AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

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ERRATA for *AASHTO LRFD Bridge Design Specifications, 8th Edition, Sections 3, 5, and 6 Academic Offer* (NCBC-8)

May 2018

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *AASHTO LRFD Bridge Design Specifications, 8th Edition, Sections 3, 5, and 6 Academic Offer*.

Please scroll down to see the full erratum.

In the event that you need to download this file again, please download from AASHTO's online bookstore at:

http://downloads.transportation.org/NCBC-8-Errata.pdf

Then, please replace the existing pages with the corrected pages to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience to our readers.

Sincerely,

Erin Grady
Publications Director

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(Editor-
ial)</td>
<td><table>
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A_c in this equation should be A_{ct}</td></tr>
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<td>6.13.6.1.3c (excerpted)

Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force located at the mid-depth of the web that provides the necessary moment resistance in conjunction with the flange splices.
The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flange splices divided by the appropriate moment arm to the mid-depth of the web. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.</td>
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6-258	*6.13.6.1.3c—Web Splices* As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force located at the mid-depth of the web that provides the necessary moment resistance in conjunction with the flange splices. The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flange splices divided by the appropriate moment arm to the mid-depth of the web. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.	*6.13.6.1.3c—Web Splices* As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force ~~located at the mid-depth of~~ in the web that provides thet necessary moment resistance in conjunction with the flange splices. The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flange splices divided by the appropriate moment arm ~~to the mid-depth of the web~~. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as one-quarter of the web depth.~~the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.~~
6-258	*C6.13.6.1.3c* … Figure C6.13.6.1.3c-1 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to positive flexure taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices divided by the moment arm, A_w:	*C6.13.6.1.3c* … Figure C6.13.6.1.3c-1 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to positive flexure. The web moment is taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices. H_w is then taken as the web moment divided by the moment arm, A_w, taken from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch.

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<tr><td>6-259</td><td>

Figure C6.13.6.1.3c-1

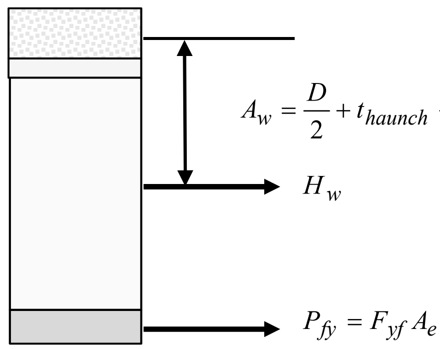

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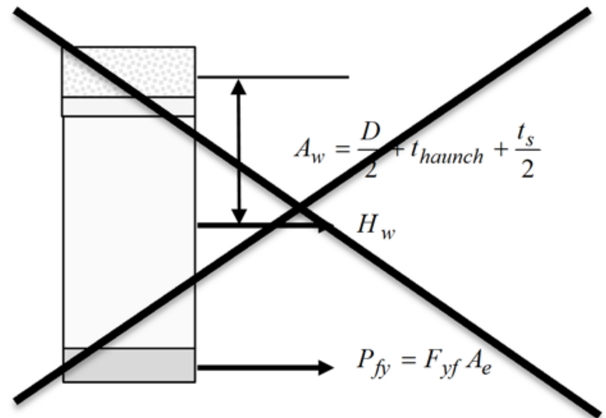
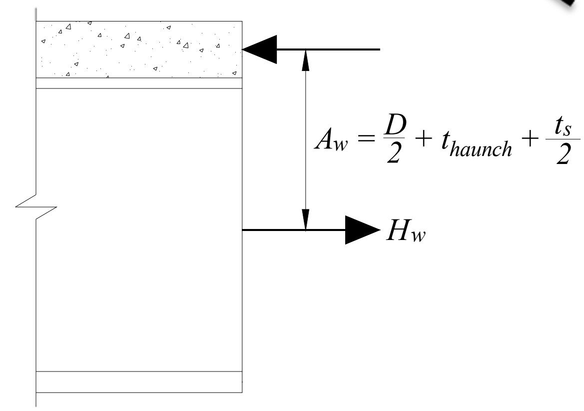
Figure C6.13.6.1.3c-1





$$Web\ Moment = H_w A_w$$

$$H_w = \frac{Web\ Moment}{A_w}$$

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<tr><td>6-259</td><td>

C6.13.6.1.3c

…

Figure C6.13.6.1.3c-2 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to negative flexure and noncomposite sections, taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices divided by the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:

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C6.13.6.1.3c

…

Figure C6.13.6.1.3c-2 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to negative flexure and noncomposite sections~~,~~. The web moment is again taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices. In this case, however, H_w is taken as the web moment divided by D/4, as shown in Figure C6.13.6.1.3c-2.~~the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:~~

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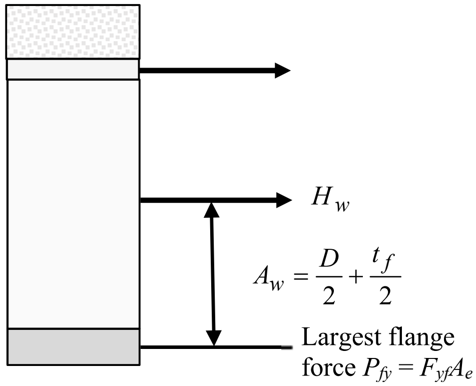
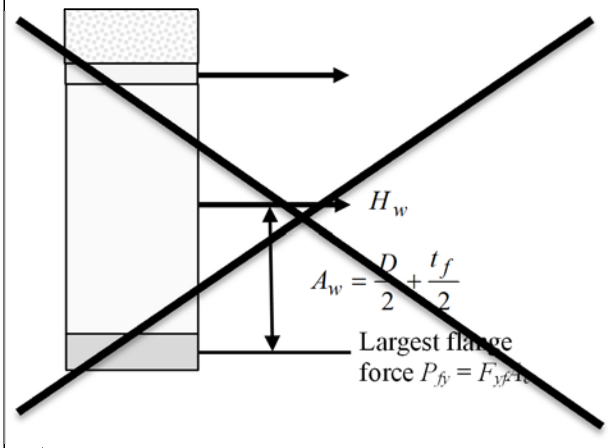
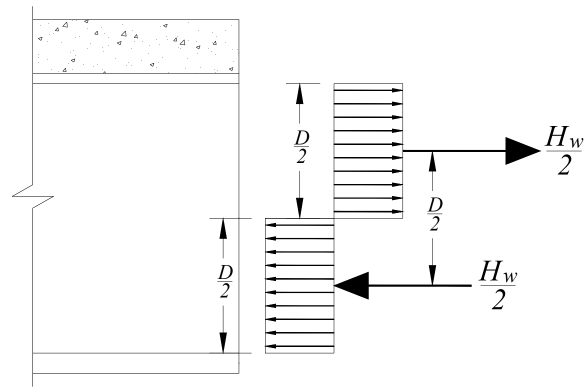
Page	Existing Text	Corrected Text
6-259	**Figure C6.13.6.1.3c-2**  	**Figure C6.13.6.1.3c-2**   $Web\ Moment = \frac{Hw}{2}\left(\frac{D}{2}\right)$ $H_w = \frac{WebMoment}{D/4}$

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6-259	*C6.13.6.1.3c* .. Because the resultant web force in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment is not applied.	*C6.13.6.1.3c* .. <u>The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-1 is provided by a horizontal tensile force, H_w, assumed acting at the mid-depth of the web that is equilibrated by an equal and opposite horizontal compressive force in the concrete deck. The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-2 is provided by two equal and opposite horizontal tensile and compressive forces, $H_w/2$, assumed acting at a distance $D/4$ above and below the mid-height of the web. In each case, there is no net horizontal force acting on the section.</u> Because the resultant web force in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment is not applied.

exposed to noncorrosive soil, where the minimum cover shall be 3.0 in.

Cover to epoxy-coated steel may be as shown for interior exposure in Table 5.10.1-1.

Table 5.10.1-1—Cover for Unprotected Main Reinforcing Steel (in.)

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments ○ General ○ Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0
Precast concrete box culverts • Top slabs used as a driving surface • Top slabs with less than 2.0 ft of fill not used as a driving surface • All other members	 2.5 2.0 1.0

"Corrosive" water or soil contains greater than or equal to 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than or equal to 2,000 ppm, a pH of less than or equal to 5.5, or both shall be considered noncorrosive in determining minimum cover.

5.10.2—Hooks and Bends

5.10.2.1—Standard Hooks

For the purpose of these Specifications, the term "standard hook" shall mean one of the following:

- For longitudinal reinforcement:
  - (a) 180-degree bend, plus a $4.0d_b$ extension, but not less than 2.5 in. at the free end of the bar, or
  - (b) 90-degree bend, plus a $12.0d_b$ extension at the free end of the bar.

C5.10.2.1

These requirements are similar to the requirements of ACI 318-14 and CRSI's *Manual of Standard Practice*.

Tests by Shahrooz et al. (2011) showed that standard hooks are adequate for reinforcement with specified minimum yield strengths between 75.0 and 100 ksi if transverse, confining reinforcement as specified in Article 5.10.8.2.4 is provided.

- For transverse reinforcement:

  (a) No. 5 bar and smaller—90-degree bend, plus a $6.0d_b$ extension at the free end of the bar;
  (b) No. 6, No. 7 and No. 8 bars—90-degree bend, plus a $12.0d_b$ extension at the free end of the bar; and
  (c) No. 8 bar and smaller—135-degree bend, plus a $6.0\ d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

Standard hooks may be used with reinforcing steel having a specified minimum yield strength between 75.0 and 100 ksi for elements and connections specified in Article 5.4.3.3 only if ties specified in Article 5.10.8.2.4 are provided.

5.10.2.2—Seismic Hooks

Seismic hooks meeting the requirements of Article 5.11.4.1.4 shall be used for transverse reinforcement in regions of expected plastic hinges and elsewhere as indicated in the contract documents.

5.10.2.3—Minimum Bend Diameters

The diameter of a bar bend, measured on the inside of the bar, shall not be less than that specified in Table 5.10.2.3-1.

Table 5.10.2.3-1—Minimum Diameters of Bend

Bar Size and Use	Minimum Diameter
No. 3 through No. 5—General	$6.0d_b$
No. 3 through No. 5—Stirrups and Ties	$4.0d_b$
No. 6 through No. 8—General	$6.0d_b$
No. 9, No. 10, and No. 11	$8.0d_b$
No. 14 and No. 18	$10.0d_b$

The inside diameter of bend for stirrups and ties in plain or deformed welded wire reinforcement shall not be less than $4.0d_b$ for deformed wire larger than D6 and $2.0d_b$ for all other wire sizes. Bends with inside diameters of less than $8.0d_b$ shall not be located less than $4.0d_b$ from the nearest welded intersection.

APPENDIX B5—GENERAL PROCEDURE FOR SHEAR DESIGN WITH TABLES

B5.1—BACKGROUND

The general procedure herein is an acceptable alternative to the procedure specified in Article 5.7.3.4.2. The procedure in this Appendix utilizes tabularized values of β and θ instead of Eqs. 5.7.3.4.2-1, 5.7.3.4.2-2, and 5.7.3.4.2-3. Appendix B5 is a complete presentation of the general procedures in LRFD Design (AASHTO 2007) without any interim changes.

B5.2—SECTIONAL DESIGN MODEL—GENERAL PROCEDURE

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-1. In using this table, ε_x shall be taken as the calculated longitudinal strain at the middepth of the member when the section is subjected to M_u, N_u, and V_u as shown in Figure B5.2-1.

For sections containing less transverse reinforcement than specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-2. In using this table, ε_x shall be taken as the largest calculated longitudinal strain which occurs within the web of the member when the section is subjected to N_u, M_u, and V_u as shown in Figure B5.2-2.

Where consideration of torsion is required by the provisions of Article 5.7.2, V_u in Eqs. B5.2-3 through B5.2-5 shall be replaced by V_{eff}.

For solid sections:

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \qquad \text{(B5.2-1)}$$

For hollow sections:

$$V_{eff} = V_u + \frac{T_u d_s}{2A_o} \qquad \text{(B5.2-2)}$$

Unless more accurate calculations are made, ε_x shall be determined as:

- If the section contains at least the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})} \qquad \text{(B5.2-3)}$$

CB5.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.7.3.1. Such an analysis (see Figure CB5.2-1) would show that the shear stresses are not uniform over the depth of the web and that the direction of the principal compressive stresses changes over the depth of the beam. The more direct procedure given herein assumes that the concrete shear stresses are uniformly distributed over an area b_v wide and d_v deep, that the direction of principal compressive stresses (defined by angle θ) remains constant over d_v, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. See Figure CB5.2-2.

For solid cross-section shapes, such as a rectangle or an "I," there is the possibility of considerable redistribution of shear stresses. To make some allowance for this favorable redistribution it is safe to use a root-mean-square approach in calculating the nominal shear stress for these cross-sections, as indicated in Eq. B5.2-1. The 0.9 p_h comes from 90 percent of the perimeter of the spalled concrete section. This is similar to multiplying 0.9 times the lever arm in flexural calculations.

For a hollow girder, the shear flow due to torsion is added to the shear flow due to flexure in one exterior web, and subtracted from the opposite exterior web. In the controlling web, the second term in Eq. B5.2-2 comes from integrating the distance from the centroid of the section, to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying by the web height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal.

Members containing at least the minimum amount of transverse reinforcement have a considerable capacity to redistribute shear stresses from the most highly strained portion of the cross-section to the less highly strained portions. Because

The initial value of ε_x should not be taken greater than 0.001.

- If the section contains less than the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{E_sA_s + E_pA_{ps}} \tag{B5.2-4}$$

The initial value of ε_x should not be taken greater than 0.002.

- If the value of ε_x from Eqs. B5.2-3 or B5.2-4 is negative, the strain shall be taken as:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_c\underline{A_{ct}} + E_sA_s + E_pA_{ps}\right)} \tag{B5.2-5}$$

where:

A_{ct} = area of concrete on the flexural tension side of the member as shown in Figure B5.2-1 (in.^{2})

A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure B5.2-1 (in.^{2})

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

A_s = area of nonprestressed steel on the flexural tension side of the member at the section under consideration, as shown in Figure B5.2-1. In calculating A_s for use in this equation, bars which are terminated at a distance less than their development length from the section under consideration shall be ignored (in.^{2})

d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement (in.)

f_{po} = a parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete. For the usual levels of prestressing, a value of $0.7f_{pu}$ will be appropriate for both pretensioned and post-tensioned members (ksi)

of this capacity to redistribute, it is appropriate to use the middepth of the member as the location at which the biaxial stress conditions are determined. Members that contain no transverse reinforcement, or contain less than the minimum amount of transverse reinforcement, have less capacity for shear stress redistribution. Hence, for such members, it is appropriate to perform the biaxial stress calculations at the location in the web subject to the highest longitudinal tensile strain; see Figure B5.2-2.

The longitudinal strain at the middepth of the member, ε_x, can be determined by the procedure illustrated in Figure CB5.2-3. The actual section is represented by an idealized section consisting of a flexural tension flange, a flexural compression flange, and a web. The area of the compression flange is taken as the area on the flexure compression side of the member, i.e., the total area minus the area of the tension flange as defined by A_{ct}. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u - V_p$, will primarily be carried by diagonal compressive stresses in the web concrete. These diagonal compressive stresses will result in a longitudinal compressive force in the web concrete of $(V_u - V_p)\cot\theta$. Equilibrium requires that this longitudinal compressive force in the web needs to be balanced by tensile forces in the two flanges, with half the force, that is $0.5(V_u - V_p)\cot\theta$, being taken by each flange. To avoid a trial and error iteration process, it is a convenient simplification to take this flange force due to shear as $V_u - V_p$. This amounts to taking $0.5\cot\theta = 1.0$ in the numerator of Eqs. B5.2-3, B5.2-4, and B5.2-5. This simplification is not expected to cause a significant loss of accuracy. After the required axial forces in the two flanges are calculated, the resulting axial strains, ε_t and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure CB5.2-4.

For members containing at least the minimum amount of transverse reinforcement, ε_x can be taken as:

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2} \tag{CB5.2-1}$$

where ε_t and ε_c are positive for tensile strains and negative for compressive strains. If, for a member subject to flexure, the strain ε_c is assumed to be negligibly small, then ε_x becomes one half of ε_t. This is the basis for the expression for ε_x given in Eq. B5.2-3. For members containing less than the minimum amount of transverse reinforcement, Eq. B5.2-4 makes the conservative simplification that ε_x is equal to ε_t.

In some situations, it will be more appropriate to determine ε_x using the more accurate procedure of Eq. CB5.2-1 rather than the simpler Eqs. B5.2-3 through B5.2-5. For example, the shear capacity of sections near the ends of precast, pretensioned simple beams made continuous for live load will be estimated in a very conservative manner by moment arm taken as the vertical distance between the mid-thickness of the top and bottom flanges.

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_r, in compression taken as:

$$R_r = \phi_c F_y A_s \qquad (6.13.6.1.3b\text{-}3)$$

where:

ϕ_c = resistance factor for compression as specified in Article 6.5.4.2
F_y = specified minimum yield strength of the splice plate (ksi)
A_s = gross area of the splice plate (in.^{2})

Bolted connections for flange splices shall be checked for slip under a flange slip force determined as the factored moment at the point of splice divided by the appropriate moment arm defined as specified herein. The factored moment for checking slip shall be taken as the moment at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, or the moment at the point of splice due to the deck casting sequence, whichever governs.

The computed flange slip force shall be divided by the nominal slip resistance of the bolts, determined as specified in Article 6.13.2.8, to determine the total number of flange splice bolts required on one side of the splice to resist slip. For all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, longitudinal warping stresses due to cross-section distortion shall be considered when checking bolted flange splices for slip and for fatigue. Longitudinal warping stresses may be ignored at the strength limit state. The vector sum of the St. Venant torsional shear and the flange slip force or design yield resistance shall be considered in the design of box-flange bolted splices for these sections at the corresponding applicable limit state.

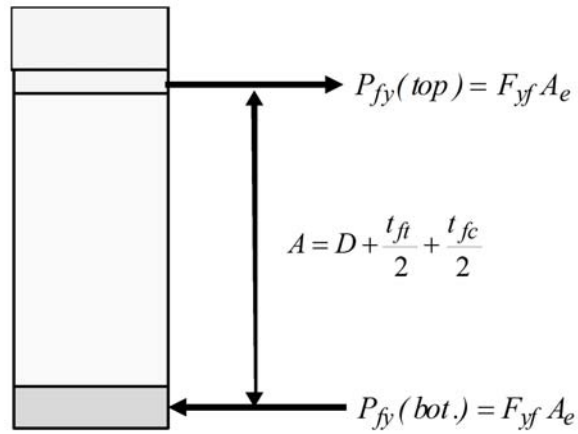


Figure C6.13.6.1.3b-2—Calculation of the Moment Resistance Provided by the Flange Splices for Composite Sections Subject to Negative Flexure and Noncomposite Sections

Flange splice plates subjected to tension are to be checked for yielding on the gross section, fracture on the net section, and block shear rupture at the strength limit state according to the provisions of Article 6.13.5.2. Block shear rupture will usually not govern the design of splice plates of typical proportion. 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 according to Eq. 6.13.6.1.3b-3. Eq. 6.13.6.1.3b-3 assumes an unbraced length of zero for the splice plates.

For a flange splice with inner and outer splice plates, P_{fy} at the strength limit state may be assumed divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than ten percent. For this case, the connections are proportioned assuming double shear. Should the areas of the inner and outer plates differ by more than ten percent, the design force in each splice plate and its connection at the strength limit state should instead be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the connections are proportioned for the maximum calculated splice-plate force acting on a single shear plane. When checking for slip of the connection for a flange splice with inner and outer splice plates, the slip resistance should always be determined by dividing the flange slip force equally to the two slip planes regardless of the ratio of the splice plate areas. Slip of the connection cannot occur unless slip occurs on both planes.

For the box sections cited in this Article, including sections in horizontally-curved bridges, longitudinal warping stresses due to cross-section distortion can be more 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.

For straight girders where flange lateral bending is deemed significant, and for horizontally-curved girders, the effects of the lateral bending need not be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections at all limit states. At the strength limit state, flange splices are to be designed to develop the full yield resistance of the flange, which cannot be exceeded in such cases under combined major-axis and lateral bending at the strength limit state. Flange lateral bending will increase the flange slip force on one side of the splice and decrease the slip force on the other side of the splice; slip cannot occur unless it occurs on both sides of the splice.

6.13.6.1.3c—Web Splices

As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable.

Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force ~~located at the mid-depth of~~ in the web that provides the necessary moment resistance in conjunction with the flange splices.

The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flange splices divided by the appropriate moment arm~~ to the mid-depth of the web~~. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as one-quarter of the web depth.~~the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance,~~ ~~P_{fy}.~~

C6.13.6.1.3c

The factored shear resistance of the bolts should be based on threads included in the shear planes, unless the web splice-plate thickness exceeds 0.5 in. As a minimum, two vertical rows of bolts spaced at the maximum spacing for sealing bolts specified in Article 6.13.2.6.2 should be provided, with a closer spacing and/or additional rows provided only as needed.

Since the web splice is being designed to develop the full factored shear resistance of the web as a minimum at the strength limit state, the effect of the small moment introduced by the eccentricity of the web connection may be ignored at all limit states. Also, for all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state.

Figure C6.13.6.1.3c-1 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to positive flexure. The web moment is taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices. H_w is then taken as the web moment divided by the moment arm, A_w, taken from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch.

The computed design web force shall be divided by the factored shear resistance of the bolts, determined as specified in Article 6.13.2.2, to determine the total number of web splice bolts required on one side of the splice at the strength limit state. The bearing resistance of the web at bolt holes shall also be checked at the strength limit state as specified in Article 6.13.2.9.

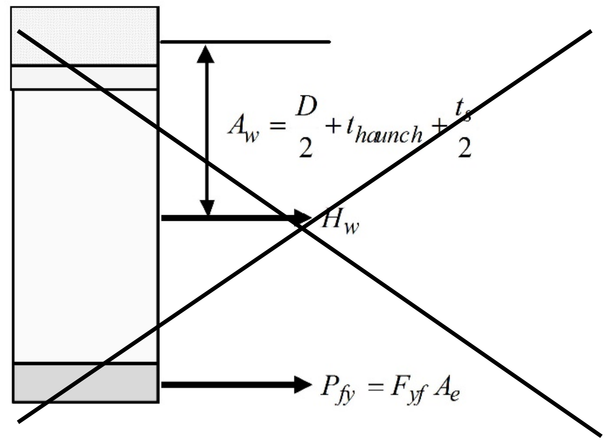
The design web force at the strength limit state shall not exceed the lesser of the factored shear resistances of the web splice plates determined as specified in Articles 6.13.4 and 6.13.5.3.

As a minimum, bolted connections for web splices shall be checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice. Should the nominal slip resistance provided by the flange bolts not be sufficient to resist the flange slip force due to the factored moment at the point of splice, determined as specified in Article 6.13.6.1.3b, the web splice bolts shall instead be checked for slip under a web slip force taken equal to the vector sum of the factored shear and the portion of the flange slip force that exceeds the nominal slip resistance of the flange bolts. The factored shear for checking slip shall be taken as the shear in the web at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, or the shear in the web at the point of splice due to the deck casting sequence, whichever governs.

For all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, the shear for checking slip shall be taken as the sum of the factored flexural and St. Venant torsional shears in the web subjected to additive shears. For boxes with inclined webs, the factored shear shall be taken as the component of the factored vertical shear in the plane of the web.

The computed web slip force shall be divided by the nominal slip resistance of the bolts, determined as specified in Article 6.13.2.8, to determine the total number of web splice bolts required on one side of the splice to resist slip.

Webs shall be spliced symmetrically by plates on each side. The splice plates shall extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances. For bolted web splices with thickness differences of 0.0625 in. or less, filler plates should not be provided.



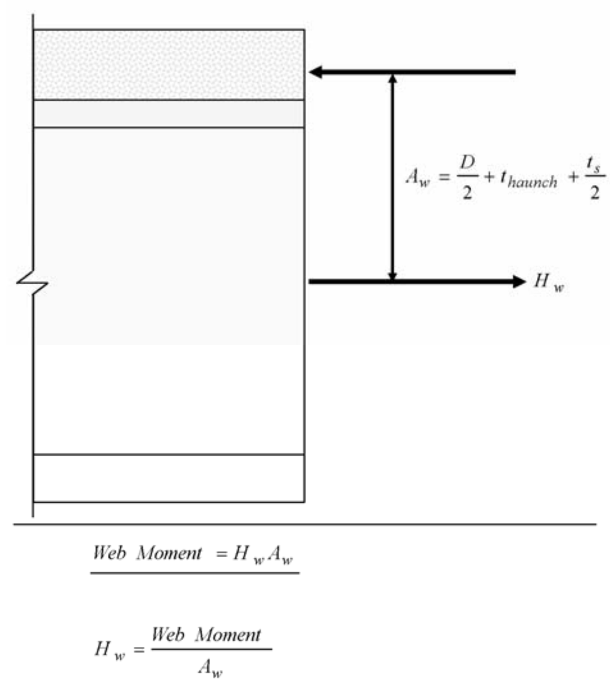


Figure C6.13.6.1.3c-1—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Positive Flexure

Figure C6.13.6.1.3c-2 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to negative flexure and noncomposite sections~~,~~. The web moment is again taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices. In this case, however, H_w is taken as the web moment divided by D/4, as shown in Figure C6.13.6.1.3c-2.~~the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:~~

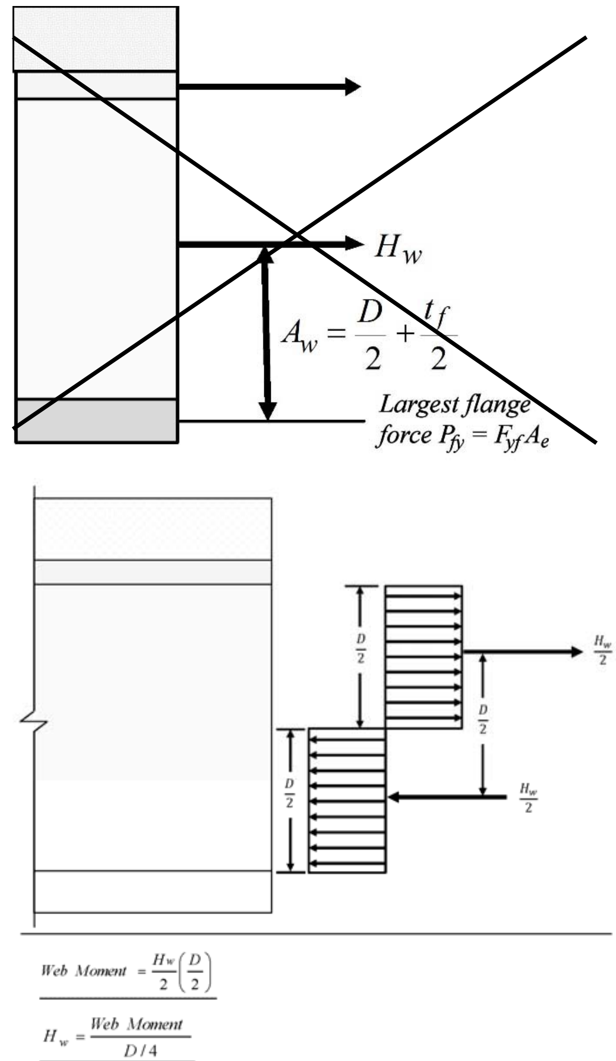


Figure C6.13.6.1.3c-2—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Negative Flexure and Noncomposite Sections

The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-1 is provided by a horizontal tensile force, H_w, assumed acting at the mid-depth of the web that is equilibrated by an equal and opposite horizontal compressive force in the concrete deck. The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-2 is provided by two equal and opposite horizontal tensile and compressive forces, $H_w/2$, assumed acting at a distance $D/4$ above and below the mid-height of the web. In each case, there is no net horizontal force acting on the section.

Because the resultant web force in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment is not applied.

Since slip is a serviceability requirement, the effect of the additional St. Venant torsional shear in the web is to be considered for the box sections described above when checking for slip.

When checking the bearing resistance of the web at bolt holes for an inclined resultant design web force, the resistance of an outermost hole, calculated using the clear edge distance, can conservatively be checked against the resultant force assumed to be acting on the extreme bolt in the connection as shown on the left of Figure C6.13.6.1.3c-3. 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 resistance be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Other options would be to calculate the bearing resistance based on the inclined distance or to resolve the resultant force in the direction parallel to the edge distance. In cases where the bearing resistance of the web splice plates controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing resistance of the outermost hole as shown on the right of Figure C6.13.6.1.3c-3.

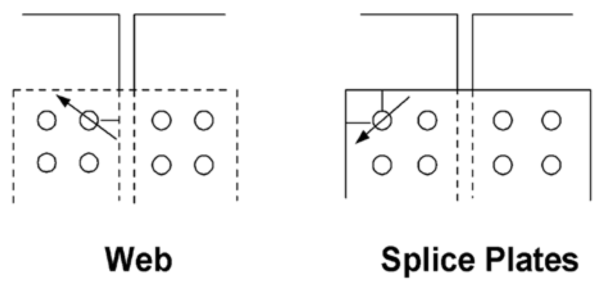


Figure C6.13.6.1.3c-3—Critical Locations for Checking Bearing Resistance of Outermost Web Splice Bolt Holes for an Inclined Resultant Design Web Force

Required bolt assembly clearances are given in AISC (2011).

6.13.6.1.4—Fillers

When bolts carrying loads pass through fillers 0.25 in. or more in thickness in axially loaded connections, including girder flange splices, either:

- The fillers shall be extended beyond the gusset or splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler or
- As an alternative, the fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the strength limit state, specified in Article 6.13.2.2, is reduced by the following factor:

C6.13.6.1.4

Fillers are to be secured by means of additional fasteners so that the fillers are, in effect, an integral part of a shear-connected component at the strength limit state. The integral connection results in well-defined shear planes and no reduction in the factored shear resistance of the bolts.

In lieu of extending and developing the fillers, the reduction factor given by Eq. 6.13.6.1.4-1 may instead be applied to the factored resistance of the bolts in shear. This factor compensates for the reduction in the nominal shear resistance of a bolt caused by bending in the bolt and will typically result in the need to provide additional bolts in the connection. The reduction factor is only to be applied on the side of the connection with the fillers. The factor in Eq. 6.13.6.1.4-1 was developed mathematically (Sheikh-Ibrahim, 2002), and verified by comparison to the results from an experimental program on axially

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \tag{6.13.6.1.4-1}$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on both sides of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate (in.^{2}); for truss gusset plate chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas

For slip-critical connections, the nominal slip resistance of a bolt, specified in Article 6.13.2.8, shall not be adjusted for the effect of the fillers.

Fillers 0.25 in. or more in thickness shall consist of not more than two plates, unless approved by the Engineer.

The specified minimum yield strength of fillers 0.25 in. or greater in thickness should not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36.0 ksi.

loaded bolted splice connections with undeveloped fillers (Yura, et al., 1982). The factor is more general than a similar factor given in AISC (2016) in that it takes into account the areas of the main connected plate, splice plates and fillers and can be applied to fillers of any thickness. Unlike the empirical AISC factor, the factor given by Eq. 6.13.6.1.4-1 will typically be less than 1.0 for connections utilizing 0.25-in. thick fillers in order to ensure both adequate shear resistance and limited deformation of the connection.

For slip-critical connections, the nominal slip resistance of a bolt need not be adjusted for the effect of the fillers. The resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if fillers were not present.

For fillers 0.25 in. or greater in thickness in axially loaded bolted connections, the specified minimum yield strength of the fillers should theoretically be greater than or equal to the specified minimum yield strength of the connected plate times the factor $[1/(1+\gamma)]$ in order to provide fully developed fillers that act integrally with the connected plate. However, such a requirement may not be practical or convenient due to material availability issues. As a result, premature yielding of the fillers, bolt bending and increased deformation of the connection may occur in some cases at the strength limit state. To control excessive deformation of the connection, a lower limit on the specified minimum yield strength of the filler plate material is recommended for fillers 0.25 in. or greater in thickness. Connections where the fillers are appropriately extended and developed or where additional bolts are provided according to Eq. 6.13.6.1.4-1 in lieu of extending the fillers, but that do not satisfy the recommended yield strength limit, will still have adequate reserve shear resistance in the connection bolts. However, such connections will have an increased probability of larger deformations at the strength limit state. For fillers less than 0.25 in. in thickness, the effects of yielding of the fillers and deformation of the connection are considered inconsequential. For applications involving the use of weathering steels, a weathering grade product should be specified for the filler plate material.

6.13.6.2—Welded Splices

Welded splice design and details shall conform to the requirements of the latest edition of AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and the following provisions specified herein.

Welded splices for tension and compression members shall be designed to resist the design axial force specified in Article 6.13.1. Tension and compression members may be spliced by means of full penetration butt welds. Flexural members shall be spliced by means of full penetration butt welds. The use of splice plates should be avoided.

Welded field splices should be arranged to minimize overhead welding.

C6.13.6.2

Flange width transition details typically show the transition starting at the butt splice. Figure 6.13.6.2-1 shows a preferred detail where the splice is located a minimum of 3.0 in. from the transition for ease in fitting runoff tabs. Where possible, constant width flanges are preferred in a shipping piece.

Material of different widths spliced by butt welds shall have symmetric transitions conforming to Figure 6.13.6.2-1. The type of transition selected shall be consistent with the detail categories of Table 6.6.1.2.3-1 for the groove-welded splice connection used in the design of the member. The contract documents shall specify that butt weld splices joining material of different thicknesses be ground to a uniform slope between the offset surfaces, including the weld, of not more than one in 2.5.

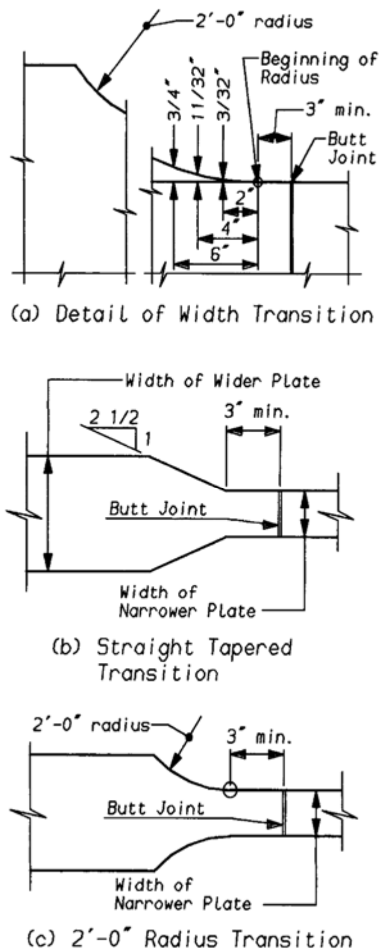


Figure 6.13.6.2-1—Splice Details

6.13.7—Rigid Frame Connections

6.13.7.1—General

All rigid frame connections shall be designed to resist the moments, shear, and axial forces due to the factored loading at the strength limit state.

C6.13.7.1

The provisions for rigid frame connections are well documented in Chapter 8 of ASCE (1971).

The rigidity is essential to the continuity assumed as the basis for design.

6.13.7.2—Webs

The thickness of an unstiffened beam web shall satisfy:

C6.13.7.2

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