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
  - Design example for cap pocket connection in SDCs C and D (additional joint reinforcement)
- 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

DESIGN FLOW CHART FOR CAST-IN-PLACE AND PRECAST BENT CAP JOINT DESIGN PER PROPOSED ARTICLE 8.13 (SDC A)

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 nonintegral
    - ***IF:*** Bent cap is Cast-In-Place
      - TBD by Others
    - ***IF:*** Bent cap is precast
      - Provide longitudinal development to satisfy:
$\ell_{ac} \geq 2d_{bl}\, f_y/f'_{cg}$ (Eq. 8.15.2.2.2-1 Modified) for grouted duct;
$\ell_{ac} \geq 2.3d_{bl}\, f_y/f'_c$ (Eq. 8.15.2.2.2-2 Modified) for cap pocket.
      - Provide transverse confinement to satisfy
$\rho_s \geq 0.11\sqrt{f'_c}/f_{yh}$ (Eq. 8.15.3.1-1)
      - Provide inside vertical joint shear stirrups:
$A_s^{jvi} \geq 0.08A_{st}$ (Eq. 8.13.4.2.2a-1) for grouted duct;
$A_s^{jvi} \geq 0.08A_{st}$ (Eq. 8.13.4.2.3a-1) for cap pocket.
        - ***IF:*** Bedding layer ≤ 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.

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Construction Specifications (BCS)

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: $f'_{cg} \geq \max[1.25(f'_{ce}+0.5)=7 \text{ ksi}, 6 \text{ ksi}]$

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)

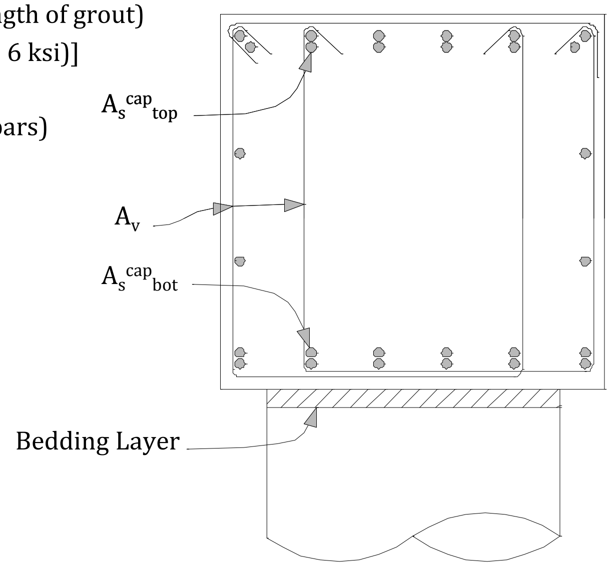
Stirrup bar size: #6

Column diameter = 4.0 ft (D_c)

A_{st} = 25.4 in^2 (#10 Tot 20)

Hoop bar size: #6

Hoop spacing = 4.0 in



Typical cap section within D_c from face of column.
Note: Bent cap reinforcement shown reflects that required by load combinations before application of joint shear design requirements.

Joint Performance

SGS 8.13.1

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)

SGS 8.13.2.2

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

Eq. 8.15.2.2.2-1 (modified)

d_{bl} = 1.27 in (#10 rebar)
f_y = 60 ksi (Article 8.13.2.2 permits use of f_y instead of f_{ye})
f'_{cg} = 7.0 ksi = min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2

l_{ac} = 21.8 in (minimum)

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

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

57.0 in ≥ 21.8 in minimum OK Extend to top face of cap less 3 in cover.

Minimum Joint Reinforcing SGS 8.13.3.2.1

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:

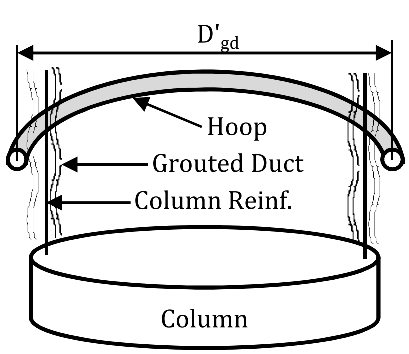
$\rho_s \geq 0.11 \sqrt{f'_c} / f_{yh} \geq 0.0037$ Eq. 8.15.3.1-1

$\rho_s \geq$ 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 in^2 (#6 hoop)
D'_{gd} = 45.92 in (confined diameter of column between centroids of #6 hoop)
s = 10.0 in (trial spacing of transverse reinforcement hoops, 12" or $0.3D_s$ maximum per Art. 8.13.3.2.1)



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

ρ_s = 0.0038

ρ_s = 0.0038 ≥ 0.0037 minimum OK 10" 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 9.37 inches, total 6 hoops.

Nonintegral Bent Cap Joint Shear Design

SGS 8.13.4

Vertical Stirrups Inside the Joint Region

SGS 8.13.4.2.2a

$A_s^{jvi} \geq 0.08\ A_{st}$ Eq. 8.13.4.2.2a-1

$A_{st} = 25.40\ \text{in}^2$

$A_s^{jvi} \geq 2.03\ \text{in}^2$

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

A_s^{jvi} = 3 patterns x 2 legs / pattern x 0.44 in^2/ leg = 2.64 in^2 ≥ 2.03 in^2 OK

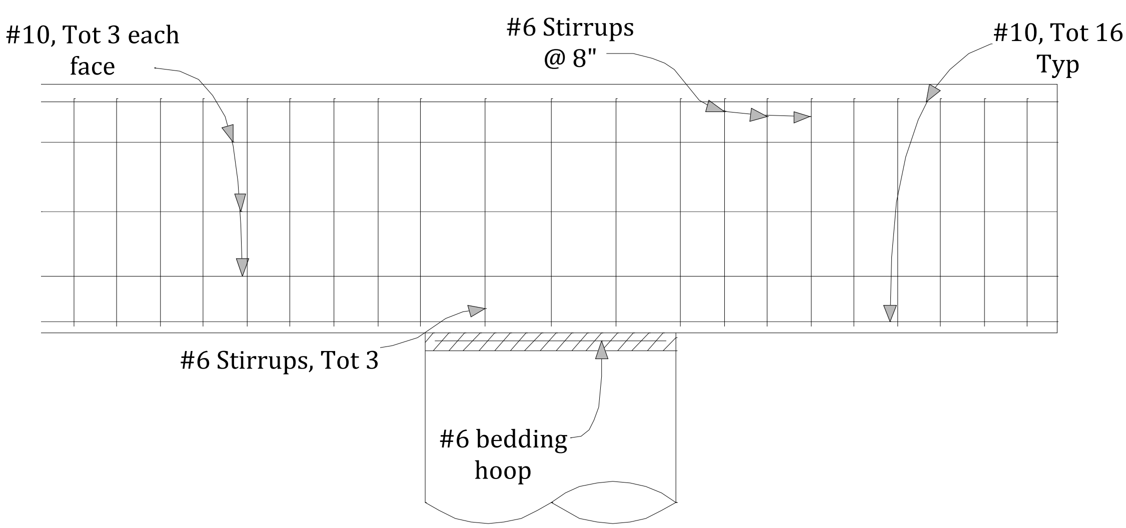
Note: Three patterns are required and used. The minimum number of stirrups is two, per Article 8.13.4.2.2a, and the bar size is no smaller than what is used in the bent cap shear stirrups. If only two stirrups were required, three may be used to reduce the spacing between stirrups to satisfy temperature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

Bedding Layer Reinforcement

SGS 8.13.4.2.1

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.

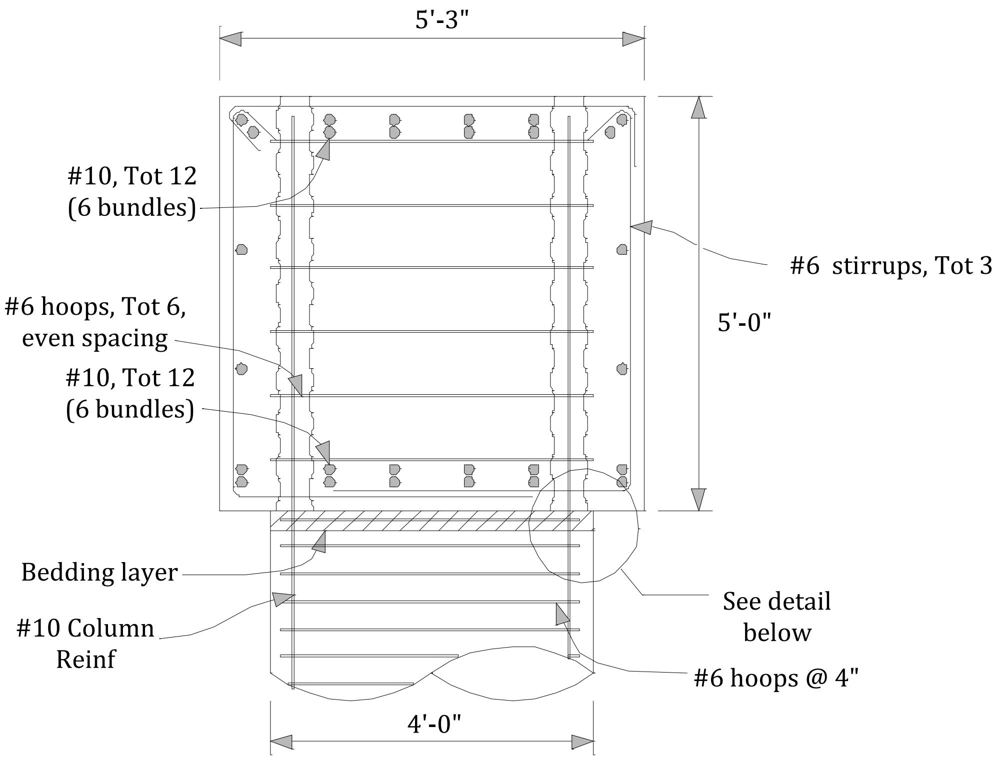
Figures Showing Final Design



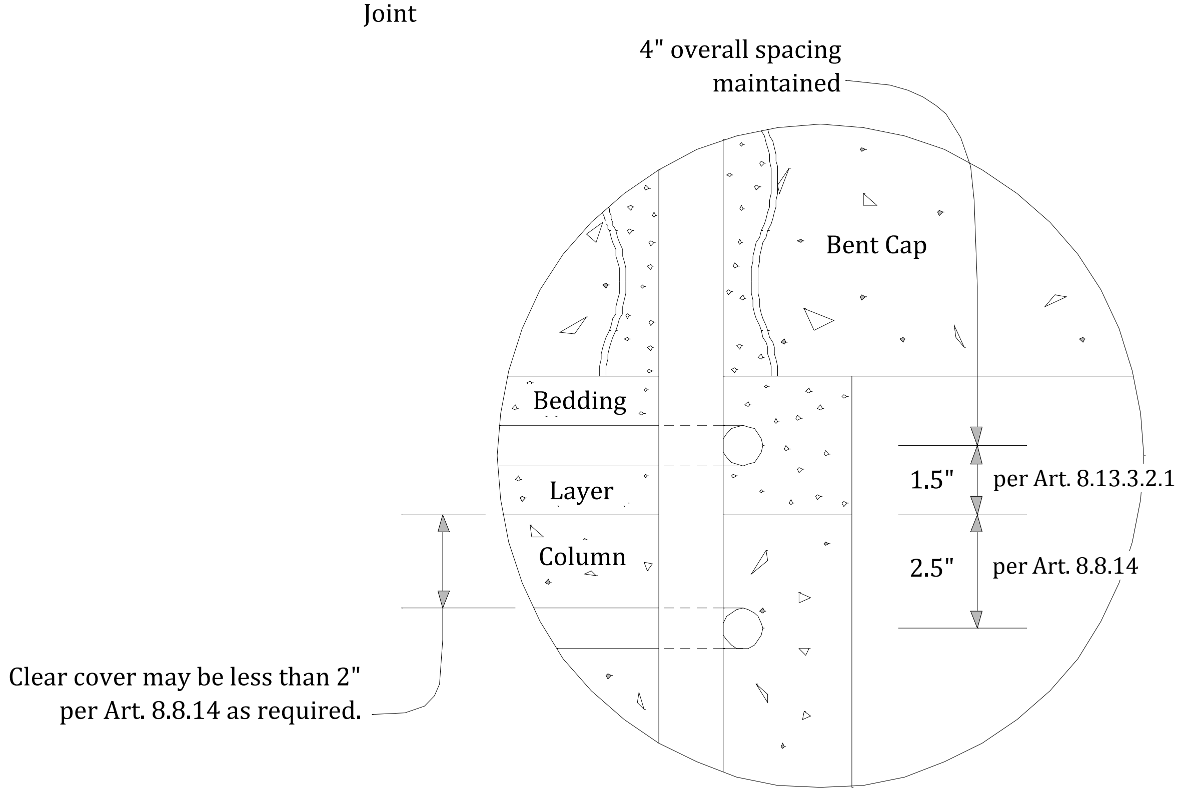
Elevation View at Column Connection

PROJECT	DESIGN EXAMPLES	SHEET NO	4	OF	5
PROJECT NO	NCHRP 12-74	DESIGNED BY	M. STILLER	DATE	11/6/2009
CLIENT	SACRAMENTO STATE UNIV.	CHECKED BY	E. MATSUMOTO	DATE	11/9/2009

SUBJECT AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL GROUTED DUCT BENT CAP JOINT DESIGN EXAMPLE - SDC A



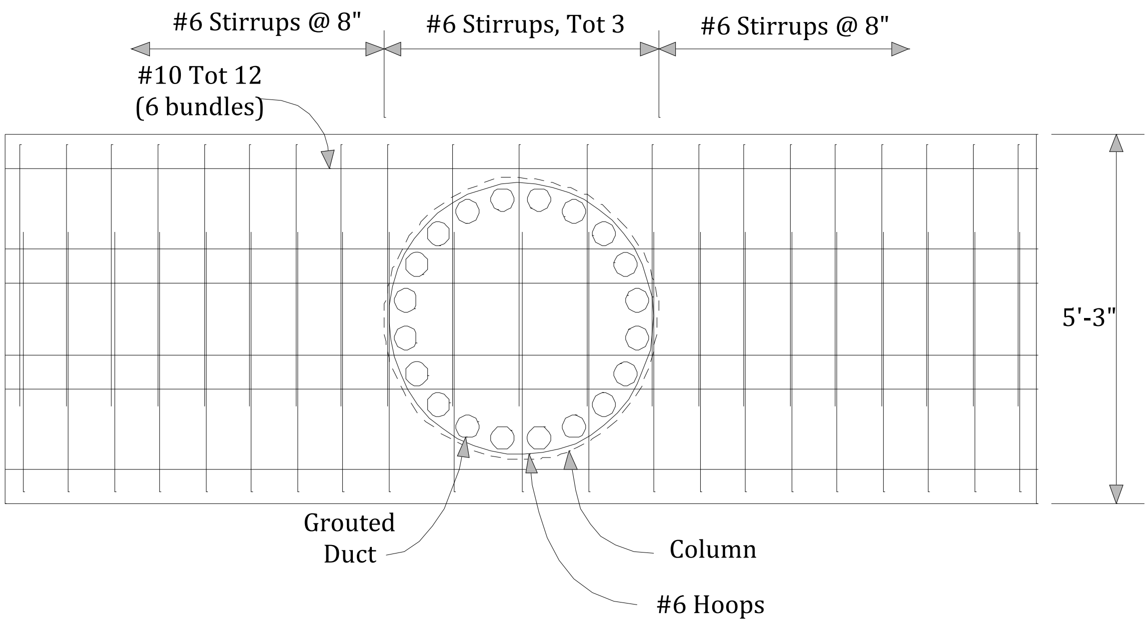
Typical Section through Joint



Simplified Section at Column Edge

PROJECT	DESIGN EXAMPLES	SHEET NO	5	OF	5
PROJECT NO	NCHRP 12-74	DESIGNED BY	M. STILLER	DATE	11/6/2009
CLIENT	SACRAMENTO STATE UNIV.	CHECKED BY	E. MATSUMOTO	DATE	11/9/2009

SUBJECT AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL GROUTED DUCT BENT CAP JOINT DESIGN EXAMPLE - SDC A



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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Construction Specifications (BCS)

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.

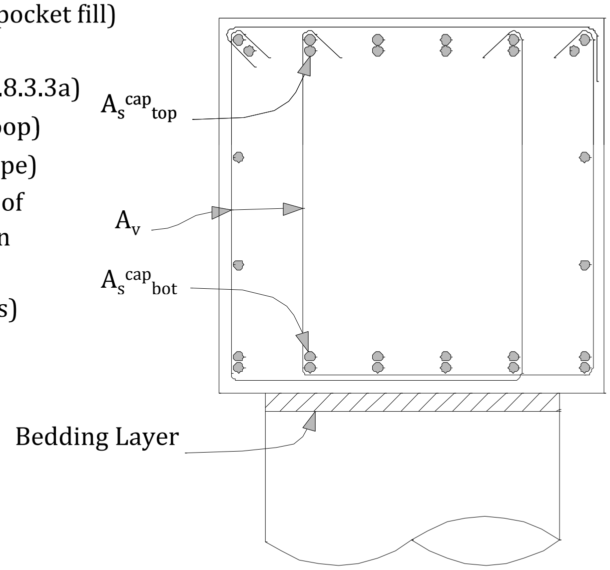
Geometry and Design Parameters

f'_c = 4.0 ksi (specified compressive strength of bent cap)
$f'_{ce} = 1.3f'_c$ = 5.2 ksi (expected f'_c of bent cap)
f'_{c_pocket}: (specified compressive strength of pocket fill)
Select cap pocket strength to satisfy BCS.
$f'_{c\_pocket} = 1.3\ f'_c + 0.5$ ksi = 5.7 ksi (BCS 8.13.8.3.3a)
f_{yh} (yield strength of equivalent hoop)
f_{yp} (nominal yield stress of steel pipe)
θ (angle between horizontal axis of cap and pipe helical corrugation or lock seam)
f_y = 60 ksi (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 in^2 (#10 Tot 20)

Hoop bar size: #6
Hoop spacing = 4.0 in



Typical cap section within D_c from face of column.
Note: Bent cap reinforcement shown reflects that required by load combinations before application of joint shear design requirements.

Joint Performance

SGS 8.13.1

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) SGS 8.13.2.2

$$l_{ac} \geq \frac{2.3 d_{bl} f_y}{f'_c}$$ Eq. 8.15.2.2.2-2 (modified)

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

l_{ac} = 30.7 in (minimum)

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

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

57.0 in ≥ 30.7 in minimum OK Extend to top face of cap less 3 in cover.

Minimum Joint Reinforcing SGS 8.13.3.2.2

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:

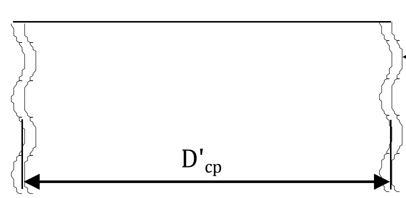
$\rho_s \geq 0.11 \sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1

$\rho_s \geq$ 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. SGS 8.15.3.2.2

$$\rho_s = \frac{4A_{sp}}{D'_{cp}\, s} \qquad s = \frac{4A_{sp}}{D'_{cp}\, \rho_s}$$

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



Corrugated steel pipe

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

s = 9.9 in max spacing
Therefore, the number of equivalent hoops per foot is 12" / s = 1.216 hoops/ft

Calculate the nominal confining hoop force of the equivalent hoops.

$F_H = n_h A_{sp} f_{yh}$ Eq. 8.15.3.2.2-2

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

F_H = 32.11 kips/ft

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

$$t_{pipe} \geq \max \begin{cases} \dfrac{F_H}{H_p f_{yp} \cos\theta} \\ 0.060 \text{ in} \end{cases}$$ Eq. 8.15.3.2.2-1

F_H =	32.1	kips/ft	
H_p =	12.0	in/ft	(specified unit length)
f_{yp} =	30.0	ksi	(manufacturer specified)
θ =	20.0	deg	(manufacturer specified)

$t_{pipe} \geq$ 0.0949 in

Use a 12 gage corrugated steel pipe, 48" nominal inside diameter. t_{pipe} = 0.105 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:

$$t_{pipe} \geq 0.04 \frac{D'_{cp}\sqrt{f'_c}}{f_{yp}\cos\theta} = 0.1381 \text{ in} \quad \text{and} \geq 0.06 \text{ in}$$ (Note: f'_c refers to cap pocket) 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

SGS 8.13.4

Vertical Stirrups Inside the Joint Region

SGS 8.13.4.2.3a

$A_s^{jvi} \geq 0.08\ A_{st}$ Eq. 8.13.4.2.3a-1

$A_{st} =$ 25.40 in^2

$A_s^{jvi} \geq$ 2.03 in^2

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

A_s^{jvi} = 3 patterns x 2 legs/ pattern x 0.44 in^2/ leg = 2.64 in^2 ≥ 2.03 in^2 OK

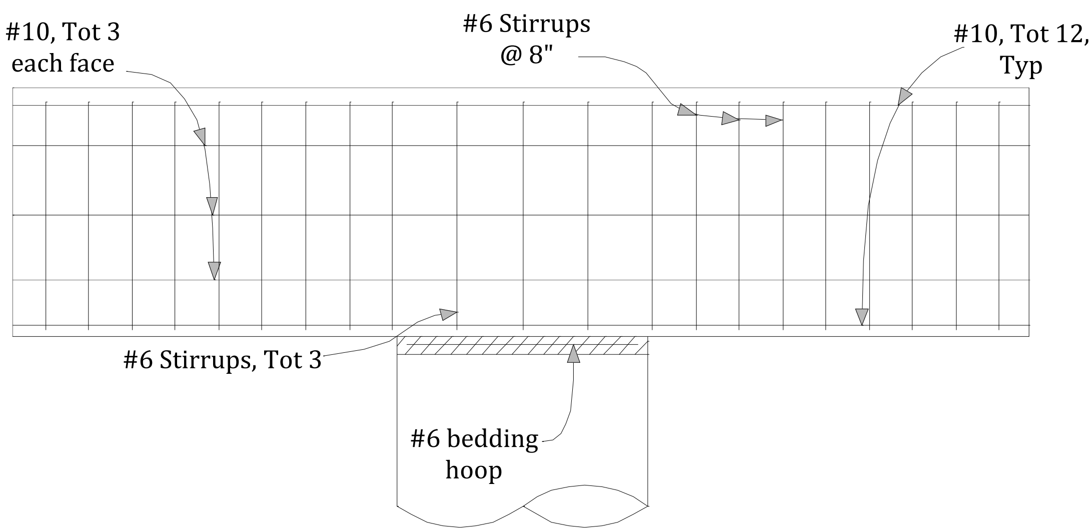
Note: Three patterns are required and used. The minimum number of stirrups is two, per Article 8.13.4.2.2a, and the bar size is no smaller than what is used in the bent cap shear stirrups. If only two stirrups were required, three may be used to reduce the spacing between stirrups to satisfy temperature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

Bedding Layer Reinforcement

SGS 8.13.4.2.1

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.

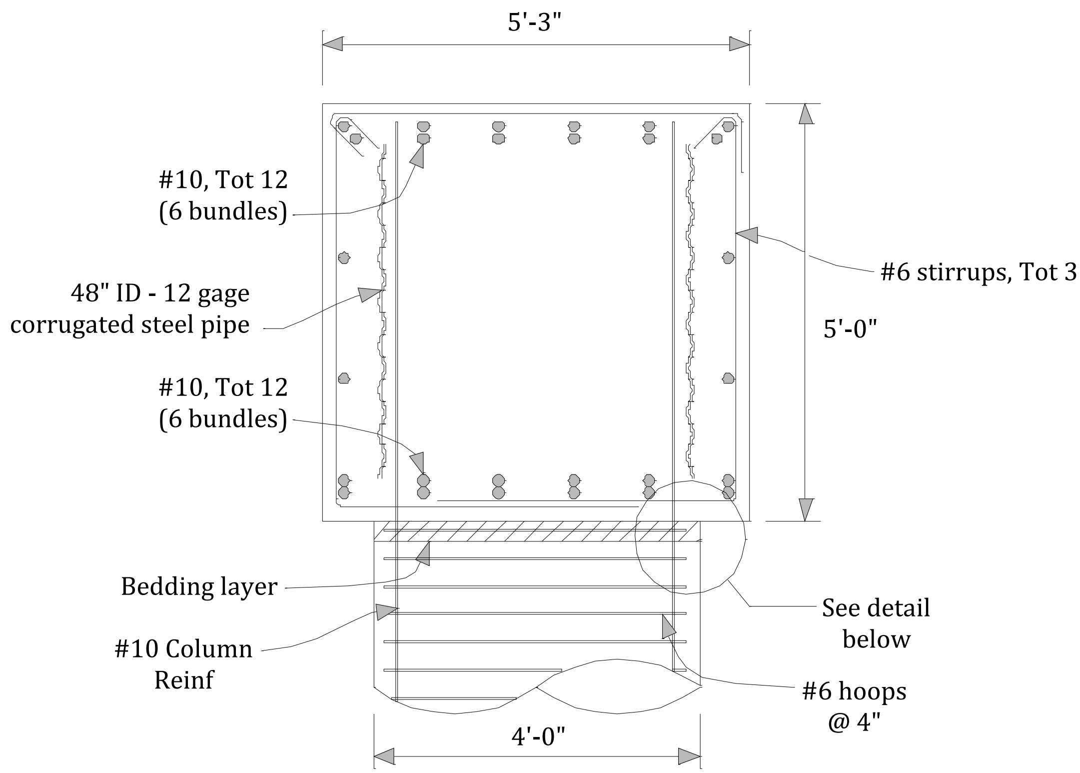
Figures Showing Final Design



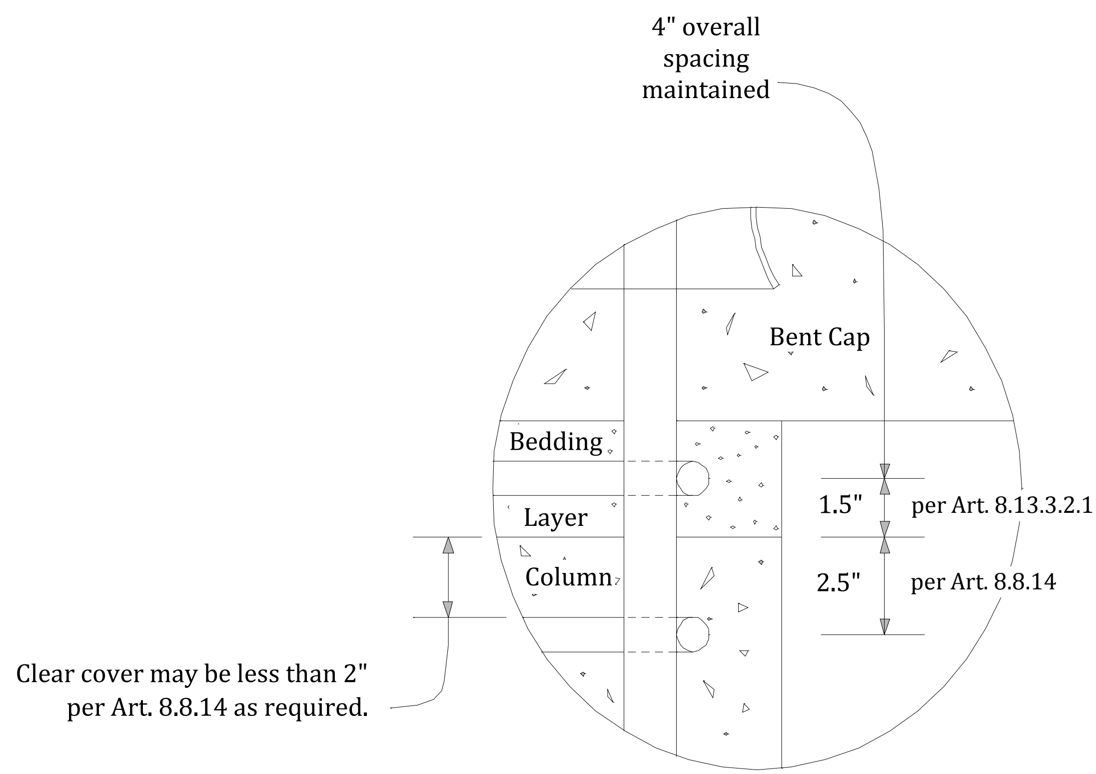
Elevation View at Column Connection

PROJECT	DESIGN EXAMPLES	SHEET NO	5	OF	6
PROJECT NO	NCHRP 12-74	DESIGNED BY	M. STILLER	DATE	11/6/2009
CLIENT	SACRAMENTO STATE UNIV.	CHECKED BY	E. MATSUMOTO	DATE	11/9/2009

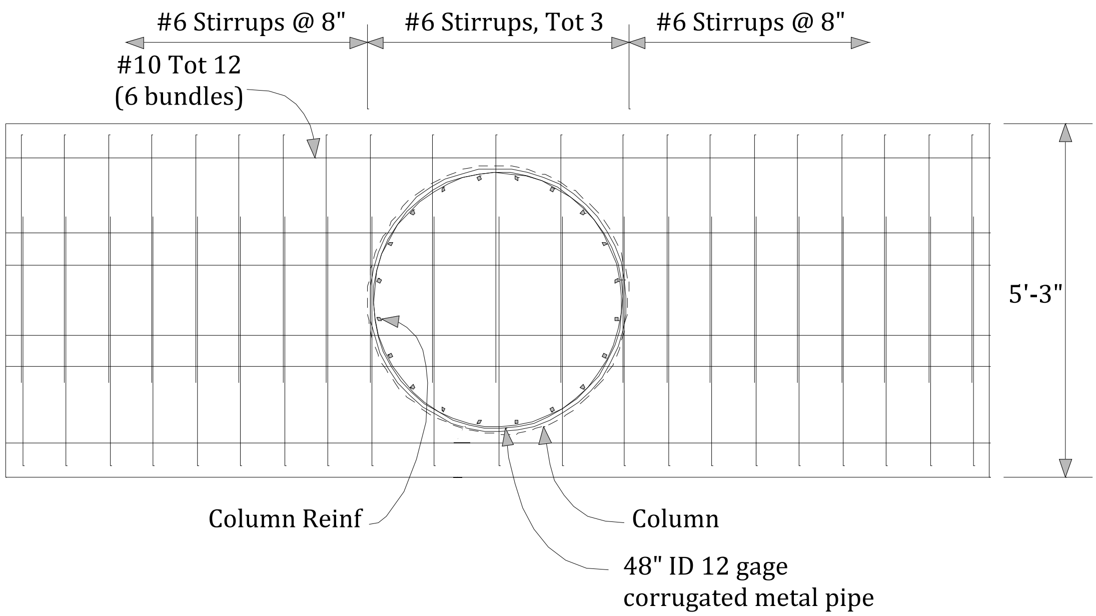
SUBJECT AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL CAP POCKET BENT CAP JOINT DESIGN EXAMPLE - SDC A



Typical Section through



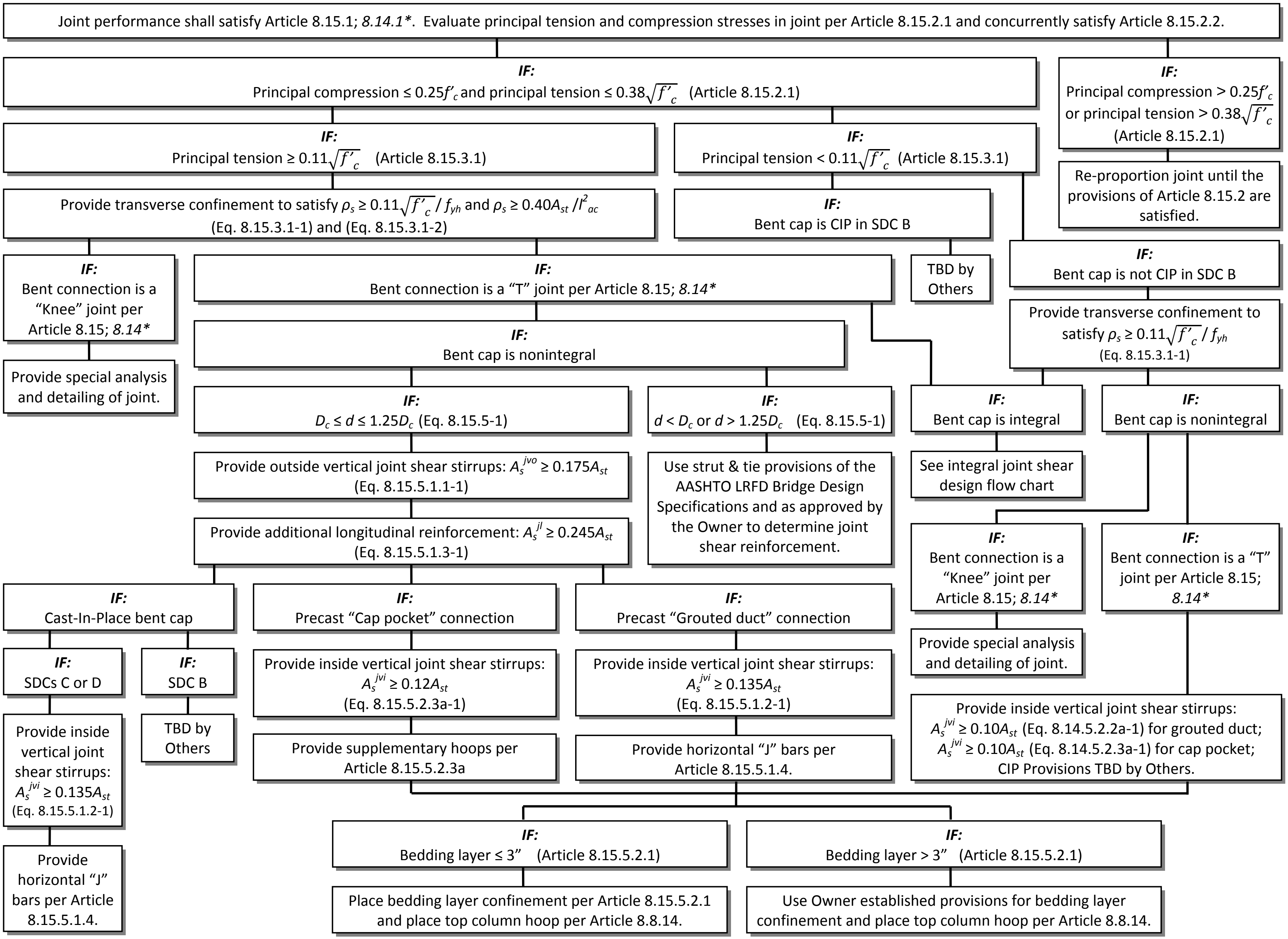
Simplified Section at Column Edge



Plan View at Column Connection

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.

DESIGN FLOW CHART FOR CAST-IN-PLACE AND PRECAST BENT CAP JOINT DESIGN PER PROPOSED ARTICLE 8.15 (SDCs C AND D) AND ARTICLE 8.14 (SDC B)*



*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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Construction Specifications (BCS)

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 $0.11\sqrt{f'c}$. 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)

$f'_{ce} = 1.3f'_c$ = 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)=7.0, 6.0)]

OK per BCS 8.13.8.3.2a

f_y = 60 ksi (yield stress of column bars)

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

f_{yh} = 60 ksi (yield stress of hoops)

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

Bent cap height = 5.00 ft (D_s)

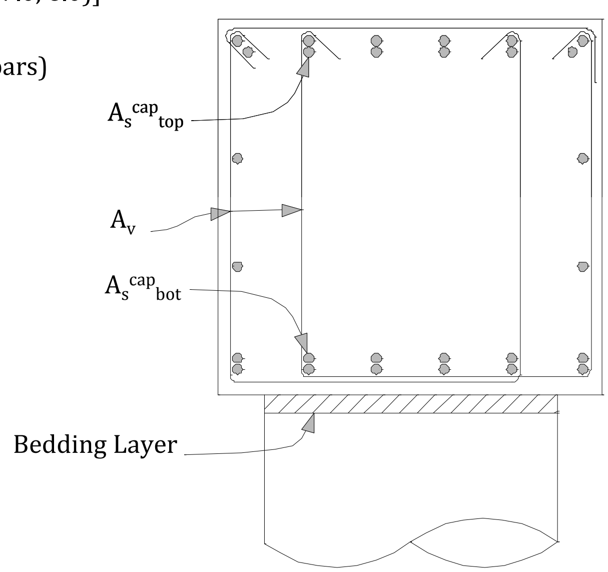
Stirrup bar size: #6

Column diameter = 4.0 ft (D_c)

A_{st} = 25.4 in^2 (#10 Tot 20)

Hoop bar size: #6

Hoop spacing = 4.0 in



Typical cap section within D_c from face of column.

Note: Bent cap reinforcement shown reflects that required by load combinations before application of joint shear design requirements.

Joint Performance

SGS 8.14.1

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)
f'_{ce} (expected concrete compressive strength) = 1.3 x f'_c 5.2 ksi SGS
f_{ye} (expected steel yield stress) = 68 ksi 8.4.4

M_p = 3769 kip-ft SGS 8.5

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

3769 kip-ft ≥ 2411 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.

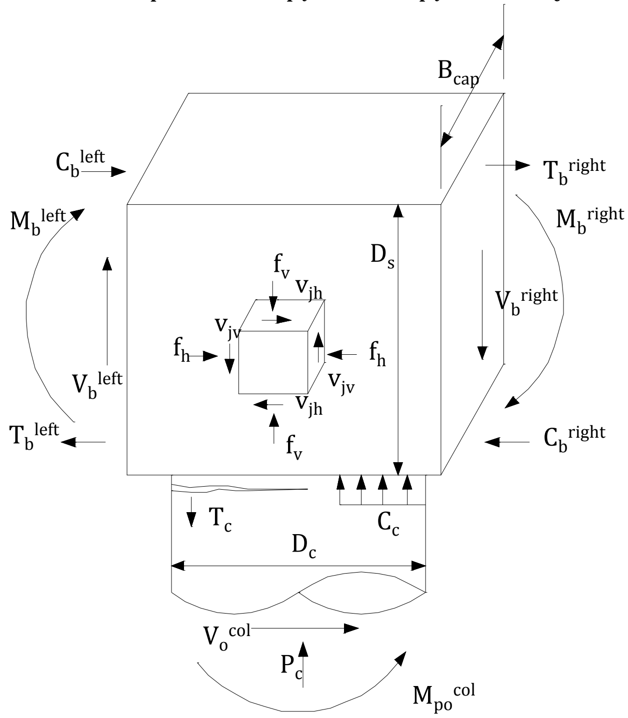
Joint Proportioning

SGS 8.14.2

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 : $p_c \le 0.25\, f'_c$ = 1.00 ksi maximum
Principal tension, p_t : $p_t \le 0.38\, \sqrt{f'_c}$ = 0.76 ksi maximum



B_{cap} =	63.0	in	(7.0 ft x 12"/ft)
D_c =	48.0	in	(5.0 ft x 12"/ft)
D_s =	60.0	in	(6.25 ft x 12"/ft)
l_{ac} =	57.0	in	Note 1
P_c =	820.0	kips	
P_b =	0	kips	Note 2
h =	4.66	ft	Note 3
M_{po} =	2411	kip-ft	Note 3

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.

$$p_t = \left| \left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} \right| = \left| \left(\frac{0.0 + 0.121}{2}\right) - \sqrt{\left(\frac{0.0 - 0.121}{2}\right)^2 + 0.144^2} \right| = 0.096 \text{ ksi}$$ Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} = \left(\frac{0.0 + 0.121}{2}\right) + \sqrt{\left(\frac{0.0 - 0.121}{2}\right)^2 + 0.144^2} = 0.216 \text{ ksi}$$ Eq. 8.15.2.1-4

$v_{jv} = T_c / A_{jv}$ = 517 k / 3591.0 in^2 = 0.144 ksi Eq. 8.15.2.1-5

$A_{jv} = l_{ac}B_{cap}$ = 57.0 in x 63.0 in = 3591.0 in^2 Eq. 8.15.2.1-6

$f_v = P_c / A_{jh}$ = 820 k / 6804 in^2 = 0.121 ksi Eq. 8.15.2.1-7

$A_{jh} = (D_c + D_s)\, B_{cap}$ = (48.0 in + 60.0 in) x 63.0 in = 6804.0 in^2 Eq. 8.15.2.1-8

$f_h = \dfrac{P_b}{B_{cap}D_s}$ = 0.0k / (63.0 in x 60.0 in) = 0.0 ksi Eq. 8.15.2.1-9

$T_c = M_{po} / h$ = 2411 kip-ft / 4.66 ft = 517 kips Eq. 8.15.2.1-10

p_c = 0.216 ksi ≤ 1.00 ksi maximum OK
p_t = 0.096 ksi ≤ 0.76 ksi maximum OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length of Column Longitudinal Reinforcement SGS 8.15.2.2

$$l_{ac} \geq \frac{2d_{bl}\, f_{ye}}{f'_{cg}}$$ Eq. 8.15.2.2.2-1

d_{bl} = 1.27 in (#10 rebar)
f_{ye} = 68 ksi
f'_{cg} = 7.0 ksi = min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2

l_{ac} = 24.7 in (minimum)

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

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

57.0 in ≥ 24.7 in minimum OK Extend to top face of cap less 3 in cover.

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections SGS 8.14.3

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_s^{jvi} and should reference Eq. 8.15.3.1-1 for ρ_s are to be satisfied. However, other joint reinforcing (A_s^{jvo}, A_s^{jl}, 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)

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

p_t = 0.096 ksi ≤ 0.220 ksi limit A_s^{jvi} joint reinf. provisions of Article 8.14.5.2.2a apply.

Calculate required volumetric ratio of transverse joint reinforcement:

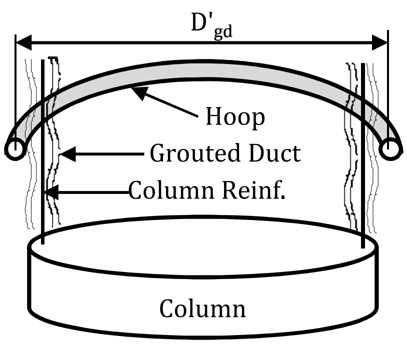
$\rho_s \geq 0.11\sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1

$\rho_s \geq 0.0037$

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 in^2 (#6 hoop)
D'_{gd} = 45.92 in (confined diameter of column between centroids of #6 hoop)
s = 10.0 in (trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or $0.3D_s$ per Art. 8.15.3.2.1)



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

ρ_s = 0.0038

ρ_s = 0.0038 ≥ 0.0037 minimum OK 10" 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 9.37 inches, total 6 hoops.

Nonintegral Bent Cap Joint Shear Design SGS 8.14.5

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 in
d = 60.0 in

48.0 in ≤ 60.0 in ≤ 60.0 in OK Provisions of 8.14.5.2 apply

Vertical Stirrups Inside the Joint Region — SGS 8.14.5.2.2a

$A_s^{jvi} \geq 0.10\ A_{st}$ — Eq. 8.14.5.2.2a-1

$A_{st} =$ 25.40 in^2

$A_s^{jvi} \geq$ 2.54 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.

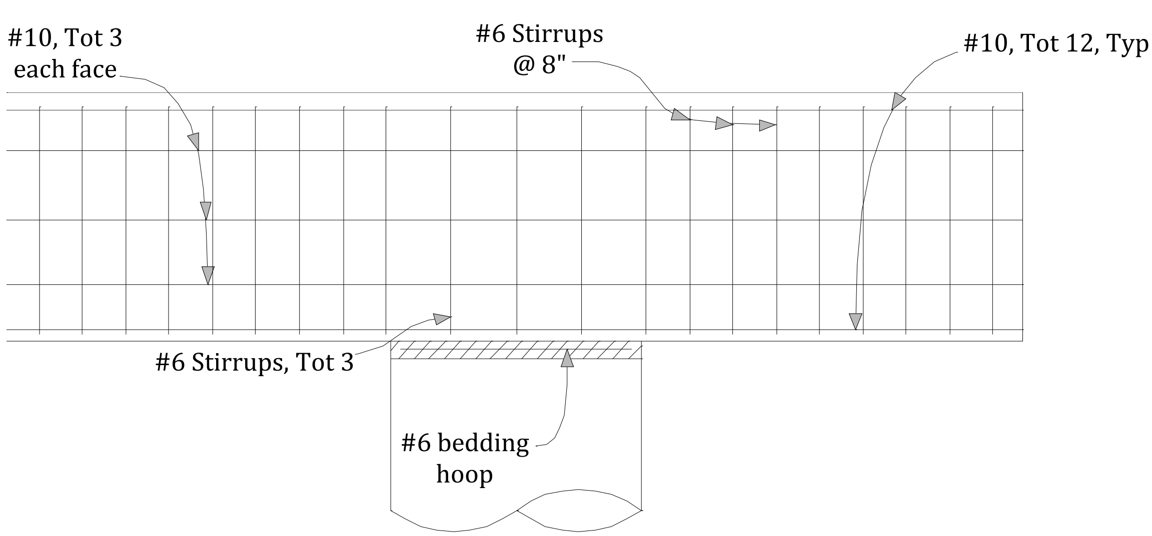
A_s^{jvi} = 3 patterns x 2 legs / pattern x 0.44 in^2/ leg = 2.64 in^2 ≥ 2.54 in^2 OK

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.

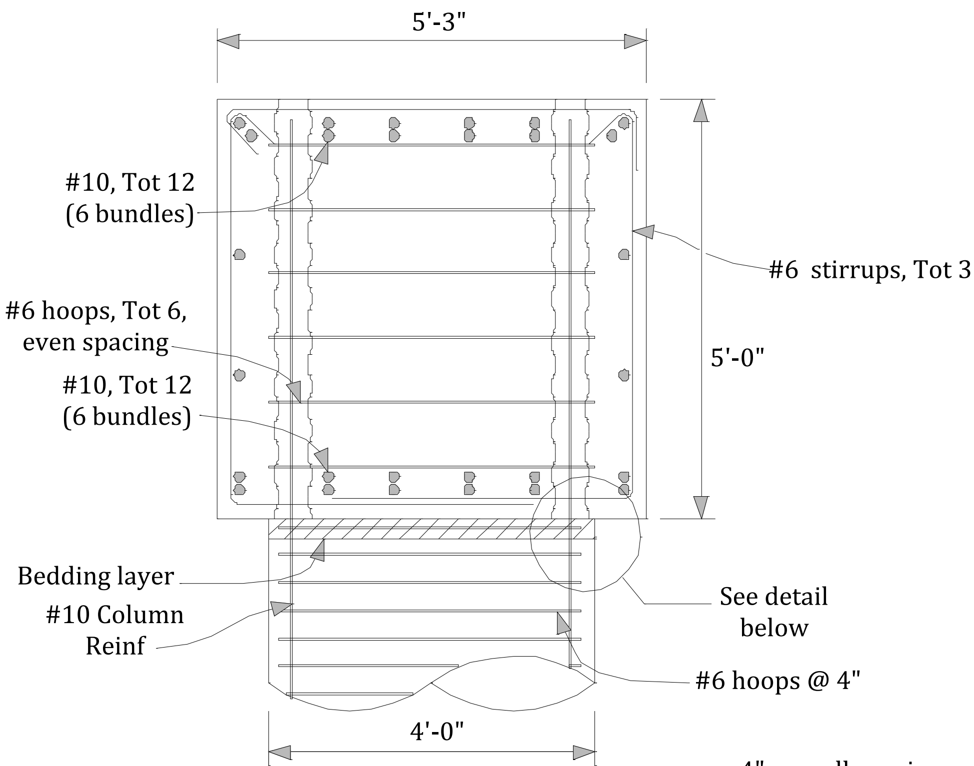
Bedding Layer Reinforcement — SGS 8.14.5.2.1

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.

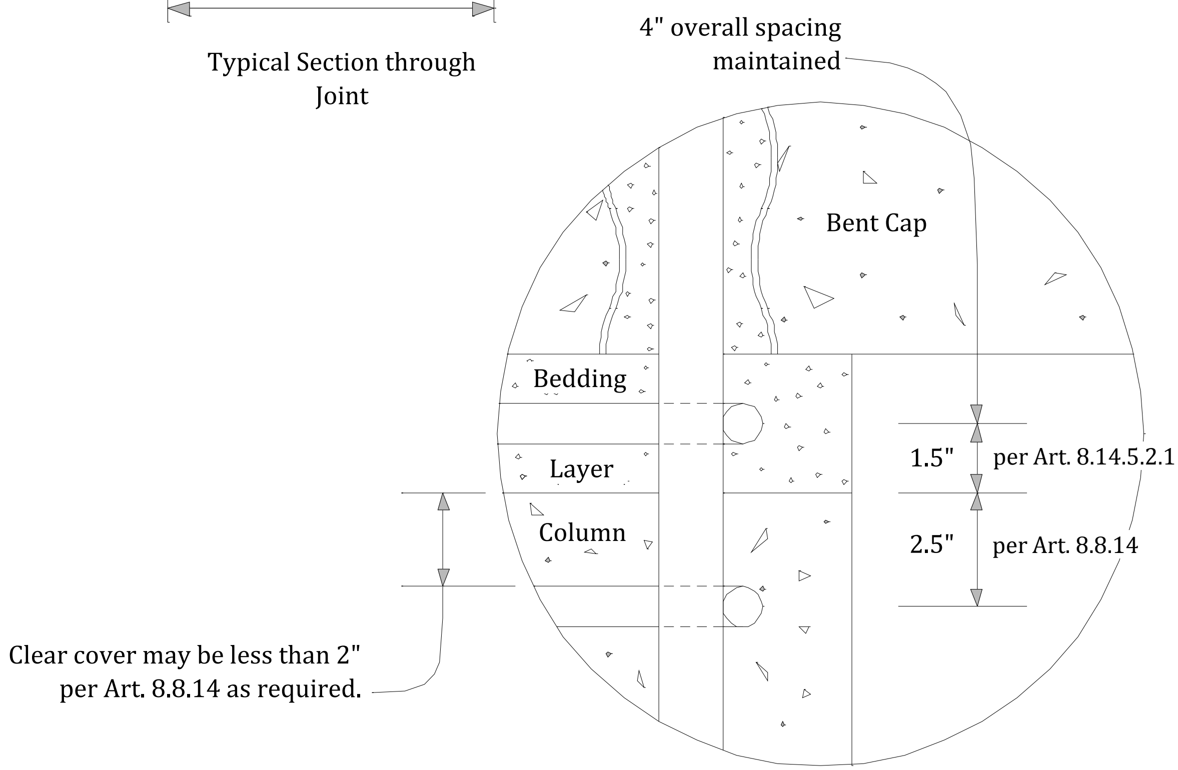
Figures Showing Final Design



Elevation View at Column Connection



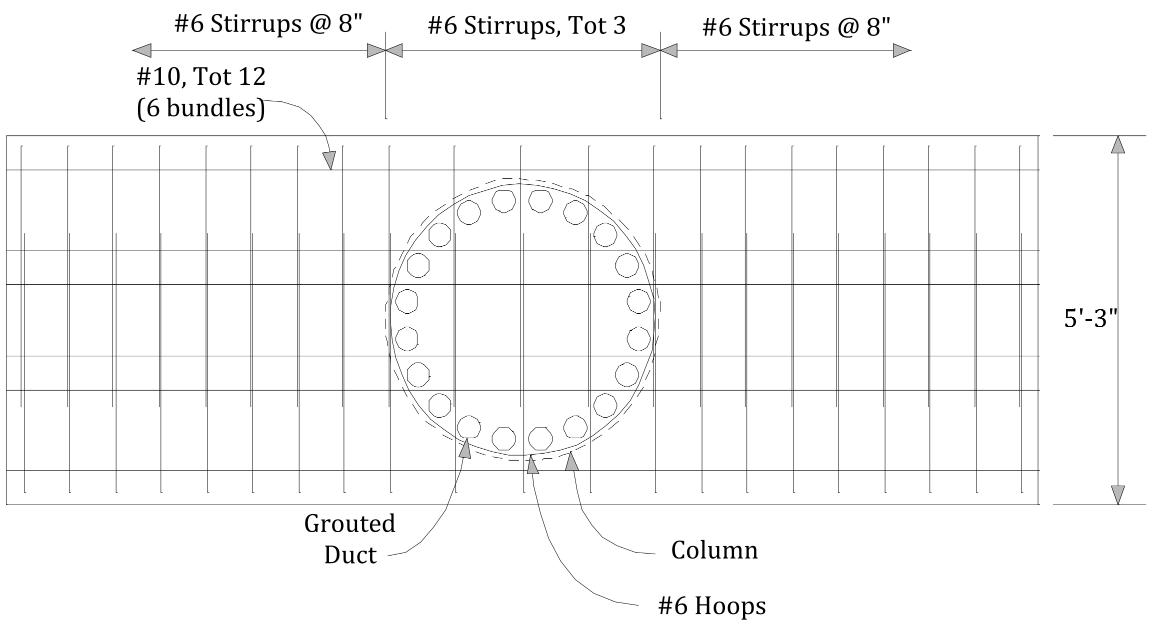
Typical Section through Joint



Simplified Section at Column Edge

PROJECT	DESIGN EXAMPLES	SHEET NO	7	OF	7
PROJECT NO	NCHRP 12-74	DESIGNED BY	M. STILLER	DATE	11/6/2009
CLIENT	SACRAMENTO STATE UNIV.	CHECKED BY	E. MATSUMOTO	DATE	11/9/2009

SUBJECT AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL GROUTED DUCT BENT CAP JOINT DESIGN EXAMPLE - SDC B



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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Construction Specifications (BCS)

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)

$f'_{ce} = 1.3f'_c$ = 5.2 ksi (expected f'_c of bent cap)

f'_{c_pocket}: (specified compressive strength of pocket fill)

Select cap pocket strength to satisfy BCS.

$f'_{c\_pocket} = 1.3\ f'_{ce} + 0.5$ ksi = 5.7 ksi (BCS 8.13.8.3.3a)

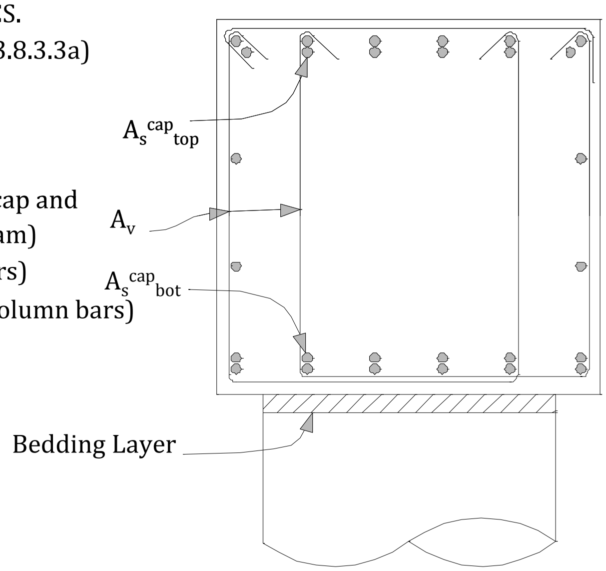
f_{yh} (yield stress of equivalent hoop)

f_{yp} (nominal yield stress of steel pipe)

θ (angle between horizontal axis of cap and pipe helical corrugation or lock seam)

f_y = 60 ksi (yield stress of column bars)

f_{ye} = 68 ksi (expected 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 in^2 (#10 Tot 20)

Hoop bar size: #6

Hoop spacing = 4.0 in

Typical cap section within D_c from face of column.

Note: Bent cap reinforcement shown reflects that required by load combinations before application of joint shear design requirements.

Joint Performance

SGS 8.14.1

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)
f'_{ce} (expected concrete compressive strength) = 1.3 x f'c 5.2 ksi SGS
f_{ye} (expected steel yield stress) = 68 ksi 8.4.4

M_p = 3769 kip-ft SGS
8.5

Maximum moment in column under seismic load application, M_u = 2411 kip-ft

3769 kip-ft ≥ 2411 kip-ft

Design moment for joint proportioning = 2411 kip-ft

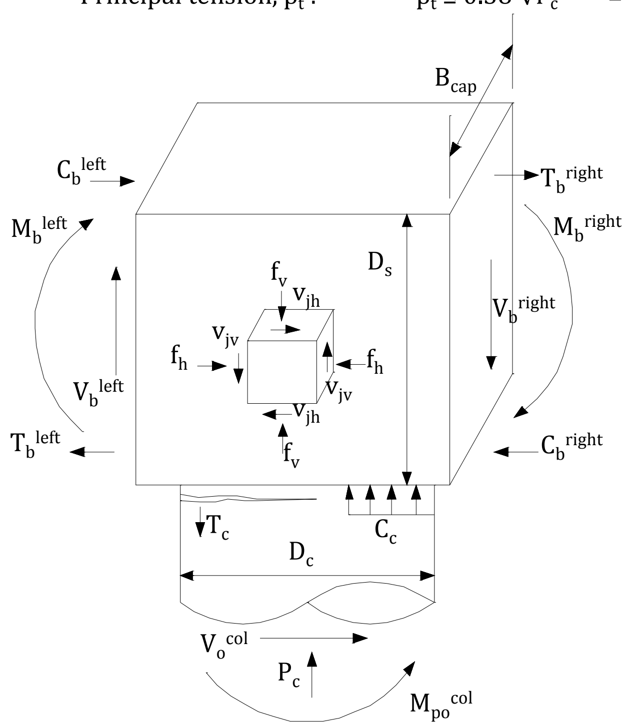
Note: This case shows the ultimate seismic moment demand, M_u, to be less than M_p; therefore, M_u is used for design. If M_u had been greater than M_p, the plastic overstrength moment capacity, M_{po}, would be determined per Eq. 8.5-1 and used for design.

Joint Proportioning SGS
8.14.2

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 : $p_c \leq 0.25\ f'_c$ = 1.00 ksi maximum
Principal tension, p_t : $p_t \leq 0.38\ \sqrt{f'_c}$ = 0.76 ksi maximum



B_{cap} =	63.0	in	(7.0 ft x 12"/ft)
D_c =	48.0	in	(5.0 ft x 12"/ft)
D_s =	60.0	in	(6.25 ft x 12"/ft)
l_{ac} =	57.0	in	Note 1
P_c =	820.0	kips	
P_b =	0	kips	Note 2
h =	4.66	ft	Note 3
M_{po} =	2411	kip-ft	Note 3

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.

$$p_t = \left| \left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} \right| = \left| \left(\frac{0.0 + 0.121}{2}\right) - \sqrt{\left(\frac{0.0 - 0.121}{2}\right)^2 + 0.144^2} \right| = 0.096 \text{ ksi}$$ Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} = \left(\frac{0.0 + 0.121}{2}\right) + \sqrt{\left(\frac{0.0 - 0.121}{2}\right)^2 + 0.144^2} = 0.216 \text{ ksi}$$ Eq. 8.15.2.1-4

$v_{jv} = T_c / A_{jv}$ = 517 k / 3591.0 in^2 = 0.144 ksi Eq. 8.15.2.1-5

$A_{jv} = l_{ac}B_{cap}$ = 57.0 in x 63.0 in = 3591.0 in^2 Eq. 8.15.2.1-6

$f_v = P_c / A_{jh}$ = 820 k / 6804 in^2 = 0.121 ksi Eq. 8.15.2.1-7

$A_{jh} = (D_c + D_s)\, B_{cap}$ = (48.0 in + 60.0 in) x 63.0 in = 6804.0 in^2 Eq. 8.15.2.1-8

$f_h = \dfrac{P_b}{B_{cap}D_s}$ = 0.0k / (63.0 in x 60.0 in) = 0.0 ksi Eq. 8.15.2.1-9

$T_c = M_{po} / h$ = 2411 kip-ft / 4.66 ft = 517 kips Eq. 8.15.2.1-10

p_c = 0.216 ksi ≤ 1.00 ksi maximum OK
p_t = 0.096 ksi ≤ 0.76 ksi maximum OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length for Column Longitudinal Reinforcement SGS 8.15.2.2

$$l_{ac} \geq \frac{2.3 d_{bl} f_{ye}}{f'_c}$$ Eq. 8.15.2.2.2-2

d_{bl} = 1.27 in (#10 rebar)
f_{ye} = 68 ksi
f'_c = 5.7 ksi (cap pocket concrete) = min (5.7 ksi, 7.0 ksi) per Article 8.15.2.2.2

l_{ac} = 34.8 in (minimum)

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

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

57.0 in ≥ 34.8 in minimum OK Extend to top face of cap less 3 in cover.

Minimum Joint Reinforcing for Precast Bent Cap Connections SGS 8.14.3

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_s^{jvi}. Eq. 8.15.3.1-1 for joint transverse reinforcement (i.e., pipe thickness) is to be satisfied. However, other joint reinforcing (A_s^{jvo}, A_s^{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 ≤ 0.220 ksi limit A_s^{jvi} joint reinf. provisions of Article 8.14.5.2.3b apply.

Calculate required volumetric ratio of transverse joint reinforcement:

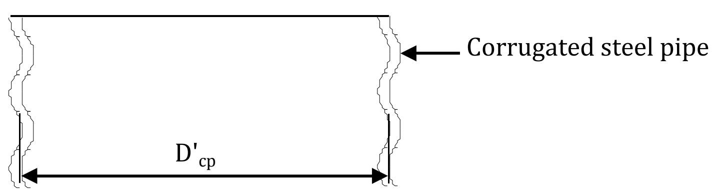
$\rho_s \geq 0.11\sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1

$\rho_s \geq$ 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. SGS 8.15.3.2.2

$$\rho_s = \frac{4A_{sp}}{D'_{cp}\, s} \qquad s = \frac{4A_{sp}}{D'_{cp}\, \rho_s}$$

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



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

s = 9.9 in max spacing
Therefore, the number of equivalent hoops per foot is 12" / s = 1.216 hoops/ft

Calculate the nominal confining hoop force of the equivalent hoops.

$F_H = n_h A_{sp} f_{yh}$ Eq. 8.15.3.2.2-2

n_h = 1.216 ea (number of equivalent hoops per unit length)
A_{sp} = 0.44 in^2 (area of #6 equivalent hoop)
f_{yh} = 60.0 ksi (yield stress of equivalent hoop)

F_H = 32.11 kips/ft

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

$$t_{pipe} \geq \max \begin{cases} \dfrac{F_H}{H_p\, f_{yp} \cos\theta} \\ 0.060 \text{ in} \end{cases}$$ Eq. 8.15.3.2.2-1

F_H = 32.1 kips/ft
H_p = 12.0 in/ft (specified unit length)
f_{yp} = 30.0 ksi (manufacturer specified)
θ = 20.0 deg (manufacturer specified)

$t_{pipe} \geq$ 0.0949 in

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

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

$$t_{pipe} \geq 0.04 \frac{D'_{cp}\sqrt{f'_c}}{f_{yp}\cos\theta} = 0.1381 \text{ in} \quad \text{and} \geq 0.06 \text{ in}$$ (Note: f'_c refers to cap pocket) 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

SGS 8.14.5

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 in
d = 60.0 in

48.0 in ≤ 60.0 in ≤ 60.0 in OK Provisions of 8.14.5.2 apply

Vertical Stirrups Inside the Joint Region

SGS

$A_s^{jvi} \geq 0.10\, A_{st}$ Eq. 8.14.5.2.3.a-1

A_{st} = 25.40 in^2

$A_s^{jvi} \geq$ 2.54 in^2

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

A_s^{jvi} = 3 patterns x 2 legs/ pattern x 0.44 in^2/ leg = 2.64 in^2 ≥ 2.54 in^2 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

SGS 8.14.5.2.1

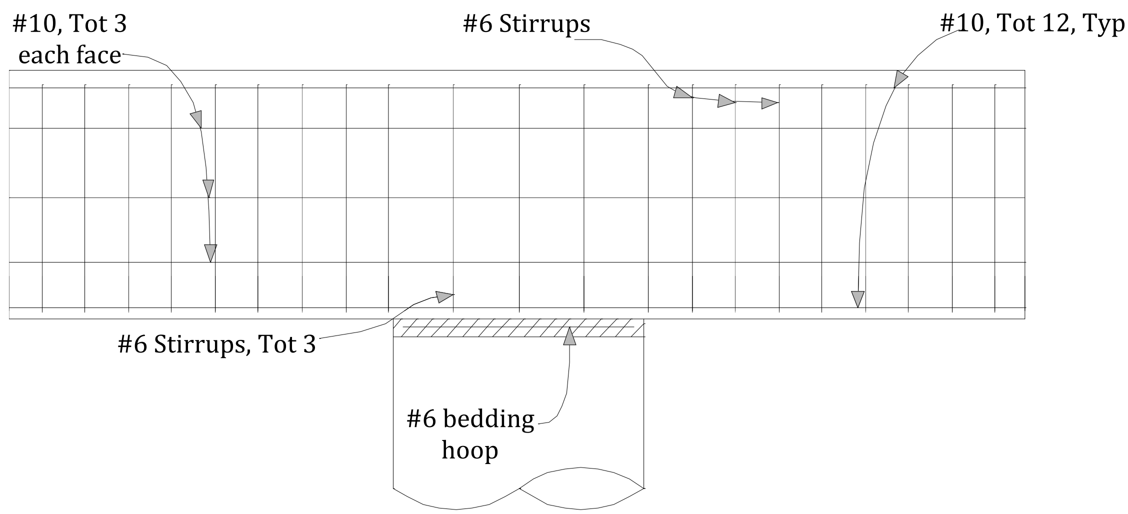
Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below and resulting center to center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

Supplementary Hoops

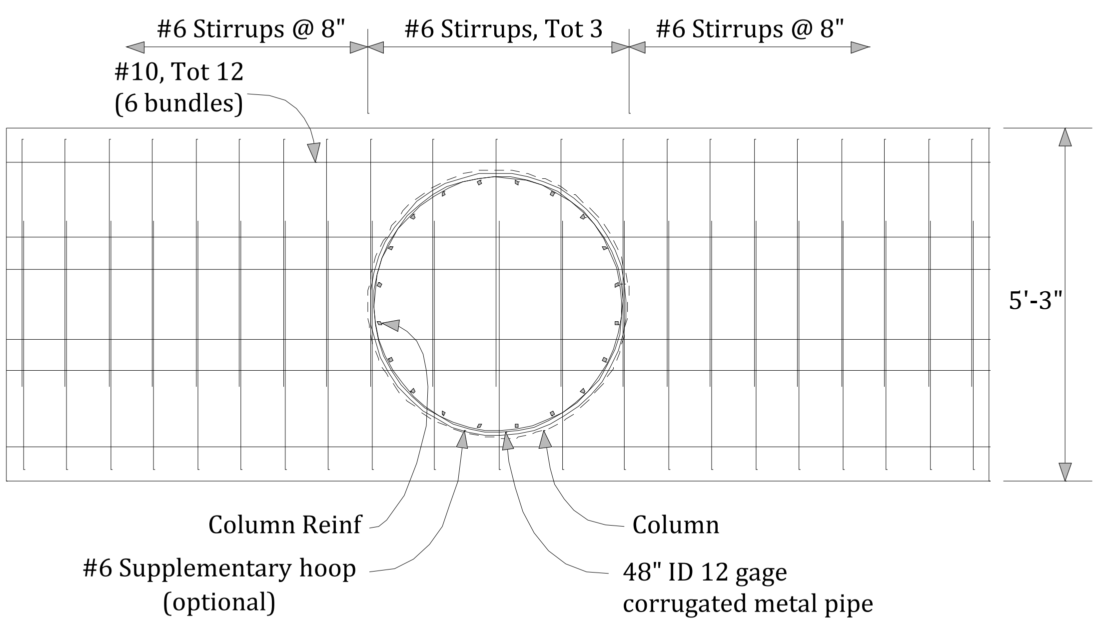
SGS 8.14.5.2.3.b

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

Figures Showing Final Design



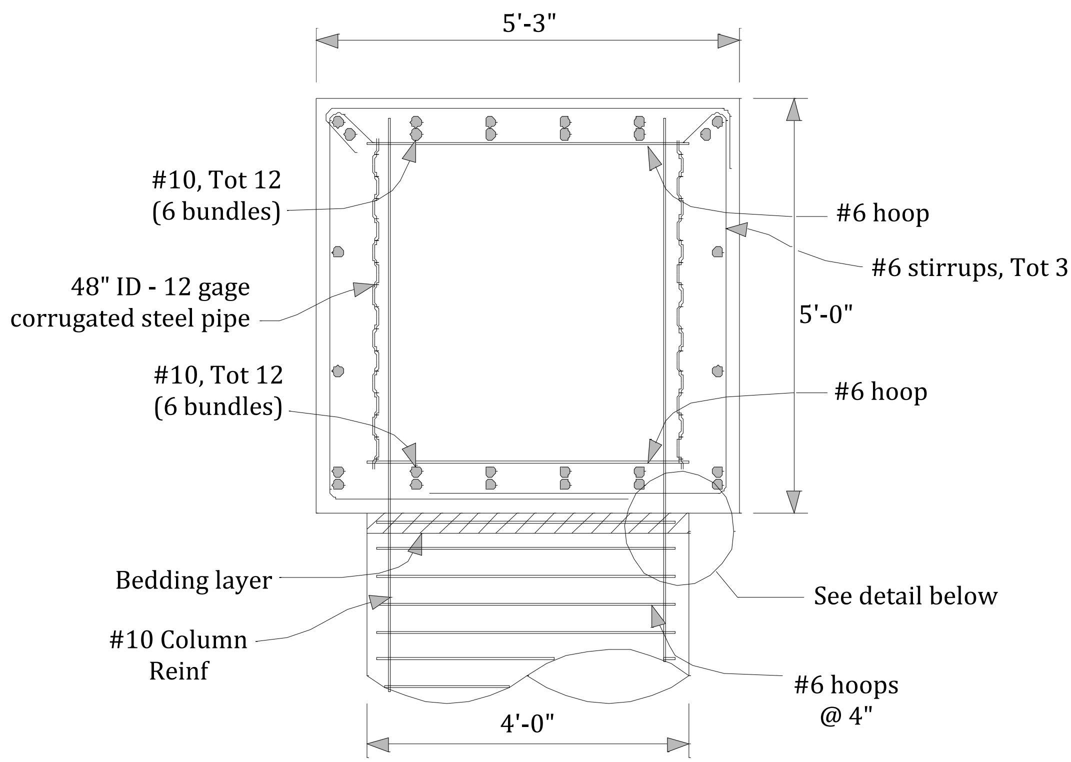
Elevation View at Column Connection



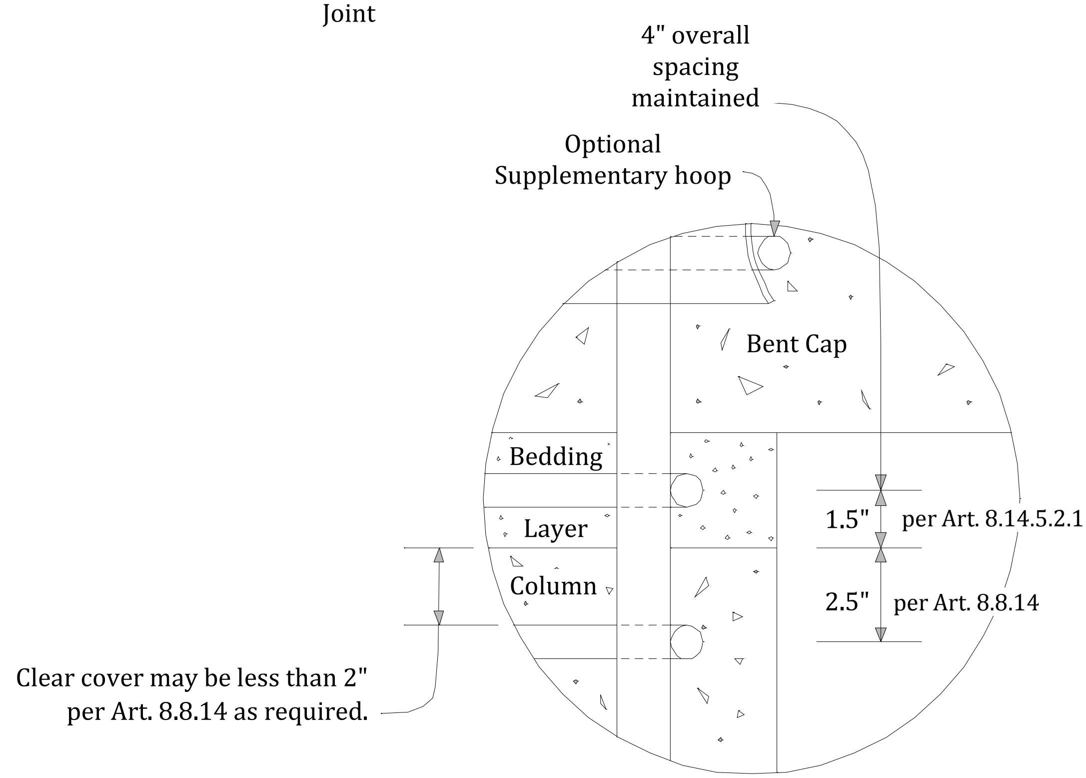
Plan View at Column Connection

PROJECT	DESIGN EXAMPLES	SHEET NO	7	OF	7
PROJECT NO	NCHRP 12-74	DESIGNED BY	M. STILLER	DATE	11/6/2009
CLIENT	SACRAMENTO STATE UNIV.	CHECKED BY	E. MATSUMOTO	DATE	11/9/2009

SUBJECT AASHTO SEISMIC GUIDE SPECIFICATIONS NONINTEGRAL CAP POCKET BENT CAP JOINT DESIGN EXAMPLE - SDC B



Typical Section through Joint



Simplified section at column edge

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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

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 $0.11\sqrt{f'c}$. 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.

AASHTO LRFD Bridge Construction Specifications (BCS)

Notes: 1. The SDC B design example illustrates the procedure to be followed when the $0.11\sqrt{f'c}$ 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_y = 60 ksi f_{ye} = 68 ksi (column bars)

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, $A_s^{cap}{}_{top}$ = 16.77 in^2

Bot cap reinf, $A_s^{cap}{}_{bot}$ = 11.40 in^2

Shear reinf, A_v = 4.12 in^2/ft

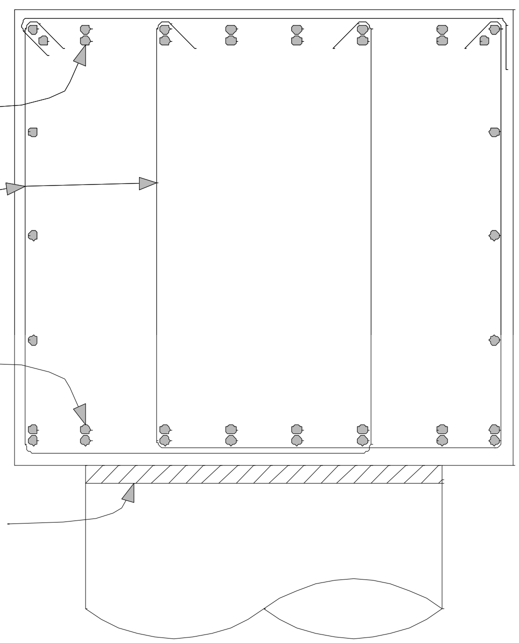
Column diameter = 5.0 ft (D_c)

A_{st} = 31.2 in^2 (#11, Tot 20)

Hoop bar size: #6

Hoop spacing = 4.0 in

f_{yh} = 60 ksi (yield stress of hoops)



Typical cap section within D_c from face of column.
Note: Bent cap reinforcement shown reflects that required by Extreme Event I load combination before application of joint shear design requirements.

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.

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 x f'_c 5.2 ksi

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

SGS 8.4.4

$M_p =$ 5970 kip-ft SGS

$M_{po} = \lambda_{mo} M_p$ 8.5

$\lambda_{mo} =$ 1.2 (ASTM A706) Eq. 8.5-1

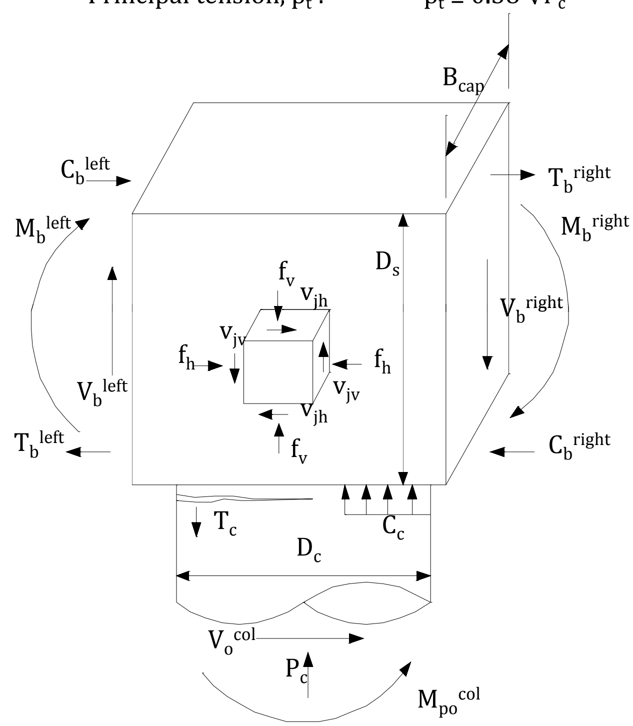
$M_{po} =$ 7164 kip-ft

Joint Proportioning SGS 8.15.2

Principal Stresses

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

Principal compression, p_c :	$p_c \le 0.25\, f'_c$	=	1.00	ksi maximum
Principal tension, p_t :	$p_t \le 0.38 \sqrt{f'_c}$	=	0.76	ksi maximum



B_{cap} =	84.0	in	(7.0 ft x 12"/ft)
D_c =	60.0	in	(5.0 ft x 12"/ft)
D_s =	75.0	in	(6.25 ft x 12"/ft)
l_{ac} =	72.0	in	Note 1
P_c =	820	kips	
P_b =	0	kips	Note 2
h =	3.79	ft	Note 3
M_{po} =	7164	kip-ft	Note 3

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.

$$p_t = \left| \left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} \right| = \left| \left(\frac{0.0 + 0.072}{2}\right) - \sqrt{\left(\frac{0.0 - 0.072}{2}\right)^2 + 0.313^2} \right| = 0.278 \text{ ksi}$$ Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} = \left(\frac{0.0 + 0.072}{2}\right) + \sqrt{\left(\frac{0.0 - 0.072}{2}\right)^2 + 0.313^2} = 0.351 \text{ ksi}$$ Eq. 8.15.2.1-4

$v_{jv} = T_c / A_{jv}$ = 1890 k / 6048.0 in^2 = 0.313 ksi Eq. 8.15.2.1-5

$A_{jv} = l_{ac} B_{cap}$ = 72.0 in x 84.0 in = 6048.0 in^2 Eq. 8.15.2.1-6

$f_v = P_c / A_{jh}$ = 820 k / 11340 in^2 = 0.0723 ksi Eq. 8.15.2.1-7

$A_{jh} = (D_c + D_s)\ B_{cap}$ = (60.0 in + 75.0 in) x 84.0 in = 11340 in^2 Eq. 8.15.2.1-8

$f_h = \dfrac{P_b}{B_{cap}D_s}$ = 0.0k / (84.0 in x 75.0 in) = 0.0 ksi Eq. 8.15.2.1-9

$T_c = M_{po} / h$ = 7164 kip-ft / 3.79 ft = 1890 kips Eq. 8.15.2.1-10

p_c = 0.351 ksi ≤ 1.00 ksi maximum OK
p_t = 0.278 ksi ≤ 0.76 ksi maximum OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length of Column Longitudinal Reinforcement SGS 8.15.2.2

$$l_{ac} \geq \frac{2\ d_{bl}\ f_{ye}}{f'_{cg}}$$ Eq. 8.15.2.2.2-1

d_{bl} = 1.41 in (#11 rebar)
f_{ye} = 68 ksi
f'_{cg} = 7.0 ksi = min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2

l_{ac} = 27.4 in (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 ft x 12"/ft - 3" = 72.0 in

72.0 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 SGS 8.15.3

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_t, is ≥ $0.11\sqrt{f'_c}$ (condition for likely joint cracking)

Calculated tension = 0.278 ksi
Limit, $0.11\sqrt{f'_c}$ = 0.220 ksi

p_t = 0.278 ksi ≥ 0.220 ksi limit

Additional joint reinforcement (A_s^{jvo}, A_s^{jvi}, A_s^{jl}, and horizontal J bars) is required.

Calculate required volumetric ratio of transverse joint reinforcement:

Maximum of: $\rho_s \geq 0.11\ \sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1
$\rho_s \geq 0.40\ A_{st} / l^2_{ac}$ Eq. 8.15.3.1-2

A_{st} = 31.2 in^2
l_{ac} = 72.0 in

Maximum of: $0.11\sqrt{f'_c} / f_{yh} =$ 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

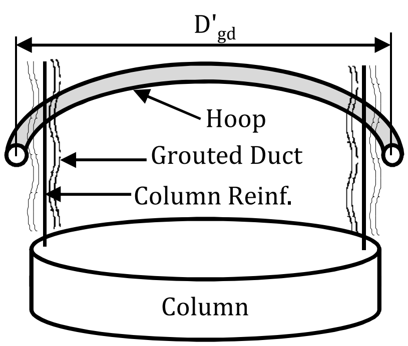
Assume transverse reinforcement in the joint is the same size as the column plastic hinging region transverse reinforcement.

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

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

$D'_{gd} =$ 57.92 in (confined diameter of column between centroids of #6 hoop)

$s =$ 8.0 in (trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or $0.3D_s$ per Art. 8.15.3.2.1)



$D'_{gd} =$ 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" x 2 - 0.88" - 1.44" + 4" + 0.12" x 2 = 57.92 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

SGS 8.15.5

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.

$D_c \leq d \leq 1.25D_c$

$D_c =$ 60.0 in

$d =$ 75.0 in

60.0 in $\leq$ 75.0 in $\leq$ 75.0 in OK Provisions of 8.15.5.2 apply

Additional Joint Shear Reinforcement

SGS 8.15.5.2.2

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:

$A_s^{jvo} \geq 0.175\ A_{st}$ Eq. 8.15.5.1.1-1

$A_{st} =$ 31.20 in^2

$A_s^{jvo} \geq$ 5.46 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_v 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 in^2 / $D_c =$ 1.092 in^2 /ft

$A_v^{total} = A_v + A_s^{jvo}$
A_v^{total} = 4.12 in^2/ft + 1.09 in^2/ft = 5.21 in^2 /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^{total}/1.76 in^2 = 2.96 stirrups / ft
Use 3 stirrups per foot, 4" spacing. A_v^{total} = 5.28 in^2/ft ≥ 5.21 in^2/ft OK

Vertical Stirrups Inside the Joint Region:

$A_s^{jvi} \geq 0.135\ A_{st}$ Eq. 8.15.5.1.2-1

$A_{st} =$ = 31.2 in^2

$A_s^{jvi} \geq$ 4.21 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 patterns x 4 legs / pattern x 0.44 in^2/ leg = 7.04 in^2 ≥ 4.21 in^2 OK

4 patterns instead of 3 are conservatively used for symmetry.

Additional Longitudinal Cap Beam Reinforcement

$A_s^{jl} \geq 0.245\ A_{st}$ Eq. 8.15.5.1.3-1

$A_{st} =$ 31.2 in^2

$A_s^{jl} \geq$ 7.64 in^2 (individual amount applied to top and bottom faces of cap)

Top cap reinf, $A_s^{cap}{}_{top}$ = 16.77 in^2
Bot cap reinf, $A_s^{cap}{}_{bot}$ = 11.40 in^2 } Per design requirements of Extreme I load case.

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

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

Note: A_s^{jl} is added to the A_s^{cap} of the bent cap required under the seismic extreme event load case only. These $A_s^{total}{}_{top}$ and $A_s^{total}{}_{bot}$ 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.

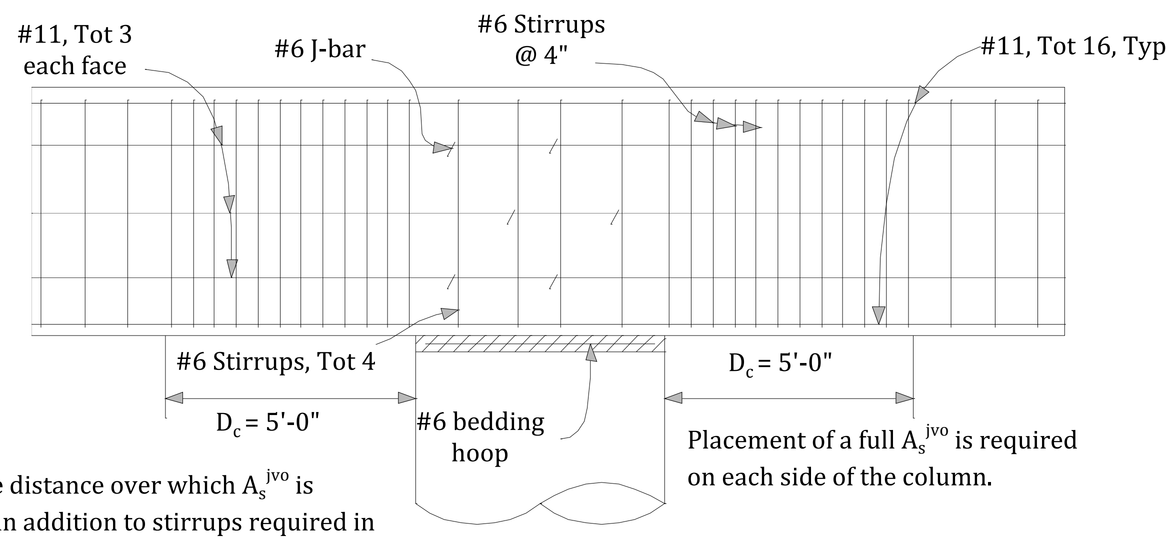
Horizontal J-Bars SGS 8.15.5.1.4

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 SGS 8.15.5.2.1

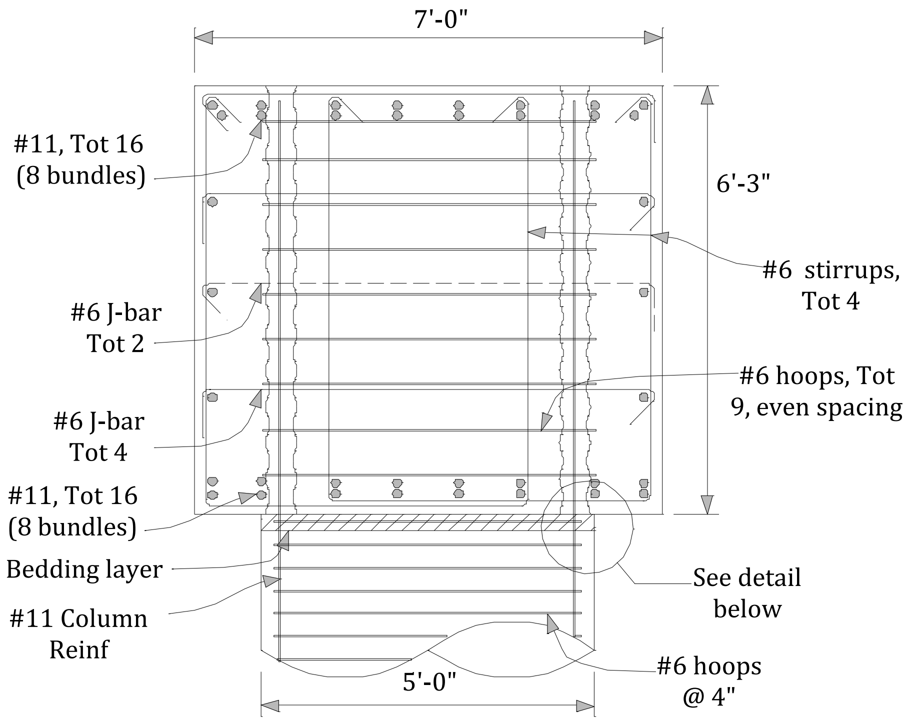
Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below, and resulting center-to-center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

Figures Showing Final Design

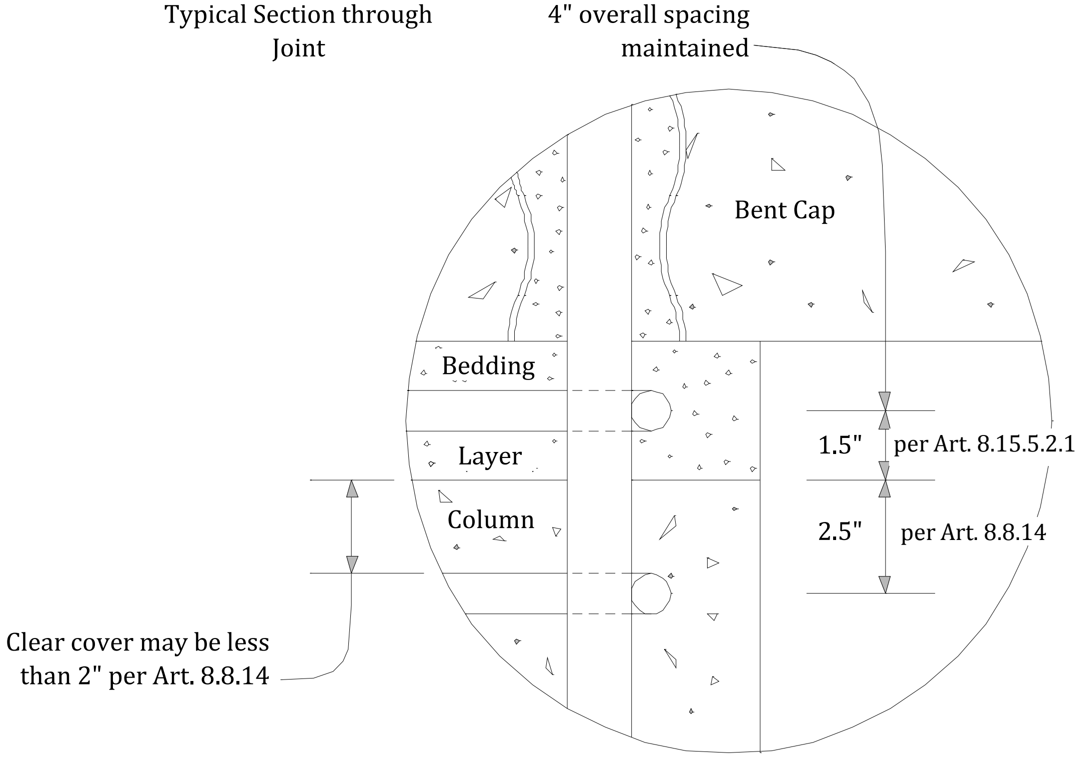


D_c is the distance over which A_s^{jvo} is spread in addition to stirrups required in the same region for other forces.
D_c = column dia.

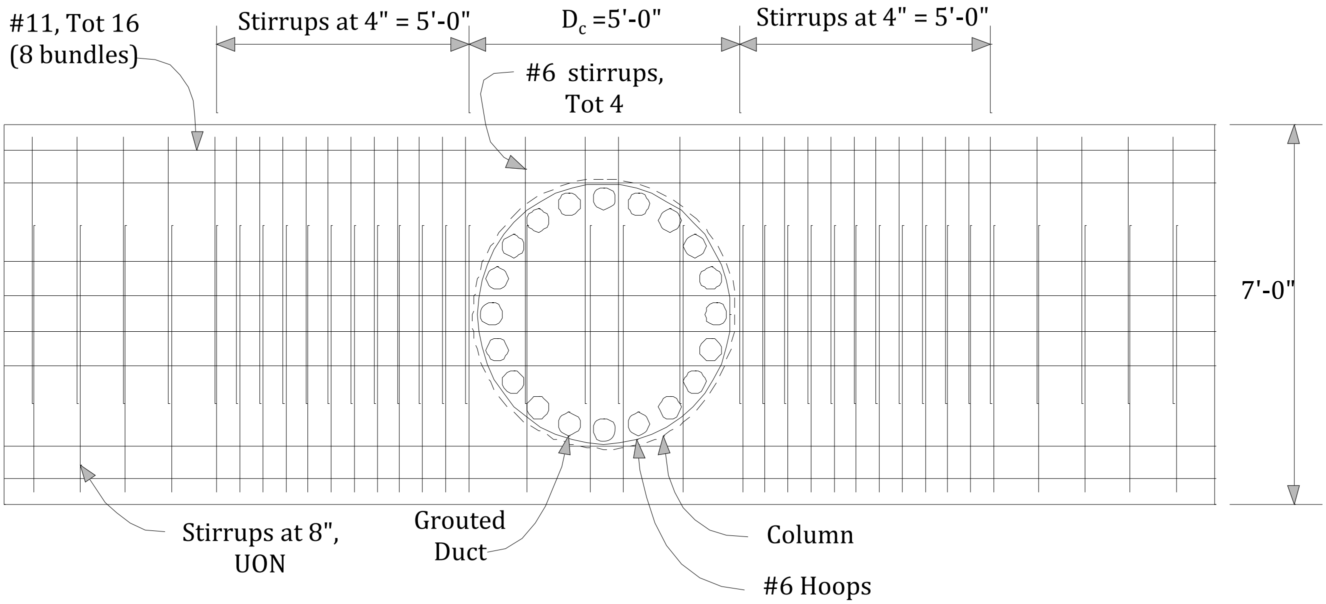
Elevation View at Column Connection



Typical Section through Joint



Simplified Section at Column Edge



Plan View at Column Connection

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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Bridge Construction Specifications (BCS)

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 $0.11\sqrt{f'_c}$. 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 $0.11\sqrt{f'c}$ 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) $f'_{ce} = 1.3f'_c$ = 5.2 ksi (expected f'_c of bent cap)

f'_{c_pocket} (specified compressive strength of pocket fill) Select cap pocket strength to satisfy BCS.

$f'_{c\_pocket} = f'_{ce}$ + 0.5 ksi = 5.7 ksi (BCS 8.13.8.3.3a)

f_{yh} (yield stress of equivalent hoop)

f_{yh} = 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)

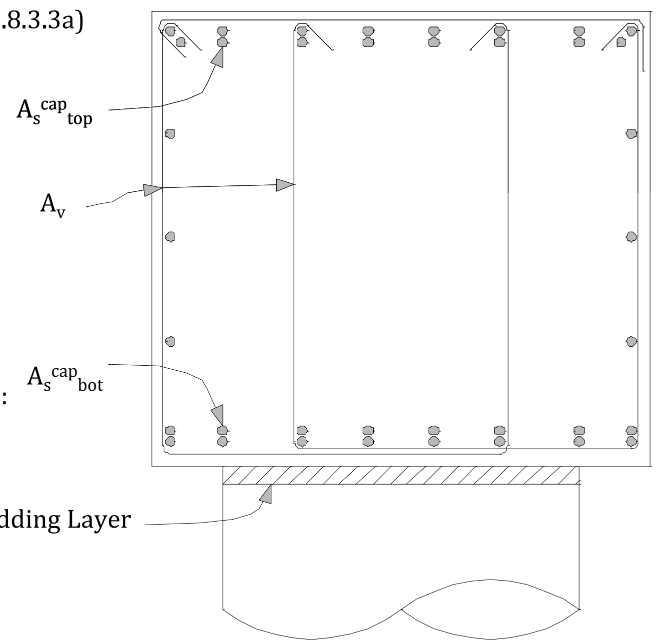
Stirrup bar size: #6

Requirements for Extreme I load case in cap:

Top cap reinf, $A_s{}^{cap}{}_{top}$ = 16.77 in^2

Bot cap reinf, $A_s{}^{cap}{}_{bot}$ = 11.40 in^2

Shear reinf, A_v = 4.12 in^2/ft



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

Typical cap section within D_c from face of column.
Note: Bent cap reinforcement shown reflects that required by Extreme Event I load combination before application of joint shear design requirements.

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.

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 x f'_c 5.2 ksi — SGS

f_{ye} (expected steel yield stress) = 68 ksi — 8.4.4

M_p = 5970 kip-ft — SGS

$M_{po} = \lambda_{mo} M_p$ — 8.5

λ_{mo} = 1.2 (ASTM A706) — Eq. 8.5-1

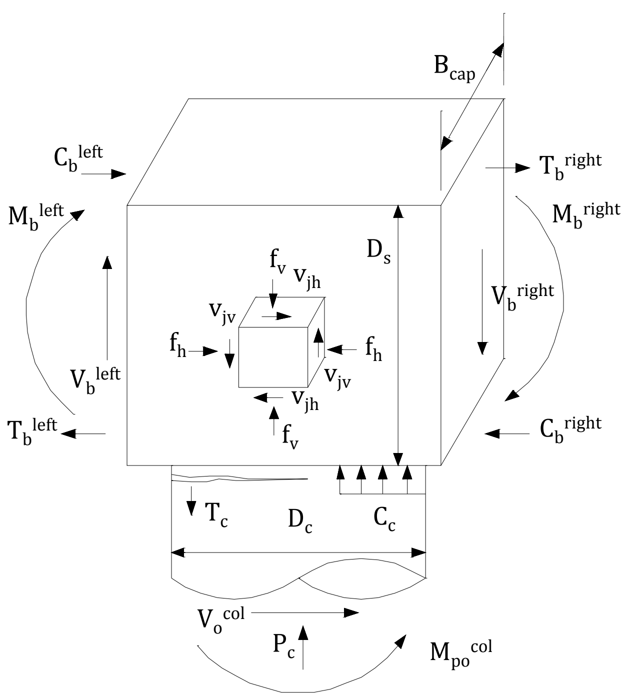
M_{po} = 7164 kip-ft

Joint Proportioning — SGS 8.15.2

Principal Stresses

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

Principal compression, p_c :	$p_c \le 0.25\, f'_c$	=	1.00	ksi maximum
Principal tension, p_t :	$p_t \le 0.38 \sqrt{f'_c}$	=	0.76	ksi maximum



B_{cap} =	84.0	in	(7.0 ft x 12"/ft)
D_c =	60.0	in	(5.0 ft x 12"/ft)
D_s =	75.0	in	(6.25 ft x 12"/ft)
l_{ac} =	72.0	in	Note 1
P_c =	820	kips	
P_b =	0	kips	Note 2
h =	3.79	ft	Note 3
M_{po} =	7164	kip-ft	Note 3

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.

$$p_t = \left|\left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}\right| = \left|\left(\frac{0.0 + 0.072}{2}\right) - \sqrt{\left(\frac{0.0 - 0.072}{2}\right)^2 + 0.313^2}\right| = 0.278 \text{ ksi}$$ Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} = \left(\frac{0.0 + 0.072}{2}\right) + \sqrt{\left(\frac{0.0 - 0.072}{2}\right)^2 + 0.313^2} = 0.351 \text{ ksi}$$ Eq. 8.15.2.1-4

$v_{jv} = T_c / A_{jv}$	$= 1890\text{ k} / 6048.0\text{ in}^2$	$= 0.313$ ksi	Eq.	8.15.2.1-5
$A_{jv} = l_{ac}B_{cap}$	$= 72.0\text{ in} \times 84.0\text{ in}$	$= 6048.0\ \text{in}^2$	Eq.	8.15.2.1-6
$f_v = P_c / A_{jh}$	$= 820\text{ k} / 11340\text{ in}^2$	$= 0.0723$ ksi	Eq.	8.15.2.1-7
$A_{jh} = (D_c + D_s)\ B_{cap}$	$= (60.0\text{ in} + 75.0\text{ in}) \times 84.0\text{ in}$	$= 11340\ \text{in}^2$	Eq.	8.15.2.1-8
$f_h = \dfrac{P_b}{B_{cap}D_s}$	$= 0.0\text{k} / (84.0\text{ in} \times 75.0\text{ in})$	$= 0.0$ ksi	Eq.	8.15.2.1-9
$T_c = M_{po} / h$	$= 7164\text{ kip-ft} / 3.79\text{ ft}$	$= 1890$ kips	Eq.	8.15.2.1-10

$p_c = 0.351$ ksi ≤ 1.00 ksi maximum OK
$p_t = 0.278$ ksi ≤ 0.76 ksi maximum OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length of Column Longitudinal Reinforcement SGS 8.15.2.2

$$l_{ac} \geq \frac{2.3 d_{bl} f_{ye}}{f'_{c\_pocket}}$$ Eq. 8.15.2.2.2-2

$d_{bl} = 1.41$ in (#11 rebar)
$f_{ye} = 68$ ksi
$f'_{c\_pocket} = 5.7$ ksi (cap pocket concrete) = min (5.7 ksi, 7.0 ksi) per Article 8.15.2.2.2

$l_{ac} = 38.7$ in (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 ft x 12"/ft - 3" = 72.0 in

72.0 in $\geq$ 38.7 in (minimum) OK Extend to top face of cap less 3 in cover.

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections SGS 8.15.3

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_t, is $\geq 0.11\sqrt{f'_c}$ (condition for likely joint cracking)

Calculated tension = 0.278 ksi
Limit, $0.11\sqrt{f'_c}$ = 0.220 ksi

$p_t = 0.278$ ksi ≥ 0.220 ksi limit

Additional joint reinforcement (A_s^{jvo}, A_s^{jvi}, and A_s^{jl}) is required.

Calculate required volumetric ratio of transverse joint reinforcement:

Maximum of: $\rho_s \geq 0.11\ \sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1

$\rho_s \geq 0.40\ A_{st} / l^2_{ac}$ Eq. 8.15.3.1-2

$A_{st} =$ 31.20 in^2

$l_{ac} =$ 72.0 in

Maximum of: $0.11\ \sqrt{f'_c} / f_{yh} =$ 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

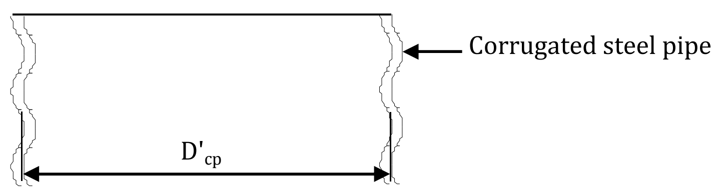
Use the minimum volumetric ratio to calculate the required number of equivalent #6 hoops per unit length of corrugated steel pipe. Use a unit length of 1 foot. Use a 54" ID corrugated pipe based on the outer diameter of column reinforcement pattern. SGS 8.15.3.2.2

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

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

$D'_{cp} =$ 54.75 in (average confined dia. of column between corrugated steel pipe walls)

$\rho_s =$ 0.0037 (minimum volumetric ratio)



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

$s =$ 8.8 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.

$F_H = n_h\ A_{sp}\ f_{yh}$ Eq. 8.15.3.2.2-2

$n_h =$ 1.369 ea (number of equivalent hoops per unit length)

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

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

$F_H =$ 36.14 kips/ft

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

$$t_{pipe} \geq \max \begin{cases} \dfrac{F_H}{H_p\ f_{yp}\ \cos\theta} \\ 0.060 \text{ in} \end{cases}$$ 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)
$\theta =$ 20.0 deg (manufacturer specified)

$t_{pipe} \geq$ 0.1068 in

Use a 12 gage corrugated steel pipe, 54" nominal inside diameter. $t_{pipe} =$ 0.1046 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:

$$t_{pipe} \geq 0.04 \frac{D'_{cp}\sqrt{f'_c}}{f_{yp}\cos\theta} = 0.1554 \text{ in} \quad \text{and} \geq 0.06 \text{ in}$$ (Note: f'_c refers to bent cap concrete) Eq. C8.15.3.2.2-1

$$t_{pipe} \geq 0.14 \frac{A_{st}D'_{cp}f_{yh}}{l_{ac}^2 f_{yp}\cos\theta} = 0.0982 \text{ in} \quad \text{and} \geq 0.06 \text{ in}$$ Eq. C8.15.3.2.2-2

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

SGS 8.15.5

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 in
$d =$ 75.0 in

60.0 in ≤ 75.0 in ≤ 75.0 in OK Provisions of 8.15.5.2 apply

Additional Joint Shear Reinforcement

SGS 8.15.5.2.3

Vertical Stirrups Outside the Joint Region:

$A_s^{jvo} \geq 0.175\, A_{st}$ Eq. 8.15.5.1.1-1

$A_{st} =$ 31.20 in^2

$A_s^{jvo} \geq$ 5.46 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_v 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 in^2 / D_c = 1.092 in^2 /ft

$A_v^{total} = A_v + A_s^{jvo}$

A_v^{total} = 4.12 in^2/ft + 1.09 in^2/ft = 5.21 in^2 /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^{total}/1.76 in^2 = 2.96 stirrups / ft

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

Vertical Stirrups Inside the Joint Region:

$A_s^{jvi} \geq 0.12\ A_{st}$ Eq. 8.15.5.2.3a-1

A_{st} = = 31.2 in^2

$A_s^{jvi} \geq$ 3.74 in^2

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

A_s^{jvi} = 5 patterns x 2 legs/ pattern x 0.44 in^2/ leg = 4.40 in^2 ≥ 3.74 in^2 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

$A_s^{jl} \geq 0.245\ A_{st}$ Eq. 8.15.5.1.3-1

A_{st} = = 31.2 in^2

$A_s^{jl} \geq$ 7.64 in^2 (individual amount applied to top and bottom faces of cap)

Top cap reinf, $A_s^{cap}{}_{top}$ = 16.77 in^2
Bot cap reinf, $A_s^{cap}{}_{bot}$ = 11.40 in^2
Per design requirements of Extreme I load case.

Total top cap reinf, $A_s^{total}{}_{top} = A_s^{cap}{}_{top} + A_s^{jl}$ = 16.77 in^2 + 7.64 in^2 = 24.41 in^2

Total bot cap reinf, $A_s^{total}{}_{bot} = A_s^{cap}{}_{bot} + A_s^{jl}$ = 11.40 in^2 + 7.64 in^2 = 19.04 in^2

Use #11 Tot 16 on top and bottom of bent cap, A_s = 24.96 in^2

Note: A_s^{jl} is added to the A_s^{cap} of the bent cap required under the seismic extreme event load case only. These $A_s^{total}{}_{top}$ and $A_s^{total}{}_{bot}$ 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. SGS 8.15.5.2.3a

Bedding Layer Reinforcement

SGS 8.15.5.2.1

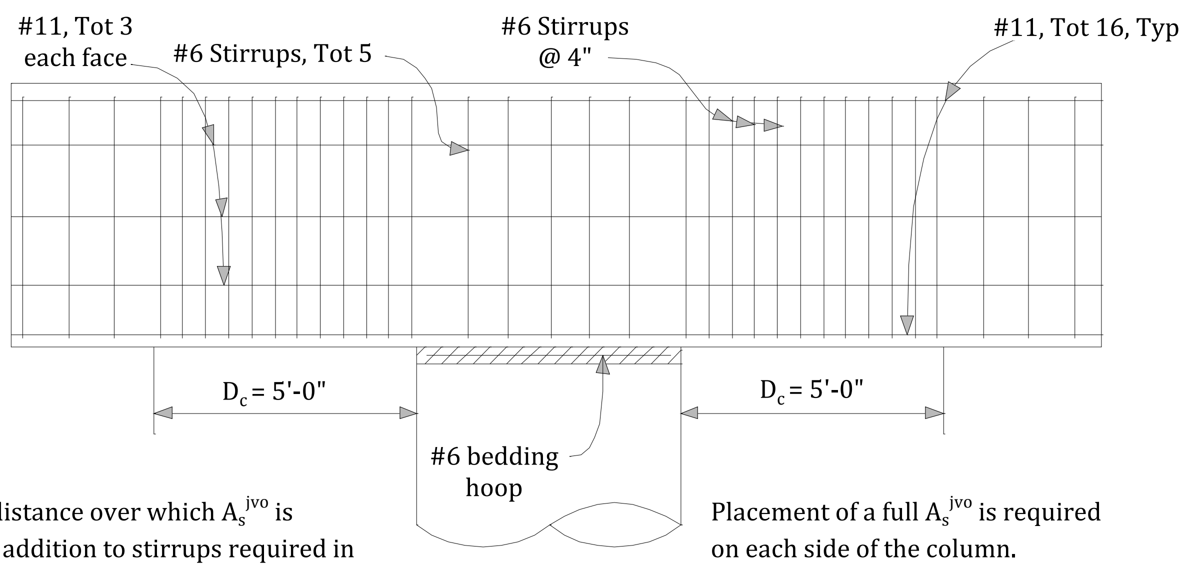
Provide bedding layer transverse reinforcement to match the size and type used in the column plastic hinging region (#6 bar). Hoop will be placed at mid-height in the bedding layer; bedding layer to be 3" thick. Cover on top column hoop to be 2" as shown in figures below, and resulting center-to-center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

Supplementary Hoops

SGS 8.15.5.2.3b

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).

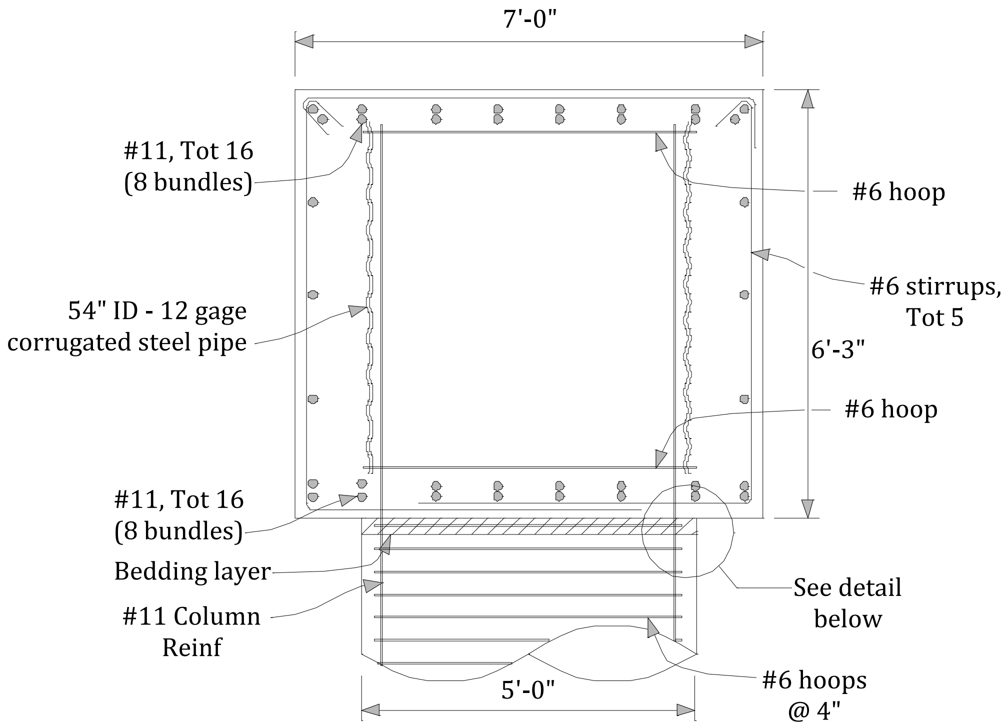
Figures Showing Final Design



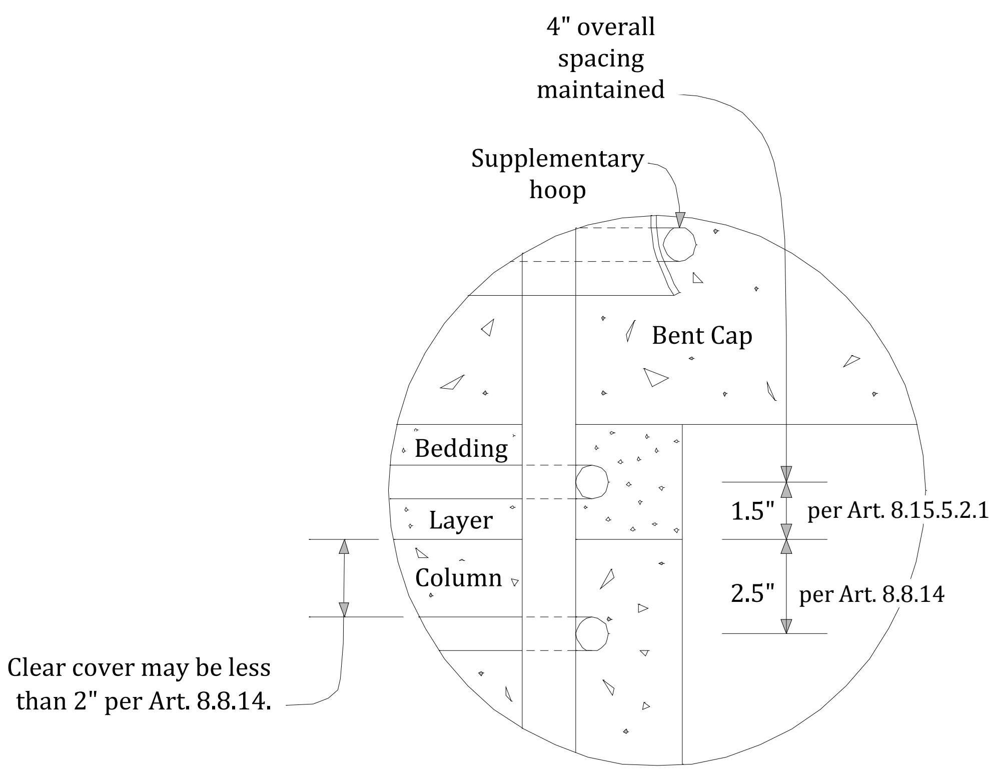
D_c is the distance over which A_s^{jvo} is spread in addition to stirrups required in the same region for other forces.
D_c = column dia.

Placement of a full A_s^{jvo} is required on each side of the column.

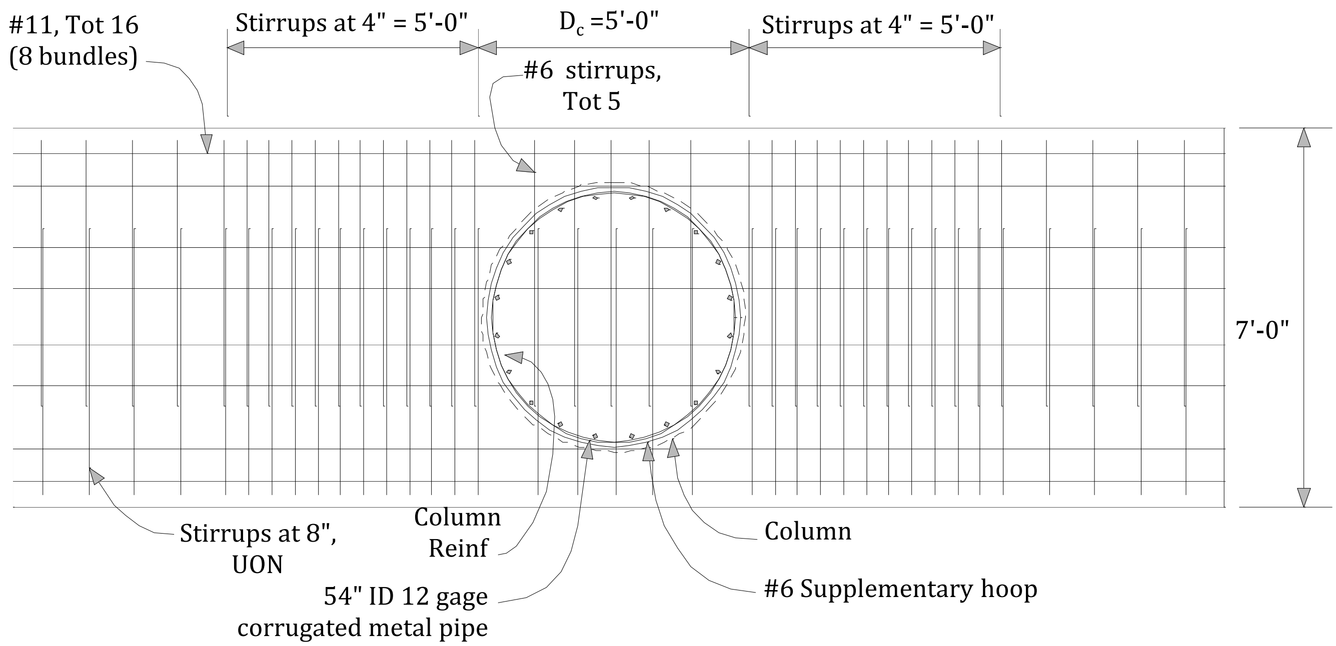
Elevation View at Column Connection



Typical Section through Joint



Simplified section at column edge



Plan View at Column Connection

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

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Bridge Design Specifications (LRFD)

AASHTO LRFD Bridge Construction Specifications (BCS)

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 estimatation of the effective yield point and the ultimate point for lateral response. The procedure presented requires itteration of the column neutral axis depth at both limit states that can be easily accomodated 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 determing the effective initial strucutural 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) — SGS § 8.4.4

f'_c= 4.0 ksi	Specified 28-day compressive strength of concrete	SGS Eq.	8.4.4-1
f'_{ce}= 1.3 f'_c = 5.2 ksi	Expected concrete compressive strength		
E_c= 4372 ksi	Modulus of elasticity	LRFD Eq.	5.4.2.4-1
G_c= 1749 ksi	Shear modulus		
β_1= 0.79	Equivalent stress block factor	LRFD §	5.7.2.2

Grout Materials — BCS § 8.13.8.3.2

f'_{cg}= 8.0 ksi	Specified compressive strength of bedding layer	BCS Eq.	8.13.8.3.2a

Check: f'_{cg} (ksi) ≥ max[1.25(f'_{ce}+0.5), 6.0] Limit = 7.1 ksi O.K.

Joint grout shall have a <u>3 lb per cy</u> fraction of polypropylene fibers for joint integrity — BCS § 8.13.8.3.5a

Mild Reinforcing Steel — SGS § 8.4.2

f_y= 60 ksi	Specified yield stress	SGS Tbl.	8.4.2-1
f_{ye}= 68 ksi	Expected yield stress	SGS Tbl.	8.4.2-1
f_{ue}= 95 ksi	Expected ultimate tensile strength	SGS Tbl.	8.4.2-1
E_s= 29000 ksi	Modulus of elasticity	SGS Tbl.	8.4.2-1
ε_y= 0.002	Effective yield strain	SGS Tbl.	8.4.2-1
ε_{su}= 0.12	Ultimate tensile strain	SGS Tbl.	8.4.2-1
ε^R_{su}= 0.06	Reduced ultimate tensile strain accounting for bar bending	SGS §	8.4.2
f_{yh}= 60 ksi	Specified minimum yield stress of confining hoops		

Post-Tensioning Steel — SGS § 8.4.3

E_{ps}= 28500 ksi	Modulus of elasticity	SGS Eq.	8.4.3-3
f_{pu}= 270 ksi	Specified ultimate tensile strength	SGS §	8.4.3
$\varepsilon_{ps,EE}$= 0.009	Essentially elastic prestress strain	SGS §	8.4.3

$f_{ps,EE}$= 245 ksi Essentially elastic prestress stress
$\varepsilon^R_{ps,u}$= 0.03 Reduced ultimate prestress strain SGS § 8.4.3

Geometric Input Parameters

Column Dimensions

D_c= 4.0 ft Gross column diameter
d_{ps}= 24.0 in Distance from edge of column to tendon centerline
c_c= 2.0 in Clear cover to hoop reinforcement
A_g= 12.57 ft^2 Gross column section area
A_e= 10.05 ft^2 Effective column shear area SGS Eq. 8.6.2-2
I_g= 12.57 ft^4 Gross column moment of inertia

H_{clr}= 19.0 ft Clear height of column
H= 24.0 ft Height from centerline bent cap to centerline foundation
n_j= 2 = 1 if fixed-pinned ; = 2 if fixed-fixed
H_{inf}= 9.5 ft Distance to point of inflection
L_{ps}= 26.5 ft Total length (unbonded) of post-tensioning tendon
N= 565 kip Total gravity force acting during seismic loading (including overturning)
$N / f'_{ce}A_g$ = 6.00 % Gravity load axial load ratio

Bent Cap Dimension

B_{cap}= 5.0 ft Bent cap width
D_s= 4.5 ft Bent cap height

Column Transverse Reinforcement Specified

Size of bars: No. 6 Butt-welded hoop reinforcement
d_{bar}= 0.75 in Diameter of reinforcing bar
A_v= 0.44 in^2 Area of reinforcing bar
s_{hoop}= 4 in Center-to-center spacing of hoops
D'= 3.60 ft Diameter of confined concrete core
A_{cc}= 10.20 ft^2 Area of confined concrete core
ρ_s= 4 A_v/[D's] = 1.02 % Volumetric ratio of confinement reinforcement SGS Eq. 8.6.2-6

Reinforcement Input Parameters

Column Mild Reinforcement Specified

Number of bars: 16 Size of bars: No. 7
d_{bar}= 0.88 in Diameter of reinforcing bar
A_{bar} 0.60 in^2 Area of reinforcing bar
A_s= 9.6 in^2 Total area of mild reinforcement
ρ_s= 0.53 % Volumetric ratio of mild reinforcement
D_{bars}= 3.47 ft Diameter about which bars are distributed
s_b= 8.17 in Clear spacing of reinforcing bars
Recommended: sb < 8.00 inches: 8.17 < 8.00, O.K. SGS § 8.6.3

Column Post-Tensioning Specified

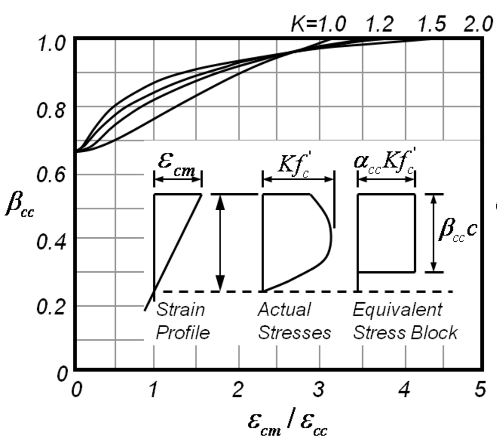
Number of tendons: 1 Number of strands per tendon: 12
Diamter of strands: 0.6 in Area of single strand 0.217 in^2
A_{ps}= 2.60 in^2

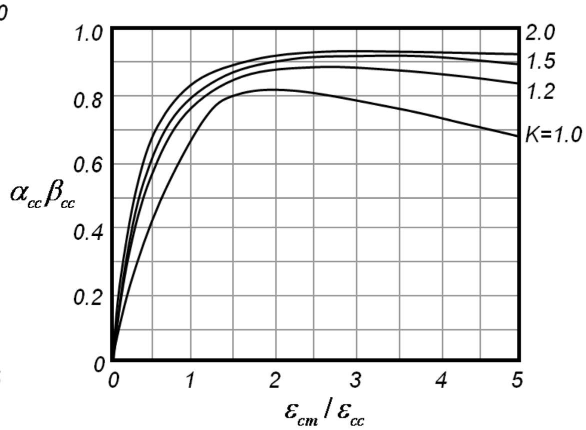
f_{pse} / f_{pu} = 0.60 Effective post-tensioning stress ratio
$A_{ps}\, f_{pse} / f'_{ce}\, A_g$ = 4.48 %

Confined Concrete Properties (based on specified reinforcement)

$\rho_{cc} = A_s / A_{cc}$ =	0.65	%	Volumetric ratio of mild reinforcement with respect to core
$\rho_s = 4\, A_v/[D's]$ =	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
$f_{lp} = k_e\, \rho_s\, f_{yh} / 2$ =	0.28	ksi	Effective confining stress
f'_{cce}=	6.92	ksi	Expected confined concrete compressive strength
ε_{cc}=	0.005		Compressive strain at peak stress
ε_{cu}=	0.019		Ultimate confined concrete compressive strain

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





from Park and Paulay Figure 3.8

β_c= 0.98 Confined concrete equivalent stress block depth factor
$\alpha_{cc}\beta_{cc}$= 0.89
α_{cc}= 0.91 Confined concrete equivalent stress block stress factor

Reference Yield Point

$c =$ 12.78 in — Neutral axis depth (determined via numerical solver)

$c / D_c =$ 26.6 %

$\theta_{conc} = \cos^{-1}\{[0.5D-\beta_1 c]/[0.5D]\} =$ 54.6 deg = 0.9529 rad

$\theta_{steel} = \cos^{-1}\{[0.5D-c]/[0.5D]\} =$ 57.4 deg = 1.0014 rad

Use radians for all equations

$T_{ps,y} = A_{ps} f_{pse}$	Tension force in post-tensioning
$T_{s,y} = A_s f_y \{[\pi-\theta_{steel}]/\pi\}$	Resultant tension force in mild reinforcement
$C_{s,y} = A_s f_y \theta_{steel}/\pi$	Resultant compression force in mild reinforcement
$C_{c,y} = 0.85 f'_{ce} D^2 [\theta_{conc} - \sin(\theta_{conc})\cos(\theta_{conc})]/4$	Resultant compression force in concrete

Tension / Axial Force Summation

$T_{ps,y} =$	422	kip	Post-tensioning
$T_{s,y} =$	445	kip	Mild reinforcement
$N =$	565	kip	Axial load
$\Sigma T =$	1431	kip	

Compression Force Summation

$C_{s,y} =$	208	kip	Mild reinforcement
$C_{c,y} =$	1224	kip	Concrete
$\Sigma C =$	1431	kip	

$\Sigma T = \Sigma C$ 1431 = 1431 Equilibrium satisfied

$y_{st} = 0.5D_{bars}\sin(\pi-\theta_{steel})/[\pi-\theta_{steel}] =$	8.19	in	Resultant steel tension eccentricity
$y_{sc} = 0.5D_{bars}\sin(\pi-\theta_{steel})/[\pi-\theta_{steel}] =$	17.50	in	Resultant steel compression eccentricity
$y_c = D\sin(\theta_{conc})^3/\{3[\theta_{conc}-\sin(\theta_{conc})\cos(\theta_{conc})]\} =$	18.03	in	Resultant concrete compression eccentricity

$M_y = C_{c,y} y_c + C_{s,y} y_{sc} + T_{s,y} y_{st} =$	2445	kip-ft	Reference yield moment
$V_y = M_y / H_{inf} =$	257	kip	Reference yield base shear
$\theta_{b,y} = 2\varepsilon_y L_{ub}/[D-D_{bars}-2c] =$	0.09	%	Fixed end rotation at joint due to opening
$\Delta_f = V_y H_{clr}^3/[3n_j^2 E_c I_g] =$	0.22	in	Deformation due to elastic bending of column
$\Delta_v = V_y H_{clr}/[G_c A_{cs}] =$	0.02	in	Deformation due to shear deformations of column
$\Delta_j = \theta_{b,y} H_{clr} =$	0.20	in	Deformation due to fixed end rotation due to opening
$\Delta_y =$	0.45	in	*Total Deformation*

DR= 0.20 % Drift ratio

Ultimate Point

$c =$ 9.80 in Neutral axis depth (determined via numerical solver)

$c / D_c =$ 20.4 %

$c / D_c < 25\%$ 20.41 < 25.00 O.K. SGS Eq. 8.8.1-4

$\theta_{conc} = \cos^{-1}\{[0.5D'-\beta_{cc}c]/[0.5D']\} =$ 57.4 deg = 1.0019 rad

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

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

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

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

$T_{ps,u} = A_{ps}[f_{pse}+\theta_{b,u}E_{ps}[d_{ps}-c_c-c]/L_{ps}$ Tension force in post-tensioning

$T_{s,u} = A_s f_u \alpha_{ut}$ Resultant tension force in mild reinforcement

$C_{s,y} = A_s f_u \alpha_{uc}$ Resultant compression force in mild reinforcement

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

Tension / Axial Force Summation

$T_{ps,u} =$	529	kip	Post-tensioning
$T_{s,u} =$	654	kip	Mild reinforcement
$N =$	565	kip	Axial load
$\Sigma T =$	1748	kip	

Compression Force Summation

$C_{s,u} =$	131	kip	Mild reinforcement
$C_{c,u} =$	1611	kip	Concrete
$\Sigma C =$	1742	kip	

$\Sigma T = \Sigma C$ 1748 /= 1742 Force imbalance, adjust c

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

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

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

$M_u = C_{c,u}y_c + C_{s,u}y_{sc} + T_{s,u}y_{st} =$ 2677 kip-ft Ultimate moment strength

$V_u = M_u / H_{inf} =$ 282 kip Ultimate shear strength

$\theta_{b,u} = \varepsilon_{cu}$ 1.88 % Fixed end rotation at joint due to opening

$\Delta_f = V_u H_{clr}^3/[3n_j^2 E_c I_g] =$	0.24	in	Deformation due to elastic bending of column
$\Delta_v = V_u H_{clr}/[G_c A_{cs}] =$	0.03	in	Deformation due to shear deformations of column
$\Delta_j = \theta_{b,u} H_{clr} =$	4.29	in	Deformation due to fixed end rotation due to opening
$\Delta_u =$	4.56	in	*Total Deformation*

DR= 2.00 % Drift ratio

PROJECT	DESIGN EXAMPLES	SHEET NO	6	OF	12
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 HYBRID BENT CAP JOINT DESIGN EXAMPLE - SDCs C and D

Performance Requirements

f_{ps}/f_{pu}= 0.75 Post-tensioning stress ratio at ultimate point

$f_{ps}/f_{pu} < f_{ps,EE}/f_{pu}$ 0.75 < 0.91 O.K.

$L_{ub,min}$= 9.98 in Minimum required unbonded length of mild reinforcement

L_{ub}= 12.00 in Specified unbonded length of mild reinforcement

$0.5D_c > L_{ub} > L_{ub,min}$ 24.00 >= 12.00 >= 9.98 O.K. SGS § 8.8.14

$\beta_{M,y}$= 0.25 Contribution of mild reinforcement to reference moment strength

$0.33 > \beta_{M,y} > 0.20$ 0.33 >= 0.25 >= 0.20 O.K. SGS Eq. 8.8.1-3

8.8.2-5

$\beta_{M,u}$= 0.21 Contribution of mild reinforcement to ultimate moment strength

β_{PT}= 1.42 Effective axial load divided by tensile steel force at ultimate point

$\beta_{PT}=(0.9N+T_{ps,y})/T_{s,u} > 1$ 1.42 > 1.00 O.K. SGS Eq. 8.8.1-2

Response Summary

Δ_y= 0.45 in

DR_y= 0.20 %

V_y= 257 kip

M_y= 2445 kip

Δ_C= 4.56 in

DR_u= 2.00 %

V_u= 282 kip

M_u= 2677 kip

k_i= 576 kip/in

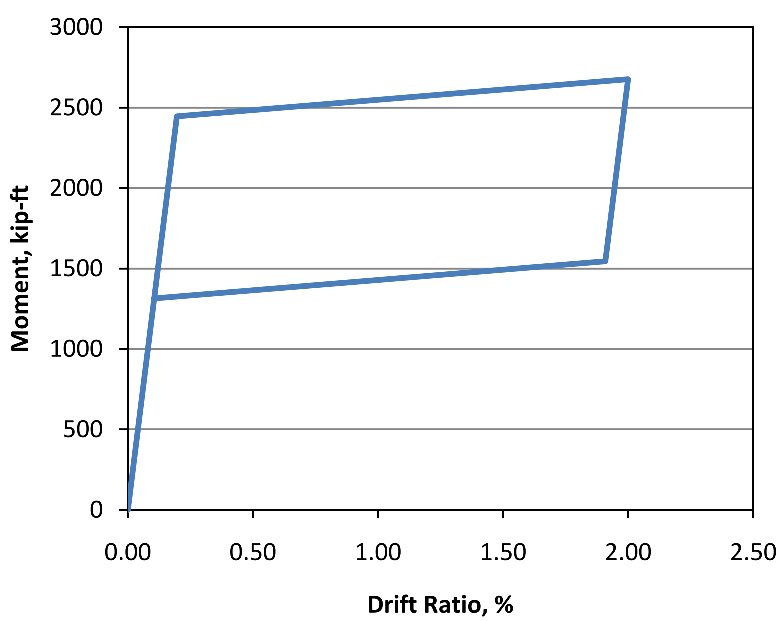
k_{pe}= 6 kip/in

α_{pe}= 1.03 %

$\mu_{\Delta,c}$= 10.2

W = N = 565 kip

T= 0.32 sec



Seismic Demand Analysis

S_S= 1.50 g 0.2 second spectral acceleration (MCE)

S_1= 0.55 g 1.0 second spectral acceleration (MCE)

Site Class: D Soil site class SGS § 3.4.2.1

F_a= 1.00 Site coefficient for 0.2 second period spectral acceleration SGS § 3.4.2.3

F_v= 1.50 Site coefficient for 1.0 second period spectral acceleration SGS § 3.4.2.3

S_{DS}= 1.50 g Design 0.2 second spectral acceleration SGS Eq. 3.4.1-1

S_{D1}= 0.83 g Design 1.0 second spectral acceleration SGS Eq. 3.4.1-2

Seismic Design Category D — SGS § 4

T_S=	0.55	sec	Response spectrum upper bound corner period	SGS Eq.	3.4.1-5
T_o=	0.11	sec	Response spectrum lower bound corner period	SGS Eq.	3.4.1-4
T^*=	0.69	sec	Characteristic ground motion period	SGS Eq.	4.3.3-3
S_a=	1.50	g	Design spectral acceleration	SGS §	3.4.1

Trial 1			*Trial 2*			*Trial 3*			*Trial 4*					
Δ_D=	1.47	in	Δ_D=	2.67	in	Δ_D=	2.91	in	Δ_D=	2.93	in			
$\mu_{\Delta,D}$=	3.29		$\mu_{\Delta,D}$=	5.98		$\mu_{\Delta,D}$=	6.51		$\mu_{\Delta,D}$=	6.56			SGS Eq.	4.9-5
R_d=	1.82		R_d=	1.98		R_d=	1.99		R_d=	1.99		<- Converged	SGS Eq.	4.3.3-1/2

$\Delta_D < \Delta_C$ 2.93 < 4.56 Demand less than capacity, O.K. — SGS Eq. 4.8-1

$\mu_{\Delta,D} < 6$ 6.56 > 6.00 Ductility demand greater than limit, NO GOOD — SGS § 4.9

$M_{P\text{-}\Delta}$= 138 kip-ft P-Δ moment demand based un seismic dispalcement demand

$M_{P\text{-}\Delta} < 0.25 \times M_u$ 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.

$M_p=M_uH/H_{clr}$ = 3382 kip-ft Effective ultimate moment capacity at CL bent cap

$M_{po} = \lambda_{mo} M_p$ — SGS § 9

λ_{mo} = 1.2 (ASTM A706) — SGS Eq. 8.5-1

M_{po} = 4058 kip-ft Overstrength moment demand acting on bent cap

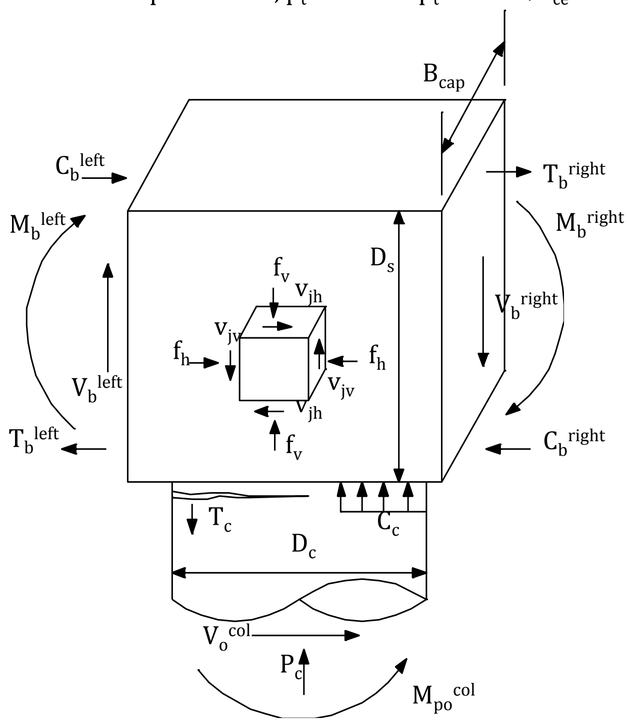
$A_s{}^{cap}{}_{top}$ = 16.55 in^2
$A_s{}^{cap}{}_{bot}$ = 12.92 in^2
Required longitudinal reinforcement required per design requirements of Extreme I load case

Joint Proportioning — SGS § 8.15.2

Principal Stresses

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

Principal compression, p_c :	$p_c \leq 0.25\, f'_{ce}$	=	1.30	ksi maximum	SGS Eq. 8.15.2.1-1
Principal tension, p_t :	$p_t \leq 0.38 \sqrt{f'_{ce}}$	=	0.87	ksi maximum	SGS Eq. 8.15.2.1-2



B_{cap} =	60.0	in	
D_c =	48.0	in	
D_s =	54.0	in	
l_{ac} =	51.0	in	Note 1
P_c =	987	kips	Includes effective PT
P_b =	0	kips	Note 2
h =	2.18	ft	Note 3
M_{po} =	4058	kip-ft	Note 3

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.

$$p_t = \left| \left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} \right| = 0.148 \text{ ksi}$$

Principal tension stress in joint — SGS Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} = 0.309 \text{ ksi}$$

Principal compression stress in joint — SGS Eq. 8.15.2.1-4

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

p_c =	0.309	ksi	≤	1.30	ksi maximum	OK
p_t =	0.148	ksi	≤	0.87	ksi maximum	OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length of Column Longitudinal Reinforcement SGS § 8.15.2.2

$l_{ac} \geq \dfrac{2\, d_{bl}\, f_{ye}}{f'_{cg}}$ SGS Eq. 8.15.2.2.2-1

$d_{bl} =$ 0.88 in No. 7
$f_{ye} =$ 68 ksi
$f'_{cg} =$ 7.0 ksi = min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2

$l_{ac} =$ 17.1 in (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 in

51.0 in ≥ 17.1 in (minimum) OK Extend to top face of cap less 3 in cover.

Minimum Joint Shear Reinforcing for Precast Bent Cap Connections SGS § 8.15.3

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_t, is ≥ $0.11\sqrt{f'_{ce}}$ (condition for likely joint cracking)

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

$p_t =$ 0.148 ksi ≤ 0.251 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:

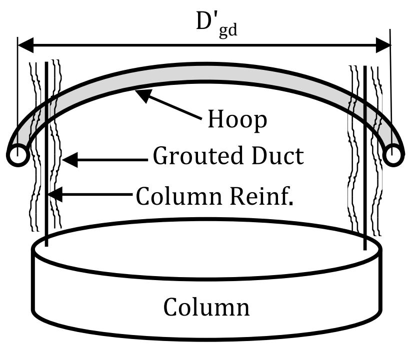
$\rho_s \geq 0.11\sqrt{f'_{ce}} / f_{yh} =$ 0.0037 SGS Eq. 8.15.3.1-1

$A_{st} =$ 9.6 in^2
$l_{ac} =$ 51.0 in

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

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

$A_{sp} =$ 0.44 in^2 (#6 hoop)
$D' =$ 43.25 in (confined diameter of column between centroids of hoop)
$s =$ 8.0 in (trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or $0.3D_s$ per Art. 8.15.3.2.1)



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

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

ρ_s = 0.0047

ρ_s = 0.0047 ≥ 0.0037 minimum OK

8" 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 SGS § 8.15.3.2

$A_s^{jvi} \geq 0.10\ A_{st}$ Required area of vertical stirrups spaced evenly over a length equal to D_c SGS Eq. 8.14.5.2.2a-1

A_{st} = 9.6 in^2

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

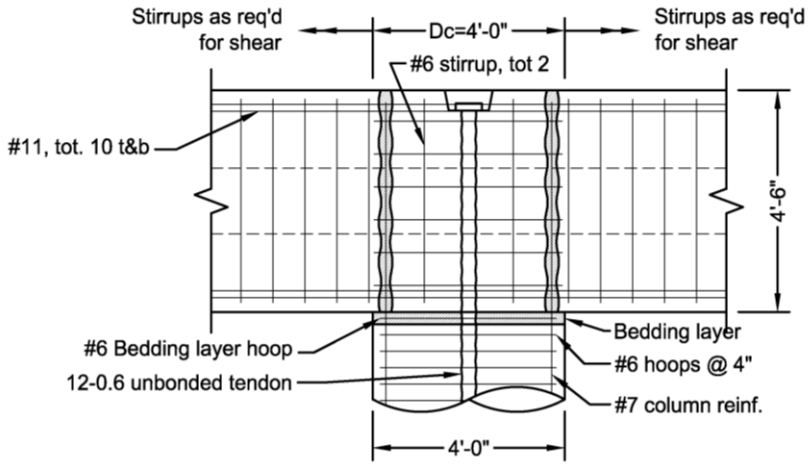
Use (2) #6 double leg stirrups spaced evenly inside joint region.

A_s^{jvi} = 2 locations x 2 legs / location x 0.44 in^2 / leg = 1.76 in^2 >= 0.96 in^2 O.K.

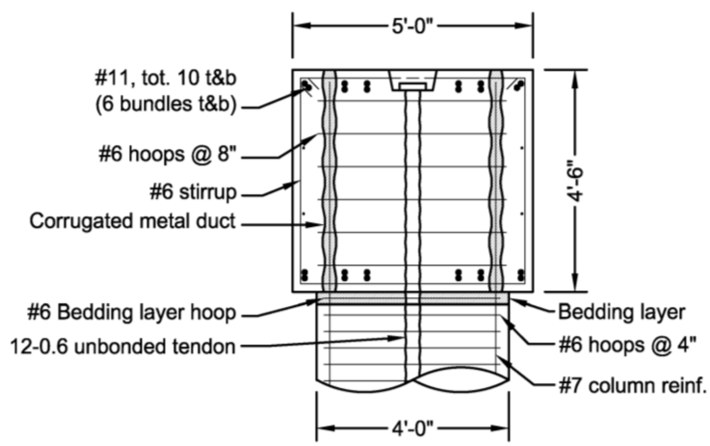
Bedding Layer Reinforcement SGS 8.15.5.2.1

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

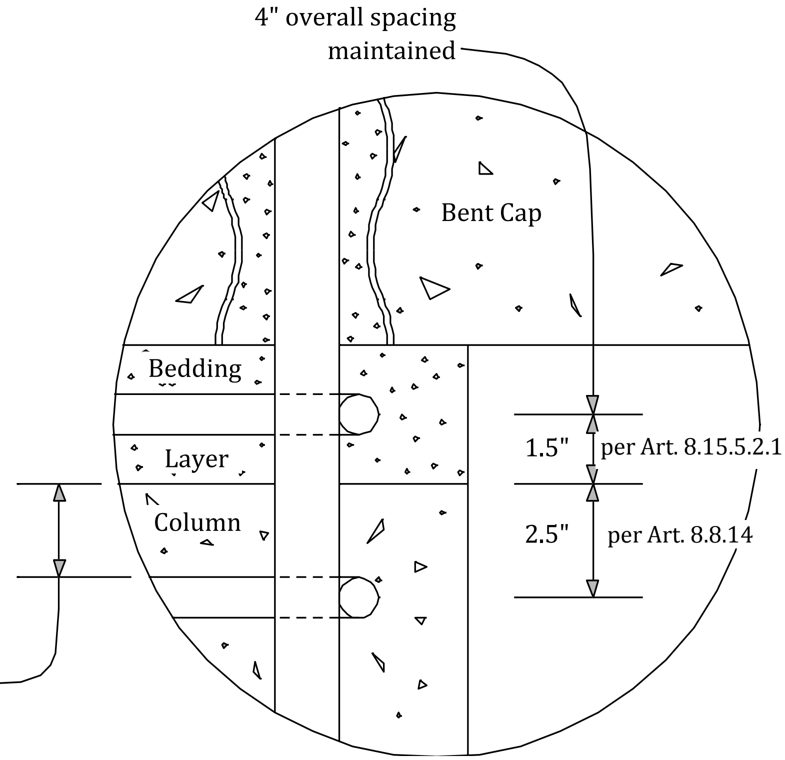
Figures Showing Final Design



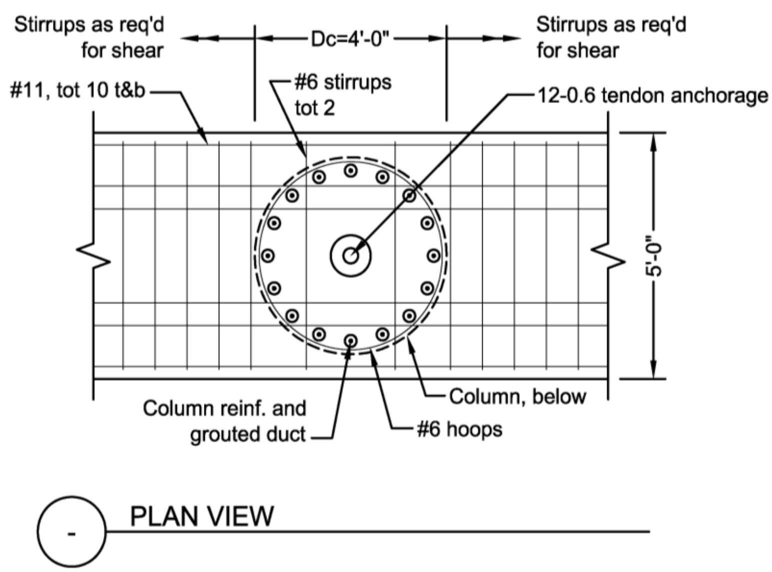
- ELEVATION AT COLUMN



- SECTION AT COLUMN CENTERLINE



Simplified Section at Column Edge



PLAN VIEW

Integral Bent Cap Design Example for SDC C and D

AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS)

AASHTO LRFD Bridge Construction Specifications (BCS)

AASHTO LRFD Bridge Design Specifications (LRFD)

This design example provides practitioners with a comprehensive example of the design activties 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) — SGS § 8.4.4

f'_c = 4.0 ksi — Specified 28-day compressive strength of column concrete — SGS Eq. 8.4.4-1

$f'_{ce} = 1.3 f'_c$ = 5.2 ksi — Expected column concrete compressive strength

f'_{cg} = 6.0 ksi — Specified 28-day compressive strength of girder concrete

Grout Materials — BCS § 8.13.8.3.2

f'_{cgb} = 8.0 ksi — Specified compressive strength of grout bedding layer

Check: f'_{cgb} (ksi) ≥ max[1.25(f'_{ce})+0.5, 6.0] — BCS Eq. 8.13.8.3.2a

8.0 ksi >= 7.0 ksi O.K.

f'_{cgv} = 8.0 ksi — Specified compressive strength of grout closure joint

Check: f'_{cgv} (ksi) ≥ max[1.25(f'_{cg})+0.5, 6.0] — BCS § 8.13.8.3.1

8.0 ksi >= 8.0 ksi O.K.

Closure joint grout shall have a 3 lb per cy fraction of polypropylene fibers for joint integrity — BCS § 8.13.8.3.6a

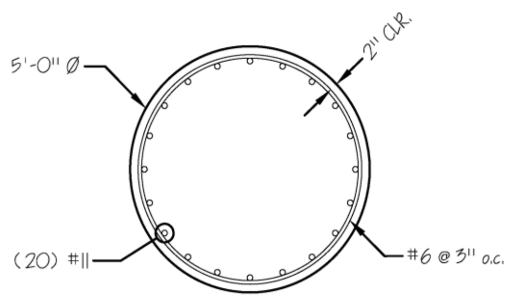
Mild Reinforcing Steel — SGS § 8.4.2

f_y =	60	ksi	Specified yield stress	SGS Tbl. 8.4.2-1
f_{ye} =	68	ksi	Expected yield stress	SGS Tbl. 8.4.2-1
f_{ue} =	95	ksi	Expected tensile strength	SGS Tbl. 8.4.2-1
E_s =	29000	ksi	Modulus of elasticity	SGS Tbl. 8.4.2-1
ε_y =	0.002		Effective yield strain	SGS Tbl. 8.4.2-1
ε^R_{su} =	0.09		Reduced ultimate tensile strain	
f_{yh} =	60	ksi	Specified yield stress of confining hoops	

Geometric Input Parameters

Column Dimensions

D_c=	5.0	ft	Gross column diameter
H_{clr}=	22.0	ft	Clear height of column
H=	11.0	ft	Clear height to point of contraflexure (fixed-fixed column)
L_p=	27.9	in	Idealized plastic hinge length
L_p/D_c=	0.465		Normalized plastic hinge length
L=	140	ft	Span length between centerline of bents



Column reinforcement

(20) No. 11 A706 Reinforcing Bars

$A_{s,bar}$ = 1.56 in^2

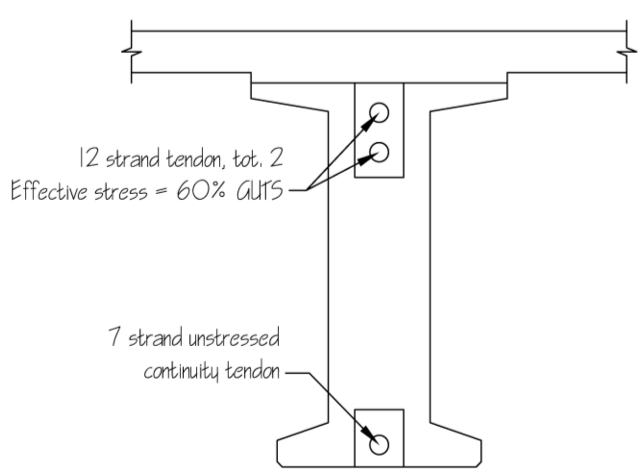
A_{st}= 31.2 in^2

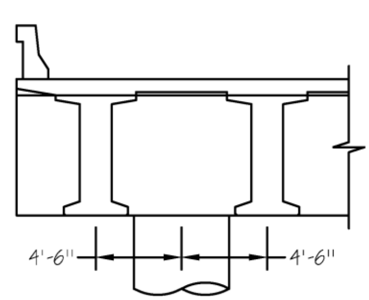
Superstructure Dimensions

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

D_{girder}=	74.0	in	Depth of post-tensioned girder
D_{haunch}=	2.0	in	Thickness of girder haunch (specified)
D_{deck}=	8.0	in	Thickness of reinforced concrete deck
D_s=	7.00	ft	Total depth of superstructure (in longitudinal direction)
s_g=	9.00	ft	Girder center-to-center spacing

A_{ps}^{Top}=	2.604	in^2	f_{pe}^{Top}=	162	ksi	Top tendon details (12-0.6" at 60% GUTS)
A_{ps}^{Mid}=	2.604	in^2	f_{pe}^{Mid}=	162	ksi	Middle tendon details (12-0.6" at 60% GUTS)
A_{ps}^{Bot}=	1.519	in^2	f_{pe}^{Bot}=	0	ksi	Bottom tendon details (7-0.6" unstressed)





w_{cj}= 2.0 in Specified closure joint dimension

Check: $w_{cj} \leq 3"$ for grout closure joints BCS § 8.13.8.3.6a

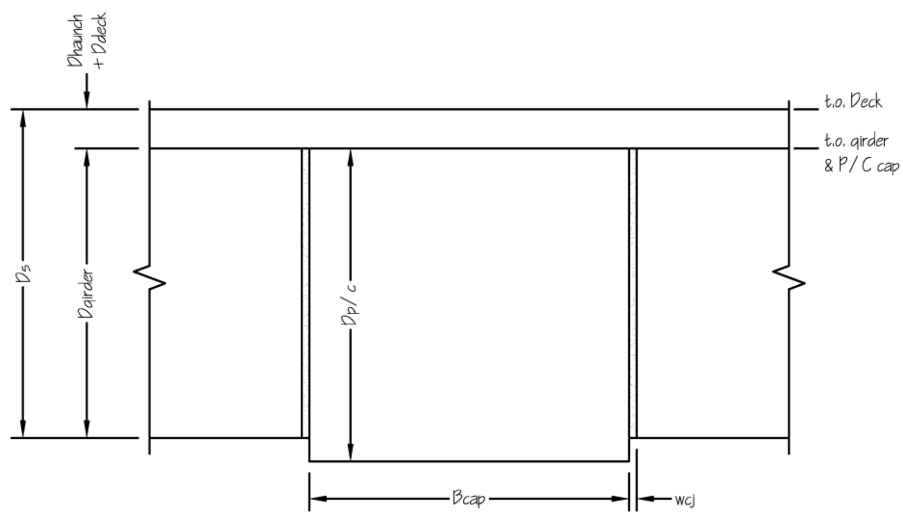
2.0 in <= 3.0 in O.K.

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

A_s^{Top}=	15.6	in^2	f_{pe}^{Top}=	162	ksi	(10) No. 11 reinforcing bars
A_s^{Bot}=	12.48	in^2	f_{pe}^{Mid}=	162	ksi	(8) No. 11 reinforcing bars
A_{ps}^{BC}=	9.11	in^2	f_{pe}^{Mid}=	162	ksi	Middle tendon details (12-0.6" at 60% GUTS)



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.

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.

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 x f'_c = 5.2 ksi — SGS
f_{ye} (expected steel yield strength) = 68 ksi (column bars) — 8.4.4
A_{st} = 31.2 in^2 Total area of column reinforcement

M_p = 6550 kip-ft Idealized bending moment capacity (from moment-curvature) — SGS § 8.5
$M_{po} = \lambda_{mo} M_p$
λ_{mo} = 1.2 (ASTM A706) — SGS Eq. 8.5-1

M_{po} = 7860 kip-ft Overstrength bending moment capacity of column (at critical section)

$V_{po} = M_{po} / H$ = 715 kip-ft Overstrength shear demand from column plastic hinge
$M_{po,cap} = V_{po} \times [H + 0.5\, D_{cap}]$ = 10540 kip-ft Overstrength bending moment demand at superstruct.

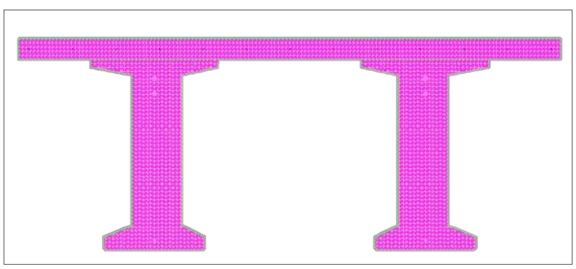
Superstructure Capacity Design for Longitudinal Direction for SDC C and D — SGS § 8.10

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 ft Effective width of superstructure (open soffit, girder-deck system) — SGS Eq. 8.10-2

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



Sample sectional analysis input image

From moment-curvature analysis:

Nominal Bending Moment Capacity

M_{ne}^+ = 8975 kip-ft
M_{ne}^- = -13015 kip-ft

Ultimate Curvature Capacity

ϕ_u^+ = 6.641E-03 1/ft
ϕ_u^- = -4.940E-03 1/ft

NOTE: Details of moment-curvature not shown

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

Design actions acting on 2 girders

$\gamma_D M_D$=	-850	kip-ft	γV_D=	415	kip	Factored dead load demand
0.5 $M_{LL,min}$=	-2248	kip-ft	0.5 $V_{LL,min}$=	90	kip	Factored live load for seismic design
0.5 $M_{LL,max}$=	414	kip-ft	0.5 $V_{LL,max}$=	-10	kip	Factored live load for seismic design

PGA_h = 0.60 g Horizontal peak ground acceleration
V / H = 1.00 Vertical to horizontal spectral acceleration ratio for zero period acceleration
Based on Bozorgnia and Campbell (2004)
PGA_v = 0.60 g Vertical peak ground acceleration

$M_{E,v} = [\ PGA_v\ V_D\ /\ L\ ]\ L^2\ /\ 12 =$ 2324 kip-ft Seismic moment demand from vertical
$V_{E,v} = PGA_v\ V_D =$ 199 kip Seismic shear demand from vertical

$M_{EEI,min} =$ -10692 kip-ft >= $M_{ne}^- =$ -13015 kip-ft O.K.
$M_{EEI,max} =$ 7158 kip-ft <= $M_{ne}^+ =$ 8975 kip-ft O.K.

Superstructure Rotation Capacity

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

$\theta_u =$ 0.017 rad Limiting superstructure rotation capacity
Check: θ_u >= 0.01 rad $\theta_u =$ 0.017 rad >= 0.010 O.K. SGS § 8.10.3

$\Delta_{settle}=\theta_u\ L =$ 29.0 in Permissible relative settlement between bents

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 SGS § 8.10.5

Seismic Shear Capacity across Closure Joint

$V_{EEI}^+ =$ 390 kip Maximum vertical shear acting on single girder

$N_{PT} =$ 844 kip Effective beam post-tensioning force
$\mu =$ 0.6 Shear friction coefficient
$\phi =$ 0.9 Strength reduction factor for shear LRFD § 5.5.4.2

$\phi V_n = \phi\ \mu\ N_{PT} =$ 456 kip

$V_{EEI}^+ =$ 390 kip <= $\phi V_n =$ 456 kip O.K.

Seismic Shear Capacity at Girder End

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

$V_{EEI}^+ =$ 170 kip Vertical shear acting on single girder during maximum positive flexure
$V_{EEI}^+ =$ 390 kip 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 in Neutral axis depth at nominal moment capacity (from moment-curvature)

$\phi =$ 0.9 Strength reduction factor for shear

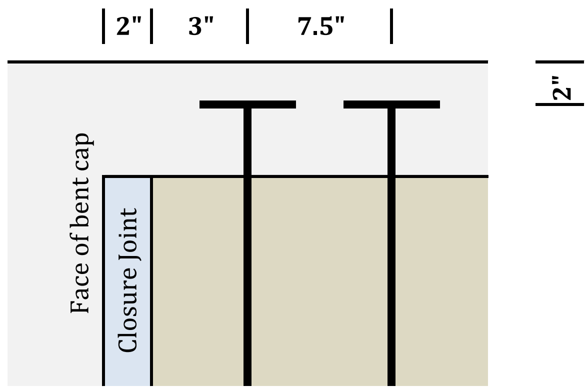
$A_{s,req} =$ 3.15 in^2 Minimum required area of shear reinforcement
length = 14.08 in Distance over which shear reinforcement shall be placed

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 in^2 Minimum required area of shear reinforcement

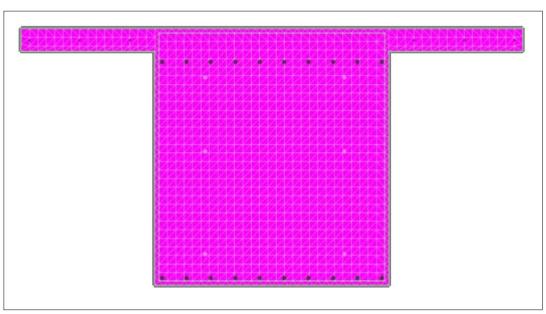
$A_{s,req}$ = 3.15 in^2 <= $A_{s,prov}$ = 3.16 in^2 O.K.



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 ft Effective width of bent cap flange SGS Eq. 8.11-1



Sample sectional analysis input image

From moment-curvature analysis:

M_{ne}^{+} = 14125 kip-ft

M_{ne}^{-} = -13105 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 kip-ft >= M_{ne}^{-} = -13105 kip-ft O.K.

$M_{EEI,max}$= 11935 kip-ft <= M_{ne}^{+} = 14125 kip-ft O.K.

Joint Performance SGS § 8.15.1

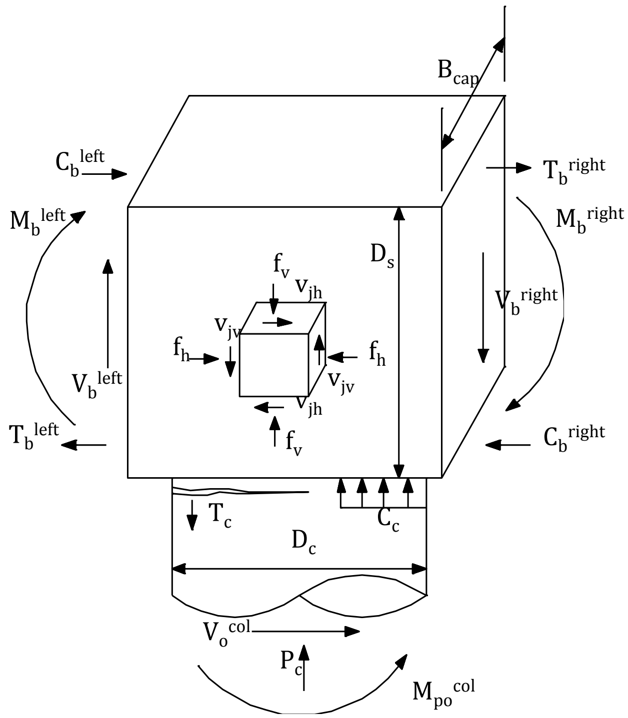
Moment resisting connections shall be designed to transmit the maximum forces produced when the column has reached its overstrength capacity, M_{po}

Joint Proportioning — SGS § 8.15.2

Principal Stresses

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

Principal compression, p_c :	$p_c \le 0.25\, f'_{ce}$	=	1.30	ksi maximum	SGS Eq. 8.15.2.1-1
Principal tension, p_t :	$p_t \le 0.38 \sqrt{f'_{ce}}$	=	0.867	ksi maximum	SGS Eq. 8.15.2.1-2



B_{cap} =	84.0	in	
D_c =	60.0	in	
D_s =	84.0	in	
l_{ac} =	71.0	in	Note 1
P_c =	825	kips	
P_b =	0	kips	Note 2
h =	3.79	ft	Note 3
$M_{po,cap}$ =	10540	kip-ft	Note 3

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.

$$p_t = \left| \left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} \right| = 0.433 \text{ ksi}$$

Principal tension stress in joint — SGS Eq. 8.15.2.1-3

$$p_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v^2_{jv}} = 0.502 \text{ ksi}$$

Principal compression stress in joint — SGS Eq. 8.15.2.1-4

$T_c = M_{po} / h$	2781	kips	Column tensile force at ultimate (from analysis above)	SGS § 8.15.2
$A_{jv} = l_{ac}\, B_{cap}$ =	5964.0	in^2	Vertical joint area	SGS Eq. 8.15.2.1-6
$v_{jv} = T_c / A_{jv}$ =	0.466	ksi	Vertical joint shear stress	SGS Eq. 8.15.2.1-5
$A_{jh} = (D_c + D_s)\, B_{cap}$ =	12096	in^2	Horizontal joint area	SGS Eq. 8.15.2.1-8
$f_v = P_c / A_{jh}$ =	0.0682	ksi	Vertical joint shear stress	SGS Eq. 8.15.2.1-7
$f_h = P_b / (B_{cap}\, D_s)$ =	0.0	ksi	Horizontal joint stress	SGS Eq. 8.15.2.1-9

p_c =	0.502	ksi	≤	1.30	ksi maximum	OK
p_t =	0.433	ksi	≤	0.867	ksi maximum	OK

Joint proportions are acceptable based on principal stress requirements.

Minimum Development Length of Column Longitudinal Reinforcement SGS § 8.15.2.2

$l_{ac} \geq \dfrac{2\, d_{bl}\, f_{ye}}{f'_{cg}}$ Eq. 8.15.2.2.2-1

$d_{bl} =$ 1.41 in (#11 rebar)
$f_{ye} =$ 68 ksi
$f'_{cg} =$ 7.0 ksi = min (7.5 ksi, 7.0 ksi) per Article 8.15.2.2.2

$l_{ac} =$ 27.4 in (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 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 SGS § 8.15.3

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_t, is ≥ $0.11\sqrt{f'_c}$ (condition for likely joint cracking)

Calculated tension = 0.433 ksi
Limit, $0.11\sqrt{f'_{ce}}$ = 0.251 ksi

$p_t =$ 0.433 ksi ≥ 0.251 ksi limit Additional joint reinforcement is required.

Calculate required volumetric ratio of transverse joint reinforcement:

Maximum of: $\rho_s \geq 0.11\sqrt{f'_c} / f_{yh}$ Eq. 8.15.3.1-1
$\rho_s \geq 0.40\, A_{st} / l^2_{ac}$ Eq. 8.15.3.1-2

$A_{st} =$ 31.20 in^2
$l_{ac} =$ 71.0 in

Maximum of: $0.11\sqrt{f'_{ce}} / f_{yh} =$ 0.0042 governs Eq. 8.15.3.1-1
$0.40\, A_{st} / l^2_{ac} =$ 0.0025 Eq. 8.15.3.1-2

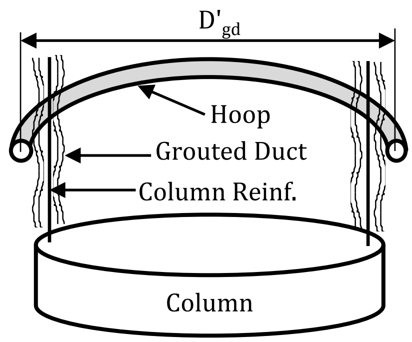
Use $\rho_s \geq$ 0.0042

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 in^2 (#6 hoop)
$D'_{gd} =$ 57.92 in (confined diameter of column between centroids of #6 hoop)

s = 6.0 in (trial spacing of transverse reinforcement hoops; spacing not to exceed 12" or $0.3D_s$ per Art. 8.15.3.2.1)



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

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

ρ_s = 0.0051

ρ_s = 0.0051 ≥ 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

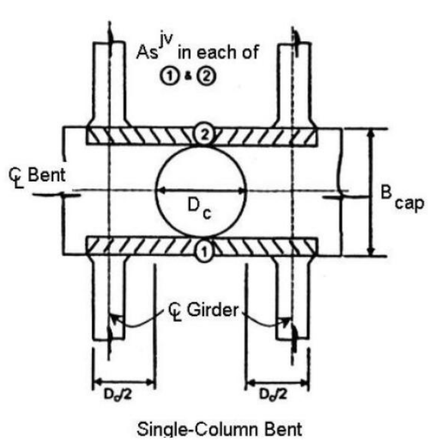
$B_{cap} \geq D_c + 24"$ 84 in >= 84 in O.K., Bent cap width adequate for integral design SGS § 8.13.4.1

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.

$A_s^{jv} \geq 0.20\, A_{st}$ A_{st} = 31.20 in^2
$A_s^{jv} \geq$ 6.24 in^2 Minimum required area of vertical reinforcement SGS Eq. 8.15.4.2.1-1



A_s^{jvo} required: in Regions 1 and 2

USE: No. 6 Stirrups , 4 legs per location

4 locations x 4 legs / location x 0.44 in^2/leg

A_s^{jvo} = 7.04 in^2 >= 6.24 in^2 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

$A_s^{jvo} \geq 0.175\ A_{st}$ $A_{st} =$ 31.20 in^2

$A_s^{jvo} \geq$ 5.46 in^2 Minimum required area of vertical reinforcement outside joint SGS Eq. 8.15.5.1.1-1

USE: No. 6 Stirrups , 4 legs per location $A_s^{jvo} =$ 7.04 in^2 >= 5.46 in^2 O.K.

Spacing outside of joint region: 10.0 in

$A_s^{jvo} \geq 0.135\ A_{st}$ $A_{st} =$ 31.20 in^2

$A_s^{jvo} \geq$ 4.21 in^2 Minimum required area of vertical reinforcement inside joint SGS Eq. 8.15.5.1.2-1

3 locations x 4 legs / location x 0.44 in^2/leg

$A_s^{jvo} =$ 5.28 in^2 >= 4.21 in^2 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 verticallys at not more than 18 inches.

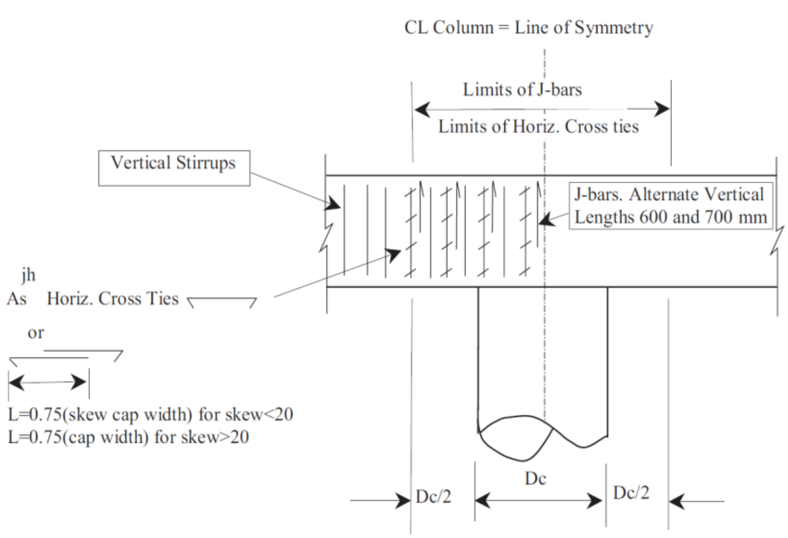
$A_s^{jh} \geq 0.10\ A_{st}$ $A_{st} =$ 31.20 in^2

$A_s^{jh} \geq$ 3.12 in^2 Minimum required area of horizontal reinforcement SGS Eq. 8.15.4.2.1-1

$D_{P/C}=$ 80.00 in Depth of precast bent cap

$N_{space,min} = D_{P/C}$ / 18in = 4.4 Minimum number of spaces between cross ties

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



SGS Figure 8.15.4.2.1-3

Space at 12" and hook on side face reinforcement

USE: No. 4 cross ties
Place at each vertical stirrup

	5	locations
x	5	ties / location
x	0.2	in^2/tie
$A_s^{jh} =$	5.00	in^2
>=	3.12	in^2 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 SGS § 8.15.4.2.3

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.

$$A_s^{sf} \geq \begin{cases} 0.10\,A^{cap}_{top} = 1.56 \text{ in}^2 \\ 0.10\,A^{cap}_{bot} = 1.25 \text{ in}^2 \end{cases} \qquad A^{cap}_{top} = 15.60 \text{ in}^2 \quad A^{cap}_{bot} = 12.48 \text{ in}^2$$ SGS Eq. 8.15.4.2.3-1

$A_s^{sf} \geq$ 1.56 in^2 Minimum horizontal side face reinforcement required

$D_{P/C}$= 80.00 in Depth of precast bent cap

$N_{space,min} = D_{P/C}$ / 18in = 6.7 Minimum number of spaces between reinforcement

Minimum number of locations of reinforcement per side of bent cap 7

USE: No. 5 side face reinforcement throughout cap with maximum spacing of 12 inches

A_s^{sf}= 7 locations x 0.31 in^2/location = 2.17 in^2 >= 1.56 in^2 O.K.

J-Bars SGS § 8.15.4.2.4

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 SGS § 8.15.4.2.5

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

$A_s^{jl} \geq 0.245\,A_{st}$ A_{st} = 31.20 in^2

$A_s^{jh} \geq$ 7.64 in^2 Additional longitudinal cap beam reinforcement SGS Eq. 8.15.5.1.3-1

Torsional Shear-Friction Requirement SGS § 8.10.5

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.

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

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

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

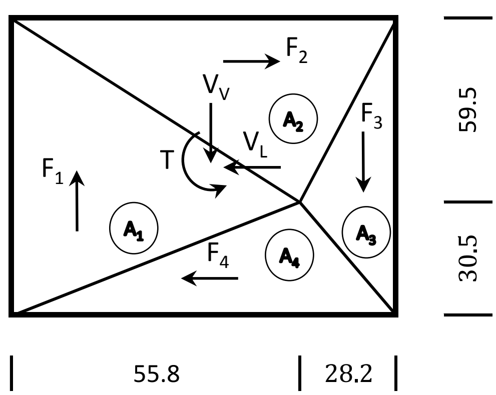
μ = 1.4 Shear friction coefficient (bent cap in continuous)
$\tau = [\mu P] / A_{cap}$ = 54.5 kip/ft^2 Shear friction stress

$T = 0.5 M_{po,cap}$ = 5270 kip-ft Torsional demand on bent cap
$V_V = 0.5 P_c$ = 413 kip Vertical shear demand on bent cap
$V_L = 0.5 V_o$ = 357 kip Longitudinal shear demand on bent cap

ϕ = 0.9 Strength reduction factor for shear LRFD § 5.5.4.2

$T = \phi \tau [A_1x_1 + A_2x_2 + A_3x_3 + A_4x_4]$ 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

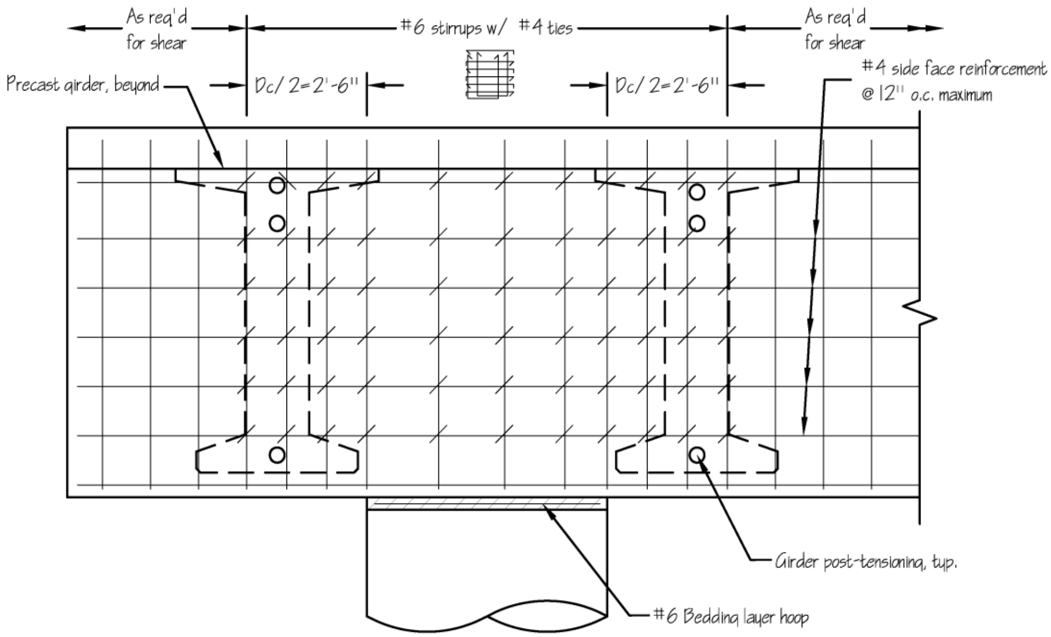
Region	Area ft^2	Distance from Centroid, ft	First moment centroid, ft^3
1	17.44	1.95	34.0
2	17.35	2.10	36.4
3	8.81	2.72	23.9
4	8.90	2.90	25.8
Total	52.50		120.2
	Capacity	**Required**	**Check**
T	5897	5270	OK
V_V	423	413	OK
V_L	415	357	OK



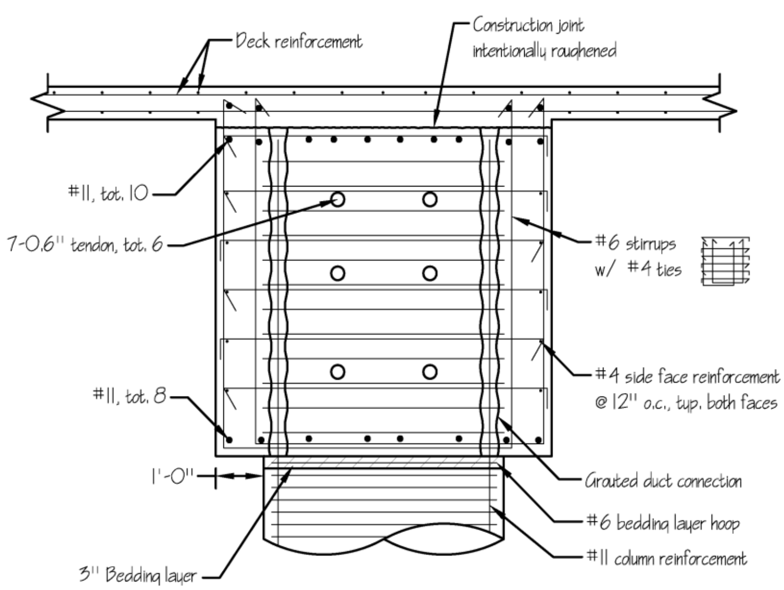
Bedding Layer Reinforcement SGS

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. 8.15.5.2.1

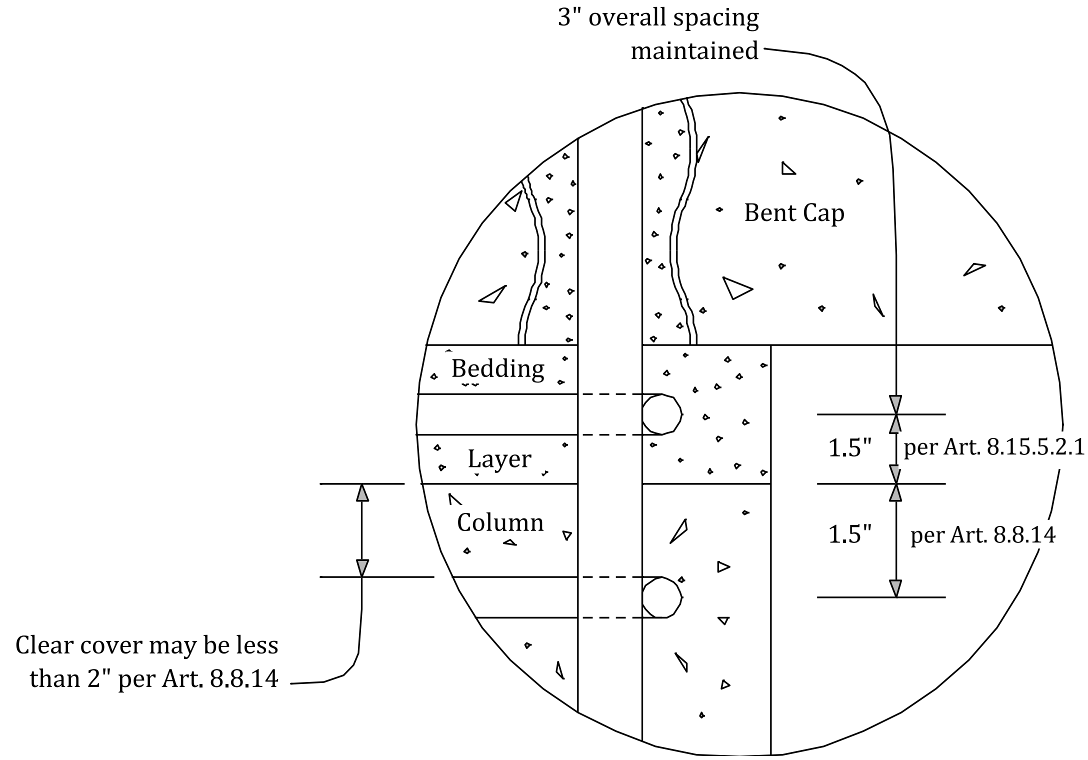
Figures Showing Final Design



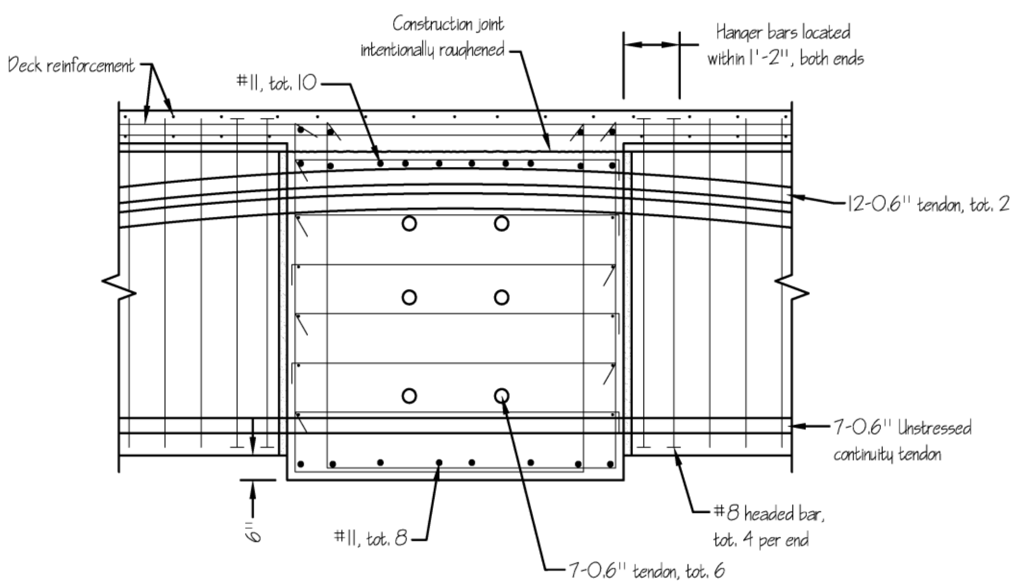
Elevation View at Column Connection



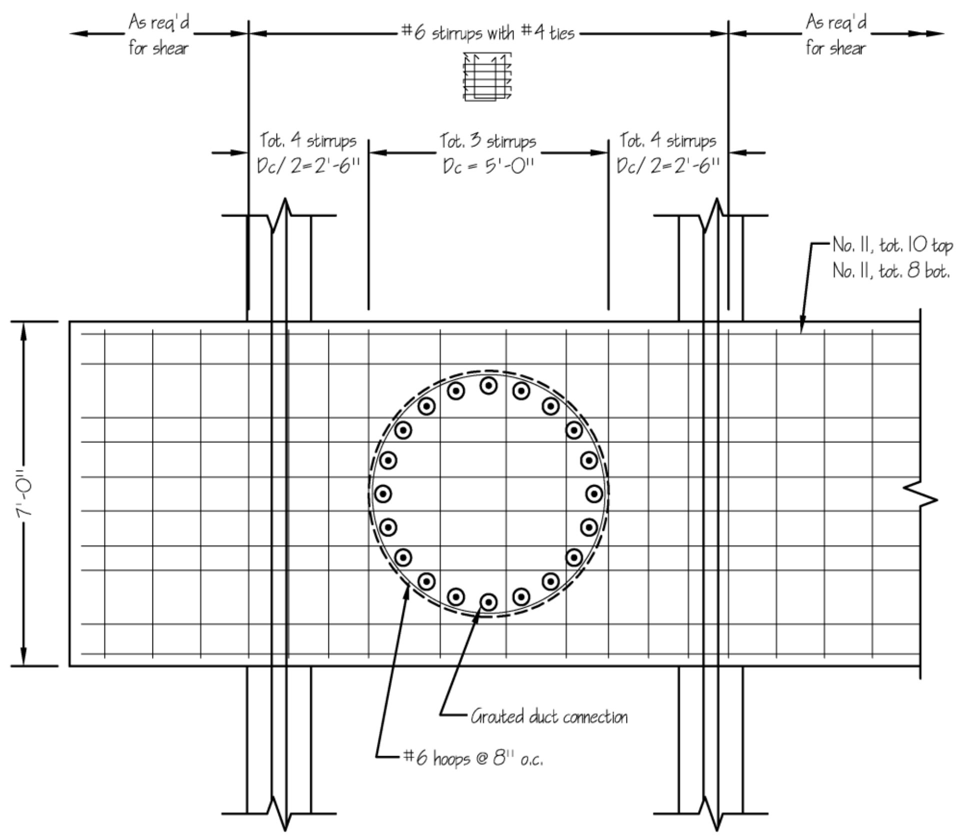
Typical Section at Column Connection



Closeup of Bedding Layer



Typical Section at Girder Connection



<u>Plan View at Connections</u>