Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-98/1410-2F		

4. Title and Subtitle	5. Report Date
A PRECAST SUBSTRUCTURE DESIGN FOR STANDARD BRIDGE SYSTEMS	September 8, 1998
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Sarah Billington, Robert Barnes and John Breen	Research Report 1410-2F
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650	11. Contract or Grant No. Research Study 0-1410
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Texas Department of Transportation Research and Technology Transfer Section/Construction Division P.O. Box 5080 Austin, TX 78763-5080	Final
	14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the Federal Highway Administration.

Research study title: **"Aesthetics and Efficient New Substructure Designs for Standard Bridge Systems"**

16. Abstract

The proposed substructure system described in this report has been developed to improve the aesthetics and reduce the construction time of the support structures for standard bridges. The form of the proposed substructures is highly attractive, and is a distinct improvement over many traditional short- and medium-span bridge substructures.

The substructure system developed is particularly well-suited for precasting, although the geometric form could be cast-in-situ. Precasting would result in increased use of high performance concrete in the substructures. The use of such concrete will bring improved durability since the high performance concrete is greatly resistant to ingress of moisture and chlorides. In addition, the greater compressive strength of the high performance concretes is utilized for reducing the handling weight and dead load of the substructure units. The bent cap units are more complex than traditional cast-in-place bent caps but appear feasible for plant production or large-scale, cast-on-site projects. The construction method proposed could shorten construction times on-site in certain applications. Shortened construction time, in turn, leads to important safety and economic advantages when traffic disruption or re-routing is necessary.

Cost studies based on input from precasters and contractors indicate that if the proposed system (or one quite similar) is actually standardized and used on several projects, the direct costs will be competitive with costs of current designs for concealed bent cap substructures, while the on-site construction time could be reduced substantially. This reduction can have important economic and safety implications on some projects.

17. Key Words	18. Distribution Statement
Bridge design, aesthetics, bridge substructure, construction time, precast concrete, segmental pier, pretensioned concrete, post-tensioned concrete	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	188	

A PRECAST SUBSTRUCTURE DESIGN FOR STANDARD BRIDGE SYSTEMS

BY

SARAH L. BILLINGTON, ROBERT W. BARNES AND JOHN E. BREEN

Research Report No. 1410-2F

Research Project 0-1410

"Aesthetic and Efficient New Substructure Designs for Standard Bridge Systems"

conducted for the

Texas Department of Transportation

IN COOPERATION WITH THE

U.S. Department of Transportation

Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH

BUREAU OF ENGINEERING RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

September 1998

This report was prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

DISCLAIMERS

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Research Supervisor

IMPLEMENTATION

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ACKNOWLEDGEMENTS

The substructure design phase of this project had many important contributors. Special recognition must be given to Professors Andy Vernooy and Dan Leary of the School of Architecture who contributed greatly to the shaping of the form of the template and encouraged a wide look at the aesthetics of the junction of the bent cap and pier shaft. Carl Holliday added a great deal with sketches and models. More than with most projects, the TxDOT project directors from the Design Division worked very closely and contributed much information, ideas, advice and encouragement. Norman Friedman's (DES) input and enthusiasm for this project contributed greatly. His successor, Dean Van Landuyt (DES), brought extremely helpful outlook and design experience that helped shape the final form.

Representatives from design, precasting, forming, and construction industries provided invaluable practical guidance. In particular, the assistance of Mr. Bill Daley of Hamilton Forms is recognized. Special thanks are given to Mr. Tom D'Arcy as a representative of the Texas Precast Manufacturers Association. His frequent meetings with the project staff and his constructive criticisms were of great assistance.

Most of the report is taken fairly directly from the doctoral dissertation of the senior author of this report. The preliminary design work by Robbie Barnes provided an excellent foundation for Sarah Billington's final designs.

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CHAPTER 1

INTRODUCTION

1.1 Background

Bridges are an essential part of any infrastructure. They span countless obstacles to connect the roads of our highway systems. Bridges can be found in a variety of settings from congested urban areas to underpopulated rural locations to beloved park environments. The vast majority of the world's bridges are of short- and moderate spans. Yet it is not these most common bridges, but rather the monumental long-span bridges that are the most noticeable and striking due to their size and often scenic settings. Many long-span monumental bridges are considered works of structural art. [1] The much more prevalent short- and moderate-span bridges simply remain functional and nondescript (Figure 1.1). Although these more moderate-sized bridges dominate our highway landscape, they typically fail to catch even the imagination of the engineers who design them.

Figure 1.1 A typical standard overpass in Texas

Rapid advances in the state of the art of engineering design, materials and construction provide engineers with many new options for short- and moderate-span bridge design. Yet, designed for economy and function alone, standard highway bridges often detract from, rather than enhance, the environment in which they are built. Such an unimaginative display of structural engineering does little to express the rapid growth and exciting developments in this profession.

High performance materials, advanced methods of fabrication and innovative construction techniques have been combined in new ways providing different forms and original solutions for bridge superstructure design. However, as shown in Figure 1.1, even the very slender pretensioned girders lose their attractiveness when they are capped with heavy parapets and are set on a forest of unattractive columns and bent cap beams. Engineers must now accept the challenge to design bridges that are not only functional and economical, but also attractive additions to their landscape.

Economics currently dictate a few standard bridge types for such a variety of settings. As a result, designers remain “prisoners of the familiar,” designing the same type bridge for sites with a variety of constraints and characters. In particular, substructures are a major visual disturbance with these standard bridges. Additionally, current cast-in-place substructure construction leads to extensive traffic delays and rerouting headaches. Little effort has been made to investigate new substructure shapes, designs and construction methods.

In this overall project (2, 3) it has been shown that many significant improvements can be made to the standard bridges of Texas. An increased awareness by highway planners and bridge designers of the visual effect of their engineering decisions is necessary for the design and construction of more attractive structures. As every element of a bridge will affect its appearance, attractive substructure designs that provide an alternative to the current common practice of cast-in-place circular columns with prismatic bent caps are needed.

Short- and moderate-span bridge design in Texas has been dominated in the past 20 years by precast pretensioned concrete superstructure bridges (Figure 1.2). The development of highly efficient plant production methods for precasting has kept this form of construction economical. State-owned bridges in particular (Figure 1.2b) are predominantly prestressed concrete because they are durable and are economically competitive. In 1996, these bridges typically cost \$310 per square meter (\$29 per square foot). Precast concrete superstructure systems were used for 75 to 80% of new highway construction let in Texas between September 1994 and August 1995. (During this time, all new construction in Texas averaged \$345/m^2 [\$32/ft^2]. In 1994, only five other states had averages below \$430/m^2 [\$40/ft^2], while the national average was \$710/m^2 [\$66/ft^2].)

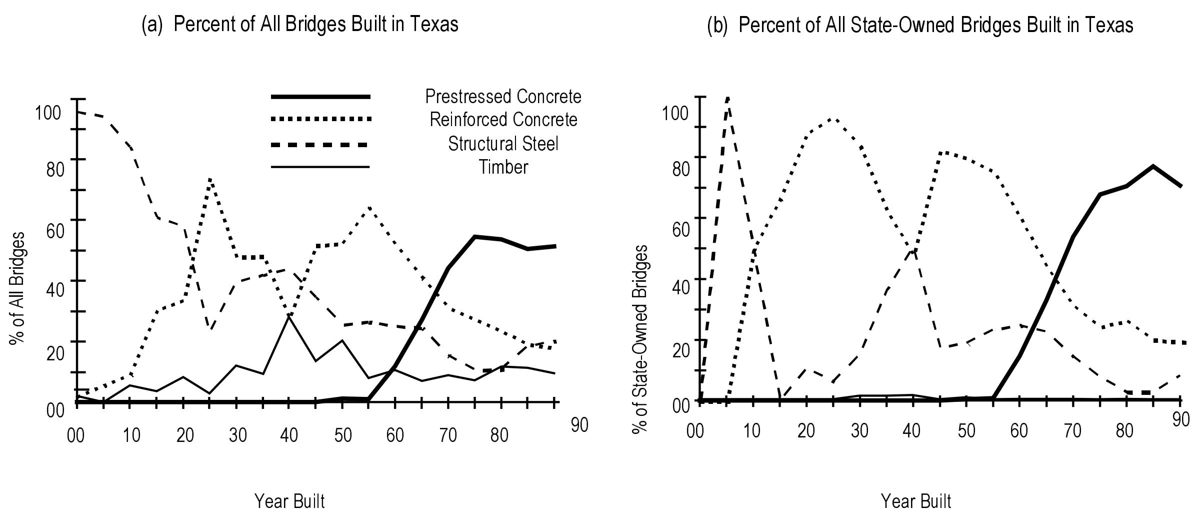


Figure 1.2 Bridge types built in Texas since 1900

The high repetition of precast superstructure elements, low cost of labor and availability of concrete all contribute to this very economical bridge type. The superstructure girders are slim, efficient and often attractive. However, some problems have been identified with the substructures of these bridges. The predominantly cast-in-place substructures are typically the least durable element of these bridges, particularly in aggressive environments. [4] Complete on-site construction of the substructure can lead to excessive and undesirable traffic delays (Figure 1.3). The unattractive forest of columns created by the multicolumn bent substructures is an unfortunate addition to most environments. For these many reasons, alternative substructure designs and construction methods are being investigated.

Figure 1.3 Extensive on-site equipment for cast-in-place multicolumn bents

Precasting offers an alternative for substructure design that can move much of substructure fabrication off-site and into the precasting plant. The efficiency of mass production and the high level of quality control of fabrication in a precasting plant have made precast superstructure elements an extremely economical form of construction. These same techniques may certainly be applied to substructure elements. On-site labor and construction time will be shortened thus reducing traffic delays and rerouting during construction. High performance concrete may be used more consistently with higher quality control in a precast plant. Use of precast high performance concrete should result in more durable substructures with a higher quality and more attractive finish. The higher strength of high performance concrete allows for hollow sections. Hollow sections result in material savings, keep hauling and erection weights low and decrease foundation costs. The use of post-tensioning with precast substructures can further improve durability by eliminating cracking under service loads and providing stiffer vertical elements that minimize lateral deflections.

Widespread introduction of precast substructures on a standardized basis presents an apparent enigma. The authors are calling for more creativity in the bridge project design process and increased designer expression in substructure design. Yet, they are proposing a set of standard designs in order to benefit from economies of scale and the improved materials, shorter construction times and enhanced durability attainable with precasting. Doesn't this standardization stifle creativity for all but those who develop the system? While it is true that pre-engineered and highly standardized systems require great creativity in their formulation and in the engineering of all of the details for prefabrication and erection, that is not the major aspect of creativity in an overall bridge project design process. Introduction of a new standardized system of substructure elements, supplementing the present standard systems, gives the project designer a new range of choices for the individual project. The authors imagine that TxDOT will continue many of their traditional substructure standards for future use. Any new systems such as those proposed herein will greatly broaden the designer's range of choices for application to a particular project. In this way creativity and freedom of expression are enhanced. Unfortunately, the need to maintain economies of scale and reuse of forms will always impose barriers to unfettered creativity.

As precasting will not be advantageous for every bridge site, new cast-in-place shapes should be investigated. These should emphasize use of a higher quality of concrete, and have careful attention paid to their details to improve durability and prevent unwanted staining (Figure 1.4).

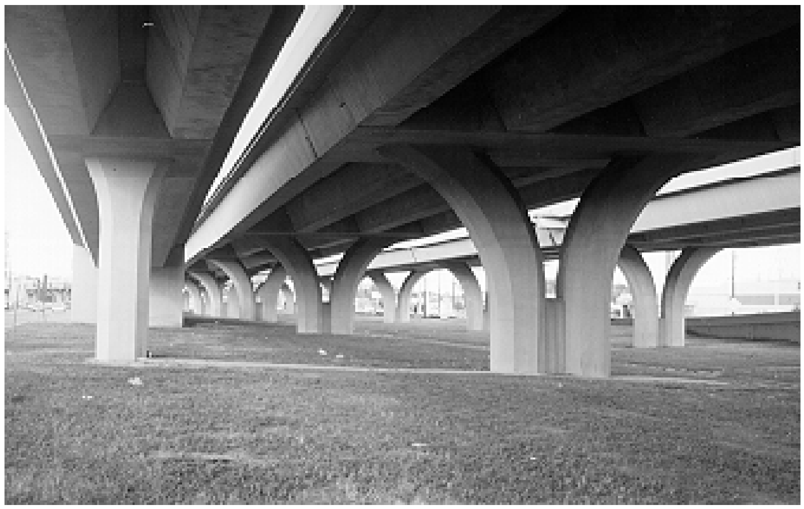


Figure 1.4 An attractive cast-in-place substructure

Recognizing that the imagination of engineers is often stifled, rather than cultivated, in many engineering offices and in typical engineering curricula, a research project was proposed to the Texas Department of Transportation (TxDOT) by the University of Texas at Austin Center for Transportation Research (CTR) to address the problem of the aesthetics and efficiency of Texas' short- and moderate-span bridges and their substructure systems. By addressing efficiency, or the minimization of wasted material, nonproductive labor and construction time in meeting the user's needs within the project constraints (functionality), the structural function and construction of the bridge is tied more closely to the economy of the bridge. The precast girder systems so commonly used throughout Texas have been proven successful through their efficiency, elegance and economy. Through CTR Project 0-1410, attention is now being turned towards improved substructure design, to advance the proud Texas tradition of building functional, economical and attractive bridges for their highway system.

1.2 Objectives

1.2.1 Objectives of the Project

The objectives of CTR Project 0–1410 as proposed to TxDOT are: [5]

1. To develop conceptual plans and visual guidelines for improving the aesthetics and efficiency of widely used moderate-span bridge systems;
2. To introduce more attractive structural forms and textures in substructures though increased use of precasting or, where appropriate, in-situ casting utilizing improved form systems similar to those used in precasting;

3. To reduce construction time, cost of traffic delay and rerouting during construction, and field concreting problems by increased precasting of bridge substructures;
4. To develop conceptual plans for several demonstration projects and to refine those plans based on field experience and observations; and
5. To provide useful design guidelines and examples for improving the aesthetics and efficiency of substructures for Standard Bridge Systems.

The objectives of this project have been carried out in detail by four graduate research assistants associated with this project. Preliminary design guidelines were developed by Listavich. [6] The background was largely completed and an initial precast single-column substructure system developed for Objectives 2 and 3 by Barnes. [7] Work towards Objectives 4 and 5 was carried out by Ratchye. [8]

Further development and restructuring of the guidelines for Objectives 1 and 5 were completed by Billington [2] and reported in Report 1410–1. [3] Development of a precast substructure system for Objectives 2 and 3 begun by Barnes [7] was completed by Billington [2] with extensive contributions from the research team, designers, precasters, form manufacturers and contractors. The original concept has been substantially modified and expanded to cover a wide range of substructure types generally found in Texas.

1.2.2 Objectives of the Report

There are two major objectives of this report. The first objective is to present suggested alternative substructure designs for use with standard superstructure systems common in Texas. With more attention paid to substructure design, the appearance, durability and time required for construction of standard precast bridges can be greatly improved. In particular, the development of a specific precast substructure system to be considered for standardization is presented. After careful investigation of available technology for different substructure fabrication and erection techniques, a family of geometries, materials and techniques judged by the authors as most appropriate for TxDOT are presented. Many design implications for the newer types of substructure systems are discussed as well. The second objective is to document the possible benefits of applying this research to practice. Benefits as well as drawbacks in terms of aesthetic consequences, safety, serviceability and economy considering both initial and life-cycle costs are demonstrated through reference to case studies presented as part of the Aesthetic Guidelines in 1410–1. [3] Further benefits and possible drawbacks of precast substructure systems are addressed through discussions of past and future applications of such systems to short- and moderate-span bridge construction.

1.3 Scope

The scope of this report is to document the completion of the objectives described in Section 1.2.2.

Chapter 2 is a literature review tracing the development and use of precast substructure systems.

Chapter 3 presents a suggested alternative substructure system for the standard short- and moderate-span bridges of Texas. The main focus of this chapter is on the development of a precast substructure system for standardization in Texas. This includes a review of past projects, a brief survey of state-of-the-art technology for precasting substructures, a discussion of concerns expressed by designers and construction industry personnel in Texas and a proposal for a new substructure system including design, fabrication, and erection sequences. Additional discussion includes alternative designs for a geometrically similar cast-in-place substructure system and a system of both precast and cast-in-place elements.

Chapter 4 presents numerous options for cast-in-place substructure design for short- and moderate-span bridges. The goal of this chapter is to show the variety of standard substructure systems available and the ways in which their appearance and efficiency can be enhanced.

Chapter 5 describes areas for further implementation of this research.

Chapter 6 provides a summary and conclusions of this work.

Appendix A is a bibliography on precast substructure design. While omitted from this report, appendices C and D to Reference 2 present detailed design calculations and drawings for a precast hammerhead and frame bent respectively, using the precast substructure system presented in Chapter 3. Copies of these two appendices have been made available to TxDOT Design Division reviewers and are available upon request for the cost of reproduction from The University of Texas at Austin, Center for Transportation Research, Austin, Texas, 78712.

CHAPTER 2

LITERATURE REVIEW—SUBSTRUCTURE DESIGN AND CONSTRUCTION

2.1 Introduction

The current common overall concept in Texas for bridge substructure design and construction is basically the same as it was forty years ago. Cast-in-place circular columns with cast-in-place rectangular bent caps constitute the most widely used system. Occasionally rectangular single or multicolumn bents with rectangular or inverted-T bent caps are used. These are frequently found in urban settings. However, substructure design in general has stagnated. Freedom of designer expression has been stifled by the desire to reuse the same shape for every project for economic savings. Poston et. al. summarized a survey of common pier designs built between 1960 and 1980. [9] The survey results represented information on over 155,000 built piers as reported by 38 organizations including 24 states and the Federal Highway Administration (FHWA). The vast majority of pier types designed by these organizations fall into two categories—category A for single columns (Figure 2.1a) and category A for multicolumn bents (Figure 2.1b). The cross sections of the various columns are predominantly one of three shapes for single-column piers and one of two shapes for multicolumn piers (Figure 2.2). The trend towards monotony in pier design has been nationwide, not just particular to Texas.

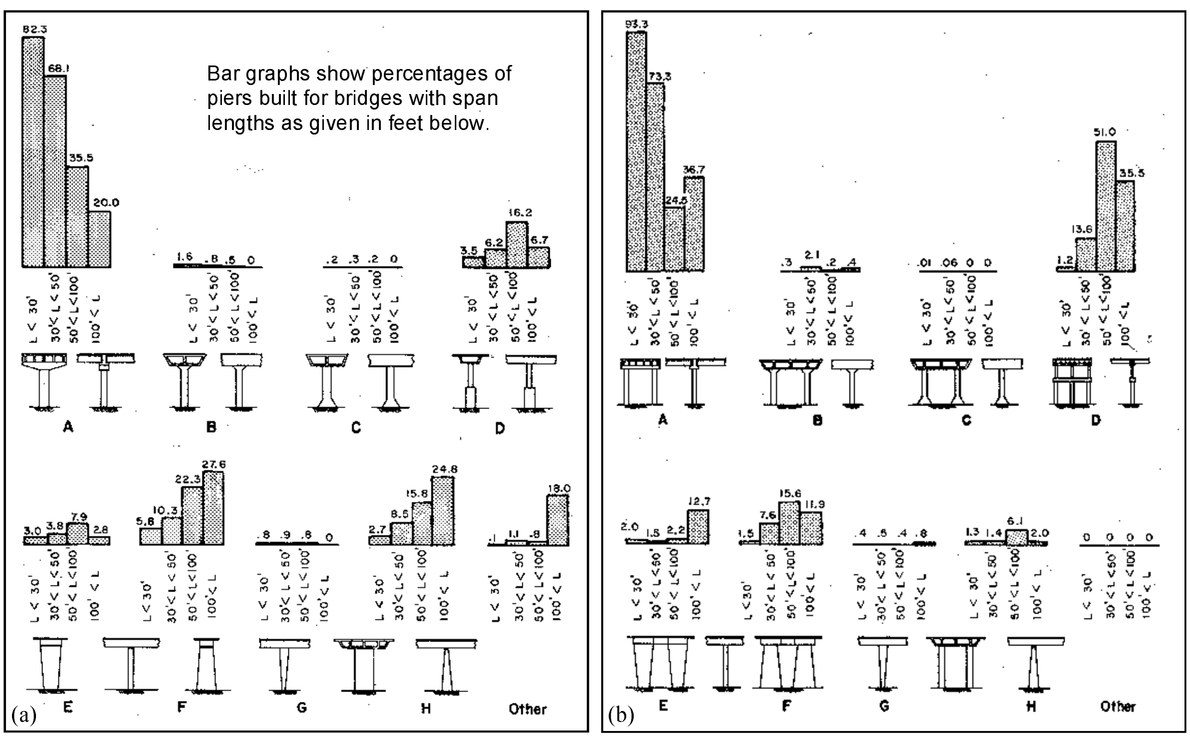


Figure 2.1 Survey results on pier types built from 1960-1980 [9]

The substructure designs most commonly used in Texas are not only fairly ugly, but they have not proven to be particularly durable. Fifty-four-percent of those state-owned bridges (“on-system bridges”) in Texas that have been classified as deficient have substructures in “poor condition” or worse. “Poor condition” is defined as advanced section loss, deterioration, spalling or scour. [10] Aside from the structures deficient because of scour, the vast majority of these deficient substructures are in coastal regions, where saltwater spray and wicking are prevalent, in the northern regions of the state where deicing salts applied to decks are carried through the joints and aggressively attack the caps and run down the columns and in regions with high sulfate soils. Frequently the distress is manifested by progressive cover cracking, spalling and severe corrosion of reinforcement. With increased understanding of durability problems and developments in state-of-the-art technology to avoid such problems, engineers must apply new designs, materials, details and methods of construction to substructure design.

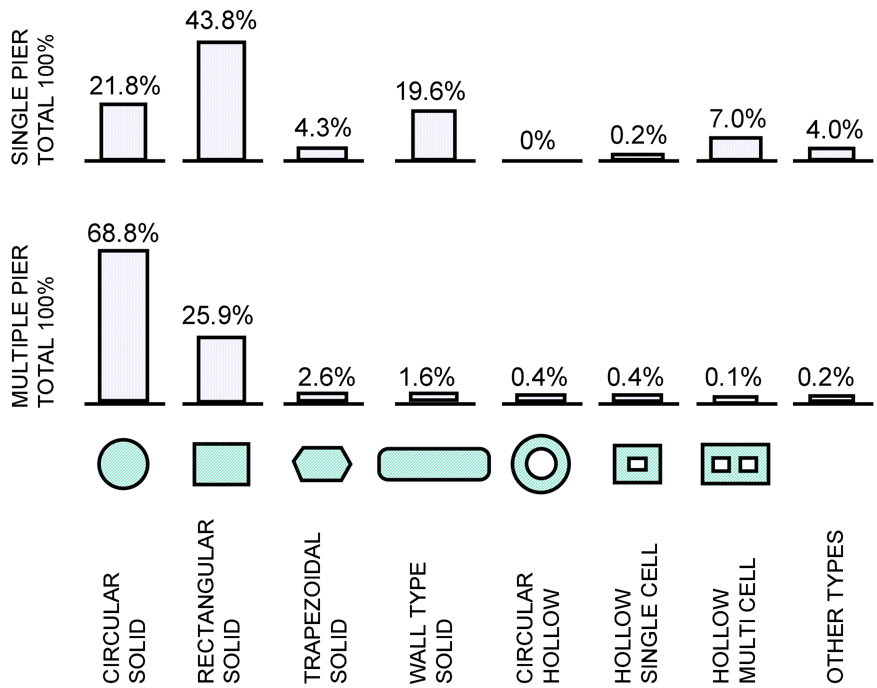


Figure 2.2 Survey results of pier cross sections built from 1960 to 1980 [9]

For the past several decades TxDOT has been addressing the substructure durability problem in both research studies and field implementation. Almost all authorities agree that two of the most effective measures to increase durability are improving concrete quality to make the concrete less permeable and increasing concrete cover to provide a greater barrier. In bridge substructures, the use of the highly permeable Class A concrete was replaced by the less permeable Class C concrete over 30 years ago. Superstructure girder concrete is now largely cast in precast plants. Precast girders have much less permeable concrete and the recently introduced High Performance Concrete (HPC) has been shown to have extremely low permeability and affords increased protection. The most practical way of introducing this HPC into substructures is to precast substructure elements.

Many other steps have been taken to improve durability. The former 38 mm (1½ in) cover was increased routinely to 51 mm (2 in) in the early 1970s. It is now routine to specify 76 mm (3 in) cover in harsh environments. Epoxy coated reinforcement and epoxy painted tops of cap surfaces are now used in some districts. Continued experimentation and innovation are underway to find improved corrosion resistant details.

Numerous improvements can be made in terms of appearance, material and construction efficiency and economics in the standard highway bridge substructures of Texas. New designs may include new shapes for both cast-in-place and precast substructure systems. Materials that may be applied to substructure design include high performance concrete and prestressing steel. Details of structural connections and of nonstructural surface treatments can and should be investigated. New methods of construction, such as precasting segmental piers, can offer economic savings for bridge designs in some locations.

2.2 Precast Substructure Systems

2.2.1 State-of-the-Art Technology

Precasting of bridge elements in the past has been primarily for superstructure elements. Precast girders of I, T, U and box cross-sections make up the majority of the short- and moderate-span highway bridges in Texas. Precast segmental box girder construction was first introduced in the United States in 1971 for the JFK Causeway in Corpus Christi, Texas. Twin segmental box girders were designed and constructed for the main spans of this causeway (Figure 2.3). Since the introduction of precast segmental construction to the United States, a large number of other segmental box girder superstructure bridges have been completed (see References 11 through 22). This method of construction for box girder superstructures has proven to be very economical, particularly for highly repetitive moderate-span as well as long-span projects. The state of the art for segmental construction is evolving and there are numerous different construction methods for segmental superstructure bridges. [11, 23] This bridge type can result in very elegant designs (Figure 2.4) and is a durable system. [24]

Figure 2.3 The JFK Causeway in Corpus Christi, Texas, 1971

Figure 2.4 The San Antonio "Y" project in Texas shows off the elegance possible with segmental concrete bridge design

Application of precast segmental technology to substructure design has been more limited. In particular, segmental construction for shorter span bridge substructures (spans less than 45 m [150 ft]) has been explored very little. Where precast substructures have been used, they have been used for a variety of project types with a variety of different substructure elements being precast. A brief survey of past projects utilizing precast substructure elements, recent trends for segmentally precast short- and moderate-span bridge elements, as well as future applications for precast substructure systems is presented in the following sections.

2.2.2 Past Projects with Precast Segmental Substructures

Precast segmental substructures have distinct advantages for certain design situations. Enhanced aesthetics, speed of construction, construction in difficult to access sites and minimization of construction's environmental impact are key reasons that precast segmental construction has been used in the past. Large projects allow for efficient production of repetitive elements in the controlled environment of a precasting yard or plant. Plant production is particularly advantageous in harsh environments where the construction season is short. Precast elements can be fabricated year round in a precast plant. Precasting pier elements minimizes the amount of on-site construction work thus reducing traffic rerouting and delays as well as the environmental impact of the construction process. References 11 and 12, 14 through 19, 21 and 25 through 28 describe a number of projects where segmental construction of piers has been used advantageously. A brief summary of many of these projects is given in Reference 7.

Different elements within substructure systems have been precast for different projects. At Redfish Bay [29], precast pile caps were placed over precast piles to construct a long low-water crossing (Figure 2.5). Precasting the pile caps saved the contractor six months of construction time by avoiding the need to place fresh concrete over water. Precasting also provided better quality control for concrete placement. At the Linn Cove Viaduct, hollow pier elements were precast and then lowered into place from the newly constructed superstructure to minimize construction impact on the site below (Figure 2.6). [11] Precast piers were used at Vail Pass to minimize site impact (Figure 2.7). [12] Precast caisson elements have been used for a number of water crossings to speed construction (Figure 2.8). [30, 31] The high quality control of concrete fabrication in a precast plant was used advantageously for hollow-column segments that support an ocean pier in South Africa. [32] The interlocking, stacked, hollow-column segments were used as a dense corrosion-resistant form that was filled with tremie concrete. The largest bridge project carried out in the Middle East as of December 1989 was the Bahrain Causeway which was constructed almost entirely out of precast elements. [33] Pile foundations, pile caps and pier shafts for six pile groups on either side of the three main spans of this causeway were all precast. Precast hammerhead caps have also been used in the past to top cast-in-place columns (Figure 2.9).

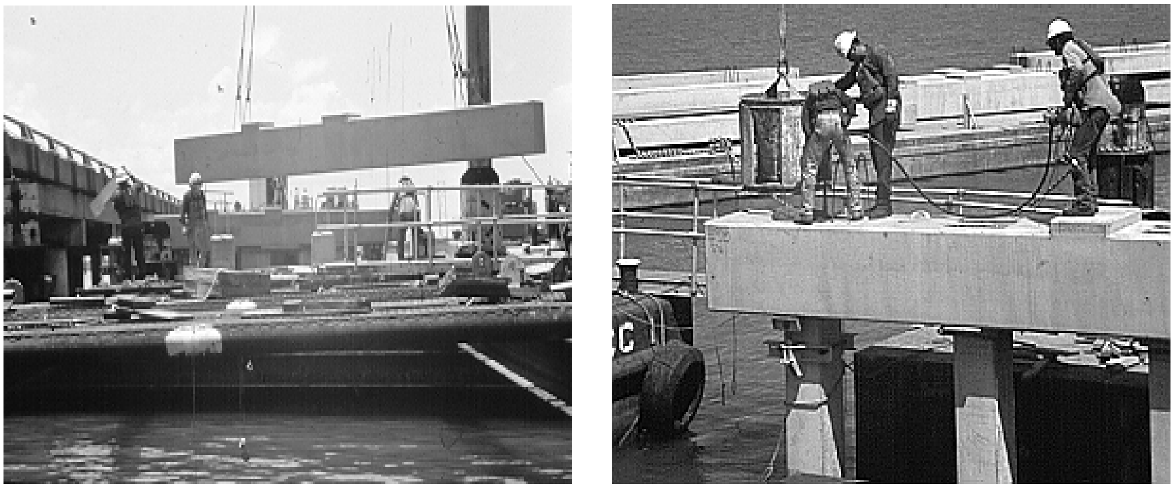


Figure 2.5 Precast pile caps for the Redfish Bay low water crossing. A minimal amount of cast-in-place concrete is used in making cap to pile connections.

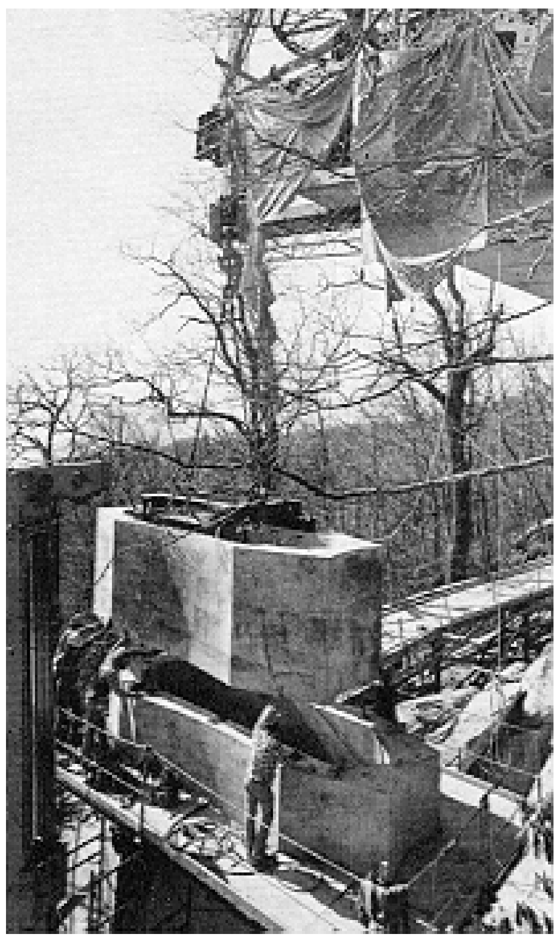
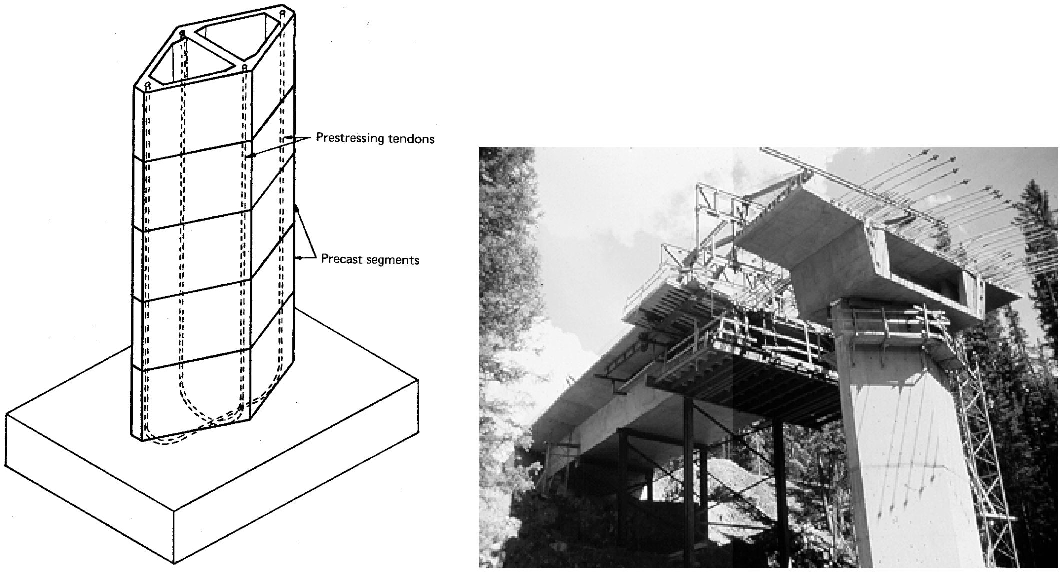


Figure 2.6 Precast hollow concrete pier segments lowered into position from the deck above at the Linn Cove Viaduct in North Carolina [10]



(a) (b)

Figure 2.7 A schematic drawing (a) and the as-built view (b) of the precast concrete piers for Vail Pass in Colorado [12]

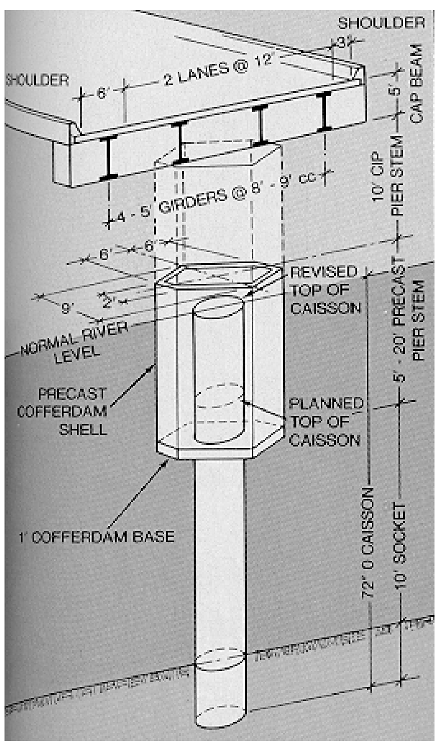


Figure 2.8 Precast cofferdams used speed construction [30]

Figure 2.9 Placement of a precast pier cap on a to cast-in-place column

Past projects, although limited in number, have shown that precasting substructure elements is feasible and advantageous for a wide variety of project types. New applications for precast substructure elements continue to be explored.

2.2.3 Recent Trends for Segmental Construction of Short- and Moderate-Span Bridges

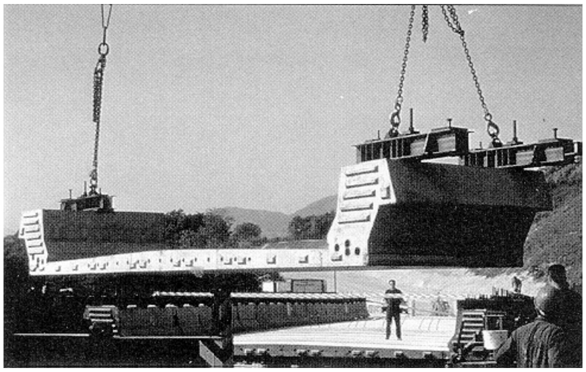
Segmental construction has traditionally been used in moderate- and long-span superstructure design. This method of construction is now being further explored for applications with short- to moderate-span superstructure and substructure design.

A joint committee of the Precast Concrete Institute (PCI) and the American Segmental Bridge Institute (ASBI) has developed standards for precast segmental box girder sections for use with spans of 30–45 m (100–150 ft) for span-by-span erection and 30–60 m (100–200 ft) for balanced cantilever construction. These standards accommodate deck widths of 8.4–13.5 m (27–44 ft). Standard design sheets with design examples have been assembled by committee members from different bridge design offices in the United States. [34] Designs similar to the standards being developed have been used successfully for moderate-span bridges constructed by the span-by-span method such as at the Florida Keys [15, 16], in San Antonio, Texas [20] and in Austin, Texas. [21]

The bridge design firm J. Muller International has developed a segmental bridge system for spans of 15–35 m (50–115 ft) named the Segmental Concrete Channel Bridge System. [35] This system of channel segments is longitudinally post-tensioned together and may be transversely pre- or post-tensioned (Figure 2.10). Key features of this system include increased clearance for standard highway overpasses, shorter construction time and lower life-cycle costs. Two such bridges have been designed for use in New York state.

The recent trend of applying segmental construction technologies to short- and moderate-span bridges has also been explored for substructure design. TxDOT has designed precast segmental piers for three moderate-span bridge projects in Texas: US Highway 183 in Austin (Figure 2.11), State Highway 249 over Louetta Road in Houston (Figure 2.12), and most recently, an overpass over Interstate Highway 10 near El Paso, TX. The designs for the latter two projects were essentially the same.

Technical sessions at industry conferences often include presentations on the use of precast substructure elements for both new design and rehabilitation projects. [36, 37] A number of practitioners attending presentations by the senior author on the substructure system presented in Chapter 3 have expressed interest in learning more about this system for possible use in their design firms. Some engineers attending commented that they frequently use precast substructures for their bridge designs for successful, rapid construction.



(a)Precast channel segments make up the super- structure. The primary load-carrying system is the parapet walls

(b) A completed channel bridge

Figure 2.10 The Segmental Concrete Channel Bridge System developed by J. Muller International for spans of 15-35m (50-115 ft.)

Figure 2.11 Precast segmental piers at US 183 in Austin, Texas

Figure 2.12 Precast piers for a grade separation in Houston, Texas

A recent project using precast substructure elements for a moderate span precast girder bridge is the Edison Bridge over the Caloosahatchee River in Fort Myers, Florida designed by HDR Engineering, Inc. [37] The precast substructure units used nonprestressed reinforcement connected with grouted sleeve couplers. Separate northbound and southbound structures were designed with 43 m (142 ft) spans made up of 1830 mm (72 in) deep Florida bulb-Ts supported by precast frame bents. The bents were made up of I-shaped columns that were cast in single pieces up to 12.5 m (41 ft) tall. The largest column segment weighed 395 kN (89 kips). The caps were up to 18.5 m (61 ft) long. In order to keep the weight low, they were inverted U-sections (Figure 2.13). The largest cap segment weighed 690 kN (155 kips). The caps had solid sections over the columns to provide horizontal shear transfer from the cap to the columns in accordance with the seismic criteria for the area. The column segments were connected to the cast-in-place footings and to the precast pier caps with mechanical couplers (grouted sleeve couplers). To ensure a "perfect" fit, similar patterns (basically metal sheets with measured openings) were used to align the dowels from the footing and to align the placement of the sleeves in the pier segment. The pattern in the field had sleeves to keep the dowels vertical during construction. Similar patterns were used for the dowels protruding from the top of the column segments and for the sleeve locations in the cap. The top of the caps were horizontal to facilitate placement over the dowels in the column segments. The 2% cross slope necessary for the deck was then achieved through variable height bearing seats cast on top of the cap at the bridge site. In 1992 dollars, the precast column bid prices were $580 per cubic meter ($445 per cubic yard) and the precast caps averaged $730 per cubic meter ($560 per cubic yard). The substructure bid prices made up roughly 7.5% of the total bridge bid price.

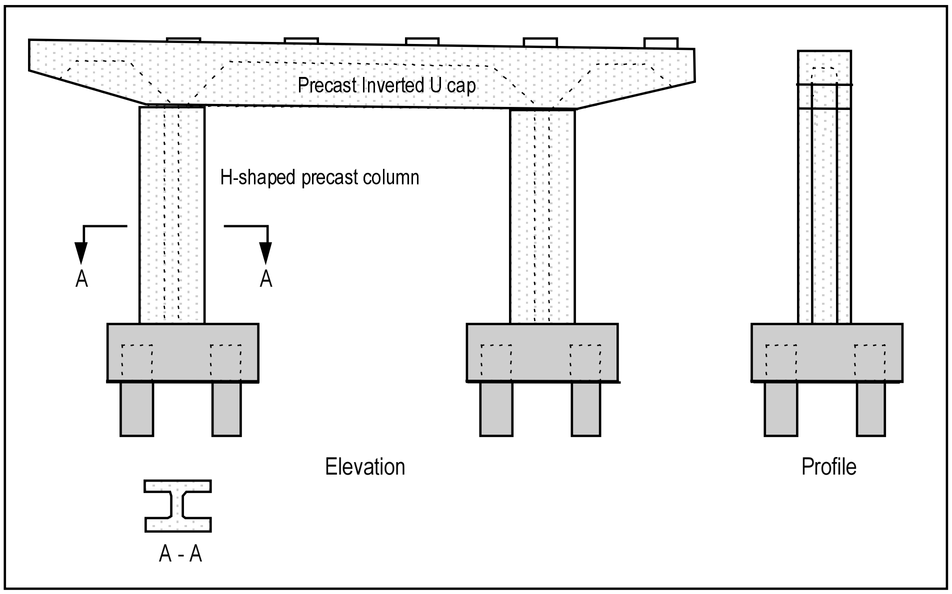
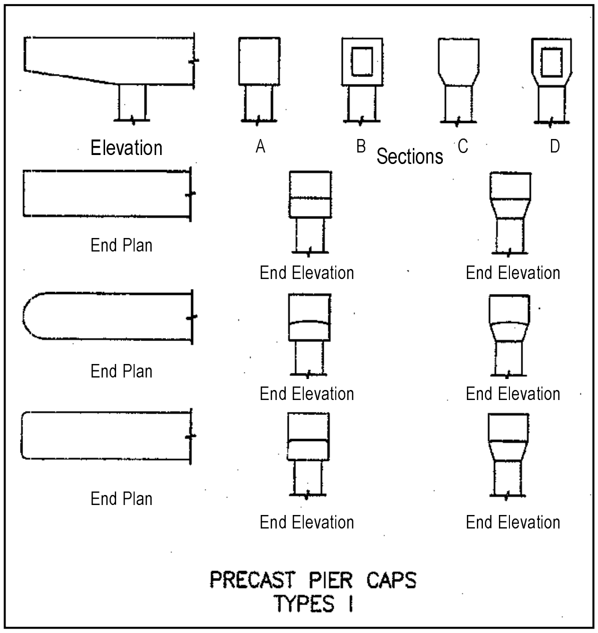
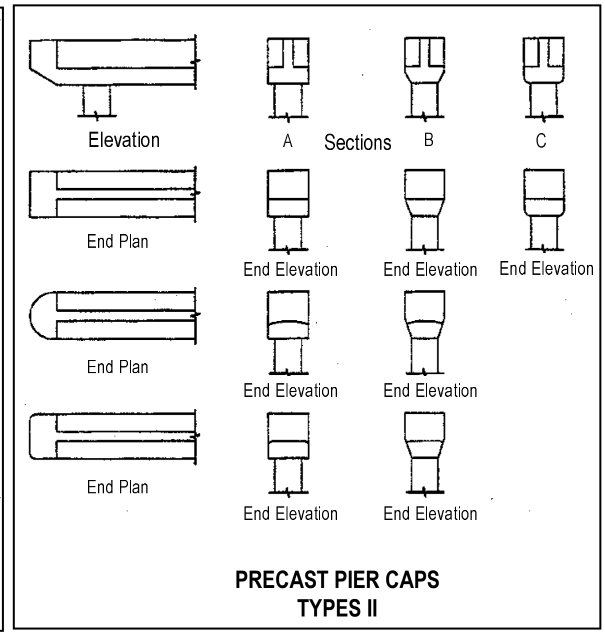


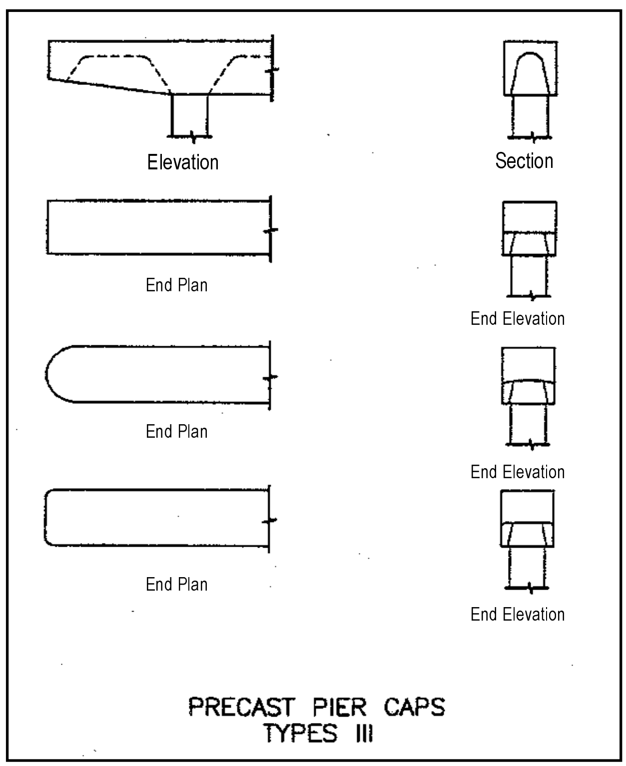
Figure 2.13 Schematic of the Edison bridge precast bents

A preliminary study for developing precast bridge substructures that could be standardized for moderate-span bridges, in particular moderate-span water crossings, was completed for the Florida DOT in May, 1996 by LoBuono, Armstrong & Associates, HDR Engineering, Inc. and Morales and Shumer Engineers, Inc. [38] The initial phase of the study involved a survey of the use of precast substructures in the United States. It was found that most state DOT's, Florida contractors and major precast concrete industries were primarily concerned with connection details for precast substructures. The second phase of the study involved identifying a number of precast substructure options for pile bent caps, and columns and caps for multicolumn and hammerhead bents. A number of shapes and fabrication options were then rated by consultants and representatives from both contracting and precasting industries. A recommendation was made to limit precast element weights to 530 kN (120 kips) and to limit the number of necessary connections. Connections discussed included mechanical couplers such as grout sleeve couplers, post-tensioned connections, welded connections and reinforcing bar lap splices. Table 2.1 and Figure 2.14 shows the section types recommended for further investigation as an outcome of the study.



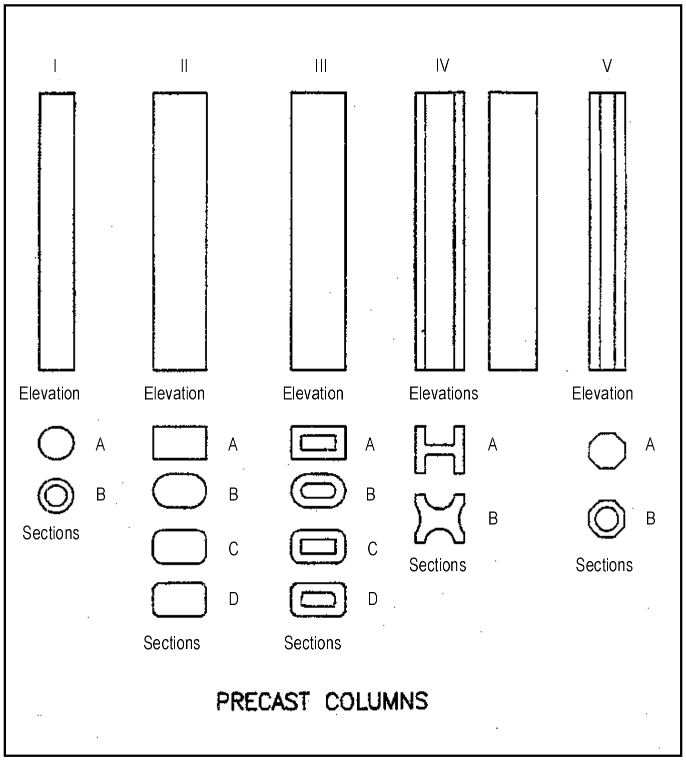


(a) **(b)**

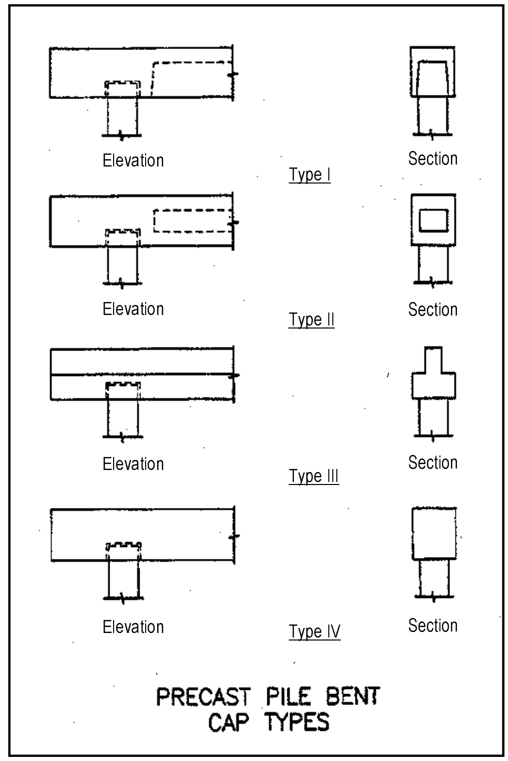


(c)

Figure 2.14 Precast bridge substructure study for the Florida DOT



(d)



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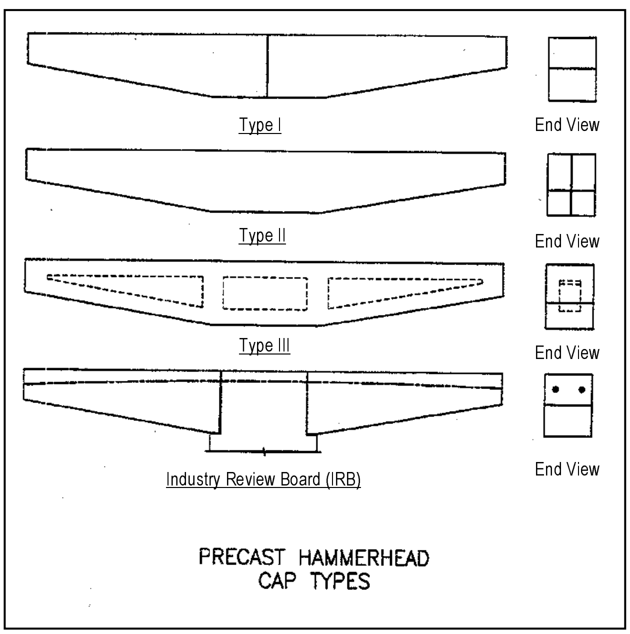
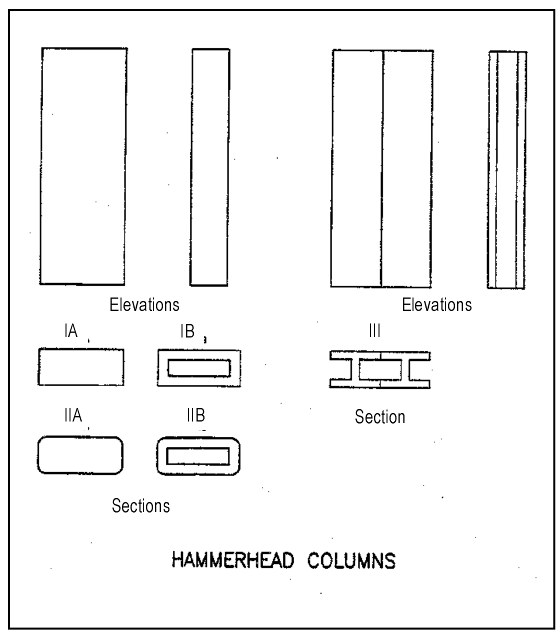


(f)



(g)

Figure 2.14 (continued)

Table 2.1 Precast substructure components chosen for further study for the Florida DOT

Component	Configuration	Configuration
Multicolumn Pier Cap *(Figure 2.14a-c)*	Solid Rectangle (IA)	Inverted U (III)
Multicolumn Pier Column *(Figure 2.14d)*	Hollow Rectangle–Rounded Corners (IIIB)	I-Shaped (IVA)
Pile Bent Caps *(Figure 2.14e)*	Solid Rectangle (IV)	Inverted U (I)
Hammerhead Pier Cap *(Figure 2.14f)*	Solid Rectangle (IRB)	
Hammerhead Pier Column *(Figure 2.14g)*	Hollow Rectangle–Rounded Corners (IIB)	Double I-Shaped (III)

CHAPTER 3

ALTERNATE SUBSTRUCTURE DESIGN—PROPOSED PRECAST CONCRETE SYSTEM

3.1 Introduction

This chapter summarizes the development of a particular precast concrete substructure system proposed by the Project 0–1410 investigators as a possible candidate for refinement, trial use and future standardization by TxDOT.

3.2 Future Applications

The benefits of precasting substructure systems for short- and moderate-span bridges are promising. Recently designed precast substructure elements have been successful in decreasing on-site construction time. As a result, traffic delays and re-routing were minimized, thus reducing an indirect cost to the public. Reducing construction time has advantages in many parts of Texas (Figure 3.1). Particularly in congested urban areas, new highway or light rail construction may cause significant traffic problems. The faster that necessary bridge structures can be built with minimum traffic disruption, the less difficulties there will be for the public, as well as the engineers and city officials.

Figure 3.1 Traffic congestion could be relieved sooner with faster highway bridge construction

The current trend of minimizing the construction industry's impact at the site of new construction is important to maintain good public relations and to protect the environment. Precasting substructures moves that portion of construction to a precasting plant and avoids the need for element fabrication, formwork assembly and concrete placement on site. The result is less equipment on-site and therefore less interference at the site during construction.

Precasting allows for higher quality control of the concrete. High performance concrete (HPC), a more durable material than normal strength concrete, requires higher quality control for proper fabrication. HPC can therefore be more efficiently and economically mixed and placed in a precasting plant than on site. Precasting also allows for more attractive finishes than cast-in-place concrete.

The positive feedback and interest from practitioners is a sign that precasting is a new option for substructure design. Recent success with precast substructures and the proven efficiency, economy and durability of precast segmental construction for superstructure design in the United States [24] point to a positive future for precast segmental substructure design.

For superstructure design, the trade-off between the construction simplicity of constant depth sections and the material efficiency of variable depth sections has shown the former to be more economical with shorter span structures. A similar trend can be seen with substructure designs. Highway structures of short- and moderate-spans with heights under 15 m (16.5 ft) are typically constructed with piers of constant cross-section. [9] The savings due to the simplicity of constructing constant column sections typically outweigh any material savings achieved by tapering the columns. As a result, standardization is particularly attractive for short- and moderate-span bridges of moderate height. Constant cross-sections lend themselves more easily to economical standardization.

As precasting substructures is a fairly new area of design, there is limited field information on their behavior and performance. More importantly, there is limited contractor experience with this form of construction. The biggest hurdles to overcome with developing new precast substructure systems are the many unfounded negative beliefs about such systems held by "prisoners of the familiar." Through attention to industry concerns and knowledge of past successes and failures, functional, economical and attractive new substructures can be designed and built. With careful implementation of this recent trend in substructure design, the advantages of precast segmental substructure systems will shine forth.

3.3 Criteria for a Texas Precast Substructure System

Looking specifically towards developing a precast substructure system for TxDOT, a number of needs and constraints must be met. The criteria for the precast substructure system are that the system must:

- be compatible with precast beam superstructures,
- be economically competitive with current practice,
- be sized for fabrication and erection with existing plants and construction equipment,
- make use of precaster and contractor experience,
- improve durability,
- be designed in accordance with *AASHTO* Bridge Design Specifications [39–41] and the TxDOT Bridge Design Manual. [42]

A precast substructure system for Texas must be compatible with precast concrete I-, T-, U- and box-beam superstructure systems as these are the predominant superstructure systems used in Texas. The substructure system must be economically competitive with current substructure systems. Economics of the design should consider not only initial dollar costs but also indirect costs and benefits to the public due to construction time and impact. While it is recognized that the initial cost of a precast system may be higher, the system must be developed with forms and details that can easily be standardized. With wide reuse over time, standardization will bring costs down and make precast systems economically competitive with cast-in-place systems for many bridge projects, particularly where rapid construction time is valued.

An additional approach to keeping the initially higher costs of a precast system down, is to develop a system that makes use of existing precast plant facilities and equipment in Texas. The construction equipment (predominantly the cranes) required for substructure erection should be compatible with the equipment requirements for superstructure erection. Therefore element weights should be kept below 700–750 kN (160–170 kips). (This is roughly the weight of the largest prefabricated beams used in Texas). Limiting element weights for erection will also keep hauling costs down.

Durability is a major concern that must be addressed by the developed system. Introduction of high performance concrete and the use of prestressing in the substructure will be advantageous. Finally, the system must be designed in accordance with the most current *AASHTO* bridge design specifications and the TxDOT Bridge Design Manual. The *AASHTO Specifications* include the 1996 *AASHTO Standard Specification for Highway Bridges, Sixteenth Edition* [40] (Standard *AASHTO*), 1994 *AASHTO LRFD Bridge Design Specifications* [39] (*LRFD AASHTO*), and 1994 Interim *Guide Specifications for Design and Construction of Segmental Concrete Bridges* [41] (*AASHTO Segmental Specifications*). Each of the issues mentioned in this section are discussed in more detail in Section 3.6 where the final proposed system is presented.

3.4 Preliminary System–Proposal I

A preliminary precast substructure system, Proposal I, was developed as a part of this research project by Barnes and is summarized in this section. A more detailed report of this work can be found in Reference 7.

Proposal I is a substructure system made up of predominantly precast elements (Figure 3.2). Details were developed for single-column (hammerhead) bents with inverted-T caps for use with *AASHTO* Type III, Type IV, and “Texas U-beam” pretensioned girders. Such single-column bents are inherently less efficient than multicolumn bents but are often desired to minimize site congestion, open up vistas, and/or provide less interference with highway lanes at the lower level. They are usually much more attractive than the standard multicolumn bents. Four different column sizes with one basic cap shape were designed to be assembled for varying heights and widths of standard bridges. These same sections were envisioned to be able to be combined to form straddle and frame bents, again for varying heights and widths (Figure 3.3). This part of the study was largely conceptual and final details were not developed for the multicolumn bents. Column segments would be precast using match-casting techniques. The segments would then be hauled to the site and post-tensioned together and to the foundations.

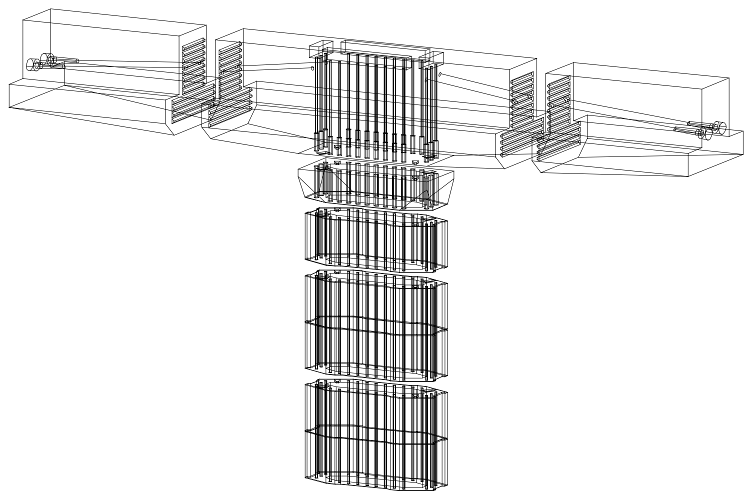


Figure 3.2 Segmental substructure of Proposal I showing longitudinal cap reinforcement and vertical column reinforcement [7]

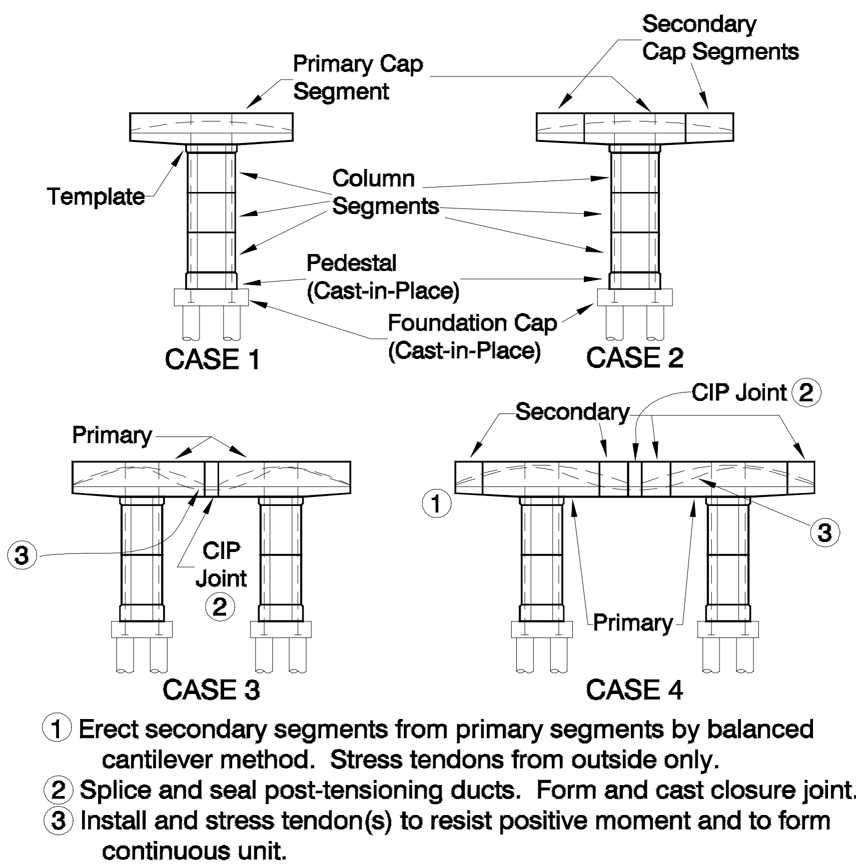


Figure 3.3 Pier configurations foreseen for Proposal I [7]

3.4.1 Elements of the Precast Substructure System–Proposal I

The precast hammerhead column developed in Proposal I is depicted in Figure 3.4. The substructure is made up of three basic segment types; column segments, a "template" segment and inverted-T cap segments. Inverted-T caps were chosen over rectangular bent caps for reasons of improved visibility through the bridge as well as increased clearance underneath the substructure. There are two areas specified for geometry control within these substructures, a joint at the base under the first column segment, and a joint at the top of the column shaft under the top column piece (the "template"). These two joints are cast-in-place with a high quality concrete. The other joints are match cast and epoxy filled. These geometry control locations are further explained in this section and in Section 3.4.3. The design criteria for the substructure units made up of these precast elements are outlined in Reference 7.

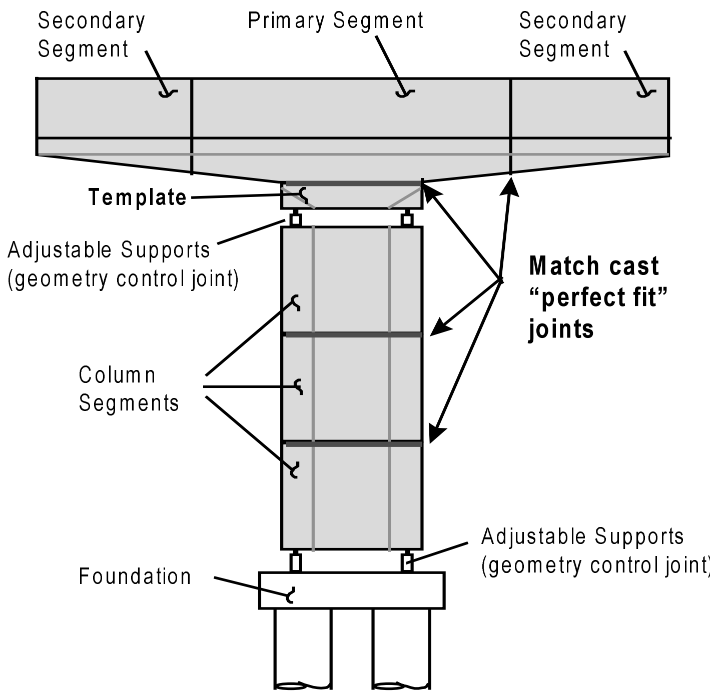


Figure 3.4 Elements of the precast pier for Proposal I [7]

Four hollow column segment sizes were designed (Figure 3.5). Column segments were hollow to reduce the weight of the elements for hauling and erection. These hollow sections were developed for use with high performance concretes having strengths of up to 69 MPa (10,000 psi). Provisions for post-tensioning ducts as well as shear keys were developed. An area was included on the side walls for an optional recess or insertion of a formliner.

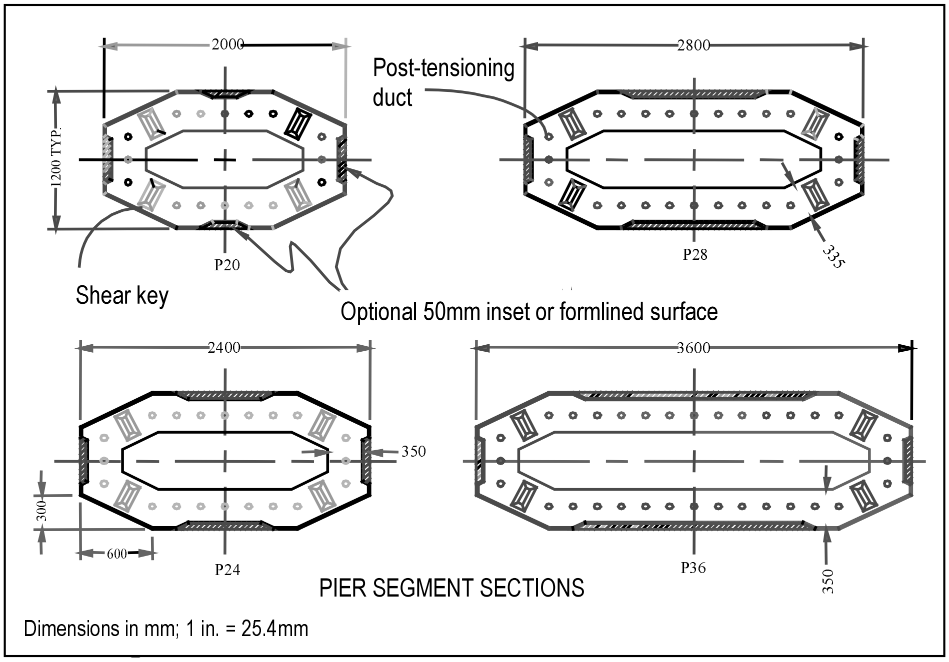


Figure 3.5 The four pier cross-sections for Proposal I [7]

Four template segments were designed, one for each of the different column sizes. The template segment is basically a construction aid. The idea for the template was adopted from the recent Northumberland Strait Crossing (NSC) project in Canada connecting New Brunswick and Prince Edward Island. [17–19] The NSC girder units, fabricated on shore, were precast segmentally and post-tensioned together into haunched span units up to 197 m (645 ft) long. The central segment of each haunched span, or the pier segment, was match cast in the casting yard to the topmost pier segment that was called the template. (A template is defined in the Webster's Dictionary as "a mold (in this case a precast element) used as a guide to form a piece being made." [43]) During NSC erection the light template piece (weighing only 890 kN (200 kips), which is light when compared to a 66.7 MN (15,000 kip) girder segment), could be quickly aligned and cast into place atop the pier. The girder segments were then floated to the site, and placed upon and post-tensioned down to the precast piers (Figure 3.6). These large girder segments were placed on the template for a "perfect" fit in under one hour. The tips of the cantilevers for these segments after placement were within 20 mm (0.8 in) of their designed positions. This is a clear testimony to the excellence of this construction method. The haunched girder segments cantilevering out from the piers were then connected with precast "drop-in" spans to complete each bridge span (Figure 3.7).

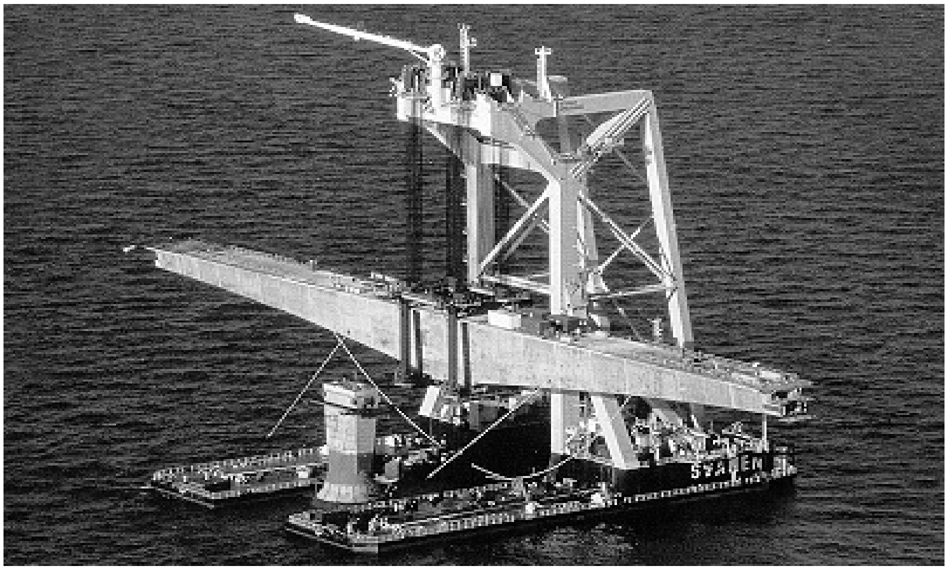


Figure 3.6 Placement of a 200 m (655 ft.) precast segmental girder for the Northumberland Strait crossing.[19] (Template segment is in position on top of the pier.)

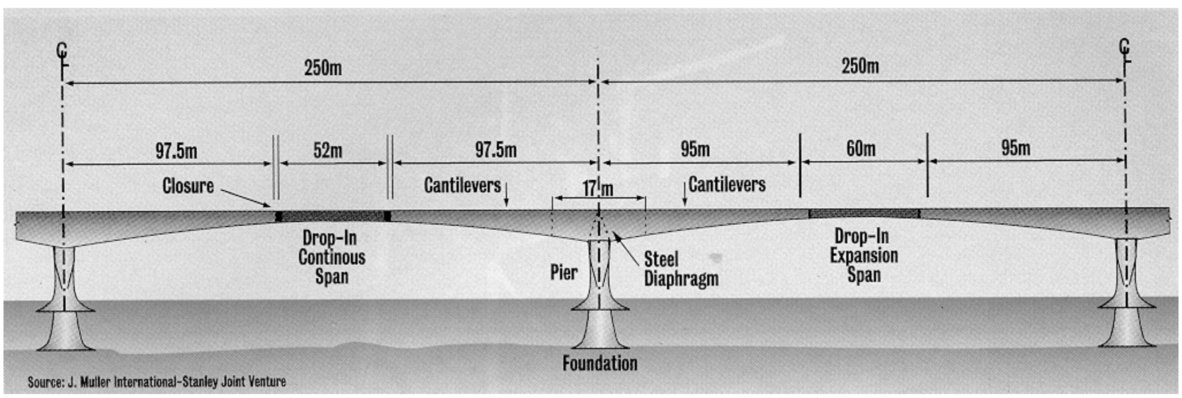


Figure 3.7 Drop-in panels connecting haunched girder segments at the Northumberland Strait Crossing [19]

Although the precast substructure system developed in Proposal I is on a much smaller scale than the NSC project, the erection time can none-the-less be greatly reduced and the geometry control improved by matchcasting the cap to the template (Figure 3.4). The lighter template piece can be properly aligned in the field to the proper deck cross-slope and set with a cast-in-place joint more quickly than the heavier, awkward-shaped cap. For Proposal I the template is essentially a smaller column segment. This small size allowed the template to be not only a construction aid, but also an artistic opportunity. An interesting shape could easily be accommodated in this small segment. A flared chamfer was chosen to visually integrate the vertical shaft with the horizontal cap (Figure 3.8).

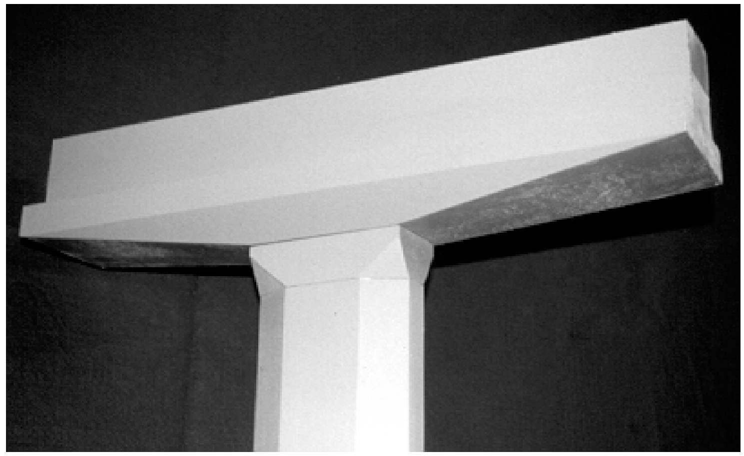


Figure 3.8 Column and cap visually integrated with flaring chamfers on the template segment of Proposal I

The inverted-T caps would be match cast segmentally (Figures 3.4 and 3.11) in a manner similar to the way segmental girders are match cast. Post-tensioning duct and shear key locations are provided. The soffit of the cap tapers in two directions. In elevation, there is a taper increasing the ledge depth from the tip of the cap to the face of the template and column (Figure 3.9a). In plan, the bottom surface of the ledge tapers from the full width of the ledge at the tip of the cap down to the narrower width of the template at the intersection of the template and column face (Figure 3.9b). The increased depth of the tapering cap at the column face provides additional moment and shear resistance at this critical section. The amount of taper provided was more than adequate structurally but was kept at this depth for an attractive visual transition, integrating the horizontal cap and the vertical column. Tapering the cap gave an interesting and striking appearance but made cap form details complex for varying cap lengths.

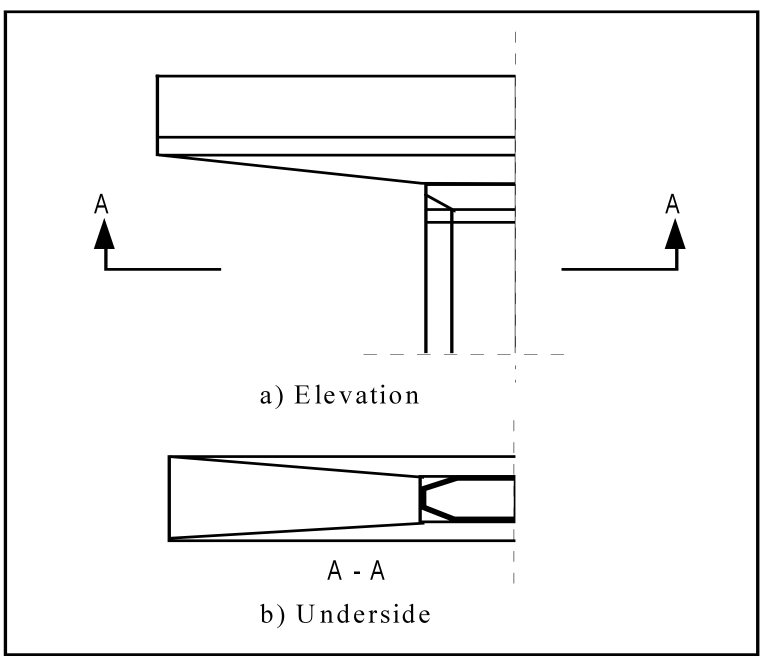


Figure 3.9 Inverted-T cap taper for Proposal I in elevation and plan

3.4.2 *Fabrication Sequence–Proposal I*

The fabrication sequence for the precast substructure elements of Proposal I is outlined in Figures 3.10 and 3.11. The column segments are match cast in their vertical position. Vertical casting has many advantages. Formed surfaces will make up all finally visible faces of the column. The concrete can be better consolidated around the ducts and the inner core form. Handling will be easier as the segments will be stored, hauled and erected in the same orientation as they were cast. Because of the advantages of vertical casting, the “short-line” method is used in which the casting equipment is never more than two segments high. Minimizing this height allows for easier assembly and lifting as well as use of equipment in existing precast plants.

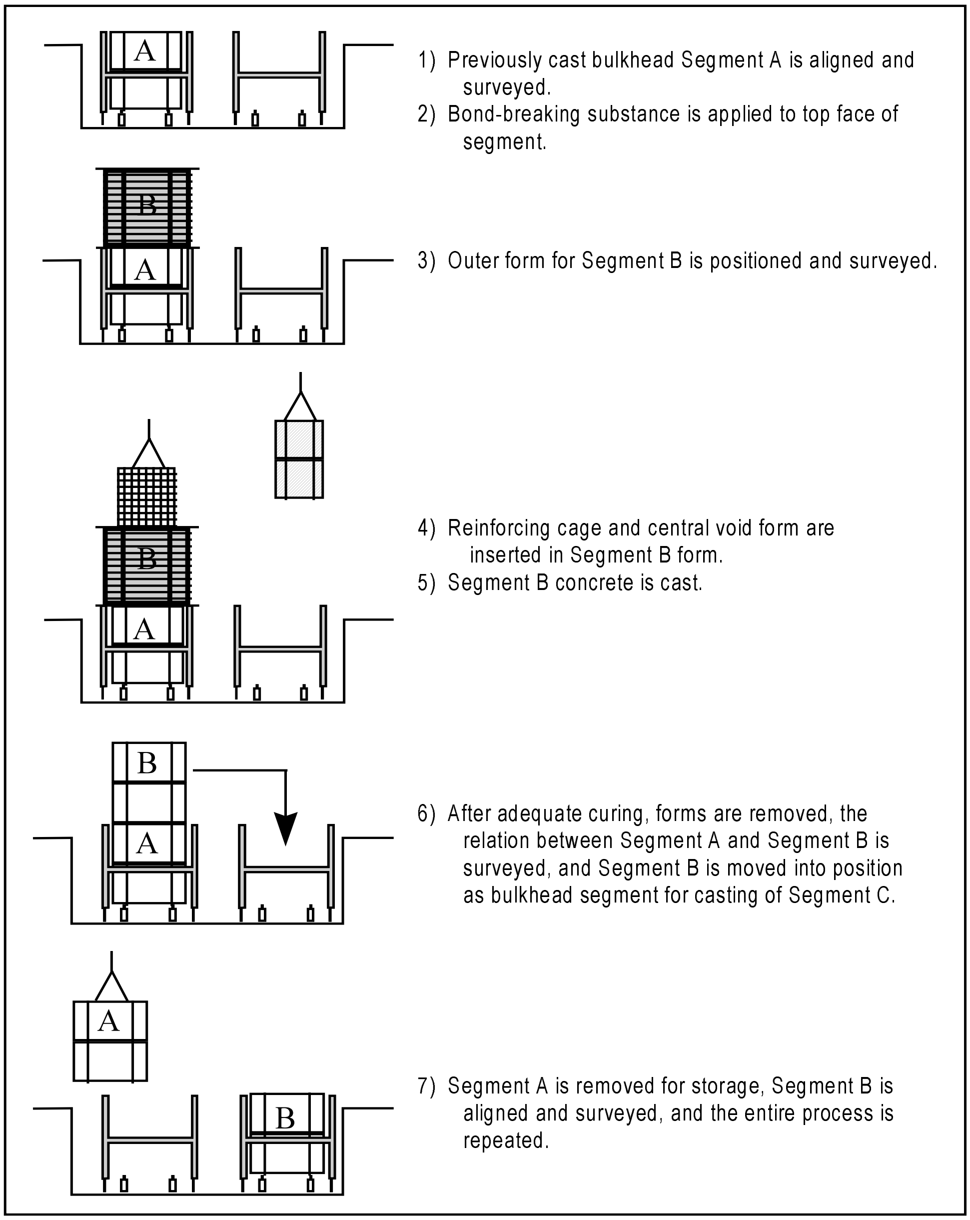


Figure 3.10 Short-line match-casting procedure for precast pier segments

Template segments are cast separately ("loose-fit"). The template is then placed under an opening in the formwork of the cap and is aligned for the proper cross-slope of the future bridge (Figure 3.11). The cap is then cast using the "long-line" method in which the entire cap is match cast in its final relative position and alignment with the template. Long-line casting allows for variations in cap lengths, in tapered soffit inserts, and in match-cast joint locations.

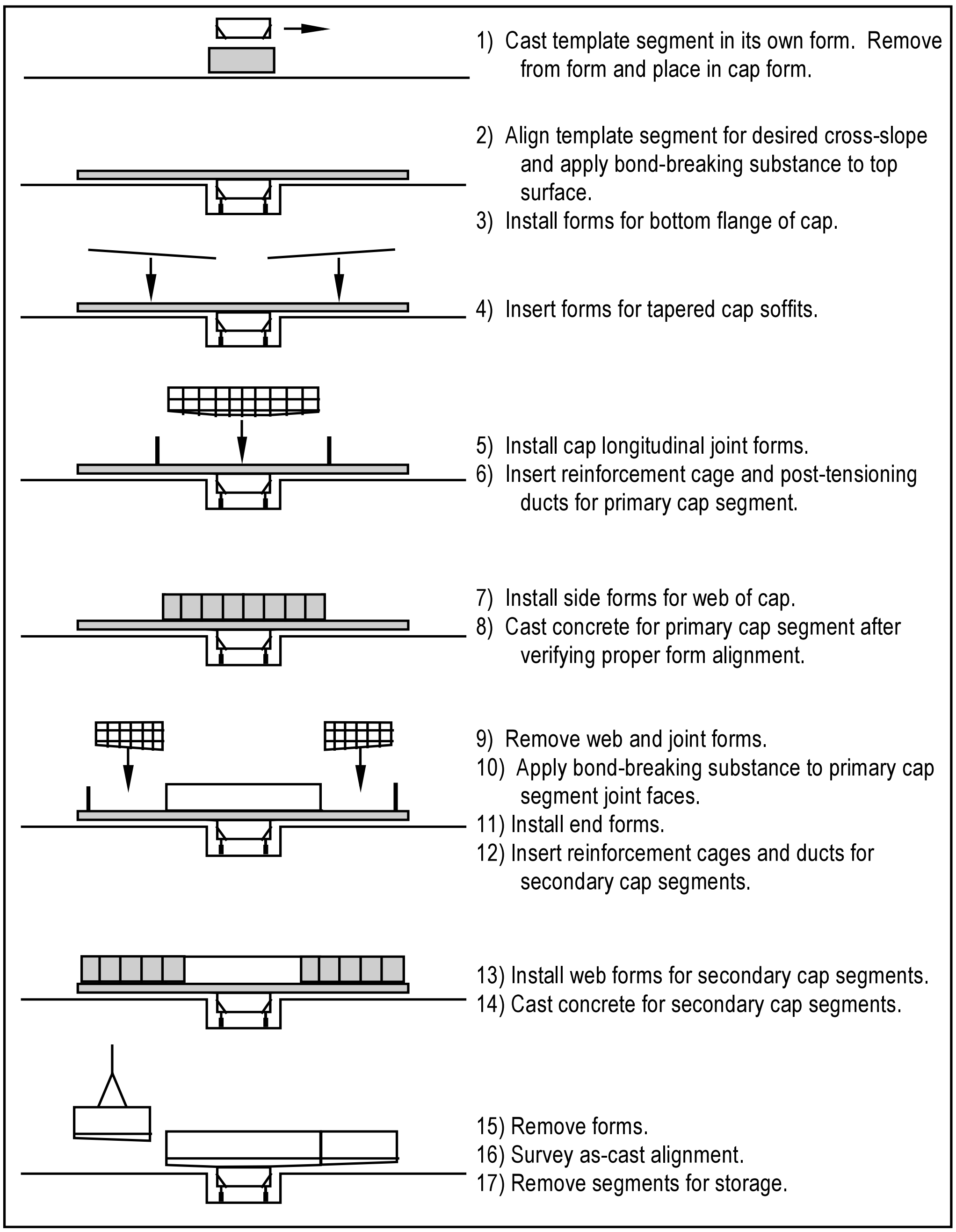


Figure 3.11 Long-line match-casting procedure for precast segmental inverted-T caps with a tapered soffit for Proposal I

3.4.3 Erection Sequence and Connection Details–Proposal I

After the drilled shaft, spread footings or pile cap foundations at the bridge site are completed, precast substructure elements can be hauled to the site for erection. The erection sequence is shown in Figure 3.12. The first segment is placed and aligned on adjustable supports on top of the footing. If needed for geometry ranges, there is also the option of casting a recess into the footing and placing the bottom of the first segment in the recess. A common adjustable support system would be a steel frame that can be adjusted with screw threads or shims. Once aligned, ducts are connected and the post-tensioning bars (PT bars) are threaded into anchors previously cast into the foundation. The bottom segment is then "locked" in position with a cast-in-place joint. The height of this cast-in-place joint can vary in accordance with the final height required for the pier. Pier segments are to be cast in 2400 mm, 1200 mm, and 600 mm (96 in, 48 in, 24 in) tall segments. Therefore the maximum required height of the cast-in-place joint will always be less than 600 mm (24 in).

Once the first segment is permanently in place, the next segment can be lifted into place above the first segment. PT bars are placed and coupled while the new segment is held above the previously placed segment. Epoxy is applied to each face of the joint and the top segment is lowered into position. Post-tensioning can then be stressed to provide a minimum 0.28 MPa (40 psi) pressure across the joint for even setting of the epoxy. The 0.28 MPa (40 psi) pressure is required by the 1994 *Interim AASHTO Segmental Specifications*. [41] TxDOT has had success in using only the segment weight to squeeze out the epoxy. Additional column segments are placed similarly.

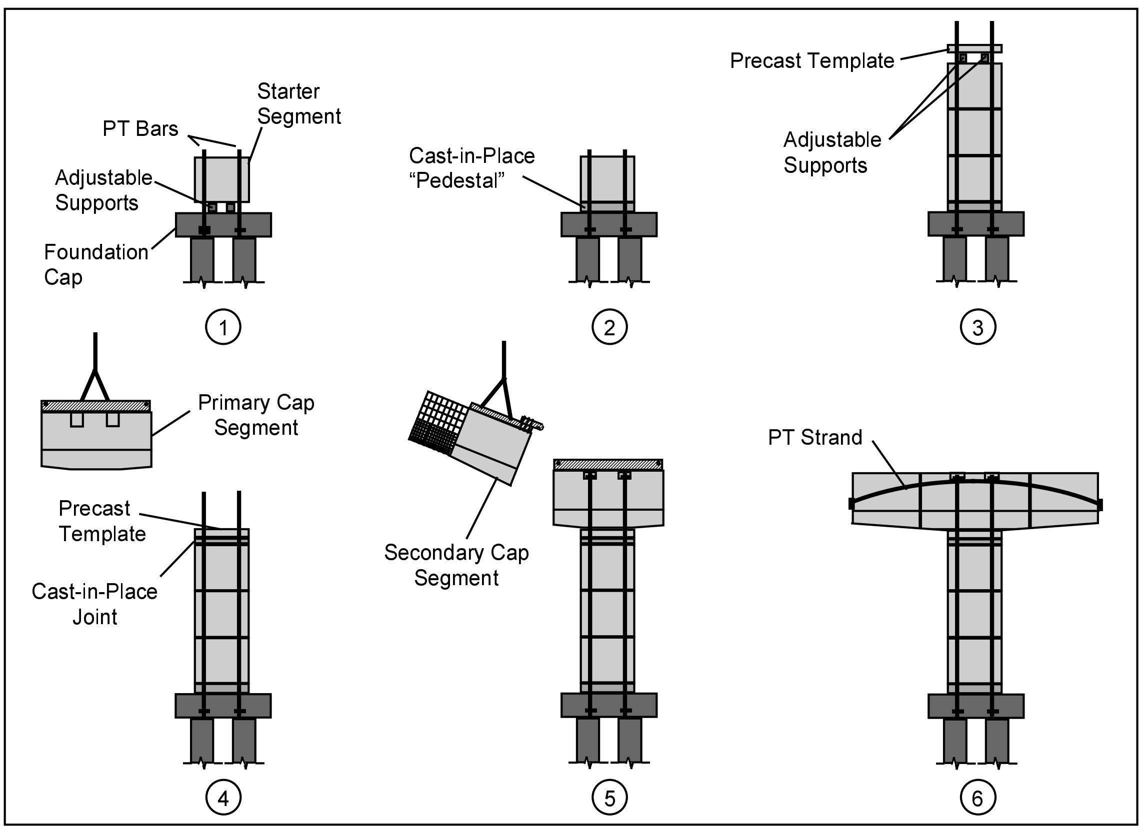
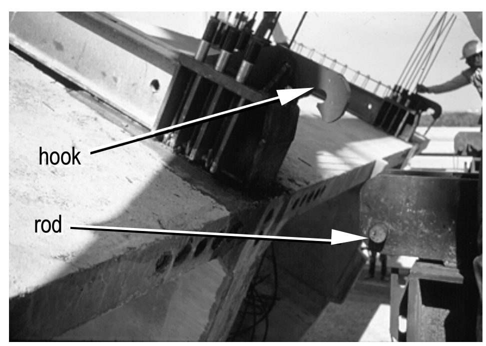


Figure 3.12 Erection sequence for the segmentally precast hammerhead pier of Proposal I

After the last ordinary column segment is placed, the template segment is placed on temporary adjustable supports above the topmost column segment and aligned to provide the necessary cross-slope and elevation for the match-cast cap. This joint is then formed and “locked” in place with a cast-in-place joint. Both cast-in-place joints are envisioned as constructed of a highly durable, strong jointing material of similar quality to the rest of the column shaft. Providing the second designated geometry control joint under the template serves two purposes. First, this geometry control joint allows for accurate cap placement. Second, any unforeseen out-of-straightness resulting from initial setting and subsequent placement of column segments can be corrected at this joint.

With the template in place, the primary cap piece can be rapidly positioned into its match-cast alignment on the epoxy joint and post-tensioned down to the column. Whether post-tensioning bars or looped post-tensioning strand are used, all post-tensioning operations can be carried out from above. Secondary cap segments can next be lowered into place with simple hooking hardware attached to the top of the cap segments. For controlled placement, one method would be to attach a hook to the secondary cap segment that can be placed over a rod between lifting beams on top of the primary segment. This method was used successfully for erecting the JFK Causeway in Corpus Christi (Figure 3.13). Once the secondary cap segments are in their match-cast position, the cap is post-tensioned together and the superstructure can be placed. Depending on the erection sequence for the superstructure, the cap may need to be post-tensioned in stages to avoid exceeding zero tension stress limitations. This possible tensile stress staged post-tensioning is of particular concern across segmental joints, where no tension is permitted at any stage.


(a)



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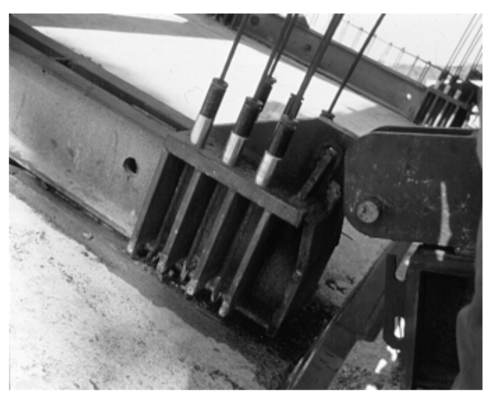

(c)

Precast box girder segments were lowered into position (a) with “hooking” hardware (b & c) that was temporarily fastened to the top of the segments to be joined.

Figure 3.13 Construction of the JFK Causeway

3.5 Evaluation of Proposal I–Industry Concerns Addressed

Upon completion of Proposal I for a precast substructure system, details were presented to a number of industry personnel for review and comments. Meetings were held with representatives from the precast industry including designers, precasters, formwork manufacturers, and material suppliers. A presentation introducing this system was also given to TxDOT engineers from across the state. Other presentations were given at national conferences including the 1996 American Concrete Institute (ACI) Fall Convention in New Orleans, Louisiana, the 1996 American Segmental Bridge Institute (ASBI) Convention in Orlando, Florida, and the 1997 Transportation Research Board (TRB) Conference in Washington, D.C.

The presentations at the national conferences were well-received. A few state DOT engineers were interested in the precast system but felt they would have difficulty finding local contractors to do the work. As mentioned in Section 2.2.3, many practitioners requested more information on the system for their own development in the future. A few practitioners commented that they have been precasting substructures for small local projects successfully for years.

The meetings with industry personnel and the presentation to TxDOT engineers were the most helpful in furthering the development of the precast substructure system. Important concerns and questions were raised that resulted in substantial subsequent improvements in the original proposal. Other concerns were recognized as commonly held beliefs that were more like “old wives’ tales.” Nonetheless, such reservations and perceptions needed to be addressed to dispel false notions. The key concerns addressed included the cap being segmental as opposed to one piece, the suitability of the system for fabrication in precasting plants more accustomed to precast girders, the relative merits of match casting vs. loose fit fabrication, the choice of epoxy joints vs. mortar joints, and the worries over problems in grouting of vertical tendon ducts.

3.5.1 Segmental Cap

One of the key concerns frequently raised about Proposal I was the configuration of the cap. The segmentally constructed cap was viewed as a large obstacle to adopting the entire system. A wide range of engineers indicated it would be more desirable to use a single cap piece in hammerhead bents or a pair of cap pieces connected with cast-in-situ joints in multicolumn bents. The segmental cap scheme was originally chosen due to the large weight of the entire cap. As discussed in Section 3.3, precast element weights needed to be restricted to less than about 700–750kN (160–170kips) to keep hauling and erection costs down. The longest cap needed for the single pier system is 13.1 m (43 ft) for a 14 m (46 ft) or three-lane roadway. If cast as one unit, such a Proposal I cap would weigh close to 1100 kN (250 kips). In addition, concerns were expressed about the form complexity with the tapering of the cap soffit. To ease the erection process, a new scheme using only single prismatic cap pieces was developed and investigated. The new scheme is presented in Section 3.6.

3.5.2 Suitability for Fabrication in Precast Plants

A major goal of this project is to develop a precast substructure system which could become a standard substructure design option. Standardization will make the system become increasingly economical through widespread use. It is important therefore to have a system that is attractive to all parties involved with fabrication and erection of standard bridge systems; both contractors and precasters.

Match-cast segmental caps require post-tensioning. The segmental caps of Proposal I could be fabricated by either precasters or general contractors. While a post-tensioned system would be attractive for general contractors, a pretensioned system, not part of Proposal I, would be most attractive for many PCI member precasting plants. A high rate of fabrication and use of existing precast plant pretensioning bulkheads and equipment will ease the investment costs of implementing a new standard for substructure design. Pretensioning the cap element will also reduce the amount of nonprestressed reinforcement required in the cap to prevent cracking during transportation and handling.

Therefore, after evaluating Proposal I, it was felt that the post-tensioned system should be secondary, and the primary option should be a pretensioning proposal for the precast caps. A new cap fabrication sequence needed to be explored that would give precasters a fabrication, forming and pretensioning option more suited to their existing plants. A major goal became development of a pretensioned cap system within weight limits which would permit handling by truck crane units of capacities associated with the placement of longer span precast girders. However, in the interest of the project's future use, a revised system should also have an alternate post-tensioned cap option for general contractors. Regardless of where the fabrication of such a system is performed, the system can still be standardized. Standard plan sheets as well as standard formwork should be developed.

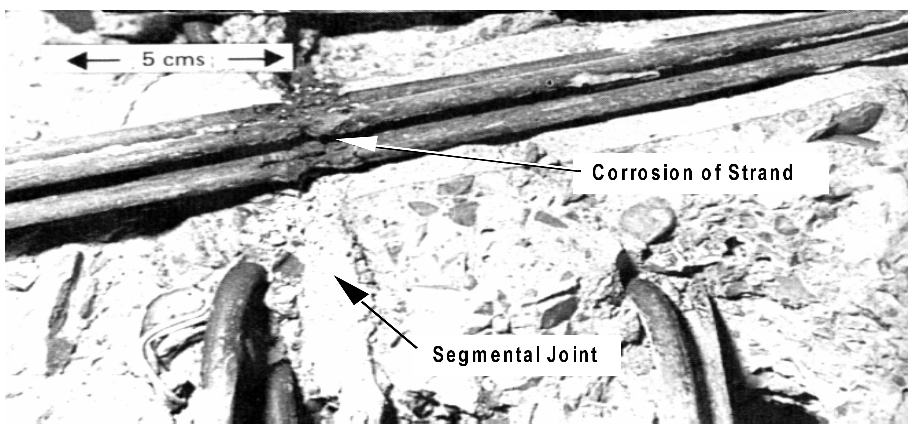
3.5.3 Match Casting vs. Loose-Fit; Epoxy vs. Mortar Joints

There were many questions and concerns about the type of joints chosen for the segments in Proposal I. The jointing material is directly related to the casting procedure. Dry joints or epoxy joints can be used with match-cast segments while mortar or cast-in-place concrete is used with segments fabricated individually ("loose-fit").

Dry joints are permitted only in mild climates that do not experience freezing or exposure to salt water. This limitation prohibits use in most locations in Texas. Dry joints also lack the uniform bearing surface provided by the epoxy filled joints and are more prone to edge crushing. Therefore dry joints were not considered. The option of using a thin coat of epoxy to seal match-cast joints has had excellent success in the past contributing to both erection ease and durability. The thin coat of epoxy lubricates the joint during segment placement and thus allows for easy, accurate joining of the segments that were aligned and match cast during fabrication, with little site measurement. The segments "auto-seat" into their match-cast positions. With loose-fit fabrication, each segment must be carefully aligned and held on site before jointing. As each joint is a geometry control joint, a thicker joint of mortar is required.

The option of match casting with epoxy joints was chosen over loose-fit fabrication with mortar joints for reasons of durability and construction speed. Although loose-fit fabrication can occur at any rate the precaster chooses (depending on how many sets of forms the precaster or contractor invests in), the procedure of placing mortar and aligning segments on site is difficult and time-consuming to perform accurately. This is a major drawback for a system developed to speed up on-site construction. Difficulties arise also with ensuring the quality and even distribution of the mortar between the segments. This can lead to partially filled joints, stress concentrations, cracking and eventual corrosion of the reinforcing steel. Mortar joints have had durability problems in the past (Figure 3.14). [44] Measures to ensure mortar durability at every joint would be required.

(a)



(b)

Figure 3.14 (a) The collapse of a bridge at Ynys-y-Gwas in Wales; (b) Collapse was attributed to corrosion of prestressing strand at mortar-filled segmental joints [from Reference 69]

Match casting of column segments can be performed at an economical rate completing at least one segment per day per casting machine. With match casting, alignment is taken care of in the casting yard. The previously aligned and "perfect fit" casting of segments allows for rapid placement on the construction site. The excellent durability record of epoxy joints is promising. Therefore to maximize both construction speed and substructure durability, a system of match-cast segments with epoxy joints was developed. With the proposal herein, it is still necessary to have two cast-in-place alignment joints in each of the otherwise match-cast piers. Concrete or mortar placed in these locations must be durable and the tendons protected.

3.5.4 Grouting of Vertical Tendons

Some practitioners have questioned the reliability of grouting vertical tendon ducts. However, many guidelines have been developed for this procedure giving successful results. [45] Pressure grouting in particular has been successful in the past. Recent experience in Texas (US Highway 183 in Austin) and Florida (State Road 430 over Seabreeze Boulevard in Volusia County) has been very positive with regard to pressure grouting vertical tendon ducts. Concerns about this procedure appear to stem from lack of experience, rather than poor performance in the past.

3.6 Final System–Proposal II

After consultation with a variety of members of the construction industry, the precast substructure system of Proposal I was modified and extended for use with single, straddle, and frame bents. The industry concerns discussed in Section 3.5 were addressed, and suggested improvements were incorporated into the new system, Proposal II. The process by which the new elements were developed is discussed in the following sections. Complete calculations and plans for several examples of Proposal II are included in Appendices C and D of Reference 2. Again, much attention was paid to the single-column hammerhead bent. The aesthetic and clearance advantages of such a pier would have to be compared to the generally lower costs of multicolumn bents. Multicolumn bents are included in this proposal, but for wider bridges.

3.6.1 Pier Segments–Proposal II

The four column sizes suggested in Proposal I were substantially revised and are shown in Figure 3.15. While four column sizes are included in Proposal II, one of these is a considerably smaller section suitable for many frame bents. The dimension transverse to the cap of all pier segments was selected as 1200 mm (48 in) to correspond with the width of the cap stem. These matching dimensions facilitate the continuation and anchorage of post-tensioning steel from the column into the cap. Column post-tensioning may be either post-tensioning bars or strands. Post-tensioning bars would be coupled to anchors cast into foundation caps. Post-tensioning strand would be threaded from the top of the cap into ducts passing through a 180-degree turn in the foundation cap resulting in both ends of the strand exiting the top of the cap (similar to Figure 2.7a and shown in Figure 3.31). Thus, all strand anchorages and post-tensioning operations are performed at the top of the cap. Column erection including foundation connections and post-tensioning options is further discussed in Section 3.6.5.

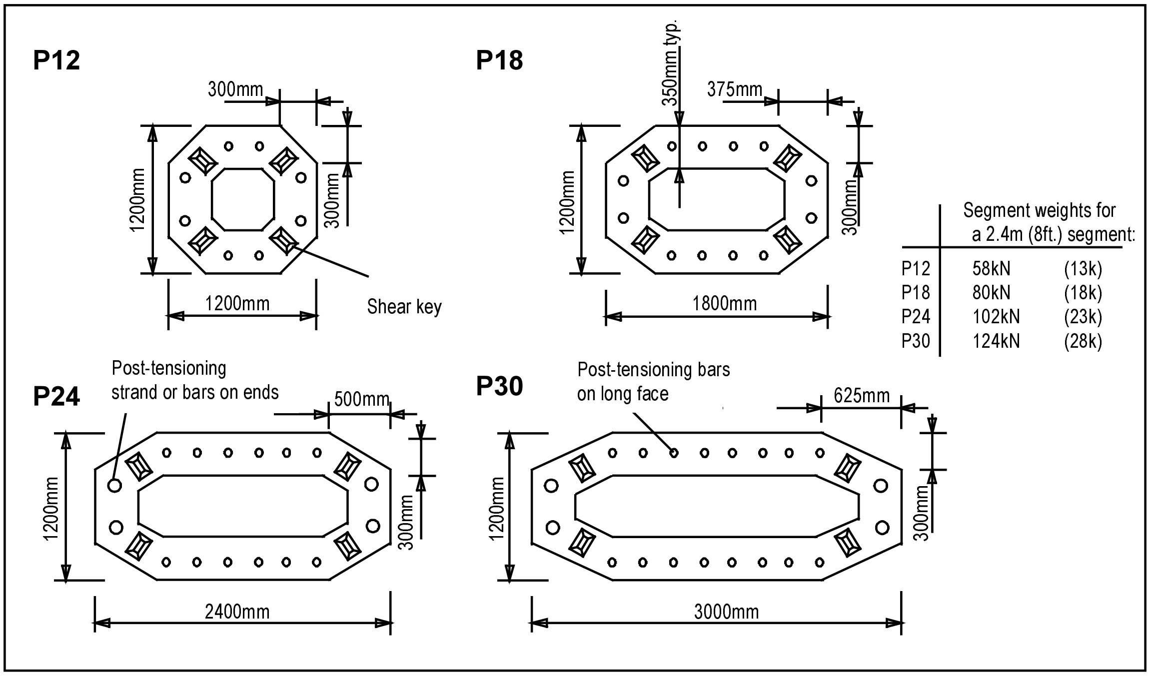


Figure 3.15 Four pier cross-sections for Proposal II

The column dimension parallel to the cap (transverse to the span of the bridge) for each column size is 1200 mm, 1800 mm, 2400 mm or 3000 mm (48 in, 72 in, 96 in, or 120 in). The column segments are designated as P12, P18, P24 and P30 to correspond with the transverse dimension of the column segment. Pier segments can be cast in 600 mm, 1200 mm and 2400 mm heights (24 in, 48 in, and 96 in) and should be cast with a high performance concrete. Figure 3.16 shows typical column detailing for a 2.4 m (96 in) tall P18 pier segment.

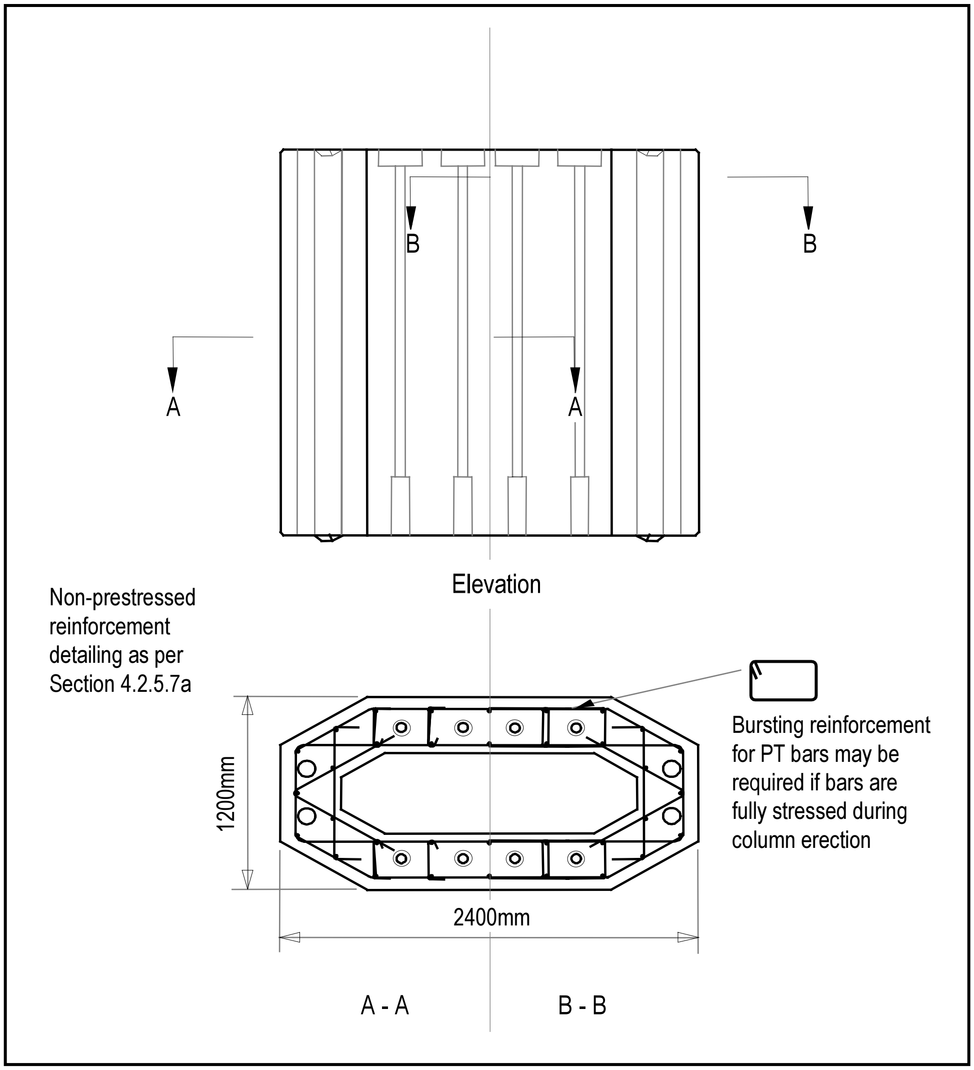


Figure 3.16 A typical 2.4 m (8 ft.) P24 segment

The basic hollow pier shape of Proposal II is similar to that of Proposal I. One detail was changed regarding corner chamfering. The corner chamfers for Proposal II vary proportionally along the column long dimension with the increasing pier dimensions. In Proposal I, the corner chamfers had the same dimensions regardless of pier size. The visual effects of different column chamfering techniques are discussed in Section 4.2.5.

The pier segments of Proposal II have no recess built into the form. The wall thickness and concrete covers for the piers can easily accommodate inserts or formliners attached to the formwork. The designer is free to choose whatever type of insert or formliner, if any, would be most appropriate for a given project. The exact type of insert should be chosen after the cap longitudinal reinforcement is determined. This longitudinal cap reinforcement (discussed further in Section 3.6.2) dictates the location of column tendon ducts. A formliner pattern will remove a certain portion of concrete cover for the column reinforcement. The amount of cover that may be removed by the insert must still leave proper cover for the column reinforcement (dictated by exposure conditions and specified *in AASHTO Specifications*).

The engineer may also choose whether or not the joints between pier segments will be accented with chamfers. Backing bars can be tack welded to standard steel forms to create chamfers. The bars can be simply ground off if chamfers are not desired for the next project. Chamfering joints helps hide staining caused by epoxy oozing out of the match-cast joint during post-tensioning operations. However, chamfering the joints calls attention to the joint and gives the pier a masonry-like appearance, which may or may not be desirable. The choice of pier appearance will be the engineer's.

Shear keys are provided at the chamfered corners. The walls of the hollow pier segments provide room for post-tensioning bars of up to 36 mm (1.375 in) with a strength of 1030 MPa (150 ksi) or 1100 MPa (160 ksi) as well as multistrand tendons up to the current 19K6 size made up of 19 25 mm (0.6 in) strands with a strength of 1860 MPa (270 ksi). The hollow core of the segments provides room for internal drainage ducts without reducing section efficiency. As shown in Table 3.1, pier segment weights for Proposal II are comparable to those of Proposal I and are easily within the capacity of cranes used for handling precast concrete girders.

Table 3.1 Pier segment weight comparison

	Pier Segment 2.4 m (8 ft)	**Weight kN (kips)**
Proposal I	P20	85 (19)
	P24	101 (23)
	P28	116 (26)
	P36	148 (33)
Proposal II	P12	58 (13)
	P18	80 (18)
	P24	102 (23)
	P30	124 (28)

3.6.2 Bent Caps and Template Segments

The bent cap design of Proposal I changed considerably for Proposal II. Match-cast segmental caps were seen by industry personnel as too cumbersome. Construction speed and efficiency would be greatly improved if the cap could be precast and handled as one segment. The cap dimensions for Proposal I resulted in extremely heavy cap elements. A cap length of 7 m (23 ft) from Proposal I would weigh 700 kN (160 kips)—around the desired maximum weight for any precast element in the system (discussed in Section 3.3). The range of design dimensions for the proposed substructure system requires a single cap element to be up to 13.1 m (43 ft) for a three-lane bridge with a shoulder. The maximum 7 m (23 ft) single cap element of Proposal I is wide enough only for a single-lane bridge with a shoulder.

To have a system whereby the cap would be one single precast element up to 13.1 m (43 ft), the weight of the caps from Proposal I needed to be reduced considerably. The taper of the cap in Proposal I had been designed for structural efficiency as well as visual integration of the horizontal cap with the vertical pier. (The taper increases where the moment and shear in the cap are highest and thus visually represents the flow of forces from the girders to the column and foundation.) By removing the taper, the cap weight could be reduced by 22%. Removing the taper also reduces the internal lever arm at the critical section at the column face. Rather than providing additional reinforcement to make up for the reduced internal lever arm, the design bending moment for the cap was substantially decreased by reducing the cap weight and by widening the template underneath the cap (Figure 3.17). Although removal of the cap taper reduced the cap's resistance to load effects such as shear and bending moment, the efficiency of the overall structure was preserved because the column flare significantly reduced the magnitude of these load effects by changing the location of the cap critical design section. Rather than deepening the cap section at the cap-column interface to better resist the moment, the column section was widened to shift the point of critical moment in the cap outwards so that lower moment capacity would be required.

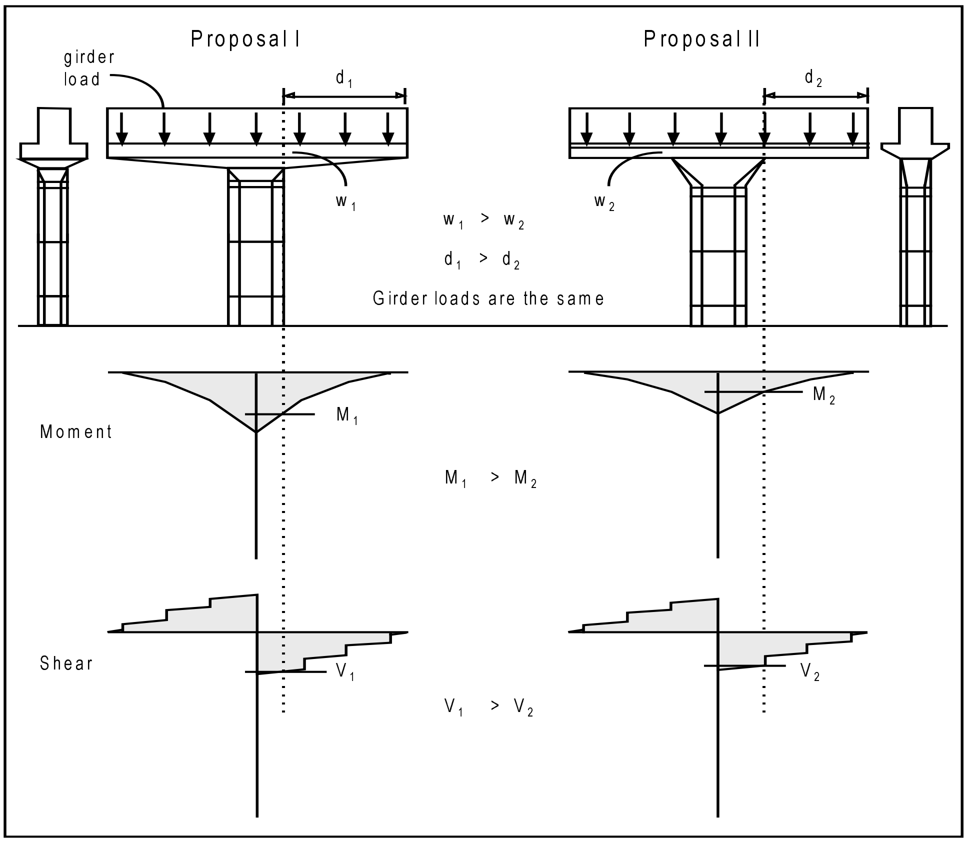


Figure 3.17 Critical cap design moment and shear are reduced when the basic structural system changes from a tapered cap (Proposal I) to a flared template (Proposal II)

Removing the cap taper also removed the need for a complex form system for the cap that could handle tapers for different length caps. Such a form system signaled to many industry personnel a higher risk and higher uncertainty of successful fabrication. It was indicated that such uncertainty could lead to substantially increased costs. Although a tapered cap is visually attractive and structurally efficient (efficient use of materials), the economy of this design alternative would be sacrificed. This dilemma was a good example of how engineers must strive to balance the three engineering disciplines of aesthetics, efficiency and economy (discussed in Reference 3) while meeting the user needs (functionalism) within the project constraints. In this case, a primary concern for the design of a prototype precast substructure system was that it could be easily standardized. Economical standardization dictates the use of simple, reusable and easily sealed forms. The tapered cap of Proposal I did not meet this criteria. The resulting uniform depth cap and tapered template for Proposal II provides a system that is attractive and efficient (in terms of material requirements, fabrication and constructability).

The small template sections of Proposal I were increased substantially in size and were made to be proportional with each pier size. The resulting template elements for Proposal II are short, flared pier segments. A flared template provides a stronger visual integration between the pier and cap than a subtle cap taper. The decreased bending moment at the critical section in the cap requires less cap reinforcement. Figure 3.18 shows the four template sizes of Proposal II. Each template size corresponds to one of the four pier segments. They are similarly designated and referred to as T12, T18, T24 and T30. As shown in Table 3.2, their weights are within the 700–750 kN (160–170 kip) maximum range.

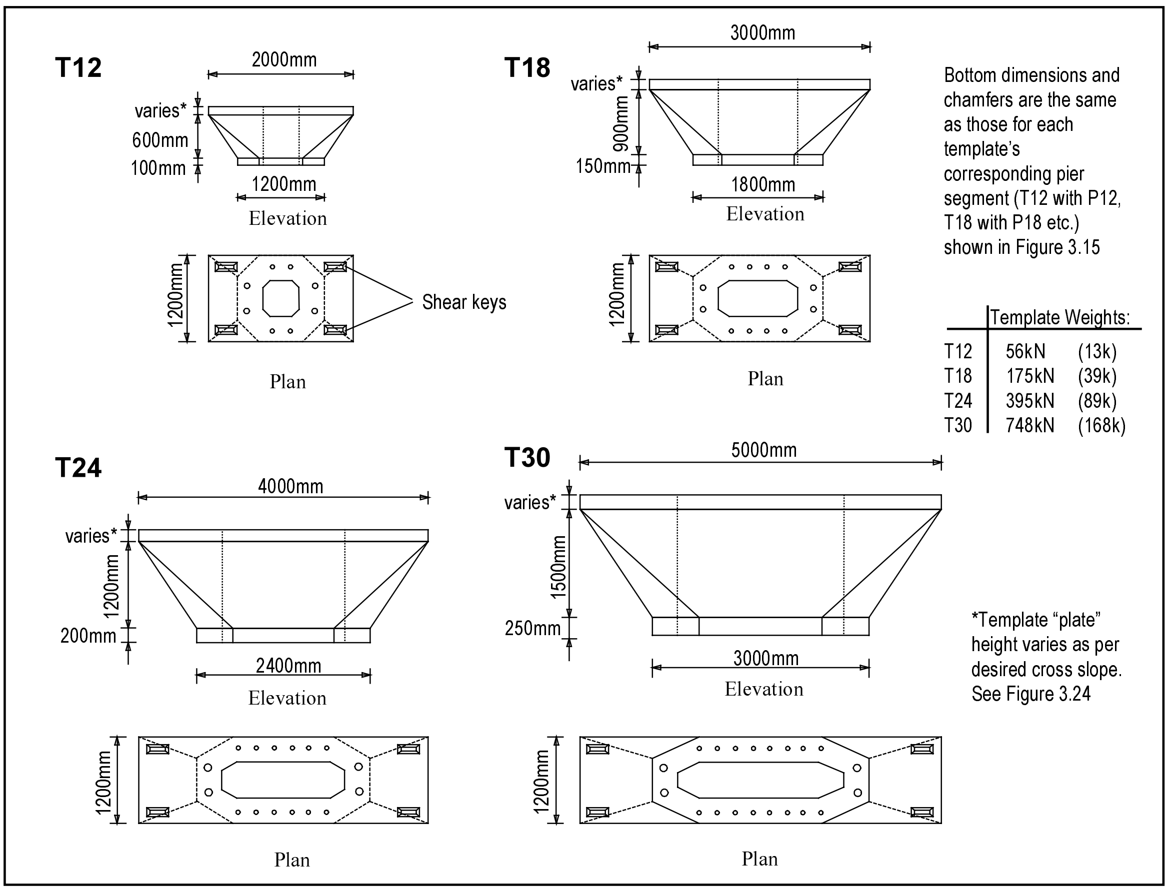


Figure 3.18 Four template segments of Proposal II

Table 3.2 Template weights for Proposal II

Template Segment Proposal II	Weight kN (kips)
T12	56 (13)
T18	175 (39)
T24	395 (89)
T30	748 (168)

To further decrease the weight of the cap, the structural function of the cap was examined. As shown in Figure 3.19, the flow of forces can be seen from simple strut- and-tie models. Examining the flow of forces for the solid cap of Proposal I (Figure 3.19c and d) shows areas where the concrete is not needed structurally—in the center of the stem and in the bottom outer corners of the ledge. Figure 3.20 shows the sequential process of removing the unwanted material and the effect the material removal has on the weight of a 13.1 m (43 ft) cap–the longest single cap piece required for the system. Removing this unnecessary dead load from the inverted-T cap further reduces the amount of reinforcement required in the cap.

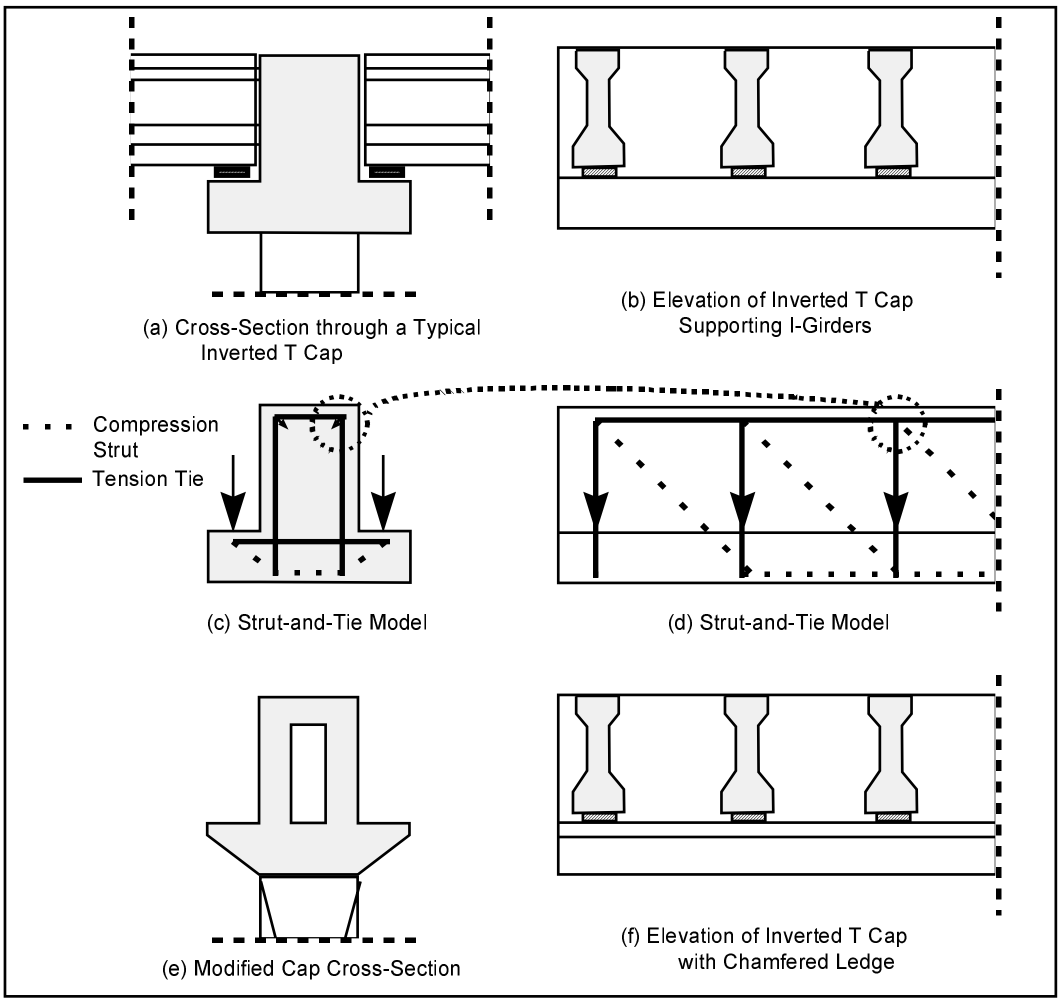


Figure 3.19 A strut-and-tie model was used to develop an efficient new shape for an inverted-T bent cap

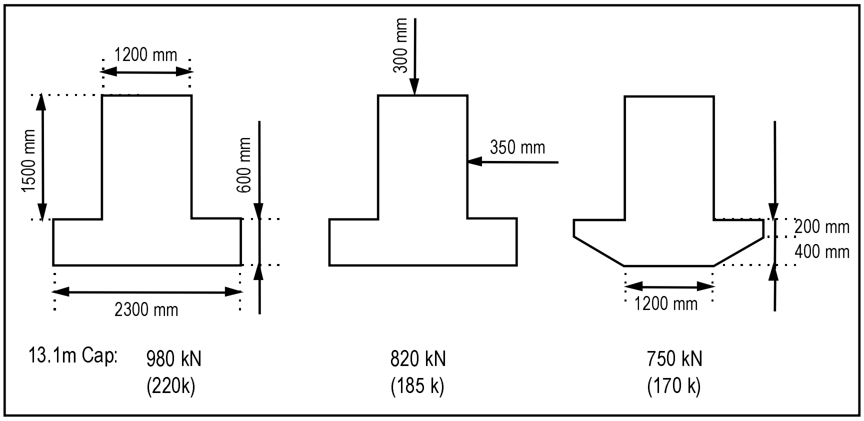


Figure 3.20 Modifying the cap cross-section led to a reduced cap weight

The precast cap elements may be longitudinally pretensioned, post-tensioned or a combination of both. The web walls of the inverted-T stem were minimized to 350 mm (14 in) in the cantilevered portions and 375 mm (15 in) at the cap-column connection. This width should provide adequate cover for anchorage zone and shear reinforcement with sufficient room for concrete placement and consolidation around the longitudinal cap reinforcement. Figure 3.21 shows details of a precast cap with two longitudinal reinforcing options (one fully pretensioned and one fully post-tensioned). Figure 3.22 shows portions of the cross-sections at the cap/column connection to show the cover and spacing of the three prestressing options. For the fully pretensioned option, each wall can accommodate four columns of 12.7 mm (0.5 in) diameter pretensioning strands on 50 mm (2 in) centers, a 50 mm (2 in) duct for column PT bars with proper coupling sheaths, and interior cover (Figure 3.22a). The walls can also accommodate 100 mm (4 in) ducts for 19K6 multistrand longitudinal tendons instead of pretensioning strand with the column PT bars located up to 50 mm (2 in) closer to the outside cover of the stem (Figure 3.22b). This will require up to a 10 degree bend in the stirrup at the base of the stem to accommodate the sheathing and coupler for the column vertical PT bars (see cross-section B2 in Figure 3.21c). A combination of cap longitudinal pretensioning and post-tensioning may also be accommodated using the two outer columns of pretensioning and a maximum of a 100 mm (4 in) duct (required for a 19K6 multistrand tendon) for post-tensioning (Figure 3.22c). For this scheme too, the column PT bars will be closer to the outside cover of the stem. The pre- and post-tensioning combination for cap longitudinal reinforcement will be typical for frame bents. When specific post-tensioning devices are selected and necessary supplementary and continuing reinforcement selected, a full-scale anchorage zone test should be performed. Due to the low column overturning moments in frame bents, the amount of vertical post-tensioning required can typically be handled in the column "ends" (as indicated on the P24 segment of Figure 3.15). In these bents, the PT bars will be located towards the center of the inverted-T stem. Thus, the coupler sheathing will not require any additional bending of stirrup cage reinforcement. The varying location of the vertical PT bars, that depend on cap longitudinal reinforcement, must be accounted for in column design (including cover requirements when form inserts are being chosen). An alternative design for the pretensioned cap would be to use 0.6 in diameter strands. The number of strands could be reduced with a probable reduction in plant labor costs and slight increase in the eccentricity of the prestress force.

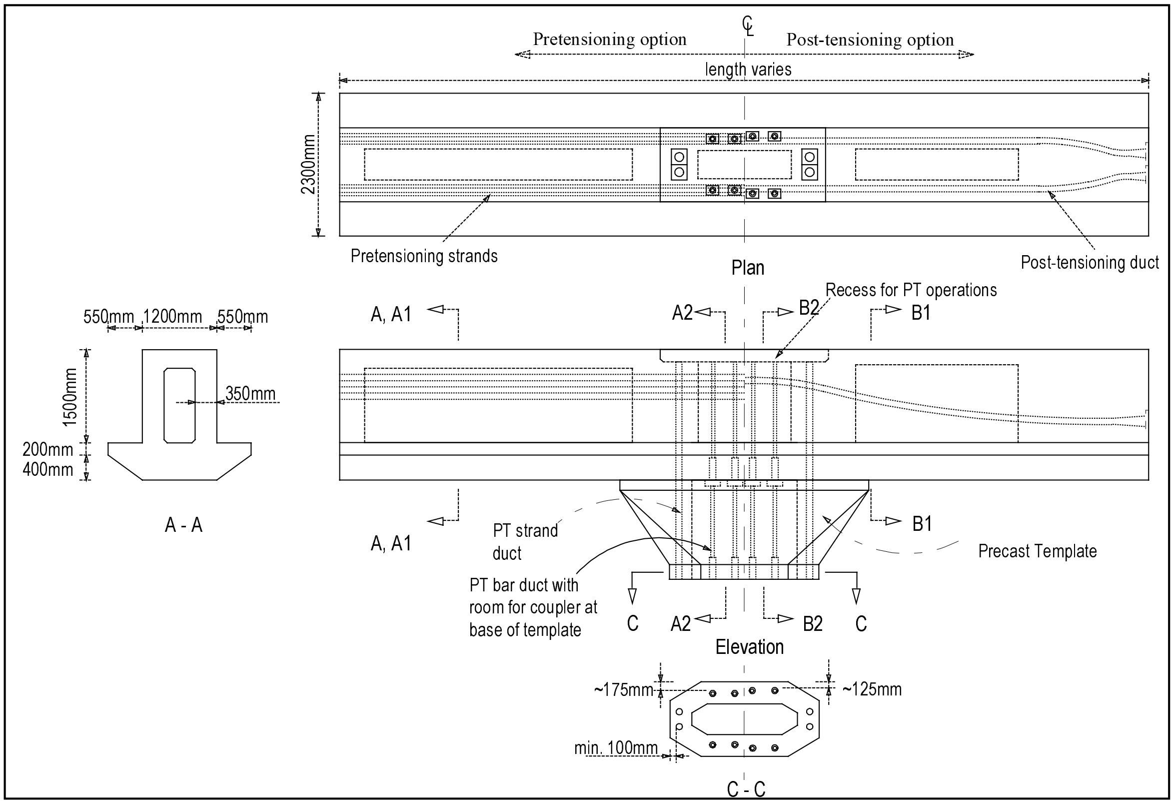


Figure 3.21a Precast cap details for Proposal II

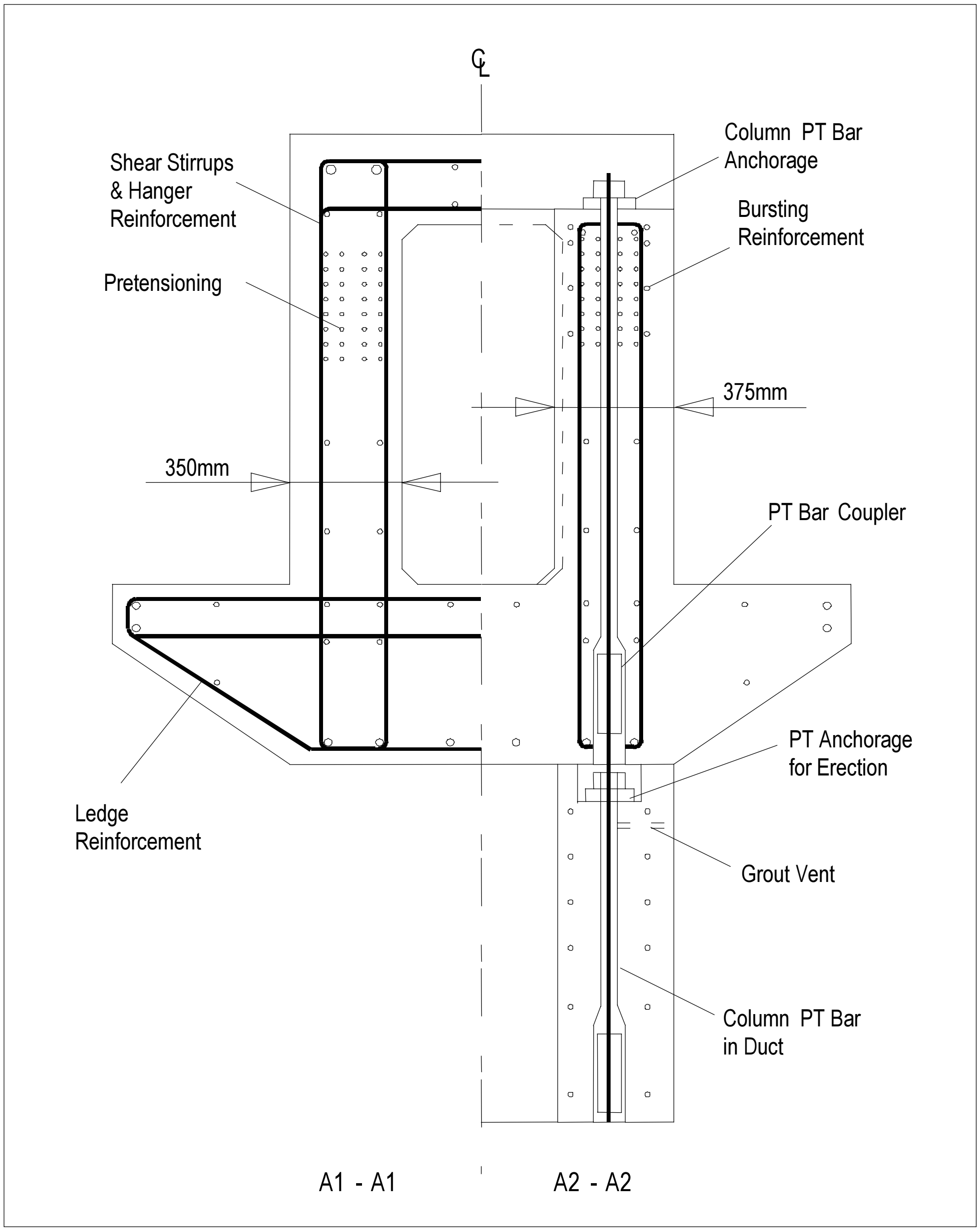


Figure 3.21b Cross-sections of cap and template for a fully pretensioned cap

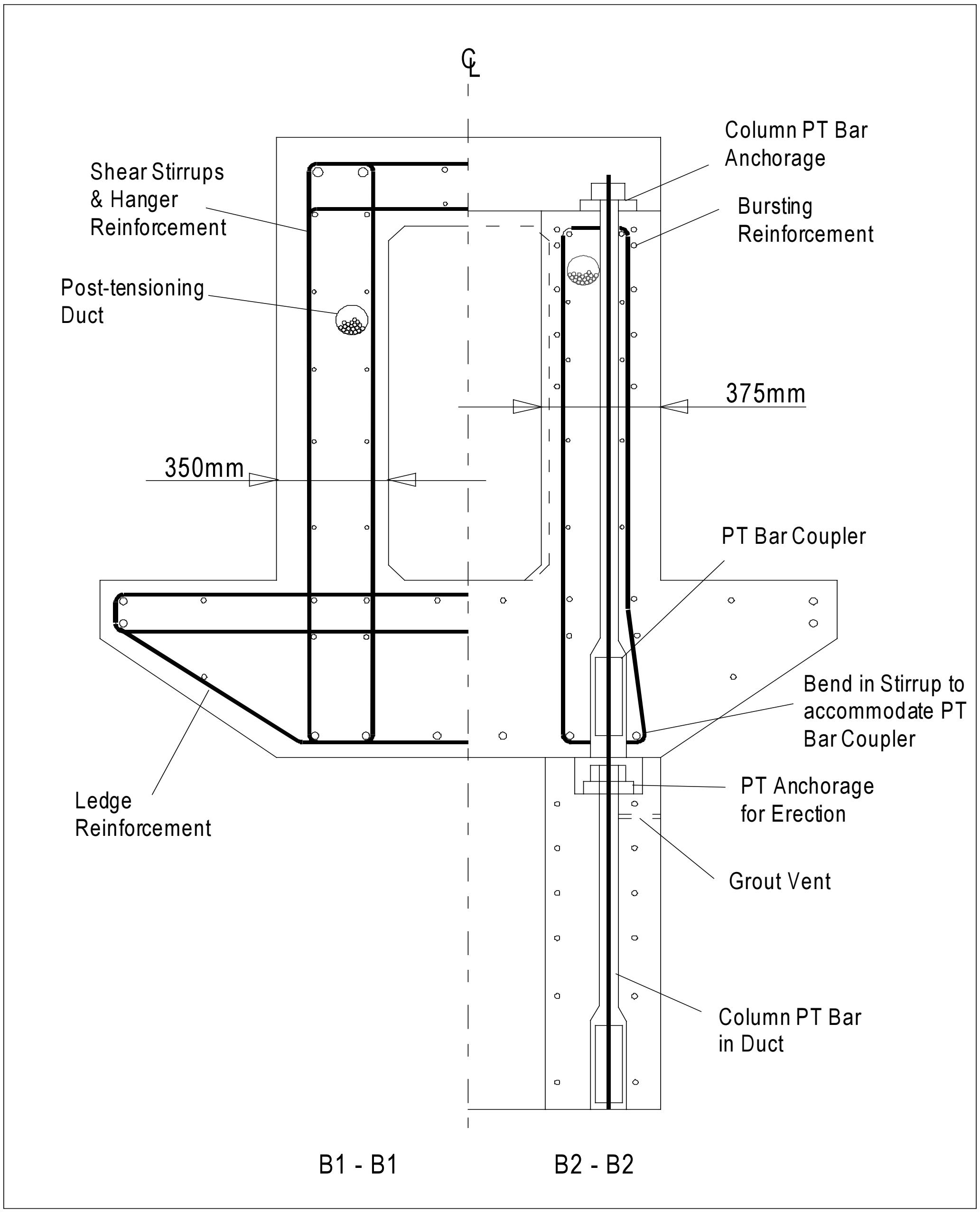


Figure 3.21c Cross-sections of cap and template for a fully post-tensioned cap

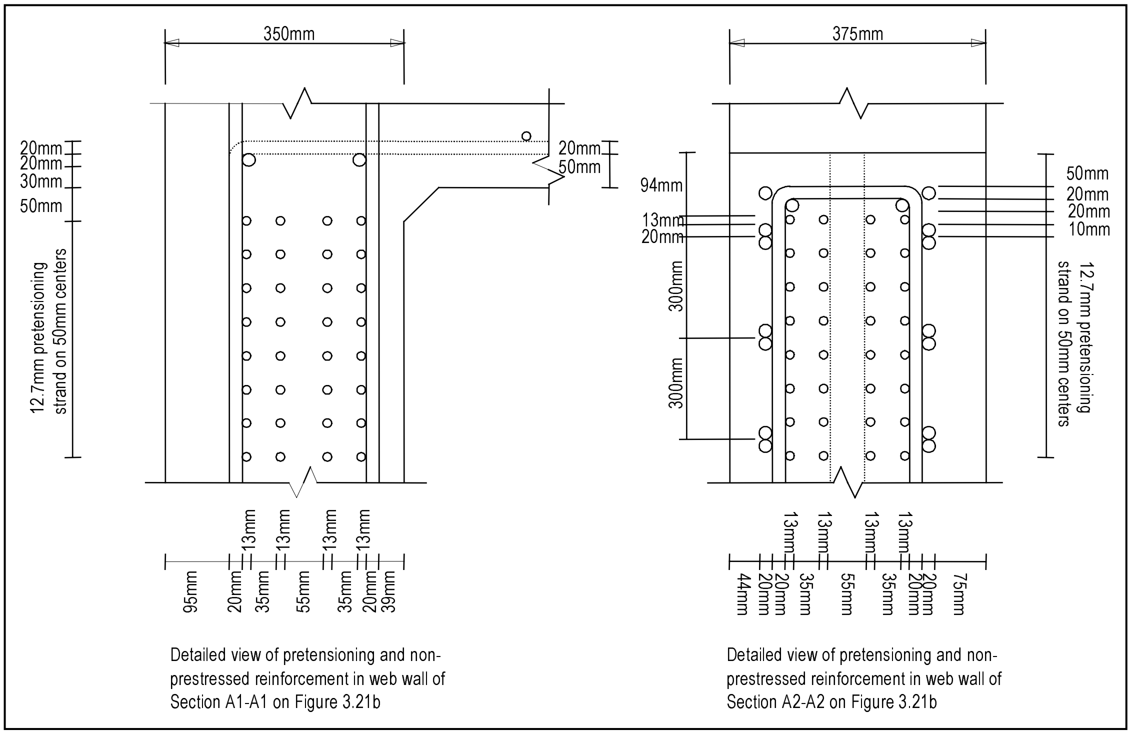


Figure 3.22a Spacing and cover details for a cap with only pretensioning strand for longitudinal reinforcement

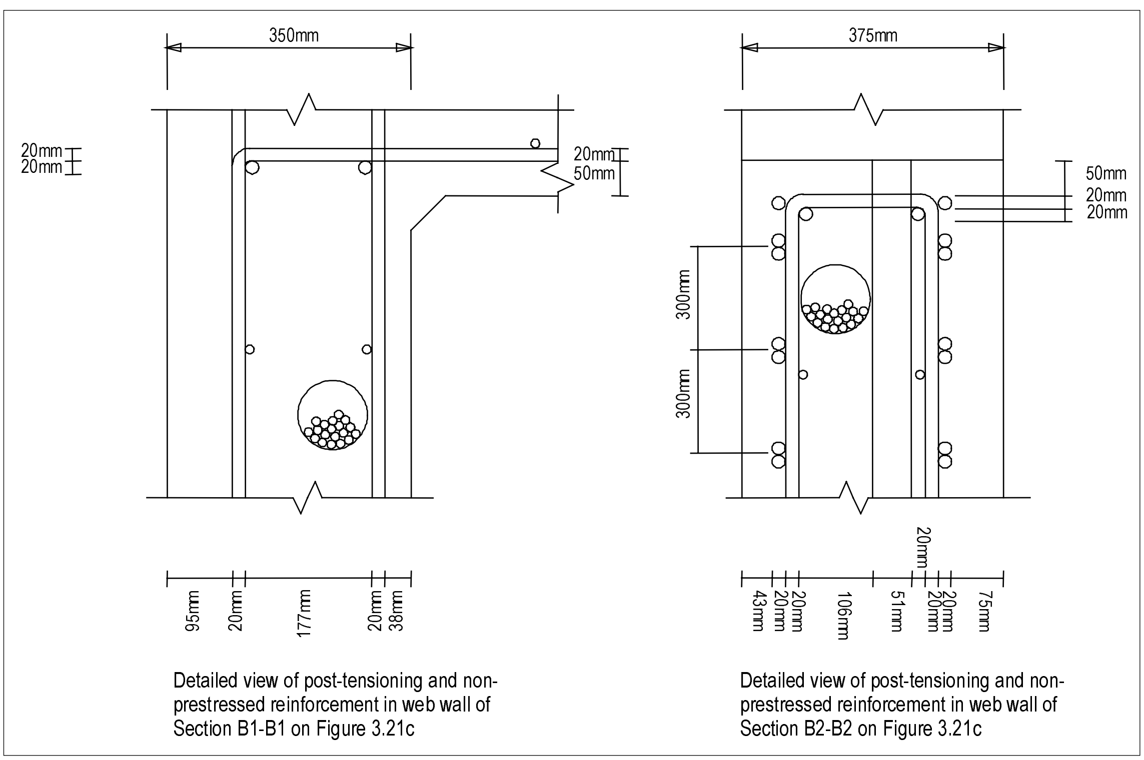
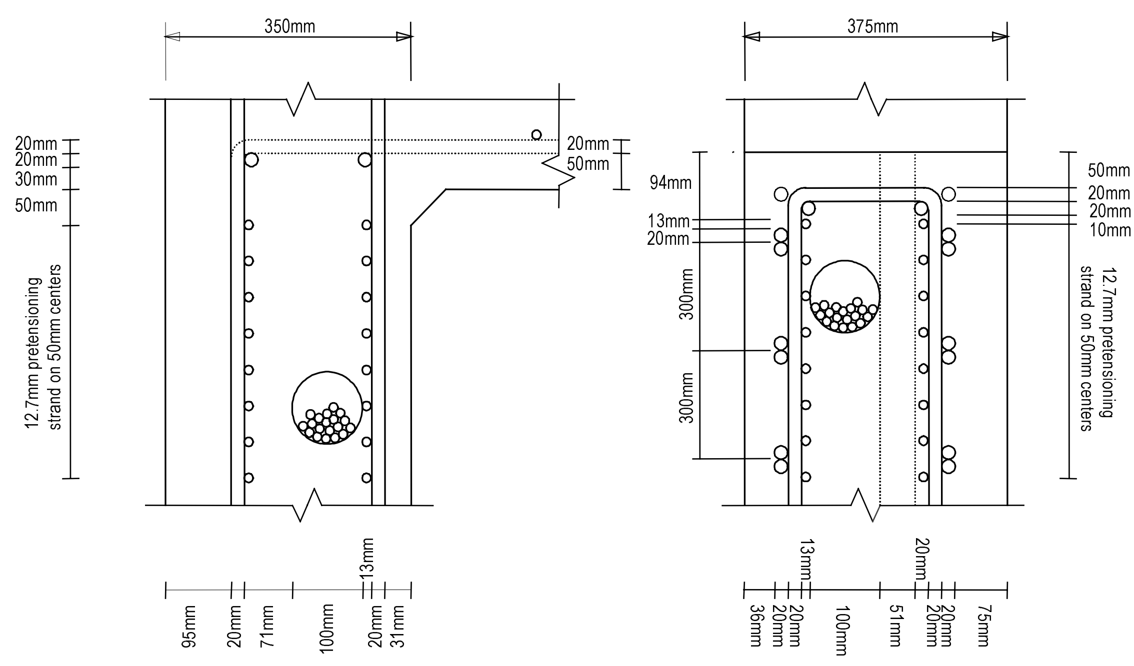


Figure 3.22b Spacing and cover details for a cap with only post-tensioning as longitudinal reinforcement



- A combination of pretensioning and post-tensioning will be a good alternative to fully post-tensioned caps for frame bents
- 19K6 tendons, shown here require the largest ducts permissible in these web walls. Should more interior cover be desired, the hollow portion of the cap could simply be reduced.

Figure 3.22c Spacing and cover details for a cap with pretensioned and post-tensioned longitudinal reinforcement

While the center of the cap stem would be hollow, solid portions are required at the ends for cap anchorage zones when longitudinal post-tensioning is used (Figure 3.21a). Anchorage zones are also required above the column for the column reinforcement not anchored in the webs of the stem. Table 3.3 gives a comparison of cap weights for one-, two- and three-lane bridges using single element caps from Proposals I and II.

Table 3.3 Single precast element bent cap weight comparison

Bridge Width	Cap Length* m (ft.)	Total Cap Weight, kN (kips)	
		Proposal I	Proposal II
1 lane (7.6m)	5.8 (19)	540 (121)	335 (75)
2 lanes (10.4m)	9.5 (31)	885 (200)	545 (123)
3 lanes (14m)	13.1 (43)	1230 (277)	750 (170)

*No skew

3.6.3 *Bent Configurations–Proposal II*

The four pier sizes, four corresponding templates and the single depth but variable length cap section can be combined in numerous ways to make up a wide range of substructure units extending from single-column bents to straddle bents and frame bents (Figure 3.23). These bents can support roadways up to 32 m (105 ft) wide and up to 18 m (59 ft) tall. A variety of skews may be accommodated as well. The inverted-T cap can support *AASHTO* Type IV Girders and 1370 mm (54 in) deep Texas U-beams. A similar cap with a shorter stem could be fabricated for use with shallower standard precast concrete girders. Shallower girders could also be supported on the deeper caps with built-up bearing seats. Deep caps supporting shallow girders should be used only when necessary, and aesthetic impact of such a detail should be carefully and completely considered. A more complete discussion of the treatment of shallow girders in combination with deep caps can be found in Section 5.2.2.3.

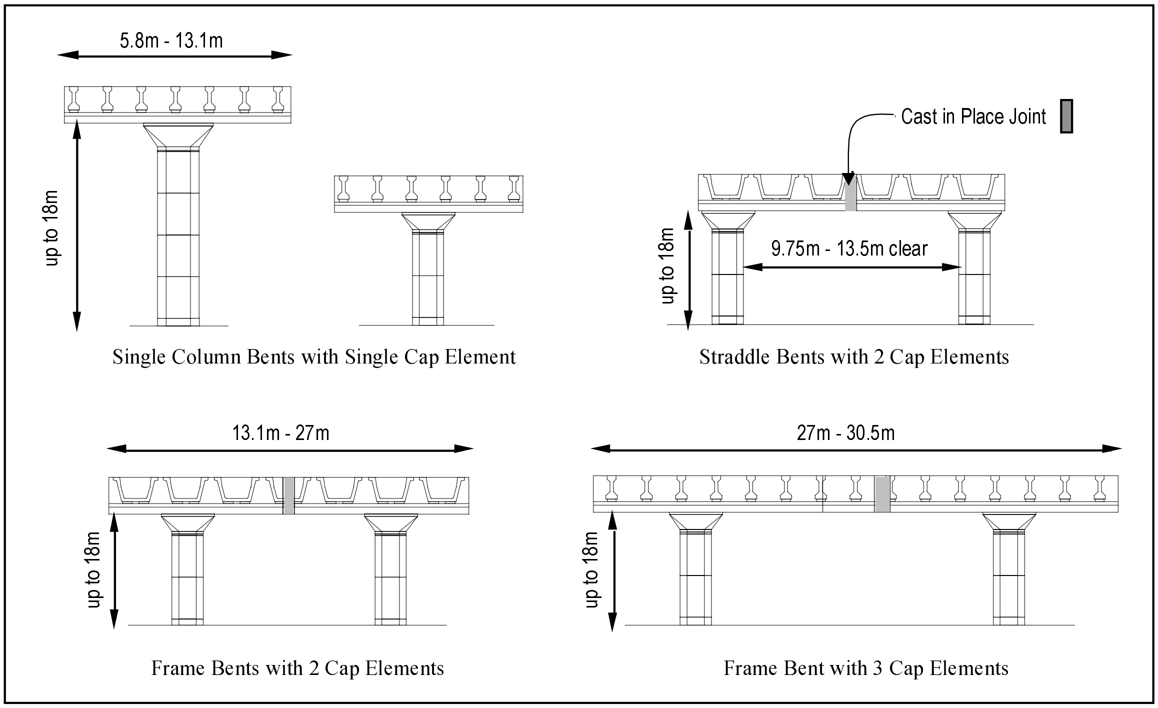


Figure 3.23 Substructure configurations for Proposal II

All bent caps may be fabricated in single elements up to 13.1 m (43 ft) long. Therefore all single-column bents are constructed with just one cap segment. Cap segments may be pretensioned or post-tensioned. Straddle and frame bents join two cap sections with a cast-in-place joint and longitudinal post-tensioning. These wider bents can accommodate caps up to 27 m (88 ft) in length with just two cap segments (two 13.1 m [43 ft] segments with an 800 mm [31 in] cast-in-place joint). Wider bents can be accommodated with two piers and two long segments and one match-cast shorter cap segment fabricated and erected as described in Sections 3.4.2 and 3.4.3. Alternatively, three piers and three caps could be utilized.

The pier segment chosen for a project will vary depending on project constraints. Roadway width, span lengths, road curvature, pier height and bent skews will all effect the pier size requirement. Other factors affecting the choice of pier size will be concrete strength and type of column reinforcement chosen. For instance, an 18m (59 ft high) single-column pier supporting a 14 m (46 ft wide) roadway could be constructed of P30 segments with a concrete strength of 56 MPa (8 ksi) or P24 segments with an 76 MPa (11 ksi) concrete. This choice is possible because the controlling factor for the pier design is the maximum compression stress resulting from column post-tensioning necessary to prevent tension across the segmental joints under biaxial bending in the service limit state.

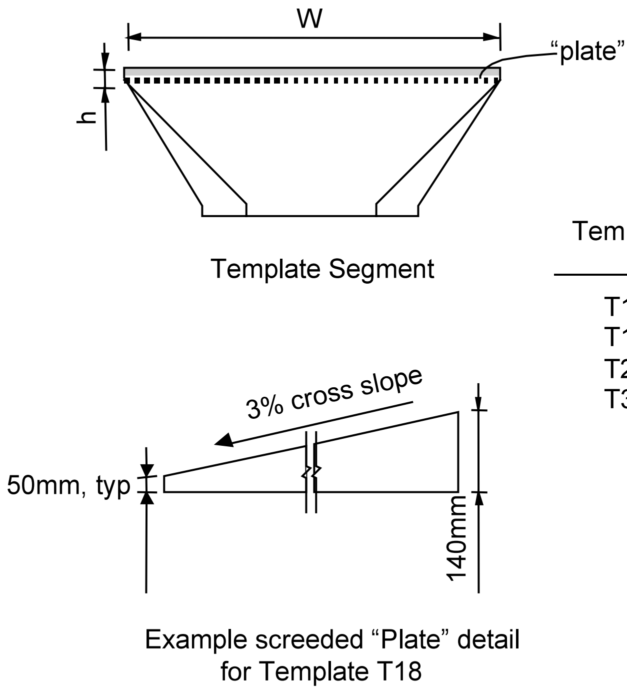
With the frame bents, the pier segment size should initially be chosen based on structural requirements. In many cases, the smallest column section may suffice for column loads. However with longer bents, a larger column section and therefore larger template section will reduce critical bending moment and shear forces in the cap. The designer must balance issues of economics and constructability when choosing between using larger column sections or more cap reinforcement. For aesthetic reasons as well, the designer may choose to use larger column sections for a long frame bent. Slender columns under a wide superstructure may appear visually weak and unsafe.

Design calculations and details for a single-column pier using P24 segments for a total clear height of 10 m (33 ft) supporting a 14 m (46 ft) roadway are shown in Appendix C of Reference 2. A combination of post-tensioned strand and bars are used for the column and the cap is designed to be pretensioned. Design calculations and details for a frame bent using P12 segments to support a 21.3 m (70 ft) roadway are given in Appendix D of Reference 2. In that example, only post-tensioning bars are used for the column while the cap segments are both pretensioned and post-tensioned. A discussion of pier design and detailing can be found in Sections 3.6.6 to 3.6.8.

3.6.4 Fabrication Sequence–Proposal II

Column segment fabrication would be the same as described in Section 3.4.2 and shown in Figure 3.10. Provisions for fasteners in the sides of the sections for bolting the upper segment forms to the existing segment should be the same ones used for attaching temporary devices for handling the segments. This dual use will minimize the number of holes or "disturbances" to the section.

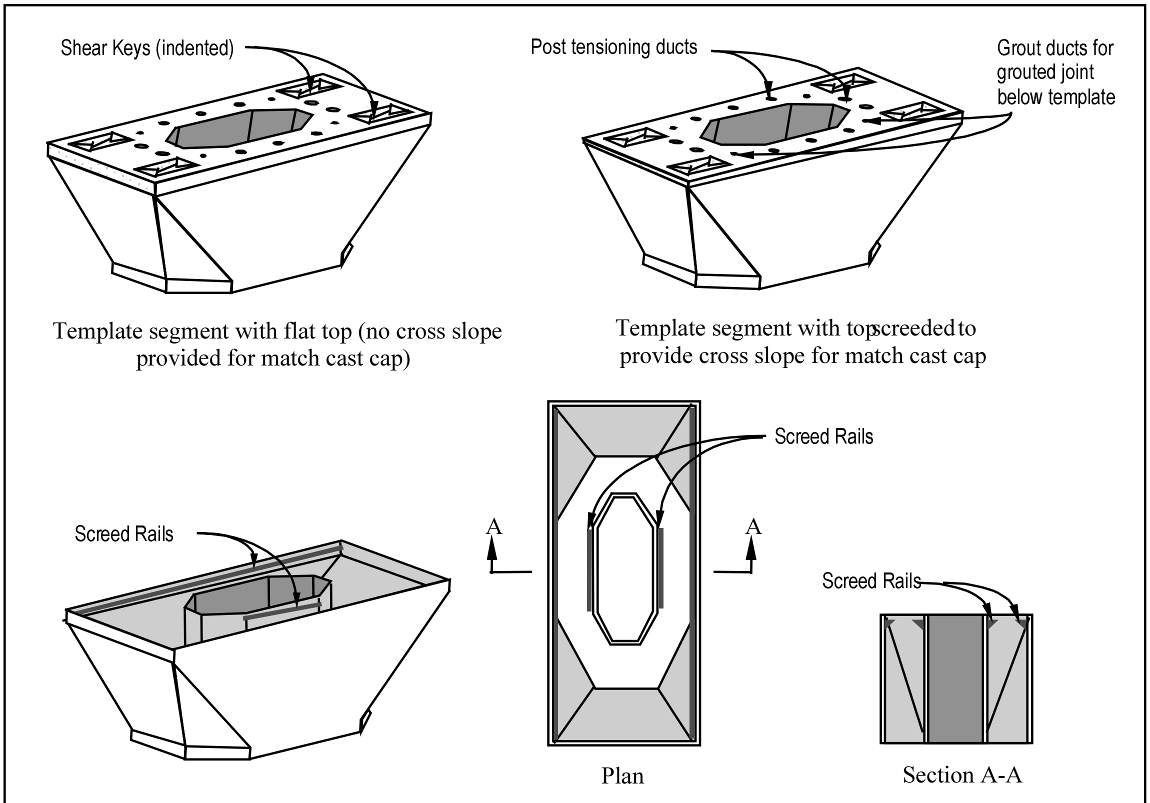
The template segments will be cast individually. The top portion of the template is essentially a quadrilateral plate. This area can be screeded to varying heights to provide necessary cross-slope for the cap. The overall plate height varies depending on the template size. This variation according to template size allows each template to readily provide up to a 3% cross-slope for the cap (Figure 3.24). The 3% value represents an arbitrary limit for the standard forms. It was chosen on the basis that about 90% of applications tend to have cross-slopes of 3% or less. The table in Figure 3.24 gives example plate heights for each template segment that can provide a 0–3% cross-slope. Required cross-slopes greater than 3% can be accommodated by developing other template designs or by using the 3% cross-slope in the template and then by varying girder bearing seat elevations for the remainder. A minimum height of 50 mm (2 in) is provided on the ends of the template "plate."



Template	Width, W mm (in)	Height, h mm (in)
T12	2000 (80 in)	110(4.25 in)
T18	3000(118 in)	140(5.5 in)
T24	4000(158 in)	170(6.75 in)
T30	5000(200 in)	200(8 in)

Figure 3.24 Example of a 3% cap cross-slope being provided for in a precast template

Screeding to the desired cross-slope can be facilitated in steel forms by providing a triangular chamfer strip, tack welded to the inside of the form. This screeding guide ("screed rail") should be welded to both the inside of the outer form and the outside of the inner core form (Figure 3.25). The rail on the outside form will create a chamfer at the site of the future cap-template joint. This chamfer should hide epoxy drippings in this awkward location which would be difficult to clean during erection. The screed rail on the inner core form would provide a ledge to support elements for forming the cap. When the cap is later match cast to the template, form surfaces are needed to prevent fresh concrete from entering the hollow portion of the template segment. The tack welded screeding rails could easily be removed (ground off) with negligible form damage for the next template segment if a different cross-slope is required. If wooden forms are used for either the inner or outer template form, wooden screed rails could be used. All inner core forms should be "drafted" (angled outward slightly as they rise to facilitate easy removal). An angle as small as 10mm over 2.4 m would suffice (0.5 in over 8 ft).



Formwork for template with screed rail tack welded to form at chosen cross slope angle

Figure 3.25 Template fabrication details

Ducts for post-tensioning bars must be cast into the template segment. A pattern sheet could be fabricated to ensure proper location and alignment of ducts. A pattern sheet would simply be a metal plate with the duct locations from adjacent column sections drilled into it. The pattern sheet is placed at the bottom of the template segment while ducts are placed to ensure that the ducts are properly aligned across this joint. Approximately four additional ducts of 40 mm (1.5 in) diameter must be cast into the template for placement of a flowable high strength grout to completely fill the joint between the top pier segment and the template segment. This joint detail is further discussed in Section 3.6.5 where the erection sequence of these piers is outlined.

Fabrication of the caps will be much more involved than fabrication of the column segments. There are a number of workable schemes for fabricating the caps with the necessary match-cast joint between the bottom of the cap and the top of the template segment. Some fabrication options are better suited for pretensioning, while others are better suited for post-tensioning.

For pretensioning, a fabrication system for the caps that can be easily adopted by existing pretensioning plants was developed. This system involves casting the cap in two-stages. In the first stage, the ledge of the cap would be match cast to the previously cast template. This operation could be done with the template supported from the ground and the ledge form supported above (Stage 1 in Figure 3.26a and Figure 3.26b). Web reinforcement would extend above the ledge. The casting operations would require concrete placement not more than 3.6 m (12 ft) the air. This imposes no problem on existing precast plant equipment. Once the ledge is cast and the concrete cured, the ledge would be transported to an existing pretensioning bed (Stage 2 in Figure 3.26a). With a number of ledges in the bed, pretensioning strands would be placed and forms for the inverted-T stems set. Post-tensioning ducts for the column and, if required, for the cap would be aligned and the concrete placed. For most of the single cap segments, pretensioning will suffice (no post-tensioning will be required) for the cap's longitudinal reinforcement. Only in cases with long caps that need to be stage tensioned will post-tensioning be required. In the case of frame caps, post-tensioning will be required to join the cap segments and provide positive moment reinforcement in the closure and span between the piers (Figure 3.23).

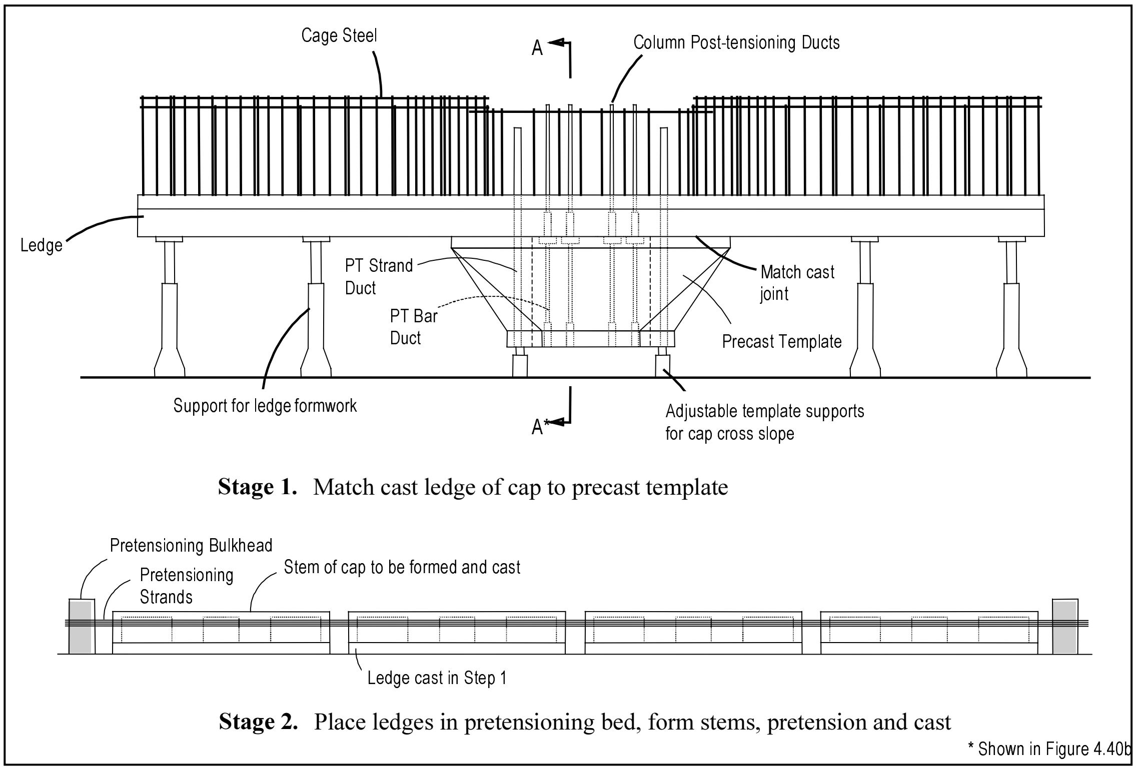


Figure 3.26a Two-stage match casting of precast, pretensioned inverted-T caps

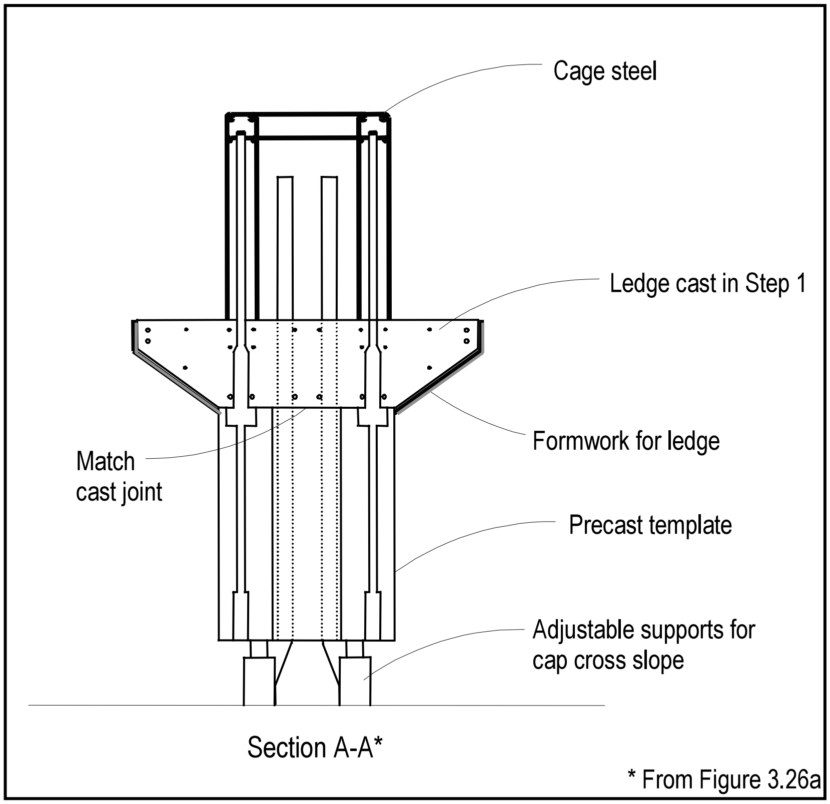


Figure 3. 26b Cross-section through the cap and template during Stage I from Figure 3.26a

The two-staged casting of the cap for this pretensioning scheme eliminates the need for self-stressing forms (forms with ends that essentially act as bulkheads) that will be 2–4 m (6.5–13 ft) in the air. Staged casting allows the precaster to use existing pretensioning beds and bulkheads. Staged casting also eases concrete placement avoiding problems due to trapped air under the ledge that could occur with a single, closed form (Figure 3.27). In single-stage casting, vibration of the ledge down through the walls of the stem would be extremely difficult with pretensioning strand running throughout. One possible solution would be to vibrate the ledges through the open top surfaces of the ledges, allow the concrete to develop an initial set for 30 to 45 minutes and then place the web concrete. With the staged casting, concrete shear keys, if needed, may be set into the ledge for shear transfer in the concrete for the final structure. Web hanger reinforcement for shear will also need to extend upward or be mechanically spliced at this location. As shown in Figure 3.26a, the stirrups required for the cap would be cast into the ledge. This detail is good, providing unspliced stirrups and also a cage with which to lift the ledge into a pretensioning bed. Consideration of the different concrete stiffnesses in the ledge and stem at release of the tendons will need to be taken into account to properly assess prestress losses due to elastic shortening (see pages C37–C40 of Appendix C in Reference 2 for example calculations).

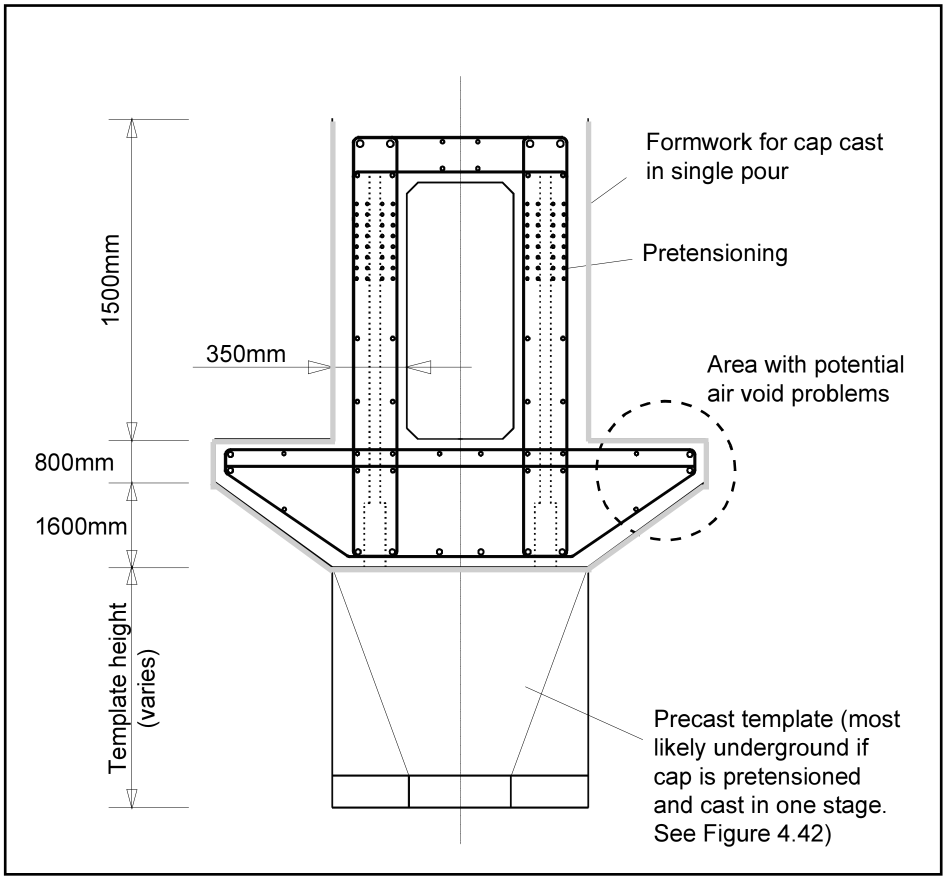


Figure 3.27 Cross-section through a cantilever of a pretensioned cap formed for a single-stage casting

Another possible method of cap construction that eliminates the need for self-stressing forms 2–4 m (6.5–13 ft) in the air would be to insert the template segment in an opening below grade and match cast the cap to the template while the cap is supported at grade level (Figure 3.28). This would require the precaster to depress the template up to 2.5 m (8 ft) below grade (for the largest template, T30). Casting could again be staged with the ledge being cast first and then placed in a pretensioning bed. Other fabrication options include match casting the cap in one stage above the template in a self-stressing form. Another option for the fabricator would be to cast a single cap in one stage resting at grade level in self-stressing forms or in a short pretensioning bed with the template inserted in an opening in the ground to facilitate the required match-cast joint.

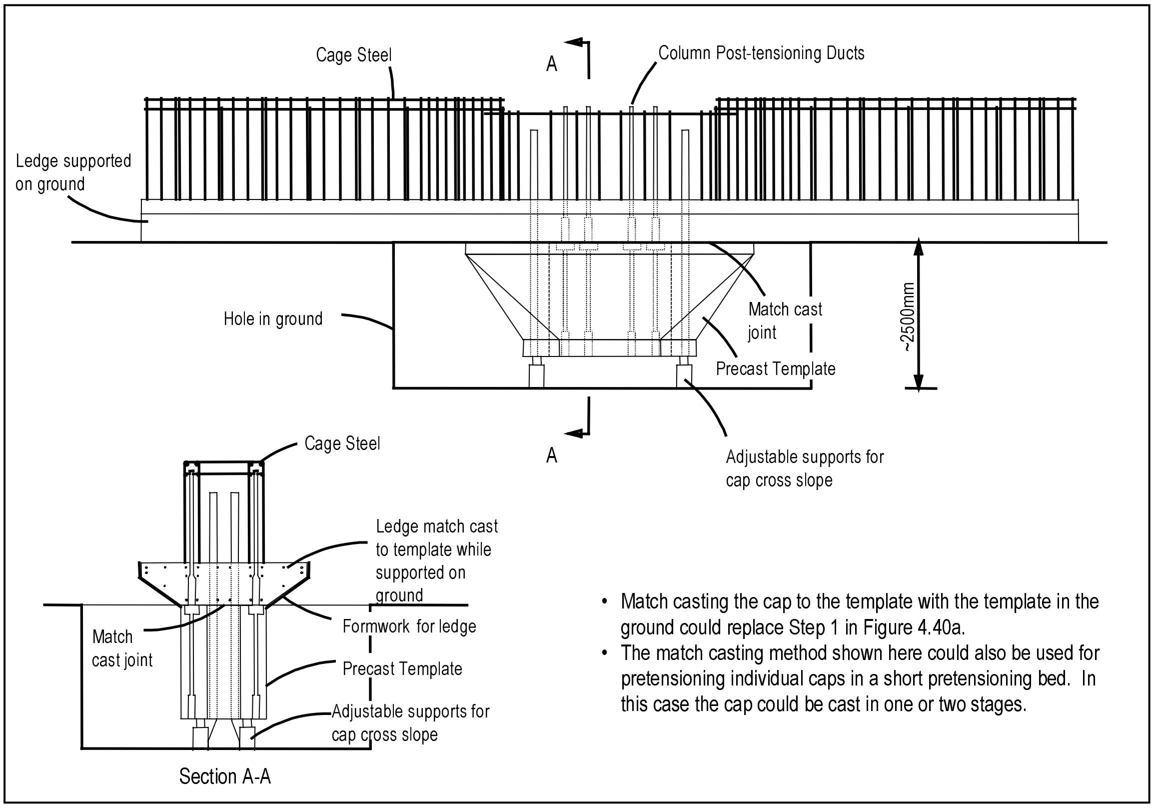


Figure 3.28 Two-stage match casting of precast caps with precast template in ground

An advantage to fabrication in a precasting plant as opposed to field concreting is that the quality of high performance concrete can be better controlled. Use of high performance concrete with its high early strength allows for faster turnover of pretensioned elements. Such higher turnover helps make element fabrication economical. Pretensioning the elements will also reduce the amount of nonprestressed reinforcement typically required to control cracking during handling of the elements to be post-tensioned onsite.

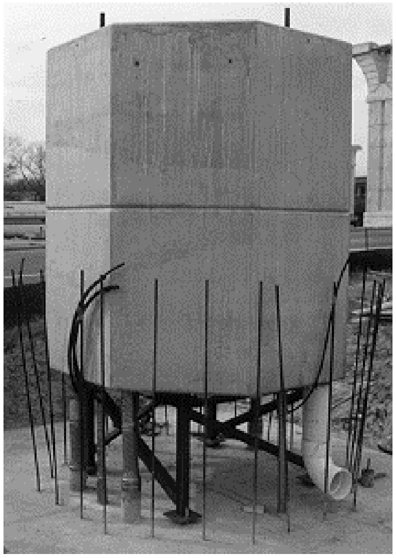
Fabrication of precast caps that will be entirely post-tensioned can be performed in a precasting plant or in a temporary precasting yard. Again, the template segment can rest on the ground with the cap formed and concrete placed 2–4 m (6.5–13 ft) in the air or the template can be placed in a recess below grade and the cap cast above while supported at grade level. The entire cap could be formed for one cast in either situation since self-stressing forms or bulkheads for pretensioning would not be used. However, as previously discussed, two-stage casting is recommended for constructability. Post-tensioning ducts running throughout the walls of the stem will add to the difficulty of vibrating the ledge concrete if placed in a single cast.

3.6.5 Erection Sequence–Proposal II

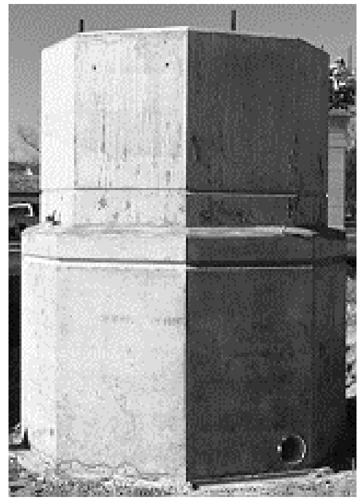
The erection sequence for the column segments for Proposal II is similar to that of Proposal I. Normal foundation construction is carried out with provisions for column post-tensioning bars and/or strands in the foundation cap, or footing. Column post-tensioning bars are anchored in the cast-in-place footing. Post-tensioning strands require ducts curving 180 degrees. The curving ducts in the footing facilitate the threading of the strands from the top of the cap down through the footing and back to the top of the cap once the cap is placed. Such a duct layout allows all final column post-tensioning operations to be performed at the top of the cap with no disruption to the completed foundations.

The first column segment is placed on adjustable supports above the previously cast footing. Where desirable for geometry flexibility with standard height segments, the footing could be designed to have a recess in which to place the first segment. With a recess however, the overall footing (or pile cap) depth may be increased. Deeper footings will be uneconomical in many locations in Texas where rock is located just below the surface. Increased excavation costs will often dictate using footings and pile caps that are as shallow as possible. For most situations, the first pier segment will therefore be placed above the footing.

The first segment is aligned on adjustable supports, such as the simple steel frame adjusted with screw threads or shims as described in Section 3.4.3. An example of this method used recently in Austin, Texas is shown in Figure 3.29a. Column post-tensioning ducts are spliced to the corresponding post-tensioning ducts in the footing, internal drainpipes are placed and joint reinforcement is tied. This first segment is then locked into position with a cast-in-place joint (Figure 3.29b). The first joint should be a concrete of similar quality to the pier segments. This joint may vary in height from 300 to 600 mm (12 to 24 in) depending on the required height of the pier. A number of first segments may be placed and aligned for a project before they are "locked" into position with these cast-in-place joints. Due to the relatively small amount of concrete needed to set each segment (less than 2.3 m^3 (3 yd^3) for the largest segment), placing concrete for more than one column at a time will be more economical and less time-consuming in terms of disruptions to the site due to field concreting.



(a) Placement of first pier segment on adjustable supports. Segment aligned, PT ducts spliced, joint reinforcement tied, internal drain pipes installed



(b) Base joint cast in place

(c) PT bars coupled, placement of subsequent segments with epoxy applied to adjoining segment faces

(d) New segment post-tensioned down to previously erected segments

Figure 3.29 Erection sequence for the precast piers at US 183 in Austin, Texas

With the first segment set, the next pier segment can be lowered into place. Before the segment is set down, PT bars are coupled, and then epoxy is placed on the faces of adjoining segments (Figure 3.29c). The segment is then lowered into position. The match-cast joints with aligning shear keys allow for rapid placement. With proper alignment control carried out during the match-casting process, no further alignment changes should be needed in the field. The newly placed segment must then be post-tensioned to the existing pier (Figure 3.29d) to provide a surface pressure of 0.28 MPa (40 psi). (The 0.28 MPa, 40 psi requirement is part of the *1994 Interim AASHTO Segmental Specifications*. This requirement may be lowered to 0.07 MPa [10 psi] in future revisions. [46] This will mean that a 3 m [10 ft] segment of any cross-section will itself provide the necessary 0.07 MPa [10 psi] pressure and temporary post-tensioning will not be required.) For shorter segments, a second segment could be placed above to apply this pressure while the epoxy sets. Avoiding the post-tensioning operations for each segment placement in the future should decrease labor costs considerably and increase construction speed.

With the final pier segment in position, the template segment can then be placed. The template is set on adjustable supports, PT ducts are spliced and the segment is aligned to provide the proper cross-slope for the match-cast cap. This joint can be very small (75–100 mm [3–4 in]) and will be filled with a durable high strength epoxy grout. Grouting the joint will typically be more economical than placing such a small amount of concrete at heights up to 18m (59 ft). A bracing system bolted to the template and the top pier segment from their inner cores can be used to hold the template in place once the proper alignment is achieved (Figure 3.30). The joint is then formed and a flowable high strength grout is placed through ducts in the template segment. After the joint has cured, the template segment can be post-tensioned to the pier, and the temporary bracing can be removed.

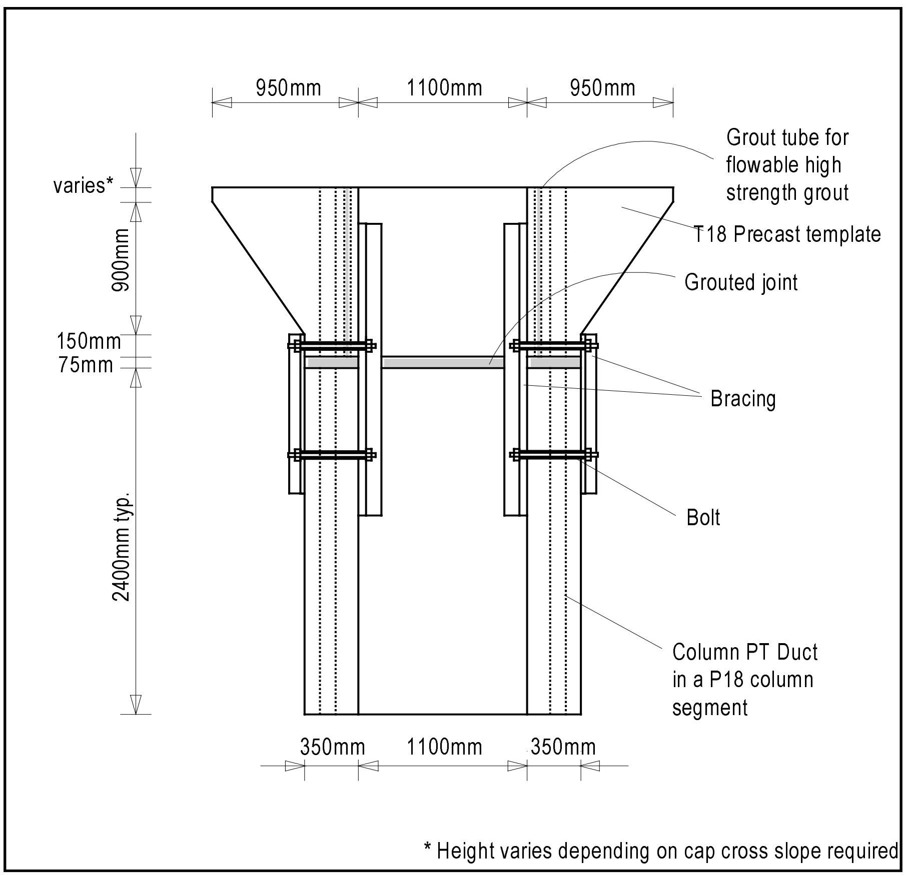


Figure 3.30 Cross-section through a P18 pier showing grouting of the template to the top column segment

The cap is placed next. Epoxy is applied to the bottom of the cap ledge and the top of the template segment. The cap is then easily set on top of the template to which it has been previously match cast. No special alignment procedures are necessary for this heavy element. It is simply set in place, self-aligned due to the match-cast shear keys and is vertically post-tensioned to the pier (Figure 3.31). The recess in the cap for the vertical post-tensioning anchor plates can then be filled with a highly durable concrete. Any required longitudinal post-tensioning of the cap is performed next. In some cases, staged post-tensioning of the cap may be necessary. This post-tensioning would be sequenced in accordance with placement of the superstructure girders.

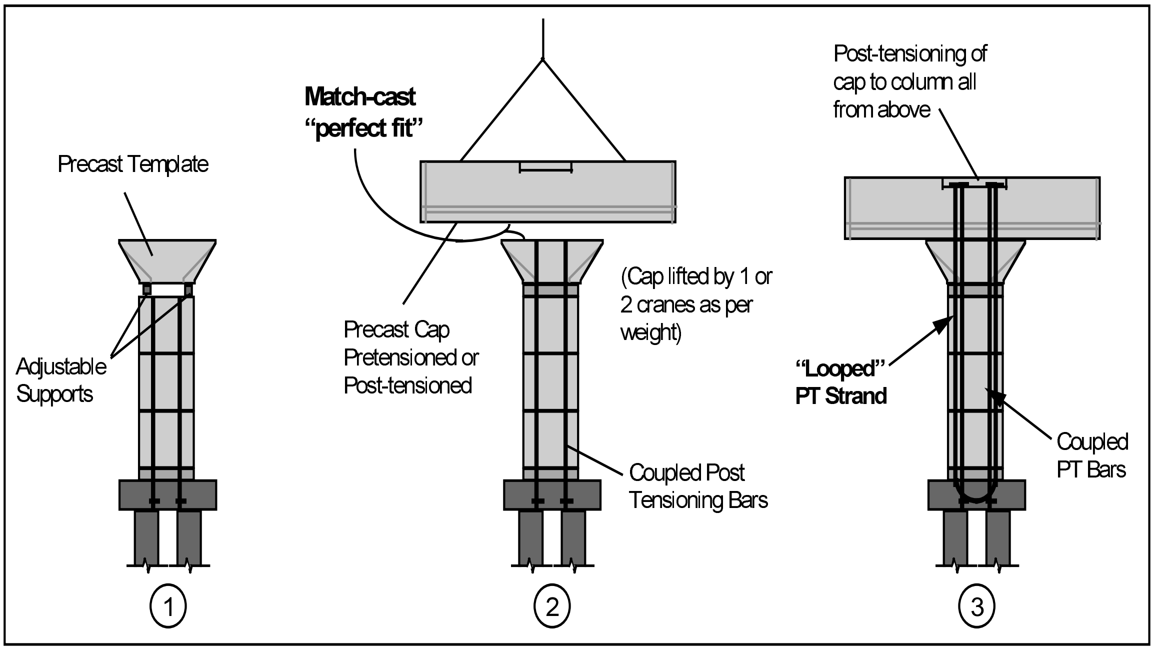


Figure 3.31 Erection sequence for a hammerhead bent with a single cap element

The erection process for frame bents less than 27 m (88 ft) wide made up of two cap segments is shown in Figure 3.32. Pier erection is the same as that for single-column bents. Each match-cast cap segment is then vertically post-tensioned to a pier. Any additional segments required between the cap segments for the frame bents wider than 27 m (88 ft) would be added in the manner described in Section 3.4.3. The additional segments are longitudinally post-tensioned to the first cap segments that were vertically post-tensioned to the piers. The remaining joint between the cap segments would then be formed, post-tensioning ducts spliced, and nonprestressed reinforcement tied. Cast-in-place concrete is then placed in the joint. Once the joint concrete has cured, the entire cap can be post-tensioned, thus providing positive moment reinforcement at the mid-span of the bent. Again, staged post-tensioning may be required during placement of the superstructure.

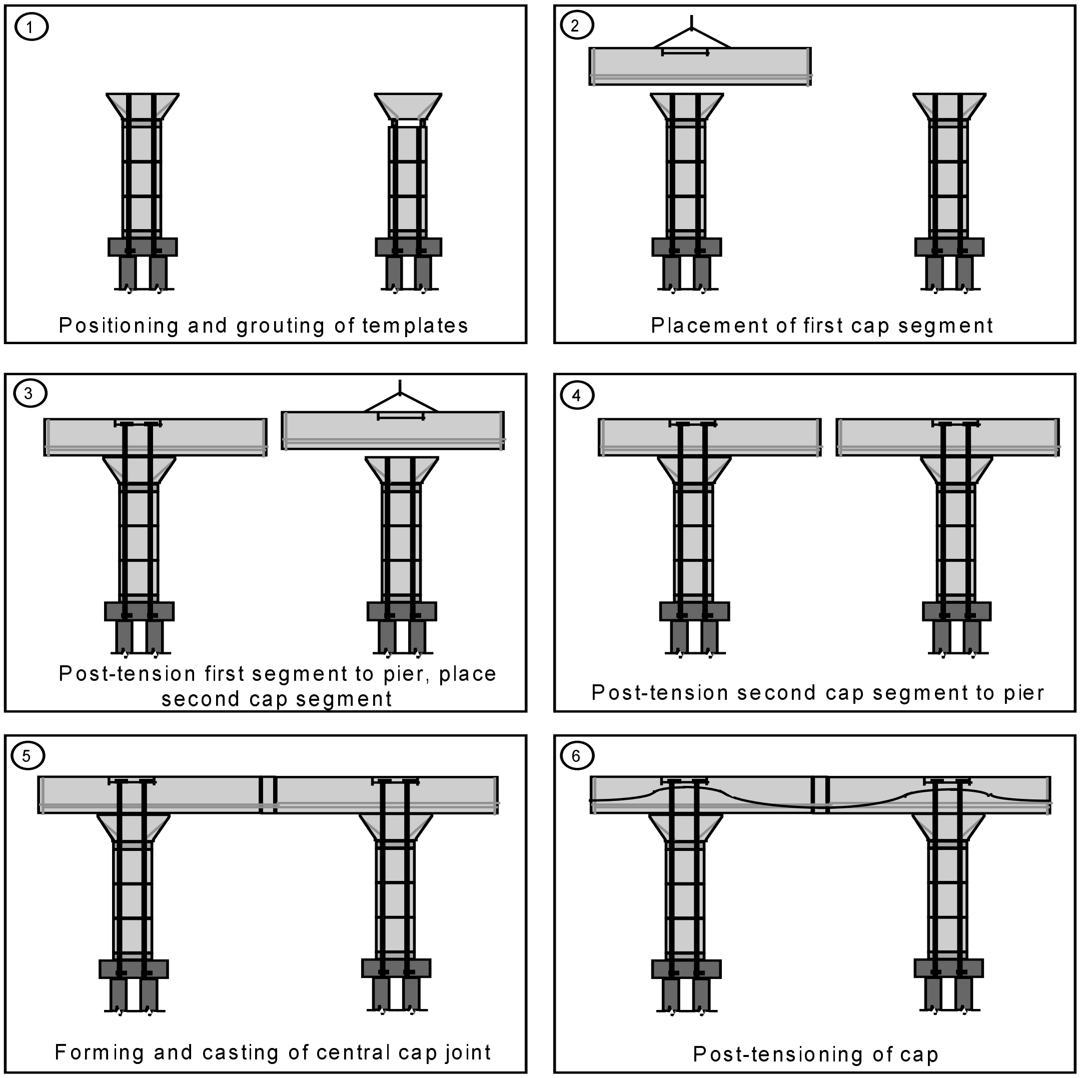


Figure 3.32 Erection sequence for a frame bent

For the frame bent caps and the single-column bent caps requiring post-tensioning, an additional cast-in-place cover could be added to the ends of the cap to cover the post-tensioned anchor plates. This addition can be attractively chamfered as shown in Figure 3.36b. A chamfered end minimizes the visual disruption to the profile of the bridge that blunt bent cap ends often create. The chamfers will also integrate the cap more attractively with skewed layouts (in a fashion similar to the octagonal columns proposed for the bents in Figure 4.39).

3.6.6 Precast Substructure Design Process–Proposal II

This section outlines the general procedure for designing a precast substructure bent. Two specific examples with detailed computations are given in Appendices C and D of Reference 2. Specific calculations and design equations mentioned in this section will be referenced to the designs in these Appendices.

The loading on a precast substructure bent should be in accordance with the prevailing code or specifications used by the engineer. In addition to satisfying serviceability and strength under service and ultimate loads respectively, any pertinent serviceability or strength requirements must be satisfied under construction loads. Construction stress limits may be critical in pier design due to the unbalanced moments imposed while girders are being placed.

Variations in the design forces will dictate the specific pier section and corresponding template size to be used. With experience, the designer will have a feel for what pier size is required for certain pier configurations, pier heights and roadway widths. The larger pier sections will generally be required for the single-column and straddle bents where column moments can be considerable. Frame bents typically experience less bending and often allow use of the smaller pier sections.

While the current practice at TxDOT is to design all multicolumn bent caps as if pin-connected to the bent columns, multicolumn (in this case two-column) precast bents should be analyzed as frames with moment connections between the cap and columns. In particular, secondary moments caused by the longitudinal post-tensioning of frame bent caps must be accounted for in design.

Slenderness effects must be considered in the pier analysis. A second order analysis that accounts for the effects of prestressing will be the most desirable solution. Approximate methods such as the moment magnifier method and the P-delta method as outlined in *AASHTO Standard* and *LRFD Specifications* can be used as well. These methods were not specifically developed for use with prestressed columns and therefore some modifications should be made when using them. The design examples in Appendices C and D of Reference 2 were based on column moments amplified using the moment magnifier method (pages C10 and D10). As the service limit state dictates no cracking of prestressed members under service loads, uncracked sections were assumed when magnifying service load moments. Therefore the only modification to the stiffness of the section at service load levels is for creep effects (β_d). When magnifying column moments under ultimate loads, a reduced stiffness which accounts for cracking, creep and shrinkage effects as specified in the *AASHTO Standard* or *LRFD Specifications* should be used. This will be conservative as it neglects the enhanced stiffening effect due to prestressing in the columns. The moment magnifier method is a conservative approximate method. It should be replaced by a refined second-order analysis when slenderness effects dominate. In such a second-order analysis the beneficial effects of prestressing can be considered. Slenderness effects will usually be more critical in the bridge longitudinal direction (weak axis bending of the column) than in the transverse direction (strong axis bending of the column) for the proposed precast substructure system if the superstructure is simply supported. With a continuous superstructure, slenderness effects for longitudinal bending should not be critical.

After the determination of critical load effects (axial, shear and moment) is completed, a final pier size should be selected. Amplified moments may have to be revised for the changed size and an iterative procedure utilized. Segmental pier design will generally be controlled by service load conditions. The column section and prestressed reinforcement are selected based on satisfying the zero tensile stress limit and the maximum service compression load stress specified in the *AASHTO* codes. Column design also will be an iterative process; selecting a section size, determining the amount of reinforcement required, and then determining the required concrete compressive strength. The ultimate capacity of the columns must then be checked by constructing an interaction diagram. The interaction diagram for a prestressed column is constructed similarly to that of a nonprestressed reinforced column by determining the failure envelope of axial and flexural load combinations through a strain compatibility analysis. One important difference with prestressed columns is that both the steel and concrete have initial strains and stresses due to the prestressing that must be accounted for in analysis. It should not be assumed that the ultimate strength of the prestressing steel will be developed at ultimate load conditions. Example calculations for an interaction diagram for a prestressed column are shown on pages C12 and D12 of Reference 2 as well as in Reference 47.

Main flexural reinforcement for the cap can be handled with pretensioned and/or post-tensioned steel combined with nonprestressed reinforcement for the ledges. Pretensioning will reduce the amount of nonprestressed reinforcement required in the cap for handling. In the case of straddle and frame bents, some post-tensioned reinforcement will be required to provide positive moment reinforcement between the supports and tie the cap segments together. Secondary moment effects due to post-tensioning in the frame and straddle bents (essentially rigid frames) must be considered. Examples of the design of cap flexural reinforcement are given in Appendices C and D of Reference 2 beginning on pages C22 and D18.

Choosing an appropriate tendon layout for frame bents in particular will be an iterative process. Stress limitations must be satisfied during construction as well as at service limit state. A controlling factor may often be that no tensile stress is permitted across a joint that has no bonded nonprestressed reinforcement passing through it. This case will often occur at the cast-in-place closure joint. There are a number of design options to handle stress limitations that are critical at this joint and when a cast-in-place closure at the center alone is not satisfactory. One option is to specify a field weld splice between nonprestressed bars that are cast extending from the end of the cap (Figure 3.33a). Precasting will provide suitable tolerance levels for the location of the nonprestressed reinforcement extending from the cap ends to allow for field welding. Another option for handling stresses across the cast-in-place joint is to move the joint away from the most critical load area (Figure 3.33b). However, moving the joint from the center will require cap elements of different lengths. The unbalanced load condition (particularly of the longer cap element) during construction may control the design of the column. The two cantilevers making up the central span of the frame bent would have to be intentionally adjusted during match casting to provide appropriate cross-slope for the deck. This becomes increasingly complex if the cantilevers have considerably different deflections due to unbalanced loads. However, in general, the caps are not very flexible. For example, in the bent shown in Appendix D of Reference 2, moving the location of the joint by one meter removed the need for staged post-tensioning during erection yet resulted in only a half-inch difference in deflection. A third option shown in Figure 3.33c is to use temporary post-tensioning bars across the top of the cap between the two cap recesses that accommodate the column post-tensioning anchorages. This option will be most useful when calculated stresses during construction do not meet stress limitations. This may occur when the stresses at midspan of the cap caused by the post-tensioning are significantly greater than the offsetting stresses from the dead load of the girders. Temporary PT bars across the top of the cap may allow for full post-tensioning initially, rather than requiring a more time-consuming staged post-tensioning scheme. Depending on the tendon layout chosen, the provisional PT bars across the top of the cap could remain in place rather than be temporary during erection. The size and number of these PT bars will dictate the necessary depth of the recess above the columns to accommodate post-tensioning operations for these bars (which will occur after the column post-tensioning is completed) and to provide them with adequate cover if they are to remain in place permanently.

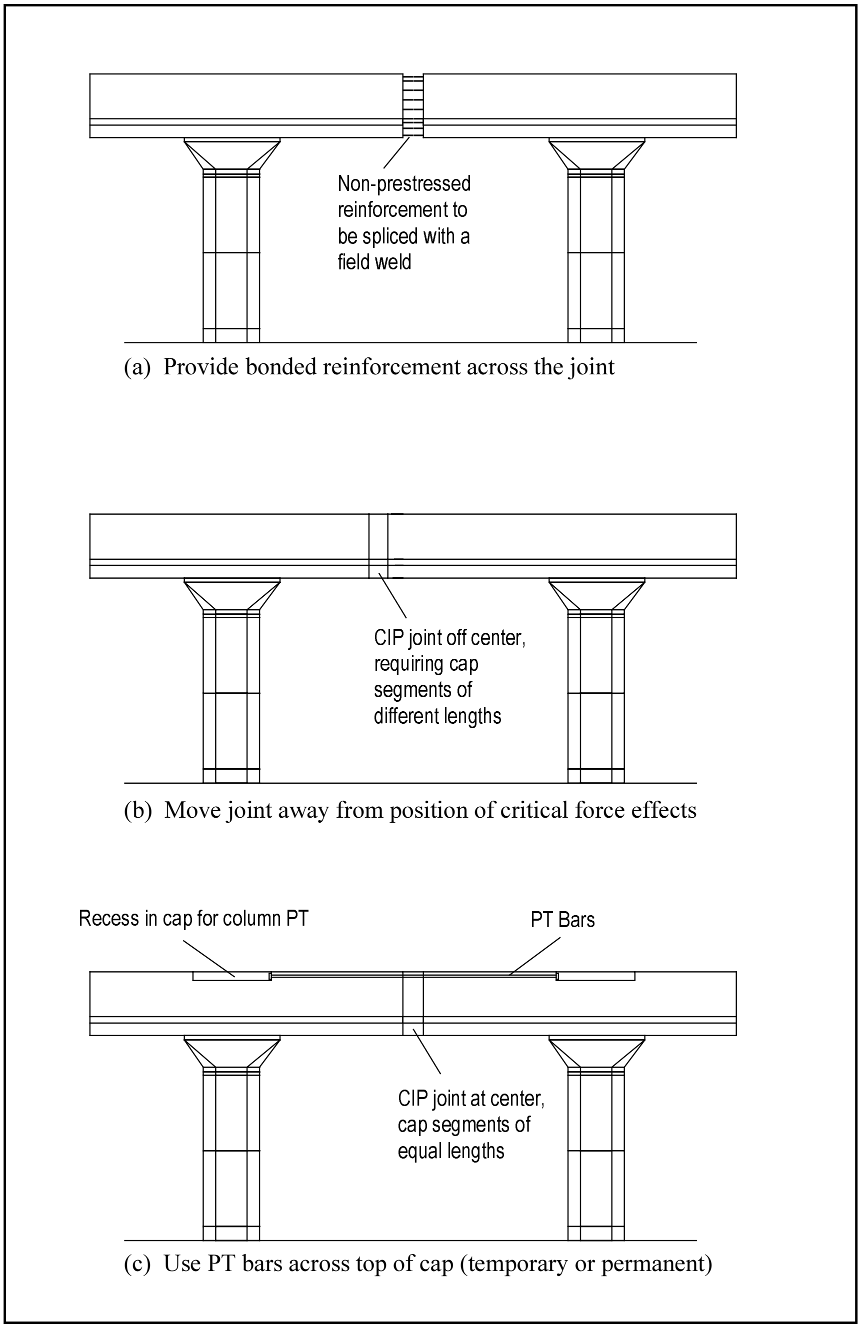


Figure 3.33 Alternate connection options for frame bents

Once the main cap flexural reinforcement is designed, the caps must then be detailed to transfer the girder loads adequately from the ledge to the stem of the inverted-T cap. The strut-and-tie method is an efficient method for designing the cap ledge and hanger reinforcement (page C28a of Reference 2). The traditional method used by TxDOT for inverted-T ledge and hanger reinforcement was developed for solid inverted-Ts with unchamfered ledges. [42] This method may be used for the proposed cap with a modification for determining punching shear resistance (pages C28b and D32 of Reference 2). The distribution requirements of the reinforcement for the strut-and-tie method will be more efficient, but the amount of required reinforcement for the two methods should be similar. The hanger reinforcement shear capacity check as per Reference 42 does not account for shear resistance provided by inclined longitudinal prestressing.

The shear capacity of the cap must be analyzed next. Shear-torsion behavior must be checked as well (pages C31 and D35 of Reference 2). Shear forces should be resisted mostly by closed stirrups. Draped post-tensioning can also aid in resisting shear forces (V_p) and the flexural reinforcement layout may be altered to better resist shear where desired. Closed stirrups should be used in each of the webs of the stem. This detail will provide both shear resistance and a support for interior side-face reinforcement for crack control. Shear reinforcement requirements must then be compared to hanger reinforcement requirements. Hanger reinforcement must be supplemented with additional stirrups where shear resistance requirements are not fully satisfied (pages C35 and D41 of Reference 2).

The ledge, hanger, and shear reinforcement may all be designed by the strut-and-tie method as outlined in Section 5.6.3 of the *LRFD AASHTO Specification*. This design method should lead to more economical design and better understanding of the flow of forces than traditional *AASHTO* methods. However, codification of strut and tie modeling is often unclear and difficult at times to apply, particularly when a large number of ultimate load cases must be checked. With more design examples and text books covering this method and with further clarification as to its advantages and limitations, the strut and tie method may become the design method of choice for these rather deep inverted-T caps. If traditional *AASHTO* methods are used, strut-and-tie models are recommended for an initial understanding of the flow of forces. Then, the traditional methods should be used carefully and will usually provide conservative detailing for familiar applications.

3.6.7 Precast Substructure Design Details

A number of separate detailing considerations are presented below:

(a) Detailing of the hollow columns should be in accordance with recommendations developed by Taylor, Rowell and Breen at the University of Texas at Austin. [48] Recommended detailing includes:

1. Two layers of longitudinal reinforcement should be provided in each pier wall, one layer near each face of the wall.
2. Maximum lateral spacing of longitudinal reinforcement should be limited to 1.5 times the wall thickness or 450 mm (18 in), whichever is smaller.
3. Maximum longitudinal spacing of transverse bars should be limited to 1.25 times the wall thickness or 300 mm (12 in), whichever is smaller.
4. Cross-ties between layers of reinforcement are recommended at maximum longitudinal and lateral spacing of 600 mm (24 in). Cross-ties should be alternated in a "checkerboard" pattern, connecting points where lateral and

longitudinal bars intersect. This reinforcement prevents buckling of longitudinal bars. Additional cross-ties are recommended at the top and bottom of each segment.

5. Lap splicing of transverse bars should be avoided, if possible. Otherwise, lap splices should be enclosed by the hooks of cross-ties.
6. Corner regions of segments should be well confined in order to enhance performance under biaxial bending.
7. Post-tensioning ducts should be grouted in order to promote integral action between post-tensioning bars and the concrete section.
8. A minimum of 1% longitudinal nonprestressed reinforcement should be provided.

The last recommendation listed above is primarily aimed at reducing the effects of creep and shrinkage in these vertical compression members. A recent study at the University of Texas in Austin has shown this minimum requirement to be appropriate for nonprestressed columns. [49] However, it may be overly conservative for prestressed columns. With the use of HPC (as proposed for the precast substructure system developed herein), concrete stiffness is enhanced. Also, it must be recognized that nonprestressed reinforcement in precast segmental substructures is not continuous across the segmental joints. Thus there are discrete regions with 0% nonprestressed reinforcement. While the nonprestressed reinforcement will be locally stressed under load, it is not required nor depended upon to carry load. A minimum amount of nonprestressed reinforcement is useful for shrinkage and temperature effects. Creep will have a more significant effect on column post-tensioning through loss of prestress. A minimum percent much less than 1% will most probably suffice.

(b) The portion of the cap above the column post-tensioning must be recessed to allow for post-tensioning operations and to provide adequate cover for the post-tensioned anchors once the recess is filled. A recess of 250 mm (10 in) should typically suffice.

(c) All shear reinforcement should be provided for in the initial stage of the two-stage cap fabrication. Full stirrups (each consisting of one continuous bar) should be cast into the ledge. The ledge may then be lifted by the cage to be placed into a pretensioning bed for casting of the stem (see Section 3.6.4). Casting the full stirrups in the first stage avoids the need for spliced hanger reinforcement.

(d) Anchorage zone detailing for the post-tensioned anchor zones is most easily handled using the strut-and-tie method as outlined in Section 5.10.9 of the *LRFD AASHTO* code (see example calculations beginning on pages C41 and D45 of Reference 2). Pretensioned anchor zones must be detailed as well as outlined in Section 5.10.10 in *LRFD AASHTO* (page C47of Reference 2). Post-tensioned anchor zones for longitudinal cap reinforcement at the ends of the cap segments will require a solid stem in these portions (Figures 3.21a and 3.34). This will increase the weight of the cap.

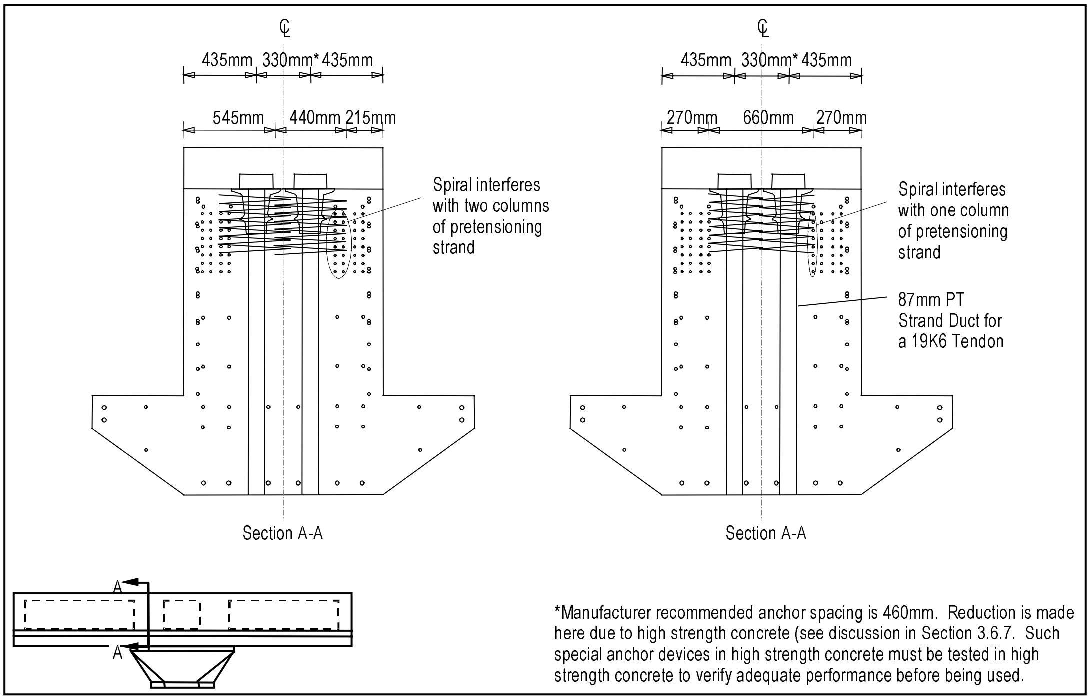


Figure 3.34 Cross-section through cap at the column post-tensioning strand anchor zone with different size spirals

Post-tensioning anchorage zone design and detailing requires attention to both the local zone and the general zone as defined in Section 5.10.9 of *the AASHTO LRFD Specifications*. Reinforcement for both the local and general zones for the vertical post-tensioned bars anchoring into the webs of the cap stem can be provided with orthogonal grid bars (bursting reinforcement). Vertical multistrand tendons will be anchored in the center of the stem, another area where the hollowed portion is filled in. Anchorage zone reinforcement for both vertical and horizontal multistrand tendons will generally require spirals for the local zone. Post-tensioning suppliers have standard details for spirals accompanying these special anchorage devices and are responsible for the specification of local zone reinforcement. The details provided to design engineers are typically for use with 28–34 MPa (4000–6000 psi) concrete. In the case of higher strength concrete, anchor spacing and the size of the spiral confining reinforcement required is sometimes reduced if tests have been performed at the higher concrete strength. Taking advantage of such reductions will be beneficial for the design of the proposed precast substructure system where space for large tendons is limited. For example, an engineer with the supplier VSL indicated to the senior author that an equation often used to estimate required anchor spacing dimensions is directly related to the concrete bearing strength at the end of the local anchor zone (Equation 3-1). [50]

$$X = \sqrt{\frac{1.15GUTS}{f'_{ci}}} \tag{3-1}$$

where: X: Anchor spacing, center-to-center

GUTS: Guaranteed ultimate tensile strength of the anchored post-tensioning tendon

f'_{ci} : Concrete compressive strength at the time of stressing

Anchor spacing will decrease with an increase in concrete compressive strength. Spiral reinforcement sizes, in particular the outer diameter of the spiral, could possibly be reduced along with anchor spacing. If the smaller spacing and spiral sizes are desired, acceptance tests *must* be performed on these special anchorage devices with higher strength concretes in accordance with the procedures outlined in *AASHTO Division II Article* 10.3.1.4.4.

In general, local zone design will be governed by the device characteristics indicated by the post-tensioning hardware supplier. However, the designer may use Equation 3-1 as a quick check to determine if certain tendon sizes will be feasible with chosen concrete strengths. It will be desirable in certain situations such as when the large 19K6 multistrand tendons are required for the vertical post-tensioning, to reduce the spiral size. Overly conservative spiral dimensions will make placement of longitudinal cap reinforcement that runs next to the spirals unnecessarily difficult (Figure 3.34). The beneficial effect of concrete strength on necessary spiral dimensions is illustrated in Figure 3.34. While the standard spiral diameter results in a conflict with two of the potential columns of pretensioning strand, a spiral sized for use with 55 MPa (8000 psi) concrete only interferes with one potential strand column. If this spiral is fabricated with a pitch of 50 mm (2 in), it may be easily placed so that the spiral loops fit between the strands. Unfortunately, the spiral requirements are hardware dependent and are dictated by the results of the manufacturer's special anchorage device tests. A few proof tests of proposed details should be performed to verify material designs and fine tune details if required. Such proof tests were required of the anchorage zones for the Bear Creek bridge and led to revised details.

e) Nonstructural details for this precast substructure system include maintenance concerns and improved substructure appearance. Internal drainpipes should be included in the design of all substructures (Figure 3.35). With thin-wall hollow piers, the drains can be placed in the void and passed through the cap. However for the drainpipes to be of use, they must be kept free of debris and blockage. Design and maintenance considerations must be coordinated so that dependable drainage methods are specified.

f) With careful quality control, the surface of precast elements can be attractive and relatively uniform, thus removing the need for painting. Other alternatives to painting which should always be considered are stained, rubbed and sandblasted surfaces (see Section 4.2.6). The grout bed under the template should be specified to have a final color that matches the color of the precast segments or provides a definite contrast as desired by the designer.

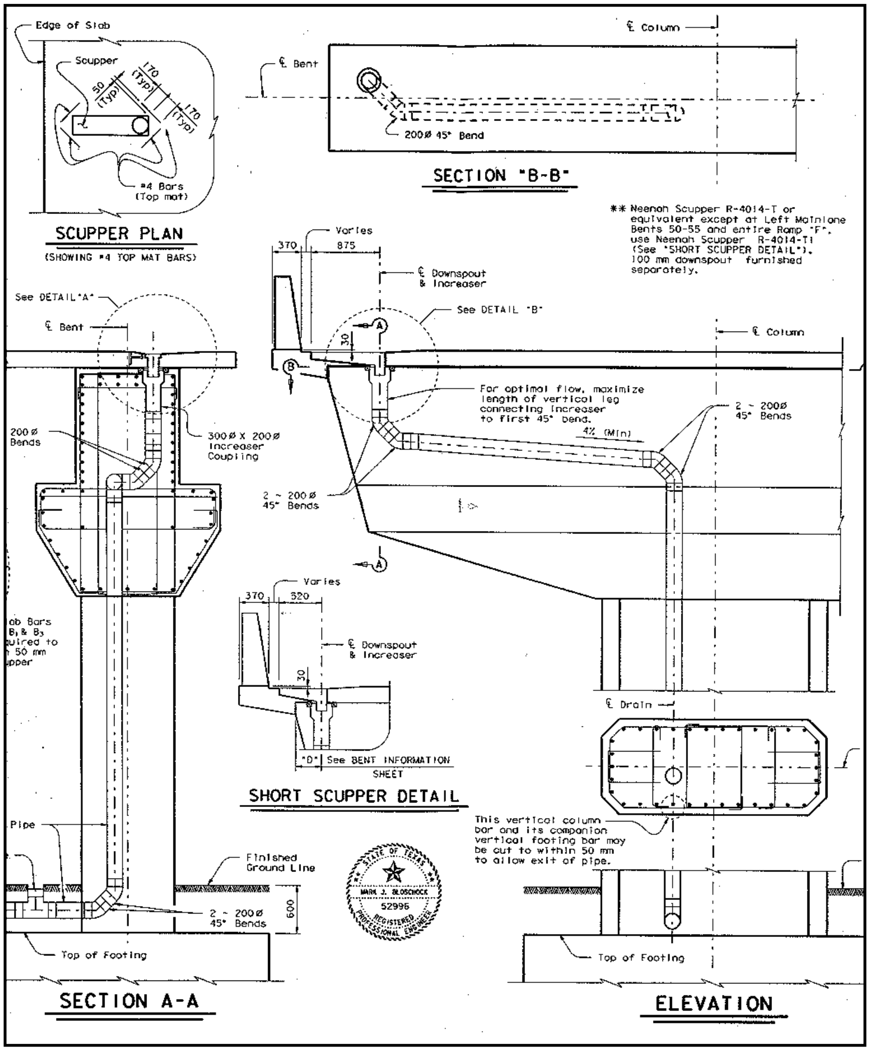


Figure 3.35 Details for an internal drain pipe in an inverted-T cap bent

3.6.8 Observations on LRFD AASHTO Bridge Specifications

In the process of designing post-tensioned substructures using the proposed system, a few important observations on the new *LRFD specification* were made. First, the longitudinal braking force in the *LRFD AASHTO* is considerably increased over the *Standard AASHTO Specification*. Table 3.4 shows the value of the braking force for simply supported bridges with different numbers of lanes comparing the *LRFD* and *Standard AASHTO* codes with the *Ontario Highway Bridge Design Code* [51] and the *Swiss (SIA) Standard 160*. [52] The equations for longitudinal braking for each code are given in Appendix E of Reference 2. The reason given in the *LRFD* version for the considerable increase is the improved braking technology of trucks as well as the fact that the new provision will now be more in line with other respected codes. The commentary to the *Ontario Code* explains that the provision is based on theoretical energy principles. However, it is noted that there have been no known failures to bridge columns due to longitudinal braking forces when designed for lower braking loads. Until specific evidence is presented to the contrary, it is recommended that the current (lower value) *Standard AASHTO* provision be used for longitudinal braking loads.

Table 3.4 Longitudinal braking force requirements for various codes

	Standard AASHTO kN (kips)		LRFD AASHTO kN (kips)		Ontario Bridge Code kN (kips)		1989 SIA Standard 160 kN (kips)	
No. of lanes considered	@ 1.8m (6ft) above deck surface		@ 1.8m (6ft) above deck surface		@ deck surface		@ deck surface	
1 lane	23	(5)	96	(22)	160	(36)	180	(40)
2 lanes	46	(10)	160	(36)	240	(54)	180	(40)
3 lanes	62	(14)	204	(46)	240	(54)	180	(40)
4 lanes	68	(15)	208	(47)	240	(54)	180	(40)

In TxDOT experience, column design in the past has rarely been controlled by longitudinal moment. Under the new *LRFD* provisions, the longitudinal bending moments in certain designs (in particular frame bents) will be greater than transverse bending moments particularly for shorter columns where transverse wind loads are not as critical. The effect on design efficiency will most probably not be large. The bending moments in frame bents are generally quite low and as seen in the design example in Appendix D of Reference 2, service stress limits are easily satisfied. Steel requirements for the cast-in-place alternative (page D15 of Reference 2) may need to be increased with increased longitudinal moments to satisfy biaxial bending requirements.

The second observation made in regard to the *AASHTO LRFD Specification* is in its treatment of design with prestressing steel. There is some debate among code-making bodies over the current prestressed concrete design philosophy in the United States. This philosophy, also found in the *AASHTO LRFD Specification*, is that stresses be limited under service loads to prevent cracking in members with prestressed reinforcement. This approach differs from the treatment of members with only nonprestressed reinforcement where cracking is permitted at service loads but must be controlled. Further discussion of these differing design philosophies can be found in References 53 through 57. Presently, the Service Limit State will almost always control the design of prestressed columns. This condition is particularly true for segmental columns because no tension is allowed across the segmental joints under service loads. The resulting amount of prestress prevents cracking under service loads and is often far in excess of what is necessary for the ultimate limit state. Regardless of one's position on the general philosophy of prestressed concrete design, there are direct positive benefits to having the Service Limit State control design. By not permitting cracking in the columns at service load levels, durability is obviously improved and fatigue will not be a problem. In addition, not allowing cracking results in increased column stiffness and reduced slenderness (P-delta) effects. Further studies are required to determine if these benefits are justified by the substantial increase in prestressing reinforcement above that required for the Strength Limit State.

3.7 Cast-in-Place Substructure Alternatives

Alternatives to the previously discussed precast substructure system include designing a substructure using the same geometric form but entirely cast-in-place or designing substructures of similar geometric form that are made up of combinations of both cast-in-place elements and precast elements. In Texas, benefits and drawbacks can be found with either an entirely cast-in-place system (CIP system), or a cast-in-place column with precast cap system (CIP column-PC cap system), as well as a precast column and cast-in-place cap system (PC column-CIP cap system).

3.7.1 Cast-in-Place Nonprestressed Alternates

The precast substructure system for Proposal II can also be constructed as an entirely cast-in-place system with nonprestressed reinforcement. To facilitate cast-in-place construction, solid sections should be used. Use of high performance concrete is recommended for improved substructure durability.

The inverted-T caps would be designed in accordance with current common practice in Texas. Such inverted-T caps can be found throughout Texas particularly in urban settings. Column design would also follow the same procedures as current common practice. The chamfered shape of Proposal II can easily accommodate required reinforcement to resist critical column forces as shown on pages C15 and D14 of Reference 2 where nonprestressed column alternates were investigated for the Proposal II precast substructure design examples. The amount of steel necessary was between 1 to 2% of the gross area of the solid column section.

3.7.2 Cast-in-Place Columns, Precast Bent Caps

With a CIP column-PC cap system, construction of cast-in-place columns can proceed directly following the casting of the foundation cap or footing. As typically used in many bridges, drilled shaft foundations may be continued above ground as the columns for a bent system. Precast caps can then be placed above the columns. This system is more efficient for forming than both a complete cast-in-place system and a PC column-CIP cap system. The column forms can be supported from the ground. The often heavy and awkward formwork for a CIP cap is avoided. The cranes required for superstructure erection can be used to place the precast caps directly before girder placement. The labor force required for this form of construction is grouped efficiently as well. A “cast-in-place” crew can work continually from the foundation to the columns. They can be replaced by the “precast placement” crew for placement of the caps, girders and, possibly, deck panels.

A disadvantage of this system is that the cap pieces are the heaviest elements of the substructure for hauling and erection. The cap is also the more cumbersome element to precast compared to column segments. With cast-in-place columns, a geometry control joint would be required underneath the precast caps to set them at appropriate cross-slopes. This would require balancing the heavy cap piece while alignment changes are performed. An alternative would be to use built-up bearing seats on the caps to provide deck cross-slope. Such a solution could be unsightly for wide caps or with large cross-slopes. Other disadvantages include disruptions to the site due to column forming, concrete placement and curing.

3.7.3 Precast Columns, Cast-in-Place Bent Caps

With a PC column-CIP cap system, a cast-in-place footing is followed by precast columns post-tensioned together as presented in Sections 3.4.3 and 3.6.5. The bent caps must then be formed and cast-in-place. The cap must be post-tensioned to the column to provide a fixed connection between the cap and column for the single-column piers. Post-tensioning of the cap to the column will also be necessary for two-column frame bents and straddle bents where moment transfer to the columns is desired. After the cap is cured and post-tensioned to the column, erection can proceed with the placement of the precast girders. This erection process alternates from cast-in-place footings to precast columns to cast-in-place bent caps to precast girders. This process requires an alternating labor force and equipment usage.

There are many advantages to a PC column-CIP cap system. Precast column segments are light and easy to haul and erect. Match-cast columns will allow for rapid column erection on site. Casting the cap in place allows the cross-slope to be provided during forming. One major disadvantage to this method of construction is that the cast-in-place portion of work is elevated. Therefore the caps require very heavy self-supporting forms. These heavy forms will often need to be assembled and the concrete placed at hazardous elevations.

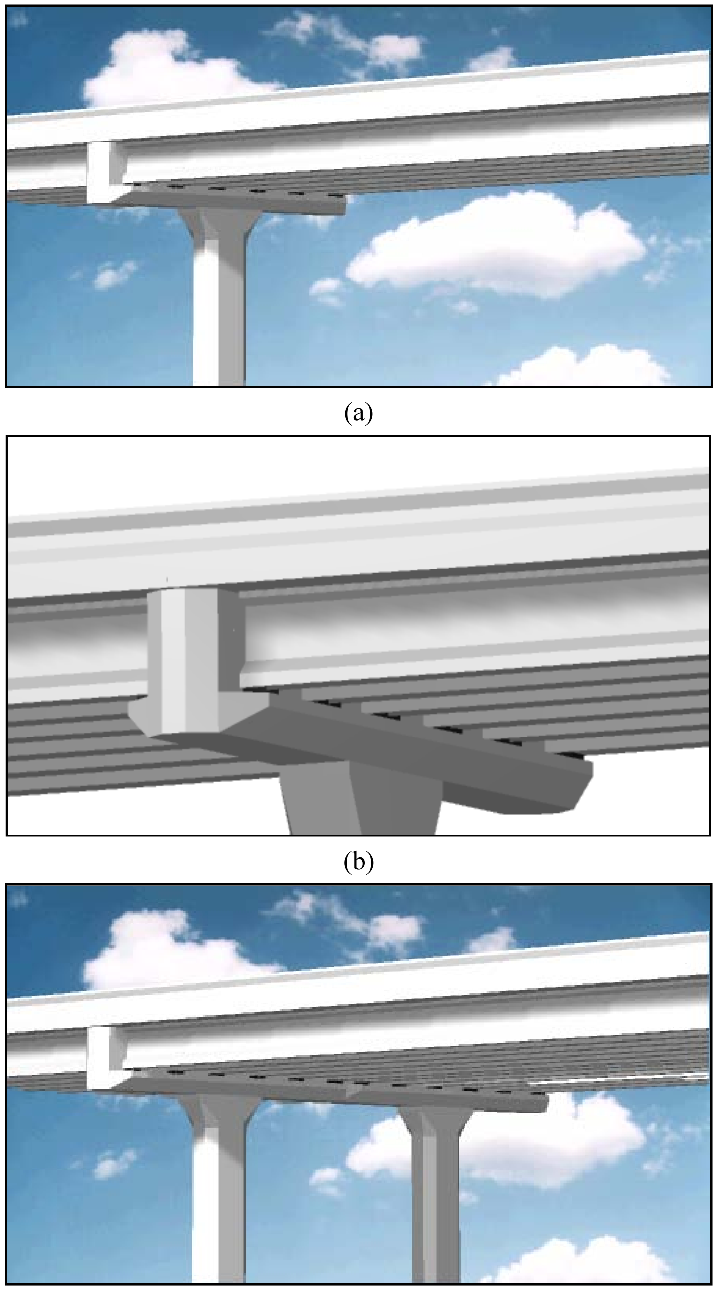
3.7.4 Section Summary

Regardless of which option is chosen, precasting any part of the substructure system should speed up construction time and reduce site disruptions when compared to an all cast-in-place substructure system. Both systems offer improved durability for the columns through use of high-performance concrete and post-tensioning. Prestressed caps would be less permeable and therefore more durable than nonprestressed caps. Both precast and cast-in-place caps could be prestressed (pretensioned and post-tensioned for precast caps and post-tensioned for cast-in-place caps). Precasting would provide higher quality control in fabrication with a resulting less permeable concrete.

3.8 Chapter Summary

Substructure design provides an opportunity for innovative design with short- and moderate-span bridge systems. New technologies and new materials can be applied for attractive and economical results. Substructures can be constructed using methods of precasting, casting in place or a combination of the two. This chapter has presented a precast substructure system for standardization. A geometrically similar system may be cast in place or be a combination of both precast and cast-in-place elements.

The proposed precast substructure system is a versatile system that can be used for a wide variety of bridge widths and heights. This system can be used with standard precast girder superstructure systems and offers a new alternative to substructure design that can increase construction speed thereby reducing costs associated with traffic delays and re-routing. The precast system of match casting with epoxy joints has provided excellent durability for structures in the past. The combination of precasting and using high performance concrete results in more durable and more attractive construction (Figure 3.36). This proposed system obviously is not a universal solution. Replacing a multicolumn bent which has a rectangular bent cap with a single-column pier will generally substantially increase costs. If the substructure is concealed from public view and does not interfere with traffic, and if construction speed is not a factor, the proposed system may be unnecessary or undesirable. The construction of the frame bent using the proposed precast substructure system can involve stage prestressing which is more complex and time-consuming than the hammerhead bent system. Thus, the frame bent will probably have less advantages than the hammerhead bent.



(a)

(b)

(c)

Figure 3.36 Computer-aided renderings of a hammerhead (a, b) and frame (c) bent using the proposed precast substructure system

Precast substructures are not an entirely new form of construction but have been used successfully in the past. Such a system will be most useful at first in Texas for large, highly repetitive, projects in highly visible locations where construction efficiency (speed of construction) and final appearance are particularly important. An initial investment in forms for a large project will lead to future savings when the forms may be reused for similar or smaller projects. Over time and with high reuse, new standard shapes for substructures may be developed to provide TxDOT designers with even more alternatives for attractive and rapidly constructed substructures.

CHAPTER 4

ALTERNATE SUBSTRUCTURE DESIGN—CAST-IN-PLACE CONCRETE OPTIONS

4.1 Introduction

Cast-in-place concrete requires formwork to be assembled in the field. Formwork can be fabricated to accommodate almost any shape the designer chooses (Figure 4.1). Such a wide variety of shapes are possible because of the ability to mold fresh concrete. The engineer's challenge is to find new forms and new shapes that are attractive and within reasonable economic limits.

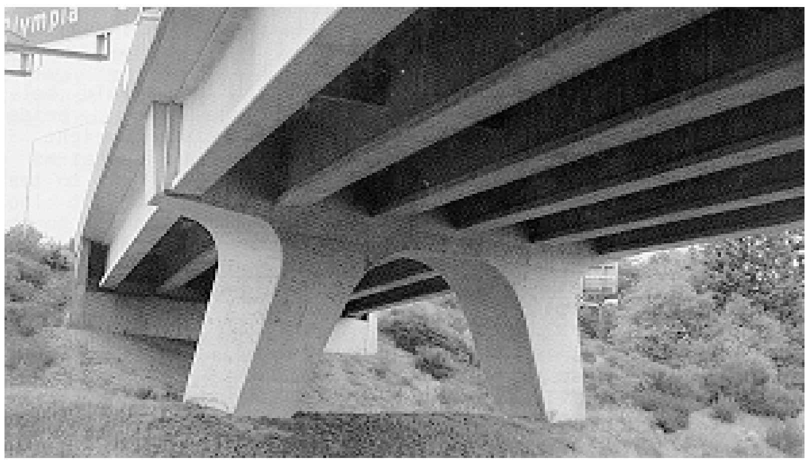


Figure 4.1 Cast-in-place substructure

Savings can be achieved with cast-in-place concrete through standardization of formwork. Unfortunately, in Texas such substructure standardization has never progressed much beyond the use of just a few shapes like circular or rectangular columns with prismatic caps (Figure 4.2). This system typically results in an ugly forest of columns in any setting. In the past, the limited variety of substructure shapes in Texas has been maintained for the sake of economics yet allows for very little designer expression. Recently, the TxDOT designers and their consultants have been using many more attractive single-column piers, particularly in urban expressways and interchanges.

Figure 4.2 Typical cast-in-place substructure system in Texas

This chapter explores new options for cast-in-place concrete substructure design. In particular, ideas are presented that are compatible with use of precast concrete superstructure elements. The goal of this chapter is to spark the imagination of engineers designing short and moderate span bridges. Ideas are presented that can be implemented in certain design situations without excessive cost increases. If the same basic substructure system (single-column, multicolumn) is maintained, very small increases and even savings are possible. Changes in structural systems such as replacement of multicolumn bents with single-column bents may lead to more substantial substructure cost increases. Recognizing that the substructure cost is only a portion of the total bridge cost (roughly 30% in Texas), any increases in substructure cost will have a lessened effect upon the overall bridge cost. For large highway projects in which the bridges typically are a small portion, any increases in substructure cost will be further minimized in relation to overall project cost.

4.2 Alternative Cast-in-Place Substructure Systems

A few substructure shapes apart from the predominant circular and rectangular columns have been experimented with in Texas. Even more shapes can be found in other states and countries. Section 4.2.1 reviews the basic substructure systems used throughout Texas. Sections 4.2.2 through 4.2.6 discuss a variety of alternative shapes including previously used shapes and new ideas for substructure design. Ideas for enhancing substructure design through nonstructural details such as concrete texture and color, and the shaping of bent cap ends are also explored. The cast-in-place options presented are discussed in terms of their aesthetic appeal or drawbacks, as well as their economic feasibility for TxDOT.

4.2.1 Substructure Systems

The substructure systems most widely used in Texas include individual columns, walls, hammerhead bents and multicolumn bents. Each system is presented and illustrated below. (Although all of the systems presented can be found in Texas, illustrations 4.6 and 4.11 are not Texas bridges.)

4.2.1.1 Individual Columns

Individual columns may be efficiently precast or cast in place. Individual columns may be used *without bent caps* to support individual girders (Figures 4.3 to 4.6).

Individual columns are most appropriate when supporting single or widely spaced girders such as segmental or trapezoidal boxes or U-beams (Figures 4.5 to 4.7). Trapezoidal box beam and U-beam superstructures require fewer longitudinal beams. As a result, it is often possible to use fewer individual supports for a given bridge width than traditional box beams or I-girders would require. Individual supports for traditional I-girders typically create a cluttered "forest of columns." Widely-spaced, large I-girders and/or girders on heavy skews however, may warrant individual columns resulting in a less congested appearance.

Figure 4.3 Individual columns supporting individual beams

Figure 4.4 Individual columns supporting individual cast-in-place continuous beams

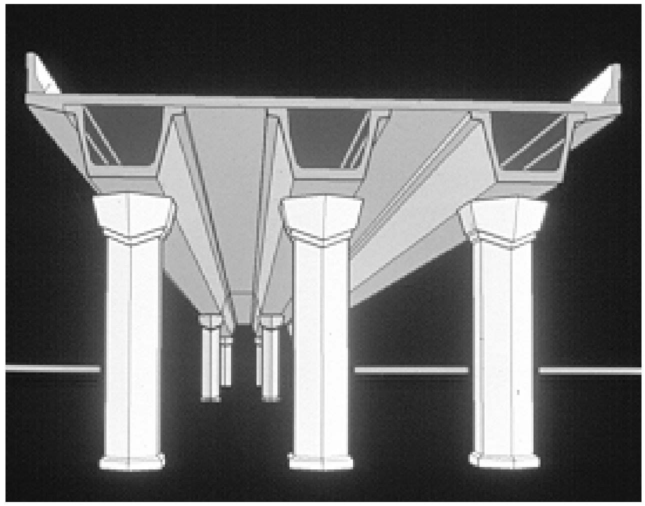


Figure 4.5 Individual columns supporting individual precast U-beams

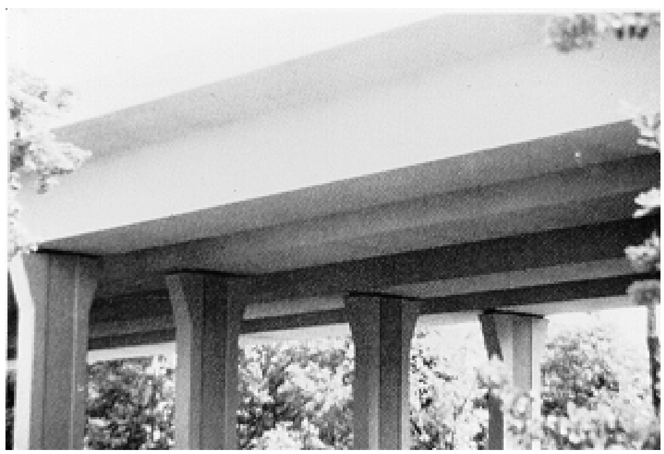


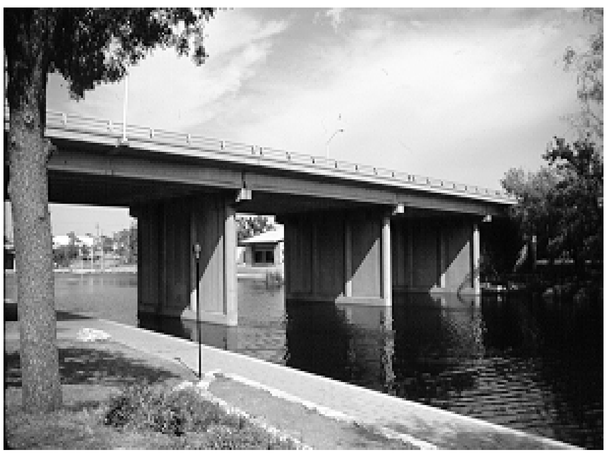
Figure 4.6 Individual columns supporting individual trapezoidal box girders

Figure 4.7 Individual columns supporting a single box girder bridge

4.2.1.2 Walls

Wall substructures are supports that are generally as wide as the superstructure which they support (Figure 4.8). Walls are typically used in waterways for minimizing blockage from debris (Figure 4.9a) or for crash protection in railroad crossings (Figure 4.9b). Wall substructures can severely obstruct visibility through a bridge when viewed from most angles, particularly if they are closely spaced (Figure 4.9a). However, when walls are used in connection with increased span lengths, the walls may add to a simple, elegant appearance (Figure 4.10a). This can be contrasted to the clutter and loss of elegance typical of multicolumn bents when used with shorter spans (Figure 4.10b). Walls and wall-type piers are typically cast-in-place and can therefore be cast with tapers or other interesting shapes (Figure 4.11).

Figure 4.8 A wall substructure unit used for a highway over-pass

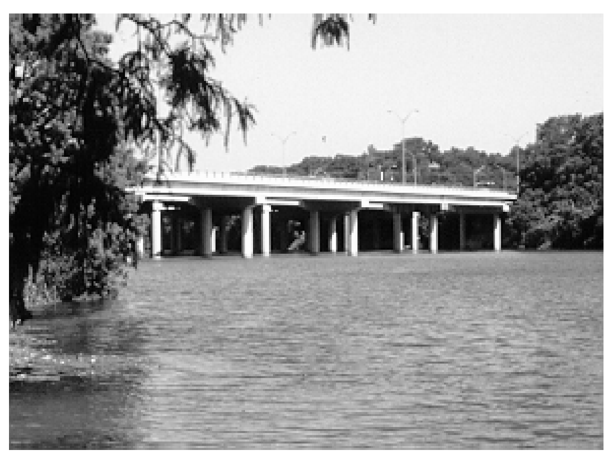


(a)

(b)

Figure 4.9 (a) Wall substructure units for a water crossing; (b) A combination multicolumn and wall substructure for a railroad crossing

a) Two single-tapered walls with long spans



b) multicolumn bents with shorter spans

Figure 4.10 Contrasts in clutter of the substructure

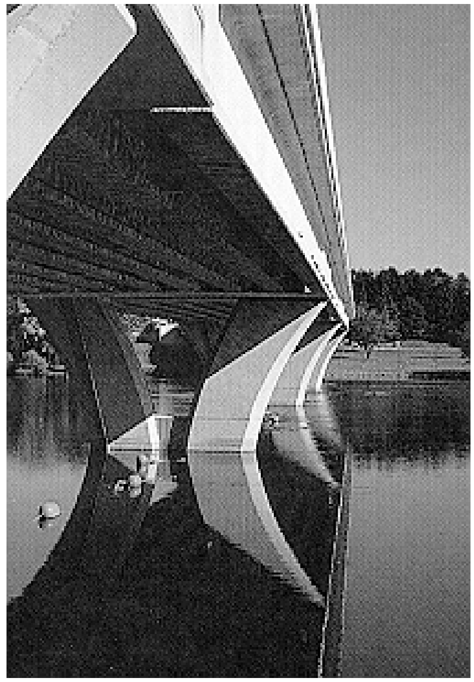


Figure 4.11 Cast-in-place substructures can be curved or tapered

4.2.1.3 Hammerhead Bents

Hammerhead bents (T-shaped single-column bents) are common for narrow bridges and bridges in locations where visibility through the structure is desired. Hammerhead bents may be a variety of shapes. They may be entirely underneath the superstructure (Figures 4.12 to 4.13) or partially or fully integrated with the superstructure (Figures 4.14 to 4.15). Circular columns are difficult to integrate visually with rectangular bent caps (Figure 4.12b, 4.14b). Rectangular columns can be integrated with rectangular bent caps more easily with attractive results (Figures 4.12a, 4.14a) Hammerhead bents may be have tapered caps and/or columns where structurally appropriate for increased efficiency and expression of the flow of forces (further discussed in Section 4.2.2).

A rectangular column and prismatic cap (a) and a rounded column joining a prismatic cap (b).

Figure 4.12 Hammerhead bents types

Figure 4.13 Hammerhead bents entirely underneath the superstructure

(a) (b)

Partially integrated bent caps with rectangular columns and prismatic caps (a) or with round columns and prismatic caps (b)

Figure 4.14 Bent cap integration with superstructure

Figure 4.15 A hammerhead bent cap fully integrated into the depth of the superstructure through the use of dapped girders

4.2.1.4 Multicolumn Bents

Multicolumn bents are common for wide bridges or in locations where the area underneath the bridge needs to be straddled. Multicolumn bents are often the most economical solution for any bridge width yet typically give a bridge a cluttered appearance (Figure 4.16). As with hammerhead bents, circular columns are more difficult to visually integrate with prismatic bent caps than are rectangular columns (Figures 4.17 to 4.18). Multicolumn bent caps may be entirely underneath the superstructure (Figure 4.17) or they may be partially or entirely integrated with the superstructure (Figure 4.18). Bent caps may be haunched or tapered where structurally appropriate for increased efficiency and expression of the flow of forces (Figure 4.19).

Figure 4.16 Cluttered multicolumn bents create "forests" of columns

Figure 4.17 Circular columns with a rectangular bent cap entirely underneath the superstructure

Figure 4.18 Rectangular columns joining a prismatic inverted-T cap

The tapered cap and column of this hammerhead bent is structurally expressive of the increased moment resistance required at the cap/column intersection and at the column base under critical loads

Figure 4.19 Structural expression in substructure design

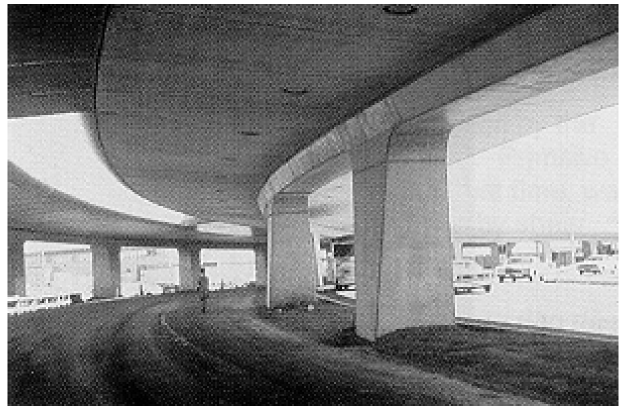
4.2.2 Alternative Shapes–Structural Expression, Curves, Tapers

There are numerous possibilities for attractive substructure design. The substructures may be designed to express visually the flow of forces from the superstructure to the foundation, often referred to as structural expression. There is a direct connection between the structural form and the structural function. Examples include deepening sections at points where larger resistance for higher moments is required or tapering columns down to pinned ends. For instance, the tapered hammerhead bent shown in Figure 4.19 is structurally expressive of the flow of forces under static loading conditions.

Figure 4.20 shows a pier that has a fixed connection at the base and a pinned connection under the superstructure. The form is structurally expressive in that the lack of moment transfer at the top of the column is expressed visually with the narrowing of the pier section towards the top. The fixed connection at the bottom is expressed by the widening of the pier towards the base. Other examples are shown in Figures 4.21a and b. [58] Structurally expressive forms must be true to their purpose. Exaggerated forms may appear contrived (Figure 4.22) while other expressive forms may be disconcerting (Figure 4.23) if the nature of a supporting cap is completely concealed.

Figure 4.20 Structural expression in substructure design

(a) These tall piers are more fixed at the base than at transversethe top. Under critical lateral loads, the point of inflection is located at about two-thirds the height from the base, allowing the columns to taper in at that location of minimum moment. [58]



(b) The tapers in these piers vary in both the longitudinal and transverse directions. In the transverse direction, the columns are vertical cantilevers fixed at the base and pinned at the top. In the longitudinal direction, the columns are pinned at the base to relieve shrinkage and creep stresses in the post tensioned deck are "fixed" at the top to provide continuity between the super- and substructure. [58]

Figure 4.21 Structural expression in substructure design

Extreme tapering in short substructure units exaggerates the flow of forces and appears contrived.

Figure 4.22 Extreme tapering design

A flared column that stops short of the outer girders leaves the viewer with a sense of uneasiness about the support of the superstructure.

Figure 4.23 Disconcerting

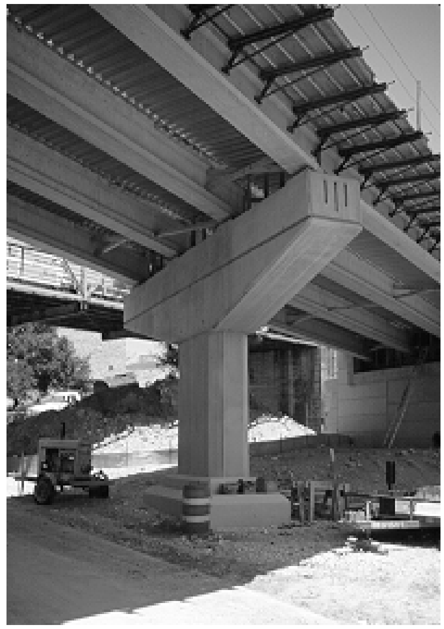
To display the flow of forces between the caps and columns of substructure units, the edges of these two elements should be continuous from one to another. Elements with abrupt changes in size whose edges do not line up give the substructure a clumsy, "building block" appearance (Figures 4.24 to 4.25). Attention to the integration of the different parts results in a more attractive form, one that demonstrates the smooth flow of forces from the superstructure to the foundation (Figures 4.26a and b).

The wide and heavy cap taper does not integrate well with this narrow column

Figure 4.24 A lack of integration

A large cap end wall contrasts strongly with narrow columns and gives the structure a weak and clumsy appearance

Figure 4.25 An awkward appearance



(a)

(b)

Figure 4.26 Attractive integration of the cap and column through simple column chamfers and cap tapers (a and b)

Expression of the construction process is another alternative for structural expression. For example, construction joints may be accented with chamfers. Accented joints however, interrupt the smooth lines expressive of concrete's quality of monolithically following any shape or form. Accented joints may make concrete appear like its structural material predecessor, masonry (Figure 4.27).

(a)
Accented joints give the appearance of a masonry structure.

(b)
A smooth surface shows off concrete's ability to perform its structural task in a flowing form.

Figure 4.27 Effect of accenting joints

Simple curves can enhance a substructure's appearance particularly for bridges in highly visible settings. Large wall supports can be shaped with a curving flare to minimize their often heavy appearance (Figure 4.28). Incorporating curves into substructure design softens the visual flow from one element to another, avoiding a building block look.

Curved tapers on these large wall supports are an elegant alternative to the typically massive appearance of wall substructures.

Figure 4.28 Reducing massive appearance

Subtle tapers can be used for attractive results as well. A two-span overpass with a single, tapered column is more elegant than a three-span bridge with two multicolumn standard Texas bents (Figures 4.29a and b). A single, tapered pier (Figure 4.30a) is more handsome than a stepped circular pier (Figure 4.30b). Obviously, cost considerations will have to be evaluated.

(a)

(b)

Figure 4.29 A simple tapered single support (a) is a more elegant solution than a series of multicolumn bents (b) for a standard overpass

(a)

(b)

Figure 4.30 A subtle column taper (a) is more attractive than abrupt changes in cross section (b) for tall columns

4.2.3 Improving Visibility through the Bridge

Multicolumn bents on short-span bridges allow for limited visibility through their forest of columns (Figure 4.31a). Where more openness, light and visibility are desired, fewer substructure elements should be used (Figure 4.31b). Such a major change in structural system may substantially increase costs. Such openness can best be obtained through the use of hammerhead bents or multicolumn bents with no more than two columns. Bent caps with more than two columns are generally cluttered and should be avoided (Figures 4.32 to 4.35). Where possible, an exceptionally wide bridge could be split into two bridges each with a two-column substructure (Figure 4.36). Using two smaller bents in place of one larger one allows more light to reach underneath and through the bridge thus avoiding dark tunnel effect. As well, two smaller frame bents may be used rather than one larger bent with many columns. When two-column bents are combined with longer spans, a lighter, more transparent bridge will result. This is a particularly good solution for congested urban areas, crime-ridden areas or park settings.

Fewer substructure elements may result in larger elements. Therefore, it is important to keep in mind the size of elements when minimizing the number of elements. Large columns may appear as walls and block visibility from certain angles. In such cases, two smaller columns in place of one wall-like column may be appropriate.

(a) (b)

Bridges of similar superstructure and roadway width with a forest of columns (a) or a single elegant tapered column (b)

Figure 4.31 Visibility through substructure

(a)

(b)

Figure 4.32 Cluttered multicolumn bents (a) replaced with more open two-column frame bents (b)

(a)

(b)

Figure 4.33 A narrow two-column bent vs. a single-column hammerhead bent (b)

(a) (b)

Replacing multicolumn bents (a) with single-column hammerhead bents (b) creates a neater, more open substructure area. The proportions of large single columns blend better with precast girder superstructures than the thin columns of multicolumn bents.

Figure 4.34 Multicolumn bents vs. hammerhead bents

(a) (b)

Replacing multicolumn bents (a) with single-column hammerhead bents (b) creates a cleaner more rhythmic appearance

Figure 4.35 Multicolumn bents vs. hammerhead bents

(a) (b)

A wide roadway (a) can be split into two separate structures with fewer columns (b). This allows more light to reach the underside and increases visibility through the bridge.

Figure 4.36 Separating wide bridges

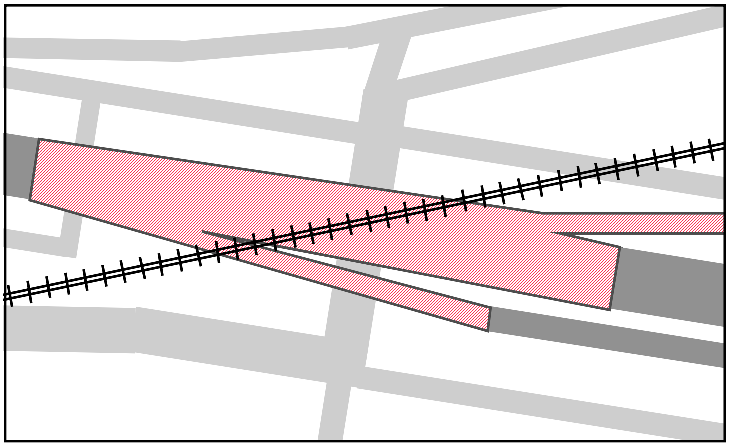
Visibility through the bridge is also effected by the type of bent cap chosen. Bent caps may be fully integrated, partially integrated or entirely underneath the substructure (Figures 4.12 through 4.15). Compared with fully and partially integrated bent caps, caps that are entirely underneath the superstructure lead to the most obstruction of visibility through the bridge (Figure 4.13). Partially integrated bent caps such as inverted-T caps, place the mass of the cap between the beams so that only the ledge supporting the beams is visible. Therefore visibility through the bridge is improved with partially integrated caps (Figure 4.14). Fully integrated caps allow for the cleanest profile. Substructure clutter is greatly reduced allowing for the maximum visibility through the bridge (Figure 4.15).

Visibility is often impaired by the use of skewed bents. In particular, a mixture of skewed and normal multicolumn bents leads to a confusing design and one that is often visually "incoherent" (Figures 4.37a and b). Such mixtures should almost always be avoided. In general, unless stream or traffic flows make them essential, skews should be avoided all together. While skewed bridges may minimize span lengths, they are typically more difficult to construct and skewed abutments often lead to increased costs. [59] Alternate solutions should be investigated (Figure 4.38). Where skews cannot be avoided, substructure shapes should be chosen that can accommodate both the deck and skew directions such as octagonal piers (Figure 4.39). (Circular columns are another option but, as stated before, do not integrate well when used with rectangular bent caps and are not recommended.)

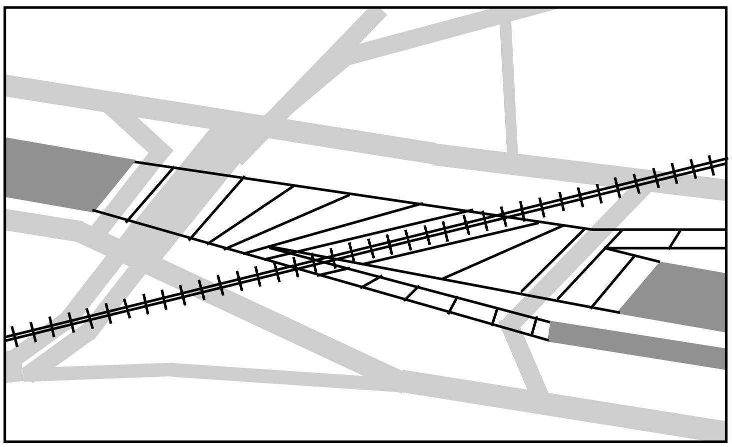
(a) (b)

Skewed bents result in confusing designs that are visually incoherent and unattractive.

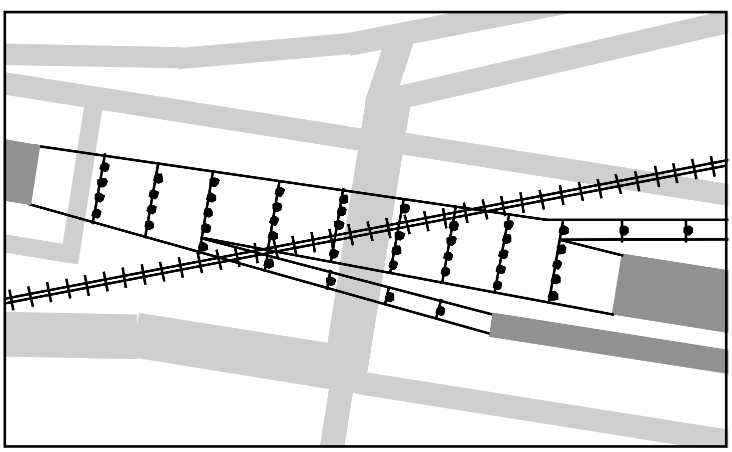
Figure 4.37 Skewed bents



(a) Right-of-way for a future bridge over a railroad track. The roadway geometry next to and underneath the future bridge is to be designed as well.



(b) The original layout consisted of eleven long skewed bents of numerous orientations for the main bridge. Some bents were parallel to the motorist traffic under the bridge, others were parallel to the train tracks under the bridge. Very few bents were perpendicular to the orientation of the superstructure.



(c) The proposed layout consisted of nine bents of similar length and orientation. The bents are parallelled to the motorist traffic passing underneath and perpendicular to the orientation of the superstructure.

Figure 4.38 Careful examination of an original skewed layout (b) shows an alternate solution possible (c) resulting in a cleaner design.

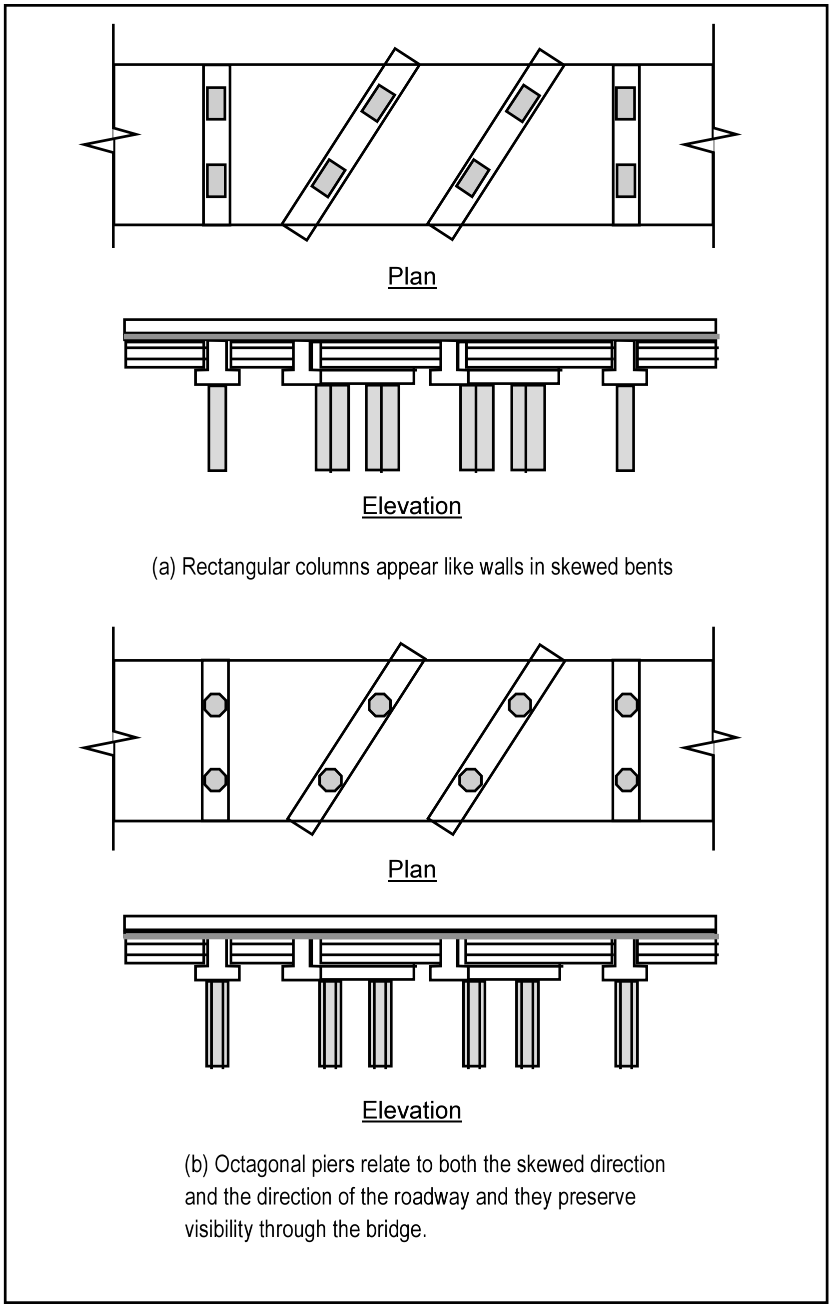


Figure 4.39 Comparison of rectangular piers and octagonal piers for skewed bents

4.2.4 Integrated Designs

Designs that integrate the superstructure and substructure constitute another attractive alternative that has been used occasionally in Texas and extensively in the Pacific Northwest. In the latter areas, such integration is often required structurally to resist earthquake loads. However in any location, a moment connection between the superstructure and substructure will allow for longer spans thus potentially decreasing the number of foundations and substructure elements required and often increasing the visual slenderness of the bridge.

Integrating the superstructure and substructure does not require a moment connection. Fully integrated caps are the most direct way to visually integrate the substructure with a simply supported superstructure. Here, inverted-T bent caps would be used to support simply supported dapped beams. This usage leads to the cleanest connection with the least amount of substructure clutter (Figures 4.15, 4.31b, 4.40).

Figure 4.40 A fully integrated cap provides a clean transition between the superstructure and substructure

Exposed and partially exposed bent caps may be shaped to express the flow of forces from the superstructure to the foundations as discussed in Section 4.2.2. Such shaping visually integrates the elements, provides visual interest to the user or passerby and imparts a feeling of stability and safety.

4.2.5 Variety with Standards

Many suggestions for improved substructure designs may require unique formwork that would not be feasible for projects with tight budgets. For such projects, standard substructure forms will almost always be required. Although not the case in Texas at this time, there can certainly be variety and appeal in standard forms. Two simple details that add visual interest and be standardized are column chamfers and column flares.

Chamfering is a technique used to remove the sharp corners of rectangular columns (Figures 4.41 and 4.42). The angle of the chamfer can be chosen by the designer and will be effected by the reinforcement layout and overall structural design. Chamfering can minimize or enhance the relative proportion between the columns and different elements of the bridge.

Figure 4.41 Small chamfers on a rectangular column

Figure 4.42 A large chamfer makes these rectangular columns appear slender

When designing different-sized chamfered columns on the same project, chamfers may change proportionally with the column size or remain the same (Figure 4.43). Particularly for columns of varying height, Option II of Figure 4.43 where the chamfers change in proportion with the column section is a more attractive solution than Option I (Figure 4.44). Keeping chamfers in proportion with the column sizes creates a smoother visual transition from one size of column to the next. Small chamfers may be attractive on tall columns, but when the same cross-section is used for a shorter column, it will appear more like a stocky wall (Figure 4.44). Column chamfers can easily be incorporated into standard designs with attractive and economical results.

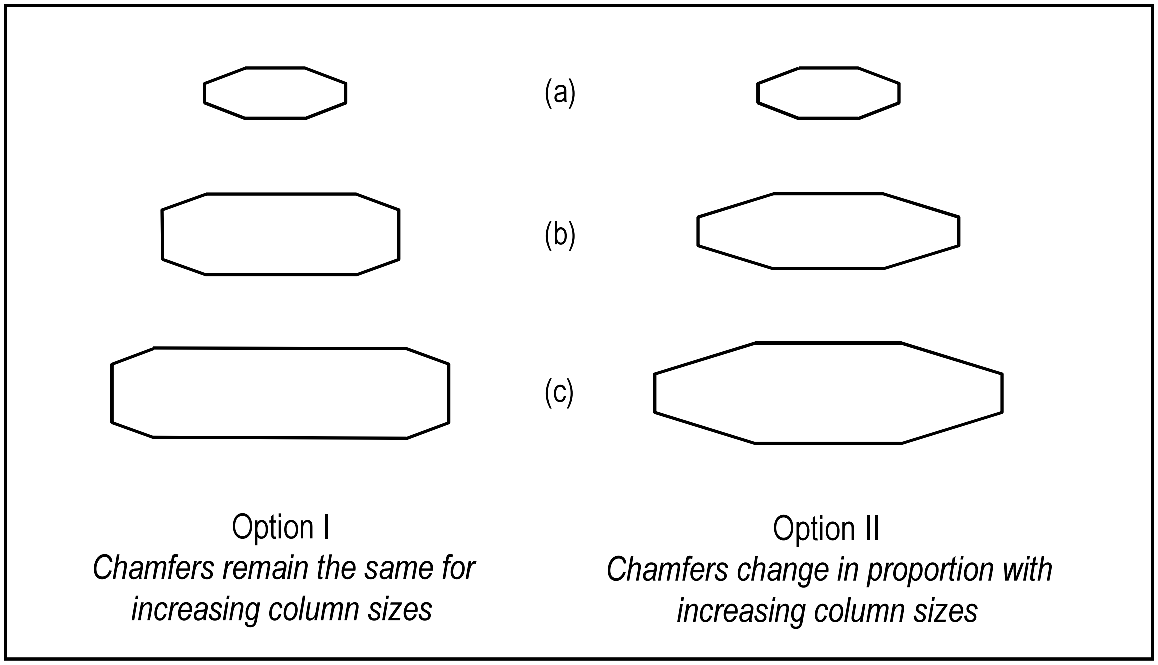


Figure 4.43 Two chamfering options for columns of varying size

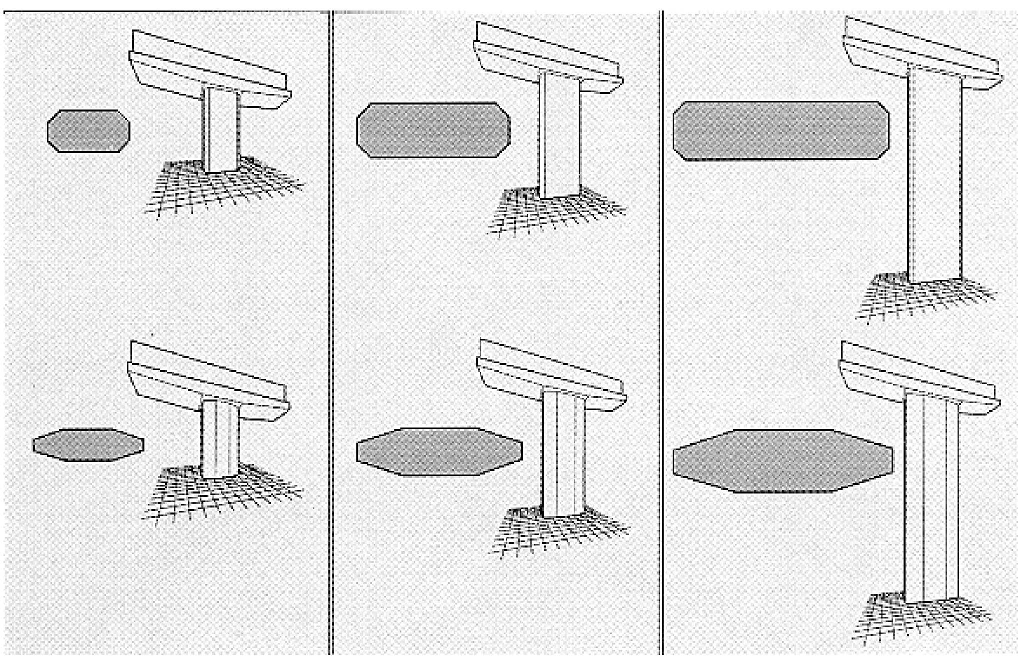


Figure 4.44 Comparison of chamfer options for piers of varying heights

Simple flares can be incorporated into standards as well. The California Department of Transportation has over 40 standard column shapes, most of which feature flares (Figure 4.45). [60] Over the years, as the standards have been used more regularly, they have become more economical.

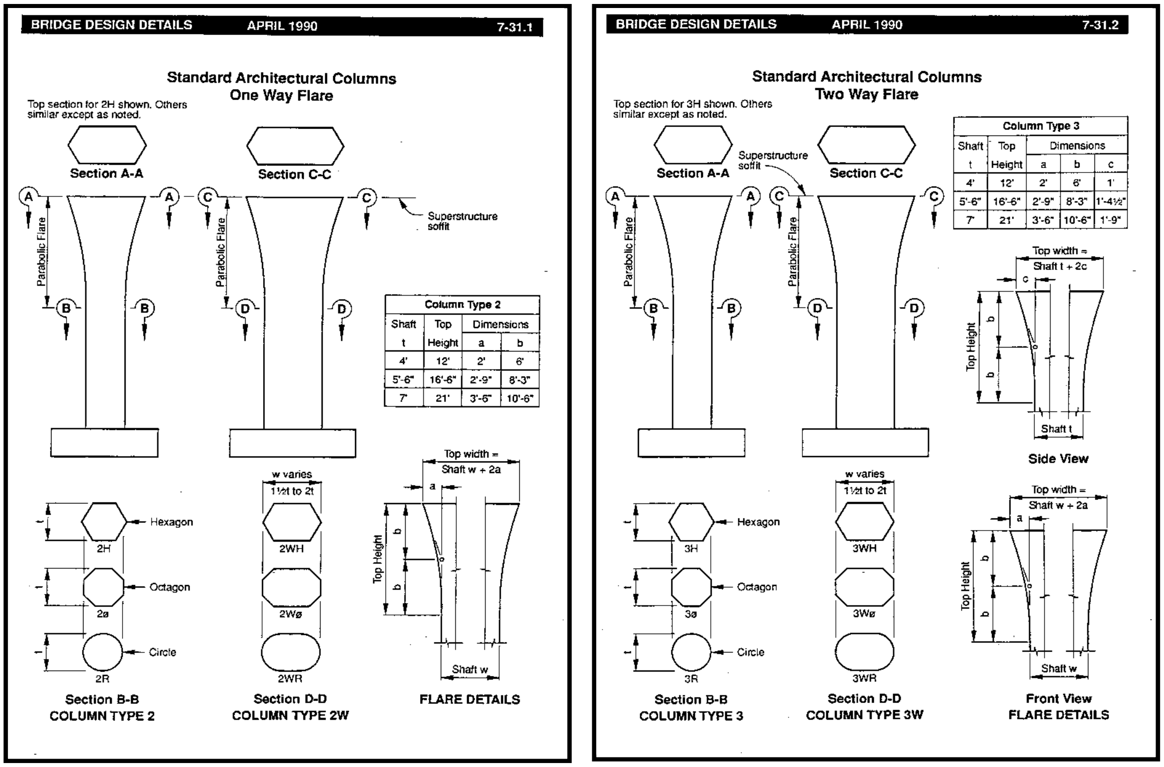


Figure 4.45 Standard column forms used by CALTRANS [60]

4.2.6 Enhancement of Substructure Designs using Nonstructural Details

Nonstructural details such as texture and color can be used in substructure design to enhance structural qualities. Attractive shaping of bent cap ends is another non-structural detail that can improve the substructure and overall bridge appearance.

Texture and color selections may be used to accent slenderness and form but are by no means necessary for attractive substructure design. Texture and color should never be used as a distraction to decorate a poorly proportioned substructure or to cover up dull forms.

There are numerous finish options. To avoid monotony, different projects should incorporate different finishes according to the design concept of each project. However, similar textures and colors should be used throughout a large project for coherence. Reference 3 gives an outline of many textures, colors and formliners available including their advantages and limitations in terms of appearance, maintenance and cost.

<u>Texture</u>

Different types of concrete texture include exposed aggregate, sand-blasted surfaces, rubbed finishes, relief and surface patterns obtained from formliners.

Exposed aggregate may be used to reflect the local geological materials, particularly when the aggregate color matches that of natural rock surroundings. Exposed aggregate is not only an attractive finish, but it may act as a graffiti deterrent (Figure 4.46). To avoid monotony from one project to the next, the numerous types of exposed aggregate finish available should be explored. Sand blasting and rubbed concrete are two other concrete finish options that have successfully enhanced attractive projects in the past (Figure 4.47).

Figure 4.46 Exposed aggregate finishes may be less likely to attract graffiti than plain concrete

Figure 4.47 An attractive pier with a rubbed concrete finish

Relief may be provided to accent different structural members. Vertical grooves in piers accent height and give piers a taller and thus more slender appearance (Figures 4.48 and 4.49). Horizontal accents on vertical members give a heavier, more massive or cut-stone-like appearance (Figures 4.50 and 4.51).

Figure 4.48 Vertical relief enhances the slenderness of this pier.

Figure 4.49 Vertical grooves give this wide pier a more slender appearance.

Figure 4.50 Horizontal grooves make concrete appear like masonry.

Figure 4.51 Horizontal accents give tall piers a stockier appearance.

A wide variety of formliner patterns are available for use on bridge projects. A single, formliner pattern can be used effectively as a harmonizing element throughout a project (Figure 4.52).

Figure 4.52 Similar formliner patterns used for the railing and the end of the bent cap

The use of texture to make concrete appear as another material should be restricted to locations where the structure is meant to replicate local structures. However, in general, concrete is best employed expressing the sculptural material that it is, capable of being formed in countless shapes and emphasizing its inherent strength in compression. Rather than imitating other structural materials (Figure 4.53), concrete is most artistically used in its own unique ways (Figures 4.54 and 4.55).

Figure 4.53 Concrete piers formed to appear like stone are foreign to the modern superstructure and urban location.

Figure 4.54 An attractively shaped pier

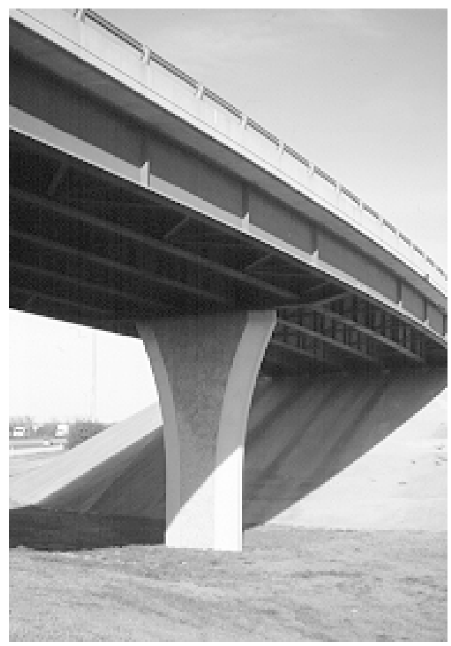


Figure 4.55 An elegantly tapered pier exposed aggregate used to accent slenderness; however, a disconcerting overall design—see Figure 4.23

Color

Color may be incorporated in concrete designs by adding pigments to a concrete mix, or by staining or painting the surface (Figures 4.56 and 4.57). Color is incorporated in steel bridges through weathering steel or paint.

Figure 4.56 The concrete for these piers was stained to blend with the local rock coloring.

Figure 4.57 Concrete painted with an earthtone to integrate with the landscape

Experience in Texas has shown that painted concrete typically peels within a few years of application. Painting of concrete therefore results in the additional maintenance needs incurred by repainting whereas concrete colored through staining does not (Figure 4.58). Colored concrete whether painted or stained, may be made to match the local natural or built environment. This technique is typically used for architectural projects. A nice example is found in Austin where retained earth wall colors are made to resemble the pink granite of the State Capitol (Figure 4.59).

Figure 4.58 Unattractive peeling of painted concrete

Figure 4.59 Low concrete retaining walls colored to match local building materials

As mentioned in the previous chapter, graffiti and staining from dirty run-off water are common problems that plague bridges, particularly the substructure. Painted concrete can be repainted to cover unwanted staining or graffiti. Stained concrete can be sand-blasted to remove unwanted staining and graffiti. A wide variety of concrete stain and paint colors are available.

Bent Cap Ends

The ends of bent caps are typically highly reflective surfaces. As a result, they call attention to themselves and often detract from the overall bridge appearance. In many cases, bent cap ends stick out like sore thumbs (Figure 4.60). Shaping the ends can prevent these eyesores, softening bulky proportions.

(a) (b)

Figure 4.60 Awkward appearance of blunt rectangular bent cap ends magnified by skewed bents

Angling the end of a bent cap to put it in a shadow will cause attention to focus on the reflective superstructure thereby accenting the horizontal flow of the structure (Figure 4.61). A more sculptural, chamfered end will add interest and may be desirable for a structure emphasizing the relationship between superstructure and substructure (Figure 4.62 and 4.63). The cap may be shaped to blend with and maintain the horizontal line of the superstructure (Figures 4.64 and 4.65). The result is the appearance of one long continuous beam. Designers must keep in mind that this technique is deceptive in terms of structural expression. The true structure is one of simply supported beams, not continuous beams. In addition, this technique becomes very complex to carry out in applications with changing cross-slopes.

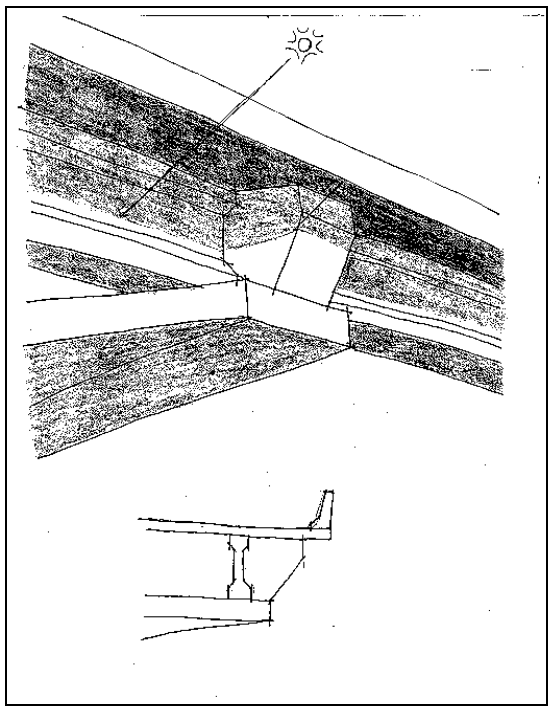
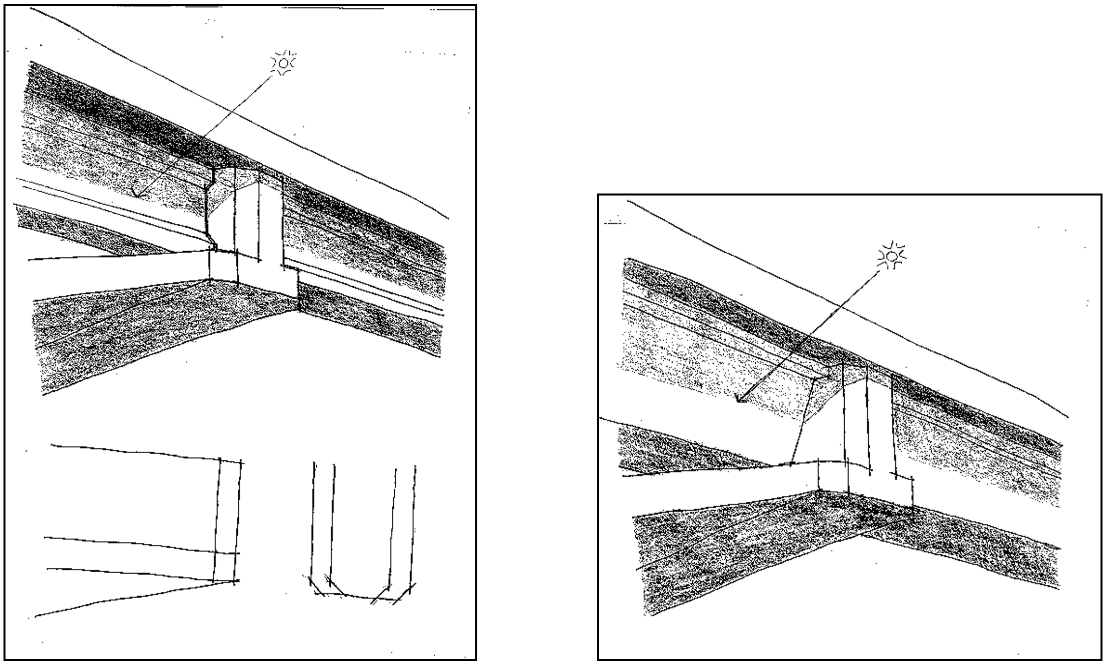


Figure 4.61 Bent cap angled back to deemphasize its typically massive appearance

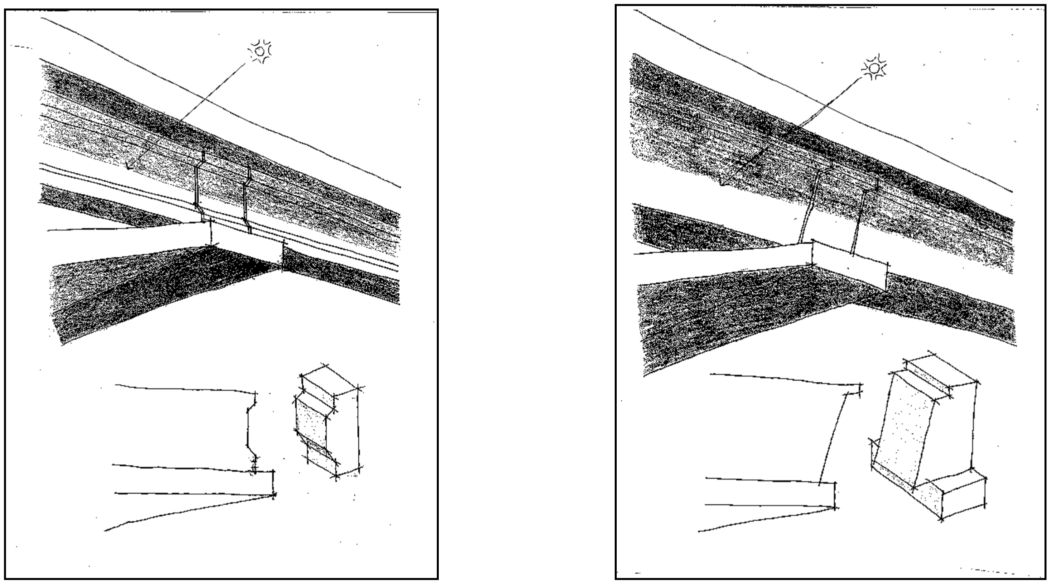


(a) (b)

Bent cap chamfering to de-emphasize large reflective surfaces and provide sculptural transition between the superstructure and substructure.

Figure 4.62 Bent cap end chamfering

Figure 4.63 Continuation of chamfers from column to bent cap for a well-integrated form



(a) (b)

Figure 4.64 Bent cap ends formed to match the shape of concrete I-girders (a) and U-beams (b)

Figure 4.65 Inverted-T bent cap stem shaped to mimic the concrete I-girders it supports

For balanced proportions between caps and columns, inverted-T bent caps should have column widths at least equal to the stem width as seen in Figure 4.66. If the inverted-T stem is wider than the column, chamfering can be used as an optical correction. The bent cap end may be chamfered so that the flat reflective surface (the surface between the chamfers) is the same width as the supporting columns (Figure 4.67). Chamfering bent cap ends to complement chamfered columns is a good option for well-integrated design.

Figure 4.66 Balanced appearance of a column and bent cap stem of equal width

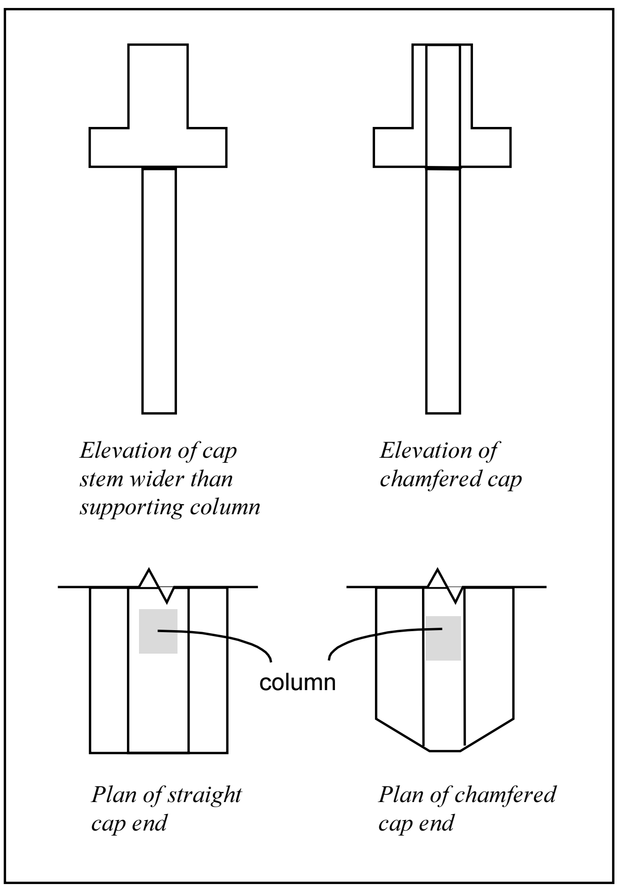


Figure 4.67 Comparison of bent cap end treatments

Nonstructural walls such as those in Figure 4.68 are sometimes used at the end of bent caps to cover an inverted-T or any gap between two simply supported beams. These walls are often poorly proportioned. These walls also call attention to the joint. The emphasized joint accents the simply supported superstructure but disrupts the horizontal visual flow of the bridge. Figure 4.69 shows the halting route of the eye across a bridge with bulky cap ends. Figure 4.70, on the other hand, shows how nonstructural walls can result in a clean, metered appearance. If nonstructural walls are desired, the walls must be kept in proportion with other elements of the bridge.

(a)

(b)

A thin wall at the end of a bent cap covers the gap between simply supported girders (a). The wall is unobtrusive in the oblique view (a) but gives the cap a disproportionately large appearance from the side (b).

Figure 4.68 Poor proportioning

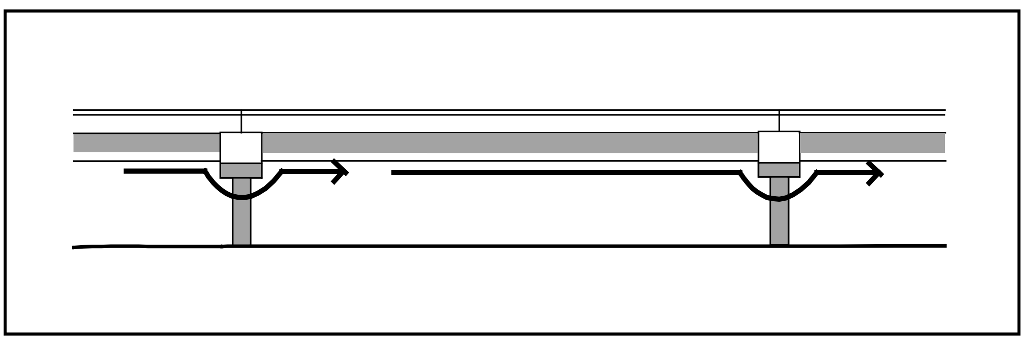


Figure 4.69 A disrupted visual flow of the horizontal superstructure

(a) (b)

These nonstructural walls (a) are the same width as the bent columns (b) giving the superstructure/substructure transition a neat and clean appearance.

Figure 4.70 Well-proportioned nonstructural walls

4.3 Chapter Summary

Designing the substructure of a bridge can be a fanciful display of imagination and expression or a simple expression of elegance. The designer's challenge is to consider the efficiency, aesthetics and economy of the design in order to meet the users' needs (functionality) within the project constraints. The choice of substructure system may be controlled by the superstructure span lengths chosen, support locations, column heights, foundation conditions or the superstructure width. In light of the constraints, a variety of economical substructure systems should be considered. The many standard options include individual columns, walls, and hammerhead or multicolumn bents. Bent caps on piers may be rectangular or inverted-Ts. Piers with fully integrated bent caps are also an option. Key aesthetic issues to consider when choosing an appropriate substructure system are structural expression, visibility through the bridge, integration of the substructure to other bridge elements and the bridge site, and enhancement through attention to nonstructural details. With thoughtful consideration, an attractive and well-suited substructure will greatly enhance the overall appearance of standard bridge systems.

CHAPTER 5

APPLICATIONS OF RESEARCH

5.1 Introduction

The research conducted for CTR Project 0–1410 has been applied to a number of bridge projects. The research team had the opportunity to apply the principles of the *TxDOT Aesthetics and Efficiency Guidelines* (*Guidelines*) [3] to two current TxDOT projects, one in San Angelo, TX and one in Wichita Falls, TX. For both projects, the use of precast substructures was also explored. In addition, the *Guidelines* were applied to four existing or potential bridges in Texas for a series of examples by a member of the research team, Steve Ratchye, who has advanced degrees in both architecture and structural engineering. [8] The examples were subsequently revised by the project staff. These studies are included in a section titled "Examples" in the *Guidelines* (Reference 3).

This chapter will describe the application of the research to the San Angelo and Wichita Falls projects. The resulting impact on the aesthetics and costs on these bridge projects will be discussed.

5.2 Research Application to US Highway 67 in San Angelo, TX

In November of 1994, the research team was invited to offer aesthetic recommendations for a bridge project in west Texas, US Highway 67 in San Angelo. Time was a major constraint for involvement in this project. The bridge contract was to be let in May 1995 requiring design plans to be finalized in January 1995. The research team's involvement was therefore limited to approximately 4 weeks.

The US 67 project had been in the planning stages for about 20 years before action was taken to finalize the design for construction. The impetus for finalizing the design came from partial federal funding to experiment with the use of high performance concrete (HPC) for pretensioned girders. Using high performance concrete with its higher than average compressive strength, would allow *AASHTO* Type IV girders to span up to 45 m (150 ft), 6 m (20 ft) more than similar normal strength concrete girders would span. Research Study 9–589 of the Center for Transportation Research was involved with the project. [61] One stipulation of this study was that attention be paid to the aesthetic impact of the bridge substructure on the Concho River park. A proposal was made to use precast caps to support the I-girders and to use concrete with a compressive strength of 55 MPa (8000 psi) in the substructure. The contractor was to be given the option to either precast the columns or cast them in place.

At the time of the research team's involvement, the preliminary *Guidelines* by Listavich [6] were being developed. Preliminary developments of the precast substructure system by Barnes [7] were also underway. The US 67 Project therefore was an excellent opportunity for the research team to explore the implementation of its research. Involvement in this project severed as a trial application of the ideas to be presented in the *Guidelines* and the suitability of precasting substructures for standard highway bridges.

5.2.1 Project Description

The US 67 Highway Project involved the design and construction of bridges and highway in southeast San Angelo. Of particular interest to the research team were the planned twin elevated bridge structures crossing the Concho river. These structures would also span a park along the river, ATSF railroad tracks, and US Highway 87 (Figure 5.1 a and b). The planned elevated structures were to run between the existing US 67 northbound and southbound highways. The existing roadways would become the new highway's frontage roads. The twin structures were to be raised above and between the two existing roadways so as to span the US Highway 87 crossing (see Section A-A in Figure 5.1b). The new bridges would therefore be approximately 4.5 m (15 ft) above the existing US 67 bridges crossing the Concho River (Figures 5.2).

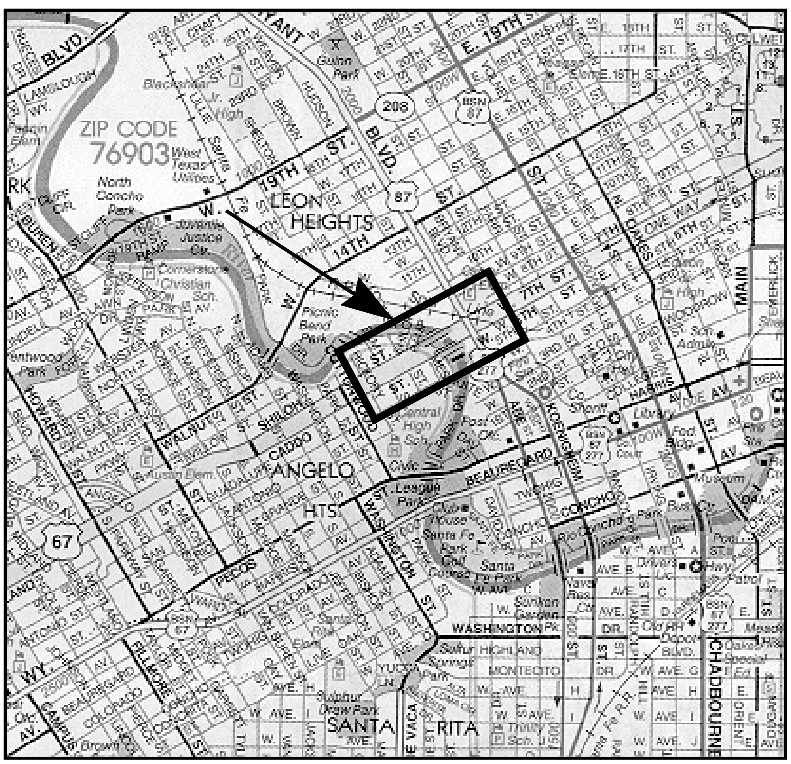


Figure 5.1a Partial map of San Angelo showing proposed bridge site

Structure: Two two-lane elevated bridge structures spanning a railroad track, US Highway 87 and the Concho River with a roadside park. Existing US 67 lower level bridges over the park to be used for frontage roads.

Project Site:

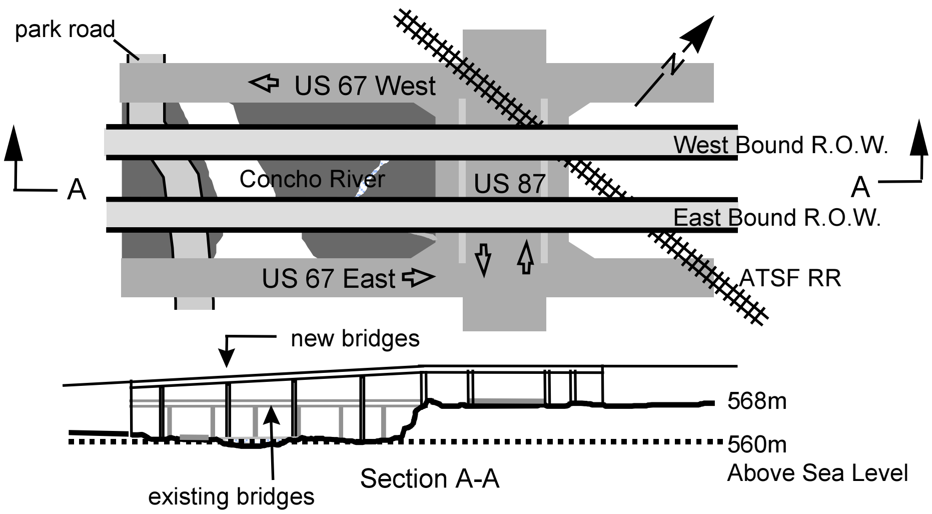


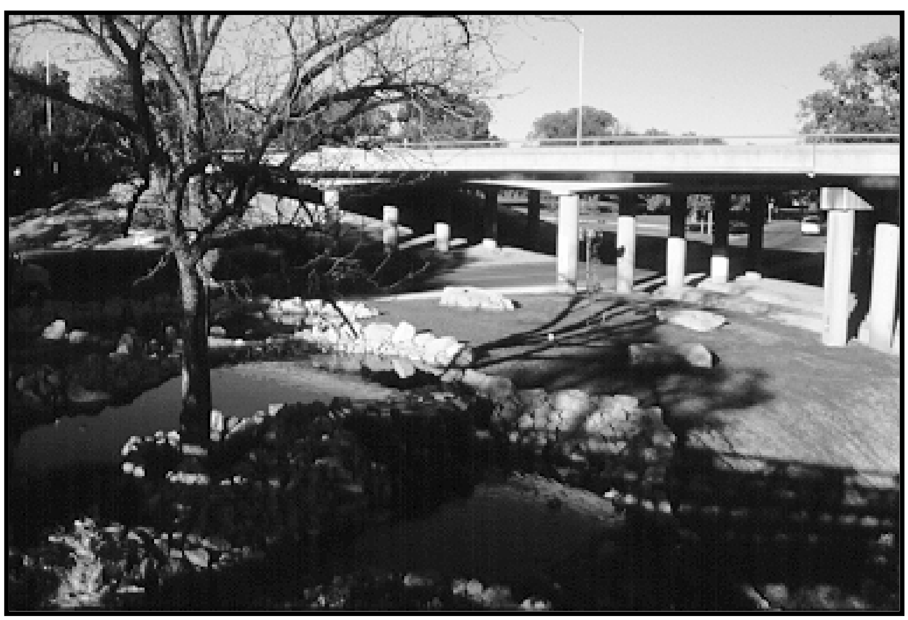
Figure 5.1b Site plan and section showing location of the future US 67 mainlanes

Figure 5.2 Location of the future US Highway 67 mainlanes

5.2.1.1 Site Characteristics

The research team walked the site in November 1994. Photographs and site observations were recorded. Site characteristics that were particularly important to the new bridge design were the attractiveness and openness of the park area (Figure 5.3), and the diagonal crossing of the ATSF Railroad through the existing US 67 and US 87 intersection (Figure 5.1b).

(a)



(b)

Figure 5.3 (a, b) An attractive park runs along the Concho River through the city of San Angelo

The park area of this bridge site is part of a larger park that winds with the Concho River through much of San Angelo. There is a small park road that runs beside the park and river. Pedestrian paths also run throughout the park with many picnic tables and benches along the way (Figures 5.2 and 5.3a). There are a number of bridges crossing the river and park as well. The age, character and function of these bridges varies. Each bridge seems to represent the era in which it was built. Some of the bridges exhibit older craftsmanship, typical of the WPA era (Figures 5.4 and 5.5). Other bridges represent the changing design trends of the Texas DOT (Figures 5.6 and 5.7). A few pedestrian bridges cross the river as well (Figures 5.8 and 5.9). Many of the engineers of these previous bridges embraced the attractive park setting by striving to enhance the park with their designs. The public too, has become actively involved with the construction of these bridges. The pedestrian bridge of Figure 5.9 is lined with plaques funded by local residents.

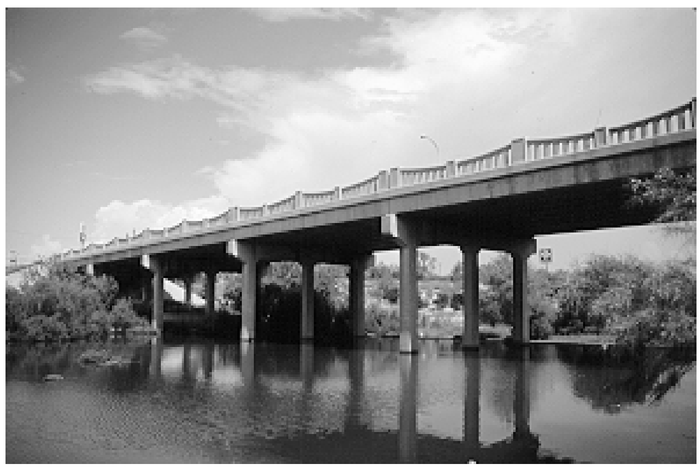


Figure 5.4 One of the older bridges crossing the Concho River in San Angelo

Figure 5.5 One of the older bridges crossing the Concho River in San Angelo

Figure 5.6 A cast-in-place slab bridge crossing the Concho River

Figure 5.7 A precast prestressed concrete I-girder bridge crossing the Concho River

Figure 5.8 One of many pedestrian bridges crossing the Concho River

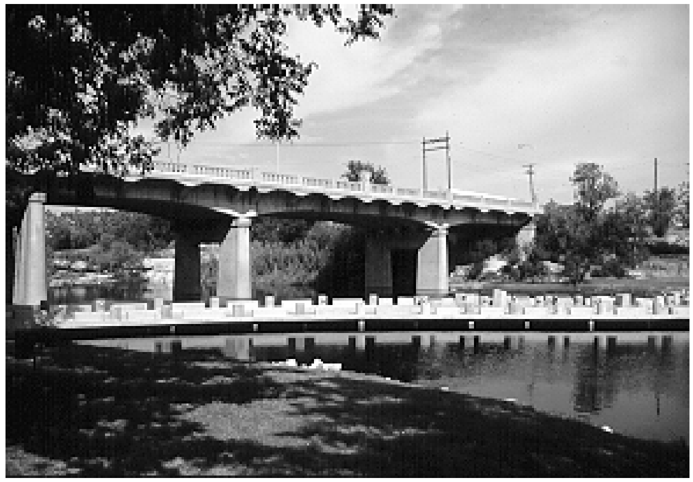


Figure 5.9 In the foreground, a low pedestrian bridge across the Concho River

Another key observation made during the site visit was the potential for danger at the intersection between the highways and the railroad at the very open and flat north end of the site. The railroad did not appear to be heavily traveled. However, the future bridge abutments to the northeast of the railroad tracks would create a wall dividing the area and restricting visibility. In particular, westbound US 67 frontage road traffic would have limited view of westbound railroad traffic with the bridge abutments in place (this is further discussed in Section 5.2.2).

The site observations gave the research team a good feel for the effect the new bridges would have on the site. The importance of the park to the city must be recognized. Attention must be given to the visual impact the future bridges will have on the site. At the same time, public safety in terms of visibility around the bridge, must be considered.

5.2.1.2 Pre-Existing Design Plans

While the project was in planning stages during the past 20 years, the geometric design (roadway alignment) was completed by engineers in the San Angelo District. When the project was revived, the plans were sent to the Design Division in Austin and preliiinary bent locations were established. This design, referred to herein as the original layout is shown in Figure 5.10.

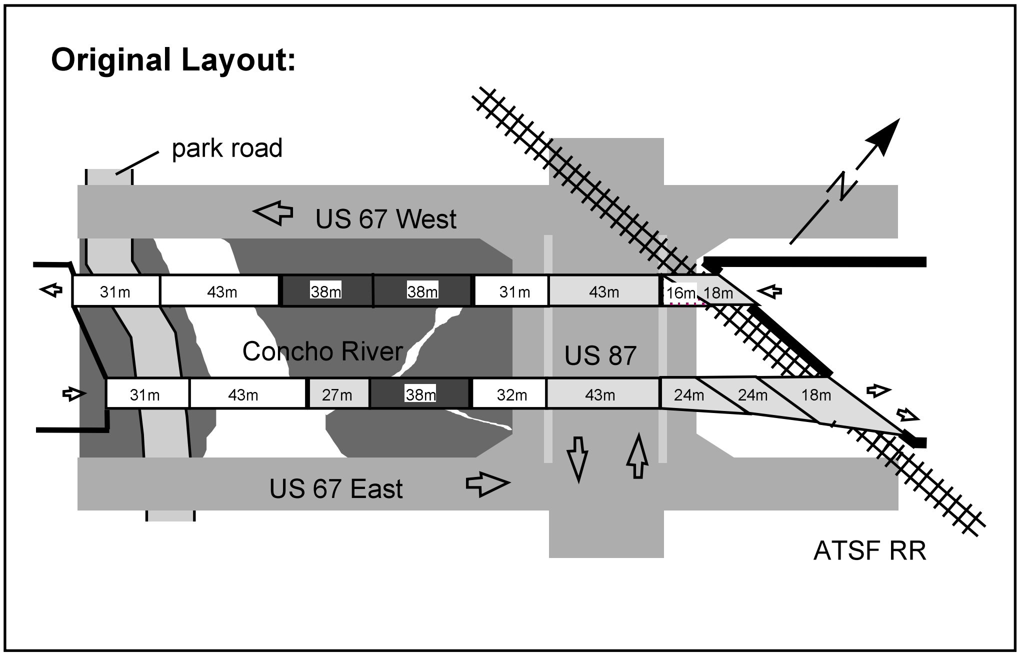


Figure 5.10 Original layout for US 67

CTR Project 9–589, introduced span lengths of up to 43 m (140 ft) for experimentation with *AASHTO* Type IV HPC girders. These HPC girders were to span the river to avoid foundation nd substructure construction in the water, and to span US 87. Type IV girders of varying span lengths were to be used throughout the project except for the spans over the railroad tracks where increased vertical clearance requirements dictated the use of shallower Texas Type C (*AASHTO* Type III) girders.

5.2.2 Application of Aesthetics Guidelines Principles

While involvement on this project occurred before the completion of *the TxDOT Aesthetics and Efficiency Guidelines* (*Guidelines*) many of the ideas and principles addressed throughout the development of the *Guidelines* were applied to the US 67 Project. The proposed improvements are presented here in the order that such improvements should be addressed in original design. This follows the order for suggested improvements in the *Guidelines* [3].

5.2.2.1 Problems Identified with the Original Layout

In balancing the observations made during the site visit with a review of the original layout, a number of problems were identified with the aesthetics and efficiency of the proposed project. Problems were identified with the layout and with both the superstructure and substructure design. In terms of aesthetics, a number of possibilities for attractive design could be explored.

The first problem was the number of different span lengths used. Roughly five general span lengths are used for a total of 17 spans (Figure 5.10). Frontage road traffic traveling alongside the bridge will experience these varying span lengths as a disharmonious jumble. In particular, rhythm is visually disrupted by the random location of bents. Another layout problem was the location of the bridge abutments.

The western abutments were directly up against the park road, looming over and dominating the otherwise open riverside park. On the eastern side, the abutments ran directly alongside the railroad tracks. The combination of this skewed abutment wall and the skewed bents would create a dark tunnel ("Thug Alley") in this open setting (Figure 5.11). As mentioned previously, the large abutment created a dangerous intersection as it restricted the views of westbound US 67 frontage road traffic and westbound train traffic.

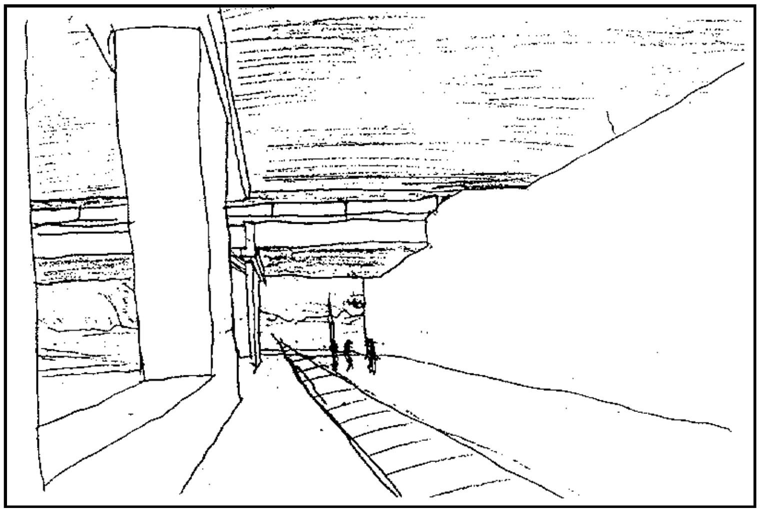


Figure 5.11 Artist's rendering of the tunnel-like space created by skewed bents (left) next to a large skewed abutment wall (right).

The skewed bents in the layout also create aesthetic problems with the superstructure and substructure design. In elevation, skewed bents create a jumbled appearance (see Figure 4.37). In the case of US 67, the skewed bents require multicolumn bents whereas the rest of the project was designed for single-column bents. (Single-column bents were part of the original plan to improve aesthetics through minimizing substructure clutter). The resulting clutter on one end of the project detracts from the project as a whole. It is inconsistent to be concerned with aesthetics only on one side (in this case, the park side) of a project. To have a relatively minor railroad track dictate such unattractive bridge design, displays a lack of awareness to aesthetic issues (and more attractive design alternatives).

A final problem identified was the use of different depth girders to provide the increased vertical clearance required for railroad traffic. When using I-girders on inverted-T bent caps, the change in superstructure depth can be particularly disruptive visually at the bent cap end (Figure 5.12).

Figure 5.12 A change in superstructure depth highlighted by the awkwardly shaped bent cap end

5.2.2.2 Layout Improvements

The first proposed improvement to the layout was to use more spans of similar length throughout the project (Figure 5.13). Reducing the variety of span lengths allows for more harmonious viewing particularly by frontage road traffic. Using consistent longer span lengths also solves other problems identified. Longer span lengths can allow the abutments to be pushed back away from the park road on the western end of the project and away from the railroad on the eastern end. Increasing superstructure lengths reduces the number of substructure supports. Pushing the abutments back keeps both the park area and the railroad intersection area more open. Safety is improved with the increased visibility between westbound US 67 frontage road traffic and westbound train traffic. Lengthening the spans when combined with slightly increased bridge elevation allows the bridge to span the railroad tracks with proper vertical and horizontal clearance without using skewed bents. With this layout solution, the bridge stands on its own, unaffected by yet safely spanning the railroad. The result is a more open and harmonious design.

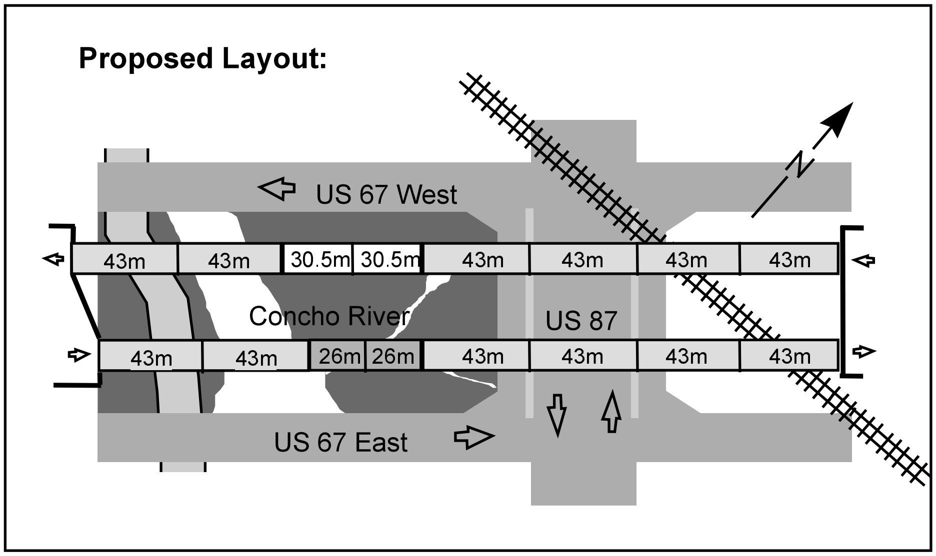


Figure 5.13 Proposed layout for US 67 with abutments pushed back, more even span lengths and no skewed bents

Using similar span lengths and avoiding skew bents removes the need for multicolumn bents. Single column hammerhead bents can be used consistently throughout the project. Asymmetrical bent caps (Figure 5.12) are avoided by raising the bridge elevation to provide adequate vertical clearance for the train. In the original design, the deck elevation dictates the need for shallow beams. This in turn dictates the need for shorter spans and unattractive, uneven bent caps. All of these drawbacks are easily removed by raising the deck elevation (between 250 and 375 mm (10 to 15 in)) and using longer spans of similar depth to the rest of the project. An additional advantage to using longer spans over the tracks is that one entire foundation and substructure unit is no longer required.

5.2.2.3 Superstructure and Substructure Design Suggestions

As discussed in the previous section, superstructure design is made more consistent through the use of similar span lengths and girder types. The shallow Texas Type C beams are no longer required thus avoiding the unattractive transition between different beam depths. Multicolumn skewed bents are avoided as well.

Due to the late introduction of the research team to this project, it was recognized that the TxDOT designers were under a strict time constraint to complete the project. TxDOT seemed resistant to the proposed raising of the deck elevation and the work this would involve in order to avoid using shorter span Type C girders. As a result, the research team also explored ways to improve the appearance of the railroad crossing in the event that the designers did choose to keep skewed bents and a combination of Type IV and Type C girders.

As bent cap ends are highly reflective surfaces, unattractive asymmetrical ends are a particular eyesore. To minimize this eyesore, the research team proposed that concrete pedestals on top of the inverted-T ledge be used to support the shallower beams. The bent cap itself would remain symmetrical and match the other bent caps in the project (Figure 5.14). The use of tapered pedestals would be a further improvement to the appearance of the shorter, shallower span (Figure 5.15). Another suggestion was to provide columns for the skewed bents that would correspond to both the bents orthogonal to the direction of the bridge traffic and the skewed railroad. This alignment would help integrate the skewed bent with the rest of the bridge supports (see Figure 4.39).

Figure 5.14 A symmetrical bent cap with pedestals to accommodate changing superstructure depths

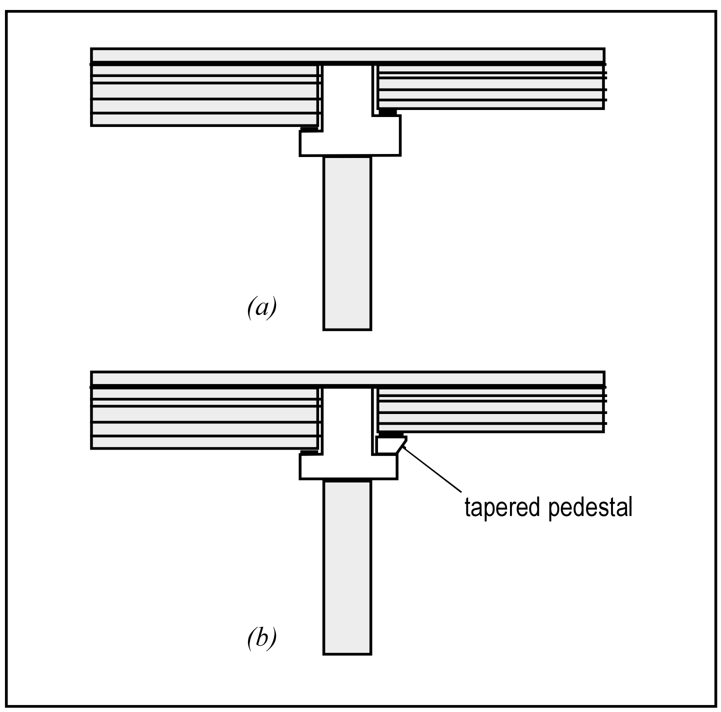


Figure 5.15 An improvement over uneven bent caps (a) is a symmetrical cap with tapered pedestals (b). Tapered pedestals offer an attractive transition between precast girders and inverted-T caps.

Aside from calling for mostly single-column hammerhead bents, the substructure design was not yet finalized before the research team became involved with the US

67 Project. The TxDOT Project Director Norman Friedman of the TxDOT Bridge Design Division in Austin designed a windowed pier for use on the San Angelo project (Figure 5.16). As mentioned previously, high performance concrete was to be used for the substructure with precast bent caps. Precasting was also an option for the columns. The research team explored a number of column shapes for precasting (see Section 5.2.3).

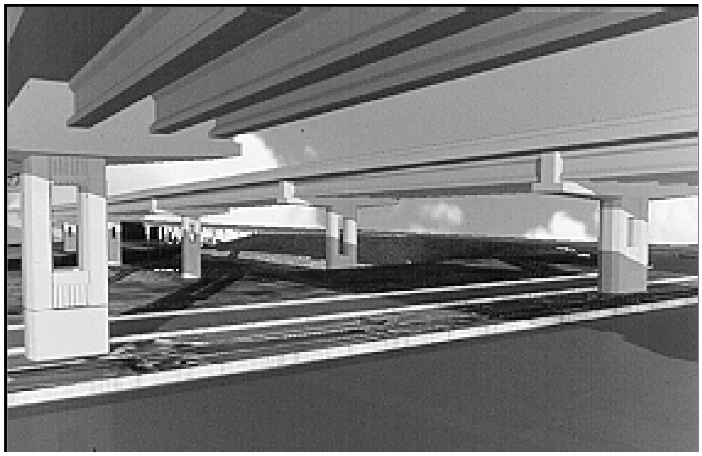


Figure 5.16 A rendering of the proposed bridge with windowed piers

5.2.2.4 Nonstructural Details

There are many ways that the bridge project as a whole could be enhanced with attention to nonstructural details. One suggestion was to use a consistent formliner pattern in the substructure. In particular, a pattern should be used which in combination with the substructure shape would have enough detail to provide a comfortable feeling for the park users. In such a narrow park, the large single columns could seem overwhelming without textural relief.

An open rail for the motorists above was suggested to allow for better views of the area and also to lighten the apparent slenderness of the structure from a distance. Finally, drainage pipes should be internal to the substructure sections.

5.2.3 Potential Use of Precast Substructures

The desire to experiment with high performance concrete (HPC) in the substructure made a precast option very attractive. The increased compressive strength of the HPC allowed for hollow sections which would reduce foundation costs and, if precast, allow for easy handling of the segments. Even precasting only the cap would simplify the erection process by avoiding the need for setting up cap formwork and placing concrete at the higher elevations (roughly 12 m, [40 ft]) over the river. The precast cap proposed at the time was too heavy to be lifted as a single unit with the cranes on site for the girders. Therefore, a segmentally precast cap was proposed similar to that of Proposal I presented in Section 3.4. Different precasting options were explored for a basic column shape that had both a solid and a windowed option (Figure 5.17).

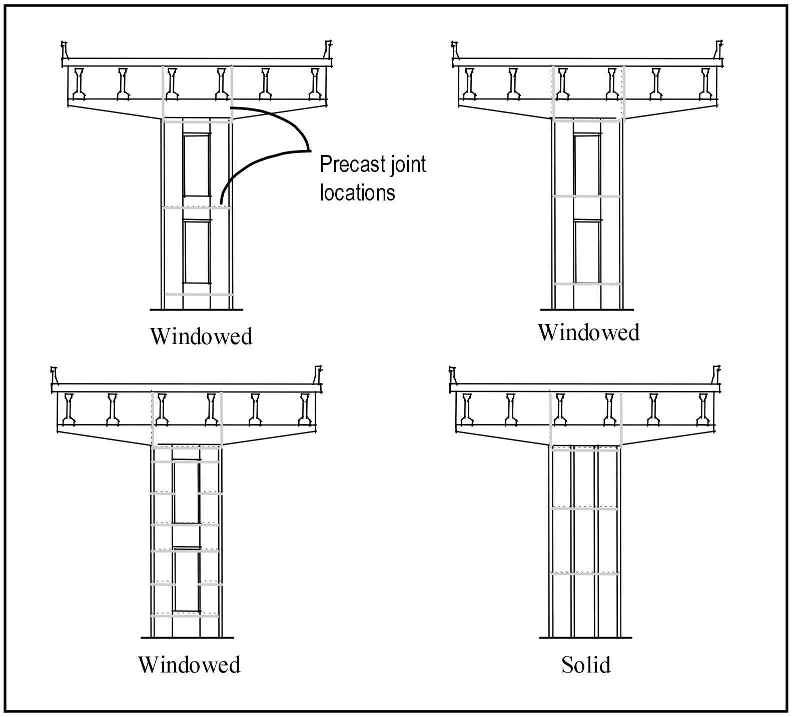


Figure 5.17 Precasting options explored for windowed and solid bents

5.2.4 Final Design

A number of the suggestions offered by the research team were used for the final TxDOT design. The final layout is shown in Figure 5.18. An effort was made to standardize the span lengths of the overall bridge. This effort was particularly successful on the western end of the project over the park and river. However, due to time constraints, TxDOT designers were indeed unwilling to change the vertical elevation of the bridge over the railroad tracks. As a result, shallower, shorter-spanning Type C beams were required. This unfortunately dictated the need for skewed bents.

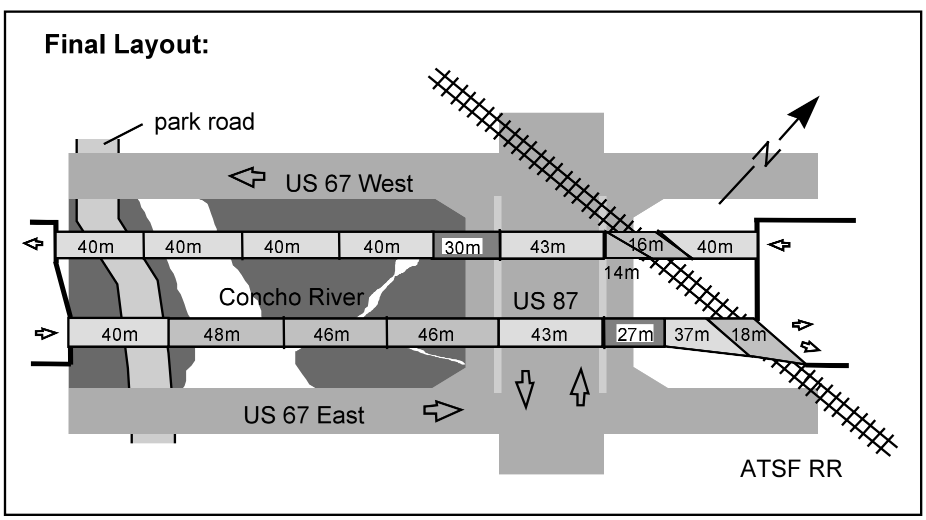


Figure 5.18 Final layout for US 67

However, pedestals to support the shallower beams were chosen in place of an asymmetrical bent cap. This change helps reduce the visual abruptness of the changing superstructure depths at the cap. Using a greater number of longer spans allowed for the removal of one substructure unit and also allowed the designers to push the abutments back from both the park road and the railroad tracks. This change lightened both of these areas and improved visibility and therefore, safety at the railroad/motorist intersection.

The details of a precast substructure system had not been fully worked out at the time design had to be finalized. Thus, this option was met with skepticism by TxDOT designers. As a result, the substructures were designed to be cast in place. A modified windowed pier was selected. The bottom "window" for the taller columns in the original design (Figure 5.16) was filled in to comply with flood provisions in the area. An attractive fractured fin formliner was chosen as an inset to the columns (Figure 5.19).

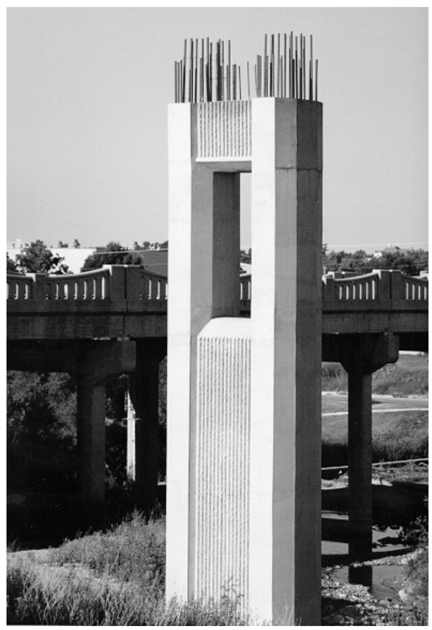


Figure 5.19 A windowed pier under construction for US 67

The final appearance of this bridge represents some of the newer design capabilities of its time, particularly with the long-span high performance concrete I-girders. Many of the less attractive aspects of the bridge could have been avoided with more forethought. A more developed and detailed precast substructure system could have been used with attractive results.

5.2.5 Economic Impact

The economic impact of the proposed improvements was examined after the project was bid. Only the proposed changes that were actually implemented were examined. The bid prices were compared to typical costs in the area for what would have been constructed had no changes been made. The three major changes examined were the trade-off between increased superstructure lengths and decreased abutment requirements, the removal of one substructure unit and the use of a windowed pier. The results are shown in Table 5.1.

Pushing the abutments back resulted in savings. The size of the retained earth wall was large enough that its cost exceeded the cost of longer girders. Removing one substructure unit certainly resulted in savings. No adjustments were considered for foundation costs although they would most likely have decreased as well. The windowed pier was bid at a higher price than normal single-column bents.

Table 5.1 Cost impact of Project 0-1410 proposed changes for US 67 in San Angelo

Cost Impact of Proposed Improvements on Final Layout for US 67 in San Angelo, Texas*

1a. Push western abutments back from the park road by 12m (40ft.)

Add superstructure	+$22,200
Remove retaining wall	-$22,620
Remove earthwork	-$22,150
Total Change	**-$22,570**

1b. Push the eastern abutment back removing 20m² (215 ft²) of retaining wall.

Add superstructure	+$72,450
Remove retaining wall	-$118,755
Remove earthwork	-$58,170
Total Change	**-$104,475**

2. New span lengths proposed - one span and therefore one substructure unit are removed.

Remove one substructure unit	-$27,500
Total Change	**-$27,500**

3. Use windowed single column piers.

Cost of one windowed pier	$36,135
Less cost of typical 3-column bent	$29,650
Add'l cost per substructure unit	+$6,500
Total Change (15 piers)	**+$97,500**

Total cost of all changes (negative values indicate savings)

1. Push abutments back	-$127,045
2. Remove one span	-$27,500
3. Use windowed pier	+$97,500
Total change in cost	**-$57,045**

Cost Impact in Terms of % of Bridge and Project Costs:

Total Bridge Cost	$3,580,000
Total Project Cost	$11,753,000

Proposal	% Total Bridge	% Total Project
1. Push abutments back	-3.6	-1.1
2. Remove one span	-0.8	-0.2
3. Use windowed pier	+2.7	+0.8
Total Change	**-1.6**	**-0.5**

*negative values indicate savings

It should be recognized that bid prices are *not* necessarily a true reflection of actual cost. Rather, contractors are able to recover potential costs elsewhere in a project by adjusting bid prices. New forms of construction provide contractors with an opportunity to increase bids in that area whether or not the work will cost them more. In the case of the San Angelo project, an overall savings was reflected in the bid prices for the sum of the proposed changes. While the unfamiliar substructure seemed to increase prices, improvements which resulted in a more harmonious layout and increased visibility around the structure afforded savings. The overall result was a probable savings in bridge cost of 1.6% and a savings in total project cost of 0.5%.

5.2.6 Section Summary

Involvement in the San Angelo project was very useful for the continuation and refinement of the research for Project 0–1410. The majority of the lessons learned were beneficial for further development of the *Guidelines*. Observations and comments on precasting of substructures provided important guidance for further development as well.

Applying principles from Listavich's *Aesthetic Guidelines* [6] gave the research team a better understanding of the design process. The importance of a site visit was clear. Physical constraints need to be recognized from the outset for successful bridge design. Physical constraints must be balanced with procedural constraints (in the case for San Angelo, time and roadway elevations) so that all of the constraints become boundaries and not obstacles.

Application of *Guidelines* principles was seen to improve aesthetics and safety while achieving monetary savings. While certain ideas led to cost increases, the amount of the increase was very little in terms of overall bridge cost and even less in terms of overall highway project cost. The considerable improvement in aesthetics outweighed the minor cost increases. The public infrastructure appearance is improved along with the quality of life in the area. Involvement in this project demonstrated the practicality and possibilities for economical and aesthetical success which thoughtful application of the *Guidelines* can provide for the design of standard short- and moderate-span bridges.

It was also clear that any new precast substructure system would need to be carefully designed and detailed for adoption into the practice. Details of constructibility, including both fabrication and erection, as well as workable connection details would be essential. The weight of substructure elements must be kept down to ranges that can be handled with conventional lifting equipment used for handling long girders in order to improve the efficiency of fabrication and erection. Difficulties were recognized in terms of assessing the costs of a proposed precast substructure system. Difficulty arises primarily in quantifying the benefits of minimizing environmental impact, avoiding traffic delays and improving bridge aesthetics.

5.3 Application of Research Findings to US Highway 287 in Wichita Falls, TX

In February of 1996, the research team was invited to make a presentation of Project 0–1410 research to the TxDOT engineers in the Wichita Falls District Office. The district office was in the planning stages for twin elevated highway structures that were to pass through a business district near downtown Wichita Falls. The existing highway is heavily traveled and the traffic signals in the area resulted in frequent traffic congestion. The research team was invited to offer comments and suggestions for the design of the planned elevated structures. Due to the businesses and parks in the area, aesthetics was an important concern of the public and of the designers. Speed of construction and avoiding extensive traffic delays or re-routing were major concerns due to the heavy traffic volume and in the interest of helping the local businesses survive.

After the initial visit and presentation, a few members of the research team returned one month later to give a similar presentation at an open civic meeting. Again, the research project was presented, but with more emphasis on suitable solutions for the design of US 287. The presentation was well received. While the audience of local engineers, politicians, businessmen and interested residents learned more about potential designs, the research team gained a better understanding of the interests of these various groups when a bridge project is developed. It was clear how important it is for engineers to be able to communicate their ideas and expertise clearly and convincingly to the public and to be sensitive to public opinion and concern in order for good bridge designs to result.

As the elevated highway ties in to a portion of interstate highway, some federal funding was available to the district. However, most of the funding for this project would come from the funding that was allocated to the district by the state. Traffic projections for the future indicated that ultimately three lanes will be required for each direction. The district therefore had to decide whether to design three-lane structures up front or to design two-lane structures with capabilities for future widening.

At the time of the research team's involvement, Listavich's preliminary *Aesthetics Guidelines* [6] were complete. A final version, *The TxDOT Aesthetics & Efficiency Guidelines* (*Guidelines*), was being developed. Barnes' precast substructure system [7] (Proposal I in Chapter 3) was nearing completion with many of the fabrication and erection details worked out. The US 287 Project therefore provided a new opportunity for the research team to explore further implementation of its research. Unlike the San Angelo project involvement, this project was still in an active, open stage of development when the research team became involved. Principles from the *Guidelines* could be more fully applied and the feasibility and suitability of using a precast substructure system for this project could be explored.

5.3.1 Project Description

A plan of the proposed project can be seen in Figure 5.20. The portion of US Highway 287 being designed had been planned for in previous highway construction. The new construction will connect previous stretches of US 287 from the south to US 287 and IH-44 to the north. Stub-outs exist on the south end of the site where the new elevated structures will tie in (Figure 5.21). As shown in Figure 5.20b, the elevated structures being designed are separated by one wide (183 m, (600 ft)) city block. At either end of the project, the elevated structures come together side by side to join the existing highway structures.

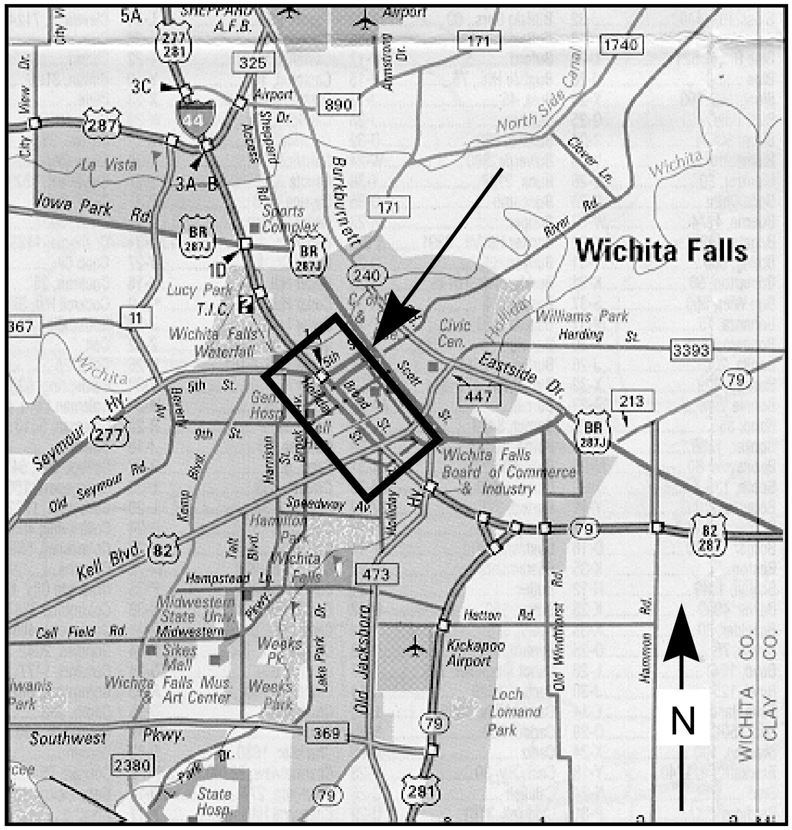


Figure 5.20a Partial map of Wichita Falls showing proposed bridge site

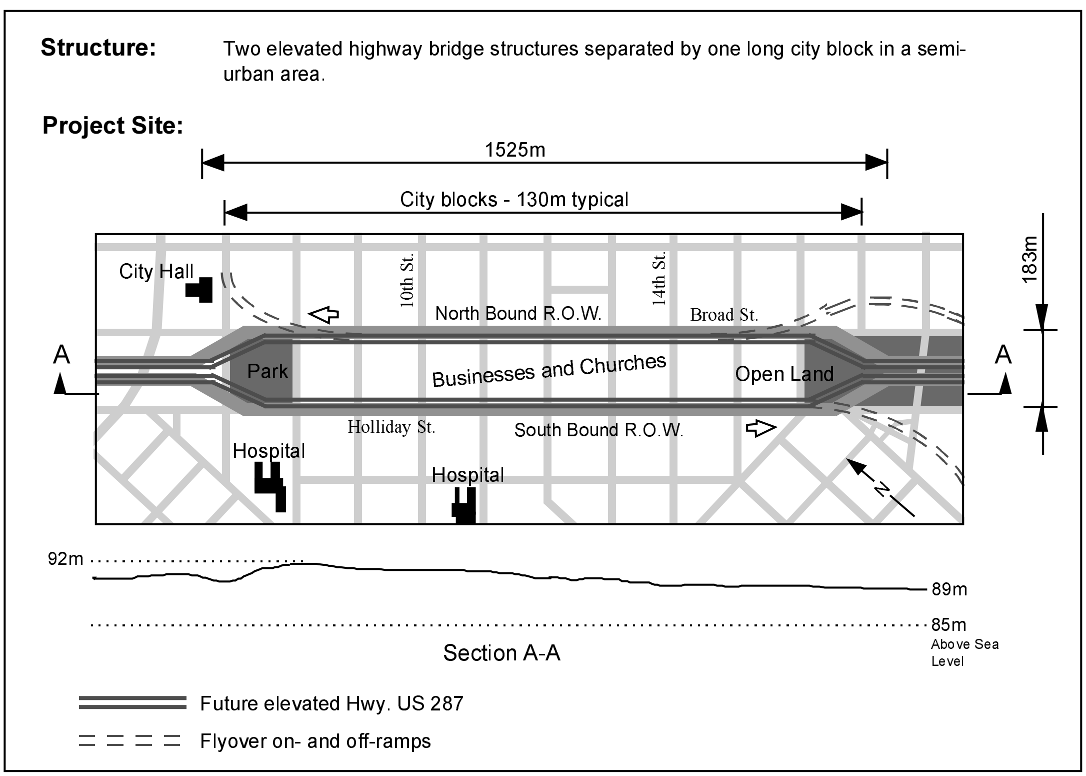


Figure 5.20b Site plan for US 287 project in Wichita Falls

Figure 5.21 Stub-outs on the south end of the US 287 project (view looking south)

Curved on and off-ramps to adjoining highways and to the downtown area are required as well. Flyover ramp structures must be provided to connect this project on the south end to an east-west highway, US 82, known locally as Kell Boulevard. At the north end of the northbound structure an off-ramp to the downtown area was requested by city officials. An on-ramp to the southbound structure at the north end is desired but not required at this time.

As shown in Figure 5.22, the new elevated structures will take up one of the four existing lanes of the city roads now being traveled on for US 287 (Broad St. for northbound traffic and Holliday St. for southbound traffic). The final three-lane structure will overhang part of the street as well as the existing sidewalk.

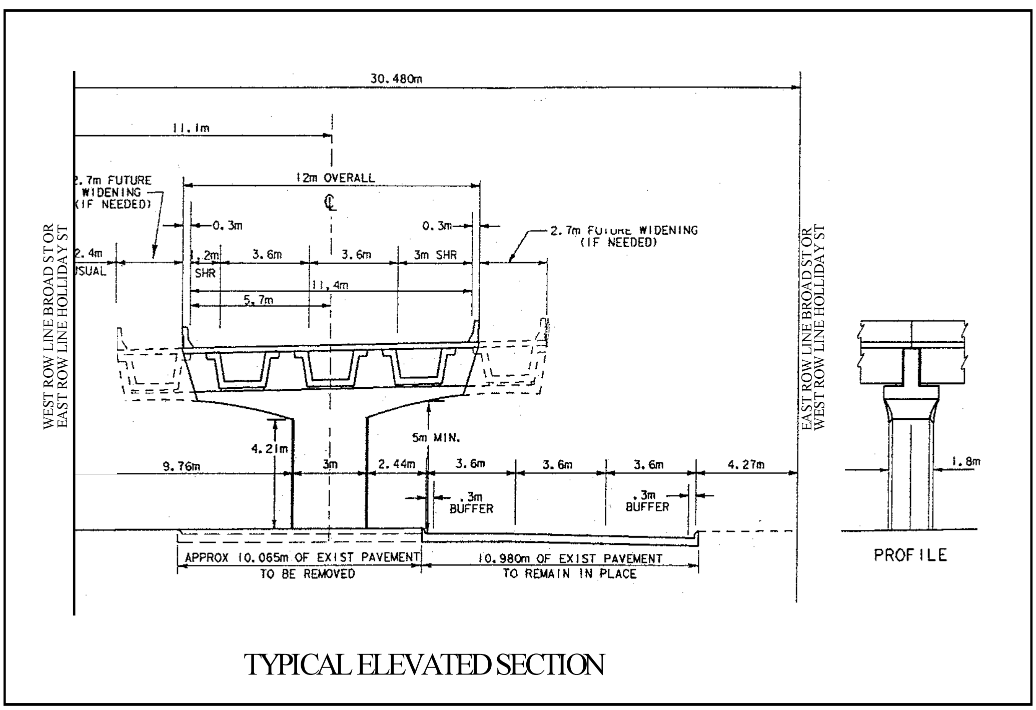


Figure 5.22 Future elevated highway built on existing roadway. This figure shows a single-column bent option with the possibility for future widening.

5.3.1.1 Site Characteristics and Constraints

The majority of the structure runs over a city grid of 130 m (430 ft) blocks. As the north and southbound structures are separated by a wider 183 m (600 ft) city block, the possibility of these structures forming a “wall” between two sides of town is reduced (see Figure 5.39). A three-lane structure will appear lighter and allow for more light to reach underneath it than a six-lane structure would.

There is an existing park on the north end, the Harold Jones Park (Figure 5.23). Although not heavily used, a number of people were seen enjoying lunch at the park's picnic tables. At the south end is a park-like area of open land that is owned by the state (Figure 5.24). The businesses in the area seemed to be surviving but with low activity. In talking with city residents, it was discovered that while the city is making attempts to revive the downtown area to the northeast of the site, most business development is occurring southwest of the site. This strip of businesses in the area of the proposed bridges is a bit of a “no-man’s land.” However, there are a number of hospitals, churches and municipal buildings located among the small businesses and restaurants in the area (Figures 5.25 and 5.26). Due to the variety of activity, the research team felt that there was potential for this area to eventually be revived (in a span of at least 20 years) and to attract more use. It was essential therefore that an aesthetically pleasing solution be found for the bridge design, one that would enhance and not detract from the site. In particular, the bridge must be attractive from underneath and up close as the future users of the site would be pedestrians and slow-moving local traffic.

Figure 5.23 The Harold Jones Park

Figure 5.24 Open land at the south end ofthe site (view from stub-out looking north)

Figure 5.25 A local church

Figure 5.26 City Hall

The existing right-of-way dictates that the elevated structure will have its supports in the current right of way and overhang a major portion of the existing streets (Figure 5.22). Therefore, part of the road will need to be closed during construction. For substructure construction, only one lane will need to be blocked off. However, if a precast concrete girder superstructure system is chosen, three and possibly all four of the lanes will need to be closed for periods of time to haul the girders to the site and lift them into place with the required two cranes (Figure 5.27). This work will have to originate from the existing road as the other side of the structure is taken up by local businesses. Another observation about the right-of-way was that the new structures will often be very close to some of the existing buildings including one attractive older church (Figure 5.28).

Figure 5.27a Two cranes are required to place precast concrete girders

Figure 5.27b Heavy traffic on the existing 287 will need to be re-routed during construction

Figure 5.28 Sketch of where the new elevated highway will be relative to some of the local architecture.

5.3.2 Application of the Guidelines

Unlike the San Angelo Project, US 287 in Wichita Falls was in the beginning stages of final design when the research team was made aware of the project. This timing gave the team the opportunity to apply the *Guidelines* from an open stage of design rather than at the very end. First, a vision, or design concept, for the bridge was developed based on the observations made during the site visit. Three major points formed the design concept of the structure and needed to be addressed. They were,

1. The structures must enhance the site without creating a wall between the older downtown and the newer developments in the city.
2. The structures should maintain as light and open an appearance as possible to be attractive for future pedestrian view.
3. The impact of the structures both during construction and once completed should have a minimum negative impact on the local businesses.

The ideas for the design concept were then carried out in suggestions for all steps in the design process to produce a coherent design. Suggestions made at these different steps of design are presented in the following sections.

5.3.2.1 Planning and Layout

To carry out the design concept, the primary concern in planning and layout was to choose a layout that maximizes visibility through the bridge. To do this, three primary suggestions were made.

The first suggestion was to use single-column substructures. Single columns minimize substructure clutter and keep the area underneath the bridge quite clear. An added benefit of single-column bents is that they minimize construction disruptions by requiring less construction area for foundation placement than multicolumn bents require. The larger elements (column and cap) do require stronger formwork.

The second suggestion for maximizing visibility was to use the longest span lengths possible to minimize the number of substructure units. Even with single-column bents, a wall-like appearance may result from oblique angles if the columns are spaced too closely together (Figure 5.29). The city grid allows for the use of even span lengths throughout the straight portions of the elevated structures. Four spans of 32.5 m (107.5 ft) or three spans of 43.3 m (142 ft) could be used for each city block. The longer span lengths would increase visibility and reduce the number of foundations and substructure units required by 25%. Although stronger foundations and bents would be required, site disruption would be decreased through fewer required excavation locations. In the end, the span lengths chosen will depend on the type of superstructure chosen. This choice is further discussed in the next section.

Figure 5.29 An example of closely spaced columns forming a wall

The third suggestion for improving visibility through the bridge was to keep the bottom of the superstructure between 6 and 7.5 m (20 to 25 ft) above ground rather than the minimum of 5 m (16.5 ft) required for vehicular clearance. Rather than crowding the site, a slightly raised bridge allows more light to reach underneath the structure thus creating a more inviting space. This open feeling is particularly important for the continued use of the area by pedestrians.

5.3.2.2 Superstructure Design

To provide a light, long-span structure, three superstructure systems were considered. These were pretensioned concrete I-girders, pretensioned concrete U-beams and a post-tensioned segmental concrete box girder. Steel girders could also provide long-spans but are considerably more expensive than the concrete alternatives and would only be considered for any longer curved spans that I-girders and U-beams could not accommodate.

Pretensioned concrete I-girders have a few advantages for this project. The first is that they are very economical. The state average for pretensioned I-girder bridges in 1995 was \$310 per square meter (\$29/ft^2) of roadway surface. Another advantage is that I-girder bridges can easily accommodate variable-width roadways. I-girder bridges usually can be easily widened in the future.

Despite the advantages of pretensioned I-girders, their many disadvantages make this system a poor option for the Wichita Falls project. I-girder bridges are particularly unattractive from below. The narrow girders and reentrant corners create dark cave-like voids when viewed from underneath (Figure 5.30). When used, diaphragms between the girders add clutter. The sloped upper surface of the bottom flange provides a perfect spot for pigeons to roost. Pretensioned I-girders are also not suited for sharply curved alignments. The discrete chords of girders require shorter spans to fit sharp curves. Not only are span lengths limited, but substructure requirements are increased and unattractive scalloped shadows are visible on the girders during parts of the day (Figure 5.31).

Figure 5.30 View of an I-girder bridge from below

Figure 5.31 Scalloped shadows on curved bridges made up of straight girders

Pretensioned concrete U-beams are another very economical option. They typically cost the same as pretensioned I-girder bridges. U-beams are more attractive from below than I-girders. Their smooth underside and angled walls reflect more light, brightening the appearance from underneath (Figure 5.32). U-beams provide no pigeon-roosting surface along their length. U-beams can accommodate a certain amount of roadway widening or narrowing but are not as flexible a system as I-girders. The reason is that one U-beam takes the place of two I-girders. For sharp transitions in plan, two wider U-beams can only be placed on 2.5 m (8 ft) centers and flare out to 4.5 m (15 ft) centers. However, four I-girders could be placed on 0.6 m (2 ft) centers to flare out to 2.5 m (8 ft) centers if necessary. In the case of the Wichita Falls project there was one area where a transition in plan could not be accommodated by U-beams. An alternative would be to use a U-beam on the exterior and I-beams on the interior to accommodate the flare (Figure 5.33). Although not particularly attractive from underneath, the continuity of the bridge elevation with U-beams could be preserved.

Figure 5.32 View of a U-beam bridge from below

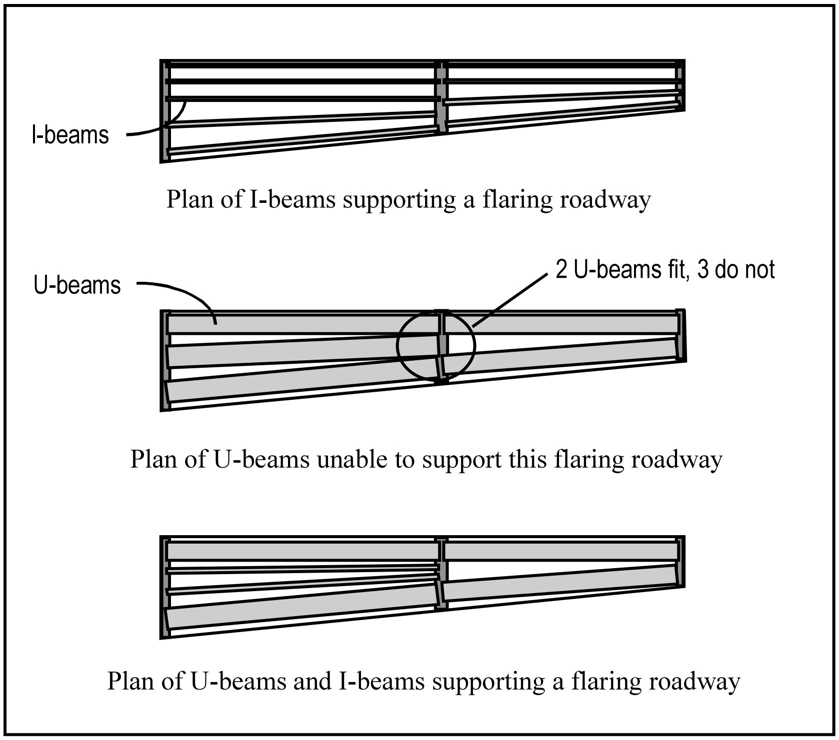


Figure 5.33 Accommodation of a flaring roadway with I-beams and U-beams

Another disadvantage is that similar to I-girders, U-beams are not well-suited for sharply curved alignments. U-beam bridges could be widened in the future. However, due to their larger width, they are not as flexible a system for future widening as I-girders.

A segmental box girder superstructure would provide the most attractive addition to the site. The smooth underside and thin wings optimize the openness and reflected light underneath the bridge (Figure 5.34). In particular, the vertical clearance is increased for the lower level frontage road motorists who experience the bridge for greater lengths than crossing traffic (Figure 5.35). Longer spans can be achieved with segmental box girders and continuous spans are possible. Segmental box girders are also very well-suited for curved alignments. The segments can be precast to fit curves. As a result, segmental box girders would provide dramatic and sweeping curved flyover ramps at the highly visible ends of the US Highway 287 project. The main drawback to segmental box girders is that it would be difficult to widen the girders. Thus, it would make the most sense to construct all three lanes initially.

Figure 5.34   A box girder bridge with thin, smooth overhangs

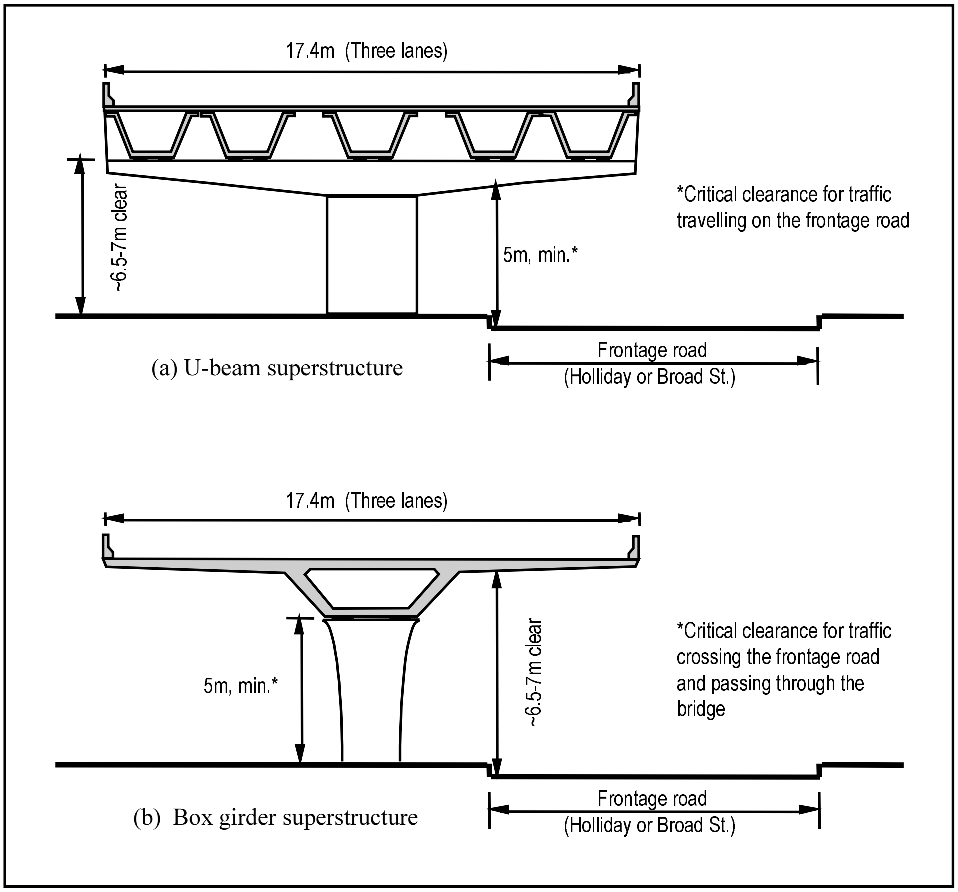


Figure 5.35 Improved frontage road clearance when changing from a U-beam design (a) to a box girder design (b).

The main issues in choosing a superstructure system revolve around the design concept and the economics of the project. The economic impact of superstructure choices is discussed in Section 5.3.5. The final bridge must be an attractive addition to the site, maintain a light, open appearance (particularly for the local traffic and pedestrians) and be constructed with a minimum amount of disruption to the site. A segmental box girder superstructure would best satisfy all of these criteria. Although more expensive than an I-girder or U-beam structure, the slender and light appearance from underneath, the ability to construct the bridge from above without disrupting traffic and the dramatic elegance of the curved ramp structures make this the most attractive option. Because segmental box girders can more easily span longer distances, constructing just three spans per city block would be possible. Use of these long-spans would improve visibility through the bridge by decreasing the number of substructure supports required. I-girders and U-beams would require the use of higher strength concrete (perhaps high performance concrete) to reach these longer spans.

The importance of this bridge enhancing rather than detracting from the site warrants the additional cost of a box girder superstructure. However, there are several complicating factors to consider. It is possible that financial constraints will dictate that only two lanes can be constructed at first and the project will need to be widened in the future. Similarly, all of the curved ramp structures may not be able to be built at the same time. In either of these cases, segmental box girders may not be a good option. Constructing additional spans at a later date using precast segmental construction would be uneconomical. Using a different superstructure system for the additional portions would be unattractive. If it is decided to build the bridge in phases of width, U-beams should be used throughout the project.

5.3.2.3 Substructure Design

To provide a clean and open design, a single-column substructure system was recommended. With a segmental box girder superstructure, single columns without bent caps could be used. This system would provide the lightest appearance. The aesthetic advantages of a box girder are apparent when the flow of forces in a box girder is compared to that of an I-beam bridge (Figure 5.36). The more efficient box girder bridge form leads to a more elegant structure.

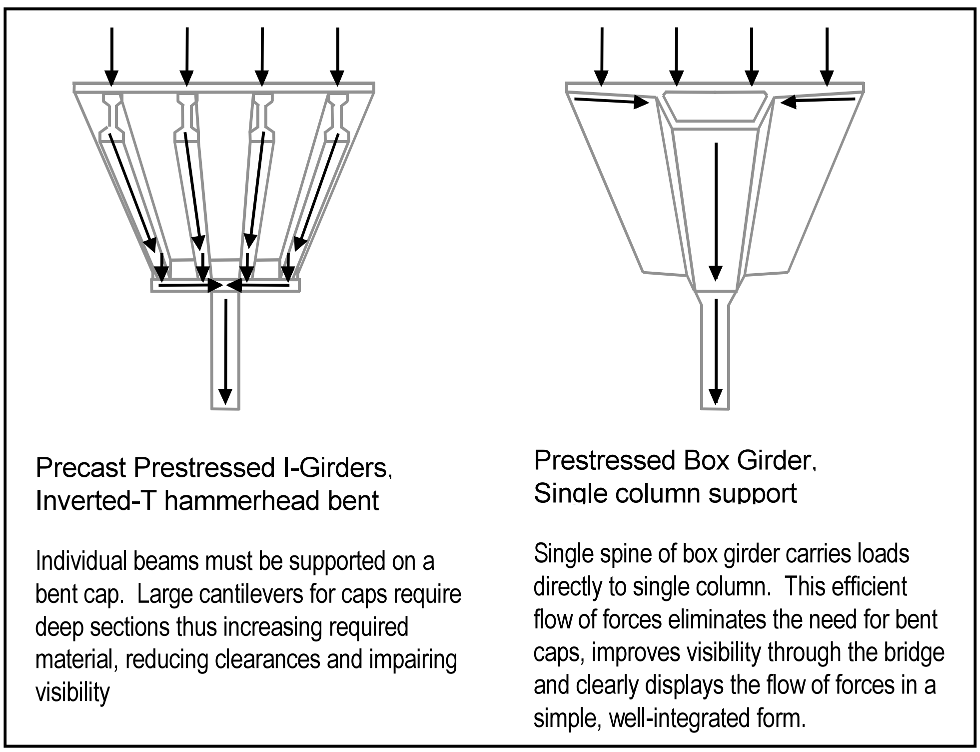


Figure 5.36 Comparison of the flow of forces in two different bridge systems

If a U-beam or I-girder system is chosen, inverted-T bent caps would help improve visibility through the substructure. The cap ends can easily be chamfered or shaped for an attractive appearance consistent with column details. Structural expression would be attractive for the hammerhead bent caps. The caps could be tapered to a deeper section at the column face or the columns could flare out to meet the cap. Both of these options provide structural efficiency and visual integration of the cap and column.

The appearance of the substructure will be greatly affected by whether or not the full three-lane bridge would be built initially or whether only a two-lane bridge would be built initially with a future widening option. If future widening is required and single columns are desired for the final bridge, the initial substructure must be built to support the full three lanes. An asymmetrical substructure would result if widening were to be only on one side on the bent (Figure 5.37a). This support would be very awkward in appearance. With widening provisions on both sides, a symmetrical substructure can be provided initially (Figure 5.37b). However, columns initially supporting two lanes but designed for three lanes will be stocky. The engineers must very seriously consider the visual impact such bridges would have until they are widened (if they are widened).

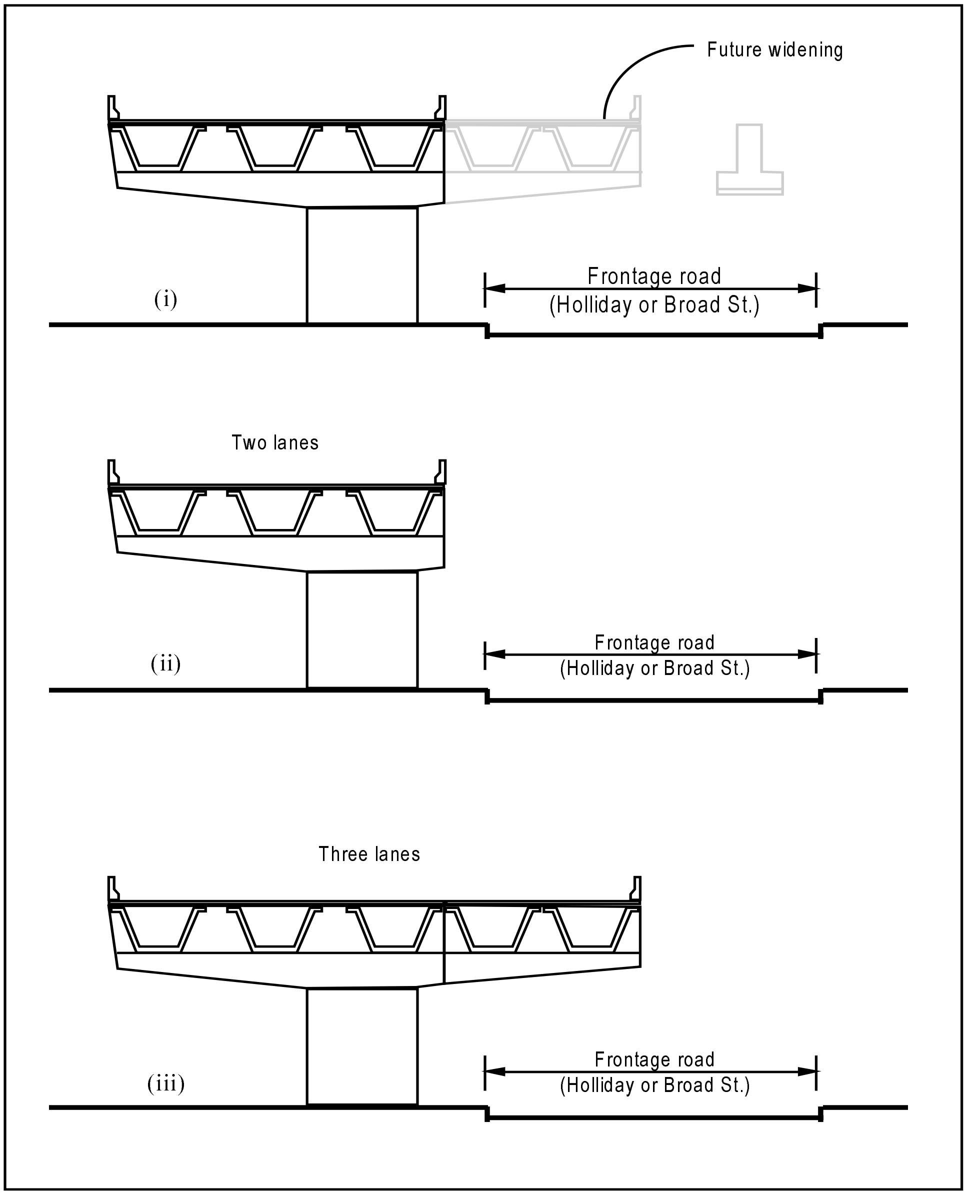


Figure 5.37a Scheme for widening to one side only (i). Appearance of the initial two-lane structure (ii) and the final three-lane structure (iii).

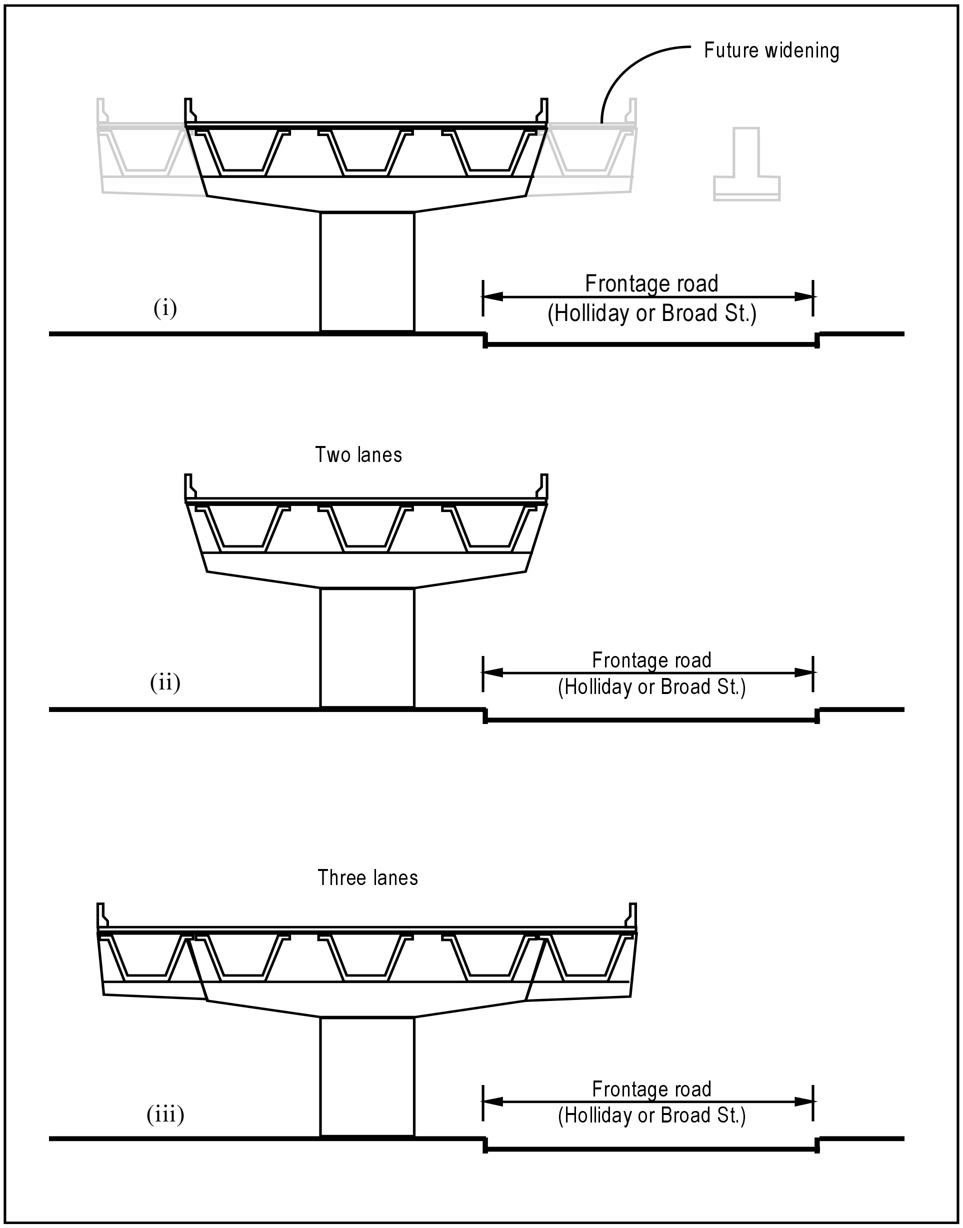


Figure 5.37 b Scheme for widening to two sides (i). Appearance of the initial two-lane structure (ii) and the final three-lane structure (iii).

For improved construction efficiency, the substructure units could be precast segmentally. Precasting the substructure is discussed further in Section 5.3.3.

5.3.2.4 Nonstructural Details

Attention to detail at this bridge site is very important as it will be primarily a pedestrian and local traffic area. The bridges must be attractive so as not to create an urban wasteland, ruining the local businesses. Attention to detail was a concern of the public particularly near the park at the north end.

Consistent surface treatment of the columns will add an attractive and unifying touch to the large structure. Using colored concrete pavers for the walkways beneath the bridge will also enhance this area and make it more attractive and welcoming to pedestrians. Designers have numerous options for column and walkway surface treatments, many of which are presented in Appendix B of the *Guidelines*.

Drainage must be carefully planned and controlled. Drain pipes should be internal. Although controversial to the businesses, no signs should be permitted to be hung from the structure as they would detract from the bridge form. An open impact barrier for the bridge roadway should be used to maintain a slender appearance for distant viewers and for traffic passing underneath orthogonal to the spans. Since attractive vegetation is desirable, particularly in the park areas, the amount of sunlight available under the bridge must be considered when choosing landscaping.

5.3.3 Potential Use of Precast Substructures

The US 287 project in Wichita Falls would benefit from using precast substructures. Perhaps most importantly, traffic disruptions could be minimized as fabrication would occur off site. Bridge erection would be faster as the need to set up formwork, place concrete and wait for it to cure would be eliminated. Accelerated construction would help relieve potential traffic delays and re-routing that the city would like to avoid. Precast substructures will have a higher quality finish which will add to the attractiveness of the final structure. A computer rendering of the proposed precast substructure system from Chapter 3 is shown on the future site in Figure 5.38.

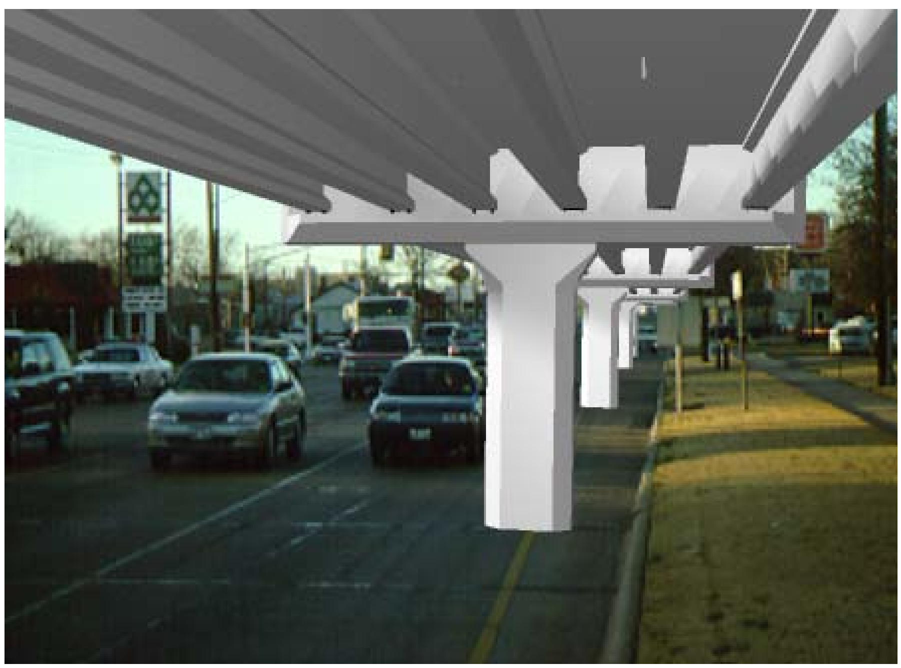


Figure 5.38 Computer rendering of the proposed precast substructure system along the right-of-way on Holliday St. (innermost lane to be removed)

Precasting the substructure would also be possible for a scheme where future widening must be feasible. Additional precast segments in the locations shown in Figures 5.22 and 5.37b, for example, can be post-tensioned to the existing bent (with a method similar to that described in Section 3.4.3 modified to provide temporary supports for the end segments while the necessary wet joints cure). Post-tensioning ducts would need to be provided initially. These ducts would need to be properly covered and protected until widening is desired.

5.3.4 City Planning Considerations

Raising the highway removes the virtual wall that traffic congestion creates during rush hour. This project elevates the fast moving traffic and allows city cross traffic to flow more easily between the newer developed areas to the southwest, and the old downtown. The separation of the twin elevated structures by a 183 m (600 ft) city block creates boundaries for the space between (Figure 5.39). There are many approaches that can be taken with the urban planning of this area to develop this space in attractive and productive ways for the city. There is potential for promenades to be developed along the frontage roads and for a district to be created within the block where the shops could be re-oriented to open towards the center. Such development could not happen overnight. Many issues would need to be addressed such as the location of some buildings directly up against the new elevated structures. However there does seem to be potential for this area to stay active. The elevated structure could provide an attractive promenade area leading people into the central district. The structure can then be seen as something that helps create an area rather than destroy it. Any city planning project such as this would be an investment over 20 to 30 years. Federal funding is often available in the form of tax relief for small businesses in older areas being revived and renovated.

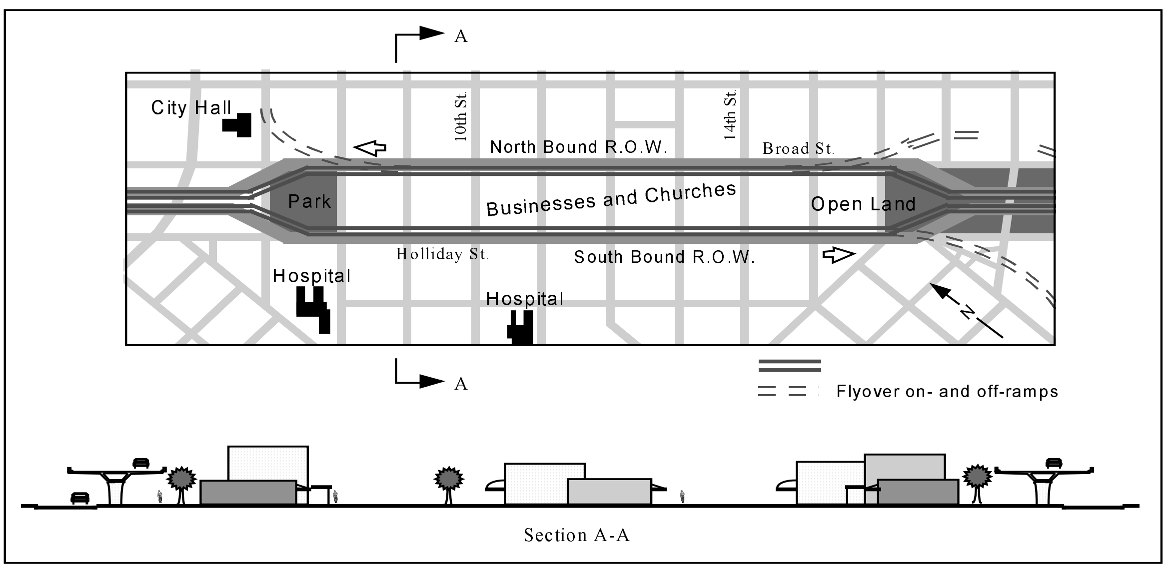


Figure 5.39 The wide area between the new elevated structures can be seen as a district

More immediate concerns regarding city planning concern the impact of the bridge on buildings directly neighboring the new structure. As mentioned previously, some structures are very close to the location of the new bridge. This will necessitate turning some of the entrances to these buildings away from the bridge.

5.3.5 Potential Economic Impact

A major factor dictating the economics of this project will be the choice of superstructure system. In 1996, I-girders and U-beam superstructure bridges averaged \$310 per square meter (\$29/ft^2) of roadway surface in Texas. As U-beams are currently only fabricated in Victoria, Texas (near Houston), the hauling costs associated with constructing such a bridge in Wichita Falls may increase the cost by a few dollars per square foot. The \$310/m^2 (\$29/ft^2) cost is largely based on unattractive substructure systems. Using more attractive substructures might increase these costs to, approximately, \$355/m^2 (\$33/ft^2) (see San Angelo project description in Section 5.2 and Table 5.1). The simply supported precast segmental box girder project for US 183 in Austin with more attractive substructure supports cost approximately \$420/m^2 (\$39/ft^2) of roadway surface. An I-girder or U-beam bridge enhanced with a precast substructure might be 20% less expensive. The construction equipment for US 183 in Austin or the San Antonio "Y" could be reused for a segmental box girder bridge in Wichita Falls which would keep the costs down. However, the start-up costs of a precasting yard would require the entire bridge project to be completed for it to be economical. An initial two-lane segmental box girder superstructure would be difficult to widen in the future. The entire three-lane bridge would therefore need to be constructed initially for segmental box girders to be an economical solution. A continuous precast segmental box girder project may cost more than \$420/m^2 (\$39/ft^2). However the longer spans possibly with a continuous superstructure will allow for savings in substructure and foundation costs.

The Wichita Falls project is a controversial one and warrants additional costs for ensuring that an attractive and efficient structure is built. The presence of the new bridges must help to enhance the area if it is to survive economically. If the bridges detract from the area, a rift will be created in the city. The bridges would essentially create a barrier between the old downtown and the newer developments. For this reason, I girders should not be considered regardless of their probably being the lowest cost choice. They would also be the least attractive option. A segmental box girder would provide the most attractive addition. As mentioned previously, the segmental design should not be considered if the entire three-lane width cannot be built at once. U-beams may be a very good economic compromise.

5.3.6 Section Summary

Many lessons were learned through the research team's involvement with this project. Offering suggestions for improved bridge design while the planning process was ongoing allowed for a more realistic view of the project's constraints. Decisions could be made after many options were considered rather than strictly based on cost and speed of design (as was the case with the San Angelo project).

Again, a site visit was essential for developing a vision and design concept for the bridge. The design concept could then be carried out while considering many different options for layout, superstructure design, substructure design and non-structural details. It was also evident that city politics had considerable impact on this urban project. It is important for the design engineers to work collaboratively with city officials. The engineers must stick to good engineering design and communicate to the city the attractive possibilities attainable with engineering design rather than allowing the city to decorate their bridges unnecessarily. The engineers must choose forms and layouts that are elegant and attractive and do not invite the desire to cover them with facades or ornamentation.

The Wichita Falls project was still being designed during the writing of this report. A decision to use U-beams where possible was most recently proposed. A segmental box girder superstructure system was considered too expensive for the District to justify constructing. In addition, the very large cantilever moments (Mu=51000 N-m) would require bent cap depths at around 4 m (160 in). Thus, the specific precast system outlined herein in Chapter 3 would not be appropriate.

5.4 Chapter Summary

Having the opportunity to implement the research while it is ongoing has been valuable for this project. For the *Guidelines*, input from designers, precasters, form suppliers and contractors across the state and the gaining of insight into the actual design process helped shape the *Guidelines* into a practical and useful manual. Through both projects, it was seen that application of the *Guidelines* from the outset of design will allow for the best use of the manual. With the San Angelo project, it was found that with simple layout changes, more attractive solutions could be found with economic savings. The Wichita Falls project showed the ease with which the *Guidelines* could be applied to allow for simple comparisons of design options. This project also showed how public interest in bridge aesthetics can interact with design options. Engineers must pay attention to the impact of their designs on the public and develop their own ideas from realistic choices so that they can proudly display their work rather than have colleagues such as landscape architects try to mask it.

Exploring the use of precast substructures provided focus for the development of a more comprehensive precast substructure system for standardization. Extensive attention to fabrication, erection and connection details is necessary for adoption of this new substructure design option. The possibilities for economical and attractive use of such a system are clear. The details must be practical and the presentation convincing to inspire the industry to overcome its resistance to change and further the potential for a new form of substructure design and construction.

CHAPTER 6

IMPLEMENTATION

6.1 Introduction

The research work described herein has potential direct applications for TxDOT (and other) highway planners and bridge engineers. Implementation of the precast substructure system proposed for standardization is discussed. Necessary follow-up steps are suggested.

6.2 Alternate Substructure System

The proposed substructure system described in Chapter 3 has been developed to facilitate adoption for standardization. This system is primarily for use with short- and moderate-span bridges with a highly standardized superstructure system, precast concrete girders. Standardization of short- and moderate-span bridge components results in economic savings primarily through highly efficient, repetitive fabrication processes and reduced design and construction time. Developing a new standard for substructure systems will be the key to successful introduction of precast substructure design for standard highway bridges.

While precast substructures are not an entirely new form of construction, they have had limited use in the past. In an effort to understand potential acceptance of a standardized precast substructure system, a sampling of industry personnel was questioned about the use of precast substructures at present and in the future. Response comments are outlined in Section 6.2.1.

As mentioned previously, a major impetus for the development of a standardized precast substructure system is the potential for savings both in dollars and in on-site construction time. Section 6.2.2 discusses the potential economic impact of a standardized precast substructure system as determined from discussions with experienced precasters and contractors in Texas.

6.2.1 Industry Comments on Precast Substructures

Direct comments from industry are summarized here to reflect the general mood in the industry concerning the acceptance of precast substructures as an upcoming form of construction. These comments indicate areas needing further study as well as areas where misconceptions need to be addressed and clarified.

6.2.1.1 LoBuono, Armstrong and Associates

Precast substructures are an acceptable method of construction but it is not apparent to this firm that precast piers offer economical advantages over cast-in-place piers. Casting piers in place is seen as a natural progression continuing up from foundation casting. Importantly, they feel that on-site substructure casting is advantageous in filling the "lag" time between the notice to proceed with superstructure erection and when the superstructure elements are ready to be erected. Exceptions are recognized in the case of large projects or projects built in difficult to access sites, where precasting has been used advantageously in the past.

Some concerns expressed about precast substructures have involved the use of looped strand tendons. Anchoring both ends at the top is viewed by them as a problem as is the ability to efficiently thread these looped tendons. (Experience on US183 indicate neither reservation is valid). They were concerned over water removal from the ducts. Post-tensioning bars are considered easier to work with but are more expensive.[62]

Authors' comments: A potential solution to congestion problems with looped tendon anchorages at the top is through use of high performance concrete with strengths of 56 MPa (8,000 psi) or higher, where smaller anchorage spacing may be permitted (see Section 3.6.7). Special anchorage devices must be tested in higher strength concrete before they can be used in such higher strength concrete in the field.

The argument that casting substructures in place is a natural progression from cast-in-place foundations can also be used for precasting substructures. A precast substructure would facilitate the natural progression of the precast superstructure which follows.

6.2.1.2 J. Muller International

To be practical, precast substructure elements must be used on large projects where it is economically justifiable to set up a casting yard and haul the segments to the site. A sufficient number of elements of the same size and shape are required for low cost fabrication. Precast substructures are particularly advantageous for sites that are difficult to access and for bridges built in harsh environments. For bridges in harsh, cold environments, fabrication indoors through the winter is advantageous. The high quality control in precasting allows for efficient use of higher quality concrete (HPC) which is less permeable and therefore more durable.[63]

Authors' comments: Another option for precast substructures to be economically justifiable is through standardization. The large volume of smaller, standard bridges exceeds the volume of many large projects. This is the rationale used to justify development of standard precast segmental box girder superstructure standards. [46]

6.2.1.3 DRC Consultants

Precast hollow box piers are considered strictly for economics and speed of construction. They have not been common in the past because projects are typically too small to gain economic advantages or because transportation costs are excessive. In a few cases, seismic criteria require a large amount of vertical post-tensioning.[64]

Authors' comments: Again, precast substructures will most probably be economically justifiable through market aggregation brought about by effective standardization. Applicability of precast substructures to seismic regions is not of major interest to TxDOT but certainly is a topic worthy of further investigation.

6.2.1.4 California Department of Transportation (CALTRANS)

Segmental substructures have not been used in California but there does seem to be a future for segmental piers particularly in nonseismic areas. The continuity between substructure and superstructure needed to satisfy the ductile design philosophy of the American seismic provisions has led CALTRANS to use cast-in-place piers with mild reinforcement predominantly. Investigations into the use of prestressing to provide continuity resulted in the conclusion that the "lack of substantial strain energy between the design load and the ultimate strength of the high strength strand could be a problem." For precast substructure design to be utilized in California, the seismic performance of such substructures will need to be further researched.[65]

6.2.1.5 Texas Department of Transportation ((TxDOT)

Precast piers have been used in Texas for at least three projects: the Neches River Bridge in Port Neches, US 183 in Austin and the Louetta Rd. Overpass on State Highway 249 in Houston. The Neches River Bridge was designed by the engineering firm of Figg and Muller. Two of the segmental piers were temporary supports that were more easily dismantled as segmental piers. The US 183 piers and the Louetta Rd. overpass piers were designed by TxDOT. For US 183, the large size of the project and high repetition of similar pier types (260 piers, 3 pier types) were reasons for proposing precast construction. The contractor chose to precast only one pier type. The decision to cast the other piers in place was based on several factors. These included the ability to quickly invoke field labor, an extreme shortage of space in the confined precast yard, generous amounts of cleared right of way which could be used for cage and form staging areas, and a very modest economic benefit computed as a 0.4% savings over the precast option. Upon completion, the contractor's superintendent indicated construction of the precast piers was the easiest way to build piers that he had ever experienced. The second-place bidder felt they would have precast all of the piers. The Louetta Rd. piers were precast because a study mandated that 69 MPa (10,000 psi) concrete be used for the substructure. To take advantage of the higher strength concrete, hollow-segmental piers were designed to be post-tensioned. No major problems ensued. The Louetta project was also seen as a study into the feasibility of precasting substructures in the future. [66,67]

Both the US 183 project and the Louetta Road project were applications without caps. This is a substantial advantage in application of precast substructures.

TxDOT is open to the development of precast substructures and is clearly in support of investigating the feasibility of such systems.

Authors' comments: The precast pier concepts developed for the Louetta Rd. overpass are being used again for a current project in El Paso. Practical implementation of the proposed precast substructure system by TxDOT will require development of specific standards for the range of applications of most interest. The recommendations from this project should be helpful in such an effort.

With the development of such standards, the possibility of precast plants being ready to furnish such elements on relatively short notice would substantially reduce project start-up time.

6.2.2 Economic Impact

The use of precast substructure systems with a great deal of field post-tensioning would be a dramatic shift away from traditional cast-in-place substructures. There are many new details and construction operations that could offset apparent savings. Types of new or additional costs include:

- Crane capacity for handling erection of caps.
- Longer hauling distances for precast elements than for ready mixed concrete.
- Extra handling and on-site storage or dependency on just-in-time delivery in congested areas.
- Difficulties in use of void forms in precasting.
- Post-tensioning incidentals (grouting, curved ducts, jack calibration, construction engineering costs, anchorage zone congestion, slightly more skilled labor, epoxy usage, etc.).
- HPC concrete costs
- Supplemental reinforcing for lifting and handling.

Thus, it was very important that the assessment of economic impact be carried out as objectively as possible and that it utilize professionals experienced with precasting and post-tensioning operations in Texas. Extensive and highly detailed discussions with precasters and erectors in Texas led to estimates for the cost of the proposed precast substructure system including necessary construction time. Specifically one major Texas precaster and one major Texas contractor who was very experienced in post-tensioned segmental construction were furnished with schematic plans of the proposed precast substructure system including dimensions, proposed fabrication description and erection sequence. Their cost estimates of the fabrication and erection of the system are outlined in Table 6.1. Cost estimates were based on the premise that the suggested system has been standardized by TxDOT and implemented by being specified in a fairly wide number of bridges. Therefore the prices listed assume that this substructure is already in production and that several precast plants possess the necessary formwork. Engineers would be able to specify standard precast piers just as they now specify standard I-beams or box beams. The estimate of construction time given in Table 6.1 was made by the authors based on observing recent field experience in Texas. The cast-in-place bents require almost twice as much time for field construction because all construction operations are in the field. In contrast, placing reinforcing, forming, and concreting operations for the precast system take place off of the site. The erection of five single precast columns (the equivalent of one five-column cast-in-place bent) for the Louetta Rd. overpass in Houston (Figure 2.12) required roughly 10 days in the field, but a number of those days did not require full crew activity. Observation at the erection of the US 183 project in Austin (Figure 2.11) indicated that two crew days per each much taller pier was a reasonable estimate. Thus, a maximum of 1½ crew days should be enough for erection of the proposed shorter pier shafts.

However, the two-column precast frame bent is considerably more complex than the Louetta or US 183 single shaft piers which had capitals but not bents. In addition to the two pier shafts, the two templates must be set and template joints cast, the two cap segments set, the cap closure cast and stressed, and then a second stage of post-tensioning applied after the girders are erected. Based on observations and reports from other segmental projects, the authors' estimate of time requirements assumed:

- 1½ crew and equipment days for erecting each shaft
- 1 crew and equipment day for erecting both templates
- 1 crew and equipment day for setting both cap segments
- 1 crew day for preparing and casting the cap closure

- ½ crew day for stage 1 stressing
- ½ crew day for stage 2 stressing

Thus, 7 crew days and 5 equipment days are required for each two-column precast frame bent (14 crew half days and 10 equipment half days). The time estimate given in half days reflects the fact that some operations such as alignment of the first column segment and casting of the base geometry control joint may occur for more than one bent before further erection operations on one bent proceed. No cost benefits are included for the substantially reduced field construction time with possible savings in traffic control and benefits in early completion of the project.

Table 6.1 Cost Estimate for the Standardized Precast Substructure System

Cost Estimate of Standardized Precast	
Segment Fabrication (including hauling)*	$260-330/m^3 ($200-250/yd)^3
Post Tensioning	10% of segment
Erection Crew **	$1500/day
Equipment *	$1000/day
Assume 7 crew days and 5 equipment days for each frame bent	
Total Cost (btwn. $515-545/m^3 ($395-50/yd^3)) say	**$565/m^3 ($452/yd^3)**

* *Source: Texas precaster*
** *Source: Texas contractor*
Note: Total cost based on the precast frame bent shown in Figure 6.2. Precast hammerhead bents with similar material quantity may see reduction due to faster erection time

Figure 6.1 gives a comparison between the estimated proposed precast substructure cost and the current average bid prices in Texas for cast-in-place substructures with rectangular caps and with inverted-T caps. The precast substructure system with its inverted-T cap would be most competitive with the cast-in-place inverted-T alternative and has both aesthetic and life-cycle cost advantages. Although the cost per cubic meter for the precast system is 16% higher, the overall cost of a bent will be closer to that of a conventional inverted-T system in many cases. This results from the precast substructure requiring less material through the use of high performance concrete in efficient hollow shapes. A comparison of material quantities, estimated cost and estimated construction time for the three types of construction shown in Figure 6.1 is given for a hypothetical multicolumn bent in Figure 6.2. Even without including the potentially highly significant cost benefits of faster on-site construction times, the proposed precast system is highly competitive (within 5%) with the inverted-T design. Neither the proposed precast system nor the presently widely used inverted-T cap system is competitive in terms of material quantities or substructure costs with the traditional rectangular cap system. The precast system could be justified as a replacement for the rectangular cap system based on reduced construction time or improved aesthetics and/or vertical height factors as are now used to justify use of the inverted-T cap system. The precast system could also be justified where bridge costs are a small portion of a large highway project, and the increases in substructure cost have a minimal effect on overall project cost (see Section 4.1). In Section 1.1, efficiency was defined as the minimization of wasted material, nonproductive labor and construction time in meeting the users needs within the project constraints (functionality). Comparing the precast system with the inverted-T cap system, the material and labor savings as reflected in volume of concrete are only in the 10% range. This is offset by a 15% increase in estimated unit cost of concrete reflecting the large number of extra operations involved in handing and connecting the precast units. Thus significant efficiencies would only be seen from minimization of construction time. This time reduction would only present savings on projects where such shorter construction times would result in reduction of user delays, improved safety, and lowered traffic rerouting costs. Aesthetic improvements would have to be justified on a project to project basis

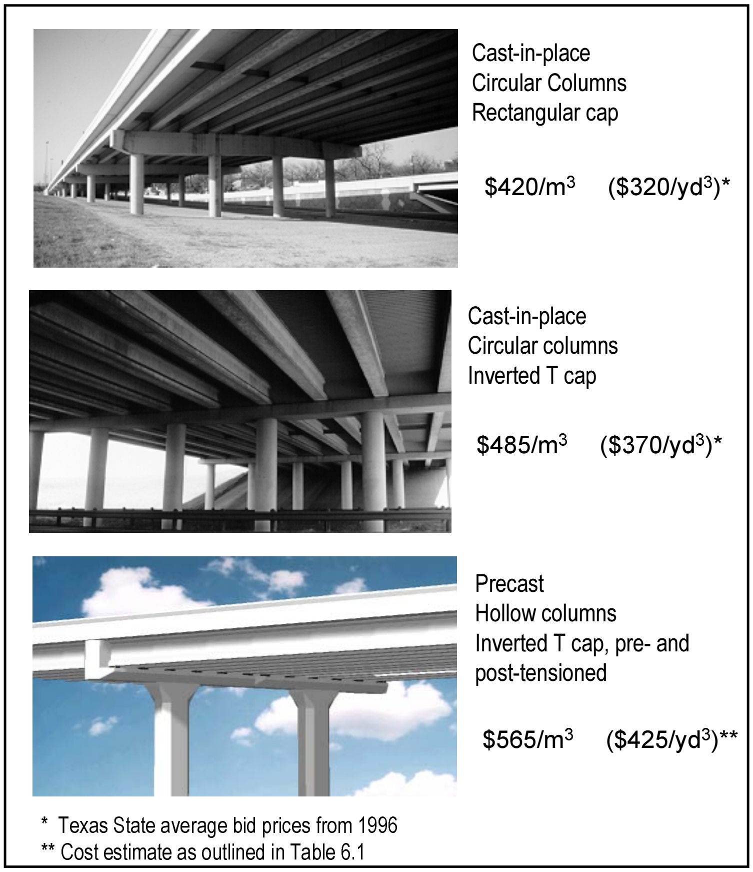
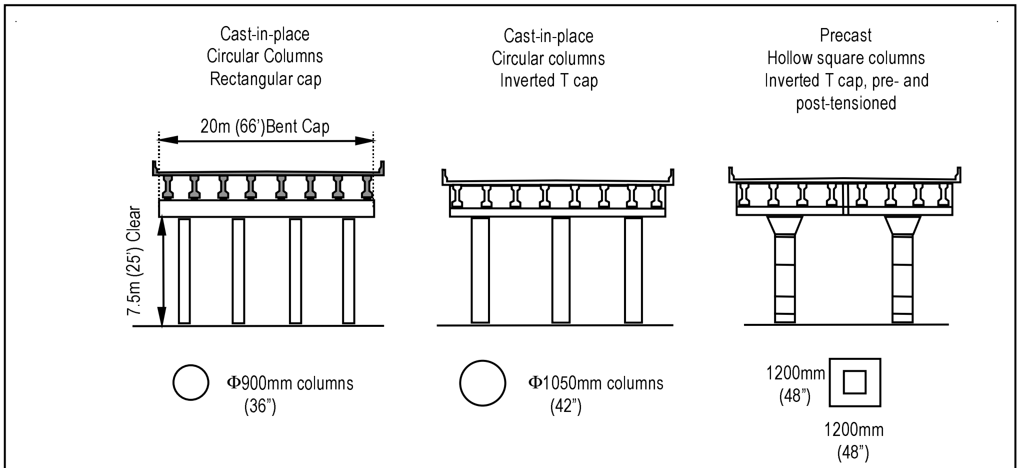


Figure 6.1 Comparison of substructure costs



Volume of concrete	37 m^3 (48 yd^3)	76 m^3 (99 yd^3)	68 m^3 (89 yd^3)
Unit cost*	\$420/m^3 (\$320/yd^3)	\$485/m^3 (\$370/yd^3)	\$565/m^3 (\$425/yd^3)
Total cost per bent	**\$15,300**	**\$36,600**	**\$38,400**
Estimated construction time	**~ 4 weeks**	**~ 4 weeks**	**~ 2 weeks**

* See Figure 6.1

Figure 6.2 Cost comparison of three designs for a 20 m (66 ft) wide bent

Advantages of the system will vary from project to project. Considering the benefits of shorter on-site construction time points to high potential for future success of precast substructure systems. A recent project in Texas (“Pierce Elevated” in Houston) utilizing 710 kN (160 k) precast pier caps to speed erection time in a congested urban site resulted in the largest early-completion bonus (\$1.6 million) ever offered by the Texas Department of Transportation.[68] The value of a day of construction time saved was estimated at \$53,000. The project manager for the contractor remarked that using falsework to tie the cast-in-place columns with the precast caps would have required six times as long as the method they used, which required no falsework. Clearly the value of construction speed in certain locations will far outweigh the construction cost differences of different substructure systems. For instance, as shown in Figure 6.2, while each precast substructure unit may cost \$23,000 more than a cast-in-place multicolumn bent with a rectangular bent cap, a project construction time savings of two weeks in the congested urban site mentioned previously would equate to a savings of \$742,000. The costs would vary depending on the size and location of the project. However, the recent experience and valuation of construction speed should give precast substructure systems in the future a highly competitive economic advantage.

Another potential economic impact of precast substructure construction is with life-cycle costs. Precasting the substructure has the advantage of higher quality control for concrete placement. The result will typically be a higher quality surface finish. The unattractive formlines and tie patterns common to cast-in-place concrete construction will be reduced with precasting. As a result, painting the surface will not be necessary to provide an attractive surface appearance. Painted concrete requires repainting roughly every five years or more often if the surface was not prepared properly before painting. Either way, painting concrete is a maintenance headache that should be avoided wherever possible. An attractive precast finish will reduce maintenance needs and costs.

Another savings in life-cycle costs achieved through the higher performance materials and improved typical quality control of precasting is enhanced durability. Cover requirements on reinforcement are more accurately achieved. Better controlled mixing, placement and curing methods can result in better quality concrete. The combination of epoxy-coated reinforcement, adequate cover achieved with controlled plant production methods, and decreased permeability through high performance concrete can provide excellent durability for a precast substructure. In addition, the compression of the concrete under service loads due to prestressing should assist in prevention of cracking and in control of crack widths if and when cracking occurs. These effects should impede the ingress of corrosive agents such as chlorides. Enhanced durability will reduce maintenance and potential replacement costs particularly for the substructure of bridges in coastal regions and areas where de-icing salts are used on the bridge decks.

6.3 Recommendations for Further Research

Further research in the development of precast substructure systems is desirable. Full-scale tests of typical bent cap post-tensioning anchorage zones should be performed to verify supplementary and confining reinforcement details for selected commercial anchorages. Investigations can be extended to include nonprestressed connection alternatives. Column segments may be joined with nonprestressed reinforcement spliced through grouted sleeve couplers. Potential applications of precast substructure systems should be examined for seismic regions. Such studies should be funded nationally or by concerned states.

Further study of the economic impact of more rapid on-site construction would benefit the future implementation of such a substructure system. While material and labor costs are easily estimated and over time become more accurate, the advantages of avoiding traffic delays and making new highways available to the public faster are less quantifiable but may in fact have a more profound impact on the economics of precast substructure systems. Recent attempts to quantify motorist inconvenience such as for the "Pierce Elevated" project discussed in Section 6.2.2, exemplify the importance of speed of construction. Different ways in which faster construction can benefit a community should be emphasized, observed and recorded.

The economic advantages of precasting year-round in harsh climates where the construction season is short should also be further examined.

6.4 Chapter Summary

The most important aspect of implementing the research presented in this report is following the success or failures of projects in which the precast substructure systems are used. This research project has been directed toward field application. The project must therefore be analyzed and judged in terms of its effectiveness, usefulness and aesthetic and economic impact on actual projects.

As precast substructure systems become more widely accepted and used, their aesthetic and economic impact on short- and moderate-span bridge design must be evaluated. Evaluations should consider both initial and life-cycle costs as well as appearance. Communication between researchers, designers and builders must remain open and cooperative for rapid and successful implementation of new ideas and for the sharing of positive and negative experiences with precast substructure systems.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1 Summary

Standard highway bridges of short- and moderate-spans are typically designed for economy and function alone. Although these short- and moderate-span bridges often dominate the highway landscape, they often detract from rather than enhance the environment in which they are built. Such an unimaginative display of structural engineering does little to express the rapid growth and exciting developments in this profession.

The improvement of standard short- and moderate-span bridges through the use of attractive, rapidly constructed concrete substructures is suggested herein. This research was conducted through the Center of Transportation Research at the University of Texas at Austin as Project No. 0-1410.

An attractive and rapidly constructed substructure system of match-cast precast segmental elements post-tensioned together on-site was developed for use with standard superstructure systems in Texas. Attractive, cast-in-place substructures were also investigated as an alternate to the current, common Texas substructure practice utilizing circular columns and rectangular bent caps. Applications of this research to two existing projects in Texas were summarized and the resulting impact reviewed. Implementation strategies for further application of the research are given.

The research presented in this report includes:

- A literature review of substructure design and construction (Chapter 2)
- The development of an attractive precast substructure system for use with standard short- and moderate-span bridges (Chapter 3)
- An investigation of new cast-in-place substructure designs (Chapter 4)
- Application of the *Guidelines* [3] and the precast substructure system to two existing projects in Texas (Chapter 5) and,
- Suggested strategies for further implementation of the research with potential impact on aesthetics, construction time and economics (Chapter 6).

This study clearly showed that the appearance and efficiency of standard short- and moderate-span bridges can be improved with more thoughtful substructure design. In the past forty years, superstructure design in Texas has continually been evaluated and improved by taking advantage of new efficient structural shapes, the efficiency of higher strength materials (both steel and concrete) and the efficiency of mass production through precasting. On the other hand, substructure design for standard I and box cross-section girder bridges has remained virtually the same. These substructures tend to utilize the same basic shapes (circular or square columns and rectangular or inverted-T bent caps) with lower grade, nonprestressed steel and cast-in-place concrete.

The standard multicolumn bent substructure system most often used in Texas requires labor intensive on-site construction, utilizes a relatively low quality concrete and creates an unattractive forest of columns. The majority of bridges with durability problems in Texas have deficient substructures. More efficient and more durable substructures would combine high performance materials and precasting. As a specific proposal, a precast substructure system was developed for use as an alternate substructure system in Texas.

The proposed precast substructure system is made up of segmentally match-cast piers with a match-cast cap that is well suited for manufacture in a precast pretensioning plant. The cap and piers would be post-tensioned together on site. The proposed system has two designated geometry control joints per pier that would require field concreting or grouting. Match-cast segmental construction of the pier shafts has been proposed largely due to its successful use for moderate- and long-span superstructure design in the past and its successful use for piers on several recent Texas projects. Segmental construction has proven to be a rapidly constructed, durable, and fairly economical system that minimizes traffic conflict and can reduce impact on the local environment.

The proposed match-cast segmental piers have been based on several examples that have been successfully constructed in Texas and nationally. Little new technology is needed for the pier shafts. On the other hand, the templates and caps would be a more pioneering innovation. The heavy weight of the caps, the relative congestion of pretensioning and post-tensioning details, and the probable need for stage post-tensioning in the multiple pier bents introduce complexity, additional costs and, until experience is obtained, uncertainty. These complications indicate that an initial use is most logical on projects with considerable repetition and where potential timesaving can provide substantial returns. The simplest initial use would be on projects utilizing the relatively straightforward hammerhead piers. The multiple-column bent would be more complex and should be considered the second stage of development.

The proposed precast substructure system is a versatile system that can be used for a wide variety of bridge widths and heights. Two specific design examples are presented in detail in Appendices C and D of Reference 2. The new system uses elements that can be handled by the types of cranes used for erecting standard girders. The precast system of match casting with epoxy joints has provided excellent durability for structures in the past. The combination of precasting and using high performance concrete results in more durable and more attractive construction. The geometric shape of the proposed system may also be used for cast-in-place construction and is recommended as an attractive alternative to current standard cast-in-place substructures. The on-site construction time required could be substantially less.

While most major concepts and dimensions were evaluated in several trial designs, actual application to a specific project will require further detailed investigation. The preliminary design was limited to applications with cross-slopes of not more than 3%. Template design and fabrication could readily be altered to accommodate cross-slopes of up to 5%. For greater cross-slopes, bearing seat buildups of unequal heights may be required. When specific post-tensioning systems are selected, the system may require a field test to verify the anchorage zone details. The drainage pipes will probably need to pass through blockouts in the cap. Particularly with the multiple column bent applications, staged prestressing of caps may be required. This would increase the complexity of equipment allocation, crew scheduling and time required for bent construction.

Numerous attractive options for cast-in-place substructure design are proposed in Chapter 4 as alternatives to the circular column, rectangular cap substructure system used most often in Texas. Key aesthetic issues identified that should be considered when choosing an appropriate substructure system and shape are structural expression, visibility through the bridge, integration of the substructure with other bridge elements and the bridge site, and enhancement of substructure design through attention to nonstructural details. Designing the substructure of a bridge is shown to be an opportunity to display ingenuity and to express the elegance possible through *engineering*. With thoughtful consideration, attractive and well-suited substructures will greatly enhance the overall appearance of standard bridge systems

The *Guidelines* [3] and preliminary versions of the precast substructures were used to develop design options for two ongoing bridge projects in Texas. The results were documented and show the ease of application, the usefulness and the practicality of the *Guidelines*. The projects also point to the future possibilities of precast substructure construction as an attractive and economical form of construction for short- and moderate-span bridge systems particularly when significantly reduced on-site construction times and reduction of congestion and traffic handling are important.

Suggestions for continued implementation of this research are presented in Chapter 6. Clearly, initial implementation of the proposed precast substructure system should be as part of several large projects in highly visible locations where construction efficiency (speed of construction) and final appearance are particularly important. An initial investment in forms for a large project will lead to future savings when the forms may be reused for similar or smaller projects. Texas precasters and contractors who were asked to give cost estimates for the fabrication and erection of the proposed precast substructure system if and when standardized, indicated that the proposed system will most probably be economically competitive with other single column piers and with cast-in-place, multicolumn inverted-T cap bents. It is not economically competitive with multicolumn bents with rectangular caps. In the future, other new standard shapes for both precast and cast-in-place substructures may be developed to provide TxDOT designers with even more alternatives for attractive substructure design.

7.2 Conclusions

7.2.1 General Conclusions

1. Current short- and moderate-span highway bridge standard systems are often unattractive, unimaginative and sometimes deficient in durability. The superstructures are generally efficient and attractive. Most problems are due to the substructures which tend to be unattractive, are not built with highly durable materials, and sometimes require unnecessarily long on-site construction times.

2. Precast substructure systems are a relatively new and versatile alternative in substructure design that can offer numerous benefits for highway construction including rapid on-site construction time, reduced traffic delays, improved use of

structural materials, improved overall appearance, reduced maintenance needs, and enhanced durability.

3. Precast substructure systems will be most efficient, attractive and economical when utilizing high-performance concrete, standardized plant production methods, match-casting and rapid field erection.

4. A prototype precast substructure system has been developed that exploits the benefits listed in Conclusion 2 and satisfies Conclusion 3. Design examples are furnished in Reference 2 to indicate typical computations and applications.

5. Initial use of precast substructures for standard highway bridges should be for large repetitive projects where the economy of scale will minimize initial form investment costs.

7.2.2 Detailed Conclusions

1. With long-span bridges, the use of minimum quantities of materials very often leads to the most economical design. In contrast, with short- and moderate-span bridges, the use of standardized, rapid and efficient construction methods is often more important for economy.

2. The overall aesthetics of bridges utilizing efficient, attractive superstructure elements is greatly affected by the layout of the bridge and the substructure system chosen. Non-structural bridge components, such as decorative ornamentation, cannot improve the appearance of a dull bridge form.

3. Significant aesthetic improvements can be made to some standard bridges with little increase in cost and sometimes with savings. (See Section 5.2)

4. On-site construction time for precast substructures in large projects with highly repetitive superstructure systems and relatively unchanging roadway geometry may be significantly reduced from that required for current cast-in-place concrete bents.

5. Costs incurred by traffic delays and benefits to the public as a result of faster highway construction are difficult to quantify but are an important advantage to precasting over cast-in-place construction particularly in congested areas.

6. For a precast substructure system, match-cast pier segments and a template that has been match cast to the cap are the key elements for speed of construction and durability.

7. Strut and tie modeling is a particularly useful tool for preliminary design where examination of the flow of forces can lead to efficient design shapes and reinforcement layouts.

8. Cast-in-place substructure design should take advantage of the moldability of concrete. Incorporation of curves, tapers and column and cap shaping for structural expression and or bridge component integration should be explored in design.

7.3 Implementation Recommendations

1. TxDOT should develop specific standards for the proposed precast substructure system.

2. Initial standards should begin with the single column and hammerhead bent cap unit so as to gain field experience with the simplest application before the multiple column bents are implemented.
3. When a specific system is detailed, a field test should be used to verify the anchorage zone details.
4. Projects in which the proposed precast substructure systems are used should be monitored so that informed judgments can be made of its effectiveness, usefulness and aesthetic and economic impact.
5. As such field application information becomes available, the system design should be revised to incorporate desirable changes to enhance constructability.

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