CHAPTER 6.1 STEEL DESIGN THEORY

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6.1.1 INTRODUCTION

Steel has higher strength, ductility, and toughness than many other structural materials such as concrete or wood, and thus makes an essential material for bridge structures. This chapter addresses basic steel design concepts and requirements for I-sections specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) and the *California Amendments* (Caltrans, 2019) for flexure, shear, compression, tension, fatigue, and serviceability and constructability. Chapter 6.2 presents design considerations, procedures, and an example for steel plate girders.

6.1.2 STRUCTURAL STEEL MATERIALS

AASHTO M 270 (Grade 36, 50, 50S, 50W, HPS 50W, HPS 70W and HPS 100W) structural steels are commonly used for bridge structures. AASHTO material property standards differ from ASTM in notch toughness and weldability requirements. When these additional requirements are specified, ASTM A709 steel is equivalent to AASHTO M 270 and is pre-qualified for use in welded steel bridges.

The use of ASTM A709 Grade 50 for all structural steel, including flanges, webs, bearing stiffeners, intermediate stiffeners, cross frames, diaphragms, and splice plates is required per Caltrans Standard Specifications (Caltrans, 2018). A hybrid section consisting of flanges with a higher yield strength than that of the web may be used to save materials and is becoming more popular due to the new high performance steels. Using HPS 70W top and bottom flanges in negative moment regions and bottom flanges in positive moment regions and Grade 50 top flanges in positive moment regions, and Grade 50 for all webs may provide the most efficient hybrid girder.

The use of HPS (High Performance Steel) and weathering steel is encouraged in the locations specified in the FHWA Technical Advisory T5140.22 (FHWA, 1989). HPS and weathering steel should not be used in the following conditions:

- The atmosphere contains concentrated corrosive industrial or chemical fumes.
- The steel is subject to heavy salt-water spray or salt-laden fog.
- The steel is in direct contact with timber decking because timber retains moisture and may have been treated with corrosive preservatives.
- The steel is used for a low urban-area overcrossing that will create a tunnel-like configuration over a road on which deicing salt is used. In these situations, road spray from traffic under the bridge causes salt to accumulate on the steel.
- The location has inadequate air flow that does not allow adequate drying of the steel.
- The location has very high rainfall and humidity or there is constant wetness.
- There is low clearance (less than 8 to 10 ft) over stagnant or slow-moving waterways.

6.1.3 DESIGN LIMIT STATES

Steel girder bridges shall be designed to meet the requirements for all applicable limit states and constructability specified by AASHTO (2017) and *California Amendments* (Caltrans, 2017). For a typical steel girder bridge, Strength I and II, Service II, and Fatigue are usually controlling limit states. For seismic design, steel bridges shall satisfy the requirements as specified in *Caltrans Seismic Design Specifications for Steel Bridges* (Caltrans, 2016).

6.1.4 FLEXURAL DESIGN

6.1.4.1 Design Requirements

The AASHTO 6.10 and its Appendices A6 and B6 provide a unified flexural design approach for steel I-girders. The provisions combine major-axis bending, minor-axis bending, and torsion into an interaction design formula and are applicable to straight bridges, horizontally curved bridges, or bridges combining both straight and curved segments. The AASHTO flexural design interaction equations for the strength limit state are summarized in Table 6.1.4-1. Those equations provide an accurate linear approximation of the equivalent beam-column resistance with the flange lateral bending stress less than $0.6F_y$ as shown in Figure 6.1.4-1 (White and Grubb 2005).

For compact sections, since the nominal moment resistance is generally greater than the yield moment capacity, it is physically meaningful to design in terms of moment. For noncompact sections, since the nominal resistance is limited to the yield strength, stress format is used. For (1) composite I-sections in negative flexure and (2) noncomposite I-sections with compact or noncompact webs in straight bridges, the provisions specified in AASHTO Appendix 6A are encouraged to be used when the web slenderness is well below the noncompact limit. However, when the web slenderness approaches the noncompact limit, Appendix 6A provides only minor increases in the nominal resistance.

Table 6.1.4-1. I-Section Flexural Design Equations (Strength Limit State)

Section Type		Design Equation
Composite Sections in Positive Flexure	Compact	$M_u + \frac{1}{3} f_l S_{xt} \le \phi_f M_n$ (AASHTO 6.10.7.1.1-1)
	Noncompact	Compression flange $f_{bu} \le \phi_f F_{nc}$ (AASHTO 6.10.7.2.1-1) Tension flange $f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nt}$ (AASHTO 6.10.7.2.1-2)
Composite Sections in Negative Flexure and Noncomposite Sections	Discretely braced	Compression flange $f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc}$ (AASHTO 6.10.8.1.1-1) Tension flange $f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nt}$ (AASHTO 6.10.8.1.2-1)
	Continuously braced	$f_{bu} \le \phi_f R_h F_{yf}$ (AASHTO 6.10.8.1.3-1)

f_{bu} = flange stress calculated without consideration of the flange lateral bending (ksi)

f_l = flange lateral bending stress (ksi)

F_{nc} = nominal flexural resistance of the compression flange (ksi)

F_{nt} = nominal flexural resistance of the tension flange (ksi)

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

M_u = bending moment about the major axis of the cross section (kip-in.)

M_n = nominal flexural resistance of the section (kip-in.)

ϕ_f = resistance factor for flexure = 1.0

R_h = hybrid factor

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

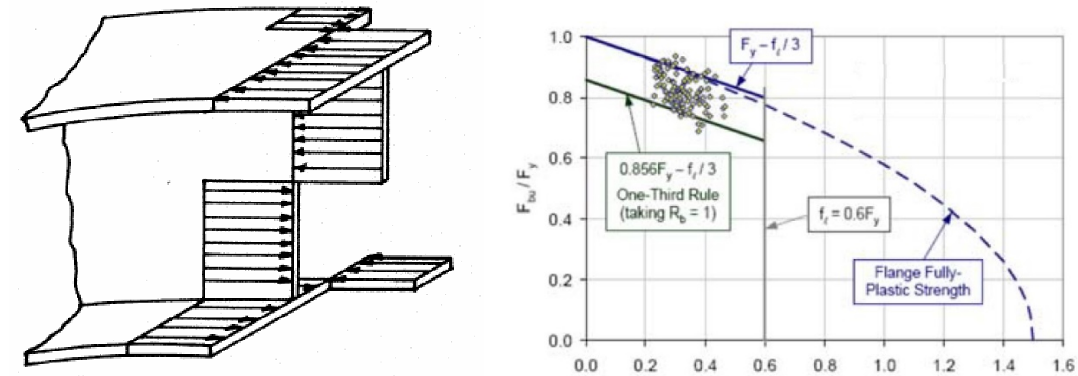


Figure 6.1.4-1 AASHTO Unified Flexural Design Interaction Equations

6.1.4.2 Composite Sections in Positive Flexure

6.1.4.2.1 Nominal Flexural Resistance

At the strength limit state, the compression flange of composite sections in positive flexure is continuously supported by the concrete deck and lateral bending does not need to be considered. For compact sections, the flexural resistance is expressed in terms of moment, while for noncompact sections the flexural resistance is expressed in terms of the elastically computed stress. The compact composite section shall meet the following requirements (AASHTO 6.10.6.2.2):

- Straight bridges
- $F_{yf} \le 70$ ksi
- $\dfrac{D}{t_w} \le 150$ (AASHTO 6.10.2.1.1-1)
- $\dfrac{2D_{cp}}{t_w} \le 3.76\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.6.2.2-1)

where

D_{cp} = the web depth in compression at the plastic moment (in.)

E = modulus of elasticity of steel (ksi)

F_{yc} = specified minimum yield strength of a compression flange (ksi)

Composite sections in positive flexure not satisfying one or more of above four requirements are classified as noncompact sections. The nominal flexural resistances are listed in Table 6.1.4-2.

Table 6.1.4-2 Nominal Flexural Resistance for Composite Sections in Positive Flexure (Strength Limit State)

Section Type	Nominal Flexural Resistance
Compact	$M_n = \min \begin{cases} M_p & \text{for } D_p \le 0.1D_t \\ M_p\left[1-\left(1-\dfrac{M_y}{M_p}\right)\left(\dfrac{D_p/D_t-0.1}{0.32}\right)\right] & \text{for } D_p > 0.1D_t \\ 1.3R_hM_y & \text{for a continous span} \end{cases}$ (AASHTO 6.10.7.1.2-1, 3) and (CA 6.10.7.1.2-2)
Noncompact	Compression flange $F_{nc} = R_b R_h F_{yc}$ (AASHTO 6.10.7.2.2-1) Tension flange $F_{nt} = R_h F_{yt}$ (AASHTO 6.10.7.2.2-2)
Ductility Requirement	For both compact and noncompact sections $D_p \le 0.42D_t$ (AASHTO 6.10.7.3-1)
D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.) D_t = total depth of the composite section (in.) F_{yt} = specified minimum yield strength of a tension flange (ksi) M_p = plastic moment of the composite section (kip-in.) M_y = yield moment of the composite section (kip-in.) R_b = web load-shedding factor	

6.1.4.2.2 Yield Moment

The yield moment M_y for a composite section in positive flexure is defined as the moment which causes the first yielding in one of the steel flanges. M_y is the sum of the moments applied separately to the appropriate sections, i.e., the steel section alone, the short-term composite section, and the long-term composite section. It is based on elastic section properties and can be expressed as:

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(AASHTO D6.2.2-2)}$$

where

M_{D1} = moment due to factored permanent loads applied to the steel section alone

(kip-in.)

M_{D2} = moment due to factored permanent loads such as wearing surface and barriers applied to the long-term composite section (kip-in.)

M_{AD} = additional live load moment to cause yielding in either steel flange applied to the short-term composite section (kip-in.):

$$F_{yf} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \quad \text{(AASHTO D6.2.2-1)}$$

$$M_{AD} = S_{ST}\left[F_{yf} - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}}\right] \quad (6.1.4\text{-}1)$$

where

S_{NC}, S_{ST}, S_{LT} = elastic section modulus for steel section alone, short-term composite, and long-term composite sections, respectively (in.^{3}).

6.1.4.2.3 Plastic Moment

The plastic moment M_p for a composite section is defined as the moment which causes the yielding of the entire steel section and reinforcement and a uniform stress distribution of $0.85f_c^{'}$ in the compression concrete slab. $f_c^{'}$ is minimum specified 28-day compressive strength of concrete. In positive flexure regions the contribution of reinforcement in the concrete slab is small and can be neglected. Table 6.1.4-3 summarizes calculations of M_p.

Table 6.1.4-3 Plastic Moment Calculation

Regions	Case and PNA	Condition and $\bar{Y}$	M_p
Positive Figure 6.1.4-2	I - In Web	$P_t + P_w \ge P_c + P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] +$ $\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t\right]$
	II - In Top Flange	$P_t + P_w + P_c \ge P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$	$M_p = \frac{P_c}{2t_c}\left[\bar{Y}^2 + \left(t_c - \bar{Y}\right)^2\right] +$ $\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t\right]$
	III- In Slab, Below P_{rb}	$P_t + P_w + P_c \ge \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(t_s\right)\left[\frac{P_w + P_c + P_t - P_{rt} - P_{rb}}{P_s}\right]$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
	IV - In Slab, Above P_{rb} Below P_{rt}	$P_t + P_w + P_c + P_{rb} \ge \left(\frac{C_{rt}}{t_s}\right) P_s + P_{rt}$ $\bar{Y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right]$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
	V - In Slab, above P_{rt}	$P_t + P_w + P_c + P_{rb} + P_{rt} < \left(\frac{C_{rt}}{t_s}\right) P_s$ $\bar{Y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t + P_{rt}}{P_s}\right]$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right) +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
Negative Figure 6.1.4-3	I - In Web	$P_c + P_w \ge P_t + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_t d_t + P_c d_c\right]$
	II - In Top Flange	$P_c + P_w + P_t \ge P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{t_t}{2}\right)\left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1\right]$	$M_p = \frac{P_t}{2t_t}\left[\bar{Y}^2 + \left(t_t - \bar{Y}\right)^2\right] +$ $\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_c d_c\right]$

$P_{rt} = F_{yrt} A_{rt}$; $P_s = 0.85 f'_c b_s t_s$; $P_{rb} = F_{yrb} A_{rb}$; $P_c = F_{yc} b_c t_c$; $P_w = F_{yw} D t_w$; $P_t = F_{yt} b_t t_t$;

f'_c = minimum specified 28-day compressive strength of concrete (ksi)

PNA = plastic neutral axis

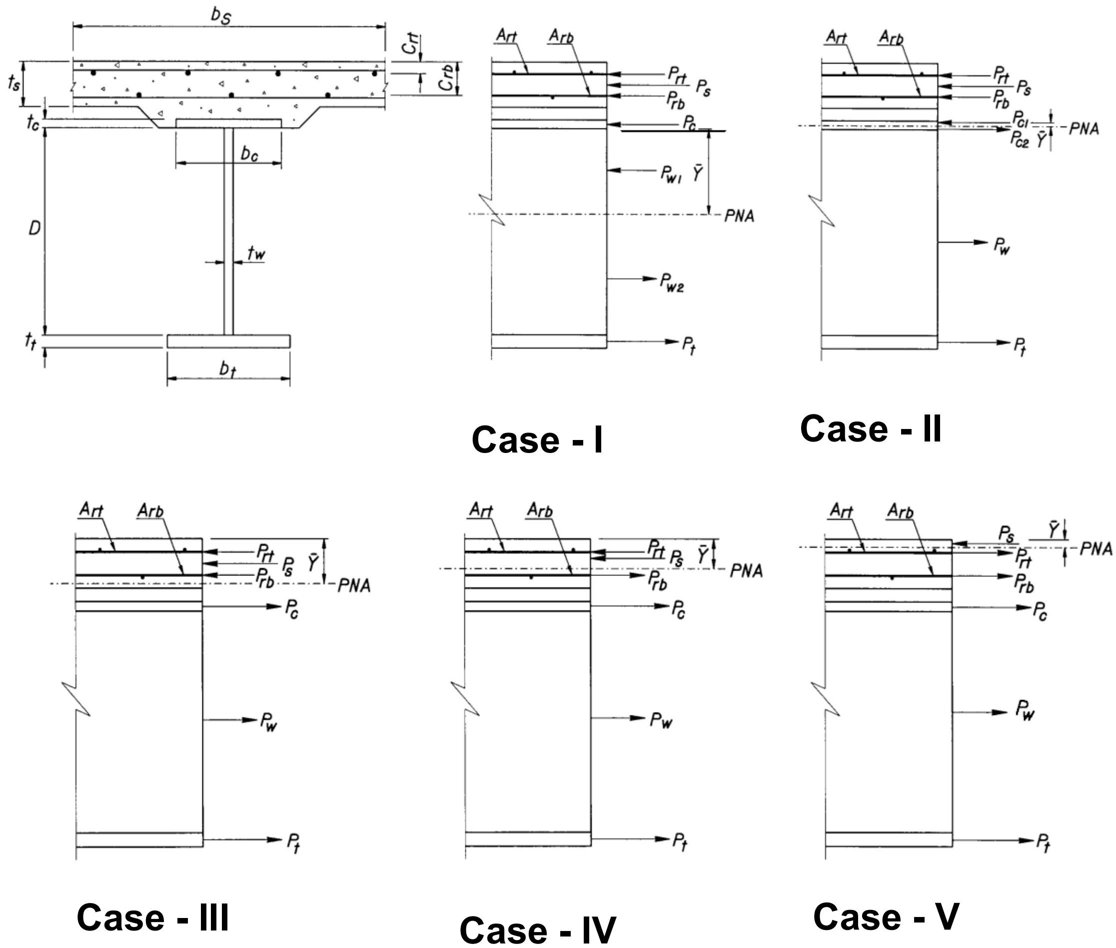
A_{rb}, A_{rt} = reinforcement area of bottom and top layer in concrete deck slab (in.^{2})

F_{yrb}, F_{yrt} = yield strength of reinforcement of bottom and top layers (ksi)

b_c, b_t, b_s = width of compression, tension steel flange, and concrete deck slab (in.)

t_c, t_t, t_w, t_s = thickness of compression, tension steel flange, web, and concrete deck slab (in.)

F_{yt}, F_{yc}, F_{yw} = yield strength of tension flange, compression flange, and web (ksi)

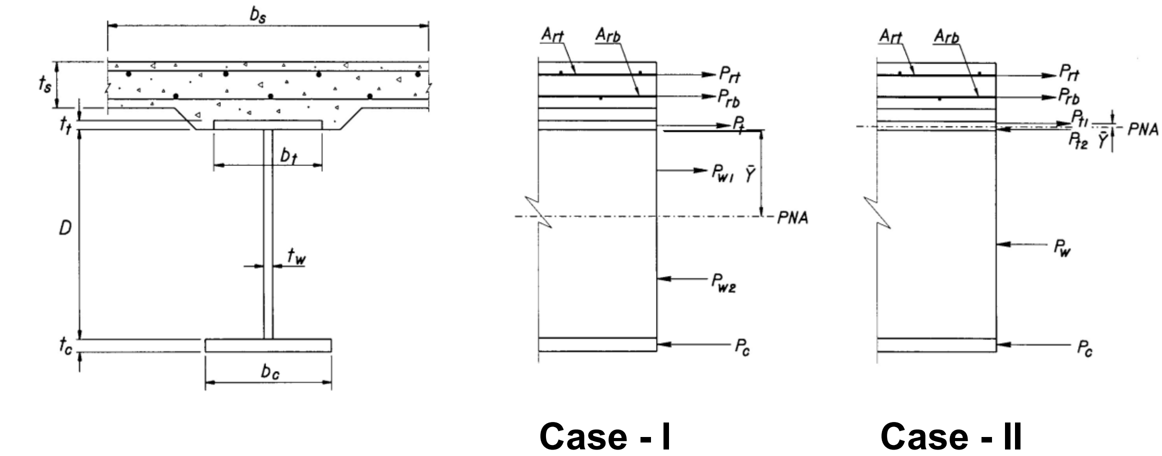


$P_{c1} = \bar{Y} b_c F_{yc}$; $\quad P_{c2} = \left(t_c - \bar{Y}\right) b_c F_{yc}$

$P_{w1} = \bar{Y} t_w F_{yw}$; $\quad P_{w2} = \left(D - \bar{Y}\right) t_w F_{yw}$

$P_{t1} = \bar{Y} b_t F_{yt}$; $\quad P_{t2} = \left(t_t - \bar{Y}\right) b_t F_{yt}$

Figure 6.1.4-2 Plastic Moment Calculation Cases for Positive Flexure



$$P_{w1} = \bar{Y} t_w F_{yw}; \quad P_{w2} = \left(D - \bar{Y}\right) t_w F_{yw}$$

$$P_{t1} = \bar{Y} b_t F_{yt}; \quad P_{t2} = \left(t_t - \bar{Y}\right) t_t F_{yt}$$

Figure 6.1.4-3 Plastic Moment Calculation Cases for Negative Flexure

6.1.4.3 Steel Sections

The flexural resistance of a steel section (i.e., composite sections in negative flexure and noncomposite sections) is governed by three failure modes or limit states: yielding, flange local buckling, and lateral-torsional buckling. The moment capacity depends on the yield strength of steel, the slenderness ratio of the compression flange, λ_f in terms of width-to-thickness ratio ($b_{fc}/2t_{fc}$) for local buckling, and the unbraced length L_b for lateral-torsional buckling. Figure 6.1.4-4 shows dimensions of a typical I-girder. Figures 6.1.4-5 and 6.1.4-6 show graphically the compression flange local and lateral torsional buckling resistances, respectively. Calculations for nominal flexural resistances are summarized in Table 6.1.4-4.

The flexural resistance in term of moments may be determined by AASHTO Appendix A6, and may exceed the yield moment for sections in straight bridges satisfying the following requirements:

- $F_{yf} \leq 70$ ksi
- $\dfrac{2D_c}{t_w} < 5.7\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.6.2.3-1)
- $\dfrac{I_{yc}}{I_{yt}} \geq 0.3$ (AASHTO 6.10.6.2.3-2)

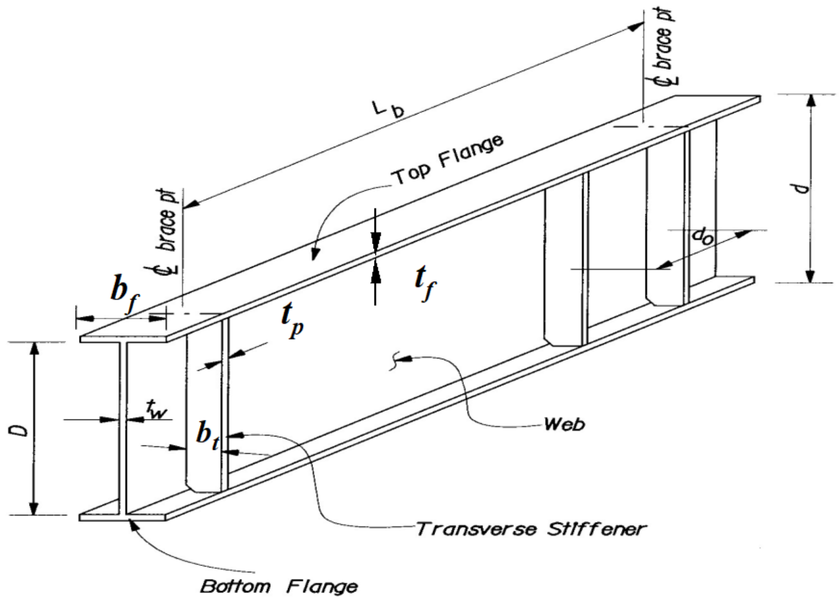


Figure 6.1.4-4 Dimensions of a Typical I-Girder

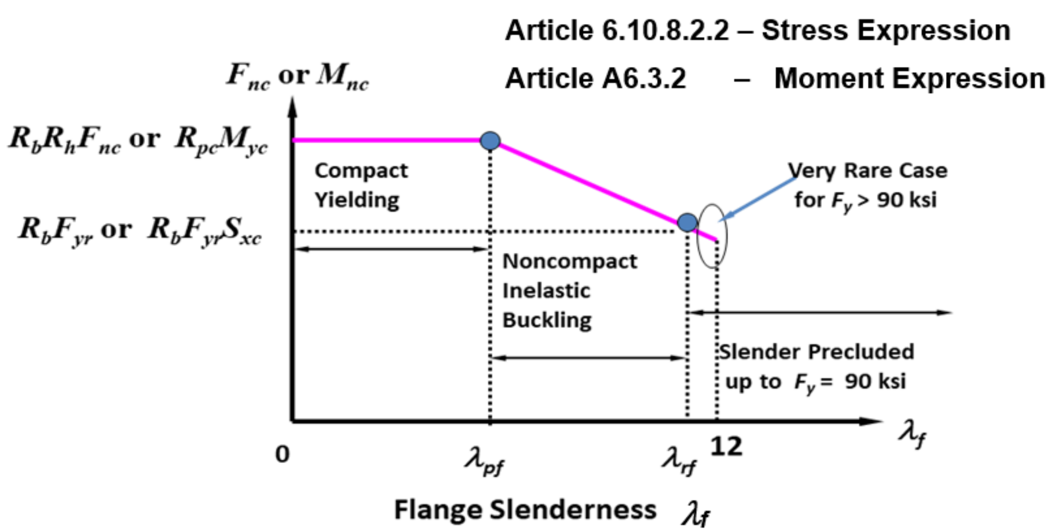


Figure 6.1.4-5 I-Section Compression-Flange Flexural Local-Buckling Resistance

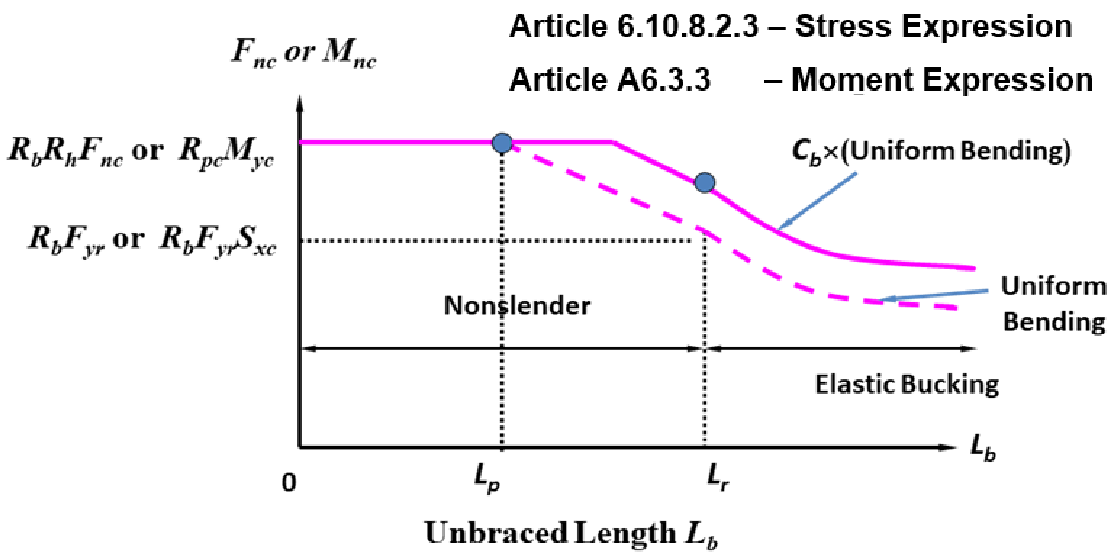


Figure 6.1.4-6 I-Section Compression-Flange Flexural Torsional Resistance

Table 6.1.4-4 Nominal Flexural Resistance for Steel Sections (Composite Sections in Negative Flexure and Noncomposite Sections)

Flange	**Nominal Flexural Resistance**
Compression	$F_{nc} = \text{smaller } [F_{nc(FLB)}, F_{nc(LTB)}]$ (AASHTO 6.10.8.2.1) $F_{nc(FLB)} = \begin{cases} R_b R_h F_{yc} & \text{for } \lambda_f \le \lambda_{pf} \\ \left[1-\left(1-\dfrac{F_{yr}}{R_h F_{yc}}\right)\left(\dfrac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b R_h F_{yc} & \text{for } \lambda_f > \lambda_{pf} \end{cases}$ (AASHTO 6.10.8.2.2-1 &2) $F_{nc(LTB)} = \begin{cases} R_b R_h F_{yc} & \text{for } L_b \le L_p \\ C_b \left[1-\left(1-\dfrac{F_{yr}}{R_h F_{yc}}\right)\left(\dfrac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \le R_b R_h F_{yc} & \text{for } L_b < L_p \le L_r \\ F_{cr} \le R_b R_h F_{yc} & \text{for } L_b > L_r \end{cases}$ (AASHTO 6.10.8.2.3-1, 2 &3)
Tension	$F_{nt} = R_h F_{yt}$ (AASHTO 6.10.8.3-1)

L_b **=** unbraced length of compression flange (in.)

L_p = limiting unbraced length to achieve $R_b R_h F_{yc} = 1.0 r_t \sqrt{E / F_{yc}}$ (AASHTO 6.10.8.2.3-4)

L_r = limiting unbraced length to achieve the onset of nominal yielding $= \pi r_t \sqrt{E / F_{yr}}$ (AASHTO 6.10.8.2.3-5)

λ_f = slenderess ratio for compression flange $= \dfrac{b_{fc}}{2t_{fc}}$ (AASHTO 6.10.8.2.2-3)

λ_{pf} = slenderess ratio for a compact compression flange $= 0.38\sqrt{\dfrac{E}{F_{yc}}}$ (AASHTO 6.10.8.2.2-4)

λ_{rf} = limiting slenderness ratio for a noncompact flange $= 0.56\sqrt{\dfrac{E}{F_{yr}}}$ (AASHTO 6.10.8.2.2-5)

F_{cr} = elastic lateral torsional buckling stress (ksi) $= \dfrac{C_b R_b \pi^2 E}{(L_b / r_t)^2}$ (AASHTO 6.10.8.2.3-8)

F_{yr} = smaller $\{0.7F_{yc}, F_{yw}\} \ge 0.5F_{yc}$ (AASHTO 6.10.8.2.2)

C_b **=** moment gradient modifier

r_t **=** effective radius of gyration for lateral torsional buckling (in.)

Moment gradient modifier C_b, AASHTO Eq. 6.10.8.2.3-7 has been used in *AASHTO LRFD Bridge Design Specifications* since 1994. This equation is only applicable to linearly varying moment diagrams between the braced points – a condition that is rare in bridge girder design. This equation can be easily misinterpreted and misapplied to moment diagrams that are not linear within the unbraced segment. AISC Specification (1993) and Caltrans BDS (2004) have adopted the Eq. (CA-6.10.8.2.3-7) originally developed by Kirby and Nethercot (1979) with slightly modifications. Eq. (CA-6.10.8.2.3-7) provides a more accurate solution for unbraced lengths in which the moment diagram deviates substantially from a straight line, such as the case of a continuous bridge girder with no lateral bracing within the span that is subjected to dead and live loads.

- For cantilevers where the free end is unbraced:

$$C_b = 1.0 \qquad \text{(CA 6.10.8.2.3-6)}$$

- For all other cases:

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \qquad \text{(CA 6.10.8.2.3-7)}$$

where:

M_{max} = absolute value of maximum moment in the unbraced segment (kip-in.)

M_A = absolute value of moment at quarter point of the unbraced segment (kip-in.)

M_B = absolute value of moment at centerline of the unbraced segment (kip-in.)

M_C = absolute value of moment in three-quarter point of the unbraced segment (kip-in.)

6.1.5 SHEAR DESIGN

6.1.5.1 Design Requirements

For I-girder web panels, the following equation shall be satisfied.

$$V_u \le \phi_v V_n \qquad \text{(AASHTO 6.10.9.1-1)}$$

where

V_u = factored shear at the section under consideration (kip)

V_n = nominal shear resistance (kip)

ϕ_v = resistance factor for shear = 1.0.

6.1.5.2 Nominal Shear Resistance

Similar to the flexural resistance, web shear resistance is also dependent on the slenderness ratio in terms of depth-to-thickness ratio (D/t_w).

For a steel girder without transverse stiffeners, shear resistance is provided by the beam action of shearing yield or elastic shear buckling. For end panels of stiffened webs adjacent to simple support, shear resistance is limited to the beam action only.

$$V_n = V_{cr} = CV_p \qquad \text{(AASHTO 6.10.9.2-1)}$$

$$V_p = 0.58F_{yw}Dt_w \qquad \text{(AASHTO 6.10.9.2-2)}$$

$$C = \begin{cases} 1.0 & \text{For } \dfrac{D}{t_w} \le 1.12\sqrt{\dfrac{Ek}{F_{yw}}} \\ \dfrac{1.12}{(D/t_w)}\sqrt{\dfrac{Ek}{F_{yw}}} & \text{For } 1.12\sqrt{\dfrac{Ek}{F_{yw}}} < \dfrac{D}{t_w} \le 1.40\sqrt{\dfrac{Ek}{F_{yw}}} \\ \dfrac{1.57}{(D/t_w)^2}\left(\dfrac{Ek}{F_{yw}}\right) & \text{For } \dfrac{D}{t_w} > 1.40\sqrt{\dfrac{Ek}{F_{yw}}} \end{cases} \qquad \text{(AASHTO 6.10.9.3.2-4,5,6)}$$

$$k = 5 + \frac{5}{(d_o/D)^2} \qquad \text{(AASHTO 6.10.9.3.2-7)}$$

where

V_{cr} = shear-buckling resistance (kip)

V_p = plastic shear force (kip)

C = ratio of the shear-buckling resistance to the shear yield strength

d_o = transverse stiffener spacing (in.)

k = shear-buckling coefficient

For interior web panels with transverse stiffeners, the shear resistance is provided by both the beam and the tension field actions as shown in Figure 6.1.5-1.

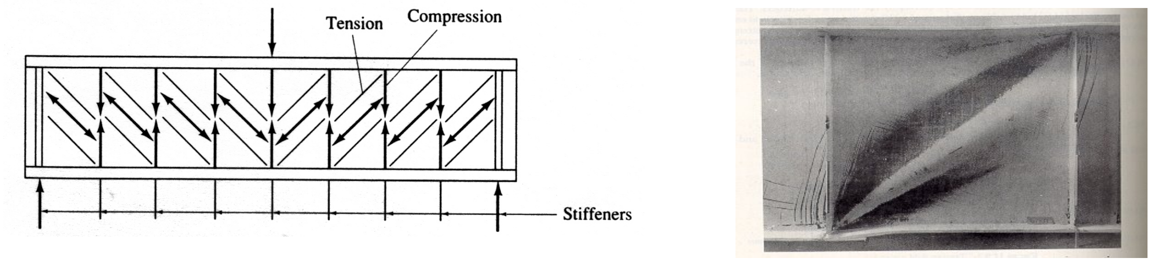


Figure 6.1.5-1 Tension Field Action

For $\frac{2Dt_w}{\left(b_{fc}t_{fc}+b_{ft}t_{ft}\right)} \leq 2.5$ (AASHTO 6.10.9.3.2-1)

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

otherwise

$$V_n = V_p \left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2} + \frac{d_o}{D}} \right] \quad \text{(AASHTO 6.10.9.3.2-8)}$$

where

b_{fc} = full width of a compression flange (in.)

b_{ft} = full width of a tension flange (in.)

t_{fc} = thickness of a compression flange (in.)

t_{ft} = thickness of a tension flange (in.)

t_w = web thickness (in.)

d_o = transverse stiffener spacing (in.)

6.1.5.3 Transverse Stiffeners

Transverse intermediate stiffeners work as anchors for the tension field force so that post-buckling shear resistance can be developed. It should be noted that elastic web shear buckling cannot be prevented by transverse stiffeners. Transverse stiffeners are designed to (1) meet the slenderness requirement of projecting elements to prevent local buckling, (2) provide stiffness to allow the web to develop its post-buckling capacity, and (3) have strength to resist the vertical components of the diagonal stresses in the web.

6.1.5.4 Shear Connectors

To ensure a full composite action, shear connectors must be provided at the interface between the concrete slab and the steel to resist interface shear. Shear connectors are usually provided throughout the length of the bridge. If the longitudinal reinforcement in the deck slab is not considered in the composite section, shear connectors are not necessary in negative flexure regions. If the longitudinal reinforcement is included, either additional connectors can be placed in the region of dead load contra-flexure points or they can be continued through the negative flexure region at maximum spacing. The fatigue and strength limit states must be considered in the shear connector design.

6.1.6 COMPRESSION DESIGN

6.1.6.1 Design Requirements

For axially loaded compression members, the following equation shall be satisfied:

$$P_u \leq P_r = \phi_c P_n \tag{6.1.6-1}$$

where

P_u = factored axial compression load (kip)

P_r = factored axial compressive resistance (kip)

P_n = nominal compressive resistance (kip)

ϕ_c = resistance factor for compression = 0.9

For members subjected to combined axial compression and flexure, the following interaction equation shall be satisfied:

For $\dfrac{P_u}{P_r} < 0.2$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \tag{AASHTO 6.9.2.2-1}$$

For $\frac{P_u}{P_r} \geq 0.2$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{(AASHTO 6.9.2.2-2)}$$

where

M_{ux}, M_{uy} = factored flexural moments (second-order moments) about the *x*-axis and *y*-axis, respectively (kip-in.)

M_{rx}, M_{ry} = factored flexural resistance about the *x*-axis and *y*-axis, respectively (kip-in.).

Compression members shall also meet the slenderness ratio requirements, $Kl/r \leq 120$for primary members, and $Kl/r \leq 140$ for secondary members. K is effective length factor; l is unbraced length; and r is radius of gyration (in.)

6.1.6.2 Axial Compressive Resistance

For noncomposite steel compression members with non-slender elements, nominal axial compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, and flexural-torsional buckling. Design equations specified in the AASHTO (2017) are identical to the column design equations AISC (2016).

For $\frac{P_e}{P_o} \geq 0.44$

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad \text{(AASHTO 6.9.4.1.1-1)}$$

For $\frac{P_e}{P_o} < 0.44$

$$P_n = 0.877 P_e \qquad \text{(AASHTO 6.9.4.1.1-2)}$$

where

P_o = nominal yield resistance = $F_y A_g$ (kip)

A_g = gross cross section area (in.^{2})

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kip)

Elastic flexural buckling resistance is as follows:

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)_{eff}^2} A_g \qquad \text{(AASHTO 6.9.4.1.2-1)}$$

where

K = effective length factor in the plane of buckling

l = unbraced length in the plan of buckling (in.)

r_s = radius of gyration about the axis normal to the plane of buckling (in.)

For compression members with slender element cross-sections, the nominal compressive resistance, P_n, shall be based on the effective area of the cross section, A_{eff}, as defined in Article 6.9.4.2.2.

6.1.7 TENSION DESIGN

6.1.7.1 Design Requirements

For axially loaded tension members, the following equation shall be satisfied:

$$P_u \le P_r \qquad (6.1.7\text{-}1)$$

where

P_u = factored axial tension load (kip)

P_r = factored axial tensile resistance (kip)

For members subjected to combined axial tension and flexure, the following interaction equation shall be satisfied:

For $\frac{P_u}{P_r} < 0.2$

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \qquad \text{(AASHTO 6.8.2.3-1)}$$

For $\frac{P_u}{P_r} \geq 0.2$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \qquad \text{(AASHTO 6.8.2.3-2)}$$

where

M_{ux}, M_{uy} = factored flexural moments about the *x*-axis and *y*-axis, respectively (kip-in.)

M_{rx}, M_{ry} = factored flexural resistance about the *x*-axis and *y*-axis, respectively (kip-in.)

Tension members shall also meet the slenderness ratio requirements, $l/r \leq 140$ for primary members subjected to stress reversal, $l/r \leq 200$ for primary members not subjected to stress reversal, and $l/r \leq 240$ for secondary members.

6.1.7.2 Axial Tensile Resistance

For steel tension members, axial tensile resistance shall be taken as the smaller of yielding on the gross section and fracture on the net section as follows:

Yielding in gross section:

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \qquad \text{(AASHTO 6.8.2.1-1)}$$

Fracture in net section:

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \qquad \text{(AASHTO 6.8.2.1-2)}$$

where

P_{ny} = nominal tensile resistance for yielding in gross section (kip)

P_{nu} = nominal tensile resistance for fracture in net section (kip)

A_n = net cross section area (in.2)

F_u = specified minimum tensile strength (ksi)

R_p = specified minimum tensile strength reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

U = reduction factor to account for shear lag

ϕ_y = resistance factor for yielding of tension member = 0.95

ϕ_u = resistance factor for fracture of tension member = 0.8.

6.1.8 FATIGUE DESIGN

There are two types of fatigue: load-induced and distortion-induced. The basic fatigue design requirement for load-induced fatigue is limiting live load stress range to fatigue resistance for each component and connection detail. Distortion-induced fatigue usually occurs at the web near a flange due to improper detailing. The design requirement for distortion-induced fatigue is to follow proper detailing practice to provide sufficient load paths. For load-induced fatigue consideration, the most common types of components and details in a typical I-girder are listed in Table 6.1.8-1 (AASHTO Table 6.6.1.2.3-1).

Table 6.1.8-1 Detail Categories for Load-Induced Fatigue for Plate Girders

Description		Category (AASHTO Table 6.6.1.2.3-1)
1	2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned high-strength bolts (bolt gusset to flange)	**B**
2	4.1 Base metal at toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	**C′**
3	5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices,	B
4	9.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding	**C**

Nominal fatigue resistance, $(\Delta F)_n$, as shown in Figure 6.1.8-1 is calculated as follows:

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

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

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

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(AASHTO 6.6.1.2.5-2)}$$

in which:

$$N = (365)(75)n(ADTT)_{ST} \qquad \text{(AASHTO 6.6.1.2.5-3)}$$

$$N_{TH} = \frac{A}{\left[\frac{0.8}{1.75}(\Delta F)_{TH}\right]} \qquad (6.1.8\text{-}1)$$

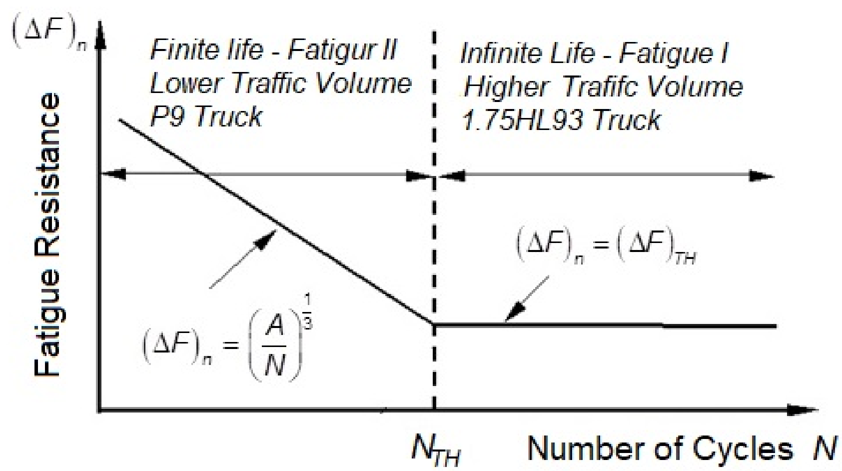


Figure 6.1.8-1 Fatigue Resistance

where A is a constant depending on detail category as specified in AASHTO Table 6.6.1.2.5-1, and $(\Delta F)_{TH}$ is the constant-amplitude fatigue threshold taken from AASHTO Table 6.6.1.2.5-3. N_{TH} is minimum number of stress cycles corresponding to constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, as calculated by Eq. (6.1.8-1).

$$ADTT_{SL} = p\,(ADTT) \qquad \text{(AASHTO 3.6.1.4.2-1)}$$

where

- p = fraction of truck traffic in a single lane (AASHTO Table 3.6.1.4.2-1) = 0.8 for three or more lanes traffic,
- n = number of stress-range cycles per truck passage = 1.0 for simple span girders (CA Table 6.6.1.2.5-2).
- $ADTT$ = number of trucks per day in one direction averaged over the design life and is specified in CA 3.6.1.4.2.

For typical highway bridges, we have:

Fatigue I: $ADTT$ = 2500, $N = (365)(75)(1.0)(0.8)(2500) = 0.5475(10)^8 > N_{TH}$

Fatigue II: $ADTT$ = 20, $N = (365)(75)(1.0)(0.8)(20) = 438{,}000 < N_{TH}$

The nominal fatigue resistances for typical Detail Categories in an I-girder are summarized in Table 6.1.8-2.

Table 6.1.8-2 Nominal Fatigue Resistance

Detail Category	**Constant –*A*** ($\times\ 10^8$) (ksi^3)	Fatigue II $(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}}$ (ksi)	Fatigue I $(\Delta F)_n = (\Delta F)_{TH}$ (ksi)
B	120.0	30.15	16.0
C	44.0	21.58	10.0
C′	44.0	21.58	12.0

6.1.9 SERVICEABILITY STATES

The service limit state design is intended to control the elastic and permanent deformations, which would affect riding ability. For steel girder, vehicular live load deflection may be limited to *L*/800 by AASHTO 2.5.2.6.

To prevent the permanent deformation due to expected severe traffic loadings, AASHTO 6.10.4 requires that for the Service II load combination, flange stresses in positive and negative bending without consideration of flange lateral bending shall meet the following requirements. They correspond to the overload check in the 2002 AASHTO *Standard Specifications* (AASHTO, 2002).

For the top steel flange of composite sections

$$f_f \le 0.95 R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-1)}$$

For the bottom steel flange of composite sections

$$f_f + \frac{f_l}{2} \le 0.95 R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-2)}$$

For both steel flanges of noncomposite sections

$$f_f + \frac{f_l}{2} \le 0.8\ R_h F_{yf} \qquad \text{(AASHTO 6.10.4.2.2-3)}$$

For compact composite sections in positive flexure in shored construction, the longitudinal compressive stress in the concrete deck shall not exceed $0.6 f_c'$.

Except for composite sections in positive flexure satisfying $D / t_w \le 150$ without longitudinal stiffeners, all sections shall satisfy

$$f_c \le F_{crw} \quad \text{(AASHTO 6.10.4.2.2-4)}$$

6.1.10 CONSTRUCTIBILITY

An I-girder bridge constructed in unshored conditions shall be investigated for strength and stability for all critical construction stages, using the appropriate strength load combination discussed in Chapter 3. All calculations shall be based on the non-composite steel section only.

AASHTO Article 6.10.3 requires checking the following requirements:

- *Compression Flange*

For discretely braced flange

$$f_{bu} + f_l \le \phi_f R_h F_{yc} \quad \text{(AASHTO 6.10.3.2.1-1)}$$

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc} \quad \text{(AASHTO 6.10.3.2.1-2)}$$

$$f_{bu} \le \phi_f F_{crw} \quad \text{(AASHTO 6.10.3.2.1-3)}$$

where

f_{bu} = flange stress calculated without consideration of the flange lateral bending (ksi)

F_{crw} = nominal bending stress determined by AASHTO 6.10.1.9.1-1 (ksi).

For sections with compact and noncompact webs, AASHTO Equation 6.10.3.2.1-3 shall not be checked. For sections with slender webs, AASHTO Equation 6.10.3.2.1-1 shall not be checked when f_l is equal to zero.

For continuously braced flanges

$$f_{bu} \le \varphi_f R_h F_{yc} \quad \text{(AASHTO 6.10.3.2.3-1)}$$

- *Tension Flange*

For discretely braced flange

$$f_{bu} + f_l \le \varphi_f R_h F_{yc} \quad \text{(AASHTO 6.10.3.2.1-1)}$$

For continuously braced flanges

$$f_{bu} \leq \varphi_f R_h F_{yt} \qquad \text{(AASHTO 6.10.3.2.3-1)}$$

- *Web*

$$V_u \leq \varphi_v V_{cr} \qquad \text{(AASHTO 6.10.3.3-1)}$$

Where

V_u = shear due to sum of factored dead loads and factored construction load applied to the non-composite section (AASHTO 6.10.3.3) (kip)

V_{cr} = shear buckling resistance (AASHTO 6.10.9.3.3-1) (kip)

NOTATION

A = fatigue detail category constant
A_{eff} = effective area of the cross section (in.^{2})
$ADTT$ = average daily truck traffic in one direction over the design life
$ADTT_{SL}$ = single lane $ADTT$ life
A_g = gross cross section area (in.^{2})
A_n = net cross section area (in.^{2})
A_{rb} = reinforcement area of bottom layer in concrete deck slab (in.^{2})
A_{rt} = reinforcement area of top layer in concrete deck slab (in.^{2})
b_c = width of compression steel flange (in.)
b_f = full width of the flange (in.)
b_{fc} = full width of a compression flange (in.)
b_{ft} = full width of a tension flange (in.)
b_s = width of concrete deck slab (in.)
b_t = width of tension steel flange (in.)
C = ratio of the shear-buckling resistance to the shear yield strength
C_b = moment gradient modifier
C_{rb} = distance from the top of the concrete deck to the centerline of the bottom layer of longitudinal concrete deck reinforcement (in.)
C_{rt} = distance from the top of the concrete deck to the centerline of the top layer of longitudinal concrete deck reinforcement (in.)
D = web depth (in.)
D_{cp} = web depth in compression at the plastic moment (in.)
D_p = distance from the top of the concrete deck to the neutral axis of the composite sections at the plastic moment (in.)
D_t = total depth of the composite section (in.)
d = total depth of the steel section (in.)
d_o = transverse stiffener spacing (in.)
E = modulus of elasticity of steel (ksi)
F_{cr} = elastic lateral torisonal buckling stress (ksi)
F_{crw} = nominal bend-buckling resistance of webs (ksi)
F_{nc} = nominal flexural resistance of the compression flange (ksi)
F_{nt} = nominal flexural resistance of the tension flange (ksi)
F_u = specified minimum tensile strength (ksi)
F_{yc} = specified minimum yield strength of a compression flange (ksi)
F_{yf} = specified minimum yield strength of a flange (ksi)
F_{yr} = compression-flange stress at the onset of nominal yielding including residual stress effects, taken as the smaller of $0.7F_{yc}$ and F_{yw} but not less than $0.5F_{yc}$ (ksi)
F_{yrb} = specified minimum yield strength of reinforcement of bottom layers (ksi)
F_{yrt} = specified minimum yield strength of reinforcement of top layers (ksi)
F_{yt} = specified minimum yield strength of a tension flange (ksi)
F_{yw} = specified minimum yield strength of a web (ksi)
f_{bu} = flange stress calculated without consideration of the flange lateral

bending (ksi)

f_c = compression-flange stress at the section under consideration due to the Service II loads calculated without consideration of flange lateral bending (ksi)

f_f = flange stresses without considering flange lateral bending (ksi)

f_l = flange lateral bending stress (ksi)

f_c' = minimum specified 28-day compressive strength of concrete (ksi)

L = span length (ft)

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.4)

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.4)

k = shear-buckling coefficient

K = effective length factor of a compression member

L = span length (ft)

L_b = unbraced length of compression flange (in.)

L_p = limiting unbraced length to achieve $R_bR_hF_{yc}$ (in.)

L_r = limiting unbraced length to onset of nominal yielding (in.)

l = unbraced length of member (in.)

M_A = absolute value of moment at quarter point of the unbraced segment (kip-in.)

M_{AD} = additional live load moment to cause yielding in either steel flange applied to the short-term composite section (kip-in.)

M_B = absolute value of moment at centerline of the unbraced segment (kip-in.)

M_C = absolute value of moment at three-quarter point of the unbraced segment (kip-in.)

M_{D1} = moment due to factored permanent loads applied to the steel section alone (kip-in.)

M_{D2} = moment due to factored permanent loads such as wearing surface and barriers applied to the long-term composite section (kip-in.)

M_{max} = absolute value of maximum moment in the unbraced segment (kip-in.)

M_n = nominal flexural resistance of the section (kip-in.)

M_{nc} = nominal flexural resistance based on the compression flange (kip-in.)

M_p = plastic moment (kip-in.)

M_{rx}, M_{ry} = factored flexural resistance about the x-axis and y-axis, respectively (kip-in.)

M_u = bending moment about the major axis of the cross section (kip-in.)

M_{ux}, M_{uy} = factored flexural moments about the x-axis and y-axis, respectively (kip-in.)

M_y = yield moment (kip-in.)

M_{yc} = yield moment with respect to the compression flange (kip-in.)

N = number of cycles of stress ranges

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

n = number of stress-range cycles per truck passage

P_c = plastic force in the compression flange used to compute the plastic moment (kip)
P_{c1} = plastic compression force in the top portion of the compression flange used to compute the plastic moment (kip)
P_{c2} = plastic tension force in bottom portion of the compression flange used to compute the plastic moment (kip)
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kip)
P_o = nominal yield resistance = $F_y A_g$ (kip)
PNA = plastic neutral axis
P_n = nominal compressive resistance (kip)
P_{ny} = nominal tensile resistance for yielding in gross section (kip)
P_{nu} = nominal tensile resistance for fracture in net section (kip)
P_r = factored axial resistance (kip)
P_{rb} = plastic force in the bottom layer of longitudinal deck reinforcement in the compression flange used to compute the plastic moment (kip)
P_{rt} = plastic force in the top layer of longitudinal deck reinforcement in the compression flange used to compute the plastic moment (kip)
P_s = plastic compressive force in the concrete deck used to compute the plastic moment (kip)
P_t = plastic force in the tension flange used to compute the plastic moment (kip)
P_{t1} = plastic tension force in the top portion of the tension flange used to compute the plastic moment (kip)
P_{t2} = plastic compression force in bottom portion of the tension flange used to compute the plastic moment (kip)
P_u = factored axial load (kip)
P_w = plastic force in the web used to compute the plastic moment (kip)
P_{w1} = plastic force in the compression web used to compute the plastic moment (kip)
P_{w2} = plastic force in the tension web used to compute the plastic moment (kip)
p = fraction of truck traffic in a single lane
R_h = hybrid factor
R_b = web load-shedding factor
R_p = specified minimum tensile strength reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size
R_{pc} = web plastification factor for the compression flange
r = radius of gyration (in.)
r_s = radius of gyration about the axis normal to the plane of buckling (in.)
r_t = effective radius of gyration for lateral torsional buckling (in.)
S_{LT} = elastic section modulus for long-term composite sections (in.^{3})
S_{NC} = elastic section modulus for steel section alone (in.^{3})
S_{ST} = elastic section modulus for short-term composite section (in.^{3})
S_{xt} = elastic section modulus about the major axis of the section to the

		tension flange taken as M_{yt}/F_{yt} (in.^{3})
S_{xc}	=	elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})
t_c	=	thickness of compression steel flange (in.)
t_f	=	thickness of the flange (in.)
t_{fc}	=	thickness of a compression flange (in.)
t_{ft}	=	thickness of a tension flange (in.)
t_t	=	thickness of tension steel flange (in.)
t_w	=	thickness of web (in.)
t_s	=	thickness of concrete deck slab (in.)
U	=	reduction factor to account for shear lag
V_{cr}	=	shear-buckling resistance (kip)
V_n	=	nominal shear resistance (kip)
V_p	=	plastic shear force (kip)
V_u	=	factored shear (kip)
$\bar{Y}$	=	distance from the plastic neutral axis to the top of the element where the plastic neutral axis is located (in.)
λ_f	=	slenderness ratio for compression flange = $b_{fc}/2t_{fc}$
λ_{pf}	=	limiting slenderness ratio for a compact flange
λ_{rf}	=	limiting slenderness ratio for a noncompact flange
$(\Delta F)_{TH}$	=	constant-amplitude fatigue threshold (ksi)
$(\Delta F)_n$	=	fatigue resistance (ksi)
ϕ_c	=	resistance factor for axial compression = 0.9
ϕ_f	=	resistance factor for flexure = 1.0
ϕ_u	=	resistance factor for tension, fracture in net section = 0.8
ϕ_v	=	resistance factor for shear = 1.0
ϕ_y	=	resistance factor for tension, yielding in gross section = 0.95

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