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Establish Representative Pier Types for Comprehensive Study: Eastern United States

by

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Technical Report NCEER-96-0005

May 28, 1996

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

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16. Abstract (Limit: 200 words)

This report describes bridge types and seismic design and detailing procedures typical of the eastern part of the United States. The report contains examples taken from state bridge plans and actual bridge designs. Some of the examples comply with current seismic provisions, while others represent older designs made before these requirements were introduced. Advantages and disadvantages of various bridge configurations and details with respect to seismic behavior are discussed. Historical accounts of changes in bridge design and detailing practices are also included. The seismic design of bridges in the eastern part of the United States follows the AASHTO Specifications for Seismic Design of Highway Bridges in conjunction with state specific policies that address the unique conditions of each state. Most bridges in the east fall in Seismic Performance Categories A and B, but in many cases part or all of the requirements of Seismic Performance Category C are also followed. The bridge pier types included in this report are divided into pile bents, column bents and solid wall piers. The bent cap types include rectangular caps, inverted T-caps and hammerhead caps. The report highlights the unique nature of the seismic response of various bridge configurations and details. Common and dissimilar elements between east and west construction practices are identified.

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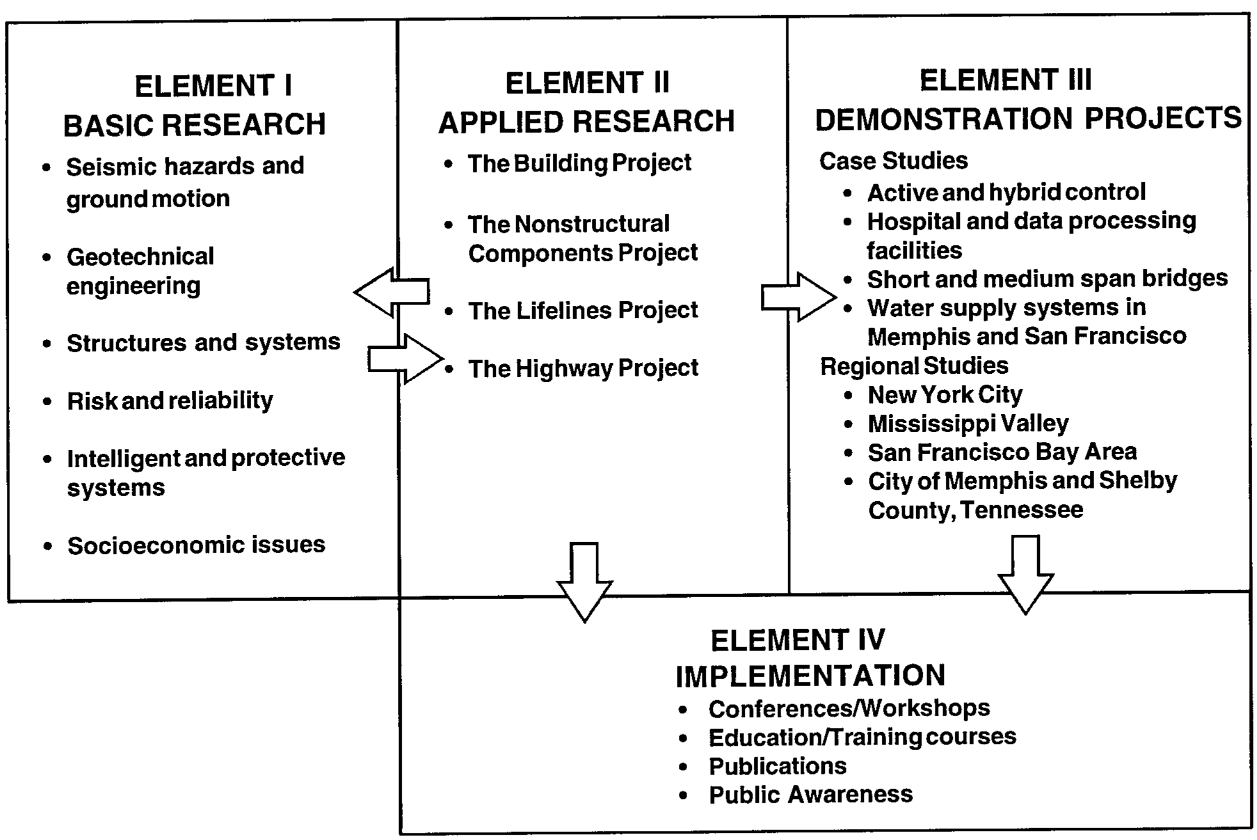
PREFACE

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.



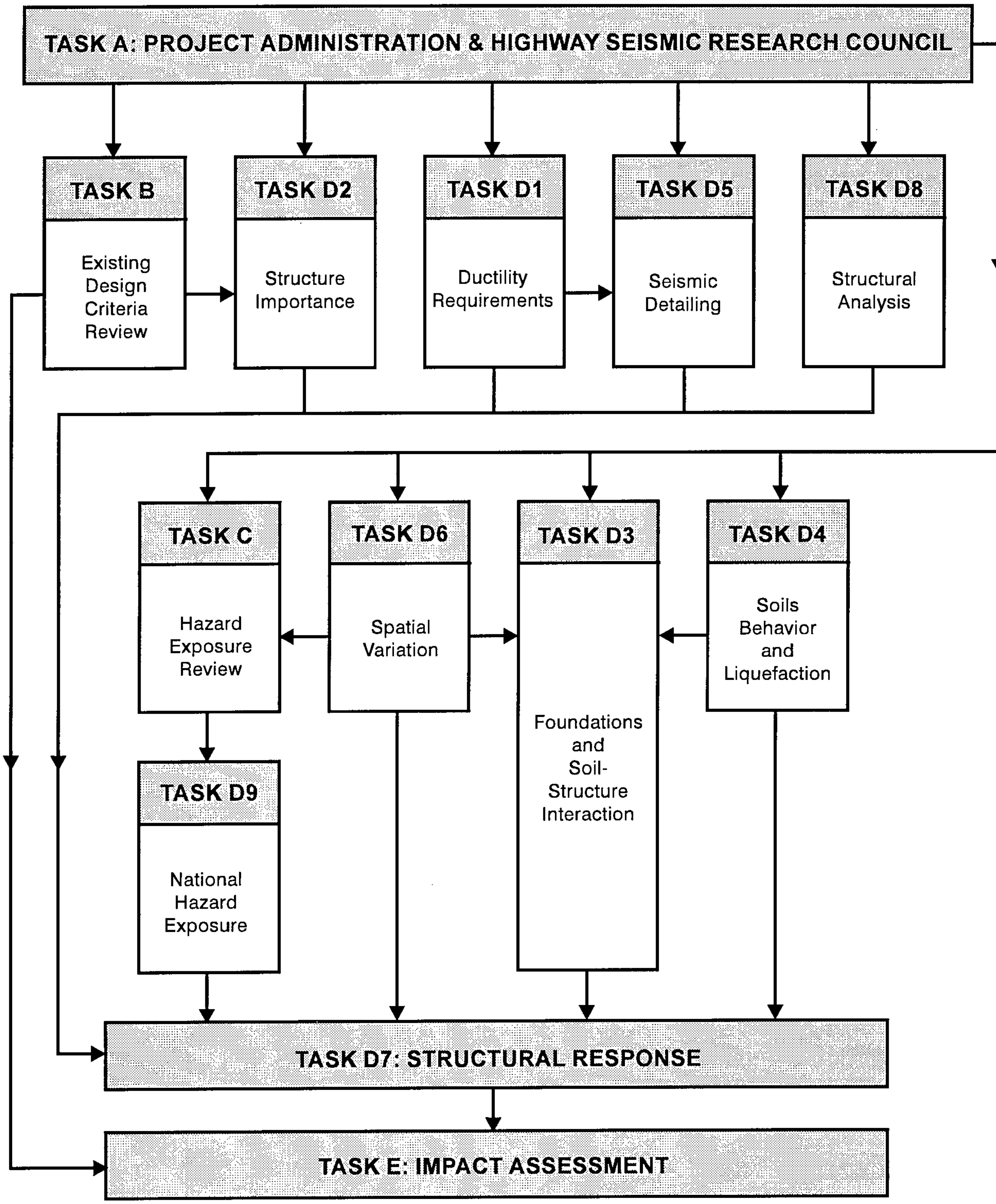
Research under the **Highway Project** develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and develops improved seismic design criteria and procedures for bridges and other highway 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(a), "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 eastern U.S. since about 1980. The companion report, NCEER-96-0006, describes pier types and seismic design and detailing procedures representative of the western U.S. since the mid-1970s.

The states contributing material include Louisiana, New York, North Carolina and Pennsylvania. Pile design issues in pile bents, bent cap design and detailing issues in pile and column bents, column and pier design and detailing issues, and footing details issues are addressed.

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



ABSTRACT

This report describes bridge types and seismic design and detailing procedures typical of the eastern part of the United States. The report contains examples taken from state bridge plans and actual bridge designs. Some of the examples comply with current seismic provisions, while others represent older designs made before these requirements were introduced. Advantages and disadvantages of various bridge configurations and details with respect to seismic behavior are discussed. Historical accounts of changes in bridge design and detailing practices are also included.

The seismic design of bridges in the eastern part of the United States follows the AASHTO Specifications for Seismic Design of Highway Bridges in conjunction with state specific policies that address the unique conditions of each state. Most bridges in the east fall in Seismic Performance Categories A and B, but in many cases part or all of the requirements of Seismic Performance Category C are also followed.

The bridge pier types included in this report are divided into pile bents, column bents and solid wall piers. The bent cap types include rectangular caps, inverted T-caps and hammerhead caps. The report highlights the unique nature of the seismic response of various bridge configurations and details. Common and dissimilar elements between east and west construction practices are identified.

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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 focussed 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-D-1.1(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 eastern U.S. since about 1980. The companion report NCEER 96-0006 describes pier types and seismic design and detailing procedures representative of the western U.S., primarily focussing on Caltrans practice.

In general, seismic policies regarding the design of the eastern bridges have followed the guidelines of the AASHTO Standard Specifications for Seismic Design of Highway Bridges in conjunction with additional state specific requirements. The extent of the AASHTO seismic design and detailing requirements varies with the Seismic Performance Category (SPC) assigned to a bridge. The SPC depends on the importance classification of the bridge and the acceleration coefficient at the site. Most bridges in the eastern part of the United States fall into either SPC A or B, but in many cases part or all of the requirements of SPC C are also followed.

The requirements of SPC A pertain only to the connection of the superstructure to the substructure design (Section 4.6) and the minimum bearing support lengths (Section 4.9.1). No other consideration of seismic forces is required for the design of structural components. The requirements of SPC B pertain to the design force requirements for superstructure and substructure structural members and connections, foundations, abutments and retaining walls (Section 4.7), and the minimum bearing support lengths (Section 4.9.2). Detailing requirements for reinforced concrete columns for SPC B are specified in Section 8.3 to ensure some level of ductility. They include minimum transverse reinforcement requirements at the top and bottom of a column (Section 8.4.1 (D)) and maximum spacing limits of transverse reinforcement (Section 8.4.1 (E)). Additional design requirements for foundations and abutments for SPC B are included in Section 6.3. The requirements for SPC C are more stringent. They pertain to the design force requirements for superstructure and substructure structural members and connections, foundations, abutments and retaining walls (Section 4.8) and minimum bearing support length (Section 4.9.3). Detailing requirements for reinforced concrete are specified in Section 8.4 for columns, piers, column connections, and construction joints in piers and columns to ensure adequate ductility capacity. Additional requirements for foundations and abutments for SPC C are included in Section 6.4.

Historically, due to the low frequency of earthquakes in the eastern United States the awareness for the need of providing adequate seismic design and detailing guidelines has been relatively low, and has generally followed the experience of the state of California. In 1973, after the 1971 San Fernando earthquake when many bridges were damaged, Caltrans adopted new seismic design criteria that have formed the basis for the modern approach to seismic design. The Caltrans specifications have been regularly refined and updated to incorporate technical developments in earthquake engineering and lessons from the 1989 Loma Prieta earthquake and the 1994 Northridge earthquake. In 1975, AASHTO published a modified version of the original Caltrans criteria as Interim Specifications, and in 1983 it adopted the ATC-6 report, a comprehensive state-of-the-art document funded by the Federal Highway Administration and Caltrans, as an approved alternate Guide Specification for seismic design. However, it was only in 1991, when AASHTO incorporated the 1983 Guide Specification into the Standard Specifications as Division I-A, that bridge designers in the low seismic zones of the eastern United States became fully aware of the various aspects inherent in the latest seismic design and detailing practices. Today, the bridge engineering community is also aware of the unique nature of the seismic hazard in the central and eastern United States where the maximum credible earthquake is expected to be significantly larger than the design earthquake, and many designs are even more conservative than the AASHTO requirements for the applicable Seismic Performance Category.

SECTION 2
REPRESENTATIVE PIER TYPES (EAST)

Typical piers, connections and details have been identified from the bridge design and construction practices of several representative eastern states (Louisiana, North Carolina, Pennsylvania and New York). The pier types include pile bents, column bents and solid wall piers. Representative examples of bent and pier type details are provided in the figures presented at the end of each section. The seismic design of bridges in Louisiana follows the requirements of the AASHTO Specifications for the Seismic Design of Highway Bridges, with the entire State of Louisiana in Seismic Performance Category (SPC) A. Bridges in North Carolina, Pennsylvania and New York fall into either the SPC A or B, as shown in figures 2-1, 2-2 and 2-3. The seismic design of bridges in North Carolina, Pennsylvania and New York is based on the AASHTO specifications for SPC A or B. In Pennsylvania and New York the SPC B requirements include adjustments that incorporate many of the SPC C requirements regarding column reinforcement, piers, column connections, foundations and abutments.

2.1 Pile Bents

Description: Pile bents consist of timber, steel or prestressed concrete piles with a cast-in-place reinforced concrete cap. The piles extend out of the ground to serve as columns and are embedded into the bottom of the cap. Double row pile bents are included when additional capacity or stability is needed. A typical trestle bridge layout with pile bents is shown in figure 2-4. Prestressed concrete piles are most common, but in some states such as North Carolina steel H-piles are also common, and steel pile bent details are provided. The minimum number of piles per bent is usually four, with the exterior piles battered in the higher bents. The reinforced concrete cap is wider than the width of the piles so that the bottom corner longitudinal reinforcing bars could be continuous along the cap. Wider bent caps are also provided to allow for possible pile mislocations. The bridge superstructure is connected to the pile bent cap mainly by elastomeric bearings. Representative pile bent details are shown in figures 4-1 through 4-10.

2.2 Column Bents

Description: Column bents consist of a reinforced concrete frame attached to a separately constructed footing. Two or three circular columns of solid cross-section with a rectangular cap are most common (see figure 2-5). One column bents and columns with a solid rectangular cross-section are also quite common. The superstructure is supported on bearings anchored to the top of the cap. In general, column spacings do not exceed 20 feet center to center of columns. Intermediate struts are usually used on the taller bents. North Carolina standards, for example, require that column struts be placed at approximately mid height in columns over 25 feet high and less than 45 feet. Two struts are required in columns in excess of 45 feet. New York standards require that reinforced concrete struts be provided near the middle half of a column when the slenderness ratio of the column in a direction parallel to the support is over 60. The rectangular reinforced concrete caps are most common (see figure 5-1). Hammerhead type caps are usually used in single column bents (see figure 6-2). Inverted T-caps are becoming more and more popular in current designs (see figure 6-5). They are usually used for aesthetic reasons, or when it is necessary to reduce the overall depth of the cap plus the superstructure. Stepped bent caps (see figure 4-10) are also common. Columns may have individual or continuous footings, depending on the soil conditions. Representative column bent details are shown in figures 5-1 through 5-10 and 6-1 through 6-6.

Historical: In the past, column struts were not required in New York, as long as the slenderness effects were considered in design.

2.3 Solid Wall Piers

Description: A vertical support is considered to be a wall pier when the ratio of the clear height to the maximum plan dimension of the support is less than 2.5. Full or partial height pier walls are often used at stream crossings to prevent the accumulation of debris between columns, and as crash walls in bridges over railways along or near the railroad track. To accommodate the geometric requirements of the superstructure, solid piers may have a hammer head type cap, or may be widened at the top (see figures 5-11 and 5-12). Representative solid wall pier details are shown in figures 5-11 through 5-13.

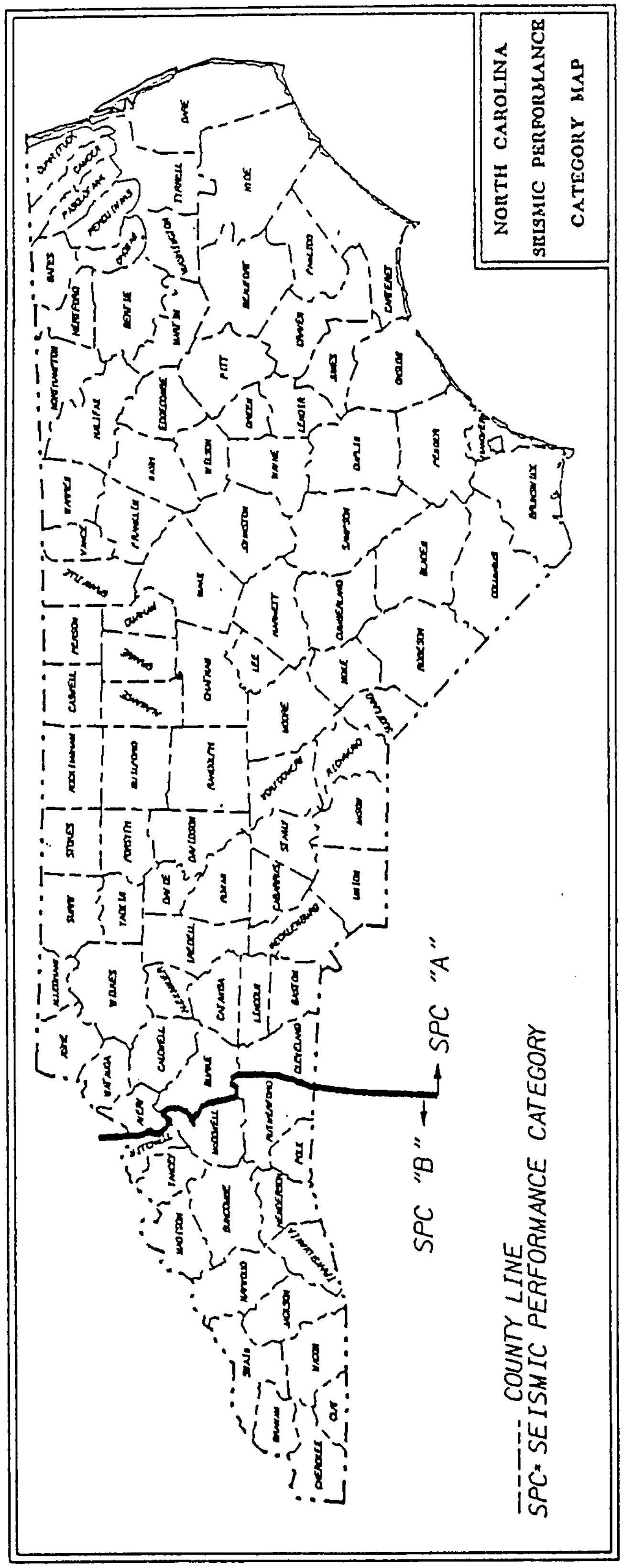


FIGURE 2-1 North Carolina Seismic Performance Category Map

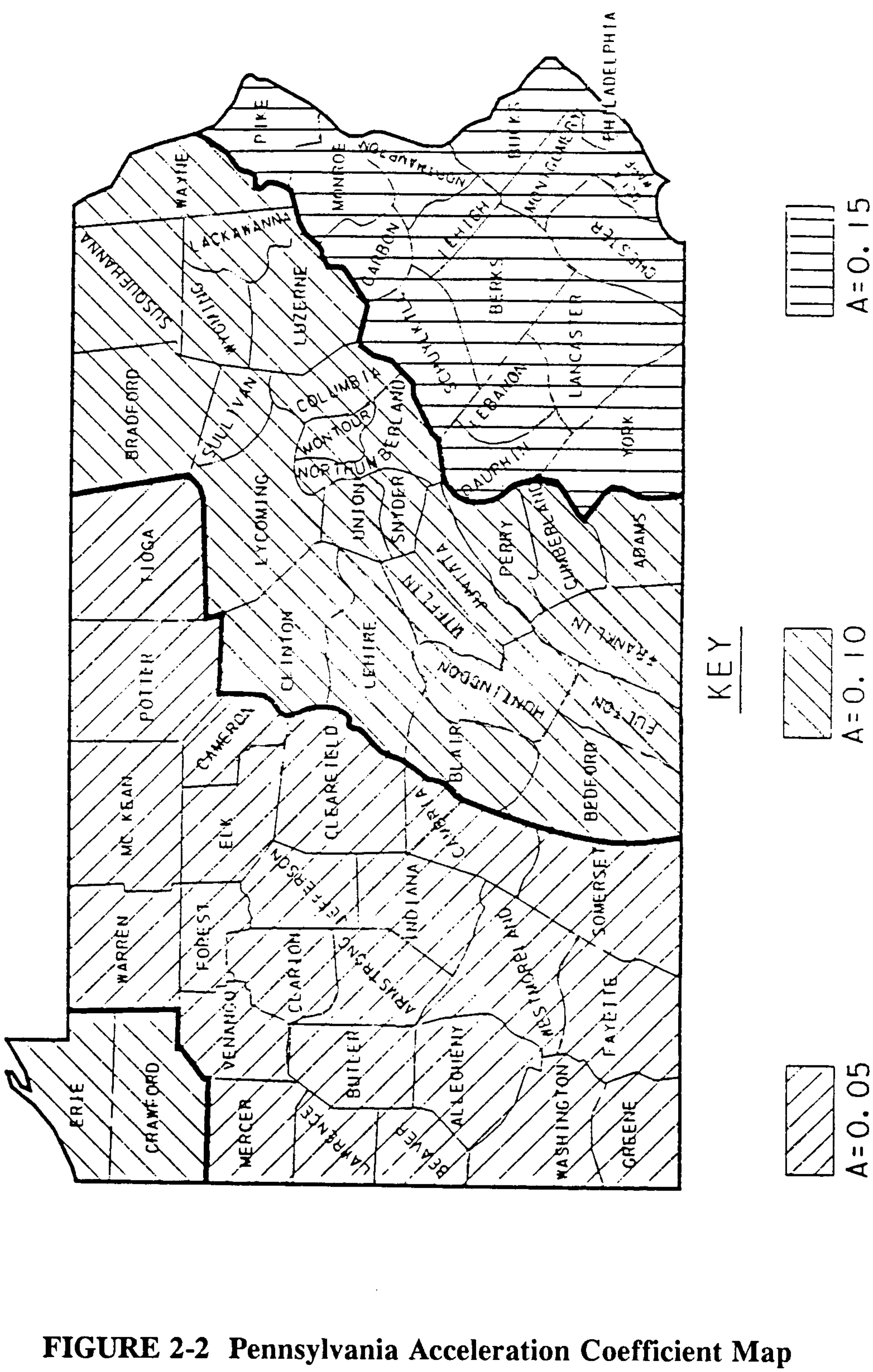


FIGURE 2-2 Pennsylvania Acceleration Coefficient Map

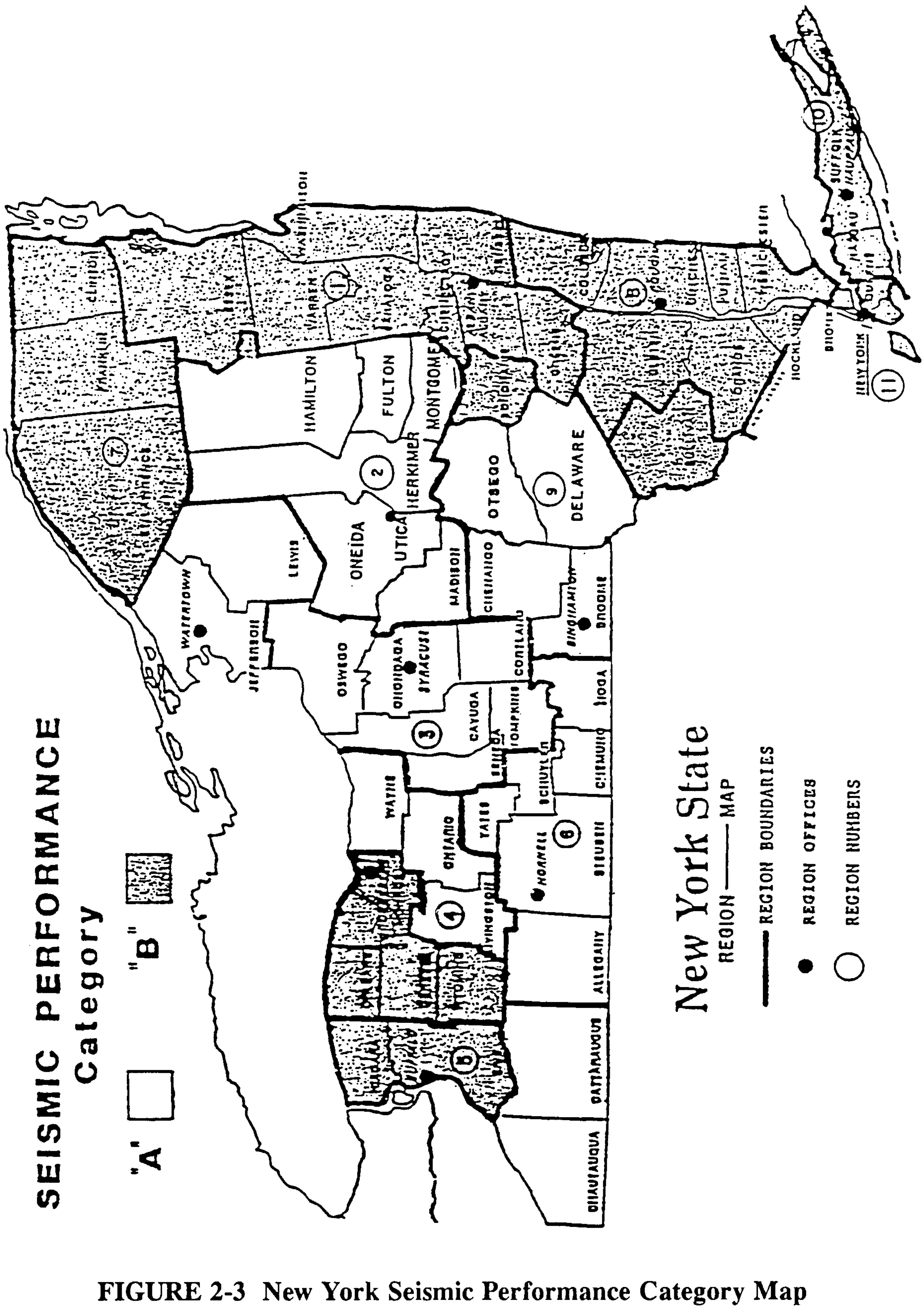
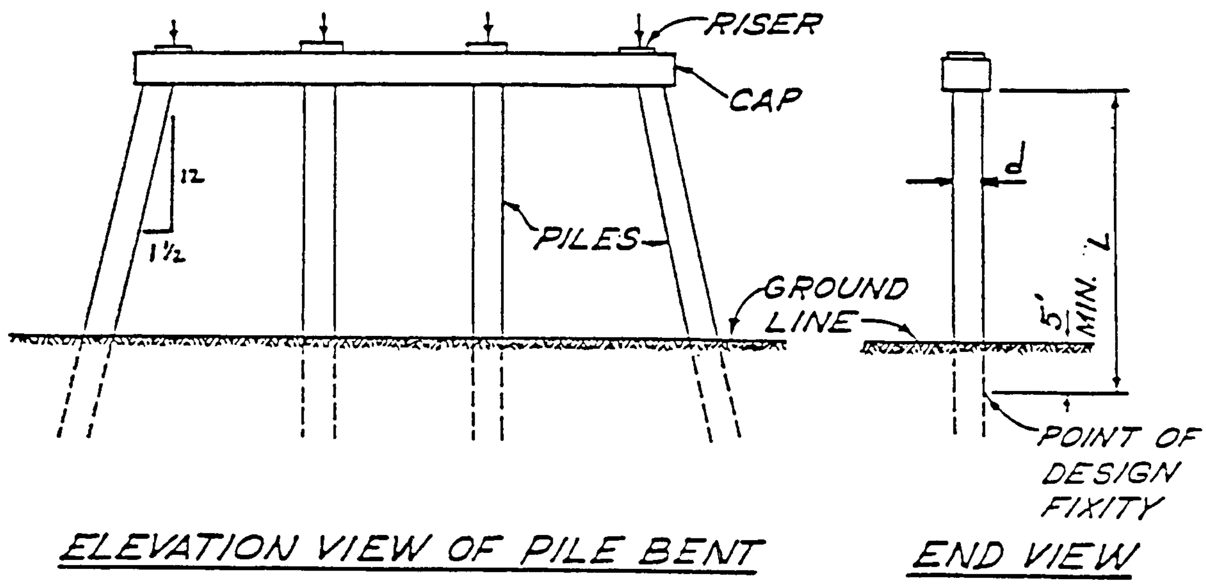


FIGURE 2-3 New York Seismic Performance Category Map



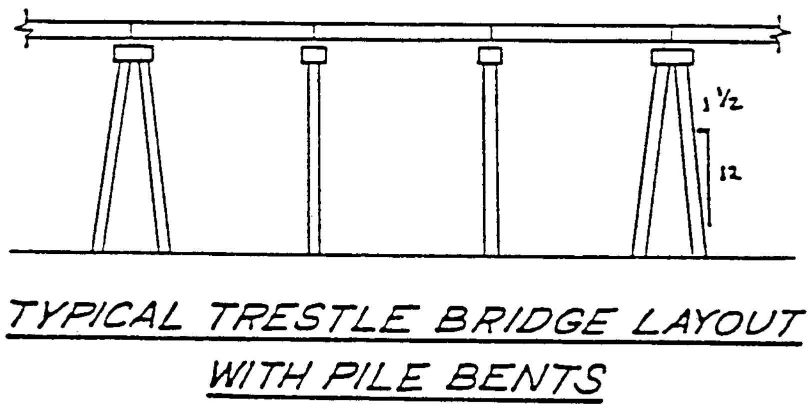


FIGURE 2-4 Typical Trestle Bridge Layout with Pile Bents

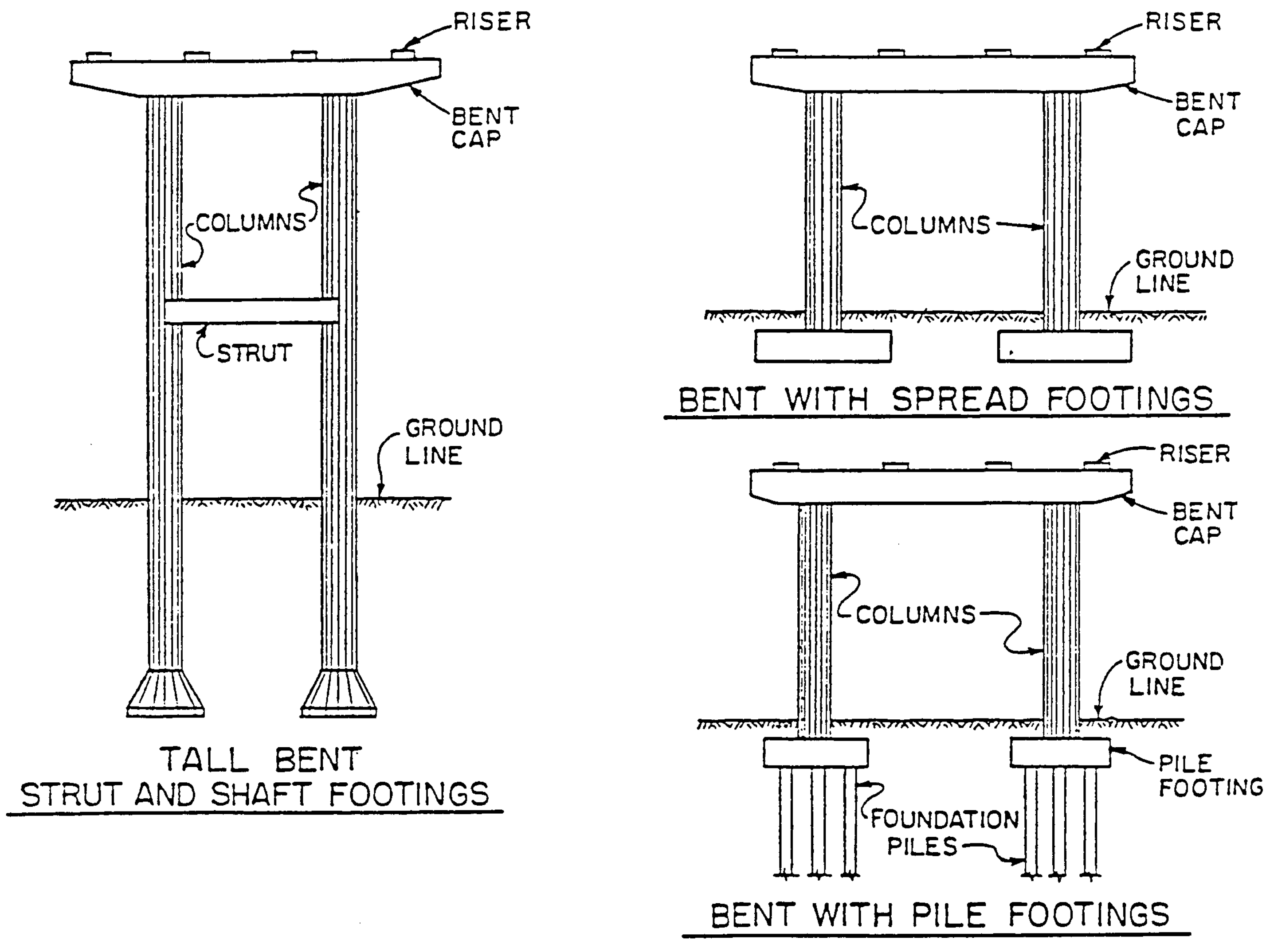


FIGURE 2-5 Typical Column Bent Details

SECTION 3
PILE DESIGN ISSUES IN PILE BENTS

3.1 Pile Batter and Maximum Slenderness Requirements

Description: Pile bents are designed according to the AASHTO Specifications with additional state specific requirements, as described below. Louisiana requires that the exterior piles be battered when the slenderness ratio L/d is over 12, where L is the unsupported pile length measured from the ground line, and d is the least dimension, or diameter of the pile section. The exterior piles are typically battered 1-1/2 on 12. The maximum slenderness ratio L/d is limited to under 20. North Carolina standards require a minimum number of 4 piles for the interior bents (bents between abutments or end bents) and a maximum pile center to center spacing of 10 feet for 12" and 14" concrete piles and 12 feet for 20" concrete piles. The North Carolina standards also specify timber pile detail requirements and provide standard steel pile bent details. Pile batter requirements are shown in figures 4-1 through 4-4.

Advantages: Pile bents are usually supported by at least 4 piles and have a larger degree of redundancy. Vertical multi-pile bents can also allow for some inelastic response, and therefore AASHTO permits a response modification factor of 3 for these systems. The response modification factors are intended to account for redundancy and ductility in structural members. Design forces are obtained by dividing the elastic forces obtained from analysis by the response modification factors.

Disadvantages: The batter of the exterior piles provides stability and good lateral support, but it can reduce the ductility capacity of the bent. Therefore, a lower response modification factor of 2 is recommended by the AASHTO seismic specifications for pile bents with one or more batter piles.

3.2 Pile-Cap Connection Details

Description: Concrete piles are typically required to penetrate into the bent cap a minimum of 9". In caps over 2'-3" deep, North Carolina requires a minimum pile penetration of 12". Pile voids that usually exist in the larger precast prestressed concrete piles (over 20") are required by Louisiana to be poured monolithic with the cap over the embedded length (see figure 4-1). To increase the strength of the pile cap connection, North Carolina standards provide for circular hoops in the cap, around the embedded portion of the piles (see figures 4-6 and 4-7). There are no state standards for positive tension anchor pile connection details indicated for the time frame being considered herein. However, many designs do provide for positive tension pile anchorage and a ductile connection to the cap through the use of a steel cage in the pile void, that extends into the cap (see figure 4-5). Representative pile-cap connection details are shown in figures 4-1 through 4-9.

Advantages: The use of a steel cage in the pile voids that extends into the cap can provide for reliable pile anchorage for tension forces and for some ductility capacity. Additional reinforcement in the cap around the embedded portion of the pile increases the local strength of the cap near the pile and improves the capacity and ductility of the pile-cap connection.

Disadvantages: In most cases, the pile connection to the bent cap has very little bending and pull out capacity and almost no ductility. Seismic damage is likely to occur during larger earthquake loading in the connection area. Better connection details are needed to improve strength and ductility, but the cost of providing such details can be high. However, for SPC A a ductile moment connection of the pile to the cap may not be needed, if properly accounted for in design.

SECTION 4
BENT CAP DESIGN AND DETAILING ISSUES IN PILE BENTS

4.1 Cap Dimensions

Description: Rectangular cast-in-place reinforced concrete pile bent caps are most common. The minimum bent cap depth requirements are generally 2'-6" for single row concrete pile systems and 3'-0" for double row concrete piles. Louisiana bent cap depth requirements vary with the pile size, as shown in table 4-1.

TABLE 4-1 Louisiana Bent Cap Depth Requirements

MINIMUM CAP DEPTH	PILES UNDER 24"	PILES 24" AND OVER
Single Row Bents	2'-0"	2'-3"
Double Row Bents	2'-6"	2'-6"

The minimum bent cap width requirements are given in terms of the pile size. Louisiana requires that the cap be larger than the pile on each side of the cap by 6" for 18" piles or less, and 9" for piles over 18". North Carolina minimum cap width requirements are shown in table 4-2.

TABLE 4-2 North Carolina Bent Cap Width Requirements

MINIMUM CAP WIDTH	12" PILES	20" PILES
Single Row Bents	2'-9"	3'-8"
Double Row Bents	4'-0"	5'-8"

A tolerance of 3" maximum mislocation of piles in the cap from the plan drawing is usually allowed. Saddle cap types are common in North Carolina (see Figure 4-8). The minimum saddle cap section is 3'-9" wide by 2'-3" deep for 12" piles and 5'-4" wide by 3'-0" deep for 20" piles. The width of the bent cap is required to allow a minimum distance of 2 1/2" from the edge of the bearing plate to the side face of the cap for a steel superstructure and a minimum distance of 5 1/2" from the centerline of anchor bolts to the side face of the cap for a prestressed girder superstructure. The minimum bent cap width is also required to meet the bearing support length requirements of the AASHTO Specifications for Seismic Design, Section 4.9.1 for various span lengths and column heights. The length of the bent cap is set so as to provide a minimum of 9" from the side face of the exterior pile to the end of the cap and a minimum of 9" from the edge or corner of bearing plate to the end of cap for steel superstructures and from the anchor bolts to the end of cap for prestressed girder superstructures. Typical bent cap dimensions are shown in figures 4-1 through 4-8.

Advantages: The wider cap section allows for the bottom corner longitudinal reinforcing bars to be continuous along the cap. It also provides a higher cap flexure capacity and it prevents damage and plastic hinging in the cap near the piles.

4.2 Longitudinal Cap Steel

Description: Both top and bottom longitudinal reinforcement, of equal area, is required in bent caps. The amount of reinforcement is determined based on the AASHTO Specifications. The minimum top and bottom reinforcement required for a rectangular cap in North Carolina is four No. 9 bars (or equivalent) for a cap width of 3'-0" or less,. five No. 9 bars (or equivalent) for a cap width greater than 3'-0" but less than or equal to 4'-0", six No. 9 bars (or equivalent) for a cap width greater than 4'-0" but less than or equal to 5'-0", and seven No. 9 bars (or equivalent) for a cap width greater than 5'-0" but less than or equal to 5'-8". The steel bar arrangement in a saddle cap section are shown in Figure 4-8. Representative longitudinal cap steel details are shown in figures 4-1 through 4-10.

Advantages: During seismic loading, load reversal in the cap may occur. Providing top and bottom reinforcement increases the positive and negative moment capacity of the cap and helps prevent damage to the cap. The saddle cap section provides steel in addition to that which is in the main cap section. Also it allows all bottom steel to be continuous along the cap.

4.3 Cap Shear Reinforcement

Description: The longitudinal steel is placed inside closed shear stirrups. The maximum stirrup spacing is 12". In Louisiana, the stirrups adjacent to piles are located at a maximum of 3" from the face of the pile and the first space is under 6". The minimum size of the stirrups is No. 4 bars. Double stirrups are used in all pile bent caps exceeding 2'-6" in width. North Carolina uses No. 4 or No. 5 stirrups placed according to AASHTO Specifications that are made of one U-type stirrup and one tie, both with hooked ends, at each location. Representative cap shear reinforcement details are shown in figures 4-2, 4-3, 4-9 and 4-10.

Advantages: The closed stirrups provide good confinement to the longitudinal steel and increase the cap ductility and torsional capacity. The closely spaced stirrups in the cap near the piles increase the local cap capacity near the pile locations. The U-shaped stirrups with hooked ends topped by a straight bar with hooked ends used in North Carolina allows for easier construction, and approaches a closed stirrup confinement effect.

Disadvantages: The closed stirrup arrangement makes it harder to place the longitudinal reinforcement and may increase the construction costs. A reinforcing steel cage assembled on site and set in place on the pile bent with all the cap reinforcement in is usually used.

4.4 Other Cap Reinforcement

Description: In North Carolina, additional reinforcement is required to be placed in the pile cap connection area. Four No. 4 longitudinal bars are required to be placed at equal spaces above each row of piles with No. 4 bars at about 4'-0" placed transversely to the cap (see figure 4-6). Two No. 4 circular hoops around each pile in single row systems and three No. 4 rectangular hoops around piles in double row systems are also required (see figures 4-6 and 4-7). In addition, for interior bents it is required that enough extra U-shaped stirrups be placed in top of cap so that there will be No. 4 bars at 6-inch centers under bearing areas of each beam or girder (see figures 4-9 and 4-10). Also, No. 9 U-shaped bars are required to be placed at the ends of caps for 16" and larger piles (see figure 4-9).

Representative cap reinforcement details used in addition to the longitudinal and transverse bars are shown in figures 4-6 through 4-10.

Advantages: The additional reinforcement around the embedded portion of the piles improves the strength and the ductility of the cap at the pile-cap connection and reduces the potential for localized seismic damage in the cap.

Disadvantages: These additional reinforcement details may increase the construction costs of bridges in SPC A.

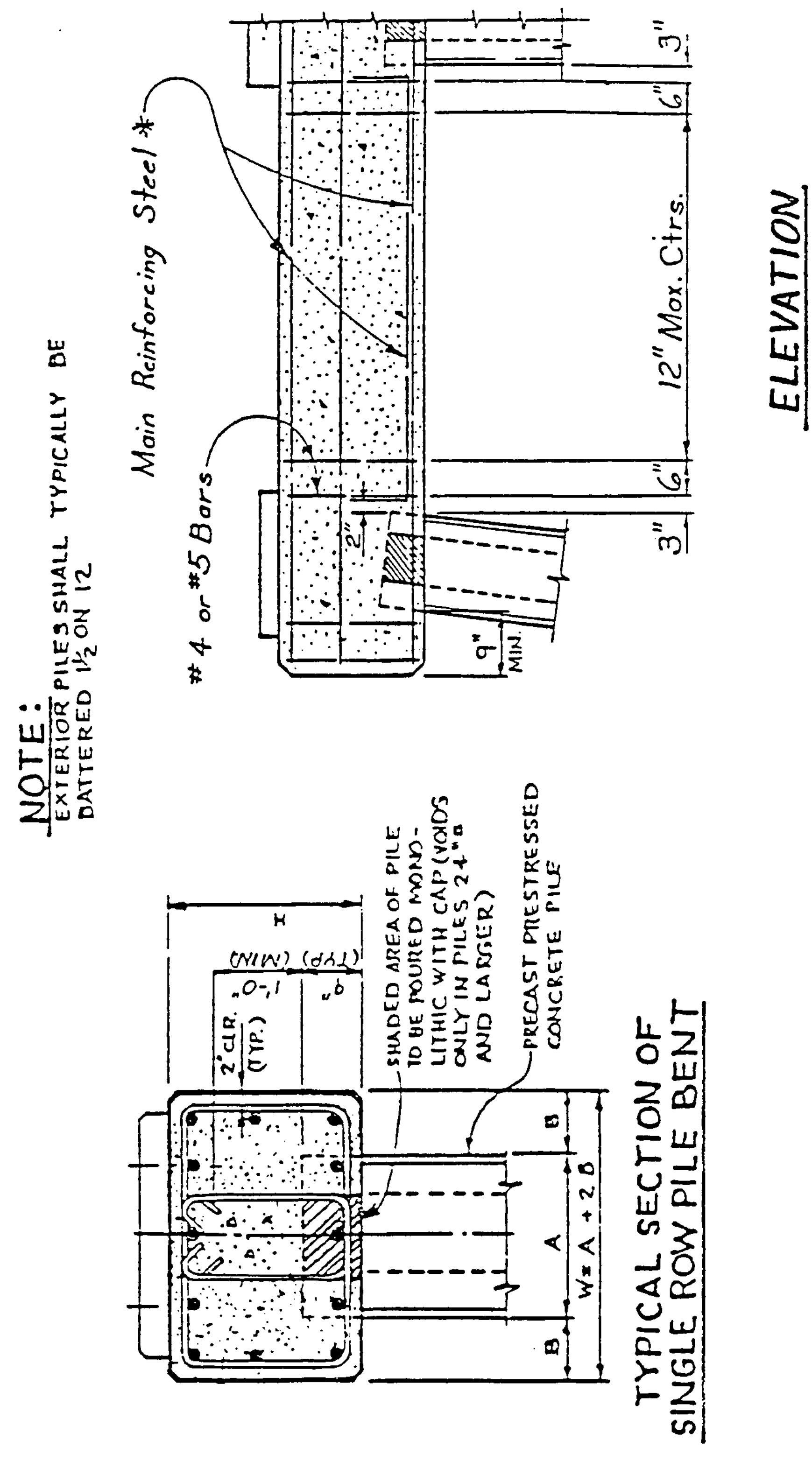


FIGURE 4-1 Louisiana Typical Single Row Pile Bent Details

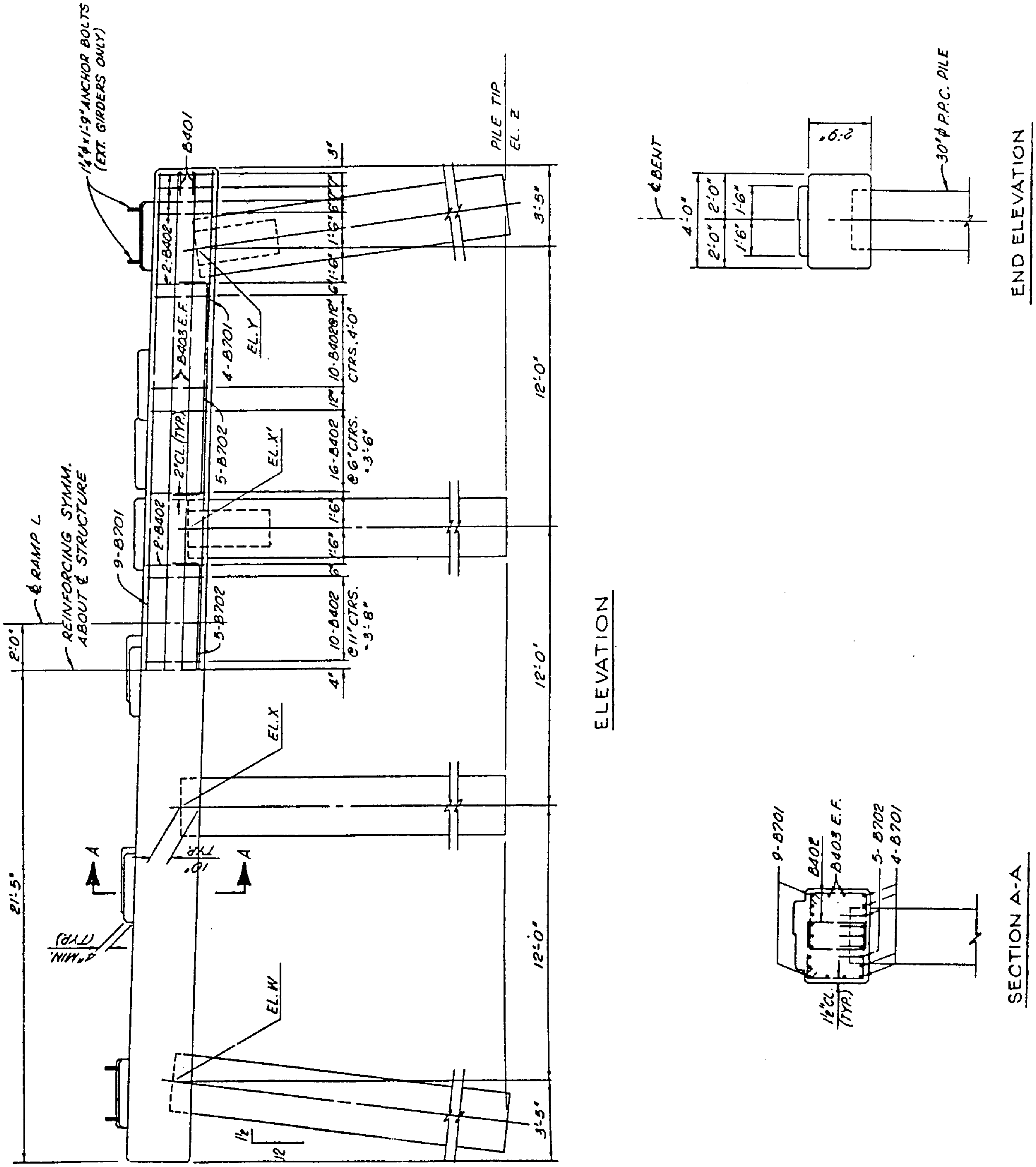


FIGURE 4-2 Single Row Pile Bent Reinforcing Details

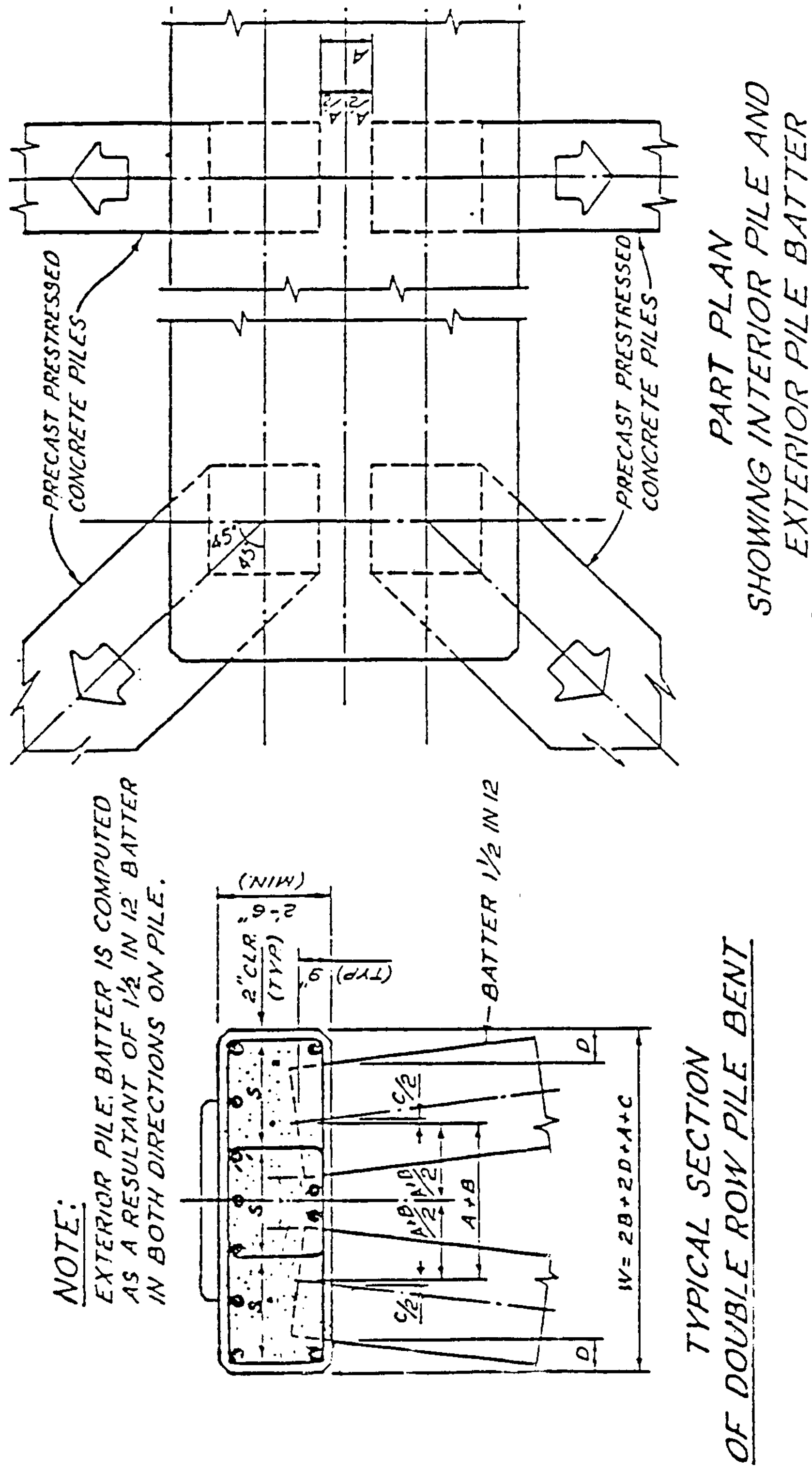


FIGURE 4-3 Louisiana Typical Double Row Pile Bent Details

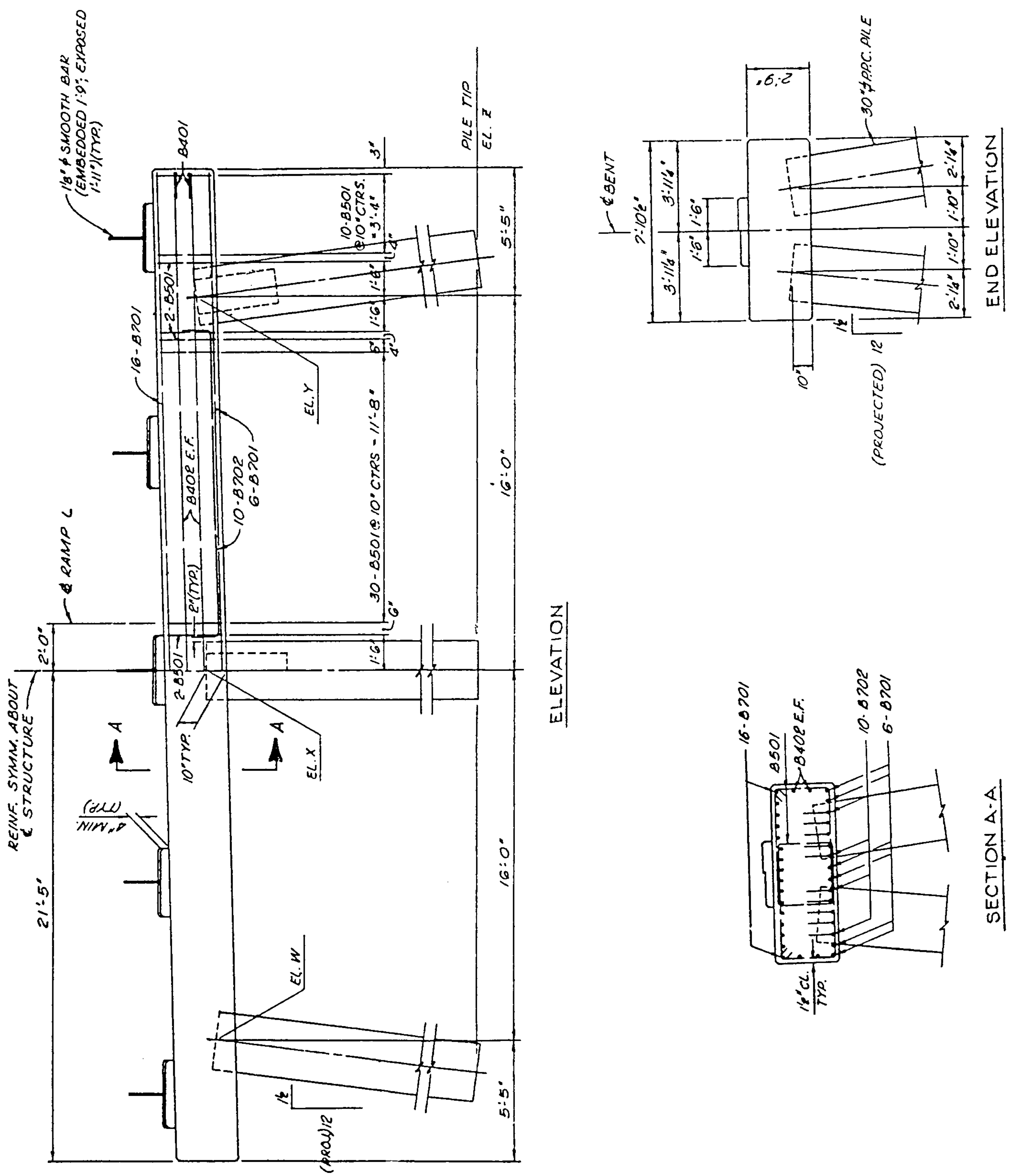


FIGURE 4-4 Double Row Pile Bent Reinforcing Details

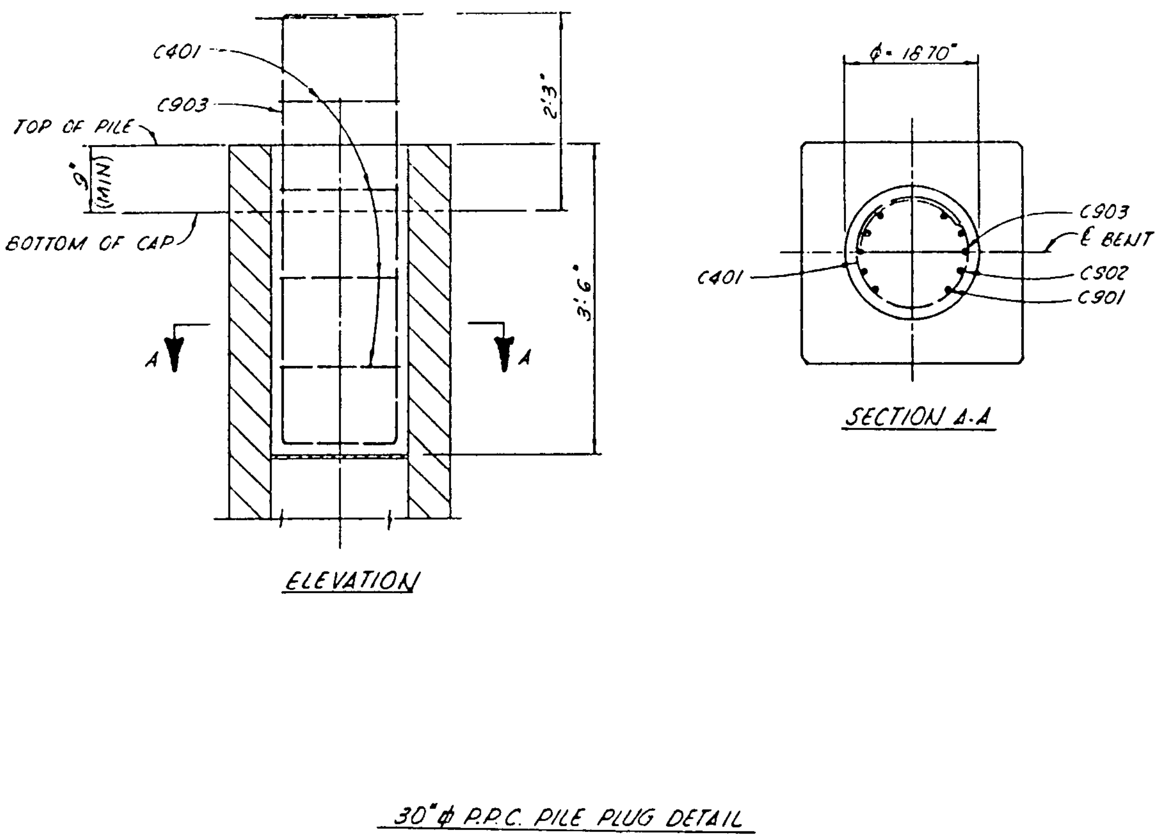
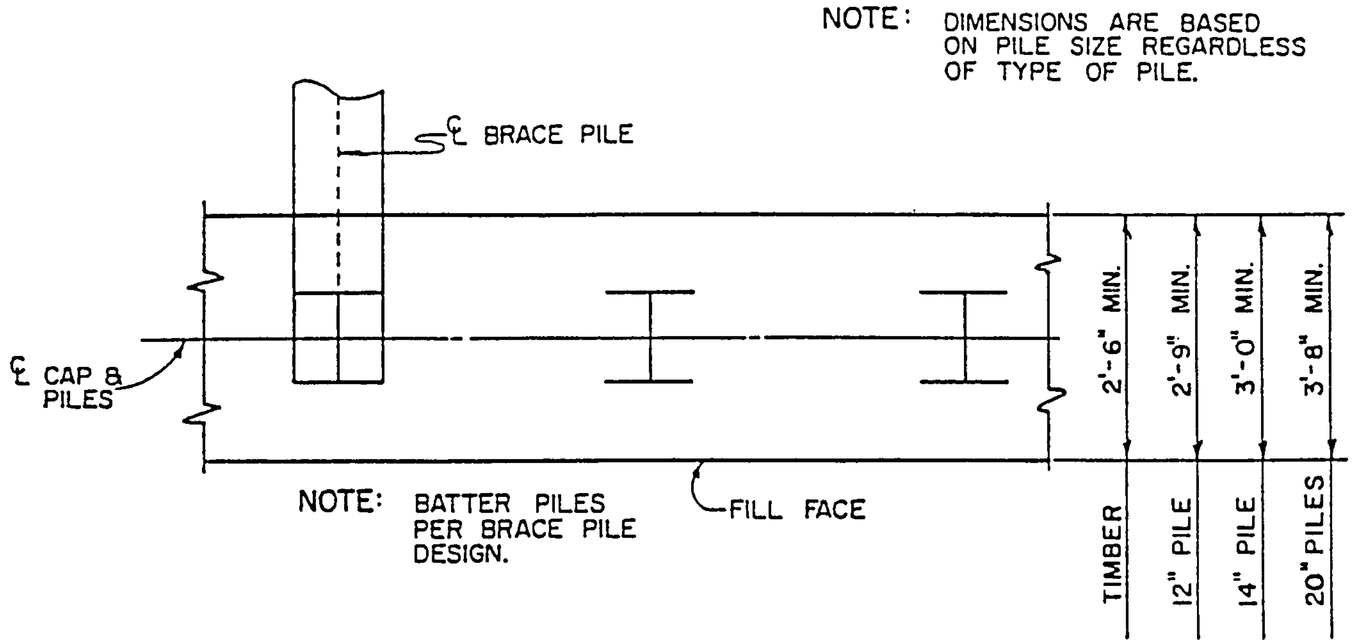


FIGURE 4-5 Louisiana Pile Plug Details



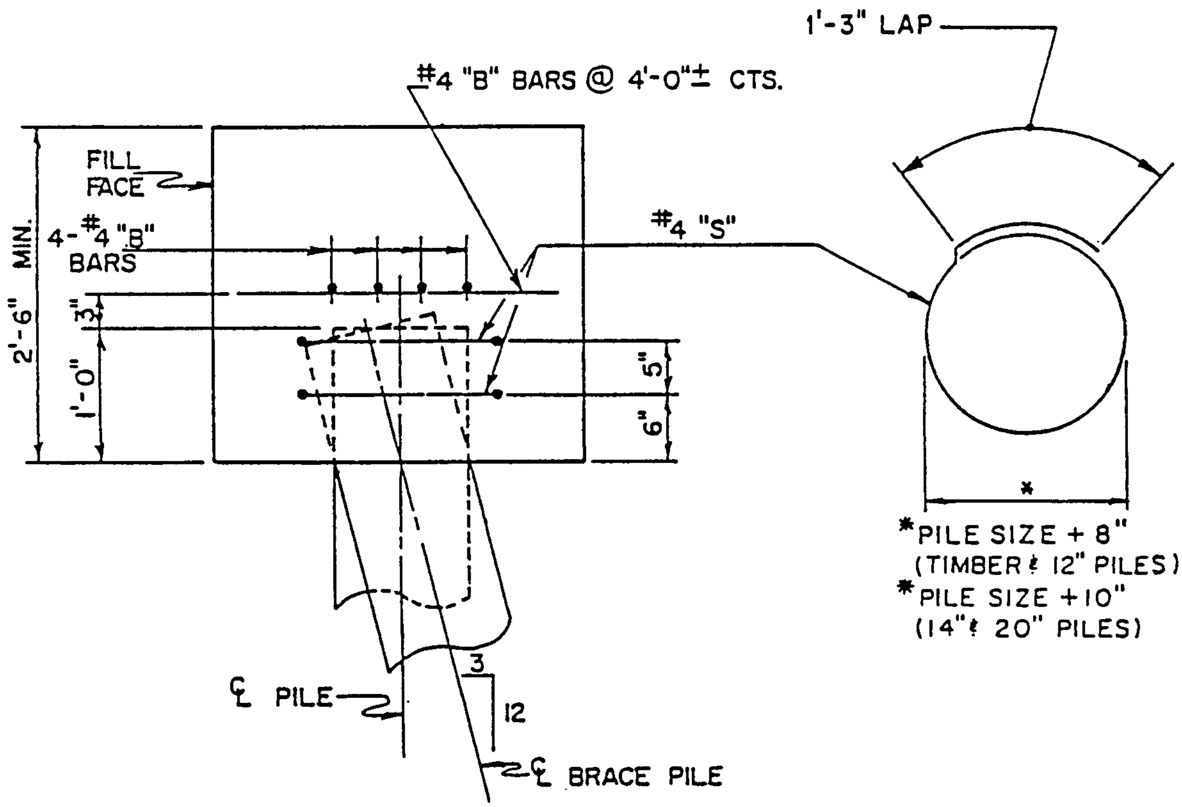


FIGURE 4-6 North Carolina Typical Single Row Pile Bent Details

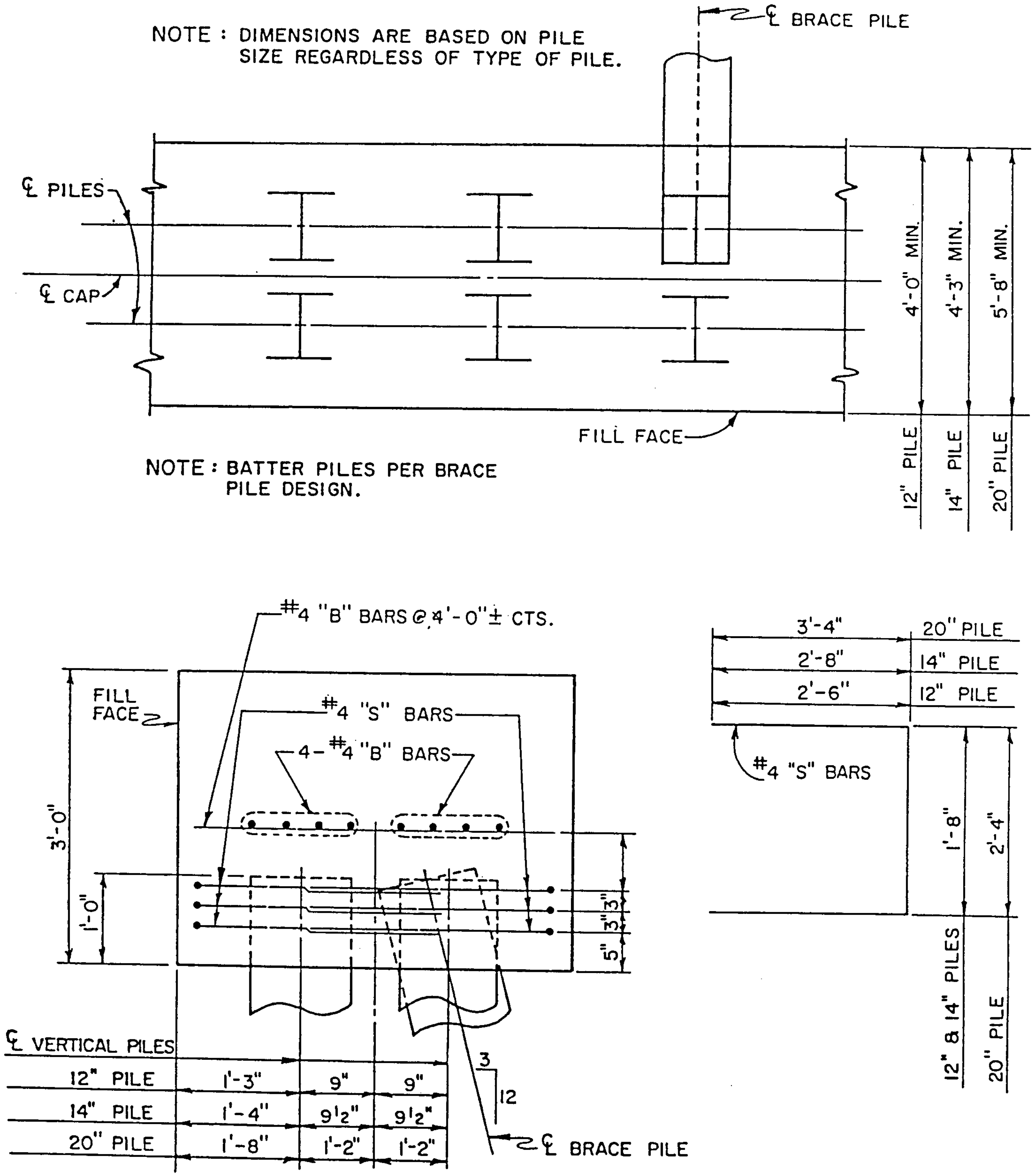


FIGURE 4-7 North Carolina Typical Double Row Pile Bent Details

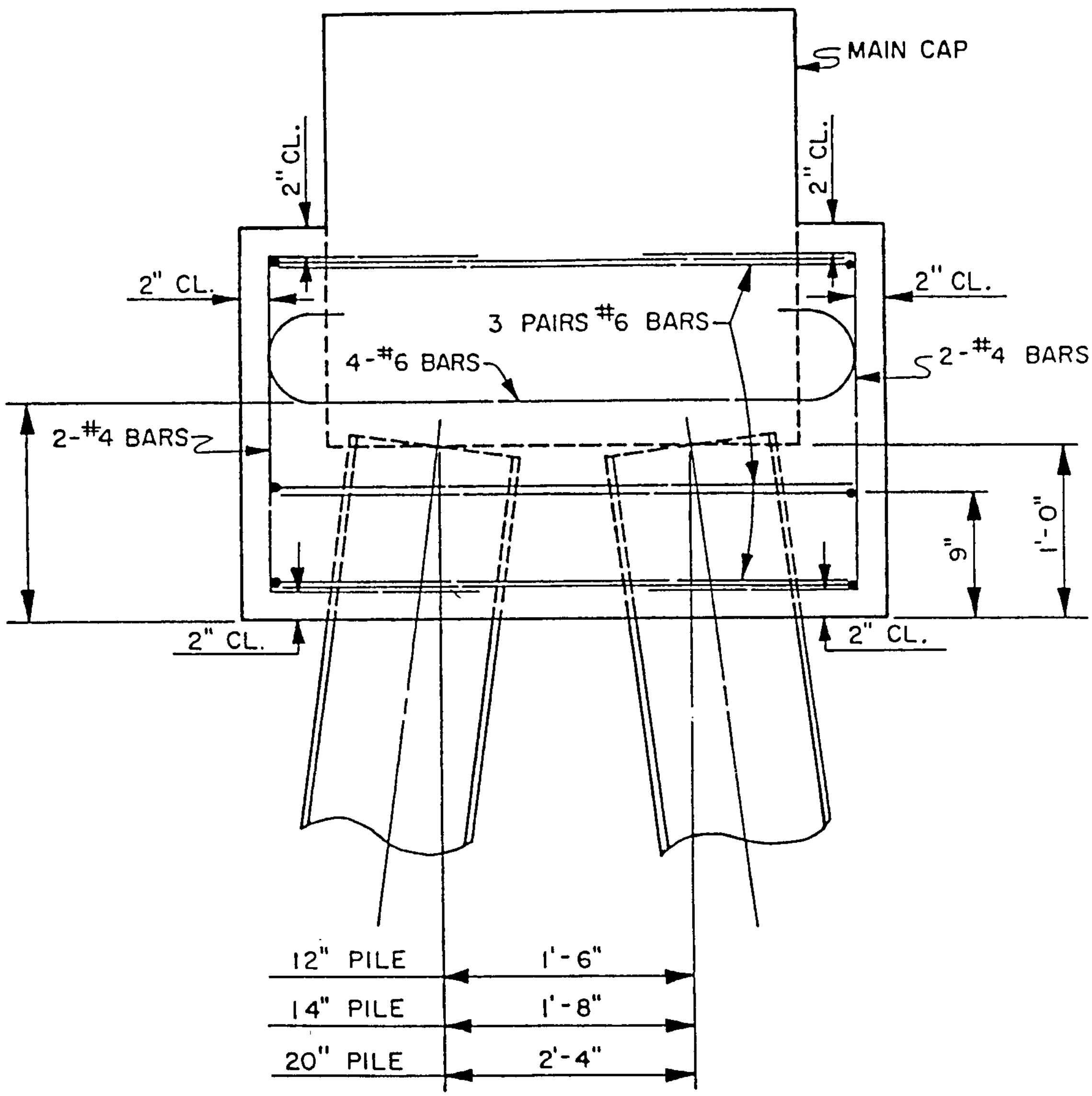


FIGURE 4-8 North Carolina Saddle Cap Details

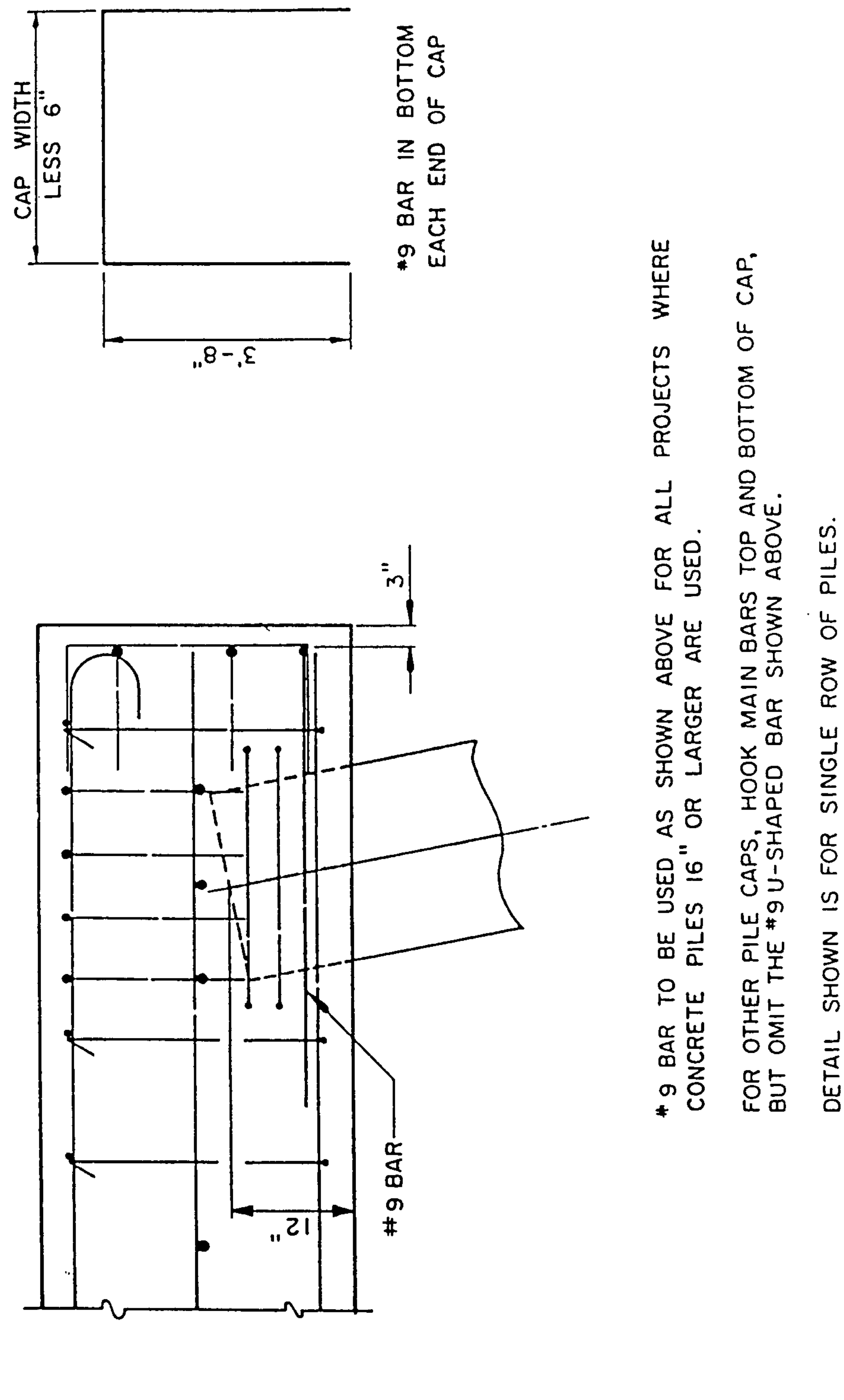


FIGURE 4-9 North Carolina Interior Bent Pile-Cap Connection Details

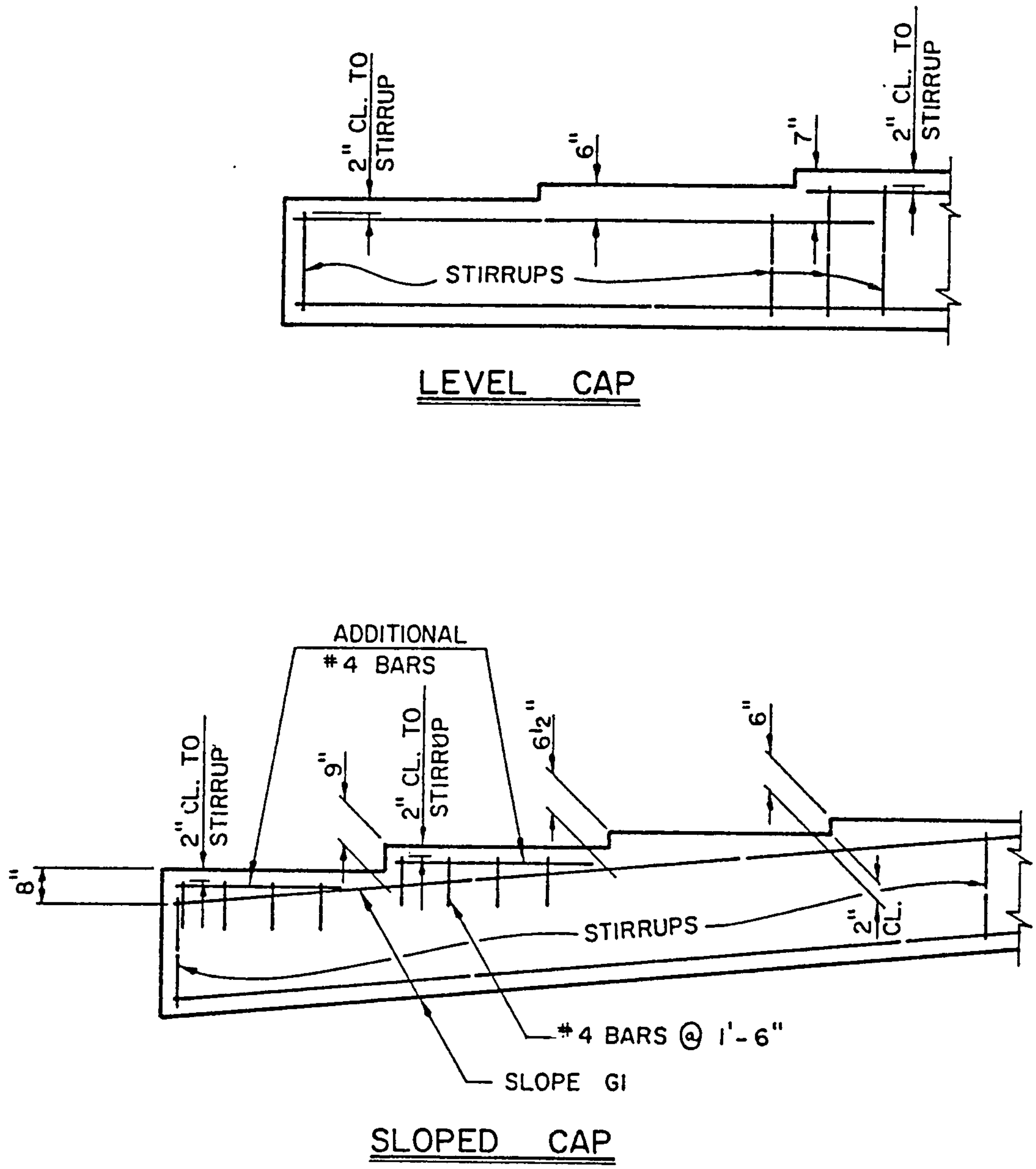


FIGURE 4-10 North Carolina Stepped Bent Cap Reinforcing Details

SECTION 5
COLUMN AND PIER DESIGN AND DETAILING ISSUES

5.1 Column Dimensions

Description: Column bents with circular columns are most common (see figure 5-1). The minimum diameter required for a circular column is usually over 30". North Carolina limits the minimum column diameter to 36". Most of the quantitative requirements below are based on North Carolina practice. Other state requirements were similar, but not as specific. For larger columns, the diameter is increased in 6" increments. When the cap width exceeds 3'-6" a minimum column diameter of over 36" is usually required. The ratio of the unsupported column length to the least column width (L/d) is generally kept under 10. Intermediate column struts are 2'-6" wide by 2'-9" deep for 36" diameter piles. For larger columns the strut size is increased accordingly. Rectangular columns of varying cross section are also common (see figure 5-2). The dimensions of rectangular columns are usually increased towards the foundation level (see figure 5-2), but sometimes they are decreased towards the base (see figure 5-3a). The overhang from the end of cap to the face of column does not exceed 4'-0" and is not less than 3'-0". In some cases column of different heights are used in the same bent (see figure 5-4a). The requirements for minimum column sizes along with other design requirements result in relatively low axial stress values. The average axial stress ratio P/f'_cA_g, significantly affects seismic capacity in terms of flexural overstrength, shear strength and ductile behavior. Representative column bent geometries and dimensions are shown in figures 5-1 through 5-13.

Advantages: Reducing the dimensions of rectangular columns towards the base reduces the loads transmitted to the footing and the potential of seismic damage in the footing. Square columns allow for an easier placement of the cap reinforcing bars.

Disadvantages: Providing minimum requirements for column dimensions and increasing the cross section of rectangular columns towards the base ensures strength and stability but may force plastic hinging to occur in the cap or the foundation structure first, which is undesirable. Using different column heights in the same bent results in nonuniform seismic load distribution and is undesirable and should be avoided where practical. The shorter columns will attract more load and may fail before plastic hinging can occur in the other columns.

5.2 Column Vertical Reinforcement

Description: Column longitudinal steel requirements generally follow the AASHTO Specifications guidelines. In Louisiana columns are usually designed as tied columns even if spiral steel is used. In North Carolina, columns on spread footings are designed for a column height that is 3'-0" longer than the actual height. The area of the longitudinal reinforcement is usually kept between 0.01 to 0.06 of the gross cross-section area of the column, which satisfies the SPC C vertical reinforcement requirement. In North Carolina, the minimum longitudinal bar sizes for 36" diameter columns are 8 No. 10 bars for columns under 19"-11", 8 No. 11 bars for columns 20'-0" and over with additional No. 11 bars added in columns over 45'-0", as per design requirements. In Pennsylvania and New York, the flexural strength of bridge columns in SPC B is determined based on the AASHTO Seismic Specification 8.4.1(B), that applies to SPC C. This provision reduces the strength reduction factor to 0.50 for both spiral and tied columns with high axial stresses (over $0.20f'_c$). This requirement was introduced to account for the lower ductility capacity of columns with high axial loads. Representative column vertical reinforcement details are shown in figures 5-1 through 5-5.

Advantages: Providing longitudinal reinforcement as per AASHTO Seismic Specification 8.4.1(B and C) that applies to SPC C to bridges in SPC B, ensures good ductility capacity and maximizes earthquake resistance.

Disadvantages: The additional reinforcement adds, albeit minimally, to the construction costs in SPC B.

5.3 Column Transverse Reinforcement

Description: The transverse column reinforcement design and detailing in bridges in SPC A follows the AASHTO Standard Specification guidelines. In addition, in Louisiana, circular columns are required to have No. 3 spiral steel at 6" pitch, and are designed as tied columns. Closed ties are used in square columns. In other states such as Pennsylvania, the use of spiral reinforcement is not as common. The spiral reinforcement described above does not meet requirements for seismic reinforcement of plastic hinge zones for SPC C or D.

Bridge columns in SPC B are designed based on AASHTO and additional, state specific, seismic design and detailing requirements. The additional requirements are often more stringent than the AASHTO Seismic Specification 8.3 for SPC B which only pertains to the amount and spacing of transverse reinforcement for confinement at plastic hinges.

Pennsylvania and New York requirements for SPC B are actually very similar to the AASHTO seismic requirements for SPC C of Sec. 8.4.1(D, E and F), as shown in figure 5-6. Sec. 8.4.1(D) pertains to the column shear capacity and was introduced to account for the reduced contribution of the concrete to the column shear capacity within the plastic hinge zone at low axial load levels, and to minimize the potential for column shear failure. Sec. 8.4.1(E) pertains to the ability of the transverse reinforcement in the expected plastic hinge regions to provide adequate confinement and prevent buckling of the longitudinal reinforcement. It defines minimum transverse steel volumetric ratios for spiral and rectangular stirrup reinforcement.

Sec. 8.4.1(F) specifies maximum spacing and extent of transverse reinforcement for confinement. The maximum spacing limit in Sec. 8.4.1(F) is the lesser of 4" or one-quarter of the minimum column dimension, but both Pennsylvania and New York allow a maximum spacing of 6". The transverse confinement reinforcement is required to extend over a length from each column end not less then the maximum cross-sectional column dimension or one-sixth of the clear height of the column, but not less than 18". Lapping of spiral reinforcement in the transverse confinement regions is not permitted. Connection of spiral reinforcement in this region must be full yield strength lap length. Similarly, rectangular stirrups must be adequately anchored by bending ends back into the core at least 135 degrees, with a minimum extension of 10 bar diameters. Transverse reinforcement outside the plastic hinge regions is provided as per AASHTO Standard Specifications.

North Carolina transverse reinforcement requirements for SPC B follow the AASHTO Seismic Specification 8.3, with the additional detailing requirements shown in figure 5-7. All round columns are required to have No. 4 spiral steel with spacing between spirals set at 3-inch centers for the whole length of the column. The splice of the spiral column reinforcement is lapped 24". Representative column transverse reinforcement details are shown in figures 5-6 through 5-9.

Advantages: Using spiral reinforcement in all circular columns, even spiral reinforcement that does not satisfy the requirements for pitch applicable to the plastic hinge zones in SPC C and D, improves the confinement of the main longitudinal steel somewhat due to the hoop stress action. Good confinement of the longitudinal steel, especially in plastic hinge areas, adds significantly to the ductility capacity of the columns and minimizes the susceptibility to damage from earthquakes. Keeping the spiral spacing constant for the entire column length simplifies construction. Closed ties are used in square columns. Square columns allow for an easier placement of the cap reinforcing bars.

Disadvantages: Providing the additional transverse reinforcing steel requirements in the plastic hinge areas and keeping the spiral pitch constant along the column length adds to the amount of steel required.

Historical: During the 1971 San Fernando and the 1989 Loma Prieta earthquakes many bridge columns failed due to a lack of confinement. Following the 1971 San Fernando earthquake, column detailing requirements have been changed to ensure adequate confinement, and after the 1989 Loma Prieta earthquake many states in the eastern U.S. have added to the transverse reinforcement requirements for SPC B. Bridges in SPC B designed prior to the current seismic requirements do not have adequate transverse reinforcement in the plastic hinge regions, but if the axial stress ratio is low ($P/f'_cA_g < 0.1$) research by Mander et al. (1993) has shown that the performance should be satisfactory.

5.4 Column Reinforcement Splices

Description: For bridges in SPC A there are no restrictions regarding the type and location of splices in column longitudinal reinforcement beyond those of the AASHTO Standard Specifications. Column reinforcement is usually lapped with dowels at the column base. Also, the AASHTO Seismic Specifications do not have any special requirements for column reinforcement splices for SPC B, but Pennsylvania and New York require that the location of the splices be limited to the center half of the column, as per AASHTO Seismic Specification 8.4.1(F) for SPC C. New York also requires that dowels from the footing extend at least 1/4 of the column height or 10 feet (see figure 5-5) and that splices in the vertical reinforcement be staggered whenever possible. North Carolina has no restrictions on lap splices for SPC B. Representative column reinforcement splice details are shown in figures 5-3a, 5-4a, 5-5 and 5-8.

Advantages: Restricting the location of lap splices away from plastic hinge regions prevents potential failure due to loss of concrete cover as a result of spalling.

Disadvantages: Splicing the longitudinal steel in the center of the column or using continuous unspliced column bars from the footing into the bent cap makes the construction more difficult.

Historical: Splicing the longitudinal reinforcement with dowels cast in the footing at the column base and the loss of bond during seismic shaking is believed to have been the main cause of failure of one of the bridges of the Golden-State-Foothills freeway interchange in the 1971 San Fernando earthquake. In most of the existing bridges in the states studied, which were designed prior to the current seismic requirements, column reinforcement is lap spliced to the footing dowels near the base of the column, which is a plastic hinge region (see figures 5-3a and 5-4a). In New York and Pennsylvania, this practice has been changed. For frame pier bents, however, lap splices at the column base may have adequate structure ductility due to redistribution of moments among the hinges within the frame.

5.5 Extension of Column Reinforcement into Bent Caps and Footings

Description: Vertical reinforcement is required to extend into bent caps for full development length. Pennsylvania requires that longitudinal reinforcement be developed for its overstrength capacity of $1.25f_y$ in SPC B columns, as per AASHTO Seismic Specification 8.4.3 for SPC C.

For SPC B Pennsylvania and New York require that column transverse reinforcement for confinement be continued into the adjoining member for a distance of at least 15" or one half the maximum column dimension (see figure 5-6), as per AASHTO Seismic Specification

8.4.3. The spacing of the reinforcement is usually kept the same as that required in the plastic hinge region. In North Carolina column connections for SPC B are detailed with No. 4 ties at 3-inch centers for a minimum of one-half the column diameter into the cap and footings (see figure 5-8). In Louisiana a minimum of four ties are continued into the footing. Extension of column reinforcement requirements in bent caps and footings are illustrated in figures 5-6 through 5-8.

Advantages: Continuing the column reinforcement into the adjoining members prevents a plane of weakness at the interface, confines the longitudinal steel beyond the face of the column and ensures joint ductility.

Disadvantages: The additional steel needed in the bent cap and footing near the column connection makes the construction more difficult.

Historical: The 1971 San Fernando earthquake when column reinforcement pulled out of footings, has pointed out the importance of continuing the transverse reinforcement into the footing. In the past the transverse reinforcement was stopped at the face of the column connection to the bent cap and the footing. Therefore, many of the existing bridge columns in SPC A and B lack proper transverse reinforcement extension details into footings and column bents (see figures 5-3, 5-12 and 6-4). Vertical bars in columns were often required to extend into cap beams a minimum of 20 bar diameters.

5.6 Column Reinforcing Details at Struts

Description: Struts are sometimes provided in high column bents to reduce the slenderness ratio and pile loading in the transverse direction. There are no specific design and detailing requirements for the column strut connection. A typical spiral column reinforcing detail at a strut joint is shown in figure 5-9. The longitudinal column reinforcement is continuous, and its confinement is also continued in the strut region. A construction joint in the column is usually permitted above the strut level. The strut reinforcement is anchored into the column.

Advantages: The column strut connection reinforcing details usually used allow for ease of construction and, depending on the design criteria used, can be adequate for SPC A and B. The smaller strut cross section and its lighter reinforcement ensures that damage during larger seismic loading occurs in the strut first.

Disadvantages: Since the strut reinforcement is not continuous through the column strut joint and the longitudinal reinforcement is lap spliced right above the strut the column strut connection can not be regarded as a moment resistant ductile joint, and it can not be expected to perform well during severe seismic shaking.

5.7 Pier Wall Design and Reinforcement Details

Description: Piers supporting bridges over railways and located within 25 feet of the centerline of railroad track are required by AREA Specifications to be protected by a reinforced concrete crash wall extending to not less than 6 feet above the top of the rail. When two or more light columns compose a pier, a wall at least 2 feet thick connects the columns. The face of the crash wall extends a distance of at least six inches beyond the face of the column on the side adjacent to the track, and is anchored to the column and footing with steel reinforcement (see figure 5-10). Pier walls used at stream crossings to prevent the accumulation of debris usually have rounded ends and extend at least one foot above design flood elevation.

AASHTO Seismic Specification 8.4.1 requires that a vertical support be designed as a column if the ratio of the clear height to the maximum plan dimensions of the support is equal to or greater than 2.5, and as a pier if this ratio is under 2.5. A pier may be designed as a wall in its strong direction and a column in its weak direction. Wall type piers have low ductility capacity and redundancy in their strong direction and are therefore assigned an R-Factor of 2. There are no special seismic requirements for pier walls in SPC A. For SPC B bridges, Pennsylvania has adopted the requirements of AASHTO Seismic Specification 8.4.2 (for SPC C) for the design of pier walls. Sec. 8.4.2 requires a minimum reinforcement ratio of 0.0025 both horizontally and vertically. The maximum reinforcement spacing either horizontally or vertically is 18", and the shear reinforcement is required to be continuous and uniformly distributed. Representative pier wall reinforcement details are shown in figures 5-10 through 5-13.

Advantages: Providing minimum requirements for horizontal and vertical reinforcement and confining ties increases the strength and the ductility of the wall. The ties usually have a 135 degree hook at one end and a 90 degree hook at the other end, so that placing of the tie would be easier. They are alternately placed on each row of main longitudinal reinforcement to optimize confinement. Using partial height walls reduces the construction costs.

Disadvantages: It is very hard to predict the response of partial height walls during strong seismic shaking. Walls are very stiff in their strong direction and attract large forces towards the connecting columns. Plastic hinging can occur in the columns near the top of the partial height wall. The shorter columns in the partial height wall will also attract larger seismic loads. A bent with a partial height wall can receive significantly higher seismic forces than the other bents that do not have a partial height wall.

Historical: The performance of partial height walls during past earthquakes (especially during the 1994 Northridge earthquake) has been very poor.

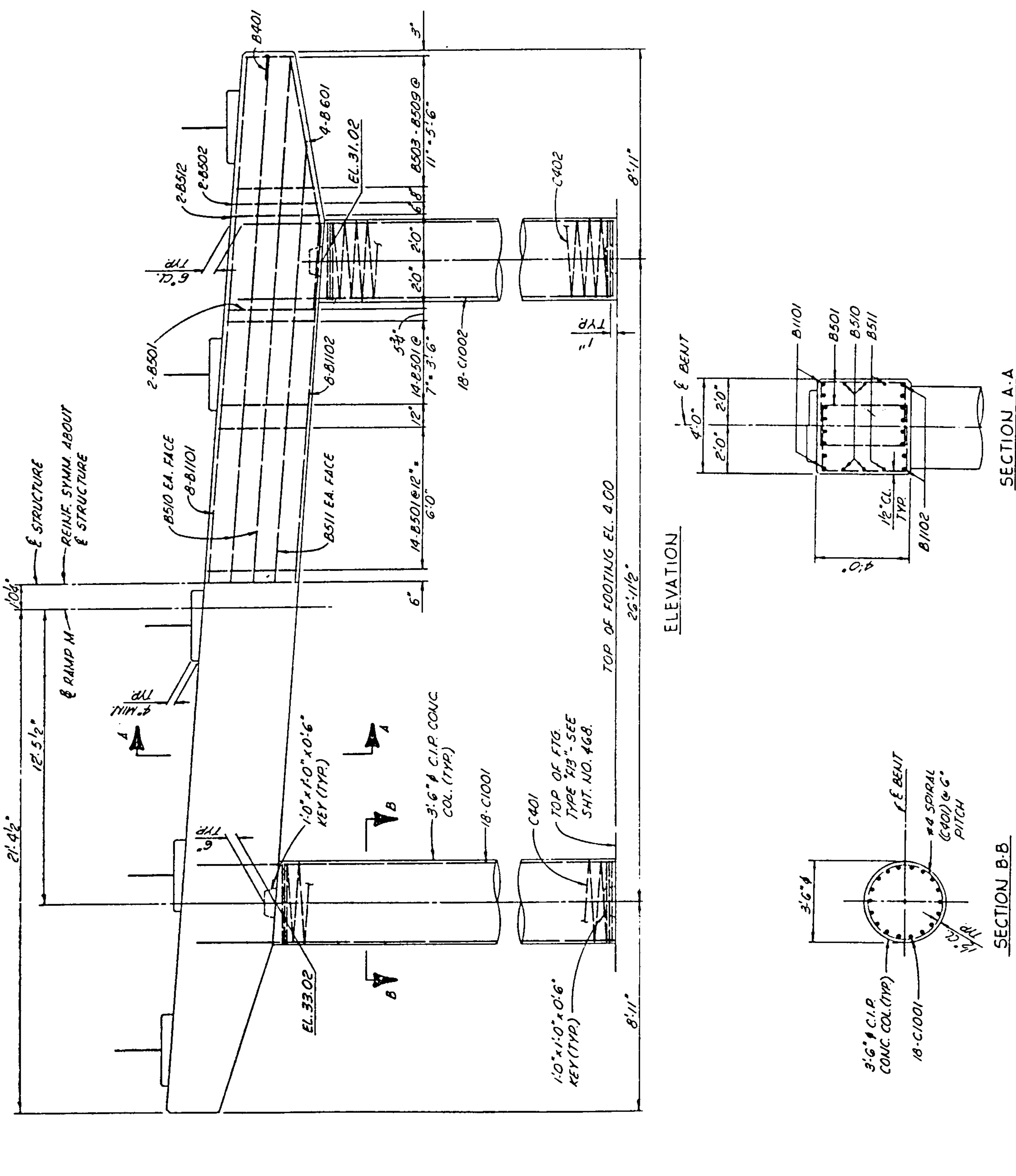


FIGURE 5-1 Louisiana Circular Column Bent Details

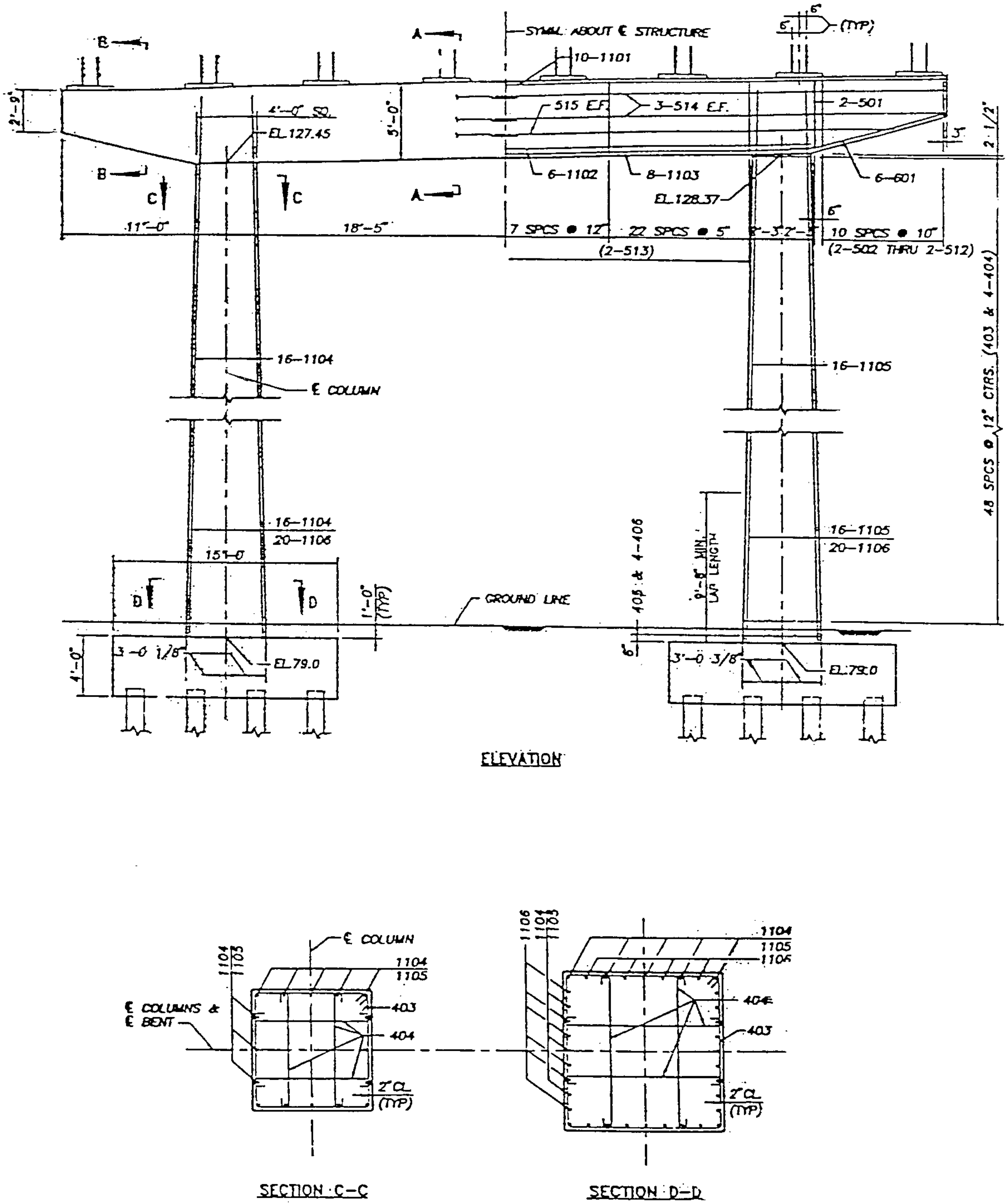


FIGURE 5-2 Louisiana Rectangular Column Bent Details

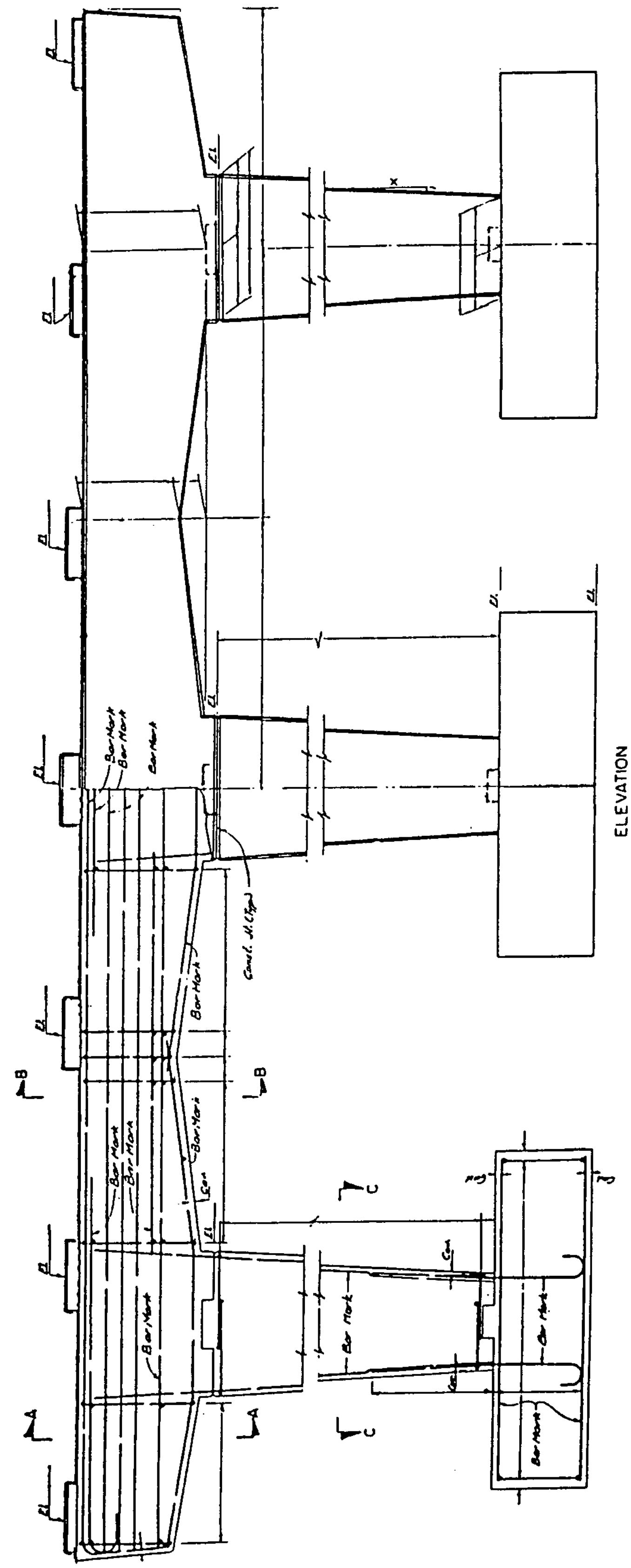


FIGURE 5-3a New York Rectangular Column Bent Details - Elevation

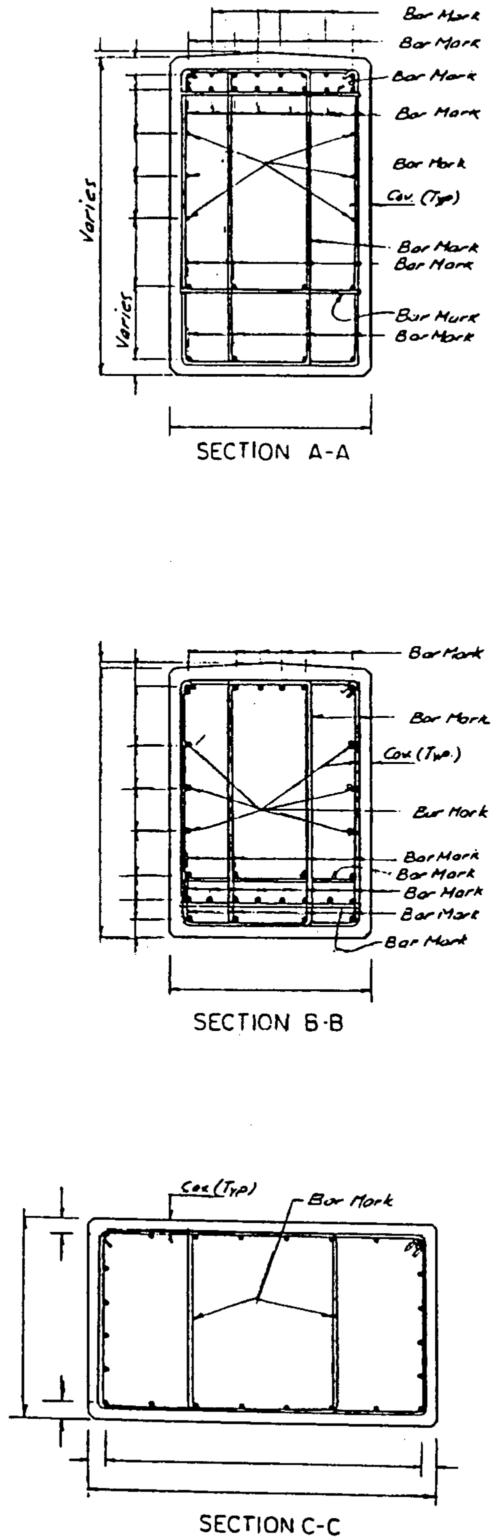


FIGURE 5-3b New York Rectangular Column Bent Details - Cross Sections

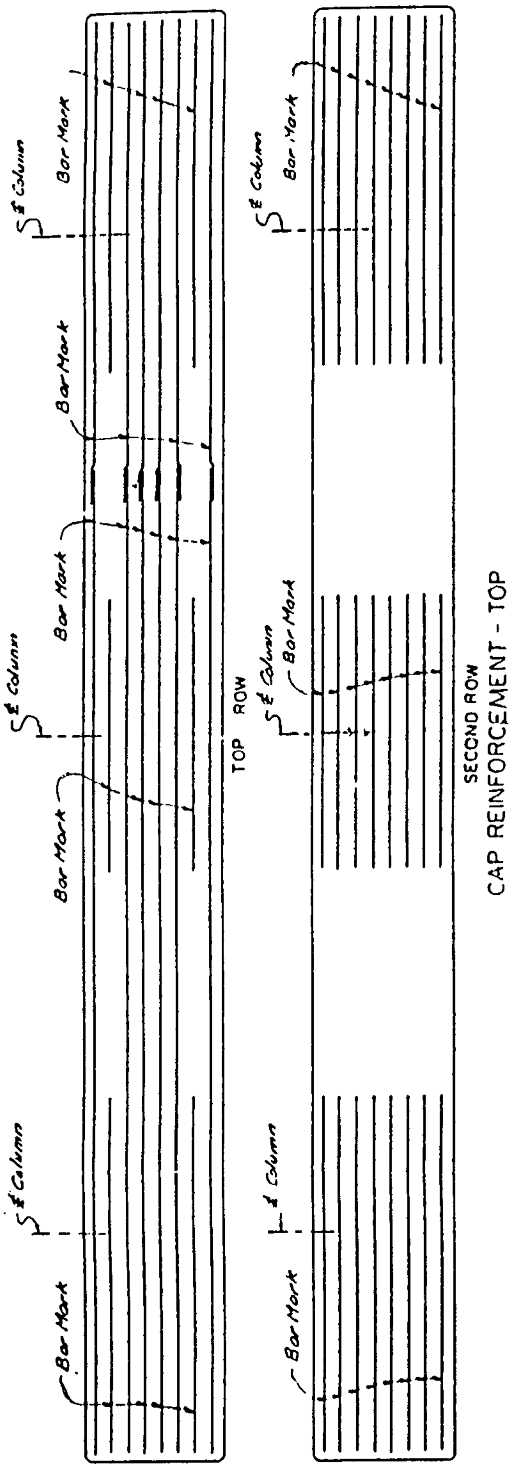


FIGURE 5-3c New York Rectangular Column Bent Details - Cap Reinforcement

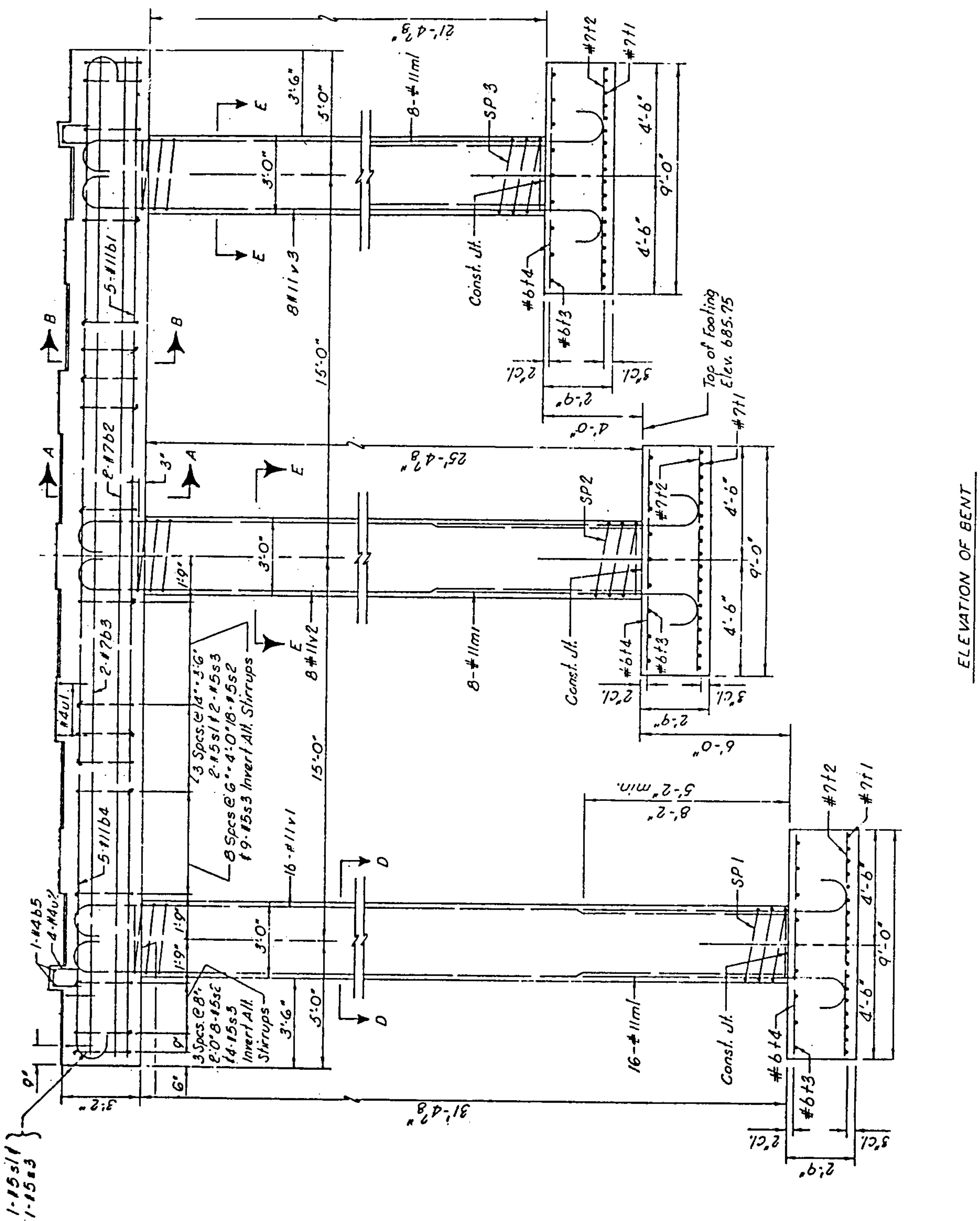


FIGURE 5-4a North Carolina Unequal Column Height Bent Details - Elevation

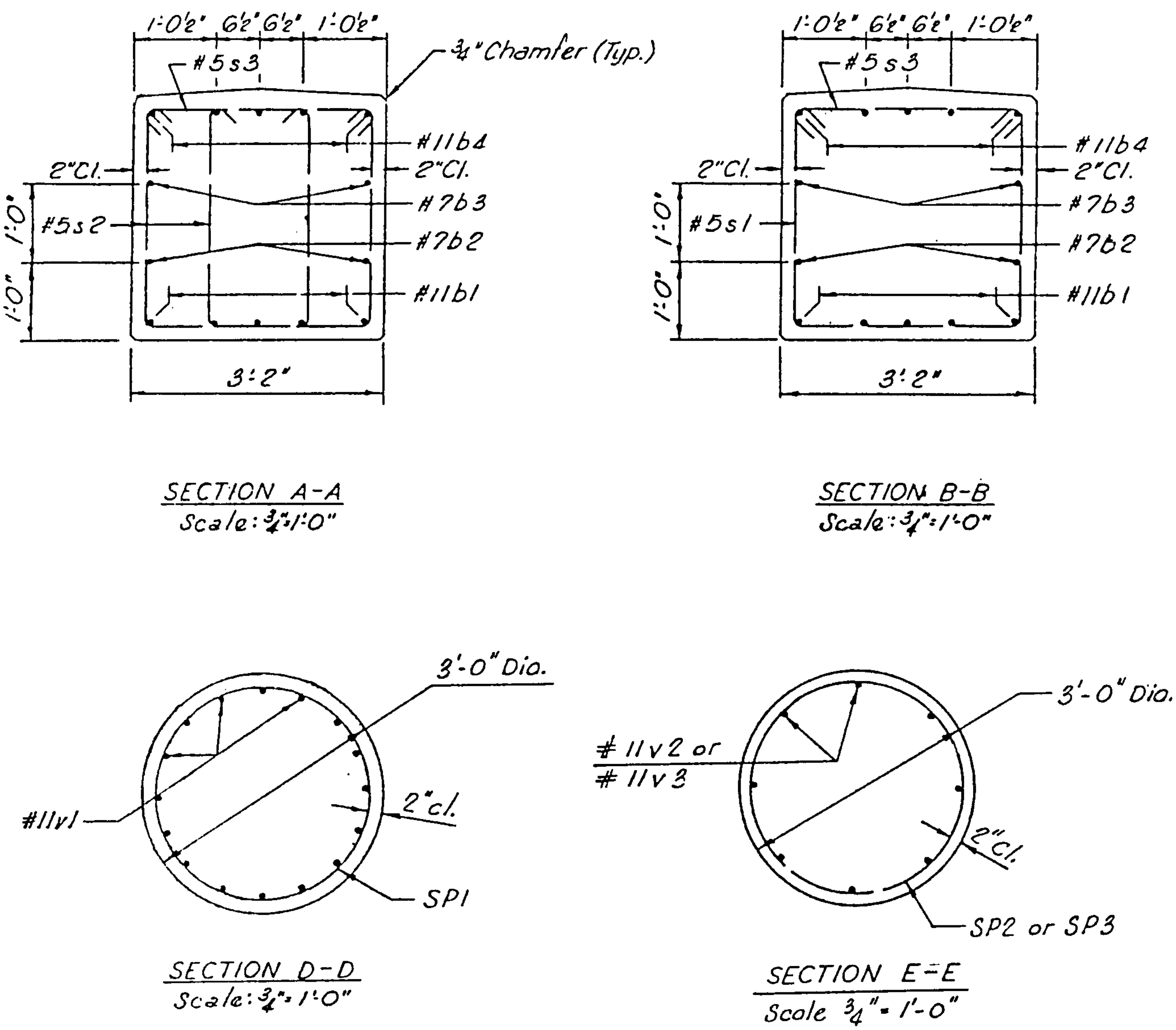


FIGURE 5-4b North Carolina Unequal Column Height Bent Details - Cross Sections

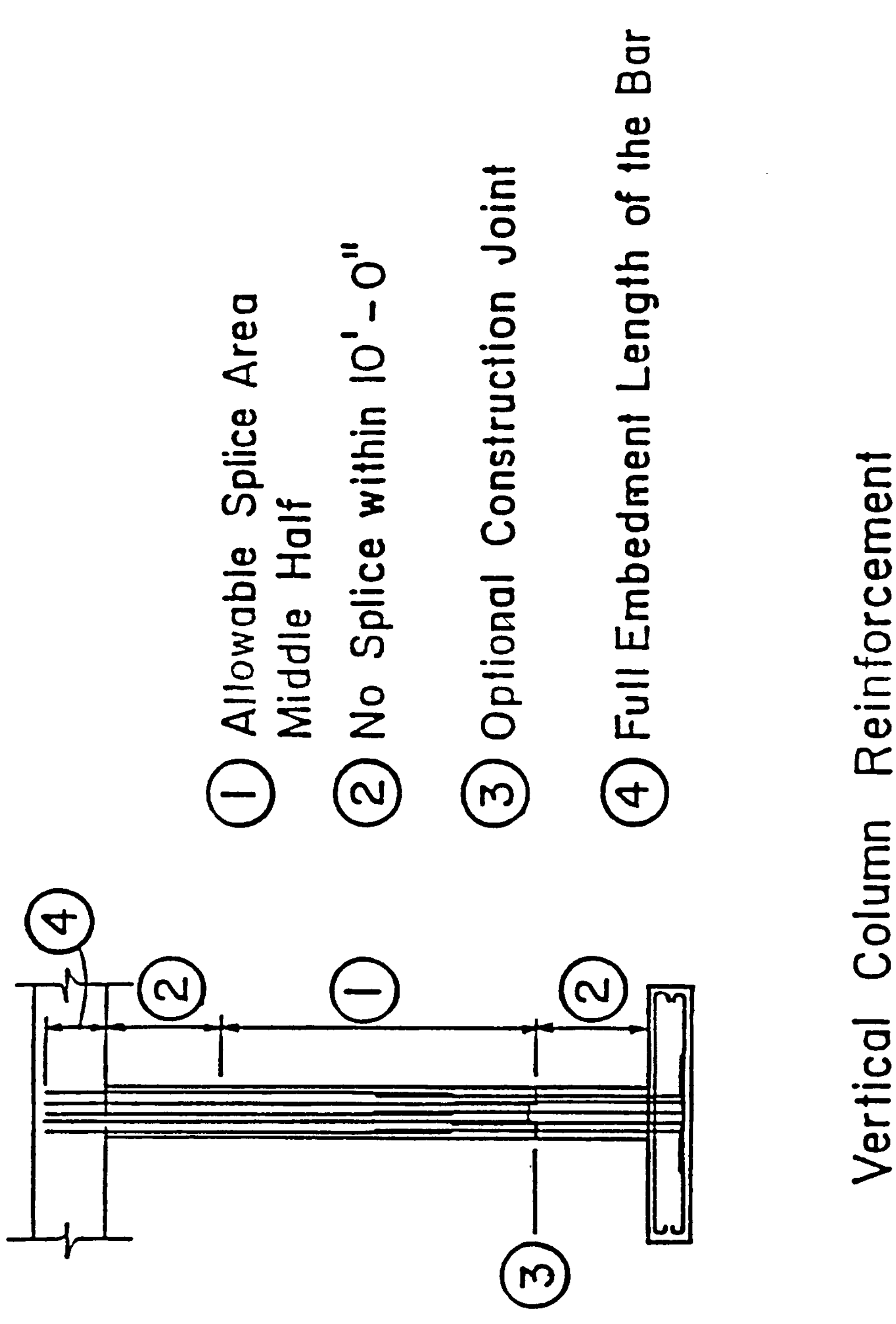


FIGURE 5-5 New York Vertical Column Reinforcement Requirements

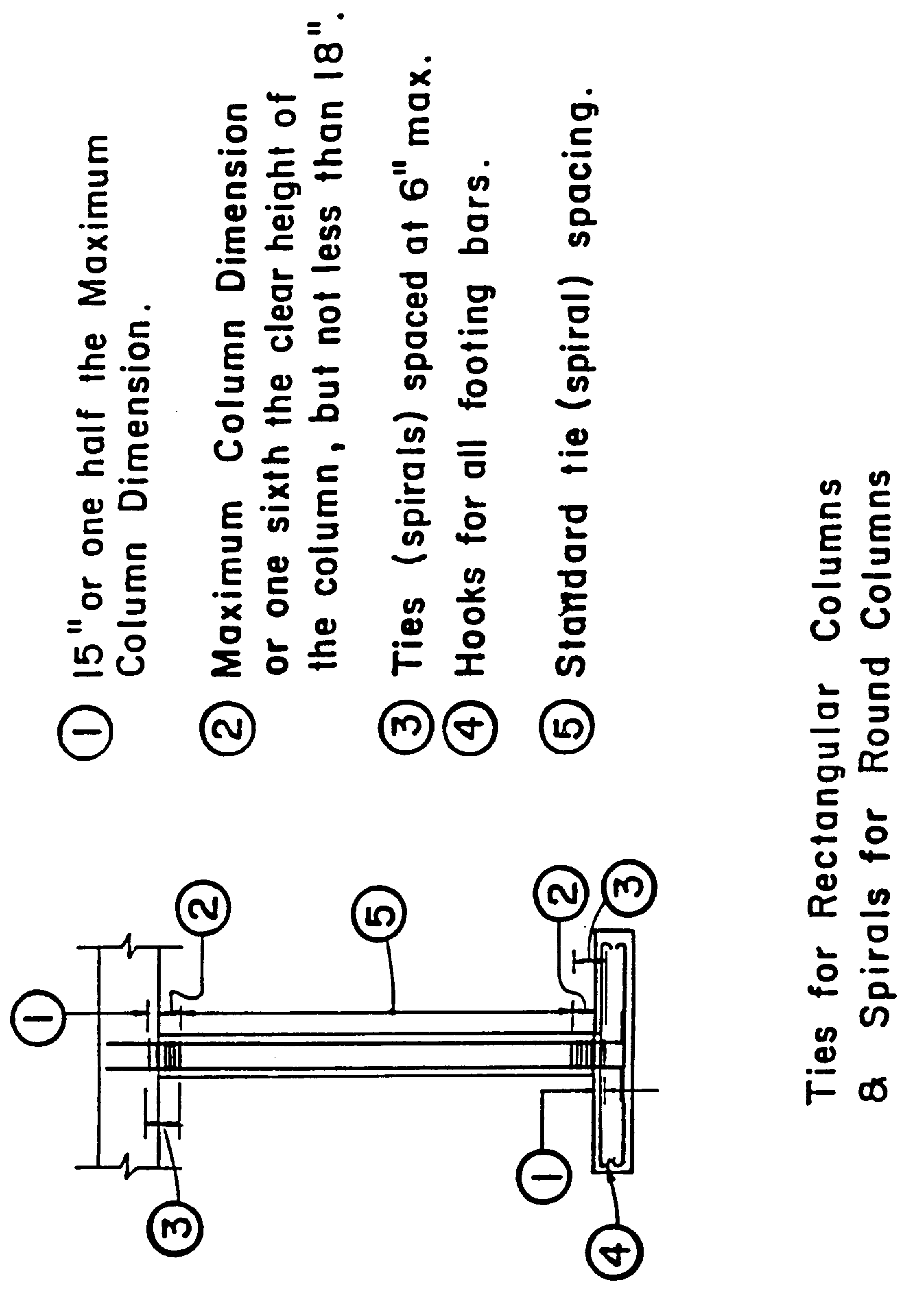


FIGURE 5-6 New York Transverse Column Reinforcement Requirements

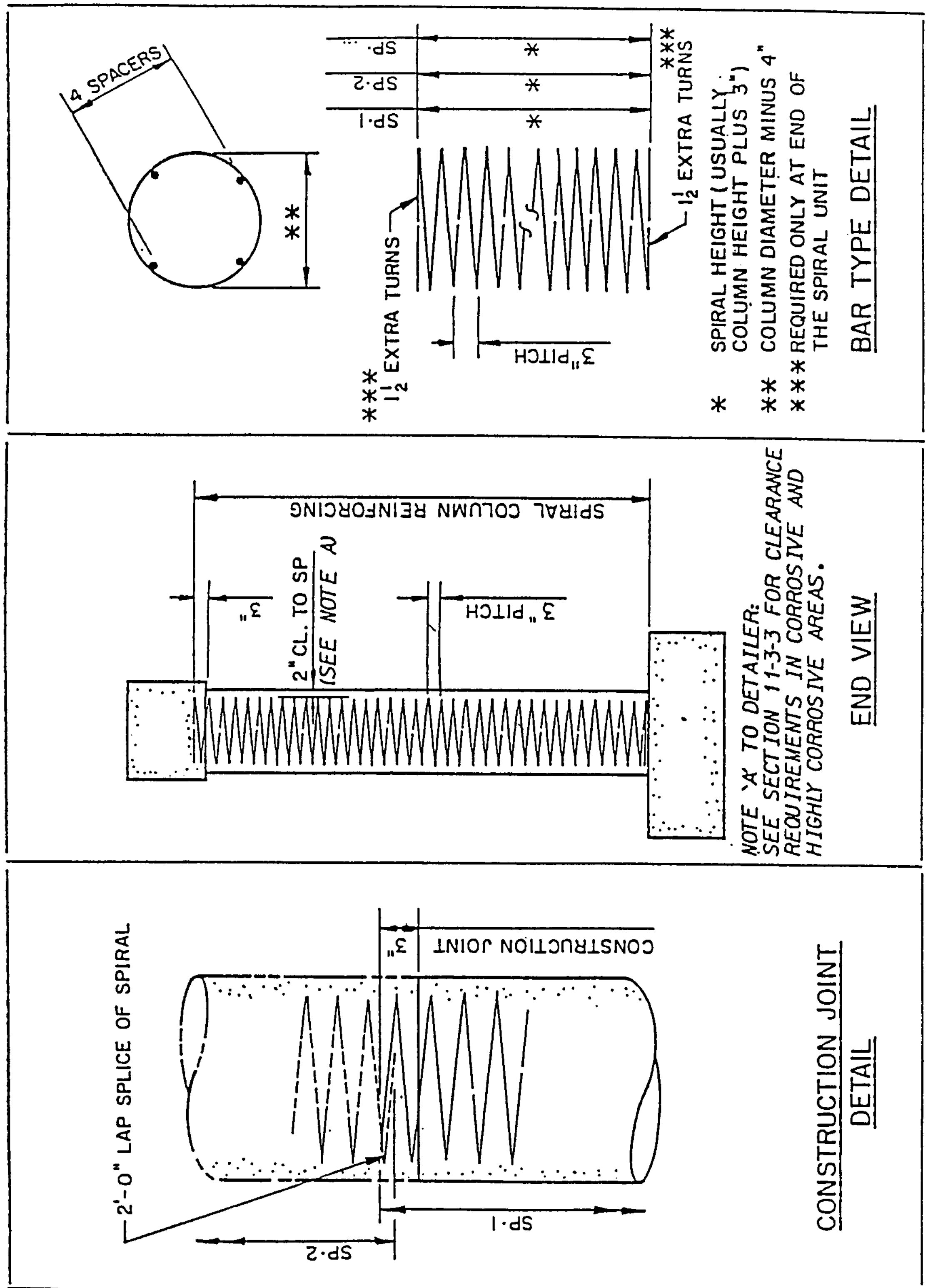


FIGURE 5-7 North Carolina Column Spiral Reinforcing Requirements

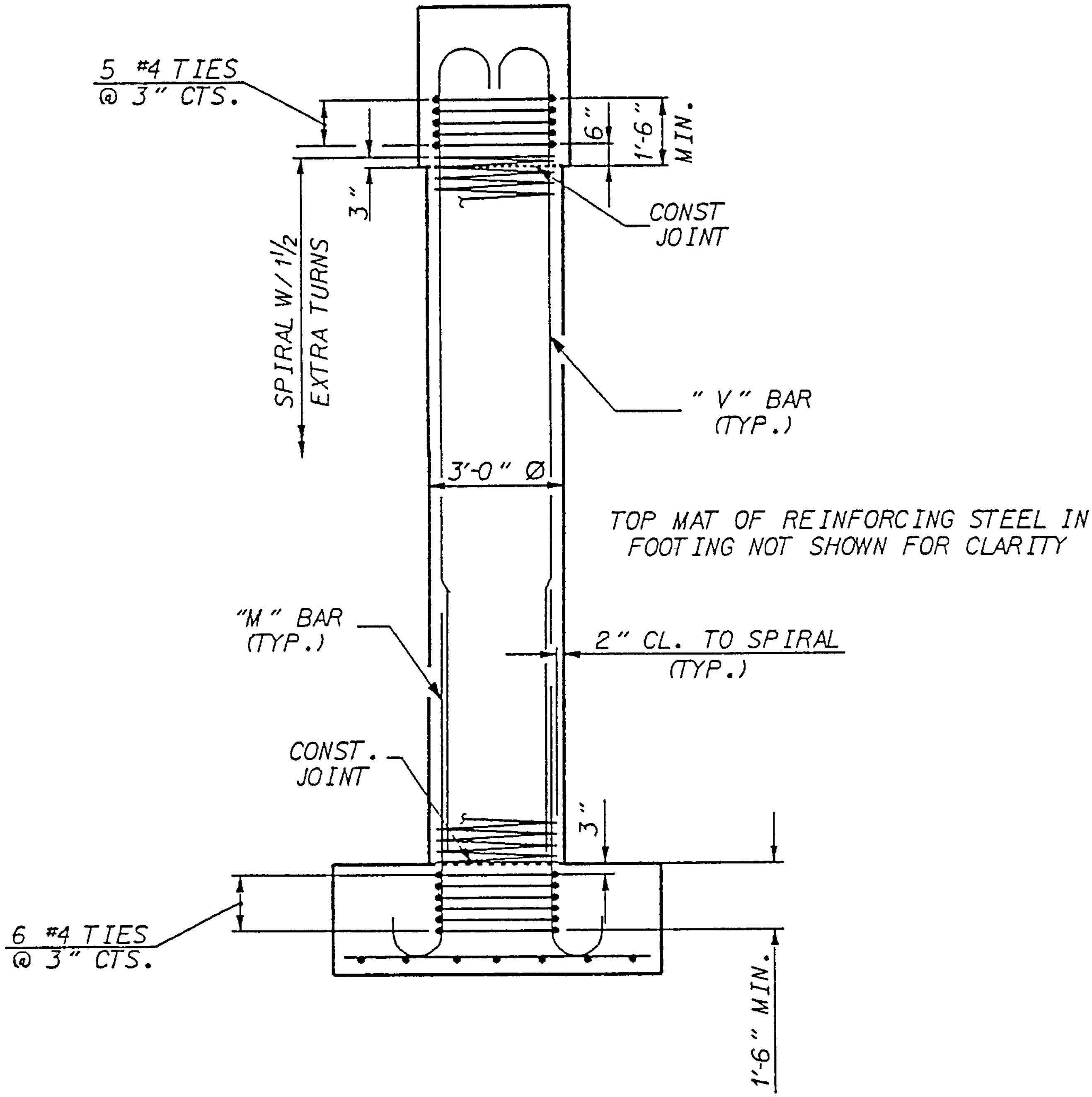


FIGURE 5-8 North Carolina Column Connection Details for SPC B

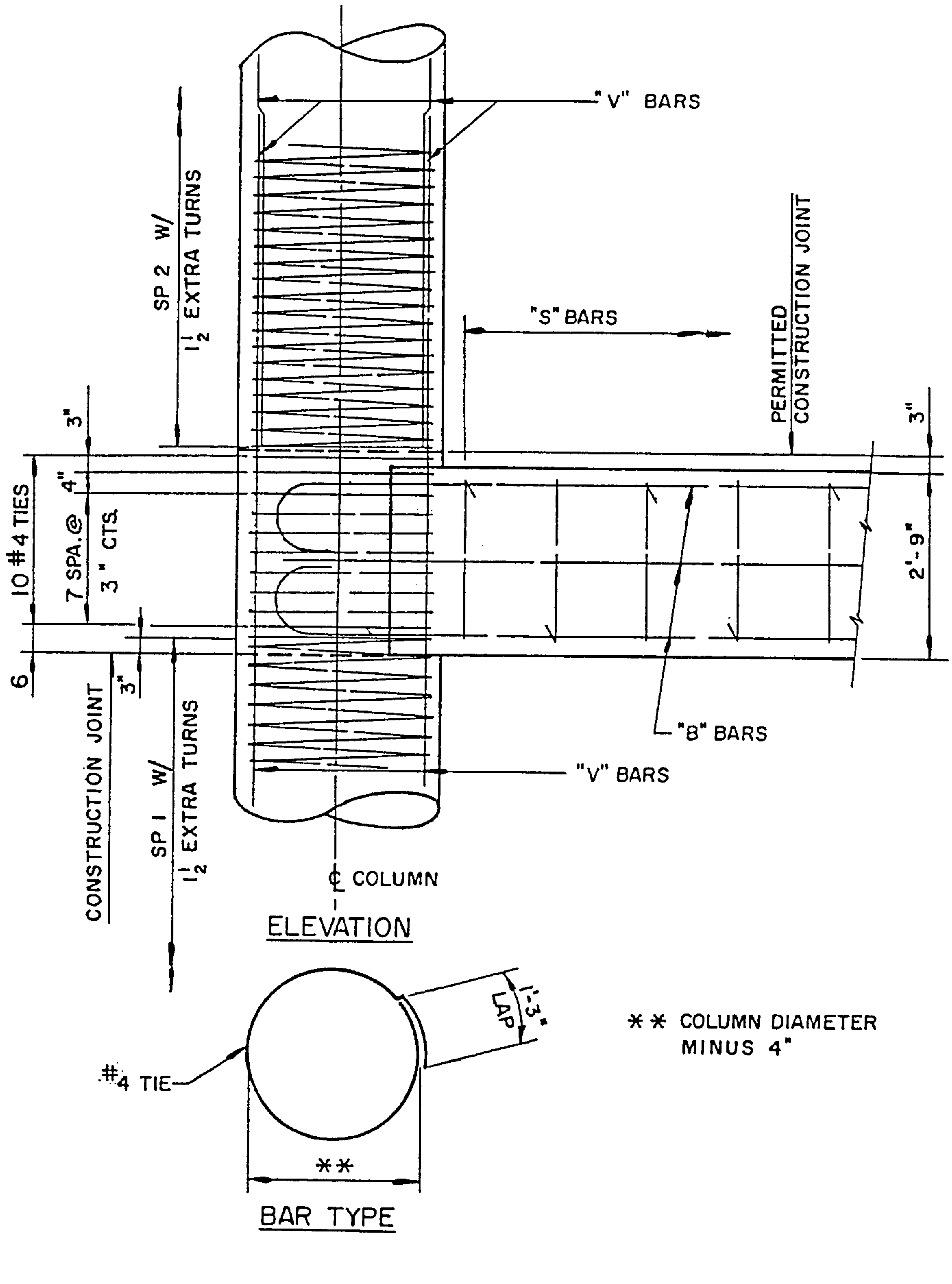


FIGURE 5-9 North Carolina Column Spiral Reinforcing at Struts

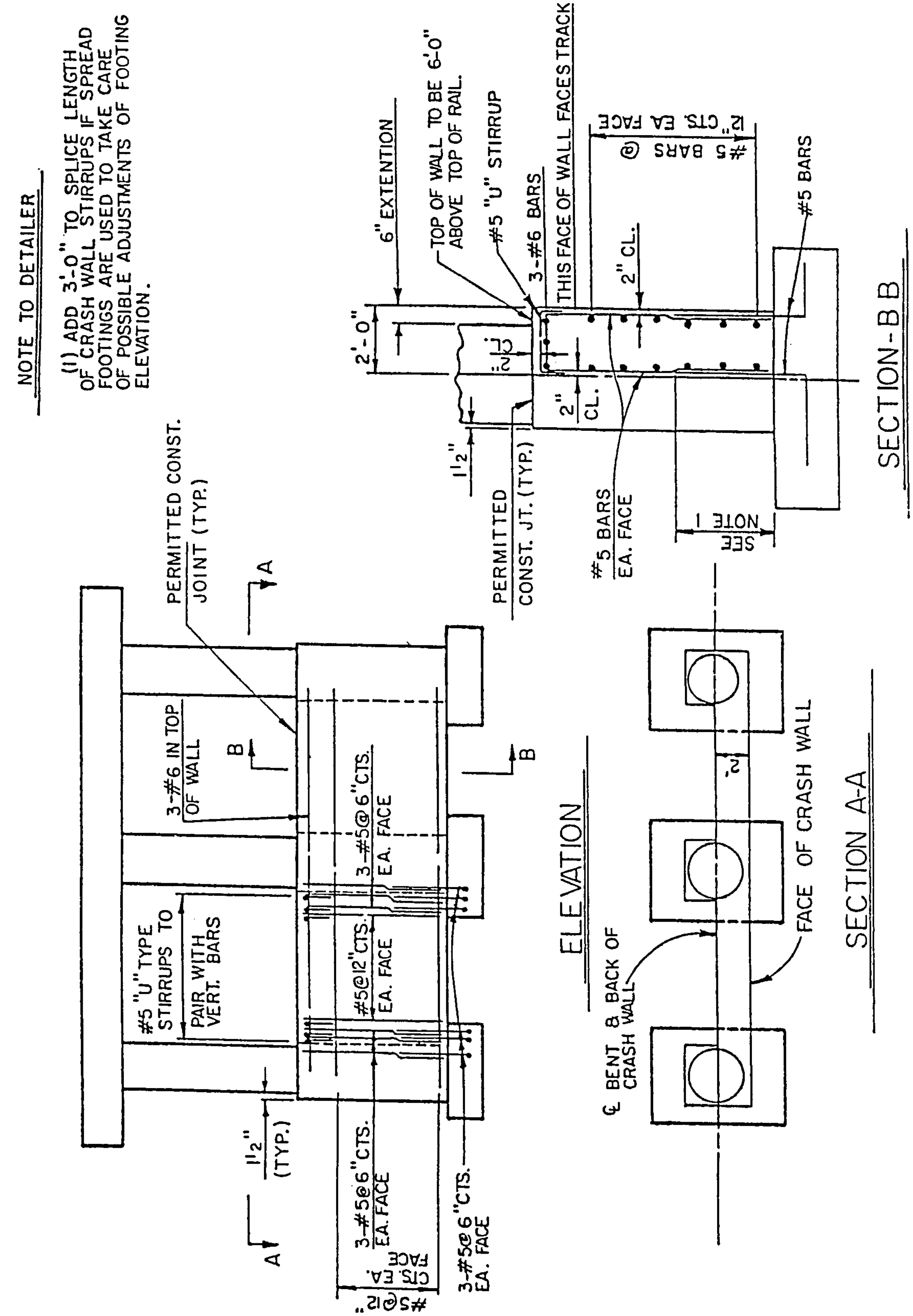


FIGURE 5-10 North Carolina Pier Crash Wall Details for Multicolumn Bents

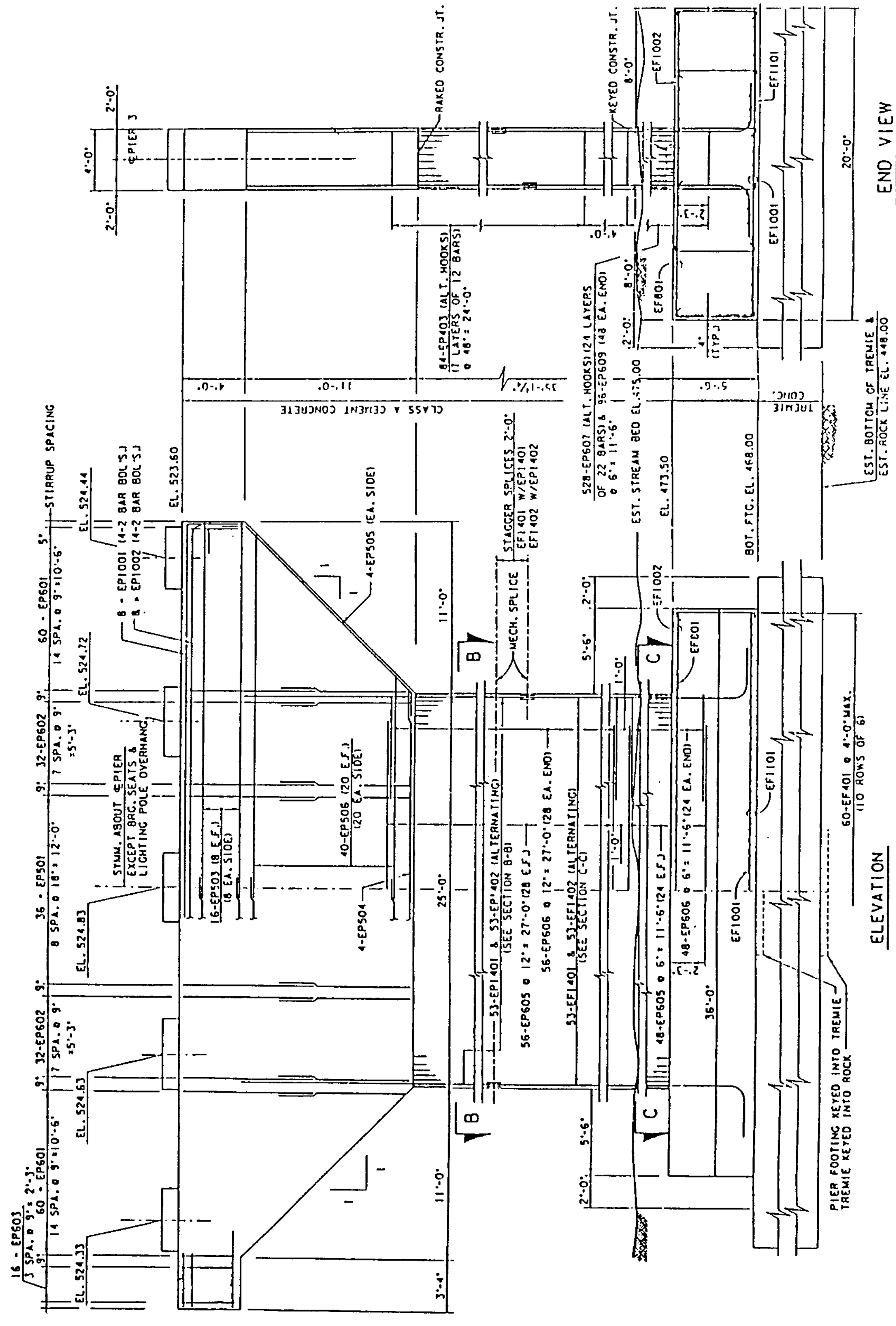
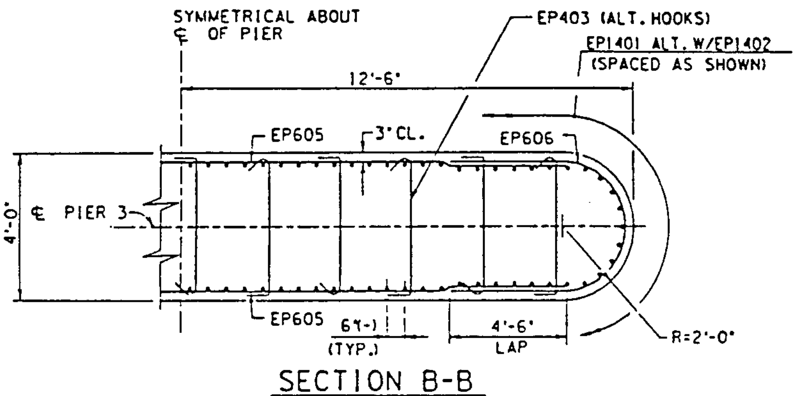


FIGURE 5-11a Pennsylvania Pier Wall with Hammerhead Details - Elevation



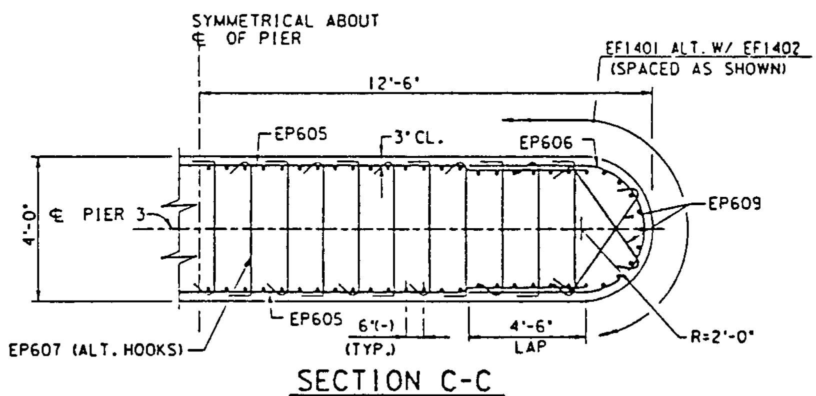


FIGURE 5-11b Pennsylvania Pier Wall with Hammerhead Cap Details - Cross Sections

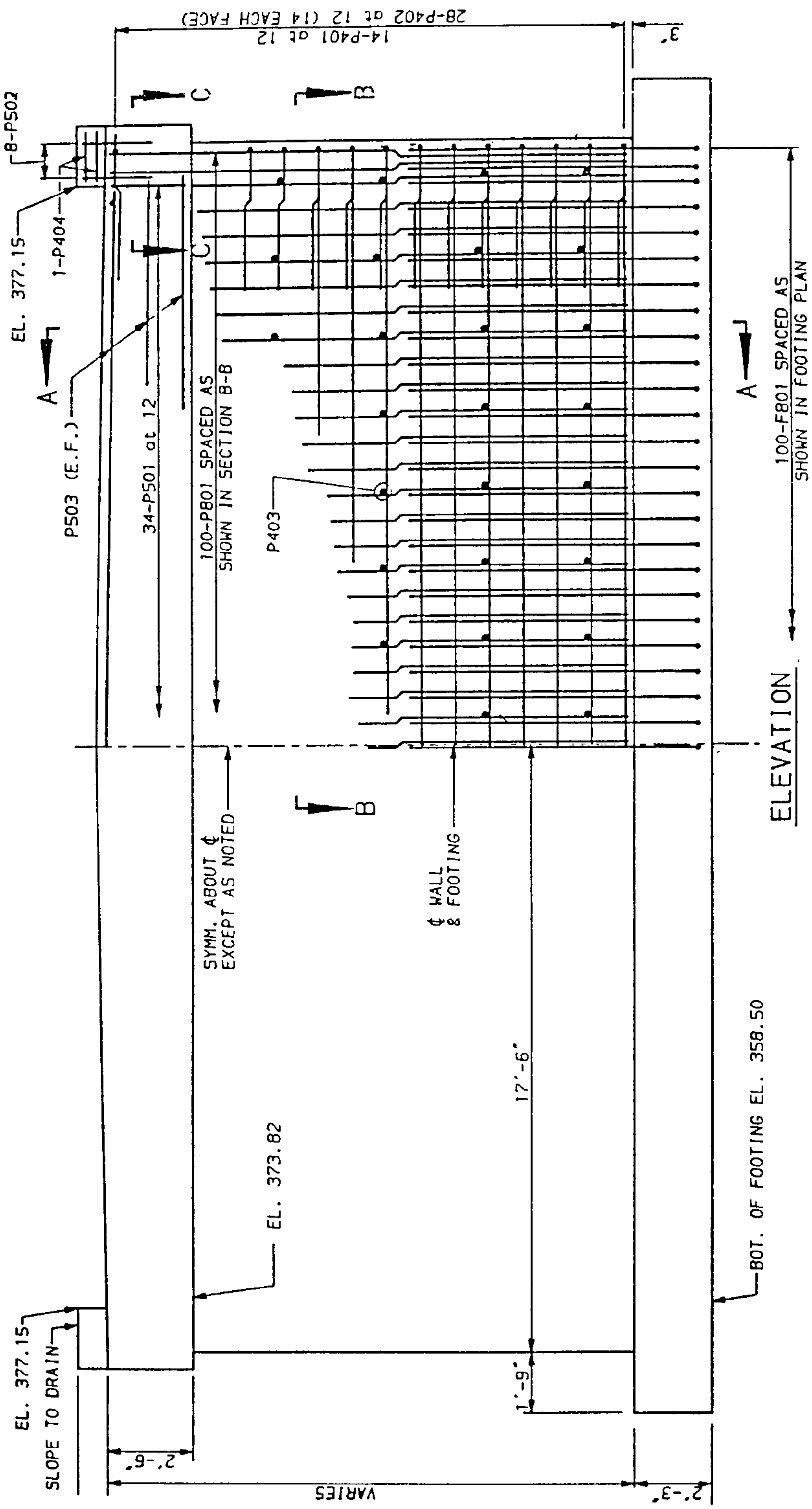


FIGURE 5-12a Pennsylvania Pier Wall with Widened Top Cap Details - Elevation

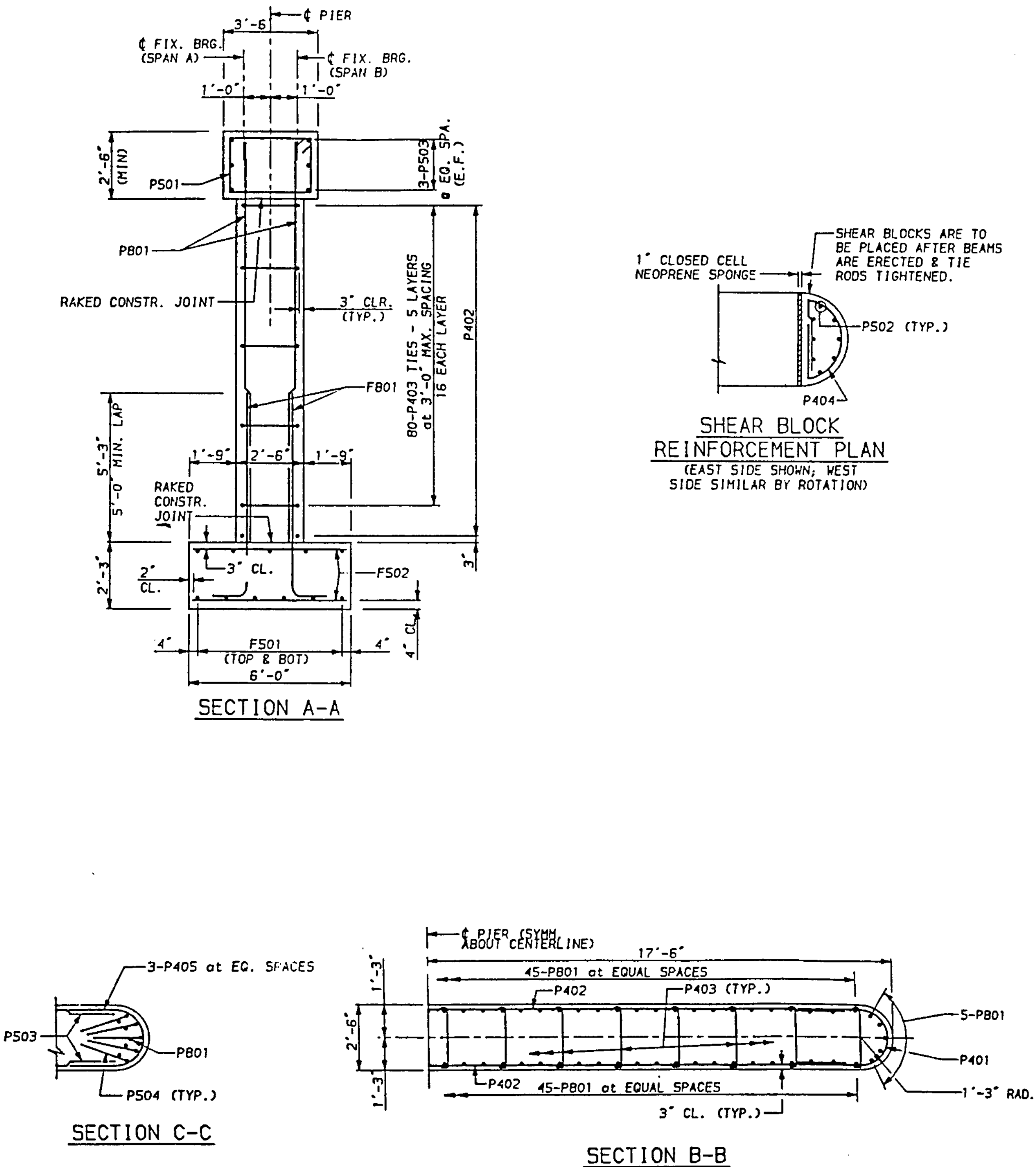


FIGURE 5-12b Pennsylvania Pier Wall with Widened Top Cap Details - Cross Sections

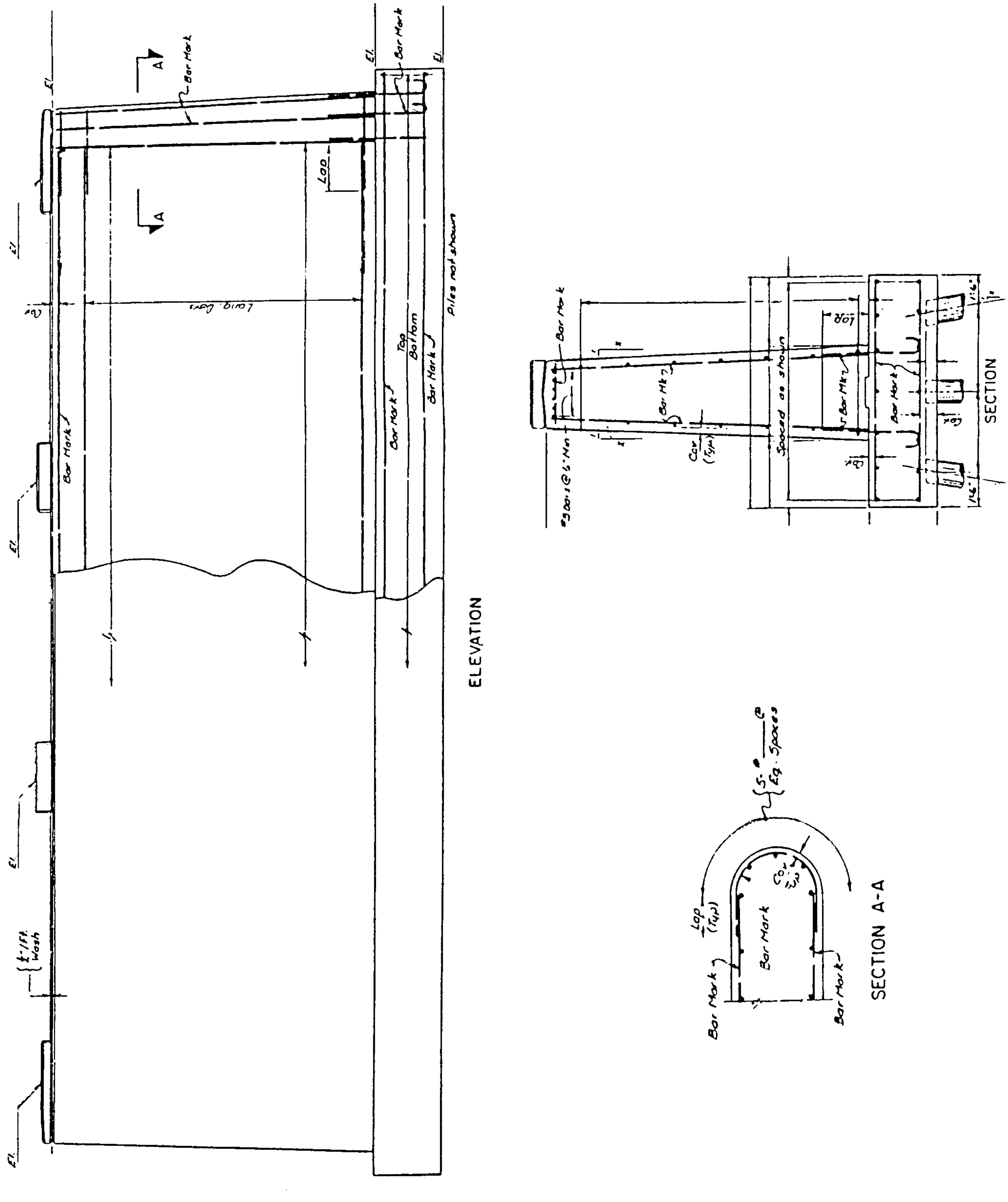


FIGURE 5-13 New York Pier Wall Details

SECTION 6
BENT CAP DESIGN AND DETAILING ISSUES IN COLUMN BENTS

6.1 Cap Dimensions

Description: There are three basic types of caps that are most common; the rectangular beam type with the superstructure on top (see figure 6-1), the hammerhead used for the single column bents (see figures 6-2, 6-3a and 6-4a), and the inverted-T cap (see figures 6-5 and 6-6). The inverted-T bent cap type is usually used for aesthetic reasons, or when it is necessary to reduce the overall depth of the cap plus the superstructure. The minimum width of the rectangular beam caps in Louisiana is at least 4" greater than the column diameter or the required clearance dictated by the bearing areas. Pennsylvania recommends up to 12" wider caps than the thickness or diameter of the columns. In North Carolina the minimum size of the bent caps is 3'-2" wide by 2'-6" deep. The minimum bent cap width is also required to meet the bearing support length provisions of the AASHTO Specifications for Seismic Design, Section 4.9.1 for various span lengths and column heights. The cap length is required to extend at least 9 inches beyond the edge of the extreme bearing plate on steel superstructures or beyond the anchor bolts on prestressed girder superstructures. Representative bent cap details are shown in figures 6-1 through 6-6.

Advantages: The wider cap section provides a higher flexure capacity, forcing plastic moments to develop in the columns. Also, it reduces the conflict between column and bent cap reinforcement.

6.2 Longitudinal Cap Steel

Description: The longitudinal cap steel is designed in accordance with the AASHTO Specifications. Both top and bottom longitudinal steel is provided. No. 4 bars at 12" spacing are required on the vertical faces of caps for shrinkage and temperature. No. 5 bars are required for massive caps. For hammer head pier caps, Pennsylvania requires that all the calculated cantilever reinforcement be extended throughout the entire length of the cap. In deep caps, additional longitudinal bars are placed at intervals throughout the depth of the cap. No. 5 bars spaced at 12" maximum are required at the bottom of hammer head caps. Representative longitudinal cap steel details are shown in figures 5-1 through 5-4 and 6-1 through 6-6.

Advantages: When the top and bottom longitudinal reinforcement is made continuous along the bent the seismic capacity of caps that may be subjected to load reversals is significantly increased.

Disadvantages: Continuous reinforcement over the cap makes construction more difficult, and may result in conflict of steel at column and bent connection.

Historical: Past performance of bent caps during earthquakes has shown that when the bottom reinforcement stops at the column, cap damage near the column may occur before the column yields as a result of the high positive moments that can be induced by seismic loading.

6.3 Cap Shear Reinforcement

Description: The maximum stirrup spacing in a bent cap is 12". In Louisiana, the first stirrup adjacent to the surface of a column is located at a maximum of 3" away from the column and the first space is under 6". Stirrups are closed, and the minimum size used is No. 4 bars. In North Carolina, stirrups consist of U-shaped bars with 135 degree hooks ends closed by a tie with 135 degree hooks. Alternate stirrups are inverted. The end of caps are reinforced with U-shaped bars. No. 4 U-shaped stirrups at 6-inch centers are placed beneath the bearing area of each line of girders or beams. Additional reinforcement requirements are specified for the top reinforcing steel of stepped bent caps (see figure 4-10). For hammer head caps, Pennsylvania requires that additional stirrups be placed in the cap within the limits of the shaft. It is recommended that stirrups be closely spaced near the ends of the shaft than in the interior region. Representative cap shear reinforcement details are shown in figures 5-1 through 5-4 and 6-1 through 6-6.

Advantages: The closed stirrups provide good confinement to the longitudinal steel and increase the cap ductility and torsional capacity. The closely spaced stirrups near the column increase the local cap capacity near the column locations.

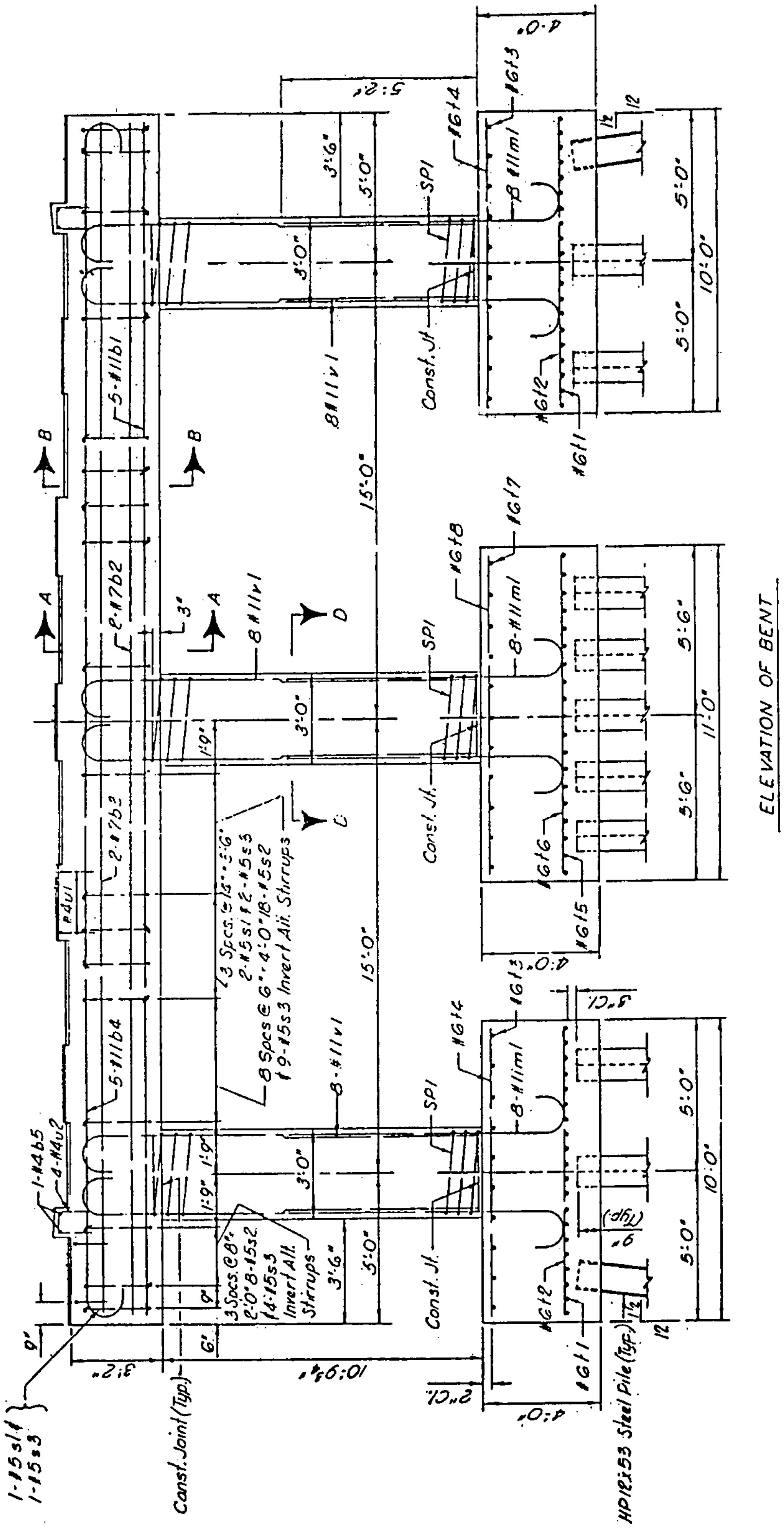


FIGURE 6-1 North Carolina Column Bent with Rectangular Cap Details

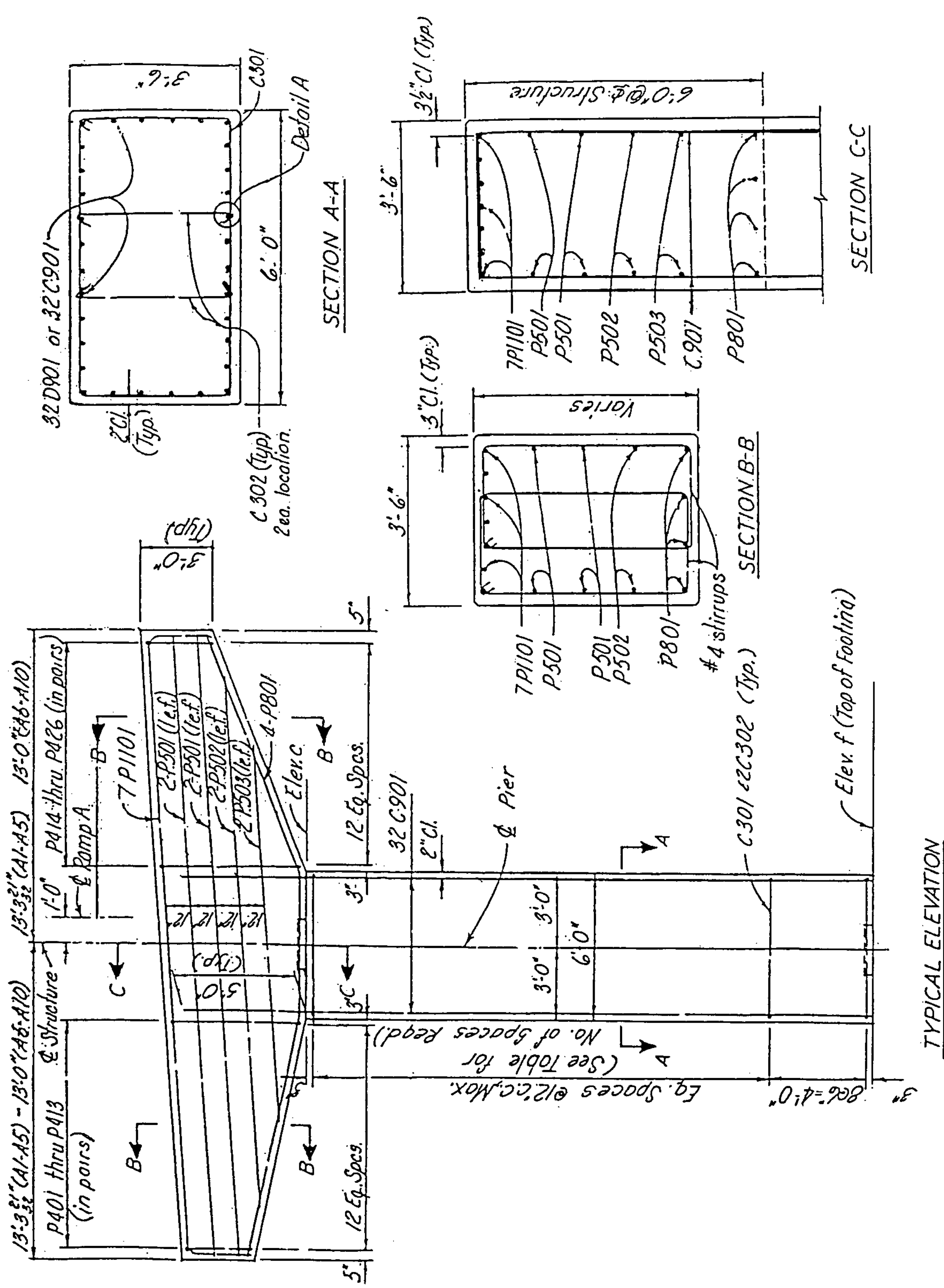


FIGURE 6-2 Louisiana Single Column Bent with Hammerhead Cap Details

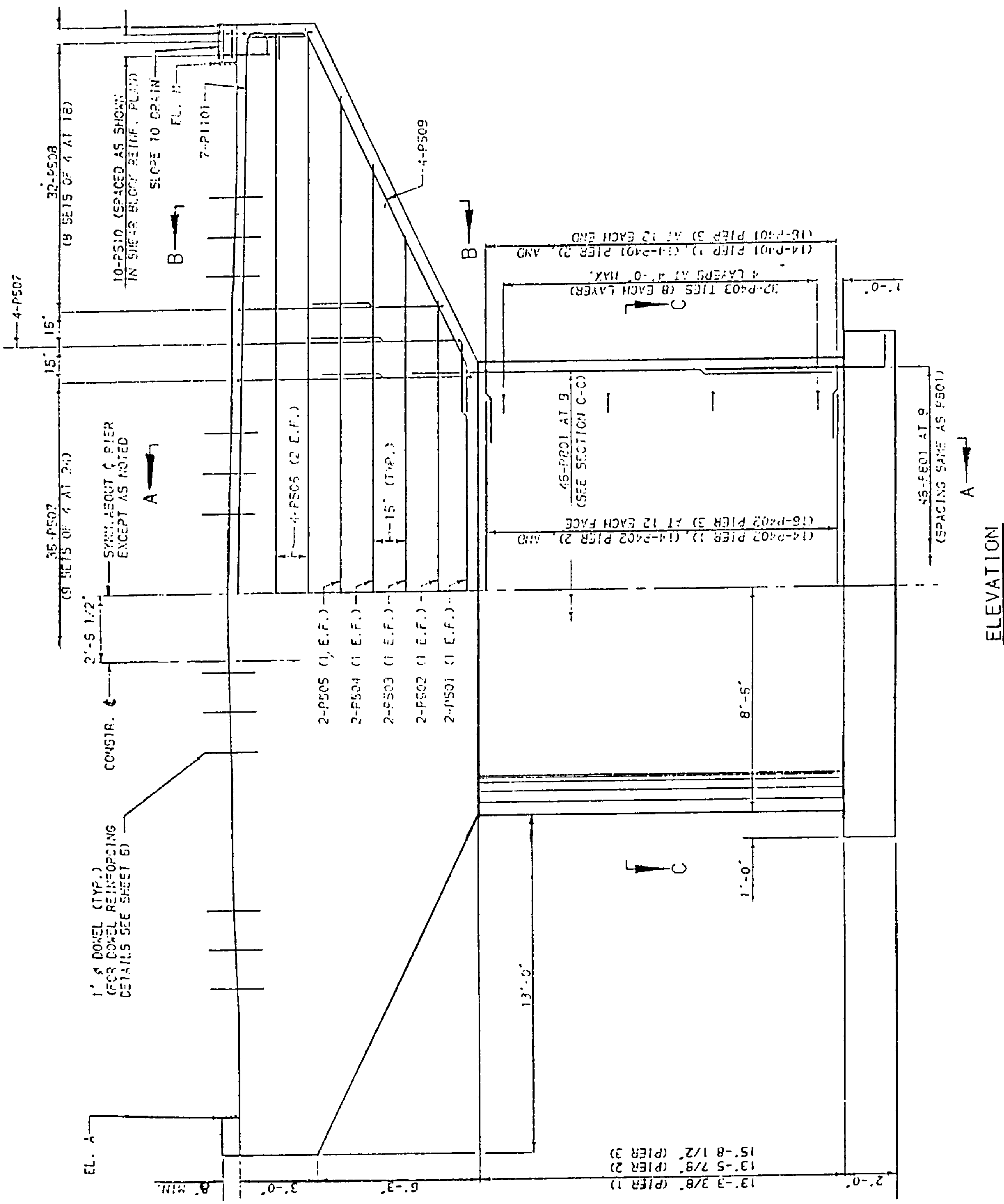


FIGURE 6-3a Pennsylvania Hammerhead Cap Details - Elevation

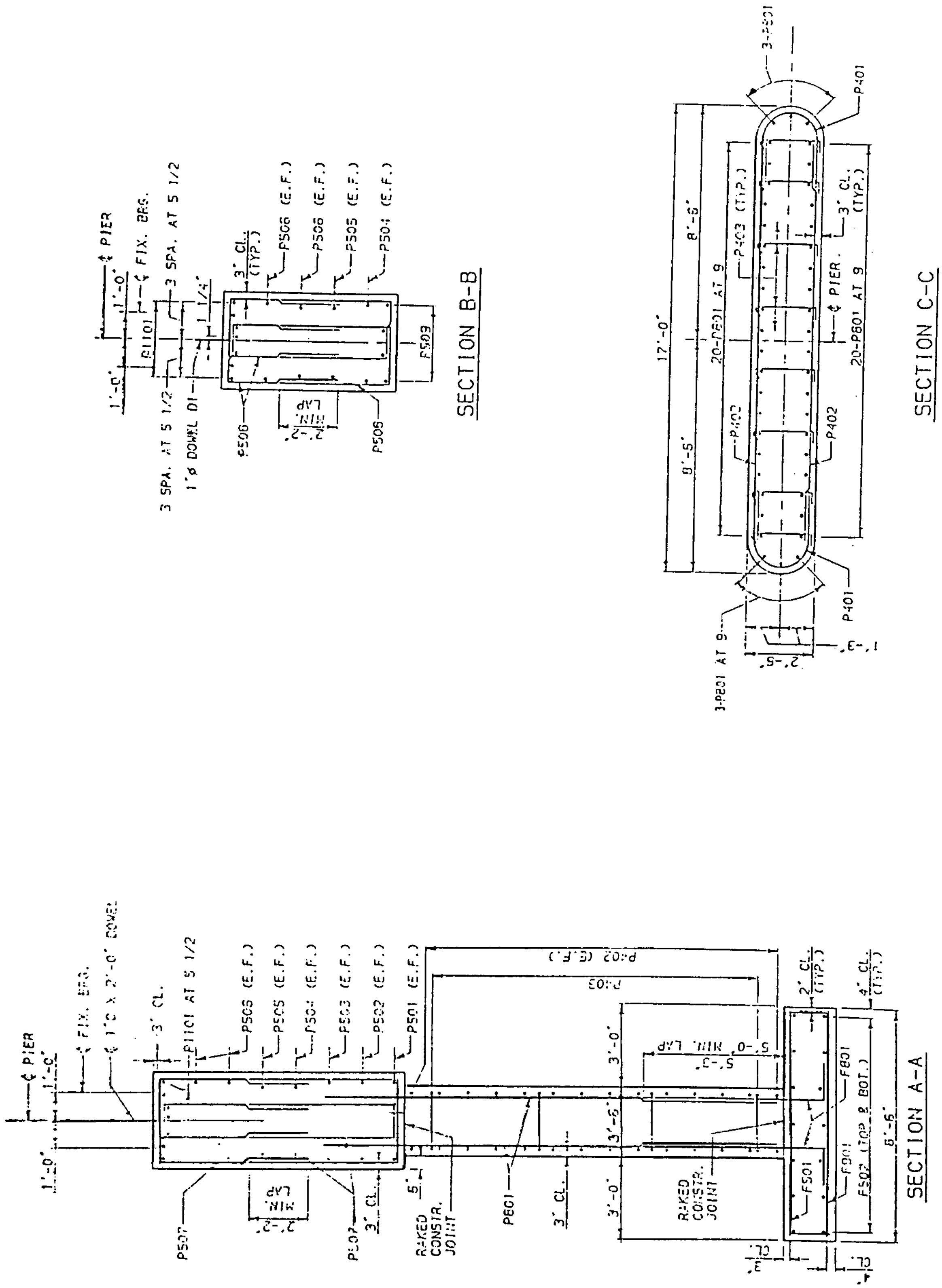


FIGURE 6-3b Pennsylvania Hammerhead Cap Details - Cross Sections

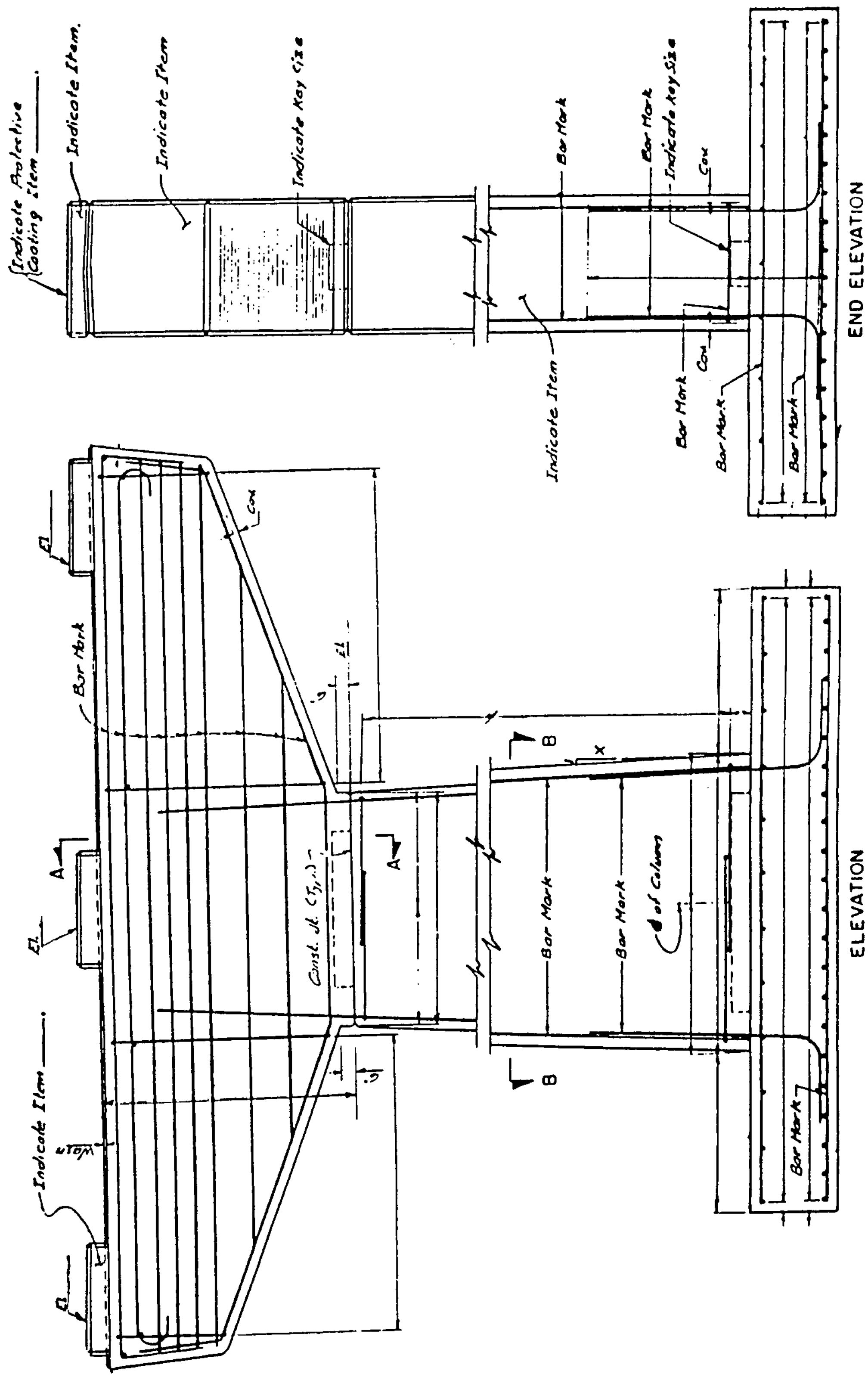
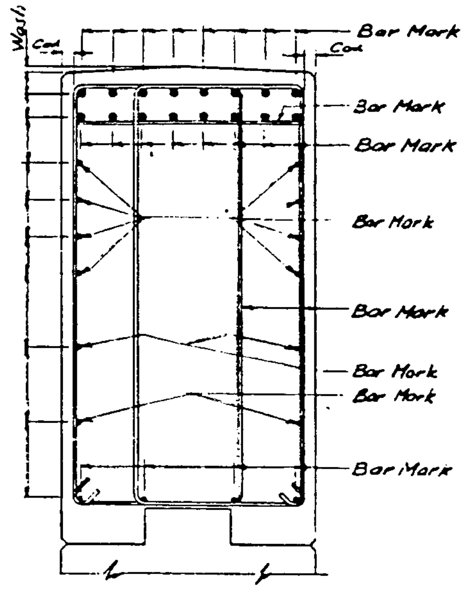
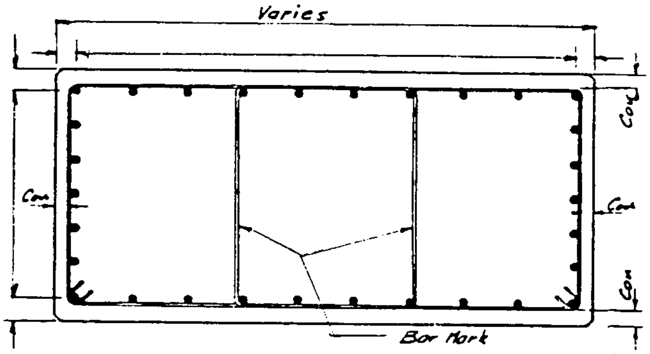


FIGURE 6-4a New York Hammerhead Cap Details - Elevations



SECTION A-A



SECTION B-B

FIGURE 6-4b New York Hammerhead Cap Details - Cross Sections

INVERTED T BENT CAP DETAILS

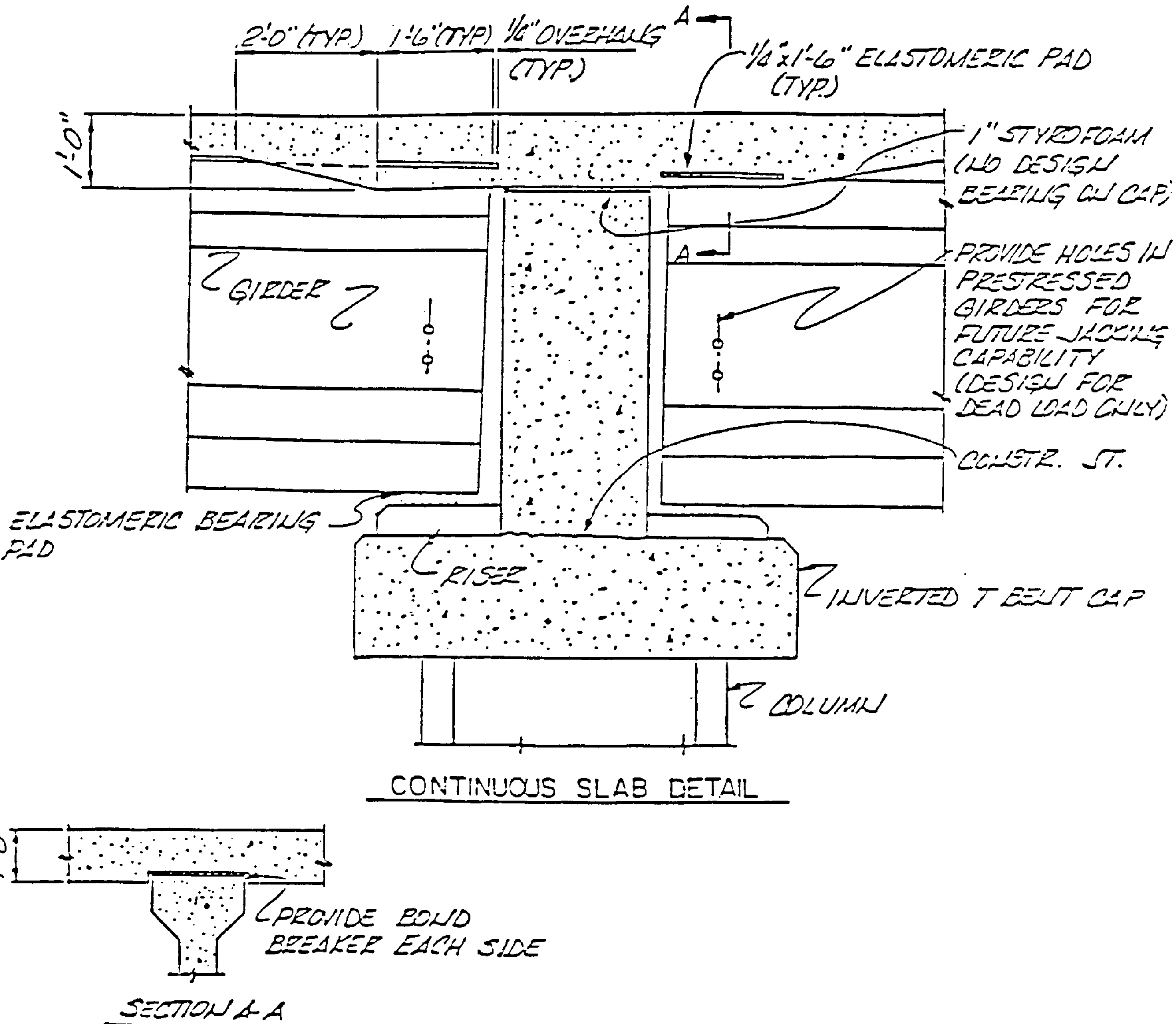


FIGURE 6-5 Louisiana Inverted T-Bent Cap Requirements

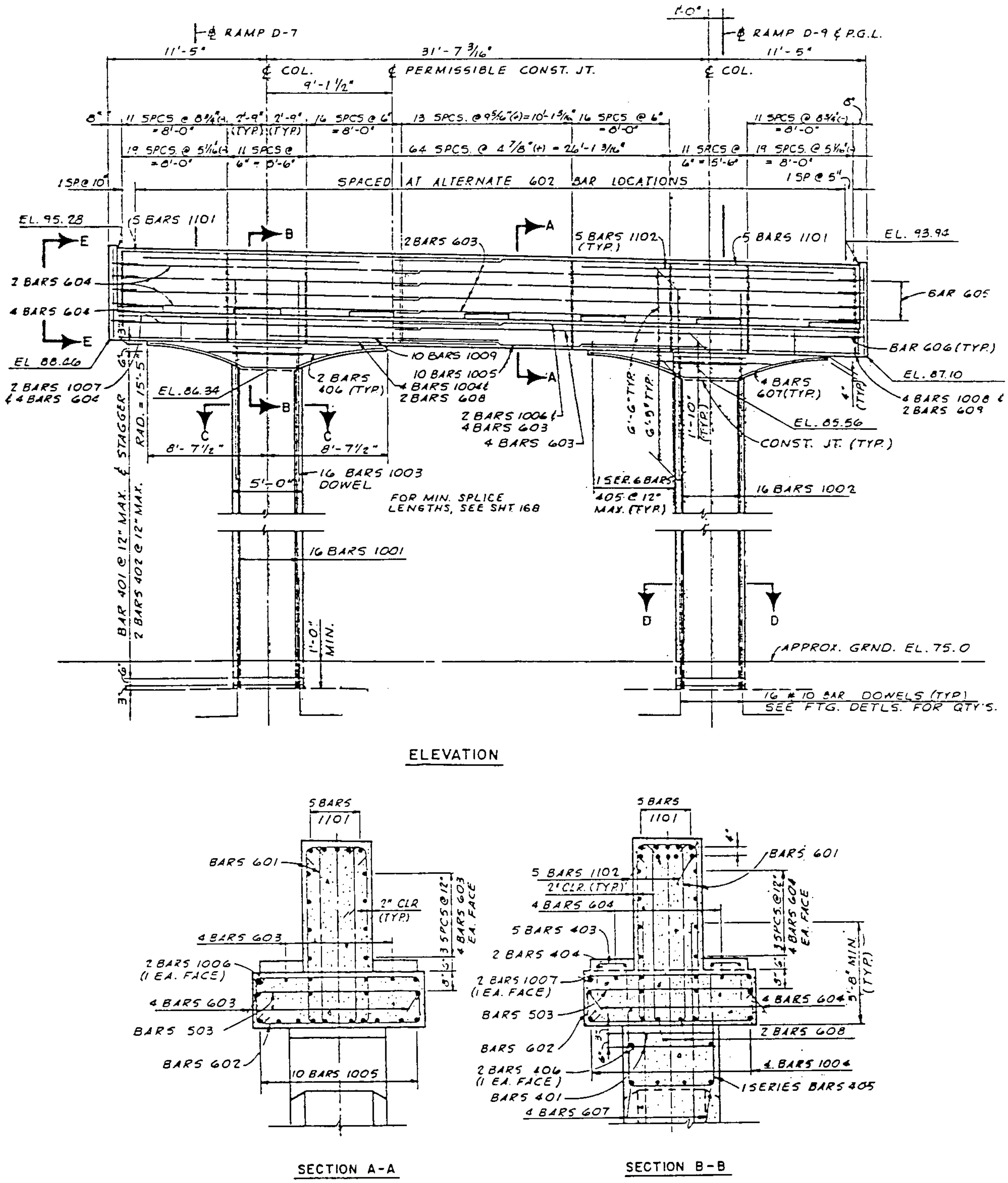


FIGURE 6-6 Louisiana Inverted T-Bent Cap Details

SECTION 7
FOOTING DETAILS ISSUES

7.1 Footing Design Details

Description: In Louisiana, the most common footings are pile supported, as shown in figures 7-1 and 7-2. Steel H-piles and timber piles are required to penetrate the bottom of the footing a minimum of 12". The minimum penetration requirement for concrete piles is 6". No positive pile tension anchorage is required. The minimum spacing of piles is 3'-0" for timber piles and 3 times the diameter (center to center) for concrete piles 18" and up. A minimum edge distance from the center of an outside pile to the face of the footing of 1'-6" is required. The exterior footing piles are sometimes battered to provide the necessary lateral support. Straps are sometimes used between isolated footings in a column bent to prevent differential horizontal movement on soft soil or where erosion is possible.

In North Carolina, the footings are either spread footings or pile footings, depending on the recommendations of the Soils Engineer. The minimum length for interior bent spread footings is 0.2 times the overall height from the bottom of footing to the crown of roadway to the next 6 inches. Minimum distance centerline to centerline of exterior piles for pile footings is 0.15 times the overall height from the bottom of footing to the crown of roadway to the next 3 inches. The minimum footing thickness in SPC A is 2 feet without piles, and 2 feet 6 inches with piles. SPC B requires increased footing thickness. The minimum pile penetration requirement into the footing is 9 inches. No positive pile tension anchorage is required. The minimum spacing of piles is 2'-6" for timber or steel piles, and 2'-9" for 12" concrete piles. A minimum of 4 piles per footing is required for pile foundations. If less than six piles are used, all piles are required to be vertical. Batter piles are used in footings with more than six piles (see figure 7-3). Struts between footings are required when foundation piles are used with laterally battered columns.

In Pennsylvania and New York, spread type individual and continuous footings are more common. Continuous footings are used whenever settlement of any magnitude is possible (see figure 7-8a).

7.2 Reinforcing Steel Details

Description: In Louisiana, footings are reinforced with a mat of steel 3" above the pile tops. The bar sizes are determined by design. Sometimes the reinforcing steel is placed between piles. Vertical dowels anchored into the footing extend into the column. The dowels are confined in the footing area by a minimum of 4 ties. The length of the dowels above the footing is determined based on the AASHTO Specifications splice length requirements. No reinforcement is required for the top of the footing and no shear stirrups are required in the footing. In special design cases top footing reinforcement and shear stirrups are provided to increase footing capacity. Shear keys at the construction joints between the column and the footing are no longer required and the shear transfer capacity is determined based on dowel action and friction on a roughened concrete surface as per AASHTO Specifications.

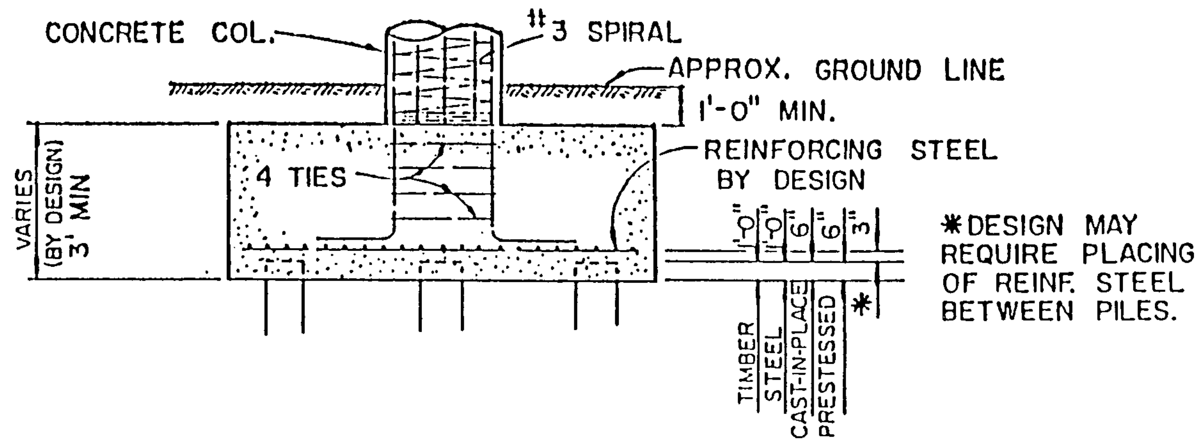
To provide for earthquake forces, North Carolina requires top footing reinforcement consisting of No. 6 bars at 12-inch centers, except in single column bents in which No. 6 bars at 12-inch centers or 50% the area of the bottom reinforcement, whichever is greater, is required. These bars are to be provided in both transverse and longitudinal directions. Confining ties for longitudinal column reinforcement are provided in SPC B footings (see figure 5-8).

A footing reinforcement layout typical to the Pennsylvania and New York seismic provisions for SPC B is shown in figure 7-9. The design and detailing requirements for piles are as per AASHTO Seismic Specification 6.3.1. New York also requires that the minimum top reinforcement for an individual footing be more than 50% of the area of the designed bottom reinforcement, but not less than No. 6 bars at 12-inch centers in the transverse and the longitudinal directions. The minimum top reinforcement for a continuous footing is required to be at least No. 6 bars at 12-inch centers in the transverse and the longitudinal directions. All top and bottom reinforcement and footing dowels are required to have 180 degree or 90 degree hooks. Both Pennsylvania and New York require that vertical stirrups connect the top and bottom mats at a maximum spacing of 48" in both directions. Representative footing reinforcing steel details are shown in figures 7-1 through 7-9.

Advantages: The requirements for minimum footing width and minimum pile spacing provide for minimum bent stability. The requirement for confining ties in the footing improves the dowel pull out capacity. The requirement for top steel reinforcement prevents cracking of the footing, forcing plastic hinging of the column which is a more desirable failure mode. The top steel reinforcement and the confining ties in the footing also improve the column reinforcement pull out capacity.

Disadvantages: When the top of footings has no steel reinforcement, during severe seismic loading cracking in the footing may occur before column plastic hinging, which is an undesirable failure mode. Also, in a lightly reinforced footing the column vertical reinforcement is more likely to pull out of the footing.

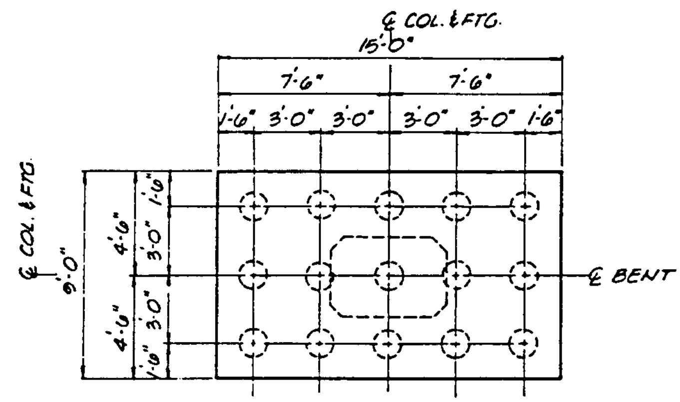
Historical: This requirement for ties confining the footing dowels was introduced in the late 1970's as a result of general changes in bridge detailing in California brought about by the 1971 San Fernando earthquake.



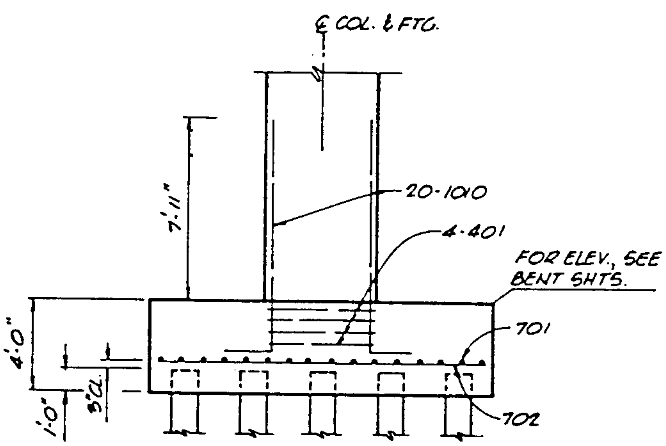
FOOTING	SHEET	CENTROID OF FOOTING		PILE GROUP PROPERTIES			PLAN AREAS	
		$\overline{X}$ (FT.)	$\overline{Y}$ (FT.)	Nº	Ixx (FT.^{2})	Iyy (FT.^{2})	CONCR (FT.^{2})	EXCAVATION (FT.^{2})
TWO PILE	2 OF 5	3.500	1.500	2	0.00	8.00	21.00	60.00
THREE "	2 OF 5	4.599	2.651	3	8.00	8.00	32.72	74.10
FOUR "	3 OF 5	3.500	3.500	4	16.00	16.00	49.00	100.00
FIVE "	3 OF 5	4.333	4.333	5	32.00	32.00	75.11	136.11
SIX "	4 OF 5	5.500	3.500	6	24.00	64.00	77.00	140.00
SEVEN "	4 OF 5	5.734	4.964	7	48.00	48.00	85.37	144.78
EIGHT "	5 OF 5	7.167	4.333	8	48.00	144.50	124.22	202.22
NINE "	5 OF 5	5.500	5.500	9	96.00	96.00	121.00	196.00

DIMENSIONS SHOWN ARE MINIMUM

FIGURE 7-1 Louisiana Footing Detail Requirements

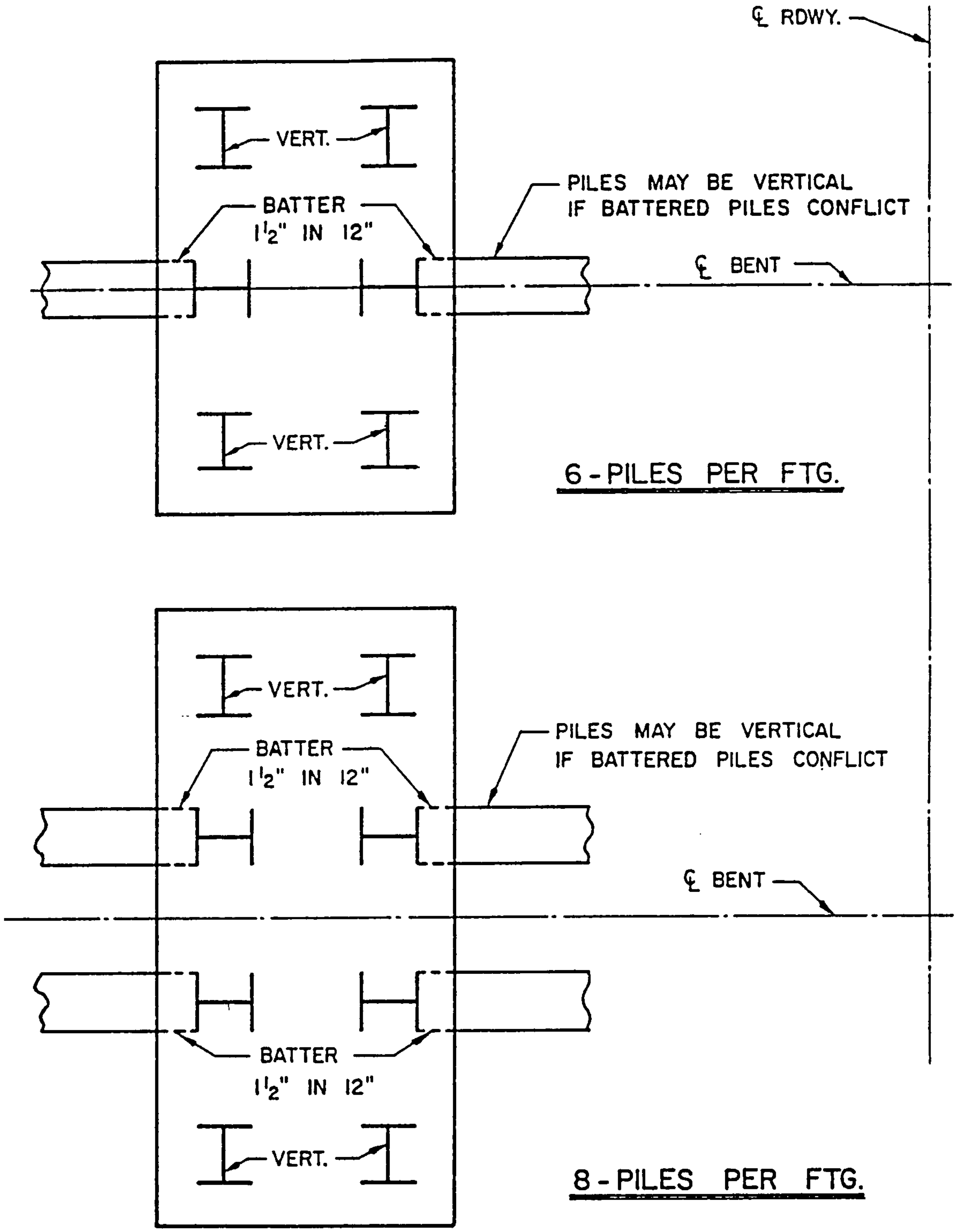


PLAN



ELEVATION

FIGURE 7-2 Louisiana Pile Supported Footing Details



DETAILS SHOW STEEL PILES. DETAILS FOR TIMBER AND CONCRETE PILES SIMILAR.

FIGURE 7-3 North Carolina Footing Detail Requirements

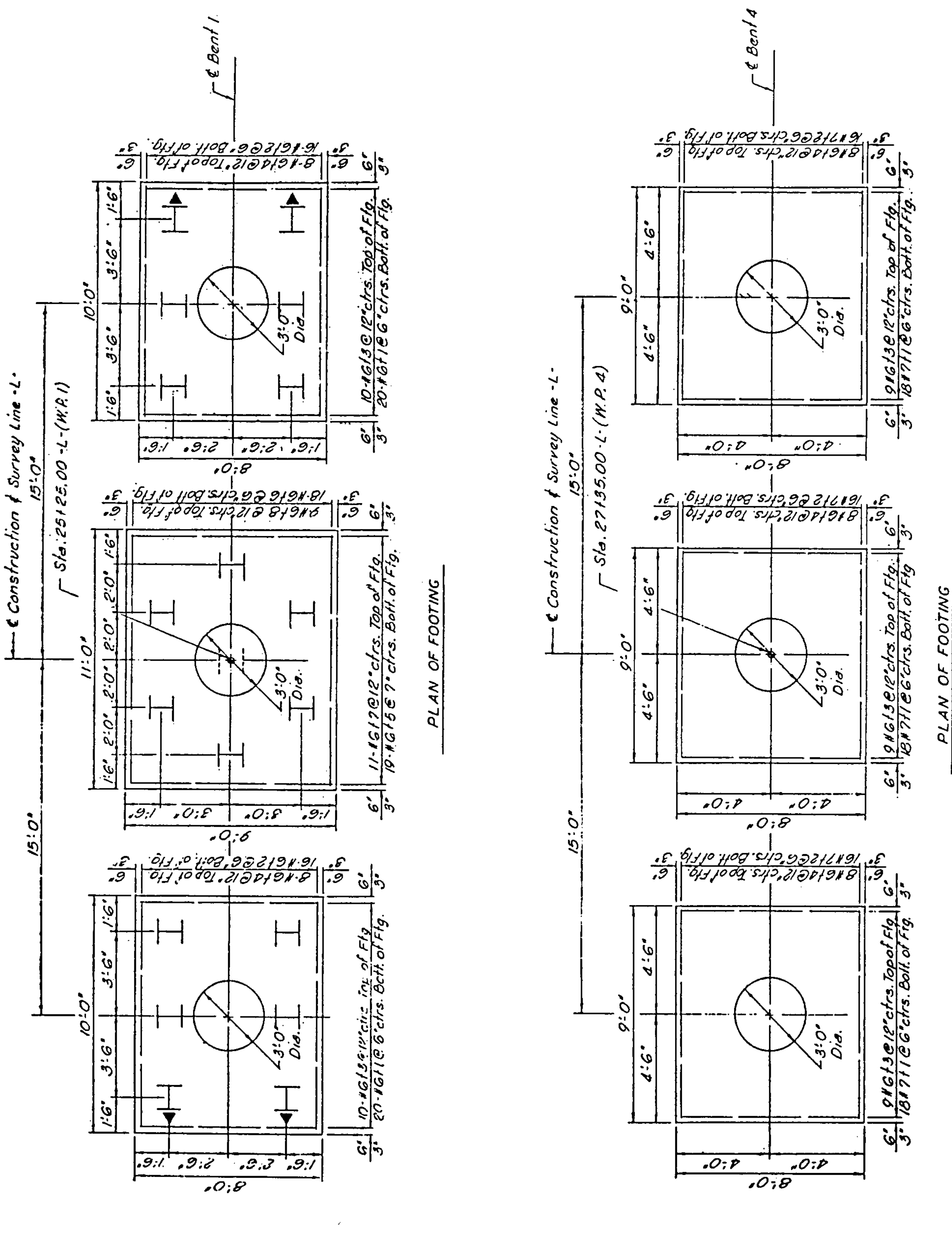
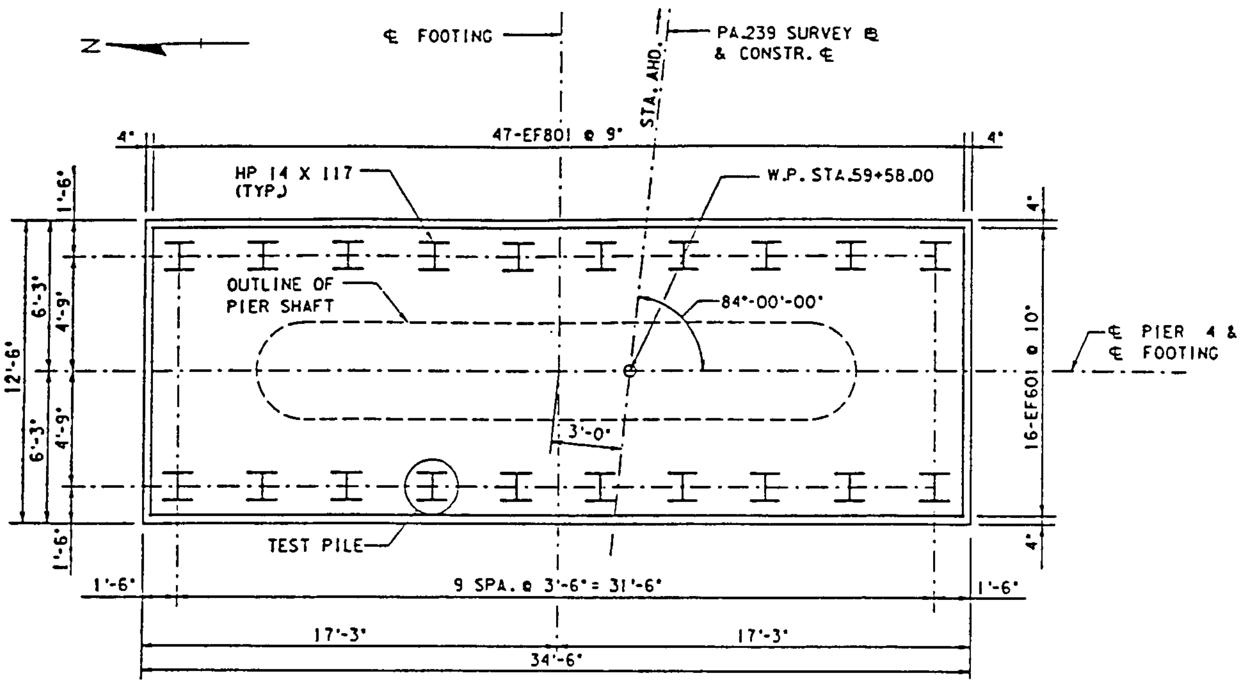
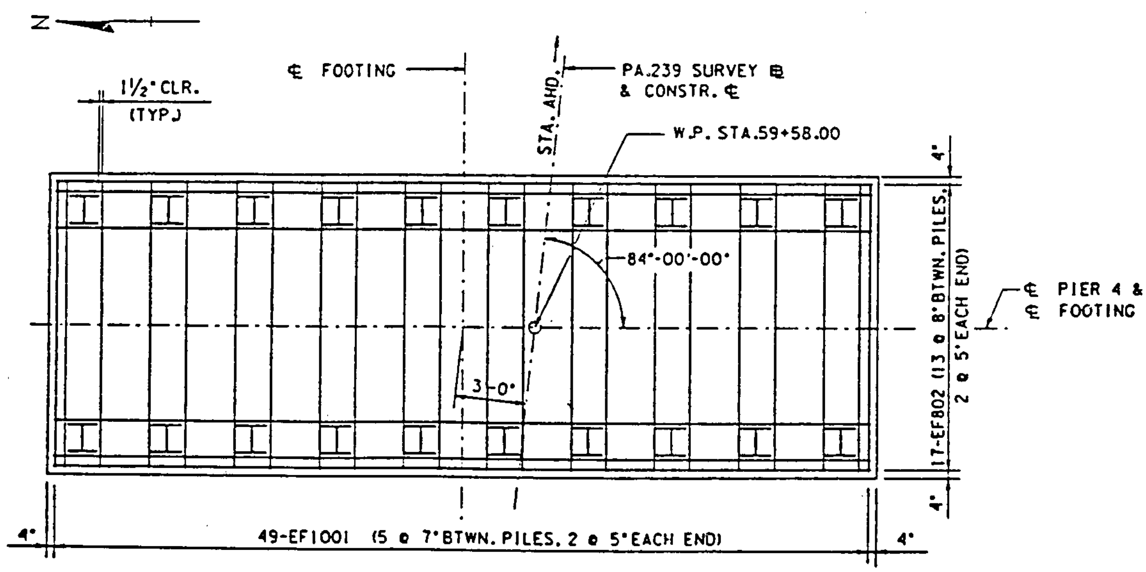


FIGURE 7-4 North Carolina Footing Details



FOOTING PLAN - PIER 4 - TOP REINFORCEMENT



FOOTING PLAN - PIER 4 - BOTTOM REINFORCEMENT

FIGURE 7-5 Pennsylvania Footing Details A

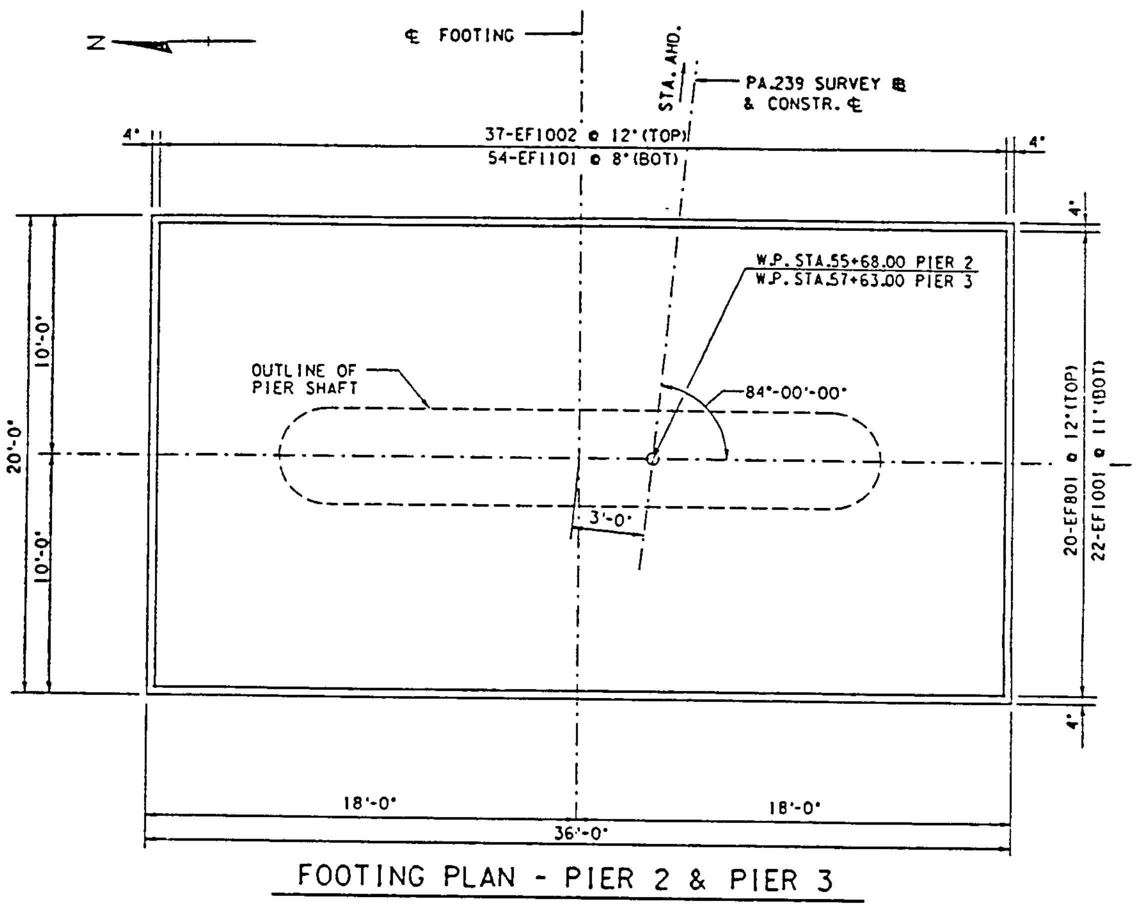


FIGURE 7-6 Pennsylvania Footing Details B

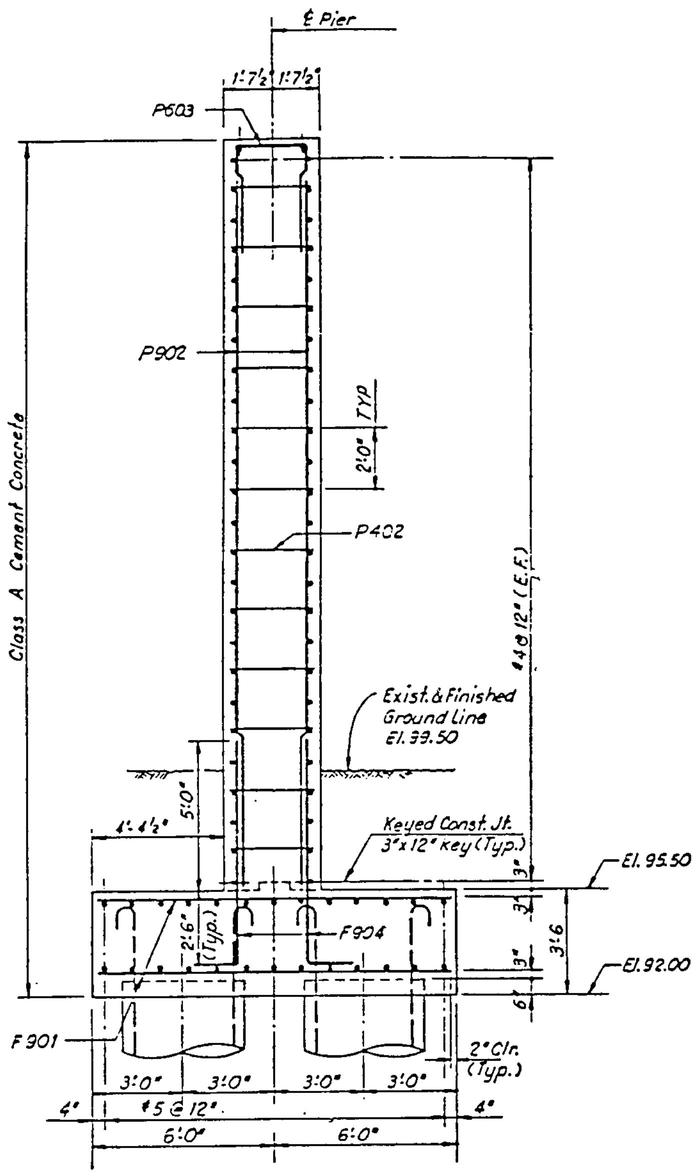


FIGURE 7-7 Pennsylvania Pier Wall Footing Details

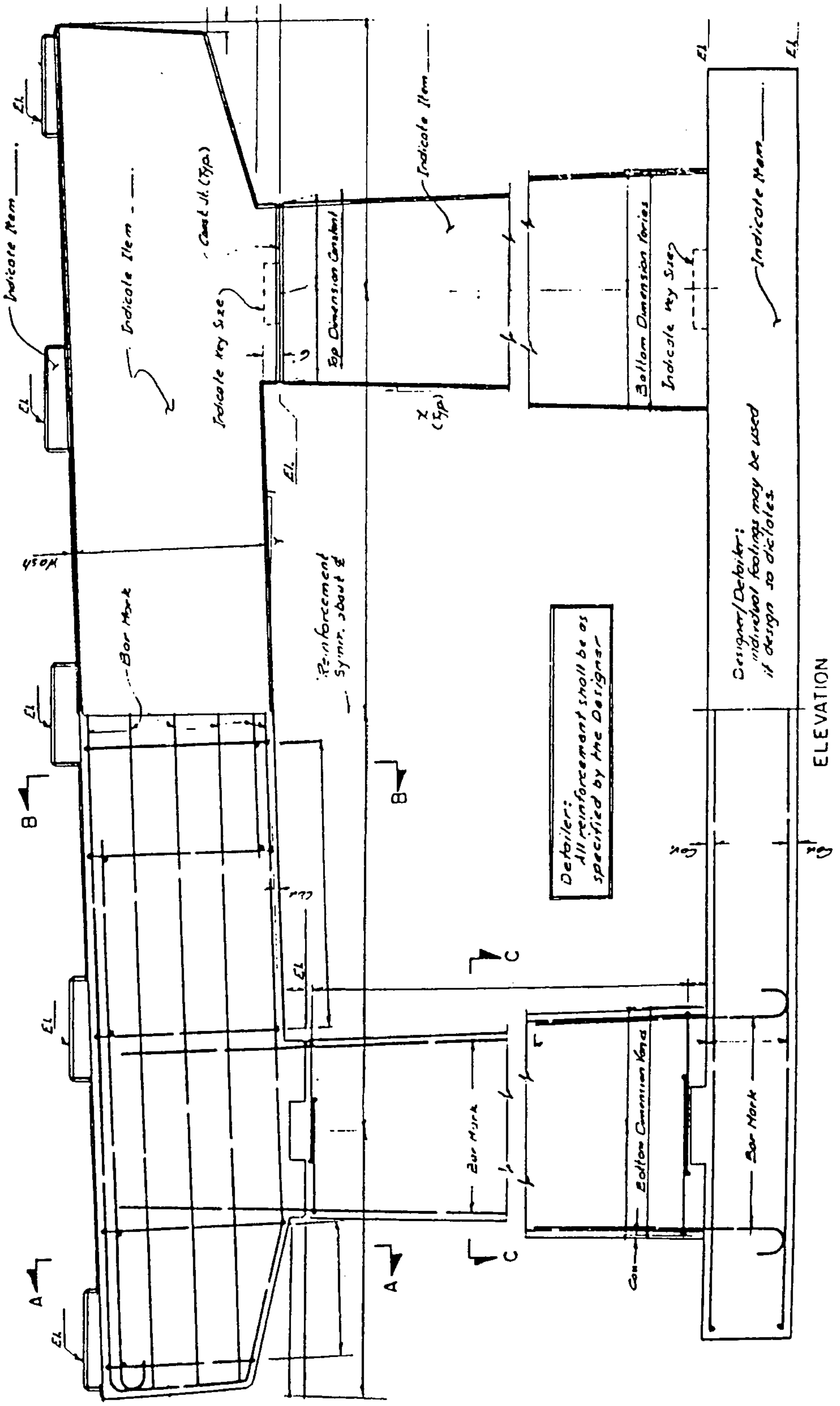


FIGURE 7-8a New York Column Bent with Continuous Footing - Elevation

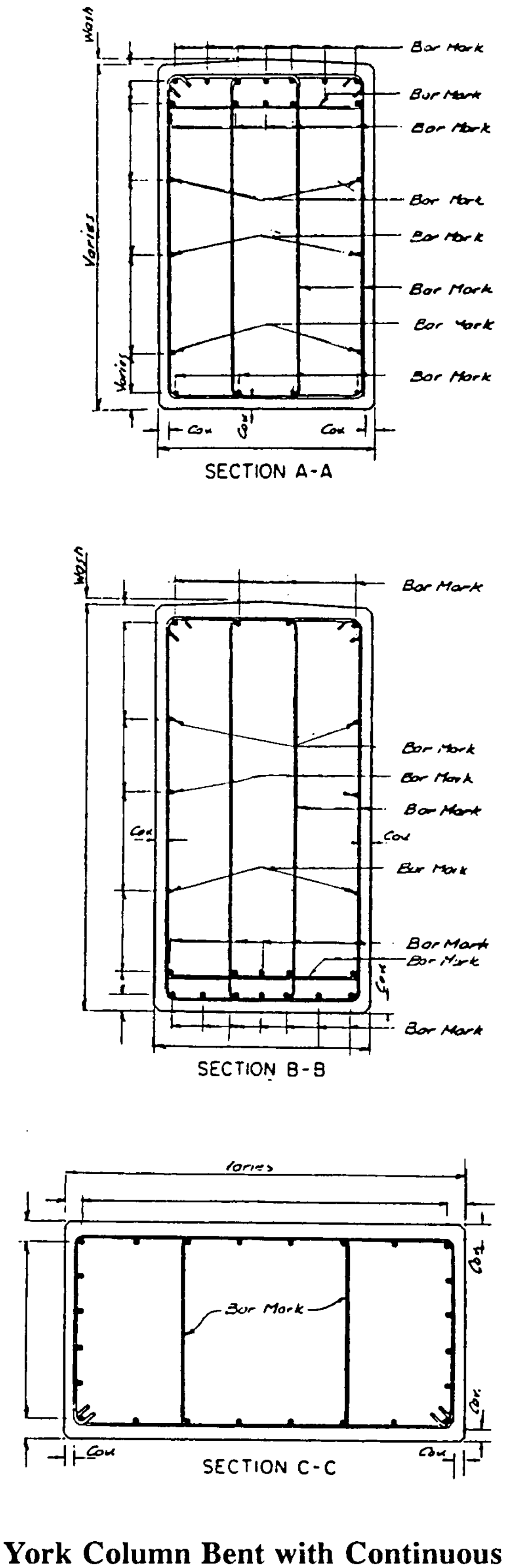


FIGURE 7-8b New York Column Bent with Continuous Footing - Cross Section

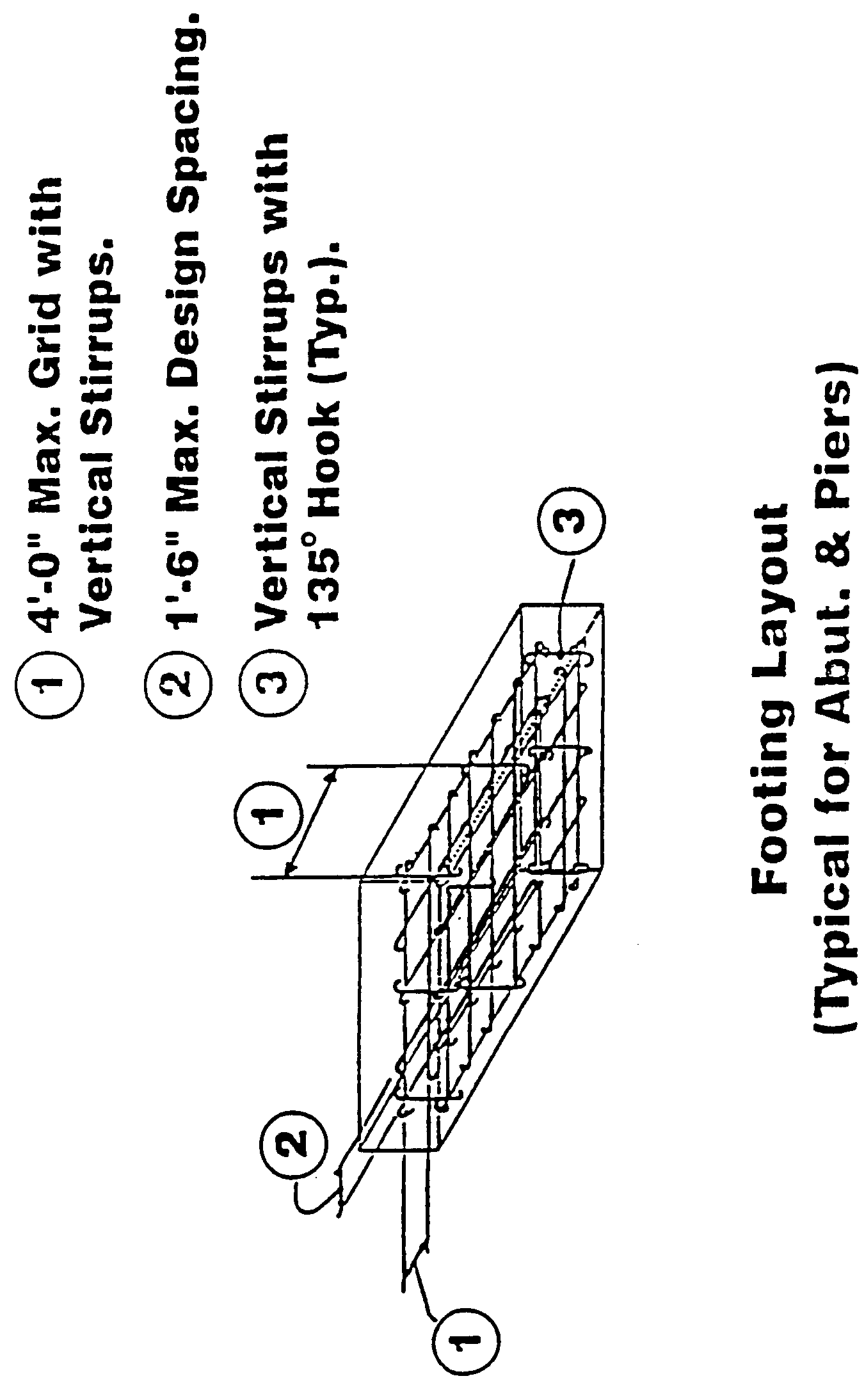


FIGURE 7-9 New York Footing Layout Reinforcement Requirements

SECTION 8
CONCLUSIONS

The requirements for seismic design of bridges in the eastern states, which are in low to moderate seismic zones, are not as rigorous as the those used for the design of western bridges. Most bridges in the east fall in Seismic Performance Categories A and B which have fewer, and less rigorous, seismic design and detailing requirements. Most bridges in the west fall in Seismic Performance Categories C and D, where more stringent design and detailing provisions apply.

Seismic design of bridges in the eastern United States currently follows the AASHTO Specifications for Seismic Design of Highway Bridges in conjunction with state specific policies designed to address the unique conditions of each state. Both the AASHTO seismic specifications and the state policies are constantly refined and updated as technical knowledge becomes available. The bridge engineering community in the east has become aware of the nature of the seismic hazard in the central and eastern United States where a very large magnitude earthquake, larger than the design earthquake, could occur. Engineers in the east have also become aware of the need to provide reinforcement details that would enhance the ductility capacity of bridges, even in regions of low seismicity where seismic loads may not govern lateral load performance and the AASHTO requirements for seismic design are not very stringent. Therefore, future bridge design and detailing practices in the east and in the west will have more and more elements in common. At present, similar detailing practices may be found in superstructures of the same construction type (e.g., concrete box girders), in bent caps, and to some extent in footings and pier walls. Changes made by several eastern states in detailing requirements for SPC B, which pertain to the confinement of longitudinal column reinforcement in plastic hinge regions, the extension of column reinforcement into bent caps and footings and the provision of top reinforcement in footings will increase the similarities between east and in the west construction details.

Historically, the development of criteria for seismic design in the east has followed the developments in the west which has already experienced several damaging earthquakes. It was only in 1983 that comprehensive seismic criteria have been included as an Alternate Guide Specification in the AASHTO Specifications, which were adopted by AASHTO in 1991. After the 1989 Loma Prieta earthquake, states like Pennsylvania and New York have made conservative adjustments to the AASHTO requirements for SPC B to improve resistance to seismic loading and reduce the potential of catastrophic bridge failure. These adjustments have resulted in a minimal increase in construction costs.

In the past, most of the bridge designs in the east were made in accordance with previous standards under which horizontal design loadings were mostly governed by wind, centrifugal or braking forces.

The bridge pier types, details and issues outlined in this report are representative of the practices and seismic policies in the Eastern U.S. In some cases, past practice has also been discussed. Typical highway bridge designs in the east are different from those in the west. In the east, most bridge steel or concrete superstructures are supported on piers and single or multi column bents through bearings. Many of the western bridges have concrete superstructures that are cast monolithically with single or multi column bents. The bridge pier types included in the report have been divided into pile bents, column bents and solid wall piers. They reflect design and construction practices in Louisiana, North Carolina, Pennsylvania and New York. Many detailing practices in these states are similar, but some pier types and details are more common in some states than others. In Louisiana, for example inverted T-caps are very common in current designs and spiral reinforcement in round columns is a standard detail. Hammerhead caps and continuous footings are more common in Pennsylvania and New York. North Carolina has standard steel pile bent details. Pile bents have been addressed separately since they are more common in the east, and the nature of their response to seismic loads is different. In the past, seismic considerations have not been included in the design of pile bents in the east. The seismic design of column bents, however, has been emphasized in design codes, especially after the 1971 San Fernando earthquake when many column bents were damaged. The seismic design and detailing of pier walls is different and the report emphasizes the unique nature of the seismic response of partial height walls. Design and detailing requirements for pile bents, column bents and pier walls are outlined and example details from actual designs are included. Some of these examples comply with the current seismic provisions, while others represent older designs made before these requirements were introduced.

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