Development of a High Performance Substructure System for Prestressed Concrete Girder Highway Bridges

by

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Thesis

Presented to the Faculty of the Graduate School

of The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

Master of Science in Engineering

The University of Texas at Austin

August, 1996

Development of a High Performance Substructure System for

Prestressed Concrete Girder Highway Bridges

APPROVED BY

SUPERVISING COMMITTEE:

To my family

yea, the work of our hands establish thou it.

—Ps. 90:17c.

ACKNOWLEDGMENTS

The author acknowledges the ongoing support of the Texas Department of Transportation for Project 0-1410. In particular, thanks go to Norman Friedman for the information and insight he has provided concerning the workings of TxDOT's Design Division. The financial support of the College of Engineering and W.A. "Tex" Moncrief, Jr. is also gratefully acknowledged.

This thesis would have been impossible without the hard work and dedication of the other student researchers involved with Project 0-1410. The author thanks Scot Listavich for his help, support and overall attitude (especially concerning baseball and golf). The ideas and architectural talents of Carl Holiday were instrumental to the completion of this work. Finally, the author is intensely grateful to Sarah Billington for assuming the leadership of this project and taking on the burden that I had neither the skills nor desire to shoulder, in addition to constantly reminding me of what I had accomplished when I was wasting time thinking about what I had not.

The author would also like to acknowledge the staff at the Phil M. Ferguson Structural Engineering Laboratory, especially Laurie Golding, Cindy McCright, April Jenkins and Sharon Cunningham, for their assistance with various phases of Projects 0-1410 and 0-1388. Thanks also go to Dr. Ned H. Burns for his tolerance and support, and Heather Jobson and Usnik Tuladhar for their hard work. All of these people made it possible to finish this thesis while pursuing doctoral studies on a different research project.

The author acknowledges his fellow students at Ferguson Lab, past and present, for fellowship and various lessons learned. Particular thanks go to those who helped with this thesis either directly or indirectly by forcing me to believe that I would finish one day. These include Jeff West, Chuck Larosche, Rodney Davis, Vince Dolan, Jeff Schmitz, Valerie Andres, Pierre Hoogenboom and Vishuan Achebe, among others. Special thanks also go to Reagan Herman and Chris Gilchrist for their assistance with the golf tournament.

The author is especially indebted to Dr. John E. Breen for supervising this work and taking the time (even when recuperating from a bad Evil Knievel impersonation) to make sure that it was done well. Teacher, advisor, golf partner and friend, Dr. Breen is a constant reminder of the symbiotic relationship between engineering research and engineering practice. Likewise, the author extends his gratitude to Dr. James O. Jirsa for finding the time to review this thesis and offer guidance.

Finally, I would like to thank my entire family, especially my parents, for their ever-present love and support. Thanks go to Sidney and Clovis for accepting me, always being happy to see me, and waking me up on the weekends. The greatest portion of my appreciation is reserved for my wife, Jessica, who has been with me throughout this ordeal. Her perseverance, faith and support are written into every word. Austin, Texas, July 9, 1996

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The University of Texas at Austin, 1996

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The precast, pretensioned concrete I-beam and cast-in-place concrete slab superstructure using stay-in-place pretensioned deck panels is the prevailing system for short- and medium-span highway bridge construction in much of the United States today. High strength materials, plant production methods, repetitive elements and standardized details all contribute to the efficiency of this system. Although this technology has been dominant for several decades, the overwhelming preponderance of substructures for these same bridges consists of cast-in-place reinforced concrete. A predominantly precast, post-tensioned substructure system for such bridges has been developed in this thesis. While cast-in-place construction techniques are utilized to a limited extent, the system was developed to benefit from the advantages inherent in precast production including: high strength and high performance materials, economies of scale, efficient standardized production and faster on-site erection times. Precast techniques also provide much needed aesthetic improvements through flexibility in utilization of attractive forms and surface textures. Environmentally sensitive sites are spared many of the disturbances that accompany cast-in-place operations. Application of the system to single and multi-column bent shapes is considered based on the general range of applications for precast, pretensioned I-beam bridges. Current Texas bridge design practice and previous uses of industrialized processes in concrete substructure production worldwide are reviewed. Aesthetic substructure design for moderate-span highway bridges is addressed. Potential element fabrication and erection techniques are discussed, and examples of standard system element designs are presented. The aesthetic and economic impact of the proposed substructure system is considered.

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CHAPTER ONE
INTRODUCTION

1.1 Background

Highway bridge construction, particularly in the state of Texas, has benefited greatly from the use of standardized processes. Precast, pretensioned concrete I-beam construction with cast-in-place deck slabs has become the most cost-effective form of moderate span superstructure in the United States. This type of superstructure system, shown under construction in Figure 1.1, presently benefits from the use of several types of components produced by industrialized processes. These include precast slab panels which also serve as work platforms and formwork for subsequent casting operations. Modular concrete railings are chosen from a few standard designs that can be readily precast or slipformed. The backbone of the system consists of the girders themselves. A few standard shapes for these I-beam sections are widely used throughout Texas. Similar sections have been standardized by AASHTO for use throughout the country. The repetitive nature of the design, fabrication and erection of this superstructure system, coupled with

Figure 1.1: Precast, Pretensioned I-Beam Superstructure Construction

over thirty years of industry experience, makes this system very difficult to beat in terms of economics and performance.

In spite of the success of this superstructure type, nearly all substructure construction occurs in the field. The actual design of the basic substructure is quite standardized, and the shapes, dimensions and reinforcement layouts are highly repetitive. Although the cast-in-place processes utilized in substructure construction are relatively straightforward and very familiar to most contractors, the present method is time-consuming. The time and effort involved in the traffic management aspects of construction add substantially to the total structure cost. The sense of frustration imposed on motorists and pedestrians by the seemingly endless construction activity (see Figure 1.2) compounds the economic losses experienced by the surrounding community due to both traffic delays and impediments to business access.

Figure 1.2: Traffic Impediment Due to Construction

Cast-in-place substructures do not feature the enhanced durability that accompanies the use of high performance materials in precast, prestressed superstructures. A survey of the Texas Department of Transportation's BRINSAP inspection reports undertaken as a portion of TxDOT-CTR Project 1405 indicates that the major deficiencies which occur with prestressed concrete bridges are in the substructures.[1] Life-cycle costs can hardly be minimized if the least durable members are those which are often subject to the most aggressive attack. In addition to the agents that wash down from the superstructure, the substructure must withstand physical and chemical attack from below: earthborne chemicals, salt spray, ice, flowing water and debris, and air pollution.

Unfortunately, the cast-in-place processes that are presently utilized often result in unattractive substructures. Efforts to reduce the cast-in-place construction costs of the columns and bents have produced shapes that are easy to form but appear ponderous and dull. The multi-column bents used for most grade separation structures result in a visual effect, shown in Figure 1.3, which is often described as a "forest" of columns — a disorderly assembly of vertical elements

Figure 1.3: "Forest" of Columns

that belies the horizontal flow of the superstructure. Water runoff from the deck usually produces extensive and unsightly staining of substructure elements relatively early in the useful life span of the structure as shown in Figure 1.4. The relative proximity of the substructure to human observers (both highway users and neighbors) compounds the visual effect of damage due to aging.

The role of aesthetics in bridge design has become increasingly important in recent years. Designers are recognizing that taxpayers perceive bridges as more than supporting devices for their travels. Bridges are an inescapable part of the human environment. In a society that is constantly on the move, highway bridges represent the largest man-made structures encountered by most humans on a regular basis. The visual and emotional impact of a bridge project is of great

Figure 1.4: Staining of Substructure

importance, especially during an era of reputed increased wariness of the role of government in our daily lives. A beautiful bridge becomes a civic asset; an ugly bridge may be perceived as more government waste. Projects that imbue societal pride and acceptance of public works create long-term economic benefits that cannot be directly computed from cost estimates or bid prices.

The technology exists to produce aesthetically pleasing substructures that reap the construction benefits already realized in superstructure construction. The repetitive nature of substructure construction in large highway interchanges (see Figure 1.5) or a series of grade separations is such that the standardization of a few cross-sections for precasting could result in substantial cost savings. On-site construction time and related traffic management and financing costs would be greatly reduced. Use of precast, high performance concretes and post-tensioning technology would increase the durability and life expectancy of substructures, especially in aggressive environments. The appearance of the substructure would be enhanced by the increased structural efficiency and through the use of high quality forms and surface textures. Such surface treatments would serve as visually and economically attractive alternatives to the painting of concrete bridges and the maintenance associated with this practice.

With the preceding considerations in mind, the Center for Transportation Research (CTR) of the University of Texas at Austin began a research project numbered 1410 and entitled "Aesthetic and Efficient New Substructure Design for Standard Bridge Systems" in the Fall of

Figure 1.5: Repetitive Substructure Construction

1993. The project, cosponsored by the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA), has the following proposed objectives:[2]

1. To develop conceptual plans and visual guidelines for improving the aesthetics and efficiency of widely used moderate-span bridges systems;

2. To introduce more attractive structural forms and textures in substructures through 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 pretensioned I-beam superstructure system has proven very efficient in its wide use throughout Texas. As usage of this system has developed, its visual attractiveness as a superstructure has become apparent. The span lengths and slenderness ratios characteristic of this structural system can be readily integrated into a complete bridge system that makes aesthetic sense. Therefore, the thrust of this project is to modify the design and construction of substructures in order to increase the overall efficiency and beauty of bridges built with concrete I-beam substructures.

Because the characteristics of the superstructure system such as span length and girder depth will not be altered, aesthetic improvements should be possible with only minor changes in construction costs. In fact, some projects may see savings. More efficient fabrication and erection of substructure components are possible through the use of high performance materials and improved form systems. Decreased on-site construction time reduces traffic disruptions and the associated financial and public relations costs. Higher quality materials and surface textures can reduce maintenance costs. Improvements in overall aesthetic appeal will bolster public acceptance of projects and reduce animosity towards future public works projects.

In the Fall of 1994, TxDOT informally presented the investigators of Project 1410 with a trial project encompassing twin bridge structures for U. S. Highway 67 to span U. S. Highway 87 and the North Concho River in San Angelo, Texas. The investigators offered to apply previously developed aesthetic guidelines to the proposed bridges and make design recommendations to TxDOT. Upon visiting the site, the investigators discovered that the bridges were to be located in a park area of civic importance. Recommendations were made to lighten the impact of the bridges on the park, imbue a sense of order in the span layout and improve the appearance of the bridge substructures from the point of view of the park and frontage roads. TxDOT implemented several of the recommendations. Unfortunately, some of the recommendations were not accepted because the project was too far along in the design process. Of particular significance concerning the topic of this thesis was TxDOT's reluctance to design the columns and bent caps as precast members.

The bridges in question were already the subject of a separate TxDOT research study Project 589 entitled "High Performance Concrete for Bridges." In accordance with that study, the precast, pretensioned I-beam superstructure of the bridge supporting westbound traffic was to be constructed using concrete with 28-day compressive strengths ranging from 69 MPa to 90 MPa (10-13 ksi). The proposed substructure system in Project 589 was to consist of precast bent caps at interior bents. The contractor was to be given the option of using precast or cast-in-place columns. Substructure concrete was to have a 28-day compressive strength of 55 MPa (8000 psi).[3]

Among the recommendations concerning aesthetics and efficiency given to TxDOT by the investigators of Project 1410, several pertained to substructure construction. First, they recommended that all interior bents have a single vertical pier shaft to minimize the physical and visual impact on the park area. Because the final bridge widths were as yet uncertain, it was noted that this recommendation might not be feasible for a few bents. In order to further increase the transparency of the substructure, the possibility of placing one or more central openings or “windows” in each pier shaft was suggested. The use of a "fractured fin" surface texture was recommended for the areas above and below the shaft windows.

The investigators agreed with the earlier proposal to design the interior bents of the westbound bridge for precasting. In addition, they recommended that all substructure elements (shafts and bent caps) on both bridges be precast. Since certain components of the westbound bridge substructure were to be precast, introducing a completely different cast-in-place system for a twin structure on the same project site did not make sense. The economic benefits of precast construction are realized through repetitive construction. Rather than taking the results of a learning curve already established on the westbound bridge and applying it to the eastbound substructure construction, the utilization of cast-in-place construction would disregard the acquired efficiency and start at the beginning of a completely new learning curve. Furthermore, the investigators believed that the use of precasting would help offset the cost of the shaft windows. The extra costs involved with forming these openings and casting high strength concrete around them could be decreased by casting the shafts as a series of match-cast segments. Precasting would also result in a more uniform and higher quality surface texture, especially on the fractured fin surfaces. The Project 1410 investigators maintained that the surface texture of the shafts would play a major role in the public acceptance of the bridges. These shafts would form the most visible intrusion on the river park area, and their attractiveness on a human scale at close range would largely define the bridges' environmental impact on this valued space. Finally, the investigators saw the pioneering use of high strength concrete in this project as an excellent opportunity to develop a new precast substructure system to enhance the advantages of this material and explore possible future applications.

For these reasons, the Project 1410 investigators proposed that all shafts and bent caps be constructed from a series of match-cast precast segments. The size of these segments would be selected so as to not require lifting equipment with a capacity beyond that already needed for placement of the pretensioned girders. The site layout was such that precasting operations might be possible at or near the site, therefore reducing transportation costs. Furthermore, it was proposed that segments be connected with post-tensioned, high strength threaded bars. These bars are easily coupled and require post-tensioning hardware that can readily be handled by one or two workers. Joint locations could be specified to ease the formation of the shaft windows.

The TxDOT reaction to these recommendations was mixed. Leaders of the TxDOT Design Division Bridge Section agreed with the recommendations concerning the outward appearance of the bents. The windowed shafts and fractured fin surface texture were incorporated in the final design. They also agreed that using two different construction systems for the two bridges did not make aesthetic or economic sense. However, instead of precasting the substructures as recommended, TxDOT chose to utilize conventional cast-in-place construction for both bridges. They indicated both funding concerns and uncertainties regarding shaft to bent cap connections as the primary reasons for this choice. A short time frame had previously been established for the funding of this project. If the project was not completely ready for bidding by a certain date, the construction would be postponed indefinitely. The deadline was such that the structural design had to be completed within two months. According to TxDOT, this time frame was not sufficient to adequately develop a precast design option. Therefore, cast-in-place construction was specified for the substructure.

Several important lessons were gleaned from this experience. Although the proposed use of high strength materials in the substructures offered an excellent opportunity to develop a new substructure system, the design and funding time frame did not. Funding pressures and an overall sense of uncertainty on the part of TxDOT regarding the design, construction and cost of a precast substructure system combined to prevent its consideration as a bid alternative. Apparently, the main shortcoming of the precast substructure concept was the lack of detail. The advantages of such a system had not been fully explained and documented. Examples of successful use of precast substructures should have been fully outlined. The necessary technology should have been explained and illustrated. Economic impact and construction costs should have been clarified. Most importantly with regard to TxDOT acceptance and support, a more detailed design should have been proposed. This design should have included member sizes and weights, reinforcement amounts, connection details and a feasible construction sequence.

As a direct result of this experience, development of a standardized precast substructure system has continued with these lessons in mind. Attempts have been made to dispel the uncertainties and reservations concerning the implementation of precast substructures. Development of a detailed, coherent system design that may be implemented economically and attractively has progressed as part of this research study. The results of this work are detailed herein.

1.2 Objectives

The research outlined in this thesis focused particularly on the second and third project objectives listed in the previous section. These objectives pertain to the development and implementation of precast substructure systems. The two primary goals for introducing such a system are evident in these two objectives. The first goal is to utilize precast techniques to improve the aesthetics of bridge substructures. The second goal is to apply construction efficiency benefits of precast superstructure systems to substructure construction. Therefore, the specific objective of this research is to apply readily available materials and technology to develop an efficient and attractive precast substructure system for precast, pretensioned I-beam bridges in the state of Texas.

Several performance criteria were identified for the development of an efficient substructure system. An ideal system would have the following qualities:

1. Flexibility
2. Repetition of Elements and Details
3. Minimal On-site Construction Time
4. Structural Efficiency
5. Durability
6. Enduring Aesthetic Value

The first four items on this list pertain to the system's efficiency in terms of construction costs. As in the case of precast, pretensioned superstructures, the ultimate economic success of a precast substructure system depends on the efficiency realized through its repeated use. Therefore, the system must be repetitive yet flexible, bearing the economic benefits of repeated use of capital and processes while offering a variety of applications to the bridge designer. Repetition of construction processes results in decreased costs. Flexibility, or the ability to adapt to geometric constraints imposed by site conditions, roadway geometries and girder placement tolerances, translates into enhanced applicability. This, in turn, provides for the repeated use of the entire system.

Reduction of on-site construction time decreases the costs associated with managing a construction site and the impact upon its immediate surroundings. No roads are built "in the middle of nowhere." The impact of construction on the surrounding environment results in costs to the environment and costs to the constructor faced with the task of mitigating this impact. Almost all highway construction projects include the costly task of managing existing traffic. However, the cost of managing traffic is very small compared to the economic burden and emotional inconvenience placed upon the traffic being managed. Traffic delays translate to wasted money, time and fuel in addition to an overall frustration with public works projects which may become manifest during the next bond referendum or election.

Structural efficiency translates into material savings. Substructure efficiency may pay material dividends in other elements as well. For example, reduced foundation costs might result from a decrease in substructure weight.

The last two criteria pertain to the cost efficiency of the bridge during its service life. The durability of a bridge determines the maintenance costs required for the structure to meet or exceed its required lifespan. Premature repair or replacement of a bridge greatly increases its overall cost to the taxpayer. Maintenance costs increase the actual lifetime bridge cost well above the original construction price.

Aesthetic issues have a bearing on bridge costs as well. Obviously, attention to visual appearance may add considerably to the bid price of a bridge, especially if the visual improvements consist of add-ons or ornamentation. However, this should not be the case if aesthetic issues are considered throughout the entire design process. Careful attempts to integrate form and function may result in a construction cost that is only slightly higher — or even lower — than that for a design which does not consider aesthetics at all.

Aesthetics can greatly affect the lifetime cost of a structure. The visual appearance of a bridge determines its psychological value to the society it serves. In addition to the costs of maintaining a bridge's function, there are costs associated with the maintenance of its visual appearance. As shown in Figure 1.6, painted bridges must be repainted. This transforms a traditionally *low* maintenance material into a *high* maintenance material. Furthermore, this increased level of maintenance is required only for aesthetic impact. Stains and graffiti must be removed or covered. Generally, graffiti artists seem to prefer large, flat surfaces as "canvases" for their graphic expressions. Introduction of textured surfaces should encourage them to look for more ideal "canvases" elsewhere. A small amount of money invested initially in the lifetime appearance of a bridge may pay substantial dividends in the reduction of maintenance costs over its entire lifespan.

Figure 1.6: Columns in Need of Repainting

The criteria outlined above were utilized to determine the characteristics of the proposed substructure system. In light of the experience gained from the trial San Angelo project, the aim of the research is not only to develop the *concept* of a precast substructure system, but to provide enough design *details* so that when the use of such a system is considered, its inherent efficiency is not overshadowed by questions and doubts regarding its constructability.

1.3 Scope

The scope of this research included three major tasks: a literature review, a photographic survey of typical highway bridge construction in the state of Texas, and the design of a standardized substructure system. The literature review and photo survey provided the necessary background information for producing a worthwhile substructure system.

1.3.1 Literature Review

Literature was reviewed for information on three main topics: current design and construction practice for concrete highway bridges, applications of standardization and precast technology to bridge substructures, and bridge aesthetics. Information on current design and construction practice was obtained primarily from TxDOT's *Bridge Design Guide*[4] and conversations with TxDOT Design Division personnel. Available technology options were identified and reviewed. Copious information concerning bridge aesthetics was identified. However, literature pertaining to the aesthetics of moderate-span highway bridges is more limited, and there are very few sources that address substructure aesthetics for this category of bridges.

1.3.2 Photographic Survey

Researchers conducted trips to various cities and regions in the state of Texas collecting photographic records of bridges. The survey concentrated on short- to medium-span concrete highway bridges. Techniques of bridge design and construction, both past and present, were noted. Particular emphasis was placed upon the visual appearance of the completed structure and its interaction with the surrounding environment.

1.3.3 Development of Standardized Substructure Design

Development of a standardized substructure design proceeded with the previously stated objectives and performance criteria in mind. Effort was concentrated on presenting a design that is both visually appealing and readily constructable. Member sizes and shapes, along with fabrication and erection methods, were evaluated and selected. Details were addressed to minimize unanswered questions regarding the feasibility of the design.

1.4 Summary of Subsequent Chapters

Chapter Two summarizes the present state of the art of the two subject areas synthesized in this research. Present TxDOT design practice for moderate-span concrete girder highway bridges is outlined while uses of standardized substructure construction, both past and present, are discussed. Theory regarding substructure aesthetics is addressed in Chapter Three. Available technology options for the standardization of bridge substructures are discussed in Chapter Four. The advantages and disadvantages of various fabrication, joining and erection methods are explained.

Chapter Five deals with the construction and structural considerations that shaped the design of the system. The candidate system itself is laid out in Chapter Six. Both the aesthetic and economic impacts of the system are discussed in Chapter Seven. In Chapter Eight, conclusions and recommendations regarding the future use of the system are presented.

1. Larosche, C. L., “Test Method for Evaluating Corrosion Mechanism in Standard Bridge Columns,” M.S. thesis, University of Texas at Austin, Dec. 1996.
2. “Aesthetic and Efficient New Substructure Design for Standard Bridge Systems,” Proposal for Texas Department of Transportation Study 0-1410, Center for Transportation Research, University of Texas at Austin, Revised Nov. 17, 1993.
3. “High Performance Concrete for Bridges,” Proposal for Texas Department of Transportation Study 9-589, Center for Transportation Research, University of Texas at Austin, Aug. 17, 1994.
4. Texas State Department of Highways and Public Transportation, *Bridge Design Guide*, First Edition, Austin, Texas, 1990.

CHAPTER TWO
STATE OF THE ART

2.1 Introduction

The initial part of this chapter consists of a brief outline of Texas Department of Transportation (TxDOT) design practice for moderate-span highway bridges, particularly those with pretensioned girder superstructure systems. This information was gleaned from TxDOT's *Bridge Design Guide*[4] and conversations with Design Division personnel. Typical geometric constraints are presented, as well as the design characteristics of the superstructure and substructure systems most commonly utilized in Texas moderate-span highway bridge construction.

Advancements in the development of industrialized processes for substructure construction are described in the final portion of this chapter. Successful application of these processes, particularly the utilization of precast members, in past and ongoing projects is described.

The successful integration of the two subject areas presented in this chapter is the ultimate goal of the research presented in this thesis. The extensive utilization of industrialized processes for substructure construction should result in more efficient and durable bridges while increasing the aesthetic flexibility available to the designer.

2.2 Overview of Texas Design Practice for Short- to Medium-Span Highway Bridges

2.2.1 Geometry

2.2.1.1 Roadway Geometry

In modern TxDOT design practice, the bridge roadway is designed to connect the approach roadways with little or no geometric discontinuities noticeable to the driver. Aesthetic and safety issues mandate that bridge travel lane and shoulder widths correspond with those of the approaches. Roadway alignment generally should not be impeded by the presence of a bridge. Bridge roadway geometry is therefore subject to the requirements of the Highway Engineer.

2.2.1.2 Roadway Width

The typical travel lane is 3.6 m (12 ft) wide. Shoulder widths vary from 0.6 m (2 ft) to 3 m (10 ft) depending on roadway volume and function. Combination of these requirements results in a considerable number of possible roadway widths. TxDOT standardized roadway widths are 7.2 m (24 ft), 8.4 m (28 ft), 9.0 m (30 ft), 11.4 m (38 ft) and 13.2 m (44 ft). The term "standardized" denotes any aspect of a bridge (roadway width, girder size, drainage detail, etc.) that has been incorporated into one or more sheets of premade drawings, or "standards," that can be readily cited and inserted into the design drawings for any bridge. Roadway widths of over 30 m (100 ft) are possible for structures carrying many lanes, such as mainlane highway grade separation structures. The overall width of the concrete deck consists of the roadway width in addition to a 300 mm (1 ft) nominal safety barrier width on each side. For example, a roadway width of 11.4 m (38 ft) results in a corresponding deck width of 12.0 m (40 ft). Separate structures are often constructed to support traffic traveling in opposite directions.

2.2.1.3 Horizontal and Vertical Clearances and Heights

Minimum horizontal and vertical clearances are specified for safety reasons and to prevent damage to bridge components. Some of the more relevant clearances are listed in Table 2.1.

These clearance requirements, in conjunction with vertical roadway alignment and site topography, result in typical bridge heights ranging from 6 m (20 ft) to 12 m (40 ft). Intracoastal Canal Bridges and multi-level interchanges frequently feature bridges or ramps with roadway heights approaching 24 to 30 m (80 to 100 ft).

2.2.1.4 Alignment and Cross-Slope

Wide bridges typically have horizontal curvatures of up to 5°. Horizontal curves of up to 10° are typically found on narrow connection structures. Tight curves with long spans may deter the use of prestressed concrete I-girder construction. Use of a straight I-girder in such cases of extreme geometry will result in unfavorable edge distances. Straight I-girders may be used for 30 m (100 ft) spans when the curvature is less than approximately 11.5°. For 40 m (130 ft) spans, the curvature must be less than approximately 6.7°. Curved structural steel plate girders are often used when horizontal curvature is beyond acceptable limits for straight I-girders, often with unpleasant aesthetic consequences.

Highway Separation Structures		
	Vertical	5 m (16.5 ft) if possible (mandatory for new interstate highway construction)
	Horizontal	3 m (10 ft) low speed, low volume roadways
		4.8 m (16 ft) medium volume roadways and freeway ramps
		9 m (30 ft) high volume roadways and freeways
Railroad Overpasses (Highway over Rail)		
	Vertical	6.9 m (23 ft) to 7.8 m (26 ft)
	Horizontal	2.6 m (8.5 ft) absolute minimum
		3.6 m (12 ft) desired minimum
		7.5 m (25 ft) without crash walls
Railroad Underpasses (Highway under Rail)		
	Vertical	Same as for highway separation structures
	Horizontal	4.8 m (16 ft) desired minimum
Stream Crossings		
	Vertical	600 mm (2 ft) above design high water level
	Horizontal	Determined by topography and hydraulics
Intracoastal Canal Bridges		
	Vertical	22 m (73 ft) above mean high water level
	Horizontal	38 m (125 ft) from center of channel

Table 2.1: Vertical and Horizontal Clearance Minimums for TxDOT Bridges (from Ref. 4)

Vertical curvatures rarely affect the selection of structure type for highway bridges. Selection of girder camber must include consideration of vertical curvature. Grades in excess of 5% can cause problems with deck concrete placement and elastomeric bearing details.

A minimum cross-slope of 10 mm/m (0.01 ft/ft) is required for effective drainage. A cross-slope of 20 mm/m (0.02 ft/ft or 1/4 in/ft) is usually specified. The maximum allowable superelevation is 80 mm/m (0.08 ft/ft). However, difficulties in placing slab concrete usually preclude the use of values greater than 50 mm/m (0.05 ft/ft). Geometrical variations imposed by vertical curvature and cross-slope are ordinarily accommodated in construction by the setting of the forms for the cast-in-place caps. Fine adjustments are made in the subsequent construction of the cast-in-place bearing seats and placement of the elastomeric bearing pads used for girder support.

2.2.1.5 Skew Angles

Skewed bridges are those for which the longitudinal axis of the superstructure is not perpendicular to the longitudinal axis of the substructure bent caps. As shown in Figure 2.1, the horizontal angle (α) between the bent cap axis and the transverse cross-section of the supported superstructure indicates the degree of skew at each bent. Standard Texas highway bridge skew angles are 0º, 15º, 30º and 45º. Skews over 60º are occasionally designed. Skew angle determines the interior bent length required to support the superstructure. For example, a bridge with no skew supporting an 11.4 m (38 ft) wide roadway would require a bent cap length of approximately 11 m (36 ft). However, the same roadway width, supported by a bridge with a skew angle of 45º, would require a cap length of 15.6 m (51 ft). A change in skew angle may also result in a different bent configuration. A single column bent could be used for the non-skewed bridge in the preceding

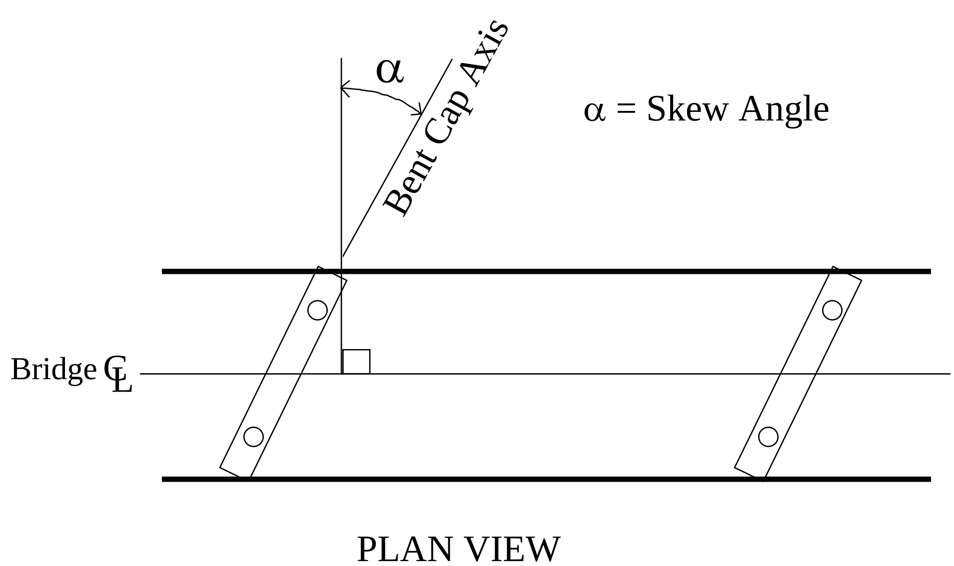

Figure 2.1: Bridge Skew Angle (α)

example, but the 45º-skew bridge would generally require more than one column to support the required length of cap. Cap length for a skewed bridge can be estimated using Equation 2.1.

$$L_c = \frac{L_{c,0}}{\cos\alpha} \qquad \text{(Eqn 2.1)}$$

L_c = Length of bent cap required for skewed bridge

$L_{c,0}$ = Length of bent cap required for bridge with no skew ($\alpha = 0$)

α = Skew angle of bridge

2.2.3 Superstructure

2.2.3.1 General

The superstructure system consists of the bridge elements that act in concert to transfer the applied loads transversely from their concentrated position on the deck to the longitudinal spine elements and then longitudinally along these girder spines to the support locations. The bridge deck, diaphragms and longitudinal stringers typically constitute the superstructure of moderate-span highway bridges in Texas. District design engineers generally choose the type of superstructure and perform the preliminary layout for the structure to satisfy the often severe boundary conditions imposed by restrictions on pier locations due to traffic, stream, or land use considerations. They are usually aware of the capabilities of the various superstructure systems available and the relative benefits and costs of each given the particularities of the district. They may also consult with their Bridge Planning Engineer or Engineer of Bridge Design.

2.2.3.2 Prestressed Concrete I-Girder Superstructure System

"Nothing has been so beneficial to the economy and durability of Texas highway bridges as precast pretensioned concrete beams." So reads the first sentence of Section 5.7 of TxDOT's *Bridge Design Guide*[4]. This section outlines the design practice of simple-span prestressed concrete I-beams in the state of Texas.

Prestressed beam spans were first offered as alternates to continuous steel girders in the 1950's during the early years of interstate highway construction. The development of reliable high-strength concrete technology and the introduction of standard shapes soon made it impossible for steel to compete economically for span lengths of up to 40 m (130 ft). The same basic shapes developed over 35 years ago are still in use today. Standard span and bent details were developed in 1957 for typical interstate highway bridge geometries of the time. These standards have been updated over time.

Advantages of prestressed I-girder bridges include:

1. Computerized design and standardized beam details,
2. Suited for most geometric conditions and bridge types,
3. Several competitive sources,
4. Most economical for span lengths ranging from 14 m (45 ft) to 44 m (145 ft),
5. Most durable superstructure for its span capability, and
6. Nearly forty years of design and construction experience.

Disadvantages of this superstructure system include:

1. Depth is not minimized,
2. The present system in Texas uses only straight girders which cannot be curved to meet requirements of extreme geometry,
3. Occasionally difficult to accurately predict deflections, and
4. Long beams sensitive to handling stresses.

Problems with unexpected deflections are less frequent now that material quality and workmanship have become more uniform. Occasional problems arise, generally under unfamiliar conditions. Clearly the advantages of prestressed I-beam construction outweigh the disadvantages with regard to constructability and the economic "bottom line."

For the first eight months of 1995, bridge projects that included prestressed I-beams represented 70 percent of the bridge contracts let by TxDOT. The average cost of these structures was $340/m^2 ($31/ft^2). New bridge projects supported by a superstructure consisting of only prestressed I-beam spans made up 54 percent of the total. The average cost of these bridges was $320/m^2 ($29/ft^2). The economic benefits of the prestressed concrete I-girder superstructure are indisputable. The resulting predominance of this bridge type places it at the focus of this project.

2.2.3.4 Material Properties

Prestressed I-girders with concrete compressive strengths of 45 MPa (6,500 psi) at release and 55 MPa (8,000 psi) at 28 days are readily available in Texas. Most precast plants can supply 83 MPa (12,000 psi) concrete if required. One plant is producing 90 MPa (13,000 psi) concrete for experimental structures. Low-relaxation, 13 mm- (0.5 in-) diameter, 1860 MPa- (270 ksi-) ultimate strength steel strand is the standard pretensioning tendon. TxDOT is a leader in the research and development of 15 mm- (0.6 in-) diameter strand. This size strand has already been used in a few pilot bridge applications, and widespread use is likely in the near future.

2.2.3.5 I-Beam Types

Figure 2.2 illustrates the standard cross-section shapes most prevalent in Texas highway bridge construction.

The Texas Type A beam has a depth of 700 mm (28 in). It is used primarily to provide uniformity of depth when widening older bridges. Span length is usually limited to approximately 15 m (50 ft).

The Texas Type B beam has a depth of 850 mm (34 in). It is also used for widening existing structures and for new structures for which depth is important. This beam can be used for span lengths ranging to approximately 20 m (65 ft).

The Texas Type C beam is one of the two most prevalent beam types used in present construction in Texas. It is a multipurpose beam with a depth of 1000 mm (40 in) that can span up to (30 m) 100 ft. The most economical span range is about 23 m (75 ft) to 27 m (90 ft).

The AASHTO Type IV beam has been the most popular beam in Texas bridge construction since 1986. It features a depth of 1350 mm (54 in) and has a reasonable span limit approaching 42 m (138 ft). Typical span lengths are 33 m (110 ft) to 40 m (130 ft).

Other beams include the Type 54, Type 72 and AASHTO Type IVM. These generally feature less lateral stability and are more difficult to transport than the previously listed types. Use of these beams is rare. The vast preponderance of prestressed concrete I-beams used in new construction consists of either Type IV or Type C beams.

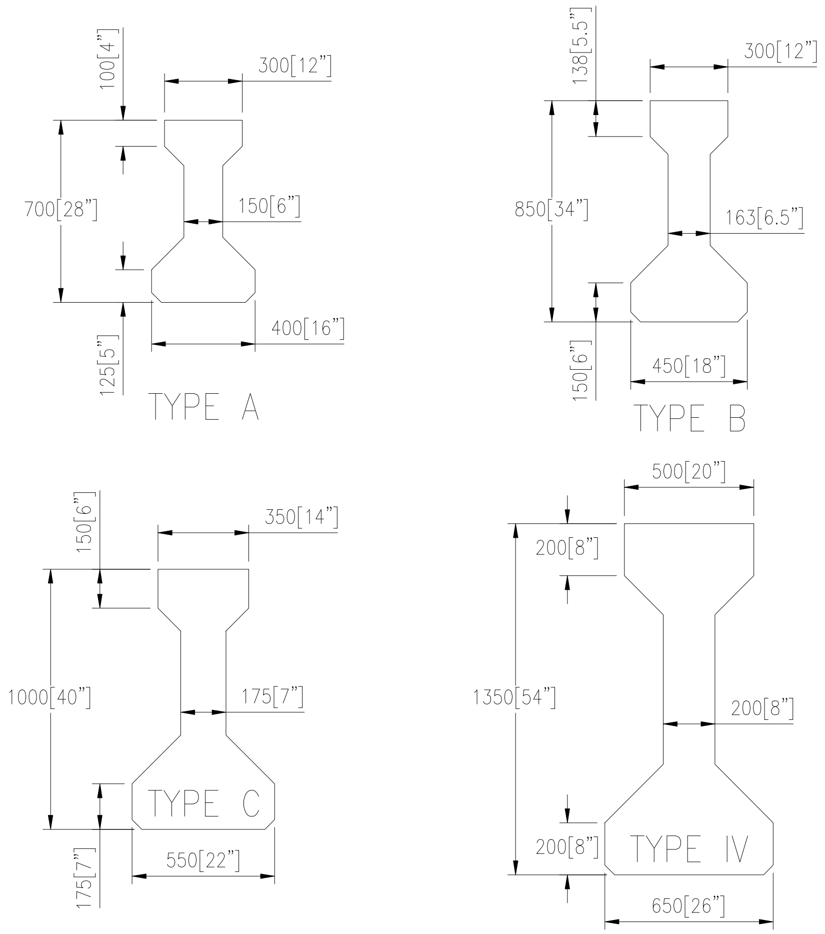

Figure 2.2: Standard I-Beam Cross-Sections

2.2.3.6 Girder Spacing

Spacing of I-beams depends on the load to be carried and the span length. The primary design limitation for prestressed concrete beam spans is the service load tensile stress at the bottom fiber of the beam. This stress is generally limited to $0.5\sqrt{f'_c}$ MPa ($6\sqrt{f'_c}$ psi). Center-to-center spacing of I-beams ranges from 1200 mm (4.0 ft) to 2900 mm (9.5 ft). The length of deck overhang measured from center of exterior beam to edge of deck slab usually varies between 560 mm (1.83 ft) and 1420 mm (4.67 ft). Overhangs of 900 mm (3.0 ft) are used for most bridges.

2.2.3.7 Deck Details

Ninety-nine percent of Texas bridges have reinforced concrete deck slabs. One-way deck slab design has been standard practice since the beginning of Texas bridge design. Slabs are designed as flexural beam strips supported by stringers (I-beams in this study). Therefore, primary deck reinforcement is placed transverse to the length of the bridge. Typical slab thicknesses range from 180 mm (7.25 in) to 210 mm (8.25 in).

Use of precast concrete panels as stay-in-place forms to span between I-beams was initiated in 1963. This technique has been so successful that use of prestressed concrete panels topped with cast in place concrete is the most preferred method of deck slab construction for concrete I-beam bridges.

In the 1960's TxDOT experimented with the design of I-girder superstructures continuous for live load. The primary goal of this design was the reduction of damage caused by the leakage of water through the deck joints over each bent. Deterioration of bent caps and columns has often resulted from the leakage of salt-laden water through deck joints. Beam compressive regions were joined by cast in place diaphragms while mild steel continuity reinforcement was placed in the deck slab across the bent. A later design featured an inverted-T bent cap with the stem cast around the I-beam ends. These designs proved to be quite complicated. Fabrication difficulties included the use of extra hardware and the accurate alignment of large diameter bars. Many diaphragms suffered from spalling. Scheduling problems resulted from the fact that stems of inverted-T bent caps could not be cast until beams were in place. Deformations proved to be difficult to predict. This type of design is no longer recommended by TxDOT.

Presently, simply-supported, precast prestressed concrete I-beam spans with only deck slab continuity, shown in Figure 2.3, is the system preferred by TxDOT. Often referred to as “poor-boy continuity,” this method of construction addresses the problem of deck joint leakage without invoking the difficulties of live load continuity. Girders are simply-supported on elastomeric bearing pads. Slab continuity is achieved by extending all longitudinal slab reinforcement across the bents. Continuous slab units are limited to lengths of 120 m (400 ft). Longitudinal movement must be provided for at the ends of the continuous slab units. Ease of construction is greatly improved because fewer end diaphragms are required and variable length reinforcement is no longer necessary at the continuous ends of skewed spans. A construction joint over each bent is recommended in order to straighten and hide the unavoidable crack. End diaphragms are used at expansion joint locations. These diaphragms are often formed with the slab in order to ease construction.

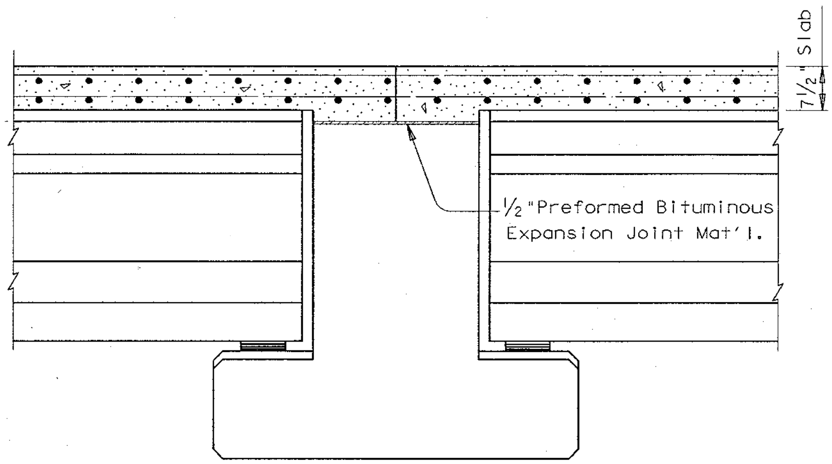

Figure 2.3: Deck Slab, or "Poor-Boy," Continuity (from Ref. 4)

2.2.3.8 Traffic Barriers and Railings

Several types of traffic barriers are used on Texas bridges. Cross-sectional shapes of the most prevalent traffic rail types are shown in Figure 2.4. The type of rail chosen can significantly affect the perceived structural slenderness of the bridge. For example, the T501 and T201 rails are completely opaque when viewed in elevation. Therefore, these rails appear to occupy a portion of

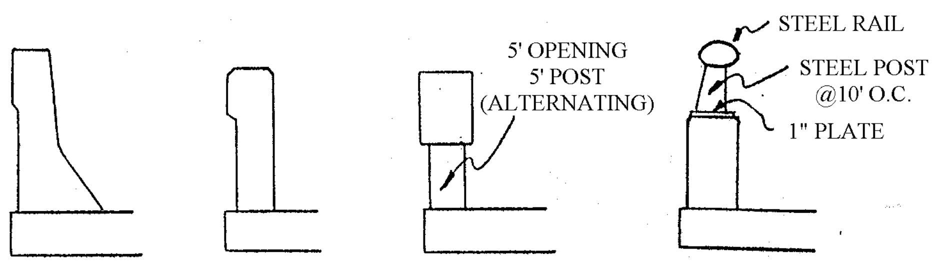

Figure 2.4: Traffic Barrier Cross-Sections

the structural depth of the superstructure, thus making the bridge appear less efficient than it actually is. However, the T202 barrier, with its alternating 1.5 m (5 ft) openings beneath the rail, and the T4 barrier, with its steel guardrail supported above shorter concrete parapet, both appear in elevation to be exactly what they are — railings that do not participate in the load-bearing function

Figure 2.5: Type T411 "Aesthetic Rail"

of the superstructure. The T411 barrier, shown in Figure 2.5 and deemed “the aesthetic rail” by TxDOT, also transmits a true expression of its non-structural function with its repetitive vertical windows and old-fashioned detail. The perceived structural slenderness ratio ranges for each of these barriers in combination with the four most common I-beam types are given in Table 2.2.

I-Beam Type	Traffic Barrier Type				
	T501	T201	T202	T4	T411
A	3.4–8.6	3.7–9.2	6.3–16	4.3–11	6.3–16
B	4.7–10	5.1–11	8.2–18	5.8–13	8.2–18
C	8.8–13	9.4–14	14–22	11–16	14–22
IV	11–16	12–17	17–24	13–19	17–24

Table 2.2: Perceived Slenderness Ratios for Various Combinations of Rail and Beam Types

2.2.3.9 Aesthetic Considerations

Presently, use of a uniform beam depth is recommended for highway grade separation structures. This provision is intended to provide a visually pleasing appearance to the motorist traveling on the roadway being spanned. For stream crossings, on the other hand, use of different beam types and depths is accepted. Current design practice asserts that there is "no significant advantage" to maintaining constant spacing between I-beams of different spans. Apparently, both the amount of time and observation effort expended beneath a bridge by motorist or pedestrian does not balance the simple economic benefit gained from varied beam spacings.

2.2.4 Substructure

2.2.4.1 General

The bridge substructure consists of the components that collect forces from the ends of each superstructure span and transfer these forces to the ground. Substructure units may include abutments, frames, or individual columns or piers. The type of substructure used on a Texas highway bridge is usually determined by the Design Division. The typical substructure system consists of abutments at the ends of the bridge and interior bents between spans.

2.2.4.2 Abutments

Texas bridges generally have two abutments, one for each end of the bridge. These are typically "stub" type abutments as shown in Figures 2.6 and 2.7. Retaining walls are often required due to the premium placed on space in urban construction. As is the case in Figure 2.8, these retaining walls often take the place of the abutment wingwalls. A U-type abutment is formed when the retaining walls pass in front of the abutment backwall. Strengthened earth retaining walls have become extremely popular in recent construction. Use of various surface texture pattern combinations has improved the visual aspect of these structures. Design of the "stub" abutment itself is completed with the use of standard details.

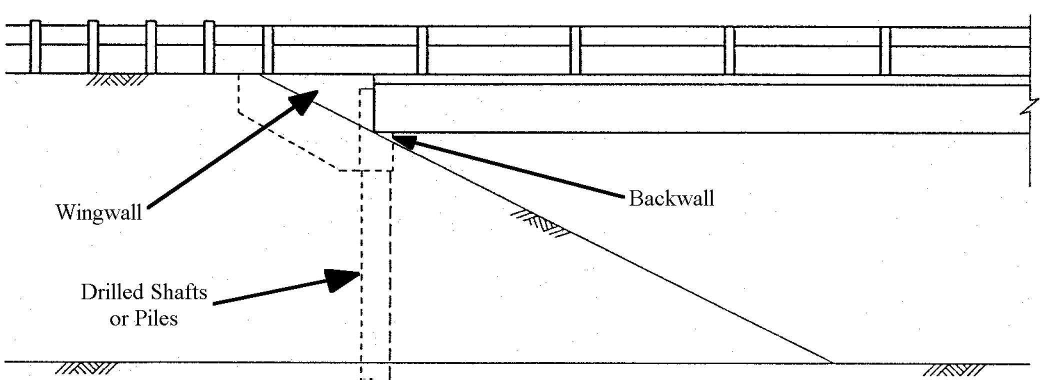

Figure 2.6: “Stub” Type Abutment (from Ref. 4)

Figure 2.7: Typical "Stub" Abutment with Wingwall

2.2.4.3 Interior Bents

Figure 2.8: "Stub" Abutment with Retaining Wall

An interior bent usually consists of a horizontal flexural member, or bent cap, that supports the stringers (prestressed concrete I-beams) and one or more vertical members, or columns, that transfer the bent cap loads to the foundation. The foundation effectively disperses the vertical and horizontal forces into the ground. There are four main types of bents used in Texas: single column frames, multiple column frames, trestle pile frames and wall piers. Some typical bents are illustrated in Figures 2.9, 2.10, 2.11, 2.12 and 2.13.

Most interior bents are multiple column frames, like that shown in Figure 2.9, consisting of circular columns and rectangular caps. These are used for the majority of stream crossings and highway grade separation structures. If a multiple column bent is very tall, an intermediate tie beam may be utilized to decrease the unbraced length of the columns.

Figure 2.9: Multiple Column Bent

Single column bents like the one in Figure 2.10 are used for ramp and connector structures in interchanges. They may also be used in other structures where aesthetics are important, so long as the deck width is not too large. Single column bents generally improve the visual quality of the bridge by lessening the impact of column clutter and increasing the apparent slenderness and efficiency of the superstructure.

Figure 2.10: Single Column Bent

Shown in Figure 2.11, the trestle pile bent is a special type of multiple column frame used for some stream crossings and low bridges where a large clearance is not required. The foundation piles are extended to the bent cap and serve as the columns for the bent.

Wall piers have been utilized as interior bents extensively in the Dallas/Ft. Worth metropolitan area. The piers generally extend the entire width of the bridge. Pier bents are often utilized for stream crossings because they offer greater resistance to hydraulic forces than column bents.

Straddle bents, like the one shown in Figure 2.12, are often used to support a ramp or roadway that passes directly over an obstacle, such as another roadway, that precludes the use of a single column bent.

Figure 2.11: Trestle Pile Bent

Reinforced concrete interior bents are usually constructed of concrete with a 28-day compressive strength of 25 MPa (3,600 psi). Reinforcing steel typically has a yield strength of 414 MPa (60 ksi).

Figure 2.12: Straddle Bent

2.2.4.4 Bent Caps

Bent caps are basically rectangular reinforced concrete beams. These caps are primarily prismatic. Soffits of cantilever ends are often chamfered. This practice is standard for multiple column bents with rectangular caps. As illustrated in Figure 2.13, the chamfered soffit better indicates the flow of forces into the column.

Figure 2.13: Chamfered End of Cap

The inverted-T cap was introduced in the 1970's in an effort to reduce the apparent thickness of the bent cap. This innovation also reduced the visual clutter in the region where the superstructure meets the substructure, allowing these two systems to combine, rather than collide. These caps are recommended for single column

Figure 2.14: Tapered Cap Soffits

bents supporting concrete I-beams. They are also frequently used for multiple column bents. Unlike rectangular caps, most inverted-T caps are of a constant depth throughout. A few structures, such as the elevated I-35 structures in Austin seen in Figure 2.14, have sloping soffits for the cantilever beam ends. As with rectangular caps, this detail imparts a sense of structural efficiency and makes a clear visual statement of the purpose of the member.

Rectangular bent caps are generally designed as non-prestressed reinforced concrete beams. The design of inverted-T caps is more complicated, largely due to the application of concentrated loads to ledges at the bottom of the beam. Hanger reinforcement must be provided to "pick up" and distribute the vertical loads to the top of the beam. The ledge must be designed to have adequate transverse capacity in flexure and shear.

Congestion of reinforcing steel is often a problem in bent cap construction. Placing concrete effectively and efficiently can be difficult. Frequently, reinforcement cages require internal stiffening, which results in even more congestion and difficulty with concrete placement.

Prestressing of bent caps has been used in rare situations. Caps of straddle bents have been post-tensioned in order to enhance flexural strength. Texas examples include the U.S. 290 / Loop 360 Interchange and the U.S. 183 Viaduct project shown in Figure 2.15, both in Austin, and the I-10/I-35 "Y" Project in San Antonio.

Figure 2.15: Post-tensioned Straddle Bent

2.2.4.5 Columns and Pier Shafts

Analysis of columns and pier shafts for interior bents is quite complex. The combination of axial and lateral load effects, the iterative nature of analyzing second-order effects, and the uncertainty in determining accurate values of the elastic modulus, E, and moments of inertia, I, throughout the column make predicting the behavior of a column or group of columns very difficult. Texas highway columns are generally designed using a moment magnification process based on elastic behavior, resulting in conservative designs. Earthquake effects are not considered.

Round columns are used in the vast majority of multiple column bents. Column diameters are standardized. Each standard column size has a predetermined height limit. A standard column size may be utilized without analysis so long as 1) the design height is less than the specified limit and 2) the cap capacity is adequate for the provided column spacing. Square columns have been used in rare situations for the purpose of enhancing aesthetics. Because design experience is more limited for square columns in multiple column bents, these columns must be analyzed, particularly for biaxial bending effects.

Single column bents usually feature rectangular columns. The corners of these columns are often rounded or chamfered. Single columns may be tapered in either the longitudinal or transverse directions (see Figure 2.16), but not in both directions.

Aside from the pretensioned driven piles used in trestle pile bents, precast and/or prestressed columns have been used sparingly in Texas practice. Two applications of precast, segmental column construction presently under construction are the Louetta Road Overpass in Houston and the U.S. 183/I-35 Interchange in Austin.

Figure 2.16: Tapered Single Column

2.2.4.6 Foundations

Foundations are usually either drilled shafts or prestressed concrete piles. The individual columns of a multiple column bent typically rest on a single drilled shaft or a footing that caps piling. Single columns are typically supported by a footing that caps a drilled shaft group or piling. Piles serve as both foundation and columns in trestle pile bents.

2.2.4.7 Substructure Aesthetics

For the vast majority of bridge substructures in Texas, aesthetic design is limited to the features incorporated in the standard elements and details chosen by the designer. These standard features include smooth round columns and chamfered rectangular cap soffits. Until recently, TxDOT has taken a minimalist approach to substructure design, promoting the use of simple shapes with clean lines as the appropriate solution to the problem of combining aesthetics and economics. This philosophy is espoused in the brief section devoted to aesthetics in the *Bridge Design Guide.*

The inverted-T cap serves as TxDOT's solution to the problem of the "forest of columns." The increased depth of this section allows for greater column spacing and, consequently, fewer columns. Placing part of the cap within the depth of the superstructure also increases the transparency of the space beneath the bridge.

Recently, TxDOT bridge designers have experimented with the use of surface texture and color. Form liners have been used to add variety and an element of human scale to bridge piers like the one shown in Figure 2.17. By provoking human interest while adhering to the philosophy of "clean lines," these attempts have been generally successful so long as form liner details have been carefully specified.

The *Bridge Design Guide* states that the artificial coloring of concrete is to be eschewed. Ironically, the painting of newly completed concrete substructures remains typical practice, effectively introducing another bridge maintenance requirement. Concrete painted with vibrant colors has been used successfully in the El Paso District. The colors reflect the Latino artistic influence in the region and greatly reduce the amount of graffiti. Careful consideration of the bridge's harmonious integration with its surroundings is paramount when choosing such color schemes.

Figure 2.17: Texture Applied with Form Liners

2.3 Use of Industrialized Processes in Substructure Construction

In the past forty years, highway bridge construction has increasingly relied upon the use of industrialized processes. Bridges are designed as combinations of parts, with emphasis on the repetitiveness of the production procedures for these parts. Savings of material quantities are sacrificed in order to receive greater returns from the minimization of labor costs. Design time and costs are reduced due to the repeated use of members and details, as are costs involved with training workers and mobilizing equipment. Prefabrication of elements also results in decreased on-site construction time.

Presently, superstructure construction in Texas benefits greatly from the use of industrialized processes. The use of a few standard, plant-produced, pretensioned girder shapes has resulted in remarkably low construction costs for Texas bridges. Precast concrete deck panels offer the same benefits of repetitive production in addition to replacing much of the labor and material costs involved with the formwork they supplant. Standardized details such as traffic rail cross-sections further increase construction savings realized through repetitive construction.

Texas bridge construction also benefits from the use of standardized processes in substructure construction, though not to the degree found in superstructure design. As mentioned previously, the vast majority of columns used in multiple column bents are smooth, circular, and selected from a set of several standard diameters. The use of these cross-sections allows for decreased design effort while allowing the contractor to use forms that are readily accessible, simple to employ, and widely reused. Similarly, rectangular bent cap cross-sections are generally selected from a few typical sizes. The soffit chamfers on these caps also feature standard dimensions, further allowing the repeated use of forms and form inserts.

Unlike superstructure construction, substructure construction has yet to fully utilize industrialized processes. Virtually all construction is performed in the field, and the potential benefits of repetitive plant production are not realized. The advantages of the prestressed I-girder superstructure system that are listed in Section 2.2.3.3 have not yet been translated to a corresponding substructure system. The following sections outline a few of the attempts to introduce the industrialized processes of precasting and segmental construction to bridge substructures in Texas and elsewhere.

2.3.1 Lake Pontchartrain, Louisiana

One of the earliest applications of precasting to bridge substructure construction in the United States was for the Lake Pontchartrain Bridge in Louisiana (see Figure 2.18). This bridge, completed in 1955, consisted of over 2200 identical spans supported at each end by a precast substructure unit. As shown in Figure 2.19, each unit consisted of two hollow prestressed spun piles connected with a rectangular precast cap.[1]

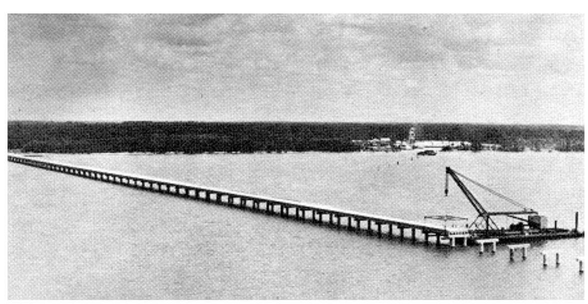

Figure 2.18: Lake Pontchartrain Bridge (from Ref. 5)

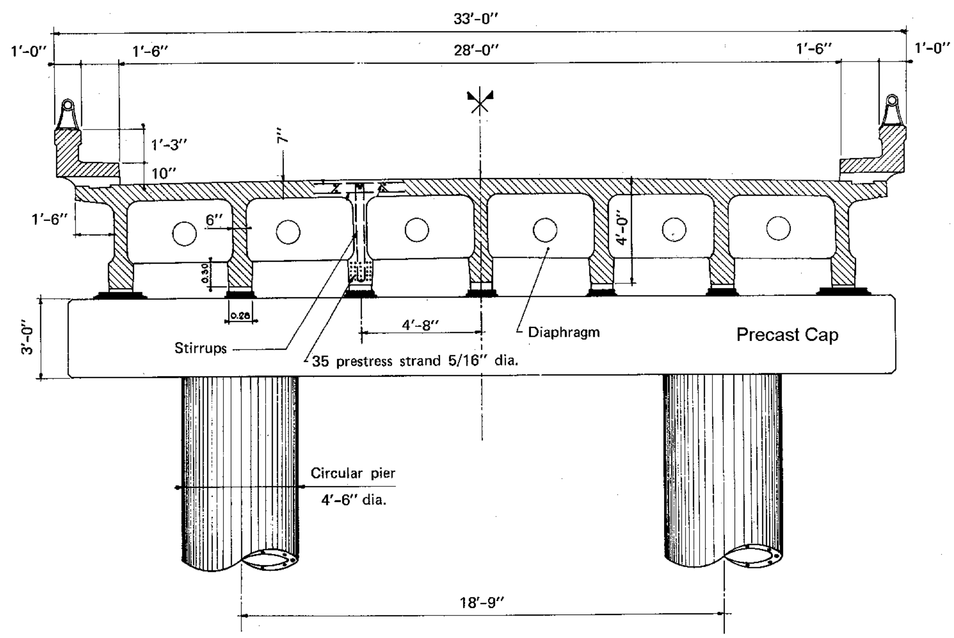

Figure 2.19: Cross-Section of Lake Pontchartrain Bridge (from Ref. 5)

2.3.2 Vail Pass, Colorado

In 1977 the construction of the Vail Pass Bridge proceeded with the use of precast, segmental piers.[2] This project, located on I-70 west of Denver, Colorado, featured pier shafts consisting of diamond-shaped, hollow segments stacked vertically and post-tensioned to the foundation with strand tendons. The foundations were cast in place with ducts for the post-tensioning strands. Each tendon was anchored at the top of the pier shaft and passed down through one corner of the shaft to the foundation as shown in Figure 2.20. The tendon was then curved back up and through the opposite corner of the shaft to be anchored once again at the top of the shaft.

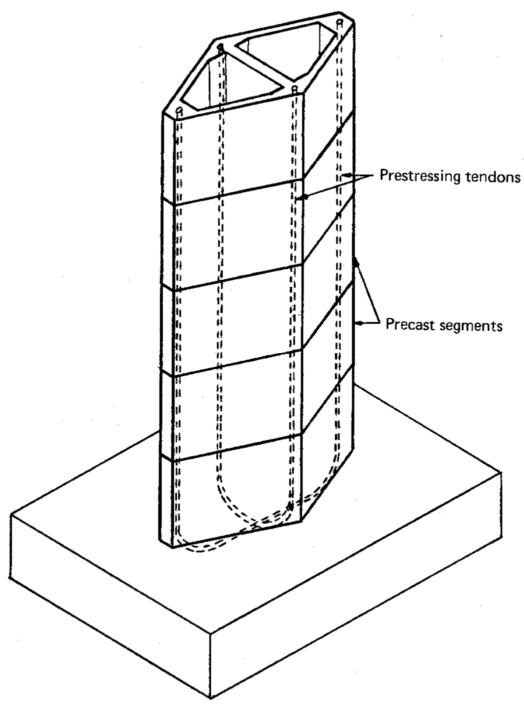

Figure 2.20: Schematic of Vail Pass Pier (from Ref. 6)

2.3.3 Linn Cove, North Carolina

Precast, segmental pier shafts were also utilized for the Linn Cove Viaduct in North Carolina, constructed between 1978 and 1983.[3] Because of the structure's environmentally sensitive location on the Blue Ridge Parkway and its proximity to Grandfather Mountain (see Figure 2.21), several spans of the bridge were erected by the progressive placement method, and

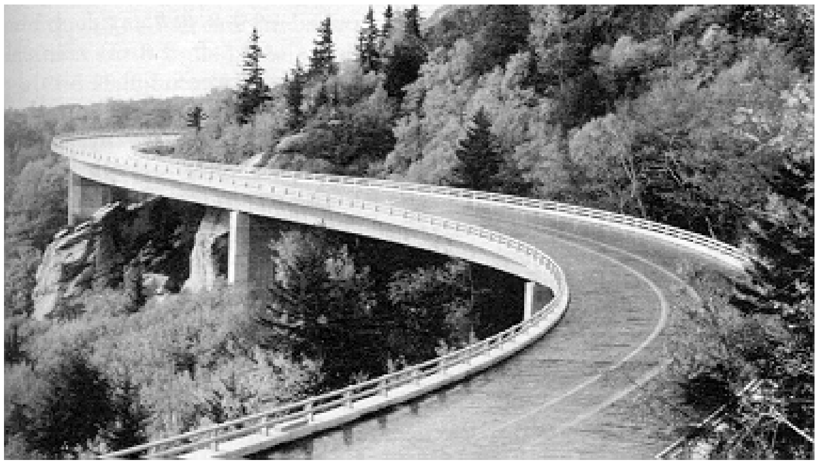

Figure 2.21: Linn Cove Viaduct (from Ref. 7)

the corresponding piers were constructed by lowering all materials from the tips of the cantilevered spans. The pier shafts consisted of precast segments stacked vertically and post-tensioned to the foundation. Each segment was either 1.8 m or 2.7 m (6 ft or 9 ft) tall and weighed up to 270 kN (60 kips). In order to increase visual harmony between the viaduct and the mountainside, the concrete was tinted with iron oxide pigment to match the existing rock color.[4] Shaft segments were match-cast vertically; each new segment was cast above the previously cast segment. As-cast geometry data were recorded and later used in the erection of the segments. No other geometry control was required for casting operations.

Field erection of segments began after forming the footings and placing the reinforcement and post-tensioning ducts. A steel frame was placed in the footing form to support the first segment. The segment was then placed on shims and flat jacks in order to allow proper alignment considering casting variations and superstructure position. After the proper alignment was achieved and verified, the footing concrete was cast to a level approximately 25 mm (1 in) above the bottom of the segment. After the concrete hardened, the joint between precast and cast-in-place concrete was waterproofed by pressure grouting with epoxy.

As depicted in Figure 2.22, each subsequent pier segment was lowered and temporarily blocked 150 mm (6 in) above the previous segment. Epoxy was then applied across the joint, and the segment was lowered into its final position. Thread bar tendons were installed and stressed in order to meet construction load demands on the pier. After placement of the final, or cap, segment for each shaft, eight 12-strand tendons were stressed and grouted. These strand tendons extended from the cap down through and out of the side of the footing.

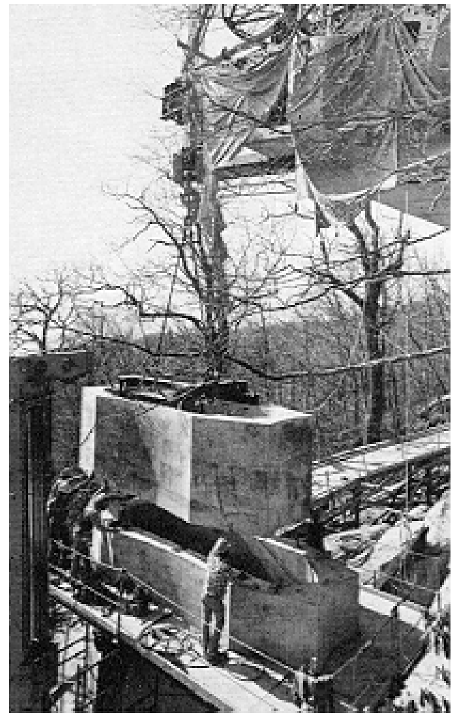

Figure 2.22: Placement of Pier Segment (from Ref. 7)

2.3.4 Florida Keys

The benefits of industrialized substructure construction were utilized in a series of bridges constructed in the late 1970’s to early 1980’s. The Long Key Viaduct, depicted in Figure 2.23 and completed in 1980, featured a V-pier substructure system that was rapidly and easily erected in advance

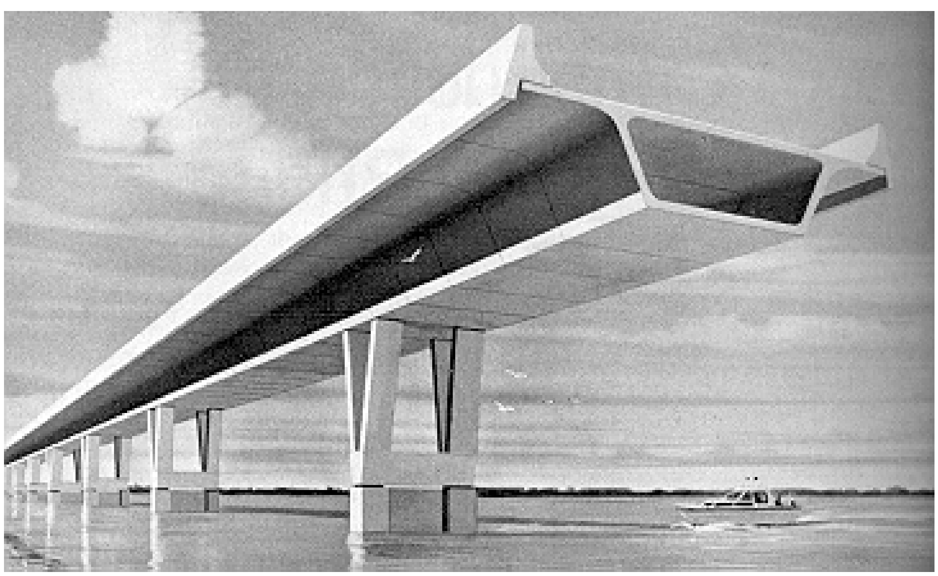

Figure 2.23: Artist's Rendering of Long Key Viaduct (from Ref. 10)

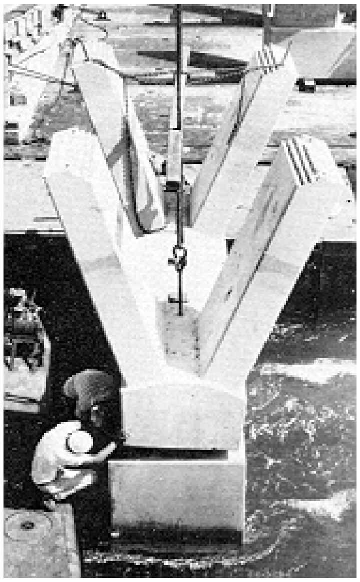

Figure 2.24: Placing Precast V-Pier on Pile Cap (from Ref. 10)

of the segmental superstructure. Each precast V-pier was supported by two 1.06 m (42 in) diameter precast piles placed in a drilled shaft. A precast strut joined the two piles by means of a cast-in-place cap at the top of each. A neoprene bearing pad was placed on this strut, and the precast V-pier assemblage was then placed on the bearing pad as shown in Figure 2.24. Temporary bolts were used to hold the V-piers to the pile caps during construction and were then removed after superstructure installation. The bid price for this substructure system was \$56 per m^2 (\$5.20 per ft^2) of deck surface in 1978. The bridge was completed 8 months ahead of schedule.[5]

The Seven Mile Bridge and Channel No. 5 Bridge, also located in the Florida Keys, successfully utilized vertically post-tensioned, precast box segmental piers.[6] The Seven Mile Bridge was constructed in record time and was completed 6 months ahead of schedule. The bridge was erected at a rate of 610 m (2000 ft) of superstructure per week.[7]

2.3.5 Dauphin Island, Alabama

The Dauphin Island Bridge, completed in 1983, featured vertically post-tensioned, precast segmental box piers (see Figures 2.25 and 2.26). The 4.88 x 2.44 x 2.74 m (16 x 8 x 9 ft) tall segments were match cast vertically and erected with epoxy joints. Four segments were produced per week in each casting machine. As described for the Linn Cove Viaduct, the initial segment of each pier was carefully positioned before the footing was cast in place around it. Post-tensioning anchorages were buried in the footing concrete.[8]

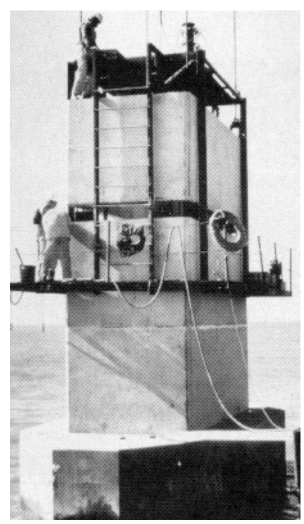

Figure 2.25: Erection of Segmental Piers for Dauphin Island Bridge (from Ref. 12)

Figure 2.26: Completed Precast Segment Piers for Dauphin Island Bridge (from Ref. 12)

2.3.6 Chesapeake & Delaware Canal Bridge, Delaware

The C & D Canal Bridge, winner of the Precast/Prestressed Concrete Institute's (PCI) Harry H. Edwards Industry Achievement Award for 1995, featured 48 precast segmental box piers ranging in height from 15 to 40 m (49 to 130 ft). The 463 box pier segments were 2.4 x 5.5 m (8 x 18 ft) in plan with a 305 mm (1 ft) wall thickness (see Figures 2.27 and 2.28). The hollow shape of the piers reduced the required number of foundation piles. Segments were match cast in 1.2, 1.5, 1.8, and 3.0 m (4, 5, 6 and 10 ft) lengths. As part of an efficient assembly line process, reinforcement cages were pre-tied and then set into place around the core form. Post-tensioning ducts were then installed, and the outer form was placed. Concrete was cast and then steam cured overnight. One segment was cast per day in each of the two casting cells used.[9]

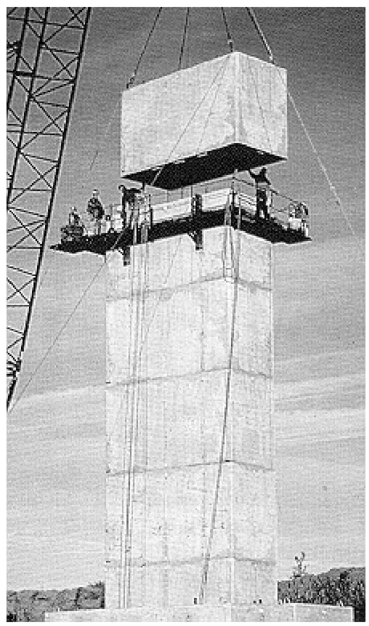

Figure 2.27: Erection of Box Piers for C&D Canal Bridge (from Ref. 13)

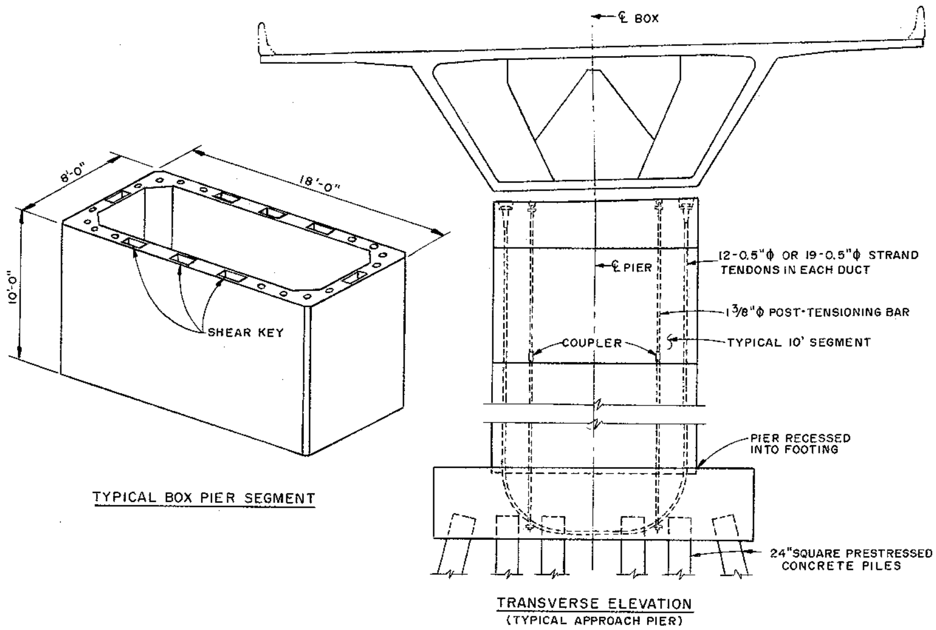

Figure 2.28: Precast Segmental Box Piers for C&D Canal Bridge (from Ref. 13)

Erection proceeded in a similar fashion as that described for the Linn Cove Viaduct above with one major exception. Each pier footing was cast prior to the placement of the initial, or starter, pier segment. The starter segment was placed in a 152 mm (6 in) recess cast in the top of the footing, and then properly aligned. Post-tensioning ducts in the segment were spliced to those in the footing, and bar tendons were coupled. A secondary concrete placement then filled the recess and locked the starter segment into its proper position. Each subsequent segment was epoxied to the previously placed segment and post-tensioned with 36 mm ($1^3/_8$ in) bar tendons. After cap placement, the strand tendons were stressed, and all ducts were grouted. As in the Vail Pass structures, each strand tendon followed a U-shaped path from one corner of the cap segment, down through the footing and back up the opposite corner of the pier to an anchorage at the opposite corner of the cap. Crews erected 30.5 m (100 ft), or ten segments, of box pier in a single day.[10]

2.3.7 Redfish Bay and Morris & Cummings Cut, Texas

The Redfish Bay Bridge and the Morris & Cummings Cut Bridge are recently completed replacement structures on State Highway 361 between Aransas Pass and Port Aransas, Texas. These bridges consist of a pretensioned double-T superstructure system supported by trestle pile bents. The original substructure design specified the use of cast-in-place bent caps on precast driven piles. However, the contractor suggested the use of precast caps in order to reduce the costs associated with concrete operations over water. This suggestion was accepted with a few modifications, and the resulting structures were completed ahead of schedule.

Figure 2.29: Connecting Precast Cap to the Piles for the Redfish Bay Bridge

The caps were precast with two slots at the location of each pile connection. After the piles were driven into final position, two #9 U-shaped dowel bars were epoxy grouted into the top of each pile. The precast cap was then lowered onto the piles so that each U-shaped bar was positioned in one of the cap slots. Cap elevations were then adjusted and verified. As shown in Figure 2.29, the void slots were filled with concrete to complete the cap-pile connection.[11]

2.3.8 Louetta Road Overpass, Houston, Texas

Depicted in Figure 2.30, this pair of three-span bridges on State Highway 249 is presently under construction as a part of joint TxDOT/FHWA study of high strength concrete applications for highway bridges. The superstructure of these structures is unique

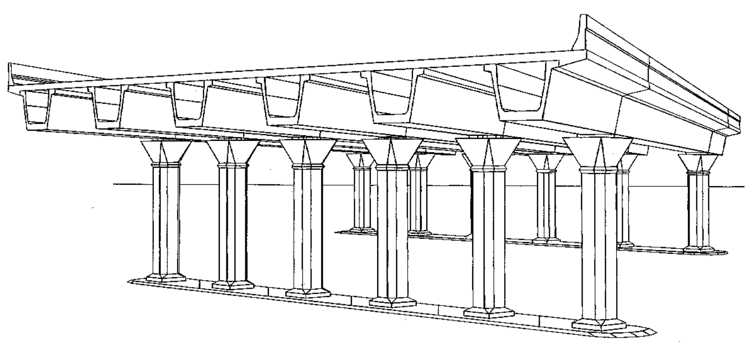

Figure 2.30: Perspective of Louetta Road Overpass (from Ref. 16)

because of the use of TxDOT's new pretensioned concrete U-beams. Each U-beam is simply supported at each end by a post-tensioned, precast segmental column. The hollow column segments are designed for 69 MPa (10 ksi) specified 28-day concrete strength. Each column is founded on a single 1.22 m- (4 ft-) diameter drilled shaft. The six 36 mm ($1^3/_8$ in) post-tensioning bar tendons are anchored within the drilled shaft as depicted in Figure 2.31. The column segments are 0.99 m (3.25 ft) square with corner chamfers of 229 mm (9 in). The columns are designed for allowable stresses of $0.45f'_c$ in compression and $0.25\sqrt{f'_c}$ MPa ($3\sqrt{f'_c}$ psi) in tension. The designers cited the system's structural efficiency, rapid erection time and aesthetic flexibility as the primary reasons for its selection.[12]

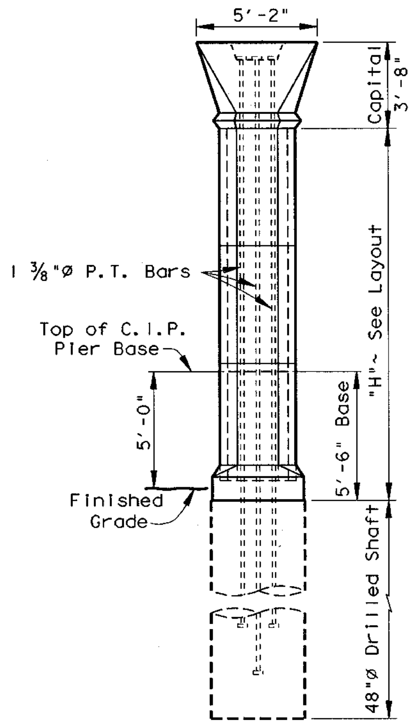

Figure 2.31: Segmental Piers for Louetta Road Overpass (from Ref. 16)

2.3.9 *U.S. 183 Viaduct, Austin, Texas*

Another Texas highway project utilizing hollow, post-tensioned, precast segmental substructure elements, the U.S. 183 Viaduct project is presently under construction in Austin. As a part of this project, one of the ramps for the new U.S. 183/ I-35 interchange is to be supported by hollow, post-tensioned, precast segmental piers (see Figures 2.32 and 2.33). The octagonal pier segments were match cast in the same casting yard as the superstructure segments. Each segment is 2.29 x 2.29 m (7.5 x 7.5 ft) in plan and 2.44 m (8 ft) tall. A perimeter groove at midheight makes each segment appear to be two 1.22 m (4 ft) segments. Segment walls are typically 406 mm (16 in) thick.

Figure 2.32: Large Ramp Piers for U.S. 183 Viaduct

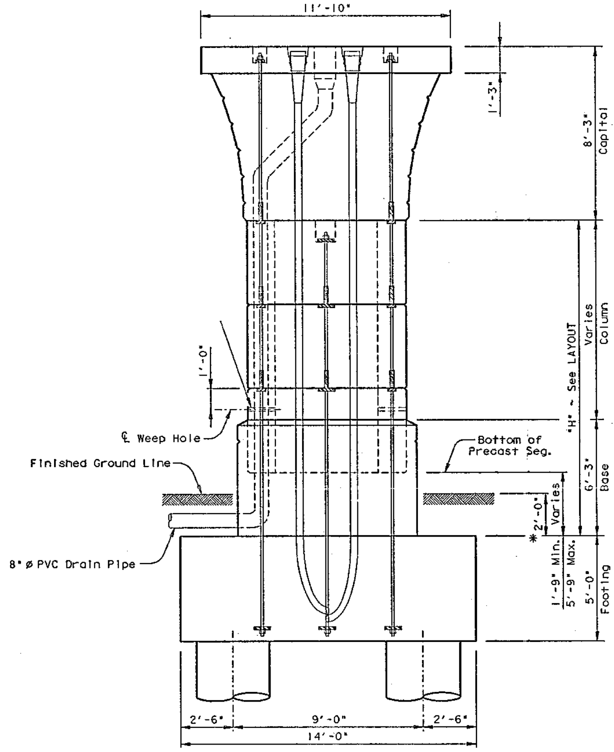

Figure 2.33: Schematic of Large Ramp Pier

Figure 2.34: Lowering Reinforcement Cage into Outer Form

Fabrication of each segment began with the placement of the outer form immediately above the previously cast segment whose top surface had been coated treated with a bond-breaking substance. As shown in Figure 2.34, the prefabricated reinforcement cage was then lowered into the outer form, and the post-tensioning ducts were installed along with a 200 mm- (8 in-) diameter PVC drain pipe. The octagonal inner form was then lowered into position. Proper alignment of the previously cast segment was achieved with the use of hydraulic jacks placed at four positions beneath this segment. Screwjacks were positioned at the four corners of the outer form's support frame to ensure proper alignment of the form. Once the proper alignment was achieved and verified (see Figure 2.35), the segment concrete was cast (see Figure 2.36) and shear keys were inset in the finished top surface. After curing overnight, the new segment was moved into position so that the next segment could be cast against it, while the previously cast segment was removed from the casting cell and placed in storage. The process was then repeated. One segment was fabricated per day. Segment fabrication involved the full-time labor of one foreman and one laborer. The foreman was quick to express his opinion that this method was the easiest form of

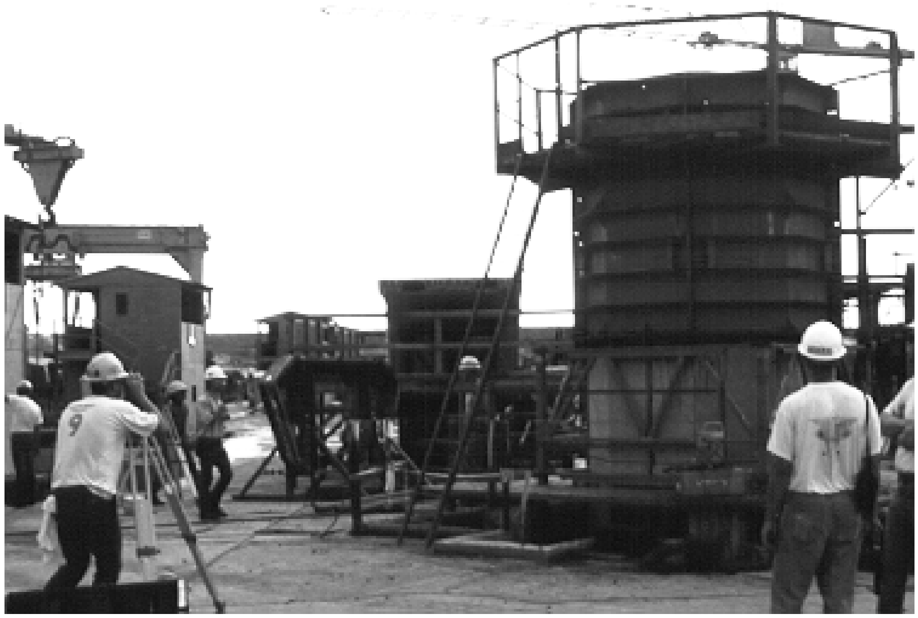

Figure 2.35: Checking Alignment of Segment and Form

substructure construction in which he had ever been a part.[13] Reinforcement cages were fabricated with the part-time labor of two ironworkers. Other plant personnel whose part-time labor was dedicated to pier segment fabrication included a crane operator, geometry control surveyors, and an extra laborer for the concrete casting. These “part-time” workers spent the rest of their day working in other production lines in the precast plant.

Like the piers of the C&D Canal Bridge and Linn Cove Viaduct, both thread bar and strand post-tensioning tendons were used as reinforcement in these piers. On-site construction began with the installation of drilled shaft foundations, the placement of all mild reinforcement, thread bars, thread bar anchorages, and strand ducts in the footing, and the casting of footing concrete.

Figure 2.36: Casting Segment Concrete

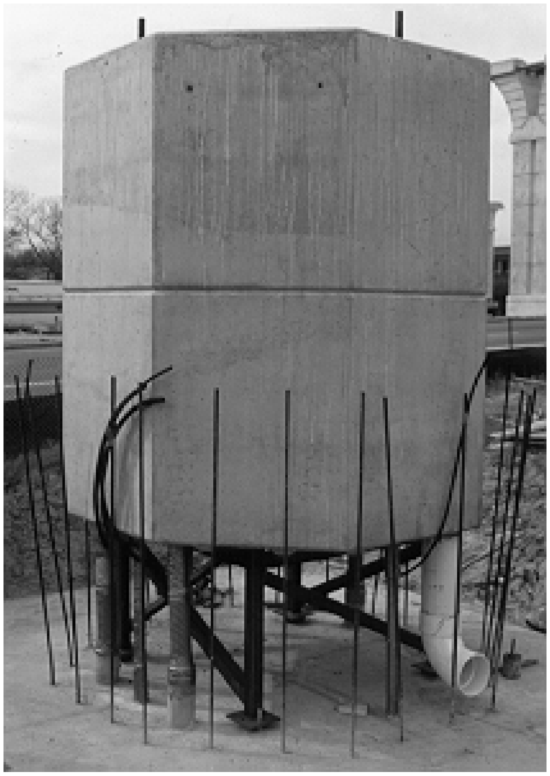

Figure 2.37: Starter Segment Supported above Footing

Figure 2.38: Starter Segment with Cast-in-Place Pedestal

Pier erection proceeded with the installation and temporary support of the starter segment above the footing at a height determined by the desired overall height of the pier. Next, the segment post-tensioning ducts were spliced to the footing ducts and mild steel reinforcement was placed as shown in Figure 2.37. The starter segment was then carefully aligned, and a pier base, or pedestal, was cast in place around the segment, effectively locking it to the footing (see Figure 2.38). After this concrete reached a predetermined strength, the four post-tensioning bars were stressed. The subsequent segment was suspended above the starter segment while the bars were coupled and the joint faces were coated with epoxy as shown in Figure 2.39. The segment was then lowered into position and the four coupled bars were stressed (see Figure 2.40). This process was then repeated until all segments had been installed. In a similar fashion, the pier capital segment was installed and post-tensioned. Two 19-strand tendons were then pulled through the U-shaped ducts that pass down the pier, through the footing, and back to the top of the capital. These two tendons were stressed from both ends, necessitating four anchorages at the top of the capital.

Figure 2.39: Applying Epoxy to Joint Faces

Finally, all post-tensioning ducts and blockouts were grouted. A completed pier is shown in Figure 2.41.

Although precast, segmental piers were used for one major ramp of this large highway project, the vast majority of the piers were cast in place. A design option that specified all piers be constructed in a precast, segmental fashion was narrowly defeated by a bid

Figure 2.40: Stressing Post-tensioning Bars

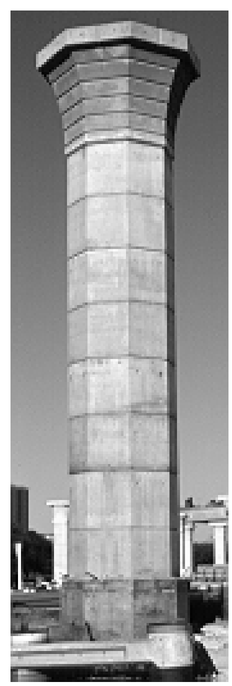

Figure 2.41: Completed Pier for U.S. 183 Viaduct

that included cast-in-place piers for most of the project. Discussions with a contractor representative revealed several of the factors that shaped the decision to bid on the cast-in-place substructure option:

1. Because the superstructure of the U.S. 183 Viaduct was to be of precast, segmental construction, the available casting yard space was too small to handle the fabrication and storage of all superstructure *and* substructure elements;
2. The project site offered exceptionally easy access to cast-in-place operations;
3. Because cast-in-place operations were to be mobilized and already on-site for the foundation construction, it seemed natural to go ahead and use them for the pier construction as well;
4. Use of the precast design for the mainlane “Y-piers” would have required a more expensive grade of steel tubing than the cast-in-place design; and
5. The precast design option offered no significant reduction in mild steel reinforcement quantity when compared to the cast-in-place design option.

In hindsight, in-place fabrication of the relatively complex reinforcement cages for the Y-pier capitals turned out to be more difficult, more time-consuming, and more costly than expected. Several of the foremen and laborers involved with constructing both the cast-in-place mainlane piers and the precast, segmental large ramp piers were eager to express their collective opinion that all of the piers should have been precast.[14]

2.3.10 Northumberland Strait Crossing, Prince Edward Island, Canada

Each pier in this structure, presently under construction, supports a 190 m (623 ft) double cantilever, precast segmental, variable depth box girder (see Figure 2.42). A single 52 or 60 m (171 or 197 ft) drop-in girder is then placed between the cantilever main girders in order to complete each span. Each pier consists of two large precast pieces: a pier base and a pier shaft. The pier base serves as a gravity foundation for the structure. The pier shaft includes a conical ice shield with a central void which matches the top of the pier base. The main girders, drop-in girders, pier bases and pier shafts are all prefabricated in the casting yard and subsequently transported to their final position using the heavy-lift catamaran-barge *Svanen*.

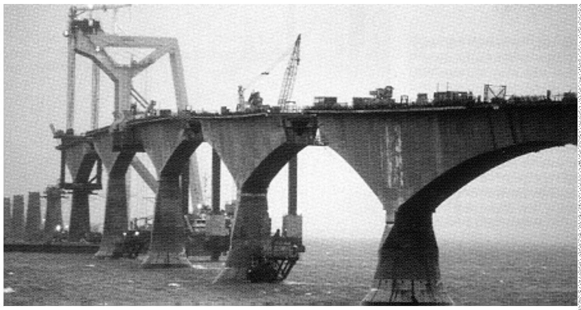

Figure 2.42: Northumberland Strait Crossing (from Ref. 22)

Figure 2.44: Template in Place atop Pier — *Svanen* with Girder in Background (from Ref. 21)

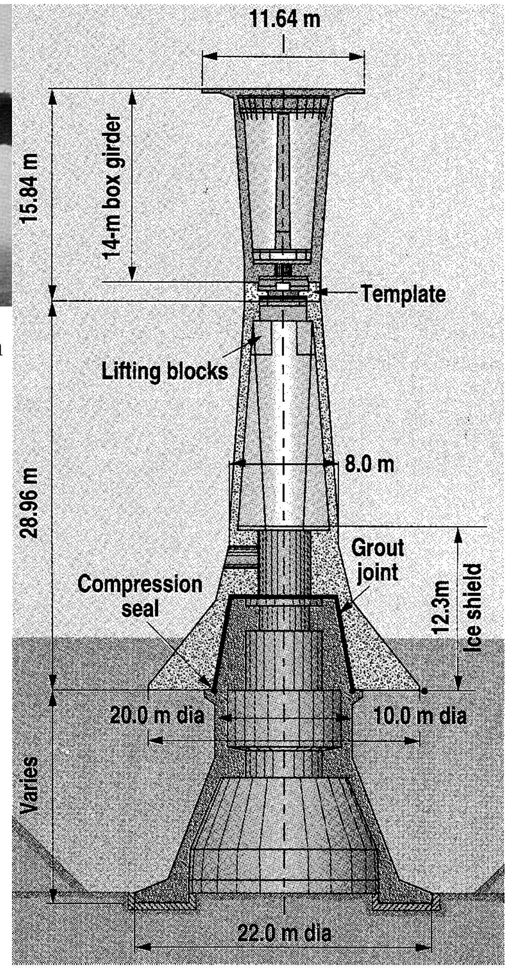

Figure 2.43: Pier Schematic for Northumberland Strait Crossing (from Ref. 21)

Because of the extreme length of the main cantilever girders, precise connection alignment at the top of the pier shaft is critical. Any misalignment at this location would be greatly magnified at the ends of the cantilever girder. For this reason, a precast concrete "template" is utilized in the pier shaft to girder connection (see Figure 2.43). The template corresponds in size to the top of the pier shaft and is match cast to the pier segment of the main girder. After the pier shaft has been installed, the template is placed on top of the pier. The template is carefully adjusted to the proper alignment and attitude with jacks and then grouted into final position. The main girder is then transported and placed in its final position by *Svanen* (see Figure 2.44). Thus, all geometric adjustments are made to the small, readily maneuverable template rather than the large and unwieldy main girder.[15,16,17,18]

2.4 Summary

After reviewing these projects that have benefited from the use of industrialized processes for substructure construction, it is apparent that the technology exists with which to develop a precast substructure system for moderate-span highway bridges. Not only does the technology exist, its applicability has been proven with the successful and efficient construction of a variety of bridge types. Many of these bridges have won awards for both their visual attractiveness and the efficiency associated with their construction.

In general, these bridges were large projects with a high degree of repetition involved in their construction. The use of precast processes was successful primarily for this reason. Efficient application of precast technology requires a relatively high degree of repetition of shapes and details. Most individual moderate-span highway bridges lack the sheer number of repetitive substructure elements required to reap the benefits of precast technology. The key to realizing these benefits is the adoption of a standardized substructure system that can be applied to a number of bridge structures in much the same manner as standard prestressed I-beams are applied at present. For example, the winning bidder on the U.S. 183 Viaduct chose not to precast most of the piers on the project. As can be seen in Figure 2.45, these piers featured a unique shape not likely to be used on subsequent projects. However, if the piers had been part of a standardized substructure system, the contractor could have been relatively certain that the forms and processes would have been reused, making the precast option a more attractive alternative.

Figure 2.45: Mainlane "Y-Pier" for U.S. 183 Viaduct

Because the use of a standardized system will produce cost benefits only if used repeatedly, it is unlikely that potential cost savings will be apparent in bid prices for the initial applications of the system. The traditional cast-in-place substructure design option will almost always have a lower bid price than any precast option until a precast system has developed a “foothold” after being used in a few applications. Thus, although a cast-in-place bid option should usually be provided, it might be desirable to mandate the use of a precast system on several initial demonstration projects. In this manner, the system learning curve can be established and form costs can be amortized through repeated use. These demonstration projects will also present an opportunity to evaluate the aesthetics, performance, and durability of the substructure system. Then, after the introductory phase is complete and the benefits of repetition have been realized, the precast substructure system can be compared on a direct cost basis with established cast-in-place methods.

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CHAPTER THREE
SUBSTRUCTURE AESTHETICS

3.1 Introduction

There is no shortage of literature that deals with the subject of bridge aesthetics. Hundreds of essays, articles and books have been written on the subject. A large proportion of the illustrations in these works are of longer span bridges or non-girder bridges. The American Concrete Institute (ACI)[1] and the Transportation Research Board (TRB)[2] have both produced excellent compilations of articles concerning various aspects of bridge aesthetics. The TRB compilation also includes an extensive annotated bibliography on the subject. At least two states have published guidelines for the aesthetic design of highway bridges.[3,4] In conjunction with the research presented in this thesis, an extensive set of guidelines for the design of moderate-span highway bridges is under development for the Texas Department of Transportation.[5] Although numerous authors have written works pertaining to aesthetic bridge design, Leonhardt[6] and Menn[7] are among the few to devote serious effort to discussing the aesthetic design of substructures. The principal ideas espoused by these two authors, particularly Leonhardt, form a common thread that is identifiable throughout most of the literature dealing with substructure aesthetics. The following sections constitute a discussion of the aesthetics of moderate span concrete bridge substructures based primarily on these principal ideas and the author's observations of Texas bridge designs, past and present.

3.2 Aesthetic Role of Substructures

Because the structural elements of girder-type bridges are generally below the roadway elevation, motorists and pedestrians are relatively insensitive to the aesthetics of a single bridge when traversing it. They are, of course, very aware of the supports and spans which they pass under or around. As a result, a large portion of the aesthetic appraisal of a girder bridge is performed by motorists when passing beneath or alongside the bridge, along with those who reside or work within sight of it. In the course of daily events, humans usually encounter the substructure elements at a closer range than the superstructure elements. Therefore, the appearance of substructure elements alone and in the context of the entire bridge is crucial to the aesthetic valuation of the structure.

3.3 Aesthetics and the Design Process

Bridge engineers often confuse aesthetics with ornamentation. This leads to the common misperception that aesthetic issues can be addressed and resolved after the overall layout and structural form of the bridge have been determined. This errant way of thinking generally leads to a second and more damaging misperception — that aesthetically appealing bridges involve "add-on" costs and so necessarily cost more than bridges designed without regard for aesthetics. Since this project intertwines improved aesthetics and improved efficiencies through the use of high quality, more durable materials, designers should consider use of more attractive *and* efficient substructures even in those fabled Texas locations where only jackrabbits or possums regularly peer under the bridge roadway surface. The importance of aesthetics in highway bridge design is stressed by Henry Petroski, who writes

> *Though most of America's more than half a million highway bridges are small and anonymous, they may not be any less important to the local traffic than the Golden Gate and Brooklyn bridges are to their hordes. The engineers of our greatest spans began by designing our smaller ones. The scale may be different, but the process is essentially the same, and so these bridges have proved to be the training grounds for dreams. Furthermore, every bridge, small or large, is also an aesthetic and environmental statement. Its lines are important beyond its span; every bridge must not only bear its burden, whether cows or coal trains, but must also be able to withstand the burden of proof that, in the final analysis, society is better served, tangibly and intangibly, by the bridge's being there at all.*
>
> *Imagine how a bridge can ruin a setting of natural beauty, whether the tranquillity of the countryside or the skyline of the city. Imagine what the wrong bridge across the Golden Gate might have done to that unique site. This is why place so often influences bridge design--for, contrary to the popular misconception, engineers are not insensitive to setting and aesthetics.*[8]

Every design decision affects the aesthetics of a bridge. Roadway geometry, materials, structural system, bent locations, member sizes, surface treatments and a host of other choices all dictate the structure's visual appearance. Thus, opportunities to increase or decrease the visual attractiveness of a bridge exist at every stage of the design process. Effective and efficient aesthetic design includes the recognition of these opportunities, the careful evaluation of available options, and the selection of the most appropriate of these options based on the resulting lifetime value to the bridge's owner and consumer — the taxpayer.

Successful and efficient aesthetic design cannot be achieved through adherence to a set of "rules." Utilizing a specific slenderness ratio or member thickness does not guarantee an attractive structure. Along the same lines, use of a certain element, either structural or non-structural, that has been labeled "aesthetic" and used with success in previous designs does not necessarily enhance the aesthetic value of a new bridge. Rather, the designer must develop an aesthetic vision as early as possible in the design process and continue to apply that vision throughout. This is not to say that a vision should be unalterable. Aesthetics is inextricably linked to structural behavior. As more is learned about the particular constraints involved, the designer's vision will likely be modified. Nonetheless, aesthetic vision should be present at every design stage, and this vision should become more well defined as the design evolves.

3.4 Harmony

Harmony is the key to the aesthetic success of a bridge design. In order to have lasting aesthetic value, the bridge must be integrated harmoniously into its surroundings. These surroundings include the geographic setting, human users/observers, and the forces of nature with which the bridge must contend for its lifespan. The entire bridge environment should be considered by the designer. The individual elements of the structure, as well as the entire bridge, should harmonize with their environment. Being of a smaller scale than the entire bridge system, individual structural and non-structural elements that are regularly in close proximity to humans should incorporate details of a human scale. This is an important facet of substructure design, especially in urban settings.

The bridge must also be in harmony with itself. Superstructure and substructure must unite to form a coherent whole. The individual elements of these systems should work together structurally and visually. Of course, these elements should also make aesthetic sense when considered alone.

The visual characteristics of a bridge structure can be broken down into two broad categories: structural form and surface features. Both are crucial to the bridge's visual harmony with its environment. Proportion, scale, order, texture and color are qualities that must be considered in order to design an aesthetically pleasing structure. Although a complete discussion of these techniques is beyond the scope of this thesis, the successful application of these ideas to the structural form and surface features of substructures is discussed in the following sections.

3.5 Structural Form

The most visually appealing overall form of any structure is one which clearly expresses efficient structural function. The most evident manifestation of a bridge's structural efficiency is the slenderness of the superstructure. However, for substructure design, structural efficiency is represented primarily by the transparency of the space beneath the superstructure and the orderliness of the elements that subdivide this space. This spatial transparency is a function of both the size and spacing of substructure elements. As with any aesthetic concern, consideration of all possible viewing angles is important. Although a series of multiple column bents each containing several small columns, such as those used throughout Texas, may appear quite transparent from a few angles and distances, the orderly expression of structural function may be obscure from other viewpoints. Both Leonhardt and Menn recommend using as few columns per bent as possible, with two as an ideal maximum.

Single shaft bents should be used for cap lengths (including the effect of skew) up to approximately 14 m (47 ft). The ratio of superstructure width to shaft breadth should preferably be between 3.5 and 5. Transparency alone does not guarantee aesthetic success. The orderly flow of the superstructure should also be evident in the progression of interior bents along the length of the bridges. Single shaft bents are especially valuable because the series of single vertical members clearly delineates the flow of the supported traffic. The versatility of the single shaft bent is useful when the designer is faced with the geometric constraints inherent in skewed crossings.

For low bridges with bent caps longer than about 14 m (47 ft), a bent supported by two shafts should be used. This generally corresponds to a roadway consisting of three lanes or more, although two-lane roadways with considerable skew may qualify. If the substructure is tall enough, a single shaft bent is still the ideal solution. However, this configuration should only be utilized if the exposed height of the shaft to the bottom of the cap is at least 2 to 3 times greater than the breadth of the column. Transparency is lost if the single shaft appears to be a wall. Bents supported by three or more shafts or columns should be used only when necessitated by very wide roadways or high degrees of skew.

Bentcaps should also be as transparent as possible, especially for low bridges. The cap should be integrated into the substructure as much as possible. The inverted-T style cap currently used widely in Texas is valuable in this regard because the web (or stem) lies within the depth of the superstructure girders (see Figures 3.1 and 3.2). Unlike

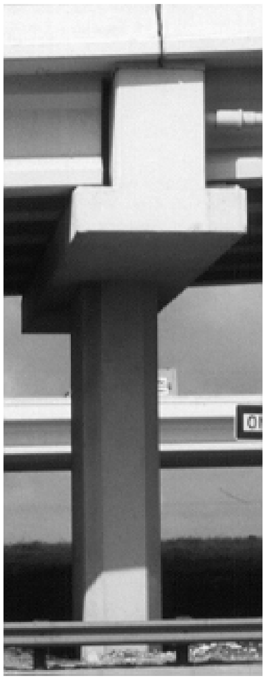

Figure 3.1: Inverted-T Bentcap

Figure 3.2: Substructure Transparency Enhanced by Inverted-T Caps

rectangular caps, only the flange (or ledge) of the bent protrudes into the space beneath the superstructure. Because of the large amount of space they occupy below the girders, hammerhead caps should only be used for tall structures (see Figure 3.3). Unless a bridge is very low, the flow of forces can be expressed by sloping the soffits of the cantilever cap overhangs of inverted-T caps. As shown in Figure 3.4, this gives the same positive visual expression of force flow as seen

Figure 3.3: Hammerhead Bents Reduce Substructure Transparency

Figure 3.4: Inverted-T Caps with Tapered Soffits

in a hammerhead cap, without sacrificing valuable space beneath the span. The depth of the inverted-T stem is generally dictated by the superstructure girder depth. The bent appears top-heavy if the stem is wider than the supporting shaft or column. In extreme situations like that depicted in Figure 3.5, this condition can make the columns appear weak or unstable. Ideally, the stem should be slightly less wide than the shaft or column. The cap stem and ledges should be in good proportion to one another as well as to the girders and shafts.

Repetition is a very important tool for establishing order in substructure design. Although the structural function of the substructure is primarily of a vertical nature, these members should also express the horizontal flow of the bridge as a whole. This is usually best accomplished by using a series of bents that are as alike as possible, composed of the same size elements and differing only in overall height and width as required (see Figure 3.6). Changes in member sizes or configuration from bent to bent should be kept to a minimum, so as not to add visual confusion to the layout.

Figure 3.5: Pier Appears Inadequate beneath Large Cap

In short, the substructure form should clearly express efficient structural function. Clean, simple force flow from superstructure to foundation should be evident in each individual bent. The progression of bents or piers, when taken as a whole, should complement the orderly, horizontal flow of the superstructure.

Figure 3.6: Use of Repetitive Elements Results in Orderly Substructure Flow

3.6 Surface Features

In addition to the bridge's abstract structural form, the designer must also consider how the shapes and surfaces of the substructure elements integrate with the environment and the rest of the bridge. In the cases of grade separations, elevated highways and interchanges, humans are frequently exposed to substructures at close range. The attractiveness of these “up-close” surfaces is vital to the acceptance of the bridge. As mentioned above, human scale should be incorporated into these elements. Large areas of smooth concrete surface should be avoided. The interest of the observer should be sparked by the use of texture.

3.6.1 Texture

Because this study focuses on concrete girder bridges, the large-scale texture of the substructure should correspond to that of the pretensioned girder superstructure. Whether composed of I-girders or box girders, many of the large-scale texture characteristics are similar. The superstructures consist of straight, prismatic members. Girder surfaces are made up of intersecting planes. The primary reason for this is obvious. Plane surfaces are much easier and cheaper to form than curved ones. Although the superstructure as a whole may be curved, it is composed of straight girders between bents.

Accordingly, the substructure elements for these bridges should feature similar large-scale texture. Surfaces should consist of intersecting planes. Shapes should be prismatic or tapered, but curves should be kept to a minimum. Adding chamfers and/or insets to rectangular shafts provides large-scale texture akin to that exhibited in the angular web recesses of box and I-girders. As shown in Figure 3.7, the resulting vertical edges enhance the slenderness and verticality of the shafts just as the horizontal edges add to the slenderness and flow of the superstructure.

A more radical method of introducing large-scale texture is the use of openings within the substructure. As shown in Figure 3.8, these openings, or "windows," can increase the number of intersecting planes composing the pier surface while increasing the transparency of the substructure as a whole.

Medium- and fine-scale textures can be produced with the use of form liners. The resulting texture may enhance the vertical nature of the shaft, in the case of the popular "fractured fin" form liner (see Figures 3.9 and 3.10), or simply express a motif particular to the bridge's setting (see Figure 3.11). Medium-scale form liners may also be used to make the concrete appear as if it is another material altogether, such as masonry or stone(see Figure 3.12). However, the aesthetic value of this type of application is debatable because it inaccurately expresses the structural function of the member, and should therefore be used only with careful consideration of the final effect. Fine-scale surface treatments such as exposed aggregate may also be used in special situations. All of these textural techniques are means of incorporating human scale into a structure and stimulating the interest of the observer. Where form liners are required, the details should be carefully specified. As shown in Figure 3.13,

Figure 3.7: Pier with Chamfered Edges

Figure 3.8: "Windowed" Pier Shafts

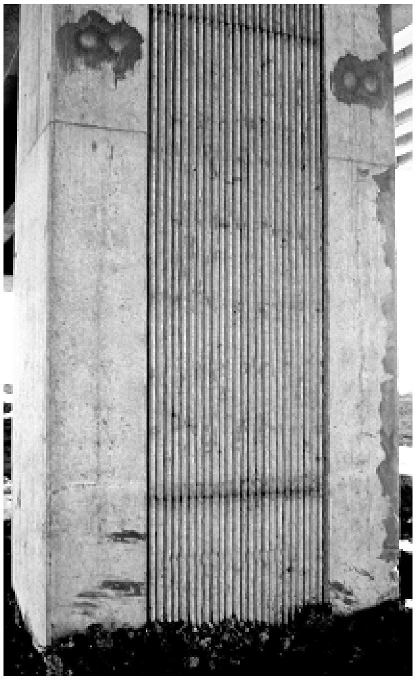

Figure 3.9: "Fractured Fin" Form Liner

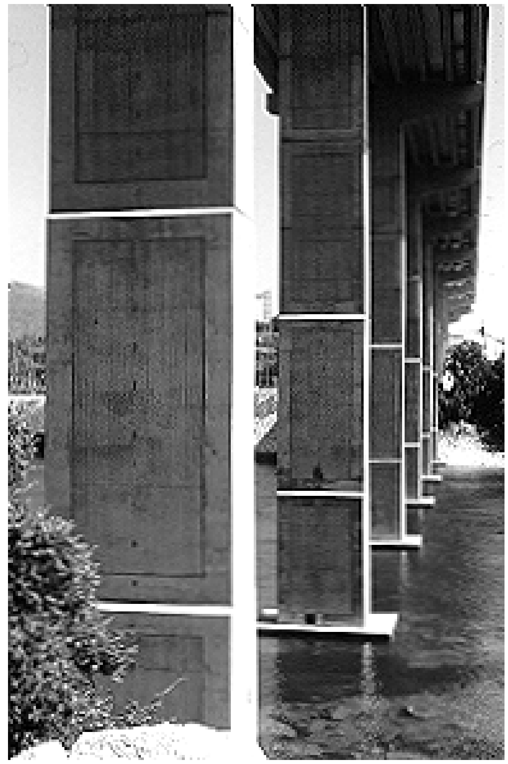

Figure 3.10: Form Liners Enhance Verticality of Pier

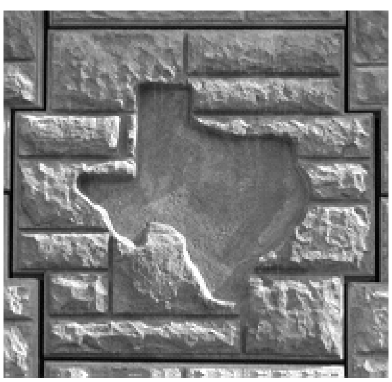

Figure 3.11: Use of Local Motif

the visual appearance can be harmed by random placement of horizontal lines when form liner joints are not controlled.

The precast plant offers an ideal environment for the application of quality surface textures through the efficient utilization of more complex form systems. The use of reusable forms and form liners in the repetitive, highly controlled plant environment results in cost-efficient production of desired textures. Other techniques, such as sandblasting or acid etching, can be more efficiently and effectively applied in the precast plant.

3.6.2 Color

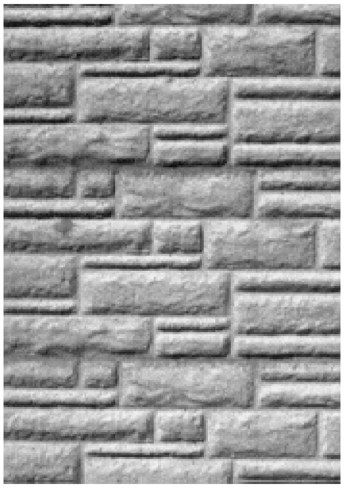

Figure 3.12: Concrete Formed to Appear as Stone Masonry

Figure 3.13: Random Placement of Form Liner Joints

Color is another surface feature that should be addressed by the designer. The concrete purist would avow that concrete is gray by nature, and any attempt to change the color of the concrete in a bridge is aesthetically detrimental. However, as with other aesthetic issues, visual harmony, rather than inherent material properties, should be the determining factor. The use of pigmented concrete to match the Linn Cove Viaduct's (Section 2.3.3) natural surroundings has certainly been hailed as a success. The vibrant concrete painting used recently in El Paso, Texas accurately reflects the culture of the region. Unfortunately, coloring of concrete can cause more trouble than it is worth. Paints and surface coatings introduce a maintenance problem that does not exist with unpainted concrete. If feasible, the color should be incorporated into the concrete mix through the properly selected aggregates and/or pigments. However, the use of these methods must be carefully tested and monitored in order to obtain a the desired color and uniformity. Coloring of concrete also adds to the cost of the constructed cost of the structure. The costs and benefits of coloring concrete must be carefully evaluated for each design situation in order to determine the proper course of action.

Coloring of concrete with aggregates and pigments is much easier in the precast plant environment than in the field. The high level of quality control and standardized processes makes it easier to achieve the desired color and uniformity. Even if no effort is made to color concrete, the finished color of precast concrete is usually more uniform than can be achieved with cast in place construction.

3.6.3 Joints and Articulations

Figure 3.14: False Joints Complement Real Joints in U.S. 183 Viaduct Piers

For the case of precast segmental piers, aesthetic treatment of horizontal joints should be considered. Because joints between column segments will be apparent, they should be accentuated and included in a regular pattern of horizontal grooves that can be repeated rhythmically in a series of shafts. This allows a more harmonious integration of adjacent shafts of varying heights such as those supporting ramp or viaduct structures. Horizontal grooves should be located so that they form a logical progression when the shafts are viewed as a series. For example, the precast ramp piers used to support the U.S. 183 Viaduct consist of segments that are 2.44 m (8 ft) tall. However, a false joint was formed at midheight of each segment (see Figure 3.14). These false joints divide each shaft into smaller 1.22 m- (4 ft-) visual units. Thus, the likelihood that the horizontal joints will appear chaotically staggered as the observer's focus shifts from pier to pier is greatly reduced. Note that the use of similar false joints in the cast-in-place "Y-piers" in the background of Figure 3.14 lends a sense of visual unity to the entire substructure system.

These horizontal articulations divide the shaft surface into individual units that may be further accentuated with the implementation of a texture scheme. Medium- or fine-scale textures may be used to visually link groups of segments together or establish them as independent entities. Whichever scheme is chosen, the result should be a coherent overall pattern.

3.7 Aging

Possibly the most overlooked factor regarding aesthetics is aging. What may have once been a beautiful bridge loses all aesthetic value and becomes another maintenance problem when plagued with drainage stains and peeling paint a few years later. A bridge is subject to constant aesthetic evaluation throughout its lifespan and should be designed accordingly.

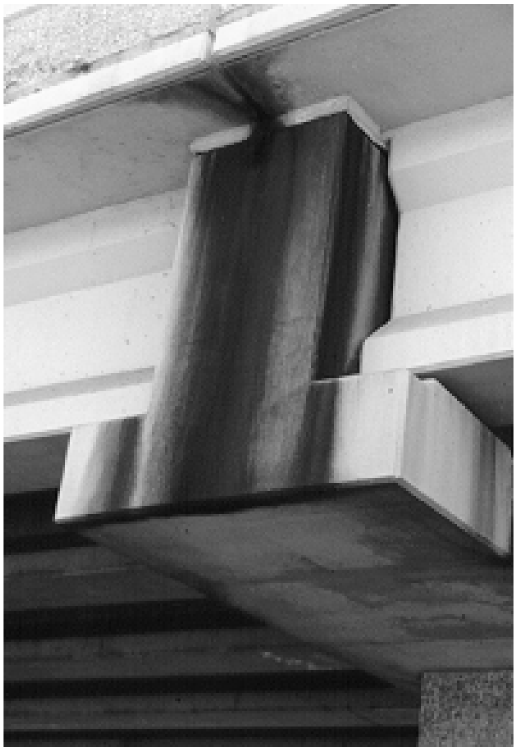

Figure 3.15: Staining Due to Runoff from Superstructure

3.7.1 Durability Effects

The substructure of a bridge is directly subjected to earth-, water- and air-borne attack. In addition, these elements are often indirectly subjected to aggressive elements that are transported down from the superstructure, generally by runoff (see Figure 3.15). The structures should be designed with prevention in mind. The first line of defense is the minimization of the concrete surface exposure to deleterious materials and chemicals, such as deicing salts and decaying expansion joint sealers. The best possible joint sealing techniques should be employed, and adequate drainage should be ensured. Of course, the best joint is no joint. Thus, appropriate continuity should be used to eliminate as many joints as possible.

Nonetheless, the substructure concrete will still be subject to direct and indirect assault. High quality concretes with low permeability and generous cover should be provided in order to protect the concrete itself and the reinforcement within. Low concrete permeability, adequate cover, and surface uniformity are most effectively produced in the tightly controlled and efficient environment of the precast plant. Hollow sections with interior drainage capability can be precast readily. The use of prestressing also greatly reduces aggressive attack by closing cracks that would normally be present in conventional reinforced concrete

Textured concrete surfaces also aid in the mitigation of damage due to aging and seem to discourage graffiti. Exposed aggregate finishes decrease the visual appearance of staining. If thoughtfully placed, grooves produced by form liners can channel water away from the more exposed flat surfaces. The resulting stains are hidden in the shadowed recesses, minimizing visual disturbance.

3.7.2 *Graffiti*

Graffiti is a direct attack on the visual appearance of a bridge by humans, and should be treated as such. The designer should consider the factors which inspire the graffiti "artist." Graffiti and "tags" are most often found on plain, smooth expanses of concrete in urban locations. The graffiti artist is much less likely to choose an interesting or aesthetically pleasing structure (or portion of a structure) as the canvas for his or her visual expression. The ideal defense against graffiti is the arousal of aesthetic interest on the part of the would be perpetrator. Both fine- and medium-scale texture have proven good deterrents against graffiti and "tagging." Such surfaces provide a less than ideal canvas for visual expression. The use of color and texture to prevent graffiti has shown promise in El Paso, a city that has been subject to rampant graffiti in the past (see Figures 3.16 and 3.17).

Figure 3.16: Smooth Traffic Barrier Serves as Graffiti "Canvas"

Figure 3.17: Colored and Textured Traffic Barrier in Same Area Is Free of Graffiti

3.8 Aesthetics and Structural Systems

The development of a standardized substructure system may seem at odds with good aesthetic design philosophy. This does not have to be the case. The use of standard sections and details in superstructure design has produced clean, slender and smooth superstructures that have inherent aesthetic value. Development of aesthetically pleasing substructures that can be efficiently constructed is the goal of this research. Providing aesthetic flexibility within a standardized, repetitive framework is the key to obtaining this goal.

Although a limited matrix of member sizes and external sections is necessary to fully realize the benefits of repetitive construction, a variety of actual surface features can be provided with only slight form modifications. Standard member silhouettes can be determined for a range of typical girder bridge applications. Insets, form liners, and color are all instruments with which to modify these shapes to produce systems that are in harmony with their specific surroundings.

Use of a particular prescribed substructure system does not necessarily make for an attractive bridge. The designer must carefully integrate the individual elements of the structure into a coherent whole. Blind selection of “ideal” column and bent cap sizes guarantees neither a beautiful nor an efficient bridge. The finished project must be visualized in concert with its surrounding environment at every stage of the design process. Any system should be used as a set of tools or ideas to be modified and applied appropriately to fit the individual bridge setting.

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CHAPTER FOUR
TECHNOLOGY OPTIONS

4.1 Introduction

In this chapter, some of the technology options available for the construction of substructure elements are outlined. Some of the advantages and disadvantages of using each option in a standardized substructure system are discussed. As discussed in Sections 2.3 and 2.4, the technology incorporated in these options has been used successfully on a wide variety of projects and is well within the grasp of bridge contractors that operate within the state of Texas.

4.2 Concrete Fabrication Options

4.2.1 Cast-in-Place Concrete

As the term implies, cast-in-place concrete is any concrete that is cast in its final service position. With few exceptions, use of cast-in-place concrete is standard practice for substructure elements in highway bridge construction. Thus, the primary advantage of this method of concrete construction is its ubiquity. Designers and constructors are familiar with cast-in-place processes. Ready-mix concrete is generally available in any locality. The collective experience gained from decades of construction practice combined with widespread material availability allows for the construction of substructure elements at low cost.

Another advantage of cast-in-place concrete is its monolithic nature, particularly that of the joints. Member continuity is inherent in cast-in-place joints, so long as they are properly detailed. Proper alignment of concrete members is facilitated by the construction of formwork in the field. Elements can be properly aligned in their final position prior to casting. Slight misalignments can be “corrected” by adjusting the position of members that are cast adjacent to the misaligned members.

4.2.2 Precast Concrete

Precast concrete is any concrete that is cast elsewhere prior to being placed in its final service position. Generally, precast concrete members are produced in the precast plant of a subcontractor that specializes in the production of these types of members and are subsequently transported to the construction site. Pretensioned I-girders are typically produced in this manner.

However, for some projects it is more cost-efficient to set up a precast plant, or yard, at or near the construction site. Members are then precast on site by the contractor or a subcontractor. If the project is large enough, transportation cost savings can offset the costs of setting up the precast yard.

The use of precast concrete offers several advantages over cast-in-place concrete. First, precasting operations are more suited for mass production of standard elements and details. The repetitive use of forms and processes results in better utilization of non-skilled labor and greater economies of scale. In general, the plant environment allows for a level of mechanization not practical in the field. A higher level of quality control is attainable in the precast plant as well. Accurate bar placement results in adequate and uniform concrete cover.

Precasting also benefits from improved concrete technology. High performance concretes are more easily produced in the plant environment. In addition to higher strength, these concretes offer greater stiffness and impermeability, resulting in more durable structures that are less susceptible to attack by chlorides and other aggressive agents. Enhanced durability can result in increased service life and decreased life-cycle costs. Steam curing and the drying that occurs prior to erection decrease the effects of shrinkage on the structure. Likewise, creep effects are reduced due to the stiffness gain acquired during the time period between casting and erection.

The precast plant environment is also ideal in terms of the visual appearance of the concrete. Concrete color and surface finish are generally more uniform due to the repetitive nature of plant production and controlled curing processes. A wide range of quality surface textures is available in the plant environment. Reusable form liners can be used to obtain attractive small- or medium-scale texture. Individual precast element sizes are usually more suited to the use of form liners than large cast-in-place members. As discussed in Section 3.6.1, the use of form liners for cast-in-place members is often marred by the "patchwork" visual effect produced when several form liners are joined together to cover an expanse of concrete larger than the available form liner dimensions. Precast elements can be sized with available form liner lengths in mind to eliminate this shortcoming. Other surface treatment techniques, including sandblasting, hydroblasting, acid-etching and color tinting, can be performed at lower cost with higher quality in the precast plant than in the field.

Precast concrete construction allows for faster on-site construction. Formwork and shoring are kept to a minimum, and precast elements can be rapidly assembled. In addition, poor weather conditions are less likely to disrupt on-site operations. Traffic interference and delays can be greatly reduced, resulting in decreased traffic management costs for the contractor as well as decreased costs to motorists and community businesses. Project costs associated with the mitigation of environmental impact necessarily decrease with reduction of on-site construction time. In sum, reduction of the overall project duration can result in decreased finance costs to the contractor and the owner.

Precasting is also conducive to the use of more efficient and attractive structural forms. Slender members result from the combination of high performance concrete and prestressing technology. A wider range of more complex shapes can be more efficiently produced than with conventional cast-in-place construction. Hollow elements can be readily precast. Use of these sections can reduce dead loads, resulting in foundation cost savings. Voids may also be used to allow for drainage through the interior of members, effectively decreasing the visual impact of surface stains.

Precast concrete construction has its disadvantages as well. Developing continuity is more complex, and connection design is often critical. Dimensional control is more critical than for cast-in-place construction. Adequate alignment of elements must be insured. Diligent geometric control must be maintained during element fabrication processes. Element handling and transportation considerations are more crucial for precast construction. Substantial costs may be involved with the transportation of precast elements. In general, a higher level of construction engineering is required for successful precast operations than for cast-in-place construction. Precast construction is further complicated by the introduction of the precaster as an additional entity in the construction hierarchy.

4.3 Reinforcement Options

The primary reinforcement options for a structural concrete substructure system include mild steel bars or mesh and prestressing tendons of bars or strands. For the case of prestressed concrete, only post-tensioned reinforcement is addressed here although individual precast superstructure segments are sometimes laterally pretensioned. Each substructure bent must consist of multiple elements if precast (due to transportation and handling constraints), and these elements must be connected to act as a continuous frame under service loads. Developing the necessary connections is quite feasible with post-tensioned tendons but impractical with pretensioned reinforcement. Therefore, pretensioned reinforcement is omitted from this discussion.

4.3.1 Mild Steel Reinforcement

The primary advantage of conventional mild steel reinforcement is the fact that it is the established industry standard. Just as almost all highway bridge substructures are constructed with cast-in-place concrete, nearly all substructure concrete is reinforced with mild steel reinforcing bars (rebars). Such practice is quite familiar to all bridge designers and constructors. This type of reinforcement has a lower unit cost than prestressing steel. Contractors have the experience and equipment necessary for this type of construction, and experienced workers are easily attainable.

Unfortunately, mild steel is not as versatile as the primary reinforcement system for precast construction. Several factors combine to make mild steel more difficult to use as the primary reinforcement for a precast system. As previously noted, any precast bent will consist of several precast components. Often the logical joint locations will be at points of maximum moment. Any mild steel used to connect elements at such joints must be fully developed on either side of the joint. Grouted splice sleeve products, mechanical connectors and welding are available for this type of connection. Unfortunately, the ability of some of these products to resist fatigue is suspect, and design provisions are accordingly conservative. The service load effects on non-prestressed substructure connections can result in large steel stress ranges. Non-prestressed rebar connections across these joints cannot prevent them from opening under typical service loads. Even if ultimate strength criteria are satisfied, this condition can result in serious durability and serviceability problems.

Use of mild steel reinforcement as primary reinforcement also prevents the efficient utilization of the strength benefits of high performance concrete. Higher strength reinforcement is necessary to fully realize the strength potential of high performance concrete.

4.3.2 Post-tensioned Reinforcement

There are several advantages to the use of post-tensioned reinforcement, particularly for precast concrete substructures. Post-tensioned concrete is generally stiffer than conventional reinforced concrete because much higher load levels can be sustained prior to cracking of the concrete. Joints and cracks that form during member fabrication and handling are closed once the effective prestress force is applied. This positive control of cracking reduces the early ingress of corrosive elements[1] and permits the protective properties of the concrete cover to function as the pricipal barrier against corrosion. This, in turn, allows one of the major benefits of high performance concrete, its highly impermeable nature, to be fully utilized. Thus, the prestressed member benefits from smaller deflections under service loads and decreased susceptibility to aggressive attack, resulting in a more durable structure. Because the section remains uncracked under service load conditions, the range of stress in the reinforcement is much smaller than that for conventionally reinforced concrete. This also increases the durability of the structure by reducing fatigue effects.

Use of post-tensioning allows the efficient utilization of high performance concrete. In addition to increased flexural and axial strength, shear capacity is enhanced by the axial compression in the member resulting from the prestress force.

Post-tensioned joints make precast substructure elements feasible. The combination of the induced prestress with the service load stresses results in little or no tension across joints under service loading. This allows the assemblage of precast components to perform as a monolithic structure. Accordingly, shear strength at the joint interface is increased, and the serviceability and durability of the structure are enhanced beyond that possible with only mild steel reinforcement.

The primary disadvantage of post-tensioning is its unit cost. As mentioned previously, the cost of post-tensioning reinforcement per ton of steel is much higher than that for mild steel. This higher material cost is substantially offset by the dramatically reduce quantity of steel required. For comparable tensile strength at ultimate, only about 25% of the required amount of Grade 60 mild steel reinforcement is necessary if Grade 270 prestressing strands are used instead. Most post-tensioning anchorage systems are proprietary, but a variety exists from which to choose. Another disadvantage is the more complicated nature of post-tensioned construction. Although the stressing hardware is widely available and is generally lightweight enough so that it can be handled without special cranes, the post-tensioning process adds additioinal steps and requires construction personnel familiar with the relatively simple operations. This complicates construction scheduling for contractors unfamiliar with the process. These complications are generally significant only on the contractor’s first post-tensioned project and can be easily overcome with thorough design and repeated use of standard connection details. Just as is true with other precasting operations, successful post-tensioning requires good construction engineering practice.

4.4 Element Jointing Options

Due to handling and transportation constraints, precast substructure bents generally consist of several precast elements that must be assembled on site. Various options are available for effectively jointing these elements so that they behave monolithically under service loading. Several of these options are discussed below.

4.4.1 Match-cast Joints

Match casting is a process in which a precast segment is formed and cast against the segment to which it will be jointed in the structure. Use of a simple bond-breaking substance between the two segments allows them to be separated after casting. The previously cast segment is placed in storage, while the newly cast segment is used to cast the next segment. The process is then repeated as necessary. Through the use of this technique, perfectly matched joints can be formed between segments.

Two types of joints are possible with match-cast elements. Dry joints (no bonding or protective substance is placed in the reassembled joint) are allowed by the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*[2] for applications with external tendons in areas which never experience freezing. Experience has shown that while these joints can develop adequate shear transfer strength and act more or less like naturally formed cracks, there is a tendency for local crushing or spalling near imperfections, and the concentrated rotation at ultimate capacity at the joint somewhat reduces the flexural capacity.[3]

The use of an epoxy adhesive in the joint is far more prevalent and highly desirable. During construction the adhesive acts as a lubricant, thus easing proper segment alignment. It serves as a local filler, somewhat like the cap on a standard compression test cylinder, relieving local hot spots and ensuring more uniform bearing. Highly agressive exposure tests on CTR Project 1264[4] have shown that the epoxy provides excellent corrosion protection for internal tendons crossing the joint. The excellent durability experience[5] of epoxy-jointed superstructures shows this to be a highly effective joint type. However, use of epoxy joints introduces several complications. The epoxies are weather sensitive. They must be protected from freezing temperatures and cannot be applied during heavy rains or to wet surfaces. The application requires reasonable care to prevent excess epoxy from dripping on traffic or leaving unsightly residue. Probably the most complicating factor is that the joint must cure under compressive stresses. This often requires temporary post-tensioning. The level of compressive stress is low (276 kPa [40 psi]), but on a substructure element with considerable surface area, this can amount to a substantial temporary post-tensioning. This temporary force is often provided by stessing temporary threaded bars which can be reused.

Only a thin layer of epoxy is required to glue the segments together when erected. Slight misalignments created during the casting of one segment are corrected during the casting of the subsequent segment. In this manner, propagation of alignment errors from segment to segment is eliminated. Match-cast, precast segmental construction has proven to be very fast and efficient since its first major application, the Choisy-le-Roi Bridge over the Seine River south of Paris, in 1962 (see Figure 4.1).[6] Application of this technology to vertical shaft substructures has been quite successful. Several projects utilizing match-cast, segmental shafts are described in Section 2.3 of this thesis. An overview of the match casting process used to fabricate pier shafts for the U.S. 183 Viaduct in Austin, Texas can be found in Section 2.3.9.

Figure 4.1: Choisy-le-Roi Bridge

This jointing system is advantageous because it minimizes the amount of work performed during the on-site erection process. Most of the dimensional control work is done in the precast plant, where it can be performed more rapidly, more accurately and more easily than in the field. Interface surfaces, shear keys and ducts are all automatically aligned during the casting process, so the segments fit together properly with no field adjustments necessary. Because the joint has minimal thickness and is sealed with the epoxy adhesive, splicing of ducts is unnecessary.

Careful geometric control must be maintained in the precast plant. Proper segment identification is necessary throughout the fabrication, transportation and erection phases of the project because each segment must be erected in the proper sequence.

4.4.2 Loose-fit, Mortar Joints

Rather than match-casting segments against one another in the precast plant, segments can be individually produced. In this manner, segments can be more rapidly produced in the plant. However, each segment must be properly aligned in its final position, and the labor involved in this alignment operation must be performed in the field.

One method of forming this type of joint involves the use of dry-pack mortar. Adjacent segments are properly aligned with a gap of approximately 12 mm (0.5 in). Mortar of dry consistency is then packed uniformly with a tool. For best results, the mortar should be packed from both sides of the joint. Unfortunately, this is impractical for hollow sections that are too small to allow interior access.

Buttered mortar joints are useful for horizontal joints between vertically stacked segments such as those used for pier shafts. This method is analogous to masonry construction. A coat of cement mortar is troweled on the surface of one segment, and the next segment is then lowered onto this bed of mortar.

Although segments with loose-fit joints can be easier to produce in the plant than match-cast segments, loose-fit joints require much more construction time and effort in the field. The tasks necessary for proper segment jointing and alignment are much more difficult to perform properly in the field than in the plant environment. Post-tensioning ducts must be spliced, mortar must be placed and the segments must be properly aligned. Applying mortar uniformly to the entire segment surface is difficult, and detrimental stress concentrations may occur. Shrinkage cracks and non-uniform placement of mortar may expose the joint reinforcement to aggressive agents and durability problems may ensue. These problems are compounded by the fact that adequate, uniform mortar placement in the joint is hard to verify visually.

Finally, the introduction of the poorly controlled mortar creates a series of weak planes in terms of durability protection for the post-tensioning tendons. The field-applied mortar is sometimes porous and does not always bond well to the precast strata. Premature failure of the Ynys-y-Gwas Bridge in Wales[7] resulted from tendon corrosion due to insufficient protection from the mortar joints used between precast segments.

4.4.3 Cast-in-Place Concrete Joints

When precast segments must be jointed to monolithic concrete, cast-in-place joints are commonly used.[8] The use of this type of joint allows for the proper field alignment of the precast element. This is particularly important if this precast element is the first of a series of match-cast segments. If the first match-cast segment is not aligned as it was during precasting, the subsequent match-cast elements will be out of alignment as well. The cast-in-place concrete also provides a uniform bond between the precast surface and the monolithic concrete element. This type of joint has been used successfully to connect match-cast segmental pier shafts to cast-in-place foundation elements for several of the projects outlined in Section 2.3. Cast-in-place concrete joints were also used to connect precast piles to precast cap beams for the Redfish Bay and Morris & Cummings Cut Bridges described in Section 2.3.7.

Post-tensioning ducts that pass through cast-in-place joints must be spliced and effectively sealed prior to concrete casting. Joint surfaces should be cleaned and roughened to improve bond. Unfortunately, the cast-in-place and precast concrete surface appearances will not match exactly. Ideally, joint concrete should be water-cured prior to post-tensioning in order to prevent shrinkage cracks.[32]

4.5 Casting Orientation

4.5.1 Pier Shafts

Pier shafts should be cast in the vertical position. Thus, each segment's finished surface (top) is a part of a joint in the completed structure, and only the formed surfaces (sides) are exposed to view. This allows for a uniform, quality appearance on all visible surfaces. Chamfers, insets and form-lined textures can be easily produced on each of these surfaces. Use of this casting position also makes it easier to insure proper consolidation around ducts, drains and the inner form (if the segment is hollow). Finally, because they are cast in the same upright position in which they will later be erected, segments can be handled and transported by simple translation. Thus, the difficult and time-consuming process of rotating a segment about one of its horizontal axes is not necessary during any stage of construction.

4.5.2 Bent Caps

Bent cap elements should be cast in the upright position such that the longitudinal axis of the cap is horizontal. In this manner the finished surface of the cap is covered by the deck slab in the completed structure. Thus, only formed surfaces are visible after erection. As mentioned above for the pier shafts, this gives the designer aesthetic flexibility in the selection of surface effects for the visible faces. Horizontal casting also provides the benefit of easier handling and transportation, because no rotation of the precast element about a horizontal axis is required.

One disadvantage to the horizontal casting position is the fact that the primary reinforcement and post-tensioning ducts are perpendicular to the direction of concrete placement. Therefore, it is more difficult to adequately consolidate the concrete around these elements. However, so long as the segment is designed with sufficient spacing between reinforcing elements and good concrete placing practice is maintained, this disadvantage is readily overcome.

4.6 Element Size

Selection of the optimum size for substructure system elements depends on many factors. In general, use of larger precast units decreases the number of units necessary for a structure. Fewer units require fewer joints. Hence, on-site construction time and costs are decreased. Because joints can be particularly prone to penetration by airborne and waterborne chemicals, decreasing the number of joints may increase the durability of a structure.

On the other hand, larger units cost more to handle and transport than smaller ones. Overload permits may be required for transport over public roads. Additionally, dimensional constraints on transportation must be considered. For example, if pier shaft elements are designed to remain upright during transportation and handling phases of construction (for reasons discussed above), the elements must be short enough to safely pass beneath highway bridges during transportation to the site. Transportation and handling stresses are more likely to be critical in the design of large members. Larger elements are more difficult to fabricate; therefore, the efficiencies normally associated with precast plant operations may be forfeited. Construction costs may increase significantly if special equipment beyond that normally used for bridge construction is required.

Larger pieces are also more difficult to incorporate into a repetitive design system. All aspects of roadway geometry will vary even for the simplest project. Required column or shaft heights are particularly variable because of roadway grade, vertical curvature and changes in ground elevation. Design flexibility decreases with increasing element size. Smaller units, or segments, can provide geometric flexibility while maintaining the repetition necessary to reap the full benefits of precast construction. Although the total number of joints is increased, the repetitive nature of these connections allows them to be completed quickly and efficiently if properly designed.

In general, precast segments should be designed as repetitive units. These units should be designed as large as practical considering casting, transportation and handling constraints while maintaining the repetitive nature of their production and erection. In addition, segments should not be so large as to require extra capacity lifting equipment beyond that normally required for precast, pretensioned concrete I-beam bridge construction.

4.7 Tendon Types

Both strand and bar tendons are readily available for post-tensioning operations. These tendon types have been used successfully on many projects in Texas, and there are a number of contractors familiar with construction using these post-tensioning systems.

4.7.1 Bar Tendons

The primary advantage of threaded bar post-tensioning tendons is their ease of use. They are readily coupled, allowing application of prestress to each segment in the structure prior to placement of subsequent sections. Bars are readily available in lengths exceeding 12 m (40 ft) with an ultimate tensile strength, f_{pu}, in excess of 1030 MPa (150 ksi). The bars are relatively rigid, making them easy to advance through straight ducts. Seating losses are minimal because end anchorages are screwed into position on the threaded bars. Post-tensioning jacks for bar tendons can be easily handled by one or two laborers.

Threaded bar tendons are ideally suited for construction of precast, segmental pier shafts. Curved tendons are usually not necessary, because shafts are post-tensioned concentrically. Each segment can be prestressed to resist imposed construction loads prior to the placement of the next segment. The remainder of the required bar tendons can be installed at intervals of up to 12 m (40 ft), or after all shaft segments are erected (if the shaft height is less than 12m). Only those bars required for construction loads need to be stressed prior to placement of the final segment. Thus, coupling and stressing operations are minimized.

4.7.2 Multiple Strand Tendons

Multiple strand tendons are available in various strand sizes and configurations. Typically, tendons consist of 13 or 15 mm- (0.5 or 0.6 in-) diameter low relaxation strands with an f_{pu} of 1860 MPa (270 ksi). Therefore, less prestressing steel is required than for bar tendons. Another advantage of strand tendons is their ability to follow a curved profile in the structure. Thus, the eccentricity of prestress can be varied along the length of member, and a “U” pattern can be used to eliminate costly anchorage in the base region, where it is difficult to protect against the effects of groundwater ingress.

Unfortunately, multiple strand tendons are more difficult to work with than bar tendons. Splicing of multiple strand tendons can be prohibitively difficult and expensive. In addition, post-tensioning jacks are typically too large for one person to handle. However, multiple strand tendons are necessary for the construction of segmental bent caps because the larger prestressing force and variable eccentricity provided by this type of tendon are needed to keep stresses at joints within allowable limits.

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5. Freyermuth, C.L., "Durability of Post-tensioned Prestressed Concrete Structures," *Concrete International*, October 1991, pp.58-65.
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7. Vignos, René P. *Test Method for Evaluating the Corrosion Protection of Internal Tendons across Segmental Bridge Joints*, Master's thesis, The University of Texas at Austin, May 1994, pp. 5-6.
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CHAPTER FIVE
TECHNOLOGY DETAILS

5.1 Introduction

Details regarding the implementation of a standardized, precast substructure system can be divided into two main groups — construction considerations and structural considerations. Construction considerations include issues pertaining to the fabrication, transportation, and erection of the substructure elements. Details related to the ability of the system to resist imposed loads and meet serviceability requirements comprise structural considerations.

5.2 Construction Considerations

The widespread use and economic success of a structural system are usually interdependent. A system must be economically feasible in order to be selected for construction. At the same time, economic benefits are not maximized until the system is widely used. Efficiency heightens applicability, which in turn enhances efficiency. The development of the precast, pretensioned I-beam superstructure is an example of this autocatalytic process. Ease of construction and standardization of girder cross-sections have led to widespread use of a few shapes. The resulting familiarity with the design and construction of this type of bridge in conjunction with the repeated use of forms and equipment has resulted in lower costs and increased use of the system.

Repetition, simplicity and flexibility are of paramount importance to the successful implementation of a precast substructure system. These three qualities allow for the minimization of labor costs and on-site construction time. Likewise, a high degree of construction engineering is required to achieve these qualities.

5.2.1 Fabrication of Substructure Elements

Successful systematic production of substructure elements requires the utilization of forms that are reusable, not only on a specific project, but on a wide range of projects. A few standardized element sections and sizes should be used. Thus, several precast producers could economically produce the entire range of system elements. The designer could then freely specify the standard elements required without a specific producer in mind. Competition between producers, each able to efficiently produce the system elements, would then produce downward pressure on fabrication costs.

Similarly, individual form parts might be interchangeable within the formwork system required for various element sizes and sections. Wherever possible, use of common dimensions maximizes the interchangeability of parts. This repetitive feature is not only applicable to the outer form of segments, but also to the inner core form (hollow members) and other features such as shear keys, chamfers, insets and placeholders for post-tensioning ducts.

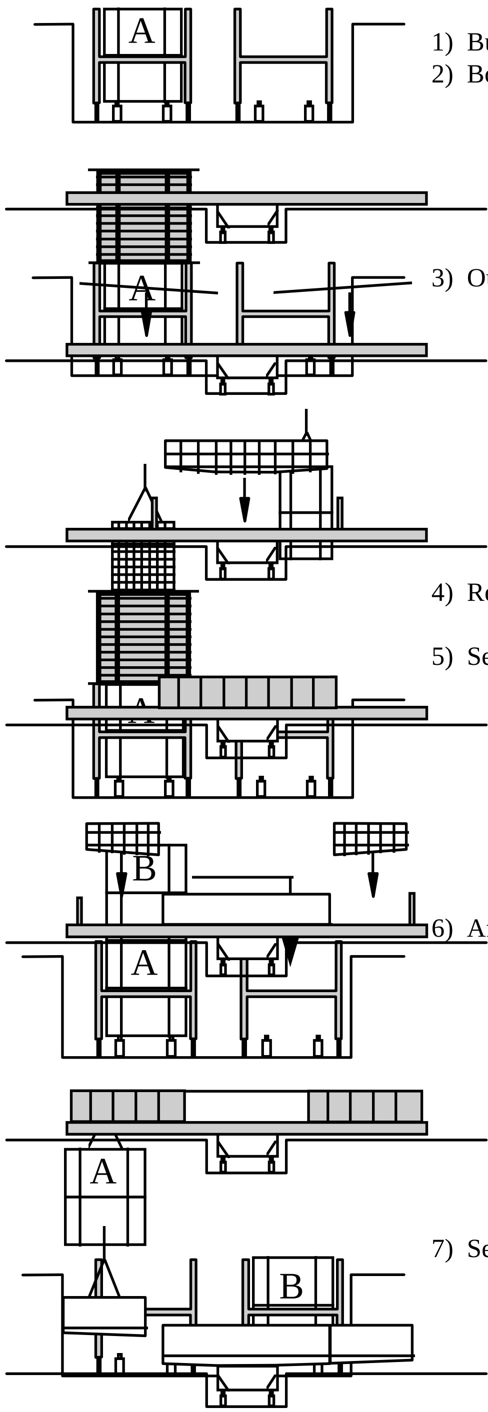
Simplicity of forms is also imperative. Inserts and form liners should be used to vary the external geometry of sections. In this manner a simple set of forms can produce a family of shapes that share the same overall dimensions but can be distinguished from one another by certain geometric features or texture. Forms must be easily constructed and readily removed from the hardened concrete. Use of planar surfaces and obtuse angles is recommended.

The fabrication process should be standardized as much as possible. Reinforcement locations and details should be constant. For example, potential locations and spacing of post-tensioning tendons should be standardized. This is analogous to the location of pretensioning tendons at regular 50 mm- (2 in-) intervals in standard beam sections. Not every potential tendon location need be utilized in every member, but the potential locations should remain the same. Similarly, mild steel reinforcement quantities and details should be as constant as possible. In this manner, element reinforcement cages can be fabricated as quickly and accurately as possible with minimum effort and time expended on deciphering complicated shop drawings and inspecting the as-built layout. Shear key sizes and locations should also be standardized.

In addition to reducing on-site erection time, use of match-cast, segmental processes can speed up the fabrication process as well. Careful geometric control need only be enforced when aligning the contiguous segments for casting. This is not to say that sloppy workmanship should be allowed. Rather, proper alignment of post-tensioning ducts is facilitated by the existence of the ducts in the previously cast segment. Similarly, meticulous geometric alignment of items such as shear keys is not necessary, because the concrete of the segment being cast will be formed against the previously cast segment, guaranteeing an excellent match. Match-casting is an "autocorrecting" process.

The fabrication of substructure elements as a series of segments also allows for the optimal use of form liners. As mentioned in Section 3.6.1, the proper use of form liners to introduce texture requires the careful consideration of the joints between form liner pieces. The joints that run perpendicular to the overall flow of the form liner texture are often difficult to conceal. Proper handling of these joints must be carefully considered by the designer and the desired layout should be explicitly specified in design drawings, lest the visual benefit of the texture be lost. The use of segmental technology offers an easy solution to this problem. As long as the segments are not longer than an individual form liner unit, typically 2.4–3.6 m (8–12 ft), the disturbing joints are not necessary within the length of a segment. Thus, each segment can be cast with a single length of reusable form liner running the length of the segment, thereby eliminating the problem. Form liner joints are merged with the articulated joint between segments.

As explained in Section 4.5.1, pier shaft segments should be cast in the vertical position. Thus, only formed surfaces are exposed in the final structure. Proper concrete consolidation is more easily achieved, and segment handling is minimized. Because of this casting alignment, “short-line” match-casting is essential (see Figure 5.1). In this process, new Segment B is cast against previously cast Segment A. After adequate curing, the forms are removed from Segment B, and the two segments are separated. Segment A is then transported to storage, and Segment B is placed in the position previously occupied by Segment A. New Segment C is then formed and cast against Segment B, and the process is repeated. In this manner, the entire casting assembly is never more than two segment heights tall, thus minimizing the scaffolding and lifting required. This method has been successfully utilized for projects such as the U.S. 183 Viaduct described in Chapter 2.



1) Bulkhead Segment A is aligned and surveyed.
2) Bond-breaking substance is applied to segment.

1) Align template segment for desired cross-slope and apply bond-breaking substance to top surface.
2) Install forms for bottom flange of cap.

3) Outer form for Segment B is positioned and surveyed.

3) Insert forms for tapered cap soffits.

4) Install joint forms.
5) Insert reinforcement cage and post-tensioning ducts for primary cap segment.

4) Reinforcing cage and central void form are inserted.
5) Segment B concrete is cast.

6) Install side forms for web of cap.
7) Cast concrete for primary segment after verifying proper form alignment.

8) Remove web and joint forms.
9) Apply bond-breaking substance to primary segment joint faces.

6) After adequate curing, forms are removed, and Segment B is moved into position as bulkhead segment for casting of Segment C.

10) Install end forms.
11) Insert reinforcement cages and ducts for secondary segments.

12) Install web forms for secondary segments.
13) Cast concrete for secondary cap segments.

7) Segment A is removed for storage, and entire process is repeated.

14) Remove forms.
15) Survey as-cast alignment.
16) Remove segments for storage.

Figure 5.1: Short-Line Match-Casting Process for Pier Segments

Figure 5.2: Long-Line Match-Casting Process for Cap Segments

Bent cap elements should be cast horizontally for the reasons given in Section 4.5.2. Unlike pier shaft elements, the cap elements should be cast in "long-line" fashion. In this manner, all the segments in a single cap assembly are cast adjacent to one another in their final relative position and alignment. This is necessary for several reasons. Ideally, bent caps, especially for single pier bents, are not prismatic because of features such as sloping cantilever soffits. Thus, identical forms cannot be used to cast every cap segment. The long-line method allows for the entire cap assembly soffit to be geometrically formed while portions of the side forms are placed along with temporary bulkheads. The individual elements within this assembly are match cast sequentially. This process also allows the designer a certain degree of flexibility in locating joints between segments. The location of these joints might be controlled by weight restrictions on the individual segments or by allowable joint stresses in the final structure. This method also allows the use of continuous reusable inserts to form chamfers or sloping soffits. These inserts can be placed in the formwork assembly whole without being cut or adjusted into pieces for the individual segments. Once used, these insets can be used again on a cap with a similar overall shape but different joint locations.

5.2.2 Transportation of Substructure Elements

As discussed in Section 4.6, the maximum allowable sizes and weights of segments are typically determined by transportation and handling considerations. If pier shaft segments are transported upright, their height will generally be limited by overhead highway clearances. Thus, the combined height of a pier segment and trailer should not be taller than 4.8 m (16 ft). Longer segments could be transported sideways. However, this practice would result in greater handling difficulties and loss of the ability to transport more than one segment per trailer. Cap segment sizes will generally not be restricted by geometric constraints because segment heights and widths will be less than 3 m (10 ft) and lengths will be considerably shorter than the lengths of pretensioned girders that are currently transported.

Segment weights should be limited to those that can be transported and handled by equipment already in use for moderate span concrete highway bridge construction. Considering the weights of I-girders and U-girders that are commonly transported and lifted into position on present construction projects, a limit of 360 kN (80 kips) per lifting crane should be used. This is also the limit recommended by ACI-ASCE Committee 343[1]. Thus, pier shaft segments, which would be lifted by only one crane, should be limited to this weight. This should not present a problem considering typical hollow pier segment dimensions. Accordingly, cap segments that would be lifted by a crane at each end should be limited to a total weight of 710 kN (160 kips). However, transportation constraints, such as allowable truck axle loads might lower this limit, depending upon the length of the segment involved. An overall segment limit weight of 670 kN (150 kips) should be used in design. Cap segments that are to be lifted by only one crane should be limited to 360 kN (80 kips). Of course, some of these limits may be increased for projects that warrant larger erection equipment or different means of segment transportation such as by barge or rail.

5.2.3 On-Site Erection of Substructure

As with segment fabrication, simplicity and repetitiveness are necessary for rapid and efficient substructure erection. Match casting enhances both the speed and simplicity of on-site connections. Thus, match-cast, epoxy joints should be used whenever possible. In some situations, where geometric flexibility is of great importance, cast-in-place joints may be used, but careful encapsulation of tendons should be provided for durability protection. Post-tensioning technology allows for rapid connections between segments.

5.2.3.1 Pier Shaft

Pier shafts should usually be designed with threaded post-tensioning bars as the primary longitudinal reinforcement system. Because transverse loads on piers are variable both in magnitude and direction, there is usually no advantage to be gained from varying the eccentricity of the prestressing force. Also, pier shafts are constructed in a progressive fashion, with each new segment being supported by the previously erected segment. Because these threaded bars can be easily coupled between segments, they present an ideal means of introducing a concentric prestressing force in a progressive fashion. The threaded bars can be post-tensioned to secure one segment, and then the reinforcement for the next segment can be coupled directly to these bars and post-tensioned in turn. The post-tensioning jacks for bars can be readily handled by one or two workers.

Pier shaft erection begins at the top of the foundation. The post-tensioning bars necessary for the longitudinal reinforcement of the pier are anchored within the cast-in-place foundation cap. The connection between the foundation cap and the first pier segment is critical to the proper alignment of the pier shaft as a whole. Unfortunately, because the foundation is cast in situ, the joint surfaces of these two elements are not match-cast. A slight alignment error at this level can result in a pier that is significantly out-of-plumb. One solution to this problem is the use of a cast-

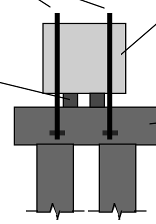


Figure 5.3: Temporary Support of Pier Starter Segment

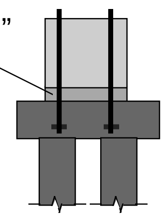


Figure 5.4: Starter Segment “Locked” with Cast-in-Place Joint

in-place concrete joint or pedestal. The first pier segment is supported either in a recess in the foundation cap or above the surface of the cap. After properly aligning the segment with shims or other supports (see Figure 5.3), post-tensioning ducts are spliced from the foundation to the segment, and mild steel joint reinforcement is placed. As depicted in Figure 5.4, cast-in-place concrete is then cast around the joint, ensuring accurate alignment. The cast-in-place portion of the joint may be hidden below grade or extended to a height of approximately 1000 mm (40 in) above grade to form an apparent pedestal. The height of this joint can be chosen to adjust the overall pier height from that given by a series of standardized segments to that required. Similar methods have been used on the Chesapeake and Delaware Canal Bridge and the U.S. 183 Elevated project described in Chapter 2.

Once the cast-in-place joint has reached adequate strength, the first pier segment is post-tensioned to the foundation. Stressing need only be applied to the bars required to counteract construction loads during this phase of the construction. Typically, only four bars (two per axis) need to be stressed prior to placement of the cap. Thus only anchorages and couplers need be used for the stressed bars at each segment level. The remaining bars may be coupled at whatever length is most practical.

After post-tensioning of the first pier segment, the subsequent segment is then epoxied into place. Post-tensioning bars may be installed and coupled either before or after placement of the section, so long as they are adequately coupled. The necessary bars are then stressed, and the process is repeated until all shaft segments are installed. Once all bars are installed, stressed and anchored, each bar and anchorage is grouted.

5.2.3.2 Template

The greatest geometric challenge is encountered at the pier-cap interface. Superelevation of the bridge deck is almost always present due to either horizontal curvature or drainage considerations. This cross-slope typically prevents the pier and cap from intersecting at a right angle. One solution to this problem is the introduction of a collar segment, or *template*, that serves a triple purpose of

1. improving the aesthetics of the system by better illustrating the flow of forces from the cap to the pier,
2. resolving the cross-slope of the cap and superstructure with the verticality of the pier, and
3. ensuring the proper vertical alignment of the cap.

The name and structural purpose of this template are inspired by the element of the same name employed successfully on the Northumberland Strait Crossing to ensure the proper vertical alignment of 190 m- (623 ft-) long cantilevered box girders (Section 2.3.10.).

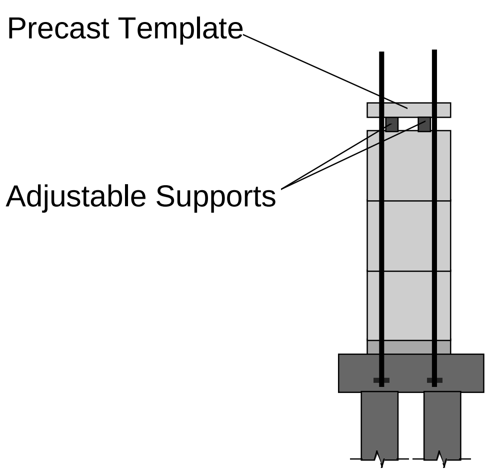


Figure 5.5: Temporary Support of Template Segment

The template segment is cast prior to casting of the cap. The top surface of the template should be sloped according to the desired superelevation of the superstructure. As discussed in Chapter 2, almost all superelevations will fall within the range of 20–50 mm/m (0.02–0.05 ft/ft). Thus, the standard template segment should have the ability to be cast with three nominal surface slopes: 0, 20 and 40 mm/m (0, 0.02 and 0.04 ft/ft). Any superstructure cross-slope that falls within the range of 0–50 mm/m (0.02–0.05 ft/ft) may be produced by utilizing these nominal template slopes

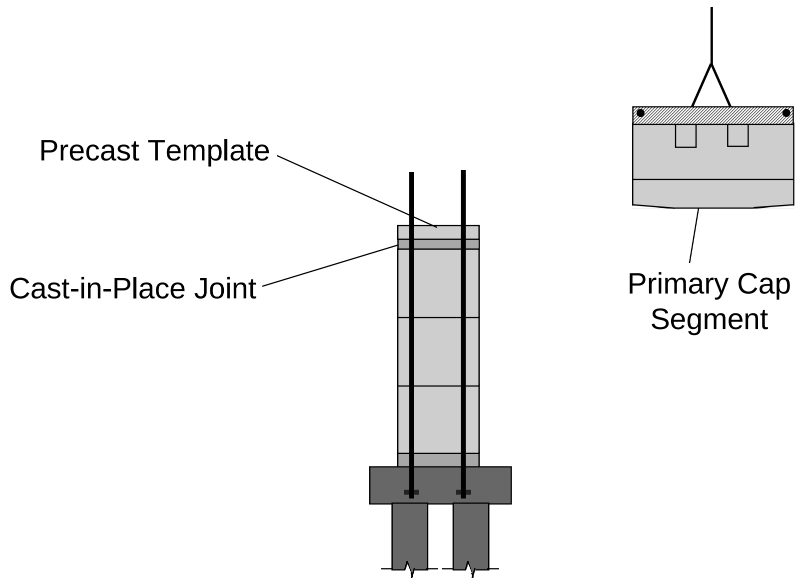


Figure 5.6: Placement of Primary Cap Segment After Template Is Fixed with Cast-in-Place Joint

and by adjusting the heights of the girder bearing seats or pads. Seat or pad height adjustments of 75 mm (3 in) or less would generally be required to form any specified cross-slope.

Once it is cast with the proper surface slope, the template is used as the bottom form for the central, or *primary*, cap segment. This primary segment is then match-cast against the template, ensuring a precise and rapid fit in the field. During erection the template segment is temporarily supported above the top pier segment and carefully aligned as depicted in Figure 5.5. After post-tensioning ducts have been spliced and proper alignment has been achieved, the template is locked into position atop the pier with a cast-in-place joint. As does the cast-in-place joint at the base of the pier, this joint presents another opportunity for the designer to adjust the overall height of the pier to the desired value without using non-standard precast units. Once the cast-in-place joint has hardened and the template has been post-tensioned to the pier shaft, the primary cap segment may be placed atop the template (see Figure 5.6). Difficult and time-consuming alignment of the unwieldy cap segment is unnecessary because of the careful alignment of the smaller and more easily handled template segment. The match-cast joint ensures a precise mutual fit and alignment between the template and cap segments.

5.2.3.3 Cap

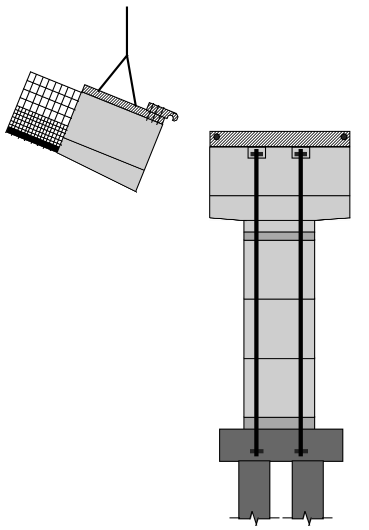


Figure 5.8: Placement of First Secondary Segment

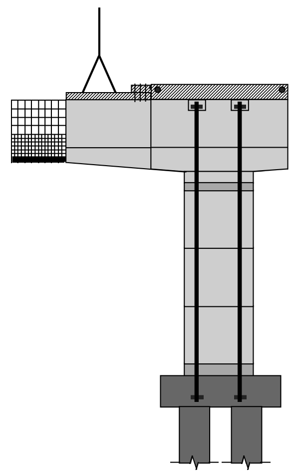


Figure 5.9: First Secondary Segment in Final Position

After the primary cap segment is erected atop the template segment, all remaining pier shaft reinforcing bars are installed and anchored at the top of the cap (see Figure 5.7). Additional cap segments, if required, are erected in balanced cantilever fashion, and epoxy is applied to each joint. These cantilever, or *secondary*, segments are lifted into position and then temporarily supported from the previously erected segment (see Figures 5.8 through 5.11). Finally, post-tensioning tendons are installed, stressed and grouted (see Figures 5.12 and 5.13).

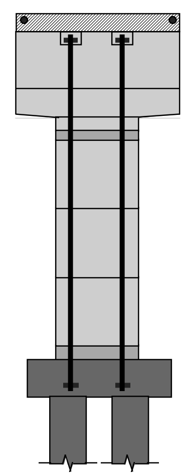


Figure 5.7: Post-tensioning of Primary Cap Segment to Pier Shaft

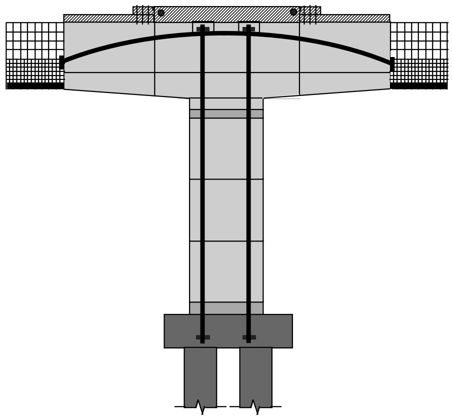


Figure 5.12: Post-tensioning of Cap

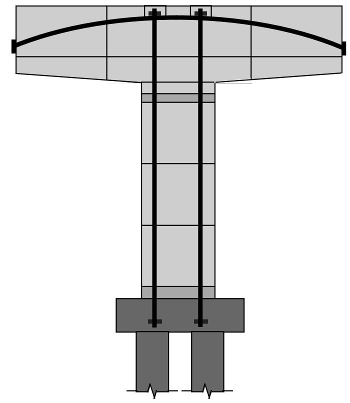


Figure 5.13: Final Substructure

Multiple strand tendons should be used as the primary longitudinal reinforcement for the cap. The large gravity forces supported by the cap require the ability to vary the eccentricity of the

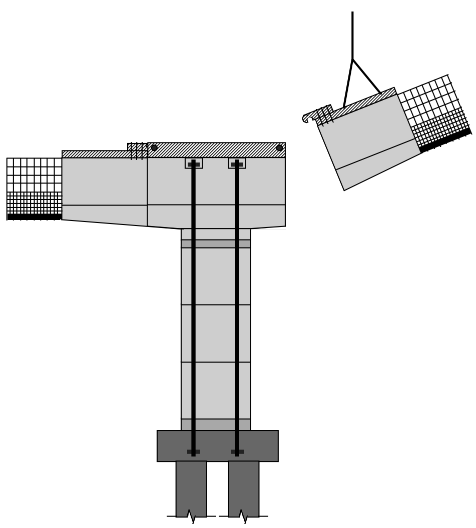


Figure 5.10: Placement of Second Secondary Segment

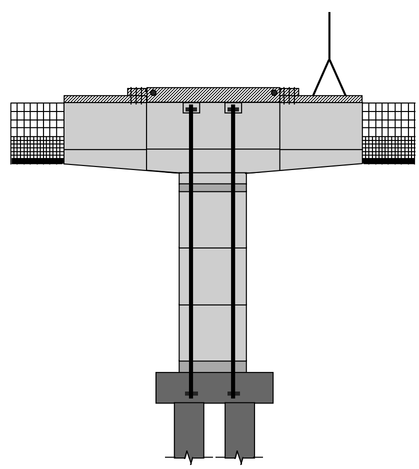


Figure 5.11: Second Secondary Segment in Final Position

resisting prestress force. Also, because the caps are constructed as balanced cantilevers, coupling of post-tensioning reinforcement is not necessary. Thus, multiple strand tendons are a natural choice for cap reinforcement.

Staged post-tensioning of the cap might be necessary in some cases where full post-tensioning prior to placement of the superstructure girders might be restricted by allowable stress

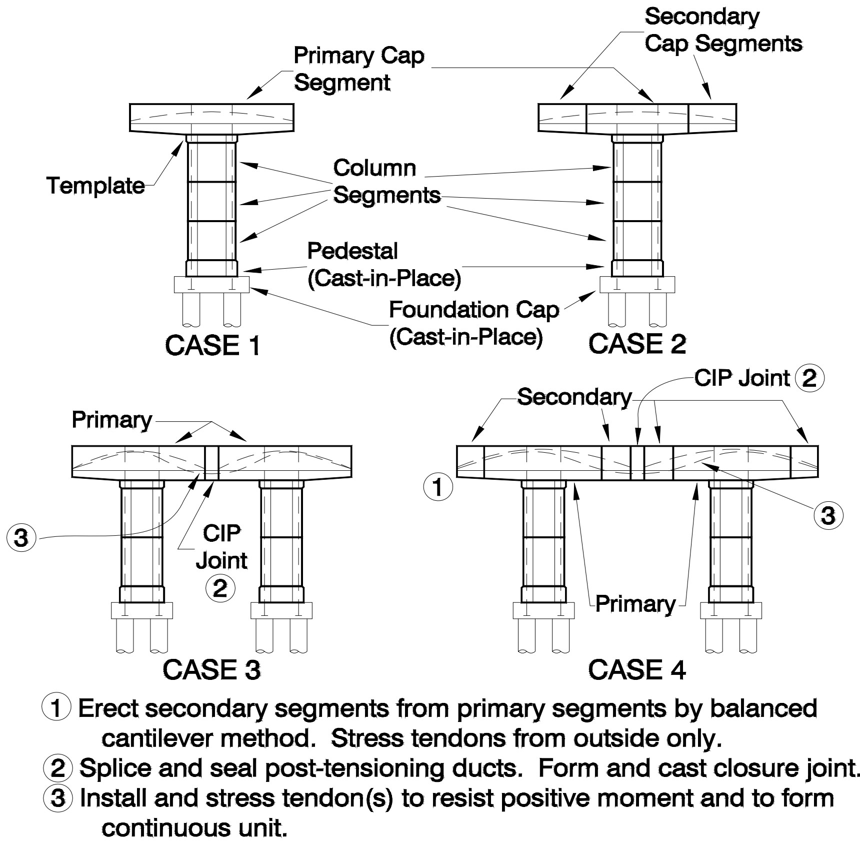


Figure 5.14: Typical Bent Configurations

limitations. If this is the case, some of the tendons may be only partially stressed or left unstressed until after some or all of the girders are in place. Once all tendons have received the final amount of prestress, the tendons and anchorages are grouted to protect against corrosion.

5.2.3.4 Bent Configurations

Four typical bent configurations are shown in Figure 5.14. Case 1 and Case 2 represent single pier bents. For some ramp structures supporting only one traffic lane, a single primary cap segment may be adequate to form the entire cap length. The resulting configuration, labeled Case 1 in Figure 5.14, requires no secondary segments. The primary segment is post-tensioned to the template and pier shaft and then multi-strand cap tendons are installed if necessary. Once all anchorages are grouted, the erection is complete.

The configuration labeled Case 2 is that which will be required for most single pier bents. This configuration requires a central primary segment on top of the template. The size of this primary segment is determined by handling and transportation requirements. Additional secondary segments, erected in balanced cantilever fashion as described in the previous section, are necessary to provide the required cap length.

For longer bents, multiple pier bent configurations are often required. The construction sequence for this type of bent can best be visualized if the bent is thought of as a series of single pier bents that are erected independently and then post-tensioned together. A primary cap segment is erected at the top of each pier and template assembly.

Case 3 represents a multiple pier bent configuration that requires two primary cap segments only. In general, a single pier bent with a pair of secondary segments can form a cap that is at least as long as any that can be formed with Case 3. Therefore, Case 3 is primarily applicable to situations that require a straddle bent. A straddle bent is necessary when the use of central single pier is precluded by an obstacle beneath the bridge such as another roadway. The Case 3 configuration can be thought of as the combination of two Case 1 bents. Each pier and template assembly is constructed as for a single pier bent. Each primary cap segment is placed into its final position atop its respective template. The cap segments are designed so that there is gap between the erected segments. This gap, which is used to form a final cast-in-place closure joint between the primary segments, should be approximately 400–1000 mm (16–39 in) long. Once the cap segments are in position, post-tensioning ducts are spliced between segments and mild steel joint reinforcement is placed. Once the closure joint concrete has been cast and reaches adequate strength, the cap is longitudinally post-tensioned to form a continuous frame.

Case 4 represents the bent configuration necessary for most cap lengths that require a multiple pier bent. This configuration can be thought of as a series of Case 2 configurations. Each pier and template assembly is erected independently, and each primary cap segment is post-tensioned to its matching template. In order to form the necessary cap length, secondary segments are required at the ends of the cap and between the piers. These secondary segments are erected as cantilevers from the primary segments as described in the previous section. One cast-in-place cap closure joint is necessary between each pair of piers to join the portions of cap supported by each pier into a continuous unit. The closure joints are formed in the same manner as those for Case 3. Once the cast-in-place concrete has reached adequate strength, the entire cap is post-tensioned together.

5.3 Structural Considerations

There are three separate specifications produced by the American Association of State Highway and Transportation Officials (AASHTO) that can be applied to the design of a precast substructure system. The latest edition of AASHTO's *Standard Specifications for Highway Bridges*[2] was published in 1992 and updated with revisions in 1993 and 1994. This code, hereafter referred to as AASHTO '94, has been used historically for concrete highway bridge design in the United States. Unfortunately, this specification gives little direction for segmental construction. However, AASHTO has a separate specification for this type of construction, *Guide Specifications for Design and Construction of Segmental Concrete Bridges*[32], hereafter referred to as the AASHTO Segmental Code. It was introduced in 1989 and updated in 1993, 1994, and 1995. The focus of this specification is on segmental superstructures. A newer *AASHTO LRFD Bridge Design Specification*[3], was introduced in 1994. This code, hereafter referred to as AASHTO LRFD, more uniformly incorporates reliability-based design and the associated load and resistance factor design concepts than AASHTO '94. Plasticity-based concepts, such as strut and tie modeling, are allowed for designing regions such as anchorage zones for which Bernoulli's "plane sections remain plane" bending hypothesis does not apply.

The purpose of this section is to help the designer understand the ramifications of each of the codes on precast, segmental *substructure* design while combining their provisions to suggest a set of rational design guidelines. Regardless of the particular code utilized, the substructure design must satisfy strength requirements when subjected to ultimate load combinations, and satisfy serviceability requirements when subjected to service load combinations.

5.3.1 Pier Shaft and Template

In general, the load combination that maximizes the biaxial bending effects in the pier and template assembly will control the combined axial-flexural design of these members at service and ultimate load levels. Either service or ultimate design criteria may control the pier design. Due to discrepancies in service load design philosophy between design codes, the choice of design code may determine whether service limit state criteria or ultimate limit state criteria control the design of the substructure. In addition, overturning moments due to construction loads may be critical and should always be considered.

5.3.1.1 Design Criteria

The applicable codes are fairly uniform in their handling of ultimate strength design. Although the nomenclature varies from code to code, there are three primary load combinations that must be considered:

1. Loads associated with normal vehicular use of bridge without wind,

2. Loads associated with exposure of bridge to wind velocities in excess of 90 km/h (55 mph) with no significant live load, and

3. Loads associated with normal vehicular use of bridge combined with 90 km/h (55 mph) wind load.

The AASHTO LRFD specification contains the most rational presentation of these load combinations and the associated load factors and is therefore recommended. Values obtained from other codes do not vary significantly and may also be used.

All of the codes agree on the basic design criterion that tends to control service load design of segmental structures. For match-cast, epoxy joints with no auxiliary mild steel reinforcement, no tensile stress is allowed at the joint under service loads. This stipulation is primarily rooted in durability concerns. Any opening of the joint under regularly occurring service loads has the potential of exposing the reinforcement to air-and water-borne aggressive agents. Although the epoxy joint sealant has a nominal tensile capacity greater than that of the surrounding concrete, its presence is ignored in an effort to take into account the possibility of poor mixing and application of the epoxy during erection. Unfortunately, considerable disparity exists between the service load combinations specified for consideration by each of the codes.

Because the service load combinations specified in AASHTO ’94 are intended for use with allowable stress design procedures, they do not take into account the transient nature of vehicular and wind loads. Thus, for the first and second load situations stipulated above, no tensile stress would be allowed under the full vehicular or full wind design loads. The third load situation would allow no tensile stress under a combination of the full vehicular design load and 30% of the full design wind load, representing the maximum wind conditions under which full traffic would be expected on the bridge. This philosophy can be deemed overly conservative when considering two facts. First, wind loads of this level are rarely, if ever, experienced by the structure. Second, even if the full vehicular or full wind loads were applied to the structure, their application would be transient in nature. The relative merit of this design philosophy seems dubious at best, especially when the joint itself, if properly constructed, should not open prior to cracking of the surrounding concrete. Unfortunately, if the AASHTO ’94 specifications are used, this overconservative service load design philosophy becomes critical in the pier design, resulting in the addition of extra post-tensioning reinforcement that is not necessary to resist the ultimate factored loads.

The AASHTO LRFD Code offers a more rational approach to service load design. As with the other codes, no tensile stress is allowed at the joints under service load combinations. However, AASHTO LRFD has one explicit service load combination, Service III, “relating only to tension in prestressed concrete structures with the objective of crack control” (pp. 3-8,9). This load combination does not include wind loads at all, due to their transient nature, and only 80 percent of the design vehicular loads are applied. This reduced vehicular load represents an event that is expected to occur about once per year for bridges with two design lanes and less often for bridges with more than two design lanes. Thus, when using this code, the pier sizing and reinforcement layout are much more likely to be controlled by ultimate strength design.

Use of the AASHTO LRFD Code is recommended for service load design with one stipulation. Designers should consider the addition of a portion of the design wind load (up to 30%) to the Service III load combination for bridges in regions that are regularly subjected to high, sustained winds. This is particularly important for tall piers, which are subjected to considerable wind-induced moments at or near the base of the shaft.

Construction loads must also be considered in the design. Construction load design for segmental structures is covered in Section 5.14.2.3 of AASHTO LRFD. This section presents several construction load combinations for which the joint tensile stress must be limited to $0.5\sqrt{f'_c}$ MPa ($6\sqrt{f'_c}$ psi). Compressive stress is limited to $0.5f'_c$. One construction load situation that might control the design of the pier shaft occurs during erection of the superstructure girders. If girder erection proceeds from one end of the bent to the other, the unbalanced moments produced by the erection of one half of the girders on one side of the bent may cause critical biaxial bending stresses in the pier shaft. The unbalanced construction dead load moments produced in the pier shaft may be greater than any service load moments experienced during the life of the structure.

5.3.1.2 Section Efficiency

Because of the allowable stress design criteria that must be satisfied under service and construction loadings, maximizing section efficiency is very important in the design of precast, segmental substructures. For a doubly symmetric, concentrically prestressed section, the concrete axial stress at a particular location based on elastic theory can be found from Equation 5.1.

$$\sigma_c = -\frac{P+F}{A} \pm \left(\frac{My}{I}\right)_u \pm \left(\frac{My}{I}\right)_v \qquad \text{(Eqn 5.1)}$$

σ_c = Axial concrete stress (+ tension, – compression)

P = Applied axial force

F = Effective prestress force

A = Area of section

M = Applied moment in direction being considered (u or v)

y = Distance from neutral axis to stress location in direction considered (u or v)

I = Moment of inertia in direction being considered (u or v)

u = Major axis bending

v = Minor axis bending

If σ_c is limited to values less than or equal to zero (no tension), then the magnitude of the compressive stress due to combined axial and prestress forces (first term) must be greater than or equal to the sum of the stresses due to bending about each axis (second and third term). Thus, it can be seen that for a given load combination and effective prestress force, increasing the ratios $\frac{I_u}{A}$ and $\frac{I_v}{A}$ results in less tension (or more compression). Therefore, hollow sections are ideal for segmental piers. The material absent from the central void of the pier section greatly reduces A while I is only slightly decreased. Considering a section's radius of gyration, r, the ability of the section to comply with the zero tensile stress design criterion is directly related to $r^2 = \frac{I}{A}$ and the amount of prestress force available.

5.3.1.3 Concrete Strength

Increased concrete compressive strength is beneficial for both service and ultimate load design. Under service load combinations, AASHTO LRFD limits the allowable concrete compressive stress to $0.45f'_c$. This limit must be satisfied under the Service I load combination, which represents the combined effects of full vehicular load and a wind velocity of 90 km/h (55 mph). Thus, the magnitude of the compressive stress contribution due to biaxial bending moments under the Service I combination will be significantly greater than the tensile stress contribution due to biaxial bending moments under the Service III combination. In addition, the compressive stress component due to axial load and effective prestress will also be increased due to addition of the remaining 20% of vehicular load not included in Service III. Figure 5.15 illustrates the different stress distributions produced by these two service load combinations. If one considers that in one quadrant of the pier section all three terms of Equation 5.1 tend to increase the compressive stress, it is apparent that large concrete compressive stresses are possible. Thus, application of the zero tensile stress criterion in one corner of the pier, combined with concentric prestressing, results in

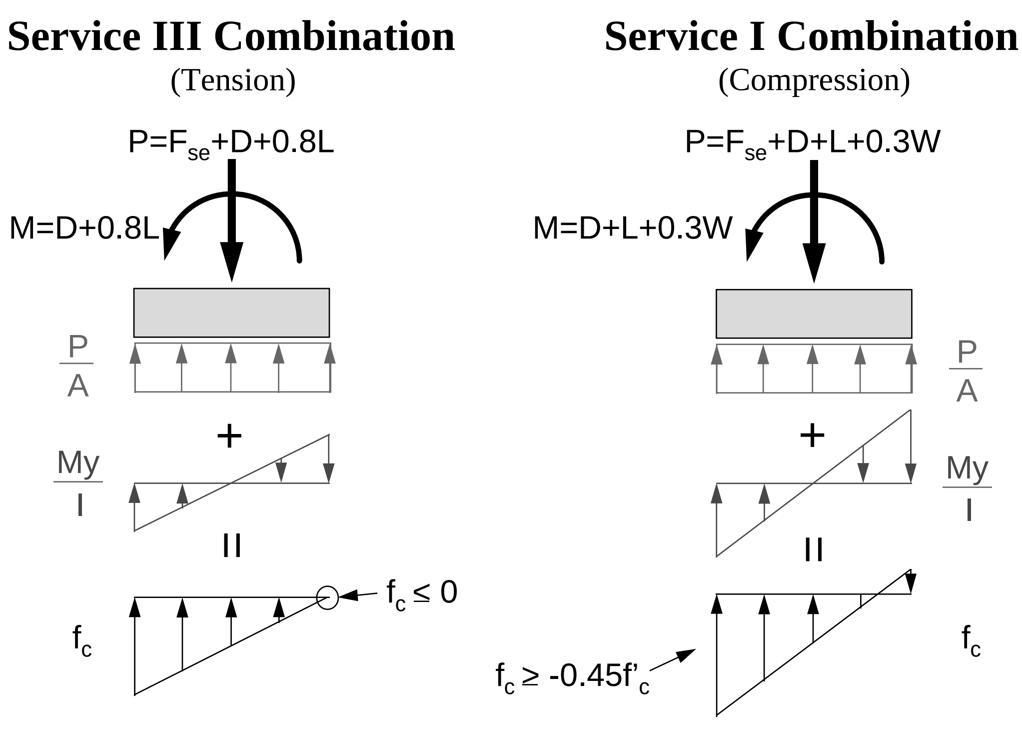


Figure 5.15: Stresses Resulting from Service Limit State Load Combinations

large compressive stresses in the opposite corner of the pier. In order to meet the compressive stress limit of $0.45f'_c$, High Performance Concretes (HPC) with compressive strengths ranging from 56 to 70 MPa (8 to 10 ksi) will often be required.

Use of HPC also reaps benefits in designing for ultimate load. Increasing the concrete compressive strength of a typical pier subjected to simultaneous bending and axial load can significantly increase the moment capacity of the pier. Typical interaction diagrams for a hollow pier section such as P28 in Figure A.2 with f'_c = 28 MPa (4 ksi) and f'_c = 56 MPa (8 ksi) are shown in Figures 5.16 and 5.17. If subjected to a typical ultimate axial load, $P_u = \phi P_n$, of 8500 kN (1910 kips), the factored major axis bending moment resistance, ϕM_n, of this section (Figure 5.16) would be 15600 kN-m (11500 kip-ft) with f'c equal to 28 MPa (4 ksi). However, if f'c is increased to 56 MPa (8 ksi), the factored moment resistance is 21700 kN-m (16000 kip-ft) — an increase of 39%.

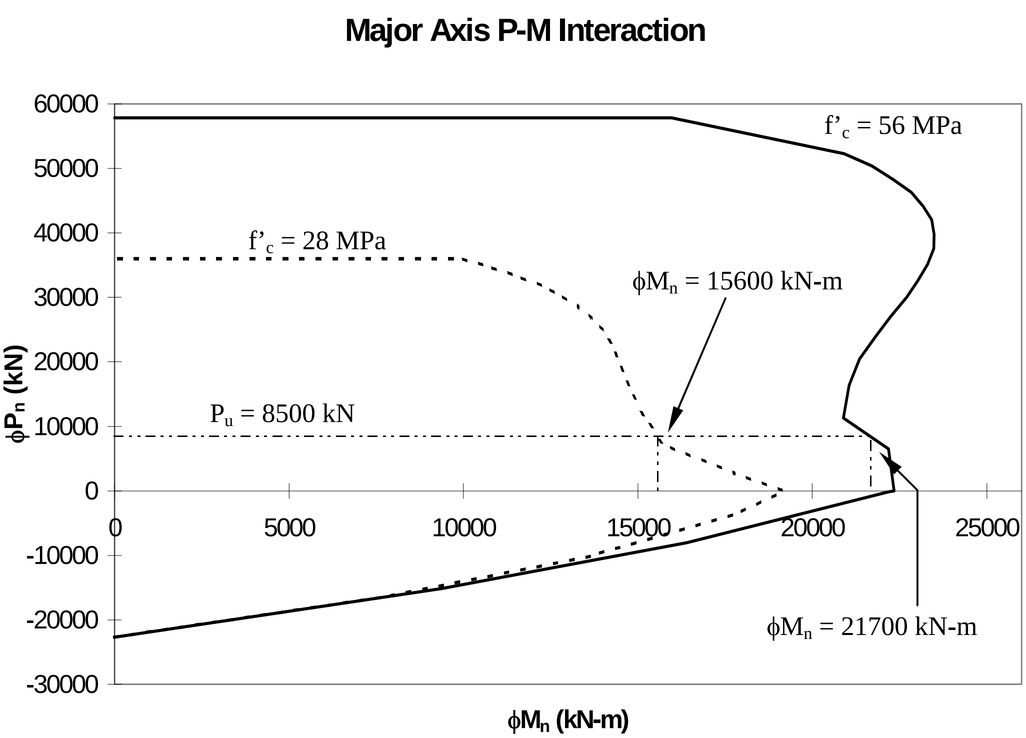


Figure 5.16: Interaction Curves for Major Axis Bending of Pier Section
(1 kN = 0.225 kips, 1 kN-m = 0.738 kip-ft)

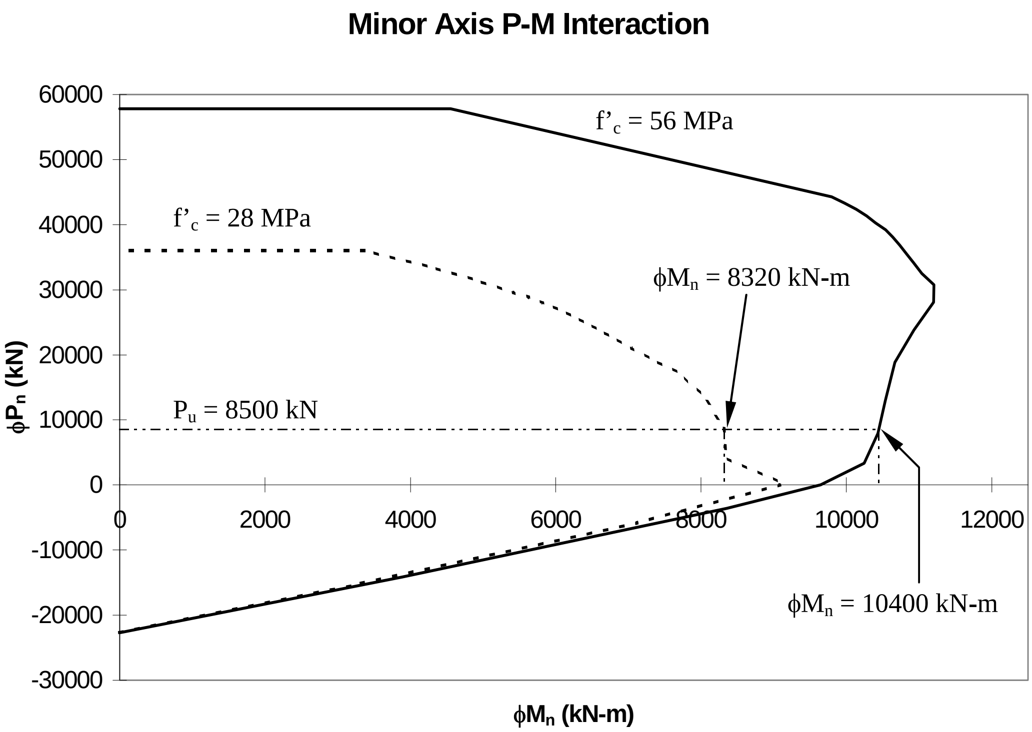


Figure 5.17: Interaction Curves for Minor Axis Bending of Pier Section
(1 kN = 0.225 kips, 1 kN-m = 0.738 kip-ft)

Subjected to minor axis bending (Figure 5.17), the section constructed of 28 MPa (4 ksi) concrete has a factored moment resistance of 8320 kN-m (6140 kip-ft). The 56 MPa section has a factored moment resistance of 10400 kN-m (7670 kip-ft) — a 25% increase.

From the interaction diagrams, it is apparent that for pure bending ($P_u = 0$) the increase in bending capacity achieved by increasing f'_c is slight (only 6% for minor axis bending). However, the relative increase in moment strength becomes quite significant under increasing axial loads. This indicates that the efficiency of using HPC to increase member capacity increases with axial load. Thus, the application of HPC to substructure design and construction should be more efficient than its application to superstructure construction. The use of hollow sections constructed of HPC allows each pier to support longer cap lengths than would be possible with normal strength sections with comparable quantities of concrete and reinforcing steel. This results in the use of fewer piers per bent; therefore, the transparency and order of the substructure space are increased.

5.3.1.4 Slenderness Effects

Local failures due to wall slenderness should not control the design of the proposed pier sections. The section capacity of these piers may be determined by application of the Whitney rectangular stress block approach so long as the wall slenderness ratio is less than 15.[4] This "wall slenderness ratio," X_u/t, is defined as the longest unsupported wall width divided by the wall thickness. For the pier sections proposed in this thesis, the maximum value of X_u/t is 8. Therefore, local instability of the compression flange need not be considered for these sections. If thinner sections are utilized, the designer should consult Reference 42 for guidance.

On the other hand, overall column slenderness effects should be considered in the design of all piers, particularly those used in tall single pier bents. Whenever possible, use of a refined second-order analysis to determine factored forces and moments is recommended. Use of such a method should allow the designer to take advantage of the enhanced stiffness of the pier due to prestressing. The use of "moment magnifiers" in accordance with ACI Code Sections 10.10, 10.11 and 10.12[5], AASHTO '94 Section 8.16.5.2, or AASHTO LRFD Section 5.7.4.3 provides a simpler but less accurate approach. The new values suggested in ACI Code Section 10.11.1 for the "smeared" or average moment of inertia under ultimate load conditions are based on *frame* tests and might be unconservative for a single pier bent. On the other hand, the older approximations for column stiffness set forth in ACI Code Section 10.12.3, AASHTO '94 Section 8.16.5.2.7, or AASHTO LRFD Section 5.7.4.3 should be conservative. Unfortunately, none of these various approximations employed in the ACI code are based upon tests of *prestressed* columns. Precompression of the concrete in prestressed columns results in a smaller extent of cracking throughout the column when one section fails. Thus, the value of the "smeared" column stiffness of a prestressed column will be greater than that of non-prestressed column with the same ultimate capacity. Because the current moment magnification method does not take this enhanced stiffness into account, a refined second-order analysis that includes the effects of prestressing is recommended.

Regardless of which method is used, inclusion of second-order effects will increase major axis bending moments by less than 10 percent for most practical cases. However, minor-axis bending moments are much more sensitive to second-order effects. Thus the choice of analysis method might affect the selection of the appropriate pier section.

5.3.1.5 Detailing

Guidelines for reinforcement detailing in hollow, post-tensioned concrete cross-sections were developed as a portion of previous research by Taylor, Rowell and Breen at the University of Texas at Austin.[6] The following recommendations are a result of that research.

1. Two layers of longitudinal reinforcement should be provided in each pier wall, one layer near each face of the wall.

2. A minimum of 1% longitudinal non-post-tensioned reinforcement should be provided. This recommendation is primarily aimed at reducing the effects of creep and shrinkage. (However, the advent of High Performance Concrete with its increased stiffness may allow a lower limit once a sufficient research database has been established. In fact, several large bridge structures have been supported on piers featuring reinforcement ratios lower than 1% without detrimental effects. A research project studying minimum reinforcement ratios for compression members is presently underway at the University of Texas at Austin. Obviously, if this limit can be safely reduced to a value such as 0.5%, great material quantity savings may be realized.)

3. Maximum lateral spacing of longitudinal reinforcement should be limited to 1.5 times the wall thickness or 450 mm (18 in), whichever is smaller.

4. Maximum longitudinal spacing of lateral reinforcement layers should be limited to 1.25 times the wall thickness or 300 mm (12 in), whichever is smaller.

5. 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 and connect 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.

6. Lap splicing of transverse bars should be avoided, if possible. Otherwise, lap splices should be enclosed by the hooks of cross ties.

7. Corner regions of segments should be well confined in order to enhance performance under biaxial bending.

8. Post-tensioning ducts should be grouted in order to promote integral action between post-tensioning bars and the concrete section.

5.3.2 Cap

Cap design is subject to the same general loads and load combinations that were discussed above for pier design. Critical sectional load effects can be determined through judicious placement of full vehicular lane load(s). Shear forces are much more likely to affect cap design than pier design, particularly in areas of low seismic risk such as Texas.

As discussed in Section 3.5, the inverted-T cap is the preferred style because of its enhanced aesthetic value. Most of the inverted-T cap resides within the depth occupied by the bridge superstructure. Because only the flange, or ledge, of the cap lies beneath the superstructure, the space defined by the substructure is more transparent and orderly. Although most of the structural considerations discussed in the following sections are applicable to all cap styles, they are focused on the inverted-T cap.

5.3.2.1 Dimensioning

Geometric constraints generally control most of the dimensions of an inverted-T style cap. Obviously, the length of the cap is closely related to the roadway width and the skew angle of the substructure. The height of the cap web, or stem, is dictated by the depth of the superstructure being supported. The thickness of the web must be large enough to enclose the longitudinal pier shaft reinforcement so that adequate moment capacity is developed in the connection between the template and the cap. The flange dimensions must be suitable to provide a bearing surface for the longitudinal beams and to transfer these loads to the web of the inverted-T.

Obviously, each individual cap in each bridge might be dimensioned in such a way as to minimize the material used. However, this would hardly be cost-efficient. Cap section dimensions should be chosen so that a variety of beam types, spans, and spacings may be supported with a minimum of changes. In this manner, repetition of production processes and standardization of details can be fully utilized to increase construction efficiency. Aesthetics may be improved by selecting dimensions that are in good proportion with one another, with the pier shaft and template below, and with the superstructure above.

5.3.2.2 Flexural Design

Flexural design involves designing the cap to satisfy both ultimate strength and serviceability requirements. First, each section must be designed such that the factored ultimate moment capacity, ϕM_n, is greater than the maximum moment, M_u, that can result from the factored load combinations given in the chosen AASHTO specification. These combinations are denoted as Strength I, Strength II, etc. in the AASHTO LRFD Code, which is recommended for use. Typically, the critical sections for negative, or hogging, moment will be located at the faces of the pier shafts or templates. Critical sections for positive, or sagging, moment will occur between the piers in caps supported by multiple piers. Varying the eccentricity of the prestressing force by utilizing curved or angled post-tensioning ducts will significantly reduce the amount of prestress required.

Amount and location of prestressing force must be determined according to serviceability requirements as well. Here again, as with pier shaft and template design, the Service I and Service III load combinations of the AASHTO LRFD Code should be used to check concrete compressive and tensile stresses, respectively. The segment joints will be the critical sections for these serviceability checks. For reasons explained above in Section 5.3.1.1, no tensile stress is allowed at the joint when match-cast, epoxy joints with no auxiliary reinforcement are subjected to the Service III load combination. Likewise, the compressive stress in the concrete is limited to $0.45f'_c$ when subjected to the Service I load combination.

In general, design for the service limit state will control the quantity and location of prestressing force required at the segment joints. Ultimate limit state design will control the selection of these parameters in other portions of the cap. The designer must be careful to keep both limit states in mind when tailoring the prestressing force.

In order to satisfy the zero tensile strength requirement at the segment joints, a large prestress force and eccentricity may be required. Allowable transfer stresses may be exceeded if this force is fully applied prior to placement of the superstructure dead load. In such a case, “staged” prestressing will be required. Only those tendons necessary to meet transfer requirements are stressed prior to placement of the superstructure. After the longitudinal beams have been placed on the cap, and the resulting dead load moments are within the desired range, the remaining tendons are stressed and grouted.

The cap must also be designed to perform adequately under construction loads. Individual segments must be reinforced in such a manner that they can be safely handled during the transportation and erection phases of construction. The designer should anticipate that support conditions will most probably change during the different construction phases. Segments should therefore be designed to withstand the load effects resulting from these various support conditions and construction loads.

5.3.2.3 Shear Design

Because the approximate cross-sectional dimensions of the inverted-T web are predetermined by the geometric constraints discussed above, effectively increasing the shear capacity of the cap can be done by

1. increasing the concrete compressive strength, f'_c,
2. adjusting the magnitude and the inclination of the prestress force, and
3. increasing the amount of transverse reinforcement, A_v.

Increasing f'_c alone is generally inefficient, because the shear capacity of the concrete is modeled as being proportional to the square root of f'_c. This method is probably only cost-efficient when a slight increase in shear capacity is required. The second method, adjusting the resultant prestress force, can be quite effective in directly counteracting the shear force. However, the tendon profile is often set by flexural design requirements, and this method lacks a certain degree of flexibility. Increasing A_v is also quite effective although its efficiency is limited due to increased steel quantities and difficult segment fabrication due to steel congestion. In addition, only a finite increase in shear capacity can be obtained before failure is controlled by f'_c. The most effective solution is often a combination of all three of the methods described.

The AASHTO LRFD method (Section 5.8.3) of calculating the concrete contribution, V_c, to the ultimate shear capacity is a relatively complicated kinematic method based on estimating the strain in the reinforcement on the flexural tension side of the member, ε_x. The "K" method specified by the AASHTO Segmental Code (Section 12.2.12) is much more straightforward and user-friendly while returning a safe estimate of the sectional capacity. Either of these methods is adequate for cap design.

Multiple shear keys should be utilized to transfer shear across segment joints. These keys should be detailed according to the AASHTO LRFD Code, Section 5.14.2.4.2.

5.3.2.4 Flange Design and Hanger Reinforcement

The use of inverted-T caps presents a few design issues which are not present in situations where longitudinal stringers are placed on top of rectangular caps. The inverted-T cap must be reinforced such that the stringer reaction forces are transferred through the cap flange into the bottom of the web and thence up into the upper portion of the web. This is necessary to ensure that the full T-section is effective in carrying the applied loads to the pier supports. There are six possible failure modes that must be considered:

1. flexure of the overall inverted-T cap,
2. flexural shear acting on the overall cap,
3. torsional shear on the overall cross section,
4. hanger tension on web stirrups,
5. flange punching shear at stringer bearings, and
6. bracket-type shear friction in the flange at the face of the web.[7]

In addition to these ultimate failure modes, a service limit state failure involving wide cracks at the web/flange interface due to local stirrup yielding must be obviated. Design formulae for all of these possible failure modes are explained in Reference 45.

5.3.2.5 Anchorage Zones

Anchorage zones should be designed with the guidance of AASHTO LRFD Section 5.10.9. Because anchorage zones typically feature complex, non-linear strain distributions, plasticity-based concepts are useful for design of these regions. Transparent strut and tie models (STM's) which satisfy equilibrium requirements provide a lower-bound design approach that is both safe and rational. Special attention must be directed towards adequately detailing the STM node regions so that the member will perform as modeled. Also, reinforcement that controls the behavior of the anchorage zone under service-level stresses must be included. An example of such behavior is the cracking of the concrete around anchorage plates due to deformation compatibility.

5.3.2.6 Concrete Strength

Unlike the case of the pier shafts, the eccentricity of the cap prestressing force can be varied along its length in order to balance the applied load effects. Thus, the extreme fiber compressive stresses under service limit state conditions are expected to be less critical than those in the pier. Concrete compressive strengths (f'_c) corresponding to approximately 42 MPa (6 ksi) should be adequate for most cap designs. Occasionally, higher strengths might be desired. In general, the required concrete compressive strength will be controlled by the concrete compressive stresses under service loads or at time of prestressing, or by the necessary concrete contribution to overall shear capacity (V_c). Anchorage zone requirements might also dictate the use of a higher concrete strength.

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3. American Association of State Highway and Transportation Officials (AASHTO), *AASHTO LRFD Bridge Design Specifications: Customary U.S. Units,* 1st ed., AASHTO, Washington, D.C., 1994.
4. Taylor, A.W., Rowell, R.B., and Breen, J.E., "Behavior of Thin-Walled Concrete Box Piers," *ACI Structural Journal*, V. 92, No. 3, May-June 1995, pp. 319-333.
5. American Concrete Institute (ACI) Committee 318, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-95 and ACI 318R-95, ACI, Detroit, 1995.
6. Taylor, A.W., "Behavior of Thin-Walled Hollow Concrete Bridge Piers and Pylons," PhD dissertation, Department of Civil Engineering, University of Texas, Austin, December 1990.
7. Mirza, S.A., and Furlong, R.W., "Design of Reinforced and Prestressed Concrete Inverted T Beams for Bridge Structures," *PCI Journal*, V. 30, No.4, July-August 1985, pp. 112-136.

CHAPTER SIX
PROPOSED SYSTEM

6.1 Introduction

This chapter outlines the proposed substructure system. Section 6.2 pertains to the selection of the overall system type. Section 6.3 details the individual system components. The development of component properties is described, and choices that must be made by the designer in order to select these components are discussed. The fabrication process for each system component is also described. The overall erection scheme was illustrated previously in Section 5.2.3. Plans for typical substructure elements are included in the Appendix.

6.2 Overall System

The proposed substructure system consists primarily of precast concrete elements. These elements are jointed in a match-cast, segmental fashion, except for a few cast-in-place joints where field geometry control is critical. The entire system is progressively post-tensioned during erection to produce a prestressed concrete substructure. Threaded post-tensioning bars form the primary reinforcing system for the vertical pier shaft portions of a system bent, while multi-strand post-tensioning tendons form the primary longitudinal reinforcement system for the horizontal cap portions of a bent. The system was developed in round SI units in anticipation of the forecasted adoption of this system of units by FHWA and TxDOT. However, the dimensions can be easily converted to round English units with little difficulty, if so desired.

6.3 System Components

6.3.1 Foundation

Although the precast portion of the substructure system exists above the foundation level, the foundation must be constructed in such a way as to allow the system erection. The foundation for each substructure bent should consist of a cast-in-place cap that encloses the tops of either drilled shafts or driven piles. In addition to transferring the substructure forces to the shafts or piles, the foundation cap must also serve to anchor the post-tensioning bars that serve as the flexural reinforcement for the pier shaft. Thus, the cap must consist of properly detailed anchorage zone reinforcement, and the initial post-tensioning bars must be accurately located within an acceptable tolerance in the field. These bar locations should be explicitly laid out on the substructure plans.

6.3.2 Foundation-to-Pier Joint

The joint connecting the initial segment of each pier to the foundation cap should be cast in place to ensure the proper field alignment of the entire shaft. This cast-in-place joint is also the first of two opportunities to adjust the pier height from that provided by an integral number of pier segment heights to the actual exact pier height desired. For example, a base joint thickness of 200 mm (8 in) might be specified to form a pier with a total height of 5.0 m (16.4 ft) with two 2.4 m- (7.87 ft-) tall segments. There are two main options in the planning of the foundation-to-pier joint.

The first option consists of forming a recess in the top of the foundation cap slightly larger than the plan dimensions of the pier segment and slightly deeper than the planned joint thickness. The first pier section is then supported at the desired height (200 mm in this case) above the base of the recess and properly aligned. Post-tensioning ducts are then spliced from the pier segment to the foundation, and mild steel joint reinforcement is placed. Once the proper alignment is verified, the recess is filled with concrete, locking the first segment in the proper alignment. A similar system was used successfully on the Chesapeake and Delaware Canal Bridge described in Section 2.3.6. One disadvantage to the implementation of this option is the fact that foundation elevations must be chosen to facilitate the placement of the proper size recess. The foundation cap must also extend to a greater depth in order to provide the necessary anchorage for the post-tensioning bars. Because difficult excavation conditions exist very close to the surface throughout much of Texas, the required degree of flexibility in locating foundations will probably result in prohibitive costs. Thus, a second option would be more desirable.

The second, more desirable option, is akin to the first except that the initial pier segment is supported above the foundation cap by a cast-in-place “pedestal” rather than in a recess within the cap. This allows the foundation cap to be freely located at an elevation where conditions and costs warrant. This system has been utilized successfully on the U.S. 183 Viaduct described in Section 2.3.9. The pier segment is properly aligned and temporarily supported on framework above the foundation. Post-tensioning ducts are then spliced as shown in Figure 2.37. Figure 2.38 illustrates the segment locked into position after the pedestal is cast.

6.3.3 Pier Shaft

The pier shaft consists of match-cast, segmental pier segments that are epoxy-jointed and post-tensioned together with threaded post-tensioning bars.

6.3.3.1 Segment Properties

Overall dimensions of the pier segment cross-sections were chosen so that the pier shafts would behave adequately under a range of load conditions imposed by girder-type superstructures characterized by various span lengths and roadway widths. A set of cross-sections was developed that would allow use of the system under a wide variety of conditions with a minimum number of cross-sections. A key factor in the selection of these dimensions was the aesthetic goal expressed in Section 3.5 that the ratio of superstructure width to shaft breadth for single pier bents range from 3.5 to 5. Pier sections were chosen such that a full practical range of superstructures could be supported by only four cross-sections. Dimensions for these four cross-sections are shown in Figure A.2 in the Appendix. The four sections share a common width of 1200 mm (47.25 in). There are four section breadths: 2000 mm (78.75 in), 2400 mm (94.5 in), 2800 mm (110.25 in) and 3600 mm (141.75 in). These four sections are designated as P20, P24, P28 and P36, respectively. The four sections can be thought of as extensions of the same shape with the only variable being the long dimension. Plans for a typical pier segment are shown in Figures A.4 through A.8.

The sections have several interesting shape features which bear discussion. First, the sections are hollow. This enhances the structural efficiency of the section by maximizing r^2 as discussed in Section 5.3.1.2. By removing unnecessary material, dead loads on the foundation are reduced. Segment weights are reduced, resulting in lower costs associated with transportation and handling. The central void also presents a means of locating drainage pipes without reducing structural efficiency.

Second, the sections are chamfered. These chamfers enhance the verticality of the pier and help to better express the flow of forces from the cap to the ground. Figure 3.7 illustrates the successful use of this technique on cast-in-place piers. When a bent must be placed on a skew, the angles presented by these chamfers help to resolve the conflict between the skewed directions of the superstructure and the feature that is being spanned. The chamfer angles also correspond to the angular nature of the I-beam, U-beam and box beam elevations. Chamfer dimensions are kept constant for all four pier sections. This promotes visual uniformity between different section sizes used in the same structure and maximizes the reuse of form parts.

A third interesting shape feature provides the designer with the opportunity to specify insets in each of the four main vertical faces of the pier. These insets would not require the use of different outer forms, but would be formed with the use of form liners or inserts. These insets also help to express the vertical nature of the pier shaft's structural function. If designed as smooth insets, they can reflect the web recesses of the pretensioned I-beam. Otherwise, form liners can be utilized in these spaces to give the pier a fine- or medium-scale texture that can express either structural function or a motif indicative of the bridge's location as discussed in Section 3.6.1.

As can be seen in the plans for a typical pier segment, there is a standard shear key size. Thus, the same size inserts can be used to form all of the pier shear keys. These keys should provide the necessary joint shear strength for epoxied joints under the full range of typical loading conditions.[1] The keys occupy the same relative position in all of the four pier segment cross-sections. Thus, the positioning and forming of these keys during segment fabrication should be a simple and repetitive process.

Potential locations for post-tensioning bars are also standardized. Bar locations are spaced on a 200 mm- (7.87 in-) grid analogous to the standard 50 mm- (2 in-) grid used for locating prestressing strands in pretensioned girders. Once again, the repetition resulting from the use of this standard layout increases the simplicity with which ducts are placed when fabricating the segments. Chances of placement error are minimal when the same standard layout is specified time after time.

The three standard segment heights are illustrated in Figure A.3. Segment heights were chosen such that transportation and handling would be relatively simple. For example, heights were limited so that transportation clearances would not be critical. In addition, segment heights were selected so that one truck could transport the weight of at least two typical segments. In order to eliminate the visual nuisance created by horizontal form liner joints, segments were sized to have a height less than or equal to the length of typical form liner panels. A 1200 mm- (47.25 in) aesthetic module was selected to size the pier segments. The full-height segment is 2400 mm (94.5 in) tall and represents two of these aesthetic modules. The false joint at the midheight of the segment delineates the two aesthetic modules that are incorporated in each full-height segment. A standard half-height segment is 1200 mm (47.25 in) tall and corresponds to one aesthetic module. Finally, the quarter-height segment is 600 mm (23.62 in) tall and represents one-half of a module. The quarter-height segment is used at the top or bottom of a pier shaft to adjust the height of the shaft to within 600 mm of the desired overall height. Thicknesses of the cast-in-place joints at the top and bottom of each pier shaft can be specified to adjust the pier height to the final desired value. Segment heights are denoted by the placement of a suffix on the cross-section type. Lack of a suffix indicates that the segment is a full-height (2400 mm-tall) segment. Half-height segments are denoted with an “H” suffix; quarter-height segments with a “Q.” For example, P28H denotes a segment with a P28 cross-section and a height of 1200 mm.

6.3.3.2 Pier Design Process

When designing the pier shafts for a bridge project, the designer should first consider the overall layout and appearance of the bridge. Roadway width and elevation should be used to determine what type of bents will be used at specific locations. Conceivably, roadway widths approaching 18 m (59 ft) might be supported on single pier bents; however, the height of the roadway will also influence this decision. For example, an 18 m-wide roadway might look awkward perched atop a single pier 5 m (16.4 ft) above the ground. On the other hand, the same pier and roadway combination might look quite slender and elegant if the pier is 18 m tall. The appearance of the structure as a whole must be considered when choosing pier dimensions.

The visual flow of the substructure must match the superstructure flow. The designer should take into account how the roadway width increases and decreases in addition to the location of on- and off-ramps. The changing height of the structure should also be considered. Suitable pier cross-sections should be chosen for each bent so that the flow of the substructure is smooth and in harmony with the flow of the complete structure and its environment.

A simple rule of thumb can be used to choose preliminary pier section sizes. The ratio of supported roadway width (including the effects of the skew angle) to the breadth of the pier section or sections supporting the roadway should be between 3.5 and 5. Thus, the P24 pier section (with a breadth of 2400 mm) might be used to support roadways ranging from 8.4 m to 12 m (28 ft to 39 ft) wide (including skew effects). A bent supported by *two* P24 pier shafts might support roadways twice as wide.

The vertical layout of the pier segments will be critical to the visual appearance of the design. The designer has a limited amount of flexibility in locating the apparent segment joints along the height of the shaft. Although the height of the pier (and thus the quantity and height of the individual segments needed to form the pier) is determined by roadway geometry, the designer can control the location of these joints by two means. First, the relative height of the cast-in-place foundation-to-pier joint and pier-to-template joint can be adjusted. Second, if a 600 mm-tall segment (such as P28Q) is required, it may be placed at either the top or the bottom of the pier shaft. By these means, the designer can make sure that the articulated joint locations flow rhythmically from pier to pier along the length of the bridge.

Once a preliminary standard section size has been chosen, the designer must ensure that the section provides the required safety and serviceability. The first step in this process is to choose the required amount of effective prestress force based on the Service Limit State and construction load stress limitations discussed in Sections 5.3.1.1 and 5.3.1.2. After the required number of post-tensioning bars has been selected to meet these criteria, the designer should check the pier's capacity to withstand the loads imposed by Ultimate Limit State load combinations. Additional bars might have to be added to provide the required capacity. These extra bars do not necessarily have to be prestressed, but they should be grouted after installation. It is essential that a strain compatibility analysis be performed to ascertain the stress in these bars at the section's ultimate capacity. Under no circumstances should it be assumed that the ultimate strength of these bars will be fully developed under ultimate loads.

Once the primary reinforcement configuration has been established, the necessary concrete strength should be determined in accordance with service, ultimate and construction load criteria. Choosing the reinforcement layout and the concrete strength may be an iterative process at first. However, with a little practice, the designer will usually be able to select an adequate concrete strength at the beginning of the process. Once a suitable reinforcement layout is chosen the adequacy of the selected concrete strength should be verified.

The standard reinforcement details provided in the typical P28 segment design included in the Appendix should be adequate for any non-seismic load combination to which the pier will be subjected. These details were designed to meet the recommendations discussed in Section 5.3.1.5 with one exception. The longitudinal mild steel provided is less than the 1% minimum recommended. This was done for several reasons. First, smaller ratios of longitudinal mild steel have been used in practice before (as recently as the U.S. 183 Viaduct). Second, a lowering of the recommended minimum is anticipated due to ongoing research of this subject. Finally, the grouted prestressing bars have the capacity to work identically to conventional mild steel reinforcement after they have been stressed and grouted. Their presence should be counted towards satisfying the 1% minimum requirement. Regardless, if a minimum of 1% reinforcement is desired, the specified longitudinal bar sizes may be increased by one standard bar size.

Shear strength of the pier should also be checked. Again, the standard details provided should be more than adequate for any non-seismic load conditions.

6.3.3.3 Fabrication of Pier Segments

Fabrication of the match-cast segmental pier segments should be rather straightforward. Casting of these segments will be very similar to the production of the pier segments for the U.S. 183 Viaduct described in detail in Section 2.3.9. At most, 12 different sets of forms are required to produce all of the standard cross-sections and heights in the system. Standard reinforcement cages can be prefabricated without regard to the actual layout of post-tensioning ducts to be placed in each individual segment. One major difference between the proposed system and that used on the U.S. 183 Viaduct is the location of the drainage system. In the U.S. 183 segments, a PVC drain pipe was installed within one wall of each segment. Aside from reducing the structural efficiency of the segment, the presence of this large void in the concrete resulted in cracking of the concrete around the pipe soon after placement. Thus, in the proposed system, it is recommended that drainage pipes be located within the central void. At the base of the pier, an elbow pipe section can be used to drain the water to the exterior (see Figures 2.37 and 2.38). This elbow would pass through the cast-in-place joint at the base of the pier.

Unlike the U.S. 183 piers, the primary longitudinal reinforcement of the proposed pier system consists entirely of threaded post-tensioning bars. For each of the four standard cross-sections, a simple jig could be used to repeatedly and accurately locate the post-tensioning ducts in the proper position prior to casting. Potential locations remain constant, so no variation of the jig would be necessary from pier to pier or project to project.

The match casting process is auto-correcting; thus, undue labor and time need not be expended on precisely locating every duct or shear key. Because each segment is cast against its predecessor, shear keys and post-tensioning ducts will necessarily match those in the adjacent segments.

6.3.4 Pier-to-Template Joint

The cast-in-place joint connecting the top pier segment to the template is analogous to the cast-in-place joint at the base of the pier. Both joints are used to adjust the overall height of the pier and ensure the proper alignment of the segment above the joint. In the case of the joint connecting the pier and the template, the joint’s purpose is to properly align the template segment, thus easing the placement of the cap with the specified cross-slope.

As explained in Section 5.2.3.2, the template is temporarily supported at the desired height above the final pier segment. Post-tensioning ducts are spliced from the pier to the template, and any necessary joint reinforcement is placed. Finally, the template is positioned exactly as desired and locked into position with a cast-in-place joint. Every effort should be taken to ensure that the cast-in-place joint matches the adjacent pier and template segments.

6.3.5 Template

Section 5.2.3.2 includes a discussion of the background and function of the template segment. As shown in Figure A.9, there are four standard template segments, designated T20, T24, T28, and T36, each corresponding to one of the four standard pier cross-sections. Standard plans for the T28 template section are included as Figures A.10 through A.14. The overall shape of the template section was chosen to express the flow of the forces from the inverted-T cap to the chamfered pier shaft. Forms for the template segment should allow the top surface to be finished at one of three standard slopes: 0, 20, and 40 mm/m. These three standard slopes allow the cap to be aligned for a range of cross-slopes. Cross-slope values within 10 mm/m of these standard values are obtained by varying the heights of the cast-in-place girder bearing seats or elastomeric bearing pads on the inverted-T flange.

Design of the template generally consists of choosing the standard template segment that corresponds to the pier cross-section and specifying the cross-slope of the template surface. The pier reinforcement is then continued up through the template into the cap. If the standard pier sections below the template are structurally adequate, the standard template section should be adequate as well, provided that the concrete strength matches that of the pier segments.

It is not necessary to match-cast the template segment against a pier segment because the template is supported by a cast-in-place joint in the final structure. However, the contractor may wish to form the template section against a pier section in order to more easily align the post-tensioning ducts.

6.3.6 Inverted-T Cap

6.3.6.1 Cap Properties

The inverted-T style bent cap was chosen for the aesthetic reasons discussed in Section 3.5. Plans for a typical segmental cap for a bent supported by a single pier are included in the Appendix as Figures A.15 through A.22. The cap is divided into two types of segments: *primary* segments, which are supported directly by pier shafts, and *secondary* segments, which are erected in balanced cantilever fashion from the primary segments. Each pier’s post-tensioning bars are anchored within the corresponding primary segment.

Several of the cross-sectional dimensions of the inverted-T cap are somewhat controlled by geometric constraints as explained in Section 5.3.2.1. The height of the cap web, or stem, is controlled by the depth of the stringers being supported. The stem height of 1500 mm (59.1 in) shown in Figure A.18 would be chosen to support a AASHTO Type IV beam or a Texas U-beam. If a Texas Type C beam (AASHTO Type III) were to be supported, a stem height of 1200 mm (47.2 in) would be specified. The thickness of the stem must be adequate to accommodate and anchor the post-tensioning bars extending from the pier shaft below. A thickness of 1200 mm (47.2 in) was chosen to match the nominal width of the standard pier shaft sections. Thus, only two standard cross-sections are necessary to form inverted-T caps for the vast preponderance of concrete girder bridges with span lengths ranging from 20 m (65 ft) to 42 m (138 ft). Should the continuing development of high performance concrete allow the cost-efficient use of Type IV I-beams and Texas U-beams for span lengths approaching 46 m (151 ft) or more, the standard cap stem height of 1500 mm (59.1 in) should still suffice. Deeper cap sections will be required only if the stringer depth exceeds 1350 mm (54 in). A stem width greater than the standard 1200 mm (47.2 in) will only be necessary if the width of the pier shaft is increased beyond the standard 1200 mm (47.2 in). This should only occur for structures that are both wide and very tall. If such an increase in pier size is required, a single standard pier and stem width of 1600 mm (63.0 in) should be adequate for even the most extreme case.

Each ledge of the inverted-T extends 600 mm (23.6 in) out from the stem. This distance was chosen both to provide an adequate seating area for the longitudinal stringers and to be in good proportion to the height of the stem, whether the stem height is 1500 mm or 1200 mm. The soffit of the inverted-T flange is tapered with increasing distance from the supporting pier shaft. This taper effectively expresses the force flow through the cantilever arms of the cap by approximating the shape of the moment diagram. The flange tapers from a total thickness of 1200 mm (47.2 in) at the intersection with the template to a minimum thickness of 600 mm (23.6 in) at the ends of the cap. In addition to the taper of the cap soffit, the cap cross-section is chamfered throughout. This chamfer expresses the force flow from the stringer reactions through the cap ledges and thence into the pier. The chamfer has a 1:1 slope with equal edge distances of 600 mm (23.6 in).

The necessary cap length is determined by the roadway width, the skew angle, and the allowable slab overhang. The slab overhang is usually around 300 mm (1 ft), which roughly corresponds to the nominal thickness of the traffic barrier above. Thus, the overall length of the cap can be estimated as the roadway width divided by the cosine of the skew angle, α (see Section

2.2.1.5). Cap lengths for single pier bents would range from 7.6 m (24.9 ft) to 18 m (59.1 ft) in standard 400 mm (15.75 in) increments. Typical ranges of supported roadway widths (magnified by skew effects) for each type of bent configuration are shown in Figure 6.1.

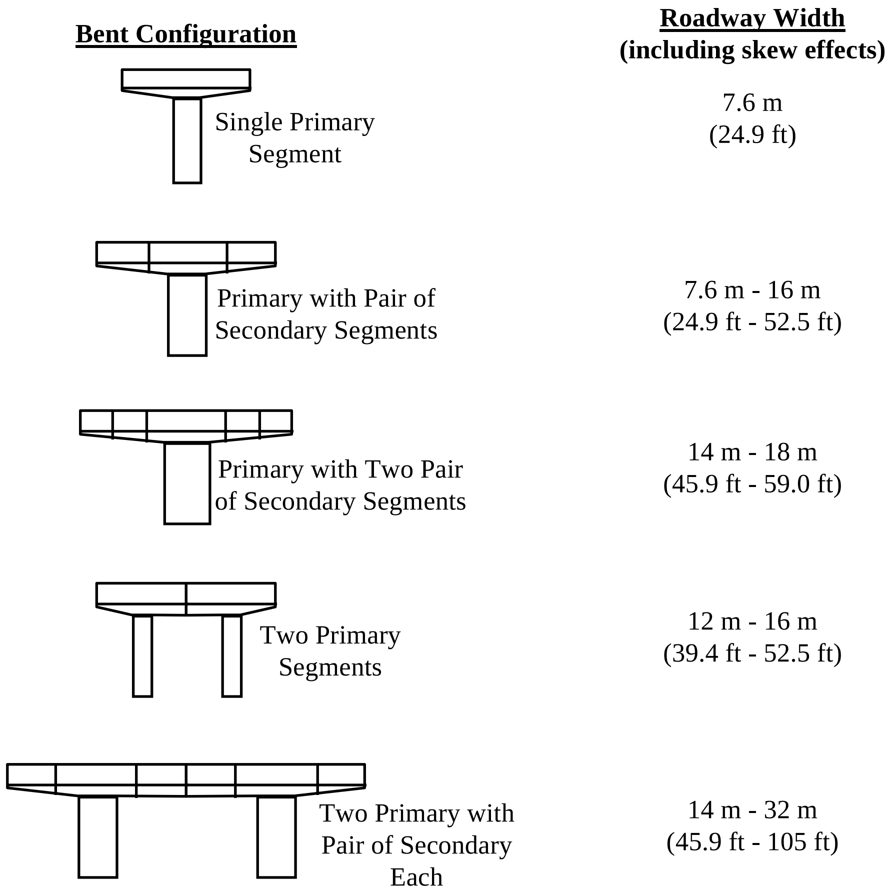

Figure 6.1: Widths of Supported Roadway for Various Bent Configurations

The primary cap segment is reinforced with bar tendons that extend from the pier shaft and with multi-strand tendons that serve as the longitudinal reinforcement for the cap. The bar tendons are anchored at the top of this segment and provide the continuous connection with the pier and template assembly. Thus, the primary segment must have a properly detailed anchorage zone for these bars. Also, because these bars extend straight up from the pier shaft, the ducts will have to be properly aligned prior to casting to allow for the cross-slope of the cap.

Both the primary and secondary segments are reinforced with longitudinal, multi-strand post-tensioning tendons. Potential ducts for these tendons can be located in two columns within the 1200 mm- (47.25 in-) width of the cap web. One, two or three layers, each containing two tendons, may be installed, giving the designer the possibility of using up to six tendons. The depth of the ducts within the cap will likely be varied in order to maximum the efficiency of the applied prestressing force.

6.3.6.2 Cap Design

Three quantities are necessary to specify the overall cap dimensions for a single pier bent. First, the depth of the supported stringers must be known in order to specify which stem height (1200 mm or 1500 mm) is needed. Second, the supporting template section is necessary to determine the length of the untapered cap soffit directly above the template. Third, the overall cap length is necessary to determine the required length of the cap cantilever arms on either side of the pier shaft.

Once the overall cap dimensions have been chosen, the designer must choose where to locate the joints between segments. Allowable stress limits under Service Limit State load combinations generally control the quantity and location of the longitudinal multi-strand tendons. Thus, location of the joints is critical to efficient cap design. Joints should generally be located as close as possible to regions of minimum moment. For the case of the single pier bent, the joints should be located as far as possible from the pier-template assembly. In other words, the primary segment should be made as large as handling and transportation considerations will allow. Beyond a point, increases in segment size will produce transportation costs that eclipse the savings in reinforcing steel.

Once the cap joint locations have been determined, the designer must assume a preliminary value of f'_c and then layout and design the anchorage zone for the bar tendons extending from the pier shaft. The standard reinforcement details shown in Figures A.16 through A.18 should be adequate for most situations; however, they should be checked. Alternate standard details might also be developed which provide the required performance with lower production costs. If necessary, f'_c might be adjusted at this stage.

Flange and hanger reinforcement details should be checked for adequate capacity against the failure modes described in Section 5.3.2.4. Here again, reinforcement details and/or f'_c might be modified.

Once these details have been checked, the cap should be designed for flexure of the overall member. The size, amount and location of multi-strand tendons must be determined by considering ultimate, service and construction load combinations. Whether or not staged prestressing will be necessary may also be determined at this time. After the post-tensioning reinforcement has been laid out, the shear capacity of the overall cap should be checked. Slight modifications of the tendon layout, f'_c, and/or the mild steel reinforcement details might be necessary at this stage.

Anchorage zones for the multi-strand tendons must be designed at the ends of the corresponding cap segments. Modifications of the reinforcement or f'_c may be necessary.

Finally, cast-in-place bearing seats should be located at the stringer reaction points. The nominal height of each bearing seat that is required to result in the proper superstructure cross-slope should be specified. These nominal heights may be adjusted after the cap has been erected in order to make up for slight alignment errors.

6.3.6.3 Fabrication of Cap Segments

As discussed in Section 5.2.1, cap segments should be cast in a horizontal, long-line manner. One precasting bed is used to form an entire cap assembly. As shown in Figure 6.2, the cap forms consist of three types of form elements. The first type (labeled “A”) includes those elements used to form the stem of the inverted-T. These interchangeable elements form the vertical surfaces of the web and the top surface of the flange. Two different sizes can be used to form stem heights of either 1200 mm (47.25 in) or 1500 mm (59.1 in). Different height forms may be produced for other cap sizes if necessary. The second element assembly is used to form the outer flange shell of the inverted-T. Labeled “B” in Figure 6.2, this outer flange shell encompasses the full dimensions of the flange at its deepest section — the template-cap interface. Therefore, this shell has a width of 2400 mm (94.5 in), a depth of 1200 mm (47.25 in), and 600 mm- (23.62 in-) square chamfers along its bottom surface. The cross-section of this assembly is typically constant for all of the caps to be used. The third set of elements is used to form the tapering soffit of the bent. Labeled “C” in Figure 6.2, these standardized stiffened plate elements are inserted into the flange shell forms in order to achieve the proper taper for the bent arms. In addition to the three groups of form parts shown in Figure 6.2, end forms are required to form the non-match-cast ends of each segment.

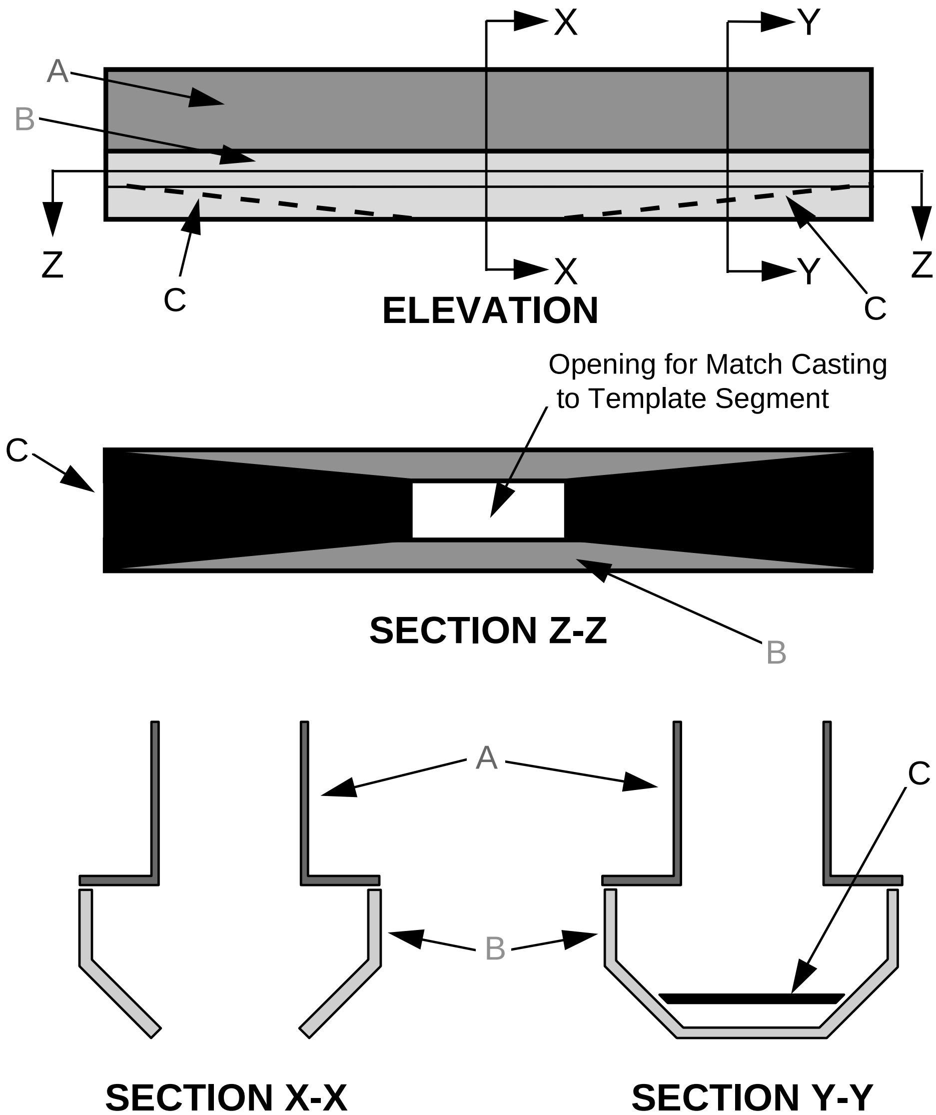

Figure 6.2: Forming System Required for Long-Line Casting of Cap Segments. Stem forms are labeled *A*. *B* represents outer flange shell. Soffit taper inserts are marked *C*.

Cap fabrication begins with the primary cap segment. The appropriate template segment is coated with a bond-breaking coating and placed in the proper position in the flange shell form. The template is aligned to result in the specified cross-slope of the cap when the bent is erected in the field. The correct soffit forms are then placed within the flange shell forms. Given the range of the four pier sections and potential cap lengths, cap cantilever arms must range in length from

2.8 m to 7.2 m (9.2 ft to 23.6 ft). Thus, with an increment of 400 mm (15.75 in), twelve standard soffit forms could produce all necessary cantilever arm lengths. Figure 6.3 and Table 6.1 show the soffit form dimensions required.

L_C (mm)	L_P (mm)
2800	2864
3200	3256
3600	3650
4000	4045
4400	4441

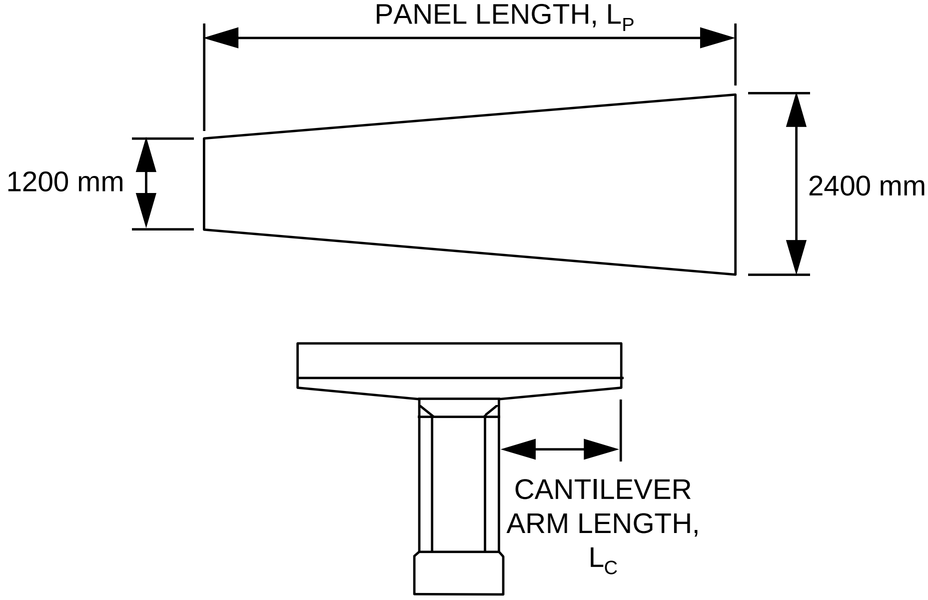

Figure 6.3: Soffit Form Insert (1 in = 25.4 mm)

L_C (mm)	L_P (mm)
4800	4837
5200	5235
5600	5632
6000	6030
6400	6428
6800	6826
7200	7225

Table 6.1: Length, L_C, of Soffit Insert Panel Required for Standard Cantilever Arm Lengths, L_P (1 in = 25.4 mm)

After the soffit forms are in place within the flange shell forms, the reinforcement cage for the primary segment is fabricated within the form, or a prefabricated cage is lowered into the form. Ducts for the pier shaft post-tensioning bars are placed within the cage and properly aligned with the corresponding ducts in the template. Drainage fixtures are installed, along with blockouts for the post-tensioning anchorages, at the top of the primary segment. Once the internal reinforcement and fixtures are in place, the stem forms are attached. The joint forms are positioned as required for the specified segment length. Joint forms are detailed to provide the necessary shear key configuration. The final alignment of the template-form assembly is verified and adjusted if necessary, and the concrete is cast.

Once the primary segment has been cast, the secondary segments can be fabricated in a similar manner. The joint surfaces of the primary segment are coated with a bond-breaker and serve as forms for the joint surfaces for the secondary segments. Once the entire cap assembly is complete, the segments can be labeled and removed for storage until they are transported to the job site and erected as discussed in Section 5.2.3.

1. Kosecki, K., and Breen, J.E., “Exploratory Study of Shear Strength of Joints for Precast Segmental Bridges,” Research Report 248-1, Center for Transportation Research, Austin, September 1983, pp. 88-89.

CHAPTER SEVEN
IMPACT

7.1 Introduction

This chapter consists of a discussion of the overall impact of the proposed substructure system. First, the aesthetic impact of the proposed system is considered. The chapter concludes with a discussion of the expected economic impact associated with implementation of the system.

7.2 Aesthetic Impact

7.2.1 Overall Structural Form

As discussed in Section 3.5, the most visually appealing overall form of any structure is one which clearly expresses efficient structural function. For bridge substructures, efficient structural function is indicated by the transparency of the space beneath the superstructure and the orderliness of the elements occupying that space. Transparency and order would both be enhanced with the use of the proposed system. The increased use of single pier bents is emphasized in the system. Single pier bents may be used for roadway widths approaching 18m (59.1 ft). Therefore, these bents may be used to support one-, two- or even three-lane roadways. These bents greatly reduce the visual clutter beneath a bridge, which results in greater transparency. Single pier bents are also quite valuable because they express an orderly substructure flow that matches that of the supported superstructure. In addition, a non-skewed single pier bent might be used where a multiple pier bent would require a visually disturbing skew angle.

Multiple pier bents constructed by means of the proposed system will also promote transparency. Roadways of up to five lanes can be supported by dual P28 pier sections. Only very wide roadways or severe skew angles would require the use of three piers per bent. Use of the proposed system would result in a marked improvement in transparency and order over the multiple column bents (often supported by as many as five or six columns) used widely at present.

The transparency of the space beneath the superstructure is also enhanced by the inverted-T style cap of the proposed system. Most of the inverted-T cap occupies space within the depth of the superstructure. Thus, less of the cap intrudes on the space beneath the superstructure, providing more transparency of this space. This is especially true when the bridge is viewed from

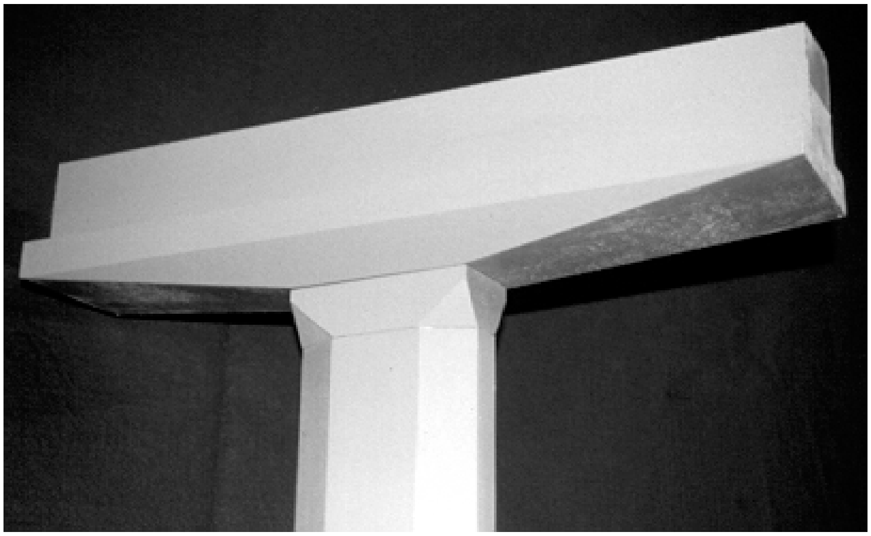

Figure 7.1: Template Segment Expresses Flow of Forces from Cap to Pier

oblique angles. As shown in Figure 7.1, the template segment visually expresses the flow of forces from the inverted-T cap to the pier shaft.

The proposed system also reaps the aesthetic benefits of repetition. All pier sections have the same width (1200 mm [47.25 in]), and there are four standard breadths from which to choose. Cap dimensions are relatively constant from bent to bent, as long as the supported stringers are of the same depth. Such repetition enhances the inherent rhythm of the superstructure and therefore increases the orderliness of the entire bridge structure.

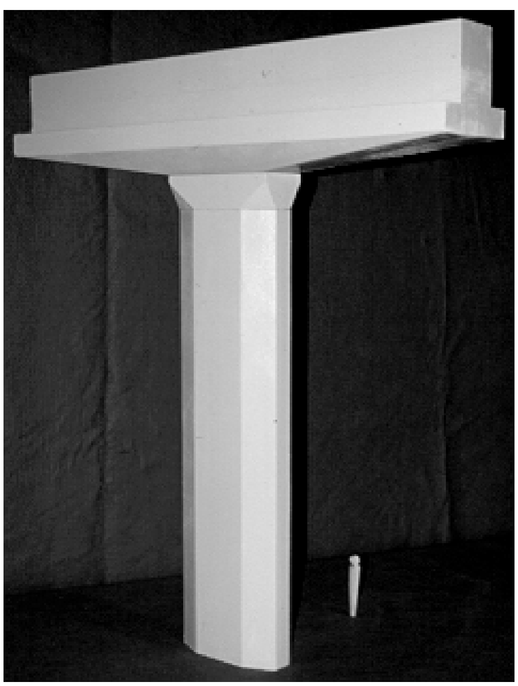

Figure 7.2: Model of Single Pier Bent

As can be seen in Figure 7.2, the system is well proportioned. The defining dimensions of the system elements consist of multiples of simple unit modules. For example, pier shaft dimensions are based upon a 400 mm (15.75 in) module. Pier segment widths, breadths and heights are multiples of this module. The system bent cap cross-sections are dimensioned in accordance with a 300 mm (11.8 in) module. This 300 mm module can also be found in the pier shaft chamfer dimensions. The incorporation of this 300 mm module in the pier serves to increase the harmony between the various system elements and to better express the continued flow of forces from the cap to the foundation.

The substructure is usually subjected to closer scrutiny than other portion of a bridge. Thus, the human scale has been incorporated into the proposed system with the use of chamfers and the surface features discussed in the next section.

7.2.2 Surface Features

The proposed system offers the designer a range of aesthetic flexibility regarding surface features. The optional pier insets allow the designer to enhance the verticality of the pier through the use of 50 mm (2 in) recesses that reflect the web insets of the superstructure I-beams or box beams. A range of form liners might also be utilized in these regions to incorporate fine- or medium-scale texture in the pier faces. The resulting texture may enhance the vertical nature of the column or simply express a motif particular to the bridge's environmental setting. These form liners can be utilized more efficiently in the precast plant. Use of precast, segmental construction also eliminates the problems with horizontal form liner joints that are often experienced in cast-in-place construction.

Texture of a larger scale is automatically incorporated in the specified dimensions of the proposed system. The tapered soffits of the inverted-T caps appear to emerge from the superstructure and flow into the template and pier. The beveled cap soffits express the flow of forces from the girders through the template to the pier. Pier chamfers continue this flow to the foundation.

Fine-scale surface treatments such as exposed aggregate may be used in special situations. The precast plant offers an ideal environment for the application of quality surface textures through the efficient utilization of more complex form systems. The finished color of precast substructures should be more uniform than of those that are cast in place because of the greater control of mix proportions and curing conditions in the precast plant. If desired, special concrete tinting may also be applied successfully in the precast plant environment.

7.2.3 Aging

The proposed system offers several defenses against the deleterious visual effects of aging. Bridges are subjected to continuous aesthetic evaluation throughout their lifespan.

Therefore, the effects of aging are of paramount importance to the overall lifetime value of a bridge.

Because the proposed system is precompressed under service loads and utilizes high performance concrete, there should be no staining due to corrosion of elements within the concrete itself. However, the designer should seriously consider drainage of runoff from the roadway above. This runoff can be channeled from the deck through a drainage pipe embedded at the proper slope within the inverted-T cap. The cap drainage pipe empties into a vertical drainage pipe located within the central void of the pier-template assembly. The runoff flowing through this pipe is then removed to the exterior of the pier by means of an elbow placed in the cast-in-place pedestal at the base of the pier shaft. Use of textured surfaces on the pier faces or insets will reduce the potential negative visual impact caused by the direct assault of rain and spray on the face of the pier, as well as any runoff that escapes the deck drainage system.

As discussed in Section 3.7.2, use of textured concrete surfaces should reduce both the frequency and visual impact of graffiti applications. Graffiti artists are likely to look elsewhere for a more suitable "canvas" for their work, particularly if the bridge as a whole is perceived as an object of civic pride rather than a neighborhood intrusion.

The various surface features available to the bridge designer makes the painting of concrete substructures unwarranted. Painting of concrete surfaces magnifies the visual effects of aging by covering a virtually maintenance-free surface with a material that deteriorates under service conditions and must be maintained on a periodic basis. The use of white paints or coatings provides a perfect background for amplifying stains and offers temptation to the passing graffiti artist.

7.3 Economic Impact

The overall lifetime value of a bridge must be evaluated in terms of its relative costs and benefits to the bridge owner — the taxpayer. The most obvious cost associated with a bridge is its construction cost. The total construction cost consists primarily of the amount of taxpayer funds paid to a general contractor for the construction of the bridge. At present, construction cost is the yardstick of choice for federal and state agencies when comparing the relative value of bridge designs. Other costs associated with the construction of a bridge which are not directly figured into the total construction cost include design costs and the costs associated with right-of-way acquisition.

Unfortunately, the construction cost of a bridge, though easily tabulated, falls short of being an accurate indicator of the overall lifetime value of the bridge. Issues relating to the service life of the structure must also be considered. For example, a bridge with a service lifespan of eighty years will almost certainly be of greater value to the taxpayer than a bridge that has the same construction cost but only has a lifespan of forty years. Likewise, a bridge that must undergo maintenance every ten years is more valuable than one that must undergo a comparable amount of maintenance every five years. Thus, the durability and service performance of a bridge can figure quite significantly in the overall value of a bridge.

Estimating the life-cycle costs of a bridge relative to its construction cost can be quite difficult for two reasons. First, very little historical data has been kept regarding these costs. Second, the present value of life-cycle costs depends on inflation rates throughout the lifespan of the bridge. Such rates can be difficult to predict. The present value of the total life-cycle cost of a bridge might range from 125% to 150% of its construction cost.[1]

As described in Chapter Two, precast segmental substructure systems have been used efficiently and successfully on large bridge projects. They have yet to be implemented on more typical, moderate-span highway bridge projects; therefore, forecasting the relative construction cost of the proposed system is difficult. Initially, the proposed system of bent construction will cost more than the cast-in-place methods currently used. However, due to the level of standardization and repetition built into the system, construction cost will decrease markedly with repeated application.

A survey of bridge projects let in Texas during the first eight months of 1995 reveals that the cost of bent construction is generally less than 15% of the total construction cost of the bridge. Supposing that bents constructed with the proposed system initially cost 20% more than those produced by presently used methods, the total construction cost of the bridge would increase approximately 2 to 3%. When projected to the total life-cycle cost of the bridge, the cost increase is only 1 to 2%. Considering the beneficial effects of the increased performance and durability of the high performance system on maintenance costs, the difference in life-cycle costs between traditional substructure construction and the proposed system approaches insignificance. Once the proposed system is in regular use, construction costs should approach those of the presently used system, and life-cycle costs could conceivably decrease. This is especially true if use of the proposed substructure system makes the painting and repainting of concrete bridges obsolete.

The relative cost of the proposed system is a matter of tradeoffs. Unit prices of the high performance concrete and prestressing reinforcement will be higher than the unit prices of the materials presently used. However, increased structural efficiency will reduce the amount of material required. The unit costs of the materials and labor involved with the fabrication and erection of the proposed system will decrease with the repeated use of standard sections and details.

It is extremely important to note that while the additional costs associated with the implementation of the proposed system increase only the portion of the total construction cost devoted to bent construction, some of the cost savings inherent in the system apply to the total cost of the bridge. For example, the increased speed of construction and the rapid on-site erection time produce cost savings throughout the total construction budget. Enhanced service life performance and durability greatly decrease the costs associated with the annual upkeep of the bridge throughout its lifespan.

Thus far, this discussion has focused solely on one portion of the overall lifetime value of a bridge — its direct monetary cost. Indirect costs to the taxpayer have not yet been addressed. Millions of dollars are lost from decreased productivity due to traffic congestion. The sooner bridge construction is completed; the sooner traffic flows more rapidly on a new section of highway. Faster on-site erection benefits the taxpayer by reducing the amount of time he or she must sit in traffic jams caused by lane closures. Local merchants are often dealt a serious economic blow when construction impedes easy access to their places of business. Thus, the overall taxpayer cost savings due to faster bridge construction can be enormous.

It is likely that use of a largely precast substructure system may result in construction costs that are at worst slightly higher than those presently incurred. At best, these direct construction costs might decrease. Total life-cycle costs of the overall bridge are likely to improve. Indirect costs to the affected communities will diminish. Therefore, the use of the proposed system should prove beneficial when evaluated solely on the basis of monetary economics.

The enhanced economic value of the substructure system is only part of its overall lifetime value to the taxpayer. The remaining factor in assessing the lifetime value of the structure is its perceived worth to the taxpayer. Therefore, the lifetime aesthetic value of the bridge is integral to its overall value. While the visible substructure (bents and abutments) generally comprises less than one-quarter of the bridge's construction cost, it receives the preponderance of the public's visual scrutiny. Therefore, improving the aesthetics of substructures is a very efficient means of improving the aesthetics of the entire bridge. The public identifies with beautiful structures and accepts them as symbols of successful and productive applications of good government. Bridges that are perceived as ugly or ungraceful accelerate the erosion of the taxpayers' faith in public works and the government that produces these works.

Therefore, while the proposed substructure system can be constructed and maintained with either insignificant extra cost or possible savings, its implementation can produce indirect economic returns and intangible aesthetic benefits to the taxpayer that are as yet unrealized. An informal survey of Texans from various geographic regions and socio-economic backgrounds was undertaken as a part of Project 1410. Of particular interest to this research is the fact that over three-fifths of the respondents stated a desire to see the state devote more tax dollars to bridge aesthetics. One-third of the respondents indicated that a greater than 5% increase in spending is warranted. It should be made clear that the investigators involved with Project 1410 do not endorse an "aesthetics surcharge" of 5% or any other value. However, it is interesting to note that there is a willingness among the taxpayers to support increased attention to bridge aesthetics. When one considers that the implementation of the proposed system can provide increased aesthetic value while resulting in little or no increase of life-cycle costs, the overall value to the taxpayer is apparent.

1. Menn, C., *Prestressed Concrete Bridges*, Birkhäuser, Boston, 1990, pp. 50-52.

CHAPTER EIGHT
CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The objective of this research was to apply readily available materials and technology to develop an efficient and attractive precast substructure system for precast, pretensioned I-beam bridges in the state of Texas. A literature review was performed with regard to three topics vital to this objective: current design and construction practice for concrete highway bridges, applications of standardization and precasting technology to bridge substructures, and bridge aesthetics. The first two topics are discussed in Chapter Two; the aesthetics of bridge substructure design is addressed in Chapter Three.

Various available technology options were investigated and evaluated for their applicability in a precast substructure system. These options are explained and their relative merits discussed in Chapter Four. Details regarding both the construction of a precast substructure system, and the structural design and performance of such a system were investigated as a part of the development process. These details are outlined in Chapter Five.

Chapter Six explains the proposed substructure system. The selection, design and fabrication of individual system elements and connections are described. Although the system was developed initially to support I-beam superstructures, it provides the designer enough aesthetic and technical flexibility to prescribe the system for use with pretensioned box beam or U-beam bridges. Plans for typical system elements were developed and are included in the Appendix. The proposed system consists predominantly of match-cast, epoxy-jointed segmental construction. Cast-in-place concrete is used for a few connections where precise field alignment is critical. Traditional cast-in-place procedures are also used for the foundation cap and girder bearing seats.

The system was developed with the overall goal of providing increased overall lifetime value when compared with presently used cast-in-place construction. Elements were sized and designed to provide increased lifetime aesthetic value. Although construction costs will likely be higher initially, use of standardized elements and processes should lead to increased cost efficiency with repeated implementation. Use of precasting reduces the required on-site construction time and associated costs. The system was also designed for enhanced service performance and durability in order to reduce life-cycle costs. Chapter Seven includes a discussion of the aesthetic and economic impact associated with implementation of the proposed substructure system.

8.2 Conclusions and Recommendations

The principal conclusions reached as a result of this research are as follows:

1. Precast concrete has great, yet unrealized, potential for improving the aesthetics, efficiency and durability of moderate-span highway bridge substructures.

2. At present the major durability deficiencies of prestressed concrete bridges occur in the substructure elements. A precast, post-tensioned substructure system that incorporates high performance concrete and improved drainage systems can result in decreased maintenance costs and increased service lifespan for the entire bridge structure.

3. The efficiencies inherent in precast concrete construction are most readily realized through repeated use; therefore, the best method of obtaining these benefits is systematic implementation of the system on a number of bridge projects.

4. Given the technology at hand, the most feasible precast concrete substructure system is one which incorporates a limited amount of cast-in-place concrete at critical joint locations to facilitate geometry adjustments.

5. The visual appearance of a bridge's substructure is critical to the public perception of the bridge; therefore, aesthetics should be considered at ever

6. y stage of the design process, particularly during substructure design.

7. Significant aesthetic improvements can be achieved with only slight increases — or even decreases — in construction cost.

8. Once fully implemented, the proposed substructure system can result in improved bridge aesthetics and increased lifetime value to the taxpayer.

Because many of the cost savings inherent in the proposed system will only be realized after repeated use, implementation of the proposed system should be mandated on a few trial bridge projects. These projects will provide an opportunity to evaluate the construction cost of the system and estimate the potential cost savings that would be realized through its repeated use. As mentioned in Chapter Two, contractors often have a better attitude regarding precast concrete after a project's completion than they had on its bid date. Performance and durability of the substructure system may be evaluated on trial projects in order to recognize long term cost benefits. Utilization of the system on several projects would also provide the opportunity to gauge the effect of the new system on the aesthetic sensitivities of the taxpayer. The system can be modified in order to further improve its aesthetic value or to overcome previously unanticipated difficulties encountered during construction.

Once the system has been implemented and evaluated on several projects, it may be offered as a bid option to compete with traditional cast-in-place substructures. Over time, the successful widespread implementation of this system for moderate-span highway bridge substructures will depend upon its aesthetic and economic success.

APPENDIX A

PLANS FOR VARIOUS SYSTEM ELEMENTS

Figure A.1: Schematic of Bent Segments

Figure A.2: Standard Pier Segment Cross-Sections

Figure A.3: Standard Pier Segment Heights

Figure A.4: Pier Segment P28

Figure A.5: Plan View of Segment P28

Figure A.6: Front Elevation of Segment P28

Figure A.7: Side Elevation of Segment P28

Figure A.8: Reinforcing Bars for Segment P28

Figure A.9: Standard Template Cross-Sections

Figure A.10: Template T28

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Figure A.10: Template T28

11

Figure A.11: Plan View of Template T28

Figure A.12: Front Elevation of Template T28

Figure A.13: Side Elevation of Template T28

Figure A.14: Reinforcing Bars for Template T28

Figure A.15: 13.2 m-Long Cap for P28 Pier Shaft

Figure A.16: Plan View of Primary Cap Segment

Figure A.17: Front Elevation of Primary Cap Segment

Figure A.18: Side Elevation of Primary Cap Segment

Figure A.19: Plan View of Secondary Cap Segment

Figure A.20: Front Elevation of Secondary Cap Segment

Figure A.21: Side Elevation of Secondary Cap Segment

Figure A.22: Reinforcing Bars for Primary and Secondary Cap Segments

REFERENCES