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12

Steel-Concrete Composite I-Girder Bridges

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12.1 Introduction

An I-section is the simplest and most effective solid section of resisting bending and shear. In this chapter straight, steel–concrete composite I-girder bridges are discussed (Figure 12.1). Materials and components of I-section girders are described. Design considerations for flexural, shear, fatigue, stiffeners, shear connectors, diaphragms and cross frames, and lateral bracing with examples are presented. For a more detailed discussion, reference may be made to recent texts by Xanthakos [1], Baker and Puckett [2], and Taly [3].

12.2 Structural Materials

Four types of structural steels (structural carbon steel, high-strength low-alloy steel, heat-treated low-alloy steel, and high-strength heat-treated alloy steel) are commonly used for bridge structures. Designs are based on minimum properties such as those shown in Table 12.1. ASTM material property standards differ from AASHTO in notch toughness and weldability requirements. Steel meeting the AASHTO-M requirements is prequalified for use in welded bridges.

Concrete with 28-day compressive strength f_c' = 16 to 41 MPa is commonly used in concrete slab construction. The transformed area of concrete is used to calculate the composite section properties. The short-term modular ratio n is used for transient loads and long-term modular ratio

FIGURE 12.1 Steel–concrete composite girder bridge (I-880 Replacement, Oakland, California)

TABLE 12.1 Minimum Mechanic Properties of Structural Steel

Material	Structural Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High Yield Strength Quenched and TemperedLow-Alloy Steel	
AASHTO designation	M270 Grade 250	M270 Grade 345	M270 Grade 345W	M270 Grade 485W	M270 Grades 690/690W	
ASTM designation	A709M Grade 250	A709M Grade 345	A709M Grade 345W	A709M Grade 485W	M709M Grades 690/690W	
Thickness of plate (mm)	Up to 100 included				Up to 65 included	Over 65–100 included
Shapes	All Groups			Not Applicable		
F_u (MPa)	400	450	485	620	760	690
F_y (MPa)	250	345	485	485	690	620

F_y = minimum specified yield strength or minimum specified yield stress; F_u = minimum tensile strength; E = modulus of elasticity of steel (200,000 MPa).

Source: American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications, Washington, D.C., 1994. With permission.

$3n$ for permanent loads. For normal-weight concrete the short-term ratio of modulus of elasticity of steel to that of concrete are recommended by AASHTO-LRFD [4]:

$$n = \begin{cases} 10 & \text{for } 16 \le f_c' < 20 \text{ MPa} \\ 9 & \text{for } 20 \le f_c' < 25 \text{ MPa} \\ 8 & \text{for } 25 \le f_c' < 32 \text{ MPa} \\ 7 & \text{for } 32 \le f_c' < 41 \text{ MPa} \\ 6 & \text{for } f_c' \le 41 \text{ MPa} \end{cases} \tag{12.1}$$

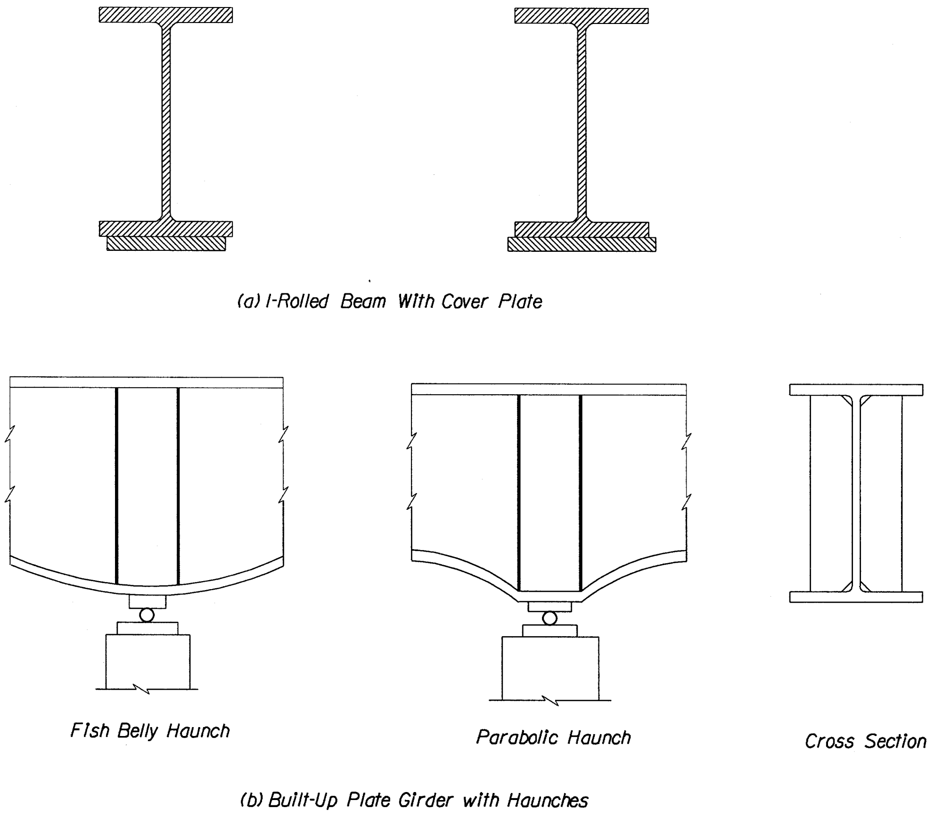


FIGURE 12.2 Typical sections.

12.3 Structural Components

12.3.1 Classification of Sections

I-sectional shapes can be classified in three categories based on different fabrication processes or their structural behavior as discussed below:

1. A steel I-section may be a *rolled* section (*beam*, Figure 12.2a) with or without cover plates, or a *built-up* section (*plate girder*, Figure 12.2b) with or without haunches consisting of top and bottom flange plates welded to a web plate. Rolled steel I-beams are applicable to shorter spans (less than 30 m) and plate girders to longer span bridges (about 30 to 90 m). A plate girder can be considered as a deep beam. The most distinguishing feature of a plate girder is the use of the transverse stiffeners that provide tension-field action increasing the postbuckling shear strength. The plate girder may also require longitudinal stiffeners to develop inelastic flexural buckling strength.
2. I-sections can be classified as *composite* or *noncomposite*. A steel section that acts with the concrete deck to resist flexure is called a composite section (Figure 12.3a). A steel section disconnected from the concrete deck is noncomposite (Figure 12.3b). Since composite sections most effectively use the properties of steel and concrete, they are often the best choice. Steel–concrete composite girder bridges are recommended by AASHTO-LRFD [4] whereas noncomposite members are not and are less frequently used in the United States.

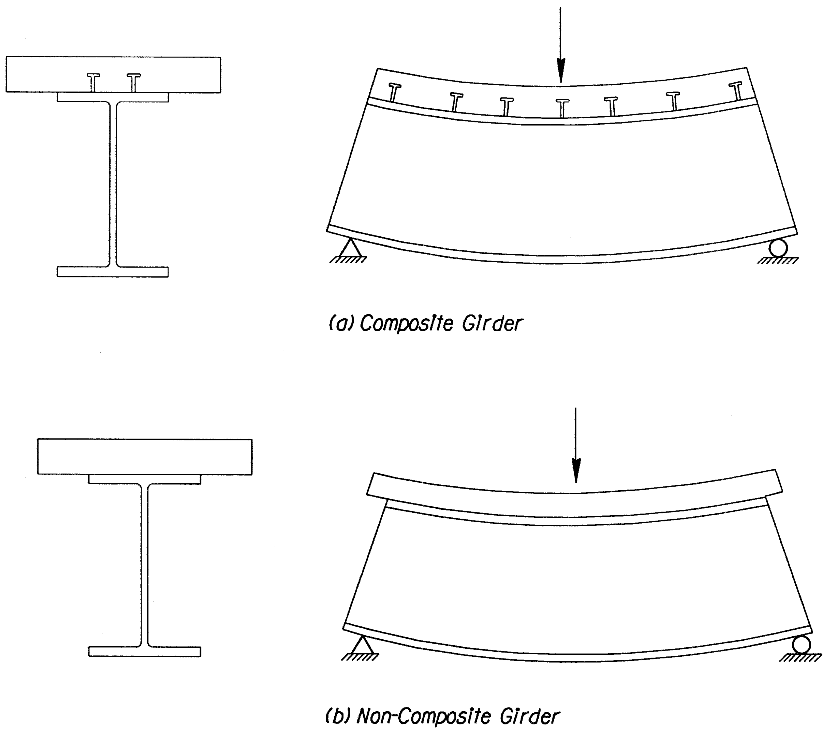


FIGURE 12.3 Composite and noncomposite section.

3. Steel sections can also be classified as *compact, noncompact,* and *slender* element sections [4-6]. A qualified compact section can develop a full plastic stress distribution and possess a inelastic rotation capacity of approximately three times the elastic rotation before the onset of local buckling. Noncompact sections develop the yield stress in extreme compression fiber before buckling locally, but will not resist inelastic local buckling at the strain level required for a fully plastic stress distribution. Slender element sections buckle elastically before the yield stress is achieved.

12.3.2 Selection of Structural Sections

Figure 12.4 shows a typical portion of a composite I-girder bridge consisting of a concrete deck and built-up plate girder I-section with stiffeners and cross frames. The first step in the structural design of an I-girder bridge is to select an I-rolled shape or to size initially the web and flanges of a plate girder. This section presents the basic principles of selecting I-rolled shapes and sizing the dimensions of a plate girder.

The ratio of overall depth (steel section plus concrete slab) to the effective span length is usually about 1:25 and the ratio of depth of steel girder only to the effective span length is about 1:30. I-rolled shapes are standardized and can be selected from a manual such as the AISC-LRFD [7]. It should be noted that the web of a rolled section always meets compactness requirements while the flanges may not. To increase the flexural strength of a rolled section, it is common to add cover plates to the flanges. The I-rolled beams are usually used for simple-span length up to 30 m for highway bridges and 25 m for railway bridges. Plate girder sections provide engineers freedom and flexibility to proportion the flanges and web plates efficiently. Plate girders must have sufficient

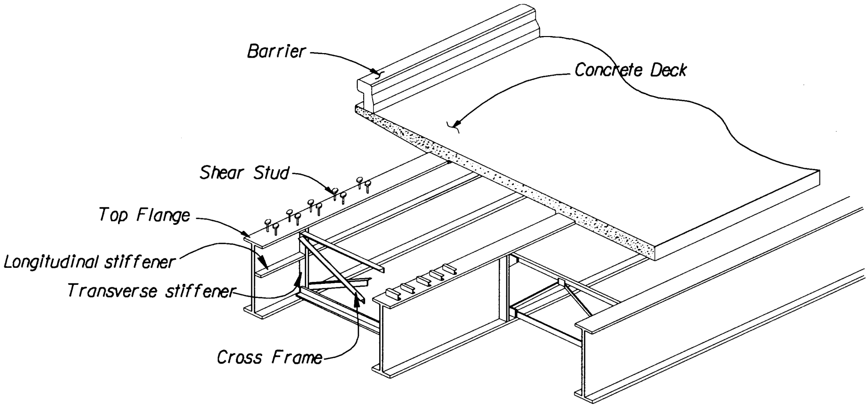


FIGURE 12.4 Typical components of composite I-girder bridge.

flexural and shear strength and stiffness. A practical choice of flange and web plates should not result in any unusual fabrication difficulties. An efficient girder is one that meets these requirements with the minimum weight. An economical one minimizes construction costs and may or may not correspond to the lowest weight alternative [8].

- *Webs*: The web mainly provides shear strength for the girder. The *web height* is commonly taken as $1/18$ to $1/20$ of the girder span length for highway bridges and slightly less for railway bridges. Since the web contributes little to the bending resistance, its thickness (t) should be as small as local buckling tolerance allows. Transverse stiffeners increase shear resistance by providing tension field action and are usually placed near the supports and large concentrated loads. Longitudinal stiffeners increase flexure resistance of the web by controlling lateral web deflection and preventing the web bending buckling. They are, therefore, attached to the compression side. It is usually recommended that sufficient web thickness be used to eliminate the need for longitudinal stiffeners as they can create difficulty in fabrication. Bearing stiffeners are also required at the bearing supports and concentrated load locations and are designed as compression members.
- *Flanges*: The flanges provide bending strength. The width and thickness are usually determined by choosing the area of the flanges within the limits of the width-to-thickness ratio, b/t, and the requirement as specified in the design specifications to prevent local buckling. Lateral bracing of the compression flanges is usually needed to prevent lateral torsional buckling during various load stages.
- *Hybrid Sections*: The hybrid section consisting of flanges with a higher yield strength than that of the web may be used to save materials; this is becoming more promoted because of the new high-strength steels.
- *Variable Sections*: Variable cross sections may be used to save material where the bending moment is smaller and/or larger near the end of a span (see Figure 12.2b). However, the manpower required for welding and fabrication may be increased. The cost of manpower and material must be balanced to achieve the design objectives. The designer should consult local fabricators to determine common practices in the construction of a plate girder.

Highway bridges in the United States are designed to meet the requirements under various limit states specified by AASHTO-LRFD [4,5] such as strength, fatigue and fracture, service, and extreme events (see Chapter 5). Constructibility must be considered. The following sections summarize basic concepts and AASHTO-LRFD [4,5] requirements for composite I-girder bridges.

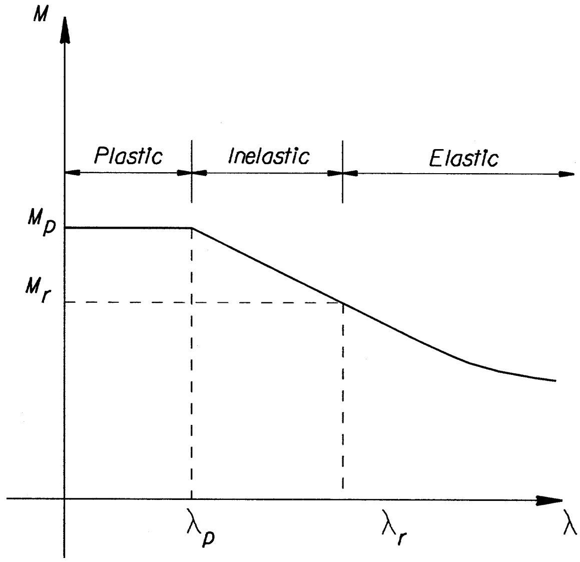


FIGURE 12.5 Three-range design format for steel flexural members.

12.4 Flexural Design

12.4.1 Basic Concept

The flexural resistance of a steel beam/girder is controlled by four failure modes or limit states: yielding, flange local buckling, web local buckling, and lateral-torsional buckling [9]. The moment capacity depends on the yield strength of steel (F_y), the slenderness ratio λ in terms of width-to-thickness ratio (b/t or h/t_w) for local buckling and unbraced length to the radius of gyration about strong axis ratio (L_b/r_y) for lateral-torsional buckling. As a general design concept for steel structural components, a three-range design format (Figure 12.5): plastic yielding, inelastic buckling, and elastic buckling are generally followed. In other words, when slenderness ratio λ is less than λ_p, a section is referred to as compact, plastic moment capacity can be developed; when $\lambda_p < \lambda < \lambda_r$, a section is referred to as noncompact, moment capacity less than M_p but larger than yield moment M_y can be developed; and when $\lambda > \lambda_r$, a section or member is referred to as slender and elastic buckling failure mode will govern. Figure 12.6 shows the dimensions of a typical I-girder. Tables 12.2 and 12.3 list the AASHTO-LRFD [4,5] design formulas for determination of flexural resistance in positive and negative regions.

12.4.2 Yield Moment

The yield moment M_y for a composite section is defined as the moment that causes the first yielding in one of the steel flanges. M_y is the sum of the moments applied separately to the steel section only, the short-term composite section, and the long-term composite section. It is based on elastic section properties and can be expressed as

$$M_y = M_{D1} + M_{D2} + M_{AD} \tag{12.6}$$

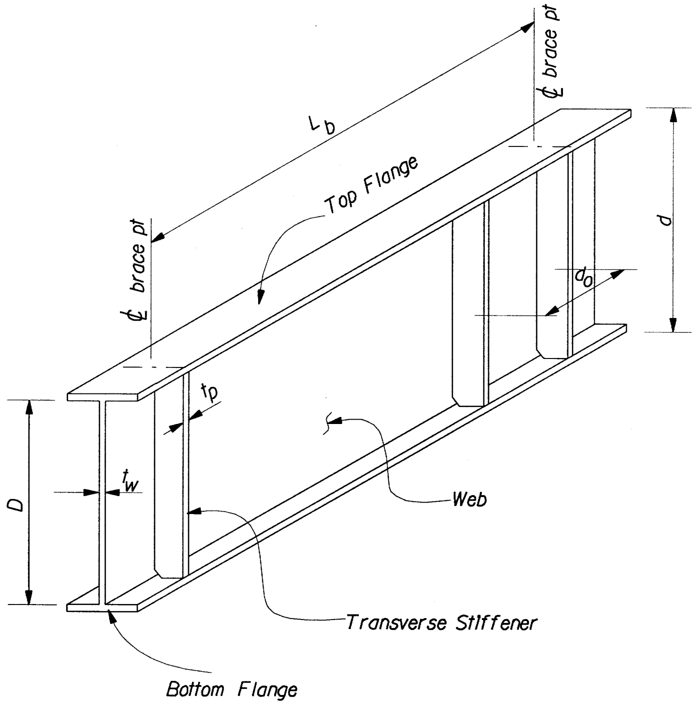


FIGURE 12.6 Typical girder dimensions.

where M_{D1} is moment due to factored permanent loads on steel section; M_{D2} is moment due to factored permanent loads such as wearing surface and barriers on long-term composite section; M_{AD} is additional live-load moment to cause yielding in either steel flange and can be obtained from the following equation:

$$M_{AD} = S_n \left[F_y - \frac{M_{D1}}{S_s} - \frac{M_{D2}}{S_{3n}} \right] \tag{12.7}$$

where S_s, S_n, and S_{3n} are elastic section modulus for steel, short-term composite, and long-term composite sections, respectively.

12.4.3 Plastic Moment

The plastic moment M_p for a composite section is defined as the moment that causes the yielding in the steel section and reinforcement and a uniform stress distribution of 0.85 f'_c in compression concrete slab (Figure 12.7). In positive flexure regions, the contribution of reinforcement in concrete slab is small and can be neglected.

The first step of determining M_p is to find the plastic neutral axis (PNA) by equating total tension yielding forces in steel to compression yield in steel and/or concrete slab. The plastic moment is then obtained by summing the first moment of plastic forces in various components about the PNA. For design convenience, Table 12.4 lists the formulas for $\overline{Y}$ and $M_{p.}$

TABLE 12.2 AASHTO-LRFD Design Formulas of Positive Flexure Ranges for Composite Girders (Strength Limit State)

Items	Compact Section Limit, λ_p	Noncompact Section Limit, λ_r	Slender Sections
Web slenderness $2D_{cp}/t_w$	$3.76\sqrt{E/F_{yc}}$	$\alpha_{st}\sqrt{E/f_c}$	N/A
Compression flange slenderness b/t	No requirement at strength limit state		
Compression flange bracing L_b/r_t	No requirement at strength limit state, but should satisfy $1.76\sqrt{E/F_{yc}}$ for loads applied before concrete deck hardens		$> 1.76\sqrt{E/F_{yc}}$
Nominal flexural resistance	For simple spans and continuous spans with compact interior support section: For $D_p \le D'$, $M_n = M_p$ If $D' < D_p \le 5D'$ $M_n = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$ For continuous spans with noncompact interior support section: $M_n = 1.3R_hM_y$ but not taken greater than the applicable values from the above two equations. **Required section ductility** $D_p/D' \le 5$ $D' = \beta\left(\frac{d+t_s+t_h}{7.5}\right)$ $\beta = \begin{cases} 0.9 \text{ for } F_y = 250 \text{ MPa} \\ 0.7 \text{ for } F_y = 345 \text{ MPa} \end{cases}$	For compression flange: $F_n = R_bR_hF_{yc}$ For tension flange: $F_n = R_bR_hF_{yt}\sqrt{1-3\left(\frac{f_v}{F_{yt}}\right)}$ R_b = load shedding factor, for tension flange = 1.0; for compression flange = 1.0 if either a longitudinal stiffener is provided or $2D_c/t_w \le \lambda_b\sqrt{E/f_c}$ is satisfied; otherwise see Eq. (12.2)	Compression Flange: Eq. (12.4) Tension Flange: $F_n = R_bR_hF_{yt}$

A_{fc} = compression flange area
d = depth of steel section
D_{cp} = depth of the web in compression at the plastic moment
D_p = distance from the top of the slab to the plastic neutral axis
f_c = stress in compression flange due to factored load
f_v = maximum St. Venant torsional shear stress in the flange due to the factored load
F_n = nominal stress at the flange
F_{yc} = specified minimum yield strength of the compression flange
F_{yt} = specified minimum yield strength of the tension flange
M_p = plastic flexural moment

$$R_b = 1 - \left(\frac{a_r}{1200+300a_r}\right)\left(\frac{2D_c}{t_w} - \lambda_b\sqrt{\frac{E}{f_c}}\right) \quad (12.2)$$

$$a_r = \frac{2D_ct_w}{A_{fc}} \quad (12.3)$$

M_y = yield flexural moment
R_h = hybrid factor, 1.0 for homogeneous section, see AASHTO-LRFD 6.10.5.4
t_h = thickness of concrete haunch above the steel top flange
t_s = thickness of concrete slab; t_w= web thickness
α_{st} = 6.77 for web without longitudinal stiffeners and 11.63 with longitudinal stiffeners
λ_b = 5.76 for compression flange area ≥ tension flange area, 4.64 for compression area < tension area

TABLE 12.3 AASHTO-LRFD Design Formulas of Negative Flexure Ranges for Composite I Sections (Strength Limit State)

Items	Compact Section Limit, λ_p	Noncompact Section Limit λ_r	Slender Sections
Web slenderness, $2D_{cp}/t_w$	$3.76\sqrt{E/F_{yc}}$	$\alpha_{st}\sqrt{E/F_{yc}}$	N/A
Compression flange slenderness, $b_f/2t_f$	$0.382\sqrt{E/F_{yc}}$	$1.38\sqrt{\dfrac{E}{f_c\sqrt{\dfrac{2D_c}{t_w}}}}$	$>1.38\sqrt{\dfrac{E}{f_c\sqrt{\dfrac{2D_c}{t_w}}}}$
Compression flange unsupported length, L_b	$\left[0.124-0.0759\left(\dfrac{M_l}{M_p}\right)\right]\left[\dfrac{r_yE}{F_{yc}}\right]$	$1.76r_t\sqrt{\dfrac{E}{F_{yc}}}$	$>1.76r_t\sqrt{\dfrac{E}{F_{yc}}}$
Nominal flexural resistance	$M_n = M_p$	$F_n = R_bR_hF_{yf}$	Compression flange Eq. (12.4) Tension flange $F_n = R_bR_hF_{yt}$

b_f = width of compression flange
t_f = thickness of compression flange
M_l = lower moment due to factored loading at end of the unbraced length
r_y = radius of gyration of steel section with respect to the vertical axis (mm)
r_t = radius of gyration of compression flange of steel section plus one third of the web in compression with respect to the vertical axis (mm)

For lateral torsional buckling AASHTO-LRFD 6.10.5.5:

$$F_n = \begin{cases} C_bR_bR_hF_{yc}\left[1.33-0.18\left(\dfrac{L_b}{r_t}\right)\sqrt{\dfrac{F_{yc}}{E}}\right] \le R_bR_hF_{yc} & \text{for } L_p < L_b < L_r \\ C_bR_bR_h\left[\dfrac{9.86E}{\left(L_b/r_r\right)^2}\right] \le R_bR_hF_{yc} & \text{for } L_b \ge L_r \end{cases} \tag{12.4}$$

$$C_b = 1.75-1.05\left(\frac{P_1}{P_2}\right)+0.3\left(\frac{P_1}{P_2}\right)^2 \le 2.3 \tag{12.5}$$

P_1 = smaller force in the compression flange at the braced point due to factored loading
P_2 = larger force in the compression flange at the braced point due to factored loading

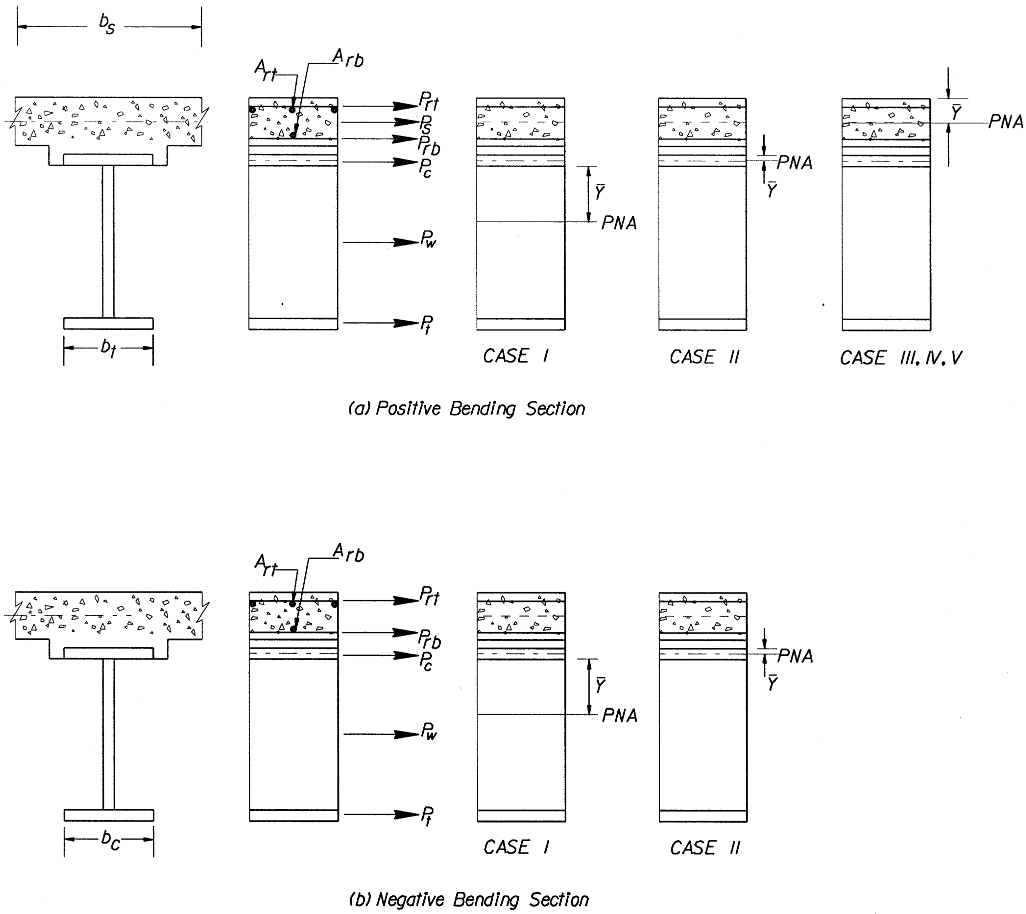


FIGURE 12.7 Plastic moments for composite sections.

Example 12.1: Three-Span Continuous Composite Plate-Girder Bridge

Given

A three-span continuous composite plate-girder bridge has two equal end spans of length 49.0 m and one midspan of 64 m. The superstructure is 13.4 m wide. The elevation, plan, and typical cross section are shown in Figure 12.8.

Structural steel:	A709 Grade 345; $F_{yw} = F_{yt} = F_{yc} = F_y = 345$ MPa
Concrete:	$f'_c = 280$ MPa ; $E_c = 25{,}000$ MPa; modular ratio $n = 8$
Loads:	Dead load = steel plate girder + concrete deck + barrier rail + future wearing 75 mm AC overlay
	Live load = AASHTO HL-93 + dynamic load allowance
Deck:	Concrete deck with thickness of 275 mm has been designed

Steel section in positive flexure region:

Top flange:	$b_{fc} = 460$ mm	$t_{fc} = 25$ mm
Web:	$D = 2440$ mm	$t_w = 16$ mm
Bottom flange:	$b_{ft} = 460$ mm	$t_{ft} = 45$ mm

Construction: Unshored; unbraced length for compression flange $L_b = 6.1$ m.

TABLE 12.4 Plastic Moment Calculation

Regions	Case	Condition and $\bar{Y}$	$\bar{Y}$ and M_p
	I — PNA in web	$P_r + P_w \ge P_c + P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right]$ $+\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t\right]$
	II — PNA in top flange	$P_t + P_w + P_c \ge P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_c - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$	$M_p = \frac{P_c}{2t_c}\left[\bar{Y}^2 + \left(t_c - \bar{Y}\right)^2\right]$ $+\left[P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t\right]$
Positive Figure 12.7a	III — PNA in slab, below P_{rb}	$P_r + P_w + P_c \ge \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rb} + P_{rt}$ $\bar{Y} = \left(t_s\right)\left[\frac{P_w + P_c + P_s - P_{rt} - P_{rb}}{P_s}\right]$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right)^2$ $+\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
	IV — PNA in slab at P_{rb}	$P_r + P_w + P_c + P_{rb} \ge \left(\frac{C_{rb}}{t_s}\right) P_s + P_{rt}$ $\bar{Y} = C_{rb}$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right)^2$ $+\left[P_{rt} d_{rt} + P_c d_c + P_w d_w + P_t d_t\right]$
	V — PNA in slab, above P_{rb}	$P_r + P_w + P_c + P_{rb} \ge \left(\frac{C_{tb}}{t_s}\right) P_s + P_{rt}$ $\bar{Y} = \left(t_s\right)\left[\frac{P_{rb} + P_c + P_w + P_t - P_{rb}}{P_s}\right]$	$M_p = \left(\frac{\bar{Y}^2 P_s}{2t_s}\right)^2$ $+\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$
Negative Figure 12.7b	I — PNA in web	$P_{cr} + P_w \ge P_c + P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_{ct} - P_{rt} - P_{trb}}{P_s} + 1\right]$	$M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right]$ $+\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_t d_t + P_c d_c\right]$
	II — PNA in top flange	$P_r + P_w + P_t \ge P_{rb} + P_{rt}$ $\bar{Y} = \left(\frac{t_t}{2}\right)\left[\frac{P_{rb} + P_c - P_w - P_{rb}}{P_t} + 1\right]$	$M_p = \frac{P_t}{2t_t}\left[\bar{Y}^2 + \left(t_t - \bar{Y}\right)^2\right]$ $+\left[P_{rt} d_{rt} + P_{rb} d_{rb} + P_t d_t + P_c d_c\right]$

P_{rt} $= F_{yrt} A_{rt}$; $P_s = 0.85 f_c' b_s t_s$; $P_{rb} = F_{yrb} A_{rb}$

P_c $= F_{yc} b_c t_t$; $P_w = F_{yw} D t_w$; $P_t = F_{yt} b_t t_t$

A_{rb}, A_{rt} = reinforcement area of bottom and top layer in concrete deck slab

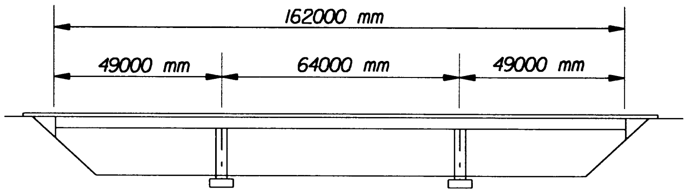
F_{yrb}, F_{yrt} = yield strength of reinforcement of bottom and top layers

b_c, b_t, b_s = width of compression, tension steel flange, and concrete deck slab

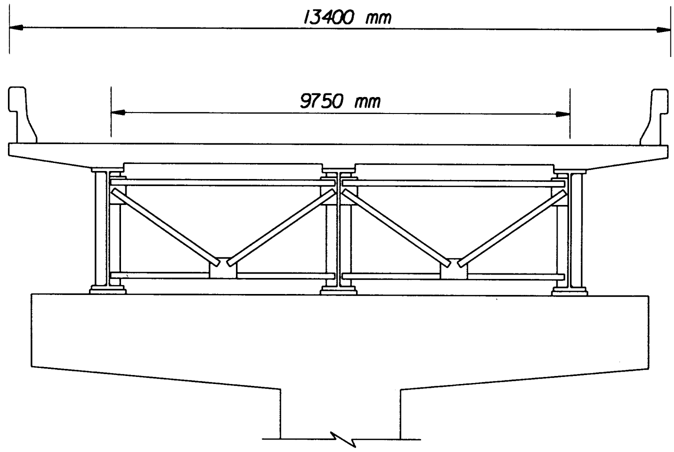
t_c, t_t, t_w, t_s = thickness of compression, tension steel flange, web, and concrete deck slab

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

Source: American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications, Washington, D.C., 1994. With permission.



(a) Elevation



(b) Typical section

FIGURE 12.8 Three-spans continuous plate-girder bridge.

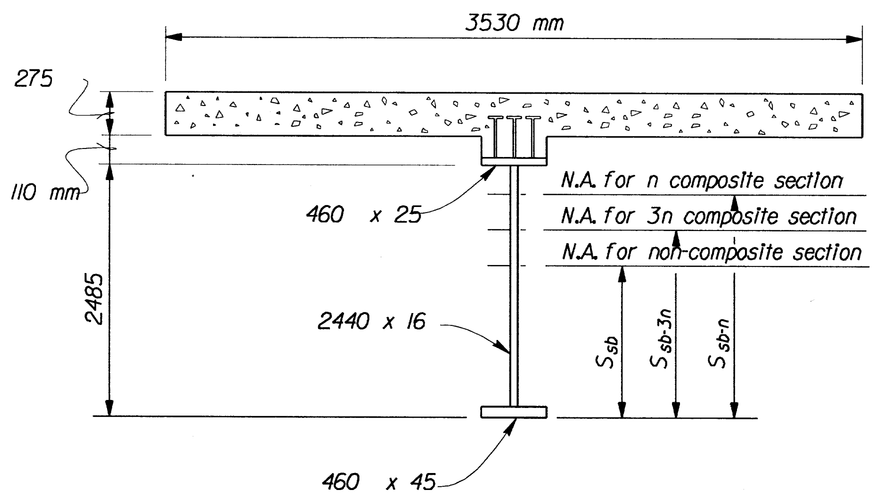


FIGURE 12.9 Cross section for positive flexure region.

Maximum positive moments in Span 1 due to factored loads applied to the steel section, and to the long-term composite section are M_{D1} = 6859 kN-m and M_{D2} = 2224 kN-m, respectively.

Requirement

Determine yield moment M_y, plastic moment M_p, and nominal moment M_n of an interior girder for positive flexure region.

Solutions

1. **Determine Effective Flange Width (AASHTO Article 4.6.2.6)**
For an interior girder, the effective flange width is

$$b_{\text{eff}} = \text{the lesser of} \begin{cases} \dfrac{L_{\text{eff}}}{4} = \dfrac{35,050}{4} = 8763 \text{ mm} \\ 12t_s + \dfrac{b_f}{2} = (12)(275) + \dfrac{460}{2} = 3530 \text{ mm} \quad \text{(controls)} \\ S = 4875 \text{ mm} \end{cases}$$

where L_{eff} is the effective span length and may be taken as the actual span length for simply supported spans and the distance between points of permanent load inflection for continuous spans (35.05 m); b_f is top flange width of steel girder.

2. **Calculate Elastic Composite Section Properties**
For the section in the positive flexure region as shown in Figure 12.9, its elastic section properties for the noncomposite, the short-term composite ($n = 8$), and the long-term composite ($3n = 24$) are calculated in Tables 12.5 to 12.7.

TABLE 12.5 Noncomposite Section Properties for Positive Flexure Region

Component	A (mm²)	y_i (mm)	$A_i y_i$ (mm³)	$y_i - y_{sb}$ (mm)	$A_i(y_i - y_{sb})^2$ (mm⁴)	I_o (in⁴)
Top flange 460 × 25	11,500	2498	28.7 $(10)^6$	1395	22.4 $(10)^9$	1.2 $(10)^6$
Web 2440 × 16	39,040	1265	49.4 $(10)^6$	162	3.0 $(10)^8$	19.4 $(10)^9$
Bottom flange 460 × 45	20,700	22.5	4.7 $(10)^5$	−1081	24.2 $(10)^9$	3.5 $(10)^6$
Σ	71,240	—	78.6 $(10)^6$		46.8 $(10)^9$	19.4 $(10)^9$

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{78.6(10)^6}{71240} = 1103 \text{ mm} \qquad y_{st} = (45 + 2440 + 25) - 1103 = 1407 \text{ mm}$$

$$I_{\text{girder}} = \sum I_o + \sum A_i (y_i - y_{sb})^2$$

$$= 19.4(10)^9 + 46.8(10)^9 = 66.2(10)^9 \text{ mm}^4$$

$$S_{sb} = \frac{I_{girder}}{y_{sb}} = \frac{66.2(10)^9}{1103} = 60.0(10)^6 \text{ mm}^3 \qquad S_{st} = \frac{I_{girder}}{y_{st}} = \frac{66.2(10)^9}{1407} = 47.1(10)^6 \text{ mm}^3$$

3. **Calculate Yield Moment M_y**
The yield moment M_y corresponds to the first yielding of either steel flange. It is obtained by the following formula:

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

TABLE 12.6 Short-Term Composite Section Properties ($n = 8$)

Component	A (mm²)	y_i (mm)	$A_i y_i$ (mm³)	$y_i - y_{sb\text{-}n}$ (mm)	$A_i(y_i - y_{sb-n})^2$ (mm⁴)	I_o (mm⁴)
Steel section	71,240	1103	78.6 (10)⁶	–1027	75.1 (10)⁹	19.4 (10)⁹
Concrete slab 3530/8 × 275	121,344	2733	3.3 (10)⁸	603	44.1 (10)⁹	2.3 (10)⁸
Σ	192,584	—	4.1 (10)⁸	—	119.2 (10)⁹	19.6 (10)⁹

$$y_{sb-n} = \frac{\sum A_i y_i}{\sum A_i} = \frac{4.1(10)^8}{192{,}584} = 2130 \text{ mm} \qquad y_{st-n} = (45 + 2440 + 25) - 2130 = 380 \text{ mm}$$

$$I_{com-n} = \sum I_o + \sum A_i (y_i - y_{sb-n})^2$$

$$= 19.6(10)^9 + 119.2(10)^9 = 138.8(10)^9 \text{ mm}^4$$

$$S_{sb-n} = \frac{I_{con-n}}{y_{sb-n}} = \frac{138.8(10)^9}{2130} = 65.2(10)^6 \text{ mm}^3 \qquad S_{st-n} = \frac{I_{com-n}}{y_{st-n}} = \frac{138.8(10)^9}{380} = 365.0(10)^6 \text{ mm}^3$$

TABLE 12.7 Long-Term Composite Section Properties ($3n = 24$)

Component	A (mm²)	y_i (mm)	$A_i y_i$ (mm³)	$y_i - y_{sb\text{-}3n}$ (mm)	$A_i(y_i - y_{sb-3n})^2$ (mm⁴)	I_o (mm⁴)
Steel section	71,240	1103	78.6 (10⁶)	–590	24.8 (10⁹)	19.4 (10⁹)
Concrete slab 3530/24 × 275	40,448	2733	1.1 (10⁸)	1040	43.7 (10⁹)	2.3 (10⁸)
Σ	111,688	—	10846.4	—	68.5 (10⁹)	19.6 (10⁹)

$$y_{sb-3n} = \frac{\sum A_i y_i}{\sum A_i} = \frac{88.1(10)^9}{111{,}688} = 1693 \text{ mm} \qquad y_{st-3n} = (45 + 2440 + 25) - 1693 = 817 \text{ mm}$$

$$I_{com-3n} = \sum I_o + \sum A_i (y_i - y_{sb-3n})^2$$

$$= 19.6(10)^9 + 68.5(10)^9 = 88.1(10)^9 \text{ mm}^4$$

$$S_{sb-3n} = \frac{I_{con-3n}}{y_{sb-3n}} = \frac{88.1(10)^9}{1693} = 52.0(10)^6 \text{ mm}^3 \qquad S_{st-3n} = \frac{I_{com-3n}}{y_{st-3n}} = \frac{88.4(10)^9}{817} = 107.9(10)^6 \text{ mm}^3$$

$$M_{AD} = S_n \left(F_y - \frac{M_{D1}}{S_s} - \frac{M_{D2}}{S_{3n}} \right)$$

$$M_{D1} = 6859 \text{ kN-m}$$

$$M_{D2} = 2224 \text{ kN-m}$$

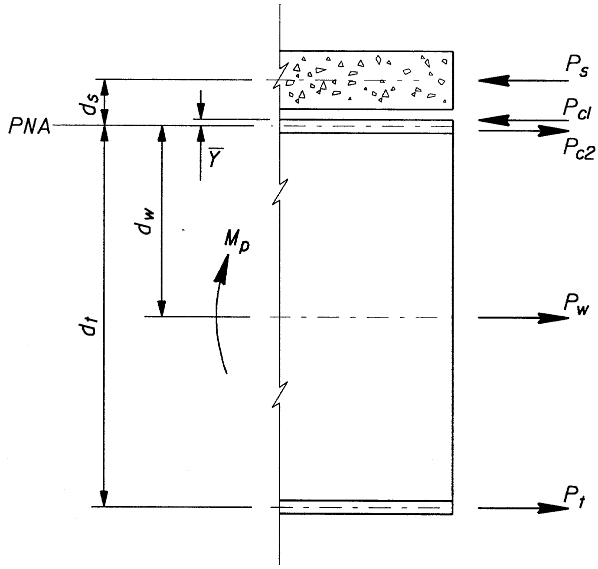


FIGURE 12.10 Plastic moment state.

For the top flange:

$$M_{AD} = (368.4)10^{-3}\left(345(10)^3 - \frac{6859}{47.1(10)^{-3}} - \frac{2224}{108.6(10)^{-3}}\right)$$
$$= 65,905 \text{ kN-m}$$

For the bottom flange:

$$M_{AD} = (65.2)10^{-3}\left(345(10)^3 - \frac{6859}{60.0(10)^{-3}} - \frac{2224}{52.1(10)^{-3}}\right)$$
$$= 12,257 \text{ kN-m} \quad \text{(controls)}$$

$$\therefore \quad M_y = 6859 + 2224 + 12,257 = 21,340 \text{ kN-m}$$

4. **Calculate Plastic Moment Capacity M_p**
   For clarification, the reinforcement in slab is neglected. We first determine the location of the PNA (see Figure 12.10 and Table 12.4).

$$P_s = 0.85 f_c' b_{\text{eff}} t_s = 0.85(28)(3530)(275) = 23,104 \text{ kN}$$

$$P_{c1} = \bar{Y} b_{fc} F_{yc}$$

$$P_{c2} = A_c F_{yc} - P_{c1} = (t_c - \bar{Y}) b_{fc} F_{yc}$$

$$P_c = P_{c1} + P_{c2} = A_{fc} F_{yc} = (460)(25)(345) = 3967 \text{ kN}$$

$$P_w = A_w F_{yw} = (2440)(16)(345) = 13,469 \text{ kN}$$

$$P_t = A_{ft} F_{yt} = (460)(45)(345) = 7141 \text{ kN}$$

Since $P_t + P_w + P_c > P_s$, the PNA is located within top of flange (Case II, Table 12.4).

$$\bar{Y} = \frac{t_c}{2}\left(\frac{P_w + P_t - P_s}{P_c} + 1\right)$$

$$= \frac{25}{2}\left(\frac{13,469 + 7141 - 23,104}{3967} + 1\right) = 4.6 \ \text{mm} < t_c = 25 \ \text{mm}$$

Summing all forces about the PNA (Figure 12.5 and Table 12.4), obtain

$$M_p = \sum M_{\text{PNA}} = P_{c1}\left(\frac{y_{\text{PNA}}}{2}\right) + P_{c2}\left(\frac{t_{fc} - y_{\text{PNA}}}{2}\right) + P_s d_s + P_w d_w + P_t d_t$$

$$= \frac{P_c}{2t_c}\left[\bar{Y}^2 + \left(t_c - \bar{Y}\right)^2\right] + P_s d_s + P_w d_w + P_t d_t$$

$$d_s = \frac{275}{2} + 110 - 25 + 4.0 = 227 \ \text{mm}$$

$$d_w = \frac{2440}{2} + 25 - 4.6 = 1240 \ \text{mm}$$

$$d_t = \frac{45}{2} + 2440 + 25 - 4.6 = 2483 \ \text{mm}$$

$$M_p = \frac{3967}{2(0.025)}\left[0.0046^2 + (0.025 - 0.004)^2\right] + (23,206)(0.227)$$

$$+ (13,469)(1.24) + (7141)(2.483)$$

$$= 39,737 \ \text{kN-m}$$

5. Calculate Nominal Moment

a. *Check compactness of steel girder section*:

- Web slenderness requirement (Table 12.2)

$$\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{F_{yc}}}$$

  Since the PNA is within the top flange, D_{cp} is equal to zero. The web slenderness requirement is satisfied.

- It is usually assumed that the top flange is adequately braced by the hardened concrete deck; there is, therefore, no requirements for the compression flange slenderness and bracing for compact composite sections at the strength limit state.

∴ The section is a compact composite section.

b. Check ductility requirement (Table 12.2) $D_p/D' \le 5$:

The purpose of this requirement is to prevent permanent crashing of the concrete slab when the composite section approaches its plastic moment capacity.

D_p = the depth from the top of the concrete deck to the PNA

$$D_p = 275 + 110 - 25 + 4.6 = 364.6 \text{ mm}$$

$$D' = \beta\left(\frac{d + t_s + t_h}{7.5}\right) = 0.7\left(\frac{2485 + 275 + 110}{7.5}\right) = 267.9 \text{ mm}$$

$$\frac{D_p}{D'} = \frac{364.6}{267.9} = 1.36 < 5 \qquad \text{OK}$$

c. Check moment of inertia ratio limit (AASHTO Article 6.10.1.1):

The flexural members shall meet the following requirement:

$$0.1 \le \frac{I_{yc}}{I_y} \le 0.9$$

where I_{yc} and I_y are the moments of inertia of the compression flange and steel girder about the vertical axis in the plane of web, respectively. This limit ensures that the lateral torsional bucking formulas are valid.

$$I_{yc} = \frac{(25)(460)^3}{12} = 2.03(10^8) \text{ mm}^4$$

$$I_y = 2.03(10^8) + \frac{(2440)(16)^3}{12} + \frac{(45)(460)^3}{12} = 5.69(10^8) \text{ mm}^4$$

$$0.1 < \frac{I_{yc}}{I_y} = \frac{2.03(10^8)}{5.69(10^8)} = 0.36 < 0.9 \qquad \text{OK}$$

d. *Nominal flexure resistance* M_n (Table 12.2):

Assume that the adjacent interior pier section is noncompact. For continuous spans with the noncompact interior support section, the nominal flexure resistance of a compact composite section is taken as

$$M_n = 1.3\, R_h M_y \le M_p$$

with flange stress reduction factor $R_h = 1.0$ for this homogenous girder, we obtain

$$M_n = 1.3(1.0)(21{,}340) = 27{,}742 \text{ kN-m} < M_p = 39{,}712 \text{ kN-m}$$

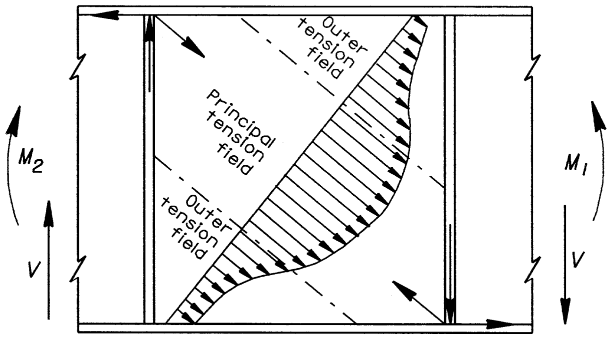


FIGURE 12.11 Tension field action.

12.5 Shear Design

12.5.1 Basic Concept

Similar to the flexural resistance, web shear capacity is also dependent on the slenderness ratio λ in term of width-to-thickness ratio (h/t_w). In calculating shear strength, three failure modes are considered: shear yielding when $\lambda \leq \lambda_p$, inelastic shear buckling when $\lambda_p < \lambda < \lambda_r$, and elastic shear buckling when $\lambda > \lambda_r$. For the web without transverse stiffeners, shear resistance is contributed by the beam action of shearing yield or elastic shear buckling. For interior web panels with transverse stiffeners, shear resistance is contributed by both beam action (the first term of the C_s *equation in Table 12.8) and tension field action* (the second term of the C_s equation in Table 12.8). For end web panels, tension field action cannot be developed because of the discontinuous boundary and the lack of an anchor. It is noted that transverse stiffeners provide a significant inelastic shear buckling strength by tension field action as shown in Figure 12.11. Table 12.8 lists the AASHTO-LRFD [4,5] design formulas for shear strength.

Example 12.2: Shear Strength Design — Strength Limit State I

Given
For the I-girder bridge shown in Example 12.1, factored shear V_u = 2026 and 1495 kN are obtained at the left end of Span 1 and 6.1 m from the left end in Span 1, respectively. Design shear strength for the Strength Limit State I for those two locations.

Solutions

1. **Nominal Shear Resistance V_n**
   a. V_n *for an unstiffened web* (Table 12.8 or AASHTO Article 6.10.7.2):
   For D = 2440 mm and t_w = 16 mm, we have

$$\text{Q} \quad \frac{D}{t_w} = \frac{2440}{16} = 152.5 > 3.07\sqrt{\frac{E}{F_{yw}}} = 3.07\sqrt{\frac{200{,}000}{345}} = 73.9$$

$$\therefore V_n = \frac{4.55 t_w^3 E}{D} = \frac{4.55\,(16)^3(200{,}000)}{2440} = (1528)10^3 \text{ N} = 1528 \text{ kN}$$

TABLE 12.8 AASHTO-LRFD Design Formulas of Nominal Shear Resistance at Strength Limit State

Unstiffened homogeneous webs	$V_n = \begin{cases} V_p = 0.58 F_{yw} D t_w & \text{for } \frac{D}{t_w} \le 2.46\sqrt{\frac{E}{F_{yw}}} \quad - \text{Shear yielding} \\ 1.48 t_w^2 \sqrt{E F_{yw}} & \text{for } 2.46\sqrt{\frac{E}{F_{yw}}} < \frac{D}{t_w} \le 3.07\sqrt{\frac{E}{F_{yw}}} \quad - \text{Inelastic buckling} \\ \frac{4.55 t_w^3 E}{D} & \text{for } \frac{D}{t_w} > 3.07\sqrt{\frac{E}{F_{yw}}} \quad - \text{Elastic buckling} \end{cases}$
Stiffened interior web panels of compact homogeneous sections	$V_n = \begin{cases} C_s V_p & \text{for } M_u \le 0.5\phi_f M_p \\ R C_s V_p \ge C V_p & \text{for } M_u > 0.5\phi_f M_p \end{cases}; \quad C_s = C + \frac{0.87(1-C)}{\sqrt{1+(d_o/D)^2}}$ $R = 0.6 + 0.4\left(\frac{M_r - M_u}{M_r - 0.75\phi_f M_y}\right) \le 1.0$
Stiffened interior web panels of noncompact homogeneous sections	$V_n = \begin{cases} C_s V_p & \text{for } f_u \le 0.5\phi_f F_y \\ R C_s V_p \ge C V_p & \text{for } f_u \le 0.5\phi_f F_y \end{cases} \quad R = 0.6 + 0.4\left(\frac{F_r - f_u}{F_r - 0.75\phi_f F_y}\right) \le 1.0$
End panels and hybrid sections	$V_n = C V_p$

d_o = stiffener spacing (mm)
D = web depth
F_r = factored flexural resistance of the compression flange (MPa)
f_u = factored maximum stress in the compression flange under consideration (MPa)
M_r = factored flexural resistance
M_u = factored maximum moment in the panel under consideration
ϕ_f = resistance factor for flexure = 1.0 for the strength limit state
C = ratio of the shear buckling stress to the shear yield strength

$$C = \begin{cases} 1.0 & \text{For } \frac{D}{t_w} \le 1.1\sqrt{\frac{Ek}{F_{yw}}} \\ \frac{1.1}{D/t_w}\sqrt{\frac{Ek}{F_{yw}}} & \text{For } 1.1\sqrt{\frac{Ek}{F_{yw}}} \le \frac{D}{t_w} \le 1.38\sqrt{\frac{Ek}{F_{yw}}} \\ \frac{1.52}{(D/t_w)^2}\sqrt{\frac{Ek}{F_{yw}}} & \text{For } \frac{D}{t_w} > 1.38\sqrt{\frac{Ek}{F_{yw}}} \end{cases} \quad (12.8)$$

$$k = 5 + \frac{5}{(d_o/D)^2}$$

b. V_n *for end-stiffened web panel* (Table 12.8 or AASHTO Article 6.10.7.3.3c):
Try the spacing of transverse stiffeners $d_o = 6100$ mm. In order to facilitate handling of web panel sections, the spacing of transverse stiffeners shall meet (AASHTO Article 6.10.7.3.2) the following requirement:

$$d_o \le D\left[\frac{260}{(D/t_w)}\right]^2$$

$$d_o = 6100 \text{ mm} < D\left[\frac{260}{(D/t_w)}\right]^2 = 2440\left[\frac{260}{2440/16}\right]^2 = 7090 \text{ mm} \qquad \text{OK}$$

Using formulas in Table 12.8, obtain

$$k = 5 + \frac{5}{(d_o/D)^2} = 5 + \frac{5}{(6100/2440)^2} = 5.80$$

$$\text{Q} \quad \frac{D}{t_w} = 152.5 > 1.38\sqrt{\frac{Ek}{F_{yw}}} = 1.38\sqrt{\frac{200{,}000(5.8)}{345}} = 80$$

$$\text{Q} \quad C = \frac{1.52}{(152.5)^2}\sqrt{\frac{200{,}000(5.80)}{345}} = 0.379$$

$$V_p = 0.58F_{yw}Dt_w = 0.58(345)(2440)(16) = 7812(10)^3 \text{ N} = 7812 \text{ kN}$$

$$V_n = CV_p = 0.379\ (7812) = 2960 \text{ kN}$$

2. Strength Limit State I

AASHTO-LRFD [4] requires that for Strength Limit State I

$$V_u \le \phi_v V_n$$

where ϕ_v is the shear resistance factor = 1.0.

a. *Left end of Span* 1:

$$\text{Q} \quad V_u = 2026 \text{ kN} > \phi_v V_n \text{ (for unstiffened web)} = 1528 \text{ kN}$$

$$\therefore \text{ Stiffeners are needed to increase shear capacity}$$

$$\phi_v V_n = (1.0)\ 2960 = 2960 \text{ kN} > V_u = 2026 \text{ kN} \qquad \text{OK}$$

b. *Location of the first intermediate stiffeners, 6.1 m from the left end in Span* 1:
Since $V_u = 1459$ kN is less than the shear capacity of the unstiffened web $\phi_v V_n = 1528$ kN the intermediate transverse stiffeners may be omitted after the first intermediate stiffeners.

TABLE 12.9 AASHTO-LRFD Design Formulas of Stiffeners

Location	Stiffener	Required Project Width and Area	Required Moment of Inertia
Compression flange	Longitudinal	$b_l \leq 0.48\, t_s \sqrt{E/F_{yc}}$	$I_s \geq \begin{cases} 0.125k^3 & \text{for } n = 1 \\ 0.07k^3n^4 & \text{for } n = 2,3,4 \text{ or } 5 \end{cases}$ k see Table 12.5
	Transverse	Same size as longitudinal stiffener; at least one transverse stiffener on compression flange near the dead load contraflexure point	
Web	Longitudinal	$b_l \leq 0.48 t_s \sqrt{E/F_{yc}}$	$I_l \geq Dt_w^3\left[2.4\left(d_o/D\right)^2 - 0.13\right]$ $r \geq 0.234 d_o \sqrt{F_{yc}/E}$
	Transverse intermediate	$50 + d/30 \leq b_t \leq 0.48 t_s \sqrt{E/F_{ys}}$ $16t_p \geq b_t \geq 0.25 b_f$ $A_s \geq \left[0.15BDt_w \dfrac{(1-C))V_u}{V_r} - 18t_w^2\right]\dfrac{F_{yw}}{F_{ys}}$ B = 1 for stiffener pairs, 1.8 for single angle and 2.4 for single plate	$I_t \geq d_o t_w^3 J$ $J = 2.5\left(D_p/d_o\right)^2 - 2 \geq 0.5$
	Bearing	$b_t \leq 0.48 t_p \sqrt{E/F_{ys}}$ $B_r = \phi_b A_{pn} F_{ys}$	Use effective section (AASHTO-LRFD 6.10.8.2.4) to design axial resistance

b_f = width of compression flange
t_f = thickness of compression flange
f_c = stress in compression flange due to the factored loading
F_{ys} = specified minimum yield strength of the stiffener
ϕ_b = resistance factor of bearing stiffeners = 1.0
A_{pn} = area of the projecting elements of the stiffener outside of the web-to-flange fillet welds, but not beyond the edge of the flange

12.5.2 Stiffeners

For built-up I-sections, the longitudinal stiffeners may be provided to increase bending resistance by preventing local buckling while transverse stiffeners are usually provided to increase shear resistance by the tension field action [10,11]. The following three types of stiffeners are usually used for I-sections:

- *Transverse Intermediate Stiffeners*: These work as anchors for the tension field force so that postbuckling 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 present local buckling, (2) provide stiffness to allow the web to develop its postbuckling capacity, and (3) have strength to resist the vertical components of the diagonal stresses in the web. These requirements are listed in Table 12.9.
- *Bearing Stiffeners*: These work as compression members to support vertical concentrated loads by bearing on the ends of stiffeners (see Figure 12.2). They are transverse stiffeners and connect to the web to provide a vertical boundary for anchoring shear force from tension

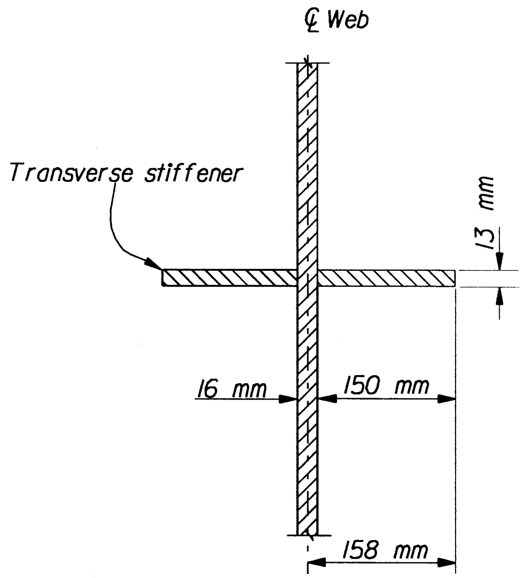


FIGURE 12.12 Cross section of web and transverse stiffener.

field action. They should be placed at all bearing locations and at all locations supporting concentrated loads. For rolled beams, bearing stiffeners may not be needed when factored shear is less than 75% of factored shear capacity. They are designed to satisfy the slenderness, bearing, and axial compression requirements as shown in Table 12.9.

- *Longitudinal Stiffeners*: These work as restraining boundaries for compression elements so that inelastic flexural buckling stress can be developed in a web. It consists of either a plate welded longitudinally to one side of the web, or a bolted angle. It should be located a distance of $2D_c/5$ from the inner surface of the compression flange, where D_c is the depth of web in compression at the maximum moment section to provide optimum design. The slenderness and stiffness need to be considered for sizing the longitudinal stiffeners (Table 12.9).

Example 12.3: Transverse and Bearing Stiffeners Design

Given

For the I-girder bridge shown in Example 12.1, factored shear V_u = 2026 and 1495 kN are obtained at the left end of Span 1 and 6.1 m from the left end in Span 1, respectively. Design the first intermediate transverse stiffeners and the bearing stiffeners at the left support of Span 1 using F_{ys} = 345 MPa for stiffeners.

Solutions

1. **Intermediate Transverse Stiffener Design**

   Try two 150 × 13 mm transverse stiffener plates as shown in Figure 12.12 welded to both sides of the web.

   a. *Projecting width b_t requirements* (Table12.9 or AASHTO Article 6.10.8.1.2):

   To prevent local buckling of the transverse stiffeners, the width of each projecting stiffener shall satisfy these requirements listed in Table 12.9.

$$b_t = 150 \text{ mm} > \begin{cases} 50 + \dfrac{d}{30} = 50 + \dfrac{2510}{30} = 134 \text{ mm} \\ 0.25 b_f = 0.25(460) = 115 \text{ mm} \end{cases} \quad \text{OK}$$

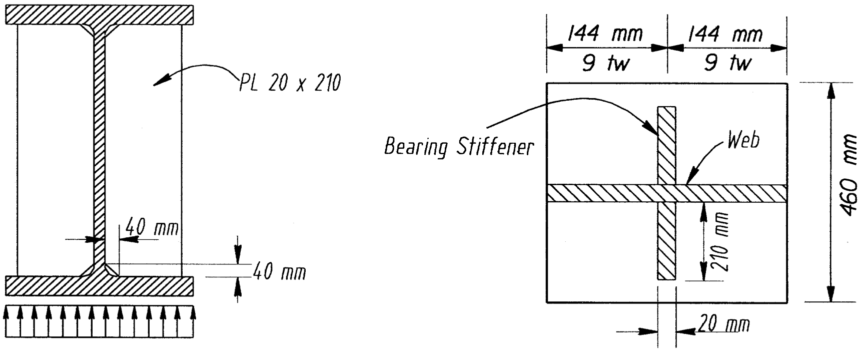


FIGURE 12.13 (a) Bearing stiffeners; (b) effective column area.

$$b_t = 150 \text{ mm} < \begin{cases} 0.48t_p\sqrt{\dfrac{E}{F_{ys}}} = 0.48(13)\sqrt{\dfrac{200{,}000}{345}} = 150 \text{ mm} \\ 16t_p = 16(13) = 208 \text{ mm} \end{cases} \quad \text{OK}$$

b. *Moment of inertia requirement* (Table 12.9 AASHTO Article 6.10.8.1.3):
The purpose of this requirement is to ensure sufficient rigidity of transverse stiffeners to develop adequately a tension field in the web.

$$\because J = 2.5\left(\frac{2440}{6100}\right)^2 - 2.0 = -1.6 < 0.5 \qquad \because \underline{\text{Use } J = 0.5}$$

$$I_t = 2\left(\frac{150^3(13)}{3}\right) = 29.3(10)^6 \text{ mm}^4 > d_o t_w^3 J = (6100)(16)^3(0.5) \quad \text{OK}$$

$$= 12.5(10)^6 \text{ mm}^4$$

c. *Area requirement* (Table 12.9 or AASHTO Article 6.10.8.1.4):
This requirement ensures that transverse stiffeners have sufficient area to resist the vertical component of the tension field, and is only applied to transverse stiffeners required to carry the forces imposed by tension field action. From Example 12.2, we have $C = 0.379$; $F_{yw} = 345$ MPa; $V_u = 1460$ kN; $\phi_v V_n = 1495$ kN; $t_w = 16$ mm; $B = 1.0$ for stiffener pairs. The requirement area is

$$A_{sreqd} = \left(0.15(1.0)(2440)(16)(1-0.379)\frac{1459}{1495} - 18(16)^2\right)\left(\frac{345}{345}\right) = -1060 \text{ mm}^2$$

The negative value of A_{sreqd} indicates that the web has sufficient area to resist the vertical component of the tension field.

2. Bearing Stiffener Design
Try two 20 × 210 mm stiffness plates welded to each side of the web as shown in Figure 12.13a.
a. *Check local buckling requirement* (Table 12.9 or AASHTO Article 6.19.8.2.2):

$$\frac{b_t}{t_p} = \frac{210}{20} = 10.5 \leq 0.48\sqrt{\frac{E}{F_y}} = 0.48\sqrt{\frac{200,000}{345}} = 11.6 \qquad \text{OK}$$

b. *Check bearing resistance* (Table 12.9 or AASHTO Article 6.10.8.2.3):
Contact area of the stiffeners on the flange $A_{pn} = 2(210 - 40)20 = 6800 \text{ mm}^2$

$$B_r = \phi_b A_{pn} F_{ys} = (1.0)(6800)(345) = 2346\,(10)^3 \text{ N} = 2346 \text{ kN} > V_u = 2026 \text{ kN} \quad \text{OK}$$

c. *Check axial resistance of effective column section* (Table 12.9 or AASHTO 6.10.8.2.4):
Effective column section area is shown in Figure 12.13b:

$$A_s = 2\left[210(20) + 9(16)(16)\right] = 13,008 \text{ mm}^2$$

$$I = \frac{(20)(420+16)^3}{12} = 138.14(10)^6 \text{ mm}^4$$

$$r_s = \sqrt{\frac{I}{A_s}} = \sqrt{\frac{138.14(10)^6}{13,008}} = 103.1 \text{ mm}$$

$$\lambda = \left(\frac{KL}{r_s\pi}\right)^2 \frac{F_y}{E} = \left(\frac{0.75(2440)}{103.1\pi}\right)^2 \frac{345}{200,000} = 0.055$$

$$P_n = 0.66^{\lambda} F_y A_s = 0.66^{0.055}(345)(13,008) = 4386\,(10)^3 \text{ N}$$

$$P_r = \phi_c P_n = 0.9(4386) = 3947 \text{ kN} > V_u = 2026 \text{ kN} \qquad \text{OK}$$

Therefore, using two 20 × 210 mm plates are adequate for bearing stiffeners at abutment.

12.5.3 Shear Connectors

To ensure a full composite action, shear connectors must be provided at the interface between the concrete slab and the structural 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 contraflexure points or they can be continued over the negative flexure region at maximum spacing. The two types of shear connectors such as shear studs and channels (see Figure 12.4) are most commonly used in modern bridges. The fatigue and strength limit states must be considered in the shear connector design. The detailed requirements are listed in Table 12.10.

Example 12.4: Shear Connector Design

Given
For the I-girder bridge shown in Example 12.1, design the shear stud connectors for the positive flexure region of Span 1. The shear force ranges V_{sr} are given in Table 12.11 and assume number of cycle $N = 7.844(10)^7$.

TABLE 12.10 AASHTO-LRFD Design Formulas of Shear Connectors

Connector Types	Stud	Channel
Basic requirement	$\frac{h_s}{d_s} \geq 4.0$	Fillet welds along the heels and toe shall not smaller than 5 mm
	$6d_s <$ pitch of connector $p = (nZ_r I)/V_{sr}Q < 600$ mm	
	Transverse spacing $\geq 4d_s$ Clear distance between flange edge of nearest connector ≥ 25 mm Concrete cover over the top of the connectors ≥ 50 mm and $d_s \geq 50$ mm	
Special requirement	For noncomposite negative flexure region, additional number of connector: $n_{ac} = (A_r f_{sr})/Z_r$	
Fatigue resistance	$Z_r = \alpha d_s^2 \geq 19 d_s^2$ $\alpha = 238 - 29.5 \log N$	—
Nominal shear resistance	$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u$	$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c}$
Required shear connectors	$n = \frac{V_h}{\phi_{sc}Q_n}$; $V_h = \text{smaller}\begin{cases}0.85 f'_c b_{\text{eff}} t_s \\ \sum A_{si}F_{yi}\end{cases}$	
	For continuous span between each adjacent zero moment of the centerline of interior support: $V_h = A_r F_{vr}$	

A_{si} = area of component of steel section
b_{eff} = effective flange width
h_s = height of stud
d_s = diameter of stud
n = number of shear connectors in a cross section
E_c = modulus of elasticity of concrete
f_c = stress in compression flange due to the factored loading
f'_c = specified compression strength of concrete
F_{yi} = specified minimum yield strength of the component of steel section
f_{sr} = stress range in longitudinal reinforcement (AASHTO-LRFD 5.5.3.1)
F_u = specified minimum tensile strength of a stud
L_c = length of channel shear connector
Q = first moment of transformed section about the neutral axis of the short-term composite section
I = moment of inertia of short-term composite section
N = number of cycles (AASHTO-LRFD 6.6.1.2.5)
V_{sr} = shear force range at the fatigue limit state
t_s = thickness of concrete slab
t_f = flange thickness of channel shear connector
Z_r = shear fatigue resistance of an individual shear connector

Solutions

1. **Stud Size (Table 12.10 AASHTO Article 6.10.7.4.1a)**
   Stud height should penetrate at least 50 mm into the deck. The clear cover depth of concrete cover over the top of the shear stud should not be less than 50 mm. Try

$$H_s = 180 \text{ mm} > 50 + (110 - 25) = 135 \text{ mm (min)} \qquad \text{OK}$$

$$\text{stud diameter } d_s = 25 \text{ mm} < H_s/4 = 45 \text{ mm} \qquad \text{OK}$$

TABLE 12.11 Shear Connector Design for the Positive Flexure Region in Span 1

Span	Location (x/L)	V_{sr} (kN)	p_{required} (mm)	p_{final} (mm)	$n_{\text{total-stud}}$
	0.0	267.3	253	245	3
	0.1	229.5	295	272	63
	0.2	212.6	318	306	117
	0.3	205.6	329	326	165
1	0.4	203.3	333	326	**210**
	0.4	203.3	333	326	**144**
	0.5	202.3	334	326	99
	0.6	212.6	318	306	51
	0.7	223.7	302	306	3

Notes:

1. $V_{sr} = \left|+(V_{LL+IM})_u\right| + \left|-(V_{LL+IM})_u\right|$.
2. $p_{\text{required}} = \dfrac{n_s Z_r I_{com-n}}{V_{sr} Q} = \dfrac{67\,634}{V_{sr}}$.
3. $n_{\text{total-stud}}$ is summation of number of shear studs between the locations of the zero moment and that location.

2. Pitch of Shear Stud, *p*, for Fatigue Limit State

a. *Fatigue resistance* Z_r (Table 12.10 or AASHTO Article 6.10.7.4.2):

$$\alpha = 238 - 29.5\log(7.844\times10^7) = 5.11$$

$$Z_r = 19d_s^2 = 19(25)^2 = 11{,}875 \text{ N}$$

b. *First moment Q and moment of initial I* (Table 12.6):

$$Q = \left(\frac{b_{\text{eff}}\,t_s}{8}\right)\left(y_{st-n} + t_h + \frac{t_s}{2}\right)$$

$$= \left(\frac{3530(275)}{8}\right)\left(380 + 85 + \frac{275}{2}\right) = 73.11\,(10^6)\text{ mm}^3$$

$$I_{com-n} = 138.8(10^9)\text{ mm}^4$$

c. *Required pitch for the fatigue limit state*:
Assume that shear studs are spaced at 150 mm transversely across the top flange of steel section (Figure 12.9) and using $n_s = 3$ for this example and obtain

$$p_{\text{reqd}} = \frac{n_s Z_r I}{V_{sr} Q} = \frac{3(11.875)(138.8)(10)^9}{V_{sr}(73.11)(10)^6} = \frac{67634}{V_{sr}}$$

The detailed calculations for the positive flexure region of Span 1 are shown in Table 12.11.

3. Strength Limit State Check

a. *Nominal horizontal shear force* (AASHTO Article 6.10.7.4.4b):

$$V_h = \text{the lesser of} \begin{cases} 0.85 f_c' b_{\text{eff}} t_s \\ F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc} \end{cases}$$

$$V_{h-concrete} = 0.5 f_c' b_{\text{eff}} t_s = 0.85(28)(3530)(275) = 2.31(10)^6 \text{ N}$$

$$V_{h-steel} = F_{yw} D t_w + F_{yt} b_{ft} t_{ft} + F_{yc} b_{fc} t_{fc}$$

$$= 345[(2440)(16) + (460)(45) + (460)(25)] = 2.458(10)^6 \text{ N}$$

$$\therefore \quad V_h = 23\,100 \text{ kN}$$

b. *Nominal shear resistance* (Table 12.10 or AASHTO Article 6.10.7.4.4c):
Use specified minimum tensile strength F_u = 420 MPa for stud shear connectors

$$Q \quad 0.5\sqrt{f_c' E_c} = 0.5\sqrt{28(25{,}000)} = 418.3 \text{ MPa} < F_u = 420 \text{ MPa}$$

$$\therefore Q_n = 0.5 A_{sc}\sqrt{f_c' E_c} = 418.3\left(\frac{\pi(25)^2}{4}\right) = 205\,332 \text{ N} = 205 \text{ kN}$$

c. *Check resulting number of shear stud connectors* (see Table 12.11):

$$n_{\text{total-stud}} = \begin{Bmatrix} 210 & \text{from left end } 0.4\,L_1 \\ 144 & \text{from } 0.4L_1 \text{ to } 0.7L_1 \end{Bmatrix} > \frac{V_h}{\phi_{sc} Q_n} = \frac{23\,100}{0.85(205)} = 133 \qquad \text{OK}$$

12.6 Other Design Considerations

12.6.1 Fatigue Resistance

The basic fatigue design requirement limits live-load stress range to fatigue resistance for each connection detail. Special attention should be paid to two types of fatigue: (1) load-induced fatigue for a repetitive net tensile stress at a connection details caused by moving truck and (2) distortion-induced fatigue for connecting plate details of cross frame or diaphragms to girder webs. See Chapter 53 for a detailed discussion.

12.6.2 Diaphragms and Cross Frames

Diaphragms and cross frames, as shown in Figure 12.14, are transverse components to transfer lateral loads such as wind or earthquake loads from the bottom of girder to the deck and from the deck to bearings, to provide lateral stability of a girder bridge, and to distribute vertical loads to the longitudinal main girders. Cross frames usually consist of angles or WT sections and act as a truss, while diaphragms use channels or I-sections as a flexural beam connector. End cross frames or diaphragms at piers and abutments are provided to transmit lateral wind loads and/or earthquake load to the bearings, and intermediate one are designed to provide lateral support to girders.

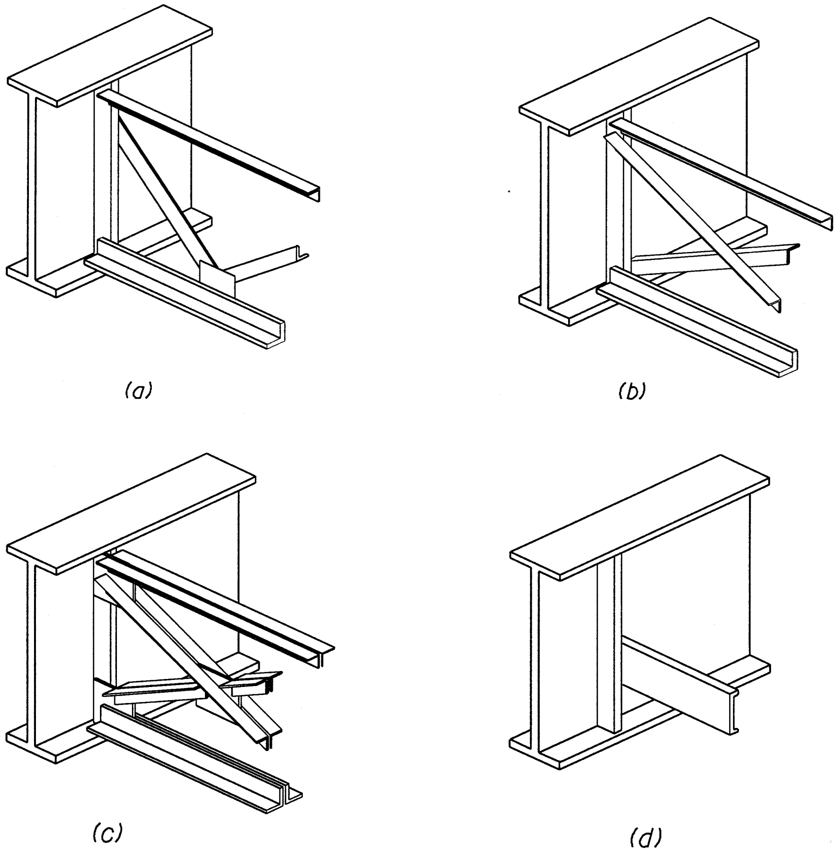


FIGURE 12.14 Cross frames and diaphragms.

The following general guidelines should be followed for diaphragms and cross frames:

- The diaphragm or cross frame shall be as deep as practicable to transfer lateral load and to provide lateral stability. For rolled beam, they shall be at least half of beam depth [AASHTO-LRFD 6.7.4.2].
- Member size is mainly designed to resist lateral wind loads and/or earthquake loads. A rational analysis is preferred to determine actual lateral forces.
- Spacing shall be compatible with the transverse stiffeners.
- Transverse connectors shall be as few as possible to avoid fatigue problems.
- Effective slenderness ratios (KL/r) for compression diagonal shall be less than 140 and for tension member (L/r) less than 240.

Example 12.5: Intermediate Cross-Frame Design

Given

For the I-girder bridge shown in Example 12.1, design the intermediate cross frame as for wind loads using single angles and M270 Grade 250 Steel.

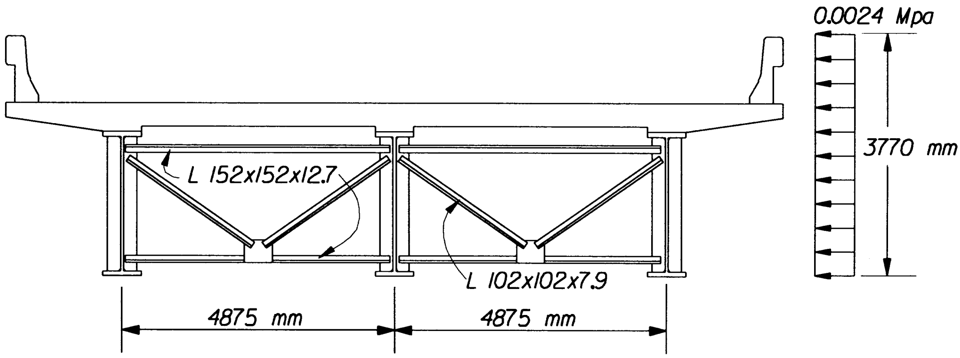


FIGURE 12.15 Wind load distribution.

Solutions:

1. **Calculate Wind Load**
   In this example, we assume that wind load acting on the upper half of girder, deck, and barrier is carried out by the deck slab and wind load on the lower half of girder is carried out by bottom flange. From AASHTO Table 3.8.1.2, wind pressure P_D = 0.0024 MPa, d = depth of structure member = 2,510 mm, and γ = load factor = 1.4 (AASHTO Table 3.4.1-1). The wind load on the structure (Figure 12. 15) is

$$W = 0.0024(3770) = 9.1 \text{ kN/m} > 4.4 \text{ kN/m}$$

Factored wind force acting on bottom flange:

$$W_{bf} = \frac{\gamma p_D d}{2} = \frac{1.4\ (0.0024)(2510)}{2} = 4.21 \text{ kN / m}$$

Wind force acting on top flange (*neglecting concrete deck diaphragm*):

$$W_{tf} = 1.4(0.0024)\left(3770 - \frac{2510}{2}\right) = 8.45 \text{ kN / m}$$

2. **Calculate forces acting on cross frame**
   For cross frame spacing:

$$L_b = 6.1 \text{ m}$$

Factored force acting on bottom strut:

$$F_{bf} = W_{bf} L_b = 4.21(6.1) = 25.68 \text{ kN}$$

Force acting on diagonals:

$$F_d = \frac{F_{tf}}{\cos\phi} = \frac{8.45(6.1)}{\cos 45^o} = 72.89 \text{ kN}$$

3. Design bottom strut

Try $\angle$ 152 × 152 × 12.7; A_s = 3710 mm^2; $r_{\min}$ = 30 mm; L = 4875 mm

Check member slenderness and section width/thickness ratios:

$$\frac{KL}{r} = \frac{0.75(4875)}{30} = 121.9 < 140 \qquad \text{OK}$$

$$\frac{b}{t} = \frac{152}{12.7} = 11.97 < 0.45\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{200{,}000}{250}} = 12.8 \qquad \text{OK}$$

Check axial load capacity:

$$\lambda = \left(\frac{0.75(4875)}{30.0\pi}\right)^2 \frac{250}{200{,}000} = 1.88 < 2.25 \quad \text{(ASSHTO 6.9.4.1-1)}$$

$$P_n = 0.66^{\lambda} A_s F_y = 0.66^{1.88}(3710)(250) = 424{,}675 \text{ N} = 425 \text{ kN}$$

$$P_r = \phi_c P_n = 0.9(425) = 382.5 \text{ kN} > F_{bf} = 25.68 \text{ kN} \qquad \text{OK}$$

4. Design diagonals

Try $\angle$ 102 × 102 × 7.9, A_s = 1550 mm^2; $r_{\min}$ = 20.1 mm; L = 3450 mm

Check member slenderness and section width/thickness ratios:

$$\frac{KL}{r} = \frac{0.75(3450)}{20.1} = 128.7 < 140 \qquad \text{OK}$$

$$\frac{b}{t} = \frac{102}{7.9} = 12.9 \approx 0.45\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{200{,}000}{250}} = 12.8 \qquad \text{OK}$$

Check axial load capacity:

$$\lambda = \left(\frac{0.75(3450)}{20.1\pi}\right)^2 \frac{250}{200{,}000} = 2.1 < 2.25 \quad \text{(ASSHTO 6.9.4.1-1)}$$

$$P_n = 0.66^{\lambda} A_s F_y = 0.66^{2.1}(1550)(250) = 161{,}925 \text{ N} = 162 \text{ kN}$$

$$P_r = \phi_c P_n = 0.9(162) = 145.8 \text{ kN} > F_d = 72.89 \text{ kN} \qquad \text{OK}$$

5. **Top strut**
   The wind force in the top strut is assumed zero because the diagonal will transfer the wind load directly into the deck slab. To provide lateral stability to the top flange during construction, we select angle $\angle$ 152 × 152 × 12.7 for top struts.

12.6.3 Lateral Bracing

The lateral bracing transfers wind loads to bearings and provides lateral stability to compression flange in a horizontal plan. All construction stages should be investigated for the need of lateral bracing. The lateral bracing should be placed as near the plane of the flange being braced as possible. Design of lateral bracing is similar to the cross frame.

12.6.4 Serviceability and Constructibility

The service limit state design is intended to control the permanent deflections, which would affect riding ability. AASHTO-LRFD [AASHTO-LRFD 6.10.3] requires that for Service II (see Chapter 5) load combination, flange stresses in positive and negative bending should meet the following requirements:

$$f_r = \begin{cases} 0.95\,R_h F_{yf} & \text{for both steel flanges of composite section} \\ 0.80\,R_h F_{yf} & \text{for both flanges of noncomposite section} \end{cases} \tag{12.9}$$

where R_h is a hybrid factor, 1.0 for homogeneous sections (AASHTO-LRFD 6.10.5.4), f_f is elastic flange stress caused by the factored loading, and F_{yf} is yield strength of the flange.

An I-girder bridge constructed in unshored conditions shall be investigated for strength and stability for all construction stages, using the appropriate strength load combination discussed in Chapter 5. All calculations should be based on the noncomposite steel section only.

Splice locations should be determined in compliance with both contructibility and structural integrity. The splices for main members should be designed at the strength limit state for not less than (AASHTO 10.13.1) the larger of the following:

- The average of flexural shear due to the factored loads at the splice point and the corresponding resistance of the member;
- 75% of factored resistance of the member.

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