In the negative flexure regions, shear connectors shall be provided where the longitudinal reinforcement is considered to be a part of the composite section.

Otherwise, shear connectors need not be provided in negative flexure regions, but additional connectors shall be placed in the region of the points of permanent load contraflexure.

\[ n_{AC} = \frac{A_s f_{sr}}{Z_r} \]  

(1) Fatigue Limit State

\[ p \leq \frac{n Z_r}{V_{sr}} \]  

(2) Strength Limit State

\[ Q_r = \phi_{s} Q_n \]  

(a) Nominal Shear Force,

- Simple & continuous spans that are noncomposite for negative flexure:

\[ P = \sqrt{P_p^2 + F_p^2} \]  

where

\[ P_p = \text{min. of} \left\{ \begin{array}{c} 0.85 f'_c b_s t_s \\ F_{wy} t_w + F_{y} b_f t_f + F_{yc} b_f t_f \end{array} \right\} \]  

(For straight spans or segments, \( F_p \) may be taken equal to zero)

- Continuous spans that are composite for negative flexure:
\[ P = \sqrt{P_r^2 + F_r^2} \]  

where

\[ P_r = P_p + P_n \]  

\[ P_n = \min \left\{ F_{yw} D t_w + F_{yw} b_f t_f + 0.45 f'_c b_f t_f \right\} \]  

(b) Shear Resistance, \( Q_n \)

- Stud shear connector

\[ Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq A_{sc} F_u \]  

- Channel shear connector

\[ Q_n = 0.3 (t_f + 0.5 t_w) L_c \sqrt{f'_c E_c} \]
Design Step 5.1 - Design Shear Connectors

Since the steel girder has been designed as a composite section, shear connectors must be provided at the interface between the concrete deck slab and the steel section to resist the interface shear. For continuous composite bridges, shear connectors are normally provided throughout the length of the bridge. In the negative flexure region, since the longitudinal reinforcement is considered to be a part of the composite section, shear connectors must be provided.

Studs or channels may be used as shear connectors. For this design example, stud shear connectors are being used throughout the length of the bridge. The shear connectors must permit a thorough compaction of the concrete to ensure that their entire surfaces are in contact with the concrete. In addition, the shear connectors must be capable of resisting both horizontal and vertical movement between the concrete and the steel.

The following figure shows the stud shear connector proportions, as well as the location of the stud head within the concrete deck.

![Stud Shear Connectors](image)

**Figure 5-1  Stud Shear Connectors**

<table>
<thead>
<tr>
<th>Shear Connector Embedment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure Region</td>
</tr>
<tr>
<td>Positive</td>
</tr>
<tr>
<td>Intermediate</td>
</tr>
<tr>
<td>Negative</td>
</tr>
</tbody>
</table>

**Table 5-1  Shear Connector Embedment**
The parameters \( I \) and \( Q \) are based on the short-term composite section and are determined using the deck within the effective flange width.

\[
In + QnZr \leq S6.10.7.4.1b
\]

The pitch of the shear connectors must be determined to satisfy the fatigue limit state as specified in S6.10.7.4.2 and S6.10.7.4.3, as applicable. The resulting number of shear connectors must not be less than the number required to satisfy the strength limit states as specified in S6.10.7.4.4.

The pitch, \( p \), of the shear connectors must satisfy the following equation:

\[
p \leq \frac{n \cdot z_r \cdot l}{V_{sr} \cdot Q}
\]

The ratio of the height to the diameter of a stud shear connector must not be less than 4.0. For this design example, the ratio is computed based on the dimensions presented in Figure 5-1, as follows:

\[
\begin{align*}
\text{Height}_{\text{stud}} &= 6.0 \text{-in} \\
\text{Diameter}_{\text{stud}} &= 0.875 \text{-in} \\
\frac{\text{Height}_{\text{stud}}}{\text{Diameter}_{\text{stud}}} &= 6.86 \quad \text{OK}
\end{align*}
\]

The pitch of the shear connectors must be determined to satisfy the fatigue limit state as specified in S6.10.7.4.4. As applicable. The resulting number of shear connectors must not be less than the number required to satisfy the strength limit states as specified in S6.10.7.4.4.

The pitch, \( p \), of the shear connectors must satisfy the following equation:

\[
p \leq \frac{n \cdot z_r \cdot l}{V_{sr} \cdot Q}
\]

The parameters \( l \) and \( Q \) are based on the short-term composite section and are determined using the deck within the effective flange width.
In the positive flexure region:

\[ n = 3 \quad \text{(see Figure 5-1)} \]

\[ l = 66340.3 \cdot \text{in}^4 \quad \text{(see Table 3-4)} \]

\[ Q = \left[ \frac{(8.0 \cdot \text{in}) \cdot (103.0 \cdot \text{in})}{8} \right] \cdot (62.375 \cdot \text{in} - 50.765 \cdot \text{in}) \]

\[ Q = 1195.8 \cdot \text{in}^3 \]

In the positive flexure region, the maximum fatigue live load shear range is located at the abutment. The factored value is computed as follows:

\[ V_{sr} = 0.75 \cdot (41.45 \cdot \text{K} + 5.18 \cdot \text{K}) \]

\[ V_{sr} = 34.97 \cdot \text{K} \]

(see live load analysis computer run)

\[ Z_r = \alpha \cdot d^2 \geq \frac{5.5 \cdot d^2}{2} \quad \text{S6.10.7.4.2} \]

\[ N = 82125000 \quad \text{(see Design Step 3.14 at location of maximum positive flexure)} \quad \text{S6.6.1.2.5} \]

\[ \alpha = 34.5 - 4.28 \cdot \log(N) \]

\[ \alpha = 0.626 \]

\[ d = 0.875 \cdot \text{in} \]

\[ \alpha \cdot d^2 = 0.48 \quad \frac{5.5 \cdot d^2}{2} = 2.11 \]

Therefore, \[ Z_r = 2.11 \cdot \text{K} \]

\[ p = \frac{n \cdot Z_r \cdot l}{V_{sr} \cdot Q} \quad p = 10.04 \cdot \text{in} \]
In the negative flexure region:

\[ n = 3 \quad \text{(see Figure 5-1)} \]

In the negative flexure region, the parameters \( I \) and \( Q \) may be determined using the reinforcement within the effective flange width for negative moment, unless the concrete slab is considered to be fully effective for negative moment in computing the longitudinal range of stress, as permitted in S6.6.1.2.1. For this design example, \( I \) and \( Q \) are assumed to be computed considering the concrete slab to be fully effective.

\[ I = 130196.1 \cdot \text{in}^4 \quad \text{(see Table 3-5)} \]

\[ Q = \left[ \frac{(8.0 \cdot \text{in}) \cdot (103.0 \cdot \text{in})}{8} \right] \cdot (64.250 \cdot \text{in} - 46.702 \cdot \text{in}) \]

\[ Q = 1807.4 \cdot \text{in}^3 \]

\[ V_{sr} = 0.75 \cdot (0.00 \cdot \text{K} + 46.53 \cdot \text{K}) \]

\[ V_{sr} = 34.90 \text{K} \]

(see Table 3-1 and live load analysis computer run)

\[ Z_r = \alpha \cdot d^2 \geq \frac{5.5 \cdot d^2}{2} \]

\[ Z_r = 2.11 \cdot \text{K} \quad \text{(see previous computation)} \]

\[ p = \frac{n \cdot Z_r \cdot I}{V_{sr} \cdot Q} \quad p = 13.07 \text{ in} \]

Therefore, based on the above pitch computations to satisfy the fatigue limit state, use the following pitch throughout the entire girder length:

\[ p = 10 \cdot \text{in} \]
Shear Connector Pitch

The shear connector pitch does not necessarily have to be the same throughout the entire length of the girder. Many girder designs use a variable pitch, and this can be economically beneficial.

However, for this design example, the required pitch for fatigue does not vary significantly over the length of the bridge. Therefore, a constant shear connector pitch of 10 inches will be used.

In addition, the shear connectors must satisfy the following pitch requirements:

\[ p \leq 24\text{ in} \quad \text{OK} \]
\[ p \geq 6 \cdot d \]
\[ d = 0.875 \cdot \text{in} \quad 6 \cdot d = 5.25 \text{ in} \quad \text{OK} \]

For transverse spacing, the shear connectors must be placed transversely across the top flange of the steel section and may be spaced at regular or variable intervals.

Stud shear connectors must not be closer than 4.0 stud diameters center-to-center transverse to the longitudinal axis of the supporting member.

\[ 4 \cdot d = 3.50 \text{ in} \quad \text{OK} \]

\[ \text{Spacing}_{\text{transverse}} = 5.0 \text{ in} \quad \text{(see Figure 5-1)} \quad \text{OK} \]

In addition, the clear distance between the edge of the top flange and the edge of the nearest shear connector must not be less than 1.0 inch.

\[ \text{Distance}_{\text{clear}} = \frac{14\text{ in}}{2} - 5\text{ in} - \frac{d}{2} \quad \text{(see Figure 5-1)} \]

\[ \text{Distance}_{\text{clear}} = 1.56 \text{ in} \quad \text{OK} \]

The clear depth of concrete cover over the tops of the shear connectors should not be less than 2.0 inches, and shear connectors should penetrate at least 2.0 inches into the deck. Based on the shear connector penetration information presented in Table 5-1, both of these requirements are satisfied.
For the strength limit state, the factored resistance of the shear connectors, $Q_r$, is computed as follows:

$$Q_r = \phi_{sc} \cdot Q_n$$

$\phi_{sc} = 0.85$

The nominal shear resistance of one stud shear connector embedded in a concrete slab is computed as follows:

$$Q_n = 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} \leq A_{sc} \cdot F_u$$

$$A_{sc} = \pi \cdot \frac{d^2}{4}$$

$$A_{sc} = 0.601 \text{ in}^2$$

$$f'_c = 4.0 \cdot \text{ksi}$$ (see Design Step 3.1)

$$E_c = 3834 \cdot \text{ksi}$$ (see Design Step 3.3)

$$F_u = 60.0 \cdot \text{ksi}$$

$$0.5 \cdot 0.601 \cdot \sqrt{4.0 \cdot 3834} = 37.21 \text{ K}$$

$$0.601 \cdot 60.0 = 36.06 \text{ K}$$

Therefore, $Q_n = 36.06 \cdot \text{K}$

$$Q_r = \phi_{sc} \cdot Q_n$$

Therefore, $Q_r = 30.65 \text{K}$

The number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment or between each adjacent point of 0.0 moment and the centerline of an interior support must not be less than the following:

$$n = \frac{V_h}{Q_r}$$

The total horizontal shear force, $V_h$, between the point of maximum positive moment and each adjacent point of 0.0 moment is equal to the lesser of the following:

$$V_h = 0.85 \cdot f'_c \cdot b \cdot t_s$$

or

$$V_h = F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_t \cdot t_t + F_{yc} \cdot b_f \cdot t_f$$
where \( f'_c = 4.0 \text{ksi} \) (see Design Step 3.1) \( \text{S5.4.2.1} \)
\( b = 103.0 \text{-in} \) (see Design Step 3.3) \( \text{S}5.4.2.1 \)
\( t_s = 8.0 \text{-in} \) (see Design Step 3.1) \( \text{S}5.4.2.1 \)
\( F_{yw} = 50 \cdot \text{ksi} \) (see Design Step 3.1) \( \text{STable 6.4.1-1} \)
\( D = 54 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)
\( t_w = 0.50 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)
\( F_{yt} = 50 \cdot \text{ksi} \) (see Design Step 3.1) \( \text{STable 6.4.1-1} \)
\( b_t = 14 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)
\( t_t = 0.875 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)
\( F_{yc} = 50 \cdot \text{ksi} \) (see Design Step 3.1) \( \text{STable 6.4.1-1} \)
\( b_f = 14 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)
\( t_f = 0.625 \cdot \text{in} \) (see Design Step 3.18) \( \text{STable 6.4.1-1} \)

\[
0.85 \cdot f'_c \cdot b \cdot t_s = 2802 \text{K}
\]
\[
F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_t \cdot t_t + F_{yc} \cdot b_f \cdot t_f = 2400 \text{K}
\]

Therefore, \( V_h = 2400 \cdot \text{K} \)

Therefore, the number of shear connectors provided between the section of maximum positive moment and each adjacent point of 0.0 moment must not be less than the following:

\[
n = \frac{V_h}{Q_r}
\]

\[
n = 78.3
\]

The distance between the end of the girder and the location of maximum positive moment is approximately equal to:

\[
L = 48.0 \cdot \text{ft}
\]

(see Table 3-7)

Similarly the distance between the section of the maximum positive moment and the point of dead load contraflexure is approximately equal to:

\[
L = 83.6 \cdot \text{ft} - 48.0 \cdot \text{ft}
\]

(see Table 3-7)

\[
L = 35.6 \text{ft}
\]
Using a pitch of 10 inches, as previously computed for the fatigue limit state, and using the minimum length computed above, the number of shear connectors provided is as follows:

\[
\frac{L \cdot \left( \frac{12 \text{ in}}{\text{ft}} \right)}{p} = 3 \cdot \frac{L}{p}
\]

\[L = 35.6 \text{ ft}, \quad p = 10 \text{ in}\]

\[n = 128.2 \quad \text{OK}\]

For continuous span composite sections, the total horizontal shear force, \(V_h\), between each adjacent point of 0.0 moment and the centerline of an interior support is equal to the following:

\[V_h = A_r \cdot F_{yr}\]

where \(A_r = 12.772 \text{ in}^2\) (see Design Step 3.3)

\[F_{yr} = 60 \text{ ksi}\] (see Design Step 3.1)

\[V_h = A_r \cdot F_{yr}\]

\[V_h = 766 \text{ K}\]

Therefore, the number of shear connectors provided between each adjacent point of 0.0 moment and the centerline of an interior support must not be less than the following:

\[n = \frac{V_h}{Q_r}\]

\[n = 25.0\]

The distance between the point of dead load contraflexure and the centerline of the interior support is approximately equal to:

\[L = 120 \cdot \text{ft} - 83.6 \cdot \text{ft}\] (see Table 3-7)

\[L = 36.4 \text{ ft}\]

Using a pitch of 10 inches, as previously computed for the fatigue limit state, the number of shear connectors provided is as follows:

\[
\frac{L \cdot \left( \frac{12 \text{ in}}{\text{ft}} \right)}{p} = 3 \cdot \frac{L}{p}
\]

\[p = 10 \text{ in}\]

\[n = 131.0 \quad \text{OK}\]
Therefore, using a pitch of 10 inches for each row, with three stud shear connectors per row, throughout the entire length of the girder satisfies both the fatigue limit state requirements of S6.10.7.4.1 and S6.10.7.4.2 and the strength limit state requirements of S6.10.7.4.4.

Therefore, use a shear stud spacing as illustrated in the following figure.

![Figure 5-2 Shear Connector Spacing](image)

**Figure 5-2 Shear Connector Spacing**

**Design Step 5.2 - Design Bearing Stiffeners**

Bearing stiffeners are required to resist the bearing reactions and other concentrated loads, either in the final state or during construction.

For plate girders, bearing stiffeners are required to be placed on the webs at all bearing locations and at all locations supporting concentrated loads.

Therefore, for this design example, bearing stiffeners are required at both abutments and at the pier. The following design of the abutment bearing stiffeners illustrates the bearing stiffener design procedure.

The bearing stiffeners in this design example consist of one plate welded to each side of the web. The connections to the web will be designed to transmit the full bearing force due to factored loads and is presented in Design Step 5.3.