DE Attachments

Design Examples

- **Attachment DE1**: SDC A Design Flow Chart
  - Flow chart for design of precast bent cap connections in SDC A
- **Attachment DE2**: SDC A Design Example—Grouted Duct Connection
  - Design example for grouted duct connection in SDC A (minimum joint reinforcement)
- **Attachment DE3**: SDC A Design Example—Cap Pocket Connection
  - Design example for cap pocket connection in SDC A (minimum joint reinforcement)
- **Attachment DE4**: SDCs B, C, and D Design Flow Chart
  - Flow chart for design of precast bent cap connections in SDCs B, C, and D
- **Attachment DE5**: SDC B Design Example—Grouted Duct Connection
  - Design example for grouted duct connection in SDC B (minimum joint reinforcement)
- **Attachment DE6**: SDC B Design Example—Cap Pocket Connection
  - Design example for cap pocket connection in SDC B (minimum joint reinforcement)
- **Attachment DE7**: SDCs C and D Design Example—Grouted Duct Connection
  - Design example for grouted duct connection in SDCs C and D (additional joint reinforcement)
- **Attachment DE8**: SDCs C and D Design Example—Cap Pocket Connection
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- **Attachment DE9**: SDCs C and D Design Example—Hybrid Connection
  - Design example for hybrid connection in SDCs C and D
- **Attachment DE10**: SDCs C and D Design Example—Integral Connection
  - Design example for integral connection in SDCs C and D
Joint performance shall satisfy Article 8.13.1.

**IF:** Bent connection is a "Knee" joint per Article 8.13

- Provide special analysis and detailing of joint.

**IF:** Bent connection is a "T" joint per Article 8.13

**IF:** Bent cap is integral

- See integral joint shear design flow chart

**IF:** Bent cap is Cast-In-Place

**IF:** Bent cap is nonintegral

**IF:** Bent cap is precast

- Provide longitudinal development to satisfy:
  \[ \sigma_c \geq 2d_{ct} f_y / f'_{cg} \] (Eq. 8.15.2.2.2-1 Modified) for grouted duct;
  \[ \sigma_c \geq 2.3d_{ct} f_y / f'_{c} \] (Eq. 8.15.2.2.2-2 Modified) for cap pocket.

- Provide transverse confinement to satisfy
  \[ \rho_s \geq 0.11 \sqrt{f''_{c}} / f_{nh} \] (Eq. 8.15.3.1-1)

**IF:**

- Bedding layer \(\leq 3"\) (Article 8.15.5.2.1)
  - Place bedding layer confinement per Article 8.15.5.2.1 and place top column hoop per Article 8.8.14.

**IF:**

- Bedding layer \(> 3"\) (Article 8.15.5.2.1)
  - Use Owner established provisions for bedding layer confinement and place top column hoop per Article 8.8.14.
Grouted Duct Joint Design Example For SDC A

An SDC A grouted duct connection between a column and a precast bent cap is not expected to be subjected to significant seismic demand. Therefore, the joint stresses in the connection are not checked; however, limited joint shear reinforcement is required. This SDC A design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Note: 1. The provisions of 8.13.1 apply to all levels within SDC A, but where $S_{D1}$ is less than 0.10, the designer has the option of using alternative precast bent cap connections such as those detailed in Matsumoto et al., 2001.

Geometry and Design Parameters

- $f'_c = 4.0$ ksi (specified compressive strength of bent cap)
- $f'_{ce} = 1.3f'_c = 5.2$ ksi (expected $f'_c$ of bent cap)
- $f'_{cg} = 7.5$ ksi (specified compressive strength of grout)
  - Check: $f'_{cg} \geq \max\{1.25(f'_{ce}+0.5)=7$ ksi, $6$ ksi\}
  - OK per BCS 8.13.8.3.2a
- $f_y = 60$ ksi (yield stress of column bars)
- $f_{yh} = 60$ ksi (yield stress of hoops)
- Bent cap width = 5.25 ft ($B_{cap}$)
- Bent cap height = 5.00 ft ($D_s$)
- Stirrup bar size: #6
- Column diameter = 4.0 ft ($D_c$)
- $A_{st} = 25.4$ in² (#10 Tot 20)
- Hoop bar size: #6
- Hoop spacing = 4.0 in

Joint Performance

Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases. The bent cap section capacity is not investigated in this design example.

Joint Proportioning (Minimum Development Length)

$$l_{ac} \geq 2d_{ch}f_y$$
$$\frac{f'_{cg}}{}$$

Eq. SGS 8.13.2.2
(modified)
dbl = 1.27 in (#10 rebar)

$ f_y = 60 \text{ ksi} $ (Article 8.13.2.2 permits use of $ f_y $ instead of $ f_{y,h} $)

$ f'_{cg} = 7.0 \text{ ksi} = \text{min (7.5 ksi, 7.0 ksi)} $ per Article 8.15.2.2

$l_{ac} = 21.8 \text{ in} \quad \text{(minimum)}$

Extend column reinforcement as far as practically possible; assume 3” clear from opposite face.

$l_{ac} = D_s - 3" = 5.0 \text{ ft x 12"/ft} - 3" = 57.0 \text{ in}$

$57.0 \text{ in} \geq 21.8 \text{ in minimum} \quad \text{OK} \quad \text{Extend to top face of cap less 3 in cover.}$

**Minimum Joint Reinforcing**

Transverse reinforcement in the form of hoops will be placed to encompass the ducts and thereby satisfy the minimum joint reinforcement requirement. The size and spacing of this reinforcement is determined by applying the provisions of Article 8.13.3.2.1 and Eq. 8.15.3.1-1.

Calculate required volumetric ratio of transverse joint reinforcement:

$$ \rho_s \geq 0.11 \sqrt{f'_{c}} / f_{y,h} \geq 0.0037 \quad \text{Eq. 8.15.3.1-1} $$

$$ \rho_s \geq 0.004 $$

Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

$$ \rho_s = \frac{4A_{sp}}{D'_{gd} s} $$

$ A_{sp} = 0.44 \text{ in}^2 \quad \text{(#6 hoop)}$

$ D'_{gd} = 45.92 \text{ in} \quad \text{(confined diameter of column between centroids of #6 hoop)}$

$ s = 10.0 \text{ in} \quad \text{(trial spacing of transverse reinforcement hoops, 12" or 0.3D_s maximum per Art. 8.13.3.2.1)}$

$ D'_{gd} = \text{Column diameter - clear cover x 2} - \text{hoop diameter - long}$

$ + \text{column reinf diameter + grouted duct inside diameter +}$

$ \text{corrugation amplitude x 2.}$

$ D'_{gd} = 48.0" - 2" x 2 - 0.88" - 1.44" + 4" + 0.12" x 2 = 45.92 \text{ in}$

(deformed diameters are used for clearance calculations)

$ \rho_s = 0.0038 \quad \text{OK} \quad 10" \text{ spacing is adequate (Note: Common practice is to}$

$\text{carry hoop spacing from column into cap, or s=4" in}$

$\text{this case.)}$

$\text{Actual even spacing is 9.37 inches, total 6 hoops.}$
Nonintegral Bent Cap Joint Shear Design

Vertical Stirrups Inside the Joint Region

\[ A_{svj} \geq 0.08 A_{st} \]

\[ A_{st} = 25.40 \text{ in}^2 \]

\[ A_{svj} \geq 2.03 \text{ in}^2 \]

Use #6 single U stirrups Tot 3 patterns placed evenly through joint

\[ A_{svj} = 3 \text{ patterns} \times 2 \text{ legs / pattern} \times 0.44 \text{ in}^2 / \text{leg} = 2.64 \text{ in}^2 \geq 2.03 \text{ in}^2 \text{ OK} \]

Note: Three patterns are required and used. The minimum number of stirrups is two, per Article 8.13.4.2.2a, and the bar size is no smaller than what is used in the bent cap shear stirrups. If only two stirrups were required, three may be used to reduce the spacing between stirrups to satisfy temperature and shrinkage requirements for side faces of the bent cap per AASHTO LRFD Article 5.10.8.

Bedding Layer Reinforcement

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3” thick. Cover on top column hoop to be 2” as shown in figures below, and resulting center to center spacing of transverse reinforcement is 4” throughout, in accordance with Article 8.8.14.

Figures Showing Final Design

Elevation View at Column Connection
Typical Section through Joint

- #10, Tot 12 (6 bundles)
- #6 hoops, Tot 6, even spacing
- #10, Tot 12 (6 bundles)

See detail below

#6 hoops @ 4"

4" overall spacing maintained

Simplified Section at Column Edge

- Bedding: 1.5" per Art. 8.13.3.2.1
- Layer: 2.5" per Art. 8.8.14

Clear cover may be less than 2" per Art. 8.8.14 as required.
Plan View at Column Connection

For additional details, see Figures 8.13.4.2.2-1, 8.13.4.2.2-2, and 8.13.4.2.2-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Cap Pocket Joint Design Example For SDC A

An SDC A cap pocket connection between a column and a precast bent cap is not expected to be subjected to significant seismic demand. Therefore, the joint stresses in the connection are not checked; however, limited joint shear reinforcement is required. This SDC A design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Note: 1. The provisions of 8.13.1 apply to all levels within SDC A, but where SD1 is less than 0.10, the designer has the option of using alternative precast bent cap connections such as those detailed in Matsumoto et al., 2001.

Geometry and Design Parameters

\[ f'c = 4.0 \text{ ksi} \]  \hspace{1cm} \text{specified compressive strength of bent cap}

\[ f'_{ce} = 1.3 f'c = 5.2 \text{ ksi} \]  \hspace{1cm} \text{expected } f'c \text{ of bent cap}

\[ f'_{c,pocket} \]  \hspace{1cm} \text{specified compressive strength of pocket fill}

Select cap pocket strength to satisfy BCS.

\[ f'_{c,pocket} = 1.3 f'c + 0.5 \text{ ksi} = 5.7 \text{ ksi} \]  \hspace{1cm} \text{(BCS 8.13.8.3.3a)}

\[ f_{yh} \]  \hspace{1cm} \text{yield strength of equivalent hoop}

\[ f_{yp} \]  \hspace{1cm} \text{nominal yield stress of steel pipe}

\[ \theta \]  \hspace{1cm} \text{angle between horizontal axis of cap and pipe helical corrugation or lock seam}

\[ f_y = 60 \text{ ksi} \]  \hspace{1cm} \text{yield stress of column bars}

Bent cap width = 5.25 ft \( B_{cap} \)

Bent cap height = 5.00 ft \( D_s \)

Stirrup bar size: #6

Column diameter = 4.0 ft \( D_c \)

\[ A_{st} = 25.4 \text{ in}^2 \]  \hspace{1cm} \#10 Tot 20

Hoop bar size: #6

Hoop spacing = 4.0 in

Joint Performance

Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases. The bent cap section capacity is not investigated in this design example.
Joint Proportioning (Minimum Development Length)

\[ l_{ac} \geq \frac{2.3d_{bl} f_y}{f'_c} \]  

\[ d_{bl} = 1.27 \text{ in} \quad (#10 \text{ bar diameter}) \]
\[ f_y = 60 \text{ ksi} \quad \text{(Article 8.13.2.2 permits use of } f_y \text{ instead of } f_{ye}) \]
\[ f'_c = 5.7 \text{ ksi} \quad \text{(cap pocket concrete) } = \min (5.7 \text{ ksi, } 7.0 \text{ ksi}) \text{ per Article 8.15.2.2.2} \]

\[ l_{ac} = 30.7 \text{ in} \quad \text{(minimum)} \]

\[ l_{ac} = D_s - 3" = 5.0 \text{ ft x } 12"/\text{ft} - 3" = 57.0 \text{ in} \]

\[ 57.0 \text{ in} \geq 30.7 \text{ in minimum } \quad \text{OK} \quad \text{Extend to top face of cap less 3 in cover.} \]

Minimum Joint Reinforcing

Minimum thickness of the helical corrugated steel pipe confinement is determined from Eq. 8.15.3.2.2-1 and Eq. 8.15.3.2.2-2. This more involved calculation can be replaced by Eq. 8.13.3.2.2-1, a simplified equation that provides a more conservative wall thickness.

Calculate required volumetric ratio of transverse joint reinforcement:

\[ \rho_s \geq 0.11 \sqrt{\frac{f'_c}{f_{yh}}} \quad \text{Eq. 8.15.3.1-1} \]

\[ \rho_s \geq 0.004 \]

Use the minimum volumetric ratio to calculate the required number of equivalent #6 hoops per unit length of corrugated steel pipe. Use a unit length of 1 foot. Use a 48" ID corrugated pipe based on the outer diameter of column reinforcement pattern.

\[ \rho_s = \frac{4A_{sp}}{D'_{cp} s} \quad s = \frac{4A_{sp}}{D'_{cp} \rho_s} \]

\[ A_{sp} = 0.44 \text{ in}^2 \quad \text{(assume #6 hoop to match column transverse reinforcement)} \]
\[ D'_{cp} = 48.65 \text{ in} \quad \text{(average confined dia. of column between corrugated steel pipe walls)} \]
\[ \rho_s = 0.0037 \quad \text{(minimum volumetric ratio)} \]

\[ D'_{cp} = \text{Nominal inside diameter of corrugated pipe + average wall corrugation width.} \]

\[ s = 9.9 \text{ in max spacing} \]

Therefore, the number of equivalent hoops per foot is \( 12" / s = 1.216 \text{ hoops/ft} \)
Calculate the nominal confining hoop force of the equivalent hoops.

\[ F_H = n_h A_{sp} f_{yh} \]  
\[ n_h = 1.216 \text{ ea} \quad \text{(number of equivalent hoops per unit length)} \]
\[ A_{sp} = 0.44 \text{ in}^2 \quad \text{(area of #6 equivalent hoop)} \]
\[ f_{yh} = 60.0 \text{ ksi} \quad \text{(yield strength of equivalent hoop)} \]

\[ F_H = 32.11 \text{ kips/ft} \]

Calculate the thickness of the corrugated steel pipe that provides the same nominal confinement as standard hoop reinforcement.

\[ t_{pipe} \geq \max \left[ \frac{F_H}{H_p f_{yp} \cos \theta} \right] \quad \text{Eq. 8.15.3.2.2-1} \]

\[ F_H = 32.1 \text{ kips/ft} \]
\[ H_p = 12.0 \text{ in/ft} \quad \text{(specified unit length)} \]
\[ f_{yp} = 30.0 \text{ ksi} \quad \text{(manufacturer specified)} \]
\[ \theta = 20.0 \text{ deg} \quad \text{(manufacturer specified)} \]

\[ t_{pipe} \geq 0.0949 \text{ in} \]

Use a 12 gage corrugated steel pipe, 48" nominal inside diameter. \[ t_{pipe} = 0.105 \text{ in} \]

As a check, compare \( t_{pipe} \) from the simplified Eq. C8.13.3.2.2-1 to that from Eq. 8.15.3.2.2-1:

\[ t_{pipe} \geq 0.04 \frac{D'}{f_{cp}} \sqrt{f_{c}'} = 0.1381 \text{ in} \quad \text{and} \geq 0.06 \text{ in} \quad \text{(Note: } f_{c}' \text{ refers to cap pocket)} \quad \text{Eq. C8.13.3.2.2-1} \]

If the refined equation (Eq. 8.15.3.2.2-1) is not used, the simplified equation may be used because it provides a more conservative value. Note that the thickness of 0.1381" from Eq. 8.13.3.2.2-1 is considerably larger than the 0.0949" calculated from the refined equation. Use of the refined equation reduces the required pipe thickness, especially for larger diameter pipes.
Nonintegral Bent Cap Joint Shear Design

Vertical Stirrups Inside the Joint Region

\[ A_{s}^{jvi} \geq 0.08 \ A_{st} \]

\[ A_{st} = 25.40 \ \text{in}^2 \]

\[ A_{s}^{jvi} \geq 2.03 \ \text{in}^2 \]

Use #6 double U stirrups, Tot 3 patterns placed evenly through joint.

\[ A_{s}^{jvi} = 3 \ \text{patterns} \times 2 \ \text{legs/pattern} \times 0.44 \ \text{in}^2/\ \text{leg} = 2.64 \ \text{in}^2 \geq 2.03 \ \text{in}^2 \quad \text{OK} \]

Note: Three patterns are required and used. The minimum number of stirrups is two, per Article 8.13.4.2.2a, and the bar size is no smaller than what is used in the bent cap shear stirrups. If only two stirrups were required, three may be used to reduce the spacing between stirrups to satisfy temperature and shrinkage requirements for side faces of the bent cap per AASHTO LRFD Article 5.10.8.

Bedding Layer Reinforcement

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below and resulting center to center spacing of transverse reinforcement is 4" throughout, in accordance with Article 8.8.14.

Figures Showing Final Design

[Diagram of bedding layer reinforcement and stirrups]

Elevation View at Column Connection
#10, Tot 12 (6 bundles)

#6 stirrups, Tot 3

5'-3"

5'-0"

#10, Tot 12 (6 bundles)

48" ID - 12 gage corrugated steel pipe

See detail below

#6 hoops @ 4"

Bedding layer

#10 Column Reinf

Typical Section through

4" overall spacing maintained

Bent Cap

Bedding

Layer

Column

1.5" per Art. 8.13.3.2.1

2.5" per Art. 8.8.14

Clear cover may be less than 2" per Art. 8.8.14 as required.
For additional details, see Figures 8.13.4.2.3-1, 8.13.4.2.3-2, and 8.13.4.2.3-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Joint performance shall satisfy Article 8.15.1; 8.14.1*. Evaluate principal tension and compression stresses in joint per Article 8.15.2 and concurrently satisfy Article 8.15.2.2.

**IF:**
Principal compression \( \leq 0.25f'_c \) and principal tension \( \leq 0.38\sqrt{f''_c} \) (Article 8.15.2.1)

**IF:**
Principal tension \( \geq 0.11\sqrt{f''_c} \) (Article 8.15.3.1)

Provide transverse confinement to satisfy \( \rho_i \geq 0.11\sqrt{f''_c} / f_{yh} \) and \( \rho_i \geq 0.40A_{st} / l^2 \) (Eq. 8.15.3.1-1) and (Eq. 8.15.3.1-2)

**IF:**
Bent cap is CIP in SDC B

provide transverse confinement to satisfy \( \rho_i \geq 0.11\sqrt{f''_c} / f_{yh} \) (Eq. 8.15.3.1-1)

**IF:**
Bent connection is a "Knee" joint per Article 8.15; 8.14*

Provide special analysis and detailing of joint.

**IF:**
Bent connection is a "T" joint per Article 8.15; 8.14*

Provide transverse confinement to satisfy \( \rho_i \geq 0.11\sqrt{f''_c} / f_{yh} \) (Eq. 8.15.3.1-1)

**IF:**
Bent cap is CIP in SDC B

Provide transverse confinement to satisfy \( \rho_i \geq 0.11\sqrt{f''_c} / f_{yh} \) (Eq. 8.15.3.1-1)

**IF:**
Bent cap is not CIP in SDC B

Provide transverse confinement to satisfy \( \rho_i \geq 0.11\sqrt{f''_c} / f_{yh} \) (Eq. 8.15.3.1-1)

**IF:**
Bent cap is nonintegral

**IF:**
Bent cap is nonintegral

**IF:**
Bent cap is integral

See integral joint shear design flow chart

**IF:**
Bent connection is a "Knee" joint per Article 8.15; 8.14*

Provide special analysis and detailing of joint.

**IF:**
Bent connection is a "T" joint per Article 8.15; 8.14*

Provide special analysis and detailing of joint.

**IF:**
Cast-In-Place bent cap

**IF:**
SDCs C or D

Provide inside vertical joint shear stirrups: \( A_{jvi}^{v} \geq 0.12A_{st} \) (Eq. 8.15.5.2.3a-1)

**IF:**
SDC B

Provide supplementary hoops per Article 8.15.5.2.3a

**IF:**
TBD by Others

Provide horizontal "J" bars per Article 8.15.5.1.4.

**IF:**
Precast “Cap pocket” connection

Provide inside vertical joint shear stirrups: \( A_{jvi}^{v} \geq 0.12A_{st} \) (Eq. 8.15.5.2.3a-1)

**IF:**
Precast “Grouted duct” connection

Provide inside vertical joint shear stirrups: \( A_{jvi}^{v} \geq 0.135A_{st} \) (Eq. 8.15.5.1.2-1)

**IF:**
Provide horizontal “J” bars per Article 8.15.5.1.4.

**IF:**
Bedding layer \( \leq 3" \) (Article 8.15.5.2.1)

Place bedding layer confinement per Article 8.15.5.2.1 and place top column hoop per Article 8.8.14.

**IF:**
Bedding layer \( > 3" \) (Article 8.15.5.2.1)

Use Owner established provisions for bedding layer confinement and place top column hoop per Article 8.8.14.

*Articles referenced in flow chart are for SDCs C and D; alternate article references for SDC B are shown italicized following the related SDCs C and D reference as applicable.
Grouted Duct Joint Design Example For SDC B

An SDC B grouted duct connection between a column and a precast bent cap is designed to produce performance similar to an SDC B cast-in-place connection; however, it is required that the principal stress in the connection be checked. This SDC B design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns for which the principal tensile stress in the connection does not exceed 0.11√f’c. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Notes: 1. When the principal tensile stress exceeds the specified limit, the joint design follows the procedure found in the corresponding SDC C and D design example. 2. Knee joints such as that represented in C8.15.1 are not addressed in the current SGS, and special design provisions should be identified by the designer.

Geometry and Design Parameters

\[ f'_c = 4.0 \text{ ksi} \quad \text{(specified compressive strength of bent cap)} \]
\[ f'_{ce} = 1.3f'_c = 5.2 \text{ ksi} \quad \text{(expected f’c of bent cap)} \]
\[ f'_{cg} = 7.5 \text{ ksi} \quad \text{(specified compressive strength of grout)} \]

Check: \[ f'_{cg} (\text{ksi}) \geq \max[1.25(f'_{ce}+0.5)=7.0, 6.0)] \]
OK per BCS 8.13.8.3.2a

\[ f_y = 60 \text{ ksi} \quad \text{(yield stress of column bars)} \]
\[ f_{ye} = 68 \text{ ksi} \quad \text{(expected yield stress)} \]
\[ f_{yh} = 60 \text{ ksi} \quad \text{(yield stress of hoops)} \]

Bent cap width = 5.25 ft (B_{cap})
Bent cap height = 5.00 ft (D_{s})
Stirrup bar size: #6

Column diameter = 4.0 ft (D_{c})
\[ A_{st} = 25.4 \text{ in}^2 \quad (#10 \text{ Tot 20}) \]

Hoop bar size: #6
Hoop spacing = 4.0 in

Joint Performance

Column sections (possibly governed by load cases other than Extreme Event (seismic) load case for SDC B) are analyzed to determine the idealized plastic moment capacity, M_p. Connections are designed for the lesser of two forces: 1) those produced by the column plastic hinging overstrength moment capacity, M_{po} or 2) unreduced elastic seismic moment in a column (i.e., the ultimate moment demand for seismic load combination, M_u). In the case where M_u exceeds M_p but not M_{po}, M_{po} is conservatively used for design because significant plastic hinging may develop. See Article 8.5 and C8.14.1 for further discussion.

Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.

Note: Bent cap reinforcement shown reflects that required by load combinations before application of joint shear design requirements.
Determine the idealized plastic moment capacity of the column, $M_p$, using section analysis program such as xSECTION, and compare to the ultimate seismic moment demand, $M_u$

\[ M_p = 3769 \text{ kip-ft} \]

Ultimate moment demand in column under seismic load combination, $M_u = 2411$ kip-ft (given)

\[ 3769 \text{ kip-ft} \geq 2411 \text{ kip-ft} \]

Design moment for joint proportioning = 2411 kip-ft

Note: This case shows the ultimate seismic moment demand, $M_u$, to be less than $M_p$; therefore, $M_u$ is used for design. If $M_u$ had been greater than $M_p$, the plastic overstrength moment capacity, $M_{po}$, would be determined per Eq. 8.5-1 and used for design.

**Joint Proportioning**

Joint proportioning for SDC B follows the provisions of Article 8.15.2 for SDCs C and D.

Principal stresses in the joint are limited by the following equations:

\[ p_c \leq 0.25 f'_c = 1.00 \text{ ksi maximum} \]

\[ p_t \leq 0.38 \sqrt{f'_c} = 0.76 \text{ ksi maximum} \]

Notes:

1) Length of column longitudinal rebar extended into cap. See calculations on Sheet 3.
2) No prestressing in section.
3) Not necessarily $M_{po}$, but that determined from 8.14.1. Tension in column longitudinal rebar may also be derived from sectional analysis.
Minimum Development Length of Column Longitudinal Reinforcement

\[ l_a \geq \frac{2d_{bl}f_{ye}}{f'_{cg}} \]  

\[ d_{bl} = 1.27 \text{ in (2#10 rebar)} \]
\[ f_{ye} = 68 \text{ ksi} \]
\[ f'_{cg} = 7.0 \text{ ksi} \]

\[ l_a = 24.7 \text{ in (minimum)} \]

Extend column reinforcement as far as practically possible, assume 3” clear from opposite face.

\[ l_{ac} = D_s - 3" = 5 \text{ ft x 12"/ft} - 3" = 57.0 \text{ in} \]

\[ 57.0 \text{ in} \geq 24.7 \text{ in minimum} \]

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections

Where the principal tension in the joint is less than 0.11\(\sqrt{f'_{c}}\), and joint proportioning is acceptable per Article 8.14.2 (8.15.2), the provisions of Article 8.14.5.2.2a for \(A_{jvi}\) and should reference Eq. 8.15.3.1-1 for \(\rho_s\) are to be satisfied. However, other joint reinforcing \((A_{jvo}, A_{jl}, \text{and horizontal J-bars})\) is not required.
Check if principal tension, p_t, is \( \geq 0.11\sqrt{f'c} \) (condition for likely joint cracking):

\[
\text{Calculated tension} = \frac{0.096}{\text{ksi}} \\
\text{Limit, } 0.11\sqrt{f'c} = \frac{0.220}{\text{ksi}}
\]

\[ p_t = \frac{0.096}{\text{ksi}} \leq \frac{0.220}{\text{ksi}} \text{ limit } \]

Joint reinf. provisions of Article 8.14.5.2a apply.

Calculate required volumetric ratio of transverse joint reinforcement:

\[
\rho_s \geq 0.11\sqrt{f'c}/f_{yh} \quad \text{Eq. 8.15.3.1-1}
\]

\[
\rho_s \geq 0.0037
\]

Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

\[
\rho_s = \frac{4A_{sp}}{D'gd \cdot s}
\]

\[ A_{sp} = 0.44 \text{ in}^2 \quad \text{(6 hoop)} \]

\[ D'gd = 45.92 \text{ in} \quad \text{(confined diameter of column between centroids of #6 hoop)} \]

\[ s = 10.0 \text{ in} \quad \text{(trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or 0.3Ds per Art. 8.15.3.2.1)} \]

\[
D'gd = \text{Column diameter - clear cover x 2 - hoop diameter - long, column reinf diameter + grouted duct inside diameter + corrugation amplitude x 2.}
\]

\[ D'gd = 48.0" - 2" x 2 - 0.88" - 1.44" + 4" + 0.12" x 2 = 45.92 \text{ in}} \]

(deformed diameters are used for clearance calculations)

\[
\rho_s = \frac{0.0038}{0.0037} \geq \text{minimum } \quad \text{OK} \quad 10" \text{ spacing is adequate (Note: Common practice is to carry hoop spacing from column into cap, or } s = 4" \text{ in this case.)}
\]

Actual even spacing is 9.37 inches, total 6 hoops.

**Nonintegral Bent Cap Joint Shear Design**

The depth of the cap with respect to the column diameter determines the method of joint shear design. When the following equation is satisfied, the joint shear design provisions of 8.14.5.2 apply. If the equation below is not satisfied, Strut-and-Tie model provisions of AASHTO LRFD Bridge Design Specifications apply.

\[
D_c \leq d \leq 1.25D_c
\]

\[ D_c = 48.0 \text{ in} \]

\[ d = 60.0 \text{ in} \]

\[ 48.0 \text{ in} \leq 60.0 \text{ in} \leq 60.0 \text{ in} \quad \text{OK} \quad \text{Provisions of 8.14.5.2 apply} \]
Vertical Stirrups Inside the Joint Region

\[ A_s^{jvi} \geq 0.10 \ A_{st} \]

\[ A_{st} = 25.40 \ \text{in}^2 \]

\[ A_s^{jvi} \geq 2.54 \ \text{in}^2 \]

Use #6 double U stirrups, Tot 3 patterns placed evenly through joint. Note that spacing may vary slightly to avoid conflict with grouted ducts.

\[ A_s^{jvi} = 3 \ \text{patterns} \times 2 \ \text{legs/pattern} \times 0.44 \ \text{in}^2/\text{leg} = 2.64 \ \text{in}^2 \geq 2.54 \ \text{in}^2 \quad \text{OK} \]

Note: The minimum number of stirrups is two, per Article 8.14.5.2.2a with a bar size no smaller than that used for bent cap stirrups.

Bedding Layer Reinforcement

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below, and resulting center to center spacing of transverse reinforcement is 4" throughout, in accordance with Article 8.8.14.

Figures Showing Final Design
AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL GROUTED DUCT BENT CAP JOINT DESIGN EXAMPLE - SDC B

Typical Section through Joint

- #10, Tot 12 (6 bundles)
- #6 hoops, Tot 6, even spacing
- #10, Tot 12 (6 bundles)
- Bedding layer
- #10 Column Reinf

Simplified Section at Column Edge

- Bent Cap
- Bedding Layer
- Column

4" overall spacing maintained

- 1.5" per Art. 8.14.5.2.1
- 2.5" per Art. 8.8.14

Clear cover may be less than 2" per Art. 8.8.14 as required.

See detail below

- #6 hoops @ 4"
- #6 stirrups, Tot 3

4'‐0"

5'‐3"

5'‐0"
#6 Stirrups @ 8"

#10, Tot 12
(6 bundles)

#6 Stirrups, Tot 3

#6 Stirrups @ 8"

Grouted Duct

Column

#6 Hoops

Plan View at Column Connection

For additional details, see Figures 8.14.5.2.2-1, 8.14.5.2.2-2, and 8.14.5.2.2-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Cap Pocket Joint Design Example For SDC B

An SDC B cap pocket connection between a column and a precast bent cap reinforces the joint by means of a helical lock-seam, corrugated steel pipe, eliminating conventional hoops and J-bars. This SDC B design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns for which the principal tensile stress in the connection does not exceed 0.11\(\sqrt{f'c}\). The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

**Notes:** 1. When the principal tensile stress exceeds the specified limit, the joint design follows the procedure found in the corresponding SDC C and D design example. 2. Knee joints such as that represented in C8.15.1 are not addressed in the current SGS, and special design provisions should be identified by the designer.

**Geometry and Design Parameters**

- \(f_c = 4.0\) ksi (specified compressive strength of bent cap)
- \(f'c = 1.3 f_c = 5.2\) ksi (expected \(f_c\) of bent cap)
- \(f'_c_{\text{pocket}}\) (specified compressive strength of pocket fill)

  Select cap pocket strength to satisfy BCS.
  \(f'_c_{\text{pocket}} = 1.3 f'c + 0.5 ksi = 5.7 ksi\) (BCS 8.13.8.3a)

- \(f_{yh}\) (yield stress of equivalent hoop)
- \(f_{yp}\) (nominal yield stress of steel pipe)
- \(\theta\) (angle between horizontal axis of cap and pipe helical corrugation or lock seam)
- \(f_p = 60\) ksi (yield stress of column bars)
- \(f_{ye} = 68\) ksi (expected yield stress of column bars)

Bent cap width = 5.25 ft \((B_{\text{cap}})\)
Bent cap height = 5.00 ft \((D_s)\)
Stirrup bar size: #6

Column diameter = 4.0 ft \((D_c)\)
\(A_{st} = 25.4\) in\(^2\) \(#10\ \text{Tot}\ 20\)

Hoop bar size: #6
Hoop spacing = 4.0 in

**Joint Performance**

Column sections (possibly governed by load cases other than Extreme Event (seismic) load case for SDC B) are analyzed to determine the idealized plastic moment capacity, \(M_p\). Connections are designed for the lesser of two forces: 1) those produced by the column plastic hinging overstrength moment capacity, \(M_{po}\) or 2) unreduced elastic seismic moment in a column (i.e., the ultimate moment demand for seismic load combination, \(M_u\)). In the case where \(M_u\) exceeds \(M_p\) but not \(M_{po}\), \(M_{po}\) is conservatively used for design because significant plastic hinging may develop. See Article 8.5 and C8.14.1 for further discussion.

Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.
Determine the idealized plastic moment capacity of the column, $M_p$, using section analysis program such as xSECTION, and compare to the ultimate seismic moment demand, $M_u$.

Axial load acting on column per extreme event load case = 820.0 kips ($P_c$)

$\sigma_{ce}$ (expected concrete compressive strength) = 1.3 x $\sigma_{ce}$, 5.2 ksi

$\sigma_{ye}$ (expected steel yield stress) = 68 ksi

$M_p = 3769 \text{ kip-ft}$

Maximum moment in column under seismic load application, $M_u = 2411 \text{ kip-ft}$

$3769 \text{ kip-ft} \geq 2411 \text{ kip-ft}$

Design moment for joint proportioning = 2411 kip-ft

Note: This case shows the ultimate seismic moment demand, $M_u$, to be less than $M_p$, therefore, $M_u$ is used for design. If $M_u$ had been greater than $M_p$, the plastic overstrength moment capacity, $M_{po}$, would be determined per Eq. 8.5-1 and used for design.

**Joint Proportioning**

Joint proportioning for SDC B follows the provisions of Article 8.15.2 for SDCs C and D.

Principal stresses in the joint are limited by the following equations:

- Principal compression, $p_c$: $p_c \leq 0.25 \sigma_{ce} = 1.00 \text{ ksi maximum}$
- Principal tension, $p_t$: $p_t \leq 0.38 \sqrt{\sigma_{ce}} = 0.76 \text{ ksi maximum}$

Notes:

1) Length of column longitudinal rebar extended into cap. See calculations on Sheet 3.
2) No prestressing in section.
3) Not necessarily $M_{po}$ but that determined from 8.14.1. Tension in column longitudinal rebar may also be derived from sectional analysis.


\[ p_t = \left( \frac{f_h + f_v}{2} \right) - \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v^2_{jv}} \]
\[ p_c = \left( \frac{f_h + f_v}{2} \right) + \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v^2_{jv}} \]

\[ v_{jv} = \frac{T_c}{A_{jv}} = \frac{517 \text{k}}{3591.0 \text{in}^2} = 0.144 \text{ksi} \quad \text{Eq. 8.15.2.1-5} \]
\[ A_{jv} = l_{ac} B_{cap} = 57.0 \text{in x 63.0 in} = 3591.0 \text{in}^2 \quad \text{Eq. 8.15.2.1-6} \]
\[ f_v = \frac{P_c}{A_{jh}} = \frac{820 \text{k}}{6804 \text{in}^2} = 0.121 \text{ksi} \quad \text{Eq. 8.15.2.1-7} \]
\[ A_{jh} = (D_c + D_s) B_{cap} = (48.0 \text{in x 60.0 in}) \times 63.0 \text{in} = 6804.0 \text{in}^2 \quad \text{Eq. 8.15.2.1-8} \]
\[ f_h = \frac{P_h}{B_{cap} D_s} = \frac{0.0 \text{k}}{63.0 \text{in x 60.0 in}} = 0.0 \text{ksi} \quad \text{Eq. 8.15.2.1-9} \]
\[ T_c = \frac{M_{po}}{h} = 2411 \text{kip-ft} / 4.66 \text{ft} = 517 \text{kips} \quad \text{Eq. 8.15.2.1-10} \]

\[ p_t = 0.216 \text{ksi} \leq 1.00 \text{ksi maximum} \quad \text{OK} \quad \text{Joint proportions are acceptable based on principal stress requirements.} \]

\[ p_c = 0.096 \text{ksi} \leq 0.76 \text{ksi maximum} \quad \text{OK} \]

**Minimum Development Length for Column Longitudinal Reinforcement**

\[ l_{ac} \geq 2.3 d_{bl} f_{ye} \]
\[ f'_{c} \]

\[ d_{bl} = 1.27 \text{ in} \quad (\#10 \text{rebar}) \]
\[ f_{ye} = 68 \text{ ksi} \]
\[ f'_{c} = 5.7 \text{ ksi} \quad (\text{cap pocket concrete}) = \min (5.7 \text{ksi}, 7.0 \text{ksi}) \text{ per Article 8.15.2.2.2} \]
\[ l_{ac} = 34.8 \text{ in} \quad (\text{minimum}) \]

Extend column reinforcement as far as practically possible, assume 3” clear from opposite face.

\[ l_{ac} = D_s - 3" = 5.0 \text{ft x 12”/ft} - 3" = 57.0 \text{ in} \]

\[ 57.0 \text{ in} \geq 34.8 \text{ in minimum} \quad \text{OK} \quad \text{Extend to top face of cap less 3 in cover.} \]

**Minimum Joint Reinforcing for Precast Bent Cap Connections**

Where the principal tension in the joint is less than \(0.11 \sqrt{f'_{c}}\) and joint proportioning is acceptable per Article 8.14.2 (8.15.2), the provisions of Article 8.14.5.2.3 for \(A_{jvi}\) Eq. 8.15.3.1-1 for joint transverse reinforcement (i.e., pipe thickness) is to be satisfied. However, other joint reinforcing \((A_{jvo}, A_{jl})\) is not required.
Check if principal tension, $p_t$, is $\geq 0.11\sqrt{f'_c}$ (condition for likely joint cracking)

Calculated tension = 0.096 ksi
Limit, $0.11\sqrt{f'_c}$ = 0.220 ksi

$p_t = 0.096$ ksi $\leq 0.220$ ksi limit $A_{vi}$ joint reinf. provisions of Article 8.14.5.2.3b apply.

Calculate required volumetric ratio of transverse joint reinforcement:

$$\rho_s \geq 0.11 \sqrt{f'_c} / f_{yh}$$

Eq. 8.15.3.1-1

$$\rho_s \geq 0.004$$

Use the minimum volumetric ratio to calculate the required number of equivalent #6 hoops per unit length of corrugated steel pipe. Use a unit length of 1 foot. Use a 48” ID corrugated pipe based on the outer diameter of column reinforcement pattern.

$$\rho_s = \frac{4A_{sp}}{D_{cp}'s}$$

$s = \frac{4A_{sp}}{D_{cp}' \rho_s}$

$$A_{sp} = 0.44 \text{ in}^2 \quad \text{(assume #6 hoop to match column transverse reinforcement)}$$

$$D_{cp}' = 48.65 \text{ in} \quad \text{(average confined dia. of column between corrugated steel pipe walls)}$$

$$\rho_s = 0.0037 \quad \text{(minimum volumetric ratio)}$$

$D_{cp}' = $ Nominal inside diameter of corrugated pipe + average wall corrugation width.

$s = 9.9 \text{ in max spacing}$

Therefore, the number of equivalent hoops per foot is $12" / s = 1.216 \text{ hoops/ft}$

Calculate the nominal confining hoop force of the equivalent hoops.

$$F_H = n_h A_{sp} f_{yh}$$

Eq. 8.15.3.22-2

$$n_h = 1.216 \text{ ea} \quad \text{(number of equivalent hoops per unit length)}$$

$$A_{sp} = 0.44 \text{ in}^2 \quad \text{(area of #6 equivalent hoop)}$$

$$f_{yh} = 60.0 \text{ ksi} \quad \text{(yield stress of equivalent hoop)}$$

$$F_H = 32.11 \text{ kips/ft}$$

Calculate the thickness of the corrugated steel pipe that provides the same nominal confinement as standard hoop reinforcement.

$$t_{pipe} \geq \max \left\{ \frac{F_H}{f_{ypp} \cos \theta}, 0.060 \text{ in} \right\}$$

Eq. 8.15.3.22-1
\[ F_H = 32.1 \text{ kips/ft} \]
\[ H_p = 12.0 \text{ in/ft} \text{ (specified unit length)} \]
\[ f_{yp} = 30.0 \text{ ksi} \text{ (manufacturer specified)} \]
\[ \theta = 20.0 \text{ deg} \text{ (manufacturer specified)} \]

\[ t_{pipe} \geq 0.0949 \text{ in} \]

Use a 12 gage corrugated steel pipe, 48" nominal inside diameter. \[ t_{pipe} = 0.105 \text{ in} \]

As a check, compare minimum \( t_{pipe} \) from Eq. 8.15.3.2.2-1 to simplified equation in the commentary:

\[ t_{pipe} \geq 0.04 \frac{D'_{cp} \sqrt{f_c}}{f_{yp} \cos \theta} = 0.1381 \text{ in} \text{ and } \geq 0.06 \text{ in} \text{ (Note: } f_c \text{ refers to cap pocket)} \text{ Eq. C8.15.3.2.2-1} \]

If the refined equation (Eq. 8.15.3.2.2-1) is not used, the maximum of these two simplified equations may be used because they provide a more conservative value. Note that the controlling thickness of 0.1381" from commentary equations is considerably larger than the 0.0949" calculated from the refined specification equation. Use of the refined equation reduces the required pipe thickness, especially for larger diameter pipes.

### Nonintegral Bent Cap Joint Shear Design

Depth of the cap with respect to the column diameter determines the method of joint shear design. When the following equation is satisfied, the joint shear design provisions of Article 8.14.5.2 apply. If the equation below is not satisfied, Strut-and-Tie model provisions of AASHTO LRFD Bridge Design Specifications apply.

\[ D_c \leq d \leq 1.25D_c \]

\[ D_c = 48.0 \text{ in} \]
\[ d = 60.0 \text{ in} \]

48.0 in \( \leq \) 60.0 in \( \leq \) 60.0 in OK Provisions of 8.14.5.2 apply

### Vertical Stirrups Inside the Joint Region

\[ A_s^{jvi} \geq 0.10 A_{st} \text{ Eq. 8.14.5.2.3a-1} \]

\[ A_{st} = 25.40 \text{ in}^2 \]

\[ A_s^{jvi} \geq 2.54 \text{ in}^2 \]

Use #6 double U stirrups, Tot 3 patterns placed evenly through joint.

\[ A_s^{jvi} = 3 \text{ patterns} \times 2 \text{ legs/ pattern} \times 0.44 \text{ in}^2 / \text{ leg} = 2.64 \text{ in}^2 \geq 2.54 \text{ in}^2 \text{ OK} \]

Note the minimum number of stirrups is two per Article 8.15.5.2.3a with a bar size no smaller than that used for bent cap stirrups.
Bedding Layer Reinforcement

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below and resulting center to center spacing of transverse reinforcement is 4" throughout, in accordance with Article 8.8.14.

Supplementary Hoops

A supplementary hoop may be optionally placed one inch from each end of the corrugated steel pipe. The area of this bar is to be no less than that provided in the column plastic region (use #6 supplementary hoop)

Figures Showing Final Design

- Elevation View at Column Connection
- Plan View at Column Connection
#10, Tot 12 (6 bundles)

48" ID - 12 gage corrugated steel pipe

#10, Tot 12 (6 bundles)

Bedding layer

#10 Column Reinf

#10, Tot 12 (6 bundles)

#6 hoop

#6 hoops @ 4"

5'-3"

5'-0"

4' overall spacing maintained

Optional Supplementary hoop

For additional details, see Figures 8.14.5.2.3-1, 8.14.5.2.3-2, and 8.14.5.2.3-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Grouted Duct Joint Design Example For SDCs C and D

Grouted duct connection between a column and a precast bent cap in SDC’s C and D is designed to produce performance similar to a cast-in-place connection in SDC’s C and D. This SDCs C and D design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns for which the principal tension stress in the connection exceeds 0.11√f’c. In this case, the connection requires additional joint reinforcement. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Notes: 1. The SDC B design example illustrates the procedure to be followed when the 0.11√f’c limit is not exceeded. 2. Knee joints such as that represented in C8.15.1 are not addressed in the current SGS, and special design provisions should be identified by the designer.

Geometry and Design Parameters

- f’c = 4.0 ksi (specified compressive strength of bent cap concrete)
- f’ce = 1.3f’c = 5.2 ksi (expected f’c of bent cap)
- f’cg = 7.5 ksi (specified compressive strength of grout)
  
  Check: f’cg (ksi) ≥ max[1.25(f’ce+0.5)=7.1, 6.0]
  OK per BCS 8.13.8.3.2a
- f’c = 60 ksi  fye = 68 ksi (column bars)
- Bent cap width = 7.0 ft (Bcap)
- Bent cap height = 6.25 ft (Dc)
- Stirrup bar size: #6

Requirements for Extreme I load case in cap:

- Top cap rein, As topp = 16.77 in²
- Bot cap rein, As bott = 11.40 in²
- Shear rein, Av = 4.12 in²/ft

Column diameter = 5.0 ft (Dc)
Ast = 31.2 in² (#11, Tot 20)

Hoop bar size: #6
Hoop spacing = 4.0 in
f’th = 60 ksi (yield stress of hoops)

Joint Performance

Moment resisting connections are designed to transmit the maximum forces produced when the column reaches its overstrength moment capacity, Mpo.

Note: Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.

Determine the idealized plastic moment capacity of the column, Mpo, using section analysis program such as xSECTION, and calculate the overstrength moment capacity, Mpo, per Article 8.5.

Axial load acting on column per extreme event load case = 820.0 kips (Pc)
f’ce (expected concrete compressive strength) = 1.3 x f’c = 5.2 ksi
f’ye (expected steel yield stress) = 68 ksi (column bars)
Joint Proportioning

Principal Stresses

Principal stresses in the joint are limited by the following equations:

\[ p_c \leq 0.25 f'_c = 1.00 \text{ ksi maximum} \]
\[ p_t \leq 0.38 \sqrt{f'_c} = 0.76 \text{ ksi maximum} \]

Notes:

1) Length of column longitudinal rebar extended into cap. See calculations below.
2) No prestressing in section.
3) Determined from sectional analysis. Tension in column longitudinal rebar may also be derived from sectional analysis.

\[ p_t = \left( \frac{f_h + f_v}{2} \right) - \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{jv}^2} = \begin{cases} 0.0 + 0.072 \frac{2}{2} - \sqrt{\left( 0.0 - 0.072 \right)^2 + 0.313^2} = 0.278 \text{ ksi} & \text{Eq. 8.15.2.1-3} \\ \end{cases} \]
\[ p_c = \left( \frac{f_h + f_v}{2} \right) + \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{jv}^2} = \begin{cases} 0.0 + 0.072 \frac{2}{2} + \sqrt{\left( 0.0 - 0.072 \right)^2 + 0.313^2} = 0.351 \text{ ksi} & \text{Eq. 8.15.2.1-4} \\ \end{cases} \]
\[ v_{jv} = \frac{T_c}{A_{jv}} = 1890 \text{ k} / 6048.0 \text{ in}^2 = 0.313 \text{ ksi} & \text{Eq. 8.15.2.1-5} \]
\[ A_{jv} = l_{ac} B_{cap} = 72.0 \text{ in} \times 84.0 \text{ in} = 6048.0 \text{ in}^2 & \text{Eq. 8.15.2.1-6} \]
\[ f_v = \frac{P_c}{A_{jv}} = 820 \text{ k} / 11340 \text{ in}^2 = 0.0723 \text{ ksi} & \text{Eq. 8.15.2.1-7} \]
\[
A_{jh} = (D_c + D_s) \cdot B_{cap} = (60.0 \text{ in} + 75.0 \text{ in}) \times 84.0 \text{ in} = 11340 \text{ in}^2 \quad \text{Eq. 8.15.2.1-8}
\]
\[
f_s = \frac{P_b}{B_{cap}D_s} = \frac{0.0k}{(84.0 \text{ in} \times 75.0 \text{ in})} = 0.0 \text{ ksi} \quad \text{Eq. 8.15.2.1-9}
\]
\[
T_c = \frac{M_{po}}{h} = \frac{7164 \text{ kip-ft}}{3.79 \text{ ft}} = 1890 \text{ kips} \quad \text{Eq. 8.15.2.1-10}
\]
\[
p_c = 0.351 \text{ ksi} \leq 1.00 \text{ ksi maximum OK} \quad \text{Joint proportions are acceptable based on principal stress requirements.}
\]
\[
p_t = 0.278 \text{ ksi} \leq 0.76 \text{ ksi maximum OK}
\]

**Minimum Development Length of Column Longitudinal Reinforcement**

\[
l_{ac} \geq \frac{2 \cdot d_{sl} \cdot f_{ye}}{f'_{cg}} \quad \text{Eq. 8.15.2.2.1}
\]
\[
d_{sl} = 1.41 \text{ in} \quad (\#11 \text{ rebar})
\]
\[
f_{ye} = 68 \text{ ksi}
\]
\[
f'_{cg} = 7.0 \text{ ksi} \quad \text{Min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2}
\]
\[
l_{ac} = 27.4 \text{ in} \quad (\text{minimum})
\]

Extend column reinforcement as far as practically possible; assume 3\" clear from opposite face, per Article 8.15.2.2.2.

\[
l_{ac} = D_s - 3" = 6.25 \text{ ft} \times 12"/\text{ft} - 3" = 72.0 \text{ in}
\]

72.0 in \(\geq\) 27.4 in (minimum) OK Extend to top face of cap less 3 in cover.

**Minimum Joint Shear Reinforcing for Precast Bent Cap Connections**

Where principal tension in the joint is greater than or equal to 0.11\(\sqrt{f'_{cg}}\) and joint proportioning is acceptable per Article 8.15.2, additional transverse joint reinforcement for cap pocket connections per Article 8.15.5.2.3 is to be added, and Eqs. 8.15.3.1-1 and 8.15.3.1-2 are to be satisfied.

Check if principal tension, \(p_c\), is \(\geq 0.11\sqrt{f'_{cg}}\) (condition for likely joint cracking)

\[
\text{Calculated tension} = 0.278 \text{ ksi}
\]
\[
\text{Limit,} 0.11\sqrt{f'_{cg}} = 0.220 \text{ ksi}
\]
\[
p_t = 0.278 \text{ ksi} \geq 0.220 \text{ ksi limit}
\]

Additional joint reinforcement (\(A_{s}^{jho}, A_{s}^{jvi}, A_{s}^{jl}\), and horizontal \(J\) bars) is required.

Calculate required volumetric ratio of transverse joint reinforcement:

\[
\rho_s \geq 0.11 \sqrt{f'_{cg}} / f_{yh} \quad \text{Eq. 8.15.3.1-1}
\]
\[
\rho_s \geq 0.40 \frac{A_{st}}{l_{ac}^2} \quad \text{Eq. 8.15.3.1-2}
\]
\[
A_{st} = 31.2 \text{ in}^2
\]
\[
l_{ac} = 72.0 \text{ in}
\]
Maximum of: \(0.11 \sqrt{f'_c / f_{ph}} = 0.0037\) governs Eq. 8.15.3.1-1

\(0.40 A_{st} / l^2_{ac} = 0.0024\) Eq. 8.15.3.1-2

Use \(\rho_s \geq 0.0037\)

Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

\[
\rho_s = \frac{4A_{sp}}{D'_{gd} s}
\]

\(A_{sp} = 0.44\text{ in}^2\) (confined diameter of column between centroids of #6 hoop)

\(D'_{gd} = 57.92\text{ in}\) (trial spacing of transverse reinforcement hoops; spacing not to exceed 12” or 0.3Ds per Art. 8.15.3.2.1)

\(s = 8.0\text{ in}\)

\(D'_{gd} = \text{Column diameter - clear cover x 2 - hoop diameter - long column reinf diameter + grouted duct inside diameter + corrugation amplitude x 2.}\)

\(D'_{gd} = 60.0" - 2" \times 2 - 0.88" - 1.44" + 4" + 0.12" \times 2 = 57.92\text{ in}\) (deformed diameters are used for clearance calculations)

\(\rho_s = 0.0038\)

\(\rho_s = 0.0038 \geq 0.0037\) minimum OK 8” spacing is adequate (Note: Common practice is to carry hoop spacing from column into cap, or s=4” in this case.)

Actual even spacing is 7.89 inches, total 9 hoops.

**Nonintegral Bent Cap Joint Shear Design**

The depth of the cap with respect to the column diameter determines the method of joint shear design. When the following equation is satisfied, the joint shear design provisions of Article 8.15.5.2 apply. If the equation below is not satisfied, Strut-and-Tie model provisions of AASHTO LRFD Bridge Design Specifications apply.

\[D_c \leq d \leq 1.25D_c\]

\[
\begin{align*}
D_c &= 60.0\text{ in} \\
d &= 75.0\text{ in}
\end{align*}
\]

\(60.0\text{ in} \leq 75.0\text{ in} \leq 75.0\text{ in}\) OK Provisions of 8.15.5.2 apply

**Additional Joint Shear Reinforcement**

Grouted duct connections follow essentially the same joint reinforcing requirements as for cast-in-place connections specified in Art. 8.15.5.1, when additional reinforcement is required per Art. 8.15.3.2.1
Vertical Stirrups Outside the Joint Region:

\[ A_s^{jvo} \geq 0.175 \ A_{st} \]

\[ A_{st} = 31.20 \ \text{in}^2 \]

\[ A_s^{jvo} \geq 5.46 \ \text{in}^2 \]

\( A_s^{jvo} \) is placed transversely within a distance \( D_c \) extending from each face of the column. This is in addition to the \( A_s \) of 4.12 in\(^2\) provided for Extreme I load case analysis per Article 8.15.5.1.1.

\[ A_s^{jvo} \geq 5.46 \ \text{in}^2 \ / \ D_c = 1.092 \ \text{in}^2 / \text{ft} \]

\[ A_v^{\text{total}} = A_v + A_s^{jvo} \]

\[ A_v^{\text{total}} = 4.12 \ \text{in}^2 / \text{ft} + 1.09 \ \text{in}^2 / \text{ft} = 5.21 \ \text{in}^2 / \text{ft} \]

Find the spacing of the #6 stirrups within the distance \( D_c \) on both sides of the column with the assumption of #6 stirrups having four vertical legs in each pattern.

Area of one stirrup pattern = 4 legs \( \times 0.44 \ \text{in}^2 / \text{leg} = 1.76 \ \text{in}^2 \)

Number of stirrup patterns required per foot = \( A_v^{\text{total}} / 1.76 \ \text{in}^2 = 2.96 \) stirrups / ft

Use 3 stirrups per foot, 4" spacing. \( A_v^{\text{total}} = 5.28 \ \text{in}^2 / \text{ft} \geq 5.21 \ \text{in}^2 / \text{ft} \) OK

Vertical Stirrups Inside the Joint Region:

\[ A_s^{jvi} \geq 0.135 \ A_{st} \]

\[ A_{st} = 31.2 \ \text{in}^2 \]

\[ A_s^{jvi} \geq 4.21 \ \text{in}^2 \]

Use #6 double U stirrups, Tot 4 patterns placed evenly through joint. Note that spacing may vary slightly to avoid conflict with grouted ducts. The minimum number of stirrups is two, per Article 8.15.5.2.2, with a bar size no smaller than that used for bent cap stirrups.

\[ A_s^{jvi} = 4 \ \text{patterns} \times 4 \ \text{legs} / \text{pattern} \times 0.44 \ \text{in}^2 / \text{leg} = 7.04 \ \text{in}^2 \geq 4.21 \ \text{in}^2 \] OK

4 patterns instead of 3 are conservatively used for symmetry.

Additional Longitudinal Cap Beam Reinforcement

\[ A_s^{l} \geq 0.245 \ A_{st} \]

\[ A_{st} = 31.2 \ \text{in}^2 \]

\[ A_s^{l} \geq 7.64 \ \text{in}^2 \] (individual amount applied to top and bottom faces of cap)
Top cap reinf, \( A_s^{\text{cap}}_{\text{top}} = \frac{16.77}{\text{in}^2} \)  
Bot cap reinf, \( A_s^{\text{cap}}_{\text{bot}} = \frac{11.40}{\text{in}^2} \)  

Per design requirements of Extreme L load case.

Total top cap reinf, \( A_s^{\text{total}}_{\text{top}} = A_s^{\text{cap}}_{\text{top}} + A_s^{\text{jl}} = 16.77 \text{ in}^2 + 7.64 \text{ in}^2 = 24.41 \text{ in}^2 \)  
Total bot cap reinf, \( A_s^{\text{total}}_{\text{bot}} = A_s^{\text{cap}}_{\text{bot}} + A_s^{\text{jl}} = 11.40 \text{ in}^2 + 7.64 \text{ in}^2 = 19.04 \text{ in}^2 \)

Use #11 Tot 16 on top and bottom of bent cap, \( A_s = 24.96 \text{ in}^2 \)

Note: \( A_s^{\text{jl}} \) is added to the \( A_s^{\text{cap}} \) of the bent cap required under the seismic extreme event load case only. These \( A_s^{\text{total}}_{\text{top}} \) and \( A_s^{\text{total}}_{\text{bot}} \) values are to be compared to the respective requirements of the applicable Strength load cases, and the larger value governs. Thus, the value shown above may not necessarily govern required bent cap flexural reinforcement.

**Horizontal J-Bars**

Provide horizontal J-bars hooked around every other vertical-to-longitudinal side face bar intersection within the joint as shown in the figure below. Bar size to be #4 minimum. #6 bar is used.

**Bedding Layer Reinforcement**

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3” thick. Cover on top column hoop to be 2” as shown in figures below, and resulting center-to-center spacing of transverse reinforcement is 4” throughout, in accordance with

**Figures Showing Final Design**

\( D_c \) is the distance over which \( A_s^{\text{jvo}} \) is spread in addition to stirrups required in the same region for other forces. \( D_c = \text{column dia.} \)
Typical Section through Joint

7'-0"

6'-3"

5'-0"

4" overall spacing maintained

#11, Tot 16 (8 bundles)

#6 J-bar
Tot 2

#6 J-bar
Tot 4

#11, Tot 16 (8 bundles)

Bedding layer

#11 Column Reinf

See detail below

#6 stirrups,
Tot 4

#6 hoops, Tot
9, even spacing

#6 hoops
@ 4"

#6, Tot 4

5'‐0"

2.5" per Art. 8.8.14

1.5" per Art. 8.15.5.2.1

Clear cover may be less than 2" per Art. 8.8.14

Simplified Section at Column Edge

Bent Cap

Bedding

Layer

Column

SACRAMENTO STATE UNIV.

E. MATSUMOTO

DATE 11/9/2009

M. STILLER

DATE 11/6/2009

CLIENT SACRAMENTO STATE UNIV.

PROJECT NO NCHRP 12-74

DESIGN EXAMPLES

AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL GROUTED DUCT BENT CAP JOINT DESIGN EXAMPLE - SDCs C and D
For additional details, see Figures 8.15.5.2.2-1, 8.15.5.2.2-2, and 8.15.5.2.2-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Cap Pocket Joint Design Example For SDCs C and D

Cap pocket connection between a column and a precast bent cap in SDCs C and D reinforces the bent cap joint by means of a helical lock-seam, corrugated steel pipe, eliminating conventional hoops and J-bars. This SDC C/D design example applies to a T-joint of a rectangular, nonintegral precast bent cap supported by circular columns for which the principal tensile stress in the connection exceeds $0.11\sqrt{f'}$. In this case, the connection requires additional joint reinforcement. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Notes: 1. The SDC B design example illustrates the procedure to be followed when the $0.11\sqrt{f'}$ limit is not exceeded. 2. Knee joints such as that represented in C8.15.1 are not addressed in the current SGS, and special design provisions should be identified by the designer.

Geometry and Design Parameters

- $f'_c = 4.0$ ksi (specified compressive strength of bent cap)
- $f'_{c,pocket}$ (specified compressive strength of pocket fill)
- $f'_{c,pocket} = f'_{ce} + 0.5$ ksi = 5.7 ksi (BCS 8.13.8.3.3a)
- $f_y$ (yield stress of equivalent hoop)
- $f_y = 60$ ksi
- $f_ye$ (expected yield stress of column bars)
- $f_ye = 68$ ksi
- $f_{yp}$ (nominal yield stress of steel pipe)
- Bent cap width = 7.0 ft ($B_{cap}$)
- Bent cap height = 6.25 ft ($D_s$)
- Stirrup bar size: #6

Requirements for Extreme I load case in cap:
- Top cap reinf, $A_{s,\text{top}} = 16.77$ in$^2$
- Bot cap reinf, $A_{s,\text{bot}} = 11.40$ in$^2$
- Shear reinf, $A_v = 4.12$ in$^2$/ft Bedding Layer

Column diameter = 5.0 ft ($D_c$)

- $A_{st} = 31.2$ in$^2$ (#11 Tot 20)
- Hoop bar size: #6
- Hoop spacing = 4.0 in

Joint Performance

Moment resisting connections are designed to transmit the maximum forces produced when the column reaches its overstrength moment capacity, $M_{por}$

Note: Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.
Determine the idealized plastic moment capacity of the column, \( M_p \), using section analysis program such as xSECTION, and calculate the overstrength moment capacity, \( M_{po} \), per Article 8.5.

Axial load acting on column per extreme event load case = 820.0 kips \( (P_c) \)

\( f'_{ce} \) (expected concrete compressive strength) = 1.3 \( f' \) 5.2 ksi

\( f_{ye} \) (expected steel yield stress) = 68 ksi

\[
M_p = 5970 \text{ kip-ft} \\
M_{po} = \lambda_{mo} M_p \\
\lambda_{mo} = 1.2 \quad \text{(ASTM A706)} \\
M_{po} = 7164 \text{ kip-ft}
\]

**Joint Proportioning**

**Principal Stresses**

Principal stresses in the joint are limited by the following equations:

- Principal compression, \( p_c \): \( p_c \leq 0.25 f' c = 1.00 \text{ ksi maximum} \)
- Principal tension, \( p_t \): \( p_t \leq 0.38 \sqrt{f' c} = 0.76 \text{ ksi maximum} \)

\[
\begin{align*}
p_c &= \left( \frac{f_h + f_v}{2} \right) - \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{jv}^2} \\
p_t &= \left( \frac{f_h + f_v}{2} \right) + \sqrt{\left( \frac{f_h - f_v}{2} \right)^2 + v_{jv}^2}
\end{align*}
\]

**Notes:**

1) Length of column longitudinal rebar extended into cap. See calculations below.
2) No prestressing in section.
3) Determined from sectional analysis. Tension in column longitudinal rebar may also be derived from sectional analysis.
\[ v_{ve} = \frac{T_c}{A_{jv}} = \frac{1890 \text{ kips} \cdot \text{ft}}{6048.0 \text{ in}^2} = 0.313 \text{ ksi} \]  
\[ A_{jv} = l_{ac}B_{cap} = 72.0 \text{ in} \times 84.0 \text{ in} = 6048.0 \text{ in}^2 \]  
\[ f_v = \frac{P_c}{A_{jh}} = \frac{820 \text{ kips}}{11340 \text{ in}^2} = 0.0723 \text{ ksi} \]  
\[ A_{jh} = (D_c + D_s)B_{cap} = (60.0 \text{ in} + 75.0 \text{ in}) \times 84.0 \text{ in} = 11340 \text{ in}^2 \]  
\[ f_h = \frac{P_b}{B_{cap}D_s} = \frac{0.0 \text{ kips}}{(84.0 \text{ in} \times 75.0 \text{ in})} = 0.0 \text{ ksi} \]  
\[ T_c = \frac{M_{po}}{h} = \frac{7164 \text{ kips} \cdot \text{ft}}{3.79 \text{ ft}} = 1890 \text{ kips} \] 

\[ p_c = 0.351 \text{ ksi} \leq 1.00 \text{ ksi maximum} \quad \text{OK} \]  
\[ p_t = 0.278 \text{ ksi} \leq 0.76 \text{ ksi maximum} \quad \text{OK} \]  

\textbf{Minimum Development Length of Column Longitudinal Reinforcement}  

\[ l_{ac} \geq \frac{2.3d_{ml}f_{ye}}{f'_{c,\text{pocket}}} \]  
\[ d_{ml} = 1.41 \text{ in} \quad \#11 \text{ rebar} \]  
\[ f_{ye} = 68 \text{ ksi} \]  
\[ f'_{c,\text{pocket}} = 5.7 \text{ ksi} \]  

\[ l_{ac} = 38.7 \text{ in} \quad \text{(minimum)} \]  

Extend column reinforcement as far as practically possible, assume 3" clear from opposite face, per Article 8.15.2.2.2.  

\[ l_{ac} = D_s - 3\text{"} = 6.25 \text{ ft} \times 12\text{"}/\text{ft} - 3\text{"} = 72.0 \text{ in} \]  

72.0 in \[ \geq 38.7 \text{ in} \quad \text{(minimum)} \quad \text{OK} \] Extend to top face of cap less 3 in cover.  

\textbf{Minimum Joint Shear Reinforcing for Precast Bent Cap Connections}  

Where principal tension in the joint is greater than or equal to \(0.11\sqrt{f'_c}\), and joint proportioning is acceptable per Article 8.15.2, additional transverse joint reinforcement for cap pocket connections per Article 8.15.5.2.3 is to be added, and Eqs. 8.15.3.1-1 and 8.15.3.1-2 are to be satisfied.  

Check if principal tension, \(p_v\), is \(\geq 0.11\sqrt{f'_c}\) (condition for likely joint cracking)  

\[ \text{Calculated tension} = 0.278 \text{ ksi} \]  
\[ \text{Limit, } 0.11\sqrt{f'_c} = 0.220 \text{ ksi} \]  
\[ p_v = 0.278 \text{ ksi} \leq 0.220 \text{ ksi limit} \]  

Additional joint reinforcement \((A_{jvo}, A_{jvi}, \text{and } A_{jl})\) is required.
Calculate required volumetric ratio of transverse joint reinforcement:

Maximum of:
\[ \rho_s \geq 0.11 \sqrt{f'_c / f_{yh}} \]  \hspace{1cm} Eq. 8.15.3.1-1
\[ \rho_s \geq 0.40 \frac{A_{st}}{l_{ac}^2} \]  \hspace{1cm} Eq. 8.15.3.1-2

\[ A_{st} = 31.20 \text{ in}^2 \]
\[ l_{ac} = 72.0 \text{ in} \]

Maximum of:
\[ 0.11 \sqrt{f'_c / f_{yh}} = 0.0037 \text{ governs} \]  \hspace{1cm} Eq. 8.15.3.1-1
\[ 0.40 \frac{A_{st}}{l_{ac}^2} = 0.0024 \]  \hspace{1cm} Eq. 8.15.3.1-2

Use \( \rho_s \geq 0.0037 \)

Use the minimum volumetric ratio to calculate the required number of equivalent #6 hoops per unit length of corrugated steel pipe. Use a unit length of 1 foot. Use a 54” ID corrugated pipe based on the outer diameter of column reinforcement pattern.

\[ \rho_s = \frac{4A_{sp}}{D'_{cp} s} \]
\[ s = \frac{4A_{sp}}{D'_{cp} \rho_s} \]

\[ A_{sp} = 0.44 \text{ in}^2 \] (assume #6 hoop to match column transverse reinforcement)
\[ D'_{cp} = 54.75 \text{ in} \] (average confined dia. of column between corrugated steel pipe walls)
\[ \rho_s = 0.0037 \] (minimum volumetric ratio)

\[ D'_{cp} = \text{Nominal inside diameter of corrugated pipe + average wall corrugation width.} \]
\[ s = 8.8 \text{ in max spacing} \]
Therefore, the number of equivalent hoops per foot is \( 12'' / s = 1.369 \) hoops/ft

Calculate the nominal confining hoop force of the equivalent hoops.

\[ F_H = n_h A_{sp} f_{yh} \]  \hspace{1cm} Eq. 8.15.3.2.2-2

\[ n_h = 1.369 \text{ ea (number of equivalent hoops per unit length)} \]
\[ A_{sp} = 0.44 \text{ in}^2 \text{ (area of #6 equivalent hoop)} \]
\[ f_{yh} = 60.0 \text{ ksi (yield stress of equivalent hoop)} \]
\[ F_H = 36.14 \text{ kips/ft} \]

Calculate the thickness of the corrugated steel pipe that provides the same nominal confinement as standard hoop reinforcement.

\[ t_{pipe} \geq \max \left[ \frac{F_H}{H_p f_{yp} \cos \theta} \right] \]
\[ 0.060 \text{ in} \]  \hspace{1cm} Eq. 8.15.3.2.2-1
F_H = 36.1 kips/ft
H_p = 12.0 in/ft (specified unit length)
f_{yp} = 30.0 ksi (manufacturer specified)
θ = 20.0 deg (manufacturer specified)

\( t_{pipe} \geq 0.1068 \text{ in} \)

Use a 12 gage corrugated steel pipe, 54" nominal inside diameter.  \( t_{pipe} = 0.1046 \text{ in} \) (2% under, Say OK)

As a check, compare minimum \( t_{pipe} \) from Eq. 8.15.3.2.2-1 to simplified equations in the SGS commentary:

\[
\begin{align*}
t_{pipe} \geq 0.04 \frac{D'c_p \sqrt{f_c}}{f_{yp} \cos \theta} & = 0.1554 \text{ in} \quad \text{and} \quad \geq 0.06 \text{ in} \\
t_{pipe} \geq 0.14 \frac{A_{st} D'c_p f_{yh}}{I_{x}^2 f_{yp} \cos \theta} & = 0.0982 \text{ in} \quad \text{and} \quad \geq 0.06 \text{ in}
\end{align*}
\]

(Note: \( f'_c \) refers to bent cap concrete) Eq. C8.15.3.2-1

Eq. C8.15.3.2-2

If the refined equation (Eq. 8.15.3.2.2-1) is not used, the maximum of these two simplified equations may be used because they provide a more conservative value. Note that the controlling thickness of 0.1554" from commentary equations is considerably larger than the calculated 0.1068" from the more accurate specification equation. Use of the refined equation reduces the required pipe thickness, especially for larger diameter pipes.

**Nonintegral Bent Cap Joint Shear Design**

Depth of the cap with respect to the column diameter determines the method of joint shear design. When the following equation is satisfied, the joint shear design provisions of Article 8.15.5.2 apply. If the equation below is not satisfied, Strut-and-Tie model provisions of AASHTO LRFD Bridge Design Specifications apply.

\[
D_c \leq d \leq 1.25D_c
\]

\[
D_c = 60.0 \text{ in} \\
d = 75.0 \text{ in}
\]

60.0 in \( \leq \) 75.0 in \( \leq \) 75.0 in OK Provisions of 8.15.5.2 apply

**Additional Joint Shear Reinforcement**

**Vertical Stirrups Outside the Joint Region:**

\[
A_{st}^{ivo} \geq 0.175 A_{st}
\]

\[
A_{st} = 31.20 \text{ in}^2
\]

\[
A_{st}^{ivo} \geq 5.46 \text{ in}^2
\]
\( A_{sv} \) is placed transversely within a distance \( D_c \) extending from each face of the column. This is in addition to the \( A_v \) of 4.12 in\(^2\) provided for Extreme I load case analysis per Article 8.15.5.1.1.

\[
A_{sv} \geq 5.46 \text{ in}^2 / D_c = 1.092 \text{ in}^2 / \text{ft}
\]

\[
A_v^{\text{total}} = A_v + A_{sv}
\]

\[
A_v^{\text{total}} = 4.12 \text{ in}^2 / \text{ft} + 1.09 \text{ in}^2 / \text{ft} = 5.21 \text{ in}^2 / \text{ft}
\]

Find the spacing of the #6 stirrups within the distance \( D_c \) on both sides of the column with the assumption of #6 stirrups having four vertical legs in each pattern.

Area of one stirrup pattern = 4 legs x 0.44 in\(^2\) / leg = 1.76 in\(^2\)

Number of stirrup patterns required per foot = \( A_v^{\text{total}} / 1.76 \text{ in}^2 = 2.96 \text{ stirrups / ft} \)

Use 3 stirrups per foot, 4" spacing. \( A_{sv}^{\text{total}} = 5.28 \text{ in}^2 / \text{ft} \geq 5.21 \text{ in}^2 / \text{ft} \) OK

**Vertical Stirrups Inside the Joint Region:**

\[
A_{sv} \geq 0.12 A_{st}
\]

\[
A_{st} = 31.2 \text{ in}^2
\]

\[
A_{sv} \geq 3.74 \text{ in}^2
\]

Use #6 single U stirrups, Tot 5 patterns placed evenly through joint.

\[
A_{sv} = 5 \text{ patterns x 2 legs / pattern x 0.44 in}^2 / \text{leg} = 4.40 \text{ in}^2 \geq 3.74 \text{ in}^2 \text{ OK}
\]

There must be a minimum of 2 stirrups per Article 8.15.5.2.3c with a bar size no smaller than that used for bent cap stirrups.

**Additional Longitudinal Cap Beam Reinforcement**

\[
A_{s}^{\|} \geq 0.245 A_{st}
\]

\[
A_{st} = 31.2 \text{ in}^2
\]

\[
A_{s}^{\|} \geq 7.64 \text{ in}^2 \text{ (individual amount applied to top and bottom faces of cap)}
\]

Top cap reinf, \( A_{s}^{\text{cap top}} = 16.77 \text{ in}^2 \)
Bot cap reinf, \( A_{s}^{\text{cap bot}} = 11.40 \text{ in}^2 \)

\[
\left\{ \begin{array}{l}
\text{Per design requirements of Extreme I load case.}
\end{array} \right. \]

Total top cap reinf, \( A_{s}^{\text{total top}} = A_{s}^{\text{cap top}} + A_{s}^{\|} = 16.77 \text{ in}^2 + 7.64 \text{ in}^2 = 24.41 \text{ in}^2 \)

Total bot cap reinf, \( A_{s}^{\text{total bot}} = A_{s}^{\text{cap bot}} + A_{s}^{\|} = 11.40 \text{ in}^2 + 7.64 \text{ in}^2 = 19.04 \text{ in}^2 \)

Use #11 Tot 16 on top and bottom of bent cap, \( A_s = 24.96 \text{ in}^2 \)
Note: $A_s^{\text{cap}}$ is added to the $A_s^{\text{cap}}$ of the bent cap required under the seismic extreme event load case only. These $A_s^{\text{total, top}}$ and $A_s^{\text{total, bot}}$ values are to be compared to the respective requirements of the applicable Strength load cases, and the larger value governs. Thus, the value shown above may not necessarily govern required bent cap flexural reinforcement.

Note: Horizontal J-bars are not required in the joint region, as confirmed by research.

**Bedding Layer Reinforcement**

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below, and resulting center-to-center spacing of transverse reinforcement is 4" throughout, in accordance with Article 8.8.14.

**Supplementary Hoops**

Provide a supplementary #6 hoop placed one inch from each end of the corrugated steel pipe. The area of this bar is to be no less than that provided in the column plastic region (#6 bar).

**Figures Showing Final Design**

Dc is the distance over which $A_s^{jvo}$ is spread in addition to stirrups required in the same region for other forces. $D_c$ = column dia.

Elevation View at Column Connection
Typical Section through Joint

54" ID - 12 gage corrugated steel pipe

#11, Tot 16 (8 bundles) Bedding layer
#11 Column Reinf

#11, Tot 16 (8 bundles) #6 stirrups, Tot 5 6'-3"

#6 hoops @ 4"

#6 hoop 5'-0"

See detail below

4" overall spacing maintained

Supplementary hoop

Bent Cap

Bedding Layer

Column

1.5" per Art. 8.15.5.2.1

2.5" per Art. 8.8.14

Clear cover may be less than 2" per Art. 8.8.14.

Simplified section at column edge
For additional details, see Figures 8.15.5.2.3-1, 8.15.5.2.3-2, and 8.15.5.2.3-3 in the AASHTO Guide Specifications for LRFD Seismic Bridge Design.
Hybrid Bent System Design Example For SDCs C and D

This design example is developed to show a practitioner a methodology to design a multi-column hybrid precast bridge column system. Complete development of simplified lateral force-displacement predictions are presented. This includes the estimation of the effective yield point and the ultimate point for lateral response. The procedure presented requires iteration of the column neutral axis depth at both limit states that can be easily accommodated with modern computer software.

A lateral demand analysis is conducted for a bridge located in a region of high seismic demand. The developed lateral properties are used to determine the effective initial structural period for the development of lateral seismic demands. The short-period displacement modification factor from AASHTO are used to account for potential amplification of displacement demands at low periods.

The calculated flexural capacity is used to investigate the required seismic design required for the precast bent cap. Demands on the bent cap joint are investigated to determine the required detailing within the joint region.

Material Input Parameters

**Concrete Materials (Unconfined)**

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<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c= $</td>
<td>4.0 ksi</td>
<td>Specified 28-day compressive strength of concrete</td>
</tr>
<tr>
<td>$f'_c = 1.3 f'_c = $</td>
<td>5.2 ksi</td>
<td>Expected concrete compressive strength</td>
</tr>
<tr>
<td>$E_c= $</td>
<td>4372 ksi</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$G_c= $</td>
<td>1749 ksi</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>$\beta_1= $</td>
<td>0.79</td>
<td>Equivalent stress block factor</td>
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</tbody>
</table>

**Grout Materials**

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<tr>
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<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_{cg}= $</td>
<td>8.0 ksi</td>
<td>Specified compressive strength of bedding layer</td>
</tr>
<tr>
<td>Check: $f'_{cg} (ksi) \geq \max[1.25(f'_c+0.5), 6.0]$</td>
<td>Limit = 7.1 ksi</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

Joint grout shall have a 3 lb per cy fraction of polypropylene fibers for joint integrity

**Mild Reinforcing Steel**

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<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
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<td>Specified yield stress</td>
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<tr>
<td>$f_{ye}= $</td>
<td>68 ksi</td>
<td>Expected yield stress</td>
</tr>
<tr>
<td>$f_{ue}= $</td>
<td>95 ksi</td>
<td>Expected ultimate tensile strength</td>
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<tr>
<td>$E_s= $</td>
<td>29000 ksi</td>
<td>Modulus of elasticity</td>
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<tr>
<td>$\varepsilon_y= $</td>
<td>0.002</td>
<td>Effective yield strain</td>
</tr>
<tr>
<td>$\varepsilon_{su}= $</td>
<td>0.12</td>
<td>Ultimate tensile strain</td>
</tr>
<tr>
<td>$\varepsilon_{ru}= $</td>
<td>0.06</td>
<td>Reduced ultimate tensile strain accounting for bar bending</td>
</tr>
<tr>
<td>$f_{yh}= $</td>
<td>60 ksi</td>
<td>Specified minimum yield stress of confining hoops</td>
</tr>
</tbody>
</table>

**Post-Tensioning Steel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{ps}= $</td>
<td>28500 ksi</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$f_{pu}= $</td>
<td>270 ksi</td>
<td>Specified ultimate tensile strength</td>
</tr>
<tr>
<td>$\varepsilon_{ps,EE}= $</td>
<td>0.009</td>
<td>Essentially elastic prestress strain</td>
</tr>
</tbody>
</table>
essentially elastic prestress stress

\( f_{ps,eq} = 245 \text{ ksi} \)

Reduced ultimate prestress strain

\( \varepsilon_{ps,u} = 0.03 \)

### Geometric Input Parameters

#### Column Dimensions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_c )</td>
<td>4.0 ft</td>
</tr>
<tr>
<td>( d_{ps} )</td>
<td>24.0 in</td>
</tr>
<tr>
<td>( c_c )</td>
<td>2.0 in</td>
</tr>
<tr>
<td>( A_g )</td>
<td>12.57 ft²</td>
</tr>
<tr>
<td>( A_c )</td>
<td>10.05 ft²</td>
</tr>
<tr>
<td>( I_g )</td>
<td>12.57 ft⁴</td>
</tr>
</tbody>
</table>

Gross column diameter

Distance from edge of column to tendon centerline

Clear cover to hoop reinforcement

Gross column section area

Effective column shear area

Gross column moment of inertia

Clear height of column

Height from centerline bent cap to centerline foundation

= 1 if fixed-pinned ; = 2 if fixed-fixed

Distance to point of inflection

Total length (unbonded) of post-tensioning tendon

Total gravity force acting during seismic loading (including overturning)

Gravity load axial load ratio

#### Bent Cap Dimension

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( B_{cap} )</td>
<td>5.0 ft</td>
</tr>
<tr>
<td>( D_s )</td>
<td>4.5 ft</td>
</tr>
</tbody>
</table>

Bent cap width

Bent cap height

#### Column Transverse Reinforcement Specified

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size of bars:</td>
<td>No. 6</td>
</tr>
<tr>
<td>( d_{bar} )</td>
<td>0.75 in</td>
</tr>
<tr>
<td>( A_v )</td>
<td>0.44 in²</td>
</tr>
<tr>
<td>( s_{hoop} )</td>
<td>4 in</td>
</tr>
<tr>
<td>( D' )</td>
<td>3.60 ft</td>
</tr>
<tr>
<td>( A_{cc} )</td>
<td>10.20 ft²</td>
</tr>
<tr>
<td>( \rho_c = 4 A_v /[D's] )</td>
<td>1.02 %</td>
</tr>
</tbody>
</table>

Butt-welded hoop reinforcement

Diameter of reinforcing bar

Area of reinforcing bar

Center-to-center spacing of hoops

Diameter of confined concrete core

Area of confined concrete core

Volumetric ratio of confinement reinforcement

#### Column Mild Reinforcement Specified

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of bars:</td>
<td>16</td>
</tr>
<tr>
<td>( d_{bar} )</td>
<td>0.88 in</td>
</tr>
<tr>
<td>( A_{bar} )</td>
<td>0.60 in²</td>
</tr>
<tr>
<td>( A_t )</td>
<td>9.6 in²</td>
</tr>
<tr>
<td>( \rho_s )</td>
<td>0.53 %</td>
</tr>
<tr>
<td>( D_{bars} )</td>
<td>3.47 ft</td>
</tr>
<tr>
<td>( s_b )</td>
<td>8.17 in</td>
</tr>
</tbody>
</table>

Size of bars: No. 7

Diameter of reinforcing bar

Area of reinforcing bar

Total area of mild reinforcement

Volumetric ratio of mild reinforcement

Diameter about which bars are distributed

Clear spacing of reinforcing bars

Recommended: \( s_b < 8.00 \) inches: 8.17 < 8.00, O.K.
Column Post-Tensioning Specified

Number of tendons: 1  Number of strands per tendon: 12
Diameter of strands: 0.6 in  Area of single strand 0.217 in²
\[ A_{ps} = 2.60 \text{ in}^2 \]

\[ f_{pse} / f_{pu} = 0.60 \]  Effective post-tensioning stress ratio
\[ A_{ps} f_{pse} / f'_{ce} A_g = 4.48 \% \]

Confined Concrete Properties (based on specified reinforcement)

\[ \rho_{cc} = \frac{A_s}{A_{cc}} = 0.65 \% \]  Volumetric ratio of mild reinforcement with respect to core
\[ \rho_s = 4 A_s/[D's] = 1.02 \% \]  Volumetric ratio of confinement reinforcement  SGS Eq. 8.6.2-6
\[ k_e = [1-0.5s/D']^2/[1-\rho_{cc}] = 0.916 \]  Confinement effectiveness coefficient
\[ f'_{cse} = \frac{k_e \rho_s f_{yh}}{2} = 6.92 \text{ ksi} \]  Expected confined concrete compressive strength
\[ \varepsilon_{cc} = 0.005 \]  Compressive strain at peak stress
\[ \varepsilon_{cu} = 0.019 \]  Ultimate confined concrete compressive strain

\[ \varepsilon_{cu} / \varepsilon_{cc} = 3.55 \]  
\[ K = f'_{cce} / f'_{ce} = 1.33 \]

Confined concrete equivalent stress block depth factor
\[ \beta_c = 0.98 \]
\[ \alpha_{cc} \beta_c = 0.89 \]
\[ \alpha_{cc} = 0.91 \]  Confined concrete equivalent stress block stress factor

from Park and Paulay Figure 3.8
Reference Yield Point

\[ c = 12.78 \text{ in} \]
\[ \frac{c}{D} = 26.6 \% \]
\[ \theta_{\text{conc}} = \cos^{-1}\left(\frac{0.5D - \beta_1 c}{0.5D}\right) = 54.6 \text{ deg} = 0.9529 \text{ rad} \]
\[ \theta_{\text{steel}} = \cos^{-1}\left(\frac{0.5D - c}{0.5D}\right) = 57.4 \text{ deg} = 1.0014 \text{ rad} \]

Use radians for all equations

\[ T_{\text{ps},y} = A_{\text{ps}} f_{\text{ps}} \]
Tension force in post-tensioning

\[ T_{\text{sy}} = A_{\text{f}} f_{\text{y}} \pi / \theta_{\text{steel}} \]
Resultant tension force in mild reinforcement

\[ C_{\text{sy}} = A_{\text{f}} f_{\text{y}} \theta_{\text{steel}} / \pi \]
Resultant compression force in mild reinforcement

\[ C_{\text{cy}} = 0.85 f_{\text{ce}} D^2 \left[ \theta_{\text{conc}} - \sin(\theta_{\text{conc}}) \cos(\theta_{\text{conc}}) \right] / 4 \]
Resultant compression force in concrete

### Tension / Axial Force Summation

| \( T_{\text{ps},y} \) | 422 kip | Post-tensioning |
| \( T_{\text{sy}} \) | 445 kip | Mild reinforcement |
| \( N \) | 565 kip | Axial load |
| \( \Sigma T \) | 1431 kip | |

\[ \Sigma T = \Sigma C = 1431 \text{ kip} \]
Equilibrium satisfied

\[ y_{\text{st}} = 0.5D \text{bars} \sin(\pi - \theta_{\text{steel}}) / (\pi - \theta_{\text{steel}}) = 8.19 \text{ in} \]
Resultant steel tension eccentricity

\[ y_{\text{sc}} = 0.5D \text{bars} \sin(\pi - \theta_{\text{steel}}) / (\pi - \theta_{\text{steel}}) = 17.50 \text{ in} \]
Resultant steel compression eccentricity

\[ y = D \sin(\theta_{\text{conc}}) / (3[\theta_{\text{conc}} \sin(\theta_{\text{conc}}) \cos(\theta_{\text{conc}})]) = 18.03 \text{ in} \]
Resultant concrete compression eccentricity

\[ M_y = C_{\text{cy}} y_c + C_{\text{sy}} y_{\text{sc}} + T_{\text{sy}} y_{\text{st}} = 2445 \text{ kip-ft} \]
Reference yield moment

\[ V_y = M_y / H_{\text{inf}} = 257 \text{ kip} \]
Reference yield base shear

\[ \theta_{\text{by}} = 2 \varepsilon_{\text{ub}} / [D - D_{\text{bars}} - 2c] = 0.09 \% \]
Fixed end rotation at joint due to opening

\[ \Delta_e = V_y H_{\text{clf}}^3 / [3n_i^2 E_i A_i] = 0.22 \text{ in} \]
Deformation due to elastic bending of column

\[ \Delta_s = V_y H_{\text{clf}} / [G_i A_i] = 0.02 \text{ in} \]
Deformation due to shear deformations of column

\[ \Delta_j = \theta_{\text{by}} H_{\text{clf}} = 0.20 \text{ in} \]
Deformation due to fixed end rotation due to opening

\[ \Delta_y = 0.45 \text{ in} \]
Total Deformation

\[ \text{DR} = 0.20 \% \]
Drift ratio
Ultimate Point

\[ c = 9.80 \text{ in} \]  
Neutral axis depth (determined via numerical solver)

\[ \frac{c}{D_c} = 20.4\% \]  
\[ \frac{c}{D_c} < 25\% \]

\[ \theta_{\text{conc}} = \cos^{-1}\left\{\frac{0.5D' - \beta_{cc}c}{0.5D'}\right\} = 57.4\text{ deg} = 1.0019\text{ rad} \]

\[ \alpha_{ut} = 0.8 - 0.35 \frac{c}{D_{bars}} = 0.72 \]  
Shape factor for ultimate tension force in mild reinforcement

\[ \alpha_{uc} = 0.05 + 0.4 \frac{c}{D_{bars}} = 0.14 \]  
Shape factor for ultimate compression force in mild reinforcement

\[ \psi_{ut} = 0.1 + 0.24 \frac{c}{D_{bars}} = 0.16 \]  
Shape factor for location of resultant tension force

\[ \psi_{uc} = 0.5 - 0.16 \frac{c}{D_{bars}} = 0.46 \]  
Shape factor for location of resultant compression force

\[ T_{ps,u} = A_{ps}f_{ps}\left[\theta_{b,u}E_{ps}\left(d_{ps} - c\right)c\right]/L_{ps} \]  
Tension force in post-tensioning

\[ T_{s,u} = A_{s}\alpha_{ut} \]  
Resultant tension force in mild reinforcement

\[ C_{s,y} = A_{s}\alpha_{uc} \]  
Resultant compression force in mild reinforcement

\[ C_{c,y} = \alpha_{cc}f'_{cc}D'^2\left[\theta_{\text{conc}}\sin(\theta_{\text{conc}})\cos(\theta_{\text{conc}})\right]/4 \]  
Resultant compression force in concrete

T = 1748 kip  
\[ \Sigma C = 1742 \text{ kip} \]  
Force imbalance, adjust c

\[ y_{st} = \psi_{ut}D_{bars} = 6.51 \text{ in} \]  
Resultant steel tension eccentricity

\[ y_{sc} = \psi_{uc}D_{bars} = 19.24 \text{ in} \]  
Resultant steel compression eccentricity

\[ y_c = D'\sin(\theta_{\text{conc}})^3/(3[\theta_{\text{conc}}\sin(\theta_{\text{conc}})\cos(\theta_{\text{conc}})]) = 15.73 \text{ in} \]  
Resultant concrete compression eccentricity

\[ M_u = C_{c,y}y_c + C_{s,y}y_sc + T_{s,u}y_{st} = 2677 \text{ kip-ft} \]  
Ultimate moment strength

\[ V_u = M_u / H_{inf} = 282 \text{ kip} \]  
Ultimate shear strength

\[ \theta_{b,u} = \epsilon_{cu} = 1.88 \% \]  
Fixed end rotation at joint due to opening

\[ \Delta = V_uH_{clf}^3/[3n^2E_{clf}] = 0.24 \text{ in} \]  
Deformation due to elastic bending of column

\[ \Delta_{sc} = V_uH_{clf}/[G_cA_{clf}] = 0.03 \text{ in} \]  
Deformation due to shear deformations of column

\[ \Delta_{fr} = \theta_{b,u}H_{clf} = 4.29 \text{ in} \]  
Deformation due to fixed end rotation due to opening

\[ \Delta_u = 4.56 \text{ in} \]  
Total Deformation

\[ DR = 2.00 \% \]  
Drift ratio
Performance Requirements

\[ \frac{f_{pu}}{f_{pu}} = 0.75 \quad \text{Post-tensioning stress ratio at ultimate point} \]
\[ \frac{f_{pu}}{f_{pu}} < \frac{f_{ps,EE}}{f_{pu}} \quad 0.75 < 0.91 \quad \text{O.K.} \]

\[ L_{ub,min} = 9.98 \text{ in} \quad \text{Minimum required unbonded length of mild reinforcement} \]
\[ L_{ub} = 12.00 \text{ in} \quad \text{Specified unbonded length of mild reinforcement} \]
\[ 0.5D_c > L_{ub} > L_{ub,min} \quad 24.00 >= 12.00 >= 9.98 \quad \text{O.K.} \quad \text{SGS § 8.8.14} \]

\[ \beta_{M,y} = 0.25 \quad \text{Contribution of mild reinforcement to reference moment strength} \]
\[ 0.33 > \beta_{M,y} > 0.20 \quad 0.33 >= 0.25 >= 0.20 \quad \text{O.K.} \quad \text{SGS Eq. 8.8.1-3} \]

\[ \beta_{M,u} = 0.21 \quad \text{Contribution of mild reinforcement to ultimate moment strength} \]

\[ \beta_{PT} = 1.42 \quad \text{Effective axial load divided by tensile steel force at ultimate point} \]
\[ \beta_{PT} = \frac{0.9N + T_{ps,y}}{T_{s,u}} > 1 \quad 1.42 > 1.00 \quad \text{O.K.} \quad \text{SGS Eq. 8.8.1-2} \]

Response Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta_y )</td>
<td>0.45 in</td>
</tr>
<tr>
<td>DR( _y )</td>
<td>0.20 %</td>
</tr>
<tr>
<td>( V_y )</td>
<td>257 kip</td>
</tr>
<tr>
<td>( M_y )</td>
<td>2445 kip</td>
</tr>
<tr>
<td>( \Delta_c )</td>
<td>4.56 in</td>
</tr>
<tr>
<td>DR( _c )</td>
<td>2.00 %</td>
</tr>
<tr>
<td>( V_c )</td>
<td>282 kip</td>
</tr>
<tr>
<td>( M_c )</td>
<td>2677 kip</td>
</tr>
<tr>
<td>( k )</td>
<td>576 kip/in</td>
</tr>
<tr>
<td>( k_{pe} )</td>
<td>6 kip/in</td>
</tr>
<tr>
<td>( \alpha_{pe} )</td>
<td>1.03 %</td>
</tr>
<tr>
<td>( \mu_{A,c} )</td>
<td>10.2</td>
</tr>
<tr>
<td>( W = N )</td>
<td>565 kip</td>
</tr>
<tr>
<td>( T )</td>
<td>0.32 sec</td>
</tr>
</tbody>
</table>

Seismic Demand Analysis

\( S_0 = 1.50 \text{ g} \quad \text{0.2 second spectral acceleration (MCE)} \)
\( S_I = 0.55 \text{ g} \quad \text{1.0 second spectral acceleration (MCE)} \)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class:</td>
<td>D</td>
</tr>
<tr>
<td>( F_s )</td>
<td>1.00</td>
</tr>
<tr>
<td>( F_v )</td>
<td>1.50</td>
</tr>
<tr>
<td>( S_{DS} )</td>
<td>1.50 g</td>
</tr>
<tr>
<td>( S_{D1} )</td>
<td>0.83 g</td>
</tr>
</tbody>
</table>

SGS § 3.4.2.1

\( S_0 \) Site coefficient for 0.2 second period spectral acceleration
\( S_I \) Site coefficient for 1.0 second period spectral acceleration

SGS § 3.4.2.3

Design 0.2 second spectral acceleration
Design 1.0 second spectral acceleration
Seismic Design Category  D  SGS § 4

\[ T_s = 0.55 \text{ sec} \]  Response spectrum upper bound corner period  SGS Eq. 3.4.1-5
\[ T_o = 0.11 \text{ sec} \]  Response spectrum lower bound corner period  SGS Eq. 3.4.1-4
\[ T^* = 0.69 \text{ sec} \]  Characteristic ground motion period  SGS Eq. 4.3.3-3

\[ S_a = 1.50 \text{ g} \]  Design spectral acceleration  SGS § 3.4.1

<table>
<thead>
<tr>
<th>Trial 1</th>
<th>Trial 2</th>
<th>Trial 3</th>
<th>Trial 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta_0 = )</td>
<td>1.47 in</td>
<td>( \Delta_0 = )</td>
<td>2.91 in</td>
</tr>
<tr>
<td>( \mu_{A,0} = )</td>
<td>3.29</td>
<td>( \mu_{A,0} = )</td>
<td>5.98</td>
</tr>
<tr>
<td>( R_d = )</td>
<td>1.82</td>
<td>( R_d = )</td>
<td>1.98</td>
</tr>
</tbody>
</table>

\[ \Delta_0 < \Delta_C = 2.93 < 4.56 \]  Demand less than capacity, O.K.  SGS Eq. 4.8-1

\[ \mu_{A,D} < 6 = 6.56 > 6.00 \]  Ductility demand greater than limit, NO GOOD  SGS § 4.9

\[ M_{P,\Delta} = 138 \text{ kip-ft} \]  P-\( \Delta \) moment demand based on seismic displacement demand
\[ M_{P,\Delta} < 0.25 \times M_u = 138 < 669 \]  O.K.  SGS Eq. 4.11.5-1

Joint Performance  SGS § 8.15.1

Moment resisting connections are designed to transmit the maximum forces produced when the column reaches its overstrength moment capacity, \( M_{po} \).

Note:  Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.

\[ M_p = M_{H} / H_{clr} = 3382 \text{ kip-ft} \]  Effective ultimate moment capacity at CL bent cap
\[ M_{po} = \lambda_{mo} M_p \]  SGS § 9
\[ \lambda_{mo} = 1.2 \]  (ASTM A706)
\[ M_{po} = 4058 \text{ kip-ft} \]  Overstrength moment demand acting on bent cap

\[ A_s^{\text{cap\,top}} = 16.55 \text{ in}^2 \]  \( \{ \) Required longitudinal reinforcement required per design requirements of Extreme I load case
\[ A_s^{\text{cap\,bot}} = 12.92 \text{ in}^2 \]
Joint Proportioning

**Principal Stresses**

Principal stresses in the joint are limited by the following equations:

- **Principal compression,** $p_c$:  
  
  $$p_c = 0.25 f'_{ce} = 1.30 \text{ ksi maximum}$$  
  
  SGS Eq. 8.15.2.1-1

- **Principal tension,** $p_t$:  
  
  $$p_t = 0.38 \sqrt{f'_{ce}} = 0.87 \text{ ksi maximum}$$  
  
  SGS Eq. 8.15.2.1-2

**Note:**  

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Expression</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B_{cap}$</td>
<td>60.0 in</td>
<td></td>
</tr>
<tr>
<td>$D_c$</td>
<td>48.0 in</td>
<td></td>
</tr>
<tr>
<td>$D_s$</td>
<td>54.0 in</td>
<td></td>
</tr>
<tr>
<td>$l_{as}$</td>
<td>51.0 in</td>
<td></td>
</tr>
<tr>
<td>$P_c$</td>
<td>987 kips</td>
<td></td>
</tr>
<tr>
<td>$P_b$</td>
<td>0 kips</td>
<td></td>
</tr>
<tr>
<td>$h$</td>
<td>2.18 ft</td>
<td></td>
</tr>
<tr>
<td>$M_{po}$</td>
<td>4058 kip-ft</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

1. Length of column longitudinal rebar extended into cap. See calculations below.
2. No prestressing in section.
3. Determined from sectional analysis. Tension in column longitudinal rebar may also be derived from sectional analysis.

**Equations:**

- Principal tension stress in joint:  
  
  $$p_t = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = 0.148 \text{ ksi}$$  
  
  SGS Eq. 8.15.2.1-3

- Principal compression stress in joint:  
  
  $$p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = 0.309 \text{ ksi}$$  
  
  SGS Eq. 8.15.2.1-4

**Results:**

- $T_c = T_{su} = 654 \text{ kips}$ Column tensile force at ultimate (from analysis above) SGS § 8.15.2
- $A_{jv} = l_{as}B_{cap} = 3060.0 \text{ in}^2$ Vertical joint area SGS Eq. 8.15.2.1-6
- $v_{jv} = T_c / A_{jv} = 0.214 \text{ ksi}$ Vertical joint shear stress Eq. 8.15.2.1-5
- $A_{jh} = (D_c + D_s)B_{cap} = 6120 \text{ in}^2$ Horizontal joint area SGS Eq. 8.15.2.1-8
- $f_v = P_c / A_{jh} = 0.1612 \text{ ksi}$ Vertical joint shear stress SGS Eq. 8.15.2.1-7
- $f_h = P_b / (B_{cap}D_s) = 0.0 \text{ ksi}$ Horizontal joint stress SGS Eq. 8.15.2.1-9

**Joint Proportions:**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Expression</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_c$</td>
<td>0.309 ksi</td>
<td>≤ 1.30 ksi maximum</td>
</tr>
<tr>
<td>$p_t$</td>
<td>0.148 ksi</td>
<td>≤ 0.87 ksi maximum</td>
</tr>
</tbody>
</table>

*Joint proportions are acceptable based on principal stress requirements.*
Minimum Development Length of Column Longitudinal Reinforcement

\[ l_{ac} \geq 2 \frac{d_{sl} f_{ye}}{f'_{ce}} \]

\[ d_{sl} = 0.88 \text{ in} \quad \text{No. 7} \]

\[ f_{ye} = 68 \text{ ksi} \]
\[ f'_{ce} = 7.0 \text{ ksi} \]  
\[ = \text{min (7.5 ksi, 7.0 ksi)} \text{ per Article 8.15.2.2.2} \]

\[ l_{ac} = 17.1 \text{ in} \quad \text{(minimum)} \]

Extend column reinforcement as far as practically possible; assume 3" clear from opposite face, per Article 8.15.2.2.2.

\[ l_{ac} = D_s - 3" = 51.0 \text{ in} \]

\[ 51.0 \text{ in} \geq 17.1 \text{ in} \quad \text{(minimum)} \quad \text{OK} \quad \text{Extend to top face of cap less 3 in cover.} \]

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections

Where principal tension in the joint is greater than or equal to 0.11\( \sqrt{f'_{ce}} \) and joint proportioning is acceptable per Article 8.15.2, additional transverse joint reinforcement for cap pocket connections per Article 8.15.5.2.3 is to be added, and Eqs. 8.15.3.1-1 and 8.15.3.1-2 are to be satisfied.

Check if principal tension, \( p_t \), is \( \geq 0.11\sqrt{f'_{ce}} \) (condition for likely joint cracking)

\[ \text{Calculated tension} = 0.148 \text{ ksi} \]
\[ \text{Limit, } 0.11\sqrt{f'_{ce}} = 0.251 \text{ ksi} \]

\[ p_t = 0.148 \text{ ksi} \leq 0.251 \text{ ksi limit} \]

Only transverse reinforcement per SGS Eq. 8.15.3.1-1 is required. No additional joint reinforcement is required.

For this HYBRID DESIGN EXAMPLE, only minimum transverse reinforcement is required. Due to the use of a combination of both post-tensioning and mild reinforcement, it is common for the principal tensile stress in the joint to be below the condition for likely joint cracking. For an example calculation with full joint detailing, see GROUTED DUCT DESIGN EXAMPLE FOR SDC C and D.

Calculate required volumetric ratio of transverse joint reinforcement:

\[ \rho_s \geq 0.11 \sqrt{f'_{ce}} / f_{yh} = 0.0037 \quad A_{st} = 9.6 \text{ in}^2 \]

\[ l_{ac} = 51.0 \text{ in} \]

Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

\[ \rho_s = \frac{4A_{sp}}{D' f_{sl} s} \]
\[ A_{sp} = 0.44 \text{ in}^2 \quad \text{(#6 hoop)} \]
\[ D' = 43.25 \text{ in} \quad \text{(confined diameter of column between centroids of hoop)} \]
\[ s = 8.0 \text{ in} \quad \text{(trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or } 0.3D_s \text{ per Art. 8.15.3.2.1)} \]
$D'_{gd} = \text{Column diameter} - 2 \times \text{clear cover} - \text{hoop diameter} - \text{long. column reinf diameter} + \text{grouted duct inside diameter} + \text{corrugation amplitude} \times 2.$

$D'_{gd} = 46.61 \text{ in}$
(deformed diameters are used for clearance calculations)

$\rho_s = 0.0047$

$\rho_s = 0.0047 \geq 0.0037 \text{ minimum} \quad \text{OK} \quad 8'' \text{ spacing is adequate (Note: The common practice for cast-in-place construction to carry hoop spacing from column into cap is not expected for precast bent caps as cages are built independently)}$

Note, must also check bursting requirements at post-tensioning anchorage per LRFD §5.10.9.6.3

**Vertical Stirrups Inside the Joint Region**

$A_s^{jvi} \geq 0.10 A_{st} \quad \text{Required area of vertical stirrups spaced evenly over a length equal to } D_c$

$A_{st} = 9.6 \text{ in}^2$

$A_s^{jvi} \geq 0.96 \text{ in}^2 \text{ spaced over } 4.0 \text{ ft}$

Use (2) #6 double leg stirrups spaced evenly inside joint region.

$A_s^{jvi} = 2 \text{ locations} \times 2 \text{ legs/location} \times 0.44 \text{ in}^2/\text{leg} = 1.76 \text{ in}^2 \geq 0.96 \text{ in}^2 \text{ O.K.}$

**Bedding Layer Reinforcement**

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3” thick. Cover on top column hoop to be 2” as shown in figures below, and resulting center-to-center spacing of transverse reinforcement is 4” throughout, in accordance with
Figures Showing Final Design
Simplified Section at Column Edge

4" overall spacing maintained

Column
Layer
Bedding
Bent Cap

1.5" per Art. 8.15.5.2.1
2.5" per Art. 8.8.14

Clear cover may be less than 2" per Art. 8.8.14

PLAN VIEW

Stirrups as req'd for shear
#11, tot 10 & b

Dc=4'-0"

#6 stirrups tot 2

12-0.6 tendon anchorage

Column reinf. and grouted duct

Column, below

#6 hoops
Integral Bent Cap Design Example for SDC C and D

This design example provides practitioners with a comprehensive example of the design activities required to design the integral, precast bent cap system presented in NCHRP 12-74. This structural system is developed for use in high seismic regions where the advantages of integral superstructure response may be desirable.

In this example, the integral connection between the superstructure and bent cap is made through post-tensioned spliced girders, which are discontinuous at the bent cap. Flexural continuity is provided through the splicing of girders through the bent cap using post-tensioning tendons. This structural type and seismic response was validated through experimental work under the NCHRP 12-74 project.

The connection between the bent cap and column is made through a grouted duct connection. This connection type was validated for high seismic regions under the NCHRP 12-74 project. The applicable commentary sections in the AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS) provide additional background.

Material Input Parameters

**Concrete Materials (Unconfined)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$</td>
<td>4.0 ksi</td>
<td>Specified 28-day compressive strength of column concrete</td>
</tr>
<tr>
<td>$f'_{ce}$</td>
<td>1.3 $f'_c$</td>
<td>Expected column concrete compressive strength</td>
</tr>
<tr>
<td>$f'_{cg}$</td>
<td>6.0 ksi</td>
<td>Specified 28-day compressive strength of girder concrete</td>
</tr>
</tbody>
</table>

**Grout Materials**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_{cgeb}$</td>
<td>8.0 ksi</td>
<td>Specified compressive strength of grout bedding layer</td>
</tr>
<tr>
<td>Check: $f'<em>{cgeb}$ (ksi) $\geq$ $\max[1.25(f'</em>{ce})+0.5, 6.0]$</td>
<td>8.0 ksi $\geq$ 7.0 ksi</td>
<td>O.K.</td>
</tr>
<tr>
<td>$f'_{cgv}$</td>
<td>8.0 ksi</td>
<td>Specified compressive strength of grout closure joint</td>
</tr>
<tr>
<td>Check: $f'<em>{cgv}$ (ksi) $\geq$ $\max[1.25(f'</em>{cg})+0.5, 6.0]$</td>
<td>8.0 ksi $\geq$ 8.0 ksi</td>
<td>O.K.</td>
</tr>
</tbody>
</table>

Closure joint grout shall have a 3 lb per cy fraction of polypropylene fibers for joint integrity

**Mild Reinforcing Steel**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_y$</td>
<td>60 ksi</td>
<td>Specified yield stress</td>
</tr>
<tr>
<td>$f_{ye}$</td>
<td>68 ksi</td>
<td>Expected yield stress</td>
</tr>
<tr>
<td>$f_{ue}$</td>
<td>95 ksi</td>
<td>Expected tensile strength</td>
</tr>
<tr>
<td>$E_s$</td>
<td>29000 ksi</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>0.002</td>
<td>Effective yield strain</td>
</tr>
<tr>
<td>$\varepsilon_{su}$</td>
<td>0.09</td>
<td>Reduced ultimate tensile strain</td>
</tr>
<tr>
<td>$f_{yh}$</td>
<td>60 ksi</td>
<td>Specified yield stress of confining hoops</td>
</tr>
</tbody>
</table>
Geometric Input Parameters

**Column Dimensions**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_c$</td>
<td>5.0 ft</td>
<td>Gross column diameter</td>
</tr>
<tr>
<td>$H_{clr}$</td>
<td>22.0 ft</td>
<td>Clear height of column</td>
</tr>
<tr>
<td>$H$</td>
<td>11.0 ft</td>
<td>Clear height to point of contraflexure (fixed-fixed column)</td>
</tr>
<tr>
<td>$L_p$</td>
<td>27.9 in</td>
<td>Idealized plastic hinge length</td>
</tr>
<tr>
<td>$L_p/D_c$</td>
<td>0.465</td>
<td>Normalized plastic hinge length</td>
</tr>
<tr>
<td>$L$</td>
<td>140 ft</td>
<td>Span length between centerline of bents</td>
</tr>
</tbody>
</table>

**Column reinforcement**

(20) No. 11 A706 Reinforcing Bars

- $A_{s,bar} = 1.56$ ft$^2$
- $A_{nt} = 31.2$ ft$^2$

**Superstructure Dimensions**

Superstructure consists of 72 inch tall post-tensioned girders with local end blocks at bent

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{girder}$</td>
<td>74.0 in</td>
<td>Depth of post-tensioned girder</td>
</tr>
<tr>
<td>$D_{haunch}$</td>
<td>2.0 in</td>
<td>Thickness of girder haunch (specified)</td>
</tr>
<tr>
<td>$D_{deck}$</td>
<td>8.0 in</td>
<td>Thickness of reinforced concrete deck</td>
</tr>
<tr>
<td>$D_s$</td>
<td>7.00 ft</td>
<td>Total depth of superstructure (in longitudinal direction)</td>
</tr>
<tr>
<td>$s_g$</td>
<td>9.00 ft</td>
<td>Girder center-to-center spacing</td>
</tr>
</tbody>
</table>

- $A_{ps, Top} = 2.604$ ft$^2$  $f_{p,e, Top} = 162$ ksi  Top tendon details (12-0.6" at 60% GUTS)
- $A_{ps, Mid} = 2.604$ ft$^2$  $f_{p,e, Mid} = 162$ ksi  Middle tendon details (12-0.6" at 60% GUTS)
- $A_{ps, Bot} = 1.519$ ft$^2$  $f_{p,e, Bot} = 0$ ksi  Bottom tendon details (7-0.6" unstressed)
**PROJECT DESIGN EXAMPLES**

**PROJECT NO:** NCHRP 12-74  
**DESIGNED BY:** Tobolski, MJ  
**DATE:** 12/31/2009

**CLIENT:** UNIVERSITY OF CALIFORNIA SAN DIEGO  
**CHECKED BY:** Restrepo, JI  
**DATE:** 12/31/2009

**SUBJECT:** AASHTO SEISMIC GUIDE SPECIFICATIONS INTEGRAL BENT CAP DESIGN EXAMPLE - SDCs C and D

- $w_{cj} = 2.0\text{ in}$  Specified closure joint dimension
- Check: $w_{cj} \leq 3''$ for grout closure joints
  - $2.0\text{ in} \leq 3.0\text{ in}\quad\text{O.K.}$

**NOTE:** For larger closure joints, concrete shall replace grout material and joint shall be reinforced to ensure the integrity of the concrete is maintained when the joint opens.

### Bent Cap Dimension

- $B_{cap} = 7.0\text{ ft}$ Bent cap width
- $h_{extra} = 6.0\text{ in}$ Additional depth of precast cap below girder (for splicing)
- $D_{P/C} = 6.67\text{ ft}$ Bent cap height
- $D_{cap} = 7.50\text{ ft}$ Total depth of bent cap including deck

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{sTop}$</td>
<td>$15.6\text{ in}^2$</td>
</tr>
<tr>
<td>$f_{peTop}$</td>
<td>162 ksi</td>
</tr>
<tr>
<td>$A_{sMid}$</td>
<td>$12.48\text{ in}^2$</td>
</tr>
<tr>
<td>$f_{peMid}$</td>
<td>162 ksi</td>
</tr>
<tr>
<td>$A_{psBC}$</td>
<td>$9.11\text{ in}^2$</td>
</tr>
<tr>
<td>$f_{peMid}$</td>
<td>162 ksi</td>
</tr>
</tbody>
</table>

**Moment resisting connections** are designed to transmit the maximum forces produced when the column reaches its overstrength moment capacity, $M_{po}$.

**Seismic Demands**

The focus of this design example is on detailing of the bent cap and superstructure. Traditional seismic displacement checks, etc. shall be carried out but are not included herein.

**Note:** Capacity of the bent cap section needs to satisfy applicable load combinations, including the Strength load cases and capacity protection during Extreme Event load cases. The bent cap section capacity is not investigated in this design example.

Determine the idealized plastic moment capacity of the column, $M_p$, using a sectional analysis program, and calculate the overstrength moment capacity, $M_{po}$, per Article 8.5.
Axial load acting on column per extreme event load case = 825.0 kips \( (P_c) \)

\( f'_{ce} \) (expected concrete compressive strength) = 1.3 \* \( f'_{c} \) = 5.2 ksi \( \text{SGS} \)

\( f'_{ye} \) (expected steel yield strength) = 68 ksi \( \text{column bars} \) \( \text{8.4.4} \)

\[ A_{st} = 31.2 \text{ in}^2 \text{ Total area of column reinforcement} \]

\[ M_p = 6550 \text{ kip-ft} \]

\[ M_{po} = \lambda_{mo} M_p \]

\[ \lambda_{mo} = 1.2 \text{ (ASTM A706)} \]

\[ M_{po} = 7860 \text{ kip-ft} \]

Overstrength bending moment capacity of column \( \text{at critical section} \) \( \text{SGS Eq. 8.5-1} \)

\[ V_{po} = M_{po} / H = 715 \text{ kip-ft} \]

Overstrength shear demand from column plastic hinge

\[ M_{po,cap} = V_{po} \times [H + 0.5 D_{cap}] = 10540 \text{ kip-ft} \]

Overstrength bending moment demand at superstructure

**Superstructure Capacity Design for Longitudinal Direction for SDC C and D** \( \text{SGS \# 8.10} \)

The superstructure shall be designed as a capacity protected member. Flexural capacity of the section shall be determined via moment-curvature analysis per SGS §8.5

**Seismic Moment Capacity of Superstructure**

\[ B_{eff} = 12.0 \text{ ft} \]

Effective width of superstructure (open soffit, girder-deck system) \( \text{SGS Eq. 8.10-2} \)

\[ B_{eff} \] will capture (2) girder webs. Perform moment-curvature on superstructure with (2) girders

From moment-curvature analysis:

**Nominal Bending Moment Capacity**

\[ M_{ne}^+ = 8975 \text{ kip-ft} \]

\[ M_{ne}^- = -13015 \text{ kip-ft} \]

**Ultimate Curvature Capacity**

\[ \phi_u^+ = 6.641 \times 10^{-3} \text{ 1/ft} \]

\[ \phi_u^- = -4.940 \times 10^{-3} \text{ 1/ft} \]

**Maximum Extreme Event I Demands Acting on 2 Girders from Plastic Hinging**

Design actions acting on 2 girders

\[ \gamma_D M_D = -850 \text{ kip-ft} \]

\[ \gamma V_D = 415 \text{ kip} \]

Factored dead load demand

\[ 0.5 M_{LL,min} = -2248 \text{ kip-ft} \]

\[ 0.5 V_{LL,min} = 90 \text{ kip} \]

Factored live load for seismic design

\[ 0.5 M_{LL,max} = 414 \text{ kip-ft} \]

\[ 0.5 V_{LL,max} = -10 \text{ kip} \]

Factored live load for seismic design

\[ PGA_h = 0.60 \text{ g} \]

Horizontal peak ground acceleration

\[ V / H = 1.00 \]

Vertical to horizontal spectral acceleration ratio for zero period acceleration

\( \text{Based on Bozorgnia and Campbell (2004)} \)

\[ PGA_v = 0.60 \text{ g} \]

Vertical peak ground acceleration
Seismic moment demand from vertical
\[ M_{E,v} = \frac{PGA \cdot V_D}{L} \cdot \frac{L^2}{12} = 2324 \text{ kip-ft} \]
Seismic shear demand from vertical
\[ V_{E,v} = PGA \cdot V_D = 199 \text{ kip} \]

\[ M_{EEI,max} = -10692 \text{ kip-ft} \leq M_{ne} = 13015 \text{ kip-ft} \quad \text{O.K.} \]
\[ M_{EEI,min} = 7158 \text{ kip-ft} \geq M_{ne} = 8975 \text{ kip-ft} \quad \text{O.K.} \]

**Superstructure Rotation Capacity**

\[ \phi_u = 4.940E-03 \text{ 1/ft} \quad \text{Limiting superstructure curvature capacity from moment-curvature} \]
\[ L_{p,s} = \frac{D_s}{2} = 3.5 \text{ ft} \quad \text{Superstructure effective hinge length} \quad \text{SGS Eq. 4.11.6.2-1} \]

\[ \theta_u = 0.017 \text{ rad} \quad \text{Limiting superstructure rotation capacity} \]
Check: \[ \theta_u = \frac{0.017 \text{ rad}}{0.010 \text{ rad}} > 0.010 \quad \text{O.K.} \quad \text{SGS § 8.10.3} \]

\[ \Delta_{settle} = \theta_u \cdot L = 29.0 \text{ in} \quad \text{Permissible relative settlement between bents} \quad \text{SGS § 8.10.5} \]

Closed hoops shall be placed in the bottom flange of the precast girder spaced with the vertical shear reinforcement, not to exceed 8” spacing. Closed hoops shall be placed within a distance equal to the depth of the precast girder extending from the end of the precast girder. Closed hoops shall be of the same size as vertical shear reinforcement within this distance (except

**Seismic Shear Capacity across Closure Joint**

\[ V_{EEI} = 390 \text{ kip} \quad \text{Maximum vertical shear acting on single girder} \]
\[ N_{PT} = 844 \text{ kip} \quad \text{Effective beam post-tensioning force} \]
\[ \mu = 0.6 \quad \text{Shear friction coefficient} \]
\[ \phi = 0.9 \quad \text{Strength reduction factor for shear} \quad \text{LRFD § 5.5.4.2} \]

\[ \phi V_n = \phi \mu N_{PT} = 456 \text{ kip} \]

\[ V_{EEI} = 390 \text{ kip} \leq \phi V_n = 456 \text{ kip} \quad \text{O.K.} \]

**Seismic Shear Capacity at Girder End**

Investigate seismic shear capacity during joint opening for design of hanger bars

\[ V_{EEI} = 170 \text{ kip} \quad \text{Vertical shear acting on single girder during maximum positive flexure} \]
\[ V_{EEI} = 390 \text{ kip} \quad \text{Vertical shear acting on single girder during maximum negative flexure} \]

Place shear reinforcement based on a 30d fan considering shear depth equal to neutral axis depth

\[ c = 8.13 \text{ in} \quad \text{Neutral axis depth at nominal moment capacity (from moment-curvature)} \]
\[ \phi = 0.9 \quad \text{Strength reduction factor for shear} \]

\[ A_{s,req} = 3.15 \text{ in}^2 \quad \text{Minimum required area of shear reinforcement} \]
\[ \text{length} = 14.08 \text{ in} \quad \text{Distance over which shear reinforcement shall be placed} \]
PROJECT DESIGN EXAMPLES
PROJECT NO  NCHRP 12-74
CLIENT  UNIVERSITY OF CALIFORNIA SAN DIEGO
SUBJECT  AASHTO SEISMIC GUIDE SPECIFICATIONS INTEGRAL BENT CAP DESIGN EXAMPLE - SDCs C and D

USE:  No. 8 headed reinforcing bars spaced at 7.5" (2) per location

NOTE: Maintain minimum 1" clear cover from face of head on bottom rebar

\[ A_{s,prov} = 3.16 \text{ in}^2 \]  Minimum required area of shear reinforcement

\[ A_{s,req} = 3.15 \text{ in}^2 \leq A_{s,prov} = 3.16 \text{ in}^2 \]  O.K.

Superstructure Capacity Design for Transverse Direction for SDC C and D  

SGS § 8.11

The bent cap shall be designed as a capacity protected member. Bent caps are considered integral if they terminate outside of the exterior girder and respond monolithically with the girder system.

\[ B_{eff} = 15.0 \text{ ft} \]  Effective width of bent cap flange  

SGS Eq. 8.11-1

From moment-curvature analysis:

\[ M_{ne}^+ = 14125 \text{ kip-ft} \]
\[ M_{ne}^- = -13105 \text{ kip-ft} \]

NOTE: Details of moment-curvature not shown

Maximum Extreme Event I Demands Acting on Bent Cap from Plastic Hinging

\[ M_{EEI,min} = -12383 \text{ kip-ft} \]
\[ M_{EEI,max} = 11935 \text{ kip-ft} \]

O.K.

Joint Performance  

SGS § 8.15.1

Moment resisting connections shall be designed to transmit the maximum forces produced when the column has reached its overstrength capacity, \( M_{po} \)}
**Joint Proportioning**

**Principal Stresses**

Principal stresses in the joint are limited by the following equations:

- **Principal compression,** \( p_c \):
  \[ p_c \leq 0.25 \, f'_c e \leq 1.30 \text{ ksi maximum} \quad \text{SGS Eq. 8.15.2.1-1} \]

- **Principal tension,** \( p_t \):
  \[ p_t \leq 0.38 \sqrt{f'_c e} \leq 0.867 \text{ ksi maximum} \quad \text{SGS Eq. 8.15.2.1-2} \]

### Notes:

1. Length of column longitudinal rebar extended into cap. See calculations below.
2. No prestressing in section.
3. Determined from sectional analysis. Tension in column longitudinal rebar may also be derived from sectional analysis.

\[
p_c = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = 0.433 \text{ ksi} \quad \text{Principal tension stress in joint} \quad \text{SGS Eq. 8.15.2.1-3}
\]

\[
p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = 0.502 \text{ ksi} \quad \text{Principal compression stress in joint} \quad \text{SGS Eq. 8.15.2.1-4}
\]

- **Tensile force at ultimate:** \( T_c = M_{po, col} / h = 2781 \text{ kips} \)
- **Vertical joint area:** \( A_{jv} = l_{ac}B_{cap} = 5964.0 \text{ in}^2 \)
- **Vertical joint shear stress:** \( v_{jv} = T_c / A_{jv} = 0.466 \text{ ksi} \)
- **Horizontal joint area:** \( A_{jh} = (D_c + D_s)B_{cap} = 12096 \text{ in}^2 \)
- **Horizontal joint shear stress:** \( f_{jh} = P_c / A_{jh} = 0.0682 \text{ ksi} \)
- **Horizontal joint stress:** \( f_h = P_b / (B_{cap} D_s) = 0.0 \text{ ksi} \)

\[
p_c = 0.502 \, \text{ksi} \leq 1.30 \, \text{ksi maximum} \quad \text{OK} \quad \text{Joint proportions are acceptable based on principal stress requirements.}
\]

\[
p_t = 0.433 \, \text{ksi} \leq 0.867 \, \text{ksi maximum} \quad \text{OK}
\]
Minimum Development Length of Column Longitudinal Reinforcement

\[ l_{ac} \geq 2 \frac{d_{bl} f_{ye}}{f'_{cg}} \]
\[ d_{bl} = 1.41 \text{ in} \quad (\#11 \text{ rebar}) \]
\[ f_{ye} = 68 \text{ ksi} \]
\[ f'_{cg} = 7.0 \text{ ksi} \quad = \text{min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2} \]

\[ l_{ac} = 27.4 \text{ in} \quad \text{(minimum)} \]

Extend column reinforcement as far as practically possible into precast bent cap portion; assume 3” clear from opposite face of precast bent cap. Additional cover provided after casting of deck.

\[ l_{ac} = D_{girder} - 3" = 71.0 \text{ in} \]

71.0 in ≥ 27.4 in (minimum) OK Extend to top face of cap less 3 in cover.

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections

Where principal tension in the joint is greater than or equal to 0.11√f’_c and joint proportioning is acceptable per Article 8.15.2, additional transverse joint reinforcement for cap pocket connections per Article 8.15.5.2.3 is to be added, and Eqs. 8.15.3.1-1 and 8.15.3.1-2 are to be satisfied.

Check if principal tension, p_t, is ≥ 0.11√f’_c (condition for likely joint cracking)

Calculated tension = 0.433 ksi
Limit, 0.11√f’_ce = 0.251 ksi

\[ p_t = 0.433 \text{ ksi} \geq 0.251 \text{ ksi limit} \quad \text{Additional joint reinforcement is required.} \]

Calculate required volumetric ratio of transverse joint reinforcement:

Maximum of:
\[ \rho_s \geq 0.11 \sqrt{f’_c / f_{yh}} \]
\[ \rho_s \geq 0.40 A_{st} / l_{ac}^2 \]

\[ A_{st} = 31.20 \text{ in}^2 \]
\[ l_{ac} = 71.0 \text{ in} \]

Maximum of:
\[ 0.11 \sqrt{f’_ce / f_{yh}} = 0.0042 \quad \text{governs} \]
\[ 0.40 A_{st} / l_{ac}^2 = 0.0025 \]

Use \[ \rho_s \geq 0.0042 \]

Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

\[ \rho_s = \frac{4A_{sp}}{D'_{gd}s} \]
\[ A_{sp} = 0.44 \text{ in}^2 \quad (\#6 \text{ hoop}) \]
\[ D'_{gd} = 57.92 \text{ in} \quad (\text{confined diameter of column between centroids of \#6 hoop}) \]
s = 6.0 in

D'_{gd} = Column diameter - 2 x clear cover - hoop diameter - long. column rein diameter + grouted duct inside diameter + corrugation amplitude x 2.

D'_{gd} = 58.05 in
(deformed diameters are used for clearance calculations)

\( \rho_s = 0.0051 \)

\( \rho_s = 0.0051 \geq 0.0042 \) minimum OK 6" spacing is adequate (Note: The common practice for cast-in-place construction to carry hoop spacing from column into cap is not expected for precast bent caps as cages are built independently)

**Integral Bent Cap Joint Description**

SGS § 8.15.4.1

The system considered is classified as a "T" joint for joint shear analysis because:
- Exterior column joints with adequate development length of bent cap longitudinal reinforcement.

Bent cap width is required to extend 12" past column on both sides

\( B_{cap} \geq D_c + 24" \quad 84 \text{ in} \geq 84 \text{ in} \) O.K., Bent cap width adequate for integral design SGS § 8.13.4.1

**Joint Shear Reinforcement**

SGS § 8.15.4.2

**Vertical Stirrups**

SGS § 8.15.4.2.1

Vertical stirrups or ties shall be placed transversely within a distance equal to the column diameter, \( D_c \), extending from either side of the column centerline.

\[ A^v_s \geq 0.20 \ A_{st} \]

\[ A^v_s \geq 6.24 \text{ in}^2 \]

Minimum required area of vertical reinforcement SGS Eq. 8.15.4.2.1-1

\( A^v_s \) required: in Regions 1 and 2

USE: No. 6 Stirrups, 4 legs per location

\( 4 \text{ locations} \times 4 \text{ legs/location} \times 0.44 \text{ in}^2/\text{leg} \)

\[ A^v_s = 7.04 \text{ in}^2 \geq 6.24 \text{ in}^2 \text{ O.K.} \]
SGS Figure 8.15.4.2.1-1

For multi-column bents, vertical stirrups shall additionally be provided in accordance with Article 8.15.5. Overlapping vertical stirrups from shall count towards meeting the requirements

\[ A_{sv} \geq 0.175 A_{st} \]
\[ A_{sv} \geq 5.46 \text{ in}^2 \]
Minimum required area of vertical reinforcement outside joint  

\( A_{sv} \geq \text{in}^2 \)

SGS Eq. 8.15.5.1.1-1

USE: No. 6 Stirrups, 4 legs per location

\[ A_{sv} = 7.04 \text{ in}^2 \geq 5.46 \text{ in}^2 \text{ O.K.} \]

Spacing outside of joint region: 10.0 in

\[ A_{sv} \geq 0.135 A_{st} \]
\[ A_{sv} \geq 4.21 \text{ in}^2 \]
Minimum required area of vertical reinforcement inside joint  

\( A_{sv} \geq \text{in}^2 \)

SGS Eq. 8.15.5.1.2-1

3 locations x 4 legs / location x 0.44 in²/leg

\[ A_{sv} = 5.28 \text{ in}^2 \geq 4.21 \text{ in}^2 \text{ O.K.} \]

Horizontal Stirrups

SGS § 8.15.4.2.2

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches.

\[ A_{sh} \geq 0.10 A_{st} \]
\[ A_{sh} \geq 3.12 \text{ in}^2 \]
Minimum required area of horizontal reinforcement  

\( A_{sh} \geq \text{in}^2 \)

SGS Eq. 8.15.4.2.1-1

\[ D_{p/c} = 80.00 \text{ in} \]

Depth of precast bent cap

\[ N_{\text{space, min}} = \frac{D_{p/c}}{18} = 4.4 \]
Minimum number of spaces between cross ties

Minimum number of cross ties (excluding bottom of vertical stirrups) 5

Space at 12” and hook on side face reinforcement

USE: No. 4 cross ties

Place at each vertical stirrup

\[ \frac{5 \text{ locations} \times 5 \text{ ties / location}}{0.2 \text{ in}²/tie} \]

\[ A_{sh} = 5.00 \text{ in}^2 \]
\[ \geq 3.12 \text{ in}^2 \text{ O.K.} \]

horizontal stirrups shall be No. 4 reinforcement per SGS §8.15.4.2.2
For multi-column bents, horizontal stirrups shall additionally satisfy the requirements of Article 8.15.5.

From inspection, this provision is satisfied.

**Horizontal Side Reinforcement**

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the following two requirements with a maximum spacing of 12 inches.

\[
A_{sf}^{\text{top}} \geq \begin{cases} 0.10 A_{\text{cap}_{\text{top}}} = 1.56 \text{ in}^2 & A_{\text{cap}_{\text{top}}} = 15.60 \text{ in}^2 \\ 0.10 A_{\text{cap}_{\text{bot}}} = 1.25 \text{ in}^2 & A_{\text{cap}_{\text{bot}}} = 12.48 \text{ in}^2 \end{cases}
\]

\[
A_{sf} \geq 1.56 \text{ in}^2 \quad \text{Minimum horizontal side face reinforcement required}
\]

\[
D_{P/C} = 80.00 \text{ in} \quad \text{Depth of precast bent cap}
\]

\[
N_{\text{space,min}} = D_{P/C} / 18 \text{in} = 6.7 \quad \text{Minimum number of spaces between reinforcement}
\]

Minimum number of locations of reinforcement per side of bent cap \(7\)

**J-Bars**

For integral cap of bents skewed greater than 20° vertical J-bars hooked around the longitudinal top deck steel are required.

Skew is less than 20° therefore no J-bars are required.

**Additional Longitudinal Cap Beam Reinforcement**

For multi-column bents, additional longitudinal cap beam reinforcement shall satisfy the requirements of Article 8.15.5.

\[
A_{sj}^{\text{v}} \geq 0.245 A_{st} \quad A_{st} = 31.20 \text{ in}^2
\]

\[
A_{sj}^{\text{h}} \geq 7.64 \text{ in}^2 \quad \text{Additional longitudinal cap beam reinforcement}
\]

**Torsional Shear-Friction Requirement**

For precast girder systems, there is no bottom soffit slab present to aide in the transfer of moment and shear demands from the superstructure to the column. Therefore, these demands are transferred through the bent cap by a torsional mechanism. The small distance between the face of the column and beams will not facilitate the development of traditional torsional spiral cracks, thus conventional torsional design provisions are not valid. Torsional capacity is calculated using a plastic torsional shear friction mechanism.
PROJECT DESIGN EXAMPLES

PROJECT NO NCHRP 12-74

CLIENT UNIVERSITY OF CALIFORNIA SAN DIEGO

CHECKED BY Restrepo, JI

DATE 12/31/2009

Subject: AASHTO SEISMIC GUIDE SPECIFICATIONS INTEGRAL BENT CAP DESIGN EXAMPLE - SDCs C and D

- \( B_{\text{cap}} = 84.0 \text{ in} \) Bent cap width
- \( D_{\text{cap}} = 90.0 \text{ in} \) Total depth of bent cap including deck
- \( A_{\text{cap}} = 7560 \text{ in}^2 \) Total area of bent cap

- \( A_{\text{s,mild}} = 30.08 \text{ in}^2 \) Total area of mild reinforcement in bent cap (+ side face)
- \( A_{ps} = 9.11 \text{ in}^2 \) Total area of post-tensioning in bent cap
- \( A_{\text{total}} = A_{\text{s,mild}} + A_{ps} = 39.19 \text{ in}^2 \) Total area of steel in bent cap (mild and post-tensioning)

- \( f_{pe} = 162 \text{ ksi} \) Effective post-tensioning force including losses
- \( P = A_{ps}f_{pe} + A_{\text{total}}E_s(0.0005) = 2045 \text{ kip} \) Total compression force including steel force developed from dilation of section

- \( \mu = 1.4 \) Shear friction coefficient (bent cap in continuous)
- \( \tau = \frac{\mu P}{A_{\text{cap}}} = 54.5 \text{ kip/ft}^2 \) Shear friction stress

- \( T = 0.5 M_{ps,\text{cap}} = 5270 \text{ kip-ft} \) Torsional demand on bent cap
- \( V_V = 0.5 P_c = 413 \text{ kip} \) Vertical shear demand on bent cap
- \( V_L = 0.5 V_o = 357 \text{ kip} \) Longitudinal shear demand on bent cap

- \( \phi = 0.9 \) Strength reduction factor for shear

- \( T = \phi \tau [A_1x_1 + A_2x_2 + A_3x_3 + A_4x_4] \) Torsional shear friction resistance
- \( V_V = \phi \tau [A_1 - A_4] \) Vertical shear friction resistance
- \( V_L = \phi \tau [A_2 - A_3] \) Longitudinal shear friction resistance

<table>
<thead>
<tr>
<th>Region</th>
<th>Area</th>
<th>Distance from Centroid</th>
<th>First moment centroid</th>
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<tr>
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<td>8.90</td>
<td>2.90</td>
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<tr>
<td>Total</td>
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<td>120.2</td>
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Bedding Layer Reinforcement

Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3” thick. Cover on top column hoop to be 1.5” as shown in figures below, and resulting center-to-center spacing of transverse reinforcement is 3” throughout, in accordance with Article 8.8.14.
Elevation View at Column Connection

Typical Section at Column Connection
Clear cover may be less than 2" per Art. 8.8.14

3" overall spacing maintained

Closeup of Bedding Layer

Typical Section at Girder Connection
Plan View at Connections