6.4.3.1 Bolts

Revise the 1st sentence as follows:

Bolts <u>used as structural fasteners</u> shall conform to one of the following:

- The Standard Specification for Carbon Steel Bolts and Studs, 60 ksi Tensile Strength, ASTM A 307, <u>Grade A or B,</u>

Revise the 2nd to 5th Paragraph as follows:

Type 1 bolts should be used with steels other than weathering steel. Type 3 bolts conforming with either AASHTO M 164 (ASTM A 325) or AASHTO M 253 (ASTM A 490) shall be used with weathering steels. AASHTO M 164 (ASTM A 325) Type 1 bolts may be either hot-dip galvanized in accordance with <u>AASHTO M 232M/M 232 (ASTM A 153/A 153M),</u> Class C, or mechanically galvanized in accordance with AASHTO M 298 (ASTM B695), Class 50 when approved by the Engineer. Galvanized bolts shall be <u>retested</u> after galvanizing, as required by AASHOT M 164 (ASTM A 325).

AASHTO M 253 (ASTM A 490) bolts shall not be galvanized.

Washers, nuts and bolts of any assembly shall be galvanized by the same process. The nuts should be overtapped to the minimum amount required for the fastener assembly and shall be lubricated with a lubricant containing a visible dye.

<u>Anchor Bolts shall conform to one of the following:</u>

- <u>ASTM A 307 Grade C, or</u>
- <u>ASTM F 1554</u>

C6.4.3.1

Revise the 1st Paragraph as follows:

The ASTM standard for A 307 bolts covers ~~two~~ <u>three</u> grades of fasteners<u>, A, B, and C. Grade A and B bolts may be used under this Specification as appropriate. There is no AASHTO standard corresponding to ASTM A 307</u>. ~~There is no corresponding AASHTO standard. Either grade may be used under these Specifications; however, Grade B is intended for pipeflange bolting, and Grade A is the quality traditionally used for structural applications.~~

<u>ASTM A 307 Grade C are nonheaded anchor bolts intended for structural anchorage purposes. There is no AASHTO standard corresponding to ASTM F 1554.</u>

6.4.3.2 Nuts

Revise as follows:

6.4.3.2.1 Nuts Used with Structural Fasteners

Nuts used with structural fasteners shall conform to the following as appropriate.

Except as noted below, nuts for AASHTO M 164 (ASTM A 325) bolts shall conform to the Standard Specification for Carbon and Alloy Steel Nuts, AASHTO M 291 (ASTM A 563), Grades DH, DH3, C, C3, and D.

Nuts for AASHTO M 253 (ASTM A 490) bolts shall conform to the requirements of AASHTO M 291 (ASTM A 563), Grades DH and DH3.

Nuts to be galvanized shall be heat treated Grade DH. The provisions of Article 6.4.3.1 shall apply. All galvanized nuts shall be lubricated with a lubricant containing a visible dye.

Plain nuts shall have a minimum hardness of 89 HRB.

Nuts to be used with AASTHO M 164 (ASTM A 325) Type 3 bolts shall be of Grade C3 or DH3. Nuts to be used with AASTHO M 253 (ASTM A 490) Type 3 bolts shall be of Grade DH3.

6.4.3.2.2 Nuts Used with Anchor Bolts

Nuts used with anchor bolts shall conform to the following as appropriate.

Nuts for ASTM A 307 Grade C and for ASTM F 1554 anchor bolts shall conform to AASHTO M 291 (ASTM A 563) for appropriate grade and size of anchor bolt.

Nuts to be galvanized shall be heat treated Grade DH or DH3. The provisions of Article 6.4.3.1 shall apply. All galvanized nuts should be lubricated with a lubricant containing a visible dye.

6.5.4.2 Resistance Factors

Revise as follows:

Resistance factors, ϕ, for the strength limit state shall be taken as follows:

- For flexure $\phi_f = 1.00$
- For shear $\phi_v = 1.00$
- For axial compression, steel only $\phi_c = 0.90$
- For axial compression, composite $\phi_c = 0.90$
- For tension, fracture in net section $\phi_u = 0.80$
- For tension, yielding in gross section $\phi_y = 0.95$
- For bearing on pins in reamed, drilled or bored holes on milled surfaces $\phi_b = 1.00$
- For bolts bearing on material $\phi_{bb} = 0.80$
- For shear connectors $\phi_{sc} = 0.85$
- For A 3258 and A 490 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in tension $\phi_t = 0.80$
- <u>For F 1554 bolts in tension</u> <u>$\phi_t = 0.80$</u>
- For A 307 bolts in shear <u>$\phi_s = 0.75$</u>
- <u>For F 1554 bolts in shear</u> <u>$\phi_S = 0.75$</u>
- For A 325 and A 490 bolts in shear $\phi_s = 0.80$
- For block shear $\phi_{bs} = 0.80$
- For web crippling $\phi_w = 0.80$
- For weld metal in complete penetration welds:
  - shear on effective area $\phi_{e1} = 0.85$
  - tension or compression normal to effective area same as base metal
  - tension or compression parallel to axis of the weld same as base metal
- For weld metal in partial penetration welds:
  - shear parallel to axis of weld $\phi_{e2} = 0.80$
  - tension or compression parallel to axis of weld same as base metal
  - compression normal to the effective area same as base metal
  - tension normal to the effective area $\phi_{e1} = 0.80$
- For weld metal in fillet welds:
  - tension or compression parallel to axis of the weld same as base metal
  - shear in the throat of weld metal $\phi_{e2} = 0.80$
- For resistance during pile driving $\phi = 1.00$

- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_c = 1.00$

6.5.5 Extreme Event Limit State

Revise as follows:

All applicable extreme event load combinations in Table 3.4.1-1 shall be investigated.

All resistance factors for the extreme event limit state, except for bolts, shall be taken to be 1.0.

<u>All resistance factors for ASTM A 307 Grade C and ASTM F 1554 bolts used as anchor bolts for the extreme event limit state shall be taken to be 1.0.</u>

Bolted joints not protected by capacity design or structural fuses may be assumed to behave as bearing-type connections at the extreme event limit state, and the values of resistance factors for bolts given in Article 6.5.4.2 shall apply.

6.6.1.2.5 Fatigue Resistance

Revise as follows:

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

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

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

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

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

in which:

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

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

where:

A = constant taken from Table 1 (ksi^3)

n = number of stress range cycles per truck passage taken from Table 2

($ADTT$) = single-lane *ADTT* as specified in Article 3.6.1.4

$(\Delta F)_{TH}$ = constant-aplitude fatigue threshold taken from Table 3 (ksi)

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

C6.6.1.2.5

Revise the 1st 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 6th 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.

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

Revise as follows:

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

C6.6.1.2.5

Revise the 7th 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.

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 Griders	5.0	
Trusses	1.0	
Transverse Members	Spacing	
	> 20.0 ft.	≤ 20.0 ft
	1.0	2.0

C6.6.1.2.5

Add a new last Paragraph as follows:

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.

C6.10.1 General

Revise the 1st Paragraph as follows:

This Article addresses the 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 lateral bending, usually extend 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 horizontally curved girders or segments.

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6.10.5.3 Special Fatigue Requirement for Webs

Revise the 1st Paragraph as follows:

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

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6.10.6.2.2 Composite Sections in Positive Flexure

Revise the 3rd Paragraph as follows:

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

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6.10.7.1.2 Nominal Flexural Resistance

Revise equation 6.10.7.1.2-2 as follows:

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

$$\cancel{M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)} \qquad (6.10.7.1.2\text{-}2)$$

C6.10.7.1.2

Revise the 2nd 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, ~~as also suggested in Yakel and Azizinamini (*2005*). 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 parametic 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.~~

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6.10.10.2 Fatigue Resistance

Revise Equation 6.10.10.2-1 as follows:

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

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

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

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

$$\cancel{Z_r = \alpha d^2 \geq \frac{5.5d^2}{2}} \qquad \text{~~(6.10.10.2-1)~~}$$

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 the fatigue live load ttaken as specified in Article 3.6.1.4 (ksi)

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6.10.11.1.1 General

Revise the 2nd and 4th 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 flanges.~~

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 ~~but shall not exceed the lesser of~~ $6t_w$. In no case shall the distance exceed ~~and~~ 4.0 in.

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6.10.11.2.1 General

Revise the 4th 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.

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6.13.1 General

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

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

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

6.13.2.1 General

Add a 3rd paragraph as follows:

Anchor bolts shall be designed for tension, shear and combined tension and shear as applicable. For global design of anchorages to concrete, refer to *Building Code Requirements for Structural Concrete* (ACI 318-05) Appendix D.

6.13.2.7 Shear Resistance

Revise the 1st Paragraph as follows:

The nominal shear resistance of a high strength bolt **(ASTM A 325 or ASTM A 490)** or an ASTM A 307 Bolt **(Grade A or B)** at the strength limit state in joints whose length between extreme fasteners measured parallel to the line of action of the force is less than 50.0 shall be taken as:

Add a 2nd Paragraph just following definition of N_s as follows:

The nominal shear resistance of an anchor bolt (ASTM A307 Grade C or ASTM F 1554) at the strength limit state in joints whose length between extreme anchor bolts measured parallel to the line of action of the force is less than 50.0 in. shall be in accordance with Eq. 2.

C6.13.2.7

Revise the 1st sentence of the 2nd Paragraph as follows:

The average value of the nominal resistance for bolts with threads in the shear plane has been determined by a series tests to be 0.833 (0.6F_{ub}), with a standard deviation of 0.03 (Yura et al. 1987).

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6.13.2.10.3 Fatigue Resistance

Revise as follows:

Where high-strength bolts in axial tension are subject to fatigue, the stress range, $\cancel{\Delta_f}$, $\underline{\Delta f}$, in the bolt, due to the fatigue design live load, plus the dynamic load allowance for fatigue loading specified in Article 3.6.1.4, plus the prying force resulting from cyclic application of the fatigue load, shall satisfy Eq. 6.6.1.2.2-1.

The nominal diameter of the bolt shall be used in calculating the bolt stress range. In no case shall the calculated prying force exceed ~~60~~ <u>30</u> percent of the externally applied load.

Low carbon ASTM A 307 bolts shall not be used in connections subjected to fatigue.

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6.13.6.1.4b Web Splices

Revise the 2nd Paragraph as follows:

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

- ~~If $V_u < 0.5\phi_v V_n$, then:~~

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

- ~~Otherwise:~~

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

where:

ϕ_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 1st Paragraph

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C6.13.6.1.4b

Revise as follows:

For compact sections:

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

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

For noncompact sections:

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

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

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

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

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_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

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

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

~~f_{ncf} = flexural stress due to the factored loads at the midthicknes of the noncontrolling flange at the point of splice concurrent with f_{cf}; 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)

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

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

In Eqs. C1 and C2, it is suggested that M_{uw} and H_{uw} be computed by conservatively using the flexural resistance ~~stresses at the midthickness~~ of the compression flanges and specified minimum yield strentgth 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 C6.13.6.1.4c.~~

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_{cf} with the maximum flexural stress, f_s, due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of splice, replace f_{ncf} with the flexural stress, f_{os}, due to Load Combination Service II at the midthickness of the other flange at the point of splice concurrent with f_s in the flange under consideration, and~~
- ~~set the factors R_h and R_{cf} 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.~~

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

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

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_{uw} and H_{uw} 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.

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6.13.6.1.4c Flange Splices

Revise as follows:

At the strength limit state, splice plates and their connections to ~~on~~ the ~~controlling~~ flange 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:

$$\underline{F_{cf} = \alpha \phi_f F_{yf}}$$

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

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where:

~~f_{cf}~~ = ~~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~~

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

ϕ_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)

ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2

ϕ_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.^{2})

A_g = gross area of the tension flange (in.^{2})

F_u = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)

F_y = specified minimum yield strength of the tension flange (ksi)

Delete the 2nd Paragraph

C6.13.6.1.4c

Delete the 3rd and 5th Paragraph

6.13.6.1.4c Flange Splices

Delete Eq. (6.13.6.1.4c-3)

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

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

where:

C6.13.6.1.4c

Delete the 7th Paragraph

6.13.6.1.4c Flange Splices

Revise as follows:

f_s = maximum flexural stress due to Load Combination Service II at the extreme fiber ~~midthickness~~ of the flange under consideration for the small 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

C6.13.6.1.4c

Revise as follows:

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.

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 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_{cf}, 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.

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6.13.6.2 Welded Splices

Revise the 2nd Paragraph as follows:

Welded splices shall be designed ~~to resist the design moment, shear, or axial force specified in Article 6.13.1.~~ <u>at the strength limit state for not less than 100 percent of the factored flexural, shear, or axial resistance of the member or element.</u> Tension and compression members may be spliced by means of full penetration butt welds; splice plates should be avoided.

Revise the 3rd Paragraph as follows:

~~Welded field splices should be arranged tom minimize overhread welding.~~ <u>Splices shall not be welded in field.</u>

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6.14.2.8 Gusset Plates

Revise as follows:

6.14.2.8.1 General

Gusset plates, fasteners and welds connecting main members shall be designed at the strength limit state for not less than 100 percent of the factored flexural, shear or axial resistance of the member.

Gusset plates, fasteners and welds connecting other members shall be designed at the strength limit state for not less than the factored flexural, shear or axial force effects of the member.

The nominal resistance of a gusset plate shall be based on the effective width as shown in Figure C6.14.2.8.1-1. The gross and net cross-section area shall be on the effective width.

The provisions of Articles 6.13.2, 6.13.3, 6.13.4 and 6.13.5 shall apply, as applicable.

Gusset or connection plates should be used for connecting main members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration.

Re-entrant cuts, except curves made for appearance, should be avoided as far as practicable. ~~The maximum stress from combined factored flexural and axial loads shall not exceed~~ ~~$\phi_f F_y$~~ ~~based on the gross area.~~

~~The maximum shear stress on a section due to the factored loads shall be~~ ~~$\phi_v F_u/\sqrt{3}$~~ ~~for uniform shear and~~ ~~$\phi_v 0.74 F_u/\sqrt{3}$~~ ~~for flexural shear computed as the factored shear force divided by the shear area.~~

~~If the length of the unsupported edge of a gusset plate exceeds~~ ~~$2.06(E/F_y)^{1/2}$~~ ~~times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.~~

C6.14.2.8

Revise as follows:

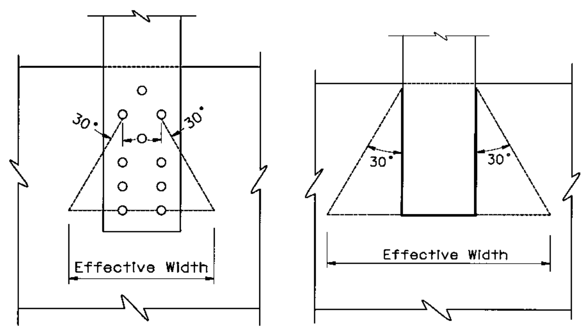
C6.14.2.8.1

~~Gusset plates may be designed for shear, bending, and axial force effects by the conventional "Method-of-Section" procedures or by continuum methods.~~

~~Plastic shape factors or other parameters that imply plastification of the cross-section should not be used.~~

Major revisions are based on Caltrans successful practice and Caltrans *Guide Specifications for Seismic Design of Steel Bridges* (*Caltrans 2001).*

Figure C6.14.2.8.1-1 shows the effective width for a gusset plate in accordance with Whitmore's method (*Whitmore 1952*).



(a) Bolted Gusset Plate (b) Welded Gusset Plate

Figure C6.14.2.8.1-1 Effective Width of Gusset Plate

6.14.2.8.2 Limiting Unsupported Edge Length to Thickness Ratio

The unsupported edge length to thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \le 2.06\sqrt{\frac{E}{F_y}} \qquad (6.14.2.8.2\text{-}1)$$

where

- L_g = unsupported edge length of a gusset plate (in.)
- t = thickness of a gusset plate (in.)
- E = modulus of elasticity of steel (ksi)
- F_y = specified minimum yield strength of the gusset plate (ksi)

For stiffened edge, the following requirements shall be satisfied:

- For welded stiffeners, slenderness ratio of the stiffener plus a width of gusset plate equal to ten times its thickness shall be $l/r \le 40$.
- For bolted stiffeners, slenderness ratio of the stiffener between fasteners shall be $l/r \le 40$.
- The moment of inertia of the stiffener shall be

$$I_s \ge \begin{cases} 1.83t^4\sqrt{(b/t)^2 - 144} \\ 9.2t^4 \end{cases} \qquad (6.14.2.8.2\text{-}2)$$

where

- I_s = moment of inertia of a stiffener about its strong axis (in.^{4})
- b = width of a gusset plate perpendicular to the edge (in.)
- t = thickness of a gusset plate (in.)

The moment of inertia of the stiffener that is required to develop the post buckling strength of a long plate has been experimentally determined by Eq. (6.14.2.8.2-2) (*AISI 1962*)

6.14.2.8.3 Tensile Resistance

The tensile resistance of a gusset plate shall be:

$$\phi P_n = \phi_y A_g F_y \le \begin{cases} \phi_u A_n F_u \\ R_r \end{cases} \qquad (6.14.2.8.3\text{-}1)$$

where

- A_n = net cross-section area of a gusset plate (in.^{2})
- A_g = gross cross-section area of a gusset plate (in.^{2})
- F_u = specified minimum tensile strength of the gusset plate (ksi)
- ϕ_u = resistance factor for tension fracture in net section = 0.80

C6.14.2.8.3

This requirement is to ensure that the tensile strength is governed by yielding in the gross section, and fracture in the net section and block shear rupture are prevented.

ϕ_y = resistance factor for tension yielding in gross section = 0.95

R_r = factored block shear rupture resistance specified by Article 6.13.4

6.14.2.8.4 Compressive Resistance

The nominal compressive resistance of a gusset plate, P_n, shall be calculated in accordance with Article 6.9.4.1.

C6.14.2.8.4

The effective length factor, *K* in Eqs. (6.9.4.1-1) and (6.9.4.1-2) may be taken as 0.6 for the gusset supported by both edges, and 1.2 for the gusset supported by one edge only (*AISC 2001*); A_s is the average effective cross section area defined by Whitmore's method; *l* is the perpendicular distance from the Whitmore section to the interior corner of the gusset. For members that are not perpendicular to each other as shown in Figure C6.14.8.2.4-1 (*AISC 2001*), *l* can be alternatively determined as the average value of

$$l = \frac{L_1 + L_2 + L_3}{3} \qquad \text{(C6.14.2.8.4-1)}$$

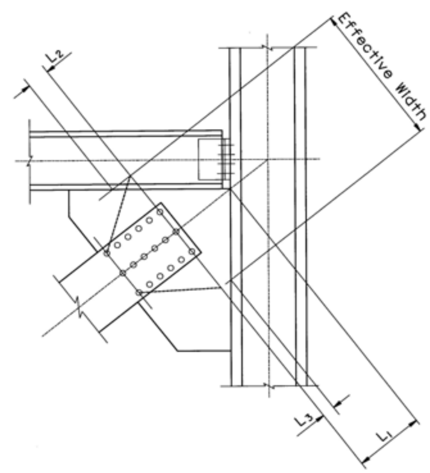


Figure C6.14.2.8.4-1Gusset Plate Connection

where

L_1 = distance from the centerline of the Whitmore section to the interior corner of a gusset plate (in.)

L_2, L_3 = distance from the outside corner of the Whitmore section to the edge of a member; negative value shall be used when the part of Whitmore section enters into the member (in.)

6.14.2.8.5 Flexural Resistance

The nominal flexural resistance of a gusset plate, M_n, shall be determined by:

$$M_n = S F_y \qquad \text{(6.14.2.8.5-1)}$$

where

S = elastic section modulus of the cross section of a gusset plate (in.^{3})

6.14.2.8.6 Shear Resistance

The nominal shear resistance of a gusset plate, V_n, shall be determined by:

$$V_n = 0.58 F_y A_g \tag{6.14.2.8.6-1}$$

where

A_g = gross cross-section area of a gusset plate (in.^{2})

6.14.2.8.7 Yielding Resistance under Combined Flexural and Axial Force Effects

The Whitmore's effective area and other critical areas of a gusset plate subjected the combined flexural and axial force effects shall satisfy the following equation:

$$\frac{M_{ux}}{\phi_f S_x F_y} + \frac{M_{uy}}{\phi_f S_y F_y} + \frac{P_u}{\phi F_y A_g} \leq 1 \tag{6.14.2.8.7-1}$$

where

ϕ_f = resistance factor for flexural

ϕ = resistance factor for axial compression = 0.9, for axial tension yielding = 0.95

M_{ux} = factored moment about x-x axis of the gusset plate (k-in.)

M_{uy} = factored moment about y-y axis of the gusset plate (k-in.)

P_u = factored axial force (kip)

S_x = elastic section modulus about x-x axis of the gusset plate (in.^{3})

S_y = elastic section modulus about y-y axis of the gusset plate (in.^{3})

A_g = gross area of the gusset plate (in.^{2})

F_y = specified minimum yield strength of the gusset plates (ksi)

6.14.2.8.8 Out-of-Plane Forces Consideration

For double gusset plate connections, out-of-plane moment shall be resolved into a couple of tension and compression forces acting on the near and far side plates.

For single gusset plate connections, out-of-plane moment and shear are about the weak axis.

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Add References

ACI. 2005. *Building Code Requirements for Structural Concrete*. 318-05. American Concrete Institute, Farmington Hills, MI

AISC. 2001. *Manual of Steel Construction, Load and Resistance Factor Design*, 3rd Ed., American Institute of Steel Construction, Chicago, IL.

AISI. 1962. *Light Gage Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, DC.

ASCE. 1971. *Plastic Design in Steel, A Guide and Commentary*, American Society of Civil Engineers, New York, NY.

Caltrans, 2001. *Guide Specifications for Seismic Design of Steel Bridges*, First Edition, California Department of Transportation, Sacramento, CA.

Whitmore, R. E. 1952. "Experimental Investigation of Stresses in Gusset Plates," *Bulletin 16*, University of Tennessee, Knoxville, TN.

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