DS Attachments

Design Specifications

- Attachment DS1: Revised Article 2.1 Definitions
  - Revision of current article to include definitions of emulative and hybrid systems
- Attachment DS2: Revised Article 4.3.3 Displacement Magnification for Short Period Structures
  - Revised Article to account for hybrid systems
- Attachment DS3: Revised Article 4.7.2 Vertical Ground Motion, Design Requirements for SDC D
  - Expanded Article to include explicit requirements for consideration of vertical excitation with integral precast bent caps discontinuous at bent
- Attachment DS4: Revised Article 4.11.6 Analytical Plastic Hinge Length
  - Revised Article to account for integral concrete superstructures
- Attachment DS5: Proposed Article 5.6.6 I_{eff} for Hybrid Systems
  - New Article for hybrid systems
- Attachment DS6: Revised Article 8.4.2 Reinforcing Steel Modeling
  - Revised Article for hybrid systems
- Attachment DS7: Proposed Article 8.8.14 Lateral Reinforcement Requirement for Columns Connecting to a Precast Bent Cap
  - New Article to ensure spacing between the hoop at top of column and the bedding layer hoop does not compromise system ductility.
- Attachment DS8: Revised Article 8.5 Plastic Moment Capacity for SDC B, C and D
  - Revised Article for hybrid systems
- Attachment DS9: Revised Article 8.8.1 Maximum Longitudinal Reinforcement
  - Revised Article for hybrid systems
- Attachment DS10: Revised Article 8.8.2 Minimum Longitudinal Reinforcement
  - Revised Article for hybrid systems
- Attachment DS11: Proposed Article 8.8.14 Minimum Debonded Length of Longitudinal Reinforcement for Hybrid Columns
  - New Article for hybrid systems
- Attachment DS12: Revised Article 8.10 Superstructure Capacity Design for Longitudinal Direction for SDC C and D
  - Revised Article for integral precast systems
- Attachment DS13: Proposed Article 8.13 Joint Design for SDC A
  - New Article for SDC A precast bent cap connection design.
- Attachment DS14: Proposed Article 8.14—Joint Design for SDC B
  - New Article for SDC B precast bent cap connection design.
- Attachment DS15: Revised Article 8.15—Joint Design for SDCs C and D
  - Revision of current Article 8.13 for SDCs C and D to Article 8.15 for precast bent cap connection design.
- Attachment DS16: Revised Article 5.10.11.4.3—Column Connections
  - Revised Article to ensure AASHTO LRFD SGS is used for emulative precast bent cap-to-column connection design.
- Attachment DS176: Proposed Article 5.11.1.2.4—Moment Resisting Joints
  - Revised Article to ensure AASHTO LRFD SGS is used for emulative precast bent cap-to-column connection design.

Attachment DS1. Revised Article 2.1—Definitions (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

2.1 DEFINITIONS

Capacity Design – A method of component design that allows the designer to prevent damage in certain components by making them strong enough to resist loads that are generated when adjacent components reach their overstrength capacity.

Capacity Protected Element – Part of the structure that is either connected to a critical element or within its load path and that is prevented from yielding by virtue of having the critical member limit the maximum force that can be transmitted to the capacity protected element.

Collateral Seismic Hazard – Seismic hazards other than direct ground shaking such as liquefaction, fault rupture, etc.

Complete Quadratic Combination (CQC) – A statistical rule for combining modal responses from an earthquake load applied in a single direction to obtain the maximum response due to this earthquake load.

Critical or Ductile Elements – Parts of the structure that are expected to absorb energy, undergo significant inelastic deformations while maintaining their strength and stability.

Damage Level – A measure of seismic performance based on the amount of damage expected after one of the design earthquakes.

Displacement Capacity Verification – A design and analysis procedure that requires the designer to verify that his or her structure has sufficient displacement capacity. It generally involves a non-linear static (i.e. "pushover") analysis.

Ductile Substructure Elements – See Critical or Ductile Elements

Earthquake Resisting Element (ERE) – The individual components, such as columns, connections, bearings, joints, foundation, and abutments, that together constitute the Earthquake Resisting System (ERS).
Earthquake Resisting System (ERS) – A system that provides a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Emulative Precast – Precast concrete systems that are designed and detailed to perform in a similar manner as compared to a cast-in-place system.

Hybrid System (or column) – A concrete bent system in which the column employs a combination of unbonded post-tensioning and reinforcement. Properly designed and detailed hybrid systems will result in controlled rocking response of the bent system producing appreciable lateral capacity with a significant reduction in damage and residual displacements as compared to traditional cast-in-place systems.

Life Safety Performance Level – The minimum acceptable level of seismic performance allowed by this specification. It is intended to protect human life during and following a rare earthquake.

Liquefaction – Seismically induced loss of shear strength in loose, cohesionless soil that results from a build up of pore water pressure as the soil tries to consolidate when exposed to seismic vibrations.

Liquefaction-Induced Lateral Flow – Lateral displacement of relatively flat slopes that occurs under the combination of gravity load and excess pore water pressure (without inertial loading from earthquake). Lateral flow often occurs after the cessation of earthquake loading.

Liquefaction-Induced Lateral Spreading – Incremental displacement of a slope that occurs from the combined effects of pore water pressure buildup, inertial loads from the earthquake, and gravity loads.

Maximum Considered Earthquake (MCE) – The upper level, or rare, design earthquake having ground motions with a 3% chance of being exceeded in 75 years. In areas near highly-active faults, the MCE ground motions are deterministically bounded to ground motions that are lower than those having a 3% chance of being exceeded in 75 years.

Minimum Support Width – The minimum prescribed width of a bearing seat that is required to be provided in a new bridge designed according to these specifications.

Nominal Resistance – Resistance of a member, connection or structure based on the expected yield strength (F_{ye}) or other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Operational Performance Level – A higher level of seismic performance that may be selected by a bridge owner who wishes to have immediate service and minimal damage following a rare earthquake.

Overstrength Capacity – The maximum expected force or moment that can be developed in a yielding structural element assuming overstrength material properties and large strains and associated stresses.
Performance Criteria – The levels of performance in terms of post earthquake service and damage that are expected to result from specified earthquake loadings if bridges are designed according to this specification.

Plastic Hinge – The region of a structural component, usually a column or a pier in bridge structures, that undergoes flexural yielding and plastic rotation while still retaining sufficient flexural strength.

Pushover Analysis – See Displacement Capacity Verification

Plastic Hinge Zone – Those regions of structural components that are subject to potential plastification and thus shall be detailed accordingly.

Response Modification Factor (R-Factor) – Factors used to modify the element demands from an elastic analysis to account for ductile behavior and obtain design demands.

Seismic Design Category (SDC) – one of four Seismic Design Categories (SDC), A through D, based on the one second period design spectral acceleration for the Life Safety Design Earthquake

Service Level – A measure of seismic performance based on the expected level of service that the bridge is capable of providing after one of the design earthquakes.

Site Class – One of six classifications used to characterize the effect of the soil conditions at a site on ground motion.

Tributary Weight – The portion of the weight of the superstructure that would act on a pier participating in the ERS if the superstructure between participating piers consisted of simply supported spans. A portion of the weight of the pier itself may also be included in the tributary weight.

Attachment DS2. Revised Article 4.3.3—Displacement Magnification for Short Period Structures (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

4.3.3 Displacement Magnification for Short Period Structures

Displacement demand, Δ_D, calculated from elastic analysis shall be multiplied by the factor R_d obtained from Eq. 1 or 2 to obtain the design displacement demand specified in Article 4.3. This magnification is greater than one (1.0) in cases where the fundamental period of the structure T is less than the characteristic ground motion period T^*, corresponding to the peak energy input spectrum.

$$R_d = \left(1 - \frac{1}{R}\right)\frac{T^*}{T} + \frac{1}{R} \geq 1.0 \quad \text{for } \frac{T^*}{T} > 1.0 \qquad (4.3.3\text{-}1)$$

$$R_d = 1.0 \quad \text{for } \frac{T^*}{T} \leq 1.0 \qquad (4.3.3\text{-}2)$$

in which:

T^* = $1.25T_s$

R = maximum expected displacement ductility of the structure

= 2 for SDC B
= 3 for SDC C
= μ_D for SDC D

where:

T_s = period determined from Article 3.4.1 (sec.)

μ_D = maximum local member displacement ductility demand determined in accordance with Article 4.9. In lieu of detailed analysis, μ_D may be taken as 6.

The displacement magnification is applied separately in both orthogonal directions prior to obtaining the orthogonal combination of seismic displacements specified in Article 4.4.

C4.3.3

The assumption that displacements of an elastic system will be the same as those of an elasto-plastic system is not valid for short-period structures which are expected to perform inelastically. The adjustment factor, R_d, is a method for correction for the displacement determined from an elastic analysis for short-period structures.

This displacement magnification factor is valid for both cast-in-place and precast concrete substructure systems. For hybrid substructure systems, this equation is appropriate for displacement ductility levels allowed by these *Guide Specifications*. (Tobolski*, 2010).

* Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS3. Revised Article 4.7.2—Vertical Ground Motion, Design Requirements for SDC D (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

4.7.2 Vertical Ground Motion, Design Requirements for SDC D	**C4.7.2**
The effects of vertical ground motions for bridges in Seismic Design Category D located within six (6) miles of an active fault as described in Article C3.4, shall be considered. For integral precast bridge superstructures with primary members that are discontinuous at the face of the bent cap (i.e. precast segmental, integral spliced girder systems, etc.), vertical seismic demand shall be explicitly considered in superstructure design for both moment and shear using equivalent static, response spectrum or time history analysis. Demands from vertical ground motion shall be combined with horizontal seismic demands based on plastic hinging forces developed in accordance with Article 4.11.2. Seismic demands shall be combined considering 100% of the demand in the vertical direction added with 30% of the seismic demand resulting from flexural hinging in one of the horizontal perpendicular directions (longitudinal) and 30% of the seismic demand resulting from flexural hinging in the second perpendicular horizontal direction (transverse).	The most comprehensive study (Button et al., 1999) performed to date on the impact of vertical acceleration effects indicated that for some design parameters (superstructure moment and shear, and column axial forces) and for some bridge types, the impact can be significant. The study was based on vertical response spectra developed by Silva (1997) from recorded western United States ground motions. Specific recommendations for assessing vertical acceleration effects will not be provided in these *Guide Specifications* until more information is known about the characteristics of vertical ground motion in the central and eastern United States, and those areas impacted by subduction zones in the Pacific. However, it is advisable for designers to be aware that vertical acceleration effects may be important and should be assessed for essential and critical bridges. See Caltrans Seismic Design Criteria (Caltrans 2006). Capacity design approaches are not applicable for vertical seismic demand due to the lack of specified mechanisms of inelastic response. Therefore, the importance of reasonable estimates of vertical seismic demand are essential to the design of all bridge systems in regions with significant vertical seismic demand (cast-in-place and precast systems). Precast systems that have discrete connections at the face of bridge substructure elements (i.e. that rely on shear friction mechanisms) are more susceptible to seismic vertical demands and care must be taken in their design (Veletzos, 2007; Tobolski*, 2010). * Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS4. Revised Article 4.11.6—Analytical Plastic Hinge Length (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

4.11.6 Analytical Plastic Hinge Length

4.11.6.1 Concrete Columns

The analytical plastic hinge length for reinforced concrete columns, L_p, is the equivalent length of column over which the plastic curvature is assumed constant for estimating the plastic rotation. The plastic rotation is then used to calculate the plastic displacement of an equivalent member from the point of maximum moment to the point of contra-flexure.

For columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft, the plastic hinge length, L_p in in., may be determined as:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \qquad (4.11.6.\underline{1}\text{-}1)$$

where:

L = length of column from point of maximum moment to the point of moment contra-flexure (in.)

f_{ye} = expected yield strength of longitudinal column reinforcing steel bars (ksi)

d_{bl} = nominal diameter of longitudinal column reinforcing steel bars (in.)

For non-cased prismatic pile shafts, the plastic hinge length, L_p in in., may be determined as:

$$L_p = 0.08H' + D^* \qquad (4.11.6.\underline{1}\text{-}2)$$

where:

D^* = diameter of circular shafts or cross section dimension in direction under consideration for oblong shafts (in.)

H' = length of pile shaft/column from point of maximum moment to point of contraflexure above ground (in.)

For horizontally isolated flared columns, the plastic hinge length, L_p in in., may be determined as:

$$L_p = G_f + 0.3 f_{ye} d_{bl} \qquad (4.11.6.\underline{1}\text{-}3)$$

where: G_f = gap between the isolated flare and the soffit of the bent cap (in.) f_{ye} = expected yield strength of longitudinal column reinforcing steel bars (ksi) d_{bl} = nominal diameter of longitudinal column reinforcing steel bars (in.) **4.11.6.2 Integral Concrete Superstructures** The analytical plastic hinge length for integral concrete superstructures, L_{ps}, is the equivalent length of superstructure over which the post-yield curvature is assumed constant for estimating the superstructure inelastic rotation. The superstructure plastic hinge length, L_{ps} in in., may be determined as: $L_{ps} = 0.5D_s$ (4.11.6.2-1) where: D_s = total depth of superstructure (in.)	

Attachment DS5. Proposed Article 5.6.6—I_{eff} for Hybrid Systems (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

5.6.6 I_{eff} for Hybrid Systems	**C5.6.6**
The effective moment of inertia for calculation of elastic flexural deformations for hybrid bridge columns can be taken equal to the gross moment of inertia. For mathematical modeling, the increase in flexibility at reference yield due to joint opening shall be considered. The influence of joint rotation shall be determined in accordance with the provisions of Article 8.5 using moment-rotation analysis. For equivalent elastic analysis, I_{eff} shall be decreased to account for the additional flexibility due to joint rotation.	At the reference yield point, low magnitude fixed end rotations are expected at column ends for hybrid systems. These rotations can have a significant influence on the effective elastic response of the bridge system and must be considered in the development of seismic demands. Additional rotation at reference yield caused by joint rotation can be approximated as the rotation at two times the yield strain of the reinforcing bar considering the debonded length (Tobolski*, 2010). * Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS6. Revised Article 8.4.2—Reinforcing Steel Modeling (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.4.2 Reinforcing Steel Modeling	**C8.4.2**
Reinforcing steel shall be modeled with a stress-strain relationship that exhibits an initial elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain, as shown in Figure 1. In lieu of specific data, the steel reinforcing properties provided in Table 1 should be used. Within the elastic region the modulus of elasticity, E_s, shall be taken as 29,000 ksi. For hybrid connections the reduced ultimate tensile strain, ε_{su}^{R}, shall equal one-half the ultimate tensile strain, ε_{su}.	The steel reinforcement properties provided in Table 1 are based upon data collected by Caltrans. For hybrid systems, localized joint opening at end of the column will impose additional strain demands on the reinforcing bar due to localized bar rotation demands. This geometrically induced loading is accounted for through a further decrease in the reduced ultimate tensile strain.

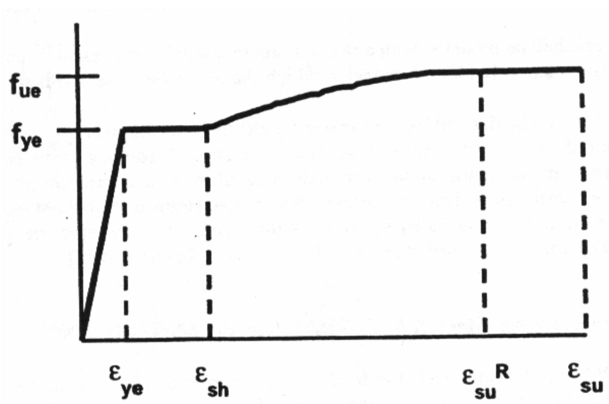


Figure 8.4.2-1 Reinforcing Steel Stress-Strain Model

Table 8.4.2-1 Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3 - #18	60	60
Expected yield stress (ksi)	f_{ye}	#3 - #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3 - #18	95	95
Expected yield strain	ε_{ye}	#3 - #18	0.0023	0.0023
Onset of strain hardening	ε_{sh}	#3 - #8	0.0150	0.0150
		#9	0.0125	0.0125
		#10 - #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^{R}	#4 - #10	0.090	0.060
		#11 - #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4 - #10	0.120	0.090
		#11 - #18	0.090	0.060

Attachment DS7. Proposed Article 8.8.14—Lateral Reinforcement Requirement for Columns Connecting to a Precast Bent Cap (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

Specifications	Commentary
8.8.14—Lateral Reinforcement Requirement for Columns Connecting to a Precast Bent Cap The spacing between the first hoop at the top of the column and the bedding layer hoop shall not exceed the spacing used for hoops in the plastic hinge region. The concrete cover above the first hoop at the top of the column shall be permitted to be less than that specified in Article 5.12.3 of *AASHTO LRFD Bridge Design Specifications*. Transverse reinforcement used for piles shall be similarly detailed.	**C8.8.14** Uniform spacing between hoops at the top of the column and the bedding layer is critical to ensure that the system ductility is not compromised. A smaller cover than that used for typical column applications is permitted for the top hoop because the bedding layer provides additional cover after placement of the precast bent cap. Plan sheets should show the intended placement of the first hoop at the top of the column. The associated requirement for shop drawings is addressed in Article 8.13.8.4.4 of the *AASHTO LRFD Bridge Construction Specifications.*

Attachment DS8. Revised Article 8.5~~—~~Plastic Moment Capacity ~~for Ductile Concrete Members~~ for SDC B, C and D (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.5 PLASTIC MOMENT CAPACITY FOR ~~DUCTILE CONCRETE MEMBERS FOR~~ SDC B, C AND D

8.5.1 Ductile Concrete Members

The plastic moment capacity of all ductile concrete members shall be calculated using a moment-curvature (M-ϕ) analysis based on the expected material properties. The moment curvature analysis shall include the axial forces due to dead load together with the axial forces due to overturning as given in Article 4.11.4.

The M-ϕ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve passes through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by equating the areas between the actual and the idealized M-ϕ curves beyond the first reinforcing bar yield point as shown in Figure 1.

8.5.2 Hybrid Concrete Members

For hybrid concrete members, the plastic moment capacity shall be calculated using a moment-rotation (M-θ) analysis based on the expected material properties. The moment rotation analysis shall include the axial forces due to dead load together with the axial forces due to overturning as given in Article 4.11.4.

The M-θ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve passes through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by equating the areas between the actual and the idealized M-θ curves beyond the first reinforcing bar yield point similar to as shown in Figure 1.

C8.5

C8.5.1

Moment curvature analysis obtains the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. A moment-curvature analysis based on strain compatibility and nonlinear stress-strain relations can be used to determine plastic limit states. The results from this rational analysis are used to establish the rotational capacity of plastic hinges as well as the associated plastic deformations. The process of using the moment-curvature sectional analysis to determine the lateral load-displacement relationship of a frame, column or pier is known as a "pushover analysis."

C8.5.2

Moment-curvature analysis for hybrid systems requires modification to assumed material models to account for debonded lengths of reinforcement and post-tensioning. Moment-rotation analysis provides a simple method to determine the force-deformation response of hybrid systems where discrete joint opening occurs at the column end. The actual strain lengths for reinforcing steel and post-tensioning can be considered when relating a given rotation to force in individual elements. At the column end, the strain profile can be conservatively assumed linear across the section. Concrete strains can be considered constant for a distance equal to the neutral axis depth within the height of the column. See Tobolski* (2010) for more detailed information on moment-rotation analysis of hybrid systems and for additional information on simplified analysis methods for approximation of hybrid response.

* Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS9. Revised Article 8.8.1—Maximum Longitudinal Reinforcement (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for cast-in-place and emulative precast compression members shall satisfy:

$$A_l \leq 0.04\, A_g \qquad (8.8.1\text{-}1)$$

where:

A_g = gross area of member cross section (in^2)

A_l = area of longitudinal reinforcement in member (in^2)

The maximum longitudinal reinforcement for hybrid compression members shall be proportioned to satisfy Equations 2 through 4

$$\frac{0.9P_D + P_{pse}}{T_s} > 1.0 \qquad (8.8.1\text{-}2)$$

where:

P_D = dead load axial load acting on column (kip)

P_{pse} = effective force in post-tensioning tendon at end of service life (kip)

T_s = resultant column reinforcement tension force associated with ultimate moment capacity (kip)

$$\frac{M_s}{M_y} \leq 0.33 \qquad (8.8.1\text{-}3)$$

where:

M_s = flexural moment capacity provided by longitudinal reinforcement at reference yield moment (kip-ft)

M_y = reference yield moment (kip-ft)

$$\frac{c}{D_c} \leq 0.25 \qquad (8.8.1\text{-}4)$$

where:

C8.8.1

This requirement is intended to apply to the full section of the columns. The maximum ratio is to avoid congestion and extensive shrinkage and to permit anchorage of the longitudinal steel, but most importantly, the small amount of longitudinal reinforcement, the greater ductility of the column.

Hybrid systems are intended to provide a self-centering response where column residual deformations are significantly less than a comparable cast-in-place or emulative precast system. To promote self-centering response, the effective axial load acting on the column must be capable of forcing the flexural reinforcement to a zero strain state (Toranzo, et. al, 2009; Tobolski*, 2010).

Additionally, for hybrid systems the flexural contribution of longitudinal reinforcement must be limited to produce a system with the hybrid characteristics. If the flexural contribution is greater than the specified limit, the overall response will begin to resemble a traditional cast-in-place or emulative precast system with increasing residual displacements and damage (Tobolski*, 2010).

* Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

c = distance from extreme compression fiber to the neutral axis at the reference yield point (in.) D_c = column diameter or smallest dimension in the direction of loading (in.)	

Attachment DS10. Revised Article 8.8.2—Minimum Longitudinal Reinforcement (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.8.2 Minimum Longitudinal Reinforcement	**C8.8.2**
The minimum area of longitudinal reinforcement for cast-in-place and emulative precast compression members shall not be less than: For columns in SDC B and C, $A_l \geq 0.007\, A_g$ (8.8.2-1) For columns in SDC D, $A_l \geq 0.010\, A_g$ (8.8.2-2) For pier walls in SDC B and C $A_l \geq 0.0025\, A_g$ (8.8.2-3) For pier walls in SDC D, $A_l \geq 0.005\, A_g$ (8.8.2-4) where: A_g = gross area of member cross section (in^2) A_l = area of longitudinal reinforcement in member (in^2) The minimum area of longitudinal reinforcement for hybrid compression members shall satisfy: $\frac{M_s}{M_y} \geq 0.20$ (8.8.2-5) where: M_s = flexural moment capacity provided by longitudinal reinforcement at reference yield moment (kip-ft) M_y = reference yield moment (kip-ft)	This requirement is intended to apply to the full section of the columns. The lower limit on the column or wall reinforcement for cast-in-place and emulative precast columns reflects the traditional concern for the effect of time-dependant deformations as well as the desire to avoid sizeable differences between the flexural cracking and yield moments. The requirement for hybrid systems reflects the need for a minimum amount of flexural reinforcement to facilitate the reliable prediction of the reference yield point. For hybrid systems with lesser reinforcement, significant variability in the reference yield point have been observed. Furthermore, this minimum amount of reinforcement is required for hybrid systems in order for the inelastic displacement modification factor specified in Article 4.3.3 to be valid (Tobolski*, 2010). * Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS11. Proposed Article 8.8.14—Minimum Debonded Length of Longitudinal Reinforcement for Hybrid Columns (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.8.14 Minimum Debonded Length of Longitudinal Reinforcement for Hybrid Columns Longitudinal reinforcement in hybrid columns shall be intentionally debonded from the surrounding concrete at hybrid column end connections. The minimum debonded length shall be such to ensure the strain in the longitudinal reinforcement does not exceed the reduced ultimate tensile strain specified in Article 8.4.2 at the column ultimate rotation capacity.	**C8.8.14** Localized joint opening associated with hybrid systems will cause concentrated straining in the longitudinal reinforcement during lateral response. To preclude the premature fracture of the reinforcement, intentional debonding is used to prevent tensile fracture prior to the ultimate rotation capacity.

Attachment DS12. Revised Article 8.10—Superstructure Capacity Design for Longitudinal Direction for SDC C and D (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.10 SUPERSTRUCTURE CAPACITY DESIGN FOR LONGITUDINAL DIRECTION FOR SDC C AND D

8.10.1 Superstructure Demand

The superstructure shall be designed as a capacity protected member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire width of the superstructure. The column overstrength moment, M_{po}, in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure shall be distributed to the spans framing into the bent based on their stiffness distribution factors. This moment demand shall be considered within the effective width of the superstructure.

Vertical seismic demands determined in accordance with Article 4.7.2 shall be distributed to the entire width of the superstructure. The demands associated with the column overstrength moment, M_{po}, shall be considered concurrently with vertical seismic demands as specified in Article 4.7.2.

C8.10.1

Capacity design procedures cannot be used for vertically induced seismic demands due to the lack of a designate mechanism of inelastic response. For regions subject to strong vertical accelerations, these demands must be explicitly considered in conjunction with demands generated by column flexural hinging (Veletzos, 2007; Tobolski*, 2010).

8.10.2 Superstructure Effective Width

The effective width of the superstructure resisting longitudinal seismic moments, B_{eff}, is defined in Eqs. 1 and 2:

For box girders and solid superstructures:

$$B_{eff} = D_c + 2D_s \qquad (8.10.\underline{2}\text{-}1)$$

For open soffit, girder-deck superstructures:

$$B_{eff} = D_c + D_s \qquad (8.10.\underline{2}\text{-}2)$$

where:

D_c = diameter of column (in.)

D_s = depth of superstructure (in.)

C8.10.2

The effective width for open soffit superstructures (i.e. T-Beams and I Girders) is reduced because they offer less resistance to the torsional rotation of the bent cap. The effective superstructure width can be increased at a 45° angle away from the bent cap until the full section becomes effective. On skewed bridges, the effective width shall be projected normal to the girders where the centerline of the girder intersects the face of the bent cap. (see Figure C1).

Additional superstructure width can be considered effective if the designer verifies that the torsional stiffness of the cap can distribute the rotational demands beyond the effective widths stated in Eqs. 1 and 2.

8.10.3 Minimum Superstructure Rotation Capacity

The superstructure to bent-cap connection shall have plastic rotation capacity equal to or greater than 0.01 radians. The plastic rotation capacity shall be

C8.10.3

Seismically induced ground motions can generate demands that are greater than those considered or not considered in design. These demands can be due to calculated using the moment-curvature analysis required per Article 8.9 and the analytical plastic hinge length for superstructures as defined in Article 4.11.6.

vertical ground motions which cannot be considered via capacity design procedures, or due to seismically induced relative settlements between bents. These seismic demands may result in superstructure loading in excess of the flexural capacity. In order to prevent superstructure failure, a minimum level of inelastic rotation capacity is desirable to prevent structural collapse or loss of life (Tobolski*, 2010).

8.10.4 Torsional Design for Open Soffit Superstructures

The transfer of column overstrength moment, M_{po}, and associated shear and axial load via torsional mechanisms must be explicitly considered in the superstructure design for open soffit structures.

C8.10.4

For open soffit superstructures, column flexural demands must be transferred into the superstructure completely through torsional response of the bent cap and superstructure system. Traditional torsional mechanisms cannot develop due to the short distance between the face of column and girder. The torsional plastic shear friction mechanism presented in the PCI Bridge Design Manual is one acceptable method for torsional design (PCI, 2004).

8.10.5 Shear Design for Integral Precast Superstructures

The bottom flange of integral precast girders shall be reinforced with closed hoops within the region from the face of bent cap equal to a distance equal to the superstructure depth. These hoops shall be spaced with the girder shear reinforcement, with spacing not to exceed 8 in. The hoops shall be the same size as the girder shear reinforcement with a minimum bar size of No. 4.

For integral precast superstructures with girders discontinuous at the face of the bent cap, headed shear reinforcement shall be placed within a distance from the face of the bent cap equal to 1.75 times the neutral axis depth at nominal capacity as determined in accordance with Article 8.9. The headed shear reinforcement within this distance shall be capable of resisting the factored shear demand including effects of vertical seismic loading in accordance with Article 4.7.2. The shear demand shall be calculated considering the direction of loading and shears generated during positive flexural loading of the superstructure. This reinforcement shall extend as close to the top of deck as possible while maintaining required concrete cover dimensions.

C8.10.5

The use of closed hoops at the end of precast girders in integral systems is aimed at providing integrity and robustness in the event of inelastic rotation demands. The integrity of the bottom flange is essential to the overall structural performance and should be protected from degradation through a minimal level of confinement (Tobolski*, 2010).

Discontinuous precast superstructures are expected to experience localized superstructure rotations at the joint when subjected to positive post-yield flexural demands. This localized rotation will result in discrete joint opening during post-yield loading which may occur due to relative settlement or vertical loading. To ensure a stable shear resistance mechanism is present, a shear transfer mechanism considering the reduction in shear depth with joint opening is required. The shear requirement is based on a shear transfer mechanism considering a maximum 30° compression strut angle (Tobolski*, 2010).

* Note: Reference to Tobolski PhD dissertation will be replaced by refereed journal publication upon acceptance.

Attachment DS13. Proposed Article 8.13—Joint Design for SDC A (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.13—JOINT DESIGN FOR SDC A

Interior joints of multicolumn bents shall be considered “T” joints for joint shear analysis. Exterior joints shall be considered knee joints and require special analysis and detailing that are not addressed herein, unless special analysis determines that “T” joint analysis is appropriate for an exterior joint based on the actual bent configuration.

C8.13

This Article addresses joint design of cast-in-place, emulative precast and hybrid bent caps that are connected to cast-in-place columns, precast columns, or prestressed piles in SDC A.

Owners need to establish appropriate design and detailing provisions for exterior joints. Efforts are currently underway to develop provisions for the AASHTO Guide Specifications.

8.13.1—Joint Performance

Moment-resisting connections shall be designed to transmit the unreduced elastic seismic forces in columns.

C8.13.1

Bridges designed for SDC A are expected to be subjected to only minor seismic displacements and forces; therefore, a force-based approach is specified to determine unreduced elastic seismic forces, in lieu of a more rigorous displacement-based analysis. (Articles 4.1 and 4.2).

However, some SDC A bridges may be exposed to seismic forces that may induce limited inelasticity, particularly in the columns. For this reason, Article 8.2 states that, when S_{D1} is greater than or equal to 0.10 but less than 0.15, minimum column shear reinforcement shall be provided according to the requirements of Article 8.6.5 for SDC B subject to Article 8.8.9 for the length over which this reinforcement is to extend. Although Article 8.8.9 does not specify placement of transverse column reinforcement into the joint, Articles 5.10.11.4.1e and 5.10.11.4.3 of the *AASHTO LRFD Bridge Design Specifications* referenced by the alternative provisions in Articles 8.2 and 8.8.9 specify placement of transverse reinforcement into the joint for a distance not less than one-half the maximum column dimension or 15.0 in. from the face of the column connection into the adjoining member.

Therefore, according to these alternative provisions, when S_{D1} is greater than or equal to 0.10 but less than 0.15, minimum transverse reinforcement is required for cast-in-place joints. When S_{D1} is less than 0.10, transverse joint shear reinforcement is not required for cast-in-place joints.

Precast bent cap connections for SDCs B, C, and D are designed and detailed to provide sufficient reinforcement for force transfer through the joint and bent cap. The precast bent cap design provisions for SDC A, including minimum joint reinforcement, are more liberal than those for SDC B, but are deemed appropriate for all values of S_{D1} in SDC A.

When S_{D1} is less than 0.10, alternative nonseismic precast bent cap connections may be used, such as those detailed in Matsumoto et al. (2001) and Brenes et al. (2006). However, it is recommended that alternative connection details include minimum joint transverse reinforcement and minimum joint stirrups. In addition, column longitudinal reinforcement should be extended into the connection as close as practically possible to the opposite face of the bent cap.

8.13.2—Joint Proportioning

C8.13.2

For SDC A, joint proportioning of bent caps requires the designer to provide sufficient length to anchor column longitudinal reinforcement in the joint.

8.13.2.1—Cast-in-place Connections

To be completed by others.

8.13.2.2—Precast Bent Cap Connections

The provisions of Article 8.15.2.2.2 for anchorage of column longitudinal reinforcement shall be used for joint proportioning, except that the nominal yield stress of the column longitudinal reinforcement shall be permitted to be used in lieu of the expected yield stress.

C8.13.2.2

Minimum development length provisions under Article 8.15.2.2 for column longitudinal reinforcement anchored in precast bent cap connections are appropriate with minor modification.

8.13.3—Minimum Joint Shear Reinforcing

8.13.3.1—Cast-in-place Connections

To be completed by others.

8.13.3.2—Precast Connections

C8.13.3.2

Minimum transverse reinforcement in the joint is required to help in the force transfer mechanism and ensure that the connection does not become a weak link in a precast bent system.

8.13.3.2.1—Grouted Duct Connection

Transverse reinforcement in the form of tied column reinforcement, spirals, hoops, or intersecting spirals or hoops shall be provided. Minimum transverse reinforcement in the joint for grouted duct connections shall satisfy Equation 8.15.3.1-1 of Article 8.15.3.1. Spacing of transverse reinforcement shall not exceed $0.3D_s$ or 12 in.

8.13.3.2.2—Cap Pocket Connection

Cap pocket connections shall use a helical, lock-seam, corrugated steel pipe conforming to ASTM A 760 to form the bent cap pocket. A minimum thickness of corrugated steel pipe shall be used to satisfy the transverse reinforcement ratio requirement. The thickness of the steel pipe, t_{pipe}, shall satisfy

C8.13.3.2.2

Article C8.15.3.2.2 discusses minimum pipe thickness requirements.

The following simplified equation may be conservatively used in lieu of satisfying Eq. 8.15.3.2.2-1.

Eq. 8.15.3.2.2-1.

$$t_{pipe} \geq \max \begin{cases} 0.04 \dfrac{\sqrt{f'_c} D'_{cp}}{f_{yp} \cos\theta} \\ 0.06\, in. \end{cases} \qquad \text{C8.13.3.2.2-1}$$

where:

f'_c = nominal compressive strength of cap pocket concrete (ksi)

D'_{cp} = average diameter of confined cap pocket fill between corrugated steel pipe walls (in.)

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

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

This simpler equation may result in a thicker pipe size in design than that obtained using Eq. 8.15.3.2.2-1.

8.13.4—Nonintegral Bent Cap Joint Shear Design

C8.13.4

The design of nonintegral cast-in-place bent cap joints is summarized in Sritharan (2005). The design of nonintegral emulative precast bent cap joints is summarized in Matsumoto (2009).

8.13.4.1—Cast-in-Place Connections

To be completed by others.

8.13.4.2—Precast Connections

C8.13.4.2

Minimum vertical joint stirrups are required to limit opening of potential joint cracks, help in the force transfer mechanism, and ensure that the connection does not become a weak link in a precast bent system.

8.13.4.2.1—Bedding Layer Reinforcement

Bedding layers between columns and precast bent caps shall be reinforced with transverse reinforcement, as shown in Figure 8.13.4.2.2-1 and Figure 8.13.4.2.3-1. Provisions of Article 8.15.5.2.1 shall be satisfied.

C8.13.4.2.1

Article C8.15.5.2.1 provides an explanation of the bedding layer and reinforcement.

8.13.4.2.2—Grouted Duct Connection

Grouted duct connections shall be reinforced in accordance with the requirements of Article 8.13.4.2.2a and shall be detailed as shown in Figure 1 through Figure 3. Details of the connection include ducts, vertical stirrups inside the joint, and bedding layer reinforcement, in addition to the joint transverse reinforcement.

C8.13.4.2.2

Article C8.15.5.2.2 provides a description of the grouted duct connection.

8.13.4.2.2a—Vertical Stirrups Inside the Joint Region Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over a length, D_c, through the joint shall satisfy: $A_s^{jvi} \geq 0.08A_{st}$ (8.13.4.2.2a-1) where: A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2}) Vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used.	*C8.13.4.2.2a* Research has demonstrated that a limited number of vertical stirrups distributed within the joint region of a grouted duct connection assists in the joint force transfer mechanism and limits opening of joint cracks, helping ensure that the connection does not become a weak link in a precast bent system.

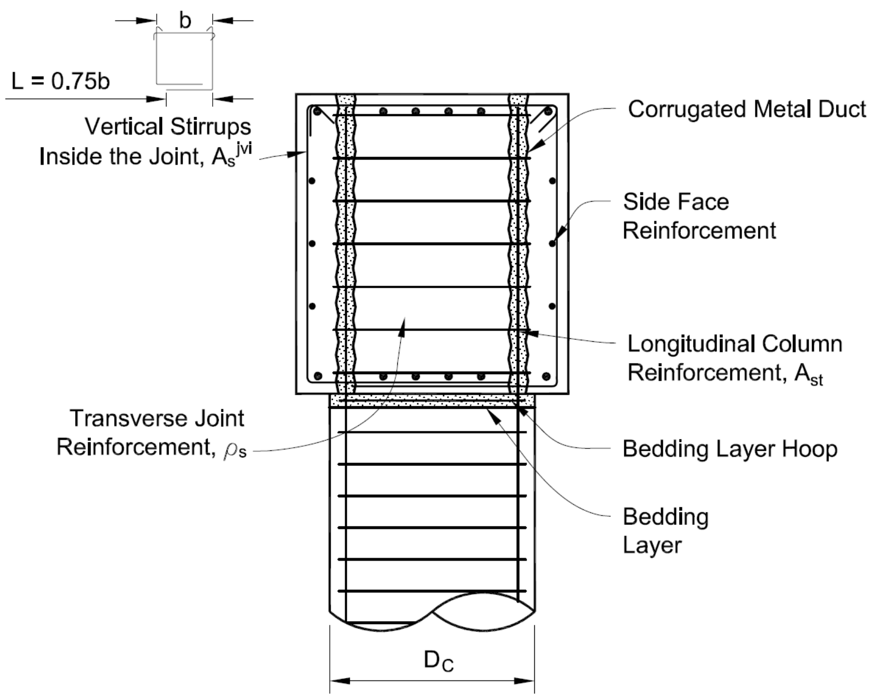


Figure 8.13.4.2.2-1—Grouted Duct Joint Shear Reinforcement Details (Section)—SDC A

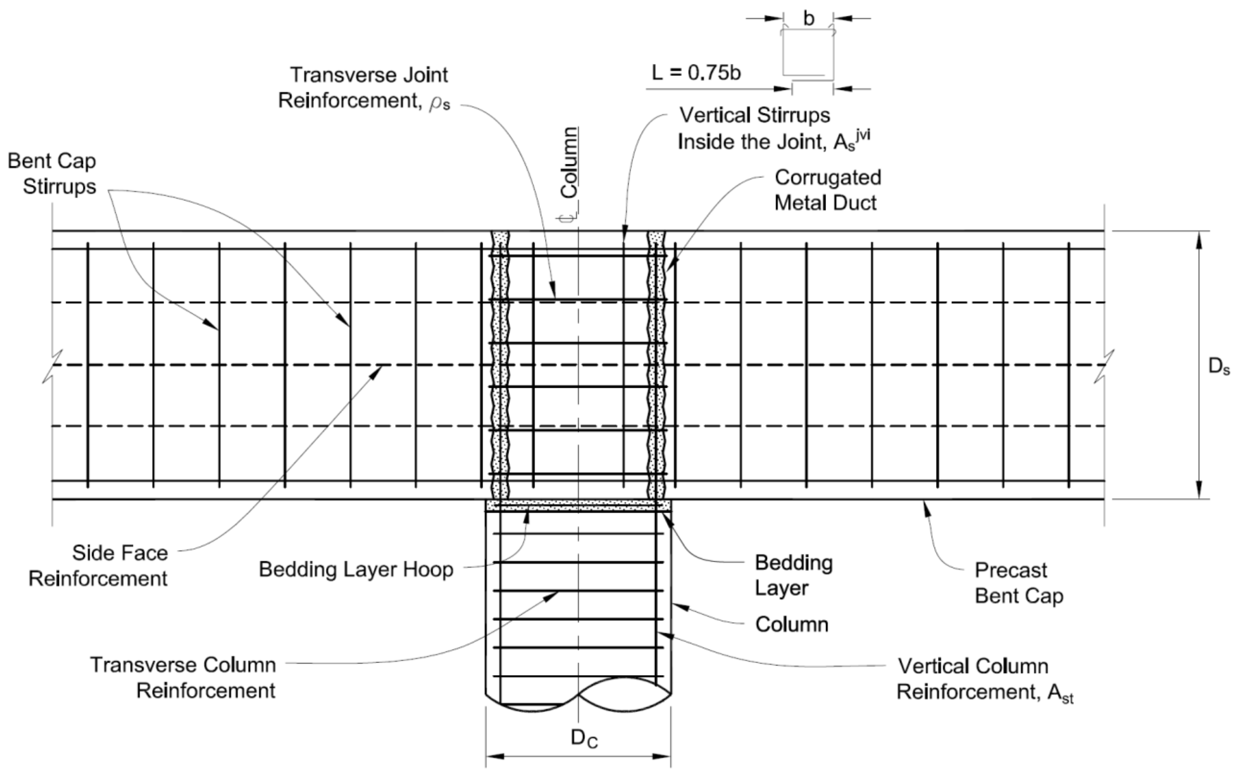
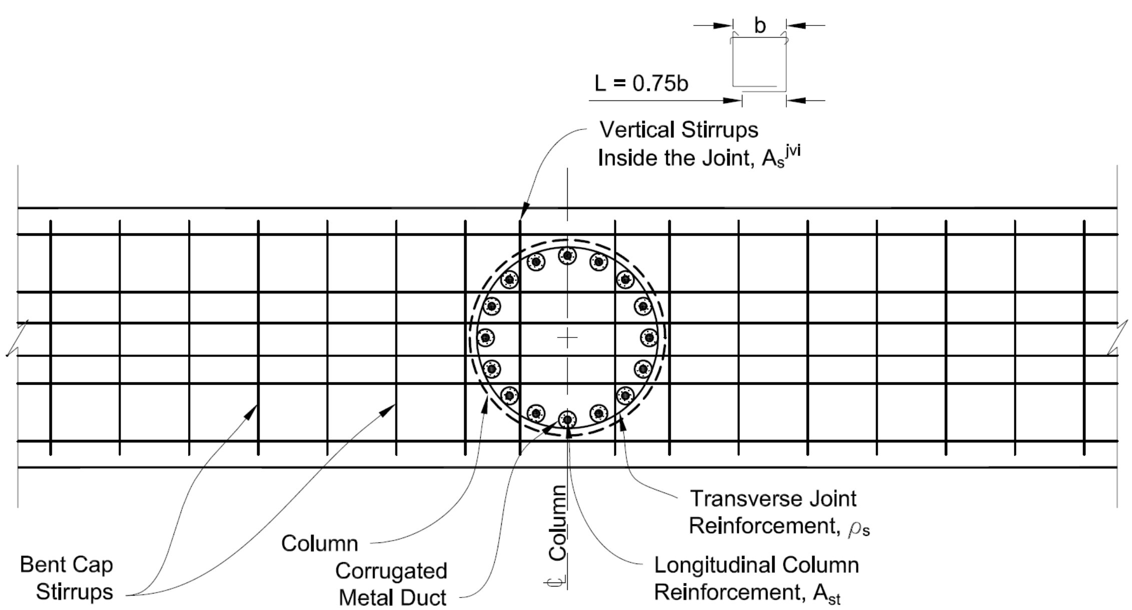


Figure 8.13.4.2.2-2—Grouted Duct Joint Shear Reinforcement Details (Elevation)—SDC A



Note: "b" is equal to the cap width minus clear cover

Figure 8.13.4.2.2-3—Grouted Duct Joint Shear Reinforcement Details (Plan)—SDC A

8.13.4.2.3—Cap Pocket Connection	C8.13.4.2.3
	C8.15.5.2.3 provides a description of the cap pocket connection.
8.13.4.2.3a—Joint Shear Reinforcement	*C8.13.4.2.3a*
Cap pocket connections shall be detailed to include vertical stirrups inside the joint, as follows: Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over a length, D_c, through the joint shall satisfy: $A_s^{jvi} \geq 0.08A_{st}$ (8.13.4.2.3a-1) where: A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})	
Vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used. Figure 1 through Figure 3 show details of the connection, including the steel pipe, vertical stirrups inside the joint, and bedding layer reinforcement.	Due to the presence of the pipe, use of overlapping double-leg vertical stirrups in the joint is not practical.

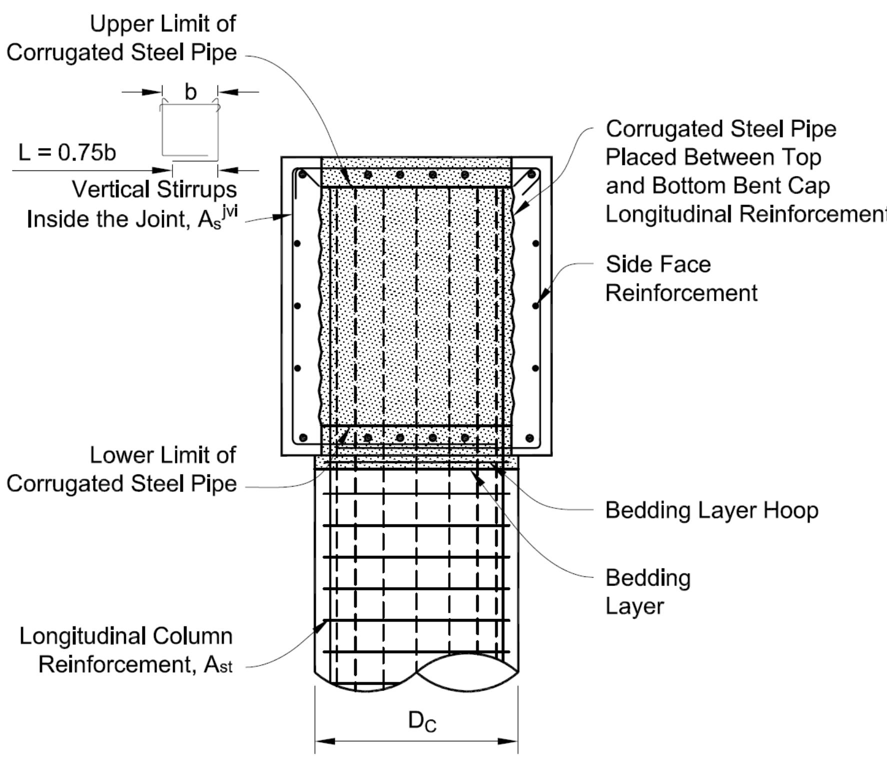


Figure 8.13.4.2.3-1—Cap Pocket Joint Shear Reinforcement Details (Section)—SDC A

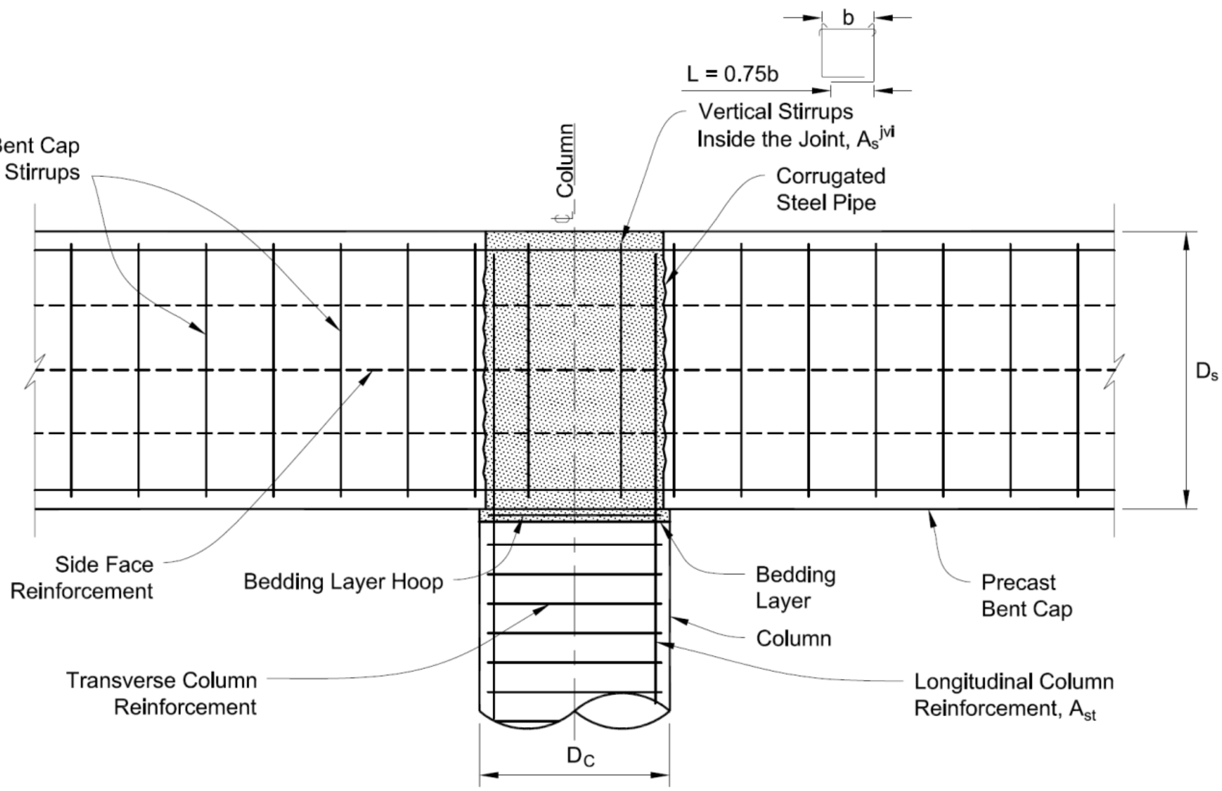


Figure 8.13.4.2.3-2— Cap Pocket Joint Shear Reinforcement Details (Elevation)—SDC A

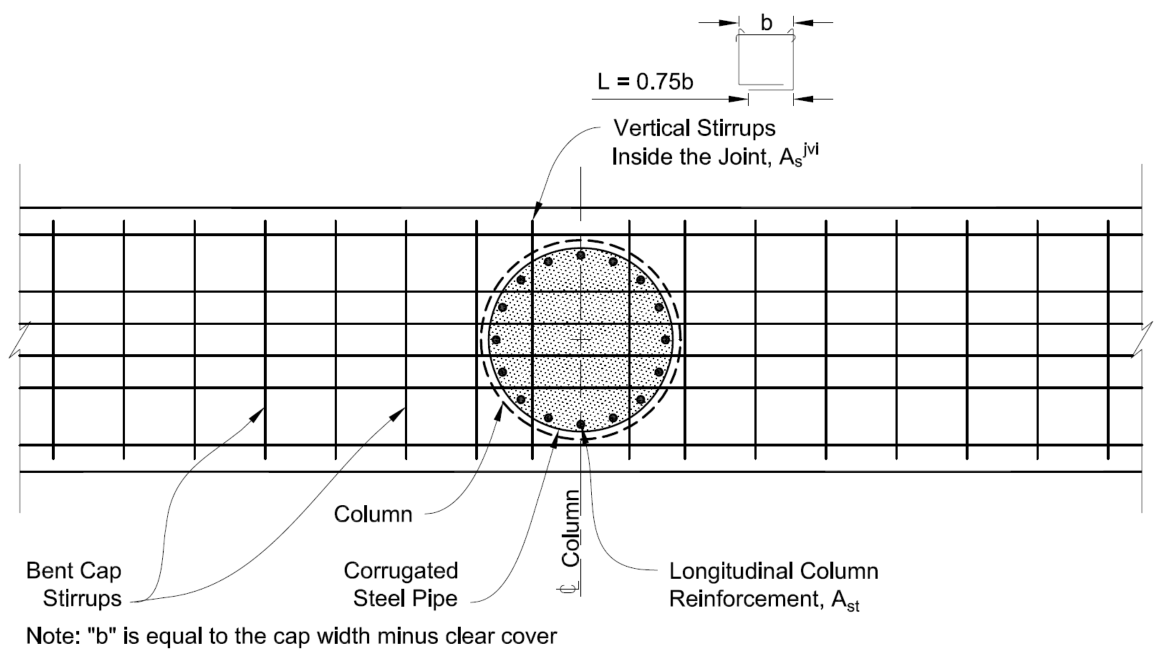


Figure 8.13.4.2.3-3— Cap Pocket Joint Shear Reinforcement Details (Plan)—SDC A

Attachment DS14. Proposed Article 8.14—Joint Design for SDC B
(AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.14—JOINT DESIGN FOR SDC B

Interior joints of multicolumn bents shall be considered "T" joints for joint shear analysis. Exterior joints shall be considered knee joints and require special analysis and detailing that are not addressed herein, unless special analysis determines that "T" joint analysis is appropriate for an exterior joint based on the actual bent configuration.

C8.14

This Article addresses joint design of cast-in-place, emulative precast and hybrid bent caps that are connected to cast-in-place columns, precast columns, or prestressed piles in SDC B.

Owners need to establish appropriate design and detailing provisions for exterior joints. Efforts are currently underway to develop provisions for the AASHTO Guide Specifications.

8.14.1—Joint Performance

Moment-resisting connections shall be designed to transmit the unreduced elastic seismic forces in columns where the column moment does not reach the plastic moment, M_p, and shall be designed to transmit the column forces associated with the column overstrength capacity, M_{po}, where the plastic moment, M_p is reached.

C8.14.1

Bridges designed for SDC B may be subjected to seismic forces that can cause yielding of the columns and limited plastic hinging. According to Articles 4.11.1 and C4.11.1, joint shear checks and full capacity design using plastic overstrength forces are not required. This practice is adopted for cast-in-place joints. However, the owner may also choose to implement the more conservative capacity-protection requirements of Article 8.9.

Precast bent cap connections adopt the more conservative provisions mentioned in Articles 4.11.1 and C4.11.1, i.e., apply explicit capacity checks to ensure that no weak link exists in the Earthquake Resisting System. This is achieved by implementing the provisions of Articles 8.14.2, 8.14.3, and 8.14.5. These provisions help ensure that joints accommodate forces in an essentially elastic manner.

Based on Article 8.3.2, Article 8.14.1 requires that connections be designed to transmit the forces associated with the column overstrength capacity, M_{po}, when the column elastic seismic moment reaches the plastic moment, M_p. This recognizes that significant plastic hinging may potentially develop in such cases.

8.14.2—Joint Proportioning

The provisions of Article 8.15.2 shall be used for joint proportioning, except that the design moment to be used in Eq. 8.15.2.1-10 shall be that determined from Article 8.14.1.

C8.14.2

As mentioned in th Article C8.3.2, SDC B structures are designed and detailed to achieve a displacement ductility, μ_D, of at least 2. In contrast to cast-in-place joint design, precast joints in SDC B are more conservatively proportioned based on a check of principal stress levels.

8.14.3—Minimum Joint Shear Reinforcing for Precast Bent Cap Connections

For precast bent cap connections, the provisions of Article 8.15.3.2 shall be used for determining minimum joint shear reinforcing.

Other joint reinforcing shall be provided in

C8.14.3

In contrast to cast-in-place joint design, precast connections require minimum joint shear reinforcing. In addition, a rational design procedure is required for the additional joint shear reinforcement when the principal accordance with Article 8.14.5.2.

tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$ as stipulated in Article 8.14.5.2.

8.14.4—Integral Bent Cap Joint Shear Design

Where the principal tension stress in the joint, p_t, as specified in Article 8.15.2.1 is greater than or equal to $0.11\sqrt{f_c'}$, then joint shear reinforcement shall satisfy the requirement for SDC C and D bridges as specified in Article 8.15.

8.14.5—Nonintegral Bent Cap Joint Shear Design

Cast-in-place, emulative precast and hybrid bent caps shall satisfy Eq. 8.15.5-1 or be designed on the basis of the strut and tie provisions of the *AASHTO LRFD Bridge Design Specifications* and as approved by the Owner.

8.14.5.1—Cast-in-Place Connections

To be completed by others.

8.14.5.2—Precast Connections

C8.14.5.2

The design of nonintegral cast-in-place bent cap joints is summarized in Sritharan (2005). The design of nonintegral emulative precast bent cap joints is summarized in Matsumoto (2009). Articles 8.15.5.2 and C8.15.5.2 provide related design provisions and background on nonintegral bent cap systems using grouted duct or cap pocket connections.

8.14.5.2.1—Bedding Layer Reinforcement

Bedding layers between columns and precast bent caps shall be reinforced with transverse reinforcement, as shown in Figure 8.14.5.2.2-1 and Figure 8.14.5.2.3-1. Provisions of Article 8.15.5.2.1 shall be satisfied.

C8.14.5.2.1

Article C8.15.5.2.1 provides an explanation of the bedding layer and its reinforcement.

8.14.5.2.2—Grouted Duct Connection

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, the provisions of Article 8.15.5.2.2 shall be satisfied.

Where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$, grouted duct connections shall be reinforced in accordance with Article 8.14.5.2.2a and shall be detailed as shown in Figure 1 through Figure 3. Details of the connection include ducts, vertical stirrups inside the joint, and bedding layer reinforcement, in addition to the joint transverse reinforcement.

C8.14.5.2.2

Article C8.15.5.2.2 provides a description of the grouted duct connection.

The more conservative provisions of Article 8.15.5.2.2 may be optionally used when an additional margin of safety in design is desired, even where principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$.

8.14.5.2.2a—Vertical Stirrups Inside the Joint Region

C8.14.5.2.2a

Research has demonstrated that a limited number of

Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over a length, D_c, through the joint shall satisfy: $A_s^{jvi} \geq 0.10 A_{st}$ (8.14.5.2.2a-1) where: A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2}) Vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used.	vertical stirrups distributed within the joint region of a grouted duct connection assists in the joint force transfer mechanism and limits opening of joint cracks, helping ensure that the connection does not become a weak link in a precast bent system.

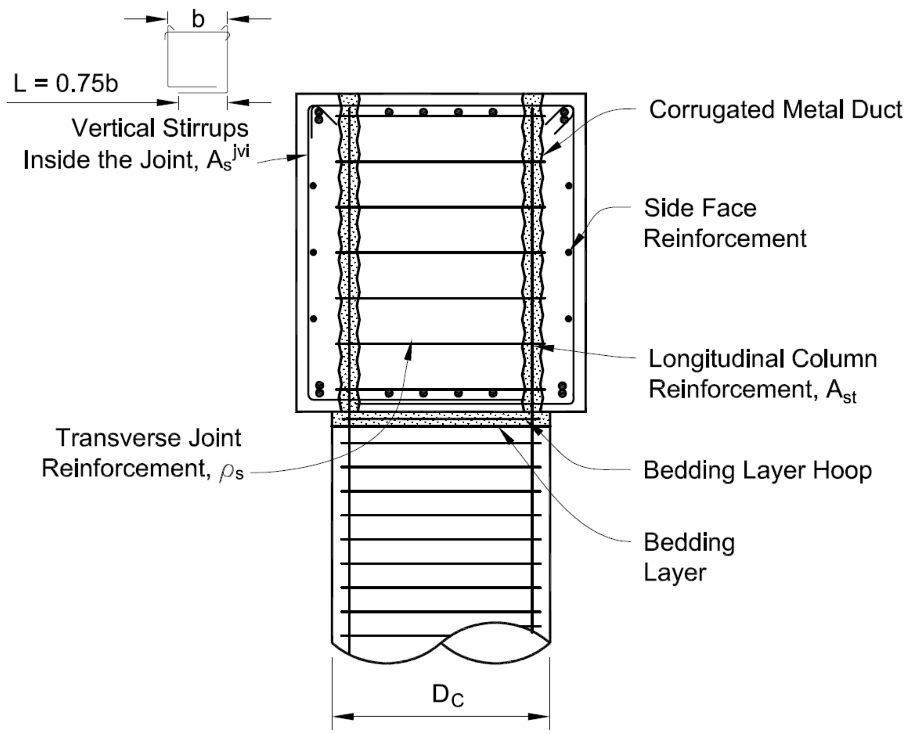


Figure 8.14.5.2.2-1—Grouted Duct Joint Shear Reinforcement Details (Section)—SDC B

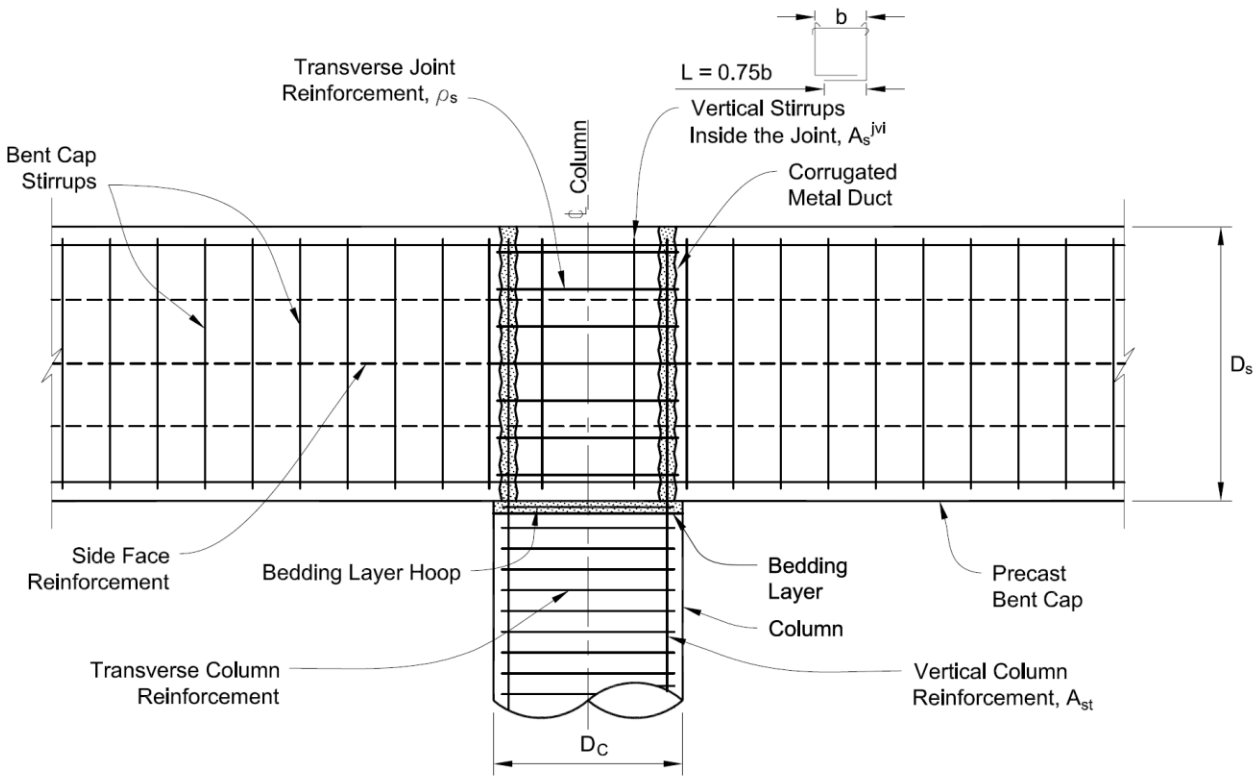


Figure 8.14.5.2.2-2—Grouted Duct Joint Shear Reinforcement Details (Elevation)—SDC B

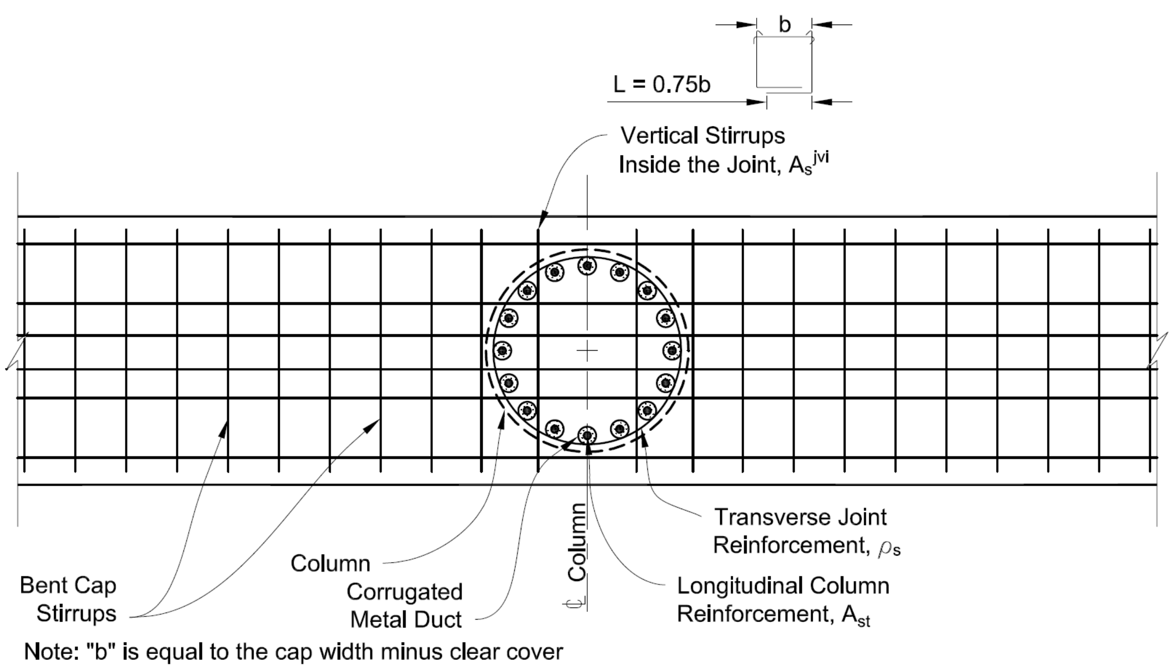


Figure 8.14.5.2.2-3—Grouted Duct Joint Shear Reinforcement Details (Plan)—SDC B

8.14.5.2.3—Cap Pocket Connection

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, the provisions of Article 8.15.5.2.3 shall be satisfied.

Where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$, the provisions of Article 8.14.5.2.3a and Article 8.14.5.2.3b shall be satisfied.

C8.14.5.2.3

C8.15.5.2.3 provides a description of the cap pocket connection.

The more conservative provisions of Article 8.15.5.2.3 may be optionally used when an additional margin of safety in design is desired, even where principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$.

8.14.5.2.3a—Joint Reinforcement

Cap pocket connections shall be reinforced with vertical stirrups inside the joint in accordance with the following:

Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over a length, D_c, through the joint shall satisfy:

$$A_s^{jvi} \geq 0.10A_{st} \tag{8.14.5.2.3a-1}$$

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

Vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used.

Joints shall be detailed as shown in Figure 1 through Figure 3. Details of the connection include the steel pipe, vertical stirrups inside the joint, and bedding layer reinforcement.

C8.14.5.2.3a

Research has demonstrated that limited vertical stirrups within the joint are required for SDC B cap pocket connections. These stirrups limit potential opening of joint cracks and help ensure sufficient displacement ductility capacity. C8.15.5.2.3a summarizes constructability issues and the use of double-leg vertical stirrups.

8.14.5.2.3b—Supplementary Hoops

A supplementary hoop shall be permitted to be placed one inch from each end of the corrugated pipe. The bar size of the hoop shall match the size of the bedding layer reinforcement required by Article 8.15.5.2.1.

C8.14.5.2.3b

A supplementary hoop may be optionally placed at each end of the pipe to limit dilation and potential unraveling. This reinforcement may be included as a simple, inexpensive, and conservative measure.

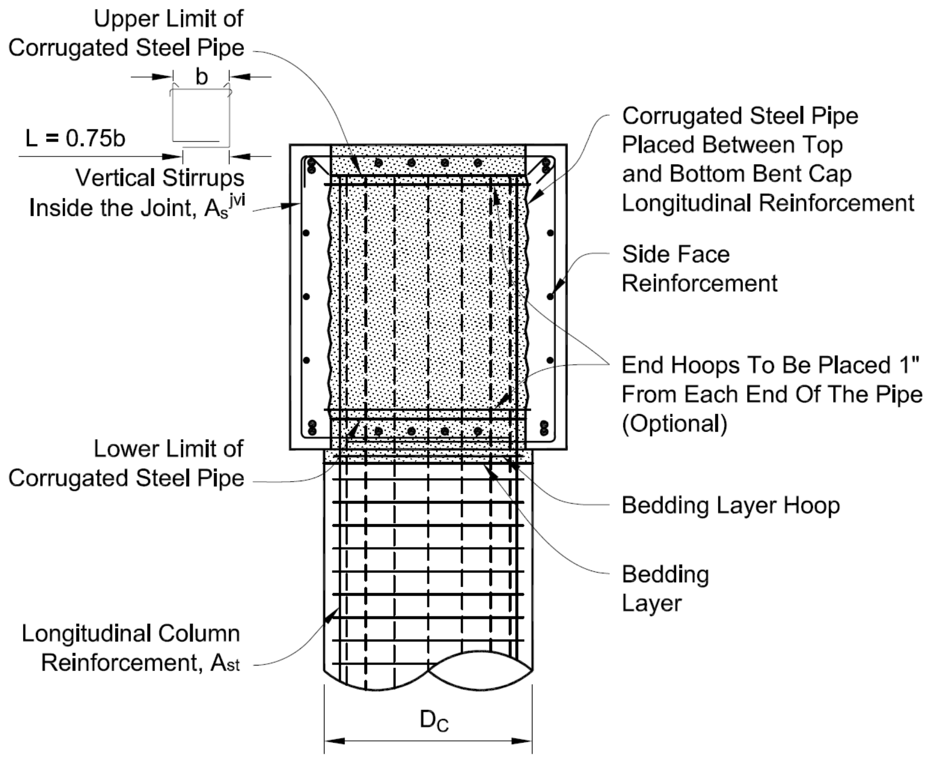


Figure 8.14.5.2.3-1—Cap Pocket Joint Shear Reinforcement Details (Section)—SDC B

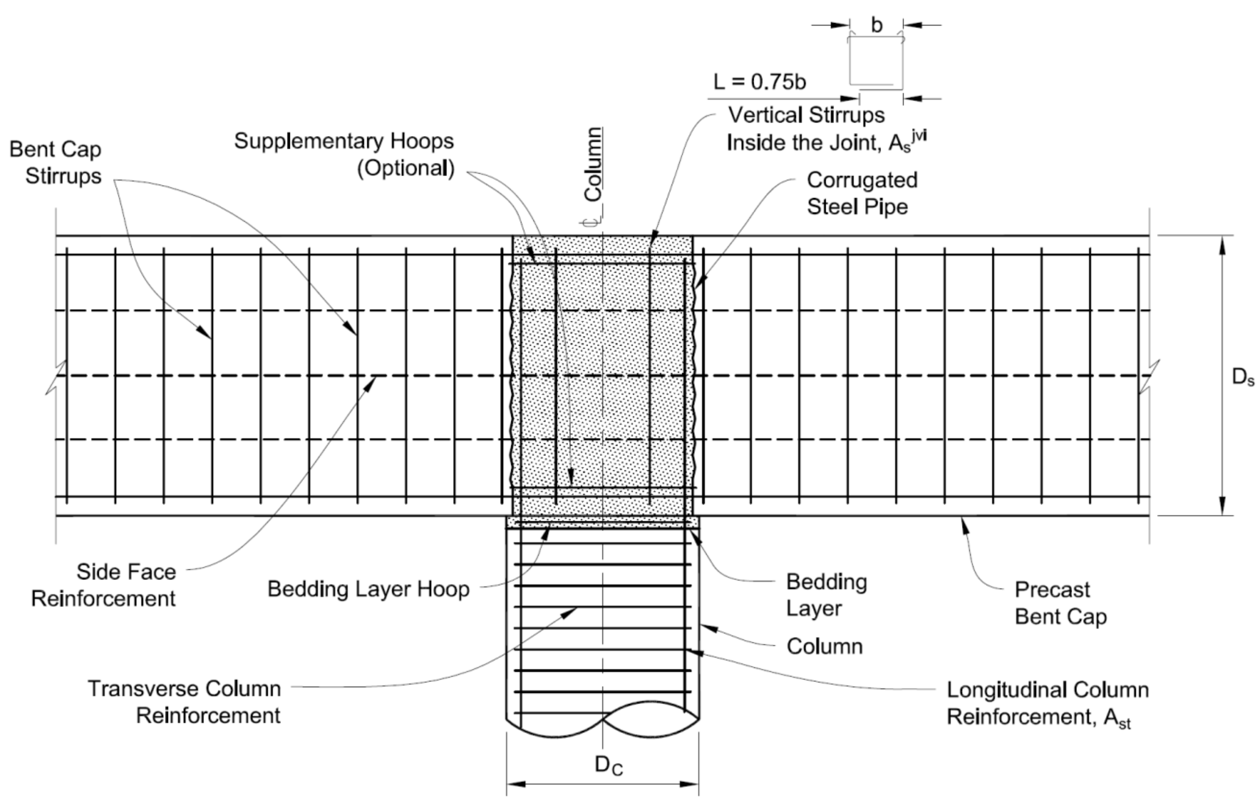


Figure 8.14.5.2.3-2— Cap Pocket Joint Shear Reinforcement Details (Elevation)—SDC B

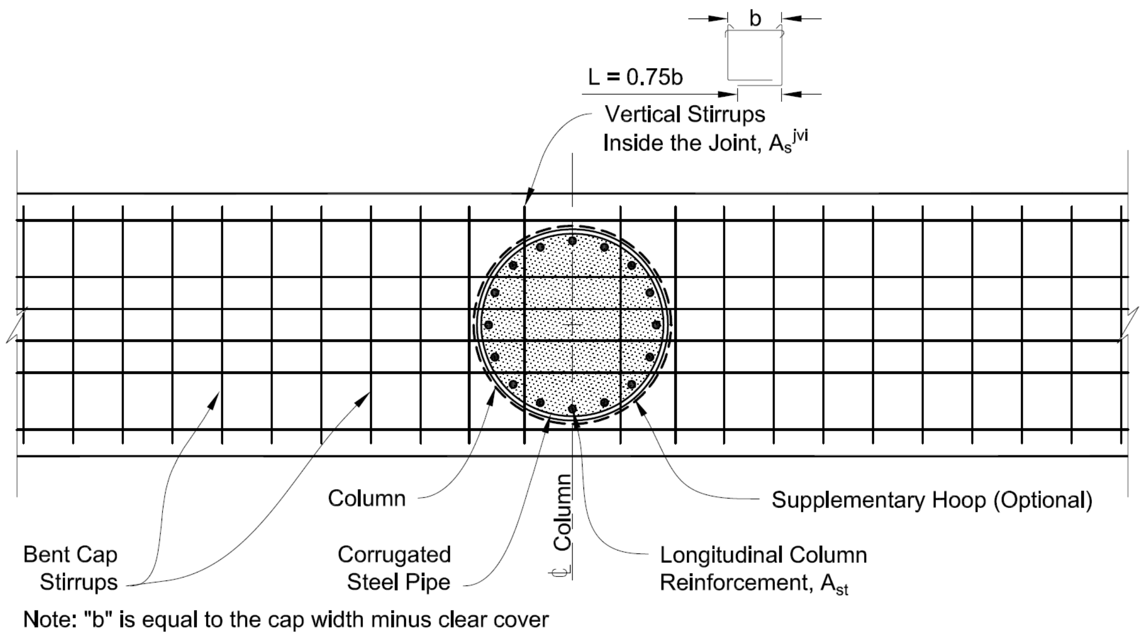


Figure 8.14.5.2.3-3— Cap Pocket Joint Shear Reinforcement Details (Plan)—SDC B

Attachment DS15. Revised Article 8.15—Joint Design for SDCs C and D (AASHTO Guide Specifications for LRFD Seismic Bridge Design, 1st Ed., 2009)

8.~~13~~15—JOINT DESIGN FOR SDCS C AND D

Interior joints of multicolumn bents shall be considered "T" joints for joint shear analysis. Exterior joints shall be considered knee joints and require special analysis and detailing that are not addressed herein, unless special analysis determines that "T" joint analysis is appropriate for an exterior joint based on the actual bent configuration.

C8.15

This Article addresses joint design of cast-in-place emulative precast, and hybrid bent caps that are connected to cast-in-place columns, precast columns, or prestressed piles in SDCs C and D. Both integral connections (girder-to-bent cap) and nonintegral connections (bent cap-to-column) are addressed. Columns are defined broadly to include cast-in-place columns, precast columns, and prestressed piles.

Owners need to establish appropriate design and detailing provisions for exterior joints. Efforts are currently underway to develop provisions for the AASHTO Guide Specifications.

8.~~13~~15.1—Joint Performance

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

C8.~~13~~15.1

A "rational" design is required for joint reinforcement when principal tension stress levels become excessive. The amounts of reinforcement required are based on a strut and tie mechanism similar to that shown in Figure C1.

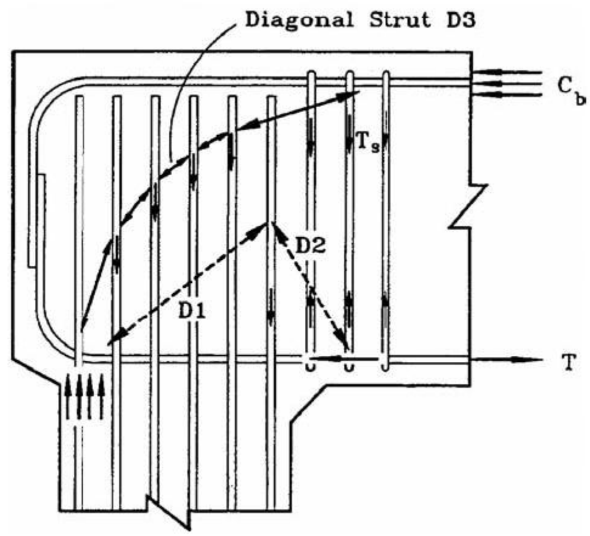


Figure C8.~~13~~15.1-1—External Vertical Joint Reinforcement for Joint Force Transfer

8.~~13~~15.2—Joint Proportioning

C8.~~13~~15.2

This section addresses two provisions necessary for proportioning bent caps: 1) cross sectional dimensions to satisfy limits on principal tension and compression stresses, and 2) sufficient length to anchor column longitudinal reinforcement in the joint.

8.15.2.1—Principal Stress Requirements

Moment-resisting joints shall be proportioned so that the principal stresses satisfy the requirements of Eq. 1 and Eq. 2.

- For principal compression, p_c:

$$p_c \le 0.25 f'_c \qquad (8.\cancel{13}\underline{15}.2.1\text{-}1)$$

- For principal tension, p_t:

$$p_t \le 0.38\sqrt{f'_c} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}2)$$

in which:

$$P_t = \left|\left(\frac{f_h + f_v}{2}\right) - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2}\right| \qquad (8.\cancel{13}\underline{15}.2.1\text{-}3)$$

$$P_c = \left(\frac{f_h + f_v}{2}\right) + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}4)$$

$$v_{jv} = \frac{T_c}{A_{jv}} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}5)$$

$$A_{jv} = l_{ac} B_{cap} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}6)$$

$$f_v = \frac{P_c}{A_{jh}} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}7)$$

$$A_{jh} = (D_c + D_s) B_{cap} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}8)$$

$$f_h = \frac{P_b}{B_{cap} D_s} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}9)$$

$$T_c = \frac{M_{po}}{h} \qquad (8.\cancel{13}\underline{15}.2.1\text{-}10)$$

where:

B_{cap} = bent cap width (in.)

D_c = cross-sectional dimension of column in the direction of bending (in.)

D_s = depth of superstructure at the bent cap for integral joints or depth of cap beam for nonintegral bent caps (in.)

l_{ac} = length of column reinforcement embedded into

C8.15.2.1

Figure C1 illustrates the forces acting on the joint as well as the associated principal stresses.

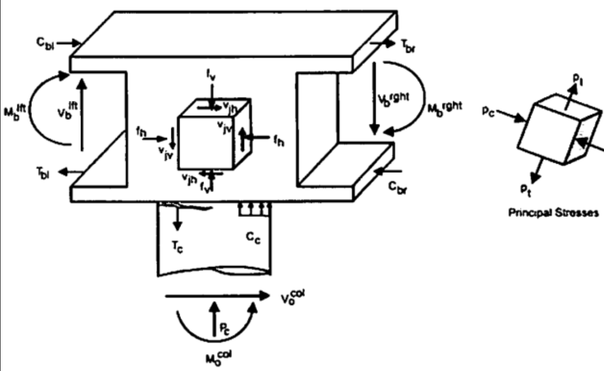


Figure C8.~~13~~15.2.1-1—Stress in T-Joints

The substitution of f'_{ce} for f'_c throughout Article 8.~~13~~15 may be acceptable provided that historic concrete test data and the Owner's approval support this action.

Unless a horizontal prestressing force is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly affecting the principal stress calculation.

the bent cap (in.)

P_c = column axial force including the effects of overturning and column post-tensioning anchored near the top of the bent cap (kips)

P_b = beam axial force at the center of the joint including the effects of prestressing and the shear associated with plastic hinging (kips)

h = distance from the center of gravity of the tensile force to the center of gravity of the compressive force of the column section (in.)

T_c = column tensile force associated with the column overstrength plastic hinging moment, M_{po} (kips)

M_{po} = overstrength plastic moment capacity of column determined in accordance with Article 8.5 (kip-in.)

In lieu of Eq. 10, T_c may be obtained directly from the moment-curvature analysis.

8.15.2.2—Minimum Anchorage Length for Column Longitudinal Reinforcement

C8.15.2.2

Column longitudinal bars should be extended into joints a sufficient depth to ensure the bars can achieve approximately 1.4 times the expected yield strength of the reinforcement, a level associated with extensive plastic hinging and strain hardening of the bars to their expected tensile strength.

8.15.2.2.1—Cast-in-place Connections

Column longitudinal reinforcement shall satisfy the anchorage length requirements of Article 8.8.4.

8.15.2.2.2—Precast Bent Cap Connections

Column longitudinal reinforcement shall be extended into precast bent caps as close as practically possible to the opposite face of the bent cap.

For grouted duct connections, the anchorage length for longitudinal column bars developed into the precast bent cap shall satisfy:

$$l_{ac} \geq \frac{2 d_{bl} f_{ye}}{f'_{cg}} \qquad (8.15.2.2.2\text{-}1)$$

where:

l_{ac} = anchored length of longitudinal column reinforcing bars into the precast bent cap (in.)

C8.15.2.2.2

Research on anchorage of rebar in grouted ducts indicates that Equation 8.15.2.2.2-1 is conservative for seismic applications. Equation 8.15.2.2.2-2 can similarly be used for cap pocket connections. (Matsumoto et al., 2008; Matsumoto, 2009) Limitations are imposed on grout or concrete fill compressive strength and longitudinal column bar diameter, based on test data. Where the specified compressive strength of the grout or concrete fill exceeds 7000 psi, Eq. 8.15.2.2.2-1 and Eq. 8.15.2.2.2-2 should use 7000 psi.

Compressive strength of the grout for grouted duct connections and compressive strength of the concrete fill for cap pocket connections should be based on the requirements of Article 8.13.8.3 of the *AASHTO LRFD Bridge Construction Specifications* to ensure the grout

d_{bl} = diameter of longitudinal column reinforcement (in.)

f_{ye} = expected yield stress of longitudinal column reinforcement (ksi)

f'_{cg} = nominal compressive strength of grout (cube strength) (ksi)

Maximum grout compressive strength used in Eq. 8.15.2.2.2-1 shall be limited to 7000 psi. The longitudinal column reinforcement shall be limited to # 11 reinforcing bars or smaller.

For cap pocket connections, the anchorage length for longitudinal column bars developed into the precast bent cap shall satisfy:

$$l_{ac} \geq \frac{2.3 d_{bl} f_{ye}}{f'_c} \qquad (8.15.2.2.2\text{-}2)$$

where:

l_{ac} = anchored length of longitudinal column reinforcing bars into the precast bent cap (in.)

d_{bl} = diameter of longitudinal column reinforcement (in.)

f_{ye} = expected yield stress of longitudinal column reinforcement (ksi)

f'_c = nominal compressive strength of cap pocket concrete fill (ksi)

Maximum compressive strength of the cap pocket concrete fill used in Eq. 8.15.2.2.2-2 shall be limited to 7000 psi. The longitudinal column reinforcement shall be limited to # 11 reinforcing bars or smaller.

8.~~13~~15.3—Minimum Joint Shear Reinforcing

8.15.3.1—Cast-in-Place Connections

Transverse reinforcement in the form of tied column reinforcement, spirals, hoops, or intersecting spirals or hoops shall be provided. The joint shear reinforcement may also be provided in the form of column transverse steel or exterior transverse reinforcement continued into the bent cap.

Where the principal tension stress in the joint, p_t, as specified in Article 8.~~13~~15.2.1 is less than $0.11\sqrt{f'_c}$, the transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 1 and no additional reinforcement within the joint is

and concrete fill do not become a weak link in the system. The grout specification in Article C8.13.8.3.2a accounts for the difference in compressive strengths when comparing 2-in grout cubes to standard concrete cylinders.

required:

$$\rho_s \ge \frac{0.11\sqrt{f_c'}}{f_{yh}}$$ (8.~~13~~15.3.1-1)

where:

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f'_c = nominal concrete compressive strength (ksi)

ρ_s = volumetric reinforcement ratio of transverse reinforcing provided within the cap as defined by Eq. 8.6.2-7

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, then transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 1 and Eq. 2, and additional joint reinforcement is required as indicated in Article 8.~~13~~15.4 for integral bent cap beams or Article 8.~~13~~15.5 for nonintegral bent cap beams:

$$\rho_s \ge 0.40\frac{A_{st}}{l_{ac}^2}$$ (8.~~13~~15.3.1-2)

where:

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

l_{ac} = length of column reinforcement embedded into the bent cap (in.)

For interlocking cores, ρ_s, shall be based on the total area of reinforcement of each core.

8.15.3.2—Precast Connections

8.15.3.2.1—Grouted Duct Connection

Grouted duct connections shall satisfy the requirements of Article 8.15.3.1 for minimum transverse reinforcement in the joint. Spacing of transverse reinforcement shall not exceed $0.3D_s$ or 12 in.

Where the principal tension stress in the joint, p_t, as specified in Article 8.15.2.1 is less than $0.11\sqrt{f_c'}$, the transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 8.15.3.1-1, and the provisions of Article 8.14.5.2.2a for vertical stirrups inside the joint shall be satisfied.

Where the principal tension stress in the joint, p_t, is

C8.15.3.2

Precast grouted duct and cap pocket connections are described in C8.15.5.2.2 and C8.15.5.2.3, respectively.

C8.15.3.2.1

Grouted duct connections require the same minimum transverse reinforcement as used for cast-in-place connections, except that spacing limitations are included. The additional joint shear reinforcement is similarly required where the principal tension stress in the joint is greater than or equal to $0.11\sqrt{f_c'}$. However, where the principal tension stress in the joint is less than $0.11\sqrt{f_c'}$, minimum vertical joint stirrups are required, as explained in Article C8.14.5.2.2a.

greater than or equal to $0.11\sqrt{f'_c}$, the transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 8.15.3.1-1 and Eq. 8.15.3.1-2, and additional joint reinforcement is required as indicated in Article 8.15.5.2.

8.15.3.2.2—Cap Pocket Connection

Cap pocket connections shall use a helical, lock-seam, corrugated steel pipe conforming to ASTM A 760 to form the bent cap pocket. The thickness of the corrugated steel pipe, t_{pipe}, shall satisfy the transverse reinforcement ratio requirements specified in Article 8.15.3.1 and shall not be taken less than that determined by Eq. 1:

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

in which:

$$F_H = n_h A_{sp} f_{yh} \qquad (8.15.3.2.2\text{-}2)$$

where:

F_H = nominal confining hoop force in the joint (kips)

H_p = height of steel pipe (in.)

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

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

n_h = number of transverse hoops in equivalent cast-in-place joint

A_{sp} = area of one hoop reinforcing bar (in.^{2})

f_{yh} = nominal yield stress of transverse reinforcement (ksi)

Where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f'_c}$, the number of hoops, n_h, shall be based on Eq. 1. In addition, the provisions of Article 8.14.5.2.3a for vertical stirrups inside the joint shall be satisfied.

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f'_c}$, the number of hoops, n_h, shall be based on the larger value of transverse joint reinforcement, ρ_s, given by Eq. 8.15.3.1-1 and Eq. 8.15.3.1-2. Additional joint reinforcement is required as indicated as indicated in Article 8.15.5.2.

C8.15.3.2.2

The equations for determining the thickness of the corrugated steel pipe in a cap pocket connection are based on providing an average confining hoop force to the joint approximately the same as that provided by hoops required for cast-in-place joints. The maximum spacing requirements of $0.3D_s$ and 12 in. do not apply to the determination of n_h. The minimum thickness of the steel pipe, t_{pipe}, of 0.060 in. corresponds to 16 gage steel pipe, which was used in the cap pocket research and is the thinnest gage steel corrugated pipe that is typically readily-available.

Equations C8.15.3.2.2-1 and C8.15.3.2.2-2 may be used to conservatively determine the required pipe thickness, in lieu of calculating the number of hoops in an equivalent cast-in-place joint, n_h, as the basis for determining the required thickness of the steel pipe. These simpler equations may result in a thicker gage pipe used in design.

Where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f'_c}$, the thickness of the steel pipe, t_{pipe}, may be determined from the following equation:

$$t_{pipe} \geq 0.04\frac{\sqrt{f'_c}\,D'_{cp}}{f_{yp}\cos\theta} \qquad (C8.15.3.2.2\text{-}1)$$

where:

f'_c = nominal concrete compressive strength of the bent cap (ksi)

D'_{cp} = average diameter of confined cap pocket fill between corrugated steel pipe walls (in.)

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

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

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f'_c}$, the thickness of the steel pipe, t_{pipe}, may be determined from the larger of Equation C8.15.3.2.2-1 and the following equation:

$$t_{pipe} \geq 0.14\frac{A_{st} D'_{cp} f_{yh}}{l_{ac}^2 f_{yp}\cos\theta} \qquad (C8.15.3.2.2\text{-}2)$$

where:

8.~~13~~15.4 Integral Bent Cap Joint Shear Design

8.~~13~~15.4.1 Joint Description

The following types of joints are considered “T” joints for joint shear analysis:

- Integral interior joints of multi-column bents in transverse direction
- All column/superstructure joints in the longitudinal direction
- Exterior column joints for box girder superstructures if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement

All other exterior joints are considered knee joints in the transverse direction and require special analysis and detailing that is not addressed in these *Guide Specifications*.

The bent cap width shall extend 12 in. on each side of the column as shown in Figure 8.~~13~~15.4.2.1-2.

8.~~13~~15.4.2 Joint Shear Reinforcement

8.~~13~~15.4.2.1 Vertical Stirrups

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. The vertical stirrup area, A_s^{jv} is required on each side of the column or pier wall, see Figures 1, 2 and 3. ~~The stirrups~~

A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

D'_{cp} = average diameter of confined cap pocket fill between corrugated steel pipe walls (in.)

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

l_{ac} = length of column reinforcement embedded into the bent cap (in.)

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

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

In design calculations, the spacing of transverse joint hoops can be directly related to the number of hoops, n_h, by the volumetric reinforcement ratio for transverse joint hoops, ρ_s, using Eq. 8.6.2-7.

C8.~~13~~15.4.1

The design of beam-column joints is based upon research and experiments for circular columns framing into rectangular beams. Although no specific requirements have been developed for rectangular columns framing into rectangular beams, the requirements of this article may be used.

~~provided in the overlapping areas shown in Figure 1 shall count towards meeting the requirements of both areas creating the overlap.~~ These stirrups can be used to meet other requirements documented elsewhere including the shear in the bent cap. Vertical joint shear reinforcement shall be placed within the joint region with a maximum spacing of 18 in. A minimum of two vertical joint shear reinforcement stirrups shall be placed within the joint.

$$A_s^{jv} \geq 0.20\, A_{st} \qquad (8.\cancel{13}15.4.2.1\text{-}1)$$

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

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

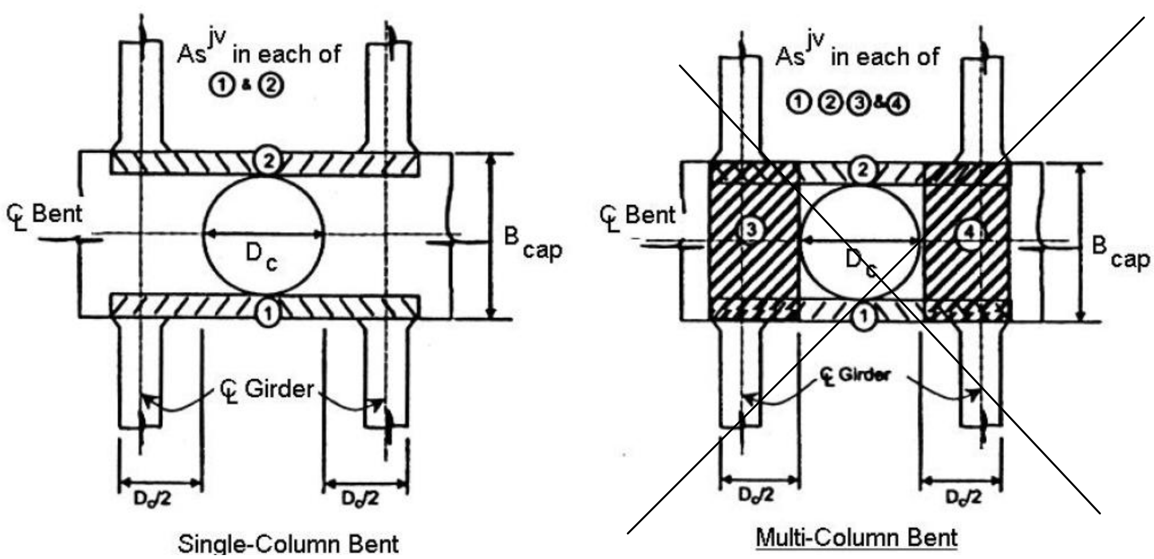


Figure 8.~~13~~15.4.2.1-1 Location of Vertical Joint Shear Reinforcement

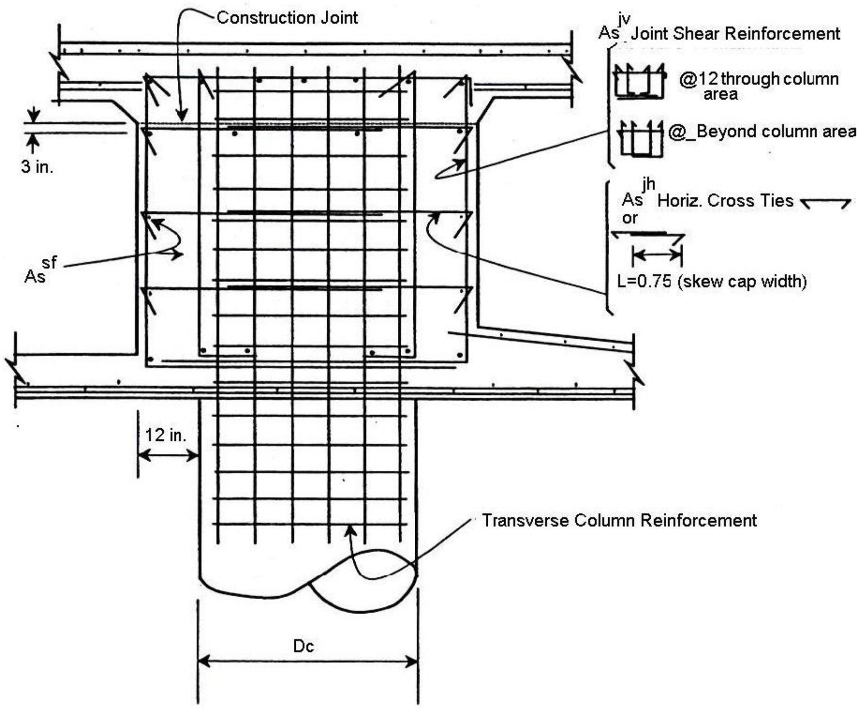


Figure 8.~~13~~15.4.2.1-2 Joint Shear Reinforcement Details

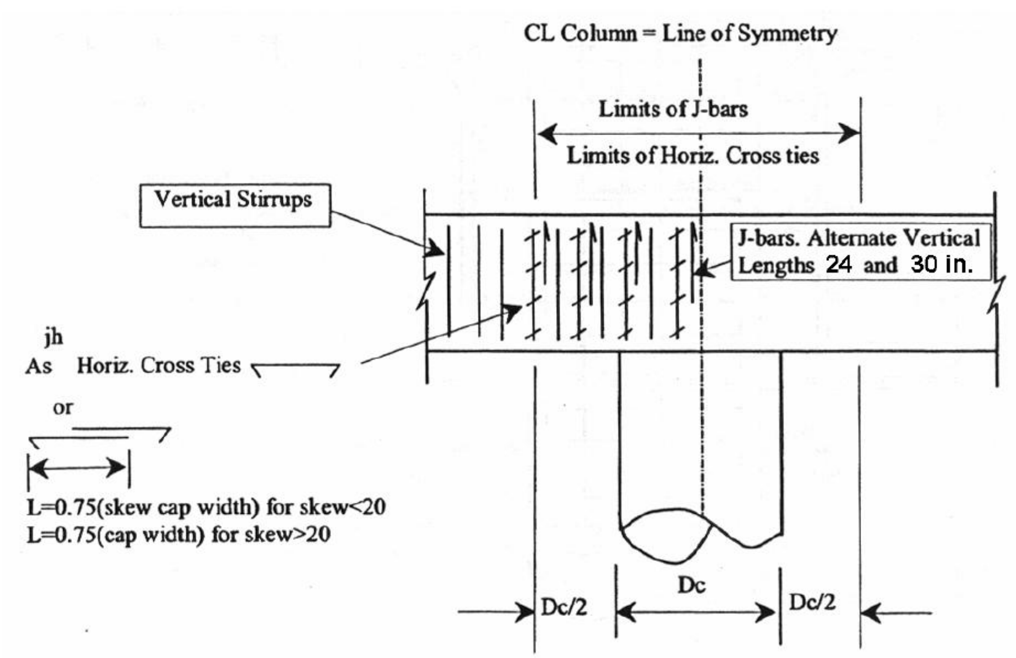


Figure 8.~~13~~15.4.2.1-3 Location of Horizontal Joint Shear Reinforcement

8.~~13~~15.4.2.2 Horizontal Stirrups

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 in. The horizontal reinforcement, A_s^{jh}, shall be placed within a distance D_c from each side of the column centerline as shown in Figure 8.~~13~~15.4.2.1-3.

$$A_s^{jh} \geq 0.10\, A_{st} \qquad (8.\cancel{13}\underline{15}.4.2.2\text{-}1)$$

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

For multi-column bents, horizontal stirrups shall additionally satisfy the requirement of Article 8.15.5.

8.~~13~~15.4.2.3 Horizontal Side Reinforcement

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in Eq. 1 and shall be placed near the side faces of the bent cap with a maximum spacing of 12 in. as shown in Figure 8.~~13~~15.4.2.1-2. Any side face reinforcement placed to meet other requirements shall count towards meeting the requirements of this article.

$$A_s^{sf} \geq max \begin{cases} 0.10\, A_{cap}^{top} \\ 0.10\, A_{cap}^{bot} \end{cases} \qquad (8.\cancel{13}\underline{15}.4.2.3\text{-}1)$$

where:

A_s^{sf} = area of longitudinal side reinforcement in the bent cap (in.^{2})

A_{cap}^{top}= area of bent cap top flexural steel (in.^{2})

A_{cap}^{bot}= area of bent cap bottom flexural steel (in.^{2})

8.~~13~~15.4.2.4 J-Bars

For integral cap of bents skewed greater than 20°, vertical J-bars hooked around the longitudinal top deck steel extending alternatively 24 in. and 30 in. into the bent cap are required. The J-dowel reinforcement shall satisfy:

$$A_s^{j\text{-}bar} \geq 0.08\, A_{st} \qquad (8.\cancel{13}\underline{15}.4.2.2\text{-}1)$$

The J-bars shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column, see Figure 1 and Figure 8.~~13~~15.4.2.1-3.

8.15.4.2.5 Additional Longitudinal Cap Beam

Reinforcement

For multi-column bents, additional longitudinal cap beam reinforcement shall satisfy the requirement of Article 15.5.

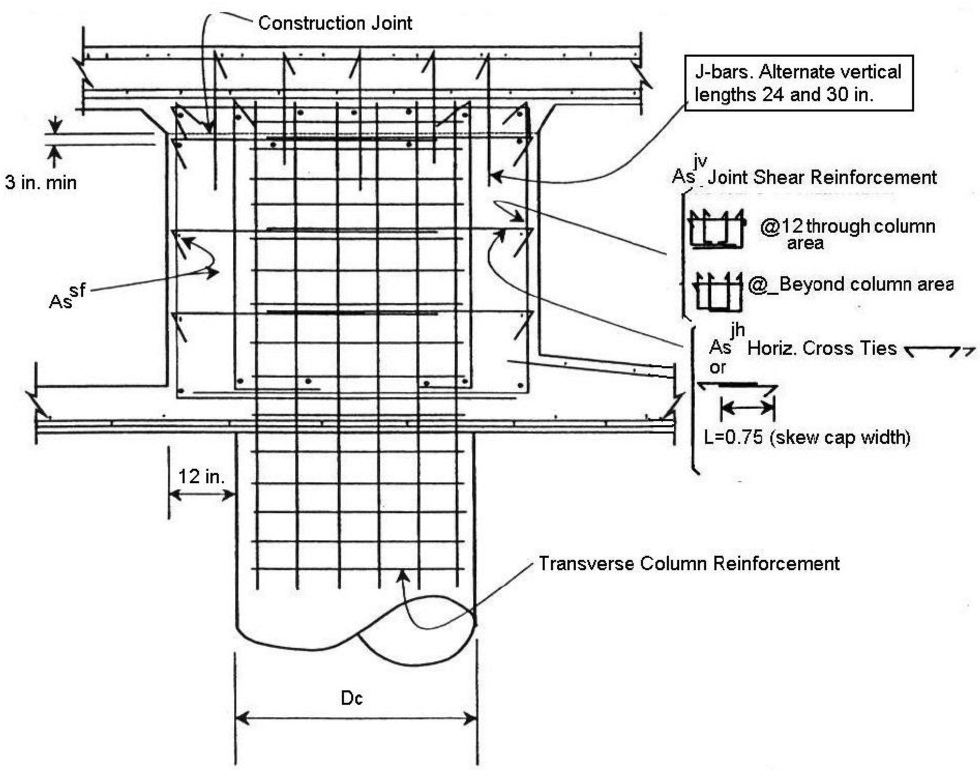


Figure 8.~~13~~15.4.2.4-1 Additional Joint Shear Steel for Skewed Bridges

8.~~13~~15.5—Nonintegral Bent Cap Joint Shear Design

Cast-in-place, emulative precast and hybrid bent caps ~~beams~~ satisfying Eq. 1 shall be reinforced in accordance with the requirements of Articles 8.~~13~~15.5.1 and 8.~~13~~15.5.2. Bent caps ~~beams~~ not satisfying Eq. 1 shall be designed on the basis of the strut and tie provisions of the *AASHTO LRFD Bridge Design Specifications* and as approved by the Owner.

$$D_c \leq d \leq 1.25 D_c \qquad (8.~~13~~15.5\text{-}1)$$

where:

D_c = column diameter (in.)

d = total depth of the bent cap beam (in.)

C8.~~13~~15.5

Nonintegral bent cap systems addressed in this Article use connections between the bent cap and column that are classified as either cast-in-place connections, emulative precast connections or hybrid connections. Emulative precast bent cap-to-column connections are designed and detailed to emulate joint performance and system ductility achieved by monolithic, cast-in-place construction. Emulative systems use a cast-in-place concrete fill or high strength, non-shrink grout to join the precast bent cap to the columns. Associated construction specifications are detailed in Article 8.13.8 of the *AASHTO LRFD Bridge Construction Specifications*.

Detailing of hybrid bridge systems can use the same provisions as those presented for cast-in-place and emulative systems. The assumed joint force transfer mechanism for all systems provide a conservative and reliable means to ensure adequate joint performance.

Cast-in-place, emulative precast, and hybrid bent cap-to-column connections are designed to accommodate the forces associated with the column's overstrength plastic hinging moment capacity in an essentially elastic manner.

The design of nonintegral cast-in-place bent cap joints is summarized in Sritharan (2005). The design of nonintegral emulative precast bent cap joints is summarized in Matsumoto (2009).

The design of multi-column integral bent caps must also satisfy the requirements presented in this Article.

8.~~13~~15.5.1—Cast-in-Place Connections

8.~~13~~15.5.1.1—Vertical Stirrups Outside the Joint Region

Vertical stirrups with a total area, A_s^{jvo}, provided to each side of the column shall satisfy:

$$A_s^{jvo} \geq 0.175\, A_{st} \qquad (8.~~13~~15.5.1.1\text{-}1)$$

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

Vertical stirrups or ties shall be placed transversely within a distance equal to the column diameter, D_c, extending from each face of the column as shown in Figure 1 and Figure 2. The area of these stirrups shall not be used to meet other requirements such as shear in the bent cap.

8.~~13~~15.5.1.2—Vertical Stirrups Inside the Joint Region

Vertical stirrups with a total area, A_s^{jvi}, spaced evenly over the column shall satisfy:

$$A_s^{jvi} \geq 0.135 A_{st}$$ (8.~~13~~15.5.1.2-1)

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

8.~~13~~15.5.1.3—Additional Longitudinal Cap Beam Reinforcement

Longitudinal reinforcement, A_s^{jl}, in both the top and bottom faces of the cap beam shall be provided in addition to that required to resist other loads. The additional area of the longitudinal steel shall satisfy:

$$A_s^{jl} \geq 0.245 A_{st}$$ (8.~~13~~15.5.1.3-1)

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

8.~~13~~15.5.1.4—Horizontal J-Bars

Horizontal J-bars hooked around the longitudinal reinforcement on each face of the cap beam shall be provided as shown in Figure 8.~~13~~15.5.1.1-1. At a minimum, horizontal J-bars shall be located at every other vertical-to-longitudinal bar intersection within the joint. The J-dowel reinforcement bar shall be at least a #4 size bar.

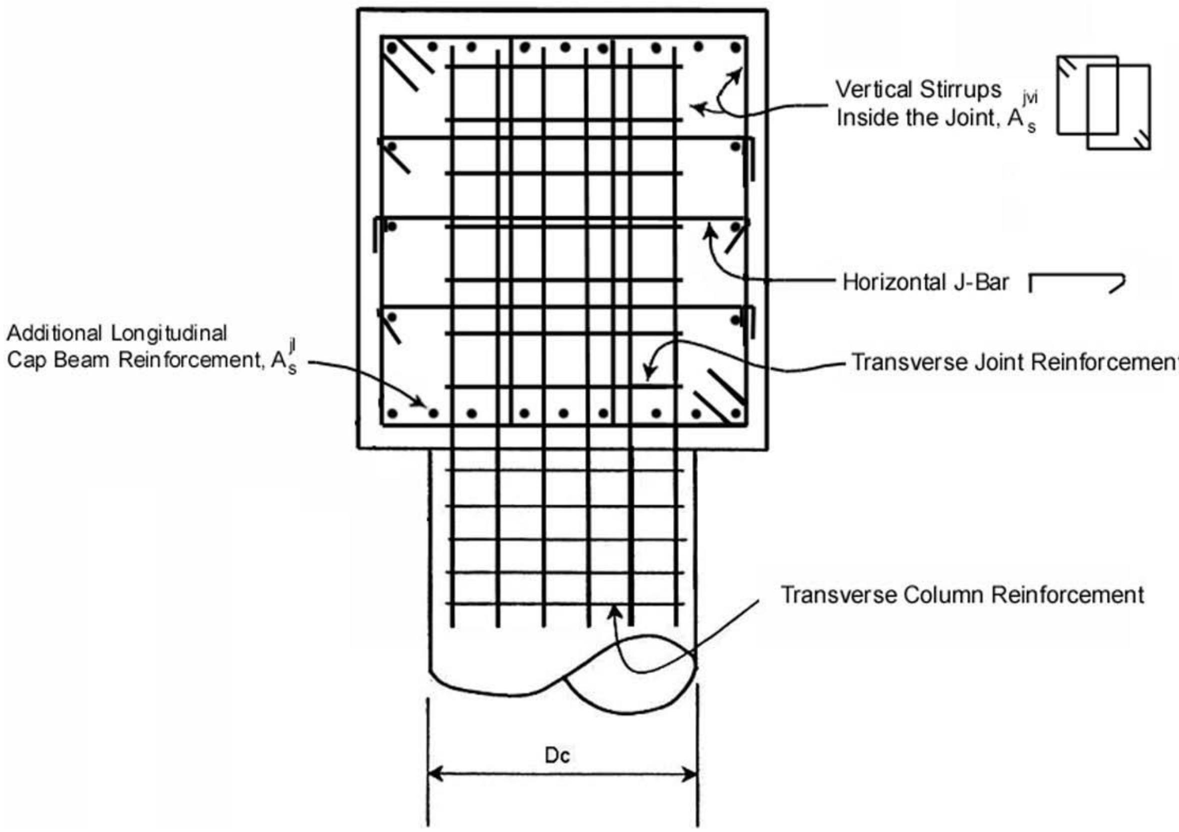


Figure 8.~~13~~15.5.1.1-1—Cast-in-place Joint Shear Reinforcement Details

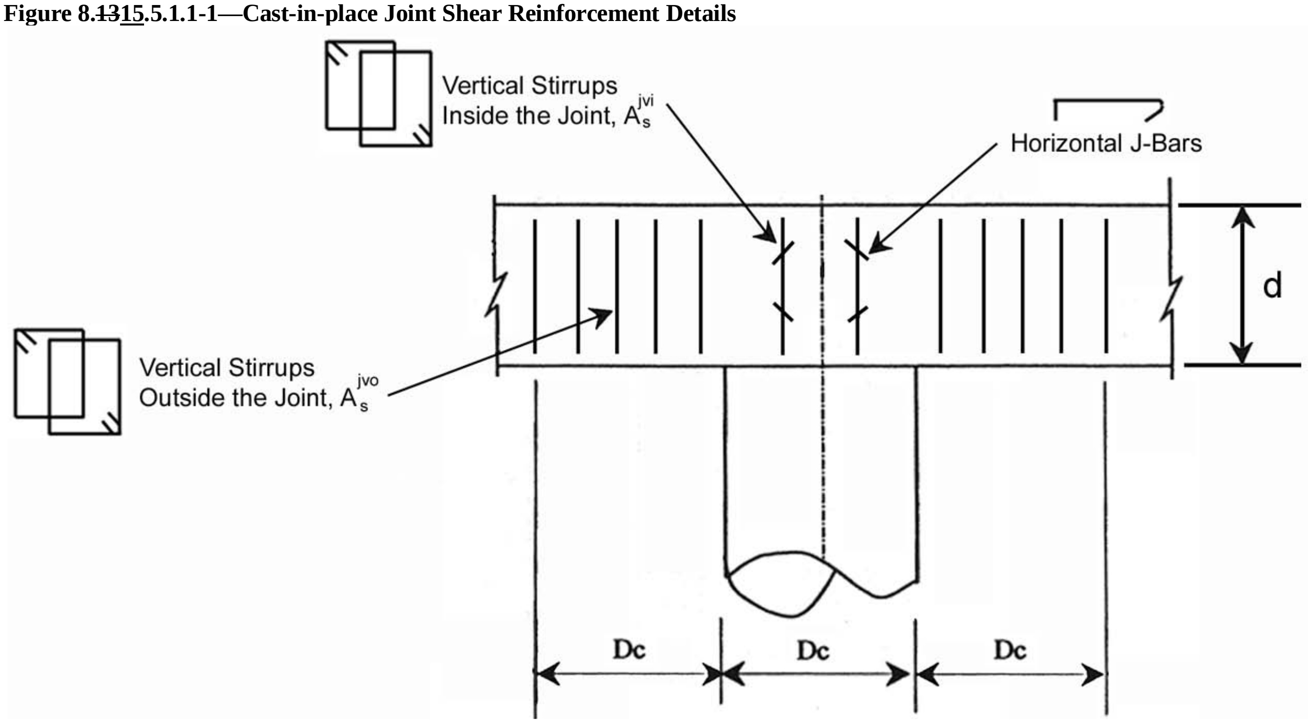


Figure 8.~~13~~15.5.1.1-2—Location of Cast-in-place Vertical Joint Shear Reinforcement

8.15.5.2—Precast Connections

C8.15.5.2

This Article applies to precast bent caps using either grouted duct or cap pocket connections including hybrid connections employing grouted duct connections for column reinforcement.

8.15.5.2.1—Bedding Layer Reinforcement

Bedding layers between columns and precast bent caps shall be reinforced with transverse reinforcement, as shown in Figure 8.15.5.2.2-1 and Figure 8.15.5.2.3-1. Bedding layer reinforcement shall match the size and type of transverse reinforcement required for the column plastic hinging region and shall be placed evenly through the depth of the bedding layer.

Grout bedding layer heights shall not exceed 3 in

C8.15.5.2.1

A bedding layer between the bent cap soffit and the top of column is used to accommodate fabrication and placement tolerances. Transverse reinforcement around the column bars within the bedding layer provides confinement and reduces the unsupported length of column bars, thereby limiting the potential for buckling of these bars at the bedding layer during plastic hinging of the column. Accurate placement of reinforcement is, therefore, essential to achieve the expected system ductility capacity. Reinforcement should normally be placed evenly through the depth of the bedding layer. However, in some cases, an uneven bedding layer (e.g., a sloping bent cap on top of a large diameter column) or a bedding layer of an unusual shape may be encountered, requiring placement of bedding layer reinforcement that is not uniformly distributed, to minimize the unsupported length of column bars. Placement of the hoop at the top of the column below the bedding layer hoop is addressed in Article 8.8.14.

Plan sheets should show the intended placement of the bedding layer reinforcement. The associated requirement for shop drawings is addressed in Article 8.13.8.4.4 of the *AASHTO LRFD Bridge Construction Specifications.*

Adequate flowability of the concrete fill or grout should not be prevented by the size and placement of the bedding layer reinforcement.

8.15.5.2.2—Grouted Duct Connection

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, grouted duct connections shall satisfy the additional joint shear reinforcement required by Article 8.15.5.1. In addition, the vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used. Figure 1 through Figure 3 show details of the connection, including ducts, vertical stirrups inside the joint, and bedding layer reinforcement, in addition to the joint transverse reinforcement.

C8.15.5.2.2

The grouted duct connection uses corrugated ducts embedded in the precast bent cap to anchor individual column longitudinal bars. The ducts and bedding layer between the cap and column or pile are grouted with high strength, non-shrink cementitious grout to complete the precast connection. Ducts are sized to provide adequate tolerance for bent cap fabrication and placement and should be accounted for in sizing the bent cap to minimize potential congestion.

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, joint shear reinforcement requirements are essentially the same as those for cast-in-place connections. However, where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$, minimum vertical stirrups are required in the joint per Article 8.14.5.2.2a. See Article 8.15.3.2.1.

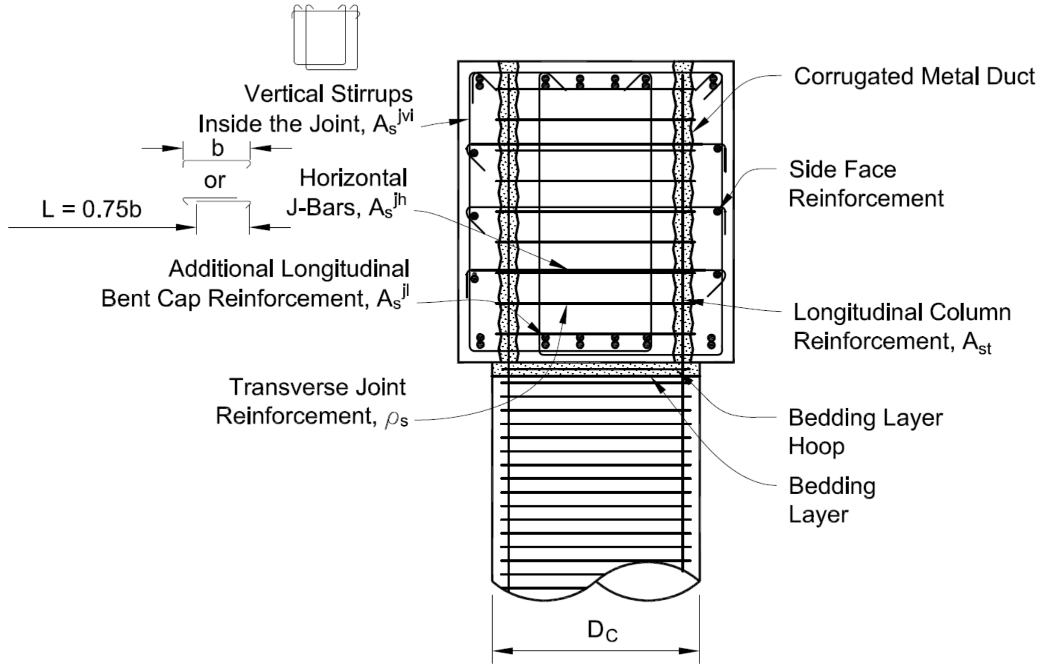


Figure 8.15.5.2.2-1—Grouted Duct Full Ductility Joint Shear Reinforcement Details (Section)—SDCs C and D

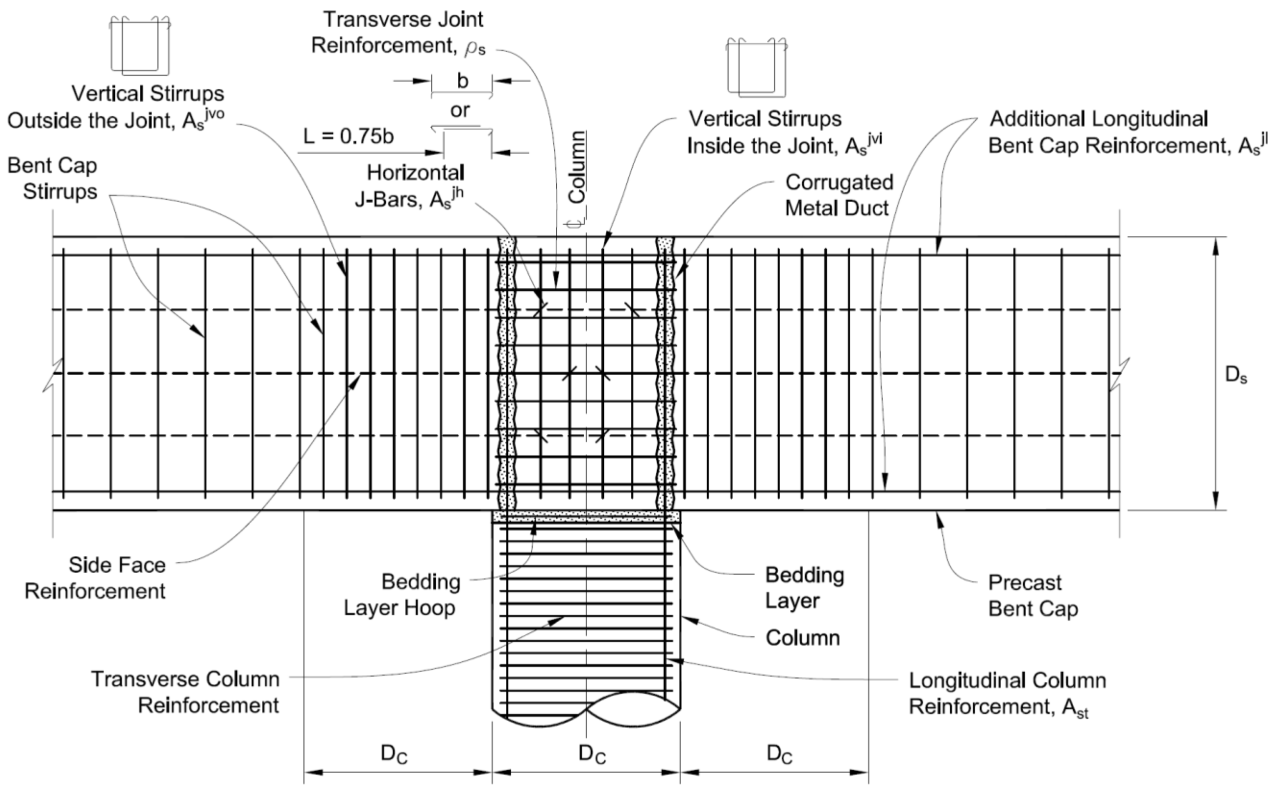


Figure 8.15.5.2.2-2—Grouted Duct Full Ductility Joint Shear Reinforcement Details (Elevation)—SDCs C and D

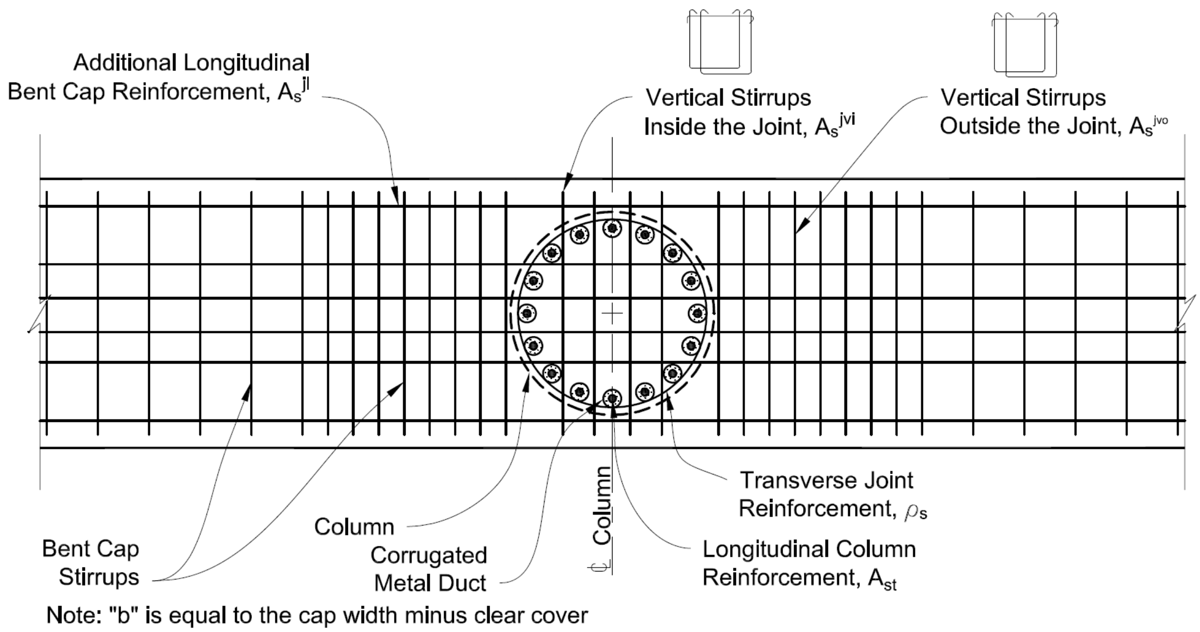


Figure 8.15.5.2.2-3—Grouted Duct Full Ductility Joint Shear Reinforcement Details (Plan)—SDCs C and D

8.15.5.2.3—Cap Pocket Connection

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, cap pocket connections shall be satisfy the additional joint shear reinforcement provisions of Article 8.15.5.2.3a and Article 8.15.5.2.3b.

C8.15.5.2.3

The cap pocket connection uses a single, helical, corrugated steel pipe embedded in the precast bent cap to form the cap pocket, which anchors the column longitudinal bars. This pipe, placed between top and bottom bent cap longitudinal reinforcement, serves as both a stay-in-place form and joint transverse reinforcement. A flowable cast-in-place concrete is used to fill the void and complete the precast connection. The pipe diameter is sized to provide adequate field tolerance for placement of the precast bent cap over column longitudinal bars, and the pipe thickness is based on satisfying transverse joint reinforcement requirements.

8.15.5.2.3a—Joint Shear Reinforcement

Cap pocket connections shall be detailed to include additional joint shear reinforcement as specified in Article 8.15.5.1, except as follows:

Horizontal J-bars specified in Article 8.15.5.1.4 shall not be required in the joint region.

Vertical stirrups inside the joint with a total area, A_s^{jvi}, spaced evenly over a length, D_c, through the joint shall satisfy:

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

where:

A_{st} = total area of column longitudinal reinforcement anchored in the joint (in.^{2})

Vertical stirrups inside the joint shall consist of double leg stirrups or ties of a bar size no smaller than that of the bent cap transverse reinforcement. A minimum of two stirrups or equivalent ties shall be used.

Figure 1 through Figure 3 show details of the connection, including the steel pipe, vertical stirrups inside the joint, and bedding layer reinforcement.

C8.15.5.2.3a

Where the principal tension stress in the joint, p_t, is greater than or equal to $0.11\sqrt{f_c'}$, joint shear reinforcement requirements inside the joint differ from those for cast-in-place connections: the pipe serves as the required transverse joint reinforcement, a smaller area of vertical joint stirrups and supplementary hoops are required, and j-bars are not used. However, outside the joint, the same vertical stirrups and additional bent cap longitudinal reinforcement are used.

Due to the presence of the pipe, use of overlapping double-leg vertical stirrups in the joint is not practical.

As shown in Figures 1 and 3, inverted U-bars or hairpins may be optionally placed within the pocket to help restrain potential splitting cracks and buckling of top bent cap flexural bars within the joint.

Where the principal tension stress in the joint, p_t, is less than $0.11\sqrt{f_c'}$, minimum vertical stirrups are required in the joint per Article 8.14.5.2.3a. See Article 8.15.3.2.2.

8.15.5.2.3b—Supplementary Hoops

A supplementary hoop shall be placed one inch from each end of the corrugated pipe. The bar size of the hoop shall match the size of the bedding layer reinforcement required by Article 8.15.5.2.1.

8.15.5.2.3b

A supplementary hoop is required at each end of the pipe to limit dilation and potential unraveling. Cap pocket connection research demonstrated the effectiveness of this reinforcement.

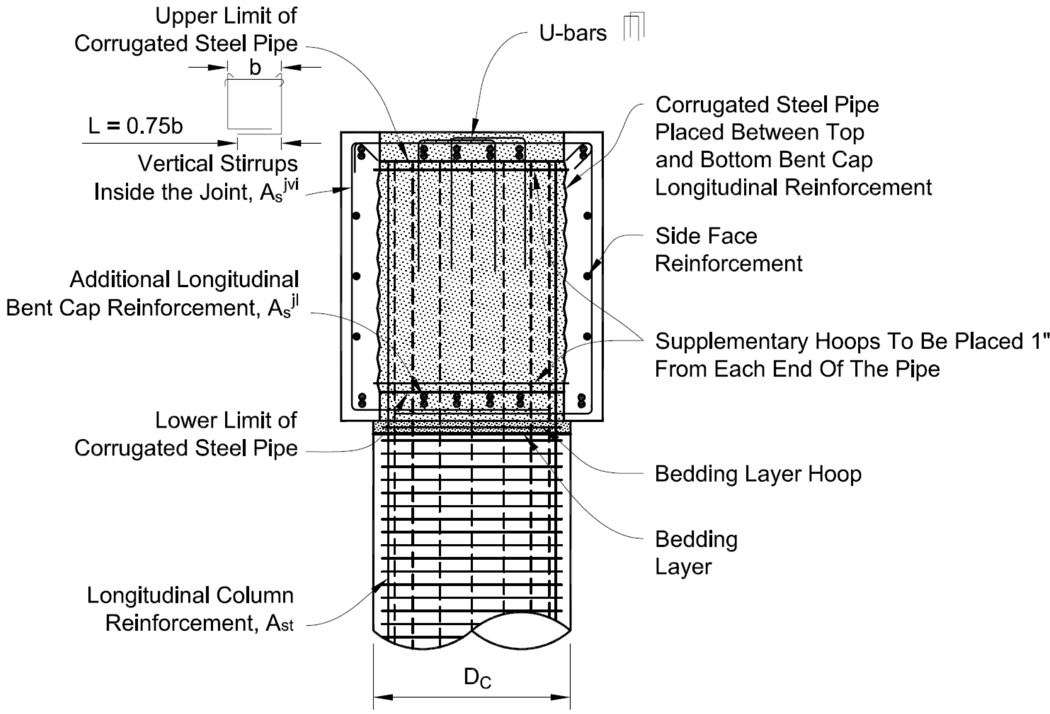


Figure 8.15.5.2.3-1—Cap Pocket Full Ductility Joint Shear Reinforcement Details (Section)—SDCs C and D

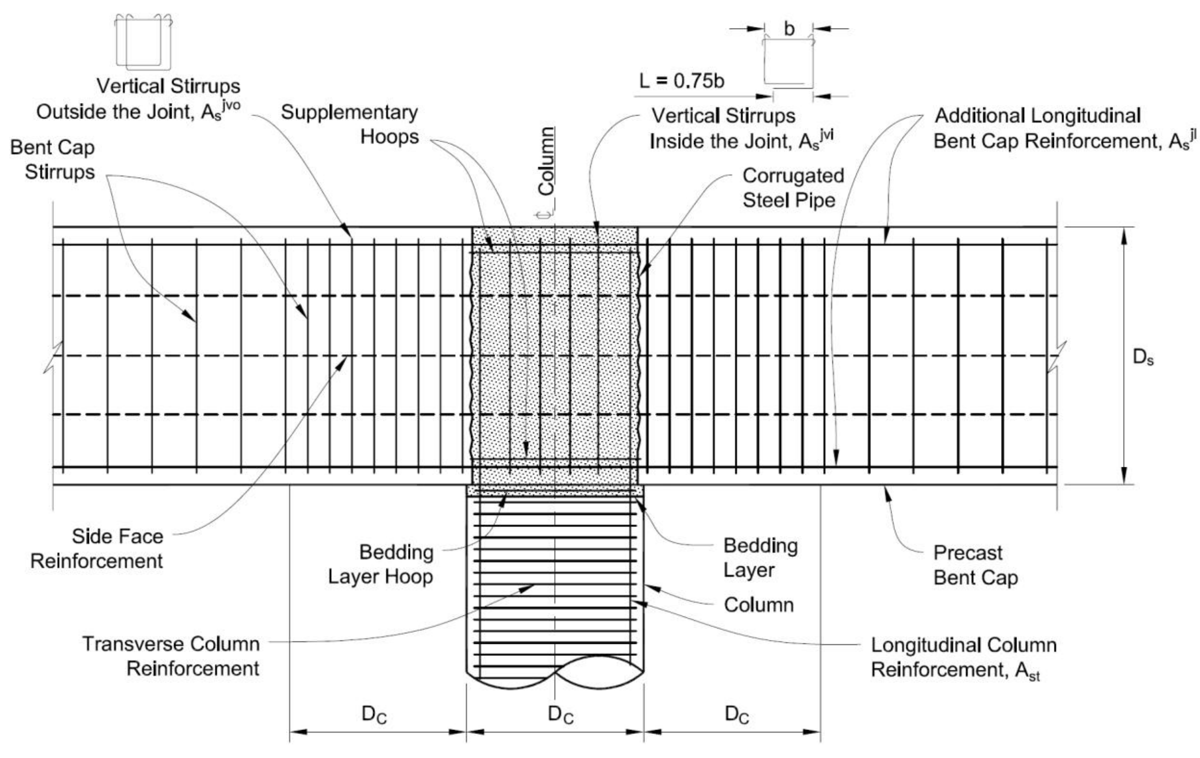


Figure 8.15.5.2.3-2— Cap Pocket Full Ductility Joint Shear Reinforcement Details (Elevation)—SDCs C and D

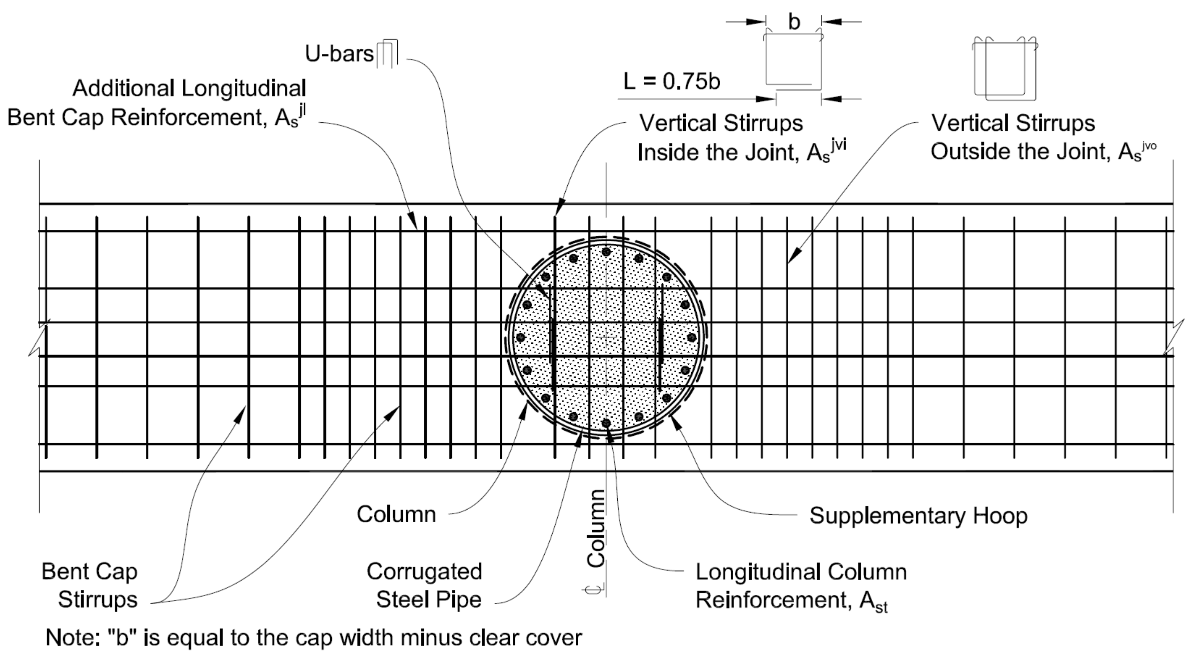


Figure 8.15.5.2.3-3— Cap Pocket Full Ductility Joint Shear Reinforcement Details (Plan)—SDCs C and D

Attachment DS16. Proposed Article 5.10.11.4.3—Column Connections
(AASHTO LRFD Bridge Design Specifications, 4th Ed., 2007 with 2008 and 2009 Interims)

Specifications	Commentary
5.10.11.4.3—Column Connections Cast-in-place concrete connections shall satisfy the provisions of this Article. Precast bent cap connections shall be based on the provisions of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. The design force for the connection between the column and the cap beam superstructure, pile cap, or spread footing shall be as specified in Article 3.10.9.4.3. The development length for all longitudinal steel shall be 1.25 times that required for the full yield strength of reinforcing as specified in Article 5.11. Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance not less than one-half the maximum column dimension or 15.0 in. from the face of the column connection into the adjoining member. The nominal shear resistance, V_n, provided by the concrete in the joint of a frame or bent in the direction under consideration, shall satisfy: • For normal weight aggregate concrete: $V_n \leq 0.380\, bd \sqrt{f_c^{'}}$, and (5.10.11.4.3-1) • For lightweight aggregate concrete: $V_n \leq 0.285\, bd \sqrt{f_c^{'}}$ (5.10.11.4.3-2)	*C5.10.11.4.3* This Article applies to cast-in-place concrete connections. Special seismic provisions for emulative precast bent cap-to-column connections are provided in Articles 8.13 through 8.15 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. A column connection, as referred to in this Article, is the vertical extension of the column area into the adjoining member. The integrity of the column connection is important if the columns are to develop their flexural capacity. The longitudinal reinforcement should be capable of developing its overstrength capacity of 1.25 f_y. The transverse confining reinforcement of the column should be continued a distance into the joint to avoid a plane of weakness at the interface. The strength of the column connections in a column cap is relatively insensitive to the amount of transverse reinforcement, provided that there is a minimum amount and that shear resistance is limited to the values specified. The factored shear resistance for joints made with lightweight aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75 percent of that in normal weight aggregate concrete.

Attachment DS17. Proposed Article 5.11.1.2.4—Moment Resisting Joints
(AASHTO LRFD Bridge Design Specifications, 4th Ed., 2007 with 2008 and 2009 Interims)

Specifications	Commentary
5.11.1.2.4—Moment Resisting Joints Cast-in-place concrete connections shall satisfy the provisions of this Article. Precast bent cap connections shall be based on the provisions of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint. In Seismic Zones 3 and 4, joints shall be detailed to resist moments and shears resulting from horizontal loads through the joint.	*C5.11.1.2.4* This Article applies to cast-in-place concrete connections. Special seismic provisions for emulative precast bent cap-to-column connections are provided in Articles 8.13 through 8.15 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Reinforcing details for developing continuity through joints are suggested in the ACI Detailing Manual. As of this writing (*Fall 1997*), much research on moment resisting joints and especially on the seismic response thereof is in progress. The reports on this work should be consulted as they become available.