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Establish Representative Pier Types for Comprehensive Study: Western United States

by

R.A. Imbsen, R.A. Schamber and T.A. Osterkamp

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Technical Report NCEER-96-0006

May 28, 1996

This research was conducted at Imbsen and Associates, Inc. and was supported by the Federal Highway Administration under contract number DTFH61-92-C-00112.

NOTICE

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16. Abstract (Limit: 200 words)

This report describes bridge pier types and seismic design and detailing procedures for new construction typical of the western U.S. since the mid-1970s. The companion report, NCEER 96-0005, describes pier types and seismic design and detailing procedures representative of the eastern U.S. since about 1980. Representative bridge types and pier type details for nine western states are referenced in Section 2. The states contributing material include Alaska, Arizona, California, Idaho, Montana, Nevada, Oregon, Wyoming and Washington. Section 3 of the report includes column and pier design and detailing issues which include: column spiral reinforcement; column shear reinforcement within the outside potential plastic hinge zones; spirals, hoops or tie requirements into bent caps and footings of fixed columns; column reinforcement lap splice restrictions; column reinforcement recommendations into knee joints; modification of linearly elastic seismic forces by R or Z factors; multi-column bent and pier wall foundation supports; column flares reinforced with minimal transverse and longitudinal reinforcement; fully confined column flares; distinction between a column and pier wall; pier wall confinement reinforcement; and partial height pier walls. Section 4 of the report includes bent cap detailing issues and Section 5 footing detailing issues. Section 6 addresses construction material issues.

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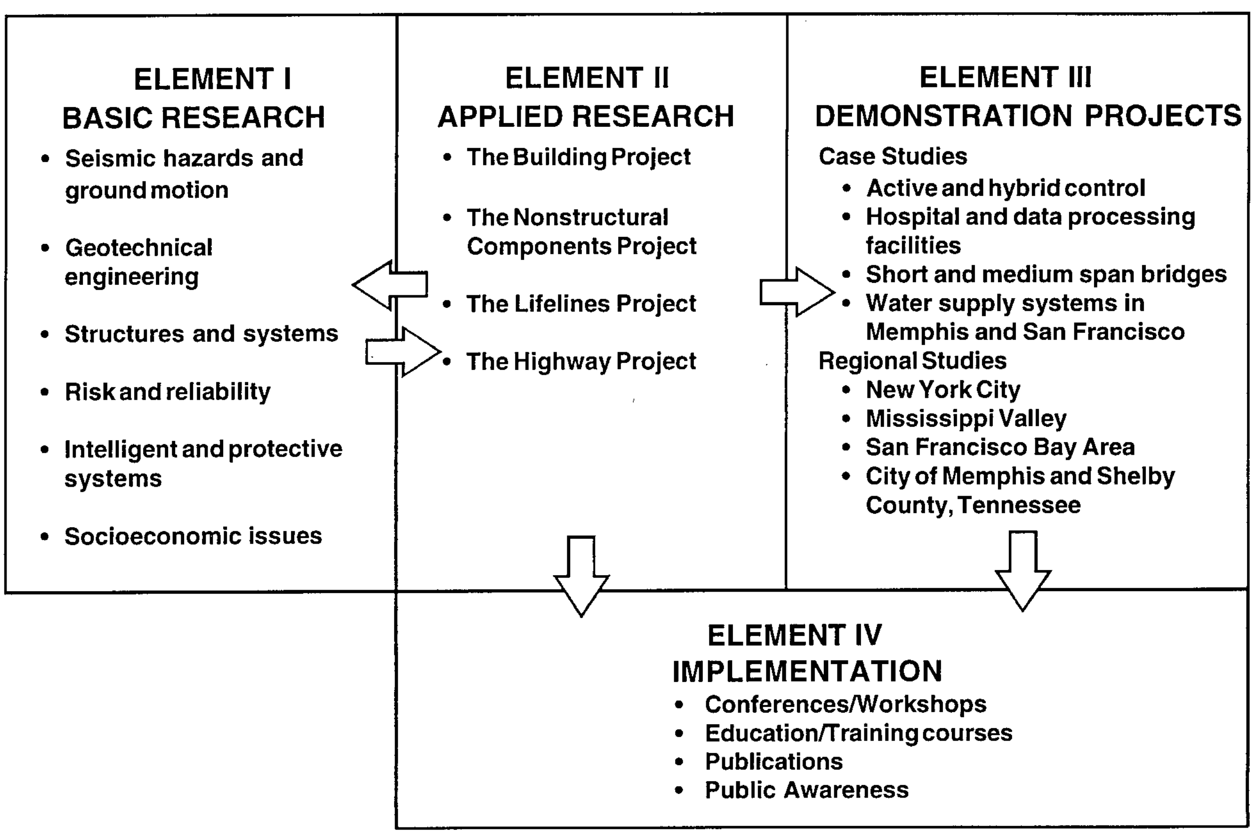
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established in 1986 to develop and disseminate new knowledge about earthquakes, earthquake-resistant design and seismic hazard mitigation procedures to minimize loss of life and property. The emphasis of the Center is on eastern and central United States *structures*, and *lifelines* throughout the country that may be exposed to any level of earthquake hazard.

NCEER's research is conducted under one of four Projects: the Building Project, the Nonstructural Components Project, and the Lifelines Project, all three of which are principally supported by the National Science Foundation, and the Highway Project which is primarily sponsored by the Federal Highway Administration.

The research and implementation plan in years six through ten (1991-1996) for the Building, Nonstructural Components, and Lifelines Projects comprises four interdependent elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten for these three projects. Demonstration Projects under Element III have been planned to support the Applied Research projects and include individual case studies and regional studies. Element IV, Implementation, will result from activity in the Applied Research projects, and from Demonstration Projects.



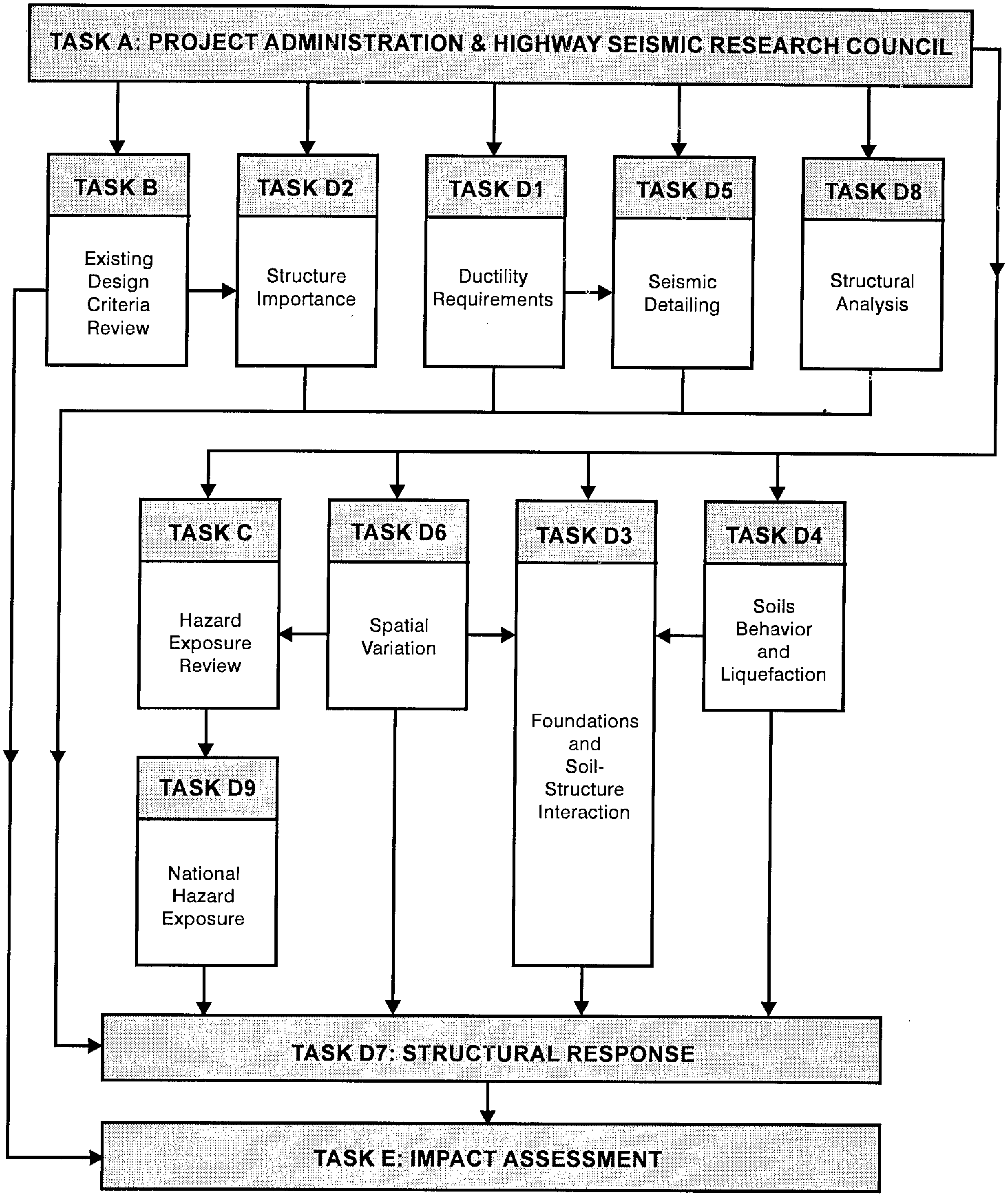
Research under the **Highway Project** develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and develops improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to: (1) assess the vulnerability of highway systems and structures; (2) develop concepts for retrofitting vulnerable highway structures and components; (3) develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, with particular emphasis on soil-structure interaction mechanisms and their influence on structural response; and (4) review and improve seismic design and performance criteria for new highway systems and structures.

Highway Project research focuses on one of two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the new highway construction project, and was performed within Task 112-D-1.1(b), "Establish Representative Pier Types for Comprehensive Study" of the project as shown in the flowchart.

The overall objective of this task is to identify and establish pier designs and details currently in use throughout the U.S. The task was split into two parts, one focused on collecting and establishing representative pier types in the eastern U.S. and the other concerned with western U.S. practice. This report describes bridge pier types and seismic design and detailing procedures typical of the western U.S. since the mid-1970s. The companion report, NCEER-96-0005, describes pier types and seismic design and detailing procedures representative of the eastern U.S. since about 1980.

The states contributing material include Alaska, Arizona, California, Idaho, Montana, Nevada, Oregon, Washington and Wyoming. Column and pier design and detailing issues, bent cap detailing issues, footing detailing issues and construction material issues are addressed.

SEISMIC VULNERABILITY OF HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00112



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ABSTRACT

This report describes bridge pier types and seismic design and detailing procedures for new construction typical of the western U.S. since the mid-1970s. The companion report, NCEER 96-0005, describes pier types and seismic design and detailing procedures representative of the eastern U.S. since about 1980. Representative bridge types and pier type details for nine western states are referenced in Section 2. The states contributing material include Alaska, Arizona, California, Idaho, Montana, Nevada, Oregon, Washington and Wyoming. Section 3 of the report includes column and pier design and detailing issues which include: column spiral reinforcement; column shear reinforcement, within the outside potential plastic hinge zones; spirals, hoops or tie requirements into bent caps and footings of fixed columns; column reinforcement lap splice restrictions; column reinforcement recommendations into knee joints; modification of linearly elastic seismic forces by R or Z factors; multi-column bent and pier wall foundation supports; column flares reinforced with minimal transverse and longitudinal reinforcement; fully confined column flares; distinction between a column and pier wall; pier wall confinement reinforcement; and partial height pier walls. Section 4 of the report includes bent cap detailing issues and Section 5 footing detailing issues. Section 6 addresses construction material issues.

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SECTION 1
INTRODUCTION

In the summer of 1993, the National Center for Earthquake Engineering Research initiated a research program directed at developing improved seismic analysis and design procedures for highway infrastructure. The research program is sponsored by the Federal Highway Administration of the U.S. Department of Transportation and consists of a series of special studies, each focused on the seismic analysis or design of specific highway system components (e.g., bridges or tunnels) and structural elements (e.g., foundations or substructures).

As a basis for developing improved bridge design standards, an early task within this program was conducted to identify and establish pier designs and details currently in use throughout the U.S. The task was split into two parts, one focused on collecting and establishing representative pier types in the eastern U.S. (Task 112-D1.1(a)) and the other concerned with western U.S. practice (Task 112-D-1.1(b)).

This report describes bridge pier types and seismic design and detailing procedures typical of the western U.S. since the mid-1970s. The companion report, NCEER 96-0005, describes pier types and seismic design and detailing procedures representative of the eastern U.S. since about 1980.

Column and pier seismic typical design and detailing procedures are outlined in this report for new construction. Design and detailing procedures for bridges which fall into Seismic Performance Categories (SPC) C and D of the AASHTO Standard Specifications for Seismic Design of Highway Bridges are emphasized (AASHTO, 1992). The California Department of Transportation (Caltrans) has been conducting research and intensely studying earthquake effects on bridges since the 1971 San Fernando (Sylmar) earthquake, the 1989 Loma Prieta earthquake, and the 1994 Northridge earthquake. The lessons learned from these earthquakes, and subsequent research, have shaped Caltrans' current seismic design requirements. California's experience and detailing requirements influenced the development of the AASHTO Standard Specifications for Seismic Design of Highway Bridges.

Therefore, Caltrans' seismic design procedures are also extensively referenced throughout the report (Caltrans, 1996). These two specifications are the only complete guidelines for the seismic design of western bridges. Most western states follow the AASHTO specifications, but some have adopted many of Caltrans' procedures and detailing practices.

SECTION 2
REPRESENTATIVE PIER TYPES (WEST)

A table has been prepared identifying representative bridge types for nine western states. Common substructure types or pier types are included in Table 2-1. In addition, several states submitted representative pier type details which are included in this report.

TABLE 2-1 Representative Pier Types for Nine Western States

State	Seismicity Categories	Common Superstructure Types	Common Substructure Types
Alaska	Mostly higher categories C & D	1. P/S precast bulb tees, up to 140' spans.	1. Concrete pier cap with high capacity steel pipe or H pile extending from cap to pile tip.
Arizona	Mostly A & B	1. P/S precast I-girder. 2. Steel I-girders. 3. Box Girder.	1. Concrete columns on spread footings; flares avoided in categories C & D. 2. Drilled shafts.
California	Do not use AASHTO. Accec. 0.1g to 0.7g.	1. CIP Box Girder – longer spans. 2. Slab – short spans.	1. Round, oblong or hexagon concrete columns on footings. 2. Commonly have concrete pile extension bents.
Idaho	A, B & C Majority of bridges A < 0.29	1. P/S precast concrete I-girders. 2. Steel welded plate girder.	1. River crossings (large ice or debris flows) require solid shafts or hammer head style pier. 2. Interchanges utilize P/S precast I-girder with multi-column bent or hammer head style bent concrete bridges < 300' in length.
Montana	Mostly A & B, but some up to D	1. P/S I-girder simply supported. 2. Steel girder continuous.	1. Concrete columns on footings. 2. Recently using steel pipe pile or drilled shaft bents in low seismic zones.
Nevada	A – D	1. Post-tensioned box girder. 2. Steel beam.	1. Concrete columns on footings – no longer using flares.
Oregon	Mostly B & C Use revised AASHTO Map	1. P/S precast beams. 2. Cast-in-place concrete.	1. Oblong concrete columns on pile or spread footings.
Washington	A – D	1. P/S precast I-girder.	1. Round concrete columns on pile or spread footings – no flares.
Wyoming	Mostly A, with some up to C.	1. Steel beams. 2. P/S precast I-girder.	1. Round concrete columns on footings. 2. Steel pile extension bents.

TABLE 2-1 Representative Pier Types for Nine Western States (cont.)

State	Common Superstructure to Substructure Connection	Substructure to Foundation Connection	Common Abutment Types	Comments
Alaska	Pile has moment resisting connection to cap. Beams set on bearing pads.	NA. Mostly pile extension bents.	Seat type.	
Arizona	Steel girders are set on bearing pads.	Fixed column base.	Seat type – fixed end diaphragms are avoided.	Generally follow AASHTO, but include some Caltrans criteria.
California	Monolithic integral bent caps. Spiral continues into cap.	Fixed column base for single column bents, pinned for multi-column base.	Box girders on seat type abutments, slab has integral abutment.	Do not follow AASHTO, have own seismic specifications.
Idaho	Pin or expansion style connection.	Fixed column base.	Concrete structure less than 300' and with skews less than 30 degrees an integral abutment is used. All other bridges a standard backwall style abutment is used.	
Montana	Spiral continuous into cap, beams set on bearing pads.	Fixed column base.	Seat type.	
Nevada	Monolithic integral bent caps, spiral continuous into cap.	Fixed column base.	Diaphragm abutment most common. Steel girders set on bearing pads.	
Oregon	Spiral continuous into cap.	Fixed column base.	Mostly seat type, then integral abutment.	Are beginning to introduce base isolation systems.
Washington	Monolithic super to substructure connection. Spiral into cap.	Fixed column base.	Integral abutments, then seat type.	
Wyoming	Steel beams set on bearing pads. Concrete I-girders cast monolithic with cap.	Pile extension bents not usually designed for pile tension.	Monolithic cast. Some are seat type.	~90% of state is Category A. Category D is in Yellowstone Park.

SECTION 3
COLUMN AND PIER DESIGN AND DETAILING ISSUES

3.1 Column Spiral Reinforcement

Description: Preference is given to columns with a circular main longitudinal reinforcement layout, confined with transverse spiral or welded hoop reinforcement. The helically formed spiral cage usually maintains a constant diameter throughout the column length. Only if the column has very unusual geometry is rectangular reinforcement layout appropriate. Oblong columns should be reinforced with two or more interlocking spiral cages. In order to insure adequate shear transfer between the spirals, special details are needed in the interlock area. Spiral reinforcement is the most preferred transverse reinforcement. For bar sizes larger than #6 for transverse reinforcement, hoops are generally used.

See Figures 3-1, 3-2, 3-3, 3-4, 3-5, 3-6, and 3-7.

Advantages: Spiral reinforcement provides excellent main longitudinal reinforcement confinement by taking advantage of hoop stress. Columns that are properly confined have been demonstrated to possess several times the ductility of inadequately confined columns. The amount of transverse reinforcement required to properly confine the column core concrete can be considerably reduced by using spirals instead of rectangular hoops and cross ties, and the open core area facilitates the placement and consolidation of concrete.

Disadvantages: Spiral Columns are somewhat more difficult to construct than tied columns, especially interlocking spirals. Non-prismatic shapes are more difficult to accommodate.

Historical: Following the San Fernando (Sylmar) Earthquake in 1971, the seismic detailing of columns was significantly changed. Since many columns failed due to a lack of confinement, procedures for determining the required transverse reinforcement and column shear reinforcement details were revised. Greater emphasis was placed on ensuring that the columns are adequately confined, particularly in the plastic hinge zone.

Between 1983 and 1988, the National Institute of Standards (NIST) conducted full-scale and prototype (1/6 scale) tests on 30 and 15 foot columns (Stone & Cheok, 1989). These tests confirmed that ductility ratios in the 6 to 10 range can be obtained for properly confining columns. The columns eventually failed when the confining reinforcement fractured, followed by buckling of longitudinal reinforcement.

3.2 Column Shear Reinforcement, Within and Outside the Potential Plastic Hinge Zone (AASHTO Standard Specifications for Seismic Design of Highway Bridges 8.4.1)

Description: AASHTO Seismic Specifications require the column, or piles in column bents, transverse reinforcement in the plastic hinge region to meet three criteria's: shear capacity to resist applied loads; have a minimum volumetric ratio; and have a spacing not to exceed the lesser of 4 inches (6 inches, Category B) or one-quarter the minimum member dimension. This

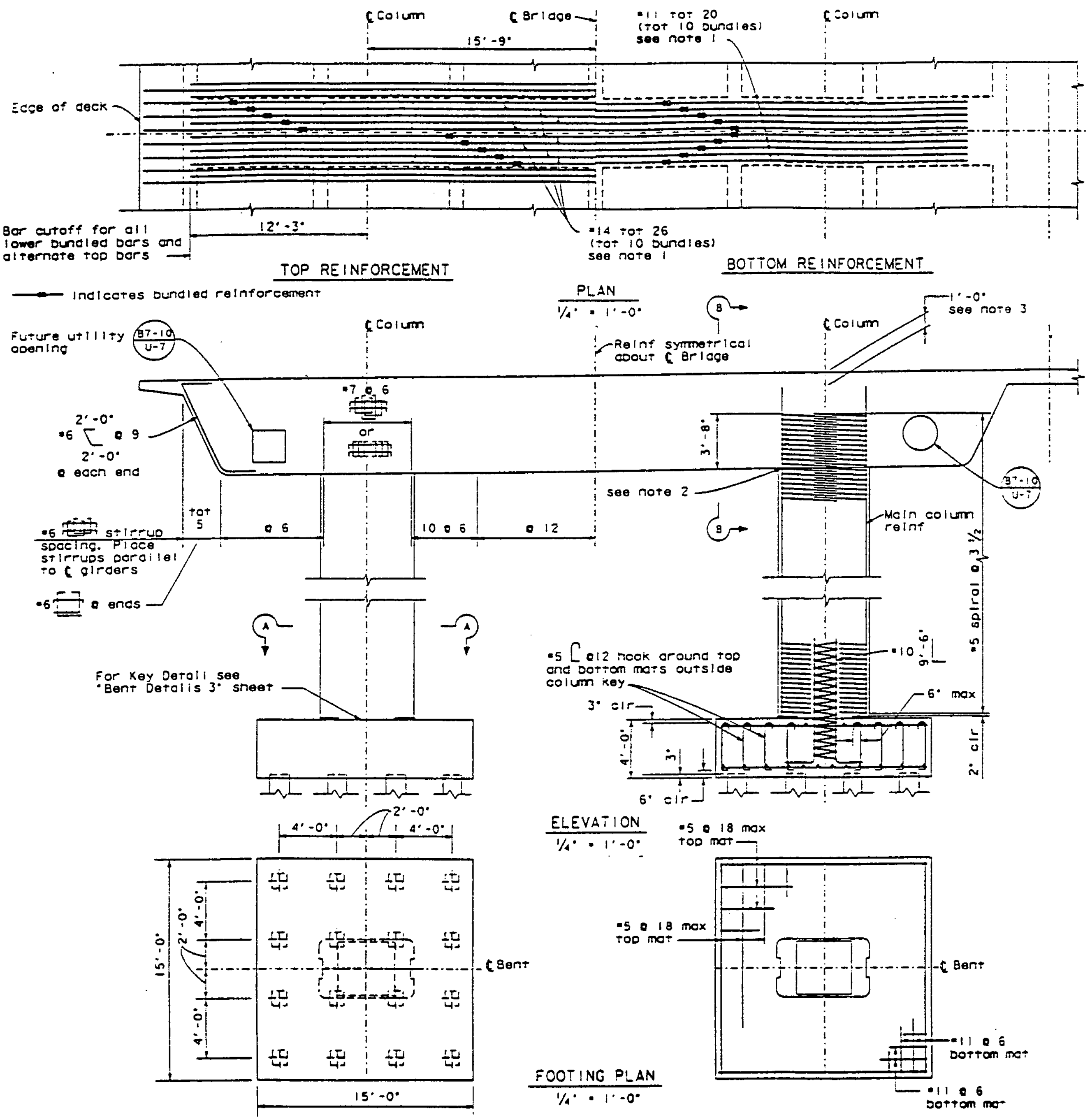
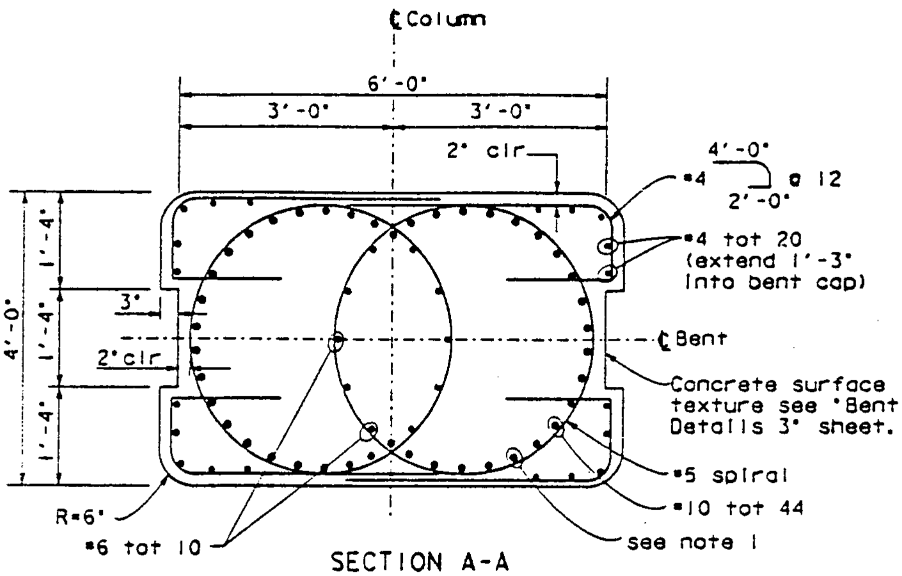


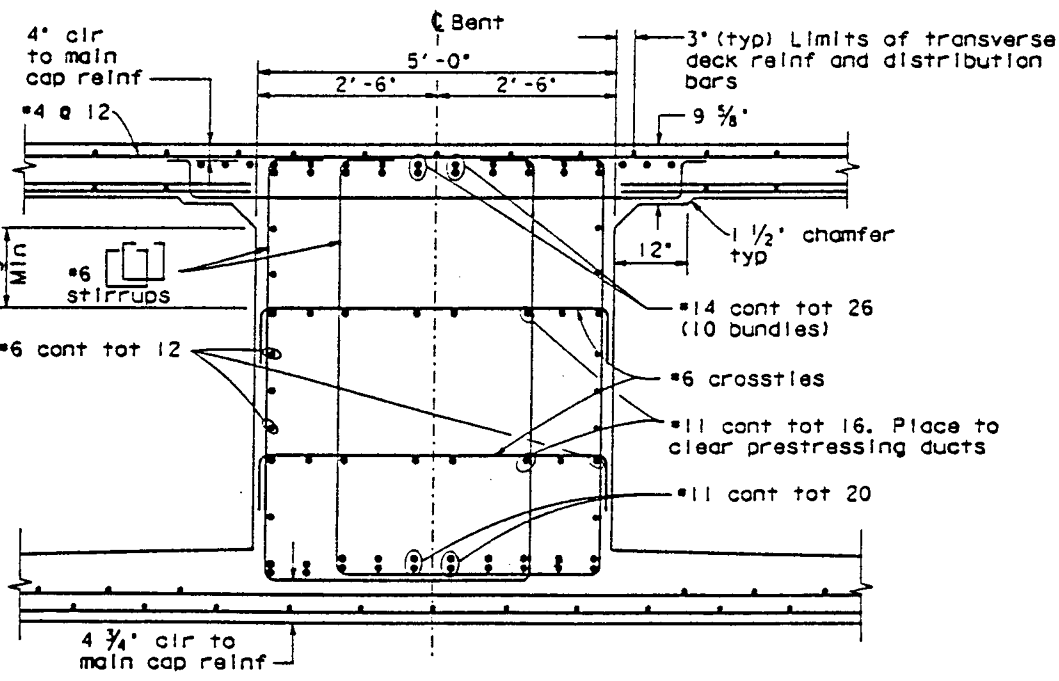
FIGURE 3-1A California Detail 1



SECTION A-A
¾" = 1'-0"

NOTES:

1. No lap splices allowed in main reinforcement.
2. Spiral may be discontinuous at bottom cap reinforcement.
3. Cut bars as necessary to clear prestressing.



SECTION B-B
¾" = 1'-0"

FIGURE 3-1B California Detail 1

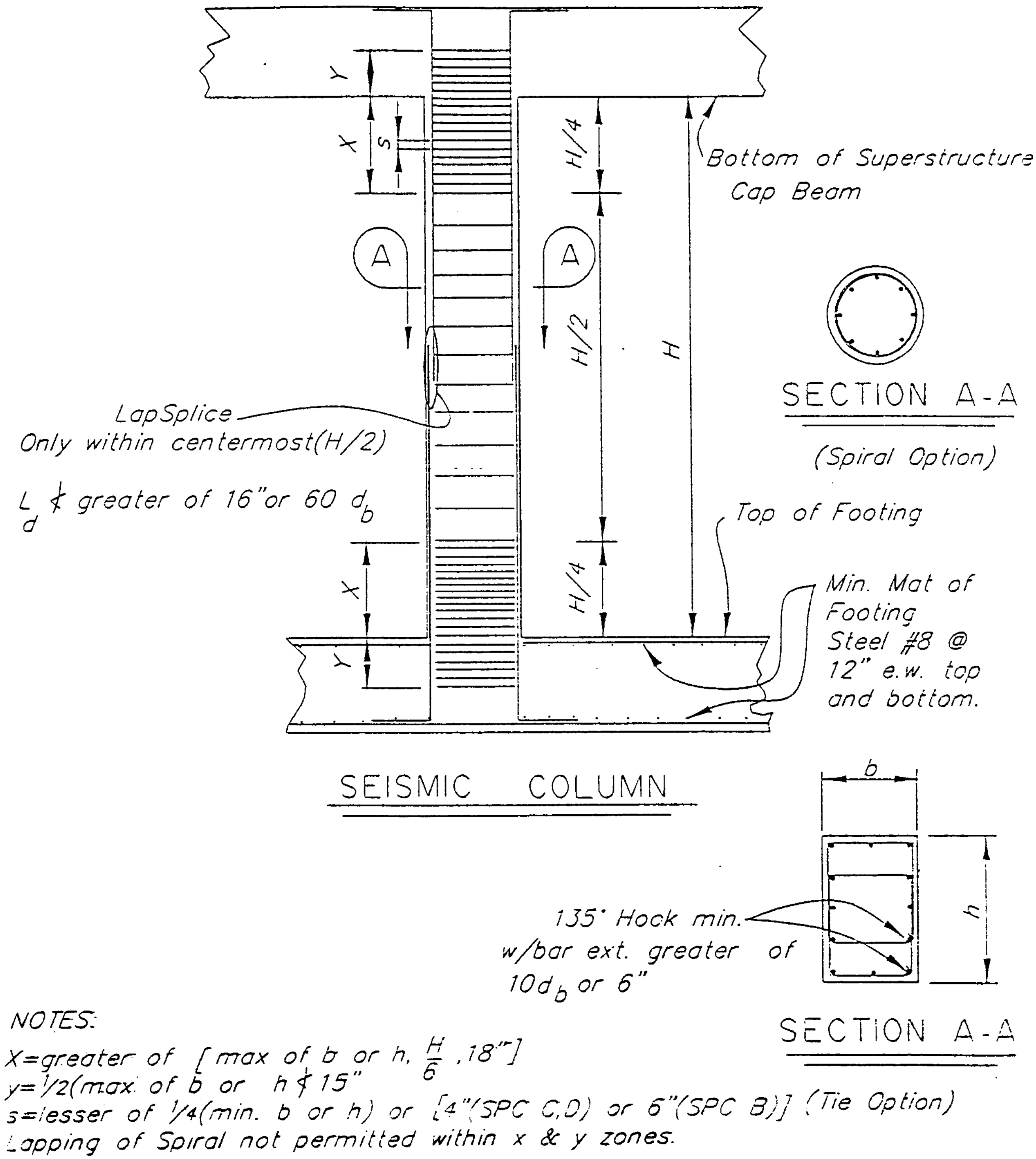


FIGURE 3-2 Alaska Detail 1

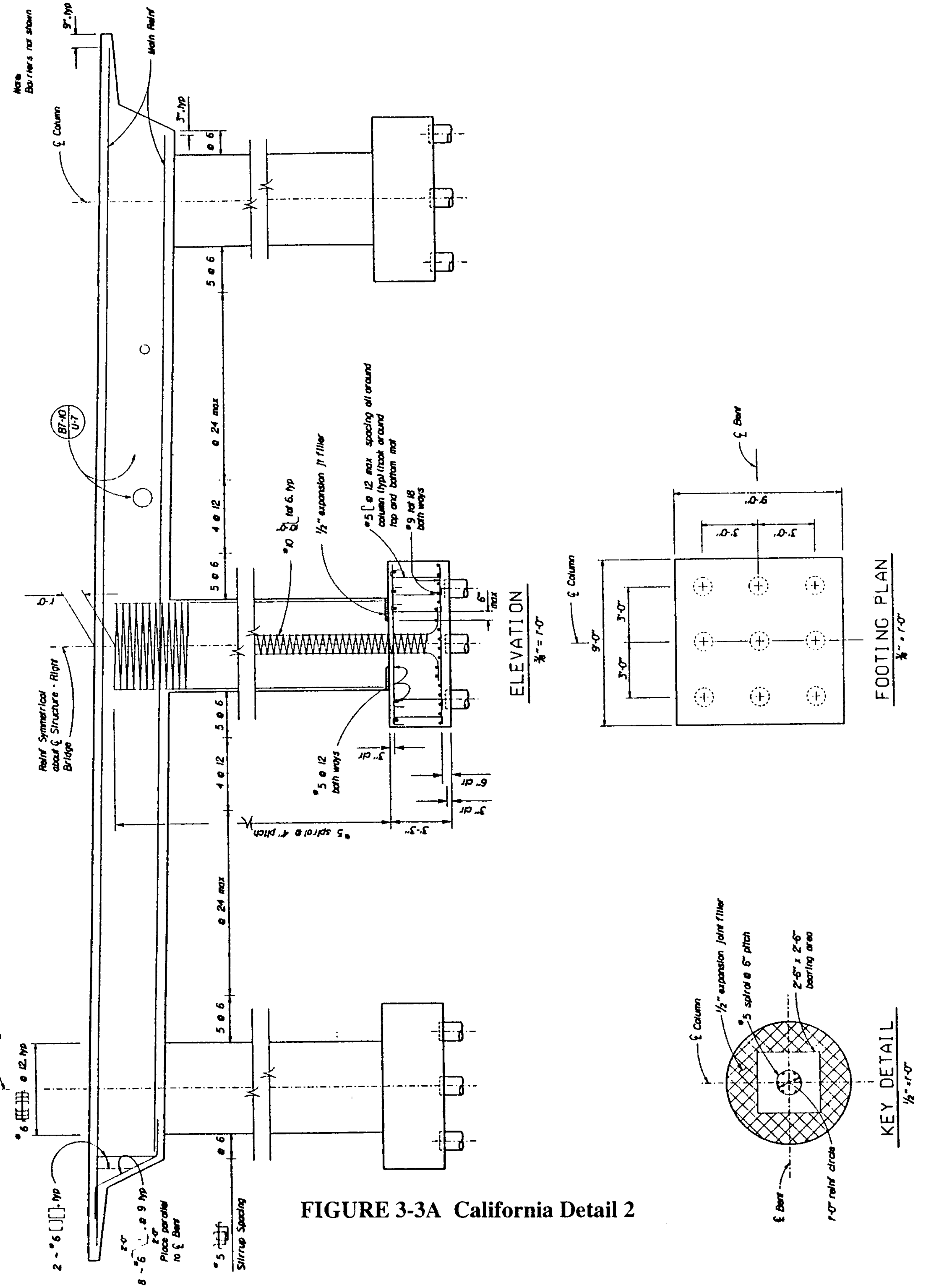


FIGURE 3-3A California Detail 2

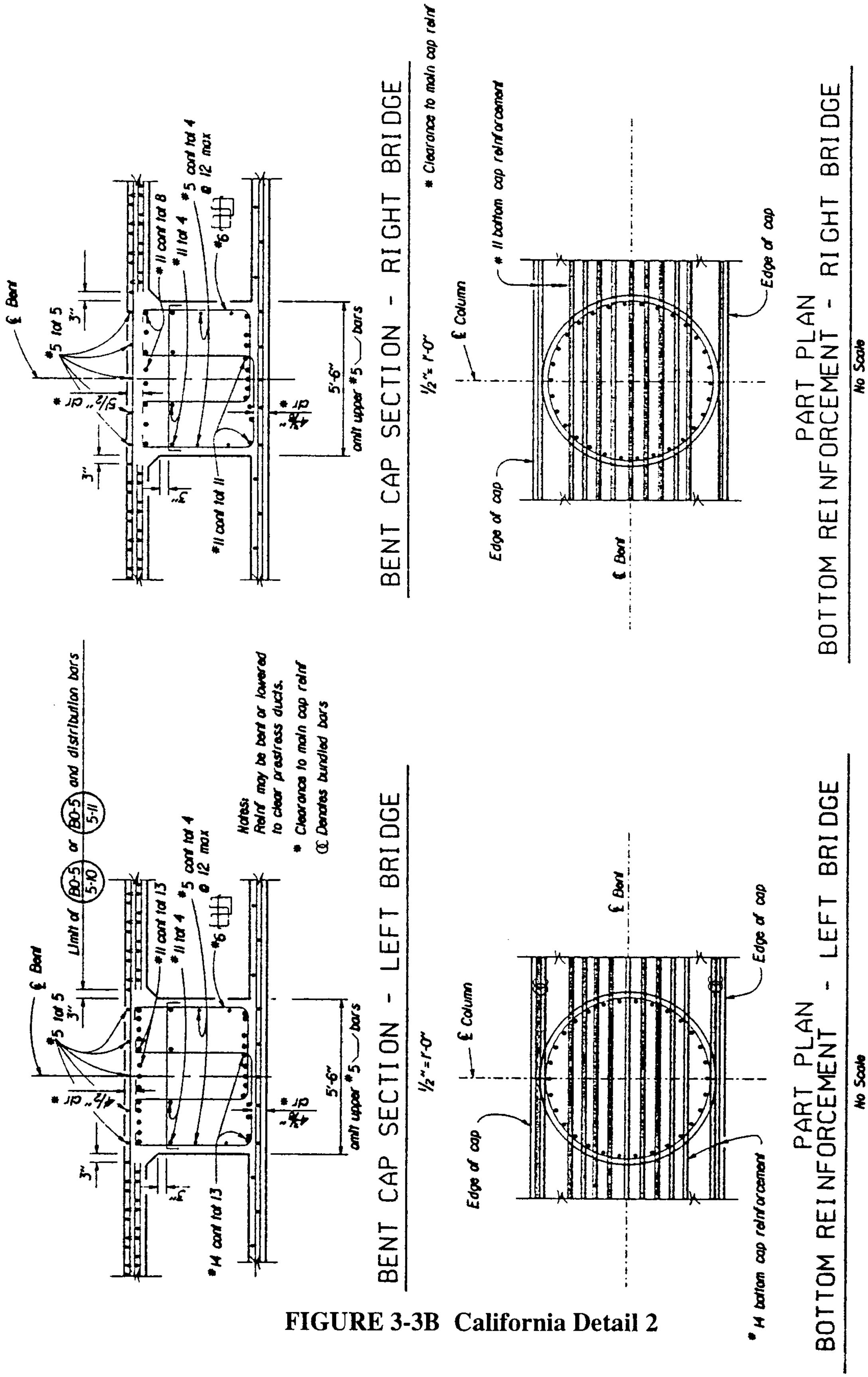


FIGURE 3-3B California Detail 2

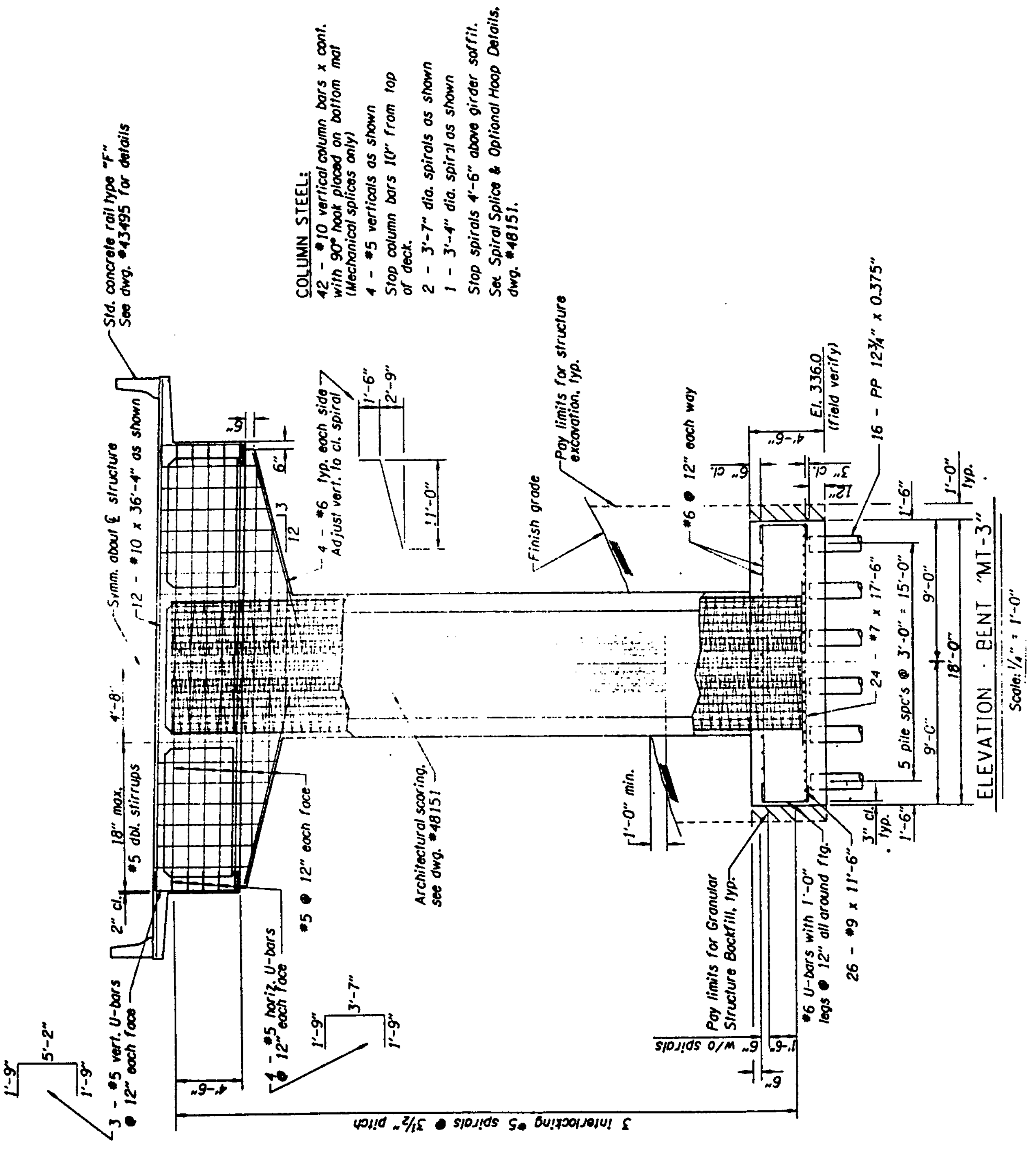
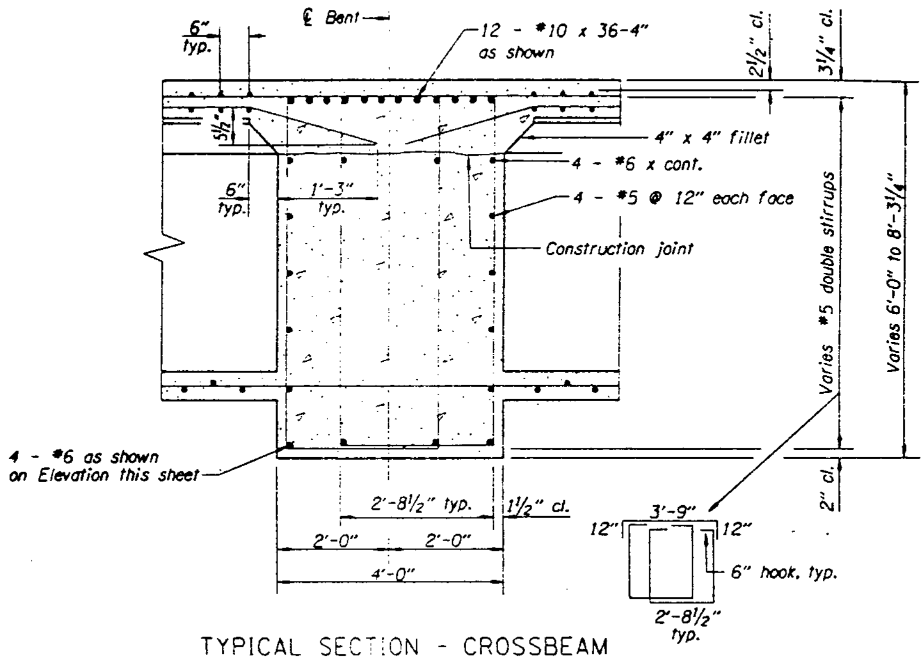
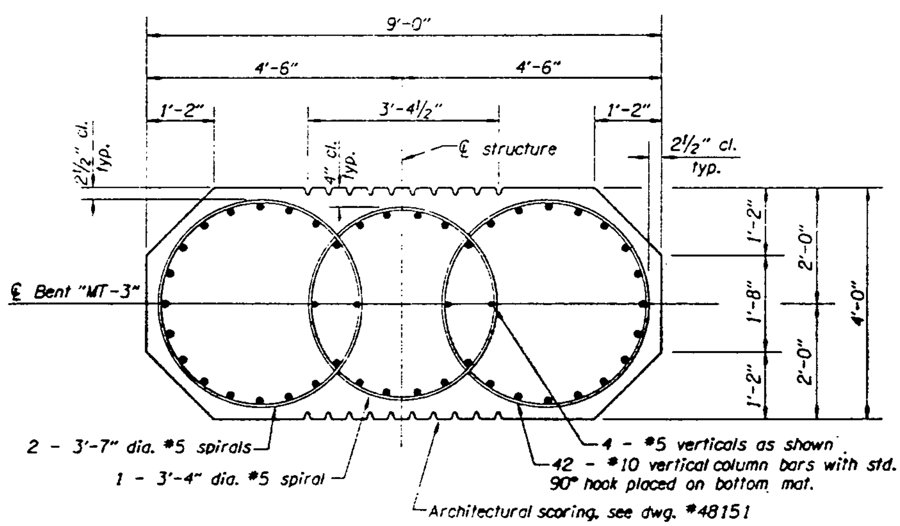


FIGURE 3-4A Oregon Detail 1



TYPICAL SECTION - CROSSBEAM

Scale: ¾" = 1'-0"



COLUMN SECTION

Scale: ¾" = 1'-0"

FIGURE 3-4B Oregon Detail 1

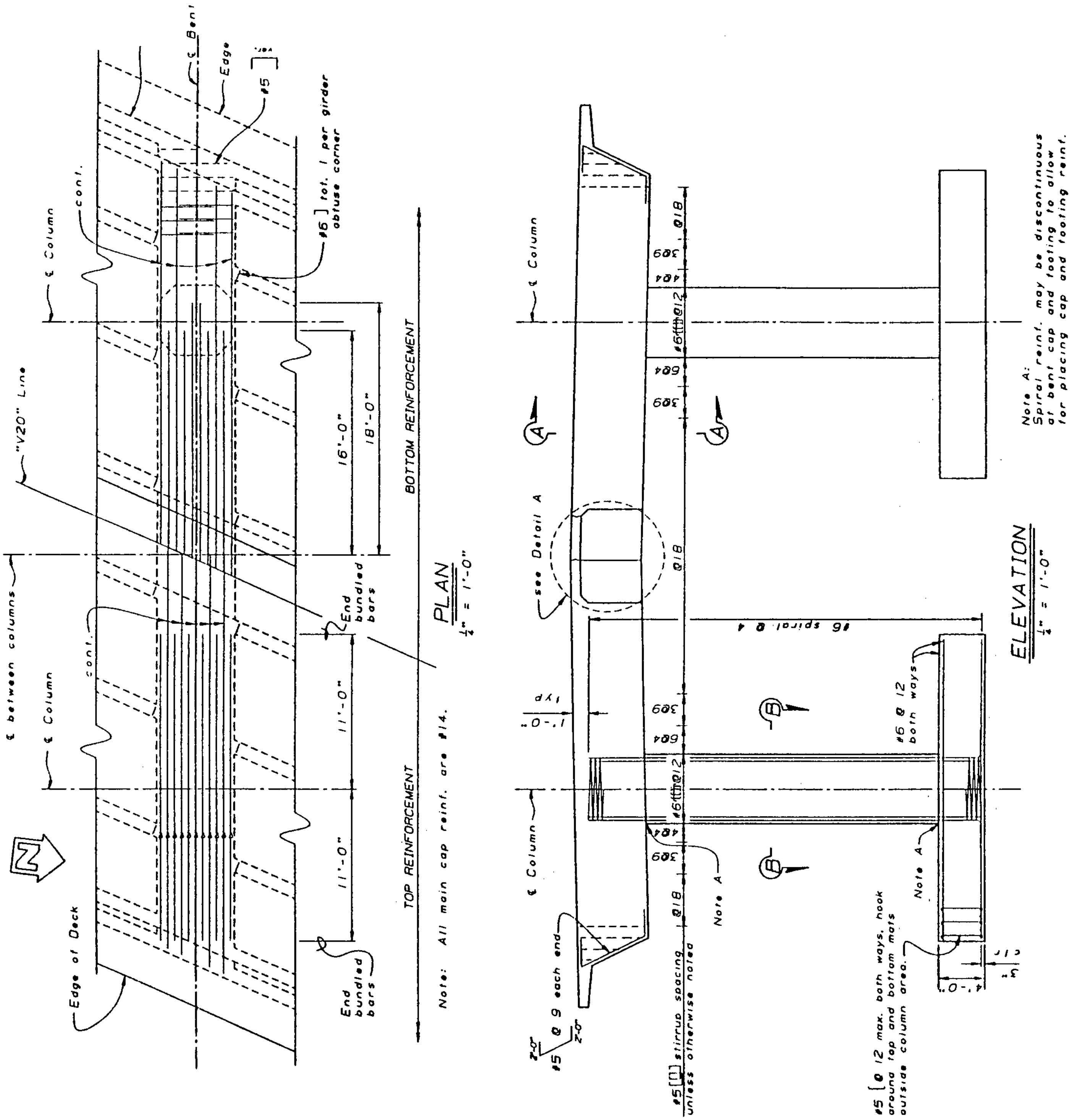


FIGURE 3-5A California Detail 3

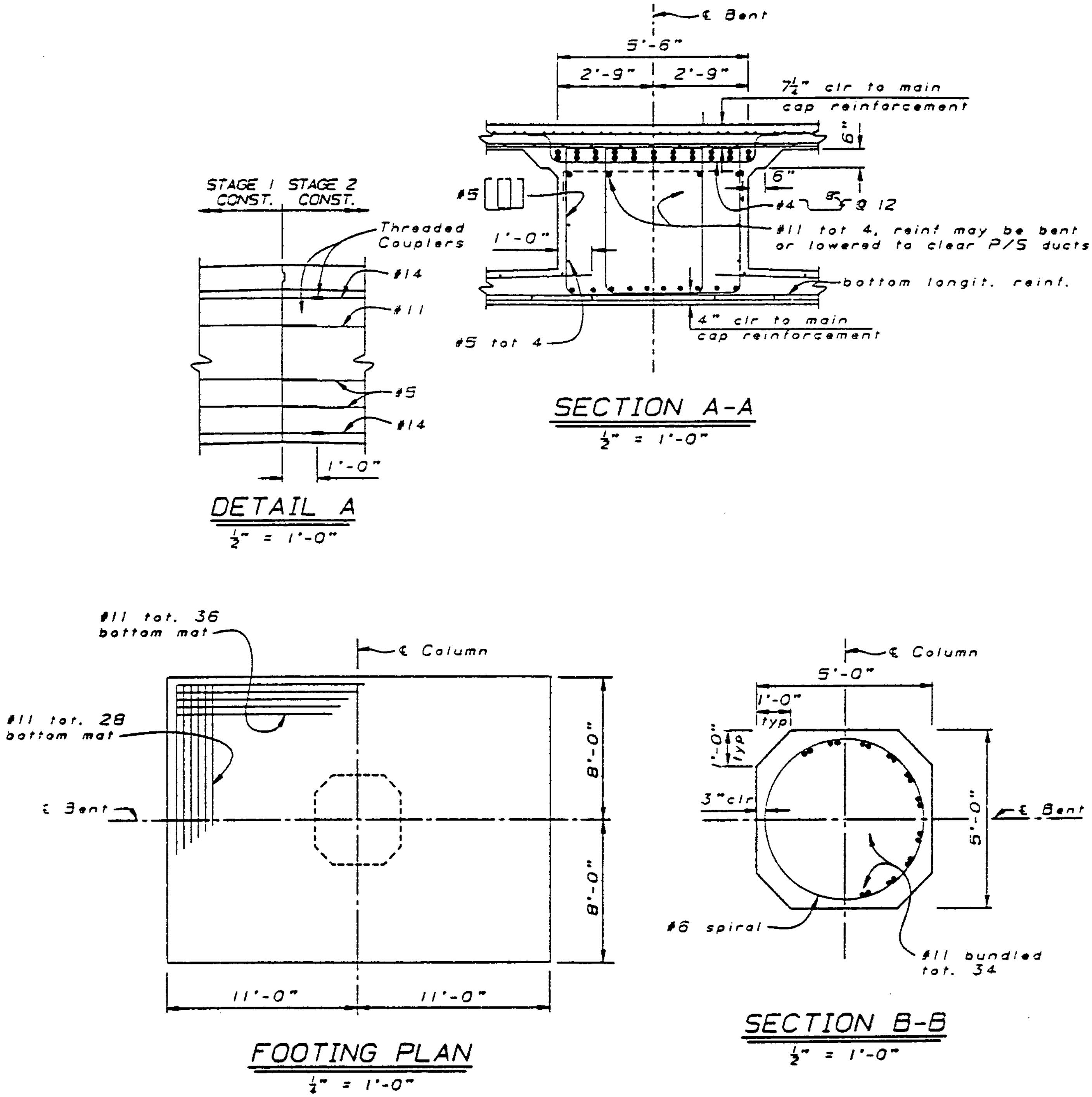


FIGURE 3-5B California Detail 3

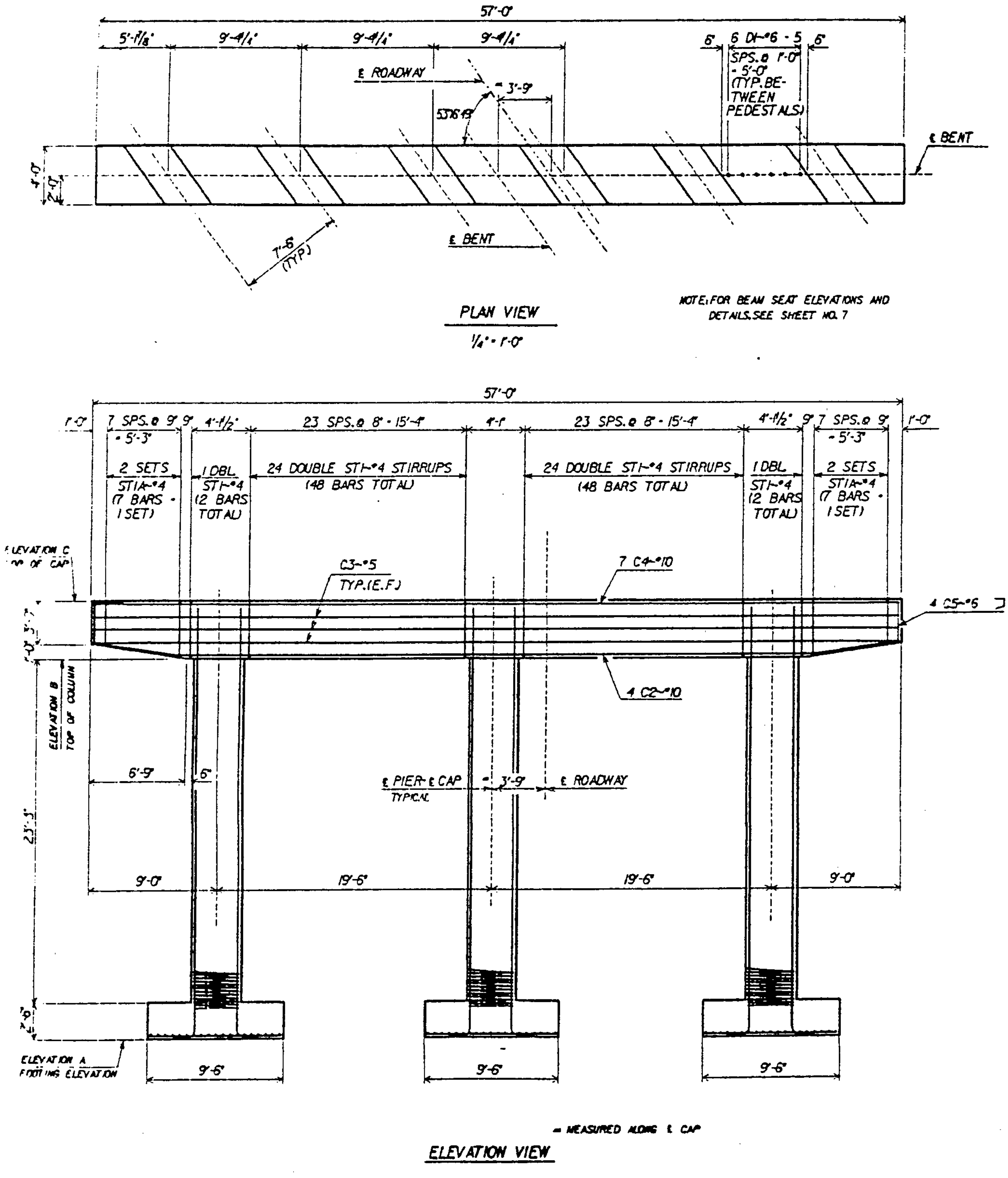
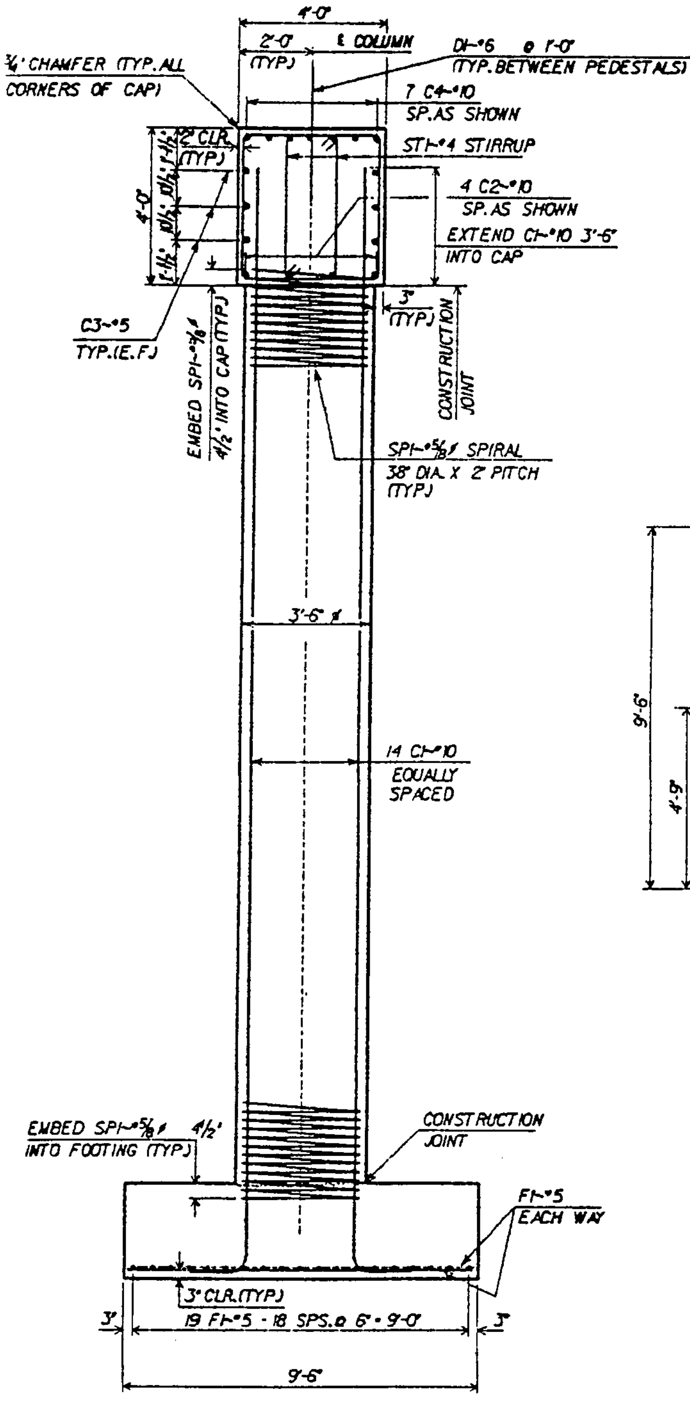


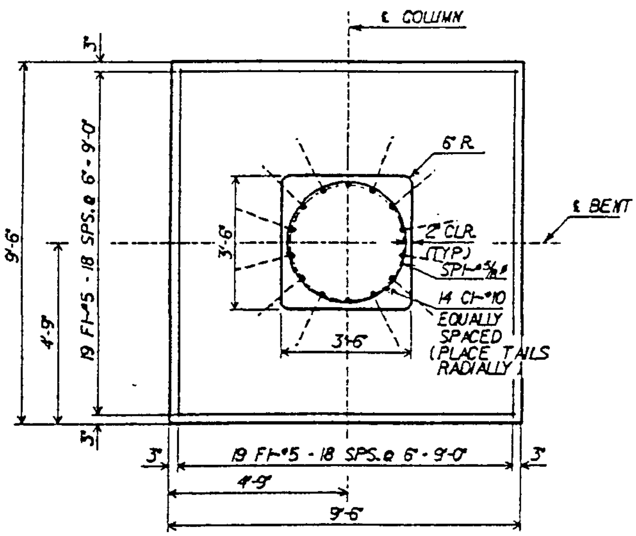
FIGURE 3-6A Idaho Detail 1



TYPICAL SECTION

ELEVATION TABLE

		ELEV. A	ELEV. B	ELEV. C
NORTH BOUND LANE	BENT NO. 1	4730.13	4756.88	4768.88
	BENT NO. 2	4730.30	4756.05	4760.05
SOUTH BOUND LANE	BENT NO. 1	4730.30	4756.05	4760.30
	BENT NO. 2	4730.14	4755.89	4759.89



FOOTING PLAN

FIGURE 3-6B Idaho Detail 1

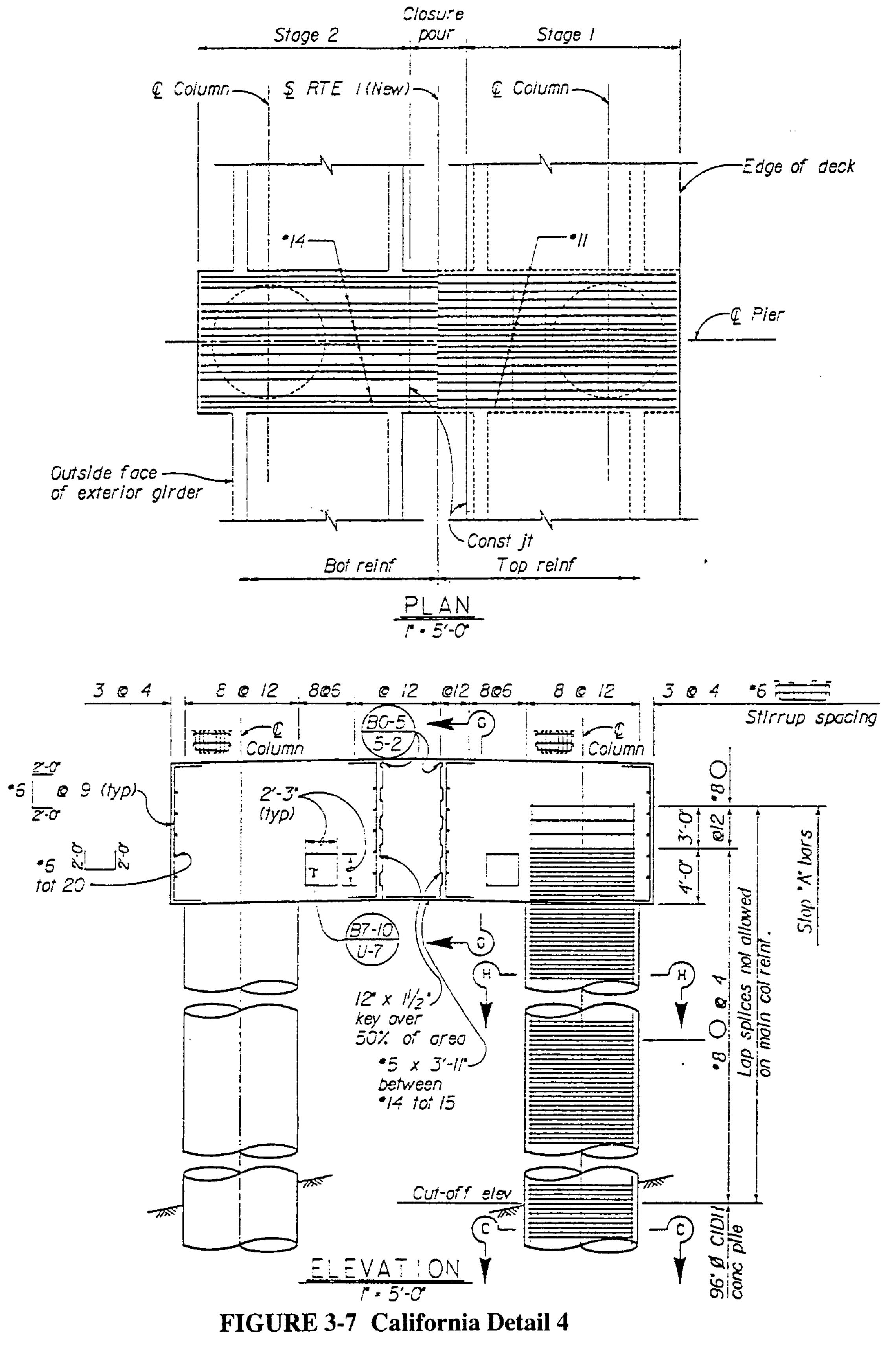


FIGURE 3-7 California Detail 4

reinforcement must extend over a length from each column end not less than the maximum cross-sectional column dimension or one-sixth the column clear height, but not less than 18 inches.

The specifications do not specify a required percentage of confinement reinforcement outside the plastic hinge region.

Caltrans' transverse reinforcement requirements are similar to AASHTO, the minimum reinforcement percentage in the column is taken as the largest of the steel based on the following criteria: minimum steel, confinement, and shear capacity (Caltrans' Bridge Design Specification (Caltrans' B.D.S.) 8.19.1, 8.18.2, and 8.16.3, respectively). These three requirements must be met throughout the column length. Within the plastic hinge zone, a more stringent criterion is used for the confinement condition. The plastic hinge zone is defined as the larger of the column or pier wall region within 24 inches of the support, one-sixth the column length, or the maximum horizontal dimension of the prismatic column portion (Caltrans' B.D.S. 3.21.8.2). Once the required spiral pitch or hoop spacing has been determined, this spacing is generally kept constant for the entire column length in order to simplify construction.

See Figures 3-2, 3-8, 3-9, and 3-10.

Advantages: These procedures ensure that adequate transverse steel has been provided to allow for the formation of plastic hinges without the loss of structural integrity. This confining steel allows for ductile column design.

Disadvantages: Slightly more steel than required is usually used outside the plastic hinge zone.

3.3 Spirals, Hoops, or Ties are Required to Continue into Bent Caps, and Footings of Fixed Columns (AASHTO Standard Specifications for Seismic Design of Highway Bridges 8.4.3)

Description: The column or pile bent transverse reinforcement required for column confinement must continue into the adjoining member a distance equal to one-half the maximum column dimension, but not less than 15 inches. The reinforcement may be discontinuous at the footing top mat reinforcement and bent cap bottom mat reinforcement to simplify construction. The reinforcement spacing is usually kept the same as that required in the plastic hinge zone.

Caltrans' criterion is similar but more stringent than AASHTO's (Caltrans' B.D.S. 8.18.2.1.4 and 8.18.2.1.3). The transverse reinforcement is required to continue into the footing of fixed columns to the point of tangency of the column bar hooks. Transverse reinforcement is also required to extend into the cap a distance equal to the lesser of one-half the confined core diameter, or the development length of the main column reinforcement, either straight or hooked. If the compression member (column or pier) has a larger cross section than that required by loading, the requirement that the transverse reinforcement must extend into the joint may be waived if structural analysis or tests demonstrate that the transverse reinforcement is not required. In order to take advantage of the main longitudinal reinforcement development length reduction

(AASHTO and Caltrans' B.D.S. 8.25.3.3), the maximum allowable transverse reinforcement spacing in the joint is limited to 4 inches.

See Figures 3-1, 3-2, 3-3, 3-4, 3-5, 3-6, 3-7, 3-8, 3-10, 3-11, 3-12, 3-13, 3-14, and 3-15.

Advantages: The joint becomes more ductile due to the confining steel in the cap and footing region. A more secure embedment bond is created between the column reinforcement and the adjoining member. The column main longitudinal reinforcement embedment length is reduced since the required development length is 25 percent less when a 4" maximum spiral spacing is used.

Disadvantages: Construction is more difficult since a large amount of steel must be placed in the column-bent cap and column-footing intersection regions. Special cap stirrup detailing is required through the column area. In prestressed bridges with monolithic bent caps, the column spiral reinforcement may have to be adjusted to clear prestress ducts.

Historical: During the 1971 San Fernando earthquake, column reinforcement pulled out of the footings at the I-5/14 Interchange. One of the precautions instituted to prevent this failure mode was to require the spiral reinforcement to continue into the joint.

3.4 Column Reinforcement Lap Splices are Restricted (AASHTO Standard Specifications for Seismic Design of Highway Bridges 8.4.1(F))

Description: The column main longitudinal reinforcement may only be spliced in the center half of the column. The splice length must be at least 60 bar diameters, but not less than 16 inches.

Caltrans' does not permit lap splices in the column main longitudinal reinforcement within the column plastic hinge zone (Caltrans' B.D.S. 8.32.1). Outside the plastic hinge zone, lap splices are only permitted if the column height is greater than 34 feet clear, and then only for No. 11 bars and smaller. In these taller columns, lap splices are not allowed within ten feet of the column ends if the column is fixed. If the column is pinned at the bottom, lap splices are allowed only in the bottom two-thirds of the column. Continuous unspliced column bars from the footing into the bent cap is the preferred detail.

AASHTO Seismic Specification 8.4.1(E) does not allow lap splicing of transverse shear reinforcement within the plastic hinge zone. Caltrans' does not permit lap splices within or outside the plastic hinge region in spiral or hoop reinforcement, only welding or mechanical couplers are allowed (Caltrans' B.D.S. 8.18.2.2.6).

See Figures 3-1, 3-2, 3-7, 3-14, 3-16, 3-17, and 3-18.

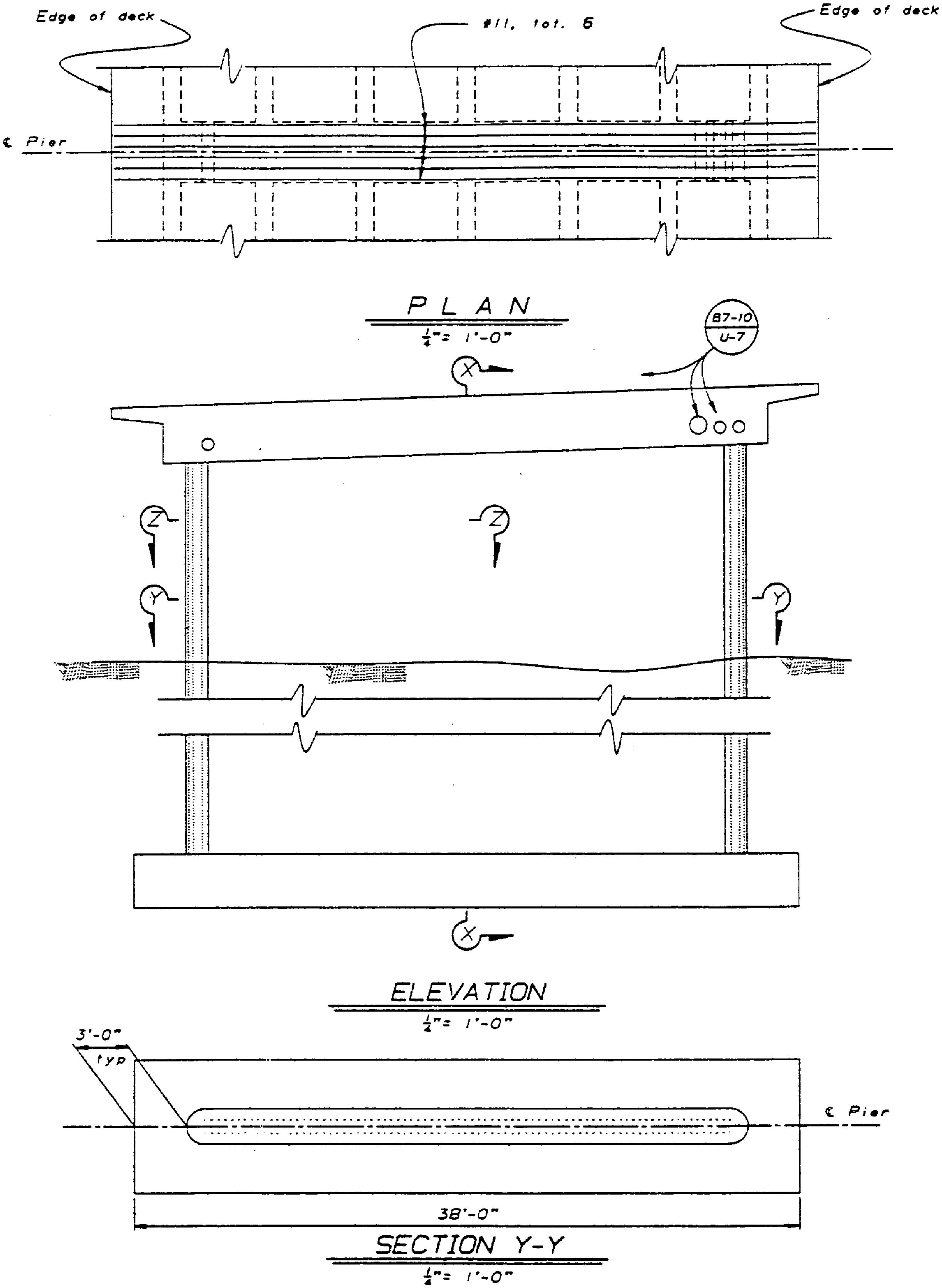


FIGURE 3-8A California Detail 5

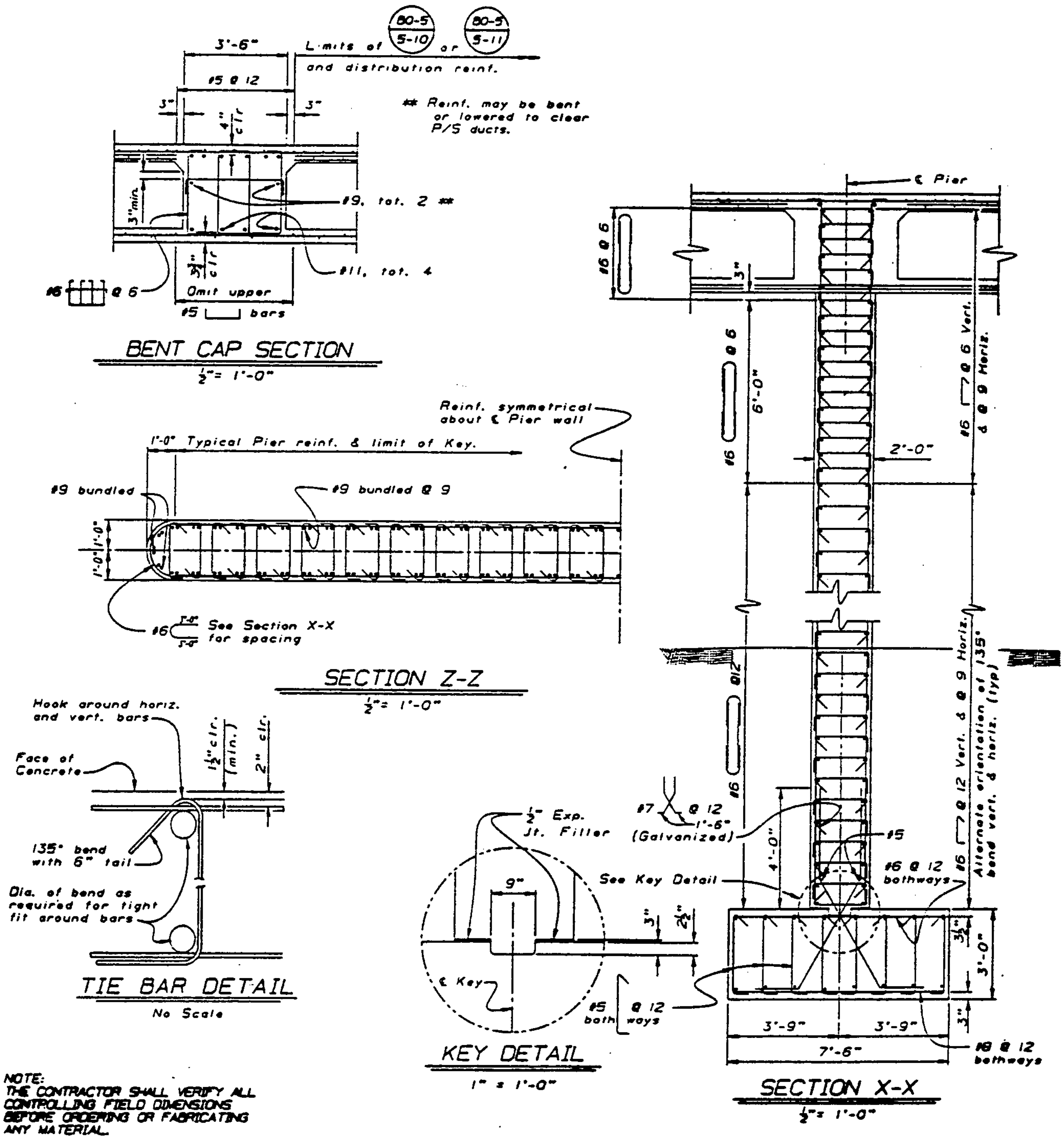


FIGURE 3-8B California Detail 5

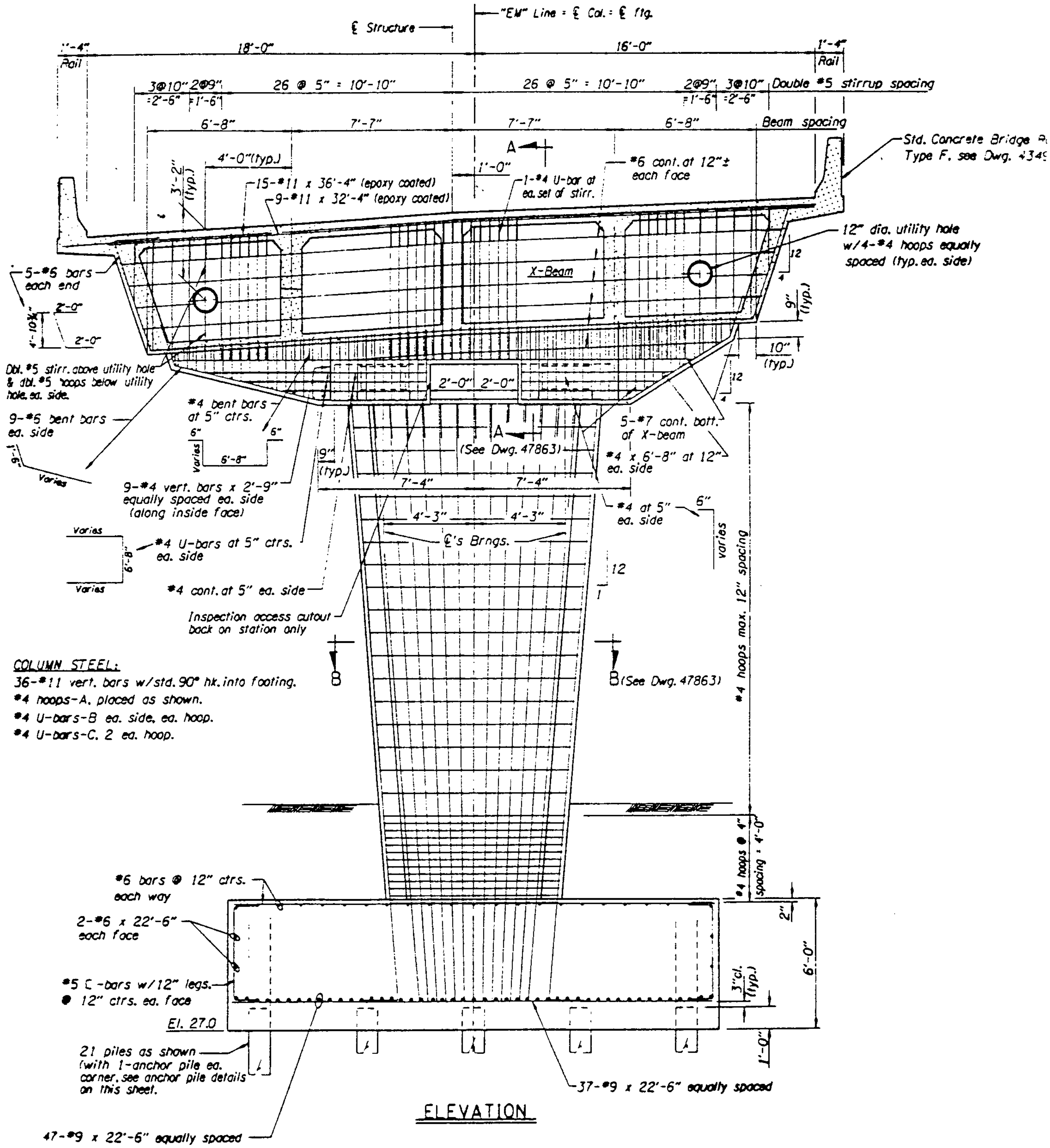


FIGURE 3-9A Oregon Detail 2

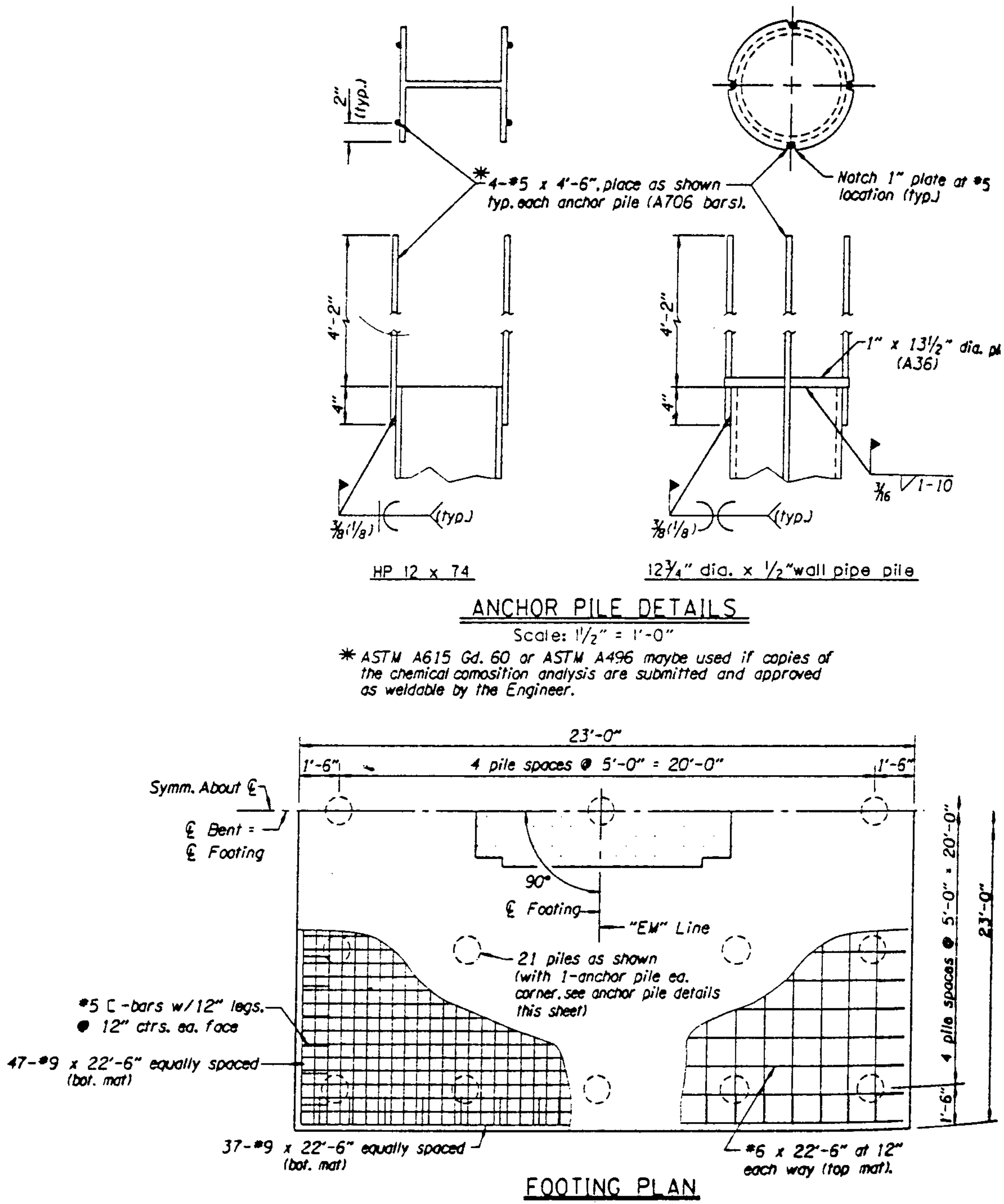
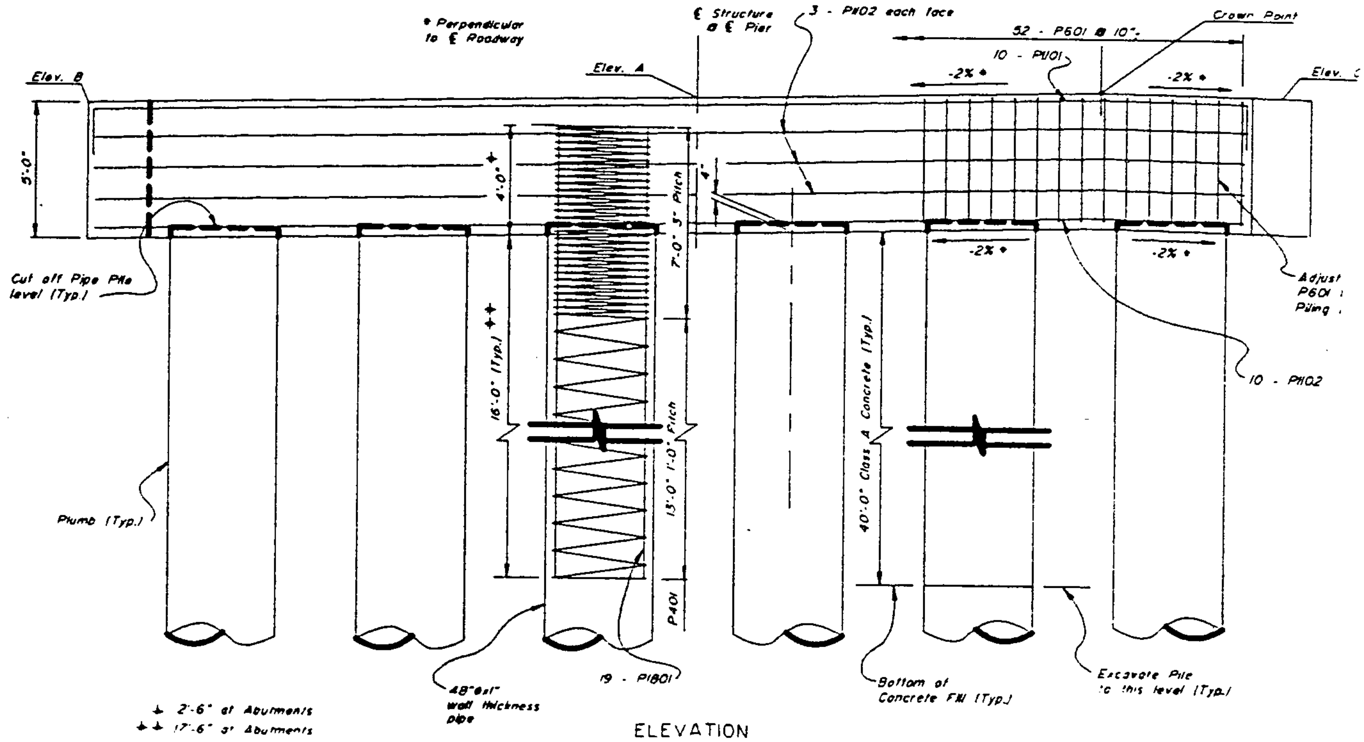
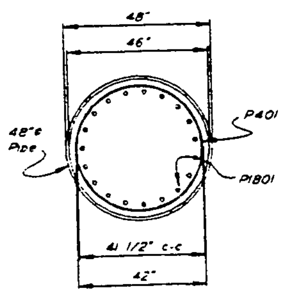
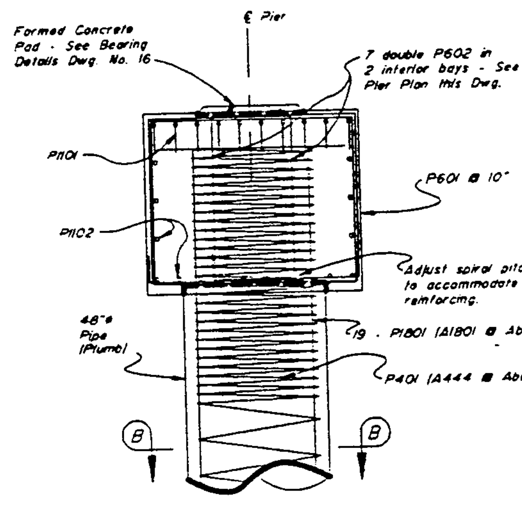


FIGURE 3-9B Oregon Detail 2







REINFORCING STEEL (One Pier)

MARK	SIZE	NO.	LENGTH	TYPE
P401	4	6	467'-10" *	Spiral *
P601	6	52	22'-4"	Bent
P602	6	28	18'-10"	Bent
P1101	11	10	46'-0"	Bent
P1102	11	16	42'-8"	—
P1801	18	114	21'-0"	Bent

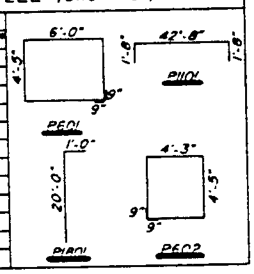


FIGURE 3-10 Alaska Detail 2

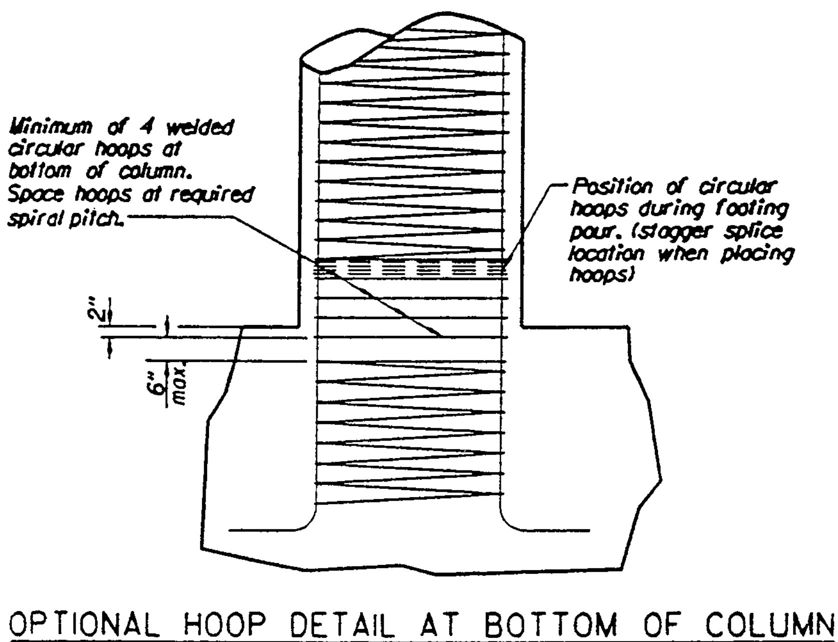


FIGURE 3-11 Oregon Detail 3

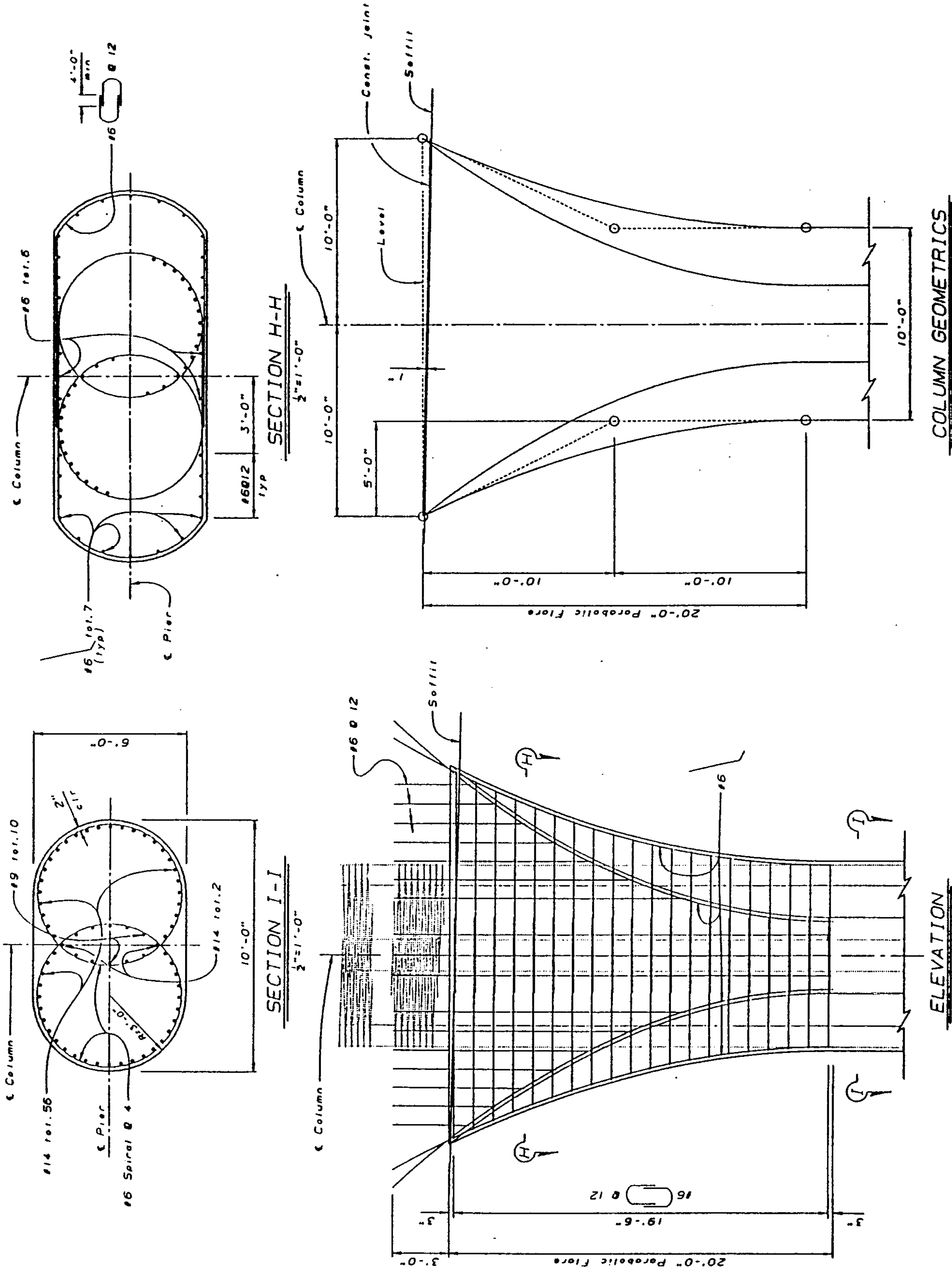


FIGURE 3-12 California Detail 6

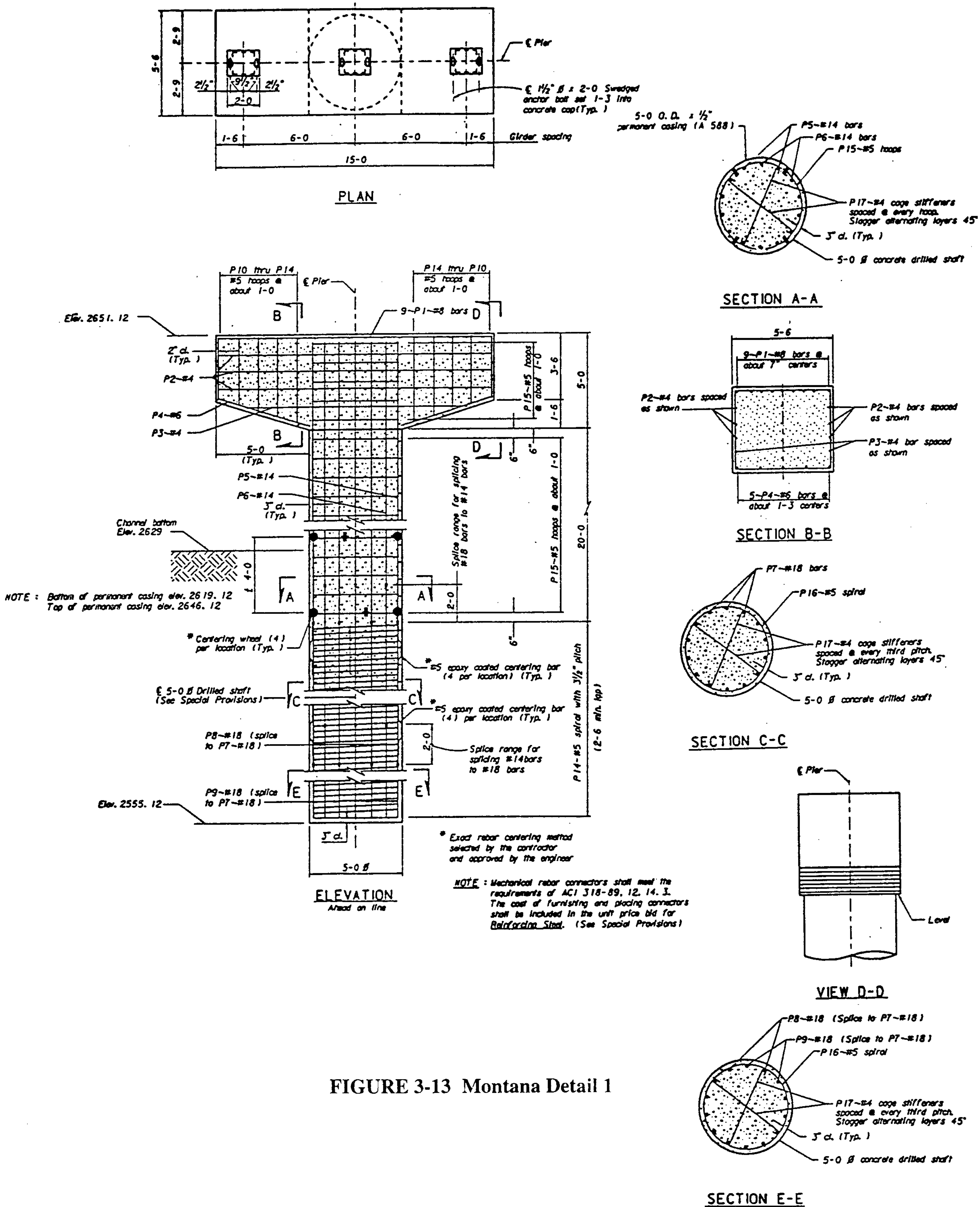
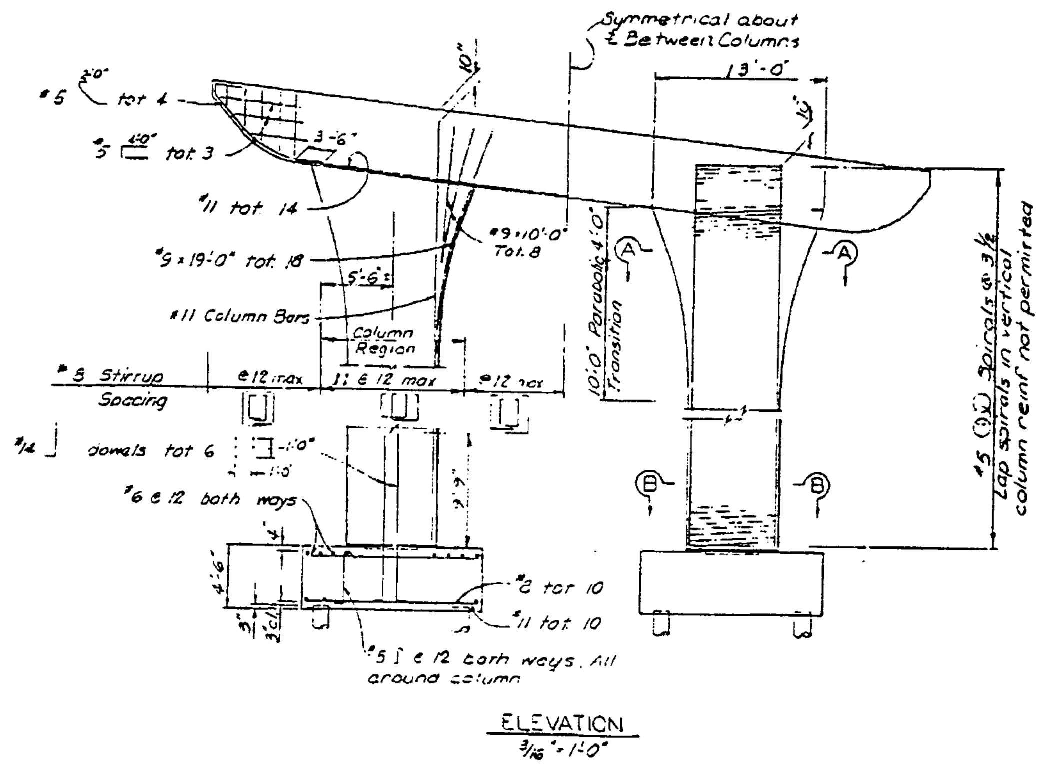
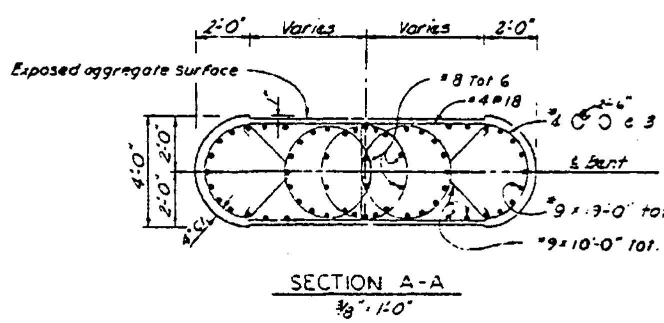


FIGURE 3-13 Montana Detail 1





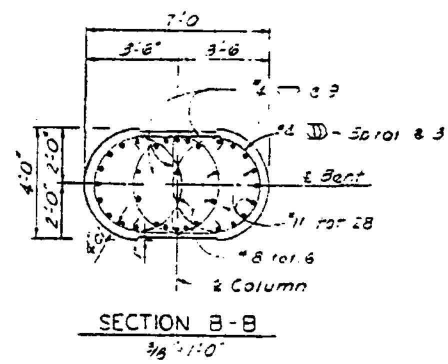


FIGURE 3-14 California Detail 7

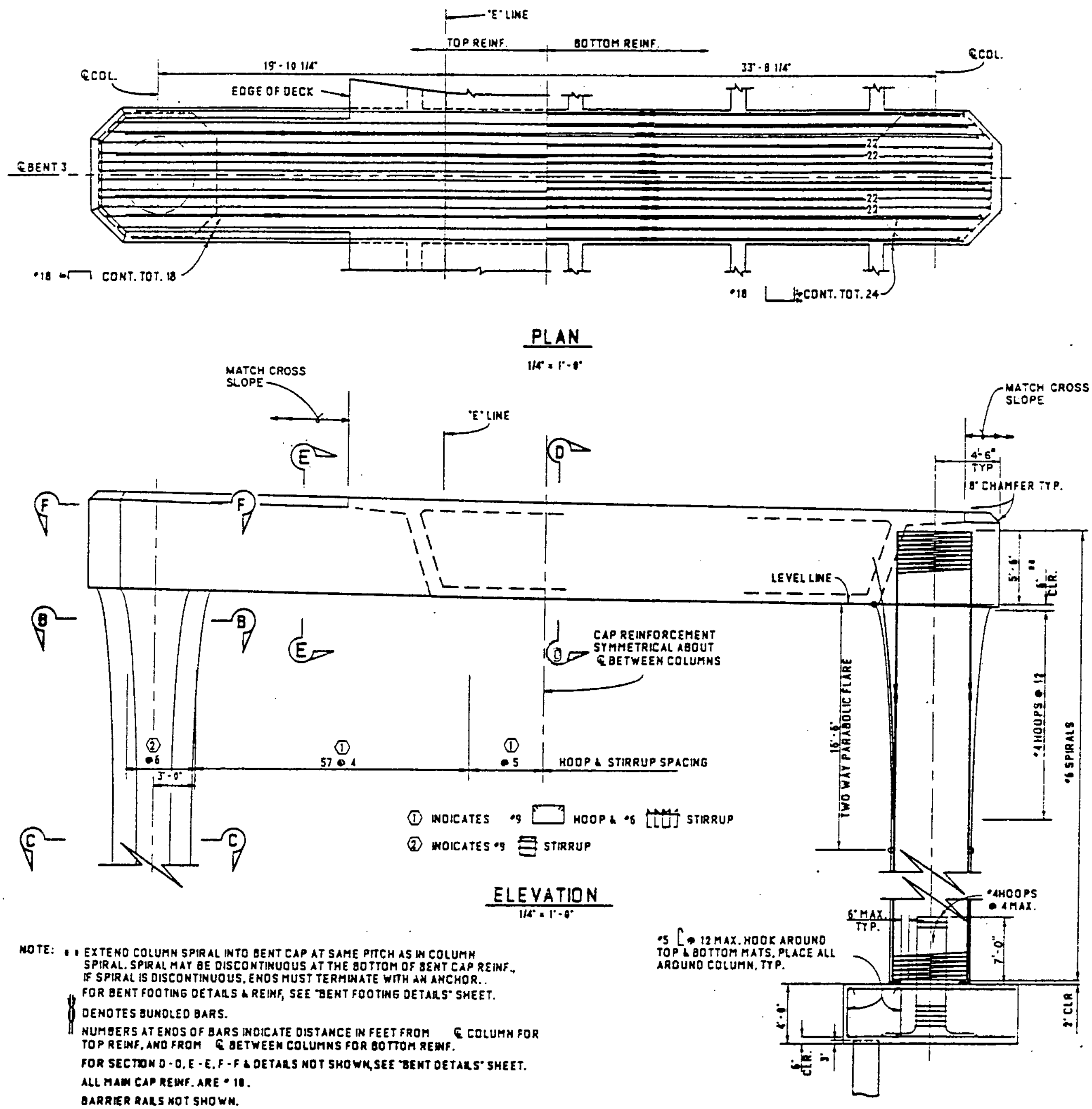


FIGURE 3-15 California Detail 8

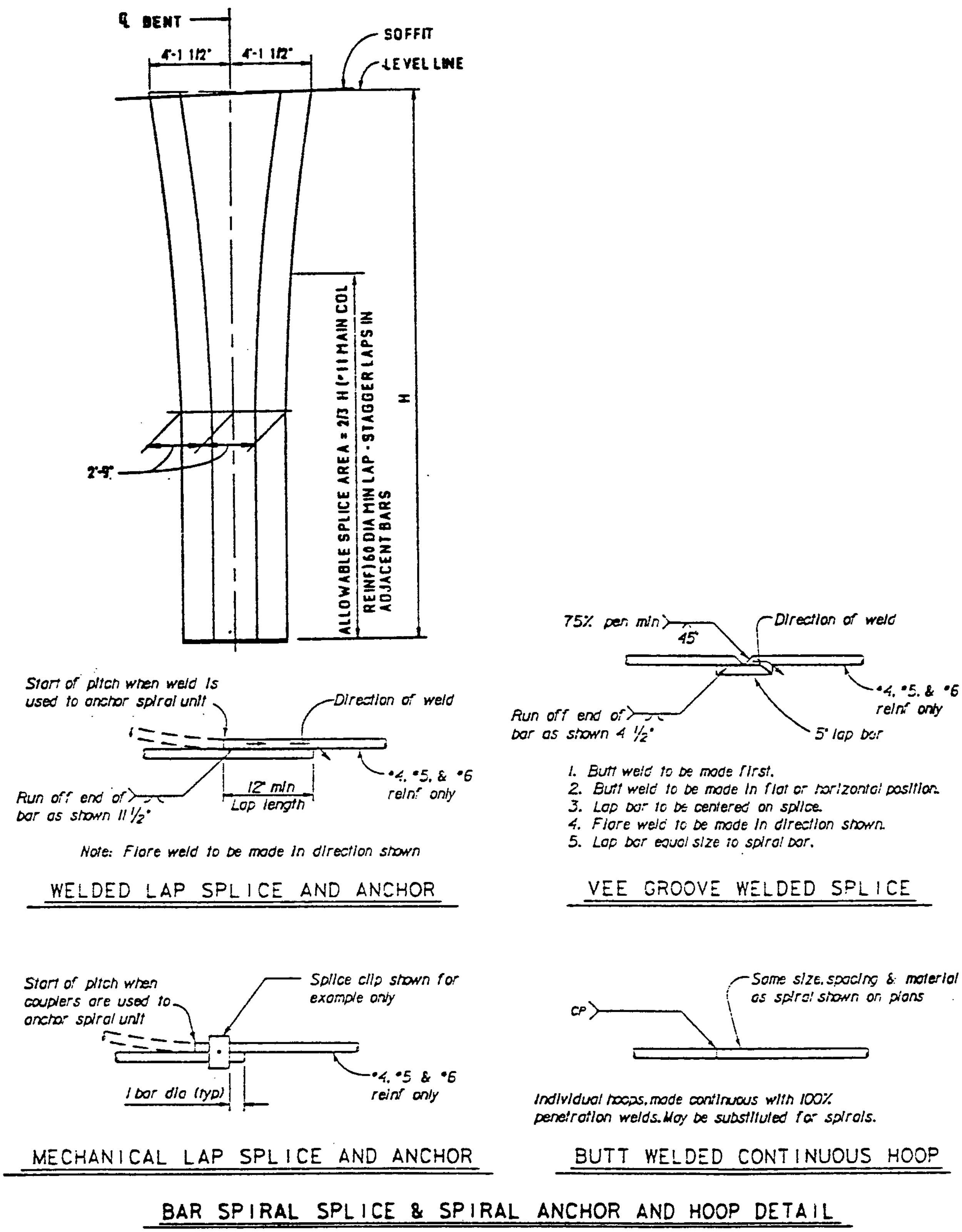


FIGURE 3-16 California Detail 9

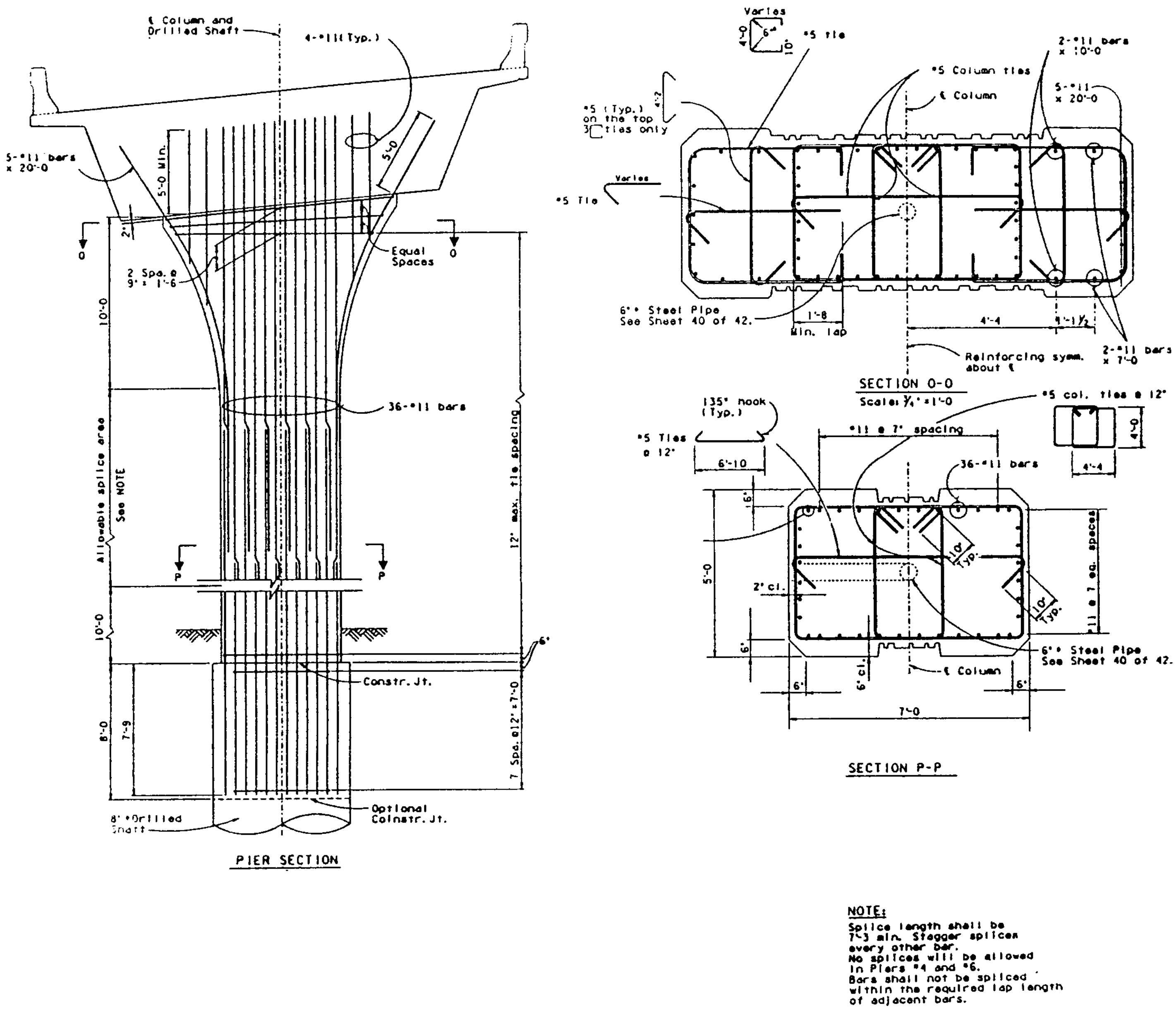
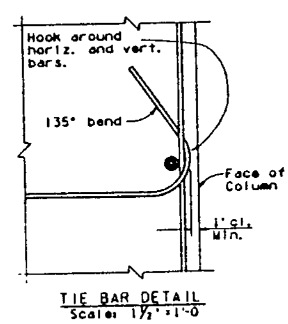


FIGURE 3-17 Arizona Detail 1



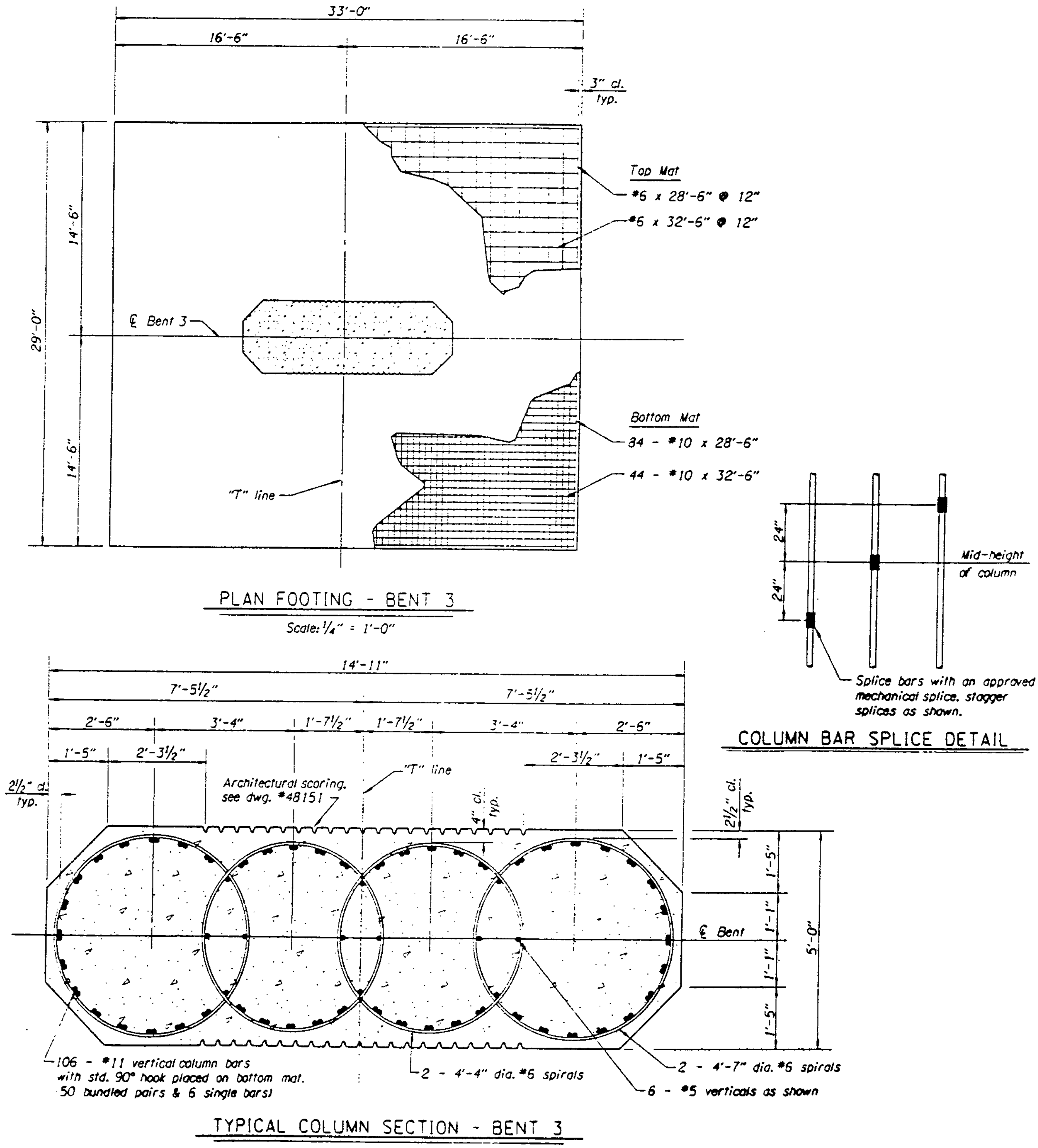


FIGURE 3-18 Oregon Detail 4

Advantages: By prohibiting the use of lap splices in column main longitudinal reinforcement, potential anchorage failures are avoided during column degradation from seismic shaking. The loss of the concrete cover around the column will not adversely affect the main reinforcement if the transverse reinforcement is welded or mechanically spliced.

Disadvantages: The column cage must be in position before the footing concrete is placed, or butt welding or mechanical splices must be used to join the column reinforcement to column reinforcement in the footing. Supporting tall column reinforcement cages can be difficult during construction.

Historical: Prior to the 1971 San Fernando earthquake it was common practice to splice column reinforcement to dowels cast into the footing. The recommendation to make column reinforcement continuous into the footing was one of the first changes implemented after this earthquake.

3.5 Column Reinforcement is Recommended to be Continuous Through Knee Joints

Description: In rigid frame structures where the vertical support member is continuous with the edge of the deck, such as outrigger bents and vehicle carrying culverts, the joint must be reinforced to develop the full nominal capacity of the joint. The joint must be able to resist moments and shears from horizontal and vertical loads through the joint. This often requires the vertical member reinforcement along the outside face of the support to bend and extend the full development length into the supported horizontal member.

The detailing of knee joints is not addressed in the current AASHTO or Caltrans' bridge codes, but is in the recently released LRFD Specifications (AASHTO, 1994).

See Figure 3-19.

Advantages: Extending the vertical reinforcement into the horizontal member helps develop a moment resisting ductile joint.

Disadvantages: During construction, the vertical bars that continue into the horizontal member must be bent to match the superstructure slope and camber. Since these bars usually have large diameters, it is difficult to field bend for alignment adjustments.

Historical: One of the causes that led to the collapse of the Oakland, California, Cypress Street Viaduct during the 1989 Loma Prieta earthquake, was the lack of reinforcement continuity through the joints. The vertical reinforcement continued straight up the outside face of the columns. Horizontal reinforcement terminated into the columns with a standard length hook. This detail did not provide the required strength and ductility during the severe seismic shaking.

This type of joint failure led directly to the addition of a specification in the new LRFD code (Section 5.11.1.2.4), which states that all joints must be detailed to assure continuity of reinforcement through the joint.

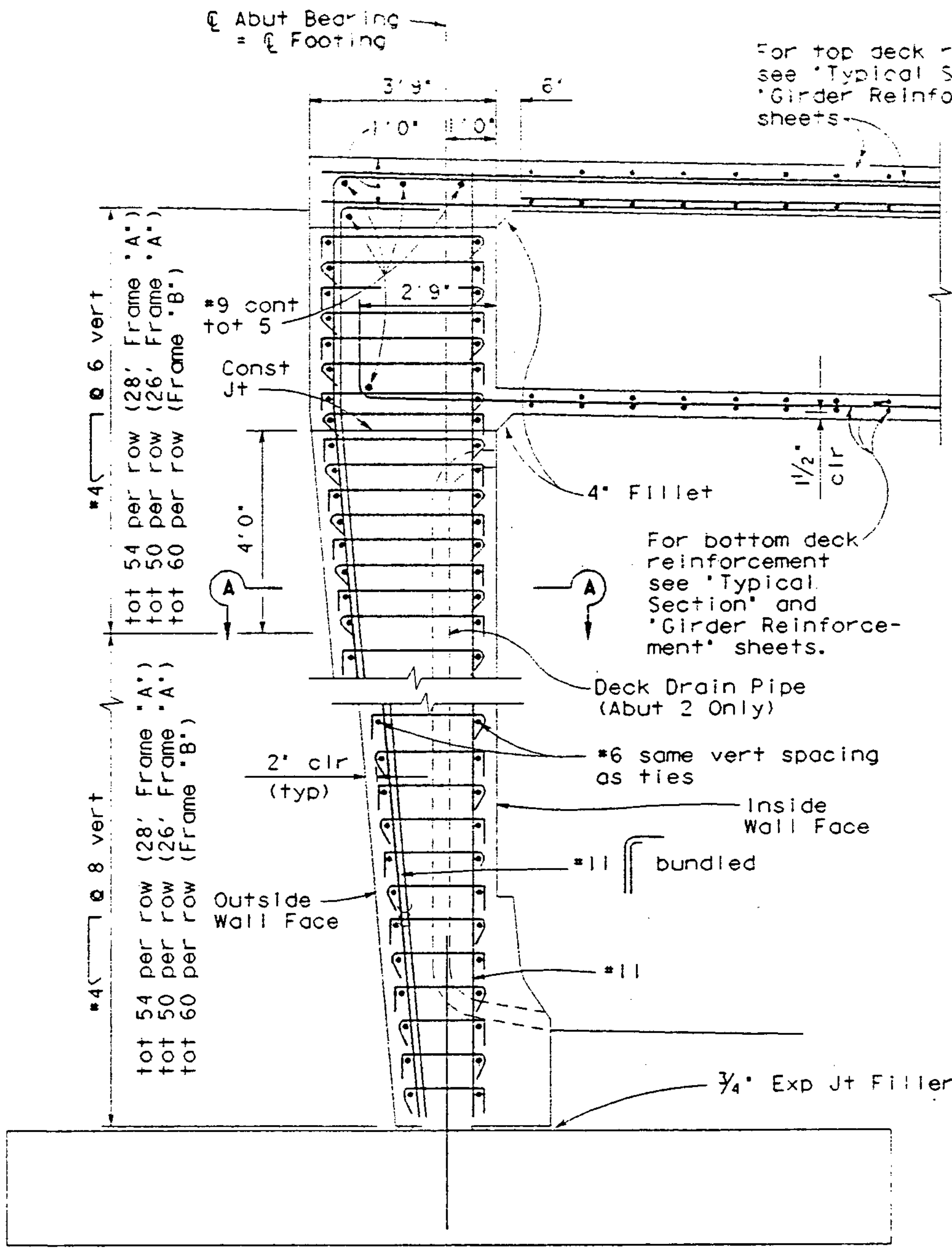


FIGURE 3-19 California Detail 10

3.6 Modification of Linearly Elastic Seismic Forces by R or Z Factors. (AASHTO Standard Specifications for Seismic Design of Highway Bridges 3.6; Caltrans' B.D.S. 3.21.1.2)

Description: Seismic design forces are determined in a similar manner by both the AASHTO and Caltrans' design specifications. Seismic design forces are determined from response spectrum analysis and modified by displacement ductility reduction factors. In the AASHTO approach, small to moderate earthquakes are resisted in the elastic range. Inelastic deformation is allowed for larger earthquakes. Response Modification Factors (R) are dependent on the component being designed, and range from 2 to 5 for columns and piers. In Seismic Performance Category A, only connections must be designed to resist earthquake forces. In Category B, axial, shear, and moment forces are modified by the R factor. In Categories C and D, only seismic moment forces are modified.

In California, bridges are designed to resist forces determined from response spectrum analysis for the maximum credible earthquake. The response spectrum forces are divided by the reduction/risk factor (Z) which is a function of the member component type and fundamental period of the structure. For example, the maximum Z factors for single and multiple column bents are 6 and 8, respectively. Pier walls are usually designed for a Z factor of 2 unless special tie criterion is met, in which case a ductility up to 6 can be used. Well-confined columns are expected to resist larger forces than they are designed for by accounting for ductile behavior.

Advantages: The method of taking elastic seismic forces and reducing them by a ductility factor greatly simplifies the analysis process. Elastic seismic forces may be calculated by single mode or response spectrum analysis. If R or Z factors are not used, nonlinear seismic analysis would have to be performed on the structure, which involves a more complicated analysis. Normally, conventional bridge structures with T>0.7 seconds produce linear analysis results comparable to non-linear analysis (Neward and Rosenblueth, 1971). Therefore, the response spectrum method is the preferred design method for these structures.

Disadvantages: All empirical methods inherently have approximations built into them. The seismic design accuracy is greatly dependent on the reduction (and risk) factor accuracy.

3.7 Multi-column Bent and Pier Wall Foundation Support Condition

Description: The AASHTO and Caltrans' Bridge Design Specifications do not make recommendations regarding the type of support condition to be used at the base of columns and pier walls. Caltrans' does, however, recommend that a multi-column bent be pinned to the foundation in Memo to Designers 6-1. This condition is attained by stopping the column main longitudinal reinforcement, providing dowels near the column center, and supporting the column on a footing bearing surface. Pier walls are pinned in the longitudinal direction by providing a single row of dowels and reducing the contact area with the footing. Arizona and Washington recommend that columns and pier walls be fixed to the foundation by continuing the longitudinal and transverse reinforcement into the foundation. Other states evaluate the support condition when a foundation is present on a case by case basis.

See Figures 3-1, 3-2, 3-3, 3-4, 3-5, 3-6, 3-8, 3-9, 3-14, 3-15, 3-19, and 3-20.

Advantages: The greatest advantage to pinning the base of the column or pier wall is the reduction in loads transmitted to the footing, resulting in reduced footing sizes. The pinned columns will also soften the response to an earthquake, effectively lengthening the longitudinal structure period. The longer period can be advantageous by moving the structure away from the peak of the response spectrum curve. An additional construction advantage is that the column cage does not have to be placed prior to the footing pour, only the shear key dowels.

Disadvantages: In pinned columns, larger seismic forces may be transferred to the abutments, and moments and displacements may increase at the top of the columns. Also, the column magnification factors applied to the service loads will increase due to the pinned base condition, which may increase the moment demand from service loads. If the design moment of the column increases, and subsequently the main longitudinal reinforcement must be increased, larger plastic hinging moments and shears may result. Depending on how the structure's fundamental period (T) is affected, it may become more prudent to fix the column to the footing. In addition, fixed columns offer greater redundancy in resisting lateral loads. Pinned columns will also have to be braced during construction.

Historical: In the past, column and pier wall supports have been pinned to the foundation as a means to reduce thermal stresses in multi-span rigid framed bridges. Some states have extended this philosophy to bridge seismic design.

3.8 Column Flares Reinforced with Minimal Transverse and Longitudinal Reinforcement

Description: Column flares are added for architectural enhancement of the bridge. The columns are usually only flared in one direction (transverse), but when required for aesthetics, may be flared in two directions. Reinforcement in the flare is usually nominal, with No. 6 or No. 7 longitudinal bars spaced around the flare parameter at about 12 inches, and having No. 4 ties at 12 inch spacing. In these columns, the flares are not expected to contribute to the column capacity during an earthquake. In Washington, and sometimes in California, the column flare is discontinued at the bottom of the cap by the use of expansion material. Washington also requires flare longitudinal reinforcement to be unbonded in the column-cap intersection region (to prevent strain hardening). This detail prevents the column flare from contributing to the column-cap joint capacity.

See Figures 3-12, 3-15, 3-17, and 3-21.

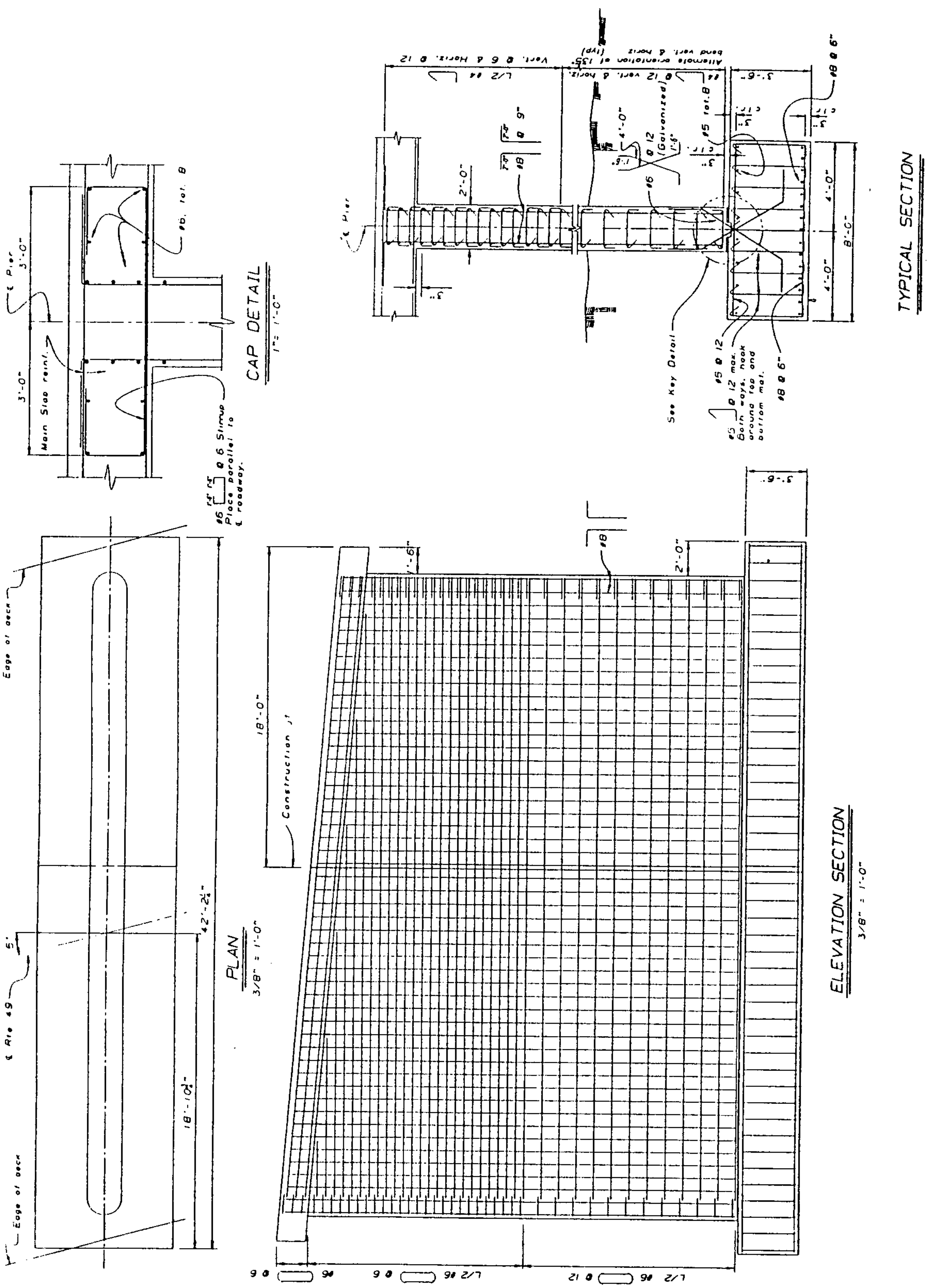


FIGURE 3-20 California Detail 11

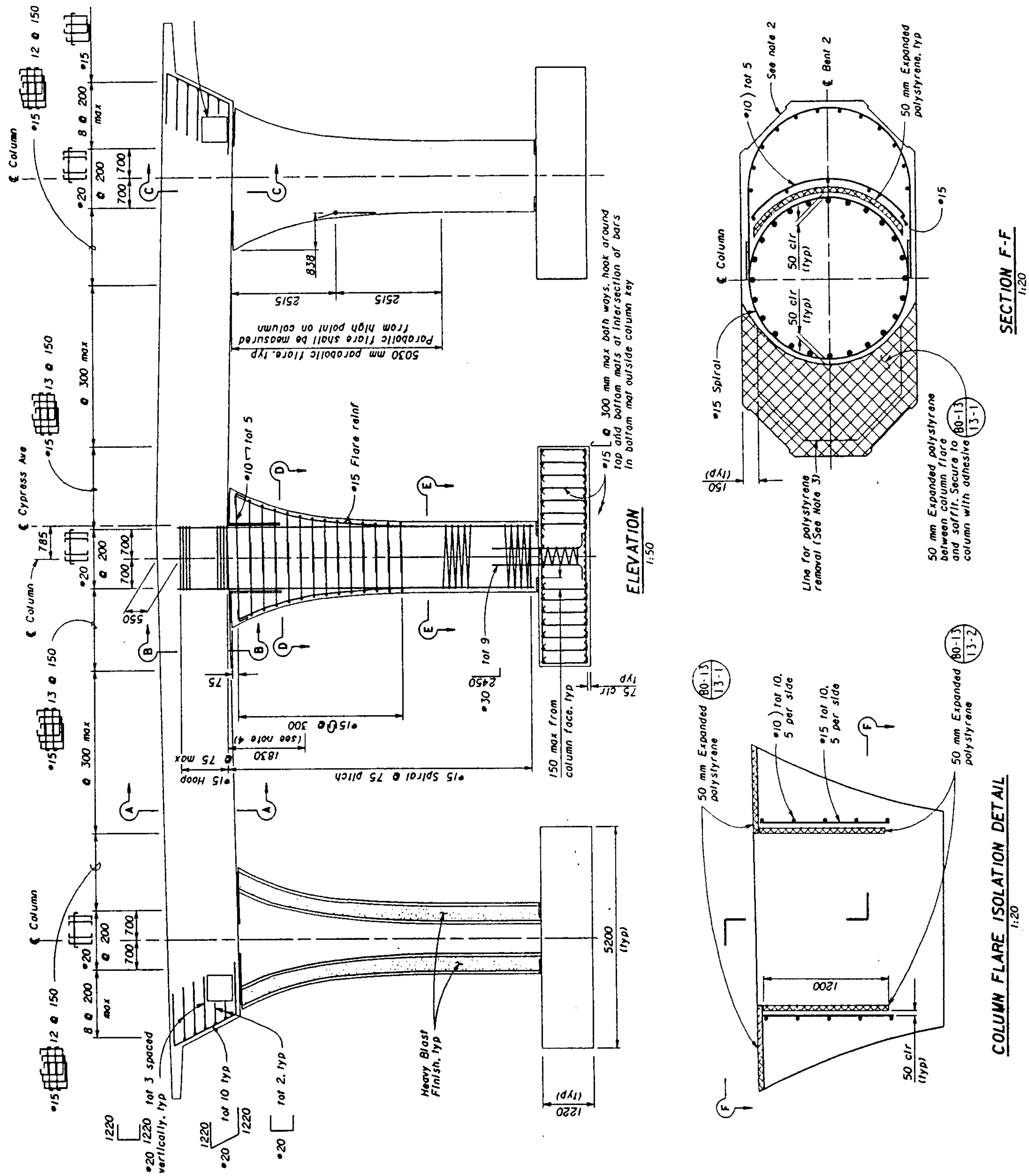


FIGURE 3-21 California Detail 12

Advantages: The column flares enhance the appearance of the bridge, creating a softer and more pleasing structure.

Disadvantages: The column must be designed to provide sufficient capacity both before and after an earthquake in which the flares fail. In addition, it may be difficult to accurately predict the ductility and plastic hinging capacity of lightly to moderately reinforced flares. If the flares do not fail as expected, the shear associated with the nominal moment capacity of the section may be significantly greater than that associated with hinging of the prismatic column section. This complicates the seismic analysis for both the column stiffness and the location along the column at which the plastic hinge may develop. The increased capacity of the column due to the flare may force the plastic hinge to form further down the column than expected, and result in larger plastic hinge moment and shear forces than predicted.

Historical: In California, the use of architectural flares began in the 1970's in locations of high pedestrian traffic and low speed vehicle traffic in an attempt to beautify the state highways. However, many of the early columns were highly reinforced even when the flares were not to be considered effective during an earthquake. During the Northridge earthquake in 1994, several architectural flares that were expected to become non-effective did not, causing plastic hinges to form much lower than expected on the column. Therefore, Caltrans is currently reviewing the appropriateness of using column flares.

3.9 Fully Confined Column Flares

Description: In these columns, the flares are designed to contribute strength during earthquakes. These flares are properly confined with ties and cross ties, and the flare longitudinal reinforcement tends to be larger size bars, such as No. 11. California and Nevada appear to be the only western states that have used fully-confined flares.

See Figure 19.

Advantages: Again, the aesthetic appearance of the bridge is enhanced. This detail can add significant strength to the column without increasing the column size. The flares can be especially useful on columns that are pinned on the bottom and all resisting moment capacity must be supplied at the top of the column.

Disadvantages: The designer must ensure that the confining steel has been properly detailed in order for the column to behave as a ductile unit. This will require that cross ties are included in the flare lateral reinforcement. Since the column has larger load carrying capacity than a column with only a confined core, the nominal moment capacity and, therefore, the plastic moment capacity will increase substantially. This will create much larger plastic shears, which will increase the lateral reinforcement requirements. In addition, the cross ties must be detailed to insure that they do not "slip down" as the cover concrete spalls during seismic activity.

When the column flares do not have a definitive transition point from prismatic to flared section, such as in a parabolic flare, it may become very difficult to predict the location at which a plastic hinge may form. This in turn further complicates the plastic shear design.

Caltrans bridge design is based on a "strong-bent cap weak-column" approach. Large column flares may make it difficult to force plastic hinges into the columns as opposed to the bent cap members. When the column section is increased in size, a careful analysis of the bent cap is warranted to insure that the cap is capable of resisting these large column moments. If a plastic moment is likely to develop in the cap, the cap must be resized or made more highly reinforced to force the plastic moment back into the column section. This may lead to impractical or uneconomical designs.

Historical: Same as minimally reinforced concrete flares. For the reasons listed above, Caltrans' has discouraged using this type of detail in recent years.

3.10 Distinction Between a Column and Pier Wall (2.5 Times Width). (AASHTO Standard Specification for Seismic Design of Highway Bridges 8.4.1, Caltrans' B.D.S. 7.5.1)

Description: A member is considered a column when the ratio of the clear height to width is greater than 2.5, otherwise, the member is considered a pier. A pier wall may be designed as a column about the weak axis and as a pier in the strong direction.

Advantages: Pier wall use is normally dictated by site conditions. One common location is in waterways. The pier wall prevents debris from becoming tangled on individual columns, which could restrict stream flow and significantly increase lateral loads on the foundation. A pier wall also decreases the impediment of stream flow, which helps to reduce backwater curves.

Disadvantages: In California, when a pier wall is designed with a ductility/risk (Z) factor less than 2, a thicker pier may be required. When Z factors are greater than 2, a substantial amount of ties are required.

3.11 Pier Wall Confinement Reinforcement (AASHTO Standard Specification for Seismic Design of Highway Bridges 8.4.2)

Description: Pier walls may be designed as columns about the weak axis and as piers about the strong axis. AASHTO requires a minimum reinforcement ratio, r, of not less than 0.0025 in both the horizontal and vertical directions. In California, if a pier wall is axially loaded below $0.4P_b$ (nominal axial strength at balanced strain) for Load Case VII, and is designed for a ductility of 2 or less, the wall is exempt from the stringent B.D.S. 8.18.2 confinement criteria. If the pier wall does satisfy this confinement criteria, the wall can be defined for ductility factors from 3 to 6 about the weak axis. The confining ties must have 135° hooks on one end and 90° hooks on the opposite end. (Caltrans' Memo to Designers 6-5)

See Figures 3-8, 3-20, 3-22 and 3-23.

ELEVATION / DIMENSION TABLE								
PIER	(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)
1	4143.66	4143.85	4143.88	4143.74	11'-3½"	29'-0½"	4128.38	4139.66
2	4144.14	4144.33	4144.36	4144.22	9'-11¾"	28'-4¼"	4130.18	4140.14
3	4144.62	4144.81	4144.84	4144.70	10'-1¾"	28'-5¼"	4130.48	4140.62
4	4145.10	4145.29	4145.32	4145.18	11'-7¾"	29'-2¾"	4129.48	4141.10
5	4145.59	4145.78	4145.81	4145.67	11'-5"	29'-1½"	4130.18	4141.59
6	4146.07	4146.26	4146.29	4146.15	14'-1"	30'-6½"	4127.98	4142.07
7	4146.55	4146.74	4146.77	4146.63	15'-8"	31'-4¾"	4126.88	4142.55
8	4147.03	4147.22	4147.25	4147.11	16'-0¾"	31'-7½"	4126.98	4143.03
9	4147.51	4147.70	4147.73	4147.59	18'-5¼"	32'-10¾"	4125.08	4143.51
10	4147.99	4148.18	4148.21	4148.07	18'-8½"	33'-0½"	4125.28	4143.99
11	4148.48	4148.67	4148.70	4148.56	18'-8½"	33'-0½"	4125.78	4144.48
12	4148.96	4149.15	4149.18	4149.04	19'-11¾"	33'-8½"	4124.98	4144.96

NOTES

1. FOR VERTICAL PILE CUT-OFF ELEVATIONS, SEE SHEET 5. PIER PILE CUTOFF ELEVATIONS HAVE A 1"(±) CONSTRUCTION TOLERANCE
2. PIERS 1 THRU 5 SHALL BE BUILT FOLLOWING DETAIL 4 ON SHEET 5. PIERS 6 THRU 12 SHALL BE BUILT FOLLOWING EITHER DETAIL 4 OR 5 ON SHEET 5, EXCEPT THAT ANY PIER SHALL BE BUILT USING DETAIL 4 ON SHEET 5 IF THE WATER DEPTH AT THE PIER IS LOW (3' OR LESS) OR AS DETERMINED BY THE ENGINEER. FOR ESTIMATING ONLY, PIERS 1 THRU 5 ARE CONSIDERED BUILT IN THE DR
3. FOR ESTIMATING PURPOSES, SEAL CONCRETE WILL BE REQUIRED TO BE PLACED TO ONE FOOT ABOVE AVERAGE (NORMAL) WATER ELEVATION OF 4135.0' AT PIERS 6 THRU 12. IF AT TIME OF CONCRETE PLACEMENT WATER LEVEL IS LOWER, SEAL CONCRETE SHALL BE PLACED TO ONE FOOT ABOVE ACTUAL WATER LEVEL.
4. ACTUAL STREAMBED ELEVATION AND BOTTOM OF PIER ELEVATION SHALL BE FIELD VERIFIED BY THE CONTRACTOR AT EACH PIER PRIOR TO ORDERING PIER REINFORCEMENT.
5. PLACE P9~#8 AT PIERS 9, 10, 11, AND 12. PLACE BETWEEN P8~#8 AND P3~#8, 14" FROM P8~#8.

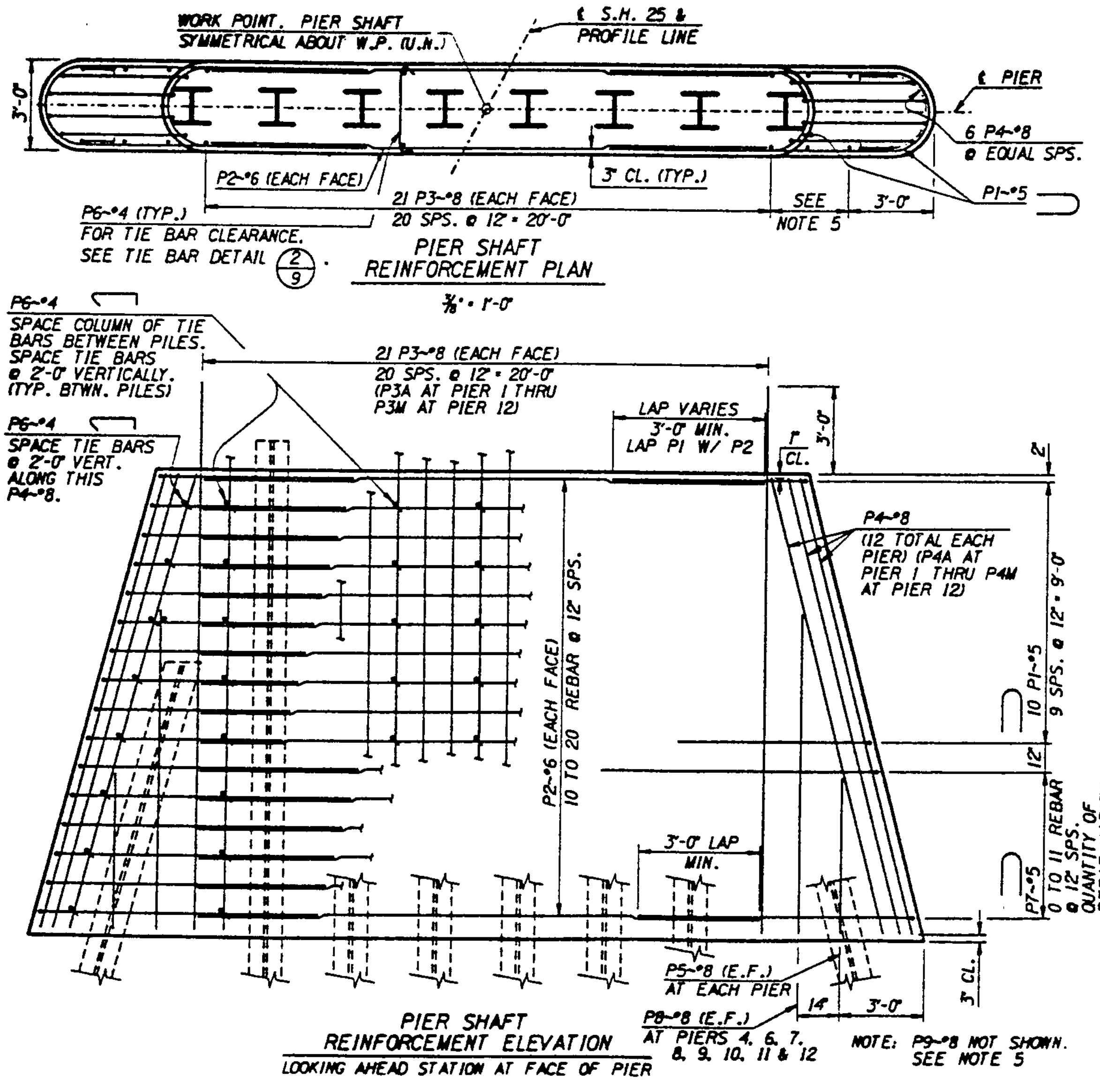


FIGURE 3-22A Idaho Detail 2

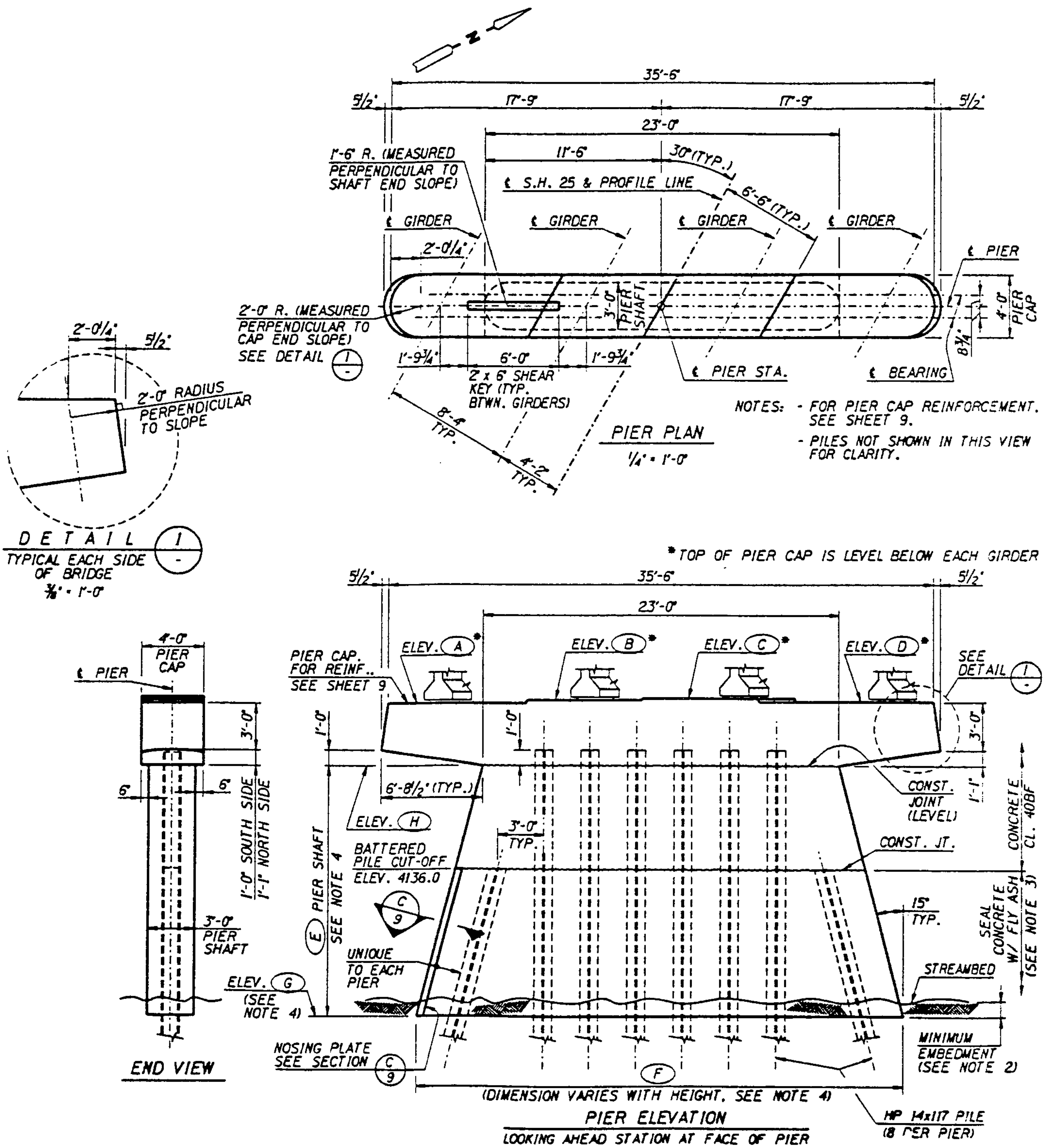


FIGURE 3-22B Idaho Detail 2

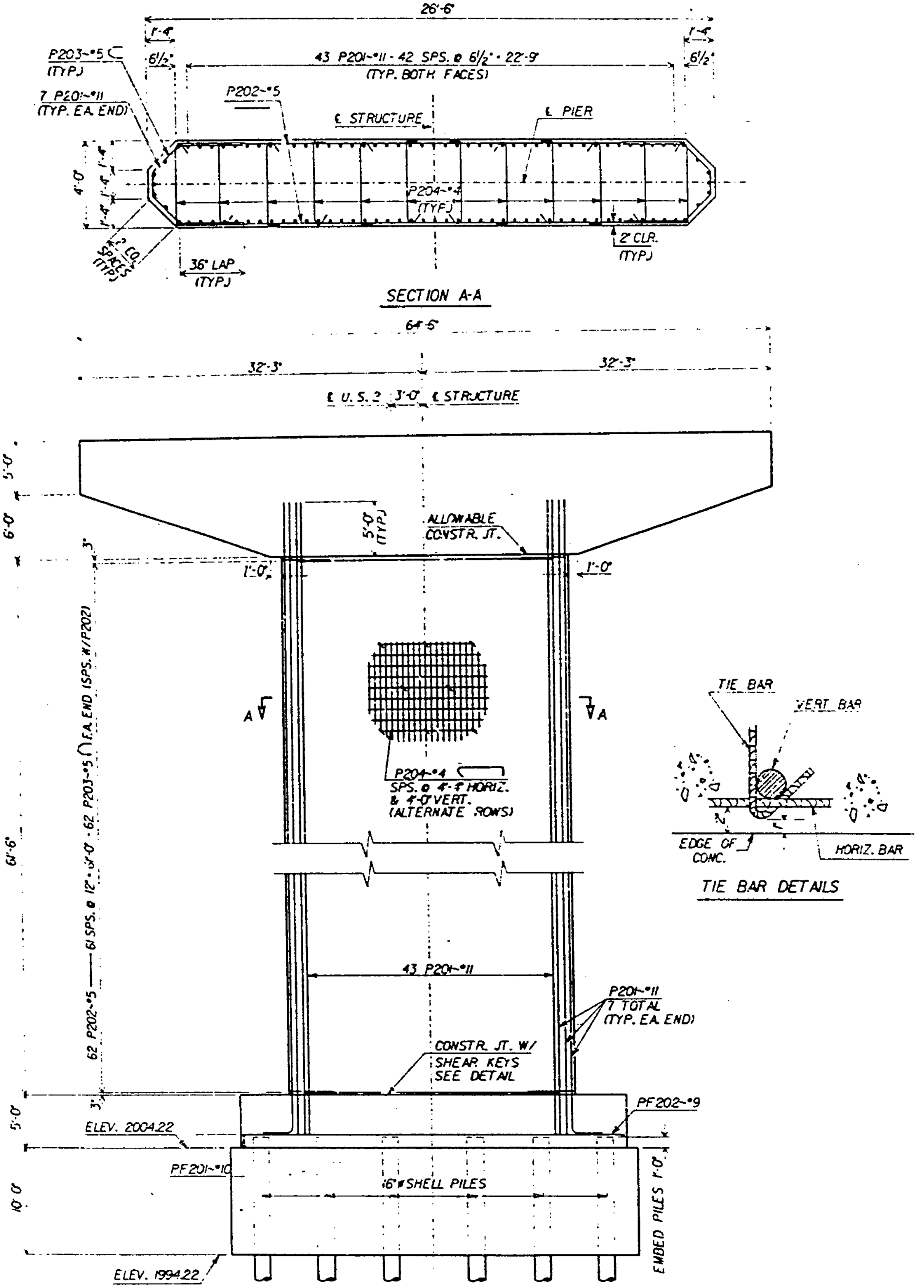


FIGURE 3-23 Idaho Detail 3

Advantages: By properly confining the main longitudinal reinforcement, pier walls can be designed for much larger seismic loads without increasing the longitudinal steel in the wall. This in turn reduces the footing size for fixed connections. Since ties are used for confinement, the tie spacing can easily be varied to optimize the spacing in and outside the plastic region. The ties are placed so that the 135° and 90° hooks are alternated horizontally and vertically. The use of 90° hooks allows easy placement, instead of having to "thread" the ties down from the top of the cage.

Historical: The San Fernando earthquake brought about the use of seismic tie details for pier walls. Initially 135° hooks were used on both ends of the ties, but this created serious constructibility problems. Either field bending or rebending of the ties was required, both of which sometimes resulted in breaking off the hook. Therefore, Caltrans began to require 135° hook only on one end, and 90° hook on the other. The 135° hook is required to be alternately placed on each row of the pier wall main longitudinal reinforcement.

3.12 Discourage Partial Height Pier Walls in Multi-column Bents

Description: Partial height pier walls or curtain walls are sometimes placed between columns of multi-column bents. These walls may or may not be structurally attached to the columns. The partial height walls are often used in waterways or as traffic barriers between the columns.

See Figures 3-24 and 3-25.

Advantages: The partial height walls often perform the same function as full height pier walls. In waterways, the partial walls help prevent debris from snagging on the columns, while saving considerable material above the high waterline where the wall is no longer needed.

Disadvantages: The partial height walls can dramatically change the predicted behavior of columns during seismic events. In the longitudinal direction, the wall will have only a small effect on the structure response. However, in the transverse direction, the wall can force a plastic hinge to develop at or near the top of the partial height wall. The walls also greatly stiffen the column for the height of the wall, which will attract larger forces to the columns. Because the column effectively has different column lengths in the longitudinal and transverse direction, the plastic hinge region could occur anywhere along or above the partial height wall.

Historical: Due to their unpredictable behavior during earthquakes, California and Alaska highly discourage the use of partial height walls. The recommendation is based on the unsatisfactory performance of these walls in past earthquakes. Poor performance occurred during the 1994 Northridge earthquake (e.g., Bull Creek Canyon Channel Bridge; see Figure 3-25).

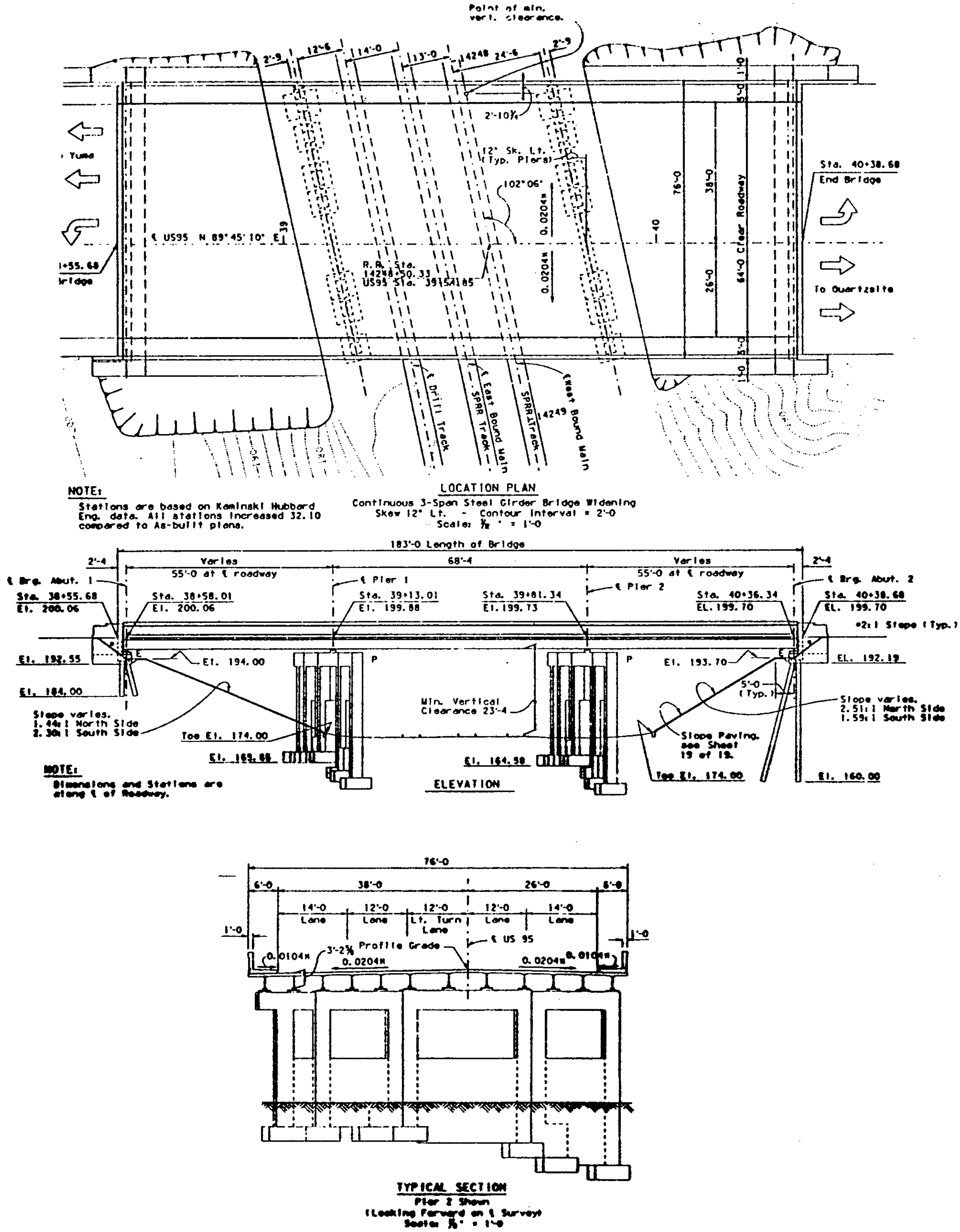


FIGURE 3-24 Arizona Detail 2

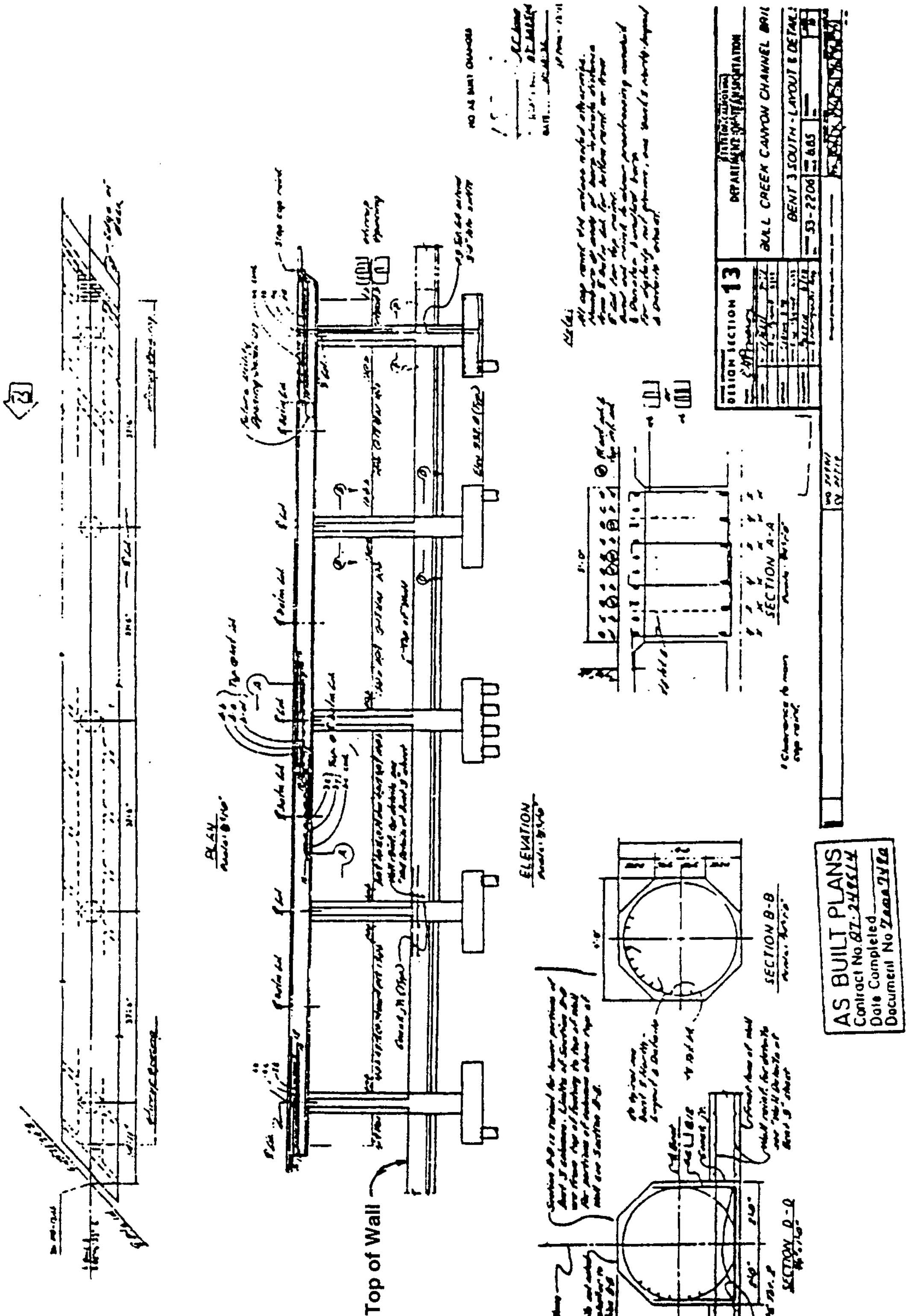


FIGURE 3-25 California Detail 13

SECTION 4
BENT CAP DETAILING ISSUES

4.1 Bent Cap is at Least 6 Inches Wider Than the Column

Description: When reinforced concrete columns are used, the reinforced concrete bent cap should be at least 3 inches wider than the column on each side of the cap.

See Figures 3-1, 3-3, 3-5, 3-6, 3-8, 3-10, 3-22, and 4-1.

Advantages: The larger cap section provides for better column main longitudinal reinforcement anchorage and reduces the conflict between column and bent cap reinforcement. The larger cap section also helps to provide greater cap flexure capacity than the column, forcing plastic moments to develop in the column.

4.2 All Bent Cap Longitudinal Steel Placed Inside Shear Reinforcement

Description: The bent cap longitudinal reinforcement is placed inside the shear stirrups and cross ties.

See Figures 3-1, 3-3, 3-4, 3-6, 3-8, 3-10, 4-1, 4-2, and 4-3.

Advantages: The stirrups provide confinement and, therefore, a degree of ductility in the bent cap. When closed stirrups are used, the cap torsional capacity is increased.

Disadvantages: A small increase in the construction effort is required to place the longitudinal reinforcement.

4.3 Longitudinal Bent Cap Reinforcement

Description: When force reversal under seismic loading can be expected in the bent cap, the cap reinforcement must be sufficient to resist these forces. In moderate to high seismic zones, this may require that substantial bent cap longitudinal reinforcement, both top and bottom mats, be continuous along the entire length of the cap. In California, all longitudinal cap reinforcement must be continuous, and lap splices are not allowed. Therefore, butt welds or mechanical couples are required to splice the reinforcement. If it is physically impossible to place all bars continuously along the cap, the maximum possible percentage of both the top and bottom mat reinforcement should be made continuous. The AASHTO Seismic Specifications do not require additional reinforcement than that required in the Standard Specifications to be made continuous in the cap.

See Figures 3-1, 3-3, 3-4, 3-6, 3-8, and 3-15.

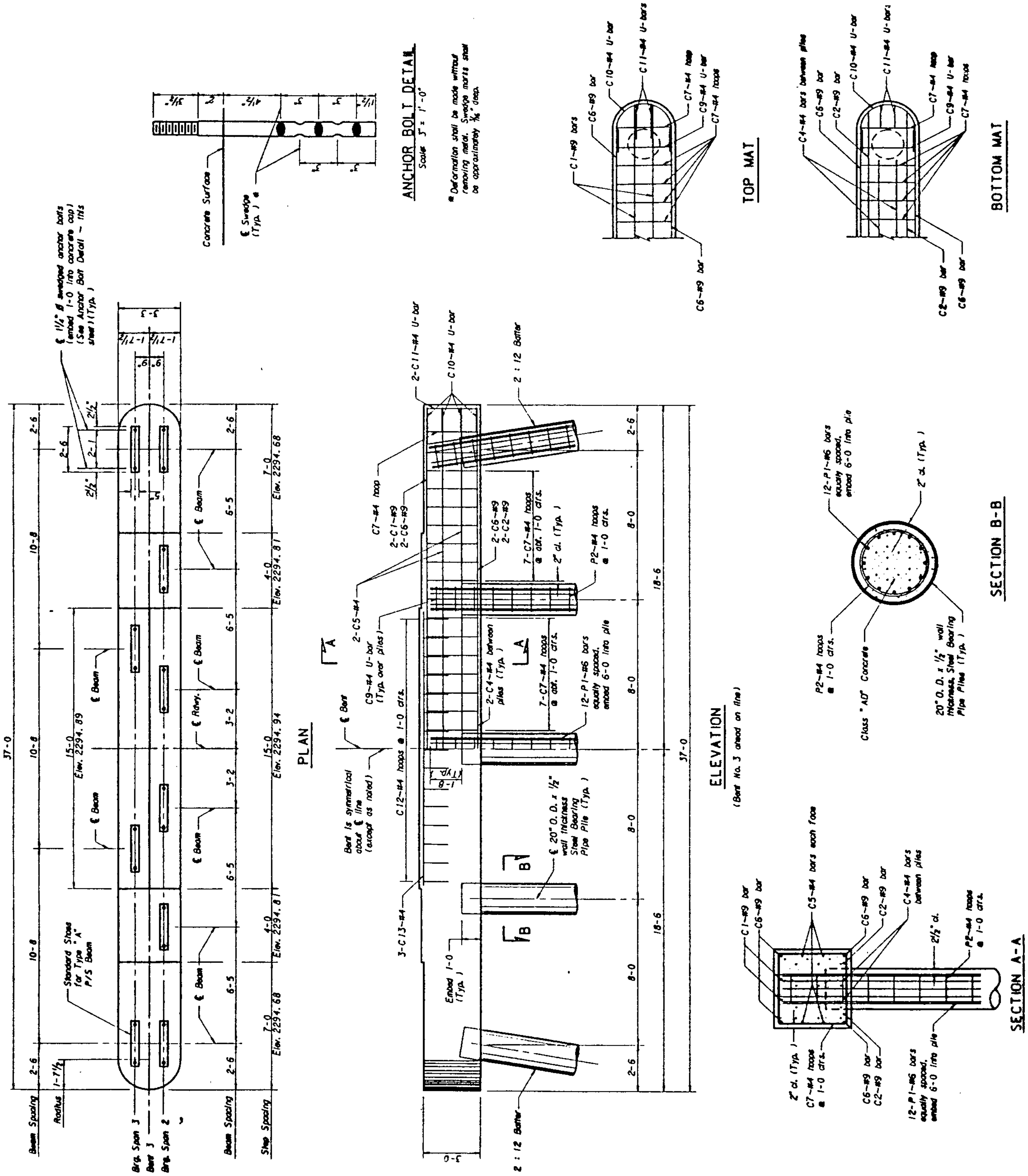


FIGURE 4-1 Montana Detail 2

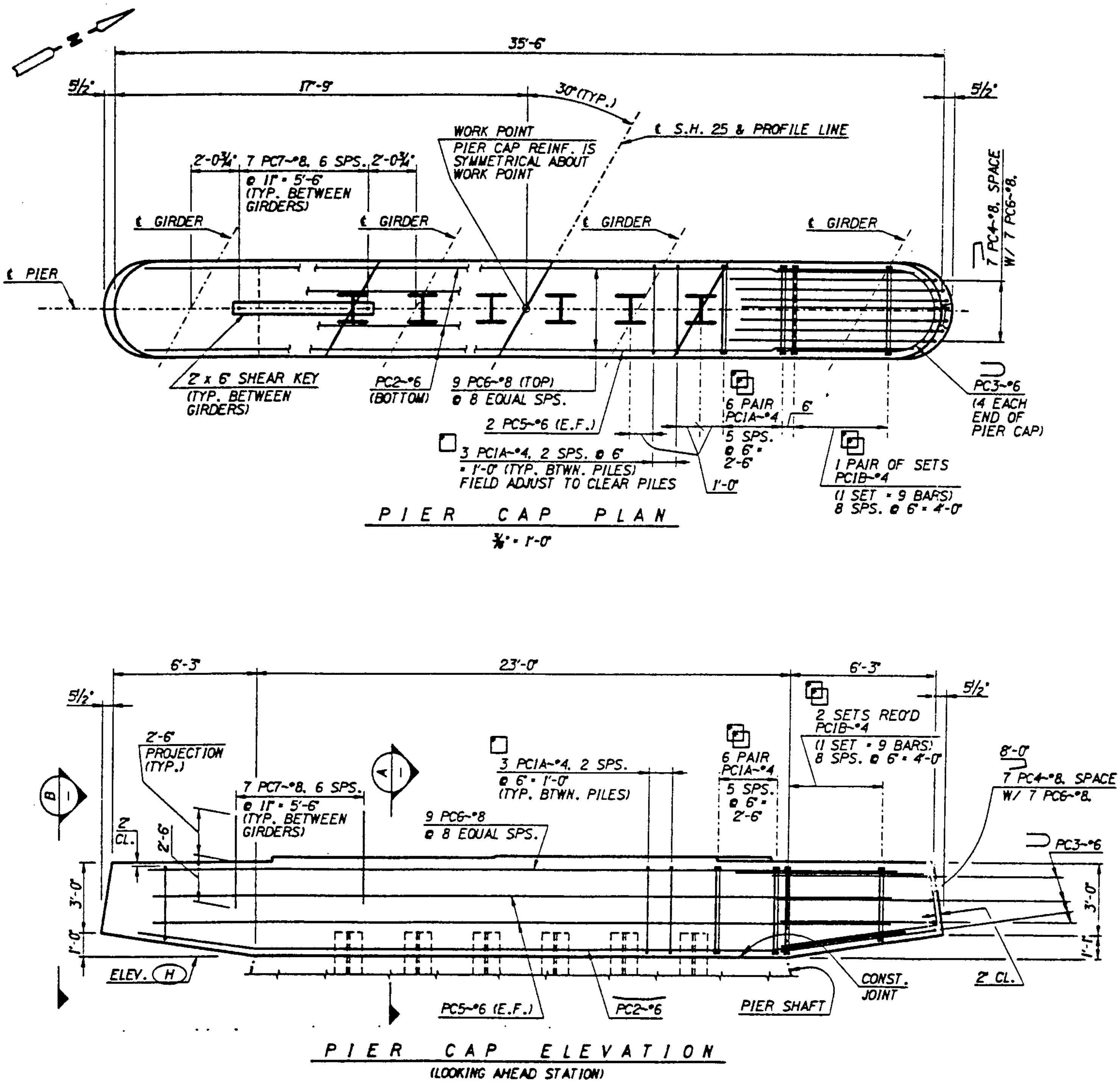


FIGURE 4-2A Idaho Detail 4

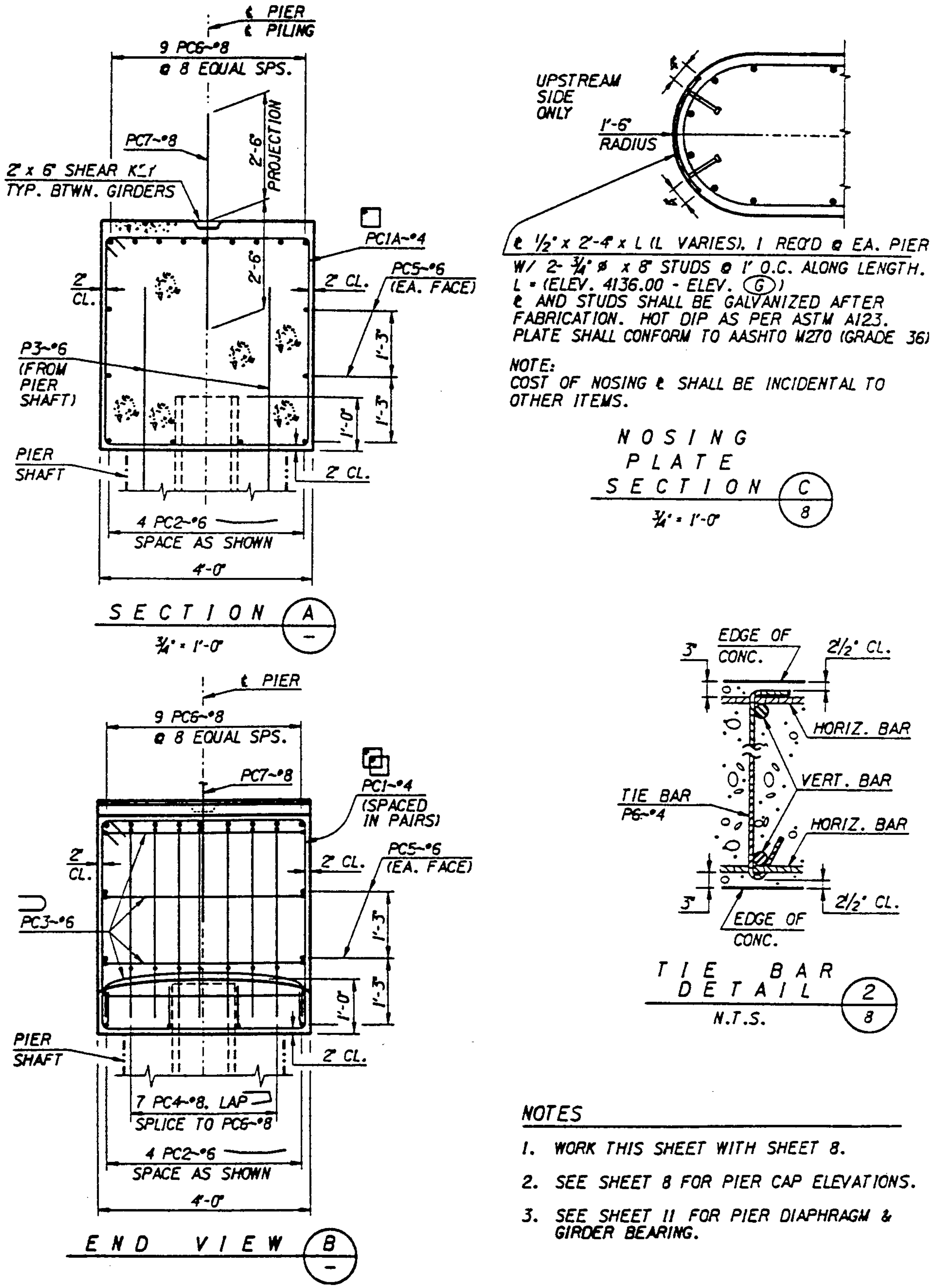


FIGURE 4-2B Idaho Detail 4

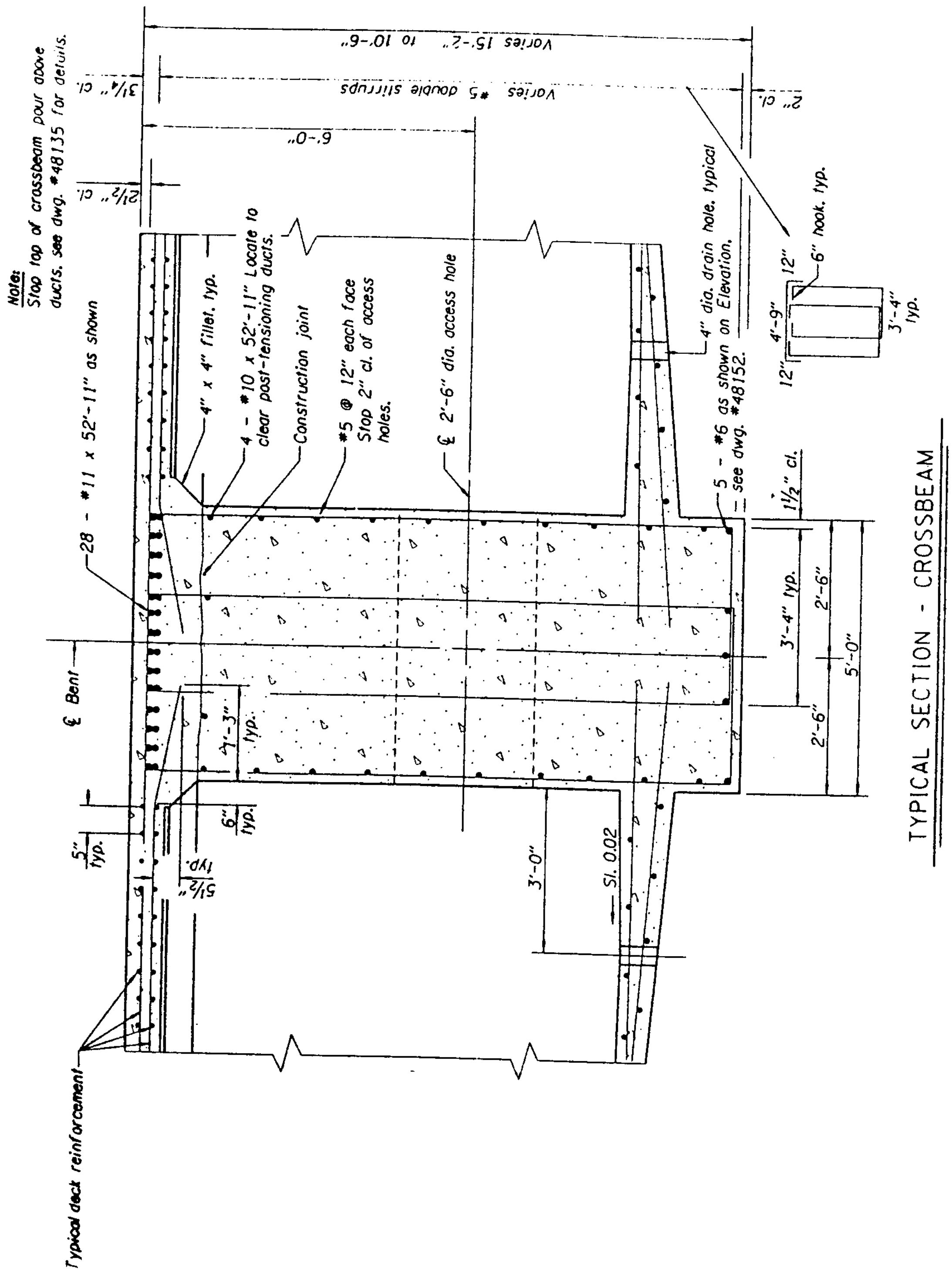


FIGURE 4-3 Oregon Detail 5

Advantages: The positive moment capacity at the face of the column support is significantly increased when all bottom mat reinforcement is continuous. In multiple column bents the increased strength helps prevent plastic hinges from forming at this location during seismic shaking.

Disadvantages: Placing the bars may become very difficult, especially through the column core area. Shifting of some column bars from their planned location may be necessary. Bar lengths must be increased, which may require welds or mechanical connectors, in order to span across the entire bent cap. In order to pass the cap longitudinal bars through the column core, bending of the cap bars may be necessary.

Historical: After the 1989 Loma Prieta earthquake, it became apparent that most bent caps were seriously under-reinforced at the cap bottom near the column. Under static load conditions, negative moment exists in the cap on each side of the column. Therefore, most bottom cap bars have traditionally been stopped at the face of the column in order to avoid conflicts with the column main longitudinal reinforcement. It is now known that large positive moments may develop at this location of multiple column bents due to seismic loading. This may cause plastic hinges to develop in the cap well before the column yields. On some bridges, this can lead to collapse mechanisms forming well below the column demand moment.

4.4 Shear Reinforcement Within Column Intersection Area. (Joint Shear)

Description: Caltrans' Interim Memo to Designers 20-6 requires a minimum transverse reinforcement equal to 0.25% of the joint area within the column joint region, where the column core frames into the bent cap. The joint region is defined as the region within the column core length along the cap centerline, and the width of the cap. Typically, the stirrup groups in this region are spaced at twelve inches on center. As many stirrup legs as necessary are placed across the cap to produce the minimum reinforcement ratio. The AASHTO Specifications do not specify additional reinforcement requirements for joint shear.

See Figures 3-1, 3-3, 3-5, and 3-10.

Advantages: The increased stirrup reinforcement in the joint helps prevent the sudden and potentially catastrophic failure of the joint.

Disadvantages: In addition to the column main longitudinal and spiral reinforcement, as well as the continuous bent cap longitudinal reinforcement stated above, the requirement for additional stirrups to be placed in the joint region creates a very congested area. Placement of all the reinforcement during construction becomes a very difficult, if not impossible, task.

Historical: The University of San Diego, under the direction of Dr. Nigel Priestly, performed tests on joint vulnerability due to shear following the Loma Prieta earthquake. These tests indicated that column-to-cap joints may be susceptible to sudden shear failure. In order to insure that this type of catastrophic failure does not occur, Caltrans increased the required shear reinforcement within the joint region. The current requirement of 0.25% reinforcement is still under review. When all ongoing testing and research is complete, Caltrans is planning to reevaluate the minimum reinforcement requirements for the purpose of writing a final specification.

4.5 Torsional Reinforcement, Especially in Outrigger Bents

Description: In outrigger bents, large torsional moment in the cap can exist. Neither Caltrans or AASHTO address torsion in their specifications. Therefore, requirements based on the American Concrete Institute (ACI) code are followed for beam torsion design (ACE, 1995). In order to resist the torsional moment, closed rectangular hoops are used in place of open leg shear stirrups.

Torsion can rarely be addressed without considering the combined effect of shear and moment. The reinforcement percentage in the cap will be dependent on the torsion interaction with these other forces.

Caltrans design is based on the columns hinging before the bent cap. Therefore, the bent cap torsional moment capacity must be larger than the column longitudinal moment in order to force plastic hinging in the columns. This may require outrigger bent caps to be quite large. Because of the difficulties involved in forcing plastic hinges into the columns, and the complex interaction of torsion with shear and moment forces, Caltrans discourages the use of outrigger bent caps.

See Figure 3-15.

Advantages: By following the ACI code, the torsional moment can be adequately accounted for and resisted by the torsion reinforcement.

Disadvantages: Closed hoops are more difficult to install during construction.

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SECTION 5
FOOTING DETAILING ISSUES

5.1 Minimum Top Mat Reinforcement Required in Footings

Description: All foundations, including spread footings supporting pinned columns, are required to have a minimum top mat reinforcement percentage.

See Figures 3-1, 3-2, 3-3, 3-4, 3-5, 3-8, 3-9, 3-14, 3-15, 3-18, and 3-20.

Advantages: This requirement is to prevent the footing from cracking during seismic loading. When columns are fixed to the footings, sizable negative moments can be generated at the face of the column. However, all footings must be able to resist column uplift forces, which also generate negative moments at the column face. By preventing the footing from cracking during seismic activity, detrimental effects to the footing shear capacity is prevented.

Disadvantages: Slightly more reinforcement is required in the footing, as well as slightly more difficult to construct.

Historical: The top mat reinforcement requirement was another of the detailing changes brought about by the 1971 San Fernando earthquake. During this earthquake, column reinforcement pulled out of footings at the I-5/14 Interchange. The addition of the top mat of reinforcement was one of the steps taken, in addition to requiring column transverse reinforcement to continue into the footing, to prevent a repeat of this failure type.

5.2 Footing Shear Stirrups

Description: The AASHTO specifications do not require footing shear reinforcement in the footing. The Caltrans' Specifications require shear stirrups within the distance d (core diameter of the column) from the face of the column. These stirrups must begin within 6 inches from the column face. The maximum spacing is 12 inches, and minimum #5 bar must be used.

See Figures 3-1, 3-3, 3-5, 3-8, 3-14, 3-15, and 3-20.

Advantages: Most footings are designed for the concrete to completely carry the shear force in the footing. The addition of the shear stirrups provides ductility in the footing during seismic loading. The stirrups help prevent the footing from degrading, preventing potential footing shear failure and possible loss of bond strength of the embedded column main longitudinal reinforcement. An additional benefit is the footing reinforcement top mat can be easily supported by the stirrups. For this reason, the stirrup bars are usually placed throughout the entire footing.

Historical: The footing stirrup requirement was also brought about by the 1971 San Fernando earthquake. Prior to this event, stirrups were seldom placed in footings in California.

5.3 Pile Tension

Description: The AASHTO specifications has some provisions for considering foundation uplift and proper development of the pile uplift forces into the footing cap. These provisions will have to be supplemented to properly address the issue of tension in piles. AASHTO Standard Specifications for Seismic Design of Highway Bridges 6.3.1(B) allows foundation uplift up to one-half of an end bearing foundation pile group or up to one-half of the contact area of a spread footing provided that foundation soils are adequate.

Caltrans' B.D.S. 4.3.4.6 allows friction piles to be considered to resist seismic tension loads up to 50% of the pile ultimate compression load. In order to rely on tension in the pile adequate pile anchorage into the footing or cap, and adequate pile tensile strength and soil-column skin friction must be present.

See Figures 3-9, 3-22, 5-1 and 5-2.

Advantages: Since the piles carry both tension and compression forces, a fewer number of piles can be used in the pile cap which produces the same total capacity as a footing with compression piles only. This results in a smaller footing size as well.

Disadvantages: The total uplift force on the pile is difficult to predict and control. Therefore, the pile-to-cap connection must be designed to resist the maximum possible tension force unless the possibility of a load fuse (i.e. zero tension capacity) at the top of the pile has been considered in the design. Not only must the connection be designed for the uplift load, but the pile itself must be able to resist tension.

End bearing piles can only be relied on to resist the weight of the pile. Skin friction piles should only be considered tension piles if they are placed in cohesive soils, but not if placed below the water line. Testing may be required to insure that the piles can resist the tension force assumed in design.

5.4 Design Footings for Lesser of Plastic Hinge and Unreduced Elastic Earthquake Forces (AASHTO Standard Specifications for Seismic Design of Highway Bridges 4.8.6, Caltrans' B.D.S. 3.21.7)

Description: The required footing seismic capacity may be based on either the plastic moment capacity at the base of the column and the unreduced elastic earthquake forces. The smaller of these two forces is recommended to be used in design. The controlling seismic force is compared to service load forces for the final footing design.

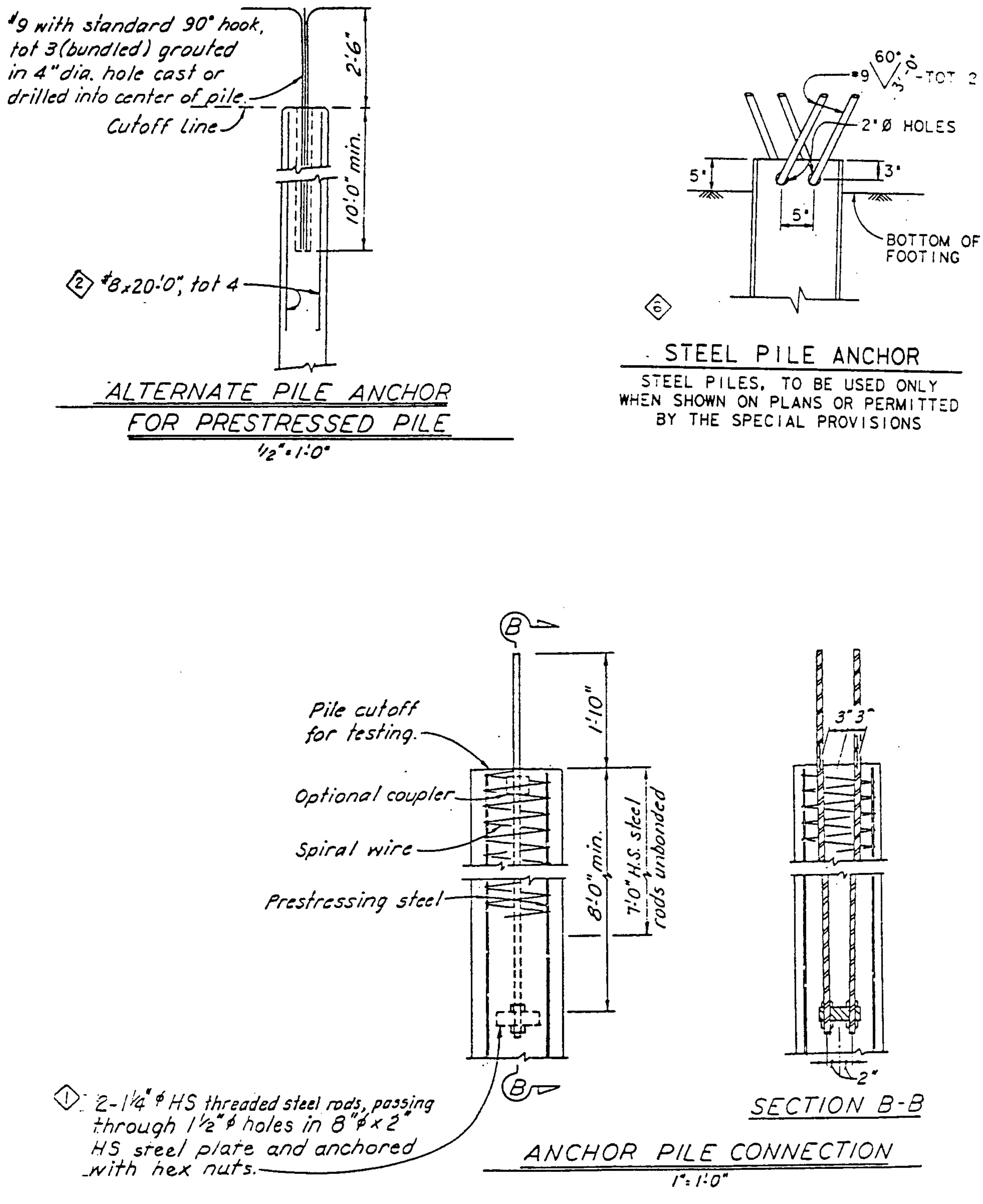
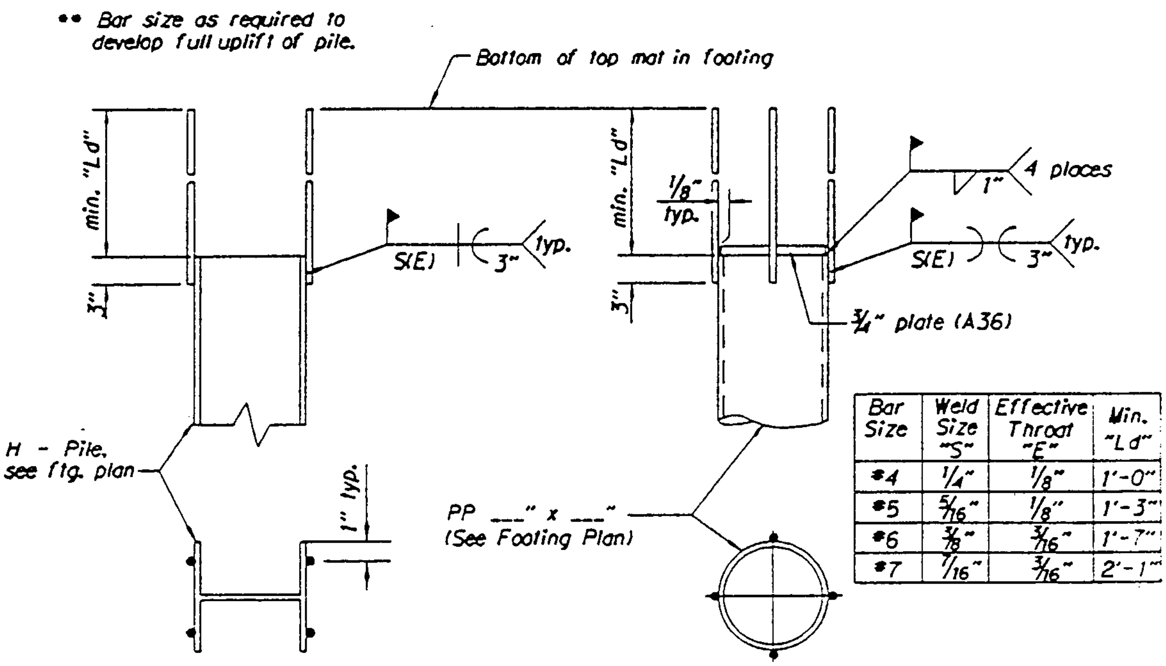


FIGURE 5-1 California Detail 14



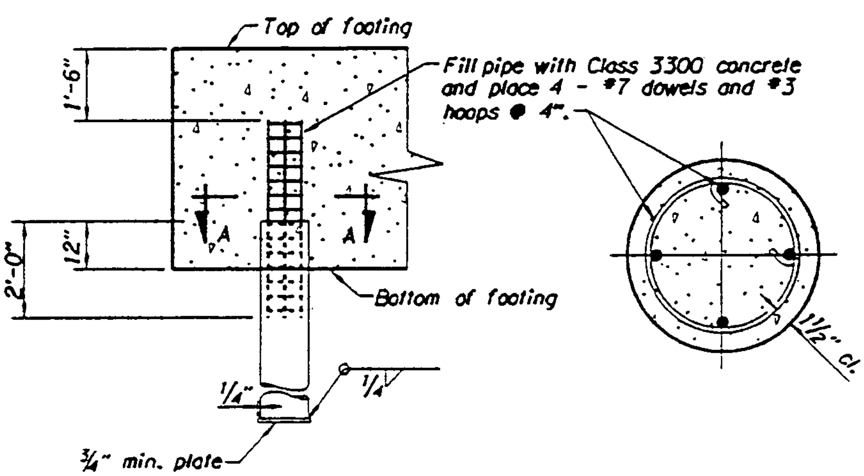
Bar Size	Weld Size "S"	Effective Throat "E"	Min. "Ld"
*4	1/4"	1/8"	1'-0"
*5	5/16"	1/8"	1'-3"
*6	3/8"	3/16"	1'-7"
*7	7/16"	3/16"	2'-1"

STEEL H-PILE

STEEL PIPE PILE

* ASTM A706 shall be used, except ASTM A615 Grade 60 or ASTM A496 may be used if copies of the chemical composition analysis are submitted and approved as weldable by the engineer.

ANCHOR PILE DETAILS



FILLED PIPE PILE ANCHOR DETAILS

FIGURE 5-2 Oregon Detail 6

Advantages: The footing is kept to the smallest possible size required to resist the maximum potential load. Caltrans has recognized that there is no benefit to designing footings for elastic earthquake forces if the column plastic capacity can not transfer this force to the footing.

Historical: Along with the detailing changes implemented after the 1971 San Fernando earthquake, this footing design philosophy was implemented.

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SECTION 6
CONSTRUCTION MATERIAL ISSUES

6.1 A 706 Steel Reinforcement is Primarily Specified for 60 ksi Bars (Caltrans' Special Provision 52.60 B)

Description: Recently, Caltrans' is requiring reinforcement bars in most concrete components to be low alloy steel designated by ATM A 706. This is especially required for the larger size bars, No. 11 or larger.

Advantages: The A 706 steel is approximately 50% more ductile than A 615 steel for No. 11, 14 and 18 bars, based on ASTM testing. This will obviously lead to more ductile columns and joints. In addition, the A 706 steel is easier to weld, and due to its greater ductility fewer bars are broken during fabrication bending and field rebending.

Disadvantages: The major drawback is the substantially increased cost of the A 706 steel. The steel is also not as widely available as the traditionally used A 615 steel. However, with the increased demand for A 706 steel, it is becoming more readily available.

History: Following the 1971 San Fernando earthquake, Caltrans began to investigate and recommend the use of A 706 steel in place of A 615 steel. However, only No. 6 reinforcement bars and smaller were available at the time. Therefore, Caltrans did not pursue the issue further. Following the 1989 Loma Prieta earthquake, Caltrans again began to investigate the use of A 706 steel. In order to promote and eventually require its use, Caltrans contacted reinforcement suppliers and requested that the A 706 steel be made available for larger size bars. Now that the reinforcement is being required, its availability is increasing.

SECTION 7
CONCLUSION

The requirements for seismic design of bridges in the western states, which are in moderate to high seismic zones, are more rigorous than those used for the design of eastern bridges. Many bridges in the West fall into Seismic Performance Categories C and D.

Seismic design of bridges in the western United States currently follows the AASHTO Specifications for Seismic Design of Highway Bridges in conjunction with state specific policies. Caltrans has developed a seismic design specification which it uses for California bridge design. Caltrans has been conducting research and intensely studying earthquake effects on bridges since the 1971 San Fernando (Sylmar) earthquake, the 1989 Loma Prieta earthquake, and the 1994 Northridge earthquake. The lessons learned from these earthquakes, and subsequent research, have shaped Caltrans' current seismic design requirements. California's experience and detailing requirements influenced the development of the AASHTO Standard Specifications for Seismic Design of Highway Bridges.

The bridge pier types, details and issues outlined in this report are representative of the practices and seismic policies in the western U.S. Typical highway bridge designs in the West are different from those in the East. Many western bridges have concrete superstructure that are cast monolithically with single or multi-column bents. Some states that construct those types of bridges are Arizona, California, Oregon and Washington. However, many western bridges like those in the east, construct steel or concrete superstructures supported on bearings. The pier caps are supported on piers, single or multi-column bents. In this report, the column and pier design and detailing issues were divided into these subdivisions:

- Column Spiral Reinforcement
- Column Shear Reinforcement, Within and Outside the Potential Plastic Hinge Zone
- Spirals, Hoops or Tie Requirements to Continue into Bent Caps and Footings of Fixed Columns
- Column Reinforcement Lap Splice Restrictions
- Column Reinforcement Recommendations through Knee Joints
- Modification of Linearly Elastic Seismic Forces by R or Z Factors
- Multi-column Bent and Pier Wall Foundation Support Conditions
- Column Flares Reinforced with Minimal Transverse and Longitudinal Reinforcement
- Fully Confined Column Flares
- Pier Wall Confinement Reinforcement
- Discourage Partial Height Pier Walls in Multi-column Bents

In addition, the report has included bent cap detailing issues, footing detailing issues and construction material issues. The report gives comparisons between Caltrans and other western states' seismic design procedures. For example, in designing the support condition at a foundation, Caltrans recommends that multi-column bents and pier walls be pinned to the foundation. Arizona and Washington recommends that columns and pier walls be fixed to the foundation.

Other western states evaluate the support condition when a foundation is present on a case-by-case basis. Also, advantages, disadvantages and historical background is given for each detailing issue.

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