Table 4-11. LRFD Interaction Check							
Opening No.	X_i, ft	Local Forces on Tee		LRFD Interaction Check			
		P_r, kips	M_{vr}, kip-in.	P_r/P_c	*Spec.* Eq. H1-1a	*Spec.* Eq. H1-1b	Interaction*
End	0.000	0.000	10.0	0.000	0.000	0.000	0.000
1	0.885	4.24	9.57	0.062	0.378	**0.387**	0.387
2	2.28	10.5	8.87	0.155	0.448	**0.407**	0.407
3	3.68	16.4	8.17	0.241	**0.511**	0.424	0.511
4	5.07	21.7	7.47	0.320	**0.566**	0.437	0.566
5	6.47	26.6	6.77	0.391	**0.615**	0.447	0.615
6	7.87	31.0	6.07	0.456	**0.657**	0.454	0.657
7	9.26	34.9	5.37	0.514	**0.691**	0.457	0.691
8	10.7	38.3	4.68	0.564	**0.719**	0.456	0.719
9	12.1	41.3	3.98	0.608	**0.739**	0.452	0.739
10	13.4	43.7	3.28	0.644	**0.753**	0.444	0.753
11	14.8	45.7	2.58	0.674	**0.759**	0.433	0.759
12	16.2	47.3	1.88	0.696	**0.758**	0.418	0.758
13	17.6	48.3	1.18	0.712	**0.751**	0.400	0.751
14	19.0	48.9	0.485	0.720	**0.736**	0.378	0.736
Bm. CL	20.0	49.0	0.000	0.722	**0.722**	0.361	0.722
						I_{max}:	**0.759**

* Reflects bold face value of controlling interaction equation.

Table 4-12. ASD Interaction Check							
Opening No.	X_i, ft	Local Forces on Tee		ASD Interaction Check			
		P_r, kips	M_{vr}, kip-in	P_r/P_c	*Spec.* Eq. H1-1a	*Spec.* Eq. H1-1b	Interaction*
End	0.000	0.000	7.35	0.000	0.000	0.000	0.000
1	0.885	3.10	7.02	0.069	0.417	**0.427**	0.427
2	2.28	7.71	6.51	0.171	0.494	**0.449**	0.449
3	3.68	12.0	6.00	0.265	**0.563**	0.468	0.563
4	5.07	15.9	5.49	0.351	**0.624**	0.482	0.624
5	6.47	19.4	4.97	0.430	**0.677**	0.493	0.677
6	7.87	22.6	4.46	0.502	**0.723**	0.500	0.723
7	9.26	25.5	3.95	0.565	**0.761**	0.503	0.761
8	10.7	28.0	3.42	0.622	**0.792**	0.502	0.792
9	12.1	30.2	2.90	0.670	**0.814**	0.497	0.814
10	13.4	32.0	2.43	0.707	**0.828**	0.489	0.828
11	14.8	33.5	1.91	0.740	**0.835**	0.477	0.835
12	16.2	34.6	1.40	0.765	**0.834**	0.461	0.834
13	17.6	35.3	0.882	0.782	**0.826**	0.440	0.826
14	19.0	35.8	0.367	0.792	**0.810**	0.416	0.810
Bm. CL	20.0	35.8	0.000	0.794	**0.794**	0.397	0.794
						I_{max}:	**0.835**

* Reflects bold face value of controlling interaction equation.

Table 4-13. ASD and LRFD Web Post Buckling Check							
		ASD			LRFD		
Post No.	X_i, ft	$T_{r(i)}$, kips	$T_{r(i+1)}$, kips	V_{ah}, kips	$T_{r(i)}$, kips	$T_{r(i+1)}$, kips	V_{uh}, kips
1	1.58	3.10	7.71	4.61	10.5	4.24	6.26
2	2.98	7.71	12.0	4.29	16.4	10.5	5.90
3	4.38	12.0	15.9	3.90	21.7	16.4	5.30
4	5.77	15.9	19.4	3.50	26.6	21.7	4.90
5	7.17	19.4	22.6	3.30	31.0	26.6	4.40
6	8.56	22.6	25.5	2.80	34.9	31.0	3.90
7	9.96	25.5	28.0	2.50	38.3	34.9	3.40
8	11.4	28.0	30.2	2.20	41.3	38.3	3.00
9	12.8	30.2	32.0	1.60	43.7	41.3	2.40
10	14.1	32.0	33.5	1.50	45.7	43.7	2.00
11	15.5	33.5	34.6	1.20	47.3	45.7	1.60
12	16.9	34.6	35.3	0.700	48.3	47.3	1.00
13	18.3	35.3	35.8	0.500	48.9	48.3	0.600
14	19.7	35.8	35.8	0.000	49.0	48.9	0.100
			Maximum	4.61		Maximum	6.26

$$C1 = 5.097 + 0.1464\left(\frac{D_o}{t_w}\right) - 0.00174\left(\frac{D_o}{t_w}\right)^2 \tag{3-33}$$

$$= 5.097 + 0.1464\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right) - 0.00174\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right)^2$$

$$= 7.54$$

$$C2 = 1.441 + 0.0625\left(\frac{D_o}{t_w}\right) - 0.000683\left(\frac{D_o}{t_w}\right)^2 \tag{3-34}$$

$$= 1.441 + 0.0625\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right) - 0.000683\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right)^2$$

$$= 2.70$$

$$C3 = 3.645 + 0.0853\left(\frac{D_o}{t_w}\right) - 0.00108\left(\frac{D_o}{t_w}\right)^2 \tag{3-35}$$

$$= 3.645 + 0.0853\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right) - 0.00108\left(\frac{12.3 \text{ in.}}{0.200 \text{ in.}}\right)^2$$

$$= 4.81$$

$$\frac{M_{allow}}{M_e} = C1\left(\frac{S}{D_o}\right) - C2\left(\frac{S}{D_o}\right)^2 - C3 \tag{3-36}$$

$$= 7.54\left(\frac{16.8 \text{ in.}}{12.3 \text{ in.}}\right) - 2.70\left(\frac{16.8 \text{ in.}}{12.3 \text{ in.}}\right)^2 - 4.81$$

$$= 0.450$$

The available flexural strength is:

LRFD	ASD
From Equation 3-37a, $\phi_b \left(\frac{M_{allow}}{M_e} \right) M_e = 0.90(0.450)(218 \text{ kip-in.})$ $= 88.3 \text{ kip-in.} > M_u = 34.6 \text{ kip-in.}$ **o.k.**	From Equation 3-37b, $\frac{M_{allow}}{M_e} \left(\frac{M_e}{\Omega_b} \right) = 0.450 \left(\frac{218 \text{ kip-in.}}{1.67} \right)$ $= 58.7 \text{ kip-in.} > M_a = 25.5 \text{ kip-in.}$ **o.k.**

Check horizontal and vertical shear

The available horizontal shear strength is calculated using AISC *Specification* Section J4.2.

LRFD	ASD
From Table 4-13, $V_{uh} = 6.26 \text{ kips}$ $\phi_v V_{n\text{-}horiz} = \phi_v 0.6 F_y (e t_w)$ (from *Spec*. Eq. J4-3) $= 0.6(50 \text{ ksi})[(4.50 \text{ in.})(0.200 \text{ in.})]$ $= 27.0 \text{ kips} > 6.26 \text{ kips}$ **o.k.**	From Table 4-13, $V_{ah} = 4.61 \text{ kips}$ $\frac{V_{n\text{-}horiz}}{\Omega_v} = \frac{0.6 F_y (e t_w)}{\Omega_v}$ (from *Spec*. Eq. J4-3) $= \frac{0.6(50 \text{ ksi})[(4.50 \text{ in.})(0.200 \text{ in.})]}{1.50}$ $= 18.0 \text{ kips} > 4.61 \text{ kips}$ **o.k.**

Check vertical shear at beam net section

From AISC *Specification* Section G3:

$$\frac{h}{t_w} = \frac{d_{t\text{-}net}}{t_w} = \frac{2.65 \text{ in.}}{0.200 \text{ in.}} = 13.3$$

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{1.2(29{,}000 \text{ ksi})}{50 \text{ ksi}}} = 29.0$$

Because $h/t_w < 1.10\sqrt{k_v E/F_y}$

$$C_{v2} = 1.0 \quad (\textit{Spec}. \text{ Eq. G2-9})$$

The available vertical shear strength at the net section is calculated using AISC *Specification* Equation G2-3.

LRFD	ASD
From Table 4-10, $V_u = 6.25 \text{ kips}$ From *Spec*. Eq. G2-3, $\phi V_{n\text{-}net} = \phi 0.6 F_y (2 d_{t\text{-}net} t_w) C_{v2}$ $= 1.00(0.6)(50 \text{ ksi})(2)(2.65 \text{ in.})(0.200 \text{ in.})(1.0)$ $= 31.8 \text{ kips} > 6.25 \text{ kips}$ **o.k.**	From Table 4-10, $V_a = 4.57 \text{ kips}$ From *Spec*. Eq. G2-3, $\frac{V_{n\text{-}net}}{\Omega_v} = \frac{0.6 F_y (2 d_{t\text{-}net} t_w) C_{v2}}{\Omega_v}$ $= \frac{0.6(50 \text{ ksi})(2)(2.65 \text{ in.})(0.200 \text{ in.})(1.0)}{1.50}$ $= 21.2 \text{ kips} > 4.57 \text{ kips}$ **o.k.**

Check vertical shear at beam gross section

From AISC *Specification* Section G2.1(b)(1)

$$\frac{h}{t_w} = \frac{17.6 \text{ in.} - 2(0.525 \text{ in.})}{0.200 \text{ in.}}$$
$$= 82.8$$

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{5.34(29{,}000 \text{ ksi})}{50 \text{ ksi}}}$$
$$= 61.2$$

Because $h/t_w > 61.2$,

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad \text{(Spec. Eq. G2-4)}$$
$$= \frac{1.10\sqrt{5.34(29{,}000 \text{ ksi})/50 \text{ ksi}}}{82.8}$$
$$= 0.739$$

From AISC *Specification* Section G1, because $h/t_w > 2.24\sqrt{E/F_y} = 53.9$

$\phi_v = 0.90$ (LRFD) $\qquad \Omega_v = 1.67$ (ASD)

LRFD	ASD
From Table 4-10, $V_u = 6.54$ kips From *Spec*. Eq. G2-1, $\phi_v V_{n\text{-}gross} = \phi_v 0.6F_y\left(d_g t_w\right)C_{v1}$ $= 0.90(0.6)(50 \text{ ksi})(17.6 \text{ in.})(0.200 \text{ in.})(0.739)$ $= 70.2 \text{ kips} > 6.54 \text{ kips}$ **o.k.**	From Table 4-10, $V_a = 4.78$ kips From *Spec*. Eq. G2-1, $\dfrac{V_{n\text{-}gross}}{\Omega_v} = \dfrac{0.6F_y\left(d_g t_w\right)C_{v1}}{\Omega_v}$ $= \dfrac{0.6(50 \text{ ksi})(17.6 \text{ in.})(0.200 \text{ in.})(0.739)}{1.67}$ $= 46.7 \text{ kips} > 4.78 \text{ kips}$ **o.k.**

The following is a summary of the beam shear strengths:

LRFD	ASD
Horizontal shear $V_{uh}/\phi_v V_{n\text{-}horiz} = 6.26 \text{ kips}/27.0 \text{ kips}$ $= 0.232 < 1.0$ **o.k.**	*Horizontal shear* $V_{ah}\Omega_v/V_{n\text{-}horiz} = 4.61 \text{ kips}/18.0 \text{ kips}$ $= 0.256 < 1.0$ **o.k.**
Vertical shear—net section $V_u/\phi_v V_{n\text{-}net} = 6.25 \text{ kips}/31.8 \text{ kips}$ $= 0.197 < 1.0$ **o.k.**	*Vertical shear—net section* $V_a\Omega_v/V_{n\text{-}net} = 4.57 \text{ kips}/21.2 \text{ kips}$ $= 0.216 < 1.0$ **o.k.**

LRFD	ASD
Vertical shear—gross section $V_u/\phi_v V_{n\text{-}gross} = 6.54 \text{ kips}/70.2 \text{ kips} = 0.093 < 1.0$ **o.k.**	*Vertical shear—gross section* $V_a\Omega_v/V_{n\text{-}gross} = 4.78 \text{ kips}/46.7 \text{ kips} = 0.102 < 1.0$ **o.k.**

Check Deflection

Deflections are calculated using 90% of the moment of inertia as discussed in Section 3.7.

From AISC *Manual* Table 3-23, Case 1, the live load and dead load deflections are:

$$\Delta_{LL} = \frac{5wl^4}{384EI_{x\text{-}net}(0.90)}$$

$$= \frac{5(0.1 \text{ kip/ft})(1 \text{ ft/12 in.})[(40 \text{ ft})(12 \text{ in./ft})]^4}{384(29{,}000 \text{ ksi})(190 \text{ in.}^4)(0.90)}$$

$$= 1.16 \text{ in.}$$

$$= \frac{L}{410} < \frac{L}{240} \quad \textbf{o.k.}$$

$$\Delta_{DL} = \frac{5wl^4}{384EI_{x\text{-}net}(0.90)}$$

$$= \frac{5(0.139 \text{ kip/ft})(1 \text{ ft/12 in.})[(40 \text{ ft})(12 \text{ in./ft})]^4}{384(29{,}000 \text{ ksi})\left(190 \text{ in.}^4\right)(0.90)}$$

$$= 1.61 \text{ in.}$$

Because Δ_{DL} = 1.61 in., a 1½-in. camber is required.

Total load deflection is:

$$\Delta_{TL} = \Delta_{LL} + \Delta_{DL}$$

$$= 1.16 \text{ in.} + 1.61 \text{ in.}$$

$$= 2.77 \text{ in.}$$

$$= \frac{L}{172} > \frac{L}{180} \quad \textbf{n.g.}$$

This beam does not meet the deflection criteria. Either a larger section (LB18×16) should be considered, or the cutting pattern could be modified to increase the stiffness of the section.

Example 4.3—Composite Castellated Beam Design

Given:

A 50-ft-long floor beam with simple supports, shown in Figure 4-5, will be evaluated as a composite castellated section subject to uniform loading.

Beam span: 50 ft
Beam spacing: 8 ft
Trial beam: Asymmetric Section: W21×44 (top) + W21×57 (bottom) → CB30×44/57

Loading:	Live load = 100 psf Dead load = 75 psf (not including beam self-weight) Metal deck and concrete weight = 55 psf Total load = 800 lb/ft + 600 lb/ft + 51 lb/ft = 1,450 lb/ft
Deflection limits:	$L/360$ live load, $L/240$ total load
Bracing:	Beam is fully braced by concrete deck, L_b =0
Material:	ASTM A992
Metal deck:	depth = 2 in., rib width = 6 in., flutes perpendicular to beam
Studs:	diameter = ¾ in., height = 4 in., F_u = 65 ksi
Concrete:	f'_c = 3,000 psi, w_c = 145 lb/ft^3, t_c = 3 in. (5 in. total deck thickness)
Connections:	Assume that connections exist on either end to provide stability during construction (prior to deck being attached) and that the connections are sufficiently rigid to prevent web post buckling at the first web post on each end. Assume that the beam has been checked in its pre-composite stage for the wet concrete weight and construction loads.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Top Root Beam:
W21×44

A = 13.0 in.^{2}	d_{top} = 20.7 in.	t_w = 0.350 in.	b_f = 6.50 in.	t_f = 0.450 in.
S_x = 81.6 in.^{3}	Z_x = 95.4 in.^{3}	I_x = 843 in.^{4}		

Bottom Root Beam:
W21×57

A = 16.7 in.^{2}	d_{bot} = 21.1 in.	t_w = 0.405 in.	b_f = 6.56 in.	t_f = 0.650 in.
S_x = 111 in.^{3}	Z_x = 129 in.^{3}	I_x = 1,170 in.^{4}		

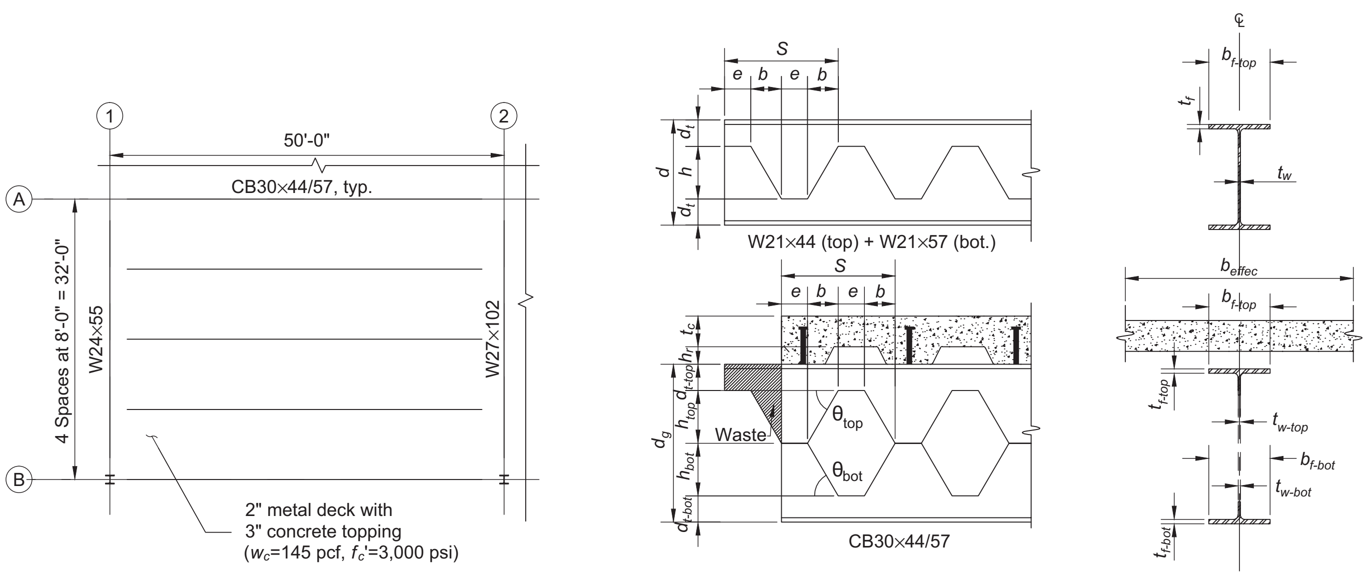

Fig. 4-5. Structural framing layout and composite castellated beam nomenclature for Example 4.3.

Resultant shape section properties for the CB30×44/57 are determined as follows:

The values of e, b and d_t are designated based on the depth of the root beam section and a trial opening size.

$$e = 8.00 \text{ in.}$$

$$b = 5.50 \text{ in.}$$

$$d_t = 5.50 \text{ in.}$$

$$\begin{aligned} h_{top} &= d_{top} - 2d_t \\ &= 20.7 \text{ in.} - 2(5.50 \text{ in.}) \\ &= 9.70 \text{ in.} \end{aligned} \quad \text{(from Eq. 4-1)}$$

$$\begin{aligned} h_{bot} &= d_{bot} - 2d_t \\ &= 21.1 \text{ in.} - 2(5.50 \text{ in.}) \\ &= 10.1 \text{ in.} \end{aligned} \quad \text{(from Eq. 4-1)}$$

$$\begin{aligned} h_o &= h_{top} + h_{bot} \\ &= 9.70 \text{ in.} + 10.1 \text{ in.} \\ &= 19.8 \text{ in.} \end{aligned} \quad \text{(from Eq. 4-2)}$$

$$\begin{aligned} d_g &= h_o + 2d_t \\ &= 19.8 \text{ in.} + 2(5.50 \text{ in.}) \\ &= 30.8 \text{ in.} \end{aligned} \quad \text{(4-3)}$$

$$\begin{aligned} \theta_{top} &= \tan^{-1}\left(\frac{h_{top}}{b}\right) \\ &= \tan^{-1}\left(\frac{9.70 \text{ in.}}{5.50 \text{ in.}}\right) \\ &= 60.4^\circ \end{aligned} \quad \text{(from Eq. 4-4)}$$

$$\begin{aligned} \theta_{bot} &= \tan^{-1}\left(\frac{h_{bot}}{b}\right) \\ &= \tan^{-1}\left(\frac{10.1 \text{ in.}}{5.50 \text{ in.}}\right) \\ &= 61.4^\circ \end{aligned} \quad \text{(from Eq. 4-4)}$$

$$\begin{aligned} S &= 2e + 2b \\ &= 2(8.00 \text{ in.}) + 2(5.50 \text{ in.}) \\ &= 27.0 \text{ in.} \end{aligned} \quad \text{(4-5)}$$

Calculate section properties of top and bottom tee and beam

Relevant cross sections are provided in Figure 4-6, and the section properties for the top and bottom tees are reported in Tables 4-14 and 4-15, respectively.

Beam net section properties

$$\begin{aligned} A_{net} &= A_{tee\text{-}top} + A_{tee\text{-}bot} \\ &= 4.70 \text{ in.}^2 + 6.22 \text{ in.}^2 \\ &= 10.9 \text{ in.}^2 \end{aligned} \quad \text{(3-7)}$$

$$\begin{aligned} \bar{y}_{bs} &= \frac{A_{tee\text{-}top}(d_t + h_o + \bar{y}_{tee\text{-}top}) + A_{tee\text{-}bot}\bar{y}_{tee\text{-}bot}}{A_{net}} \\ &= \frac{4.70 \text{ in.}^2 (5.50 \text{ in.} + 19.8 \text{ in.} + 4.24 \text{ in.}) + (6.22 \text{ in.}^2)(1.19 \text{ in.})}{10.9 \text{ in.}^2} \\ &= 13.4 \text{ in.} \end{aligned} \quad \text{(4-25)}$$

Table 4-14. Top Tee Section Properties at Center of Opening			
$A_{tee\text{-}top}$ = 4.70 in.^{2}	x = 5.14 in.	r_x = 1.61 in.	r_y = 1.48 in.
$\bar{y}_{tee\text{-}top}$ = 4.24 in.	$S_{x\text{-}top}$ = 9.63 in.^{3}	$S_{x\text{-}bot}$ = 2.86 in.^{3}	Z_x = 5.07 in.^{3}
$I_{x\text{-}tee\text{-}top}$ = 12.1 in.^{4}	I_y = 10.3 in.^{4}	J = 0.266 in.^{4}	y_o = 4.01 in.
Note: The fillet radius is assumed to be zero in the section properties calculations.			

Table 4-15. Bottom Tee Section Properties at Center of Opening			
$A_{tee\text{-}bot}$ = 6.22 in.^{2}	x = 0.475 in.	r_x = 1.51 in.	r_y = 1.57 in.
$\bar{y}_{tee\text{-}bot}$ = 1.19 in.	$S_{x\text{-}top}$ = 3.29 in.^{3}	$S_{x\text{-}bot}$ = 11.9 in.^{3}	Z_x = 5.95 in.^{3}
$I_{x\text{-}tee\text{-}bot}$ = 14.2 in.^{4}	I_y = 15.3 in.^{4}	J = 0.685 in.^{4}	y_o = 0.870 in.
Note: The fillet radius is assumed to be zero in the section properties calculations.			

$$\begin{aligned}\bar{y}_{ts} &= d_g - \bar{y}_{bs} \\ &= 30.8\text{ in.} - 13.4\text{ in.} \\ &= 17.4\text{ in.}\end{aligned} \tag{4-26}$$

$$\begin{aligned}d_{effec} &= d_g - \left[\left(d_t - \bar{y}_{tee\text{-}top}\right) + \bar{y}_{tee\text{-}bot}\right] \\ &= 30.8\text{ in.} - \left[(5.50\text{ in.} - 4.24\text{ in.}) + 1.19\text{ in.}\right] \\ &= 28.4\text{ in.}\end{aligned} \tag{4-27}$$

$$\begin{aligned}I_{x\text{-}net} &= I_{x\text{-}tee\text{-}top} + A_{tee\text{-}top}\left[\bar{y}_{ts} - \left(d_t - \bar{y}_{tee\text{-}top}\right)\right]^2 + I_{x\text{-}tee\text{-}bot} + A_{tee\text{-}bot}\left(\bar{y}_{bs} - \bar{y}_{tee\text{-}bot}\right)^2 \\ &= 12.1\text{ in.}^4 + \left(4.70\text{ in.}^2\right)\left[17.4\text{ in.} - (5.50\text{ in.} - 4.24\text{ in.})\right]^2 + 14.2\text{ in.}^4 + \left(6.22\text{ in.}^2\right)(13.4\text{ in.} - 1.19\text{ in.})^2 \\ &= 2{,}180\text{ in.}^4\end{aligned} \tag{4-28}$$

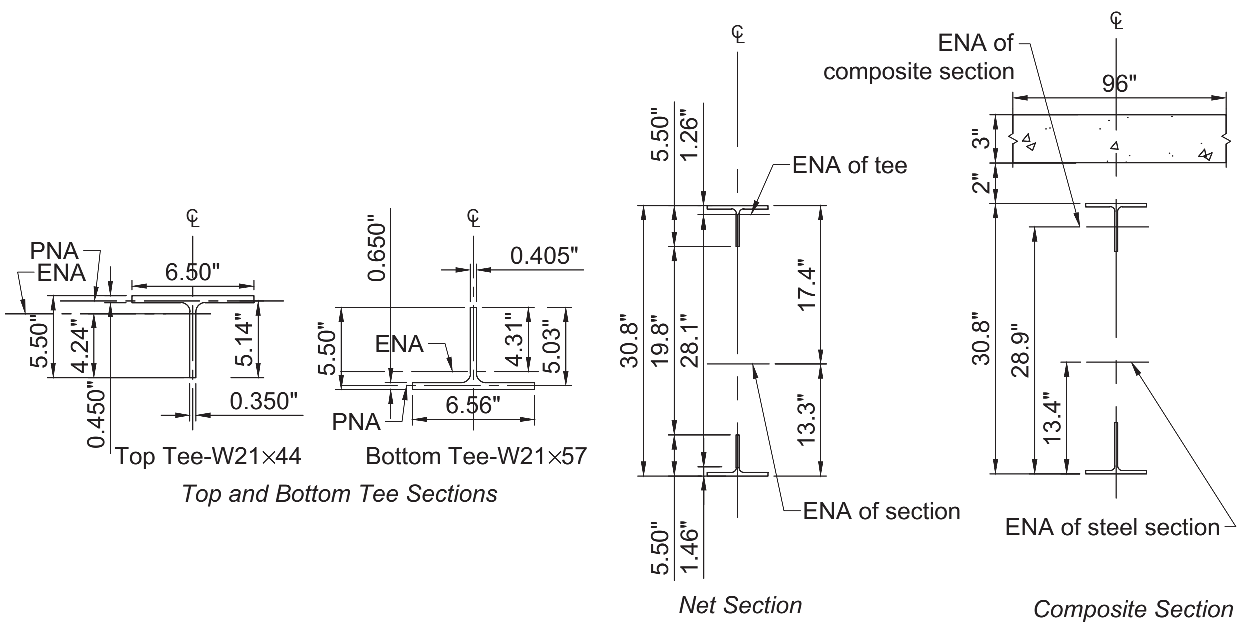

Fig. 4-6. Tee, net and composite sections for castellated beam for Example 4.3.

$$S_{x\text{-}net\text{-}top} = \frac{I_{x\text{-}net}}{\bar{y}_{ts}} \tag{4-29}$$

$$= \frac{2{,}180 \text{ in.}^4}{17.4 \text{ in.}}$$

$$= 125 \text{ in.}^3$$

$$S_{x\text{-}net\text{-}bot} = \frac{I_{x\text{-}net}}{\bar{y}_{bs}} \tag{4-30}$$

$$= \frac{2{,}180 \text{ in.}^4}{13.4 \text{ in.}}$$

$$= 163 \text{ in.}^3$$

Composite section properties in accordance with The Structural Engineer's Handbook *(Gaylord and Gaylord, 1992)*

$$n = \frac{E_s}{E_c} \tag{4-31}$$

$$= \frac{29{,}000{,}000 \text{ psi}}{33(145 \text{ pcf})^{1.5}\sqrt{3{,}000 \text{ psi}}}$$

$$= 9.19$$

$$b_{effec} = \min\{Span/4,\ Spacing\} \tag{3-4}$$

$$= \min\left\{\frac{50 \text{ ft}}{4}, \frac{8 \text{ ft} + 8 \text{ ft}}{2}\right\}(12 \text{ in./ft})$$

$$= 96.0 \text{ in.}$$

$$A_c = b_{effec}t_c \tag{4-32}$$

$$= (96.0 \text{ in.})(3.00 \text{ in.})$$

$$= 288 \text{ in.}^2$$

$$A_{ctr} = \frac{A_c}{n} \tag{4-33}$$

$$= \frac{288 \text{ in.}^2}{9.19}$$

$$= 31.3 \text{ in.}^2$$

$$K_c = \frac{A_{ctr}}{A_{ctr} + A_{net}} \tag{4-34}$$

$$= \frac{31.3 \text{ in.}^2}{31.3 \text{ in.}^2 + 10.9 \text{ in.}^2}$$

$$= 0.741$$

$$e_c = h_r + \frac{t_c}{2} \tag{4-35}$$

$$= 2.00 \text{ in.} + \frac{3.00 \text{ in.}}{2}$$

$$= 3.50 \text{ in.}$$

Assuming that the neutral axis is in the concrete.

$$y_{cc} = \left(\frac{A_{net} t_c}{A_{ctr}}\right)\left[\sqrt{1+\frac{2A_{ctr}}{A_{net} t_c}\left(\bar{y}_{ts} + e_c + \frac{t_c}{2}\right)} - 1\right] \tag{4-36}$$

$$= \left[\frac{\left(10.9 \text{ in.}^2\right)(3.00 \text{ in.})}{31.3 \text{ in.}^2}\right]\left[\sqrt{1+\frac{2\left(31.3 \text{ in.}^2\right)}{\left(10.9 \text{ in.}^2\right)(3.00 \text{ in.})}\left(17.4 \text{ in.} + 3.50 \text{ in.} + \frac{3.00 \text{ in.}}{2}\right)} - 1\right]$$

$$= 5.87 \text{ in.}$$

$$t_c + h_r = 3.00 \text{ in.} + 2.00 \text{ in.}$$

$$= 5.00 \text{ in.} < 5.87 \text{ in.}$$

Because $t_c + h_r < y_{cc}$, the neutral axis is in the steel.

$$\bar{y}_c = \left(\bar{y}_{ts} + e_c\right) K_c \tag{4-37}$$

$$= (17.4 \text{ in.} + 3.50 \text{ in.})0.741$$

$$= 15.5 \text{ in.}$$

$$I_{x\text{-}comp} = \left(\bar{y}_{ts} + e_c\right)\bar{y}_c A_{net} + I_{x\text{-}net} + \frac{A_{ctr} t_c^2}{12} \tag{4-38}$$

$$= (17.4 \text{ in.} + 3.50 \text{ in.})(15.5 \text{ in.})\left(10.9 \text{ in.}^2\right) + 2{,}180 \text{ in.}^4 + \frac{\left(31.3 \text{ in.}^2\right)(3.50 \text{ in.})^2}{12}$$

$$= 5{,}740 \text{ in.}^4$$

$$S_{x\text{-}comp\text{-}conc} = \frac{I_{x\text{-}comp}}{\bar{y}_{ts} - \bar{y}_c + e_c + 0.5t_c} \tag{4-39}$$

$$= \frac{5{,}740 \text{ in.}^4}{17.4 \text{ in.} - 15.5 \text{ in.} + 3.50 \text{ in.} + 0.5(3.00 \text{ in.})}$$

$$= 832 \text{ in.}^3$$

$$S_{x\text{-}comp\text{-}steel} = \frac{I_{x\text{-}comp}}{\bar{y}_{bs} + \bar{y}_c} \tag{4-40}$$

$$= \frac{5{,}740 \text{ in.}^4}{13.4 \text{ in.} + 15.5 \text{ in.}}$$

$$= 199 \text{ in.}^3$$

For the first iteration,

$$d_{effec\text{-}comp} = d_g - \bar{y}_{tee\text{-}bot} + h_r + 0.5t_c \tag{3-8}$$

$$= 30.8 \text{ in.} - 1.19 \text{ in.} + 2.00 \text{ in.} + 0.5(3.00 \text{ in.})$$

$$= 33.1 \text{ in.}$$

Check Vierendeel bending

The governing load cases are: