



## 16.6 DESIGN CRITERIA FOR STRENGTHENING STEEL GIRDERS FOR LIVE LOADS

### 16.6.1 GENERAL

This STP addresses design criteria for strengthening existing steel girders for live loads. The criteria specify requirements for flexural strengthening by adding torsional bracing, shear connectors, cover plates, post-tensioning, and longitudinal stiffeners; and for shear strengthening by adding transverse stiffeners and web plates.

### 16.6.2 DEFINITIONS

Unshored Construction—A construction method in which the steel girders are not shored before deck concrete hardens.

### 16.6.3 NOTATION

- $A_{fc}$  = area of the existing compression flange (in.<sup>2</sup>)
- $A_{ps}$  = area of the prestressing steel (in.<sup>2</sup>)
- $A_r$  = area of the existing longitudinal reinforcement within the effective width of the concrete deck (in.<sup>2</sup>)
- $A_{sce}$  = effective cross-sectional area of one threaded stud shear connector, taken as 80% of the gross unthreaded cross-sectional area of the connector (in.<sup>2</sup>)
- $A_{ft}$  = area of the existing tension flange (in.<sup>2</sup>)
- $A_w$  = area of the existing web (in.<sup>2</sup>)
- $b_s$  = effective width of concrete deck (in.)
- $C$  = ratio of the shear-buckling resistance to the shear yield strength determined by Article 6.10.9.3
- $C_b$  = moment gradient modifier determined by Article 6.10.8.2.3
- $c$  = distance from the extreme compression fiber to the neutral axis (in.)
- $D$  = depth of the web (in.)
- $d_e$  = effective web depth (in.)
- $d_o$  = transverse stiffener spacing (in.)
- $E$  = modulus of elasticity of steel (ksi)
- $e$  = distance from the inside face of the bearing stiffener to the end of the girder,



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for calculation purposes, shall not exceed  $0.84t_w\sqrt{E/F_{yw}}$  (in.)

- $F_y$  = specified minimum yield strength of steel (ksi)
- $F_u$  = specified minimum tensile strength of steel; specified minimum tensile strength of shear connector material (ksi)
- $F_{ua}$  = actual tensile strength of the existing steel (ksi)
- $F_{ya}$  = actual yield strength of the existing steel (ksi)
- $F_{yar}$  = actual yield strength of the existing reinforcement (ksi)
- $F_{yfs}$  = equivalent yield strength of the strengthened flange (ksi)
- $F_{yc}$  = specified minimum yield strength of the new cover plate (ksi)
- $F_{yl}$  = specified minimum yield strength of the connecting angle (ksi)
- $F_{yw}$  = specified minimum yield strength of the web (ksi)
- $F_{yaf}$  = actual yield strength of the existing flange (ksi)
- $F_{yaw}$  = actual yield strength of the existing web (ksi)
- $F_{yw}$  = specified minimum yield strength of the new web (ksi)
- $F_{yws}$  = equivalent yield strength of the strengthened web (ksi)
- $f'_c$  = specified minimum 28-day compressive strength of concrete (ksi)
- $f'_{ca}$  = actual concrete compressive strength of the existing concrete (ksi)
- $f_{ps}$  = stress in the unbonded prestressing steel (ksi)
- $I_{yeff}$  = effective out-of-plane moment of inertia of the existing girder section (in.<sup>4</sup>)
- $I_{yc}$  = moment of inertia of the existing compression flange about the vertical axis in the plane of the web (in.<sup>4</sup>)
- $I_{yt}$  = moment of inertia of the existing tension flange about the vertical axis in the plane of the web (in.<sup>4</sup>)
- $L$  = length of the girder span under consideration (in.)
- $M_n$  = nominal flexural resistance (kip-in.)
- $M_{US}$  = largest of the factored moments at the strength limit state of the strengthened girders within the unbraced lengths adjacent to the bracing (kip-in.)
- $M_{ys}$  = yield moment of a strengthened steel girder section (kip-in.)
- $M_{pf}$  = plastic moment of a section composed of the top flange and the segment of the web with the depth,  $d_e$ , about the plastic neutral axis as shown in Figure 16.6.7.1.1 (kip-in.)
- $M_{prm}$  = smaller of  $M_{pst}$  and  $M_{pf}$  (kip-in.)



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- $M_{pst}$  = plastic moment of a section composed of the bearing stiffener and the segment of the web with depth,  $d_e$ , plus the distance from the inside face of the stiffener to the end of the girder,  $e$ , about the plastic neutral axis as shown in Figure 16.6.7.1.1 (kip-in.)
- $n$  = number of braced points within the span
- $Q_n$  = nominal shear resistance of a single stud shear connector (kip)
- $t$  = distance from the extreme tension fiber to the neutral axis (in.)
- $t_c$  = thickness of the new cover plate (in.)
- $t_{fc}$  = thickness of the existing compression flange (in.)
- $t_s$  = thickness of the concrete deck (in.)
- $t_w$  = thickness of the web (in.)
- $t_{we}$  = thickness of the existing web (in.)
- $t_{wl}$  = thickness of the connecting angle (in.)
- $t_{wn}$  = thickness of the new web (in.)
- $V_n$  = nominal shear resistance of the steel web (kip)
- $V_p$  = plastic shear force (kip) as specified in Article 6.10.9.3
- $\phi_{br}$  = resistance factor for the bracing = 0.75
- $\alpha$  = parameter to consider partial tension-field action
- $\beta_{br}$  = flexural stiffness of the bracing (kip-in/rad)

### 16.6.4 GENERAL REQUIREMENTS

Except as specified herein, a strengthened steel girder shall be designed in accordance with the following requirements and order of hierarchy:

- (1) Project specific design criteria
- (2) *AASHTO-CA BDS* for the limit states: Strength, Service, and Fatigue, as applicable. The fatigue design life shall be not less than the service life remaining for the existing girder.

For existing steel components and reinforcement, actual yield and tensile strength,  $F_{ya}$ ,  $F_{yar}$ , and  $F_{ua}$ , should be used if available, in lieu of as-built specified minimum yield and tensile strength,  $F_y$  and  $F_u$ , respectively.

For existing concrete deck slabs, actual concrete compressive strength,  $f'_{ca}$  should be used if available, in lieu of as-built specified minimum 28-day compressive strength of concrete,  $f'_c$ .

Stress at the service limit state and the fatigue limit state shall be based on the as-built



sequential loading history, strengthening construction sequences, and corresponding section properties. Where as-built information is not available, unshored construction shall be assumed. For composite sections in negative flexure, section properties shall include the longitudinal reinforcement within the effective width of the concrete deck provided that shear connectors meet requirements specified in Article 6.10.10. Section properties of a steel girder strengthened by post-tensioning shall include prestressing steel.

For an existing steel girder using ASTM A7 with  $F_y$  of 30 ksi, the nominal flexural resistance,  $M_n$ , shall be taken as the yield moment,  $M_{ys}$ .

### 16.6.5 MATERIAL PROPERTIES FOR EXISTING STRUCTURES

Material properties for existing structures,  $F_{ya}$ ,  $F_{yar}$ ,  $F_{ua}$ , and  $f'_{ca}$ , should be obtained from physical tests if feasible, in accordance with *the Manual for Bridge Evaluation* (AASHTO, 2019). Other existing concrete material properties including the modulus of elasticity, the creep coefficient, and the modulus of rupture, shall be determined in accordance with *AASHTO-CA BDS*, except that  $f'_{ca}$  if available is used in lieu of  $f'_c$ .

### 16.6.6 FLEXURAL STRENGTHENING

#### 16.6.6.1 Bracing

Discrete bracing, such as cross frames and diaphragms, may be added to increase the flexural resistance of an existing girder that is controlled by lateral torsional buckling.

The strength of the bracing shall provide torsional restraint greater than  $0.02M_{us}$ , where  $M_{us}$  is the largest of the factored moments at the strength limit state of the strengthened girders within the unbraced lengths adjacent to the bracing (kip-in.). The flexural stiffness of the bracing,  $\beta_{br}$ , shall satisfy:

$$\beta_{br} \geq \frac{1}{\phi_{br}} \frac{2.4L}{nEI_{yeff}} \left( \frac{M_{us}}{C_b} \right)^2 \quad (16.6.6.1.1)$$

Where:

$$I_{yeff} = I_{yc} + \left( \frac{t}{c} \right) I_{yt} \quad (16.6.6.1.2)$$

$E$  = modulus of elasticity of steel (ksi)

$I_{yeff}$  = effective out-of-plane moment of inertia of the existing girder section (in.<sup>4</sup>)

$I_{yc}$  = moment of inertia of the exiting compression flange about the vertical axis in the plane of the web (in.<sup>4</sup>)

$I_{yt}$  = moment of inertia of the existing tension flange about the vertical axis in the plane of the web (in.<sup>4</sup>)



- $L$  = length of the girder span under consideration (in.)
- $C_b$  = moment gradient modifier determined by Article 6.10.8.2.3
- $c$  = distance from the extreme compression fiber to the neutral axis (in.)
- $n$  = number of braced points within the span
- $t$  = distance from the extreme tension fiber to the neutral axis (in.)
- $\phi_{br}$  = resistance factor for the bracing = 0.75
- $\beta_{br}$  = flexural stiffness of the bracing (kip-in/rad)

Where girders are noncomposite, existing bracing systems shall be supplemented or stiffened to satisfy Eq. 16.6.6.1.1.

## 16.6.6.2 Composite Sections

### 16.6.6.2.1 General

Shear connectors may be used on the top flange of non-composite steel girders to increase flexural resistance by making a composite section. Shear connectors shall be installed by welding or bolting.

The pitch of post-installed shear connectors and shear connector groups along the line of force shall satisfy Article 6.10.10.1.2.

### 16.6.6.2.2 Welded Stud Shear Connectors

Pockets for shear connectors shall be roughened to ¼ inch amplitude or beveled such that the top of the pocket is wider than the bottom. The side slope of beveled pockets shall not exceed 5:1. The clear distance between the edges of the pocket and nearest shear stud shall not be less than 2 inches.

The requirement that the pocket shall be filled with nonshrink grout or any construction material meeting strength, modulus of elasticity, durability, and fatigue resistance equivalent to structural concrete shall be specified in the project special provisions or shown on the plans.

### 16.6.6.2.3 Non-welded Stud Shear Connectors

The nominal shear resistance of one non-welded stud shear connector shall be taken as (Kwon, et al. 2007):

$$Q_n = 0.5A_{sce}F_u \quad (16.6.6.2.1)$$

where:

$Q_n$  = nominal shear resistance of a single stud shear connector (kip)



$A_{sce}$  = effective cross-sectional area of one threaded stud shear connector, taken as 80% of the gross unthreaded cross-sectional area of the connector (in.<sup>2</sup>)

$F_u$  = specified minimum tensile strength of shear connector material (ksi)

### 16.6.6.3 Cover Plates

#### 16.6.6.3.1 Flexural Resistance

A cover plate may be used to increase the flexural resistance of an existing steel girder that is controlled by either flange yielding or flange local buckling.

For a compact section with only the compression flange strengthened by a cover plate, the nominal flexural resistance,  $M_n$ , shall be taken as the moment of the plastic forces about the plastic neutral axis.

For a compression flange strengthened by a cover plate, the combined flange thickness and the equivalent yield strength shall be used in determining the local buckling resistance.

$$F_{yfs} = \frac{t_{fc}F_{yaf} + t_cF_{yc}}{t_{fc} + t_c} \quad (16.6.6.3.1)$$

where:

$F_{yfs}$  = equivalent yield strength of the strengthened flange (ksi)

$F_{yaf}$  = actual yield strength of the existing flange (ksi)

$F_{yc}$  = specified minimum yield strength of the new cover plate (ksi)

$t_{fc}$  = thickness of the existing compression flange (in.)

$t_c$  = thickness of the new cover plate (in.)

For a tension flange strengthened by a cover plate, the nominal flexural resistance,  $M_n$ , shall be taken as the yield moment,  $M_{ys}$ .

$M_{ys}$  shall be taken as the moment that causes yielding in deck reinforcement or any component of the strengthened girder.  $M_{ys}$  shall be determined in accordance with Article D6.2.

#### 16.6.6.3.2 Cover Plate Connections

Welded cover plates are not permitted. Bolted connections shall be designed as slip-critical.

At the strength limit state, bolted connections on either side of the location of the maximum stress of the cover plate shall be designed for the plastic force of the cover plate.

### 16.6.6.4 Post-Tensioning Steel Girders

#### 16.6.6.4.1 General

Post-tensioning may be used to increase the flexural resistance of an existing steel girder that is controlled by tension flange yielding or compression flange overstress.

Prestressing steel should be placed in the tension zone at the strength limit state, as far as possible from the neutral axis. For continuous steel girders, secondary effects due to post-tensioning shall be considered. Prestressing steel shall be anchored beyond the point at which it is no longer required.

Long term prestress losses may be taken as 5 ksi for strands and 3 ksi for bars unless a refined analysis is performed. Short term prestress losses shall be determined in accordance with Article 5.9.3.

#### 16.6.6.4.2 Flexural Resistance

The nominal flexural resistance,  $M_n$ , shall be taken as the moment of the component forces including post-tensioning forces about the neutral axis based on the assumptions specified in Articles 6.10.7 and 6.10.8.

Stress in the unbonded prestressing steel,  $f_{ps}$ , should be determined in accordance with Article 5.6.3.1.2 using the distance from the extreme compression fiber to the neutral axis,  $c$ .

For compact sections,  $c$  should be determined as follows:

(1) For composite sections in positive flexure:

- For the neutral axis within the concrete deck

$$c = \frac{A_{ps}f_{ps} + A_{ft}F_{yaf} + A_wF_{yaw} + A_{fc}F_{yaf}}{0.85f'_{ca} b_s} \quad (16.6.6.4.1)$$

- For the neutral axis within the top compression flange

$$c = \left( \frac{t_{fc}}{2} \right) \left[ \frac{A_{ps}f_{ps} + A_{ft}F_{yaf} + A_wF_{yaw} - 0.85f'_{ca} b_s t_s}{A_{fc}F_{yaf}} + 1 \right] + t_s \quad (16.6.6.4.2)$$

- For the neutral axis within the web

$$c = \left( \frac{D}{2} \right) \left[ \frac{A_{ps}f_{ps} + A_{ft}F_{yaf} - 0.85f'_{ca} b_s t_s - A_{fc}F_{yaf}}{A_wF_{yaw}} + 1 \right] + t_{fc} + t_s \quad (16.6.6.4.3)$$

where:

$A_{fc}$  = area of the existing compression flange (in.<sup>2</sup>)



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- $A_{ps}$  = area of the prestressing steel (in.<sup>2</sup>)
- $A_{ft}$  = area of the existing tension flange (in.<sup>2</sup>)
- $A_w$  = area of the existing web (in.<sup>2</sup>)
- $b_s$  = effective width of concrete deck (in.)
- $D$  = depth of the web (in.)
- $F_{yaf}$  = actual yield strength of the existing flange (ksi)
- $F_{yaw}$  = actual yield strength of the existing web (ksi)
- $t_{fc}$  = thickness of the existing compression flange (in.)
- $t_s$  = thickness of the concrete deck (ksi)

(2) For composite sections in negative flexure:

- For the neutral axis within the bottom compression flange

$$c = \left( \frac{t_{fc}}{2} \right) \left[ \frac{A_{ps}f_{ps} + A_rF_{yar} + A_{ft}F_{yaf} + A_wF_{yaw}}{A_{fc}F_{yaf}} + 1 \right] \quad (16.6.6.4.4)$$

- For the neutral axis within the web

$$c = \left( \frac{D}{2} \right) \left[ \frac{A_{ps}f_{ps} + A_rF_{yar} + A_{ft}F_{yaf} - A_{fc}F_{yaf}}{A_wF_{yaw}} + 1 \right] + t_{fc} \quad (16.6.6.4.5)$$

where:

- $A_r$  = area of the existing longitudinal reinforcement within the effective width of the concrete deck (in.<sup>2</sup>)
- $F_{yar}$  = actual yield strength of the existing reinforcement (ksi)

(3) For noncomposite sections. Eqs. 16.6.6.4.4 and 16.6.6.4.5 apply except that  $A_r$  is taken as zero.

For noncompact sections,  $c$  should be determined by the following assumptions:

- Stress in prestressing steel is taken as  $f_{ps}$
- Stresses in flanges are taken as nominal flexural resistances for the compression or tension flanges as specified in Article 6.10.7.2.1.
- Stresses in the web are linearly distributed between flange stresses and compression stress shall not exceed the web bend-buckling resistance as specified in Article 6.10.1.9.
- Compression stress in concrete deck is taken as  $0.6 f'_{ca}$
- Tensile stress in the longitudinal reinforcement is taken as  $F_{yar}$





#### 16.6.6.4.3 Stress Limitations

Stresses in post-tensioned steel girders shall be investigated for two Service II limit state load cases: (1) before losses due to the sum of initial prestress and permanent loads, and (2) after losses due to the sum of effective prestress, permanent loads, and transient loads.

Stress limits for prestressing steel specified in Article 5.9.2.2 shall apply.

Stress limits for concrete deck and steel girders specified in Article 6.10.4.2 shall apply.

#### 16.6.6.4.4 Anchorage Bracket

Steel components of the bracket shall be designed to remain elastic. At the strength limit state, the design force shall be taken as 1.2 times the maximum jacking force.

#### 16.6.6.4.5 Anchorage Bracket Connections

A post-tensioning anchorage bracket shall be connected to the existing girders by high strength bolts. An anchorage bracket connection shall be designed as slip-critical for combined shear and tension.

- At the strength limit state, bolted connections shall be designed for 1.2 times the maximum jacking force.
- At the jacking stage, bolted connections shall be designed to prevent slip for the maximum jacking force.

#### 16.6.6.5 Longitudinal Stiffeners

Longitudinal stiffeners may be added to increase the flexural resistance of an existing steel girder that is controlled by web bend-buckling.

Longitudinal stiffeners shall be bolted to the existing webs.

### 16.6.7 SHEAR STRENGTHENING

#### 16.6.7.1 Nominal Shear Resistance of Simply Supported End Panel

The nominal shear resistance,  $V_n$ , for a stiffened web end panel (Uang and Kim 2019; Kim and Uang 2015) shall be taken as:

$$V_n = V_p \left[ C + \alpha \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}} \right] \quad (16.6.7.1.1)$$



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where:

$$\alpha = \frac{2.8(\sqrt{M_{pf} + M_{pm}} + \sqrt{M_{pst} + M_{pm}})}{D\sqrt{t_w F_{yw}(1-C)}} \leq 1.0 \quad (16.6.7.1.2)$$

$C$  = ratio of the shear-buckling resistance to the shear yield strength determined by Article 6.10.9.3

$d_o$  = transverse stiffener spacing (in.)

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

$M_{pf}$  = plastic moment of the section composed of the top flange and the segment of the web with the depth,  $d_e$ , about the plastic neutral axis as shown in Figure 16.6.7.1.1 (kip-in.)

$M_{pm}$  = smaller of  $M_{pst}$  and  $M_{pf}$  (kip-in.)

$M_{pst}$  = plastic moment of the section composed of the bearing stiffener and the segment of the web with depth,  $d_e$ , plus the distance from the inside face of the stiffener to the end of the girder,  $e$ , about the plastic neutral axis as shown in Figure 16.6.7.1.1 (kip-in.)

$V_p$  = plastic shear force (kip) as specified in Article 6.10.9.3

$t_w$  = thickness of the web (in.)

$\alpha$  = parameter to consider partial tension-field action

$e$  = distance from the inside face of the bearing stiffener to the end of the girder, for calculation purposes, shall not exceed  $0.84t_w\sqrt{E/F_{yw}}$  (in.)

$d_e$  = effective web depth (in.)

If  $C \leq 0.8$ , then:

$$d_e = 35t_w(0.8 - C)^2 \quad (16.6.7.1.3)$$

Otherwise:

$$d_e = 0 \quad (16.6.7.1.4)$$

The effective web areas and the plastic neutral axes for  $M_{pf}$  and  $M_{pst}$  are shown in Figure 16.6.7.1.1.

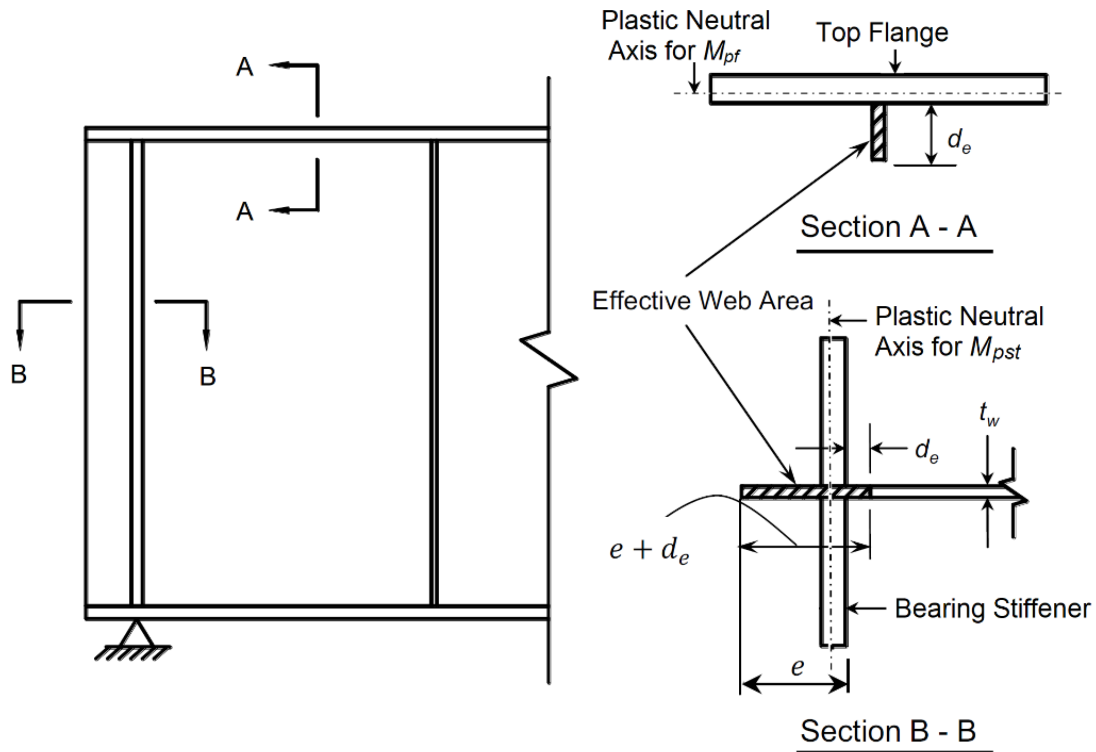


Figure 16.6.7.1.1 Effective Web Area and Plastic Neutral Axis

### 16.6.7.2 Transverse Stiffeners

Transverse stiffeners may be added to increase the shear resistance of an existing steel girder that is controlled by shear buckling.

Transverse stiffeners should be bolted to the existing webs.

### 16.6.7.3 Web Plates

#### 16.6.7.3.1 General

A web doubler plate may be added to increase the shear resistance of an existing steel girder that is controlled by either shear yielding or shear buckling.

#### 16.6.7.3.2 Shear Resistance

At the strength limit state, the combined thickness and the equivalent yield strength shall be used in determining the ratio of the shear-buckling resistance to the shear yield strength,  $C$ .



$$F_{yws} = \frac{t_{we} F_{yaw} + t_{wn} F_{yw}}{t_{we} + t_{wn}} \quad (16.6.7.3.1)$$

where:

$F_{yws}$  = equivalent yield strength of the strengthened web (ksi)

$F_{yaw}$  = actual yield strength of the existing web (ksi)

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

$t_{we}$  = thickness of the existing web (in.)

$t_{wn}$  = thickness of the new web (in.)

### 16.6.7.3.3 Web Plate Connections

A new web plate shall be connected to the existing web by high strength bolts.

A web plate connection shall be designed as slip-critical for combined shear and moment.

At the strength limit state, a web plate connection shall be designed for a shear force taken as  $0.58F_{yw}t_{wn}D$ , combined with shear induced moments.

### 16.6.7.3.4 Web-to-Flange Connections

Bolted angles may be used to increase the strength of existing web-to-flange connections. Web-to-flange connections shall be designed as slip-critical.

At the strength limit state, a web-to-flange connection shall be designed for a horizontal shear flow,  $0.58F_{yw}t_{wn}$ .

The connecting angle shall satisfy

$$t_{wl} F_{yl} \geq t_{wn} F_{yw} \quad (16.6.7.3.2)$$

Where:

$F_{yl}$  = specified minimum yield strength of the connecting angle (ksi)

$t_{wl}$  = thickness of the connecting angle (in.)

## 16.6.8 REFERENCES

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