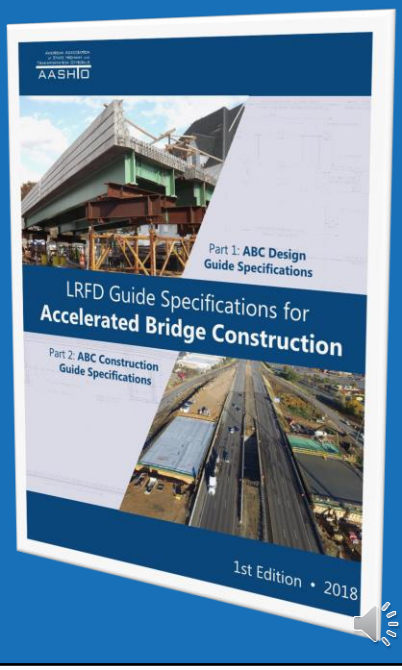
Module 10: Integral Cap Design and Detailing

This module covers:

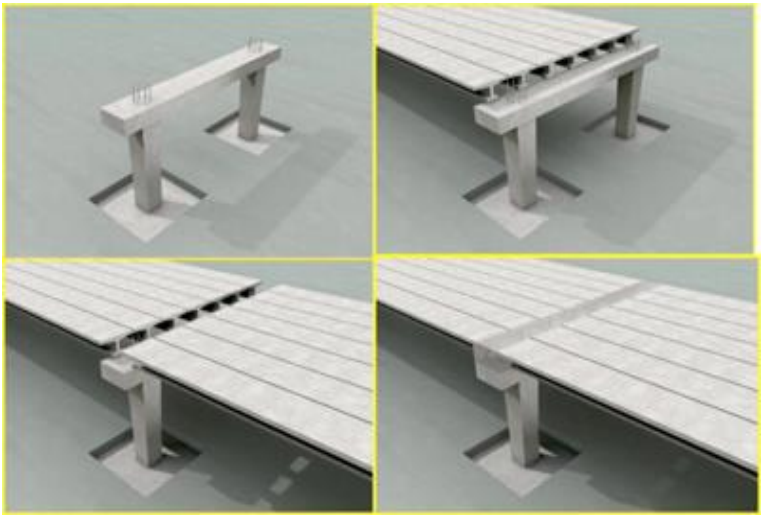
- Integral Substructure-to-Superstructure Connections
- Two-Stage Integral Pier Cap



This module covers:

- Integral Substructure-to-Superstructure Connections
- Two-Stage Integral Pier Caps

3.6.11 Integral Substructure-to Superstructure Connections



The recommended approach to design and detailing of full integral superstructure-to-substructure connections is to incorporate a reinforced closure joint in the design. The Designer shall identify the load path through the element to provide proper detailing for force transfer to adjacent elements. This type of connection, or system of connections, is always a CP connection.

Designers shall consider non-integral or semi-integral designs for ABC bridges with very short construction durations.

Semi-integral abutment connections may be used to facilitate the construction of bridge system installations using SPMT and lateral sliding methods.

Article 3.6.11 of the Guide Specifications for ABC covers integral substructure to superstructure connections. The graphics show the details of this connection.

Integral substructure to superstructure connections are varied throughout different states, but the principles apply the same, and the process is captured here and should be applied to each specific situation with care and understanding.

We will review the basic principles of this design in the following slides. Being an emulative connection, the design provisions of the applicable DOT should also be followed.

This example includes the following features:

1. Precast columns with a socket connection into a spread footing (covered in module 9)
2. Precast semi-dropped (Stage 1) pier-cap with grouted ducts to allow penetration of column flexural steel into the cap diaphragm
3. Precast girders with precast partial depth panels
4. CIP footings
5. CIP deck topping
6. And the upper (Stage 2) section of the pier cap, which is always CIP

3.6.12 Two-stage Integral Pier Cap

In two-stage integral bents, a lower pier cap is first integrated with the columns to provide support for the girders during construction. Then, an upper-stage cap beam is cast around bars and prestressing strands that project from the girder ends and possibly from the column ends to rigidly connect the superstructure with the substructure. This connection is always a CP connection. However, prescriptive joint shear steel is required in SDC C and D (Zones 3 and 4) immediately adjacent to the columns.

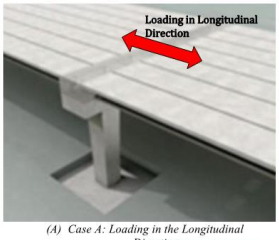
Column longitudinal reinforcement shall extend into the upper cap as far as practical. In the event that some column bars need to be terminated in the lower cap, supplemental shear reinforcement shall be provided to ensure force transfer into the joint at the girder/deck level. The entire load path in the joint seismic subsystem shall be verified.

The photo on the slide shows the integral cap during the placement of the precast partial depth deck panels. The deck is placed over the majority of the superstructure and cured before the closure pour in Stage 2 of the pier cap that integrates the entire sub-structure to super-structure connection. Deck reinforcement has not yet been placed in this photo.

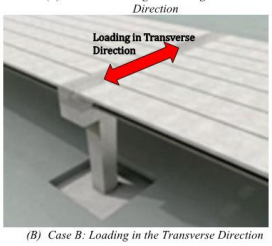
Note this same concept can be used with pre-assembled superstructure spans brought in using SPMT or lateral slide methods and only the CIP upper (Stage 2) pour would be required.

Also, note that the column (long.) flexural reinforcing is extended to just below the deck reinforcement.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4



(A) Case A: Loading in the Longitudinal Direction



(B) Case B: Loading in the Transverse Direction

Figure 3.6.12.1-1—Loading Cases for Evaluation of Joint Principal Stresses

Moment resisting joints shall be proportioned so that principal stresses satisfy the requirements of Eq. 3.6.12.1-1 and Eq. 3.6.12.1-2:

- For principal compression, p_c:

$$p_c \leq 0.25 f'_c \qquad (3.6.12.1\text{-}1)$$

- For principal tension, p_t:

$$p_t \leq 0.38\sqrt{f'_c} \qquad (3.6.12.1\text{-}2)$$

in which:

$$p_c = \left|\left(\frac{f_h+f_v}{2}\right) + \sqrt{\left(\frac{f_h-f_v}{2}\right)^2 + \left(v_{jv}\right)^2}\right| \qquad (3.6.12.1\text{-}3)$$

$$p_t = \left|\left(\frac{f_h+f_v}{2}\right) - \sqrt{\left(\frac{f_h-f_v}{2}\right)^2 + \left(v_{jv}\right)^2}\right| \qquad (3.6.12.1\text{-}4)$$

where:

f_h = average normal stress in the horizontal direction within a moment resisting joint (ksi)
f_v = average normal stress in the vertical direction within a moment resisting joint (ksi)
v_{jv} = vertical shear stress in a moment resisting joint (ksi)

Joint principal stresses shall be independently evaluated for loading in the longitudinal direction (Loading Case A) and loading in the transverse direction (Loading Case B) as shown in Figure 3.6.12.1-1.

Article 3.6.12.1 of the Guide Specifications for ABC covers joint proportioning for a two-stage integral pier cap for SDCs C & D or Seismic Zones 3 & 4.

For those familiar with the AASHTO Seismic Guide Specifications, the process is identical to the “Joint Design for SDCs C and D” provisions in that specification. The only difference is that the stress terms (f’s) are modified for joint geometry consistent with the load path in each direction of analysis.

We will identify the area that resists the over-strength forces and determine if the principle stresses in tension and compression are met. If they are, there is one set of detailing requirements. If they are not, there are additional detailing requirements.

We will quickly go over how to determine the average normal stress in the horizontal direction, the average normal stress in the vertical direction, and the vertical shear stresses applied to the joint.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4

FOR LONGITUDINAL LOADING

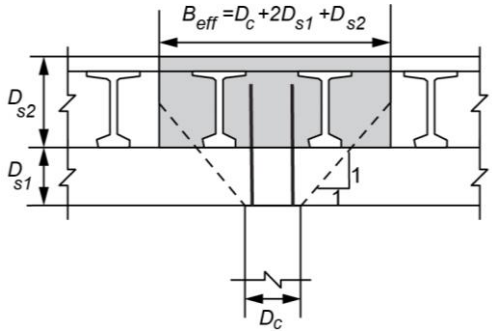
Loading Case A

For longitudinal loading, joint stress components shall be calculated as follows:

$$f_h = \frac{P_b}{D_{s2}(D_c + 2D_{s1} + D_{s2})} \qquad (3.6.12.1\text{-}5)$$

where:

f_h = average normal stress in the horizontal direction within a moment resisting joint (ksi)
D_c = diameter of the column (in.)
D_{s1} = depth of dropped portion of cap (in.)
D_{s2} = depth of superstructure (in.)
P_b = superstructure axial force at the center of the joint including effects of prestressing-positive in compression (kips)



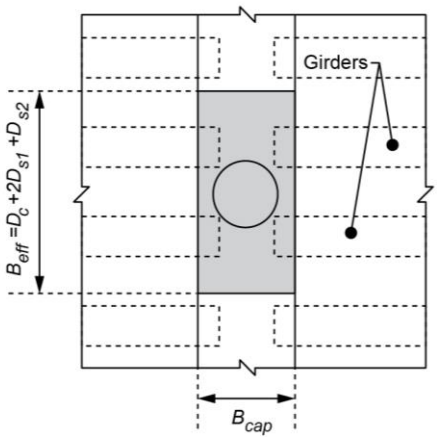
For multi-column bents the term $(D_c + 2D_{s1} + D_{s2})$ in Equations 3.6.12.1-5, 3.6.12.1-6, and 3.6.12.1-7 shall not be taken greater than the width that is geometrically tributary to the column.

In the longitudinal direction, force transfer only occurs in the upper pier cap or diaphragm where the superstructure frames into the pier cap. Although this reduces the depth of the joint region, the effect is offset by the increase in the effective width parallel to the pier cap. This increase arises due to the ability of the lower pier cap to distribute forces along its length before introducing forces into the actual joint region at the upper pier cap. The increase in effective width is based on a spreading of the load at a 45-degree angle starting at the face of the column and stopping at the center of the superstructure, which has a depth of D_{S2}.

The longitudinal horizontal stress is the superstructure axial force at the center of the joint divided by the effective area defined in the equation.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4

FOR LONGITUDINAL LOADING



$$f_v = \frac{P_c}{B_{cap}(D_c+2D_{s1}+D_{s2})} \qquad (3.6.12.1\text{-}6)$$

where:

P_c = column axial force- positive in compression (kips)

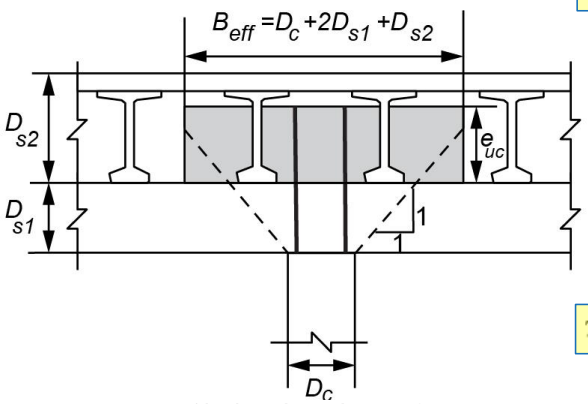
B_{cap} = pier cap width (in.)

For multi-column bents the term $(D_c + 2D_{s1} + D_{s2})$ in Equations 3.6.12.1-5, 3.6.12.1-6, and 3.6.12.1-7 shall not be taken greater than the width that is geometrically tributary to the column.

In the longitudinal direction, the effective area is still the same, but the vertical stress is the column axial force in compression divided by the effective area defined in equation 3.6.12.1-6.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4

FOR LONGITUDINAL LOADING



For multi-column bents the term $(D_c + 2D_{s1} + D_{s2})$ in Equations 3.6.12.1-5, 3.6.12.1-6, and 3.6.12.1-7 shall not be taken greater than the width that is geometrically tributary to the column.

$$v_{jv} = \frac{T_c}{e_{uc}(D_c + 2D_{s1} + D_{s2})} \qquad (3.6.12.1\text{-}7)$$

where:

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

e_{uc} = length of column and additional effective pier cap reinforcement embedded into upper stage pier cap (in.)

The column tensile force resultant, T_c, shall be obtained from the moment-curvature analysis of the column section framing into the joint. In lieu of moment-curvature analysis, T_c may be approximate as:

$$T_c = 0.7 A_{st} f_{ye} \qquad (3.6.12.1\text{-}8)$$

where:

A_{st} = total area of column reinforcement that extends into the upper cap (in^2)

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

In the next step, the designer determines the shear stress for the longitudinal loading of the joint. The effective width is still the same, but the area is determined using only the distance to the top of the column (longitudinal) flexural reinforcement extended into the upper (stage 2) pier cap section, defined as (e_{uc}).

The column tensile force is most accurately obtained from the moment-curvature analysis of the column section framing into the joint. However, there is an approximate (conservative) method provided in equation 3.6.12.1-8 that is permitted to be used.

The designer would then divide the column tensile force by the effective area defined in equation 3.6.12.1-7 to get the longitudinal shear stress.

Note that if the column space gets tighter and the pier cap depth gets larger, there is a limit on what should be used. Geometry should control all of these equations, so the designer need not double count any stress distribution.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4

FOR TRANSVERSE LOADING

Loading Case B

For loading in the transverse direction, the expressions for joint stress components f_h and v_{jv} shall be modified as follows:

$$f_h = \frac{P_{pc}}{B_{cap}(D_{s1}+D_{s2})} \qquad (3.6.12.1\text{-}9)$$

where:

P_{pc} = axial force in the integral pier cap at the center of the joint (kips).

$$v_{jv} = \frac{T_c}{(e_{uc}+D_{s1})B_{eff}} \qquad (3.6.12.1\text{-}10)$$

where:

B_{eff} = effective width of the integral pier cap joint (in.)

e_{uc} = length of column and additional effective pier cap reinforcement embedded into upper stage pier cap (in.)

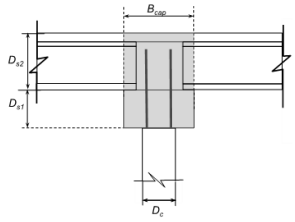


Figure C3.6.12.1-4—Critical Area for the Calculation of f_h under Loading in the Transverse Direction

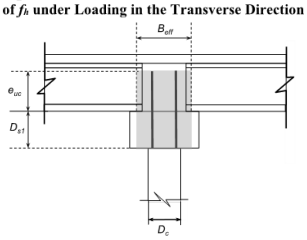


Figure C3.6.12.1-5—Critical Area for the Calculation of v_{jv} under Loading in the Transverse Direction

The next step is to go through the same process again but in the transverse direction. The vertical stress equation is the same. Here the stress in the horizontal and shear planes is reduced by the geometry of the pier cap. It is important to remember that the forces applied are specific to the direction of loading.

Note that the horizontal area is based on B_{cap}. However, the shear stress is only applied over the effective pier cap width. We will review this next.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4

FOR TRANSVERSE LOADING

The effective width of the integral pier cap joint, B_{eff}, shall consider the shape of the column framing into the joint as shown in Figure 3.6.12.1-2 and shall be determined as:

- For circular column sections:

$$B_{eff} = \sqrt{2}D_c \qquad (3.6.12.1\text{-}11)$$

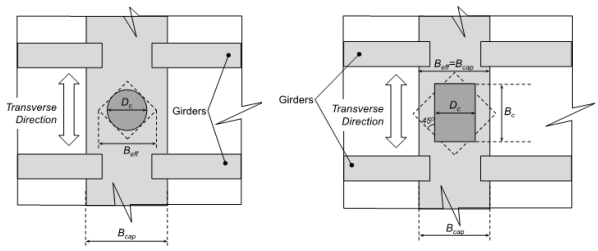
- For rectangular column sections:

$$B_{eff} = B_c + D_c \qquad (3.6.12.1\text{-}12)$$

where:

B_c = width of column measured parallel to the transverse direction.

The effective width of the integral pier cap joint, B_{eff}, shall not exceed the width of the pier cap, B_{cap}.



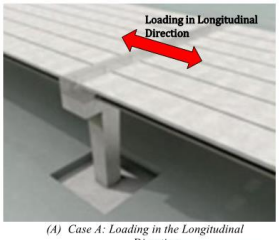
(a) Circular Column (Plan View)

(b) Rectangular Column (Plan View)

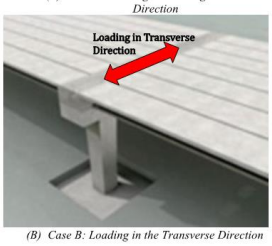
Figure 3.6.12.1-2—Effective Width of Integral Pier Cap Joint

The effective width of the integral pier cap joint (B_{eff}) shall consider the shape of the column and the orientation as it frames into the pier cap. The equations are different for circular or rectangular columns, but the upper limit is still the width of the pier cap.

3.6.12.1 Joint Proportioning for Two-stage Integral Pier Cap for SDCs C & D or Seismic Zones 3 & 4



(A) Case A: Loading in the Longitudinal Direction



(B) Case B: Loading in the Transverse Direction

Figure 3.6.12.1-1—Loading Cases for Evaluation of Joint Principal Stresses

Moment resisting joints shall be proportioned so that principal stresses satisfy the requirements of Eq. 3.6.12.1-1 and Eq. 3.6.12.1-2:

- For principal compression, p_c:

$$p_c \leq 0.25 f_c' \qquad (3.6.12.1\text{-}1)$$

- For principal tension, p_t:

$$p_t \leq 0.38\sqrt{f_c'} \qquad (3.6.12.1\text{-}2)$$

in which:

$$p_c = \left|\left(\frac{f_h+f_v}{2}\right) + \sqrt{\left(\frac{f_h-f_v}{2}\right)^2 + \left(v_{jv}\right)^2}\right| \qquad (3.6.12.1\text{-}3)$$

$$p_t = \left|\left(\frac{f_h+f_v}{2}\right) - \sqrt{\left(\frac{f_h-f_v}{2}\right)^2 + \left(v_{jv}\right)^2}\right| \qquad (3.6.12.1\text{-}4)$$

where:

f_h = average normal stress in the horizontal direction within a moment resisting joint (ksi)
f_v = average normal stress in the vertical direction within a moment resisting joint (ksi)
v_{jv} = vertical shear stress in a moment resisting joint (ksi)

Joint principal stresses shall be independently evaluated for loading in the longitudinal direction (Loading Case A) and loading in the transverse direction (Loading Case B) as shown in Figure 3.6.12.1-1.

Now the designer can calculate the principle stresses and verify that they meet the limits specified in Equations 3.6.12.1-1 for compression and 3.6.12.1-2 for tension.

3.6.12.2 Minimum Joint Shear Reinforcing for SDCs C & D or Seismic Zones 3 & 4

Transverse reinforcement in the form of ties, spiral, or hoops shall be provided.

Where principal tension in the joint, p_t, as specified in Article 3.6.12.1 is less than $0.11\sqrt{f_c'}$, the transverse reinforcement in the joint, ρ_s, shall satisfy Eq. 3.6.12.2-1 and no additional reinforcement within the joint is required.

$$\rho_s \geq \frac{0.11\sqrt{f_c'}}{f_{yh}} \qquad (3.6.12.2\text{-}1)$$

where:

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

f_c' = nominal concrete compressive strength (ksi)

ρ_s = volumetric reinforcement ratio of transverse reinforcement

where principal tension stress in the joint, p_t, is greater than $0.11\sqrt{f_c'}$, transverse reinforcement in the joint shall satisfy Eq. 3.6.12.2-1 and Eq. 3.6.12.2-2 in addition to the provisions of Article 8.13.4 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

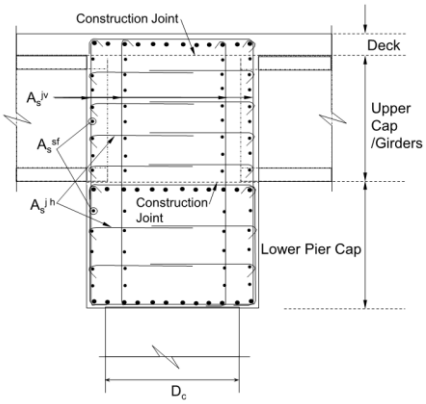


Figure C3.6.12.2-1—Joint Shear Reinforcement Details for Two-Stage Integral Pier Cap

The required vertical stirrups, A_s^{jv}, shall extend over the full depth of the pier cap, $D_{s1} + D_{s2}$. The full depth of the integral pier cap shall be considered when laying out the required additional reinforcement.

$$\rho_s \geq \frac{0.4A_{st}}{e_{uc}^2} \qquad (3.6.12.2\text{-}2)$$

where:

A_{st} = total area of column reinforcement that extends into the upper cap (in.^{2})

e_{uc} = length of column and additional effective pier cap reinforcement embedded into upper stage cap (in.)

The designer would then verify if the principle tension in the joint (in each direction) is less than 0.11*sqrt(f`c). If it is, then in each direction, apply transverse reinforcement that meets equation 3.6.12.2-1.

However, if the principle tension in the joint is greater than 0.11*sqrt(f`c), then the design needs to meet the additional requirements in the applicable direction. The design must satisfy both Equations 3.6.12.2-1 and 3.6.12.2-2 for reinforcement, but also make sure that the design is under the limits of Equations 3.6.12.1-1 and 3.6.12.1-2.

Figure 3.6.12.2-1 shows the orientation of vertical stirrups (A_{sjv}) that must extend the full depth of D_{s1} and D_{s2}. The horizontal side reinforcement (A_s^{sf}) and horizontal reinforcement (A_s^{jh}) are also shown. All detailing must be shown to fully develop these bars. Note the blue underlined text. The designer should use the AASHTO Seismic Guide Specifications provisions for additional details and equations for these bars.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Material Properties

Concrete Properties:

- $f'_c = 4$ ksi
- $f'_{ce} = 5.2$ ksi

Steel Properties (ASTM A615 Gr 60):

- $f_y = 60$ ksi
- $f_{ye} = 68$ ksi
- $f_{ul} = 95$ ksi
- $\varepsilon_{ye} = 0.0023$
- $f_{yh} = f_{tr} = 60$ ksi

On the following slides, we will demonstrate a design example of a joint design and principle stress calculations.

Material properties are shown on the slide. They are based on conventional concrete and A615 Gr 60 steel reinforcement. The expected properties are also included here, based on the over-strength plastic forces applied to the joint.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Column Geometry and Reinforcement

D_c = 6'-0"

Longitudinal Reinforcement:

- $\rho_l = 0.015$

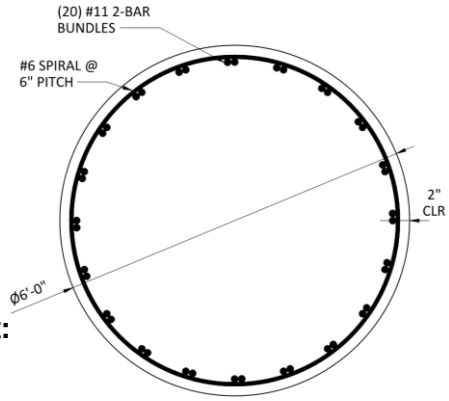
$A_{st} = 62.4\ in^2$

Transverse Reinforcement:

- $\rho_s = 0.004$

Approx. Column Tensile Force Resultant:

- $T_c = 0.7*A_{st}*f_{ye} = 2970$ kips



The column geometry is based on a 6 foot diameter column with 1.5% longitudinal reinforcement and 0.4% transverse reinforcement.

We will use the approximate column tensile force equation shown, which equals 2970 kips. Recall that a moment-curvature analysis will give you a value that is reduced from this if you are having issues getting the checks to pass and the amount of reinforcement you need in the joint area.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

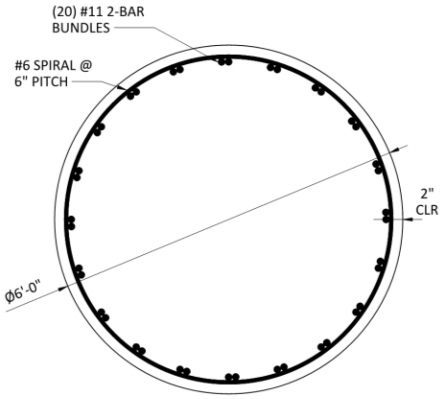
Maximum Joint Loading

Case A: Bridge Longitudinal Direction

- P_{cA} = 2080 kips
- M_{poA} = 17400 kip*ft
- $P_{bA} = V_{poA}$ = 870 kips

Case B: Bridge Transverse Direction

- P_{cB} = 3070 kips
- M_{poB} = 18800 kip*ft
- $P_{bB} = V_{poB}$ = 940 kips



We will look at two cases;

- Case A is in the longitudinal bridge direction and
- Case B is in the transverse bridge direction.

The corresponding over-strength forces are shown for each. We are not covering how to generate those forces in this example, but be mindful to get the resulting forces at the right location in the joint, not just at the top of the column.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Crossbeam Geometry

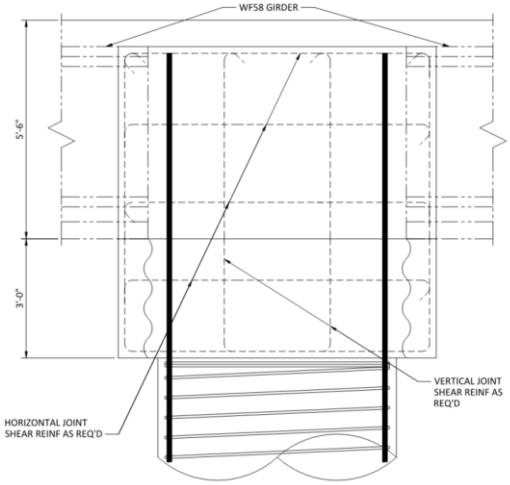
B_{cap} = 8’-0”

$B_{eff} = \min(\sqrt{2}*D_c, B_{cap})$ = 8’-0”

D_{S1} = 3’-0”

D_{S2} = 5’-6”

e_{uc} = 4’-6”



The next step is to define the crossbeam geometry.

For a 6-foot diameter column, we will use an 8-foot wide pier cap.

Since we have a circular column shape, the effective width is 8 feet for all calculations in the transverse direction.

The Semi-dropped (Stage 1) pier cap will be 3 feet high. The upper (or Stage 2) pier cap will be 5.5 feet high and the column longitudinal (flexural) reinforcement is extended 4.5 feet into the Stage 2 pier cap.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

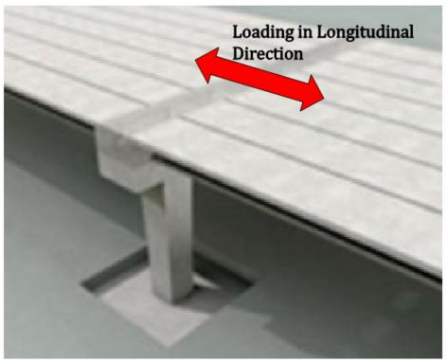
Case A: Longitudinal Loading

Average Joint Normal Stresses

$$f_{hA} := \frac{P_{bA}}{D_{s2} \cdot (D_c + 2 \cdot D_{s1} + D_{s2})} = 0.074 \cdot ksi$$

$$f_{vA} := \frac{P_{cA}}{B_{cap} \cdot (D_c + 2 \cdot D_{s1} + D_{s2})} = 0.107 \cdot ksi$$

$$v_{jvA} := \frac{T_{cA}}{e_{uc} \cdot (D_c + 2 \cdot D_{s1} + D_{s2})} = 0.306 \cdot ksi$$



We can now check Case A for Longitudinal Loading Average Joint Normal Stresses

The average stress in the horizontal and vertical plane, as well as the vertical shear stress, are calculated as shown.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Case A: Longitudinal Loading

Principal Stresses

$$p_{cA} := \left| \frac{(f_{hA} + f_{vA})}{2} + \sqrt{\left(\frac{f_{hA} - f_{vA}}{2}\right)^2 + v_{jvA}^2} \right| = 0.398 \cdot ksi$$ < 0.25*f'$_c$ = 1 ksi OK

$$p_{tA} := \left| \frac{(f_{hA} + f_{vA})}{2} - \sqrt{\left(\frac{f_{hA} - f_{vA}}{2}\right)^2 + v_{jvA}^2} \right| = 0.216 \cdot ksi$$ < 0.38* √ f'$_c$ = 0.76 ksi OK

We now calculate the principle compressive and tensile stresses using the equations in the Guide Specifications for this provision. The result is less than the specified limits, therefore we can move on to the next step.

3.6.12 Two-stage Integral Pier Cap

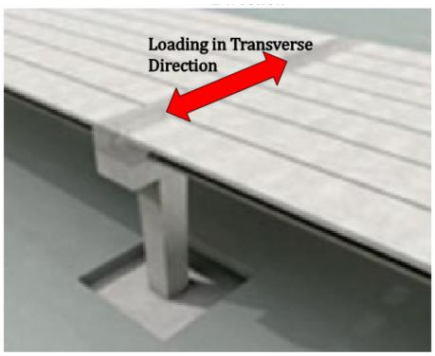
Joint Design - Principal Stress Example

Case B: Transverse Loading
Average Joint Normal Stresses

$$f_{hB} := \frac{P_{bB}}{B_{cap} \cdot (D_{s1} + D_{s2})} = 0.104 \cdot ksi$$

$$f_{vB} := \frac{P_{cB}}{B_{cap} \cdot (D_c + 2 \cdot D_{s1} + D_{s2})} = 0.158 \cdot ksi$$

$$v_{jvB} := \frac{T_{cA}}{(e_{uc} + D_{s1}) \cdot B_{eff}} = 0.368 \cdot ksi$$



We can now check Case B for Transverse Loading Average Joint Normal Stresses

The average stress in the horizontal and vertical plane, as well as the vertical shear stress, are calculated as shown.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Case B: Transverse Loading

Principal Stresses

$$p_{cB} := \left| \frac{(f_{hB} + f_{vB})}{2} + \sqrt{\left(\frac{f_{hB} - f_{vB}}{2}\right)^2 + v_{jvB}^2} \right| = 0.501 \cdot ksi$$ < 0.25*f'$_c$ = 1 ksi OK

$$p_{tB} := \left| \frac{(f_{hB} + f_{vB})}{2} - \sqrt{\left(\frac{f_{hB} - f_{vB}}{2}\right)^2 + v_{jvB}^2} \right| = 0.238 \cdot ksi$$ < 0.38* √ f'$_c$ = 0.76 ksi OK

We now calculate the principle compressive and tensile stresses using the equations in the Guide Specifications for this provision. The result is less than the specified limits, therefore we can move on to the next step.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Minimum Transverse Reinforcement in the Joint

$$\rho_{.s} := \begin{cases} 0.11 \cdot \dfrac{\sqrt{f'_{.c}}}{f_{.vh}} & \text{if } \max(p_{.tA}, p_{.tB}) \le 0.11 \cdot \sqrt{f'_{.c}} \\ \text{Max}\left(0.11 \cdot \dfrac{\sqrt{f'_{.c}}}{f_{.yh}}\right), \left(0.4 \cdot \dfrac{A_{.st}}{e_{.uc}^{2}}\right) & \text{Otherwise} \end{cases}$$

$\rho_s \ge 0.000856$

$\max(p_{.tA}, p_{.tB}) = 0.238$ ksi

$0.11 \cdot \sqrt{f'_{.c}} = 0.22$ ksi

$0.11 \cdot \dfrac{\sqrt{f'_{.c}}}{f_{.yh}} = 0.000367$

$0.4 \cdot \dfrac{A_{.st}}{e_{.uc}^{2}} = 0.000856$

Now we verify if the maximum principle tensile stress is less than or greater than 0.11*sqrt(f`c).

The result exceeds this value, therefore the transverse reinforcement in the joint also must satisfy both equations 3.6.12.2-1 and 3.6.12.2-2. In this case, the controlling case is based on the amount of longitudinal steel that extends into the Stage 2 cap.

Recall that p_{tA} is 0.216 ksi, so it is less than the principle stress limit in the longitudinal direction, therefore the minimum amount of reinforcement can be applied in that direction.

This example will not get into how to distribute reinforcement. But other checks still apply since our principle tension exceeds 0.11*sqrt(f`c) in the transverse direction.

3.6.12 Two-stage Integral Pier Cap

Joint Design - Principal Stress Example

Additional Required Reinforcement in the Joint

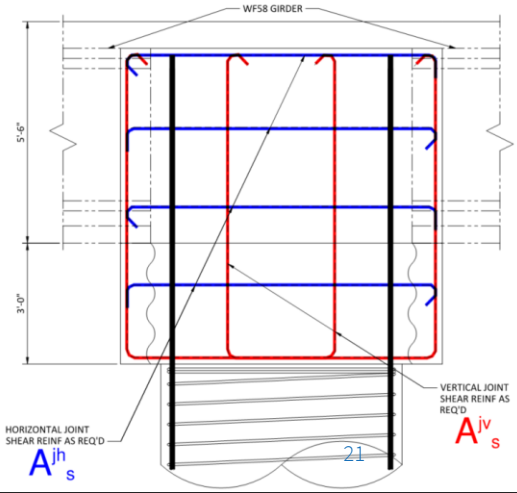
From Previous Slide:

$$\max(p_{.tA}, p_{.tB}) \geq 0.11 \cdot \sqrt{f'_{.c}}$$

Therefore:

$A^{jv}_s = 0.20 * A_{st} = 12.48 \text{ in}^2$

$A^{jh}_s = 0.12 * A_{st} = 7.49 \text{ in}^2$



As specified in the Guide Specification provisions, the designer must refer to the AASHTO Seismic Guide Specification, Section 8.4 for additional checks and criteria. We are not covering all of them in this training, but in this case, we show the amount of steel required for the vertical and horizontal stirrups. The geometry of the pier cap and column reinforcement configuration will also impact how this reinforcement is distributed through the effective area, but this should be as even as possible.

Note that the surface reinforcement is not shown, but this is based on 10% of the maximum upper or lower flexural reinforcement in the cap.

J-bars, knee joint details, and other criteria are also found in the AASHTO Seismic Guide Specifications.

End of Module 10

Available Modules

Non-seismic

1. General Provisions and Common Connections
2. Substructure Design
3. Superstructure Design
4. Bridge Systems
5. Durability of ABC Designs
6. ABC Construction Specifications

Seismic

7. Introduction to ABC Seismic
8. Seismic Connection Design and Detailing
9. Seismic Pocket and Socket Connections
10. Seismic Integral Connections

This concludes Module 10 of the ABC Training