

## 6.6.1.2.5 Fatigue Resistance

Revise Eq. (6.6.1.2.5-1) and (6.6.1.2.5-2) as follows:

For finite fatigue life ( $N \leq N_{TH}$ )

$$\underline{(\Delta F_n) = \left(\frac{A}{N}\right)^{\frac{1}{3}} \quad (6.6.1.2.5-1a)}$$

For infinite fatigue life ( $N > N_{TH}$ )

$$\underline{(\Delta F_n) = (\Delta F)_{TH} \quad (6.6.1.2.5-1b)}$$

in which:

$$N = (365)(75)n(ADTT)_{SL} \quad (6.6.1.2.5-2a)$$

$$\underline{N_{TH} = \frac{A}{[(\Delta F)_{TH}]^3} \quad (6.6.1.2.5-2b)}$$

where:

$N_{TH}$  = minimum number of stress cycles corresponding to constant-amplitude fatigue threshold,  $(\Delta F)_{TH}$

## C6.6.1.2.5

Revise the 2<sup>nd</sup> paragraph as follows:

The requirement on higher-traffic-volume bridges that the maximum stress range experienced by a detail be less than the constant-amplitude fatigue threshold provides a theoretically infinite fatigue life. ~~The maximum stress range is assumed to be twice the live load stress range due to the passage of the fatigue load, factored in accordance with the load factor in Table 3.4.1-1 for the fatigue load combination.~~

Revise the 6<sup>th</sup> paragraph as follows:

When the maximum design stress range is less than ~~one-half of the~~ constant-amplitude fatigue threshold, the detail will theoretically provide infinite life. When the number of stress cycles induced by HL-93 fatigue truck exceeds  $N_{TH}$ , the details shall be checked for the infinite life. Except for Categories E and E', for higher traffic volumes, the design will most often be governed by the infinite life check. Table C1 shows the values of  $N_{TH}$  and  $(ADTT)_{SL}$  above which the infinite life check governs, assuming a 75-year design life and one cycle per truck. It is obvious that when  $ADTT$  is taken as 2500, all details should be checked for the infinite life.

Revise Table C6.6.1.2.5-1 as follows:

Table C6.6.1.2.5-1  $N_{TH}$  and 75-Year  $(ADTT)_{SL}$  Equivalent to Infinite Life.

Detail Category	$N_{TH}$ (Number of Cycles Equivalent to Infinite Life)	75-Year $(ADTT)_{SL}$ Equivalent to Infinite Life (Trucks per Day)
A	1,825,000	65
B	2,953,000	110
B'	3,536,000	130
C	4,383,000	160
C'	2,546,000	90
D	6,399,000	230
E	12,118,000	440
E'	22,318,000	815
M 164 (A 325) Bolts in Axial Tension	57,000	2
M 253 (A 490) Bolts in Axial Tension	57,000	2

Revise Table 6.6.1.2.5-2 as follows:

**Table 6.6.1.2.5-2 Cycles per Truck Passage,  $n$ .**

Longitudinal Members	Span Length	
	> 40.0 ft.	≤ 40.0 ft.
Simple Span Girders	1.0	2.0
Continuous Girders		
1) Near Interior Support	Fatigue I - 1.5 Fatigue II - 1.2	2.0
2) elsewhere	1.0	2.0
Cantilever Girders	5.0	
Trusses	1.0	
Transverse Members	Spacing	
	> 20.0 ft.	≤ 20.0 ft.
	1.0	2.0

(C6.6.1.2.5, cont.)

Revise the 7<sup>th</sup> paragraph as follows:

The values in the above table have been computed by Eq. 2b using the values for  $A$  and  $(\Delta F)_{TH}$  specified in Tables 1 and 3, respectively. The resulting values of  $N_{TH}$  and the 75-year  $(ADTT)_{SL}$  differ slightly when using the values for  $A$  and  $(\Delta F)_{TH}$  given in the Customary US Units and SI Units versions of the Specifications. The values in the above table represent the larger value from either version of the Specifications rounded up to the nearest 1,000 cycles and 5 trucks per day.

Add a new last paragraph:

Cycles per design fatigue Permit Truck (Fatigue II limit state) passage are evaluated by the rainflow method. The numbers of cycles induced by the fatigue Permit Truck passage are somewhat similar to the cycles induced by the HL-93 fatigue truck used for Fatigue I Limit State, except in the case of near interior support of bridges that spans greater than 40 feet.

## 6.10 I-SECTION FLEXURAL MEMBERS

### 6.10.1 General

#### C6.10.1

##### Revise Paragraph 1 as follows:

This article addresses general topics that apply to all types of steel I-sections in either straight bridges, horizontally curved bridges, or bridges containing both straight and curved segments. For the application of the provisions of Article 6.10, bridges containing both straight and curved segments are to be treated as horizontally curved bridges since the effects of curvature on the support reactions and girder deflections, as well as the effects of flange bending, usually extended beyond the curved segments. Note that kinked (chorded) girders exhibit the same actions as curved girders, except that the effect of the noncollinearity of the flanges is concentrated at the kinks. Continuous kinked (chorded) girders should be treated as horizontally curved girders with respect to these Specifications. Simply supported straight (chorded) girders in horizontally curved bridges should be treated as straight skewed girders. Straight bridges are intended to mean bridges containing only straight girders or segments. Horizontally curved bridges are intended to mean bridges containing only horizontally curved girders or segments.

The five bullet items in this article indicate the overarching organization of the subsequent provisions for the design of straight I-section flexural members. Each of the sub-articles throughout Article 6.10 are written such that they are largely self-contained, thus minimizing the need for reference to multiple articles to address any one of the essential design considerations. For the strength limit state, Article 6.10.6 directs the Engineer to the subsequent Articles 6.10.7 through 6.10.12, and optionally for sections in straight I-girder only to Appendices A and B, for the appropriate design requirements based on the type of I-section. The specific provisions of these Articles and Appendices are discussed in the corresponding Articles of the Commentary.

Add a new 3<sup>rd</sup> paragraph as follows:

For horizontally curved bridges, in addition to the potential sources of flange lateral bending discussed in the preceding paragraph, flange lateral bending effects due to curvature must always be considered at all limit states and also during construction.

### 6.10.5.3 Special Fatigue Requirement for Webs

Revise the 1<sup>st</sup> paragraph as follows:

For the purposes of this article, the factored fatigue load shall be ~~taken as twice that~~ calculated using the Fatigue I Limit State for infinite fatigue life load combination specified in Table 3.4.1-1, with the fatigue live load taken as specified in Article 3.6.1.4.

#### 6.10.6.2.2 Composite Sections in Positive Flexure

Revise the 3<sup>rd</sup> Paragraph as follows:

~~Compact sections shall satisfy the requirements of Article 6.10.7.1. Otherwise, the section shall be considered non-compact and shall satisfy the requirements of Article 6.10.7.2.~~

#### 6.10.7.1.2 Nominal Flexural Resistance

Revise Equation (6.10.7.1.2-2) as follows:

$$M_n = \left[ 1 - \left( 1 - \frac{M_y}{M_p} \right) \left( \frac{D_p / D_t - 0.1}{0.32} \right) \right] M_p$$

(6.10.7.1.2-2)

#### C6.10.7.1.2

Revise the 2<sup>nd</sup> Paragraph as follows:

Eq. 2 defines the inelastic moment resistance as a straight line between the ductility limits  $D_p/D_t = 0.1$  and 0.42. It gives approximately the same results as the comparable equation in previous Specifications, but is a simpler form that depends only on the plastic moment resistance  $M_p$ , the yield moment resistance  $M_y$ , and on the ratio  $D_p/D_t$ , also suggested in Yakel and Azizinamini (2004). ~~Both equations implement the above philosophy justified by Wittry (1993). Eq. 2 is somewhat more restrictive than the equation in previous Specifications for sections with small values of  $M_p/M_y$ , such as sections with hybrid webs, a relatively small deck area and a high strength tension flange. It is somewhat less restrictive for sections with large values of  $M_p/M_y$ . Wittry (1993) considered various experimental test results and performed a large number of parametric cross-section analyses. The smallest experimental or theoretical resistance of all the cross sections considered in this research and in other subsequent studies is  $0.96M_p$ . Eq. 2 is based on the target additional margin of safety of 1.28 specified by Wittry at the maximum allowed value of  $D_p$  combined with an assumed theoretical resistance of  $0.96M_p$  at this limit. At the maximum allowed value of  $D_p$  specified by Eq. 6.10.7.3 1, the resulting nominal design flexural resistance is  $0.78M_p$ .~~

**6.10.10.2 Fatigue Resistance**

Revise Eq. (6.10.10.2-1) as follows:

For finite fatigue life ( $N \leq 5.966(10)^6$ )

$$\underline{Z_r = \alpha d^2} \quad (6.10.10.2-1a)$$

For infinite fatigue life ( $N > 5.966(10)^6$ )

$$\underline{Z_r = 5.5d^2} \quad (6.10.10.2-1b)$$

**6.10.10.3 Special Requirements for Points of Permanent Load Contraflexure**

Revise definition of  $f_{sr}$  as follows:

$f_{sr}$  = stress range in the longitudinal reinforcement over the interior support under the Fatigue I load combination for infinite fatigue life specified in Table 3.4.1-1 with fatigue live load taken as specified in Article 3.6.1.4 (ksi)

**6.10.11.1.1 General**

Revise the 2<sup>nd</sup> and 4<sup>th</sup> Paragraphs as follows:

~~Stiffeners in straight girders not used as connection plates shall be welded to tight fit at the compression flange and fitted tightly to the tension flange. but need not be in bearing with the tension flange.~~ Single-sided stiffeners on horizontally curved girders should be attached to both flanges. ~~When pairs of transverse stiffeners are used on horizontally curved girders, they shall be fitted tightly to both flange~~

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange or longitudinal stiffener-to-web weld shall not be less than  $4t_w$  nor more than the lesser of  $6t_w$ . In no case shall the distance exceed and 4.0 in.

### 6.10.11.2 Bearing Stiffeners

#### 6.10.11.2.1 General

Revise the 4<sup>th</sup> paragraph as follows:

Each stiffener shall be either finish (mill or grind) ~~milled~~ to bear plus a fillet weld against the flange through which it receives its load or attached to that flange by a full penetration groove weld.

## 6.13 CONNECTIONS AND SPLICES

### 6.13.1 General

Revise the 1<sup>st</sup> paragraph as follows:

Except as specified otherwise, connections and splices for main members shall be designed at the strength limit state for not less than ~~the larger of:~~

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

~~75~~ **100** percent of the factored flexural, shear, or axial resistance of the member or element.

## 6.13.6 Splices

### 6.13.6.1 Bolted Splices

#### 6.13.6.1.4 Flexural Members

##### 6.13.6.1.4b Web Splices

Revise the 1<sup>st</sup> and 2<sup>nd</sup> paragraphs as follows:

Web splice plates and their connections shall be designed for shear, the moment due to the eccentricity of the shear at the point of splice and the portion of the flexural moment assumed to be resisted by the web at the point of splice. For all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3 or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, including horizontally curved bridges, the shear shall be taken as the sum of the flexural and St. Venant torsional shears in the web subjected to additive shears. For boxes with inclined webs, the web splice shall be designed for the component of the vertical shear in the plane of the web.

As a minimum, at the strength limit state, the design shear,  $V_{uw}$ , shall be taken as follows: the factored shear resistance of the smaller web at the point of splice,  $\phi_v V_n$ .

If  $V_u < 0.5\phi_v V_n$ , then;

$$V_{uw} = 1.5V_u \quad (6.13.6.1.4b-1)$$

otherwise:

$$V_{uw} = \frac{(V_u + \phi_v V_n)}{2} \quad (6.13.6.1.4b-2)$$

where:

$\phi_v$  = resistance factor for shear specified in Article 6.5.4.2

$V_u$  = ~~shear due to the factored loading at the point of splice (kip)~~

$V_n$  = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively (kip)

##### C6.13.6.1.4b

Delete the 1<sup>st</sup> paragraph.

~~Eqs. 1 and 2 provide a more consistent design shear to be used for designing web splice plates and their connections at the strength limit state than that given in past editions of the Standard Specifications and the First Edition of the LRFD Specifications. Eq. 1 arbitrarily limits the increase in the shear at the point of splice to 50 percent of the shear due to the factored loading,  $V_u$ , where  $V_u$  is less than 50 percent of the factored shear resistance,  $V_r = \phi_v V_n$ , at the point of splice. The increase in the shear is limited to 50 percent of  $V_u$  because the possibilities for  $V_u$  to change from its calculated value are less than for moment; large unintended shifts in the shear at the splice are unlikely. In addition, the maximum shear is usually not concurrent with the maximum moment at the splice. Thus, the use of a lower value of the design shear in regions where the applied shear is low is deemed reasonable. A lower value of the design shear is also more reasonable for rolled beams, which have significantly higher values of factored shear resistance. For cases where  $V_u$  is greater than 50 percent of  $V_r$ , the design shear is determined from Eq. 2 as the average of  $V_u$  and  $V_r$ . For checking slip of the bolted connections, the design shear is simply taken as the shear at the point of splice under Load Combination Service II defined in Table 3.4.1-1.~~

Revise Eqs C6.13.6.1.4b-1 and C6.13.6.1.4b-2 as follows:

For compact sections:

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

$$H_{uw} = \frac{t_w D}{2} (R_h F_{cf} + R_{cf} f_{ncf}) \quad (C6.13.6.1.4b-2)$$

$$M_{uw} = \phi_f \frac{t_w F_{yw}}{4} (D^2 - 4y_o^2) \quad (C6.13.6.1.4b-1a)$$

$$H_{uw} = \phi_f (2 t_w y_o F_{yw}) \quad (C6.13.6.1.4b-2a)$$

For noncompact sections:

$$M_{uw} = \phi_f \frac{t_w D^2}{12} (F_{nc} + F_{yw}) \quad (\text{C6.13.6.1.4b-1b})$$

$$H_{uw} = \phi_f \frac{t_w D}{2} (F_{yw} - F_{nc}) \quad (\text{C6.13.6.1.4b-2b})$$

where:

$t_w$  = web thickness (in.)

$D$  = web depth (in.)

$R_h$  = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in

which  $F_{ef}$  does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0

$F_{ef}$  = design stress for the controlling flange at the point of splice specified in Article 6.13.6.1.4e; positive for tension, negative for compression (ksi)

$R_{ef}$  = the absolute value of the ratio of  $F_{ef}$  to the maximum flexural stress,  $f_{ef}$ , due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in Article 6.13.6.1.4e

$f_{nef}$  = flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with  $f_{ef}$ ; positive for tension, negative for compression (ksi)

$F_{nc}$  = nominal flexural resistance of the compression flange at the point of splice as specified in Article 6.10.8.2 (ksi)

$F_{yw}$  = specified minimum yield strength of the web at the point of splice (ksi)

$v_o$  = distance from the mid-depth of the web to the plastic neutral axis (in.)

$\phi_f$  = resistance factor for flexure specified in Article 6.5.4.2

Revise the 4<sup>th</sup> paragraph as follows:

In Eqs. C1 and C2, it is suggested that  $M_{uw}$  and  $H_{iw}$  be computed by conservatively using the flexural resistance stresses at the midthickness of the compression flanges and specified minimum yield strength of the web. ~~By utilizing the stresses at the midthickness of the flanges, the same stress values can be used for the design of both the web and flange splices, which simplifies the calculations. As an alternate, however, the stresses at the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross sections supporting those loadings. In Eqs. C1 and C2, the concurrent flexural stress at the midthickness of the noncontrolling flange is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. The controlling and noncontrolling flanges are defined in Article C6.13.6.1.4e.~~

Revise the 5<sup>th</sup> and 6<sup>th</sup> paragraphs as follows:

The stresses in Eqs. C1 and C2 are to be taken as signed quantities. For convenience, absolute value signs are applied to the resulting difference of the stresses in Eq. C1. In actuality, the sign of  $M_{uw}$  corresponds to the sign of the flexural moment for the loading condition under consideration.  $H_{uw}$  in Eq. C2 is taken as a signed quantity; positive for tension, negative for compression. For sections where the neutral axis is located at the middepth of the web,  $H_{uw}$  will equal zero. For all other sections,  $M_{uw}$  and  $H_{uw}$  applied together will yield a combined stress distribution equivalent to the unsymmetrical stress distribution in the web.

Eqs. C1c and C2c can also be used to compute values of  $M_{uw}$  and  $H_{uw}$  to be used when checking for slip of the web bolts. However, the following substitutions must first be made in both equations:

- replace  $F_{ef}$  with the maximum flexural stress,  $f_s$ , due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of splice,
- replace  $f_{nef}$  with the flexural stress,  $f_{os}$ , due to Load Combination Service II at the midthickness of the other flange at the point of splice concurrent with  $f_s$  in the flange under consideration, and
- set the factors  $R_h$  and  $R_{ef}$  equal to 1.0. It is not necessary to determine a controlling and noncontrolling flange when checking for slip.

The same sign convention applies to the stresses.

$$M_{uw} = \frac{t_w D^2}{12} |f_s - f_{os}| \quad \text{(C6.13.6.1.4b-1c)}$$

$$H_{uw} = \frac{t_w D}{2} (f_s + f_{os}) \quad \text{(C6.13.6.1.4b-2c)}$$

where:

$f_s$  = maximum flexural stress due to Load Combination Service II at the extreme fiber of the flange under consideration for the smaller section at the point of splice (positive for tension and negative for compression) (ksi)

$f_{os}$  = flexural stress due to Load Combination Service II at the extreme fiber of the other flange at the point of splice with  $f_s$  in the flange under consideration (positive for tension and negative for compression) (ksi)

In Eqs. C1c and C2c, it is suggested that  $M_{ow}$  and  $H_{ow}$  be computed by conservatively using the stresses at the extreme fiber of the flanges. As an alternate, however, the stresses at the midthickness of the flanges or the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross-sections supporting those loadings.

~~$R_{ef}$  = the absolute value of the ratio of  $F_{ef}$  to the maximum flexural stress,  $f_{ef}$ , due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in Article 6.13.6.1.4e~~

$f_{ncf}$  = flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with  $f_{cf}$ ; positive for tension, negative for compression (ksi)

## 6.13.6.1.4c Flange Splices

Revise as follows:

At the strength limit state, splice plates and their connections on the controlling flanges shall be proportioned to provide a minimum resistance taken as the design stress,  $F_{cf}$ , times the smaller effective flange area,  $A_e$ , on either side of the splice, where  $F_{cf}$  is defined as:

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

$$F_{cf} = \alpha \phi_f F_{yf} \quad (6.13.6.1.4c-1)$$

in which:

$A_e$  = effective area of the flange (in.<sup>2</sup>). For compression flanges,  $A_e$  shall be taken as the gross area of the flange. For tension flanges,  $A_e$  shall be taken as:

$$A_e = \left( \frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n \leq A_g \quad (6.13.6.1.4c-2)$$

where:

~~$f_{cf}$  = design maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (ksi)~~

~~$R_h$  = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which  $F_{cf}$  does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

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

$\phi_f$  = resistance factor for flexure specified in Article 6.5.4.2

$F_n$  = nominal flexural resistance of the flange (ksi)

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

$\phi_u$  = resistance factor for fracture of tension members as specified in Article 6.5.4.2

$\phi_y$  = resistance factor for yielding of tension members as specified in Article 6.5.4.2

$A_n$  = net area of the tension flange determined as specified in Article 6.8.3 (in.<sup>2</sup>)

## C6.13.6.1.4c

Delete the 3<sup>rd</sup> paragraph:

~~The controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its extreme fiber midthickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the noncontrolling flange. In areas of stress reversal, the splice must be checked independently for both positive and negative flexure. For composite sections in positive flexure, the controlling flange is typically the bottom flange. For sections in negative flexure, either flange may qualify as the controlling flange.~~

Revise the 5<sup>th</sup> paragraph as follows:

~~Eq. 3 defines a design stress for the noncontrolling flange at the strength limit state. In Eq. 3, the flexural stress at the midthickness of the noncontrolling flange, is concurrent with the stress in the controlling flange, is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. However, as a minimum, the factored up stress must be equal to or greater than  $0.75 \alpha \phi_f F_{yf}$ .~~

Delete the 7<sup>th</sup> paragraph:

~~Since flanges of hybrid girders are allowed to reach  $F_{yf}$ , the applied flexural stress at the midthickness of the flange in Eqs. 1, 3 and 5 is divided by the hybrid factor,  $R_h$ , instead of reducing  $F_{yf}$  by  $R_h$ . In actuality, yielding in the web results in an increase in the applied flange stress. When the flange design stress is less than or equal to the specified minimum yield strength of the web,  $R_h$  is taken equal to 1.0 since there is theoretically no yielding in the web. The load shedding factor,  $R_b$ , is not included in these equations since the presence of the web splice precludes the possibility of local web buckling.~~

Revise the 10<sup>th</sup> paragraph as follows:

- $A_g$  = gross area of the tension flange (in.<sup>2</sup>)  
 $F_u$  = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)  
 $F_{yt}$  = specified minimum yield strength of the tension flange (ksi)

~~Splice plates and their connections on the noncontrolling flange at the strength limit state shall be proportioned to provide a minimum resistance taken as the design stress,  $F_{ncf}$ , times the smaller effective flange area,  $A_{e_s}$ , on either side of the splice, where  $F_{ncf}$  is defined as:~~

Revise Eq. (6.13.6.1.4c-3) as follows:

$$F_{ncf} = R_{cf} \left| \frac{f_{ncf}}{R_h} \right| \geq 0.75 \alpha \phi_f F_{yf} \quad (6.13.6.1.4c-3)$$

where:

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

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

$R_h$  = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which  $f_{cf}$  does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0

At the strength limit state, the design force in splice plates subjected to tension shall not exceed the factored resistance in tension specified in Article 6.13.5.2. The design force in splice plates subjected to compression shall not exceed the factored resistance,  $R_c$ , in compression taken as:

(no change to Eq. 4)

For the box sections cited in this article, longitudinal warping stresses due to cross-section distortion can be significant under construction and service conditions and must therefore be considered when checking the connections of bolted flange splices for slip and for fatigue. The warping stresses in these cases can typically be ignored in checking the top-flange splices once the flange is continuously braced. The warping stresses can also be ignored when checking splices in both the top and bottom flanges at the strength limit state. For these sections, St. Venant torsional shear must also be considered in the design of box-flange bolted splices at all limit states. St. Venant torsional shears are typically neglected in top flanges of tub sections once the flanges are continuously braced. The bolts for box-flange splices may be designed for the effects of the torsional shear using the traditional elastic vector method that is typically applied in the design of web splices. Depending on the limit state under investigation, the shear on the flange bolt group is assumed caused by either the flange force due to the factored loads, or by the appropriate flange design force, as applicable. The moment on the bolt group is taken as the moment resulting from the eccentricity of the St. Venant torsional shear due to the factored loads, assumed applied at the centerline of the splice. At the strength limit state, a design torsional shear due to factored loads should be used, ~~which can be taken as the torsional shear due to the factored loads multiplied by the factor,  $R_{cf}$ , from Eq. 3.~~ The box-flange splice plates in these cases should also be designed at the strength limit state for the combined effects of the calculated design shear and design moment acting on the bolt group.

Revise the 11<sup>th</sup> paragraph as follows:

In cases for straight girders where flange lateral bending is deemed significant, and for horizontally curved girders, the effects of the lateral bending must be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections. The traditional elastic vector method may also be used in these cases to account for the effects of flange lateral bending on the design of the splice bolts. The shear on the flange bolt group is assumed caused by the flange force, calculated as described in the preceding

Revise Eq. (6.13.6.1.4c-5) as follows:

$$F_s = \frac{f_s}{R_h} \quad (6.13.6.1.4c-5)$$

where:

$f_s$  = maximum flexural stress due to Load Combination Service II at the extreme fiber ~~midthickness~~ of the flange under consideration for the smaller section at the point of splice (ksi)

$R_h$  = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which  $f_s$  in the flange with the larger stress does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0.

paragraph. The flange force is calculated without consideration of the flange lateral bending. The moment on the bolt group is taken as the flange lateral bending moment due to the factored loads. At the strength limit state, a design lateral bending moment ~~due to the factored loads~~ should be used, ~~which can be taken as the lateral bending moment due to the factored loads multiplied by the factor,  $R_{ef}$ , from Eq. 3.~~ Splice plates subject to flange lateral bending should also be designed at the strength limit state for the combined effects of the calculated design shear and design moment acting on the bolt group. Lateral flange bending can be ignored in the design of top flange splices once the flange is continuously braced.