Precast Bent System for High Seismic Regions

Final Report, Appendix C: Design Example No. 2

Publication No. FHWA-HIF-13-037-C

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HIGHWAYS FOR LIFE

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16. Abstract

The final report for this project provides information on the system concept, as well as the background physical testing for the upper level grouted duct connections and the lower socket type connections. Additionally, the report provides design and construction specifications, describes a demonstration project constructed in Washington State, and discusses lessons learned from the demonstration project. This report is a technical resource that provides background information on the use of a precast bent system for use in high seismic regions. The system is designed for, and intended to be used with, prestressed girder bridges that are built integrally with the supporting intermediate piers.

Appendix C is the second of two design examples that were prepared to guide engineers through the application of the AASHTO Seismic Guide Specifications to the Highways for LIFE precast bridge bent system, along with the supplemental design provisions that have been proposed as an appendix to the AASHTO Seismic Guide Specifications. Particular emphasis is given to the design and detailing of the connections between precast elements to ensure ductile energy dissipating behavior within the plastic hinge regions of the columns and capacity protection throughout the remainder of the structure. This design example uses an actual bridge recently designed by WSDOT, the US 101 Bone River Bridge replacement near Raymond. The new bridge will be a three-span structure, with the two-column intermediate bents and the short-stem seat abutments each supported by two oversized pile shaft foundations. The superstructure will consist of four prestressed concrete wide flange girders that will be made continuous at the integral dropped cap beam at the intermediate piers. The precast concrete columns were connected to the pile shaft foundations using a socket connection similar to the large-scale laboratory tests described in the final project report.

There are two additional appendixes to the final report, published as stand-alone documents, as well as two companion reports that cover the detailed testing and modeling of the spread footing and drilled shaft foundation versions of the precast column-to-foundation connection:

- Appendix A: Design Provisions (FHWA-HIF-13-037-A)
- Appendix B: Design Example No. 1 (FHWA-HIF-13-037-B)
- Laboratory Tests of Column-to-Drilled Shaft Socket Connections (FHWA-HIF-13-038)
- Laboratory Tests of Column-to-Footing Socket Connections (FHWA-HIF-13-039)

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CHAPTER 1. INTRODUTION

PURPOSE

This is the second of a set of two seismic design examples created for this project. A different bridge is used in each design example. The goal of the design examples is to cover the features that must be addressed in the seismic design process for a fully precast bridge system. Table 1 is a matrix of the features in this seismic design example.

Table 1. Design example matrix.

Description	Three-span continuous
Plan Geometry	Straight. No skew
Superstructure Type	Prestressed precast concrete girder
Pier Type	Precast two-column, dropped cap bent
Abutment Type	Short abutment with overhanging end diaphragm
Foundation Type	Pile shaft
Connections and Joints	Socket connection to shaft, integral at intermediate piers, elastomeric bearing pads at abutments

APPLICABLE SPECIFICATIONS

The design examples conform to the following specifications:

- American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for LRFD [Load and Resistance Factor Design] Seismic Bridge Design, Second Edition (herein called "Seismic Guide Specifications").(1)
- AASHTO LRFD Bridge Design Specifications, Fifth Edition (herein called "Bridge Design Specifications").(2)
- Washington State Department of Transportation, Bridge Design Manual, M23-50.04, 2010 (herein called "WSDOT BDM").(3)

Additionally, the design of the precast column and its connections to the foundation and cap beam and the seismic design of the cap beam are supplemented by draft design specifications contained in appendix A of this report, in addition to the University of Washington research reports supporting this Highways for LIFE project.(4,5)

EMPHASIS

This design example follows the procedures of the Seismic Guide Specifications with a special focus on the connection from a precast reinforced concrete column to a cast-in-place oversized pile shaft for a fully precast integral bent system for high seismic areas. All other design considerations are not explicitly addressed here, as they were addressed in the first design example (appendix B).

BRIDGE DESCRIPTION

The Bone River bridge replacement is located south of the city of Raymond, Washington, on Highway 101. Structural details are described below and shown in figures 1 through 4.

Bridge Length/Span

The bridge has three spans and is 334 feet long from back to back of pavement seats at the abutments. The end spans are 97 feet long, and the central span in 140 feet long.

Curvature

The bridge is straight. No horizontal curvature exists.

Roadway Width

The roadway is 36 feet wide, measured from curb line to curb line.

Pier Skew

All piers are perpendicular to the bridge centerline.

Superstructure

The superstructure is made up of four 6-foot 2-inch deep prestressed precast wide flange girders (WSDOT Series WF74G) spaced at 9 feet 6 inches on center. The girders are made integral with the substructure using a full depth cast-in-place diaphragm. Cast-in-place intermediate diaphragms spanned between the girders at three locations in the central span and two locations at the end spans. The roadway is a 7 1/2-inch-thick cast-in-place slab with a total width of 37 feet 9 inches.

Substructure

The substructure consists of an abutment at each end and two precast concrete intermediate piers, which were made up of a 5-foot-deep by 7-foot-wide dropped precast concrete cap beam supported by two square precast concrete columns. The full cap beams consist of a precast lower stage integral with the full depth cast-in-place diaphragm above. The columns are 5-foot-wide square sections. To ensure symmetric performance under biaxial loading, the bottom 2 inches of the columns are reduced to a 5-foot octagonal section, and the top 3 inches are reduced to a 5-foot-diameter circular section.

The height of the columns at the intermediate pier locations is 17 feet 1 inch, measured from the top of the pile shaft to the soffit of the dropped cap beam.

Foundations

Each column at the intermediate piers is supported by a 10-foot-diameter cast-in-place pile shaft. Each cast-in-place abutment is supported by two cast-in-place pile shafts. The pile shafts for the columns and the abutments vary in length from 40 to 90 feet, depending on the location.

Connections

The ends of the precast girders are integrally connected to the diaphragm at the intermediate piers. The abutments provide full restraint in the direction transverse to the bridge centerline and no restraint in the direction parallel to the bridge centerline.

Materials

The concrete has a nominal compressive strength of 4,000 psi for the precast and cast-in-place elements. The prestressed girders use 8,500 psi concrete with a minimum of 7,000 psi at release. Mild steel reinforcement is ASTM A706 Grade 60, and prestressing strands are ASTM A416 (AASHTO M203) Grade 270.

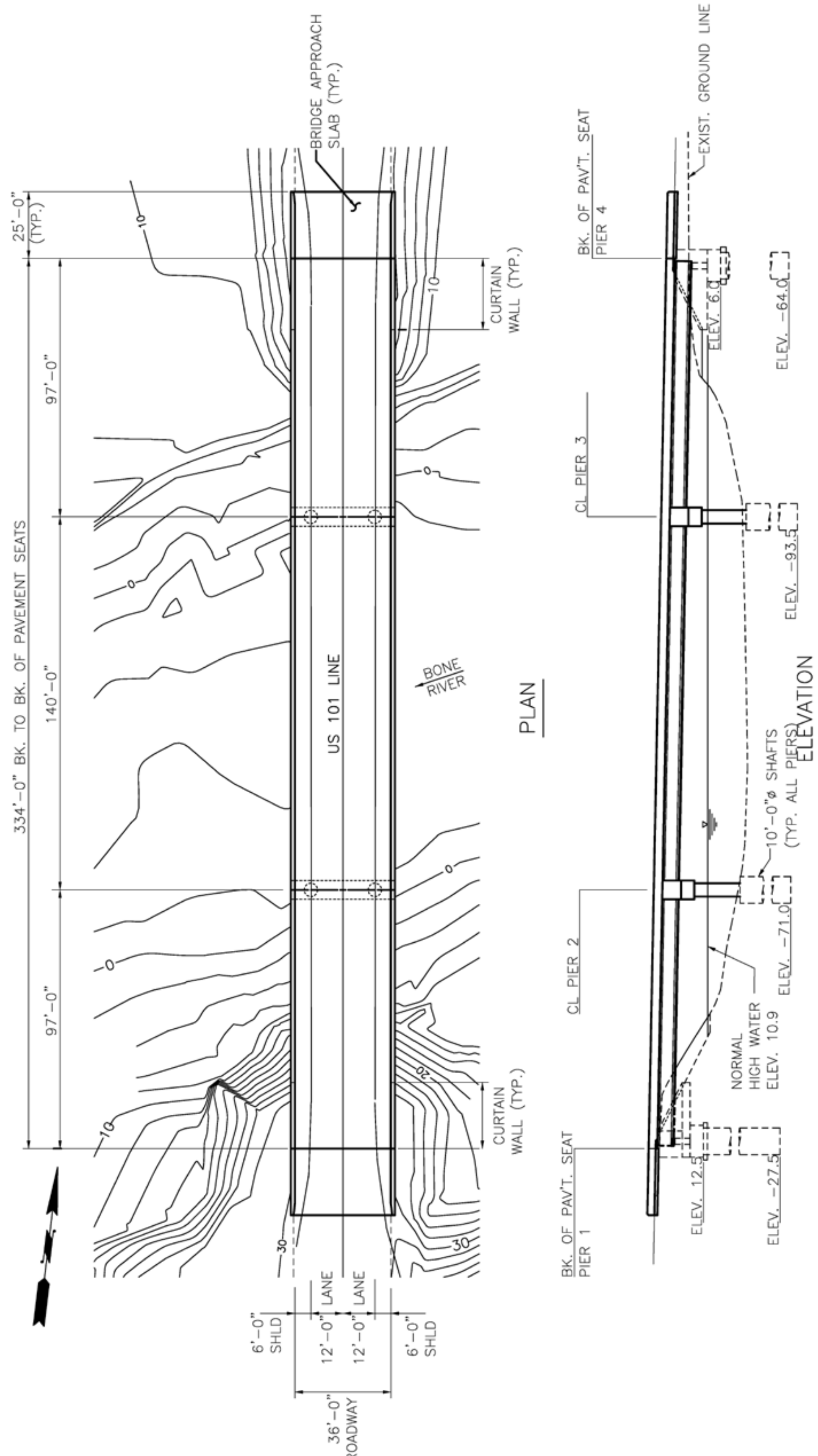

Figure 1. Diagram. Bridge plan and elevation.

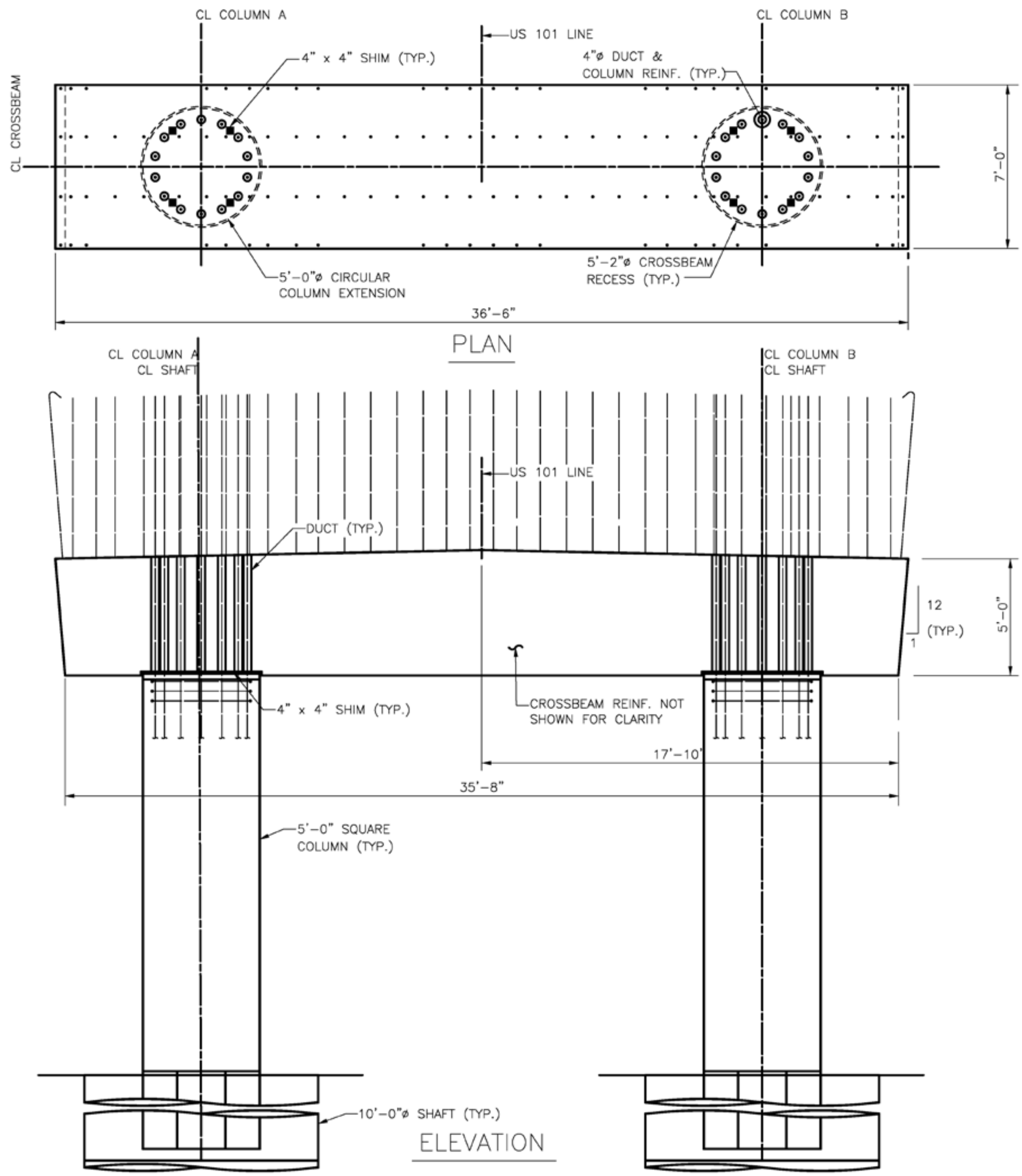

Figure 2. Diagram. Typical precast pier and oversized shaft.

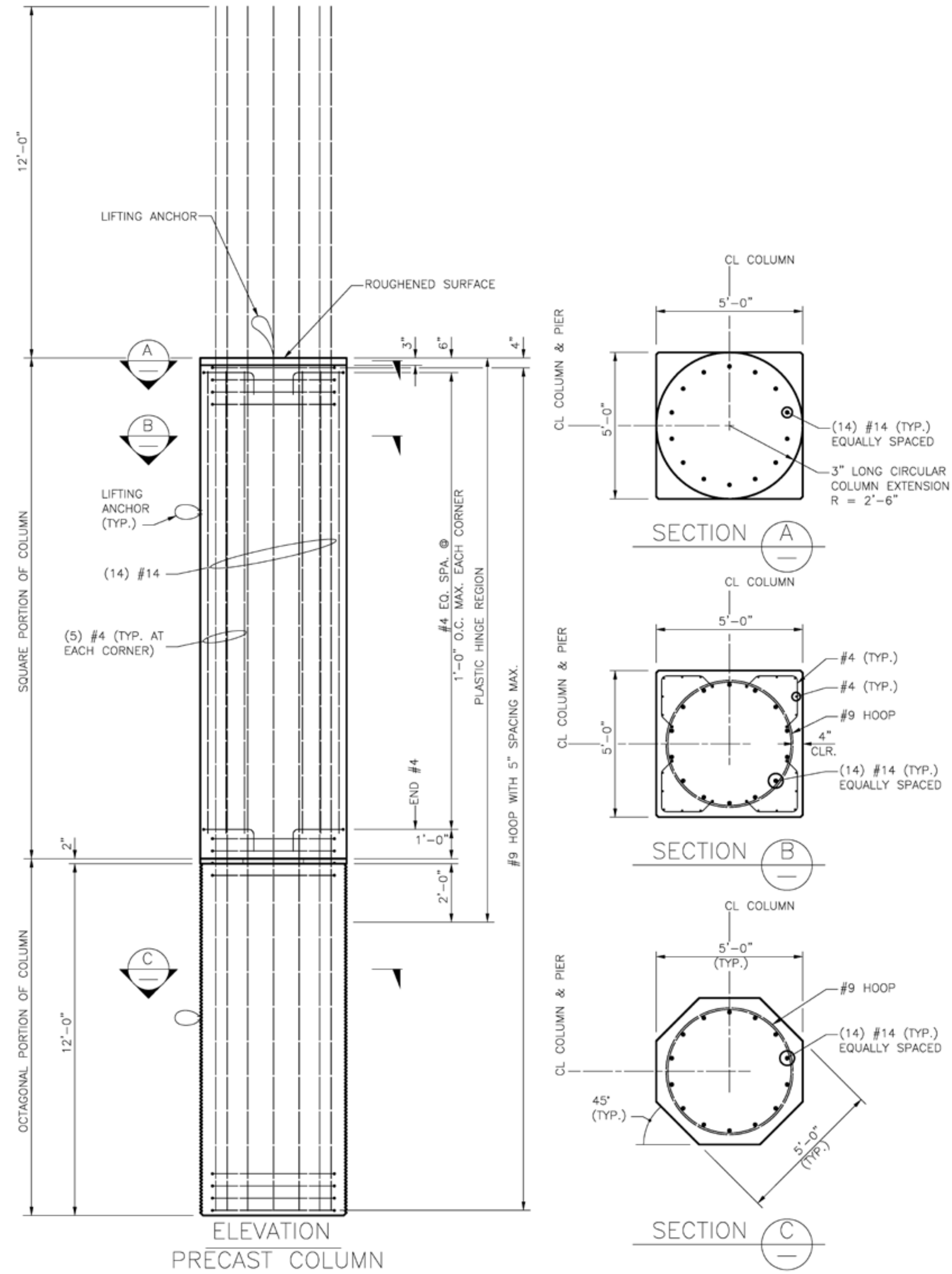

Figure 3. Diagram. Typical precast column details.

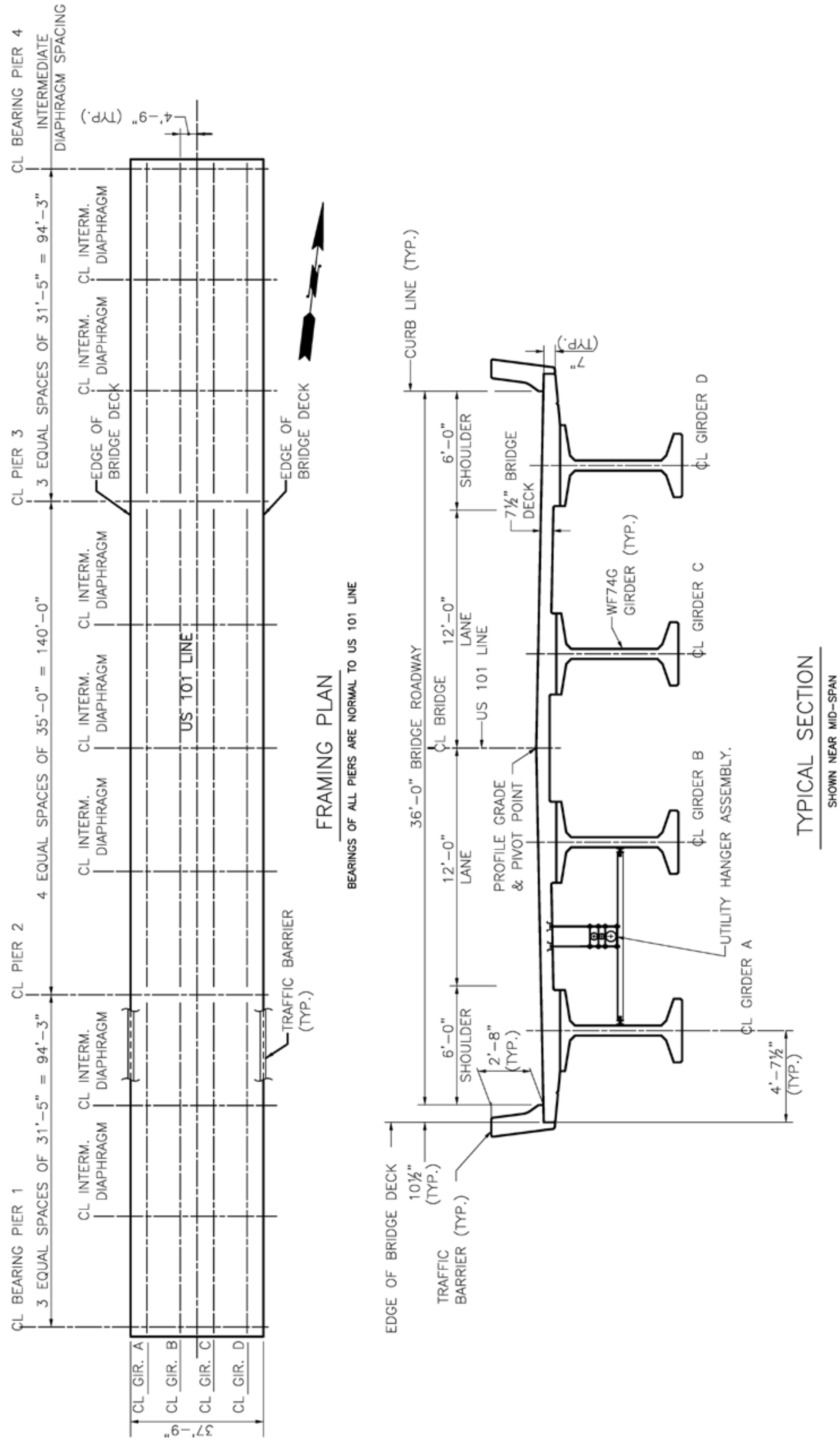

Figure 4. Diagram. Typical section of bridge.

CHAPTER 2. DEMAND ANALYSIS

To determine the seismic demands on the bridge, a sufficiently accurate analysis model must be generated. The development of such a model includes the calculation of member properties, geometry, boundary conditions, foundation stiffnesses, seismic mass/weight, input spectra, and internal releases. The column plastic hinge interaction surfaces and curvature limitations need to be defined and entered into the analysis model or need to be considered by hand analysis. A gravity dead load check should be made by hand calculations against the seismic model output. The model should also be checked to ensure that it has adequate mass participation (at least 90 percent mass participation in both the transverse and longitudinal directions) and produces the anticipated directional base shear based upon the period of the structure in the direction under consideration. In this case, the analytical model consists of centerline beam-column elements for the columns and cap beams, while the superstructure is a grillage representation. Rigid offsets are used to connect the centerline elements.

Figure 5 shows a 3-dimensional rendering of the bridge analytical model. The ground line is at the top of the oversized pile shaft section. In the actual analysis model, the pile shafts are represented by linear rotational and translational springs at the bottom of the column, but they are shown in figure 5 to show their dimensions in relation to the rest of the bridge. It can also be seen that the 5-foot-diameter columns have a relatively low aspect ratio, indicating that there will be high plastic shear demands; therefore, it is necessary to watch out for shear critical behavior. The abutments (which are not visible in figure 5) are treated as linear springs that prevent translation in the vertical and transverse directions while the superstructure is unrestrained by the abutments in the longitudinal direction.

Figure 5. Diagram. Analytical model rendering.

The seismic demand analysis typically consists of a multimodal linear response spectrum analysis. These analyses are used to define the displacement demands used to assess the ductile structural elements and the elastic demands on the capacity-protected structural elements. The plastic overstrength capacity of the ductile elements (i.e., plastic hinging forces in the columns) will become the plastic overstrength demands on the capacity-protected elements. The minimum of the elastic and plastic overstrength demands will define the seismic demands on the capacity-protected elements.

CHAPTER 3. CAPACITY ANALYSIS

After the demand analyses are completed, the capacity analyses should be run. The bridge in this design example is located in seismic design category (SDC) D, where the Seismic Guide Specifications require that a nonlinear static (pushover) analyses be used to determine the displacement capacity of the bridge and the plastic overstrength demands on the capacity-protected elements. The displacement capacity of each pier for a given direction of loading was defined as the top of column displacement when one of the hinges first reaches an ultimate curvature limit as defined in article 8.5 of the Seismic Guide Specifications.

The pushover models were generated from the global linear elastic response spectrum model, modified to include plastic hinges at the top and bottom of each pier. Three quantities were needed to convert the demand model into the capacity model. First, the locations of plastic hinges were determined, then the nonlinear moment-curvature relationships are determined. These must incorporate the variations in plastic moment capacity and ultimate curvature with variations in the axial load. This is typically accomplished with P-M and P-ϕ interaction relationships. Finally, the analytical plastic hinge length must be determined to relate curvature to rotation so that the displacement capacity can be determined by the structural model.

Due to the column detailing, where the square precast columns were reduced to circular and octagonal sections at the top and the bottom of the column, respectively, elastic column properties were based on the square cross-section, while the plastic hinge properties were based on the circular/octagonal cross-section.

The pier displacement capacity was evaluated in the pier transverse and longitudinal directions independently, which are defined as follows:

- Pier-Longitudinal: Perpendicular to the pier centerline.
- Pier-Transverse: Parallel to the pier centerline.

A pushover model was created for the intermediate piers to define the pier transverse and longitudinal seismic displacement capacities. The pier pushover models included the unfactored dead load pier reactions from the global analysis model. The pushover analyses were run ignoring P-Δ effects.

MOMENT-CURVATURE AND AXIAL FORCE-MOMENT INTERACTION RELATIONSHIPS

In the analysis model, the column moment-curvature responses were approximated as elastic perfectly plastic. Sectional responses were developed for multiple axial loads to account for associated changes in moment capacity and ultimate curvature limits. The actual moment-curvature relationship used by the analysis program interpolated the moment capacity using the P-M interaction surface for the cross-section. There are several commercially available sectional analysis programs that can determine moment-curvature and P-M interaction relationships. Any program or calculation method that is based on strain compatibility and uses appropriate nonlinear material constitutive relationships is deemed acceptable by the Seismic Guide Specifications.

The P-M interaction curve for the column plastic hinges was generated by taking the moment values as the plastic capacities from the idealized moment curvature response for each given axial load. As the columns were circular within the plastic hinge region, the P-M interaction was taken to be symmetric. In the case of rectangular columns, biaxial bending affects should be considered. The P-M interaction curve is shown in figure 6.

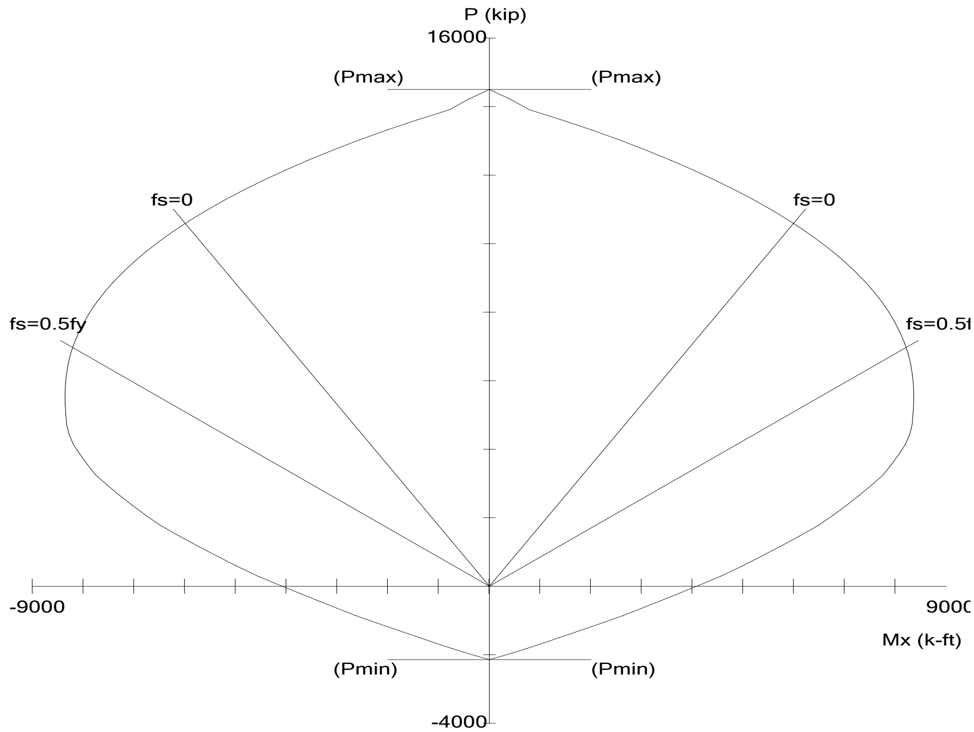

Figure 6. Diagram. Column P-M interaction curve (plastic moment capacity).

The P-M interaction curve was then entered into the structural analysis program to define the behavior of the plastic hinges at the top and bottom of the columns. If reinforcement or column sections were to differ at the top and bottom of the column, separate P-M interactions would be required for the top and bottom plastic hinges. The 5-foot-diameter circular section at the top of the column and the 5-foot-wide octagonal section at the bottom of the column were taken to be equivalent analytically; thus, only one P-M interaction curve was used.

PLASTIC HINGE LENGTHS

The final step in determining the displacement capacity and plastic overstrength forces was to define the analytical plastic hinge lengths (L_p), which were calculated in accordance with article 4.11.6 of the Seismic Guide Specifications, as shown in figure 7.

$$L_p = 0.08L + 0.15\, f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} + G_f$$

Figure 7. Equation. Calculating the analytical plastic hinge lengths.

In figure 7:

- L = Length of column from point of maximum moment to the point of moment contraflexure (inches)
- f_{ye} = Expected yield strength of longitudinal reinforcing steel (68 ksi for A706 steel; see Seismic Guide Specification table 5.1.2.1.1)
- d_{bl} = Nominal diameter of longitudinal column reinforcing steel bars (inches)
- G_f = Gap between the isolated flare and the soffit of the cap beam or the top of foundation

The example column will be designed with No. 14 longitudinal column reinforcing (A706) steel (d_{bl} = 1.693 inch and f_{ye} = 68 ksi). Because the columns are restrained against rotation in both the longitudinal and transverse directions, the column will go into double curvature regardless of the direction of loading; therefore, the plastic hinge lengths will be equal and can be calculated using the column height (H = 17 feet 1 inch).

For a column in double curvature:

$$L = \frac{H}{2} = \frac{17.08\text{ ft}}{2} = 102\text{ in.}$$

$$L_p = 0.08(102\text{ in.}) + 0.15(68\text{ ksi})(1.693\text{ in.}) = 25.4\text{ in.}$$

greater than or equal to

$$L_p = 0.3(68\text{ ksi})(1.693\text{ in.}) + 3\text{ in.} = 37.5\text{ in.}$$

Therefore, use:

$$L_p = 37.5\text{ in.}$$

INELASTIC COLUMN MOMENT AND SHEAR DEMANDS

In Type I structures, the substructure components act as fuses to limit the seismically induced forces on the superstructure and foundation components (i.e., capacity-protected elements). To restrict inelastic action to within the ductile components (i.e., the fuses), all other members within the structure must be able to resist the maximum forces that can be generated by the ductile fuses without damage. These design forces are known as the plastic overstrength forces. In this design example, the fuses are the plastic hinges that are allowed to form at the top and bottom of the columns. The plastic overstrength design forces are the plastic capacities of the column plastic hinges multiplied by a strength magnification factor (in this case, 1.2 for A706 reinforcing steel). The design forces for a bent with two or more columns must be calculated for displacements in the plane of the bent and perpendicular to the bent using expected material properties.

In-Plane Pier Design Forces

The plastic overstrength demands were determined following the procedures of article 4.11.4 of the Seismic Guide Specifications. Article 4.11.4 gives four steps to calculate the plastic hinging forces for bents with two or more columns within the plane of the bent. In this design example, the calculations used to determine the overstrength demands from the column plastic hinging are omitted, as they are not unique to a fully precast integral bent system. Of note, however, is that the length used to calculate the plastic shear demand is reduced by $L_p/2$ at each end based on the conservative assumption in the Seismic Guide Specifications that M_p is developed at the center of the plastic hinge. The axial, flexural, and column shear demands are provided in table 2. The axial demands are asymmetric due to the overturning axial forced generated due to frame action within the bent. Piers 2 and 3 have the same column length, so the plastic overstrength forces are equal.

Table 2. In-plane pier design forces.

	East Column	**West Column**
Axial Load (kip)	400	2,000
Length (ft)	14.0	14.0
Plastic Overstrength Moment (k-ft)	5,470	7,700
Plastic Column Shear (kip)	781	1,100

Out-of-Plane Pier Design Forces

Article 4.11.3 of the Seismic Guide Specifications covers plastic hinging perpendicular to the plane of the bent. The shear associated with plastic hinging is the sum of the plastic moment at the top and the bottom of the column, divided by the effective column height. In this example, the intermediate calculations are omitted. The overstrength forces are provided in table 3.

Table 3. Out-of-plane pier design forces.

	Pier 2	**Pier 3**
Axial Load (kip)	1,200	1,200
Length (ft)	14.0	14.0
Plastic Overstrength Moment (k-ft)	5,500	5,500
Plastic Column Shear (kip)	786	786

Once the plastic overstrength demands of the ductile elements are determined, the nominal shear capacity of the ductile elements must be checked to ensure the shear can be transferred to the capacity protected elements. This check is not detailed for this design example, as it was covered in detail in the first design example (see appendix B).

CHAPTER 4. DESIGN CHECKS

While a fully precast bridge bent system behaves seismically in much the same way as conventional cast-in-place reinforced concrete, special detailing considerations must be made to ensure ductile behavior. The most critical considerations are the connections from the precast column to the oversized pile shaft and the precast cap beam.

COLUMN DESIGN

The precast columns were designed according the Seismic Guide Specifications to provide adequate stiffness, curvature ductility capacity, and strength (axial, flexural, and shear). The details of these calculations are not presented here, as they are included in the first design example. The final column design was a 5-foot-wide square column reinforced with 14 No. 14 bars placed in a circular pattern within No. 9 circular hoops at 5 inches on center. All reinforcing had a minimum of 4 inches of clear cover. This column has a significant amount of transverse reinforcement due to the very high plastic shear demands. This is a product of the column's low aspect ratio (length divided by column width or diameter).

COLUMN-TO-PILE SHAFT SOCKET CONNECTION

In the fully precast bridge bent system, a socket connection is made from the column to the oversized pile shaft by casting the splice region of the shaft around a portion of the precast column, the length of which must be long enough to develop a force transfer mechanism that can resist column plastic overstrength forces and, if applicable, the column tension reaction. The column-to-pile shaft connection must be able to withstand the plastic overstrength demands associated with column plastic hinging in an essentially elastic manner, as the Seismic Guide Specifications require that oversized pile shafts be capacity protected. The design procedure here has been developed through large-scale experimental testing at the University of Washington. The critical components that must be checked to ensure plastic action within the column and essentially elastic pile-shaft behavior are as follows and will be addressed in turn:

- Oversized pile shaft flexural, shear, and axial capacity.
- Column embedment (i.e., the non-contact lap splice length between column and shaft longitudinal reinforcement).
- Pile shaft lateral confinement reinforcement in the connection region.

Pile Shaft Strength

In the Seismic Guide Specifications, the oversized pile shaft is considered a capacity-protected element, and as such, it must resist the plastic overstrength forces developed by the column plastic mechanism in an essentially elastic manner. There is nothing unique about the shaft strength calculations for a precast column with a socket connection. Per article 8.8.12 of the Seismic Guide Specifications, the oversized pile shaft will have to resist 125 percent of the flexural demand generated within the pile shaft by the column plastic moment and shear demand. The applied loads on the pile shaft are included in table 4.

Table 4. Pile shaft applied loads.

Column Axial Load, *P*	2,000 kip
Column M_{po}	7,700 k-ft
Column V_{po}	1,100 kip
1.25M_{po}	9,625 k-ft
1.25V_{po}	1,375 kip

The shaft demands are determined using a nonlinear soil-structure interaction analysis commonly utilizing p-y spring methods. At this point in the design, it may be necessary to consider the effects of liquefaction. The details of the pile shaft analysis will not be presented here, as they are standard practice and not unique to the precast bent system. The demands from the pile shaft analysis are given in table 5.

Table 5. Pile shaft design demands.

Column Axial Load, *P*	2,000 kip
M_{max}	51,000 k-ft
V_{max}	1,700 kip

Once these demands have been generated, the shaft reinforcement can be designed. For this design example, this resulted in a 10-foot-diameter shaft with 6 inches of clear cover to the spiral, 58 No. 18 bars in bundled pairs, and No. 8 circular hoops at 7 inches on center. This shaft has a substantial amount of both longitudinal and transverse reinforcement because of the capacity protection requirements. The factored P-M interaction curve for the pile shaft is shown in figure 8. Note the markers indicating the design points.

Column Embedment Length

To transfer the flexural forces from the column to the pile shaft, the column longitudinal bars must be fully developed at the top of the shaft. This requires a non-contact lap splice with the pile shaft longitudinal reinforcement. The length of column embedment into the pile shaft is then governed by the non-contact splice length of the column longitudinal steel. The minimum embedment length can then be calculated using the equation in figure 9, which uses the Class C lap splice length and assumes a 45-degree compression strut between the column and shaft longitudinal reinforcement. Because the shaft has a considerably larger diameter than the column and includes a high transverse reinforcement content, the column longitudinal bars are being developed within highly confined conditions. Accordingly, the development equation from article 8.8.4 in the Seismic Guide Specifications may be used for the development length.

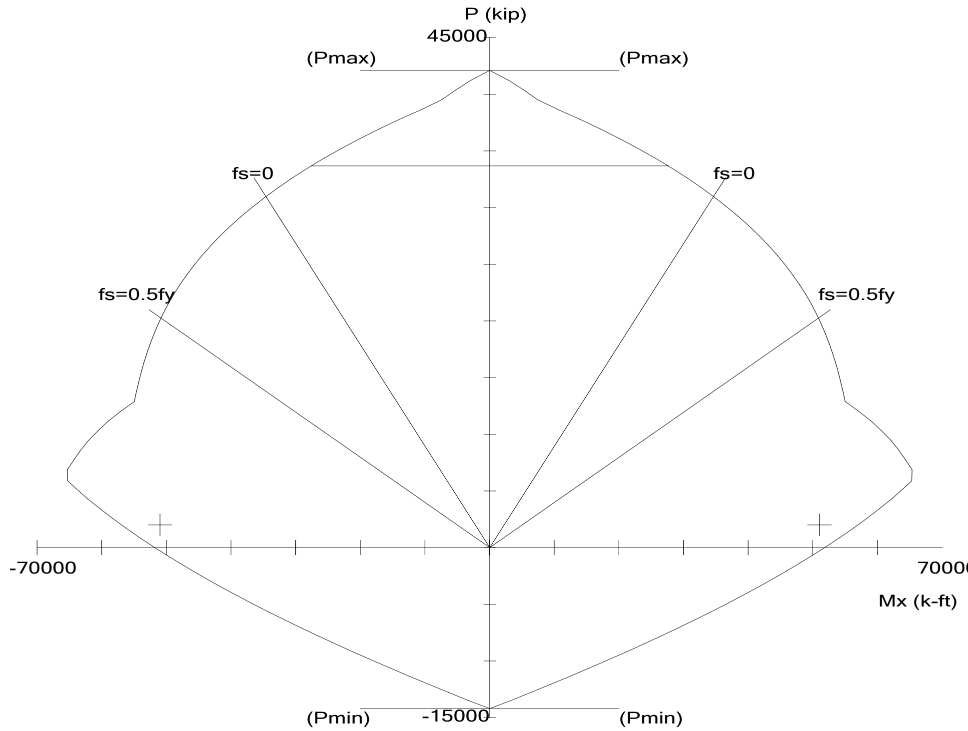

Figure 8. Diagram. Pile shaft factored P-M interaction curve.

$$l_e = l_s + e + c$$

Figure 9. Equation. Calculating minimum embedment length.

In figure 9:

l_e = Total embedment length of precast column

l_s = $1.7(l_{ac})$ = Class C splice length of controlling bar (inches)

l_{ac} = Anchorage length of the column longitudinal reinforcing (per article 8.8.4 of the Seismic Guide Specifications)

e = Largest center-to-center distance between column and shaft bars (inches)

c = Total bar end cover distance of both column and shaft bars (inches)

The column embedment length geometry and reinforcement arrangement are shown in figure 10.

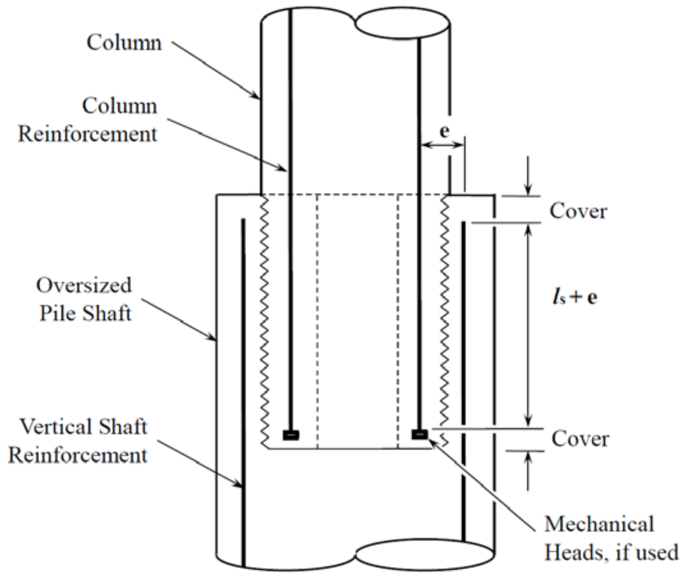

Figure 10. Diagram. Column-to-shaft longitudinal bar arrangement.

For the columns in this design example, No. 14 longitudinal bars must be anchored into the pile shaft. Therefore, the anchorage length is calculated as follows, according to the Seismic Guide Specifications:

$$l_{ac} = \frac{0.79 d_{bl} f_{ye}}{\sqrt{f'_c}} = \frac{0.79(1.693 \text{ in.})(68 \text{ ksi})}{\sqrt{4 \text{ ksi}}} = 45.5 \text{ in.}$$

Thus, the Class C splice length is taken as:

$$l_s = 1.7(l_{ac}) = 1.7(45.5 \text{ in.}) = 77.4 \text{ in.}$$

For this bridge, the largest center-to-center distance between column and shaft longitudinal reinforcement is 27.7 inches, and the end cover distance for the column and shaft bars is a total of 5 inches. The column embedment length is then:

$$l_e = l_s + e + c = 77.4 \text{ in.} + 27.7 \text{ in.} + 5 \text{ in.} = 110.0 \text{ in.}$$

This was rounded up to 12 feet for the final design. The entire length of the column embedded into the shaft was an octagonal section with 1-inch saw-tooth castellations to provide positive force transfer across the interface between the column and the shaft.

Pile Shaft Lateral Confinement

To prevent the end of the pile shaft from splitting open due to the prying forces imposed by the embedded precast column, a minimum amount of lateral confinement reinforcement must be included. This requirement is based on research conducted at the University of Washington. The equation for the minimum lateral confinement of the shaft surrounding the embedded portion of the precast column is shown in figure 11.

$$\frac{A_{sh}}{s_{max}} \geq \frac{k f_{ul} A_l}{2\pi f_{ytr} l_s}$$

Figure 11. Equation. Minimum lateral confinement of the shaft surrounding the embedded portion of the precast column.

In figure 11:

A_{sh}	=	Area of lateral confinement steel – one leg of spiral or welded hoop (in.^{2})
s_{max}	=	Spacing between lateral confinement steel (inches)
k	=	Efficiency factor, taken as 1.0 for the upper half of the embedment length and 0.5 for the lower half
f_{ul}	=	Tensile strength of column longitudinal reinforcement (ksi)
A_l	=	Total area of column longitudinal reinforcement (in.^{2})
f_{ytr}	=	Yield strength of lateral or transverse reinforcement (ksi)
l_s	=	Length of required Class C splice (inches)

To further reduce the size of cracks that may develop at the very top of the pile shaft, the upper 1 foot of the shaft confinement length should have additional reinforcement that is double the transverse reinforcement content that is provided in the upper half of the embedment length. This reinforcement can be positioned in one of several ways, as indicated in figure 12. Option A bundles the hoops or spirals, and Option B halves the spacing of the upper half requirement.

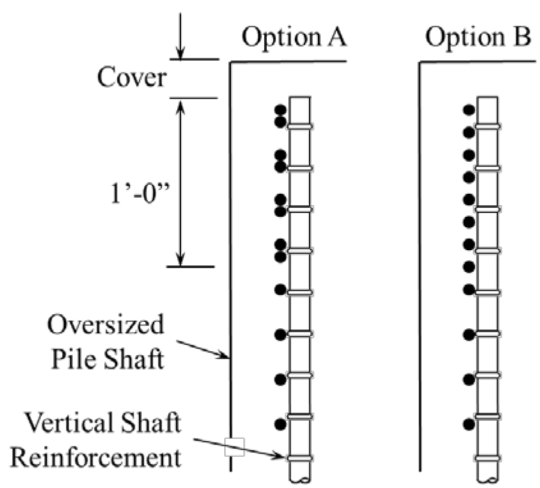

Figure 12. Diagram. Options for placement of additional lateral confinement reinforcement in the top 1 foot of oversized pile shafts.

The overall layout for the reinforcement of the splice region of the socket type connection is shown in figure 13.

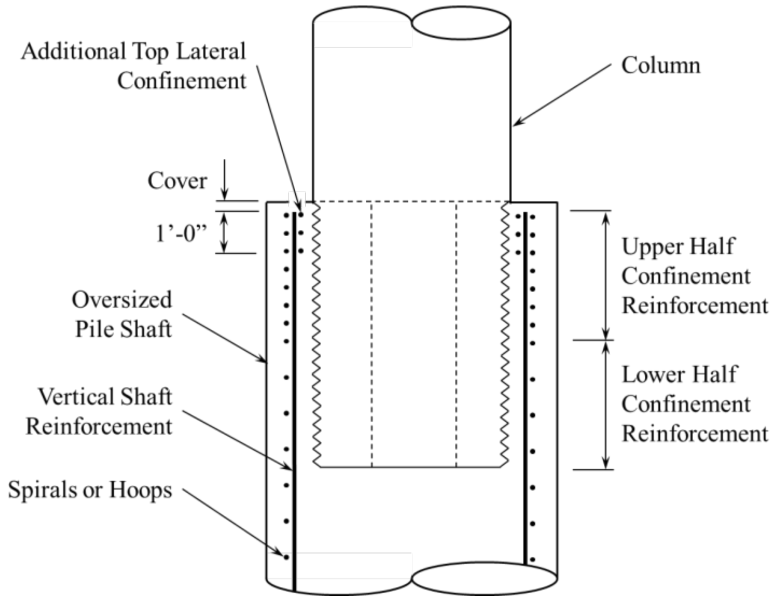

Figure 13. Diagram. Tie reinforcement for a socket-type column-to-pile shaft connection.

For the shafts in this design example, the following spacing must be used given the No. 8 circular hoops, expected material properties, and the distance from column to shaft longitudinal bars. By rearranging terms, the maximum spacing for the lower half of the embedment length is:

$$s_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{0.5 f_{ul} A_l} = \frac{2\pi(0.79 \text{ in.}^2)(60 \text{ ksi})(77.4 \text{ in.})}{0.5(68 \text{ ksi})(31.5 \text{ in.}^2)} = 21.5 \text{ in.}$$

For the upper half of the embedment length, the maximum spacing of the No. 8 hoops is:

$$s_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{f_{ul} A_l} = \frac{2\pi(0.79 \text{ in.}^2)(60 \text{ ksi})(77.4 \text{ in.})}{(68 \text{ ksi})(31.5 \text{ in.}^2)} = 10.8 \text{ in.}$$

And finally, the top 1 foot of the upper half of the embedment length must have double the reinforcement. Because the hoop spacing for the upper one half is not particularly onerous, Option B, where the maximum spacing of the upper half is halved, will be used. Thus:

$$s_{max} = \frac{1}{2}\left(\frac{2\pi A_{sh} f_{ytr} l_s}{f_{ul} A_l}\right) = \frac{1}{2}\left(\frac{2\pi(0.79 \text{ in.}^2)(60 \text{ ksi})(77.4 \text{ in.})}{(68 \text{ ksi})(31.5 \text{ in.}^2)}\right) = 5.4 \text{ in.}$$

However, as discussed earlier, the maximum spacing for the No. 8 hoops to resist the column plastic shear demand is 7 inches on center for the entire length of the shaft. Therefore, the requirements for lateral confinement of the socket connection are not critical, and the spacing for shear resistance governs the design for the embedded length of the column. However, the upper 1 foot of the shaft requires a reduced spacing interval. Here, three No. 8 hoops are included in the upper 1 foot of the shaft, providing a spacing of 4 inches on center.

COLUMN-TO-CAP BEAM CONNECTION

The connection from the column to the integral precast cap beam poses several challenges to ensure adequate force transfer from the substructure to the superstructure. The detailing for the column-to-cap beam connection and the cast-in-place diaphragm at the intermediate piers for this design example are very similar to those used in the design example detailed in appendix B. As such, the special design and detailing considerations will not be discussed in any detail here.

CLOSING REMARKS

The design checks shown in this design example provide an overview of the specific considerations that must be made to ensure controlled ductile performance for a precast column connected to an oversized cast-in-place pile shaft.

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