Saleh, Y., Duan, L. "Conceptual Bridge Design."
Bridge Engineering Handbook.
Ed. Wai-Fah Chen and Lian Duan
Boca Raton: CRC Press, 2000

13

Steel–Concrete Composite Box Girder Bridges

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13.1 Introduction

Box girders are used extensively in the construction of urban highway, horizontally curved, and long-span bridges. Box girders have higher flexural capacity and torsional rigidity, and the closed shape reduces the exposed surface, making them less susceptible to corrosion. Box girders also provide smooth, aesthetically pleasing structures.

There are two types of steel box girders: steel–concrete composite box girders (i.e., steel box composite with concrete deck) and steel box girders with orthotropic decks. Composite box girders are generally used in moderate- to medium-span (30 to 60 m) bridges, and steel box girders with orthotropic decks are often used for longer-span bridges.

This chapter will focus on straight steel–concrete composite box-girder bridges. Steel box girders with orthotropic deck and horizontally curved bridges are presented in Chapters 14 and 15.

13.2 Typical Sections

Composite box-girder bridges usually have single or multiple boxes as shown in Figure 13.1. A single cell box girder (Figure 13.1a) is easy to analyze and relies on torsional stiffness to carry eccentric loads. The required flexural stiffness is independent of the torsional stiffness. A single box girder with multiple cells (Figure 13.1b) is economical for very long spans. Multiple webs reduce the flange

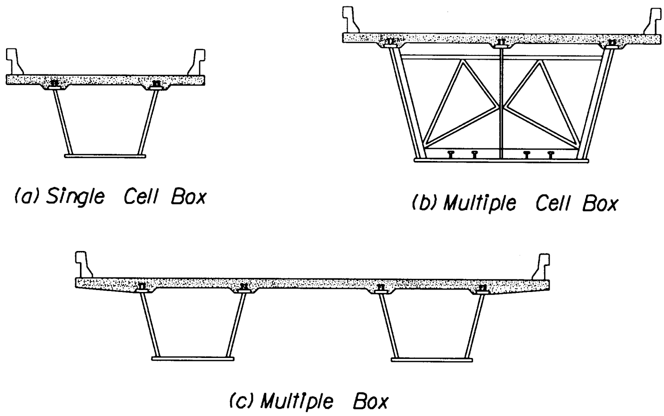

FIGURE 13.1 Typical cross sections of composite box girder.

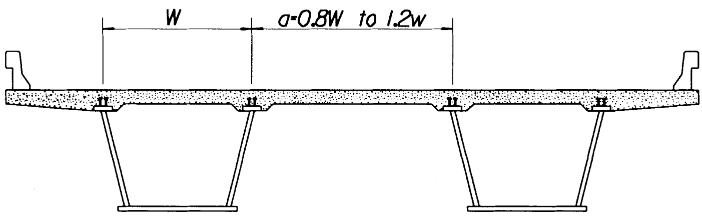

FIGURE 13.2 Flange distance limitation.

shear lag and also share the shear forces. The bottom flange creates more equal deformations and better load distribution between adjacent girders. The boxes in multiple box girders are relatively small and close together, making the flexural and torsional stiffness usually very high. The torsional stiffness of the individual boxes is generally less important than its relative flexural stiffness. For design of a multiple box section (Figure 13.1c), the limitations shown in Figure 13.2. should be satisfied when using the AASHTO-LRFD Specifications [1,2] since the AASHTO formulas were developed from these limitations. The use of fewer and bigger boxes in a given cross section results in greater efficiency in both design and construction [3].

A composite box section usually consists of two webs, a bottom flange, two top flanges and shear connectors welded to the top flange at the interface between concrete deck and the steel section (Figure 13.3). The top flange is commonly assumed to be adequately braced by the hardened concrete deck for the strength limit state, and is checked against local buckling before concrete deck hardening. The flange should be wide enough to provide adequate bearing for the concrete deck and to allow sufficient space for welding of shear connectors to the flange. The bottom flange is designed to resist bending. Since the bottom flange is usually wide, longitudinal stiffeners are often required in the negative bending regions. Web plates are designed primarily to carry shear forces and may be placed perpendicular or inclined to the bottom flange. The inclination of web plates should not exceed 1 to 4. The preliminary determination of top and bottom flange areas can be obtained from the equations (Table 13.1) developed by Heins and Hua [4] and Heins [6].

13.3 General Design Principles

A box-girder highway bridge should be designed to satisfy AASHTO-LRFD specifications to achieve the objectives of constructibility, safety, and serviceability. This section presents briefly basic design principles and guidelines. For more-detailed information, readers are encouraged to refer to several texts [6–14] on the topic.

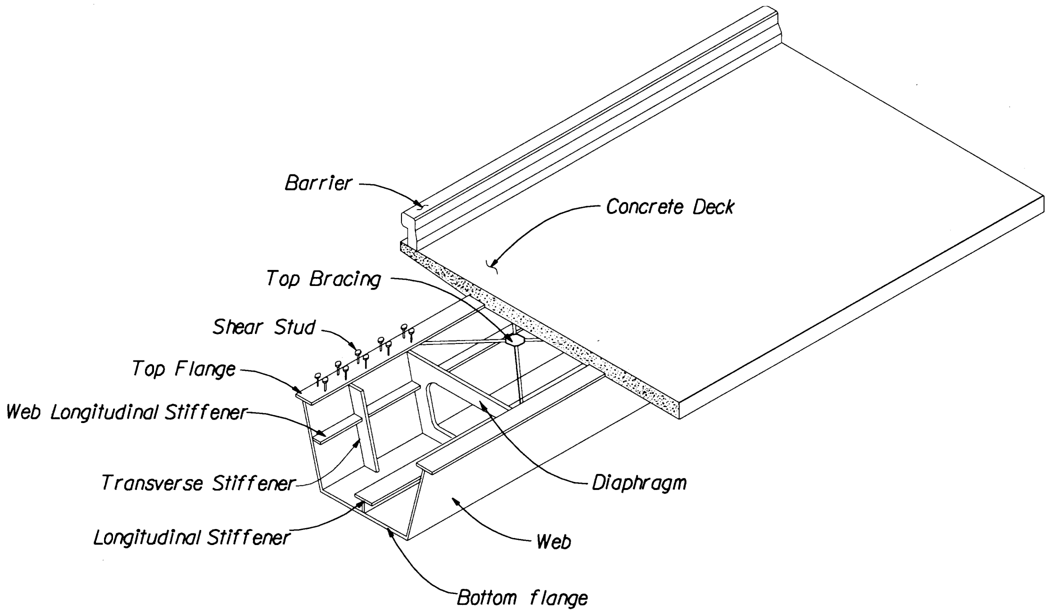

FIGURE 13.3 Typical components of a composite box girder.

In multiple box-girder design, primary consideration should be given to flexure. In single box-girder design, however, both torsion and flexure must be considered. Significant torsion on single box girders may occur during construction and under live loads. Warping stresses due to distortion should be considered for fatigue but may be ignored at the strength limit state. Torsional effects may be neglected when the rigid internal bracings and diaphragms are provided to maintain the box cross section geometry.

13.4 Flexural Resistance

The flexural resistance of a composite box girders depends on the compactness of the cross sectional elements. This is related to compression flange slenderness, lateral bracing, and web slenderness. A "compact" section can reach full plastic flexural capacity. A "noncompact" section can only reach yield at the outer fiber of one flange.

In positive flexure regions, a multiple box section is designed to be compact and a single box section is considered noncompact with the effects of torsion shear stress taken by the bottom flange (Table 13.2). In general, in box girders non-negative flexure regions design formulas of nominal flexure resistance are shown in Table 13.3.

In lieu of a more-refined analysis considering the shear lag phenomena [15] or the nonuniform distribution of bending stresses across wide flanges of a beam section, the concept of effective flange width under a uniform bending stress has been widely used for flanged section design [AASHTO-LRFD 4.6.2.6]. The effective flange width is a function of slab thickness and the effective span length.

13.5 Shear Resistance

For unstiffened webs, the nominal shear resistance V_n is based on shear yield or shear buckling depending on web slenderness. For stiffened interior web panels of homogeneous sections, the postbuckling resistance due to tension-field action [16,17] is considered. For hybrid sections, tension-field action is

TABLE 13.1 Preliminary Selection of Flange Areas of Box-Girder Element

Items	Top Flange		Bottom Flange	
	A_T^+	A_T^-	A_B^+	A_B^-
Single span	$254d\left(1-\frac{26}{L}\right)$	—	$328d\left(1-\frac{28}{L}\right)$	—
Two span	$0.64A_B^+$	$1.60A_B^+\frac{F_y^-}{F_y^+}$	$\frac{645}{k}(1.65L^2-0.74L+13)$	$1.17A_B^+\frac{F_y^-}{F_y^+}$
Three span	$\frac{330n}{k}(L_1-22)$	$\frac{814n}{k}(L_1-31)$	$\frac{423n}{k}(L_1-16)$	$\frac{645}{kn}(3.16L_2-0.018L_2^2-70)$
	$0.95A_T^- - \frac{A_T^{-2}}{58650}-\frac{3484}{k}$		$\frac{211.67n}{k}(L_2-14.63)$	

A_T^+, A_T^- = the area of top flange (mm²) in positive and negative region, respectively

A_B^+, A_B^- = the area of bottom flange (mm²) in positive and negative region, respectively

d = depth of girder (mm)

L, L_1, L_2 = length of the span (m); for simple span ($27 \le L \le 61$)

for two spans ($30 \le L_2 \le 67$)

for three spans ($27 \le L_1 \le 55$)

W_R = roadway width (m)

N_b = number of boxes

n = L_2/L_1

k = $\frac{N_B F_y d}{W_R(344{,}750)}$

F_y = yield strength of the material (MPa)

not permitted and shear yield or elastic shear buckling limits the strength. The detailed AASHTO-LRFD design formulas are shown in Table 12.8 (Chapter 12). For cases of inclined webs, the web depth D shall be measured along the slope and be designed for the projected shear along inclined web.

To ensure composite action, shear connectors should be provided at the interface between the concrete slab and the steel section. For single-span bridges, connectors should be provided throughout the span of the bridge. Although it is not necessary to provide shear connectors in negative flexure regions if the longitudinal reinforcement is not considered in a composite section, it is recommended that additional connectors be placed in the region of dead-load contraflexure points [AASHTO-LRFD 1.10.7.4]. The detailed requirements are listed in Table 12.10.

13.6 Stiffeners, Bracings, and Diaphragms

13.6.1 Stiffeners

Stiffeners consist of longitudinal, transverse, and bearing stiffeners as shown in Figure 13.1. They are used to prevent local buckling of plate elements, and to distribute and transfer concentrated loads. Detailed design formulas are listed in Table 12.9.

TABLE 13.2 AASHTO-LRFD Design Formulas of Nominal Flexural Resistance in Negative Flexure Ranges for Composite Box Girders (Strength Limit State)

Compression flange with longitudinal stiffeners	$F_n = \begin{cases} R_b R_h F_{yc} & \text{for } \frac{w}{t} \le 0.57\sqrt{\frac{kE}{F_{yc}}} \\ 0.592 R_b R_h F_{yc}\left(1 + 0.687 \sin\frac{c\pi}{2}\right) & \text{for } 0.57\sqrt{\frac{kE}{F_{yc}}} < \frac{w}{t} \le 1.23\sqrt{\frac{kE}{F_{yc}}} \\ 181\ 000\, R_b R_h k\left(\frac{t}{w}\right)^2 & \text{for } \frac{w}{t} > 1.23\sqrt{\frac{kE}{F_{yc}}} \end{cases}$ $c = \dfrac{1.23 - \frac{w}{t}\sqrt{\frac{F_{yc}}{kE}}}{0.66}$ $k = \text{buckling coefficnt} = \begin{cases} \frac{8I_s}{wt^3} \le 4.0 & \text{for } n = 1 \\ \frac{14.3 I_x}{wt^3 n^4} \le 4.0 & \text{For } n = 2, 3, 4 \text{ or } 5 \end{cases}$
Compression flange without longitudinal stiffeners	Use above equations with the substitution of compression flange width between webs, b for w and buckling coefficient k taken as 4
Tension flange	$F_n = R_b R_h F_{yt}$

E = modulus of elasticity of steel
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
n = number of equally spaced longitudinal compression flange stiffeners
I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and taken at the base of the stiffener
R_b = load shedding factor, $R_b = 1.0$ — if either a longitudinal stiffener is provided or $2D_c / t_w \le \lambda_b \sqrt{E / f_c}$ is satisfied
R_h = hybrid factor; for homogeneous section, $R_h = 1.0$, see AASHTO-LRFD (6.10.5.4)
t_h = thickness of concrete haunch above the steel top flange
t = thickness of compression flange
w = larger of width of compression flange between longitudinal stiffeners or the distance from a web to the nearest longitudinal stiffener

13.6.2 Top Lateral Bracings

Steel composite box girders (Figure 13.3) are usually built of three steel sides and a composite concrete deck. Before the hardening of the concrete deck, the top flanges may be subject to lateral torsion buckling. Top lateral bracing shall be designed to resist shear flow and flexure forces in the section prior to curing of concrete deck. The need for top lateral bracing shall be investigated to ensure that deformation of the box is adequately controlled during fabrication, erection, and placement of the concrete deck. The cross-bracing shown in Figure 13.3 is desirable. For 45° bracing, a minimum cross-sectional area (mm^2) of bracing of 0.76× (box width, in mm) is required to ensure closed box action [11]. The slenderness ratio (L_b/r) of bracing members should be less than 140.

AASHTO-LRFD [1] requires that for straight box girders with spans less than about 45 m, at least one panel of horizontal bracing should be provided on each side of a lifting point; for spans greater than 45 m, a full-length lateral bracing system may be required.

13.6.3 Internal Diaphragms and Cross Frames

Internal diaphragms or cross frames (Figure 13.1) are usually provided at the end of a span and interior supports within the spans. Internal diaphragms not only provide warping restraint to the box girder, but improve distribution of live loads, depending on their axial stiffness which prevents distortion. Because rigid and widely spaced diaphragms may introduce undesirable large local forces, it is generally good practice to provide a large number of diaphragms with less stiffness than a few very rigid diaphragms. A recent study [18] showed that using only two intermediate diaphragms per span results in 18% redistribution of live-load stresses and additional diaphragms do not significantly improve the live-load redistribution. Inverted K-bracing provides better inspection access than X-bracing. Diaphragms shall be designed to resist wind loads, to brace compression flanges, and to distribute vertical dead and live loads [AASHTO-LRFD 6.7.4].

For straight box girders, the required cross-sectional area of a lateral bracing diagonal member A_b (mm^2) should be less than 0.76× (width of bottom flange, in mm) and the slenderness ratio (L_b/r) of the member should be less than 140.

For horizontally curved boxes per lane and radial piers under HS-20 loading, Eq. (13.1) provides diaphragm spacing L_d, which limits normal distortional stresses to about 10% of the bending stress [19]:

$$L_d = \sqrt{\frac{R}{200L - 7500}} \le 25 \tag{13.1}$$

where R is bridge radius, ft, and L is simple span length, ft.

To provide the relative distortional resistance per millimeter greater than 40 [13], the required area of cross bracing is as

$$A_b = 750\left[\frac{L_{ds}a}{h}\right]\left[\frac{t^3}{h+a}\right] \tag{13.2}$$

where t is the larger of flange and web thickness; L_{ds} is the diaphragm spacing; h is the box height, and a is the top width of box.

13.7 Other Considerations

13.7.1 Fatigue and Fracture

For steel structures under repeated live loads, fatigue and fracture limit states should be satisfied in accordance with AASHTO 6.6.1. A comprehensive discussion on the issue is presented in Chapter 53.

13.7.2 Torsion

Figure 13.4 shows a single box girder under the combined forces of bending and torsion. For a closed or an open box girder with top lateral bracing, torsional warping stresses are negligible. Research indicates that the parameter ψ determined by Eq. (13.3) provides limits for consideration of different types of torsional stresses.

$$\psi = L\sqrt{GJ/EC_w} \tag{13.3}$$

where G is shear modulus, J is torsional constant, and C_w is warping constant.

For straight box girder (ψ is less than 0.4), pure torsion may be omitted and warping stresses must be considered; when ψ is greater than 10, it is warping stresses that may be omitted and pure

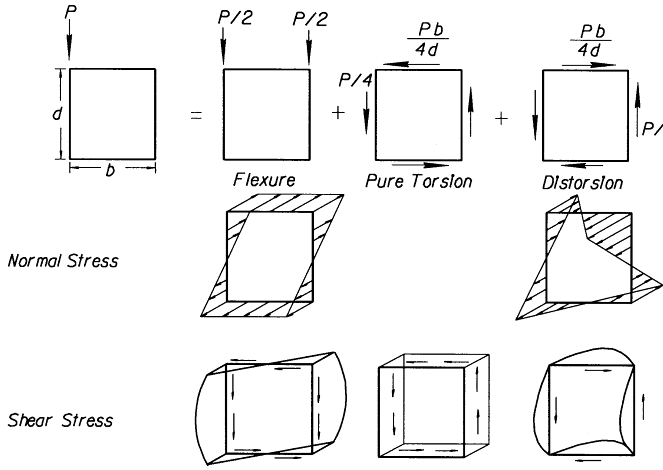

FIGURE 13.4 A box section under eccentric loads.

torsion that must be considered. For a curved box girder, ψ must take the following values if torsional warping is to be neglected:

$$\psi \geq \begin{cases} 10 + 40\theta & \text{for } 0 \leq \theta \leq 0.5 \\ 30 & \text{for } \theta > 0.5 \end{cases} \tag{13.4}$$

where θ is subtended angle (radius) between radial piers.

13.7.3 Constructibility

Box-girder bridges should be checked for strength and stability during various construction stages. It is important to note that the top flange of open-box sections shall be considered braced at locations where internal cross frames or top lateral bracing are attached. Member splices may be needed during construction. At the strength limit state, the splices in main members should be designed for not less than the larger of the following:

- The average of the flexure moment, the shear, or axial force due to the factored loading and corresponding factored resistance of member, and
- 75% of the various factored resistance of the member.

13.7.4 Serviceability

To prevent permanent deflections due to traffic loads, AASHTO-LRFD requires that at positive regions of flange flexure stresses (f_f) at the service limit state shall not exceed $0.95R_h F_{yf}$.

13.8 Design Example

Two-Span Continuous Box-Girder bridge

Given

A two-span continuous composite box-girder bridge that has two equal spans of 45 m. The superstructure is 13.2 m wide. The elevation and a typical cross section are shown in Figure 13.5.

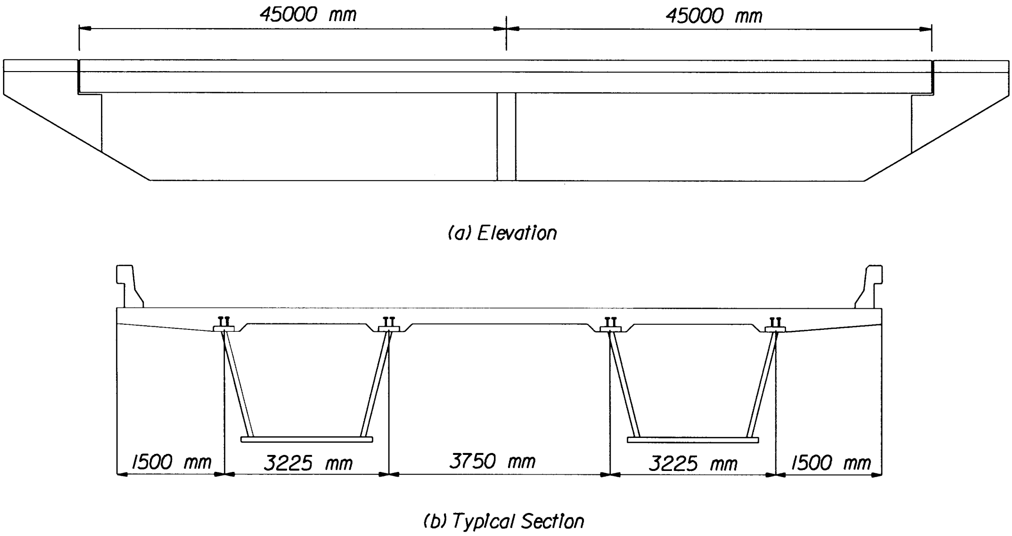

FIGURE 13.5 Two-span continuous box-girder bridge.

Structural steel:	AASHTO M270M, Grade 345W (ASTM A709 Grade 345W) uncoated weathering steel with $F_y = 345$ MPa
Concrete:	$f_c' = 30.0$ MPa; $E_c = 22{,}400$ MPa; modular ratio $n = 8$
Loads:	Dead load = self weight + barrier rail + future wearing 75 mm AC overlay Live load = AASHTO Design Vehicular Load + dynamic load allowance Single-lane average daily truck traffic ADTT in one direction = 3600
Deck:	Concrete slabs deck with thickness of 200 mm
Specification:	AASHTO-LRFD [1] and 1996 Interim Revision (referred to as AASHTO)
Requirements:	Design a box girder for flexure, shear for Strength Limit State I, and check fatigue requirement for web.

Solution

1. **Calculate Loads**
   a. *Component dead load* — DC *for a box girder*:
   The component dead-load DC includes all structural dead loads with the exception of the future wearing surface and specified utility loads. For design purposes, assume that all dead load is distributed equally to each girder by the tributary area. The tributary width for the box girder is 6.60 m.
   - *DC*1: acting on noncomposite section
     Concrete slab = (6.6)(0.2)(2400)(9.81) = 31.1 kN/m
     Haunch = 3.5 kN/m
     Girder (steel-box), cross frame, diaphragm, and stiffener = 9.8 kN/m
   - *DC*2: acting on the long term composite section
     Weight of each barrier rail = 5.7 kN/m

   b. *Wearing surface load* — *DW*:
   A future wearing surface of 75 mm is assumed to be distributed equally to each girder
   - *DW*: acting on the long-term composite section = 10.6 kN/m

2. **Calculate Live-Load Distribution Factors**
   a. *Live-load distribution factors for strength limit state* [AASHTO Table 4.6.2.2.2b-1]:

$$LD_m = 0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L} = 0.05 + 0.85\frac{3}{2} + \frac{0.425}{3} = 1.5 \text{ lanes}$$

b. *Live-load distribution factors for fatigue limit state*:

$$LD_m = 0.05 + 0.85\frac{N_L}{N_b} + \frac{0.425}{N_L} = 0.05 + 0.85\frac{1}{2} + \frac{0.425}{1} = 0.9 \text{ lanes}$$

3. **Calculate Unfactored Moments and Shear Demands**
The unfactored moment and shear demand envelopes are shown in Figures 13.8 to 13.11. Moment, shear demands for the Strength Limit State I and Fatigue Limit State are listed in Table 13.3 to 13.5.

TABLE 13.3 Moment Envelopes for Strength Limit State I

Span	Location (x/L)	M_{DC1} (kN-m); Dead Load-1	M_{DC2} (kN-m); Dead Load-2	M_{DW} (kN-m); Wearing Surface	M_{LL+IM} (kN-m)		M_u (kN-m)	
					Positive	Negative	Positive	Negative
	0.0	0	0	0	0	0	0	0
	0.1	3,058	372	681	3338	–442	**10,592**	4,307
	0.2	5,174	629	1152	5708	–883	**18,023**	7,064
	0.3	6,350	772	1414	7174	–1326	**22,400**	8,268
	0.4	6,585	801	1466	7822	–1770	**23,864**	7,917
1	0.5	5,880	715	1309	7685	–2212	**22,473**	6,018
	0.6	4,234	515	943	6849	–2653	**18,369**	2,571
	0.7	1,647	200	367	5308	–3120	**11,540**	–2,472
	0.8	–1,882	–229	–419	3170	–3822	2,168	**–9,457**
	0.9	–6,350	–772	–1414	565	–4928	–9,533	**–18,745**
	1.0	–11,760	–1430	–2618	–1727	–7640	–22,264	**–32,095**

Notes:
1. Live load distribution factor $LD = 1.467$.
2. Dynamic load allowance $IM = 33\%$.
3. $M_u = 0.95\ [1.25(M_{DC1} + M_{DC2}) + 1.5\ M_{DW} + 1.75\ M_{LL+IM}]$.

TABLE 13.4 Shear Envelopes for Strength Limit State I

Span	Location (x/L)	V_{DC1} (kN); Dead Load-1	V_{DC2} (kN); Dead Load-2	V_{DW} (kN); Wearing Surface	V_{LL+IM} (kN)		V_u (kN)	
					Positive	Negative	Positive	Negative
	0.0	784	95	87	877	–38	**2626**	1104
	0.1	575	70	64	782	–44	**2158**	784
	0.2	366	44	41	711	–58	**1727**	449
	0.3	157	19	18	601	–91	**1233**	83
	0.4	–53	6	–6	482	–138	724	**–307**
1	0.5	–262	–32	–29	360	–230	208	**–773**
	0.6	–471	–57	–52	292	–354	–216	**–1290**
	0.7	–680	–83	–76	219	–482	–648	**–1815**
	0.8	–889	–108	–99	145	–612	–1083	**–2342**
	0.9	–1098	–133	–122	67	–750	–1524	**–2882**
	1.0	–1307	–159	–145	22	–966	–1910	**–3553**

Notes:
1. Live load distribution factor $LD = 1.467$.
2. Dynamic load allowance $IM = 33\%$.
3. $V_u = 0.95\ [1.25(V_{DC1} + V_{DC2}) + 1.5V_{DW} + 1.75V_{LL+IM}]$.

TABLE 13.5 Moment and Shear Envelopes for Fatigue Limit State

Span	Location (x/L)	M_{LL+IM} (kN-m) Positive	M_{LL+IM} (kN-m) Negative	V_{LL+IM} (kN) Positive	V_{LL+IM} (kN) Negative	$(M_{LL+IM})_u$ (kN-m) Positive	$(M_{LL+IM})_u$ (kN-m) Negative	$(V_{LL+IM})u$ (kN) Positive	$(V_{LL+IM})u$ (kN) Negative
	0.0	0	0	286	–31	0	0	214	–23
	0.1	1102	–137	245	–31	827	–102	184	–23
	0.2	1846	–274	205	–57	1385	–206	154	–38
	0.3	2312	–412	167	–79	1734	–309	125	–59
	0.4	2467	–550	130	–115	1851	–412	98	–86
1	0.5	2405	–687	97	–153	1804	–515	73	–115
	0.6	2182	–824	67	–190	1636	–618	50	–143
	0.7	1716	–962	45	–226	1287	–721	33	–169
	0.8	1062	–1099	25	–257	1796	–824	19	–193
	0.9	414	–1237	9	–286	311	–928	7	–215
	1.0	0	–1373	0	–309	0	–1030	0	–232

Notes:
1. Live load distribution factor $LD = 0.900$.
2. Dynamic load allowance $IM = 15\%$.
3. $(M_{LL+IM})_u = 0.75(M_{LL+IM})_u$ and $(V_{LL+IM})_u = 0.75(V_{LL+IM})_u$.

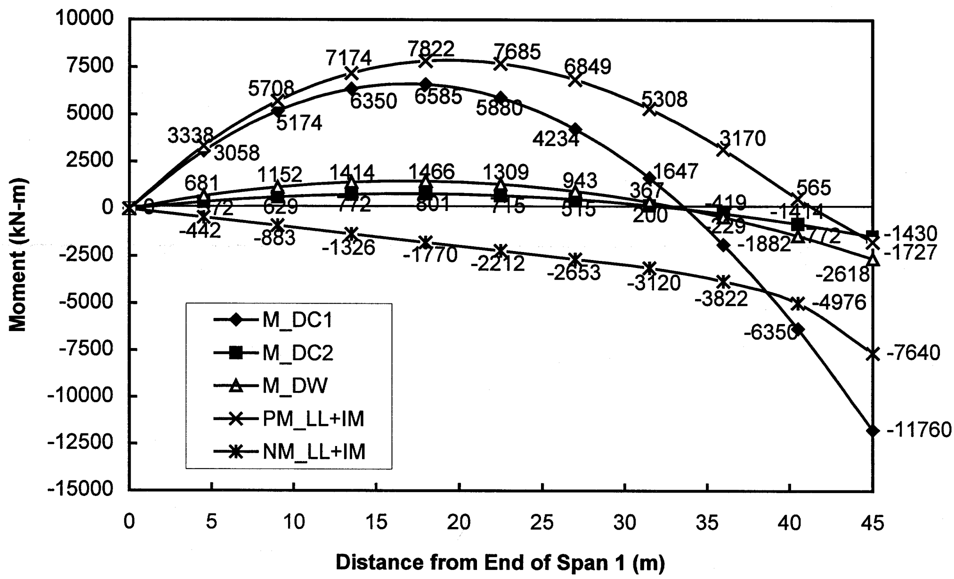

FIGURE 13.6 Unfactored moment envelopes.

4. Determine Load Factors for Strength Limit State I and Fracture Limit State

Load factors and load combinations

The load factors and combinations are specified as [AASHTO Table 3.4.1-1]:

Strength Limit State I: $1.25(DC1 + DC2) + 1.5(DW) + 1.75(LL + IM)$

Fatigue Limit State: $0.75(LL + IM)$

a. *General design equation* [AASHTO Article 1.3.2]:

$$\eta \sum \gamma_i Q_i \leq \phi R_n$$

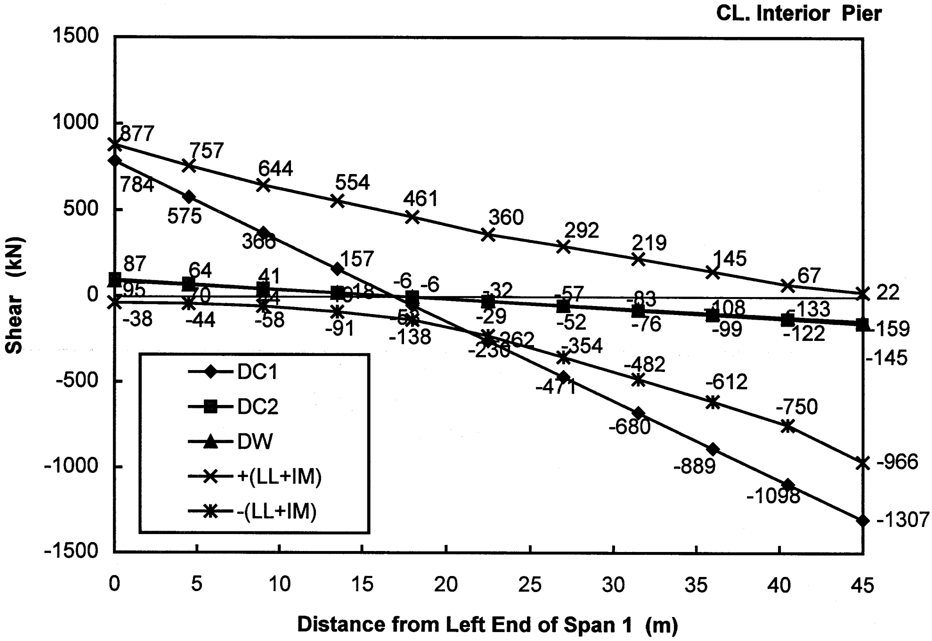

FIGURE 13.7 Unfactored shear envelopes.

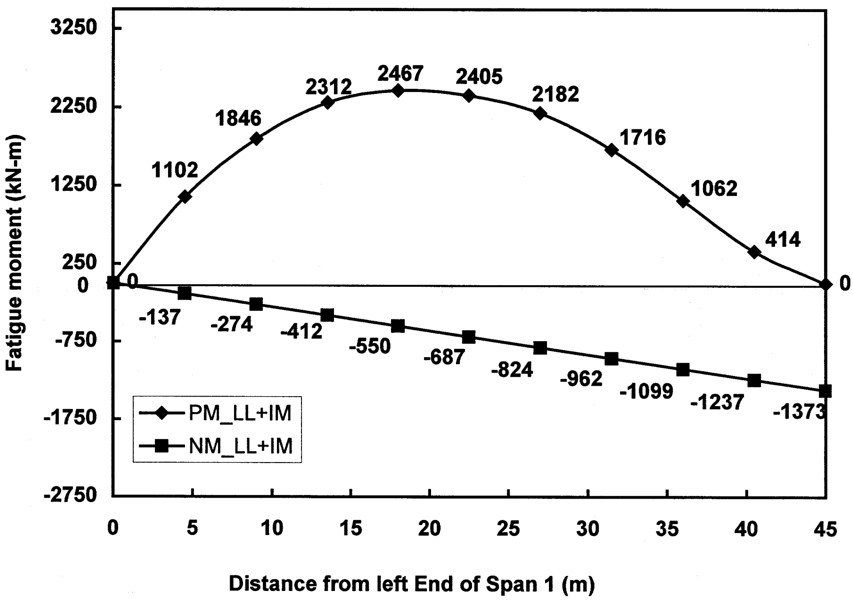

FIGURE 13.8 Unfactored fatigue load moment.

where γ_i is load factor and ϕ resistance factor; Q_i represents force effects or demands; R_n is the nominal resistance; η is a factor related ductility η_D , redundancy η_R , and operational importance η_I of the bridge (see Chapter 5) designed and is defined as:

$$\eta = \eta_D \eta_R \eta_I \geq 0.95$$

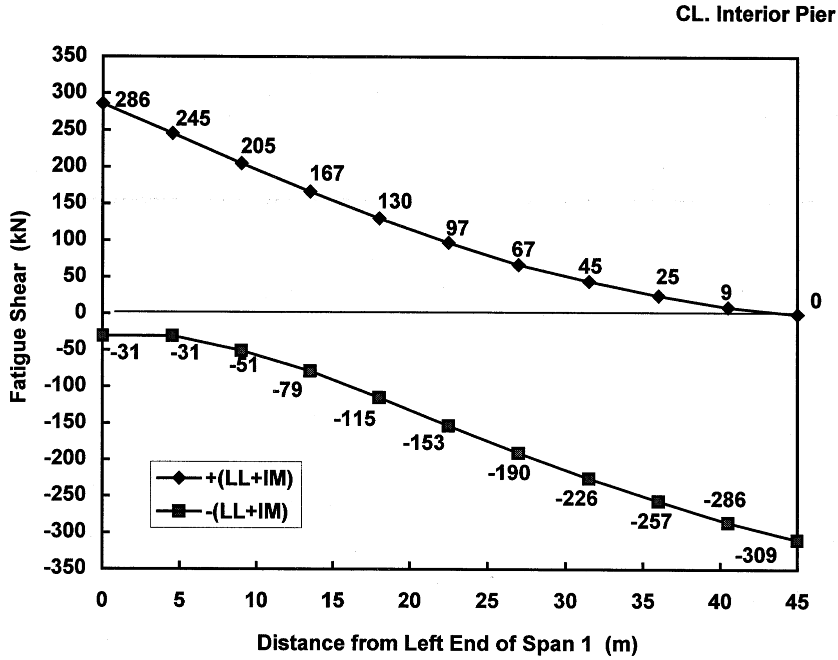

FIGURE 13.9 Unfactored fatigue load shear.

For this example, the following values are assumed:

Limit States	Ductility η_D	Redundancy η_R	Importance η_I	η
Strength limit state	0.95	0.95	1.05	0.95
Fatigue limit state	1.0	1.0	1.0	1.0

5. **Calculate Composite Section Properties**:
Effective flange width for positive flexure region [AASHTO Article 4.6.2.6]
a. For an interior web, the effective flange width:

$$b_{\text{eff}} = \text{the lesser of} \begin{cases} \dfrac{L_{\text{eff}}}{4} = \dfrac{33750}{4} = 8440 \text{ mm} \\ 12t_s + \dfrac{b_f}{2} = (12)(200) + \dfrac{450}{2} = 2625 \text{ mm} \qquad \text{(controls)} \\ S = 3750 \text{ mm} \end{cases}$$

b. For an exterior web, the effective flange width:

$$b_{\text{eff}} = \text{the lesser of} \begin{cases} \dfrac{L_{\text{eff}}}{8} = \dfrac{33750}{8} = 4220 \text{ mm} \\ 6t_s + \dfrac{b_f}{4} = (6)(200) + \dfrac{450}{4} = 1310 \text{ mm} \qquad \text{(controls)} \\ \textit{The width of the overhang} = 1500 \text{ mm} \end{cases}$$

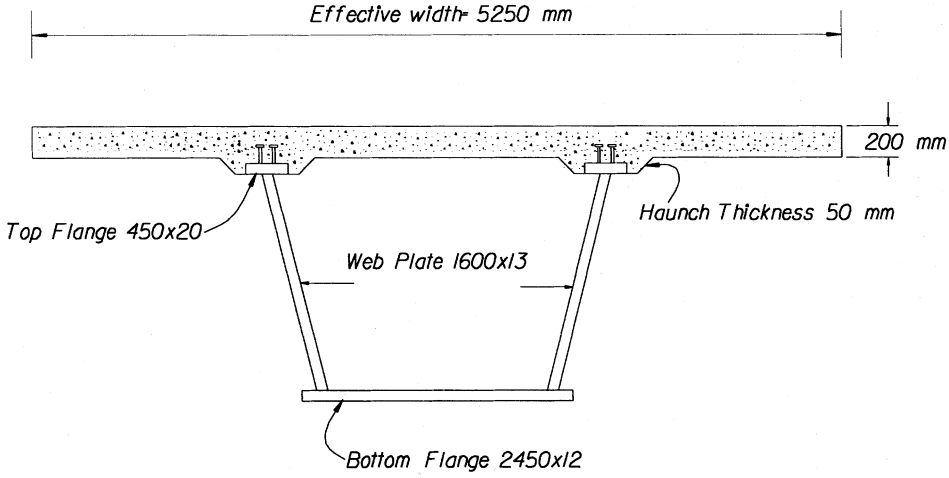

FIGURE 13.10 Typical section for positive flexure region.

$$\textit{Total effective flange width for the box girder} = 1310 + \frac{2625}{2} + 2625 = 5250 \text{ mm}$$

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; b_f is top flange width of steel girder.

Elastic composite section properties for positive flexure region:
For a typical section (Figure 13.10) in positive flexure region of Span 1, its elastic section properties for the noncomposite, the short-term composite ($n = 8$), and the long-term composite ($3n = 24$) are calculated in Tables 13.6 to 13.8.

TABLE 13.6 Noncomposite Section Properties for Positive Flexure Region

Component	A (mm^2)	y_i (mm)	A_iy_i (mm^3)	$y_i - y_{sb}$ (mm)	$A_i(y_i - y_{sb})^2$ (mm^4)	I_o (mm^4)
2 top flange 450 × 20	18,000	1574.2	28.34 (10^6)	885	141 (10^9)	0.60 (10^6)
2 web 1600 × 13	41,600	788.1	32.79 (10^6)	99	0.41 (10^9)	8.35 (10^9)
Bottom flange 2450 × 12	29,400	6.0	0.17 (10^6)	–683	13.70 (10^9)	0.35 (10^6)
Σ	89,000	—	61.30 (10^6)	—	28.23 (10^9)	8.35 (10^9)

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{61.30(10^6)}{89{,}000} = 688.7 \text{ mm} \qquad y_{st} = (12 + 1552.5 + 20) - 688.7 = 895.5 \text{ mm}$$

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

$$= 8.35(10^9) + 28.23(10^9) = 36.58(10^9) \text{ mm}^4$$

$$S_{sb} = \frac{I_{\text{girder}}}{y_{sb}} = \frac{36.58(10^9)}{688.7} = 53.11(10^6)\text{mm}^3 \qquad S_{st} = \frac{I_{\text{girder}}}{y_{st}} = \frac{36.58(10^9)}{895.5} = 40.85(10^6)\text{ mm}^3$$

Effective flange width for negative flexure region:
The effective width is computed according to AASHTO 4.6.2.6 (calculations are similar to Step 5a) The total effective of flange width for the negative flexure region is 5450 mm.

TABLE 13.7 Short-Term Composite Section Properties ($n = 8$)

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 (mm⁴)
Steel section	89,000	688.7	61.30 (10⁶)	–611	33.24 (10⁹)	36.58 (10⁹)
Concrete Slab 5250/8 × 200	131,250	1714.2	225.0 (10⁶)	414	22.54 (10⁹)	0.43 (10⁹)
Σ	220,250	—	386.3 (10⁶)	—	55.77 (10⁹)	37.02 (10⁹)

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{92.79(10^6)}{220\ 250} = 1299.8 \text{ mm} \qquad y_{st} = (12 + 1552.5 + 20) - 1299.8 = 284.4 \text{ mm}$$

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

$$= 37.02(10^9) + 55.77(10^9) = 92.79(10^9) \text{ mm}^4$$

$$S_{sb} = \frac{I_{com}}{y_{sb}} = \frac{92.79(10^9)}{1299.8} = 71.39(10^6) \text{ mm}^3 \qquad S_{st} = \frac{I_{com}}{y_{st}} = \frac{92.79(10^9)}{284.4} = 326.30(10^6) \text{ mm}^3$$

TABLE 13.8 Long-Term Composite Section Properties ($3n = 24$)

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 (mm⁴)
Steel section	89,000	688.4	61.3 (10⁶)	–338	10.2 (10⁹)	36.58 (10⁹)
Concrete slab 5250/24 × 200	43,750	1714.2	75.0 (10⁶)	688	20.7 (10⁹)	5.40 (10⁶)
Σ	132,750	—	136.0 (10⁶)	—	30.85 (10⁹)	36.59 (10⁹)

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{136.0(10^6)}{132\ 750} = 1026.7 \text{ mm} \qquad y_{st} = (12 + 1552.5 + 20) - 1026.7 = 557.5 \text{ mm}$$

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

$$= 36.59(10^9) + 136.0(10^9) = 67.43(10^9) \text{ mm}^4$$

$$S_{sb} = \frac{I_{com}}{y_{sb}} = \frac{67.43(10^9)}{1026.7} = 65.68(10^6) \text{ mm}^3 \qquad S_{st} = \frac{I_{com}}{y_{st}} = \frac{67.43(10^9)}{557.5} = 121.0(10^6) \text{ mm}^3$$

Elastic composite section properties for negative flexure region:
AASHTO (6.10.1.2) requires that for any continuous span the total cross-sectional area of longitudinal reinforcement must not be less than 1% of the total cross-sectional area of the slab. The required reinforcement must be placed in two layers uniformly distributed across the slab width and two thirds must be placed in the top layer. The spacing of the individual bar should not exceed 150 mm in each row.

$$A_{s\ \text{reg}} = 0.01(200) = 2.00 \text{ mm}^2/\text{mm}$$

$$A_{s\ \text{top-layer}} = \frac{2}{3}(0.01)(200) = 1.33 \text{ mm}^2/\text{mm} \ \ (\#16 \text{ at } 125 \text{ mm} = 1.59 \text{ mm}^2/\text{mm})$$

$$A_{s\ \text{bot-layer}} = \frac{1}{3}(0.01)(200) = 0.67 \text{ mm}^2/\text{mm} \ \ (\text{alternate } \#10 \text{ and } \#13 \text{ at } 125 \text{ mm} = 0.80 \text{ mm}^2/\text{mm})$$

Figure 13.11 shows a typical section for the negative flexure region. The elastic properties for the noncomposite and the long-term composite ($3n = 24$) are calculated and shown in Tables 13.9 and 13.10.

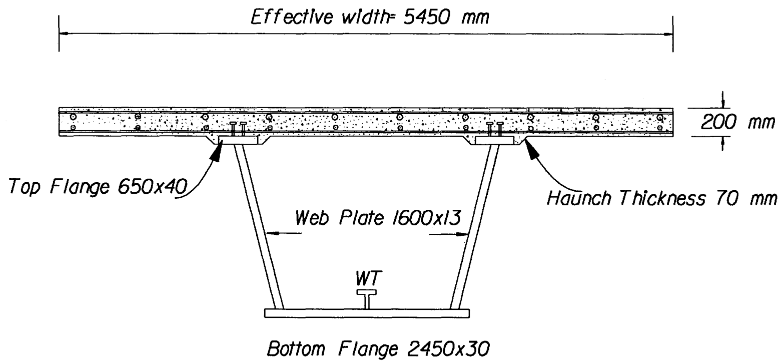

FIGURE 13.11 Typical section for negative flexure region.

TABLE 13.9 Noncomposite Section Properties for Negative 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 (mm⁴)
2 Top flange 650 × 40	52,000	1602	83.32 (10⁶)	911	43.20 (10⁹)	6.93 (10⁶)
2 Web 1600 × 13	41,600	806	33.53 (10⁶)	115	0.55 (10⁹)	8.35 (10⁹)
Stiffener WT	5,400	224.3	1.21 (10⁶)	–466	1.18 (10⁹)	37.63 (10⁶)
Bottom flange 2450 × 30	73,500	15	1.10 (10⁶)	–676	33.57 (10⁹)	5.51 (10⁶)
Σ	172,500	—	119.2 (10⁶)		78.49 (10⁹)	8.40 (10⁹)

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{119.2(10^6)}{172500} = 690.8 \text{ mm} \qquad y_{st} = (30 + 1552.5 + 40) - 690.8 = 931.4 \text{ mm}$$

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

$$= 8.40(10^9) + 78.50(10^9) = 86.90(10^9) \text{ mm}^4$$

$$S_{sb} = \frac{I_{\text{girder}}}{y_{sb}} = \frac{86.90(10^9)}{690.8} = 125.8(10^6) \text{ mm}^3 \qquad S_{st} = \frac{I_{\text{girder}}}{y_{st}} = \frac{86.90(10^9)}{931.4} = 93.29(10^6) \text{ mm}^3$$

TABLE 13.10 Composite Section Properties for Negative 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 (mm⁴)
Steel section	172 500	690.8	119.2 (10⁶)	73.2	0.92 (10⁹)	86.90 (10⁹)
Top reinforcement	8,665	1762.2	15.27 (10⁶)	998.2	8.63 (10⁹)	—
Bottom reinforcement	4,360	1677.2	7.31 (10⁹)	913.2	3.64 (10⁹)	—
Σ	185 525	—	141.7 (10⁶)	—	13.19 (10⁹)	86.90 (10⁹)

$$y_{sb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{141.7(10^6)}{185\,525} = 764 \text{ mm} \qquad y_{st} = (30 + 1552.5 + 40) - 764 = 858.2 \text{ mm}$$

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

$$= 86.90(10^9) + 13.19(10^9) = 100.09(10^9) \text{ mm}^4$$

$$S_{sb} = \frac{I_{com}}{y_{sb}} = \frac{100.09(10^9)}{764} = 131.00(10^6) \text{ mm}^3 \qquad S_{st} = \frac{I_{com}}{y_{st}} = \frac{100.09(10^9)}{858.2} = 116.63(10^6) \text{mm}^3$$

6. Calculate Yield Moment M_y and Plastic Moment Capacity M_p

a. *Yield moment M_y* [AASHTO Article 6.10.5.1.2]:

The yield moment M_y corresponds to the first yielding of either steel flange. It is obtained by the following formula

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

where M_{D1}, M_{D2}, and M_{AD} are moments due to the factored loads applied to the steel, the long-term, and the short-term composite section, respectively. M_{AD} can be obtained by solving the equation:

$$F_y = \frac{M_{D1}}{S_s} + \frac{M_{D2}}{S_{3n}} + \frac{M_{AD}}{S_n}$$

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

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

$$M_{D1} = (0.95)(1.25)(M_{DC1}) = (0.95)(1.25)(6585) = 7820 \text{ kN-m}$$

$$M_{D2} = (0.95)(1.25M_{DC2} + 1.5M_{DW})$$

$$= (0.95)[1.25(801) + 1.5(1466)] = 3040 \text{ kN-m}$$

For the top flange:

$$M_{AD} = (329.3)10^{-3}\left((345)10^3 - \frac{7.820}{40.85(10)^{-3}} - \frac{3.040}{120(10)^{-3}}\right)$$

$$= 41.912(10)^3 \text{ kN-m}$$

For the bottom flange:

$$M_{AD} = (71.39)10^{-3}\left((345)10^3 - \frac{7.820}{53.11(10)^{-3}} - \frac{3.040}{64.68(10)^{-3}}\right)$$

$$= 10.814(10)^3 \text{ kN-m} \qquad \text{(control)}$$

$$M_y = 7820 + 3040 + 10814 = 21{,}674 \text{ kN-m}$$

b. *Plastic moment M_p* [AASHTO Article 6.1]:

The plastic moment M_p is determined using equilibrium equations. The reinforcement in the concrete slab is neglected in this example.

- Determine the location of the plastic neutral axis (PNA), $\bar{Y}$
  From the Equation listed in Table 12.4 and Figure 12.7.

$$P_s = 0.85 f'_c b_{\text{eff}} t_s = 0.85(30)(5250)(200) = 26,775 \text{ kN}$$

$$P_c = A_{fc} F_{yc} = 2(450)(20)(345) = 6,210 \text{ kN}$$

$$P_w = A_w F_{yw} = 2\ (1600)(13)(345) = 14,352 \text{ kN}$$

$$P_t = A_{ft} F_{yt} = 2450(12)(345) = 10,143 \text{ kN}$$

$$\because P_t + P_w + P_c = 10,143 + 14,352 + 6,210 = 30\ 705 \text{ kN} > P_s = 26,755 \text{ kN}$$

$\therefore$ PNA is located within the top flange of steel girder and the distance from the top of compression flange to the PNA, $\bar{Y}$ is

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

$$\bar{Y} = \frac{20}{2}\left(\frac{14,352 + 10,143 - 26,775}{6,210} + 1\right) = 6.3 \text{ mm}$$

- *Calculate M_p:*
  Summing all forces about the PNA, obtain:

$$M_p = \sum M_{\text{PNA}} = \frac{P_c}{2t_c}\left(\bar{Y}^2 + (t_c - \bar{Y})^2\right) + P_s d_s + P_w d_w + P_t d_t$$

where

$$d_s = \frac{200}{2} + 50 - 20 + 6.3 = 136.3 \text{ mm}$$

$$d_w = \frac{1552.5}{2} + 20 - 6.3 = 789.8 \text{ mm}$$

$$d_t = \frac{12}{2} + 1552.5 + 20 - 6.3 = 1571.9 \text{ mm}$$

$$M_p = \frac{6210}{2(20)}\left(6.3^2 + (20 - 6.3)^2\right) + (26,775)(136.3) + (14,352)(789.8) + (10,143)(1571.9)$$

$$M_p = 30,964 \text{ kN-m}$$

7. **Flexural Strength Design — Strength Limit State I:**
   a. *Positive flexure region*:
      - *Compactness of steel box girder*
        The compactness of a multiple steel boxes is controlled only by web slenderness. The purpose of the ductility requirement is to prevent permanent crushing of the concrete slab when the composite section approaches its plastic moment capacity. For this example, by referring to Figures 13.2 and 13.4, obtain:

$\frac{2D_{cp}}{t_w} \le 3.76\sqrt{\frac{E}{f_c}}$, PNA is within the top flange $D_{cp} = 0$, the web slenderness requirement is satisfied

$D_p = 200 + 50 - 20 + 6.3 = 236.3$ mm

(depth from the top of concrete deck to the PNA)

$$D' = \beta\left[\frac{d + t_s + t_h}{7.5}\right] \qquad \beta = 0.7 \text{ for } F_y = 345 \text{ MPa}$$

$$D' = 0.7\left(\frac{1552.5 + 12 + 200 + 50}{7.5}\right) = 169.3 \text{ mm}$$

$$\left(\frac{D_p}{D'}\right) = \left(\frac{236.3}{169.3}\right) = 1.4 \le 5 \text{ OK}$$

- *Calculate nominal flexure resistance, M_n* (see Table 12.2)

$$1 < \left(\frac{D_p}{D'}\right) = 1.4 < 5$$

$$M_n = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$$

$$M_n = \frac{5(30,964) - 0.85(21,674)}{4} + \frac{0.85(21,674) - (30,964)}{4}(1.4)$$

$$M_n = 28,960 \text{ kN-m} \ge 1.3(1.0)(21674) = 28,176 \text{ kN-m}$$

$\therefore \quad M_n = 28,176$ kN-m

From Table 13.3, the maximum factored positive moments in Span 1 occurred at the location of $0.4L_1$.

$$\eta\Sigma\gamma_i M_i \le \phi_f M_n$$

$$23,864 \text{ kN-m} < 1.0\ (28,176) \text{ kN-m} \qquad \text{OK}$$

b. *Negative flexure region*:
For multiple and single box sections, the nominal flexure resistance should be designed to meet provision AASHTO 6.11.2.1.3a (see Table 13.2)

i. Stiffener requirement [AASHTO 6.11.2.1-1]:
Use one longitudinal stiffener (Figure 13.11), try WT 10.5 × 28.5.
The projecting width, b_ℓ of the stiffener should satisfy:

$$b_\ell \le 0.48 t_p \sqrt{\frac{E}{F_{yc}}}$$

where

$t_p =$ the thickness of stiffener (mm)

$b_\ell =$ the projected width (mm)

$$b_\ell = \frac{267}{2} = 133.5\,\text{mm}$$

$$I_s = 33.4 \times 10^6 + 4748(190.1)^2 = 20.5 \times 10^6\ \text{mm}$$

$$133.5 \leq 0.48(16.5)\sqrt{\frac{2 \times 10^5}{345}} = 190\ \ \text{mm} \qquad \text{OK}$$

ii. Calculate buckling coefficient, k:

For $n = 1$

$$k = \left(\frac{8I_s}{Wt^3}\right)^{1/3} = \left(\frac{8(20.5 \times 10^6}{1225(24)^3}\right)^{1/3} = 2.13 < 4.0$$

iii. Calculate nominal flange stress (see Table 13.2):

$$0.57\sqrt{\frac{Ek}{F_{yc}}} = 0.57\sqrt{\frac{(2.13)(2)10^5}{345}} = 20.03$$

$$1.23\sqrt{\frac{Ek}{F_{yc}}} = 1.23\sqrt{\frac{(2.13)(2)10^5}{345}} = 43.22$$

$$20.03 < \frac{w}{t} = \frac{1225}{30} = 40.83 < 43.22$$

The nominal flexural resistance of compression flange is controlled by inelastic buckling:

$$F_{nc} = 0.592\ R_b R_h F_{yc}\left(1 + 0.687\sin\frac{c\pi}{2}\right)$$

$$c = \frac{1.23 - \frac{w}{t}\sqrt{\frac{F_{yc}}{kE}}}{0.66} = \frac{1.23 - 40.8\sqrt{\frac{345}{(3.9)2(10^5)}}}{0.66} = 0.56$$

Longitudinal stiffener is provided, $R_b = 1.0$, for homogenous plate girder $R_h = 1.0$:

$$F_{nc} = 0.592.(1.0)(1.0)(345)\left(1 + 0.687\sin\frac{(0.56)\pi}{2}\right) = 313.4\ \text{MPa}$$

For tension flange:

$$F_{nt} = R_b R_h R_{yt} = (1.0)(1.0)(345) = 345 \text{ MPa}$$

iv. Calculate M_{AD} at Interior Support

$$M_{D1} = (0.95)(1.25)(M_{DC1}) = (0.95)(1.25)(11760) = 13{,}965 \text{ kN-m}$$

$$M_{D2} = (0.95)(1.25M_{DC2} + 1.5M_{DW})$$

$$= (0.95)[1.25(1430) + 1.5(2618)] = 5428 \text{ kN-m}$$

$$M_{AD} = S_n \left(F_n - \frac{M_{D1}}{S_s} - \frac{M_{D2}}{S_n} \right)$$

$$M_{AD-\text{comp}} = (0.131)\left(312.4 \times 10^3 - \frac{13\,965}{0.1258} - \frac{5428}{0.131} \right)$$

$$= 20\,954 \text{ kN-m}$$

$$M_{AD-\text{tension}} = (0.1166)\left(345 \times 10^3 - \frac{13\,965}{9.33(10)^{-2}} - \frac{5428}{0.1166} \right)$$

$$= 17\,346 \text{ kN-m} \quad (\text{control})$$

- Calculate nominal flexure resistance, M_n:

$$M_n = 13{,}965 + 5{,}428 + 17{,}346 = 36{,}739 \text{ kN-m}$$

From Table 13.3, maximum factored negative moments occurred at the interior support

$$\eta \Sigma \gamma_i M_i \leq \phi_f M_n$$

$$\underline{32{,}095 \text{ kN-m} < 1.0\,(36{,}739) \text{ kN-m}} \qquad \text{OK}$$

8. Shear Strength Design — Strength Limit State I

a. *End bearing of Span 1*

- Nominal shear resistance V_n:

  For inclined webs, each web shall be designed for shear, V_{ui} due to factored loads taken as [AASHTO Article 6.11.2.2.1]

$$\therefore\ V_{ui} = \frac{V_u}{\cos\theta} = \frac{2626}{2\cos(14)} = 1353 \text{ kN} \quad (\text{per web})$$

where θ is the angle of the web to the vertical.

$$\because \quad \frac{D}{t_w} = \frac{1600}{13} = 123.1 > 3.07\sqrt{\frac{E}{F_{yw}}} = 3.07\sqrt{\frac{(2.0)10^5}{345}} = 73.9$$

$$\therefore V_n = \frac{4.55 t_w^3 E}{D} = \frac{4.55(13)^3(2.0)10^5}{1600} = 1249.5 \text{ kN}$$

$$\because \quad V_{ui} = 1353 \text{ kN} > \phi_v V_n = (1.0)\,(1249.5) \text{ kN}$$

$$\therefore \text{Stiffeners are required}$$

- V_n for end-stiffened web panel [AASHTO 6.10.7.3.3c]

$$V_n = CV_p$$

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

in which d_o is the spacing of transverse stiffeners

$$\text{For } d_o = 2400 \text{ mm} \quad \text{and} \quad k = 5 + \frac{5}{(2400/1600)^2} = 7.22$$

$$\because \quad \frac{D}{t_w} = 123.1 > 1.38\sqrt{\frac{Ek}{F_{yw}}} = 1.38\sqrt{\frac{200{,}000(7.22)}{345}} = 89.3$$

$$\because \quad C = \frac{152}{(123.1)^2}\sqrt{\frac{200{,}000(7.22)}{345}} = 0.65$$

$$V_p = 0.58 F_{yw} D t_w = 0.58(345)(1600)(13) = 4162 \text{ kN}$$

$$V_n = CV_p = 0.65(4162) = 2705 \text{ kN} > V_{ui} = 1353 \text{ kN} \qquad \text{OK}$$

b. *Interior support*:
- The maximum shear forces due to factored loads is shown in Table 13.4

$$V_u = \frac{3553}{2} = 1776.5 \text{ kN (per web)} > \phi_v V_n = (1.0)\,(1249.5) \text{ kN}$$

$\therefore$ Stiffeners are required for the web at the interior support.

c. *Intermediate transverse stiffener design*

The intermediate transverse stiffener consists of a plate welded to one of the web. The design of the first intermediate transverse stiffener is discussed in the following.

- Projecting Width b_t Requirements [AASHTO Article 6.10.8.1.2]

  To prevent local bucking of the transverse stiffeners, the width of each projecting stiffener shall satisfy these requirements:

$$\left\{\begin{matrix} 50 + \dfrac{d}{30} \\ 0.25 b_f \end{matrix}\right\} \le b_t \le \left\{\begin{matrix} 0.48 t_p \sqrt{\dfrac{E}{F_{ys}}} \\ 16 t_p \end{matrix}\right\}$$

where b_f is full width of steel flange and F_{ys} is specified minimum yield strength of stiffener. Try stiffener width, b_t = 180.0 mm.

$$b_t = 180 > \begin{cases} 50 + \dfrac{d}{30} = 50 + \dfrac{1600}{30} = 103.3 \text{ mm} \\ 0.25 b_f = 0.25(450) = 112.5 \text{ mm} \end{cases} \qquad \text{OK}$$

Try t_p = 16 mm

$$b_t = 180 < \begin{cases} 0.48 t_p \sqrt{\dfrac{E}{F_{ys}}} = 0.48(16)\sqrt{\dfrac{200\ 000}{345}} = 185 \text{ mm} \\ 16 t_p = 16(14) = 224 \text{ mm} \end{cases} \qquad \text{OK}$$

Use 180 mm × 16 mm transverse stiffener plates.

- Moment of inertia requirement [AASHTO Article 6.10.8.1.3]
  The purpose of this requirement is to ensure sufficient rigidity of transverse stiffeners to develop tension field in the web adequately.

$$I_t \ge d_o t_w^2 J$$

$$J = 2.5\left(\frac{D_p}{d_o}\right)^2 - 2.0 \ge 0.5$$

where I_t is the moment of inertia for the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the midthickness of the web for stiffener pairs and D_p is the web depth for webs without longitudinal stiffeners.

$$\because J = 2.5\left(\frac{1600}{2400}\right)^2 - 2.0 = -0.89 < 0.5 \qquad \because \underline{\text{Use } J = 0.5}$$

$$I_t = \frac{(180)^3(16)}{3} = 31.1(10)^6 \text{ mm}^4 > d_o t_w^2 J = (2400)(16)^3(0.5) = 4.9(10)^6 \text{ mm}^4 \quad \text{OK}$$

- Area Requirement [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.

$$A_s \ge A_{s\min} = \left(0.15 B D t_w (1-C)\frac{V_u}{\phi_v V_n} - 18 t_w^2\right)\left(\frac{F_{yw}}{F_{ys}}\right)$$

where $B = 1.0$ for stiffener pairs. From the previous calculation:

$C = 0.65$ $\quad F_{yw} = 345$ MPa $\quad F_{ys} = 345$ MPa

$V_u = 1313$ kN (per web) $\quad \phi_f V_n = 1249.5$ kN $\quad t_w = 13$ mm

$$A_s = (180)(16) = 2880 \text{ in.}^2$$

$$> A_{s\min} = \left(0.15(2.4)(1600)(13)(1-0.65)\frac{1313}{1249.5} - 18(13)^2\right)\left(\frac{345}{345}\right)$$

$$= -288\,\text{mm}^2$$

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

9. Fatigue Design — Fatigue and Fracture Limit State

a. *Fatigue requirements for web in positive flexure region* [AASHTO Article 6.10.4]:

The purpose of these requirements is to control out-of-plane flexing of the web due to flexure and shear under repeated live loadings. The repeated live load is taken as twice the factored fatigue load.

$$D_c = \frac{f_{DC1} + f_{DC2} + f_{DW} + f_{LL+IM}}{\dfrac{f_{DC1}}{y_{st}} + \dfrac{f_{DC2} + f_{DW}}{y_{st-3n}} + \dfrac{f_{LL+IM}}{y_{st-n}}} - t_{fc}$$

$$= \frac{\dfrac{M_{DC1}}{S_{st}} + \dfrac{M_{DC2} + M_{DW}}{S_{st-3n}} + \dfrac{2(M_{LL+IM})_u}{S_{st-n}}}{\dfrac{M_{DC1}}{I_{\text{girder}}} + \dfrac{M_{DC2} + M_{DW}}{I_{\text{com}-3n}} + \dfrac{2(M_{LL+IM})_u}{I_{\text{com}-n}}} - t_{fc}$$

$$D_c = \frac{\dfrac{6585}{40.9(10)^6} + \dfrac{(801+1466)}{121(10)^6} + \dfrac{2(2467)}{326.3(10)^6}}{\dfrac{6585}{36.58(10)^9} + \dfrac{(801+1466)}{67.43(10)^9} + \dfrac{2(2467)}{92.79(10)^9}} - 20$$

$$D_c = 710 \text{ mm}$$

$$\frac{2D_c}{t_w} = \frac{2(710)}{13(\cos 14)} = 113 < 5.76\sqrt{\frac{E}{F_{yc}}} = 5.76\sqrt{\frac{2(10)^5}{345}} = 137.2$$

$$\therefore \quad f_{cf} = F_{yw}$$

f_{cf} = maximum compression flexure stress in the flange due to unfactored permanent loads and twice the fatigue loading

$$f_{cf} = \frac{M_{DC1}}{S_{st}} + \frac{M_{DC2} + M_{DW}}{S_{st-3n}} + \frac{2\left(M_{LL+IM}\right)_4}{S_{st-n}} = 161 + 18.7 + 15.3$$

$= 195$ Mpa $< F_{yw} = 435$ MPa OK

References

1. AASHTO, *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C., 1994.
2. AASHTO, *AASHTO LRFD Bridge Design Specifications*, 1996 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C., 1996.
3. Price, K. D., Big Steel Boxes, in *National Symposium on Steel Bridge Construction*, Atlanta, 1993, 15-3.
4. Heins, C. P. and Hua, L. J., Proportioning of box girder bridges girder, *J. Struct. Div. ASCE,* 106(ST11), 2345, 1980.
5. Subcommittee on Box Girders of the ASCE-AASHTO Task Committee on Flexural Members, Progress report on steel box girder bridges, *J. Struct. Div. ASCE,* 97(ST4), 1971.
6. Heins, C. P., Box girder bridge design — state of the art, *AISC Eng. J.*, 15(4), 126, 1978.
7. Heins, C. P., Steel box girder bridges — design guides and methods, *AISC Eng. J.*, 20(3), 121, 1983.
8. Wolchuck, R., Proposed specifications for steel box girder bridges, *J. Struct. Div. ASCE,* 117(ST12), 2463, 1980.
9. Wolchuck, R., Design rules for steel box girder bridges, in *Proc. Int. Assoc, Bridge Struct. Eng.*, Zurich, 1981, 41.
10. Wolchuck, R., 1982. Proposed specifications for steel box girder bridges, Discussion, *J. Struct. Div. ASCE,* 108(ST8), 1933, 1982.
11. Seim, C. and Thoman, S., Proposed specifications for steel box girder bridges, discussion, *J. Struct. Div. ASCE,* 118(ST12), 2457, 1981.
12. AISC, *Highway Structures Design Handbook*, Vol. II, AISC Marketing, Inc., 1986.
13. Heins, C. P. and Hall, D. H., *Designer's Guide to Steel Box Girder Bridges*, Bethlehem Steel Corporation, Bethlehem, PA, 1981.
14. Wolchuck, R., Steel-plate deck bridges, in *Structural Engineering Handbook*, 3rd ed., Gaylord, E. H., Jr. and Gaylord, C. N., Eds., McGraw-Hill, New York, 1990, Sect. 113.
15. Kuzmanovic, B. and Graham, H. J., Shear lag in box girder, *J. Struct. Div. ASCE,* 107(ST9), 1701, 1981.
16. Balser, Strength of plate girders under combined bending and shear, *J. Struct. Div. ASCE,* 87(ST7), 181, 1971.
17. Balser, Strength of plate girders in shear, *J. Struct Div. ASCE,* 87(ST7), 151, 1971.
18. Foinquinos, R., Kuzmanovic, B., and Vargas, L. M., Influence of diaphragms on live load distribution in straight multiple steel box girder bridges, in *Building to Last, Proceedings of Structural Congress XV,* Kempner, L. and Brown, C. B., Eds., American Society of Civil Engineers, 1997, Vol. I., 89–93.
19. Olenik, J. C. and Heins, C. P., Diaphragms for curved box beam bridges, *J. Struct. Div.*, ASCE, 101(ST10), 1975.