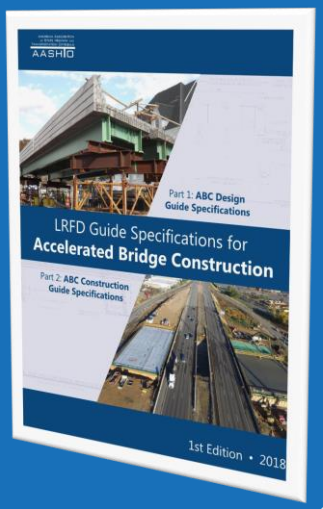
Module 2: Substructures

This Module Covers:

- Grouted couplers used in substructures
- Abutments
  - Precast integral abutment example
  - Precast cantilever abutment example
- Precast Pier Bents
  - Grouted Couplers Connection Example
  - Socket Connection Example



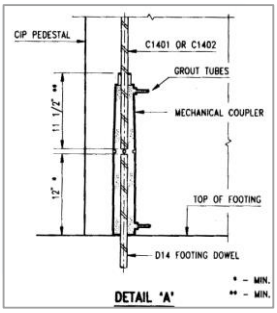
Module 2 covers design of substructures. This includes:

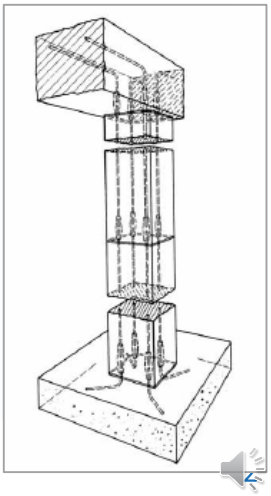
- The use of grouted couplers in substructure elements, and the design of prefabricated abutments.
- Two design examples for abutments.
  - The first example will be for a precast integral abutment bridge.
  - The second example will be for a precast cantilever abutment.
- Design of Piers including
  - Grouted coupler connections
  - One type of socket connection

Section 3: Design of PBEs

Common design approach

- Emulative design
  - The same as non-prefabricated element





Grouted couplers are mechanical devices that can connect reinforcing steel bars. These devices have been in the concrete market for well over 30 years. Grouted couplers are popular in prefabrication since connections can be made quickly with minimal changes to the design of the elements.

The coupler is a steel device that is cast into one element. The connection is made by inserting a reinforcing bar projecting from the 2nd element into the coupler. The coupler is then filled with a special high strength grout to complete the connection. The force from one bar is transferred to the coupler, which is then transferred to the bar in the second element. These devices can connect very large bars in very short distances. As you can see in the diagram to the left, a #14 bar can be developed in approximately 12 inches within the coupler.

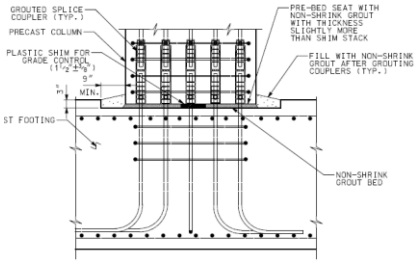
These couplers can be used to connect any type of precast concrete element. They are most commonly used in pier columns and pier caps. The Guide Specifications for ABC include provisions for use of these couplers in seismic zones. They can also be used in non-seismic applications. These couplers can be specified as either Type 1 or Type 2 as we discussed earlier today.

The couplers can be placed within the footing, at the base of the column, anywhere along the column, the top of the column, or within the pier cap.

Section 3: Grouted Bar Couplers

Couplers have a minor impact on the design

- Concrete cover on non-coupler designs
  - Design section based on required cover
  - Detail bars accordingly
- Coupler Designs
  - The diameter of the coupler is larger than the bar
  - There are multiple coupler manufacturers, and each has different dimensions (diameter)
  - In order to maintain cover over the coupler, the designer needs to:
    - Move the rebar cage for design
    - The distance has to be bracketed to all manufacturers



6 OPTIONAL RECESSED FOOTING - PREFERRED

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Grouted couplers can be used in an emulative design. They essentially take the place of a common lap splice. The couplers have a minor impact on the design of the elements. In order to provide adequate cover over the reinforcing steel and the coupler, it is necessary to move the reinforcing bar cage inward. This results in a minor reduction in the design depth of the element.

There are several manufacturers of these couplers. Each has slightly different dimensions. In typical reinforced concrete design, we simply specify concrete cover. With couplers, we need to specify the distance from the face of the element to the centerline of the bar taking into account the various coupler sizes in the market. The Guide Specifications for ABC include provisions in the construction section that includes dimensions that all manufacturers can meet. Armed with this information, the designer can locate the bars to ensure proper cover.

This slight shift of the bars affects the structural capacity of the element That the designer must take into account.

Section 3: Bar Couplers

Special detailing with bar couplers

5.5.2.1 Concrete Cover with Mechanical Connectors

The detailing of reinforcement in elements with mechanical connectors shall account for the size of the coupler. Reinforcing bars shall be detailed to provide adequate concrete cover over the connector.

The Designer shall specify the distance to the center of the spliced bars. The specified dimension shall be based on the maximum diameter of the available mechanical connectors plus the required cover.

C5.5.2.1

The mechanical connector will be larger than the connected bar. There are several manufacturers of mechanical connectors, however there is no standard connector size. This issue requires a different approach to the detailing of concrete cover.

Table C5.5.2.1-1 shows the recommended detailing dimensions for commonly available grouted connectors. These connectors are well suited for prefabrication and have been used on many successful projects. These dimensions encompass the dimensions of these common connectors. These dimensions are preliminary and not guaranteed. Designers should contact approved companies to verify these dimensions.

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Article 5.5.2.1 of the Guide Specifications for ABC describes how to detail concrete cover with mechanical connectors. The commentary refers to Table C5.5.2.1-1 For the dimensions of commonly available couplers.

This provision also requires designers to specify the distance to the center of these spliced bars.

Section 3: Bar Couplers

Special detailing with bar couplers

Table C5.5.2.1-1—Dimensions of Common Grouted Reinforcing Bar Connectors

Bar Size	Outside Diameter (in.)	Length of Connector (in.)
4	2.63	14.13
5	3.00	14.13
6	3.00	14.13
7	3.00	18.75
8	3.50	18.75
9	3.50	18.75
10	3.50	23.50
11	4.00	23.50
14	4.00	28.38
18	4.50	39.63

Other mechanical connectors can be used, provided that they can meet the strength requirements of Article 3.6.4. Designers should contact manufacturers to determine the required dimensions for detailing.

This is the table specified in the previous article. The table includes both the outside diameter of the couplers and the overall length of the coupler. The length is important for detailing elements such as pier caps. The designer needs to ensure that the coupler, combined with the projecting bar and bar bend can fit within the element.

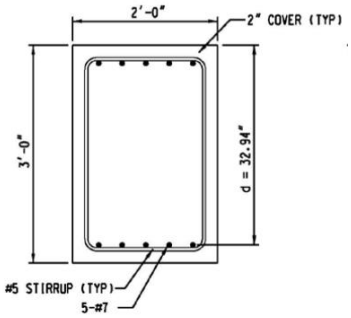
Example: Emulative Coupler Design

Example Calculation: Emulative Detailing of Reinforcement with Grouted Splice Couplers

Approach: Calculate equivalent beam sections

Note: A Beam Section is shown for simplicity. Emulative column calculations are more complex, but follow a similar approach.

Given: Section shown to the left below
Factored applied moment = 5100 in-kips



CAST-IN-PLACE DESIGN

Source: FHWA ABC Design Manual

The following is a design example for the emulative use of grouted couplers. We'll go through the redesign of a bending element from cast-in-place concrete to precast concrete.

The dimensions, details, and forces used in this example are taken from the Federal Highway Administration ABC Design Manual that we discussed previously. This is denoted by the orange header at the top of the slide.

The reinforcing where this connection will be detailed includes 5-#7 bars. The overall depth of the element is 36”. The design depth for the cast-in-place concrete section is 32.94”. The factored applied bending moment on the section is 5,100 inch-kips.

Example: Emulative Couplers Design

Using standard beam design calculations, the flexural resistance of the sections can be calculated. Adding grouted couplers affects the location of the flexural reinforcing, which affects the bending resistance. The following table shows the key variables that are affected by this change and several options for the emulative design:

	Reinforcement	Coupler dia.	Flexural Bar Cover	Design d	ΦMn	Result
Original Design	#5-#7	N/A	2.625	32.94	5157	OK
Trial 1	5-#7 w/ coupler	3	3.6875	31.88	4985	NG
Trial 2	4-#8 w/ coupler	3.5	3.875	31.81	5198	OK

Table C5.5.2.1-1—Dimensions of Common Grouted Reinforcing Bar Connectors

Bar Size	Outside Diameter (in.)	Length of Connector (in.)
4	2.63	14.13
5	3.00	14.13
6	3.00	14.13
7	3.00	18.75
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10	3.50	23.50
11	4.00	23.50
14	4.00	28.38
18	4.50	39.63

Other mechanical connectors can be used, provided that they can meet the strength requirements of Article 3.6.4. Designers should contact manufacturers to determine the required dimensions for detailing.

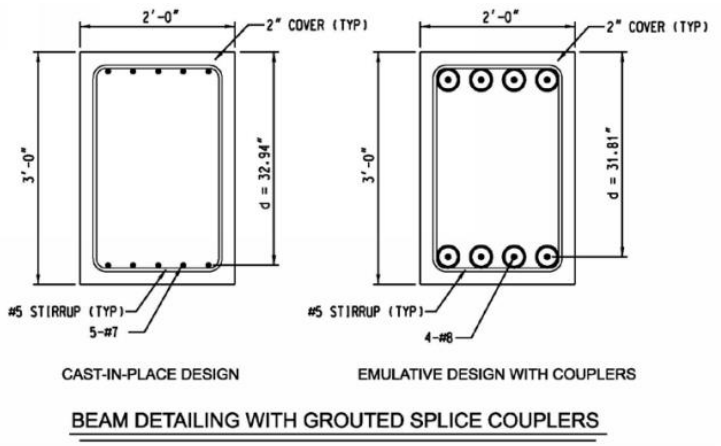
Source: FHWA ABC Design Manual

Two trial designs are shown in this example. The first is using the same bars that are detailed in the cast-in-place concrete section. This is done to demonstrate the impact that the couplers have on the flexural capacity of the section. The second trial demonstrates a redesign with larger bars to achieve the desired strength. It is common to use larger bars at wider spacing in coupler connections, since the devices are somewhat expensive and they take up more room in the element.

The flexural bar cover in the table is the distance from the face of the element to the outside edge of the flexural bar. A review of the table shows that the design depth varies based on the size of the bar and the coupler.

The results of the design show that Trial 1 design with the same bars results in a 3.3% decrease in the flexural capacity that is not acceptable. The Trial 2 design with fewer larger bars meet the flexural capacity requirements for the connection.

Example: Emulative Couplers Design

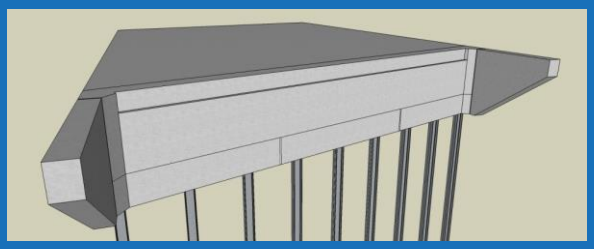


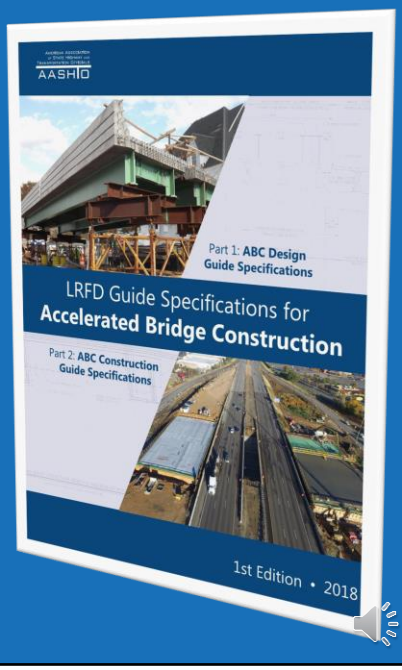
Source: FHWA ABC Design Manual

These details show the original design and the emulative redesign with couplers. The redesigned shows the reduced design depth and the location of the couplers within the section.

This is the limit of the redesign requirements. The same approach can be used for any coupler connection design.

Precast Integral Abutment Example



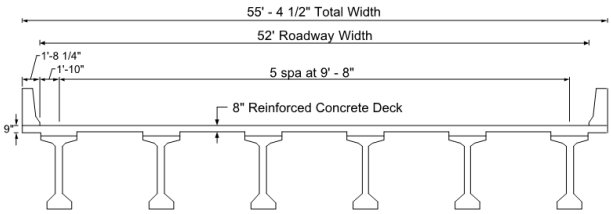


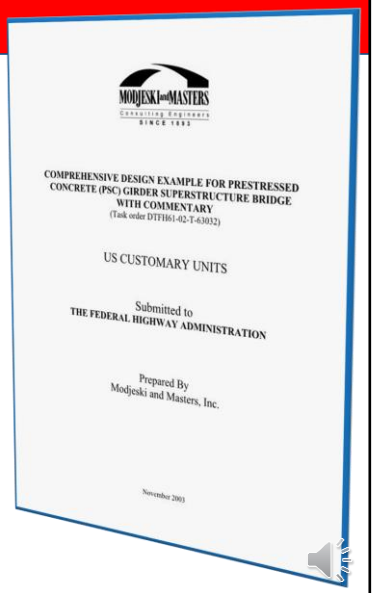
We will now go through a design example for a precast integral abutment supported by steel H piles. It is important to note that the same design would apply abutments with different pile types.

Example: Integral Abutment Design

- Use FHWA LRFD Design Examples for the basis
- Re-design using precast: Emulative design

- Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary (in U.S. customary units) [74].
- CIP concrete deck.
- Concrete parapets.
- Two-span prestressed concrete girders (AASHTO Type VI).
- Integral abutments with integral wingwalls.
- Four-column concrete open frame pier bent.

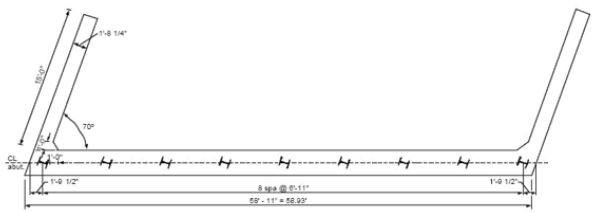


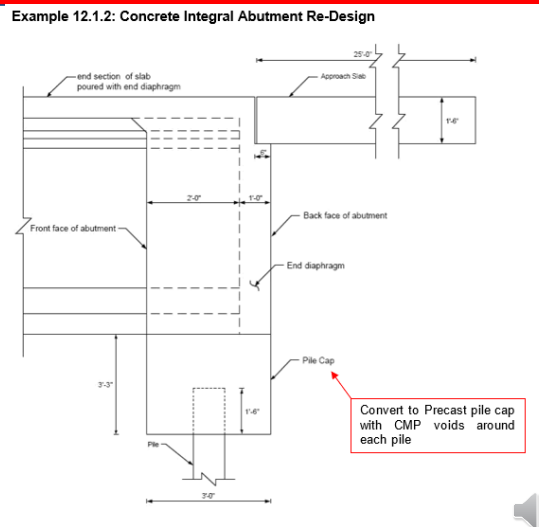


This example is taken from the Federal Highway Administration design examples for LRFD, which is denoted by the red header on the top of the slides.

This example design is for a two-span prestressed I-girder bridge with integral abutments.

Example: Integral Abutment Design





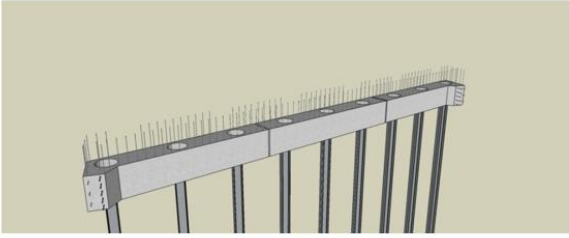
These are the details included in the design example.

For this example, we will convert the lower portion of the abutment pile cap from cast-in-place concrete to precast concrete. It is not feasible to completely convert this abutment to precast concrete. The cast-in-place concrete diaphragm is used to connect multiple elements with significant tolerances. Therefore, the opportunity for prefabrication is limited to the lower portion of the abutment stem where it connects to the piles.

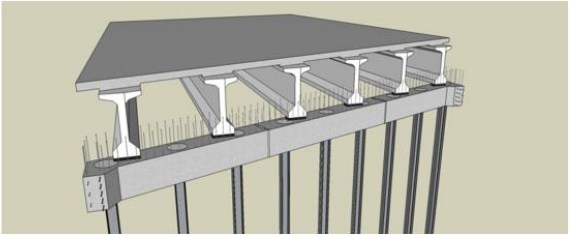
Example: Integral Abutment Design

The following figures show the proposed construction sequence:

Step 1:
Install piles
Set precast cap elements



Step 2:
Set Girders
Cast Deck
Note: Barrier Reinf.
not shown



These details show the proposed construction method for this integral abutment.

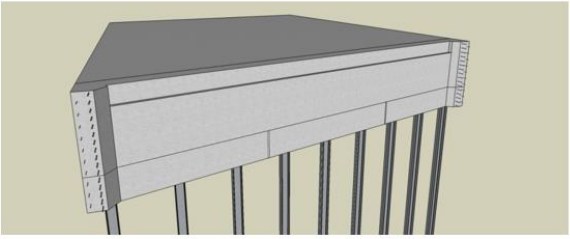
After installation of the piles, precast cap elements with corrugated metal pipe voids are placed over the piles . These are typically temporarily supported by a bedding layer of crushed stone. Once set and leveled, the cap voids are filled with normal concrete to complete the connection to the piles. High early strength concrete may also be used for this connection.

Once the void concrete gains sufficient strength, the erection of girders and installation of the deck can follow.

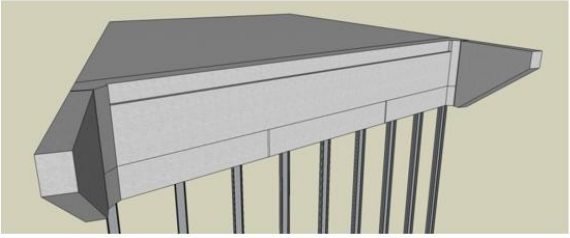
You will note that joints are shown between the elements. There are several ways to detail the connections between these elements. A reinforced concrete closure joint could be specified. Some states have used transverse post tensioning combined with a match-cast epoxied shear key. For this example, we propose to use a simple grouted shear key joint. This joint will not transmit bending moment, but will transfer horizontal shear across the joint. In this case, the connection is not truly emulative of the cast-in-place concrete option. As you will see in the example, this does not have a significant impact on the overall design or performance. This is especially true since the cast-in-place concrete diaphragm will provide additional force transfer mechanisms.

Example: Integral Abutment Design

Step 3:
Cast integral diaphragm



Step 4:
Install precast wingwalls



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Step three of the construction involves casting of the integral diaphragm.

Step four of the construction would involve the construction of the integral wingwalls. These wingwalls could be made with cast-in-place concrete or precast concrete. If they're made of precast concrete, the connection to the abutment could be made with a reinforced concrete closure joint or with grouted couplers.

We will limit this example to the design of the pile cap.

Example: Integral Abutment Design

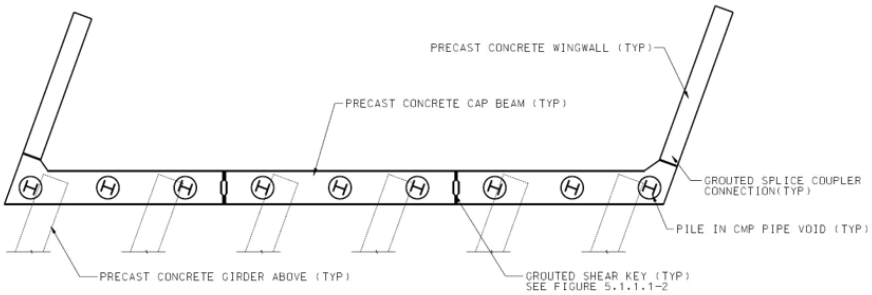

Figure 12.1.2-3 Proposed Precast Integral Abutment Pile Cap (stem)

This is a plan view of the proposed integral pile cap Showing the grouted keys between the elements. To ensure stability of the system, it is recommended that each element engage at least two piles.

The corrugated metal pipe voids are shown at each pile.

Additional voids may also be detailed between the piles to reduce the shipping and handling weight of the element. Those voids would be filled with concrete at the same time as the voids around the piles.

CMP Pipe Void Size: Tolerances

4.6.4.1 Specified Pile Pocket Width

The specified pile pocket width defined on the plans is the absolute minimum tolerable width that accounts for the pocket width tolerance, the required edge clearances for the pile, and the largest width of the pile.

Specified width of pile pocket connection:

$$S_w = 2t_{min_cl} + O.D. + 2T_w \qquad (4.6.4.1\text{-}1)$$

where:

t_{min_cl} = minimum tolerable clearance (in.)
S_w = specified width of pile or drilled shaft void (in.)
T_w = pile pocket width tolerance (in.)
O.D. = outer diameter of drilled shaft and pipe pile or measured diagonally from corners of H-pile (in.)

Table 4.6.4.1-1 – Pile Pocket Width Tolerance, T_w

G (in.)	T_w (in.)
3	3 1/4
6	6 1/8
9	9 1/8

C4.6.4.1

The value for T_w is based on fabrication and erection tolerances from Articles 2.6 and 3.9. The Designer may wish to modify the values in Articles 2.6 and 3.9. When this occurs the Designer should use Table C4.6.4.1 to calculate the pocket width tolerances. The factor associated with T_w in this table is based on using the probability occurrence of the maximum additive tolerances.

Table C4.6.4.1 – Pocket Width Tolerance, T_w

G (in.)	T_w (in.)
3	± 0.72 (P+G+L_H)
6	± 0.81 (P+G+L_H)
9	± 0.86 (P+G+L_H)

where:
G = maximum pile or drilled shaft lateral installation tolerance (in.)
P = maximum fabrication pocket location tolerance (in.)
L_H = horizontal erection tolerance of precast member (in.)
T_w = width of pile void connection tolerance (in.)

Note the maximum pile or drilled shaft lateral installation tolerance, G, typically varies between States. Table C4.6.4.1 gives a typical range of values of G to determine the void width tolerance.

In order to size the corrugated metal pipe voids, we need to investigate the impact of pile driving tolerances.

The header on the top of the slide is now purple, which indicates that we are using the NCHRP Project 12-98 reference guideline document for tolerances for Prefabricated Bridge Elements and Systems. This document provides the pile pocket width tolerance in a table that accounts for the pile driving tolerance that is specified.

For this example, we will assume a pile driving tolerance of plus or minus 3 inches. In this scenario, the pile pocket width tolerance equals 3¼”.

The specified width of the pile pocket is shown in Equation 4.6.4.1-1 and is equal to two times the minimum tolerable clearance at the edge of the pile plus the outside diameter of the pile , plus two times the pile pocket width tolerance.

CMP Pipe Void Size: Tolerances

Givens: Refer to Design Step 7.1.3 of the reference document [74]
Width of cap: 36"
Size of pile: HP12x53
Depth = 11.78"
Width = 12.05"

Assumptions:
Pile Installation Tolerance = ± 3" total in any direction
(assume driving template is used)

Minimum Clear Distance between pile corner and void = 0"

CMP Corrugation amplitude = 0.75 in.

Effective pile diameter = *Dpile* = $\sqrt{11.78^2 + 12.05^2} = 16.85\ in.$

Pile driving tolerance = *Tpile* = *3 in.*

Specified Minimum width of socket = *2(0") + 16.85" + 2(3.35") = 23.35"*

Round to next largest 3" increment (pipe standard), use D = 24"

Specified width of pile pocket connection:

$$S_w = 2t_{min_cl} + O.D. + 2T_w \qquad (4.6.4.1\text{-}1)$$

where:

t_{min_cl} = minimum tolerable clearance (in.)
S_w = specified width of pile or drilled shaft void (in.)
T_w = pile pocket width tolerance (in.)
O.D. = outer diameter of drilled shaft and pipe pile or measured diagonally from corners of H-pile (in.)

Table 4.6.4.1-1 – Pile Pocket Width Tolerance, T_w

G (in.)	T_w (in.)
3	3 1/4
6	6 1/8
9	9 1/8

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For this design example, the following details are called out in the FHWA design example :

1. The size of the pile is an HP12X53. The effective pile diameter of this section is equal to the diagonal dimension from the edges of the flanges. It is calculated as 16.85 inches as shown.
2. The width of the pile cap is 36 inches

The following assumptions are made for this example :

1. The pile driving tolerance is limited to plus or minus 3 inches in any direction. This tolerance is smaller than what some states use in their standard specifications. However, contractors have proven that this tolerance can be maintained if a driving template is used during construction. If a larger tolerance is desired, a larger void would be required, leading to a wider abutment cap.
2. We will assume that the minimum clear distance between the corner of the pile and the side of the void is 0". Since the pile is an H section, this will not prohibit the placement of concrete in the void.
3. We will assume the amplitude of the corrugations in the pipe is limited to ¾". This does not affect the capacity of the connection. It is used in this example to check if the void can fit within the pile cap width of 36 inches.

With this data we can calculate the specified minimum width of the socket, which equals 23.35 inches. We will round this up to the next largest 3 inch increment, which is 24 inches. Manufacturers use a 3 inch increment for pipe sizes.

CMP Pipe Void Size: Tolerances

Distance from outside edge of CMP to face of cap = CMP_{cov}

CMP_{cov} = (36" stem – (24" CMP + 2*0.75" amplitude) / 2

= 5.25" $\geq$ 5.125" OK

Note:

The assumptions for clear distances were chosen as small values in order to fit the CMP within the 36" wide stem section. If larger tolerances were desired, the CMP would need to be larger and the stem width would need to be increased in order to accommodate the void. This would necessitate a re-design of the stem and piles.

Figure 12.1.2-4 shows the details of the final CMP void.

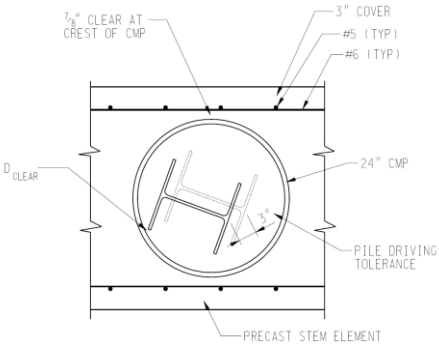


Figure 12.1.2-4 Detail of CMP Void and Pile Tolerance

We now check to ensure that the CMP void will fit within the 36 inch width of the pile cap.

Review the calculation on the top of the slide

Note that some of the assumptions, such as driving tolerance and minimum clear distance for this example were set to fit the pile void within the 36 inch section. If larger values for these assumptions were chosen, the void size would need to increase, requiring a wider pile cap.

The figure shows the final details of the cap void showing the offset of the pile due to pile driving tolerances.

Example: Integral Abutment Design

The pile cap is designed to support the girders, deck, haunches, and abutment diaphragm. In the FHWA design example, this is referred to as "Stage 1". The abutment end diaphragm is designed to support all other loads and is referred to as "Stage 2".

Punching shear design is based on AASHTO LRFD ABC GS Article 3.6.6.6

Givens:
f.c = 4 ksi (assumed strength of void concrete)

*Circumference of pipe = 24 in. * π = 75.4 in.*

Length of pipe above pile top = 39" – 18" = 21"
(Figure 7.1-1 [74])

Punching Shear area per linear foot of pipe
Acv = 75.4"(12 in./ft) = 905 in²

Nominal Punching Shear Resistance
Vn = 0.13(f.c)^1/2 Acv
= 0.13(4)^1/2(905)
= 226 kips per linear foot of pipe

Nominal Resistance of CMP Void for this pile cap
*= 226k/ft * 21"/(12"/ft) = 396 kips*

$$V_n = 0.13\sqrt{f'_{cp}}\,Acv \qquad (3.6.6.6\text{-}1)$$

where:

f'_{cp} = nominal compressive stress of the pocket concrete (ksi)
A_{cv} = cylindrical shaft area of the pocket (in.^{2})

The cylindrical shaft area of the pocket A_{cv}, shall be calculated as:

$$A_{cv} = \pi\, d_v\, h_v \qquad (3.6.6.6\text{-}2)$$

where:

d_v = inside diameter of the CMP pocket (in.)
h_v = effective height of the CMP pocket (in.)

The next step in the design is to calculate the vertical shear resistance of the CMP void.

The assumed strength of the concrete in the void is 4ksi.

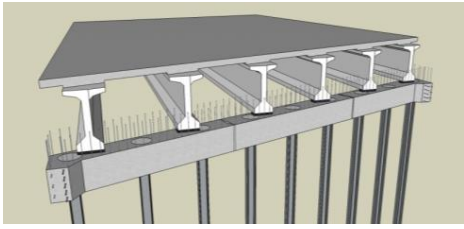
The length of the CMP pipe above the pile top is equal to the overall cap height minus the embedment of the pile into the void, which equals 21 inches. The overall cap height is taken from the FHWA design example.

The guide specification provisions are shown on the right side of the slide. The next step is to calculate the cylindrical shaft area of the pocket. The circumference of the pipe equals 75.4 inches. For this example, we will calculate the resistance per linear foot of pipe. Therefore the cylindrical shaft area is equal 75.4x12 = 905 square inches per foot of pipe.

Equation 3.6.6.6-1 is then used to calculate the nominal punching shear resistance, which is 226 kips per foot of pipe. Therefore, the total nominal resistance is equal to 226*21/12 = 396 kips.

Example: Integral Abutment Design

The actual load on the CMP void is the dead load of the beams, deck, and abutment diaphragm. The diaphragm can help to resist other loads that are applied to the structure after the diaphragm is cast.



The factored maximum total girder reactions for Stage 1

PSI (TOTAL) = 2(147.4) + 4(158) = 926.8 kips
(Design step 7.1.3.2 [74])

Stage 1 load per pile = 926.8/9 piles = 103 kips per pile

Check capacity of CMP Void

Vri = ΦVni

And the design shall satisfy:

Vri ≥ Vui
Vui = factored shear force = 103 kips
*Vri = ΦVni = 0.90*396 kips = 356 kips*
356 kips ≥ 103 kips

Therefore, the CMP void shear resistance is adequate

Using simple statics we can calculate the factored force on each pile. the result is 103 kips per pile.

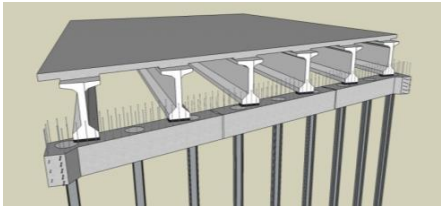
The ultimate resistance of the void connection is equal to the resistance factor times the nominal capacity, which equals 356 kips.

As you can see, the resistance of these voids is significant. The height of this pile cap is quite small, yet there is ample resistance provided. This is a very conservative calculation in that we are neglecting the resistance of the concrete diaphragm cast over the voids, which will provide in additional load path for superstructure forces into the piles.

Example: Integral Abutment Design

Step 3: Design of Stem Reinforcement

The intent is to use a precast concrete pile cap combined with precast concrete wingwalls. Figures 12.1.2-2 &3 shows the intended change from CIP concrete to precast concrete for the abutment pile cap. The only significant change from the CIP option is the use of two grouted shear keys. The CIP pile cap was designed as a continuous beam. The precast cap is similar to a continuous beam; however, the shear keys create a hinge in the cap. Figure 12.1.2-5 shows two free body diagrams for the cap element. The roller supports represent the piles and the beam represents the cap.



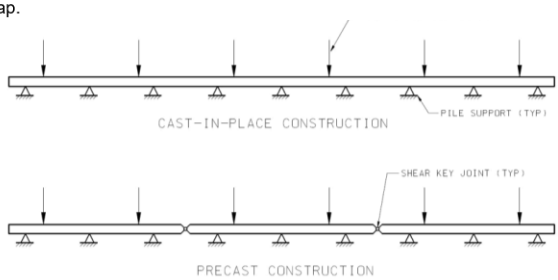


Figure 12.1.2-5 Free Body Diagrams for Pile Cap
Top: CIP Construction
Bottom: Precast construction based on the details shown in Figures 12.1.2-2&3

The next step in the design process is to check the reinforcing in the cap. This is an example where true emulation is not followed. Minor changes to the free body diagram for the cap are called for due to the grouted shear keys that are proposed.

The top diagram shows the cast-in-place option with a continuous pile cap.

The bottom diagram shows the proposed precast construction. In this free body diagram, we insert hinges at these locations. This structure is still stable and determinant, therefore we can calculate forces using standard statics. A standard computer frame analysis program could also be used.

Example: Integral Abutment Design

The FHWA design example treated the cap a simple span. The resulting answer for positive moment bending caused by vertical forces was multiplied by 0.8 to account for continuity. The same moment was applied to the negative moment section. A continuous beam design would give more accurate results.

In this re-design, the positive moment will be calculated assuming simple span action, which is conservative. The following is a re-calculation of the cap beam positive moment due to vertical loads. The re-design of negative flexure can be re-calculated based on the hinge located mid-way between the piles. AASHTO Section 5.6.3 recommends that a strut and tie model be used for the design of a pile cap. This can be accomplished using the free body diagram shown above.

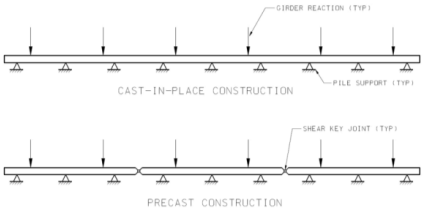


Figure 12.1.2-5 Free Body Diagrams for Pile Cap
Top: CIP Construction
Bottom: Precast construction based on the details shown in Figures 12.1.2-2&3

The FHWA design example treated the cap as simple spans. To account for continuity, the moments were multiplied by 0.8.

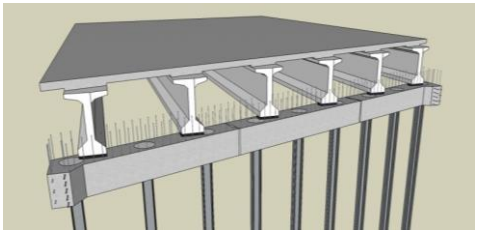
For this example, we will conservatively assume simple spans between piles to simplify the calculations. As previously stated, a more detailed analysis could also be performed.

We will be using a sectional approach for this example. It is understood that AASHTO LRFD BDS Article 5.6.3 recommends the use of strut and tie modeling for pile caps. The lower free body diagram could be used for this purpose as well.

Example: Integral Abutment Design

Positive Flexural design due to vertical forces
(refer to page 7-17 of the FHWA design example [74])

Assuming the beam is located mid-way between the piles.
Do not use 0.8 factor to adjust for continuity due to the presence of the shear keys.



$Mu = Pul/4 + wul^2/8$

l = spacing of piles = 6.917 ft

$Mu = 189.6(6.917)/4 + 6.96(6.917)^2/8$

$= 369.5$ k-ft

Resisting moment of the cap section

$Mr = 490$ k-ft > $Mu = 369.5$ k-ft **OK**

We will now check the flexural design of the pile cap due to vertical forces using the equations in the Federal Highway design example.

The bending moment is conservatively calculated assuming that the girder reaction is midway between piles. The dead load of the diaphragm is applied as a uniformly distributed load.

The 0.8 factor for continuity is conservatively not used in this example due to the presence of the shear keys.

We are not proposing to change the reinforcing shown in the Federal Highway pile cap example. This means that the bending resistance is the same at 490 ft-kips. As you can see the resisting bending moment exceeds the ultimate moment.

Example: Integral Abutment Design

Negative Flexural design due to vertical forces near shear key
(refer to page 7-17 of the FHWA design example [74])

Mu = *wu* $(l/2)^2/2$

Cantilever beam with span of l/2
There are no beam reactions in this region

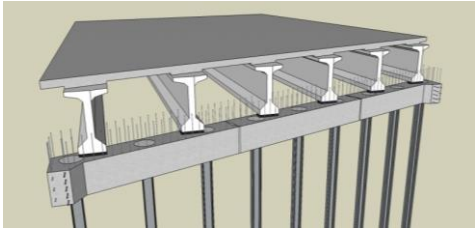
l = *spacing of piles*

l/2 = *6.917/2 = 3.46 ft*

Mu = $6.96(3.46)^2/2$

= *41.67 k-ft OK by inspection*

Shear design check
The shear design equation for the FHWA LRFD design example is based on the beam being located directly adjacent to a pile. This is still the case with the precast option chosen, therefore the cap design for shear is adequate for the precast option.



The next step is to check the area of the cap from the shear key to the nearest pile. As you can see from the diagram, there are no beam reactions in these areas with the proposed layout. If a beam were to be located in these areas, the reaction forces would need to be added to this calculation.

This section of the cap is treated as a simple cantilever with an applied uniform load. As you can see, the section has ample capacity to resist the forces.

The force is used for shear in the Federal Highway example have not changed due to this configuration, therefore the shear reinforcing would be adequate.

Again, a more complex strut and tie model could be used for this design.

Example: Integral Abutment Design

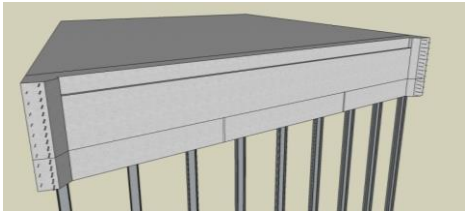
Step 4: Design of Integral Abutment Diaphragm

The FHWA LRFD design uses a built-up composite concrete beam that is made up of the pile cap and the integral diaphragm. The use of the precast cap with the shear key leads to a discontinuity of the longitudinal cap reinforcement at the key. Therefore, the design of the diaphragm will not account for the composite action of the pile cap.

Design of diaphragm for flexure

The maximum positive moment is calculated assuming the girder reaction is applied at mid-span between piles taking 80% of the simple span moment.

Note that this approach is conservative since it does not take into account the dead loads transferred into the cap beam below or the composite action between the cap beam and the diaphragm.



The FHWA example treats the cap and diaphragm as a composite section for vertical loads. The shear keys will result in a discontinuity in this diaphragm.

For this design, we will conservatively assume that the pile cap provides no flexural resistance for vertical loads for the diaphragm. We will simply reinforce the cast-in-place diaphragm assuming that the cap is not present.

Example: Integral Abutment Design

Mu = 498.2 k-ft (see FHWA example page 7-21)

Mn = As fy (ds – a/2)

Where:

As = use 4 #8 bars
= 4(0.79) = 3.16 in²

fy = 60 ksi

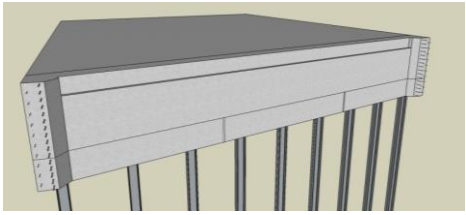
ds = depth of dia. – cover – stirrup – ½ bar dia.
= 84.75" – 3" – 0.625 – ½(1.0)
= 80.625"

a = As fy / 0.85 f'c b
= 3.16(60)/[0.85(3)(36)]
= 2.07"

Mn = 3.16(60)(80.625 – 2.07/2)/12
= 1257 k-ft

Therefore,

Mr = 0.9(1257)
= 1132 k-ft > Mu = 498.2 k-ft ***OK***



By inspection:

- Mr > 4/3(Mu). This means that the minimum reinforcement requirements of S5.7.3.3.2 are satisfied.

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We will assume that it will be necessary to add 4-#8 bars to provide positive flexural resistance over the cap joints.

The calculations shown are standard reinforced concrete design provisions. The result is that the resistance is significantly higher than the applied ultimate moment.

Example: Integral Abutment Design

Step 5: Design of backwall as a horizontal beam resisting passive earth pressure

The resistance of the backwall can be conservatively checked assuming that the pile cap has no resistance to passive earth pressure. The pile cap will transmit the passive pressures into the diaphragm, therefore the backwall moments can be applied to the section that is above the pile cap.

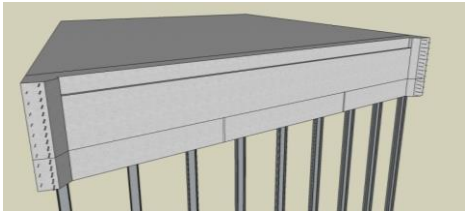
Check reinforcing in diaphragm for total backwall moments

Mu = 300 k-ft/ft
(see page 7-28 of the FHWA LRFD Design Example)

Mn = As fy (ds – a/2)

Where:
As = area of longitudinal reinforcement bars at front face (tension side) of the abutment diaphragm (7 #6 bars)
= 7(0.44)
= 3.08 in^2

fy = 60 ksi



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The next step is to check the backwall for resistance to passive earth pressure that is brought on by thermal expansion of the integral bridge, which is pushing against the backfill soils.

Again, we will conservatively assume that the pile cap provides no resistance to these forces due to the presence of the shear keys.

We will apply the ultimate moment from the FHWA design example to the diaphragm section only.

Example: Integral Abutment Design

ds = width of backwall – cover – vert. bar dia. – ½ bar dia.
= 36" – 3" – 0.625" – ½ (0.75)
= 32.0"

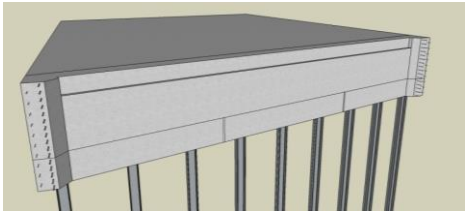
a = As fy / 0.85 f'c b (b = height of diaphragm = 80.75")
= 3.08(60)/[0.85(3)(80.75)]
= 0.897"

Mn = 3.08(60)(32" – 0.897"/2)/12
= 486 k-ft

Mr = 0.9(486)
= 437 k-ft > Mu = 300 k-ft ***OK***

By inspection:

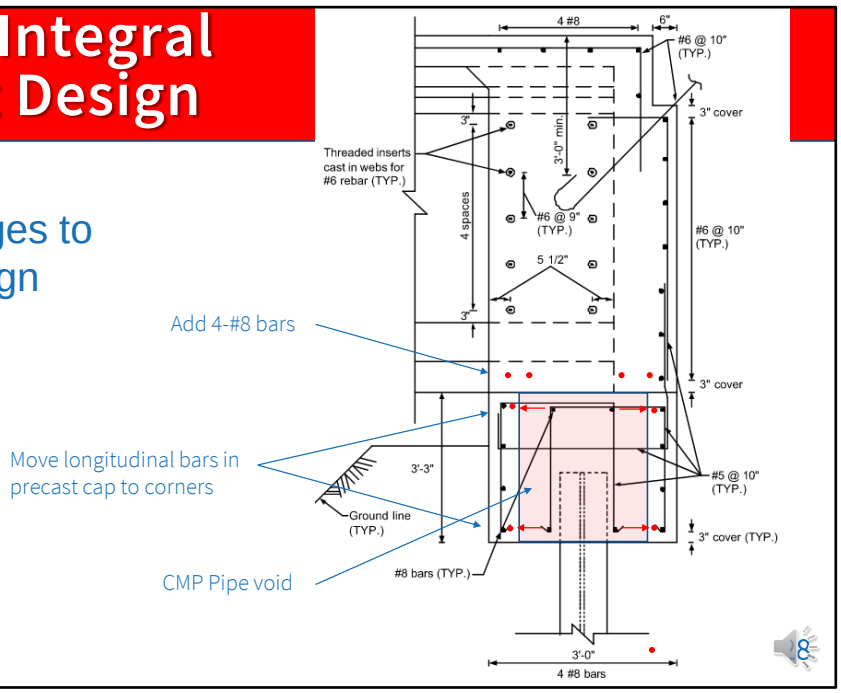
- Mr > 4/3(Mu). This means that the minimum reinforcement requirements of S5.7.3.3.2 are satisfied.



Using standard reinforced concrete design principles, the diaphragm was found to be adequate to resist the forces.

Example: Integral Abutment Design

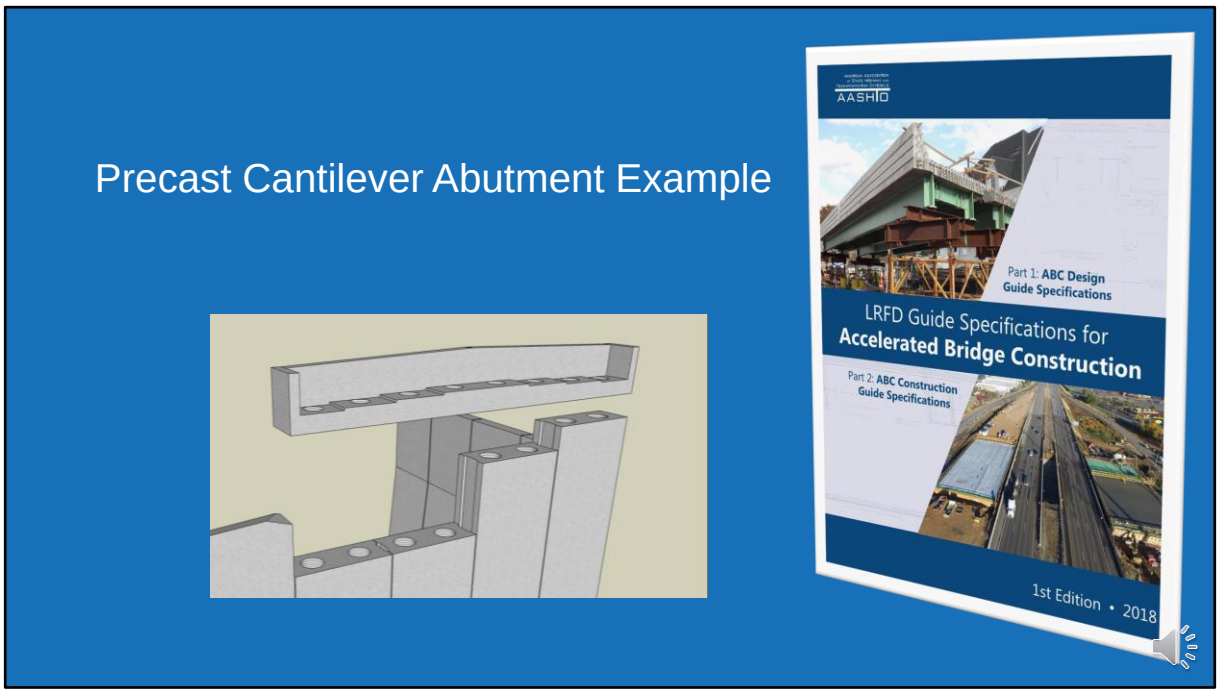
Reinforcing changes to cast-in-place design



This is a detail showing the modifications to the design with a precast cap. The following changes are required :

- Add 4-#8 bars along the length of the diaphragm below the beam.
- Adjust location of the longitudinal #8 bars in the cap to the corners to provide room for the corrugated metal pipe void.
- Minor detailing adjustments would also be required for the shear reinforcing around the pipe void in the cap.

This concludes the design example for an integral abutment. Before we move on, are there any questions about what we have just covered?



We will now go through a second design example for a substructure made with precast concrete. In this example we will investigate design of a cantilever abutment built with precast elements.

Example: Cantilever Abutment Design

Reverse engineer FHWA LRFD design example

In general, the change of a CIP cantilever abutment to precast concrete involves the connection of the wall stem to the footing. The use of a precast cap beam is also proposed. This is done to make the abutment stem elements similar and to provide a continuous beam seat.

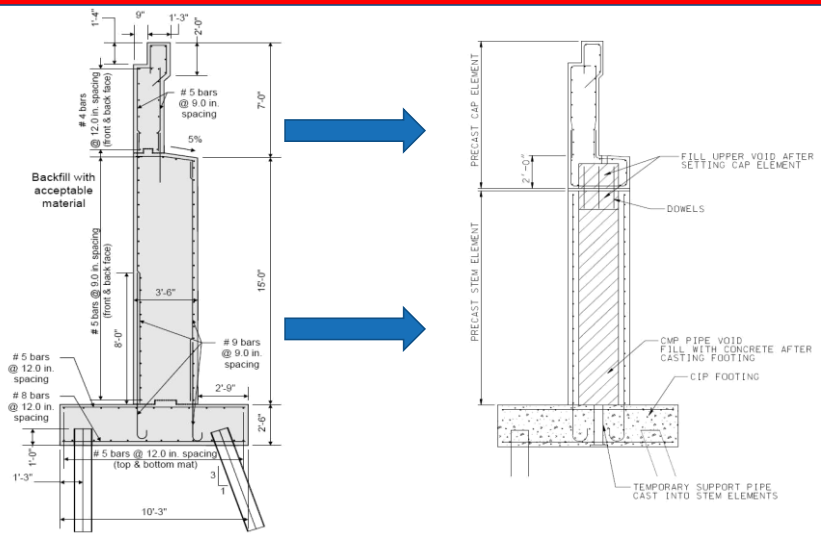
The abutment is a standard cantilever abutment supported on spread footings. The proposed re-design involves the re-design of the following:

Element	Proposed lower connection type
Wall Stem	Connection to footing by placing the wall stem on temporary supports with projecting reinforcing dowels and casting the footing under the stem.
Bridge-seat/backwall element	Corrugated Metal Pipe void with reinforcing dowels

In this example, we will take a cast-in-place concrete cantilever abutment that is included in the Federal Highway design examples and convert it to precast concrete. Again, the header at the top of the slide is red to indicate that we are using the Federal Highway design example as a reference.

In this example, we will look to convert the wall stem and bridge seat portions of the abutment from cast-in-place concrete to precast concrete. We will keep the cast-in-place concrete pile cap footing, as it is difficult to detail a precast footing supported on piles. There are ways to build a concrete pile cap quickly without precasting. We will demonstrate this in the example

Example: Cantilever Abutment Design

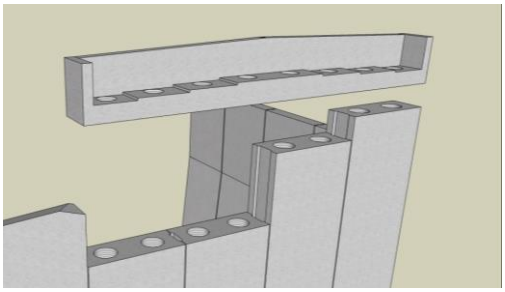


These details show the proposed approach. The concept is to precast the stem of the abutment with corrugated metal pipe voids. The purpose of the voids is twofold. First it will reduce the shipping and handling weight of these stem pieces , which can get very heavy due to their thickness. The second reason for the corrugated metal pipe voids is to provide a means of connecting a precast abutment seat element on top of the wall stem.

Example: Cantilever Abutment Design

Upper portion concept

- Keep reinforcing above bridge seat the same
- Detail a cap element with all the complexities of the bridge seat
  - One complex element
  - Lower elements are simple and less expensive
  - The cap element will tie the lower elements together
  - Moment demand on cap connection is minimal, which makes design easy and inexpensive
  - Connect the two using a pocket connection
  - Based on PCI northeast recommended details



This approach of using an abutment seat element is included in the PCI Northeast Recommended Guide Details for Precast Substructures that can be found at www.pcine.org.

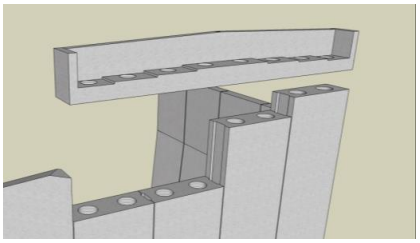
The concept of this approach is to keep all the wall elements simple and similar to reduce cost. The complexities of the beam seat detailing including variable beam seat elevations, anchor rods, and shear keys are left to one complex element. This approach simplifies the overall fabrication process for the abutment.

The location of the connection between the wall panels and the beam seat element is located just below the beam seat where there is very low moment demand on the section. A corrugated metal pipe void can also be cast in the beam seat element. The connection is made by inserting reinforcing steel dowels in the void and filling the void with concrete.

Example: Cantilever Abutment Design

Upper portion concept

The approach for this design is to verify whether or not there is tension across the connection. If there is no tension, then the connection will not need a significant amount of reinforcement. This approach is not new to the bridge construction industry. Stone masonry gravity abutments with un-reinforced joints have been in service for hundreds of years. The calculation of overturning for this example will be at the Service I limit state (un-factored loads).



We will start with the connection of the beam seat element to the wall panels.

The approach for this design is to verify whether or not there is tension across the connection. In most cases tension will not be present.

The idea of having a lightly reinforced joint in an abutment comes with historical similarities. Stone masonry abutments have many unreinforced horizontal joints. They are designed as gravity sections where all joints are in compression under all loads. Our approach will be similar.

Example: Cantilever Abutment Design

Calculation of forces: All forces are calculated on a "per foot of wall" basis

Girder Reactions
(see page 7-34 of the FHWA LRFD Design Example [75])

$R_{DC\ TOTAL}$ = 7.66 k/ft
$R_{DW\ TOTAL}$ = 1.20 k/ft

R_{DL} = 7.66 + 1.20 = 8.86 k/ft

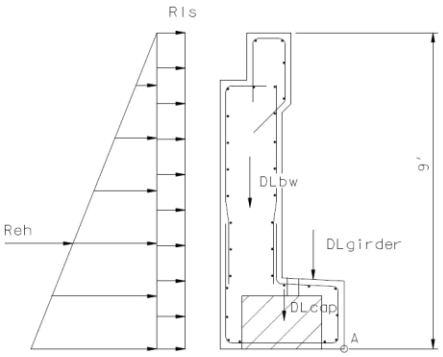
Active Soil Pressure on rear face of the cap element
(see page 7-19 of the FHWA LRFD Design Example [75])

$p = k_a \gamma_s z$

where:
$k_a = 0.3$
$\gamma_s = 0.120$ kcf
z = height of cap = 9 ft

$p = 0.3(0.120)(9) = 0.324$ k/ft^2

Reh = Horizontal soil force acting on wall
$= \frac{1}{2} p z$
$= \frac{1}{2} (0.324)(9)$
$= 1.46$ k/ft



In this calculation we will calculate the overturning forces and the resisting forces.

The overturning forces consist of active soil pressure and surcharge.

The resisting forces include the dead load of the cap and the dead load girder reaction, assuming that the girders are installed prior to backfilling, which would need to be specified.

Example: Cantilever Abutment Design

Live Load Surcharge

H = height of wall = 9 feet

Use AASHTO Table 3.11.6.4.1 to obtain equivalent height of soil

Interpolate

H = 5 ft, → heq = 4.0 ft

H = 10 ft, → heq = 3.0 ft

Therefore, for H = 9 ft → heq = 3.8 ft

Change in earth pressure due to LL surcharge

$\Delta_p = k_a \gamma_s heq$

$= 0.3(0.120)(3.8)$

$= 0.137\ k/ft^2$

Rls = Force due to live load surcharge

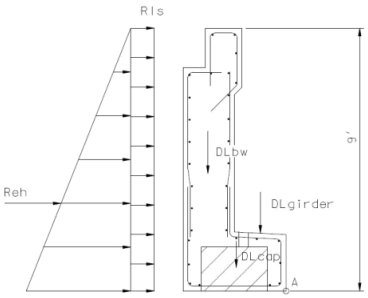
$= \Delta_p H$

$= 0.137(9)$

$= 1.23\ k/ft$

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0



These calculations show the calculation of live load surcharge using AASHTO Article 3.11.6.4.

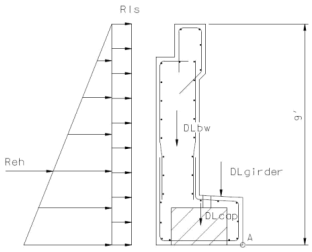
Review the calculation

Example: Cantilever Abutment Design

Dead load of precast cap

DLbw = Backwall Dead Load
(see page 7-34 of the FHWA LRFD Design Example [75])
= 1.71 k/ft

DLcap = Seat Dead Load
= 2'(3.5')(0.15kcf)
= 1.05 k/ft



Calculate Overturning Moment

Sum moments about point A

$\Sigma M_{OT\,A} = Reh(Yeh) + Rls(Yls)$

Where:
Yeh = H/3 = 3ft
Yls = H/2 = 4.5ft

$\Sigma M_{OT\,A} = 1.46(3) + 1.23(4.5) = 9.92$ *k-ft*

Calculate Resisting Moment

Sum moments about point A

$\Sigma M_{RESIST\,A} = DLbw(Xbw) + DLcap(Xcap) + R_{DL}(X_{Girder})$

Where:
Xbw = 3.5-0.833 = 2.67ft
Xcap = 3.5/2 = 1.75ft
X_{Girder} *= 7.5"/12 = 0.625ft*

$\Sigma M_{RESIST\,A} = 1.71(2.67) + 1.05(1.75) + 8.86(0.625)$
= 11.81 k-ft

$\Sigma M_{RESIST} > \Sigma M_{OT}$ ***Therefore the connection is in compression***

The top portion of this slide shows the calculation of the dead load of the cap and bridge seat. The download reaction of the girder is taken from the Federal Highway design example.

The sum of the overturning moments equals 9.92 kip-ft.

The sum of the resisting moments equals 11.81 kip-ft

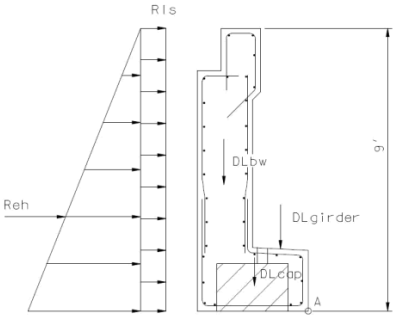
Therefore the connection is in compression across the entire width.

If tension was found at the rear of the joint, the design would change by adding reinforcing within the corrugated metal pipe void to resist the overturning moment.

Example: Cantilever Abutment Design

The result of this calculation is that the connection does not need reinforcement. Dowel bars will be used in the connection to provide positive reinforcement for shear in addition to the sear capacity of the concrete within the CMP void. By inspection, the void concrete and dowel bars will be sufficient to resist the lateral forces acting on the cap.

Construction sequence: This calculation assumes the girder and deck are placed prior to backfilling. Alternatively, the dowel bars could be designed to take the moment if the section is backfilled prior to placing girders. Designers should note the assumed construction sequence on the plans.



The result of this calculation shows that the connection does not need reinforcement. Dowel bars will be used to simply provide a positive connection between the cap and the wall panels below.

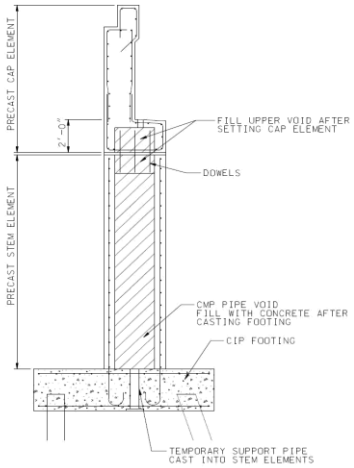
If the dead load of the superstructure is not applied prior to backfilling, then the dowel connection could be designed for the overturning flexural moments.

Example: Cantilever Abutment Design

Step 2: Design of Stem Element and Connection to footing

The approach for the abutment stem design is to make all the stem sections a uniform size. By separating the bridge seat from the stem sections, it is possible to keep the detailing of the stem simple and consistent. Corrugated Metal Pipe (CMP) voids are used for this element for two reasons:

- They can be used to reduce the weight of the element for shipping and handling.
- The void can be used to make the connection to the cap element.



The design of the stem reinforcing is identical to the cast-in-place concrete option. We are choosing a simple connection developed by The Washington State DOT where a precast stem is fabricated with the flexural steel projecting from the bottom of the stem. A temporary support is also cast into the stem to provide a means of setting the panel before casting of the pile cap footing.

Corrugated metal pipe voids will be used for two purposes :

1. They will reduce is shipping and handling weight of the wall stem panels
2. They provide a means of connecting the beam seat cap to the panels as previously discussed

Individual wall panels could be connected with a simple grouted shear key, which is similar to a standard contraction joint in cast-in-place concrete construction.

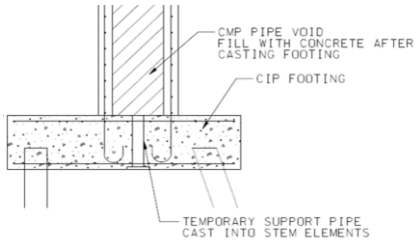
Example: Cantilever Abutment Design

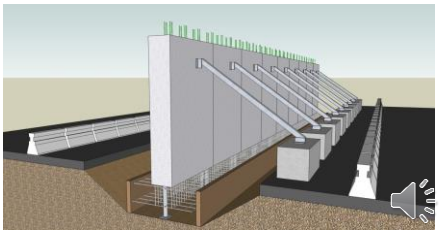
Footing (pile cap) options:

- Precast with CMP pile sockets:
  - Difficult to detail sockets
  - Difficult to manage pile tolerance
  - Reinforcement layout can be problematic
- CIP Concrete
  - Easy to detail
  - Accommodates pile tolerances
  - Almost as fast

Stem Design

- If CMP is filled, the design is unchanged





The details for connecting the wall panels to the concrete pile cap footing are shown here. The reason for the cast-in-place footing is due to the complexities of connecting a precast footing to piles. Corrugated metal pipe voids have been used in precast footings; however, pile tolerances lead to large voids which leads to unusual reinforcing layouts in the footing.

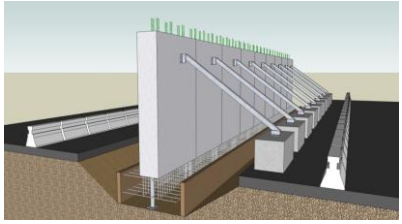
The proposed detail can accommodate normal pile tolerances with only a minor sacrifice in construction speed. The construction of this type of footing can be completed in as little as one day.

The corrugated metal pipe void in the stem will be filled after installation , therefore the design of the stem element reinforcing is unchanged from the cast-in-place concrete design.

Example: Cantilever Abutment Design

The construction sequence for this concept is as follows:

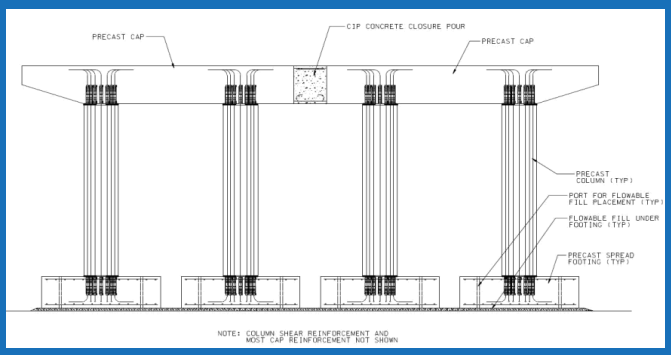
1. Install the piles
2. Set the stem elements to line and grade supporting the stem elements on the temporary integral posts. Shim under the posts to provide the proper grade.
3. Install the footing reinforcing and form the footing
4. Cast the footing and cure the concrete until it attains adequate strength to support the stems.
5. Fill the CMP void with concrete. Fill the void to the elevation of the bottom of the cap dowels.
6. Grout or concrete the vertical connections between the stem elements.

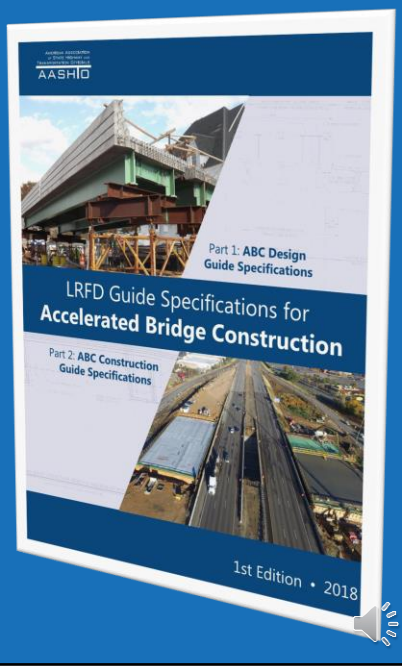


The construction sequence for the abutment is as follows:

Review the sequence shown on the slide

- Precast Pier Bents
  - Grouted Couplers Connection Example
  - Socket Connection Example





Continuing our look prefabricated substructures, we will now look at several options that are available for precast piers.

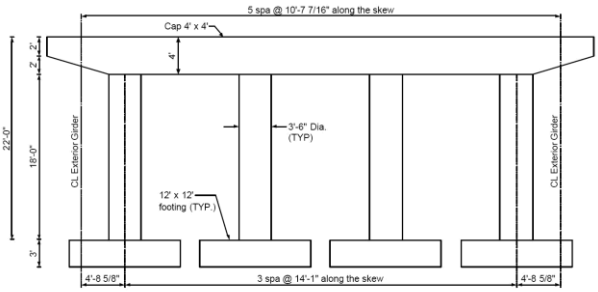
We will go through the design of several connections using different ABC technologies. These will include the use of grouted couplers for column connections, and an innovative socket connection to connect the column to a footing.

The Guide Specifications for ABC contain other options for connections. Due to limitations of time, we will not be covering all connections. We have included information to designers in the commentary, including references to the research that led to each provision. The final report for NCHRP Project 12-102 also includes more information on each provision. This document is available at the project website (search “NCHRP 12-102”). If designers have questions or uncertainty about a provision, they can locate and review the reference material to gain a better understanding of each provision.

Example: Pier Bent Design

This example:

- Use FHWA LRFD Design Example as basis
- Use emulation for the design
- Detail connections with grouted couplers
  - Place outside of the hinge zone
- Analysis and design of the bent is the same as CIP



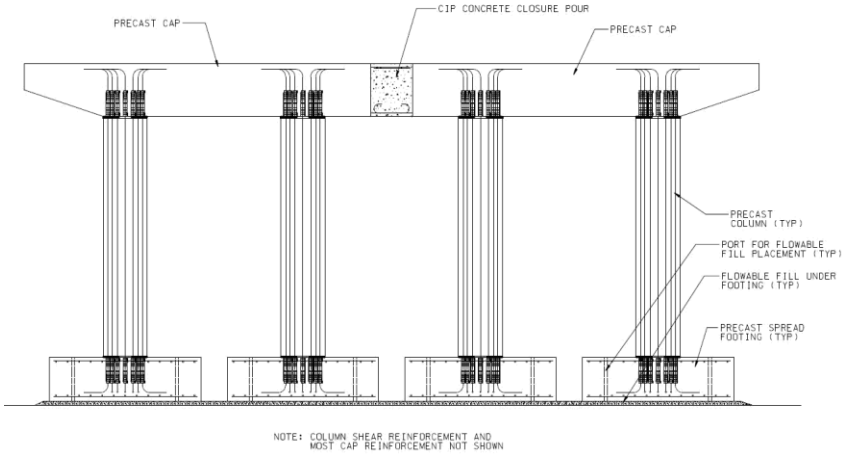
FHWA Example Pier Bent

We will now go through a design example for a pier bent with grouted couplers for column connections.

This will be another example of an emulative approach. By using emulation, the analysis of the pier is the same as with a cast-in-place concrete pier. Therefore, we will not go through the detailed calculations for calculating pier forces. For this we will use the Federal Highway LRFD design example as a basis, which is denoted by the red header at the top of the slide.

Example: Pier Bent Design

Proposed details:



The current AASHTO LRFD Bridge Design Specifications prohibit the use of mechanical connectors in the plastic hinge zone for seismic design. The specifications also require Staggering of mechanical connectors in columns. It is important to note that this is based on the current provisions for what we now call Type 1 connectors. The ACI 318 Building Code does allow Type 2 couplers in hinge zones and does not require staggering of Type 2 couplers.

Bridge research has shown that the higher strength Type 2 connectors can be placed in a single cross section and within a potential hinge zone. In displacement based seismic design, the geometry of the hinge is an important aspect. The couplers will affect the stiffness of the hinge, which would affect the displacements during seismic events. Provisions are included in the Guide Specifications for ABC to address this.

One way to mitigate this is to locate the couplers within the connected elements and not in the column. In this case, the design of the connections is identical cast-in-place concrete design. There are additional provisions in the Guide Specification for this type of detail in seismic design.

Example: Pier Bent Design

Column design

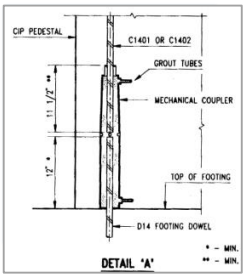
- Same as CIP

Detailing with couplers

- Larger diameter than bars
- Potential conflicts with cap reinforcing

Design

- Type 1: Meet AASHTO requirements for mechanical splices (125% Fy)
- Type 2: Strong as bar (100% Fu)



Grouted couplers can be placed within columns , however they do have a minor impact on the design of the column. Earlier today we went through the design of a beam with couplers. The design of a column with couplers is similar. The bar cage needs to be moved inward to provide adequate concrete cover over the coupler. From there, the design is the same as a cast-in-place concrete column.

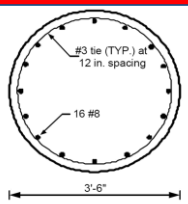
We will also look at the impact of placing couplers within the pier cap, specifically the effect on reinforcement layout.

To reiterate, the Guide Specifications for ABC contain two types of couplers. Type 1 Couplers meet the current AASHTO specification requirements for mechanical splices. Type 2 Couplers are essentially the same strength as the bar.

Example: Pier Bent Design

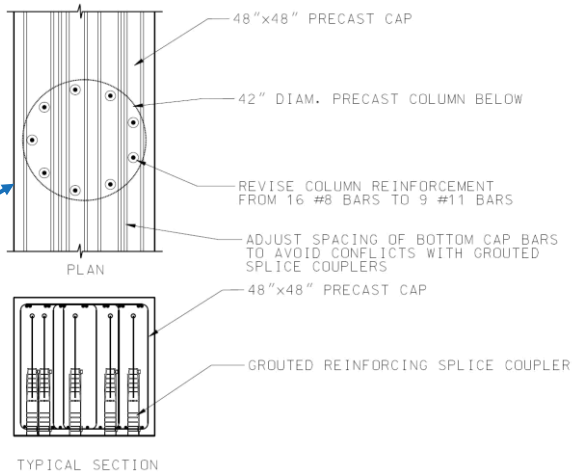
CIP column

- Same as CIP



Trial precast column with coupler

- Use fewer larger bars
- Easier to connect
- More cost efficient



In the previous design example for couplers in beams, we noted that it is often desirable to use fewer but larger bars to mitigate congestion of reinforcing within the elements. One of the most congested elements in a bridge is the pier cap. It is important to detail all the bars and couplers in a pier cap to ensure that everything will fit.

The detail to the left shows the column reinforcement designed in the Federal Highway example. The detail to the right shows a potential redesign using couplers. We are proposing to use fewer but larger bars to facilitate the placement of the bottom flexural reinforcement in the cap. The plan view of the cap shows the interaction of the couplers with the cap flexural steel. You can see that even with fewer bars modifications to the cap steel are required.

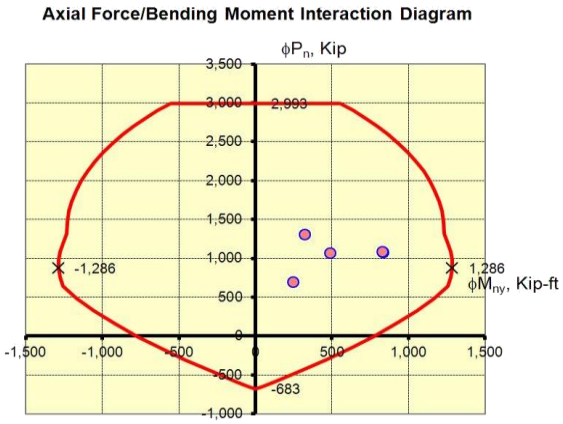
This is also cost effective in that the increase in cost of couplers from one bar size to the next is not significant. In fact, the total cost maybe less with fewer but larger couplers when you factor in the labor required to grout the couplers.

The proposed design change is to replace the 16-#8 bars with 9-#11 bars.

Example: Pier Bent Design

Column capacity check

- Use column interaction design approach
- Red lines are the interaction diagram for the revised design
- Pink dots are the column demands
- By inspection, the new column is adequate



Revised Column Interaction Diagram

The design of the column is based on the axial/moment interaction approach.

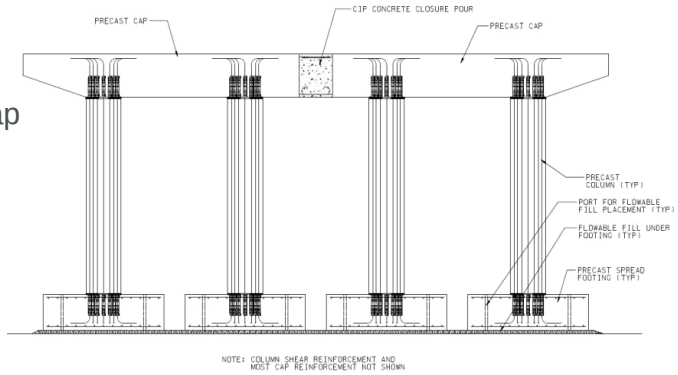
The diagram to the right is the interaction diagram for the redesigned column. the red lines indicate the column capacity limits. The pink dots represent the force demands on the columns. As you may already know, if the forces are within the confines of the capacity limits, the column is adequate.

By inspection, the new column design is adequate to resist the forces.

Example: Pier Bent Design

Precast cap

- Use 2 cap elements to facilitate construction
- Detail connection between two cap elements as a reinforced closure joint



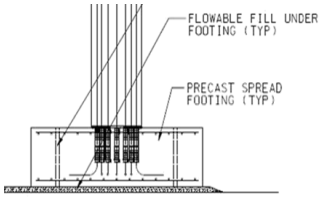
The pier cap in this example is quite long. This makes shipping and handling more difficult. It would also complicate the connection with the cap in that a large number of couplers would need to be engaged with the erection of the cap. It is common to connect a cap to two columns. Therefore, I would recommend using a two piece cap, With a closure joint connecting the two cap elements.

Another option is to redesign the pier with an open joint between these two cap elements, eliminating the need for a connection. Splitting the cap also reduces the thermal bending moments in the columns. This approach is likely to be the most cost effective design.

Example: Pier Bent Design

Precast footing

- The only change is the layout of bars in the top mat of reinforcing steel to make room for couplers
- Use flowable fill or grout to seat footing



In this design example, the maximum factored bearing stress under the footings is:

σ = 11.71 ksf
(see page 7-79 of the example problem [74]).

Expressed in pounds per square inch

σ = 11.71 ksf (1000)/144 = 81.3 psi

This demonstrates the lack of need for a high strength grout material. Several agencies are not using flowable fill to span the gap between the footing and the substrate. Most flowable fills can easily attain this strength level; therefore, they would be acceptable for this situation.

3.6.3—GROUTING OR CONCRETING UNDER PRECAST SPREAD FOOTINGS, PRECAST SLABS-ON-GRADE, AND APPROACH SLABS

The use of flowable grouts and flowable fills is recommended to provide proper seating of the precast footing or precast slab. The strength of the grout or fill shall be consistent with the anticipated loads.

C3.6.3

The bearing of a spread footing or slab on earth is a form of a connection. The interface between the spread footing or slab needs to transmit vertical and horizontal loads to the subsurface.

It is possible to place a precast concrete footing directly on top of soil; however, experience has shown that this can be a difficult and time consuming process. The use of flowable grout or fill materials can ensure that uniform bearing is achieved. The spread footing or slab can be detailed with height adjustment devices or shims to provide the required elevations. The void under the element can then be filled with a flowable material to complete the connection. The material used to fill the void should be selected to be commensurate with the anticipated loads. High-strength non-shrink grouts have been successfully used; however the strength of these grouts normally far exceeds the anticipated loads. A more cost effective solution is to use a flowable cementitious fill. The strength of these fills are more consistent with the magnitude of loading on a typical spread footing or slab.

The FHWA design example includes independent footings for each column. It also shows a shallow spread footing foundation. This facilitates the use of a precast footing. by using couplers within the footing, the designer would need to potentially rearrange the top bar's in the footing to avoid conflicts with the couplers. Otherwise, the footing design is unchanged.

The PCI Northeast Guide Details for precast substructures recommends the use of low strength flowable fill under footings to provide uniform contact with the substrate. We have seen designers specify high strength grout for this purpose. This is an effective, but expensive option. Let's investigate the use of a lower cost alternative with flowable fill. This material can typically achieve an ultimate compressive strength of between 500 and 1000 psi.

Article 3.6.3 describes the use of flowable fills under footings.

The calculation shown from the FHWA design example shows the maximum factored bearing stress under the footing, which is 11.71ksf. if we convert that to pounds per square inch we get a value of 81.3psi. For this case, and for most cases, flowable fill will be adequate for bearing capacity.

Socket Connection Example

Alternate connection for column to footing

- Precast column with CIP footing
- Round or multi-facetted columns

Benefits

- Simplified detailing
- No dowels between column and footing
- Tolerances are not an issue
- Great for pile supported footings
- Easy to design

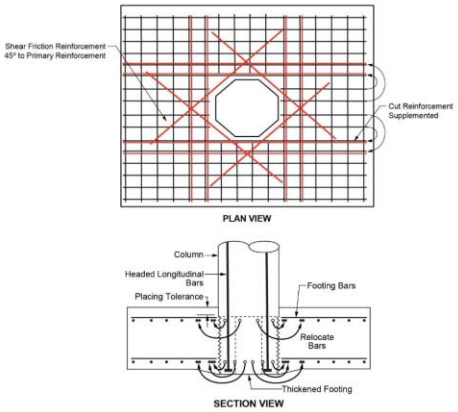


Figure C3.6.7.2-1—Placement of Footing Bars with a Socket Connection

We will now explore an alternate detail for the connection of the column to the footing. This detail is classified as a socket connection in that a portion of one element is inserted into the connected element.

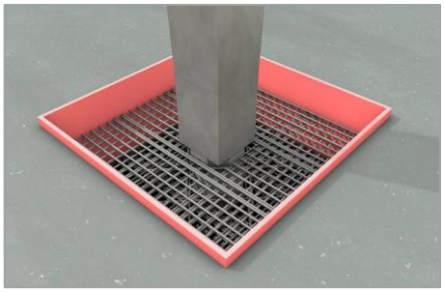
This connection is one of my favorite ABC connections for the following reasons :

- The details are clean and simple
- There is no need for any reinforcing bars to project from the column into the footing
- Fit-up and tolerances are not an issue
- This detail works very well for pile supported footings
- And, as you will see in the next few slides, the design is very simple.

Socket Connection Example

Roughened Column End cast into Footing

- Based on FHWA Highways for LIFE Research in Washington



This socket connection was developed under a research project funded by the Federal Highway Administration under the “Highways for LIFE” program.

The left graphics shows a computer rendering of the column and footing prior to casting.

The right photo shows the column being installed in an opening between the adjusted reinforcing bar cage. You will note that the portion of the column embedded in the footing is cast with a roughened surface to transfer column axial forces into the footing via interface shear.

The center photo shows a column that is plumbed and braced and the placement of the footing concrete.

Research has shown that this detail can fully develop the bending capacity of the column, even for seismic applications.

Section 3: Socket Connection

Column to footing socket connection

- Article 3.6.7.2

3.6.7.2—Precast Concrete Column in Spread Footing or Pile Cap Socket Connection

Where socket connections are used to connect precast columns to spread footings or pile caps, the following requirements shall apply.

To maintain integrity of aggregate interlock across the interface, the embedded element and associated construction loads shall be supported until the cast-in-place concrete or grout around the footing gains sufficient strength. If the column is temporarily supported directly by soil, a sacrificial slab shall be used to prevent relative settlement during construction unless it can be shown that the supporting soil has adequate resistance.

The embedment depth of the column shall not be less than 1.0 times its diameter. Column longitudinal bars shall be fully anchored within the depth of the footing or pile cap.

C3.6.7.2

The provisions in this article are based on the design specifications included in report FHWA-HIF-13-037-A (Marsh et al. 2013).

Article 3.6.7.2 of the Guide Specifications for ABC covers the design of this connection. In the next few slides, we will review the main provisions.

The first provision is that the embedment depth of the column in the footing must be equal to or greater than the diameter of the column.

Section 3: Socket Connection

Column to footing socket connection

- Article 3.6.7.2

Column bars shall be terminated with mechanical anchorages capable of developing the required bar strength. The anchorages shall be placed below the bottom mat of footing reinforcement or top mat of pile cap reinforcement.

Bent-out bars are commonly used in cast-in-place construction to support the column cage while the footing is cast, but this is not necessary with a precast column because the column can rest directly on the soil or a sacrificial slab. In addition, for a socket-type connection it is generally not practical to turn the column bars with 90-degree hooks.

Joint shear performance has been shown to be greatly improved when column longitudinal bars are anchored through ASTM A970 Class HA heads, rather than bent-out bars. This is particularly the case when mechanical anchorages are placed below the bottom mat of the footing reinforcement (Haraldsson et al. 2013). The latter may require thickening the footing around the column as shown in Figure C3.6.7.2-1.

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Another provision this article is that the column longitudinal bars shall be terminated with mechanical anchorages capable of developing the required bar strength within the footing.

The anchorages need to be located below the bottom mat of footing reinforcement or the top map of pile cap reinforcement.

Section 3: Socket Connection

Column to footing socket connection

- Article 3.6.7.2

For socket connections with an embedment depth l_e, of less than $1.5D_c$, the interface of the precast column with the footing or pile cap shall be intentionally roughened to an amplitude of not less than 0.25 inches.

Results from tests and analyses (Osanai et al. 1996) showed that for socket base connections, performance of the column base was improved by the addition of a roughened surface at the interface. With embedment depth of $1.5D_c$ or more, the column base could be regarded as a rigid connection, even without a roughened surface. Socket base connections with roughened surfaces and embedment depths of $1.0D_c$ or more reached the ultimate strength of the columns and gave results similar to those with an embedment depth of $1.5D_c$.

Footing or pile cap reinforcement shall be distributed uniformly across the width of the element, but those bars that conflict with the column shall be supplemented with the same number of bars/size on both sides of the column.

The placement of footing bars is shown in Figure C3.6.7.2-1. Supplemental bars that replace cut bars to accommodate the column may be bundled as necessary.

For columns that have embedment depth that is less than 1.5 times the column diameter, the portion of the column embedded in the footing needs to be roughened to an amplitude of not less than 0.25 inches.

This provision also requires that footing reinforcement be uniformly distributed across the width of the footing , however bars that conflict with the column maybe shifted to both sides of the column.

Section 3: Socket Connection

Column to footing socket connection

- Article 3.6.7.2.1

3.6.7.2.1—Interface Shear Transfer in Socket Connections

The socket connection shall be designed to adequately transfer shear forces across the interface planes per the interface shear requirements of *AASHTO LRFD Bridge Design Specifications*. The cohesion factor, c, the friction factor, μ, and the factors K1 and K2 shall be taken as those for normal-weight concrete placed against a clean concrete surface, except that μ shall not exceed 0.5 where embedment depth is less than 1.1 times the column diameter.

C3.6.7.2.1

Evaluation of shear capacity along the interface should consider aggregate interlock and shear friction. Shrinkage of the concrete around the embedded portion of the column increases the normal pressure across the interface. This is conservatively accounted for by specifying a cohesion factor, c, corresponding to concrete placed against a clean concrete surface. Flexural steel that provides a net compression force across the concrete-concrete interface and not used to resist the flexural demands may be used to satisfy the shear friction requirements of the connection. If axial tension force exists (as in a corbel with beam shrinkage), then provide additional steel to resist the axial tension force also.

Osanai et al. (1996) evaluated friction coefficient recommendations for use in modeling force transfer across the interface of socket connections. Based on correlation

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The final design check for this article ensures that the column axial forces can be transferred to the cast-in-place concrete footing via interface shear.

This provision sets limits on the factors used for that calculation.

Socket Connection Example

Step 1: Check embedment depth (GS 3.6.7.2)

- Column diameter in FHWA example = 3’-6”
- Footing depth = 3’-0”
- Therefore, increase footing thickness to 3’-6”

> The embedment depth of the column shall not be less than 1.0 times its diameter. Column longitudinal bars shall be fully anchored within the depth of the footing or pile cap.

5

Using the Guide Specification Article 3.6.7.2, we can design the connection for the FHWA design example.

Step one of this process involves checking the embedment depth of the column. The FHWA example has a column that is 3’-6” in diameter, and a footing that is only three feet thick.

Therefore, in order to meet this provision, we will need to increase the thickness of the footing to match the column diameter which is 3’-6”.

Socket Connection Example

Step 2: Check embedment depth (GS 3.6.7.2)

- Need to anchor column bars in footing portion
- No room for hooks
- Use headed reinforcing bars

> Column bars shall be terminated with mechanical anchorages capable of developing the required bar strength. The anchorages shall be placed below the bottom mat of footing reinforcement or top mat of pile cap reinforcement.

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Step two of the connection design involves anchoring the longitudinal column bars within the footing portion of the column.

There is not sufficient room for hooks, therefore we will use headed reinforcing bars.

Development of Headed Bars

3.6.2.3.1—Development of Headed and Mechanically Anchored Deformed Reinforcing Bars in Tension

The basic development length for headed and mechanically anchored deformed reinforcing bars in tension, l_{dth}, measured from the underside of the head to the point of development shall satisfy:

$$l_{dth} \geq \frac{0.51 f_y d_b}{\sqrt{f'c}} \qquad (3.6.2.3.1\text{-}1)$$

but not less than 6 in., where all of the following criteria are met:

- f'_c used in the calculation shall not exceed 6 ksi;
- Concrete shall be normal weight;
- Clear cover for the bar shall not be less than $2d_b$;
- Clear spacing between bars being developed shall not be less than $4d_b$
- f_y shall not exceed 60 ksi;
- Bar size shall not exceed No. 11;
- A_{brg} shall not be less than $4A_b$;

where:

f'_c = nominal compressive stress of concrete (ksi)
f_y = specified minimum yield stress of reinforcing bars (ksi)

Article 3.6.2.3.1

A_{brg} = net bearing area under the head of the bar
A_b = area of individual bar (in^2)
d_b = nominal diameter of a reinforcing bar (in.)

The basic development length, l_{dth}, shall be multiplied by the following factors:

- Epoxy coated reinforcement……....1.2
- Anchorage or development for the full yield stress of reinforcement is not required, or where reinforcement in flexural members is in excess of that required by analysis…….....$\frac{(A_s \text{ required})}{(A_s \text{ provided})}$

The Guide Specifications for ABC include provisions for development of headed reinforcing bars.

Article 3.6.3.1 includes the design requirements, which were derived from equations in the ACI 318 Code. We will use these requirements to check the development of headed bars in our example.

Socket Connection Example

Anchorage of headed bars

- f'c = 3 ksi
- db = 1.0 inches (#8 bars)
- fy = 60 ksi
- Therefore $l_{dth} \geq \frac{0.51 f_y d_b}{\sqrt{f'c}}$ = 17.7 inches
- Actual = 42 inches ✓
- Check spacing > 4db = 4"
- Actual = 7" ✓

3.6.2.3.1—Development of Headed and Mechanically Anchored Deformed Reinforcing Bars in Tension

The basic development length for headed and mechanically anchored deformed reinforcing bars in tension, l_{dth}, measured from the underside of the head to the point of development shall satisfy:

$$l_{dth} \geq \frac{0.51 f_y d_b}{\sqrt{f'c}} \quad (3.6.2.3.1\text{-}1)$$

but not less than 6 in., where all of the following criteria are met:

- f'_c used in the calculation shall not exceed 6 ksi;
- Concrete shall be normal weight;
- Clear cover for the bar shall not be less than $2d_b$;
- Clear spacing between bars being developed shall not be less than $4d_b$
- f_y shall not exceed 60 ksi;
- Bar size shall not exceed No. 11;
- A_{brg} shall not be less than $4A_b$;

where:

f'_c = nominal compressive stress of concrete (ksi)
f_y = specified minimum yield stress of reinforcing bars (ksi)

The FHWA design example is based on the following specifications :

- The concrete ultimate strength is 3 ksi
- The diameter of the bars is 1 inch
- The bars are grade 60 reinforcing

Using these values the development length is calculated at 17.7 inches.

The provided development length is 42 inches minus cover, which would equal approximately 38 inches. Therefore, the design is adequate.

The next check is on the clear spacing of the bars, which needs to be greater than four bar diameters. The actual bar spacing is approximately 7 inches, therefore this provision is met.

Socket Connection Example

Step 3: Surface treatment

- If the embedment depth is less than 1.5Dc
- Roughen interface to amplitude of 0.25”

- Our embedment = 1.0Dc
- Therefore, roughen the surface

- Distribution of footing bars
- Redistribute conflict bars adjacent to the column

For socket connections with an embedment depth l_e, of less than $1.5D_c$, the interface of the precast column with the footing or pile cap shall be intentionally roughened to an amplitude of not less than 0.25 inches.

Footing or pile cap reinforcement shall be distributed uniformly across the width of the element, but those bars that conflict with the column shall be supplemented with the same number of bars/size on both sides of the column.

The next step is a check on the required surface treatment for the embedded portion of the column in the footing.

If the column embedment is less than 1.5 times the column diameter, the surface needs to be roughened to an amplitude of 0.25 inches.

Our design embedment is 1.0 times the column diameter , therefore we need to specify a roughened surface.

This provision also specifies the distribution of footing reinforcing. Several bars will need to be cut to allow for the installation of the precast column. an equal number or greater number of bars will be added to the footing adjacent to the column.

Diagonal bars will also be added adjacent to the column as shown in the article details.

Socket Connection Example

Step 4: Interface shear

- Use AASHTO LRFD BDS Provisions
- Use factors for normal weight concrete placed against clean concrete
- μ < 0.5 if embedment depth is less than 1.1 Dc
- This example falls under this criteria

3.6.7.2.1—Interface Shear Transfer in Socket Connections

The socket connection shall be designed to adequately transfer shear forces across the interface planes per the interface shear requirements of *AASHTO LRFD Bridge Design Specifications*. The cohesion factor, c, the friction factor, μ, and the factors K1 and K2 shall be taken as those for normal-weight concrete placed against a clean concrete surface, except that μ shall not exceed 0.5 where embedment depth is less than 1.1 times the column diameter.

The last step in the process is to check the interface shear resistance of the roughened portion of the embedded column , to ensure that the interface can resist the factor column axial forces.

We will use the AASHTO LRFD Bridge Design Specifications for the interface shear check.

The Guide Specifications for ABC specify the use of factors for normal weight concrete placed against clean concrete.

The value for μ will be 0.5, since our embedment is less than 1.1 times the column diameter.

Socket Connection Example

Step 4: Interface shear

- f'_c = 3 ksi
- A_{cv} = circumference*D_c
  = (π*D)*D_c = 3.14*42"*42" = 5544in^2
- P_c = 0
- A_{vf} = 0
- μ = 0.5 (from GS Article)
- c = 0.24
- K_1 = 0.25
- K_2 = 1.5

$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$ = 1330 kips ✓

$V_{ni} \le K_1 f'_c A_{cv},$ = 4158 kips

$V_{ni} \le K_2 A_{cv}$ = 8316 kips

5.7.4.3—Interface Shear Resistance

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \quad (5.7.4.3\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \ge V_{ui} \quad (5.7.4.3\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)
V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip)
ϕ = resistance factor for shear specified in Article 5.5.4.2. For the extreme limit state event ϕ may be taken as 1.0.

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \quad (5.7.4.3\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not exceed either of the following:

$$V_{ni} \le K_1 f'_c A_{cv}, \quad (5.7.4.3\text{-}4)$$

$$V_{ni} \le K_2 A_{cv} \quad (5.7.4.3\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \quad (5.7.4.3\text{-}6)$$

where:

b_{vi} = interface width considered to be engaged in shear transfer (in.)
L_{vi} = interface length considered to be engaged in shear transfer (in.)
c = cohesion factor specified in Article 5.7.4.4 (ksi)
μ = friction factor specified in Article 5.7.4.4

P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)

f'_c = design concrete compressive strength of the weaker concrete on either side of the interface (ksi)
K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.7.4.4.
K_2 = limiting interface shear resistance specified in Article 5.7.4.4 (ksi)

For normal weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.:

c = 0.24 ksi
μ = 1.0
K_1 = 0.25
K_2 = 1.5 ksi

This slide shows the calculations used to calculate the nominal interface shear resistance.

Review the calculations *as shown*

Socket Connection Example

Step 4: Interface shear cont.

- Vni= 1330 kips
- ϕ = 0.9 (AASHTO Article 5.5.4.2)
- Vri= 0.9*1330 = 1197 kips

- FHWA Design Example
- Pu = 1374 kips N.G.

- Increase footing thickness to 50”
- Vri= 1197 kips * 50/42 = 1425 kips ✓

- No check for flexure is required if these provisions are met
- The footing can develop the flexural capacity of the column
- Footing reinforcing can be re-designed due to increased thickness

5.7.4.3—Interface Shear Resistance

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \quad (5.7.4.3\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \quad (5.7.4.3\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)
V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip)
ϕ = resistance factor for shear specified in Article 5.5.4.2. For the extreme limit state event ϕ may be taken as 1.0.

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Now we can calculate the factored interface shear resistance by multiplying the nominal interface shear resistance by the appropriate resistance factor of 0.9.

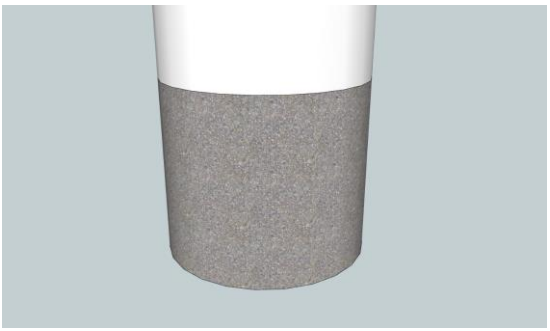
The factored interface shear resistance with found to be 1197 kips.

The ultimate axial load in the column taken from the Federal Highway design example is 1347 kips. since the ultimate load is greater than the resistance, the connection needs to be revised.

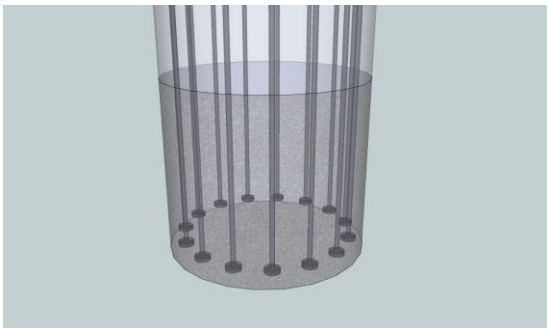
The resistance can be easily adjusted by increasing the thickness of the footing from 42 to 50 inches. It would be appropriate to redesign the reinforcing in the footing based on this revised dimension.

It is important to note that flexural resistance need not be calculated with this connection. This is based on the fact that the research that led to this provision was based on developing the full plastic bending moment in the column. Again, this connection can be used for seismic applications as well as non-seismic applications.

Socket Connection Example



Exposed aggregate column base



Internal main column bars
Note: Confinement bars are not shown

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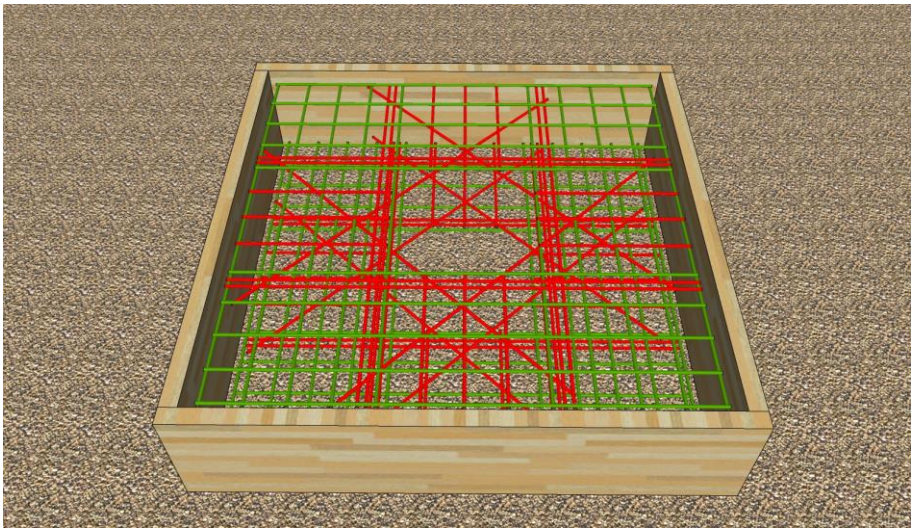
The next few slides show the details of this connection.

The sketch on the left shows the exposed aggregate base of the column.

The sketch on the right shows the headed reinforcing bars in the column. Note that the spiral reinforcement is not shown for clarity.

Socket Connection Example

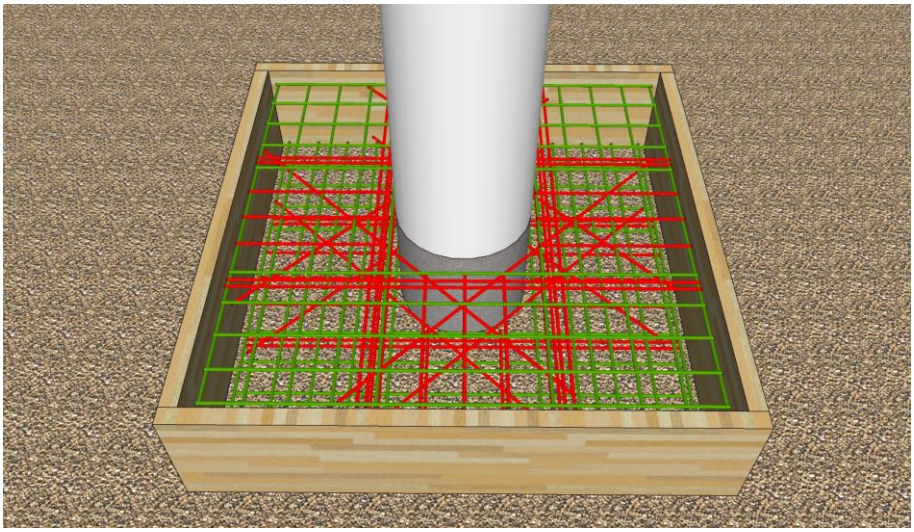


This sketch shows the footing reinforcement in the forms prior to column installation.

The green bars show the original footing reinforcement. The red bars indicate reinforcement that has been cut or added to the footing. 3 bars were cut in each direction each mat. To make up for this, 4 bars were added, 2 on each side of the column.

Diagonal interface shear bars are added as per the provisions of the Guide Specification.

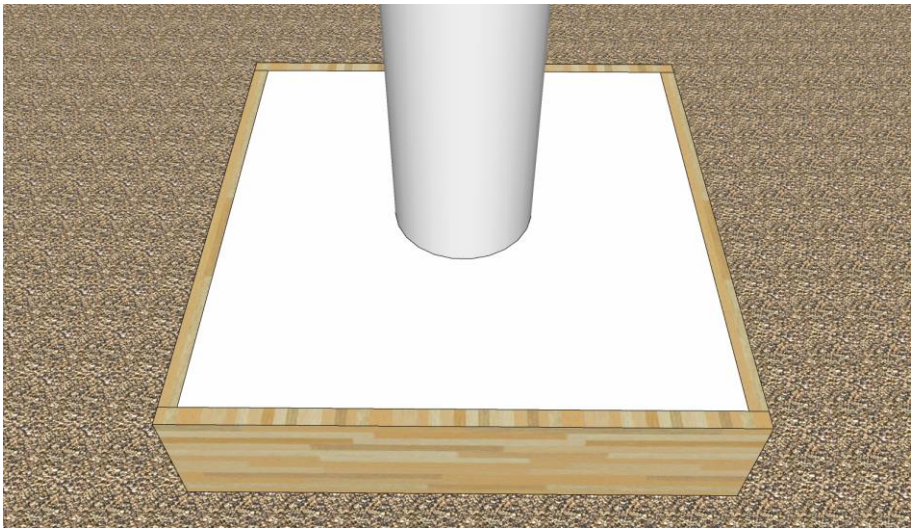
Socket Connection Example



This sketch shows the column set in place. Typically a small leveling pad of concrete would be used under the column provide a solid temporary base.

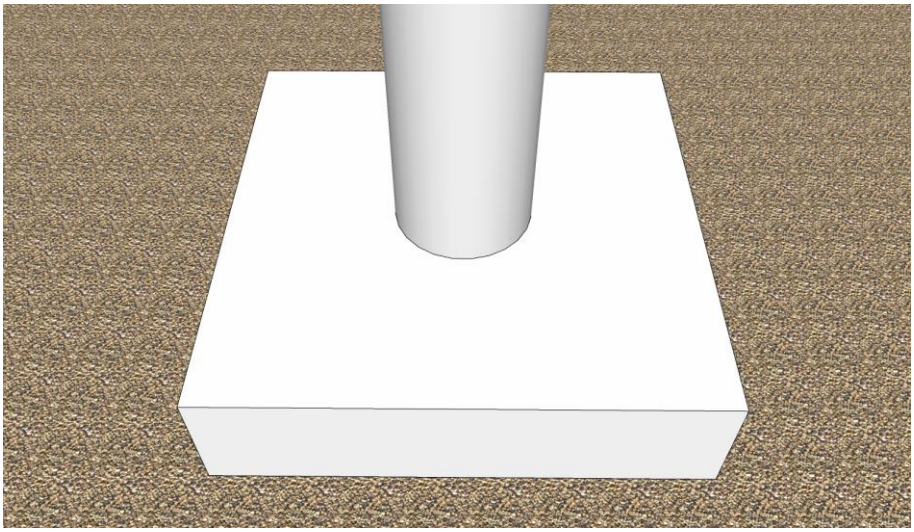
The column would be braced along its length to ensure that it is plumb and in the correct position.

Socket Connection Example



This sketch shows the footing after placement of the concrete.

Socket Connection Example



This sketch shows the completed footing.

End of Module 2

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

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