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CHAPTER 15

* Indicates 11 x 17 sheet; all others are 8½ x 11.

TABLE OF CONTENTS – PIER DETAILS

CHAPTER 15

FILE NO. **TITLE** **DATE**

SUPERSTRUCTURE FORCES

DESIGN / DETAILING FOR COLLISION FORCE

PIER PROTECTION SYSTEM

* Indicates 11 x 17 sheet; all others are 8½ x 11.

INTRODUCTION

Piers transmit loads from the superstructure to the foundation.

The intent of this chapter is to establish the practices and specific requirements of the Structure and Bridge Division for the design and detailing of piers and pile bents.

It is expected that users of this chapter will adhere to the practices and requirements stated herein. A design waiver will be required for areas that are indicated as "minimum" standards or where the term "shall" is indicated. The designer shall be responsible for investigation, analysis and calculations necessary to secure a waiver from the State Structure and Bridge Engineer. The designer must indicate in the waiver why the standard cannot be met.

References to AASHTO LRFD specifications refer to the current AASHTO *LRFD Bridge Design Specifications*, current Interims and VDOT Modifications (current IIM-S&B-80). References to AASHTO standard bridge specifications refer to the AASHTO *Standard Specifications for Highway Bridges*, 16^{th} Edition, 1996, including the 1997 and 1998 Interims and VDOT Modifications.

Several major changes to past practices are as follows:

1. Temperature and shrinkage steel required in footing.

2. Rounded cap ends are only to be used when any portion of a cap is below the high water or mean high tide elevation.

3. Added collision clearance criteria for double overpasses.

NOTE:
Due to various restrictions on placing files in this manual onto the Internet, portions of the drawings shown do not necessarily reflect the correct line weights, line types, fonts, arrowheads, etc. Wherever discrepancies occur, the written text shall take precedence over any of the drawn views.

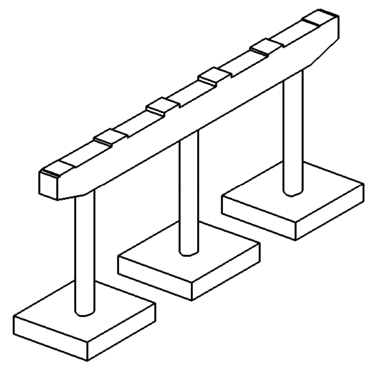
GENERAL INFORMATION:

Specific types of piers and details are more cost effective or necessary due to aspects of the particular bridge grade and location. In general, it is beneficial to keep cap size, column size and pile type and size the same for all piers/bents on a project and/or corridor to enable the reuse of forms and to avoid ordering small quantities. On larger projects, additional column sizes or pile types may be warranted where heights, depths or design loads vary substantially across the spans.

Drainage: See File No. 22.04-2 for criteria on downspout / collector pipe placement.

Multi-Column Piers:

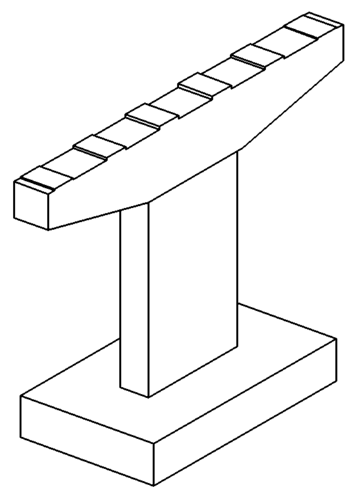
Multi-column piers are typically used where column heights are below 30 feet. Column spacing between 15 and 20 feet is generally cost effective. Cap ends shall not be rounded, but should be tapered for aesthetic purposes. Concrete Class A3 (f'_c = 3,000 psi) should be used. Class A4 (f'_c = 4,000 psi) may be used where warranted.



On wide structures with more than five columns and/or cap lengths greater than 80 feet, designers should consider whether to split a multi-column pier into two piers especially where columns are short and contraction/expansion of the pier cap results in large internal forces. For piers with more than six columns and/or cap lengths greater than 100 feet, two piers are required.

Hammerhead Piers:

Hammerhead piers are typically used where column lengths on multi-column piers will require larger column sizes due to slenderness. Hammerhead piers are also an option where stream flow could result in debris build-up between columns of a multi-column pier.



Where stream flow is present, hammerhead piers shall be oriented parallel to the direction of flow. Small skews to the direction of flow are acceptable (0 to 5 degrees) where superstructure skew can be eliminated completely or reduced for specific design purposes. Specific design purposes include reducing skew to 10 degrees so V-load method can be used to determine cross frame forces on steel superstructure and reducing skew to 20 degrees so finite element analysis is not required. Small skews parallel to the direction of flow require concurrence of the hydraulic engineer.

Wall/Solid Piers:

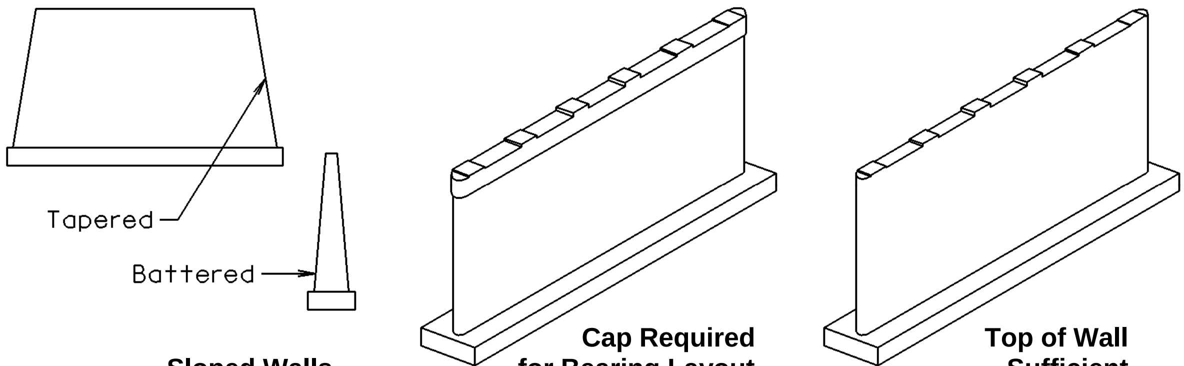
Wall piers are typically used where multi-column piers may be used, but stream flow will result in debris build-up between columns or where efficient design for collision force is required.

Wall/Solid Piers continued on next sheet

GENERAL INFORMATION (continued):

Wall/Solid Piers (continued):

Concrete Class A3 (f'_c = 3,000 psi) shall be used. Sloped (tapered/battered) walls shall not be used. Non-reinforced walls are prohibited. Wall piers shall be oriented parallel to the direction of flow. Caps are required where the width of wall is not sufficient for bearing layout.

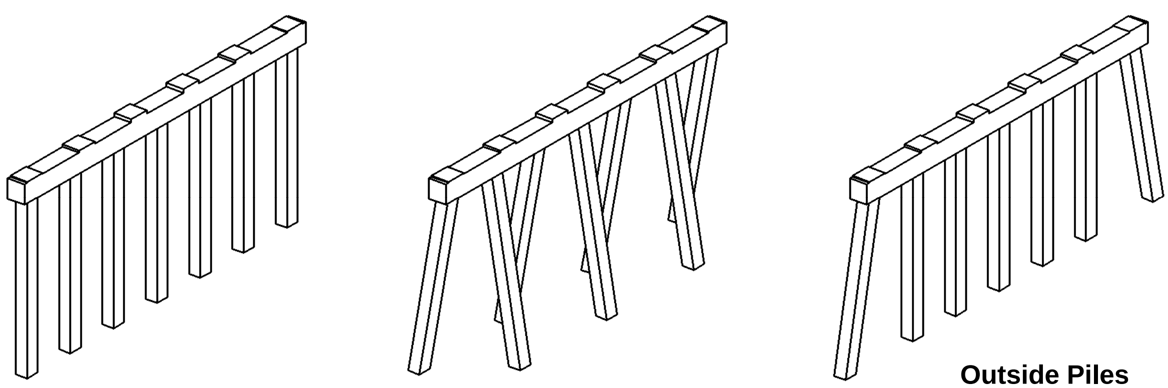


Sloped Walls | **Cap Required for Bearing Layout** | **Top of Wall Sufficient**

Pile Bents:

Pile bents are typically used over wetlands and/or bodies of water where driven square or cylinder prestressed concrete piles can reduce the environmental impact and pile or spread footings may not be feasible. Plumb piles are preferred. The outside pile on both sides of the bent may be battered to improve resistance to movement due to transverse forces only where future widening is not a concern. Likewise, piles may be battered longitudinally at a fixed bent or bents to better fix the thermal center of a unit and handle longitudinal forces. Batter on piles should not exceed 1 : 6. Driving difficulties increase with the length of the pile. Before battering piles, designers should consider whether fixing more bents can adequately handle design loads and movement.

Pile bents are often used in scourable areas. Designers should investigate both the existing ground and scoured condition as the assumed point of fixity for the piles can vary substantially. Additionally, pile driveability must be evaluated. The designers must determine if a pile can be driven deep enough (without overstressing the pile) so that it is stable after the scour occurs. Shorter design lengths will attract more load while longer lengths will increase slenderness. Assuming the number of piles equals the number of beam/girder lines is a good starting point for preliminary design.



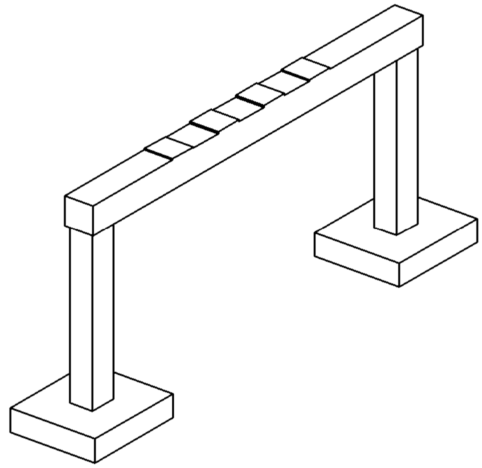
Piles Plumb | **Piles Battered Longitudinally** | **Outside Piles Battered Transversely**

GENERAL INFORMATION (continued):

Straddle Bents:

Straddle bents are typically used where column/footing placement would interfere with the road below (e.g. ramp flyovers).

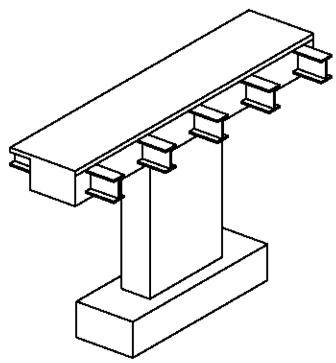
Straddle bents are non-redundant structures. In addition, steel straddle bents are considered to be fracture critical.

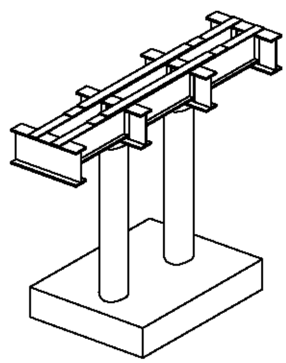


Integral Caps:

Integral caps are used where finished grade limitations on the structure provide insufficient vertical clearance above the underpass. Steel integral caps are considered fracture critical including a two-stringer transverse cap system where each stringer is designed to carry all forces independently.

Note that the deck was cut at the near face of integral cap for clarity for concrete example. Deck not shown for two-stringer transverse cap system example. Parapet not shown.



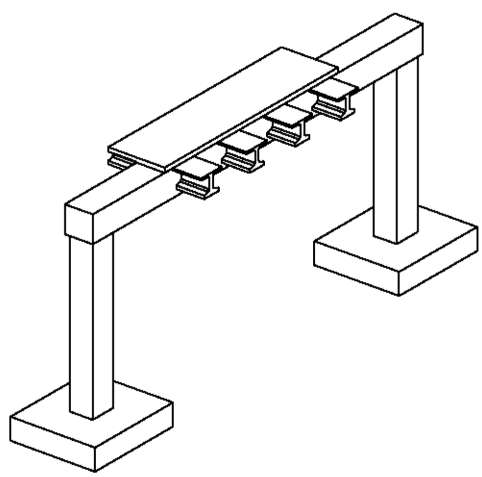


Integral Straddle Bents:

Integral straddle bents are used where column/footing placement would interfere with the road below (e.g. ramp flyovers) and finished grade limitations on the structure provide insufficient vertical clearance above the underpass.

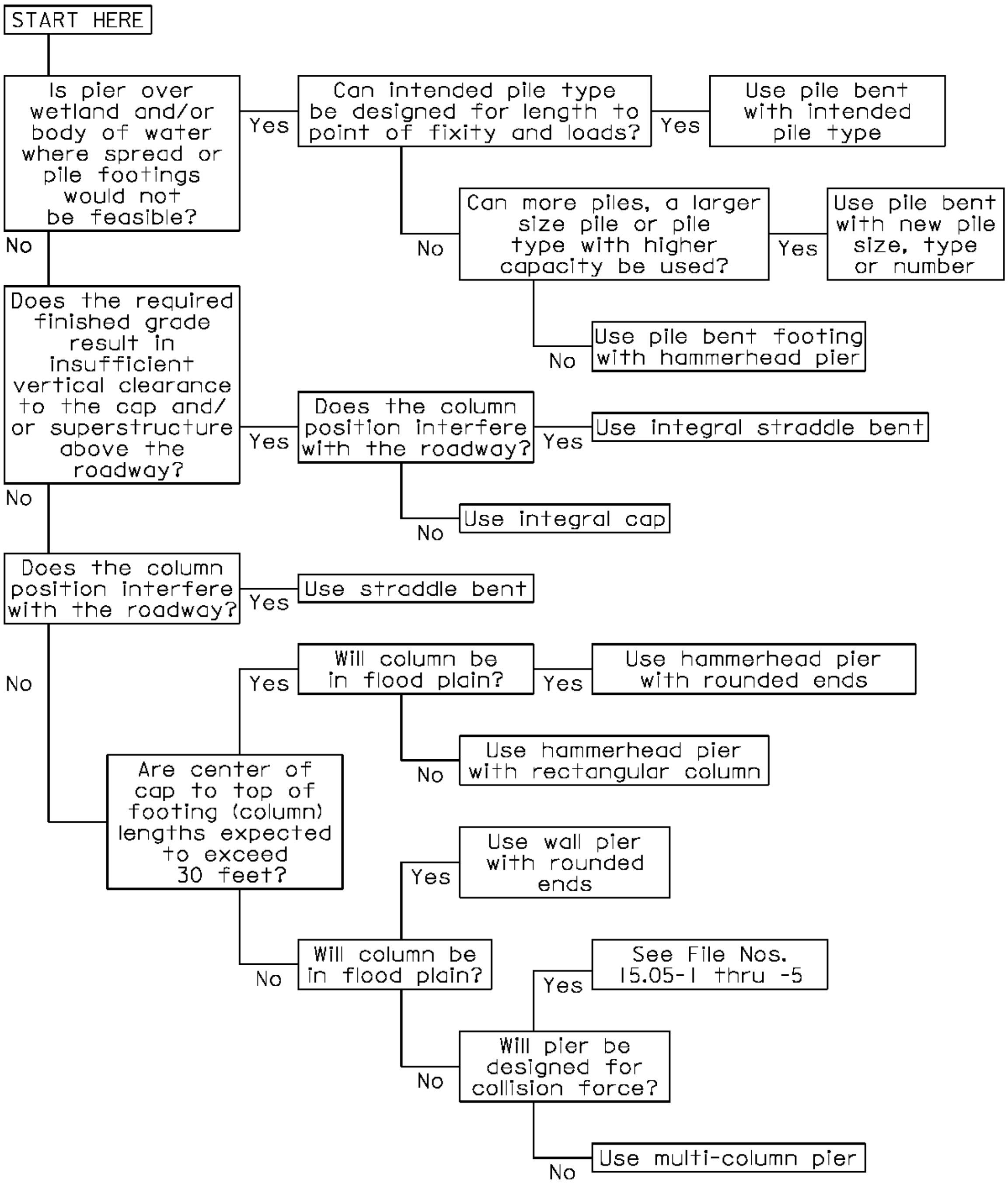
Integral straddle bents are non-redundant structures. In addition, steel integral straddle bents are considered to be fracture critical.

Note that the deck was cut at the near face of integral straddle cap for clarity. Parapet not shown.



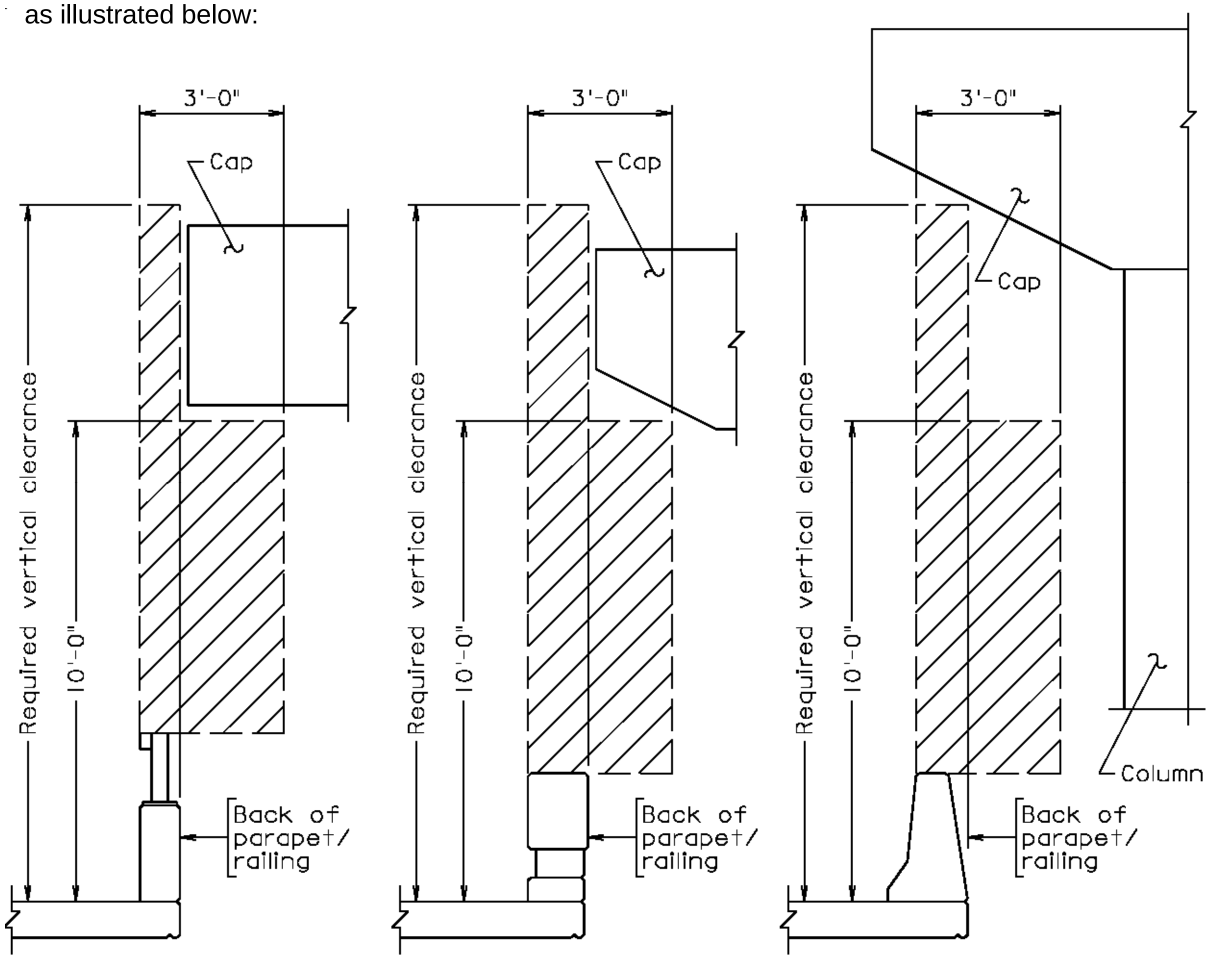
TYPE SELECTION:

This flowchart is provided to aid pier type selection and is intended to provide general guidelines of when to use or not use a particular type.



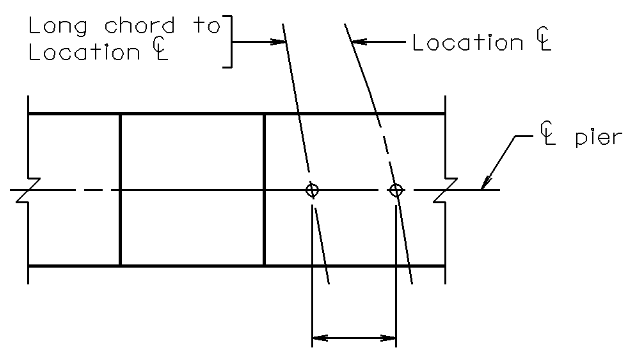
DOUBLE OVERPASS CLEARANCE:

No portion of a cap or column on a double overpass shall be within 3 feet clear of the top front face of parapet/rail extended 10 feet above the top of deck or protrude beyond the back of parapet/rail for the remaining distance to the required vertical clearance for the road classifiction as illustrated below:

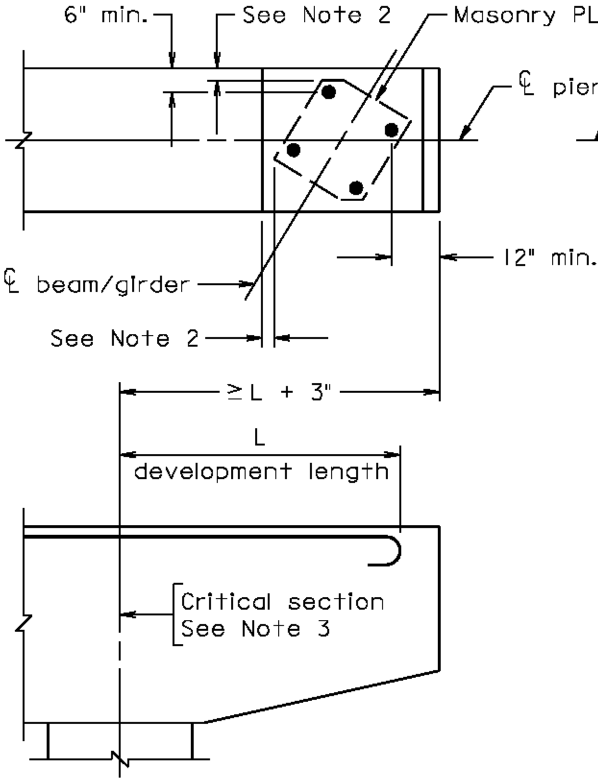


BRIDGE LAYOUT ON LONG CHORD:

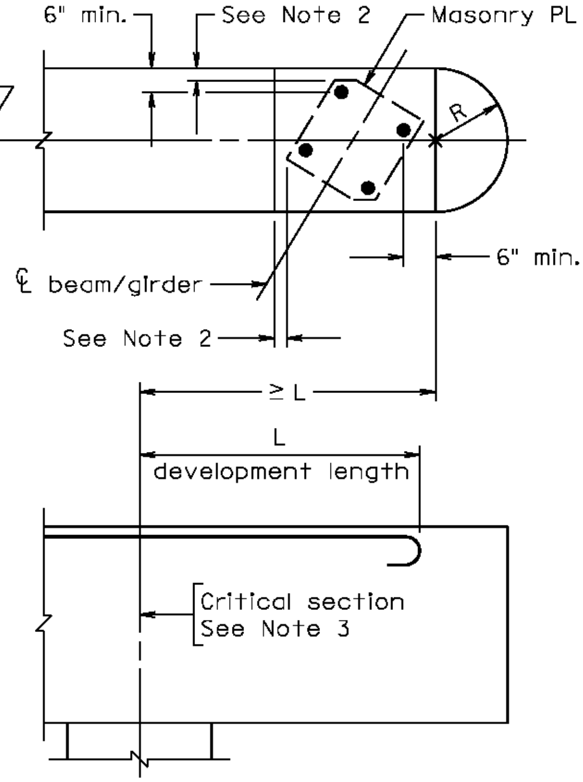
Use details only if pier is laid out on the long chord. Details shown here must agree with BRIDGE LAYOUT and SUBSTRUCTURE LAYOUT shown on the plans.



PART PLAN OF CAP
BRIDGE LAYOUT ON LONG CHORD



SQUARED CAP END



ROUNDED CAP END
Only when within hydraulic opening

CRITERIA FOR PIER CAP LENGTH

Notes:

1. Set location of the center of the rounded cap end as follows:
   - at least 6” beyond center of the extreme anchor bolt.
   - at least a length equal to the development length (L) for the main bars beyond the critical section.

   Use whichever gives the greater cap length.

2. The width of the pier seat shall be such that the edge of seat shall be at least 3” beyond the extreme point of the masonry plate, where applicable, and 6” beyond the anchor bolt(s). Where masonry plate criteria controls cap width or length, the masonry plate may be clipped to provide the necessary clearance.

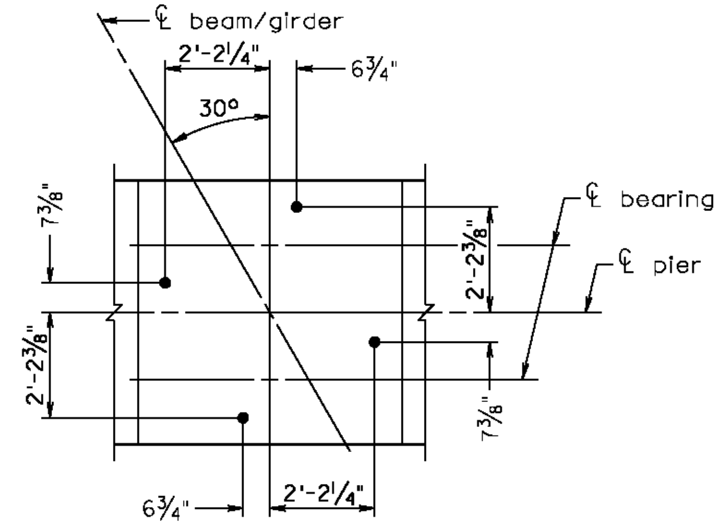
3. The critical section for multi-column cap bending moment (cantilever) with concentrated load(s) outside the centerline of the extreme column is the centerline of that column. For critical section for shear in cap, see AASHTO LRFD specifications, Articles 5.8.1 and 5.6.3.

4. For basic development length and modification factors, see File Nos. 07.101-2 and -3.

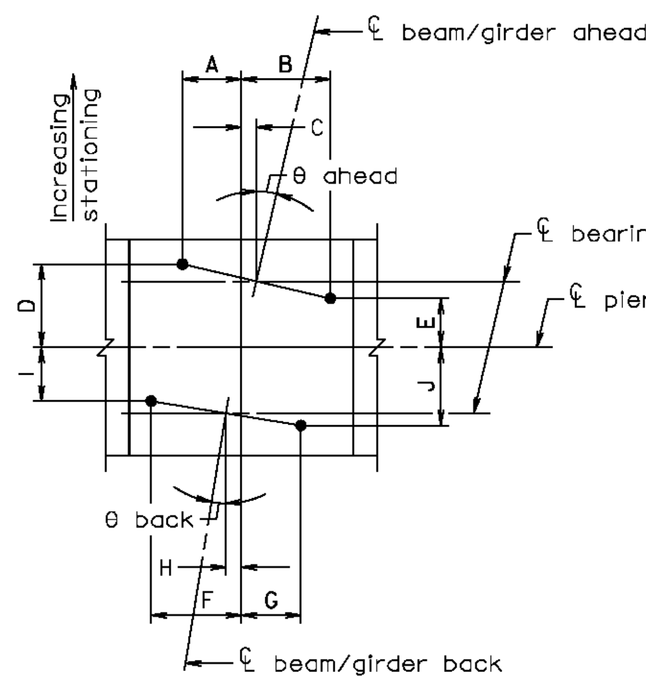
5. Top reinforcement shall be of sufficient length for supporting the stirrups.

SAMPLE ANCHOR BOLT LAYOUTS:

TYPICAL ANCHOR BOLT LAYOUT shall include vertical and horizontal offset dimensions for each anchor bolt from one reference point and angles. Variable tables should accompany anchor bolt layouts where necessary. Cap width, centerline bearing to centerline pier, seat width and centerline of seat to edge of seat dimensions are normally shown in the cap PLAN view.

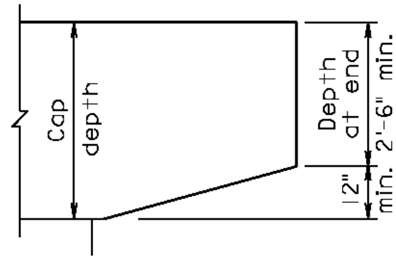


TYPICAL ANCHOR BOLT LAYOUT
(For skewed structures)



TYPICAL ANCHOR BOLT LAYOUT
(For curved structures)

BOTTOM CAP SLOPE:



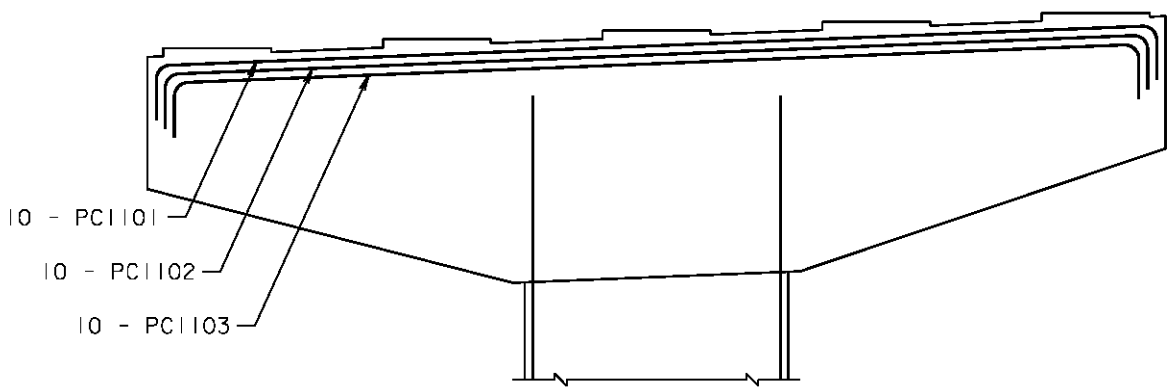
Bottom slopes for pier cap cantilevers should generally be limited to a range between 3 : 1 and 6 : 1 with the ratio of (cap depth) to (depth at end) generally limited to a range between 2 : 1 and 1.3 : 1. Deviations from these limits should be made only in cases of unusual pier geometry or to achieve architectural effects. Slopes shall not be used where the end of cap is less than 2'-6" deep and/or slope depth is less than 12". Slopes shall not be used on caps with rounded ends.

CAP REINFORCEMENT:

Top Bars:

All top main cap bars shall be hooked at the ends of the cap. Use 180 degree hooks with a single layer. Use 90 degree bends with two or more layers of top main bars. Laps should be avoided. If required, laps should be located near area(s) of maximum positive moment.

With two or more layers of top reinforcement, additional bar series are required (each successively shorter) to provide the required clear distance between bends. Cap dimensions shall be reevaluated if more than five layers are required.



Bottom Bars:

Multi-column piers: Lowest bottom main cap bars in corners of stirrups shall be extended into cantilever. Others in lowest row and bars in other bottom rows may be terminated at interior end of cantilever (i.e column side) except where additional bars require extension to meet shrinkage and temperature requirements. Laps should be avoided. If required, laps should be located near area(s) of maximum negative moment. See File No. 15.03-1 for example.

Hammerhead piers: Laps should be avoided. At a minimum, bottom main cap bars shall be placed in the corners of stirrups, consist of #6 bars or larger and meet shrinkage and temperature requirements.

CAP REINFORCEMENT (Continued):

Shrinkage and Temperature Bars:

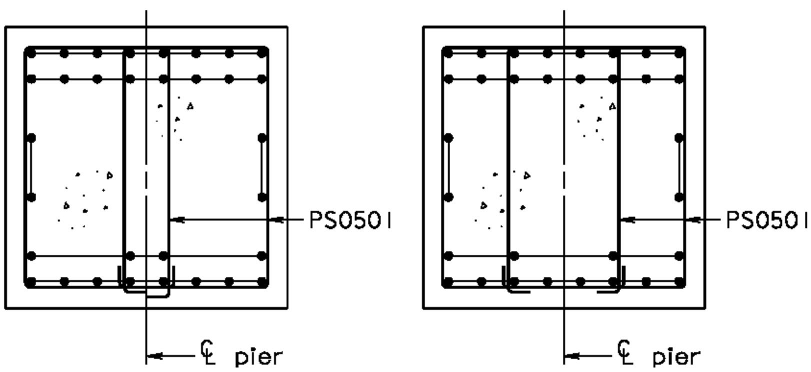
For area and spacing requirements for shrinkage and temperature reinforcement, see AASHTO LRFD specifications, Article 5.10.8.

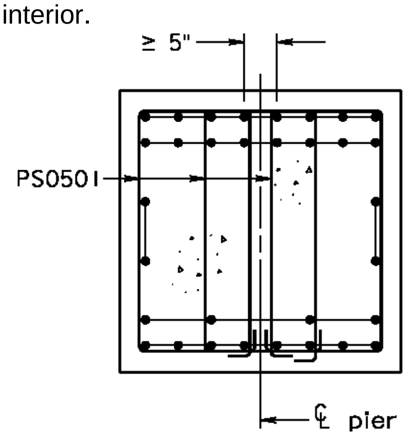
Stirrups:

Use #4 stirrups unless the spacing required is less than 4” (in which case #5 stirrups should be used). When two #5 stirrups do not satisfy the steel requried, three stirrups in one plane may be used. When the stirrup area becomes excessive, the cap depth should be reevaluated.

Vertical legs of stirrups should be distributed across the cap width (Example 1). Two vertical legs may be placed between two adjacent main bars where the distance between main bars is greater than or equal to 5” (Example 2). Two vertical legs shall not be about the same main bar (Example 3).

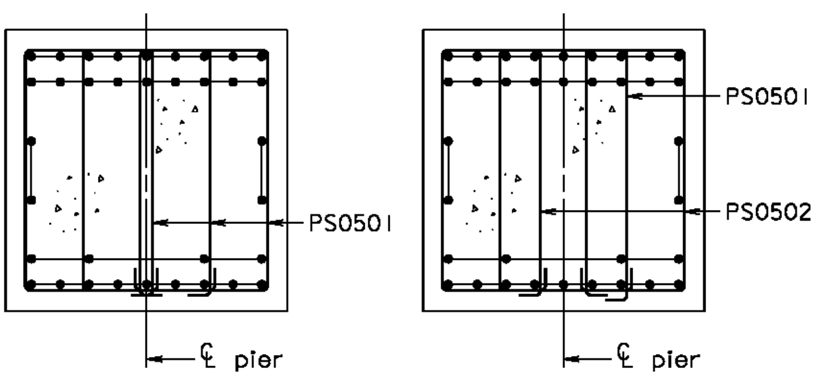
Where designers are concerned with high torsion and splitting forces in pile bent caps (e.g. bents with staggered battered piles), stirrups may be arranged as shown in Example 4 where one stirrup encloses all the main bars and one or more stirrups provided on the interior.

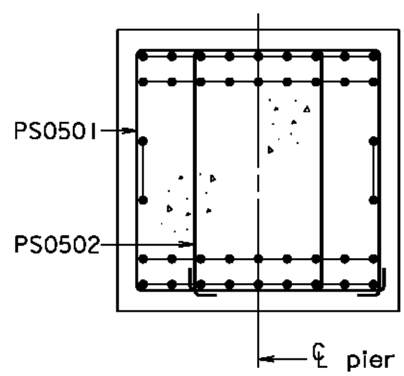




Prohibited **Acceptable** **Pile Bent**

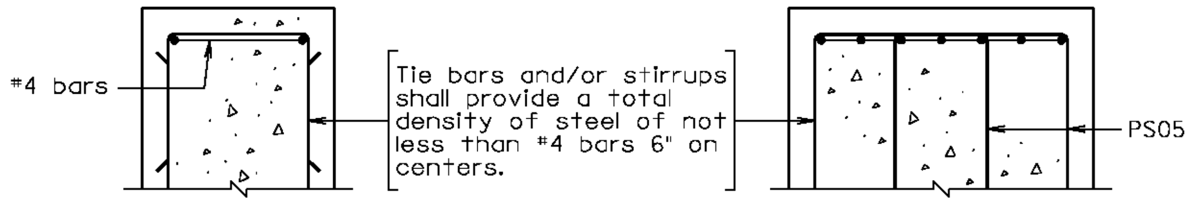
EXAMPLE 1 **EXAMPLE 2**





Prohibited **Acceptable** **Pile Bent**

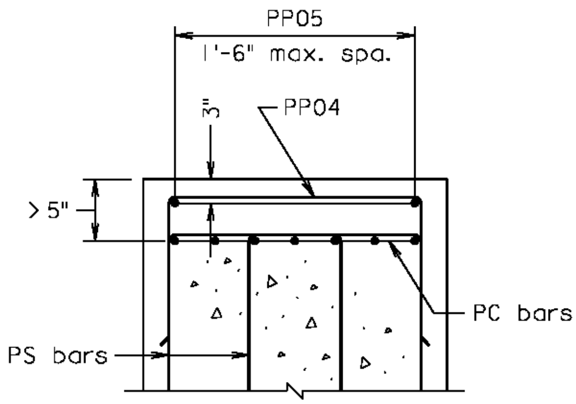
EXAMPLE 3 **EXAMPLE 4**

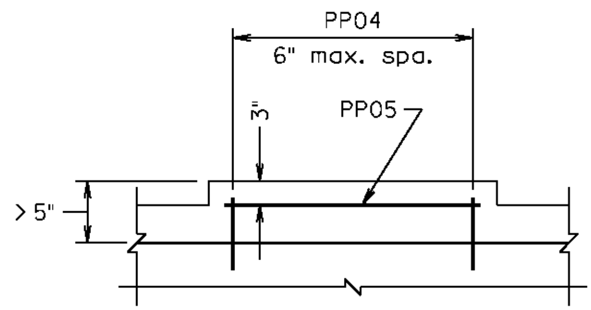


PART SECTION THRU WALL / SOLID PIER WITHOUT CAP

PART SECTION THRU PIER CAP

MAIN REINFORCEMENT ≤ 5” TO TOP OF SEAT

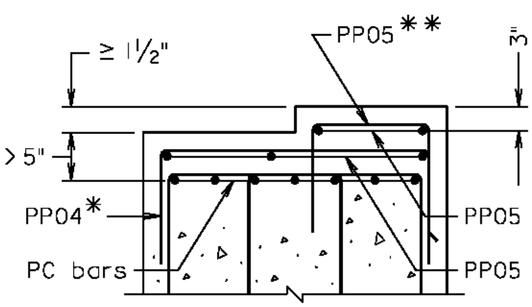


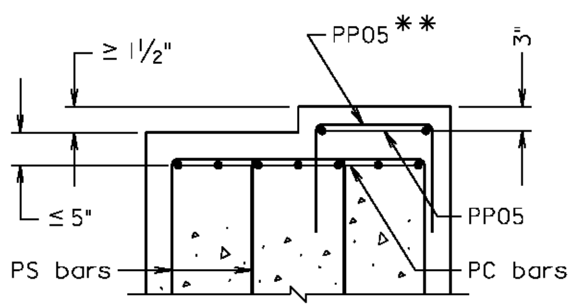


PART ELEVATION

PART SECTION

MAIN REINFORCEMENT > 5” TO TOP OF SEAT





PART SECTION

PART SECTION

* Minimum #4 bars @ 6" on centers as shown in PART ELEVATION for Main Reinforcement > 5" to Top of Seat above.

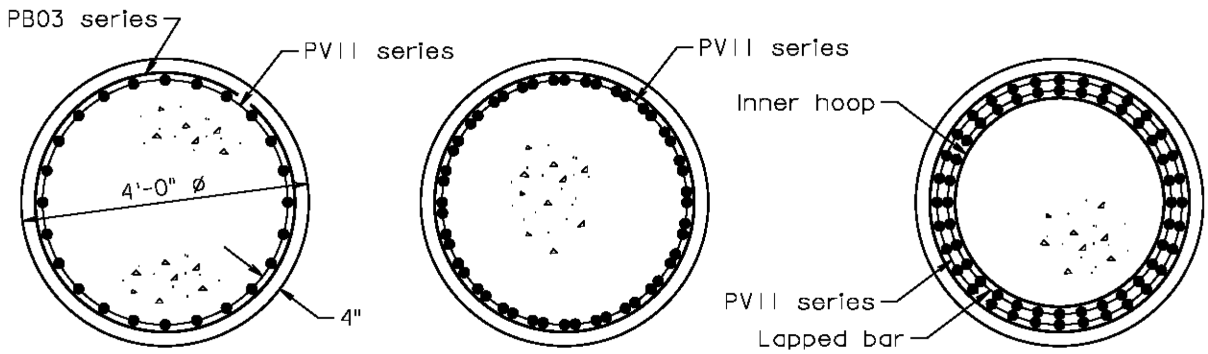
** Minimum #5 bars @ 6" on centers where difference between top of seats is greater than or equal to 1½" similar to PART ELEVATION shown above for Main Reinforcement > 5" to Top of Seat.

SPLIT PADS

PIER DETAILS **PIER COMPONENTS AND MISCELLANEOUS DETAILS** **SEAT REINFORCEMENT**	VOL. V - PART 2 DATE: 17Dec2010 SHEET 6 of 15 FILE NO. 15.02-6

Columns:

Vertical reinforcement in columns (PV series) shall equal at least 1% of the gross concrete area of the column. Maximum reinforcement shall be limited to 8% of the gross concrete area. The minimum spacing between the vertical reinforcement from AASHTO LRFD specifications, Article 5.10.3.1, shall be provided with bars spliced side by side. Radial splicing to provide the required spacing for additional bars shall only be permitted where tied to inner hoops to maintain position.



Side by Side **Radial Splicing**

For spiral and tie requirements, see AASHTO LRFD specifications, Article 5.7.4.6. Pitch shall be detailed using Article 5.10.6.3 (ties) unless a spirally reinforced column, Article 5.10.6.2, is warranted. See also current IIM-S&B-80.

Multi-Column Piers:

The minimum column diameter shall be 3'-0". Where columns larger than 3'-0" are required, they shall be selected in 6" increments from 3'-0" to 5'-0". For diameters greater than 5'-0", 12" increments shall be used. The use of A4 (f'_c =4,000 psi) concrete should be considered when its use will result in a reduction in column diameter.

The minimum bar size for vertical reinforcement is #9. Use ten (10) #9 bars for 3'-0" diameter column unless strength requirements indicate a larger area is needed.

Round columns are to be designed as tied columns, with continuous spiral ties or welded wire fabric unless special circumstances warrant the use of spirally reinforced columns.

Hammerhead Piers:

Main vertical reinforcement shall be detailed such that maximum concrete lifts will not exceed 30 feet.

The use of hollow column sections requires additional details for inspection including elevation markings on the interior, access ladders, lighting and power supply.

Wall/Solid Piers:

The minimum wall thickness shall be 2'-6". Bar size of vertical reinforcement extending from the footing into the wall shall not be less than #5 bars @ 12". Horizontal reinforcement in the wall shall not be less than #4 bars @ 12".

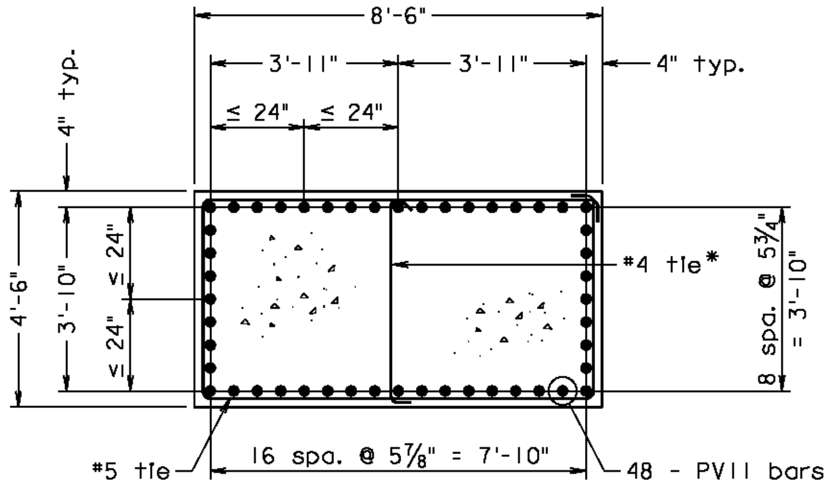
COLUMN TIES:

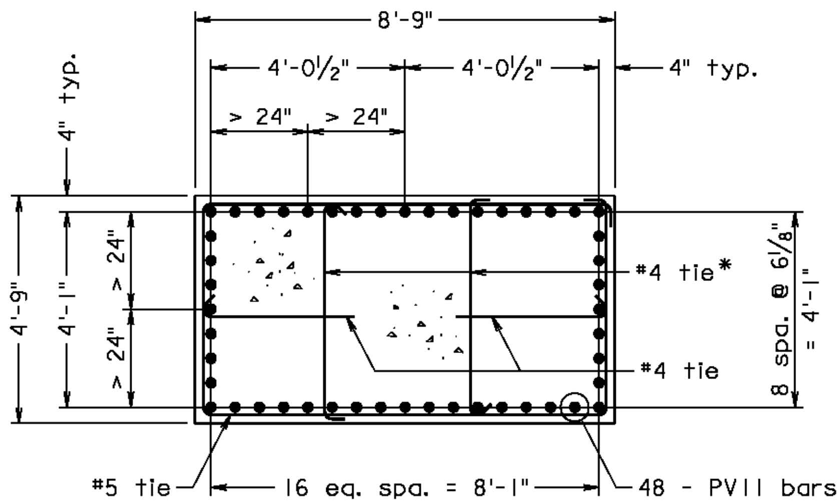
For spiral and tie requirements, see Article 5.7.4.6 of the AASHTO LRFD specifications. The column sections to the right depict additional tie arrangements adhering to the requirement of Article 5.10.6.3 that no vertical bar or bundle shall be more than 24” measured along the tie from a restrained bar or bundle.

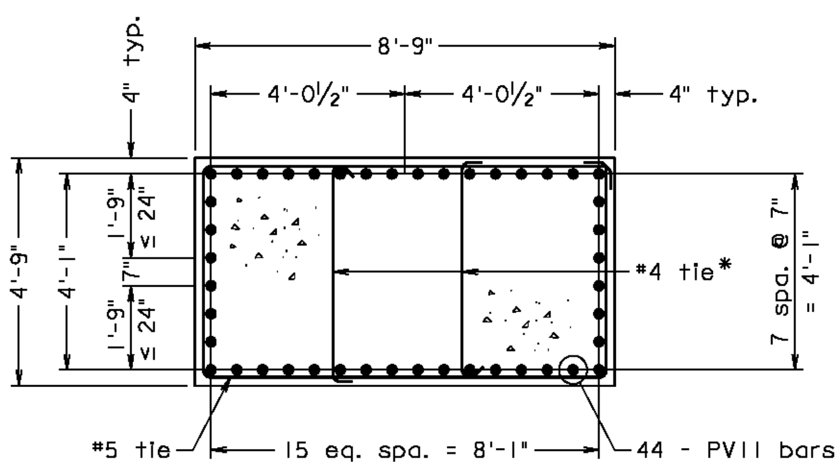
In the first example (top), the dimensioned bars are less than 24” from a restrained bar and do not require an additional tie.

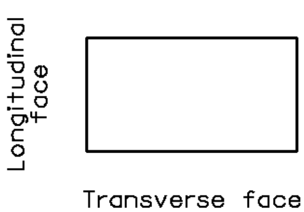
Comparing the second example (middle) to the first, the column dimensions were increased and the dimensioned bars now exceed 24” requiring an additional tie. Instead of adding two additional ties along the transverse face, one was added and the original tie position was moved so that all bars are less than 24” from a restrained bar.

Comparing the third example (bottom) to the second, the column dimensions remain the same, but the number of vertical bars is one less in each longitudinal face. The dimensioned bars are less than 24” from a restrained bar and do not required an additional tie.





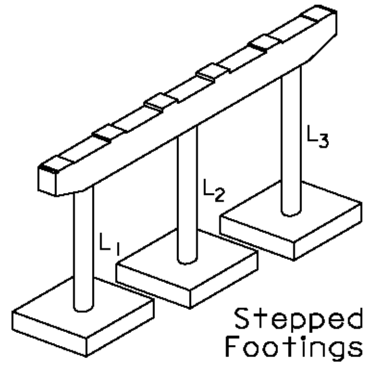




*Alternate position of hooks and bends at each level

FOOTING TYPES AND DETAILS:

Minimum footing depth shall be 3'-0" except for footings with square prestressed concrete piles where the minimum footing depth shall be increased as indicated in File No. 15.02-10.



Footing dimensions for each column (including pile layout where applicable) at a multi-column pier should be identical. With stepped footings, the shorter column length is stiffer and draws more load. The longer column carries less load, but is more slender. The required footing sizes should have similar dimensions and one footing size used.

Footing dimensions should be kept the same between all similar types of piers except on larger projects where design loads, geometry or geotechnical requirements vary substantially across the spans or where specific need exists (e.g. minimum footing size is necessary to provide required clearance for sheet piling during construction). In these situations, footing sizes should be grouped.

Tops of footings shall be a minimum of 12" below existing ground or finished grade. Tops of footings for overpasses shall be a minimum of 12" below the invert of adjacent ditch or top of fill slope. For stream crossings, footing shall be located at a depth to provide for safety against scour. In certain cases, such as when footings might interfere with proposed or future construction, the following note shall be used:

"The footing elevations shall not be varied from that shown on plans so that (give reason)."

Spread footing:

A spread FOOTING PLAN is shown on File No. 15.03-2 for a hammerhead pier. A TYPICAL FOOTING PLAN for a multi-column pier is similar.

To allow for field adjustment due to actual subsurface conditions, piers utilizing spread footings shall be designed for an additional two feet of column length (i.e. two foot drop in bottom of footing elevation). Reinforcing steel shall be detailed accordingly such that, at the lower footing elevation, the minimum lap for the column main vertical steel is maintained and the spiral pitch or tie spacing can be adjusted and satisfy design requirements.

Pile footing:

For a hammerhead pier or multi-column pier where piles are used in the footing, rest the bottom reinforcement mat on top of the piles. This allows the entire mat of reinforcing steel to be tied off and lowered onto the pile group. See File No. 15.03-1 for an example of a TYPICAL FOOTING PLAN for a multi-column pier using steel piles. A FOOTING PLAN for a hammerhead pier is similar.

For wall piers with piles, the footing reinforcement is likely to be tied in place as lowering a long narrow flexible mat on top of the piles may not be feasible. Reinforcement should be detailed at the bottom of the footing allowing the designer to take advantage of the full footing depth for moment and shear calculations. Due to the low number and size of bars typically required, conflicts with pile spacing could exist, but slight field adjustments can be made. Additional bars shall be detailed over the piles transversely and longitudinally. See File No. 15.03-3 for an example of typical footing reinforcement for a wall pier using steel piles.

PILE TYPES:

Minimum tip elevation, where required for scour, lateral stability or to attain minimum pile lengths/embedment, and design capacity for all piles shall be specified in the General Notes on the Title Sheet. See standard General Notes in Chapter 2, Section 3. All elevations in notes shall be provided to an even foot interval (i.e. not tenths/hundredths). Base estimated quantities on the estimated tip elevation, not the minimum tip elevation.

Driving Test for C-I-P Concrete Pile, Prestressed Concrete Pile, Steel Pile and Timber Pile shall be broken out of the overall production pile quantity and the driving test quantity provided shall be 10 feet longer than the estimated pile lengths. See Chapter 3 for the List of Bid Items.

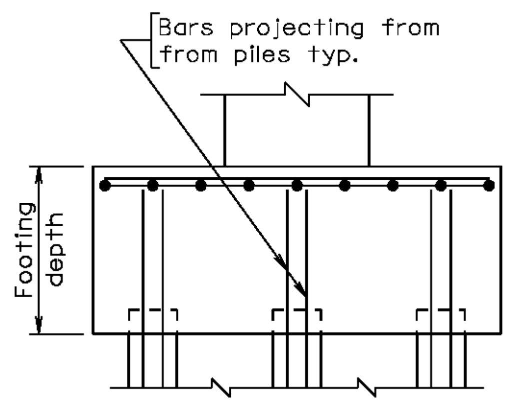
Steel H-piles:

Use HP10x42 as bearing and/or friction piles unless HP12x53 is needed for design. Steel H-piles are better suited to penetrate weathered rock or natural/man-made obstructions (e.g., boulders, buried rubble) than prestressed piles. Pile points should be used if driven to refusal. Grade 50 steel shall be used and indicated in the General Notes. If the lateral resistance of an H-pile is important, the strong and weak axes of the pile shall be considered.

Prestressed piles:

For square prestressed concrete pile details, see VDOT Manual of the Structure and Bridge Division, Volume V – Part 3, Current Details, standard BPP-1. The footing depth shall be determined so that the bars projecting from the piles will not interfere with the top mat of reinforcement in the footing as shown in the ELEVATION view. Cap depth for pile bents shall be determined so that the bars projecting from the piles will not interfer with the top main reinforcement. The bars projecting from the piles shall not be detailed as bent bars.

Prestressed piles are better suited as friction piles as they cause higher soil displacement during driving as compared to steel H-piles. Additionally, concrete-to-soil friction is typically higer than steel-to-soil friction. Prestressed piles have the same section properties/capacity in both axes (i.e. do not have a weak direction).



ELEVATION

Cast-in-place (C-I-P) concrete piles:

Substitution of prestressed piles with C-I-P concrete piles shall only be allowed when noted in the General Notes on the title sheet. C-I-P concrete piles shall not be used on pile bents or where driving may induce shell collapse. These areas include, but are not limited to, the Fredericksburg District and the zone from Mercury Boulevard to Hampton Roads Harbor in the Hampton Roads District. See standard General Notes in Chapter 2, Section 3.

PILE TYPES (Continued):

Cylinder piles:

Cylinder piles have hollow cross-sections capable of resisting large combined axial loads and bending moments and are suitable for larger unbraced lengths such as deep water applications. Designers should check with local suppliers for available diameters, wall thicknesses, number of tendons, lengths and other properties for design so an efficient cross-section can be selected to meet specific project needs. The hollow cross-section facilitates connection to either precast concrete or cast-in-place caps.

Timber piles:

Use of timber piles shall be limited to timber pedestrian bridges, shared use path bridges and support for box culverts on soft ground. Timber piles above ground water, in the splash zone or partially submerged are subject to decay. Timber piles shall be treated with preservatives unless permanently and completely submerged.

PILE REQUIREMENTS:

See AASHTO LRFD specifications, Article 10.7.1.2 with VDOT Modifications, current IIM-S&B-80.

Spacing: Steel H-piles: as friction piles, not less than 3.5 x nominal size of the piles
as bearing piles, not less than 2'-6"

Concrete piles: not less than 3 x diameter or side dimension of the piles

Timber piles: not less than 2'-6"

Edge distance: The distance from the side of any pile to the nearest edge of the footing shall not be less than 9".

Driving tolerance: Driving tolerance(s) for piles shall be considered when determining shear at the critical section.

Steel and concrete piles:

For elements supported by a single row of plumb, battered or staggered piles including, but not limited to, bent caps: +/- 3" about the long axis of the footing

All other piers except as noted above: +/- 6" about both major axes.

For driving tolerances of timber piles and other information, see VDOT *Road and Bridge Specifications*, Section 403 and Table IV-1.

Projection (embedment) in caps/footings:

Steel piles: 12" into footing
18" into footing when subjected to intermittent uplift (under any loading condition)
18" into footing plus a positive method of anchoring the pile to the footing when subject to uplift under seismic loading

Cast-in-place or precast concrete piles including cylinder piles: 6" into footing with reinforcement projecting to obtain development as required by the design

Timber piles: 12" into footings

DRILLED SHAFT REQUIREMENTS:

Drilled shaft spacing and driving tolerance shall be in accordance with VDOT *Special Provision for Drilled Shafts* and AASHTO LRFD specifications with VDOT Modifications, current IIM-S&B-80.

Projection (embedment) in caps/footings: 6" into footing with reinforcement projecting to obtain development as required by the design

DRILLED SHAFTS:

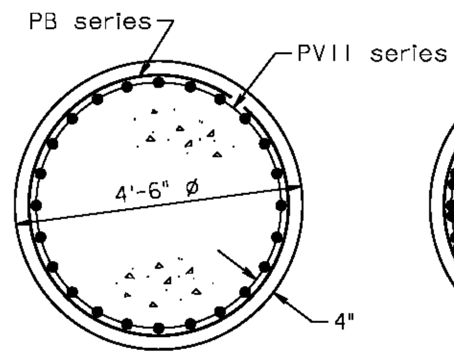
General:

Drilled shafts have a high axial and lateral capacity and may be economical where large numbers of steel or prestressed piles would be required. When weathered rock prevents conventional pile driving a sufficient distance below the scour elevation, drilled shafts may be a solution. Since there is no significant vibration during construction, drilled shafts can be used when there is risk of disturbing existing structure(s) by pile driving. Drilled shafts often do not require a footing (i.e. columns can be individually supported by a drilled shaft). Concerns to consider when determining whether drilled shafts are appropriate include lack of redundancy, quality is sensitive to construction procedure and the presence of groundwater can make construction difficult.

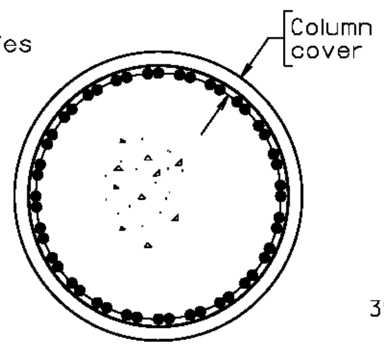
The minimum concrete cover for main vertical (principal) reinforcement and ties and spirals is 1" more than for pier columns. See current IIM-S&B-80. Radial splicing to provide the required minimum spacing for additional bars shall only be permitted where tied to inner hoops to maintain position. See File No. 15.02-7 for similar details in columns.

Drilled Shaft Directly Supporting Column:

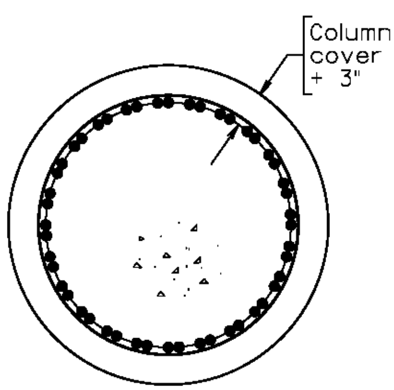
When a single column is individually supported by a drilled shaft, the drilled shaft diameter shall be a minimum of 6" greater than the column diameter. However, the vertical reinforcement in the drilled shaft shall align with the position of the vertical reinforcement in the column, i.e. the cover for the vertical reinforcement will be at least 3" more in the drilled shaft than in the column.



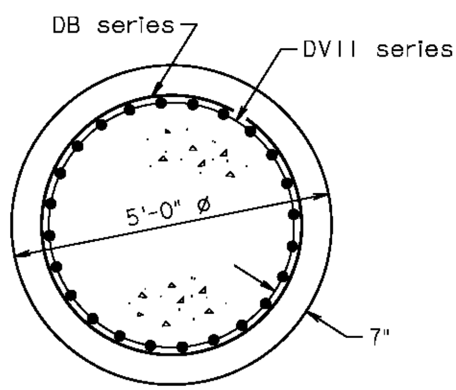
SECTION A-A



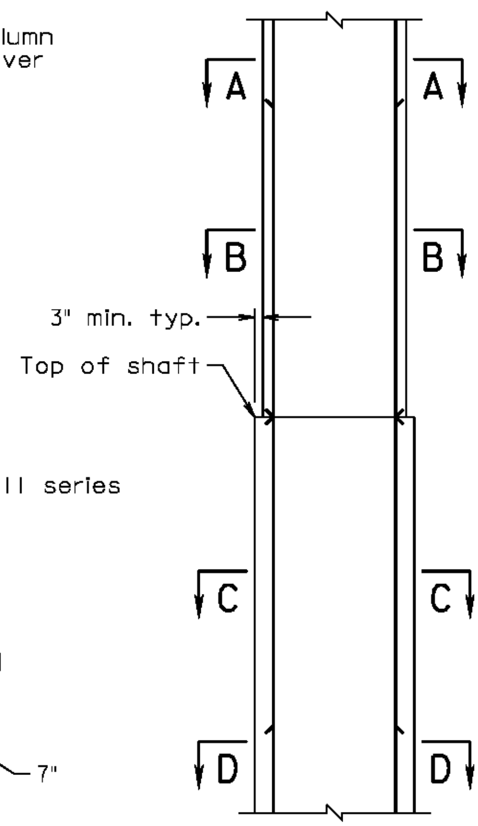
SECTION B-B



SECTION C-C



SECTION D-D



PART ELEVATION

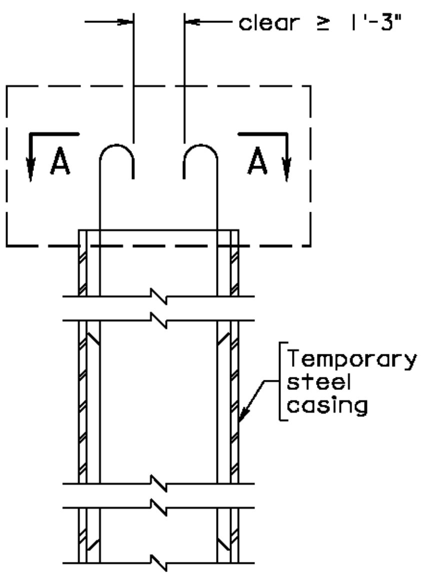
PIER DETAILS
PIER COMPONENTS AND MISCELLANEOUS DETAILS
DRILLED SHAFTS

DRILLED SHAFTS (Continued):

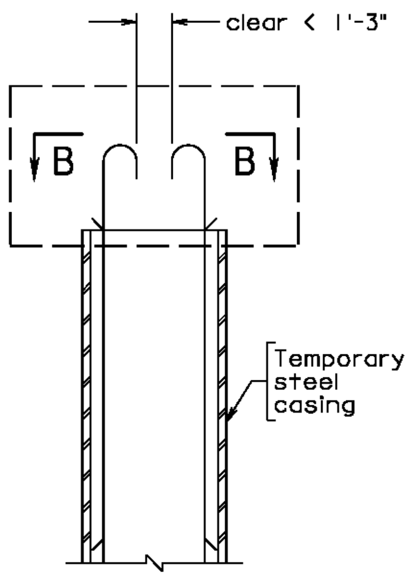
Drilled Shafts Supporting Footing:

When drilled shafts support a footing, the drilled shaft bars must be fully developed into the footing. Typically, large bar sizes and minimum footing depth require the main drilled shaft reinforcement to be hooked. If turned outward, these hooked bars will interfere with removing the temporary steel casing used to form drilled shafts. If turned inward, these hooked bars may interfere with the tremie tube and/or collar(s) during concrete placement when the clear opening in the drilled shaft reinforcement cage is less than 1'-3".

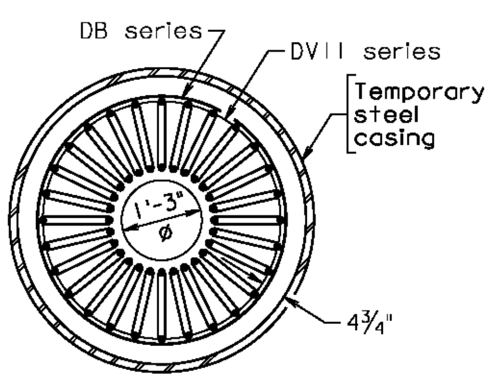
Where clear distance between hooks is greater than or equal to 1'-3", hooks shall be oriented toward the interior and main vertical reinforcement spliced away from the top of drilled shaft (Case 1). Where clear distance between hooks is less than 1'-3", splice main vertical reinforcement at the top of drilled shaft and rotate the main vertical reinforcement hooks to provide 1'-3" clear where feasible (Case 2).



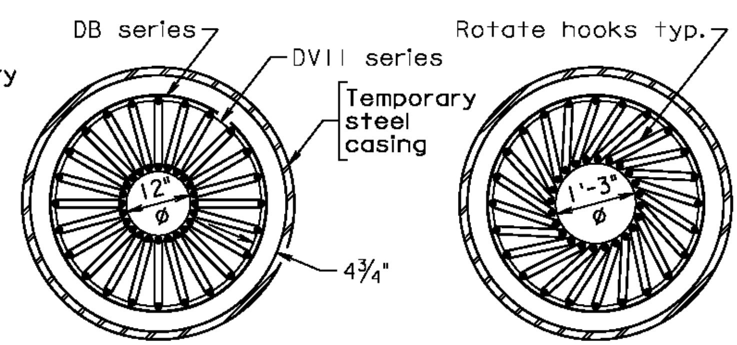
CASE 1



CASE 2



VIEW A-A (CASE 1)



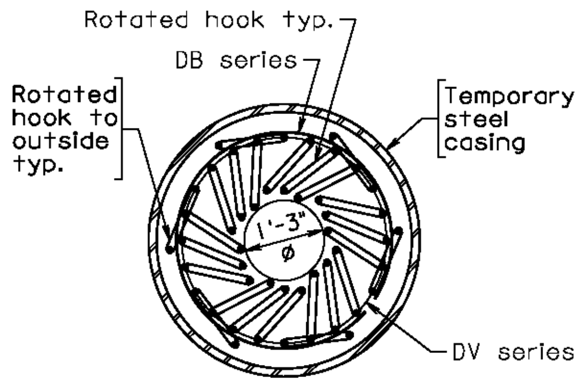
Radial **Rotated**

VIEW B-B (CASE 2)

DRILLED SHAFTS (Continued):

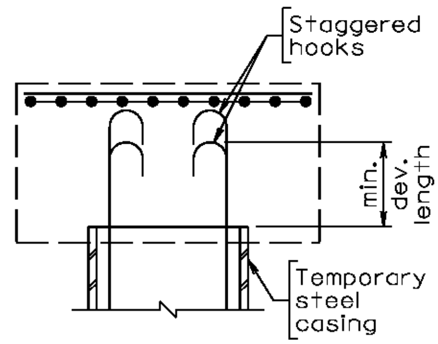
Drilled Shafts Supporting Footing (continued):

Where there is insufficient room to rotate the hooks to provide the minimum 1'-3" clear opening (bar overlap), the configuration is considered too tight for field implementation or it is desirable to open up spacing, some hooks may be rotated so the ends fall outside the reinforcement cage, but within the limits of the temporary steel casing. Rotate the remaining hooks on the interior to provide the minimum 1'-3" clear opening and relatively even spacing between hook ends.



VIEW B-B (CASE 2 ALTERNATE)

Where the minimum 1'-3" clear opening cannot be obtained by methods discussed in Case 2, options include increasing drilled shaft diameter and increasing footing depth. With sufficient footing depth, main vertical drilled shaft bars can be developed without hooks (Seismic Zone 1 only). With sufficient footing depth, hooks can also be staggered beyond the minimum development length, but must clear the top reinforcement mat (see PART ELEVATION). Where staggered hooks are used, alternate the position of adjacent bars.



PART ELEVATION

Where permanent casing will be used, hooks shall be turned outward.

Details similar to those shown in this section (ELEVATION and VIEW) shall be provided on the plan sheets to show the intended layout of the hooks.

STATE	FEDERAL AID		STATE		SHEET
	ROUTE	PROJECT	ROUTE	PROJECT	NO.
VA.	—		88	0088-099-101, B635	24

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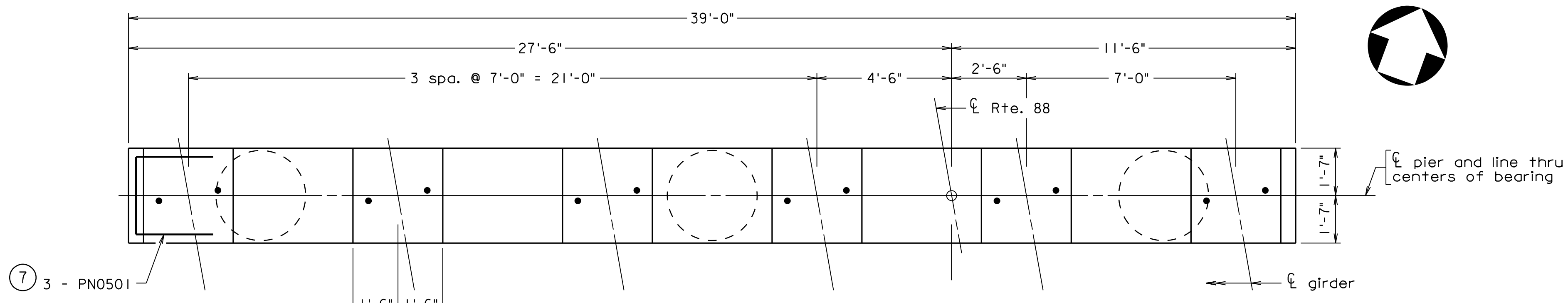
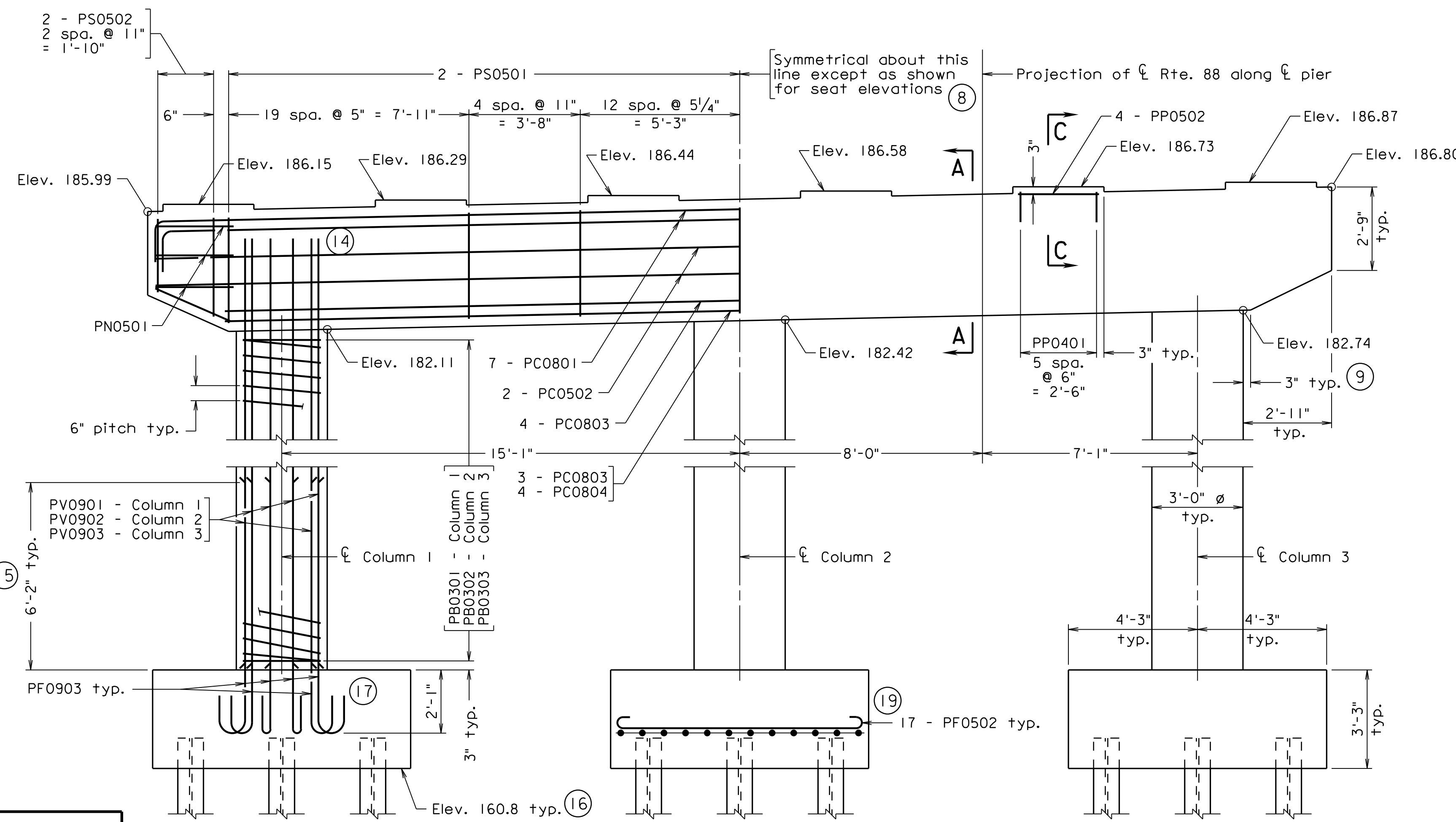


PLAN OF CAP

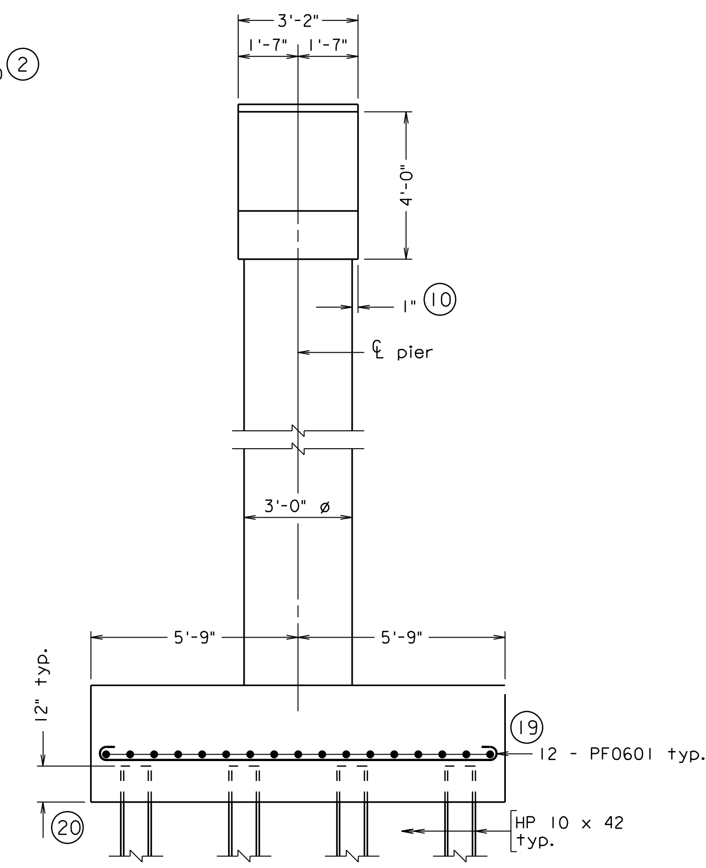
Notes:

When finishing concrete between and beyond pads, float surface to drain from ℄ pier to edges of cap.

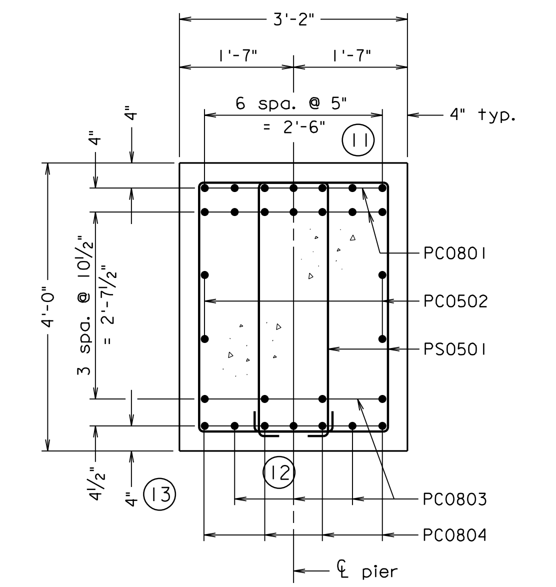
Welded wire fabric may be substituted for spiral reinforcement in pier columns, maintaining an equivalent area of reinforcement, subject to approval by the Bridge Engineer.



ELEVATION

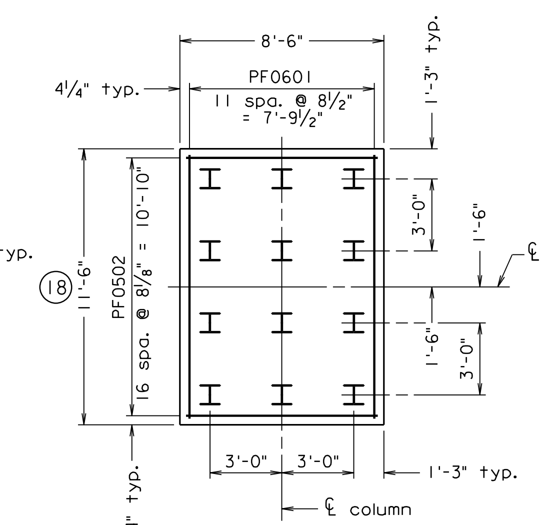


END VIEW



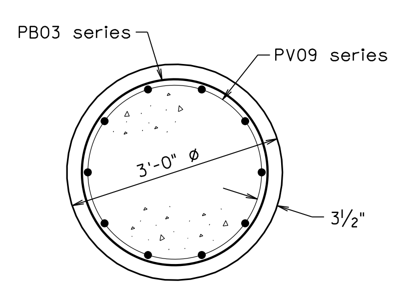
SECTION A-A

Scale: ¾" = 1'-0"



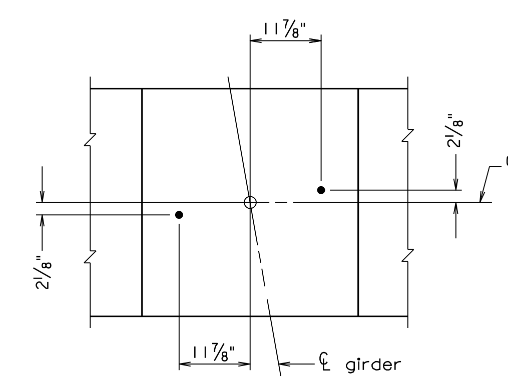
TYPICAL FOOTING PLAN

Scale: ¼" = 1'-0"



TYPICAL SECTION OF COLUMN

Scale: ¾" = 1'-0"



(5) TYPICAL ANCHOR BOLT LAYOUT

Scale: ¾" = 1'-0"

PIER DETAILS
SUGGESTED DESIGN AND DETAILING
SAMPLE MULTI-COLUMN PIER SHEET

(1)

Scale: ⅜" = 1'-0" unless otherwise noted

(21)

No.	Description	Date
	Revisions	

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

STRUCTURE AND BRIDGE DIVISION

PIER

Designed: SJB
Drawn: SJB
Checked: SJB

Date: Oct. 2010

Plan: 999-

VOL. V - PART 2
DATE: 17Dec2010
SHEET 1 of 6
FILE NO. 15.03-1

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(24)

VDOT S&B DIVISION
RICHMOND, VA
STRUCTURAL ENGINEER

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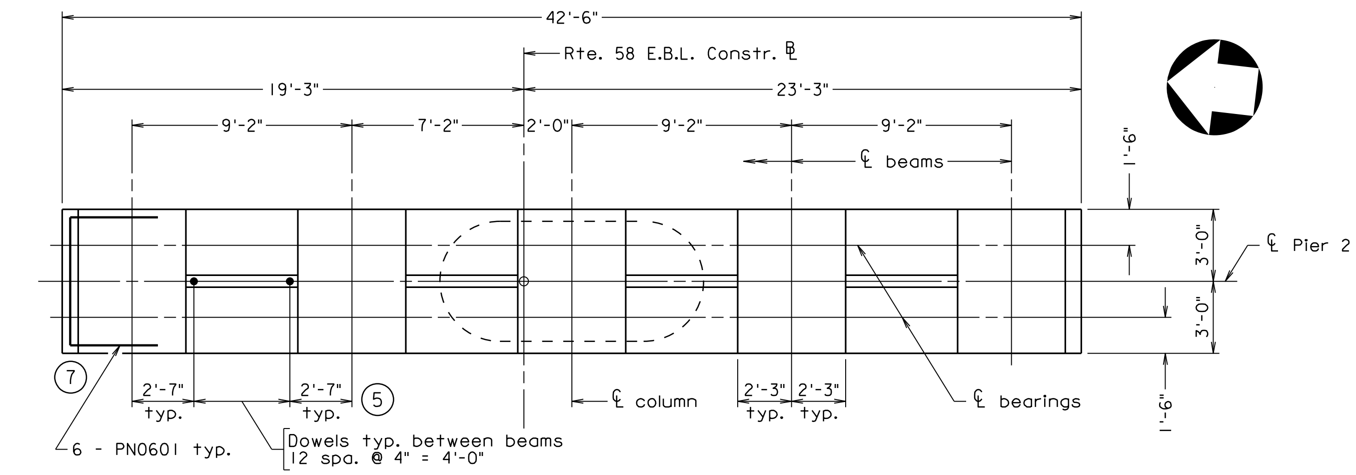
STATE	FEDERAL AID ROUTE	FEDERAL AID PROJECT	STATE ROUTE	STATE PROJECT	SHEET NO.
VA.	—		58	0058-041-116, B607	16

(22)

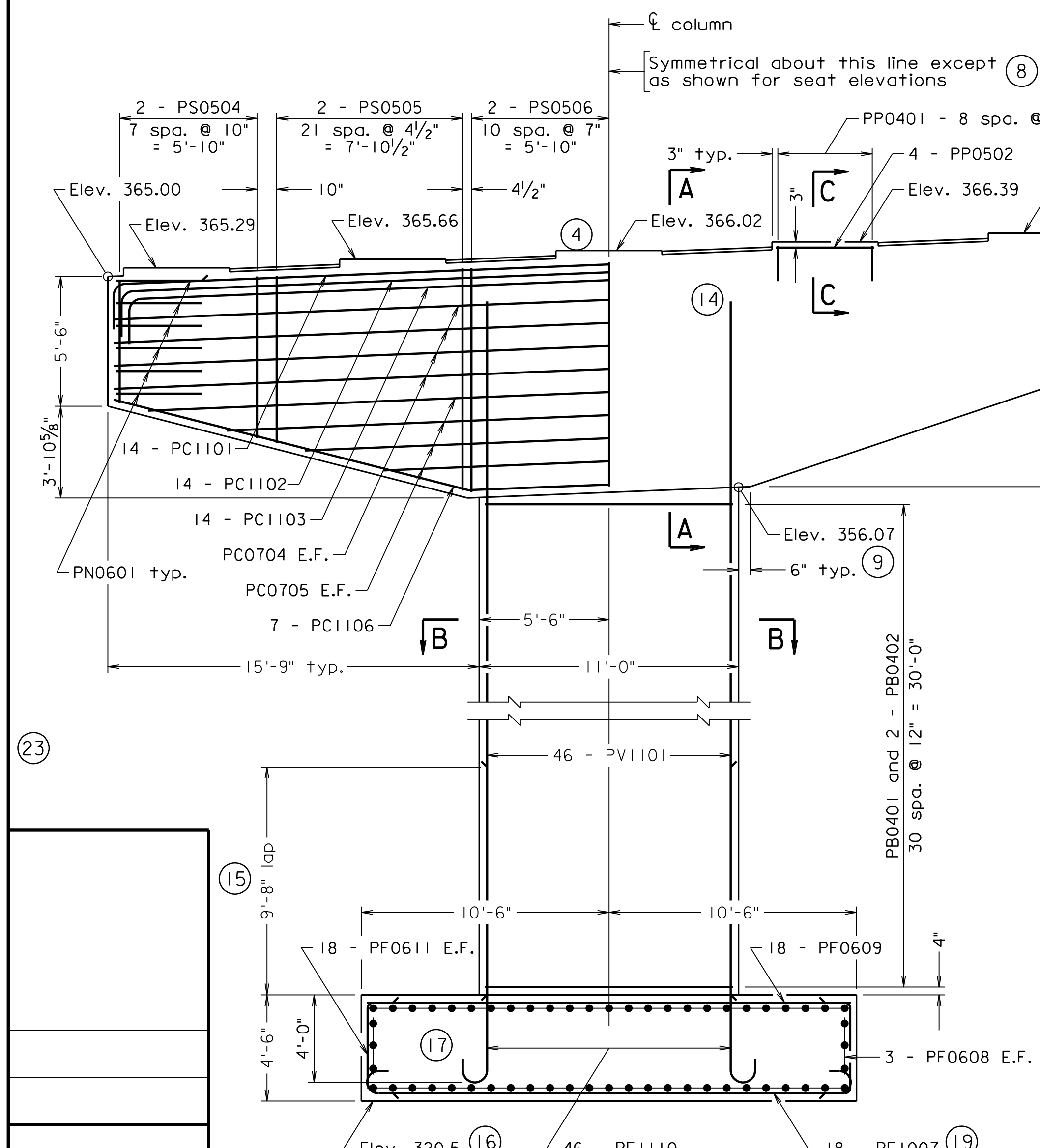
Notes:

When finishing concrete between and beyond pads, float surface to drain from ℄ pier to edges of cap.

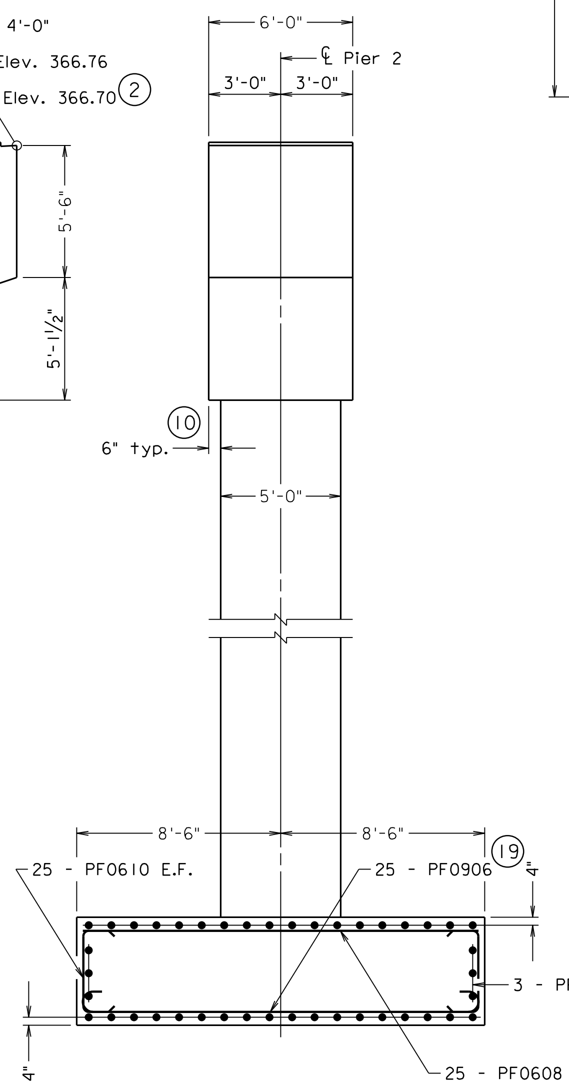
For closure diaphragm details, see sheet 12.



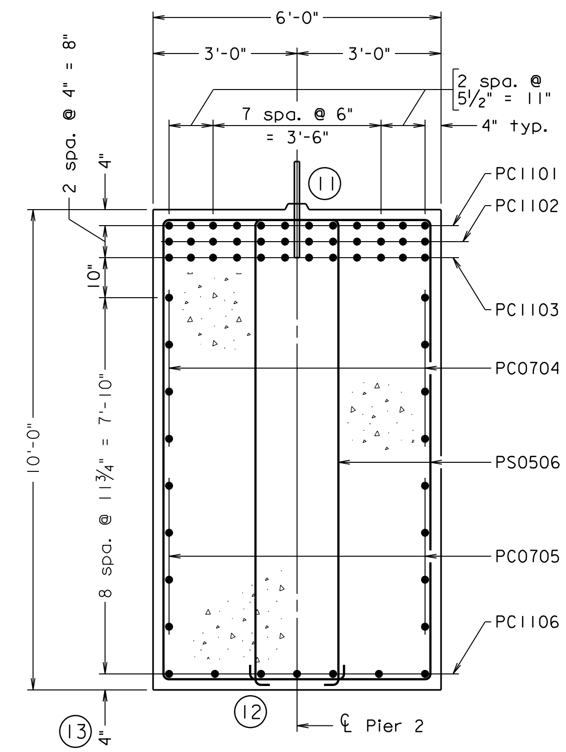
PLAN OF CAP



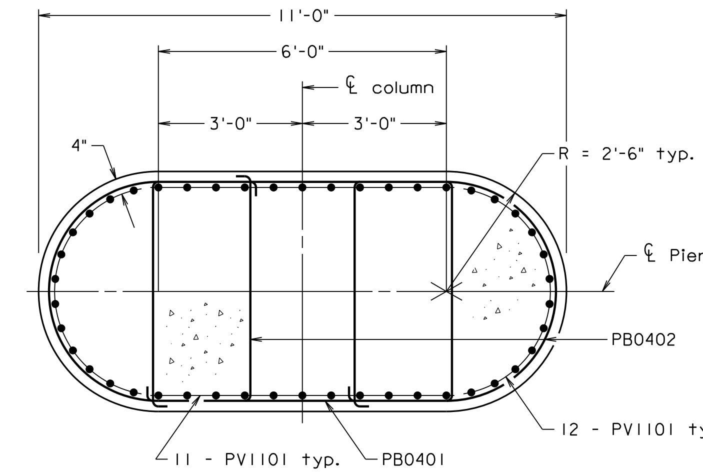
ELEVATION



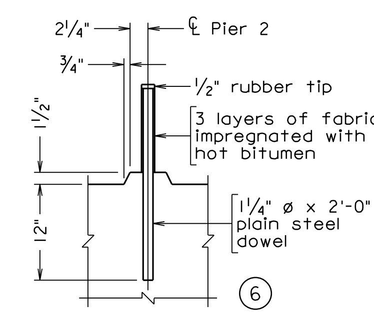
END VIEW



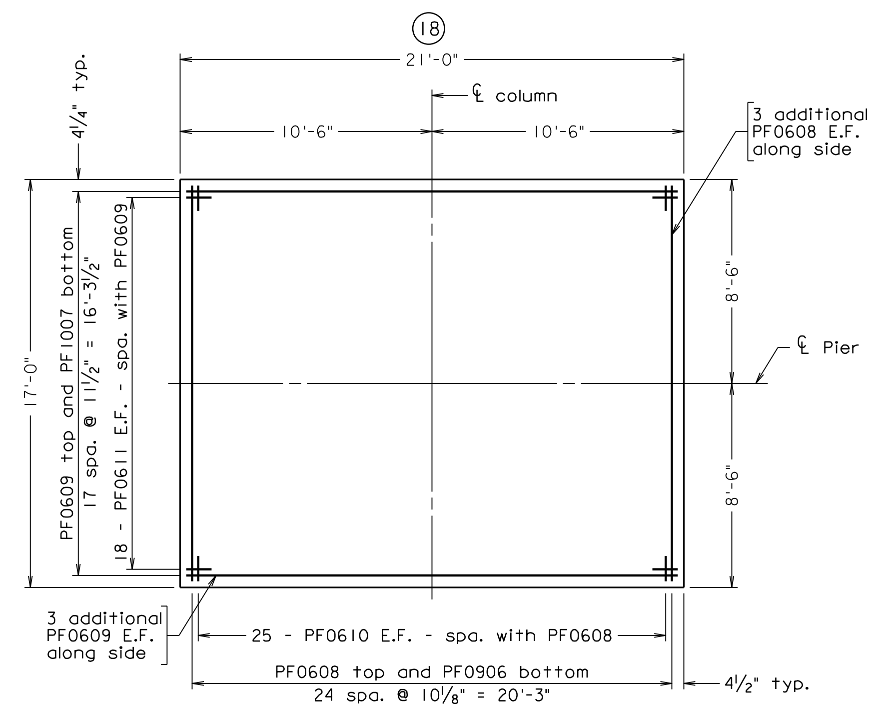
SECTION A-A
Scale: ½" = 1'-0"



SECTION B-B
Scale: ½" = 1'-0"

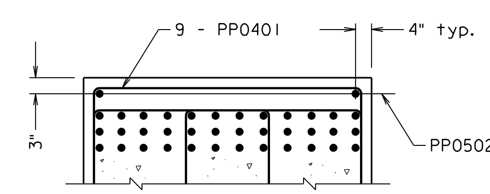


DOWEL DETAIL
Not to scale



FOOTING PLAN

(21)



SECTION C-C
Scale: ½" = 1'-0"

PIER DETAILS
SUGGESTED DESIGN AND DETAILING
SAMPLE HAMMERHEAD PIER SHEET

(1) Scale: ¼" = 1'-0" unless otherwise noted

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(23)

(24)

VDOT S&B DIVISION
RICHMOND, VA
STRUCTURAL ENGINEER

No.	Description	Date
	Revisions	

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DIVISION

PIER 2

Designed: SJB Drawn: SJB Checked: SJB	Date: Oct. 2010	Plan: 999-

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SHEET 2 of 6
FILE NO. 15.03-2

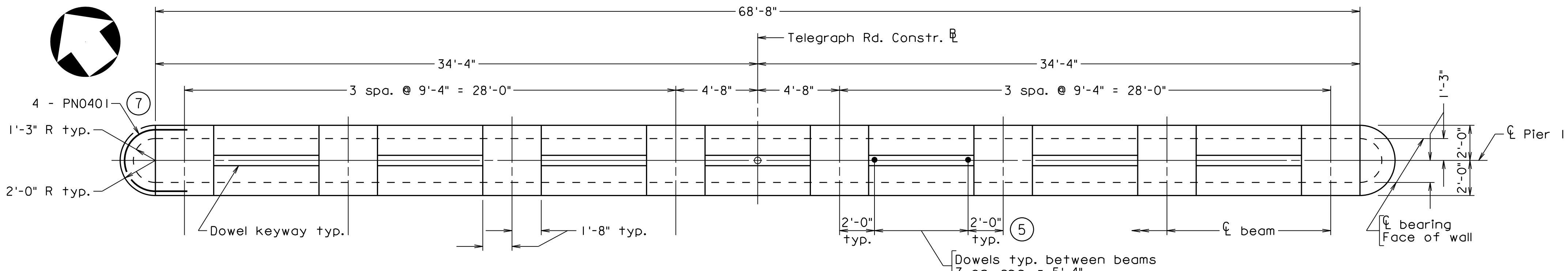
STATE	FEDERAL AID		STATE		SHEET
	ROUTE	PROJECT	ROUTE	PROJECT	NO.
VA.	—		58	0058-041-116, B607	16

22

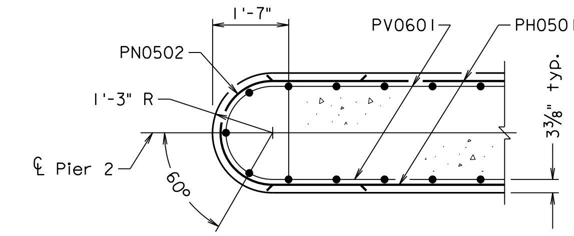
Notes:

When finishing concrete between and beyond pads, float surface to drain from ℄ pier to edges of cap.

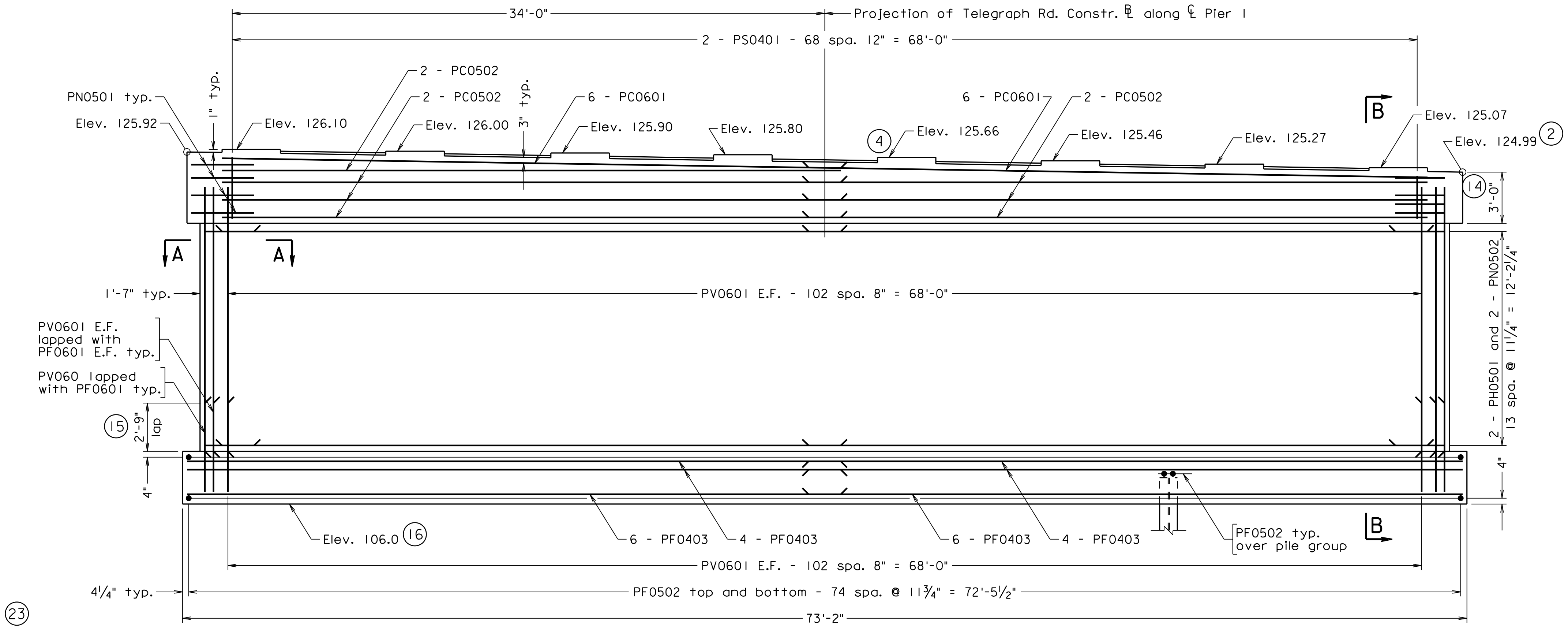
For closure diaphragm details, see sheet 12.



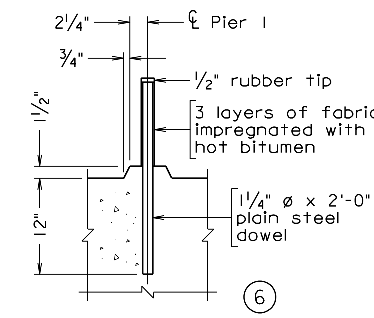
PLAN OF CAP



SECTION A-A

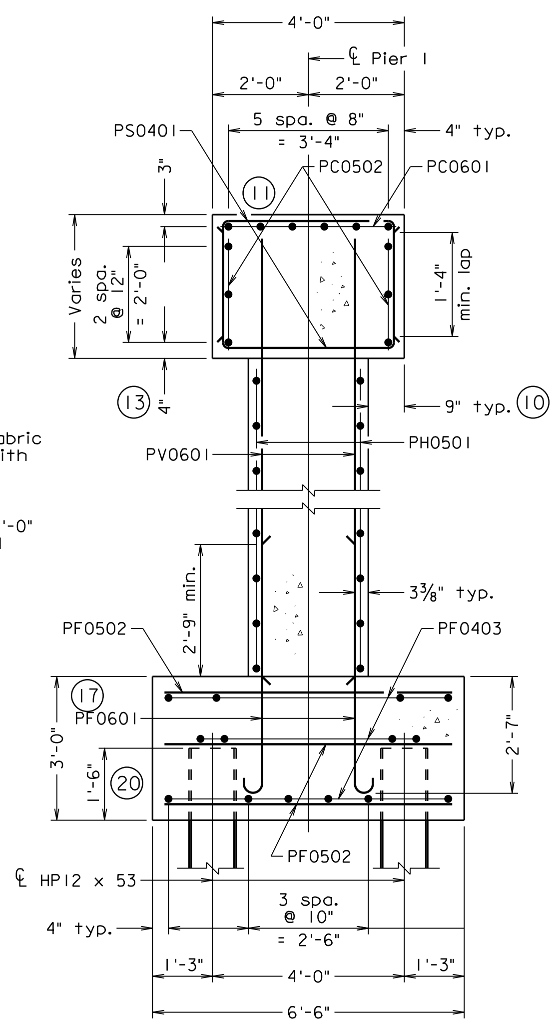


ELEVATION



DOWEL DETAIL

Not to scale



SECTION B-B

23

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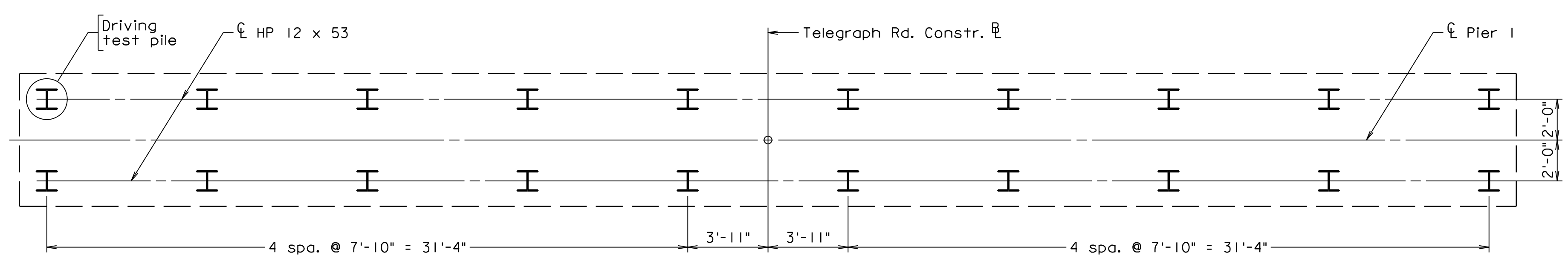

PILE PLAN

21

PIER DETAILS
SUGGESTED DESIGN AND DETAILING
SAMPLE WALL PIER SHEET

24

VDOT S&B DIVISION
RICHMOND, VA
STRUCTURAL ENGINEER

1

Scale: ¼" = 1'-0" unless otherwise noted

No.	Description	Date
	Revisions	

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

STRUCTURE AND BRIDGE DIVISION

PIER 1

Designed: SJB
Drawn: SJB
Checked: SJB

Date	Plan
Oct. 2010	999-

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SHEET 3 of 6
FILE NO. 15.03-3

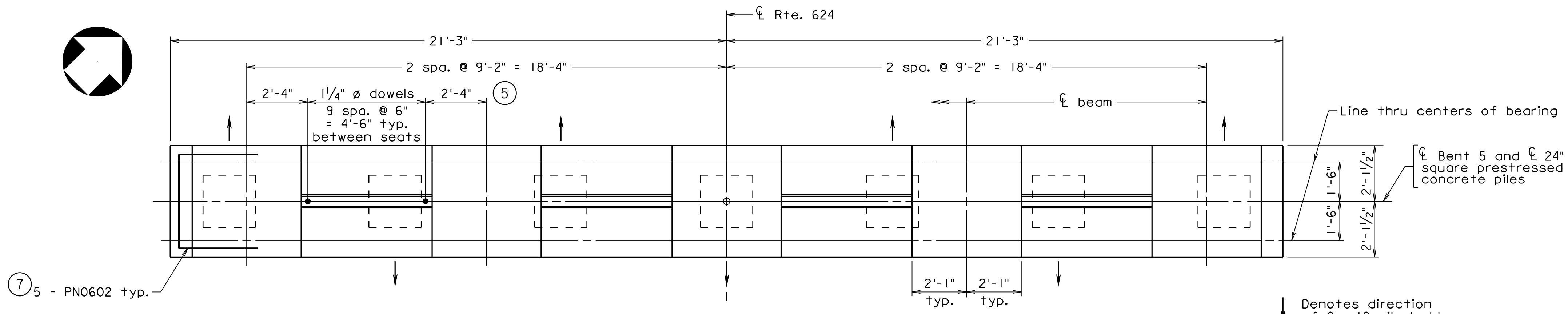
STATE	FEDERAL AID ROUTE	FEDERAL AID PROJECT	STATE ROUTE	STATE PROJECT	SHEET NO.
VA.	—		624	624-079-148, B611	12

(22)

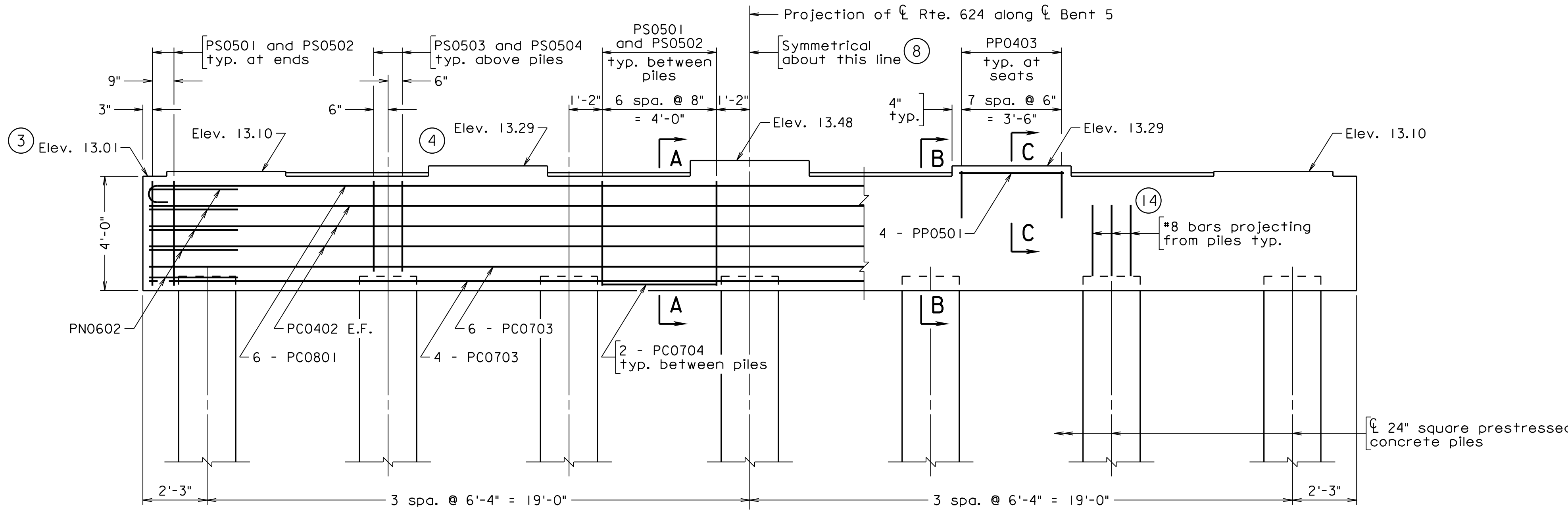
Notes:

When finishing concrete between and beyond pads, float surface to drain from ℄ bent to edges of cap.

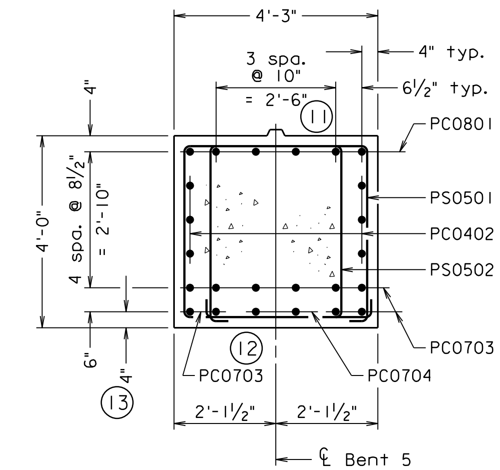
For closure diaphragm details, see sheet 19.



PLAN OF CAP

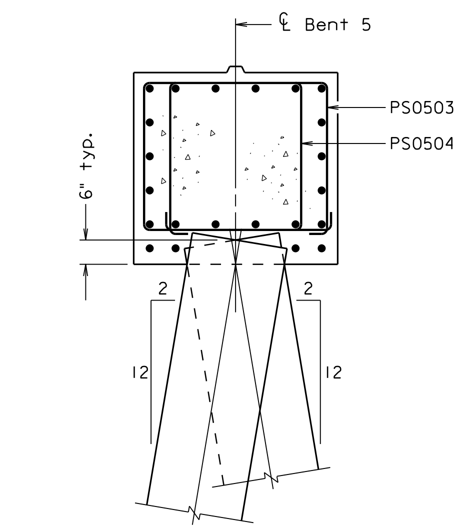


ELEVATION



SECTION A-A

Scale: ½" = 1'-0"

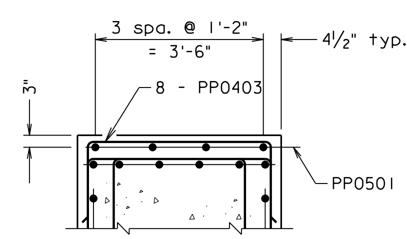


SECTION B-B

For details not shown, see Section A-A

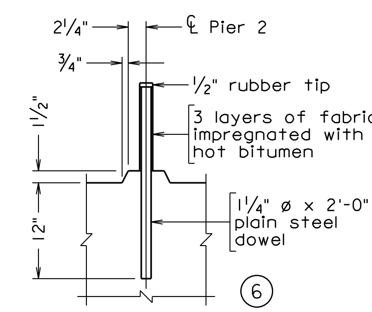
Scale: ½" = 1'-0"

(21)



SECTION C-C

Scale: ½" = 1'-0"



DOWEL DETAIL

Not to scale

(23)

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(24)

VDOT S&B DIVISION
RICHMOND, VA
STRUCTURAL ENGINEER

PIER DETAILS
SUGGESTED DESIGN AND DETAILING
SAMPLE PILE BENT SHEET

(1)

Scale: ⅜" = 1'-0" unless otherwise noted

No.	Description	Date
	Revisions	

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION

STRUCTURE AND BRIDGE DIVISION

BENT 5

Designed: SJB
Drawn: SJB
Checked: SJB

Date: Oct. 2010

Plan: 999-

VOL. V - PART 2
DATE: 17Dec2010
SHEET 4 of 6
FILE NO. 15.03-4

DETAILING CHECK LIST FOR PIERS / BENTS

1. Show Plan of Cap, Elevation, End View and (Typical) Footing Plan at a scale of $^3/_8$" = 1'-0", but no smaller than $^1/_4$" = 1'-0". Show remaining details at a scale of $^3/_4$" = 1'-0", but no smaller than $^1/_2$" = 1'-0".

2. Slope cap from end-to-end when seat heights of 4" or more can be avoided. Slope bottom of cap parallel to top of cap. Minimum pad heights should be 1" at the edge of pad and cap elevations should be established on this basis. For sloping cap, provide elevation at top of column on higher side.

3. When cap is horizontal, the top of cap elevation should be established 1" below the lowest pad and dimensioned in the elevation view.

4. Where "arriving" and "departing" beam/girders result in calculated differences in pad elevations, the difference is to be applied as follows:

   difference = $^1/_8$" : apply to bolster thickness.

   difference > $^1/_8$" to $^1/_2$" : apply to bearing height.

   difference > $^1/_2$" : apply to pad elevations or to superstructure members taking aesthetics into consideration.

5. The anchor bolt layout is to be shown in plan view if space allows. If dimensions vary, add table.

6. Include DOWEL DETAIL for piers/bents with prestressed beams and fixed closure pours.

7. Length of PN bars should be determined to obtain a Class A splice with top PC bar.

8. For a symmetrical pier cap, reinforcing steel may be shown in half of cap and pier labeled "Symmetrical about this line except as shown (for superelevation) (etc.)."

9. Use minimum of 3".

10. Use minimum of 1" unless bearing layout requires larger cap.

11. Space bars to clear anchor bolts by 1" minimum and to allow ample room for concrete vibrators. To alleviate spacing problems, consider adding another row of bars.

12. Hooks in stirrups should be shown at bottom of cap so as to preclude any possible interference with anchor bolts and shall be placed away from exterior corners.

13. Dimension must allow for fabrication tolerances for stirrups.

14. Length of main column reinforcement should be determined so as to clear lowest layer(s) of top main cap bars by at least 2".

15. Use Class C splice for main column reinforcement lap with footing bars.

16. Provide bottom of footing elevation to the tenth of a foot (i.e. not hundredths).

DETAILING CHECK LIST FOR PIERS / BENTS (Continued)

17 Footing-to-column bars shall be hooked in footing. Bars are to rest on bottom mat of main footing reinforcement and shall provide the minimum embedment length into footing. See File Nos. 07.101-2 and -3. Embedment length shall be not less than basic development length for compression.

18 Where a rectangular footing is required by design, the longer dimension should be used perpendicular to centerline of pier.

19 Main footing bars shall be hooked. Bars shall be located above piles.

20 For minimum pile embedment, see File No. 15.02-12. Note the H-piles depicted on File Nos. 15.03-1 and -3 are shown for seismic Zone 1 with no uplift and intermittent uplift respectively.

21 For instructions on completing the title block, see File No. 03.03.

22 For instructions on completing the project block, see File No. 03.02.

23 For instructions on developing the CADD sheet number, see File Nos. 01.01-7 and 01.14-4.

24 For instructions on sealing and signing, see File No. 01.16.

GENERAL INFORMATION:

Forces on the superstructure are transferred to the substructure through the bearings. Bearing design, joint design, unit length (i.e. length of continuous spans between joints) and subsurface conditions affect pier design and a holistic approach is required to ensure the bridge functions properly.

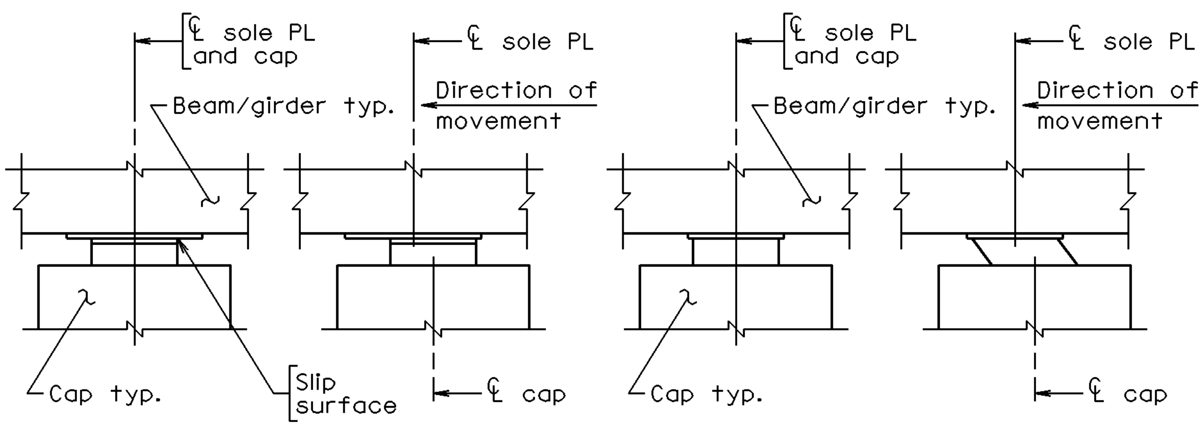
Grade (vertical profile) effects can typically be ignored in design.

Bearings:

Bearings can be fixed, guided expansion or expansion. Fixed bearings do not allow movement in any direction. Guided expansion (slotted) bearings allow movement in one direction (typically longitudinal to bridge, transverse to bridge or in direction of long chord) and no movement perpendicular to the expansion direction. Expansion bearings allow movement in any direction.

Although a fixed bearing is attached to the substructure to prevent movement, the substructure may deflect based on its stiffness and the force transferred from the superstructure.

Expansion bearings either slip or distort. Slip bearings accommodate movement when temperature and shrinkage (where applicable) forces overcome the coefficient of friction for the particular bearing type. Substructures with slip bearings should be designed for the slip force.



As set **Expanded/Contracted** **As set** **Expanded/Contracted**

SLIP **DISTORTION**

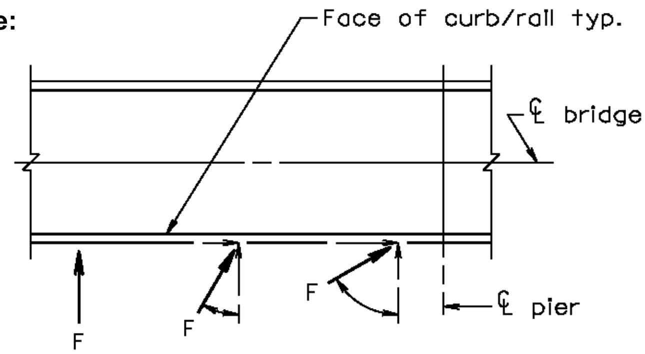
Typical values for slip coefficients are 0.25 for steel bearings, 0.10 for self lubricating bronze plates and 0.08 for steel bearings with PTFE surfaces. When determining the slip force, use dead load only. Live load reactions should not be included.

Elastomeric bearings accommodate movement by distortion. Total elastomer thickness in the various layers are sized to equal or exceed twice the anticipated movement. Distortion of the elastomeric bearing creates internal forces that are transferred to the substructure. These internal forces can be large enough to cause substructure deflection and the overall movement becomes a function of both pad distortion and substructure deflection. See examples starting on File No. 15.04-4.

GENERAL INFORMATION (Continued):

Wind and Braking Forces on Superstructure:

Wind forces on the superstructure and live load can have both a transverse and longitudinal component. These forces are computed for various skew angles of wind perpendicular to the longitudinal direction of the bridge. Thus 0 degree skew angle is perpendicular to the bridge and consists of the largest transverse force wind pressure.



0 Degree 30 Degree 60 Degree

Wind on Superstructure and Live Load

Braking Force (Traction) consists of only a longitudinal force on a straight bridge, but will have a transverse component on a curved bridge due to centrifugal force.

Longitudinal forces and transverse forces are typically distributed among the number of bearings fixed in that direction. For example, on a straight four-span bridge with fixed bearings at Pier 2 and guided expansion (slotted) bearings at the abutments and Piers 1 and 3, Pier 2 should conservatively be designed for all the longitudinal force. Piers 1, 2 and 3 should be designed for their portion of the transverse force (i.e., one-half span back and one-half span up station). The design of Piers 1 and 3 would include longitudinal temperature and shrinkage forces, where applicable, and Pier 2 any unbalance force as discussed below.

Temperature (Expansion, Contraction) and Shrinkage Forces (where applicable):

Expansion/contraction of the superstructure develops forces which are transferred to the substructure through the bearings. Bridges with concrete superstructures must also be designed for shrinkage. A shrinkage coefficient of 0.0003 may be used except for segmentally constructed bridges. For typical multi-beam/girder prestressed concrete superstructures not integral with the substructure, the shrinkage coefficient can be considered to include superstructure creep effects.

The total design movement at each bent/pier shall be 0.65 times the design thermal movement range. The design thermal movement range shall be obtained from AASHTO LRFD specifications, Table 3.12.2.1-1 using the moderate climate range for steel superstructures (120 °F) and the cold climate range for concrete superstructures (80 °F). See current IIM-S&B-80. Therefore, +/- 78 °F should be used to determine temperature forces for pier/bent design for steel superstructures and +/- 52 °F for concrete superstructures. A 52 °F contraction combined with shrinkage will control for concrete superstructures. Where elastomeric expansion bearings are used, the maximum elastomeric shearing resistance shall be used to determine temperature and shrinkage forces.

On a two-span continuous symmetric structure with fixed bearings at the pier, the temperature and shrinkage (where applicable) force would be zero at the pier. With unsymmetric spans, an unbalanced force will exist due to the unbalanced expansion lengths and/or bearing design.

Similarly for three or more span continuous symmetric structures with fixed bearings at the middle pier(s), the thermal center (neutral point) can be considered the center. However, relative stiffness between piers may shift the thermal center of a bridge and/or create unbalanced forces at fixed piers.

GENERAL INFORMATION (Continued):

Temperature (Expansion, Contraction) and Shrinkage Forces (where applicable):

When pier/bent heights differ by more than 25 percent between piers/bents within a continuous structural unit, the designer shall take into account the effects of substructure stiffness when determining the expansion length at any point (including joints) and the forces to be resisted at any substructure unit. Pier height is measured from the top of footing to the top of cap. For pile bents, height is measured from the assumed point of pile fixity to the top of cap. See Example 1.

When the summation of span lengths left or right of a fixed pier differ more than 25 percent, the designer shall take into account the effects of unsymmetric spans when determining the expansion length at any point (including joints) and the forces to be resisted at any substructure unit. See Example 2.

The design of pile bents is further complicated in that the magnitude of the temperature and shrinkage (where applicable) forces directly affects the location of the assumed point of fixity for the pile along with varying geotechnical conditions at each location. See Example 3.

Pier programs typically apply load factors internally. The load factors for force effects for Uniform Temperature, TU, and Shrinkage, SH, in the strength limit state depend on whether I_g or $I_{effective}$ was used to derive forces and are the same for both. Designers shall ensure that the appropriate load factors from AASHTO LRFD Article 3.4.1 are used in pier program runs for TU and SH.

All three examples use simplified analysis in conjuction with the gross moment of inertia for the pier columns. As such, a load factor of 0.5 and 1.0 would be used for strength and service limit states respectively for both TU and SH for load combinations within the pier program. Since the load factors are the same, TU and SH are considered together in the examples and the combined force is intended to be input as TU in the pier program.

Bridges with curved girders, skews greater than 20 degrees, integral piers, and/or column/pile lengths varying by more than 50 percent between extremes within an individual pier/bent should be modeled using a finite element analysis program to determine overall structure behavior due to thermal and other loads before designing piers, bearings or joints.

Bridge Layout:

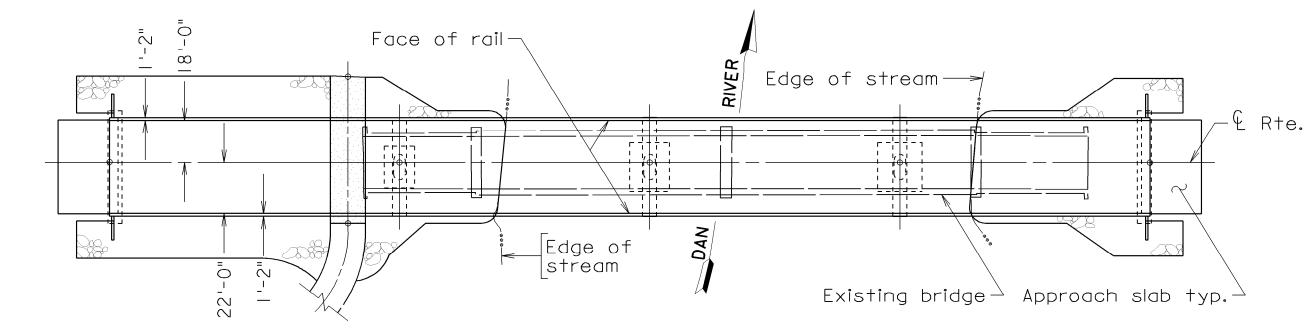
To obtain a satisfactory and efficient bridge design, the distribution of wind and traction forces needs to be balanced against the temperature and shrinkage demands. If too many piers are fixed, the temperature and shrinkage forces for the fixed piers will disproportionally control design. If too few are fixed, the wind and traction forces will disproportionally control design.

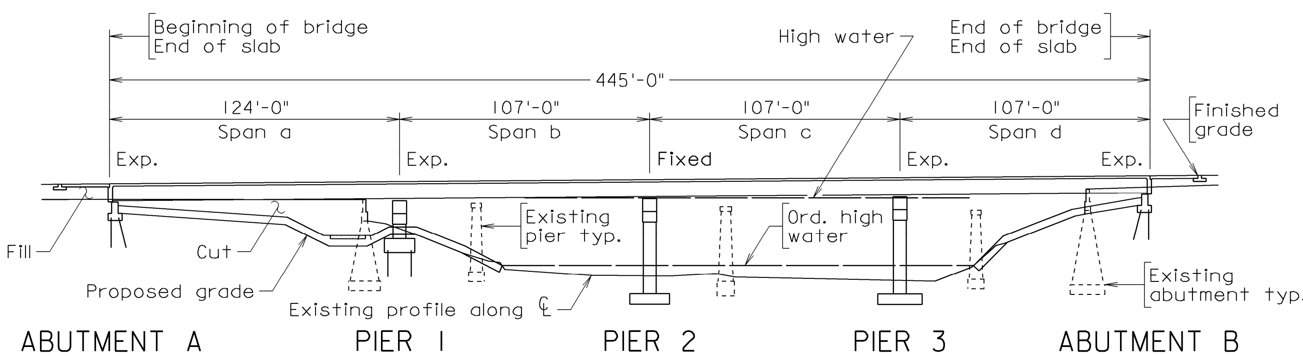
Joint type and size, where applicable, should be determined considering the overall bridge or unit temperature and shrinkage modeling.

Where temperature and shrinkage forces in the exterior piers/bents of a unit are too large or control column/pile size, various methods may be used to reduce forces. Increasing the elastomer thickness for elastomeric bearings or switching to slip bearings with low friction coefficients can lower design forces. Oversizing or slotting holes in fixed bearing sole plates can lower design forces by allowing some movement before engaging and facilitate fit-up during erection for steel superstructures. For pile bents, predrilling and placing select backfill around the pile once it is in place can lower the assumed point of fixity by increasing the pile flexibility and reducing the temperature/shrinkage forces. Adding a joint should only be done when all options are exhausted.

EXAMPLE 1: Hammerhead pier on spread/pile footing of varying height

This example consists of a 4-span continuous prestressed Bulb-T bridge with semi-integral abutments. Elastomeric bearings are used and dimensions shown in the calculation table are those from the final bearing designs. The preliminary hammerhead pier design uses a 5'-0" by 11'-0" column with circular ends. The superstructure typical section consists of 5 beam lines. Skew = 0°.





Sample calculations for the temperature loads are provided below and on the following sheet. Note that the thermal center of the bridge was determined by an iterative process (i.e. adjusting the position of the thermal center until opposing forces balance) and is located 14 feet to the left of Pier 2. Elastomeric deformation, Δelast, was also determined by an iterative process for each expansion pier (i.e. adjusting the value until the summation of the elastomeric deformation and pier deflection equals the total movement required).

Variables and equations:

∂ = coefficient of thermal expansion/contraction for normal weight concrete = 0.000006 per °F

ΔT = Temperature change, °F = 0.65 x 80 °F = 52 °F

L = Length from thermal center of bridge, feet

L_{elast} = Length of elastomeric bearing pad, inches

W_{elast} = Width of elastomeric bearing pad, inches

G_{max} = maximum elastomeric shearing resistance (range = 95 to 130 psi) = 0.130 ksi

Example continued on next sheet

EXAMPLE 1: Hammerhead pier on spread/pile footing of varying height (cont'd.)

Variables and equations (continued):

No_{pads} = No. of elastomeric bearings (for PS beams continuous for LL, count both bearing lines) = 10

t_{elast} = Total thickness of internal and external elastomeric layers (total pad height – shims), inches

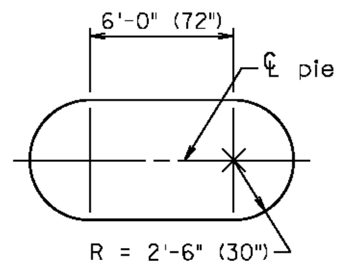
Δ_{elast} = Assumed/actual elastomeric deformation in direction of expansion/contraction, inches

P_{elast} = Force created internally in elastomeric pad for assumed/actual deformation, kips

$= L_{elast} \times W_{elast} \times G_{max} \times No_{pads} \times \Delta_{elast} / t_{elast}$

H_{ss} = Top of cap to top of footing (pier on pile or spread footing), feet

f'_c = Column (pile for pile bent) concrete strength = 3,000 psi



E_c = Modulus of elasticity for concrete, psi

$= w_c^{1.5} \times 33 \times f'^{0.5}_c = 145^{1.5} \times 33 \times 3000^{0.5} = 3{,}156{,}000$ psi

I_c = Uncracked moment of inertia for column(s) (piles for pile bent), in^4

$= [72" \times (60")^3 / 12] + [\pi \times (30")^4 / 4] = 1{,}932{,}200\ in^4$

P_{SS} = Force required to deflect substructure required distance at fixed pier/bent, kips

$= 3 \times E_c \times I_c \times \Delta_{temp} / (H_{SS} \times 12)^3$

Δ_{SS} = Substructure deflection in direction of exp./contraction, inches = $P_{elast} \times (H_{SS} \times 12)^3 / (3 \times E_c \times I_c)$

Δ_{temp} = Total movement required at pier/bent including shrinkage [prestressed superstructure], inches

$= (\partial \times L \times \Delta T \times 12) + (0.0003 \times L \times 12)$

Sample calculation:

With longer spans and shorter (stiffer) piers on the left of Pier 2, assume the thermal center of the bridge is 8.5 feet left of Pier 2.

The required movement at Piers 1 and 3 will be a combination of pier deflection and pad distortion. Pier 2 is fixed and the pier must deflect the required movement. The abutments have two rows of piles and are assumed rigid, so the elastomeric pad must distort the required distance.

Using the values found in the calculation table on the following sheet, the numbers for Pier 1 are:

$\Delta_{temp} = (\partial \times L \times \Delta T \times 12) + (0.0003 \times L \times 12) = (0.000006 \times 98.5 \times 52 \times 12) + (0.0036 \times 98.5) = 0.723"$

Try Δ_{elast} = 0.600"

$P_{elast} = L_{elast} \times W_{elast} \times G_{max} \times No_{pads} \times \Delta_{elast} / t_{elast} = 26 \times 10 \times 0.130 \times 10 \times 0.600 / 2.152 = 94.2$ kips

$\Delta_{SS} = P_{elast} \times (H_{SS} \times 12)^3 / (3 \times E_c \times I_c) = 94.2 \times (13.5 \times 12)^3 / (3 \times 3{,}156 \times 1{,}932{,}200) = 0.022"$

Example continued on next sheet

EXAMPLE 1: Hammerhead pier on spread/pile footing of varying height (cont'd.)

Sample calculation (continued):

$\Delta_{elast} + \Delta_{SS}$ = 0.600" + 0.022" = 0.622" < Δ_{temp} = 0.723"

Since the summation of Δ_{elast} and Δ_{SS} does not equal Δ_{temp}, **try Δ_{elast} = 0.698"**

$P_{elast} = L_{elast} \times W_{elast} \times G_{max} \times No_{pads} \times \Delta_{elast} / t_{elast}$ = 26 x 10 x 0.130 x 10 x 0.698 / 2.152 = 109.6 kips

$\Delta_{SS} = P_{elast} \times (H_{SS} \times 12)^3 / (3 \times E_c \times I_c)$ = 109.6 x (13.5 x 12)^3 / (3 x 3,156 x 1,932,200) = 0.025"

$\Delta_{elast} + \Delta_{SS}$ = 0.698" + 0.025" = 0.723" = Δ_{temp}

Perform a similar iterative calculation process for Pier 3. Directly compute P_{elast} for both abutments and P_{SS} for Pier 2.

Calculation table (thermal center assumed 8.5 feet left of Pier 2):

	L	L_{elast}	W_{elast}	t_{elast}	Δ_{elast}	P_{elast}	H_{ss}	P_{SS}	Δ_{SS}	Δ_{temp}
Abutment A	220.1	26	16	3.924	1.617	111.4	N/A	N/A	N/A	1.617
Pier 1	98.5	26	10	2.152	0.698	109.6	13.5	N/A	0.025	0.723
Pier 2	8.5	N/A	N/A	N/A	N/A	N/A	37.9	12.1	0.062	0.062
Pier 3	115.5	26	10	2.152	0.456	71.6	38.7	N/A	0.392	0.848
Abutment B	220.1	26	16	3.924	1.617	111.4	N/A	N/A	N/A	1.617

The summation of the shaded columns, Δ_{elast} and Δ_{SS}, must equal the total movement required, Δ_{temp}.

111.4 kips + 109.6 kips = **221.0 kips** > 12.1 kips + 71.6 kips + 111.4 kips = **195.1 kips**

The thermal center of the bridge is further from Pier 2 in the direction of Abutment A. Recalculate with the thermal center of the bridge 14.5 feet from Pier 2.

Calculation table (thermal center assumed 14.5 feet left of Pier 2):

Location	L	L_{elast}	W_{elast}	t_{elast}	Δ_{elast}	P_{elast}	H_{ss}	P_{SS}	Δ_{SS}	Δ_{temp}
Abutment A	214.1	26	16	3.924	1.573	108.4	N/A	N/A	N/A	1.573
Pier 1	92.5	26	10	2.152	0.651	102.2	13.5	N/A	0.024	0.679
Pier 2	14.5	N/A	N/A	N/A	N/A	N/A	37.9	20.7	0.106	0.106
Pier 3	121.5	26	10	2.152	0.480	75.3	38.7	N/A	0.412	0.892
Abutment B	226.1	26	16	3.924	1.661	114.4	N/A	N/A	N/A	1.661

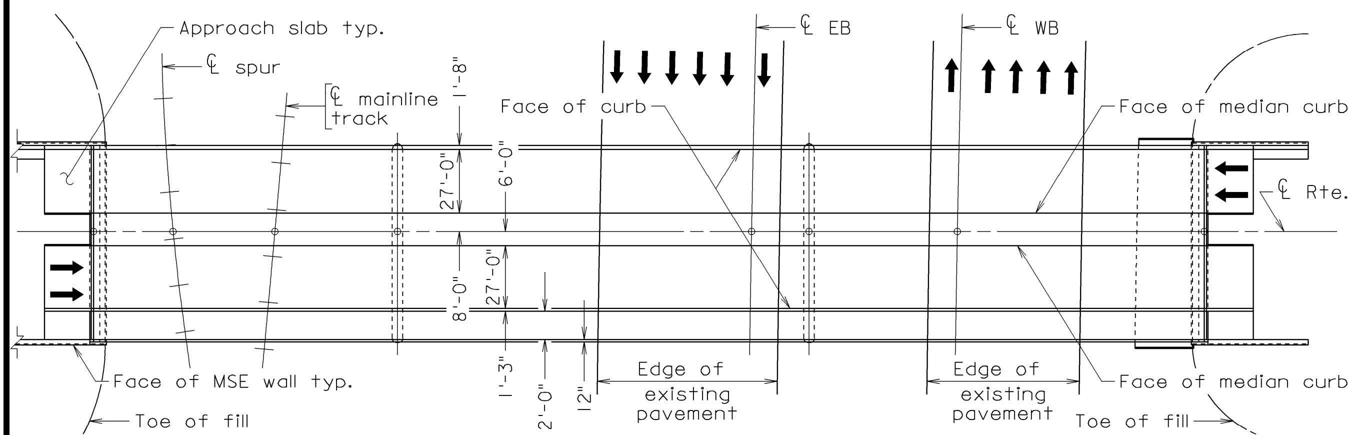
The summation of the shaded columns, Δ_{elast} and Δ_{SS}, must equal the total movement required, Δ_{temp}.

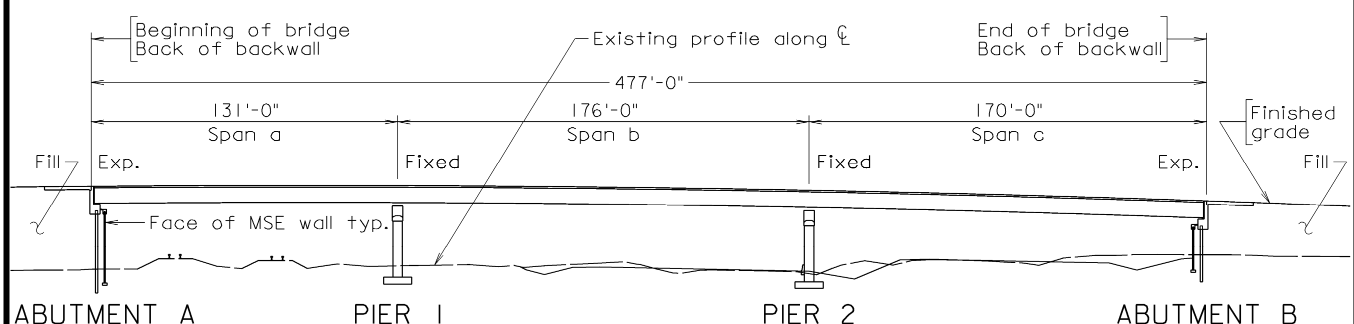
108.4 kips + 102.2 kips = **210.6 kips** ≈ 20.7 kips + 75.3 kips + 114.4 kips = **210.4 kips**

Note that the resulting temperature and shrinkage force at Pier 1 is 102.2 kips while 75.3 kips at Pier 3. Although Pier 2 is fixed, it is expected to deflect 0.106" to the right and the temperature and shrinkage force is 20.7 kips. Design requirements for all three piers will be relatively balanced as Pier 2 will be designed for a small temperature and shrinkage force, but all the longitudinal wind and braking force. Pier 1 has a larger temperature and shrinkage force, but shorter column length than Pier 3.

EXAMPLE 2: Multi-column pier on spread footings with unsymmetric spans

This example consists of a 3-span continuous steel bridge with joints at both ends. Elastomeric bearings will be used and dimensions shown in the calculation table are those from the final bearing designs. There is a longitudinal joint along the route centerline and the piers left and right of the joint are separate structures. The preliminary multi-column pier designs consist of two 4'-0" diameter columns left of the longitudinal joint and three 4'-0" diameter columns right. The superstructure typical section consists of four beam lines left of the longitudinal joint and five right. 4,000 psi concrete is used in this example as it was required for the column design. Skew = 0°.





Fixed bearings at both piers are used in this example. Fixing the bearings at only one pier is an option. However, different joint types were required at each abutment due to the unsymmetric expansion/contraction and applying all the longitudinal superstructure forces to one pier was found to require larger column sizes.

Sample calculations for the temperature loads are provided below and on the following sheet for the structure right of the longitudinal joint. Note that the structure left of the longitudinal joint should be modeled separately to determine forces.

Variables and equations:

∂ = coefficient of thermal expansion/contraction for steel = 0.0000065 per °F

ΔT = Temperature change, °F = 0.65 x 120 °F = 78 °F

Example continued on next sheet

EXAMPLE 2: Multi-column pier on footings with unsymmetric spans (cont'd.)

Variables and equations (continued):

L = Length from thermal center of bridge, feet

L_{elast} = Length of elastomeric bearing pad, inches

W_{elast} = Width of elastomeric bearing pad, inches

G_{max} = maximum elastomeric shearing resistance (range = 95 to 130 psi) = 0.130 ksi

No_{pads} = No. of elastomeric bearings (for PS beams continuous for LL, count both bearing lines) = 5

t_{elast} = Total thickness of internal and external elastomeric layers (total pad height – shims), inches

Δ_{elast} = Assumed/actual elastomeric deformation in direction of expansion/contraction, inches

P_{elast} = Force created internally in elastomeric pad for assumed/actual deformation, kips

$= L_{elast} \times W_{elast} \times G_{max} \times No_{pads} \times \Delta_{elast} / t_{elast}$

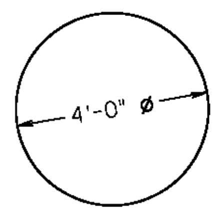
H_{ss} = Top of cap to top of footing (pier on pile or spread footing), feet

f'_c = Column (pile for pile bent) concrete strength = 4,000 psi

E_c = Modulus of elasticity for concrete, psi = $w_c^{1.5} \times 33 \times f'^{0.5}_c = 145^{1.5} \times 33 \times 4000^{0.5}$ = 3,644,000 psi

I_c = Uncracked moment of inertia for column(s) (piles for pile bent), in^4

$= 3 \times \pi \times (24")^4 / 4 = 781{,}700\ in^4$



P_{SS} = Force required to deflect substructure required distance at fixed pier/bent, kips

$= 3 \times E_c \times I_c \times \Delta_{temp} / (H_{SS} \times 12)^3$

Δ_{SS} = Substructure deflection in direction of exp./contraction, inches = $P_{elast} \times (H_{SS} \times 12)^3 / (3 \times E_c \times I_c)$

Δ_{temp} = Total movement required at pier/bent [steel superstructure, no shrinkage], inches

$= \partial \times L \times \Delta T \times 12$

Calculation table (Right of longitudinal joint): Thermal center determined to be 63.75' left of Pier 2

Location	L	L_{elast}	W_{elast}	t_{elast}	Δ_{elast}	P_{elast}	H_{ss}	P_{SS}	Δ_{SS}	Δ_{temp}
Abutment A	241.25	18	18	4.434	1.468	69.7	N/A	N/A	N/A	1.468
Pier 1	112.50	22	22	2.147	0.375	55.0	30.3	N/A	0.309	0.684
Pier 2	63.50	24	24	2.522	0.220	32.7	29.3	N/A	0.166	0.386
Abutment B	231.18	22	22	4.804	1.407	92.1	N/A	N/A	N/A	1.407

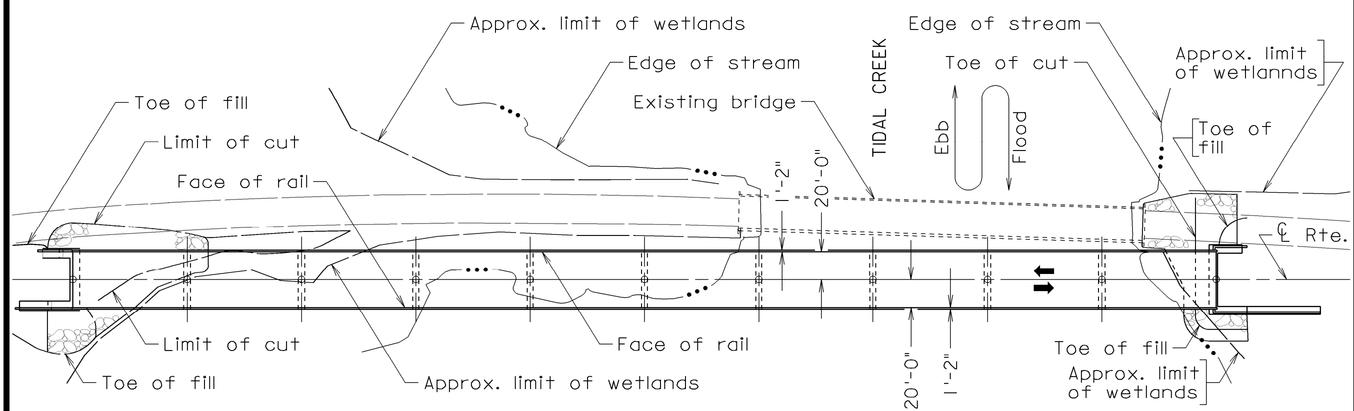
The summation of the shaded columns, Δ_{elast} and Δ_{SS}, must equal the total movement required, Δ_{temp}.

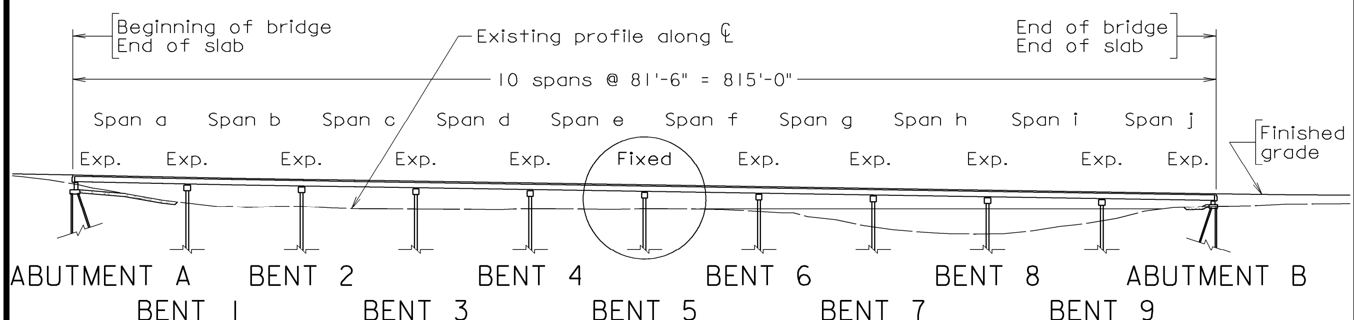
69.7 kips + 55.0 kips = **124.7 kips** ≈ 32.7 kips + 92.1 kips = **124.8 kips**

EXAMPLE 3: PILE BENT

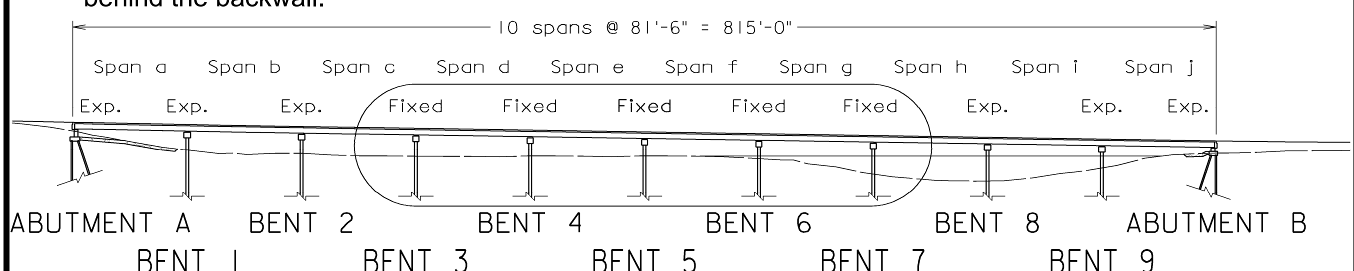
Pile bents and drilled shafts have an additional consideration when compared to columns on spread or pile footings. The geotechnical conditions at each location require modeling and the magnitude of the forces directly affects the location of the assumed point of fixity for the pile/shaft.

This example consists of a 10-span continuous prestressed Bulb-T bridge with semi-integral abutments at both ends. Elastomeric bearings will be used. Due to the length of the bridge elastomeric bearings with a PTFE sliding surface will be used at the abutments. The preliminary pile bent designs consist of 24” square prestressed concrete piles with seven piles per bent. Skew = 0°.





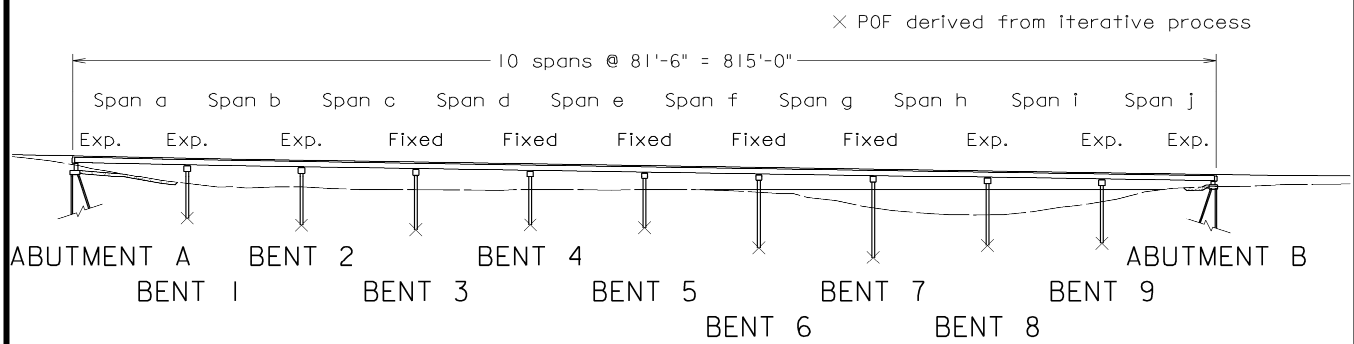
Applying all the longitudinal forces to Bent 5 will likely be infeasible. Fixing three piers may provide an efficient design. This example spreads the load further by fixing five piers as shown below. Check to ensure that the piles can handle the forces generated by forcing them to displace for temperature and shrinkage. Slip bearings exist at both abutments and should not be part of the unit modeling. Passive pressure will develop at both abutments, but can be neglected since EPS is used behind the backwall.



Example continued on next sheet

EXAMPLE 3: PILE BENT (cont'd.)

The assumed point of fixity (POF) is dependent on the geotechnical data at each bent location and is sensitive to the horizontal force exerted on the piles. In the sketch below, the points of fixity for the piles are derived by modeling the geotechnical data in the L-Pile compute program for the calculated forces. This is an iterative process in which POF's are assumed, forces calculated and then entered into L-Pile. The new POF's resulting from the L-Pile run are input back into the model and the process repeated until closure.



Sample calculations for the temperature loads in the scoured condition are provided below and continued on the next sheet for the sketch above.

Variables and equations:

∂ = coefficient of thermal expansion/contraction for normal weight concrete = 0.000006 per °F

ΔT = Temperature change, °F = 0.65 x 80 °F = 52 °F

L = Length from thermal center of bridge, feet

L_{elast} = Length of elastomeric bearing pad, inches

W_{elast} = Width of elastomeric bearing pad, inches

G_{max} = maximum elastomeric shearing resistance (range = 95 to 130 psi) = 0.130 ksi

No_{pads} = No. of elastomeric bearings (for PS beams continuous for LL, count both bearing lines) = 10

t_{elast} = Total thickness of internal and external elastomeric layers (total pad height – shims), inches

Δ_{elast} = Assumed/actual elastomeric deformation in direction of expansion/contraction, inches

P_{elast} = Force created internally in elastomeric pad for assumed/actual deformation, kips

$= L_{elast} \times W_{elast} \times G_{max} \times No_{pads} \times \Delta_{elast} / t_{elast}$

H_{ss} = Top of cap to assumed point of fixity for piles (pile bent), feet

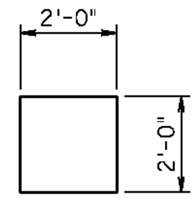
f'_c = Column (pile for pile bent) concrete strength = 5,000 psi

Example continued on next sheet

EXAMPLE 3: PILE BENT (cont'd.)

Variables and equations (continued):

E_c = Modulus of elasticity for concrete, psi = $w_c^{1.5}$ x 33 x $f'_c{}^{0.5}$ = $145^{1.5}$ x 33 x $5000^{0.5}$ = 4,074,000 psi



I_c = Uncracked moment of inertia for column(s) (piles for pile bent), in^4

= 7 piles x [24" x $(24")^3$ / 12] = 193, 536 in^4

P_{SS} = Force required to deflect substructure required distance at fixed pier/bent, kips

= 3 x E_c x I_c x Δ_{temp} / $(H_{SS} \times 12)^3$

Δ_{SS} = Substructure deflection in direction of expansion/contraction, inches

= P_{elast} x $(H_{SS} \times 12)^3$ / (3 x E_c x I_c)

Δ_{temp} = Total movement required at pier/bent including shrinkage [prestressed superstructure], inches

= (∂ x L x ΔT x 12) + (0.0003 x L x 12)

Calculation table (Scoured):

Location	L	L_{elast}	W_{elast}	t_{elast}	Δ_{elast}	P_{elast}	H_{ss}	PSS	Δ_{SS}	Δ_{temp}
Bent 1	277.0	25	19	5.279	0.422	49.3	35.5	N/A	1.612	2.034
Bent 2	195.5	22	15	4.043	0.255	27.1	39.1	N/A	1.181	1.436
Bent 3	114.0	N/A	N/A	N/A	N/A	N/A	41.1	16.5	0.837	0.837
Bent 4	32.5	N/A	N/A	N/A	N/A	N/A	36.3	6.8	0.239	0.239
Bent 5	49.0	N/A	N/A	N/A	N/A	N/A	37.2	9.6	0.360	0.360
Bent 6	130.5	N/A	N/A	N/A	N/A	N/A	50.3	10.3	0.958	0.958
Bent 7	212.0	N/A	N/A	N/A	N/A	N/A	56.0	12.1	1.557	1.557
Bent 8	293.5	22	15	4.043	0.252	26.8	46.0	N/A	1.903	2.155
Bent 9	375.0	25	19	5.279	0.349	40.8	43.2	N/A	2.405	2.754

The summation of the shaded columns, Δ_{elast} and Δ_{SS}, must equal the total movement required, Δ_{temp}.

Bent 5 must be designed for the unbalanced temperature and shrinkage force.

(49.3 + 27.1 + 16.5 + 6.8) kips = **99.7 kips** ≈ (9.6 + 10.3 + 12.1 + 26.8 + 40.8) kips = **99.6 kips**

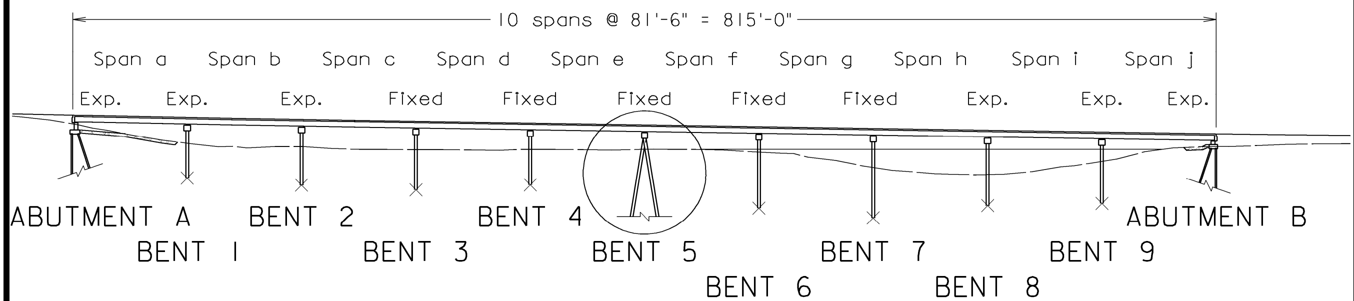
Note that the thermal center of the structure is 49 feet to the left of the actual midpoint of the bridge (Bent 5). The bearings were originally designed assuming expansion from the actual midpoint of the bridge. An argument may be made that the bearings at Bents 8 and 9 need to be re-designed so that $t_{elast} \geq$ 2 x the anticipated movement in one direction with the appropriate load factors for deformation. See current IIM-S&B-80. After re-design of the bearings, the temperature and shrinkage forces would need to be recalculated.

However, the original design did not account for flexibility of the substructure and since the deformation of the elastomeric pad is much less than originally designed for, the bearing designs are O.K. by inspection. Bearing re-design to take advantage of substructure flexibility is not recommended in this case as smaller bearings with less thickness would increase the already large forces on Bents 1 and 9.

Example continued on next sheet

EXAMPLE 3: PILE BENT (cont'd.):

The location of the thermal center of the bridge is of particular importance in this jointless bridge example. A design waiver was approved for this structure as it exceeds the selection criteria on File No. 20.01-1 for semi-integral bridges. To minimize any negative effects due to additional movement at Abutment B, the piles at Bent 5 were battered to ensure that the thermal center is as close to the actual center of the bridge as possible. The updated calculation table with Bent 5 as the thermal center is provided below.



Calculatation table (Scoured – Bent 5 battered)

Location	L	L_{elast}	W_{elast}	t_{elast}	Δ_{elast}	P_{elast}	H_{ss}	PSS	Δ_{SS}	Δ_{temp}
Bent 1	326.0	25	19	5.279	0.496	58.1	35.5	N/A	1.898	2.394
Bent 2	244.5	22	15	4.043	0.319	33.8	39.1	N/A	1.477	1.796
Bent 3	163.0	N/A	N/A	N/A	N/A	N/A	41.1	23.6	1.197	1.197
Bent 4	81.5	N/A	N/A	N/A	N/A	N/A	36.3	17.1	0.599	0.599
Bent 5	0.0	N/A	N/A	N/A	N/A	N/A	37.2	TBD	0	0
Bent 6	81.5	N/A	N/A	N/A	N/A	N/A	50.3	6.4	0.599	0.599
Bent 7	163.0	N/A	N/A	N/A	N/A	N/A	56.0	9.3	1.197	1.197
Bent 8	244.5	22	15	4.043	0.210	22.3	46.0	N/A	1.586	1.796
Bent 9	326.0	25	19	5.279	0.303	35.5	43.2	N/A	2.091	2.394

The summation of the shaded columns, Δ_{elast} and Δ_{SS}, must equal the total movement required, Δ_{temp}.

Bent 5 must be designed for the unbalanced temperature and shrinkage force.

(58.1 + 33.8 + 23.6 + 17.1) kips = **132.6 kips** > (6.4 + 9.3 + 22.3 + 35.5) kips = **73.5 kips**

Unbalanced force = 132.6 kips – 73.5 kips = 59.1 kips

The non-scoured condition must also be modeled to ensure that the reduced lengths to the assumed POF do not control design. The non-scoured results are not provided for this example.

GENERAL INFORMATION:

Piers having less than the horizontal clearance from the edge of a roadway or centerline of a railway track specified in AASHTO LRFD Article 3.6.5.2 shall be protected as per Article 3.6.5.1 or designed for the collision force.

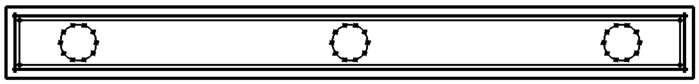
The information in File Nos. 15.05-2 thru -5 is provided to assist the designer in determining acceptable design concepts and procedures for piers with pile footings adjacent to a railway track with less than 50 feet horizontal clearance and not protected as per Article 3.6.5.1. The dimensional minimums provided incorporate crash wall requirements from the AREMA *Manual for Railway Engineering* and those set by individual railways. Spread footings may require a shear key to develop the lateral resistance to the collision force.

Piers with less than the horizontal clearance from the edge of a roadway and not protected as specified in Article 3.6.5.1 may use similar concepts as shown for railway track (e.g. multi-column/wall criteria, reinforcement design assumptions/layouts, combined footings, battered piles, etc.). Where vehicular traffic is protected from the piers with traffic barrier, a wall is not required between columns as long as the individual column(s) are independently capable of resisting the collision force. If a wall is used to connect the columns, it shall extend a minimum of 4'-6" above ground and a minimum of 12" below ground.

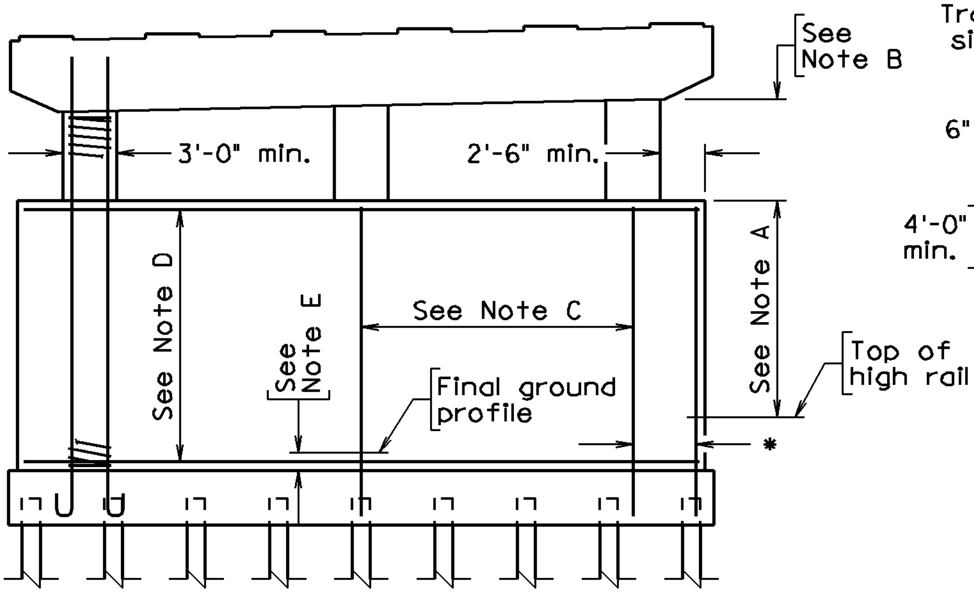
Pile bents shall not be used where highway vehicle or railroad collision forces are applicable.

Piers designed for the collision force may be larger and stiffer than other piers in a continuous unit and may attract additional loads. Careful attention shall be given to temperature and seismic modeling to obtain satisfactory overall bridge performance and economy for joint, bearing and substructure design.

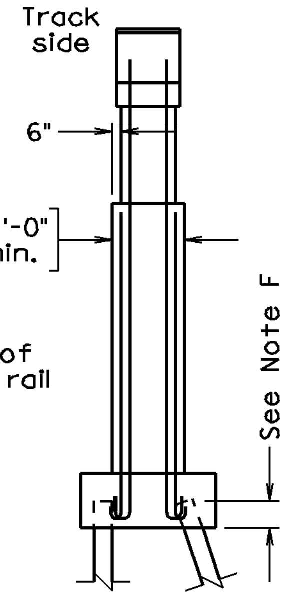
* Reinforcement beyond exterior column



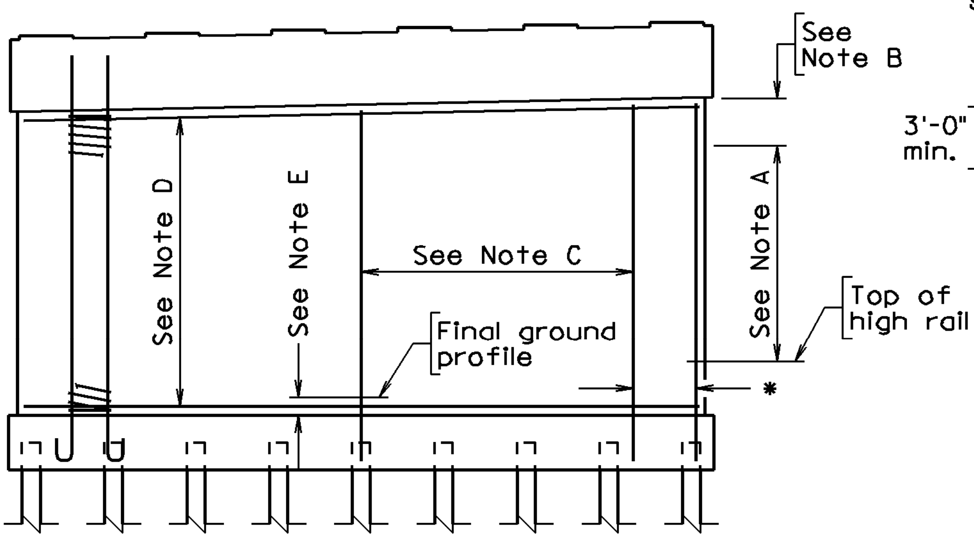
COLUMN / WALL SECTION



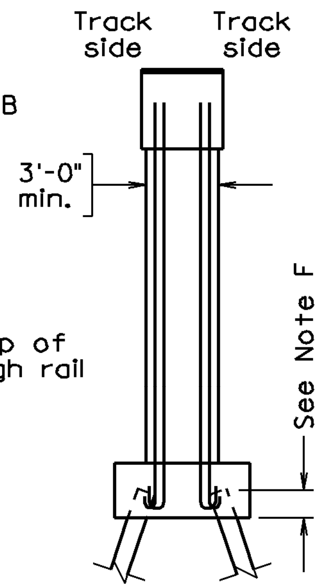
ELEVATION



END VIEW



ELEVATION



END VIEW

For notes, see next sheet.

PIER DETAILS
DESIGN / DETAILING FOR COLLISION FORCE
MULTI-COLUMN / WALL PIERS ADJACENT TO RAILWAY

VOL. V - PART 2
DATE: 17Dec2010
SHEET 2 of 5
FILE NO. 15.05-2

Notes:

A. The crash wall shall extend to not less than 6 feet (10 feet for Norfolk Southern Railway) above top of rail for piers 12 to 25 feet from the centerline and 12 feet above top of rail for piers less than 12 feet from the centerline.

B. Design as a multi-column pier with struts between columns. Where column extension from required top of wall to bottom of cap is less than 6 feet, extend crash wall full height. For either case, column reinforcement shall extend through the wall and be fully developed into the footing. Combined footings will likely be most economical, minimize bar sizes in the wall reinforcement and assist in load distribution.

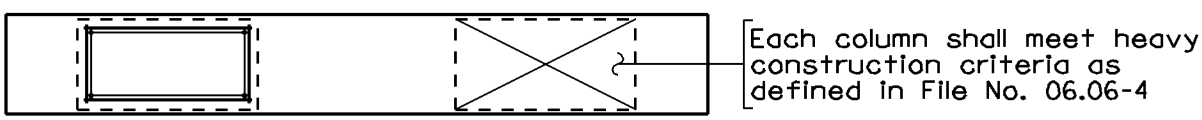
C. Provide vertical wall reinforcement sufficient to resist 75 percent of the collision force independently assuming the distance between column centers as the design section. Reinforcement shall be fully developed into the footing. Reinforcement beyond exterior columns shall provide an area of reinforcement per foot not less than between columns.

D. Provide horizontal wall reinforcement sufficient to resist 75 percent of the collision force independently assuming the full depth of wall as the design section. Horizontal reinforcement shall be fully developed past the column centers and identical on both sides of the wall. The maximum positive and negative moment in the wall can both be approximated as 0.125xPxL where P is 75 percent of the collision force and L is the column spacing.

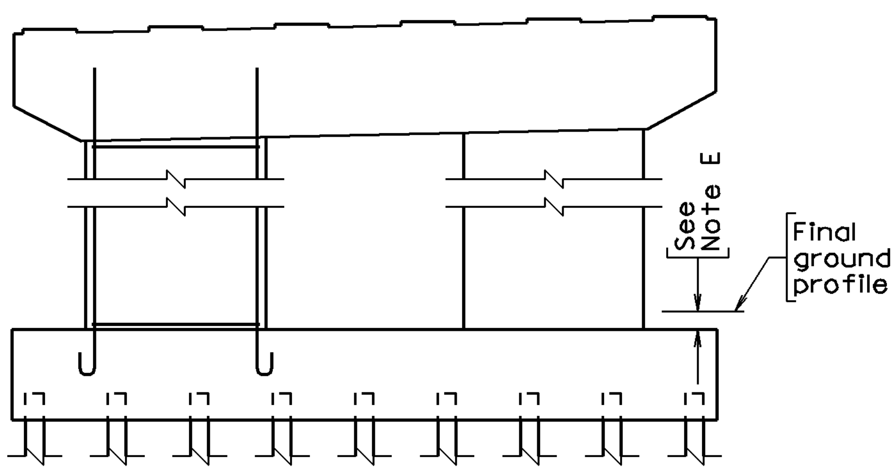
E. Top of footing shall be a minimum 12” below surrounding ground on track side.

F. Provide pile embedment for seismic design. Batter piles as required to resist the collision force assuming 4 kips horizontal resistance per vertical pile unless analysis supports a higher lateral capacity. Maximum pile batter is 4 : 12.

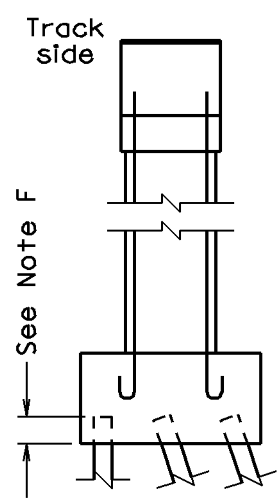
G. Multi-column piers with 25 feet, but less than 50 feet horizontal clearance from the centerline of track may be designed/detailed as described in Note B or without the wall between columns as shown below.



PLAN OF CAP



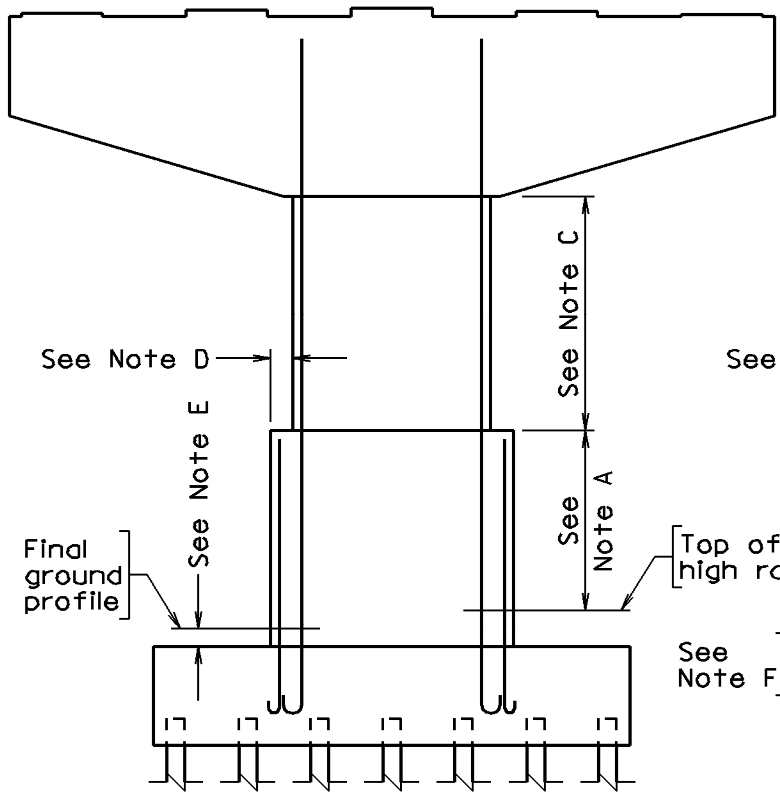
ELEVATION (See Note G)



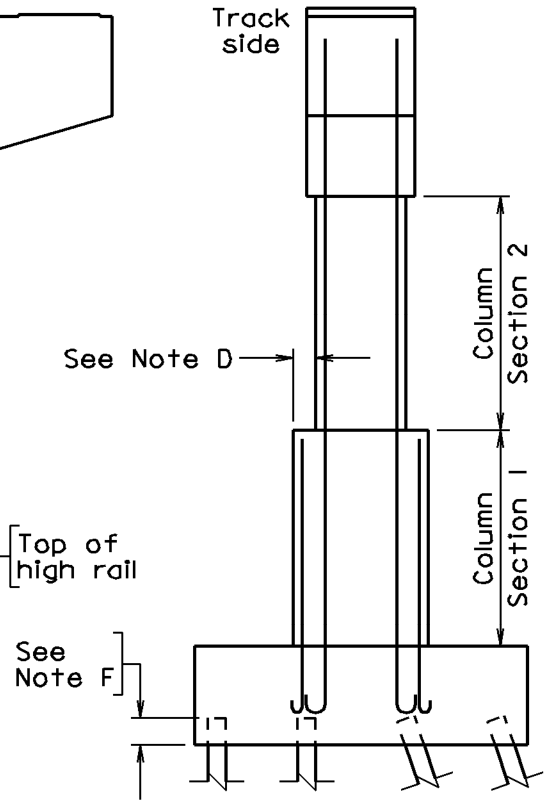
END VIEW

PIER DETAILS
DESIGN / DETAILING FOR COLLISION FORCE
MULTI-COLUMN / WALL PIERS ADJACENT TO RAILWAY

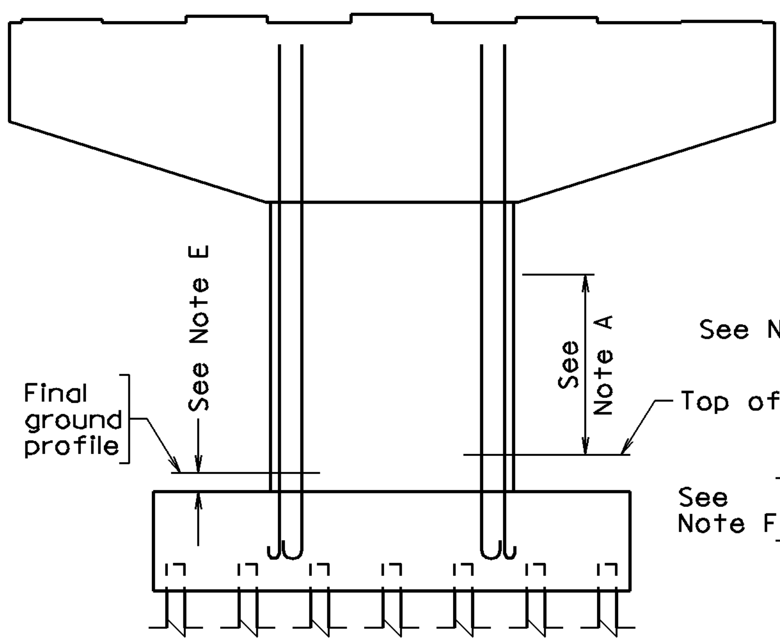
VOL. V - PART 2
DATE: 17Dec2010
SHEET 3 of 5
FILE NO. 15.05-3



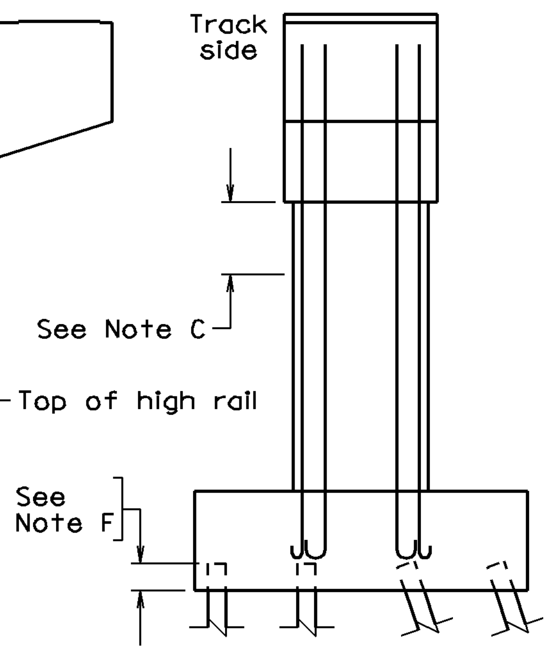
ELEVATION



END VIEW



ELEVATION



END VIEW

For notes, see next sheet.

PIER DETAILS
DESIGN / DETAILING FOR COLLISION FORCE
HAMMERHEAD PIERS ADJACENT TO RAILWAY

VOL. V - PART 2
DATE: 17Dec2010
SHEET 4 of 5
FILE NO. 15.05-4

Notes:

A. Hammerhead piers with less than 25 feet horizontal clearance from the centerline of a railway track shall be comprised of two column sections. The change in section shall occur not less than 6 feet (10 feet for Norfolk Southern Railway) above top of rail for piers 12 to 25 feet from the centerline and 12 feet above top of rail for piers less than 12 feet from the centerline.

B. The upper, smaller, column shall be sized and reinforced sufficient to carry all loads at the top of footing excluding the collision force. The column base shall be sized and reinforced sufficient to satisfy the Extreme Event II load combination limit state and shall meet the criteria for heavy construction. Reinforcing steel from both sections shall be developed into the footing.

C. When column extension from the required distance above top of rail to bottom of cap is less than 6 feet, a constant section may be used full height provided the reinforcement is in two layers as detailed in Note B.

D. Where two column sections are used, the column base shall extend a minimum of 6” beyond the upper column face in each direction.

E. Top of footing shall be a minimum 12” below surrounding ground on track side.

F. Provide pile embedment for seismic design. Batter piles as required to resist the collision force assuming 4 kips horizontal resistance per vertical pile unless analysis supports a higher lateral capacity. Maximum pile batter is 4 : 12.

G. Hammerhead piers with 25 feet, but less than 50 feet horizontal clearance from the centerline of a railway track may be designed/detailed as described in Note A or with a constant column section full height. For bridges with multiple piers, using two column sections may be beneficial to standardize cap dimensions/design since the upper column size may be more in line with design requirements at other pier locations with greater than 50 feet horizontal clearance to a track.

For details, see previous sheet.

GENERAL INFORMATION:

AASHTO LRFD specifications*, Article 11.7.2.1,* require "Where the possibility of collision exists from highway and river traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system."

The pier protection requirements below apply to all new construction. Pier protection shall be considered for all widenings.

The requirement for pier protection shall adhere to the following hierarchy:

1. For vehicular collision, locate pier ≥ 30'-0" from edge of roadway where feasible.
2. For vehicular collision, evaluate whether site conditions qualify for exemption from protection. See AASHTO LRFD specifications, Article C3.6.5.1.
3. Design pier for collision loads as specified in AASHTO LRFD specifications, Article 3.6.5 (vehicular collision) and 3.14 (vessel collision).
4. Use barrier protection (Manual of the Structure and Bridge Division, Volume V - Part 3, standards BPPS-1 thru -3) (vehicular collision) or an appropriate protection system (vessel collision).

Pier protection using barrier protection has the following requirements (AASHTO):

1. Crashworthy ground mounted, Test Level 5.
2. 54" high barrier when located within 10'-0" from component being protected.
3. 42" high barrier when located ≥ 10'-0" from component being protected.

The Structure and Bridge Division has developed standards to meet the barrier requirements noted above. See Manual of the Structure and Bridge Division, Volume V - Part 3. Standard BPPS-1 is to be used when the distance between the back edge of the barrier footing and the pier column/stem is less than 10'-0". Standard BPPS-3 is to be used when the distance between the back edge of the barrier footing and the pier column/stem is ≥ 10'-0". Standard BPPS-2 is required to be included with standard BPPS-1.

The face of the barrier curb shall align with the guardrail offset for the roadway classification where sufficient clearance is available. The minimum distance from the edge of the roadway (pavement) to the face of the barrier curb shall be 2'-0". Barrier shall be aligned parallel to the edge of the roadway.

The minimum distance from the edge of the roadway (pavement) to the face of a pier column or pier stem is 4'-0".

2'-0" minimum distance from edge of pavement to barrier curb
1'-8" barrier width at base
4" distance from barrier to back edge of footing
4'-0" = minimum distance from edge of roadway to face of pier column/stem

In this case the back edge of the barrier footing would be cast against the pier column/stem face. This would also require the pier footing to be sufficiently depressed to clear the barrier footing.

GENERAL INFORMATION (continued):

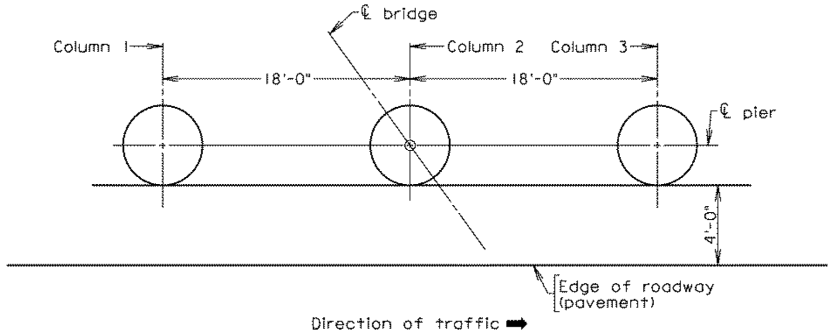
Where traffic is on both sides of the pier and at differing elevation, top of pier footing elevation shall be based on the lower roadway elevation.

NOTE: The 10’-0” clearance is set from the pier column/stem to the back of the barrier footing while the 30’-0” is set from the pier column/stem to the edge of the roadway.

BARRIER LAYOUT EXAMPLES:

EXAMPLE 1:

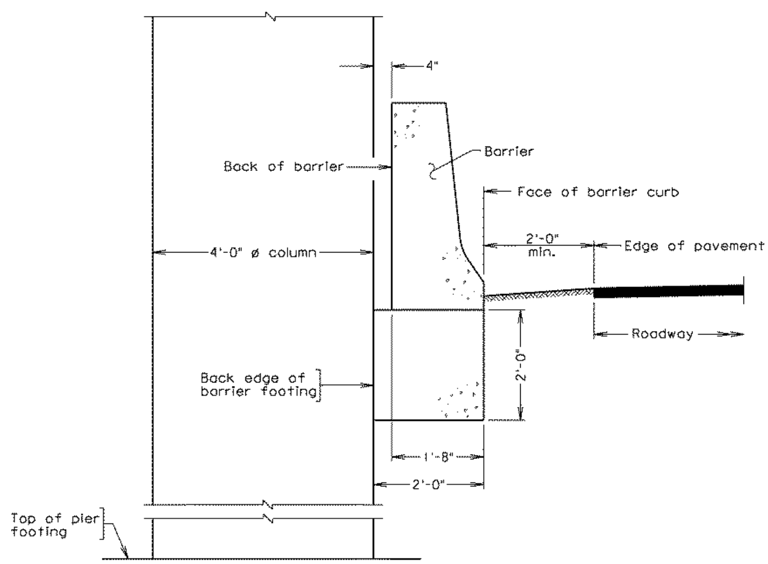
Traffic is on one side of the pier and the ℄ pier and ℄ roadway are parallel. The 3-column pier *consists* of 4'-0" diameter columns with 18'-0" column spacing, is not designed for collision and is not exempt. Clearances from the edge of roadway to pier column are as shown.



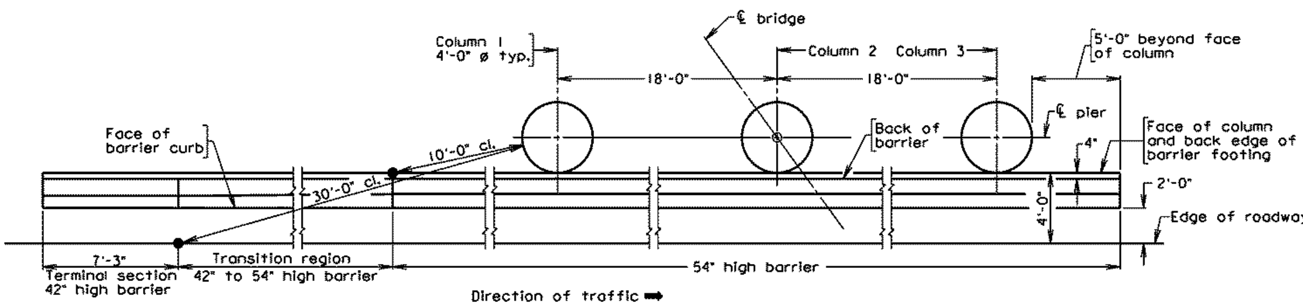
Barrier protection is required as the pier columns are < 30' from the edge of the roadway (pavement) and the pier is not designed for collision load. For the purposes of this example, the minimum 2'-0" distance from the edge of pavement to the face of barrier curb also coincides with the guardrail offset for the roadway classification involved. The width of barrier footing is 2'-0".

4.00' – (2.00' + 2.00') = 0.00' clearance to the back edge of the barrier footing.

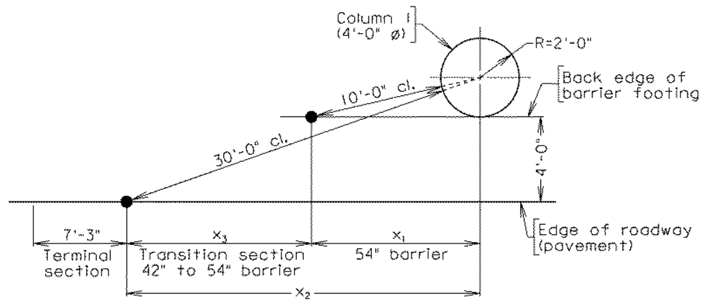
The distance from the face of the barrier curb to the pier columns is < 10'-0". Therefore a 54" barrier is required and Standards BPPS-1 and -2 will be used.



EXAMPLE 1 (continued):



Using Standard BPPS-1, determine at which point there is a 10'-0" clearance to the back edge of barrier footing in order to reduce the barrier height to 42" and where clearance is 30'-0" to the edge of roadway to terminate the 42" barrier. Add 7'-3" terminal section on run-on side for guardrail, etc. and extend barrier 5'-0" past face of stem at run-off end.



Round up barrier dimensions in 6" multiples.

$X_1 = \sqrt{(10.0\text{ft} + 2.0\text{ft})^2 - (0.0\text{ft} + 2.0\text{ft})^2}$
$= 11.83\text{ft}$ **Use:** 12'-0"

$X_2 = \sqrt{(30.0\text{ft}+2.0\text{ft})^2 - (4.0\text{ft}+2.0\text{ft})^2}$
$= 31.43\text{ft}$ **Use:** 31'-6"

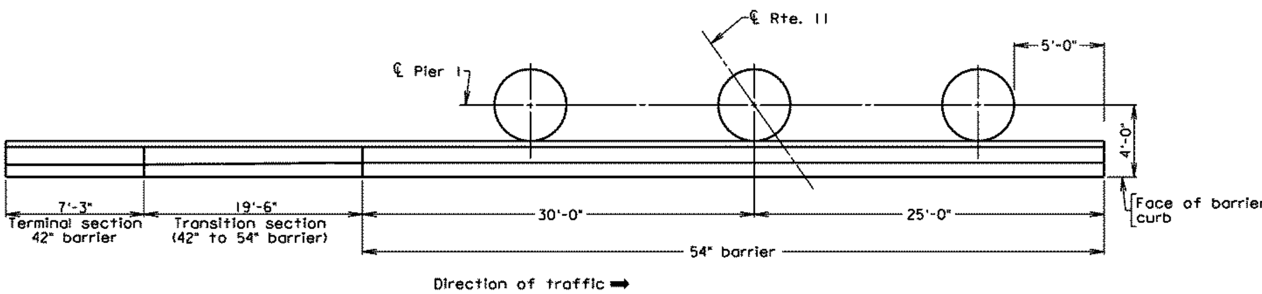
$X_3 = X_2 - X_1$
= (31'-6") - (12'-0")
= 19'-6" **Use:** 19'-6"

EXAMPLE 1 (continued):

Calculate required barrier dimensions and add LAYOUT OF BARRIER FOR PIER PROTECTION to standard sheet. Show terminal section and lengths for 42" and 54" high barrier as well as 42" to 54" transition section. Add lengths to PLAN and ELEVATION on standard sheet.

L_1 = X_1 + c-c spacing of column
= (12'-0") + (18'-0")
= 30'-0"

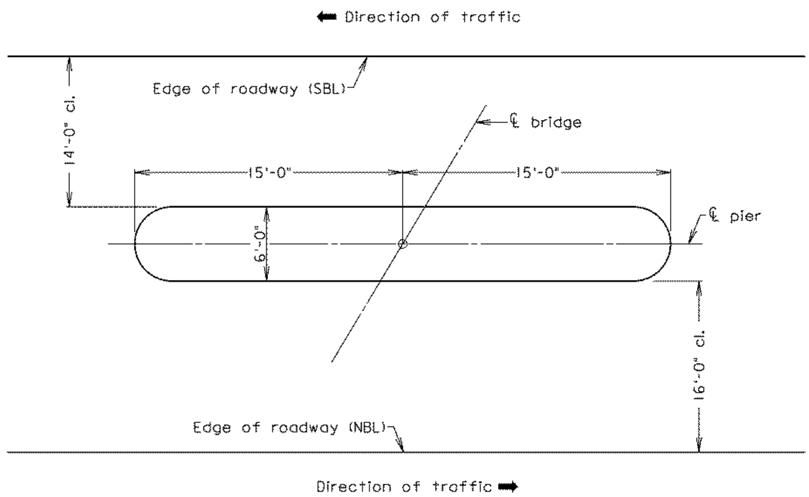
L_2 = c-c spacing of column + column ø/2 + 5.0ft
= 18.0ft + 2.0ft + 5.0ft
= 25'-0"



LAYOUT OF BARRIER FOR PIER PROTECTION
Scale: $^1/_8$" = 1'-0"

EXAMPLE 2:

Traffic is on <u>both sides</u> of the pier and the ℄ pier and ℄ roadway are parallel. The pier stem is 30'-0" long and 6'-0" wide with rounded ends, is not designed for collision and is not exempt. Clearances from the edge of roadway to the pier stem are as shown.



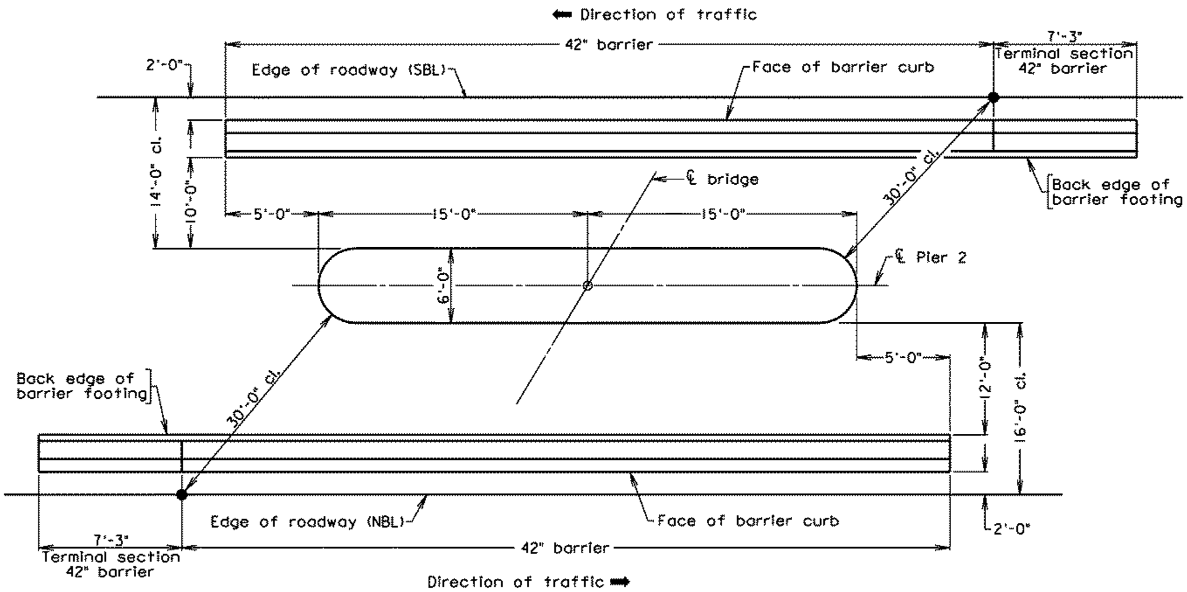
Barrier protection is required on both sides of the pier as the pier stem is < 30' from the edge of the roadways (pavements) and the pier is not designed for collision load. For the purposes of this example, the minimum 2'-0" distance from the edge of pavement to the face of barrier curb also coincides with the guardrail offset for the roadway classification involved. The width of barrier footing is 2'-0".

SBL: 14.00' – (2.00' + 2.00') = 10.00' clearance to the back edge of the barrier footing

NBL: 16.00' – (2.00' + 2.00') = 12.00' clearance to the back edge of the barrier footing

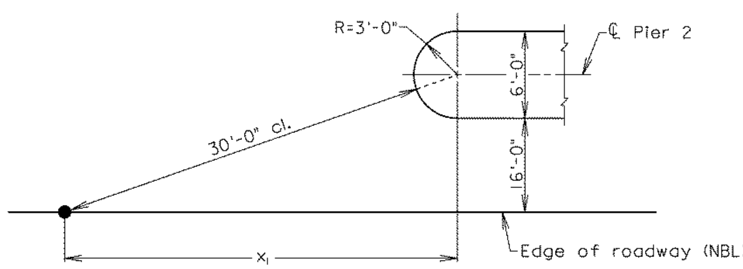
The distance from the back edge of the barrier footing to the pier stem is ≥ 10'-0" for both the SBL and NBL. Therefore a 42" barrier is sufficient for both and Standard BPPS-3 will be used.

EXAMPLE 2 (continued):



Using standard BPPS-3, calculate the distances at which the pier stem has a clearance of 30'-0" to the edge of roadways to terminate the barrier. Add 7'-3" terminal section on run-on side for guardrail, etc. and extend barrier 5'-0" past face of stem at run-off end.

NBL side:

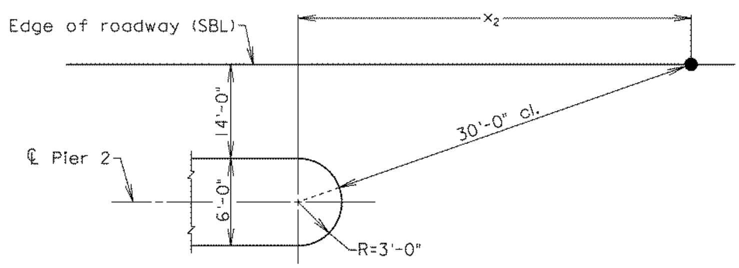


$X_1 = \sqrt{(30.0\text{ft} + 3.0\text{ft})^2 - (16.0\text{ft} + 3.0\text{ft})^2}$

$= 26.98$ **Use:** 27'-0"

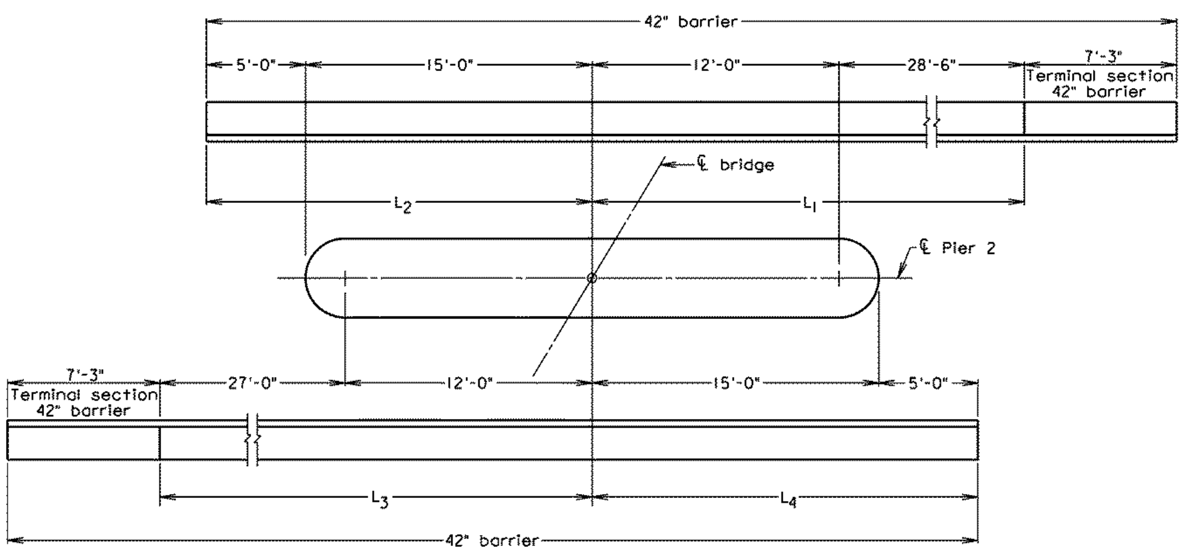
EXAMPLE 2 (continued):

SBL side:



$X_2 = \sqrt{(30.0\text{ft} + 3.0\text{ft})^2 - (14.0\text{ft} + 3.0\text{ft})^2}$
= 28.28ft **Use:** 28'-6"

Calculate required barrier dimensions and add LAYOUT OF BARRIER FOR PIER PROTECTION to standard sheet. Show terminal section and lengths for 42" high barrier. Add lengths to PLAN and ELEVATION on standard sheet.



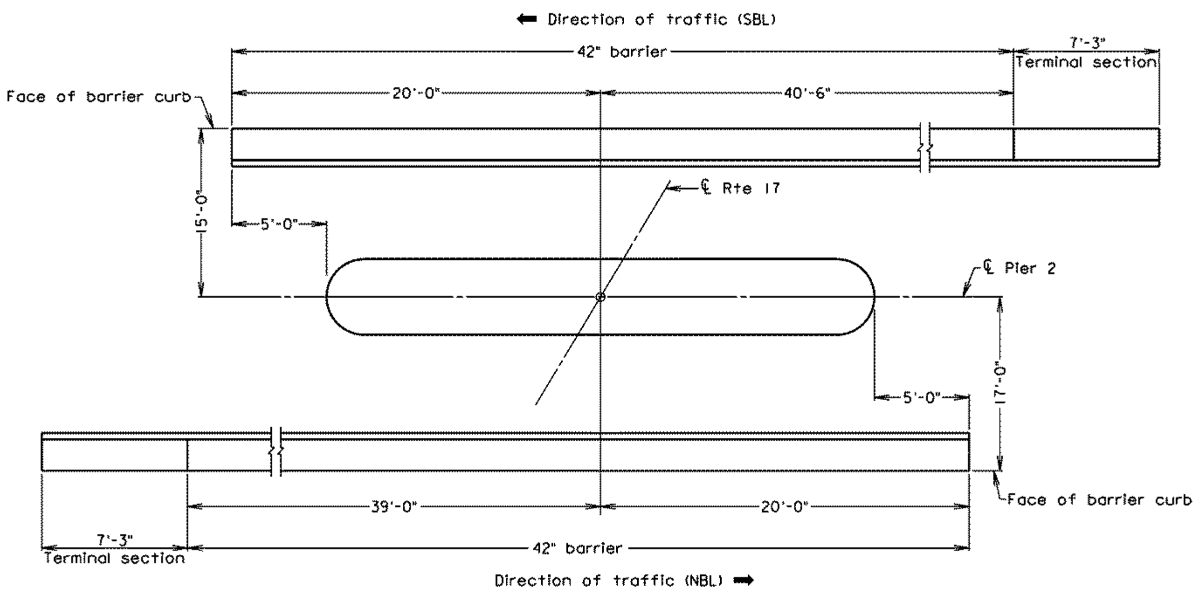
L_1 = 12'-0" + 28'-6"
= 40'-6"

L_2 = 15'-0" + 5'-0"
= 20'-0"

EXAMPLE 2 (continued):

L_3 = 27'-0" + 12'-0"
= 39'-0"

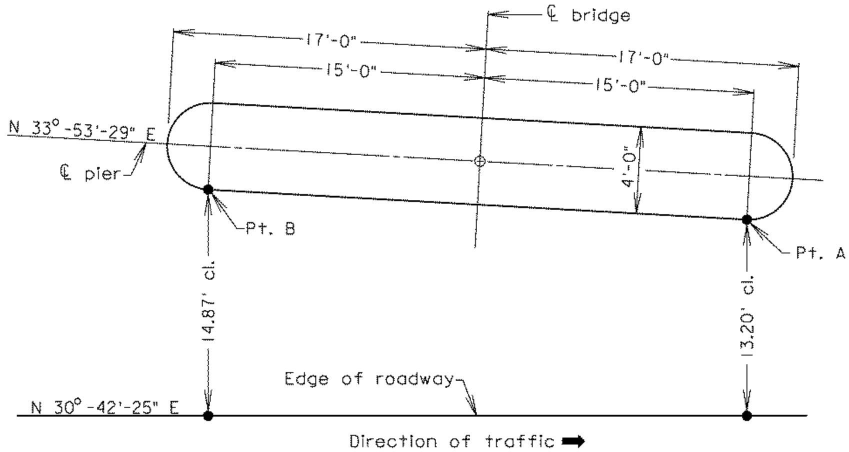
L_4 = 15'-0" + 5'-0"
= 20'-0"



LAYOUT OF BARRIER FOR PIER PROTECTION
Scale: $^1/_8$" = 1'-0"

EXAMPLE 3:

Traffic is on one side of the pier and the ℄ pier and ℄ roadway are <u>not parallel</u>. The pier stem is 30'-0" long and 4'-0" wide with rounded ends, is not designed for collision and is not exempt. Clearances from the edge of roadway to pier stem are as shown.



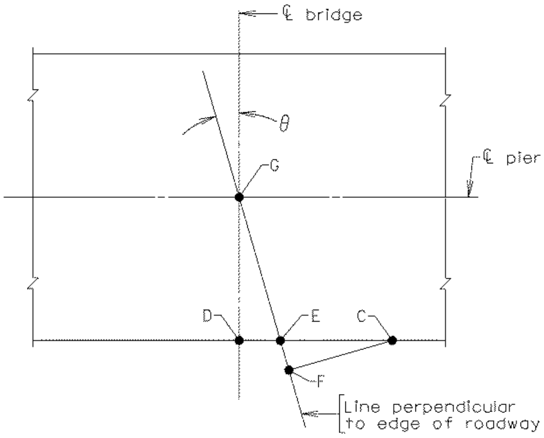
Barrier protection is required as the pier stem is < 30'-0" from the edge of the roadway (pavement) and the pier is not designed for collision load. For the purposes of this example, the minimum 2'-0" distance from the edge of pavement to the face of barrier curb also coincides with the guardrail offset for the roadway classification involved. The width of barrier footing is 2'-0".

At Point A: 13.20' – (2.00' + 2.00') = 9.20' clearance to the back edge of the barrier footing.

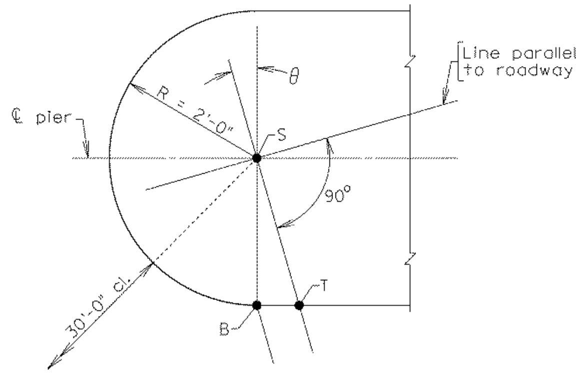
At Point B: 14.87' – (2.00' + 2.00') = 10.87' clearance to the back edge of the barrier footing.

The distance from the face of the barrier curb to the pier stem at Point A < 10'-0". Therefore, a 54" barrier is required and Standards BPPS-1 and -2 will be used. At Point B, the distance is ≥ 10'-0" in which case a 42" barrier is sufficient.

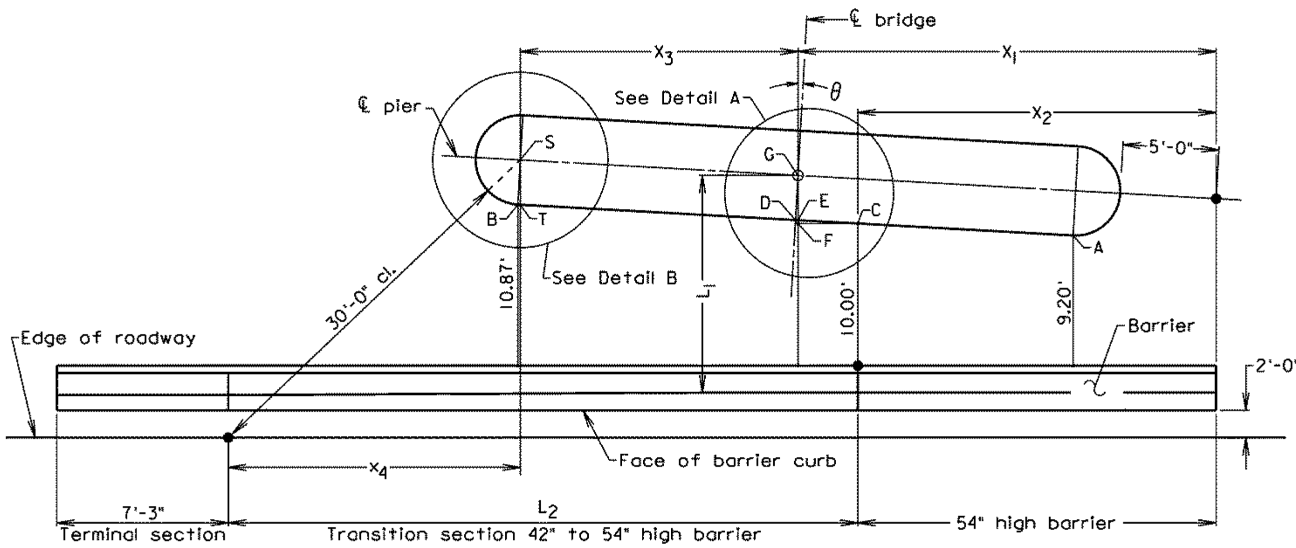
EXAMPLE 3 (continued):



DETAIL A



DETAIL B



Determine location of point where there is 10'-0" clearance to the back edge of barrier footing. This is needed in order to set the point at which the barrier can be transitioned from 54" to 42" height.

θ = (33°-53'-29") – (30°-42'-25") = 3°-11'-04" = angle difference between pier centerline and edge of roadway.

L_{AB} = 30.00ft

L_{AC} = [(10.00-9.20)/(10.87-9.20)] (30.00)
= 14.37ft

EXAMPLE 3 (continued):

L_{CD} = 15.00 – 14.37
= 0.63ft

L_{DE} = 2.00 (tanθ)
= 0.11ft

$L_{CE} = L_{CD} - L_{DE}$
= 0.63 – 0.11
= 0.52ft

L_{EF} = 0.52 (sinθ)
= 0.029ft

L_{EG} = 2.00/(cosθ)
= 2.00ft

Calculate dimensions from tie point (centerline of bridge (route, etc.) and centerline pier) to face of barrier curb.

$L_1 = L_{EG} + 10.00 + L_{EF} + 2.00$
= 2.00 + 10.00 + 0.029 + 2.00
= 14.029ft **Use:** 14'-0$^3/_8$"

L_{CF} = 0.52 (cosθ)
= 0.52ft **Use:** 6"

Extend barrier 5'-0" past end of column

X_1 = (15.00 + 2.00 + 5.00) (cosθ)
= 21.97

X_2 = 21.97 – 0.52
= 21.45ft **Use**: 21'-6"

Calculate the distance at which the pier stem has a clearance of 30'-0" to the edge of the roadway.

X_3 = 15.00 (cosθ)
= 14.98ft

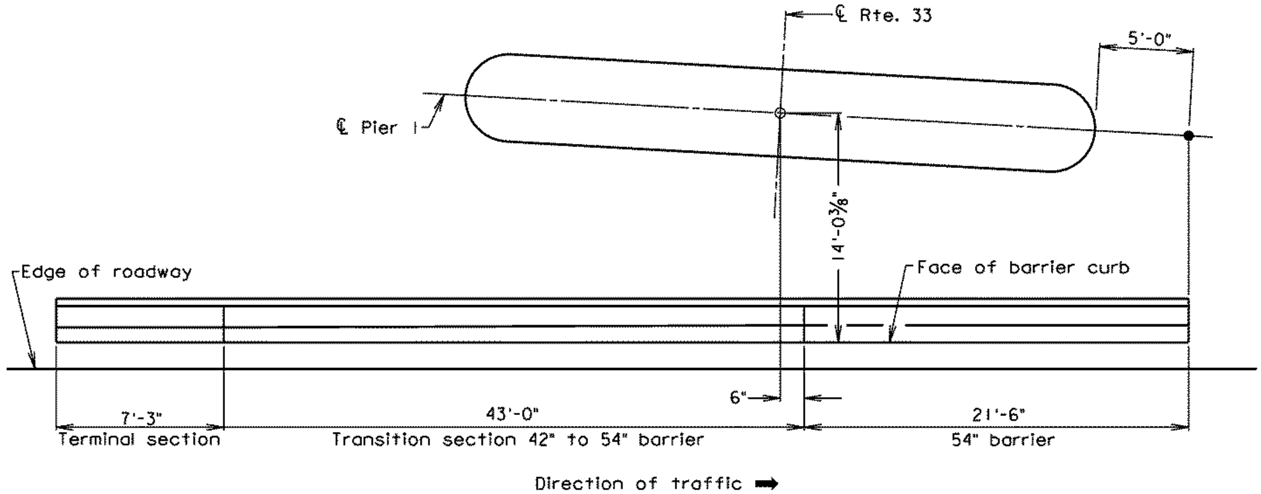
Line BT is perpendicular to SB.

ST = 2.00/(cosθ)
= 2.00ft

$X_4 = \sqrt{(30.00\text{ft} + \text{R})2 \quad - (\text{ST} + 10.87 + 2.00 + 2.00)^2}$
= 27.19ft

EXAMPLE 3 (continued):

$L_2 = L_{CF} + X_3 + X_4$
$= 0.52 + 14.98 + 27.19$
$= 42.69$ft **Use:** 43'-0"



LAYOUT OF BARRIER FOR PIER PROTECTION

Scale: $^1/_8$" = 1'-0"