moment region," over the pier, and terminated in the next span at a point where it is no longer required, determined in a similar fashion as the steps described above.

If it is desired to lower the concrete stress at a given location, the deck placement sequence could be modified.

7.5 Girder Check: Section G4-1, Service Limit State (Article 6.10.4)

Article 6.10.4 contains provisions related to the control of elastic and permanent deformations at the Service Limit State. For the sake of brevity, only the calculations pertaining to permanent deformations will be presented for this example.

7.5.1 Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made under the Service II load combination.

Article 6.10.4.2.2 states that flanges of composite sections must satisfy the following requirements:

Top flange of composite sections: $f_f \leq 0.95 R_h F_{yf}$ Eq. (6.10.4.2.2-1)

Bottom flange of composite sections: $f_f + \frac{f_\ell}{2} \leq 0.95 R_h F_{yf}$ Eq. (6.10.4.2.2-2)

However, according to Article C6.10.4.2.2, under the load combinations specified in Table 3.4.1-1, Eqs. (6.10.4.2.2-1) and (6.10.4.2.2-2) need only be checked for compact sections in positive flexure. For sections in negative flexure and noncompact sections in positive flexure, these two equations do not control and need not be checked. Composite sections in all horizontally curved girder systems are to be treated as noncompact sections at the strength limit state, in accordance with Article 6.10.6.2.2. Therefore, for Section G4-1, Eqs. (6.10.4.2.2-1) and (6.10.4.2.2-2) do not need to be checked but are demonstrated below for illustrative purposes only.

The term f_f is the flange stress at the section under consideration due to the Service II load combination calculated without consideration of flange lateral bending. The f_ℓ term, the flange lateral bending stress, in Eq. (6.10.4.2.2-2) is to be determined in accordance with Article 6.10.1.6. A resistance factor is not included in these equations because Article 1.3.2.1 specifies that the resistance factor be taken equal to 1.0 at the service limit state.

It should be noted that in accordance with Article 6.10.4.2.2, redistribution of negative moment due to the Service II loads at the interior-pier sections in continuous span flexural members using the procedures specified in Appendix B6 is not to be applied to horizontally curved I-girder sections. The applicability of the Appendix B6 provisions to horizontally curved I-girder sections has not been demonstrated; hence the procedures are not permitted for this type of girder.

Check the flange stresses due to the Service II loads at Section G4-1. η is always specified to equal 1.0 at the service limit state (Article 1.3.2):

$$0.95R_hF_{yf} = 0.95(1.0)(50) = 47.50 \text{ ksi}$$

Top Flange:

$$f_f \leq 0.95R_hF_{yf} \qquad \text{Eq. (6.10.4.2.2-1)}$$

$$f_f = 1.0\left[\frac{1.0(661+2{,}682)}{2{,}477} + \frac{1.0(510+583)}{6{,}900} + \frac{1.3(5{,}125)}{18{,}102}\right]12 = -22.51 \text{ ksi}$$

$$f_f = \left|-22.51\right| \text{ksi} \leq 0.95R_hF_{yf} = 47.50 \text{ ksi} \quad \text{OK} \qquad (\text{Ratio} = 0.474)$$

Bottom Flange:

$$f_f + \frac{f_\ell}{2} \leq 0.95R_hF_{yf} \qquad \text{Eq. (6.10.4.2.2-1)}$$

Compute f_ℓ similarly to how it was calculated for the top flange constructibility checks. First determine the flange lateral moment, M_{lat}, due to the Service II load combination:

$$M_{lat} = \frac{M\ell^2}{NRD} \qquad \text{Eq. (C4.6.1.2.4b-1)}$$

$$= \frac{[1.0(661+2{,}682+510+583)+1.3(5{,}125)](20)^2}{12(716.5)(7)} = 73.8 \text{ kip} - \text{ft}$$

The factored Service II flange stress is:

$$f_\ell = \frac{M_{lat}}{S_{bot\_fl}} \frac{73.8(12)}{110.3} = 8.03 \text{ ksi}$$

Therefore:

$$f_f + \frac{f_\ell}{2} = 1.0\left[\frac{1.0(661+2{,}682)}{3{,}093} + \frac{1.0(510+583)}{3{,}835} + \frac{1.3(5{,}125)}{4{,}187}\right]12 + \frac{8.03}{2} = 39.50 \text{ ksi}$$

$$f_f + \frac{f_\ell}{2} = 39.50 \text{ ksi} < 47.50 \text{ ksi} \quad \text{OK} \quad (\text{Ratio} = 0.832)$$

7.5.2 Web Bend-Buckling

With the exception of composite sections in positive flexure in which the web satisfies the requirement of Article 6.10.2.1.1 ($D/t_w \leq 150$), web bend-buckling of all sections under the Service II load combination is to be checked as follows:

$$f_c \leq F_{crw} \qquad \text{Eq. (6.10.4.2.2-4)}$$

The term f_c is the compression-flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9.

At Section G4-1:

$$\frac{D}{t_w} = \frac{80}{0.5625} = 142.2 \leq 150$$

Because Section G4-1 is a composite section subject to positive flexure satisfying $D/t_w \leq 150$, Eq. (6.10.4.2.2-4) need not be checked. An explanation as to why these particular sections are exempt from the above web bend-buckling check is given in Article C6.10.1.9.1.

7.6 Girder Check: Section G4-1, Fatigue Limit State (Article 6.10.5)

Article 6.10.5 indicates that details in I-girder section flexural members must be investigated for fatigue as specified in Article 6.6.1. For horizontally curved I-girder bridges, the fatigue stress range due to major-axis bending plus lateral bending is to be investigated. As appropriate, the Fatigue I and Fatigue II load combinations specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 are to be employed for checking load-induced fatigue in I-girder sections. The Fatigue I load combination is used when investigating infinite load-induced fatigue, and the Fatigue II load combination is used when investigating finite load-induced fatigue.

According to Table 3.6.2.1-1, the dynamic load allowance for fatigue loads is 15 percent. Centrifugal force effects are considered and are included in the fatigue moments. For the purpose of this design example, the 75-year single lane ADTT is assumed to be 1,000 trucks per day.

7.6.1 Fatigue in Bottom Flange

At Section G4-1, it is necessary to check the bottom flange for the fatigue limit state. The base metal at the transverse stiffener weld terminations and interior cross frame connection plate welds at locations subject to a net tensile stress must be checked for fatigue. This detail corresponds to Condition 4.1 in Table 6.6.1.2.3-1 and is classified as a Category C′ fatigue detail. Only the bottom flange is checked herein, as a net tensile stress is not induced in the top flange by the fatigue loading at this location.

According to Eq. (6.6.1.2.2-1), the factored fatigue stress range, $\gamma(\Delta f)$, must not exceed the nominal fatigue resistance, $(\Delta F)_n$. In accordance with Article C6.6.1.2.2, the resistance factor, ϕ, and the load modifier, η, are taken as 1.0 for the fatigue limit state.

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{Eq. (6.6.1.2.2-1)}$$

From Table 6.6.1.2.3-2, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category C′ fatigue detail is 745 trucks per day. Therefore, since the assumed $(ADTT)_{SL}$ for this design example of 1,000 trucks per day is greater than this limit of 745 trucks per day, the detail must be checked for infinite fatigue life using the Fatigue I load combination. Per Article 6.6.1.2.5, the nominal fatigue resistance for infinite fatigue life is equal to the constant-amplitude fatigue threshold:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \text{Eq. (6.6.1.2.5-1)}$$

where $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold and is taken from Table 6.6.1.2.5-3. For a Category C′ fatigue detail, $(\Delta F)_{TH}$ = 12.0 ksi, and therefore:

$$(\Delta F)_n = 12.0 \text{ ksi}$$

As shown in Table 9, the unfactored negative and positive moments due to fatigue, including centrifugal force effects and the 15 percent dynamic load allowance, at Section G4-1 are -603 kip-ft and 1,603 kip-ft, respectively. As shown in Table 13, the short-term composite section properties (n = 7.56) used to compute the stress at the bottom of the web (top of the bottom flange, where the weld in question is located) are:

$I_{NA(n)}$ = 294,158 in.^{4}

$d_{BOT\ OF\ WEB} = d_{BOT\ OF\ STEEL} - t_{f\_BOT\ FLANGE}$ = 70.25 in. – 1.5 in. = 68.75 in.

Therefore, the unfactored stress range at the bottom of the web due to vertical loads only is:

$$f_{range\_vert} = \left(\frac{(|-603| + 1{,}603)(12)(68.75)}{294{,}158} \right) = 6.19 \text{ ksi}$$

The flange lateral bending stress at the connection plate must also be considered according to Article C6.10.5.1. The connection plates are assumed to be 6 inches wide. To compute the flange lateral bending stress range at the top of the bottom flange due to curvature, it is first necessary to compute the flange lateral moment of inertia:

$$I_{flg} = \frac{1.5(21)^3}{12} = 1{,}158 \text{ in.}^4$$

Using Eq. (C4.6.1.2.4b-1), compute the range of flange lateral moment at the connection plate:

$$M_{lat} = \frac{M\ell^2}{NRD} = \frac{(|-603| + 1{,}603)(20^2)}{12(716.5)(7)} = 14.66 \text{ kip} - \text{ft}$$

Compute the distance from the centerline of the web to the edge of the connection plate, and then compute the stress at this point:

$$c = 6 + \frac{0.5625}{2} = 6.3 \text{ in.}$$

$$f_{lat} = \frac{14.66(6.3)}{1{,}158}(12) = 0.96 \text{ ksi}$$

Per Table 3.4.1-1, the load factor, γ, for the Fatigue I load combination is 1.5. The total factored stress range at the edge of the connection plate due to both major-axis bending stress and flange lateral bending stress is therefore:

$$\gamma(\Delta f) = (1.5)(6.19 + 0.96) = 10.73 \text{ ksi}$$

Checking Eq. (6.6.1.2.2-1),

$$\gamma(\Delta f) = 10.73 \text{ ksi} < (\Delta F) = 12.00 \text{ ksi} \quad \text{OK} \quad (\text{Ratio} = 0.894)$$

7.6.2 Special Fatigue Requirement for Webs

In accordance with Article 6.10.5.3, interior panels of stiffened webs must satisfy:

$$V_u \leq V_{cr} \qquad \text{Eq. (6.10.5.3-1)}$$

where: V_u = shear in the web at the section under consideration, due to unfactored permanent loads plus the factored fatigue load (Fatigue I live load)

V_{cr} = shear buckling resistance determined from Eq. (6.10.9.3.3-1)

Satisfaction of Eq. (6.10.5.3-1) is intended to control elastic flexing of the web, and the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect. The live load shear in the special requirement is supposed to represent the heaviest truck expected to cross the bridge in 75 years.

Only interior panels of stiffened webs are investigated because the shear resistance of end panels of stiffened webs and the shear resistance of unstiffened webs are limited to the shear buckling resistance at the strength limit state.

The unfactored shears at Section G4-1 are shown below. These results are taken directly from the three-dimensional analysis as reported in Table 10:

Steel Dead Load:	$V_{DC1\text{-}STEEL}$	= -5 kips
Concrete Deck Dead Load:	$V_{DC1\text{-}CONC}$	= -23.8 kips
Composite Dead Load:	V_{DC2}	= -4 kips
Future Wearing Surface Dead Load:	V_{DW}	= -2.9 kips
Total Permanent Load		= -35.7 kips
Fatigue Live Load + Impact:	V_{FAT}	= -20 kips

Therefore, the Fatigue I shear in the web is:

$$V_u = -35.7 + 1.5(-20) = \text{-65.7 kips}$$

Next, compute the shear-buckling resistance:

$$V_{cr} = CV_p \qquad \text{Eq. (6.10.9.3.3-1)}$$

where: C = ratio of the shear-buckling resistance to the shear yield strength
V_p = plastic shear force

Compute the plastic shear force:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.3-2)}$$

$$V_p = 0.58(50)(84)(0.5625) = 1{,}370 \text{ kips}$$

To determine the ratio C, the shear-buckling coefficient, k, must first be computed as follows:

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

At this particular location, the transverse stiffener spacing is assumed to be 82 inches. Therefore, d_o = 82 in.

$$k = 5 + \frac{5}{\left[\frac{82}{84}\right]^2} = 10.2$$

Check the following relation in order to select the appropriate equation for computing C:

$$\frac{D}{t_w} = \frac{84}{0.5625} = 149.3 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(10.2)}{50}} = 108$$

Since the above relation is true, the ratio C is computed using Eq. (6.10.9.3.2-6) as follows:

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

$$C = \frac{1.57}{\left(\frac{84}{0.5625}\right)^2}\left(\frac{29{,}000(10.2)}{50}\right) = 0.416$$

The shear-buckling resistance is then computed in accordance with Eq. (6.10.9.3.3-1):

$$V_{cr} = (0.416)(1{,}370) = 570 \text{ kips}$$

Using the above results, check the requirement of Article 6.10.5.3, $V_u \le V_{cr}$:

$$V_u = |-65.7| \text{ kips} \le V_{cr} = 570 \text{ kips} \quad \text{OK}$$

Therefore, the web is satisfactory for fatigue at the maximum positive moment location.

7.7 Girder Check: Section G4-1, Strength Limit State (Article 6.10.6)

7.7.1 Flexure (Article 6.10.6.2)

According to Article 6.10.6.2.2, sections in positive flexure in horizontally curved steel girder bridges are to be considered noncompact sections and are to satisfy the requirements of Article 6.10.7.2. Furthermore, both compact and noncompact sections in positive flexure must satisfy the ductility requirement specified in Article 6.10.7.3. The ductility requirement is intended to protect the concrete deck from premature crushing. The section must satisfy:

$$D_p \le 0.42\,D_t$$ Eq. (6.10.7.3-1)

Where D_p is the distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment, and D_t is the total depth of the composite section. Reference the section property computations for the location of the neutral axis of the composite section at the plastic moment. At Section G4-1:

$$D_p = 9.0 + 4.0 - 1.0 + 0.77 = 12.77 \text{ in.}$$

$$D_t = 1.5 + 84.0 + 4.0 + 9.0 = 98.50 \text{ in.}$$

$$0.42D_t = 0.42(98.50) = 41.37 \text{ in.} > 12.77 \text{ in.} \quad \text{OK (Ratio = 0.309)}$$

Noncompact sections in positive flexure must satisfy the provisions of Article 6.10.7.2. At the strength limit state, the compression flange must satisfy:

$$f_{bu} \leq \phi_f F_{nc} \qquad \text{Eq. (6.10.7.2.1-1)}$$

where:

f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6
ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.7.2.2

As explained in Article C6.10.7.2.1, flange lateral bending is not considered for the compression flanges at the strength limit state because the flanges are continuously supported by the concrete deck.

At the strength limit state, the tension flange must satisfy:

$$f_{bu} + \frac{1}{3} f_{\ell} \leq \phi_f F_{nt} \qquad \text{Eq. (6.10.7.2.1-2)}$$

where:

f_{ℓ} = flange lateral bending stress determined as specified in Article 6.10.1.6
F_{nt} = nominal flexural resistance of the tension flange determined as specified in Article 6.10.7.2.2

Additionally, the maximum longitudinal compressive stress in the concrete deck at the strength limit state is not to exceed $0.6f_c'$. The longitudinal compressive stress in the deck is to be determined in accordance with Article 6.10.1.1.1d, which allows the permanent and transient load stresses in the deck to be computed using the short-term section properties (i.e. modular ratio taken as *n*).

7.7.1.1 Strength I Flexural Stress in Top and Bottom Flange

The unfactored bending moments at Section G4-1 are shown below. These results are directly from the three-dimensional analysis as reported in Table 9. The live load moment includes the centrifugal force and dynamic load allowance effects.

Noncomposite Dead Load:	M_{DC1}	= 661 + 2,682 = 3,343 kip-ft
Composite Dead Load:	M_{DC2}	= 510 kip-ft
Future Wearing Surface Dead Load:	M_{DW}	= 583 kip-ft
Live Load (including IM and CF):	M_{LL+IM}	= 5,125 kip-ft

Compute the factored flange flexural stresses at Section G4-1 for the Strength I load combination, without consideration of flange lateral bending. As discussed previously, the η factor is taken equal to 1.0 in this example. Therefore:

For Strength I, the bending stresses due to vertical loads are as follows:

Top Flange (compression):

$$f_{bu} = \left[\frac{(\gamma_{DC1} M_{DC1})}{S_{nc}} + \frac{[(\gamma_{DC2} M_{DC2} + \gamma_{DW} M_{DW})]}{S_{3n}} + \frac{(\gamma_{LL} M_{LL})}{S_n} \right](12)\eta$$

$$= -\left[\frac{1.25(3{,}343)}{2{,}477} + \frac{[1.25(510) + 1.5(583)]}{6{,}900} + \frac{1.75(5{,}125)}{18{,}102} \right](12)(1) = -28.82 \text{ ksi}$$

Bottom Flange (tension):

$$f_{bu} = \left[\frac{(\gamma_{DC1} M_{DC1})}{S_{nc}} + \frac{[(\gamma_{DC2} M_{DC2} + \gamma_{DW} M_{DW})]}{S_{3n}} + \frac{(\gamma_{LL} M_{LL})}{S_n} \right](12)\eta$$

$$= \left[\frac{1.25(3{,}343)}{3{,}093} + \frac{[1.25(510) + 1.5(583)]}{3{,}835} + \frac{1.75(5{,}125)}{4{,}187} \right](12)(1) = 46.65 \text{ ksi}$$

As required to check the discretely braced tension flange, the lateral bending stress must also be calculated for the bottom flange. Using the moments shown above, the unfactored lateral bending moment and corresponding lateral bending stress are calculated as follows:

$$M_{lat} = \frac{M\ell^2}{NRD} \qquad \text{Eq. (C4.6.1.2.4b-1)}$$

$$f_\ell = \frac{M_{lat}}{S_{bot\_flange}}, \text{ where } S_{bot\_flange} = \frac{(1.5)(21)^2}{6} = 110.3 \text{ in.}^3$$

$$M_{lat\_DC1} = \frac{3{,}343(20)^2}{12(716.5)(7)} = 22.22 \text{ kip} - \text{ft} \qquad f_{\ell\_DC1} = \frac{M_{lat\_DC1}}{S_{bot\_fl}} = \frac{22.22(12)}{110.3} = 2.42 \text{ ksi}$$

$$M_{lat\_DC2} = \frac{510(20)^2}{12(716.5)(7)} = 3.39 \text{ kip} - \text{ft} \qquad f_{\ell\_DC2} = \frac{M_{lat\_DC2}}{S_{bot\_fl}} = \frac{3.39(12)}{110.3} = 0.37 \text{ ksi}$$

$$M_{lat\_DW} = \frac{583(20)^2}{12(716.5)(7)} = 3.88 \text{ kip} - \text{ft} \qquad f_{\ell\_DW} = \frac{M_{lat\_DW}}{S_{bot\_fl}} = \frac{3.88(12)}{110.3} = 0.43 \text{ ksi}$$

$$M_{lat\_LL} = \frac{5{,}125(20)^2}{12(716.5)(7)} = 34.06 \text{ kip} - \text{ft} \qquad f_{\ell\_LL} = \frac{M_{lat\_LL}}{S_{bot\_fl}} = \frac{34.06(12)}{110.3} = 3.71 \text{ ksi}$$

Therefore, the total factored lateral bending stress in the bottom flange is:

$$f_{\ell} = 1.25(2.42 + 0.37) + 1.5(0.43) + 1.75(3.71) = 10.63 \text{ ksi}$$

7.7.1.2 Top Flange Flexural Resistance in Compression

Per Article 6.10.7.2.2, the nominal flexural resistance of the compression flange of noncompact composite sections in positive flexure is to be taken as:

$$F_{nc} = R_b R_h F_{yc} \qquad \text{Eq. (6.10.7.2.2-1)}$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1.

For a homogenous girder, the hybrid factor, R_h, is equal to 1.0. In accordance with Article 6.10.1.10.2, the web load-shedding factor, R_b, is equal to 1.0 for composite section in which the web satisfies the requirement of Article 6.10.2.1.1, such that $D/t_w \leq 150$.

$$\frac{D}{t_w} = \frac{84}{0.5625} = 149.3 \leq 150$$

Therefore:

$$F_{nc} = (1.0)(1.0)(50.00) = 50.00 \text{ ksi}$$

For Strength I:

$$f_{bu} \le \phi_f F_{nc} \qquad \text{Eq. (6.10.7.2.1-1)}$$

$$f_{bu} = \left|-28.82\right| \text{ksi} \le \phi_f F_{nc} = (1.0)(50.00) = 50.00\ \text{ksi} \qquad \text{OK} \qquad (\text{Ratio} = 0.576)$$

7.7.1.3 Bottom Flange Flexural Resistance in Tension

Article 6.10.7.2.2 states that the nominal flexural resistance of the tension flange of noncompact composite sections is to be taken as:

$$F_{nt} = R_h F_{yt} \qquad \text{Eq. (6.10.7.2.2-2)}$$

Therefore:

$$F_{nt} = (1.0)(50.00) = 50.00\ \text{ksi}$$

For Strength I:

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nt} \qquad \text{Eq. (6.10.7.2.1-2)}$$

$$f_{bu} + \frac{1}{3} f_\ell = 46.65 + \frac{1}{3}(10.63) = 50.19\ \text{ksi} \approx \phi_f F_{nt} = (1.0)(50.00) = 50.00\ \text{ksi} \quad (\text{Ratio} = 1.004)$$

Ratio is slightly greater than 1.00, but say OK for the purpose of this design example.

In practice, the flange thickness could be increased at this field section to eliminate the overstress in the bottom flange.

According to the provisions of Article 6.10.1.6, lateral bending stresses in discretely braced flanges are to satisfy the following requirement:

$$f_\ell \le 0.6 F_{yf} \qquad \text{Eq. (6.10.1.6-1)}$$

Although this check also applies to both the top flange and the bottom flange before the deck has cured, it is only demonstrated in this example for the bottom flange in the final condition at the strength limit state.

$$f_\ell = 10.63\ \text{ksi} < 0.6 F_{yf} = 0.6(50) = 30\ \text{ksi} \quad \text{OK} \quad (\text{Ratio} = 0.354)$$

7.7.2 Web Flexural Resistance

Article C6.10.1.9.1 states that composite sections subjected to positive flexure need not be checked for web bend-buckling in their final composite condition when the web does not require longitudinal stiffeners, as is the case for this design example.

7.7.3 Concrete Deck Stresses

According to Article C6.10.7.2.1, the maximum longitudinal compressive stress in the concrete deck at the strength limit state is not to exceed $0.6f_c'$. This limit is to ensure linear behavior of the concrete, which is assumed in the calculation of steel flange stresses. The longitudinal compressive stress in the deck is to be determined in accordance with Article 6.10.1.1.1.d, which allows the permanent and transient load stresses in the deck to be computed using the short-term section properties (*n* = 7.56 composite section properties). Referring to Table 13 of the section property calculations, the section modulus to the top of the concrete deck is:

$$S_{deck} = \frac{294{,}158}{9.0 + 4.0 + \frac{84}{2} - 26.75} = 10{,}413\,\text{in.}^3$$

Calculate the Strength I factored longitudinal compressive stress in the deck at this section, noting that the concrete deck is not subjected to noncomposite dead loads. The stress in the concrete deck is obtained by dividing the stress acting on the transformed section by the modular ratio, *n*.

$$f_{deck} = 1.0\left[\frac{1.25(510) + 1.5(583) + 1.75(5{,}125)}{(10{,}413)(7.56)}\right]12 = -1.60\text{ ksi}$$

$$f_{deck} = \left|-1.60\text{ ksi}\right| < 0.6f'_c = 0.6(4.0) = 2.40\text{ ksi} \qquad \text{OK}$$

7.8 Girder Check: Section G4-2, Constructibility (Article 6.10.3)

Although not required, the bottom flange at Section G4-2, which is a discretely braced flange in compression, may be checked to ensure that it satisfies the requirements of Eqs. (6.10.3.2.1-1), (6.10.3.2.1-2), and (6.10.3.2.1-3) for critical stages of construction, if desired. Generally these provisions will not control because the size of the bottom flange in negative flexure regions is normally governed by the Strength Limit State. In regard to construction loads, the maximum negative moment reached during the deck placement analysis, plus the moment due to the self-weight, typically does not significantly exceed the calculated noncomposite negative moments assuming a single stage deck placement. Nonetheless, the constructibility check is performed herein for completeness, and to illustrate the constructibility checks for a negative moment region. For this constructibility check, it is assumed that the concrete deck has not yet hardened at Section G4-2. The following equations are checked for the compression flange:

$$f_{bu} + f_{\ell} \le \phi_f R_h F_{yc} \qquad \text{Eq. (6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_{\ell} \le \phi_f F_{nc} \qquad \text{Eq. (6.10.3.2.1-2)}$$

$$f_{bu} \le \phi_f F_{crw} \qquad \text{Eq. (6.10.3.2.1-3)}$$

Additionally, the top flange, which is considered discretely braced for constructibility (i.e. the deck is not hardened), may be checked for the following requirement specified in Article 6.10.3.2.2.

$$f_{bu} + f_{\ell} \le \phi_f R_h F_{yt} \qquad \text{Eq. (6.10.3.2.2-1)}$$

To illustrate this constructibility check, it is assumed that the unfactored major-axis bending moment due to the deck placement is -7,272 kip-ft and moment due to steel self-weight is -1,917 kip-ft at this section (see Table 9).

Calculate the factored major-axis flexural stresses in the flanges of the steel section due to the factored load resulting from the steel self-weight and the assumed deck placement sequence.

For the Strength I Load Combination:

Top Flange: $$f_{bu} = \frac{1.0(1.25)[(-1{,}917) + (-7{,}272)](12)}{6{,}689} = 20.61 \text{ ksi}$$

Bot. Flange: $$f_{bu} = \frac{1.0(1.25)[(-1{,}917) + (-7{,}272)](12)}{7{,}377} = -18.68 \text{ ksi}$$

For the Special Load Combination specified in Article 3.4.2.1:

Top Flange: $$f_{bu} = \frac{1.0(1.4)[(-1{,}917) + (-7{,}272)](12)}{6{,}689} = 23.08 \text{ ksi}$$

Bot. Flange: $$f_{bu} = \frac{1.0(1.4)[(-1{,}917) + (-7{,}272)](12)}{7{,}377} = -20.93 \text{ ksi}$$

The Special Load Combination controls in this case.

For this example and for illustration purposes, the V-load equation is used to compute the flange lateral bending moments due to curvature.

$$M_{LAT} = \frac{M\ell^2}{NRD} = \left| \frac{[(-1{,}917) + (-7{,}272)](20)^2}{(12)(716.5)(7)} \right| = 61.1 \text{ kip} - \text{ft} \qquad \text{Eq. (C4.6.1.2.4b-1)}$$

Combine the factored flange lateral bending moment computed using the V-load equation with the lateral moment due to the overhang brackets which was computed in earlier calculations. The factored flange lateral bending moment and flange lateral bending stress are computed as:

$$M_{TOT\_LAT} = (1.4)[61.1 + 8.4] = 97.3 \text{ kip} - \text{ft}$$

Top Flange:

$$f_\ell = \frac{M_{TOT\_LAT}}{S_\ell} = \frac{(97.3)(12)}{(2.50)(28)^2/6} = 3.57 \text{ ksi}$$

Bot. Flange:

$$f_\ell = \frac{M_{TOT\_LAT}}{S_\ell} = \frac{(97.3)(12)}{(3.00)(27)^2/6} = 3.20 \text{ ksi}$$

7.8.1 Constructibility of Top Flange

For critical stages of construction, the following requirement must be satisfied for discretely braced tension flanges according to Article 6.10.3.2.2.

$$f_{bu} + f_\ell \leq \phi_f R_h F_{yt} \qquad \text{Eq. (6.10.3.2.2-1)}$$

The tensile flange stress for the Special Load Combination specified in Article 3.4.1.2, calculated without consideration of the lateral bending, f_{bu}, in the top flange is:

$$f_{bu} = 23.08 \text{ ksi} \quad \text{(factored, calculated previously)}$$

The total lateral bending stress due to overhang brackets and curvature effects in the top flange is:

$$f_\ell = 3.57 \text{ ksi} \quad \text{(factored, calculated previously)}$$

The resistance is calculated as follows:

$$\phi_f R_h F_{yt} = (1.0)(1.0)(50.0) = 50.0 \text{ ksi}$$

Therefore,

$$f_{bu} + f_\ell = 23.08 + 3.57 = 26.65 \text{ ksi} < \phi_f R_h F_{yt} = 50.0 \text{ ksi} \quad \text{OK} \quad \text{(Ratio = 0.533)}$$

7.8.2 Constructibility of Bottom Flange

7.8.2.1 Bottom Flange Lateral Bending Amplification

As checked for the top flange in the positive moment region, the bottom flange in the negative moment region must also be checked to determine if a first-order or second-order analysis is appropriate for computing lateral bending stresses since the bottom flange is in compression. According to Article 6.10.1.6, lateral bending stresses determined from a first-order analysis may be used in discretely braced compression flanges for which:

$$L_b \leq 1.2 L_p \sqrt{\frac{C_b R_b}{f_{bu}/F_{yc}}} \qquad \text{Eq. (6.10.1.6-2)}$$

L_p is the limiting unbraced length specified in Article 6.10.8.2.3 determined as:

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = \frac{1.0(7.43)\sqrt{\frac{29{,}000}{50}}}{12} = 14.9 \quad \text{ft} \qquad \text{Eq. (6.10.8.2.3-4)}$$

where r_t is the effective radius of gyration for lateral torsional buckling specified in Article 6.10.8.2.3 determined as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{27}{\sqrt{12\left[1+\frac{1}{3}\frac{(39.56)(0.625)}{27(3)}\right]}} = 7.43 \text{ in.} \qquad \text{Eq. (6.10.8.2.3-9)}$$

C_b is conservatively taken as 1.0 for this computation. Article C6.10.1.10.2 indicates that the web load-shedding factor, R_b, is taken as 1.0 for constructibility.

Check the relation given in Eq. (6.10.1.6-2):

$$L_b = 20 \text{ ft} < 1.2(14.9)\sqrt{\frac{1.0(1.0)}{\frac{20.93}{50}}} = 27.6 \text{ ft}$$

Because Eq. (6.10.1.6-2) is satisfied, Article 6.10.1.6 allows the flange lateral bending stress to be determined directly from a first-order elastic analysis. Therefore, no amplification is required, and as computed earlier forthe Special Load Combination specified in Article 3.4.1.2, the total flange stress due to lateral bending is:

$$f_\ell = 3.20 \text{ ksi}$$

7.8.2.2 Flexure in Bottom Flange (Article 6.10.3.2.1)

During construction (as well as in the final condition), the bottom flange at Section G4-2 is a discretely based compression flange, so the provisions of Article 6.10.3.2.1 apply. Each of the following requirements are checked. The article indicates that if the section has a slender web, Eq. (6.10.3.2.1-1) is not checked when f_ℓ is zero, and for sections with compact or noncompact webs, Eq. (6.10.3.2.1-3) is not checked. In this case, the web is noncompact (as demonstrated later), so only the first two equations must be checked.

$$f_{bu} + f_\ell \leq \phi_f R_h F_{yc} \qquad \text{Eq. (6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \qquad \text{Eq. (6.10.3.2.1-2)}$$

$$f_{bu} \leq \phi_f F_{crw} \qquad \text{Eq. (6.10.3.2.1-3)}$$

where: ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
R_h = hybrid factor specified in Article 6.10.1.10.1 (1.0 at homogeneous Section G4-2)
F_{crw} = nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2 (i.e. local or lateral torsional buckling resistance, whichever controls). The provisions of Article A6.3.3 are not to be used to determine the lateral torsional buckling resistance of top flanges of curved I-girder bridges, per Article 6.10.3.2.1.

Check Eq. (6.10.3.2.1-1) using the previously calculated values of factored flange stresses:

$$f_{bu} + f_\ell = 20.93 + 3.20 = 24.13 \text{ ksi} < \phi_f R_h F_{yc} = 1.0(1.0)(50) = 50 \text{ ksi} \quad \text{OK (Ratio = 0.483)}$$

Secondly, check Eq. (6.10.3.2.1-2). The equation must be satisfied for both local buckling and lateral torsional buckling using the the appropriate value of the nominal flexural resistance, F_{nc}, for local buckling (Article 6.10.8.2.2) or for lateral torsional buckling (Article 6.10.8.2.3), as applicable.

Determine the local buckling resistance of the compression flange. First, check the flange slenderness.

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{27}{2(3)} = 4.5$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15$$

Since $\lambda_f < \lambda_{pf}$, the flange is compact and the nominal flexural resistance is determined using Eq. (6.10.8.2.2-1).

R_b is taken as 1.0 for constructibility checks per Article 6.10.3.2.1, and R_h is taken as 1.0 per Article 6.10.1.10.1. Therefore, F_{nc} for the local buckling resistance is calculated as:

$$F_{nc} = R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.2-1)}$$

$$= (1.0)(1.0)(50) = 50.00 \text{ ksi}$$

Determine the lateral torsional buckling resistance of the compression flange, noting that the unbraced length, L_b, at this location is 20 ft.

L_p = 14.9 ft (calculated previously)

$$L_r = \pi r_t \sqrt{\frac{E}{F_{yr}}} = \frac{\pi(7.43)\sqrt{\frac{29{,}000}{0.7(50)}}}{12} = 56.0 \text{ ft.} \qquad \text{Eq. (6.10.8.2.3-5)}$$

Since $L_p < L_b < L_r$, use Eq. (6.10.8.2.3-2) to calculate the lateral torsional buckling resistance.

$$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.3-2)}$$

$$= 1.0\left[1 - \left[1 - \frac{0.7(50)}{1.0(50)}\right]\left(\frac{20 - 14.9}{56.0 - 14.9}\right)\right](1.0)(1.0)(50) = 48.14 \text{ ksi}$$

Therefore, check Eq. (6.10.3.2.1-2) for local buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell = 20.93 + \frac{1}{3}(3.20) = 22.00 \text{ ksi} < \phi_f F_{nc} = 1.0(50.00) = 50.00 \text{ ksi} \quad \text{OK}$$

(Ratio = 0.440)

Check Eq. (6.10.3.2.1-2) for lateral torsional buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell = 20.93 + \frac{1}{3}(3.20) = 22.00 \text{ ksi} < \phi_f F_{nc} = 1.0(48.14) = 48.14 \text{ ksi} \quad \text{OK}$$

(Ratio = 0.457)

Third, determine if Eq. (6.10.3.2.1-3) must be checked. The slenderness is checked according to Article 6.10.6.2.3 for noncomposite sections:

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{Eq. (6.10.6.2.3-1)}$$

$$\frac{2(39.56)}{0.625} = 126.6 < 5.7\sqrt{\frac{29{,}000}{50}} = 137.3$$

Because the web is noncompact, Eq. (6.10.3.2.1-3) need not be checked.

7.9 Girder Check: Section G4-2, Service Limit State (Article 6.10.4)

Article 6.10.4 contains provisions related to the control of elastic and permanent deformations at the Service Limit State.

7.9.1 Permanent Deformations (Article 6.10.4.2)

Article 6.10.4.2 contains criteria intended to control permanent deformations that would impair rideability. As specified in Article 6.10.4.2.1, these checks are to be made using the Service II load combination.

As stated previously for the Service limit state check of Section G4-1, Article 6.10.4.2.2 requires that flanges of composite sections satisfy the following relations:

Top flange of composite sections: $f_f \le 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-1)

Bottom flange of composite sections: $f_f + \frac{f_\ell}{2} \le 0.95R_hF_{yf}$ Eq. (6.10.4.2.2-2)

However, according to Article C6.10.4.2.2, for composite sections in negative flexure designed as slender-web sections at the strength limit state according to the provisions of Article 6.10.8, and for composite sections in positive flexure designed as noncompact sections at the strength limit state, these two equations do not control and need not be checked. Composite sections in all horizontally curved girder systems are to be treated as slender-web sections in negative flexure and as noncompact sections in positive flexure at the strength limit state, in accordance with Article 6.10.6.2.2 (regardless of their web slenderness). Therefore, for Section G4-2, Eqs. (6.10.4.2.2-1) and (6.10.4.2.2-2) do not need to be checked and are not demonstrated in this example.

7.9.2 Web Bend-Buckling

With the exception of composite sections in positive flexure in which the web satisfies the requirement of Article 6.10.2.1.1 ($D/t_w \le 150$), web bend-buckling of all sections under the Service II load combination is to be checked as follows:

$$f_c \le F_{crw} \qquad \text{Eq. (6.10.4.2.2-4)}$$

The term f_c is the compression-flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending, and F_{crw} is the nominal elastic bend-buckling resistance for webs determined as specified in Article 6.10.1.9. Because Section G4-2 is a section in negative flexure, it must be checked for Eq. (6.10.4.2.2-4).

Determine the nominal web bend-buckling resistance, F_{crw}, for Section G4-2 in accordance with Article 6.10.1.9.1, as follows:

$$F_{crw} = \frac{0.9\,E\,k}{\left(\frac{D}{t_w}\right)^2} \qquad \text{Eq. (6.10.1.9.1-1)}$$

However, F_{crw} shall not exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$. The bend-buckling coefficient, k, is computed as:

$$k = \frac{9}{\left(D_c / D\right)^2} \qquad \text{Eq. (6.10.1.9.1-2)}$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c is to be determined as specified in Article D6.3.1.

In accordance with Article 6.10.4.2.1, for members with shear connectors provided throughout the entire length of the girder that also satisfy Article 6.10.1.7, the concrete deck may be assumed to be effective for both positive and negative flexure, provided that the corresponding longitudinal stresses in the concrete deck at the section under consideration are smaller than $2f_r$, where f_r is the modulus of rupture of concrete specified in Article 5.4.2.6. Article 6.10.1.7 is in regard to the minimum of one percent of longitudinal reinforcement provided in the concrete deck and is satisfied for Section G4-2 in this design example.

$$f_r = 0.24\sqrt{f'_c} \qquad \text{Article 5.4.2.6}$$

Therefore,

$$2f_r = 2\left(0.24\sqrt{4}\right) = 0.960\text{ ksi}$$

In accordance with Article 6.10.1.1.1d, the longitudinal flexural stresses in the concrete deck due to all permanent and transient loads are to be computed using the short-term modular ratio, *n*. The calculated stress on the transformed section is divided by *n* to obtain the longitudinal stress in the concrete deck. Since the deck is not subjected to noncomposite dead loads, the longitudinal stress in the deck at Section G4-2 is due to DC2, DW, and LL+I moments only. The unfactored major-axis bending moments at Section G4-2 are (see Table 9):

Noncomposite Dead Load:	M_{DC1}	= -1,917 + (-7,272) = -9,189 kip-ft
Composite Dead Load:	M_{DC2}	= -1,537 kip-ft
Future Wearing Surface Dead Load:	M_{DW}	= -1,478 kip-ft
Live Load (including IM and CF):	M_{LL+IM}	= -6,726 kip-ft

The longitudinal compressive stress in the deck is computed using the short-term section properties (*n* = 7.56 composite section properties) in accordance with Article 6.10.1.1.1d. Referring to Table 16 of the section property calculations and noting that the total depth of the composite Section G4-2 is 100 inches, the section modulus to the top of the concrete deck is:

$$S_{deck} = \frac{539{,}403}{100.00 - 63.40} = 14{,}738 \text{ in.}^3$$

Calculate the Service II factored longitudinal compressive stress in the deck at this section, noting that the concrete deck is not subjected to noncomposite dead loads. The stress in the concrete deck is obtained by dividing the stress acting on the transformed section by the modular ratio, *n*.

$$f_{deck} = 1.0\left[\frac{1.00(1{,}537) + 1.00(1{,}478) + 1.30(6{,}726)}{(14{,}738)(7.56)}\right]12 = 1.266 \text{ ksi}$$

$$f_{deck} = 1.266 \text{ ksi} > 2f_r = 0.960 \text{ ksi}$$

Since f_{deck} is greater than $2f_r$, for this Service limit state check, the concrete deck cannot be assumed to be effective for negative flexure and the flexural stresses in the steel section caused by the Service II load combination are to be computed using the section consisting of the steel girder and the longitudinal reinforcement within the effective width of the concrete deck. Refer to Table 17 and Table 18 for the composite section properties with longitudinal steel reinforcement. The major-axis bending stress in the top and bottom flange for the Service II load combination are computed as follows (f_t = tension flange, f_c = compression flange):

For Service II:

Top Flange:

$$f_t = 1.0\left[\frac{1.00(9{,}189)}{6{,}689} + \frac{1.00(1{,}537)}{6{,}944} + \frac{1.00(1{,}478)}{6{,}944} + \frac{1.30(6{,}726)}{7{,}146}\right]12 = 36.38 \text{ ksi}$$

Bottom Flange:

$$f_c = 1.0\left[\frac{1.00(9{,}189)}{7{,}377} + \frac{1.00(1{,}537)}{7{,}429} + \frac{1.00(1{,}478)}{7{,}429} + \frac{1.30(6{,}726)}{7{,}523}\right]12 = -33.76 \text{ ksi}$$

In order to compute F_{crw}, it is first necessary to determine D_c, the depth of the web in compression. In accordance with Article D6.3.1, for composite sections in negative flexure where the concrete deck is not permitted to be considered effective in tension at the service limit state, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement. As explained in Article CD6.3.1, for composite sections in negative flexure, the distance between the neutral axis locations for the steel and composite sections is small, and the location of the neutral axis for the composite section is largely unaffected by the dead-load stress. Therefore, D_c is simply computed for the section consisting of the steel girder plus the longitudinal reinforcement. In this example, the section properties from Table 18 are used to compute D_c as follows, where the thickness of the bottom flange is 3 in. (the short-term section is conservatively used):

$$D_c = 44.55 - 3.00 = 41.55 \text{ in.}$$

Compute the bend-buckling coefficient, k:

$$k = \frac{9}{(D_c / D)^2} = \frac{9}{(41.55/84)^2} = 36.78$$

Therefore, the nominal web bend-buckling resistance, F_{crw}, is computed as:

$$F_{crw} = \frac{0.9\,E\,k}{\left(\dfrac{D}{t_w}\right)^2} = \frac{0.9(29{,}000)(36.78)}{\left(\dfrac{84}{0.625}\right)^2} = 53.14 \text{ ksi} > \min\left(R_h F_{yc}, F_{yw}/0.7\right) = 50.0 \text{ ksi}$$

Therefore use F_{crw} = 50.0 ksi.

Verify Eq. (6.10.4.2.2-4):

$$f_c = |-33.76| \text{ ksi} \le F_{crw} = 50.0 \text{ ksi} \qquad \text{OK} \qquad \text{(Ratio = 0.675)}$$

7.10 Girder Check: Section G4-2, Fatigue Limit State (Article 6.10.5)

Article 6.10.5 indicates that details in I-girder section flexural members must be investigated for fatigue as specified in Article 6.6.1. For horizontally curved I-girder bridges, the fatigue stress range due to major-axis bending plus lateral bending is to be considered. As appropriate, the Fatigue I and Fatigue II load combinations specified in Table 3.4.1-1 and the fatigue live load specified in Article 3.6.1.4 are to be employed for checking load-induced fatigue in I-girder sections. The Fatigue I load combination is used when investigating infinite load-induced fatigue life, and the Fatigue II load combination is used when investigating finite load-induced fatigue life.

According to Table 3.6.2.1-1, the dynamic load allowance for fatigue loads is 15%. Centrifugal force effects are considered, and included in the fatigue moments. As discussed previously, the 75-year single lane ADTT is assumed to be 1,000 trucks per day.

7.10.1 Fatigue in Top Flange

At Section G4-2, it is necessary to check the top flange for the fatigue limit state. The base metal at the transverse stiffener weld terminations and interior cross frame connection plate welds at locations subject to a net tensile stress must be checked as a Category C′ fatigue detail per Condition 4.1 in Table 6.6.1.2.3-1. Only the top flange is checked herein, as a net tensile stress is not induced in the bottom flange by the fatigue loading at this location. Also, it should be noted that lateral bending stress in the top flange is not a concern for the fatigue limit state at this section since the deck is in place and continuously braces the top flange.

According to Eq. (6.6.1.2.2-1), the factored fatigue stress range, $\gamma(\Delta f)$, must not exceed the nominal fatigue resistance, $(\Delta F)_n$. In accordance with Article C6.6.1.2.2, the resistance factor, ϕ, and the load modifier, η, are taken as 1.0 for the fatigue limit state.

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad \text{Eq. (6.6.1.2.2-1)}$$

From Table 6.6.1.2.3-2, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category C′ fatigue detail is 745 trucks per day. Therefore, since the assumed $(ADTT)_{SL}$ for this design example of 1,000 trucks per day is greater than this limit of 745 trucks per day, the detail must be checked for infinite fatigue life using the Fatigue I load combination. Per Article 6.6.1.2.5, the nominal fatigue resistance for infinite fatigue life is equal to the constant-amplitude fatigue threshold:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \text{Eq. (6.6.1.2.5-1)}$$

where $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold and is taken from Table 6.6.1.2.5-3. For a Category C′ fatigue detail, $(\Delta F)_{TH}$ = 12.0 ksi, and therefore:

$$(\Delta F)_n = 12.0\ \text{ksi}$$

As shown in Table 9, the unfactored negative and positive moments due to fatigue, including centrifugal force effects and the 15 percent dynamic load allowance, at Section G4-2 are -1,315 kip-ft and 351 kip-ft, respectively.

In accordance with Article 6.6.1.2.1, for flexural members that utilize shear connectors throughout the entire length that also have concrete deck reinforcement satisfying the provisions of Article 6.10.1.7, it is permissible to compute the flexural stresses assuming the concrete deck to be effective for both positive and negative flexure at the fatigue limit state.

As required by Articles 6.10.10.1, shear connectors are necessary along the entire length of horizontally curved continuous composite bridges. Also, earlier calculations in this design example show that the deck reinforcement is in compliance with Article 6.10.1.7. Therefore, the concrete deck is assumed effective in computing the major-axis bending stresses for the fatigue limit state at Section G4-2. From Table 16, the short-term composite section properties (*n* = 7.56) used to compute the stress at the top of the web (bottom of the top flange, where the weld in question is located) are:

$I_{NA(n)}$ = 539,403 in.^{4}

$d_{TOP\ OF\ WEB} = d_{TOP\ OF\ STEEL} - t_{f\_TOP\ FLANGE}$ = 26.10 in. – 2.50 in. = 23.60 in.

Per Table 3.4.1-1, the load factor, γ, for the Fatigue I load combination is 1.5. The factored stress range at the top of the web is computed as follows:

$$\gamma(\Delta f) = (1.5)\left(\frac{(|-1{,}315| + 351)(12)(23.60)}{539{,}403}\right) = 1.31\,\text{ksi}$$

Checking Eq. (6.6.1.2.2-1),

$$\gamma(\Delta f) = 1.31\,\text{ksi} \le (\Delta F)_n = 12.0\,\text{ksi} \quad \text{OK} \quad (\text{Ratio} = 0.109)$$

7.10.2 Special Fatigue Requirement for Webs

In accordance with Article 6.10.5.3, interior panels of stiffened webs must satisfy:

$$V_u \le V_{cr} \qquad \text{Eq. (6.10.5.3-1)}$$

where: V_u = shear in the web at the section under consideration, due to unfactored permanent loads plus the factored fatigue load (Fatigue I live load)

V_{cr} = shear buckling resistance determined from Eq. (6.10.9.3.3-1).

Satisfaction of Eq. (6.10.5.3-1) is intended to control elastic flexing of the web, and the member is assumed to be able to sustain an infinite number of smaller loadings without fatigue cracking due to this effect. The live load shear in the special requirement is supposed to represent the heaviest truck expected to cross the bridge in 75 years.

Only interior panels of stiffened webs are investigated because the shear resistance of end panels of stiffened webs and the shear resistance of unstiffened webs are limited to the shear buckling resistance at the strength limit state.

The unfactored shears at Section G4-1 are shown below. These results are directly from the three-dimensional analysis as reported in Table 10.

Steel Dead Load:	$V_{DC1\text{-}STEEL}$	= -45 kips
Concrete Deck Dead Load:	$V_{DC1\text{-}CONC}$	= -144 kips
Composite Dead Load:	V_{DC2}	= -36 kips

Future Wearing Surface Dead Load: V_{DW} = -28 kips
Total Permanent Load = -253 kips

Fatigue Live Load (incl. IM + CF): V_{LL+IM} = -55 kips

Therefore, the Fatigue I shear in the web is:

$$V_u = -253 + 1.5(-55) = -336 \text{ kips}$$

Next, compute the shear-buckling resistance:

$$V_{cr} = CV_p \qquad \text{Eq. (6.10.9.3.3-1)}$$

where: C = ratio of the shear-buckling resistance to the shear yield strength
V_p = plastic shear force

Compute the plastic shear force:

$$V_p = 0.58 F_{yw} D t_w \qquad \text{Eq. (6.10.9.3.3-2)}$$

$$= 0.58(50)(84)(0.625) = 1{,}523 \text{ kips}$$

To determine the ratio C, the shear-buckling coefficient, k, must first be computed as follows:

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

At this particular location, the transverse stiffener spacing is assumed to be 82 inches. Therefore, d_o = 82 in.

$$k = 5 + \frac{5}{\left[\frac{82}{84}\right]^2} = 10.2$$

Check the following relation in order to select the appropriate equation for computing C:

$$\frac{D}{t_w} = \frac{84}{0.625} = 134.4 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(10.2)}{50}} = 108$$

Since the above relation is true, the ratio C is computed using Eq. (6.10.9.3.2-6) as follows:

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

$$C = \frac{1.57}{\left(\frac{84}{0.625}\right)^2}\left(\frac{29{,}000(10.2)}{50}\right) = 0.514$$

The shear-buckling resistance is then computed in accordance with Eq. (6.10.9.3.3-1):

$$V_{cr} = (0.514)(1{,}523) = 783 \text{ kips}$$

Using the above results, check the requirement of Article 6.10.5.3, $V_u \le V_{cr}$:

$$V_u = |-336| \text{ kips} \le V_{cr} = 783 \text{ kips} \quad \text{OK}$$

Therefore, the web is satisfactory for fatigue at the maximum negative moment location.

7.11 Girder Check: Section G4-2, Strength Limit State (Article 6.10.6)

7.11.1 Flexure (Article 6.10.6.2)

According to Article 6.10.6.2.3, composite sections in negative flexure in horizontally curved steel girder bridges are to be treated as slender-web sections at the strength limit state regardless of their web slenderness and must therefore satisfy the requirements of Article 6.10.8.

Composite sections in negative flexure must satisfy the provisions of Article 6.10.8.1. At the strength limit state, the compression flange must satisfy:

$$f_{bu} + \frac{1}{3}f_\ell \le \phi_f F_{nc} \qquad \text{Eq. (6.10.8.1.1-1)}$$

where:

- f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6
- ϕ_f = resistance factor for flexure = 1.0 (Article 6.5.4.2)
- F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.8.2

Per Article 6.10.8.1.3 for continuously braced flanges, at the strength limit state, the tension flange must satisfy:

$$f_{bu} \le \phi_f R_h F_{yf} \qquad \text{Eq. (6.10.8.1.3-1)}$$

It should be noted that flange lateral bending is not considered for the tension flange at the strength limit state in this case because the flange is continuously supported by the hardened concrete deck.

7.11.1.1 Strength I Flexural Stress in Top and Bottom Flange

The unfactored bending moments at Section G4-2 from the analysis are shown below (see Table 9). The live load moment includes the centrifugal force and dynamic load allowance effects.

Noncomposite Dead Load:	M_{DC1}	= -1,917 + (-7,272) = -9,189 kip-ft
Composite Dead Load:	M_{DC2}	= -1,537 kip-ft
Future Wearing Surface Dead Load:	M_{DW}	= -1,478 kip-ft
Live Load (including IM and CF):	M_{LL+IM}	= -6,726 kip-ft

Compute the factored flange flexural stresses at Section G4-2 for the Strength I load combination, without consideration of flange lateral bending. As discussed previously, the η factor is taken equal to 1.0 in this example. In accordance with Article 6.10.1.1.1c, the flexural stresses are computed using section properties based on a composite section consisting of the steel section and the longitudinal reinforcement within the effective width of the concrete deck (refer to Table 17 and Table 18). Therefore:

For Strength I, the bending stresses due to vertical loads are as follows:

Top Flange (tension):

$$f_{bu} = \left[\frac{(\gamma_{DC1} M_{DC1})}{S_{nc}} + \frac{[(\gamma_{DC2} M_{DC2} + \gamma_{DW} M_{DW})]}{S_{3n}} + \frac{(\gamma_{LL} M_{LL})}{S_n}\right](12)\eta$$

$$= \left[\frac{1.25(9{,}189)}{6{,}689} + \frac{[1.25(1{,}537) + 1.5(1{,}478)]}{6{,}944} + \frac{1.75(6{,}726)}{7{,}146}\right](12)(1) = 47.52 \text{ ksi}$$

Bottom Flange (compression):

$$f_{bu} = \left[\frac{(\gamma_{DC1} M_{DC1})}{S_{nc}} + \frac{[(\gamma_{DC2} M_{DC2} + \gamma_{DW} M_{DW})]}{S_{3n}} + \frac{(\gamma_{LL} M_{LL})}{S_n}\right](12)\eta$$

$$= \left[\frac{1.25(9{,}189)}{7{,}377} + \frac{[1.25(1{,}537) + 1.5(1{,}478)]}{7{,}429} + \frac{1.75(6{,}726)}{7{,}523}\right](12)(1) = -44.14 \text{ ksi}$$

As required to check the discretely braced compression flange, the lateral bending stress must also be calculated for the bottom flange. Using the moments shown above, the unfactored lateral bending moment and corresponding first-order lateral bending stress are calculated as follows:

$$M_{lat} = \frac{M\ell^2}{NRD} \quad \text{Eq. (C4.6.1.2.4b-1)}$$

$$f_\ell = \frac{M_{lat}}{S_{bot\_flange}}, \text{ where } S_{bot\_flange} = \frac{(3.0)(27)^2}{6} = 364.5 \text{ in.}^3$$

$$M_{lat\_DC1} = \frac{9{,}189(20)^2}{12(716.5)(7)} = 61.07 \text{ kip} - \text{ft} \qquad f_{\ell\_DC1} = \frac{M_{lat\_DC1}}{S_{bot\_fl}} = \frac{61.07(12)}{364.5} = 2.01 \text{ ksi}$$

$$M_{lat\_DC2} = \frac{1{,}537(20)^2}{12(716.5)(7)} = 10.22 \text{ kip} - \text{ft} \qquad f_{\ell\_DC2} = \frac{M_{lat\_DC2}}{S_{bot\_fl}} = \frac{10.22(12)}{364.5} = 0.34 \text{ ksi}$$

$$M_{lat\_DW} = \frac{1{,}478(20)^2}{12(716.5)(7)} = 9.83 \text{ kip} - \text{ft} \qquad f_{\ell\_DW} = \frac{M_{lat\_DW}}{S_{bot\_fl}} = \frac{9.83(12)}{364.5} = 0.32 \text{ ksi}$$

$$M_{lat\_LL} = \frac{6{,}726(20)^2}{12(716.5)(7)} = 44.70 \text{ kip} - \text{ft} \qquad f_{\ell\_LL} = \frac{M_{lat\_LL}}{S_{bot\_fl}} = \frac{44.70(12)}{364.5} = 1.47 \text{ ksi}$$

As investigated for the bottom flange constructibility checks for Section G4-2, the bottom flange for the strength limit state may be subject to lateral bending amplification. The flange lateral bending stress, f_ℓ, may be determined directly from first-order elastic analysis if the following relation is satisfied:

$$L_b \le 1.2 L_p \sqrt{\frac{C_b R_b}{f_{bu}/F_{yc}}} \qquad \text{Eq. (6.10.1.6-2)}$$

The limiting unbraced length, L_p, was computed previously in the constructibility check as 14.9 ft. Per Article 6.10.1.10.2, R_b is to be taken as 1.0 if the web satisfies:

$$\frac{2D_c}{t_w} \le \lambda_{rw} \qquad \text{Eq. (6.10.1.10.2-2)}$$

For the strength limit state and in accordance with Article D6.3.1, for composite sections in negative flexure, D_c is to be computed for the section consisting of the steel girder plus the longitudinal reinforcement (the short-term section is conservatively used). Referring to Table 18, D_c is taken as:

D_c = 44.55 - 3.0 = 41.55 in.

Therefore,

$$\frac{2D_c}{t_w} = \frac{2(41.55)}{0.625} = 133.0$$

Compute the limiting slenderness ratio for a noncompact web:

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29{,}000}{50}} = 137.3 \qquad \text{Eq. (6.10.1.10.2-4)}$$

Eq. (6.10.1.10.2-2) is satisfied:

$$\frac{2D_c}{t_w} = 133.0 < \lambda_{rw} = 137.3$$

Therefore, $R_b = 1.0$. Check Eq. (6.10.1.6-2) assuming $C_b = 1.0$:

$$L_b = 20\text{ ft} > 1.2(14.9)\sqrt{\frac{(1.0)(1.0)}{(44.14)/(50)}} = 19.0\text{ ft}$$

Since Eq. (6.10.1.6-2) is not satisfied, the second-order elastic compression-flange lateral bending stresses must be considered. The first-order values may be amplified as follows:

$$f_\ell = \left(\frac{0.85}{1 - \frac{f_{bu}}{F_{cr}}}\right) f_{\ell 1} \geq f_{\ell 1} \quad \text{(second-order analysis)} \qquad \text{Eq. (6.10.1.6-4)}$$

where: f_{bu} = bottom flange stress calculated without consideration of flange lateral bending
F_{cr} = elastic lateral torsional buckling stress for the flange under consideration determined using Eq. (6.10.8.2.3-8)

$$F_{cr} = \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \qquad \text{Eq. (6.10.8.2.3-8)}$$

where r_t is the effective radius of gyration for lateral torsional buckling specified in Article 6.10.8.2.3 determined as:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = \frac{27}{\sqrt{12\left[1+\frac{1}{3}\frac{(41.55)(0.625)}{27(3)}\right]}} = 7.41 \text{ in.} \qquad \text{Eq. (6.10.8.2.3-9)}$$

Using Eq. (6.10.8.2.3-8), compute the elastic lateral torsional buckling stress, F_{cr}:

$$F_{cr} = \frac{1.0(1.0)(\pi^2)(29{,}000)}{\left[\frac{20(12)}{7.41}\right]^2} = 272.8 \text{ ksi}$$

The amplification factor (AF) is then determined as follows:

$$AF = \left(\frac{0.85}{1-\frac{44.14}{272.8}}\right) = 1.01 > 1.00 \text{ OK}$$

Therefore, the total factored lateral bending stress at the bottom flange, including the amplification factor, is:

$$f_\ell = 1.01[1.25(2.01+0.34)+1.5(0.32)+1.75(1.47)] = 6.05 \text{ ksi}$$

7.11.1.2 Top Flange Flexural Resistance in Tension

As stated previously, the continuously braced top flange must satisfy:

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad \text{Eq. (6.10.8.1.3-1)}$$

For Strength I:

$$f_{bu} = 47.52 \text{ ksi} < \phi_f R_h F_{yf} = 1.0(1.0)(50) = 50 \text{ ksi} \qquad \text{OK (Ratio = 0.950)}$$

7.11.1.3 Bottom Flange Flexural Resistance in Compression

For discretely braced compression flanges at the strength limit state, Eq. (6.10.8.1.1-1) must be satisfied for both local buckling and lateral torsional buckling using the the appropriate value of the nominal flexural resistance, F_{nc}, for local buckling (Article 6.10.8.2.2) or for lateral torsional buckling (Article 6.10.8.2.3), as applicable.

Per Article 6.10.8.2.2, if $\lambda_f \le \lambda_{pf}$, then the local buckling resistance of the compression flange is to be taken as:

$$F_{nc} = R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.2-1)}$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1.

Compute the slenderness ratio for the compression flange:

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{27}{2(3.0)} = 4.50 \qquad \text{Eq. (6.10.8.2.2-3)}$$

Compute the limiting slenderness ratio for a compact flange:

$$\lambda_{pf} = 0.38\sqrt{\frac{29{,}000}{50}} = 9.15 \qquad \text{Eq. (6.10.8.2.2-4)}$$

$$\lambda_f = 4.50 < \lambda_{pf} = 9.15$$

Therefore, $F_{nc} = R_b R_h F_{yc}$

For a homogenous girder, the hybrid factor, R_h, is equal to 1.0. As shown earlier, the web load-shedding factor, R_b, is equal to 1.0. Therefore, F_{nc} for the local buckling resistance is calculated as:

$$F_{nc} = (1.0)(1.0)(50.00) = 50.00\text{ ksi}$$

Next, determine the lateral torsional buckling resistance of the compression flange, noting that the unbraced length, L_b, is 20 ft.

L_p = 14.9 ft (calculated previously in the bottom flange lateral bending amplification check for constructibility)

$$L_r = \pi r_t\sqrt{\frac{E}{F_{yr}}} = \frac{\pi(7.43)\sqrt{\dfrac{29{,}000}{0.7(50)}}}{12} = 56.0\text{ ft.} \qquad \text{Eq. (6.10.8.2.3-5)}$$

Since $L_p < L_b < L_r$, use Eq. (6.10.8.2.3-2) to calculate the lateral torsional buckling resistance. C_b is conservatively assumed as 1.0.

$$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] R_b R_h F_{yc} \leq R_b R_h F_{yc} \qquad \text{Eq. (6.10.8.2.3-2)}$$

$$= 1.0 \left[1 - \left[1 - \frac{0.7(50)}{1.0(50)} \right] \left(\frac{20 - 14.9}{56.0 - 14.9} \right) \right] (1.0)(1.0)(50) = 48.14 \text{ ksi}$$

Check Eq. (6.10.8.1.1-1) for local buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \qquad \text{Eq. (6.10.8.1.1-1)}$$

$$f_{bu} + \frac{1}{3} f_\ell = \left| -44.14 \right| + \frac{1}{3}(6.05) = 46.16 \text{ ksi} < \phi_f F_{nc} = 1.0(50.00) = 50.00 \text{ ksi} \quad \text{OK (Ratio = 0.923)}$$

Check Eq. (6.10.8.1.1-1) for lateral torsional buckling as follows:

$$f_{bu} + \frac{1}{3} f_\ell = \left| -44.14 \right| + \frac{1}{3}(6.05) = 46.16 \text{ ksi} < \phi_f F_{nc} = 1.0(48.14) = 48.14 \text{ ksi} \quad \text{OK (Ratio = 0.959)}$$

7.11.2 Web Shear Strength (Article 6.10.9)

According to the provisions of Article 6.10.9.1, at the strength limit state, straight and curved web panels must satisfy:

$$V_u \leq \phi_v V_n \qquad \text{Eq. (6.10.9.1-1)}$$

where:

ϕ_v = resistance factor for shear = 1.0 (Article 6.5.4.2)
V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively
V_u = factored shear in the web at the section under consideration

Since the web at Support 1 is an interior panel, Article 6.10.9.3.2 applies, and the nominal shear resistance is to be taken as:

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

where:

d_o = transverse stiffener spacing
V_n = nominal shear resistance of the web panel
V_p = plastic shear force
C = ratio of the shear-buckling resistance to the shear yield strength

The above shear resistance applies provided that the following proportional requirement is satisfied:

$$\frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} \leq 2.5 \qquad \text{Eq. (6.10.9.3.2-1)}$$

Checking the above equation for Section G4-2:

$$\frac{2(84)(0.625)}{\left[(27)(3.0) + (28)(2.5)\right]} = 0.70 < 2.5 \quad \text{OK}$$

Therefore, the equation for V_n shown above applies for the web panel of Section G4-2.

7.11.2.1 Applied Shear

The unfactored shears for Girder G4 at Support 2 are shown below. These results are taken directly from the three-dimensional analysis as reported in Table 10.

Steel Dead Load:	$V_{DC1\text{-}STEEL}$	= -45 kips
Concrete Deck Dead Load:	$V_{DC1\text{-}CONC}$	= -144 kips
Composite Dead Load:	V_{DC2}	= -36 kips
Future Wearing Surface Dead Load:	V_{DW}	= -28 kips
Live Load (including IM + CF):	V_{LL+IM}	= -159 kips

The maximum Strength I factored shear is computed as:

$$V_u = 1.25(-45 - 144 - 36) + 1.50(-28) + 1.75(-159) = -602\ \text{kips}$$

7.11.2.2 Shear Resistance

Compute the plastic shear force:

$$V_p = 0.58F_{yw}Dt_w \qquad \text{Eq. (6.10.9.3.2-3)}$$

$$= 0.58(50)(84)(0.625) = 1{,}523 \text{ kips}$$

To determine the ratio C, the shear-buckling coefficient, k, must first be computed as follows:

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

At this particular location, the transverse stiffener spacing is assumed to be 82 inches. Therefore, d_o = 82 in.

$$k = 5 + \frac{5}{\left[\frac{82}{84}\right]^2} = 10.2$$

Check the following relation in order to determine the appropriate equation for computing C:

$$\frac{D}{t_w} = \frac{84}{0.625} = 134.4 > 1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{29{,}000(10.2)}{50}} = 108$$

Since the above relation is true, the ratio C is computed using Eq. (6.10.9.3.2-6) as follows:

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

$$C = \frac{1.57}{\left(\frac{84}{0.625}\right)^2}\left(\frac{29{,}000(10.2)}{50}\right) = 0.514$$

The nominal shear resistance is then computed in accordance with Eq. (6.10.9.3.2-2):

$$V_n = V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] = (1{,}523)\left[0.514 + \frac{0.87(1-0.514)}{\sqrt{1+\left(\frac{82}{84}\right)^2}}\right] = 1{,}244 \text{ kips}$$

Using the above results, check the requirement of Article 6.10.9.1, $V_u \le \phi_v V_n$:

$$V_u = \left|-602 \text{ kips}\right| \le \phi_v V_n = (1.0)(1{,}244) = 1{,}244 \text{ kips} \quad \text{OK (Ratio = 0.484)}$$

Therefore, the web of Section G4-2 is satisfactory for shear at Support 2.

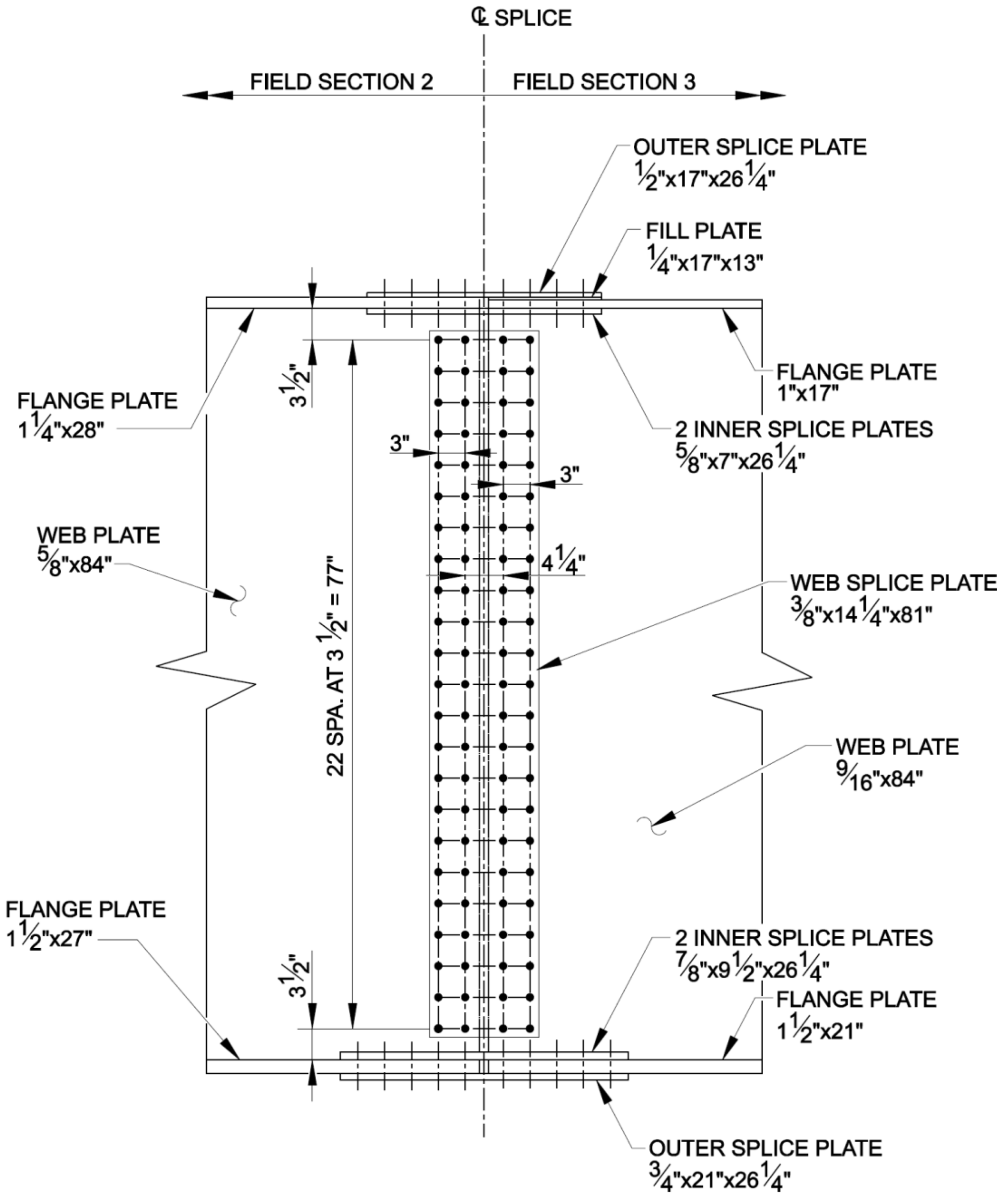
7.12 Bolted Field Splice

7.12.1 General

This section will show the design of a bolted field splice in accordance with the provisions of Article 6.13.6. The design computations will be illustrated for the Field Splice #2 on Girder G4. First, single bolt capacities are computed for slip resistance (Article 6.13.2.8) and shear resistance (Article 6.13.2.7), and then the bearing resistance on the connected material is computed (Article 6.13.2.9). The tensile resistance (Article 6.13.2.10) of a single bolt is also computed for completeness but is not used in this example. The field splice is then checked for constructibility, the service limit state, and the strength limit state.

All bolts used in the field splice are 0.875-inch diameter ASTM A325 bolts. Table 6.13.2.4.2-1 shows that a standard hole diameter size for a 0.875-inch diameter bolt is 0.9375 inch. The connection is designed assuming that a Class B surface condition is provided and that the surface is unpainted and blast-cleaned. Also, it is assumed that the bolt threads are excluded from the shear planes in all connections.

Article 6.13.6.1.4a requires at least two rows of bolts on each side of the joint. Thus, four rows of four bolts are selected for each flange, and two vertical rows of 23 bolts per web are selected for the web splice plate. Oversize or slotted holes in either the member or the splice plates are not permitted. The elevation view of the bolted field splice being investigated is shown in Figure 9, and views of the top and bottom flange splice plates are shown in Figure 10 and Figure 11, respectively.



NOTES:

1. ALL BOLTS ARE ⅞" DIA (ASTM A325) H.S. BOLTS.
2. A 0.25 IN GAP IS ASSUMED BETWEEN THE EDGES OF THE FIELD PIECES.

Figure 9 Bolted Field Splice in Span 2 of G4 – Elevation View

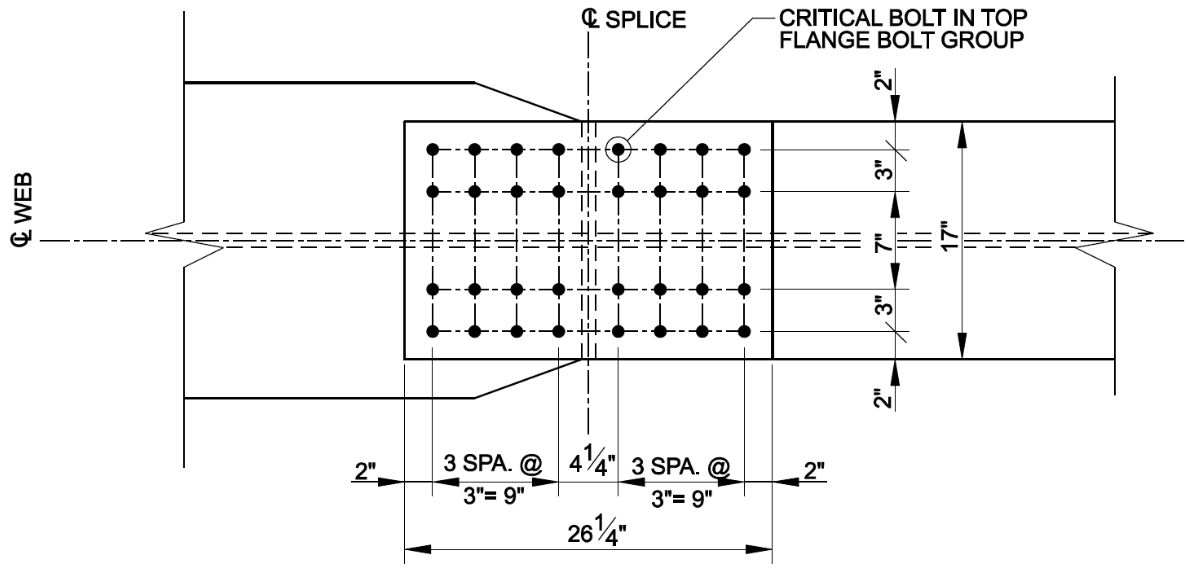


Figure 10 Bolted Field Splice in Span 2 of G4 – Top Flange

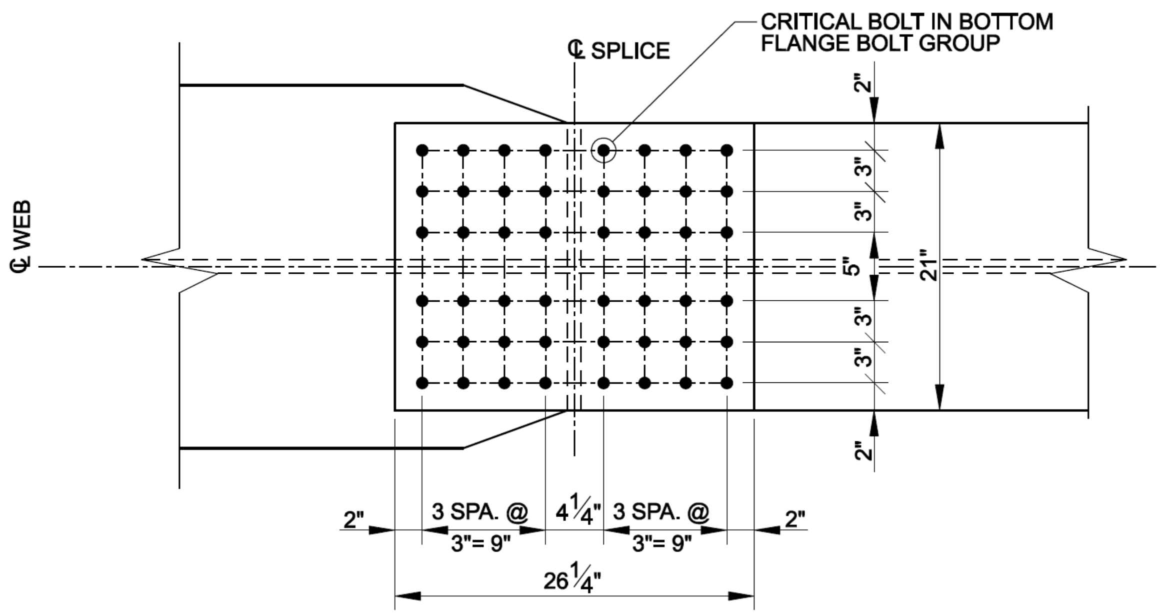


Figure 11 Bolted Field Splice in Span 2 of G4 – Bottom Flange

7.12.2 Resistance Calculation for the Service Limit State and Constructibility

For slip-critical connections, the factored resistance, R_r, of a bolt at the Service II load combination is taken as:

$R_r = R_n$ Eq. (6.13.2.2-1)

where: R_n = the nominal resistance as specified in Article 6.13.2.8

The nominal slip resistance of a bolt in a slip-critical connection shall be taken as:

$$R_n = K_h K_s N_s P_t$$ Eq. (6.13.2.8-1)

where: N_s = number of slip planes per bolt
P_t = minimum required bolt tension specified in Table 6.13.2.8-1
K_h = hole size factor specified in Table 6.13.2.8-2
K_s = surface condition factor specified in Table 6.13.2.8-3

For all bolts in this connection:

- N_s = 2 since each connection has two slip planes.
- P_t = 39 kips for A325, 0.875-inch bolts.
- K_h = 1.0 since standard size holes are used.
- K_s = 0.50 since a Class B surface preparation is assumed for this design example.

Therefore, the slip resistance of a single bolt for service and constructibility checks is:

$$R_r = R_n = (1.0)(0.50)(2)(39) = 39 \text{ kips/bolt}$$

7.12.3 Resistance Calculations for the Strength Limit State

The factored resistance, R_r, of a bolted connection at the strength limit state is taken as

$R_r = \phi R_n$ Eq. (6.13.2.2-2)

where: ϕ = resistance factor for bolts specified in Article 6.5.4.2

The nominal resistance of the bolted connection must be computed for three types of strength: shear, bearing, and tension, where applicable.

Article 6.13.6.1.4a states that the factored flexural resistance of the flanges at the point of the splice at the strength limit state must satisfy the applicable provisions of Article 6.10.6.2, which relate to flexure. The girder satisfies the provisions of Article 6.10.6.2 at the splice location; however, the checks at this particular location are not included in this example.

7.12.3.1 Bolt Shear Resistance (Article 6.13.2.7)

The nominal shear resistance, R_n, of a high-strength bolt at the strength limit state where threads are excluded from the shear plane is computed as follows:

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

where: A_b = area of bolt corresponding to the nominal diameter
F_{ub} = specified minimum tensile strength of the bolt per Article 6.4.3
N_s = number of shear planes per bolt

R_n = 0.48(0.601)(120)(2) = 69.2 kips/bolt

The factored shear resistance at the strength limit state is taken as:

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

where: ϕ_s = shear resistance factor for bolts in shear from Article 6.5.4.2

R_r = 0.8(69.2) = 55.4 kips/bolt

7.12.3.2 Bearing Resistance on Connected Material (Article 6.13.2.9)

The nominal bearing resistance of interior and end bolt holes at the strength limit, R_n, is taken as one of the following two terms, depending on the bolt clear distance and the clear end distance.

(1) With bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

$$R_n = 2.4dtF_u \qquad \text{Eq. (6.13.2.9-1)}$$

(2) If either the clear distance between holes is less than 2.0d or the clear end distance is less than 2.0d:

$$R_n = 1.2L_ctF_u \qquad \text{Eq. (6.13.2.9-2)}$$

where: d = nominal diameter of the bolt (in.)
t = thickness of the connected material (in.)
F_u = tensile strength of the connected material specified in Table 6.4.1-1 (ksi)
L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied force

For example, in the case of the web, the end distance is 2.0 inches. According to Article 6.8.3, the width of each standard bolt hole for design is to be taken as the nominal diameter of the hole = 0.9375″, creating a clear end distance of 1.53 inches, which is less than 2.0d. Therefore, Eq. (6.13.2.9-2) applies. The thinner of the two webs is used for the thickness, t. The nominal bearing resistance for the end row of bolts in the web is:

R_n = 1.2(1.53)(0.5625)(65) = 67.13 kips/bolt

The factored resistance is:

$R_r = \phi_{bb}R_n$ Eq. (6.13.2.2-2)

where: ϕ_{bb} = shear resistance factor for bolts bearing on material (Article 6.5.4.2)

$R_r = 0.8(67.13) = 53.7$ kips/bolt

7.12.3.3 Tensile Resistance (Article 6.13.2.10)

The nominal tensile strength of a bolt, T_n, independent of any initial tightening force, is to be taken as:

$$T_n = 0.76A_bF_{ub} \qquad \text{Eq. (6.13.2.10.2-1)}$$

$$T_n = 0.76(0.601)(120) = 54.8 \text{ kips/bolt}$$

The tensile bolt strength is not used in this example.

7.12.4 Constructibility Checks

According to Article 6.13.6.1.4a, connections must be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck. Since Cast #1 causes a negative moment at the splice location that is larger than the moment assuming a single placement of the entire deck, Steel + Cast #1 controls. For constructibility, the load factor is 1.4 according to the special load combination specified in Article 3.4.2.1.

Article 6.13.6.1.4c requires that lateral bending effects be considered in the design of curved girder splices. Since the flange is discretely braced for this case, flange lateral bending must be considered. To account for the effects of flange lateral bending, the flange splice bolts will be designed for the combined effects of shear and moment using the traditional elastic vector method. The shear on the bolts is caused by the flange force calculated from the average major-axis bending stress in the flange, and the moment on the bolt group is caused by the flange lateral bending moment.

7.12.4.1 Constructibility of Top Flange

To check constructibility of the top flange, first compute the polar moment of inertia of the top flange bolt pattern, shown in Figure 10. The bolt pattern consists of the 16 bolts in the flange on one side of the connection.

$$I_p = 2(4)\left(3.5^2 + 6.5^2\right) + 2(4)\left(1.5^2 + 4.5^2\right) = 616 \text{ in.}^2$$

Compute the unfactored major-axis bending moment and the unfactored flange lateral moment. Using the results listed in Table 9, the major-axis moment for Steel + Cast #1 is computed as follows:

$$\text{Major-Axis Moment} = -382 + (-1{,}910) = -2{,}292 \text{ kip} - \text{ft}$$

Using the major-axis bending moments from Table 9 and Eq. (C4.6.1.2.4b-1), compute the flange lateral bending moment for Steel + Cast #1

$$M_{lat} = \frac{[-382 + (-1{,}910)](20)^2}{12(716.5)(7)} = 15.2 \text{ kip} - \text{ft}$$

The section properties of Field Section 3 of Girder 4 are used to compute the bending stresses since Field Section 3 is the smaller of the two girder sections connected by the splice. The calculations of the section properties are not shown here, but they are computed as demonstrated earlier in this example for other girder sections.

The flange stress due to major-axis bending can be computed at the midthickness of the flange. Herein, this flange stress is computed by taking the average of the stress at the top of the top flange and the top of the web. It is conservative to use only the flange stresses at the outer edge of the flange and not at the midthickness. The factored major-axis bending stresses for the special load combination specified in Article 3.4.2.1 are computed as follows:

$$f_{topflg} = \left(\frac{-2{,}292}{-2{,}262}\right)(12)(1.4) = 17.02 \text{ ksi}$$

$$f_{topweb} = \left(\frac{-2{,}292}{-2{,}308}\right)(12)(1.4) = 16.68 \text{ ksi}$$

Compute the force in the top flange using the average major-axis bending stress in the flange. The gross section of the flange is used to check for slip.

$$F_{top} = \left(\frac{17.02 + 16.68}{2}\right)(17.0)(1.0) = 286 \text{ kips}$$

Compute the longitudinal force in each bolt resulting from the major-axis bending stress by dividing by the number of bolts on one side of the top flange splice:

$$F_{Longvert} = \frac{286}{16} = 17.9 \text{ kips/bolt}$$

Compute the factored longitudinal component of force in the critical bolt due to the flange lateral moment, noting that the transverse distance from the centroid of the bolt group to the critical bolt is 6.5 inches:

$$F_{Longlat} = \frac{15.2(6.5)}{616}(12)(1.4) = 2.69 \text{ kips/bolt}$$

Therefore, the total longitudinal force is computed as:

$$F_{Longtot} = 17.9 + 2.69 = 20.6 \text{ kips/bolt}$$

Compute the factored transverse component of force in the critical bolt due to the flange lateral bending moment, noting that the longitudinal distance from the centroid of the bolt group to the critical bolt is 4.5 inches:

$$F_{Trans} = \frac{15.2(4.5)}{616}(12)(1.4) = 1.87 \text{ kips/bolt}$$

Compute the resultant force on the critical bolt:

$$\sum F = \sqrt{20.6^2 + 1.87^2} = 20.7 \text{ kips/bolt}$$

Check $R_u \leq R_r$, where R_r equals the factored slip resistance of one bolt (calculated previously):

R_u = 20.7 kips/bolt < R_r = 39 kips/bolt OK

7.12.4.2 Constructibility of Bottom Flange

As stated previously, Cast #1 causes a negative moment at the splice location that is larger than the moment assuming a single placement of the entire deck, so Steel + Cast #1 controls for constructibility, and the appropriate load factor is 1.4 per the special load combination specified in Article 3.4.2.1.

Similar to the check of the top flange, the section properties of Field Section 3 of Girder 4 are used to compute the bending stresses at the bottom flange (section property calculations not shown). The factored major-axis bending stresses for the special load combination specified in Article 3.4.2.1 are computed as follows:

$$f_{botflg} = \left(\frac{-2{,}292}{3{,}029}\right)(12)(1.4) = -12.71 \text{ ksi}$$

$$f_{botweb} = \left(\frac{-2{,}292}{3{,}157}\right)(12)(1.4) = -12.20 \text{ ksi}$$

Compute the force in the bottom flange using the average major-axis bending stress in the flange. The gross section of the flange is used to check for slip.

$$F_{bot} = \left[\frac{-12.71 + (-12.20)}{2}\right](21)(1.5) = -392 \text{ kips}$$

Compute the longitudinal force in each bolt resulting from the major-axis bending stress by dividing by the number of bolts on one side of the bottom flange splice:

$$F_{Longvert} = \frac{392}{24} = 16.33 \text{ kips/bolt}$$

Compute the polar moment of inertia of the bottom flange bolt pattern shown in Figure 11. The bolt pattern consists of the 24 bolts in the flange on one side of the connection.

$$I_p = 2(6)\left(1.5^2 + 4.5^2\right) + 2(4)\left(2.5^2 + 5.5^2 + 8.5^2\right) = 1{,}140 \text{ in.}^2$$

Compute the factored longitudinal component of force in the critical bolt due to the flange lateral moment, noting that the transverse distance from the centroid of the bolt group to the critical bolt is 8.5 inches:

$$F_{Longlat} = \frac{15.2(8.5)}{1{,}140}(12)(1.4) = 1.90 \text{ kips/bolt}$$

Therefore, the total longitudinal force is computed as:

$$F_{Longtot} = 16.33 + 1.90 = 18.2 \text{ kips/bolt}$$

Compute the factored transverse component of force in the critical bolt due to the flange lateral bending moment, noting that the longitudinal distance from the centroid of the bolt group to the critical bolt is 4.5 inches:

$$F_{Trans} = \frac{15.2(4.5)}{1{,}140}(12)(1.4) = 1.01 \text{ kips/bolt}$$

Compute the resultant force on the critical bolt:

$$\sum F = \sqrt{18.2^2 + 1.01^2} = 18.2 \text{ kips/bolt}$$

Check $R_u \leq R_r$, where R_r equals the factored slip resistance of one bolt (calculated previously):

R_u = 18.2 kips/bolt < R_r = 39 kips/bolt OK

7.12.4.3 Constructibility of Web

Article 6.13.6.1.4a directs the designer to check the bolted splice to prevent slip in the bolts during the erection of the steel and during the casting of the concrete deck. A pattern of two rows of 7/8 in. diameter bolts spaced vertically at 3.5 in. are selected for the web splice. There are 46 bolts on each side of the web splice. The pattern is shown in Figure 9. Although not illustrated here, the number of bolts in the web splice could be decreased by spacing a group of bolts closer to the mid-depth of the web (where flexural stress is relatively low) at the maximum specified spacing for sealing (see Article 6.13.2.6.2), and by spacing the remaining two groups of bolts near the top and bottom of the web at a closer spacing. Note that there are 3.5 inches between the inside of the flanges and the first bolt to provide sufficient assembly clearance. 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.

Compute the polar moment of inertia of the web bolts about the centroid of the bolt group on one side of the connection.

$$I_p = \left[2(2)\left(3.5^2 + 7.0^2 + 10.5^2 + 14.0^2 + 17.5^2 + 21.0^2 + 24.5^2 + 28.0^2 + 31.5^2 + 35.0^2 + 38.5^2\right)\right]$$
$$+ \left[46(1.5)^2\right] = 24{,}898\ \text{in}^2$$

An alternate equation to compute I_p is provided in Article C6.13.6.1.4b.

Compute the factored shear at the splice due to Steel plus Cast #1 and Cast #2 for the special load combination specified in Article 3.4.2.1, as this is the governing shear for constructibility. The unfactored shears are taken from Table 10, and Cast #2 is conservatively included in the calculation.

$$V = (1.4)(27 + 7 + 92) = 176\ \text{kips}$$

Compute the factored moment, M_v, due to the eccentricity of the factored shear about the centroid of the connection (refer to the web bolt pattern in Figure 9).

$$M_V = V \text{ x } e = 176\left(\frac{3}{2} + 2.125\right)\left(\frac{1}{12}\right) = 53.2\ \text{kip} - \text{ft}$$

Determine the portion of the major-axis bending moment resisted by the web, M_{uw}, and the horizontal force resultant in the web, H_{uw}, using the equations provided in Article C6.13.6.1.4b. M_{uw} and H_{uw} are assumed to be applied at the middepth of the web. Using the factored major-axis bending stresses calculated previously, the average factored bending stresses in the top and bottom flanges for Steel plus Cast #1 are computed as follows:

Top flange:

$$F_{cf} = \left(\frac{17.02 + 16.68}{2}\right) = 16.85\ \text{ksi (T) (controlling flange)}$$

Bottom flange:

$$f_{ncf} = \left[\frac{-12.71 + (-12.20)}{2}\right] = -12.45 \text{ ksi (C)}$$

where: F_{cf} = design stress for controlling flange at the point of splice specified in Article 6.13.6.1.4c; positive for tension, negative for compression

f_{ncf} = flexural stress due to the factored loads at midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf}; positive for tension, negative for compression

Since the absolute stress in the top flange is greater than the absolute stress in the bottom flange, the top flange is the controlling flange.

Using these bending stresses, compute M_{uw} and H_{uw}:

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

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

where: R_{cf} = the absolute value of the ratio of F_{cf} to the maximum flexural stress, f_{cf}, due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in Article 6.13.6.1.4c

R_h = hybrid factor, equal to 1.0 in this example

As indicated in Article C6.13.6.1.4b, the ratio R_{cf} is equal to 1.0 in this case since the equations for M_{uw} and H_{uw} are being used to check slip.

$$M_{uw} = \frac{0.5625(84)^2}{12}\left|1.0(16.85) - 1.0(-12.45)\right|\left(\frac{1}{12}\right) = 808 \text{ kip} - \text{ft}$$

$$H_{uw} = \frac{0.5625(84)}{2}\left[1.0(16.85) + 1.0(-12.45)\right] = 104.0 \text{ kips}$$

The total moment on the web splice is computed as follows:

$$M_{tot} = M_v + M_{uw} = 53.2 + 808 = 861 \text{ kip} - \text{ft}$$

Compute the vertical bolt force due to the factored shear by dividing the shear by the number of bolts on one side of the web splice:

$$F_s = \frac{V}{N_b} = \frac{176}{46} = 3.83 \text{ kips/bolt}$$

Compute the bolt force due to the horizontal force resultant by dividing the horizontal force by the number of bolts on one side of the web splice:

$$F_H = \frac{H_{uw}}{N_b} = \frac{104.0}{46} = 2.26 \text{ kips/bolt}$$

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

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{861(12)\left(\frac{3}{2}\right)}{24{,}898} = 0.62 \text{ kips/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{861(12)(38.5)}{24{,}898} = 15.98 \text{ kips/bolt}$$

Compute the resultant bolt force:

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(3.83 + 0.62)^2 + (2.26 + 15.98)^2} = 18.77 \text{ kips/bolt}$$

Check $R_u \leq R_r$, where $R_u = F_r$, and R_r equals the slip resistance of one bolt (calculated previously):

$$R_u = F_r = 18.77 \text{ kips/bolt} < R_r = 39 \text{ kips/bolt} \quad \text{OK}$$

The preceding check is conservative since the maximum factored moment after Cast #1 is assumed to be concurrent with the maximum factored shear after Cast #2.

7.12.5 Service Limit State, Top and Bottom Flange

According to the provisions of Article 6.13.6.1.4c, bolted connections for flange splices are to be designed as slip-critical connections for the flange design force. As a minimum, for checking slip of the flange splice bolts, the design force for the flange under consideration must be taken as the Service II design stress, F_s, times the smaller gross flange area on either side of the splice. F_s is calculated as follows:

$$F_s = \frac{f_s}{R_h} \qquad \text{Eq. (6.13.6.1.4c-5)}$$

where: f_s = maximum flexural stress due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of the splice (ksi)

R_h = 1.0 for homogeneous girders

Compute the flexural stresses for the top and bottom flanges (at the flange midthickness) for both the negative and positive live load bending cases and using the load factors for the Service II load combination from Table 3.4.1-1. The section properties of Field Section 3 of Girder 4 are used to compute the flange stresses; however, as noted earlier, the section property calculations are not shown for this particular section. It is assumed in these calculations that the concrete deck is not permitted to be considered effective in tension at this section for the negative live load bending case.

Negative live load bending case

$$f_s = \left[\frac{\gamma_{DC}\left(M_{DC1_{STEEL}} + M_{DC1_{CONC}}\right)(c)}{I_{NC}} + \frac{\left(\gamma_{DC}M_{DC2} + \gamma_{DW}M_{DW}\right)(c)}{I_{LongTerm(-M)}} + \frac{\gamma_{LL+I}\left(M_{LL+I}\right)(c)}{I_{ShortTerm(-M)}}\right]$$

$$f_{s,topflg} = -\left[\frac{1.0(-1,967)(49.02)}{111,996} + \frac{1.0[-250+(-237)]47.48}{120,299} + \frac{1.30(-2,772)(44.66)}{135,580}\right](12) = 26.9 \text{ ksi (T)}$$

$$f_{s,botflg} = \left[\frac{1.0(-1,967)(36.23)}{111,996} + \frac{1.0[-250+(-237)]37.77}{120,299} + \frac{1.30(-2,772)(40.59)}{135,580}\right](12) = -22.4 \text{ ksi (C)}$$

Positive live load bending case

$$f_s = \left[\frac{\gamma_{DC}\left(M_{DC1_{STEEL}} + M_{DC1_{CONC}}\right)(c)}{I_{NC}} + \frac{\left(\gamma_{DC}M_{DC2} + \gamma_{DW}M_{DW}\right)(c)}{I_{LongTerm(+M)}} + \frac{\gamma_{LL+I}\left(M_{LL+I}\right)(c)}{I_{ShortTerm(+M)}}\right]$$

$$f_{s,topflg} = -\left[\frac{1.0(-1,967)(49.02)}{111,996} + \frac{1.0[-250+(-237)]31.05}{210,369} + \frac{1.30(2,054)(15.96)}{293,406}\right](12) = 9.5 \text{ ksi (T)}$$

$$f_{s,botflg} = \left[\frac{1.0(-1,967)(36.23)}{111,996} + \frac{1.0[-250+(-237)]54.20}{210,369} + \frac{1.30(2,054)(69.29)}{293,406}\right](12) = -1.6 \text{ ksi (C)}$$

The negative live load bending case governs since it results in the larger absolute flange stresses. It should be noted that the total moment associated with the positive live load bending case results in overall negative moment at the splice location.

It is also necessary to include the force resultant in the bolt group due to the flange lateral bending stress. Apply only the noncomposite dead load lateral moment to the top flange since this moment is locked-in when the deck hardens. No other loads deflect the top flange in the transverse direction after the deck hardens since the deck acts as a diaphragm between girders. The lateral moment due to all loadings is applied to the bottom flange.

Determine the unfactored lateral moments using Eq. (C4.6.1.2.4b-1):

$$M_{lat\_DC1} = \frac{[-382 + (-1{,}585)](20)^2}{12(716.5)(7)} = -13.07 \text{ kip} - \text{ft}$$

$$M_{lat\_DC2} = \frac{-250(20)^2}{12(716.5)(7)} = -1.66 \text{ kip} - \text{ft}$$

$$M_{lat\_DW} = \frac{-237(20)^2}{12(716.5)(7)} = -1.57 \text{ kip} - \text{ft}$$

$$M_{lat\_LL-} = \frac{-2{,}772(20)^2}{12(716.5)(7)} = -18.42 \text{ kip} - \text{ft}$$

$$M_{lat\_LL+} = \frac{2{,}054(20)^2}{12(716.5)(7)} = 13.65 \text{ kip} - \text{ft}$$

7.12.5.1 Top Flange Critical Bolt Shear

Determine the force on the critical bolt, which is taken as the bolt farthest from the centroid of the bolt group. See Figure 9 for location of the critical bolt in the top flange bolt group. The shear force in the critical bolt has two sources – shear force induced by lateral bending and shear force induced by major-axis bending. Each must be computed, and then the force resultant is determined. The lateral bending induced shear force for the critical bolt due to noncomposite dead load has two components and is calculated as follows:

$$P_{long} = \frac{Mx}{I_p} \qquad\qquad P_{lat} = \frac{My}{I_p}$$

where: M = lateral bending moment (kip-in)
x = transverse distance from centroid of bolt group to critical bolt
y = longitudinal distance from centroid of bolt group to critical bolt
I_p = bolt group polar moment of inertia

Compute the factored lateral bending moment due to noncomposite dead load (DC1) only since only DC1 applies to the top flange, as discussed previously:

$$M = 1.0(13.07)(12) = 156.8 \text{ kip} - \text{in.}$$ (the sign is not needed in these calculations)

The bolt group polar moment of inertia was computed previously as:

$$I_p = 616\ \text{in.}^2$$

The longitudinal and lateral components of the lateral bending induced shear force are computed as follows:

$$P_{long} = \frac{156.8(6.50)}{616} = 1.65\ \text{kips}$$

$$P_{lat} = \frac{156.8(4.50)}{616} = 1.15\ \text{kips}$$

The controlling flange force due to major-axis bending is equal to the maximum of the top flange flexural stresses, f_s, multiplied by the gross area of the flange. Since the girder is homogeneous, $F_s = f_s$. The longitudinal force in each bolt resulting from flexure is determined by dividing the controlling flange force by the number of bolts on one side of the top flange splice.

$$P_{long\_vertbend} = \frac{F_s A_{top\_fl}}{N_b} = \frac{26.9(17)(1)}{16} = 28.6\ \text{kips/bolt}$$

The total force on the critical bolt is the resultant of the controlling flange force due to major-axis bending and the shear forces due to lateral bending. Therefore, the total force resultant on the critical bolt is:

$$F_{crit} = \sqrt{(28.6 + 1.65)^2 + 1.15^2} = 30.3\ \text{kips}$$

7.12.5.2 Bottom Flange Critical Bolt Shear

Determine the force on the critical bolt, which is taken as the bolt farthest from the centroid of the bolt group. See Figure 9 for location of the critical bolt in the bottom flange bolt group. The calculations for the bottom flange are similar to the previous calculations for the top flange. The lateral bending induced shear force for the critical bolt due to Service II dead load and live load is calculated as follows:

$$P_{long} = \frac{Mx}{I_p} \qquad P_{lat} = \frac{My}{I_p}$$

Compute the factored lateral bending moment due to all loadings, as discussed previously:

$$M = [1.0(-13.07 - 1.66 - 1.57) + 1.30(-18.42)](12) = 482.9\ \text{kip-in.}$$

(the sign is not needed in these calculations)

The bolt group polar moment of inertia was computed previously as:

$$I_p = 1{,}140\ \text{in.}^2$$

The longitudinal and lateral components of the lateral bending induced shear force are computed as follows:

$$P_{long} = \frac{482.9(8.50)}{1{,}140} = 3.60\ \text{kips}$$

$$P_{long} = \frac{482.9(4.50)}{1{,}140} = 1.91\ \text{kips}$$

The controlling flange force due to major-axis bending is equal to the maximum of the bottom flange flexural stresses, f_s, multiplied by the gross area of the flange. Since the girder is homogeneous, $F_s = f_s$. The longitudinal force in each bolt resulting from flexure is determined by dividing the controlling flange force by the number of bolts on one side of the bottom flange splice.

$$P_{long\_vertbend} = \frac{F_s A_{bot\_fl}}{N_b} = \frac{|-22.4|(21)(1.5)}{24} = 29.4\ \text{kips/bolt}$$

The total force on the critical bolt is the resultant of the controlling flange force due to major-axis bending and the shear forces due to lateral bending. Therefore, the total force resultant on the critical bolt is:

$$F_{crit} = \sqrt{(29.4 + 3.60)^2 + 1.91^2} = 33.0\ \text{kips}$$

The critical bolt shear force is greater for the bottom flange than for the top flange, so the bottom flange controls. For slip-critical connections, the factored resistance, R_r, was calculated previously as 39 kips/bolt.

$$F_{crit} = 33.0\ \text{kips} < R_r = 39\ \text{kips/bolt} \quad \text{OK}$$

It should be noted that by including the effects of the flange lateral bending stress, the resultant force in the top and bottom flange bolts increases from 28.6 kips/bolt to 30.3 kips/bolt (6.0%) and 29.4 kips/bolt to 33.0 kips/bolt (12.0%), respectively.

7.12.6 Strength Limit State

Bolted splices are designed at the strength limit state to satisfy the requirements specified in Article 6.13.1. In basic terms, Article 6.13.1 indicates that a splice shall be designed for the larger of (a) the average of the factored applied stresses and the factored resistance of the member or (b) 75 percent of the factored resistance of the member. The intent of this provision is to provide reasonably sized connections. Where the section changes at the splice, the smaller of the two connected sections is to be used.

7.12.6.1 Flange Splice General Calculations

The effective area, A_e, of a flange when it is in tension is computed using Eq. (6.13.6.1.4c-2). The net area, A_n, is calculated using the provisions of Article 6.8.3.

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g \qquad \text{Eq. (6.13.6.1.4c-2)}$$

Where: ϕ_u = resistance factor for tension, fracture in net section, specified in Article 6.5.4.2
ϕ_y = resistance factor for tension, yielding in gross section, specified in Article 6.5.4.2

There are 4 bolts per row in the top flange splice, and 15/16-inch bolt holes are assumed in the splice design calculations. The net area of the top flange is computed as follows:

$$A_n = [17.0 - 4(0.9375)](1.0) = 13.25 \text{ in.}^2$$

The gross area of the top flange is computed as follows:

$$A_g = (17.0)(1.0) = 17.0 \text{ in.}^2$$

The effective area of the top flange is then computed using Eq. (6.13.6.1.4c-2):

$$A_e = \left(\frac{0.8(65)}{0.95(50)}\right) 13.25 = 14.5 \text{ in.}^2$$

Since the effective area does not exceed the gross area, use the computed effective area, A_e = 14.5 in.^{2}

In accordance with Article 6.13.6.1.4c, the effective area is used to compute the force in the flange when the flange is subject to tension, and the gross area is used when a flange is subject to compression.

According to the provisions of Article 6.13.6.1.4a, the flexural stresses due to the factored loads at the strength limit state shall be determined using the gross section properties. The factored bending stresses for Strength I at the midthickness of the flanges are computed as follows:

<u>Negative live load bending case</u>

$$f = \left[\frac{\gamma_{DC}\left(M_{DC1_{STEEL}} + M_{DC1_{CONC}}\right)(c)}{I_{NC}} + \frac{\left(\gamma_{DC} M_{DC2} + \gamma_{DW} M_{DW}\right)(c)}{I_{LongTerm(-M)}} + \frac{\gamma_{LL+I}\left(M_{LL+I}\right)(c)}{I_{ShortTerm(-M)}}\right]$$

$$f_{topflg} = -\left[\frac{1.25(-1{,}967)(49.02)}{111{,}996} + \frac{[1.25(-250)+1.5(-237)]47.48}{120{,}299} + \frac{1.75(-2{,}772)(44.66)}{135{,}580}\right](12)$$

$$f_{topflg} = 35.3 \text{ ksi (T)}$$

$$f_{botflg} = \left[\frac{1.25(-1{,}967)(36.23)}{111{,}996} + \frac{[1.25(-250)+1.5(-237)]37.77}{120{,}299} + \frac{1.75(-2{,}772)(40.59)}{135{,}580}\right](12)$$

$$f_{botflg} = -29.5 \text{ ksi (C)}$$

Positive live load bending case

$$f = \left[\frac{\gamma_{DC}\left(M_{DC1_{STEEL}} + M_{DC1_{CONC}}\right)(c)}{I_{NC}} + \frac{(\gamma_{DC}M_{DC2} + \gamma_{DW}M_{DW})(c)}{I_{LongTerm(+M)}} + \frac{\gamma_{LL+I}(M_{LL+I})(c)}{I_{ShortTerm(+M)}}\right]$$

$$f_{topflg} = \left[\frac{0.90(-1{,}967)(49.02)}{111{,}996} + \frac{[0.90(-250)+0.65(-237)]31.05}{210{,}369} + \frac{1.75(2{,}054)(15.96)}{293{,}406}\right](12)$$

$$f_{topflg} = 7.6 \text{ ksi (C)}$$

$$f_{botflg} = \left[\frac{0.90(-1{,}967)(36.23)}{111{,}996} + \frac{[0.90(-250)+0.65(-237)]54.20}{210{,}369} + \frac{1.75(2{,}054)(69.29)}{293{,}406}\right](12)$$

$$f_{botflg} = 2.1 \text{ ksi (T)}$$

The negative live load bending case governs since it results in the larger absolute flange stresses. For the negative live load bending case, the top flange is the controlling flange since it has the largest ratio of the flexural stress to the corresponding flange resistance (based on calculations not shown here). Splice plates and their connections on the controlling flange need to be proportioned to provide a minimum resistance taken as the design stress times the smaller effective flange area, A_e, on either side of the splice. Article 6.13.6.1.4c defines the design stress, F_{cf}, for the controlling flange as:

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

where f_{cf} is the maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the splice, the top flange in this case. The hybrid factor R_h is taken as 1.0

since all plates have the same yield strength, and α is taken as 1.0. The flange resistance modification factor, R_g is calculated as follows:

$$R_g = \frac{\lfloor \alpha A_e F_{yf} \rfloor_{LS}}{\lfloor \alpha A_e F_{yf} \rfloor_{SS}} \leq 1.0$$

Where $\lfloor \alpha A_e F_{yf} \rfloor_{LS}$ is for the flange under consideration in the larger section at the splice point and $\lfloor \alpha A_e F_{yf} \rfloor_{SS}$ is for the flange under consideration in the smaller section at the splice point.

$$R_g = \frac{[(1)(33.2)(50)]}{[(1)(14.5)(50)]} = 2.29 > 1.0$$

Therefore:

$R_g = 1.0$

Note the effective area for the larger flange section is equal to 33.2 in^2. This calculation is not shown but is similar to the effective area calculation shown previously for the smaller flange section.

$$F_{cf} = \frac{\left|\frac{35.3}{1.0}\right| + 1.0(1.0)(50)(1.0)}{2} = 42.65\,\text{ksi}$$

$$0.75\alpha\varphi_f F_{yf} = 0.75(1.0)(1.0)(50)(1.0) = 37.5\,\text{ksi}$$

Therefore, F_{cf} = 42.65 ksi controls.

The area of the smaller top flange is used to ensure that the design force does not exceed the strength of the smaller flange. The design force in the top flange is:

$F_{cf}A_e$ = 42.62 (14.5) = 618 kips (T)

Splice plates and their connections on the noncontrolling flange at the strength limit state must be proportioned to provide a minimum resistance taken as the design stress, F_{ncf}, times the smaller of the effective flange area, A_e, on either side of the splice. The bottom flange is the noncontrolling flange in this case. F_{ncf} is calculated as follows:

$$F_{ncf} = R_{cf}\left|\frac{f_{ncf}}{R_h}\right| \geq 0.75\alpha._f F_{yf} R_g \qquad \text{Eq. (6.13.6.1.4c-4)}$$

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 factored loads at the midthickness of the noncontrolling flange at the splice concurrent with f_{cf}

R_h = 1.0 for a homogeneous girder

$$R_{cf} = \left|\frac{F_{cf}}{f_{cf}}\right| = \left|\frac{42.65}{35.3}\right| = 1.208$$

$$F_{ncf} = 1.208\left|\frac{-29.5}{1.0}\right| = 35.64\text{ksi}$$

$$0.75\alpha\varphi_f F_{yf}R_g = 0.75(1.0)(1.0)(50)(1.0) = 37.5\text{ ksi}$$

Therefore, F_{ncf} = 37.5 ksi.

The minimum design force for the noncontrolling flange, $F_{ncf}A_e$, is computed as follows:

$F_{ncf}A_e$ = 37.5(21.0)(1.5) = 1,181 kips (C)

In the above equation, the effective flange area, A_e, is taken equal to the smaller gross flange area, A_g, on either side of the splice. The gross flange area is used since the flange is subjected to compression.

7.12.6.2 Bolt Shear in Top Flange

For the top flange splice plates, use a 0.5″ x 17″ outer plate and two 0.625″ x 7″ inner plates.

The difference in thickness of the two top flanges being joined is ¼ in., so a ¼ in. thick fill plate is required. As permitted by Article 6.13.6.1.5, fillers need not be extended beyond the splice material and developed provided that the factored resistance at the bolts in shear at the strength limit state is reduced by the following factor:

$$R = \frac{(1+\gamma)}{(1+2\gamma)} \quad \text{Eq. (6.13.6.1.5-1)}$$

where: A_f = sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

γ = A_f/A_p

Compute the above terms as follows:

$$A_f = 17(0.25) = 4.25\text{in.}^2$$

Area of top flange splice plates = (17)(0.5)+2(7)(0.625) = 17.25 in.^{2}

Area of connected plate = (17)(1.0) = 17.0 in.^{2} (controls)

A_p = 17.0 in.^{2}

$$\gamma = \frac{A_f}{A_p} = \frac{4.25}{17.0} = 0.25$$

$$R = \frac{(1+0.25)}{[1+2(0.25)]} = 0.83$$

Therefore, reduce the bolt design shear strength by 0.83 for the strength limit state check only.

Flange lateral bending is not considered in the top flange after the deck has hardened and the flange is continuously braced. The factored bolt shear resistance, R_r, was previously computed as 55.4 kips/bolt. Therefore, the required number of bolts is computed as follows:

$$\text{No. bolts required} = \frac{F_{cf}\,A_e}{R(R_r)} = \frac{618}{0.83(55.4)} = 13.4 \text{ bolts, use } 16 \text{ bolts}$$

$$\frac{618}{16} = 38.6 \text{ kips/bolt} < R(R_r) = 0.83(55.4) = 46.0 \text{ kips/bolt} \quad \text{OK}$$

7.12.6.3 Bolt Shear in Bottom Flange

For the bottom flange, flange lateral bending must be considered since the flange is discretely braced. The following dead and live load values have been taken directly from the analysis:

	Lateral Bending Moment
Steel Dead Load	2.9 kip-ft
Concrete Deck Dead Load	-12.1 kip-ft
Composite Dead Load	-1.89 kip-ft
Future Wearing Surface Dead Load	-1.81 kip-ft
Live Load (including (IM + CF)	-16.8 kip-ft

The Strength I total factored lateral bending moment is:

M_{lat} = 1.25(2.9 + 12.1 + 1.89) + 1.5(1.81) + 1.75(16.8) = 53.2 kip-ft

The longitudinal and lateral components of the lateral bending induced shear force are computed as follows:

$$P_{long} = \frac{53.2(8.50)}{1{,}140}(12) = 4.76 \text{ kips/bolt}$$

$$P_{lat} = \frac{53.2(4.50)}{1{,}140}(12) = 2.52\text{ kips/bolt}$$

The total force on the critical bolt is the resultant of the shear force due to major-axis bending and the shear forces due to lateral bending. The shear force in each bolt due to major-axis bending is equal to the minimum design force, $F_{ncf}A_e$, divided by the number of bolts:

$$P_{vert\,bend} = \frac{1{,}181}{24} = 49.21\text{ kips/bolt}$$

The total force resultant on the critical bolt is therefore:

$$F_{crit} = \sqrt{(4.76 + 49.21)^2 + 2.52^2} = 54.0\text{ kips} = R_u$$

The factored shear resistance at the strength limit state, R_r, was calculated previously as 55.4 kips/bolt.

$$R_u = 54.0\text{ kips/bolt} < R_r = 55.4\text{ kips/bolt OK}$$

It should be noted that a fill plate is not required for the bottom flange splice. Therefore, no reduction in the bolt design shear strength is necessary.

7.12.6.4 Web Splice Design

7.12.6.4.1 Design Shear

As demonstrated for the flange splice design, the design shear is based on a portion of the applied stress and/or a portion of the factored resistance per Article 6.13.1 in order to ensure reasonably sized connections. The web splice is designed based on the conservative assumption that the maximum moment and shear at the splice occur simultaneously.

In order to determine the design shear, V_{uw}, first determine the factored shear, V_u. Using the values from Table 10, the factored shear at the splice for Strength I is computed as:

$$V_u = 1.25(27 + 112 + 19) + 1.5(22) + 1.75(139) = 474\text{ kips}$$

The factored shear resistance of the 0.5625 in. web at the splice (the smaller web) was determined to be 617 kips according to the provisions of Article 6.10.9.1. Although not shown, the calculations are similar to the calculations shown earlier for computing the shear resistance of the web at Sections G4-2 and G4-3.

$$V_u \le \phi_v V_n = 617\text{kips} \qquad \text{Eq. (6.10.9.1-1)}$$

As needed to determine the design shear V_{uw}, compute half the factored shear resistance as follows:

$$0.5\phi_v V_n = 0.5(617) = 309\text{kips} < V_u = 474\text{kips}$$

Therefore, according to Article 6.13.6.1.4b, since $V_u > 0.5\phi_v V_n$, the design shear is computed as follows:

$$V_{uw} = \frac{(V_u + \phi_v V_n)}{2} = \frac{(474 + 617)}{2} = 546\ \text{kips} \qquad \text{Eq. (6.13.6.1.4b-2)}$$

In the checks that follow, the design shear is shown not to exceed the factored block shear rupture resistance of the web splice plates specified in Article 6.13.4 or the factored shear resistance of the web splice plates specified in Article 6.13.5.3.

7.12.6.4.2 Design Moment and Design Horizontal Force Component

First, compute the moment, M_{uv}, due to the eccentricity of the design shear from the centerline of the splice to the centroid of the web splice bolt group as follows:

$$M_{uv} = V_{uw} e$$

$$M_{uv} = 546\left[\frac{3}{2} + 2.125\right]\left(\frac{1}{12}\right) = 165\text{kip} - \text{ft}$$

Determine the portion of the major-axis bending moment resisted by the web, M_{uw}, and the horizontal design force resultant in the web, H_{uw}, according to the provisions of Article C6.13.6.1.4b. M_{uw} and H_{uw} are assumed to act at the middepth of the web. As stated earlier, negative live load bending condition controls, so only this condition will be investigated.

As computed earlier for the flange splice design, the negative live load bending stresses are as follows:

f_{cf} = 35.3 ksi
F_{cf} = 42.65 ksi
f_{ncf} = -29.5 ksi
R_{cf} = 1.208

Using these bending stresses, compute the portion of the flexural moment assumed to be resisted by the web and the horizontal design force resultant in the web:

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

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

$$M_{uw} = \frac{0.5625(84)^2}{12}\left|1.0(42.65) - 1.208(-29.5)\right|\left(\frac{1}{12}\right) = 2{,}158\ \text{kip} - \text{ft}$$

$$H_{uw} = \frac{0.5625(84)}{2}\left[1.0(42.65) + 1.208(-29.5)\right] = 166\text{kips}$$

The total moment on the web splice is computed as follows:

$$M_{tot} = M_{uv} + M_{uw} = 165 + 2{,}158 = 2{,}323\ \text{kip - ft}$$

7.12.6.4.3 Block Shear Rupture (Article 6.13.4)

Block shear rupture resistance normally does not govern for typical web splice plates, but the check is illustrated here for completeness. The assumed block shear failure plane for the web splice plate is shown in Figure 12.

According Article 6.13.4, the factored resistance of the combination of parallel and perpendicular planes shall be taken as:

$$R_r = \phi_{bs} R_p \left(0.58 F_u A_{vn} + U_{bs} F_u A_{tn}\right) \le \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right) \qquad \text{Eq. (6.13.4-1)}$$

where: R_p = reduction factor for holes taken equal to 1.0 for bolt holes drilled full size
A_{vg} = gross area along the plane resisting shear stress (in.^{2})
A_{vn} = net area along the plane resisting shear stress (in.^{2})
U_{bs} = reduction factor for block shear rupture resistance taken equal to 1.0 when the tension stress is uniform
A_{tn} = net area along the plane resisting tension stress (in.^{2})
ϕ_{bs} = resistance factor for block shear specified in Article 6.5.4.2

First, compute the area terms, based on the assumed block shear failure planes shown in Figure 12:

$$A_{vg} = 2(81)(0.375) = 60.75\ \text{in.}^2$$

$$A_{vn} = 2[79 - 22.5(0.9375)](0.375) = 43.43\ \text{in.}^2$$

$$A_{tn} = 2[3 + 2 - 1.5(0.9375)](0.375) = 2.69\ \text{in.}^2$$

Compute the factored resistance as follows:

$$R_{r1} = 0.80(1.0)\left[0.58(65)(43.43) + (1.0)(65)(2.69)\right] = 1{,}450 \text{ kips (controls)}$$

$$R_{r2} = 0.80(1.0)\left[0.58(50)(60.75) + 1.0(65)(2.69)\right] = 1{,}549 \text{ kips}$$

$$V_{uw} = 546 \text{ kips} < R_r = 1{,}450 \text{ kips} \quad \text{OK}$$

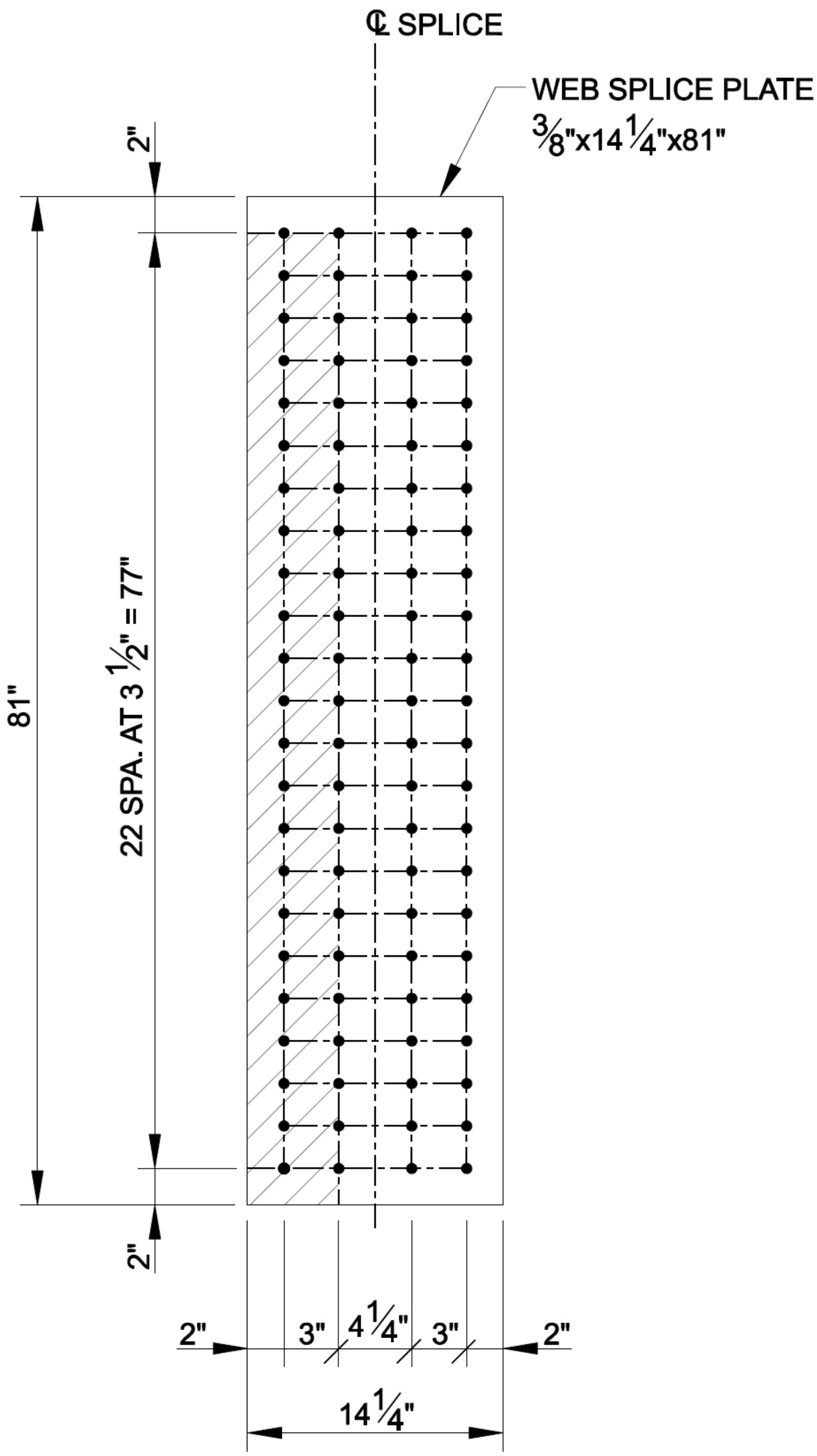


Figure 12 Assumed Block Shear Failure Planes for Web Splice Plate

7.12.6.4.4 Flexural Yielding

It is also necessary to check for flexural yielding on the gross section of the web splice plates at the strength limit state. The flexural stress is limited to $\phi_f F_y$. From Figure 9, the web splice plate length is 81 in. Therefore, the section modulus and gross area are computed as follows:

$$S_{web\ PL} = \frac{2(0.375)(81)^2}{6} = 820\ \text{in.}^3$$

$$\text{Gross Area} = A_g = 2(0.375)(81) = 60.75\ \text{in.}^2$$

Using the design moment and horizontal force resultant computed previously at this location, the bending stress in the splice plate is computed as follows:

$$f_g = \frac{M_{uv} + M_{uw}}{S_{PL}} + \frac{H_{uw}}{A_g} = \frac{(165 + 2{,}158)(12)}{820} + \frac{166}{60.75} = 36.73\ \text{ksi}$$

$$f_g = 36.73\ \text{ksi} < \phi_f F_y = 1.0(50) = 50\ \text{ksi} \quad \text{OK}$$

The splice plates are therefore adequate for flexure.

7.12.6.4.5 Shear Yielding and Shear Rupture (Article 6.13.5.3)

According to the provisions of Article 6.13.5.3, the factored shear resistance, R_r, of the connection element shall be taken as the smaller value based on shear yielding or shear rupture. For shear yielding, the factored shear resistance of the connection element is computed as follows:

$$R_r = \phi_v 0.58 F_y A_{vg} \qquad \text{Eq. (6.13.5.3-1)}$$

$$R_r = 1.0(0.58)(50)(60.75) = 1{,}762\ \text{kips}$$

For shear rupture, the factored resistance of the connection element is computed as follows:

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn} \qquad \text{Eq. (6.13.5.3-2)}$$

$$R_r = 0.80(0.58)(1.0)(65)(43.43) = 1{,}310\ \text{kips (controls)}$$

Therefore, the lesser of the factored shear resistances is checked against the design shear as follows:

V_{uw} = 546 kips < R_r = 1,310 kips OK

7.12.6.4.6 Shear in Web Splice Bolts at Strength Limit State

Compute the vertical bolt force by dividing the design shear by the number of web splice bolts on one side of the connection:

$$F_s = \frac{V_{uw}}{N_b} = \frac{546}{46} = 11.87 \text{ kips/bolt}$$

Compute the bolt force due to the horizontal design force resultant by dividing the horizontal force by the number of web splice bolts on one side of the connection:

$$F_H = \frac{H_{uw}}{N_b} = \frac{166}{46} = 3.61 \text{ kips/bolt}$$

Compute the horizontal and vertical components of the force on the extreme bolt due to the total moment on the splice. The polar moment of inertia is 24,898 in.^{2} (calculation not shown).

$$F_{Mv} = \frac{M_{tot}x}{I_p} = \frac{2{,}323(12)(1.5)}{24{,}898} = 1.68 \text{ kips/bolt}$$

$$F_{Mh} = \frac{M_{tot}y}{I_p} = \frac{2{,}323(12)(38.5)}{24{,}898} = 43.10 \text{ kips/bolt}$$

Compute the resultant bolt force:

$$F_r = \sqrt{(F_s + F_{Mv})^2 + (F_H + F_{Mh})^2} = \sqrt{(11.87 + 1.68)^2 + (3.61 + 43.10)^2} = 48.64 \text{ kips/bolt}$$

$$R_r = \phi_s R_n = 55.4 \text{kips (calculated previously)} \qquad \text{Eq. (6.13.2.2-2)}$$

$$F_r = 48.64 \text{ kips/bolt} < R_r = 55.4 \text{ kips/bolt OK}$$

7.12.6.4.7 Bearing Resistance of Web

As shown in Figure 9, 0.375 in. thick splice plates are used. As permitted in Article 6.13.6.1.5, 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 in.

Checking the provision of Article 6.13.2.6.2, the spacing of the bolts for sealing is less than the maximum permissible spacing:

$$s \le 4 + 4t \le 7.0 \text{in.} \qquad \text{Eq. (6.13.2.6.2-1)}$$

$$s = 3\text{in.} < 4 + 4(0.375) = 5.5\text{in. OK}$$

It is necessary to check the bearing resistance at the web splice bolt holes at the strength limit state. The assumption is that at the strength limit state, the bolts have slipped and gone into bearing. The bearing strength of the web controls since the web thickness is less than the sum of the two splice plate thicknesses. The bearing strength of the outermost hole in the thinner web at the splice, calculated using the clear edge distance, will conservatively be checked against the maximum resultant force acting on the extreme bolt in the connection. This check is conservative since the resultant force acts in the direction of an inclined distance that is larger than the clear edge distance. Should the bearing strength be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Another option would be to calculate the bearing strength based on the inclined distance or resolve the resultant force in the direction parallel to the edge distance. In cases where the bearing strength of the web splice plate controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing strength of the outermost hole.

Again for a hole diameter of 15/16 inch, the clear distance between the edge of the hole and the edge of the field piece is computed as follows:

$$L_c = 2.0 - \frac{0.9375}{2} = 1.53 \text{ in.}$$

Since the clear end distance is less than 2.0d, the nominal bearing resistance at the bolt holes is computed as follows:

$$R_n = 1.2 L_c t F_u \qquad \text{Eq. (6.13.2.9-2)}$$

$$R_n = 1.2(1.53)(0.5625)(65) = 67.13 \text{ kips/bolt}$$

The factored bearing resistance is computed as:

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

$$R_r = 0.8(67.13) = 53.7 \text{ kips/bolt}$$

The maximum force on the extreme bolt was computed previously for strength as:

$$F_r = 48.64 \text{ kips/bolt} < R_r = 53.7 \text{ kips/bolt} \quad \text{OK}$$

7.12.6.5 Top Flange (Controlling Flange) Splice Plate Design

The width of the outside splice plate should be as wide as the width of the narrowest flange at the splice. Therefore, 17 inches is selected for the width of the outer plate. See Figure 9 for the plate sizes and bolt patterns.

The following plate sizes are selected for the top flange splice plates, and the gross and net areas are computed. Again, the bolt holes are assumed to be 15/16 inch for design purposes.

Outer Plate: 17 in. x 0.5 in.

A_s = (17) (0.5) = 8.50 in.^{2}

A_n = 8.50 – 4 (0.9375) (0.5) = 6.625 in.^{2}

Inner Plates: 2 – 7 in. x 0.625 in.

A_s = 2 (7) (0.625) = 8.75 in.^{2}

A_n = 8.75 – 4 (0.9375) (0.625) = 6.406 in.^{2}

As specified in Article C6.13.6.1.4c, if the areas of the inner and outer splice plates are within 10 percent, then the flange design force at the strength limit state may be divided equally to the inner and outer plates and their connections. Double shear may then be assumed in designing the bolts. If the areas differ by more than 10 percent, the flange design force is to be proportioned to the inner and outer plates by the ratio of the area(s) of the splice plate under consideration to the total area of the splice plates. In that case, the shear strength of the bolts would be checked assuming the maximum calculated splice plate force acts on a single shear plane.

At Splice 2, the top flange is subjected to tension. The design force, $F_{cf}A_e$, for the top flange was computed previously to be 618 kips. Flange lateral bending need not be considered in the top flange after the deck has hardened. The capacity of the splice plates to resist tension is computed according to Article 6.8.2.1.

In accordance with Article 6.13.5.2, the factored tensile resistance, P_r, is taken as the lesser of the values given by Eqs. (6.8.2.1-1) and (6.8.2.1-2).

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \quad \text{(yielding on gross section)} \qquad \text{Eq. (6.8.2.1-1)}$$

$$P_r = 0.95(50)(8.50 + 8.75) = 819 \text{kips}$$

or

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \quad \text{(fracture on net section)} \qquad \text{Eq. (6.8.2.1-2)}$$

where: A_n = 6.625 + 6.406 = 13.03 in.^{2} < 0.85A_g = 0.85(8.5 + 8.75) = 14.66 in.^{2}

R_p = reduction factor for holes taken equal to 1.0 for bolt holes drilled full size

U = reduction factor to account for shear lag taken equal to 1.0 when force effects are transmitted to all elements

$$P_r = 0.80(65)(13.03)(1.0)(1.0) = 678 \text{ kips} > 618 \text{ kips} \quad \text{Controlling flange is OK.}$$

Note that per Article 6.13.5.2, the net area, A_n, used in Eq. (6.8.2.1-2) is not to be taken greater than 85 percent of the gross area of the plate.

Next, check the inner and outer plates for adequate resistance against block shear rupture according to Article 6.13.4. The factored resistance of the combination of parallel and perpendicular planes is taken as:

$$R_r = \phi_{bs} R_p \left(0.58 F_u A_{vn} + U_{bs} F_u A_{tn}\right) \leq \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right) \qquad \text{Eq. (6.13.4-1)}$$

First, compute the area terms, based on the assumed block shear failure planes of the top flange splice plates shown in Figure 13 and Figure 14:

$$A_{vg} = 2(2+9)(0.5) + 2(2+9)(0.625) = 24.75 \text{ in.}^2$$

$$A_{vn} = 2[2+9-3.5(0.9375)](0.5) + 2[2+9-3.5(0.9375](0.625) = 17.37 \text{ in.}^2$$

$$A_{tn} = 2[3+2-1.5(0.9375)](0.5) + 2[3+2-1.5(0.9375)](0.625) = 8.09 \text{ in.}^2$$

Compute the factored resistance as follows:

$$R_{r1} = 0.8(1.0)[0.58(65)(17.37) + 1.0(65)(8.09)] = 945 \text{ kips (controls)}$$

$$R_{r2} = 0.8(1.0)[0.58(50)(24.75) + 1.0(65)(8.09)] = 995 \text{ kips}$$

R_u = 618 kips < R_r = 945 kips OK

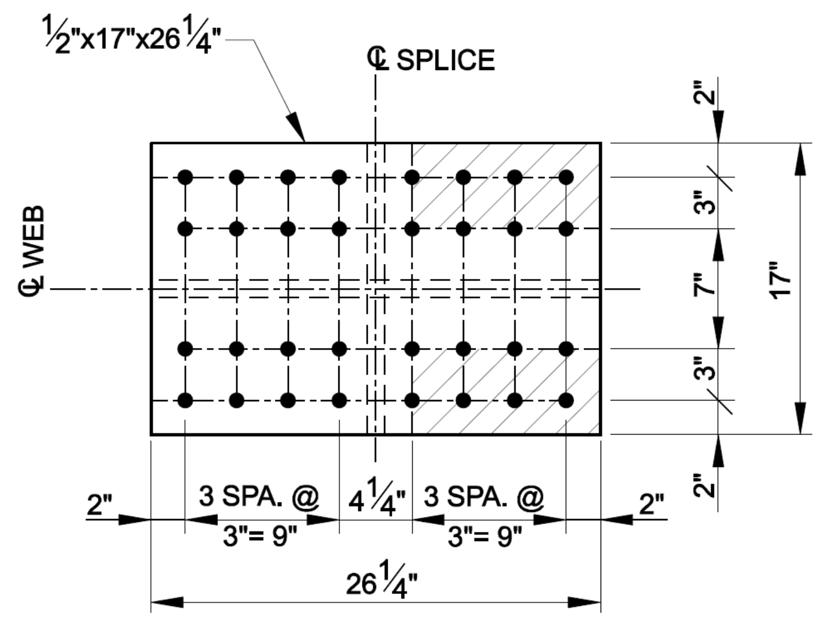


Figure 13 Assumed Block Shear Failure Planes for Top Flange Outer Splice Plate

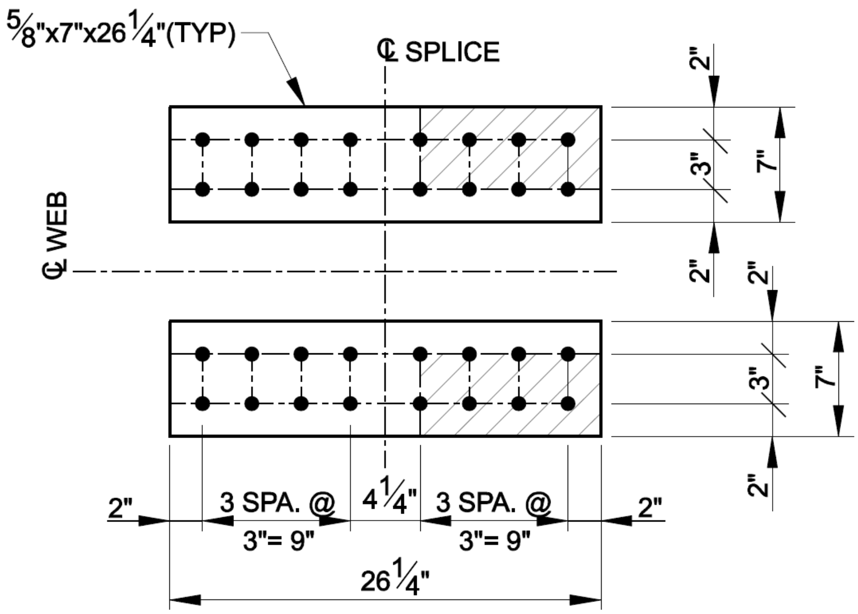


Figure 14 Assumed Block Shear Failure Planes for Top Flange Inner Splice Plates

7.12.6.6 Bottom Flange (Noncontrolling Flange) Splice Plate Design

According to Article 6.13.6.1.4c, flange splice plates subjected to compression at the strength limit state are to be checked only for yielding on the gross section of the plates. Therefore, check the bottom flange which is in compression, the noncontrolling flange in this case. The design force, $F_{ncf}A_e$, was computed previously to be 1,181 kips.

The following plate sizes are selected for the bottom flange splice plates, and the gross areas are computed.

Outer Plate: 21 in. x 0.75 in.

$A_g = (21)\ (0.75) = 15.75\ \text{in.}^2$

Inner Plates: 2 – 9.5 in. x 0.875 in.

$A_g = 2\ (9.5)\ (0.875) = 16.63\ \text{in.}^2$

The factored resistance in compression is taken as:

$$R_r = \phi_c F_y A_s \qquad \text{Eq. (6.13.6.1.4c-5)}$$

where: ϕ_c = resistance factor for compression as specified in Article 6.5.4.2

A_s = gross area of splice plate

$R_r = 0.95(50)(15.75+16.63) = 1{,}538$ kips > 1,181 kips OK

Since the splice plates are on a partially braced flange and subjected to compression, check for yielding on the gross section of the splice plates under their portion of the minimum design force, $F_{ncf}A_e$, plus the factored flange lateral bending moment.

The flange design force, $F_{ncf}A_e$, was computed previously to be 1,181 kips (compression). The flange lateral moment for strength was computed previously to be -53.2 kip-ft (factored).

The gross section modulus in the lateral direction of the inner and outer splice plates together is:

$$S_{lat} = \frac{(0.75 + 0.875)(21)^2}{6} - \frac{(0.875)(2)^2}{6} = 118.9 \text{ in.}^3$$

Check for flexural yielding on the gross section of the flange splice plates at the strength limit state due to flexure and flange lateral bending. The flexural stress is limited to $\phi_f F_y$.

$$f_g = \frac{1{,}181}{(15.75 + 16.63)} + \frac{53.2(12)}{118.9} = 41.84\text{ksi}$$

$$41.84 \text{ ksi} < \phi_f F_y = 1.0(50) = 50 \text{ ksi} \quad \text{OK}$$

If the difference in area of the inner splice plates had not been within 10 percent of the area of the outside splice plate, the factored design force would then be proportioned to the inner and outer splice plates accordingly (see Article C6.13.6.1.4c).

7.12.6.7 Bearing Resistance at Bolt Holes (Bottom Flange)

Check bearing of the bolts on the connected material under the minimum design force. The design bearing strength, R_n, is computed using the provisions of Article 6.13.2.9. The bottom flange governs the bearing strength of the connection, as the sum of the splice plate thicknesses times the tensile strength of the plates is greater than the bottom flange thickness times its tensile strength.

The bottom flange splice is subject to lateral bending; thus, the resultant force, R_u, in the extreme outer bolt acts in the direction of an inclined distance that is larger than the clear end distance. In lieu of calculating the bearing resistance based on this inclined distance, the resistance of the outermost hole, calculated using the clear end distance, will conservatively be checked against the maximum resultant bolt force, R_u.

According to specifications, the bearing strength for the end row of bolts is computed using Eq. (6.13.2.9-1) or Eq. (6.13.2.9-2). Calculate the clear distance between holes and the clear end distance and compare to 2.0d to determine the equation to be used to compute the bearing strength (where "d" is the diameter of the bolt).

For the four bolts adjacent to the edge of the splice plate, the edge distance is 2 in. as shown in Figure 11. Therefore, the clear distance between the edge of the holes and the end of the splice plate is:

$$L_c = 2.0 - \frac{0.9375}{2} = 1.53 \text{ in.}$$

The value 2.0d is equal to 1.75 in. Since the clear end distance is less than 2.0d, use Eq. (6.13.2.9-2) to compute the nominal bearing resistance at the strength limit state. Note that t is the thickness of the connected material, the bottom flange thickness.

$$R_n = 1.2L_c tF_u = 1.2(1.53)(1.5)(65) = 179.0 \text{ kips/bolt} \qquad \text{Eq. (6.13.2.9-2)}$$

The factored bearing resistance is computed as:

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

$$R_r = 0.8(179.0) = 143.2 \text{ kips/bolt}$$

R_u = 54.0 kips < R_r = 143.2 kips OK

Although not included here, similar calculations show that bearing of the bolts on the top flange is less than the computed resistance as well.

7.13 Cross Frame Member and Connection

7.13.1 Cross Frame Diagonal Design

Evaluation of the cross frame analysis results shows that the diagonal member between G4 and G3 at Support 2 has the largest force. The largest factored load of the Load Combinations examined is -88 kips (compression). Compression members are designed according to Article 6.9. According to Article 6.7.4.1, cross frames in horizontally curved bridges are considered primary members.

Using the girder spacing and web height, determine the effective length of the diagonal member:

$$\ell = \sqrt{11^2 + 7^2} = 13 \text{ ft}$$

Use a L8x8x3/4 single angle with a yield stress of 50 ksi and with the following properties taken from the AISC *Steel Construction Manual* [6].

$r_{xx} = r_{yy}$ = 2.46 in.
r_{zz} = 1.57 in.
A_s = 11.5 in.^{2}

Check the slenderness provision of Article 6.9.4.2.1 for the cross frame diagonal member:

$$\frac{b}{t} \leq k\sqrt{\frac{E}{F_y}} \qquad \text{Eq. (6.9.4.2.1-1)}$$

where: k = plate buckling coefficient, 0.45 for outstanding legs of single angles, from Table 6.9.4.2.1-1
b = the full width of the outstanding leg for a single angle (in.)
t = plate thickness (in.)

$$\frac{b}{t} = \frac{8}{0.75} = 10.7 < 0.45\sqrt{\frac{29{,}000}{50}} = 10.8 \quad \text{OK. Member is nonslender.}$$

Check the limiting slenderness ratio of Article 6.9.3. As a primary member, the angle must satisfy the following:

$$\frac{K\ell}{r} \leq 120$$

where: K = effective length factor specified in Article 4.6.2.5 as 1.0 for single angles regardless of end connection (in.)
ℓ = unbraced length (in.)
r = minimum radius of gyration (in.)

$$\frac{K\ell}{r} = \frac{1.0(13)(12)}{1.57} = 99 < 120 \quad \text{OK}$$

In an actual design, an additional iteration of the analysis would be necessary since the cross frame member area used in the model was 5.0 in.^{2} and the design area is 11.5 in^2. Since the cross frames are truss members in the 3D analysis, the area of the cross frame elements affects the structure rigidity, which in turn alters the girder moments and shears as well as cross frame forces.

Having satisfied the basic slenderness provisions, the angle is then checked for the strength limit state in accordance with Article 6.9.4.4 regarding single-angle members.

Single angles are commonly used as members in cross frames of steel girder bridges. Since the angle is typically connected through one leg only, the member is subjected to combined axial load and flexure. In other words, the eccentricity of the applied axial load induces moments about both principal axes of the angle. As a result, it is difficult to predict the nominal compressive resistance of these members. The provisions of Article 6.9.4.4 provide a simplified approach by permitting the effect of the eccentricities to be neglected when the single angles are evaluated as axially loaded compression members for flexural buckling only using an appropriate specified effective slenderness ratio, $(K\ell/r)_{eff}$, in place of $(K\ell/r_s)$ in Eq. (6.9.4.1.2-1). By following this approach, the single angles are designed as axially loaded compression members for flexural buckling only according to the provisions of Articles 6.9.2.1, 6.9.4.1.1, and 6.9.4.1.2. It should be noted that according to Article 6.9.4.4, the actual maximum slenderness ratio of the angle, not the effective slenderness ratio, is not to exceed the limiting slenderness ratio specified in Article 6.9.3 as checked above. Also, per Article 6.9.4.4, single angles designed using $(K\ell/r)_{eff}$ shall not be checked for flexural-torsional buckling.

Compute the effective slenderness ratio per Article 6.9.4.4 based on the criteria for equal-leg angles. The length, ℓ, is defined as the distance between the work points of the joints measured along the length of the angle, which is conservatively assumed to be equal to the full diagonal distance of 13 feet in this example. First, check the ℓ/r_x limit of 80:

$$\frac{\ell}{r_x} = \frac{(13)(12)}{2.46} = 63.4 < 80$$

where: r_x = radius of gyration about the geometric axis of the angle parallel to the connected leg (Although not relevant for equal-leg angles, the term r_x may actually equal r_{yy} when unequal-leg angles are used.)

Therefore, compute the effective slenderness ratio as follows:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} \qquad \text{Eq. (6.9.4.4-1)}$$

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{(13)(12)}{2.46} = 120$$

In accordance with the provisions for single-angle members in Article 6.9.4.4 and using the effective slenderness ratio, $(k\ell/r)_{eff}$, the factored resistance of the angle in compression is to be taken as:

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance determined using the provisions of Article 6.9.4.1.1
ϕ_c = resistance factor for compression as specified in Article 6.5.4.2

To compute P_n, first compute P_e and P_o. P_e is the elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, which is the applicable buckling mode for single angles. P_o is the equivalent nominal yield resistance equal to QF_yA_g, where Q is the slender element reduction factor determined as specified in Article 6.9.4.2. Q is taken as 1.0 in this case according to Article 6.9.4.2.1 since the angle member is nonslender per Eq. (6.9.4.2.1-1).

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g$$ Eq. (6.9.4.1.2-1)

where $(K\ell/r)_{eff}$ is used in place of $(K\ell/r_s)$ in the denominator.

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r}\right)_{eff}^2} A_g = \frac{\pi^2 (29{,}000)}{(120)^2}(11.5) = 229 \text{ kips}$$

$$P_o = QF_y A_g = (1.0)(50)(11.5) = 575 \text{ kips}$$

Since

$$\frac{P_e}{P_o} = \frac{229}{575} = 0.40 < 0.44,$$

the nominal axial resistance in compression is computed as:

$$P_n = 0.877 P_e$$ Eq. (6.9.4.1.1-2)

$$P_n = 0.877(229) = 201 \text{ kips}$$

Compute the factored axial resistance of the angle in compression as follows:

$P_r = \phi_c P_n = 0.95(201) = 191$ kips

$P_u = |-88\text{kips}| < P_r = 191$ kips OK

7.13.2 Cross Frame Fatigue Check

The fatigue of the cross frame member is checked assuming that the diagonal is connected to a gusset plate with fillet welds. From the analysis, the maximum range of unfactored fatigue force due to one cycle of stress in any diagonal in the bridge is 22.7 kips. To cause one cycle of this computed stress range requires two vehicles to traverse the bridge in separate transverse positions with one vehicle leading the other. Per Article C6.6.1.2.1, unless otherwise prescribed by the owner, it is recommended that the one cycle of stress be taken as 75 percent of the stress range in the member determined by the passage of the factored fatigue load in two different transverse positions as just described. This 75 percent is not related to, and is in addition to the live load factor for the Fatigue load combination. The reduction in load (to 75% percent) is intended to approximate the low probability of two vehicles being located in the critical relative positions that cause the maximum stress range, over millions of cycles. However, in no case should the calculated range of stress be less than the tension stress caused by loading one lane.

In the case of this example, the tension force caused by the loading of one lane is taken as 15 kips. The following comparison is made (note that the preceding language in Article C6.6.1.2.1 has been superseded in the 2015 Interims to the 7th Edition):

Unfactored calculated fatigue force range = (0.75)(22.7 kips) = 17.0 kips (governs)
Unfactored fatigue tension force caused by loading one lane = 15.0 kips

Thus, the calculated fatigue force range of 17.0 kips will be used for the fatigue design.

Condition 7.2 from Table 6.6.1.2.3-1 applies, which corresponds to the base metal in an angle section connected to a gusset or connection plate by longitudinal fillet welds along both sides of the connected element of the member cross-section. Therefore Detail Category E applies. From Table 6.6.1.2.3-2, the 75-year $(ADTT)_{SL}$ equivalent to infinite fatigue life for a Category E fatigue detail is 3,530 trucks per day. Therefore, since the assumed $(ADTT)_{SL}$ for this design example of 1,000 trucks per day is less than this limit of 3,530 trucks per day, the detail should be checked for finite fatigue life using the Fatigue II load combination. Table 3.4.1-1 requires that a load factor of 0.75 be applied to the force range for checking Fatigue II.

Factored fatigue force range = (0.75)(17.0) = 12.8 kips

To account for shear lag effects in the single angle cross frame member, the factored fatigue force range should be divided by the effective area. The effective area is calculated in accordance with Table 6.6.1.2.3-1, Description 7.2, where the effective area is computed as:

$$A_{eff} = \left(1 - \frac{\bar{x}}{L}\right) A_g$$

where: $\bar{x}$ = connection eccentricity (in.)
L = maximum length of longitudinal welds (in.)

The length of the longitudinal weld on each side of the angle is taken as 7.0 inches, based on calculations in the following section. Therefore the effective area and factored fatigue stress range are computed as:

$$A_{eff} = \left(1 - \frac{2.26}{7.0}\right)(11.5) = 7.79\ \text{in.}^2$$

$$\text{Factored fatigue stress range} = \frac{12.8}{7.79} = 1.64\ \text{ksi}$$

Per Article 6.6.1.2.5, the nominal fatigue resistance for finite fatigue life is equal to:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}}$$

in which:

$$N = (365)(75)n(ADTT)_{SL}$$

From Table 6.6.1.2.5-2, the number of cycles per truck passage, n, for transverse members spaced at 20 ft or less is 2.0. Therefore:

$$N = (365)(75)(2.0)(1000) = 54.75 \text{ x } 10^6 \text{ cycles}$$

From Table 6.6.1.2.5-1, the detail category constant, A, for a Category E detail is 11 x 10^8 $ksi^{3.}$ Therefore,

$$(\Delta F)_n = \left(\frac{11 \text{ x } 10^8}{54.75 \text{ x } 10^6} \right)^{\frac{1}{3}} = 2.72 \text{ ksi} > 1.64 \text{ ksi} \quad \text{OK}$$

Based on the relatively low performance ratios for compressive resistance and fatigue calculated above, the use of a smaller-size angle might possibly be considered.

7.13.3 Cross Frame Welded Connection

According to Article C6.13.3.2.4b, the factored resistance of fillet-welded connections subjected to shear along the length of the weld is taken as the lesser of the factored resistance of the base metal or weld metal. A 5/16" fillet weld (w = 5/16") and E70XX electrodes are assumed.

According to the provisions of Article 6.13.3.2.4b, the resistance of the welded connection is taken as the product of the effective area of the weld and the factored resistance, R_r, of the welded connection in terms of stress. More commonly, the effective throat (0.707w) is multiplied by the factored resistance, R_r, to get strength in terms of force per length.

The factored resistance of the weld metal is:

$$R_r = 0.6\phi_{e2}F_{exx} \qquad \text{Eq. (6.13.3.2.4b-1)}$$

$$R_r = 0.6(0.80)(70) = 33.6 \text{ ksi}$$

Weld failure rarely occurs in the base metal. However, as explained in Article C6.13.3.2.4b, since “overstrength” weld metal is used, the capacity can be governed by the weld leg and the shear rupture resistance of the base metal. The factored shear rupture resistance of the base metal is (Article 6.13.5.3):

$$R_r = \phi_{vu}(0.58F_u)$$

$$R_r = 0.80(0.58)(65) = 30.2 \text{ ksi (controls)}$$

Therefore, the base metal governs in this case, and the factored resistance of the welded connection per length of weld is:

$$0.707wR_r = 0.707(0.3125)(30.2) = 6.67 \text{ kips/in.}$$

Therefore, the length of weld required to resist the Strength I factored axial load is computed as:

$$\frac{88}{6.67} = 13.2 \text{ in.} \quad \text{use 7.0 in. longitudinal welds on each side of the angle}$$

It is generally preferable to weld the angle all around to the gusset plate to provide the best seal against moisture. The sealing weld wrapping the end of the angle should not be considered in determining the resistance of the connection. The gusset plate must be sized appropriately to allow for the minimum required weld length to be provided.

The gusset plate should be of at least the same thickness as the angle, have at least the same equivalent net area, and have sufficient capacity to transfer resultant cross frame forces to the girder. The design of the gusset plate is not covered in this design example. However the gusset plate should be designed for shear, compression, tension, or a combination thereof, as applicable.

The gusset plate is bolted to the connection plate, which is welded to the girder web and flanges. The diagonal is attached near the bottom flange of G4. The bottom chord carries 40 kips out of the connection. The resultant force from the bottom chord and diagonal forces, and moment due to eccentricity must be considered in the design of the bolt group connecting the gusset plate to the connection plate. Also, the welds between the connection plate and bottom flange must be able to transfer the resultant force. The design of the bolt group and the welded connection of the connection plate to the girder are not covered in this design example

7.14 Shear Connector Design

Shear connectors are to be provided throughout the entire length of a curved continuous composite bridge according to the provisions of Article 6.10.10.1. In order to demonstrate the design of shear connectors, the required number of shear connectors will be determined for Girder 4 of Span 1. The following calculations illustrate the design for the strength and the fatigue limit states.

7.14.1 Shear Connector Design for Strength – Girder G4, Span 1

Compute the number of shear connectors required for the strength limit state in Span 1 according to the provisions of Article 6.10.10.4.

The factored shear resistance of a single connector, Q_r, at the strength limit state is taken as:

$$Q_r = \phi_{sc} Q_n \qquad \text{Eq. (6.10.10.4.1-1)}$$

where: Q_n = nominal shear resistance of a single shear connector determined as specified in Article 6.10.10.4.3 (kips)

ϕ_{sc} = resistance factor for shear connectors specified in Article 6.5.4.2

Shear connectors that are 6 in. long by 7/8 in. diameter are selected for design. Compute the nominal resistance of one shear connector embedded in the concrete deck using Article 6.10.10.4.3.

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \qquad \text{Eq. (6.10.10.4.3-1)}$$

where: A_{sc} = cross-sectional area of a stud shear connector

E_c = modulus of elasticity of the deck concrete = 3,834 ksi (calculated previously)

F_u = specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 (ksi)

$$A_{sc} = \frac{\pi (0.875)^2}{4} = 0.60 \text{ in.}^2$$

$$Q_n = 0.5(0.60)\sqrt{(4)(3{,}834)} = 37.2 \text{ kips}$$

$$A_{sc} F_u = 0.60(60) = 36 \text{ kips} \quad \text{(controls)}$$

Therefore, use Q_n = 36 kips.

Compute the nominal shear force, P, according to the provisions of Article 6.10.10.4.2. For the shear connector design, Span 1 is divided into two regions: 1) the portion between the end of the span and the location of maximum positive live load moment and 2) the portion between the maximum positive live load moment and the adjacent interior support.

7.14.1.1 End of Span to Maximum Positive Moment Location

Between the end of Span 1 and the location of maximum positive live load plus impact moment, Eq. (6.10.10.4.2-1) is applicable. For this portion of Span 1, the total nominal shear force and required pitch are computed in the following calculations.

The total nominal shear force in this portion of the span is computed as follows:

$$P = \sqrt{P_p^{\,2} + F_p^{\,2}} \qquad \text{Eq. (6.10.10.4.2-1)}$$

where: P_p = total longitudinal force in the concrete deck at the point of maximum positive live load plus impact moment (kips) taken as the lesser of either:

$$P_{1p} = 0.85f'_c b_s t_s \qquad \text{Eq. (6.10.10.4.2-2)}$$

or

$$P_{2p} = F_{yw}Dt_w + F_{yt}b_{ft}t_{ft} + F_{yc}b_{fc}t_{fc} \qquad \text{Eq. (6.10.10.4.2-3)}$$

F_p = total radial force in the concrete deck at the point of maximum positive live load plus impact moment (kips) taken as:

$$F_p = P_p \frac{L_p}{R} \qquad \text{Eq. (6.10.10.4.2-4)}$$

b_s = effective width of the concrete deck (in.)
L_p = arc length between an end of the girder and an adjacent point of maximum positive live load plus impact moment (ft)
R = minimum girder radius over the length, L_p (ft)

The effective width of the concrete deck, b_s, is calculated according to Article 4.6.2.6.1 for an exterior girder, calculated previously as 111 in. Conservatively, since G4 is an exterior girder with an overhang less than half of the girder spacing, the width of the deck could have been assumed to be equal to the interior girder effective width so that all girders would have the same stud spacing. That approach is not taken here.

$$P_{1p} = 0.85(4)(111)(9) = 3{,}397 \text{ kips}$$

$$P_{2p} = 50(84)(0.5625) + 50(21)(1.5) + 50(20)(1.0) = 4{,}938 \text{ kips}$$

The total longitudinal force in the deck, $P_{p,}$ is the lesser of P_{1p} or P_{2p}; therefore, P_p is taken to be 3,397 kips.

The arc length, L_p, between the end of the girder and the point of maximum positive live load plus impact moment is 73 feet. The total radial shear force in the concrete deck, F_p, at the point of maximum positive live load plus impact moment is computed as follows.

$$F_p = (3{,}397)\left(\frac{73}{716.5}\right) = 346.1 \text{ kips}$$

Therefore, the total nominal shear force in this portion of the span is:

$$P = \sqrt{3{,}397^2 + 346.1^2} = 3{,}415 \text{ kips}$$

The minimum number of shear connectors, n, over the region under consideration is taken as:

$$n = \frac{P}{Q_r} = \frac{P}{\phi_{sc} Q_n}$$ Eq. (6.10.10.4.1-2)

$$n = \frac{3{,}415}{0.85(36)} = 112$$

Compute the required pitch, p, with 3 studs per row.

$$\text{No. of rows} = \frac{112}{3} = 37.3 \text{, say 38 rows}$$

$$p = \frac{73(12)}{(38-1)} = 23.7 \text{ in.}$$

The shear connector pitch for strength is less critical than for fatigue in this region, which is demonstrated later in this example.

7.14.1.2 Maximum Positive Moment Location to Adjacent Interior Support

Between the location of maximum positive live load plus impact moment and the adjacent interior support, Eq. (6.10.10.4.2-5) is applicable. For this portion of Span 1, the total nominal shear force and required pitch are computed in the following calculations.

The total nominal shear force in this portion of the span is computed as follows:

$$P = \sqrt{P_T^{\,2} + F_T^{\,2}}$$ Eq. (6.10.10.4.2-5)

where: P_T = total longitudinal force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips) taken as:

$$P_T = P_p + P_n$$ Eq. (6.10.10.4.2-6)

P_n = total longitudinal force in the concrete deck over an interior support (kips) taken as the lesser of either:

$$P_{1n} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc}$$ Eq. (6.10.10.4.2-7)

or

$$P_{2n} = 0.45 f'_c \, b_s t_s$$ Eq. (6.10.10.4.2-8)

F_T = total radial force in the concrete deck between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (kips) taken as:

$$F_T = P_T \frac{L_n}{R} \qquad \text{Eq. (6.10.10.4.2-9)}$$

L_n = arc length between the point of maximum positive live load plus impact moment and the centerline of an adjacent interior support (ft)

R = minimum girder radius over the length, L_n (ft)

The following two terms were computed previously and are applicable here as well:

P_p = 3,397 kips

b_s = 111 in.

Using the plate girder dimensions at Support 2 (Field Section 2), compute P_{1n} as follows:

$$P_{1n} = 50(84)(0.625) + 50(28)(2.5) + 50(27)(3) = 10{,}175 \text{ kips}$$

$$P_{2n} = 0.45(4)(111)(9) = 1{,}798 \text{ kips}$$

The total longitudinal force in the deck over the interior support, $P_{n,}$ is the lesser of P_{1n} or P_{2n}; therefore, P_n is taken to be 1,798 kips.

Therefore, the total longitudinal force in the concrete deck in the region under consideration is:

$$P_T = 3{,}397 + 1{,}798 = 5{,}195 \text{ kips}$$

Next, compute the arc length, L_n, and the total radial force in the concrete deck, F_T, in the region under consideration. The total arc length along girder G4 in Span 1 is 163.8 ft.

$$L_n = 163.8 - 73 = 90.8 \text{ ft}$$

$$F_T = 5{,}195\left(\frac{90.8}{716.5}\right) = 658 \text{ kips}$$

The total nominal shear force in this portion of the span is:

$$P = \sqrt{5{,}195^2 + 658^2} = 5{,}237 \text{ kips}$$

The minimum number of shear connectors, n, over the region under consideration is taken as:

$$n = \frac{P}{Q_r} = \frac{P}{\phi_{sc} Q_n}$$ Eq. (6.10.10.4.1-2)

$$n = \frac{5{,}237}{0.85(36)} = 171.1\text{, say 172}$$

Compute the required pitch, p, with 3 studs per row.

$$\text{No. of rows} = \frac{172}{3} = 57.3\text{, say 58 rows}$$

$$p = \frac{90.8(12)}{(58-1)} = 19.1\,\text{in.}$$

The shear connector pitch for strength is less critical than for fatigue in this region, which is demonstrated later in this example.

7.14.2 Shear Connector Design for Fatigue – Girder G4, Span 1

To demonstrate the fatigue requirements for shear connectors, fatigue will be checked at the maximum positive moment location and at the first interior support (Support 2).

7.14.2.1 Maximum Positive Moment Location

Determine the required pitch of the shear connectors for fatigue at this section according to the provisions of Article 6.10.10.1.2. The pitch, p, of shear connectors must satisfy the following:

$$p \le \frac{nZ_r}{V_{sr}}$$ Eq. (6.10.10.1.2-1)

where: n = number of shear connectors in a cross-section

Z_r = shear fatigue resistance of an individual shear connector determined as specified in Article 6.10.10.2 (kips)

V_{sr} = horizontal fatigue shear range per unit length (kips/in.)

The 75-year single lane Average Daily Truck Traffic $(ADDTT)_{SL}$ is assumed to be 1,000 trucks per day. Where the projected 75-year $(ADDT)_{SL}$ is greater than or equal to 960 trucks per day, the fatigue resistance for an individual stud shear connector, Z_r, is defined in Article 6.10.10.2 as follows:

$$Z_r = 5.5d^2$$ Eq. (6.10.10.2-1)

The Fatigue I load combination is to be used for this case according to Article 6.10.10.2. As stated earlier, shear connectors that are 6 in. long by 7/8 in. diameter are selected for design, with 3 studs per row. The fatigue resistance of one shear connector is computed as follows:

$$Z_r = 5.5(0.875)^2 = 4.21 \text{ kips}$$

The fatigue resistance for 3 shear connectors is:

$$nZ_r = 3(4.21) = 12.63 \text{ kips/row}$$

From Table 10, the unfactored shear force range at this location due to one fatigue truck is:

$$20 + |-20| = 40 \text{ kips}$$

The Fatigue I factored shear force range is:

$$V_f = 1.5(40) = 60 \text{ kips}$$

According to the provisions of Article 6.6.1.2.1, the live load stress range may be calculated using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure. The structural deck thickness, t_s, is 9.0 inches; the modular ratio, n, equals 7.56; and the effective flange width is 111 inches (calculated previously).

In order to compute the longitudinal shear range, first compute the transformed deck area as follows:

$$\text{Transformed deck area} = \frac{\text{Area}}{n} = \frac{(111)(9)}{7.56} = 132.1 \text{ in.}^2$$

Compute the first moment of the transformed short-term area of the concrete deck, Q, with respect to the neutral axis of the uncracked live load short-term composite section. Determine the distance from the center of the deck to the neutral axis. Section properties are taken from Table 13. The neutral axis of the short-term composite section is 16.25 in. measured from the top of the top flange.

Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_s/2$

$$\text{Moment arm of the deck} = 16.25 - 1 + 4 + \frac{9}{2} = 23.75 \text{ in.}$$

$$Q = 132.1(23.75) = 3{,}137 \text{ in.}^3$$

Compute the longitudinal fatigue shear range per unit length, V_{fat}:

$$V_{fat} = \frac{V_f Q}{I} = \frac{60(3{,}137)}{294{,}158} = 0.64 \text{ kips/in. (factored)}$$

It is also necessary to compute F_{fat}, the radial fatigue shear range per unit length. Article 6.10.10.1.2 directs the designer to compute F_{fat} by taking the larger of two computed values from Eqs. (6.10.10.1.2-4) and (6.10.10.1.2-5). The first equation is an approximation based on the stress in the flange and the radius of curvature. The second equation is a more exact calculation based on the actual cross frame force from the analysis. As explained in Article C6.10.10.1.2, the first equation typically governs unless torsion is caused by effects other than curvature, such as skew. In this example, the two equations are expected to yield similar results since all the torsion is due to curvature. As permitted in Article 6.10.10.1.2, for straight or horizontally curved bridges with skew not exceeding 20 degrees, the radial fatigue shear range from Eq. (6.10.10.1.2-5) may be taken equal to zero. Therefore, in this case, $F_{fat2} = 0$ and $F_{fat} = F_{fat1}$.

$$F_{fat1} = \frac{A_{bot} \sigma_{flg} \ell}{wR} \qquad \text{Eq. (6.10.10.1.2-4)}$$

where: σ_{flg} = range of longitudinal fatigue stress in the bottom flange without consideration of flange lateral bending (ksi)

ℓ = distance between brace points (ft)

w = effective length of deck (in.) taken as 48.0 in.

The stress range σ_{flg} is based on the range of fatigue moment taken from Table 9:

Unfactored fatigue moment range = $|-603| + 1{,}603 = 2{,}206 \text{ kip-ft}$

The section properties are again taken from Table 13. Using the load factor of 1.5 for Fatigue I, the range of longitudinal fatigue stress in the bottom flange is computed as follows:

$$\sigma_{flg} = (1.5)\left(\frac{2{,}206}{4{,}187}\right)(12) = 9.48 \text{ ksi (factored)}$$

A_{bot} = (21)(1.5) = 31.5 in.^{2}

$$F_{fat1} = \frac{31.5(9.48)(20)}{48(716.5)} = 0.17 \text{ kips/in.}$$

$F_{fat} = F_{fat1}$ = 0.17 kips/in. (factored)

The positive and negative longitudinal shears due to major-axis bending are due to the fatigue vehicle located in Span 1 with the back axle on the left and then on the right of the point under consideration. This means that the truck actually has to turn around to produce the computed longitudinal shear range. The positive and negative radial shear ranges are produced by loading first in Span 1 and then in Span 2. Again, this is not a realistic loading case to combine with the longitudinal shear case but has been done to be practical and to be conservative. Combining the longitudinal and radial fatigue shear ranges vectorially, the total horizontal fatigue shear range per unit length is computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2} \qquad \text{Eq. (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{(0.64)^2 + (0.17)^2} = 0.66 \text{ kips/in.}$$

Compute the required shear connector pitch for fatigue for 3 studs per row.

$$p \le \frac{nZ_r}{V_{sr}} \qquad \text{Eq. (6.10.10.1.2-1)}$$

$$p \le \frac{12.63}{0.66} = 19.1 \text{ in./row}$$

As shown earlier, the number of shear connectors was also checked for the strength limit state according to the provisions of Article 6.10.10.4. The required pitch for fatigue, 19.1 in./row, governs.

7.14.2.2 Interior Support Location (Support 2)

Using the same procedure illustrated at the maximum positive moment location, fatigue requirements for shear connectors are investigated at the first interior support (Support 2).

Determine the required pitch of the shear connectors for fatigue at this section according to the provisions of Article 6.10.10.1.2. As before, the pitch, p, of shear connectors must satisfy the following:

$$p \le \frac{nZ_r}{V_{sr}} \qquad \text{Eq. (6.10.10.1.2-1)}$$

The calculation of the fatigue resistance, nZ_r, is the same as performed at the maximum positive moment location. For 3 shear connectors per row, nZ_r = 12.63 kips/row.

From Table 10 at Section G4-2, the unfactored shear force range at this location due to one fatigue truck is:

$$3 + |-55| = 58 \text{ kips}$$

The Fatigue I factored shear force range is:

$$V_f = 1.5(58) = 87 \text{ kips}$$

According to the provisions of Article 6.6.1.2.1, the live load stress range may be calculated using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure. The structural deck thickness, t_s, is 9.0 inches; the modular ratio, n, equals 7.56; and the effective flange width is 111 inches (calculated previously).

Compute the first moment of the transformed short-term area of the concrete deck, Q, with respect to the neutral axis of the uncracked live load short-term composite section. Determine the distance from the center of the deck to the neutral axis. Section properties are taken from Table 16. The neutral axis of the short-term composite section is 26.10 in. measured from the top of the top flange.

Moment arm of the deck = Neutral axis - t_{flg} + haunch + $t_s/2$

$$\text{Moment arm of the deck} = 26.10 - 2.5 + 4 + \frac{9}{2} = 32.10\ \text{in.}$$

Transformed deck area = 132.1 in.^{2} (computed previously)

$$Q = 132.1(32.10) = 4{,}240\ \text{in.}^3$$

Compute the longitudinal fatigue shear range per unit length, V_{fat}:

$$V_{fat} = \frac{V_f Q}{I} = \frac{87(4{,}240)}{539{,}403} = 0.68\ \text{k/in. (factored)}$$

Compute the radial shear range, F_{fat}, based on Eq. (6.10.10.1.2-4). As explained previously, per Article 6.10.10.1.2 the radial fatigue shear range from Eq. (6.10.10.1.2-5) may be taken equal to zero in this case. Therefore, in this case, $F_{fat2} = 0$ and $F_{fat} = F_{fat1}$.

$$F_{fat1} = \frac{A_{bot}\sigma_{flg}\ell}{wR} \qquad \text{Eq. (6.10.10.1.2-4)}$$

The stress range σ_{flg} is based on the range of fatigue moment taken from Table 9:

$$\text{Unfactored fatigue moment range} = |-1{,}315| + 351 = 1{,}666\ \text{kip-ft}$$

The section properties are again taken from Table 16. Using the load factor of 1.5 for Fatigue I, the range of longitudinal fatigue stress in the bottom flange is computed as follows:

$$\sigma_{flg} = (1.5)\left(\frac{1{,}666}{8{,}508}\right)(12) = 3.52\ \text{ksi (factored)}$$

A_{bot} = (27)(3.0) = 81.0 in.^{2}

$$F_{fat1} = \frac{81.0(3.52)(20)}{48(716.5)} = 0.17 \text{ kips/in.}$$

$F_{fat} = F_{fat1} = 0.17$ kips/in. (factored)

Combining the longitudinal and radial fatigue shear ranges vectorially, the total horizontal fatigue shear range per unit length is computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2} \qquad \text{Eq. (6.10.10.1.2-2)}$$

$$V_{sr} = \sqrt{(0.68)^2 + (0.17)^2} = 0.70 \text{ kips/in.}$$

Compute the required shear connector pitch for fatigue for 3 studs per row.

$$p \leq \frac{nZ_r}{V_{sr}} \qquad \text{Eq. (6.10.10.1.2-1)}$$

$$p \leq \frac{12.63}{0.70} = 18.0 \text{ in./row}$$

As shown earlier, the number of shear connectors was also checked for the strength limit state according to the provisions of Article 6.10.10.4. The required pitch for fatigue, 18.0 in./row, governs.

7.15 Bearing Stiffener Design

Bearing stiffeners are designed as columns to resist the reactions at bearing locations. According to Article 6.10.11.2.1, bearing stiffeners must be placed on the webs of built-up sections at all bearing locations. At bearing locations on rolled shapes and at other locations on built-up sections or rolled shapes subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, either bearing stiffeners must be provided or else the web must be investigated for the limit states of web crippling or web local yielding according to the provisions of Article D6.5 (Appendix D6). It should be noted that the provisions of Article D6.5 should be checked whenever girders are incrementally launched over supports.

Bearing stiffeners must extend the full depth of the web and as closely as practical to the outer edges of the flanges. Each stiffener must be either milled to bear against the flange through which it receives its load or attached to that flange by a full penetration groove weld. Typical practice is for the bearing stiffeners to be milled to bear plus fillet welded to the appropriate flange, regardless of whether or not a cross frame or diaphragm is connected to the stiffeners. Full penetration groove welds are costly and often result in welding deformation of the flange.

The design of bearings stiffeners at Support 1 for Girder G4 is illustrated in this example. Grade 50 (F_{ys} = 50 ksi) steel is selected for the bearing stiffeners.

Girder G4 has the largest total reaction at the simple end support (Support 1). Unfactored reactions are shown below. These results are directly from the three-dimensional analysis as presented in Table 10.

Steel Dead Load:	$R_{DC1-STEEL}$	= 23 kips
Concrete Deck Dead Load:	$R_{DC1-CONC}$	= 92 kips
Composite Dead Load:	R_{DC2}	= 23 kips
Future Wearing Surface Dead Load:	R_{DW}	= 19 kips
Live Load (including IM + CF):	R_{LL+IM}	= 143 kips

The Strength I factored reaction is computed as:

$$R_u = 1.25(23 + 92 + 23) + 1.50(19) + 1.75(143) = 451 \text{ kips}$$

7.15.1 Projecting Width

The width, b_t, of each projecting stiffener element must satisfy:

$$b_t \le 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \qquad \text{Eq. (6.10.11.2.2-1)}$$

Use a bearing stiffener thickness of 0.75 inches.

$$b_t \le 0.48(0.75)\sqrt{\frac{29{,}000}{50}} = 8.7$$

Select two 7.0-inch wide by 0.75-inch thick stiffeners, one stiffener on each side of the web.

7.15.2 Bearing Resistance

According to Article 6.10.11.2.3, the factored bearing resistance for the fitted ends of bearing stiffeners is taken as:

$$(R_{sb})_r = \phi_b (R_{sb})_n \qquad \text{Eq. (6.10.11.2.3-1)}$$

where: $(R_{sb})_n$ = nominal bearing resistance for the fitted ends of the bearing stiffeners (kips)

$$(R_{sb})_n = 1.4 A_{pn} F_{ys} \qquad \text{Eq. (6.10.11.2.3-2)}$$

ϕ_b = resistance factor for bearing specified in Article 6.5.4.2

A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds but not beyond the edge of the flange (in^2)
F_{ys} = specified minimum yield strength of the stiffener (ksi)

$$A_{pn} = 2(7-1)(0.75) = 9.0\,\text{in.}^2$$ (Assume 1 in. for the stiffener clip.)

The nominal bearing resistance is:

$$(R_{sb})_n = 1.4(9)(50) = 630\,\text{kips}$$

The factored bearing resistance is:

$$(R_{sb})_r = 1.0(630) = 630\,\text{kips} > R_u = 451\,\text{kips} \quad \text{OK}$$

7.15.3 Axial Resistance

Determine the axial resistance of the bearing stiffener according to Article 6.10.11.2.4. This article directs the Engineer to Article 6.9.2.1 for calculation of the factored axial resistance, P_r. The yield strength is F_{ys}, the radius of gyration is computed about the midthickness of the web, and the effective length is 0.75 times the web depth ($K\ell$ = 0.75D).

$$P_r = \phi_c P_n \qquad \text{Eq. (6.9.2.1-1)}$$

where: P_n = nominal compressive resistance determined using the provisions of Article 6.9.4
ϕ_c = resistance factor for compression as specified in Article 6.5.4.2

As indicated in Article 6.9.4.1.1, P_n is the smallest value of the applicable modes of buckling, and in the case of bearing stiffeners, torsional buckling and flexural-torsional buckling are not applicable. Therefore, P_n is computed for flexural buckling only.

To compute P_n, first compute P_e and P_o. P_e is the elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling. P_o is the equivalent nominal yield resistance equal to QF_yA_g, where Q is the slender element reduction factor, taken equal to 1.0 for bearing stiffeners per Article 6.9.4.1.1

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad \text{Eq. (6.9.4.1.2-1)}$$

Compute the effective length of the bearing stiffener according to Article 6.10.11.2.4.

$$K\ell = 0.75(84) = 63\,\text{in.}$$

Compute the radius of gyration about the midthickness of the web.

$$r_s = \sqrt{\frac{I_s}{A_s}}$$

According to the provisions of Article 6.10.11.2.4b, for stiffeners welded to the web, a portion of the web is to be included as part of the effective column section. For stiffeners consisting of two plates welded to the web, the effective column section is to consist of the two stiffener elements, plus a centrally located strip of web extending $9t_w$ on each side of the outer projecting elements of the group. The area of the web that is part of the effective section is computed as follows:

$$A_w = 2(9)(0.5625)(0.5625) = 5.7 \text{ in.}^2$$

Use the full area of the stiffeners to compute the axial resistance.

$$A = 2(7)(0.75) = 10.5 \text{ in.}^2$$

The total area of the effective section is therefore:

$$A_s = 5.7 + 10.5 = 16.2 \text{ in.}^2$$

Next, compute the moment of inertia of the effective section, conservatively neglecting the web strip:

$$I = \frac{0.75(7.0 + 0.5625 + 7.0)^3}{12} = 193 \text{ in.}^4$$

Compute the radius of gyration:

$$r_s = \sqrt{\frac{193}{16.2}} = 3.45 \text{ in.}$$

The elastic critical buckling resistance is computed as follows:

$$P_e = \frac{\pi^2 (29{,}000)}{\left(\frac{63}{3.45}\right)^2}(16.2) = 13{,}905 \text{ kips}$$

The equivalent nominal yield resistance is computed as follows, with A_s used for A_g:

$$P_e = QF_yA_g = (1.0)(50)(16.2) = 810 \text{ kips}$$

Since,

$$\frac{P_e}{P_o} = \frac{13{,}905}{810} = 17.2 > 0.44,$$

the nominal axial compression resistance is computed as:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad \text{Eq. (6.9.4.1.1-1)}$$

$$P_n = \left[0.658^{\frac{1}{17.2}}\right](810) = 790 \text{ kips}$$

The factored resistance of the bearing stiffeners is computed as follows:

$Pr = \phi_c P_n = 0.95(790) = 750$ kips

$P_u = 451$ kips $< P_r = 750$ kips OK

The bearing stiffeners selected for Girder G4 at Support 1 satisfy the requirements for design.

8.0 SUMMARY OF DESIGN CHECKS AND PERFORMANCE RATIOS

The results for this design example at each limit state are summarized below for the maximum positive moment and maximum negative moment locations. The results for each limit state are expressed in terms of a performance ratio, defined as the ratio of a calculated value to the corresponding resistance.

8.1 Maximum Positive Moment Region, Span 1 (Section G4-1)

Constructibility	
Flexure (STRENGTH I)	
Eq. (6.10.3.2.1-1) – Top Flange, yielding	0.852
Eq. (6.10.3.2.1-2) – Top Flange, local buckling	0.726
Eq. (6.10.3.2.1-2) - Top Flange, lateral torsional buckling	0.792
Eq. (6.10.3.2.1-3) – Top Flange, web bend buckling	0.927
Eq. (6.10.3.2.2-1) – Bottom Flange, yielding	0.618
Service Limit State	
Permanent Deformations (SERVICE II)	
Eq. (6.10.4.2.2-1) – Top Flange	0.474
Eq. (6.10.4.2.2-2) – Bottom Flange	0.832
Fatigue Limit State	
Flexure (FATIGUE I)	
Eq. (6.6.1.2.2-1) – Bottom Flange	0.894
Strength Limit State	
Ductility Requirement – Eq. (6.10.7.3-1)	0.309
Flexure (STRENGTH I)	
Eq. (6.10.7.2.1-1) – Top Flange	0.576
Eq. (6.10.7.2.1-2) – Bottom Flange	1.004
Eq. (6.10.1.6-1) – Bottom Flange	0.354

8.2 Interior Support, Maximum Negative Moment (Section G4-2)

Constructibility	
Flexure (STRENGTH I)	
Eq. (6.10.3.2.2-1) – Top Flange, yielding	0.533
Eq. (6.10.3.2.1-1) – Bottom Flange, yielding	0.483
Eq. (6.10.3.2.1-2) – Bottom Flange, local buckling	0.440
Eq. (6.10.3.2.1-2) – Bottom Flange, lat. torsional buckling	0.457
Service Limit State (SERVICE II)	
Web Bend-Buckling - Eq. (6.10.4.2.2-4)	0.675
Fatigue Limit State	
Flexure (FATIGUE I)	
Eq. (6.6.1.2.2-1) – Top Flange	0.109

Strength Limit State

Flexure (STRENGTH I)

Eq. (6.10.8.1.1-1) – Bottom Flange, local buckling	0.923
Eq. (6.10.8.1.1-1) – Bottom Flange, lat. torsional buckling	0.959
Eq. (6.10.8.1.3-1) – Top Flange, yielding	0.950
Shear (STRENGTH I) – Eq. (6.10.9.1-1)	0.484

8.3 End Support (Section G4-3)

Strength Limit State (STRENGTH I)

Shear – Eq. (6.10.9.1-1)	0.791

9.0 REFERENCES

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