Example I-1  Composite Beam Design

Given:

A series of 45-ft. span composite beams at 10 ft. o/c are carrying the loads shown below. The beams are ASTM A992 and are unshored. The concrete has $f'_c = 4$ ksi. Design a typical floor beam with 3 in. 18 gage composite deck, and 4½ in. normal weight concrete above the deck, for fire protection and mass. Select an appropriate beam and determine the required number of shear studs.

Solution:

Material Properties:

- Concrete  
  \[ f'_c = 4 \text{ ksi} \]

- Beam  
  \[ F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi} \]

Loads:

Dead load:
- Slab  
  \[ = 0.075 \text{ kip/ft}^2 \]
- Beam weight  
  \[ = 0.008 \text{ kip/ft}^2 \] (assumed)
- Miscellaneous  
  \[ = 0.010 \text{ kip/ft}^2 \] (ceiling etc.)

Live load:
- Non-reduced  
  \[ = 0.10 \text{ kips/ft}^2 \]

Since each beam is spaced at 10 ft. o.c.

Total dead load  
\[ = 0.093 \text{ kip/ft}^2(10 \text{ ft.}) = 0.93 \text{ kips/ft.} \]
Total live load  
\[ = 0.10 \text{ kip/ft}^2(10\text{ ft}) = 1.00 \text{ kips/ft.} \]

Construction dead load (unshored)  
\[ = 0.083 \text{ kip/ft}^2(10 \text{ ft}) = 0.83 \text{ kips/ft} \]
Construction live load (unshored)  
\[ = 0.020 \text{ kip/ft}^2(10 \text{ ft}) = 0.20 \text{ kips/ft} \]
**Determine the required flexural strength**

<table>
<thead>
<tr>
<th></th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_u = 1.2(0.93 \text{ kip/ft}) + 1.6(1.0 \text{ kip/ft})$</td>
<td>$w_u = 0.93 \text{ kip/ft} + 1.0 \text{ kip/ft}$</td>
<td>$w_u = 1.93 \text{ kip/ft}$</td>
</tr>
<tr>
<td></td>
<td>$= 2.72 \text{ kip/ft}$</td>
<td></td>
</tr>
<tr>
<td>$M_u = \frac{2.72 \text{ kip/ft}(45 \text{ ft})^2}{8}$</td>
<td>$M_u = \frac{1.93 \text{ kip/ft}(45 \text{ ft})^2}{8}$</td>
<td>$= 687 \text{ kip-ft.}$</td>
</tr>
<tr>
<td></td>
<td>$= 687 \text{ kip-ft.}$</td>
<td></td>
</tr>
</tbody>
</table>

Use Tables 3-19, 3-20 and 3-21 from the Manual to select an appropriate member.

**Determine $b_{\text{eff}}$**

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:

1. One-eighth of the beam span, center to center of supports
   $$\frac{45 \text{ ft}}{8} = 11.3 \text{ ft.}$$

2. One-half the distance to center-line of the adjacent beam
   $$\frac{10 \text{ ft}}{2} = 10.0 \text{ ft.}$$  **Controls**

3. The distance to the edge of the slab
   Not applicable

*Calculate the moment arm for the concrete force measured from the top of the steel shape, $Y_2$.*

Assume $a = 1.0$ in. (Some assumption must be made to start the design process. An assumption of 1.0 in. has proven to be a reasonable starting point in many design problems.)

$$Y_2 = t_{\text{slab}} - a/2 = 7.5 - 0.5 = 7.0 \text{ in.}$$

Enter Manual Table 3-19 with the required strength and $Y_2 = 7.0$ in. Select a beam and neutral axis location that indicates sufficient available strength.

Select a W21×50 as a trial beam.

When PNA location 5 (BFL), this composite shape has an available strength of:

<table>
<thead>
<tr>
<th></th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_o M_a = 770 \text{ kip-ft} &gt; 687 \text{ kip-ft}$</td>
<td>$M_a/\Omega_b = 512 \text{ kip-ft} &gt; 489 \text{ kip-ft}$</td>
<td>$\text{o.k.}$</td>
</tr>
</tbody>
</table>

Note that the required PNA location for ASD and LRFD differ. This is because the live to dead load ratio in this example is not equal to 3. Thus, the PNA location requiring the most shear transfer is selected to be acceptable for ASD. It will be conservative for LRFD.
Check the beam deflections and available strength
Check the deflection of the beam under construction, considering only the weight of concrete as contributing to the construction dead load.

Limit deflection to a maximum of 2.5 in. to facilitate concrete placement.

\[
I_{eq} = \frac{5}{384} \frac{w_{ud} t^4}{E \Delta} = \frac{5(0.83 \text{ kip/ft})(45 \text{ ft})^4 (1728 \text{ in.}^3/\text{ft}^4)}{384(29,000 \text{ ksi})(2.5 \text{ in.})} = 1,060 \text{ in.}^4
\]

From Manual Table 3-20, a W21×50 has \(I_e = 984 \text{ in.}^4\), therefore this member does not satisfy the deflection criteria under construction.

Using Manual Table 3-20, revise the trial member selection to a W21×55, which has \(I_e = 1140 \text{ in.}^4\), as noted in parenthesis below the shape designation.

Check selected member strength as an un-shored beam under construction loads assuming adequate lateral bracing through the deck attachment to the beam flange.

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Calculate the required strength</strong></td>
<td><strong>Calculate the required strength</strong></td>
</tr>
<tr>
<td>(1.4 DL = 1.4 \times (0.83 \text{ kips/ft}) = 1.16 \text{ kips/ft} )</td>
<td>(DL + LL = 0.83 + 0.20 = 1.03 \text{ kips/ft} )</td>
</tr>
<tr>
<td>(1.2DL + 1.6LL = 1.2 \times (0.83) + 1.6(0.20) = 1.32 \text{ klf} )</td>
<td></td>
</tr>
<tr>
<td>(M_u_{(unshored)} = \frac{1.31 \text{ kip/ft}(45 \text{ ft})^2}{8} = 331 \text{ kip-ft} )</td>
<td>(M_a_{(unshored)} = \frac{1.03 \text{ kips/ft}(45 \text{ ft})^2}{8} = 260 \text{ kip-ft} )</td>
</tr>
<tr>
<td>The design strength for a W21×55 is 473 kip-ft &gt; 331 kip-ft o.k.</td>
<td>The allowable strength for a W21×55 is 314 kip-ft &gt; 260 kip-ft o.k.</td>
</tr>
</tbody>
</table>

For a W21×55 with \(Y^2=7.0 \text{ in.}\), the member has sufficient available strength when the PNA is at location 6 and \(\sum Q_n = 292 \text{ kips}\).

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\phi \cdot M_e = 767 \text{ kip-ft} &gt; 687 \text{ kip-ft} ) o.k.</td>
<td>(M_a/\Omega_b = 510 \text{ kip-ft} &gt; 489 \text{ kip-ft} ) o.k.</td>
</tr>
</tbody>
</table>

Check \(a\)

\[
a = \frac{\sum Q_n}{0.85f_y b} = \frac{292 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft})(12 \text{ in./ft.})} = 0.716 \text{ in.}
\] 

0.716 in. < 1.0 in. assumed o.k.

Check live load deflection

\[
\Delta_L < \frac{l}{360} = \frac{(45 \text{ ft})(12 \text{ in./ft})}{360} = 1.5 \text{ in.}
\]

A lower bound moment of inertia for composite beams is tabulated in Manual Table 3-20.
For a W21×55 with \( f_y = 7.0 \) and the PNA at location 6, \( I_{LB} = 2440 \text{ in.}^4 \)

\[
\Delta_{LL} = \frac{5}{384} \frac{w_{oLL} t^4}{EI_{LB}} = \frac{5(1.0 \text{ kip/ft})(45 \text{ ft})^4 (1728 \text{ in.}^3/\text{ft}^4)}{384(29,000 \text{ ksi})(2440 \text{ in.}^4)} = 1.30 \text{ in.}
\]

1.30 in. < 1.5 in. \textbf{o.k.}

**Determine if the beam has sufficient available shear strength**

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_u = \frac{45\text{ft}}{2} (2.72\text{kip/ft}) = 61.2 \text{ kips} )</td>
<td>( V_u = \frac{45\text{ft}}{2} (1.93\text{kip/ft}) = 43.4 \text{ kips} )</td>
</tr>
<tr>
<td>( \phi V_u = 234 \text{ kips} &gt; 61.2 \text{ kips} ) \textbf{o.k.}</td>
<td>( V_u/\Omega = 156 \text{ kips} &gt; 43.4 \text{ kips} ) \textbf{o.k.}</td>
</tr>
</tbody>
</table>

**Determine the required number of shear stud connectors**

Using perpendicular deck with one \( \frac{3}{4} \)-in. diameter weak stud per rib in normal weight 4 ksi concrete. \( Q_n = 17.2 \text{ kips/stud} \)

\[
\frac{\sum Q_n}{Q_n} = \frac{292 \text{ kips}}{17.2 \text{ kips}} = 17 \text{, on each side of the beam.}
\]

Total number of shear connectors; use 2(17) = 34 shear connectors.

**Check the spacing of shear connectors**

Since each flute is 12 in., use one stud every flute, starting at each support, and proceed for 17 studs on each end of the span.

\( 6d_{stud} < 12 \text{ in.} < 8t_{slab} \), therefore, the shear stud spacing requirements are met.

The studs are to be 5 in. long, so that they will extend a minimum of 1½ in. into slab.
Example I-2  Filled Composite Column in Axial Compression

Given:

Determine if a 14-ft long HSS10×6×3/8 ASTM A500 grade B column filled with $f'_c = 5$ ksi normal weight concrete can support a dead load of 56 kips and a live load of 168 kips in axial compression. The column is pinned at both ends and the concrete at the base bears directly on the base plate. At the top, the load is transferred to the concrete in direct bearing.

Solution:

Calculate the required compressive strength

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_u = 1.2(56 \text{ kips}) + 1.6(168 \text{ kips})$</td>
<td>$P_a = 56 \text{ kips} + 168 \text{ kips}$</td>
</tr>
<tr>
<td>= 336 kips</td>
<td>= 224 kips</td>
</tr>
</tbody>
</table>

The available strength in axial compression can be determined directly from the Manual at $KL = 14$ ft as:

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_Pu = 353 \text{ kips}$</td>
<td>$P_u/\Omega_c = 236 \text{ kips}$</td>
</tr>
<tr>
<td>353 kips &gt; 336 kips</td>
<td>236 kips &gt; 224 kips</td>
</tr>
<tr>
<td>o.k.</td>
<td>o.k.</td>
</tr>
</tbody>
</table>

Supporting Calculations

The available strength of this filled composite section can be most easily determined by using Table 4-14 of the Manual. Alternatively, the available strength can be determined by direct application of the Specification requirements, as illustrated below.

Material Properties:

<table>
<thead>
<tr>
<th>HSS10×6×3/8</th>
<th>$F_y = 46$ ksi</th>
<th>$F_u = 58$ ksi</th>
</tr>
</thead>
</table>

Concrete    $f'_c = 5$ ksi

$$E_c = w^{0.5} \sqrt{f'_c} = (145)^{0.5} \sqrt{5} = 3,900 \text{ ksi}$$
Geometric Properties:
HSS 10 x 6 x 3/8  \( t = 0.375 \) in.  \( b = 10.0 \) in.  \( h = 6.0 \) in.

Concrete:
The concrete area is calculated as follows

\[
r = 2t = 2(0.375 \text{ in.}) = 0.75 \text{ in.} \quad \text{(outside radius)}
\]
\[
b_f = b - 2r = 10 - 2(0.75) = 8.50 \text{ in.}
\]
\[
h_f = h - 2r = 6 - 2(0.75) = 4.50 \text{ in.}
\]
\[
A_c = b_f h_f + \frac{\pi}{2} \left( r - t \right)^2 + 2 b_f (r - t) \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) + 2 \left( \frac{\pi (r - t)^2}{2} \right) \left( \frac{h_f}{2} + \frac{4(r - t)}{3\pi} \right)
\]
\[
= 48.4 \text{ in.}^2
\]
\[
I_c = \frac{b_f h_f^3}{12} + \frac{2(b_f)h_f^3}{12} + 2(r - t) \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) + 2 \left( \frac{\pi (r - t)^2}{2} \right) \left( \frac{h_f}{2} + \frac{4(r - t)}{3\pi} \right)
\]
\[
= 111 \text{ in.}^4
\]

For this shape, buckling will take place about the weak axis, thus

\[
h_1 = 6 - 2(0.375) = 5.25 \text{ in.}
\]
\[
b_1 = 10 - 4(0.375) = 8.5 \text{ in.}
\]
\[
h_2 = 6 - 4(0.375) = 4.5 \text{ in.}
\]
\[
b_2 = 0.375 \text{ in.}
\]
\[
(r - t) = 0.75 - 0.375 = 0.375 \text{ in.}
\]
\[
I_c = \frac{(8.5)(5.25)^3}{12} + 2(0.375)(4.50)^3 + 2(0.375)^4 \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) + 2 \left( \frac{\pi (0.375)^2}{2} \right) \left( \frac{4.5}{2} + \frac{4(0.375)}{3\pi} \right)
\]
\[
= 111 \text{ in.}^4
\]

HSS 10 x 6 x 3/8:
\( A_s = 10.4 \) in.\(^2\)  \( I_s = 61.8 \) in.\(^4\)  \( h/t = 25.7 \)

Limitations:

1) Normal weight concrete 10 ksi \( \geq f'_c \geq 3 \) ksi  \( f'_c = 5 \) ksi  \text{ o.k.}

2) Not Applicable.

3) The cross-sectional area of the steel HSS shall comprise at least one percent of the total composite cross section.

\[
10.4 \text{ in.}^2 > (0.01)(48.6 \text{ in.}^2 + 10.4 \text{ in.}^2) = 0.590 \text{ in.}^2 \quad \text{o.k.}
\]

4) The maximum b/t ratio for a rectangular HSS used as a composite column shall be equal to 2.26 \( \sqrt{\frac{E}{F_y}} \).

\[
b/t = 25.7 \leq 2.26 \sqrt{\frac{E}{F_y}} = 2.26 \sqrt{29,000 \text{ksi}/46 \text{ksi}} = 56.7 \quad \text{o.k.}
\]

User note: For all rectangular HSS sections found in the Manual the b/t ratios do not exceed 2.26 \( \sqrt{\frac{E}{F_y}} \).

5) Not Applicable.
Calculate the available compressive strength

\( C_2 = 0.85 \) for rectangular sections

\[
P_o = A_y F_y + A_y F_y + C_2 A_c f_c'
\]

\[
= (10.4 \text{ in.}^2)(46 \text{ ksi}) + 0.85(48.4 \text{ in.}^2)(5 \text{ ksi}) = 684 \text{ kips}
\]

\[
C_3 = 0.6 + 2 \left( \frac{A_c}{A_y + A_y} \right) = 0.6 + 2 \left( \frac{10.4 \text{in.}^2}{48.4 \text{in.}^2 + 10.4 \text{in.}^2} \right) = 0.954 \geq 0.90
\]

Therefore use 0.90

\[
E I_{eff} = E I_s + E I_{cr} + C_3 E c I_c
\]

\[
= (29,000 \text{ ksi})(61.8 \text{ in.}^4) + (0.90)(3,900 \text{ ksi})(111 \text{ in.}^4)
\]

\[= 2,180,000 \text{ kip-in.}^2\]

User note: \( K \) value is from Chapter C and for this case \( K = 1.0. \)

\[
P_e = \pi^2 (E I_{eff}) / (K L)^2 = \pi^2 (2,180,000 \text{ kip-in.}^2) / ((1.0)(14\text{ft})(12\text{in./ft}))^2 = 762 \text{ kips}
\]

\[
\frac{P_o}{P_e} = \frac{684 \text{ kips}}{762 \text{ kips}} = 0.898
\]

\( 0.898 \leq 2.25 \) Therefore use Eqn. I2-2 to solve \( P_n \)

\[
P_n = P_o \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] = (684 \text{ kips}) \left[ 0.658(0.898) \right] = 470 \text{ kips}
\]

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi_c = 0.75 )</td>
<td>( \Omega_c = 2.00 )</td>
</tr>
<tr>
<td>( \phi_c P_n = 0.75(470 \text{kips}) = 353 \text{kips} )</td>
<td>( P_o/\Omega_c = 470 \text{ kips} / 2.00 = 235 \text{ kips} )</td>
</tr>
<tr>
<td>353 kips ( &gt; 336 \text{kips} ) <strong>o.k.</strong></td>
<td>235 kips ( &gt; 224 \text{kips} ) <strong>o.k.</strong></td>
</tr>
</tbody>
</table>

Section I2.1b
Example I-3  Encased Composite Column in Axial Compression

Given:

Determine if a 14 ft tall W10×45 steel section encased in a 24 in. × 24 in. concrete column with $f'_c = 5$ ksi, is adequate to support a dead load of 350 kips and a live load of 1050 kips in axial compression. The concrete section has 8-#8 longitudinal reinforcing bars and #4 transverse ties @ 12 in. o/c. The column is pinned at both ends and the load is applied directly to the concrete encasement.

Solution:

Material Properties:

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column W10×45</td>
<td>$F_y$</td>
<td>50 ksi</td>
</tr>
<tr>
<td></td>
<td>$F_u$</td>
<td>65 ksi</td>
</tr>
<tr>
<td>Concrete</td>
<td>$f'_c$</td>
<td>5 ksi</td>
</tr>
<tr>
<td></td>
<td>$E_c$</td>
<td>3,900 ksi (145 pcf concrete)</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>$F_{yst}$</td>
<td>60 ksi</td>
</tr>
</tbody>
</table>

Geometric Properties:

<table>
<thead>
<tr>
<th>Section</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>W10×45</td>
<td>$A_s$</td>
<td>13.3 in.$^2$</td>
</tr>
<tr>
<td></td>
<td>$I_y$</td>
<td>53.4 in.$^4$</td>
</tr>
<tr>
<td>Reinforcing steel:</td>
<td>$A_{sr}$</td>
<td>6.32 in.$^2$</td>
</tr>
<tr>
<td></td>
<td>$I_{sr}$</td>
<td>$\frac{\pi r^4}{4} + Ad^2 = \frac{8 \pi (0.50)^4}{4} + 6(0.79)(9.5)^2 = 428$ in.$^4$</td>
</tr>
<tr>
<td>Concrete:</td>
<td>$A_c = A_{cg} - A_t - A_{sr} = 576$ in.$^2$ - 13.3 in.$^2$ - 6.32 in.$^2 = 556$ in.$^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$I_c = I_{cg} - I_t - I_{sr} = 27,200$ in.$^4$</td>
<td></td>
</tr>
</tbody>
</table>

Note: The weak axis moment of inertia is used as part of a slenderness check.
Limitations:

1) Normal weight concrete $10 \text{ ksi} \geq f'_c \geq 3 \text{ ksi}$ $f'_c = 5 \text{ ksi}$ o.k. Section I1.2

2) $F_{yon} \leq 75 \text{ ksi}$ $F_{yon} = 60 \text{ ksi}$ o.k.

3) The cross-sectional area of the steel core shall comprise at least one percent of the total composite cross section.

$$13.3 \text{ in.}^2 > (0.01)(576 \text{ in.}^2) = 5.76 \text{ in.}^2 \text{ o.k.}$$ Section I2.1a

4) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum transverse reinforcement shall be at least $0.009 \text{ in.}^2$ of tie spacing.

$$0.20 \text{ in.}^2 / 12 \text{ in.} = 0.0167 \text{ in.}^2 / \text{ in.} > 0.009 \text{ in.}^2 / \text{ in.} \text{ o.k.}$$

5) The minimum reinforcement ratio for continuous longitudinal reinforcing, $\rho_{sr}$, shall be 0.004.

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32 \text{ in.}^2}{576 \text{ in.}^2} = 0.011 > 0.004 \text{ o.k.} \text{ Eqn. I2-1}$$

Calculate the total required compressive strength

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_u = 1.2(350 \text{ kips}) + 1.6(1050 \text{ kips})$ $= 2100 \text{ kips}$</td>
<td>$P_u = 350 \text{ kips} + 1050 \text{ kips}$ $= 1400 \text{ kips}$</td>
</tr>
</tbody>
</table>

Calculate the available compressive strength

$$P_a = A_dF_y + A_{sr}F_{yr} + 0.85A_cf'_c$$

$$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) + 0.85(556 \text{ in.}^2)(5 \text{ ksi}) = 3410 \text{ kips} \text{ Eqn. I2-2}$$

$$C_i = 0.1 + 2 \left( \frac{A_c}{A_g + A_s} \right) = 0.1 + 2 \left( \frac{13.3 \text{ in.}^2}{556 \text{ in.}^2 + 13.3 \text{ in.}^2} \right) = 0.15 \text{ Eqn. I2-5}$$

$$EI_{eff} = EI_c + 0.5E_dI_{sr} + C_iE_dI_c$$

$$= (29,000 \text{ ksi})(53.4 \text{ in.}^4) + 0.5(29,000 \text{ ksi})(428 \text{ in.}^4) + (0.15)(3,900 \text{ in.}^4)(27,200) = 23,700,000 \text{ kip-in.}^2 \text{ Eqn. I2-4}$$

User note: $K$ value is from Chapter C and for this case $K = 1.0$.

$$P_a = \pi^2(EI_{eff})/(KL)^2 = \pi^2(23,700,000 \text{ kip-in.}^2) / ((1.0)(14\text{ in.})(12\text{ in./ft}))^2 = 8,290 \text{ kips} \text{ Eqn. I2-3}$$

$$\frac{P_u}{P_a} = \frac{3,410 \text{ kips}}{8,290 \text{ kips}} = 0.411$$

$0.411 \leq 2.25$ Therefore use Eqn. I2-2 to solve $P_u$ Section I2.1b
\[ P_n = P_o \left[ 0.658 \left( \frac{P_o}{P_p} \right) \right] = (3,410 \text{ kips}) \left[ 0.658^{(0.411)} \right] = 2870 \text{ kips} \]

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi_c = 0.75 )</td>
<td>( \Omega_c = 2.00 )</td>
</tr>
<tr>
<td>( \phi_c P_n = 0.75(2870\text{kips}) = 2150 \text{ kips} )</td>
<td>( P_o / \Omega_c = 2870 \text{ kips} / 2.00 = 1440 \text{ kips} )</td>
</tr>
<tr>
<td>2150 kips &gt; 2100 kips o.k.</td>
<td>1440 kips &gt; 1400 kips o.k.</td>
</tr>
</tbody>
</table>

Because the entire load in the column was applied directly to the concrete, accommodations must be made to transfer an appropriate portion of the axial force to the steel column. This force is transferred as a shear force at the interface between the two materials.

Determine the number and spacing of \( \frac{1}{2} \) in. diameter headed shear studs to transfer the axial force.

**Solution:**

**Material Properties:**
- Conc. \( f'_c = 5 \text{ ksi} \) \( E_c = 4070 \text{ ksi} \)
- Shear Studs \( F_u = 65 \text{ ksi} \)

**Geometric Properties:**
- W10x45 \( A_d = 13.3 \text{ in.}^2 \) \( A_{flange} = 4.97 \text{ in.}^2 \) \( A_{web} = 3.36 \text{ in.}^2 \) \( d_d = 10.1 \text{ in.} \)
- Shear Studs \( A_{sc} = 0.196 \text{ in.}^2 \)
- Conc. \( d_c = 24 \text{ in.} \)

**Calculate the shear force to be transferred**

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V = \frac{P_o}{\phi_c} = \frac{2,100}{0.75} = 2,800 \text{ kips} )</td>
<td>( V = P_o \Omega_c = 1,400(2) = 2,800 \text{ kips} )</td>
</tr>
</tbody>
</table>

\[ V' = V(A_d/F_u / P_o) = 2800 \text{ kips} \left[ \left( \frac{13.3 \text{ in.}^2}{50 \text{ ksi}} \right) / (3410 \text{ kips}) \right] = 546 \text{ kips} \]

**Calculate the nominal strength of one \( \frac{1}{2} \) in. diameter shear stud connector**
\[ Q_n = 0.5A_n \sqrt{f'cE_c} \leq A_nF_u \]

\[ 0.5A_n \sqrt{f'cE_c} = 0.5(0.196 \text{ in.}^2)\sqrt{(5 \text{ksi})(3900 \text{ksi})} = 13.7 \text{ kips} \]

\[ A_nF_u = (0.196 \text{ in.}^2)(65 \text{ ksi}) = 12.7 \text{ kips} \]

Eqn. I2-12

Therefore use 12.7 kips.

Calculate the number of shear studs required to transfer the total force, \( V' \)

\[ V' / Q_n = 546 \text{ kips} / 12.7 \text{ kips} = 43 \]

An even number of studs are required to be placed symmetrically on two faces. Therefore use 22 studs minimum per flange

Determine the spacing for the shear studs

The maximum stud spacing is 16 in.

The available column length is 14ft (12 in./ft) = 168 in. and the maximum spacing is

\[ = 168 \text{ in.}/(22+1) = 7.3 \text{ in.} \]

Therefore, on the flanges, use single studs @ 7 in.

Stud placement is to start 10.5 in. from one end.

Determine the length of the studs for the flanges:

\[ \left( \frac{d_s - d_u}{2} \right) - 3 \text{ in.} = \left( \frac{24 \text{ in.} - 10.1 \text{ in.}}{2} \right) - 3 \text{ in.} = 3.95 \text{ in.} \]

Therefore use 3 ½ in. in length for the flanges.

Note: The subtraction of 3 in. is to ensure sufficient cover.

Summary: use ½-in. diameter shear stud connectors as shown on each flange, spaced @ 7 in.