The required kicker axial strength is,

$$\begin{aligned} F_{uK} &= 1.2F_K \\ &= 1.2(6.02 \text{ kips}) \\ &= 7.22 \text{ kips} \end{aligned}$$

From AISC Manual Table 4-12, the available axial compression strength of the single angle, assume that the kicker has an effective length factor of $K = 1.0$ with the ends pinned is,

$\phi_c P_n = 9.82 \text{ kips} > F_{uK}$ **o.k.**

The deflection of the assembly at the center of the span is calculated as the vertical deflection at the HSS midspan assuming that it is simply supported at each kicker. The deflection at the center of the HSS span with point loads at one-third points of the span is derived from AISC Manual Table 3-23, case 9.

$$\begin{aligned} \Delta_{HSS} &= \frac{P_{cw} L_T^3}{28EI_x} + \frac{5w_{HSS} L_T^4}{384EI_x} \\ &= \frac{(1.40 \text{ kips})(10 \text{ ft})^3 (12 \text{ in./ft})^3}{28(29{,}000 \text{ ksi})(10.7 \text{ in.}^4)} \\ &\quad + \frac{5(0.01218 \text{ kip/ft})(10 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(10.7 \text{ in.}^4)} \\ &= 0.287 \text{ in.} \end{aligned}$$

Torsional rotation of the HSS at midspan is,

$$\begin{aligned} \theta_{HSS} &= \frac{T\left(\dfrac{L_T}{3}\right)}{GJ} \\ &= \frac{0.700 \text{ kip-ft}\left(\dfrac{10 \text{ ft}}{3}\right)(12 \text{ in./ft})^2}{11{,}200 \text{ ksi}(11.0 \text{ in.}^4)} \\ &= 0.00273 \text{ rad. (or } 0.156°) \end{aligned}$$

The vertical displacement of the HSS due to rotation of the HSS between kickers at midspan is,

$$\begin{aligned} \Delta_{HSS\theta} &= \theta_{HSS}\left(\frac{B}{2}\right) \\ &= 0.00273 \text{ rad.}\left(\frac{3 \text{ in.}}{2}\right) \\ &= 0.00409 \text{ in.} \end{aligned}$$

This deflection is small and has been neglected in the rest of this example.

The vertical deflection of the spandrel beam at the kicker (reduced from the AISC Manual Table 3-23, cases 1 and 9) is,

$$\begin{aligned} \Delta_s &= \frac{5P_K L^3}{162EI_x} + \frac{22(w_D + w_S)L^4}{1{,}944EI_x} \\ &= \frac{5(5.45 \text{ kips})(30 \text{ ft})^3 (12 \text{ in./ft})^3}{162(29{,}000 \text{ ksi})(800 \text{ in.}^4)} \\ &\quad + \frac{22(0.144 \text{ kip/ft} + 0.188 \text{ kip/ft})(30 \text{ ft})^4 (12 \text{ in./ft})^3}{1{,}944(29{,}000 \text{ ksi})(800 \text{ in.}^4)} \\ &= 0.565 \text{ in.} \end{aligned}$$

The axial shortening of the kicker due to the compression load is,

$$\begin{aligned} \Delta'_K &= \frac{F_K L'_K}{A_K E} \\ &= \frac{6.02 \text{ kips}(10.1 \text{ ft})(12 \text{ in./ft})}{2.09 \text{ in.}^2 (29{,}000 \text{ ksi})} \\ &= 0.0120 \text{ in.} \end{aligned}$$

This deflection is small and has been neglected in the rest of this example.

The service wind point load transmitted to the roof deck at the stiffener is calculated using the reaction at an interior support of a three span beam (AISC Manual, Table 3-22c),

$$\begin{aligned} P_{wind} &= \frac{11}{10} p_w \left(\frac{h}{2}\right) L_T \\ &= 1.1(0.030 \text{ kip/ft}^2)\left(\frac{14 \text{ ft}}{2}\right)(10 \text{ ft}) \\ &= 2.31 \text{ kips} \end{aligned}$$

Based on how much deck is tributary to each kicker, the effective width of the metal roof deck is,

$b_{eff} = 3$ ft

The horizontal elongation of the metal roof deck is,

$$\begin{aligned} \Delta_D &= \frac{(H_K + P_{wind}) s_{deck}}{b_{eff} t_{deck} E} \\ &= \frac{(5.96 \text{ kips} + 2.31 \text{ kips})(10 \text{ ft})}{3 \text{ ft}(0.0358 \text{ in.})(29{,}000 \text{ ksi})} \\ &= 0.0266 \text{ in.} \end{aligned}$$

Using Δ_D as the lateral translation of the top flange of the spandrel beam, the rotation of the spandrel beam at the kicker due to load in the roof deck is,

$$
\begin{aligned}
\theta_K &= \frac{\Delta_D}{d} \\
&= \frac{0.0266 \text{ in.}}{18.0 \text{ in.}} \\
&= 0.00149 \text{ rad. (or } 0.0850^\circ)
\end{aligned}
$$

The rigid body translation of the corner of the HSS due to rotation of the kicker assuming small angle rotation (where $\theta = \sin\theta$) is,

$$
\begin{aligned}
\Delta_K &= \theta_K l_{eod} \\
&= 0.00149 \text{ rad.} \left(15 \text{ in.}\right) \\
&= 0.0224 \text{ in.}
\end{aligned}
$$

Both Δ_D and Δ_K are small and will be neglected.

From Example 6.1, the vertical deflection is limited to 0.90 in. The total vertical displacement of the HSS at the center of its span is,

$$
\begin{aligned}
\Delta_{total} &= \Delta_{HSS} + \Delta_s \\
&= 0.287 \text{ in.} + 0.565 \text{ in.} \\
&= 0.852 \text{ in.} \le 0.90 \text{ in.} \quad \textbf{o.k.}
\end{aligned}
$$

For the welds from the roof deck to the spandrel beam and adjacent beams, try seven deck-to-beam puddle welds within the deck effective width ($n_{weld} = 7$). Note that this is more welding than the typical deck perimeter attachment. The weld size is $d_{weld} = ⅝$ in.

The factored shear per weld is,

$$
\begin{aligned}
V_{uw} &= \frac{1.2H_K + 0.8P_{wind}}{n_{weld}} \\
&= \frac{1.2\left(5.96 \text{ kips}\right) + 0.8(2.31 \text{ kips})}{7} \\
&= 1.29 \text{ kips}
\end{aligned}
$$

The weld shear strength in accordance with the Steel Deck Institute Diaphragm Design Manual (SDI, 2004), page 4-4 is,

$$
\begin{aligned}
\phi &= 0.70 \\
Q_f &= 2.2\, t_{deck} F_u \left(d_{weld} - t_{deck}\right) \\
&= 2.2\left(0.0358 \text{ in.}\right)\left(45 \text{ ksi}\right)\left(0.625 \text{ in.} - 0.0358 \text{ in.}\right) \\
&= 2.09 \text{ kips} \\
\phi Q_f &= 0.70\left(2.09 \text{ kips}\right) \\
&= 1.46 \text{ kips} > V_{uw}
\end{aligned}
$$

Seven puddle welds between the roof deck and the spandrel beam within a 3-ft width adjacent to the stiffeners and kickers will carry the forces from torsion and wind loads from the curtain wall into the roof deck. The same number of welds is also required at the adjacent roof framing beam (the first interior beam). The design of the stiffener plate was illustrated in Example 6.3.

Comments:

The edge HSS allows for a substantial eccentricity between the cladding and the spandrel framing without a substantial penalty on the weight of the framing. In this example, using kickers in place of slightly heavier roll beams has reduced the steel tonnage. However, the kickers require additional puddle welds between the roof deck and the spandrel and adjacent roof framing beams than might otherwise be required. The connections between the kicker and the adjacent beams are most often field-welded and these welds can add some cost to the project. The best choice between these various options depends upon finding the right mix of material costs and labor costs. Thus, it is ideal to discuss the options with the steel fabricator to determine which one will provide the least total cost.

Example 6.5—Floor Spandrel Beam with Eccentric Precast Panel Loads

The 30-ft-long spandrel beam shown in Figures 6-33 and 6-34 supports the floor framing shown in addition to 6-in.-thick precast concrete cladding panels weighing 75 psf. The panels are supported by steel brackets bearing on each floor slab 2 in. from the edge of the slab. The 6¼-in.-thick floor system consists of 3¼ in. of 3,000 psi lightweight concrete on 3-in. metal floor deck. The floor live load is 100 psf, and the superimposed (post-composite) dead load is 15 psf. The maximum vertical deflection due to superimposed loads is ½ in.

Design a bearing connection for the precast panel by evaluating the slab for flexural loads induced by the precast cladding. If the slab cannot resist the flexural loads, evaluate whether the bent-plate pour stop is capable of resisting the flexural loads.

Determine the total vertical deflection at the support points for the precast panels. Also, determine the adequacy of the spandrel beam including the effects of normal stresses due to torsional warping.

Given:

For this example assume the controlling load combination for strength is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection). The floor framing beams perpendicular to the spandrel provide some torsional restraint for the spandrel. The contribution of the composite floor system to the torsional stiffness of the spandrel is conservatively ignored. The horizontal eccentricity between the centroid of the precast panel and the edge of the pour stop is resolved by a couple between adjacent floors.

The spandrel beam is a W24×76 with a length of $L = 30$ ft for flexure and a length of $L_T = 10$ ft for torsion. The span of the perpendicular floor framing beams is $L_{fb} = 25$ ft with the spacing of the supports for the precast panels at $s_p = 10$ ft.

The post-composite properties of the spandrel beam are, $S_{eff} = 217$ in.^{3} and $I_{eff} = 3{,}410$ in.^{4} The pre-composite factored moment demand $M_{pre} = 93.0$ kip-ft (with a load

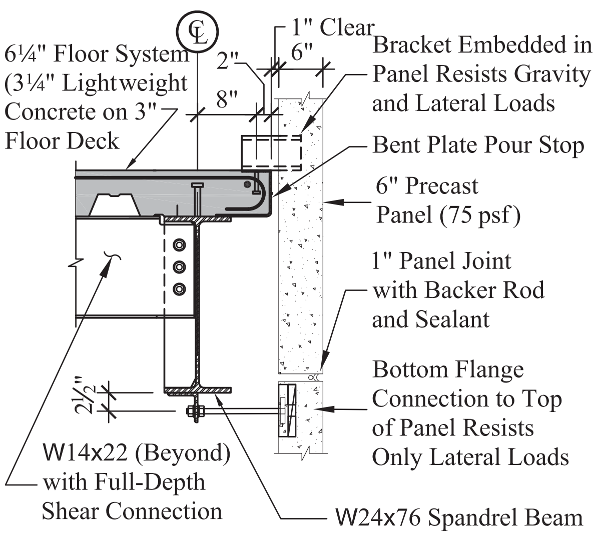

Fig. 6-33. Section of floor spandrel beam with precast panel supported on slab.

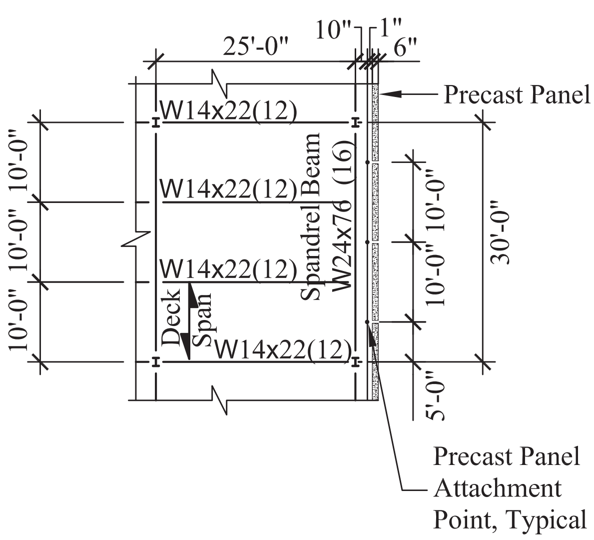

Fig. 6-34. Floor plan at spandrel beam.

factor of 1.2) with a total factored moment demand M_u = 414 kip-ft. The post-composite deflection of the spandrel beam due to live load and superimposed dead load at mid-span is 0.377 in.

The story height is $h = 12$ ft, with the eccentricity between the point of precast load and the centerline of the beam, $e = 8$ in. The horizontal eccentricity between the point of the precast load and the centerline of the precast panels, $e_p = 6$ in. The vertical eccentricity between the bottom flange of the spandrel beam and the point of horizontal panel load, $e_r = 2\frac{1}{2}$ in.

The width of the precast embed parallel to the slab edge, $b_{pc} = 3$ in.

The W24×76 spandrel beam has the following properties:

$$d = 23.9 \text{ in.}$$
$$b_f = 8.99 \text{ in.}$$
$$t_f = 0.680 \text{ in.}$$

$$I_y = 82.5 \text{ in.}^4$$
$$S_x = 176 \text{ in.}^3$$
$$S_y = 18.4 \text{ in.}^3$$

$$J = 2.68 \text{ in.}^4$$
$$a = 104 \text{ in.}$$
$$W_{no} = 52.2 \text{ in.}^2$$

The W14×22 floor framing beams have $I_x = 199$ in.^{4}

The pour stop plate is made from ASTM A36 material.

Solution:

For the loads on the spandrel beam, see Figures 6-35, 6-36, and 6-37. The service precast panel point load at each support point on the spandrel is,

$$\begin{aligned} P_{pc} &= \left(0.075 \text{ kip/ft}^2\right)hs_p \\ &= \left(0.075 \text{ kip/ft}^2\right)\left(12 \text{ ft}\right)\left(10 \text{ ft}\right) \\ &= 9.00 \text{ kips} \end{aligned}$$

The factored precast panel point load at each support point on the spandrel beam is,

$$\begin{aligned} P_{upc} &= 1.2P_{pc} \\ &= 1.2\left(9.00 \text{ kips}\right) \\ &= 10.8 \text{ kips} \end{aligned}$$

The service wind point load on the spandrel beam at each precast panel support point location is,

$$\begin{aligned} P_w &= p_w s_p h \\ &= 0.030 \text{ kip/ft}^2\left(10 \text{ ft}\right)\left(12 \text{ ft}\right) \\ &= 3.60 \text{ kips} \end{aligned}$$

The factored wind point load at each precast panel support point on the spandrel is,

$$\begin{aligned} P_{uw} &= 0.8P_w \\ &= 0.8(3.60 \text{ kips}) \\ &= 2.88 \text{ kips} \end{aligned}$$

The service horizontal force on the spandrel beam due to eccentricity of the panel with respect to the pour stop is,

$$\begin{aligned} P_H &= \frac{P_{pc}e_p}{h-d-e_r} \\ &= \frac{9.00 \text{ kips}(6 \text{ in.})}{12 \text{ ft}(12 \text{ in./ft}) - 23.9 \text{ in.} - 2.5 \text{ in.}} \\ &= 0.459 \text{ kip} \end{aligned}$$

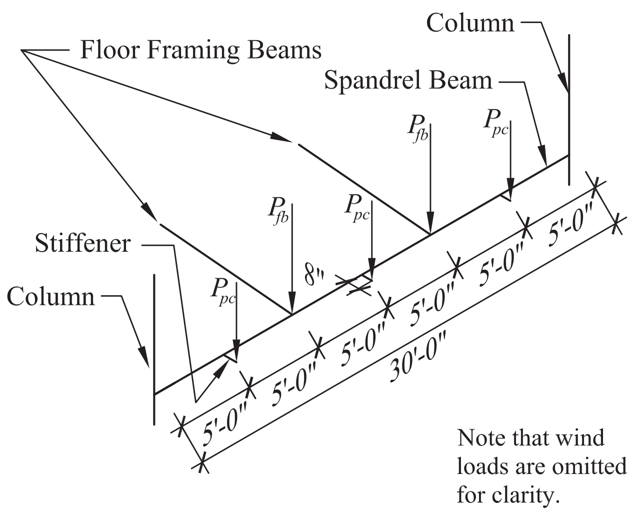

Fig. 6-35. Loads on spandrel beam.

The factored horizontal force on the spandrel beam due to eccentricity of the panel with respect to the pour stop is,

$$\begin{aligned} P_{uH} &= 1.2(P_H) \\ &= 1.2(0.459 \text{ kip}) \\ &= 0.551 \text{ kip} \end{aligned}$$

For the design of the bearing support for the precast panel, the slab strength to support the gravity load of the precast panel will be evaluated assuming that the panel load spreads over an effective width at 45° to each side.

The effective width of the slab to resist the precast load is,

$$\begin{aligned} b_{eff} &= 2\tan 45^\circ\left(e - \frac{b_f}{2}\right) + b_{pc} \\ &= 2\tan 45^\circ\left(8 \text{ in.} - \frac{8.99 \text{ in.}}{2}\right) + 3 \text{ in.} \\ &= 10.0 \text{ in.} \end{aligned}$$

The factored moment in the slab due to the precast gravity load is,

$$\begin{aligned} M_{us} &= P_{upc}\left(e - \frac{b_f}{2}\right)\frac{b}{b_{eff}} \\ &= 10.8 \text{ kips}\left(8 \text{ in.} - \frac{8.99 \text{ in.}}{2}\right)\frac{12 \text{ in./ft}}{10.0 \text{ in.}} \\ &= 45.4 \text{ kip-in./ft} \end{aligned}$$

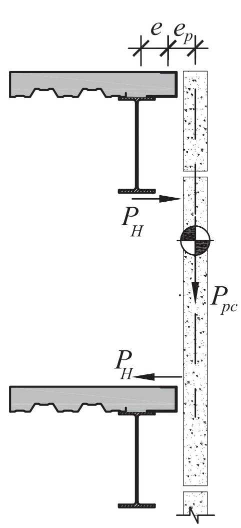

Fig. 6-36. Resolution of precast panel eccentricity.

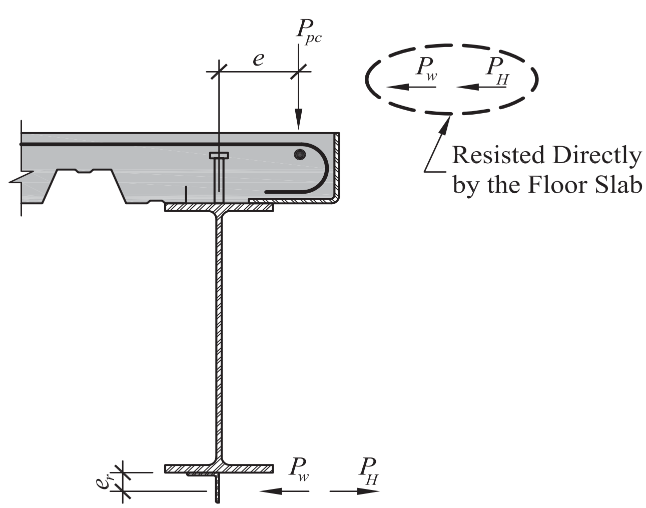

Fig. 6-37. Vertical and lateral forces on spandrel beam.

From Table 5-4, it will be difficult to provide enough reinforcement in the slab to resist the flexure created by the panel load. As an alternative, determine if the bent plate can resist all flexure induced by the precast panels (see Figure 6-38). The horizontal force is resisted by the studs in tension and developed into the slab through the slab reinforcing steel. For flexure, AISC Specification Section F11 provides the flexural strength of rectangular bars bent about the minor axis.

Try a ½-in.-thick bent plate where the effective width of the plate tributary to the precast attachment, assuming the load spreads at 45° to each side, is,

$$b_{eff} = 2\tan 45^\circ\left(e - \frac{b_f}{2}\right) + b_{pc}$$
$$= 2\tan 45^\circ\left(8 \text{ in.} - \frac{8.99 \text{ in.}}{2}\right) + 3 \text{ in.}$$
$$= 10.0 \text{ in.}$$

The moment on the plate is,

$$M_{up} = M_{us} = 45.4 \text{ kip-in./ft}$$

The plastic section modulus of the plate per foot is,

$$Z_p = \frac{bt^2}{4}$$
$$= \frac{12 \text{ in./ft }(0.5 \text{ in.})^2}{4}$$
$$= 0.750 \text{ in.}^3\text{/ft}$$

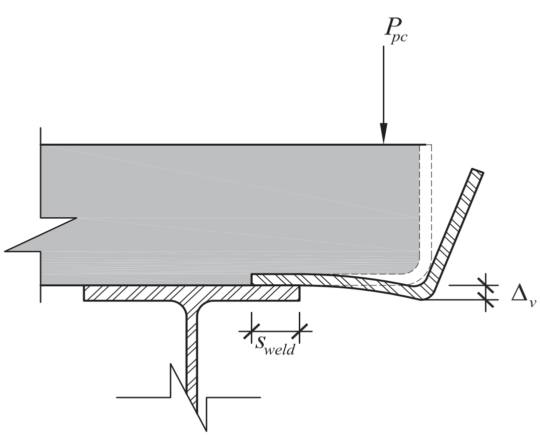

Fig. 6-38. Deflection of bent plate pour stop.

The available flexural strength of the bent plate is,

$$\phi M_{np} = \phi F_y Z_p$$
$$= 0.90(36 \text{ ksi})\left(0.750 \text{ in.}^3\text{/ft}\right)$$
$$= 24.3 \text{ kip-in./ft} < M_{us} \quad \textbf{n.g.}$$

The precast panel loads are too large for a reasonable thickness bent plate to cantilever off of the spandrel beam flange. The ½-in. plate alone does not have adequate flexural strength. Additionally, as shown below, the deflection of the plate (which combines with the deflection of the spandrel) is excessive.

The distance between welds connecting the plate to the beam flange is,

$$s_{weld} = 2 \text{ in.}$$

The effective moment of inertia of the plate is,

$$I = \frac{b_{eff}t^3}{12}$$
$$= \frac{10.0 \text{ in.}(0.50 \text{ in.})^3}{12}$$
$$= 0.104 \text{ in.}^4$$

The vertical deflection of the end of the plate, neglecting the weight of the concrete (from AISC Manual Table 3-23, case 26), is,

$$\Delta_V = \frac{P_{pc}e^2}{3EI}\left(e + s_{weld}\right)$$
$$= \frac{9.00 \text{ kips }(8 \text{ in.})^2}{3(29{,}000 \text{ ksi})(0.104 \text{ in}^4)}(8 \text{ in.} + 2 \text{ in.})$$
$$= 0.637 \text{ in.}$$

Therefore, stiffener plates must be added to the spandrel at each precast panel support point and the spandrel designed to take torsion. See Figure 6-39. Adding the stiffeners eliminates the need to design the bent plate pour stop for any loads other than the wet load of the concrete. From Table 5-11, a ½-in.-thick bent plate pour stop will be sufficient.

For the purposes of the analysis, consider the spandrel beam restrained for torsion at each of the perpendicular floor framing beams and the columns. Use the provisions from AISC Design Guide No. 9 to determine the maximum angle of twist. Assume the rotation is about the center of the top flange of the spandrel beam since it is restrained by the concrete slab.

The service torsional moment at the midspan of L_T due to the precast load is,

$$T_{pc} = P_{pc}e - P_H(d + e_r)$$
$$= \frac{9.00 \text{ kips}(8 \text{ in.}) - 0.459 \text{ kip}(23.9 \text{ in.} + 2\tfrac{1}{2} \text{ in.})}{12 \text{ in./ft}}$$
$$= 4.99 \text{ kip-ft}$$

The service torsional moment at the midspan of L_T due to the wind load applied at the bottom of the beam is,

$$T_w = \frac{P_w}{2}(d)$$
$$= \frac{\left(\frac{3.60 \text{ kips}}{2}\right)(23.9 \text{ in.})}{12 \text{ in./ft}}$$
$$= 3.59 \text{ kip-ft}$$

The total service torsional moment applied at the midspan of L_T is,

$$T = T_{pc} + T_w$$
$$= 4.99 \text{ kip-ft} + 3.59 \text{ kip-ft}$$
$$= 8.58 \text{ kip-ft}$$

The service torsion at each end of L_T is,

$$T_e = \frac{T}{2}$$
$$= \frac{8.58 \text{ kip-ft}}{2}$$
$$= 4.29 \text{ kip-ft}$$

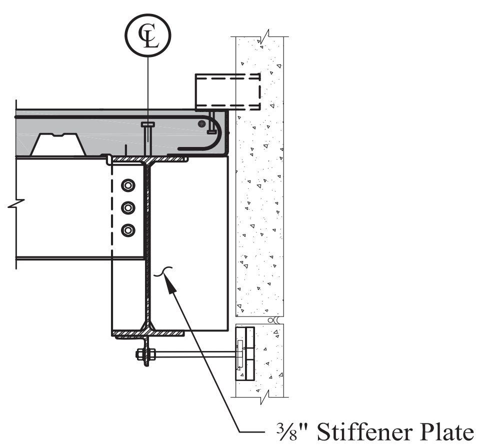

Fig. 6-39. Section of spandrel beam with stiffener plate at bottom flange connection to precast panel.

To obtain the service moment applied to the ends of the floor-framing beams, multiply T_e by 2 because the precast panels are symmetric about the floor framing beams. The service moment applied to the end of the floor framing beam is,

$$M_{fb} = 2T_e$$
$$= 2(4.29 \text{ kip-ft})$$
$$= 8.58 \text{ kip-ft}$$

The rotation at the end of the floor framing beam (see Figure 6-40) is,

$$\theta_{fb} = \frac{M_{fb}L_{fb}}{3EI_{x\,fb}}$$
$$= \frac{8.58 \text{ kip-ft}(25 \text{ ft})(12 \text{ in./ft})^2}{3(29{,}000 \text{ ksi})(199 \text{ in.}^4)}$$
$$= 0.00178 \text{ rad.} \quad (\text{or } 0.102°)$$

The service vertical reaction on the spandrel beam due to the end moment on the floor framing beam is,

$$R_{fb} = \frac{M_{fb}}{L_{fb}}$$
$$= \frac{8.58 \text{ kip-ft}}{25 \text{ ft}}$$
$$= 0.343 \text{ kip}$$

In this case, R_{fb} is small. In many practical cases it may be neglected.

$$\frac{L_T}{a} = \frac{10 \text{ ft}}{104 \text{ in./}(12 \text{in./ft})} = 1.15$$

From AISC Design Guide No. 9, Appendix B, case 3 for $\alpha = 0.5$, the torsional function at the center of the span L_T (between roll beams) is,

$$\theta\left(\frac{GJ}{TL_T}\right) = 0.025 \text{ rad.}$$

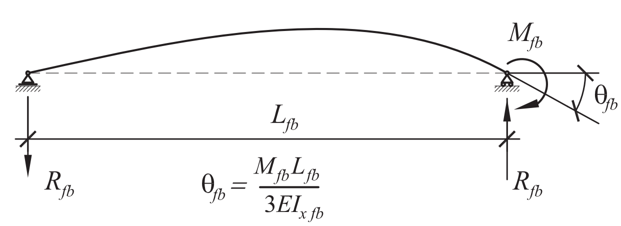

Fig. 6-40. Rotation of roll beam.

The rotation at the center of the span, L_T, is,

$$\theta = \frac{(0.025\,\text{rad.})TL_T}{GJ}$$
$$= \frac{(0.025\,\text{rad.})(8.58\text{ kip-ft})(10\text{ ft})(12\text{ in./ft})^2}{(11{,}200\text{ ksi})(2.68\text{ in.}^4)}$$
$$= 0.0102\text{ rad.}\ \ (\text{or } 0.585°)$$

Note that this torsional rotation θ assumes the spandrel beam can rotate about its centroid. In reality, the spandrel beam is restrained by the slab and will rotate about the center of its top flange. Thus, the actual rotation will be slightly smaller.

Adding the rotation of the spandrel beam between the roll beams to the rotation of the roll beams,

$$\theta_{total} = \theta + \theta_{fb}$$
$$= 0.585° + 0.102°$$
$$= 0.687°$$

The vertical deflection at the precast support point due to rigid body rotation,

$$\Delta_{vrb} = e\sin(\theta_{total})$$
$$= (8\text{ in.})\sin(0.687°)$$
$$= 0.0959\text{ in.}$$

The total vertical deflection is,

$$\Delta_{total} = \Delta_{vrb} + \Delta_{pcl}$$
$$= 0.0959\text{ in.} + 0.377\text{ in.}$$
$$= 0.473\text{ in.} \le \tfrac{1}{2}\text{ in. } \textbf{o.k.}$$

The available strength of the spandrel beam is calculated using case 3 from AISC Design Guide No. 9, Appendix B to determine the normal stresses due to warping. The factored torsional moment at the middle of the torsion span is,

$$T_u = 1.2T_{pc} + 0.8T_w$$
$$= 1.2(4.99\text{ kip-ft}) + 0.8(3.59\text{ kip-ft})$$
$$= 8.86\text{ kip-ft}$$

The torsional function at the center of the span L_T (between roll beams) is,

$$\theta_u''\left(\frac{GJa}{T_u}\right) = 0.25$$

$$\theta_u'' = \frac{0.25T_u}{GJa}$$
$$= \frac{0.25(8.86\text{ kip-ft})(12\text{ in./ft})}{(11{,}200\text{ ksi})(2.68\text{ in.}^4)(104\text{ in.})}$$
$$= 8.51\times10^{-6}\text{ rad./in.}^2$$

The normal stress at midspan due to warping is,

$$\sigma_{wsu} = EW_{no}\theta_u''$$
$$= 29{,}000\text{ ksi}(52.2\text{ in.}^2)(8.51\times10^{-6}\text{rad./in.}^2)$$
$$= 12.9\text{ ksi}$$

The factored post composite moment demand is,

$$M_{post} = M_u - M_{pre}$$
$$= 414\text{ kip-ft} - 93.0\text{ kip-ft}$$
$$= 321\text{ kip-ft}$$

The interaction of combined normal stresses at the midspan of the beam is based on the AISC Design Guide No. 9, Equation 4.16a, using the elastic combination of stresses,

$$\sigma_{total} = \frac{M_{pre}}{S_x} + \frac{M_{post}}{S_{eff}} + \sigma_{wsu}$$
$$= \frac{93\text{ kip-ft}(12\text{ in./ft})}{176\text{ in.}^3} + \frac{321\text{ kip-ft}(12\text{ in./ft})}{217\text{ in.}^3} + 12.9\text{ ksi}$$
$$= 37.0\text{ ksi}$$

$$\phi F_y = 0.90F_y$$
$$= 0.90(50\text{ ksi})$$
$$= 45.0\text{ ksi} > 37.0\text{ ksi}\quad \textbf{o.k.}$$

The beam has adequate flexural strength.

Comments:

As an alternative approach, the rotation of the beam can be determined using the "Flexural Analogy," which converts the applied torsion into a couple acting at the top and bottom flanges of the beam. The force at the top flange is assumed to be resisted by the slab. The force at the bottom flange is assumed to be resisted by weak-axis bending of the bottom half of the W24. The calculated deflection is then converted into an equivalent rotation about the top flange.

The moment of inertia of the bottom flange is,

$$I_{y\ FA} = \frac{I_y}{2} = \frac{82.5\text{ in.}^4}{2} = 41.3\text{ in.}^4$$

The equivalent force to apply to the bottom flange is,

$$F_{FA} = \frac{T}{d} = \frac{8.58\text{ kip-ft}\left(12\text{ in./ft}\right)}{23.9\text{ in.}} = 4.31\text{ kips}$$

The deflection of the bottom flange is,

$$\begin{aligned}\Delta_{bf\ FA} &= \frac{F_{FA} L_T^3}{48 E I_{y\ FA}} \\ &= \frac{\left(4.31\text{ kips}\right)\left(10\text{ ft}\right)^3\left(12\text{ in./ft}\right)^3}{48\left(29{,}000\text{ ksi}\right)\left(41.3\text{ in.}^4\right)} \\ &= 0.129\text{ in.}\end{aligned}$$

Because the beam is forced to rotate about its top flange the designer can take advantage of the full beam depth when calculating θ. The equivalent rotation about the top flange using the flexural analogy is,

$$\begin{aligned}\theta_{FA} &= \sin^{-1}\left(\frac{\Delta_{bf\ FA}}{d}\right) \\ &= \sin^{-1}\left(\frac{0.129\text{ in.}}{23.9\text{ in.}}\right) \\ &= 0.310°\end{aligned}$$

In this case the rotation predicted by the flexural analogy is approximately half of that predicted by analyzing the twist of the beam about its centroid. When the torsional length is less than about 15 ft, the flexural analogy is easier to calculate and provides a more accurate result.

The 0.337-in. deflection due to post composite load is approximately 80% of the total deflection. The rest of the deflection is due to twist of the beam between the roll beams. This example illustrates that the presence of roll beams will help to significantly reduce deflections at the precast panels. It also shows that under certain loading conditions one must still consider torsional deflections between roll beams.

When using the LRFD methodology with a commercially available computer program, it may be necessary to make two "runs" of the analysis—the first in LRFD-mode to obtain factored moments and the second in ASD-mode to obtain the effective elastic section modulus and post-composite deflections. To investigate the interaction of the stresses properly, only elastic stresses are used here. Using $M_r/\phi M_n$ in place of the first two terms in the stress interaction equation may be unconservative.

Example 6.6—Precast Panel Loads at Floor Opening

This example illustrates the design of a spandrel beam adjacent to an opening with eccentric cladding load. The example also shows how to analyze a built-up section consisting of a wide flange and supplemental side plate for torsion.

The 30-ft-long wide-flange spandrel beam shown in Figures 6-41 and 6-42 is adjacent to a stair opening over a portion of its span. The spandrel beam supports the floor framing shown in addition to 6-in.-thick precast concrete cladding panels weighing 75 psf. The panels bear on the top

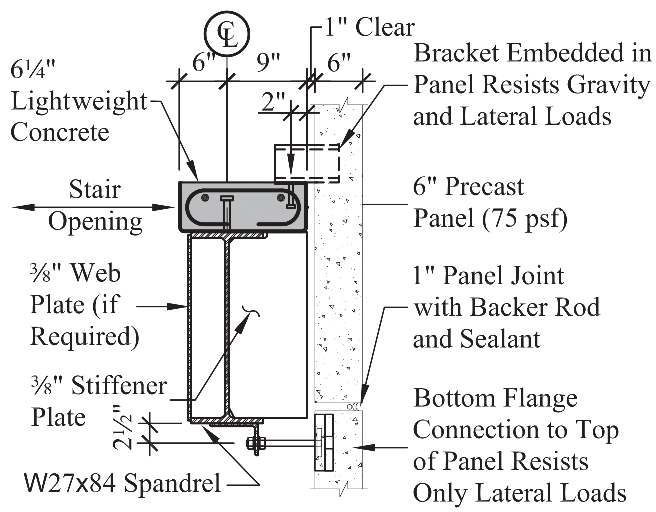

Fig. 6-41. Section of floor spandrel beam at opening with precast panel supported on slab.

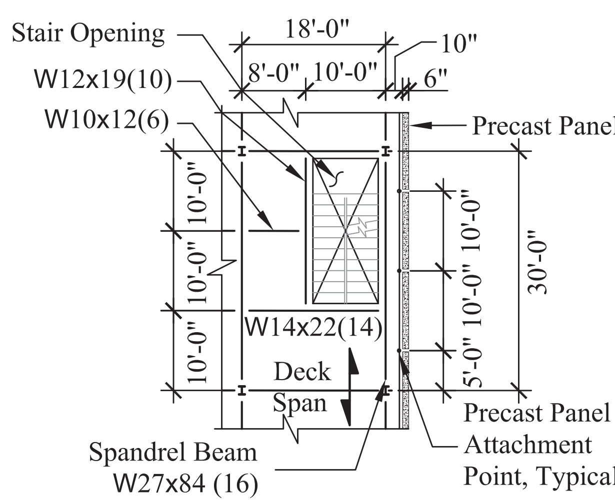

Fig. 6-42. Floor plan at spandrel beam.

of the slab 2 in. from the edge of the slab. The 6¼-in.-thick floor system consists of 3¼-in. lightweight concrete on 3-in. metal floor deck. The floor live load is 100 psf, and the superimposed dead load is 15 psf. The maximum vertical deflection due to superimposed loads is ½ in.

Determine the total vertical deflection at the support points for the precast panels considering torsion and the bare W27 section. If the deflection of the bare W27 section is excessive, add a second web to create a built-up box section to increase torsional stiffness. Check the strength and the horizontal displacement of the spandrel beam—for this case, use a horizontal deflection limit of L/480.

Given:

For this example assume the controlling load combination for strength is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection). The spandrel beam is a W27×84, while the W14×22 floor framing beam perpendicular to the spandrel provides torsional restraint for the spandrel. For the spandrel beam, L = 30 ft for flexure and L_T = 20 ft for torsion. The length of the W14×22 floor beam, L_{fb} = 10 ft. The story height, h = 12 ft.

The horizontal eccentricity between the centroid of the precast panel and the edge of the pour stop is resolved by a couple between adjacent floors. The pour stop is a ¼-in.-thick bent plate. The spandrel has web stiffeners at the precast attachment locations. Because of the opening, assume that the spandrel beam does not act compositely with the floor slab. The spandrel beam deflects a total of 0.48 in. due to live loads and superimposed dead loads, including the cladding.

The supports for the precast panels are spaced at s_p = 10 ft apart. The horizontal eccentricity between the point of the precast panel attachment and the centerline of the beam, e = 7 in. The horizontal eccentricity between the point of the precast panel attachment and centerline of the precast panels is e_p = 6 in. The vertical eccentricity between the bottom flange of the spandrel beam and the panel attachment point is e_r = 2½ in. Any additional contribution of the concrete slab strip to the torsional constant, J, of the section has been neglected.

The factored moment demand, M_u = 414 kip-ft. The available moment strength of the bare beam, considering a laterally unbraced length L_b = 20 ft, is $\phi_b M_n$ = 579 kip-ft, based on AISC Manual Table 3-10.

For the loads on the spandrel beam, see Figures 6-43, 6-44 and 6-45. The W27×84 spandrel beam has the following properties:

d = 26.7 in.
t_w = 0.460 in.
b_f = 10.0 in.
t_f = 0.640 in.
I_x = 2,850 in.^{4}
I_y = 106 in.^{4}
S_y = 21.2 in.^{3}
Z_y = 33.2 in.^{3}
J = 2.81 in.^{4}
a = 128 in.

The deflection due to superimposed dead load at midspan is Δ_{pcl} = 0.48 in.

The W14×22 floor framing beam has I_x = 199 in.^{4}

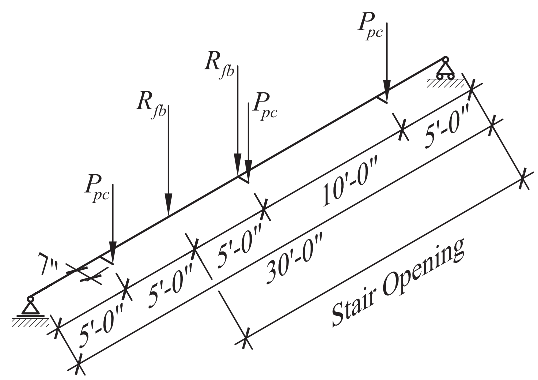

Fig. 6-43. Loads on spandrel beam.

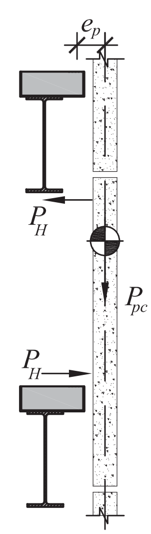

Fig. 6-44. Resolution of precast panel eccentricity.

Solution:

The service precast panel point load at each support point on the spandrel is,

$$\begin{aligned} P_{pc} &= \left(0.075\text{ kip/ft}^2\right)hs_p \\ &= \left(0.075\text{ kip/ft}^2\right)\left(12\text{ ft}\right)\left(10\text{ ft}\right) \\ &= 9.00\text{ kips} \end{aligned}$$

The service wind point load on the spandrel beam at each precast panel support point location is,

$$\begin{aligned} P_w &= p_w s_p h \\ &= 0.030\text{ kip/ft}^2\left(10\text{ ft}\right)\left(12\text{ ft}\right) \\ &= 3.60\text{ kips} \end{aligned}$$

The service horizontal force on the spandrel beam due to eccentricity of panel with respect to pour stop is,

$$\begin{aligned} P_H &= \frac{P_{pc}e_p}{h-d-e_r} \\ &= \frac{9.00\text{ kips}\left(6\text{ in.}\right)}{12\text{ ft}\left(12\text{ in./ft}\right)-26.7\text{ in.}-2\tfrac{1}{2}\text{ in.}} \\ &= 0.470\text{ kip} \end{aligned}$$

The factored horizontal force on the spandrel beam due to the eccentricity of the panel with respect to the pour stop is,

$$\begin{aligned} P_{uH} &= 1.2\left(P_H\right) \\ &= 1.2\left(0.470\text{ kip}\right) \\ &= 0.564\text{ kip} \end{aligned}$$

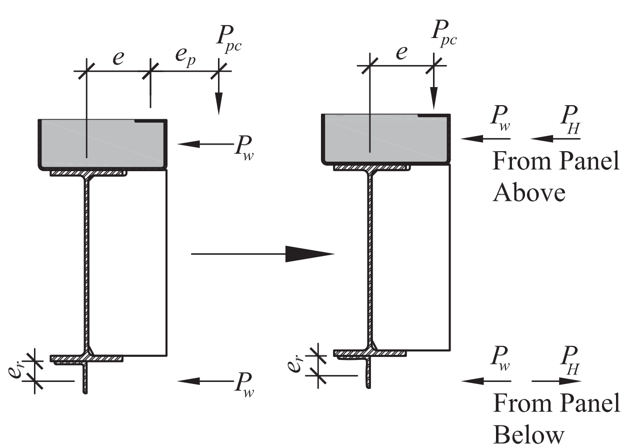

Fig. 6-45. Vertical and lateral forces on spandrel beam.

The deflection of the W27 section will be evaluated without the slab to determine its contribution to the deflection at the tip of the bent plate pour stop considering torsion. For the purpose of this analysis, the perpendicular floor framing beam and columns will be considered to restrain the torsion of the spandrel beam. Using AISC Design Guide No. 9 (Seaburg and Carter, 1997), the maximum angle of twist can be determined. Unlike the previous example; however, in this example torsion is calculated with respect to the centroid of the spandrel beam since the top flange is not restrained laterally by the floor slab.

Note that because of the horizontal eccentricity, e_p, between the centroid of the panel and the point of vertical support, the weight of the panel creates a couple between the flanges of the spandrel that opposes the torsion caused by P_{pc}. The service torsional moment due to the precast load at the point of attachment is,

$$\begin{aligned} T_{pc} &= P_{pc}e - 2P_H\left(\frac{d}{2}+e_r\right) \\ &= \frac{9.00\text{ kips}\left(7\text{ in.}\right)-2\left(0.470\text{ kip}\right)\left(\dfrac{26.7\text{ in.}}{2}+2\tfrac{1}{2}\text{ in.}\right)}{12\text{ in./ft}} \\ &= 4.01\text{ kip-ft} \end{aligned}$$

Unlike the previous example in which the wind loads caused twist on the bottom flange of the spandrel, here the wind loads are applied equally to both the top and bottom flange of the beam. Although there is no resulting twist, the beam is loaded in weak-axis flexure between the column and the W14×22 floor framing beam. The total service torsional moment at the point of the precast attachment is,

$$\begin{aligned} T &= T_{pc} \\ &= 4.01\text{ kip-ft} \end{aligned}$$

To obtain the service flexural moment applied to the end of the W14×22 floor-framing beam, T from the span adjacent to the stair is added to $T/2$ from the span adjacent to the stair. The service moment applied to the end of the floor framing beam is,

$$\begin{aligned} M_{fb} &= T + \frac{T}{2} \\ &= 4.01\text{ kip-ft} + \frac{4.01\text{ kip-ft}}{2} \\ &= 6.02\text{ kip-ft} \end{aligned}$$

The rotation at the end of the floor framing beam is (see Figure 6-46),