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Extending Span Ranges of Precast Prestressed Concrete Girders

REID W. CASTRODALE
CHRISTOPHER D. WHITE
Ralph Whitehead Associates, Inc.
Charlotte, NC

TRANSPORTATION RESEARCH BOARD

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NOTICE

THE NATIONAL ACADEMIES
Advisers to the Nation on Science, Engineering, and Medicine

FOREWORD

By David B. Beal
Staff Officer
Transportation Research
Board

This report contains the findings of research performed to develop recommended load and resistance factor design procedures for achieving longer spans using precast prestressed concrete bridge girders. Standard details and examples illustrating the design of three typical bridges were also developed. The material in this report will be of immediate interest to bridge designers.

Each year, 200 to 300 bridges are built with spans exceeding 160 feet. Generally, these spans are not designed with precast prestressed concrete girders because of limited experience and limited information. In addition, the size and weight of many precast sections precludes land transportation. Limited choices lead to decreased competition between bridge types and materials, which results in higher costs to owners.

Solutions to increase the span range of precast prestressed concrete girders used in the United States and abroad include splicing segments for longer simple or continuous spans, connecting simple spans for continuity, and assembling haunched pier segments in the field. However, this experience and the technology for applying these solutions are fragmented and job-specific.

The objective of this research was to develop recommended load and resistance factor design procedures, standard details, and design examples for achieving longer spans using precast prestressed concrete bridge girders. This research was performed by Ralph Whitehead Associates, Inc. The report fully documents the research leading to the conclusion that spliced-girders are the design option with the greatest potential for extending span length. Detailed design examples for one-, two-, and three-span spliced-girder bridges are included as appendixes.

CONTENTS

EXTENDING SPAN RANGES OF PRECAST PRESTRESSED CONCRETE GIRDERS

SUMMARY

Precast prestressed concrete girder bridges have proven to be economical, exhibiting good structural performance and requiring minimal maintenance. Improvements in materials, the introduction of new girder shapes, and advances in design methods have increased the span range of precast prestressed concrete girders over the years. However, these girders are still used infrequently for spans in excess of 160 feet. This upper limit of practical application exists for several reasons, including material limitations, structural considerations, size and weight limitations on girder shipping and handling, and a general lack of information and design aids necessary to design longer spans using concrete girders.

Eliminating precast prestressed concrete girders from consideration for spans beyond 160 feet limits the options available to designers for this span range. This lack of available options results in decreased competition between bridge types and materials, which can lead to higher costs for bridge owners. It also denies owners the long-term benefits of prestressed concrete girder structures, such as durability, low maintenance, and aesthetics.

Some designers, fabricators, and contractors, however, have successfully collaborated to extend span lengths for precast prestressed concrete girders to distances greater than 300 feet and expand their use to other applications not normally associated with precast prestressed concrete girder construction. In these cases, the introduction of another material and structure type into the long-span bridge market has already improved the economy of long-span bridges. Unfortunately, the experience, information, and methods used on these projects have tended to be job-specific, and the knowledge gained has not been made widely available for use on similar projects.

NCHRP Project 12-57, "Extending Span Ranges of Precast, Prestressed Concrete Girders," was initiated to address these issues by collecting prior experience and information and then developing design recommendations and specifications so that owners, designers, fabricators, and contractors would be prepared to make more effective use of precast prestressed concrete girder bridges for new applications and longer spans. The objectives of the research project are specifically stated in the Research Project Statement:

The objective of this research is to develop recommended load and resistance factor design (LRFD) procedures, standard details, and design examples for achieving longer spans using precast, prestressed concrete bridge girders.

The full spectrum of approaches for extending the span ranges of precast prestressed concrete girders was considered in the research. However, it was found that incremental changes in conventional design methods and materials generally result in relatively small increases in the span range for precast prestressed concrete girders. Information required to implement these techniques is also generally available in the literature. Therefore, although the research findings include listings and comparisons of these conventional techniques, the further development of these techniques was not the focus of the study.

It was found that the splicing of precast prestressed concrete girders could increase span ranges significantly. A spliced girder is a precast prestressed concrete member that is fabricated in several relatively long pieces (i.e., girder segments) that are assembled into a single girder for the final bridge structure. Post-tensioning is generally used to reinforce the connection between girder segments. Splicing allows designers to overcome limitations of fabrication, shipping, and erection that have prevented the use of very long precast prestressed concrete girders in the past.

Spliced girder construction shares several features with segmental box girder construction, including the fact that spans are made up of several pieces or segments and the significant issues of post-tensioning and grouting. Where possible, experience and design methods from segmental box girder construction should be used for spliced girder construction. However, the researchers found that a distinction needs to be made between the two types of construction to avoid confusion of terminology and to clarify the application of the design specifications. Several ways in which spliced girder construction differs from segmental box girder construction include the following: girder segments are a greater portion of the span length; joints between girder segments are generally cast-in-place, rather than match-cast and epoxied; spliced girder segments do not incorporate a full-width deck; and bulb-tee or U-beam sections are used rather than closed-box girder shapes.

This study identified over 250 spliced precast prestressed concrete girder bridges that have been designed in the United States, Canada, and other areas of the world. Although this number is significant, it is a very small fraction of the total inventory of bridges in these areas. Use of spliced girders is also not widespread, with most spliced girders located in a few regions. Implementation of this technology requires consideration of various issues with which the designer of conventional precast prestressed concrete girders typically is not familiar. Furthermore, information regarding the implementation of spliced girder construction is limited and the design specifications do not clearly address the design of spliced girder bridges. Therefore, the main focus of this research was to address issues related to the design and construction of spliced precast prestressed concrete girder bridges.

The products of this research project include the following:

- A listing of spliced girder bridges was developed to provide ideas and a level of comfort regarding the design and performance of these structures. Key information is given for each project in the list. The data are summarized by region. It was found that few precast prestressed concrete spliced girder bridges have been constructed in other countries.
- Listings of design options for extending span ranges of precast prestressed concrete girder bridges are given. The options include material-related options, design enhancements, methods using post-tensioning, and spliced girder construction. A limited comparative design study was performed to demonstrate the effectiveness of several of the most promising material-related and design enhancement options for a specific bridge.

- An extensive list of design issues and details that should be considered when designing extended spans for both conventional and spliced girder construction is provided. The issues and details are discussed, giving possible options along with advantages and disadvantages. Some of the major issues are cost data, deck replacement, post-tensioning and grouting, staged construction, curved bridges, and diaphragms at closure pours. A method for splicing haunched pier segments is proposed. An overview of the development of spliced girder construction in Florida is also provided.
- The tabulated results of a review of the *AASHTO LRFD Bridge Design Specifications,* conducted to identify obstacles to implementation of long-span solutions, are included. The most significant issues identified related to the refinement of the *Specifications* for the design of spliced girders. Proposed revisions to the *Specifications* are included and address some of the concerns related to clarifying the *Specifications* for application to spliced girder construction.
- Comprehensive spliced girder design examples were developed to illustrate spliced girder design for a long simple-span bridge, a two-span bridge in a seismic region, and a three-span bridge with haunched pier segments.
- A list of issues that need to be addressed in order to encourage and permit wider use of extended-span precast prestressed concrete girder bridges was developed. Resolution of these issues will require action by owners and industry rather than designers. Types of issues discussed include general items, specifications, design and detailing, fabrication, transportation and construction. Hauling restrictions were identified as a major issue.
- A list of design resources, including design programs and other documents, is provided.

This research provides the information and tools required for implementation of spliced girder technology and other methods for extending the span ranges of precast prestressed concrete girders. Implementation of the findings of this research should be aggressively pursued through a partnership between owners and industry.

CHAPTER **1**

INTRODUCTION AND RESEARCH APPROACH

1.1 PROBLEM STATEMENT

Since its first use for bridges in 1950 *(1)*, precast prestressed concrete has gained rapid acceptance as the preferred material for short- to medium-span bridges in the United States *(2)*. Bridges built using prestressed concrete girders have proven to be economical, exhibit good structural performance, and typically require minimal maintenance. Through the use of improved materials, the introduction of more efficient girder shapes, and the development of other enhancements, the range of spans for which precast prestressed concrete girders are used has increased over the years. However, such girders are still used infrequently for spans in excess of 160 feet. This upper limit of practical use exists for several reasons, including material limitations, structural considerations, size and weight limitations on girder shipping and handling, and a general lack of information necessary to design and build longer spans.

Eliminating precast prestressed concrete girders from consideration for spans beyond 160 feet limits the options available to designers for this span range. This lack of available options results in decreased competition between bridge types and materials, which can lead to higher costs for bridge owners. It also denies owners the long-term benefits of prestressed concrete girder structures, namely durability, low maintenance, and aesthetics.

Some designers, fabricators, and contractors, however, have collaborated successfully, using various approaches, to extend the span lengths for precast prestressed concrete girders to distances in excess of 300 feet. In other cases, where shorter spans were more appropriate, special techniques have enabled the use of precast concrete girders in situations where they typically would not have been considered. In many cases, these precast prestressed concrete bridges have been constructed as alternate designs, where the bid documents indicated a different type of structure. Unfortunately, the experience, information, and methods used on these projects have tended to be job-specific, and, in general, the knowledge gained has not been made widely available for use on similar projects.

1.2 RESEARCH OBJECTIVES AND TASKS

NCHRP Project 12-57, "Extending Span Ranges of Precast Prestressed Concrete Girders," was initiated to address these issues by extending the practical use of prestressed concrete girders to longer spans and to applications not normally associated with precast prestressed concrete girder construction. The major goal of the research was to provide the necessary information to owners, designers, fabricators, and contractors so as to enable them to consider and design precast prestressed concrete girders for these new applications and longer spans.

The objective of the research project is clearly stated in the Research Project Statement:

> *The objective of this research is to develop recommended load and resistance factor design (LRFD) procedures, standard details, and design examples for achieving longer spans using precast, prestressed concrete bridge girders.*

To accomplish this objective, NCHRP Project 12-57 included the following tasks:

- Task 1. Review relevant foreign and domestic practice, performance data, research findings, design examples, design software, and other information related to long-span precast prestressed girder bridges. This information shall be assembled from technical literature and from unpublished experiences of engineers, bridge owners, precasters, and others. Information on actual field experience is of particular interest.
- Task 2. Identify and describe existing or new methods for extending the span length of precast prestressed bridge girders. These methods shall include, as a minimum, splicing, post-tensioning, and enhanced concrete properties. List the advantages and limitations of each method, and identify critical issues for design, fabrication, transportation, and construction.
- Task 3. Determine if revisions to the *AASHTO LRFD Bridge Design Specifications (3)* are needed in order to address the issues identified in Task 2. Recommend a course of action to develop each specification change identified.
- Task 4. Identify other actions needed in order to address the issues identified in Task 2 and make recommendations on implementation of these actions.
- Task 5. Submit an interim report to document Tasks 1 through 4 for review by the NCHRP. The report shall include a prioritized list of alternatives for extending the span length of concrete girders.

- Task 6. Develop comprehensive design procedures for each of the alternatives approved by the project panel in Task 5. Illustrate these procedures with design examples.
- Task 7. Prepare recommended changes to the *AASHTO LRFD Bridge Design Specifications (3)* to address the issues identified in Task 3 as approved by the project panel.
- Task 8. Revise the Task 6 and 7 deliverables in accordance with project panel comments.
- Task 9. Submit a final report that documents the entire research effort and includes the design procedures and recommended specification changes as appendixes. In addition, provide a companion executive summary that outlines the research results.

The execution of the project was divided into two phases. Phase I, which included Tasks 1 through 5, was concluded with the submittal of the Interim Report. Phase II, which included Tasks 6 through 9, was concluded with the submittal of the Final Report. The contents of the Interim Report have been incorporated into the Final Report.

1.3 MAIN FOCUS OF RESEARCH— SPLICED GIRDERS

The project statement and tasks address the full spectrum of possible approaches for extending the span ranges of precast prestressed concrete girders. Although this wide focus was retained for portions of the study, it was determined that narrowing the focus of the study would provide the greatest benefit. This decision was based on the following findings from the early stages of the research:

- Most of the techniques and approaches for extending span ranges involve incremental changes in conventional design methods and materials. These changes generally result in relatively small increases in the span range for precast prestressed concrete girders. Information required to implement these techniques is generally available in the literature or from commercial sources. Therefore, although it is important to identify and evaluate these approaches to make more designers aware of the potential benefit in their use, a significant research effort is not required before these techniques can be used.
- One technique, the splicing of girders, was found to allow significantly increased span ranges for precast prestressed concrete girder bridges. This technique involves the fabrication of the girders in segments that are then assembled into the final structure. Although many spliced girder bridges have been constructed, the use of this technique is not widespread. Use of this technology also requires consideration of various issues with which the designer of conventional precast prestressed concrete girders typically is not familiar. Furthermore, the infor-

mation available in the literature regarding the implementation of spliced girder construction is limited.

Based on these findings, it was determined that the main focus of this study would be to address issues related to the design and construction of spliced precast prestressed concrete girder bridges.

1.3.1 Definition of Spliced Girders

For the purposes of this study, a spliced girder is a precast prestressed concrete member fabricated in several relatively long pieces (i.e., girder segments) that are assembled into a single girder for the final bridge structure. Post-tensioning is generally used to reinforce the connection between girder segments.

Other characteristics of precast concrete spliced girders are discussed in the next few sections. Splicing girders is simply an approach to make possible the use of longer or heavier girders than can otherwise be used. Therefore, the technique is often used in conjunction with other approaches for extending span ranges to achieve the maximum benefit.

1.3.2 Reasons for Using Spliced Girders

1.3.2.1 Design Issues

Structural design issues are often significant factors in the selection of a spliced girder system for a bridge. Common issues that may lead a designer to consider the use of precast prestressed concrete spliced girders for a project are as follows:

- Increasing span lengths to reduce the number of substructure units and the total project cost;
- Increasing the girder spacing to reduce the number of girder lines and the total project cost;
- Increasing span lengths to improve safety by eliminating shoulder piers or interior supports;
- Minimizing structure depth through the use of long, continuous members to obtain required vertical clearance for traffic, waterways, and so forth;
- Avoiding the placement of piers in water to reduce environmental impact and total project cost;
- Placing piers to avoid obstacles on the ground, such as railroad tracks, roadways, and utilities;
- Improving aesthetics through various design enhancements, such as more slender superstructures, longer spans, or haunched sections at piers;
- Eliminating joints for improved structural performance, reduced long-term maintenance/increased service life, and improved rideability;

- Eliminating falsework that may restrict the flow of traffic beneath a structure and may pose a safety hazard for traffic and construction workers; and
- Increasing speed of construction through the use of precast elements to reduce congestion and traffic delays.

1.3.2.2 Construction Issues

Spliced precast concrete girders may also be selected as the preferred superstructure system for a bridge in order to deal with significant construction issues. The most common is that the girders, if manufactured and installed as full-span elements, are too large to handle at one or more of the following points in the construction sequence:

- Manufacturing,
- Handling,
- Transportation, and
- Erection.

Contractor preferences and/or the availability of equipment, as well as the means of delivery and access to the site, may also affect the use of spliced girders.

Although span lengths for spliced girder bridges may be comparable to those of segmental box girder bridges, construction is generally more conventional. This allows bridge contractors to pursue spliced girder projects rather than requiring specialty contractors to perform the work. A specialty contractor typically is still required to perform the operations related to post-tensioning and grouting. Furthermore, fabrication of girder segments can be performed in existing precasting plants with existing or modified forms, rather than requiring the purchase and setup of custom forms for a segmental box girder project. Erection typically also can be accomplished with standard equipment, rather than the specialized equipment often required for segmental box girder construction. These advantages may lead to better competition and reduced project cost.

1.3.2.3 Cost Issues

Typically, a compelling reason is needed to justify the added costs for post-tensioning and temporary supports, if required. Reduced construction costs, resulting from the advantages listed in Section 1.3.2.1, and reduced fabrication time may help off set these increased costs. There are many examples of spliced concrete girders providing economical solutions when bid in competition with other solutions and materials.

With the increased use of design/build contracts and value engineering proposals, it appears likely that more spliced girder bridges will be used as designs are tailored to contractors' preferred methods and equipment. The increased speed of fabrication and erection for spliced concrete girders is def-

initely attractive to contractors participating in design/build ventures or for contractors seeking cost or schedule reductions through value engineering proposals.

1.3.3 Typical Applications for Spliced Girders

1.3.3.1 Simple Spans

Simple-span prestressed concrete girders have been widely used across the country. However, in situations where full-length girders cannot be easily handled or hauled to the project site, splicing can enable the use of precast prestressed concrete girders.

Some examples where simple spans may require splicing are as follows:

- Remote sites or locations where access to the site is restricted. In these cases, the roadway access to the bridge site may not be adequate to support the load of a full-span girder, or the alignment of the road may not allow the transportation of a full-span girder.
- Situations where available or preferred equipment is inadequate to handle, transport, and/or erect a full-span girder. An example is where a contractor would prefer to use a smaller crane at the erection site, rather than rent a large crane only required for setting the girders.
- Single-point urban interchanges (SPUIs). Long, single-span bridges are required to span an intersection beneath the structure. A bridge with a minimum structure depth is often a requirement. If a one-piece girder were used to provide the required span, it would be well beyond typical hauling and handling capabilities.

1.3.3.2 Continuous Spans

The most common application for spliced concrete girders has been for multiple-span bridges where the girders are continuous for the deck, superimposed dead loads, and live loads. Typical examples where continuous spans use splicing are as follows:

- Crossings where long spans are required, especially over waterways or for viaducts;
- Sites where overhead clearances are restricted;
- Sites with limited substructure locations; and
- Situations where the number of girders or substructure units can be reduced to improve economy.

Spliced concrete girders that are continuous over interior supports may be constructed using a wide variety of configurations. The most common configurations are as follows:

- Full-span girders that are spliced at interior supports;
- Partial-span girder segments that are spliced within the span and over the interior piers;

- Partial-span girder segments that are continuous over the interior piers, but spliced within the span; and
- A special case of the preceding configuration where the pier segments are haunched to improve structural performance.

In some cases, a single bridge may use several of the configurations listed above to address special design requirements. An example is the Main Street Viaduct in Pueblo, Colorado (see Section 1.3.4.2.2).

1.3.3.3 Seismic Applications

Spliced concrete girders are becoming more widely used where seismic activity must be considered. Attractive features of spliced girder bridges for these locations include the following:

- Continuous superstructure to resist seismic loads and
- Integral connection (cap) between superstructure and substructure.

For seismic locations where cast-in-place concrete bridges are commonly used, precast concrete spliced girder bridges also offer the following advantages:

- Elimination of falsework that may restrict traffic beneath a structure or pose a safety hazard to traffic and construction workers and
- Increased speed of construction, which reduces congestion and traffic delays.

1.3.4 Examples of Spliced Girders

Several examples of projects using spliced girders are presented in the following sections for each of the three categories of applications discussed in Section 1.3.3. Only a brief description of each bridge is given here. Details are available in the references cited.

1.3.4.1 Simple Spans

These bridges illustrate some of the issues that affect the decision to use spliced girders in special situations and highlight the versatility of splicing for simple spans—with applications for long and short spans and with locations at remote sites or in the middle of major cities.

1.3.4.1.1 Klickitat County, Washington. A spliced girder bridge was built in Klickitat County, Washington, in 1954, making it one of the first spliced girder bridges constructed in the United States *(4)*. The simple-span bridge is also prob-

ably one of the shortest spliced girder bridges in the United States with a final girder length of 90 feet. The girder was fabricated in three 30-foot segments by inserting separator plates in the girder forms. Splicing was used because the contractor did not have a large enough crane to lift the entire girder. The segments were erected on falsework as shown in Figure 1, then post-tensioned.

1.3.4.1.2 Rock Cut Bridge, Washington. The Rock Cut Bridge, which crosses the Kettle River in a remote area of Washington State, was constructed in just 3½ months in 1995 *(5, 6)*. The total girder length was 190.5 feet, which was divided into three nearly equal girder segments. Access to the site was very difficult, eliminating the use of full-length girders. However, because of stringent environmental constraints, no construction could take place in the river. Therefore, the three segments for each girder were assembled on the ground near the bridge site, then trucked to the site and launched across the river using a launching truss, as shown in Figure 2. The use of this innovative construction approach to meet difficult site conditions and a very tight construction schedule and budget earned the bridge the PCI Harry H. Edwards Industry Advancement Award in 1997, as well as the PCI Award in the "Bridges with Spans Greater than 135 ft" category *(5)*.

Figure 1. Bridge in Klickitat County, WA (4).
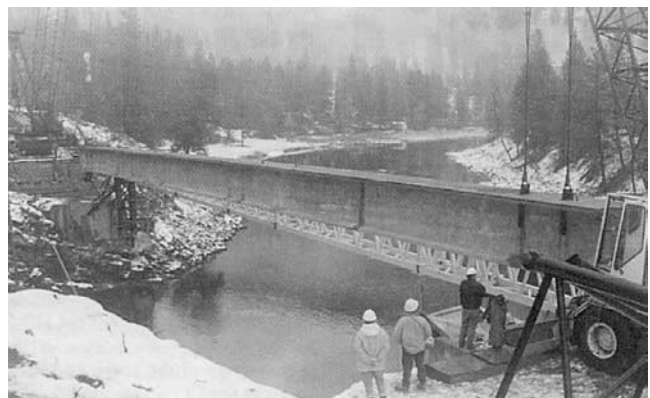
Figure 2. Rock Cut Bridge, WA (6).

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1.3.4.1.3 I-15 Bridges, Salt Lake City, Utah. The reconstruction of Interstate 15 through the heart of Salt Lake City was a huge design/build project performed on a very tight schedule. Bridges over 17 SPUIs were to be constructed as part of this project *(7, 8)*. These bridges had simple spans of up to 220 feet that were required to span all of the traffic flow patterns of an interchange beneath the bridge. The design/build team, which could have selected any available material or concept to construct these bridges, selected spliced concrete girders to satisfy the design and scheduling requirements. For each line of girders, the three segments were erected on falsework as shown in Figure 3. Girder splices and intermediate diaphragms were then cast, followed by placement of the concrete deck before the girders were post-tensioned.

1.3.4.2 Continuous Spans

These bridges illustrate additional concepts and advantages of spliced girder bridges where multiple continuous spans are required. These projects also indicate that the spliced girders may use constant depth or haunched sections, or even a combination of the two, to achieve the spans necessary to satisfy the design requirements.

1.3.4.2.1 US 231 over the White River, Indiana. The bridge carrying US 231 over the White River in Indiana was bid as an alternate to steel plate girders in the early 1990s *(9)*. The multi-span spliced concrete girder alternate used constant depth, full-span girders spliced at interior piers and post-tensioned for continuity as indicated in Figure 4. The low bid for the concrete alternate was more than 10 percent below the low bid for the steel alternate. Using this system, the bridge was divided into three-span continuous units. The continuity allowed a very wide girder spacing to be employed, which added to the economy of the system. Semi-lightweight concrete was used in the girders to reduce the dead load. Several bridges in Florida have also been constructed using a similar approach *(10)*.

1.3.4.2.2 Main Street Viaduct, Pueblo, Colorado. The Main Street Viaduct in downtown Pueblo, Colorado, crosses 12 railroad tracks, the Arkansas River and its dike, and a city street *(11)*. The locations for piers were severely limited because of the obstacles beneath the bridge. Full-span pre-

Figure 3.   I-15 Bridge, Salt Lake City, UT (7).

stressed concrete girders were not used because close girder spacings were required for the long spans. Steel plate girders were also eliminated because of higher initial costs and the requirement for long-term maintenance. The bridge has five spans ranging from 88 to 174 feet. The designers creatively used both haunched girders and thickened webs at piers to satisfy vertical under-clearance and structural requirements. Two falsework towers were used during erection of the pier segments, one of which is shown in Figure 5. The remaining girder segments were erected using strong-backs attached to the girders. Construction of the bridge was completed in 1995.

1.3.4.2.3 Highland View, Florida. The Highland View Bridge is a three-span, high-rise bridge over the Gulf Intracoastal Waterway in Florida. It is typical of several similar bridges constructed in recent years in coastal areas *(10, 12)*. The bridge has a main span of 250 feet, which was a record for this type of structure at the time of its construction. The constant depth end and drop-in girder segments are 6 feet deep, with the haunch over the piers increasing to 10 feet deep. Two falsework towers were erected outside the channel span to stabilize the pier segments, to support the reactions from the end span girders, and to resist uplift when the drop-in segments were placed (see Figure 6). Strong-backs were attached to the drop-in segments to support them from the ends of the pier segments. This erection scheme kept all falsework out of the channel. The bridge construction was completed in 1994. Recently, a similar spliced concrete girder bridge with a main span of approximately 320 feet was constructed in Florida.

1.3.4.3 Seismic Applications

Bridges constructed in seismic regions often require a rigid or integral connection between the superstructure and
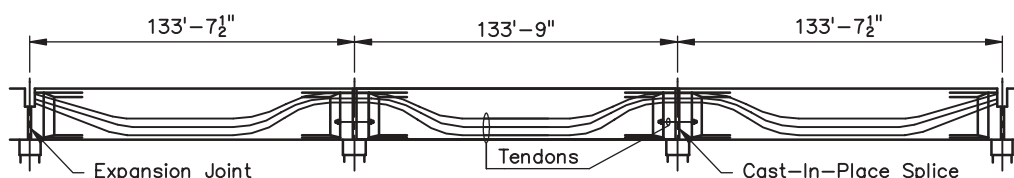
Figure 4.   Schematic of bridge over White River, IN (9).

Figure 5. *Main Street Viaduct, Pueblo, CO (11).*

Figure 6. *Highland View Bridge, FL.*

the supporting substructure. The following examples show that spliced girder concepts can be used to satisfy this requirement.

1.3.4.3.1 Research at the University of California at San Diego. Between 1996 and 1998, tests were conducted at the University of California at San Diego (UCSD) on the connection between precast prestressed concrete spliced bridge girders and their supporting columns *(13, 14)*. Two types of bridges were tested: (1) bulb-tee girders with a girder segment crossing the pier and splices within the span and (2) open-top trapezoidal box beams (bathtubs) with splices at the pier and at midspan. Both systems used an integral pier cap with the girders passing through or embedded into the pier cap. A schematic of the prototype structure for the test program is shown in Figure 7.

The tests of these spliced concrete girder concepts demonstrated that these structural systems significantly exceeded the minimum ductility requirements. Furthermore, the superstructure and bent cap response was essentially elastic during the simulated seismic loading. These results are very promising and should open up a significant new area of application for spliced girder bridges.

1.3.4.3.2 H-3/Lower Halawa Valley Viaduct, Hawaii. A good application of one of the systems tested at UCSD is the mile-long Lower Halawa Valley Viaduct in Hawaii *(15)*. The spliced girder was a value engineering solution proposed by the contractor's engineer to reduce the cost of the dual bridges and the time required to build them. Each bridge was
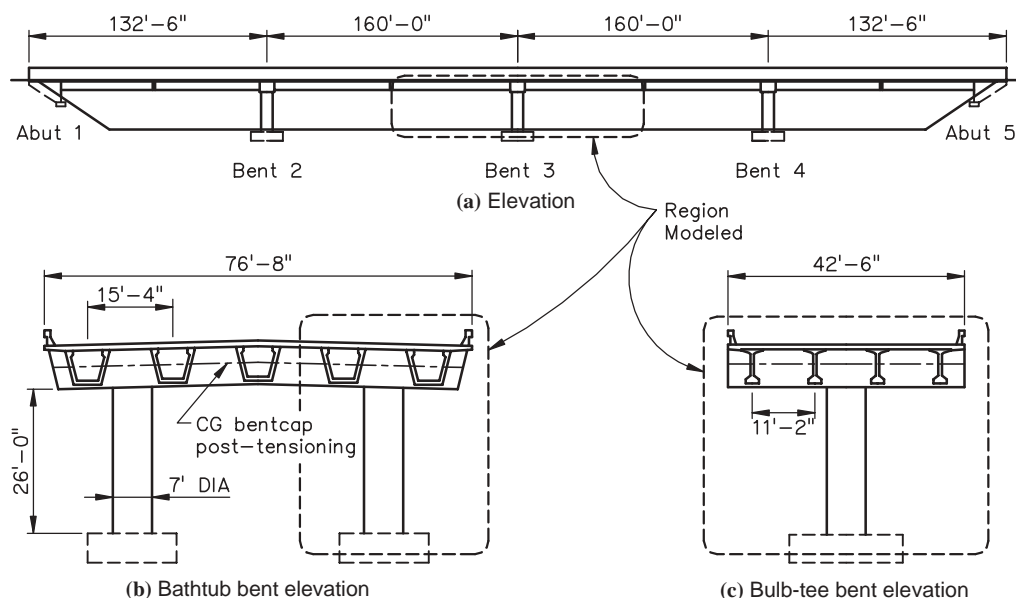
Figure 7. *Schematic of prototype structure for tests at UCSD (14).*

divided into five-span units, with each span approximately 200 feet long. The pier segments were haunched. A single falsework tower was used at each pier to stabilize the pier segments, as shown in Figure 8. Integral pier caps were used to make a positive connection between the superstructure and substructure to resist the seismic design forces.

1.3.5 Comparison of Spliced Girders and Segmental Box Girders

Although spliced girder construction is similar to segmental box girder construction in many ways, there are significant differences. It became clear during this study that it is important to distinguish between the types of construction where appropriate and to emphasize the similarities in other areas.

A first step in comparing the two types of construction is establishing practical terminology that distinguishes spliced girder construction from segmental box girder construction. It is recommended that the individual elements of a spliced girder be called "girder segments" to distinguish them from the segments used for segmental box girder construction and to avoid potential confusion. This terminology is used throughout this report.

To distinguish further between the types of construction, the following definitions are proposed for use in the *AASHTO LRFD Bridge Design Specifications (3)*. (It should be noted that the definition for "Spliced Precast Girder" is only intended to apply to superstructure elements, while the definition for "Segmental Construction" is intended to be more general, applying to superstructure and other types of elements.)

- **Spliced Precast Girder**—A type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using post-tensioning, to form the completed girder. The bridge cross section is typically a conventional structure in which multiple precast girders support a cast-in-place concrete composite deck. This type of construction is not considered to be segmental construction for the purposes of these Specifications.
- **Segmental Construction**—The fabrication and erection of a structural element (i.e., superstructure and/or substructure) using individual elements, which may be

Figure 8.   Lower Halawa Valley Viaduct, HI (15).

either precast or cast-in-place. The completed structural element acts as a monolithic unit under some or all design loads. Post-tensioning typically is used to connect the individual elements. For superstructures, the individual elements typically are short (with respect to the span length) box-shaped segments with monolithic flanges that comprise the full width of the structure.

1.3.5.1 Differences

The typical differences between spliced girder and segmental box girder technology and construction are summarized in Table 1.

Several of the differences listed in Table 1 have special significance for the corrosion resistance and, therefore, the long-term performance of a bridge. Differences that may provide an added measure of corrosion resistance for spliced girder bridges compared to segmental box girder bridges include fewer joints, joints protected by the cast-in-place (and often post-tensioned) deck, internal fully bonded tendons, and the redundancy of multiple webs and tendons.

1.3.5.2 Similarities

Despite the many differences between spliced girders and segmental box girders, the two types of girders share the following:

- Longitudinal post-tensioning with multi-strand tendons,
- Grouting of tendons,
- Staged construction methods, and
- The occasional use of match casting for spliced girder construction.

1.3.5.3 Recommendations

Based on these comparisons, three significant recommendations are as follows:

1. Make use of similar technology wherever possible to advance the use of spliced girder construction by using proven technology from the segmental construction industry.
2. Distinguish between the types of construction to eliminate confusion for the designer.
3. Distinguish between the types of construction to avoid unnecessary focus on corrosion issues for spliced girder construction.

1.4 RESEARCH APPROACH

Research objectives were fulfilled by addressing each of the research topics listed in Table 2. Each research topic

TABLE 1 List of differences between spliced girders and segmental box girders

Issue	Spliced Girders	Segmental Box Girders
Size of segments relative to span	Large - 2 to 3 segments / span	Small - 8 to 10 segments / span
Joints between segments	Field cast with concrete 6 to 24 in. wide	Match cast, with epoxy Essentially no joint width
Girders	Standard girder shapes	Customized girder shapes
Number of girders	Multiple lines of girders in each span	Single girder line in each span
Deck	A composite deck is typically cast on spliced girders	The deck is cast as an integral part of the box girder segments
Hauling issues	Length and sometimes weight and height	Weight and height
Contractors	Bridge contractors	Specialty bridge contractors
Fabricators	Bridge girder fabricators	Specialty bridge fabricators
Redundancy	Many webs and post-tensioning tendons	Generally two webs with post-tensioning tendons
Corrosion	Splices are typically protected by the composite deck	Splices penetrate through the entire depth of the structure
Erection	Conventional equipment	Often specialized equipment
Post-tensioning	Typically one or two stages for the full length of bridge	May include many stages of post-tensioning as segments are erected
Longitudinal prestressing	Combined pretensioning and post-tensioning	Only post-tensioning
Number of splices in ducts	Few, with splices accessible during construction	Many, with inaccessible splices at match-cast joints
Tendon locations	Internal	Combination of internal and external (within the box)
Section type	Bulb tee or U-beam	Box

focused on a different area of investigation and development required to extend the span ranges of precast prestressed concrete bridges. The topics were selected as the best way to organize the findings of the study to facilitate their practical use. The topics are related to the project tasks listed in Section 1.2, but there is no one-to-one correspondence between tasks and research topics. The tasks related to each research topic are indicated in Table 2.

For each research topic, appropriate activities were performed such as the collection of information or the development of recommendations. Two or more activities were defined for most of the research topics to improve access to the information and recommendations that have been developed. Brief descriptions of the research topics and activities are given in this section.

A listing of the research topics with their activities and the chapters in which the results are described and summarized is shown in Table 3. This table also indicates the appendix that contains the complete presentation of related results for each topic or subtopic.

A brief discussion of each research topic and its associated activities follows. The discussion of the first research topic, Project Information (Section 1.4.1), includes a detailed discussion of the data collection activity that provides the foundation for many of the research findings and applications reported in Chapters 2 and 3.

1.4.1 Project Information

One of the goals of the project was to collect data on projects that had been designed or constructed using various methods for extending span ranges of precast prestressed

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TABLE 2   List of research topics and tasks

Research Topic	Related Task
Project Information	1
Design Options	2
Issues and Details for Extended Span Bridges	2
Design Examples	6
Design Specifications	3 & 7
Issues Requiring Additional Action	4
Design Software and Other Resources	1

concrete girders. This information is valuable to demonstrate the variety of applications and approaches for which extended-span concepts have been used. Data collection efforts focused on spliced concrete girders. A list of spliced girder bridges with basic project information was developed. A summary of the bridge list with comments on the significance of the data was also prepared. A discussion of the process used to collect project information follows.

TABLE 3   List of research topics and location of discussion and results

Research Topic or Activity	Chapter	Appendix
Project Information		
Data Collection Form	1	A1
Evaluation of Data Collection Results	2	A2
Listing of Spliced Girder Bridge Projects	2	A3
Design Options		
Design Options for Extending Spans	2	B1
Comparative Design Study	3	B2
Issues and Details for Extended Span Bridges		
Issues and Details for Extended Span Bridges	2	C1
History of Florida's Spliced Girder Bridges	2	C2
Design Examples		
Design Example 1 – Single Span Spliced PCI BT-96 Girder	3	D1
Design Example 2 – Two-Span Spliced U-Beam Girder	3	D2
Design Example 3 – Continuous Three Span Girder	3	D3
Design Specifications		
Proposed Revisions to the *AASHTO LRFD Bridge Design Specifications*	2	E1
List of Potential Issues in the *AASHTO LRFD Specifications*	2	E2
Issues Requiring Additional Action	2	F
Design Software and Other Resources		
Design Software and Other Resources	2	G1
Bibliography	2	G2

A literature search identified published sources of project information in technical journals, presentations, marketing literature, and project award submissions for extended-span prestressed concrete girder projects. Some references were project specific, while others addressed more generally applicable details of a particular design or construction method. References used to obtain project data are included in the bibliography provided in Appendix G2. In some cases, references may not be generally accessible to the public.

Published information on projects using extended spans for precast prestressed concrete girders is limited. Therefore, data for many projects could only be collected through direct interaction with the participants in the project or others who had access to project documents. Details of projects, including information on the selection of the particular method or methods for extending girder spans, design methods, details of fabrication, and methods of construction and erection were collected. An attempt was also made to collect the experiences of participants in the projects in order to use this information to identify important factors that need to be considered in the design and construction of spliced girder and other long-span concrete girder bridges.

A data collection form was developed as a Microsoft Excel spreadsheet. The single-page form and two pages of instructions that accompanied it are shown in Appendix A1. Macros in the spreadsheet enabled the user to enter data for up to nine bridges in a single file. The form was distributed via email to project participants, including owners, designers, contractors, fabricators, and post-tensioning suppliers. A form was sent to the department of transportation in each state. Initially, the form was sent to a list of contacts compiled by the research team. Additional contacts were identified as the data collection effort progressed.

The form was used to collect project information, including dimensions, design procedures, costs, project participants and methods of construction. Respondents were also asked to provide comments regarding reasons for selecting spliced girders and any other special project features. When provided, this information gave valuable insight into the process of design and selection of extended-span or spliced girder bridges. Where possible, information on the performance of the completed structure was obtained. Because of the great variety in types of bridges that have been designed and constructed, some difficulties were encountered in obtaining consistent and meaningful data.

Although the form was titled "Spliced I-Girder Data Collection Form," indicating its primary focus on spliced girder bridges, information was also collected for bridges where other methods were used to extend spans beyond the limits of conventional simple-span prestressed girders, such as shoring for deck placement and continuity for live load.

Use of the data collection form enabled the research team to collect detailed project data in a uniform format for a large number of extended-span and spliced girder bridges and to accomplish the following:

- Facilitate the compilation and organization of significant data on a large body of bridge projects, both domestic and foreign;
- Facilitate classification of projects according to parameters such as by location (state, country, region), chronologically by design/construction year, by splice location (in-span, at piers or both), and by contract method (design/bid/build, design/build, and value engineering change proposals [VECP]);
- Facilitate evaluation of project data to determine the evolution of trends and common practices, both globally and within particular regions;
- Provide insight into challenges in the areas of design, detailing, construction, specifications, and software use; and
- Serve as a springboard for gathering additional and more specific information concerning details and methods used on individual projects of interest.

In some cases, telephone interviews were conducted to obtain additional information about projects for which data collection forms were received. Designer respondents were asked to provide information concerning the type of design software used and their opinions concerning it, whether they thought available specifications addressed all design issues sufficiently, and how they handled time-dependent effects, temperature gradient, composite sections, and stress and camber histories during design. Conversations with owner and on-site owner representative respondents were used to gather information concerning the quality of the design plans, the adequacy of plan details, particular problems during construction, and what aspects they would like to see improved on future jobs. Similar inquiries were made of contractors and precast concrete fabricators to obtain their perspectives and suggestions regarding ways to make a particular construction method more competitive in the future.

Response to the data collection effort was generally good, resulting in the collection of completed data forms, contract plans, plans from VECPs, photographs, other documents, and personal recollections. In some cases, the research team became aware of an existing project but was not successful in obtaining data.

1.4.2 Design Options

Combining information collected from various sources regarding projects and design procedures with their experience, the research team developed a discussion of design options for extending span ranges of precast prestressed concrete girders. These options included both enhanced techniques and spliced girder construction. A prioritized list of these options was then developed for members with and without post-tensioning. To better understand how different options affect designs, a limited comparative design study

using several design options from the prioritized list was conducted for a single bridge typical section by determining the maximum span for different design options.

1.4.3 Issues and Details for Extended Span Bridges

This activity was also based on the compilation and evaluation of information collected from various sources and the experience of the research team. A comprehensive listing of topics to be considered in the design, fabrication, and construction of spliced girder bridges was developed. Several options are presented where available, with advantages and disadvantages given.

1.4.4 Design Examples

Based on the listing of design options, three spliced girder bridges were selected for the development of comprehensive design examples. The examples focused on aspects of design unique to spliced girder construction and illustrate the application of many of the design issues listed in the Design Issues and Details section.

1.4.5 Design Specifications

Design specifications can affect the success of a design methodology significantly. Therefore, the evaluation of the current *LRFD Specifications* was given a high priority in the project.

Early in the project, the existing *LRFD Specifications* were thoroughly reviewed to identify any provisions that may need revision, deletion, and/or reorganization and to identify provisions that may need to be added to the *Specifications* to address issues related to extended-span ranges and spliced girder construction properly. The results of the review were tabulated.

Based on the review of the *Specifications* and the experience of the research team developing the design examples, it was determined that the *Specifications* needed revision. Proposed revisions were prepared to clarify the applicable provisions for spliced girder construction and to improve the organization of the provisions that apply to spliced girder construction. Few new provisions were introduced in the proposed revisions.

1.4.6 Issues Requiring Additional Action

During the course of the research, issues that needed to be addressed for spliced girder construction to advance quickly were identified and compiled.

1.4.7 Design Software and Other Resources

The design of spliced girder bridges can be quite complex. Therefore, the design is generally executed using a computer program or series of spreadsheets. Information was obtained from users and suppliers of several computer programs. Specific information on commercially available software is not provided in this report.

Design resources in the literature and on the web were identified and have been listed. These include design examples for spliced girders and DOT websites that have design manuals, details, or specifications for spliced girder construction. Website links are given rather than repeating the information in this report. The use of website links will enable readers to find current information on the websites rather than information that will rapidly become out of date if repeated in this report.

A literature review was conducted to identify design guides, design examples, project information, and other existing information that could be used by the designer of spliced girder bridges. All cited references and other identified resources are included in the bibliography in Appendix G2.

1.5 REPORT ORGANIZATION

This report presents the results of the work conducted by the research team on NCHRP Project 12-57. Given that the goal of the project was to provide practical guidance to designers regarding the design of longer span precast prestressed concrete bridge girders, the report is organized to make the products of the research easily accessible to those who may use it. This was accomplished by separating the body of the report, which describes and summarizes the design resources, from the resources themselves, which are presented in the appendixes.

The body of this report presents the approach and evaluation of the activities of the project. An overview of the contents of each chapter follows.

Chapter 1 (this chapter) provides an overview of the project objectives and the activities used to achieve the objectives.

Chapter 2 contains the findings of the study. The chapter contains summaries of most of the major activities of the research project as well as prioritized lists of design options and detailed discussions of several items of special interest, such as cost comparisons, post-tensioning and grouting, deck replacement issues, and splicing of haunched pier girder segments. The issues are presented in the same order as the activities are introduced in Chapter 1.

Chapter 3 presents summaries of the applications of the findings that were developed in the research. These include a discussion of the results of the comparative design study, an introduction to the three design examples, and a summary of the proposed revisions to the *AASHTO LRFD Bridge Design Specifications.*

Chapter 4 contains conclusions and issues for additional study or further action. Implementation of the project products is also discussed.

A list of references cited in the body of the report follows Chapter 4. A separate reference listing is provided for each appendix, where required.

The appendixes contain the products and supporting documentation for this research project. In most cases, an appendix is divided into several sections.

Appendix A contains a listing of spliced girder projects and the survey form used to collect the data. A summary with comments on the data is also given.

Appendix B contains a discussion of design options that have been or may be used to extend the span ranges for precast prestressed concrete girders. A prioritized listing of the design options is presented and discussed. A comparative study is included to demonstrate the effect of using different options for a single-span bridge.

Appendix C contains a detailed discussion of issues that need to be considered in the design and construction of spliced girder bridges. An overview of the extensive experience with spliced girder bridges in Florida is provided for historical perspective.

Appendix D contains the three design examples prepared for this project.

Appendix E contains proposed revisions to the *LRFD Specifications* addressing the design of spliced girder bridges. A list of provisions in the *Specifications* that may need revision to enhance design for extended-span precast prestressed concrete bridges is also included.

Appendix F contains a list of issues that need to be addressed to maximize the potential for extending the span ranges of precast prestressed concrete girders.

Appendix G contains information on design software and additional resources for the design of spliced girder bridges and a bibliography.

1.6 OTHER ITEMS

1.6.1 Design Specifications

The *AASHTO LRFD Bridge Design Specifications*, 2nd Edition, with interims through 2003 *(3)* was used as the document governing design for this report, including the design examples. The AASHTO *Standard Specifications for Highway Bridges*, 16th Edition *(16)* may be cited as a reference in some cases.

1.6.2 Terminology

The following terminology is used in this report:

- The *AASHTO LRFD Bridge Design Specifications* cited above are referred to as the "*LRFD Specifications,*" or simply as the "*Specifications.*"
- The AASHTO *Standard Specifications for Highway Bridges* cited above are referred to as the "*Standard Specifications.*"

- The commonly used terminology of "post-tensioning" or "post-tensioned" is used in this report. However, the *LRFD Specifications* depart from this standard terminology by using "posttensioning" and "posttensioned." Where quoting from the *LRFD Specifications*, the usage of the *Specifications* is retained.

- The term "prestressing" is taken to be a general term which includes both pre-tensioning and post-tensioning.
- The terms "beam" and "girder" are used interchangeably.
- The terms "fabricator," "producer," and "precaster" are used interchangeably to indicate the party responsible for the manufacture of precast prestressed concrete girders.

CHAPTER **2**

FINDINGS

The findings of this project address the broad range of design options that can be used to extend span ranges of precast prestressed concrete bridges. However, as discussed in Chapter 1, spliced girder construction was identified as providing the greatest potential for increasing spans and has the greatest need for design guidance, so much of the following discussion is focused on spliced girder construction.

Items appear in this chapter in the same order in which they were introduced in Chapter 1.

2.1 PROJECT INFORMATION

Information was collected to document existing projects in which extended-span precast prestressed concrete girders have been used and to document the experience related to these structures.

2.1.1 Results of Data Collection

Through the process of gathering completed data collection forms and following up with individual respondents, information on between 185 and 190 bridges was obtained from 57 survey participants. Combining these projects with additional projects identified using other means, the research team identified more than 250 extended-span or spliced girder bridges around the world. Table 4 (from Appendix A3) summarizes the number of extended-span bridges for which data were collected by country, state, region, and decade of construction. Additional details for each bridge represented in Table 4 are given in Appendix A3.

In compiling a quantitative list of project data reported, an attempt has been made to eliminate overlap from bridges identified by more than one individual, although each individual's comments have been retained in the database. Appendix A2 contains an evaluation and discussion of the collected data.

2.1.2 Focus on Spliced Girder Construction

The research team determined that data from spliced girder bridges were most significant (compared with other extended-span methods) for the following reasons:

- Other methods for extending span ranges, such as those involving material enhancements or improved girder sections, are generally modifications of current technology and procedures and can be used without significant changes to normal design and construction methods.
- For some of the materials-related issues, such as high-performance concrete and specified density concrete, information is already available or research is underway by other agencies to make information available to designers to assist them in implementing these methods.
- The evolution of experience and necessity has led the industry in the direction of spliced girder construction as a means to achieve significantly longer spans than those previously achieved using single-piece, full-span precast prestressed concrete girders.
- Spliced girder construction requires consideration of a significant number of additional issues with which many designers are not familiar.
- Many owners, designers, and contractors are not familiar with the large number of projects that have been constructed using spliced girders, nor with the great variety of situations and conditions in which they have been successfully used.

Therefore, data collection efforts concentrated on gathering information on spliced girder bridges without neglecting the contribution of other methods in achieving longer spans. A concerted effort was made to collect data on the entire range of extended-span girder bridge projects from the earliest examples [North Africa, 1944*(17)*] to the most recent (several currently under design); from the shortest spans (three segments for a 60-ft-long span in Medicine Hat, AB, built in 1954 and still in service *[18, 19]*) to the longest span (Moore Haven, Florida) with a main span of 320 feet *(20)*; from constant-depth simple-span structures to five-span continuous structures with haunched girders and multi-staged post-tensioning *(20)*; and from a bridge in Pike County, Kentucky *(21)* to ones in Taiwan and New Zealand *(22)*.

2.1.3 Foreign Projects

Data collection for foreign projects was difficult, and information gathered on such projects was limited. Part of the

TABLE 4 List of bridges—summary

Region	State/Province	Total Number of Bridges	1950's	1960's	1970's	1980's	1990's	2000's	Design/Bid/Build	Design/Build	VECP
West		**26**	**0**	**0**	**0**	**1**	**19**	**6**	**4**	**8**	**0**
	AZ	1						1	1		
	CO	17				1	11	5	3		
	UT	8					8			8	
Midwest		**20**	**0**	**0**	**2**	**5**	**8**	**5**	**6**	**0**	**0**
	IL	2			2			0	1		
	IN	4					4				
	KY	3				2	1				
	TN	1						1	1		
	MO	2				2					
	NE	1					1				
	OH	3						3	3		
	MN	1					1		1		
	TX	1				1					
	WV	2					1	1			
Northeast		**23**	**1**	**3**	**3**	**1**	**6**	**9**	**4**	**4**	**0**
	CT	0						0			
	NJ	4						4	2	2	
	NY	12	1	3	2	1	4	1	1		
	MA	4					1	3	1		
	PA	1			1		0			1	
	RI	1						1			
	VT	1					1			1	
Northwest		**41**	**0**	**2**	**3**	**7**	**23**	**6**	**1**	**0**	**0**
	OR	21		2	2	1	13	3			
	WA	20			1	6	10	3	1		
Far West		**20**	**0**	**1**	**6**	**0**	**5**	**8**	**13**	**0**	**1**
	CA	14		1	5		2	6	10		
	NV	3					1	2	3		
	HI	3			1		2				1
Southeast		**51**	**0**	**2**	**3**	**9**	**19**	**18**	**18**	**5**	**5**
	AL	3					3			2	1
	FL	36		2	3	7	14	10	14	2	3
	GA	1					1				
	LA	1						1			
	NC	2						2			
	MS	3					1	2	1	1	1
	SC	1				1	0				
	VA	4				1		3	3		
Canada		**67**	**0**	**0**	**3**	**31**	**24**	**9**	**0**	**0**	**0**
	AB	31			1	16	5	9			
	BC	29				13	16				
	MB	2				1	1				
	ON	4			2	1	1				
	SK	1					1				
Other Countries		**3**	**0**	**0**	**0**	**1**	**2**	**0**	**0**	**0**	**0**
	New Zealand	1					1				
	Taiwan	1				1					
	Bahamas	1					1				
Total		**251**	**1**	**8**	**20**	**55**	**106**	**61**	**46**	**17**	**6**

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problem was a general lack of knowledge concerning the existence or location of such projects in other countries. Historically, different methods of contracting bridge projects (i.e., design/build versus design/bid/build) have been used in European countries as compared with the United States, which means that a different course of action needed to be taken and different sources had to be investigated to gather information concerning projects in these and other countries. Literature searches of foreign projects or construction methods were often not successful because of differences in nomenclature—in part, because of confusion between the U.S. use of the term "spliced girders" and the foreign literature definition of "segmental" to define a particular type of construction. The lack of understanding about what constitutes a "spliced girder" bridge became apparent when engineers from other countries were contacted. After pursuing all known leads, correspondence with post-tensioning personnel and engineers in Switzerland, France, New Zealand, and Japan eventually led to the conclusion that splicing of precast multi-girder bridges is not very common in other parts of the world. Although the Data Collection Form served as a starting point in generating a large inventory of data on domestic projects built using extended-span methods, it was of little value in gathering information on projects in other countries.

2.2 DESIGN OPTIONS FOR EXTENDING SPANS

Design options for extending span ranges of precast prestressed concrete girders are summarized in this section, followed by prioritized lists of the options. The correlation between design options in the prioritized lists and the design options used in the comparative design study is also presented, although the summary of the comparative study appears in Chapter 3.

2.2.1 Design Options for Extending Spans

From the data collected about projects and design procedures, a wide variety of design options were identified that can be used to extend span ranges of precast prestressed concrete girders. Additional design options were developed by the design team by considering the range of possible variables involved in bridge design. Characteristics of each design option are described in Appendix B1, with advantages and disadvantages. An introduction to and brief summary of the information presented in Appendix B1 are presented in the following.

The variety of identified design options is the result of differences in experience, equipment, site constraints, and other conditions.

The design options identified for extending spans vary significantly in degree of complexity, effectiveness, and practicality. It was found that combining several design options may

prove more effective than using the design options alone. Some design options listed may not be feasible in all areas or they may fail to produce a significant economical advantage for a particular set of circumstances. Therefore, designers are encouraged to consult with owners, fabricators, and contractors before implementing extended-span design options to ensure that the use of a selected option will offer the potential for greater economy without introducing unmanageable risk, uncertainty, or adverse effects.

The design options identified are divided into four groups:

- Material-related options,
- Design enhancements,
- Methods utilizing post-tensioning, and
- Spliced girder construction.

A brief discussion of the design options in each group is presented below.

2.2.1.1 Material-Related Options

Design options for extending span ranges related to material properties can usually be implemented with standard design procedures and software.

- **High-Strength Concrete.** High-strength concrete (HSC) has been used successfully to extend the span ranges of precast prestressed concrete bridge girder bridges. It is generally used to the greatest advantage in girders, but may also be used in decks. This option has also been used successfully in conjunction with other design options.
- **Specified Density Concrete.** Specified density concrete (SDC) has been used to reduce design loads to extend span ranges of precast prestressed concrete bridge girder bridges, but has also been used to reduce the shipping weights to facilitate use of longer girders. SDC may be traditional lightweight concrete, or it may be used to reduce the unit weight of the concrete as required for shipping.
- **Increased Strand Size.** The use of a larger strand size at the same strand spacing improves the efficiency of pre-tensioned girders. This design option frequently is combined with HSC to obtain a significant increase in girder spans. Standard strand sizes must still be used.
- **Increased Strand Strength.** Strand producers have been developing strands with strengths of 300 ksi or greater. Increased strand strength, accompanied with a proportional increase in initial strand stress, will increase span ranges for precast prestressed concrete girder bridges. However, revisions to material specifications and research in areas such as transfer and development length must be completed before use of higher strength strand will be approved.

- **Decks of Composite Materials.** In recent years, several concepts have been introduced for using composite materials, typically developed in the defense industry, for the construction of highway bridge decks. Although several important issues remain to be addressed, bridge decks constructed using composite materials may be used to extend span ranges of precast prestressed concrete beams by significantly reducing the weight of the deck structure.

2.2.1.2 Design Enhancements

Several design options for extending span ranges are related to modifications or enhancements of design parameters or procedures. In most cases, these methods can be implemented with standard design procedures and software.

- **Modified Standard Girder Sections.** A wide variety of options exist for modifying existing girder cross sections to extend span ranges of precast prestressed concrete beams. These include the following:
  – Moving side forms in or out to reduce section weight or increase section properties;
  – Increasing the depth of the bottom flange to add a row of strands;
  – Increasing the depth of a girder section for increased section properties;
  – Increasing the width of the top flange to reduce deck forming, improve lateral stability of the girder for handling, and increase section properties;
  – Increasing the thickness of the top flange to improve section properties;
  – Casting some or all of the deck with the girder, which improves efficiency of prestressing and allows for more rapid construction by eliminating deck forming. This type of section is called a "decked bulb tee."
- **New Standard Girder Sections.** Precast prestressed concrete girders have become an economical bridge solution in large part because girder shapes are standardized. However, in recent years, several new girder cross sections have been introduced that generally are more efficient than the existing shapes, but the difference is not large. Given that precasters will have to purchase forms to manufacture new girder shapes, the cost of the forms must be considered when evaluating the benefit of new girder sections.
- **Modify Strand Pattern or Utilization.** Several design options involving strands have been identified. These include the following:
  – Reducing strand spacing to increase the eccentricity of the prestress force;
  – Bundling strands at drape points to increase the eccentricity of the prestress force;
  – Debonding top strands at the center of a girder to improve stress conditions at ends without disturbing stress conditions at the center of the girder (this option also improves lateral stability of long girders); and
  – Transforming strands to improve section properties.
- **Enhanced Structural Systems.** Several design options for increasing span ranges have been identified that involve modifications of or improvements to structural systems. These include the following:
  – Making girders continuous for live loads by placing a continuity diaphragm betweens ends of girders at interior piers;
  – Shoring girders during placement of deck concrete;
  – Fixing the ends of spans at abutments for frame action;
  – Using cantilevered girders to extend spans beyond interior piers;
  – Using pier tables to shorten the span of precast prestressed girders; and
  – Using integral caps to provide a fixed connection between superstructure and substructure for frame action.
- **Enhanced Design and Analysis Methods.** Several design options for increasing span ranges have been identified that involve modifications or improvements to design and analysis methods. These include the following:
  – Improving deck design to reduce required deck thickness and
  – Using refined methods for computing live load distribution to reduce design loads.

2.2.1.3 Methods Using Post-Tensioning

The addition of post-tensioning significantly increases span lengths beyond conventional prestressed girder limits. However, the design and construction of such bridges is significantly more complex. Several design options have been identified that have been used successfully on projects to extend span ranges of precast prestressed concrete girder bridges.

A major disadvantage of adding post-tensioning to a girder is increased complexity in forming and the additional work that must be done in either the plant or field to stress and grout the post-tensioning tendons. Where post-tensioning tendons are anchored in the girder, anchor blocks must be formed to provide additional section width and reinforcement fabricated to resist concentrated loads at the anchorages. If post-tensioning is done at the site, the contractor should hire a specialty subcontractor to perform the tendon installation, stressing, and grouting. This adds complexity to the construction process and may add uncertainty to the construction schedule.

- **Combined Pre- and Post-Tensioning.** Post-tensioning can be effectively combined with pre-tensioning to increase span ranges for single-piece girders or for spliced girders. The post-tensioning may be applied to the final composite cross section where it is most effective.

- **Staged Post-Tensioning.** Staged post-tensioning permits a significant increase in the amount of prestress that can be applied to a girder cross section, which makes possible longer spans. This increase is possible because a portion of the post-tensioning is applied to the bare girder, with the remainder of post-tensioning applied to the completed structure, which also places the deck into compression, enhancing its durability.
- **External Post-Tensioning.** External post-tensioning of girder bridges, where the tendons are stressed after the girders are erected and diaphragms with anchorages and deviation points are installed, is not common in the United States. This method may hold promise for some special situations, but construction appears to be more complex and post-tensioning ducts may be exposed.
- **Longitudinal Post-Tensioning in the Deck over Interior Piers.** Longitudinal post-tensioning in the deck over interior piers has been used in a few projects to counteract the tension that develops in continuous structures over interior supports. Significant issues exist regarding location and protection of tendon anchorages.

2.2.1.4 Spliced Girder Construction

Several types of spliced girder construction and the advantages associated with spliced girder construction have been identified. Given that most of the issues presented in Appendix B1 regarding spliced girder construction are discussed in Section 1.3, they are not repeated here.

2.2.2 Prioritized Lists of Design Options for Extending Spans

From the design options identified, two prioritized lists are presented below. The first list is for design options without post-tensioning and the second list is for design options with post-tensioning and splicing. Prioritization of these lists of design options for extending the span ranges of precast prestressed concrete girders is an uncertain task because of the great variety and number of alternatives available. Comments on use of the prioritized lists are also given to close this section.

2.2.2.1 Design Options Without Post-Tensioning

This prioritized list contains methods that can be used for bridges without the addition of post-tensioning to the structure. These techniques use improved material properties, improved analysis methods, and new construction methods. No spliced girder techniques are included in this list, although some of the methods can be used with splicing to achieve even greater spans.

The list was prioritized considering the benefits of the technique (i.e., the relative increase in maximum span length); the effect of the technique on design or construction; and how

available, acceptable, and understandable the approach is expected to be to the typical owner, designer, and contractor. Comparative analyses were not performed to assist in the prioritization of the list, so the placement in the list is completely subjective.

Using this approach, the prioritized list in Table 5 was developed. All design options included in the "Material-Related Methods" and "Design Enhancements" groups are represented by at least one entry in the table. The highest priority methods are listed first. The advantages and disadvantages of each design option have been identified in Appendix B1.

Some design options appear lower on the list than would be expected by the anticipated benefit. Several factors that resulted in lower rankings are listed below, with examples.

- **Uncertainty.** An example is the use of increased strand strength where the benefit of using 300-ksi strand instead of the standard 270 ksi strand would be similar to using an increased strand size, which is a highly ranked design option. However, the availability and acceptance of the higher strength strand is uncertain, so it is not ranked as a high-potential design option. A second example is the use of composite material for decks, which has only been used for demonstration projects at the writing of this report.
- **Complexity.** Several options introduce significant additional complexity to design or construction. Such options include refined methods for computing live load distribution factors, improved deck design, and the use of cantilever spans or pier tables.
- **Acceptance.** The design specifications or local practice may not allow options such as decreased strand spacing. Although a strand spacing of 1.75 inches is allowed by the *LRFD Specifications* for 0.5-inch-diameter strands, the cost to implement this closer spacing would be prohibitive for fabricators. It would be better to use larger strand at the current 2.0-inch spacing.

2.2.2.2 Design Options with Post-Tensioning and Splicing

Post-tensioning and splicing appear to have the greatest potential for extending span ranges of precast prestressed concrete girder bridges. The benefits of using post-tensioning and splicing have been demonstrated many times, as indicated by the data that have been collected on existing spliced girder bridges. These design options are significantly more complex than are those that do not employ post-tensioning and splicing. Design and construction details can have a significant effect on the success and economy of a spliced concrete girder bridge project.

A separate prioritized list, shown in Table 6, was developed for these methods to emphasize these design options

TABLE 5   Prioritized list of alternatives without post-tensioning

Highest Rank	1	Deck cast with girders (decked bulb tee)
	2	Touch shoring under girders during placement of deck
	3	High-strength concrete girder
	4	Deeper sections in existing girder families
	5	Girders made continuous for composite loads
	6	Increased strand size
	7	Lightweight concrete deck
	8	Lightweight concrete girder
	9	Bundle strands drape points
	10	Deeper bottom flange to add a row of strands
	11	Top strands debonded
	12	Increased beam width (side forms moved apart)
	13	Increased top flange width
	14	Transformed strands
	15	High-strength concrete deck
	16	Increased top flange thickness
	17	Integral cap with fixed connection between superstructure and substructure
	18	Reduced strand spacing
	19	Refined methods for computing live load distribution factors
	20	Cantilever spans
	21	Pier tables
	22	Improved deck design
	23	Ends of spans fixed at abutments
	24	Decks composed of composite materials
Lowest Rank	25	Increased strand strength

TABLE 6   Prioritized list of alternatives with post-tensioning and splicing

Highest Rank	1	Pier segments with end segments and drop-in spans
	2	Simple-span, spliced girder bridges
	3	Multiple segments in each span with splices at piers
	4	Full-span girders with splices at piers
	5	Staged post-tensioning of simple-span, full-length girders
	6	Combined pre- and post-tensioning in simple-span, full-length girders with post-tensioning applied after deck is placed and cured
	7	Combined pre- and post-tensioning in simple-span, full-length girders with post-tensioning applied to bare girder
	8	Longitudinal post-tensioning in the deck over interior supports
Lowest Rank	9	External post-tensioning

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and to recognize their significantly greater complexity compared with the simpler methods shown in Table 5. Again, comparative analyses were not performed. However, the method used to extend the span range of girders for a given project is generally selected based on consideration of specific project criteria rather than a quantitative comparison.

The prioritized list of design options for these design options shown in Table 6 was developed using the same approach used to develop Table 5. All design options included in the "Methods Utilizing Post-Tensioning" and "Spliced Girder Construction" groups are represented in Table 6. The highest priority methods are listed first. The advantages and disadvantages of each design option have been identified in Appendix B1.

For the design options in this list, complexity was not considered as significant a factor in the ranking as it was for the list in Table 5. The most complex design options were ranked the highest, because all of the design options in this group require greater complexity, but the greater complexity results in greater benefit.

2.2.2.3 Comments on Prioritized Listings of Design Options

The prioritization of design options in Tables 5 and 6 was performed without a quantitative basis. Furthermore, the effect of each design option was considered individually. A significant finding of the comparative design study was that combinations of design options may provide significant advantages, possibly well beyond the advantage gained from individual design enhancements. It is also the experience of the research team that specific project characteristics, such as girder cross section, girder spacing, or span lengths, significantly affect the relative improvement achieved using different design options. One example is that bundling of strands does not provide a significant improvement in the comparative design study, although it has been shown to be very beneficial in other designs.

Therefore, designers are urged to consider these lists not as indicating tested and definite rankings, but as approximate and subject to change when the effect of specific project requirements and the combination of different design options is considered. Designers should evaluate the benefits for the listed design options by performing their own comparative design studies using specific project requirements and considering any limitations on available design options that may apply because of local conditions or practice. Practical combinations of design options should also be considered to gain the greatest benefit.

When performing a comparative design study, designers should consider the effect of relatively small incremental increases in quantities such as the girder concrete strength rather than making a large increase in the quantity. This is encouraged because, in some cases, a smaller increase in concrete strength may provide essentially the same benefit as a larger increase because of limitations of the capabilities of the girder cross section or some other design parameter.

2.2.3 Comparative Design Study

The design options considered in the comparative design study, which is discussed in Chapter 3, were taken from the prioritized lists of Tables 5 and 6. The correlation between the design options in the comparative design study and the prioritized design options is shown in Table 7.

2.3 ISSUES AND DETAILS FOR EXTENDED-SPAN BRIDGES

Using information collected from various sources and the experience of the research team, a comprehensive listing of issues that should be considered in the design, fabrication, and construction of spliced girder bridges was developed. When available, several options are presented along with advantages and disadvantages. The complete results of this portion of the study are presented in Appendix C1. Appendix C2 is included to provide a historical perspective on the development of spliced girder construction in Florida.

TABLE 7   Correlation between comparative design study cases and prioritized listings

No.	Description	Priority Ranking
1	Base Design	-
2	Increased f'_{ci}	-
3	High Strength Concrete (HSC)	3
4	Lightweight Concrete Deck	7
5	Lightweight Concrete Beam	8
6	Lightweight Concrete Deck & Beam	7 & 8
7	Deeper Bottom Flange	10
8	Increased Beam Width	12
9	Bundled Strands	9
10	Top Strands Debonded	11
11	0.6"φ Strands	6
12	0.6"φ Strands and HSC	3 & 6
13	Decked Bulb Tee	1
14	Touch Shoring	2
15	Added Post-Tensioning	6

All Priority Rankings shown are for design options appearing in Table 5 "Prioritized List of Alternatives without Post-Tensioning" except for Design Case 15, which appears in Table 6 "Prioritized Listing of Alternatives with Post-Tensioning and Splicing."

Several issues of special significance are discussed below. These discussions are not repeated in Appendix C1, although some additional information may be provided on the issues.

2.3.1 Cost Data

Detailed construction cost information for post-tensioned, spliced girder bridges is somewhat difficult to obtain. Most states only track cost data on a project basis, or at best, separate bridge and roadway costs. Detailed cost tabulations seem to disappear quickly once a bridge has been constructed. Even when bid tabulations are available for a particular project, it is often difficult to determine which pay items represent the spliced girder unit and which ones represent the approach units. Other items, such as concrete and reinforcing steel, may be lumped together without regard to their function in the bridge, so it is hard to distinguish superstructure costs from substructure costs. Alternative project delivery methods, such as design/build and VECPs, complicate matters further when trying to isolate costs for a certain portion of the bridge.

Nonetheless, the research team was able to gather cost data from a sampling of post-tensioned spliced girder bridges around the country, including a significant number of bridges in Florida. The data collected in Florida is sufficient to characterize the state of practice and assign costs for estimating purposes in the Southeast region of the country. However, because construction costs differ significantly between regions of the country (e.g., labor costs, materials costs, and special design requirements), between different methods of project delivery (e.g., design/bid/build, design/build, and VECP), and as the result of local contracting practices (e.g., expertise, experience, and available equipment), it is difficult to assign meaningful cost parameters that are universally relevant.

2.3.1.1 Available Cost Data

The most relevant parameter for gauging the competitiveness of a spliced girder bridge alternative and for estimating construction costs on future projects is the square foot (of bridge deck) cost for the spliced girder superstructure. Using this parameter greatly reduces the potential for skewed results because of unusual substructure/foundation conditions or from unrelated approach structure costs. The square foot superstructure cost can be used to compare various superstructure types with similar span lengths. Because different superstructure alternatives impose different loads on substructure units, the cost of the substructure should still be considered when comparing bridge alternatives.

Table 8 shows square foot superstructure costs obtained during the data collection phase of the project for a number of spliced girder bridges. Data shown as "Construction Year $" were extracted directly from project cost data or respondent data collection forms and represent dollar values at the time of construction. Data shown as "2002 $" have been indexed to represent Year 2002 dollar values using the cost indexing methods described in "Civil Works Construction Cost Index System (CWCCIS)"*(23)*.

In general, the square foot superstructure costs shown in Table 8 include the following items (if applicable), unless noted otherwise:

- Deck, buildup and diaphragm concrete (volume);
- Deck, buildup and diaphragm steel (weight);
- Prestressed girders (length);
- Traffic railing (length);
- Post-tensioning tendons (weight of strand);
- Bridge floor grooving (area);
- Reinforced elastomeric pads (volume) or multi-rotational bearings (each);
- Expansion joints (length); and
- Mobilization (percentage of project lump sum amount).

To facilitate the comparison of similar bridges, additional features are presented for the bridges listed in Table 8, namely:

- The configuration of the bridge (e.g., three-span river crossing; two-span structure with integral piers; single-span urban interchange; long, multi-unit viaduct structure over land; long, multi-unit water crossing);
- The maximum span length used;
- The type of girders used (e.g., bulb tees, AASHTO girders, open top tub girders, closed box girders); and
- The number and uniformity of spliced girder spans (i.e., quantity of girder, repetition).

Other data from past projects that may be relevant for evaluating costs on future projects include bid prices for competing alternative structure types, such as steel plate or box girder bridges or concrete segmental bridges. Where this information is available, it has also been shown in Table 8.

2.3.1.2 Preliminary Cost Estimating

When developing a preliminary cost estimate for a post-tensioned spliced girder bridge alternative, it may seem logical to begin with an estimate for a conventional prestressed girder bridge to which is added the supplemental cost of post-tensioning. However, this type of incremental cost approach is rarely valid. In most instances where a spliced girder bridge is being considered, the primary benefit of such an alternative comes from being able to extend the span lengths beyond the capabilities of a conventional pre-tensioned girder bridge. For a post-tensioned spliced girder alternative to be competitive against a conventional girder design, the cost of splicing and post-tensioning must be offset by savings in substructure costs due to longer spans and/or in girder costs by reducing the number of girder lines or some other significant cost item. Alternatively, a spliced girder superstructure may be the preferred choice when compared with other viable

TABLE 8 Comparative cost data

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
US 90 Over East Pascagoula River	Pascagoula, MS	2002	$3,350,362	87,954	$38.09	$39.27	250	10,206	3-span over river; 72" bulb tee w/haunched pier	Calculated from bid tabs, inc. mobilization
SR 10 Over the Escambia River	Pensacola, FL	2003	$2,140,310	27,494	$77.85	$77.85	210	2,832	3-span over river; modified 78" bulb tee w/haunched pier	Calculated from bid tabs, inc. mobilization
Fuller Warren Bridge over St. John's River	Jacksonville, FL	2001		102,700	$39.53	$41.19	250	10,400	3-span over river; modified 78" bulb tee w/haunched pier	Calculated from owner info., inc. mobilization; all 11 bidders chose concrete alt. over steel
Moore Haven Bridge / US27 over Caloosahat-chee River	Moore Haven, FL	2000					320	3,175	3-span over river; modified 78" bulb tee	VECP saved $621,000 over steel superstructure
SR 312 Bridge Over Intracoastal Waterway	St. Augustine, FL	1999	$1,235,103	27,305	$45.23	$48.81	250	2,070	3-span over ICWW; 78" bulb tee w/haunched pier	Calculated from bid tabs, inc. mobilization
Gandy Bridge WB / SR-600 Over Tampa Bay	Tampa / St. Petersburg, FL	1998		22,500	$50 for simple-span 78" BT		235	29,600	3-span over water; Type IV simple spans, but VECP w/ 78" BT simple spans was used	FDOT cost data, inc. mobilization

[1] 2002 costs are derived from Construction Year costs by factoring the latter value by the ratio of the 2002 cost index over the index from the fiscal year prior to the completion of construction. Index values are from the "Civil Works Construction Cost Index System (CWCCIS)", Manual EM 1110-2-1304, U.S. Army Corps of Engineers, March 31, 2000 w/Tables Revised as of March 31, 2003.
[2] Left-aligned values indicate two- to five-span structures, usually over water.
 Center-aligned values indicate long repetitive-span structures, usually over water.
 Right-aligned values indicate single-span structures.
[3] Data derived from two different sources resulted in vastly different average costs for the same structure.

TABLE 8   (Continued)

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
MacArthur Causeway Bridge / US-41 Over ICWW	Miami, FL	1997		290,000	$30.00	$33.54	145	6,000	Approx. 15 spans x 145' over water; 72" bulb tee; Span-by-span segmental alternative (no bidders)	FDOT cost data, inc. mobilization
Flagler Beach Bridge EB/WB / SR-100 Over ICWW	Flagler Beach, FL	1997		51,000	$50.00	$55.90	230	23,000	3-span over ICWW; 72" bulb tee w/haunched pier	FDOT cost data, inc. mobilization
Vilano Beach Bridge / SR-A1A Over ICWW	St. Augustine, FL	1996		204,000	$30.00	$38.23	148	23,000	Approx. 26 spans x 148' or 137' over water; 72" bulb tee	FDOT cost data, inc. mobilization
Merrill Barber Bridge NB/SB / SR-60 Over ICWW	Vero Beach, FL	1995		376,000	$33.00	$38.45	142	38,200	27spans x 142' over water, 72" bulb tee	FDOT cost data, inc. mobilization
US-17/92 Bridge Over St. Johns River at Lake Monroe [3]	Seminole / Volusia Cos., FL	1994	$1,752,250	44,243	$39.61	$47.57	200	4,680	3-span over river; 72" bulb tee w/haunched pier	Calculated by respondent; mobilization probably not included

[1] 2002 costs are derived from Construction Year costs by factoring the latter value by the ratio of the 2002 cost index over the index from the fiscal year prior to the completion of construction. Index values
   are from the "Civil Works Construction Cost Index System (CWCCIS)", Manual EM 1110-2-1304, U.S. Army Corps of Engineers, March 31, 2000 w/Tables Revised as of March 31, 2003.

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   Right-aligned values indicate single-span structures.

[3] Data derived from two different sources resulted in vastly different average costs for the same structure.

TABLE 8 **(Continued)**

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
US-17/92 Bridge Over St. Johns River at Lake Monroe [3]	Seminole / Volusia Cos., FL	1994	$1,752,250	44,243	$39.61	$47.57	200	4,680	3-span over river; 72" bulb tee w/haunched pier	Calculated by respondent; mobilization probably not included
US-17/92 Bridge Over St. Johns River at Lake Monroe [3]	Seminole / Volusia Cos., FL	1994		44,200	$52.00	$62.45	200	4,650	3-span over river; 72" bulb tee w/haunched pier	FDOT cost data, inc. mobilization
Highland View Bridge / SR-71 Over ICWW	Port St. Joe, FL	1993		30,200	$40.00	$50.12	250	3,200	3-span over ICWW; 72" bulb tee	FDOT cost data, inc. mobilization
Edison Bridge SB / US-41 Over Caloosahat-chee River	Ft. Myers, FL	1994		298,000	$27.00	$32.43	143	27,300	32 spans x 143' over water; 72" bulb tee	FDOT cost data, inc. mobilization
Edison Bridge NB / US-41 Over Caloosahat-chee River	Ft. Myers, FL	1992		304,000	$27.00	$34.72	143	30,700	36 spans x 143' over water; 72" bulb tee	FDOT cost data, inc. mobilization; Segmental alternative: 1 of 8 bids (25% higher)

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TABLE 8   (Continued)

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
US-331 Over Choctaw-hatchee Bay	Destin/Pana-ma City, FL	1990		1,127,000	$30.00	$40.23	200	1,110,000	42 spans x 143' & 160'-200'-160' main unit over water; 72" bulb tee	FDOT cost data, inc. mobilization
Howard Frankland Bridge / I-275 Over Tampa Bay	Tampa to St. Petersburg, FL	1990		1,127,000	$20.00	$26.82	143	1,110,000	111 spans x 143' over water; 72" bulb tee	FDOT cost data, inc. mobilization; Alternatives: steel plate girders / segmental box (5% higher)
Eau Gallie Bridge / SR-518 Over Indian River	Melbourne, FL	1988		263,900	$20.00	$28.88	145	25,900	20 spans x 145' over water; 72" bulb tee	FDOT cost data, inc. mobilization
Trout Creek Under-crossing / I-80	Truckee, CA	2003	$1,090,000	15,515	$70.25	$70.25	145	2,030	145' single-span over road; 84" bulb tee	Probably includes abutment costs but no mobilization (Caltrans practice)
Firestone Boulevard Bridge over Los Angeles River	Southgate, CA	2000	$850,000	11,825	$75.88	$75.91	132	1,892	4-span over river, 72" bulb tees	Probably includes abutment costs but no mobilization (Caltrans practice)

[1] 2002 costs are derived from Construction Year costs by factoring the latter value by the ratio of the 2002 cost index over the index from the fiscal year prior to the completion of construction. Index values
  are from the "Civil Works Construction Cost Index System (CWCCIS)", Manual EM 1110-2-1304, U.S. Army Corps of Engineers, March 31, 2000 w/Tables Revised as of March 31, 2003.
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TABLE 8   (Continued)

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
Via de la Amistad Undercross-ing	San Diego, CA	1999	$440,000	6,150	$71.54	$77.20	148.17	444	148' single-span w/50 deg. skew; tub girders	Includes "assumed 5' deep" abutment costs (actual abutments 22' deep)
South 38th St / I-5 Interchange	Tacoma, WA	2001	$1,500,000	35,100	$42.74	44.53	164	1,950	2-span tub girders w/integral piers	Lump sum superstructure bid, no mobilization
Trent River Bridge No. 3078 - Inland Island Highway	Courtenay, BC	1998		28,300				2,053	3-span w/int. piers over river; BC Type 6 I-girder	Steel plate girder (price was $300K over $1.8M winning bid)
Duke Point Underpass No. 3121 - Trans Canada Highway No. 1	Nanaimo, BC	1997		15,325			124.67	455	3-span highway interchange; BC 54" U-girder	Twin CIP boxes (price was $200K over $1.0M winning bid)
Nile Creek Bridge	Vancouver, BC	1996		39,840			174	3,984	3-span over creek; 90" bulb tee	Steel plate girder price was CAD-$500K (US$364K) over CAD$5.5M (US$4.0M) winning bid
King George Flyover No. 2908 - King George Flyover	Surrey, BC	1990		13,850			141	734	3-span highway interchange; BC 54" U-girder	Steel plate girder (price was $125K over $1.2K winning bid)

[1] 2002 costs are derived from Construction Year costs by factoring the latter value by the ratio of the 2002 cost index over the index from the fiscal year prior to the completion of construction. Index values are from the "Civil Works Construction Cost Index System (CWCCIS)", Manual EM 1110-2-1304, U.S. Army Corps of Engineers, March 31, 2000 w/Tables Revised as of March 31, 2003.

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TABLE 8 (Continued)

Bridge Name	Location	Comp-letion Year	Spliced Girder Superstructure Total Cost	Spliced Girder Bridge Deck Area (SF)	Spliced Girder SF Superstructure Cost (ConstructionYear $)	Spliced Girder SF Superstructure Cost (2002 $)[1,2]	Max Span Length (ft)	Total Quantity of spliced girders	General Bridge Configuration	Cost Comparison w/ Alternative; Cost Data Comments
Esker Overhead No. 2928 - Yellowhead Highway 16	Terrace, BC	1990	$230,000 (girders)	6,600			164.04	644	164' simple span over RR tracks; BC Type 7 I-girder	Steel plate girder (price was $100K over $800K winning bid)
Husky Entrance Road Overpass	Milton, VT	1998	$709,200	5,950	$119.19	$129.68	140	700	140' single-span over RR track & access road	Details of cost data unavailable
Cranston Viaduct (twin bridge), Rt. 10 over Amtrak R/W	Cranston, RI	2002	$4,750,000	50,000	$95.00	$97.95	175	4,840	4-span over RR tracks; modified 82" bulb tee	Cost data extrapolated by reviewer, probably does not include mobilization
US 231 over the White River	Indiana	1993?	$1,902,560	67,908	$28.02	$35.10	133.75	6,500	9 spans x 133.75' + 2 Type IV simple spans over river	$2,784,041 / 67,908 SF=$41.00/SF ($944,330 total savings on bridge)
I-80 over Cuyahoga River Valley	Summit Co., OH	2004		113,700			200	12,600	5-span over river valley; 102" modified bulb tee	Steel plate girder (price was $1.432M over $38.543M winning bid)
I-80 over Cuyahoga River Valley	Auckland, New Zealand	1997	NZ$2,200,000	21,100	NZ$104.30 (appr. US$59.50)	(appr. US$66.50)	138	3,480	4-span over highway; 71" bulb tee	Details of cost data unavailable

[1] 2002 costs are derived from Construction Year costs by factoring the latter value by the ratio of the 2002 cost index over the index from the fiscal year prior to the completion of construction. Index values are from the "Civil Works Construction Cost Index System (CWCCIS)", Manual EM 1110-2-1304, U.S. Army Corps of Engineers, March 31, 2000 w/Tables Revised as of March 31, 2003.

[2] Left-aligned values indicate two- to five-span structures, usually over water.
Center-aligned values indicate long repetitive-span structures, usually over water.
Right-aligned values indicate single-span structures.

[3] Data derived from two different sources resulted in vastly different average costs for the same structure.

superstructure options capable of achieving the desired span length, such as steel girders or concrete segmental box girders. In either event, it is difficult to generate valid cost estimates for spliced girder bridges based on general cost data or "rules-of-thumb" derived from pre-tensioned girder bridges.

The square foot cost data presented in Table 8 can be used to develop preliminary cost estimates for post-tensioned spliced girder bridges. In order for the estimates to be reasonable, the following guidelines should be adhered to:

- Because different states and owner agencies use different methods of tracking project costs, it is important to understand which items are included in a bridge cost estimate. Some state DOTs include abutment costs in their superstructure cost summaries; others may include bridge bearings, expansion joints, or wearing surfaces.
- The historic cost data should come from a structure that is similar in span length, span configuration, and project size to the structure being designed. The uniqueness or repetitiveness involved in the construction of a particular bridge unit affects the average cost. For example, the design of a three-span, spliced girder unit over a river should be compared with a similar three-span structure, whereas a long, repetitive span bridge over open water should be compared with a bridge of similar length. A single-span bridge with approach roadways on fill at either end should only be compared with another single-span bridge with a similar span length and geometric conditions.
- Where a large body of historic cost data exists for a particular type of spliced girder structure (e.g., a three-span channel unit), the average of values near the median of the cost distribution should be used unless special conditions warrant otherwise. Individual bridge costs that are significantly higher or lower than the main distribution of data for a particular bridge configuration should be disregarded as anomalies. Where only a few sample projects exist for a particular bridge type, the estimator should make a special effort to ensure that these projects offer a valid comparison with the bridge being designed.
- Project cost estimates should be based on past projects in the same geographic region whenever possible. Different regions of the country are characterized by specific construction practices, material costs, and labor rates. Therefore, construction costs will differ from region to region. Despite these differences, it is possible to extrapolate cost data from some regions to other regions based on regional cost indices. In addition to indexing costs on a year-to-year basis, CWCCIS also presents "state adjustment factors," which break down labor, material, and equipment costs for the purpose of comparing construction costs between different states. These adjustment factors can be used satisfactorily in many situations. Nonetheless, it is important to ensure that the necessary equipment, manufacturing capabilities, and expertise exist in a particular region when assessing the viability of a spliced girder bridge alternative.

- Superstructure costs should never be evaluated independently of substructures costs, or more importantly, total project costs. The proposed spliced girder alternative must provide an overall project benefit in order to be competitive.

Despite the documented advantages of post-tensioned spliced girder bridges, as evidenced by the large number that have been constructed across the country, there are still regions of the country where contractors and/or owners are resistant to their use. In these areas, it is very difficult to assign meaningful costs to an alternative that may not attract any bidders. Some state DOTs have designed spliced girder bridges with the result being that all the contractors bid the other alternative, largely because the contractors were not familiar with spliced girder construction. Owners in this situation may find it necessary to prevent alternative structure types from being used on the first few projects until contractors develop a certain level of familiarity and expectation. The long-term benefit should be increased competition with greater contractor participation on future projects involving spliced girder alternatives.

2.3.2 Deck Replacement

In many northern and mountainous areas of the country, corrosion of deck reinforcement resulting from the use of deicing chemicals and multiple freeze/thaw cycles has caused serious premature deterioration of many decks. Since the supporting girders remain intact and serviceable, many decks have been completely removed and replaced. Because of the widespread deterioration of decks, the complete removal and replacement of decks has come to be expected as a future maintenance procedure for bridges in some areas. Therefore, many owners now require bridges to be designed so that the decks can be completely removed and replaced at some point in the future when the anticipated deterioration occurs.

However, for bridges where post-tensioning tendons have been stressed after the deck has been cast, the complete removal of the deck in the future has significant consequences for the bridge. The removal of the prestressed area of the deck, and its associated dead load, may lead to significant overstress or even possible failure in the girders. Furthermore, the precompression that existed in the original deck is lost, so the replacement deck is not expected to have the same life expectancy as the original precompressed deck.

In regions where deck replacement is typically considered in bridge design, there are three basic approaches to address this issue for spliced girders:

1. Require design for deck removal and prohibit stressing of post-tensioning tendons with the deck in place.
2. Require design for deck removal but allow stressing of post-tensioning tendons with the deck in place.
3. Allow stressing of post-tensioning tendons with the deck in place but use alternate deck protection or reha-

bilitation measures to avoid complete removal of deck during service life of the structure.

Each of these approaches is discussed below.

2.3.2.1 Require Deck Replacement and Prohibit Stressing of Deck

The approach of requiring complete removal and replacement of the deck and prohibiting the stressing of post-tensioning tendons with the deck in place could be considered as the conventional approach, because it does not differ from other conventional types of composite construction. Because the post-tensioning force was only applied to the girders, the removal of the deck has little or no effect on the girders. If the stresses were within allowable limits during construction, they should remain within the limits during deck removal and replacement. However, it is recommended that stress conditions at deck removal be evaluated since additional prestress losses have occurred.

Although removal of the deck does not affect the girder significantly, prohibiting stressing of the post-tensioning tendons with the deck in place does have a significant detrimental impact on the efficiency of the bridge design. In most cases, a design with post-tensioning applied only to the girder will require an increased quantity of post-tensioning (more strands or tendons), an increased girder depth, added girder lines, or other design enhancements to achieve the same span as a design in which some or all of the post-tensioning is applied to the girder with the deck in place.

A second significant disadvantage to this approach is that the deck is not precompressed. Without precompression of the deck from post-tensioning, the deck can be expected to experience transverse cracking similar to other conventional bridges. This cracking may be a significant contributor to the deterioration of the deck. If the deck is in place when post-tensioning tendons are stressed, the resulting precompression of the deck reduces or eliminates the transverse deck cracking, thus improving the durability of the deck.

Although it is generally possible to design spliced girders for complete deck removal and replacement if the girders are post-tensioned before the deck concrete is placed, this approach results in reduced efficiency and economy in design and reduced durability of the deck.

2.3.2.2 Require Deck Replacement and Allow Stressing of Deck

When an owner requires spliced girder bridges to be designed for the future complete removal and replacement of the deck but allows the stressing of post-tensioning tendons in the girders with the deck in place, the designer must show a strategy on the contract drawings for removal of the deck while maintaining stresses in the girders within allowable limits. The plans should state any special conditions required to accomplish the complete deck removal and replacement, which may include items such as providing a deck removal sequence, installation of temporary supports, installation of ballast or counter weights, and installation of temporary external prestressing. In most cases, the spliced girder design will have to be altered from the optimum design in order to make future deck removal feasible.

Allowing the deck to be present when some or all of the post-tensioning tendons are stressed precompresses the deck. This precompression is expected to extend the life span of the original deck by preventing or limiting transverse cracks in the deck that may hasten the deterioration of the deck. However, the replacement deck will generally not be precompressed, so it cannot be expected to be as durable as the original deck.

Spliced girders may be designed for complete deck removal and replacement when the original deck was precompressed by stressing the post-tensioning tendons in the girders after the deck concrete was cast and cured. However, this approach results in significant complications during the deck replacement, may reduce the efficiency of the spliced girder design, and is expected to reduce the durability of the replacement deck.

2.3.2.3 Allow Stressing of Deck and Use Alternate Deck Protection and Rehabilitation Measures

Several options for extending and maintaining the life of the bridge deck have been developed that allow the optimal design of spliced girder bridges, including the stressing of post-tensioning tendons with the deck in place, without requiring the complexities and inefficiencies of complete deck removal. Implementation of these alternate strategies greatly enhances the potential for spliced girder construction.

The alternate strategies to complete deck replacement are based on improving the durability of the decks through increased resistance to corrosion and the resulting deterioration. These alternate strategies include the following:

- Precompress the deck concrete with post-tensioning after the deck is poured.
- Use highly durable high-performance concrete (HPC) for decks.
- Include an "integral" overlay with the initial deck placement. This additional thickness of deck concrete can be removed and replaced as chlorides penetrate the concrete.
- Add a non-structural sacrificial concrete overlay on the deck. This overlay can be removed and replaced if conditions indicate an increasing potential for initiation of corrosion, but the structural deck is not affected.
- Add an asphaltic concrete pavement (ACP) overlay with a continuous waterproof membrane system. This approach has been widely used in Europe *(24)*. The long-term performance of the waterproof membrane is critical to the success of this system, because the ACP allows penetration of water and salt.

- Remove only the top cover of the deck as chlorides penetrate the concrete (also known as "partial deck replacement"). The concrete cover is removed to the top layer of reinforcement; then the concrete cover is replaced. This does not have a significant effect on the structure even if the deck was present when post-tensioning tendons were stressed.
- Transversely post-tension deck to reduce or eliminate potential longitudinal cracks.
- Use other measures, such as corrosion-inhibiting admixtures in the concrete, protective coatings on steel reinforcement, or alternate deck reinforcement materials.

Using one or more of these alternate strategies, it appears possible to design spliced girders to provide the desired durability so that complete replacement of the deck is unnecessary. This will allow for the greatest economy of design for spliced girder bridges, including the application of post-tensioning after the deck is cast to provide precompression in the deck.

2.3.2.4 Example of Alternate Deck Protection Measures with Stressing of Deck

An example of an approach where alternate deck protection measures are used is found in Section 9.17 of the New York State DOT *Bridge Manual (25)*. This section, which addresses requirements for post-tensioned spliced girder designs, recognizes the benefit of precompressing the deck by stating the following:

> Whenever possible, part of the longitudinal post-tensioning shall be applied after the hardening of the deck concrete so that net tension on top of the deck surface is less than or equal to the modulus of rupture as per section 8.15.2.1.1 of the *Standard Specifications for Highway Bridges* for the life of the bridge at service load limits, with effects of creep and shrinkage of deck concrete also considered.

The New York State DOT *Bridge Manual (25)* also discusses deck replacement and durability in Section 9.4.4, which addresses segmental precast box girders. This section recognizes the difficulty of requiring deck removal for replacement:

> Since the deck slab is an integral part of the box girder system, the complete replacement of the deck is nearly impossible. To reduce this risk, the structure should be designed so there is no tensile stress at the top surface of the segment under service load conditions both including and excluding time dependent effects.

Although the *Bridge Manual* expects segmental box girders to have decks post-tensioned in both directions, a feature not generally found in spliced girder bridges, the reasoning used in this section could be applied to spliced girder construction as well. The requirements are stricter for segmental construction, because no tension is allowed in the deck.

Other sections of the New York State DOT *Bridge Manual (25)* address additional issues related to the durability of the deck.

2.3.2.5 General Issues Related to Deck Removal

Where the deck must be completely removed for replacement, care must be taken to avoid distress in diaphragms or girders due to differential movements of girders as the deck is removed. If the deck removal is staged and the deck will be left on part of the structure, the designer should evaluate whether a portion of any diaphragm connecting the girders should be removed to allow relative movement between the two parts of the bridge. The diaphragm connection would then be restored when the deck closure is poured after the deck has been replaced on both sides of the structure.

2.3.2.6 Conclusion

The efficiency of spliced girder designs is impaired by the requirement that a bridge must be designed to allow the complete future removal and replacement of the deck. Alternate strategies for extending the service life of bridge decks are available that may be used instead of relying on complete deck replacement. Additional studies should be conducted to demonstrate the effect of the three approaches discussed above on spliced girder designs.

2.3.3 Post-Tensioning and Grouting

Corrosion of prestressing steel has always been an area of concern for prestressed concrete bridges. However, over 50 years of performance data clearly demonstrate that corrosion of prestressing steel is not a significant issue in any type of prestressed concrete bridge. The concern about corrosion is heightened for post-tensioning tendons and anchorages, which appear to have a greater potential for corrosion when compared with pretensioned strands for several reasons:

1. Strands are exposed within ducts for several days prior to grouting. During this period, the strands cannot be inspected.
2. Ducts are grouted after the tendons are stressed, but the quality of grouting cannot be determined along the length of the tendon.
3. Anchorages are encased after grouting, but may be subject to infiltration by water.
4. Where metal ducts are used, the duct can become the site where corrosion is initiated.

Several instances of problems with corrosion in post-tensioning tendons have recently been discovered in Florida and other states. These discoveries have caused the industry

to reexamine practices. The problems have highlighted the need for improved grouting materials and procedures, and significant progress has been made in addressing the concerns. The cost of implementing improved practices during design and construction is small compared with the costs of repairing grouting deficiencies. Therefore, the best possible practices should be used and specified during design and construction.

The following discussion provides historical background to current grouting concerns and focuses on issues related to post-tensioning tendons used in spliced girder construction.

2.3.3.1 Historical Background for Current Concerns

Corrosion has not been a significant problem with prestressed concrete bridges, either pre- or post-tensioned. No known structural failures of bridges in the United States have been attributable to corrosion of post-tensioning tendons. However, a few isolated, but well-publicized, instances of tendon corrosion have occurred in both the United Kingdom and the United States. In the United Kingdom, the failure of the Bickton Meadows footbridge in 1967 and the collapse of the Ynys-Y-Gwas Bridge in 1985 resulted in a ban on the construction of post-tensioned bridges in the United Kingdom from 1992 to 1996. The ban was subsequently lifted based on recommendations published in Concrete Society Technical Report No. 47, "Durable Bonded Post-Tensioned Bridges" *(26)*.

In the United States, problems with a few grouted post-tensioned bridges have recently been discovered, most notably in Florida, a state with more than 70 major post-tensioned bridges *(27)*. Inspections beginning in 1992 of the post-tensioned segmental box girder bridges in the I-595/I-75 interchange near Fort Lauderdale routinely reported water leakage through epoxy joints in the deck and efflorescence coming from the top slab continuity tendons. Additional investigations revealed one 300-foot-long top slab continuity tendon substantially without grout. During the spring of 1999, a corrosion-related failure of an external tendon was found in the Niles Channel Bridge, a low-level segmental box girder bridge in the Florida Keys. An investigation by the Florida Department of Transportation (FDOT) indicated that cyclical recharge of water in a grout void in the anchor head was the primary cause of the tendon corrosion. Water contaminated with salt spray leaked through the expansion joints and ran down the inside faces of the segment diaphragms onto the anchorages. In August 2000, corrosion-induced failures of one external tendon and part of a second external tendon were found in the Mid-Bay Bridge, a post-tensioned segmental box girder bridge near Destin, Florida. The failure of one of the tendons involved corrosion of strands at the anchor head, similar to the Niles Channel Bridge. Failure of the other tendon was traced to a breach in the duct itself, which permitted access to moisture and the development of isolated corrosion. In September 2000, corrosion damage was found in two of the four legs of vertical looped tendons in a seg-

mentally constructed pier on the Sunshine Skyway Bridge over Tampa Bay, Florida *(28)*. The corrosion resulted from several deficiencies, including the intrusion of salt water into the ducts during construction, poor-quality grout at the anchor heads, splitting of the tendon ducts, and leaky segment joints in the pier.

As a result of these isolated incidents, FDOT investigated the cause of the problems at each of the bridge locations and took measures to repair the deficient tendons. FDOT implemented a series of interim measures beginning in 1999 to strengthen design, specification, construction, and inspection requirements for all types of new post-tensioned bridge construction. Post-tensioning tendons in all existing structures were subsequently inspected using nondestructive testing methods. It was determined that corrosion in grouted post-tensioning tendons was not a widespread problem and, in fact, showed a very low frequency of occurrence. However, the Department realized that the potential for corrosion existed in other locations (depending on conditions at the tendon location) and that improvements in design details, materials, and grouting practices were necessary to decrease the likelihood of corrosion in post-tensioning tendons and anchorages on future bridge projects.

As a result of the isolated problems discovered in Florida, several other states with post-tensioned bridges have inspected the post-tensioning tendons and anchorages in their bridges. Inspections performed by the Texas DOT on several post-tensioned bridges identified grouting deficiencies in some structures. The deficiencies included grout voids, split external tendon ducts, and tendon corrosion similar to the deficiencies noted in some of the Florida bridges. The Massachusetts Highway Department also investigated the post-tensioning tendons on the newly constructed segmental bridges for the Central Artery and Tunnel project (CA/T or "The Big Dig"). They discovered several locations where the potential for corrosion existed because of poor-quality grout and/or voids in the tendon anchor heads. The Department proceeded to repair the deficiencies at those locations and instituted new grouting practices for the remainder of the project.

On a worldwide basis, experience with grouted post-tensioning tendons over more than one-half a century has proven that post-tensioned concrete structures are inherently cost-effective to build and provide excellent long-term durability. On a national basis, the American Segmental Bridge Institute (ASBI) published a report, "Durability Survey of Segmental Concrete Bridges" *(29)*, in September 2000, which indicated that the National Bridge Inventory inspection reports indicated no structurally deficient segmental bridges in the United States. Despite the excellent performance of most of the post-tensioned bridges in service around the world, the discovery of any corrosion problem tends to generate concern with regard to all post-tensioned construction.

Although it is not the intent of this research report to present a complete discussion of the issues surrounding corrosion in post-tensioning tendons, it is important to address concerns as they apply to the use of post-tensioning in spliced

girder bridges. Corrosion of post-tensioning tendons is not a widespread problem and is believed to be a concern primarily for segmental bridges. Although there were dramatic examples of tendon corrosion and tendon failure in some of the Florida segmental bridges and evidence of grouting deficiencies in some of the Texas and Boston CA/T structures, other states have not reported similar problems.

2.3.3.2 Differences between Spliced Girder and Segmental Construction

Spliced girder bridges differ from segmental bridges with regard to post-tensioning as follows:

- The joints between the precast elements in spliced girder bridges do not extend through the deck, as they do for segmental box girder construction. Instead, the joints (splices) are protected by the cast-in-place deck.
- Spliced girder bridges use relatively few joints between the precast elements, so leakage at joints and resulting contaminant intrusion into post-tensioning ducts is minimized.
- Spliced girder bridges do not generally use external ducts, which have experienced some problems with damage during construction and splitting with time.
- Spliced girder bridges offer additional redundancy (multiple girder webs) instead of concentrating tendons in only a few webs.

2.3.3.3 Progress in Addressing Concerns

The recent problems in Florida related to grouting have caused the industry to reexamine past material specifications and grouting practices. The problems reported, most of which are specific to precast segmental construction, have been traced to identifiable causes associated with poor design details, low-quality materials, and improper grouting procedures combined with inadequate inspection practices. This has highlighted the need for using improved technology in the quality of grouting materials and procedures.

Industry and owners have addressed the challenge of improving post-tensioning details and practices in the following ways:

- The Post-Tensioning Institute (PTI) has published a guide specification, "Specification for Grouting of Post-Tensioned Structures" (30), which addresses many of the issues of proper grouting and inspection practices.
- The American Segmental Bridge Institute (ASBI) formed a grouting committee consisting of owners, designers, contractors, and suppliers to address the problem and

issued an interim statement on grouting practices in December 2000 (31).
- ASBI now offers a grouting certification training course, which follows the recommendations of the PTI Guide Specifications.

FDOT recently published "New Directions for Florida Post-Tensioned Bridges" (27), which summarizes their new policies and procedures developed in response to the discovery of the problems and describes FDOT's five-part strategy to produce more durable post-tensioned bridges:

1. Enhanced post-tensioning systems: All post-tensioning tendons shall be fabricated using enhanced post-tensioning systems.
2. Fully grouted tendons: All post-tensioning tendons shall be completely filled with grout during construction.
3. Multi-level anchor protection: All post-tensioning tendon anchors shall have a minimum of four levels of corrosion protection.
4. Watertight bridges: All bridge decks of post-tensioned bridges shall be watertight.
5. Multiple tendon paths: Post-tensioned bridges shall be designed to provide multiple tendon paths using a greater number of smaller sized tendons.

Volume 4 of the FDOT publication is "Design and Construction Inspection of Precast Concrete Spliced I-Girder Bridges." This comprehensive document provides detailed recommendations for the application of the five-part strategy specifically to post-tensioned spliced girder bridges. Specific details are provided in many areas, including duct spacing and clearances in the web of post-tensioned girders (Figure 9), grout vent and drain locations (Figure 10), and anchorage encasement (Figure 11). It is strongly recommended that the designer of a spliced girder bridge obtain a copy of the report (27), which is available at no charge on the FDOT website.

Grouting specifications and design recommendations are also available from other state DOTs, including California, Massachusetts, Texas, and Washington, which have considerable experience with post-tensioned structures.

Recent research (32, 33) has demonstrated that satisfactory tendon grouting can be achieved with a high level of performance assurance.

2.3.3.4 Conclusion

Although concerns have been raised about the durability and construction of post-tensioned concrete bridges, it has been found that corrosion of post-tensioning tendons and anchorages is not widespread. However, in response to the concerns raised, improved grouting materials and procedures

Previous Experience

Ducts too large for available width

Ducts in contact

honeycomb or void

longitudinal crack

Proposed

Provide clearance for concrete placement and consolidation

Minimum (center to center) spacing of ducts to be the greatest of:
- **2*inside diameter of ducts**
- **Outside diameter of duct (across rib) + 2"**
- **O.D. duct + 1.5*maximum aggregate size**
- **4"**
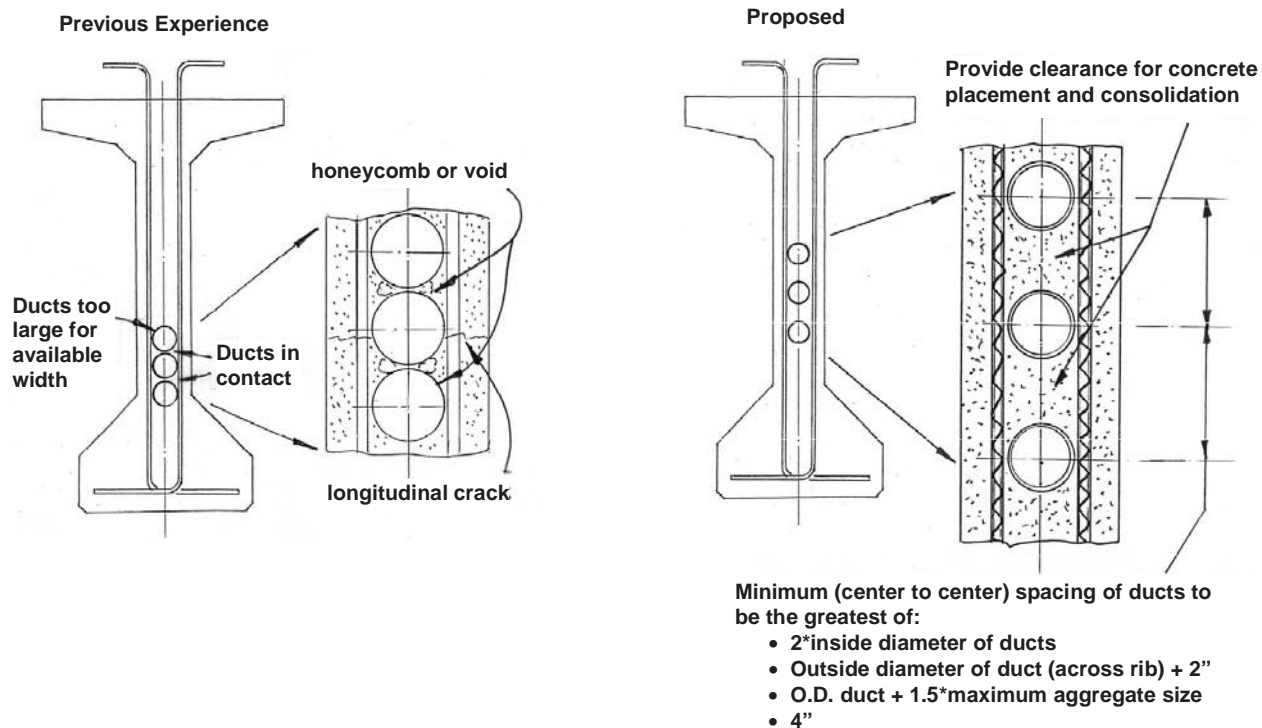
Figure 9.   Duct spacing and clearances in post-tensioned girders (27).

have been developed and research has been conducted on the performance of grouts. In addition, owners and industry have recognized the need for appropriate attention to grouting control during the construction process. Many states now require grouting supervisors and inspectors to have a requisite level of training and experience to ensure compliance with recommended practices and specifications.

Based on these efforts, it appears that the overall excellent performance of post-tensioned concrete bridges, specifically spliced girder bridges, should continue and be enhanced by the improvements now being made.

2.3.4 Staged Construction

Spliced girders may be used for bridges where staged construction is required. Eventually, spliced girder bridges may also be widened, which will require some of the same considerations as required for staged construction.

As with any staged construction, the main concern is matching grades between the stages. However, matching grades with post-tensioned members is complicated by the fact that the elevations of the girder change when post-tensioning tendons are stressed. Therefore, the construction sequence affects the

Provide vent at each crest and 3 to 6 feet each side of crest (C, E, H, J)

Continuously draped tendon profile

Provide sealed duct splices at each cast-in-place joint
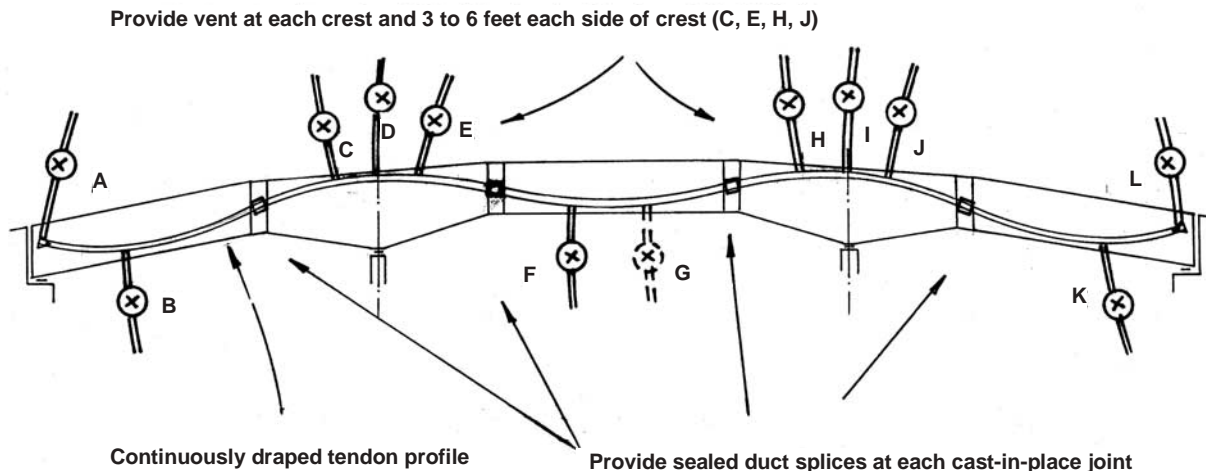
Figure 10.   Grout vent and drain locations (27).
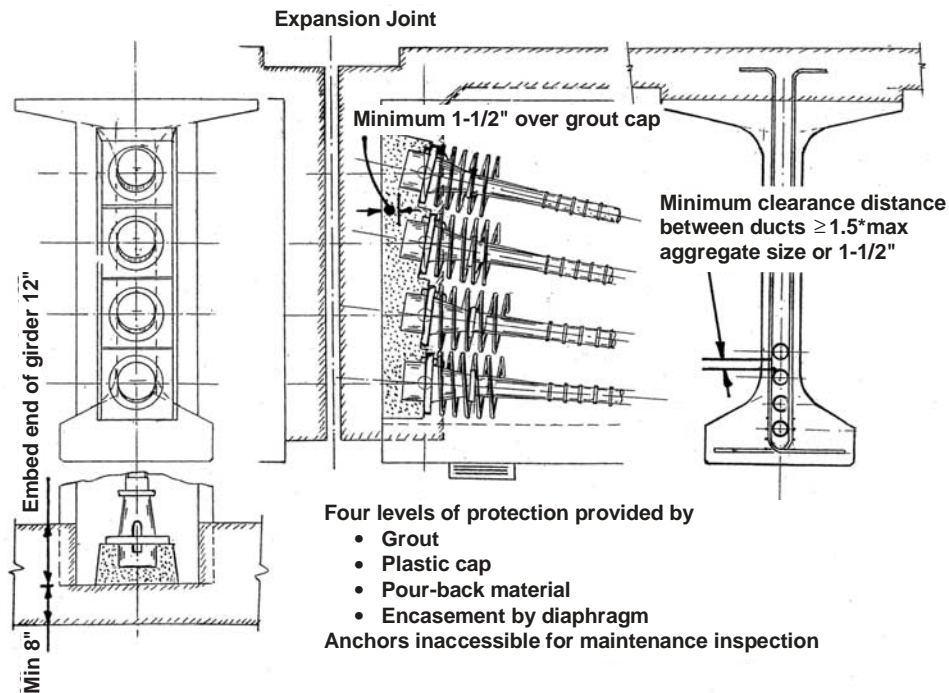
Figure 11. Anchorage encasement details (27).

issues that must be addressed when using spliced girders with staged construction.

2.3.4.1 Single-Stage Post-Tensioning Applied Before Deck Is Cast

If all of the post-tensioning tendons are stressed before the deck is cast, the design of a spliced girder bridge for staged construction is essentially the same as staged construction for conventional girders because all of the deformations caused by stressing the post-tensioning tendons occur before the deck is cast. The variation of the camber along the length of the spliced girder can then be accounted for when the deck forms and screed elevations are set, as is done in construction of conventional precast prestressed concrete girder bridges. An additional buildup thickness and additional allowances for grinding the deck to match the profile should be considered, to allow for any unexpected variation in the girder profile.

The spliced girder may be subject to deflections because of time-dependent effects (e.g., creep and shrinkage) during the period between completion of the first stage and erection of the second stage. A survey of the first stage should be conducted as described in the following section.

2.3.4.2 Post-Tensioning Applied After Deck Is Cast

If some or all of the post-tensioning tendons are stressed after the deck is cast, the spliced girder needs to be designed

with details that provide for possible elevation adjustments during construction and that allow for the movement of the spliced girder during stressing of the post-tensioning tendons.

All post-tensioning tendons must be stressed before making any permanent connections between the stages of construction. Any temporary connections between new and existing construction must be capable of allowing relative movement during stressing of post-tensioning tendons or must be loosened or disconnected to allow movement during tendon stressing.

A detailed survey of first stage or existing structure should be made to define the grades that must be matched. A survey should be conducted when the first stage is completed, and then a second survey prior to erecting the second-stage girder segments. The design and details of the second or later stages should then be reviewed and adjustments made to grades, seat elevations, and temporary support elevations to account for the actual conditions.

The designer should consider one or more of the following options to allow for adjustments during construction for any unexpected variation in the girder profile:

- Setting the girders in the second or later stages at slightly lower elevations;
- Designing the post-tensioning tendons with the capacity for slightly increasing or decreasing the post-tensioning force (by adding or removing strands from the tendon or partially stressing the tendon) to adjust the camber;
- Providing additional buildup thickness;

- Providing additional slab thickness and cover to allow for grinding the deck to match the profile; and
- Providing a wearing surface that can be used to match the final grades.

It is much easier to provide extra allowances in the design and concrete in the deck and buildup than to try to correct a problem in the field after all bearing elevations have been established and the girder segments have been set.

2.3.4.3 Reducing Differential Elevations

In some cases, it may be desirable or necessary to reduce or eliminate the differential elevation between the stages of construction. Although this is not generally done, options to do so exist. In many cases, these actions could be avoided by providing adequate allowances for construction tolerances as discussed above.

Depending on the details of the bridge and its flexibility, differences in elevation between the stages could be reduced or eliminated by attaching temporary cross beams to the bridge and pulling the two stages into alignment. This could be accomplished by attaching a cross beam to the bottom of the diaphragms, spanning across several girders in each stage. The adjusted alignment would be maintained until the closure between the stages was poured and cured. A similar system has been widely used to eliminate the differential camber between decked bulb tee girders before making the connection between adjacent units.

Other options for reducing the differential elevation between stages include hanging weights from one structure or changing the temperature of some portion of one structure (this has been done in a few cases to improve the alignment of very-long-span structures at the closure of joints between cantilevers).

Long-term effects of pulling out the difference in elevation should not be significant because of the flexibility of spliced girder structures. Furthermore, it is expected that the stresses would equalize with time between the stages because of creep effects.

2.3.5 Curved Bridges

Spliced girders have been used successfully on bridges with a radius as small as 500 feet *(22)* (Figure 12). The basic concept remains the same as for straight spliced girders, with only a few additional details and design considerations.

To provide curvature in the plan, straight girder segments are generally used to approximate the curve. Therefore, angle changes occur at the splice locations (Figure 13). This means that the splice must be wide enough for the post-tensioning ducts to be curved at an acceptable radius in the plan. Estimates of friction losses must also account for the loss at this curvature in the plan.

Figure 12.   Curved spliced girder bridge (22).

The tendon curvature in the plan will cause a transverse force that must be considered in the design. This force will be most significant at the splice for the interior girder where the surface of the concrete could spall off without proper detailing.

A three-dimensional structural analysis may be required for small-radius ramps.

2.3.6 Splicing of Haunched Girder Segments

Some of the longer-span spliced girder bridges have used a haunched pier girder segment to improve the structural efficiency of the girder. With the increased depth, these girder segments often become very heavy and can also be very tall. As a result, most haunched pier girder segments cannot be transported on land and must be delivered by barge, which limits their use to sites crossing navigable waterways.

Figure 13.   Curved spliced girder bridge using straight girder segments (34).

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Spliced girders that are haunched at the interior piers may be an economical solution to long-span bridges at other types of sites. Therefore, the issue of transportation of large pier segments on land must be solved if this is to be feasible. The idea of splicing the pier girder segment to enable shipping has been suggested as one solution. Pier girder segments could be spliced using either a vertical or a horizontal joint.

2.3.6.1 Type of Splice

Although a horizontal joint could be used, several issues complicate its use. With a horizontal splice, the simplest concept would be to use a girder with the same depth as the other girder segments for the pier girder segment and splice a piece on the bottom of the girder segment. This would require a connection at the bottom of the girder capable of transferring horizontal shear between the two elements. This is expected to present design and fabrication problems. Erecting the long, shallow-tapered haunch and then making the necessary connections between the haunch and girder along a lengthy interface appear to be problematic. Furthermore, it appears that such a design would require testing to demonstrate its performance before it could be used.

Use of a vertical joint allows any type of section to be used, such as sections where the web height varies and the bottom flange dimensions remain constant or nearly constant. Existing forms for one-piece haunched segments could easily be used or modified for use with vertically spliced girder segments.

Standard techniques can be used for erection of the girder segments. For the vertical splice, one-half of the pier girder segment would be erected on the pier and a temporary support, as is typically done. Then, the other half of the girder segment would be raised into position and attached to the first half, completing the pier segment. With the halves of the pier girder segment being relatively deep for their short length, it is possible that the halves could be conventionally reinforced, eliminating the need for pre-tensioning the pier girder segments. Given that the use of the vertical splice appears to have more benefits and fewer complications, only the use of a vertical splice is considered further in this study.

A haunched pier girder segment can be spliced with a vertical splice using standard splicing procedures. Two options are available: girder segments with a cast-in-place closure, and match-cast girder segments (typical of segmental box girder construction). With proper consideration of construction issues, either detail may be used. However, a major concern for detailing either type of splice is the ease and speed with which the contractor can make the connection in the field, because a crane would be required to hold the girder segment until a connection adequate to hold the second girder segment in place is made.

2.3.6.2 Cast-in-Place Vertical Splice

A possible set of details for a vertical splice in a haunched girder segment is shown in Figure 14. These details demonstrate a splice with a cast-in-place closure joint (splice) between girder segments, which would be very similar to other splices on the bridge. To facilitate erection, the use of alignment plates is recommended. The alignment plates are machined with one or more shear keys that provide fast and
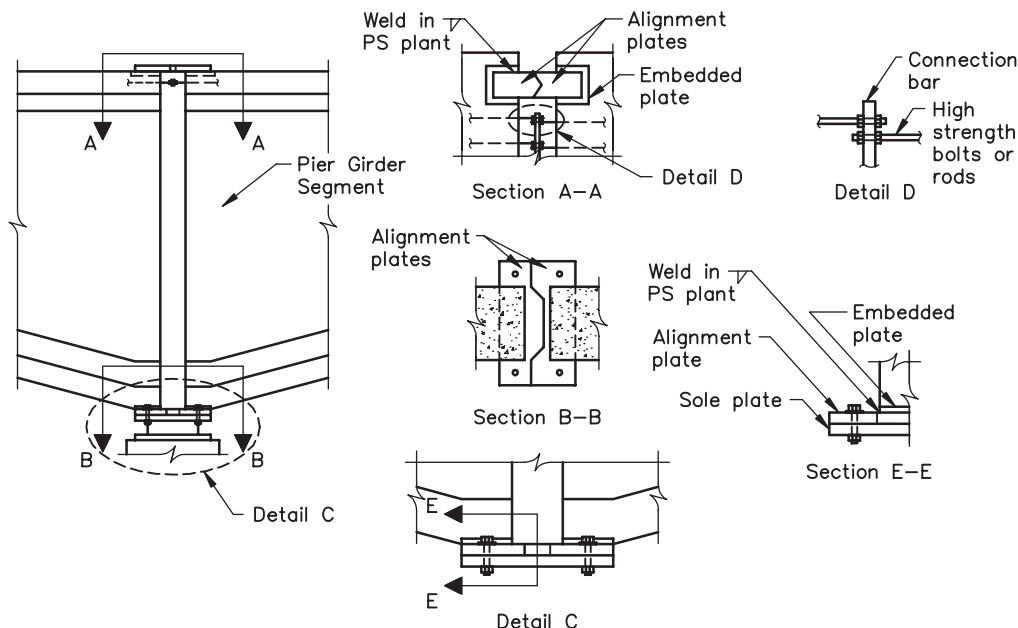
Figure 14. Proposed details for cast-in-place vertical splice for pier girder segment.

accurate alignment of the segments in the field. The two halves of the pier girder segment are temporarily erected and carefully aligned in the fabrication plant. Then alignment plates at both the top and bottom flange are welded to plates embedded in the two halves of the pier segment. Although additional effort is required in the plant, the alignment plates facilitate erection, because the alignment of the halves in the field will be quick and accurate, allowing the crane to release the girder segment after a high-strength bolted connection is made. The speed of the connection is important for efficiency in erection.

Erection of the haunched girder segments begins by placing one-half of the girder segment on the interior pier with its other end resting on and tied down to a temporary support tower. The second half is then erected, using bolted connections and alignment plates at the top and bottom of the girder to ensure proper positioning. Formwork would then be installed for the splice, which would typically be contained within a pier diaphragm. Once the splice and diaphragm have been cast and have reached minimum strength, the two pieces of the haunched girder segment would be post-tensioned using straight tendons in the top flange to provide the necessary prestress to support the weight of the drop-in span. The pier girder segment tendons must be located so they do not conflict with the main continuous post-tensioning tendons. After the girder segments are erected, the contractor would form and pour the cast-in-place splice, then wait for curing before stressing the post-tensioning tendons in the pier girder segment. The time required for these steps would delay the erection schedule for the drop-in girder segments, which might pose a difficulty for the contractor.

2.3.6.3 Match-Cast Vertical Splice

This method uses established procedures from segmental box girder construction. When the girder segments are match-cast, alignment plates are not required because the faces of the matching girder segments have alignment keys cast into them.

Use of match-casting of the pier girder segment pieces would require careful alignment of the halves of the pier girder segment as the second half is cast. After erection of the first half of the haunched pier girder segment, the mating surfaces of the splice would be coated with epoxy as the second half is erected. When positioned, post-tensioning tendons in the top flange would be stressed to hold the two girder segments and apply a minimum clamping pressure across the epoxy joint. As with the splice using the cast-in-place closure, post-tensioning would be used to provide the necessary capacity for the girder segment to resist the load of the drop-in girder segment.

2.3.6.4 Conclusion

The concepts presented here have not been used for construction of haunched pier girder segments, but both types of

vertical splices use standard procedures that could be easily implemented for a spliced girder project. Therefore, it appears that the use of such a concept is reasonable, with proper attention to the details required to facilitate quick and accurate erection of the pieces of the haunched pier girder segment.

With either method of making the vertical girder segment splice, the geometry of the girders can be adjusted to compensate for the deflection in the tips of the pier girder segment when the drop-in is placed or to make allowances or corrections for the profile of the structure. This feature allows the pier girder segment to better fit the final profile of the completed structure.

2.3.7 Diaphragms at Closure Pours

A diaphragm has typically been installed between girders at splice locations, with the diaphragm and splice cast at the same time.

Providing a diaphragm at the splice location provides the following benefits:

- Improved consolidation of concrete in the splice, given that the volume of concrete being cast with the splice is much larger and more accessible for placement and vibration. (Splice locations tend to be congested with spliced post-tensioning ducts and possibly other reinforcement, so having the splice within a diaphragm provides for much better flow of concrete into the splice area. It also gives much better access for vibration, without having to pass the vibrator between the side form and spliced duct.)
- Additional stability for girders during stressing of post-tensioning tendons.
- Improved confinement of the concrete in the splice, because the splice is surrounded on all sides with concrete.
- Opportunity to provide a visual break in the exterior face of the girder if the diaphragm is extended beyond the face of the girder (Figure 15). (This can mask any difference in color of the concrete in the girder and splice.)

Figure 15.   Diaphragm at splice extended beyond face of exterior girder.

Potential disadvantages of providing a diaphragm at the splice location are as follows:

- Additional weight of concrete in the diaphragm;
- Difficult detailing for skewed bridges; and
- Potential distress in diaphragms during stressing of post-tensioning tendons as girders camber.

In addition, diaphrams are unnecessary for added stability for cross sections with greater stability, such as U-beams.

Although potential disadvantages exist to providing diaphragms at spliced locations, it is recommended that diaphragms be provided at splice locations unless special provisions are made to enhance the consolidation of concrete.

2.4 DESIGN SPECIFICATIONS

Design specifications can affect the success of a design methodology significantly. Therefore, the evaluation of the current *LRFD Specifications* was given a high priority in the project. Two activities addressed the *Specifications*. A summary of the results of a review of the *Specifications* is presented here, while a summary of the proposed revisions to the *Specifications* is presented in Chapter 3.

2.4.1 Review Existing *LRFD Specifications*

The existing *LRFD Specifications* were thoroughly reviewed to identify any provisions that may need revision, deletion, or reorganization and to identify provisions that may need to be added to the *Specifications* to address issues related to extended-span ranges and spliced girder construction. The complete list of items identified and a discussion of the format of the list are given in Appendix E2.

During the review of the *Specifications*, the following issues were identified as having special significance. Several items listed in Appendix E2 may be associated with each of the following issues.

2.4.1.1 Integration of Provisions for Segmentally and Other Than Segmentally Constructed Bridges

It is not clear how spliced concrete girders are to be considered in the *Specifications*. It appears that spliced girder construction would currently be classified as "Segmentally Constructed" or "Segmental Components," which would appear to make them subject to all the requirements of segmental bridges. However, many of the articles in the *Specifications* that apply to segmentally constructed bridges appear to be intended for bridges where the segments are relatively short, and match-cast joints are typically used. Many provisions for this type of construction are not appropriate, necessary, or applicable for spliced concrete girder bridges.

A particular area of concern is the limiting stresses, which are given in Article 5.9.4. Limiting stresses are provided for two categories of bridges: "Other than Segmentally Constructed Bridges" and "Segmentally Constructed Bridges." By definition, spliced girders appear to fall into the category of "Segmentally Constructed Bridges." However, the only limiting tensile stresses given for this type of construction are for the joints. This is appropriate for bridges with short segments, but is not appropriate for bridges with long segments where limiting stresses for "Other than Segmentally Constructed Bridges" should apply at locations away from the splices.

A second significant area of concern is Article 5.14, which presents provisions for specific structure types. As identified in the listing of issues in Appendix E2, spliced girder construction is mentioned in the *Specifications*, but the designer is left to choose which of the articles apply. There is an article for "Beams and Girders" (5.14.1) and an article for "Segmental Construction" (5.14.2). Both articles contain provisions that can be applied to spliced girder construction.

2.4.1.2 Definitions of Lightweight Concrete

The *Specifications* are not consistent or clear in their treatment of lightweight concrete. This includes definitions of the different types of lightweight concrete and corrections for different quantities, such as shear capacity and resistance factors, that depend on the type of lightweight concrete.

This lack of clarity and consistency makes it difficult to use specified density concrete, which may have a range of unit weights and compositions. The *Specifications* need to be revised so that any special considerations required for using a reduced-density concrete to extend span ranges will be simple, clear, and rational.

2.4.1.3 Partial Prestressing

Partial prestressing has been introduced to U.S. bridge design practice for the first time in the *LRFD Specifications*. However, complete direction for its implementation has not been provided. There have been no known problems to date concerning this because owners and designers do not have applications where the concept can be applied.

One possible application where the implementation of partial prestressing may be beneficial is the design of bridge decks in the negative moment region over interior supports. Using this concept could clarify the issue concerning tensile stress limits in the deck at these locations for continuous bridges (see next item). However, given that the concept of partial prestressing has not been well developed in the *Specifications*, considerable effort may be required to identify design methods and understand the advantages and disadvantages of this approach.

2.4.1.4 Deck Design for Continuous Members

In continuous members post-tensioned after the deck has been added, the deck may be subject to tensile stresses as a result of continuity moments at interior pier locations. As a result, tensile stress limits may be applied to the deck. This may lead to additional prestress (post-tensioning) being provided to satisfy the tensile stress limits in the deck. However, tensile stresses in the deck are not subject to limiting stresses in other types of continuous bridges, such as those constructed on steel girders.

2.4.1.5 Conclusion

The first of the issues listed above, which relates to the clear integration of spliced girder into the *Specifications*, was selected as the highest priority for continued work. The remaining issues discussed above are significant, but effort focused on them will not bring the benefits that would be realized by improving the *Specifications* for the design of spliced girder bridges.

2.5 ISSUES REQUIRING ADDITIONAL ACTION

During the course of this research project, issues and obstacles were identified that need to be addressed and resolved before the full potential of extended-span and spliced girder designs can be realized. In most cases, resolution of these issues will involve owners, fabricators, and other industry interests, rather than designers.

The issues can be categorized as follows:

- General,
- Specifications,
- Design and detailing,
- Fabrication,
- Transportation, and
- Construction.

One of the more significant issues is discussed below. The remaining issues are presented in Appendix F.

2.5.1 Hauling Issues

An important factor to be considered when evaluating the cost-effectiveness of spliced girders or other very long precast girders is the hauling requirements of individual states through which the girders have to be transported to reach the project site. Because of the wide variety of hauling requirements and restrictions, the potential for the effective use of extended spans and spliced girder technology varies greatly from state to state.

A brief discussion of this issue is presented to highlight these differences and to provide suggestions regarding what actions are needed to encourage the use of spliced girders where appropriate. This discussion includes information from several girder producers and designers regarding their perspectives on this issue and recommendations to owners to avoid possible pitfalls encountered if the hauling issue is not addressed properly.

2.5.1.1 Girder Producer Perspective

From a producer's perspective, the varying hauling requirements for different states are frustrating and can prove very costly. For example, one producer had to haul a girder from a plant in Ohio to a project site in West Virginia. Because of different trailer requirements, the girder was hauled from the plant where the girder was fabricated in Ohio to anther plant in Kentucky, where it was loaded on a different trailer and then transported to West Virginia. In other instances, where girders needed to be hauled across several states, permits were issued in some states and not others for the same load on the same route. In addition, states that have a length restriction, such as North Carolina (currently 120 feet), force shorter piece lengths and therefore more hauls are necessary to a particular site.

Differences in weight limitations, axle arrangements, length restrictions, and even escort requirements cause many problems and increase the costs for producers, contractors, and owners.

Some of the differences in permit approvals relate to the method used by the bridge engineer to evaluate the effect of the overweight loads. Without getting into a detailed discussion of the different methods, it is apparent that some states are allowing much heavier loads on the same type of facility than others. The design codes should be the same, but there are large variations of allowable loads and axle configurations. Allowable girder weights range from 120 kips to over 200 kips. To date, the longest single-piece girder delivered by truck appears to be a 209-foot, 260-kip NU 2800 girder in British Colombia. Another recent example is the transporting of 14 precast prestressed concrete girders, each of which were 170 feet long and weighed 190,000 pounds, 250 miles across the state of Washington *(35)*. It may be helpful to investigate methods used by states like Washington and Pennsylvania, which seem to allow larger loads, and compare these methods with those of other states. However, where higher loads are allowed, it is necessary to have enough truckers with adequate trailers in the area. If there is not enough competition among hauling companies, the transport costs could consume some of the savings from an economical design. For example, Washington State had been allowing girders to be designed with weights up to 200 kips, but, given the limited availability of adequate trailers, the state has decided to reduce the maximum girder weight to 165 kips.

Hauling issues should be an important part of the designer's bridge type study. However, even though most states have adequate instruction in policy memos or design manuals for selection of girder type, many designers are not familiar with

the different states' hauling requirements and methods used to evaluate overweight loads. This can result in an efficient design being completed that cannot be economically transported to the site. The cost of transporting the longer girders may, in some instances, outweigh the savings supposedly realized by the design of longer spans. For this reason, communication between the designer and producer is very important when determining girder types, especially when crossing state lines.

2.5.1.2 Suggestions for Owners

Some suggestions for owners to ensure that hauling issues are addressed adequately when considering use of longer or spliced girder design are as follows:

1. Make designers aware of hauling requirements and encourage them to communicate with the producers when there is a question about getting a certain length and weight of girder to the site. This should be a regular part of the project scoping procedure and included in the girder type selection decision.
2. Get input from producers and contractors when developing details for longer or spliced girders. Their experience can provide valuable information and avoid unnecessary costs.
3. Work through industry groups, such as PCI and AGC, to encourage the states to consider standardization of some of the hauling requirements. In North Carolina, DOT representatives meet regularly with PCI and AGC to discuss common issues and solve problems. This is an excellent forum to collect ideas and work toward getting them implemented.
4. Owners such as DOTs need to work within their own organizations to revise any hauling requirements that are restricting the designer's ability to use the latest technology to provide the most economical design. For example, in states where shorter lengths are required, the design office needs to work with the permit office to set a reasonable length.
5. Look at methods used to evaluate overweight loads and ensure that they are based on a rational design procedure. Compare with other states where larger loads are allowed and revise methods if necessary.

2.5.1.3 Conclusion

Because of the complexity of issues surrounding this topic, it is impractical to require all states to use the same criteria. However, any effort that can be made to relieve some of the problems will benefit the producers, designers, owners, and ultimately, the taxpayers. The hauling issues need to be addressed in order to take full advantage of any innovations involved with extending span or girder lengths.

2.6 DESIGN SOFTWARE AND OTHER RESOURCES

The final area in which information was collected in this study was related to software and other design resources or tools that are available for designers of spliced girder bridges.

2.6.1 Review of Design Software

In most cases, conventional bridge design software can be used to design bridges that use the identified design options for extending span ranges of precast prestressed concrete bridges. Simplified design methods can be used to design some spliced girder bridges, but most are complex enough that the design is executed using specialized computer programs or spreadsheets. Custom spreadsheets and commercially available programs were used to complete the design examples and comparative design study. Both types of software products served their intended function, simplifying the design process.

As part of this study, information on design software used for past and ongoing spliced girder bridge projects was obtained from the project data collection effort, from experiences of the researchers and other designers, and from suppliers of several computer programs. However, specific information concerning commercially available software is not provided in this report.

The findings of this study regarding software resources used to date for spliced girder bridge design are summarized in Appendix G1. Appendix G1 also provides direction regarding when the use of software may be beneficial and what features should be considered when purchasing software for spliced girder bridge design.

2.6.2 Other Resources

A literature review was conducted to identify existing design guides and examples that could be used by the designer of spliced girder bridges. In most cases, the existing resources are not readily available or address only limited portions of the design. A discussion and listing of the other resources identified appear in Appendix G1.

2.6.3 Bibliography

Appendix G2 is a bibliography containing all cited references and other identified resources.

CHAPTER **3**

INTERPRETATION, APPRAISAL, AND APPLICATIONS

Several of the products of this research study have immediate or future application for the design of extended span precast prestressed concrete bridge girders. These include a comparative design study, three design examples, and proposed revisions to the *AASHTO LRFD Specifications*. These products are discussed in this section.

3.1 COMPARATIVE DESIGN STUDY

The comparative study was prepared in order to demonstrate and compare the effectiveness of several of the highest ranked design options that have been identified for extending span ranges of precast prestressed concrete girders. A correlation between the design options in the comparative design study and the design options in the prioritized lists of Section 2.3.2 was given in Table 8. The complete comparative design study is presented in Appendix B2. Portions of the study are repeated here to summarize its content.

A single-span, simply supported bridge was selected as the subject of the comparative design study. The bridge's typical section and dimensions are shown in Figure 16, with PCI BT-72 precast prestressed concrete girders at a constant spacing of 8 feet, 0 inches. For the comparative design study, maximum spans were computed for the bridge using the selected design variations. Most design variations considered the effect of changing individual design parameters, but several combinations of design variations were also evaluated. Two additional girder types were also considered in the study. Calculations were performed using a commercially available software package.

The results were compiled and compared to give an indication of the effectiveness of each design option for increasing the maximum span. Results from this study should be considered as giving an indication of trends. For other situations, where girder spacing, section type, and other conditions are changed, the relative effect of different alternatives may vary from what is reported in this study. Combinations of different design cases are expected to provide additional benefit, although the effect may vary.

This study only considered flexural design of the precast prestressed girders. For the parameters considered in this study, the service limit state governed all designs. Conditions along the girder were evaluated.

A brief summary of the study follows, with an introduction to the design parameters considered and a summary of the results. Complete details of the study and a listing of the design assumptions and parameters used in preparing the study are given in Appendix B2.

3.1.1 Design Cases for PCI BT-72 Girder

The first set of designs uses the PCI BT-72 girder. Fourteen design variations on the base design were considered for this section. The variations ranged from examining the effect of concrete strength to modifying the girder cross section. The basic design information for the base design and each variation is summarized in Table 9. See Appendix B1 for a more detailed description of each case. Brief descriptions of the less obvious variations are as follows:

- **Deeper Bottom Flange (Case 7)**—The girder cross section was modified by adding 2 inches to the depth of the bottom flange of the precast girder, which also increases the total height of the section by 2 inches. The increased depth of the bottom flange allowed the use of an additional row of strands.
- **Increased Beam Width (Case 8)**—The girder cross section was modified by the addition of 2 inches to the width of the girder web along the full height of the girder. Therefore, the top and bottom flange widths were also increased by 2 inches.
- **Bundling of Strands (Case 9)**—The draped strands were bundled between hold-down points, rather than maintaining the 2-inch spacing. This approach, which is permitted by the current *Specifications*, increases the eccentricity, and therefore the effectiveness, for the draped pre-tensioning strands.
- **Debonding of Top Strands (Case 10)**—To reduce concrete stresses at release, girders were designed with additional top strands that were debonded at the center of the span with 30 feet bonded at each end. After erection of the girders, these strands were de-tensioned by torch through an access hole using typical de-tensioning procedures.
- **Decked Bulb Tee (Case 13)**—The cross section of the girder was modified by the addition of an 8-inch-thick (structural thickness) deck to the top of the standard PCI

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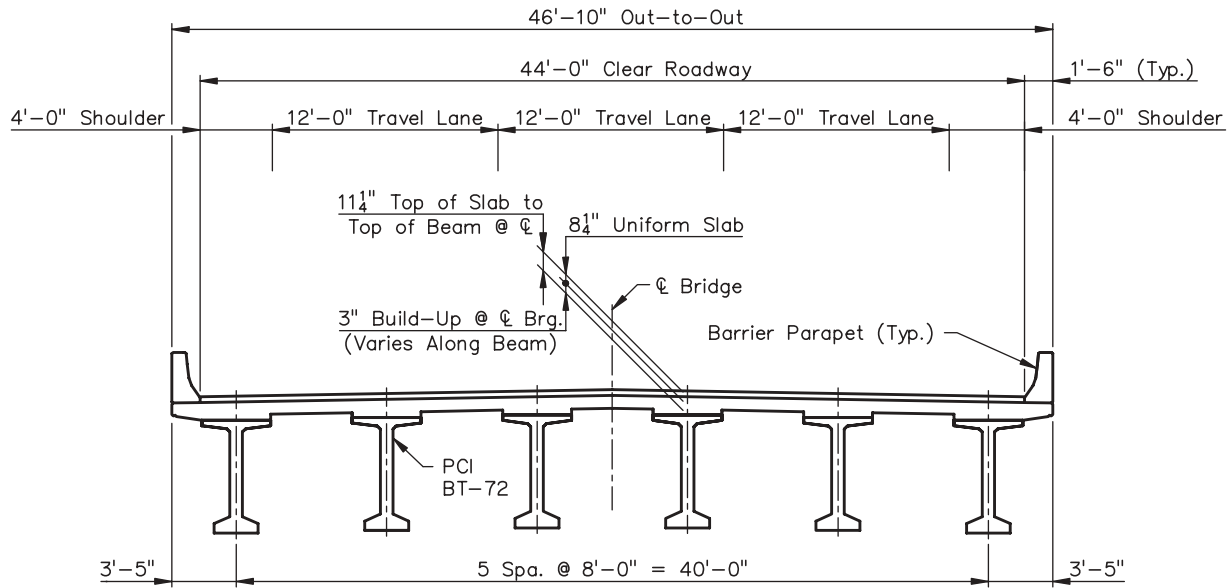
Figure 16.   Typical section of bridge.

BT-72 as shown in Figure 17. The deck is present when the girder is prestressed at the release of the pre-tensioned strands.

- **Touch Shoring (Case 14)**—Before placement of the reinforced concrete deck, temporary shoring towers are installed to support the girder at midspan. The tempo-

rary tower is placed so it supports the girders without applying an upward load prior to placement of the deck concrete. This changes the span resisting the weight of the fresh deck concrete from a simple span to a two-span continuous structure, which reduces the effect of the weight of the deck on the bare girder. After the deck

TABLE 9   Basic design information for designs using PCI BT-72 girder

Design Case		f'_{ci}	f'_c	f'_{cd}	w_c	w_{cd}	Web Width	Bottom Flange Depth	Strand ϕ
No.	Description	(ksi)	(ksi)	(ksi)	(kcf)	(kcf)	(in.)	(in.)	(in.)
1	Base Design	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
2	Increased f'_{ci}	6.0	6.5	4.5	0.150	0.150	6.0	6.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	6.0	6.0	½
4	Lightweight Conc. Deck	5.0	6.5	4.5	0.150	0.120	6.0	6.0	½
5	Lightweight Conc. Beam	5.0	6.5	4.5	0.120	0.150	6.0	6.0	½
6	Lightweight Conc. Dk. & Bm.	5.0	6.5	4.5	0.120	0.120	6.0	6.0	½
7	Deeper Bottom Flange	5.0	6.5	4.5	0.150	0.150	6.0	8.0	½
8	Increased Beam Width	5.0	6.5	4.5	0.150	0.150	8.0	6.0	½
9	Bundled Strands	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
10	Top Strands Debonded	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	6.0	6.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	6.0	6.0	0.6
13	Decked Bulb Tee	5.0	6.5	6.5	0.150	0.150	6.0	6.0	½
14	Touch Shoring	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
15	Added Post-Tensioning	5.0	6.5	4.5	0.150	0.150	8.0	6.0	½

Shaded cells indicate values that differ from Base Design.

Figure 17.   Girder dimensions for decked bulb tee.

cures and reaches a specified minimum strength, the temporary shoring is removed and the bridge acts again as a simple span. The reaction present in the shoring is then transferred to the composite girder section. Therefore, the stresses at midspan caused by the deck concrete are reduced, because some of the effect of the weight of the concrete had been placed and had reached a specified strength. Applying the post-tensioning to the full cross section, rather than to the bare girder, makes it more effective. To accommodate the post-tensioning ducts, the cross section of the girder was modified by the addition of 2 inches to the width of the girder along its full height as described in Case 8.

The results of designs for all of the design cases are listed in Table 10 and include maximum spans, the absolute and percentage increase in maximum spans compared with the Base Design, and notes concerning governing design condition(s). The maximum spans for each design case are presented graphically in Figure 18 and the percent increase in maximum spans compared with the Base Design are shown in Figure 19.

A discussion of the results for each design case is given in Appendix B1. The general conclusions and comments are summarized as follows:

1. The greatest increase in maximum span length was obtained by casting the deck with the girder (decked bulb tee—Case 13) and by adding post-tensioning to a pre-tensioned girder (Case 15), with increases in maximum spans of 33.1 and 24.6 percent, respectively. These design variations are out of the ordinary and require significant additional effort in design and construction. However, they have been used successfully in some situations and regions.
2. The next most effective strategy for increasing the maximum span length was the combination of increased

strand size with high-strength concrete (Case 12), with an increase in the maximum span of 16.9 percent. The use of this strategy has become common and has served to increase span ranges for precast concrete girders significantly. Other design comparisons achieved increases in maximum spans from 0 to 10 percent.

3. A significant finding was the increase shown in Case 12, where two strategies were combined to produce a much higher increase in maximum span than either strategy alone. Although other combinations of strategies were not investigated in this study, it appears possible that the combination of different strategies could provide impressive additional gains in maximum spans. Therefore, designers are encouraged to consider combinations of strategies.
4. A final comment addresses the limitations of this study. These results are based on maximum span designs. The use of maximum spans results in designs that are governed by several stress conditions simultaneously. For most designs, only one design condition governs, or steps may be taken to prevent other conditions from governing (such as draping strands). Furthermore, this study is limited in scope, considering only one bridge cross section. Therefore, the results from these comparisons, while instructive, may not reflect the potential for bridges where the parameters differ and the design span is not at the maximum span. Each situation must be considered individually.

3.1.2 Comparison of PCI BT-72 with Different Girder Types

The second set of designs compares designs using the PCI BT-72 girder to designs using the NEBT 1800 and AASHTO Type VI girders. These girders are of equal or nearly equal height, as shown in Figure 20, but they differ in other dimensions.

To compare the effect of design cases on different girder types, four of the cases discussed above were repeated using NEBT 1800 and AASHTO Type VI girders. These four cases were as follows:

1. Base Design   *Case 1*
2. High-Strength Concrete   *Case 3*
3. 0.6-inch-Diameter Strands   *Case 11*
4. 0.6-inch-Diameter Strands and   *Case 12*
   High-Strength Concrete

In the discussion of the results of these designs, the cases will be identified using the numbers of the cases as used in the PCI BT-72 girder designs. The design information for each design combination is summarized in Table 11.

The results of the comparison of different girder types are shown in Table 12, which include maximum spans, span

TABLE 10 Summary of results for PCI BT-72 girder design cases

Design Case		Maximum Span	Span Increase	Percent Increase in Span	Notes
No.	Description	(ft)	(ft)	(%)	
1	Base Design	118	0	0.0	A
2	Increased f'_{ci}	126	8	6.8	A
3	High Strength Conc. (HSC)	130	12	10.2	B
4	Lightweight Conc. Deck	119	1	0.8	A
5	Lightweight Conc. Beam	122	4	3.4	A
6	Lightweight Conc. Dk. & Bm.	124	6	5.1	A
7	Deeper Bottom Flange	126	8	6.8	A
8	Increased Beam Width	124	6	5.1	A
9	Bundled Strands	118	0	0.0	A
10	Top Strands Debonded	122	4	3.4	A
11	0.6"ϕ Strands	118	0	0.0	A
12	0.6"ϕ Strands and HSC	138	20	16.9	A
13	Decked Bulb Tee	157	39	33.1	A,E
14	Touch Shoring	130	12	10.2	C
15	Added Post-Tensioning	147	29	24.6	D

Notes:

A   Critical stresses occur at release at midspan bottom in compression and at Service III at midspan bottom in tension.

B   Critical stresses occur at midspan at Service III despite room for additional strands. Additional strands do not provide sufficient precompression to offset the tension stresses due to applied moments.

C   Critical stresses occur at release at end of girder in compression in spite of draping and debonding.

D   Critical stresses occur at midspan in compression at the top of the girder.

E   Total weight of girder is 251 kips. Splicing may be required to address handling and transportation issues.

Figure 18.   Maximum span for each design case.
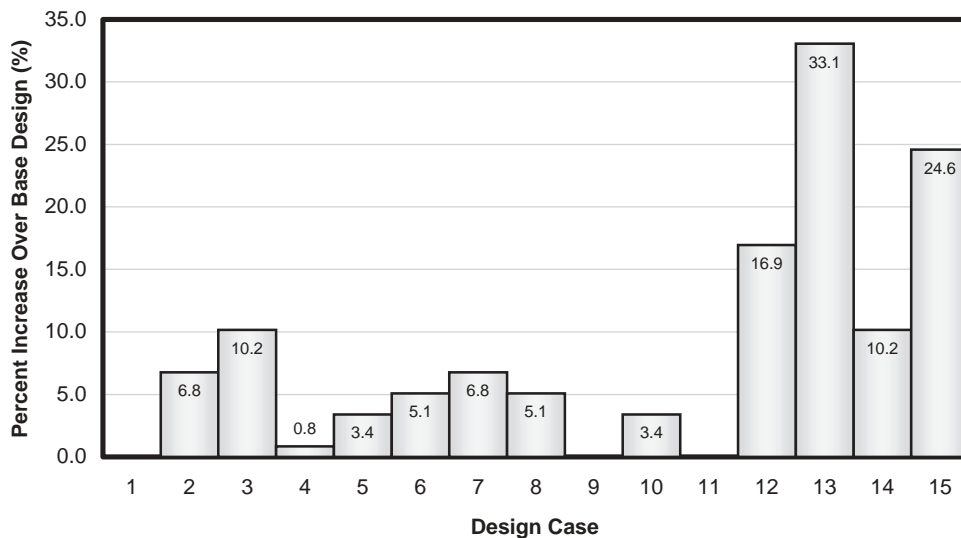
Figure 19. Percentage span increase for each design case.

increase, and percent increase in maximum span over the Base Design for each girder type, and over the same design case for the PCI BT-72 girder. Results for the PCI BT-72 girder design cases are identical to those presented in the preceding section.

3.1.2.1 Comparisons for Different Girder Types

The maximum spans for each design case, grouped by girder type, are presented graphically in Figure 21, and the percent increase in maximum spans compared with the Base Design for each girder type is shown in Figure 22.

The increase in maximum span for the three design variations over the Base Design for each girder type indicates a

similar relationship for the different girder types. The trends can be summarized as follows:

1. High-strength concrete increased the maximum span length. The span increase ranged from 4.4 percent to 10.2 percent with both the PCI BT-72 and AASHTO Type VI girders experiencing the same increase of 10.2 percent.
2. The use of 0.6-inch-diameter strands with normal-strength concrete was not effective for increasing maximum span for these girders for the bridge cross section considered.
3. The combination of high-strength concrete and 0.6-inch-diameter strands was most effective for increasing the maximum span for all girder types. The span increase ranged from 14.8 percent to 16.9 percent. The PCI
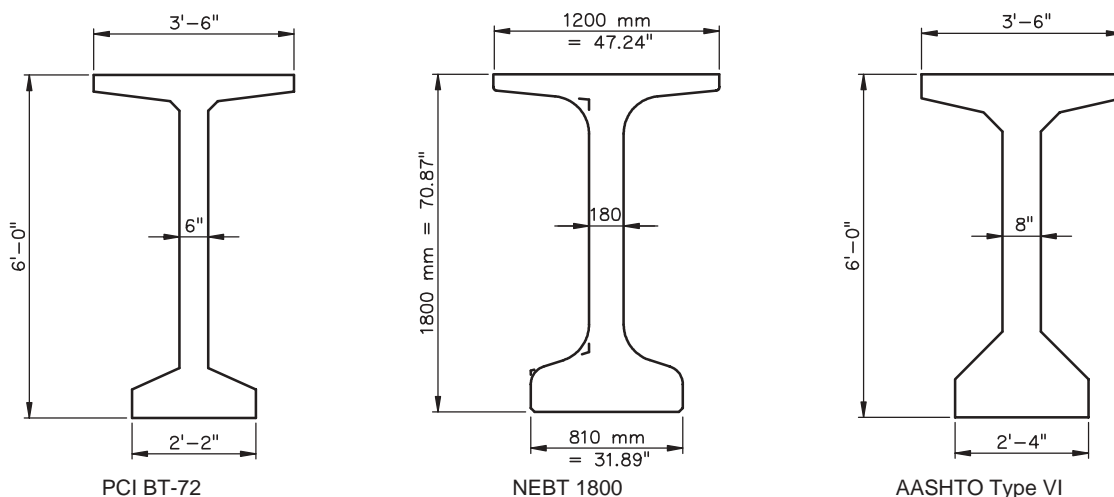
Figure 20. Dimensions for different girer types.

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TABLE 11 Basic design information for designs using different girder types

Girder Type and Design Case		f'_{ci}	f'_c	f'_{cd}	w_c	w_{cd}	Web Width	Bottom Flange Width	Strand ϕ
No.	Description	(ksi)	(ksi)	(ksi)	(kcf)	(kcf)	(in.)	(in.)	(in.)
PCI BT-72 Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	6.0	26.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	6.0	26.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	6.0	26.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	6.0	26.0	0.6
NEBT 1800 Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	7.1	31.9	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	7.1	31.9	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	7.1	31.9	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	7.1	31.9	0.6
AASHTO Type VI Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	8.0	28.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	8.0	28.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	8.0	28.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	8.0	28.0	0.6

Shaded cells indicate values that differ from Base Designs.

TABLE 12 Summary of results for different girder types

Girder Type and Design Case		Maximum Span	Span Increase for Each Girder Type	Percent Increase for Each Girder Type	Span Increase Compared to PCI BT-72 Same Case	Percent Increase Compared to PCI BT-72 Same Case
No.	Description	(ft)	(ft)	(%)	(ft)	(%)
PCI BT-72 Girder						
1	Base Design	118	----	----	----	----
3	High Strength Conc. (HSC)	130	12	10.2	----	----
11	0.6" ϕ Strands	118	0	0.0	----	----
12	0.6" ϕ Strands and HSC	138	20	16.9	----	----
NEBT 1800 Girder						
1	Base Design	135	----	----	17	14.4
3	High Strength Conc. (HSC)	141	6	4.4	11	8.5
11	0.6" ϕ Strands	135	0	0.0	17	14.4
12	0.6" ϕ Strands and HSC	155	20	14.8	17	12.3
AASHTO Type VI Girder						
1	Base Design	137	----	----	19	16.1
3	High Strength Conc. (HSC)	151	14	10.2	21	16.2
11	0.6" ϕ Strands	138	1	0.7	20	16.9
12	0.6" ϕ Strands and HSC	159	22	16.1	21	15.2

Figure 21. Maximum spans for different girder types.

BT-72 girder showed the greatest increase despite its having the smallest bottom flange, which limits the possible number of strand locations.

3.1.2.2 Comparisons of Different Girder Types with PCI BT-72 Girder

The maximum spans for each design case, grouped by design case, are presented graphically in Figure 23, and the percent increase in maximum spans compared with the Base Design for each girder type is shown in Figure 24.

The increase in maximum span for the three girder types over the Base Design for each design variation indicates a similar relationship for each design variation. The trends can be summarized as follows:

1. The NEBT 1800 and AASHTO Type VI girders have a greater maximum span for all design cases investigated.
2. The NEBT 1800 and AASHTO Type VI girders have maximum spans 17 feet and 19 feet larger, respectively, than the Base Design for the PCI BT-72 girder.
3. The use of high-strength concrete and 0.6-inch-diameter strands increased the maximum spans of the NEBT 1800 and AASHTO Type VI girders by 20 and 22 feet, respectively, over the Base Design for the PCI BT-72 girder. This strategy for increasing span range is clearly the most effective of those considered.
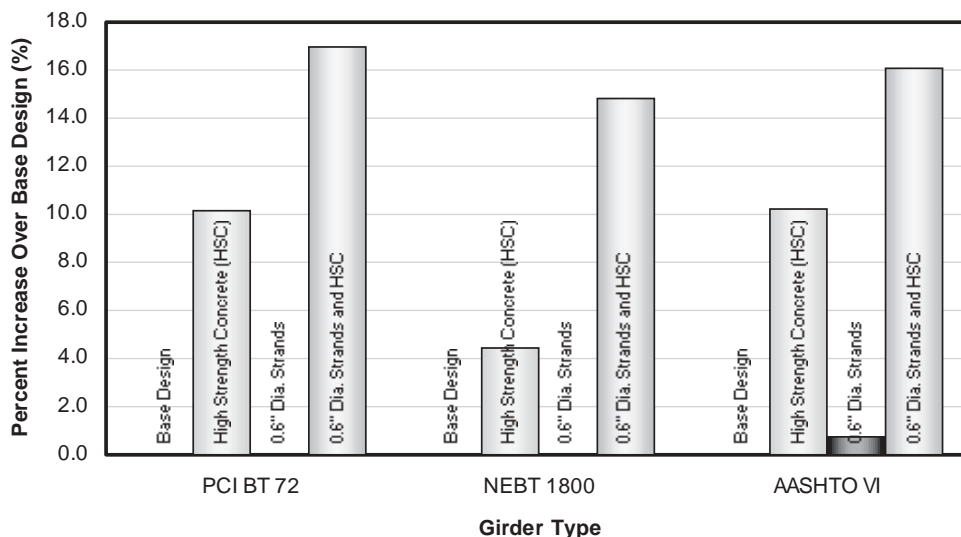
Figure 22. Percent increase in maximum spans for different girder types.

50
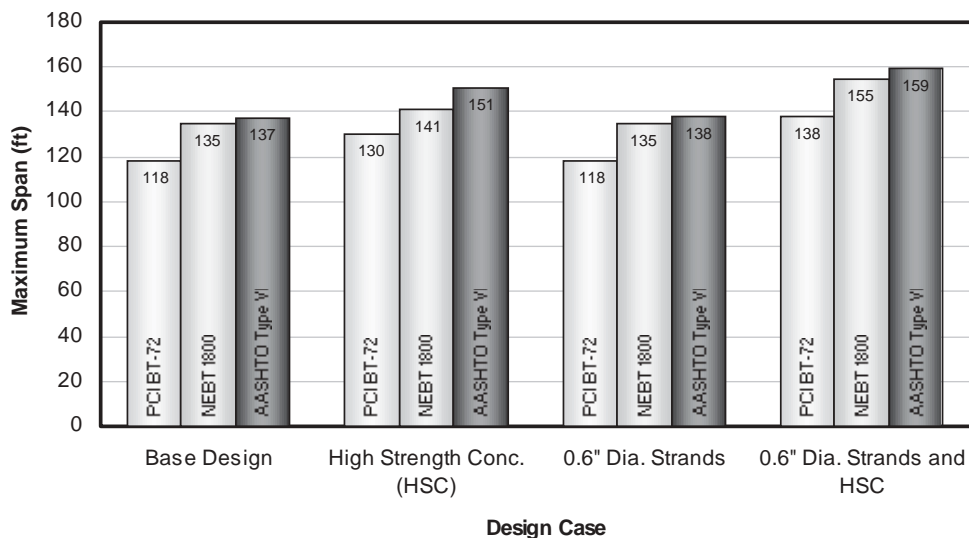
Figure 23.   Maximum spans for different girder types for each design case.

3.1.2.3 General Conclusions and Comments

The comparison of designs for different girder types indicates that the AASHTO Type VI girder has the largest maximum span and the PCI BT-72 girder has the smallest maximum span for all design cases. The NEBT 1800 girder does not experience as great a percentage increase in maximum spans from the Base Design for any of the design cases considered when compared with the other girders (see Figure 22).

These conclusions tend to indicate that the AASHTO Type VI girder would provide the most efficient designs given that it had the longest maximum spans. However, the Type VI girder is a much larger cross section. Therefore, more information must be considered when comparing the perfor-

mance of different girder sections to determine the most efficient and economical designs.

3.2 DESIGN EXAMPLES

Design examples are important resources that enable a designer to approach a new type of design with increased confidence because the examples present clearly all of the required issues and procedures necessary to complete a design. Design of spliced girder bridges, which is the major focus of this study, involves greater complexity in design than is required for conventional precast prestressed concrete girder designs. Therefore, design examples were developed to facilitate the
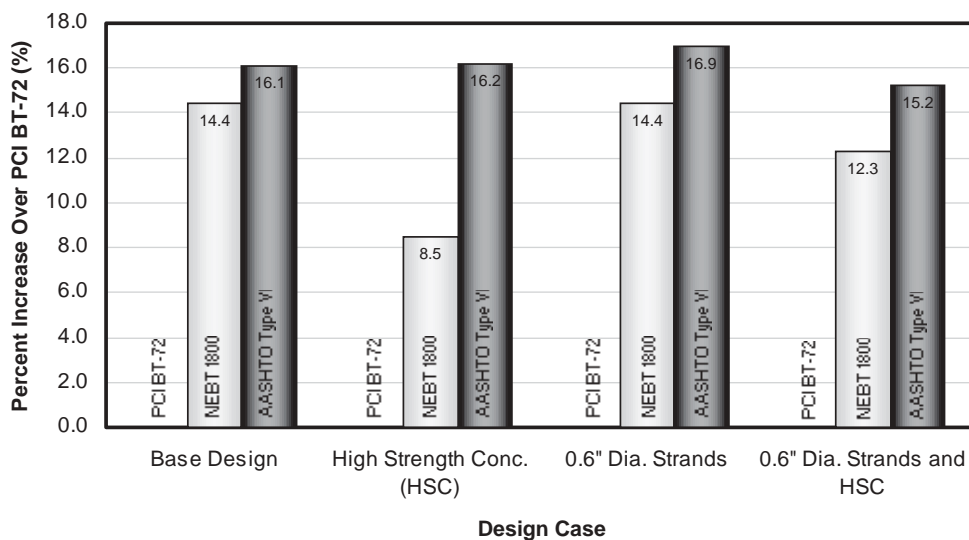
Figure 24.   Percent increase in maximum spans for different girder types compared with PCI BT-72 girder.

use of this type of construction, which has great potential for extending the span ranges of precast prestressed concrete bridges.

Three design examples are presented in Appendix D. Each design example represents a spliced girder bridge concept that can be applied to a wide range of applications. Following several general comments, a brief introduction to each design example is presented.

3.2.1 General Comments

The level of detail differs among the three design examples. In the first design example, which represents the simplest structure, the basic concepts are thoroughly presented for all aspects of spliced girder design. For aspects of design where the design of spliced girders is the same as conventional precast prestressed concrete girders, detailed design calculations are generally not given. The subsequent design examples focus on the issues specific to multiple-span spliced girder bridges and generally refer to Design Example 1 rather than repeating detailed discussions of the basic concepts that have already been presented. In all examples, an effort has been made to state all assumptions, so that the examples will be as clear and useful as possible for the user.

The examples are intended to be a reasonable solution for the site and conditions described. Many aspects of design can be altered, such as the cross-section type, the sequence and method of construction, and the sequence and quantity of post-tensioning. For example, priorities of design, such as deck removal for replacement, may dictate the use of a different construction sequence. Therefore, the most appropriate choice for these and other details must be established considering the specific constraints for each project. To illustrate this point, some details of design and construction differ between the design examples. Several options that could have been employed in the design are discussed at the end of Design Examples 1 and 2. The user is encouraged to consider the design options in light of the specific requirements and constraints to obtain the best solution for the project rather than

to repeat the design solutions presented here. The concepts presented should enable the user to extend the use of the example to other situations.

The design examples were prepared using the *AASHTO LRFD Bridge Design Specifications* with Interims through 2003. Where the *Specifications* are unclear or do not provide requirements necessary for the design, assumptions are made and explained. Article and equation references are given where appropriate to assist the designer in applying the *Specifications*.

Each design example is numbered separately and has its own table of contents to assist the user in finding sections of interest and to give an overview of the organization of the design process. Each design example also has a list of the references cited in the example.

3.2.2 Design Example 1— Single-Span Spliced PCI BT-96 Girder

The first design example is the simplest example, addressing a long simple-span bridge. The elevation of the girder is shown in Figure 25 with the typical section shown in Figure 26. This could represent a long-span bridge over a stream crossing or over a wide "single-point urban interchange" (SPUI). The specific application illustrated is an SPUI over an existing roadway where traffic must be maintained during construction. Details of the construction sequence are summarized in Figure 27.

Details of this design example include the following:

- The span is simple.
- There is a 96-inch-deep modified PCI BT girder with an 8-inch web
- The three girder segments are pre-tensioned.
- Girder segments are erected on permanent abutments and temporary towers.
- Splices between girders are cast when the deck is cast on the girder segments.
- Six post-tensioning tendons are used with 12- to 0.6-inch-diameter strands in each tendon.
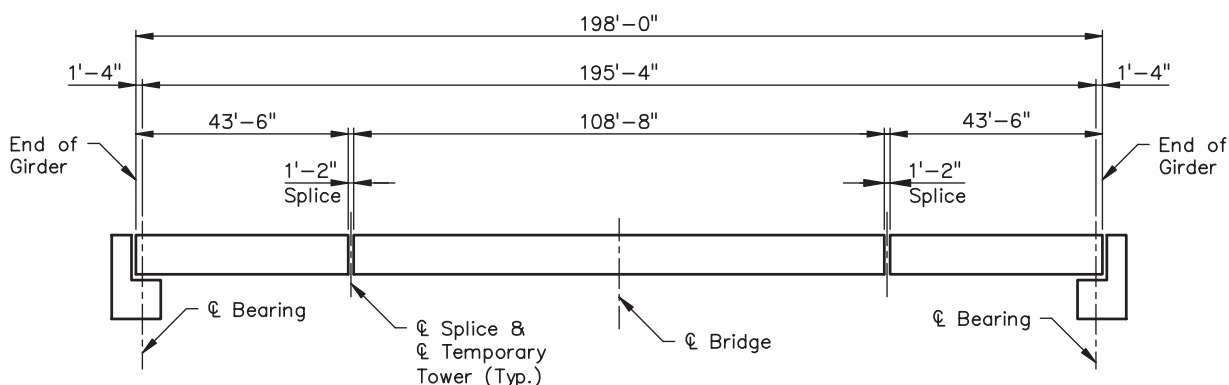
Figure 25.   Design Example 1: elevation.

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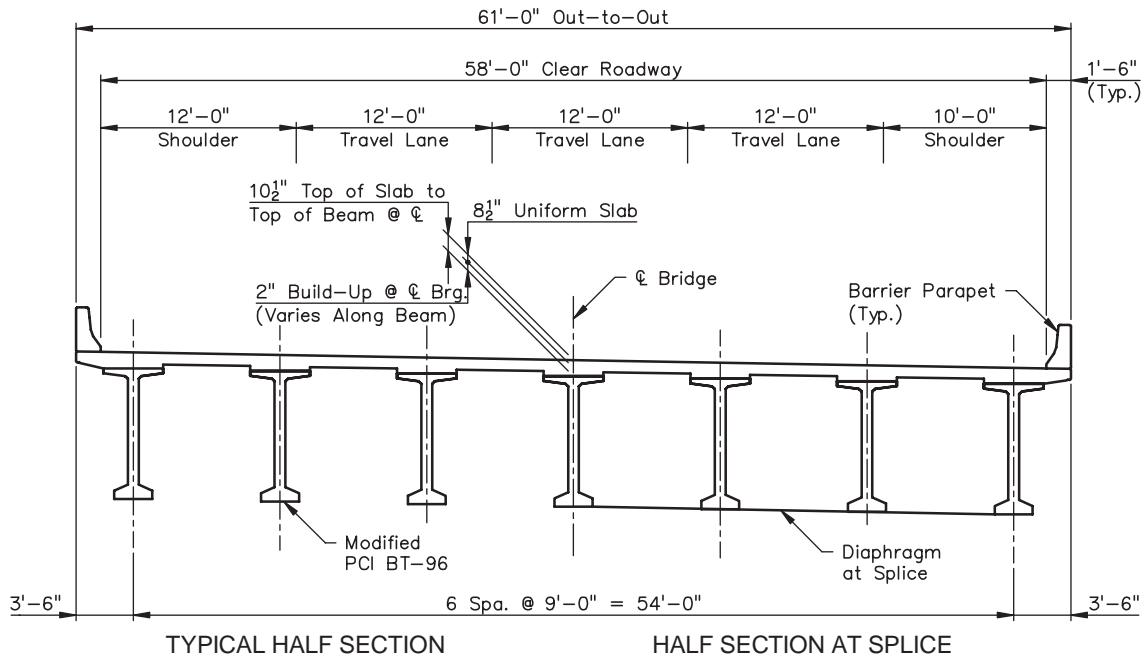
Figure 26.   Design Example 1: typical section of bridge.

- All post-tensioning tendons are stressed after the deck and splice concrete are placed.
- Post-tensioning tendon anchorages are located in end blocks at the end of each spliced girder.
- Conventional abutments and bearings are used.
- Design was performed using a custom spreadsheet.

Several special issues or features encountered during the development of this design example are discussed in the following.

1. The original bridge length and splice locations were set considering minimum traffic clearance requirements. However, it was found during design that the service limit state design of the girder was being governed at the splice locations, where only post-tensioning tendons were available to prestress the cross section. Therefore, the splice locations were moved closer to the ends of the bridge to improve the stress conditions at the splice. However, as the splice locations were moved outward, the center girder segment length increased. The center girder segments in the final bridge layout weigh approximately 125 kips, which is close to the maximum practical shipping weight for a girder segment. Therefore, the final girder segment layout was affected by traffic clearances, service limit state stress conditions, and girder weights for shipping.

2. Post-tensioning ducts were laid out using straight segments with short curves connecting them. All straight duct segments were parallel as shown in Figure 28. Also, the splice location was placed so that all ducts were straight as they crossed the splice, which simplifies splicing of the ducts. Given that this duct layout closely approximates a parabolic tendon profile, as shown in Figure 29, a parabolic profile was used to simplify friction and anchorage set loss computations. Although this type of duct layout has not been used for any bridge, it appears that fabrication of girder segments would be simplified by its use.

3. The number of pre-tensioned strands in the girder segments was selected to satisfy stress limits for handling and placing of deck concrete.

4. Stress limits for intermediate construction stages are not provided in the *LRFD Specifications*. Stress limits were assumed for the design based on the stress limits provided in the *Specifications*.

5. Equations in the *LRFD Specifications* for estimating prestress losses were modified to include the effects of using two types of prestressing. Instantaneous and long-term losses in both pre-tensioned strands and post-tensioning tendons were increased by the presence of the other type of prestressing.

6. The elevation of the bearings at the temporary shoring towers was raised to create additional camber in the structure. This additional camber offset the deflection resulting from creep and shrinkage to provide an acceptable deformation of the bridge at the end of the service life, as shown in Figure 30. The build-up required during construction was also reduced as a result of this elevation adjustment.

7. Post-tensioning tendons were only stressed from one end of the bridge. Stressing from both ends was investigated, but this approach did not provide a significant improvement in tendon force.

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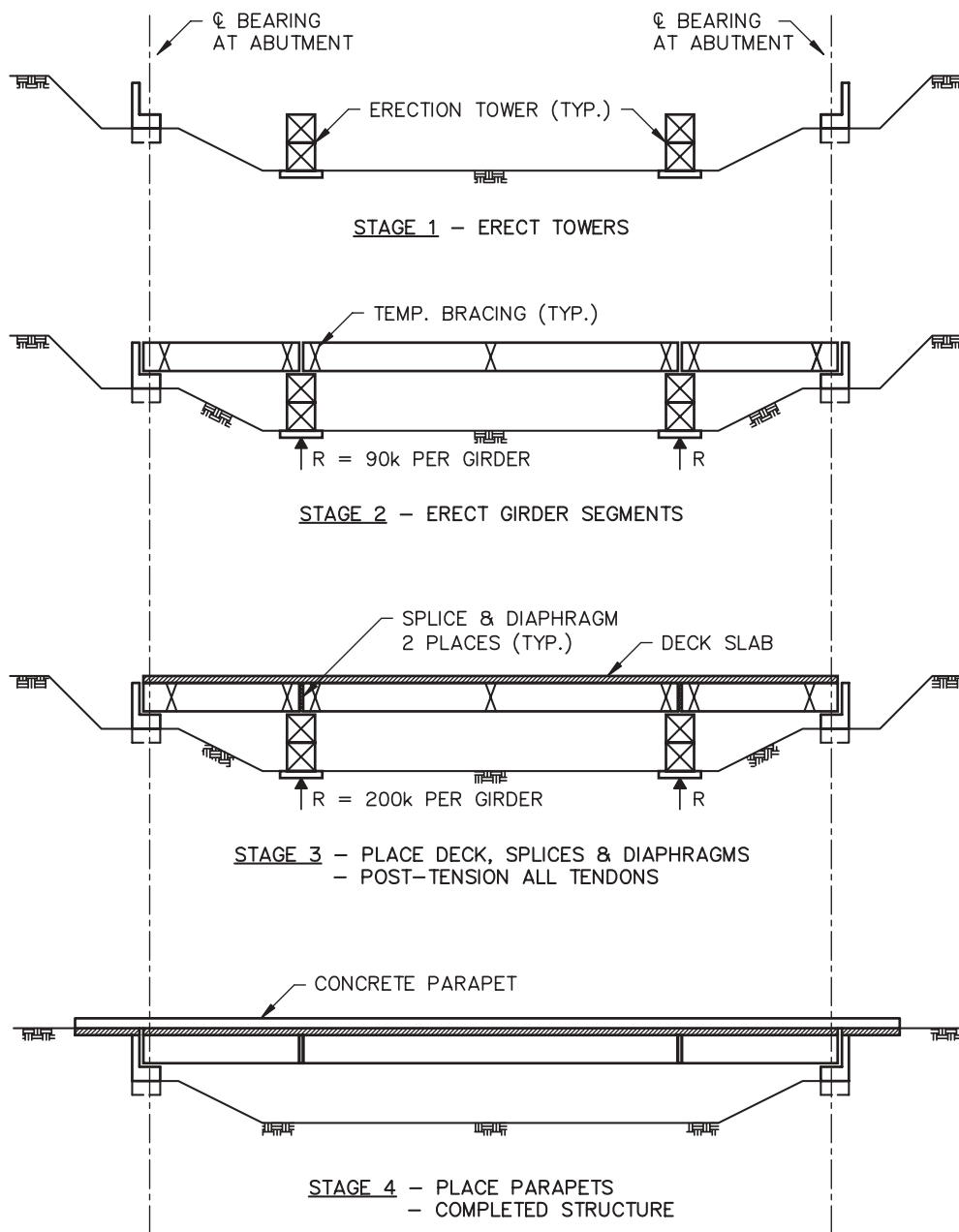
Figure 27.   Design Example 1: construction sequence.

8. The jacking stress for post-tensioning tendons was reduced below the limiting stress because the stress limit at the dead end governed.
9. It was found that the total moment due to deck dead load was the same with the girder shored at splice locations as it would be if the girder were not shored, since the shoring locations were symmetrical about the center of the span. With the bare girder segments supported by shoring, the moments caused by the deck dead load were a relatively small fraction of the total deck dead load moments. Therefore, most of the moment caused by the deck dead load was resisted by the composite section, which improved stress conditions significantly.

Although shoring during placement of deck concrete was unavoidable for the construction sequence used in the example, this information is useful when considering use of shoring for other bridges.

3.2.3 Design Example 2— Two-Span Spliced U-Beam Girder

The second design example is more complex, representing a two-span bridge crossing an existing interstate, where maintenance of traffic issues is very important. The elevation of the girder is shown in Figure 31 with the typical section shown

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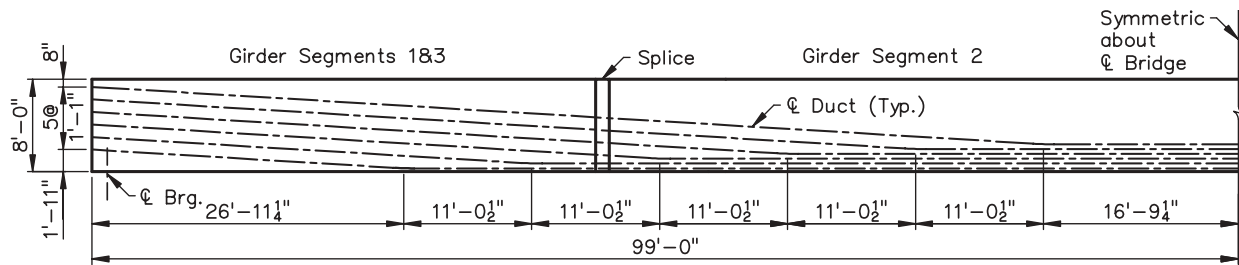
Figure 28.   Post-tensioning duct layout—elevation.
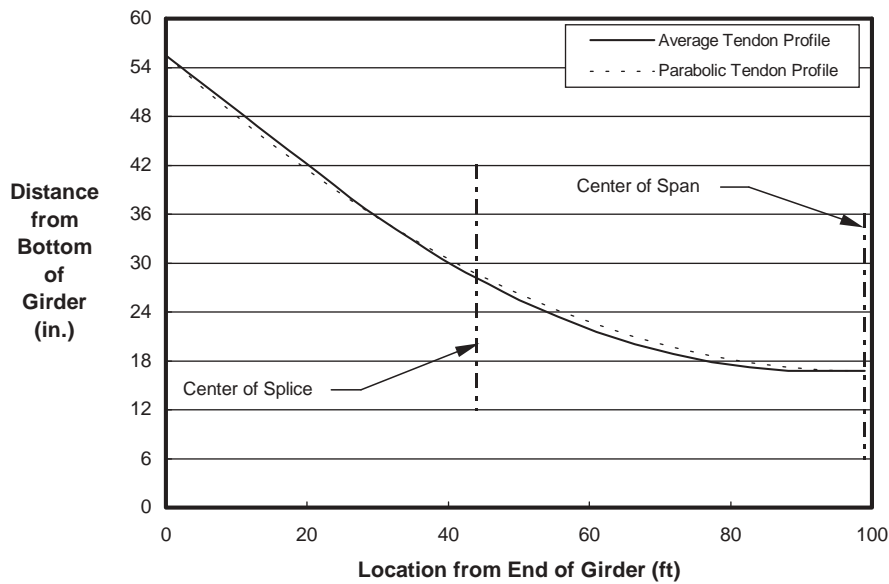
Figure 29.   Comparison of average and parabolic tendon profiles.
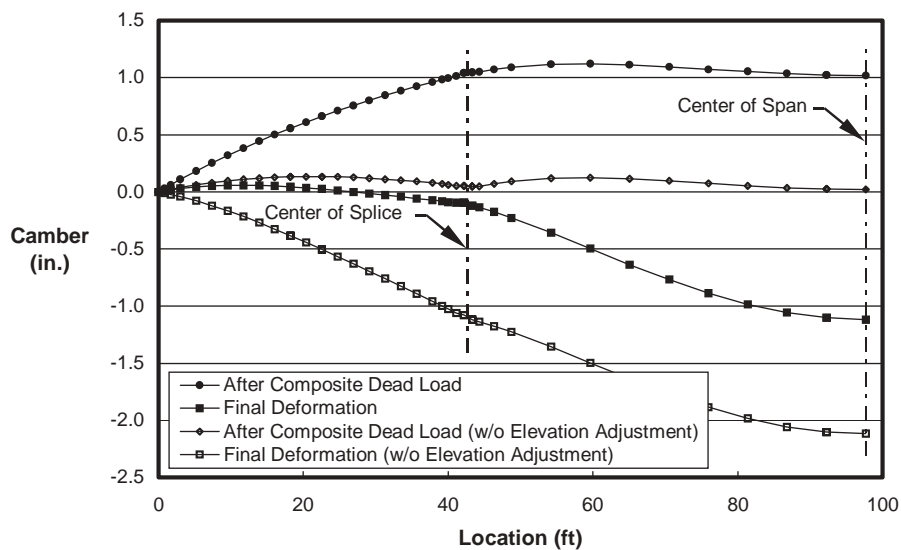
Figure 30.   Cambers with and without elevation adjustment.
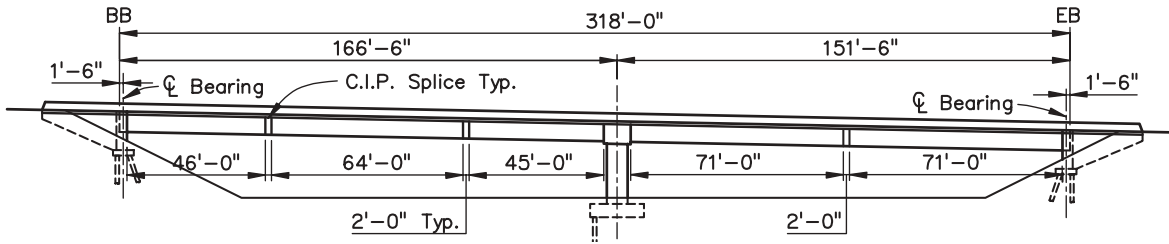
Figure 31.   Design Example 2: elevation.

in Figure 32. This example also illustrates the application of spliced girder construction to sites where seismic design must be considered. Details of the construction sequence are summarized in Figure 33.

Details of this design example include the following:

- Two unequal spans are used.
- There are 6-foot-deep U-beams.
- There are five girder segments, with several lengths. Three are pre-tensioned.
- Girder segments are erected on temporary towers and the interior pier.

- Splices between girders are cast after the girder segments are erected.
- Eight post-tensioning tendons are used with ten 0.6-inch diameter strands in each tendon.
- After splicing, first stage post-tensioning tendons are stressed.
- The deck is cast on the continuous girders.
- Second-stage post-tensioning tendons are stressed after the deck concrete is placed.
- Post-tensioning tendon anchorages are located in the cast-in-place end diaphragm, which rests on conventional bearings.
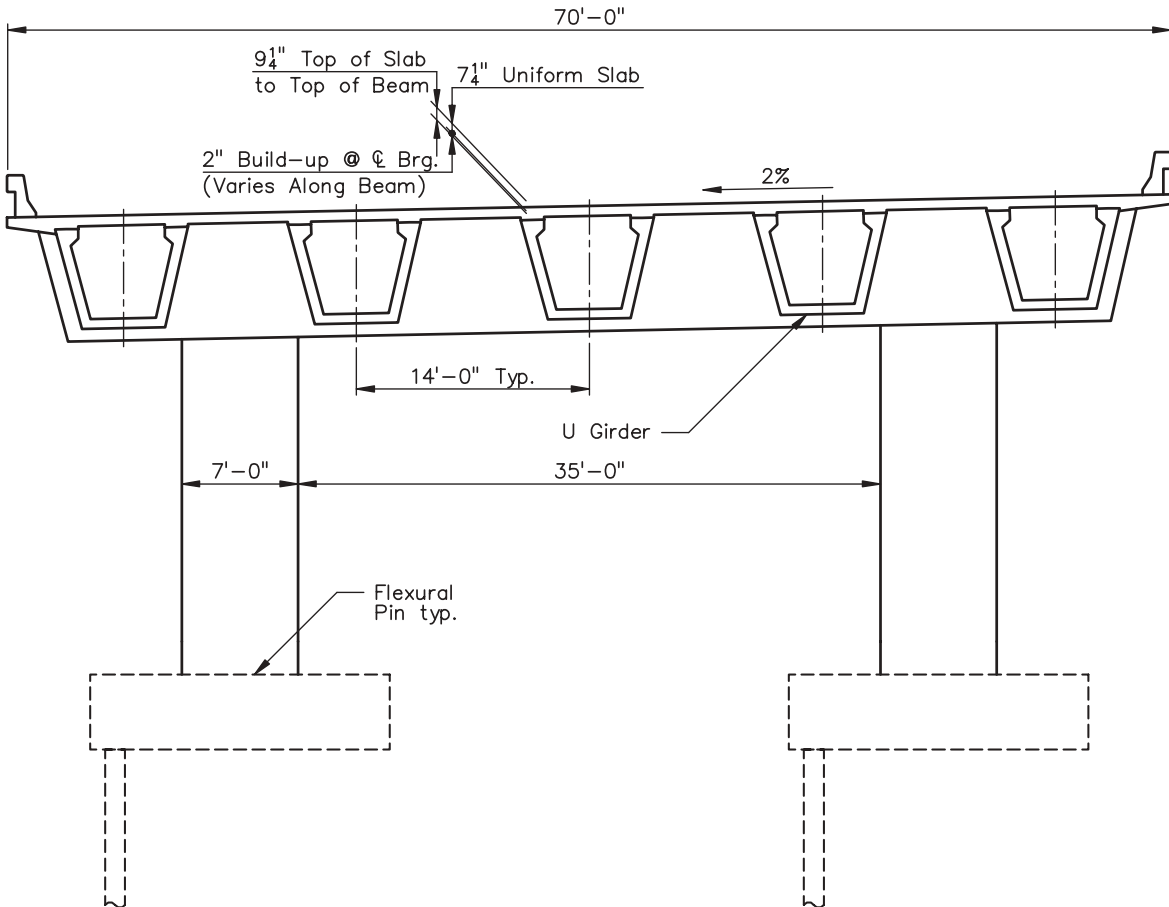
Figure 32.   Design example 2: typical section.

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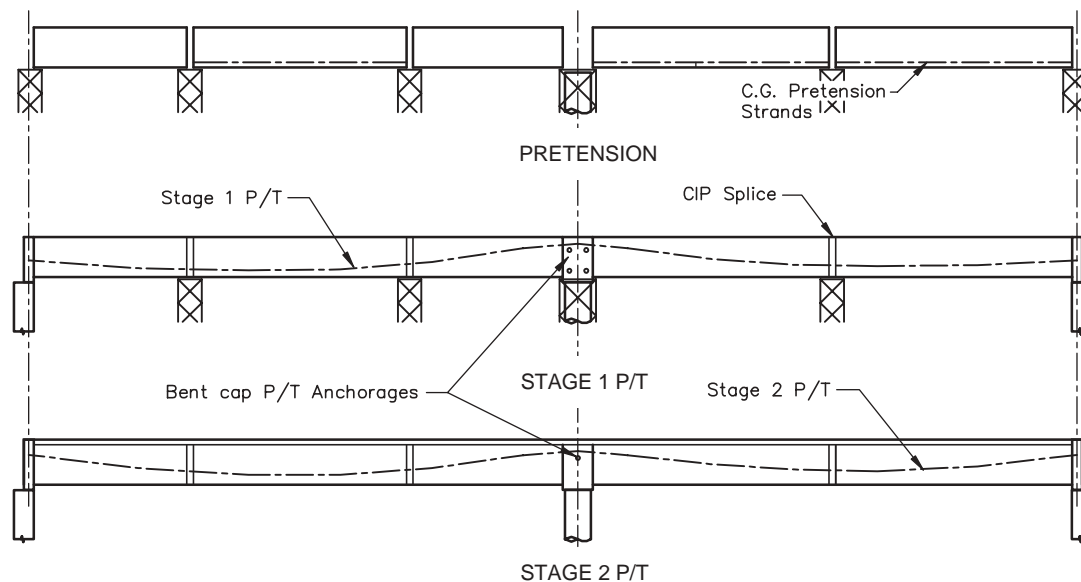
Figure 33.   Design Example 2: construction sequence.

- The interior pier cap is integral, providing a fixed connection between the superstructure and substructure.

Several special issues or features encountered during the development of this design example are discussed in the following.

1. Careful attention must be paid to the post-tensioning path and the influence on deflections. In particular, the prestress eccentricity or drape must be reduced in shorter spans to balance the prestress deflections and camber.
2. The effect of unbalanced prestressed deflection can result in an initial downward deflection that has the effect of pinning or holding down the falsework shoring towers, which could result in overloading and difficulty in removal of the towers after post-tensioning.
3. Several construction sequences are suitable for this example. Multi-stage post-tensioning was selected for this example to minimize shoring costs, because the falsework towers are designed to support girder self-weight only.
4. Continuity of the columns with the superstructure results in substantial economy in high-seismic regions because the foundations are designed to resist the reactions developed in the bent frame due to seismic forces and not the overstrength moment due to plastic hinging. In high-seismic regions, it is not uncommon for ordinary freeway bridges to have the substructure make up more than one-half of the overall bridge cost.
5. The LRFD equations for calculating nominal flexural resistance can result in the section being over-reinforced in the negative bending regions. Although over-reinforcement is allowed for prestress sections, meeting the demand requirements could be difficult and may control the design. Prior to increasing the dimensions of the concrete sections to meet this requirement, the designer should consider alternatives such as using strain compatibility analysis (moment-curvature programs) in lieu of code equations or adding mild reinforcement in the compression flange.
6. Given that U-beam sections are heavier than I or bulb-tee sections, weight is an important consideration in selecting segment joint locations. The upper limit for standard lifting and hauling equipment is approximately 80 tons (U.S. Customary). With the segment weighing approximately 1 ton per foot, the upper limit is 80 feet. Although heavier sections could be constructed, their use may slow the construction schedule and substantially increase costs.
7. With shorter segments, some girder segments may not require pre-tensioning, as demonstrated in this example. However, if the smaller girder segments are to be cast with the longer girder segments, equal pre-tensioning for all girder segments may be desirable to improve constructability.
8. An elastic analysis was performed to develop design moments and stresses, where time-dependent losses were estimated as a lump-sum value. A time-dependent analysis (TDA) substantially increases the design effort and complexity. This level of complexity has prevented some owners and designers from using this technology. One way to get around this is use a dead-load envelope (i.e., design for both full continuity and simply supported conditions of the dead load). This envelope could be used in place of the full TDA. Obviously, more efficient designs could result by performing a TDA, but using an envelope could allow for a more widespread use of this technology.

9. Post-tensioning should be designed with standard hardware in mind.

3.2.4 Design Example 3—
Continuous Three-Span Girder

The third design example, the most complex of the three, addresses a three-span bridge. A half-elevation of the bridge is shown in Figure 34 with typical sections shown in Figures 35 and 36. This type of structure has most often been used for crossings over navigable waterways, but can be used for any multi-span crossing where spans are greater than can be achieved using a constant depth cross section. The concepts of the example can also be extended for use on bridges with more than three spans. Details of the construction sequence are summarized in Figure 37.

Details of this design example include the following:

- There were three spans, with equal end spans (symmetrical about the center of the bridge).
- There were 78- and 90-inch-deep Florida-modified bulb-tees with a 9-inch web, haunched to 15-feet deep at the interior piers
- The five girder segments are each pretensioned.
- The girder segments are erected on permanent piers, temporary towers, and strongbacks.
- Splices between girders are cast after the girder segments are erected.
- Four post-tensioning tendons are used with two tendons with twelve 0.6-inch-diameter strands and two tendons with fifteen 0.6-inch-diameter strands.
- After splicing, first-stage post-tensioning tendons are stressed.
- The deck is cast on the continuous spliced girders.
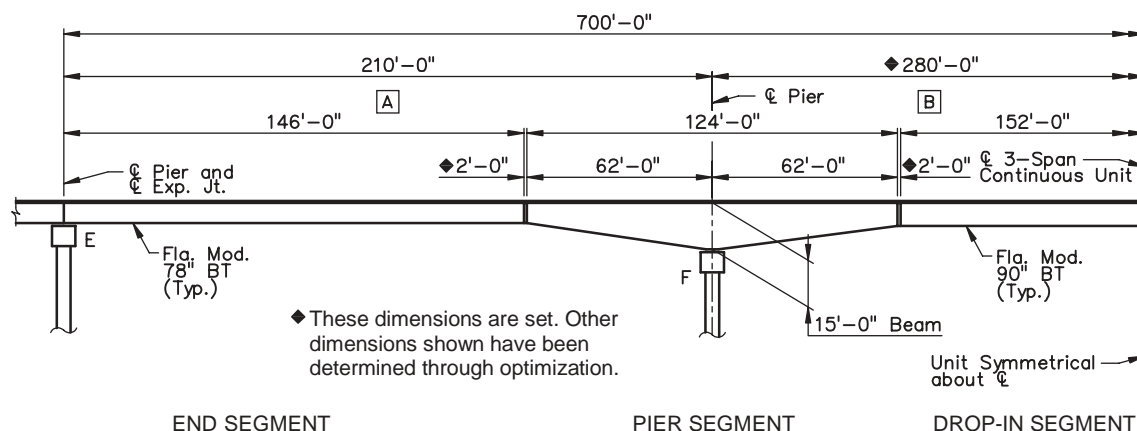
Figure 34.   Design Example 3: half elevation of bridge girder layout.
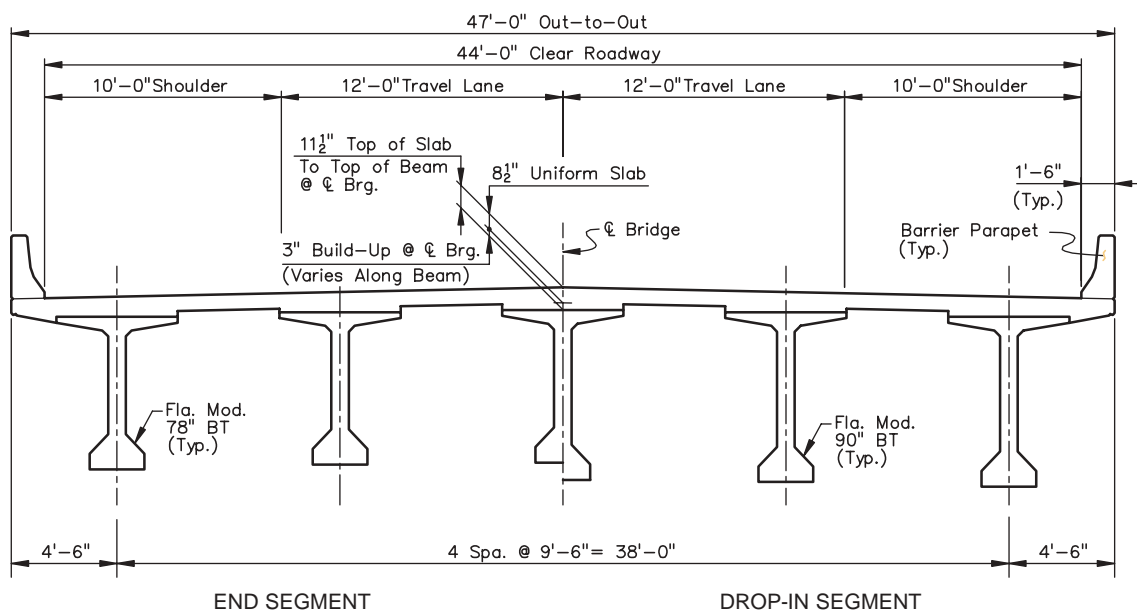
Figure 35.   Design Example 3: typical section at positive moment locations

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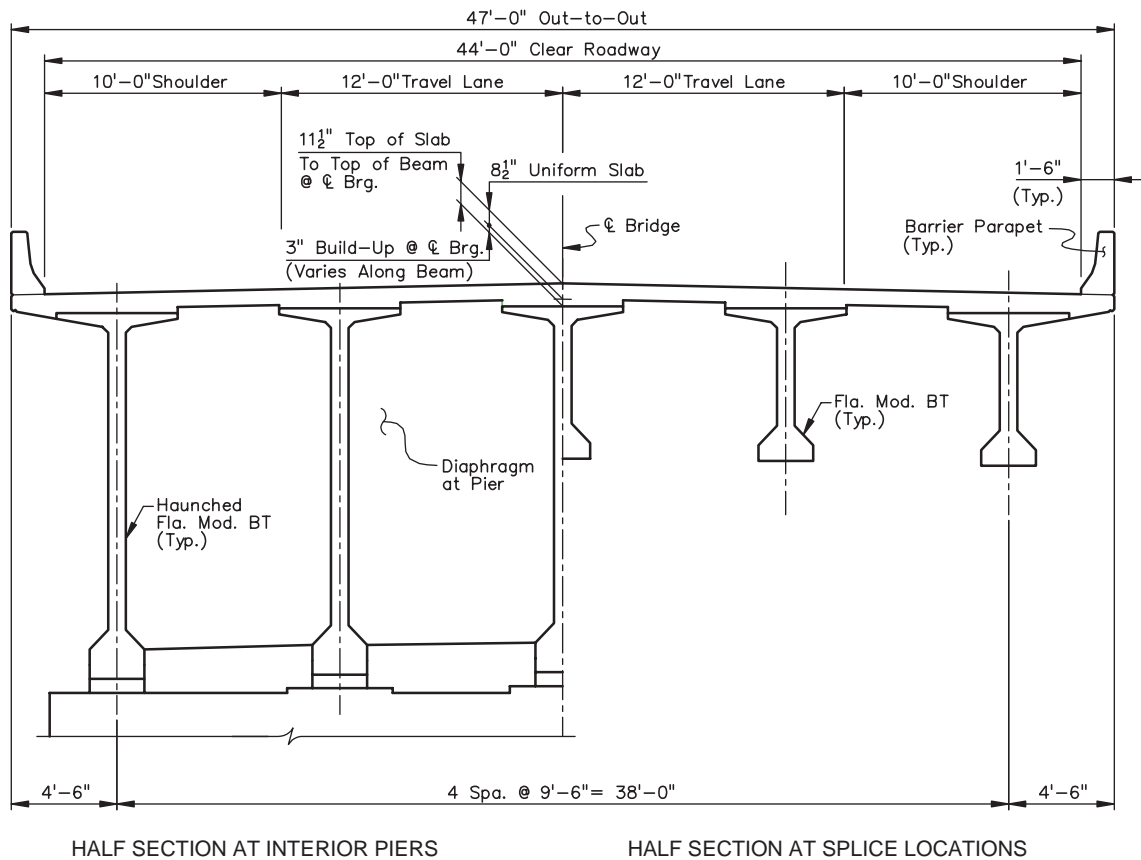
Figure 36. Design Example 3: sections at critical locations.

- Second-stage post-tensioning tendons are stressed after the deck concrete is placed.
- Post-tensioning tendon anchorages are located in end blocks at the end of each girder.
- Design was performed using a commercial software program that employed time-dependent analysis, including the effects of creep and shrinkage of concrete.

Several special issues or features that were encountered during the development of this design example are discussed in the following.

Despite meeting service limit state requirements, the design of the bridge girders was governed by live load deflection limits and the strength limit state. To address the live load deflections, the depth of the pier girder segments was increased from 12 feet 6 inches to 15 feet 0 inches. To improve the strength of the girder at the interior pier, the depth of the bottom flange was made to vary along the length of the girder segment so that it was slightly deeper at the interior pier in order to provide an increased compression area that could handle the negative moments (Figure 38). The drop-in girder segments were initially assumed to be the same cross sections as the end girder segments (78 inches deep), but the live load

deflection at the center of the bridge exceeded the optional limit specified by the *LRFD Specifications* (Table 13). The depth of the drop-in girder segment was increased to 90 inches to bring the deflection of the structure within the allowable limit (Figure 39).

In order to minimize the amount of build-up required and to better match the girder profile to the roadway profile, the bearing elevations at the interior piers and temporary support towers were adjusted. The bearing elevations at the temporary support towers were lowered by 0.5 inches while the bearing elevations at the interior piers were raised 0.5 inches (Figure 40). A major reason for the correction is the nearly 1-inch downward deflection of the tip of the pier girder segment when the drop-in girder segment is placed.

Calculations using the LRFD equations for nominal flexural resistance indicated that the section was over-reinforced in the negative moment regions ($c/d > 0.42$). Although over-reinforcement is allowed for prestressed sections, meeting the demand requirements could be difficult and may control the design. Prior to increasing the dimensions of the concrete sections to meet these criteria, the designer should consider alternatives such as adding mild reinforcement in the compression flange. In some cases, the amount of mild reinforcement

Erect Piers and Temporary Supports
Erect Pier Girder Segments [Stage 1]

Sym. about
Center of
Bridge

Erect End Girder Segments [Stage 1]

Erect Drop-in Girder Segments [Stage 1]

Cast Splices [Stage 2]
Stress Stage 1 Tendons [Stage 3]

Cast Deck Slab [Stage 4]
Stress Stage 2 Tendons [Stage 5]

Cast Barriers/Parapets [Stage 6]
Open Bridge to Traffic [Stage 7]
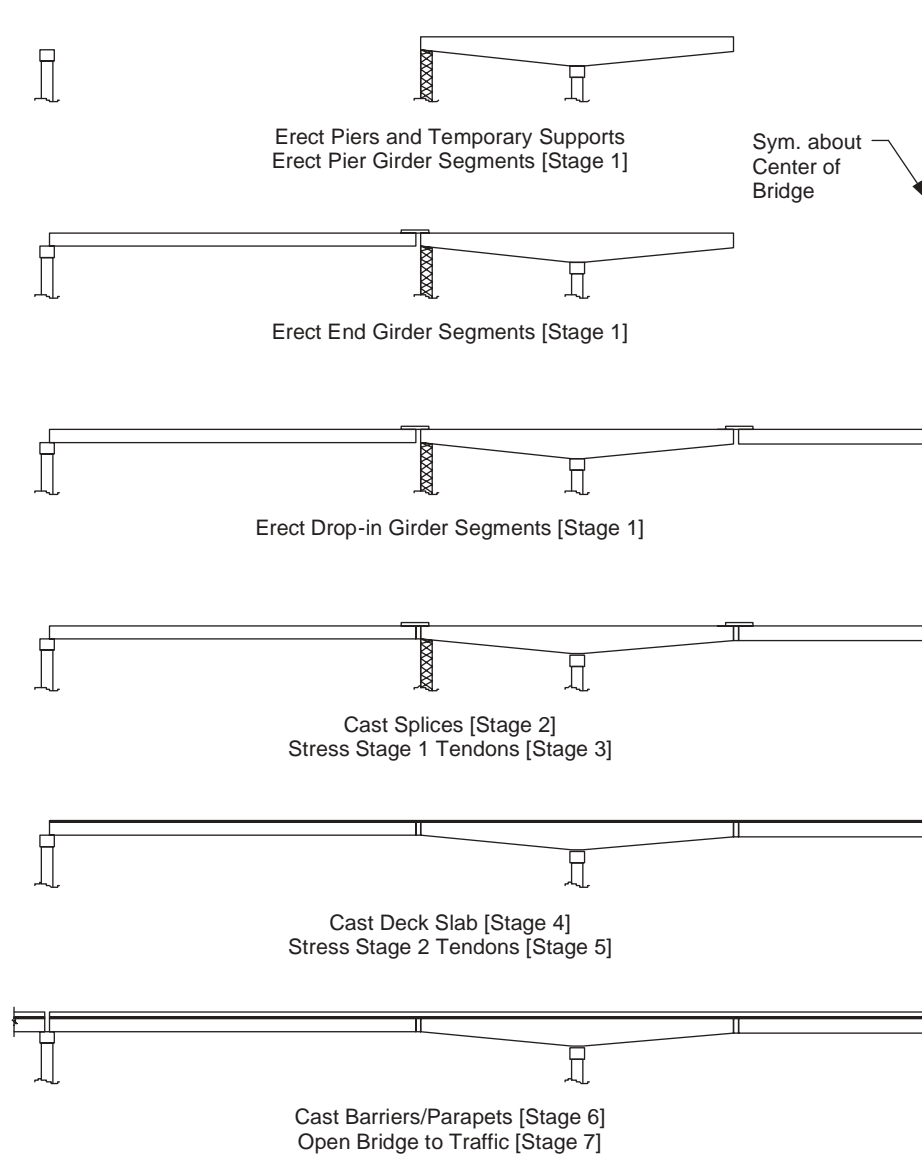
Figure 37.   Design Example 3: construction sequence.

required in the compression flange can hinder constructability, and the designer should consider modifying the compression flange.

Pre-tensioned strands were added to the drop-in girder segment to increase the camber and improve the final girder profile.

Under full dead loads, the deck remained in compression, except at the ends of the bridge where the prestress effect was apparently reduced because of the end blocks. Under full service loads with live load, the deck only went into tension at the splice locations. Therefore, the goal of providing a deck that is always in compression (longitudinally) or is subject to only a minor tension under service loads was achieved.

Post-tensioning tendons for both stages were stressed from both ends of the bridge. Stressing from one end was investigated, but this approach did not provide an acceptable tendon force along the bridge.

Because of the long spans and deep girder segments, the weight of the girder segments ranged from 109 to 121 tons. These weights significantly exceed typical weight restrictions for highways, so these girder segments would have to be shipped by barge to the job site. Special lifting equipment might be required to place the girder segments.

Before using a software package for design, the designer must understand the methodology behind the analysis performed by the software and the proper method of constructing a model. This may require communicating with the technical

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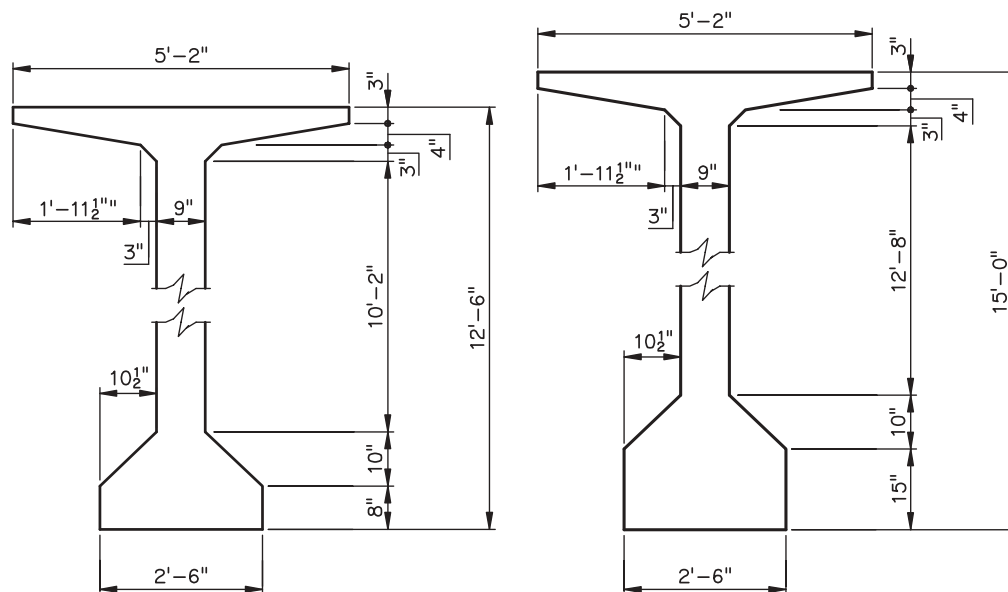
Figure 38.   Cross sections of pier girder segments.

support for the software or seeking the advice of a person familiar with that particular package. Details of the results should be carefully reviewed to satisfy the designer that the software is performing the analysis and design properly.

3.3 PROPOSED REVISIONS TO
THE *AASHTO LRFD SPECIFICATIONS*

A major goal for this research project was to develop proposed revisions to the *AASHTO LRFD Bridge Design Specifications* where needed to allow, facilitate, or clarify the use of extended spans for precast prestressed concrete bridge girders. To fulfill this project goal, the *Specifications* were reviewed to identify provisions that need revision. The review revealed that the most significant issues were related to the refinement of the *Specifications* for the design of spliced girders. Significant revisions do not appear necessary to implement strategies for extending span ranges other than the splicing of girders. Therefore, proposed revisions were prepared to clarify the applicable provisions for spliced girder construction and to improve the organization of the provisions

that apply to spliced girder construction. Few new provisions were introduced in the proposed revisions. Additional revisions are needed to refine the specifications for use with spliced girder bridges.

The proposed revisions are presented in Appendix E1 using the same format as the *LRFD Specifications*, including a column for commentary. The proposed revisions are presented in two formats: one with revisions marked using standard typographical conventions to indicate areas of change and the second with revisions incorporated to present the intended form and content of the revised *Specifications*.

The most important issue identified to improve the *Specifications* for application to spliced girder design is to clarify the distinction between spliced girder and segmental construction. Revision of the *Specifications* in this area is essential so that designers can easily find and interpret the requirements that apply to spliced girder construction. The most reasonable approach to making the distinction was to treat spliced girder bridges as conventional prestressed concrete girder bridges with additional design requirements for the splice locations where some provisions related to segmental construction are applicable. This distinction prevents the

TABLE 13   Live load deflections and limits

Span	Maximum Live Load Deflections Initial Design (in.)	Maximum Live Load Deflection Final Design (in.)	Allowable Live Load Deflection (in.)
Exterior Span	3.7	3.16	3.15
Interior Span	5.2	4.00	4.20
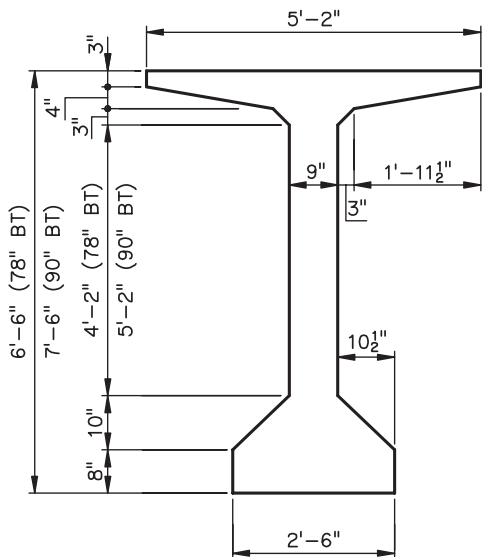
Figure 39.   Cross sections of end girder and drop-in girder segments.

general, applying to superstructure and other types of elements. The definition of "Segmental Construction" is proposed to replace the definition of "Segmental Components" currently appearing in Article 5.2.

- **Spliced Precast Girder**—A type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using post-tensioning, to form the completed girder. The bridge cross section is typically a conventional structure in which multiple precast girders support a cast-in-place concrete composite deck. This type of construction is not considered to be segmental construction for the purposes of these Specifications.
- **Segmental Construction**—The fabrication and erection of a structural element (superstructure and/or substructure) using individual elements, which may be either precast or cast-in-place. The completed structural element acts as a monolithic unit under some or all design loads. Post-tensioning typically is used to connect the individual elements. For superstructures, the individual elements are typically short (with respect to the span length), box-shaped segments with monolithic flanges that comprise the full width of the structure.

The major points of the proposed revisions may be summarized as follows:

unnecessary and inappropriate application of requirements for segmental construction, which appear to be intended for short-segment cast-in-place or match-cast box girder bridges, to spliced girder bridges. Revisions have been worded to preserve as much flexibility in design options as possible to allow cases in which the boundaries between spliced girder and segmental construction, and other concepts, may be blurred.

Definitions have been proposed for the *LRFD Specifications* to clarify the distinction between the two types of construction. The definition for "Spliced Precast Girder" is only intended to apply to superstructure elements, while the definition for "Segmental Construction" is intended to be more

- Article 5.14.1.3, Spliced Precast Girders, has been added to address the detailing and design of spliced girder bridges specifically. This new article contains provisions from Article 5.14.1.2.6, Transverse Construction Joints, and some appropriate provisions related to spliced girder construction from Article 5.14.2, Segmental Construction. Additional information from other articles in the *Specifications* and other sources has also been added.
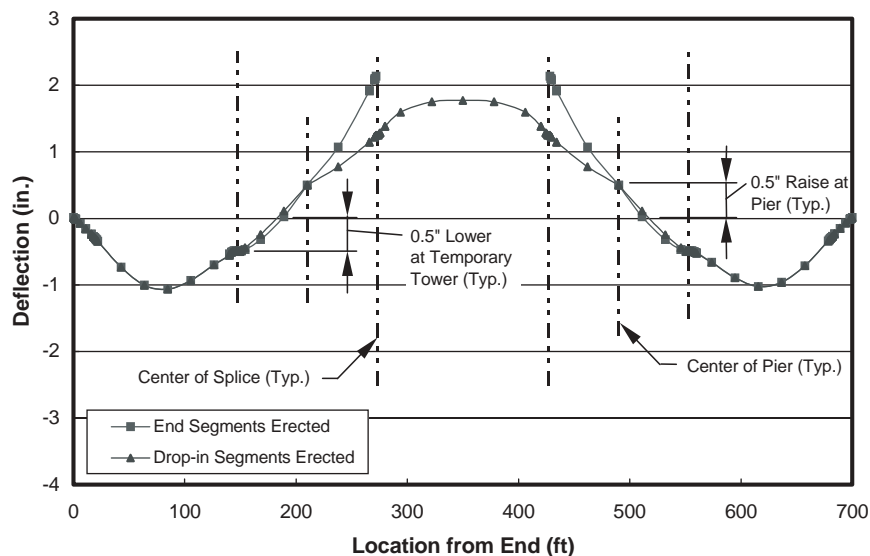
Figure 40.   Final design—deflection of girder segments at erection (adjusted).

- Article 5.14.2.4.7, Precast Segmental Beam Bridges, has been deleted, with its contents distributed to the new Article 5.14.1.3 as appropriate.
- The provisions of Article 5.14.1.2.7, which address precast girders made continuous, have been moved to a new Article 5.14.1.4, Simple Span Precast Girders Made Continuous. The introduction to the article has been revised to reflect the intent of this type of construction more clearly, limiting it to simple-span construction with closure joints at interior piers and to coordinate its requirements with the new section on spliced precast girders.

- Article 5.14.1.2.8, Longitudinal Construction Joints, has been moved to Article 5.14.4.3.3d, which is a sub-article related to precast slab bridges.
- With the addition of Articles 5.14.1.3 and 5.14.1.4, current Article 5.14.1.3, Cast-in-Place Girders and Box and T-Beams, must be renumbered as Article 5.14.1.5.
- Several other articles have been revised to address additional issues and concerns. Some others have been included for comment or to note ongoing revision activity by the AASHTO Subcommittee on Bridges and Structures Technical Committee for Concrete Structures, T-10.

CHAPTER **4**

CONCLUSIONS AND SUGGESTED RESEARCH

4.1 CONCLUSIONS

4.1.1 General Conclusions

The objective of this research project was to identify options and provide resources that would encourage the use of longer spans for precast prestressed concrete girders. The research has focused on spliced girder technology because it has the greatest potential for increasing the span ranges of precast prestressed concrete girders.

This research has fulfilled the objective of the research by bringing together into a single report the full range of information required for designing and constructing precast prestressed concrete girders with extended spans, including spliced girder bridges. This report has demonstrated that this assembled information can be applied to design a wide range of applications from single spans to multiple spans, from urban interchanges to remote highways. This report reflects the experience and recommendations of a team of experts having extensive experience in the design, fabrication, and construction of different types of spliced girder bridges in different parts of the country. The report also includes significant contributions from other experts in this type of construction from across the country and Canada.

Much of the information compiled in this report has not been previously available. Without this information, the use of extended-span and spliced girder technology has been limited in many areas, which denies owners and the public the benefits of spliced girder construction. However, the successful implementation of the products of this research can be expected to result in an increase in the number of projects designed using longer-span precast prestressed concrete girders and spliced girders, especially in states or regions where they have not been previously used.

The use of the technology presented in this report will result in extending the spans achievable by precast prestressed concrete girder structures, meeting the needs of longer spans that are more frequently being required for safety, aesthetic, environmental, and economic reasons. Furthermore, with longer spans from precast prestressed concrete girder bridges, the economy of long-span bridges will be improved by increased competition between different types of long-span structures.

The design and construction recommendations provided by this research project will be useful to owners, designers, fabricators, and contractors (1) to learn how to use spliced girder technology or (2) to improve their current understanding of spliced girder technology. Many aspects of design and construction have been addressed to give a complete understanding of spliced girder technology.

4.1.2 Specific Conclusions

The following specific conclusions can be drawn from the research performed as part of NCHRP Project 12-57:

- Spliced girder bridges have a proven track record, with more than 250 spliced girder bridges having been constructed, some of which were constructed as early as 1952, shortly after the first prestressed concrete bridge in the United States was built.
- Various options are available for increasing the spans of precast prestressed concrete bridge girders. These options range from the enhancement of material and design parameters to the addition of post-tensioning and splicing of girders.
- For the specific conditions of the comparative design study (Appendix B2), an increase in the maximum span of up to 17 percent was achieved using a combination of enhanced material properties, and an increase of 33 percent was achieved using a decked bulb tee.
- Spliced girders can be successfully designed using the existing *AASHTO LRFD Specifications*.
- The *AASHTO LRFD Specifications* should be revised to clarify and improve their usefulness for spliced girder bridges. An improved specification will facilitate the application of the *LRFD Specifications* to the design of spliced girder bridges.
- Design resources are available for design of spliced girder bridges, including computer design programs.

4.2 SUGGESTED RESEARCH

It appears that research to remove major obstacles to the use of the various approaches to extending the span ranges of precast prestressed concrete bridge girders is not urgently

needed because most approaches to increasing span lengths are extensions of current materials and procedures, rather than introductions of new concepts or materials. The only significant exception to this appears to be decked bulb-tee girders, where research is needed to develop and demonstrate a connection between units that will allow the use of such girders on primary roads. This research has already been approved for FY 2004 as NCHRP Project 12-69, "Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges."

Research is needed in some areas to refine design procedures or results of design requirements, such as the following:

- Continue review and revision of *AASHTO LRFD Specifications* to refine the *Specifications* for use with spliced girder and other long-span girder bridges.
- Develop and/or evaluate alternatives to full deck removal and replacement for extending the service life of bridge decks.
- Perform a comparative design study of post-tensioned bridges designed for full deck replacement, with and without the deck in place when the post-tensioning tendons are stressed. The goal of the research would be to quantify any increased cost and decreased structural efficiency that results from prohibiting the stressing of post-tensioning tendons on bridges with the deck in place.
- Develop design procedures and specifications for reduced (or specified) density concrete, including evaluation of prestress losses and long-term deflections.
- Develop and/or evaluate methods to estimate long-term deflections (currently multipliers are used).
- Verify prestress losses, especially for long-span post-tensioned members.
- Review methods for evaluating the effect of overweight loads on roads and bridges leading to recommendations that could result in more uniform hauling requirements across the country.

4.3 OTHER ACTION ITEMS

This study has identified several issues that are not material or design related and would require cooperation from all of the parties involved in the design and construction of bridges to resolve. Many of these issues are covered in Appendix F, so they will not be repeated here.

It is also recognized that impediments to the implementation of spliced girder technology in areas where it is not currently used may include the following:

- Reluctance of owners to use technology with which they and their consultants, fabricators, and contractors are unfamiliar;
- Reluctance of designers, fabricators, and contractors to use technology with which they are unfamiliar;
- Reluctance of contractors to use technology that may add risk, additional subcontractors, and other complicating factors to a project;
- Limitations on transportation of girder segments, because of weight or length, in the jurisdiction of the project or in adjacent jurisdictions through which girder segments must be transported;
- Reluctance of owners to use post-tensioned concrete structures because of concerns regarding inspection of ducts after grouting and for the life of the structure;
- Lack of availability of design tools (software);
- Concern about performance of spliced girders in extreme events, such as earthquakes and vessel impact;
- Concern about availability of erection equipment for large segments or use of temporary supports;
- Lack of cost-estimating data to allow appropriate comparisons of design alternatives; and
- Lack of clarity or restrictive provisions in the *LRFD Specifications* addressing spliced girder technology.

These concerns have been addressed by this report. Therefore, an aggressive implementation and distribution strategy is recommended in order to fully realize the potential for the technologies discussed in this report.

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Listing of Spliced Girder Bridge Projects

Table of Contents

APPENDIX A
COLLECTION OF PROJECT INFORMATION

Information was collected to describe existing projects in which extended span precast, prestressed concrete girders have been used and to document the experience related to these structures. The data collection was limited to spliced girder bridge projects.

This appendix contains an evaluation and detailed listing of the collected data, and the form, which was used to collect the data.

Appendix A1   Data Collection Form

This appendix contains the three-page data collection form and instructions.

Appendix A2   Evaluation of Data Collection Results

This appendix evaluates and summarizes the collected project data by region. For each region, the general usage of spliced girders, type of girders, reasons for using extended span girders, challenges, and special features are discussed. The appendix closes with an evaluation of general trends that can be observed in the data.

Appendix A3   Listing of Spliced Girder Bridge Projects

This appendix contains a table that summarizes the collected project data by region. A second table contains detailed data for each bridge for which data was collected.

APPENDIX A1
Data Collection Form

SPLICED I-GIRDER DATA COLLECTION FORM
NCHRP 12-57: EXTENDING SPAN RANGES OF PRECAST, PRESTRESSED CONCRETE GIRDERS

Name: _____   **Phone:** _____

Organization: _____   **E-mail:** _____

Address: _____

_____   Please return completed file to:

_____   *reid.castrodale@rwhitehead.com*

_____

SPLICED GIRDER DEFINITION:

The use of precast, prestressed concrete girders that are fabricated in pieces shorter than the final span lengths or structure length, which are then joined together using post-tensioning to form the final structure. Elements of spliced girder technology typically differ from "segmental" construction as follows: segments are a greater portion of the span length; joints between segments are generally not match-cast and epoxied; deck segments are not full deck width; and bulb-tee or u-shaped sections are used rather than box shapes.

INSTRUCTIONS:

General:

Please fill in the Name, Organization and contact information boxes above. This information need only be entered once. It will be copied to all individual data sheets. Use the buttons below to navigate through the file, create new worksheets, save worksheets and print worksheets.

Section A: General Data / Description

Enter the Bridge Name or a descriptive locator for the bridge (preferably the roadway route the bridge carries -- i.e., US19 -- and the roadway route, river, etc. that the bridge crosses), the Location (including State and/or Country) and the Estimated Year of Completion. If the bridge will not be built or was built using a different alternative, enter the year that design was completed and see Item F.10 in the Data worksheet.

Section B: Involvement

Enter as much information as possible concerning the parties involved with the different aspects of the project. This information is necessary to give proper credit to all parties involved and to follow up concerning details of design and construction.

Section C: Span / Geometric Data

Enter Span Dimensions and Arrangement for spliced girder units (e.g., 200'-260'-200' units w/integral piers, etc.). Enter whether the Geometric Feature crossed is a river, road or other and the Total Length of spliced girder bridge portion (For the previous example, four units of 200'-260'-200' would give 2640'.) Enter out-to-out Deck Width. Enter Component Lengths for Pier Sections and Drop-In Girders, and check correct box for Splice Locations. For In-Span Splice Locations, the Pier

Section E: Design Tools / Trials

This information is primarily directed to designers. Enter any information concerning Software, Specifications and Design References used. Also, enter a brief description of any challenges that were met during design (e.g., no approved specifications available for spliced girder design; codes do not address certain areas of design; no software available to analyze time-dependent stage construction using composite sections, particularly regarding stress checks).

Section F: Cost / Economics

Enter overall Cost of Structure and the corresponding Area of Bridge Deck (for entire project, not just spliced girder portion) -- Area of Spliced Girder Bridge Deck and Total Length of Spliced Girders are calculated automatically by program based on previous entries, but can be overridden (Check "Calculation: Manual" above Box F.3. Enter the Superstructure Cost and Substructure Cost for the Total Structure (entire bridge) and the Superstructure Cost and Substructure Cost for just the Spliced Girder Portion. Enter the Alternative Structure Type or base bid alternative (e.g., steel plate girder; concrete segmental; etc.) and check whether the spliced girder superstructure was built as the result of a Design/Bid/Build process, a Design/Build process or a Value Engineering Contractor Proposal. Enter the low bid for the Alternative Structure. If the spliced girder alternative was designed but not built due to lack of funding or because the Alternative Structure was selected, check the box in Section F.10. Enter information concerning Design Costs and CEI Costs as either an Absolute $ amount or as a percentage of construction cost (Relative).

Section G: Precast Production / Shipping

Enter the Casting Location (city or location of casting yard relative to bridge site).

Enter Span Dimensions and Arrangement for spliced girder units (e.g., 200'-260'-200' units w/integral piers, etc.). Enter whether the Geometric Feature crossed is a river, road or other and the Total Length of spliced girder bridge portion (For the previous example, four units of 200'-260'-200' would give 2640'.) Enter out-to-out Deck Width. Enter Component Lengths for Pier Sections and Drop-In Girders, and check correct box for Splice Locations. For In-Span Splice Locations, the Pier Section Component Length will be the distance from the splice in one span to the splice in the next span. For Pier Splice Locations, the Pier Section Component Length will be 0 (zero), i.e., Drop-In Girder Component Length is the entire span

Section D: Girder / Material Data

Enter girder Concrete Strength and any Special Concrete Admixtures used. Enter Girder Type or Description of Girder (e.g., Florida 78" Bulb-Tee; Modified PCI 72" Bulb-Tee w/9" web thickness; etc.). Enter Size of Tendons and Number of Tendons per girder (e.g. 7 - 12 x 0.6"-dia. would indicate the use of 7 tendons per girder consisting of 12-strand tendons with 0.6"-dia. strands. etc.)

Enter the Casting Location (city or location of casting yard relative to bridge site). Enter the Method of Shipping and approximate Shipping Distance between the casting yard and the bridge site.

Section H: Justification / Performance / Comments

Enter a brief description or Reason for Selecting Spliced I-Girders and any Comments concerning design or construction challenges, biddability aspects or Performance of the structure.

Section I: Precast Erection Methods / Details

Enter a brief description of the Erection Sequence and Post-Tensioning Sequence (e.g., Haunched pier segments were erected at either end of span; drop-in girders were erected; first-stage P.T. tendons were stressed; deck was cast; second-stage P.T. tendons were erected; etc.) Enter Method of Erecting Girders (e.g., barge-mounted crane; deck mounted crane; ground based crane; etc.) and Method of Supporting Drop-In Girders prior to Splicing (e.g., temporary tower under splice; strongbacks supporting drop-in girder with backspan tied to a temporary tower; etc.).

SPLICED I-GIRDER DATA COLLECTION FORM
NCHRP 12-57: EXTENDING SPAN RANGES OF PRECAST, PRESTRESSED CONCRETE GIRDERS

Survey Date: July 27, 2001

Name of Respondent: _____ Phone: _____ Project No.: _____ Records Available: *(check, as applicable)*

Organization: _____ E-mail: _____ Proj. No.Type: ☐ FAP ☐ Federal ☐ State ☐ Other *(check, as applicable)*

☐ Plans
☐ Specs.
☐ Photos
☐ Inspection Reports

A. GENERAL DATA / DESCRIPTION

1. Bridge Name / Description	2. Location (City & State or Country)	3. Est. Completion Year

C. SPAN / GEOMETRIC DATA

1. Span Dimensions / Arrangement	2. Geometric Feature Crossed	3. Total Length (ft)	4. Deck Width (ft)

5. Component Lengths		6. Splice Locations			7. No.Girders / Deck Width
a.Pier Section	b.Drop-In Girder	a.In-Span ☐	b.Piers ☐	c.Both ☐	

<----- (check one) ----->

8. Horizontal Curvature	
a. Degree of Curv.	b. Radius of Curv.

F. COST / ECONOMICS

Automatic: ◉ <----- Calculation? -----> Manual: ○

1. Project Totals		2. Area Spliced Girder Bridge Deck (ft²)	3. Total Quantity Spliced Girders (ft)
a. Cost of Structure	b. Area Bridge Deck		

3. Total Structure		4. Spliced Girder Portion Only	
a. Superstructure Cost	b. Substructure Cost	a. Superstructure Cost	b. Substructure Cost

5. Alternative Structure Type (if applicable)	6.Des./Bid /Build ☐	7.Design /Build ☐	8.VECP by Contractor ☐

<----- (check one) ----->

9. Alternative Structure Bid Price	10. Designed, but Never Built	11. Design Costs (Absolute or Relative)	12. CE&I Costs (Absolute or Relative)

(check, if applicable)

I. PRECAST ERECTION METHODS / DETAILS

1. Erection Sequence / Post-Tensioning Sequence Details (Description)

2. Method of Erecting Girders	3. Method of Supporting Drop-In Girders / Splicing

---

B. INVOLVEMENT

	a. Organization	b. Contact Person	c. Phone Number	d. E-Mail Address
1. Owner				
2. Designer				
3. Contractor				
4. Precaster				
5. P.T.Supplier				

D. GIRDER / MATERIAL DATA

1. Concrete Strength (psi)	2. Special Concrete Admixtures	3. Girder Type/Description	4. Number & Size of Tendons

E. DESIGN

1. Software / Other Tools	2. Specifications

3. Other Design References	4. Challenges / Obstacles

G. PRECAST PRODUCTION / SHIPPING

1. Casting Location	2. Method of Shipping to Site	3. Shipping Distance (mi)

H. JUSTIFICATION / PERFORMANCE / COMMENTS

1. Reason for Selecting Spliced I-Girders

2. Comments / Special Project Features (I.e., construction difficulties, special considerations, biddability aspects, other benefits/detriments)

7/15/03 8:36 AM

N:\PROJ\2657-142\Bridge Info\[NCHRP 12-57 Blank Data Collection Form.xls]Data (1)

APPENDIX A2
EVALUATION OF DATA COLLECTION RESULTS

APPENDIX A2
EVALUATION OF DATA COLLECTION RESULTS

Based on the spliced girder bridge data collected, an attempt was made to draw some conclusions concerning general trends in usage, types of methods used, frequency of use in different regions of the country, what constitutes standard practice in various regions, and the competitiveness of spliced girder alternatives in particular locations or situations.

Because this evaluation was based largely on information compiled from data collection forms into an organized database, it reflects the opinions of a limited number of individuals – namely the survey participants and the research project team members (approximately 55 individuals). It represents a partial survey because complete information has not been received for all spliced girder bridges. To a certain degree, the evaluation of the survey is biased because it reflects the input of bridge designers and precasters to a greater extent than owners, contractors, or post-tensioning suppliers. Though intended to present the input and viewpoint of all parties involved, this was not practical for a number of reasons.

First of all, it was easier to identify a known project with a particular design firm and then track down the individual engineer responsible for the project. This individual was generally knowledgeable concerning the overall process and the design methods used. Oftentimes they were knowledgeable concerning significant aspects of project construction. Most importantly, as far as the survey was concerned, they were willing to take the time to share their experiences on the project, as is evidenced by their heavy participation in the survey.

Precasters are also heavily represented in the data collection results. As with designers, they could be fairly easily identified with a particular project and were generally willing to participate in the survey. Fortunately, several of them have been involved in a number of projects in a particular region of the country, so they were able to offer additional experience and perspective. The contribution of precasters was extremely important to the data collection process because of their direct experience with the bridge construction.

The process of collecting "written" data from contractors, field personnel and post-tensioning suppliers was largely unsuccessful because it was difficult to identify or locate the individual responsible for a particular project. Because of the nature of their day-to-day work, they were generally

more reluctant to put their experience in writing in the form of a survey, but would usually share their opinions when asked specific questions. Although information collected from contractors and other field personnel was limited, a number of individuals were interviewed concerning their personal experience on a particular project.

1 Findings by Region

Experience with and details of spliced girder bridge methods vary greatly around the country. In evaluating the data results, it has been very difficult to categorize the state of the practice on a national level, but has been possible to identify general trends applicable to individual states or regions of the country. The following assessment of results has therefore been divided into specific regions of the country where the use of extended span girder bridges has been reported.

As was previously stated, the data collected represents a large cross-section of the spliced girder bridge population, but the quantity and quality of data differs for individual bridges. For example, conclusions concerning a particular feature (e.g., type of girders used, span arrangement, etc.) may be based on data from twenty-three bridges in a region overall, but from only seven bridges for which that information was provided on the data collection form. It is important to note that not all of the bridges reported on the data collection forms have actually been built. Some are still under design and some have been abandoned in favor of an alternative structure type. For the latter case, the collected data has been "lined out" in Tables A3-2 through A3-9 to indicate that the structure has not and will not be built as a spliced girder bridge.

Findings are presented for the following regions:
- West
- Midwest
- Northeast
- Northwest
- Far West
- Southeast
- Canada

1.1 West

States in the region with spliced girder bridges: Arizona, Colorado, and Utah.

1.1.1 General Usage

Total bridges: 27, of which 8 are dual bridges built as part of the I-15 reconstruction in Utah and 8 are part of the I-70 project in Colorado. All of the I-15 bridges are simple-span "single point urban interchanges" (SPUIs) built since 1998 with span ranges between 206 ft and 222 ft. The remaining 12 bridges are two-span to five-span continuous, structures built over rivers, railroads and highways with maximum spans of 170 ft to 209 ft. The bridges built for the I-70 project have variability in that some are spliced over piers while others are spliced within the spans and some utilize bearings while others are integral with the substructure. The Colorado DOT is actively engaged in adding to their spliced girder bridge inventory. They currently have half-dozen new bridges under design or construction.

1.1.2 Type of Girders

All bridges built to date utilize constant depth bulb tees ranging from 72 in. to 96 in. deep, but a number of new bridges under design or construction will use trapezoidal tub girders.

1.1.3 Reason for Using Extended Span Girders

Span long distances over rivers, railroads, etc.; safety owing to the elimination of piers in medians; aesthetics; long-term serviceability; low initial cost; economy; schedule.

1.1.4 Challenges

Proper concrete consolidation when casting deep, thin-webbed girders; shipping deep, long and heavy girders; highly skewed structure with varying span lengths.

1.2 Midwest (plus West Virginia and Texas)

States in the region with spliced girder bridges: Illinois, Indiana, Kentucky, Tennessee, Missouri, Nebraska, Minnesota, Ohio, West Virginia, and Texas.

1.2.1 General Usage

18 bridges total, generally two-span to five-span continuous except for two single-span bridges in Minnesota. Two of the Illinois bridges (maximum span of 125 ft) date back to the early 1970s; five bridges (Kentucky, Missouri and Texas) were built in the late 1980s; the remaining bridges were built in the 1990s or are currently under design (Ohio 3, Tennessee 1, West Virginia 1). Maximum span ranges vary from 160 ft to 235 ft.

1.2.2 Type of Girders

I-girders (AASHTO, bulb tees)

1.2.3 Reason for Using Extended Span Girders

Span long distances over rivers, railroads, etc.; minimize/avoid piers in river or R/R ROW; aesthetics; low initial cost compared to cast-in-place box girder; economy; difficulty in transporting large girders; to evaluate cost effectiveness (Illinois, 1974).

1.2.4 Challenges

Providing navigational clearances and matching profile of existing bascule structure; spanning over deep valley and railroad crossing; splicing at midspan where flexural stresses are greatest.

1.2.5 Special Features

Steel girder alternative selected instead of spliced girder alternative with haunched pier segment due to unusually low steel prices, inexperience of local contractors with spliced girders and unequal aesthetic treatment between two alternatives (Illinois, 2002); spliced girder alternative selected over steel plate girders in Indiana, Ohio and Minnesota; 160 ft girders spliced on barge and erected as single unit; lightweight concrete used for drop-in girder; girders made integral with substructure.

1.3 Northeast

States in the region with spliced girder bridges: Connecticut, New Jersey, New York, Massachusetts, Pennsylvania, Rhode Island, and Vermont.

1.3.1 General Usage

23 bridges total with nearly half of them located in New York. Other than the 8 bridges in New York and the one in Pennsylvania, the remaining bridges have all been constructed since 1996. Of these, there is at least one single span (140 ft) structure; the remaining ones are two-span to five-span with maximum span ranges of 80 ft to 236 ft. The eight bridges in New York have been constructed over an extremely long range of years (1952, 1960, 2 in 1965, 1978, 1983, 1995, 2003). They have very little in common with each other besides the fact that they are all post-tensioned. They vary from simple-span to multi-span; they have maximum span ranges from less than 100 ft to 240 ft; they use trapezoidal, box, tee, AASHTO, or bulb-tee girders; and they vary in splice locations from over the piers to within the spans to splices at both locations.

1.3.2 Type of Girders

Older structures – trapezoidal, box, tee, or AASHTO. Newer structures – New England Bulb tees from 39 in. to 71 in. deep or Modified AASHTO Type VI (90 in. deep.)

1.3.3 Reason for Using Extended Span Girders

Span long distances over rivers, railroads, etc.; less live load deflection; transporting smaller precast units; reduce number of substructure elements; reduced schedule; client preference; minimal depth of structure; aesthetics; lower initial cost; long-term serviceability (life-cycle costs); sensitive environment; difficulty in transporting large girders; deck durability enhanced by post-tensioning compression; general site constraints; safety owing to the elimination of piers in medians.

1.3.4 Challenges

Calculating and controlling time-dependent cambers and deflections; stress balancing on unit with unbalanced span arrangement; approved specifications for spliced girder construction unavailable; sample details for spliced girder construction unavailable; design criteria for evaluating stresses in cast-in-place pier diaphragms unavailable; limited girder depths; temporary wind loads on splice support towers; erection in 50-ft deep water near ocean, which required temporary tie-down of girder to pier cap

1.3.5 Special Features

The New Jersey structures were built with integral piers for seismic considerations; a continuous bridge with three 80-ft spans is currently under design in Massachusetts (spliced for deck durability purposes); owner opted for spliced girder superstructure over continuous steel plate girder due to lower life-cycle costs (Massachusetts); Vermont bridge used precast panels on bottom flanges for aesthetics (hide utilities) and precast stay-in-place deck forms.

1.4 Northwest

States in the region with spliced girder bridges: Oregon and Washington.

1.4.1 General Usage

41 bridges total with 21 in Oregon and 20 in Washington. The bridges in Oregon represent two generations: 5 were built from 1960 to 1984, and 16 more were constructed (or are being constructed) from 1993 to present. The first generation includes a 220 ft simple-span bridge built in 1970 and a multi-span structure with 70 ft maximum span length built in 1967. Information is presently

unavailable for the remaining structures. The Washington bridges include one bridge built in 1974; the remaining bridges were built from 1985 to present. Span information is unavailable for most of the structures, but the list includes a 196 ft simple-span and several multi-span bridges with two to four spans and maximum span lengths of 62 ft to 200 ft. The majority of bridges in both states cross rivers but a number of the Washington bridges traverse Interstate 5.

1.4.2 Type of Girders

The overwhelming majority of bridges built prior to 1986, plus one built in 1993, utilize a combination of cast-in-place box girder section over the piers with drop in I-girders between pier tables. Most of the bridges built from 1986 through the early 1990s utilize precast U-beams, as did a bridge built in Washington in 1996. Since the mid-1990s, bridges have been built with either trapezoidal tubs or with bulb tees (modified, Washington Super Girder) ranging in depth from 72 in. to 96 in.

1.4.3 Reason for Using Extended Span Girders

Eliminate falsework over traffic or in river; eliminate piers; minimize structure depth; quality of plant precast products; span long distances over rivers; aesthetics and adherence to corridor theme; long-term serviceability; improved durability; economy (low initial cost); schedule (reduced construction window due to weather conditions); ease of construction (compared to cast-in-place, other structure types); adaptability to future widening.

1.4.4 Challenges

Highly skewed structure at one location; remove existing bridge and construct new bridge within one year time frame while minimizing traffic disruption on interstate

1.4.5 Special Features

As noted previously, several of the bridges utilized cast-in-place box sections over the piers combined with precast I-girder drop-ins; girders for one bridge in Washington were assembled on site and launched across the river to construct 190 ft simple span; one bridge was built on a 1263 ft radius curve, another on a 3000 ft radius curve (both were three-span structures spliced at pier locations only); several bridges utilized cast-in-place pier tables (seismic, familiarity considerations); 40° skew on one bridge; 10-month construction schedule for removal of existing bridge and construction of new spliced girder bridge over I-5 in Washington

1.5 Far West

States in the region with spliced girder bridges: California, Nevada, and Hawaii.

1.5.1 General Usage

19 bridges total, including 3 in Nevada, 2 in Hawaii and the rest in California. The first bridge in Nevada was a 16 ft wide, 164 ft simple-span pedestrian bridge built in 1997. The remaining two bridges in Nevada are two- and four-span structures with precast tub/box girders and integral pier connections. The H-3 bridges in Hawaii consist of long viaduct structures (twenty-seven spans inbound, thirty spans outbound) over environmentally sensitive terrain built in 1997. The maximum span for these structures is 200 ft. The spliced girder bridges in California were built in two distinct phases: 6 bridges from 1966 to 1981; 8 bridges from 1994 to 2004. The earlier bridges consist of single-span and two-span structures and one long viaduct with maximum spans of 92 ft to 188 ft; the later generation bridges consist of 3 single-span bridges in the 144 ft to 148 ft range and two- to five-span bridges with maximum spans varying between 120 ft and 200 ft.

1.5.2 Type of Girders

All bridges built prior to the 1990s, except 1 bridge in California, use precast box or trapezoidal girders. Remaining California bridges (1994 to present) utilize bulb tees ranging in depth from 55 in. to 108 in. All Nevada bridges use trapezoidal boxes. Hawaii bridge uses a 78 in deep modified AASHTO Type VI girder.

1.5.3 Reason for Using Extended Span Girders

Erection over freeway with maintenance of traffic; span long distances over freeway, river, etc.; elimination of piers in river; limited vertical clearance; elimination of falsework; schedule (speed of construction; reduced construction window due to weather conditions); elimination of piers (expensive due to seismic requirements); ability to use integral substructure thereby reducing foundation costs for seismic design (pinned at footing); durability; potential for redecking; owner preference for precast concrete; low initial cost; cast-in-place construction confined to decks only in winter months

1.5.4 Challenges

Varying deck widths, span lengths with bents over railroads, utilities; construction at Elevation 6000 (severe freeze-thaw) and three-stage construction to replace existing structure; review of shop drawings was cumbersome; stability of columns while setting 108 in. deep girder, 200 ft long; designing for different prestress losses between pretensioning and post-tensioning (stressed at different

times); detailing effort at integral pier connection is cumbersome; deflections; staged post-tensioning; post-tensioning contractor placed ducts incorrectly; exterior girder racking problem under first-stage post-tensioning; alignment issues not addressed in the specifications; reduced aggregate size/flowability for concrete in splices; Modifications to pier diaphragm/drop cap connections; detailing complications for girders kinked at splice for 3000 ft radius; misalignment of PT ducts while casting closure due to inadequate support of ducts.

1.5.5 Special Features

First bridge built in California (1966) utilized haunched pier segments with splice at midspan; polyester concrete deck overlay used to preserve deck; California bridge combined cast-in-place pier tables with two unequal drop-in segments with splices located away from traffic lanes; piers at 15° to 41° skew; VECP used to redesign prestressed girders with match-cast splices, two-piece girders and post-tensioning for two projects (1977, 1981); several bridges were built with integral substructure connections; two projects (1975, 1981) used lightweight concrete.

1.6 Southeast

States in the region with spliced girder bridges: Alabama, Florida, Georgia, Louisiana, North Carolina, Mississippi, South Carolina, and Virginia.

1.6.1 General Usage

53 bridges total: 38 in Florida, maximum of 4 in remaining states. With the exception of a handful of overpass structures in Florida and one in Mississippi, nearly all bridges are over the Intra-Coastal Waterway (ICWW) or navigable rivers. All are multi-span structures with the majority being three-span; others are two-span to five-span. Maximum span ranges are between 115 ft and 320 ft. Earliest bridge was 1964 (Florida) with subsequent generations in Florida as follows (1 more in 1960s, 3 in 1970s, 7 in 1980s, 14 in 1990s, and 11 planned for construction in 2000s). 1 bridge built in South Carolina in 1983 and 1 bridge built in Virginia in 1984 (no information available); remaining bridges were (or will be) constructed since 1990 with the majority since 1998.

1.6.2 Type of Girders

All bridges utilize I-girders with the exception of 1 bridge in Florida built with box beams (1985); earlier bridges utilized AASHTO girder sections while bridges constructed since the late 1980s use bulb-tee sections, generally 72 in., or 78 in. deep. Recent bridges have used haunched sections with either variable bottom flange depths or variable web depths.

1.6.3   Reason for Using Extended Span Girders

Sensitive environment; owner's desire to try them; span long distances over rivers, ICCW, etc.; safety owing to the elimination of piers in medians; aesthetics; durability (salt water environment); long-term serviceability; more economical for smaller projects than segmental; more economical and better serviceability than steel structure; delivery schedule; elimination of (expensive) foundations designed for ship impact; preferred building material for economy and durability in Florida; because of success on earlier spliced girder projects; transportation limitations; potential for future widening.

1.6.4   Challenges

Ship impact design considerations; short end spans required due to site conditions; lateral bracing of girders during construction; time-dependent software not (readily) available for earliest bridges (pre-1982); 108 in. deep girder with 23 in. wide bottom flange required complete re-tooling of precaster's stressing frame; early experience with Florida bulb-tees had difficulty fitting post-tensioning ducts in 6½ in. and 7 in. webs and still maintaining cover requirements; also had difficulty with oval ducts, so eventually went to thicker web; longer spans with deeper haunch girders have resulted in extremely heavy lifting weights when variable depth bottom flange was used (now using variable web depth instead); welded wire fabric used for haunch segment vertical steel on one job; contractor damaged one bridge with equipment and one when removing shoring (both satisfactorily repaired); 150 ft. drop-in girder fell due to inadequate support at splice locations prior to splicing (damage several other girders on barge below); blocked post-tensioning ducts and web cracking on early bridge; cracking in deck after second stage post-tensioning (eliminated on future units by modifying stressing sequence); restrictions on contractor's work area due to proximity to town (Virginia)

1.6.5   Special Features

Several bridges designed using post-tensioned 72 in. deep Florida bulb-tees were VECP redesigned for 78 in. deep simple-span girders; several continuous steel superstructures were VECP redesigned to use 72 in. or 78 in. deep post-tensioned spliced girders; touch-shoring used at midspan to cast deck for 1985 bridge; haunched girders with variable depth bottom flanges or variable depth webs used on several bridges; continuous steel plate girder bridge redesigned as three-span spliced girder structure with 320 ft. haunched main span and 15 ft. girder depth at piers (longest known spliced girder span using multi-girder cross-section); use on non-symmetrical cantilevers about centerline of main pier to produce favorable reactions on side-span temporary supports; commonly use Type II cement with 20% fly ash (Florida); lightweight concrete deck being used on lightweight concrete bulb-tees in Virginia

1.7   Canada

Provinces in Canada with spliced girder bridges: Alberta, British Columbia, Manitoba, and Ontario.

1.7.1   General Usage

68 bridges total: 31 in Alberta, 30 in British Columbia, 7 between Manitoba, Ontario, and Saskatchewan. First bridge built in 1970, second and third in 1977; steady usage of spliced girder technology since 1980. Large number of single-span structures ranging from 118 ft. to 164 ft. Remaining bridges are two-span to five-span continuous with maximum span ranges from 87 ft. to 230 ft.

1.7.2   Type of Girders

Mix of I-girders (44 in. to 110 in. deep) and trapezoidal box girders (42 in. to 78 in. deep).

1.7.3   Reason for Using Extended Span Girders

Mountain environment; schedule (short construction season); design commissioned and promoted by precaster (create family of forms); span long distances over rivers, railroads, etc.; speed of construction; staged construction with minimal traffic disruption; minimal impact on surrounding natural environment; minimal shoring requirements over river bed; elimination of falsework; facilitate girder transportation; design efficiency; durability; deck compression from post-tensioning provides less cracking and better durability; reduced number of girders; contractor and precaster VECP; aesthetics; long-term serviceability; lower initial cost.

1.7.4   Challenges

No center pier allowed; must maintain traffic during construction; no shores allowed in river; Maintaining zero tension in precast/cast-in-place joints at diaphragms; casting diaphragms in stages to accommodate girder displacements during deck pour; casting deck in transverse strips to act as ties in stabilizing girder webs in compression; 40° skew; river crossing required temporary shores on berms; use of three oval ducts in web; 3" wide unreinforced girder splices; transportation of 148 ft. long pier girder; construction over busy freeway; shipping of heavy girders; use of high-strength concrete due to high stresses in girder; unusually shallow girders; poor soil conditions under structure; bracing to prevent girder twisting due to highly skewed piers; shoring had to be worked around spring runoff; heavy map cracking in deck due to large restraint of heavy diaphragms at integral piers; erection from deck level due to environmental constraints; cracking in anchor blocks on BC bridge; difficulty in

matching color of architectural edge profile between CIP splice and precast girder; well-distributed pattern cracking in deck near integral diaphragms on BC bridge

1.7.5 Special Features

138 ft. simple span on 312 ft. radius built in 1986; five-span structure with 154 ft. maximum span length built on 662 ft. radius in 1986; other bridges built on 1286 ft. to 1680 ft. radii and supported on single point bearings (no end diaphragms between boxes); several bridges built with integral piers; used 10,000 psi concrete on a couple of different jobs; 300T marine crane required for erection;

2 General Trends and Observations

Although it is hard to categorically describe the state of spliced girder bridge design and construction in the United States, it is possible to note some general trends based on a review of the data collected and presented in Appendix A2*. Where appropriate, observations have also been noted concerning practices in individual states or regions of the country.

(* Note: Project data collected for this study has been summarized for the sake of brevity in Appendix A2. Complete data, where available, is contained in the Data Collection Forms currently retained by the NCHRP project team.)

2.1 Trends

2.1.1 General

1. Although a handful of projects utilized 28-day concrete strengths on the order of 10,000 psi, the vast majority of bridges were not built with high-strength concrete (5500 psi to 8500 psi)

2. The majority of bridges were spliced at in-span locations only (mid-span or quarter point). The number of bridges spliced at both in-span and pier locations or at the piers alone was about the same. These trends did show variations between different regions of the country.

3. Cost data was available for only a limited number of projects and has been presented in Chapter 2. Quantitative analysis of cost data is extremely difficult to perform for a number of reasons, including: (1) It is difficult to ensure what the cost data includes (i.e., cannot compare apples to apples); (2) The contract method (i.e., design/bid/build, design/build, value engineering cost proposal   VECP) has a great impact on the cost of an alternative and makes it difficult to make valid comparisons between different

contract types; (3) The cost of a bridge is greatly influenced by the equipment a contractor already has available; (4) Other project parameters, such as schedule, owner preference, or the availability of other methods of construction, may drive project selection and final costs. Nonetheless, it is possible to use the cost information presented in Chapter 2 and the indexing methods presented in Reference *(1)* to develop legitimate preliminary cost estimates for a variety of span configurations, span lengths and girder types that are applicable to different regions of the country.

4. The majority of spliced girder bridges have been built using design/bid/build contract methods, but bridges constructed under design/build and VECP contracts have become more prevalent in recent years. This trend also varies by region.

5. Schedule is often an important construction consideration for using spliced girder bridges. This applies to both speed of construction and time required to fabricate girders (as compared to structural steel).

6. The number of bridges built using spliced girder methods has increased exponentially over the years, as has the number of states with spliced girder bridges. Presently, two-thirds of the states have already built or have plans to build one.

2.1.2 *Specifications / Design Guidelines*

1. The majority of bridges designed in the past ten to fifteen years utilized the AASHTO *Standard Specifications* for general design considerations, the AASHTO *Segmental Guide Specifications* for design involving post-tensioning, anchor zone and time-dependent effects, and the CEB-FIP Concrete Model Code to determine time-dependent and material properties.

2. A large number of respondent referenced state highway department design specifications in addition to the specifications noted in Item 1.

3. A fair number of respondents referenced PCI's "State-of-the-Art of Precast/Prestressed Concrete Spliced-Girder Bridges" as a design guide.

4. Several respondents referenced PCI New England's "New England Bulb Tee (NEBT) Post-Tensioned Design Guidelines" and the Precast/Prestressed Concrete Manufacturer's Association of California (PCMAC) technical information as design guides.

2.1.3 Construction Techniques

Most of the earlier spliced girder bridges (1950s through late 1980s) utilized temporary towers to support girder segments prior to splicing and making continuous. The majority of bridges built since 1990 utilize strongbacks to support drop-in girders from the previously erected pier segment when possible. Temporary towers are now mainly used where necessary to stabilize pier segments when they cannot be stabilized by the previously erected structure. At least a few bridges have used dapped end pier segments to support the drop-in segment during erection.

2.1.4 Design Details

Splices are generally located based on shipping/handling restrictions or structural considerations, but sometimes also on site geometric constraints, such as obstructions below the bridge or away for traffic lanes underneath the bridge.

2.1.5 Special Considerations

Items identified that require special consideration during design and construction include:

1. Wind loads on temporary towers must be given special attention

2. Girder stability during construction is very important

3. Proper grouting of post-tensioning ducts is extremely important

4. Consolidation of concrete during casting of deep bulb-tee sections can present a challenge

5. Standardization of girder casting details and erection details would facilitate construction greatly

2.2 Regional/State Trends

2.2.1 Florida / Southeast

The general state of post-tensioned spliced girder practice in the Southeast can be characterized by its development in Florida. Florida leads the nation with 38 spliced girder bridges, most of which are three- to five-span water crossings over the Intracoastal Waterway or navigable rivers. The state of practice in Florida is unique in that there has been a distinct and consistent evolution of design practices and details over the years. This evolution has been documented in a number of papers listed in the bibliography.

Other trends in the Southeast include:

1. Concrete spliced girder alternatives were often selected over continuous steel superstructure alternatives

2. Concrete spliced girder alternatives were often selected over segmental concrete alternatives, usually on smaller projects where segmental construction is less competitive

3. Concrete spliced girder superstructures (72-in. Florida bulb tee) were often redesigned as simple-span prestressed girder bridges using a deeper girder cross-section (78-in. Florida modified bulb tee) when it was permissible to raise the roadway grade

4. Florida is the only location where the justification given for using spliced girder bridges at a number of locations was because of past experience and success in using this method of construction

2.2.2 New York

New York is on the other end of the spectrum from Florida with regard to consistency in application and evolution of usage and details. As noted previously, the eight bridges in New York have been constructed over an extremely wide range of years (1952, 1960, 2 in 1965, 1978, 1983, 1995, 2003). They have very little in common with each other besides the fact that they are all post-tensioned. They vary from simple span to multi-span; they have maximum span ranges from less than 100 ft to 240 ft; they use trapezoidal, box, tee, AASHTO, or bulb-tee girders; and they vary in splice locations from over the piers to within the spans to splices at both locations.

2.2.3 California

The trend of spliced girder construction in California can be characterized with respect to two separate eras: the handful of bridges constructed in the 1960s through early 1980s utilized precast box girder sections while the remaining 9 or 10 bridge constructed since 1990 and currently under construction use bulb-tee sections. The current use of post-tensioned spliced girders is indicative of the state of bridge construction in California, where cast-in-place, post-tensioned box girders are the superstructure of choice. Spliced girder bridges are currently being used in situations where falsework is not permissible (over busy freeways, in river beds, etc.) or where the piers are too tall to make cast-in-place construction economically feasible. The use of spliced girder construction has been promoted in California through the efforts of the Precast/Prestressed Concrete Manufacturers Association of

California (PCMAC), and the technology has been adapted for use with integral substructure connections to better resist seismic forces.

2.2.4 Oregon

Oregon is unique in that a number of spliced girder bridges there were built using a hybrid cross-section involving prestressed drop-in I-girders spliced with a cast-in-place box section at the piers. There were also a few of these structures reported in Canada. It may offer potential for use in California by combining the contractor preference for cast-in-place construction at the piers with the speed and convenience of precast construction.

2.2.5 British Columbia, Canada

British Columbia is unique in that it is one of the few places where match-cast joints have been used between precast girder members. This procedure was used at a couple of different bridge locations.

3 References

1. "Civil Works Construction Cost Index System (CWCCIS)," U.S. Army Corps of Engineers, Washington DC, March 31, 2000 with tables revised March 31, 2003.

APPENDIX A3

Listing of Spliced Girder Bridge Projects

Table A3-1 List of Bridges - Summary

Region	State/ Province	Total Number of Bridges	Constructed by Decades						Contract Type		
			1950's	1960's	1970's	1980's	1990's	2000's	Design/ Bid/Build	Design/ Build	VECP
West		**27**	**0**	**0**	**0**	**1**	**19**	**6**	**4**	**8**	**0**
	AZ	1						1	1		
	CO	18				1	11	5	3		
	UT	8					8			8	
Midwest		**20**	**0**	**0**	**2**	**5**	**8**	**5**	**6**	**0**	**0**
	IL	2			2			0 (1)	1		
	IN	4					4				
	KY	3				2 (1)	1				
	TN	1						1	1		
	MO	2				2					
	NE	1					1				
	OH	3						3	3		
	MN	1					1		1		
	TX	1				1					
	WV	2					1	1			
Northeast		**23**	**1**	**3**	**3**	**1**	**6**	**9**	**4**	**4**	**0**
	CT	0						0 (1)			
	NJ	4						4	2	2	
	NY	12	1	3	2	1	4	1	1		
	MA	4					1	3	1		
	PA	1			1		0 (1)			1	
	RI	1						1			
	VT	1					1			1	
Northwest		**41**	**0**	**2**	**3**	**7**	**23**	**6**	**1**	**0**	**0**
	OR	21		2	2	1	13	3			
	WA	20			1	6	10	3	1		
Far West		**19**	**0**	**1**	**6**	**0**	**5**	**8**	**13**	**0**	**1**
	CA	14		1	5		2	6	10		
	NV	3					1	2	3		
	HI	3			1		2				1

Table A3-1 (cont.)   List of Bridges - Summary

Reg ion	State/ Province	Total Number of Bridges	Constructed by Decades						Contract Type		
			1950's	1960's	1970's	1980's	1990's	2000's	Design/ Bid/Build	Design/ Build	VECP
Southeast		**57**	**0**	**2**	**3**	**9**	**19**	**18**	**18**	**5**	**5**
	AL	3					3			2	1
	FL	36		2	3	7	14 (1)	10	14	2	3
	GA	1					1				
	LA	1						1 (2)			
	NC	2						2			
	MS	3					1	2	1	1	1
	SC	1				1	0 (1)				
	VA	4				1		3	3		
Canada		**68**	**0**	**0**	**3**	**31**	**24**	**9**	**0**	**0**	**0**
	AB	19			1	16	5	9			
	BC	30				13	16				
	MB	2				1	1				
	ON	5			2	1	1				
	SK	1					1				
Other Countries		**3**	**0**	**0**	**0**	**1**	**2**	**0**	**0**	**0**	**0**
	New Zealand	1					1				
	Taiwan	1				1					
	Bahamas	1					1				
Total		**252**	**1**	**8**	**20**	**55**	**105**	**61**	**46**	**17**	**6**

TABLE A3-2   LIST OF BRIDGES – WEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Tatum Blvd Overpass (SR101L EB & WB over Tatum Blvd)	Phoenix	AZ	USA	2001	72.2 – 170.6	72.2' + 170.6' + 72.2' with integral piers.	X			AASHTO Modified Type VI	X		
Main Street Viaduct over Arkansas R. and railroad yard	Pueblo	CO	USA	1995	152 – 174	152'-174'-164.5'-155.5'-88'	X			Modified Colorado 72" Bulb-T	X		
I-70 Viaduct, Washington St. to Brighton Blvd, Unit 1	Denver	CO	USA	2000	105 – 188.3	113'-188.3'-105' (Unit 1)	X			Modified Colorado 72" Bulb-T	X		
I-70 Viaduct	Denver	CO	USA	2002	138 - 209	138-209-138, (plus other units)				Colorado BT72 modified	X		
I-70 Replacement Phase I - Unit 2	Denver	CO	USA	1998	130.4 – 173.8	130.4-173.8-130.3				BT-72/96			
I-70 Replacement Phase I - Unit 3	Denver	CO	USA	1998	129.6 – 150.9	144.4-5 @ 150.9-129.6				N/A			
Colo SH 34/ I-36	Ft. Morgan	CO	USA	1995	174.5	2 @ 174.5				BT-72/96			
Colo SH 44/ I-76	Adams County	CO	USA	1988	164 - 183	164-183-164				BT-72/96			
Buckley Road over I-76	Brighton	CO	USA	1996	171.5	2 @ 171.5				BT-72/96			
I270/ I76 Flyover		CO	USA		230	230				84" Tub			
I25/ Nevada Tejon		CO	USA	2001	125	125				60" Tub			
I25/ Woodman		CO	USA	2000	113	113				57" Tub			
SH52/ I25		CO	USA	1999	65 - 190	65-190-65				Box 74"			
I225/ Parker		CO	USA	2000	258	258				Box 103"			

TABLE A3-2 (cont.) LIST OF BRIDGES – WEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
I25 38th & Fox		CO	USA	1995	225	225				Box 103"			
Harmony/ I25		CO	USA	1998	65-190	65-190-65				BT 72 to 74			
Federal/US 36		CO	USA	1999	171	171-171				BT 84			
104th SH44/ I76		CO	USA	1994	170	170-170				BT 72 to 96			
US34 Ramp/ I76		CO	USA	1994	173	173-173				BT 72 to 96			
Buckley/I76		CO	USA	1992	164-183	164-183-164				BT 72 to 96			
SLC bridge 20 & 22	Salt Lake City	UT	USA	1998	216	216	X			BT - 7.875' deep		X	
SLC bridge 30 & 32	Salt Lake City	UT	USA	1998	216	216	X			BT - 7.875' deep		X	
SLC bridge 46 & 48	Salt Lake City	UT	USA	1998	207	207	X			BT - 7.875' deep		X	
SLC bridge 54 & 56	Salt Lake City	UT	USA	1998	222	222	X			BT - 7.875' deep		X	
SLC bridge 8 & 10	Salt Lake City	UT	USA	1998	215	215 ft simple Span	X			BT - 7.875' deep		X	
SLC bridge 64 & 66	Salt Lake City	UT	USA	1998	215	215 ft simple Span	X			BT - 7.875' deep		X	
SLC bridge 74 & 76	Salt Lake City	UT	USA	1998	215	215 ft simple Span	X			BT - 7.875' deep		X	
SLC bridge 130 & 132	Salt Lake City	UT	USA	1998	215	215 ft simple Span	X			BT - 7.875' deep		X	

TABLE A3-3 LIST OF BRIDGES – MIDWEST

| Name of Bridge | Location | | | Year Complete | Span Range | Span Arrangement | Splice Location | | | Girder Type | Type of Contract | | |
	Place	State	Country				In-Span	Pier	Both		Design/Bid/Build	Design/Build	VECP
FA Rt. 72 Sec. 74-68-HB Sta. 1256+11.76	Piatt County	IL	USA	1974	106.5	Multiple	X			48" I-Beam	X		
FA Rt. 72	Clark County	IL	USA	1974	106.5'-125'	Multiple	X			48" I-Beam	X		
~~South Damen Ave./F.A.U. Rt. 2850~~	~~Chicago~~	~~IL~~	~~USA~~	~~2002~~	~~185-235.6~~	~~Multiple~~	X			~~Modified PCI BT-72~~	X		
I-469 / I-69	Allen County	IN	USA	1994	259.54	126.5, 131.5				54" Mod.AASHTO Type III			
Dupont Road / I-69	Allen County	IN	USA	1995	231	2 @ 115.5				48" Mod.AASHTO Type III			
US 421 / Wabash River	Carroll County	IN	USA	1995	802	2 @ 133.83				73" Mod.AASHTO Type IV			
US 231 over Wabash River	Indiana	IN	USA	1991	210.6.5	12 @ 175.5				Bulb Tee	X		
~~Salt River Bridge~~	~~Fort Knox~~	~~KY~~	~~USA~~	~~1987~~	~~85-160~~	~~Multiple~~		X		~~I girder~~			
Twelve Mile Creek Bridge	Campell County	KY	USA	1987						I girder			
Shelby Creek Bridge	Pike County	KY	USA	1989	162.25218.5	Multiple				I girder			
KY 38/ Upstream Portal of Clover Fork of Cumberland River	Harlan County	KY	USA	1991	311.5	2 @ 155				72" Mod.AASHTO Type IV			

Notes (Typical, all sheets):
1) All dimensions in feet unless shown otherwise
2) ~~Name of Bridge~~ indicates bridge that was not built or was built using other alternative type.

TABLE A3-3 (cont.)   LIST OF BRIDGES – MIDWEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Missouri River Bridge #1		MO	USA	1989						I girder			
Missouri River Bridge #2		MO	USA	1989						I girder			
North Platte Bridge over I-80	North Platte and I-80	NE	USA	1997	286	2 @ 143				NU1350			
SUM-271-0802 Lt. Over Cuyahoga River Valley	Peninsula/ Summit Cos.	OH	USA	2008	182-232	185-232-232-185	X			AASHTO Type IV Mod. (VI Mod.?)	X		
SUM-271-0802 Rt. Over Cuyahoga River Valley	Peninsula/ Summit Cos.	OH	USA	2008	182-232	185-232-232-185	X			AASHTO Type IV Mod. (VI Mod.?)	X		
I-80 over Cuyahoga River Valley	Summit County	OH	USA	2004	150-200	Multiple	X			I-Girders 102" deep	X		
Holden Bridge	Holden Bridge	WV	USA	1997	1240	222				Mod.AASHTO Type IV			
SR 75 over Holston River	Sullivan County	TN	USA	2006									
Tug Fork Bridge	Williamson	WV	USA	Under Design	1100								
Loy Lake Road Underpass (Sherman)	Grayson County	TX	USA	1985	150-160					72" I-Girder			
Br. No. 70037 & 70038, TH 101 EB/WB over CSAH 16	Shakopee	MN	USA	1995	170	170 ft single span	X			Modified Mn/DOT 81" deep I-girder	X		

A3-6

TABLE A3-4 LIST OF BRIDGES - NORTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
~~Route 68 over Interstate 91~~	~~Wallingford~~	~~CT~~	~~USA~~	~~2002~~	~~167-167~~	~~Multiple~~	X			~~New England Bulb Tee~~			
Route 9 (Edison) Bridge -- South Approach	Middlesex County	NJ	USA	2002		Multiple		X		2m Bulb Tee		X	
Route 9 (Edison) Bridge -- North Approach	Middlesex County	NJ	USA	2002		Multiple	X			2m Bulb Tee		X	
Route 35 over Shark River	Monmouth County	NJ	USA	In final design at this time	566 m	51.82 m				Modified AASHTO Type IV	X		
Ocean City-Longport Bridge Replacement	Ocean City	NJ	USA	2002	184-22	184-222-184	X			Mod. AASHTO Type VI (90" deep)	X		
Rtes. 29 and 40 over the Battenkill	Washington County	NY	USA	1978	178	Single	X			AASHTO Mod.Type VI			
Rte. 203 over Kinderhook Creek	Valatie, Columbia County	NY	USA	1983	175-175	2 Simple 175' spans	X			AASHTO Modified Type IV			
Nepperhan Ave. Pedestrian Bridge	Yonkers	NY	USA	1995	140	Single	X			Trapezoidal U-shaped precast			
Rte. 284 Bridge	Orange County	NY	USA	1965	120	Multiple	X			T-Girders			
Interstate I-81 Oneida Lake Bridge	Brewerton	NY	USA	1960	3 Span 240' center span	Multiple	X						

Notes (Typical, all sheets):
1) All dimensions in feet unless shown otherwise
2) ~~Name of Bridge~~ indicates bridge that was not built or was built using other alternative type.

TABLE A3-4 (cont.)   LIST OF BRIDGES - NORTHEAST

| Name of Bridge | Location | | | Year Complete | Span Range | Span Arrangement | Splice Location | | | Girder Type | Type of Contract | | |
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
State University Albany Health and PE Building	Albany	NY	USA	1965	120	Single	X						
Cross Westchester Expy (I-287) over Saw Mill River Rd (Rt 9A)	Green-burgh	NY	USA	2003	137.78	Single	X			NEBT 1800 (70.87")	X		
Bloomingburg Road Bridge	Orange County	NY	USA	1998	238'-5"	2 @ 119				Channnel Section			
Carpenter Road over Metro North Railroad	Dutchess County	NY	USA	1997	86'	86				Channnel Section			
Nepperhan Avenue Pedestrian Bridge	West-chester County	NY	USA	1994	140	140				(2) Adjacent Trapezoidal Box Section			
Oneonta, NY	Oneonta	NY	USA	1972	130	130				AASHTO Type V (63")			
Muller's Bridge		NY	USA	1952									
Providence Highway over Harris St. and High St	Dedham	MA	USA	200?	74-123	Multiple	X			NEBT 1600			
MHD Bridge No. W-04-025, Winter Street over I-95	Waltham	MA	USA	2004	80-80	Multiple		X		New England Bulb Tee 1000 (39.4" deep)	X		
Evelyn F. Moakley Bridge (Northern Ave. Bridge)	South Boston	MA	USA	1996	67-236	Multiple	X			Box Beams, AASHTO Type IV			
Evelyn F. Moakley Bridge (Northern Ave. Bridge)	South Boston	MA	USA	2000	67-236	Multiple	X			Box Beams, AASHTO Type IV			

TABLE A3-4 (cont.)   LIST OF BRIDGES - NORTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Middleport Bridge	Schuylkill County	PA	USA	1975		Single	X						
Walnut Lane	Phila-delphia,	PA	USA	1990 +/-		Multiple	X					X	
Cranston Viaduct (twin bridge), Rt 10 over Amtrak ROW	Cranston	RI	USA	2002	163-175	Multiple	X			Mod.NEBT (6'-10 5/8" deep)			
Husky Entrance Road Overpass	Milton	VT	USA	1998	140	Single	X			NEBT 1800 / 1800mm deep		X	

TABLE A3-5   LIST OF BRIDGES – NORTHWEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Royal Avenue (Beltline)	Eugene	OR	USA	2001	57-57	Multiple	X			ODOT BT 84 w/ 7.5" web			
Jefferson Bridge over Breitenbush River	Near Detroit	OR	USA	2001	64.33-127.66	Multiple	X			ODOT BT 84 w/ 7.5" web			
Carmine-Smith Bridge	McKenzie River	OR	USA	1960						I girder			
Stewart Park Bridge	Douglas County	OR	USA	1967	60-70	Multiple			X	Hybrid of CIP box girders and I girders			
Umpqua River Bridge	Sutherlin	OR	USA	1970	220	Single	X			Hybrid of CIP box girders and I girders			
Oak Avenue Bridge	Douglas County	OR	USA	1971						Hybrid of CIP box girders and I girders			
McKenzie River Bridge	Armitage	OR	USA	1984						Hybrid of CIP box girders and I girders			
Big Nestucca River Bridge	Tillamook, County	OR	USA	1999	67-69.5	Multiple		X		ODOT BI 2740 (108")			
S. Fork Coquille Rver	Coss County	OR	USA	1997	74.75-124.7	Multiple			X	BI84			
Dillard Bridge	Douglas County	OR	USA	1998	96.5-115.8	Multiple	X			BT 72			
NW 185th Ave Interchange	Washing-ton County	OR	USA	1993	130-135	Multiple	X			BI75			
Little Elk Creek Bridge No. 7	Eddyville	OR	USA	1998	58.4-65.6	Multiple	X			BI 96			

TABLE A3-5 (cont.)   LIST OF BRIDGES – NORTHWEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Royal Avenue (Beltline)	Eugene	OR	USA	2001	118	17.45m-17.45m	X			Trapezoidal Tub			
Eddyville-Cline Hill Little Elk Cr. BR # 2	Corvallis	OR	USA	1997	180					96" BI			
Eddyville-Cline Hill Little Elk Cr. BR # 7	Corvallis	OR	USA	1997	184					96" BI			
Eddyville-Cline Hill Little Elk Cr. BR # 8	Corvallis	OR	USA	1997	176					96" BI			
Eddyville-Cline Hill Little Elk Cr. BR # 9	Corvallis	OR	USA	1997	164					96" BI			
South Umpqua River Dillard Bridge	Roseburg	OR	USA	1997	164-200					72" Bulb Tee			
Biehn Street Bridge	Klamath Falls	OR	USA	1997	192					108" BI			
Chenoweth St. Bridge	The Dalles	OR	USA	1997	192					Varying Trap. Tub			
Washburn Way Bridge	Klamath Falls	OR	USA	1997	182					Varying Trap. Tub			
Harvard Rd. Bridge	Spokane	WA	USA	1997	175	Multiple		X		Modified W74G			
Green River Bridge	Kent	W	USA	1999	131.75-166	Multiple		X		New 82.7" WA Supergirder			
TJ Meenach Bridge	Spokane	WA	USA	1995	125-183	Multiple		X		WSDOT W74G w/ wider web			
Rock Cut Bridge	Orient	WA	USA	1996	160-200	Single	X						

TABLE A3-5 (cont.)   LIST OF BRIDGES – NORTHWEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/Bid/Build	Design/Build	VECP
Nine-mile Bridge	Spokane County	WA	USA	1975	190.5					1			
128 Street Bridge	Snohomish	WA	USA	1985	172-172	Multiple	X			Hybrid of CIP box girders and I girders			
Henderson Blvd. Undercrossing	Olympia	WA	USA	1985						U			
College Street Undercrossing	Lacey	W	USA	1986						U			
Sleater Kinney Interchange	Olympia	WA	USA	1986						U			
Lilly Road Undercrossing	Olympia	WA	USA	1987						U			
Pullen Creek Bridge	WA	WA	USA	1988						I girder			
Airdustrial Road Interchange	Olympia	WA	USA	1990						U			
Trosper Road Underpass	Thurston County	WA	USA	1991						Precast Box girder			
Sunset Drive Interchange	Bellingham	W	USA	1996						U			
MT. Hood Meadows Bridge	MT. Hood National Forest	WA	USA	1999	61.7'-61.7'	Multiple	X			Trapezoidal Tub			
Saxon Road Bridge 127A	Whatcom County	W	USA	1998	276	37-202-37				89" Decked Bulb Tee			
SR 20 Twisp River Bridge Replacement	Twisp, Okanogan County,	WA	USA	2000	196		X			W95PTMG Girder			

TABLE A3-5 (cont.)   LIST OF BRIDGES – NORTHWEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
I-90 Sunset I/C	Issaquah	WA	USA	2003		Multiple			X	Tub	X		
S 200th St. over Green River	Kent	WA	USA	1998	125'-170'	125'-170'	X			WSDOT 74G girder	X		
S. 38th St./I-5 Interchange	Tacoma	WA	USA	2001	161'-164'	161'-164'				Trapezoidal Tub			

TABLE A3-6   LIST OF BRIDGES – FAR WEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Fibreboard Undercrossing	Truckee	CA	USA	2003		Single	X			Bulb-Tee	X		
Mococo OH / Rte 680, crosses Mococo RD and RR	Concord	CA	USA	2004	121 -187	Multiple			X	Modified CA Bulb Tee 9ft (9.8" web)			
Trout Creek Undercrossing / I-80	Truckee	CA	USA	2003	145	Single	X			CA 2150 mm Bulb-Tee			
Firestone Boulevard Bridge over Los Angeles River	Southgate	CA	USA	2000	104.5 - 132	Multiple			X	AASHTO 72"	X		
Mojave Dr. Over-crossing, over I-15	Victorville	CA	USA	2002	119.75 – 197.75	Multiple		X		Caltrans 1.4 m (55" +/-) BT Caltrans BT, 78" deep			
RT 57 HOV Widening: Brea Canyon Rd Undercrossing	Los Angeles County	CA	USA	1994	160 - 200	Multiple	X			Caltrans BT, 78" deep	X		
El Camino Avenue Overcrossing	West Sacra-mento	CA	USA	1977		Multiple			X	5.75 ft deep trapazoidal box	X		
E Street Overcrossing	Chula Vista	CA	USA	1970	123 - 123				X	Trapazoidal Box	X		
H Street Overcrossing	Chula Vista	CA	USA	1973	92.2 – 92.2	Multiple			X	3.75 ft. deep trapazoidal box	X		
Harbor Drive Overhead	Ocean-side	CA	USA	1975		Single			X	3.75 ft Trapazoidal Box	X		

TABLE A3-6 (cont.)   LIST OF BRIDGES – FAR WEST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/Bid/Build	Design/Build	VECP
Sacramento River Bridge (Anderson, CA)	Anderson	CA	USA	2003	154 - 154	Multiple			X	CALTRANS Modified BT1770 (70")	X		
Harbor Boulevard Overcrossing	West Sacra-mento	CA	USA	1970	139 - 139	Multiple			X	6.0 ft trapazoidal box	X		
Napa River Bridge	Vallejo	CA	USA	1966	120 - 188	Multiple			X		X		
Via de La Amistad Undercrossing	San Diego	CA	USA	1999	148.17	Single	X						
Lower Halawa Valley Viaduct, Interstate H-3 Inbound , Unit 1 Phase 1B	Honolulu	HI	USA	1997	150 - 200	Multiple	X			Modified Keehi Type VI - 78" deep			X
Lower Halawa Valley Viaduct, Interstate H-3 Outbound, Unit 1 Phase 1B	Honolulu	HI	USA	1997	150 - 200	Multiple	X			Modified Keehi Type VI - 78" deep			X
Kaimakani Street Separation (Interstate Rte H-1)	Honolulu	HI	USA	1970		Multiple				AASHTO type IV			
Desert Inn Road Pedestrian Bridge P-1450	Las Vegas	NV	USA	1997	164	Single	X			Trapazoidal Box	X		
H-933 Oakey Blvd. Grade Separation Widening	Las Vegas	NV	USA	2002	55.42 – 75.56	55.42 - 75.56 - 75.56 - 55.42 with integral piers	X			5' x 3.38' Box Girders	X		
H-1211 Washington Ave. Grade Separation	Las Vegas	NV	USA	2000	120 - 130	130' - 120' with Integral Piers	X			Precast U-Girders	X		

TABLE A3-7   LIST OF BRIDGES - SOUTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Bridge Over Alabama River	Mont-gomery	AL	USA	1998	175-215	Multiple	X			FBT-72 / 7" web		X	
Bridge Over Black Warrior River	Tuscaloosa	AL	USA	1998	208-260	Multiple	X			FBT-72 / 8" web		X	
Bridge on U.S. 11 over Black Warrior River	Tuscaloosa	AL	USA	1999	208-262.5	Multiple	X			FBT-72 / 8" web			X
Wonderwood Intracoastal Crossing	Jackson-ville	FL	USA	2003	195-250	Multiple	X			FBT -78	X		
S.R. 10 (U.S. 90A) over the Escambia River	Pensacola	FL	USA	2003	130.75-210.47	Multiple	X			FBT -78	X		
SR 312 Bridge Over Intracoastal Waterway	St. Augustin	FL	USA	1999	192.5-250	Multiple		X		78" Mod. AASHTO VI			
US-17/92 Bridge Over St. Johns River at Lake Monroe	Sanford / Lake Monroe	FL	USA	1994	160-200	Multiple	X			FBT72			
Palm Valley Bridge	Palm Valley	FL	USA	2002	119-290	Multiple	X			Mod. FL BT 78; 81" deep	X		
SR 528 over the Indian River	Brevard County	FL	USA	2004	161-210	Multiple	X				X		
Midpoint Bridge over Caloosahatchee River	Fort Myers/Cape Coral	FL	USA	1998	160-200	Multiple	X			78" FBT w/108" deep haunch	X		
S.R. 44 Over I-4	Volusia County	FL	USA	2003	182-182	Multiple	X			Mod. FBT 78	X		
SR 574A over I-75	Tampa	FL	USA	1982	143-143	Multiple			X	AASHTO Type VI	X		

TABLE A3-7 (cont.)   LIST OF BRIDGES - SOUTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Harbour Island People Mover over Crosstown Expressway	Tampa	FL	USA	1985	80-140	Multiple		X		48" wide x 54" deep box beam		X	
Florida's Turnpike over I-595 and North New River	Ft. Lauderdale	FL	USA	1989						AASHTO Type V			X
Highland View Bridge / SR-71 Over ICWW	Port St. Joe	FL	USA	1993	196-250	Multiple		X		FBT-72	X		
Gandy Bridge WB / SR-600 Over Tampa Bay	Tampa / St. Petersburg	FL	USA	1998	144-235	Multiple		X		Mod. FBT 78	X		
~~Anna Maria Island Bridge / SR-64 Over ICWW~~	~~Bradenton/ Anna Maria Island~~	~~FL~~	~~USA~~	~~N/A~~	~~140-140~~	~~Multiple~~		~~X~~		~~FBT-72~~			
Bayou Chico Bridge (Pace Blvd) / SR-292 Over ICWW	Pensacola	FL	USA	1999	200-250	Multiple	X			Mod. FBT 78			X
Moore Haven Bridge / US-27 Over Caloosahatchee	Moore Haven	FL	USA	2000	215-320	Multiple	X			Mod. FBT 78			X
Fuller Warren Bridge / I-95 Over St. Johns River	Jackson-ville	FL	USA	2002	200-260	Multiple	X			Mod. FBT 78			
St. George Island Bridge / SR-300 over Apalachicola Bay	Eastpoint	FL	USA	2004	208-258	Multiple	X			Mod. FBT 78	X		

Notes (Typical, all sheets):
1) All dimensions in feet unless shown otherwise
2) ~~Name of Bridge~~ indicates bridge that was not built or was built using other alternative type.

TABLE A3-7 (cont.)   LIST OF BRIDGES - SOUTHEAST

| Name of Bridge | Location | | | Year Complete | Span Range | Span Arrangement | Splice Location | | | Girder Type | Type of Contract | | |
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
SR-210 Over ICWW	Jackson-ville	FL	USA	2001			X			Mod. FBT 78			
SR-10 / Atlantic Blvd. Bridge Over ICWW	Jackson-ville	FL	USA	2002			X			Mod. FBT 78			
US-331 Over Choctawhatchee Bay	Destin/ Panama City	FL	USA	1990	142-200	Multiple			X	FBT-72			
Edison Bridge NB / US-41 Over Caloosahatchee River	Ft. Myers	FL	USA	1992	143-143	Multiple		X		FBT-72	X		
Edison Bridge SB / US-41 Over Caloosahatchee River	Ft. Myers	FL	USA	1994	143-143	Multiple		X		FBT-72	X		
Merrill Barber Bridge NB/SB / SR-60 Over ICWW	Vero Beach	FL	USA	1995	142-142	Multiple		X		FBT-72			
Vilano Beach Bridge / SR-A1A Over ICWW	St. Augustine	FL	USA	1996	137-148	Multiple		X		FBT-72			
Flagler Beach Bridge EB/WB / SR-100 Over ICWW	Flagler Beach	FL	USA	1997	161-230	Multiple	X			FBT-72			
MacArthur Causeway Bridge / US-41 Over ICWW	Miami	FL	USA	1997	145-145	Multiple		X		FBT-72	X		
SR-510 Over Sebastian Inlet	Sebastian	FL	USA	1965	100-180	Multiple	X			AASHTO Type IV Modified			

TABLE A3-7 (cont.) LIST OF BRIDGES - SOUTHEAST

| Name of Bridge | Location | | | Year Complete | Span Range | Span Arrangement | Splice Location | | | Girder Type | Type of Contract | | |
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Duplicate of Sebastian Inlet Bridge	Pensacola	FL	USA	1968	100-180	Multiple	X			AASHTO Type IV Modified			
Duplicate of Sebastian Inlet Bridge	Ft. Lauderdale	FL	USA	1970	100-180	Multiple	X			AASHTO Type IV Modified			
Chipola Nursery Road Over I-10	Marianna	FL	USA	1977	127-127	Multiple			X	AASHTO Type IV Modified			
Buffalo Ave. Over I-75	Tampa	FL	USA	1979	127-127	Multiple			X	AASHTO Type IV Modified			
Martin Luther King Blvd. Over I-75	Tampa	FL	USA	1980	115-115	Multiple	X			AASHTO Type IV Modified			
Howard Frankland Bridge / I-275 Over Tampa Bay	Tampa to St. Petersburg	FL	USA	1990	143-143	Multiple		X		FBT-72			
3 Bridges on I-75 Over Miscellaneous Roadways	Between Tampa to Naples	FL	USA	1980-83		Multiple	X			AASHTO Type IV Modified			
Eau Gallie Bridge / SR-518 Over Indian River	Melbourne	FL	USA	1988	145-145	Multiple		X		FBT-72			
Apalachicola River Bridge	Franklin County	FL	USA	1986						I girder (var.)			
Talmadge Bridge Approaches	Savannah	GA	USA	1990	180	Multiple	X			Mod. Bulb Tee	X		
~~US 165 across the Ouachita River, Columbia~~	~~Caldwell Parish~~	~~LA~~	~~USA~~	~~2000~~	~~190.3-249.3~~	~~Multiple~~	X			~~FBT (?)~~	X		

Notes (Typical, all sheets):
1) All dimensions in feet unless shown otherwise
2) ~~Name of Bridge~~ indicates bridge that was not built or was built using other alternative type.

TABLE A3-7 (cont.)   LIST OF BRIDGES - SOUTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
~~US 84 across the Black River~~	~~Jonesville~~	~~LA~~	~~USA~~	~~2000~~	~~190.3-249.3~~	~~Multiple~~	X			~~FBT (?)~~	X		
US90 @ Wrigley's Pass	New Orleans	LA	USA	2006									
Manteo Bypass	US 64 over Croatan Sound Dare County	NC	USA	2001	27,573.16'	137.76-252.62-137.76				78" Bulb Tee			
Oak Island Bridge	Brunswick County	NC	USA	Under Design	1793	154-227-154				Bulb Tee			
U.S. Highway 90 High Level Bridge		MS	USA	2002	196-250	Multiple	X			FBT-72 / 8" web	X		
S.R. 605 Across Biloxi River		MS	USA	2002	147.56-196.85	Multiple	X			FBT-78			X
Grand Casino Parking Garage Bridge		MS	USA	1994	68'-160'	Multiple	X			PCI/AASHTO BT-72		X	
Congaree River Bridge		SC	USA	1983						I girder (var.)			
~~Dawhoo River Bridge~~		~~SC~~	~~USA~~	~~1991~~						~~I girder (var.)~~			
Great Neck Road Bridge		VA	USA	1984						I girder (var.)			
Route 123 over Occoquan River		VA	USA	2006	180-240	180'-240'-180'	X			Mod.VA 77" Bulb-Tee		X	
Bridge on Route 33 over Pamunkey River		VA	USA	2007	200-240	2 Units -200'-240'-240'-200	X			PCEF 89 95.5 60		X	

Notes (Typical, all sheets):
1)   All dimensions in feet unless shown otherwise
2)   ~~Name of Bridge~~ indicates bridge that was not built or was built using other alternative type.

TABLE A3-7 (cont.)   LIST OF BRIDGES - SOUTHEAST

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Bridge on Route 33 over Mattaponi River		VA	USA	2006	200-240	2 Units -200'-240'-240'-200'	X			PCEF 89 95.5 60		X	

TABLE A3-8   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Whitehorn Overpass	Banff National Park	AB	CANADA	1985	153.5	Single	X			4'-5" Bath-Tub			
Pipestone Creek	Banff National Park	AB	CANADA	1985	128.9	Single	X			3'-7" Bath-Tub			
MacLeod Trail / Anderson Road Interchange	Calgary	AB	CANADA	2000	114-126	Multiple		X		"U" Girder			X
Bow River Bridge (on Highway 2 over Bow River)	Calgary	AB	CANADA	2000	174-213	Multiple	X			NU2800 (9.19' deep)		X	
North Scenic Drive Bridge	Lethbridge	AB	CANADA	1984	134.5	Single	X			5'-6" Deep Bath-Tub			
Crowsnest Trail	Lethbridge	AB	CANADA	1985	118	Single	X			5'-6" Deep Bath-Tub			
Contract M	Lethbridge	AB	CANADA	1984	130.5	Single	X			5'-6" Deep Bath-Tub			
33rd Avenue S.W.	Calgary	AB	CANADA	1982	87-87	Multiple			X	3'-6" Bath-Tub Girder			
17th Avenue S.E.	Calgary	AB	CANADA	1980	134.5-138	Multiple			X	4'-4" Bath-Tub Girder			
10th Street LRT Bridge	Calgary	AB	CANADA	1986	65.33-124.17	Multiple	X			6'-0" (var.depth) Bath-Tub			
Bow River Bridge @ Deerfoot Trail	Calgary	AB	CANADA	1982	114.8-157.4	Multiple			X	6'-6" Bath-Tub Girder			
James McDonald Bridge	Edmonton	AB	CANADA	1970	185-250	Multiple				I girder (var.)			
Rainbow Valley Bridge	Edmonton	AB	CANADA	1980						Precast Box girder			

TABLE A3-8 (cont.)   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Airport Hwy 2 Bridge		AB	CANADA	1982						Trapezoidal tub			
36 Street Bridge	Calgary	AB	CANADA	1982						Trapezoidal tub			
Bow River Bridge	Calgary	AB	CANADA	1985						Trapezoidal tub 6' Bath Tub			
10th Street LRT Bridge	Calgary	AB	CANADA	1986	610	87-110-110-114-124-65				6' Bath Tub			
Norquay O/C	Banff	AB	CANADA	1984	145	145				6' Bath Tub			
Eisenhaer Junction O/C	Banff	AB	CANADA	1985	320	160				6' Bath Tub			
Oldman River Bridge	Taber	AB	CANADA	2001?	189'-203'	189'-203'-203'-203'-189'		X		2800 mm BT Nu			
Highway 2 over Bow River	Calgary	AB	CANADA	2002	174'-213'	174'-213'-213'-174'		X		2800 mm BT Nu			
Benchlands Trail	Canmore	AB	CANADA	1996	134'-138'	134'-138'		X		1755 mm Bulb Tee			
Deerfoot Trail over 196th Avenue	Calgary	AB	CANADA	2002		161'-161'		X		2000 mm Bath-tub			
Anthony Henday Drive	Edmonton	AB	CANADA	1998		144'-131'		X		1700 mm Bulb Tee			
Stoney Trail over CP Rail	Calgary	AB	CANADA	1999?	106'-110'	106'-110'-110'-106'		X		1800 mm Decked Bulb Tee			

TABLE A3-8 (cont.)   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Highway 927 over Pine Coulee Reservoir	Pine Coulee	AB	CANADA	1998		90'-123'		X		1500 mm Bulb Tee			
Highway 16 over Highway 14	Stoney Plain	AB	CANADA	1998?		135'-135'		X		1755 mm Bulb Tee			
53rd Street over Crowchild Trail	Calgary	AB	CANADA	2002		119'-118'				1600 mm Bath-Tub			
Whitemud Drive over Highway 14	Edmonton	AB	CANADA	2001		96'-121'-121'-96'				1700 mm Bulb-Tee			
18th Street over Glenmore Trail	Calgary	AB	CANADA	2002		128'-128'				1650 mm Bath-Tub			
Country Hills Blvd over Deerfoot Trail	Calgary	AB	CANADA	2002		190'-190'				2000 mm Bath-Tub			
Trent River Bridge No. 3078 - Inland Island Highway	Courtenay	BC	CANADA	1998	108.3-131.2	Multiple		X		BC Type 6 I-girders (78.75" deep)			
Moberly Bridge No. 1176 - Highway No. 29 - Moberly River	Chetwynd	BC	CANADA	1991	55.77-68.9	Multiple		X		BC Type 3 I-girder (44" deep)	X		
Esker Overhead No. 2928 - Yellowhead Highway 16	Terrace	BC	CANADA	1990	164.04	Single	X			BC Type 7 I-girder (90.55" deep)			X
Floods Underpass No. 2716 - Trans Canada Highway No. 1	Hope	BC	CANADA	1986	83.66-116.47	Multiple		X		BC Type 5 I-gider (68" deep)	X		
Thacker Creek Overpass No. 2826 - Trans Canada Highway No. 1	Hope	BC	CANADA	1986	59.06-98.42	Multiple		X		BC Type 4 I-girder (58" deep)	X		

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TABLE A3-8 (cont.)   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
YVR Overpass No. 2. Grant McConachie Way.	Richmond	BC	CANADA	1986	137.8	Single	X			Curved trapez. spine-box: 79" high, 114" wide - top; 312' radius.	X		
Jones Hill Underpass No. 2851 - Trans Canada Highway No. 1	Hope	BC	CANADA	1986	114.83	Single		X		BC Type 5 I-girder (68" deep)			X
King George Flyover No. 2908 - King George Highway	Surrey	BC	CANADA	1990	118.1-141.08	Multiple	X			BC 54" deep U-Girder	X		
Duke Point Underpass No. 3121 - Trans Canada Highway No. 1	Nanaimo	BC	CANADA	1997	68.9-124.67	Multiple	X			BC 54" deep U-girder			X
Bloedel Creek Bridge	Near Courtenay	BC	CANADA	1998	147	Single				"I" Girder			X
Commercial Drive Bridge	Vancouver	BC	CANADA	1989	62.34-121.39	Multiple	X			BC 54" deep U-girder	X		
Nile Creek Upstream Bridge	Nanaimo	BC	CANADA	1996	155.2-174	Multiple		X		2.3m BCMoT I-girder	X		
Murray Bridge	Merritt	BC	CANADA	1986	46-131.2	Multiple	X			BCMoT 2.3m I-girder	X		
Brook Bridge	Merritt	BC	CANADA	1986	46-121.4	Multiple	X			BCMoT 2.3m I-girder	X		
Pothole Creek Bridge	Merritt	BC	CANADA	1990	101.7-114.9	Multiple		X		BC MoT 1727girder	X		

TABLE A3-8 (cont.)   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
Pennask Creek Bridge	Merritt	BC	CANADA	1990	101.7-114.10	Multiple							
208 St Overpass -- Bridge #7561	Township of Langley	BC	CANADA	1999	148-148	Multiple	X			Type 6 (200mm deep) I-girders	X		
Rosewall Creek Bridge	Comox	BC	CANADA	1996	146	Single	X			90.5" deep MOTH BCType 7 I-Girder	X		
McKenzie Avenue Overpass on Highway 17	Victoria	BC	CANADA	1993	147	Single	X			54" Deep MOTH BC Trapez. Box Girders	X		
Sproat Falls Bridge on Highway No. 4	Port Alberni	BC	CANADA	1991	85-141	Multiple			X	68" Deep MOTH BC Type 5 - I Girders	X		
Nile Creek Bridge	Vancouver Island Highway	BC	CANADA	1996	152-174	Multiple			X	I-Girder/7.55' Deep	X		
Annacis Channel West Bridge - Highway 91	Vancouver	BC	CANADA	1997	152-230	Multiple			X	108" Deep MOTH BC Type 8 I Girders	X		
Annacis Channel Bridge	Vancouver	BC	CANADA	1984	196-233	Multiple	X			I-girder			
Floods Underpass	Hope	BC	CANADA	1985						I-girder			
Thacker Creek Overpass	Chilliwack District	BC	CANADA	1985						I-girder			
Overpass No. 2	Vancouver	BC	CANADA	1986						Precast box girder			

TABLE A3-8 (cont.)   LIST OF BRIDGES – CANADA

Name of Bridge	Location			Year Complete	Span Range	Span Arrangement	Splice Location			Girder Type	Type of Contract		
	Place	State	Country				In-Span	Pier	Both		Design/ Bid/Build	Design /Build	VECP
King George Flyover	Surrey	BC	CANADA	1989						Trapezoidal tub			
Esker Overhead	Skeena	BC	CANADA	1990	173.25	Single	X			I-girder			
Colony Farm Overhead	Vancouver	BC	CANADA	1982	900	6 @ 100				Curved Trapezoidal Girder 5.5' deep			
Kildonan Corridor	Winnipeg	MB	CANADA	1989						I-girder (var.)			
Provencher Bridge	Winnipeg	MB	CANADA	1990	166-195	Multiple	X			I-girder			
Cumberland House	Cumber-land House	SK	CANADA	1996?		194'-194'-194'-194'			X	3000 mm Bulb Tee			
Cockshutt Road Bridge	Brantford	ON	CANADA	1977	126.25-159.17	Multiple	X			I-girder			
Kingston Road Bridge	Scar-borough	ON	CANADA	1977	130-166	Multiple	X			I-girder			
Credit River Bridge	Mississ-auga	ON	CANADA	1982						I-girder			
Eglinton Ave. Bridge	Mississ-auga	ON	CANADA	1990						I-girder			
Highway #403 over credit river		ON	CANADA		119-160	Multiple	X			I-girder			

TABLE A3-9   LIST OF BRIDGES – OTHER COUNTRIES

Name of Bridge	Location Place	State	Country	Year Complete	Span Range	Span Arrangement	Splice Location In-Span	Pier	Both	Girder Type	Type of Contract Design/ Bid/Build	Design /Build	VECP
SH16 - Northwest motorway - Rosebank Patiki Interchange	Auckland		New Zealand	1997	84-138	Multiple			X	5.9' deep T-girder	X		
Tan Shui River Bridge / Spiced girder with 'Y' piers	Taipei		Taiwan	1982	170-262	Multiple			X	T-girder			
New Providence to Paradise Island	Common-wealth of the Bahamas		Bahamas	1999	190	Multiple			X	Modified Type VI		X	

OPTIONS FOR EXTENDING SPAN RANGES

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APPENDIX B
OPTIONS FOR EXTENDING SPAN RANGES

To utilize the potential of precast prestressed concrete girders for longer span bridges, designers must be aware of the design options available to reach the longer spans. Therefore, a goal of this research project was the identification of existing and potential options for extending span ranges.

The findings on this topic are presented in this appendix in two parts. The options presented here apply to extending conventional precast prestressed concrete girder spans as well as spliced girder designs.

Appendix B1   Design Options for Extending Spans

A significant number of design options for extending spans have been identified based on the information collected from owners, designers, contractors and producers and from a review of relevant literature. The research team has added several additional alternatives by considering the range of possible variables involved in bridge design. The focus of this appendix is on methods for extending span ranges without using splicing, with only a short section on spliced girder construction.

Appendix B2   Comparative Design Study

A comparative design study was performed to assist designers in identifying the potential benefits of several of the most promising design options for extending spans identified in the study. This was illustrated using a typical simple span bridge with a set girder spacing, for which maximum possible span lengths were computed using the selected options. While the study is limited in scope, it does demonstrate trends that may help designers identify potentially beneficial concepts for a given project.

APPENDIX B1

DESIGN OPTIONS FOR EXTENDING SPANS

APPENDIX B1
DESIGN OPTIONS FOR EXTENDING SPANS

Using information collected from owners, designers, contractors, and producers and from a review of relevant literature, the research team has identified a wide variety of design options for achieving extended span lengths for precast prestressed concrete girder bridges. Additional design options have been developed by the design team by considering the range of possible variables involved in bridge design.

The variety of possible options is the result of differences in experience, equipment, site constraints and other conditions. Therefore, methods listed in this appendix may not be feasible in all areas or they may fail to produce a significant economical advantage for a particular set of circumstances. Designers should confer with owners, fabricators and contractors before implementing extended span methods to ensure that the use of a selected technique will offer the potential for greater economy without introducing unmanageable risk, uncertainty or adverse effects.

The different design options vary significantly in degree of complexity, effectiveness and practicality. It was also found that designers often combine several options for extending spans to achieve design objectives.

The following listing of design options is divided into four groups:

- Material related methods
- Design enhancements
- Methods utilizing post-tensioning
- Spliced girder construction

Within each group, the list is arranged in approximate order of increasing complexity or design effort. Advantages and limitations of each design option are given where appropriate.

The treatment of spliced girders in this listing is relatively brief. Because they offer the greatest potential for significantly increasing span ranges beyond conventional limits, the many details associated with spliced girder construction are presented in **Appendix C1**. Design and construction details are very important to spliced girder bridges and may make the difference between a competitive, economical alternate that is selected for use on a project and one that is never built.

1 Material Related Options

Several design options for extending span ranges are related to material properties. These options can be implemented with standard design procedures and software.

1.1 High-Strength Concrete or Enhanced Concrete Properties

While the use of high performance concrete (HPC) has been encouraged by the FHWA and others in recent years, much of the increase in performance has been directed toward durability. Improvements in this area will not directly affect the span ranges of precast, prestressed concrete bridge girders. However, accompanying increases in other material properties of high performance concrete, as mentioned below, can have a noticeable impact on span ranges of precast concrete girders.

It has been demonstrated by a number of researchers that high strength concrete can be used to extend the span ranges of precast, prestressed concrete bridge girders (1, 2, 3). The increased concrete strength is most beneficial when used in the girder, but may also provide some benefit when used for the deck.

Some designers have specified concrete with an increased modulus of elasticity. This provides some benefits, but they are minor compared to the increase in strength.

While not a direct factor in extending span ranges, improved concrete workability has proven essential to facilitating other improvements that result in increased span ranges. Workability is a major factor in the ability to fabricate girders with narrower and/or deeper webs and with post-tensioning ducts. Without improved workability of the concrete, proper consolidation of concrete for tall, narrow cross-sections with and without post-tensioning ducts would not be possible.

Concrete with enhanced properties typically has an increased cost compared to standard concretes. However, the increased material cost is frequently offset by a reduction in the number of girders or spans required for a given bridge. Workability and production issues must also be considered.

1.2 Specified Density Concrete

Reduced density concrete, or lightweight concrete, has been used for many years to reduce the weight of bridge members. This reduced density concrete, or specified density concrete (SDC) as it is now being called, can be used in decks and girders to reduce the weight for design purposes as well as to reduce the weight of girders or girder segments to be transported. In some states, where load limits for transporting girders on highways are restrictive, lightweight aggregates may be combined with normal weight aggregates to reduce the unit weight of the girder concrete just enough to satisfy shipping requirements. In other cases, the same method has been used to reduce the dead load of the structure. Bridge decks constructed with lightweight concrete have been found to have equal or better durability when compared to normal weight concrete decks subjected to the same environmental and traffic conditions (4).

If SDC is used for girders, consideration of the potential for increased losses and reduced modulus of elasticity must be considered. Issues of concrete production and finishing with lightweight aggregates must also be considered. The addition of lightweight aggregate as another type of aggregate

with special care requirements may be difficult in some batching operations. Finishing of lightweight concrete for decks may also require special attention, but has been used successfully in many projects.

It should be noted that there is significant variability in the quality and performance of lightweight aggregates from different sources. Availability of aggregates capable of producing the desired properties of fresh and hardened concrete should be investigated prior to use, and project specifications should clearly indicate the required aggregate or concrete mix properties.

1.3 Increased Strand Size

It has been well documented that increased strand size in a given girder cross-section and strand pattern will increase the span length of precast, prestressed concrete bridge girders. The 42% increase in strand area from 0.5-in.-diameter strand to 0.6-in.-diameter strand can significantly increase the maximum span attainable for bridge girders, especially when used in conjunction with increased concrete strength. It does not appear likely that strand diameters greater than 0.6 inches will be used for pretensioned construction in the near future.

The use of 0.6-in.-diameter strands on the same grid as is used for 0.5 in. diameter strands makes possible a greater level of precompression in the girder upon release. This provides increased resistance to tensile stresses under service loads, provided the girder is not overstressed prior to the application of loads. To take full advantage of the increased precompression it may be necessary to debond or shield additional strands at the ends, drape strands near the midpoint of the girder, or take other measures to control stresses at release. When using larger diameter strands, the increase in stress transfer length at the ends of the girder should also be consideration during design, particularly with regard to shear.

Larger strand sizes are stiffer and are therefore more difficult to handle in the prestressing plant. However, many plants have successfully used the larger strand. There are typically some one-time costs incurred by a prestress plant when using larger strand for the first time, because of required modifications. These may include modifying or replacing the stressing plates, upgrading the stressing equipment, and purchasing new chucks for anchoring the strands.

1.4 Increased Strand Strength

Producers of prestressing strand are now capable of producing standard sizes of strand with an ultimate tensile strength of 300 ksi. The 11% increase in strand capacity will allow an increase in the span lengths of bridge girders, similar to those obtained using larger diameter strands, provided that similar design considerations are taken into account.

Several revisions to the material specifications governing strands (ASTM A 416) are currently being considered, one of which is the introduction of 300-ksi strand. However, research on the transfer and development characteristics for the strand, as well as on stress corrosion characteristics, have yet to be performed. Since questions remain concerning some of the strand properties, approval of

the specification revisions appears questionable at the current time. As a result, it appears that use of higher strength strand in pretensioned bridge construction may not occur in the near future.

1.5   Decks Composed of Composite Materials

In recent years several concepts have been introduced for using composite materials, typically developed in the defense industry, for the construction of highway bridge decks. While this technology is untested, pilot bridge applications are being installed. If issues such as the attachment of the panels to precast, prestressed girders, details for attachment of barrier rails to the deck, long-term durability and current high material cost can be adequately addressed, bridge decks constructed using composite materials may be used to extend span ranges of precast, prestressed concrete beams by significantly reducing the weight of the deck structure. However, the potential increase in span length due to the decrease in deck weight may be offset by the lack of composite action between the deck and girder.

2   Design Enhancements

Several methods for extending span ranges are related to modifications or enhancements of design parameters or procedures. In most cases, these methods can be implemented with standard design procedures and software.

2.1   Modified Standard Girder Sections

Designers and precasters have frequently modified standard girder forms to increase the span range for a particular girder type. Designers should confer with local producers before proposing modifications to forms so that only economically feasible changes are made. The modifications that are used most often are discussed below.

2.1.1   Moving Side Forms In or Out

Side forms for girders can be moved closer together or farther apart relatively easily by modifying or replacing soffit (bottom) forms. Without further modifications, this changes all horizontal dimensions of the cross-section by the same dimension.

Moving the forms apart (e.g., for sections with 6 inch webs, such as the PCI bulb tee) can provide a sufficient web thickness to permit the use of post-tensioning ducts while maintaining adequate cover over the vertical shear reinforcement and post-tensioning duct. This modification will also increase the girder section properties for increased shear and bending resistance. This method has been used to extend the span range of simple-span girders and to significantly increase the span length of spliced girders without the use of haunched segments over the piers. It can also be used locally along sections of the girder where increased shear resistance is required, such as over interior supports of continuous bridges.

An example of this modification is shown in **Figure B1-1** for a PCI BT-72 girder.

Moving the side forms in (e.g., for typical girder cross-sections with 8 inch webs, such as AASHTO Types IV, V and VI) can reduce the weight of a girder without significantly reducing its structural properties.

In some areas, girder side forms are moved out to increase the web thickness, but then shim plates are installed on the inside faces of the bottom flange forms so the width of the bottom flange remains unchanged from the basic girder. This can be an advantage for a fabricator because the soffit form does not have to be changed from standard to modified girder designs, because the cost of new or modified soffit forms and the cost to remove and reinstall soffit forms can be significant.

2.1.2 Increased Bottom Flange Depth

The typical increase in bottom flange depth (and therefore the girder height) is 2 inches. This modification increases the section properties of the girder, but more importantly, it allows an additional row of pretensioned strands to be placed in the bottom flange. This modification has been used most often with PCI bulb-tee sections where the small bottom flange size limits the number of prestressing strand positions available. This revision is also demonstrated in **Figure B1-1**.

This modification is generally a fairly simple one-time modification to the girder forms, allowing the forms to "step up" for the increased bottom flange or to drop back down for standard designs.

2.1.3 Deeper Sections in Existing Girder Families

The deepest sections in the current AASHTO and PCI bulb-tee girder families are 72 inches deep. However, states such as Pennsylvania have extended this family to include girder depths up to 96 inches. On occasion, even deeper girders have been used. Use of the same shape for the top and bottom flange and increasing the height of the web allows use of the same reinforcement details (except for a taller stirrup) and possibly reuse of some or all of the existing side forms. A PCI bulb tee girder modified to achieve an 8 ft deep section for splicing is shown in **Figure B1-2.**

In many cases, precasters will be able to find economical methods to utilize existing forms for these deeper sections. Some fabricators have forms for which the same bottom piece is used for all girder heights and different upper pieces are bolted to it to form the varying web height and top flange. Other fabricators have forms with the same top and bottom flange form sections and different height filler sections to vary the height of the web. In either case, there is a significant cost attached to modifying the form.

2.1.4 Increased Top Flange Width

This modification can also be accomplished relatively easily in most cases. Increased top flange width improves section properties, increases the eccentricity of prestressing on the bare girder, and improves lateral stability of the bare girder during handling, transportation and erection. It also reduces the span of the deck slab, which can result in reduced forming costs, reduced deck

reinforcement and possibly reduced deck weight if a thinner deck can be used. The wider top flange does require more concrete in the buildup, especially with increased superelevation or curvature of the deck.

2.1.5 *Increase Top Flange Thickness*

Thickening the top flange has been used to accomplish two main objectives. The first is to increase the girder section properties to extend the span *(5)*. The second is to provide more space in the top flange so that post-tensioning could be more easily installed in a haunched pier segment *(6)*. An additional benefit of increased top flange thickness is the ability for the section to better resist construction loads such as deck form loads that are applied at the flange tips.

2.1.6 *Deck Cast With Girder (Decked Bulb-Tee Girders)*

Casting all or significant portions of the deck slab with the girder prior to release of the pretensioning strands has been used to great advantage in some parts of the country. An example of this type of girder cross-section, generally referred to as a "decked bulb tee," is shown in **Figure B1-3.** Having the deck in place when the prestress is applied reserves more of the prestress force to resist applied loads, rather than using it to resist the deck dead load when applied later as in conventional construction. Also, there is a larger area for the prestress to act upon, which allows the application of a greater prestress force to the section. The prestress eccentricity is also greater due to elevated position of the neutral axis in the web. Very long or very shallow structures are possible with this type of cross-section *(7)*.

Decked members offer great potential for rapid construction because the deck is placed with the girder. Therefore, no deck forming is required where the full deck is cast on the girders. Once erected, only connections between the girders have to be made before a wearing surface can be applied and the bridge is ready for traffic.

In some cases, where a cast-in-placed concrete composite deck is desired, only a portion of the flange depth may be cast with the girder. While this type of solution has some benefits, especially since there is no deck forming, some of the speed of construction is lost because a cast-in-place deck must be placed.

Several issues must be considered when using this type of cross-section. Since there is no composite deck slab, camber growth can become a problem unless considered in the design. Also, the sections are quite heavy, which can be a significant obstacle where shipping weights or equipment capacities are restricted. Because of the increased weight of these sections and the possibility that pretensioning alone may not be sufficient to provide the desired prestress force, they are often candidates for the use of post-tensioning and splicing.

Cross-slopes or super-elevation must also be accommodated. This requires that the deck must taper across the flange, or the girder is set normal to the cross-slope. There are also related issues that need to be considered when these girders are skewed.

2.2 New Standard Girder Sections

Precast prestressed concrete girders have become an economical bridge solution in large part because of the standardization of girder shapes. However, in recent years, several new girder cross sections have begun to be used in different parts of the country. The new girder shapes are generally more efficient in some ways than the existing shapes, but the difference is generally not large.

A new and an existing girder shape are compared with the PCI BT-72 in **Figure B1-4**. Information on other new section shapes can be found in several articles in the PCI Journal in the last several years. *(8,9,10)*

Since precasters will have to purchase new girder forms to accommodate new girder shapes, the economy of new girder shapes is an important consideration. If a project has a significant number of prestressed beams, the job may be able to bear a significant portion or even all of the cost of the new forms. However, if the project is relatively small, and no new projects using the same shape are ready for bid, precasters may be reluctant to bid the job competitively.

2.3 Modify Strand Pattern or Utilization

2.3.1 Reduced Strand Spacing

In the past, reduced strand spacings were used in some areas of the country. These strand spacings did not satisfy the requirements of the *Standard Specifications*. However, performance of the girders was good. For a time, use of these reduced spacings was disallowed by FHWA pending additional research. It has now been shown that reduced spacings can provide adequate structural performance, so the *LRFD Specifications* have been revised to allow reduced strand spacings (LRFD Art. 5.10.3.3.1). Further reduction in strand spacing is not expected. While a significant number of states are using 0.6-in.-diameter strand at the reduced spacing of 2 inches on center, few states have used the reduced spacing of 1.75 in. on center for 0.5-in.-diameter strand.

2.3.2 Draped Strands Bundled at Drape Points

The minimum spacing requirements for strands are only required at or near the ends of pretensioned members. The spacing may be reduced at other locations. This allows the use of bundled strands, where draped strands are depressed in a group with strands touching. Typical hardware for bundling is shown in **Figure B1-5**, with standard draping hardware with rollers that maintain the 2 in. grid spacing at the drape point shown at the left side of the figure. In many cases, the fabricators manufacture their own hold-down devices, or have the devices manufactured for them. Therefore, the hardware varies as demonstrated by the two devices shown in the figure.

In most cases, the strands are placed in the bed at their highest position and an initial prestress force is applied. The hold-down devices are then pushed down using hydraulic equipment to the desired location. One or more strands running through the hold-down device are typically used to push the hold-down devices into position and hold them there. An example of strands that have been bundled appears in **Figure B1-6**.

Most states do not use this technique but maintain the spacing between strands for the full length of the girders. Bundling of strands slightly increases the eccentricity of draped strands between the drape points, which improves the efficiency of the prestress force applied by the strands. However, designers must confer with owners and fabricators to determine whether this technique is acceptable. Precasters may have to modify their stressing procedures and possibly their prestress bed to accommodate bundling of strands.

2.3.3   Debonded Top Strands

For long span girders, the quantity of prestressing that can be applied to a girder may be limited by the allowable concrete stresses at the ends of the girder at release. One method for improving the stress conditions at the ends of the girders is to provide additional top strands. However, these strands are detrimental at midspan. Therefore, to eliminate the force in the top strands, the strands must be debonded for the center region of the girder. A length of strand is bonded at each end to anchor the strand and provide the needed force at the end of the beam to satisfy the stress requirements at release. This is shown schematically in **Figure B1-7**. To make the debonding effective at the center of the girder, access ports are provided at midspan so the strands can be detensioned by heating. The top strands are frequently left tensioned in the girder following release and detensioned after erection to take advantage of the camber control and improved lateral stability provided by the additional prestress in the top flange.

This option requires no modifications to design or fabrication procedures other than the unusual use of debonding for the center portion of top strands, and the provision of a port in the top flange to access the strands for detensioning.

2.3.4   Transformed Strands and/or Reinforcement in Section Properties

Some owners allow designers to transform the area of mild reinforcement and/or prestressing steel into an effective concrete area for stress calculations. This provides slightly improved section properties for resisting applied loads. Designers should exercise caution when using transformed sections to ensure that elastic shortening losses are properly considered.

2.4   Girders Made Continuous for Live Loads

A popular approach to gain additional span length from precast, prestressed concrete girders is to use full-span girders that are connected by continuity diaphragms at interior supports within a continuous unit. The continuity diaphragms close the joint between ends of girders to provide resistance to loads placed on the completed structure. Either mild steel reinforcement or prestressing strands are

typically extended from the ends of the girders into the continuity diaphragm to make a positive connection between the girders and the diaphragm. The continuity diaphragms are generally placed at the same time as the deck slab.

This simple construction method has been widely used for many years. Methods have been developed to simplify design, which seem to have worked well. However, there are situations where cracking has developed in the continuity diaphragms and occasionally cracking occurs in the girders near the continuity diaphragms. Recently completed NCHRP Project 12-53 has studied the behavior of bridges using this type of detail.

2.5   Girders Shored During Placement of Deck

By shoring precast, prestressed concrete girders when the deck concrete is placed, the span length of the girders can be extended. This technique was successfully used on a project in Florida *(11)*.

The concept is simple. With the girder shored during placement of the deck concrete, the stress in the girder due to the weight of the deck can be greatly reduced, eliminated, or even reversed from what it would be if the girder were unshored when the deck was cast. When the shores are removed, the weight of the deck must be carried by the girders, but it is resisted by the entire composite section instead of by the girder alone. The net effect is that the dead load of the deck uses much less of the girder prestress capacity, leaving more "capacity" to resist the applied live load. Because the prestressing is utilized more efficiently, the same girder cross-section can be used to span a greater distance. See additional discussion in **Section 2.10.2** of **Appendix C1**.

The main disadvantage to this system is the additional cost associated with the shoring tower. The tower must be rigid so that significant deflections do not result from deformation of the tower or settlement of the supporting soil or substructure elements when the load from the deck concrete is added. On the Florida project cited above, the cost was reduced by utilizing "touch" shoring, located only at mid-span. An additional advantage of limiting the shoring to this location is that the deck load induces negative moment curvature at mid-span, thereby increasing the positive moment bending capacity under service loads. In some cases, shoring may not be practical because of requirements for maintenance of traffic or other concerns beneath the bridge.

2.6   Fixed End Condition at Abutments

Some bridges have been designed and built with details at the abutments that provide fixity. If bridges are fixed at the ends, longer spans can be achieved since positive moments are reduced.

However, the details at the abutments are unusual, requiring a cantilever to extend beyond the support to a location where it is tied down to the substructure unit. This short cantilever must be designed for high shear and moment. The tie-down must be capable of resisting significant uplift, and the reaction at the abutment wall will be significantly increased.

2.7   Cantilever Spans

For multi-span bridges, the end spans, or possibly interior spans as well, can be constructed to cantilever beyond an interior pier to extend the span between interior piers. A simple-span girder can then be placed between the cantilevers. This concept was employed in Florida *(12)* and elsewhere as some of the first attempts to extend span ranges of precast, prestressed concrete girders.

While not always required, the cantilever girders are often spliced. In some cases the splice has been achieved using mild steel reinforcement alone (i.e., no post-tensioning). The cantilever girders are typically fabricated in two parts: a uniform depth segment, and a haunched segment that forms the cantilever over the pier. These two segments may be joined on the ground so that the cantilever girder can be lifted as a unit, or the two pieces may be spliced in place. Once the completed cantilever girder is in place, a cantilever extends past the pier on both sides of the center span (assuming a three-span bridge).

To complete the bridge, a drop-in girder is placed across the main span, between the two cantilevers. Generally, the simple-span drop-in girder is supported on permanent bearings on the ends of the cantilever spans. There are expansion joints in the deck at each end of the drop-in span. The ends of the drop-ins and cantilevers are usually thickened with end blocks to accommodate the bearings and the transfer of forces in the dapped section.

The maximum span achievable using this method, without establishing continuity at the joint between the cantilever span and drop-in span, is limited to the maximum simple span for a girder plus the lengths of the two cantilevers.

2.8   Pier Tables

Cast-in-place pier tables can also be used to cantilever from a pier in order to extend the span ranges of precast, prestressed concrete girders. This concept is similar to the cantilever span described previously except that the connection between the pier table and the pier is fixed, allowing the pier to resist unbalanced moments as precast girders are set on both sides of the pier table to complete the spans. The pier table may be conventionally reinforced or post-tensioned, and the precast concrete girders may be simply supported or made continuous with the pier table.

This concept is similar to spliced girder designs where a pier girder segment is placed on the interior pier, cantilevering on both sides of the pier. It is recommended that the use of spliced girders with precast pier girder segments be used rather than a cast-in-place pier table to reduce the cost and time of construction.

2.9   Integral Caps

Integral caps can be used to introduce frame action at the connection between the superstructure and substructure. This distributes moments from the superstructure into the substructure, which enables longer spans to be used. This can be very important for bridges in seismic locations.

Integral caps are often used with spliced girders because continuity of the superstructure at interior piers is desirable. The integral caps are often post-tensioned longitudinally (above the length of the cap), but may also be designed using only mild reinforcement.

2.10 Improved Deck Design

Improved deck designs can be used to reduce the deck thickness and therefore the weight of the deck acting on the girder. It is unlikely that significant thickness reductions can be practically achieved for conventionally reinforced decks simply because of the cover and spacing requirements for the deck reinforcement. However, the use of transverse post-tensioning could make this strategy feasible, if the additional cost of transverse post-tensioning can be justified. An additional benefit of transverse deck post-tensioning is the reduction or elimination of longitudinal deck cracking and the accompanying deterioration that may occur with a thin deck and/or wide girder spacing. When transverse deck post-tensioning is coupled with other strategies for compressing the deck longitudinally, deck durability should be improved.

2.11 Refined Methods for Computing Live Load Distribution Factors

The *LRFD Specifications* allow designers to use refined methods to compute live load distribution factors for girder design. In most cases, the use of refined methods will lead to a significant decrease in the live load effect placed on a girder. The reduced live load effect will result in an increase in girder span lengths.

3 Methods Utilizing Post-Tensioning

The addition of post-tensioning to a bridge is the single most important way to significantly increase span lengths beyond conventional prestressed girder limits. However, this increase comes with a price in terms of increased design and construction complexity. Fortunately, methods and details that have been used successfully on numerous past projects can be used to facilitate design processes and minimize difficulties during construction.

3.1 Combined Pre - and Post-Tensioning

Post-tensioning can be effectively combined with pretensioning to maximize efficiency of a girder section in order to increase span ranges. This can be done either for single piece girders or for spliced girders. The use of post-tensioning allows for flexibility in tendon placement with greater eccentricity at midspan (like bundling previously mentioned), and reduced eccentricity at the ends. With post-tensioning, the prestress can also be applied to the final cross-section (composite or decked) where it is most effective.

A major disadvantage of adding post-tensioning to a girder is increased complexity in forming and the additional work that must be done in either the plant or field to stress and grout the post-tensioning tendons. Where post-tensioning tendons are anchored in the girder, anchor blocks must be formed to provide additional section width and reinforcement fabricated to resist concentrated loads

at the anchorages. The anchorage zones typically cannot be contained within the standard girder cross-section, so the forms must be modified or special forms purchased to form the anchorage zones (see discussion in **Section 3.2 in Appendix C1**). If post-tensioning is done at the site, the contractor may need to hire a sub-contractor to perform the tendon installation, stressing and grouting. This adds complexity to the construction process and uncertainty to his schedule.

3.2   Staged Post-Tensioning

Staged post-tensioning permits a significant increase in the amount of precompression that can be applied to a girder cross-section without overstressing the precast component prior to external load application. In staged post-tensioning, a portion of the post-tensioning (stage 1) is applied to the bare girder, generally after splicing, to provide the necessary prestress to resist the weight of the deck concrete. This also provides compression in any girder splices, if present. After the deck concrete has been placed and has reached a specified strength, a second stage of post-tensioning is applied, which completes the amount of prestress required to resist service loads. This post-tensioning applied to the completed structure also places the deck into compression, which enhances its durability.

One drawback to this method of construction is that access may be required for inspection of the post-tensioning anchor even after the deck concrete has been placed. This can complicate construction details and scheduling. However, generally adequate encasement of the anchorage is provided so that access is not required.

3.3   External Post-Tensioning

While uncommon in the United States, external post-tensioning can be applied to the bridge after the girders are erected and diaphragms with anchorages and deviation points are installed.

This method may hold promise for some special situations, but construction appears to be more complex. There may also be concerns with leaving post-tensioning ducts exposed to the elements unless they are protected inside a closed section, such as in a precast box girder or "tub" section.

3.4   Longitudinal Post-Tensioning in the Deck over Interior Piers

Longitudinal post-tensioning in the deck over interior piers has been used in some projects to counteract the tension that develops in continuous structures over interior supports. To use this method, the designer must address the issues of where to locate the anchorages and how to gain access to them for stressing. Providing access to anchorages from the top of the deck slab is undesirable because of the potential for recharge through the pourbacks and eventual corrosion in the anchorages and tendons. Providing access from below may not be feasible without imposing additional equipment and access requirements on the contractor.

4   Spliced Girder Construction

The use of post-tensioning enables prestressed girder systems to achieve spans significantly beyond conventional limits. As a result, precast prestressed concrete bridge girders have become longer, deeper and heavier. While this trend does not generally present a problem in substructure design or with the performance of the bridge in its final service condition, it does create practical problems regarding how to fabricate, transport and erect long and heavy precast components for these long-span bridges.

Almost since the very beginning of the use of prestressing in bridge structures in the early 1950s, engineers and contractors that have encountered limits on handling, transportation, and erection have resorted to splicing precast components together to obtain longer bridge girders. This concept has endured through the years, but now offers new potential to achieve significantly longer spans as a result of improvements in materials, design capabilities and construction methods in recent years. The range of possibilities for different methods of spliced girder construction to be used in conjunction with post-tensioning is great.

Basic concepts of spliced girder construction are presented in this section, while design and construction details are presented in **Appendix C1**.

4.1   Simple-Span Bridges

Simple-span spliced girder bridges are usually composed of two or more segments. These are generally spliced on falsework.   They may be bulb-tee, open-topped trapezoidal box (tub) or conventional AASHTO girder type structures with composite decks.

4.2   Multiple Span Continuous Bridges

There are three main concepts for this type of construction.   Other possible concepts for multiple span bridges are typically variations on the three concepts described below.

4.2.1   Full-Span Girders with Splices at Piers

This spliced girder concept has been used in a number of locations. It provides ease of erection consistent with the use of full span girders, but maximizes span lengths by utilizing continuous post-tensioning through multiple spans once the girders from adjacent spans are spliced together at pier locations. Post-tensioning is generally staged as described previously.

In the past, construction of this type has been closely scrutinized by contractors. Where the potential has existed to raise the bridge profile, deeper simple-span pretensioned girders have frequently been substituted for the continuous girders. This is typically done to eliminate the need for a specialty post-tensioning contractor, particularly where the volume of post-tensioning is small. However, this concept still remains an economical solution in a number of situations where extended length prestress girder bridges are required and full-span girders can be transported and erected.

4.2.2 Pier Segments with End Segments and Drop-In Spans

This spliced girder concept is generally used where the spans are too long for full-span segments.

Pier segments are generally erected first. Once erected, they must be stabilized prior to placement of the other segments. Temporary support towers are often used to provide stability. These are typically placed at splice locations. Pier segments may also be attached to the pier, either through temporary connections or through an integral cap, so that the pier segment is stabilized without falsework. This is most often employed where use of falsework may be prohibitively expensive, such as for very tall piers. Piers must be designed to resist moments caused by unbalanced conditions during erection.

End segments may be connected to pier segments on the ground, but are most often set on the falsework tower used to stabilize the pier segment. Drop-in segments are frequently supported from the end of the pier segments using strongbacks attached to the drop-in segments. Staged post-tensioning, as discussed earlier, is often used for this method to allow for removal of the false work towers before placing the deck concrete.

4.2.3 Multiple Segments in Each Span with Splices at Piers

This spliced girder concept can take two forms. The first is where erection equipment is available to erect full-span girders, but full-span girders cannot be transported to the job site. In this case, the girder segments are transported to the jobsite where they are assembled and post-tensioned prior to erection. The second is where girder segments are erected on falsework and post-tensioned in place, similar to the simple-span concept described above.

This concept may also be used where the girder cross-section is inappropriate for use as a pier segment (i.e., temporary cantilever condition), such as an open-topped trapezoidal box beam (U-beam or tub). It can provide the ease of erection attributable to full span girders, but permits span length increases by utilizing post-tensioning, which is usually stressed in stages.

4.3 General Advantages of Spliced Girder Bridges

There are many different reasons why post-tensioned spliced girder bridges are used instead of conventional construction methods. The following is a summary of some of the most prevalent reasons for using spliced precast, prestressed concrete girders.

4.3.1 Design and Site Constraints

Constraints due to elements of design or restrictions on the site are often motivations for the use of spliced girders.

- Eliminate piers, especially where piers and/or foundations are costly due to poor soils, tall piers and vessel impact or seismic requirements

- Span long distances where limitations on pier placement exist (e.g., over wider roadways, railway yards, shipping channels, rivers, etc.)

- Allow reduced superstructure depth when vertical clearances are restrictive

- Speed up construction, particularly in areas where it is necessary to minimize road closings or where construction seasons are short due to extreme weather conditions

- Enable construction when whether conditions are unfavorable to other materials or methods of construction (e.g., steel, cast-in-place concrete)

- Eliminate falsework where not permitted or desirable, or where soil support conditions are unfavorable

- Enable transport and erection of long, heavy precast, prestressed girders in pieces where longer spans are required or where environmental or access conditions (e.g., winding mountain roads) prohibit full length girders from being delivered to the site

4.3.2 *Improve Resistance for Extreme Events*

Seismic events and vessel impact are considered to be extreme events in the *LRFD Specifications*. Bridges subject to these effects are usually designed as continuous structures and often with integral substructure connections in order to maximize the resistance of the structure to extreme events.

- Provide continuity to increase safety and probability of survival in extreme event

- Enable load redistribution from the piers to the superstructure in vessel impact and seismic situations by providing superstructure continuity

- Enhance two previous advantages through the use of integral connections (i.e., moment transfer) between the superstructure and substructure

- Reduce foundation sizes and costs in seismic regions by using integral connections at the deck level and pinned connections at the footing level

4.3.3 *Minimize Inconvenience to Public*

Spliced girder methods can be used to improve the speed of construction in and around heavily traveled thoroughfares because they typically have shorter delivery times when compared to other design alternates. The use of precast elements can also reduce or eliminate falsework requirements during construction, which provides improved access and safety to the public.

4.3.4 *Eliminate Joints*

Splicing girders allows for long, multi-span bridges where the number of expansion joints is minimized. By eliminating expansion joints, the initial cost to construct the bridge is reduced by the cost and labor required to provide and install the expansion joints. With fewer joints, the long-term cost is also reduced, since drainage from leaking joints onto substructure elements is reduced or eliminated. Additionally, drainage leaking onto pier caps can lead to premature deterioration of the pier.

4.3.5   Owner Preference

Owners may encourage or specify consideration of precast, prestressed girder construction, even when span lengths exceed the capacity of conventional simple-span girder methods, for the following reasons:

- Obtain the advantages of precast, prestressed concrete construction (e.g., low initial cost, increased durability, reduced maintenance costs, quality from precasting, aesthetics, etc.) where it would otherwise not be possible

- Provide a competitive alternative for span ranges normally reserved for other material types or methods of construction

- Obtain the durability advantages of additional compression in the deck as a result of longitudinal post-tensioning

- Increase safety by eliminating piers in the medians or along the shoulders at grade crossings and interchanges

- Provide for deck widening or replacement, which may not be feasible with other alternatives (e.g., segmental concrete construction)

- Reduce live load deflections through increased stiffness as compared to other materials (e.g., steel girders)

- Improve aesthetics

- Reduce long-term maintenance in areas where repainting (steel structures) may prohibitively costly or undesirable (e.g., sensitive environments, mountain regions, over busy freeways) or where corrosion is a concern (e.g., salt water environments, freeze/thaw areas)

- Reduce construction schedules where lane closures would unduly inconvenience the traveling public

- Reduce construction schedules where fabrication of members from other materials may not be available to meet compressed project deadlines

4.3.6   Contractor Preference

With the increased use of alternate designs through value engineering cost proposals (VECPs), contractor alternates and design build projects, more bridges are being built under the direction of a contractor. These designs reflect what the contractor desires to see in a bridge rather than what the owner or designer may expect. In these situations, the use of spliced girders has increased as contractors select the system and material for the superstructure that they can construct most economically and rapidly.

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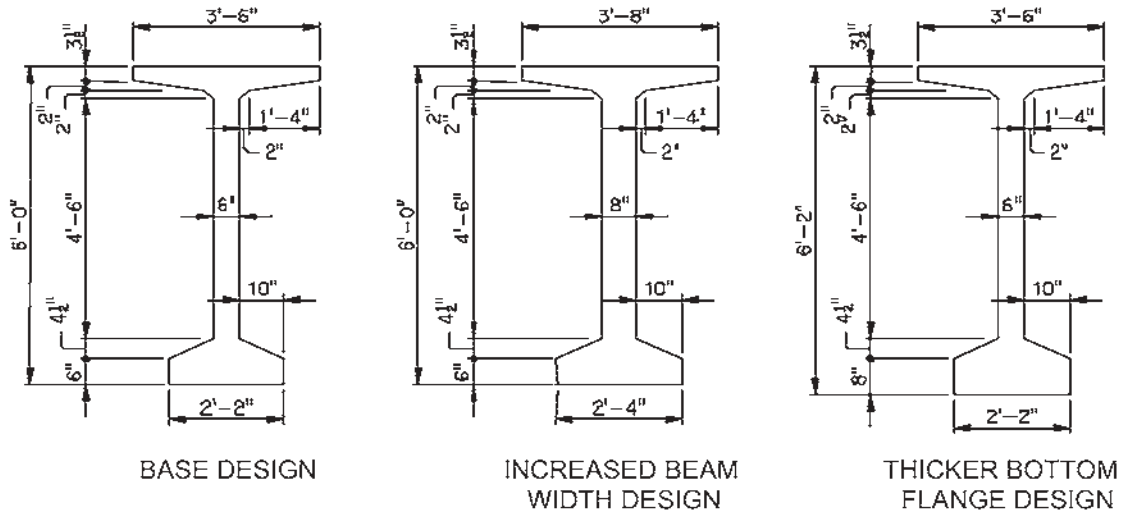
Figures

BASE DESIGN   INCREASED BEAM WIDTH DESIGN   THICKER BOTTOM FLANGE DESIGN

Figure B1-1   **Girder Dimensions for PCI BT-72 Modifications**
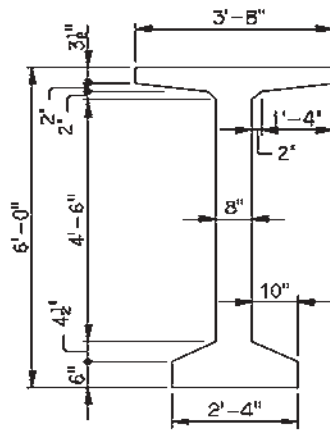
Figure B1-2   **Modified Standard PCI BT-72 Girder Section**
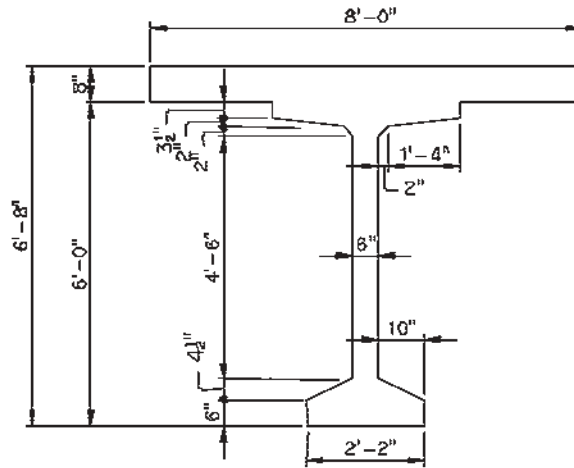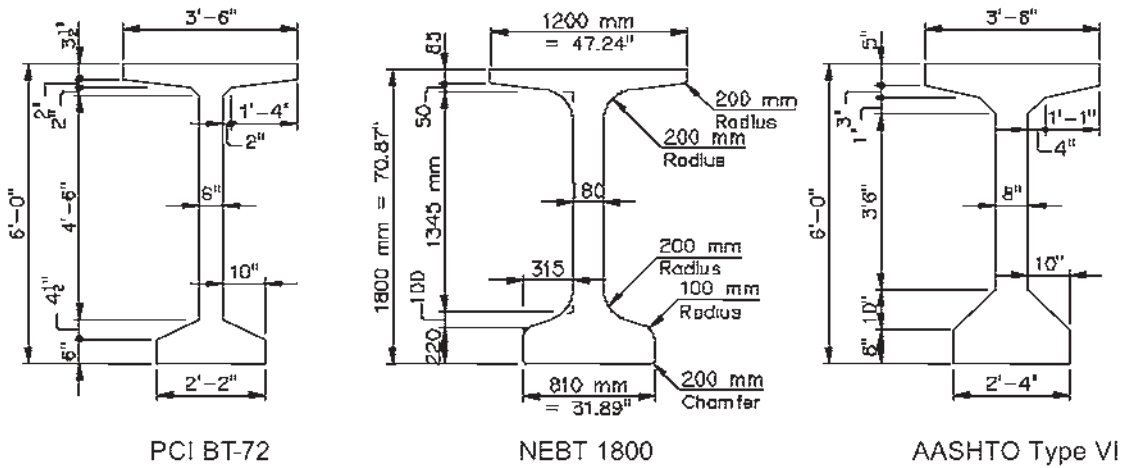
Figure B1-3   Girder Dimensions for Decked Bulb Tee

PCI BT-72   NEBT 1800   AASHTO Type VI

Figure B1-4   Girder Dimensions for Different Girder Types
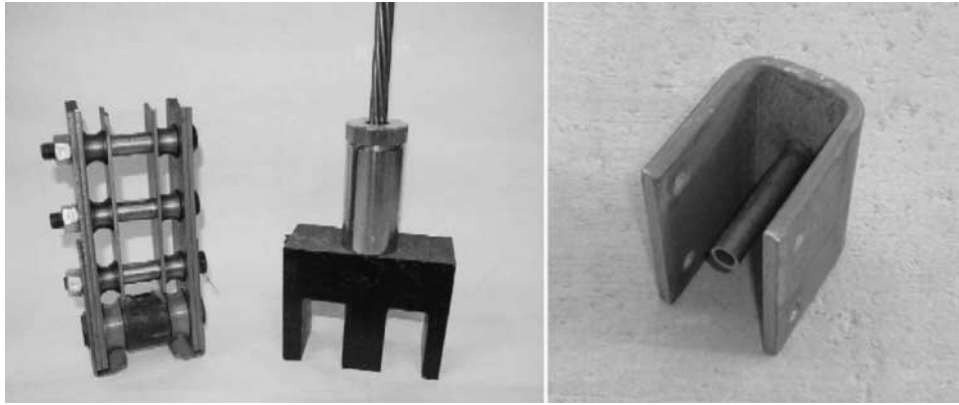
Figure B1-5   Standard and Bundled Strand Hardware

Figure B1-6   Bundled Strands in Bed
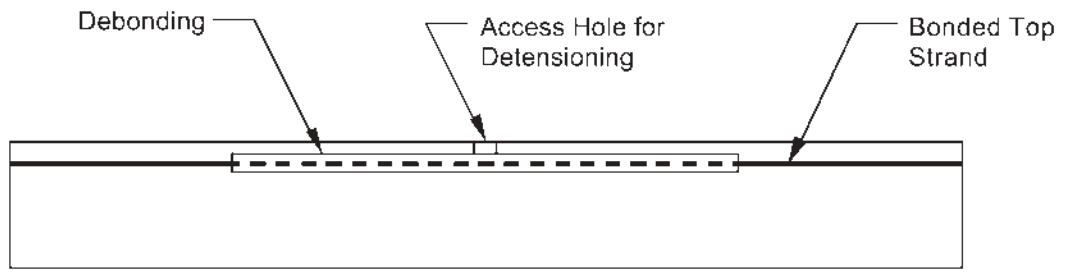
Figure B1-7   Schematic of Debonded Top Strands

COMPARATIVE DESIGN STUDY

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1	INTRODUCTION

This comparative study was prepared to demonstrate and compare the effectiveness of a number of the highest ranked techniques that have been identified for extending span ranges of precast prestressed concrete girders.

A single span, simply supported bridge was selected as the subject of the comparative design study. The typical cross section was comprised of precast prestressed concrete girders at a fixed spacing. Maximum spans were then computed for the bridge using several design cases. Some design cases were also compared using different girder types. In most cases, design cases were considered individually. Calculations were performed using a commercially available software package.

The results were compiled and compared to give an indication of the effectiveness of each design variation for increasing the maximum span. Results from this study should be considered as giving an indication of trends. For other situations, where girder spacing, section type and other conditions are changed, the relative effect of different alternates may vary from what is reported here. Combinations of different design cases are expected to provide additional benefit, although the effect may vary.

This study only considered flexural design of the precast prestressed girders. For the parameters considered in this study, the service limit state governed all designs. Conditions all along the beam were evaluated. Shear and deflections were not considered for this comparative study.

The presentation of this study follows, with an introduction of the design parameters considered, a description of the bridge being studied, a summary of the results. The details of the assumptions and parameters used in the comparative design study follow.

The study was conducted in two parts. The first considered a PCI BT-72 girder design with fourteen design variations. The second compared four design cases from the PCI BT-72 girder design with results using two other girder cross-sections of similar depth: A NEBT 1800 girder and an AASHTO Type VI girder.

1.1	DESIGN VARIATIONS FOR PCI BT-72 GIRDER

The first set of designs uses the PCI BT-72 girder. The fourteen design variations considered for this section are listed below. **Table 1.1-1** summarizes the basic design information for the base design and each variation. See **Section 3** for a detailed discussion of each variation.

- Increased f'_{ci}
- High Strength Concrete
- Lightweight Concrete Deck
- Lightweight Concrete Beam
- Lightweight Concrete Deck and Beam
- Deeper Bottom Flange
- Increased Beam Width
- Bundled Strands
- Top Strands Debonded
- 0.6 in. Diameter Strands
- 0.6 in. Diameter Strands and High Strength Concrete
- Decked Bulb Tee

- Touch Shoring
- Post-Tensioning of Base Girder

Table 1.1-1   Basic Design Information for Designs using PCI BT-72 Girder

Design Case		f'_{ci}	f'_c	f_{cd}	w_c	w_{cd}	Web Width	Bottom Flange Depth	Strand ϕ
No.	Description	(ksi)	(ksi)	(ksi)	(kcf)	(kcf)	(in.)	(in.)	(in.)
1	Base Design	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
2	Increased f'_{ci}	6.0	6.5	4.5	0.150	0.150	6.0	6.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	6.0	6.0	½
4	Lightweight Conc. Deck	5.0	6.5	4.5	0.150	0.120	6.0	6.0	½
5	Lightweight Conc. Beam	5.0	6.5	4.5	0.120	0.150	6.0	6.0	½
6	Lightweight Conc. Dk. & Bm.	5.0	6.5	4.5	0.120	0.120	6.0	6.0	½
7	Deeper Bottom Flange	5.0	6.5	4.5	0.150	0.150	6.0	8.0	½
8	Increased Beam Width	5.0	6.5	4.5	0.150	0.150	8.0	6.0	½
9	Bundled Strands	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
10	Top Strands Debonded	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	6.0	6.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	6.0	6.0	0.6
13	Decked Bulb Tee	5.0	6.5	6.5	0.150	0.150	6.0	6.0	½
14	Touch Shoring	5.0	6.5	4.5	0.150	0.150	6.0	6.0	½
15	Added Post-Tensioning	5.0	6.5	4.5	0.150	0.150	8.0	6.0	½

Shaded cells indicate values that differ from Base Design.

1.2   COMPARISON WITH DIFFERENT GIRDER TYPES

The second set of designs compares designs using the PCI BT-72 girder to designs using the NEBT 1800 and AASHTO Type VI girders. These girders are of equal or nearly equal height, as shown in **Figure 1.2-1**, but they differ in other dimensions (see **Figure 6.4.2.1-1**).
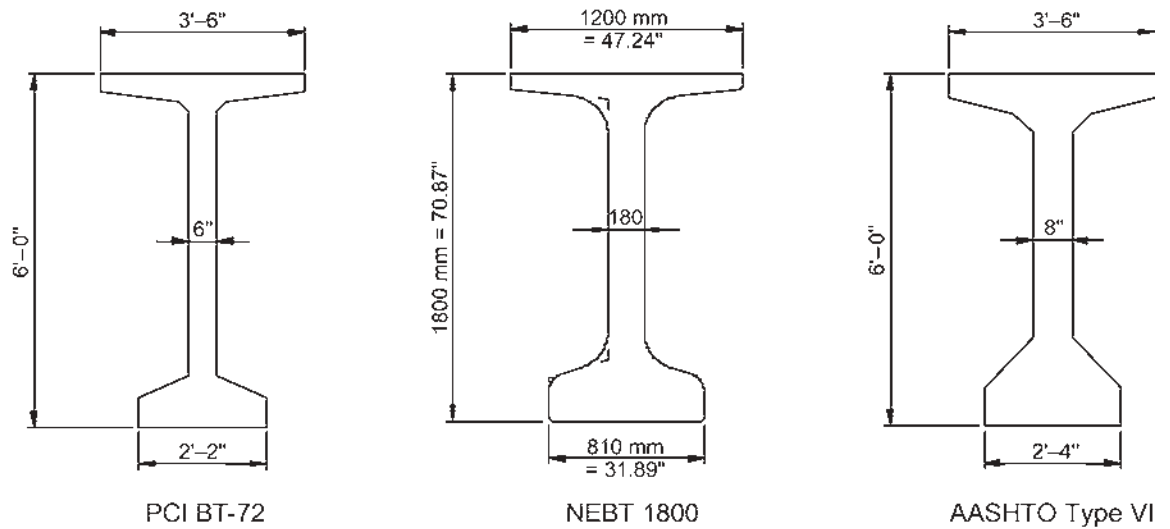
Figure 1.2-1   **Basic Girder Dimensions**

The four design cases considered for each cross-section type are listed below. The case numbers correspond to the case numbers in the comparison using the PCI BT-72 girder. **Table 1.2-1** summarizes the basic design information for each design combination. See **Section 3** for a detailed discussion of each variation.

- Base Design
- High Strength Concrete
- 0.6 in. Diameter Strands
- 0.6 in. Diameter Strands and High Strength Concrete

Table 1.2-1   Basic Design Information for Designs using Different Girder Types

Girder Type and Design Case		f'_{ci}	f'_c	f'_{cd}	w_c	w_{cd}	Web Width	Bottom Flange Width	Strand ϕ
No.	Description	(ksi)	(ksi)	(ksi)	(kcf)	(kcf)	(in.)	(in.)	(in.)
PCI BT-72 Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	6.0	26.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	6.0	26.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	6.0	26.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	6.0	26.0	0.6
NEBT 1800 Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	7.1	31.9	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	7.1	31.9	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	7.1	31.9	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	7.1	31.9	0.6
AASHTO Type VI Girder									
1	Base Design	5.0	6.5	4.5	0.150	0.150	8.0	28.0	½
3	High Strength Conc. (HSC)	8.0	10.0	4.5	0.150	0.150	8.0	28.0	½
11	0.6"ϕ Strands	5.0	6.5	4.5	0.150	0.150	8.0	28.0	0.6
12	0.6"ϕ Strands and HSC	8.0	10.0	4.5	0.150	0.150	8.0	28.0	0.6

Shaded cells indicate values that differ from Base Designs.

2   DESCRIPTION OF BRIDGE

The bridge is comprised of precast, prestressed concrete girders with a composite deck slab. The basic geometry of the bridge is shown in **Figures 2-1** through **2-3**. The span length of the bridge (center to center of bearings) varies for each case depending on the results of the design.

This comparative study considers only the design of an interior girder. Design of an exterior girder would be similar except for loads and section properties. For this bridge, the design of the interior girder governs.
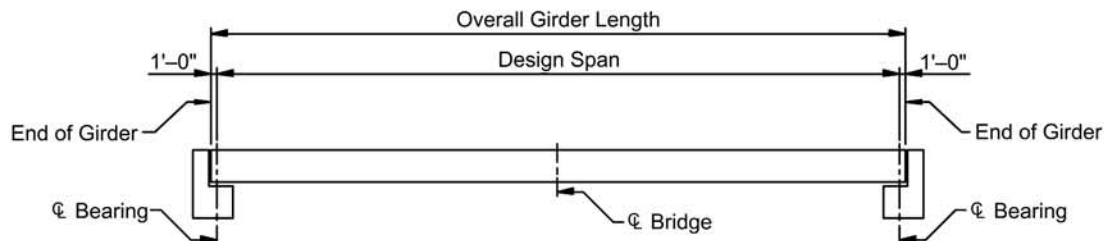
Figure 2-1   Elevation View of Girder
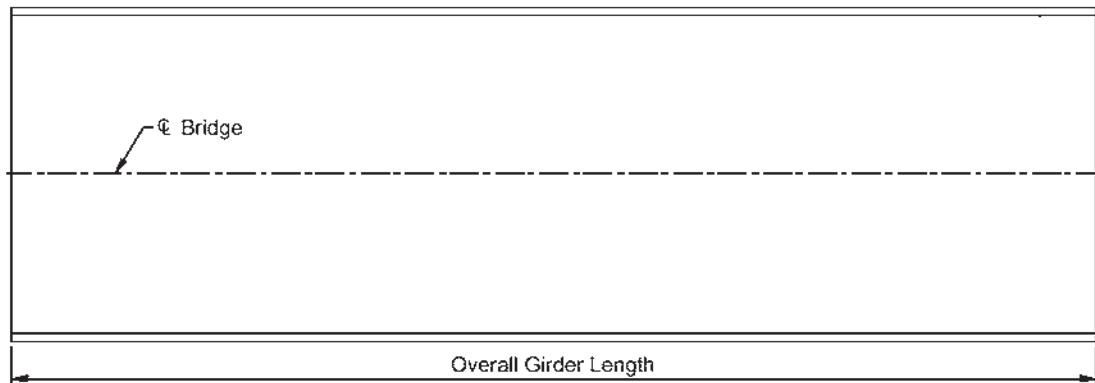
Figure 2-2   Plan View of Bridge
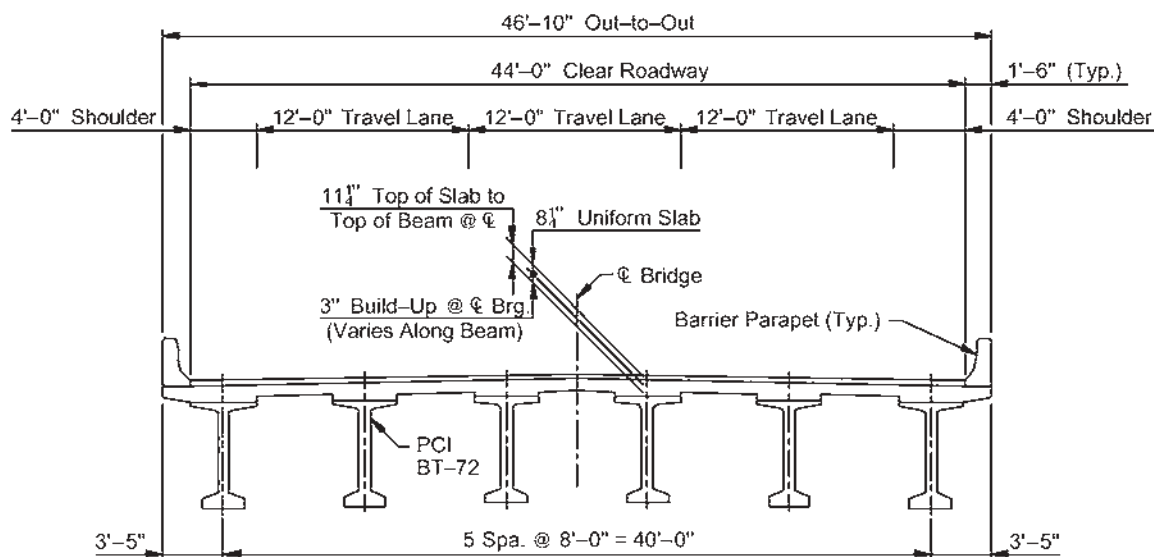
Figure 2-3   Typical Section of Bridge

3   DETAILS OF DESIGN CASES

Each of the design cases evaluated in this comparative study is described below. The variations in the basic design parameters create secondary effects, which include adjusted loading conditions, section properties, and adjustments of live load distribution factors. These additional variations from the base design are noted in the later sections of this study where further details of the design parameters are presented.

3.1   DESIGN CASES FOR PCI BT-72 GIRDER

This comparative study investigated fourteen variations on the base design using a PCI BT-72 girder as mentioned in **Section 1.1**. The variations ranged from examining the effect of concrete strength to modification of the girder cross-section. The fifteen design cases (the Base Design plus the fourteen variations) can be grouped into the eight categories of design cases listed below.

1. Base Design   *Case 1*

2. Concrete Strength Variations   *Cases 2 and 3*

3. Lightweight Concrete   *Cases 4, 5 and 6*

4. Modifications to Girder Cross-Section   *Cases 7, 8 and 13*

5. Stress Control with Respect to Strand Layout *Cases 9 and 10*

6. Variation in Strand Size   *Cases 11 and 12*

7. Touch Shoring   *Case 14*

8. Post-Tensioning   *Case 15*

Each design case is described below. Refer to **Table 1.1-1** for a summary of the basic design information for each case.

3.1.1 **Base Design (*Case 1*)**

The Base Design is representative of a typical design using the PCI BT-72 girder. Typical concrete strengths are used for the girder and deck. All concrete is normal weight concrete. Other aspects of design, such as strand patterns and design loads, are typical. See later sections of this study for details.

3.1.2 **Increased f'_{ci} (*Case 2*)**

The concrete strength at release for the precast girder was increased by 1.0 ksi to $f'_{ci} = 6.0$ ksi since conditions at release govern designs in some cases.

3.1.3 **High Strength Concrete (*Case 3*)**

A high strength concrete was used for the precast girder for this design case.

The 28 day concrete strength of the precast girder was increased by 3.5 ksi to $f'_c = 10.0$ ksi.

The concrete strength at release for the precast girder was increased by 3.0 ksi to $f'_{ci} = 8.0$ ksi.

3.1.4 **Lightweight Concrete Deck (*Case 4*)**

The unit weight of the concrete for the cast-in-place bridge deck was reduced from 150 pcf to 120 pcf. The strength of the deck concrete remained unchanged from the base design.

This option reduces the load placed on the bridge from the weight of the deck.

3.1.5 **Lightweight Concrete Girder (*Case 5*)**

The unit weight of the concrete for the precast PCI BT-72 girder was reduced from 150 pcf to 120 pcf. The strength of the girder concrete remained unchanged from the Base Design.

This option reduces the load placed on the bridge from the weight of the girder. Material properties change as a result of the use of lightweight concrete, which affects several aspects of the behavior of the bridge, including prestress losses and deflections.

3.1.6 **Lightweight Concrete Deck & Beam (*Case 6*)**

The unit weight of the concrete for both the deck and precast PCI BT-72 girder was reduced from 150 pcf to 120 pcf. The strength of both the deck and girder concrete remained unchanged from the base design.

This case combines the effect of Cases 4 and 5. Material properties change as indicated for Case 5.

3.1.7	**Deeper Bottom Flange (*Case 7*)**

The cross-section of the girder was modified by adding 2 in. to the depth of the bottom flange of the precast girder, which also increases the total height of the section by 2 in. See **Figure 6.4.1.1-1**.

The increased depth of the bottom flange allowed the use of an additional row of strands. See **Figure 7.1.1-1**.

3.1.8	**Increased Beam Width (*Case 8*)**

The cross-section of the girder was modified by the addition of 2 in. to the width of the girder web along the full height of the girder. Therefore, the top and bottom flange widths were also increased by 2 in. This adjustment can be easily accomplished by using a 2 in. wider soffit (bottom) form which spreads the girder side forms apart by 2 in. See **Figure 6.4.1.1-1**.

3.1.9	**Bundling of Strands (*Case 9*)**

The draped strands were bundled between hold-down points. This approach, which is permitted by the *LRFD Specifications*, increases the eccentricity, and therefore the effectiveness, for the draped pretensioning strands. For this design, the draped strands were bundled with four strands in a row (two groups of two) and with strands touching both vertically and horizontally. See **Figure 7.1.1-1**.

At the girder ends, the draped strands were positioned in the 2 in. grid used for other design cases, which is required by the *Specifications*.

3.1.10	**Debonding of Top Strands (*Case 10*)**

To reduce concrete stresses at release, girders were designed with additional top strands that were debonded at the center of the span with 30 ft bonded at each end. The bonded length depends on design requirements to satisfy stress limits. After erection of the girders, the debonded top strands were disabled by cutting according to typical detensioning procedures. Access holes were provided in the top flange to allow the strands to be cut. Cutting the top strands increases the total eccentricity of all strands in the center region of the beam, making the remaining strands more effective in resisting tensile stresses at midspan at the service limit state.

3.1.11	**0.6 in. Diameter Strands (*Case 11*)**

The ½ in. diameter strands were replaced with 0.6 in. diameter strands of the same strength. The larger strands provide over 40 percent more force than the smaller strands. The same strand pattern is used for the larger strands (2 in. by 2 in. grid), so the efficiency of the strands may be improved significantly.

3.1.12	**0.6 in. Diameter Strands and High Strength Concrete (*Case 12*)**

This case combines the effects of Cases 3 and 11.

The ½ in. diameter strands were replaced with 0.6 in. diameter strands.

The 28 day concrete strength of the precast girder was increased by 3.5 ksi to $f'_c = 10.0$ ksi.

The concrete strength at release for the precast girder was increased by 3.0 ksi to $f'_{ci} = 8.0$ ksi.

3.1.13　　**Decked Bulb Tee (*Case 13*)**

The cross-section of the girder was modified by the addition of an 8 in. (structural thickness) deck to the top of the standard PCI BT-72 as shown in **Figure 6.4.1.1-2**. The deck is present when the girder is prestressed at the release of the pretensioning strands. Therefore, it has the same concrete strength as the girder and is prestressed along with the girder.

Girders of this type are erected side-by-side. After erection, connections are made between the girders, generally at the edge of the slabs. This type of construction allows very rapid completion of the superstructure of a bridge.

Since the deck is present with the girder at release, the weight of the decked section is much greater than the weight of the girder alone. This may pose significant issues for the handling and transportation of the girder. In many cases, the girder will have to be fabricated in segments that are spliced at the site to complete the structure. Splicing normally requires the use of post-tensioning, which would require an increased web width. Neither the addition of post-tensioning nor the accompanying increase in web width have been included in this case.

No build up was included in this design since the deck is cast with the girder. The camber and deflections of this girder must be carefully considered as is required for any precast deck system.

3.1.14　　**Touch Shoring (*Case 14*)**

Before placement of the reinforced concrete deck, temporary shoring towers are installed to support the girder at midspan. The temporary tower is placed so it supports the girders without applying an upward load prior to placement of the deck concrete. This changes the span resisting the weight of the fresh deck concrete from a simple span to a two-span continuous structure, which reduces the effect of the weight of the deck on the girder. After the deck cures and reaches a specified minimum strength, the temporary shoring is removed and the bridge acts again as a simple span. The reaction present in the shoring is then transferred to the composite girder section. Therefore, the stresses at midspan caused by the deck concrete are reduced, since some of the effect of the weight of the deck is placed on the composite section rather than only the bare beam, as in conventional construction.

The success of this method is dependent on the ability of the temporary support to provide unyielding support at midspan. Any shortening of the tower, either due to shortening in the tower or compression of the supporting soil or girder bearing material, will result in a greater transfer of load into the bare girder than was assumed in the design.

3.1.15　　**Post-Tensioning of Bare Girder (*Case 15*)**

In this case, post-tensioning was added to the girder to increase the potential prestress that could be applied to the girder. The post-tensioning tendons consisted of up to 19 - ½ in. diameter strands placed in each of four 3.15 in. diameter ducts cast into the girders in a draped profile. The ducts were located along the center of the web and were placed in the section as low as possible at midspan. See **Figure 7.1.2-1**. End blocks would be required for anchorage of the post-tensioning tendons. The ducts would be grouted after tensioning.

The post-tensioning was applied after the deck concrete had been placed and had reached a specified strength. Applying the post-tensioning to the full cross-section rather than to the bare girder makes it more effective.

To accommodate the post-tensioning ducts, the cross-section of the girder was modified by the addition of 2 in. to the width of the girder along its full height as described in Case 8.

An amount of pretensioning steel was included in the design to satisfy stress requirements at release. The pretensioned strands could not be located in the web because of the presence of the post-tensioning ducts. Post-tensioning was used to provide the remaining prestress required to satisfy the design requirements.

3.2 DESIGN CASES FOR DIFFERENT GIRDER TYPES

To compare the effect of design cases on different girder types, four of the cases discussed above were repeated using NEBT 1800 and AASHTO Type VI girders. These four cases were:

1. Base Design   *Case 1*

2. High Strength Concrete   *Case 3*

3. 0.6 in. Diameter Strands   *Case 11*

4. 0.6 in. Diameter Strands and High Strength Concrete   *Case 12*

The four cases considered are identical to the cases listed above, so their descriptions are not repeated here. In the discussion of the results of these designs, the cases will be identified using the numbers of the cases as used in the PCI BT-72 girder designs.

4 SUMMARY OF RESULTS

4.1 DESIGN CASES FOR PCI BT-72 GIRDER

The results of designs for all cases are listed in **Table 4.1-1** which include maximum spans, the absolute and percentage increase in maximum spans compared to the Base Design, and notes concerning governing design condition(s). The maximum spans for each design case are presented graphically in **Figure 4.1-1** and the percent increase in maximum spans compared to the Base Design are shown in **Figure 4.1-2**.

Table 4.1-1   Summary of Results for PCI BT-72 Girder Design Cases

Design Case		Maximum Span	Span Increase	Percent Increase in Span	Notes
No.	Description	(ft)	(ft)	(%)	
1	Base Design	118	0	0.0	A
2	Increased f'_{ci}	126	8	6.8	A
3	High Strength Conc. (HSC)	130	12	10.2	B
4	Lightweight Conc. Deck	119	1	0.8	A
5	Lightweight Conc. Beam	122	4	3.4	A
6	Lightweight Conc. Dk. & Bm.	124	6	5.1	A
7	Deeper Bottom Flange	126	8	6.8	A
8	Increased Beam Width	124	6	5.1	A
9	Bundled Strands	118	0	0.0	A
10	Top Strands Debonded	122	4	3.4	A
11	0.6"ϕ Strands	118	0	0.0	A
12	0.6"ϕ Strands and HSC	138	20	16.9	A
13	Decked Bulb Tee	157	39	33.1	A,E
14	Touch Shoring	130	12	10.2	C
15	Added Post-Tensioning	147	29	24.6	D

Notes:

A   Critical stresses occur at release at midspan bottom in compression and at Service III at midspan bottom in tension.

B   Critical stresses occur at midspan at Service III despite room for additional strands. Additional strands do not provide sufficient precompression to offset the tension stresses due to applied moments.

C   Critical stresses occur at release at end of girder in compression in spite of draping and debonding.

D   Critical stresses occur at midspan in compression at the top of the girder.

E   Total weight of girder is 251 kips. Splicing may be required to address handling and transportation issues.

Figure 4.1-1   Maximum Span for Each Design Case
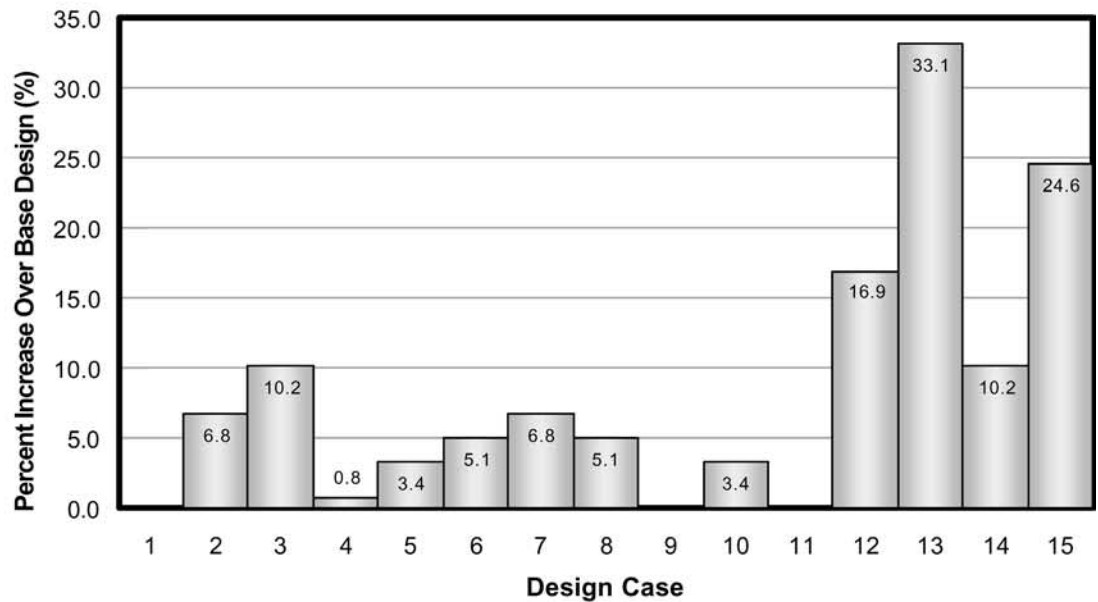
Figure 4.1-2   Percentage Span Increase for Each Design Case

4.1.1   Base Design (*Case 1*)

The Base Design for the PCI BT-72 girder yielded a maximum span of 118 ft. Even though locations were available for additional pretensioning strands, the addition of any strands caused stresses to exceed the tensile stress limit at the bottom of the girder at midspan under Service III conditions or compression at release. This combination of governing stress conditions was found to occur in almost all of the design cases for the PCI BT-72 girder.

4.1.2   **Concrete Strength Variations (*Cases 2 and 3*)**

Increasing the concrete strength at release by 1.00 ksi (Case 2), increased the maximum span by 8 ft (6.8 percent). The same over stressing mode experienced by the Base Design governs in this design variation as well. In spite of the initial cost increase associated with a higher concrete strength, this variation has the potential of being the easiest modification to increase span length without requiring post-tensioning, or touch shoring. This would not be effective if release conditions did not govern the design.

Increasing the concrete strength of the PCI BT-72 girders from $f'_c = 6.5$ ksi and $f'_{ci} = 5$ (Base Design) to $f'_c = 10$ ksi and $f'_{ci} = 8$ ksi (Case 3), yields a 10.2 percent increase in span length with a maximum span of 130 ft. The increase in span length would have to be compared to the increased cost for the higher strength concrete. In spite of room for additional pretensioning strands, any additional strands resulted in no significant reduction in the tensile stress experienced by the girder resulting in the specified maximum span.

4.1.3   **Lightweight Concrete (*Cases 4, 5 and 6*)**

The utilization of light weight concrete for the beam, deck, or both provided an increase of maximum span because of the 30 pcf reduction in concrete unit weight. The use of lightweight concrete in the deck proved to be a better option than using lightweight concrete in the beam. The lightweight beam (Case 5) had a span increased of only 0.8 percent (1 ft) while the lightweight deck (Case 4) provided an additional 3.2 percent of span (4 ft).

The best lightweight variation consisted of both lightweight deck and beam (Case 6). The maximum span was increased an additional 2 ft beyond that obtained using the lightweight deck (6 ft total). The number of strands remained consistent between all three cases. These cases experienced the same over stressing mode as the Base Design.

4.1.4   **Modifications to Girder Cross-Section (*Cases 7, 8 and 13*)**

Physical modification of the PCI BT-72 girder dimensions resulted in span increases of 5.1 to 6.8 percent. Increasing the web width of the girder (Case 8) increases the girder section properties , which results in an increase of 6 ft in the maximum span length. This result is comparable with the lightweight concrete cases.

Adding 2 in. of depth to the bottom flange (Case 7) allowed for an additional row of strands along the bottom of the girder increasing possible number of strand locations and the eccentricity. This modification resulted in an additional 8 ft in maximum span. Governing stress conditions were the same as experienced by the Base Design. Comparing these two cases of modifying the girder dimensions, the increased bottom flange depth proved to be more effective in increasing maximum span with the least amount of additional strands when compared with the Base Design.

The use of a decked bulb tee (Case 13) increased the span length by 33.1 percent (39 ft) for a total maximum span of 157 ft. This alternative produced the greatest increase in maximum span. However, the large weight of the section introduces significant issues for handling and transportation. The stress conditions at the girder ends at release governed design despite debonding and draping.

4.1.5   **Stress Control with Respect to Strand Layout (*Cases 9 and 10*)**

Bundling strands at hold-down locations (Case 9) increases the eccentricity of the strand pattern and decreased the number of required strands. However, this case yielded no increase in span length. The same stress conditions controlled as for the Base Design.

Placing four debonded strand in the top flange of the girder (Case 10), which were detensioned at the center of the span after girder erection, yielded a 3.4 percent increase of maximum span length. The governing stress conditions were the same as for the Base Design.

4.1.6   **Variation in Strand Size (*Cases 11 and 12*)**

Using 0.6 in. diameter strands instead of ½ in. diameter strands (Case 11) resulted in no increase in span length using the base concrete strength. The only advantage for this design is that the fabricator would have to handle fewer strands. The governing stress conditions were the same as the Base Design.

The combination of high strength concrete and 0.6 in. diameter strands (Case 12) proved to be a much more attractive design variation. The maximum span was increased to 138 ft with 54 strands. The governing stress conditions were the same as for the Base Design.

The effect of combining the 0.6 in. diameter strands and high strength concrete results in a 16.9 percent increase in maximum span, which is a much greater increase than for either the strand size (0 percent) or concrete strength (6.8 percent) increase individually. This illustrates that the combination of strategies for increasing span ranges may be significantly different from the effect of individual strategies. Therefore, the designer should carefully consider the combination of different options where they are available.

4.1.7   **Touch Shoring (*Case 14*)**

Providing temporary support at midspan using touch shoring proved to be effective for the PCI BT 72 girder. This design case provided a 10.2 percent increase in maximum span length, to a span of 130 ft, which is the same as the case with increased concrete strength at release (Case 2). The governing stress conditions were the same as for the Base Design.

4.1.8   **Post-Tensioning (*Case 15*)**

The utilization of both pretensioning and post-tensioning increased the maximum span by 24.6 percent (29 ft). This was the second most successful strategy for increasing span length. Stresses at midspan under Service I conditions at the top of the girder governed.

4.1.9   **General Conclusions and Comments**

The greatest increase in maximum span length was obtained by casting the deck with the girder (decked bulb tee – Case 13) and by adding post-tensioning to a pretensioned girder (Case 15), with increases in maximum spans of 33.1 and 24.6 percent, respectively. These design variations are out of the ordinary and require significant additional effort in design and construction. However, they have been used successfully in some situations and regions.

The next most effective strategy for increasing the maximum span length was the combination of increased strand size with high strength concrete (Case 12), with an increase in the maximum span of 16.9 percent. The use of this strategy has become common and has served to significantly increase span ranges for precast concrete girders. Other design comparisons achieved increases in maximum spans from 0 to 10 percent.

A significant finding was the increase shown in Case 12, where two strategies were combined to produce a much higher increase in maximum span than either strategy alone. While other combinations of strategies were not investigated in this study, it appears possible that the combination of different strategies could provide impressive additional gains in maximum spans. Therefore, designers are encouraged to consider combinations of strategies.

A final comment addresses the limitations of this study. These results are based on maximum span designs. The use of maximum spans results in designs that are governed by several stress conditions simultaneously. For most designs, only one design condition governs, or steps may be taken to prevent other conditions from governing (such as draping strands). Furthermore, this study is limited in scope, considering only one bridge cross-section. Therefore, the results from these comparisons, while instructive, may not reflect the potential for bridges where the parameters differ and the design span is not at the maximum span. Each situation must be considered individually.

4.2   **DESIGN CASES FOR DIFFERENT GIRDER TYPES**

The results of the comparison of different girder types are shown in **Table 4.2-1**, which include maximum spans, span increase, and percent increase in maximum span over the Base Design for each girder type, and over the same design case for the PCI BT-72 girder. Results for the PCI BT-72 girder design cases are identical to those presented in the preceding section.

Table 4.2-1   **Summary of Results for Different Girder Types**

Girder Type and Design Case		Maximum Span	Span Increase for Each Girder Type	Percent Increase for Each Girder Type	Span Increase Compared to PCI BT-72 Same Case	Percent Increase Compared to PCI BT-72 Same Case
No.	Description	(ft)	(ft)	(%)	(ft)	(%)
PCI BT-72 Girder						
1	Base Design	118	----	----	----	----
3	High Strength Conc. (HSC)	130	12	10.2	----	----
11	0.6" ϕ Strands	118	0	0.0	----	----
12	0.6" ϕ Strands and HSC	138	20	16.9	----	----
NEBT 1800 Girder						
1	Base Design	135	----	----	17	14.4
3	High Strength Conc. (HSC)	141	6	4.4	11	8.5
11	0.6" ϕ Strands	135	0	0.0	17	14.4
12	0.6" ϕ Strands and HSC	155	20	14.8	17	12.3
AASHTO Type VI Girder						
1	Base Design	137	----	----	19	16.1
3	High Strength Conc. (HSC)	151	14	10.2	21	16.2
11	0.6" ϕ Strands	138	1	0.7	20	16.9
12	0.6" ϕ Strands and HSC	159	22	16.1	21	15.2

4.2.1   **Results for Different Girder Types**

The maximum spans for each design case, grouped by girder type, are presented graphically in **Figure 4.2.1-1** and the percent increase in maximum spans compared to the Base Design for each girder type are shown in **Figure 4.2.1-2**.

Figure 4.2.1-1   Maximum Spans for Different Girder Types

Figure 4.2.1-2   Percent Increase in Maximum Spans for Different Girder Types

The increase in maximum span for the three design variations over the Base Design for each girder type indicates a similar relationship for the different girder types. The trends can be summarized as follows:

- High strength concrete increased the maximum span length. The span increase ranged from 4.4 percent to 10.2 percent with both the PCI BT-72 and AASHTO Type VI girders experiencing the same increase of 10.2 percent.

- The use of 0.6 in. diameter strands with normal strength concrete was not effective for increasing maximum span for these girders for the bridge cross-section considered.

- The combination of both high strength concrete and 0.6 in. diameter strands was most effective for increasing the maximum span for all girder types. The span increase ranged from 14.8 percent to 16.9 percent. The PCI BT-72 girder showed the greatest increase in spite of the fact that it has the smallest bottom flange, limiting the possible number of strand locations.

4.2.2   **Comparisons of Different Girder Types to the PCI BT-72 Girder**

The maximum spans for each design case, grouped by design case, are presented graphically in **Figure 4.2.2-1** and the percent increase in maximum spans compared to the Base Design for each girder type are shown in **Figure 4.2.2-2**.
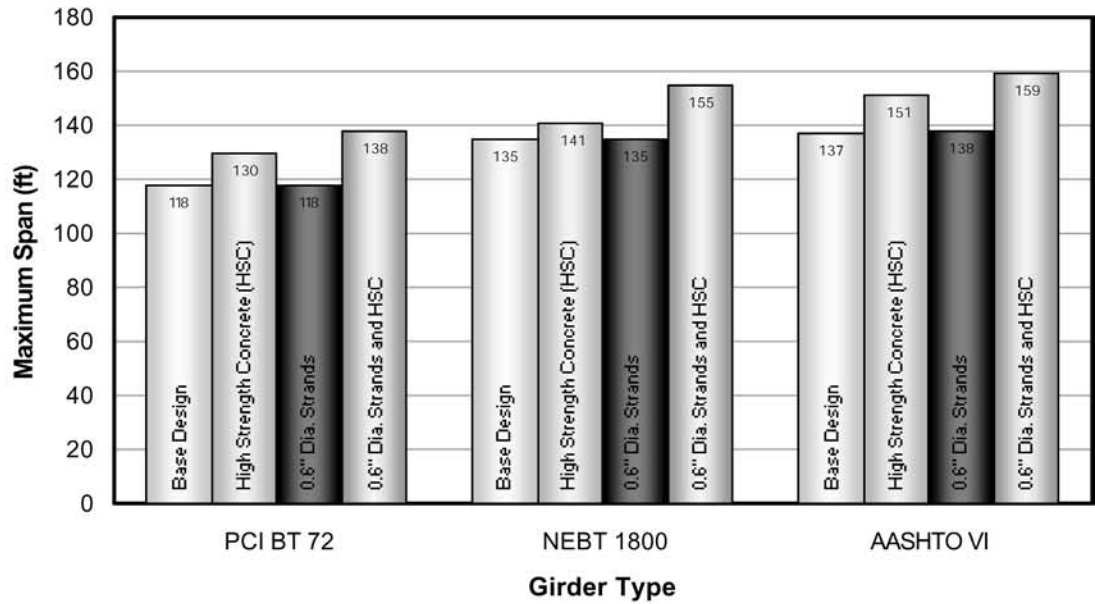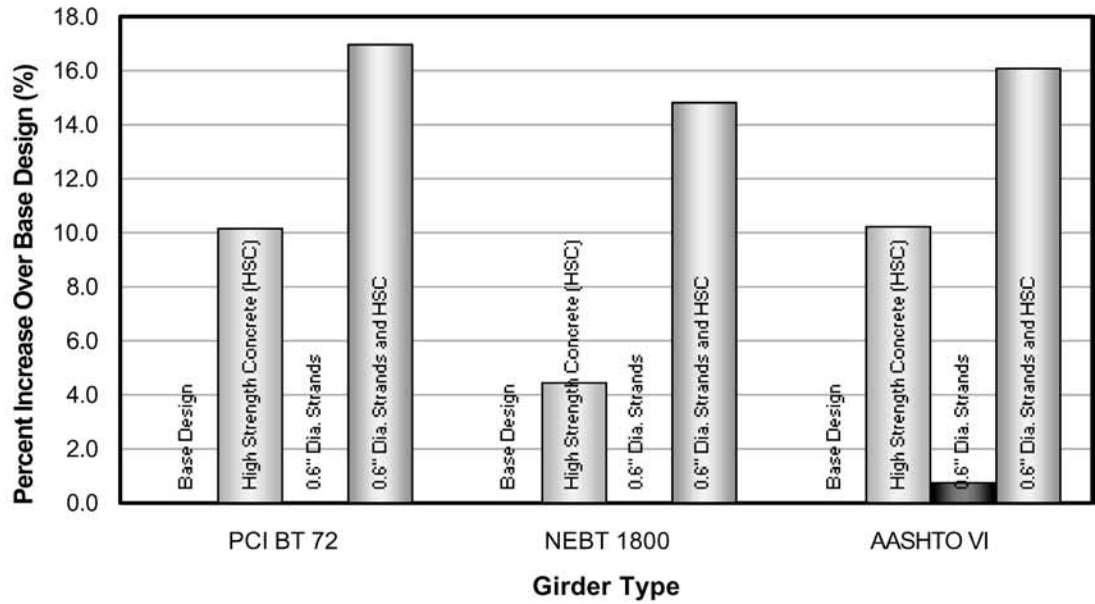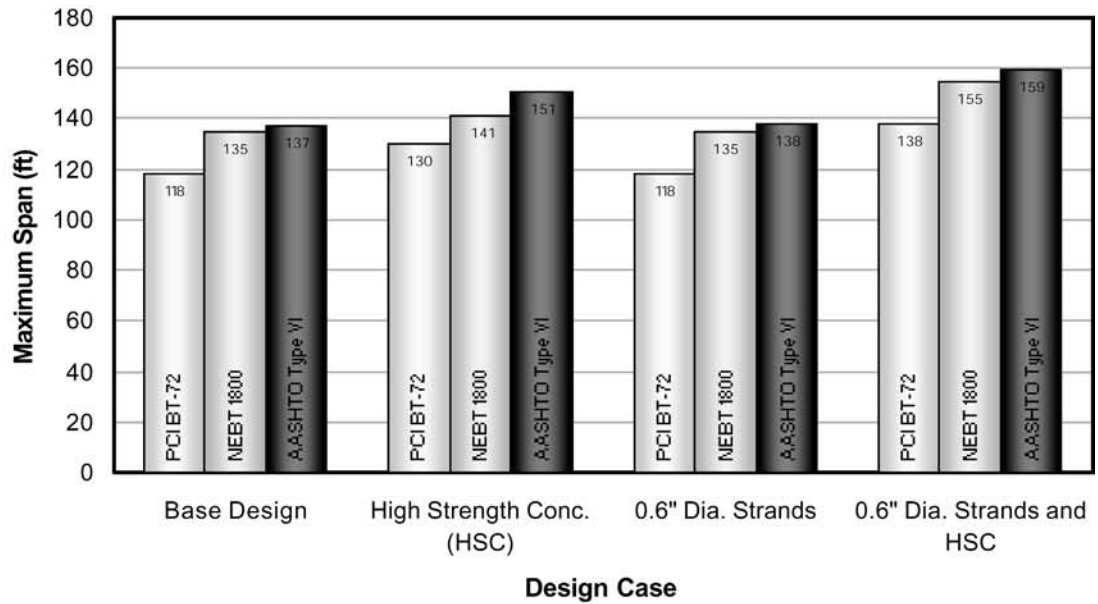

Figure 4.2.2-1   Maximum Spans for Different Girder Types for Each Design Case

The increase in maximum span for the three design variations over the Base Design for each girder type indicates a similar relationship for the different girder types. The trends can be summarized as follows:

- The NEBT 1800 and AASHTO Type VI girders have a greater maximum span for all design cases investigated.

- The NEBT 1800 and AASHTO Type VI girders have maximum spans 17 ft and 19 ft larger, respectively, than the Base Design for the PCI BT-72 girder.

- The use of high strength concrete and 0.6 in. diameter strands increased the maximum spans of the NEBT 1800 and AASHTO Type VI girders by 20 and 22 ft, respectively,

over the Base Design for the PCI BT-72 girder. This strategy for increasing span range is clearly most effective of those considered.

Figure 4.2.2-2   Percent Increase in Maximum Spans for Different Girder Types Compared to PCI BT-72 Girder

4.2.3   **General Conclusions and Comments**

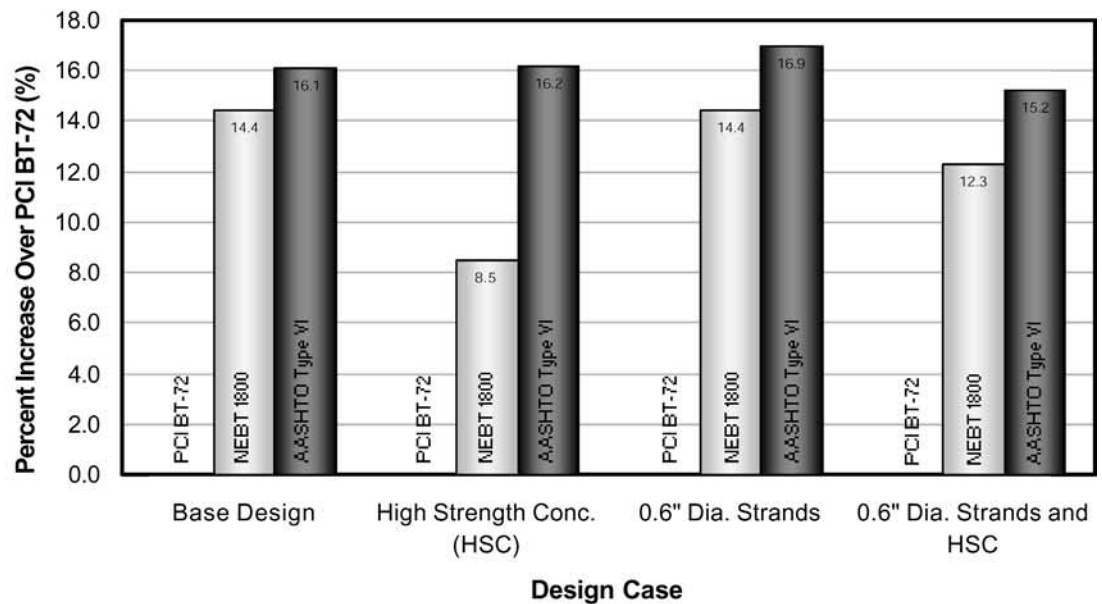The comparison of designs for different girder types indicates that the AASHTO Type VI girder has the largest maximum span and the PCI BT-72 girder had the smallest maximum span for all design cases. The NEBT 1800 girder did not experience as great an increase in maximum spans for any of the design cases considered when compared to the other girders.

These conclusions tend to indicate that the AASHTO Type VI girder would provide the most efficient designs since it had the longest maximum spans. However, the Type VI girder is a much larger cross-section. Therefore, more information must be considered when comparing the performance of different girder sections to determine the most efficient and economical designs.

5   DESIGN ASSUMPTIONS

The following basic assumptions are used for the design of this bridge. Additional discussion of some items is provided later in the design example.

1. The roadway section is set as shown in the preceding figures.

2. No permanent intermediate diaphragms are used. Temporary bracing is required along the girders for stability during construction.

3. A standard composite reinforced concrete deck is used. The deck is 8¼" thick, including ¼" that is considered to be an integral wearing surface. A 3 in. build-up is assumed to accommodate construction tolerances and variation in camber. However, the build-up is neglected when computing composite section properties.

4. Girders are set on standard elastomeric bearings at the abutments

5. All girders are a single unit with no splicing.

6. The strand pattern is completely filled from the bottom in most cases. In a few cases, strands are removed from bottom to achieve the maximum span.

7. When post-tensioning is used, tendons are placed in a single vertical plane. Post-tensioning ducts are grouted after tendons are stressed to provide corrosion protection and to improve performance by providing bond between the strands and concrete.

6   BASIC DESIGN INFORMATION

6.1   GOVERNING SPECIFICATIONS

AASHTO LRFD Bridge Design Specifications, 2nd Edition with Interims through 2002. References to articles, equations and tables in these *specifications* will be preceded by the prefix "LRFD" to differentiate these references from references to items in this design example.

6.2   LOADS

6.2.1   Live Load

The design live load is HL-93 with 33% dynamic allowance (IM) on the design truck, as specified in LRFD Article 3.6.1.2.

6.2.1.1   **Live Load Distribution Factors for PCI BT-72 Girder**

Live load distribution factors for moment and shear computed for each design case are shown in **Table 6.2.1.1-1**. Design parameters for this girder fall within the limits for the range of applicability for these equations.

Table 6.2.1.1-1   Live Load Distribution Factors for PCI BT-72 Girder

Case	Design Case	DFM One Lane Loaded	DFM Two or More Lanes Loaded	DFS One Lane Loaded	DFS Two or More Lanes Loaded
1	Base Design	0.458	0.663	0.681	0.825
2	Increased f'_{ci}	0.447	0.651	0.681	0.825
3	High Strength Conc. (HSC)	0.451	0.658	0.681	0.825
4	Lightweight Conc. Deck	0.467	0.677	0.681	0.825
5	Lightweight Conc. Beam	0.443	0.642	0.681	0.825
6	Lightweight Conc. Dk. & Bm.	0.450	0.654	0.681	0.825
7	Deeper Bottom Flange	0.454	0.661	0.681	0.825
8	Increased Beam Width	0.456	0.663	0.681	0.825
9	Bundled Strands	0.458	0.663	0.681	0.825
10	Top Strands Debonded	0.452	0.657	0.681	0.825
11	0.6"φ Strands	0.458	0.663	0.681	0.825
12	0.6"φ Strands and HSC	0.441	0.648	0.681	0.825
13	Decked Bulb Tee	----	0.602	---	0.814
14	Touch Shoring	0.442	0.646	0.681	0.825
15	Added Post-Tensioning	0.427	0.630	0.681	0.825

Shaded cells indicate values that differ from Base Design.

6.2.1.2   **Live Load Distribution Factors for Different Girder Types**

Live load distribution factors for moment and shear for each design case are shown in **Table 6.2.1.2-1**. Design parameters for these girders fall within the limits for the range of applicability for these equations.

Table 6.2.1.2-1   Live Load Distribution Factors for Different Girder Types

Girder Type and Design Case		DFM One Lane Loaded	DFM Two or More Lanes Loaded	DFS One Lane Loaded	DFS Two or More Lanes Loaded
PCI BT-72 Girder					
1	Base Design	1.458	0.663	0.681	0.825
3	High Strength Conc. (HSC)	0.451	0.658	0.681	0.825
11	0.6" φ Strands	0.458	0.663	0.681	0.825
12	0.6" φ Strands and HSC	0.451	0.658	0.681	0.825
NEBT 1800 Girder					
1	Base Design	0.447	0.655	0.681	0.825
3	High Strength Conc. (HSC)	0.449	0.660	0.681	0.825
11	0.6" φ Strands	0.447	0.655	0.681	0.825
12	0.6" φ Strands and HSC	0.449	0.660	0.681	0.825
AASHTO Type VI Girder					
1	Base Design	0.447	0.656	0.681	0.825
3	High Strength Conc. (HSC)	0.440	0.651	0.681	0.825
11	0.6" φ Strands	0.447	0.656	0.681	0.825
12	0.6" φ Strands and HSC	0.449	0.651	0.681	0.825

6.2.2   **Noncomposite Dead Loads**

6.2.2.1   **Girder**

A summary of girder dead loads is provided in **Tables 6.2.2.5-1** and **6.2.2.5-2,** reflecting the variation in the unit weight of girder concrete and the cross-sectional area of the precast girders for the different design cases and girders evaluated.

LRFD Article 5.14.1.2.1 specifies that an allowance for impact during handling and transportation equal to 50% of the girder self-weight should be used for design of the girders. However, in this comparative study, the girder is not evaluated for the increased load for handling and transportation since this does not govern conventional designs. The girders are only evaluated for service and release conditions to determine maximum span.

6.2.2.2   **Deck**

The girder spacing of 8 ft (96 in.) and the total thickness of the deck of 8¼ in. are held constant for all designs. Since the unit weight of the deck concrete varies, the total deck dead load for an interior girder has been computed for each case as summarized in **Tables 6.2.2.5-1** and **6.2.2.5-2** for each design case and girder type. The load on an exterior girder would have to be adjusted for the overhang width and the overhang taper.

6.2.2.3   **3 in. Build-Up**

The full build-up thickness of 3 in. over the width of the girder top flanges, considering the unit weight of the deck concrete, is used for dead load computations. This applies to both interior and exterior girders. A build-up is not used for the decked bulb tee design (Case 13).

6.2.2.4 | **SIP Deck Forms**

The stay-in-place (SIP) deck forming system contributes an additional load of 0.016 ksf on the formed area between girders, resulting in a load on an interior girder as indicated in **Tables 6.2.2.5-1** and **6.2.2.5-2**. The load on an exterior girder would be one half of the load on an interior girder.

6.2.2.5 | **Total Noncomposite (NC) Dead Load**

The total noncomposite dead load for an interior girder is indicated in **Table 6.2.2.5-1** and **6.2.2.5-2**.

Table 6.2.2.5-1   Noncomposite Dead Loads for PCI BT-72 Girder

No.	Design Case – Description	Girder Dead Load (klf)	Build-Up Dead Load (klf)	Deck Dead Load (klf)	SIP Dead Load (klf)	Total NC Dead Load (klf)
1	Base Design	0.799	0.131	0.825	0.072	1.827
2	Increased f'_{ci}	0.799	0.131	0.825	0.072	1.827
3	High Strength Conc. (HSC)	0.799	0.131	0.825	0.072	1.827
4	Lightweight Conc. Deck	0.799	0.105	0.660	0.072	1.636
5	Lightweight Conc. Beam	0.639	0.131	0.825	0.072	1.667
6	Lightweight Conc. Dk. & Bm.	0.639	0.105	0.660	0.072	1.476
7	Deeper Bottom Flange	0.853	0.131	0.825	0.072	1.881
8	Increased Beam Width	0.949	0.138	0.825	0.069	1.981
9	Bundled Strands	0.799	0.131	0.825	0.072	1.827
10	Top Strands Debonded	0.799	0.131	0.825	0.072	1.827
11	0.6"ϕ Strands	0.799	0.131	0.825	0.072	1.827
12	0.6"ϕ Strands and HSC	0.799	0.131	0.825	0.072	1.827
13	Decked Bulb Tee	1.599	---	---	---	1.599
14	Touch Shoring	0.799	0.131	0.825	0.072	1.827
15	Added Post-Tensioning	0.949	0.138	0.825	0.069	1.981

Shaded cells indicate values that differ from Base Design.

Table 6.2.2.5-2   Noncomposite Dead Loads for Different Girders

Girder Type	Girder Dead Load (klf)	Build-Up Dead Load (klf)	Deck Dead Load (klf)	SIP Dead Load (klf)	Total Noncomposite Dead Load (klf)
PCI BT-72	0.799	0.131	0.825	0.072	1.827
NEBT 1800	1.009	0.148	0.825	0.065	2.047
AASHTO Type VI	1.130	0.131	0.825	0.072	2.158

6.2.3   **Composite Dead Loads**

Dead loads placed on the composite girder are distributed equally to the six girders in the cross-section (LRFD Art. 4.6.2.2.1).

6.2.3.1   **Barrier Parapet**

Each barrier parapet is assumed to contribute 0.371 klf, or 0.742 klf for both parapets. Therefore the load per girder is

$$w = 0.742 / 6 = 0.124 \text{ klf}$$

6.2.3.2   **Future Wearing Surface**

The future wearing surface is assumed to contribute 0.025 ksf over the roadway width of 44 ft. Therefore the load per girder is

$$w = 0.025 (44) / 6 = 0.183 \text{ klf}$$

6.3   **MATERIALS AND MATERIAL PROPERTIES**

Material properties used for design are given below.

6.3.1   **Concrete**

6.3.1.1   **Basic Properties**

Basic properties for both girder and deck concrete are listed for the different design cases in **Table 6.3.1.1-1**.

Table 6.3.1.1-1   Basic Concrete Properties

Variation		f'_{ci}	f'_c	f_r	w_c	w_{cd}	E_{ci}	E_c	E_{cd}
No.	Description	(ksi)	(ksi)	(ksi)	(kcf)	(kcf)	(ksi)	(ksi)	(ksi)
1	Base Design	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
2	Increased f'_{ci}	6.0	6.5	0.612	0.150	0.150	4,696	4,888	4,067
3	High Strength Conc. (HSC)	8.0	10.0	0.759	0.150	0.150	5,423	6,063	4,067
4	Lightweight Conc. Deck	5.0	6.5	0.433	0.150	0.120	4,287	4,888	2,910
5	Lightweight Conc. Beam	5.0	6.5	0.433	0.120	0,150	3,067	3,497	4,067
6	Lightweight Conc. Dk. & Bm.	5.0	6.5	0.433	0.120	0.120	3,067	3,497	2,910
7	Deeper Bottom Flange	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
8	Increased Beam Width	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
9	Bundled Strands	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
10	Top Strands Debonded	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
11	0.6"ϕ Strands	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
12	0.6"ϕ Strands and HSC	8.0	10.0	0.759	0.150	0.150	5,423	6,063	4,067
13	Decked Bulb Tee	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,888
14	Touch Shoring	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067
15	Added Post-Tensioning	5.0	6.5	0.612	0.150	0.150	4,287	4,888	4,067

Shaded cells indicate values that differ from Base Design.

f'_{cd} = 4.5 ksi except for Decked Bulb Tee (Case 13), which used $f'_{cd} = f'_c$ = 6.5 ksi.

6.3.1.2 **Time-Dependent Properties**

Since simplified loss computations are used for this design (LRFD Art. 5.9.5.4.1), it is not necessary to compute specific time-dependent properties for the girder or deck concrete.

6.3.2 **Pretensioning Strands**

The girders are pretensioned with either ½ in. diameter low relaxation seven-wire strands or 0.6 in. diameter low relaxation seven-seven wire strands.

A_{ps} = 0.153 in^2 (For ½ in. diameter strands)

A_{ps} = 0.217 in^2 (For 0.6 in. diameter strands)

f_{pu} = 270 ksi

f_{py} = 0.90 f_{pu} = 243 ksi

f_{pj} = 0.75 f_{pu} = 202.5 ksi

E_p = 28,500 ksi

6.3.3 **Post-Tensioning Materials**

6.3.3.1 **Post-Tensioning Strands**

The girders are post-tensioned with ½ in. diameter low relaxation seven-wire strands.

$$A_{ps} = 0.153 \text{ in}^2$$
$$f_{pu} = 270 \text{ ksi}$$
$$f_{py} = 0.90 \, f_{pu} = 243 \text{ ksi}$$
$$f_{pj} = 0.75 \, f_{pu} = 202.5 \text{ ksi}$$
$$E_p = 28,500 \text{ ksi}$$

6.3.3.2   **Post-Tensioning Ducts**

The tendons selected for this project are composed of 19 - ½ in. diameter strands.

LRFD Article 5.4.6.2 requires that the size of the duct shall not be greater than 0.4 times the least gross concrete thickness at the location of the duct. Therefore, the maximum duct diameter for an 8 in. web is

$$D_{max} = 0.4 \, b_w = 0.4(8) = 3.2 \text{ in.}$$

The same article requires that the area of the duct be at least 2.0 or 2.5 times the net area of the prestressing strands, depending on how the strands are installed. Using the larger factor to be conservative, the minimum duct diameter for the 19 - ½ in. diameter strands is

$$D_{min} = \sqrt{[4(2.5 \, A_{ps}) / \pi]} = \sqrt{[4(2.5 \, (19(0.153))) / \pi]} = 3.04 \text{ in.}$$

The standard diameter duct used with 19 - ½ in. strands is 85 mm or 3.35 in. This violates the maximum diameter. A reduced size duct may also be used, which has a diameter of 80 mm or 3.15 in. This size duct satisfies both the maximum and minimum criteria, and will be used in the example. With the slightly smaller diameter, extra care must be taken by the fabricator and contractor to ensure proper alignment of the ducts in the girders and proper placement of the girders when erected.

6.3.3.3   **Post-Tensioning Anchorages**

Details of post-tensioning anchorages are not considered or presented in this comparative study. The details vary with the supplier of the material. However, a supplier was consulted to obtain reasonable parameters and dimensions for the post-tensioning.

6.3.3.4   **Grout**

The type and strength of grout do not affect the design and will therefore not be specified here.

Ducts must be grouted within a specified period following stressing.

6.3.4   **Mild Reinforcement**

$$f_y = 60 \text{ ksi}$$
$$E_s = 29,000 \text{ ksi}$$

6.4   **SECTION PROPERTIES**

6.4.1   **PCI BT-72 Girder Design Cases**

6.4.1.1   **Noncomposite Section (Girder only)**

The base cross-section considered was a standard PCI BT-72 girder. Two variations of the standard girder were also considered and are shown below in **Figures 6.4.1.1-1** and **6.4.1.1-2**.

The standard PCI bulb tee section has a 6 in. web, which cannot accommodate the duct required for the post-tensioning design case. Therefore, the girder width is increased by 2 in.

by spreading the side forms 2 in. This results in an 8 in. web, as shown in **Figure 6.4.1.1-1** for the increased width, and post-tensioning design cases, which are cases 8 and 13, respectively.

For one design case, the depth of the bottom flange of the PCI BT-72 girder was increased to 8 in. from the standard 6 in. This makes possible the use of an additional row of 12 pretensioning strands.

The third variation consists of a decked BT-72 that is shown in **Figure 6.4.1.1-2**. The deck in this section is cast with the girder, so the entire section is present at release. Girders are then erected side-by-side resulting in a complete deck without any field placed concrete.
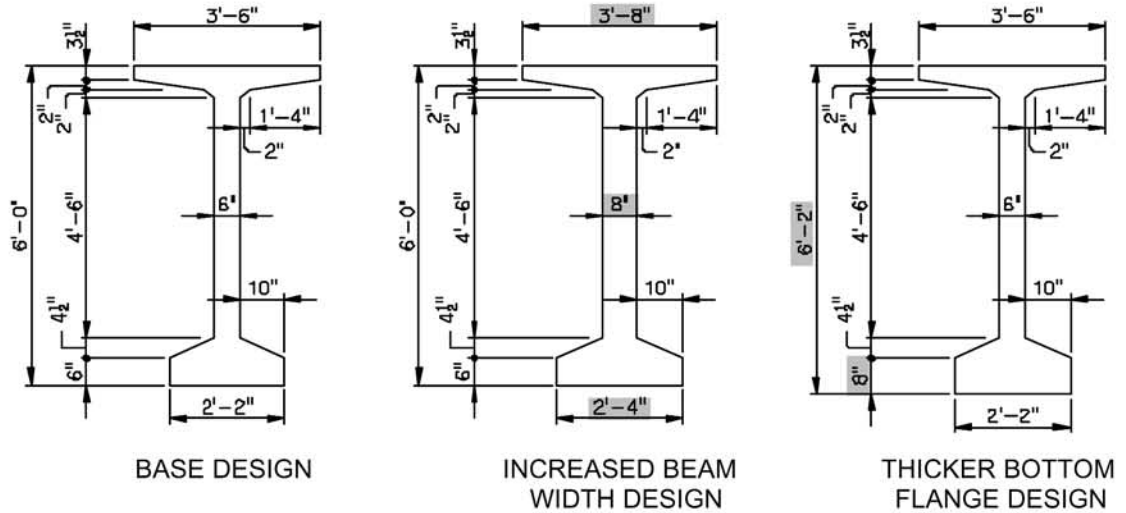

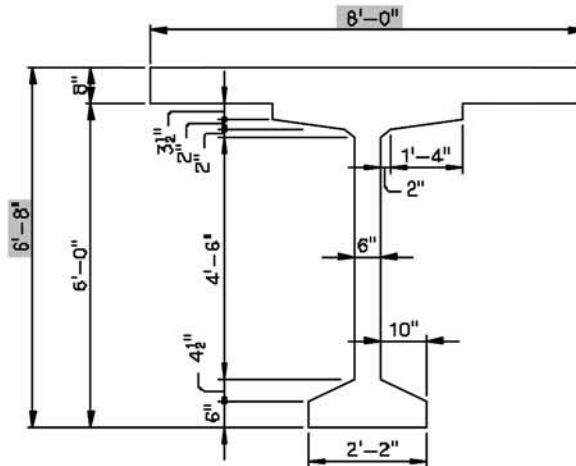**Figure 6.4.1.1-1 Girder Dimensions for Undecked PCI BT-72 Variations**

Figure 6.4.1.1-2 Girder Dimensions for Decked Bulb Tee

The noncomposite (bare girder) section properties for the design cases are shown in **Table 6.4.1.1-1**.

Table 6.4.1.1-1 Noncomposite Section Properties

	Design Case	h	y_b	A	I	Web Width	Top Flange Width
No.	Description	(in.)	(in.)	(in.^{2})	(in.^{4})	(in.)	(in.)
1	Base Design	72.0	36.60	767	545,857	6.0	42.0
2	Increased f'_{ci}	72.0	36.60	767	545,857	6.0	42.0
3	High Strength Conc. (HSC)	72.0	36.60	767	545,857	6.0	42.0
4	Lightweight Conc. Deck	72.0	36.60	767	545,857	6.0	42.0
5	Lightweight Conc. Beam	72.0	36.60	767	545,857	6.0	42.0
6	Lightweight Conc. Dk. & Bm.	72.0	36.60	767	545,857	6.0	42.0
7	Deeper Bottom Flange	74.0	36.22	819	614,736	6.0	42.0
8	Increased Beam Width	72.0	36.50	911	608,109	8.0	44.0
9	Bundled Strands	72.0	36.60	767	545,857	6.0	42.0
10	Top Strands Debonded	72.0	36.60	767	545,857	6.0	42.0
11	0.6"ϕ Strands	72.0	36.60	767	545,857	6.0	42.0
12	0.6"ϕ Strands and HSC	72.0	36.60	767	545,857	6.0	42.0
13	Decked Bulb Tee	80.0*	56.31	1,535	1,145,550	6.0	42.0*
14	Touch Shoring	72.0	36.60	767	545,857	6.0	42.0
15	Added Post-Tensioning	72.0	36.50	911	608,109	8.0	44.0

Shaded cells indicate values that differ from Base Design.

* Since deck is cast with girder, girder height includes 8 in. structural deck thickness. Top flange width shown is only for the girder portion of the cross-section. Total top flange width is 96 in.

6.4.1.2 **Composite Section (Girder with Deck Slab)**

Table 6.4.1.2-1   **Composite Section Properties**

Design Case		h_c	y_{bc}	A_c	I_c	n
No.	Description	(in.)	(in.)	(in^2)	(in^4)	
1	Base Design	80.0	54.51	1,406	1,090,404	0.832
2	Increased f'_{ci}	80.0	54.51	1,406	1,090,404	0.832
3	High Strength Conc. (HSC)	80.0	52.43	1,282	1,027,018	0.671
4	Lightweight Conc. Deck	80.0	51.32	1,224	992,995	0.595
5	Lightweight Conc. Beam	80.0	57.80	1,660	1,191,154	1.163
6	Lightweight Conc. Dk. & Bm.	80.0	54.51	1,406	1,090,403	0.832
7	Deeper Bottom Flange	82.0	54.53	1,458	1,244,835	0.832
8	Increased Beam Width	80.0	52.78	1,550	1,197,504	0.832
9	Bundled Strands	80.0	54.51	1,406	1,090,404	0.832
10	Top Strands Debonded	80.0	54.51	1,406	1,090,404	0.832
11	0.6"ϕ Strands	80.0	54.51	1,406	1,090,404	0.832
12	0.6"ϕ Strands and HSC	80.0	52.43	1,282	1,027,018	0.671
13	Decked Bulb Tee *	80.0	56.31	1,535	1,145,550	-----
14	Touch Shoring	80.0	54.51	1,406	1,090,404	0.832
15	Added Post-Tensioning	80.0	52.78	1,550	1,197,504	0.832

Shaded cells indicate values that differ from Base Design.

* Since deck is cast with girder, there are no composite section properties for this girder. Values shown are for the noncomposite girder.

6.4.2 **Different Girder Types**

6.4.2.1 **Noncomposite Section (Girder only)**

Section properties and general girder dimensions are indicated in **Table 6.4.2.1-1** and illustrated in **Figure 6.4.2.1-1**.

Table 6.4.2.1-1   **Noncomposite Section Properties for Different Girder Types**

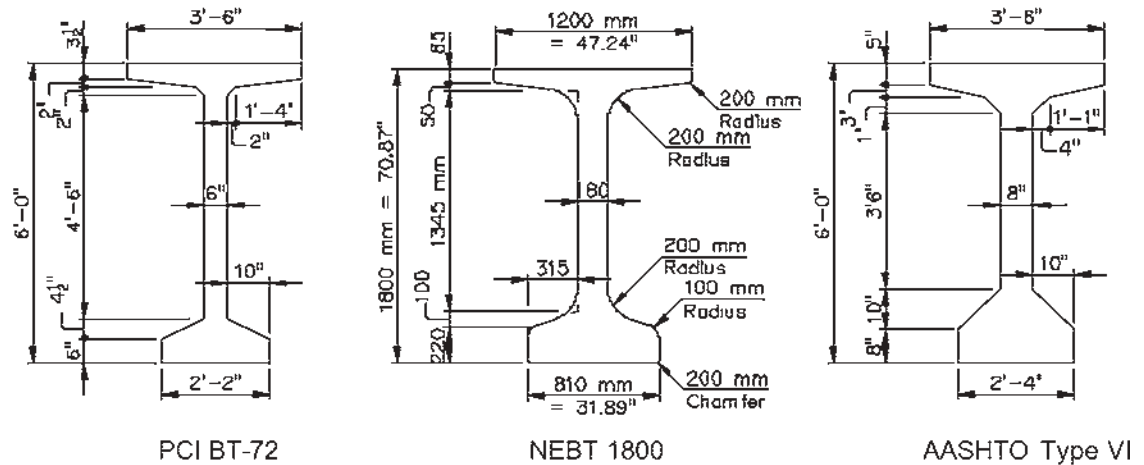| Girder Type	h	y_b	A	I	Web Width	Bottom Flange Width	Top Flange Width
	(in.)	(in.)	(in^2)	(in^4)	(in.)	(in.)	(in.)
PCI BT-72	72.00	36.60	767	545,857	6.0	26.00	42.00
NEBT 1800	70.87	33.66	969	660,449	7.1	31.89	47.20
AASHTO Type VI	72.00	36.38	1,085	733,319	8.0	28.00	42.00

Figure 6.4.2.1-1   Girder Dimensions for Different Girder Types

6.4.2.2   **Composite Section (Girder with Deck Slab)**

Table 6.4.2.2-1   Composite Section Properties for Different Girder Types

Girder Type and Design Case		h_c (in.)	y_{bc} (in.)	A_c (in.^{2})	I_c (in.^{4})	N
PCI BT-72 Girder						
1	Base Design	80.00	54.51	1,406	1,090,404	0.832
3	High Strength Conc. (HSC)	80.00	52.43	1,282	1,027,018	0.671
11	0.6" ϕ Strands	80.00	54.51	1,406	1,090,404	0.832
12	0.6" ϕ Strands and HSC	80.00	52.43	1,282	1,027,018	0.671
NEBT 1800 Girder						
1	Base Design	78.87	50.04	1,607	1,317,611	0.832
3	High Strength Conc. (HSC)	78.87	47.97	1,484	1,234,252	0.671
11	0.6" ϕ Strands	78.87	50.04	1,607	1,317,611	0.832
12	0.6" ϕ Strands and HSC	78.87	47.97	1,484	1,234,252	0.671
AASHTO Type VI Girder						
1	Base Design	80.00	51.07	1,724	1,368,017	0.832
3	High Strength Conc. (HSC)	80.00	49.14	1,600	1,284,412	0.671
11	0.6" ϕ Strands	80.00	51.07	1,724	1,368,017	0.832
12	0.6" ϕ Strands and HSC	80.00	49.14	1,600	1,284,412	0.671

7   PRESTRESS LAYOUT AND DATA

The number of prestressing strands in the girders was selected to provide enough prestress to maintain concrete stresses within stress limits at release and at service conditions at all locations along the girder, including girder ends, according the specified stress limits given in the *AASHTO LRFD Specification*.

7.1   PCI BT-72 GIRDER DESIGN CASES

7.1.1   Pretensioning Strands

Pretensioning strands are generally added by filling available positions from the bottom of the girder to the top, as shown in **Figure 7.1.1-1**. For some design cases, the maximum span design required that some of the lower strands be removed from the pattern in order to satisfy stress limits in the concrete and achieve the maximum span. All Strand patterns utilize a 2 in. grid as shown.

BASE DESIGN   MODIFIED BOTTOM
                                     FLANGE DESIGN

BUNDLED STRAND
DESIGN
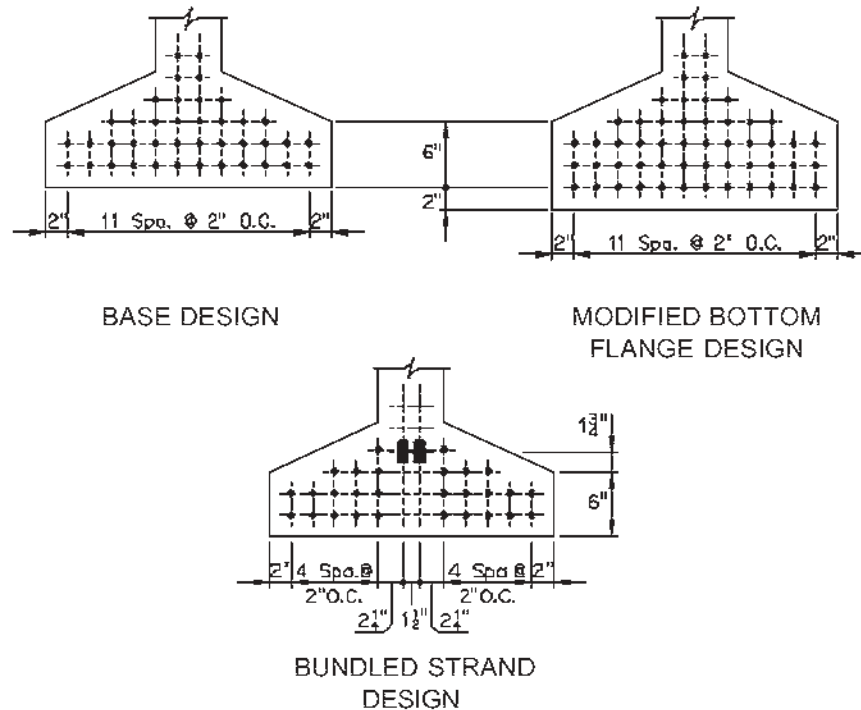
Figure 7.1.1-1   Pretensioning Strand Layouts in Bottom Flange

7.1.2   Post-Tensioning Tendons

The post-tensioning tendons are contained in four 3.15 in. diameter ducts as shown in **Figure 7.1.2-1**. Each contains a maximum of 19 - ½ in. diameter strands. The amount of post-tensioning was selected to provide enough prestress to maintain concrete stresses within stress limits at service. The difference between the center of the ducts and tendons was taken into account as specified in LRFD Art. C5.9.1.6.
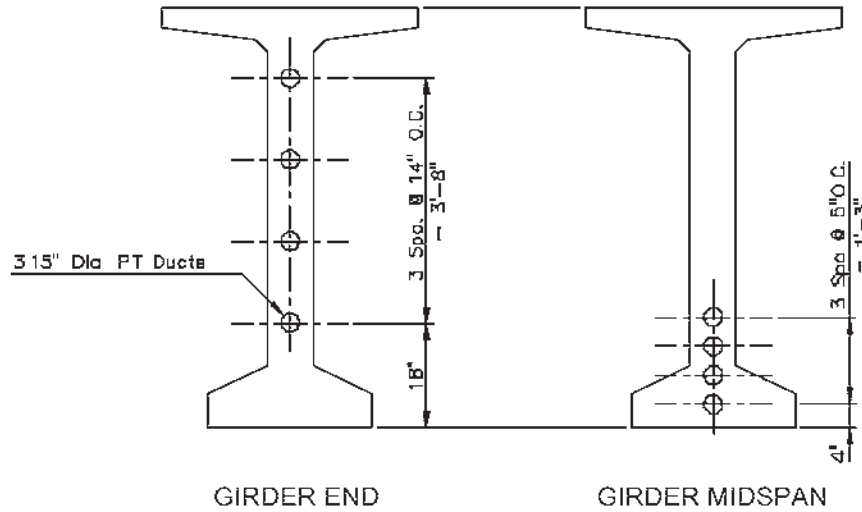
GIRDER END

GIRDER MIDSPAN

Figure 7.1.2-1   Post-Tensioning Duct Locations

7.2

DIFFERENT GIRDER TYPES

Pretensioning strands are added for these sections using the same procedures outlined for the PCI BT-72 girder design cases. Patterns for these girders are shown in **Figure 7.2-1**. Note that the bottom row of strands for the NEBT 1800 girder is located at 70 mm (2.75 in.) from the bottom of the girder which is the standard for this type girder. All strand patterns a 2 in. or 50 mm grid as shown.
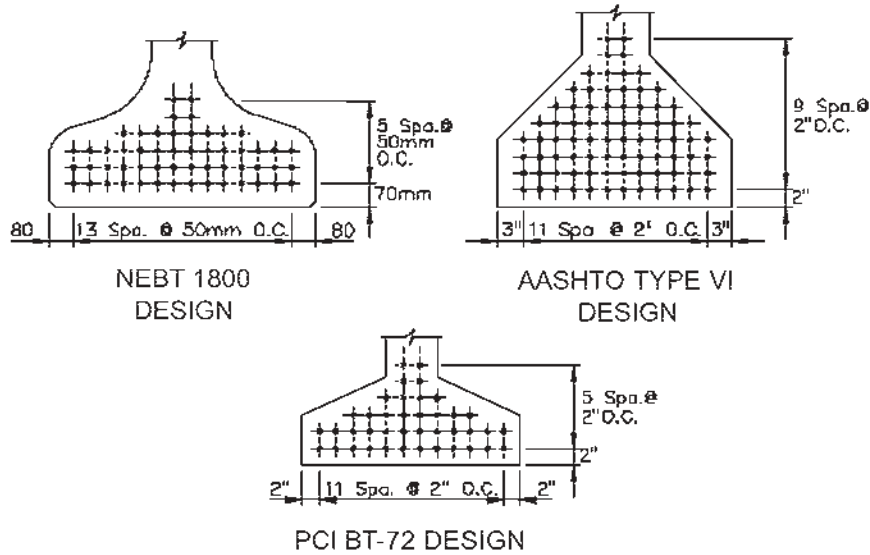

NEBT 1800
DESIGN

AASHTO TYPE VI
DESIGN

PCI BT-72 DESIGN

Figure 7.2-1   Bottom Flange Strand Layouts for Different Girders

8

PRESTRESS LOSSES

The prestress losses are computed using the provisions of the *AASHTO LRFD Specifications*. LRFD Art. 5.9.5.4.1 allows the simplified approach to be used with spans not greater than 250 ft, normal weight concrete, and a minimum concrete strength at prestressing of 3.5 ksi. This design meets all of these criteria except for design Cases 5 and 6 where lightweight concrete is

used for the girders. However, the same loss computation method is used for these cases Calculations are not shown.

9   DESIGN FOR FLEXURE

The design of this bridge for flexure is the focus of the design process. The service limit state requirements for flexure govern the required prestress force and strand layout. After the quantity of prestress has been determined, the flexural strength of the section at the strength limit state was checked at critical sections.

9.1   SERVICE LIMIT STATE

9.1.1   Stress Limits

The following stress limits are used for design at the service limit state. The stress limits at the different stages of construction are discussed in the following sections.

9.1.1.1   Temporary Stresses at Release

These stress limits apply only to the girder concrete.

Compression

f_{cR} $= 0.60\ f'_{ci}$ LRFD Art. 5.9.4.1.1

Tension

f_{tR1} $= 0.0948\ \sqrt{f'_{ci}} \leq 0.2$ ksi   LRFD Table 5.9.4.1.2-1

or

f_{tR2} $= 0.24\ \sqrt{f'_{ci}}$ with reinforcement to resist the tensile force in the concrete

For this comparative study, f_{tR2} was used as the tension stress limit.

9.1.1.2   Final Stresses at Service Limit State after Losses

Stress limits at final conditions are given below for the girder and deck concrete.

Compression

Compressive stress limits for the girder concrete are computed using the same equations provided in the *LRFD AASHTO Specification*, with the appropriate concrete strength:

f_{c1} $= 0.60\ \phi_w f'_c$, for full service loads $(\phi_w = 1)$ LRFD Table 5.9.4.2.1-1

f_{c2} $= 0.45\ f'_c$, for effective prestress (PS) and full dead loads (DL)

f_{c3} $= 0.40\ f'_c$, for live load plus one-half of effective PS and full DL

Tension

Girder Concrete:

For the precompressed compression zone, assuming moderate corrosion conditions:

f_{t1} $= 0.19\ \sqrt{f'_c}$ LRFD Table 5.9.4.2.2-1

Tensile stress limits are not given at final conditions for locations other than the precompressed compression zone because the conditions at release or post-tensioning will govern for tensile stresses in these regions for simple spans. Therefore, tensile stresses will only be checked in the precompressed tensile zone.

9.1.1.3   **Summary of Stress Limits**

The stress limit values for the different types of concrete at each of the stages of construction are given in **Table 9.1.1.3-1**. For other girder types evaluated, the same stress limits per case apply as those indicated for the PCI BT-72 girder shown (Cases 1, 3, 11, and 12).

Table 9.1.1.3-1   Summary of Concrete Stress Limits

Design Cases		Release		Service			
		f_{cR}	f_{tR2}	f_{c1}	f_{c2}	f_{c3}	f_{t1}
No.	Description	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)	(ksi)
1	Base Design	3.000	-0.492	3.900	2.925	2.600	-0.484
2	Increased f'_{ci}	3.600	-0.539	3.900	2.925	2.600	-0.484
3	High Strength Conc. (HSC)	4.800	-0.622	6.000	4.500	4.000	-0.601
4	Lightweight Conc. Deck	3.000	-0.492	3.900	2.925	2.600	-0.484
5	Lightweight Conc. Beam	3.000	-0.492	3.900	2.925	2.600	-0.484
6	Lightweight Conc. Dk. & Bm.	3.000	-0.492	3.900	2.925	2.600	-0.484
7	Deeper Bottom Flange	3.000	-0.492	3.900	2.925	2.600	-0.484
8	Increased Beam Width	3.000	-0.492	3.900	2.925	2.600	-0.484
9	Bundled Strands	3.000	-0.492	3.900	2.925	2.600	-0.484
10	Top Strands Debonded	3.000	-0.492	3.900	2.925	2.600	-0.484
11	0.6"ϕ Strands	3.000	-0.492	3.900	2.925	2.600	-0.484
12	0.6"ϕ Strands and HSC	4.800	-0.622	6.000	4.500	4.000	-0.601
13	Decked Bulb Tee *	3.000	-0.492	3.900	2.925	2.600	-0.484
14	Touch Shoring	3.000	-0.492	3.900	2.925	2.600	-0.484
15	Added Post-Tensioning	3.000	-0.492	3.900	2.925	2.600	-0.484

Shaded cells indicate values that differ from Base Design.

APPENDIX C

ISSUES AND DETAILS FOR EXTENDED SPAN BRIDGES

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APPENDIX C

ISSUES AND DETAILS FOR EXTENDED SPAN BRIDGES

As designers begin to use precast prestressed concrete girders for longer span bridges, they must be aware of issues that need to be considered for the successful application of the girders, especially when spliced girders are used. Therefore, a goal of this research project was the identification of design issues and details that are important for the successful implementation of extended span precast prestressed concrete girders. The findings on this topic are presented in this appendix in two parts. The issues and details presented here focus on spliced girder designs.

Appendix C1   Issues and Details for Extended Span Bridges

This appendix presents a list of various issues and details that need to be considered during the design and construction of extended span girders, with a focus on spliced girders. The section has an introduction to further describe its purpose.

Appendix C2   History of Florida's Spliced Girder Bridges

This appendix presents an overview of the development of spliced girder bridges in the state of Florida, where there are currently more than 30 spliced girder bridges. This information is included to provide an historical perspective on the development of spliced girder construction and design, which can provide insights into current practice.

APPENDIX C1

ISSUES AND DETAILS FOR EXTENDED SPAN BRIDGES

1   INTRODUCTION

This appendix contains a discussion of the many issues and details that need to be considered in the design, fabrication, transportation and construction of long-span precast, prestressed concrete bridge girders. A section will be devoted to each of these four categories.

While each item in the list has been categorized, one item may appear in more than one list, or an item may also need to be considered in another area. Therefore, the reader is encouraged to review all items, not just those that appear in the category that represents his role in the project. For example, the amount of pretensioning in a segment is discussed in the fabrication section because it affects on handling, but must also be considered during the design and detailing of the girder.

The list is lengthy yet should not be considered exhaustive. The focus of the list is on post-tensioned spliced girder bridges because there are so many important issues that must be considered when designing such structures.

Where appropriate, references are given for other documents that provide additional information on a subject. Information is not repeated here when it can be found in references.

For most of the topics discussed in this appendix, there may be several different solutions that could be used successfully for a given bridge, while for some other bridge, one solution may have distinct advantages that make it the preferred solution. Furthermore, a detail or option that works well for one bridge may not work well for another. Therefore, designers must avoid using details or methods from this list or another project without evaluating its suitability for use on the specific project being designed. As a result, a single recommendation is not usually given for topics discussed in this appendix; instead, several options are presented for most issues, with a discussion of features of each option.

2   Design

The following should be considered where applicable during the design of spliced girder and other long-span, post-tensioned concrete girder bridges.

2.1   Girder Cross-Section

Several types of girder cross-sections have been used for spliced girder projects. The type of section used is sometimes set by project requirements or local practice.

2.1.1   I-Girder or Bulb Tee

Standard AASHTO and PCI sections can be used for extended span ranges and splicing. In some cases, the cross-section dimensions may need to be modified to accommodate post-tensioning ducts, or to obtain other advantages, such as increased span capability, improved shear resistance or increased to improve girder stability.

In some states, new standard sections have been developed suitable for the design and construction requirements in a particular region. An example of this is the Florida MBT ("modified bulb tee"), which is 78 in. deep and has a 60 in. wide top flange *(1)*. The top flange width was increased to improve handling and reduce forming costs.

The size of the bottom flange is important when a large number of pretensioned strands are required for the design. The bottom flange size is not as important when post-tensioning is used, although a larger bottom flange will improve the composite section properties.

2.1.2   Open-Topped Trapezoidal Box Beams

Open-topped trapezoidal box beams, also known as U-beam, bathtubs, or tubs, have been used for an increasing number of projects in recent years. These sections are favored by some designers and agencies because they are considered to have improved aesthetic qualities. In some cases, pier caps have been eliminated by positioning girder lines directly over columns. The girders have significant room in the webs and bottom flange for post-tensioning tendons. Internal tendons could also be used. They are very stable for transportation and erection because of the wide bottom flange.

These sections are very heavy since they have two webs, so splicing may be necessary at shorter span lengths than those required for I-girder of bulb-tee shapes. The open-top configuration makes these segments poor candidates for use as pier segments, so bridges using these sections are typically spliced at interior piers.

Fabrication of these sections is complicated because the void must be formed in the interior of the tub. There are also significant issues related to accommodation of cross-slope and superelevation with these wide sections. In some areas, a family of tub sections has been developed with a range of section depths. A change in depth also changes the width between the tops of the webs because of the slope in the webs.

2.1.3 Decked Cross-Sections

In limited areas of the country, standard girder sections are used with some or all of the deck section cast with the girder prior to prestressing. The sections are generally pretensioned, but splicing and post-tensioning are also often used because of the weight of these girders.

This type of cross-section makes more efficient use of concrete because the dead load of the deck is present when the initial prestress is applied. This type of cross section also has a significant advantage in that it makes possible very rapid construction of a bridge since most or all of deck is already in place.

Connections between adjacent girders are made at the deck and possibly at intermediate diaphragms. Weld plates, grouted keyways and transverse post-tensioning have been used to establish the connections between adjacent girders.

The following options are available for the deck portion of the girder:

- Full width and full thickness

- Partial width and thickness, with cast-in-place structurally composite topping that also fills gaps between flange tips (eliminates open joint between flange tips)

- Full width and partial thickness

- Partial width and full thickness, with cast-in-place closure

The use of a partial thickness deck allows for placement of a composite deck that can be used to establish the required cross-slope or superelevation, and can also help control long-term camber. However, it slows construction since reinforcement must be set and concrete placed for the composite deck.

Concerns expressed by some designers regarding this type of cross-section include:

- Effectiveness of connections between adjacent girders

- Possible reflective cracking over, and leakage between, longitudinal deck joints

- Control of long-term camber without a composite deck

- Rideability of the final deck surface

- Long-term durability of the deck

- Location of tendon anchors for post-tensioning

It should be noted that conventional (undecked) bulb-tee cross-sections, with their extended flange widths and relatively shallow, tapered top flanges, offer many of the advantages of this method of construction without many of the problems.

2.2   Girder Bottom Flange Profile

When splicing girders, there are several options that have been used for the profile of the girder. These options generally are used to address the need for additional moment resistance at interior supports. However, occasionally, they have also been used to accomplish other design objectives, such as providing haunches on the ends of simple-span girders to satisfy aesthetic requirements for a project.

2.2.1   Constant Depth

When using standard cross-sections, it is typically most economical to utilize sections with a constant depth for the length of the span or spans. Where standard section dimensions are not adequate to satisfy design requirements, which may occur at the interior supports where web shear stresses or bottom flange compressive stresses may be extremely high during construction and even under service loads, the following approaches can be utilized.

- Webs may be thickened at locations of high shear and/or moment if required. In some cases, extended end block sections, which have a much thicker web, have been used to address high shears and/or moments.

- In a few cases, a bottom slab has been added between bottom flanges to provide additional moment resistance at interior supports while maintaining a constant section depth. In these situations the bottom slab is usually cast after the girders are erected and is only provided for the portion of the span on each side of the interior support where additional compression area or stiffness is required.

2.2.2   Haunched

Where design requirements force the modification of the cross-section, girders are often haunched at the interior supports. The increased depth provides additional moment and shear capacity to satisfy the design requirements. However, the increased depth can result in a haunched pier segment that may be too heavy to transport over highways.

The haunch has typically been a linear taper, which is simple to construct. Curved or parabolic haunches may also be used, but may result in an increased cost.

The fabrication of haunched girders usually requires a special bed because the depth of the section is greater than can be accommodated in a standard prestress bed.

There are several options available for forming haunched sections for I- or bulb-tee girders:

2.2.2.1 Constant Web

Existing side forms can be used by building a sloped or curved soffit beneath the forms. The increased depth is accomplished by varying the height of the bottom flange. Therefore, the portion of the cross section in which the depth is increasing is as wide as the bottom flange. An example of this type of section is shown in **Figure C1-1a**.

While this approach simplifies fabrication by allowing the precaster to utilize existing side forms as part of the form for the haunched girder segment, the increased height in the haunched portion of the wide bottom flange adds significant weight to the section that may not be necessary or desirable.

2.2.2.2 Constant Bottom Flange

Special forms can be used for the haunched girder segment that allow the bottom flange to remain a constant depth along the length of the segment. Therefore, the height of the web varies along the segment as shown in **Figure C1-1b**.

Use of this type of girder segment requires the fabricator to purchase a custom set of forms. However, the weight of the segment is reduced significantly, which may result in savings in transportation costs. As the length of the span, and ultimately the depth of the haunched girder, increases, this option becomes much more attractive than the first option because of the difference in girder segment weight.

On some projects, the bottom flange has increased in size slightly from the end of the haunched girder segment to the center of the girder segment. Since customized forms are required for the constant bottom flange approach, this approach is little different. The larger bottom flange may be required at the center of the haunched girder segment to resist the negative factored design moment.

2.2.2.3 *Splicing of Haunched Sections*

The use of spliced haunched segments has been proposed to make bridges with haunched pier segments a viable solution at locations where barge delivery is not possible or where other limitations or restrictions may apply. The use of both horizontal and vertical joints has been considered. It appears that the use of a vertical joint may provide several advantages including simplicity of design and fabrication, ease of shipping, ease of splicing, and the ability to provide some camber in the completed pier segment to counteract the sag at the segment tips that occurs when a drop-in segment is erected. This concept is discussed in **Section 2.4.5** of the Final Report.

2.3   Preliminary Design Cost Data

A critical need in the successful implementation of girder bridges with extended spans in areas where there is no experience with this type of construction is the collection of cost data to be used in preliminary cost estimates to compare design alternates. However, it is difficult to obtain meaningful information on bridge costs. In fact, because construction costs differ significantly between different regions of the country (labor costs, materials costs, special design requirements, etc.), between different methods of construction (design/bid/build, design/build, VECP), and between different contractors (skill, experience, available equipment), it does not appear possible to assign a cost figure to a general bridge type. See discussion of this topic in **Section 2.2** of the Final Report.

2.4   Bridge Layout

Preliminary design of spliced girder bridges is essentially the same as for other types of bridges. The list of example structures provided in **Appendix A** provides some guidance regarding possible span ranges of a certain bridge type or configuration. The designer also needs to be familiar with typical span to depth ratios that can be achieved by the various approaches to long span design.

The bridge layout should be developed considering the following constraints.

2.4.1 *Location of Piers and Splices*

The location of piers and splices should generally be determined by a combination of structural issues, site characteristics, fabrication and erection concerns. Segment length and weight should be considered, especially where the hauling weight limits are restrictive. Possible locations for temporary supports must also be considered. A spliced girder bridge in Pueblo, Colorado *(2)* is a good example of using spliced girders to address several obstacles for pier locations.

Where lengths of girders may vary within a span or between related bridges, the use of standard segments should be considered. For example, in a simple-span bridge, standard end segments with the same length and details could be developed with the variation in girder lengths being taken up in the simpler center segments. The Washington State DOT has developed standard based on this concept. Any such standardization will improve the economy of the bridge.

In many cases, splicing near midspan is discouraged because of the high flexural stresses in this region. However, with proper consideration in design and detailing, a midspan splice can be used without sacrificing durability, performance or structural integrity.

2.4.2 Limitations of Transportation, Access or Equipment

While this type of information may not be known with certainty during design unless it is a VECP or design/build project, the designer must be generally familiar with typical capabilities. The designer should contact fabricators and the owner agency to determine the maximum limits for girder shipping weights and lengths in the vicinity the project, considering several different producers. If producers are located in a different state, they will have to satisfy the requirements of all states through which the girder will pass on the way to the site. The design may have to be altered by splicing, the use of reduced density concrete or other measures in order to satisfy shipping requirements.

It is reported that girders up to 200,000 lbs have been shipped by truck in some states, while in others, the truck and girder weight is limited to 180,000 lbs, which limits the weight of girder to approximately 132,000 lbs for a particular case. The variability of hauling restrictions is illustrated by another case in which a girder fabricated in one state was shipped to another state where it had to be reloaded onto another trailer for shipping to a third state. The change in equipment was required because neither trailer could be used in all the states on the route.

If the bridge crosses a navigable waterway, girder segments may be delivered by barge, which may permit the transport of much larger girder segments than is possible on highways.

The designer should also be aware of commonly available lifting equipment to make sure that, unless absolutely necessary, the member sizes used in the design will be reasonable for area contractors.

2.5 Analysis and Design

A variety of issues not generally considered in standard pretensioned girder design must be taken into account when designing spliced girder bridges and other types of long-span structures. Some

of the major issues are listed below. Design issues for long-span structures which are not significantly different than those for the design of conventional bridges are not discussed.

2.5.1   Live Load Distribution

2.5.1.1   Approximate Distribution Factors

The range of applicability for the approximate distribution factors given in the *LRFD Specifications* for concrete girders with composite decks and decked bulb tees includes spans up to 240 ft. The range of applicability for the distribution factors for open-topped box beams extends only to spans of 140 ft. Therefore, for both types of bridges, long span structures may be beyond the range of applicability of the approximate equations. Therefore, the designer will need to consider the use of a refined method for estimating the distribution of live loads in long span bridges where the span exceeds the range of applicability for the appropriate equation.

The approximate distribution factors are applicable for both simple-span and continuous bridges.

2.5.1.2   Refined Analysis

If refined analysis is used to determine the distribution of live load to individual girders, the multiple presence factors given in the *Specifications* must be used.

2.5.2   Time-Dependent Effects

When bridges are indeterminate from continuity or frame action, time-dependent effects become significant. Combining indeterminacy with long spans and/or splicing of girders may increase the degree and significance of these effects. Of even greater consequence is the redistribution of forces from these effects that occurs as a result of large post-tensioning forces having been applied to a structure under changing static support conditions (i.e., built in pieces that are then made continuous).

Time-dependent effects include creep, shrinkage and relaxation of prestressing steel. These effects are generally not considered directly in typical pretensioned girder designs. Therefore, designers may not be aware of these effects or have the design tools and experience to analyze these effects.

2.5.3   Temperature Effects

Uniform axial temperature changes (seasonal) and temperature gradient effects (daily) must be considered during design.

It has been found that temperature gradients in girders caused by daily fluctuations in temperature can have a significant effect on bridges. Stresses of sufficient magnitude to cause cracking in concrete can be developed when the deformations induced by temperature gradients are restrained in indeterminate structures.

Analysis of the effects of thermal effects is not well understood by many designers. More sophisticated analysis methods or computer programs are required to consider the effects of temperature on long-span bridges.

2.5.4 Specified Concrete Strengths

2.5.4.1 High Strength Concrete

While the use of high strength concrete can extend the span of prestressed concrete girders, increases in strength should be considered in increments during the design process. A large increase in concrete strength may produce an increased span length capability, but it may be found that a smaller increase in concrete strength will provide nearly the same span at a lower cost.

2.5.4.2 Specified Strengths at Critical Stages

Because of the possibility of a number of stages of prestressing during the construction of a spliced girder bridge, it is important to clearly indicate the specified concrete strengths for different elements of the structure at each point in the construction sequence. If girders are pretensioned, an initial minimum concrete compressive strength at release must be specified. If the girders are subsequently post-tensioned, a second minimum compressive strength at post-tensioning should be indicated, even if it is the 28-day strength.

Minimum concrete compressive strengths at time of stressing of any required post-tensioning must also be specified for concrete in wet-joint splices and for composite decks. Care must be taken to ensure that the correct minimum concrete strengths are used for computations of strength and stresses, including computation of section properties at different stages of construction.

2.5.5 Effective Flange Width

For spliced concrete girders, the effective flange width is determined as for conventional bridges. The more complex considerations for segmental concrete box beams are typically not required.

2.5.6   Redistribution of Moments in Continuous Beams

The *LRFD Specifications* permit moment redistribution in "reinforced concrete beams" at the strength limit state (Article 5.7.3.5). There is some confusion as to whether the *Specifications* are intended to read "continuously reinforced concrete beams" (i.e., bonded reinforcement), thereby excluding unreinforced segment joints (i.e., precast segmental construction) only, as opposed to all prestressed superstructures. This issue has been noted in the list of comments on the *LRFD Specifications* contained in **Appendix E2**.

2.5.7   Resistance Factors

It appears that the *LRFD Specifications* intend spliced girders to be considered as segmental construction. Therefore, the resistance factors for flexure for joints in segmental construction should be used for evaluating the capacity of spliced girders at splices. These resistance factors are different from the resistance factors for conventional construction.

2.5.8   Load Factors for Locked-In Force Effects

Because construction of spliced girder bridges frequently involves staged post-tensioning and staged construction, "locked-in force effects" result from continuity restraint of time-dependent deformations (CR, SH), applied displacements (TU, TG, SE) and post-tensioning (EL). Secondary post-tensioning effects (EL) are including in all limit states with a load factor of 1.0. The remaining effects are included at the Service Limit States with a load factor of 1.0 for force design and 1.20 for displacement design (i.e., bearing and joint movements), at the Strength Limit States with a load factor of 0.50 for force design and 1.20 for displacement design, and at the Extreme Event Limit States with a load factor of 0.

2.6   Pretensioning

Spliced girder segments may or may not require pretensioning. Pretensioning may not provide any benefit to the final structure and may be necessary only to address conditions during handling. It may be possible to specify lifting locations that will eliminate the need for pretensioning for handling. This should be discussed with fabricators during the design phase. For girder segments where splices are well away from the maximum positive moment locations, pretensioned strands can be used very effectively to complement the post-tensioning tendons in meeting design requirements at maximum moment locations. In these cases, a significant number of strands may be used, as shown in **Figure C1-2** for a drop-in girder segment. Pretensioned strands cannot be located in the center positions in the bottom flange or in the web if they conflict with the post-tensioning ducts.

If match-cast splices made with precision-machined bulkheads are used, significant pretensioning will lead to end rotations that will alter the fit-up of the segments after release. If the minimum amount of pretensioning required to balance the dead load of the segment is used, the alignment of the ends of the segment should remain essentially as cast. Therefore, pretensioning for match-cast segments should be used carefully.

Where cast-in-place splices are used to connect girder segments, any relative end rotations of girder segments can be accommodated in the splice.

2.7 Post-Tensioning

Consideration of post-tensioning includes many complex issues such as:

- Ducts

- Tendon stressing

- Grouting

- Anchorages

- Inspection

While some of these issues may be more closely related to construction than designing, the proper consideration of all of these issues is critical to the successful fabrication, construction, and long-term performance of a post-tensioned concrete bridge. Many requirements related to these issues will appear in project specifications or notes on the design drawings.

The designer is directed to sample specifications and other resources developed by owner agencies that have significant experience with post-tensioned construction. Most of these documents are now available on the internet.

A major resource that provides detailed discussion of the issues related to post-tensioning that must be considered in spliced girder construction has been prepared by FDOT *(3)*. Careful consideration of this document is highly recommended.

2.7.1 Types of Post-Tensioning Tendons

When a simple-span girder is post-tensioned to extend its span, tendons generally run the full length of the girders, whether the girders are constructed as a single-piece or are spliced.

For post-tensioning tendons in continuous bridges, the following types may be used:

- Continuous tendons running the full length of the continuous unit

- Partial length tendons with intermediate anchorages. These tendons typically extend for a distance greater than the length of one segment.

- Stitched tendons, i.e., local overlapping tendons crossing a splice location and anchored a short distance each side of the splice. By avoiding long continuous tendons, design and construction may be simplified. Sample details from plans are shown in **Figure C1-3**.

Of these types of tendons, continuous tendons are used most frequently by a wide margin. The remaining types have been used only occasionally.

Post-tensioning tendons for bridge girders are normally bonded, multi-strand tendons. The tendons are normally located within the webs (i.e., internal tendons). Grouting is used to provide bond between the girder and strands and to provide corrosion protection. The size of tendon is determined by structural requirements with consideration of the available space for ducts and anchorages.

External tendons, or those running outside the confines of the concrete cross section are often used in segmental box girder bridges and may eventually see limited used in spliced girder construction. These types of tendons are normally restricted to closed cell box girder usage where the tendons can be located within the box void, out of sight and protected from the elements.

2.7.2   Web Thickness

The *LRFD Specifications* require that the diameter of post-tensioning ducts be no more than 0.4 times the minimum web thickness. However, a ratio of 0.5 has been used with success in a number of bridges. In such instances it is extremely important to consider the effects during construction of high web shears and stresses resulting from post-tensioning tendon curvature prior to the ducts being grouted. The web width must also be adequate to provide minimum cover over mild reinforcement passing the duct.

2.7.3   Section Reduction at Ducts

The reduced cross-section at ducts should be considered when determining both flexural and shear capacity of the member. The *LRFD Specifications* give provisions for subtracting duct areas

for shear and flexural computations. However, it may be found that neglecting the reduction in properties caused by the voids is conservative for many bridges. See discussion in **Design Example 1**.

Special attention should be given to reduced section properties to prevent overstressing of the cross-section when a member is tensioned.

2.7.4   Spacing of Ducts

Provide at least minimum spacing as required by the *LRFD Specifications*. The spacing must be adequate to prevent break-through between ducts at locations of significant tendon curvature.

2.7.5   Length of Tendons

Post-tensioning tendons can generally be any length required by the structure, as long as the effective prestress after all losses including friction is adequate. However, some states have recently placed restrictions on continuous tendon lengths based on concerns about obtaining adequate prestress levels along the entire tendon length, the ability to install strands in the deck without kinking or binding, and the ability to grout exceptionally long tendons. The designer may want to consult with the owner agency specifications and a post-tensioning supplier if very long tendons are being considered.

2.7.6   Staged Post-Tensioning

The application of prestress to a bridge can be staged by introducing it at different times in the life of the bridge. This is most often done by applying additional post-tensioning to the bridge cross section after the deck has been applied. This improves the structural performance of the bridge by restoring some of the prestress that has been used in supporting the weight of the deck concrete, and by precompressing the deck longitudinally for improved durability. Initial prestress can be applied either by pretensioning or by post-tensioning although handling and transportation concerns normally dictate the application of at least a minimum level of pretension.

A disadvantage of applying post-tensioning in stages is that access to post-tensioning anchorages must be maintained until late in the bridge construction. Pattern casting of the deck may be required to maintain access to tendons for second stage tensioning. This can complicate the casting of the deck. A second mobilization of the post-tensioning subcontractor (if one is used) and his equipment is also required. In some cases, anchorages for second stage post-tensioning tendons have been located in the top of the girder. This improves access for stressing the tendons, but requires that a portion of the deck cannot be cast until the tendons are stressed and grouted.

Typically, lower post-tensioning tendons are stressed in the first stage and higher tendons are stressed in the second stage. This can provide a benefit in improved precompression in the deck and in access to second stage tendons for stressing.

2.7.7 Sequencing Across Bridge

Post-tensioning of girders is accomplished sequentially across a bridge. The effect of tensioning on adjacent girders should be considered, especially where permanent diaphragms have been placed between girders prior to stressing. For unusually wide bridge decks, consideration should be given to providing guidance in the design plans concerning the transverse sequence of girder erection, temporary bracing installation and deck casting.

2.8 Ducts

2.8.1 Material

Ducts embedded in concrete (i.e., "internal ducts") should be fabricated from corrugated galvanized metal or corrugated high-density virgin polyethylene (polyethylene). Ducts located outside of the concrete (i.e., "external ducts") should be smooth polyethylene pipes. However, in either case polyethylene ducts or pipes should not be used when the radius of curvature of the tendon is less than 30 ft (9 m). External ducts would generally apply only to U-beam cross-sections.

Metal ducts should be fabricated with either welded or interlocked seams, and joints between sections should have connections that do not result in angle changes at the joints. Ducts should be sufficiently rigid to bend without crimping and to maintain their correct shape and alignment during concrete placement.

Polyethylene internal and external ducts should meet the ASTM and governing state specification requirements. The polyethylene material should not contain water-soluble chlorides. Due to corrosion concerns and the increasing desire to eliminate all nonessential, nonstructural metal components from post-tensioned bridge structures, nonmetallic ducts are becoming the preferred choice for post-tensioning duct material. Because of the greater flexibility of polyethylene ducts than metal ducts, however, they need to be supported at closer intervals to prevent misalignment and displacement of the duct during girder casting.

At locations of small tendon curvature, such as in "deviation saddles" and anchorage locations, smooth steel pipes are usually specified. In extremely aggressive environments, the pipes should be galvanized.

2.8.2 Shape (Cross-Section)

All ducts for longitudinal post-tensioning of girders should be round (i.e., not oval or "flat"). The concrete cross-section should be proportioned to allow the use of ducts of adequate diameter while maintaining minimum cover over ducts and reinforcement. "Flat" ducts may be used in thinner cross-sections, such as for transverse or longitudinal post-tensioning of bridge decks.

2.8.3 Size

Current post-tensioning practice recommends that the internal cross-sectional area of ducts for multiple strand tendons be at least 2.5 times the net area of the strand tendon. LRFD Article 5.4.6.2 also requires that the outside diameter of the duct not exceed 40% of the width of the web. If adequate cover is provided for the duct and reinforcement, this limit has been violated in several recently constructed precast, prestressed concrete girder bridges with post-tensioning.

Ducts for single strands, "flat" strand tendons or single post-tensioning bars should have an inside diameter at least 3/8 inch (10 mm) larger than the nominal diameter of the strand or bar. The dimensions are slightly larger than the *LRFD Specifications* require. However, they are recommended for realistic, proper installation of post-tensioning strand tendons or bars.

2.8.4 Profile

Minimum tendon radii are specified in LRFD Article 5.4.6.1 and other documents. It is recommended also that a bending radius of strand tendons not be less than 10 ft (3 m). The tendon profile is typically not a factor in spliced girder design and detailing while it can be in segmental box girder designs.

2.8.5 Splicing

2.8.5.1 In Segments

Duct splices within precast girder segments must be sealed and secured to prevent leakage during placement of concrete.

2.8.5.2 At Splices

Post-tensioning ducts must be spliced at wet cast joints between girder segments. The splices must be sealed to prevent leakage of cement paste into the duct. Therefore, the use of proper materials and installation methods for duct splices is essential for the successful completion of the

project. Details should be specified in the contract documents and carefully monitored during construction. Heat shrink material has been used successfully to seal duct splices at wet cast splices.

Duct locations within girder segments and the placement of girder segments during erection must be carefully controlled to provide proper alignment of ducts at splices. Wider splices may be used to provide additional room to accommodate any misalignment of ducts in the splice.

2.9   Girder Splices

2.9.1   Type

There are two prevalent types of splices for girders.

2.9.1.1   Cast-in-Place

A cast-in-place (wet-cast) splice is currently the most common type of splice. For this splice, girders are precast independently with the length adjusted to allow for a cast-in-place concrete splice.

Using a field-cast splice provides generous tolerance in the fabrication and erection of the girder segments. Erection is also simplified because the two segments do not require an exact fit. However, measures must be taken to ensure that the segments are properly aligned to enable successful splicing of the ducts.

The width of the splice must be adequate to provide for splicing of any post-tensioning ducts that cross the splice. In some cases, mild reinforcement also extends from the adjacent segments into the joint and is spliced, either by lapping or by use of mechanical splices. See **Figure C1-4**.

The strength of the concrete in the splice can be a significant issue, especially if the splice is at or near a critical section. The rate of strength gain of the splice concrete can also be important because the splice concrete must reach a specified strength before tendons can be stressed that cross the splice.

2.9.1.2   Match-Cast

Match-cast splices have also been used in a few spliced girder projects. In most cases, a match cast joint is emulated by casting pieces of girders using precision-machined bulkheads to provide the equivalent of match casting, as shown in **Figure C1-5**. True match casting is not possible if segments are pretensioned. As the segments are assembled in the field, epoxy is applied to both sides of

the joint, the segments are drawn together, and a temporary clamping force is applied across the joint to ensure bonding of the epoxy. Dry joints between segments are not permitted.

Since there is no clearance between segments to allow for splicing of ducts, a compressible gasket is used to seal the duct at the splice. The minimum effective size of seal is used to minimize the portion of the cross-section over which bond between the segments is prevented.

Since the segments are match cast, alignment of the segments is ensured. Alignment keys may be used to assist in proper positioning of the girder segments. In most cases, erection is essentially the same as for girder segments with wet joints because the segments are set in position, but are not epoxied and pulled together until later. An advantage to the use of match cast joints with epoxy is that it is not necessary to wait for strength gain of the joint material prior to post-tensioning.

Because the ends of the segments are match cast, end rotations caused by pretensioning and camber due to temperature gradient or other effects can disturb the fit-up of segments. Other tolerances must also be closely maintained because there is no opportunity for compensation or correction. Sliding bearings are used under one of the girder segments to allow the segments to be drawn together after the epoxy is applied.

Where girder segments are joined to previously erected segments using match cast splices, a wet joint is generally required somewhere within the span to allow room to position and join segments prior to casting the final closure joint in the span.

2.9.2 Shear Keys

Shear keys are often used at splice locations to assist in carrying the design shears at the splice locations. A variety of details have been proposed and used as shown in **Figure C1-5**. There is also discussion regarding whether their use is mandatory. In some cases, the force provided across the splice by post-tensioning is adequate to resist the design shear without shear keys. Use of a shear key at a splice location generally does not represent a significant increase in fabrication cost or effort.

2.9.3 Site of Girder Splicing Operations

Typically, girders are spliced in place. This generally facilitates erection by requiring smaller lifts since each girder segment is erected separately. However, it may be economical for the fabricator to splice the girder segments at the plant and to then ship the spliced girders, or for the contractor to splice the girders on site and then erect the spliced girders. While both of these alternate

approaches require much larger lifting equipment, the elimination of temporary supports or the consideration of other factors may make this economical.

Splicing girder segments prior to erection may require local post-tensioning that would run only the length of the assembled girder segments. This post-tensioning used to assemble the girder segments could either be independent of the continuity post-tensioning or be spliced to the continuity post-tensioning, providing that tendon coupling is allowed by the governing project specifications.

2.9.4   Diaphragm at Splice

A diaphragm is typically installed between girders in conjunction with the splice. This provides additional stability for the girder segments and confinement of the splice concrete. The use of a diaphragm at the splice also provides increased access to the splice area for improved consolidation of the concrete in this critical location.

2.9.5   Limiting Stresses

The *LRFD Specifications* provide limiting stresses for prestressed concrete in tension and compression. The compressive stresses are the same for conventional and segmental construction. However, the tensile stresses are different. For segmental construction, limiting stresses are only specified at the joints, while for conventional construction, limiting stresses are specified in general and applies at all locations.

While a strict interpretation of the *Specifications* in their current form would seem to indicate that spliced girders only need to satisfy the limiting stresses for segmental construction, it appears that the intent of the *Specifications* is satisfied if the limiting stresses for segmental construction are used at the joints and the limiting stresses for conventional construction are used for all other locations. This issue is noted in the review comments in **Appendix E**.

2.9.6   Mild Reinforcement in Splice

Typically, mild reinforcement is normally not provided across wet joints (the use of mild reinforcement is not applicable to match-cast joints). However, in some cases, reinforcement has been placed across splice locations. The mild reinforcement is either lap-spliced or spliced using some type of mechanical or welded connection. This reinforcement can be used to provide additional moment capacity at the strength limit state but is not required for crack control since the tensile stress in the splice is limited to less than the cracking strength of the concrete. Adding unnecessary reinforcement in the splice can complicate and slow fabrication of girder segments and completion of the splice

2.9.7   Preparation of Surfaces

Prior to splicing, for both match-cast and wet-cast splices, the surfaces to be joined should be thoroughly cleaned to remove any foreign matter or loose concrete. The type of cleaning method (sand or water blast) and location (plant or site) should be specified in the contract documents.

2.10   Temporary Shoring

Two applications of shoring are discussed below. The benefits of using shoring must be sufficient to offset the costs of the shoring and any additional foundations required. In some cases, the permanent pier foundations can be used to support the temporary shoring.

2.10.1   Towers for Erection of Girder Segments

Temporary shoring towers may be required for supporting girder segments prior to splicing. If assumed in the construction sequence used to design the bridge, these must be shown on the contract plans. Several types can be used, from pipe and HP pile sections to commercial scaffolding, as shown in **Figure C1-6**. The towers can be driven as piles, or may be supported from temporary foundations on soil, or from permanent pier footings. Additional details of temporary shoring towers are shown in **Figure C1-7**.

2.10.2   Touch Shoring for Carrying Deck Dead Load

As discussed in **Appendix B1, Section 2.5**, shoring can be used effectively to extend the span range of precast, prestressed concrete girders by supporting a girder during placement of deck concrete. After girders are erected, the shoring is adjusted to touch the bottom of each girder without applying a significant force to it. Therefore, any additional load placed on the girder, such as the deck slab, will be partially supported by the shoring. The girder must still span between temporary and permanent support locations, but the span of the girder for the deck load is reduced. This results in reduced stresses in the girders from the deck dead load. At temporary shoring locations, the additional dead load creates a negative moment, a portion of which remains in the girder after the deck is cast and hardened, which reduces positive bending stresses from loads subsequently applied to the completed structure.

After the deck slab reaches a specified compressive strength, the shoring is removed. The force that was being carried by the temporary shoring tower must then be carried by the full span girder, which is now a composite section. Therefore, the total stresses caused by loads applied with the shoring in place will be less than the stresses that occur if the shoring had not been used.

For this technique to be successful, the designer must make a realistic assessment of possible shoring tower configurations and foundation conditions and deformations that could be expected from these sources. If the deformations are significant relative to the unshored deflection of the girder under the weight of the deck concrete, the shoring will not be effective.

Shoring that was initially used for supporting girder segments prior to splicing can also be used to support the girders after splicing when the deck concrete is placed the spliced girders will rise off of the temporary supports after first stage post-tensioning. For two stage post-tensioning the shores may then be adjusted to touch the girders after stressing to provide additional benefit.

For bridges where single stage post-tensioning is applied after the deck is cast, shoring required to support girder segments will have to remain in place while the deck is cast. Therefore, the benefits of touch shoring will automatically be available.

2.11 Anchorages

Whenever post-tensioning is used, special hardware and reinforcement is required to resist the large concentrated forces that occur in the girder at the anchorage locations. There are several options that may be considered when detailing anchorages.

2.11.1 Location of Anchorages

Anchorage locations must accommodate tendon installation, stressing and grouting operations and must also be detailed so they can be adequately encased to prevent corrosion of the anchorage assembly.

Anchorages have been placed at several locations in girders.

2.11.1.1 In Girder

Most often, post-tensioning anchorages are located in the end face of the girder. This is generally the location for which fabrication is the easiest. Anchorages have also been located in blockouts in the top of the girder or in notches at the end of the girder. With proper detailing and consideration of construction requirements, anchorages placed at any of these locations can be successfully used. See **Figures C1-8** and **C1-9** for details and photos of anchorages.

A disadvantage for anchorages located at the end of a girder is that the erection of girders or the stressing of tendons in the adjoining spans will be affected, if adequate spacing for post-tensioning

operations (i.e., stressing rams, strand extensions, etc.) is not available. In some cases, girders in adjacent spans cannot be set until the post-tensioning operations have been completed.

Where anchorages are in the top of the girder or notches at the end of the girder, a portion of the deck slab cannot be cast until the post-tensioning operations have been completed. This may be a significant disadvantage for a contractor.

The fabricator should be permitted to install and cast anchorages located in girders either in the pretensioning bed with the rest of the segment, or as a secondary pour. See further discussion in **Section 3.2.**

2.11.1.2 In Endwall

The fabrication of end anchorages in girders can be avoided by placing the anchorages in an endwall that is constructed after the girders are erected. This detail has been used successfully on a number of projects. While girder fabrication will be simplified by placing the anchors in an endwall, placing the anchorages in the girder may be more economical than having the contractor install the anchors and associated reinforcement in the field. Plant fabrication generally ensures better control over the placement of anchorage hardware and reinforcement and over concrete quality in the congested anchorage area.

2.11.1.3 Intermediate Anchorages

Segmental (box girder) construction frequently uses so-called "blisters" to anchor intermediate post-tensioned anchorages for tendons that do not run the full length of the structural unit. This type of anchorage has rarely been used on spliced girder bridges since it is generally desirable for post-tensioning tendons to run the full length of the girder.

A special case of intermediate anchorages is a stitched splice where short tendons are installed across a splice.

2.11.2 Detailing of Anchors

The post-tensioning supplier is generally only required to provide reinforcement details for the "local" zone behind the anchorage. The engineer must design and detail reinforcement in the "general" zone, which is the remainder of the anchorage region. This detailing of reinforcement by two parties can lead to difficulties in fabrication since all reinforcement may not be shown on the same

drawings. Every effort should be made to identify and eliminate potential conflicts during the design and shop drawing submittal phases of a project.

2.12 Future Deck Maintenance or Replacement

In many parts of the country, the future maintenance or replacement of a bridge deck is an important issue that must be considered in the design of a bridge. In some jurisdictions, bridges must be designed and constructed so that the entire deck may be removed and replaced at some time in the future. This topic is discussed further in **Chapter 2, Section 2.4.1**.

2.12.1 Alternatives to Future Deck Removal

To design for future removal of the deck, any scheme for prestressing in which prestress is applied to the structure with some or all of the deck in place must be closely examined. Without the deck, girders may become overstressed. Therefore, to avoid the necessity of future deck removal, the following approaches have been suggested to address potential deck deterioration.

2.12.1.1 High Performance Concrete

High performance concrete (HPC) decks are now being used in many areas of the country to provide extended service life. In this case, HPC is used to significantly reduce the permeability of the deck concrete, which delays or prevents the intrusion of chlorides.

2.12.1.2 Wearing Surface

A wearing surface can be used to provide additional protection for deck reinforcement to eliminate the need for eventual removal of the entire structural deck. Before the structural deck begins to deteriorate, the wearing surface can be removed and replaced to eliminate material contaminated with chlorides. This type of approach has been used on some segmental box girder bridges.

There are several different types of wearing surfaces. There has been research studying the relative merits of the different types. In all cases, however, wearing surfaces are an expensive addition to the cost of a bridge.

2.12.1.3 Bi-Directional Deck Post-Tensioning

Transverse post-tensioning of the deck can be used to improve deck durability. When the deck is post-tensioned in both the longitudinal and transverse directions, the deck should be essentially crack free. This should prevent intrusion of corrosive agents, resulting in increased service life.

However, post-tensioning the deck results in significant added cost. This may be offset in part by the utilization of structural benefits of transverse post-tensioning, such as reducing the deck thickness or increasing the girder spacing.

2.13 Curvature in Plan

Spliced girders have been used successfully on bridges with a radius as small as 500 ft *(4)*. The basic concept remains unchanged, and only a few additional details and design considerations are required.

To provide curvature in plan, straight girder segments are used to approximate the curve. Therefore, angle changes occur at the splice locations. This means that the splice must be wide enough for the post-tensioning ducts to be curved at an acceptable radius in plan as they cross the splice. Estimates of friction losses must also account for the friction loss caused by the tendon curvature in plan.

The tendon curvature in plan will cause a transverse force that must be considered in design. This force will be most significant at the interior girder where, without proper detailing, the surface of the concrete could spall off during stressing of the post-tensioning tendons.

A three-dimensional structural analysis may be required for small radius ramps.

2.14 Integral Caps

Integral caps may be used with spliced girder bridges. Their benefits include:

- Providing a rigid connection between superstructure and substructure to improve performance in extreme events (seismic or vessel impact)

- Providing frame action to improve structural efficiency

- Minimizing the vertical separation between crossing roadways since some or all of the depth of the pier cap is contained within the depth of the superstructure. This is a benefit where the cap extends over the roadway below.

- Improve aesthetics of a bridge

Integral caps may be either conventionally reinforced or prestressed. Prestressing is preferred to improve structural performance and to reduce the potential for cracking on the top of the

cap and deck. Pier segments may pass completely through an integral cap, or segments may be spliced in the cap.

Integral caps can be detailed with concrete or fabricated steel hinges beneath the cap if rotational restraint in the longitudinal direction is not desired.

The use of an integral cap for spliced girder construction is illustrated in **Appendix D, Design Example 2**.

2.15 Typical Issues for Post-Tensioned Construction

The following are issues associated with any post-tensioned bridge design. They may be unfamiliar to designers accustomed to only designing pretensioned girders. Since guidance in these areas is available from other sources, designers will be referred to those sources rather than to repeat the same information in this report. The items are listed so designers will be aware that they should be considered.

2.15.1 Losses

Losses due to friction and anchor set must be considered for post-tensioned spliced girders. The design examples in **Appendix D** address this issue.

2.15.2 Secondary Moments

When continuous members are post-tensioned, secondary moments are caused. These moments may be significant and must be included in the analysis and design. Of even greater consequence is the redistribution of forces from these effects that occurs as a result of large post-tensioning forces having been applied to a structure under changing static support conditions. *The PCI Bridge Design Manual (5)* has a detailed discussion of this issue.

2.16 Camber and Deflections

When spliced girders are erected, segments may be supported on falsework towers or by strongbacks supported on other segments. While some segments will have cambers like ordinary pretensioned concrete girders, others will have cambers and deflections that can be very different from the ordinary. The designer must be aware of these differences and make provisions in the design and details to account for them. The design examples in **Appendix D** address this issue.

2.17 Decked Members

Decked members are described in **Appendix B1, Section 2.1.6**.

2.17.1 Accommodation of Cross-Slope and Superelevation

Decked girder sections generally do not have a composite topping that may be used to provide the desired cross-section of the riding surface. Therefore, the required cross slope can generally be provided by tapering the deck flange on the girders or by tilting the decked girders so that the top of the flange is properly oriented for the cross slope.

Use of decked members with a full-depth deck is not recommended where there are transitions in deck cross-section, unless an additional surface is applied to provide the desired surface.

2.17.2 Connections Between Girders

Details must be provided for the connection between the decks of decked members. Typically, the connection between adjacent girder flanges is made by welding plates between inserts.

An alternate deck connection detail uses spliced mild reinforcement in a cast-in-place closure strip between girder units. While the closure pour eliminates the open joint in the deck between the girder flanges that may lead to reflective cracking in the deck, it requires forming and slows the completion of the bridge.

Transverse post-tensioning may be used to make the connection between adjacent decked girders. Intermediate diaphragms may be used to provide a more positive connection between adjacent units.

2.17.3 Camber

In typical prestressed girders with composite decks, the camber does not generally change significantly once the deck is applied. But decked members do not have a composite deck, since the entire cross section including the deck is cast and prestressed at the same time. Therefore, camber growth may continue with time, which may lead to a noticeable departure of the riding surface from the intended grade line. This situation should be investigated during design. In some cases, camber growth can be reduced by providing a larger prestress force that is more nearly concentric.

Differential camber between girders can be removed as described in **Section 5.8** of this appendix.

The camber growth issue may be mitigated, and the riding surface improved, by use of a partial depth deck on the decked section. However, some of the structural benefits of using decked members are reduced. With a partial-depth decked section, the final roadway cross-section and profile can be established with the reinforced concrete that completes the deck thickness. The entire deck area can still be formed by the top flange of the decked sections, resulting in rapid construction. Camber growth may be reduced by the addition of the structural composite deck.

2.18 Stability

Designers should be aware of stability conditions that may make a long-span girder difficult to construct. References by Mast *(6,7)* should be consulted regarding the stability of long-span girders during lifting and transport. Buckling of tall webs in girders must also be considered.

The stability of erected girders must be considered as well. Plans should generally indicate that temporary intermediate diaphragms should be provided. In some cases, horizontal wind bracing may be required to maintain the stability of erected girders.

The *LRFD Specifications* indicate in a number of articles that stresses and buckling shall be investigated or considered. For example, the Specifications require that prestressed components "shall be investigated for stresses and deformations for each stage that may be critical during construction, stressing, handling, transportation and erection as well as during the service life of the structure of which they are part" (Article 5.5.1); "Buckling of precast members during handling, transportation, and erection shall be investigated" (Article 5.5.4.3); and that "Buckling of a member between points where concrete and tendons are in contact, buckling during handling and erection, and buckling of thin webs an flanges shall be investigated" (Article 5.9.1.3). However, evaluation of conditions during handling, transportation and erection are the responsibility of the contractor, as indicated in Article C5.14.1.2.1.

Therefore, it is recommended that stability and handling should be considered in a general way during design. However, the designer should not be concerned with detailed computations regarding the handling, transportation and erection of girders, issues over which he has little control and no certain knowledge.

2.19 Intermediate Diaphragms

Intermediate diaphragms at locations other than splices may not be required. Intermediate diaphragms contribute to girder stability during construction and in the final condition, and to load

distribution. However, the effect of diaphragms on load distribution is generally small. Therefore, for many bridges of typical span lengths, intermediate diaphragms have been eliminated. However, for longer spans, intermediate diaphragms may be required.

Where required, intermediate diaphragms may be cast-in-place concrete, precast concrete or steel. Precast concrete diaphragms have been used successfully when installed in one piece per bay, or when halves of a diaphragm are attached to adjacent girders.

2.20 Deck Design for Continuous Members

In continuous members that are post-tensioned after the deck has been added, the entire composite section can be categorized as a prestressed concrete member. As a result, allowable tensile stress limits are generally applied to the deck. This may lead to additional prestress being required to satisfy the stress limit. However, in continuous bridges constructed with steel, for example, the deck stress is not checked. Therefore, it is recommended that designs should satisfy stress limits to provide the benefits of a deck with little or no tension. However, this is only achievable where post-tensioning is applied after the deck is cast, which may not be permitted by some owners.

Where the deck is not post-tensioned, it cannot be considered part of a prestressed member and should not be subject to the limiting stresses for prestressed concrete. The deck design should be similar to a deck on a continuous steel girder and cracking should be expected.

2.21 Alternate Construction Methods or Designs

If permitted by the owner, the contract documents should state that alternate construction methods or designs are permitted. As is clearly demonstrated by this report, there are many different ways to design and construct a bridge for a given site. In some situations, only minor modifications of construction methods may be allowed. In other situations, modification of the superstructure design or even the entire structure may be permitted. Allowing the contractor freedom to best utilize his particular experience and equipment leads to improved economy in most cases. It also provides an opportunity for the introduction of innovative design and construction concepts.

3 Fabrication

The following issues should be considered where applicable during the design and detailing of spliced girder and other long-span post-tensioned concrete girder bridges. These issues should also be considered by fabricators of these structures.

3.1   Installation of Ducts

Post-tensioning ducts must be carefully installed to achieve the desired tendon profile, minimizing any unintentional variation in duct location. Ducts should be securely tied to supporting reinforcement or auxiliary bars to maintain alignment during placement of concrete. Duct splices must be grout tight. Where girder segments are spliced, the ducts must extend through the end form or header to ensure proper alignment with ducts in adjoining girder segments.

Strands may be placed in the ducts prior to concrete placement to assist in maintaining the tendon profile and to ensure the duct opening remains unobstructed. These strands are only used during fabrication of the segment.

3.2   Post-Tensioning Anchorages and End Blocks

The use of post-tensioning anchorages in girders typically leads to the requirement for an end block, a thickened area at the anchorage location. This enlarged portion of the girder is required to accommodate the anchorage hardware and associated reinforcement.

Where post-tensioning anchorages are required in the ends of the girders, a fabricator has two options for casting them, as described below. Designers should consider giving fabricators the option of using either approach unless special design considerations preclude the use of either approach. Different fabricators may choose different options depending on their experience, available forms, and other factors.

3.2.1   Cast in Bed

Using this option, the fabricator uses special forms on the long-line casting bed to form the end blocks containing the anchorages. This is easiest when a girder or segment requires an end block only on one end. However, fabricators can adjust form lengths to provide end blocks on both ends of a girder if necessary.

3.2.2   Secondary Cast

Using this option, a portion of the girder end where the end block will be located is not cast when the remainder of the girder is cast. After the girder is pretensioned and the beam is removed from the bed, the anchors and end block reinforcement are installed and the end block is cast in a secondary (subsequent) concrete placement. Photos of a secondary cast end block are shown in **Figure C1-10**.

3.3 Haunched Segments

See discussion in **Section 2.2.2** of this appendix.

3.4 Location of Lifting Loops for Handling

Fabricators should be given flexibility to move lifting points to locations that will be most beneficial and economical for their equipment and facility. Contractor requirements may also affect lifting loop locations, such as the desire to move the locations in from the ends of the beam to increase stability. This may need to be considered during design, since moving lifting loop locations may cause allowable stresses to be exceeded. In many cases, fabricators consider this when preparing shop drawings for girders or girder segments.

3.5 Pretensioning of Segments

Segments are typically pretensioned to counteract the self-weight of the member with additional consideration for impact during handling and transportation.

Pretensioning of spliced elements may serve little or no purpose in the completed bridge and may not be required if the segment is relatively short and lifted at appropriate locations. The use of little or no pretension minimizes or eliminates end rotations which may improve fit up at splices and may be more cost effective. If no pretensioning is used, some mild reinforcement should be provided to control potential cracking.

3.6 Assembly of Segments

If transportation and lifting of large segments is possible, it may be feasible for segments to be spliced at the plant. This may eliminate the need for temporary shoring towers for splicing. This approach has only been used for a few bridges.

Large haunched segments that have been segmented for transportation may be assembled at the plant and fitted with alignment hardware to simplify alignment in the field (see **Section 2.4.5** in the Final Report). Orientation of segments can be adjusted when alignment hardware is installed to compensate for tip deflections expected when drop-in segments are set.

4 Transportation

Designers should be aware of the following issues which are typically handled by fabricators or contractors.

4.1 Stability

Long girders must be evaluated for lateral stability during shipping. Characteristics of the truck and trailer hauling the girder and the route between the casting yard and the site must be considered in addition to the stability of the girder itself. Where the unassisted girder does not provide an adequate factor of safety, the girder may be stiffened by external trusses or by additional prestressing in the top flange. See **Section 2.18** of this appendix.

4.2 Shipping Limitations

Fabricators must be aware of shipping restrictions in the states for which they fabricate girders and through which they may have to transport girders. The limits vary significantly, even for neighboring states.

Designers must also be aware of the shipping limits that may apply for a given site so that the bridge can be constructed economically as designed.

4.3 Hauling Equipment

Where shipping limits permit larger loads, use of improved equipment can result in the shipment of larger girders, leading to increased spans for precast, prestressed concrete girders. The use of hauling equipment capable of transporting heavier and longer girders on highways is cited as one of the reasons that large pretensioned girders are now commonly used in Pennsylvania *(8)*.

4.4 Fabricate Girders on Site

For projects with a large number of girders, it may become economical to fabricate the girders at or near the site to eliminate or reduce difficulties with transportation.

The cost of setting up a plant on site is large, as well as securing reliable material sources and obtaining proper certifications and qualified personnel. Even when existing precasters set up remote operations at a job site, difficulties can be encountered with material suppliers and others. Therefore, the installation of an on-site or project-specific precasting plant is not recommended.

5 Construction and Erection

The following issues should be considered where applicable during the design of spliced girder and other long-span girder bridges. Some of the issues should also be considered by fabricators and contractors.

5.1 Falsework

Temporary supports are required in many cases for spliced girder construction. The various types of falsework that may be used are discussed below.

5.1.1 Temporary Towers

A temporary erection tower is often used to stabilize pier segments and support the girder segments in the connecting span. The pier segment must be tied to the tower to resist the uplift that occurs when the drop-in segment is placed on the other end of the segment. In some cases, both girders rest on one tower. This may introduce unbalanced forces in the tower when uplift occurs on one side while the weight of the end span is still acting down on the other side. Two separate towers may be used to address this situation. However, the most common situation is for the end span to be supported on the end of the pier girder segment by means of a strongback that rests on the pier segment. Therefore, the two reactions are combined into one, and the weight of the end span helps to balance the uplift from the placement of the drop-in span on the other end of the pier girder segment.

Temporary towers may have independent foundations, or may be supported from the permanent pier foundations. In some cases, temporary towers are braced against the permanent structure. If a permanent pier is used for support of the tower, the effect of any loads from the temporary support on the permanent structure must be evaluated.

Temporary towers typically remain in place until the girders are post-tensioned. Tie-downs to the tower may be loosened during post-tensioning to allow movement of the girders as long as stability of the girders is maintained.

5.1.2 Strongbacks

A strongback is a short structural member (usually steel) that is temporarily attached to a precast girder segment and extends beyond the end of the segment. Photographs of two examples of strongbacks are shown in **Figure C1-11** with a typical detail shown in **Figure C1-12**. The cantilever portion of the strongback rests on another segment when the girder is erected. After the girder is set, the strongback is attached to the supporting member to prevent shifting of the support. After the splice is made between girder segments and post-tensioning is applied, the strongback is removed to permit construction of the deck.

At some splice locations, such as at the connection between drop-in girders and pier girder segments, strongbacks are used instead of temporary towers. This is permissible when the pier segment

is adequately braced and supported at the opposite end by a tower or by connection to the previously completed structure. Strongbacks are also frequently used for setting end spans, as mentioned above, to simplify temporary tower design and reduce uplift at the end of the pier segment.

The connection of the strongback to both segments must be adequate to prevent relative movement between segments. Post-tensioning bars are generally used.

Web alignment bars should also be provided where strongbacks are used. These bars, which are connected as soon as the girder segment is erected, prevent rolling of the erected girder segment with respect to the supporting girder segment. Web alignment bars are shown in **Figure C1-12**.

5.1.3 *Embedded Hanger Details*

Embedded hanger assemblies have been used in a few bridges instead of strongbacks. An example is the Cazaly hanger that was used on the Shelby Creek Bridge in Kentucky *(9)*. However, use of these embedded assemblies has generally been abandoned in favor of strongbacks that are temporary, external, and reusable.

5.2 Splices

Several issues must be considered when constructing the splice for spliced girders.

5.2.1 *Placement and Curing of Concrete*

The proper consolidation of concrete in field splices is extremely important. If a diaphragm is not provided at a splice location, various measures must be taken to ensure proper consolidation of the concrete, since placement of concrete in a girder web with post-tensioning ducts is a very unusual type of concrete placement for field crews. Therefore, use of a diaphragm at splice locations is recommended to improve conditions for placement of concrete in splices.

The workability of the concrete must be controlled and proper vibration must be applied. In some instances it may be necessary to use a reduced maximum aggregate size to ensure proper consolidation. The concrete must also be properly cured in order to achieve the required strength for post-tensioning.

Forms should be removed from splice locations and the splice inspected prior to stressing tendons. Any voids or deficiencies in the splice concrete must be repaired prior to stressing post-tensioning tendons.

5.2.2 Duct Splicing

The proper splicing of post-tensioning ducts crossing the girder splice is essential for the smooth progress of construction. An improperly installed duct splice may allow grout to leak into the duct during placement of concrete in the splice. To improve the reliability of duct splices, pressure testing of ducts should be considered.

5.2.3 Mild Reinforcement Splicing

In some cases, mild reinforcement extends into splices. The reinforcement is either lap spliced or spliced using mechanical or welded connections. Methods that are most efficient in the field, considering erection and fabrication tolerances, should be used.

5.3 Stressing of Post-Tensioning Tendons

5.3.1 Qualified Personnel

The stressing of post-tensioning tendons must be performed by qualified personnel. All procedures of the project specifications must be followed closely. Any discrepancy between pressure reading, theoretical elongations and measured elongations must be evaluated and corrected before additional tendons are stressed. Safety procedures must also be developed and closely followed.

5.3.2 Staged Post-Tensioning

The use of staged post-tensioning and the associated anchorage details can have a significant influence on the construction of the bridge. Access for post-tensioning equipment must be maintained until tendons are installed, tensioned and grouted. In some cases, the erection of adjacent spans may be delayed until after the post-tensioning operations have been completed. For bridges where the girders are post-tensioned at the abutments, completion of backfilling and diaphragm construction may also be delayed until all post-tensioning operations are finished. Usually, the placement of deck concrete is also affected by the staging of post-tensioning. The anchorages are usually encased in the final deck or endwall pour.

5.4 Grouting of Ducts

Post-tensioning tendons must be grouted by qualified personnel using appropriate equipment. All procedures of the project specification must be followed closely. It is essential that grouting be performed and inspected properly to ensure the intended structural performance and durability.

A significant body of information is available on this issue from industry and owner agency sources.

5.5 Protection of Anchorages

Following grouting, the post-tensioning anchorages must be protected and encased according to the project documents. Long-term performance of the structure depends on the successful completion of this activity. See **Figure C1-13**.

5.6 Deflection of Segments

The preparation of construction details must consider the deformations of the structure as segments are added and post-tensioning is applied.

For example, as drop-in girders are placed on pier segments, the tip of the pier segment will deflect. The direction and magnitude of the slopes at the ends of the pier segment and drop-in will not be the same. This must be considered in detailing strongbacks, girder ends or other connections.

As segments are added or post-tensioning is applied, girders will move relative to adjacent girders. Any temporary or permanent bracing must be able to accommodate this movement without damage or loss of capacity. The stability of adjacent girders must not be affected by this movement.

5.7 Stability

5.7.1 Lifting Segments

The stability of girders or segments during erection must be considered. Additional stiffening of the member may be required, or special considerations or equipment used during lifting. The effect of inclined lifting leads can be significant for slender girders and should be considered.

5.7.2 Segments with Strongbacks

The stability of segments erected using strongbacks must be carefully considered. This is especially an issue when the segments are placed on a curve, so there is an angle change in plan between segments. Web bars or some other means to prevent rotation of girder segments are required to maintain stability of the segments.

5.7.3   Temporary Bracing

After erection, bracing should be provided to prevent movement of the girders. Horizontal wind bracing may also be required in some situations. The bracing may be removed after the deck is placed if not required to remain in the structure.

5.8   Leveling Decked Bulb-Tee Bridge Girders

A simple procedure has been developed for removing small differential cambers between adjacent decked members. One or more threaded inserts are cast into the top surface of all girders at a designated location or locations in the span. After the girders are erected, threaded rods are placed in the inserts and a structural member is placed across three or more girders. The structural member is typically a pair of back-to-back channels spaced apart to provide clearance for threaded rods to pass between them. At locations where a girder is low, a small center-hole ram is placed on the threaded rod and the girder is pulled up to match the adjacent girders. The welded connections between girder flanges are then made to maintain the deck at the proper elevation.

5.9   Launching Trusses

While not commonly used in many parts of the country, launching trusses can be used efficiently in many situations to erect girders or girder segments and reduce requirements for lifting equipment or to address site access limitations. See the article on the Rock Cut Bridge *(10)* for an example of a project using a launching truss.

6   References

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Figures:

a) Constant Web Depth

b) Varying Web Depth

Figure C1-1:   Haunched Girder Segments

Figure C1-2:   Combined Pretensioned Strands and Post-Tensioning Tendons
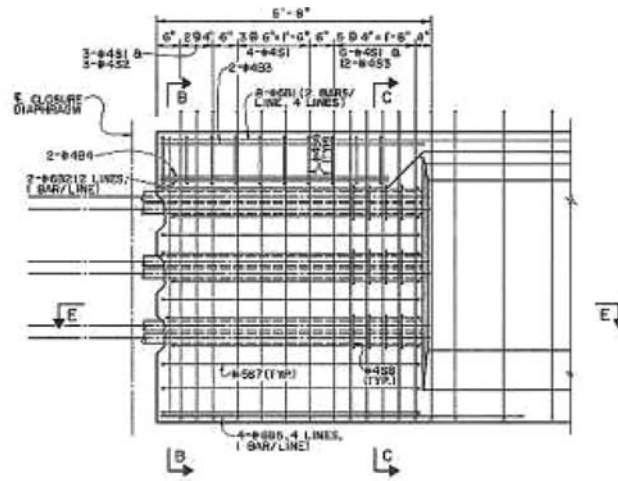
a) Elevation
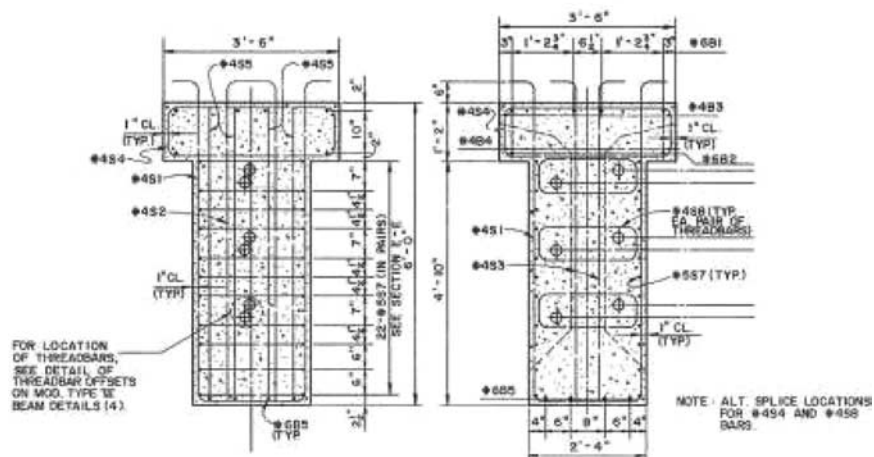
b) Section B-B   c) Section C-C
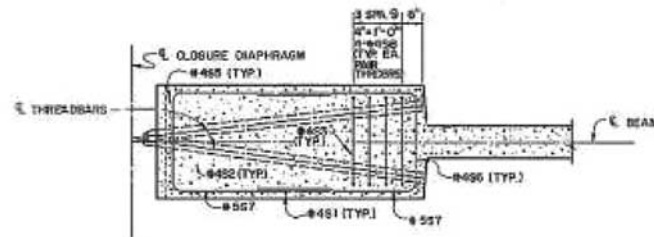
d) Section E-E

Figure C1-3:   Details of Stitched Splice

C1 -39

| a) Lap Spliced Mild Reinforcement | b) Mechanically Spliced Mild Reinforcement |

Figure C1-4:   Cast-in-Place Splice

| a) Machined Bulkhead | b) Applying Compression Across Splice |

Figure C1-5:   Match-Cast Splice

C1-40
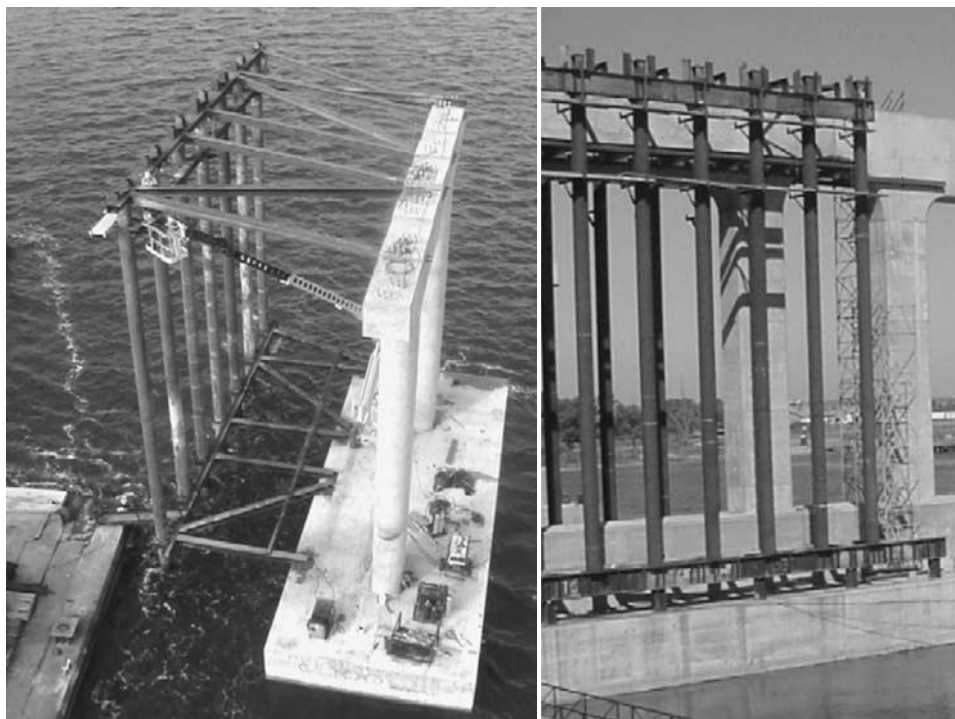
Figure C1-6:   Examples of Temporary Shoring Towers

Figure C1-7:   Details of Temporary Shoring Towers

Expansion Joint

Minimum 1-1/2" over grout cap

Minimum clearance distance between ducts 1.5 * max aggregate size or 1-1/2"

Embed end of girder 12"

Min 6"

Four levels of protection provided by
- Grout
- Plastic cap
- Pour-back material
- Encasement by diaphragm
Anchors inaccessible for maintenance inspection
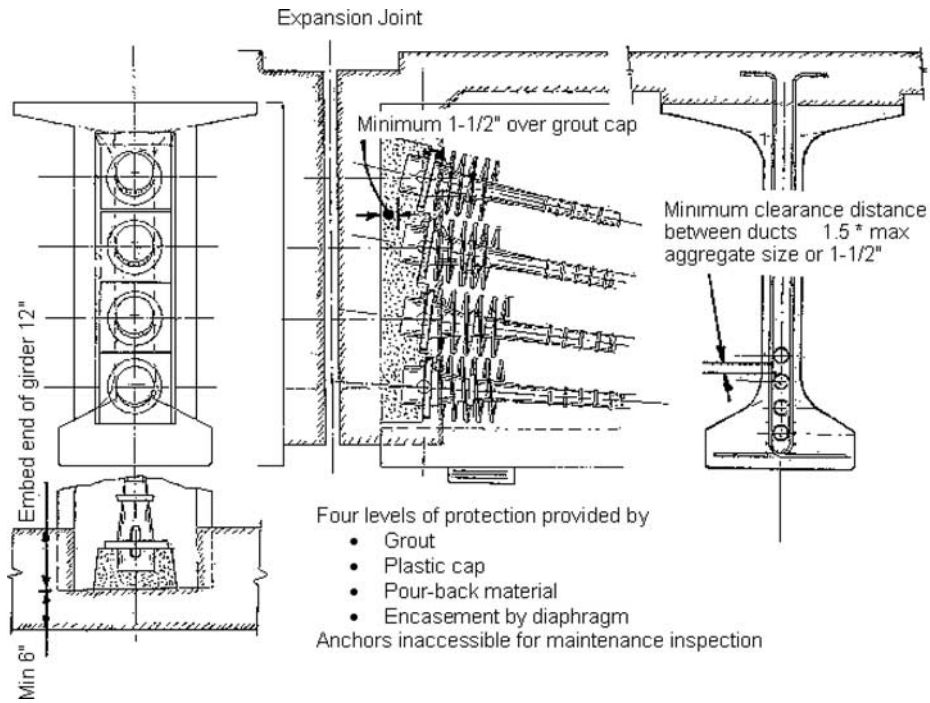
a) Anchorage in End of Girder

Deck slab – pour full width

Construction Joint

1-1/2" min cover over cap

No more than 50% of PT to anchor in top of girder

Four levels of protection provided by
- Grout
- Plastic Cap
- Pour-back Material
- Structural Deck Slab
(Anchors inaccessible for maintenance inspection)
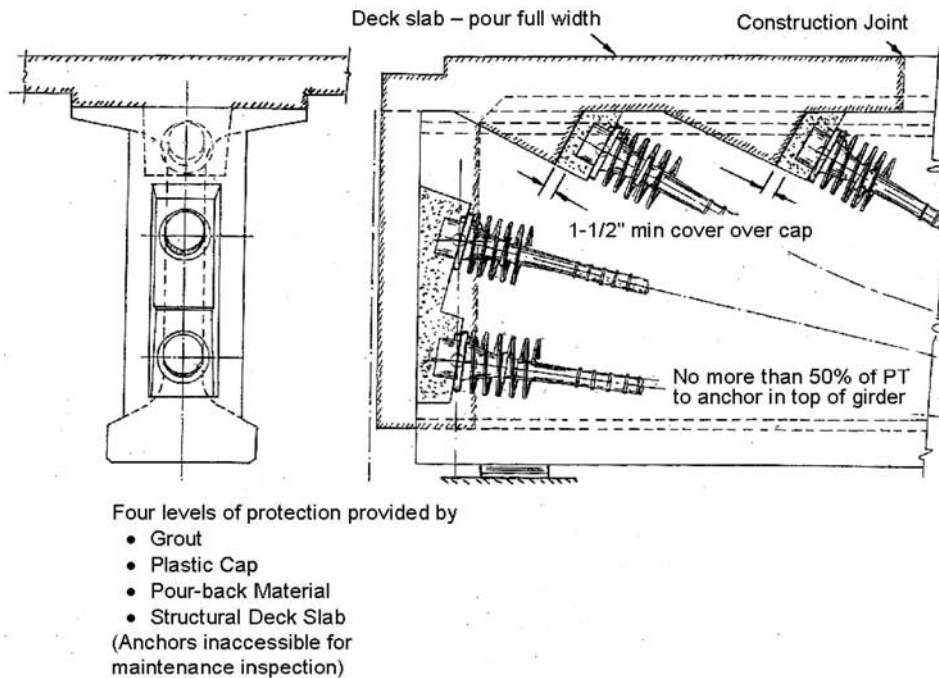
b) Anchorage in End and Top of Girder

Figure C1-8:   Details of Anchorages in Girders (FDOT, 2002)

C1-42

a) Reinforcement in Anchorage   b) Completed Anchorage

Figure C1-9:   Anchorage in Girder
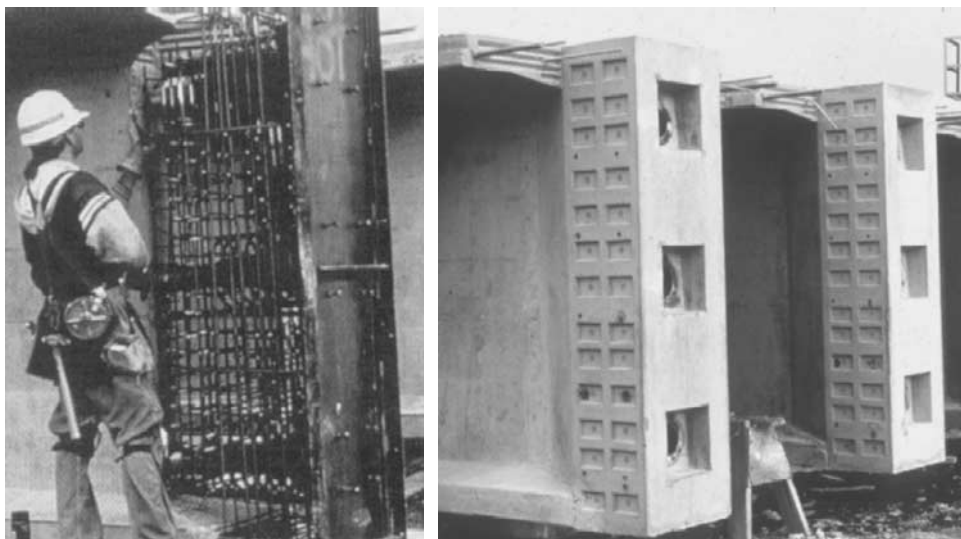
a) Anchorage Reinforcement   b) Completed Anchorage

Figure C1-10:   Secondary Cast End Block in Girder
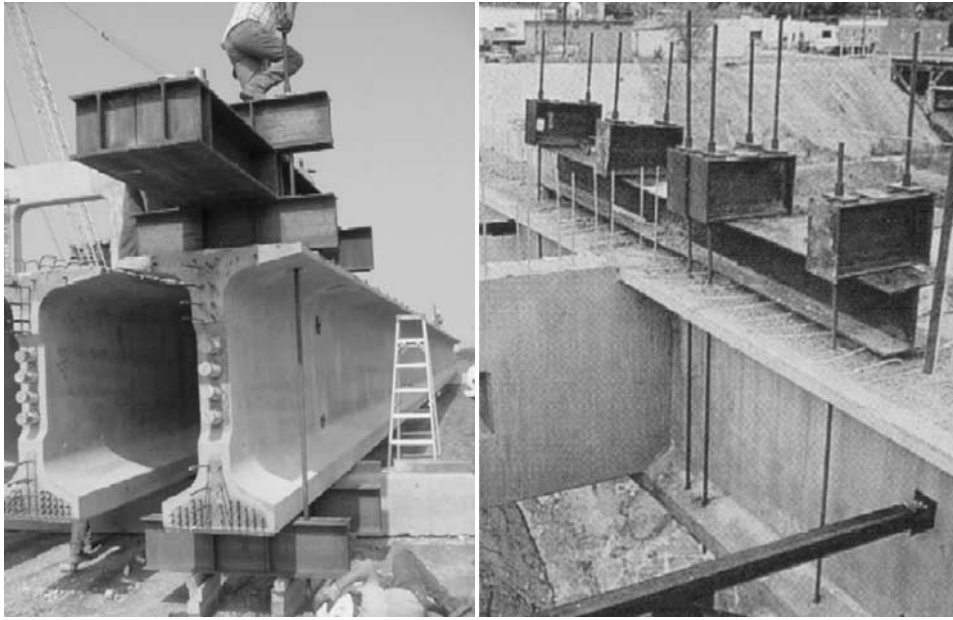
a) Installed on Girder Segment   b) After Erection and Splicing

Figure C1-11: Strongbacks

a) Elevation   b) Section A–A
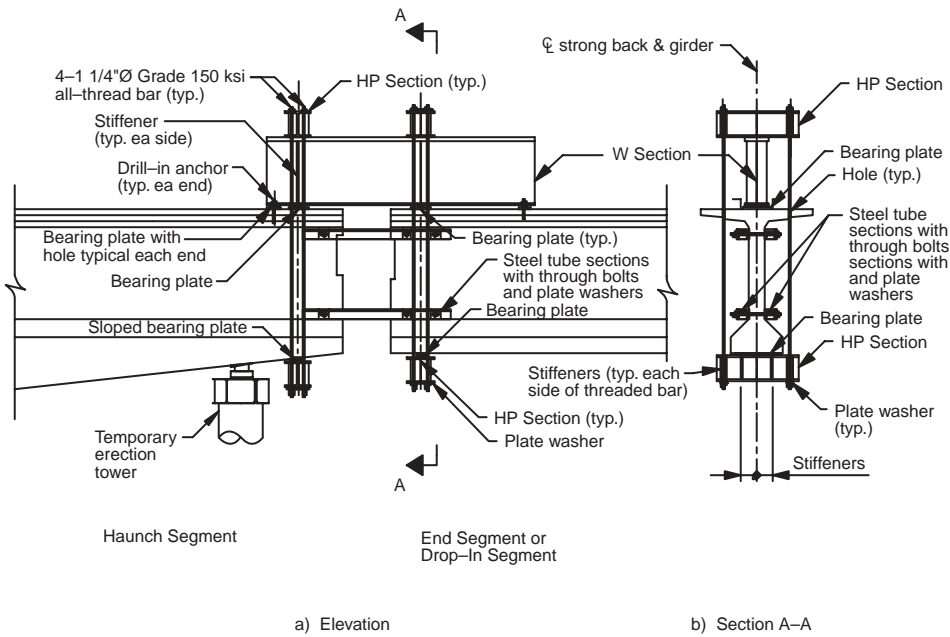
Figure C1-12: Details of Strongback
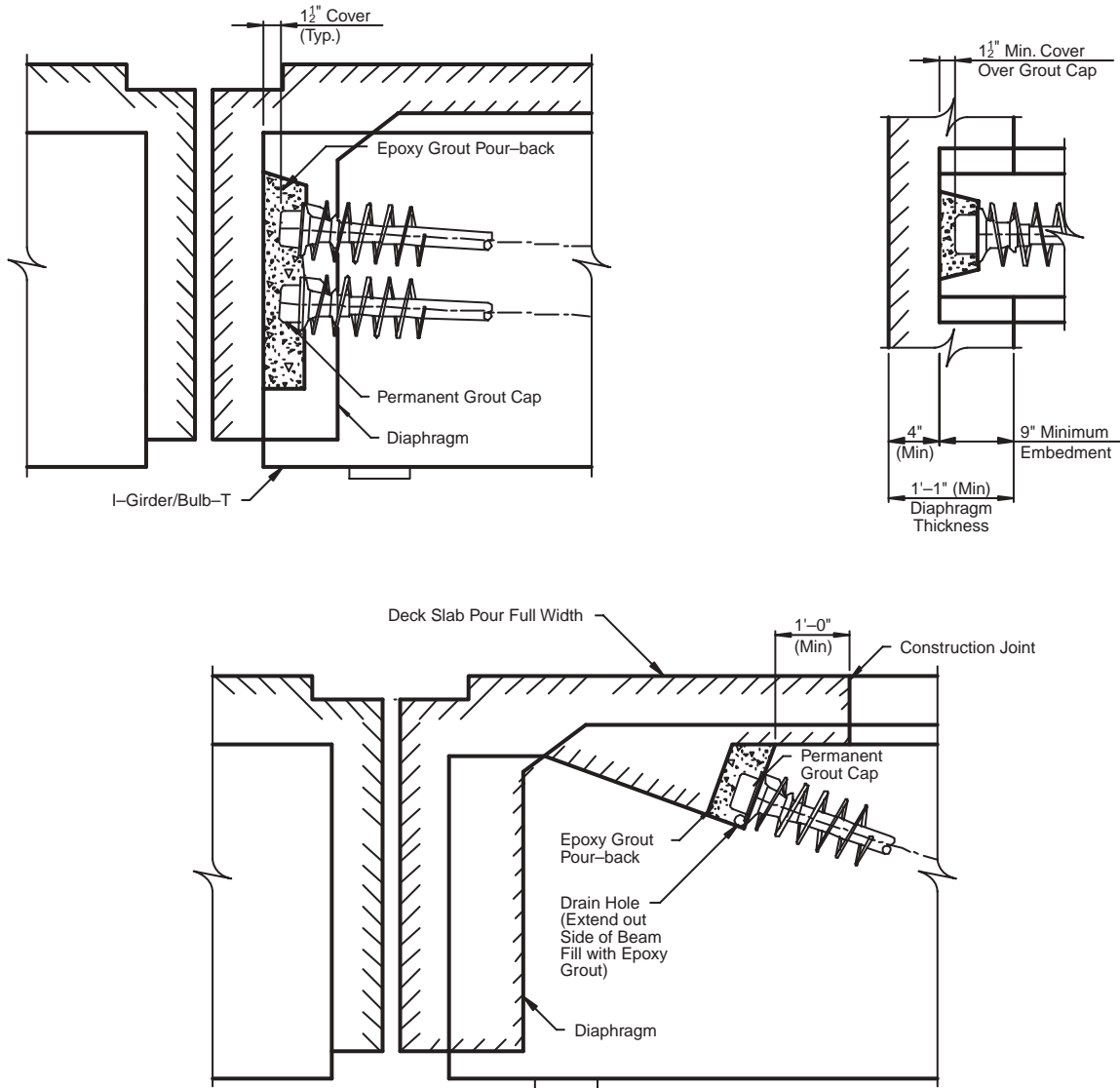
Figure C1-13: Anchorage Protection (FDOT, 2002)

APPENDIX C2

HISTORY OF FLORIDA'S SPLICED GIRDER BRIDGES

APPENDIX C2
HISTORY OF FLORIDA'S SPLICED GIRDER BRIDGES

1 INTRODUCTION

The Florida Department of Transportation (FDOT), in cooperation with the Federal Highway Administration (FHWA), has been a pioneer in the design and construction of precast prestressed spliced concrete girder bridges.

In the mid 1960's, FDOT, in conjunction with Howard Needles Tammen Bergendoff (HNTB), developed a spliced girder system utilizing reinforced end-span splices and central span hinges. The first project, SR-510 over Sebastian Inlet, was followed by the construction of two other similar bridges within the state. While these three bridges were not post-tensioned, they still utilized splicing techniques to extend the span of precast prestressed concrete bridges.

Beginning in 1977 with a two-span AASHTO Type IV spliced girder bridge over I-10 in the Florida panhandle, FDOT has completed 23 projects using precast spliced concrete girder bridges with post-tensioning. Four of these projects included dual bridges. Therefore, at the time of this publication, Florida is home to 29 spliced post-tensioned concrete girder bridges (two of which are currently under construction).

Therefore, the total number of spliced girder bridges in Florida exceeds 30.

The following is a chronological list of spliced girder bridges in Florida, with the name and location for each project. In the list, the Intracoastal Waterway is abbreviated as "ICWW".

- SR-510 Over Sebastian Inlet (Sebastian, FL) (constructed about 1968)
- Duplicate Bridges of Sebastian Inlet Bridge (Pensacola, FL / Ft. Lauderdale, FL)
- Chipola Nursery Road Over I-10 (Marianna, FL)
- Buffalo Ave. Over I-75 (Tampa, FL)
- Martin Luther King Blvd. Over I-75 (Tampa, FL)
- Three Bridges on I-75 Over Miscellaneous Roadways (Tampa to Naples, FL)
- SR-518 Over Indian River / Eau Gallie Bridge (Melbourne, FL)

- I-275 Over Tampa Bay / Howard Frankland Bridge (Tampa/St. Petersburg, FL)

- US-331 Over Choctawhatchee Bay (Destin/Panama City, FL)

- US-41 Over Caloosahatchee River / Thomas A. Edison Bridge (Ft. Myers, FL) (Dual bridges)

- SR-71 Over ICWW / Highland View Bridge (Highland View, FL)

- US-17/92 Over St. Johns River at Lake Monroe (Sanford, FL)

- SR-60 Over ICWW / Merrill Barber Bridge (Vero Beach, FL) (Dual bridges)

- SR-A1A Over ICWW / Vilano Beach Bridge (St. Augustine, FL)

- SR-100 Over ICWW / Flagler Beach Bridge (Flagler Beach, FL)

- US-41 Over ICWW / MacArthur Causeway Bridge (Miami, FL) (Dual bridges)

- SR-600 Over Tampa Bay / Westbound Gandy Bridge (Tampa/St. Petersburg, FL)

- Anna Maria Island Bridge Over ICWW (Bradenton, FL) (designed but not built)

- SR-292 (Pace Blvd.) Over Bayou Chico (Pensacola, FL)

- US-27 Over Okeechobee Waterway / Moore Haven Bridge (Moore Haven, FL)

- SR-312 Over Matanzas River (ICWW) (St. Augustine Beach, FL)

- SR-10 / Atlantic Blvd. Bridge Over ICWW (Jacksonville, FL)

- I-95 Over St. Johns River / Fuller Warren Bridge (Jacksonville, FL) (Dual bridges)

- SR-210 Bridge / Palm Valley Bridge (south of Jacksonville, FL)

- Wonderwood Connector over ICWW (Jacksonville, FL) (currently under construction)

- SR-300 Over Apalachicola Bay / St. George Island Bridge (Eastpoint, FL) (currently under construction)

During construction of the first three post-tensioned spliced girder bridges beginning in 1977, each of the bridges experienced difficulty with post-tensioning (P/T) due to misalignment of the P/T ducts. Numerous girders developed cracked webs and the construction time and cost was substantial. As a result, FDOT discontinued use of the system. In 1986, the fourth spliced girder bridge project was constructed, the Eau Gallie Bridge over Indian River, beginning a new era in spliced girder technology that continues today.

Several excellent articles (1, 2, 3, 4, 5) have been published regarding the Florida Bulb-Tee (FBT) spliced girder system. However, the text of this publication is based upon the author's personal

experience while serving for 27 years as a structural engineer with the prestressed concrete industry in Florida, design consultants and with FDOT. The author has been privileged to be associated with many of the firms and individuals that are referenced in the following text.

2   HISTORY OF EARLY SPLICED GIRDER SYSTEM

In 1975, FDOT was approached by Mario Suarez of StressSteel Corporation, a major supplier of post-tensioning products at that time. Mr. Suarez and his engineering department had developed details for a system of using prestressed AASHTO girders with P/T ducts embedded within the webs. FDOT decided to use the system, and the Chipola Nursery Road Bridge over I-10 was constructed in 1977. The bridge consisted of two 127 ft (39 m) spans of Type IV girders that were spliced with a poured concrete joint at the interior pier. The girders were simple spans at erection and post-tensioned after the deck was cast for continuity under live load and dead loads imposed on the composite section.

A desirable feature of the longer spans was the ability to remove the "shoulder piers" of the typical interstate overpass bridge while maintaining the shallower depth Type IV girder. The traditional design used three spans with end abutments and three interior piers. The spliced girder system removed the need of the piers each side of the interstate and provided improved safety.

As noted above, five other bridges were constructed in Florida using spliced AASHTO girders. Two bridges used three girders spliced at interior points of the two span structures. The other three bridges used four girders spliced at interior points and the interior pier of the two span structures. Each of these projects experienced some problem with the P/T ducts being misaligned. Many of the girders developed cracks and "pop-out" of the webs during post-tensioning. Repairs were laborious and expensive, and FDOT discontinued use of the system until advent of the Eau Gallie Bridge. However, recent maintenance inspections of these bridges have shown favorable results and proper function of the structures.

3   HISTORY OF CURRENT SPLICED GIRDER SYSTEM

During 1982, Henry T. Bollmann, serving as FDOT State Structures Design Engineer was in charge of all bridge design. Mr. Bollmann was instrumental in convincing the Florida Legislature to provide funding for a FDOT Structures Research Laboratory. He then retained Paul F. Csagoly as Chief Engineer of Structural Research and Development. Mr. Csagoly had previously held a similar position with the Ontario Ministry of Transportation and Communications in Canada and chaired the first task

force for the *Ontario Highway Bridge Design Code*. Mr. Csagoly and FDOT Engineer Paul W. Beckham were the primary designers of the Eau Gallie Bridge.

Several factors prompted Mr. Bollmann and FDOT to consider again the use of a spliced girder system. A complaint of the general public, and of course a concern to FDOT, was that many AASHTO girder bridges in Florida have a "washboard" feel of the riding surface while driving. This is due to the girders having an increased camber growth after the deck is placed during construction. Therefore, the deck surface at mid-span of the girders is higher than the intended profile. The condition is common due to conservative design by previous AASHTO bridge codes, conservative prestressing strand loss calculations, use of strand that was not low-relaxation, and the high temperature range in Florida. The public at times became alarmed when driving, in particular on the longer spans of lengthy bridges such as the Cross-town Expressway in Tampa. Mr. Bollmann believed that something must be done to alleviate this problem.

The main impetus to increase the span lengths of concrete girders was implementation of ship impact requirements. The tragic collision of a cargo ship with the Sunshine Skyway Bridge over Tampa Bay occurred in 1980. FHWA and FDOT immediately began applying design lateral forces on bridge piers that were based on studies performed by the Louisiana DOT. The use of this additional design requirement significantly increased the cost of bridge piers and foundations. The "Louisiana design loads" were even greater than the loads now specified in the AASHTO bridge design specifications. Ship impact design truly changed the manner in which designers determined the optimum bridge type for a given location.

In addition to the increased cost of bridge substructures due to lateral loads, the issue of ship impact also resulted in requiring increased spans lengths for main spans over a navigation channel. The required horizontal clearance between piers each side of the channel increased significantly due to additional offset of the foundations from the fender system. Adding to this increased span was also the factor that pier foundations were much larger due to the ship impact design loading.

All of the spliced girder bridges designed and constructed from the Eau Gallie Bridge in 1986 to the present have significant ship impact loading, or would have if the span had not been increased enough by using spliced girders to eliminate the ship impact loading *(4)*. The majority of the bridges are over the Intracoastal Waterway (ICWW) along the Florida coastlines, and all others are at locations with cargo barge traffic on rivers and interior waterways.

Another factor for desiring greater span lengths, although more philosophical in nature than protection of the traveling public, is that of aesthetics. FDOT and FHWA agreed that a sleek depth to span length ratio is desirable and worthy of moderate additional design and construction cost. Coincidentally, this factor became pronounced during the same time period as the other factors previously noted.

Therefore, in 1983 under the direction of Henry Bollmann, Paul Csagoly and FDOT engineering staff began development of what is now known as the Florida Bulb-Tee (FBT) girder shapes. Mr. Csagoly performed parametric studies *(6)* to determine the optimum cross-sections for use of the girders as prestressed, post-tensioned multi-span units with continuity. Three heights of girder, similar to the AASHTO Type IV, V and VI girders, were initially developed. However, only the 6 ft (1.8 m) height section was used extensively. The 4.5 ft (1.4 m) section was used on several bridges as a more economical simple span section than the AASHTO Type IV girder. A new Modified Bulb-Tee (MBT) girder was introduced in 1993 and will be discussed later in this text.

The Eau Gallie Bridge girder design utilized four-span units with 145 ft (44 m) span lengths. Construction occurred from 1985 to 1987 and at a cost of about $35 per square foot of bridge deck area. The girders were simple span from pier to pier under their own weight during erection. First stage P/T was performed on the girders after closure pours were made at the three interior piers. The deck was then cast and second stage P/T was performed after the required concrete strength was achieved. All other post-tensioned spliced girder bridges designed and constructed have followed the same basic procedure.

Load testing of a prototype girder line was performed in 1985 at Gate Concrete Products Company in Jacksonville prior to beginning full production. The testing showed substantial reserve structural capacity. Failure occurred by crushing of the concrete in the bottom flange at the interior support due to negative bending moment, but only after obvious visual signs of distress (deflection, cracking, etc.) appeared long before failure. The completed bridge was load tested in 1990 by the FDOT Structures Research and Testing staff under direction of Mohsen A. Shahawy *(7)* and the results were impressive. The bridge also does not exhibit the "washboard" effect of the riding surface. All spliced girder bridges constructed to date have also been without the "washboard" effect.

The next bridge to utilize spliced FBT girders was also a landmark case in demonstrating the viability of the system in all respects – structural adequacy, economy and aesthetic nature. The bridge is I-275 over Tampa Bay and is known as the Howard Frankland Bridge. Spanning between

Tampa and St. Petersburg, the length is 16,000 ft (4880 m) with a width of 71 ft (22 m). At a projected cost of over $40,000,000, FHWA and FDOT decided to provide three alternate designs for bidding: a combination of span-by-span and balanced cantilever concrete segmental box girder; a combination of spliced concrete girder approaches with a steel three-span channel span; and a spliced FBT girder system for the entire length. Bids for the project were received in 1987.

Four contractor firms bid the project, with one bidding the segmental, none bidding the steel and three bidding the spliced FBT girder. The low bid was for the FBT girder alternate with the segmental bid being 5% higher as the second low bid. The contractor bidding the segmental alternate believed that it would have been more economical if it had used only span-by-span erection rather than a combination with the balanced cantilever spans. Those spans were designed in the highest ship impact region in order to reduce the number of foundations. Regardless, at a cost of about $37 per square foot, the spliced FBT girder system had again demonstrated economy. In addition, segmental construction is at its most economical when the project is massive, such as the Howard Frankland Bridge. Therefore, segmental construction had a better chance at economy for this project than it would have for a typical, smaller sized waterway crossing.

Construction of the bridge went well with the exception of having to increase the web thickness of the girders from the design value of 7.0 in (0.18 m) to 7.5 in (0.19 m). As a result, all other projects designed have been detailed utilizing the actual diameter of reinforcing bars (i.e., with the bar deformations) and P/T ducts, and have accounted for placement tolerances.

The Choctawhatchee Bay Bridge serves as another landmark for the spliced FBT girder system by having the first three-span continuous "drop-in" FBT channel unit in the state. Design was completed in 1988 and consisted of four-span approach units of spliced girders. The channel unit was designed as a three-span unit made of five separate FBT girders: an end span girder for the first and third spans, a center girder for the main span and a "haunched depth" girder at each interior pier. The span lengths are 160 ft (49 m), 200 ft (61 m) and 160 ft (49 m).

The 200 ft (49 m) channel span would normally have been of steel plate girder for a bridge of this size. However, due to the extremely aggressive salt water environment, it was desirable to have a concrete structure. The project size was not considered large enough for an economical segmental box bridge. In addition, FDOT wanted to pursue this concept through design and construction in order to gain the experience and knowledge of the system's viability. Jack O. Evans, FDOT Bridge Engineer, had experience with a similar design and was confident of the system. Therefore, FDOT requested that

FHWA would not require alternate designs and that alternates designs would not be considered after bids were received. FHWA agreed and a very successful structure type was created as a result.

As of this date, more than 10 bridges have been constructed in Florida utilizing the drop-in spliced FBT girder system that was developed for the Choctawhatchee Bay Bridge. The economy of the system for channel span units was demonstrated dramatically during 1997 on the Moore Haven Bridge (US-27 over the Okeechobee Waterway) project. The design plans contained simple span Modified Bulb-Tee (MBT) girder approaches with a steel plate girder three-span channel unit. The span lengths of the channel unit are 215 ft (66 m), 320 ft (98 m) and 200 ft (61 m). The contractor firm proposed to FDOT through the Value Engineering Change Proposal (VECP) process to change the channel unit from structural steel to a drop-in MBT girder unit.

FDOT and FHWA studied the proposed design in great detail, as there was concern of going to a channel span much greater than previously used. The 320 ft (98 m) span would far surpass the 250 ft (76 m) span of the Highland View Bridge, the record at that time. In addition, the contractor firm's Specialty Engineer, Janssen & Spaans Engineering, Inc., proposed to use a constant depth bottom flange with a variable depth web for the haunched girders. All similar projects to this point had used a variable depth bottom flange and a constant depth web for the haunched girders, and there was concern regarding web crippling. Janssen & Spaans Engineering, Inc. had also designed the Highland View Bridge in 1991 *(4)* with great success, and FDOT retained the Engineer of Record, Beiswenger Hoch and Associates, to assist with the review.

The variable depth web was proposed by the contractor in order to reduce the weight of the haunched girders for erection equipment considerations. All review studies proved the proposed design to be adequate, and FDOT and FHWA approved the VECP. The contractor proposed savings of $621,000 to be split evenly with FDOT. Therefore, a significant savings resulted by use of a drop-in spliced concrete girder channel span unit rather than structural steel plate girder.

It should be noted that structural steel girders are expensive in Florida (and most of the Southeast) as compared to prestressed concrete girders. Concrete is a product that is "native" to Florida. Also, the prestressed concrete industry in the state has non-union labor whereas the steel industry is highly union labor. Steel plate and box girder bridges are used in Florida for highly curved and long span bridges, but steel cannot compete with concrete girders where they can be used. Regardless, the significance of the Moore Haven Bridge VECP for the channel unit must be recognized.

The bid results of the Edison Bridges and the MacArthur Bridges were also significant from another perspective. Each of these projects contained contract documents for a precast concrete segmental box girder alternate and also for the post-tensioned FBT72 girder alternate. For the Edison Bridges, only one contractor of eight bidders quoted the precast segmental box alternate. For the MacArthur Bridges, no contractors quoted the segmental box alternate. These results confirmed the economy of the spliced girder systems developed in Florida.

The MBT girder was introduced to FDOT in 1993 through a VECP proposal by the contractor firm constructing the Golden Glades Interchange Viaduct in Miami. Janssen & Spaans Engineering, Inc. was also the Specialty Engineer for the contractor and had used the 78 in (2 m) height girder elsewhere in the United States. As well as being a deeper girder than the FBT girder, the MBT section has a 5 ft (1.5 m) width top flange versus a 4 ft (1.2 m) top flange for the FBT. In the author's opinion, the MBT girder is an extremely efficient cross-section for both simple span use and for use in spliced girder continuous spans.

The Golden Glades Interchange Viaduct was designed with two alternate structure types for bidding: a concrete span-by-span segmental box alternate and an AASHTO Type VI simple span girder alternate. All bidders chose the Type VI girder option. However, the more efficient MBT girder allowed the VECP proposal to remove a girder line for the entire bridge length. The MBT cross-section contains only 2% more area (therefore, also concrete) than the AASHTO Type VI girder.

Even though the Golden Glades Interchange Viaduct did not utilize spliced P/T girders, it is significant in having introduced the MBT girder in Florida. The MBT girder has in essence replaced the FBT girder. Several projects that were designed with the four-span units of spliced FBT girder had VECP proposals approved to substitute simple span MBT girders. Contractors prefer not having the time and expense of P/T operations in the field unless absolutely necessary.

In conclusion, for waterway crossings subject to ship impact, the MBT girder is the "hands-down" winning girder in Florida for both simple spans and spliced girder continuous spans. Spliced girders have and will continue to remain the optimum structure type for channel span units. In fact, the new St. George Island Bridge in the Florida panhandle contains a five-span channel unit. The bridge is currently under construction.

4   DESIGN CONSIDERATIONS

All of the spliced post-tensioned girder bridges in Florida have been designed utilizing time-dependent analysis of the structure, from the construction phase to "day infinity". It is crucial that time-dependent properties for creep and shrinkage factors be properly estimated and utilized during the design. Failure to do so would result in underestimating moments, shear and stresses. Underestimating creep and shrinkage differential between the girders and the cast-in-place deck slab can result in excessive effects in the positive moment regions. Overestimating creep and shrinkage differential can result in excessive effects in the negative moment regions.

It is also crucial that all phases of the construction erection method be accounted for in design. The use of temporary supports, "strongback" hanger devices, tension tie-down locations and other items are significant to developing induced built-in moments, shears and stresses to the girders. Therefore, the designer must develop an assumed erection method, preferably with the assistance of a contractor experienced in the erection of spliced girders. This erection method must be clearly shown in the contract plans. If a contractor were to propose a different erection method, the contract documents should make it clear that a re-analysis may be necessary.

Other design parameters that are crucial to be estimated properly are friction and wobble coefficients for the post-tensioning tendons. Originally, FDOT used values of 0.25 for friction and 0.0015 for wobble. These values, in particular the value for wobble, are conservative. Friction tests on the bridges have shown actual values from 0.23 to as low as 0.13. FDOT continues the use of the 0.25 value for friction but reduced the wobble coefficient to a 0.0002 value. This results in a more realistic estimation of the final post-tensioning forces in the structure.

Based on variability of the parameters noted above, the proper design of a spliced girder bridge must account for "upper and lower bounds" of these factors. Therefore, the use of a time-dependent, phase sequencing and highly detailed computer program is of utmost importance. Designers of the bridges in Florida have used programs such as "BC" (Bridge Construction), "BRUCO" (Bridge Under Construction), "BD II" (Bridge Designer II), "TANGO" and "ADAPT". Other programs have been developed and are currently being developed. The author has been involved in designs performed by most of these programs. It has been interesting to compare the results of one program versus another and find the analyses to be within +/- 2% of each other.

An excellent source of information on these and other programs can be viewed at the Internet websites www.bridgepros.com and www.bridgesite.com.

5 FABRICATION CONSIDERATIONS

General fabrication techniques for spliced girders are very similar to conventional prestressed concrete girders with the exception of placing the post-tensioning ducts. The main difference versus typical fabrication is the prestressing of the top flange of the haunched pier girders. Fabricators must have stressing abutments or self-stressing forms that are capable of resisting a significant total prestressing force at a centroid location high above the bottom of the flange. This requires specialized equipment and tends to limit the production of these girders to the larger fabricator firms.

Proper placement and supporting of the post-tensioning ducts are critical and require intense care of labor and inspection personnel Most problems with spliced girder bridges in Florida have been related to improper placement of ducts during fabrication.

Another problem with girder fabrication in Florida, particularly on the initial projects, has been improper concrete cover on reinforcing steel in the flanges and web. The design and detailing of the first projects in Florida used the "nominal" rather than "actual" dimensions of reinforcing steel diameters. Also, the designers and detailers did not consider proper bar bending tolerances. Along with the factor of allowable placement tolerances of reinforcing steel, improper concrete cover and reinforcing steel placement resulted.

Therefore, the factors of reinforcing steel fabrication, placement and other tolerances must be considered when developing the girder cross-section dimensions. The author highly recommends that an additional width of the web be utilized in order to alleviate most problems. The nominal amount of resulting concrete in the cross-section can be very "cheap insurance" to problems during fabrication.

6 ERECTION AND CONSTRUCTION CONSIDERATIONS

As noted above in "Design Considerations", the development and detailing of proposed erection equipment, means and methods are crucial. However, often a contractor will propose an erection scheme that differs significantly from the one used in the design. If so, the contract documents should require that the contractor be responsible for a re-analysis of the structure if deemed necessary by the owner. In Florida, it is required that the contractor retain a "Specialty Engineer" to perform this design.

Also, a "Specialty Engineer" is required to design, detail and inspect installation of all erection equipment used by the Contractor. The Engineer-of-Record and the owner should not be liable

for the design of equipment that does not remain part of the permanent structure. FDOT has developed specifications over the years with the assistance of FHWA and other parties. These specifications have been invaluable during spliced girder bridge construction.

The use of temporary support towers and "strongback" hanger support devices has been very successful. Temporary support towers have typically consisted of steel H-piling driven at the end splice locations and towers built up to the point of the beam bearing elevation. However, recently on three projects, the temporary supports were attached to the central span pier foundation slab. This resulted in improved economy for the contractor as well as removing the structure from the waterway and work areas.

7   SPECIFICATION CONSIDERATIONS

Due to the extensive use of post-tensioned bridges in Florida, both of spliced girders and of segmental boxes, FDOT has committed significant resources to inspection and maintenance. A main consideration for such efforts is the fact that most of these bridges are located in the high chloride extremely aggressive environment of the coastal regions.

The information obtained over the years has resulted in numerous changes and additions to the FDOT specifications, design criteria and inspection procedures for post-tensioning. This information is available at no charge from the FDOT website. Specifications are available at www11.myflorida.com/specificationsoffice/ and design and inspection criteria are available at www11.myflorida.com/structures/.

8   MAINTENANCE CONSIDERATIONS

Bridges in Florida are inspected by FDOT at least every two years. Inspection records for the spliced girder bridges have shown very favorable results. The main items that differ from conventional concrete girder bridges are post-tensioning and bearing pad types. These items have shown favorable service on spliced girder bridges; however, they have not been favorable on several segmental box girder bridges and this has been the topic of much discussion throughout the engineering and construction communities. It is imperative to separate the two structure types when discussing maintenance issues, and not to group them together based on that they are each post-tensioned.

Recent inspections of several spliced girder bridges in the Tampa and southwest Florida area have demonstrated that the post-tensioning anchorages are performing well. The anchorage areas on some segmental bridges have shown evidence of improper grouting of the post-tensioning ducts.

In regard to the post-tensioning anchorage locations, the author highly recommends that the ducts be located within the center of the web and remain so, not flared at the ends of the girders. Therefore, the anchorages remain in the center of the girder and exit at the end face or through the top flange. This reduces the possibility of splitting or spalling due to lateral bursting stresses induced by a flared tendon trajectory. A flared trajectory also results in more difficult placement during fabrication.

FDOT has published extensive reports on the inspection and maintenance of the post-tensioned bridges in Florida. This information is entitled "New Directions for Florida Post-tensioned Bridges" and is contained in 10 volumes, each of which relates to a specific bridge type. For spliced girder bridges, "Volume 4 of 10: Design and Construction Inspection of Precast Concrete Spliced I-Girder Bridges" *(8)*, is applicable. This is available at no charge from the FDOT website www11.myflorida.com/structures/.

9   REFERENCES

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APPENDIX D
Design Examples

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APPENDIX D

DESIGN EXAMPLES

1   INTRODUCTION

Design examples are important resources that enable a designer to approach a new type of design with increased confidence because they clearly present all of the required issues and procedures necessary to complete the design. Since design of spliced girder bridges, which is the major focus of this study, involves greater complexity in design that is required for conventional precast prestressed concrete girder designs, design examples were developed to facilitate the use of this type of construction, which has great potential for extending the span ranges of precast prestressed concrete bridges.

Three design examples are presented in this appendix, representing configurations of spliced girder bridges that can be applied to a wide range of applications. A brief introduction to each design example is presented in this introduction, following several general comments.

The level of detail differs between the three design examples. In the first design example, which represents the simplest structure, the basic concepts are thoroughly presented for all aspects of spliced girder design. For aspects of design where the design of spliced girders is the same as conventional precast prestressed concrete girders, detailed design calculations are generally not given. The subsequent design examples focus on the issues specific to multiple span spliced girder bridges and generally refer to Design Example 1 rather than repeating detailed discussions of the basic concepts that have already been presented. In all examples, an effort has been made to state all assumptions, so that the examples will be as clear and useful as possible for the user.

The examples are intended to be a reasonable solution for the site and conditions described. They may not be the absolute best solution for the bridge type presented. There are many aspects of design that can be altered, such as the cross-section type, the sequence and method of construction and the sequence of post-tensioning. For example, priorities of design, such as deck removal for replacement, may dictate the use of a different construction sequence. Therefore, the most appropriate choice for these and other details must be established considering the specific constraints for each project. To illustrate this point, some details of design and construction differ between the design examples. Several additional options that could have been employed in the design are discussed at the

end of the design examples. The user is encouraged to consider the design options in light of the specific requirements and constraints to obtain the best solution for their project rather than to repeat the design solutions presented here. The concepts presented should enable the user to extend the use of the example to other situations.

The design examples were prepared using the *AASHTO LRFD Bridge Design Specifications* with Interims through 2003. Where the *Specifications* are unclear or do not provide requirements necessary for the design, assumptions are made and explained. Article and equation references are given where appropriate to assist the designer in applying the *Specifications.*

Each design example is numbered separately and has its own Table of Contents to assist the user in finding sections of interest and to give an overview of the organization of the design process. Each design example also has a list of references that were cited in the example.

1.1 Design Example 1

The first design example is the simplest example, addressing a long simple span bridge. This could represent a long-span bridge over a stream crossing or over a wide "single point urban interchange" (SPUI). The specific application illustrated is a SPUI over an existing roadway where traffic must be maintained during construction.

Details of this design example include:

- Simple span

- 96 in. deep modified PCI bulb-tees with an 8 in. web

- Three girder segments are used, with two lengths. All girder segments are pretensioned

- Girder segments are erected on permanent abutments and temporary towers

- Splices between girders are cast when the deck is cast on the girder segments

- All post-tensioning tendons are stressed after the deck and splice concrete are placed

- Post-tensioning tendon anchorages are located in end blocks at the end of each girder

- Conventional abutments and bearings are used

1.2 Design Example 2

The second design example is more complex, representing a two-span bridge crossing an existing interstate, where maintenance of traffic issues are very important. This example also illustrates the application of spliced girder construction to sites where seismic design must be considered.

Details of this design example include:

- Two unequal spans

- 6 ft deep U-beams

- Five girder segments are used, with several lengths. Three of the girder segments are pretensioned.

- Girder segments are erected on temporary towers and the interior pier.

- Splices between girders are cast after the girder segments are erected.

- After splicing, first stage post-tensioning tendons are stressed.

- The deck is cast on the continuous girders.

- Second stage post-tensioning tendons are stressed after the deck concrete is placed.

- Post-tensioning tendon anchorages are located in the cast-in-place end diaphragm, which rests on conventional bearings.

- The interior pier cap is integral, providing a fixed connection between the superstructure and substructure.

1.3 Design Example 3

The third design example is the most complex of the three design examples. It addresses a three span bridge. This type of structure has most often been used for crossings over navigable waterways, but can be used for any multi-span crossing where spans are greater than can be achieved using a constant depth cross-section. The concepts of the example can also be extended for use on bridges with more than three spans.

Details of this design example include:

- Three spans, with equal end spans (symmetrical about center of bridge)

- End girder segments are 78 in. deep Florida modified bulb-tee girders; pier girder segments are haunched to 15'-0" deep at the interior piers, with a slightly varying bottom flange height; the drop-in girder segments are 90 in. deep Florida modified bulb-tee girders. All girder segments have a 9 in. web.

- Five girder segments are used, with three different lengths.

- The five girder segments are each pretensioned.

- The girder segments are erected on permanent piers, temporary towers and strongbacks.

- Splices between girders are cast after the girder segments are erected.

- After splicing, first stage post-tensioning tendons are stressed.

- The deck is cast on the continuous spliced girders.

- Second stage post-tensioning tendons are stressed after the deck concrete is placed.

- Post-tensioning tendon anchorages are located in end blocks at the end of each girder.

- The depth of the haunched girder segments at the interior pier and the depth of the drop-in girder segments in the center span were increased to satisfy live load deflection requirements. The pretensioning in the girder segments was also adjusted to improve the profile of the bridge under dead load conditions.

DESIGN EXAMPLE 1:   Single Span Spliced PCI BT-96 Girder

Table of Contents

1   INTRODUCTION

The bridge in this example represents a typical long span simply supported bridge. It illustrates the design process and span capability of simple-span spliced concrete girders, focusing on issues that are specific to spliced girder design, while providing enough design information for the reader to be able to follow the design from start to finish.

Simplified design methods are employed to demonstrate a manual approach to the design of spliced girder bridges. The design is performed using designer-developed spreadsheets to facilitate computations. Losses are computed using the refined method of the *AASHTO LRFD Bridge Design Specifications (1)* with consideration of the effects of combined pretensioning and post-tensioning on the girder segments.

The outline of this design example is as complete as possible, identifying the essential elements of design. However, some of the computations are no different from other long span pretensioned bridges or other post-tensioned bridges. Where this is the case, details of computations are not shown as part of this design example.

While the design example demonstrates an excellent solution for the site, other options exist that may be used to advantage in certain situations. Other design issues will affect the design and detailing of spliced girder bridges. These issues are discussed briefly in **Section 15**.

2   DESCRIPTION OF BRIDGE

The bridge is a simply supported one-span bridge over a "single-point urban interchange" (SPUI). The same type of structure can also be used over any similar crossing, including waterways where piers are not allowed in the channel it is equally appropriate for remote rural locations and for congested urban sites.

The span length of the bridge (center of center of bearings) is 195'-4".

The bridge is comprised of precast, prestressed concrete bulb tee girders, based on the PCI bulb-tee section (see **Section 5.4.2**), with a composite deck slab.

The girders are fabricated in three pieces because of handling and shipping considerations. The end girder segments are identical. The three girder segments are erected on temporary support towers at the two splice locations. A minimum center girder segment length was established by maintenance of traffic requirements for the roadway beneath the bridge. The center segment must span four lanes of traffic and a center turning lane, plus narrow shoulders and temporary barriers to protect the temporary supports. The initial segment configuration of the bridge, which was based on a minimum center girder segment length, is shown in **Figure 2-1**. The final segment configuration and other details of the bridge are shown in **Figures 2-2** through **2-4**. The length of the center girder segment was increased because of structural design considerations, as discussed later in this example.

The girders are spliced by placing concrete in the joints between girder segments. For this example, the concrete is placed in the splice at the same time as the deck slab concrete is being placed. A diaphragm is also cast with each splice to improve stability and facilitate concrete placement and consolidation in the splice. After the concrete in the deck and splices reach a minimum strength, post-tensioning tendons running the full length of the spliced girders are stressed. After post-tensioning, the girders are continuous for all loads applied to the composite section.

Single Span Spliced PCI BT-96 Girder   Design Example 1

The number of girders in the typical section was optimized during the preliminary design process.

This design example demonstrates the design of an interior girder. Design of an exterior girder would be similar except for loads. For this bridge, the design of the interior girder governs.
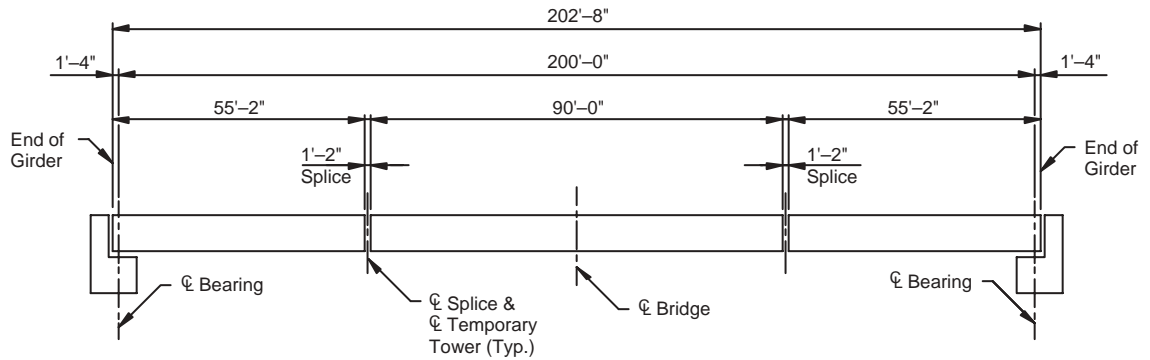
Figure 2-1   Elevation – Girder Erection – Initial Design
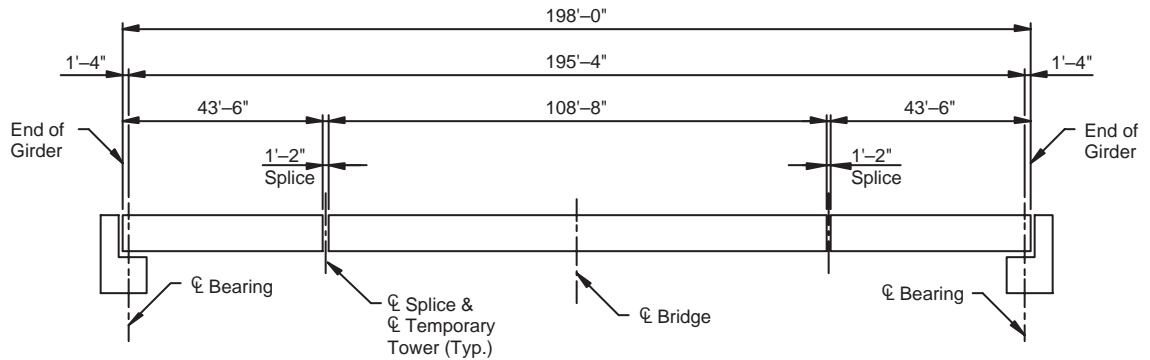
Figure 2-2   Elevation – Girder Erection – Final Design
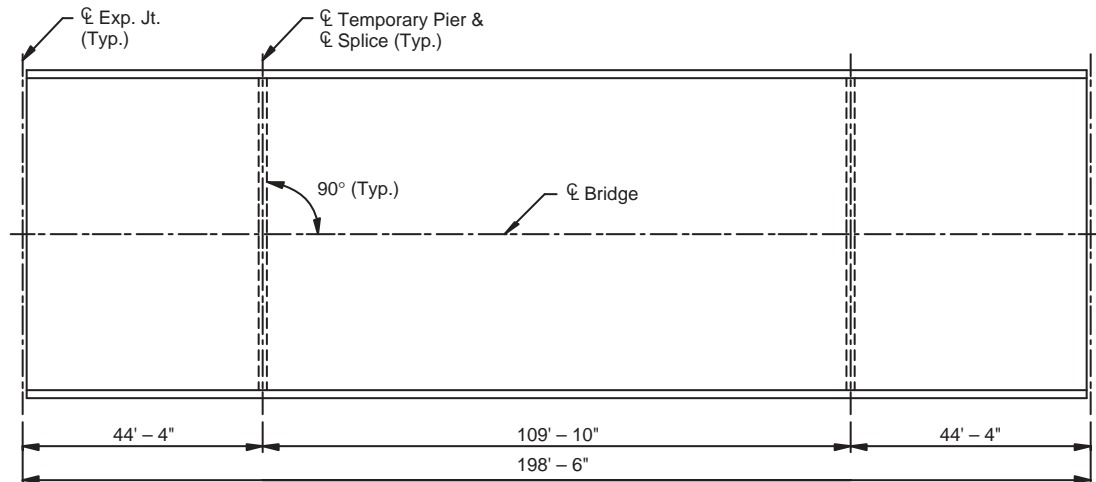
Figure 2-3   Plan View of Bridge

D1-2

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
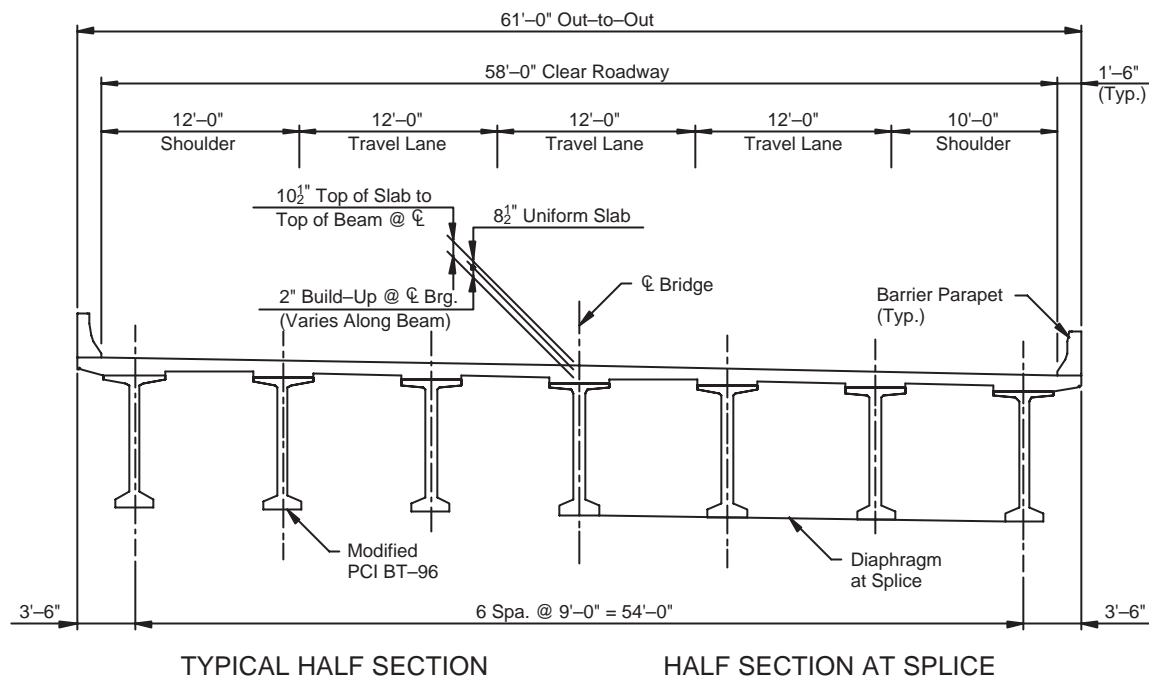
Figure 2-4   Typical Section of Bridge

3   DESIGN ASSUMPTIONS

The following assumptions are used for the design of this bridge. Additional discussion of some items is provided later in the design example.

1. The roadway section and span length are set as shown in the preceding figures.

2. Girder segments are erected on permanent abutments and temporary shoring towers at splice locations.

3. The girders are spliced and the deck concrete placed prior to post-tensioning the girders and removal of the temporary shoring towers. This allows the post-tensioning to be applied to the full girder cross-section, making it more efficient. It also eliminates the need for the post-tensioning subcontractor to return to the site for a second stage of post-tensioning which would be necessary, if the post-tensioning were performed in two stages. Single stage post-tensioning of the composite section requires that the non-composite girder segments resist the weight of the deck and related loads as simple spans rather than a continuous span. This increases the stresses in the girders near midspan requires additional pretensioning of the girder segments and increases the required capacity of the temporary supports. However, since the shoring is in place until after the deck is placed, the weight of the deck is partially resisted by the composite full span section.

4. Girder segments are pretensioned for stresses from handling, erection, and placement of deck concrete. The quantity of pretensioning is also set to balance the deflections caused by the placement of the deck concrete and related loads.

5. The width of the splice between ends of girder segments is 14 in. This width of splice is greater than the minimum required for splicing the post-tensioning ducts in order to provide for tolerance in duct and girder placement. Even with this width, care must be

D1-3

taken to properly locate the ducts in the precast girder segments and to erect the girder segments in proper alignment. While the use of a wider splice would facilitate placement and consolidation of the concrete in the splice and provide more tolerance for coupling the post-tensioning ducts, the wider splice would increase the width of the diaphragm at the splice location, increasing loads on the girder and the quantity of concrete required to build the structure.

6. A full depth diaphragm, cast with the splice, is provided at splice locations. This diaphragm provides stability at the splice locations, provides improved confinement for the concrete in the splice, and facilitates placement of concrete in the splice.

7. Splices and diaphragms are cast with the deck. Therefore, the girder segments are simply supported as the deck concrete is placed. The concrete used in the splice and diaphragm has the same compressive strength as the deck concrete. Placing the splice and diaphragm concrete with the deck concrete may slow the progress of deck placement, which may be a complication for the contractor. Special provisions may be required to accommodate beam end rotations at splices during deck placement.

8. No permanent intermediate diaphragms are used other than at splice locations. Temporary bracing is required along the girders for stability during construction.

9. Post-tensioning tendons are placed in a single vertical plane. To simplify fabrication, tendons are detailed with straight sections connected with a 30 ft radius. The straight sections are parallel. It appears that this is the first time that a tendon layout of this type has been proposed.

10. Post-tensioning tendon anchorages are contained within end blocks at the ends of each line of girders. Each tendon is comprised of 12 – 0.6 in. diameter low relaxation strands placed in a nominal 3.15 in. (80mm) inside diameter duct. This size of tendon and duct is selected to minimize the girder web width while providing a reasonable prestress force from post-tensioning tendons. Other sizes of tendons may be more economical for other projects.

11. Post-tensioning tendons are stressed from one end for simplicity and because it was found that two end stressing will not improve the effective tendon force for this bridge.

12. Post-tensioning ducts are grouted after tendons are stressed to provide corrosion protection and to improve performance by providing bond between the strands and concrete.

13. A typical composite reinforced concrete deck is used. The deck is 8½ in. thick, including ¼ in. for an integral wearing surface. A build-up is used to accommodate construction tolerances and variation in camber.

14. Girders are set on typical elastomeric bearings at the abutments. Provisions are made to accommodate shortening of girders during post-tensioning.

3.1   **SEQUENCE OF CONSTRUCTION**

The sequence of construction, with estimated age of the girders at each stage, is shown in **Table 3.1-1**.

The final stage occurs at the end of the service life of the bridge, which is taken to be 75 years (see definition for "Design Life" in LRFD Art. 1.2). At this stage, all prestress losses have occurred (effective prestress will be at its minimum value), and all dead loads will be present on the structure. Stresses at intermediate ages after all loads are in place are not as critical as

the final stage for this bridge. This is typical for simply supported spans where tension controls. If compressive stresses are close to limiting values, the stresses when the live load is first applied should also be checked.

Table 3.1-1   **Sequence of Construction and Design Stages**

Stage	Description	Girder Age (days)
1	Stress pretensioning strands	-
2	Cast girder segments	0
3	Release Pretensioned strands	1
4	Erect girder segments	50
5	Placement of deck and splice concrete	60
6	Stress Post-tensioning tendons	75
7	Add parapets	100
8	Bridge open, add live load (HL-93) with dynamic allowance	140
9	Add future wearing surface	15,000
10	Final condition, after losses, with live load (HL-93)	27,500

The main steps in the construction sequence are illustrated in **Figure 3.1-1**.

Four of the above listed construction stages are critical for the design of this bridge. These stages are listed in **Table 3.1-2** and will appear in discussions, tables, and figures throughout the remainder of this design example.

Table 3.1-2   **Critical Construction Stages for Design**

Stage	Description
1	Release Pretensioned strands
5	Placement of deck and splice concrete
6	Stress Post-tensioning tendons
10	Final condition, after losses, with live load (HL-93)

3.2   **EFFECT OF SEQUENCE OF CONSTRUCTION**

The most significant feature of the sequence of construction is that the girders are post-tensioned in a single operation after the deck has been placed. This option is feasible because all of the segments can be supported on the abutments or temporary supports until after the deck is cast. There are several benefits as a result of this sequence. One is that the use of only one stage of post-tensioning simplifies construction by requiring only one use of the post-tensioning equipment and subcontractor. A second benefit is that the deck is placed in additional compression, which can reduce or eliminate deck cracking. A possible disadvantage is that the removal of the deck, if required during the life of the bridge, is complicated since the deck was present when post-tensioning was applied.

Other sequences of construction could be used. One option is for the girders to be post-tensioned in two stages: one stage prior to and one stage after the deck is placed. Another

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

option is that the girders could be post-tensioned on the site and then lifted into place in a single piece. While this has been done, it requires either one or more very large cranes to handle the spliced girder, or a launching truss for supporting the girder as it crosses the feature below. A third option is to make the splice prior to deck placement, but delay post-tensioning until after the deck is placed. While these options were considered, the one used appears to be the most efficient and reasonable for this situation.

Placing the concrete in the splices and diaphragms during the placement of the deck concrete requires consideration of the end rotations of the center girder segment as the deck concrete is applied. Measures that can be taken to avoid possible distress at the splice includes placing only a portion of the deck at one time (see **Section 14.6**) or significantly retarding the concrete in the splice so that it remains plastic while the deck is placed on the center girder segment.
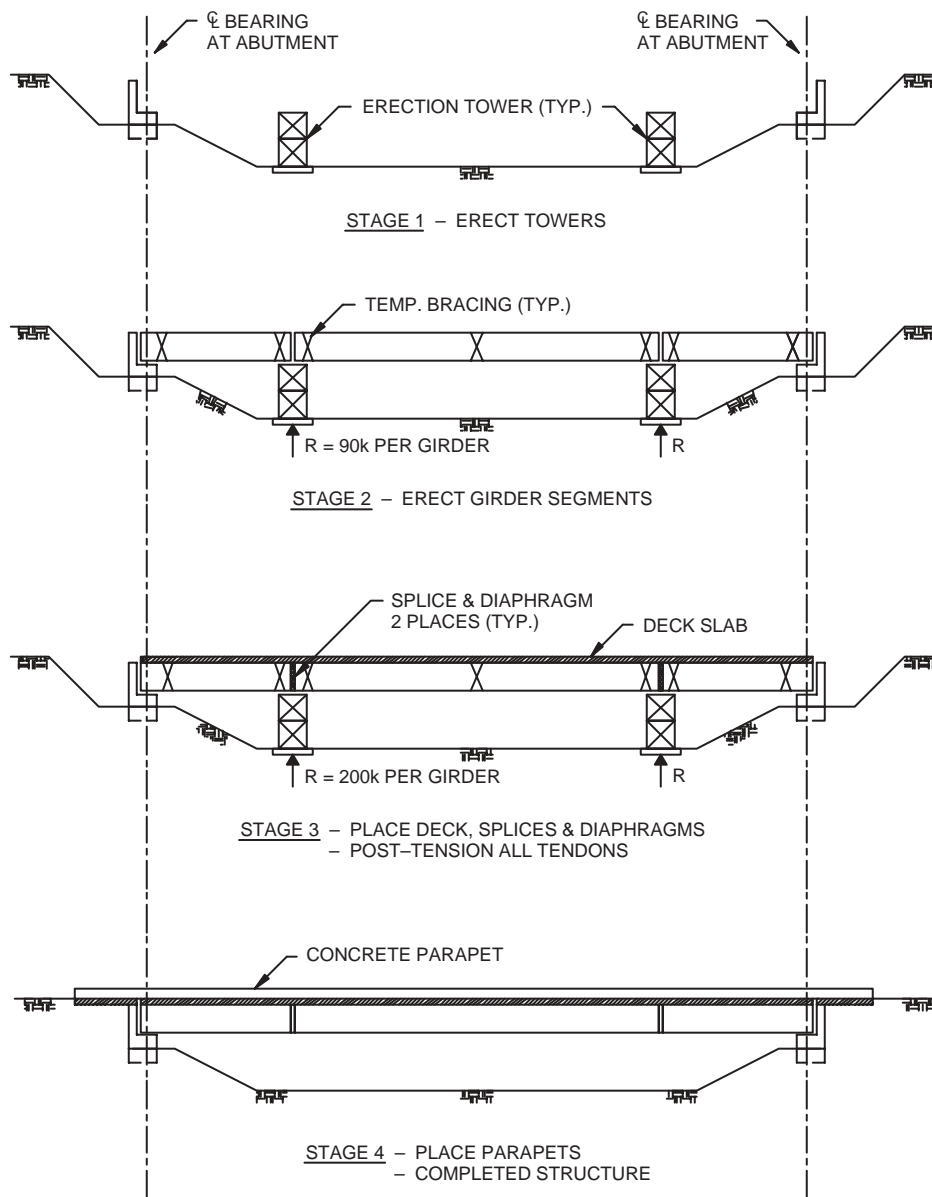
Figure 3.1-1   Construction Sequence

3.3   **OTHER DESIGN ASSUMPTIONS**

The deck thickness for the girder spacing in this example is determined to be 8½ in. using a slab design chart. The top ¼ in. of the deck is assumed to be a sacrificial wearing surface. Therefore, the structural deck thickness is taken as 8.25 in. for design purposes. The weight of the remaining ¼ in. of the deck is included as additional load on the noncomposite girder.

For simplicity, the full thickness of the build-up is applied to the full length of the girder for dead load computations. In most design situations, a value is used that is less than the specified build-up thickness at the center of the bearings because the actual thickness will vary along the length of the girder from the maximum of 2 in. at the center of bearings. The top flange width for the modified PCI BT-96 girder is 44 in.

The build-up is neglected when computing composite section properties that are used to calculate stresses for service limit state design, since the build-up will vary along the bridge. The assumed build-up is discussed further and verified in **Section 11.1.6**.

The girder span at release of pretensioning strands is taken to be the distance between centers of bearing locations, not the full length of the girder. This is slightly more conservative (stresses are more critical) and simplifies reporting of design quantities.

Intermediate diaphragms are not used in the bridge other than at splice locations. Temporary steel or timber cross frames are used during erection to stabilize girders (See **Figure 3.1-1**). However, the weight of these temporary components is minor and is neglected in these calculations.

Access must be provided at the abutments to allow stressing of the post-tensioning tendons. In this case, access is only required at one end since the girders are only stressed from one end.

4   **PRELIMINARY DESIGN**

Preliminary design aids exist for non-spliced pretensioned girders of normal sizes and spans. However, no such design aids exist for spliced girder bridges. The following indicates some of the steps taken to formulate the solution for this site.

4.1   **GIRDER TYPE SELECTION**

The selection of the girder type for this bridge was based on the following criteria:

1. Use a widely available cross-section

2. Consider the span-to-depth ratio

The consideration of these two criteria is discussed below.

4.1.1   **Section Type**

For this example, it is assumed that prestressed concrete girder fabricators located near the bridge site have been using PCI bulb tee girders for typical pretensioned girder bridges. Therefore, a girder in this family of sections was used. While this girder has a relatively small bottom flange, this should not be a problem for the following reasons:

*   The bridge is a simple span (not requiring a large bottom flange to resist high compressive stresses from negative moments at interior supports of continuous spans)

*   Pretensioning requirements are minor since pretensioning is only used to resist the self-weight of the girder and other loads applied to the noncomposite girder

- The majority of the prestress force is provided by post-tensioning tendons in the web (magnitude of force from post-tensioning tendons is not affected by size of bottom flange)

The range of standard section depths for the PCI bulb tees is not adequate for the span required. Therefore, it was necessary to use a modified section as discussed below.

4.1.2   **Section Depth**

The *AASHTO LRFD Specifications* provide optional minimum depth requirements for constant depth superstructures in LRFD Table 2.5.2.6.3-1. While the factors in the table have not been widely used in prestressed girder design, they do provide a guide for preliminary design.

Using the factor for Precast I-Beams given in the table, the minimum depth of the superstructure is:

$$h_{min} = 0.045L = 0.045(200) = 9.00 \text{ ft} \hspace{2cm} \text{LRFD Table 2.5.2.6.3-1}$$

This indicates that a superstructure depth of approximately 9.00 ft could be used for this span. It is assumed that this is the total superstructure depth. Therefore, the girder depth could be approximately 9 ft less the slab thickness, which is expected to be about 8.5 in. This provides a depth of approximately 8.3 ft or 99.5 in. Rounding down to a convenient dimension resulted in the use of a girder depth of 8 ft. It is assumed that the vertical clearance below the structure can accommodate this depth.

To verify that this section depth is adequate, the optional live load deflection limit of LRFD Art. 2.5.2.6.2 is checked in **Section 11.1.4.2**. This depth is not a standard depth in the PCI bulb tee family, but can be achieved by fabricators in several ways. See **Section 5.4.2** for the dimensions of the modified PCI BT-96 girder.

In some areas of the country, other cross-sections will be available. Locally available sections should always be used. If girders of the necessary depth have not been used in the region, prestressed girder fabricators should be consulted regarding the type of girder section that would be most economical and efficient for the project.

4.1.3   **Web Width**

The width of the web must be adequate to satisfy the post-tensioning requirements and the requirements of the *AASHTO LRFD Specifications* for the post-tensioning duct selected. The discussion in **Section 5.3.5.2** indicates that the minimum web width required for the post-tensioning tendons used for this bridge is 8 in. This is based on the requirement that the duct cannot be larger than 0.4 times the width of the web. Therefore, for this project, an 8 in. web will be used.

An 8 in. web is not standard for a PCI bulb tee girder. Therefore, the section is modified by spreading the side forms by 2 in., thus increasing the width of the web, bottom flange, and top flange by the same amount. This modification is generally simple to accomplish for fabricators.

It should be noted that girders have been fabricated and construction completed where the duct diameter is larger than 0.4 times the least web width.

4.2   **POST-TENSIONING TENDONS**

During preliminary design, post-tensioning literature or suppliers should be consulted regarding the sizes of tendons available and appropriate for the project. Selection of the tendon

size must be done early in the project since it affects the girder cross-section as noted above. Preliminary computations can be performed to estimate the number and size of tendons required.

4.2.1   **Tendon and Duct Size**

For the size of girder and span required, a tendon with a maximum of 12 – 0.6 in. diameter strands was selected as reasonable. The standard duct size for this number of strands has an inside diameter of 3.35 in. (85mm). However, this duct size does not satisfy the requirements of the *LRFD Specifications*. However, a slightly undersized duct with a 3.15 in. (80mm) inside diameter duct does satisfy the requirement and can still be used for this number of strands (see **Section 5.3.5.2**). Using this size of tendon and duct minimizes the girder web width while providing a reasonable prestress force from post-tensioning tendons. Other sizes of tendons may be more economical for other projects.

4.2.2   **Minimum Duct Spacing**

The tendons are spaced at a minimum spacing near midspan to achieve the greatest eccentricity of the prestress force. The duct spacing satisfies the *LRFD Specifications* requirement that the ducts have at least 1.5 in. clear between the ducts.

4.2.3   **Post-Tensioning Anchorages**

Typical dimensions for anchorages or the selected tendons must also be obtained in order to properly detail the end blocks and anchorage zones.

4.3   **SPLICES**

4.3.1   **Splice Location**

Initially, the splice locations for this project were determined by the requirements for maintenance of traffic beneath the bridge during construction. The requirement of a long clear span under the center of the structure during construction to accommodate traffic resulted in the use of a three-segment configuration. The roadway beneath the bridge was centered beneath the span so the end girder segment lengths would be equal. The traffic configuration beneath the bridge during construction required a distance between splices of at least 90 ft. The design process began using this length for the center girder segment.

However, other considerations are also significant in determining the splice location. These other considerations include:

1. Stresses at splice

- Splice has lower stress limits since it generally has a lower concrete strength
- Consider moving splice away from critical moment locations to keep from being controlling location for stresses
- The only prestressing available at the splice is provided by the post-tensioning tendons. The girder segments are also pretensioned which helps control stresses.

2. Hauling requirements

- Weight of segments

3. Handling and lifting requirements

    4. Temporary shoring tower requirements

During the design process, it was found that the stresses at the splice locations were critical. Design requirements could not be satisfied with the splices located using the minimum span required for maintenance of traffic beneath the bridge. The splice locations were then moved out, increasing the length of the center segment, to reduce the stress at the splices. The resulting center girder segment length is 108'-8".

With the increased length, the center girder segment weighs approximately 125 kips, which is approaching the limit for practical hauling without special permits in many areas. Therefore, use of a longer center girder segment may significantly increase the cost of transportation in some areas of the country. The larger center girder segment also increases the size of crane or cranes required for handling and erection.

It is generally more economical, for both design and fabrication, to detail a symmetrical structure with segments of equal length.

4.3.2 | **Splice Width**

The width of the splice depends on the duct splicing method used and on other construction requirements. A splice width will be shown on the plans, but it could be revised during construction due to preferences of the contractor. However, the width of the diaphragm at the splice may also change if the splice width is changed, which could affect the design.

For this project, a splice width of 14 in. was selected. A typical splice width is 12 to 24 in. Wider splices facilitate the placement and consolidation of concrete in the splice, although the use of a diaphragm at the splice also assists in these processes. Wider splices, however, also require more field-placed concrete, and if they are cast with the deck, the placement of concrete in the larger splice and diaphragm may slow the progress of concrete placement in the deck.

Wider splices also provide for more tolerance in the placement of the girders, which significantly affects the alignment and splicing of the ducts. The plans should specify a tolerance for girder placement to communicate to the contractor the importance of properly positioning the girder segments with respect to each other. The tolerance specified depends on the type of duct splice and other factors.

4.4 | **ESTABLISH TYPICAL SECTION AND PRELIMINARY POST-TENSIONING LAYOUT**

After selecting the girder cross-section and size of post-tensioning tendons, preliminary design can proceed to determine the number and spacing of the girders and to estimate the number of post-tensioning tendons. A preliminary design spreadsheet was developed that considered stresses at the splices and midspan to compare different numbers of girders in the cross-section and to evaluate pretensioning and post-tensioning requirements.

- Determine cross section properties for the girder and composite section

- Determine girder design loads (A build-up of 3 in. was initially assumed. This was later reduced to 2 in. The build-up was only used for load, not for increasing the depth of the composite section.)

- Estimate pretensioning requirements for handling of girder segments. Losses were estimated as a lump sum.

- Determine loading from removal of temporary shoring tower

- Estimate post-tensioning requirements, including consideration of lower concrete strength at splices. Losses were estimated as a lump sum.

Based on this limited evaluation, it was determined that the typical section shown in **Figure 2-4**, with seven girders, was the best design. This design would require six post-tensioning tendons. Using six girders in the typical section would require a seventh tendon, which would not be practical.

| 4.5 | **REFINEMENT OF PRELIMINARY DESIGN** |

After the preliminary design was established, final design was begun to refine the design in the following areas:

- Set final span length

- Set final splice locations

- Set tendon profiles, including tendon locations at ends (in girder end blocks) considering anchorage dimensions

- Compute prestress losses

- Determine cambers and deflections and set build-up to be used in design

As shown in this list, the span length and splice locations were adjusted to satisfy design requirements. The initial span length was 200 ft, but this could not be achieved. It was found that a slightly reduced span of 195'-4" would satisfy design requirements. Final splice locations were set as shown in **Section 2** in order to reduce stresses at the splice locations.

| 5 | # BASIC DESIGN INFORMATION |

Once the parameters for the design have been set or estimated using the preliminary design process described above, final design is performed to refine the design.

| 5.1 | **GOVERNING SPECIFICATIONS** |

AASHTO LRFD Bridge Design Specifications, 2nd Edition with Interims through 2003 *(1)*. References to articles, equations, and tables in these *Specifications* will be preceded by the prefix "LRFD" to differentiate these references from references to items in this design example.

While revisions to the *LRFD Specifications* are proposed as part of this research project (see **Appendix E**), none of the proposed revisions are used in this example, other than clarifications of the existing provisions.

| 5.2 | **LOADS** |

| 5.2.1 | **Live Load** |

The design live load is HL-93 with 33% dynamic allowance (IM) on the design truck, as specified in LRFD Article 3.6.1.2.

Live load effects (moment and shear) were computed along the length of the span using equations found in the *PCI Bridge Design Manual (2)*.

5.2.1.1 | **Live Load Distribution Factors**

Live load distribution factors are computed using equations in LRFD Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1 moment and shear, respectively, for an interior girder of section type (k) (see LRFD Table 4.6.2.2.1-1). Design parameters for this girder fall within the limits for the range of applicability for these equations.

Moment	One Design Lane Loaded	0.441 lanes / girder
	Two or More Design Lanes	0.673 lanes / girder
Shear	One Design Lane Loaded	0.720 lanes / girder
	Two or More Design Lanes	0.884 lanes / girder

5.2.2 | **Noncomposite Dead Loads**

5.2.2.1 | **Girder**

The girder dead load is computed using the unit weight of girder concrete (**Section 5.3.2.2**) and the area of the girder (**Section 5.4.2**).

$$w = w_c\,A = (0.150)(1,103/144) = 1.149 \text{ klf}$$

LRFD Article 5.14.1.2.1 specifies that an allowance for impact during handling and transportation equal to 50% of the girder self-weight should be used for design of the girders. However, in this bridge, where the bare girders must support the weight of the deck prior to post-tensioning, the load of the deck is greater than the allowance for impact. Therefore, in this example, the girder is not evaluated for the increased load for handling and transportation. It has been customary to use an impact factor of 20%, rather than 50%, when evaluating girders for handling and shipping stresses.

5.2.2.2 | **Deck**

Since the unit weight of deck concrete is 0.150 kcf, the girder spacing is 9 ft = 108 in., and the total thickness of the deck is 8½ in., the total deck dead load for an interior girder is

$$w = w_{cd}\,A_d = (0.150)(8.5(108)/144) = 0.956 \text{ klf}$$

The load on an exterior girder would be adjusted for the overhang width and the overhang taper.

5.2.2.3 | **Build-Up**

The full build-up thickness of 2 in. is used for dead load computations (see **Section 3.3**). The unit weight of the deck concrete is used over the width of the top flange of the girder.

$$w = w_{cd}\,h_{BU}\,b_{tf} = (0.150)(2(44)/144) = 0.092 \text{ klf}$$

This applies to both interior and exterior girders.

5.2.2.4 | **SIP Deck Forms**

The weight of a stay-in-place deck forming system is included in the design as an option. The forms and concrete contribute an additional load of 0.016 ksf on the formed area between girders, resulting in a load on an interior girder of

$$w = 0.016\,(GS - b_{tf}) = 0.016(108 - 44)/12 = 0.085 \text{ klf}$$

The load on an exterior girder would be one half of the load on an interior girder.

5.2.2.5 | **Temporary Construction Design Load**

Article 2.2.3.1 of the AASHTO *Guide Design Specifications for Bridge Temporary Works (3)* specifies a construction design load of 0.020 ksf on the full deck width. This distributed load is applied over the tributary area for each girder. Exterior girders have an additional load of 0.075 klf plus the weight of any equipment, such as a screed. In this example, the weight of equipment is assumed to be 0.125 klf.

The construction design load for an interior girder is

$$w = 0.020 \text{ GS} = 0.020(108)/12 = 0.180 \text{ klf}$$

The construction design load for an exterior girder is

$$w = 0.020 \text{ (GS/2 + OH)} + 0.075 + 0.125 = 0.020(9/2 + 3.5) + 0.200 = 0.360 \text{ klf}$$

This construction design load is a temporary load applied to the noncomposite girder during the placement of the deck concrete. It is only applied at the deck placement load event. This loading must be considered in this example because the girder is not fully prestressed when the deck is placed. Therefore, the construction stage at deck placement could be a critical design condition, which is not the case for typical pretensioned girder construction.

5.2.2.6 | **Total Noncomposite Dead Load**

The total noncomposite dead load for an interior girder, excluding the temporary construction design load, is

$$w_{NC} = 1.149 + 0.956 + 0.092 + 0.085 = 2.282 \text{ klf}$$

5.2.3 | **Composite Dead Loads (CDL)**

Dead loads placed on the composite girder are distributed equally to the seven girders in the cross-section (LRFD Art. 4.6.2.2.1).

5.2.3.1 | **Splices and Splice Diaphragms**

The splices and splice diaphragms are supported by the temporary towers when they are cast. Therefore, the load from them is not transferred to the bridge until the post-tensioning tendons are stressed. This load is therefore applied to the composite section.

The splice diaphragms are as wide as the splices, which are 14 in. wide. The height of the diaphragm is equal to the height of the girder plus the build-up, because the build-up is specified at the temporary support location. Since the splice and splice diaphragm are cast at the same time, the load for an interior girder is simply the product of the height of the girder, the length of the splice, the girder spacing and the unit weight of the splice concrete. This load also includes the weight of the deck above the splice, which is not considered elsewhere in the calculations.

The concentrated load on an interior girder from the splice, diaphragm, and deck above the diaphragm is

$$R_{spl} = h \, L_{sp} \, GS \, w_{cs} + (h_{BU} + h_f) \, L_{sp} \, GS \, w_{cd}$$
$$= 8(1.167)(9)(0.150) + ((2 + 8.5)/12)(1.167)(9)(0.150) = 14.0 \text{ kips}$$

The load on an exterior girder is one half of the load on an interior girder plus the weight of half of a splice and the weight of the deck overhang.

5.2.3.2

Removal of Temporary Support

When the post-tensioning tendons are stressed, the bridge is lifted off the temporary supports. The reactions at the temporary supports are then transferred to the full-span bridge. The reactions are computed by applying the total noncomposite dead load to half of the girder segment length. The reactions for interior girders at the temporary towers for the two types of girder segments are

$$R_{end} = w_{NC} (L_{end}/2) = 2.282 (43.5/2) = 49.6 \text{ kips}$$
$$R_{center} = w_{NC} (L_{center}/2) = 2.282 (108.66/2) = 124.0 \text{ kips}$$

The total reaction at pier removal, which includes the concentrated load from the splice and diaphragm, is

$$R_{total} = R_{end} + R_{center} + R_{spl} = 187.6 \text{ kips}$$

This total reaction is applied at the center of the splice.

5.2.3.3

Parapet

Each parapet is assumed to weigh 0.371 klf, or 0.742 klf for both parapets. Therefore the load per girder is

$$w = 0.742 / 7 = 0.106 \text{ klf}$$

5.2.3.4

Future Wearing Surface (FWS)

The future wearing surface is assumed to weigh 0.025 ksf over the clear roadway width of 58 ft. Therefore the load per girder is

$$w = 0.025 (58) / 7 = 0.207 \text{ klf}$$

5.3

MATERIALS AND MATERIAL PROPERTIES

Material properties used for design are given below.

5.3.1

Girder Concrete

5.3.1.1

Basic Properties

Design requirements were satisfied using concrete strengths typical for pretensioned girders. High strength concrete was not required.

The basic properties for the girder concrete are:

f'_{ci}	= 5.00 ksi (at release)	
f'_c	= 6.50 ksi	
f_r	= $0.24 \sqrt{f'_c}$	LRFD Art. 5.4.2.6
	= 0.612 ksi	
w_c	= 0.150 kcf	LRFD Table 3.5.1-1
E_{ci}	= 4,287 ksi	LRFD Eq. 5.4.2.4-1
E_c	= 4,888 ksi	LRFD Eq. 5.4.2.4-1

The 28-day girder concrete strength, f'_c, must be achieved prior to post-tensioning. A reduced unit weight of concrete, reflecting the unit weight without reinforcement, could be used to compute the modulus of elasticity values. This refinement is not significant and was neglected for this example.

5.3.1.2 | **Time-Dependent Properties**

Since simplified loss computations are used for this design, it is not necessary to compute specific time-dependent properties for the girder concrete.

5.3.2 | **Deck Concrete**

The subscript "d" is used to indicate properties related to the deck (or splice or diaphragm) concrete.

5.3.2.1 | **Basic Properties**

The basic properties for the deck concrete are:

f'_{cdi} = 3.50 ksi (at post-tensioning)

f'_{cd} = 4.50 ksi

f_r = 0.24 $\sqrt{f'_{cd}}$ LRFD Art. 5.4.2.6

     = 0.509 ksi

w_{cd} = 0.150 kcf   LRFD Table 3.5.1-1

E_{cdi} = 3,587 ksi   LRFD Eq. 5.4.2.4-1

E_{cd} = 4,067 ksi   LRFD Eq. 5.4.2.4-1

5.3.2.2 | **Time-Dependent Properties**

Since simplified loss computations are used for this design, it is not necessary to compute specific time-dependent properties for the deck concrete.

5.3.3 | **Splice and Diaphragm Concrete**

The splice and diaphragm at the splice are cast at the same time as the deck using the same concrete and concrete properties. A higher strength concrete could be used at the splice if stress limits at the splice govern the design, which would generally occur when a splice is located at or near midspan.

The subscript "s" is used to indicate properties related to the splice or diaphragm concrete.

5.3.3.1 | **Basic Properties**

The basic properties for the splice and diaphragm concrete are:

f'_{csi} = 3.50 ksi (at post-tensioning)

f'_{cs} = 4.50 ksi

f_r = 0.24 $\sqrt{f'_{cs}}$ LRFD Art. 5.4.2.6

     = 0.509 ksi

w_{cs} = 0.150 kcf   LRFD Table 3.5.1-1

E_{csi} = 3,587 ksi   LRFD Eq. 5.4.2.4-1

E_{cs} = 4067 ksi   LRFD Eq. 5.4.2.4-1

5.3.3.2 | **Time-Dependent Properties**

Since simplified loss computations are used for this design, it is not necessary to compute specific time-dependent properties for the splice and diaphragm concrete.

5.3.4	**Pretensioning Strands**

The girders are pretensioned with 0.6 in. diameter low relaxation seven-wire strands.

A_{ps} = 0.217 in^2

f_{pu} = 270 ksi

f_{py} = 0.90 f_{pu} = 243 ksi · · · · · · · · · · · · · · · · · · LRFD Table 5.4.4.1-1

f_{po} = 0.75 f_{pu} = 202.5 ksi · · · · · · · · · · · · · · · · · LRFD Table 5.9.3-1

E_p = 28,500 ksi · LRFD Art. 5.4.4.2

5.3.5	**Post-Tensioning Materials**
5.3.5.1	**Post-Tensioning Strands**

The girders are post-tensioned with 0.6 in. diameter low relaxation seven-wire strands.

A_{ps} = 0.217 in^2

f_{pu} = 270 ksi

f_{py} = 0.90 f_{pu} = 243 ksi · · · · · · · · · · · · · · · · · LRFD Table 5.4.4.1-1

f_{pj} = 0.90 f_{py} = 218.7 ksi (maximum – see **Section 7.3**) · · · LRFD Table 5.9.3-1

E_p = 28,500 ksi · LRFD Art. 5.4.4.2

5.3.5.2	**Post-Tensioning Tendons and Ducts**
5.3.5.2.1	*Maximum Duct Size for 8 in. Web*

The tendons selected for this project are composed of 12 – 0.6 in. diameter strands. This number was selected because it is the maximum number that can be placed in a round duct in the 8 in. web. An equivalent number of 0.5 in. diameter strands could also be used.

Post-tensioning ducts must satisfy the requirements of LRFD Art. 5.4.6. LRFD Article 5.4.6.2 requires that the size of the duct shall not be greater than 0.4 times the least gross concrete thickness at the location of the duct. Therefore, the maximum duct diameter for an 8 in. web is

D_{max} = 0.4 b_w = 0.4(8) = 3.2 in.

The same article requires that the area of the duct be at least 2.0 or 2.5 times the net area of the prestressing strands, depending on how the strands are installed. Using the larger factor to be conservative, the minimum duct diameter for the 12 – 0.6 in. diameter strands is

D_{min} = $\sqrt{[4(2.5 A_{ps})/\pi]}$ = $\sqrt{[4(2.5(12(0.217)))/\pi]}$ = 2.88 in.

The standard duct diameter used with 12 – 0.6 in. strands is 85mm or 3.35 in. This satisfies the minimum diameter requirement but violates the maximum diameter. A reduced size duct may also be used for a tendon of this size, which has a diameter of 80mm or 3.15 in. This size duct satisfies both the maximum and minimum criteria, and will be used in the example. With the slightly smaller diameter, extra care must be taken by the fabricator and contractor to ensure proper alignment of the ducts in the girders and proper placement of the girders when erected.

5.3.5.2.2	*Minimum Cover Requirements*

Minimum cover requirements for the post-tensioning ducts and reinforcement (stirrups) specified in LRFD Art. 5.12.3 and Table 5.12.3-1 must also be satisfied.

Assuming that the W/C ratio for the girder concrete is less than 0.40, the Specifications allow the minimum specified cover to be factored by 0.8. Therefore, the minimum cover for the duct would be 0.8(2.0) = 1.6 in., for the "Exterior other than above" condition in LRFD Table

5.12.3-1 or one half the diameter of the duct, which is 1.575 in., so 1.6 in. governs. The cover provided over the selected duct diameter of 3.15 in. is 2.43 in. > 1.6 in., so the minimum cover requirement is satisfied for the duct.

The required cover for the stirrups may be 0.5 in. less than the cover in Table 5.12.3-1 for main bars, but not less than 1.0 in. Reducing the cover, then applying the 0.8 factor results in a minimum cover over the stirrups of 0.8(2.0-0.5) = 1.2 in. > 1.0 in., so 1.2 in. governs. Allowing 0.75 in. for the stirrup diameter (the overall diameter of a No. 6 bar), the cover provided over the stirrups is 1.55 in. > 1.2 in., so the minimum cover requirement is also satisfied for the stirrups.

5.3.5.3   Post-Tensioning System Design Parameters

LRFD Table 5.9.5.2.2b-1 gives values for friction and wobble coefficients for use in design. The values selected for this design are listed below, with justification for the values chosen.

5.3.5.3.1   *Wobble Coefficient*

The wobble coefficient, K, given in the table is a typical value and is used for this example:

K   = 0.0002 / ft   LRFD Table. 5.9.5.2.26-1

5.3.5.3.2   *Friction Coefficient*

The friction coefficient, μ, is taken as

μ = 0.25   LRFD Table. 5.9.5.2.26-1

This is the maximum value given in the table. Actual values are typically closer to 0.20. The higher value is used to provide tolerance for field conditions. This would allow construction to proceed if the measured friction were above 0.20, but still below 0.25.

5.3.5.3.3   *Anchor Set*

Anchor set is mentioned in the commentary to LRFD Article C5.9.5.2.1. The commentary discusses various values that may be used. For this design, the anchor set, A_{set}, is taken as:

A_{set} = 0.375 in.

This value is at the upper end of the range of typical values was selected. This was again selected to provide tolerance for field operations.

5.3.5.4   Post-Tensioning Anchorages

Details of post-tensioning anchorage hardware are not considered or presented in this example. The details vary with the supplier of the material. A supplier was consulted to obtain reasonable parameters and dimensions which must be considered in laying out the post-tensioning. The size of the post-tensioning anchorage hardware was taken as 10.5 in. x 10.5 in. (see **Figure 6.2.1.5-1**).

5.3.5.5   Grout

While use of proper grout and grouting procedures is essential for the successful completion of a project, the type and strength of grout does not affect the design and are therefore not specified here. See the main report for additional information on grout and grouting.

Ducts must be grouted within a specified period following stressing to prevent corrosion of the tendons.

| 5.3.6 | **Mild Reinforcement** |

$$f_y \quad = \quad 60 \text{ ksi}$$
$$E_s \quad = \quad 29,000 \text{ ksi}$$

| 5.4 | **SECTION PROPERTIES** |

| 5.4.1 | **Effect of Post-Tensioning Ducts on Section Properties** |

LRFD Art. 5.9.1.4 requires that "... prior to bonding of post-tensioning tendons, effects of loss of area due to open ducts shall be considered." The article allows gross section properties to be used after post-tensioning tendons are grouted.

Section properties presented in the following are based on the gross section, that is, the voids are not considered. Neglecting the voids simplifies computation of section properties. Neglecting the voids is also based on experience, which has shown that the use of gross section properties is generally conservative for bridges of this type. Calculations have confirmed that this is the case for this bridge. The voids near the bottom of the beam result in a greater precompression of the bottom fibers, which is the location that governs this design at full service conditions. Stresses at stressing of the post-tensioning tendons should also be checked to ensure that the net section (with voids) is adequate to resist the post-tensioning forces.

LRFD Art. 5.8.2.9 requires that the width of the web effective in resisting shear shall be reduced by a fraction of the duct diameter. This reduction must be considered when computing the shear capacity of the girder.

| 5.4.2 | **Noncomposite Section (Girder only)** |

The standard PCI bulb tee section family includes only the BT-54, BT-63 and BT-72 girders. The BT-72 is not deep enough to satisfy the design requirements for this span. Therefore, a modified section is used with a 96 in. total depth. The shape of the top and bottom flanges is retained.

The standard PCI bulb tee sections also have a 6 in. web which cannot accommodate the duct required for the post-tensioning tendons. Therefore, the web width is increased to 8 in. The side forms are moved apart to increase the web width, which also increases the width of the top and bottom flanges. This makes the bottom flange width 28 in., which is standard for the AASHTO Type V and VI girders. Therefore, fabricators that have produced the AASHTO shapes will have a soffit form that can be used for this modified section.

The dimensions of the modified section are shown in **Figure 5.4.2-1**.

The section properties for the modified PCI BT-96 girder are:
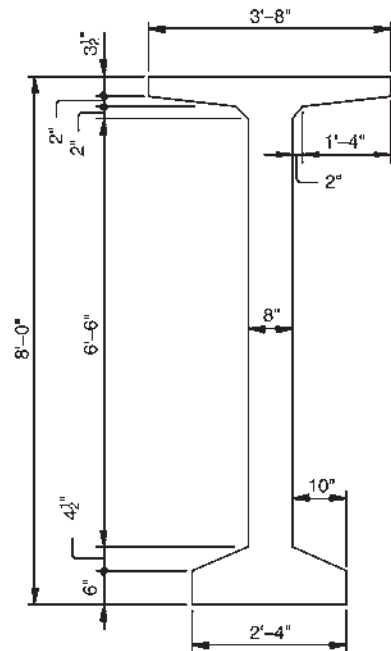
Figure 5.4.2-1   **Dimensions for Modified PCI BT-96 Girder**

$$h \quad = \quad 96.00 \text{ in.}$$
$$A \quad = \quad 1,103 \text{ in}^2$$

$$I \quad = \quad 1{,}260{,}081 \text{ in}^4$$

$$y_b \quad = \quad 48.47 \text{ in.}$$

$$y_t \quad = \quad 47.53 \text{ in.}$$

$$S_b \quad = \quad 25{,}995 \text{ in}^3$$

$$S_t \quad = \quad 26{,}514 \text{ in}^3$$

5.4.3   Composite Section (Girder with Deck Slab)

The composite deck slab is transformed using the modular ratio, n, for computing stresses at the service limit state.

$$n \quad = \quad E_{cd} / E_c \ = \ 4{,}067 / 4{,}888 \ = \ 0.832$$

The build-up is not considered in computing these section properties since the build-up may vary along the length of the girder to compensate for camber.

$$b \quad = \quad 9 \text{ ft} \ = \ 108 \text{ in.}$$

$$b_{eff} \quad = \quad 108 \text{ in.} \hspace{4cm} \text{LRFD Art. 4.6.2.6.1}$$

$$b_{eff\,tr} \ = \ n\,b_{eff} \ = \ 0.832\,(108) \ = \ 89.86 \text{ in.}$$

$$h_c \quad = \quad 104.25 \text{ in.}$$

$$A_c \quad = \quad 1{,}103 + 89.86(8.25) = 1{,}844 \text{ in}^2$$

$$A'_c \quad = \quad A_c/n = 2{,}217 \text{ in}^2 \hspace{2cm} \text{(Detransformed for computing stresses in deck)}$$

$$I_c \quad = \quad 2{,}447{,}089 \text{ in}^4$$

$$y_{bc} \quad = \quad 69.24 \text{ in.}$$

$$y_{tc} \quad = \quad 26.76 \text{ in.}$$

$$y_{tdc} \quad = \quad 35.01 \text{ in.}$$

$$S_{bc} \quad = \quad 35{,}344 \text{ in}^3$$

$$S_{tc} \quad = \quad 91{,}431 \text{ in}^3$$

$$S_{tdc} \quad = \quad (I_c/y_{tdc})/n = 83{,}995 \text{ in}^3 \hspace{1cm} \text{(Detransformed for computing stresses in deck)}$$

5.4.4   Composite Section at Splice Locations

A diaphragm is present at the splice location. Therefore, the structurally effective cross section at the splice includes some portion of the diaphragm. This makes the effective cross-section significantly larger than the girder cross-section. It is not clear how much of the diaphragm on either side of an interior girder would contribute to the behavior of the girder at the splice. Therefore, as a simplification, the composite section properties for the girder are used at the splice. This is expected to provide conservative results, because stresses computed at the splice would be higher than actual stresses since a larger section is available than is accounted for in the composite girder section properties.

It could be reasoned that, since the splice concrete is a lower strength than the girder, computed stresses should be "detransformed" by multiplying by the modular ratio prior to comparison to stress limits. It is conservative to neglect the correction, since the computed stresses would be higher than the detransformed stresses.

5.4.5   **Noncomposite Section at End Block**

An end block is provided at the end of the girders to contain the post-tensioning anchorages. See discussion and dimensions in **Section 12.2.1**.

The properties of the end block are given below.

h = 96.00 in.

A = 2,752 in^2

I = 2,196,687 in^4

y_b = 49.07 in.

y_t = 46.93 in.

S_b = 44,767 in^3

S_t = 46,807 in^3

5.4.6   **Composite Section at End Block**

The critical stage for the end block for these girders is when the post-tensioning tendons are stressed. The tendons are stressed after the deck is cast. Therefore, the composite section properties are required to compute the stresses at the end block.
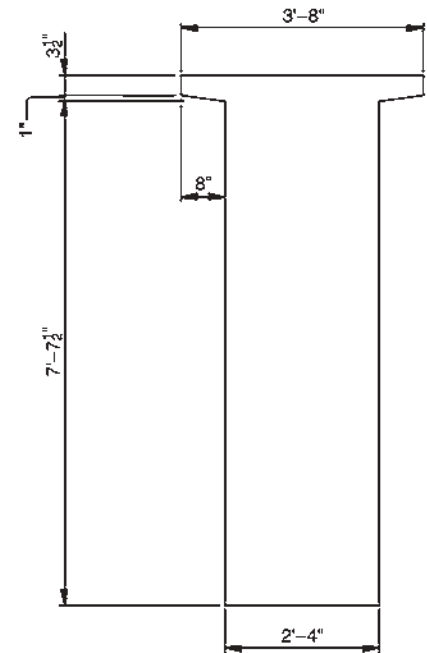
Figure 5.4.5-1   **Dimensions for Modified PCI BT-96 Girder End Block**

The composite properties of the end block are computed using the same assumptions and modular ratio used in **Section 5.4.3**.

b = 9 ft = 108 in.

b_{eff} = 108 in.   LRFD Art. 4.6.2.6.1

$b_{eff\,tr}$ = $n\, b_{eff}$ = 0.832 (108) = 89.86 in.

h_c = 104.25 in.

A_c = 2,752 + 89.86(8.25) = 3,493 in^2

A_c = A_c/n = 4,198 in^2 (Detransformed for computing stresses in deck)

I_c = 3,723,252 in^4

y_{bc} = 59.50 in.

y_{tc} = 36.10 in.

y_{tdc} = 44.35 in.

S_{bc} = 62,153 in^3

S_{tc} = 103,150 in^3

S_{tdc} = $(I_c/y_{tdc})/n$ = 100,907 in^3 (Detransformed for computing stresses in deck)

5.5   **DESIGN MOMENTS**

The following sections present the computed design moments along the span for service and strength limit states. All noncomposite loads are applied to girder segment spans. Loads applied after the deck and splices are placed act on the full span composite structure.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

5.5.1 | **Service Limit State**

The following tables list service limit state moments for each component of the applied load. **Tables 5.5.1-1** and **2** contain both the moments on the individual girder segments and the moments applied to the entire spliced girder. These moments are shown in **Figure 5.5.1-1** to illustrate the moments acting on the girder segments and those acting on the spliced girder. The total service limit state moments are not given in the tables, but the total moment, including the Service I live load moment is shown on **Figure 5.5.1-1**.
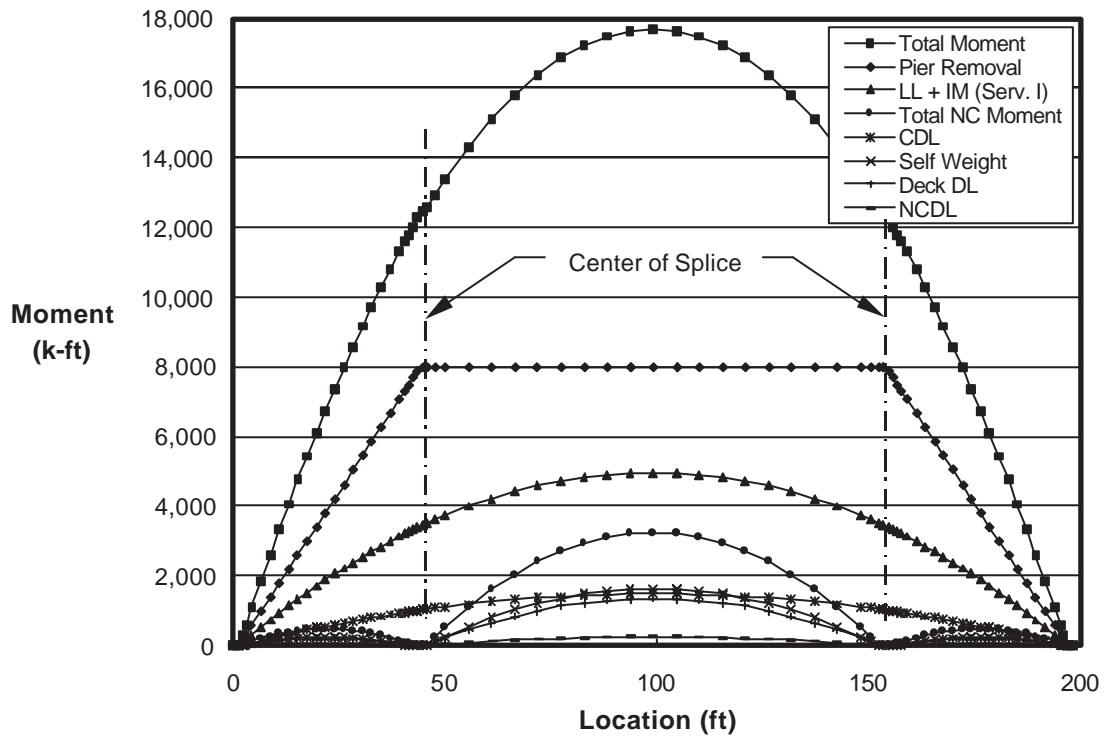
Figure 5.5.1-1   Service I Moments

Table 5.5.1-1   Service Design Moments for Girder Segments 1 and 3

Location from CL Permanent Bearing (ft)	Location from CL Temporary Bearing (ft)	Girder DL (k-ft)	Deck DL (k-ft)	NCDL (k-ft)	Constr. LL (k-ft)	Pier Removal (k-ft)	CDL (k-ft)	LL + IM Serv. I (k-ft)	LL + IM Serv. III (k-ft)
0.0	0.0	0	0	0	0	0	0	0	0
0.8	0.8	20	16	3	3	158	26	87	69
1.7	1.7	38	32	6	6	312	51	171	137
3.0	3.0	66	55	10	10	565	91	307	246
5.2	5.2	107	89	17	17	972	155	523	418
7.4	7.4	143	119	22	22	1,379	217	733	586
9.5	9.5	173	144	27	27	1,786	278	938	750
11.7	11.7	198	165	31	31	2,193	337	1,138	910
13.9	13.9	218	181	34	34	2,600	395	1,333	1,066
16.1	16.1	232	193	36	36	3,007	451	1,522	1,218
18.2	18.2	240	200	37	38	3,415	506	1,707	1,365
20.4	20.4	243	203	38	38	3,822	559	1,886	1,509
22.6	22.6	241	201	37	38	4,229	611	2,060	1,648
24.8	24.8	233	194	36	37	4,636	661	2,229	1,782
26.9	26.9	220	183	34	35	5,043	710	2,392	1,914
29.1	29.1	202	168	31	32	5,450	758	2,551	2,041
31.3	31.3	178	148	27	28	5,857	804	2,704	2,164
33.5	33.5	148	123	23	23	6,264	848	2,853	2,282
35.6	35.6	113	94	17	18	6,671	891	2,996	2,397
37.8	37.8	73	61	11	11	7,079	933	3,134	2,507
39.2	39.2	45	38	7	7	7,331	958	3,217	2,573
40.0	40.0	27	23	4	4	7,486	973	3,267	2,613
41.2	41.2	0	0	0	0	7,706	994	3,336	2,669
42.2	42.2	0	0	0	0	7,893	1,011	3,394	2,715

Note 1: Girder Segment 1 shown – Girder Segment 3 similar.

Note 2: Construction LL Moment is not included in Total Moment.

Note 3: Highlighted rows indicate the centerline of the permanent and temporary bearings.

Table 5.5.1-2   **Service Design Moments for Loads on Girder Segment 2**

Location from CL Permanent Bearing (ft)	Location from CL Temporary Bearing (ft)	Girder DL (k-ft)	Deck DL (k-ft)	NCDL (k-ft)	Constr. LL (k-ft)	Pier Removal (k-ft)	CDL (k-ft)	LL + IM Serv. I (k-ft)	LL + IM Serv. III (k-ft)
43.3	----	0	0	0	0	8,002	1,031	3,460	2,768
44.3	0.0	0	0	0	0	8,002	1,048	3,516	2,813
46.3	2.0	120	100	19	19	8,002	1,081	3,624	2,899
48.8	4.4	260	217	40	41	8,002	1,119	3,750	3,000
54.2	9.9	549	457	85	86	8,002	1,198	4,007	3,206
59.6	15.3	803	668	124	126	8,002	1,267	4,232	3,385
65.1	20.7	1,024	852	158	160	8,002	1,327	4,424	3,539
70.5	26.2	1,210	1,007	186	190	8,002	1,378	4,593	3,674
75.9	31.6	1,363	1,134	210	214	8,002	1,419	4,730	3,784
81.4	37.0	1,481	1,233	228	232	8,002	1,452	4,834	3,867
86.8	42.5	1,566	1,304	241	245	8,002	1,475	4,906	3,925
92.2	47.9	1,617	1,346	249	253	8,002	1,489	4,946	3,957
97.7	53.3	1,634	1,360	252	256	8,002	1,493	4,954	3,963
103.1	58.8	1,617	1,346	249	253	8,002	1,489	4,946	3,957
108.5	64.2	1,566	1,304	241	245	8,002	1,475	4,906	3,925
114.0	69.6	1,481	1,233	228	232	8,002	1,452	4,834	3,867
119.4	75.1	1,363	1,134	210	214	8,002	1,419	4,730	3,784
124.8	80.5	1,210	1,007	186	190	8,002	1,378	4,593	3,674
130.3	85.9	1,024	852	158	160	8,002	1,327	4,424	3,539
135.7	91.4	803	668	124	126	8,002	1,267	4,232	3,385
141.1	96.8	549	457	85	86	8,002	1,198	4,007	3,206
146.6	102.2	260	217	40	41	8,002	1,119	3,750	3,000
149.0	104.7	120	100	19	19	8,002	1,081	3,624	2,899
151.0	106.7	0	0	0	0	8,002	1,048	3,516	2,813
152.0	----	0	0	0	0	8,002	1,031	3,460	2,768
Note 1: Construction LL Moment is not included in Total Moment.									
Note 2: Highlighted rows indicate the centerline of the temporary bearings and midspan.									

5.5.2   **Strength Limit State**

The following tables list factored moments for the Strength I limit state for each component of the applied load. **Tables 5.5.2-1** and **2** contain both the moments on the individual girder segments and the moments applied to the entire spliced girder. These moments are shown in **Figure 5.5.2-1** in order to demonstrate the difference between moments acting on the girder segment and those acting on the spliced girder.

Table 5.5.2-1   Strength I Design Moments for Girder Segments 1 and 3

Location from CL Permanent Bearing (ft)	Location from CL Temporary Bearing (ft)	Girder DL (k-ft)	Deck DL (k-ft)	NCDL (k-ft)	Constr. LL (k-ft)	Pier Removal (k-ft)	CDL (k-ft)	LL + IM (k-ft)	Total Factored Moment M_u (k-ft)
Load Factor		1.25	1.25	1.25	1.75	1.25	1.50	1.75	
0.0	0.0	0	0	0	0	0	0	0	0
0.8	0.8	24	20	4	0	197	38	152	436
1.7	1.7	47	39	7	5	390	76	299	859
3.0	3.0	83	69	13	10	706	136	538	1,544
5.2	5.2	134	112	21	18	1,215	232	915	2,628
7.4	7.4	179	149	28	29	1,724	325	1,283	3,686
9.5	9.5	217	180	33	39	2,233	416	1,641	4,721
11.7	11.7	248	206	38	48	2,741	505	1,991	5,730
13.9	13.9	272	226	42	54	3,250	592	2,332	6,715
16.1	16.1	290	241	45	60	3,759	676	2,664	7,674
18.2	18.2	300	250	46	64	4,268	759	2,986	8,609
20.4	20.4	304	253	47	66	4,777	839	3,300	9,520
22.6	22.6	301	251	46	67	5,286	916	3,605	10,405
24.8	24.8	292	243	45	66	5,795	992	3,900	11,266
26.9	26.9	275	229	42	64	6,304	1,065	4,187	12,102
29.1	29.1	252	210	39	60	6,813	1,136	4,464	12,914
31.3	31.3	222	185	34	55	7,322	1,205	4,733	13,700
33.5	33.5	185	154	29	49	7,830	1,272	4,992	14,462
35.6	35.6	141	118	22	41	8,339	1,337	5,243	15,199
37.8	37.8	91	76	14	31	8,848	1,399	5,484	15,912
39.2	39.2	56	47	9	20	9,164	1,436	5,629	16,341
40.0	40.0	34	28	5	12	9,357	1,459	5,716	16,599
41.2	41.2	0	0	0	7	9,632	1,490	5,838	16,960
42.2	42.2	0	0	0	0	9,866	1,517	5,940	17,322

Note 1: Girder Segment 1 shown – Girder Segment 3 similar.

Note 2: Construction LL Moment is not included in Total Factored Moment. (Shaded Column)

Note 3: Highlighted rows indicate the centerline of the permanent and temporary bearings.

Table 5.5.2-2 **Strength I Design Moments for Girder Segment 2**

Location from CL Permanent Bearing (ft)	Location from CL Temporary Bearing (ft)	Girder DL (k-ft)	Deck DL (k-ft)	NCDL (k-ft)	Constr. LL (k-ft)	Pier Removal (k-ft)	CDL (k-ft)	LL + IM (k-ft)	Total Factored Moment M_u (k-ft)
Load Factor		*1.25*	*1.25*	*1.25*	*1.75*	*1.25*	*1.50*	*1.75*	
43.3	----	0	0	0	0	10,002	1,547	6,056	17,605
44.3	0.0	0	0	0	0	10,002	1,572	6,153	17,728
46.3	2.0	150	125	23	33	10,002	1,621	6,342	18,264
48.8	4.4	326	271	50	71	10,002	1,678	6,562	18,889
54.2	9.9	686	571	106	150	10,002	1,796	7,012	20,173
59.6	15.3	1,004	836	155	220	10,002	1,900	7,406	21,302
65.1	20.7	1,279	1,065	197	281	10,002	1,990	7,742	22,276
70.5	26.2	1,513	1,259	233	332	10,002	2,067	8,038	23,111
75.9	31.6	1,703	1,418	262	374	10,002	2,129	8,277	23,792
81.4	37.0	1,852	1,541	285	406	10,002	2,178	8,459	24,318
86.8	42.5	1,958	1,629	302	429	10,002	2,212	8,586	24,689
92.2	47.9	2,021	1,682	311	443	10,002	2,233	8,656	24,906
97.7	53.3	2,043	1,700	315	448	10,002	2,240	8,669	24,969
103.1	58.8	2,021	1,682	311	443	10,002	2,233	8,656	24,906
108.5	64.2	1,958	1,629	302	429	10,002	2,212	8,586	24,689
114.0	69.6	1,852	1,541	285	406	10,002	2,178	8,459	24,318
119.4	75.1	1,703	1,418	262	374	10,002	2,129	8,277	23,792
124.8	80.5	1,513	1,259	233	332	10,002	2,067	8,038	23,111
130.3	85.9	1,279	1,065	197	281	10,002	1,990	7,742	22,276
135.7	91.4	1,004	836	155	220	10,002	1,900	7,406	21,302
141.1	96.8	686	571	106	150	10,002	1,796	7,012	20,173
146.6	102.2	326	271	50	71	10,002	1,678	6,562	18,889
149.0	104.7	150	125	23	33	10,002	1,621	6,342	18,264
151.0	106.7	0	0	0	0	10,002	1,572	6,153	17,728
152.0	----	0	0	0	0	10,002	1,547	6,056	17,605

Note 1: Construction LL Moment is not included in Total Factored Moment. (Shaded Column)

Note 2: Highlighted rows indicate the centerline of the temporary bearings and midspan

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

6 | PRESTRESS LAYOUT AND DATA

6.1 | PRETENSIONED STRANDS

The number of pretensioned strands in the girder segments was selected to provide enough prestress to maintain concrete stresses within stresses limits at release and when deck concrete was placed. The net camber of the girder segments after the deck was placed was also considered in setting the number of strands.

All pretensioned strands are straight, since strands cannot be draped with post-tensioning ducts occupying the web. No debonding is required. Strands are placed in the bottom and top flanges as shown in **Figure 6.1-1**. All strands are fully stressed to a force of 0.75 f_{pu} or 31 kips. Four top strands were required for Girder Segment 2 to control tensile stresses near the end of the beam.

GIRDER SEGMENTS 1 AND 3   GIRDER SEGMENT 2
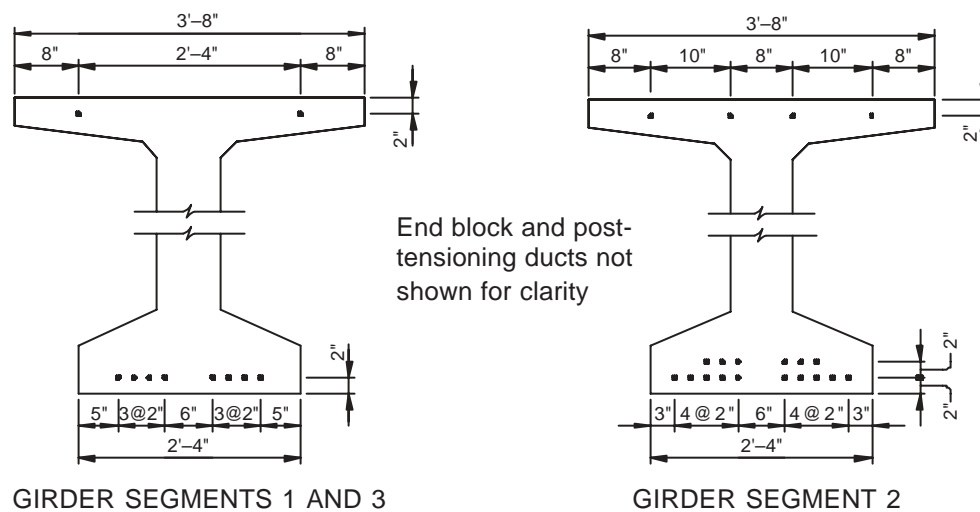
Figure 6.1-1   Layout of Pretensioned Strands

Table 6.1-1   Pretensioned Strand Data for Girder Segments

Girder Segments	No. Strands	A_{ps} (in^2)	C.G. from Bottom (in.)	Eccentricity (Noncomposite Section) (in.)	Eccentricity (Composite Section) (in.)
1 & 3	10	2.170	20.40	28.07	48.84
2	20	4.34	21.00	27.47	48.24

6.2 | POST-TENSIONING TENDONS

The post-tensioning tendons are contained in ducts. The layout of the ducts is discussed below, followed by the adjustment required to determine the location of the centroid of the post-tensioning tendons based on the duct locations.

D1-26

6.2.1   **Duct Layout**

Post-tensioning tendons in this girder are designed to provide the prestress required to resist the live load and the dead load as it is transferred from the girder segments to the full span. The moments caused by these loads follow an approximately parabolic shape, so a parabolic tendon profile is frequently used in the layout of tendons in a post-tensioned girder. The use of a parabolic profile with multiple tendons requires that the location of the ducts containing the tendons must be determined at a number of intermediate points along the girder, and that the distance between ducts will vary along the length of the girder. This is a complication during fabrication, although it is not a major issue in most cases.

6.2.1.1   **Duct Profile**

To simplify the design and fabrication of the post-tensioned girders in this example, a layout with straight runs of ducts connected with short curved sections is used. The straight portions of the tendons are parallel. The curved portion of each tendon has a minimum radius of 30 ft. A detail of the curved portion of the tendon is shown in **Figure 6.2.1.1-1**.
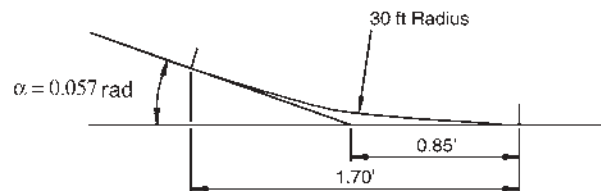
Figure 6.2.1.1-1   Detail at Curvature in Duct

The duct profile is defined so that the tendons are parallel at the center and ends of the span and that the average of the tendon profiles is approximated closely by a parabola or circular curve. The use of a circular curve allows the assumption that the friction losses are linear, which simplifies loss computations.

6.2.1.2   **Duct Spacing**

Where ducts are horizontal, they are spaced at 5 in. on center. This provides slightly more than the minimum 1.5 in. clear between ducts that is required in LRFD Article 5.10.3.3.2.

A second consideration is the spacing of ducts where ducts are curved. The bend location for each tendon occurs at a location where the other tendons are straight. When the bend occurs, the spacing to the next tendon on the inside of the radius, is much greater than the minimum requirement of one duct diameter found in the *AASHTO Segmental Guide Specifications (4)*.

6.2.1.3   **Details at Post-Tensioning Anchorages**

The spacing between the centers of anchorages at the end of the girder is defined by the dimensions and reinforcement associated with the anchorage hardware. Sufficient clearance must also be provided to allow for stressing of the tendons.

A spacing between tendons at the end of the girder of 13 in. was selected to provide space for the anchorage and avoid interference between the spirals required for each anchor.

An added benefit of having the ducts parallel that all anchorages are inclined at the same angle. Therefore, fabrication of blockouts at the anchorages is simplified since all blockouts are the same.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

6.2.1.4   Final Duct Layout

Initial duct locations were defined considering the original girder segment lengths. As the design was refined and the girder segment lengths and overall span were adjusted, the duct layout was also adjusted. The ducts were laid out so that all ducts would be straight as they crossed the splice with at least 9 in. of straight duct at the end of the girder segments. The duct layout was also adjusted slightly to improve the agreement between the average tendon profile and the parabolic tendon profile.

The final duct layout is shown in **Figure 6.2.1.4-1.**
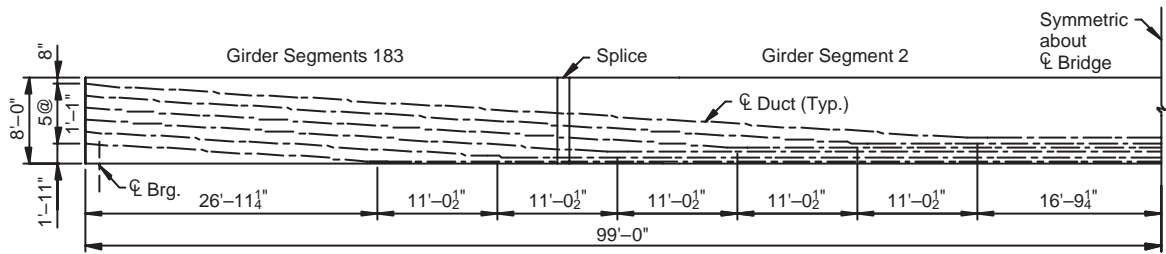
Figure 6.2.1.4-1   Post-Tensioning Duct Layout – Elevation

6.2.1.5   Details of Duct Layout

Section views showing details of the tendon layout at ends of girders segments and at midspan are presented in **Figures 6.2.1.5-1** and **6.2.1.5-2**. It can be seen that the circular and parabolic profiles are very closely approximated by the average of the profiles obtained using straight segments of duct with short curves between.
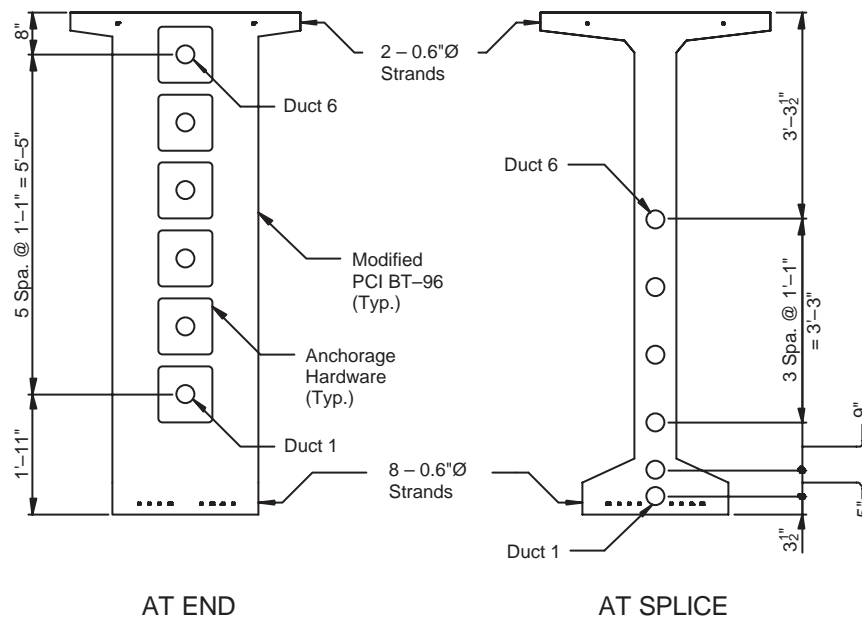
Figure 6.2.1.5-1   Post-Tensioning Duct Layout - Sections- Girder Segments 1 and 3

D1-28
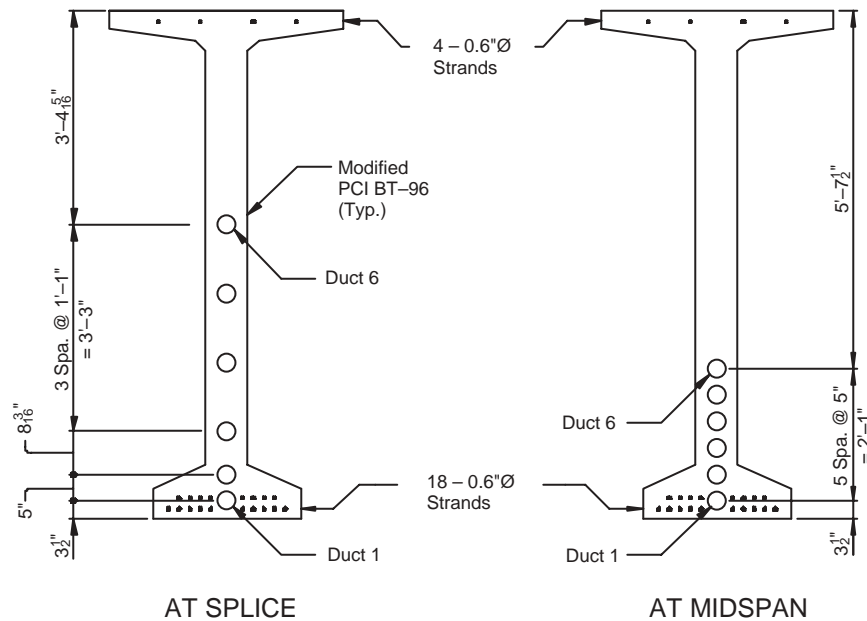
Figure 6.2.1.5-2   Post-Tensioning Duct Layout -Sections- Girder Segments 2

6.2.2 | **Tendon Profile**

6.2.2.1 | **Determination of Tendon Profile**

The center of a post-tensioning tendon is offset by a distance, Z, from the center of the duct when the tendon is curved. This is caused by the bunching of the strands on one side of the duct, as shown schematically in **Figure 6.2.2.1-1**. For a 3.15 in. diameter duct with 12-0.6 in. diameter strands, it is recommended that the offset between the center of duct and tendon (Z) be taken as 0.625 in. A different value is used from the 0.75 in. given in LRFD Figure C5.9.1.6-1 for ducts with an outside diameter between 3 and 4 in. because the duct is slightly undersized for the number of strands, which reduces the Z value.
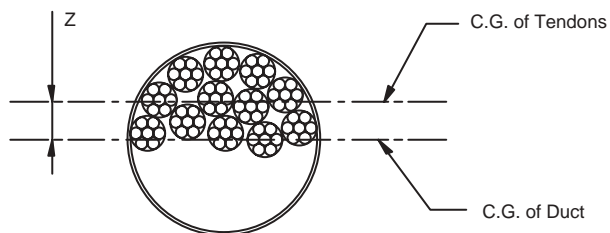
Figure 6.2.2.1-1   Offset between C.G. of Duct and C.G. of Tendon

The location of the center of the tendon is computed for each duct assuming that the full value of the offset, Z, is taken for the horizontal portion of the tendon, and that after the curve, the offset reduces linearly from the full value to zero at the end of the girder. An average eccentricity is then computed for all tendons. The eccentricity of each tendon, with the average eccentricity for all ducts are shown in **Table 6.2.2.1-1**. The eccentricity of the parabolic tendon layout discussed in the next section is also shown. The eccentricity is computed from the centroid of the composite section, since the post-tensioning is applied to the composite section.

D1-29

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

The profiles for each tendon and the average of all tendons are shown in **Figure 6.2.2.1-2.**
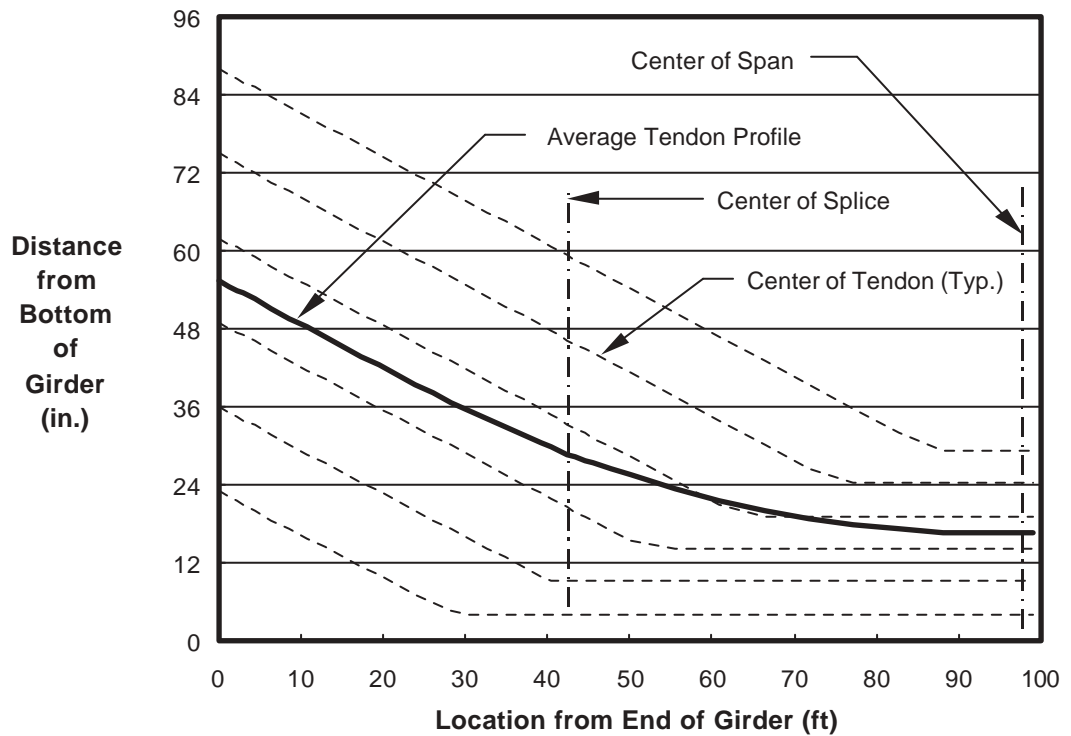
Figure 6.2.2.1-2   Post-Tensioning Tendon Profiles

The procedure for computing the average tendon eccentricity will be demonstrated for midspan.

g   = distance from bottom of girder to average tendon

$$= \frac{\sum_{i=1}^{N}(g_i + z)}{N}$$

     = [(3½ + ⅝) + (8½ + ⅝) + (13½ + ⅝) + (18½ + ⅝) + (23½ + ⅝) + (28½ + ⅝)]/6

     = 16.625 in.

e   = $y_{bc} - g$

     = 69.24 − 16.625

     = 52.62 in.

D1-30

Table 6.2.2.1-1   Eccentricity of Tendons and Average Tendon Profiles

Location from End of Girder (ft)	Eccentricity of Tendons						Average Eccentricity (in.)	Parabolic Eccentricity (in.)	Difference (in.)
	1 (in.)	2 (in.)	3 (in.)	4 (in.)	5 (in.)	6 (in.)			
0.0	46.2	33.2	20.2	7.2	-5.8	-18.8	13.7	13.7	0.0
1.3	47.1	34.1	21.1	8.1	-4.9	-17.9	14.6	14.8	0.1
4.4	49.1	36.1	23.1	10.2	-2.8	-15.8	16.6	17.1	0.4
6.5	50.5	37.6	24.6	11.6	-1.4	-14.4	18.1	18.7	0.6
10.9	53.4	40.5	27.5	14.5	1.6	-11.4	21.0	21.8	0.8
17.4	57.7	44.8	31.9	18.9	6.0	-7.0	25.4	26.2	0.8
19.6	59.1	46.3	33.3	20.4	7.4	-5.5	26.8	27.5	0.7
23.9	62.0	49.1	36.3	23.3	10.4	-2.6	29.7	30.2	0.5
26.1	63.4	50.6	37.7	24.8	11.8	-1.1	31.2	31.5	0.3
30.5	65.0	53.5	40.6	27.7	14.8	1.8	33.9	33.9	0.0
34.8	65.0	56.4	43.5	30.6	17.7	4.7	36.3	36.2	-0.1
37.0	65.0	57.8	45.0	32.1	19.2	6.2	37.5	37.3	-0.3
40.5	65.0	59.8	47.4	34.5	21.5	8.6	39.5	39.0	-0.5
44.7	65.0	60.0	50.1	37.3	24.3	11.4	41.5	40.8	-0.7
45.7	65.0	60.0	50.8	37.9	25.0	12.1	41.8	41.2	-0.6
50.1	65.0	60.0	53.8	40.9	28.0	15.1	43.8	43.0	-0.8
66.4	65.0	60.0	55.0	50.0	39.0	26.1	49.2	48.3	-0.9
77.3	65.0	60.0	55.0	50.0	45.0	33.4	51.4	50.6	-0.8
82.7	65.0	60.0	55.0	50.0	45.0	37.1	52.0	51.4	-0.6
99.0	65.0	60.0	55.0	50.0	45.0	40.0	52.6	52.5	0.0

Note 1: Highlighted rows indicate the centerline of the permanent bearing and midspan

Note 2: Eccentricity is measured from the centroid of the composite section.

6.2.2.2   **Comparison of Tendon Profiles**

The average tendon profile obtained using the six tendons shown in the previous section is compared to a parabolic tendon layout with the same eccentricities at midspan and at the end in **Figure 6.2.2.2-1**. The figure clearly demonstrates that the parabolic tendon profile is very closely approximated by the average profile obtained for tendons in ducts with straight segments connected with short curves. The maximum difference between the eccentricities for the two tendon layouts is −0.9 in., or less than 2% of the eccentricity at that location. This is an insignificant difference. Therefore, the tendon profile used is assumed to be equivalent to the parabolic profile.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
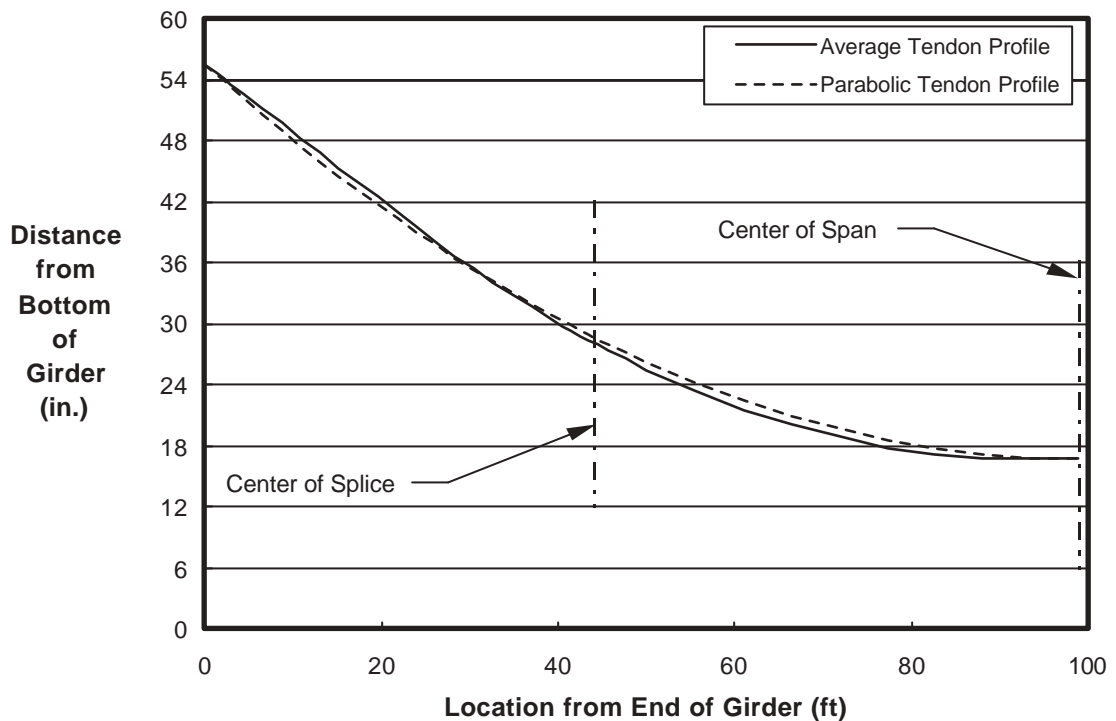
Figure 6.2.2.2-1 Comparison of Average and Parabolic Tendon Profiles

6.2.2.3   **Post-Tensioning Tendon Information at Critical Locations**

The eccentricity of the average tendon profile is shown for critical locations in **Table 6.2.2.3-1**.

Table 6.2.2.3-1   Average Tendon Profile at Critical Locations

Location	C.G. from Bottom (in.)	Eccentricity (in.)
End	55.4	13.8
Center of Girder Segments 1 and 3	40.8	28.4
Center of Splice	28.0	41.2
Midspan	16.6	52.6
Note: Eccentricity is measured from centroid of composite section		

Additional post-tensioning tendon information:

- Number of 0.6 in. diameter strands in each tendon = 12

- Number of tendons = 6

- Total number of strands = 72

- Total are of strands = 15.624 in^2

D1-32

7   PRESTRESS LOSSES

The estimation of prestress losses for the design of a spliced girder bridge at the service limit state has significant differences from typical design of pretensioned girders, because prestressing is applied at two different times and in two different ways. A rigorous time-step approach could be used to estimate losses. However, a simplified approach is employed in this example, based on the provisions of the *AASHTO LRFD Specifications*. The computations can be done manually or automated using a spreadsheet or program.

LRFD Art. 5.9.5.4.1 allows the simplified approach to be used with spans not greater than 250 ft, normal weight concrete, and a minimum concrete strength at prestressing of 3.5 ksi. This design meets all of these criteria.

This section begins by presenting basic concepts and procedures for estimating prestress losses that are required for this type of bridge which has both pretensioned and post-tensioned elements. Losses are then computed at midspan for the pretensioned strands and for the post-tensioned tendons. This section concludes with a summary of losses and effective stresses.

7.1   BASIC CONCEPTS

Losses are computed in this design example using the methods and principles of the "refined method" of the *AASHTO LRFD Specifications*. This approach only provides total losses and has no provisions for computing losses at stages prior to the final conditions. This is normally adequate when all of the prestress is applied to the member at once.

However, in this example, the prestress is applied in two stages: at release of the pretensioned strands, and at stressing of the post-tensioning tendons. A third stage could be possible if a second stage of post-tensioning were used. Concrete stresses must be computed and compared to limiting values at critical events that occur between the initial application of prestress at release of the pretensioned strands and the final conditions. Furthermore, the two stages of prestressing affect each other and the losses. Therefore, the equations in the *Specifications* must be modified to account for these issues. These modifications are described below.

7.1.1   Interaction of Pretensioning and Post-Tensioning in Loss Computations

The application of a second stage of prestress affects the initial stage of prestress. Additional elastic shortening loss in the pretensioning strands occurs as the post-tensioning tendons are stressed. Creep losses are affected for both stages of prestress because additional stresses are present to cause creep at the centroid of the prestressing strands or tendons. Details of these considerations are discussed below.

7.1.1.1   Elastic Shortening

7.1.1.1.1   *Pretensioned Strands*

The prestress force in the pretensioned strands is directly affected by the stressing of the post-tensioning tendons. The stressing of the post-tensioning tendons causes secondary elastic shortening losses in the pretensioned strands in the girder segments. This additional loss reduces the effective prestress in the pretensioned strands after the post-tensioning tendons are stressed.

The additional elastic shortening loss caused by post-tensioning is computed in the same way as the initial value for elastic shortening:

- Compute the concrete stress at the centroid of the pretensioned strands that is caused by the application of prestress (in this case, the stressing of the post-tensioning tendons)

- Compute the change in dead load stress that occurs at the centroid of the pretensioned strands when the post-tensioning tendons are stressed

- Multiply the sum of the two concrete stresses by the modular ratio to convert the total concrete stress into a steel stress

The pretensioned strands are affected by all of the post-tensioning tendons as they are stressed. Therefore, the additional elastic shortening is computed using an equation of the form of LRFD Eq. 5.9.5.2.3a-1 for pretensioned members rather than LRFD Eq. 5.9.5.2.3b-1 for post-tensioned members, in which the loss is reduced because of the effect of sequential application of post-tensioning on multiple tendons.

This additional elastic shortening is then computed using the following equation, which has the same form as LRFD Eq. 5.9.5.2.3a-1:

$$\Delta f_{pES2} = f_{cgp1} E_p / E_c \qquad \text{Eq. 7.1.1.1.1-1}$$

where:

f_{cgp1} = the stress at the centroid of the pretensioned strands (stage 1 prestress) caused by the stressing of the post-tensioning tendons (stage 2 prestress) and any accompanying change in dead load stress.

The stress f_{cgp1} in this equation is computed in the same manner as the f_{cgp} computed for the elastic shortening of the post-tensioning tendons, except that the stresses due to moments are computed at the centroid of the pretensioned strands instead of the post-tensioned tendons.

The post-tensioning force is applied to the composite section so composite section properties and eccentricities are used.

In this example, the change in dead load stress is caused when the stressing of the post-tensioning tendons lifts the girder from the temporary supports. This lifting creates moments caused by the application of the reactions at the temporary supports being resisted by the full span structure. This stress is computed using composite section properties.

Note that the modular ratio in this equation is computed using the specified concrete strength, f'_c, rather than the concrete strength at release, f'_{ci}, which is used to compute the initial elastic shortening loss. The higher value is used because the girder concrete will have achieved the full specified concrete strength by the time the second stage of prestress is applied when the post-tensioning strands are stressed.

This additional increment of elastic shortening loss is included in computing the effective stress in the pretensioned strands at all critical events including and following the stressing of the post-tensioning tendons.

7.1.1.1.2   *Post-Tensioning Tendons*

Elastic shortening of the post-tensioning tendons is unaffected by the pretensioned tendons, because the prestress effect from the pretensioned strands has already been applied to the girder.

7.1.1.2   Creep

The loss due to creep is computed using the following equation, which applies to both pretensioned strands and post-tensioning tendons:

$$\Delta f_{pCR} = 12.0\, f_{cgp} - 7.0\, \Delta f_{cdp} \qquad \text{LRFD Eq. 5.9.5.4.3-1}$$

This equation assumes that prestress is applied in only one stage.

When more than one stage of prestress is applied, the creep loss computed for both stages is affected by the other stage of prestress. This can be accounted for by adding a term to the above equation and modifying the definition of Δf_{cdp} as discussed below.

7.1.1.2.1   *Pretensioned Strands*

For the pretensioned strands, the additional compression at the level of the pretensioned strands caused by the stressing of the post-tensioning tendons increases the creep affecting the pretensioned strands. This effect is included by adding a term to LRFD Eq. 5.9.5.4.3-1 to obtain:

$$\Delta f_{pCR} = 12.0\, f_{cgp} - 7.0\, \Delta f_{cdp} + 7.0\, f_{cgp1} \qquad \text{Eq. 7.1.1.2.1-1}$$

where f_{cgp1} is defined in **Section 7.1.1.1.1**.

The additional compression at the level of the strands will increase the creep, so the new term is added to the existing equation. A coefficient of 7.0 is used for this term because the prestress is applied at approximately the same time as the additional dead loads that included in the Δf_{cdp} term. The term Δf_{cdp} includes the effect of all dead loads applied to the girder, both before and after post-tensioning, except for dead load effects that occur when additional prestress is applied, because those dead load stresses are included in f_{cgp} and Δf_{cgp1}.

7.1.1.2.2   *Post-Tensioning Tendons*

For the post-tensioning tendons, additional compression at the level of the tendons already exists when the tendons are stressed. This pre-existing stress will increase the creep affecting the post-tensioning tendons. This effect is included by adding a term to LRFD Eq. 5.9.5.4.3-1 to obtain:

$$\Delta f_{pCR} = 12.0\, f_{cgp} - 7.0\, \Delta f_{cdp} + 7.0\, f_{cgp2} \qquad \text{Eq. 7.1.1.2.2-1}$$

where:

f_{cgp2} = the stress at the centroid of the post-tensioning tendons (stage 2 prestress) caused by the pretensioned strands (stage 1 prestress) and any accompanying change in dead load stress.

As indicated in the preceding section, the additional compression at the level of the tendons increases the creep, so the new term is added to the existing equation. The compression due to the pretensioned strands is already present when the post-tensioning tendons are stressed. Therefore, some of the creep caused by the pretensioned strands has already occurred. The coefficient of 7.0 is used for this term because to reflect the reduced effect from this prestress, while not seeking to refine the analysis further or imply a higher level of accuracy.

The term Δf_{cdp} is computed using the same moments as were used to compute Δf_{cdp} for the pretensioned strands. It includes the effect of all dead loads applied to the girder, both before and after post-tensioning, except for dead load effects that occur when additional prestress is applied, because those dead load stresses are included in f_{cgp} and Δf_{cgp2}. The term Δf_{cdp} differs from the value computed for the pretensioned strands because it is computed at the centroid of the post-tensioning tendons.

7.1.2   Creep and Shrinkage Losses at Intermediate Construction Stages

To allow estimation of time-dependent losses at intermediate construction stages, a two-step process is used. First, the final creep and shrinkage losses are computed using equations

discussed above for creep losses or given in the *Specifications* for shrinkage losses. These final values are then factored by a time factor taken from the equations for creep and shrinkage in the *Specifications.*

These procedures are only applied to the pretensioned strands in this example because no critical events occur between stressing of the post-tensioning tendons and the final conditions. If a second stage of post-tensioning were used or if other intermediate stages were considered, these modifications would be applied to the creep and shrinkage losses affecting the post-tensioning tendons.

In some cases, the final creep loss includes the effect of dead loads that are not present at the event being considered. An example is that the deck load is used to compute the final loss due to creep, but that load has not been applied and therefore does not cause creep when the deck concrete is being placed. However, such details are neglected to avoid making the computations overly complex and to avoid significant fluctuations in the creep loss values. It should also be recognized that the equations in the *Specifications* are approximate and should not be taken as providing a high degree of accuracy in the losses computed.

The following equations are used to compute factors to reduce the final computed values for creep and shrinkage. These equations utilize only the time-dependent portions of the creep and shrinkage equations.

Creep, based on LRFD Eq. 5.4.2.3.2-1 and neglecting t_i:

$$k_{CRt} \;=\; \frac{t^{0.6}}{10.0 + t^{0.6}} \qquad\qquad\qquad\qquad \text{Eq. 7.1.2-1}$$

Shrinkage, based on LRFD Eq. 5.4.2.3.3-1:

$$k_{SHt} \;=\; \frac{t}{35.0 + t} \qquad\qquad\qquad\qquad\qquad \text{Eq. 7.1.2-2}$$

7.1.3   **Relaxation Losses at Intermediate Construction Stages**

Relaxation losses for intermediate construction stages are estimated using LRFD Eq. 5.9.5.4.4c-2, with modifications for low-relaxation strand. The variation of relaxation with time is provided by using the time-adjustment factors for creep and shrinkage, as discussed above, applied to the creep and shrinkage losses in the equation. Therefore, it could be recomputed at the different construction stages after the creep and shrinkage losses have been adjusted. However, this is an unnecessary refinement to a minor component of loss. Therefore, a single value of final relaxation loss will be computed using the final values for creep and shrinkage loss. This loss will not be adjusted at the different construction stages.

7.1.4   **Losses at Splice Locations**

The concrete in the splices is not pretensioned. Therefore, the only prestress effect at splice locations is caused by the post-tensioning tendons and the losses associated with them.

7.1.5   **Reduction of Effective Prestress due to Anchorage Effects**

7.1.5.1   **Pretensioned Strands**

At locations very near the ends of the pretensioned girder segments, which are within the transfer length, the effective prestress force from the pretensioned strands must be reduced, as is done for all pretensioned girders. This reduction is not actually a loss component, but has

the same effect, since it results in a reduction in the effective prestress. This reduction occurs as the prestress is applied during release of the pretensioned strands.

The effective stress in the pretensioned strands is assumed to vary linearly from zero at the end of the member to the full effective prestress force (as computed at each critical event) at a distance from each end of the girder equal to the transfer length. This variation is a result of the force in pretensioned strands being transferred to the concrete through bond between the concrete and strands.

The transfer length is defined as $60d_b$ in LRFD Article 5.11.4.1, where d_b is the nominal diameter of the pretensioned strands, or 3.0 ft for the 0.6 in. diameter strands used to pretension the girder segments.

7.1.5.2   **Post-Tensioning Tendons**

Since the post-tensioning tendons are anchored at the end of the girders by external anchorages and plates, the effective prestress force in the tendons is assumed to extend to the end of the girder.

7.2   **PRETENSIONED STRANDS**

In this section, sample calculations are shown for each component of prestress loss for the pretensioned strands. The calculations shown are for the center girder segment, so the properties for Girder Segment 2 are used. The effect of post-tensioning on the pretensioned strands is considered. The components of loss are then combined to determine losses at the critical construction stages.

As is generally done for the design of pretensioned girders, losses are computed at midspan of the member. For this bridge, with a center girder segment and two identical end girder segments, a prestress loss is computed at midspan for each of the girder segments. This loss is then used for the length of the girder segment. The effective prestress force is adjusted at the ends of the girder segments at locations within the transfer length.

7.2.1   **Loss Components**

7.2.1.1   **Elastic Shortening**

This section estimates only the elastic shortening loss at release. The simplified approach allowed by the *LRFD Specifications* was used. The additional elastic shortening from the effect of post-tensioning is computed in **Section 7.2.1.6**.

$$\Delta f_{pES1} = \frac{E_p}{E_{ci}} f_{cgp} = 5.37 \text{ ksi} \qquad\qquad \text{LRFD Eq. 5.9.5.2.3a-1}$$

$$f_{cgp} = \frac{F_p}{A} + \frac{F_p e^2}{I} - \frac{M_{GDL} e}{I} = 0.808 \text{ ksi}$$

F_p = $f_p A_{ps}$ = 820.3 k

A_{ps} = 4.34 in^2

f_p = $0.70 f_{pu}$ = 189 ksi *(Assumed)*   LRFD Art. 5.9.5.2.3a

e   = eccentricity of pretensioned strands for noncomposite section = 27.47 in.

E_p = 28,500 ksi

E_{ci} = 4,287 ksi

M_{GDL} = 1,634 k-ft = 19,608 k-in   Table 5.5.1-2

7.2.1.2   Creep

This section estimates creep losses for pretensioned strands caused by both pretensioned strands and post-tensioning tendons.

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} + 7.0 f_{cgp1} \qquad\qquad \text{Eq. 5.1.1.2.1-1}$$
$$= 12.0(0.808) - 7.0(0.775) + 7.0(2.708)$$
$$= 23.2 \text{ ksi}$$

f_{cgp} $= 0.808$ ksi (Elastic Shortening Calculation)

$$\Delta f_{cdp} = \frac{(M_{DDL} + M_{NCDL})e}{I} + \frac{M_{CDL}e_c}{I_c} = 0.775 \text{ ksi}$$

$$f_{cgp1} = \frac{F_{PT}}{A_c} + \frac{F_{PT}e_{PT}e_c}{I_c} - \frac{M_{pier}e_c}{I_c} = 2.708 \text{ ksi}$$

M_{DDL}	$= 1{,}360$ k-ft $= 16{,}320$ k-in	Table 5.5.1-2
M_{NCDL}	$= 252$ k-ft $= 3{,}020$ k-in	Table 5.5.1-2
M_{CDL}	$= 1{,}493$ k-ft $= 17{,}920$ k-in	Table 5.5.1-2
M_{pier}	$= 8{,}002$ k-ft $= 96{,}023$ k-in	Table 5.5.1-2
F_{PT}	$= 186.7(15.624) = 2{,}918$ k	Table 7.3.3-2
e	$= 27.5$ in.	

e_c $=$ eccentricity of pretensioned strands for composite section $= 48.2$ in.

e_{PT} $=$ average eccentricity of all post-tensioning tendons for composite section $= 52.6$ in.

7.2.1.3   Shrinkage

$$\Delta f_{pSR} = (17.0 - 0.150H) \qquad\qquad \text{LRFD Eq. 5.9.5.4.2-1}$$
$$= (17.0 - 0.150(75))$$
$$= 5.75 \text{ ksi}$$

H $= 75\%$

7.2.1.4   Relaxation at Release

This loss is only considered at the release of the pretensioned strands.

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right]f_{pj} = 1.98 \text{ ksi} \qquad \text{LRFD Eq. 5.9.5.4.4b-2}$$

t $= 1$ day

f_{pj} $= 202.5$ ksi

f_{py} $= 243$ ksi

7.2.1.5   Final Relaxation

This loss component depends on the other loss components. The additional elastic shortening due to post-tensioning is neglected in this calculation. This loss applies to all construction stages except the initial stage at which the pretensioned strands are released.

$$\Delta f_{pR2} = 0.3[20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})] \qquad \text{LRFD Eq. 5.9.5.4.4c-1}$$
$$= 0.3[20.0 - 0.4(5.37) - 0.2(5.75 + 23.22)]$$

$$= 3.62 \text{ ksi}$$

7.2.1.6 **Additional Elastic Shortening Loss due to Post-Tensioning**

As discussed in **Section 7.1.1.1.1**, the stressing of the post-tensioning tendons causes secondary elastic shortening losses in the pretensioned strands in the girder segments.

The stress at the centroid of the pretensioned strands caused by stressing the post-tensioning tendons and the pier removal load has already been computed as Δf_{cgp1} as part of the creep calculations in **Section 7.2.1.2**. This is the stress required in the equation below, so it will be substituted for f_{cgp} without further computations.

$$\Delta f_{pES2} = \frac{E_p}{E_c} f_{cgp}$$

LRFD Eq. 5.9.5.2.3a-1

$$= \frac{E_p}{E_c} f_{cgp1} = 15.79 \text{ ksi}$$

f_{cgp1} = 2.708 ksi   Section 7.2.1.2

E_p = 28,500 ksi

E_c = 4,888 ksi

7.2.2 **Losses at Critical Construction Stages**

Prestress losses for the pretensioned strands at the critical construction stages computed in this section are summarized in **Table 7.4-1**.

7.2.2.1 **At Release of Pretensioned Strands**

$$\Delta f_{PreT} = \Delta f_{pES1} + \Delta f_{pR1}$$
$$= 5.37 + 1.98$$
$$= 7.35 \text{ ksi}$$

7.2.2.2 **At Placement of Deck and Splice Concrete**

$$\Delta f_{PreT} = \Delta f_{pES1} + k_{CRt}\Delta f_{pCR} + k_{SHt}\Delta f_{pSR} + \Delta f_{pR2}$$
$$= 5.37 + 0.54(23.22) + 0.63(5.75) + 3.62$$
$$= 25.06$$

k_{CRt} = 0.54 for 60 Days   Eq. 7.1.2-1

k_{SHt} = 0.63 for 60 Days   Eq. 7.1.2-2

7.2.2.3 **At Stressing of Post-Tensioning Tendons**

$$\Delta f_{Pre} = \Delta f_{pES1} + k_{CRt}\Delta f_{pCR} + k_{SHt}\Delta f_{pSR} + \Delta f_{pR2} + \Delta f_{pES2}$$
$$= 5.37 + 0.57(23.22) + 0.68(5.75) + 3.62 + 15.79$$
$$= 41.93 \text{ ksi}$$

k_{CRt} = 0.57 for 75 Days   Eq. 7.1.2-1

k_{SHt} = 0.68 for 75 Days   Eq. 7.1.2-2

7.2.2.4 **At Final Conditions after Losses**

$$\Delta f_{Pre} = \Delta f_{pES1} + \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pR2} + \Delta f_{pES2}$$
$$= 5.37 + 23.22 + 5.75 + 3.62 + 15.79$$

= 53.76 ksi

7.3   **POST-TENSIONING TENDONS**

In this section, sample calculations are shown for each component of prestress loss for the post-tensioned tendons. The calculations for the friction and anchor set losses are summarized for all locations along the length of the bridge while other losses are computed at midspan. The effect of the pretensioned strands on the time dependent losses for the post-tensioning tendons is considered. The components of loss are then combined to determine losses at the critical construction stages.

The time-dependent losses for the post-tensioning tendons computed at midspan of the spliced girder are used for all locations along the spliced girder.

7.3.1   **Loss Components**

7.3.1.1   **Friction and Anchor Set**

Friction and anchor set losses are closely related. They are estimated using an iterative process that continues by reducing the temporary jacking stress, f_{pj}, until all stress limits are satisfied at the critical locations. For this design, the initial iteration used a jacking stress at the anchorage slightly less than the maximum permitted value, which resulted in all other stress limits being violated. The jacking stress was reduced until the stress limit was satisfied at the dead end anchorage, which was the location that governed this design. The critical locations, stress limits and the stresses used for the initial and final iterations are shown in **Table 7.3.1.1-1**.

To simplify the computation of friction and anchor set losses, the rate of friction loss (or the friction loss per unit length of tendon) is assumed to be constant. This assumption corresponds to an equivalent parabolic (or circular) tendon profile. This is a reasonable assumption for this spliced girder as discussed in **Section 6.2.2.2**.

Friction and anchor set losses are computed using the length of the spliced girder from anchorage to anchorage, which is 198 ft.

Table 7.3.1.1-1   Schematic of Stress Limits and Iterations on Jacking Stress

Location	Stress Limit		Initial Estimate	Final Estimate	Description
		(ksi)	(ksi)	(ksi)	
1	0.90 f_{py}	218.7	215.0	202.5	Temporary Jacking Stress at Stressing End Anchorage
2	0.70 f_{pu}	189.0	199.0	187.0	Stress after Seating at Stressing End Anchorage
3	0.74 f_{pu}	199.8	206.9	194.4	Stress after Seating at End of Anchor Set Loss
4	0.70 f_{pu}	189.0	200.9	189.2*	Stress after Seating at Dead End Anchorage
5	------	------	------	193.7	Midspan

Shaded cells govern.

* Slightly exceeds stress limit – say OK.

7.3.1.1.1   *Friction*

Friction losses are computed using the equation in the *LRFD Specifications*. The computation for the friction loss at midspan is shown.

$$\Delta f_{pF} \quad = \quad f_{pj}\left(1 - e^{-(Kx+\mu\alpha)}\right) \qquad\qquad \text{LRFD Eq. 5.9.5.2.2b-1}$$

$$= \quad 6.78 \text{ ksi}$$

f_{pj} = 202.5 ksi (final iteration)   Table 7.3.1.1-1

K   = 0.0002 per ft

x   = 99 ft (half of girder length)

μ = 0.25

α = 0.057 rads (from **Figure 6.2.1.1-1**) (half span)

7.3.1.1.2 | *Anchor Set*

Anchor set loss is computed by estimating the loss that occurs when each tendon pulls into the anchorage at seating. This pull in reduces the stress at the anchorage and for a distance that depends on the rate of friction loss in the tendon.

The constant rate of friction loss, R_F can be determined using the friction loss computed in **Section 7.3.1.1.1** and its corresponding location, x (midspan).

$$R_F = \frac{\Delta f_{pF}}{x} = \frac{6.78}{99}$$ Eq. 7.3.1.1.2-1

$$= 0.0692 \text{ ksi/ft} = 0.00577 \text{ ksi/in.}$$

The distance over which the anchor set loss extends, x_A, is computed using the equation

$$x_A = \sqrt{\frac{A_{set}E_p}{R_F}}$$ Eq. 7.3.1.1.2-2

$$= 1,361 \text{ in.} = 113.4 \text{ ft}$$

A_{set} = 0.375 in.

E_p = 28,500 ksi

This calculation indicates that the anchor set loss extends for 57 % of the length of the tendon, or 15.4 ft beyond midspan.

The anchor set loss at the stressing anchorage, which is the maximum anchor set loss, is computed using the equation

$$\Delta f_A = 2 x_A R_F$$ Eq. 7.3.1.1.2-3

$$= 2 (1,361) (0.00577)$$

$$= 15.7 \text{ ksi}$$

The anchor set loss varies linearly from this maximum value at the stressing anchorage, to zero at the distance x_A from the stressing anchorage, as shown in **Table 7.3.1.1-1**.

7.3.1.2 | **Elastic Shortening**

$$\Delta f_{pES} = \frac{N-1}{2N}\frac{E_p}{E_c}f_{cgp} = 6.94 \text{ ksi}$$ LRFD Eq. 5.9.5.2.3b-1

$$f_{cgp} = \frac{F_{PT}}{A_c} + \frac{F_{PT}e_{PT}^2}{I_c} - \frac{M_{pier}e_{PT}}{I_c} = 2.856 \text{ ksi}$$

F_{PT} = $f_p A_{ps}$ = 2,926 kips

f_p = 193.3 − 6.0 = 187.3 ksi *(Estimate Δf_{pES} = 6.0 ksi)*

A_{ps} = 15.624 in^2

e_{PT} = 52.6 in.

M_{pier} = 8,002 k-ft = 96,023 k-in

N = 6 Tendons

E_p = 28,500 ksi

E_c = 4,888 ksi

| 7.3.1.3 | **Creep** |

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \Delta f_{cdp} + 7.0\, f_{cgp2}$$
$$= 12.0(2.856) - 7.0(0.871) + 7.0(0.835)$$
$$= 34.01 \text{ ksi}$$

$f_{cgp} = 2.856 \text{ ksi } (\textit{See Elastic Shortening Calculation})$

$$\Delta f_{cdp} = \frac{(M_{DDL} + M_{NCDL})e_{PT}}{I} + \frac{M_{CDL}e_{PT}}{I_c} = 0.871 \text{ ksi}$$
Section 7.3.1.2

$$f_{cgp2} = \frac{F_{pre}}{A} + \frac{F_{pre}e\,e_{PT}}{I} - \frac{M_{self}e_{PT}}{I} = 0.835 \text{ ksi}$$

M_{DDL}	= 1,360.0 k-ft = 16,320.0 k-in	Table 5.5.1-2
M_{NCDL}	= 2,591.7 k-ft = 3,020.4 k-in	Table 5.5.1-2
M_{CDL}	= 1,493.3 k-ft = 17,919.6 k-in	Table 5.5.1-2
M_{pier}	= 8,001.9 k-ft = 96,022.8 k-in	Table 5.5.1-2
e_{PT}	= 52.5 in.	
e	= 48.2 in.	
F_{Pre}	= 820.3 kips	

| 7.3.1.4 | **Shrinkage** |

$$\Delta f_{pSR} = (13.5 - 0.123H)$$
LRFD Eq. 5.9.5.4.2-2
$$= (13.5 - 0.123(75))$$
$$= 4.28 \text{ ksi}$$

$H = 75\%$

| 7.3.1.5 | **Final Relaxation** |

$$\Delta f_{pR2} = 0.3[20.0 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})]$$
LRFD Eq. 5.9.5.4.4c-2
$$= 0.3[20.0 - 0.3(6.78) - 0.4(6.94) - 0.2(4.28 + 34.01)]$$
$$= 2.26 \text{ ksi}$$

| 7.3.2 | **Losses at Critical Construction Stages** |

Prestress losses computed in this section for each critical construction stage are summarized in **Table 7.4-1**.

| 7.3.2.1 | **At Post-Tensioning** |

$$\Delta_{PT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES}$$
$$= 2.04 + 6.78 + 6.94$$
$$= 15.76 \text{ ksi}$$

| 7.3.2.2 | **At Final Conditions after Losses** |

$$\Delta_{PT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pCR} + \Delta f_{pSR} + \Delta f_{pR2}$$
LRFD Eq. 5.9.5.1-2
$$= 2.04 + 6.78 + 6.94 + 34.01 + 4.28 + 2.26$$
$$= 56.61 \text{ ksi}$$

7.4   **SUMMARY OF PRESTRESS LOSSES**

Prestress losses for both pretensioned strands and post-tensioning tendons are presented in **Table 7.4-1**. Components of prestress loss for pretensioned strands are presented for each girder segment, while components of prestress loss for the post-tensioning tendons are presented for the ends and midspan. The table also gives the losses at the critical construction stages as appropriate.

In design, the effective prestress for either pretensioned strands or post-tensioning tendons is the quantity used in computations. Therefore, the effective prestress for both types of prestressing at the critical construction stages is presented in **Table 7.4-2**. The effective prestress for Girder Segments 1 and 3 differ because of the effect of the post-tensioning tendons.

The initial stresses from which the losses are subtracted are also shown in **Table 7.4-2**. It should be noted that the initial pretensioned strand stress, f_{po}, and the jacking stress for the post-tensioning tendons, f_{pj}, appear to be the same for the two types of prestressing. While the stresses are equal, this is coincidental, because the jacking stress for the post-tensioning tendons was determined from the friction and anchor set losses and the tendon stress limits.

Table 7.4-1   Summary of Prestress Losses

GIRDER SEGMENTS					
	Segment		1	2	3
	Loss Components:				
Pretension Strands	Δf_{pR1}	(ksi)	1.98	1.98	1.98
	Δf_{pES1}	(ksi)	3.75	5.37	3.75
	Δf_{pCR}	(ksi)	20.88	23.22	21.14
	Δf_{pSR}	(ksi)	5.75	5.75	5.75
	Δf_{pR2}	(ksi)	3.95	3.62	3.94
	Δf_{pES2}	(ksi)	12.92	15.79	13.13
	Total Loss at Stages of Construction::				
	Δf_{Pre} at Release of Pretensioned Strands	(ksi)	5.73	7.35	5.73
	Δf_{Pre} at Placement of Deck & Splice Concrete	(ksi)	25.21	25.06	25.34
	Δf_{Pre} at Stressing of Post-tensioning Tendons	(ksi)	39.06	41.93	39.40
	Δf_{Pre} at Final Conditions after Losses	(ksi)	47.25	53.76	47.70
SPLICED GIRDER					
	Location		Left End	Midspan	Right End
	Loss Components:				
Post-Tensioning Tendons	Δf_{pF}	(ksi)	0.00	6.78	13.33
	Δf_{pA}	(ksi)	15.48	2.04	0.00
	Δf_{pES}	(ksi)	6.94	6.94	6.94
	Δf_{pCR}	(ksi)	34.23	34.23	34.23
	Δf_{pSR}	(ksi)	4.28	4.28	4.28
	Δf_{pR2}	(ksi)	2.25	2.25	2.25
	Total Loss at Stages of Construction::				
	Δf_{PT} at Stressing of Post-tensioning Tendons	(ksi)	22.42	15.80	20.27
	Δf_{PT} at Final Conditions after Losses	(ksi)	63.18	56.56	61.03
Note: Left end is stressing end for post-tensioning					

Table 7.4.2-2   Summary of Initial and Effective Stresses

Girder Segment:		1	2	3
Pretensioned Strands:				
f_{po}		202.5	202.5	202.5
Δf_{Pre} at Release of Pretensioned Strands	(ksi)	196.8	195.2	196.8
Δf_{Pre} at Placement of Deck & Splice Concrete	(ksi)	177.3	177.4	177.2
Δf_{Pre} at Stressing of Post-tensioning Tendons	(ksi)	163.4	160.6	163.1
Δf_{Pre} at Final Conditions after Losses	(ksi)	155.3	148.7	154.8
Post-Tensioning Tendons:		Left End	Midspan	Right End
f_{pj}	(ksi)		202.5	
Δf_{PT} at Stressing of Post-tensioning Tendons	(ksi)	180.1	186.7	182.2
Δf_{PT} at Final Conditions after Losses	(ksi)	139.3	145.9	141.5
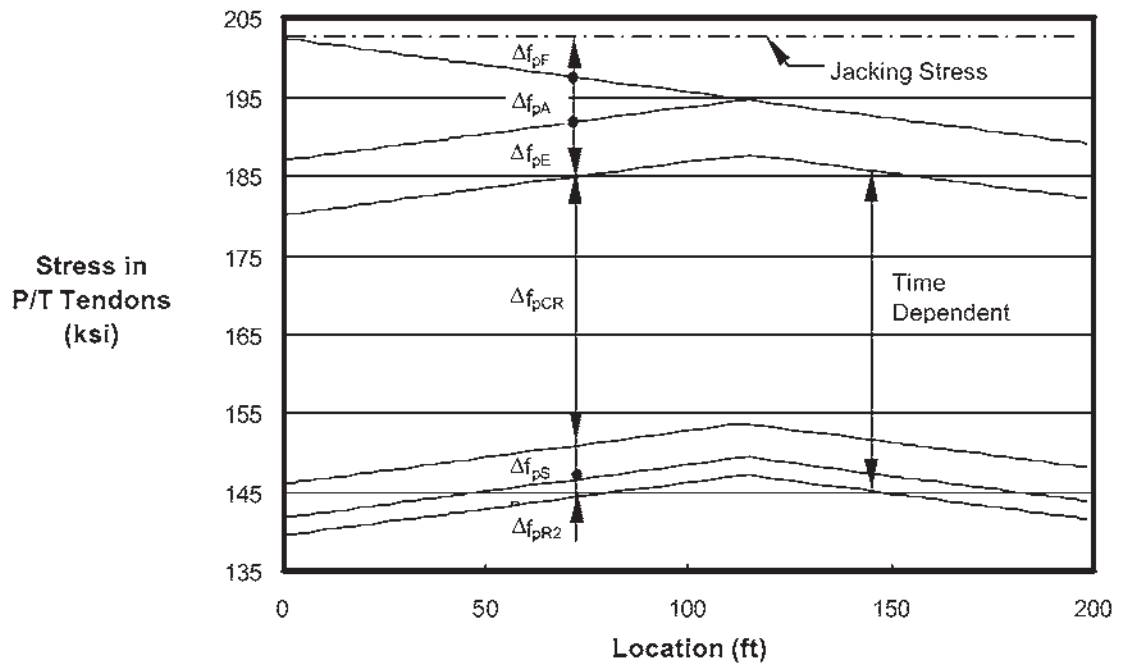
Figure 7.3.1.1-1 Prestress Remaining After Losses

8   DESIGN FOR FLEXURE

The design of this bridge for flexure is the focus of the design process. The process differs from typical designs because of the combination of pretensioned strands and post-tensioning tendons and the various stages of construction.

The service limit state requirements for flexure govern the required prestress force. After the quantity of prestress has been determined, the flexural strength of the section at the strength

limit state is checked at critical sections. The fatigue limit state is mentioned, but no computations are required for this bridge.

8.1   **SERVICE LIMIT STATE**

For a typical pretensioned girder, the critical service limit states are release (before time-dependent losses) and final conditions (after all losses).

For this spliced girder, there are four critical service limit state design stages:

1. Release Pretensioned Strands
2. Placement of Deck and Splice Concrete
3. Stress Post-Tensioning Tendons
4. Final Conditions after Losses

Each of these service limit states is addressed in the following, first to establish the limiting stresses, then the tabulation of stresses along the beam at the limit state. All limiting stresses are satisfied by the design.

If compressive stresses were close to stress limits for the computed structure, the case of full service loads but only partial losses could possibly govern. However, this is not critical for this bridge.

8.1.1   **Stress Limits**

The following stress limits are used for design at the service limit state. Where stress limits are not given in the *LRFD Specifications*, for a necessary construction stage, a limit is proposed.

8.1.1.1   **Prestressing Strands**

8.1.1.1.1   ***Pretensioning Strands***

The stress limits for low relaxation pretensioning strands are: LRFD Table 5.9.3-1

Immediately prior to transfer:

f_{pi} = 0.75 f_{pu} = 202.5 ksi

At Service Limit State after losses:

f_{pe} = 0.80 f_{py} = 0.80 (0.90f_{pu}) = 0.72 f_{pu} = 194.4 ksi

8.1.1.1.2   ***Post-Tensioning Strands***

The stress limits for low relaxation post-tensioning strands are: LRFD Table 5.9.3-1

Prior to seating:

f_{pi} = 0.90 f_{py} = 0.90 (0.90f_{pu}) = 0.81 f_{pu} = 218.7 ksi

At anchorages and couplers immediately after anchor set:

f_{pi} = 0.70 f_{pu} = 189.0 ksi

At end of seating loss zone immediately after anchor set:

f_{pi} = 0.74 f_{pu} = 199.8 ksi

At Service Limit State after losses:

f_{pe} = 0.80 f_{py} = 0.80 (0.90f_{pu}) = 0.72 f_{pu} = 194.4 ksi

It should be noted that all of these limits will not be satisfied simultaneously. Therefore, iterative computations are employed to determine the stresses that govern. See **Section 7.3.1**.

8.1.1.2   **Concrete**

The Stress Limits at the different stages of construction are discussed in the following sections. The stress limit values for the different types of concrete at each of the stages of construction are summarized in **Table 8.1.1.2.5-1**.

8.1.1.2.1   ***Temporary Stresses at Release of Pretensioned Strands***

These stress limits apply only to the girder concrete.

Compression

$\qquad$ f_{cR} $\quad = 0.60\ f'_{ci}$ $\hfill$ LRFD Art. 5.9.4.1.1

Tension

$\qquad$ f_{tR1} $\quad = 0.0948\ \sqrt{f'_{ci}} \le 0.2$ ksi $\hfill$ LRFD Table 5.9.4.1.2-1

$\qquad$ or

$\qquad$ f_{tR2} $\quad = 0.24\ \sqrt{f'_{ci}}$ with reinforcement to resist the tensile force in the concrete

Since the pretensioning in the girder segments is relatively small, the f_{tR1} limit is used. The additional two top strands in Girder Segment 2 could be eliminated if the f_{tR2} limit were used with additional reinforcement.

8.1.1.2.2   ***Temporary Stresses at Placement of Deck and Splice Concrete***

The *AASHTO LRFD Specifications* only provide stress limits for concrete at initial and final conditions. Stress limits for intermediate critical events, which occur during the construction of spliced girder bridges, are not specified.

The stress conditions at placement of the deck and splice concrete are temporary conditions. However, temporary stress limits used at release are not appropriate for all conditions. Therefore, stress limits are suggested for compression and tension in the following.

At this stage, the section is still comprised of simply supported girders with no composite deck, so these stress limits are only required for the girder concrete. The specified girder concrete strength at 28 days is used to compute these stress limits because the girders are generally required to reach this strength prior to shipment to the site.

Compression

Since this is a temporary condition, the stress limit for compression given for release is used. However, the stress limit is computed using the girder concrete strength of 28 days.

$\qquad$ f_{cD} $\quad = 0.60\ f'_{c}$ $\hfill$ LRFD Table 5.9.4.2.1-1

Tension

During placement of the deck concrete, the precompressed tensile zone of the girders will be subjected to significant tensile stresses. Because the *Specifications* do not give a tensile stress limit for the precompressed tensile zone for temporary conditions at release, the tensile stress limit for the precompressed tensile zone at final conditions is used for this temporary intermediate stage. This limit will prevent cracking of the girders from the applied dead loads.

$\qquad$ f_{tD} $\quad = 0.19\ \sqrt{f'_{c}}$ $\hfill$ LRFD Table 5.9.4.2.2-1

8.1.1.2.3 | *Temporary Stresses at Stressing of Post-Tensioning Tendons*

Concrete stresses at stressing of the post-tensioning tendons are compared to the temporary stress limits before losses given in LRFD Article 5.9.4.1 because these stresses apply to a post-tensioned member at stressing.

Since splices have been made and the composite deck is present when the post-tensioning tendons are stressed, stress limits are checked for the splice and deck concretes. As with the previous event, the specified girder concrete strength at 28 days is used to compute these stress limits. The minimum specified strengths at post-tensioning for the deck and splice concretes are used for these concretes.

At this stage, the composite section is present and the girders are spliced. Therefore, these stress limits apply to the girder, deck and splice concrete.

<u>Compression</u>

Girder Concrete: (no change from previous event)

f_{cgP} = 0.60 f'_c *LRFD Table 5.9.4.2.1-1*

Deck Concrete:

f_{cdP} = 0.60 f'_{cdi} *LRFD Table 5.9.4.2.1-1*

Splice Concrete:

f_{csP} = 0.60 f'_{csi} *LRFD Table 5.9.4.2.1-1*

<u>Tension</u>

The limiting stresses for the girder and deck concrete apply to regions other than the precompressed tensile zone and are unlikely to govern for any of the critical sections. However, they may govern at other locations.

Girder Concrete: (no change from previous event)

f_{tD1} = 0.0948 $\sqrt{f'_c} \leq$ 0.2 ksi   *LRFD Table 5.9.4.1.2-1*

or

f_{tD2} = 0.24 $\sqrt{f'_c}$ with reinforcement to resist the tensile force in the concrete

Deck Concrete:

f_{tD1} = 0.0948 $\sqrt{f'_{cdi}} \leq$ 0.2 ksi   *LRFD Table 5.9.4.1.2-1*

or

f_{tD2} = 0.24 $\sqrt{f'_{cdi}}$ with reinforcement to resist the tensile force in the concrete

Splice Concrete:

For the splice concrete, the limiting stresses for segmentally constructed bridges are used. A stress limit is given for both the precompressed tensile zone and for other areas. Both require additional bonded reinforcement.

Splice Concrete – Precompressed Tensile Zone:

f_{tD1} = 0.0948 $\sqrt{f'_{csi}}$ with auxiliary reinforcement to resist   *LRFD Table 5.9.4.1.2-1*
            the tensile force in the concrete

Splice Concrete – Other Areas:

f_{tD1} = 0.19 $\sqrt{f'_{csi}}$ with auxiliary reinforcement to resist   LRFD Table 5.9.4.1.2-1
      the tensile force in the concrete

8.1.1.2.4   ***Stresses at Final Conditions after Losses***

Stress limits at final conditions are given below for the girder, deck and splice concrete. Compressive stresses will be checked for the girder and deck slab, since post-tensioning was applied to the composite section.

Stress limits are computed for girder, deck and splice concrete. Tensile stress limits are not computed for deck concrete because it cannot be in the precompressed tensile zone in a simple span.

<u>Compression</u>

Compressive stress limits for the girder, deck and splice concrete are computed using the same equations, with the appropriate concrete strength:

f_{c1} = 0.60 $\phi_w f'_c$, for full service loads (ϕ_w = 1)   LRFD Table 5.9.4.2.1-1

f_{c2} = 0.45 f'_c, for effective prestress (PS) and full dead loads (DL)

f_{c3} = 0.40 f'_c, for live load plus one-half of effective PS and full DL

<u>Tension</u>

Girder Concrete:

For the precompressed compression zone, assuming moderate corrosion conditions:

f_{t1} = 0.19 $\sqrt{f'_c}$ LRFD Table 5.9.4.2.2-1

Tensile stress limits are not given at final conditions for locations other than the precompressed compression zone because the conditions at release or post-tensioning will govern for tensile stresses in these regions for simple spans. Therefore, tensile stresses will only be checked in the precompressed tensile zone.

Deck Concrete:

No tensile stress limits apply because no portion of the deck is a precompressed tensile zone in a simple span girder.

Splice Concrete:

For the splice concrete, limiting stresses for segmentally constructed bridges are used. The splice is a Type A joint with bonded reinforcement. For this type of joint, a stress limit is given for both the precompressed tensile zone and for other areas.

Splice Concrete − Precompressed Tensile Zone:

f_{tD1} = 0.0948 $\sqrt{f'_{cs}}$ with auxiliary reinforcement to resist   LRFD Table 5.9.4.2.2-1
      the tensile force in the concrete

Splice Concrete − Other Areas:

f_{tD1} = 0.19 $\sqrt{f'_{cs}}$ with auxiliary reinforcement to resist   LRFD Table 5.9.4.2.2-1
      the tensile force in the concrete

8.1.1.2.5   ***Summary of Stress Limits***

The stress limit values for the different types of concrete at each of the stages of construction are given in **Table 8.1.1.2.5-1**.

Table 8.1.1.2.5-1 Summary of Concrete Stress Limits

Stage of Construction	Type of Stress	Girder Concrete	Deck Concrete	Splice Concrete
1. Release of Pretensioned Strands	*Compression*	3.000	–	–
	Tension	-0.200	–	–
2. Placement of Deck and Splice Concrete	*Compression*	3.900	–	–
	Tension	-0.200	–	–
3. Stressing of Post-Tensioning Tendons	*Compression*	3.900	2.700	2.700
	Tension	-0.612	-0.09	-0.509
4. Final Conditions after Losses	*Compression 1*	3.900	2.700	2.700
	Compression 2	2.925	2.025	2.025
	Compression 3	2.600	1.800	1.800
	Tension	-0.484	–	-0.201

Compression 1 – Full service load

Compression 2 – Full dead load

Compression 3 – Full live load plus one half of the prestress and dead load effect

8.1.2   **Service Limit State Stresses for Components of Load**

In this section, concrete stresses are computed along the girder segments for each component of applied load and prestress. Concrete stresses are given for the top and bottom of the girder and for the top of the deck slab. The stresses are presented in tables for each girder segment, except where stresses are identical for Girder Segments 1 and 3, in which case a single table is presented. The stresses are computed using moments presented in tables in **Section 5.5.1** and section properties from **Section 5.4**.

The first pair of tables contains concrete stresses caused by loads applied to the noncomposite section. The second pair of tables contains concrete stresses caused by loads applied to the composite section.

A third set of tables gives concrete stresses caused by prestress. These tables give the effects of the pretensioned strands and post-tensioning tendons separately for each critical construction stage for which the prestress is active. The effects of prestress losses are included in these stresses by using the effective prestress values for each construction stage given in **Table 7.3.3-2**. Pretensioned strands are active for all four construction stages, but post-tensioning tendons are only active for the last two construction stages, so concrete stresses are only shown for the stages in which the specific prestress is active.

Stresses computed in these tables are presented graphically in **Figures 8.1.2-1** through **8.1.2-3** to show the component stresses for at the top of deck and top and bottom of girder.

The concrete stresses presented in this section are combined to determine the stresses at the critical construction stages in the next section. To assist in understanding how the values are combined, the columns for each component of applied load are given a letter designation. These letters will be used to indicate the combinations of loads included in each critical construction stage in the following section.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
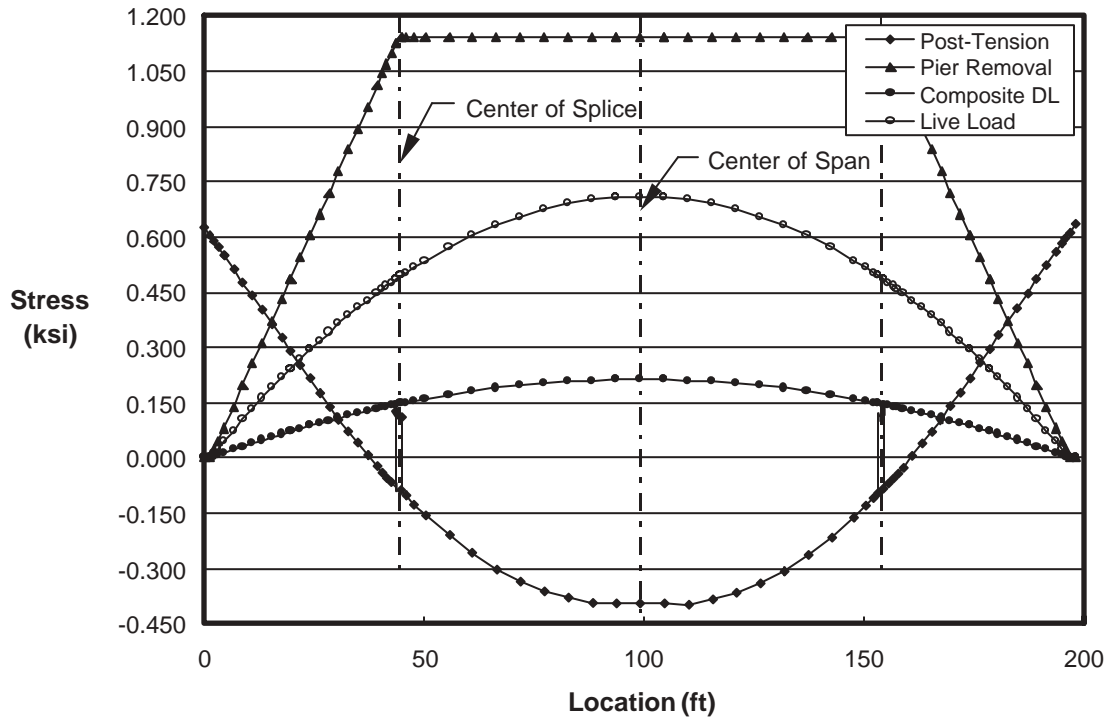
Figure 8.1.2-1   Concrete Stresses from Applied Loads and Prestress at Top of Deck
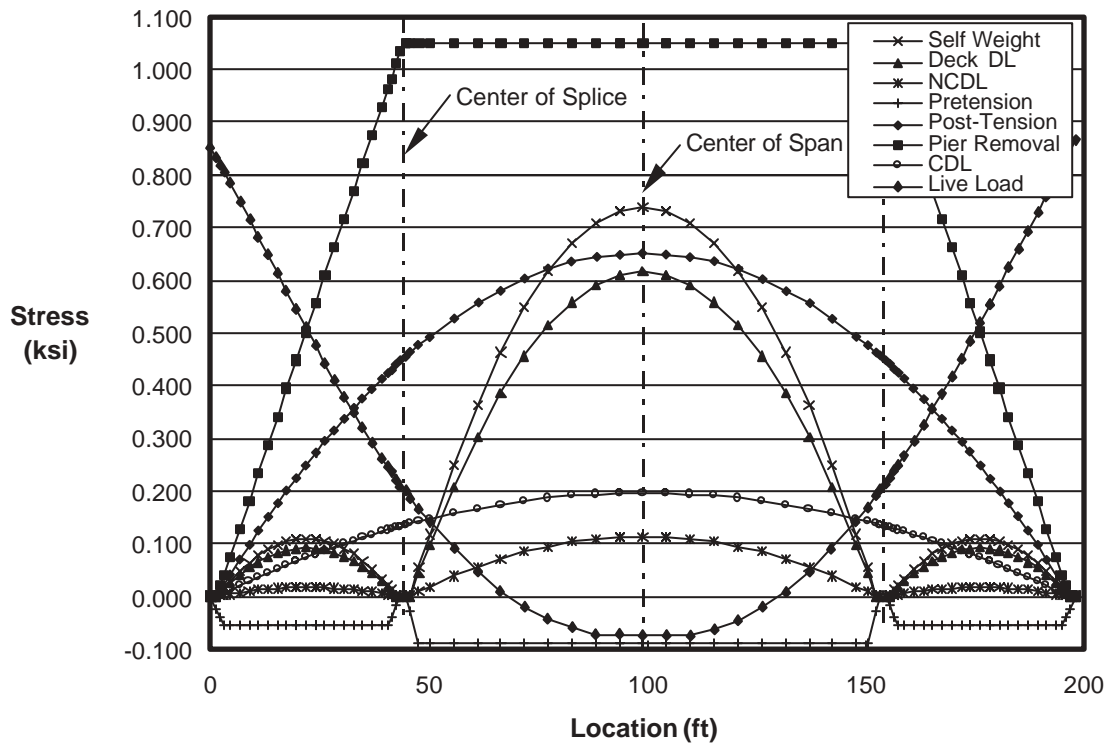
Figure 8.1.2-2   Concrete Stresses from Applied Loads and Prestress at Top of Girder

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
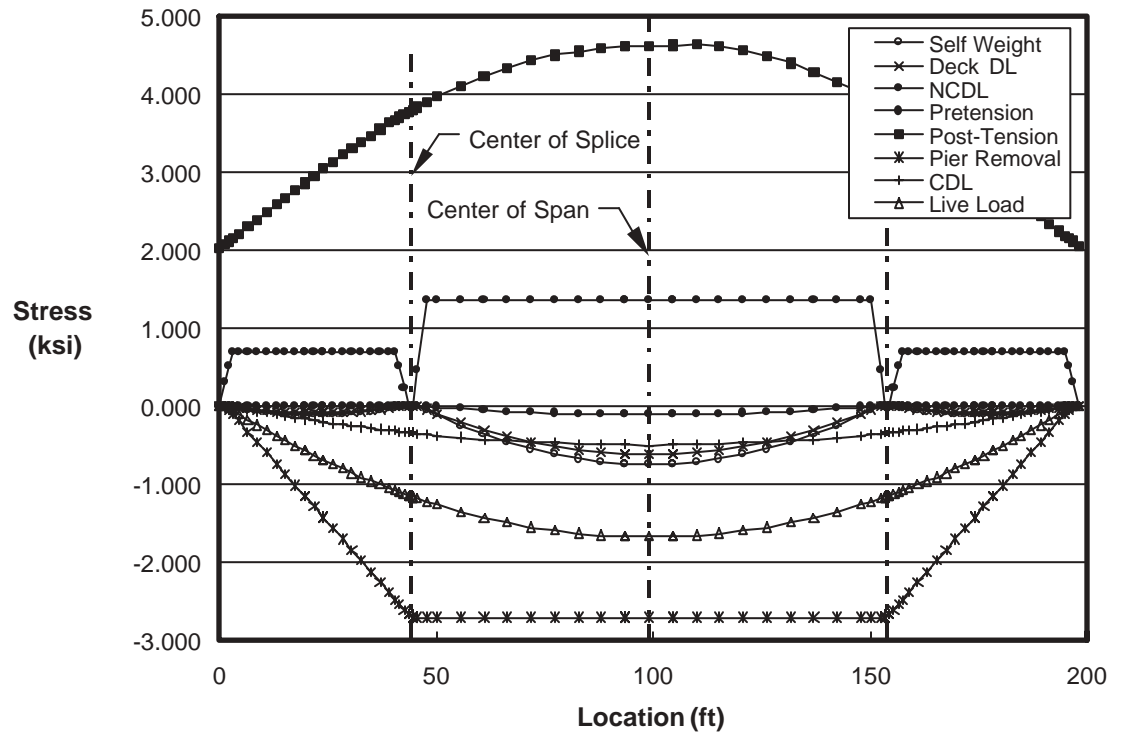
Figure 8.1.2-3   Concrete Stresses from Applied Loads and Prestress at Bottom of Girder

Table 8.1.2-1   Concrete Stresses from Components of Applied Loads on Noncomposite Section – Girder Segments 1 and 3

Location from CL Permanent Bearing (ft)	Stresses due to Self Weight (ksi)		Stresses due to Deck DL (ksi)		Stresses due to NCDL (ksi)		Stresses due to Construction LL (ksi)	
	A		B		C		D	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.8	0.009	-0.009	0.007	-0.007	0.001	-0.001	0.001	-0.001
1.7	0.017	-0.017	0.014	-0.015	0.003	-0.003	0.003	-0.003
3.0	0.030	-0.031	0.025	-0.025	0.005	-0.005	0.005	-0.005
5.2	0.049	-0.050	0.040	-0.041	0.007	-0.008	0.008	-0.008
7.4	0.065	-0.066	0.054	-0.055	0.010	-0.010	0.010	-0.010
9.5	0.078	-0.080	0.065	-0.067	0.012	-0.012	0.012	-0.013
11.7	0.090	-0.092	0.075	-0.076	0.014	-0.014	0.014	-0.014
13.9	0.099	-0.100	0.082	-0.084	0.015	-0.015	0.015	-0.016
16.1	0.105	-0.107	0.087	-0.089	0.016	-0.016	0.016	-0.017
18.2	0.109	-0.111	0.090	-0.092	0.017	-0.017	0.017	-0.017
20.4	0.110	-0.112	0.092	-0.094	0.017	-0.017	0.017	-0.018
22.6	0.109	-0.111	0.091	-0.093	0.017	-0.017	0.017	-0.017
24.8	0.106	-0.108	0.088	-0.090	0.016	-0.017	0.017	-0.017
26.9	0.100	-0.102	0.083	-0.085	0.015	-0.016	0.016	-0.016
29.1	0.091	-0.093	0.076	-0.077	0.014	-0.014	0.014	-0.015
31.3	0.080	-0.082	0.067	-0.068	0.012	-0.013	0.013	-0.013
33.5	0.067	-0.068	0.056	-0.057	0.010	-0.011	0.010	-0.011
35.6	0.051	-0.052	0.043	-0.043	0.008	-0.008	0.008	-0.008
37.8	0.033	-0.034	0.027	-0.028	0.005	-0.005	0.005	-0.005
39.2	0.020	-0.021	0.017	-0.017	0.003	-0.003	0.003	-0.003
40.0	0.012	-0.012	0.010	-0.010	0.002	-0.002	0.002	-0.002
41.2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
42.2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Note 1: Girder Segment 1 shown – Girder Segment 3 similar

Note 2: Highlighted rows indicate the centerline of the permanent and temporary bearings.

Table 8.1.2-2   Concrete Stresses from Components of Applied Loads on Noncomposite Section – Girder Segment 2

Location from CL Permanent Bearing (ft)	Stresses due to Self Weight (ksi)		Stresses due to Deck DL (ksi)		Stresses due to NCDL (ksi)		Stresses due to Construction LL (ksi)	
	A		B		C		D	
	Top	*Bottom*	*Top*	*Bottom*	*Top*	*Bottom*	*Top*	*Bottom*
43.3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
44.3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
46.3	0.054	-0.056	0.045	-0.046	0.008	-0.009	0.009	-0.009
48.8	0.118	-0.120	0.098	-0.100	0.018	-0.019	0.018	-0.019
54.2	0.248	-0.253	0.207	-0.211	0.038	-0.039	0.039	-0.040
59.6	0.363	-0.371	0.303	-0.309	0.056	-0.057	0.057	-0.058
65.1	0.463	-0.472	0.386	-0.393	0.071	-0.073	0.073	-0.074
70.5	0.548	-0.559	0.456	-0.465	0.084	-0.086	0.086	-0.088
75.9	0.617	-0.629	0.513	-0.524	0.095	-0.097	0.097	-0.099
81.4	0.670	-0.684	0.558	-0.569	0.103	-0.105	0.105	-0.107
86.8	0.709	-0.723	0.590	-0.602	0.109	-0.111	0.111	-0.113
92.2	0.732	-0.747	0.609	-0.621	0.113	-0.115	0.115	-0.117
97.7	0.740	-0.754	0.616	-0.628	0.114	-0.116	0.116	-0.118
103.1	0.732	-0.747	0.609	-0.621	0.113	-0.115	0.115	-0.117
108.5	0.709	-0.723	0.590	-0.602	0.109	-0.111	0.111	-0.113
114.0	0.670	-0.684	0.558	-0.569	0.103	-0.105	0.105	-0.107
119.4	0.617	-0.629	0.513	-0.524	0.095	-0.097	0.097	-0.099
124.8	0.548	-0.559	0.456	-0.465	0.084	-0.086	0.086	-0.088
130.3	0.463	-0.472	0.386	-0.393	0.071	-0.073	0.073	-0.074
135.7	0.363	-0.371	0.303	-0.309	0.056	-0.057	0.057	-0.058
141.1	0.248	-0.253	0.207	-0.211	0.038	-0.039	0.039	-0.040
146.6	0.118	-0.120	0.098	-0.100	0.018	-0.019	0.018	-0.019
149.0	0.054	-0.056	0.045	-0.046	0.008	-0.009	0.009	-0.009
151.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
152.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Note: Highlighted rows indicate the centerline of the temporary bearings and midspan

Table 8.1.2-3   Concrete Stresses from Components of Applied Loads on Composite Section – Girder Segments 1 and 3

Location from CL Permanent Bearing (ft)	Stresses due to Pier Removal (ksi)			Stresses due to CDL (ksi)			Stresses due to LL + IM (ksi)		
	E			F			G		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
0.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.8	0.023	0.021	-0.053	0.004	0.003	-0.009	0.012	0.011	-0.029
1.7	0.045	0.041	-0.106	0.007	0.007	-0.017	0.024	0.022	-0.058
3.0	0.081	0.074	-0.192	0.013	0.012	-0.031	0.044	0.040	-0.104
5.2	0.139	0.128	-0.330	0.022	0.020	-0.052	0.075	0.069	-0.177
7.4	0.197	0.181	-0.468	0.031	0.028	-0.074	0.105	0.096	-0.249
9.5	0.255	0.234	-0.606	0.040	0.036	-0.094	0.134	0.123	-0.318
11.7	0.313	0.288	-0.745	0.048	0.044	-0.114	0.163	0.149	-0.386
13.9	0.371	0.341	-0.883	0.056	0.052	-0.134	0.190	0.175	-0.452
16.1	0.430	0.395	-1.021	0.064	0.059	-0.153	0.217	0.200	-0.517
18.2	0.488	0.448	-1.159	0.072	0.066	-0.172	0.244	0.224	-0.579
20.4	0.546	0.502	-1.297	0.080	0.073	-0.190	0.269	0.247	-0.640
22.6	0.604	0.555	-1.436	0.087	0.080	-0.207	0.294	0.270	-0.699
24.8	0.662	0.608	-1.574	0.094	0.087	-0.225	0.318	0.293	-0.757
26.9	0.720	0.662	-1.712	0.101	0.093	-0.241	0.342	0.314	-0.812
29.1	0.779	0.715	-1.850	0.108	0.099	-0.257	0.364	0.335	-0.866
31.3	0.837	0.769	-1.989	0.115	0.105	-0.273	0.386	0.355	-0.918
33.5	0.895	0.822	-2.127	0.121	0.111	-0.288	0.408	0.374	-0.969
35.6	0.953	0.876	-2.265	0.127	0.117	-0.303	0.428	0.393	-1.017
37.8	1.011	0.929	-2.403	0.133	0.122	-0.317	0.448	0.411	-1.064
39.2	1.047	0.962	-2.489	0.137	0.126	-0.325	0.460	0.422	-1.092
40.0	1.069	0.982	-2.541	0.139	0.128	-0.330	0.467	0.429	-1.109
41.2	1.101	1.011	-2.616	0.142	0.130	-0.337	0.477	0.438	-1.133
42.2	1.128	1.036	-2.680	0.144	0.133	-0.343	0.485	0.445	-1.152
Note 1: Girder Segment 1 shown – Girder Segment 3 similar									
Note 2: Highlighted rows indicate the centerline of the permanent and temporary bearings.									

Table 8.1.2-4   Concrete Stresses from Components of Applied Loads on Composite Section – Girder Segment 2

Location from CL Permanent Bearing (ft)	Stresses due to Pier Removal (ksi)			Stresses due to CDL (ksi)			Stresses due to LL + IM (ksi)		
	E			F			G		
	Top Deck	Top	Bottom	Top Deck	Top	Bottom	Top Deck	Top	Bottom
43.3	1.143	1.050	-2.717	0.147	0.135	-0.350	0.494	0.454	-1.175
44.3	1.143	1.050	-2.717	0.150	0.138	-0.356	0.502	0.461	-1.194
46.3	1.143	1.050	-2.717	0.154	0.142	-0.367	0.518	0.476	-1.230
48.8	1.143	1.050	-2.717	0.160	0.147	-0.380	0.536	0.492	-1.273
54.2	1.143	1.050	-2.717	0.171	0.157	-0.407	0.572	0.526	-1.360
59.6	1.143	1.050	-2.717	0.181	0.166	-0.430	0.605	0.555	-1.437
65.1	1.143	1.050	-2.717	0.190	0.174	-0.451	0.632	0.581	-1.502
70.5	1.143	1.050	-2.717	0.197	0.181	-0.468	0.656	0.603	-1.559
75.9	1.143	1.050	-2.717	0.203	0.186	-0.482	0.676	0.621	-1.606
81.4	1.143	1.050	-2.717	0.207	0.191	-0.493	0.691	0.634	-1.641
86.8	1.143	1.050	-2.717	0.211	0.194	-0.501	0.701	0.644	-1.666
92.2	1.143	1.050	-2.717	0.213	0.195	-0.505	0.707	0.649	-1.679
97.7	1.143	1.050	-2.717	0.213	0.196	-0.507	0.708	0.650	-1.682
103.1	1.143	1.050	-2.717	0.213	0.195	-0.505	0.707	0.649	-1.679
108.5	1.143	1.050	-2.717	0.211	0.194	-0.501	0.701	0.644	-1.666
114.0	1.143	1.050	-2.717	0.207	0.191	-0.493	0.691	0.634	-1.641
119.4	1.143	1.050	-2.717	0.203	0.186	-0.482	0.676	0.621	-1.606
124.8	1.143	1.050	-2.717	0.197	0.181	-0.468	0.656	0.603	-1.559
130.3	1.143	1.050	-2.717	0.190	0.174	-0.451	0.632	0.581	-1.502
135.7	1.143	1.050	-2.717	0.181	0.166	-0.430	0.605	0.555	-1.437
141.1	1.143	1.050	-2.717	0.171	0.157	-0.407	0.572	0.526	-1.360
146.6	1.143	1.050	-2.717	0.160	0.147	-0.380	0.536	0.492	-1.273
149.0	1.143	1.050	-2.717	0.154	0.142	-0.367	0.518	0.476	-1.230
151.0	1.143	1.050	-2.717	0.150	0.138	-0.356	0.502	0.461	-1.194
152.0	1.143	1.050	-2.717	0.147	0.135	-0.350	0.494	0.454	-1.175

Note: Highlighted rows indicate the centerline of the temporary bearings and midspan

Table 8.1.2-5   **Concrete Stresses from Pretensioning Strands – Girder Segment 1**

Location from CL Permanent Bearing (ft)	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		Final Conditions after Losses (ksi)	
	H		I		J		K	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
0.0	-0.029	0.377	-0.026	0.340	-0.024	0.313	-0.023	0.297
0.8	-0.047	0.615	-0.042	0.554	-0.039	0.511	-0.037	0.485
1.7	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
3.0	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
5.2	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
7.4	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
9.5	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
11.7	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
13.9	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
16.1	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
18.2	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
20.4	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
22.6	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
24.8	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
26.9	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
29.1	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
31.3	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
33.5	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
35.6	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
37.8	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
39.2	-0.065	0.848	-0.059	0.764	-0.054	0.705	-0.051	0.669
40.0	-0.047	0.615	-0.042	0.554	-0.039	0.511	-0.037	0.485
41.2	-0.022	0.283	-0.020	0.255	-0.018	0.235	-0.017	0.223
42.2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings.

Table 8.1.2-6   Concrete Stresses from Pretensioning Strands – Girder Segment 2

Location from CL Permanent Bearing	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		Final Conditions after Losses (ksi)	
	H		I		J		K	
	Top	*Bottom*	*Top*	*Bottom*	*Top*	*Bottom*	*Top*	*Bottom*
43.3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
44.3	-0.037	0.554	-0.026	0.340	-0.030	0.456	-0.023	0.297
46.3	-0.110	1.663	-0.042	0.554	-0.090	1.368	-0.037	0.485
48.8	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
54.2	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
59.6	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
65.1	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
70.5	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
75.9	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
81.4	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
86.8	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
92.2	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
97.7	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
103.1	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
108.5	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
114.0	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
119.4	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
124.8	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
130.3	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
135.7	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
141.1	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
146.6	-0.110	1.663	-0.059	0.764	-0.090	1.368	-0.051	0.669
149.0	-0.110	1.663	-0.042	0.554	-0.090	1.368	-0.037	0.485
151.0	-0.037	0.554	-0.020	0.255	-0.030	0.456	-0.017	0.223
152.0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Note: Highlighted rows indicate the centerline of the temporary bearings and midspan

Table 8.1.2-7   Concrete Stresses from Pretensioning Strands – Girder Segment 3

Location from CL Permanent Bearing (ft)	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		Final Conditions after Losses (ksi)	
	H		I		J		K	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
153.2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
154.2	-0.022	0.283	-0.020	0.255	-0.018	0.234	-0.017	0.222
155.3	-0.047	0.615	-0.042	0.554	-0.039	0.510	-0.037	0.484
156.2	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
157.5	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
159.7	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
161.9	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
164.0	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
166.2	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
168.4	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
170.6	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
172.7	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
174.9	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
177.1	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
179.3	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
181.4	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
183.6	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
185.8	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
188.0	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
190.1	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
192.3	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
193.7	-0.065	0.848	-0.059	0.764	-0.054	0.703	-0.051	0.667
194.5	-0.047	0.615	-0.042	0.554	-0.039	0.510	-0.037	0.484
195.3	-0.029	0.377	-0.026	0.339	-0.024	0.312	-0.023	0.297

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Table 8.1.2-8   Concrete Stresses from Post-tensioning Tendons – Girder Segment 1

Location from CL Permanent Bearing (ft)	Stressing of Post-tensioning Tendons (ksi)			Final Conditions after Losses (ksi)		
	L			M		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
0.0	0.780	1.076	2.691	0.602	0.830	2.077
0.8	0.761	1.059	2.737	0.588	0.817	2.113
1.7	0.743	1.042	2.782	0.573	0.805	2.148
3.0	0.713	1.015	2.855	0.550	0.784	2.205
5.2	0.665	0.971	2.974	0.513	0.750	2.297
7.4	0.616	0.927	3.092	0.476	0.716	2.389
9.5	0.568	0.883	3.211	0.439	0.682	2.481
11.7	0.519	0.838	3.330	0.401	0.648	2.574
13.9	0.471	0.794	3.449	0.364	0.614	2.666
16.1	0.422	0.749	3.568	0.326	0.580	2.759
18.2	0.373	0.705	3.688	0.289	0.545	2.852
20.4	0.324	0.660	3.808	0.251	0.511	2.946
22.6	0.276	0.616	3.928	0.213	0.476	3.039
24.8	0.227	0.571	4.048	0.175	0.442	3.133
26.9	0.179	0.527	4.165	0.138	0.408	3.225
29.1	0.135	0.488	4.272	0.105	0.378	3.308
31.3	0.094	0.450	4.373	0.073	0.349	3.387
33.5	0.053	0.413	4.475	0.041	0.320	3.467
35.6	0.012	0.375	4.577	0.009	0.291	3.546
37.8	-0.030	0.338	4.679	-0.023	0.262	3.626
39.2	-0.053	0.316	4.737	-0.041	0.245	3.672
40.0	-0.067	0.304	4.771	-0.052	0.235	3.699
41.2	-0.085	0.287	4.815	-0.066	0.223	3.734
42.2	-0.100	0.273	4.853	-0.078	0.212	3.764

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings.

Table 8.1.2-9   Concrete Stresses from Post-tensioning Tendons – Girder Segment 2

Location from CL Permanent Bearing (ft)	Stressing of Post-tensioning Tendons (ksi)			Final Conditions after Losses (ksi)		
	L			M		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
43.3	-0.118	0.257	4.898	-0.091	0.199	3.798
44.3	-0.133	0.243	4.936	-0.103	0.189	3.828
46.3	-0.164	0.215	5.012	-0.127	0.167	3.888
48.8	-0.201	0.181	5.105	-0.156	0.141	3.962
54.2	-0.271	0.118	5.281	-0.211	0.091	4.100
59.6	-0.334	0.060	5.440	-0.260	0.047	4.226
65.1	-0.387	0.013	5.575	-0.301	0.010	4.333
70.5	-0.430	-0.026	5.686	-0.334	-0.020	4.422
75.9	-0.465	-0.058	5.779	-0.362	-0.045	4.497
81.4	-0.487	-0.077	5.840	-0.379	-0.060	4.547
86.8	-0.505	-0.093	5.891	-0.393	-0.072	4.590
92.2	-0.506	-0.093	5.903	-0.394	-0.072	4.601
97.7	-0.507	-0.093	5.915	-0.395	-0.072	4.613
103.1	-0.508	-0.093	5.926	-0.396	-0.073	4.624
108.5	-0.509	-0.093	5.938	-0.397	-0.073	4.636
114.0	-0.493	-0.078	5.905	-0.385	-0.061	4.612
119.4	-0.470	-0.058	5.843	-0.367	-0.045	4.561
124.8	-0.435	-0.026	5.749	-0.339	-0.020	4.485
130.3	-0.391	0.013	5.637	-0.305	0.010	4.395
135.7	-0.338	0.061	5.501	-0.264	0.048	4.287
141.1	-0.274	0.119	5.340	-0.214	0.093	4.159
146.6	-0.204	0.183	5.163	-0.158	0.143	4.019
149.0	-0.166	0.218	5.069	-0.129	0.169	3.945
151.0	-0.135	0.246	4.992	-0.105	0.191	3.884
152.0	-0.119	0.260	4.954	-0.093	0.202	3.854

Note: Highlighted rows indicate the centerline of the temporary bearings and midspan

Table 8.1.2-10   Concrete Stresses from Post-tensioning Tendons – Girder Segment 3

Location from CL Permanent Bearing (ft)	Stressing of Post-tensioning Tendons (ksi)			Final Conditions after Losses (ksi)		
	L			M		
	Top Deck	Top	Bottom	Top Deck	Top	Bottom
153.2	-0.101	0.276	4.909	-0.079	0.215	3.819
154.2	-0.086	0.291	4.870	-0.067	0.226	3.788
155.3	-0.068	0.307	4.825	-0.053	0.239	3.753
156.2	-0.054	0.320	4.791	-0.042	0.249	3.726
157.5	-0.030	0.341	4.732	-0.023	0.265	3.680
159.7	0.012	0.380	4.629	0.009	0.295	3.599
161.9	0.054	0.418	4.526	0.042	0.325	3.518
164.0	0.095	0.456	4.423	0.074	0.354	3.437
166.2	0.137	0.494	4.321	0.106	0.383	3.357
168.4	0.181	0.533	4.214	0.140	0.414	3.273
170.6	0.229	0.578	4.095	0.178	0.449	3.180
172.7	0.279	0.623	3.973	0.216	0.484	3.085
174.9	0.328	0.668	3.852	0.255	0.519	2.990
177.1	0.378	0.713	3.731	0.293	0.553	2.896
179.3	0.427	0.758	3.610	0.331	0.588	2.801
181.4	0.476	0.803	3.490	0.369	0.623	2.707
183.6	0.525	0.848	3.369	0.407	0.658	2.613
185.8	0.575	0.893	3.249	0.445	0.692	2.519
188.0	0.624	0.938	3.129	0.483	0.727	2.425
190.1	0.672	0.982	3.009	0.521	0.761	2.332
192.3	0.721	1.027	2.889	0.559	0.796	2.239
193.7	0.752	1.055	2.815	0.582	0.817	2.181
194.5	0.770	1.072	2.770	0.597	0.830	2.146
195.3	0.789	1.089	2.723	0.611	0.843	2.110

Note: Highlighted rows indicate the centerline of the temporary bearings and midspan

8.1.3   **Service Limit State Stresses for Load Combinations**

In this section, total concrete stresses at each of the four critical construction stages are given along the girder segments. These stresses are the sum of the concrete stresses caused by components of applied load and prestress presented in the previous section. As in the previous section, concrete stresses are given for the top of the deck slab and for the top and bottom of the girder.

The stresses are presented in tables for each girder segment because prestress losses for the post-tensioning tendons are slightly different between Girder Segments 1 and 3. The first set of tables contains total concrete stresses for the first three construction stages. The second set of tables contains total concrete stresses for the final construction stage for the two load combinations that include the full service loads (Tension and Compression 1, see **Table 8.1.1.2.5-1**). This set of tables is used to evaluate both compression and tension limits with full service loads. The final set of tables contains total concrete stresses for the final construction stage, but is only used to evaluate compression limits for the partial service load cases (Compression 2 and 3, see **Table 8.1.1.2.5-1**).

To indicate which concrete stresses are combined to obtain the totals reported in this section, a summation of letters is shown at the top of each group of columns representing a construction stage. These letters identify the components of applied load or prestress that are combined for the columns shown. The letters appear in the tables of the previous section.

Stresses computed in these tables are presented graphically in **Figures 8.1.3-1** through **8.1.3-3** to show the component stresses for at the top of deck and top and bottom of girder. The applicable stress limits summarized in **Table 8.1.1.2.5-1** are also shown on the figures to indicate that the stress limits are satisfied.

Table 8.1.3-1   Total Concrete Stresses at Construction Stages 1-3 – Girder Segment 1

Location from CL Permanent Bearing (ft)	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		
	A + H		A + B + C + D + I		A + B + C + E + J + L		
	Top	*Bottom*	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
0.0	-0.029	0.377	-0.026	0.340	0.780	1.052	3.004
0.8	-0.038	0.606	-0.024	0.535	0.784	1.058	3.176
1.7	-0.048	0.831	-0.022	0.727	0.787	1.063	3.346
3.0	-0.035	0.818	0.006	0.699	0.794	1.094	3.307
5.2	-0.016	0.799	0.045	0.658	0.803	1.141	3.250
7.4	0.000	0.782	0.080	0.623	0.813	1.182	3.197
9.5	0.013	0.768	0.110	0.593	0.823	1.219	3.150
11.7	0.025	0.757	0.134	0.568	0.833	1.250	3.108
13.9	0.034	0.748	0.153	0.549	0.842	1.277	3.071
16.1	0.040	0.741	0.166	0.535	0.852	1.298	3.040
18.2	0.044	0.737	0.174	0.527	0.861	1.315	3.013
20.4	0.045	0.736	0.177	0.524	0.870	1.327	2.992
22.6	0.044	0.737	0.175	0.526	0.880	1.334	2.976
24.8	0.041	0.741	0.168	0.533	0.889	1.335	2.965
26.9	0.035	0.747	0.155	0.546	0.899	1.333	2.956
29.1	0.026	0.755	0.137	0.565	0.914	1.330	2.941
31.3	0.015	0.766	0.114	0.589	0.931	1.325	2.926
33.5	0.002	0.780	0.085	0.618	0.948	1.314	2.917
35.6	-0.014	0.796	0.051	0.652	0.965	1.299	2.912
37.8	-0.032	0.815	0.012	0.692	0.982	1.278	2.913
39.2	-0.045	0.828	-0.015	0.720	0.994	1.265	2.911
40.0	-0.035	0.603	-0.016	0.527	1.003	1.271	2.715
41.2	-0.022	0.283	-0.020	0.255	1.016	1.281	2.434
42.2	0.000	0.000	0.000	0.000	1.028	1.309	2.174

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Table 8.1.3-2   **Total Concrete Stresses at Construction Stages 1-3 – Girder Segment 2**

Location from CL Permanent Bearing (ft)	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		
	A + H		A + B + C + D + I		A + B + C + E + J + L		
	Top	*Bottom*	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
43.3	0.000	0.000	0.000	0.000	1.025	1.307	2.181
44.3	-0.037	0.554	-0.033	0.504	1.010	1.263	2.675
46.3	-0.055	1.608	0.017	1.393	0.979	1.283	3.554
48.8	0.008	1.543	0.153	1.254	0.942	1.375	3.518
54.2	0.139	1.410	0.432	0.969	0.872	1.571	3.429
59.6	0.254	1.292	0.679	0.718	0.809	1.742	3.355
65.1	0.353	1.191	0.893	0.500	0.756	1.893	3.288
70.5	0.438	1.104	1.074	0.315	0.713	2.022	3.228
75.9	0.507	1.034	1.222	0.164	0.678	2.127	3.181
81.4	0.561	0.979	1.337	0.047	0.656	2.215	3.133
86.8	0.599	0.940	1.419	-0.037	0.638	2.275	3.107
92.2	0.622	0.917	1.469	-0.088	0.637	2.321	3.072
97.7	0.630	0.909	1.485	-0.104	0.636	2.336	3.068
103.1	0.622	0.917	1.469	-0.088	0.635	2.321	3.095
108.5	0.599	0.940	1.419	-0.037	0.634	2.275	3.153
114.0	0.561	0.979	1.337	0.047	0.651	2.214	3.198
119.4	0.507	1.034	1.222	0.164	0.673	2.127	3.245
124.8	0.438	1.104	1.074	0.315	0.708	2.022	3.291
130.3	0.353	1.191	0.893	0.500	0.752	1.893	3.350
135.7	0.254	1.292	0.679	0.718	0.805	1.743	3.416
141.1	0.139	1.410	0.432	0.969	0.869	1.572	3.488
146.6	0.008	1.543	0.153	1.254	0.940	1.377	3.576
149.0	-0.055	1.608	0.017	1.393	0.977	1.286	3.610
151.0	-0.037	0.554	-0.033	0.504	1.008	1.266	2.731
152.0	0.000	0.000	0.000	0.000	1.024	1.310	2.237
Note: Highlighted rows indicate the centerline of the temporary bearings and midspan							

Table 8.1.3-3   **Total Concrete Stresses at Construction Stages 1-3 – Girder Segment 3**

Location from CL Permanent Bearing (ft)	Release of Pretensioned Strands (ksi)		Placement of Deck & Splice Concrete (ksi)		Stressing of Post-tensioning Tendons (ksi)		
	A + H		A + B + C + D + I		A + B + C + E + J + L		
	Top	*Bottom*	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
153.2	0.000	0.000	0.000	0.000	1.026	1.312	2.229
154.2	-0.022	0.283	-0.020	0.255	1.015	1.284	2.488
155.3	-0.035	0.603	-0.016	0.527	1.002	1.275	2.769
156.2	-0.045	0.828	-0.015	0.719	0.994	1.268	2.964
157.5	-0.032	0.815	0.012	0.692	0.981	1.282	2.965
159.7	-0.014	0.796	0.051	0.652	0.965	1.303	2.963
161.9	0.002	0.780	0.085	0.617	0.949	1.319	2.967
164.0	0.015	0.766	0.114	0.588	0.932	1.330	2.975
166.2	0.026	0.755	0.137	0.564	0.916	1.336	2.989
168.4	0.035	0.747	0.155	0.546	0.901	1.339	3.003
170.6	0.041	0.741	0.168	0.533	0.891	1.342	3.010
172.7	0.044	0.737	0.175	0.525	0.883	1.341	3.020
174.9	0.045	0.736	0.178	0.523	0.874	1.335	3.035
177.1	0.044	0.737	0.174	0.526	0.865	1.323	3.055
179.3	0.040	0.741	0.166	0.535	0.857	1.307	3.080
181.4	0.034	0.748	0.153	0.548	0.848	1.286	3.110
183.6	0.025	0.757	0.134	0.568	0.839	1.260	3.146
185.8	0.013	0.768	0.110	0.592	0.830	1.229	3.187
188.0	0.000	0.782	0.080	0.622	0.821	1.193	3.233
190.1	-0.016	0.799	0.046	0.658	0.811	1.152	3.284
192.3	-0.035	0.818	0.006	0.698	0.802	1.107	3.340
193.7	-0.048	0.831	-0.022	0.726	0.796	1.076	3.377
194.5	-0.038	0.606	-0.024	0.534	0.793	1.071	3.208
195.3	-0.029	0.377	-0.026	0.339	0.789	1.065	3.036

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Table 8.1.3-4   Total Concrete Stresses at Construction Stage 4 – Compression 1 and Tension Limits – Girder Segment 1

Location from CL Permanent Bearing (ft)	Compression 1 (ksi)			Tension (ksi)		
	A + B + C + E + F + G + K + M			0.80*G + A + B + C + E + F + K + M		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
0.0	0.602	0.808	2.375	0.602	0.808	2.375
0.8	0.626	0.833	2.489	0.624	0.831	2.494
1.7	0.650	0.857	2.601	0.645	0.853	2.613
3.0	0.688	0.918	2.486	0.679	0.910	2.507
5.2	0.749	1.011	2.308	0.734	0.998	2.343
7.4	0.809	1.099	2.136	0.788	1.080	2.186
9.5	0.868	1.180	1.972	0.841	1.156	2.036
11.7	0.925	1.256	1.816	0.893	1.226	1.893
13.9	0.982	1.326	1.667	0.944	1.291	1.757
16.1	1.038	1.390	1.525	0.994	1.350	1.629
18.2	1.093	1.448	1.391	1.044	1.404	1.507
20.4	1.146	1.501	1.264	1.092	1.451	1.392
22.6	1.199	1.547	1.145	1.140	1.493	1.285
24.8	1.251	1.588	1.033	1.187	1.530	1.184
26.9	1.302	1.624	0.927	1.234	1.561	1.089
29.1	1.356	1.657	0.819	1.283	1.590	0.992
31.3	1.411	1.686	0.714	1.334	1.615	0.898
33.5	1.465	1.710	0.617	1.383	1.635	0.811
35.6	1.518	1.727	0.527	1.432	1.648	0.731
37.8	1.569	1.738	0.445	1.480	1.656	0.658
39.2	1.603	1.744	0.394	1.511	1.660	0.612
40.0	1.623	1.761	0.178	1.530	1.676	0.400
41.2	1.654	1.785	-0.129	1.558	1.698	0.097
42.2	1.679	1.826	-0.412	1.582	1.737	-0.181

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Table 8.1.3-5 Total Concrete Stresses at Construction Stage 4 – Compression 1 and Tension Limits – Girder Segment 2

Location from CL Permanent Bearing (ft)	Compression 1 (ksi)			Tension (ksi)		
	A + B + C + E + F + G + K + M			0.80*G + A + B + C + E + F + K + M		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
43.3	1.693	1.839	-0.443	1.595	1.748	-0.208
44.3	1.692	1.810	-0.015	1.591	1.718	0.223
46.3	1.688	1.859	0.732	1.585	1.764	0.978
48.8	1.683	1.980	0.621	1.575	1.882	0.875
54.2	1.676	2.234	0.381	1.562	2.129	0.653
59.6	1.669	2.457	0.174	1.548	2.346	0.461
65.1	1.664	2.651	-0.007	1.537	2.535	0.293
70.5	1.662	2.818	-0.164	1.531	2.697	0.148
75.9	1.660	2.954	-0.290	1.524	2.830	0.032
81.4	1.662	3.063	-0.394	1.524	2.936	-0.066
86.8	1.661	3.140	-0.462	1.521	3.011	-0.129
92.2	1.668	3.193	-0.515	1.527	3.063	-0.180
97.7	1.669	3.209	-0.524	1.527	3.079	-0.187
103.1	1.666	3.192	-0.492	1.525	3.062	-0.156
108.5	1.657	3.139	-0.416	1.517	3.010	-0.083
114.0	1.656	3.062	-0.330	1.518	2.936	-0.002
119.4	1.654	2.953	-0.226	1.519	2.829	0.096
124.8	1.657	2.818	-0.101	1.526	2.697	0.211
130.3	1.660	2.652	0.055	1.533	2.535	0.355
135.7	1.665	2.458	0.235	1.544	2.347	0.522
141.1	1.673	2.236	0.440	1.559	2.130	0.712
146.6	1.680	1.982	0.678	1.573	1.884	0.933
149.0	1.686	1.862	0.788	1.583	1.766	1.034
151.0	1.690	1.813	0.041	1.590	1.720	0.279
152.0	1.692	1.842	-0.388	1.593	1.751	-0.153
Note: Highlighted rows indicate the centerline of midspan and temporary bearings						

Table 8.1.3-6 **Total Concrete Stresses at Construction Stage 4 – Compression 1 Tension Limits – Girder Segment 3**

Location from CL Permanent Bearing (ft)	Compression 1 (ksi)			Tension (ksi)		
	A + B + C + E + F + G + K + M			0.80*G + A + B + C + E + F + K + M		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
153.2	1.678	1.829	-0.357	1.581	1.740	-0.126
154.2	1.653	1.789	-0.075	1.557	1.701	0.151
155.3	1.623	1.765	0.231	1.529	1.679	0.453
156.2	1.602	1.748	0.446	1.510	1.664	0.664
157.5	1.569	1.742	0.496	1.479	1.660	0.709
159.7	1.518	1.731	0.578	1.432	1.653	0.781
161.9	1.465	1.714	0.666	1.384	1.640	0.860
164.0	1.412	1.692	0.762	1.335	1.621	0.946
166.2	1.358	1.663	0.866	1.285	1.596	1.039
168.4	1.304	1.630	0.973	1.236	1.567	1.135
170.6	1.253	1.595	1.078	1.189	1.536	1.229
172.7	1.202	1.555	1.189	1.143	1.501	1.329
174.9	1.150	1.509	1.307	1.096	1.459	1.435
177.1	1.097	1.457	1.432	1.048	1.412	1.548
179.3	1.043	1.399	1.565	0.999	1.359	1.668
181.4	0.988	1.336	1.705	0.950	1.301	1.796
183.6	0.931	1.266	1.853	0.899	1.236	1.930
185.8	0.874	1.191	2.008	0.847	1.166	2.072
188.0	0.816	1.110	2.171	0.795	1.091	2.221
190.1	0.757	1.023	2.341	0.742	1.009	2.377
192.3	0.696	0.930	2.518	0.688	0.922	2.539
193.7	0.659	0.870	2.632	0.654	0.865	2.644
194.5	0.635	0.846	2.520	0.633	0.844	2.526
195.3	0.611	0.821	2.406	0.611	0.821	2.406

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

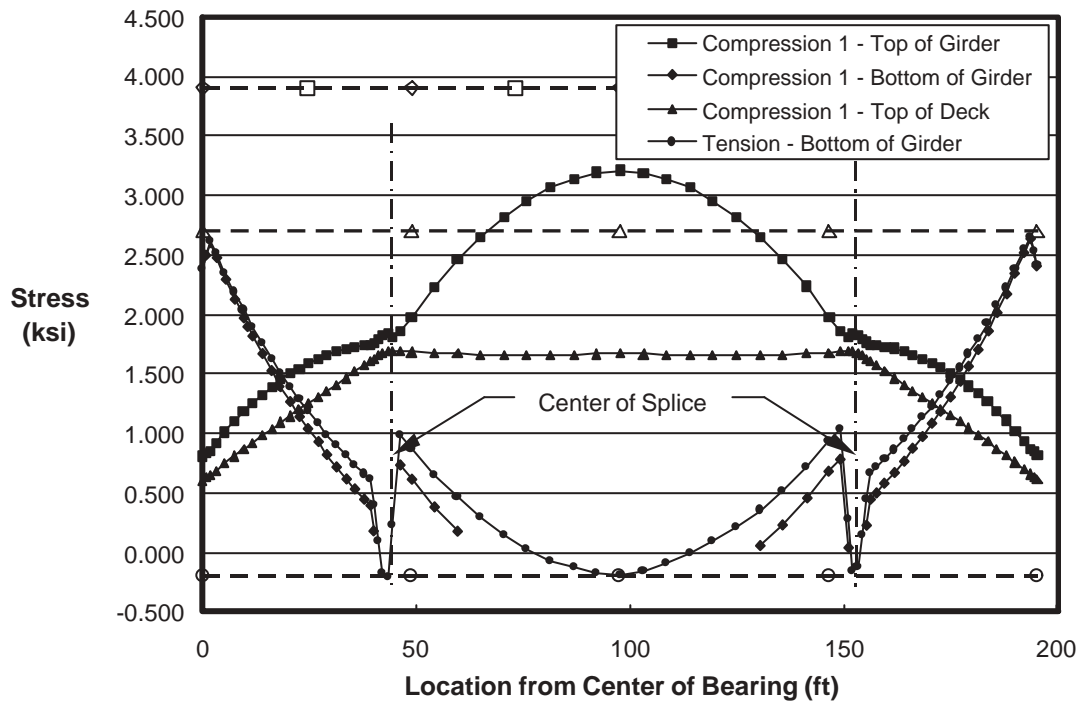**Single Span Spliced PCI BT-96 Girder**   **Design Example 1**

Figure 8.1.3-4   Total Concrete Stresses at Construction Stage 4 – Tension and Compression1

Table 8.1.3-7   **Total Concrete Stresses at Construction Stage 4 – Compression 2 and 3 Limits – Girder Segment 1**

Location from CL Permanent Bearing (ft)	Compression 2 (ksi)			Compression 3 (ksi)		
	A + B + C + E + F + K + M			G + ½(A + B + C + E + F + K + M)		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
0.0	0.602	0.808	2.375	0.301	0.404	1.187
0.8	0.614	0.822	2.518	0.319	0.422	1.230
1.7	0.625	0.835	2.659	0.337	0.440	1.272
3.0	0.644	0.878	2.591	0.366	0.479	1.191
5.2	0.674	0.943	2.485	0.412	0.540	1.065
7.4	0.704	1.003	2.385	0.457	0.598	0.944
9.5	0.734	1.057	2.291	0.501	0.652	0.827
11.7	0.763	1.107	2.202	0.544	0.703	0.715
13.9	0.792	1.151	2.119	0.586	0.750	0.607
16.1	0.820	1.190	2.042	0.628	0.795	0.504
18.2	0.849	1.224	1.970	0.668	0.836	0.406
20.4	0.877	1.253	1.905	0.708	0.874	0.312
22.6	0.905	1.277	1.844	0.747	0.909	0.223
24.8	0.932	1.296	1.790	0.784	0.940	0.138
26.9	0.960	1.310	1.739	0.822	0.969	0.057
29.1	0.992	1.322	1.685	0.860	0.996	-0.024
31.3	1.025	1.331	1.632	0.899	1.021	-0.102
33.5	1.057	1.335	1.585	0.936	1.042	-0.176
35.6	1.090	1.334	1.544	0.973	1.060	-0.245
37.8	1.122	1.327	1.509	1.009	1.075	-0.310
39.2	1.143	1.322	1.486	1.031	1.083	-0.349
40.0	1.157	1.333	1.287	1.045	1.095	-0.465
41.2	1.177	1.347	1.003	1.065	1.112	-0.631
42.2	1.194	1.381	0.741	1.082	1.136	-0.782

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Table 8.1.3-8   **Total Concrete Stresses at Construction Stage 4 – Compression 2 and 3 Limits – Girder Segment 2**

Location from CL Permanent Bearing (ft)	Compression 2 (ksi)			Compression 3 (ksi)		
	A + B + C + E + F + K + M			G + ½(A + B + C + E + F + K + M)		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
43.3	1.199	1.385	0.732	1.094	1.147	-0.809
44.3	1.190	1.348	1.178	1.097	1.136	-0.605
46.3	1.170	1.384	1.962	1.103	1.167	-0.249
48.8	1.147	1.488	1.894	1.109	1.236	-0.326
54.2	1.104	1.708	1.741	1.124	1.380	-0.490
59.6	1.064	1.902	1.611	1.137	1.506	-0.631
65.1	1.032	2.071	1.495	1.148	1.616	-0.755
70.5	1.006	2.215	1.395	1.159	1.710	-0.862
75.9	0.984	2.333	1.316	1.168	1.787	-0.948
81.4	0.971	2.429	1.247	1.176	1.849	-1.018
86.8	0.961	2.496	1.204	1.181	1.892	-1.064
92.2	0.962	2.543	1.164	1.187	1.921	-1.097
97.7	0.961	2.559	1.158	1.188	1.930	-1.103
103.1	0.960	2.543	1.187	1.186	1.921	-1.086
108.5	0.957	2.495	1.250	1.179	1.892	-1.041
114.0	0.966	2.428	1.311	1.174	1.848	-0.986
119.4	0.979	2.332	1.380	1.165	1.787	-0.916
124.8	1.001	2.215	1.459	1.157	1.710	-0.830
130.3	1.028	2.071	1.557	1.146	1.616	-0.724
135.7	1.061	1.902	1.671	1.135	1.507	-0.601
141.1	1.101	1.710	1.800	1.123	1.381	-0.460
146.6	1.145	1.490	1.951	1.108	1.237	-0.297
149.0	1.169	1.386	2.019	1.102	1.169	-0.221
151.0	1.188	1.351	1.234	1.096	1.137	-0.577
152.0	1.198	1.388	0.787	1.093	1.148	-0.781
Note: Highlighted rows indicate the centerline of midspan and temporary bearings						

Table 8.1.3-9   Total Concrete Stresses at Construction Stage 4 – Compression 2 and 3 Limits – Girder Segment 3

Location from CL Permanent Bearing (ft)	Compression 2 (ksi)			Compression 3 (ksi)		
	A + B + C + E + F + K + M			G + ½(A + B + C + E + F + K + M)		
	Top Deck	*Top*	*Bottom*	*Top Deck*	*Top*	*Bottom*
153.2	1.193	1.384	0.796	1.082	1.137	-0.755
154.2	1.176	1.351	1.057	1.065	1.113	-0.604
155.3	1.156	1.336	1.340	1.045	1.097	-0.439
156.2	1.142	1.326	1.538	1.031	1.085	-0.323
157.5	1.121	1.331	1.560	1.008	1.077	-0.284
159.7	1.090	1.338	1.595	0.973	1.062	-0.220
161.9	1.058	1.340	1.635	0.936	1.044	-0.151
164.0	1.026	1.337	1.681	0.899	1.023	-0.078
166.2	0.993	1.328	1.732	0.861	0.999	0.000
168.4	0.962	1.316	1.785	0.823	0.972	0.080
170.6	0.935	1.302	1.835	0.786	0.944	0.161
172.7	0.908	1.284	1.888	0.748	0.913	0.245
174.9	0.881	1.261	1.947	0.710	0.878	0.333
177.1	0.853	1.233	2.012	0.670	0.840	0.426
179.3	0.825	1.199	2.082	0.630	0.799	0.524
181.4	0.797	1.161	2.158	0.589	0.755	0.627
183.6	0.769	1.117	2.240	0.547	0.708	0.733
185.8	0.740	1.068	2.327	0.504	0.657	0.845
188.0	0.711	1.014	2.420	0.460	0.603	0.961
190.1	0.682	0.954	2.518	0.416	0.546	1.082
192.3	0.653	0.890	2.623	0.370	0.485	1.207
193.7	0.634	0.847	2.690	0.341	0.446	1.287
194.5	0.623	0.835	2.549	0.324	0.429	1.245
195.3	0.611	0.821	2.406	0.306	0.410	1.203

Note: Highlighted rows indicate the centerline of the permanent and temporary bearings

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

Figure 8.1.3-5   Total Concrete Stresses at Construction Stage 4 – Compression 2 and 3 Limits

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
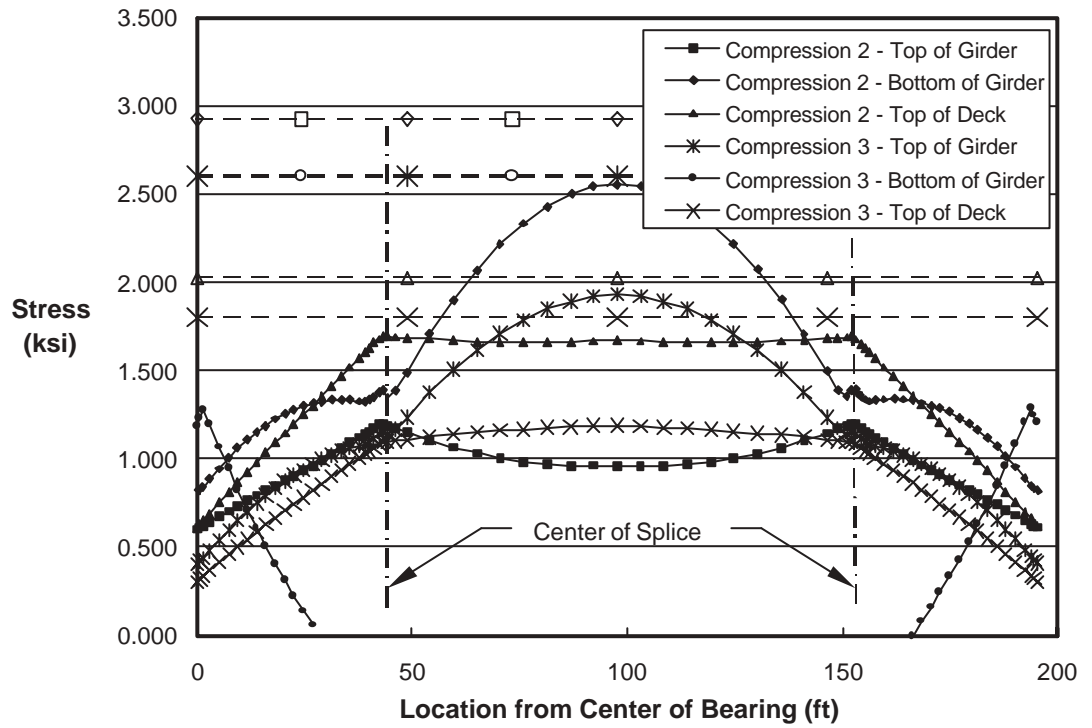
8.2   **STRENGTH LIMIT STATE**

After design is completed at the service limit state, the girder is checked at critical sections for flexural capacity. For this design example, the capacity at midspan will be considered. The contribution of the build-up to section depth is neglected in all calculations.

The strength limit state is only checked at the final condition. It is assumed that strength is adequate for intermediate loading conditions.

8.2.1   **Initial Evaluation of Flexural Capacity**

An initial computation was performed using the equations and procedures of LRFD Art. 5.7.3. Because of the large number of post-tensioning tendons, it was found that the depth of compression, c, extended well into the web of the girder. In this case, the equations in the *Specifications* do not work well because they cannot properly consider the different concrete strengths in the girder and deck, and they cannot account for the girder flange and girder web.

Therefore, it was decided to utilize a simplified strain-compatibility analysis to evaluate the flexural capacity of the cross section. The following sections present the approach and results of the analysis. Details of the analysis can be found in text books on the design of prestressed concrete members and in the *PCI Bridge Design Manual.*

8.2.2   **Stress in Prestressing Steel**

All prestressing is considered together. Differences in effective stress in the pretensioned strands and post-tensioning tendons are insignificant at the strength limit state. Strands in the top flange of the girder are neglected since the concrete surrounding them will be in the compression block when the capacity is determined. Considering the pretensioned strands and post-tensioning tendons, the effective depth to the centroid of the total prestressed reinforcement in the bottom flange, d_p, is estimated to be

$$d_p = [d_{pPT}(A_{PT}) + d_{pPreT}(A_{PreT})] / (A_{PT} + A_{PreT})$$
$$= [87.51(15.624) + 101.36(3.906)] / (15.624 + 3.906)$$
$$= 90.28 \text{ in.}$$

There is a locked in difference in strain between the concrete and the prestressed reinforcement, ε_{po}, also known as the "decompression strain". This quantity can be estimated as:

$$\varepsilon_{po} = f_{po}/E_p$$
$$= 180/28,500$$
$$= 0.00632 \text{ in./in.}$$

where

$$f_{po} = 180 \text{ ksi}$$
$$E_p = 28,500 \text{ ksi}$$

This strain must be added to the strain computed using the concrete strain, ε_{cu}, and the assumed curvature. The value of f_{po} was taken as a stress of 180 ksi (lower than the jacking stress because of post-tensioning losses), which provides a strain of 0.00632 in./in. when divided by E_{ps}. This strain must be added to the strain computed using the concrete strain, ε_{cu}, and the assumed curvature.

To apply the strain compatibility analysis, the following equation, known as the modified Ramberg-Osgood function, is used to compute the stress in the prestressing steel for a given strain:

$$f_{ps} = E_p \varepsilon_{pf} \left\{ A + \frac{1-A}{\left[1 + \left(B \varepsilon_{pf} \right)^C \right]^{1/C}} \right\} \le f_{pu}$$

where:

E_p = 28,500 ksi

ε_{pf} = value of strain for which the stress is to be computed

A = 0.0334

B = 116.4

C = 12

The values A, B and C are parameters that can be varied to define the shape of the stress-strain curve. The values here were selected to obtain a stress of approximately 243 ksi at 1% strain, and a stress of 270 ksi at 3.5% strain. Other expressions are also available for estimating the stress in the prestressing steel.

8.2.3   **Concrete Stress Block**

The equivalent rectangular stress block is used for the concrete, with the concrete strength for the deck and girder used as appropriate. The stress block is applied to the entire area of the flange, and then the depth of the girder determined using the strain compatibility analysis. The depth of the compression block is computed by dividing the depth of compression, c, by the β_1 value for the girder concrete.

The shape of the top of the girder is simplified by representing the top flange as a rectangle that is 44 in. wide by 4.5 in. tall. Below this flange, the section is taken to be the width of the web, 8 in.

The ultimate compressive strain in the concrete at the top of the deck is taken to be

ε_{cu} = 0.003 in./in.   LRFD Art. 5.7.2.1

8.2.4   **Flexural Capacity**

Strain compatibility analysis is an iterative approach that can be used to determine the flexural capacity of a cross section at the strength limit state. The following steps are followed:

- A depth of compression, c, is assumed

- The strain in the prestressing steel is computed from the curvature, $\phi = \varepsilon_{cu}/c$, plus the decompression strain

- The stress in the prestressing steel is computed using the modified Ramberg-Osgood equation

- The force is computed in the prestressing steel

- The total depth of the concrete stress block, a, is computed as c/β_1, where $\beta_1 = 0.725$ for the girder concrete.

- The total compression force in the deck, top flange and web is computed using $0.85f'_c$ with the appropriate concrete strength

- The compression force is compared to the tension force. The depth of compression, c, is adjusted and the process is repeated until the compression force = the tension force.

- After equilibrium is achieved, the flexural capacity, M_n, is computed

For this section, the following values were obtained:

c = 28.66 in.

a = c/β_1 = 28.66 / 0.725 = 20.78 in.

a_{web} = $a - h_f$ = 20.87 − 8.25 = 12.53 in.

f_{ps} = 248.7 ksi

T = 4,857 kips

C = 4,857 kips

M_n = 33,916 k-ft

Comparing the factored moment to the factored capacity (with ϕ = 1.0, from LRFD Art. 5.5.4.2.1):

M_u = 24,969 k-ft < ϕM_n = 1.0(33,916) = 33,916 k-ft   O.K.

The value for M_u is well below the value of ϕM_n, so the design is acceptable.

8.2.5   **Reinforcement Limits**

The maximum reinforcement limit (LRFD Art. 5.7.3.3.1) is checked using the following:

c/d = 28.66 / 90.28 = 0.317 < 0.42   O.K.

The minimum reinforcement limit is not checked, but is certainly satisfied by the quantity of reinforcement provided at this section.

8.3   **FATIGUE LIMIT STATE**

Since the member is designed to satisfy the limiting tensile stress at the Service III limit state, fatigue in the reinforcement does not need to be considered, as indicated in LRFD Art. 5.5.3.1.

9   DESIGN FOR SHEAR

9.1   **VERTICAL SHEAR**

When performing the shear design of girders with post-tensioning, the effect of the ducts must be considered when the ducts are located within the minimum web dimension of the member. LRFD Article 5.8.2.9 requires that one-quarter of the diameter of a grouted duct be deducted from the web width used for shear, b_v. Since the ducts are located within the web for the full length of the girder, the web width must be reduced for shear calculations.

The service load and factored shear values are presented in **Table 9.1-1**. **Figures 9.1-1** and **2** present this data graphically. The slight offset at splice locations in the figure is caused by the concentrated load from the diaphragm at the splice.

Table 9.1-1   Shear from Components of Applied Loads and Factored Shear

Location (ft)	Girder DL (kips)	Deck DL (kips)	NCDL (kips)	Pier Removal (kips)	CDL (kips)	Live Load + IM (kips)	Total Shear (kips)	Factored Shear, V_u (kips)
Load Factors	1.25	1.25	1.25	1.25	1.5	1.75	-----	-----
0.0	24.22	20.16	3.73	187.18	30.58	191.09	456.97	674.40
0.8	23.26	19.36	3.58	187.18	30.32	189.78	453.47	669.31
1.7	22.31	18.57	3.44	187.18	30.06	188.49	450.04	664.32
3.0	20.76	17.28	3.20	187.18	29.63	186.40	444.45	656.17
5.2	18.26	15.20	2.81	187.18	28.95	183.05	435.45	643.08
7.4	15.76	13.12	2.43	187.18	28.27	179.72	426.48	630.03
9.5	13.26	11.04	2.04	187.18	27.59	176.43	417.54	617.04
11.7	10.76	8.96	1.66	187.18	26.91	173.16	408.63	604.09
13.9	8.26	6.88	1.27	187.18	26.23	169.92	399.74	591.19
16.1	5.76	4.80	0.89	187.18	25.55	166.70	390.88	578.34
18.2	3.26	2.72	0.50	187.18	24.87	163.52	382.05	565.53
20.4	0.77	0.64	0.12	187.18	24.19	160.36	373.24	552.78
22.6	-1.73	-1.44	-0.27	187.18	23.51	157.22	364.47	540.07
24.8	-4.23	-3.52	-0.65	187.18	22.82	154.12	355.72	527.41
26.9	-6.73	-5.60	-1.04	187.18	22.14	151.04	346.99	514.80
29.1	-9.23	-7.68	-1.42	187.18	21.46	147.99	338.30	502.24
31.3	-11.73	-9.76	-1.81	187.18	20.78	144.97	329.63	489.72
33.5	-14.23	-11.84	-2.19	187.18	20.10	141.97	320.99	477.25
35.6	-16.73	-13.92	-2.58	187.18	19.42	139.01	312.38	464.83
37.8	-19.23	-16.00	-2.96	187.18	18.74	136.06	303.79	452.46
39.2	-20.78	-17.29	-3.20	187.18	18.32	134.25	298.48	444.80
40.0	-21.72	-18.08	-3.35	187.18	18.06	133.15	295.24	440.14
41.2	-23.07	-19.20	-3.55	187.18	17.69	131.59	290.62	433.50
42.2	-24.22	-20.16	-3.73	187.18	17.38	130.27	286.71	427.86
42.2	-24.22	-20.16	-3.73	187.18	17.38	130.27	334.82	488.01
Splice								
43.3	62.43	51.96	9.62	0.00	17.01	128.73	145.74	250.79
43.3	62.43	51.96	9.62	0.00	17.01	128.73	269.74	405.79
44.3	61.28	51.00	9.44	0.00	16.70	127.42	265.83	400.18
46.3	58.98	49.09	9.09	0.00	16.07	124.81	258.04	388.98
48.8	56.18	46.76	8.66	0.00	15.31	121.68	248.59	375.40
54.2	49.94	41.56	7.69	0.00	13.61	114.80	227.61	345.31
59.6	43.70	36.37	6.73	0.00	11.91	108.09	206.79	315.51
65.1	37.46	31.17	5.77	0.00	10.21	101.55	186.15	286.02
70.5	31.21	25.98	4.81	0.00	8.51	95.18	165.68	256.82
75.9	24.97	20.78	3.85	0.00	6.80	88.98	145.39	227.92
81.4	18.73	15.59	2.89	0.00	5.10	82.95	125.26	199.33
86.8	12.49	10.39	1.92	0.00	3.40	77.10	105.30	171.03
92.2	6.24	5.20	0.96	0.00	1.70	71.41	85.52	143.03
97.7	0.00	0.00	0.00	0.00	0.00	65.90	65.90	115.33
Midspan								
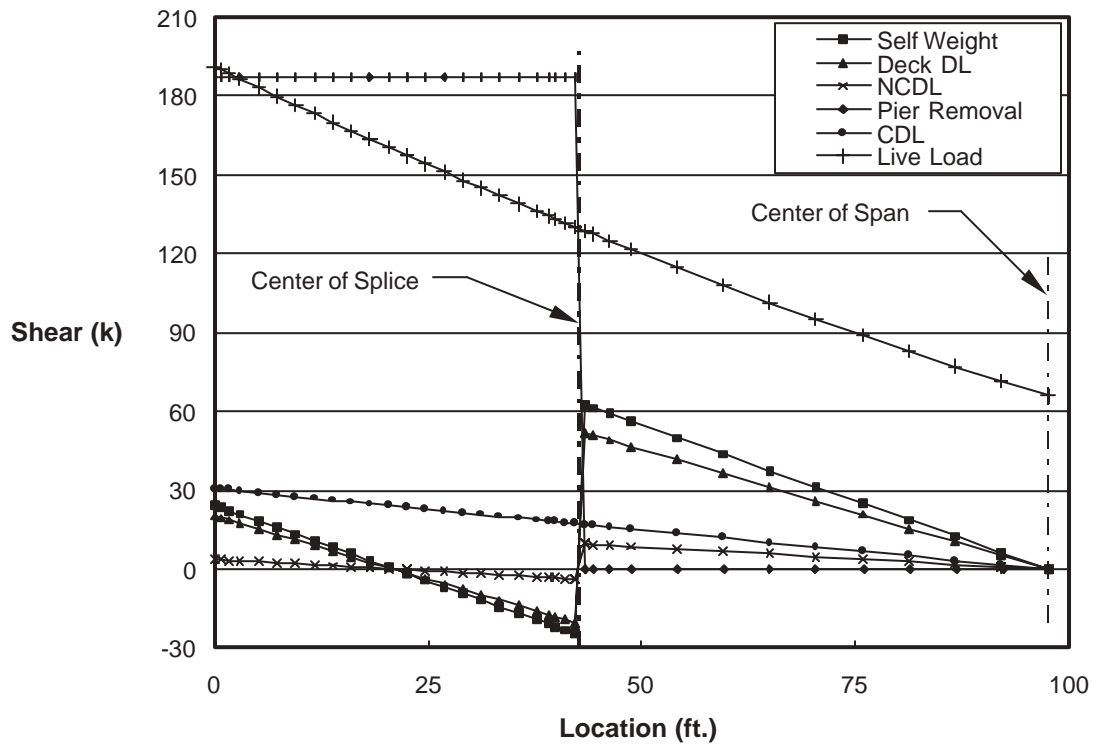
Figure 9.1-1   Vertical Shear due to Loading Components
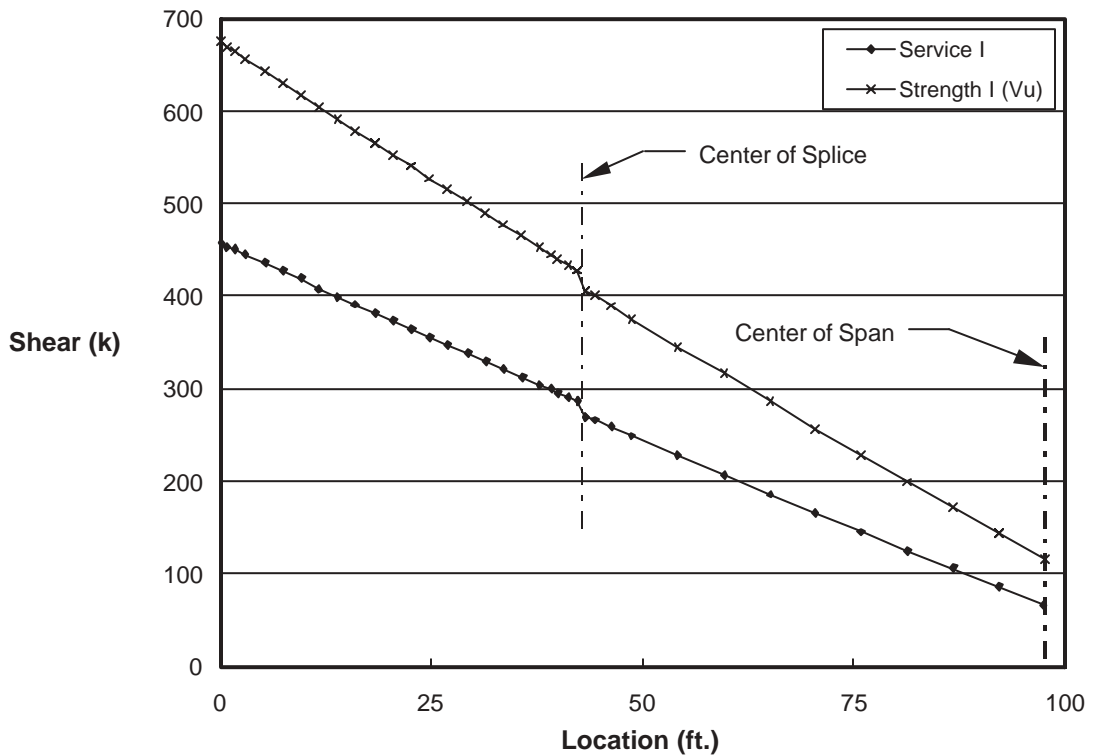

Figure 9.1-2   Total Shear with Live Load

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9.1.1 | **At End of End Block**

The critical section for shear would appear to be at the end of the end block, where the section reduces to the standard cross-section with an 8 in. web. However, the calculation of the shear capacity is no different for this spliced girder than for other members. Therefore, the computation of shear capacity at this location is not demonstrated in the design example.

9.1.2 | **At Splice Locations**

Shear at the splice location must be considered since there is a potential plane of weakness at the faces of the splice. The interface shear provisions in LRFD Art. 5.8.4 are used to evaluate the capacity at this location.

For this evaluation, the interface will be considered to be a concrete to hardened concrete surface that has not been intentionally roughened. This is a conservative assumption. Therefore, the following interface shear parameters apply:

c = 0.075 ksi   LRFD Art. 5.8.4.2

μ = 0.6λ = 0.6 for normal weight concrete where λ = 1.0   LRFD Art. 5.8.4.2

The nominal shear resistance of the interface is computed using

V_n = $cA_{cv} + \mu[A_{vf}f_y + P_c]$ LRFD Eq. 5.8.4.1-1

where:

A_{cv} = area of concrete engaged in shear transfer

= assume the web width x (total section depth – depth in tension [see **Section 13.1**])

= 8 x (104.25 – 10) = 754 in^2

A_{vf} = area of shear reinforcement crossing the shear plane

= 12 (0.31) [rebar] + 15.624 [post-tensioning tendons]

= 3.72 + 15.624 = 19.34 in^2

f_y = yield strength of reinforcement

= use 60 ksi for mild reinforcement

P_c = permanent net compressive force normal to the shear plane

= $f_{pe} A_{ps}$ [for post-tensioning tendons, using the minimum f_{pe} of say, 140 ksi]

= 140 (15.624) = 2,187 kips

Therefore,

V_n = $cA_{cv} + \mu[A_{vf}f_y + P_c]$ = 0.075(754) + 0.6[19.34(60) + 2,187]

= 56.5 + 0.6[1,160 + 2,187] = 56.5 + 2,008 = 2,065 kips

Comparing the factored shear, V_u, to the factored nominal shear resistance, ϕV_n, with ϕ = 0.90 (LRFD Art. 5.5.4.2.1), using the maximum shear at the face of a splice:

V_u = 427 kips < ϕV_n = 0.9(2,065) = 1,858 kips   O.K.

This comparison indicates that the shear resistance of the splice is adequate with a significant surplus capacity. Almost all of the capacity comes from the combination of the reinforcement crossing the shear interface (the post-tensioning tendons providing almost all of that resistance) and the permanent clamping force of the post-tensioning across the interface. Even if all of the contribution of the reinforcement crossing the interface were neglected, the post-tensioning force would provide more than twice the necessary resistance. Therefore, in this

situation, a shear key is not required to provide the necessary shear resistance at the splice location.

An additional contribution to the shear resistance at the splice is the inclination of the post-tensioning tendons crossing the splice. These tendons provide a component of prestress that resists the applied shear. This effect is considered in vertical shear calculations using the term V_p. This quantity is not large, but does contribute additional capacity.

9.2   **INTERFACE (HORIZONTAL) SHEAR**

The approach for design for interface shear between the girder and deck for this spliced girder is the same as conventional girder designs. Therefore, it will not be demonstrated in this example.

10   **LONGITUDINAL REINFORCEMENT REQUIREMENT**

This requirement, found in LRFD Article 5.8.3.5, is not critical for post-tensioned girders because the tension force is anchored at the end of the girder. Therefore, the entire capacity of the tendons is available up to the end of the girder. Since this requirement is not critical for spliced girders, the computation is not shown here.

11   **DEFORMATIONS**

11.1   **DEFLECTIONS AND CAMBERS**

The calculation of camber and deflections for this spliced girder follows the same general procedures used for conventional pretensioned or post-tensioned simple span girders. Detailed calculations for deflections and cambers are not shown because they are not unique to spliced girder design.

Elastic deflections are computed for all applied loads and stages of prestressing. The camber caused by stressing the post-tensioning tendons is computed using the assumed parabolic tendon profile. The deflection for the pier removal load, which occurs as the post-tensioning tendons are stressed, is computed assuming the load is applied at the center of the splice.

Elastic deflections are then factored for creep effects using the procedures of the PCI multiplier method. The development of deflection multipliers for this bridge is discussed in **Section 11.1.1**.

The elastic and factored deformations are then accumulated taking into account the change in structure as the girder segments are spliced and post-tensioning tendons are stressed, and any elevation adjustment at the temporary supports (see **Section 11.1.2**).

The deflection and camber components at midspan of Girder Segment 2, which is also midspan of the spliced girder, are summarized in **Tables 11.1-1** and **11.1-2** for each stage of construction. An additional construction stage is shown for the erected girder segments prior to placement of the deck, splice and diaphragm concrete. A row is added in the table to consider camber adjustments made by raising or lowering the elevation of the temporary supports with respect to the chord between permanent bearing elevations to achieve the desired final deflected shape.

The net camber along the girder segments is shown in **Figure 11.1-1** for the first three construction stages, where the girder segments are acting as individual simple spans. The net

camber along the spliced girder is shown in **Figure 11.1-2** for the construction stages after splicing the girder segments and stressing the post-tensioning tendons. The net camber along the spliced girder is shown in **Figure 11.1-3** for the completed bridge, before and after long-term deflections, and for the design condition, used to set screed elevations, which is the average of the other two deflections shown in the figure. The grade on the bridge is therefore set using half of the long-term deflection following completion of the bridge. This fraction was selected arbitrarily.
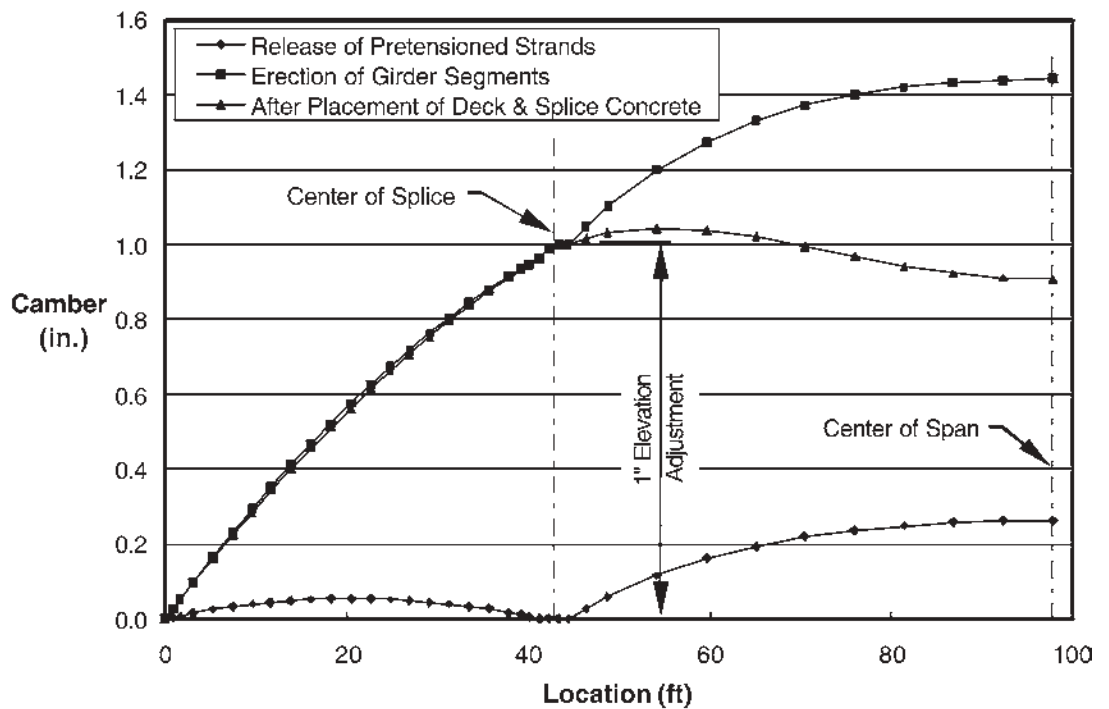
Figure 11.1-1   Camber at Early Construction Stages

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
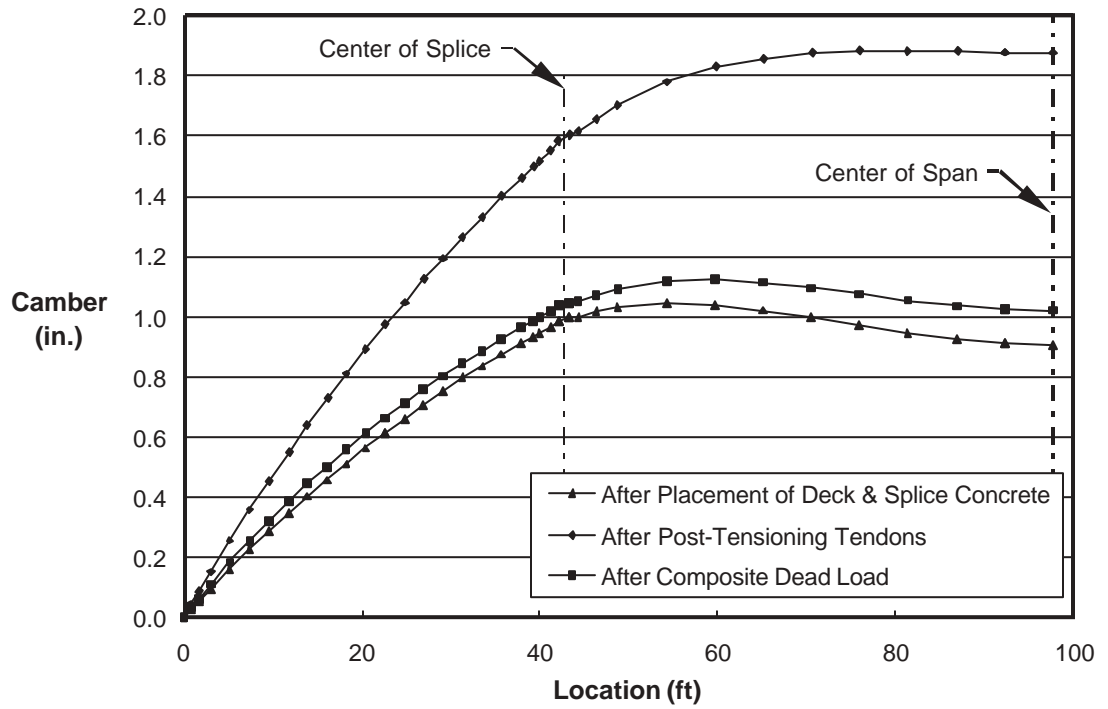
Figure 11.1-2   Camber at Later Construction Stages
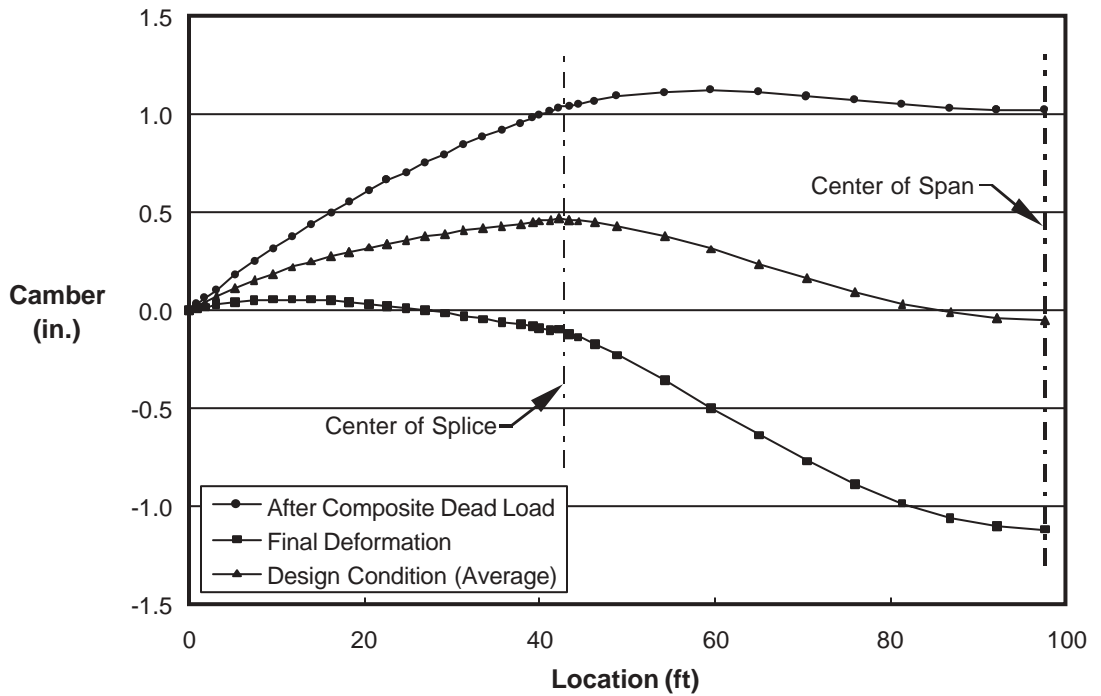
Figure 11.1-3   Camber after Completion of Bridge

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The deflection of the center girder segment under the load of the deck concrete was also considered when selecting the number of pretensioned strands to be used in the segment to keep the girder essentially flat with the deck in place. Additional pretensioned strands could have been added if additional camber were desired in Girder Segment 2.

Table 11.1-1   Camber Summary at Midspan – Early Construction Stages

	Release			Erection			After Deck Casting			After Post-Tensioning		
	Δ_e in.	k_D	$k_D\Delta_e$ in.	Δ_e in.	k_D	$k_D\Delta_e$ in.	Δ_e in.	k_D	$k_D\Delta_e$ in.	Δ_e in.	k_D	$k_D\Delta_e$ in.
Pretension	0.88	1.00	0.88	0.88	1.80	1.58	0.88	1.80	1.58	0.88	1.80	1.58
Self Weight	-0.62	1.00	-0.62	-0.62	1.85	-1.15	-0.62	1.85	-1.15	-0.62	1.85	-1.15
Elevation Adjustment	---	---	---	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Deck Dead Load	---	---	---	---	---	---	-0.45	1.00	-0.45	-0.45	1.00	-0.45
NCDL	---	---	---	---	---	---	-0.08	1.00	-0.08	-0.08	1.00	-0.08
Post-Tensioning	---	---	---	---	---	---	---	---	---	6.14	1.00	6.14
Pier Removal	---	---	---	---	---	---	---	---	---	-5.16	1.00	-5.16
CDL	---	---	---	---	---	---	---	---	---			
Total	0.26			1.44			0.91			1.88		
k_D = Deflection Multipliers												

Table 11.1-2   **Camber Summary at Midspan – Later Construction Stages**

	With CDL			Average Service Life	Final		
	Δ_e in.	k_D	$k_D\Delta_e$ in.	$k_D\Delta_e$ in.	Δ_e in.	k_D	$k_D\Delta_e$ in.
Pretension	0.88	1.80	1.58	1.67	0.88	2.00	1.76
Self Weight	-0.62	1.85	-1.15	-1.29	-0.62	2.30	-1.43
Elevation Adjustment	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Deck Dead Load	-0.45	1.00	-0.45	-0.59	-0.45	1.60	-0.72
NCDL	-0.08	1.00	-0.08	-0.11	-0.08	1.60	-0.13
Post-Tensioning	6.14	1.00	6.14	8.60	6.14	1.80	11.05
Pier Removal	-5.16	1.00	-5.16	-8.00	-5.16	2.10	-10.84
CDL	-0.86	1.00	-0.86	-1.33	-0.86	2.10	-1.81
Total	1.02			-0.05	-1.12		
k_D = Deflection Multipliers							

11.1.1   **Deflection Multipliers**

The computation of the approximate deflection multipliers used in the PCI multiplier method is based on an article by Martin *(5)*. A generalized formulation of Martin's method was used to determine the multipliers shown in **Tables 11.1-1** and **2**. The generalized formulation added a correction for age at loading (for the deck dead load, taken from LRFD Eq. 5.4.2.3.2-1 and considered a deflection reduction from the presence of mild reinforcement in the deck. A detailed discussion of the computations for the deflection multipliers is not provided here.

11.1.2   **Elevation Adjustments at Temporary Supports**

The use of temporary supports to erect the girder segments provides an opportunity to adjust the final deflected shape of the spliced girder. That is, the elevation of the bearings at the temporary supports can be set to eliminate sag deflection from the long-term girder deflection or to conform the bridge to the grade line of the roadway. The elevation adjustments made at the towers affect the camber of all girder segments for which adjustments are made at one or both ends of the girder segment. The elevation adjustment is shown in **Figure 11.1-1** for construction stages other than release and its effect is included in **Figure 11.1-2**.

The adjustment in elevation at the temporary supports is entered in **Tables 11.1-1** and **11.1-2** in the row "Elevation Adjustment." Since this is a fixed dimension, no multiplier is applied. For this bridge with a symmetrical layout of girder segments, equal adjustments are made at each temporary support. This results in a constant correction for the length of the center girder segment and a linearly varying correction for the end girder segments. With an asymmetrical

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

arrangement of girder segments, adjustments would be computed separately for each tower, and the correction would vary along the length of a center girder segment.

Without any elevation adjustment, the camber after post-tensioning and placement of the composite dead loads was only 0.02 in. upward (see **Table 11.1-2**, neglecting the elevation adjustment). With time, the bridge is expected to sag, with the final (long-term) deflection estimated to be a downward sag of 2.12 in. The bridge roadway profile was assumed to be a level grade.

Raising the bearing elevations at the temporary supports 1.00 in. changes each of the deflections mentioned above by that amount, resulting in a camber at midspan after post-tensioning and placement of the composite dead loads of 1.02 in. upward and a final deflection of 1.12 in. downward (as shown in **Table 11.1-2**). This results in the profile of the bridge being within about 1.00 in. (±) of the intended elevation at midspan for the life of the bridge. **Figure 11.1.2-1** shows the deflected shape of the bridge with and without an elevation adjustment for two stages of construction.
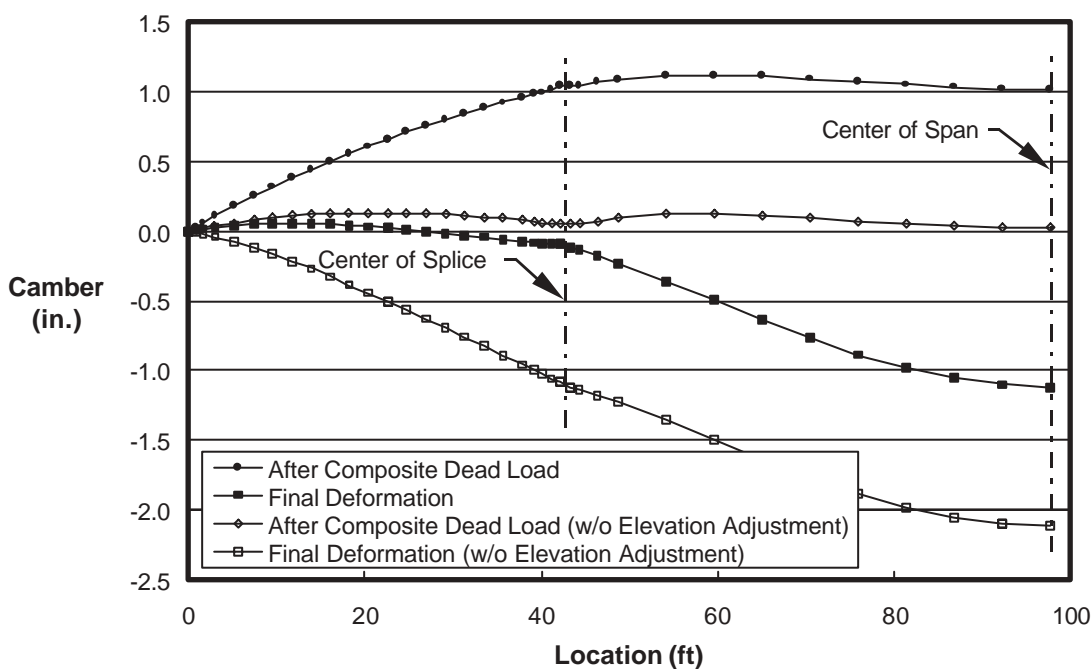
Figure 11.1.2-1   Cambers with and without Elevation Adjustment

The average of the deflections after placement of the composite dead loads and the final condition is used as the design condition for setting deck slab elevations during construction.

11.1.3   **Verification of Build-up**

Considering the average final deflection condition as the design condition for setting final grade, the variation of the camber along the length of the girder is at most about 0.5 in. This verifies that assumed value of build-up of 2 in. is adequate to provide camber tolerance during

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construction. Therefore, no modification of the assumed build-up was required for final design.

Vertical curves should also be considered when verify the build-up.

| 11.1.4 | **Live Load Deflection** |

The live load deflection for the spliced girder was computed using the procedure outlined in the *PCI Bridge Design Manual* and the provisions of LRFD Art. 2.5.2.6.2 and 3.6.1.3.2. The entire cross-section of the bridge was assumed to deflect uniformly under the live load, so the effective live load distribution factor was the number of lanes of traffic (4) divided by the number of girders (7) resulting in a distribution factor of 0.571 lanes per girder.

The computed live load deflection for this spliced girder is 1.30 in., which is significantly less than the optional deflection limit of 2.93 in. specified in LRFD Art. 2.5.2.6.2. Therefore, this bridge satisfies the optional deflection criteria of the *LRFD Specifications*. Detailed calculations are not shown for the live load deflection because they are not unique to spliced girder design.

| 11.2 | **SHORTENING** |

The shortening of the girder segments and the completed bridge due to prestress and time-dependent effects is computed below. These values are required for the precaster to adjust girder lengths, the contractor to make provisions for movement during post-tensioning, and for the designer to detail the expansion joints and bearings. The shortening caused by prestress is computed using the axial component of the prestress effect, i.e., P/A, with the force, P, being the force in the strands or tendons after release or seating, before long-term losses.

| 11.2.1 | **Pretensioning** |

Pretensioning is applied to each girder segment. The shortening of the prestressed concrete girder segments is small. These calculations are typically performed by the precaster so they can adjust the length of the girder segments prior to fabrication. The values for the end girder segments can be neglected.

$$\Delta_{gs1} = \frac{PL}{EA} = \frac{(410)(43.5)(12)}{(4{,}888)(1{,}103)} = 0.04 \text{ in.} \qquad \text{Girder Segments 1 and 3}$$

$$\Delta_{gs2} = \frac{PL}{EA} = \frac{(820)(108.7)(12)}{(4{,}888)(1{,}103)} = 0.20 \text{ in.} \qquad \text{Girder Segment 2}$$

These elastic values will not contribute to the shortening of the final structure because they occur prior to placement of the splices. However, they will contribute to the creep shortening of the final structure.

| 11.2.2 | **Post-Tensioning** |

Because the prestress force applied when the post-tensioning tendons are stressed is large, the shortening is noticeable.

$$\Delta_{PT} = \frac{PL}{EA} = \frac{(3{,}057)(198)(12)}{(4{,}888)(1{,}844)} = 0.81 \text{ in.}$$

| 11.2.3 | **Shrinkage** |

LRFD Art. 5.4.2.3.1 gives a shrinkage strain, ε_{sh}, of 0.0005 in./in. for situations where the shrinkage is not known. This strain can be reduced using the volume-to-surface ratio (V/S)

reduction factor for shrinkage, k_c. The factor can be estimated from LRFD Figure 5.4.2.3.2-1 assuming a V/S of about 4 in. (since the girder web is 8 in. and the deck is approximately 8 in. thick, and the V/S is half the thickness for a long slender member). From the table, the reduction factor for V/S = 4 in. is about 0.8 (further refinement is not warranted).

$$\Delta_{SH} = k_c \, \varepsilon_{sh} \, L = 0.8(0.0005)(198*12) = 0.95 \text{ in.}$$

where L = the full length of the complete bridge.

11.2.4   **Creep**

The effect of creep is represented by a creep coefficient. In this case, the creep coefficient, ψ, is estimated to be 2. Therefore, the total eventual shortening from an elastic load, such as application of post-tensioning, will be twice the initial elastic amount.

$$\Delta_{CR \, PT \, tot} = \psi \, \Delta_{PT} = 2 \, (0.81) = 1.62 \text{ in.}$$

The creep shortening that occurs due to post-tensioning would then be this total value minus the elastic shortening, or

$$\Delta_{CR \, PT} = (\psi - 1) \, \Delta_{PT} = (2 - 1)(0.81) = 0.81 \text{ in.}$$

For the pretensioned girder segments, only the creep deformation will be added to the long term shortening, as discussed above. The creep deformation for the pretensioned girder segments is

$$\Delta_{CR \, Pret} = (\psi - 1) \, \Delta_{gs} = (2 - 1) \, 0.28 = 0.28 \text{ in.}$$

where

$$\Delta_{gs} = \Delta_{gs1} + \Delta_{gs2} + \Delta_{gs3} = 0.04 + 0.20 + 0.04 = 0.28 \text{ in.}$$

11.2.5   **Total Long Term Shortening**

The total long term shortening, which occurs after post-tensioning, is the sum of the shrinkage and creep deformations:

$$\Delta_{Total \, LT} = \Delta_{SH} + \Delta_{CR \, PT} + \Delta_{CR \, Pret} = 0.95 + 0.81 + 0.28 = 2.04 \text{ in.}$$

The elastic and long term shortening which occurs after splicing and includes the elastic shortening that occurs during stressing of the post-tensioning tendons, is

$$\Delta_{Total} = \Delta_{Total \, LT} + \Delta_{PT} = 2.04 + 0.81 = 2.85 \text{ in.}$$

These values of shortening must be considered in the detailing of bearings and expansion joints.

11.3   **ROTATIONS**

The procedure for computing rotations for this bridge is the same as for other pretensioned and post-tensioned girders. Therefore, details of computing rotations are not shown here.

12   ANCHORAGE AND STRESSING DETAILS

12.1   **ANCHORAGE OF PRETENSIONED STRANDS**

LRFD Art. 5.10.10.1 requires that a minimum amount of reinforcement be provided at the end of pretensioned girders to resist bursting stresses. For these girder segments, which are very lightly pretensioned, the required area of reinforcement is small. To satisfy this requirement, the end girder segments require one No. 5 double-legged stirrup and the interior segment

requires two No. 5 double-legged stirrups. This quantity of reinforcement, which must be provided at both ends of each girder segment, can easily be provided by stirrups.

Nominal confinement reinforcement, as required by LRFD Art. 5.10.10.2, must also be provided at both ends of each girder segment.

Both of these requirements are typical for all pretensioned girders, so computations and details are not shown in this design example.

| 12.2 | **ANCHORAGE OF POST-TENSIONING TENDONS** |

A number of issues must be considered when detailing anchor regions for post-tensioning tendons. A brief treatment of these issues is given below.

| 12.2.1 | **Anchorage Hardware and Stressing Equipment** |

Anchorage hardware from several post-tensioning suppliers should be considered when detailing practical anchor dimensions for the size of tendon being used. Dimensions for both the anchor casting and the spiral confining reinforcement in front of the anchor should be determined, as well as the recommended spacing between anchors and the minimum edge distance from the center of the anchor to the face of concrete. For the girder in this design example, the anchor bearing flange is assumed to be 10½ in. square, with a 13 in. spacing between tendons at the anchor location.

A blockout must be provided for the anchorage hardware so that the face of the anchor is positioned perpendicular to the tendon as it exits the concrete. For this example, all the tendons exit the girder at the same angle. Therefore, all the blockouts are fabricated to the same angle, simplifying fabrication. Individual blockouts can be detailed, or, since all anchorages are oriented at the same angle, a single, continuous blockout may be used.

The blockout must be sized to provide at least the minimum dimensions required for the tendon stressing equipment. This information should be obtained from the post-tensioning supplier for the anchorage hardware to be used. Longitudinal clearances are not as critical for stressing equipment in this case because the stressing anchorage is at an abutment. For bridges with multiple post-tensioned units, space must be provided between adjacent ends of girders to accommodate the tensioning equipment or else the tendon paths at the anchors must deviate to accommodate it. This must be carefully considered when detailing the end block.

The blockout dimensions must also satisfy requirements for adequate encasement of the tendon anchor heads after stressing (see **Section 12.2.6**).

| 12.2.2 | **End Block** |

An end block is required to house the tendon anchorage hardware and accompanying reinforcement. For this girder, the end block has been sized to match the width of the bottom flange to simplify fabrication. Details of the end block are shown in **Figure 12.2.2-1**.

The length of the end block, measured from the face of the anchors to the start of the end block, should be at least as long as the height of the girder, as indicated in LRFD Art. C5.10.9.6.1. This requirement allows the end block to extend beyond the limits of the "disturbed region" at the end of the girder. Thus, the transition from the end block cross-section to the normal girder cross-section will occur in a region relatively free from stress concentrations due to concentrated loads at the post-tensioning anchorages and the reaction at the bearing. In this example, an end block length of 10'-6" is used, which satisfies the above criteria and provides additional length so that the end block extends for at least the height of the girder beyond the bearing.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**

The taper at the start of the end blocks is shown at a 2:1 slope to reduce the stress concentrations due to change in section. A gradual transition is also recommended in LRFD Art. C5.10.9.6.1.
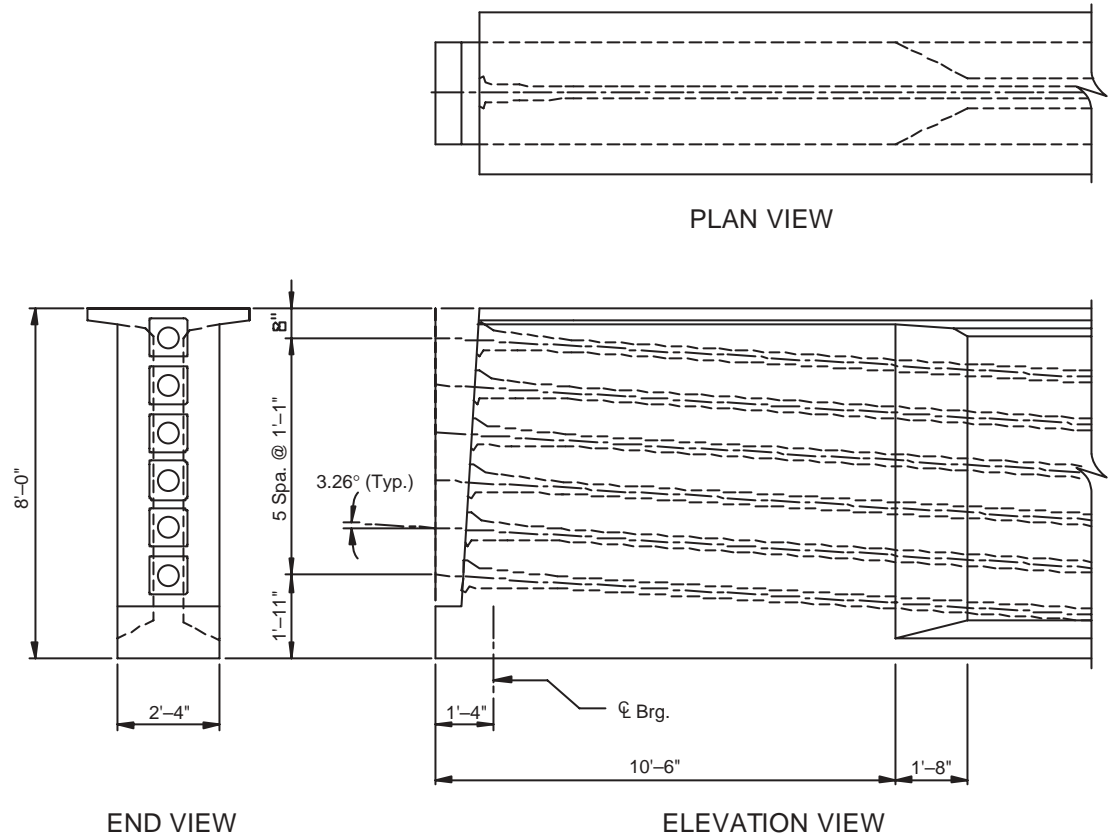
PLAN VIEW

END VIEW   ELEVATION VIEW

Figure 12.2.2-1   End Block Details

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Encasement
Concrete

℄ Brg.
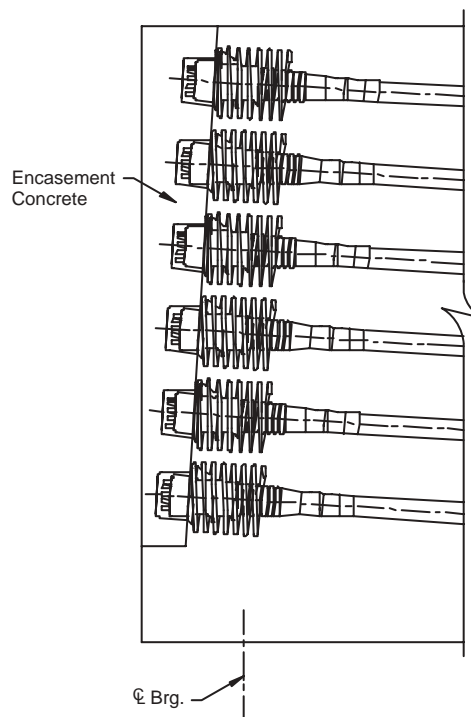
Figure 12.2.2-2　　Anchorage Detail

| 12.2.3 | **Local Zone Design** |

Reinforcement of the local zone, which is defined in LRFD Articles C5.10.9.2.3 and 5.10.9.7.1, is generally detailed by the post-tensioning supplier for a specific anchorage device. The bridge designer usually does not perform the design of this reinforcement but is still responsible for ensuring that it meets the requirements of LRFD Article 5.10.9.7.

| 12.2.4 | **General Zone Design** |

The general zone is defined in LRFD Article C5.10.9.2.1. The design and detailing of reinforcement for the general zone is the responsibility of the bridge designer. Because a detailed treatment of this subject cannot be adequately covered in this report, the reader is urged to consult other references *(6, 7, 8)* in addition to LRFD Article 5.10.9.3.

Design of the general zone includes the following three considerations, as described in LRFD Article 5.10.9.3.2:

1) Design of reinforcement to resist bursting forces in front of the anchorage device (Note: This may also include a check of compression stresses ahead of the anchorage device.)

2) Design of reinforcement to resist spalling forces on the back face (or anchor face) of the member.

3) Design of reinforcement to resist longitudinal edge tension forces for eccentrically loaded anchorage zones.

12.2.4.1

Web Bursting in Anchor Block

LRFD Articles 5.10.9.3.1 and 5.10.9.3.2 permit the bursting force in the anchorage zone, T_{burst}, and its distance from the loaded surface, d_{burst}, to be determined using one of three general approaches:

1) Strut-and-tie models

2) Refined elastic stress analyses

3) Other approximate methods (such as that given in LRFD Article 5.10.9.6)

Because the girder end block meets the requirements of LRFD Art. C5.10.9.6.1, the approximate method of Article 5.10.9.6 can be used. Since the center-to-center spacing of the anchorage devices does not exceed 1.5 times the width of the anchor, LRFD Art. C5.10.9.6.1 allows the anchors to be treated as "closely spaced."

12.2.4.1.1

Vertical Bursting

In the following computations, the term ΣP_u represents the sum of the factored force in each tendon. Since the tendons are tensioned sequentially, the force in the tendons will be different. For the purposes of this computation, the maximum value for ΣP_u occurs when the final tendon is stressed, so one tendon will be considered to be at its maximum value for jacking (0.90 f_{py}), while the other five tendons will be considered to be at the maximum stress permitted after anchoring the tendon (0.70 f_{pu}). It should be noted that these stresses give the maximum possible combination of forces allowed by the *Specifications*, and are not directly related to the actual tendon forces used in this design. This approach allows for possible field adjustments of the post-tensioning.

Since the number of pretensioned strands in the girder segment is relatively small, the force from the pretensioned strands has been neglected in this analysis.

The bursting force and its location can be computed as:

$$
\begin{aligned}
T_{burst} &= 0.25\ \Sigma P_u\ [1\text{-}a/h] + 0.5\ |\Sigma(P_u \sin \alpha)| & \text{LRFD Eq. 5.10.9.6.3-1}\\
&= 0.25(3{,}585.7)[1\text{-}0.711] + 0.5|(3{,}585.7)\sin(3.263°)|\\
&= 361.5 \text{ k}
\end{aligned}
$$

$$
\begin{aligned}
d_{burst} &= 0.5\ (h\text{-}2e) + 5e \sin \alpha & \text{LRFD Eq. 5.10.9.6.3-2}\\
&= 0.5(106.25 - 2(13.7)) + 5(13.7)\sin(3.263°)\\
&= 43.3 \text{ in.}
\end{aligned}
$$

where:

ΣP_u = Sum of factored tendon forces

= $\gamma_j [f_{pj}(1 \text{ Tendon})(A_{ps} \text{ per Tendon}) + 0.70 f_{pu}(5 \text{ Tendons})(A_{ps} \text{ per Tendon})]$

= 1.2 [0.9(0.9(270))(1)(12)(0.217) + 0.70(270)(5)(12)(0.217)]

= 3,585.7 k

γ_J = 1.2   LRFD Art. 3.4.3.2

h = Out-to-out height of cross-section upon which post-tensioning is applied

= 106.25 in.

a = Height of the combined anchorage plates

= 5(13.0) + 10.5

= 75.5 in.

$$a/h \quad = \quad 75.5/106.25$$
$$= \quad 0.711$$

$$\alpha \quad = \quad \text{Angle of inclination of tendon force}$$
$$= \quad 3.263°$$

$$e \quad = \quad \text{Eccentricity of the group of anchorage devices}$$
$$= \quad 13.7 \text{ in.}$$

The area of mild reinforcement required in the anchorage zone to resist T_{burst} is then computed as:

$$A_s \quad = \quad T_{burst}/f_y$$
$$= \quad 361.5/60$$
$$= \quad 6.02 \text{ in}^2$$

This reinforcement should be centered at d_{burst} from the end of girder. The required area of mild reinforcement can easily be provided by using No. 5 double-legged stirrups spaced at 10.0 in. throughout the region defined in LRFD Fig. C5.10.9.3.2-1. Due to the small amount of pretensioned strands in the girder segment at the anchorage zone, the force due to the pretensioned strands has been neglected in this analysis.

Additional shear reinforcement may be required in this zone, however the calculations do not differ from those of typical pretensioned girders. Thus these calculations are not shown in this example.

12.2.4.1.2 | *Transverse Bursting*

In the transverse direction, the general zone and the local zone are approximately the same. Therefore, the reinforcement requirements for the local zone will govern the design. Furthermore, the end diaphragm will be cast prior to stressing the tendons, so the diaphragm will provide additional confinement to resist bursting in this direction.

12.2.4.2 | **Spalling Force**

LRFD Art. 5.10.9.3.2 requires a minimum area of spalling reinforcement to be provided in the end of the end block, directly behind the anchorage devices. Since the center-to-center spacing of the anchorages is less than 0.4 times the depth of the girder segment section, the spalling tension force that needs to be resisted is equal to $0.02P_u$.

$$F_{spall} \quad = \quad 0.02(3,585.7)$$
$$= \quad 71.7 \text{ k}$$

The area of mild reinforcement required in the for spalling tension is then computed as:

$$A_s \quad = \quad F_{spall}/f_y$$
$$= \quad 71.7/60$$
$$= \quad 1.20 \text{ in}^2$$

12.2.4.3 | **Longitudinal Edge Tension**

Because the centroid of the tendon forces lies inside the kern of the section, longitudinal edge tension forces will not develop (LRFD Article C5.10.9.6.4).

12.2.4.4 | **Effects of Stressing Sequence**

The sequence of tendon stressing has an effect on forces induced in the anchor zone region, particularly with regard to edge tension forces (spalling and vertical.) Assuming the bottom

tendon is stressed first, the anchorage region will develop spalling and longitudinal edge tension forces due to the eccentricity of the post-tensioning force on the section. Although these forces are temporary, it is necessary to verify their magnitude during design to ensure that excessive cracking does not result.

For this example problem, the design of the anchorage zone for temporary stresses due to the post-tensioning sequence are not shown. However, edge tension forces can be determined directly from a strut-and-tie model of the anchorage zone or from LRFD Article 5.10.9.6.4 if approximate methods are being used.

12.2.5   **Tendon Stressing Sequence**

12.2.5.1   **Within Girder Segments**

For each girder, tendons must be stressed in a sequence that avoids overstressing the cross-section (See **Section 12.2.4.4**). Since tendons stressed early in the sequence will experience greater elastic shortening losses, the bottom tendons should be stressed at or near the end of the sequence to maximize the stress in the tendons. Additionally, the first tendons to be stressed should be near the center of the section to minimize the effects of eccentricity on the anchorage zone.

Considering these factors, the following stressing sequence has been selected for the tendons in each girder.

Table 12.2.4.1-1 Post-Tensioning Tendon Stressing Sequence for each Girder

Tendon Number	Stressing Sequence
6 (Top)	4
5	3
4	2
3	1 (First)
2	5
1 (Bottom)	6 (Last)

12.2.5.2   **Across Bridge**

Post-tensioning tendons must be stressed in a sequence that allows the bridge to be stressed uniformly. Therefore, the stressing must proceed across the bridge as it progresses within each girder. This is important to prevent overstressing girders or diaphragms as the bridge moves upward and loads are redistributed from the temporary supports to the abutments. The stressing sequence should be shown on the contract plans.

Generally, stressing begins at the center girder, with no more than two tendons stressed in a girder before moving to the next girder. After the center girder, stressing continues outward, alternating sides to minimize the eccentricity about the centerline of the structure. An equal number of tendons are stressed in all girders before additional tendons are stressed in any girder.

| 12.2.5.3 | Anchorage Encasement |

The post-tensioning anchorages must be encased in concrete to provide long-term protection from corrosion. The surface of the concrete must be carefully prepared to ensure excellent bonding to the encasement concrete. Dowels should extend from the girder anchorage to engage the encasement concrete.

13 SPLICE AND DIAPHRAGM DETAILS

13.1 REINFORCEMENT IN SPLICE

The reinforcement in the splice between girder segments is proportioned to satisfy the requirements for:

- Stress limits for the splice at the service limit state

- Shear in the splice

The reinforcement required to satisfy shear requirements was presented in **Section 9.1.2**, where it was noted that the mild reinforcement is not required to provide a significant portion of the shear resistance because of the contribution of other resisting mechanisms. Therefore, only nominal reinforcement is provided crossing the shear interface.

A quantity of reinforcement must be computed as part of the limiting tensile stress for the splice location (see LRFD Table 5.9.4.2.2-1). An area of reinforcement is required that resists the full tensile force in the concrete at the splice at a working stress of 0.5 f_y, where f_y is taken as 60 ksi. In this example, the required area is supplied using mild reinforcement, neglecting the contribution of the post-tensioning tendons crossing the splice.

The tensile force in the concrete is computed by determining the depth of tension at the bottom of the splice, which will be designated as x. This is accomplished using the absolute values of the computed stresses at the top and bottom of the splice taken from **Table 8.1.3-4** through **8.1.3-6** for Service III load combination. The location with the maximum tensile stress was used here, which was 43.4 ft from the center line of bearing. Therefore, x may be computed as:

$$x = [f_{bot} / (f_{top} - f_{bot})] h = [0.203 / (1.749 + 0.203)] 96 = 9.98 \text{ in., Say 10 in.}$$

This distance is greater than the thickness of the bottom flange. However, assuming that the bottom flange is a constant width up to the distance x is conservative, requiring more reinforcement, but not a lot more since it is at the end of the tensile zone where the stress is reducing to zero.

The tensile force, T, is then computed as the product of the average stress and the width of the bottom flange, which is 28 in., or

$$T = f_{ave}[\text{tensile area}] = (f_{bot} / 2) [b_{bot} x] = (0.203/2) [28(10)] = 28.4 \text{ kips}$$

The required area of reinforcement is computed by dividing this tensile force by the working stress of 0.5 f_y to obtain:

$$A_s = T / 0.5 f_y = 28.4 / (0.5(60)) = 0.95 \text{ in}^2$$

USE 4 - #5 bars with $A_s = 4(0.31) = 1.24 \text{ in}^2 > 0.95 \text{in}^2$ O.K.

This area of reinforcement must be provided within the tension zone. Four bars were used to provide symmetry in the connection. The bars are positioned in the section so that they mesh with bars from the opposite girder.

These bars must be developed within the splice. The required length to develop a hooked bar is computed to be 11.2 in. for a #5 bar with f'$_c$ = 4.5 ksi (LRFD Art. 5.11.2.4). Therefore, the bars are detailed as projecting 12 in. from the end of the girder prior to a standard hook.
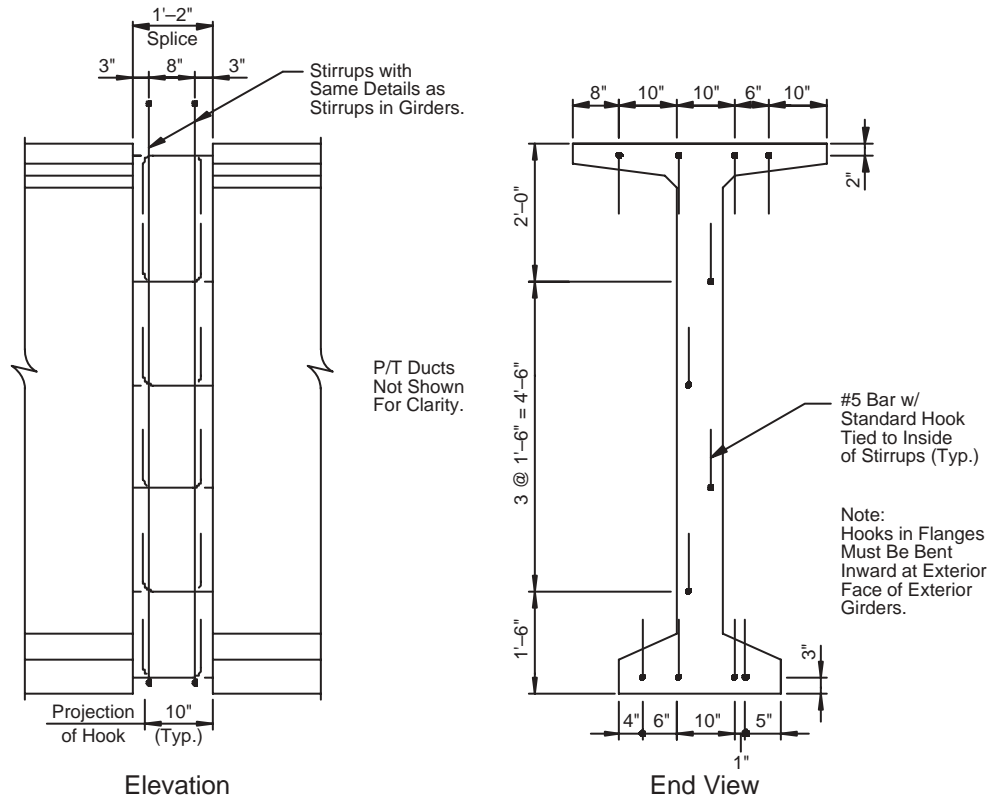

Figure 13.1-1   Details of Reinforcement in Splice

As required by LRFD Art. 5.14.1.2.6b, stirrups shall be provided in the splice with a spacing not to exceed the least of the spacing in the adjacent girder segments. The same stirrup size and detailing should be used.

Reinforcement in the splice should be detailed so that access to splicing the post-tensioning ducts will not be significantly restricted.

13.2   **DUCT SPLICING DETAIL**

The detail for splicing of the duct should be obtained from a supplier. The length of the coupler and other duct splice details are important factors in determining the width of the splice.

The system recommended for this project is a split coupler that is placed over the ducts projecting from the end of the girder segments. The coupler is then taped to seal the connection after being placed on the projecting ducts. The splice is 8 in. long. The ducts should extend approximately 3 in. into the coupler, so they must project at least 6 in. from the end of the girder segment.
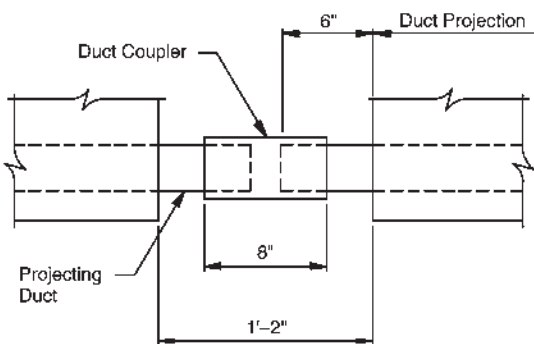
Figure 13.2-1   Schematic Detail of Duct Splice

13.3   SHEAR KEYS

Shear keys are not required for this design for shear capacity. However, shear keys are provided in some bridges as an added factor of safety at the splice location. See the body of the report for more information on details of shear keys.

13.4   REINFORCEMENT OF DIAPHRAGM AT SPLICE

The diaphragm reinforcement is nominal reinforcement. The diaphragm does not resist any design loads. Therefore, detailing of the diaphragm reinforcement is similar to any other cast-in-place diaphragm for prestressed concrete girders. Reinforcement must be detailed to avoid conflicts with post-tensioning ducts and to be adequately anchored at exterior girders.

13.5   APPEARANCE OF SPLICE CONCRETE

Several spliced girder bridges suffer from mismatched concrete coloration at splice locations. This is the result of different concrete mixes being used for the girder and splice concrete. This can be avoided by using the same concrete for the splice as was used for the girder. This is not generally practical. As an alternate, constituent materials for the splice concrete could be obtained from the same source as the girder concrete. Trial batches may be desired to provide the best possible match in color if the bridge will be constructed in a location where the splice will be visible.

Another option is to make the splice location an architectural feature by extending the diaphragm beyond the face of the exterior beam. This provides a distinct visual element that may obscure the difference in color between the concretes. A final option would be to apply a pigmental sealer or surface coat to the girder to mask the different colors. However, this may lead to maintenance issues

14   ADDITIONAL DESIGN ISSUES

14.1   SPECIAL REQUIREMENTS

LRFD Art. 5.14.1.2 addresses precast beams. It contains several requirements that apply to prestressed concrete spliced girders.

14.2   BEARING DESIGN

14.2.1   **Permanent Bearings**

The bearings at the ends of these spliced girders are no different from long-span single piece prestressed concrete girders. Large laminated elastomeric bearing pads may be adequate to support the loads and provide the required movements. Pot bearings may be required for some spliced girder designs. Embedded plates in the ends of the girders may be necessary at permanent bearing locations to secure the girders to the large bearing assemblies that may be required. Since the design of the bearings is no different than for other long-span girders, the details of the bearing design is not shown here.

14.2.2   **Accommodation of Shortening during Post-Tensioning**

As computed in **Section 11.2.2**, the girder will shorten approximately 0.81 in. This shortening must be permitted to occur without significant resistance or else the girder will not experience the full post-tensioning force as intended in the design. Therefore, the detailing of the bearings to allow this movement without significant restraint is an important aspect of design.

Several options have been used in existing projects. If the bearings have sufficient shear capacity and flexibility, they can be allowed to deform with the application of the post-tensioning force. However, this deformation will be larger than should be allowed to remain in the bearings in most cases. Therefore, the girder is jacked up off of the bearings after post-tensioning to relieve the shear deformation in the bearings. Other details include using greased bearing surfaces, or using manufactured sliding bearings with Teflon and stainless steel surfaces.

14.2.3   **Temporary Bearings**

At temporary support towers, temporary bearings or leveling pads must be provided to support the girders. These bearing must maintain the girders at the proper elevation, provide a stable bearing, and not damage the girder. Embedded plates are not typically used at the temporary bearing locations.

14.3   TEMPORARY SUPPORT TOWERS

The temporary support towers located at the splice locations must be designed to support the loads of the girders, deck and diaphragms until the girders are post-tensioned. The plans should indicate design loads for the towers. These loads are shown on **Figure 3.1-1** for the erection of girders and for the maximum load condition, which is when the deck, splice and splice diaphragm have been placed and the construction live load is also on the structure.

These loads at the two stages of construction are:

> 90 kips per girder   Girders erected
>
> 200 kips per girder   Girders with deck, splice, diaphragm and construction live load

These loads apply to both interior and exterior girders.

Temporary support towers must be capable of providing a stable unyielding support so that the girder segment elevations can be properly set and maintained.

14.4   TEMPORARY BRACING

Temporary bracing is required to provide stability for the erected girders prior to placement of the deck, splices and splice diaphragms. This bracing is typically comprised of steel shapes

that are temporarily bolted between the girders. Bracing may also be provided from the temporary support towers to the girder segments. After the deck is cast and the girders are post-tensioned, the bracing should be removed.

14.5   HANDLING CONSIDERATIONS

Contractors and fabricators are typically responsible for the handling of prestressed concrete girders. They often consider the location of lifting loops or other factors when reviewing the stability of the girders prior to manufacturing and handling. The spliced girders in this bridge have no special concerns beyond those of typical prestressed concrete girders.

14.6   DECK SLAB CASTING SEQUENCE

The design assumes that concrete is placed in the splices and diaphragms at the same time that the deck concrete is being placed. This may pose a problem for the contractor when casting the deck because of the relatively large size of the diaphragm pours, which could delay the deck concrete placement.

The total deck concrete placement consists of approximately 330 CY of concrete. This can be done in a single placement in most cases. However, since the center girder segment is long, its ends will rotate significantly as the deck concrete is placed on it. The end rotation of the center girder segment is estimated to be approximately 0.126 in. from top to bottom of the girder segment. While the movement at the top or bottom could be approximately one half of this quantity, this is still a significant movement. Therefore, steps should be taken to accommodate this end rotation in some way to avoid potential distress at the diaphragm.

One approach would be to place the entire deck from one end to the other, but to design the concrete mix for the diaphragms and splices so that the concrete remains plastic for the time it takes to place most of the concrete on the center girder segment. In this way, the concrete will not resist the end rotation, and distress will be avoided. Using this approach, it may be beneficial for the contractor to have the capability to place concrete in the diaphragm at the same time that the deck placement is progressing.

A second approach would be to use two concrete placements. The first placement would begin at or near the interior edge of one of the splices and progress across the center girder segment to the far end of the bridge. This scheme would place all of the concrete on the center girder segment before the concrete is placed in the diaphragm and splice. Therefore, all end rotations from the center girder segment would occur prior to concrete placement in the diaphragm. A second concrete placement would then begin at either end of the remaining deck, because the deflection of the end girder segment is negligible. This sequence of deck casting is consistent with the design assumptions and would not affect the performance of the bridge.

There are two possible sequences for casting of the deck slab. The first, as utilized in this design example, incorporates the deck casting at the same time as establishing continuity of the girder segments. The second, as discussed in **Section 15.2**, incorporates the deck casting after continuity has been established. There may not appear to be a large difference in design from a contractors point of view, however, as shown in **Figure 15.2-3**, the moment reduction, created by the second casting sequences, has a dramatic effect on the stress acting on the girder. Special care should be taken to insure that the proper deck casting sequence is followed.

| 15 | OTHER POSSIBLE CONFIGURATIONS |

This section presents several modifications to the design assumptions that could be considered. Each modification is briefly discussed to compare some of the advantages and disadvantages to the original design.

| 15.1 | TWO-STAGE POST-TENSIONING |

The use of staged post-tensioning, with several of the tendons being stressed on the bare girder to make it continuous prior to placement of the deck, is commonly used in multi-span spliced girder bridges. This option was investigated for the bridge in this design example. However, it was found that more post-tensioning would be required if staged post-tensioning were utilized. Furthermore, use of staged post-tensioning would require that the post-tensioning supplier return to the site a second time to complete the post-tensioning. It appears that the ability to cast the deck while the girder segments are supported on temporary towers makes the single-stage post-tensioning more economical.

| 15.2 | SPLICING GIRDERS BEFORE PLACING DECK |

| 15.2.1 | **With Post-Tensioning** |

Several spliced girder bridges have been constructed with the girders spliced and post-tensioned prior to placement of the deck. While all of the post-tensioning is applied to the smaller girder section, more post-tensioning is required to achieve the same span. Additionally, the deck is not precompressed in this scheme, which removes one of the significant benefits of the original design: that the deck is longitudinally prestressed, which reduces or eliminates the transverse cracking that often plaques bridge decks.

The most frequently cited reason for using this construction sequence is that it allows the removal of the deck for replacement at a future date. Since none of the post-tensioning is applied with the deck in place, removal of the deck does not pose the likelihood of overstressing the girders.

| 15.2.2 | **With Mild Reinforcement** |

An alternate to the construction sequence used in the example is to make the girder segments continuous prior to placing the deck concrete using a mild reinforcement connection rather than post-tensioning. The mild reinforcement continuity connections between girder segments would resist the negative moment at the splices when the deck concrete is placed. The post-tensioning would be applied on the composite section after the deck was placed as assumed in the original design.

Splicing the girder segments prior to placing the deck concrete would provide several benefits, which include:

- Precompressing the bottom of the splice, which is generally a critical location for tensile stresses

- Reducing the total deflection of the girder segments by reducing the deflection during deck placement

- Reducing the positive moment stresses locked into the girder segments during deck placement, which contribute to the critical stresses at midspan

- Allowing the use of a higher strength concrete in the splice than is used in the deck, since they are placed at different times

- Speeding placement of deck concrete, because the splice and diaphragm would not be placed at the same time

- Possibly allowing the reduction of the length of the center girder segment

Splicing the girder segments prior to placing the deck concrete also requires that additional issues be considered during design, which include:

- The negative moment connection at the splice location must resist the negative moments developed as the deck concrete is placed. Service load design may be used to determine the quantity of reinforcement, to compute the precompression in the bottom of the splice (estimated to be 0.675 ksi for this bridge), and to limit stresses in the reinforcement to control cracking. Preliminary analysis of indicates that 6.5 in^2 of reinforcement would be required which must be placed in the top flange of the girder. The reinforcement would not have to resist later loads, since the deck and post-tensioning would act to resist these loads.

- The girder segments, especially short end segments, must be designed for significant negative moments along their length (not just at splice) caused by the deck and construction loads.

- The negative moment connection between girder segments must be properly designed detailed. While bars can extend from the girders and be hooked down into the diaphragm, this detail would not be usable for the bars in the exterior girders since the diaphragm does not extend beyond the exterior face of the girder in most designs. Straight bar splices using lapped bars or a mechanical connectors could also be used.

- The load and moment caused by removal of the temporary support must be recomputed because the reactions would be different.

There are no differences in the design for subsequent construction stages, other than considering the revised stresses.

Moment diagrams for the girder segments in this design example are shown in **Figure 15.2.2-1** with and without girder continuity at deck placement. The reduction in moment at midspan of the center girder segment is significant. Concrete stresses at the bottom of the girder for the Service III limit state load combination with and without girder continuity at deck placement are shown in **Figure 15.2.2-2**. The reduction in stress at midspan of the bridge is approximately 0.239 ksi, and the reduction at the splices is approximately 0.205 ksi. This figure clearly indicates that the creation of a simple continuity connection at the splices prior to deck placement results in a significant reduction in stresses at the bottom of the girder. This reduced stress could result in a significant reduction in the required post-tensioning.

Single Span Spliced PCI BT-96 Girder   **Design Example 1**
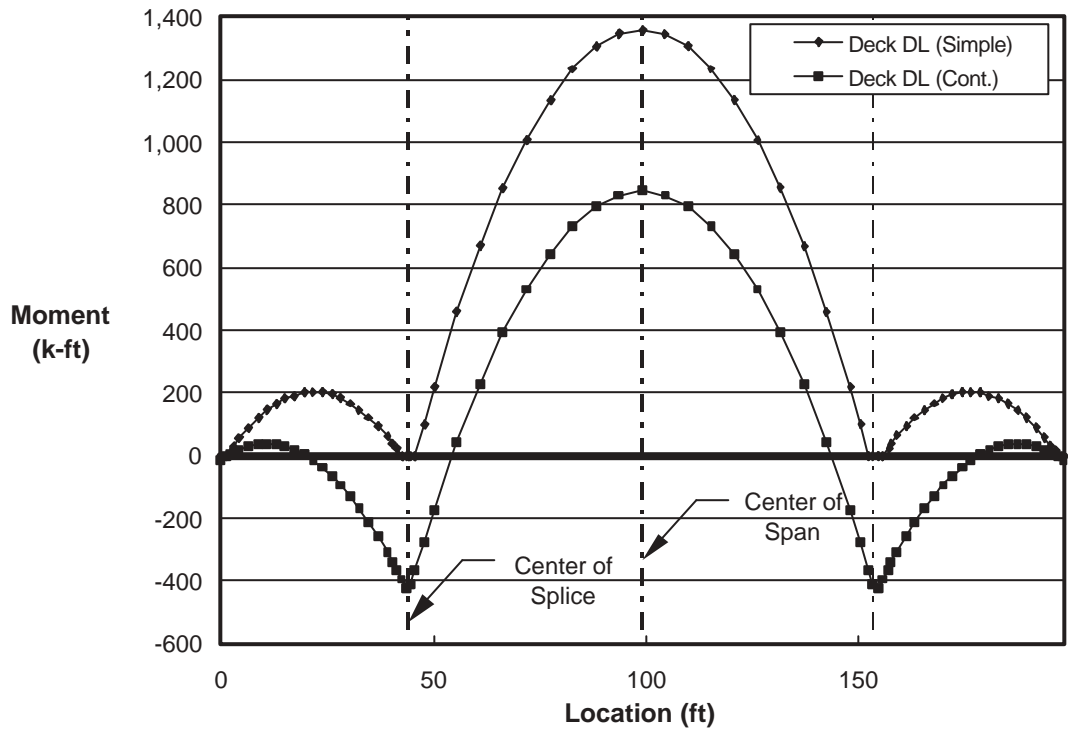
Figure 15.2-1   Deck Dead Load Moments with and without Continuity
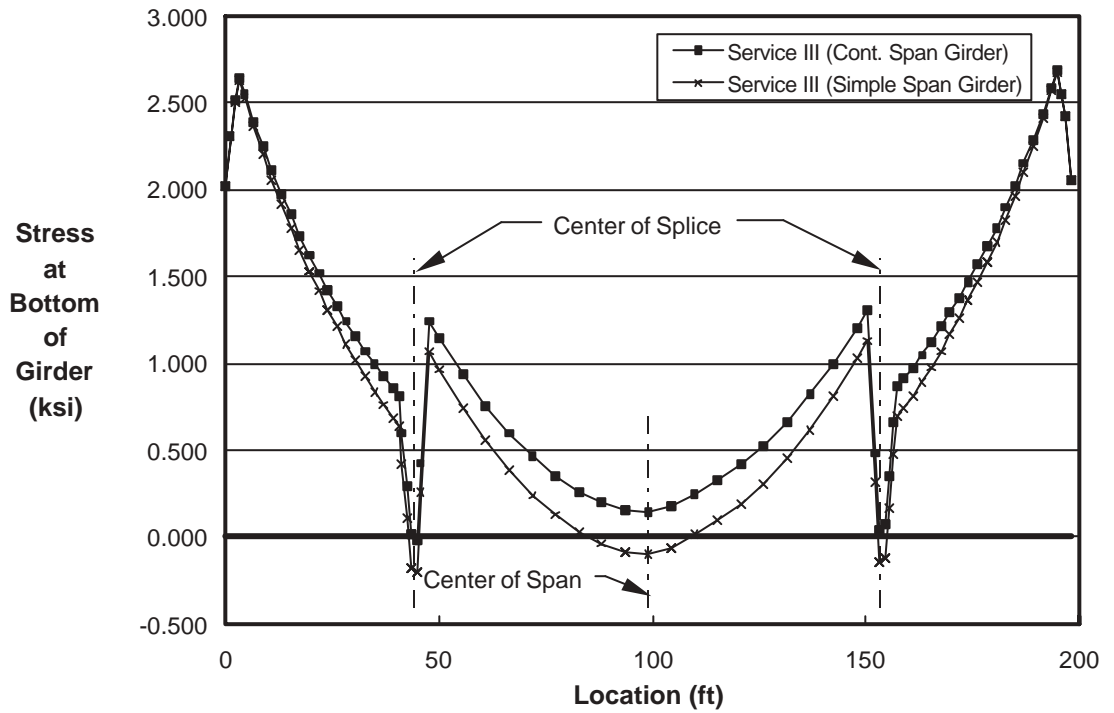
Figure 15.2-2   Bottom of Girder Concrete Stresses at Service III with and without Continuity

15.3 **STAGED CONSTRUCTION**

The design of spliced concrete girders at sites where staged construction is required has been successfully performed. The bridge should be detailed to provide adequate tolerance for differences in camber in girders or in the completed bridge, since post-tensioning is applied to the bridge with the deck in place. Considerations are essentially the same as for conventional long-span bridges.

The designer should consider using wider closure pours between stages of construction to reduce the effect of any difference in deck profile. The cover on the deck may also be increased slightly to provide a sacrificial surface for planing of the deck surface to achieve the desired final deck profile.

Any connections between girders in different stages should be delayed or properly detailed to allow relative movement of girders when post-tensioning is applied. Both shortening and camber effects should be considered.

15.4 **PROVISIONS FOR DECK DURABILITY**

In some regions, bridges are designed for the complete removal and replacement of decks in anticipation of potential deterioration of the deck. This presents difficulties for post-tensioned bridges where some or all of the post-tensioning is applied to the composite section, such as the bridge in this design example.

For the bridge in this design example, an alternate approach would be used to address the deck durability and replacement issue, rather than planning on the eventual complete replacement of the deck. Alternate strategies to addressing deck durability employ one or more methods that address the corrosion of deck reinforcement. These methods include:

- Use of high performance concrete in the deck, which has a significantly reduced permeability to the intrusion of chlorides and moisture

- Use of a sacrificial concrete overlay that protects the deck and can be removed and replaced when chloride levels become elevated

- Use of coated reinforcement or alternate reinforcement materials, which reduces, delays or eliminates the potential for corrosion

- Use of corrosion inhibiting admixtures in the deck concrete

It should be noted that applying post-tensioning to the composite section, including the deck, which reduces or eliminates deck cracking. The elimination of deck cracks, in conjunction with other methods of addressing corrosion mentioned above, provide a good strategy for improving the durability of the bridge deck.

A complete discussion of this topic is beyond the scope of this present discussion. A more complete discussion is presented elsewhere in the Final Report.

15.5 **POST-TENSIONING ANCHORAGES PLACED IN BACKWALL OR ABUTMENT**

Post-tensioning anchorages may be placed in the backwall of abutments to simplify girder fabrication. This eliminates the need for end blocks in the end of girders to house the post-tensioning anchorages. This can speed the fabrication of girders.

However, some contractors prefer having the placement of the post-tensioning anchorage done under the more controlled environment of a prestress concrete manufacturing plant. The placement of anchorages in the field will also slow the work of constructing the backwall.

Since there do not appear to be decisive advantages or disadvantages to lead designers to use only one method, designers should be open to allowing either approach when proposed by a post-tensioning contractor. Most spliced girder bridges have been constructed with anchorages installed in the girders.

15.6   **INTEGRAL ABUTMENTS**

Spliced girder bridges may utilize integral abutments if properly detailed to allow shortening of the girders during post-tensioning operations. To achieve this, the connection between the girders and the foundation should not be made until after post-tensioning is completed. The connection could be made when the anchorage encasement is placed.

One drawback for the use of integral abutments for spliced girder bridges is the lack of access to inspect the anchorage areas when they are encased in a backwall that is under an approach slab.

15.7   **SINGLE SPLICE AT OR NEAR MIDSPAN**

A splice at or near midspan will be subjected to high stresses from applied loads. The limiting tensile stresses for the splice are less than the tensile stress limits for the precast elements. Therefore, the splice will generally govern the design. A higher concrete strength may be used in the splice to increase the stress limits, but this only provides a significant benefit if compression stresses are governing.

One advantage to having the splice at or near midspan is that the splice occurs where all tendons are probably horizontal and in their lowest position, which maximizes the effect of the post-tensioning. A spliced girder can certainly be successfully designed with a splice at midspan (see **Design Example 2**).

A design issue that arises with midspan splices is where to compute the losses for the pretensioned strands, since the maximum moment location at midspan is not pretensioned. It is suggested that (the splice) ;losses be computed at the center of each girder segment.

15.8   **ASYMMETRICAL SPLICE LOCATIONS**

There are no special criteria for design of spliced girders with asymmetrical splice locations. Moments from pier removal will be unbalanced when splices are not symmetrically placed. However, this can be addressed in the design process.

15.9   **SKEW**

The main issue that must be considered with skewed spliced girder bridges is how to deal with diaphragms at the splices and abutments. The diaphragm at the splices may follow the skew if the ends of the bridge are parallel. This would keep the segments the same length and the tendon profiles the same in each girder line. If the skews are not equal, the girder lengths will differ.

15.10   **ALTERNATE GIRDER SECTIONS**

The modified PCI bulb tee girder used for this bridge provided a reasonable solution. The selection of the section was based on the assumed local use of other girders in the PCI bulb tee family. While this may be an economical solution in some cases, the use of a different girder section may provide a better solution.

The PCI bulb tee has a relatively small bottom flange, which did not pose a problem for space for enough pretensioned strands. However, the small bottom flange may limit the capability of the girder because of a smaller stiffness for the section. A section with a larger bottom flange, such as the New England bulb tee, Nebraska bulb tee or other new sections, may provide a better solution if the section is available. Tub shaped girders, such as used in **Design Example 2**, can also be used for this type of structure.

Therefore, it is suggested that the preliminary design of a spliced girder bridge should consider other sections that may be available in the local or nearby area. Owners and local prestressed concrete producers should be consulted when considering the use of new girder sections.

15.11   HIGH STRENGTH CONCRETE

High strength concrete is not generally required for spliced girders because the splice locations tend to govern the designs at the service limit state. However, high strength concrete, or high performance concrete, can have improved creep and shrinkage properties that may be beneficial for spliced girder bridges. These improved properties can reduce deflections as well as creep and shrinkage losses.

16   REFERENCES

1. *AASHTO LRFD Bridge Design Specifications*, 2nd Edition with Interims, American Association of State Highway and Transportation Officials. Washington, DC. (1998-2003).
2. *PCI Bridge Design Manual*, First Edition, Precast/Prestressed Concrete Institute, Chicago, IL (1997).
3. *"Guide Design Specifications for Bridge Temporary Works"*, American Association of State Highway and Transportation Officials. Washington, DC. (1995).
4. *"Guide Specifications for Design and Construction of Segmental Concrete Bridges"*, 2nd Edition, American Association of State Highway and Transportation Officials. Washington, DC. (1999).
5. Martin, L. D., "A Rational Method for Estimating Camber and Deflection of Precast Prestressed Members." PCI Journal, Vol. 22, No. 1. (January-February 1977) pp. 100-108.
6. Libby, J. R., *Modern Prestress Concrete*, 3rd Edition Van Nostrand Reinhold Company, NY (1984).
7. Post-Tensioning Institute, "Anchorage Zone Design" Phoenix, AZ (October 2000).
8. VSL International Ltd., "Detailing For Post-Tensioning", Bern, Switzerland (1991).

DESIGN EXAMPLE 2: TWO-SPAN SPLICED U-BEAM GIRDER

Table of Contents

Two-Span Spliced U-Beam Girder   **Design Example 2**

1   INTRODUCTION

The bridge in this example represents a structure carrying a multi-lane cross street over an interstate. An integral cap is used at the interior pier, which is an advantage for sites with significant seismicity, as discussed in **Section 11** of this example. The second splice location and asymmetrical girder segments in one span are included to represent a real-world condition where maintenance of traffic requirements during construction limit placement of temporary supports. The bridge is also skewed to illustrate another condition often found in bridges of this type. The girders are spliced over the interior pier, where the segments terminate a short distance (6 in.) beyond the face of the bent cap.

An elastic design is presented to illustrate the effects of staging prestress and gravity load applications. Prestress forces and compressive strength requirements are developed from this analysis at service limit states. Safety of the structure is evaluated at strength limit states.

Shear design is presented to illustrate the effects of splicing on the shear response. Of particular interest is the interface shear capacity at the bent caps.

The emphasis of this example is on issues related to two-span spliced girder bridges, which are not covered in **Design Example 1**. In order to prevent repetition, this example does not cover as much detail related to the basics of spliced girder design as **Design Example 1**. Where applicable, appropriate sections of **Design Example 1** will be referenced for further details.

2   DESCRIPTION OF BRIDGE

As shown in **Figure 2-1** and **2**, the location of the existing traffic lanes and orientation of the roadway improvements dictate splice layout, bent location and skew of the bridge. The new structure must accommodate future widening of both the southbound and northbound lanes of Route 78 to the east, thus resulting in unsymmetrical span lengths.
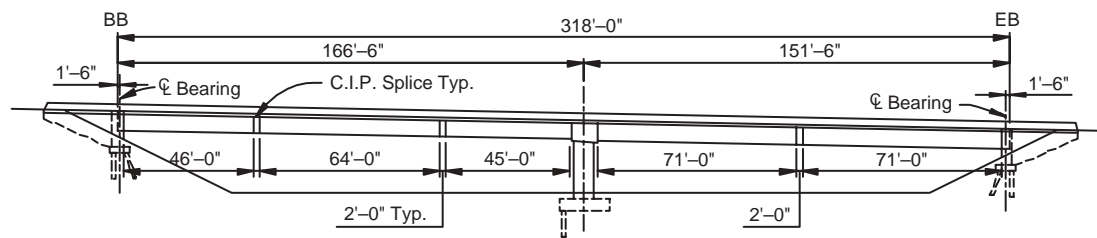

Figure 2-1   Elevation

As shown in **Figure 2-3**, the bridge is 70.0 ft wide with a constant cross-slope of 2%. The girders are spaced at 14.0 ft on center and are set with the soffit parallel to the deck cross-slope. This orientation allows the web widths and associated stirrups to have equal lengths, thus simplifying girder segment construction. The girder segments terminate a short distance into the cast-in-place bent cap and at the cast-in-place end diaphragms. This termination allows the post-tension anchorages to be located in the cast-in-place diaphragms. Also, this termination allows level bearing surfaces to be field constructed with greased galvanized bearing pads that allow initial prestress shortening displacements without a significant amount of temporary bracing. The bent cap is 9 in. deeper than the superstructure to improve constructability and seismic performance.

D2-1

Two-Span Spliced U-Beam Girder **Design Example 2**
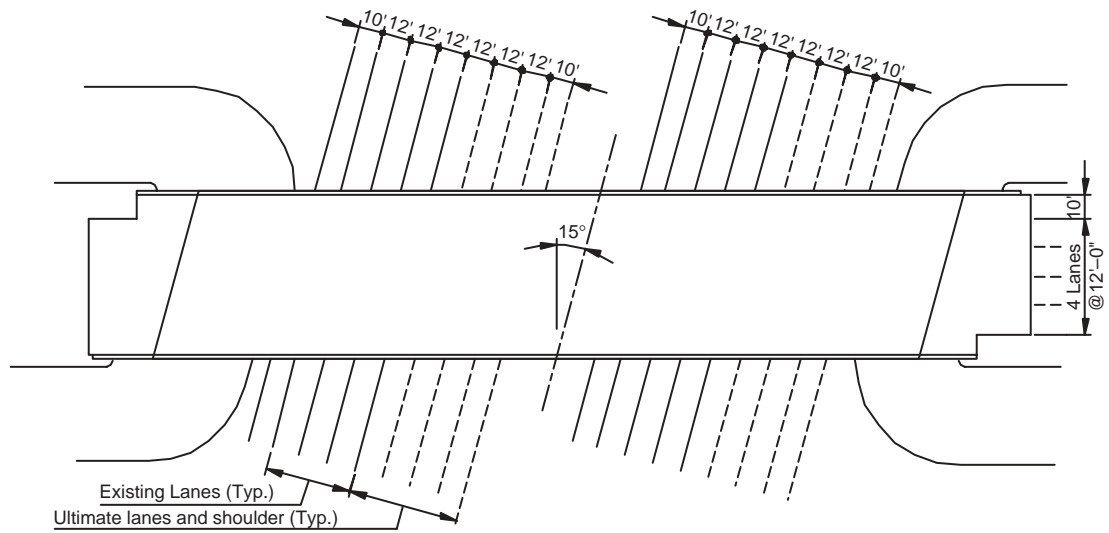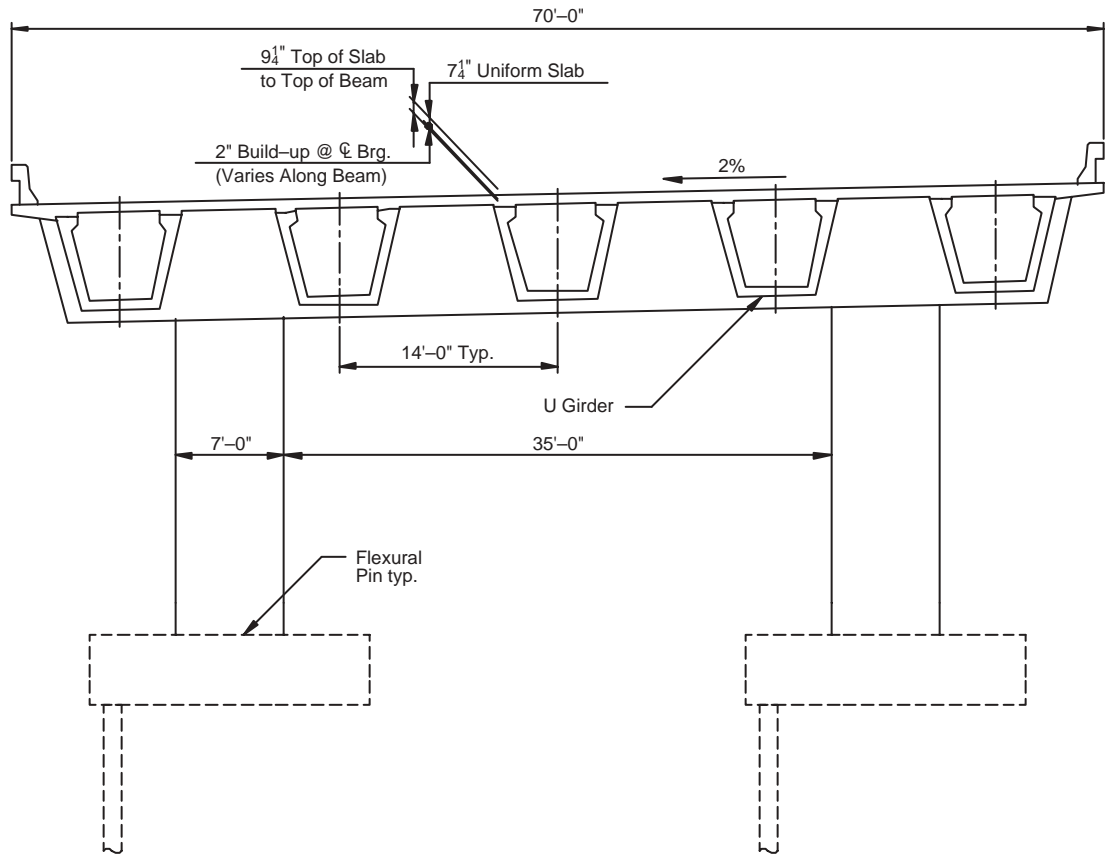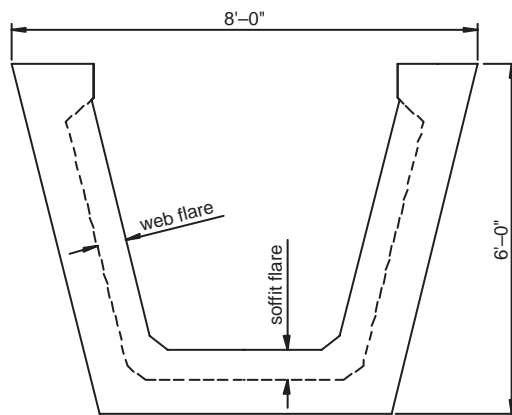
Figure 2-2   Plan View of Bridge

The two 7 ft circular columns supporting the bent cap are pinned at the footing in both the longitudinal and transverse directions. Ideally, this pin transfers column axial load and shear, but no moment to the footing, and all lateral resistance is provided by frame action. The primary benefit of using a flexural pin at the column base is the significantly reduced foundation size over a fixed base detail, because the footings do not have to resist the overstrength moments due to plastic hinging. However, the superstructure and bent cap must be designed to resist these plastic hinge moments. The structural design and detailing aspects of developing these moments are the subject of **Section 11**.

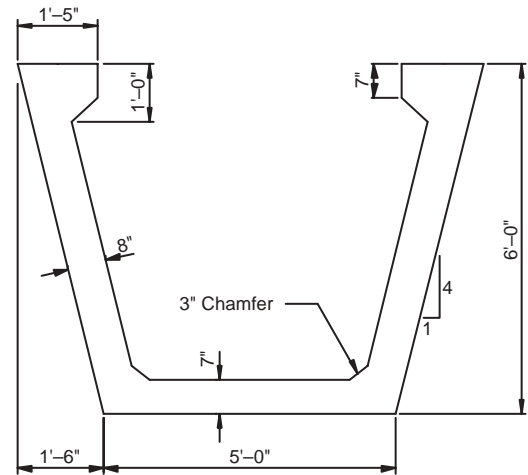**Two-Span Spliced U-Beam Girder**   **Design Example 2**

70'–0"

$9\frac{1}{4}$" Top of Slab to Top of Beam

$7\frac{1}{4}$" Uniform Slab

2" Build–up @ ℄ Brg. (Varies Along Beam)

2%

14'–0" Typ.

U Girder

7'–0"

35'–0"

Flexural Pin typ.

2-3a TYPICAL SECTION

8'–0"

web flare

soffit flare

6'–0"

Note: Web and soffit flare vary from 0 to 6" over 12'–0" from face–of–cap

2-3b AT END

1'–5"

1'–0"

7"

8"

3" Chamfer

7"

4

1

6'–0"

1'–6"

5'–0"

2-3c TYPICAL

U-BEAM CROSS SECTIONS

Figure 2-3   **Bridge Section**

D2-3

3   DESIGN ASSUMPTIONS

In this example, there are five segments per girder line. The center segment of Span 1 and all of the segments in Span 2 are pretensioned. After transfer, the girder segments making up a span are placed on falsework towers, as shown in **Figure 3-1**.
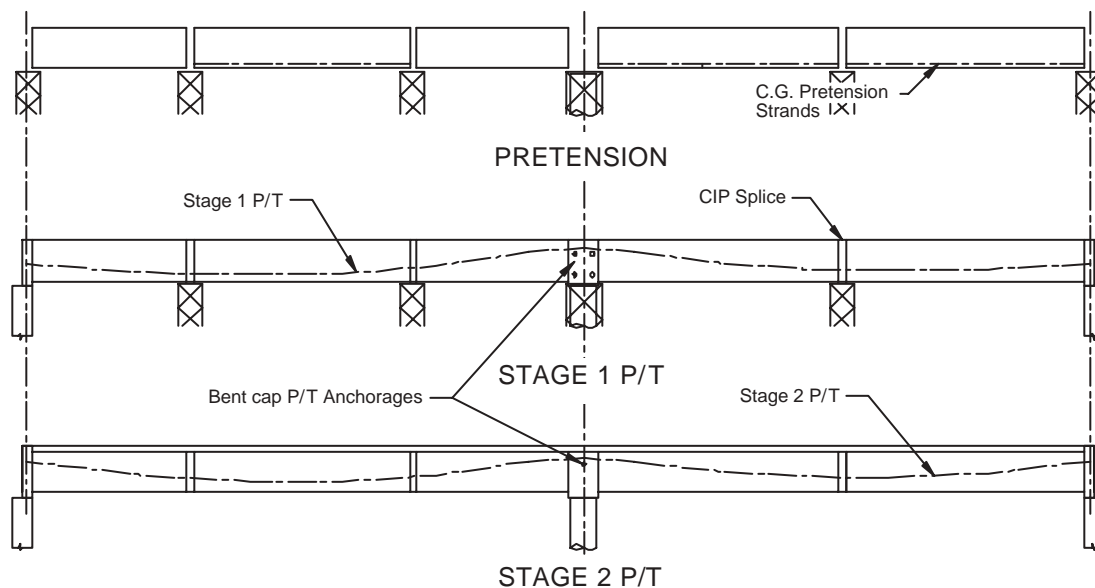

Figure 3-1   Construction Sequence

Post-tensioning is applied in two stages. After casting the bent caps, the splices and end-diaphragms, the girder segments are made continuous by applying the first stage of post-tensioning to both the girders and the bent cap, where post-tensioning is first applied to the bent cap and then the girders. This first stage of post-tensioning (Stage 1 P/T) resists moments and forces developed from releasing the falsework towers, the fluid weight of the deck and associated construction loads. After application of Stage 1 P/T, the falsework towers are removed completely (including the falsework for Bent 2) and the deck can be constructed from the girders and bent caps.

Second stage post-tensioning (Stage 2 P/T) is intended limit the tensile stress in the concrete deck below cracking levels at negative bending regions over the bent. Therefore, the use of gross section properties is correct. This post-tensioning is applied to both the superstructure and bent cap after the deck has reached the specified strength and age requirements. In this stage, post-tensioning of the bent cap can occur either before, or after stressing the girders.

The primary benefits of this construction staging are that the shoring towers only have to support the weight of the girders, and the deck remains crack free under service loading. These advantages are at the cost of an additional construction step of applying post-tensioning twice to the structure. A discussion on alternative construction staging is presented in **Section 13**.

CONSTRUCTION SCHEDULE

3.1

Table 3.1-1 lists the assumed construction schedule used for this example, with the age in days representing an average age for all girders in the bridge.

Table 3.1-1   Assumed Construction Schedule

#	Days	Event	Loads
1	1	Transfer Pretension	Girder self weight
2	30	Set Girders on Falsework	Girder self weight
3	50	Pour splice, diaphragms	N/A
4	60	Stage 1 P/T	Falsework release forces
5	80	Pour CIP Deck	Fluid weight of deck
6	90	Stage 2 P/T	N/A
7	110	Place barriers	Traffic barriers
8	120	Open to full traffic	1.29 lanes of live load per girder
9	27,375	Service	Future wearing surface

The *Specifications* and special provisions typically have no restrictions on the transfer of the pretension force, which is typically based on strength requirements. In contrast, cast-in-place post-tensioned structures often have a minimum time restriction required before prestressing can take place. For example, the California Department of Transportation (Caltrans) requires 10 days and a minimum of 3.5 ksi compressive strength before applying post-tensioning to cast-in-place concrete *(1)*. These requirements ensure that concrete behind the post-tensioning anchorages has developed sufficient strength prior to stressing.

3.2

ANALYSIS METHODS

For all service limit states, analysis is performed using gross, uncracked section properties, and loads and stress are superimposed from various load cases. Moments, shears, and axial loads used to evaluate the strength limit state are based on elastic properties. All permanent load calculations were performed using "hand methods" of analysis. Spreadsheets were used to perform repetitive calculations.

The bearing pads supporting the reactions at each abutment are assumed to perform as perfect rollers, and the pin at the base of the column is assumed to provide no rotational restraint. It is assumed that the structure can sway for all load cases, which implies there are no flexure and shear loading acting on the columns from dead load and live load. Therefore, the support at Bent 2 is assumed to be a perfect pin, and the superstructure is modeled as continuous beam supported on rollers at the exterior supports and a pin at the interior support. It should be noted that the pin at the base of the column is not perfect, and, while some moments develop at the base, they are not significant enough to warrant consideration.

In this analysis model, the loads are transferred to a "knife-edge" support at Bent 2, with the section properties of the superstructure extending to the center of Bent 2. In reality, the superstructure frames into a 9 ft wide by 7.5 ft deep bent cap that is supported by two-7 ft diameter columns. In design of the superstructure, the maximum design negative moments are assumed to occur at the face of the bent cap (4.5 ft from the centerline of Bent 2). These assumptions are discussed in further detail in references *(2, 3)*.

The structure is statically indeterminate, with continuity over Bent 2. Prestress restraint moments (secondary moments) are calculated using the moment area method. A detailed description of the secondary moment calculations is presented in **Section 7**.

4   PRELIMINARY DESIGN

Since there is no nationwide standard for U-beam girders, and the number of bridges incorporating the shape is relatively small, preliminary design and development of section dimensions is presented in this section. Although design of a U-beam girder bridge is similar to a precast I or bulb-tee, there are differences, which include placement on cross-slope, increased torsional capacity, improved structural efficiency and heavier hauling. Additional considerations for construction are presented.

4.1   STRUCTURAL DIMENSIONS

Although the overall bridge length is 318 ft measured from beginning to end of bridge, the bridge rests on bearing pads at the centerline of the abutments a distance of 1'-6" from the ends. Subtracting this distance results in a 165 ft length for Span 1 and 150 ft for Span 2.

The depth to span ratio (D/S) of the bridge is based on the value recommended in LRFD Table 2.5.2.6.3-1 for continuous precast girder bridges. Using a 6 ft deep girder with a 7.25 in. deck and a 2 in. build-up, the overall D/S ratio is 0.041, slightly greater than 0.040, as recommended for continuous precast I-beam bridges.

4.2   GIRDER SPLICES

The girders must initially span the existing lanes with two feet of distance between the edge of the traffic lane and the end of the girder to accommodate temporary traffic barriers and any additional room for falsework supports. In Span 1, a 64 ft long segment spans the active traffic lanes. A single 110 ft long segment could span these lanes from Abutment 1 and not exceed length limitations for most states. However, the weight of this segment would be over 95 tons, and specialized hauling and lifting equipment would be required slowing delivery and increasing cost (70 tons seems to be the upper limit for use of standard lifting and hauling equipment *(2)*). In Span 2, only two girder segments are required, and in order to minimize segment size, a mid-span splice was chosen. While minimizing the segment sizes, this configuration is the least favorable location in terms of minimizing post-tensioning, because the splice section, which does not benefit from the pretension strand is near the location of maximum positive moment within the span. However, this section is not likely to control the design because the span length is significantly less than Span 1.

The girder splices are 2 ft long to accommodate duct splices and development of longitudinal reinforcement. Although the width of the splice could be less, 2 ft allows for any potential misalignment of the prestress ducts.

4.3   GIRDER DIMENSIONS

The girder sections, shown in **Figure 2-3**, depict the same girder in two different locations. The typical section dimensions were adopted *(4)* with hard conversion from metric to US customary units. A 12 ft flare section at the girder-bent cap interface is provided to increase shear capacity, eccentricity of the post tensioning and reduce service load compressive stress. The prestress eccentricity is improved by lowering the center of gravity of the girder and raising the prestress profile, as illustrated in **Figure 4.3-1**. As shown, the post-tensioning throughout most of the length is stacked in a row along the web centerline, as shown in **Figure 4.3-1a**. At the flare section, ducts can spread horizontally until a configuration is developed, like the one shown in **Figure 4.3-1b**, where the ducts are placed side-by-side as they enter the bent cap.

(a) Midspan   (b) Face of Bent Cap   (c) Face of End Diaphragm
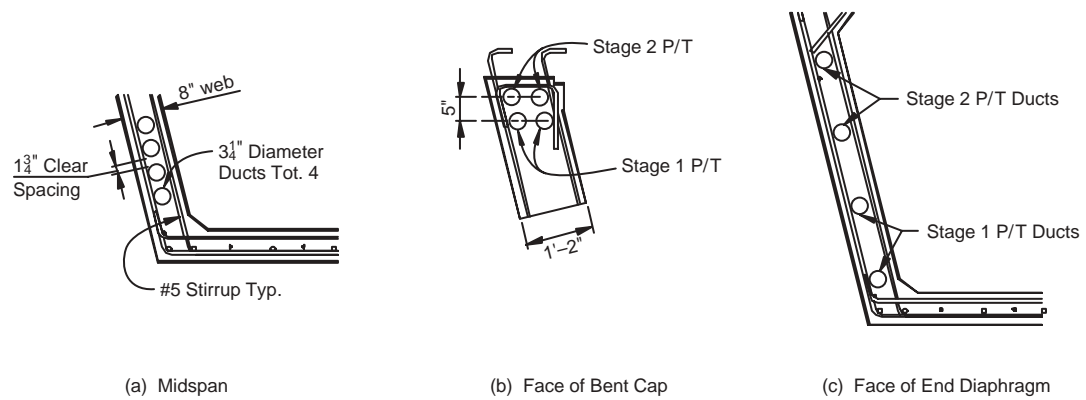
Figure 4.3-1   Post-Tensioning Duct Layout

It is assumed that standard flare forms for U-beam girders are available with the 12 ft flare starting and ending normal to the girder centerline. Hence, because of the skew, the flare for each web within the girder have unequal lengths, as shown in **Figure 4.3-2**. This figure also shows that the acute corner of the girder has been eliminated, as this corner typically spalls off during transfer of the pretension force. To further simplify the bulkhead forms, both webs terminate normal to the girder centerline, and the skew is taken up in the soffit.

Figure 4.3-2   Girder End-Diaphragm Detail

4.4   DIAPHRAGMS

In an effort to minimize girder segment lengths, the precast girder segments extend a short distance (6 in.) into the end diaphragms, as shown in **Figure 4.4-1**. This configuration requires post-tensioning anchorages to be located in cast-in-place end diaphragms, which has the primary benefit of not requiring flares or anchorage reinforcement in the girder webs. Further, the cast-in-place diaphragms allow for casting of a level bearing surface for the elastomeric pads, considering the soffit cross-slope and roadway profile and skew. Also, the end-diaphragms provide stability for the girders during stressing.
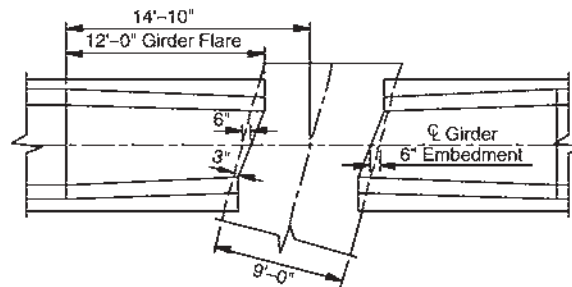
Figure 4.4-1   Girder Flare Detail

The primary drawback to termination of the girder segments is the increase in construction schedule, as cast-in-place concrete can have both time and strength requirement prior to applying post-tensioning. As an example, the California Department of Transportation requires a minimum of 10 days and 3.5 ksi strength prior to post-tensioning cast-in-place concrete *(1)*. Furthermore, the end diaphragm has to be relatively wide to accommodate the post-tensioning anchors.

Since the U-beam section can be set on an inclined surface and not tip over, and after the deck is poured is stiff under torsion loads, intermediate diaphragms are not required. A discussion on the need for girder diaphragms for handling considerations is presented in **Section 12.2**.

5   DESIGN PARAMETERS

5.1   SPECIFICATIONS

AASHTO LRFD Bridge Design Specifications, 2nd Edition with Interims through 2003 *(5)*. References to articles, equations and tables in these *Specifications* will be preceded by the prefix "LRFD" to differentiate these references from references to items in this design example.

Proposed revisions recommended as part of this research project (see **Appendix E**). References to articles and equations in the proposed *Specifications* will be preceded by the prefix *"Proposed"* to differentiate these references from references to items in the *AASHTO LRFD Specifications*.

For design and details concerning the development of superstructure-column continuity for severe seismic loads, design recommendations developed by the California Department of Transportation and others are used *(6, 7, 8, 9)*.

5.2   LOADS

5.2.1   Dead Load

The following dead loads of the structure are based on a concrete unit weight of w_c = 150 pcf.

Girder net section: 1.72 k/ft

An impact factor of 1.5 is applied to the girder to account for dynamic effects encountered during shipment and placement of girder segments (see LRFD Article 5.14.2.1).

Girder at face of cap: 2.66 k/ft

Deck and build-up: 1.34 k/ft

There are no diaphragms within the spans of the bridge, the splice has the same cross-section as the girders. Therefore, no additional weight is included for splices or diaphragms.

5.2.2   **Superimposed Dead Load (SDL)**

These loads are listed as follows:

Future wearing surface: 0.035 ksf x 14 ft of width = 0.490 k/ft

Traffic barrier: 0.380 k/ft per barrier (total 2) Assumed to be supported by exterior girder only

5.2.3   **Live Load (LL)**

Live load distribution factors are computed using equations in LRFD Table 4.6.2.2.2b-1 for Section Type (C) (see LRFD Table 4.6.2.2.1-1). Since the girder spacing of 14 ft is greater than 11.5 ft, which exceeds the range of applicability for this girder, the approximate distribution factors do not apply. Hence, the lever rule is recommended, where the deck is modeled as a simply supported beam between girders.

Interior Girder

	One Design Lane Loaded	0.94 lanes / girder
	Two or More Design Lanes	1.29 lanes / girder

Exterior Girder

	One Design Lane Loaded	1.22 lanes / girder
	Two or More Design Lanes	1.21 lanes / girder

Dynamic impact of 33% is applied to the truck load.

5.2.4   **Thermal Gradient**

An evaluation of thermal gradients is not included in the analysis.

5.2.5   **Load Combinations**

The design satisfies the following service level load combinations:

$$D + LL + P/S \qquad \text{SERVICE I}$$
$$D + 0.8LL + P/S \qquad \text{SERVICE III}$$

Service I load combination is used to evaluate all components and parameters not related to the evaluation of tensile stress. Service III load combination relates to evaluating tensile stress in design of the prestress tendons.

The following strength limit state load combinations are evaluated in **Section 8.2**:

$$\gamma_p D + 1.75LL + P/S \qquad \text{STRENGTH I}$$
$$\gamma_p D + L + P/S + EQ \qquad \text{EXTREME I}$$

where γ_p is the load factor for permanent loads. ($\gamma_p = 1.25$ for self weight of the girders and deck, and $\gamma_p = 1.50$ for future wearing surface and utilities).

5.2.6 **Seismic Loads**

Derivation of seismic loads is beyond the scope of this report. However, the distribution of overstrength moments from plastic hinging of the column into the superstructure is presented in **Section 11**.

5.2.7 **Material Properties**

The material properties used in the analysis and design are listed in **Table 5.3-1**.

Table 5.3-1   **Material Properties**

Girder concrete			
28 day strength	f_c'	6,000 psi
Transfer	f_{ci}'	5,000 psi
Elastic modulus*	E	4,890 ksi
Unit weight	γ	150 pcf
Cast-in-place concrete			
28 day strength	f_c'	4,000 psi
Transfer	f_{ci}'	3,500 psi
Elastic modulus*	E	3,600 ksi
Unit weight	γ	150 pcf
Modular ratio	n	0.82**
Prestressed steel			
Tensile strength	f_{su}	270 ksi
Elastic modulus	E_s	28,500 ksi

  * Elastic modulus at 28 Days.
  **For computing composite section properties. Cast-in-place concrete includes
     deck, build-up, bent cap, end diaphragms and splices.

5.3 **STRESS LIMITS**

5.3.1 **Final Service Conditions**

Allowable concrete tension stresses at service loads are limited to $3\sqrt{f_c'}$ (psi) at the joints and $6\sqrt{f_c'}$ (psi) elsewhere, where fully bonded reinforcement is required to resist the tensile forces at a stress of $0.5f_y$ (LRFD Table 5.9.4.2-1). These stress limits are applicable to the following Service III limit state:

$$D + P/S + 0.8L$$

for the purpose of computing the required prestress forces. Allowable compression stresses at service limit states are limited to the following (from LRFD Table 5.9.4.2.1-1):

$D + P/S + L$:	$0.60f_c'$
$D + P/S$:	$0.45f_c'$
$(D + P/S)/2 + L$:	$0.40f_c'$

5.3.2   **Temporary Service Limit States**

A temporary service limit state is any condition occurring prior to developing full prestress losses. The allowable tension stress is limited to $6\sqrt{f'_{ci}}$ in the precast segments and $3\sqrt{f'_{ci}}$ at the splices, with bonded reinforcement to resist the tensile force in concrete computed on the assumption of an uncracked section at a stress of $0.5f_y$, as per LRFD Table 5.9.4.1.2-1.

The allowable compression stress is $0.60f'_{ci}$ for the deck, splices, bent cap and end diaphragms and $0.6f'_c$ for the girders (LRFD 5.9.4.1.1).

5.4   **SECTION PROPERTIES**

The section properties computed in **Table 5.4-1** are based on the material properties listed in **Section 5.3-1**. The distances y_b and y_t are the distance from the section center of gravity to the top and bottom fiber of the girder. The section modulus S_{deck} is used to compute the stress at the top of the deck in the composite section.

Table 5.4-1   **Computed Section Properties**

Property	Units	Non-Composite Girder		Composite Girder	
		Midspan	*Face of Cap*	*Midspan*	*Face of Cap*
A	ft^2	11.48	17.74	18.77	25.02
Y_b	ft	2.69	2.59	4.15	3.71
Y_t	ft	3.31	3.41	2.62	3.06
I	ft^4	46.86	60.39	110.04	137.61
S_b	ft^3	17.41	23.32	26.52	37.07
S_t	ft^3	14.16	17.70	59.49	60.16
S_{deck}	ft^3	------	------	41.99	44.99

5.5   **DESIGN MOMENTS**

The moments profiles acting on the non-composite section shown in **Figure 5.5-1** include the segment self-weight moments spanning between falsework supports, the falsework-release moments and the moments from fluid weight of the deck. The falsework-release moments are calculated by applying equal and opposite falsework reactions to the girder including the additional weight of the girder splices. The moment at Bent 2 is calculated by first releasing continuity at the support and calculating the relative rotation of the two spans by integrating curvature (M/EI). The moment required to bring the relative rotation back to zero is the negative moment due to falsework release. The fluid deck weight moments were calculated using a similar procedure.

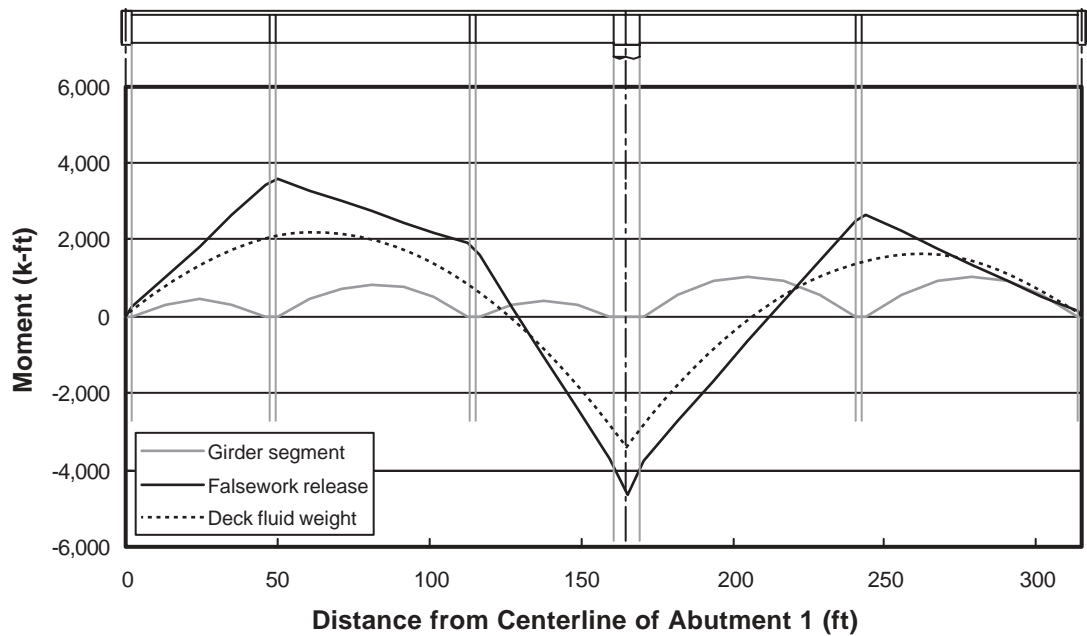**Two-Span Spliced U-Beam Girder**   **Design Example 2**

Figure 5.5-1   Dead Load Moments on Non-Composite Section

Live load moments for one lane of traffic are shown in **Figure 5.5-2**. For comparison purposes, the HL-93 loading used for this example is plotted with an HS-20 live load lane using the *AASHTO Standard Specifications for Highway Bridges (10)*. As shown, there is a considerable difference between the two *Specifications*. For positive bending, the HL-93 moments are 70% greater than the HS-20 moments, and for negative bending, the moments are 50% greater.
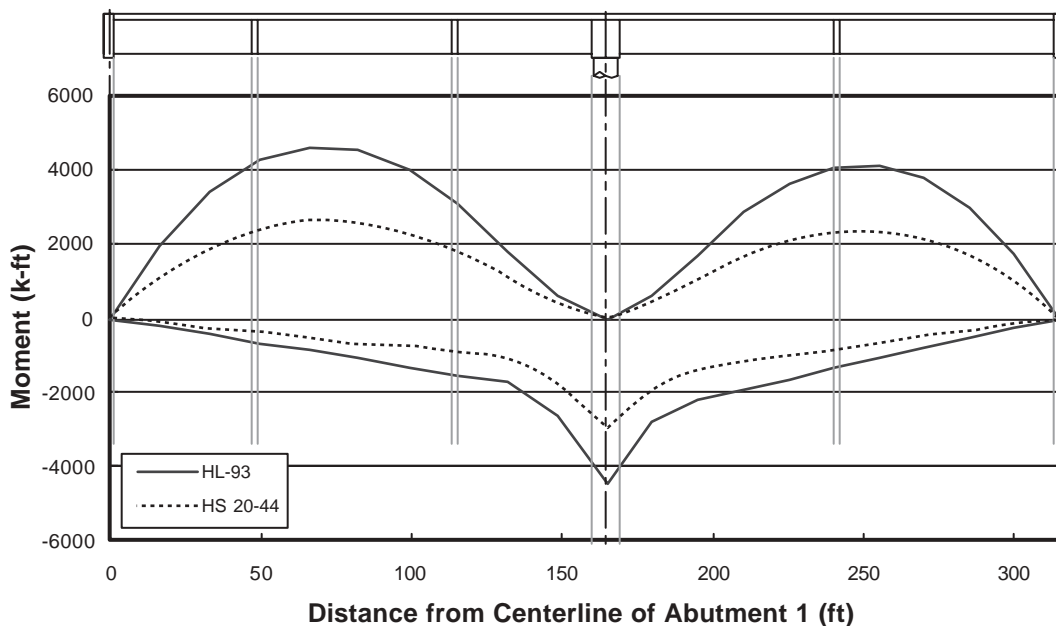

Figure 5.5-2   Live Load Moments

6   PRESTRESS DATA AND LAYOUT

A total of three intervals of prestress are applied to the structure including pretensioning, Stage 1 P/T and Stage 2 P/T, as mentioned previously. Since design is a trial and error procedure, an initial estimate of these forces must be made to determine clearances, and define the prestress profile.

6.1   PRETENSIONED STRAND

For pretensioning, the strand is centered vertically in the bottom slab of the girder. With this information and assumed prestress loss, the pretensioning force is calculated directly. Because some of the girder segments in Span 1 are so short, prestress is not required for transportation and handling of those respective segments.

6.2   POST-TENSIONING

The California Department of Transportation has developed charts for estimating the amount of post-tensioning required for cast-in-place box girder structures. The chart *(3)*, shown in **Figure 6.2-1**, was used to develop the initial post-tensioning forces. With the Span 1 length of 165 ft and D/S equal to 0.04 the amount of prestress steel is 4.3 lb/ft2 of deck area. Hence, the total number of strands N for both stages of post-tensioning is calculated using the following:

$$N = (4.3 \text{ lb/ft}^2)(14 \text{ ft spacing}) / ((490 \text{ lb/ft}^3)(0.217 \text{ in}^2/\text{strand})(1\text{ft}^2/144 \text{ in}^2))$$

$$= 81.5 \text{ Strands} \quad \text{Say 80 Strands}$$

Using a total of 8 prestress tendons per girder (4 per web), with a total of 10 strands per tendon. A 12 - 0.6 in. diameter strand tendon is a common size which can be used with 3.25 in (outer diameter) ducts. This duct is the largest diameter that can be used in an 8 in. web, as per LRFD Article 5.4.6.

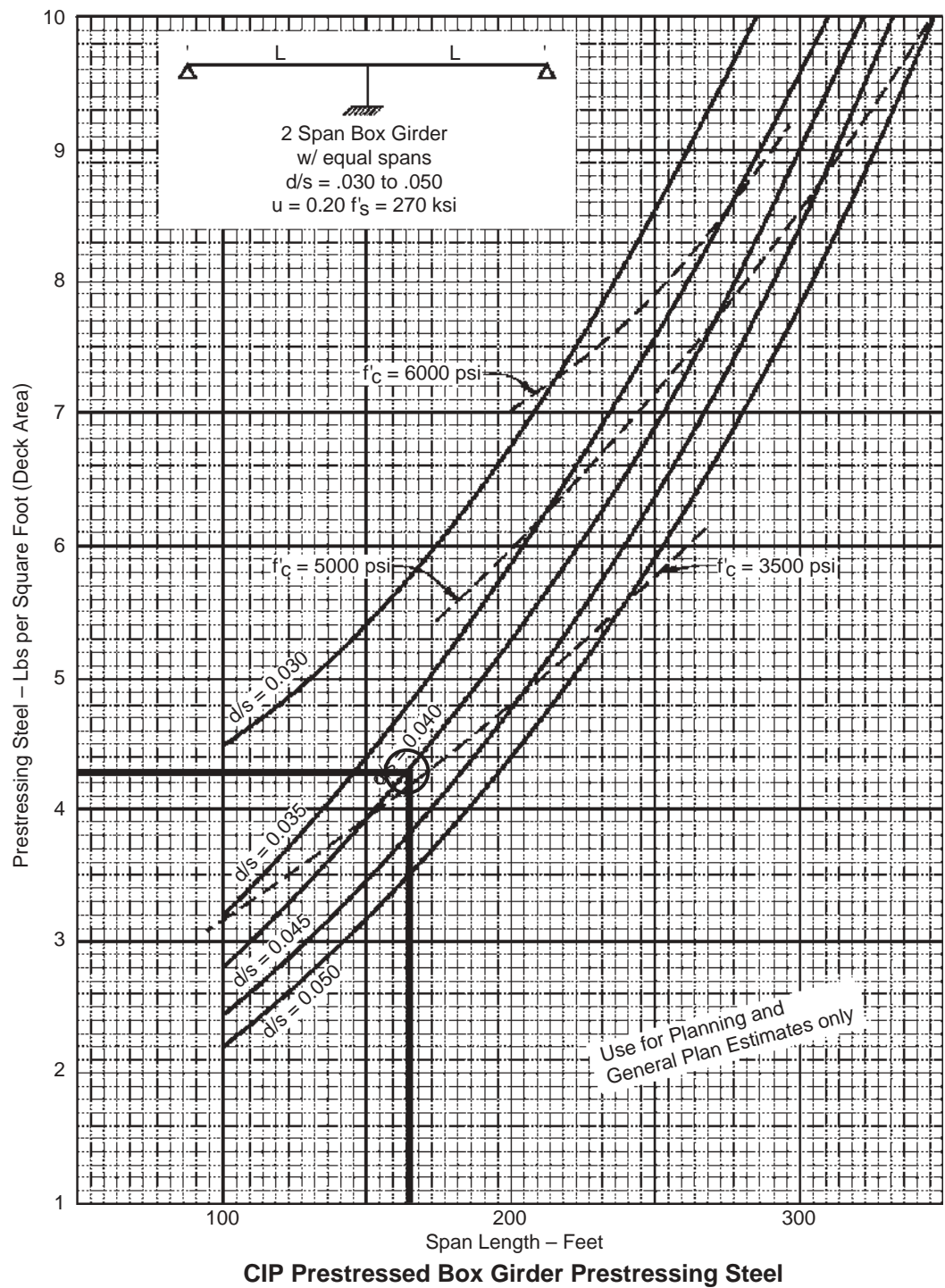**Two-Span Spliced U-Beam Girder**   **Design Example 2**

Figure 6.2-1 with labels:
- Prestressing Steel – Lbs per Square Foot (Deck Area)
- Span Length – Feet
- 2 Span Box Girder w/ equal spans, d/s = .030 to .050, u = 0.20 f'ₛ = 270 ksi
- f'_c = 6000 psi
- f'_c = 5000 psi
- f'_c = 3500 psi
- d/s = 0.030
- d/s = 0.035
- d/s = 0.040
- d/s = 0.045
- d/s = 0.050
- Use for Planning and General Plan Estimates only

CIP Prestressed Box Girder Prestressing Steel

Figure 6.2-1   Prestressing Steel Estimating Chart from [3]

Arrangement of these prestress ducts is shown in **Figure 4.3-1** at two locations. At midspan, the ducts are stacked along the web centerline, and the clearance between the ducts is 1.75 in., which exceeds the minimum requirements of LRFD Article 5.10.3.3. The lower two ducts are for Stage 1 P/T and the upper two are for Stage 2 P/T.

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Based on this configuration, the low point (66 ft from Centerline of Abutment 1) of Stage 1 P/T is calculated as:

Table 6.2-1   Post-Tensioning Considerations

Item	Distance (in.)
Cover	1.00
#5 Stirrup	0.75
Duct O.D.	3.25
Duct clear spacing/2	0.75
"Z"	0.75
Total	6.50

where "Z" is the distance between the centroid of the tendon and the center of the duct as shown in LRFD Article 5.9.1.6. At this location, the centroid of Stage 2 P/T is 10 in. above Stage 1 P/T. This relative spacing is maintained until the tendon reaches the flare where the ducts can be separated horizontally. In the girder flare region adjacent to the bent cap, the tendons for each stage of post-tensioning are side-by-side, as shown in **Figure 4.3-1 (b)**, which defines the high point of the tendon profile. Between the end of the bridge and the low points, the tendons splay out to provide the required separation between anchorages. The duct arrangement at the end diaphragm is shown in **Figure 4.3-1 (c)**.

The primary goal in developing a tendon profile is to mirror the dead load moment diagram, as this configuration provides the most efficient design if prestress losses are not considered. Therefore, the low point of the prestress cable is located at the point of maximum positive dead load moment and the high point is located at the centerline of Bent 2. The tendon is defined as a parabola extending between control points. However, since the profile has to be smooth to be continuous, the tendon curvature is reversed over Bent 2. Where the tendon curvature reverses from positive to negative curvature is called the point of inflection, which is typically located from 5% to 10% of the span length from the centerline of support.

These points and the inflection points define the parabolic tendon profiles that are shown in **Figure 6.2-2**. The inflection point is at 0.1L from the centerline of Bent 2, where L is the respective span length. As shown, the low point in Span 2 is higher than the low point in Span 1 due to differences in span lengths. With this arrangement, the deflections are balanced as a result of having a reduced eccentricity in Span 2.
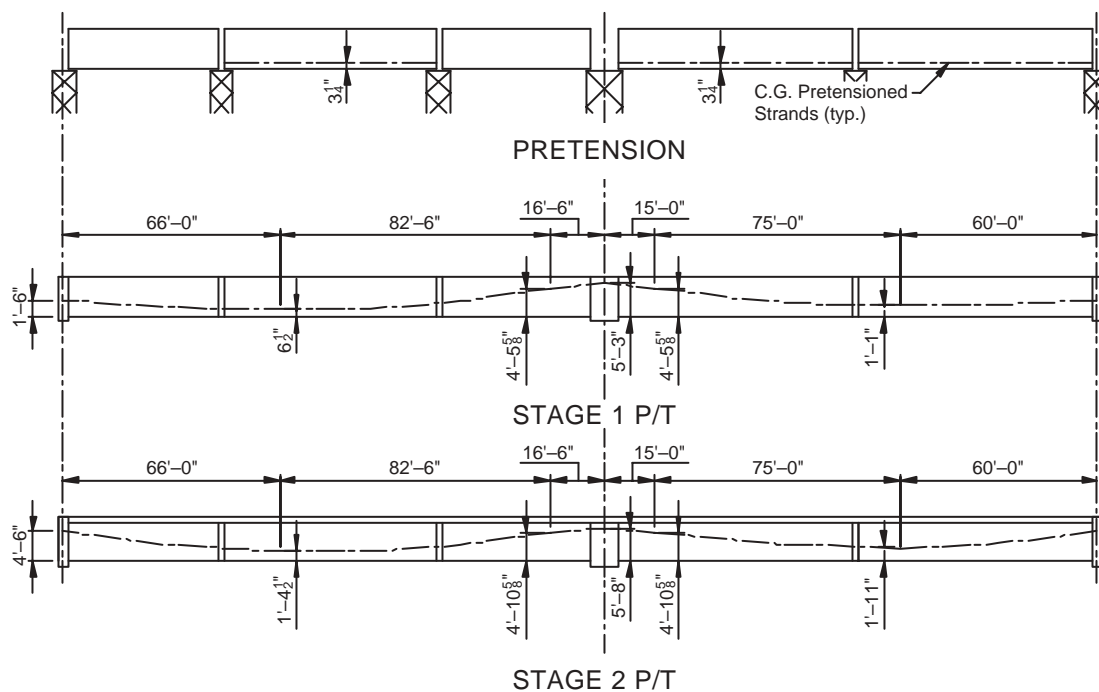
Figure 6.2-2   Prestress Layout

7   PRESTRESS FORCES AND MOMENTS

Prestress losses for both pretension and post-tension tendons are calculated using simplified long-term loss tables in the LRFD. Short-term losses including elastic shortening, friction and anchor set are calculated in this section. Verification of the elastic shortening losses is provided in **Section 8**.

Prestress losses including friction and anchor set are calculated using the assumed stress profile shown in **Figure 7-1**. Each tendon is divided into three sections per span and the losses are assumed to be linear within these sections. Friction and anchor set losses vary along the tendon length, while the elastic shortening and long term losses are assumed to be constant along the tendon length.

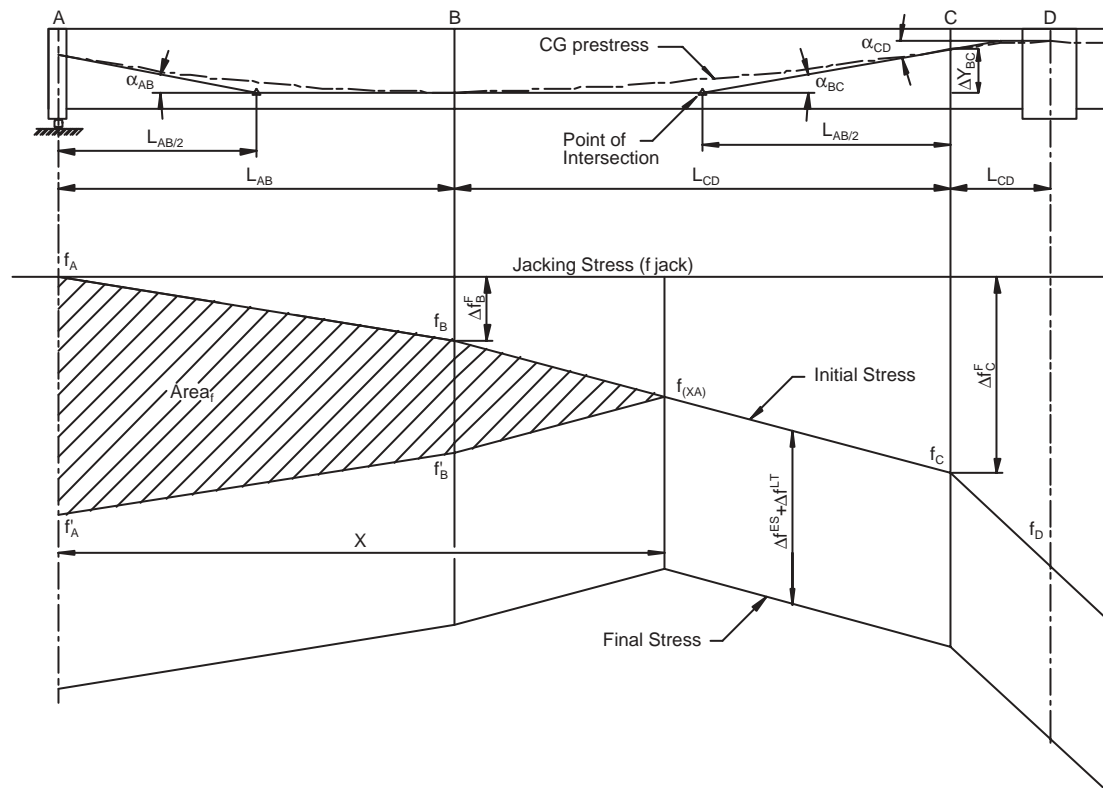**Two-Span Spliced U-Beam Girder**　　　　　　　　　　　**Design Example 2**

Figure 7-1　　　　**Post-Tensioning Tendon Stress Profile for Span 1**

7.1　　**FRICTION AND ANCHOR SET**

Friction loss is calculated along the length of the bridge with a friction coefficient $\mu = 0.2$ and wobble factor $k = 0.0002$ per foot of tendon length. Friction values are calculated at the low point, the inflection point and the high point of both spans. These values are shown in **Table 7.1-1**. The angle change is cumulative of the total angle change within the bridge. As shown in **Figure 7-1**, the angle change can be approximated for small angles as the vertical distance between points divided by half of the cable segment length (i.e. $\alpha_{BC} = 2\Delta Y_{BC}/L_{BC}$).

The total displacement of the strand associated with seating of the anchors is 0.375 in. For both tendons, the length of the anchor set loss exceeds the distance to the first control point. The release of prestress to set the wedges in the anchor plate is assumed to have the same friction loss as the initial stressing profile (i.e., $f_A' - f_B' = f_B - f_A$). The total shaded region A_F divided by the modulus of elasticity of the strand is equal to the anchor set displacement. This area can be calculated using the following:

$$A_F = 2(1 - f_X)X - (f_A - f_B)L_{AB} - (2 - f_B - f_X)(X - L_{AB})$$　　　　Eq. 7.1-1

where

$$f_X = f_B - \frac{(X - L_{AB})(f_B - f_C)}{L_{BC}}$$　　　　Eq. 7.1-2

The only unknown in this equation is X, or the length of influence the anchor set has on the prestress profile. Through trial and error, X is calculated and the initial stress profile is

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developed as a function of the jacking stress. As shown, X is larger than L_{AB}, and Equation 1 and 2 are valid. The jacking stress for both Stage 1 and 2 Post-Tensioning was determined to satisfy the allowable stress limits of LRFD Table 5.9.3-1.

Table 7.1-1   **Friction and Anchor Set Losses**

Stage 1 Post-Tensioning				
Location*	L (ft)	α	f_p/f_{pj}	f_{ps} (ksi)
A	0.0	0.000	1.000	205.0
B	66.0	0.029	0.981	201.1
C	148.5	0.124	0.947	194.1
D	165.0	0.219	0.926	189.8
E	180.0	0.324	0.906	185.8
F	255.0	0.428	0.876	179.7
G	315.0	0.460	0.864	177.0
After Anchor Set				
A'	0.0		0.920	*188.9*
B'	66.0		0.939	192.6
X	116.0		0.960	196.9
Stage 2 Post-Tensioning				
Location	L (ft)	α	f_p/f_{pj}	f_{ps} (ksi)
A	0.0	0.000	1.000	209.0
B	66.0	0.095	0.968	204.2
C	148.5	0.190	0.935	195.3
D	165.0	0.285	0.914	191.0
E	180.0	0.390	0.892	187.0
F	255.0	0.494	0.861	180.9
G	315.0	0.599	0.833	175.7
After Anchor Set				
A'	0.0		0.904	*188.9*
B'	66.0		0.935	195.4
X	106.6		0.952	198.9

*See **Figure 7-1** for locations. Controlling stress in strand = 0.7 f_{pu} = 189 ksi.

7.2   **ELASTIC SHORTENING**

For structures with multi-stage prestressing, each tendon has elastic shortening losses that occur during the initial stressing and stressing of subsequent tendons. Furthermore, the tendons are also subject to subsequent elongation due to externally applied loads that increase tendon stress (referred to as regain). For pretensioning, transfer of prestress to the concrete

results in shortening and direct loss in the tendon. The calculation of this loss requires determining stress in the concrete at the strand level f_{cgp} and, assuming perfect bond between the strand and the concrete, multiplying it by the ratio of the elastic modulus of the materials. At transfer, the concrete is subject to both prestress moments and self-weight moments. Hence, the loss is variable along the length of the segment. To simplify the problem, concrete shortening loss is computed at one location and assumed constant along the segment length.

For post-tensioning, the elastic shortening at stressing is a result of having several tendons stressed in sequence, and if one tendon is stressed, there is no elastic shortening losses. Additional losses develop when post-tensioning is applied at subsequent load stages, as mentioned previously. In both cases the stress in the concrete is computed at the level of each respective tendon f_{cgp} to get the elastic shortening losses, as shown in **Figure 7.2-1**.
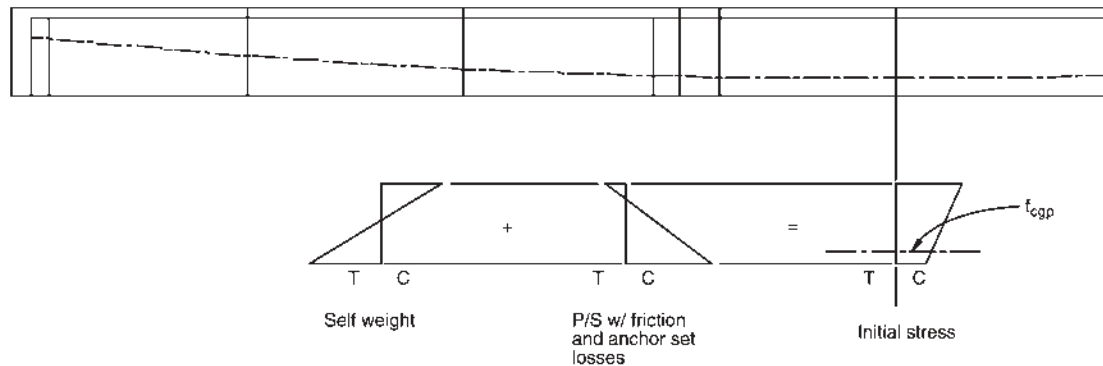

Figure 7.2-1   Girder Stress Profile for Elastic Shortening

The elastic shortening was calculated for three stages of prestressing at the assumed location of maximum positive moment 66 ft from centerline of Abutment 1 at the following intervals:

(1) transfer of pretension strands,

(2) Stage 1 post-tensioning, including regain from the fluid weight of the deck and

(3) Stage 2 post-tensioning.

Calculation of f_{cgp} requires the prestress forces for all stages of loading. Therefore, these losses are first be estimated and then verified after the design is complete. Verification is provided in **Section 8.1.4**.

7.3   TIME-DEPENDENT LOSSES

The long term losses associated with creep, shrinkage and relaxation are from LRFD Table 5.9.5.3-1. This table gives approximate lump-sum values that include all time-dependent losses. The AASHTO *Standard Specifications* has a similar table but it includes elastic shortening.

Since the approximate lump sum values are for final service conditions, time dependent losses must be estimated for each respective construction interval. The equation for the creep coefficient (LRFD Equation 5.4.2.3.2-1) is used to determine the amount of time-dependent loss that occurs during the respective construction intervals. The creep curve is a good indicator of the rate of total prestress loss because creep typically accounts for over half of the total time-dependent prestress loss. Also, elastic shortening loss is faster than the creep, which

balances prestress relaxation, which is slower. Taking the ratio of ultimate versus creep at time t, the percentage of time-dependent prestress loss can be calculated as:

$$\psi = \frac{(t)^{0.6}}{10 + (t)^{0.6}} \times 100$$

This ratio is 43%, 54% and 60% for time intervals of 30, 60 and 90 days, respectively. These ratios are used to calculate the amount of long term loss for the construction intervals shown in **Table 7.3-1**.

Table 7.3-1   Estimated Lump Sum Prestress Losses

Interval	Item	Δf_{pLT}	$\Delta f_{pES}{}^*$	Total
Day 60				
	Pretension	14	0	14
Day 90				
	Pretension	20	5	25
	Stage 1 P/T	10	3	13
Final Service				
	Pretension	33	12	45
	Stage 1 P/T	19	7	26
	Stage 2 P/T	19	2	21

* Cumulative elastic shortening losses to be verified with final design prestress forces. For further discussion see **Section 8.1.4**.

7.4   **PRESTRESS MOMENTS**

Prestress moments are calculated using the prestress force multiplied by its respective eccentricity for pretension moments. For post-tensioning, restraint (or secondary moments) develop at Bent 2. Secondary moments are calculated by integrating the curvature (M/EI) assuming free rotation at Bent 2 of both spans, and then calculating the moment required to bring this relative rotation back to zero.

In calculating the moment areas, the moments are interpolated between sections spaced no greater than 0.1L, where L is the span length. The section locations, shown in **Figure 7.4-1**, include nodes at multiples of 0.1L and at locations of abrupt changes in stress at the ends of the girder segments.

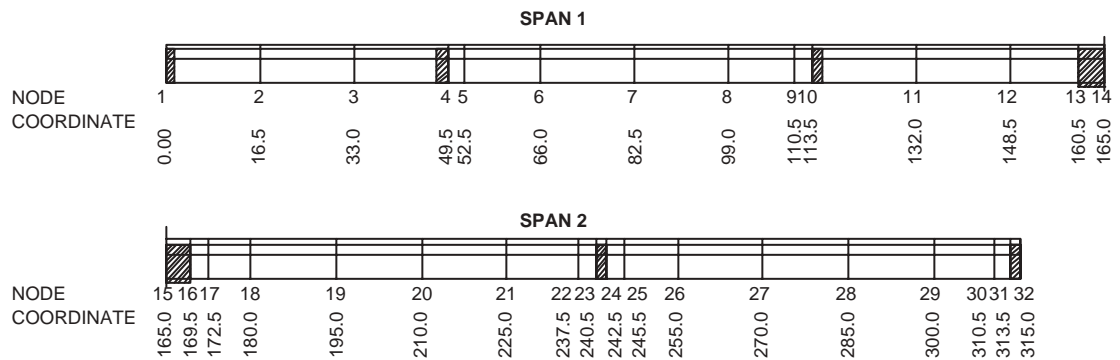**Two-Span Spliced U-Beam Girder**   **Design Example 2**

SPAN 1

NODE	1	2	3	4 5	6	7	8	9 10	11	12	13 14
COORDINATE	0.00	16.5	33.0	49.5 52.5	66.0	82.5	99.0	110.5 113.5	132.0	148.5	160.5 165.0

SPAN 2

NODE	15 16 17	18	19	20	21	22 23 24 25	26	27	28	29	30 31 32
COORDINATE	165.0 169.5 172.5	180.0	195.0	210.0	225.0	237.5 240.5 242.5 245.5	255.0	270.0	285.0	300.0	310.5 313.5 315.0

Figure 7.4-1   Node Layout and Coordinates

Calculation of the prestress secondary moments is illustrated in **Figure 7.4-2** and the tabulated moment areas are listed in **Table 7.4-1**. In this Figure, prestress moments for Span 1 are shown with only friction and anchor set losses. The simple-span moments (also referred to as primary moments) are the prestress eccentricity multiplied by the jacking stress including friction and anchor set loss. The first moment of area of the simple-span moment curve about Node 1 (total of Column 8 in **Table 7.4-1**) divided by the span length is the free rotation of Span 1. A similar procedure is performed for Span 2.
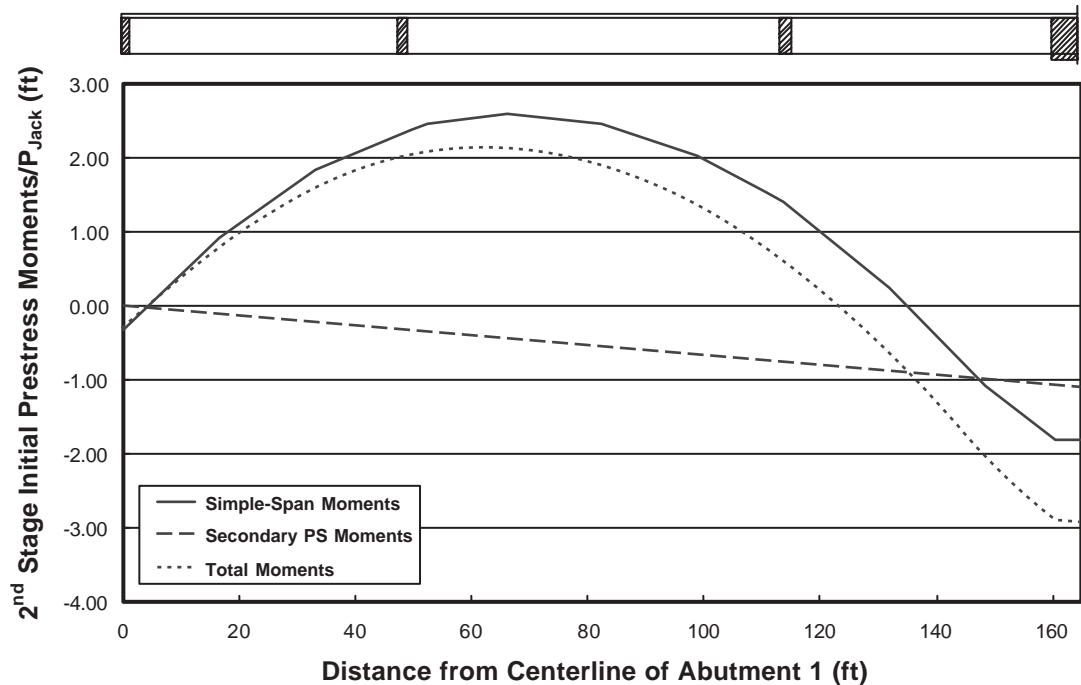

Legend:
— Simple-Span Moments
– – Secondary PS Moments
···· Total Moments

X-axis: Distance from Centerline of Abutment 1 (ft)
Y-axis: 2nd Stage Initial Prestress Moments/P_{Jack} (ft)

Figure 7.4-2   Stage 2 P/T Moment Profile for Span 1

The sum of the free rotations at Bent 2 is multiplied by the stiffness to get the secondary moment at Bent 2. This total stiffness is calculated using the following:

$$k_D = \frac{3EI}{L_{AD} + L_{DG}} = \frac{EI}{105}$$

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It should be noted that this is an approximate stiffness, because the stiffness of the flares is not included. For the Stage 2 P/T, the free rotations are $63.1P_{Jack}/EI$ and $52.1P_{Jack}/EI$ for Spans 1 and 2, respectively. Based on these rotations, the secondary moment at Bent 2 is:

M_{sec} $= -P_{Jack}(63.1+52.1)/105 = -1.097P_{Jack}$ k-ft

Figure 7.4-2 shows that the secondary moments decrease the effective eccentricity within the positive bending region and increase the prestress moments in the negative bending region.

Table 7.4-1   Span 1 Prestress Moments Calculations for Stage 2 P/T

Node	co (ft)	y_{cg} (ft)	e (ft)	F_p	M_{pr} (ft)	A^* (ft^2)	X^* (ft)	AX (ft^3)	M_{sec} (ft)	M_{coe} (ft)
	A	B	C	D	E	F	G	H	I	J
1	0.0	4.50	-0.35	0.903	-0.32				0.00	-0.32
2	16.5	3.13	1.02	0.911	0.93	5.0	13.8	70	-0.09	0.84
3	33.0	2.16	1.99	0.919	1.83	22.8	25.7	584	-0.18	1.65
4	49.5	1.57	2.58	0.927	2.39	34.9	41.6	1,451	-0.28	2.12
5	52.5	1.51	2.64	0.929	2.46	7.3	51.0	371	-0.29	2.16
6	66.0	1.38	2.78	0.935	2.60	34.1	59.3	2,022	-0.37	2.23
7	82.5	1.53	2.62	0.942	2.47	41.8	74.2	3,097	-0.46	2.08
8	99.0	2.00	2.15	0.949	2.04	37.2	90.5	3,362	-0.55	1.49
9	110.5	2.52	1.63	0.950	1.55	20.6	104.5	2,157	-0.61	0.94
10	113.5	2.68	1.47	0.949	1.40	4.4	112.0	496	-0.63	0.77
11	132.0	3.89	0.26	0.941	0.25	15.2	120.6	1,839	-0.73	-0.48
12	148.5	5.30	-1.15	0.935	-1.07	-6.8	144.7	-985	-0.82	-1.90
13	160.5	5.67	-1.96	0.930	-1.82	-17.4	155.0	-2,692	-1.89	-2.71
14	165.0	5.67	-1.96	0.928	-1.82	-8.2	162.7	-1,331	-0.92	-2.73
Total								10,438		

Legend		
Column	Symbol	Description
A	co	Distance from centerline of Abutment 1
B	y_{cg}	Distance from girder soffit to centroid of stage 2 post-tensioning tendons
C	e	Eccentricity of prestress strand
D	F_p	Prestress force divided by P_{Jack}
E	M_{pr}	Primary prestress moment = e F_p.
F	A^*	Area under primary prestress moment diagram measured between nodes
G	X^*	Distance of centroid of area to the centerline of Abutment 1
H	AX	First moment of area under the primary prestress moment diagram
I	M_{sec}	Secondary prestress moment
J	M_{coe}	Total prestress moment

Note: F_p has been divided by P_{Jack}.

The last column in **Table 7.4-1** is the total prestress moments (M_{coc}) without elastic shortening and time-dependent losses divided by P_{jack} for Stage 2 P/T. For node locations, see **Figure 7.4-1**.

8   FLEXURAL DESIGN

Prestress force and compressive strength requirements are developed from the analysis of service limit states for all stages of loading. From the previous sections, prestress moments and forces are developed as a function of P_{jack}. The jacking force is then solved for each stage of loading based on allowable tension stress requirements. Once prestress forces are developed, the compressive strength requirements are developed under service limit states. The initial assumptions used to develop these forces and compressive strengths require verification. Although several iterations are typically required, only one is presented in this example.

The flexural strength is evaluated to determine the amount of mild reinforcement required at critical locations within each span.

8.1   SERVICE LIMIT STATES

8.1.1   Girder Segment Transportation and Erection

The amount of prestressing used as pretensioning in the girder is typically a small percentage of the overall prestress force. At this stage of loading, the girder is designed to resist self-weight moments with an impact factor of 1.5 due to handling, transportation and erection as required in LRFD Article 5.14.1.2.

Based on allowable tension stress of $6\sqrt{f'_{ci}}$ (424 psi), the required number of 0.6 in. diameter strands with a jacking stress of 202.5 ksi is listed in the **Table 8.1.1-1**. Combined stresses from prestress, and self weight are shown for the top and bottom fiber of the girder. The stress due to prestressing includes the long term and elastic shortening losses at Day 60, as listed in **Table 7.3-1**.

The amount of mild reinforcement required in the bottom flange is based on the force developed in the tension zone assuming an uncracked section. This tension force is resisted by mild reinforcement assuming a stress of $0.5f_y$, as per LRFD Table 5.9.4.1.2-1. For Segments 4 and 5, 14 #5 rebar spaced along the top and bottom soffit mat is required to resist these tensile forces.

Two-Span Spliced U-Beam Girder   **Design Example 2**

Table 8.1.1-1   Pretension Calculations with Losses @ Day 60

Girder Segment	Length	Moment	No. of 0.6 in. Diameter Strands	Combined Stress		Flange Rebar
				Top	Bottom	
	(ft)	(k-ft)		(psi)	(psi)	(in^2)
1	46.5	698	0	342	-278	3.48
2	64.5	1,343	4	566	-284	3.40
3	45.5	668	0	328	-267	3.33
4	71.5	1,651	5	694	-344	4.11
5	71.5	1,651	5	694	-344	4.11

* Moments include 1.5 impact factor

8.1.2   Stage 1 P/T, Falsework Release and Deck Pour

The construction schedule shown in **Table 3.1-1** indicates Stage 1 P/T is applied at Day 60 and Stage 2 P/T is applied at Day 90. Within that interval, the falsework is released and the deck is poured. Therefore, Stage 1 P/T counters tensile stresses induced by these action including prestress losses to Day 90. The jacking force (P_{Jack}) is determined by trail and error and is equal to 2,300 kip. The jacking force is assumed, and the associated stresses are checked to verify if they are within allowable limits.
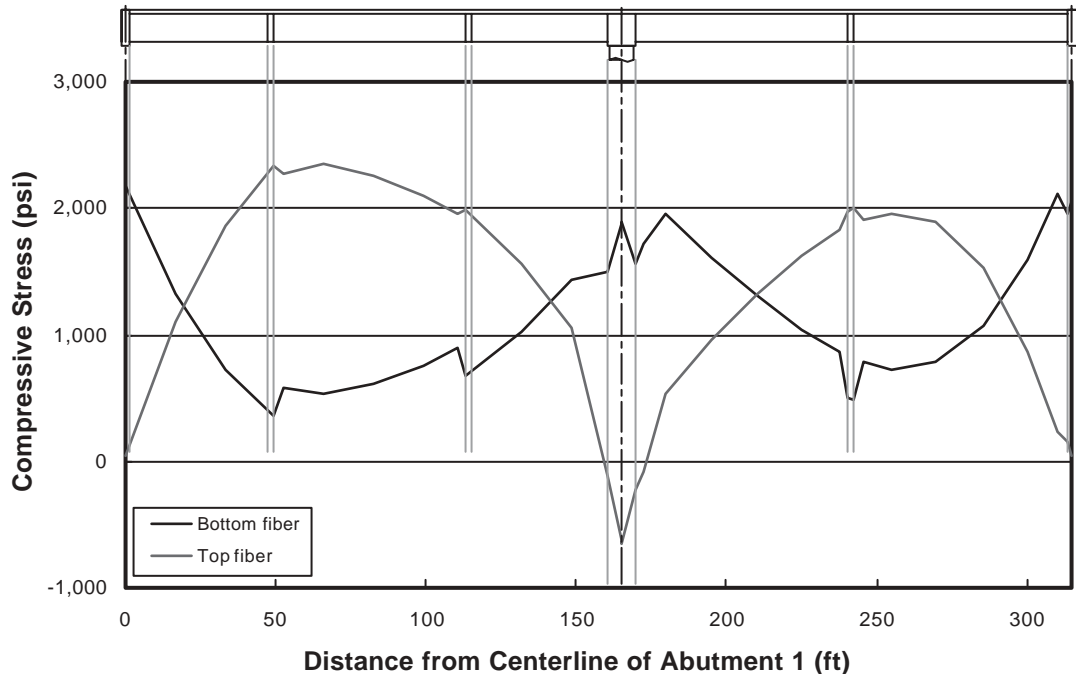
Figure 8.1.2-1   Girder Stress Immediately After Deck Pour

The combined stress profile of **Figure 8.1.2-1** shows the point of maximum tension, which is the controlling location for prestress, is the top fiber at the face of bent cap in Span 2. It

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should be noted that the stress between the faces of bent cap are artificial due to the "knife edge" support condition assumed at the center of Bent 2. Discontinuities in the stress profiles are due to termination of the pretensioning strand at the ends of the girder segment. Tension stress at this location requires reinforcement from the girder to extend into the bent cap. The tension force is calculated with the following combined stress values (compression is positive):

Top fiber stress (f_t) = -93 psi

Bottom fiber stress (f_b) = 1322 psi

The depth of the tension zone at the top of the girder, d_t, is computed as:

$$d_t = \frac{f_t H}{(f_t - f_b)} = \frac{93(72)}{(93 + 1322)} = 4.3 \text{ in.}$$

where H is the depth of the girder. As shown the tension depth falls within the top flange of the girder, and the width (b) assumed is 17 in. Based on these parameters the amount of reinforcement (A_s) required to develop this tension force is

$$A_s = \frac{f_t db}{2(f_y / 2)} = \frac{93(4.3)(17)}{2(30,000)} = 0.112 \text{ in}^2$$

Two #4 rebar extending from the face of the girder is sufficient to transfer that tension force at this temporary stage of loading.

The maximum compression stress during this interval occurs at the beginning of Segment 2 at the splice closest to Abutment 1. The maximum stress at this location is 2,350 psi, resulting in a required compressive strength at Stage 1 P/T of 3,900 psi. While the girders have attained the 28 day compressive strength, the splices have to attain this strength prior to stressing. Tabulated stress values for this interval are listed in **Appendix A**.

8.1.3 | **Stage 2 P/T and Final Service Conditions**

At this stage of loading, it is assumed all time-dependent prestress losses have occurred and the future wearing surface has been applied. The losses for the pretensioning and Stage 1 P/T are assumed to apply only to the non-composite section. In reality, these losses act on the composite section, which induce restraint (secondary) moments at the support. Although approximate, it has been standard practice for hand calculations to apply the prestress forces with all assumed losses to the non-composite section. *(3, 11)*

As with the prior load stages, the prestress force (Stage 2 P/T) is determined based on allowable tension stress at the Service III limit state. The stress profile for this load case in **Figure 8.1.3-1** shows that the top of girder stress at the face of the bent cap is the controlling location for prestress force. The Stage 2 P/T force is 1,600 kips.

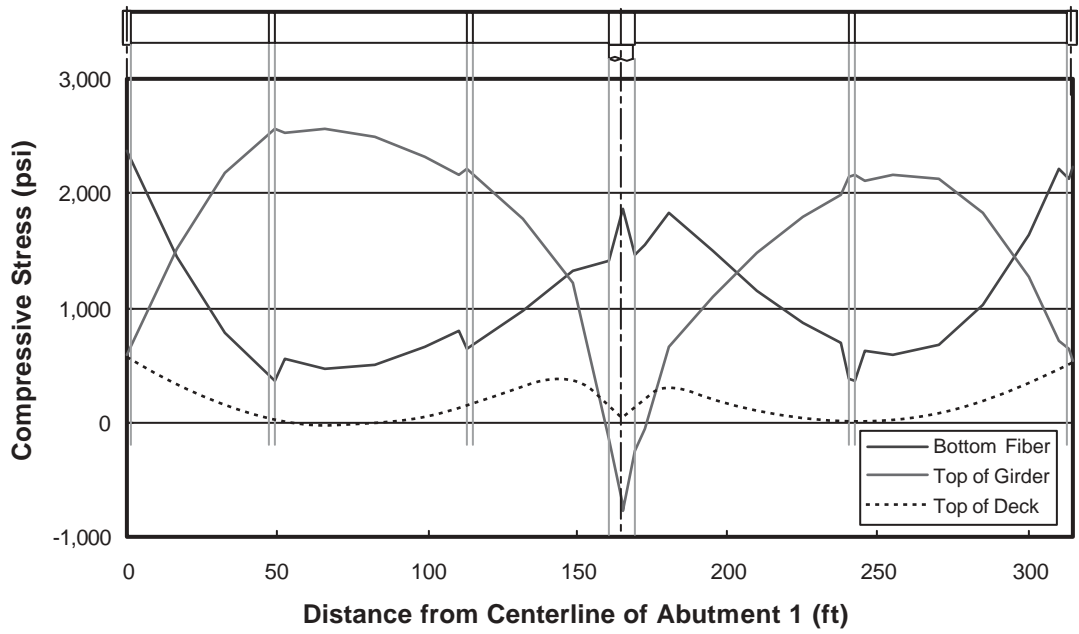**Two-Span Spliced U-Beam Girder** **Design Example 2**

Figure 8.1.3-1 **Service III (D + P/S + 0.8L) Maximum Tension Stress at Service**

Table 8.1.3-1 **Prestress Force Summary**

Prestress	Jacking Force (kips)
Pretension	
Girder Segment 1	0
Girder Segment 2	176
Girder Segment 3	0
Girder Segment 4	220
Girder Segment 5	220
Post-Tension	
Stage 1	2300
Stage 2	1600

The mild reinforcement required to pass into the bent cap is calculated as using the following stress

Top fiber stress (f_t) = -255 psi

Bottom fiber stress (f_b) = 1,464 psi

The depth of the tension zone at the top of the girder, d_t, is computed as:

$$d_t = \frac{157(66)}{(157 + 1255)} = 9.79 \text{ in.}$$

b is conservatively assumed to be 17 in.

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Two-Span Spliced U-Beam Girder   **Design Example 2**

$$A_s = \frac{f_t db}{2(f_y/2)} = \frac{157\,(9.79)(17)}{2(30,000)} = 0.71 \text{ in}^2$$

Three #5 rebar extending from the face of each web is sufficient to transfer that tension force at this final service stage of loading.

The compressive strength assumption must be verified based on the allowable stress for three different load cases. The maximum compressive stress profile for the D + L + P/S load combination shown in **Figure 8.1.3-2**, indicates that the top fiber of the girder at Node 4 (66.0 ft from Centerline of Abutment 1) controls the girder strength, and the deck compressive stress is far less than the allowable of 3,000 psi.
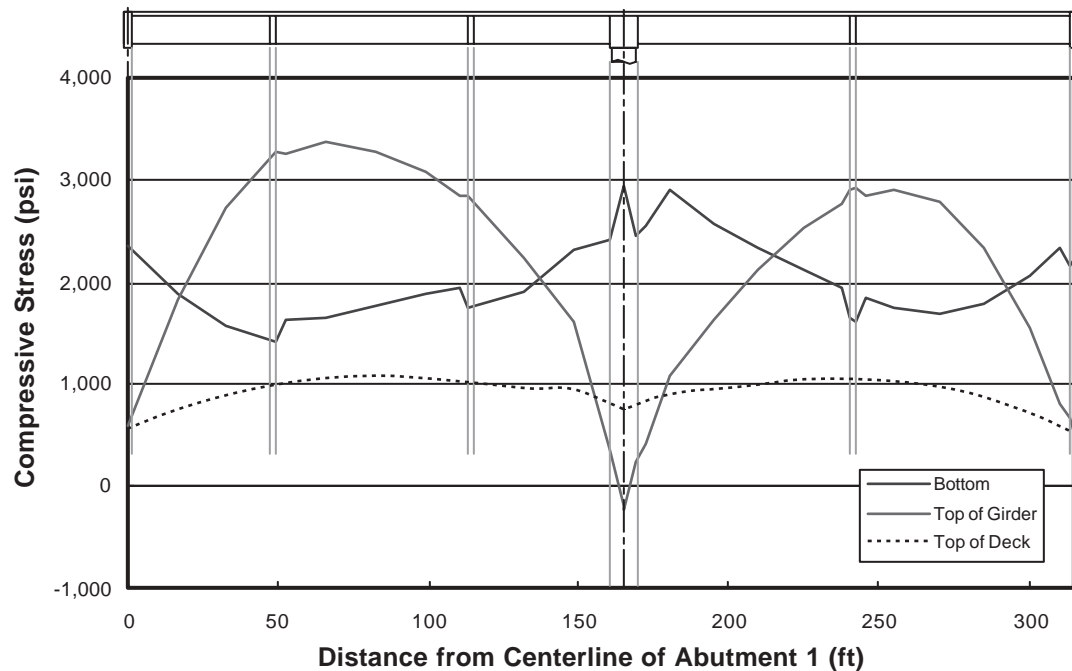
Figure 8.1.3-2   **Service I (D+P/S+L) Maximum Compressive Stress**

The maximum stress for the components within the bridge are listed in **Table 8.1.3-2** along with the corresponding required 28-day compressive strength. The underlined numbers represent controlling load cases for that particular component, and the bottom line has the required compressive strength based on these stresses.

Table 8.1.3-2   **Maximum Concrete Compressive Stress**

Load Case	Girder (psi)	Splice (psi)	Bent Cap (psi)	End Diaphragm (psi)
D + P/S	_2,681_	_2,646_	1,500	_2,160_
D + P/S + L	3,364	3,278	2,460	2,167
½(D + P/S) + L	2,028	1,951	_1,715_	1,090
Required f'$_c$	6,000	5,900	4,300	4,800

The required compressive strength of the girders is less than specified and close to the value assumed in the analysis (see **Table 5.3-1**). The cast-in-place splice compressive strength of 5,900 psi could be difficult to obtain from ready-mix plants. Also, an owner may request Type III cement for the splice to match the color of adjacent girder segments, as concrete with Type III cement is typically lighter in appearance than with Type II cement.

On the contrary, the bent cap and end-diaphragms are too massive for Type III Cement. Since these components are offset from the face of the girder, the color difference may not be an issue, and Type II Cement is required. Local ready mix suppliers should be consulted before specifying concrete strengths above 5,000 psi. If these strengths cannot be reliably obtained, girder flares can be added to the ends of the girders at the splices will reduce these required strengths at the expense of additional girder hauling and lifting weight. For the bent cap and end diaphragm, the compression rapidly diffuses into the solid diaphragm. Therefore, it is reasonable to allow these stresses to exceed the allowable stress of the bent cap and end-diaphragm concrete at these locations.

It should be noted that the allowable stresses are checked after completion of construction when traffic is first allowed on the bridge. Calculations are not shown because they are similar to the final service stress check shown previously, and stresses at this limit state did not control the design.

8.1.4   Elastic Shortening Loss Verification

The elastic shortening losses are verified based on the design prestress forces developed for each construction interval, and final service conditions. The stress used to compute the elastic shortening is shown in **Table 8.1.4-1**.

Table 8.1.4-1   Concrete Compressive Stress at Section 66 ft from CL Abutment 1

	Girder Stress		Stress at Tendon Level (f_{cgp})		
	Top (psi)	Bottom (psi)	Pretension (psi)	Stage 1 P/T (psi)	Stage 2 P/T (psi)
Day 60					
Pretension*	-94	256	239	------	------
Self Weight	304	-248	-221	------	------
Total	210	208	18	------	------
Δf_{pES}			118	------	------
Assumed			0	------	------
Day 90					
Stage 1 P/T*	-861	2,945	2,769	1,240**	------
Falsework Release	1,673	-1,361	-1,214	0	------
Deck	1,076	-882	-787	-705	------
Total	1,888	711	768	535	------
Δf_{pES}			5,047	3,543	------
From Previous Stages			118	0	------
Total Elastic Shortening Loss			5,165	3,453	------
Assumed			5,000	3,000	------
Final Service					
Stage 2 P/T*	127	1,361	1,302	1,251	1,081**
SDL	115	-259	-241	-255	-173
Total	242	1,102	1,073	1,037	344
Δf_{pES}			7,050	6,811	2,260
From Previous Stages			5,165	3,543	0
Total Elastic Shortening Loss			12,215	10,264	2,260
Assumed			12,000	7,000	2,000

*Based on initial prestress without time-dependent and elastic shortening losses.

**Reduced by (N-1)/2N, where N is the number of tendons stressed at each load stage.

It should be noted that the regain stress from the applied permanent loads is used in the elastic shortening loss calculations.

The differences between the assumed and computed elastic shortening losses are significant at the final service stage of loading. This procedure may require more than one iteration to converge on a solution. However, this difference is small, as compared to the prestress in the strand (< 2%).

8.2   **STRENGTH LIMIT STATE**

The flexural strength limit state moments are calculated in **Table 8.2-1** at selected locations within Span 1. Nodes 4, 6 and 13 are shown in **Figure 7.4-1**. These moments include prestress secondary moments assuming all losses have occurred.

Table 8.2-1   Ultimate Moment Calculations (k-ft)

Item	γ	Moment (k-ft)			Factored Moment (k-ft)		
Node		4	6	13	4	6	13
D	1.25	5,932	6,242	-7,126	7,415	7,803	-8,908
SDL	1.50	942	989	-1,311	1,413	1,484	-1,967
LL+I	1.75	5,468	5,948	-5,134	9,569	10,409	-8,985
P/S *	1.00	489	653	1,587	489	653	1,587
M_u					18,900	20,300	-18,300

* P/S (prestress secondary moments).

The flexural capacity is first analyzed assuming no mild reinforcement is in the section, and prestress provides the only resistance to the factored moments. The capacity at each section is the compared to the demands with appropriate resistance factors. Mild steel, if required is added to the section and the analysis is repeated.

8.2.1   Flexural Resistance at Splice in Span 1 (Node 4)

The flexural resistance at Node 4 (see **Figure 7.4-1** for location) is calculated using the following parameters:

Centroid of prestress measured from the soffit is calculated as follows:

Table 8.2.1-1   Post-Tensioning Tendons at Splice

Tendon	A_{ps}	y_{cg}	$A_{ps}\ y_{cg}$
Stage 1 P/T	11.2	7.25	81.2
Stage 2 P/T	7.9	18.84	148.8
Total	19.1	-----	230.0

$$y_{cg} = \frac{230}{19.1} = 12.0 \text{ in}$$

The neutral axis is calculated using the following:

$$c = \frac{A_{ps}f_{pu} + 0.85\beta_1 f_c' (b - b_w)h_f}{\beta_1 0.85 f_c' b_w + kA_{ps}f_{pu} / d_p}$$ LRFD Eq. 5.7.3.1.1-3

where

f_{pu} = 270 ksi

f_c' = 4.0 ksi

β_1 = 0.85

b   = 168 in.

b_w = 31 (Represents average width at top of girder including top bulb and build-up, assuming c = 21.25 in.)

d_p = 81.2 − 12.0 = 69.1 in.

Therefore,

$$c = \frac{(19.1)(270) - (0.85)^2(4)(168 - 31)(7.25)}{(0.85)^2(4)(31) + 0.28(19.1)(270)/(69.1)} = 20.7 \text{ in.}$$

Check reinforcement ratio

$c/d_p = 20.7/69.1 = 0.30 < 0.42$

$$f_{ps} = \left[1 - k\frac{c}{d_p}\right] \qquad\qquad \text{LRFD Eq. 5.7.3.1.1-1}$$

where $k = 0.28$ from LRFD Table C5.7.3.1.1-1

$$f_{ps} = \left[1 - 0.28\frac{20.7}{69.1}\right]270 = 247 \text{ ksi}$$

where $a = 0.85(20.7) = 17.6$ in.

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85f_c'(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad \text{LRFD Eq. 5.7.3.2.2-1}$$

$$M_n = (19.1)(247)(69.2 - (17.6)/2) + (0.85)(4)(168 - 31)(0.85)(7.25)\left[\frac{18}{2} - \frac{7.25}{2}\right]$$

$$= 22,500 \text{ k-ft}$$

The resistance factor for flexure at joints is 0.95, and the corresponding resistance at Node 4, is

$$\phi M_n = 0.95(22,500) = 21,300 \text{ k-ft}$$

This capacity is greater than the demand of 18,900 kft and 20,350 at both Node 4 and Node 6, respectively. Therefore, a capacity check at Node 6 is not required.

It should be noted that the girder was assumed to have a compressive strength of 4 ksi. In certain cases, this assumption could lead to unconservative results. However, the LRFD Equations for estimating the neutral axis depth for T-sections are considered conservative, and therefore, the analysis presented are satisfactory *(12)*. For a more refined results, a section analysis program should be used *(13)*.

8.2.2   Flexural Resistance at Face of Bent Cap in Span 1 (Node 13)

The negative flexure capacity at Node 13 is checked in a similar manner.

Table 8.2.2-1   Post-Tensioning Tendons at Face of Bent Cap

Tendon	A_{ps}	y_{cg}	$A_{ps}\,y_{cg}$
Stage 1 P/T	11.2	62.3	697
Stage 2 P/T	7.9	68.4	540
Total	19.1	-----	1238

$$y_{cg} = d_p = \frac{1238}{19.1} = 64.8 \text{ in}$$

where

f_{pu} = 270 ksi

f'_c = 6.0 ksi

β_1 = 0.75

b   = 60 in.

b_w = 28 in. (flare section)

$$c = \frac{(19.1)(270) - (0.85)(0.75)(6)(60 - 28)(13)}{(0.85)(0.75)(6)(28) + 0.28(19.1)(270)/(64.8)} = 27.6 \text{ in}$$

c/d_p = 27.6/64.8 = 0.42

This ratio was calculated assuming no mild reinforcement is in the deck. Including this reinforcement may push the section into the over-reinforced category. Therefore, the section needs to be evaluated after this reinforcement has been designed.

$$f_{ps} = \left[1 - 0.28\frac{27.6}{64.7}\right] = 238 \text{ ksi}$$

Therefore,

$$M_n = (19.1)(238)(64.7 - (20.7)/2) + (0.85)(0.75)(6)(60 - 28)(13)\left[\frac{20.7}{2} - \frac{13}{2}\right] = 21{,}100 \text{ k-ft}$$

The factored resistance is calculated as

$$\phi M_n = 0.95(21{,}100) = 20{,}040 \text{ k-ft}$$

which is larger than the demand shown in **Table 8.2-1** of 18,300 k-ft. Hence, no mild reinforcement is required at this location for strength requirements. However, the section has to be checked for seismic conditions in **Section 11**.

9   SHEAR DESIGN

The procedure for shear design is no different for spliced girder bridges than conventional prestressed concrete girder bridges with the exception of the interface shear at the splices, and in this example, at the face of the bent cap. The shear demands are evaluated in the negative bending region of the superstructure, as this is typically the location of greatest required shear reinforcement in a 2-span bridge.

9.1   DETAILING REQUIREMENTS

The concrete shear parameters shown in LRFD Table 5.8.3.4.2-1 were developed with the assumption of having well distributed reinforcement in both the longitudinal and transverse directions. The maximum longitudinal and transverse spacing of the web reinforcement cannot exceed 24 in. for shear stress less than $0.125f'_c$ and 12 in for shear stress greater than $0.125 f'_c$.

9.2   STIRRUP REQUIRMENTS

To illustrate the shear design procedure, a section at the face of the bent cap is presented in this section. To account for the shear transfer in the compression zone of the girder, the shear demand should be taken at a distance equal to the effective depth of superstructure away from

the support. However, due to the variable width of the web, the required stirrup spacing is evaluated at the beginning and end of the flare, and the minimum spacing is determined by interpolation between these two points.

The factored shear demand and corresponding moments are calculated as follows:

Table 9.2-1   Factored Shear and Associated Moments

	γ	V (k)	γV (k)	M (k-ft)	γM (k-ft)
DC	1.25	278	348	7126	8908
DW	1.50	29	44	1311	1967
L	1.75	202	354	2590	4533
P/S	1.00	10	10	1587	1587
		$V_u =$	756	$M_u =$	17,000

As calculated in **Section 8**,

c = 27.6 in.

d = 64.8 in.

b_w = 28.0 in.

f_{ps} = 241 ksi

$d_v = 64.8 - \dfrac{0.75(27.6)}{2} = 54.5 \text{ in.}$

This distance d_v need not to be taken less than 0.9d, or 58.3 in. (see LRFD Article 5.8.2.9).

ϕ = 0.9 (LRFD Table 5.5.4.2.2-1)

$b_v = b_w - \phi_{duct}/4 = 28 - 4(3.25)/4 = 24.75 \text{ in}$

$v = \dfrac{V_u - V_p}{\phi b_v d_v} = \dfrac{756 - 0}{(0.9)(58.3)(24.8)} = 0.581 \text{ ksi}$

Assume $\theta = 30°$

$\varepsilon_x = \dfrac{\left(M_u / d_v + 0.5 V_u \cot\theta - A_{ps} f_{po} \right)}{2 E_p A_{ps}}$ 　　　LRFD Eq. 5.8.4.3.2-1

$\varepsilon_x = \dfrac{(17000 / 4.85 + 0.5(756)\cot 30 - (19.1)(189))}{2(28500)(19.1)} = 0.499 \times 10^{-3}$

From LRFD Table 5.8.3.4.2-1

$\theta = 31°,\ \beta = 2.50$

$V_c = \beta \sqrt{f'c}\, b_v d_v$ 　　　LRFD Eq. 5.8.3.3 -3

$V_c = 2.50 \dfrac{\sqrt{6000}}{1000}(58.2)(24.8) = 280 \text{ kip}$

$V_s = \dfrac{V_u}{\phi} - V_c$

$V_s = 756 /(0.9) - 280 = 560 \text{ kip}$

The spacing of the #5 stirrups is computed using the following equation

$$s \le \frac{A s f_y d_v \cot \theta}{V_s}$$

$$s \le \frac{1.24(60)(58.2)\cot(31°)}{560} = 13.0 \text{ in.}$$

9.3 **LONGITUDINAL FORCES FROM SHEAR MECHANISM**

To complete the variable angle truss mechanism, the longitudinal forces developed must be checked. The longitudinal force T is calculated as:

$$T = \frac{M_u}{d_v \dot{o}} + \left[\frac{V_u}{\phi} - 0.5 V_s\right] \cot \theta \qquad \text{LRFD Eq. 5.8.3.5 -1}$$

$$T = \frac{17000(12)}{(58.3)(0.9)} + \left[\frac{756}{0.9} - (0.5)(560)\right] \cot 31° = 4,820 \text{ kip}$$

$$T = A_{ps} f_{ps} + A_s f_y$$

$$A_{ps} f_{ps} = (19.1)(238) = 4550 \text{ kip}$$

Solving for A_s:

$$A_s = \frac{T - A_{ps} f_{ps}}{f_y}$$

$$A_s = \frac{4820 - 4550}{60} = 4.50 \text{ in}^2$$

This reinforcement of #5 spaced at 12 in. on center is provided in the deck to minimize the amount of congestion in the precast girder.

9.4 **INTERFACE SHEAR**

The interface shear demand between the deck and the girder in the negative bending region is not significant, as it falls outside of the effective shear depth d_v, and this action is limited to the positive bending region of the girder. Since the girder segments terminate at the face of the bent cap, the interface shear capacity must be checked at this section.

The permanent normal force acting on the shear plane P_c is the prestress force with all losses acting on that section equal to 2,973 kip.

μ = 0.6 for smooth concrete with no shear keys.

$$\dot{o}V_n = (0.9)(0.6)(2,973) = 1,605 \text{ kip}$$

which is far greater than the shear demand of 756 kip.

However, the shear stress on the contact area is limited to $0.2 f'_c$ or 800 psi, which ever is less. The contact area A_{cv} is the area within the compression zone at ultimate, which is:

$$A_{cv} = (60 - 28)(10) + 28(27.6) = 1,188 \text{ in}^2$$

Therefore,

$$\dot{o}V_n = (0.9)(0.8)(1188) = 855 \text{ kip}$$

which is greater than the demand. Based on the interface shear capacity, no surface preparation (i.e., intentional roughening or shear keys) is required.
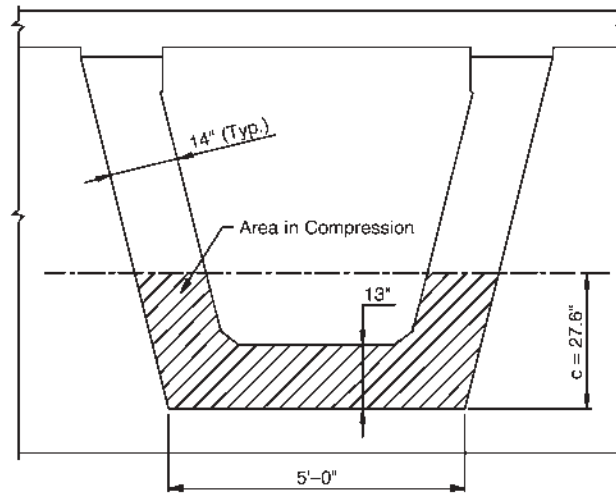
Figure 9.4-1   Compression Zone at Ultimate

10   DEFLECTIONS

For permanent loads, such as prestress and self-weight, falsework tower elevations need to be set so the build-up is the same as dimensioned on the plans. Also, deck form work is supported by the girders, and screed elevations must account for deflections due to loads placed on the structure after the deck is cast, including the fluid weight of the deck.

10.1   ESTIMATED DEFLECTIONS

The estimated deflections for various load intervals shown in **Figure 10.1-1** were calculated by integrating curvature (M/EI) along the length of the bridge. At each respective stage of loading, the deflections include all prestress losses for each respective interval of loading. To be consistent with the elastic design approach, these deflections do not account for creep and differential shrinkage.

10.1.1   Pretensioned Girder Segments

The amount of pretensioning required is only enough to prevent cracking during hauling and erection. Hence, the amount of deflection over the assumed 60 day period prior to applying Stage 1 P/T is not going to be the large upward camber that is typical of conventional precast girder bridges. In contrast, the net deflection at the midspan of the segments is downward. These deflections are based on an assumed support reaction being 12 in. from the end of the girder. For all segments, the deflections are less than 1/8 in, and therefore, not included in the deflection profiles of **Figure 10.1-1**.

Table 10.1-1   Initial Midspan Segment Deflection after Transfer

Segment	L	P_f	Δ
	(ft)	(kip)	(in.)
1	44.0	0	-0.03
2	62.0	162	-0.06
3	43.0	0	-0.03
4	69.0	202	-0.09
5	69.0	202	-0.09

Note: Positive is upward.

10.1.2   Girder Deflections after Falsework Release

When Stage 1 P/T (1) is applied, the structure will lift off of the falsework supports within each span, as shown in Profile (1) of **Figure 10.1-1**. Deflection at this load stage includes the self weight of the girders. The total cumulative deflection profile immediately after the deck pour (see Profile (2)) shows a relative downward deflection from the previous load interval to the lowest point through out the loading history. This downward deflection is followed by a net upward deflection from Stage 2 P/T [Profile (3)]. This deflection profile is from prestress forces with no Long-Term losses. A combination of prestress losses and the addition of superimposed dead load (SDL) brings the deflection to its final service condition [Profile (4)].
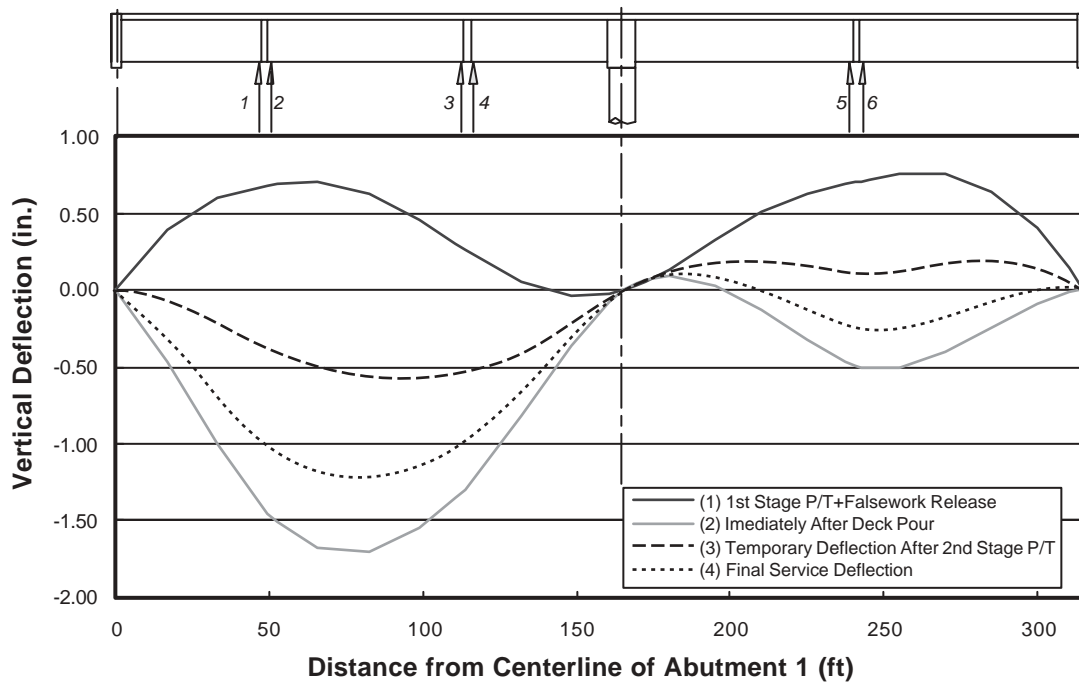
Figure 10.1-1   Girder Deflection Profiles

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10.2 **FALSEWORK AND SCREED ELEVATIONS**

From the deflection profiles presented in the previous section, the profile grade of the riding surface and the elevations of the falsework tower supports are developed.

The elevation of the falsework tower supports is the elevation of each respective support on the profile grade less the superstructure depth with the added camber listed in **Table 10.2-1**. This camber is based on the total deflection shown as Profile (4) in **Figure 10.1-1**.

Table 10.2-1 **Falsework Camber Calculations (in.)**

Falsework Sup.#	Distance (ft)	$\Delta_{(4)}$ (in.)	Camber (in.)
1	46.5	-0.96	0.96
2	50.5	-1.03	1.03
3	112.5	-1.01	1.01
4	116.5	-0.94	0.94
5	239.5	-0.23	0.23
6	243.5	-0.24	0.24

Note: Positive is upward. For falsework support locations see **Figure 10.1-1**.

The screed line elevations should be higher than the profile grade by the difference in deflections between the Stage 1 P/T and Falsework Release Deflections (1) and the Final Service Deflections (4). This difference includes all of the permanent loads on the girder that occur after the screed rails are set, which is, the fluid weight of the deck, Stage 2 P/T, the application of the superimposed dead load and all prestress losses.

The amount of vertical distance, or camber (upward is positive) between the profile grade elevation the screed line elevation is listed in **Table 10.2-2**. A more complete listing of girder deflection calculations is shown in **Table A-7** of **Appendix A**.

Table 10.2-2 **Screed Camber**

	Location	Distance (ft)	(4) (in.)	(1) (in.)	(4)-(1) (in.)	Camber (in.)
Span1	¼	41.3	-0.85	0.65	-1.50	1.50
	½	82.5	-1.22	0.63	-1.85	1.85
	¾	123.8	-0.81	0.15	-0.96	0.96
Span 2	¼	202.5	0.06	0.42	-0.36	0.36
	½	240.0	-0.23	0.70	-0.93	0.93
	¾	277.5	-0.12	0.71	-0.83	0.83

Note: Positive is upward.

11   SEISMIC CONSIDERATIONS

Seismic considerations related to development of an integral column-superstructure connection under severe seismic loads are presented in this section. It is assumed the seismic forces are derived from the overstrength moment due to column plastic hinging. Components, such as the bent cap and superstructure are designed to resist these forces with sufficient factor of safety. This section covers the seismic design and analysis in the longitudinal direction, as it relates to design of the spliced girder superstructure. For analysis in the transverse direction, see references *(7, 14)*.

The columns are designed to resist fully reversed displacement cycles up to a structural ductility of 4 ($\mu_\Delta = 4$). As a result, significant inelastic strain of the column reinforcement will occur within the bent cap-column joint region. More information on seismic design of bridge columns in severe seismic regions can be found in reference *(7)*.

In that text, a methodology for seismic design using capacity design principals is presented. Maximum plastic hinge forces are developed using overstrength material strengths listed in **Table 11-1**. The resistance is calculated using expected material properties and no strength reduction factors are necessary.

Table 11-1   **Material Strength for Seismic Design**

	Nominal (ksi)	Expected (ksi)	Overstrength (ksi)
Rebar			
f_{ye}	60	66	78
f_{ue}	80	88	104
Concrete			
f'_{ce}	4.0	5.2	6.8

11.1   SEISMIC FORCES

An idealized representation of the maximum seismic forces the bridge is designed to resist is shown in **Figure 11.1-1**. Seismic Resistance is achieved from the ground up. First the column is designed to resist the maximum plastic moment at the top M_o based overstrength material properties shown in **Table 11-1**. The analysis to attain this moment includes strain hardening of the reinforcement, and effects of the confinement on the concrete. Based on the overstrength moment, anchorage of the column reinforcement and adequacy of the joint shear reinforcement are evaluated.
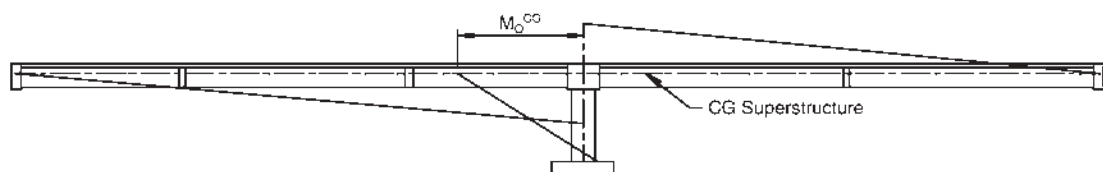
Figure 11.1-1   **Seismic Moments**

The column overstrength moments are extrapolated to the center of gravity of the superstructure M_o^{CG}, as shown in **Figure 11.1-2**.
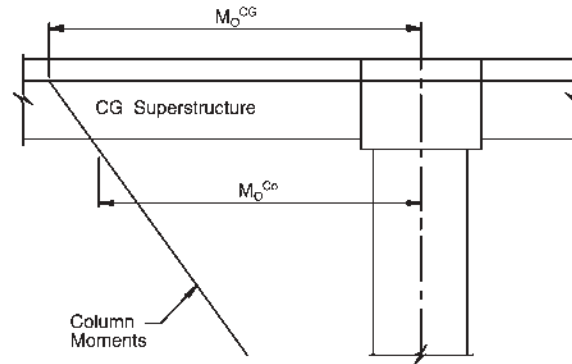
Figure 11.1-2   Column Moment Detail

From column analysis:

M_o^{Top} = 33,000 kft (top of column)

M_o^{Base} = 5,000 k-ft (base of column)

From the bridge geometry

L_c = 20.6 ft (clear distance between top of footing and face of bent cap)

Y_{CG} = 5.08 ft (distance from top of column to center of gravity of superstructure)

The design seismic moment at the center of gravity of superstructure is calculated using the following:

$$M_o^{CG} = M_o^{Top} + \frac{(M_o^{Top} + M_o^{Base})}{L_c}(Y_{CG})$$

$$M_o^{CG} = 33000 + \frac{(33000 + 5000)}{20.6}(5.08) = 42,400 \text{ k-ft}$$

This moment is resisted by the bent cap through torsion forces. The idealized torque profile for the bent cap, shown in **Figure 11-3**, indicates the maximum torque occurs at the edge of the column and is equal to $M_o^{CG}/2$. The torsion in the bent cap is distributed into the superstructure based on relative flexibility of the superstructure and the bent cap. Hence, the superstructure does not resist column overstrength moments uniformly across the width. To account for this, an effective width is used to distribute the moments along the length of the bent cap, as discussed in **Section 11.4**.
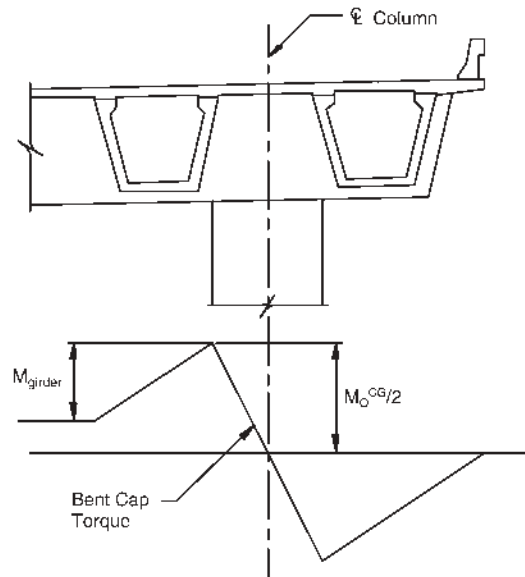
Figure 11-3   Bent Cap Torque Profile

11.2   JOINT SHEAR DESIGN

In order for the column reinforcement to be anchored into the bent cap and to develop its overstrength capacity, the column rebar has to extend into the bent cap as high as possible without interfering with the bent cap and deck reinforcement. For this example, the bars extend up to 9 in. below the deck surface. In order to develop favorable bond strut angles this extension of the reinforcement is required even if the development length equations permit a shorter length.

The joint shear mechanism in **Figure 11.2-1** shows the column longitudinal reinforcement being clamped through struts extending from the compression zone of the column and bent cap stirrups placed on both sides of the column. It has been suggested that this clamping occurs over the top 60% of the column rebar extension into the bent cap *(6)* with a maximum bond stress of $30\sqrt{f'_c}$ (psi) for well-confined joint regions. Based on this observation, the minimum anchorage length required is

$$l_a \geq 0.025 \frac{d_b f_{ye}}{\sqrt{f'_c}} \qquad\qquad \text{Eq. 11.2-1}$$

with a 1.2 multiplier for 2-bar bundles, as is the case for this example. Therefore, the required development length of the # 14 longitudinal rebar is:

$$l_a \geq 0.025 \frac{(1.69)(68,000)}{\sqrt{5000}}(1.2) = 48.75 \text{ in}$$

l_a (provided) = 81.5 in.

The joint shear force-transfer mechanism shown in **Figure 11.2-1** is based on 3 compression struts that provide clamping of the column longitudinal reinforcement. The first strut D1 provides clamping of the longitudinal reinforcement making up column tension force T_c' near to the compression zone of the column, and ties into the compression zone of the superstructure. The remaining reinforcement making up T_c is clamped by the compression

struts D2 and D3. The vertical and horizontal components of D2 are resisted by the vertical and horizontal legs of the joint shear stirrups, respectively, while the horizontal component D3 is resisted by the column hoops.

T_c' portion of column tension force clamped by D1.

T_c remaining column tension force clamped by D2 and D3 assumed to be 25% of the total column longitudinal reinforcement.
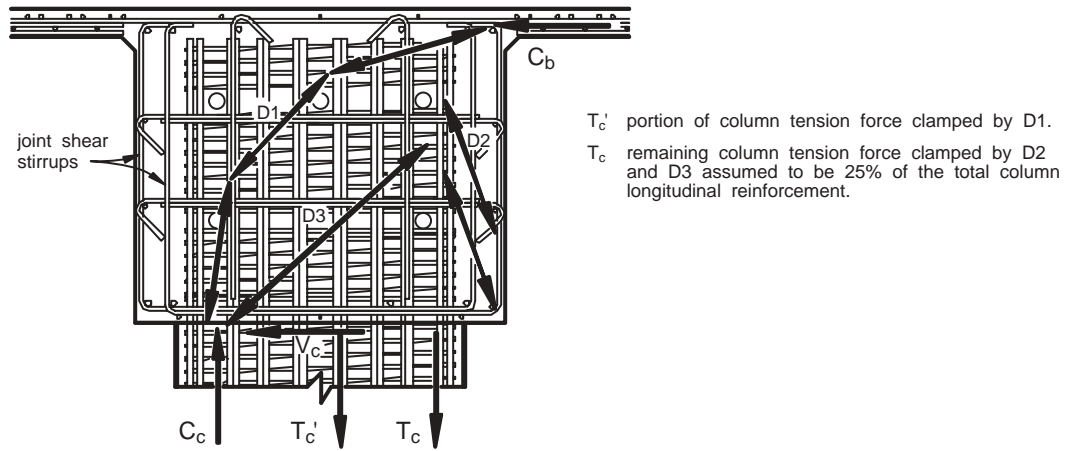
Figure 11.2-1   Joint Shear Force-Transfer Mechanism

There are several references *(6, 7, 8)* that offer prescriptive amounts of reinforcement required for standard joint dimensions assuming circular columns with column reinforcement extending to the top of the joint, and there are slight differences between them. The California Department of Transportation recommends the following *(6)*:

For joint shear reinforcement resisting the horizontal and transverse components of D2,

$$A_S^{JV} = 0.2A_{SC} = 0.2(112.5) = 22.5 \text{ in}^2 \qquad \text{Eq. 11.2-3}$$

$$A_S^{JH} = 0.1A_{SC} = 0.1(112.5) = 11.3 \text{ in}^2 \qquad \text{Eq. 11.2-4}$$

This reinforcement is required on each side of the bent cap and distributed over twice the width of the column.

To control cracking within the column core region, and to prevent buckling and assist in providing bond transfer to the top of the superstructure longitudinal reinforcement, j-bars should extend at least $2/3^{\text{rds}}$ of the height of the bent cap.

$$A_S^{J-bar} = 0.08A_{SC} = 0.08(112.5) = 9.0 \text{ in}^2 \qquad \text{Eq. 11.2-5}$$

The column hoop reinforcement ratio required in the joint region is calculated using the following:

$$\rho_s = 0.4\frac{A_{SC}}{l_a^2} = 0.4\frac{112.5}{(81.5)^2} = 0.0067 \qquad \text{Eq. 11.2-6}$$

However, it has been a general rule by Caltrans that the amount of transverse reinforcement in the joint region should be equal to or greater than amount of transverse reinforcement in the column.

To prevent rebar buckling, it is suggested that:

$$\rho_S \geq 0.0002n \qquad \text{Eq. 11.2-7}$$

where n is the number of bars. Therefore,

$$\rho_S \geq 0.0002\,(50) = 0.010$$

The transverse reinforcement ratio is

$$\rho_s = \frac{4A_h}{D_C s}$$ Eq. 11.2-8

By rearranging terms, the spacing of the column hoops within the column core region can be computed as

$$s \leq \frac{4A_h}{D\rho_s} = \frac{4(0.79)}{(79.0)(0.010)} = 4.00 \text{ in.}$$ Eq. 11.2-9

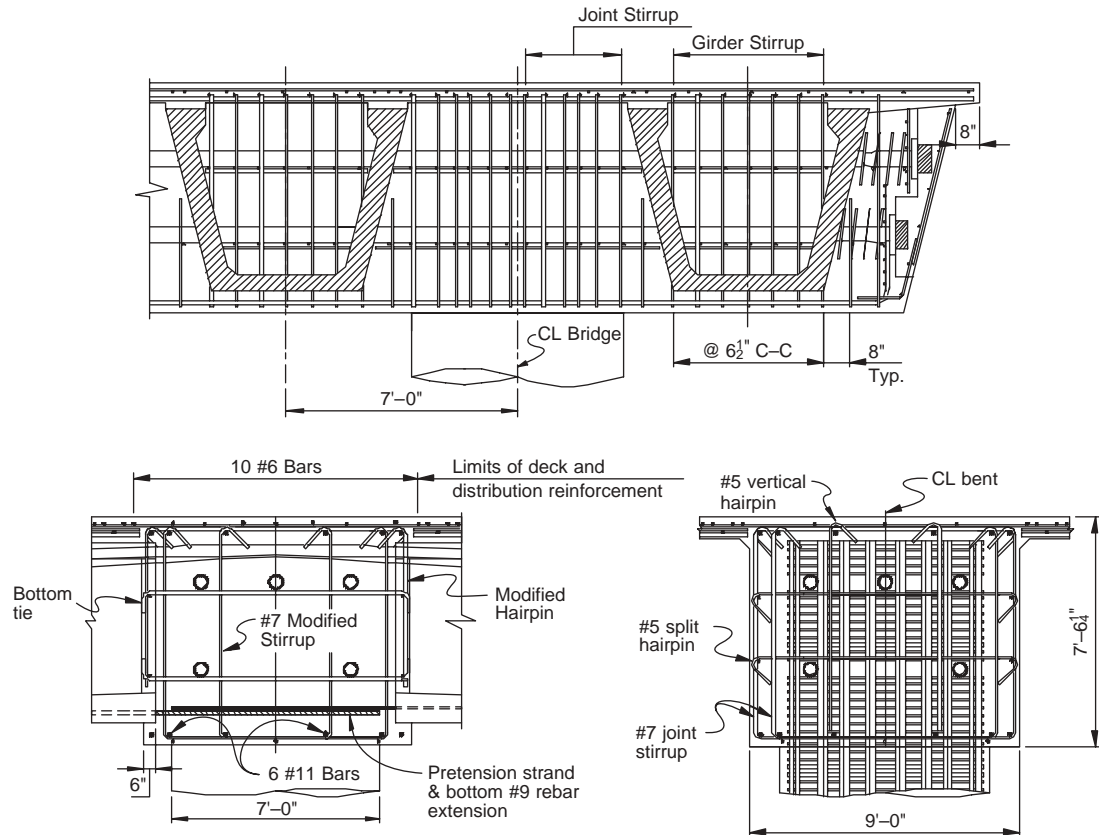The joint shear reinforcement details are shown in **Figure 11.2-2**.

Figure 11.2-2   Bent Cap Joint Reinforcement Details

11.3   BENT CAP TORSION

As mentioned previously, the column plastic moment is resisted by the bent cap through torsional mechanisms. The peak torque Mo^{CG} acts on a small section of bent cap between the column and side of the adjacent girder. This section is too small to form a spiral crack around the perimeter. Hence, a torsion-shear friction mechanism, as developed in reference *(6)* is used to compute the resistance.

Two-Span Spliced U-Beam Girder   **Design Example 2**
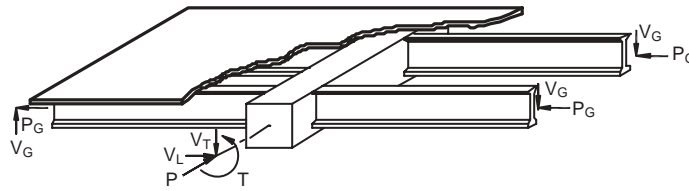
Assumptions:

1. Shearing stress is constant over section
2. Quadrant forces proportional to normal force P and respective area (e.g. $F_4 = A_4 \mu P / A$)
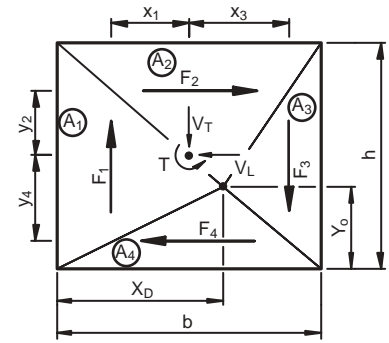
Sectional Force Equilibrium:
$T = F_1 x_1 + F_2 y_2 + F_3 x_3 + F_4 y_4$
$V_T = F_1 - F_3, \; V_L = F_2 - F_4$

Figure 11.3-1   Torsion-Shear Friction Mechanism

The mechanism shown in **Figure 11.3-1** can be described using the sand-heap analogy, where the shear stress has a constant magnitude, or slope in the analogy. However, the section resists not only torque from the plastic hinge but also, dead load shear acting vertically and the column shear acting horizontally. This requires that the apex of the triangle be set of center. The coordinates of this apex are located through trail and error. The spread sheet calculations for developing the coordinates of the apex and the associate required axial load are shown in **Tables 11.3-1** and **11.3-2**.

Table 11.3-1   Parameters for Torsion-Shear Friction Analysis

h	(ft)	7.52
b	(ft)	9
P	(kip)	6000
X_o	(ft)	5.55
Y_o	(ft)	2.93
μ		1.4

Table 11.3-2   Torsion-Shear Friction Analysis

Quad	Area (ft^2)	Force (k)	x_i, y_i	Moment (k-ft)
1	20.87	2590	2.65	6864
2	20.66	2564	2.23	5717
3	12.97	1610	3.35	5394
4	13.19	1636	2.78	4555
Total	67.68		$T_n =$	22528
	$V_v =$	980		
	$V_h =$	927		

D2-43

The torque demand of 21,200 k-ft, the dead load shear of 972 kip and the column shear of 922 kip are less than their respective capacities.

The normal force P on the friction plane consists of two components: the prestress force with all losses and the axial force created by the reinforcement passing through the shear plane. This force can be calculated with the following equation from reference *(7)*.

$$P = P_f + A_s(0.0005)E_s \qquad \text{Eq. 11.3-1}$$

where A_s includes the prestress strand and mild reinforcement passing through the section. A total of 5 tendons with 27 - 0.6 in strands each and 12 # 11 rebar is sufficient to develop the 6,000 kip required axial load. These 5 tendons are required to pass through the column core, and the column hoops have to be adjusted accordingly, where each duct has an outer diameter of 4.5 in.

11.4   SUPERSTRUCTURE DEMANDS

The superstructure seismic moments in Span 1 differ from those in Span 2 due to the unequal span lengths and resulting differences in relative stiffness. The distribution of moments to each span can be solved by comparing the relative stiffness and computing the moments using moment distribution.

$$k_{BA} = \frac{3EI}{L_{AB}}$$

$$k_{BC} = \frac{3EI}{L_{BC}}$$

where $L_{AB} = 165$ ft and $L_{BC} = 150$ ft

$$DF_{AB} = \frac{k_{BA}}{k_{BA} + k_{BC}} = 0.476$$

$$DF_{BC} = 1 - 0.476 = 0.524$$

The superstructure seismic moment at Bent 2 in Spans 1 and 2 are:

$$M_o^{BA} = 42,400(0.476) = 20,200 \text{ k-ft}$$

$$M_o^{BC} = 42,400(0.524) = 22,200 \text{ k-ft}$$

It should be noted that this moments are completely reversible and therefore should be subtracted and added to the D+P/S moments.

As stated previously, the superstructure resistance to column plastic hinging is non-uniform along the width of the bridge. To account for this action, an effective width approximation is used, where the maximum resistance per unit of superstructure width of the actual structure is distributed over an equivalent effective width to provide an equivalent resistance. Structural testing of similar details has shown that the overstrength plastic hinge moment can be resisted equally by the two girders adjacent to the column. *(9)*

$$W_S = 2 \, S_G \qquad \text{Eq. 11.4-1}$$

where S_G is the spacing of the girders.

$$H_S = 6.77 \text{ ft}$$

$$D_C = 7.00 \text{ ft}$$

$$W_S = 2(14.0) = 28.0 \text{ ft}$$

Based on the this effective width, the moment per girder is calculated as

$$M_{(seis)} = (M_o^{BC})S_G/W_S = 14.0/28.0 \ (22,200) = 11,100 \text{ k-ft.}$$ Eq. 11.4-2

where S_G is the girder spacing. The seismic moments can now be added (or subtracted) directly to (or from) the dead load and prestress secondary moments developed in **Section 5**.

12   OTHER DESIGN DETAILS

12.1   SPLICE DETAILS

Two different types of splices are used for the girder segments, one within the span and one at the bent cap. Each type has different concerns or criteria.

For the splices within the span, the cross-section shape will be the same as that within the girder segments. Care must be taken to adequately vibrate the section so that concrete can flow around the P/T ducts. Forming of this section could consist of web forms with the top of the soffit left open, which requires the soffit concrete to briefly set before pouring the webs. The open soffit allows for vibration of the concrete under the P/T ducts.

As shown, no mild reinforcement is required to meet strength requirements for the splices. However, mild reinforcement is required to pass from the girder to the splice for shear requirements (#4 @ 12). With this longitudinal reinforcement, the stirrup spacing at this location is determined using the same criteria as used for the adjacent girder. Special attention must be made to the color and texture of the concrete within the splice to ensure it matches that of the adjacent precast girder. Type III cement (commonly used at precast plants) typically has a lighter color than Type II, (commonly used in cast-in-place construction). Further, the concrete splice must have a 28 day compressive strength of 5,900 psi, as indicated in **Section 8.1.3**.

For the splice at the bent cap, the girder segments are framing into a much larger section. Therefore, the compression at service loads on the bent cap from the girders will reduce a short distance beyond the end of the girder. As a result, the compressive strength required listed in **Table 8.1.3-1** should be based on the flexure design of the bent cap, and not the superstructure. Matching the color and texture of the girder concrete may be a problem. However, since the bent cap is offset from the outside face of the girder, the difference may not be as noticeable.

12.2   HANDLING CONSIDERATIONS – LIFTING LOOP LOCATIONS, ETC.

Special care must be taken when transporting U-beams because the cross-section is subject to bending moments during lifting operations. Lifting loops embedded into the girders extend out of the top of the inclined webs. Horizontal struts are required at the lifting locations to balance the horizontal component of the reaction at the top of the girder.

Girder segment diaphragms can be used to simplify forming of the end diaphragms, bent caps and splices, as required. These diaphragms also can be serve as a horizontal brace during the lifting operations. However, the girder segment diaphragms are typically a secondary pour, and increase the weight of the girder segments. If the lifting loops are set away from the ends of the girder segment, as is the usual case, struts are required in addition to the segment diaphragms. Therefore, provisions for these diaphragms are not a part of this example.

12.3   ANCHORAGE ZONE REINFORCEMENT

The cast-in-place end diaphragm containing the post-tensioning anchorage is wide and deep enough that the bursting forces in these elements do not require a substantial amount of reinforcement in the general zone. Good performance has been observed in cast-in-place box girder bridges using a grid # 4 @ 4 in. both ways behind the prestress bearing plate that extends 6 in. beyond the anchorage recess for each girder. Details of the standards used by the California Department of Transportation for cast-in-place prestressed box girder bridges can be found in reference *(15)*.

Since the prestress anchorage details in this example are not specific to spliced girder bridges, the general zone design is not presented herein. The reader is referred to **Design Example 1** for general zone design.

12.4   **DECK FORMS AND PROVISIONS FOR OVERHANG BRACKETS**

Structural details of how the deck will be formed must be considered during design, and ultimately during construction. Both the contractor and engineer must be aware of the type of connection details that will be used to attach these items. For instance, Texas Department of Transportation has standard details metal corrugated deck forms that include a metal plate that runs continuously and vertically along the top of the girder which allows for a variable build up which ultimately occurs due to camber of the girders. This metal plate is attached to the girders with studs cast into the web.

Provisions must be made so the overhang bracket can attach to the web. Ties are required at or near the overhang brackets to avoid excessive bending moments created by the eccentric load from the deck finishing equipment. These ties should be tied into the transverse bracing in between the girder lines. Ultimately, attachment details for the overhang brackets, ties and struts need to be considered during design and during construction prior to fabrication of the girders.

12.5   **DECK FINISHING EQUIPMENT AND CONSTRUCTION LOADS ON NON-COMPOSITE GIRDER**

Even if the girder is tied in to the end-diaphragms and bent caps, the girder section is flexible due to torsional loading until the deck is placed. Under this condition, the exterior girder can experience substantial rotations due to the weight of the deck finishing machine. A deck finishing machine can develop reactions of up to 2 to 3 tons and the screed rail deflections must be checked under this condition. As mentioned previously, this reaction applies transverse bending moments to the girder. Temporary rods that tie the two webs together are required at each overhang bracket, and provisions for attachment of these rods must be made in both the design and construction phases of the project.

13   OTHER POSSIBLE CONFIGURATIONS

13.1   **ALTERNATE GIRDER CROSS-SECTIONS (BULB TEE GIRDERS)**

Bulb-T girder can be used in place of the U-beam girders using the girder segment layout of **Figure 3-1**, where girder segments terminate at the face of the bent cap. In cross-section, the spacing of the bulb-T girders would be approximately 9 ft, and the number of girders would increase to from 5 to 8. Since the weight of the bulb-T girder section is substantially lower

that the U-beam, the number of girder segments along the length of the bridge can be reduced. This results in a reduction in the number of splices, which reduces the amount of shoring and increases the speed of construction.

13.1.1   Girder Segment Bulb-T Alternative

Reducing the number of segments to 4 results in fewer shoring towers and allows the in-span splices to be located closer to the dead load moment inflection points. Relocating these splices will permit the some of the post-tensioning to be replaced with pretensioning. Further, the allowable tension stress is greater in the girder segment than in the girder splice, thus reducing the amount of post-tensioning required.

13.1.2   Girder Segment Bulb-T Alternative

The configuration requiring the least amount of post-tensioning is a 3-girder segment configuration with one of the three extending continuously through the bent cap extending across the negative bending region of the continuous structure. With this arrangement, maximum negative and positive bending locations fall within a girder segment, thus reducing the post-tensioning, as discussed previously.

All in-span shoring towers could be eliminated with the girder segments extending from the abutments supported by segments over the pier (pier segments) with the use of strong backs. This would require an additional construction step to cast and stress the bent caps prior to setting the span segments. Tie-downs fixed to the end of the pier segments, or at least one shoring tower is required to resist the unbalanced moment during erection of the girders.

13.2   STAGED CONSTRUCTION

The stages of construction used in this example could be altered. A discussion of the consequences of changing the configuration is presented.

13.2.1   100% Post-tensioning After Deck Pour

Applying all post-tensioning after the deck is poured has several advantages over multi-staged post-tensioning including a reduction in the construction schedule and reduced compressive strength. Also, a larger prestress can be applied as pretensioning, resulting in reduced anchorage hardware costs. However, the falsework system would be have to be considerably stronger to support both the weight of the girders and the deck. Loads from the deck finishing equipment would require substantial bracing to resist the overturning moments.

13.2.2   100% Post-tensioning Before Deck Pour

Stressing all post-tensioning prior to placing the deck has the advantage of speeding the construction schedule and mobilization. However, the tendon profiles would have to be adjusted to reduce temporary tension stress at the critical sections, namely the low-points and bent cap locations. Therefore, the tendon profile would not be as efficient, and more prestress is required. Also, this alternative would allow the deck to crack under service loads. Hence, the gross property assumption would not be correct.

13.2.3   **Match-Cast Girder Segments**

A construction staging plan where match casting the girder segments could allow the girders to be set on shoring towers and prestressed to allow removal of these towers in an evening shutdown period of the freeway below. In this plan, the prestress tendons are not continuous over the bent. After placement and stressing, construction is similar to that of a conventional precast girder bridge.

Since the post-tensioning is not continuous over the bent cap, substantially higher positive moments will develop over a continuous system. Also, no tension stress is allowed at a match-cast joint. In terms of seismic design, developing a fixed connection between the superstructure and the column is difficult because the positive seismic moments are not reduced by the dead load. Hence, a substantial amount of reinforcement is required to extend from the girder into the bent cap to develop positive moment resistance.

13.3   **DECK REPLACEMENT**

Deck replacement results in a redistribution of the post-tensioning from the deck to the girders that could result stresses that exceed design limits. Deck replacement design requires careful evaluation of this redistribution and provisions to counter these increases. Temporary external prestress tendons can be used during deck reconstruction to reduce this stress. Provisions for temporary external prestress tendons could be made by casting holes in the bent cap and reinforcing the end diaphragms.

13.4   **INTEGRAL ABUTMENT**

Integral abutments have been shown to result in substantial cost savings because the foundations are designed to resist axial loads and not the overturning moments. Also, the negative moments that develop at the end of the bridge results in a reduction in moment demands in the adjacent spans. *(9)*

Horizontal soil pressures developed behind the integral are resisted as a direct strut to the opposite abutment. The skew at the abutment introduces a horizontal component that is transverse to the longitudinal axis of the bridge. These transverse forces introduce a twisting action on the bridge that has no resistance other than the stiffness of the columns. As a result, undesirable lateral movements in the abutment could develop overtime.

13.5   **POST-TENSIONING ANCHORAGES PLACED IN GIRDER**

Post-tensioning anchorages can be placed in the girder as opposed to placement in the end-diaphragm. This would allow for reduction in the end-diaphragm size and reduce the time required to apply the first stage of post-tensioning. However, the disadvantages include: increased segment weight, because the anchorages do require a girder flare and increased congestion of the reinforcement.

14   REFERENCES

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14. Sritharan, S., Priestley, M.J.N., Seible, F., Seismic Performance of Prestressed Concrete Bridge Joints, *PCI Journal*, V. 44, No. 2, March-April 1999.(14)

15. Caltrans, *Standard Plans*, California Department of Transportation, Sacramento, CA, July 2000.(15)

APPENDIX A

Table A-1   Girder Moments

Node	Coordinate	Self Weight	False-work Release	Deck	SDL	Live Load		Secondary P/S	
						max -ve	max +ve	Stage 1	Stage 2
	(ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)	(k-ft)
		A	B	C	D	E	F	G	H
1	0.0	0	0	0	0	0	0	0	0
2	16.5	362	1,387	1,222	447	-283	2,548	28	135
3	33.0	355	2,674	2,080	761	-565	4,359	55	271
4	49.5	0	3,831	2,573	942	-846	5,468	83	406
5	52.5	103	3,800	2,623	960	-897	5,558	88	431
6	66.0	621	3,416	2,702	989	-1,129	5,948	111	542
7	82.5	827	2,946	2,466	903	-1,410	5,828	139	677
8	99.0	564	2,477	1,866	683	-1,693	5,158	166	813
9	110.5	103	2,149	1,232	451	-1,891	4,305	186	907
10	113.5	0	2,010	1,038	380	-1,940	4,086	191	932
11	132.0	384	-360	-428	-157	-2,260	2,300	222	1,083
12	148.5	263	-2,516	-2,121	-776	-3,376	746	249	1,219
13	160.5	0	-4,202	-3,582	-1,311	-5,134	203	269	1,317
14	165.0	0	-4,986	-4,179	-1,530	-5,771	0	277	1,354
15	165.0	0	-4,986	-4,179	-1,530	-5,767	0	277	1,354
16	169.5	0	-4,254	-3,615	-1,323	-5,143	218	269	1,314
17	172.5	115	-3,901	-3,255	-1,191	-4,707	365	263	1,287
18	180.0	487	-3,170	-2,406	-881	-3,634	737	249	1,219
19	195.0	939	-1,706	-934	-342	-2,850	2,203	222	1,083
20	210.0	1,003	-243	237	87	-2,494	3,639	194	948
21	225.0	680	1,220	1,107	405	-2,137	4,665	166	813
22	237.5	115	2,440	1,602	586	-1,837	5,122	143	700
23	240.5	0	2,672	1,689	618	-1,782	5,215	138	673
24	242.5	0	2,733	1,741	637	-1,714	5,228	134	655
25	245.5	115	2,684	1,808	662	-1,651	5,244	128	628
26	255.0	569	2,338	1,943	711	-1,425	5,291	111	542
27	270.0	970	1,791	1,909	699	-1,069	4,843	83	406
28	285.0	983	1,245	1,574	576	-712	3,851	55	271
29	300.0	608	698	937	343	-357	2,247	28	135
30	310.5	115	315	313	115	-113	670	8	41
31	313.5	0	146	107	39	-39	228	3	14
32	315.0	0	-3	0	0	0	0	0	0

Legend		
Col.	Title	Description
A	Self Weight	Self Weight Moments of Girder Segments spanning Falsework Supports
B	Falsework Release	Release of Falsework Support on Continuous Structure
C	Deck	Fluid Weight of Deck acting on Continuous Structure
D	SDL	Superimposed Dead Load acting on Composite Structure
E & F	Live Load	Maximum Negative and Positive Live Load (HL-93) Moments
G & H	Secondary P/S	Secondary Prestress Moments for Stage 1 and Stage 2 Post-Tensioning

Table A-2   Girder Concrete Stresses Immediately After Deck Pour

Node	Coordinate (ft)	Dead Load (ksf)		Pretension (ksf)		Stage 1 P/T (ksf)		Total (ksf)	
		A		B		C		D	
		Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
1	0.0	0	0	0	0	6	313	6	313
2	16.5	210	-171	0	0	-52	362	158	191
3	33.0	361	-293	0	0	-93	397	268	103
4	49.5	452	-368	0	0	-117	418	335	50
5	52.5	461	-375	-14	37	-120	420	328	82
6	66.0	476	-387	-14	37	-124	425	338	75
7	82.5	440	-358	-14	37	-100	408	327	87
8	99.0	346	-282	-14	37	-30	353	303	108
9	110.5	246	-200	-14	37	48	292	280	129
10	113.5	215	-175	0	0	72	273	287	98
11	132.0	-29	23	0	0	252	125	223	148
12	148.5	-309	251	0	0	459	-46	150	205
13	160.5	-440	334	0	0	423	-117	-16	217
14	165.0	-518	393	0	0	430	-122	-88	271
15	165.0	-518	393	0	0	424	-121	-94	272
16	169.5	-445	337	0	0	413	-114	-31	223
17	172.5	-415	319	-17	35	421	-107	-12	247
18	180.0	-359	292	-17	46	451	-55	75	284
19	195.0	-120	98	-17	46	274	88	137	232
20	210.0	70	-57	-17	46	136	198	189	187
21	225.0	212	-173	-17	46	38	276	233	150
22	237.5	293	-239	-17	46	-14	317	262	124
23	240.5	308	-251	0	0	-23	323	285	73
24	242.5	316	-257	0	0	-27	327	288	70
25	245.5	325	-265	-17	46	-33	331	275	113
26	255.0	342	-279	-17	46	-42	337	283	105
27	270.0	330	-268	-17	46	-41	35	272	113
28	285.0	268	-218	-17	46	-32	327	219	155
29	300.0	158	-129	-17	46	-17	313	125	230
30	310.5	52	-43	-17	46	-2	300	34	304
31	313.5	18	-15	0	0	3	296	21	281
32	315.0	0	0	0	0	6	294	6	294

Legend		
Col.	Title	Description
A	Dead Load	Segment Weight + Falsework Release + Fluid Weight of Deck
B	Pretension	Includes elastic shortening and long term prestress losses up to Day 90
C	Stage 1 P/T	Includes elastic shortening and long term prestress losses up to Day 90
D	Total	Sum of dead load and prestress immediately prior to Stage 2 P/T

Table A-3   Final Girder Component Stresses – Part 1

Node	Coordinate	Dead Load		Pretension		Stage 1 P/T		Stage 2 P/T		
	(ft)	(ksf)		(ksf)		(ksf)		(ksf)		
		A		B		C		D		
		Top	Bottom	Top	Bottom	Top	Bottom	Top Deck	Top	Bottom
1	0.0	0	0	0	0	5	287	82	79	54
2	16.5	210	-171	0	0	-47	332	42	51	118
3	33.0	361	-293	0	0	-85	364	14	31	164
4	49.5	452	-368	0	0	-107	383	-2	20	191
5	52.5	461	-375	-11	31	-110	385	-3	19	194
6	66.0	476	-387	-11	31	-114	390	-5	18	198
7	82.5	440	-358	-11	31	-92	375	3	24	186
8	99.0	346	-282	-11	31	-27	324	22	38	158
9	110.5	246	-200	-11	31	44	268	42	52	127
10	113.5	215	-175	0	0	66	250	48	56	118
11	132.0	-29	23	0	0	231	114	91	86	47
12	148.5	-309	251	0	0	421	-42	141	121	-32
13	160.5	-440	334	0	0	388	-107	144	122	-53
14	165.0	-518	393	0	0	394	-112	145	122	-54
15	165.0	-518	393	0	0	389	-111	143	121	-54
16	169.5	-445	337	0	0	379	-105	141	119	-53
17	172.5	-415	319	-14	29	386	-98	142	119	-52
18	180.0	-359	292	-14	39	414	-51	136	117	-33
19	195.0	-120	98	-14	39	251	81	94	87	32
20	210.0	70	-57	-14	39	125	182	61	63	84
21	225.0	212	-173	-14	39	35	253	36	46	121
22	237.5	293	-239	-14	39	-13	290	22	36	142
23	240.5	308	-251	0	0	-21	296	20	34	146
24	242.5	316	-257	0	0	-25	300	19	33	148
25	245.5	325	-265	-14	39	-31	304	17	32	151
26	255.0	342	-279	-14	39	-39	309	13	29	156
27	270.0	330	-268	-14	39	-37	307	15	30	152
28	285.0	268	-218	-14	39	-29	299	26	38	133
29	300.0	158	-129	-14	39	-15	287	47	53	98
30	310.5	52	-43	-14	39	-2	275	67	67	66
31	313.5	18	-15	0	0	3	271	73	71	55
32	315.0	0	0	0	0	5	269	77	74	50

Legend		
Col.	Title	Description
A	Dead Load	Segment Weight + Falsework Release + Fluid Weight of Deck
B	Pretension	Includes all losses
C	Stage 1 P/T	Includes all losses
D	Stage 2 P/T	Includes all losses

Table A-4   Final Girder Component Stresses – Part II

Node	Coordinate (ft)	SDL (ksf)			Maximum Negative Live Load (ksf)			Maximum Positive Live Load (ksf)		
		A			B			C		
		Top Deck	Top	Bottom	Top Deck	Top	Bottom	Top Deck	Top	Bottom
1	0.0	0	0	0	0	0	0	0	0	0
2	16.5	11	8	-17	-7	-5	11	57	42	-69
3	33.0	18	13	-29	-13	-10	21	97	73	-118
4	49.5	22	16	-36	-20	-14	32	122	91	-148
5	52.5	23	16	-36	-21	-15	34	124	92	-150
6	66.0	24	17	-37	-27	-19	43	132	99	-161
7	82.5	21	15	-34	-34	-24	53	130	97	-157
8	99.0	16	11	-26	-40	-29	64	115	86	-139
9	110.5	11	8	-17	-45	-32	71	96	72	-116
10	113.5	9	6	-14	-46	-33	73	91	68	-110
11	132.0	-4	-3	6	-54	-38	85	51	38	-62
12	148.5	-18	-13	29	-80	-57	127	17	12	-20
13	160.5	-29	-22	35	-114	-85	139	5	3	-6
14	165.0	-34	-25	41	-128	-96	156	0	0	0
15	165.0	-34	-25	41	-128	-96	156	0	0	0
16	169.5	-29	-22	36	-114	-86	139	5	4	-6
17	172.5	-27	-20	34	-106	-78	135	8	6	-11
18	180.0	-21	-15	33	-87	-61	137	16	12	-20
19	195.0	-8	-6	13	-68	-48	108	49	37	-59
20	210.0	2	1	-3	-59	-42	94	81	61	-98
21	225.0	10	7	-15	-51	-36	81	104	78	-126
22	237.5	14	10	-22	-44	-31	69	114	85	-138
23	240.5	15	10	-23	-42	-30	67	116	87	-141
24	242.5	15	11	-24	-41	-29	65	116	87	-141
25	245.5	16	11	-25	-39	-28	62	117	87	-142
26	255.0	17	12	-27	-34	-24	54	118	88	-143
27	270.0	17	12	-26	-26	-18	40	108	81	-131
28	285.0	14	10	-22	-17	-12	27	86	64	-104
29	300.0	8	6	-13	-9	-6	14	50	37	-61
30	310.5	3	2	-4	-3	-2	4	15	11	-18
31	313.5	1	1	-1	-1	-1	2	5	4	-6
32	315.0	0	0	0	0	0	0	0	0	0

Legend		
Col.	Title	Description
A	SDL	Superimposed Dead Load acting on composite structure
B	Live Load (max -ve)	From maximum Live Load Moments that create tension on the top fiber
C	Live Load (max +ve)	From maximum Live Load Moments that create tension on the bottom fiber

Table A-5   Girder Stress: Live Load Combinations

Node	Coordinate (ft)	D + P/S + 0.8LL (ksf) A			D + P/S + LL (ksf) B		
		Top Deck	Top	Bottom	Top Deck	Top	Bottom
1	0.0	82	84	341	341	84	82
2	16.5	48	217	208	273	263	110
3	33.0	22	312	112	227	392	129
4	49.5	5	370	53	203	472	142
5	52.5	3	363	79	233	468	143
6	66.0	-3	370	66	237	484	151
7	82.5	-2	358	74	253	473	155
8	99.0	6	334	94	269	443	153
9	110.5	17	312	116	280	409	148
10	113.5	20	317	91	252	411	148
11	132.0	44	255	141	276	324	139
12	148.5	58	174	190	333	232	139
13	160.5	24	-20	204	347	52	119
14	165.0	8	-104	268	424	-27	111
15	165.0	6	-111	269	425	-34	109
16	169.5	20	-37	211	354	35	117
17	172.5	30	-7	224	368	62	123
18	180.0	46	93	265	418	155	131
19	195.0	31	160	215	370	234	135
20	210.0	15	212	165	338	306	144
21	225.0	5	257	125	306	363	150
22	237.5	1	287	100	280	397	150
23	240.5	1	308	56	235	419	151
24	242.5	1	312	53	231	422	150
25	245.5	1	301	90	266	411	149
26	255.0	3	312	84	252	419	149
27	270.0	11	306	98	243	401	139
28	285.0	26	263	148	258	337	125
29	300.0	48	182	234	296	225	105
30	310.5	67	104	318	337	116	84
31	313.5	73	92	306	312	96	79
32	315.0	77	78	319	319	78	77

Legend		
Col.	Title	Description
A	D + P/S + 0.8LL	Load combination used to determine Stage 2 P/T force
B	D + P/S + LL	Load combination for developing compressive strength

Table A-6   Girder Stress: Live Load Combination and D + P/S

Node	Coordinate (ft)	½(D + P/S) + LL (ksf) A			Total D + P/S (ksf) B			
		Top Deck	Top	Bottom	Top Deck	Bottom Deck	Top	Bottom
1	0.0	41	42	170	82	79	84	341
2	16.5	83	153	142	53	59	221	263
3	33.0	113	232	124	32	44	320	206
4	49.5	132	281	117	21	36	381	171
5	52.5	133	280	133	20	36	375	199
6	66.0	142	292	140	19	35	386	195
7	82.5	142	285	153	25	39	377	199
8	99.0	134	264	166	39	49	357	205
9	110.5	122	240	176	53	59	337	209
10	113.5	119	239	163	57	62	343	179
11	132.0	95	181	180	87	84	286	190
12	148.5	78	122	230	122	108	220	206
13	160.5	62	27	243	115	100	48	209
14	165.0	55	-14	290	111	97	-27	268
15	165.0	54	-17	290	109	95	-34	269
16	169.5	61	19	247	112	97	32	216
17	172.5	66	34	251	115	100	56	232
18	180.0	74	83	277	115	102	142	281
19	195.0	92	136	239	86	81	198	263
20	210.0	112	183	216	63	65	245	244
21	225.0	127	220	193	46	53	285	225
22	237.5	132	241	174	36	46	312	210
23	240.5	133	253	151	35	45	332	168
24	242.5	133	254	148	34	44	335	166
25	245.5	133	249	164	32	43	324	203
26	255.0	133	253	153	30	41	331	198
27	270.0	123	241	142	31	42	320	203
28	285.0	105	200	142	40	48	272	231
29	300.0	77	131	155	55	58	187	282
30	310.5	50	64	171	69	69	105	333
31	313.5	42	50	157	74	72	92	311
32	315.0	28	39	160	77	74	78	319

Legend		
Col.	Title	Description
A	½ (D + P/S) + LL	Load combination for developing compressive strength
B	Total D + P/S	Sum of dead load and prestress in final service conditions

Table A-7   Girder Deflections

Node	Coordinate	Post-Tensioning		False-work Release	Deck	SDL	Total Deflection			
		Stage 1	Stage 2				(1)	(2)	(3)	(4)
	(ft)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
		A	B	C	D	E	F	G	H	I
1	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	16.5	1.64	0.48	1.15	0.87	0.14	0.40	-0.47	-0.05	-0.31
3	33.0	2.91	0.89	2.16	1.61	0.25	0.60	-1.01	-0.22	-0.69
4	49.5	3.74	1.18	2.87	2.13	0.33	0.68	-1.45	-0.39	-1.01
5	52.5	3.84	1.22	2.96	2.20	0.34	0.69	-1.51	-0.42	-1.06
6	66.0	4.09	1.32	3.19	2.37	0.37	0.70	-1.67	-0.50	-1.19
7	82.5	3.96	1.28	3.14	2.33	0.36	0.63	-1.70	-0.55	-1.22
8	99.0	3.38	1.10	2.76	2.01	0.31	0.46	-1.55	-0.57	-1.15
9	110.5	2.77	0.90	2.33	1.67	0.26	0.30	-1.36	-0.56	-1.03
10	113.5	2.59	0.84	2.20	1.56	0.24	0.27	-1.30	-0.55	-0.99
11	132.0	1.42	0.46	1.29	0.88	0.14	0.06	-0.82	-0.41	-0.66
12	148.5	0.49	0.16	0.50	0.32	0.05	-0.04	-0.36	-0.22	-0.31
13	160.5	0.08	0.02	0.10	0.05	0.01	-0.02	-0.08	-0.06	-0.07
14	165.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15	165.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	169.5	-0.03	-0.01	-0.06	-0.02	0.00	0.03	0.05	0.04	0.05
17	172.5	-0.02	-0.01	-0.07	-0.02	0.00	0.05	0.07	0.07	0.07
18	180.0	0.07	0.03	-0.06	0.04	0.01	0.13	0.09	0.12	0.11
19	195.0	0.52	0.17	0.17	0.29	0.05	0.33	0.03	0.19	0.10
20	210.0	1.11	0.37	0.55	0.63	0.10	0.51	-0.13	0.20	0.01
21	225.0	1.66	0.54	0.95	0.96	0.15	0.63	-0.33	0.16	-0.12
22	237.5	2.00	0.65	1.21	1.16	0.18	0.69	-0.47	0.12	-0.22
23	240.5	2.06	0.67	1.25	1.19	0.19	0.70	-0.49	0.11	-0.24
24	242.5	2.09	0.68	1.28	1.21	0.19	0.71	-0.50	0.11	-0.24
25	245.5	2.13	0.69	1.30	1.23	0.19	0.72	-0.51	0.11	-0.25
26	255.0	2.18	0.70	1.32	1.26	0.20	0.75	-0.51	0.12	-0.25
27	270.0	2.04	0.64	1.19	1.16	0.18	0.75	-0.41	0.16	-0.18
28	285.0	1.62	0.49	0.89	0.89	0.14	0.65	-0.24	0.19	-0.07
29	300.0	0.93	0.26	0.48	0.49	0.08	0.41	-0.08	0.15	0.01
30	310.5	0.30	0.08	0.15	0.15	0.02	0.14	-0.01	0.06	0.02
31	313.5	0.10	0.03	0.05	0.05	0.01	0.05	0.00	0.02	0.01
32	315.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Legend		
Col.	Title	Description
A	P/T Stage 1	Stage 1 post-tensioning with no losses on non-composite continuous structure
B	P/T Stage 2	Stage 2 post-tensioning with no losses on composite continuous structure
C	Falsework	Effect of releasing falsework supports on continuous girder
D	Deck	Fluid weight of deck acting on non-composite continuous girder
E	SDL	Super imposed dead load acting on composite continuous girder
F	(1)	Total deflection after Stage 1 P/T (falsework release + Stage 1 P/T)
G	(2)	Total deflection after placement of the deck
H	(3)	Total deflection after Stage 2 P/T
I	(4)	Final service load deflection (includes all prestress losses and SDL)

DESIGN EXAMPLE 3: CONTINUOUS THREE SPAN GIRDER

Table of Contents

1   **INTRODUCTION**

The bridge in this example represents a typical high-level Intracoastal Waterway crossing. The bridge consists of five sections of prestressed concrete girders that are spliced with post-tensioning to create a three-span continuous unit. To date, the longest span for this type of girder unit is 320 feet, with center spans of 200 to 250 feet having been used on a number of bridges. From discussions with prestressed girder manufacturers and contractors involved in these bridges, the 320 foot span appears to be the maximum span possible using standard construction equipment for girder handling, transportation and erection.

This three-span unit design example uses a center span of 280 feet. This span length falls within the range of existing structures and will provide a reasonable example of a structure that is realistically constructible. The same structure type may also be used over wide interchanges, multiple railroad tracks and similar locations.

Bridges of this type have been bid competitively in many instances against alternate designs using structural steel plate girder or precast segmental concrete boxes erected by the balanced cantilever method. The bridge with the 320 foot span was proposed by the contractor as an alternate to the original steel plate girder design after the contract was awarded. The contractor shared a $400,000 savings with the owner through a Value Engineering Change Proposal.

The design was performed using a commercially available computer program. The program utilizes time-dependent creep and shrinkage methodology for an accurate estimation of prestress losses, effects of differential creep and shrinkage between the girders and the deck slab, and camber and deflection effects.

Due to the complexity of this type design, a designer and an owner should consider the use of a time dependent computer program as mandatory. Several such programs are currently available. This design example demonstrates what input data has to be developed in order to use these programs, but it does not repeat the voluminous output that is generated. Rather, graphs and charts are used to present the results.

2   **DESCRIPTION OF BRIDGE**

The cross-section for the design example is based on the typical Intracoastal Waterway crossing. This section consists of two 12'-0" wide travel lanes, one in each direction, with a 10'-0" wide shoulder on each side. Therefore with a 1'-6" wide traffic barrier on each side, the out-to-out width of the bridge is 47'-0". The bridge is designed for a total of three traffic lanes in accordance with AASHTO design criteria. The bridge length is taken to be on a tangent alignment with no horizontal curvature.

The bridge cross-section uses five girder lines. The girders are spaced at 9'-6" on center with 4'-6" overhangs each side. This spacing and overhang configuration result in optimum distribution of dead load and live load to the interior and exterior girders. Therefore, all girders have the same design requirements. Optimizing the girder spacing is an important part of the preliminary design of this structure type.

The three-span unit uses concrete diaphragms at the piers but no intermediate diaphragms. Temporary intermediate diaphragms of structural steel shapes are used at about 40 feet on center during erection and during casting of the bridge deck slab. These temporary diaphragms provide lateral bracing of the compression flange and are designed to resist a

Continuous Three Span Girder   **Design Example 3**

minimum of 5% of the compression force in the flange, similar to lateral bracing of a column. After the deck slab has been cast and cured, the temporary diaphragms are removed.

If the bridge were on an alignment with horizontal curvature, permanent concrete intermediate diaphragms at the closure pour locations may be required to resist "kinks" at the post-tensioning tendons at those locations. However, experience with bridges constructed has proven that permanent intermediate diaphragms are otherwise not required nor or beneficial for stability or live load distribution.

The design example uses a center span (Span 2) length of 280 feet. This is a fixed parameter based on the required horizontal clearance of the main pier locations. For a waterway crossing, the clearance is based on the minimum navigational channel width required by the U.S. Coast Guard for the fender system and the offset of the centerline of piers from the fender system. For other locations, the main span length may be determined by clearances needed over a roadway corridor or a railroad right-of-way. Also, the main span length may be affected by the cost of foundations due to ship impact or extremely tall piers.

The end spans (Spans 1 and 3) used in the example are 210 feet each, or 0.75 of the main span. Existing bridges of this type have used end spans ranging from about 0.70 to 0.85 of the main span, the differences being due to end span pier location restrictions or preferences of the designers. The ratio of 0.75 used is based on the researchers' personal experiences with this type structure and on the culmination of research done for this report. The layout and cross-sections of the five sections of concrete girder to create the three-span continuous unit are shown in **Figures 2-1**, **2-2** and **2-3**.
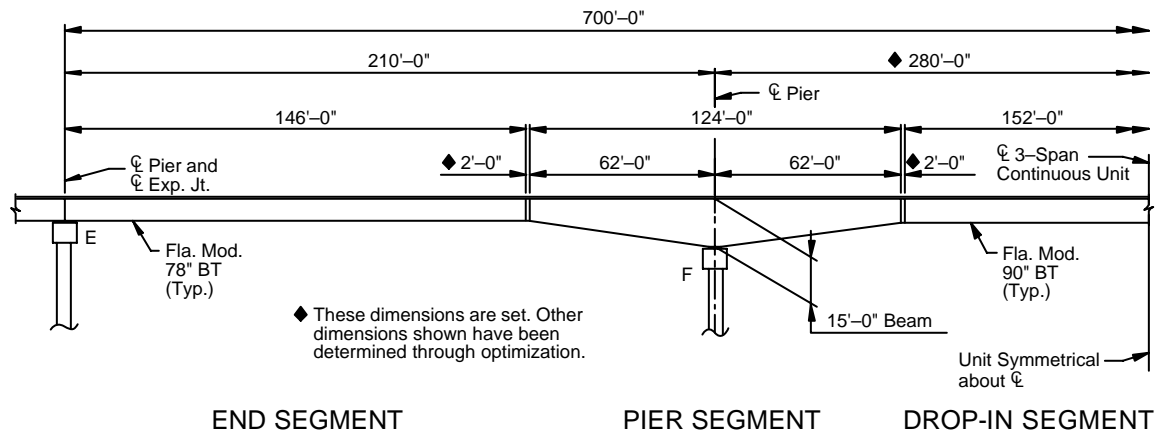
Figure 2-1   **Half Elevation of Bridge Girder Layout**

Continuous Three Span Girder

Design Example 3
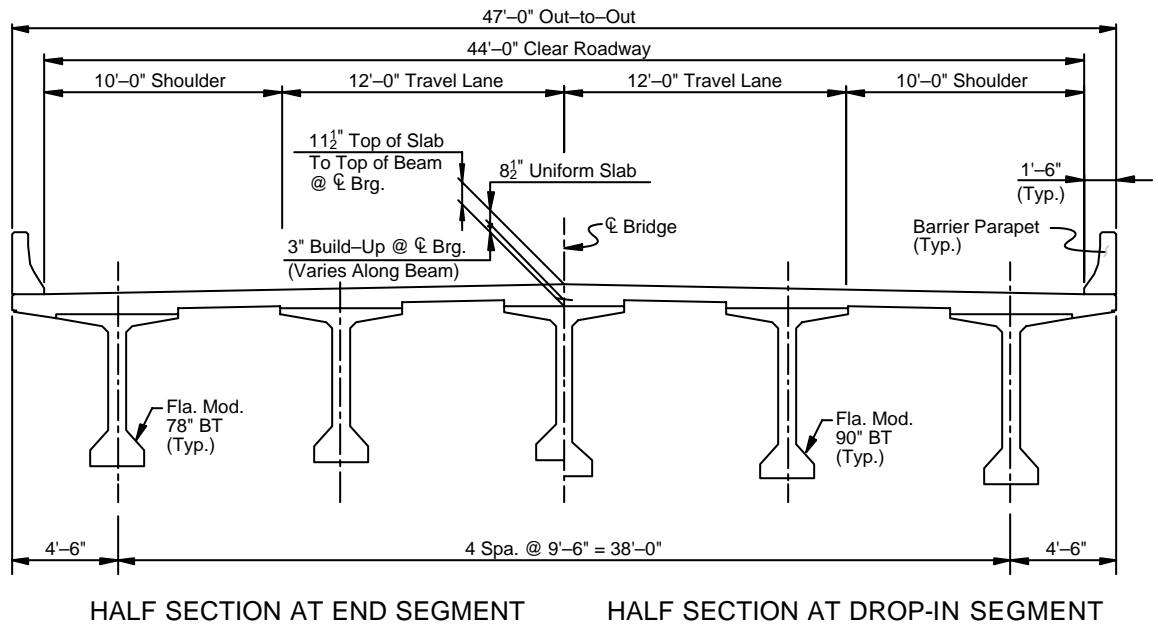
Figure 2-2 Typical Section for Constant Depth Girders

47'–0" Out–to–Out
44'–0" Clear Roadway
10'–0" Shoulder 12'–0" Travel Lane 12'–0" Travel Lane 10'–0" Shoulder

$11\frac{1}{2}$" Top of Slab To Top of Beam @ ℄ Brg.
$8\frac{1}{2}$" Uniform Slab
℄ Bridge
1'–6" (Typ.)
Barrier Parapet (Typ.)
3" Build–Up @ ℄ Brg. (Varies Along Beam)

Fla. Mod. 78" BT (Typ.)
Fla. Mod. 90" BT (Typ.)

4'–6" 4 Spa. @ 9'–6" = 38'–0" 4'–6"

HALF SECTION AT END SEGMENT   HALF SECTION AT DROP-IN SEGMENT
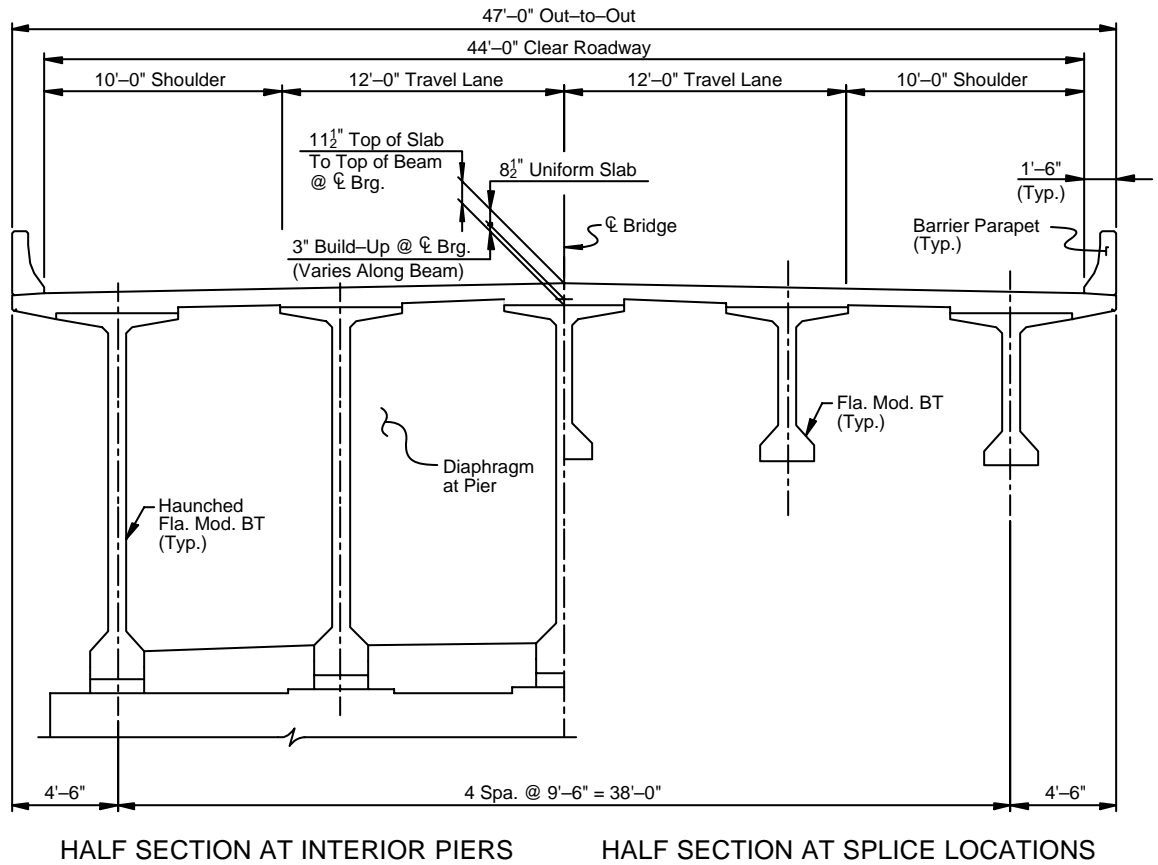
Figure 2-3 Sections at Critical Locations

47'–0" Out–to–Out
44'–0" Clear Roadway
10'–0" Shoulder 12'–0" Travel Lane 12'–0" Travel Lane 10'–0" Shoulder

$11\frac{1}{2}$" Top of Slab To Top of Beam @ ℄ Brg.
$8\frac{1}{2}$" Uniform Slab
℄ Bridge
1'–6" (Typ.)
Barrier Parapet (Typ.)
3" Build–Up @ ℄ Brg. (Varies Along Beam)

Diaphragm at Pier
Fla. Mod. BT (Typ.)
Haunched Fla. Mod. BT (Typ.)

4'–6" 4 Spa. @ 9'–6" = 38'–0" 4'–6"

HALF SECTION AT INTERIOR PIERS   HALF SECTION AT SPLICE LOCATIONS

D3-3

The girders are made continuous by casting a splice between the ends of the girder segments. Post-tensioning is performed in two stages: Stage 1 prior to casting of the deck slab and Stage 2 after casting the deck slab. Therefore, the girders are considered continuous for the deck slab dead load and for all loads applied to the composite section.

The following design example demonstrates the design of an interior girder. Design of an exterior girder would be similar except for loads. For this bridge, the design of the interior girder governs slightly. With proper determination of the optimum girder spacing and overhang length, it is possible that one design can suffice for all girders.

3   DESIGN ASSUMPTIONS AND COMPUTER MODELING

The following basic assumptions are used for the design example:

1. The roadway section, girder cross-sections and span lengths are established as shown in **Figures 2-1, 2-2 and 2-3**.

2. Girder segments are erected on permanent piers, on temporary shoring towers, and using temporary "strongback" hanger systems. Temporary intermediate diaphragms of structural steel shapes are used during erection and during casting of the deck slab.

3. After the girders are spliced, Stage 1 post-tensioning tendons are stressed. The temporary shoring and hanger systems are then removed (or towers are detached and left in-place until later). The deck slab is then poured and Stage 2 post-tensioning tendons are stressed after the deck slab has reached the required strength.

4. Girder segments are pretensioned for stresses mainly from handling and erection but the pretensioning is also part of the net compression provided with the post-tensioning. A minimum criterion for pretensioning is to provide sufficient force in order to resist the girder weight with a 50% impact factor. Also, the use of greater pretensioning can reduce the amount of post-tensioning required.

5. The 28-day concrete strength of the girders is taken as 8,500 psi. The 28-day concrete strength of the bridge deck slab concrete is taken as 4,500 psi.

6. Post-tensioning tendons are placed in a single vertical plane. Experience has shown that use of a 9 in. web width for the girders is optimum. Therefore, 4 in. diameter galvanized corrugated metal ducts are used in order to provide adequate placement tolerances during girder fabrication and for corrosion protection. Post-tensioning tendons may contain up to (19) – 0.60 in. diameter strands for this duct size. It is anticipated that four tendons will be used.

7. Post-tensioning tendon anchorages are contained within end blocks in the end girder segments at the ends of each line of girders (i.e., at the ends of the three span unit). The end blocks are the full width of the bottom flange (2'-6") and extend 16'-0" from the end with an additional 4'-0" transition length to the 9 in. web width. Typical end block lengths have ranged from 1.5 to 2.5 times the girder depth.

8. Post-tensioning tendons are stressed from both ends during Stage 1 and during Stage 2. Even though design may be accomplished using single-end stressing, stressing from both ends is recommended to provide symmetry of stresses in the structure. Also, the additional labor cost of stressing at the second end is nominal.

9. Post-tensioning ducts are grouted after tendons are stressed to provide for corrosion protection and to improve performance.

10. The width of the splice between girder segments is 2'-0". Post-tensioning tendon ducts will be spliced with a separate splice section between the ends of the ducts projecting from the girder segments.

11. A standard composite reinforced concrete deck is used. The deck is 8½ in. thick, including ½ in. as a sacrificial thickness to be removed by planing if required for bridge deck smoothness and "ride-ability". The ½ in. sacrificial thickness is not considered part of the structural composite section and is treated as additional superimposed dead load. Composite section properties are based on the transformed effective width of the composite deck slab based on AASHTO requirements and the different modulus of elasticity values for the girders and the deck slab.

12. A 3 in. concrete build-up is specified at pier locations between the top of girders and the bottom of the deck slab. A 1 in. minimum build-up is used to accommodate construction tolerances and variation in camber.

13. Girders are set on laminated neoprene/steel plate elastomeric bearings at all permanent bearing locations. Bearings at the ends of the spliced girder unit will have stainless steel/Teflon-coated sliding surfaces to allow for girder elastic shortening and creep due to post-tensioning and for shrinkage.

14. The sign convention for this design is opposite of that utilized in **Design Example 1** and **2**. Tensile stresses are considered positive and compression is negative. This confirms to the sign convention of the computer program used.

3.1   SEQUENCE OF CONSTRUCTION

The sequence of construction, with estimated age of the girders at each stage, is shown in **Table 3.1-1** and illustrated in **Figure 3.1-1**.

Table 3.1-1   Sequence of Construction

Stage	Description	Girder Age (Days)
0	Precast girders	1
1	Erect girders on piers and temporary falsework and using strongbacks	60
2	Cast splices	70
3	Stress post-tensioning Tendons 1 and 2 (PT Stage 1)	84
4	Cast deck slab	98
5	Stress post-tensioning Tendons 3 and 4 (PT Stage 2)	119
6	Cast traffic barriers	133
7	Add live load (HL-93) with dynamic allowance	161
8	Add future wearing surface (FWS)	4,161
9	End of Service (i.e., "Final Conditions After All Losses")	27,500

The final design stage is taken at the time that all losses, creep and shrinkage have occurred (effective prestress will be at its minimum value), and all dead loads will be present on the structure. Stresses at intermediate stages must be checked in detail, as construction loads during erection can greatly affect both stresses during erection and at the final stage. The temporary supporting of the end and drop-in girder segments on the ends of the haunched pier girder segments has a significant effect on the pier girder segments and the negative moment region in general.

Erect Piers and Temporary Supports
Erect Pier Girder Segments [Stage 1]

Sym. about
Center of
Bridge

Erect End Girder Segments [Stage 1]

Erect Drop-in Girder Segments [Stage 1]

Cast Splices [Stage 2]
Stress Stage 1 Tendons [Stage 3]

Cast Deck Slab [Stage 4]
Stress Stage 2 Tendons [Stage 5]

Cast Barriers/Parapets [Stage 6]
Open Bridge to Traffic [Stage 7]
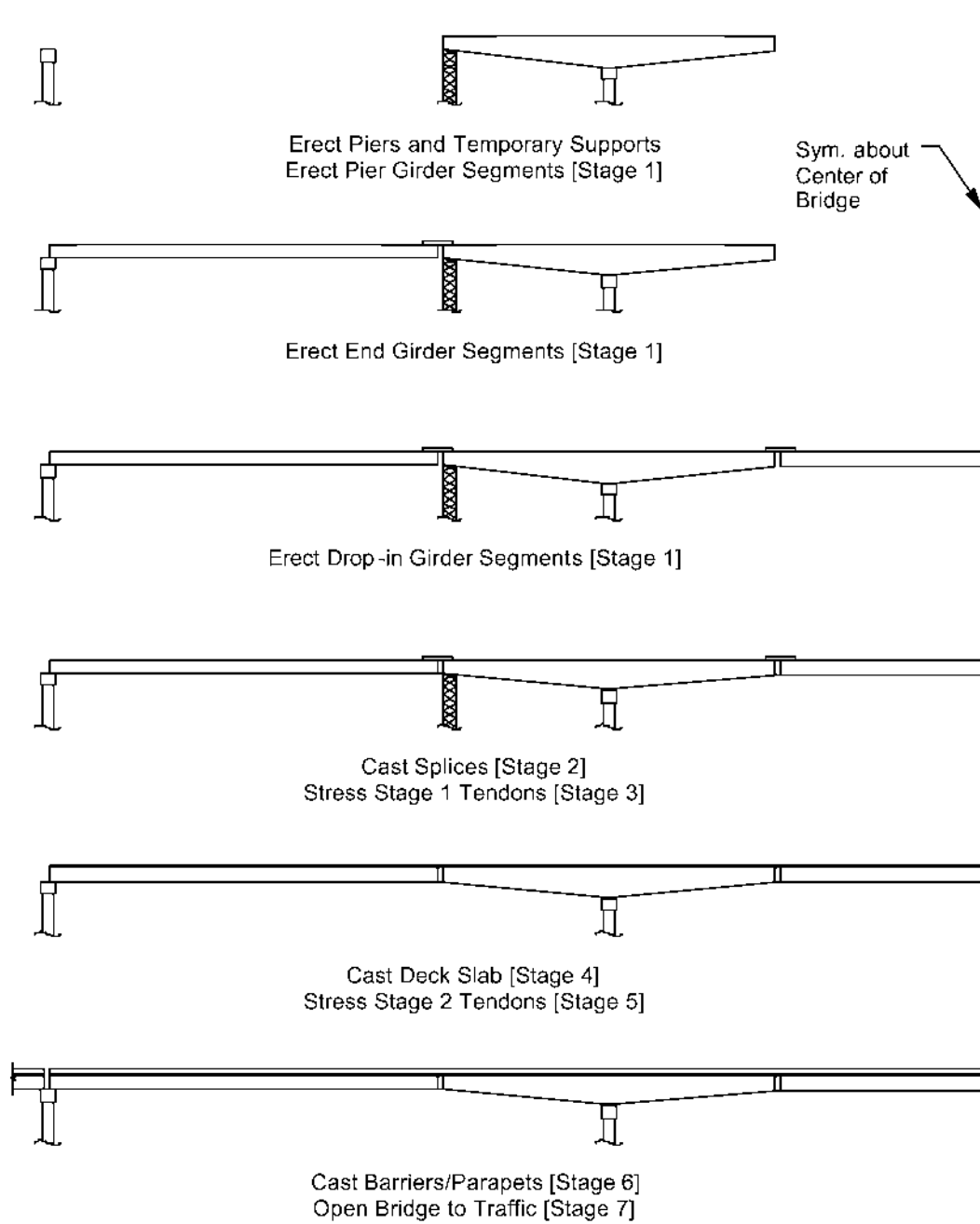
Figure 3.1-1   Stages of Construction

3.2   EFFECT OF CONSTRUCTION SEQUENCE

The sequence of construction has a significant effect on the design and behavior of the bridge. The construction sequencing noted above is based on the structure being built at a rapid pace. This results in the "worst case scenario" for the effects on pretensioning and post-tensioning losses, creep and shrinkage. If a longer time period is assumed between girder segment fabrication and casting of the deck, the effect from these important design parameters is reduced.

It is recommended that designers assume a similar construction sequence for the base design. It is also recommended that, after establishing the design for this sequence, the designer increase the time periods in order to obtain an envelope of the results and to be able to review the effects of the change in construction schedule. This has been of value during construction of existing bridges when design issues arose when the construction time varied from the sequence assumed in the contract plans.

3.3   COMPUTER MODELING

Several software packages are available for the design of post-tensioned spliced concrete girders. Each software package has advantages and disadvantages based on the assumption and opinions of the computer programmer. Before using any software package, the theory behind the analysis performed by the software and how to utilize the voluminous output generated must be understood. The software package selected for this design example was based on many assumptions by the software developer and required some discussion with the technical support to fully understand how to correctly model the bridge.

In addition to the basic input required by all prestressed concrete bridge design programs, such as geometry and material properties, this software package required additional input to analyze staged construction and time-dependent effects. This included the sequence of construction stages, methods of applying loading, post-tensioning layouts, and other similar items.

This software package required construction stages in addition to those shown in **Table 3.1-1** for the analysis to be accurately performed. **Table 3.3-1** shows the 22 stages required for the analysis. The additional stages are shown in italics. A load stage with a duration of 0 days was used whenever a load was applied. The load stage numbers from **Table 3.1-1**, which are used in discussion of results, are shown in the last column of **Table 3.3-1**.

Additional consideration for a designer before creating a model can include the following:

- The number and locations of analysis sections
- The process of applying loads in addition to member self weight
- Ease of modifying a model

All software packages can include errors, inconsistencies, or confusing issues in the programming, and the logic behind the analysis, input and output. The designer must verify the results of the software before using it for design.

Table 3.3-1   Construction Schedule for Computer Model (C.M.)

C.M. Stage	Action	Start Time (Days)	Duration (Days)	Girder Age (Days)	Loads Applied in Addition to Member Self Weight	DE 3 Stage (Days)
0	Release of Pretensioning Strands	----	----	1		0
1	Erect Pier Segments	0	0	60		
2	Erect End Segments	0	0	60		1
3	Erect Drop-in Segment	0	0	60		
4	Time Step: Form Splice	0	10	----		
5	Cast Splice	10	0	70		2
6	Time Step: Cure Splice	10	14	----		
7	PT Stage 1: Tension Tendons 1 and 2	24	0	84	Remove Temporary Support and Strongbacks	3
8	Time Step: Form Deck	24	14	----	Temporary Construction Load	
9	Cast Deck	38	0	98	Add Build-up and Sacrificial Wearing Surface Load	4
10	Time Step: Cure Deck	38	21	----	Remove Temporary Construction Load	
11	PT Stage 2: Tension Tendons 3 and 4	59	0	119		5
12	Time Step: Form Barriers/Parapets	59	14	----		
13	Cast Barriers/Parapets	73	0	133	Add Barrier Loading	6
14	Time Step: Cure Barriers/Parapets	73	28	---		
15	Open Bridge to Traffic	101	0	161	Add Live Load	7
16	Time Step	101	4000	----		
17	Apply Future Wearing Surface	4,101	0	4,161	Add Future Wearing Surface (FWS)	8
18	Time Step	4,101	6000	----		
19	Time Step	10,101	6000	----		
20	Time Step	16,101	6000	----		
21	Time Step	22,101	5399	----		
22	Service Life of Bridge	27,500	0	27,560		9

3.4　　**OTHER DESIGN ASSUMPTIONS**

Intermediate diaphragms are not used in the bridge. Temporary steel cross-frame diaphragms are required during erection to stabilize the girders and to provide lateral restraint of the compression flange. The weight of these temporary components is minor and is neglected in these calculations.

For simplicity, the full thickness of the build-up is applied to the full length of the girder for dead load computations. In most design situations, a value is used that is less than the specified build-up thickness at the center of the bearings because the actual thickness varies along the length of the girders.

The build-up is neglected when computing composite section properties that are used to calculate stresses for service limit state design, since the build-up will vary along the bridge. However, for computation of section properties and strength calculations related to the reinforcement at the pier diaphragms, the build-up is included. This is done because the build-up is specified at the center of bearings, so the full build-up will be provided at the pier diaphragm locations.

The design of this example provides for a "crack free" deck slab by limiting the tensile stress in the top of the deck slab to that allowed by the *AASHTO LRFD Specifications* for moderate corrosion conditions. The design can also be performed to provide for a "zero tension" condition in the deck slab by slightly increasing the post-tensioning effect from the Stage 2 tendons, by increasing the number of strands in the tendons, and/or by adjusting the eccentricities of the tendons.

4　　**PRELIMINARY DESIGN**

Several aspects of the bridge girder layout and girder cross-section are very important to be determined accurately in the preliminary design process. As noted previously, the main span length is a fixed parameter that all other factors are predicated upon. The main purpose of the preliminary design process is to predict the optimum end span lengths, the haunched pier segment depth and the optimum post-tensioning tendon size, the number of tendons and the configuration.

A preliminary frame analysis of the continuous girder is advantageous to determine the optimum locations of inflection points and points of maximum positive moments. Maximum negative moments are, of course, located at the centerline of the interior piers. The objective is to determine a balanced condition of the structure under dead loads of the completed three-span girder and the deck slab. As these dead loads represent about 70% of the total loads, the effects of the live load and superimposed dead loads on the locations of inflection points and maximum positive moments are minor.

The cross-section of the girder at the locations of maximum positive moment is usually predicated on existing girder sections available in the vicinity of the bridge location. Throughout the United States and other countries, prestressed concrete "Bulb Tee" girder sections have found significant use during the past two decades. Most Departments of Transportation have developed standard sections or have adopted standard sections developed by national organizations such as AASHTO and the Precast/Prestressed Concrete Institute (PCI).

It is recommended that designers of this type of structure contact the client, local contractors and prestressed concrete firms to investigate the existence of girder sections that are most

economical for the bridge location. If desirable sections are not available in the area, designers may benefit from considering other existing sections described in this report.

The cross-section of the haunched pier segment girders is then developed from the use of the constant depth girder section used at the positive moment locations. The depth of the girder at the interior pier is usually the last aspect of the optimization effort. The determination of the post-tensioning tendon size affects the width required for the web of all girders. The use of a very large tendon size can be a disadvantage as it would result in a large web for all girders and increase the structure weight unnecessarily. Therefore, it is best to first decide upon the tendon size to be used and then to determine the optimum depth of the girder at the interior piers.

4.1 **GIRDER TYPE SELECTION**

1. As noted previously, designers should investigate available girder cross-sections in the vicinity of the project site.

2. The design example uses the cross-section of the Florida Department of Transportation Bulb-Tee girders. These sections have been used on numerous post-tensioned spliced girder bridges with great success. These girders have been used throughout the Southeast, so forms are readily available.

3. The girder web width is increased to 9 in. by spreading the standard girder side forms. This web width is required to accommodate 4 in. diameter post-tensioning ducts. The width of the top and bottom flanges also increase by 2 in. (i.e., a 5'-2" wide top flange and a 2'-6" wide bottom flange).

4.2 **SPLICE LOCATION**

Splices are located at the joints between girder sections (See **Figure 2-1**). Splice locations have varied on projects built to date. It is important to determine the best possible location for each project. Major items of consideration are the location of inflection points under total dead loads and attempting to balance the weight of the individual girders.

4.3 **ESTIMATE NUMBER AND SPACING OF GIRDERS**

The optimum girder spacing and overhang dimensions must be determined accurately, as discussed in **Section 2** previously.

4.4 **TYPICAL SECTION AND PRELIMINARY POST-TENSIONING LAYOUT**

The steps to be considered in preliminary design can be summarized as:

1. Determine girder section properties

2. Determine girder design loads

3. Determine loading from removal of temporary shoring tower

4. Set tendon locations at ends (in girder end blocks) considering anchorage dimensions

5. Set tendon profiles

6. Estimate prestress losses

5   BASIC DESIGN INFORMATION

5.1   GOVERNING SPECIFICATIONS

AASHTO LRFD Bridge Design Specifications, 2nd Edition with Interims through 2003 *(1)*. References to articles, equations and tables in these *Specifications* will be preceded by the prefix "LRFD" to differentiate these references from references to items in this design example.

5.2   LOADS

Live Load	HL-93 with 33% dynamic allowance (IM) on the design truck. Live load distribution factors are computed using equations in LRFD Table 4.6.2.2.2b-1 for section type (k) (see LRFD Table 4.6.2.2.1-1):
Girder Self Weight	The unit weight of girder concrete is 0.150 kcf.
Deck Slab (structural)	The deck slab has a structural thickness of 8 in. (see **Section 3.1.10**), with a unit weight of 0.150 kcf. = 0.1005 ksf on the tributary area for girders
Wt. of Add'l Deck Thk.	The additional deck slab thickness is ½ in. (see **Section 3.1.10**). = 0.0062 ksf on the tributary area for girders
3 in. Build-up	The full build-up thickness of 3 in. is used for dead load computations (see **Figure 2-3**). = 0.084 klf for all girders (non-composite section)
Parapet Load	= 0.371 klf per parapet, or 0.742 klf for both parapets = 0.1484 klf for each girder (composite section)
Future Wearing Surface	= 0.025 ksf on roadway width = 0.238 klf for each girder (composite section)

Dead loads placed on the composite girder are distributed equally to all girders in the cross-section (LRFD Art. 4.6.2.2.1).

5.3   MATERIALS AND MATERIAL PROPERTIES

Material properties used for design are given below.

5.3.1   Girder Concrete

5.3.1.1   Basic Properties

f'_c = 8.5 ksi

f'_{ci} = 7.0 ksi

w_c = 150 pcf

5.3.1.2   **Time-Dependent Properties**

Time-dependent concrete properties (creep and shrinkage) are significant in the analysis and design of these girders. These properties are critical to proper design of this type of continuous post-tensioned bridge due to the girders being precast/prestressed and the deck slab being cast-in-place. As noted in **Section 3.1**, the time lapsed between casting of the girders and casting of the deck slab can have a significant effect due to these parameters. The program computed these properties according to the *AASHTO LRFD Specifications* with modifications for time-dependent analysis.

5.3.2   **Deck Concrete**

5.3.2.1   **Basic Properties**

f'_{cd} = 4.5 ksi

f'_{cdi} = 3.5 ksi

w_{cd} = 150 pcf

5.3.2.2   **Time-Dependent Properties**

See **Section 5.3.1.2** for discussion.

5.3.3   **Splice Concrete**

Girder splices occur near inflection points and require less concrete strength than the girders. The splice concrete is placed before the deck slab, so the strength can be different than the deck slab concrete.

5.3.3.1   **Basic Properties**

f'_{cs} = 5.5 ksi

f'_{cs} = 4.0 ksi

w_{cs} = 150 pcf

5.3.3.2   **Time-Dependent Properties**

See **Section 5.3.1.2** for discussion.

5.3.4   **Pretensioning Strands and Post-Tensioning Tendons**

Pretensioned strands and post-tensioning tendons utilize 0.6 in. diameter low relaxation seven-wire strand.

A_{ps} = 0.217 in^2

f_{pu} = 270 ksi

f_{py} = 0.90 f_{pu} = 243 ksi

f_{pj} = 0.75 f_{pu} = 202.5 ksi

E_p = 28,500 ksi

5.3.4.1   **Post-Tensioning System Design Parameters**

LRFD Table 5.9.5.2.2b-1 gives values for friction and wobble coefficients for use in design. The values selected for this design are listed below, with justification for the values chosen.

5.3.4.1.1   *Wobble Coefficient*

The wobble coefficient, K, given in the table is a typical value and is used for this example:

    K   = 0.0002 / ft   LRFD Table. 5.9.5.2.2b-1

5.3.4.1.2   *Friction Coefficient*

The friction coefficient, μ, is taken as

    μ = 0.25   LRFD Table. 5.9.5.2.2b-1

This is the maximum value given in the table. Actual values are typically closer to 0.20. The higher value is used to provide tolerance for field conditions. This would allow construction to proceed if the measured friction were above 0.20, but still below 0.25.

5.3.4.1.3   *Anchor Set*

Anchor set is mentioned in the commentary to LRFD Article C5.9.5.2.1. The commentary discusses various values that may be used. For this design, the anchor set, A_{set}, is taken as:

    A_{set} = 0.375 in.

This value is at the upper end of the range of typical values selected. This was again selected to provide tolerance for field operations.

5.3.4.2   **Grout**

While use of proper grout and grouting procedures is essential for the successful completion of a project, the type and strength of grout do not affect the design and are therefore not specified here. See the main report for additional information on grout and grouting.

Ducts must be grouted within a specified period following stressing to prevent corrosion of the tendons.

5.3.5   **Mild Reinforcement**

    f_y = 60 ksi
    E_s = 29,000 ksi

Continuous Three Span Girder   **Design Example 3**

5.4 SECTION PROPERTIES

5.4.1 Noncomposite Section (Girder only)

Table 5.5.1-1   Section Properties of Florida 78 in. Bulb Tee with Variable Web Depth

Section	Depth	Area	Moment of Inertia	Y_b	Y_t	S_b	S_t
	(in.)	(in^2)	(in^4)	(in.)	(in.)	(in^3)	(in^3)
BT-78	78	1,261	1,014,899	40.21	37.79	25,237	26,859
BT-84	84	1,315	1,223,135	43.19	40.81	28,317	29,974
BT-90	90	1,369	1,455,041	46.18	43.82	31,511	33,201
BT-96	96	1,423	1,711,587	49.16	46.84	34,818	36,540
BT-102	102	1,477	1,993,747	52.14	49.86	38,237	39,989
BT-108	108	1,531	2,302,492	55.13	52.87	41,767	43,548
BT-114	114	1,585	2,638,794	58.11	55.89	45,407	47,217
BT-120	120	1,639	3,003,626	61.10	58.90	49,159	50,996
BT-126	126	1,693	3,397,960	64.09	61.91	53,020	54,884
BT-132	132	1,747	3,822,766	67.08	64.92	56,990	58,882
BT-138	138	1,801	4,279,019	70.07	67.93	61,071	62,989
BT-144	144	1,855	4,767,689	73.06	70.94	65,260	67,204
BT-150	150	1,909	5,289,749	76.05	73.95	69,559	71,529
BT-156	156	1,963	5,846,170	79.04	76.96	73,966	75,962
BT-162	162	2,017	6,437,925	82.03	79.97	78,483	80,504
BT-168	168	2,071	7,065,986	85.02	82.98	83,108	85,155
BT-174	174	2,125	7,731,324	88.01	85.99	87,842	89,914
BT-180	180	2,179	8,434,913	91.01	88.99	92,684	94,782

Section properties of the precast prestressed girders may be required as input data to the computer program being used in design.

OTHER LOCATIONS   AT ANCHORAGES
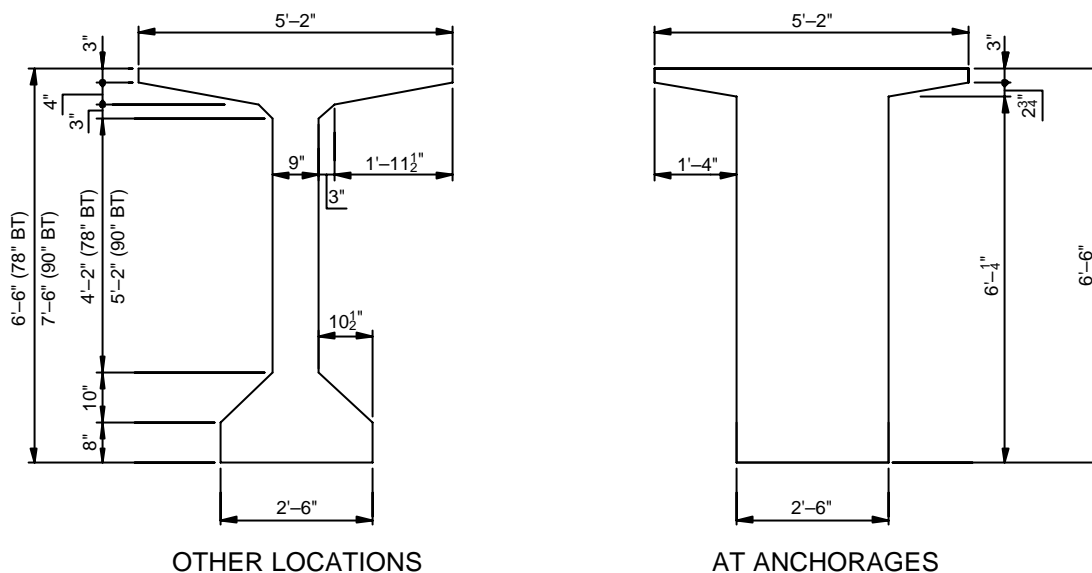
Figure 5.4.1-1   Cross Sections of End Girder and Drop-in Girder Segments

D3-14

Continuous Three Span Girder   **Design Example 3**

AT SPLICE   AT PIER
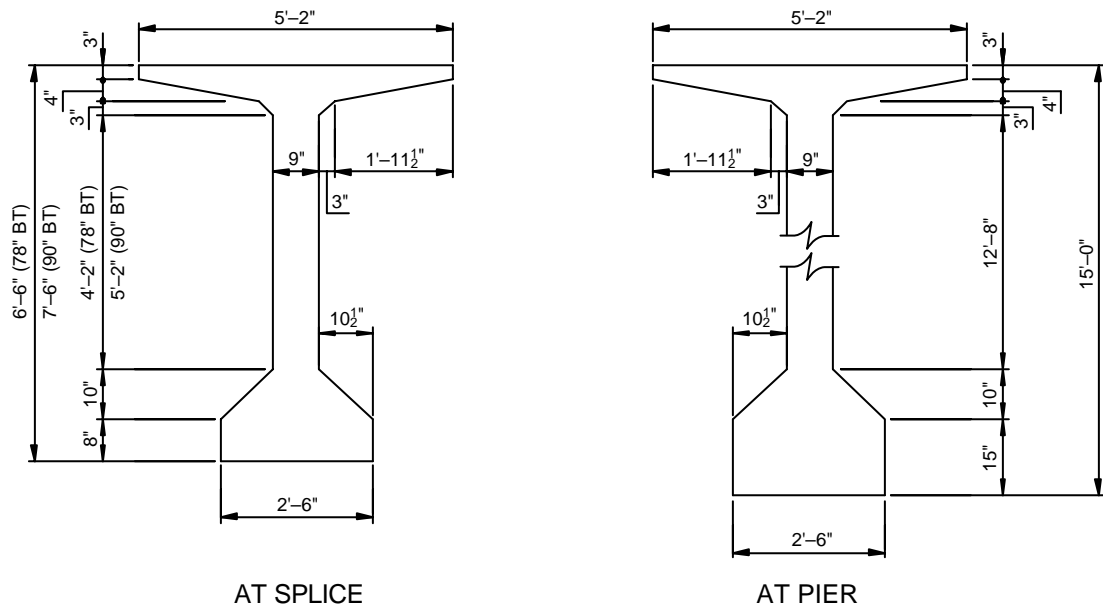
Figure 5.4.1-2   Cross Sections of Pier Girder Segments

The height of the bottom flange varies in the pier girder segment to provide an increased compression zone at the pier for design at the strength limit state.

5.4.2   Composite Section (Girder with Deck Slab)

The composite deck slab is transformed using the modular ratio, n, for computing stresses at the service limit state. Composite section properties are calculated by the computer program being used in design.

The build-up is not considered in computing these section properties (see **Section 3.2**).

5.5   DESIGN MOMENTS

The following sections present the computed design moments along the span for service and strength limit states.

5.5.1   Service Limit State

The following figures show service limit state moments for several of the construction stages. The moments due to self weight of the spliced girder and additional dead loads applied to the structure are shown in **Figure 5.5.1-1.** The moments due to the HL-93 live load, as computed by the computer program, are shown in **Figure 5.5.1-2**. The total service limit state moments are shown in **Figure 5.5.1-3** for Stages 7 and 9.

D3-15

Continuous Three Span Girder   **Design Example 3**
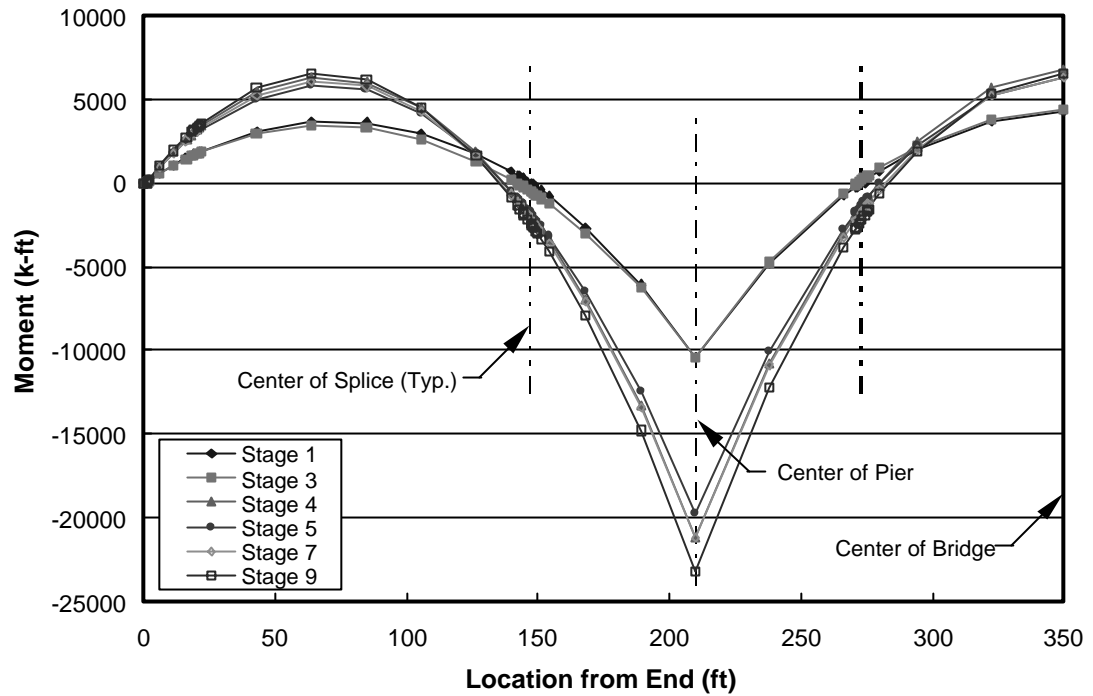
Figure 5.5.1-1   Service Dead Load Moments
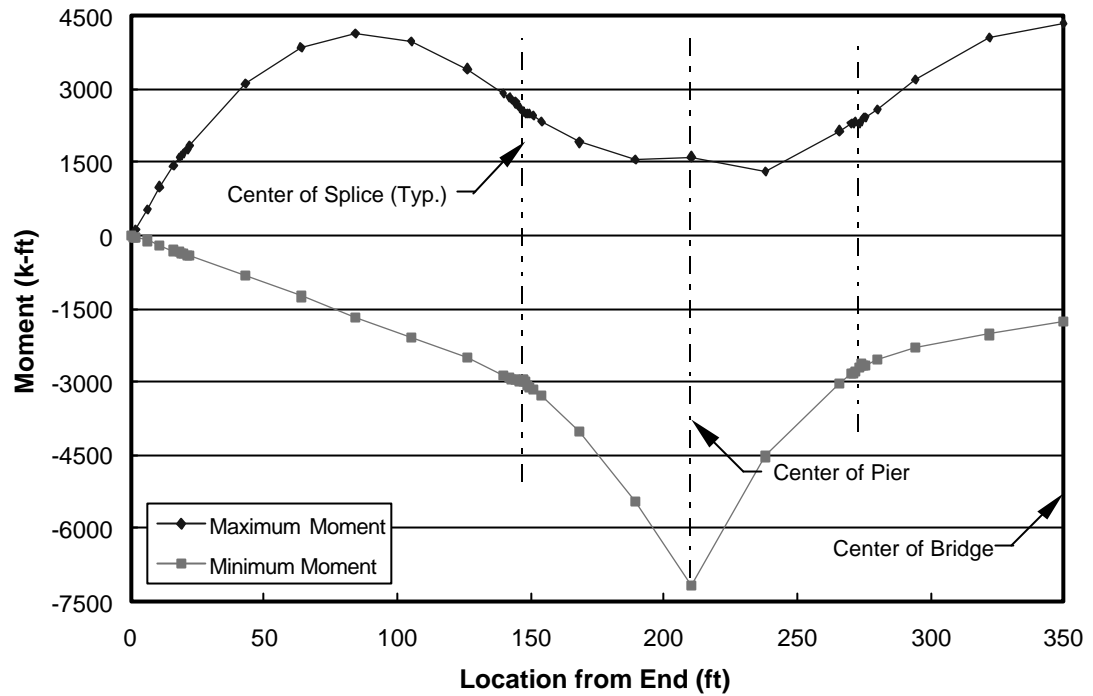
Figure 5.5.1-2   Service Moments Due to HL-93 (LL + IM) per Girder

D3-16
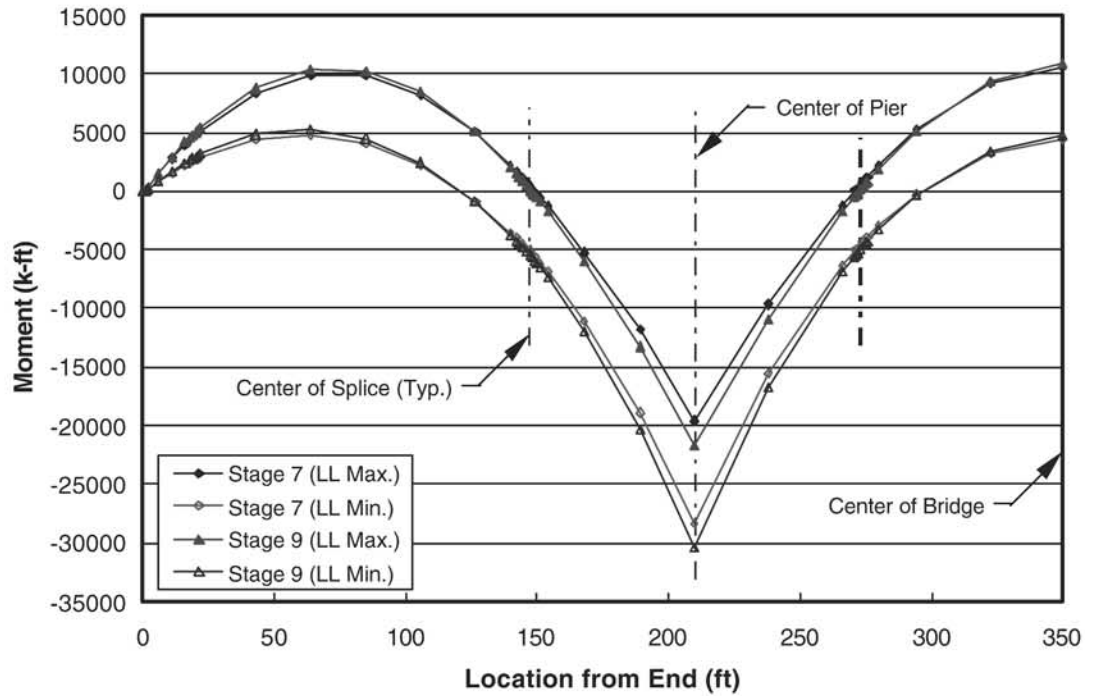
Figure 5.5.1-3   Total Service Moments for Stages 7 and 9

5.5.2   **Strength Limit State**

The following figures show Strength I limit state moments for several of the construction stages. The factored moments due to self weight of the spliced girder and additional dead loads applied to the structure are shown in **Figures 5.5.2-1** and **5.5.2-2.** The maximum and minimum values shown result from the load factors given in LRFD Table 3.4.1-2. The factored moments due to the HL-93 live load, as computed by the computer program, are shown in **Figure 5.5.2-3**. The maximum combined Strength I limit state moments are shown in **Figure 5.5.2.4**.

Continuous Three Span Girder
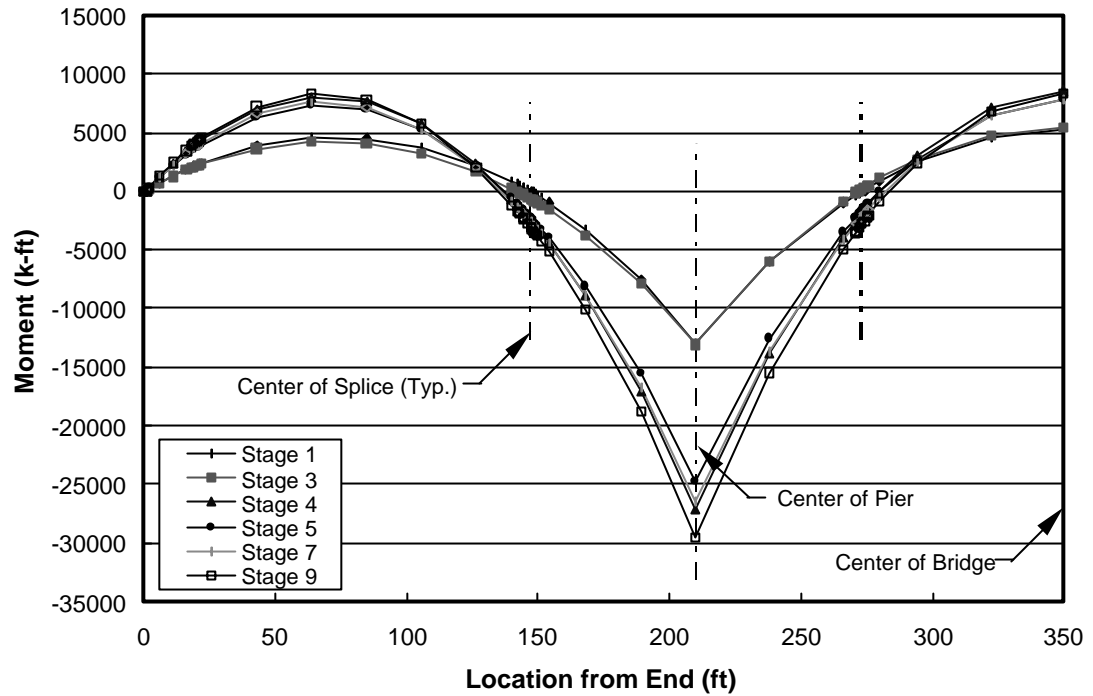
Design Example 3

Figure 5.5.2-1   Total Strength Dead Load Moments- Maximum
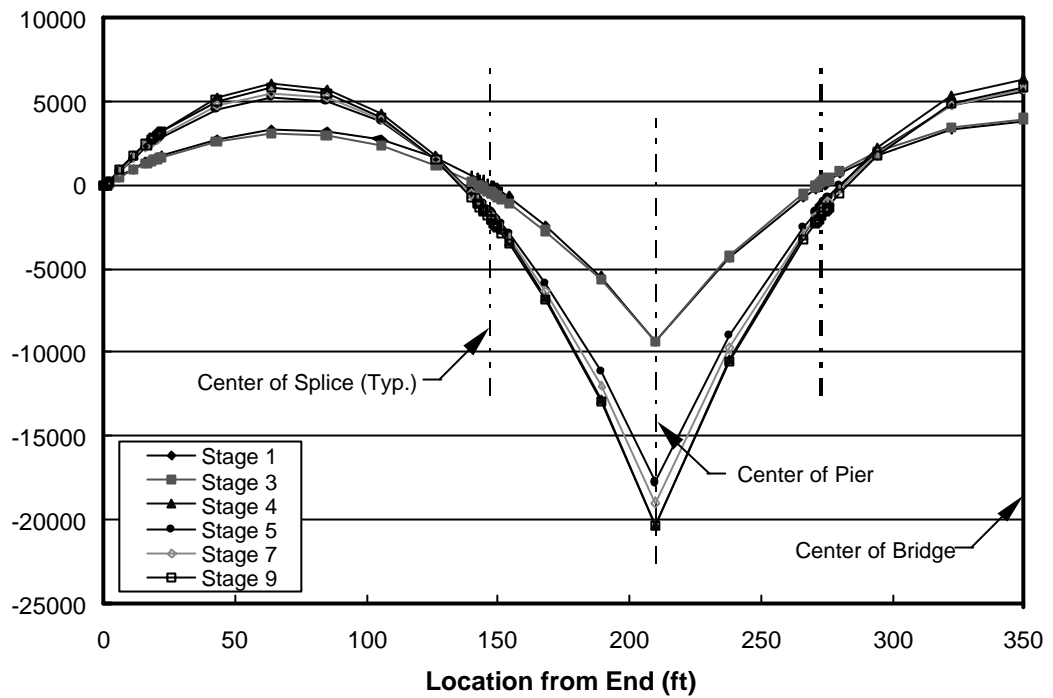
Figure 5.5.2-2   Total Strength Dead Load Moments - Minimum

Continuous Three Span Girder   **Design Example 3**
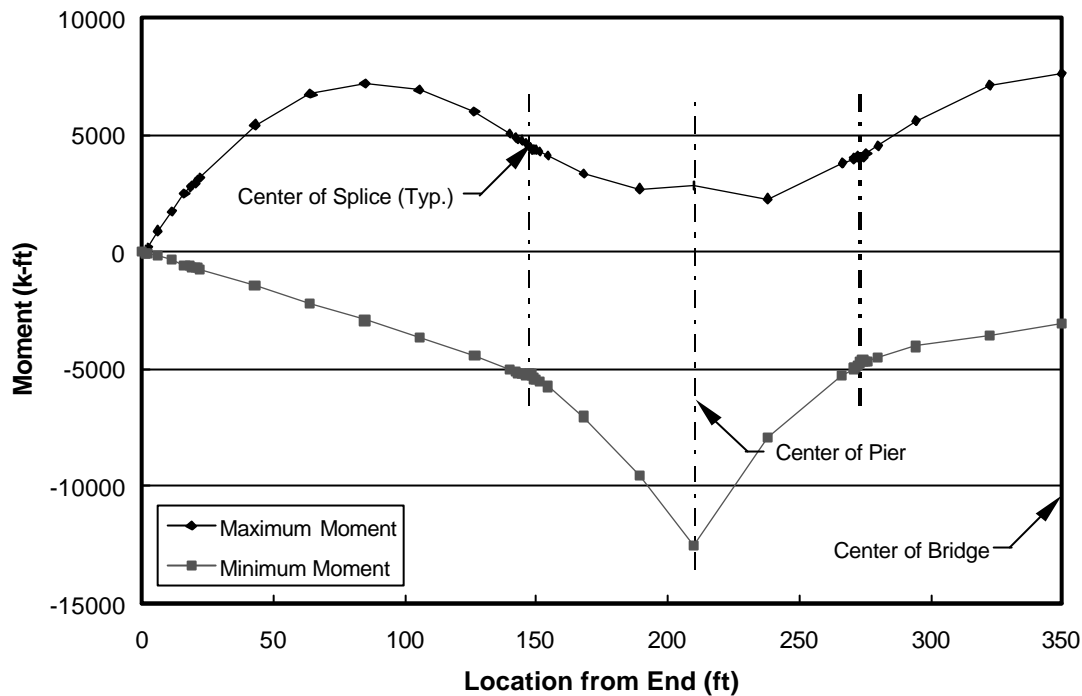
Figure 5.5.2-3   Strength Moments Due to HL-93 (LL + IM) per Girder
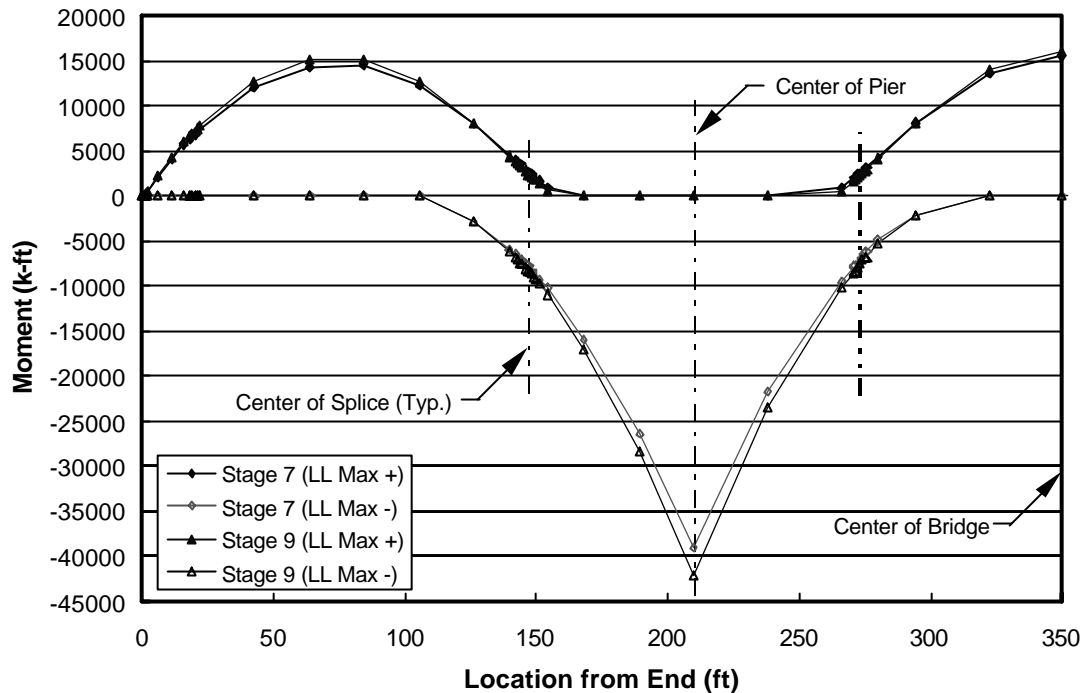
Figure 5.5.2-4   Total Strength Moments for Stages 7 and 9

6   PRESTRESS LAYOUT AND DATA

The first step in establishing the prestress arrangement is to determine the optimum post-tensioning duct size. This is crucial as the width of the girder web is predicated on the diameter of the duct. This design example uses 4 in. diameter ducts within a 9 in. web. This results in a duct/web ratio of 0.44, even though the *AASHTO LRFD Specifications* states that a maximum ratio of 0.40 should be used. Based on much successful use of the 4 in. ducts within a 9 in. web, a recommendation is being made to modify the 0.40 ratio. Any increase beyond 9 in. to the web width begins to be counter-productive, as the additional dead load of the girder may limit the use of longer spans due to handling, transportation and erection considerations. Also, the increased width provides only a nominal increase to the section properties of the cross-section.

The number of post-tensioning tendons to be used is the second most important parameter. Some early designs of these structures used three versus four tendons and "oval shaped" ducts in the belief that a substantial cost savings would result. Experience has proven that only round ducts should be used. For the span range of this type structure, it is recommended that a minimum of four ducts be considered.

Based on the selected post-tensioning tendon duct size and number of tendons, the maximum amount of post-tensioning is determined. The total number of 0.60 in. diameter strands per tendon is then based upon compressive stress requirements at the splice locations for positive moments and at the centerline of interior piers for negative moments.

The amount of pretensioning is then based upon "supplementing" the post-tensioning as required to produce the total net compressive stress needed at any point along the span lengths. The amount of pretensioning is also checked to provide for handling, transportation and erection stresses. It is recommended that at least a 50% impact factor be used on the girder weights.

The amount of pretensioning in the top flange of the haunched girders at interior piers is affected by the method of girder erection. This example utilizes a "strongback" system for supporting the drop-in girder of the main span. This method is beneficial for maintenance of traffic over an active waterway or roadway during construction but results in a substantial load on the cantilevers of the haunched girder segments. The negative moment produced then controls the pretensioning, since no post-tensioning tendons have been stressed at that time. If temporary support towers are used at all splice locations, then the amount of pretensioning in the top flange is reduced.

6.1   PRETENSIONED STRAND LAYOUT

The number of pretensioned strands in the girder segments was selected to provide enough prestress to maintain concrete stresses within stresses limits at release and erection. All pretensioned strands are straight with no debonding required. Strands are placed as shown in **Figures 6.1-1** to **6.1-4**. All strands are initially stressed to 0.75 f_{pu}.

It is recommended that an amount of pretensioning be used that is larger than required for only stress considerations. The pretensioning assists not only for post-tensioning factors but also with assuring adequate camber in the completed structure.

As Mentioned in **Section 11.1**, pretensioned strands were added in the drop-in girder segment to improve the final profile of the spliced girder.

Continuous Three Span Girder

Design Example 3

AT ANCHORAGE

MIDSPAN
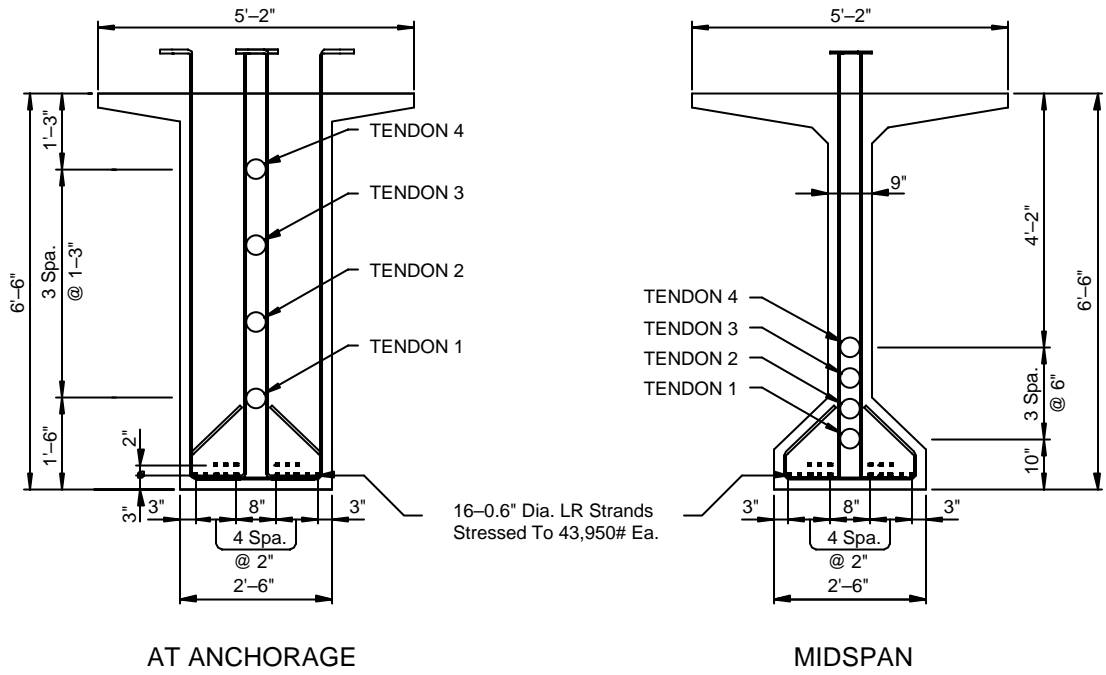
Figure 6.1-1   Prestress Layout for End Girder Segments

END GIRDER SEGMENT

PIER GIRDER SEGMENT
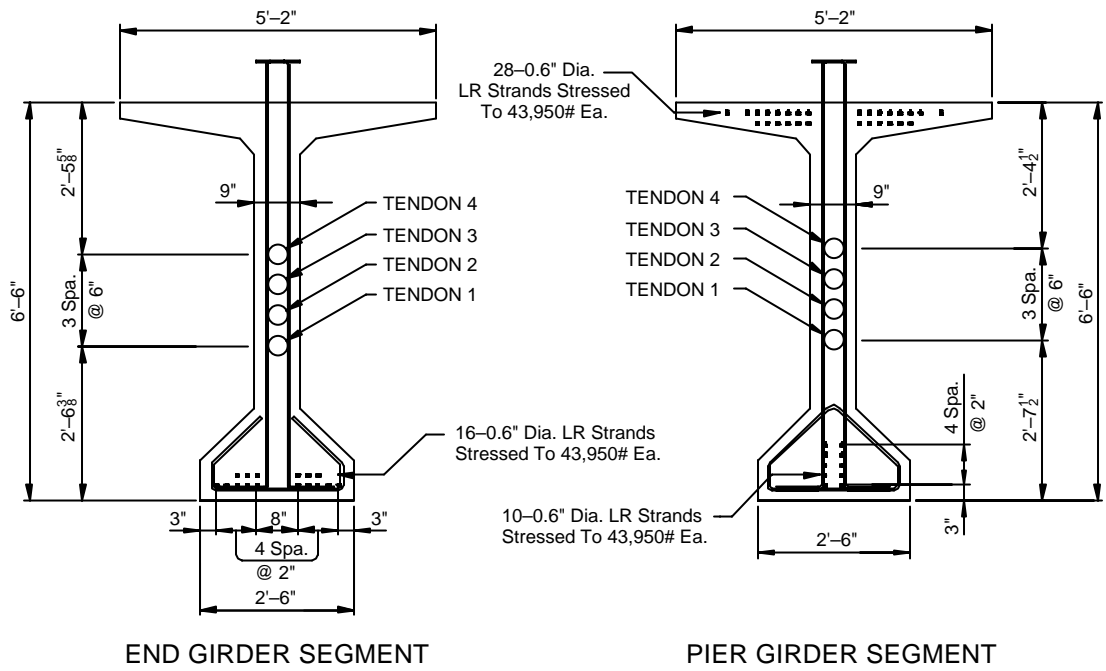
Figure 6.1-2   Prestress Layout for Girder Segments at Splices in Spans 1 and 3

D3-21

Continuous Three Span Girder   **Design Example 3**
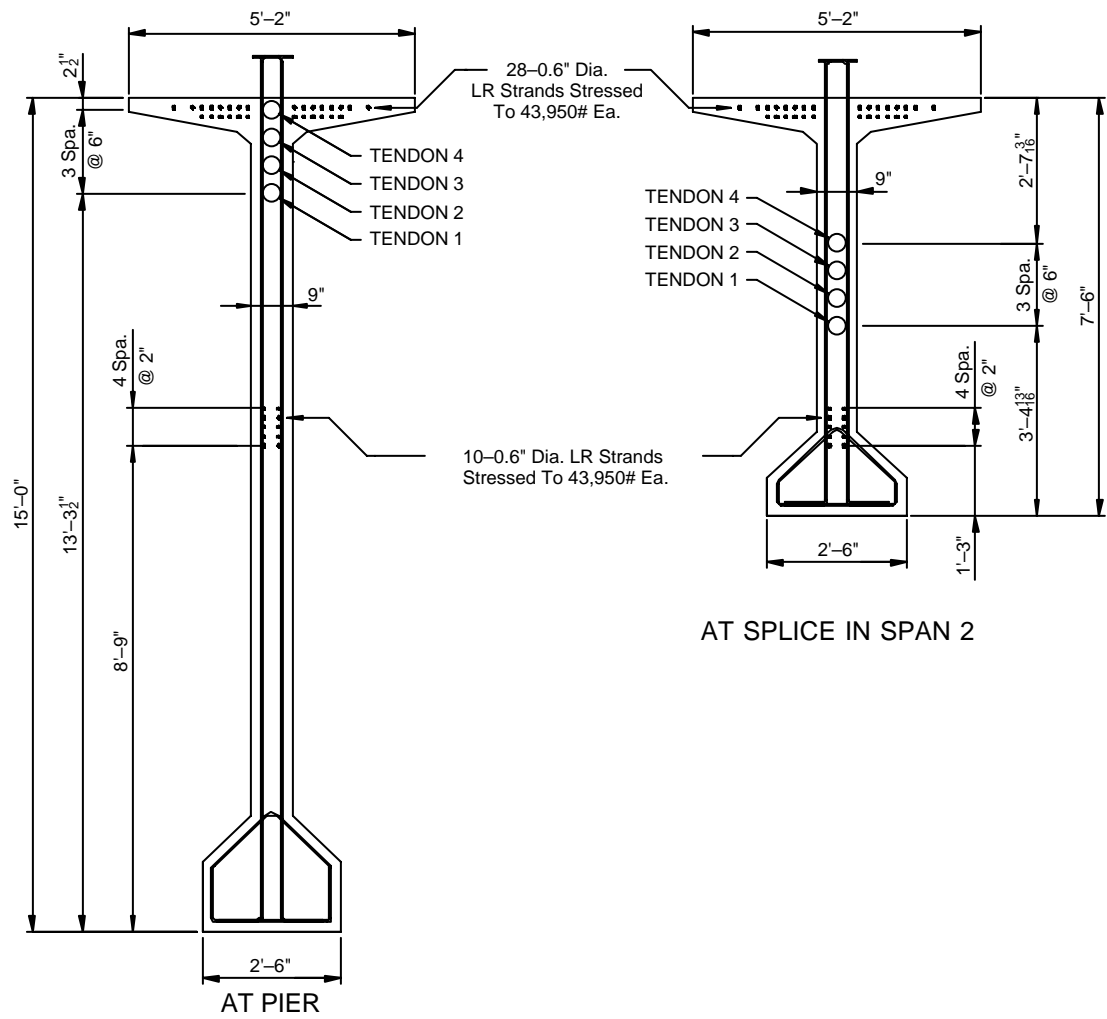
Figure 6.1-3   **Prestress Layout for Pier Girder Segments**

D3-22
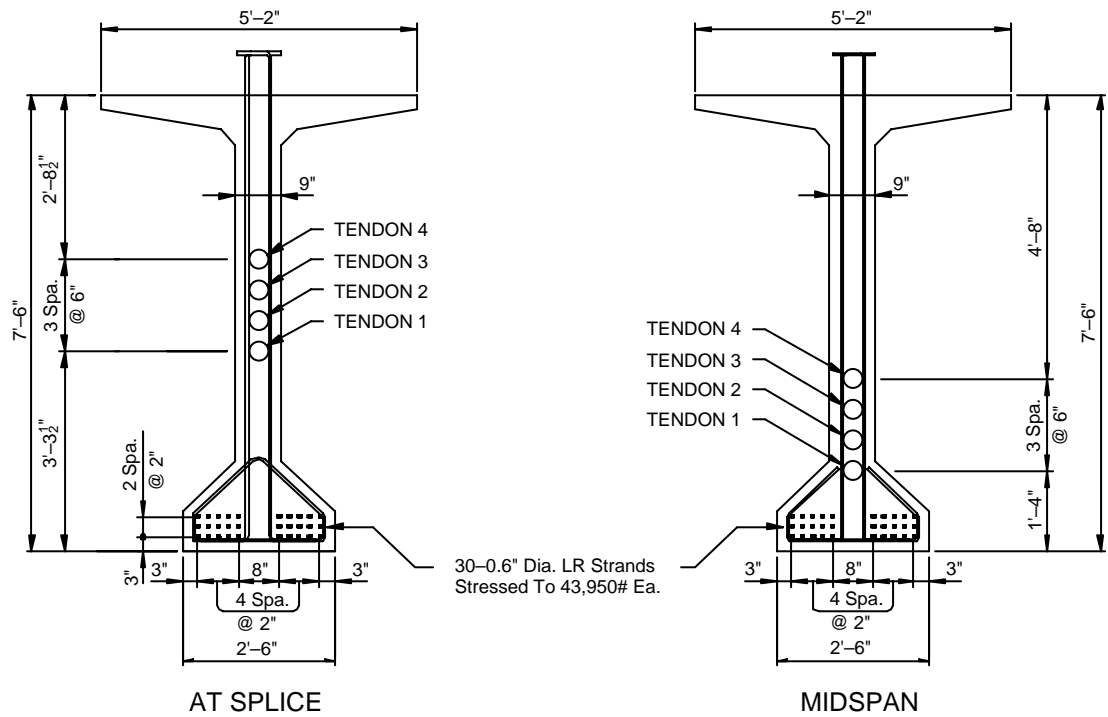
Figure 6.1-4   Prestress Layout for Drop-in Girder Segments

6.2   POST-TENSIONING TENDON LAYOUT

Post-tensioning tendons in this girder are designed to provide the prestress required to resist the live load and the dead load. Section views showing details of the tendon layout at ends of girders segments and at midspan are presented in **Figures 6.1-1** to **6.1-4**. The tendon profile is shown in **Figure 6.2-1**. The layout shown is the tendon layout used for analysis. An adjustment is required to determine the duct layout. See **Section 6.2.1** for a brief discussion of the adjustment.

D3-23
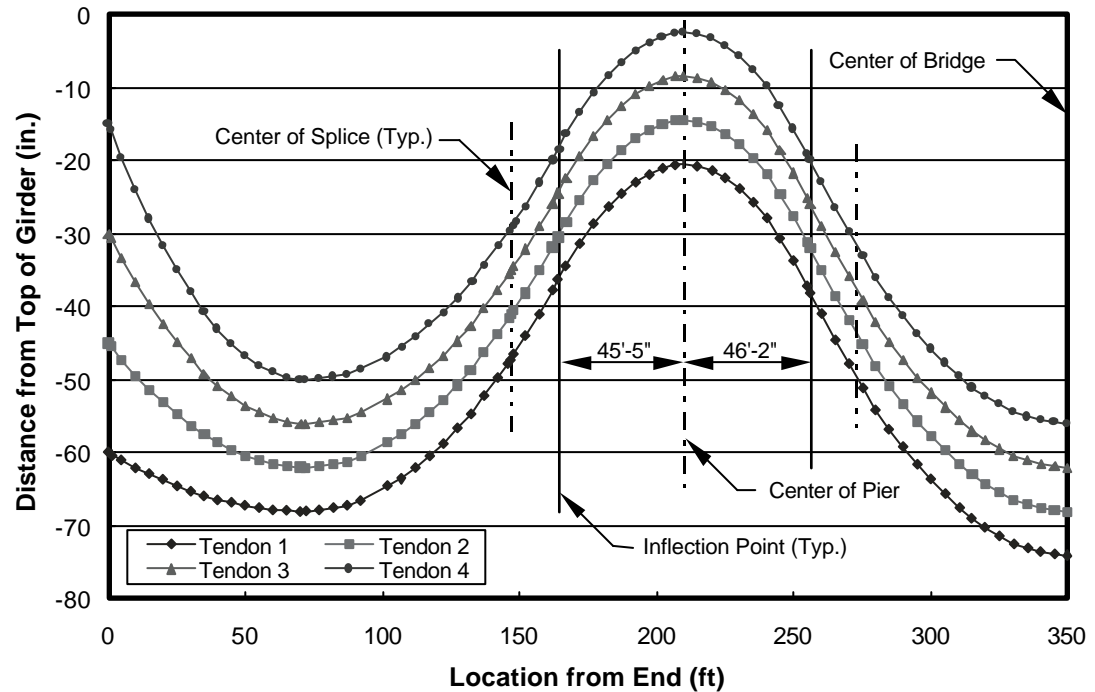
Figure 6.2-1   Post-Tensioning Tendon Layout

6.2.1   Determination of Duct Profile

The center of a post-tensioning tendon is offset by a distance, Z, from the center of the duct when the tendon is curved. This is caused by the bunching of the strands on one side of the duct, as shown schematically in **Figure 6.2.1-1**.
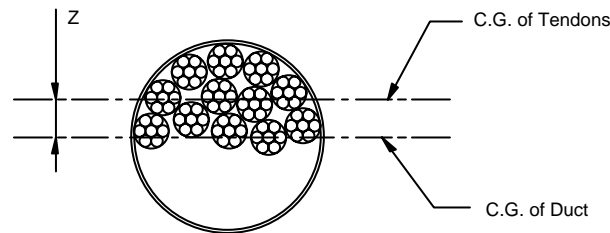
Figure 6.2.1-1   Offset between C.G. of Duct and C.G. of Tendon

The location of the center of the duct is computed for each tendon assuming that the full value of the offset, Z, is taken for the extreme portions of the tendon layout, and a proportional fraction of Z between the high and low points along the tendon layout.

7   PRESTRESS LOSSES AND EFFECTIVE PRESTRESS

The estimation of prestress losses for the design of a spliced girder bridge is significantly different from the design of typical pretensioned girders because prestressing is applied at

D3-24

three different times and in two different ways. A rigorous time-step approach is used by the software to estimate losses. The results are discussed in the following sections.

7.1   LOSSES IN PRETENSIONED STRANDS

Prestress losses in the pretensioned strands are computed according to the time step analysis utilized by the software package. The effects of the staged post-tensioning are included in the loss analysis for the pretensioned strands.

The effective stress in the pretensioned strands along the spliced girder for selected load stages is shown in **Figure 7.1-1**. The losses increase with time as is expected due to time dependent losses such as creep and shrinkage. The only exception to the increase in prestress loss occurs when the deck is cast. The additional dead load increases the stress in the pretensioned strands. The effect of the transfer length is also clearly seen as the stress drops to zero at the end of each girder segment.
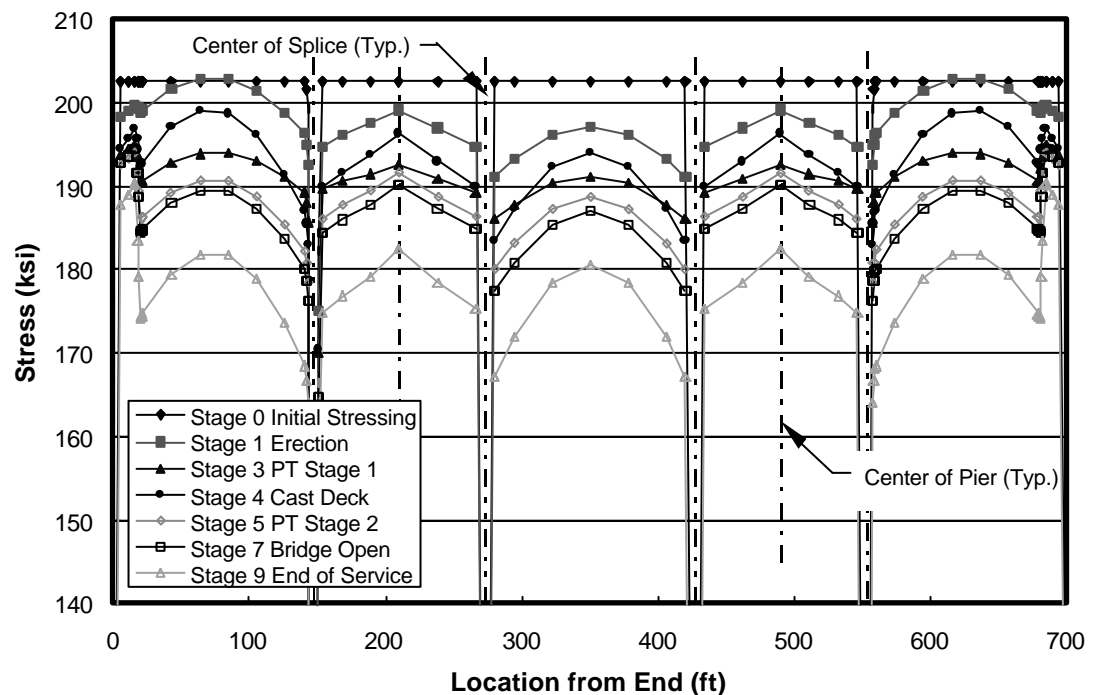
Figure 7.1-1   Stress in Pretensioned Strands

7.2   LOSSES IN POST-TENSIONING TENDONS

The effective stresses in Tendons 1 and 3 are shown along the length of the spliced girder for selected load stages in **Figures 7.2-1** and **7.2-2**, respectively. The stresses are only shown for one tendon in each stage of post-tensioning to more clearly demonstrate the anchor set loss and the effect of stressing the second tendon in a stage. Stresses are shown for intermediate stages as tendons are stressed and anchored for each stage of post-tensioning. The significant jump in the stress in tendons at each end of the continuous girder are apparently caused by the change in cross-section properties at the end block. The increased section reduces both short and long-term losses.

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Continuous Three Span Girder   **Design Example 3**
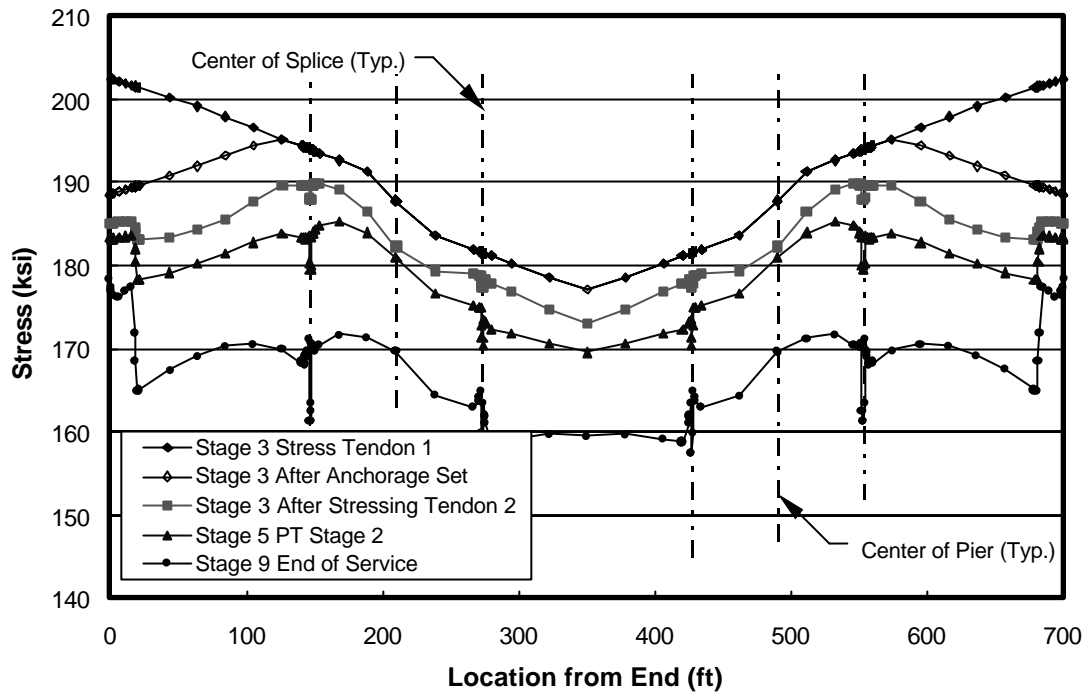
Figure 7.2-1   Stress in Tendon 1
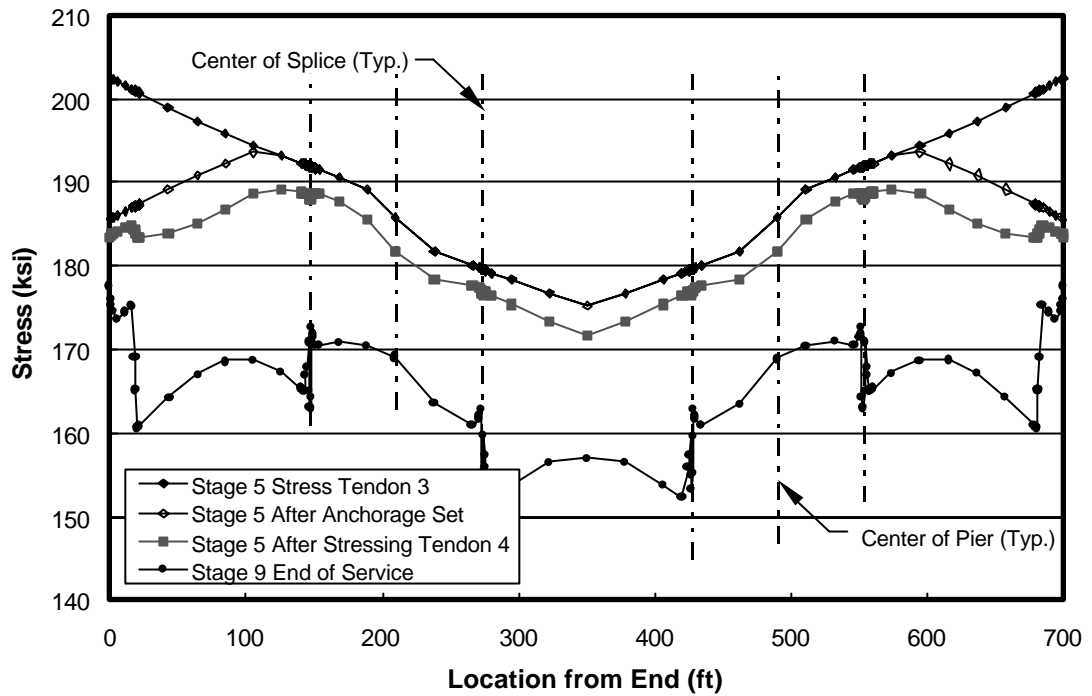
Figure 7.2-2   Stress in Tendon 3

Continuous Three Span Girder   **Design Example 3**

7.2.1   Anchorage Set Loss

Friction and anchorage set losses are closely related. They are estimated using an iterative process that continues by reducing the assumed temporary jacking stress, f_{pj}, until all stress limits are satisfied at the critical locations.

Anchorage set loss is computed by estimating the loss that occurs when each strand pulls into the anchorage at seating. This pull in reduces the stress at the anchorage and for a distance that depends on the rate of friction loss in the tendon. The jacking stress was initially assumed to be its maximum value, 218.7 ksi (see **Table 7.2.1-1**). It was then reduced until the stress limit was satisfied at the stressing end anchorage, which was the location that governed this design. The stress limits, critical locations, and the stresses after the final iteration are shown in **Table 7.2.1-1**. The variation in tendon stress before and after anchorage set are shown in **Figure 7.2.1-1** for the final jacking stress.

Table 7.2.1-1   Limiting Anchorage Set Stresses

Location or Condition	Stress Limit (ksi)		Final Stress (ksi)	Description
1	0.90 f_{py}	218.7	202.5	Temporary Jacking Stress at Stressing End Anchorage
2	0.70 f_{pu}	189.0	188.5	Stress after Seating at Stressing End Anchorage
3	0.74 f_{pu}	199.8	195.5	Stress after Seating at End of Anchorage Set Loss

Shaded cells govern.
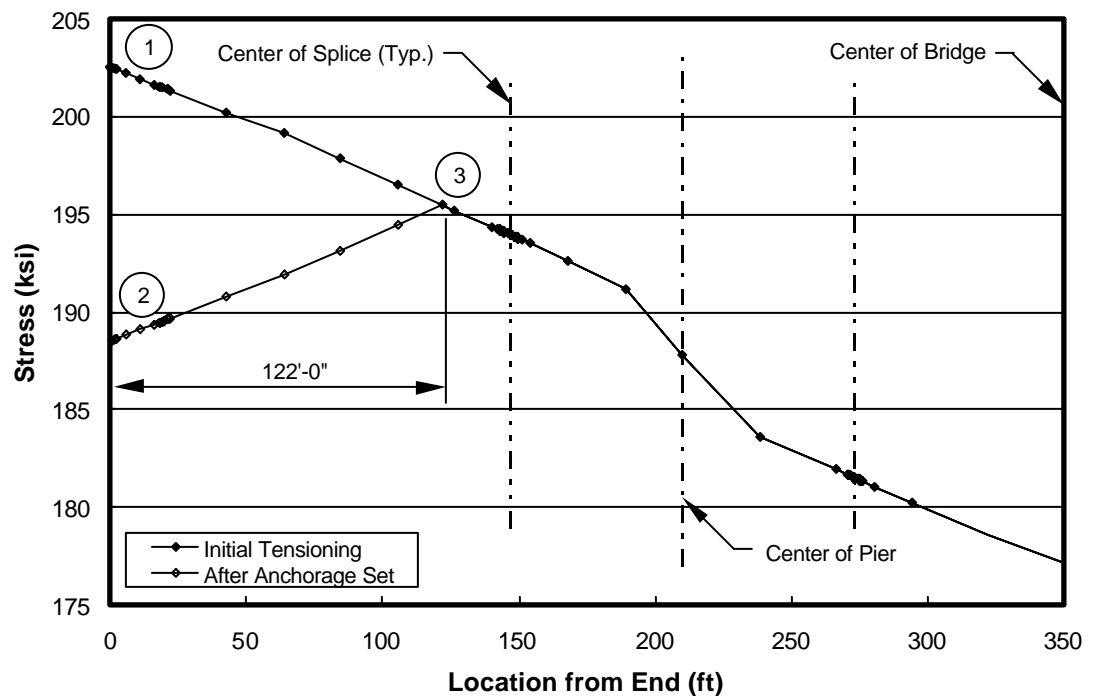
Figure 7.2.1-1   Stress in Tendon 1 Before and After Anchorage Set

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The anchorage set loss is the difference between the stress in the tendon before and after anchorage set. The loss varies with the location within the length of tendon affected by anchorage set or 122 ft in this case. The anchorage set loss at the anchorage was:

202.5 − 188.5 = 14.0 ksi

The anchorage set distance varied for each tendon due to the change in curvature associated with the increased tendon spacing in the anchorage zone region. The anchorage set distance for each tendon is shown in **Table 7.2-2**.

Table 7.2-2   Anchorage Set Distances

Tendon	Anchorage Set Distance (ft)
1	122.0
2	117.4
3	112.6
4	107.4

7.3   **EFFECTIVE PRESTRESS FORCE**

The effective prestress force remaining in each of the three stages of prestressing is shown in **Figures 7.3-1** to **7.3-3** for several construction stages. The effective prestress force is simply the stress in the prestressing strands or tendons, shown in **Figures 7.1-1** to **7.2-2**, multiplied by the area of prestressed reinforcement present.

The effective prestress force in the pretensioned strands at Stages 1, 7, and 9 is shown in **Figure 7.3-1**. As expected, the effective force decreases as the girder age increases. The effective prestress force differs between girder segments because of the different number of pretensioned strands in each segment. (See **Figures 6.1-1** through **6.1-4**.)

Figures 7.3-2 and **7.3-3** indicate the effective prestress force in Stage 1 and Stage 2 post-tensioning tendons (two tendons in each stage). The effects of PT Stage 2 on PT Stage 1 are seen in **Figure 7.3-2** as a reduction in effective force along the entire length of the spliced girder. The total effective force due to the post-tensioning is shown in **Figure 7.3-4**. The effective force in the initial stages, before PT Stage 2, indicates a small reduction in force due to prestress loss. After PT Stage 2, the magnitude of the force increases significantly.

As mentioned in **Section 7.2**, the increased force in the tendons at the ends of the exterior spans is the result of the increased section properties created by the end block.

Continuous Three Span Girder   **Design Example 3**
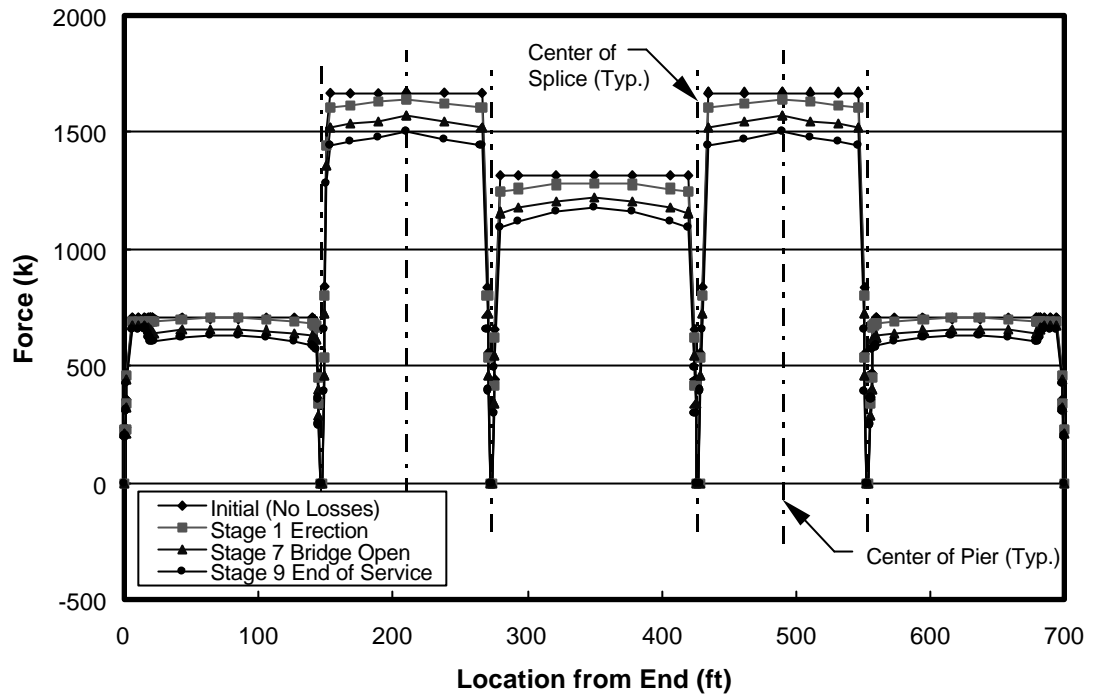
Figure 7.3-1   Effective Force in Pretensioned Strands – Stages 1, 7, and 9
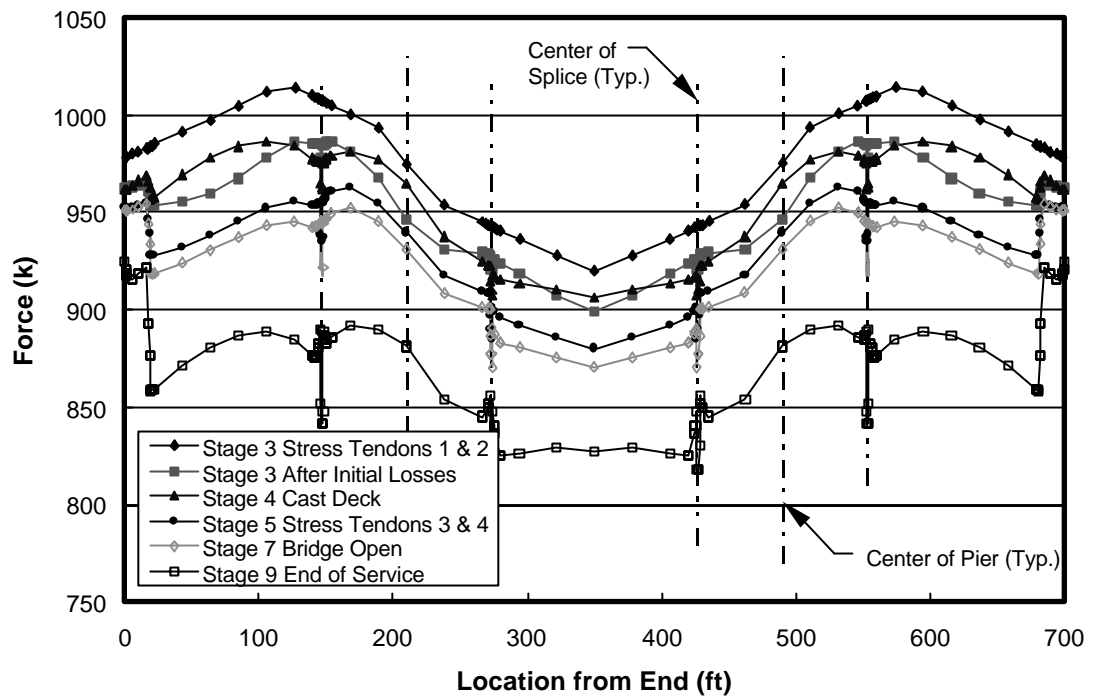
Figure 7.3-2   Effective Force in Post-Tensioning Tendons 1 and 2

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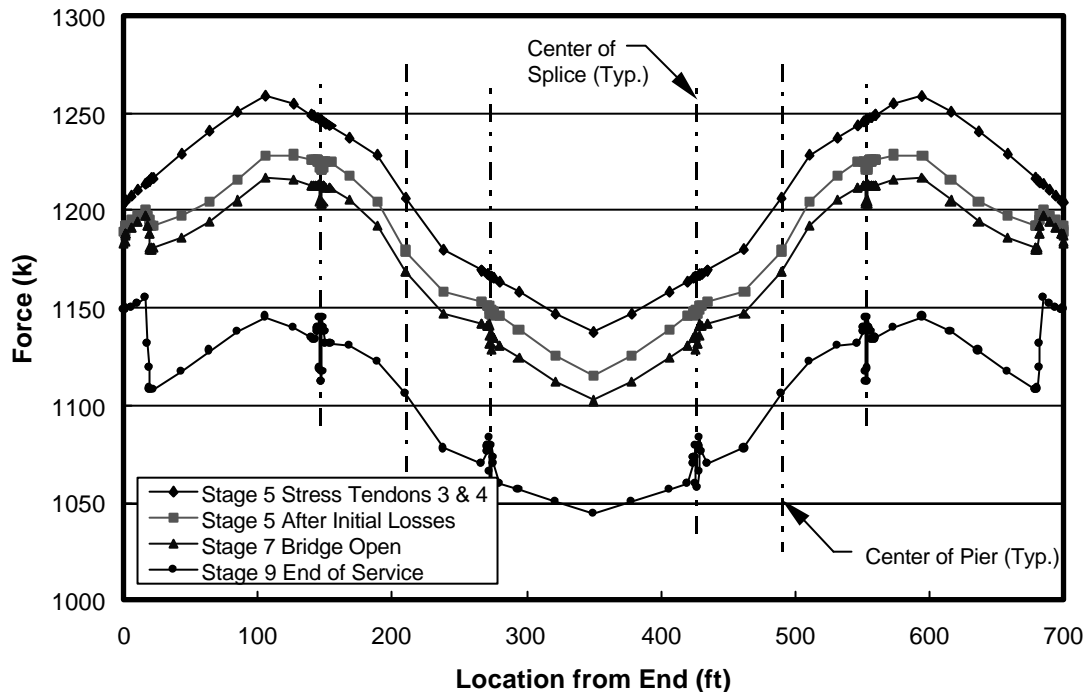
Figure 7.3-3   Effective Force in Post-Tensioning Tendons 3 and 4

8   DESIGN FOR FLEXURE

The design of this bridge for flexure is the focus of the design process. The process differs from typical designs because of the combination of pretensioned strands and post-tensioning tendons and the various stages of construction.

The service limit state requirements for flexure govern the required prestress force during the early stages. In the latter stages, flexural strength and deflection limits governed the prestress force required.

As noted in **Section 1**, due to the voluminous output generated by design programs for this type of structure, the most important data has been transferred to a commonly used spreadsheet/chart generating program. The information contained in this section provides a concise visual summary of the design quantities along the structure.

Information is provided in this section for the majority of the stages of construction, which are defined in **Section 3.1** and summarized in **Table 3.1-1**.

8.1   SERVICE LIMIT STATE

For the service limit state, concrete stresses in the girder segments and deck slab must remain within specified stress limits for tension and compression. Therefore, for this bridge, as with most pretensioned and post-tensioned concrete structures, providing adequate prestress force to satisfy the specified stress limits is the first step in design.

D3-30

In general, the prestress force required to satisfy the stress limits will provide the required ultimate moment capacity and will provide adequate control of camber and deflection of the structure. However, in this structure, additional prestress force was required for the strength limit state and also to control deflections.

8.1.1   Stress Limits

The following stress limits are used for the design of the girder segments and deck slab for the service limit state.

8.1.1.1   Prestressing Strands

8.1.1.1.1   *Pretensioned Strands*

The stress limits for low relaxation strands in the pretensioned girder segments are:

Immediately prior to transfer: LRFD Table 5.9.3-1

f_{pi} = 0.75 f_{pu} = 202.5 ksi

At Service Limit State after losses: LRFD Table 5.9.3-1

f_p = 0.80 f_{py} = 194.4 ksi

These stress limits are not discussed in this example because they do not govern designs.

8.1.1.1.2   *Post-Tensioning Tendons*

The stress limits for low relaxation strands in the post-tensioning tendons are:

Prior to seating: LRFD Table 5.9.3-1

f_{pi} = 0.90 f_{py} = 0.90 (0.90f_{pu}) = 0.81 f_{pu} = 218.7 ksi

At anchorages and couplers immediately after anchor set: LRFD Table 5.9.3-1

f_{pi} = 0.70 f_{pu} = 189.0 ksi

At end of seating loss zone immediately after anchor set: LRFD Table 5.9.3-1

f_{pi} = 0.74 f_{pu} = 199.8 ksi

At Service Limit State after losses: LRFD Table 5.9.3-1

f_{pe} = 0.80 f_{py} = 0.80 (0.90f_{pu}) = 0.72 f_{pu} = 194.4 ksi

It should be noted that all of these limits will not be satisfied simultaneously.

8.1.1.2   Concrete

8.1.1.2.1   *Temporary Stresses at Release*

Compression: LRFD Art. 5.9.4.1.1

f_{cR} = 0.60 f'_{ci}

Tension: LRFD Table 5.9.4.1.2-1

f_{tR1} = 0.0948 $\sqrt{f'_{ci}}$ ≤ 0.200 ksi

or f_{tR2} = 0.24 $\sqrt{f'_{ci}}$ with reinforcement to resist the tensile force in the concrete

8.1.1.2.2   **Intermediate Stresses During Construction**

Stress limits are given below for the girder concrete. Compressive stresses may be checked for the deck slab, but never govern, so they are not included here.

Compression: LRFD Table 5.9.4.2.1-1

f_{c1} = 0.60 $\phi_w f'_c$, for full service loads (ϕ_w = 1 for girders) [*Compression 1*]

Tension: LRFD Table 5.9.4.2.2-1

For the precompressed tensile zone (including the top of deck slab):

f_{t1} = 0.19 $\sqrt{f'_c}$, assuming moderate corrosion conditions

8.1.1.2.3   **Final Stresses at Service Limit State after Losses**

Stress limits are given below for the girder concrete. Compressive stresses may be checked for the deck slab, but never govern, so they are not included here.

Compression: LRFD Table 5.9.4.2.1-1

f_{c1} = 0.60 $\phi_w f'_c$, for full service loads (ϕ_w = 1 for girders) [*Compression 1*]

f_{c2} = 0.45 f'_c, for effective prestress (PS) and full dead loads (DL) [*Compression 2*]

f_{c3} = 0.40 f'_c, for live load plus one-half of effective PS and full DL [*Compression 3*]

Tension: LRFD Table 5.9.4.2.2-1

For the precompressed tensile zone (including the top of deck slab):

f_{t1} = 0.19 $\sqrt{f'_c}$, assuming moderate corrosion conditions

8.1.1.2.4   **Summary of Stress Limits for Concrete**

The stress limit values for the different types of concrete at the major stages of construction are given in **Table 8.1.1.2.4-1**.

Table 8.1.1.2.4-1 Summary of Concrete Stress Limits

Stage of Construction	Type of Stress	Girder Concrete (ksi)	Deck Concrete (ksi)	Splice Concrete (ksi)
1. Release of Pretensioned Strands	*Compression*	-4.200	–	–
	Tension	-0.200	–	–
2. Intermediate Stages	*Compression*	-5.100	-2.400	-3.000
	Tension	0.554	0.380	0.425
3. Final Conditions after Losses	*Compression 1*	-5.100	-2.400	-3.000
	Compression 2	-3.825	-1.800	-2.250
	Compression 3	-3.400	-1.600	-2.000
	Tension	0.554	0.380	0.425

8.1.2   **Stresses in Concrete**

Stresses in the girders and deck slab have been computed along the bridge for construction stages 1, 3, 4, 5, 7, and 9 shown in **Table 3.1.1-1**. These stresses are compared with the specified stress limits in **Figures 6.1-1 to 6.1-21**.

For stages including live loads, figures for Service I show compression stress limits and figures for Service III show tension stress limits. Separate figures are used for girder and deck stresses, and for the different compression stress limits. Where applicable, stress limits for splices are shown on the figures for girder stresses.

Reviewing the figures for each stage and comparing them to figures for the other stages provides a global view of how the structure behaves during construction and after all time-dependent effects have occurred.

It may be noted that the splice locations and the midspan of the center span typically control the design of the post-tensioning force for tensile stresses in the positive moment regions. In turn, the allowable tensile stress in the top of the top slab often controls the design of the post-tensioning force in the negative moment region over the piers.

Therefore, the amount of post-tensioning in the bottom two tendons (Tendons 1 and 2 – PT Stage 1) is affected most by the positive moment regions while the amount in the top two tendons (Tendons 3 and 4 – PT Stage 2) is affected most by the negative moment region. The final design of this example utilizes 12 – 0.60 in. strands in each of the bottom two tendons (PT Stage 1) and 15 – 0.60 in. strands in each of the top two tendons (PT Stage 2).

Figures 8.1-9 and **8.1-17** show stresses in the deck in the completed structure under dead load only. While the deck is subject to slight tension when the bridge is open to traffic (**Figure 8.1-9**), the tension is gone after all prestress losses have occurred (**Figure 8.1-17**). It is desirable to have the deck in compression to close potential cracks.

As shown in **Figures 8.1-1 to 8.1-21**, the design falls within the allowable stress limits for both the deck slab and the girder segments for all stages.

Continuous Three Span Girder   **Design Example 3**
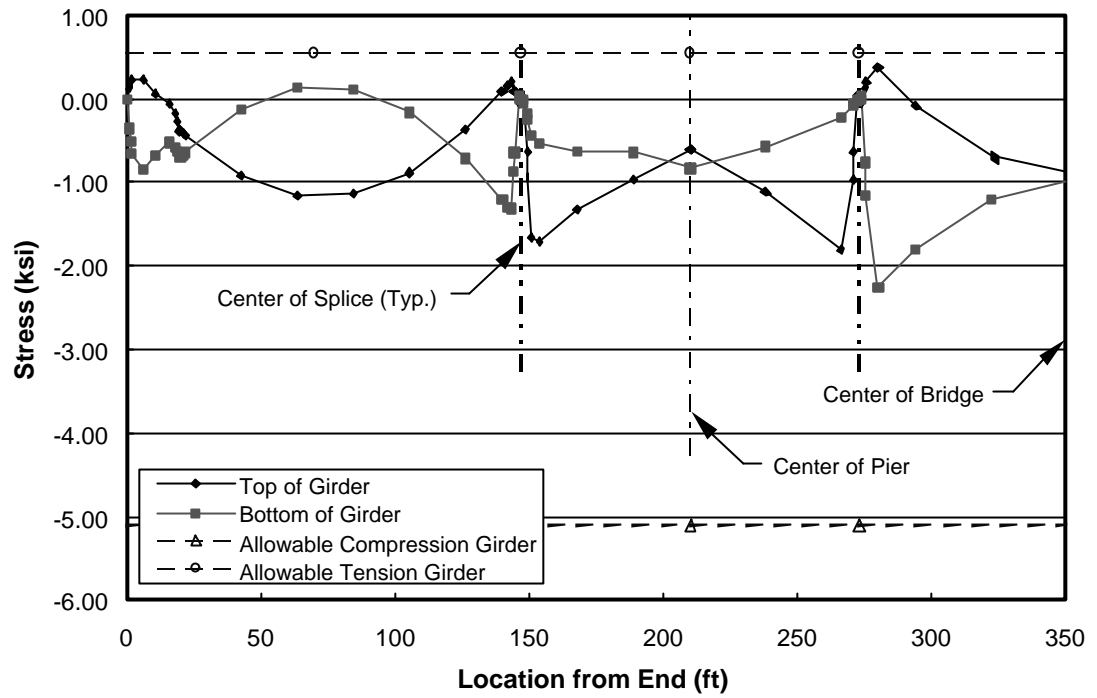
Figure 8.1-1   Girder Stresses for Service I - Stage 1 [Girder Segment Erection]
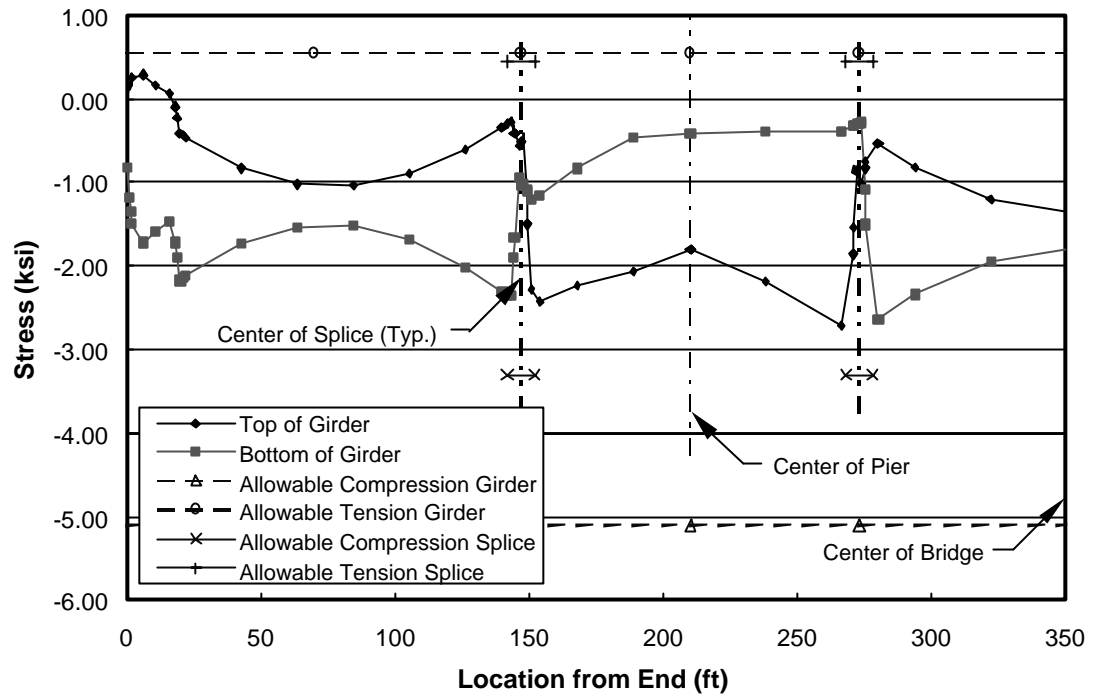
Figure 8.1-2   Girder Stresses for Service I - Stage 3 [PT Stage 1]

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Continuous Three Span Girder   **Design Example 3**
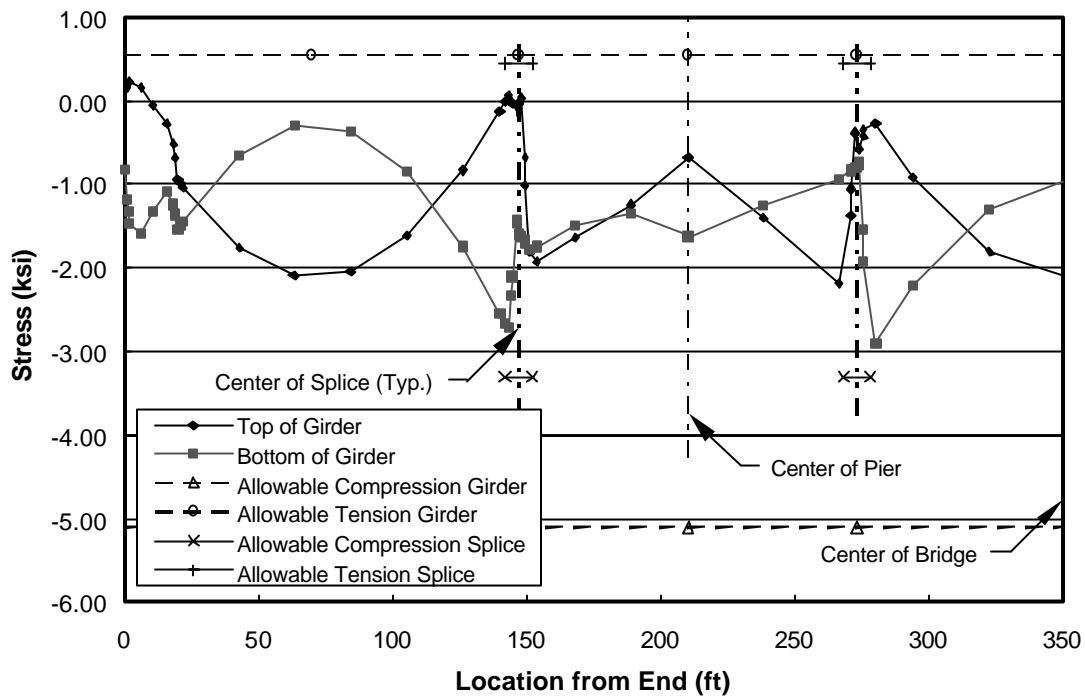
Figure 8.1-3   Girder Stresses for Service I - Stage 4 [Deck Slab Cast]
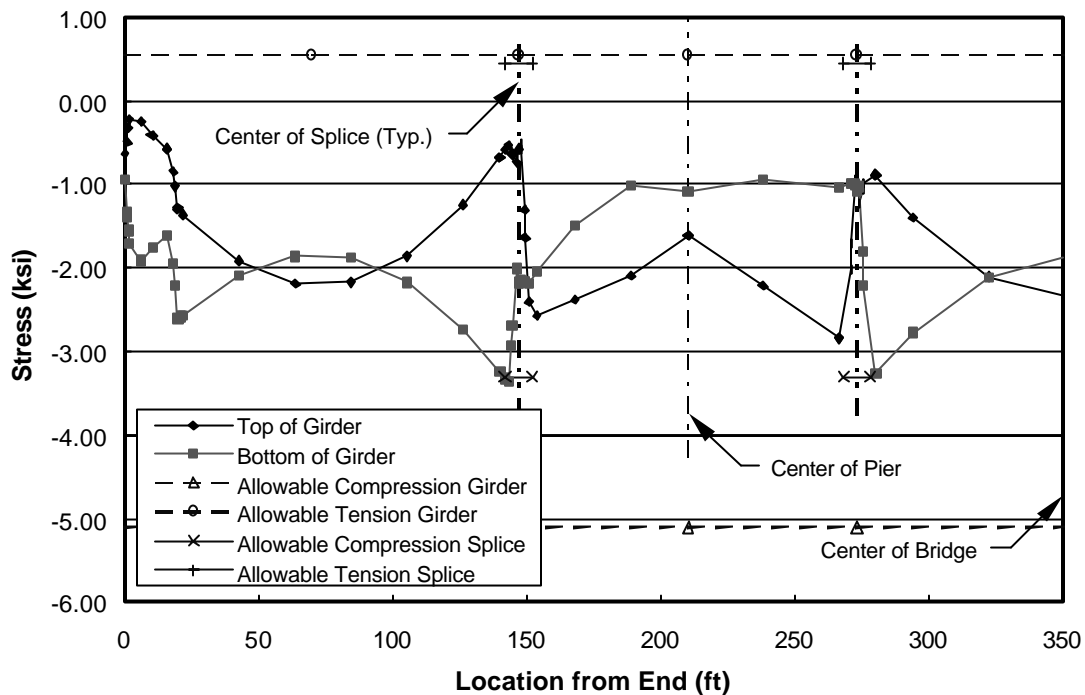
Figure 8.1-4   Girder Stresses for Service I - Stage 5 [PT Stage 2]

D3-35

Continuous Three Span Girder   **Design Example 3**
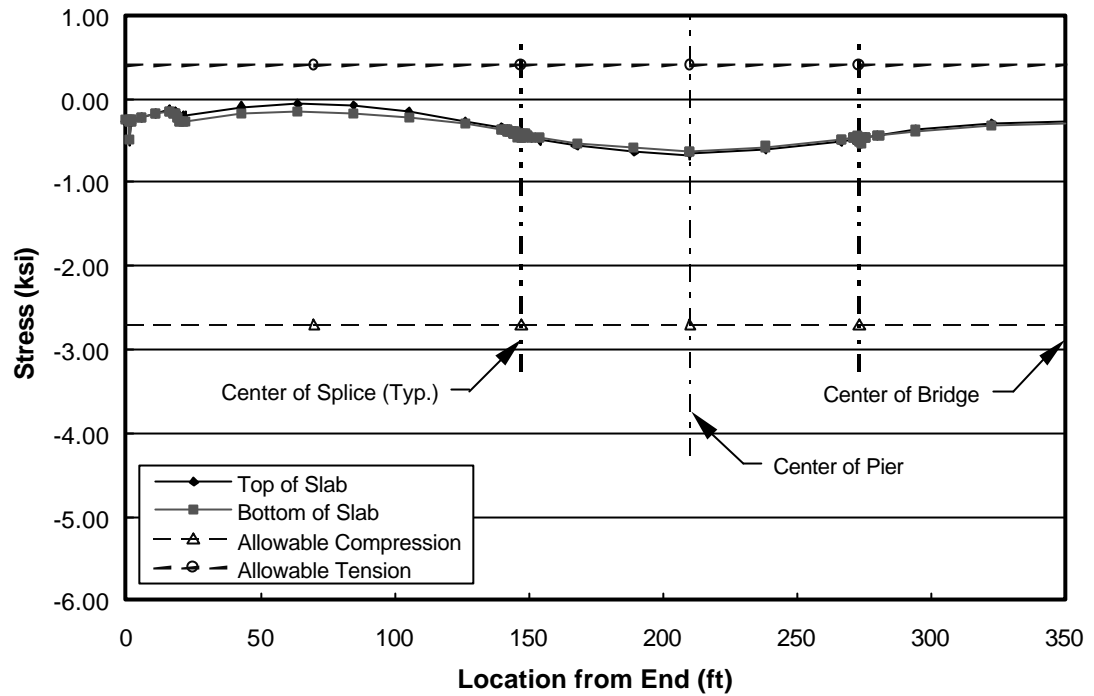
Figure 8.1-5   Slab Stresses for Service I - Stage 5 [PT Stage 2]
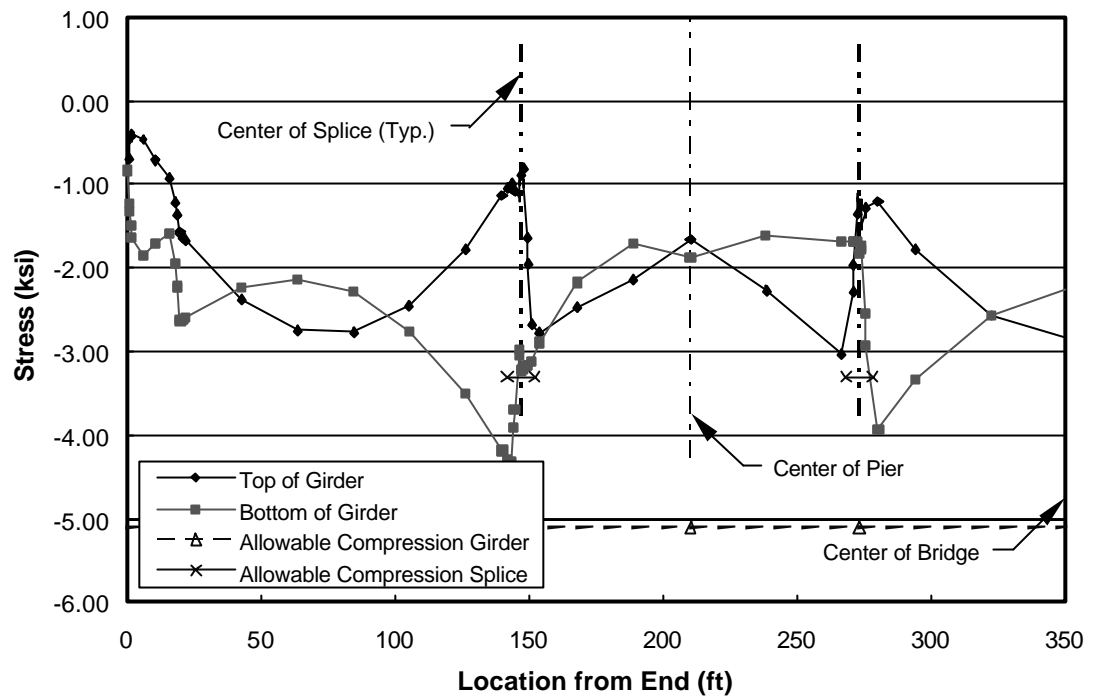
Figure 8.1-6   Girder Stresses for Service I (*DL + PS + LL*) - Stage 7 [Bridge Open]

D3-36

Continuous Three Span Girder   **Design Example 3**

Figure 8.1-7   Slab Stresses for Service I (*DL + PS + LL*) - Stage 7 [Bridge Open]
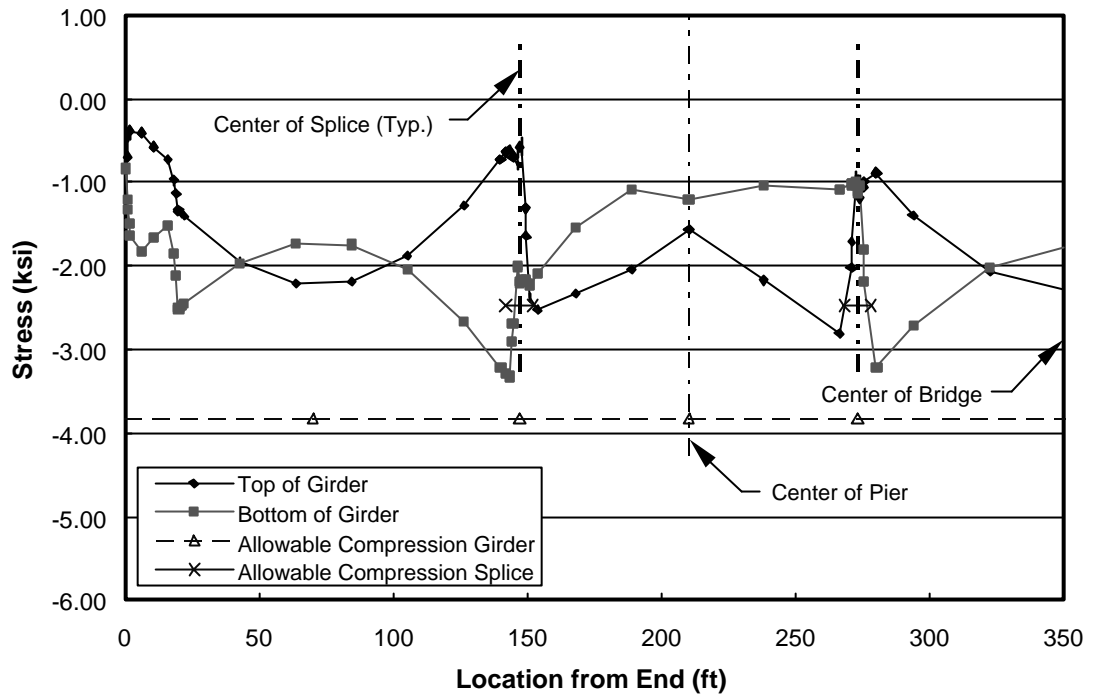
Figure 8.1-8   Girder Stresses for Service I (*DL + PS*) - Stage 7 [Bridge Open]

Continuous Three Span Girder   Design Example 3
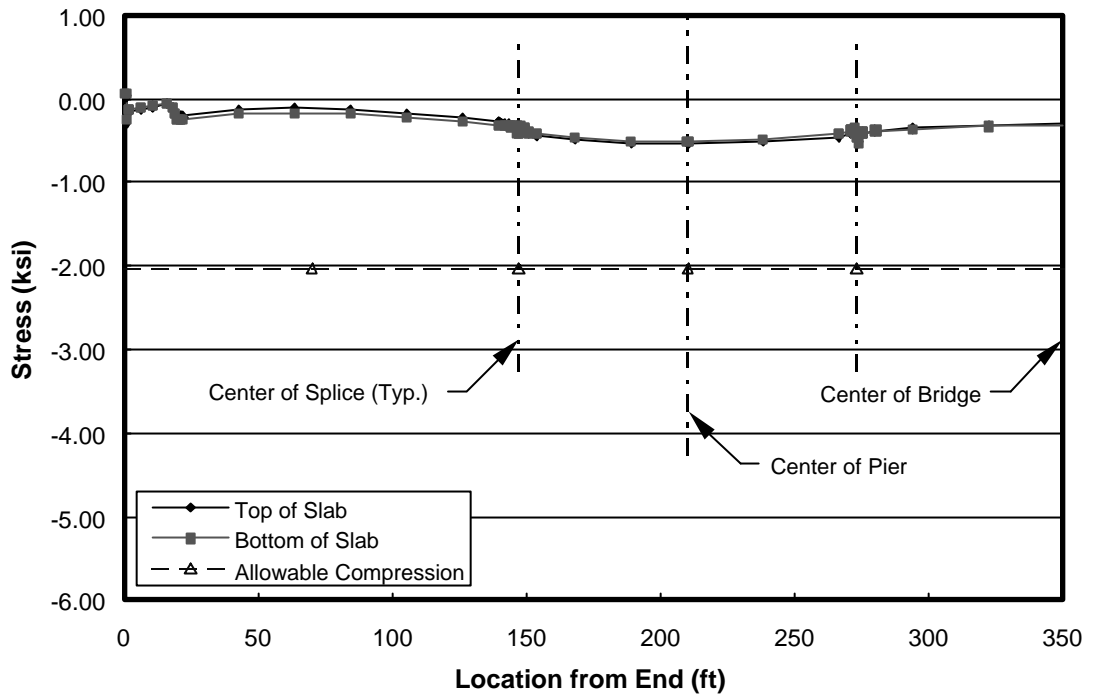
Figure 8.1-9   Slab Stresses for Service I (*DL* + *PS*) - Stage 7 [Bridge Open]
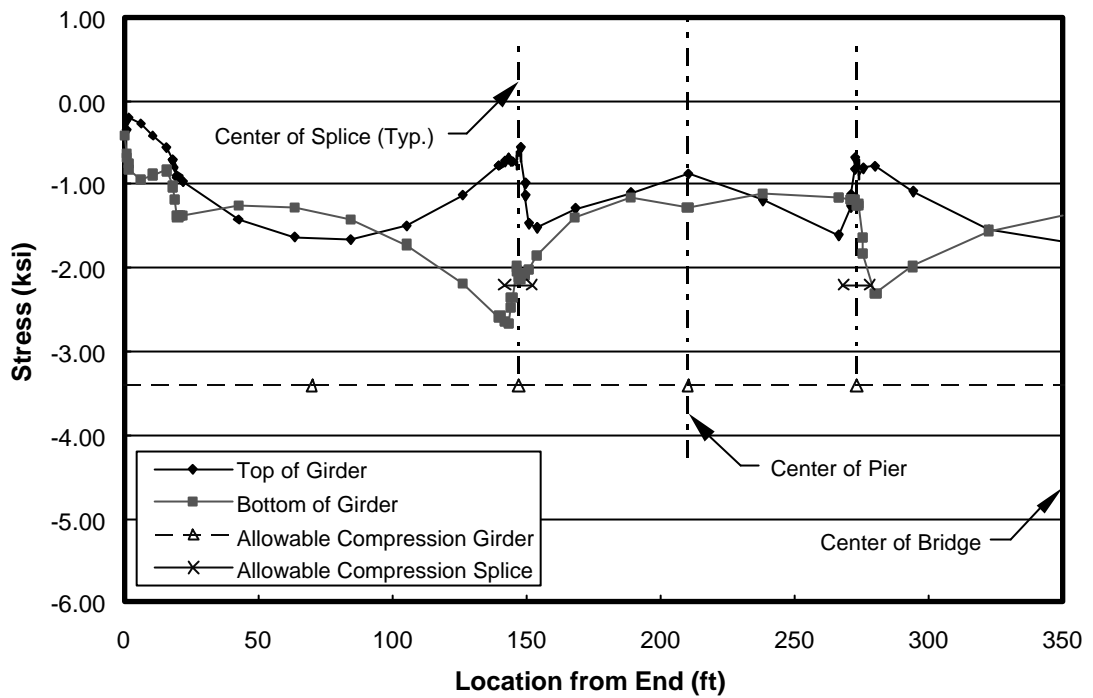
Figure 8.1-10   Girder Stresses for Service I (0.5(*DL* + *PS) + LL*) - Stage 7 [Bridge Open]

D3-38

Continuous Three Span Girder   **Design Example 3**
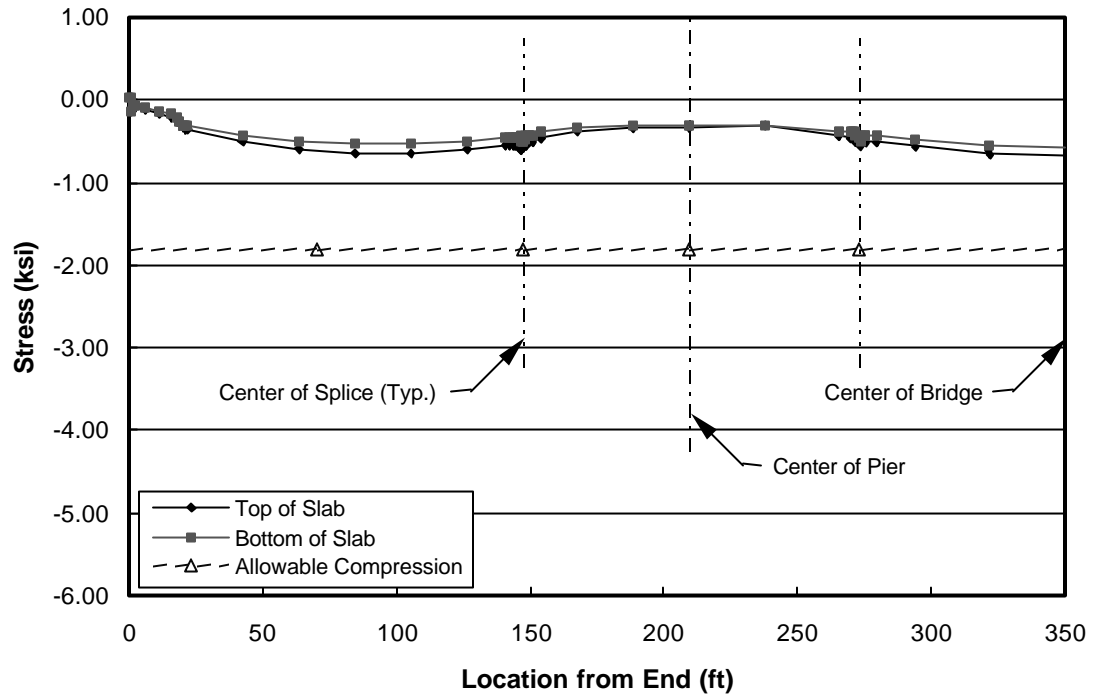
Figure 8.1-11   Slab Stresses for Service I (0.5(*DL + PS) + LL*) - Stage 7 [Bridge Open]
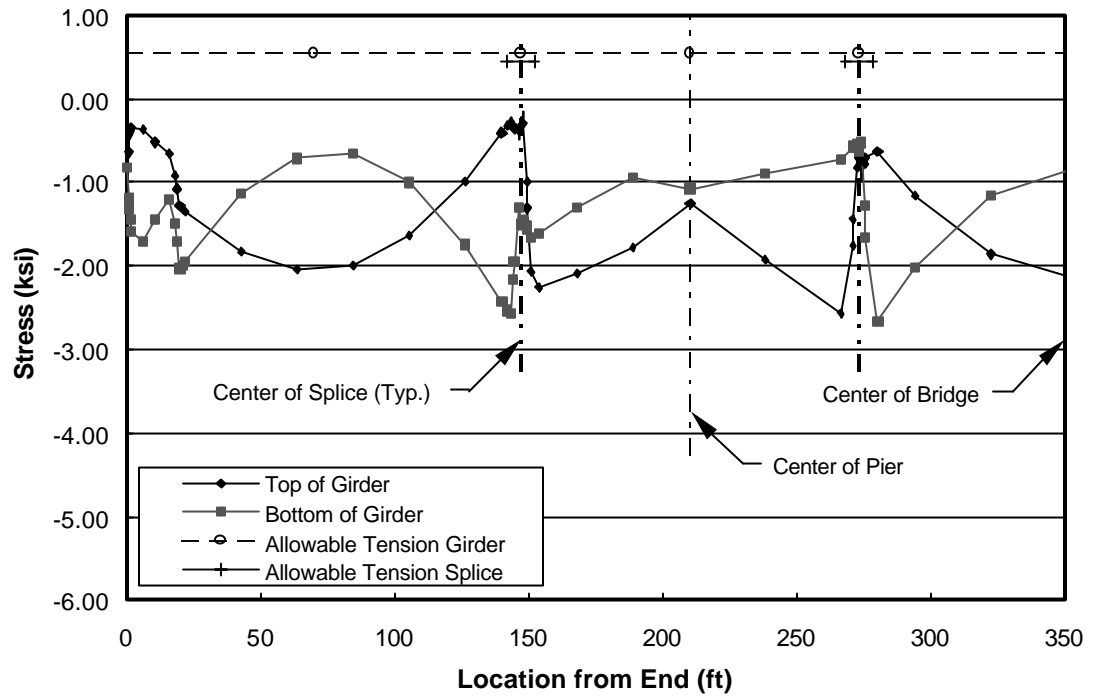
Figure 8.1-12   Girder Stresses for Service III - Stage 7 [Bridge Open]

D3-39

Continuous Three Span Girder   **Design Example 3**
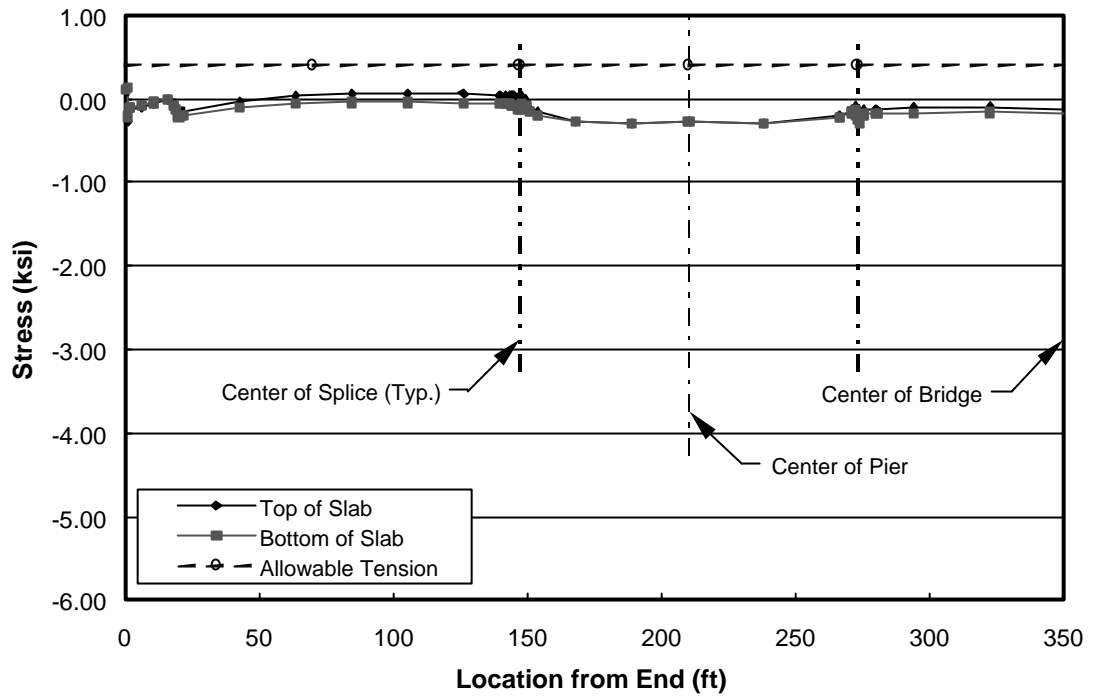
Figure 8.1-13   Slab Stresses for Service III- Stage 7 [Bridge Open]
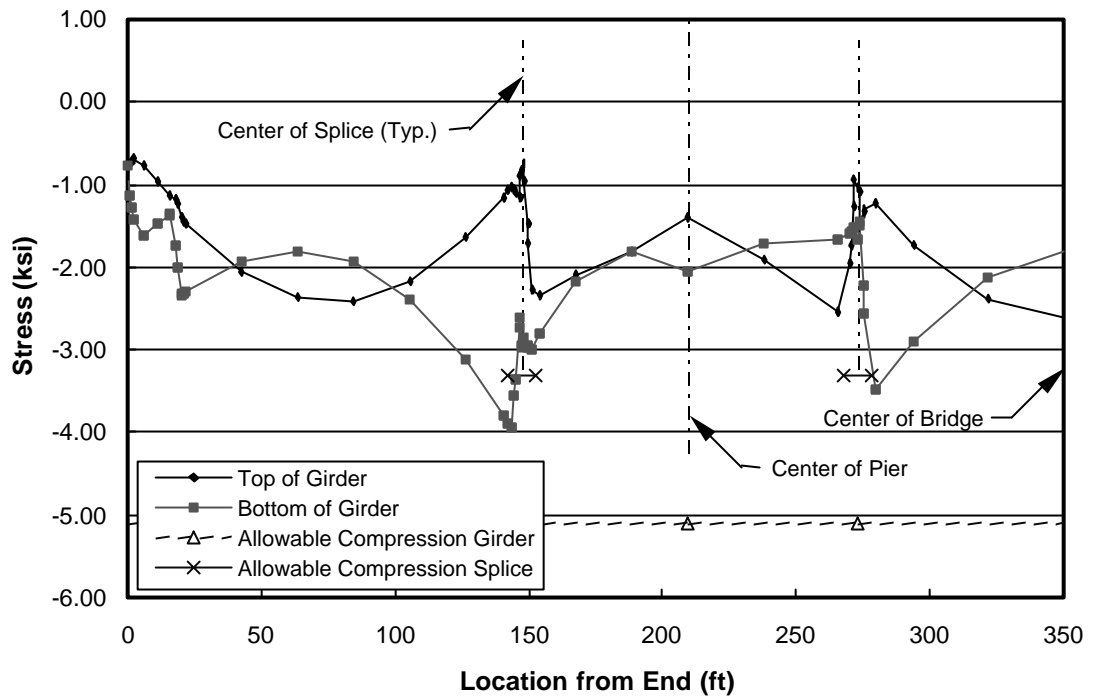
Figure 8.1-14   Girder Stresses for Service I (*DL* + *PS* + *LL*) - Stage 9 [Service Life]

D3-40

Continuous Three Span Girder
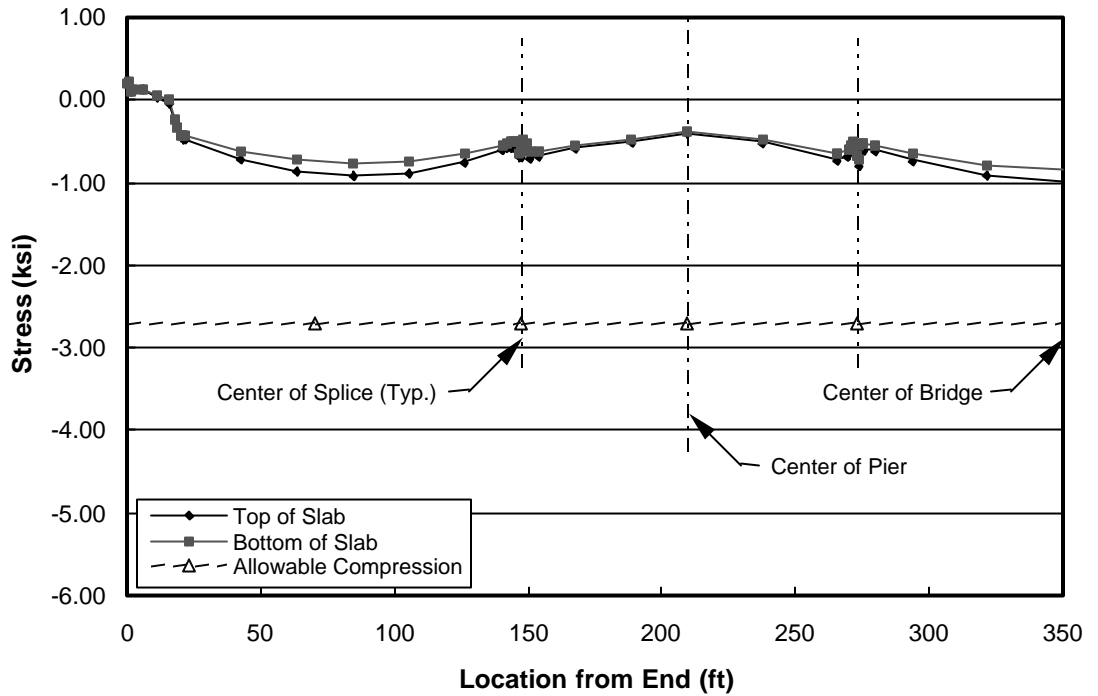
Design Example 3

Figure 8.1-15   Slab Stresses for Service I (*DL + PS + LL*) - Stage 9 [Service Life]
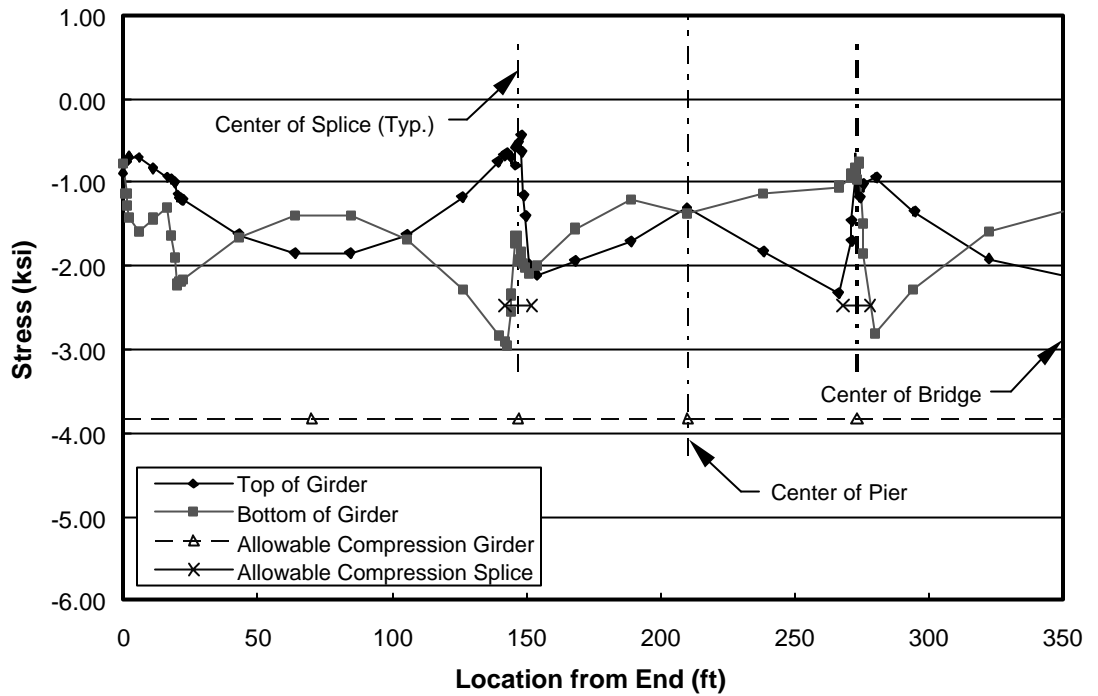
Figure 8.1-16   Girder Stresses for Service I (*DL + PS*) - Stage 9 [Service Life]

D3-41

Continuous Three Span Girder   **Design Example 3**
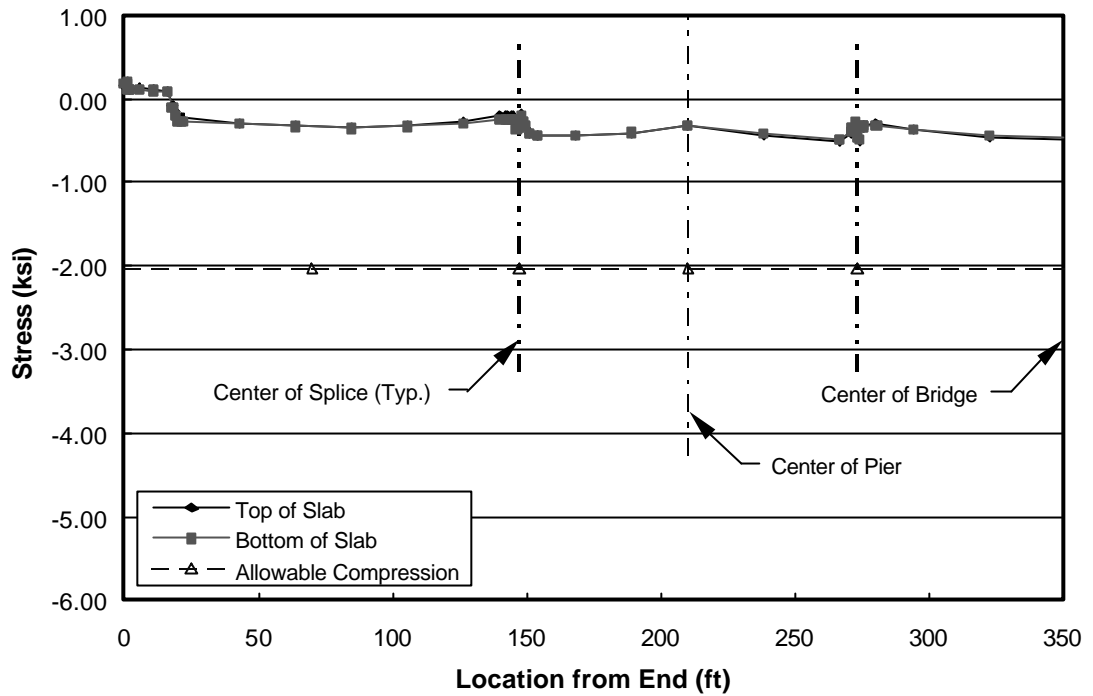
Figure 8.1-17   **Slab Stresses for Service I (*DL + PS*) - Stage 9 [Service Life]**
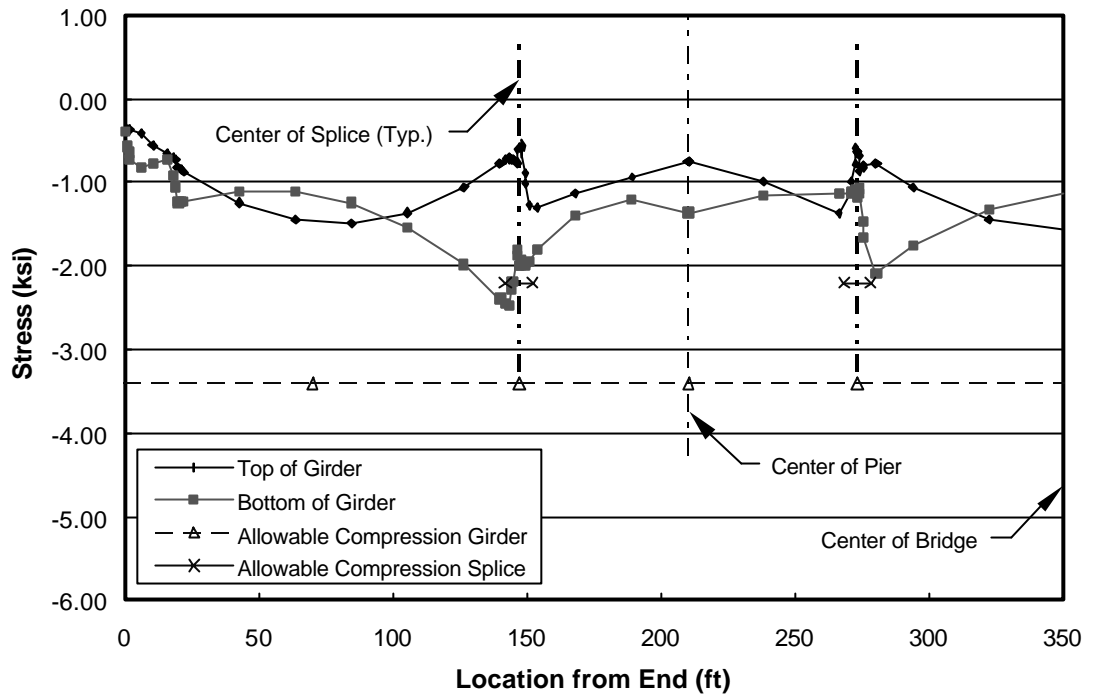
Figure 8.1-18   **Girder Stresses for Service I (0.5(*DL + PS) + LL*) - Stage 9 [Service Life]**

D3-42

Continuous Three Span Girder   **Design Example 3**
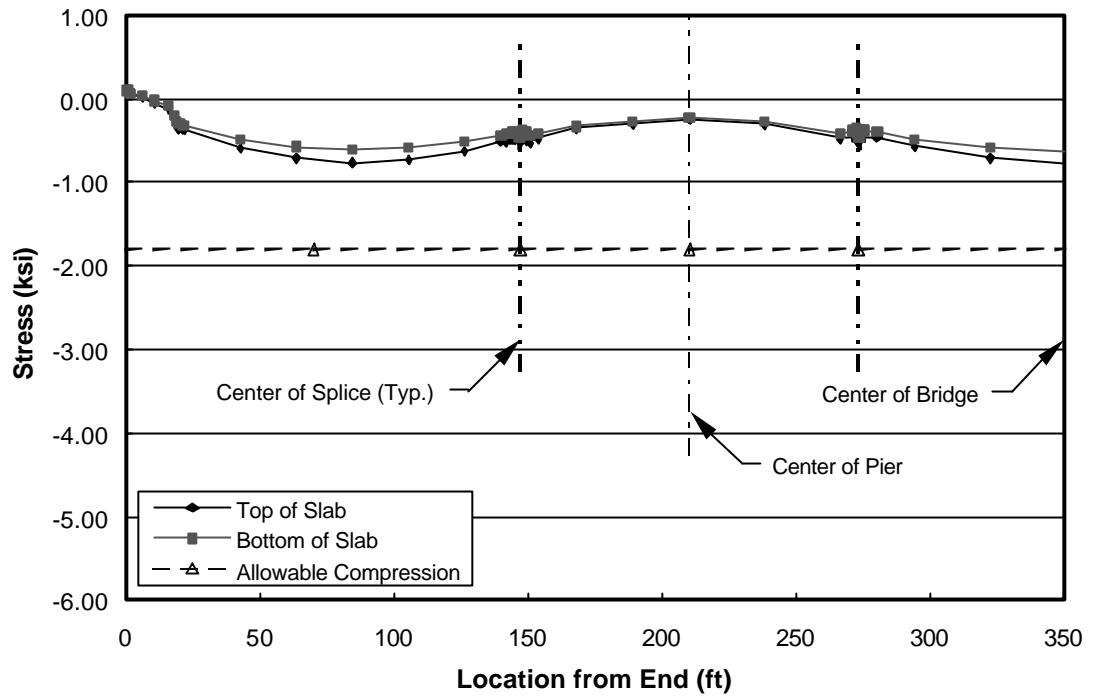
Figure 8.1-19   Slab Stresses for Service I (0.5(*DL* + *PS)* + *LL*) - Stage 9 [Service Life]
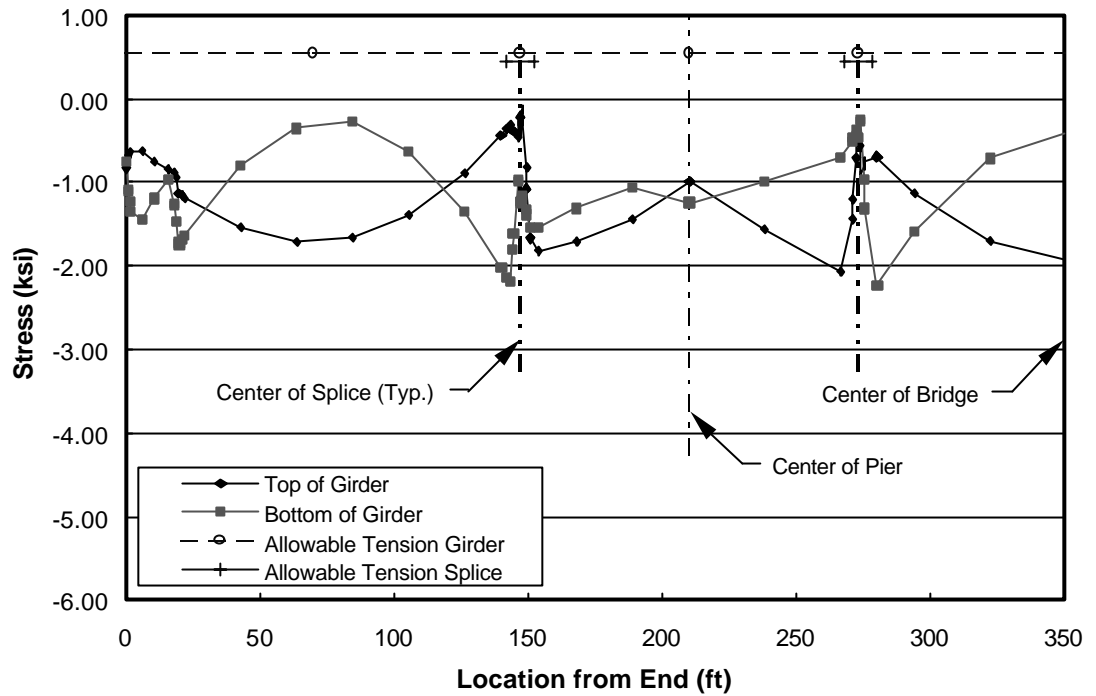
Figure 8.1-20   Girder Stresses for Service III- Stage 9 [Service Life]

D3-43
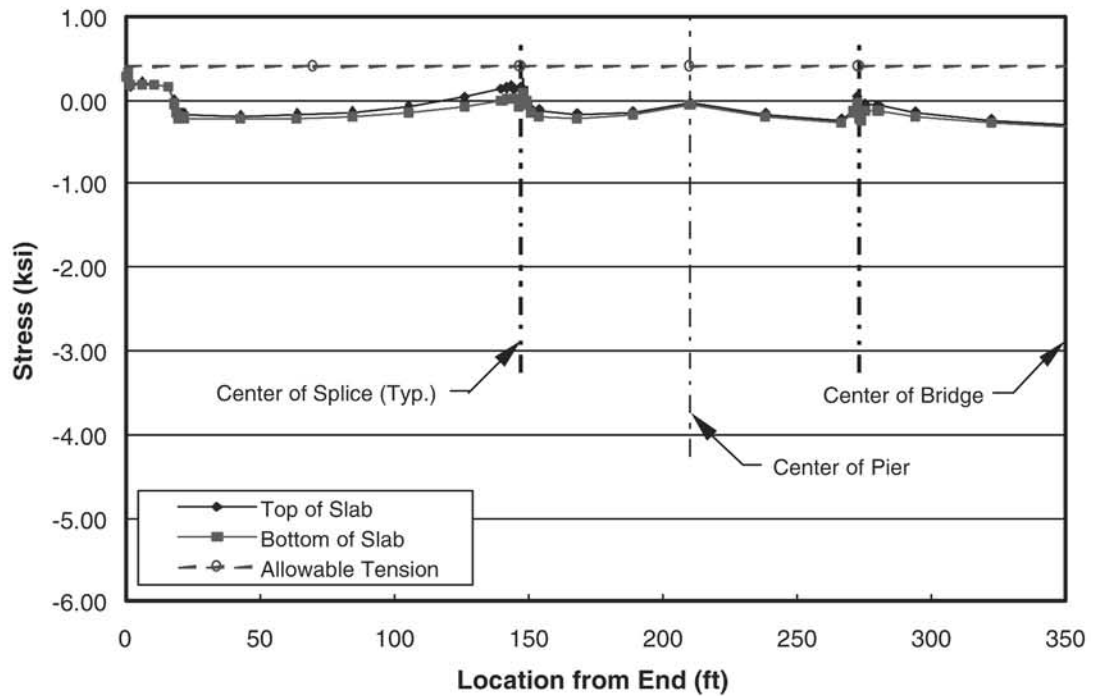
Figure 8.1-21   Slab Stresses for Service III - Stage 9 [Service Life]

8.2   STRENGTH LIMIT STATE

After design is completed at the service limit state, the girder is checked at critical sections for flexural capacity. The contribution of the build-up to section depth is neglected in all calculations.

The strength limit state is only checked at the condition at which the bridge is first opened to traffic and at the end of the service life of the bridge. It is assumed that strength is adequate for intermediate loading conditions.

The factored moment capacity required versus the capacity provided at Stages 7 and 9, which are with full live load, are shown in **Figures 8.2-1** and **8.2-2**. The splice locations are the controlling points, because only the post-tensioning tendons cross the joints. Mild reinforcement could be added to supplement the moment capacity provided by post-tensioning tendons.

Continuous Three Span Girder   **Design Example 3**
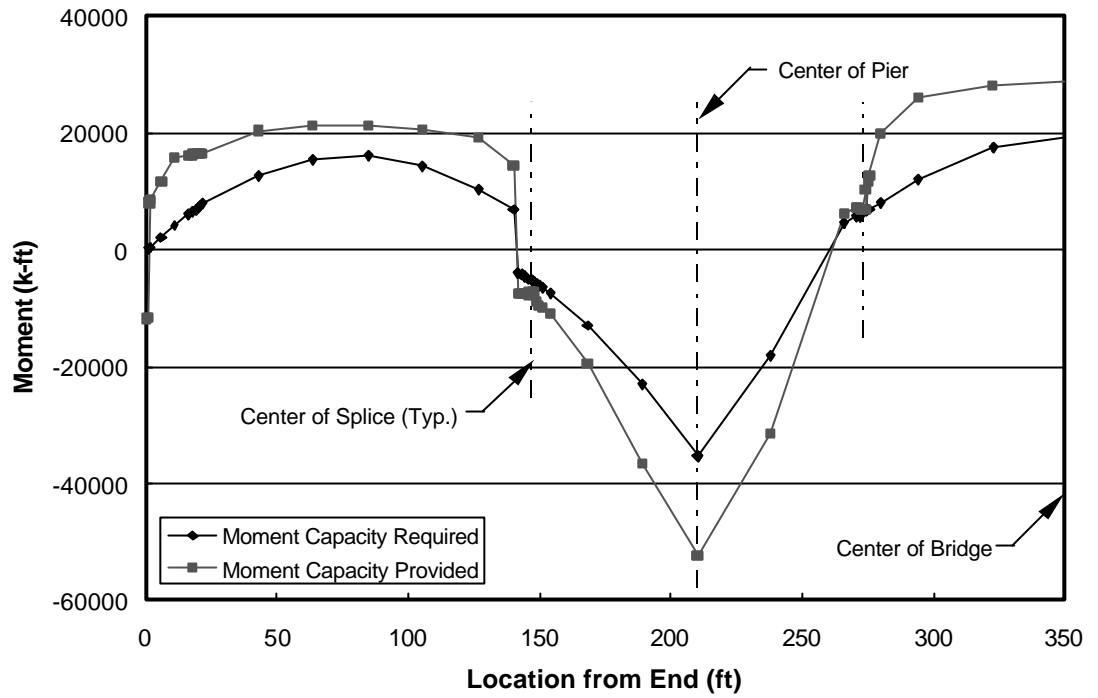
Figure 8.2-1   Moment Capacity and Demand for Load Combination Strength I –
                     Stage 7
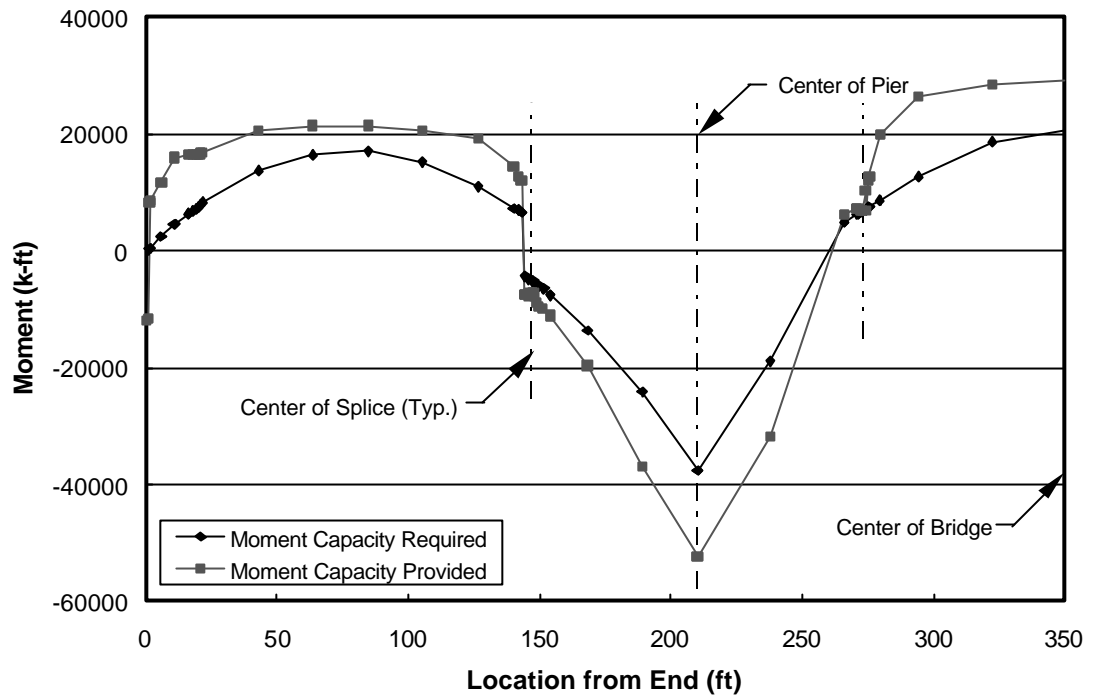
Figure 8.2-2   Moment Capacity and Demand for Load Combination Strength I –
                     Stage 9

D3-45

9   DESIGN FOR SHEAR

When performing the shear design of girders with post-tensioning, the effect of the ducts must be considered when the ducts are located within the web. LRFD Article 5.8.2.9 requires that one-quarter of the diameter of a grouted duct be deducted from the web width used for shear, b_w. Since the ducts are located within the web for the full length of the girder, the web width must be reduced for shear calculations.

Shear calculations are not shown for this design example. Consult **Design Example 1** for a discussion of shear design.

10   LONGITUDINAL REINFORCEMENT REQUIREMENT

This requirement, found in LRFD Article 5.8.3.5, is not critical for post-tensioned girders because the tension force is anchored at the end of the girder. Therefore, the entire capacity of the post-tensioning tendons is available up to the end of the girder. Since this requirement is not critical for spliced girders, the computation is not shown here.

11   **DEFLECTIONS**

The software package generates a finite element (FE) model to obtain displacements.

The analysis is performed incrementally by forming element stiffness and element loadings interval-by-interval and then solving for the incremental displacements

11.1   INITIAL DESIGN

The initial design for the bridge (See **Appendix A**), met all the service and strength limits specified by the *LRFD Specifications*. However, the bridge failed the optional live load limit specified in LRFD Art. 2.5.2.6.2. Thus a redesign with increased stiffness and prestress force was required. The dead load deflection of the initial design is shown in **Figure 11.1-1**. The live load deflection, as computed by the software package, is shown in **Figure 11.1-2**.

As shown in **Figure 11.1-1**, the maximum dead load deflection was 1.25 in. when the bridge open to traffic and 2.8 in. at the end of the service life. These values are fairly large, but could be dealt with by modifying the build-up or by adjustment of the support elevations

The maximum live load deflection for this bridge was 3.7 in. for the exterior spans and 5.2 in. for the interior span. Both of these values exceed the allowable live load deflection specified in LRFD Art. 2.5.2.6.2 as the span length divided by 800 (See **Table 11.1-1**). Therefore, the bridge required additional prestress force and section stiffness to meet the requirement.

Table 11.1-1   Initial Design Live Load Deflections

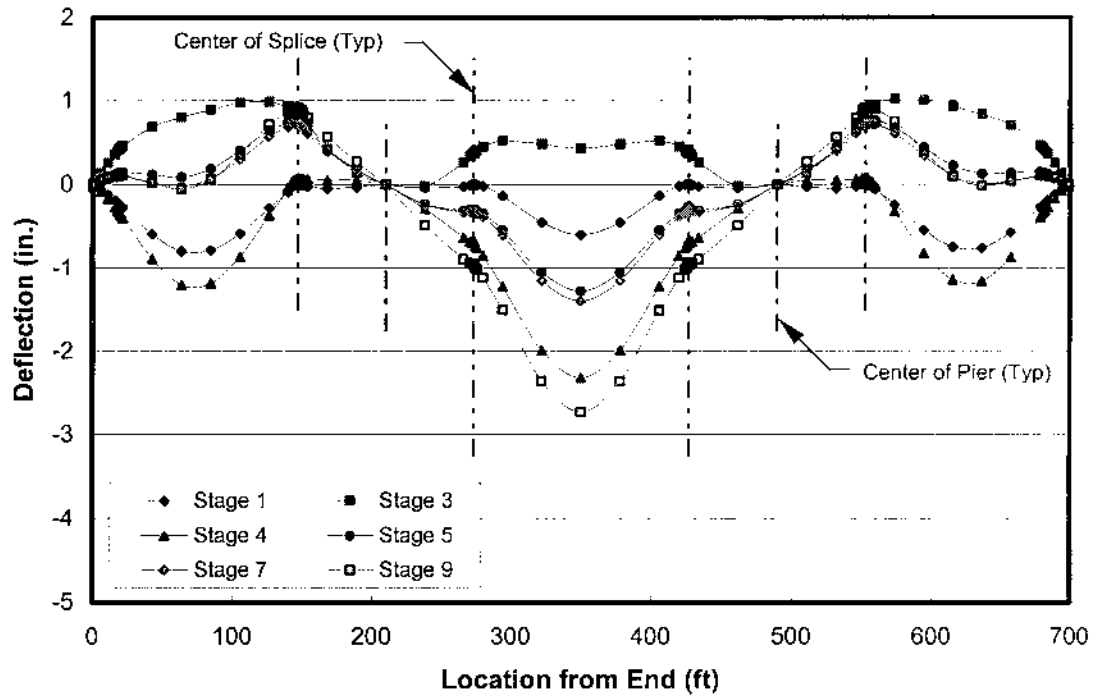| Span	Maximum Live Load (in.)	Allowable Live Load (in.)
Exterior Span	3.7	3.15
Interior Span	5.2	4.20

Figure 11.1-1   Initial Design – Dead Load Deflections of Spliced Girder
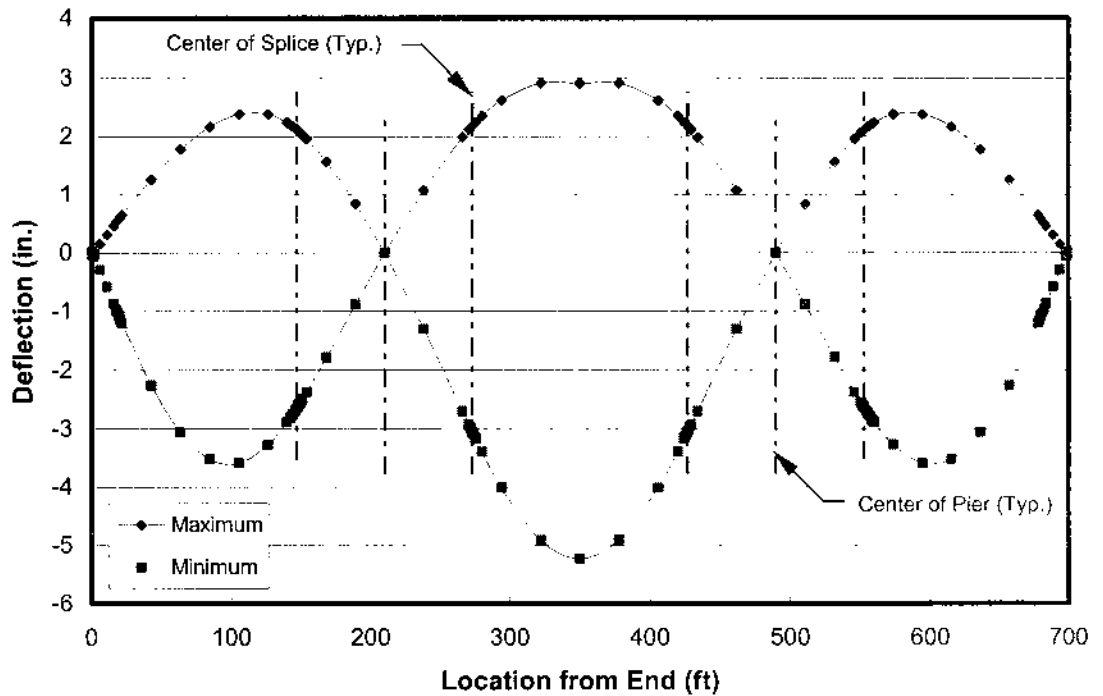

Figure 11.1-2   Initial Design – Live Load Deflections of Spliced Girder

Continuous Three Span Girder   **Design Example 3**

11.2	**FINAL DESIGN**

11.2.1	**Dead Load Deflection**

The dead load deflection of the final design improved over that of the initial design shown in **Figure 11.1-1**. The dead load deflection of the bridge when first opened is 0.4 in. and at the end of the service life is only 1.2 in. as shown in Figure **11.2.1.1-2**. These deflection can be accommodated by the build-up initially estimated for the design. However, it is preferable to have camber when the bridge opens to traffic and at the end of the service life. This can be achieved by adjusting bearing elevations at the permanent piers and the temporary support towers.

11.2.1.1	**Unadjusted Dead Load Deflections**

Figures 11.2.1.1-1 and **11.2.1.1-2** show the unadjusted dead load deflection of the bridge. The first figure shows the deflections during the stages of the girder erection process used for this design. The second figure shows the deflections at different stages of construction similar to **Figure 11.1-1**.
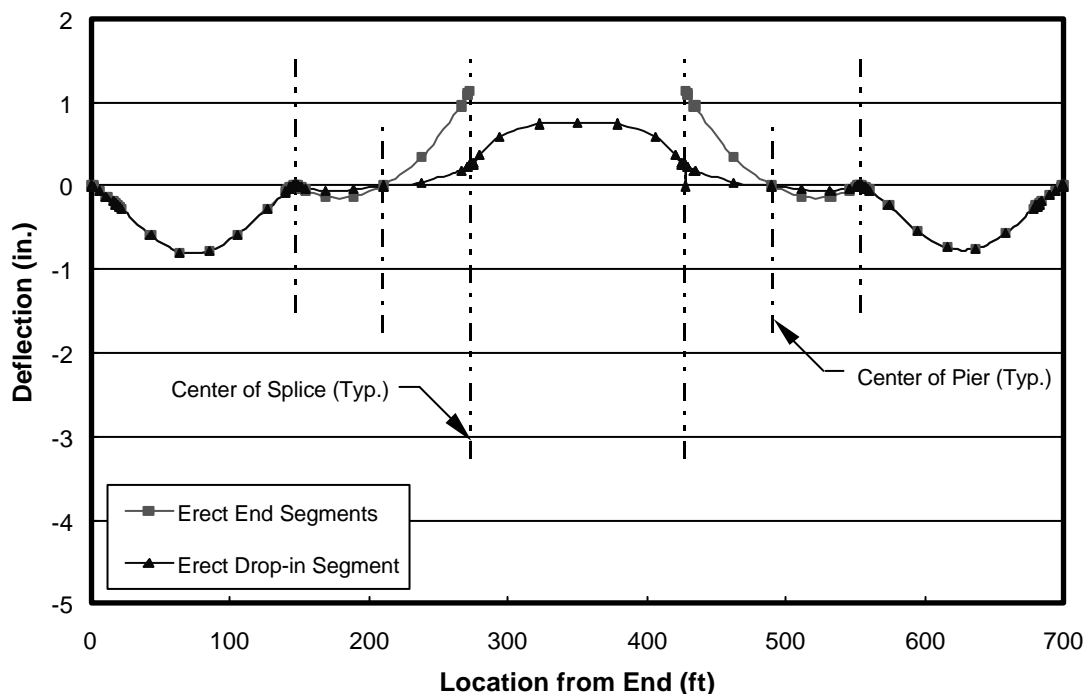
Figure 11.2.1.1-1 Final Design - Deflection of Girder Segments at Erection (Unadjusted)

D3-48
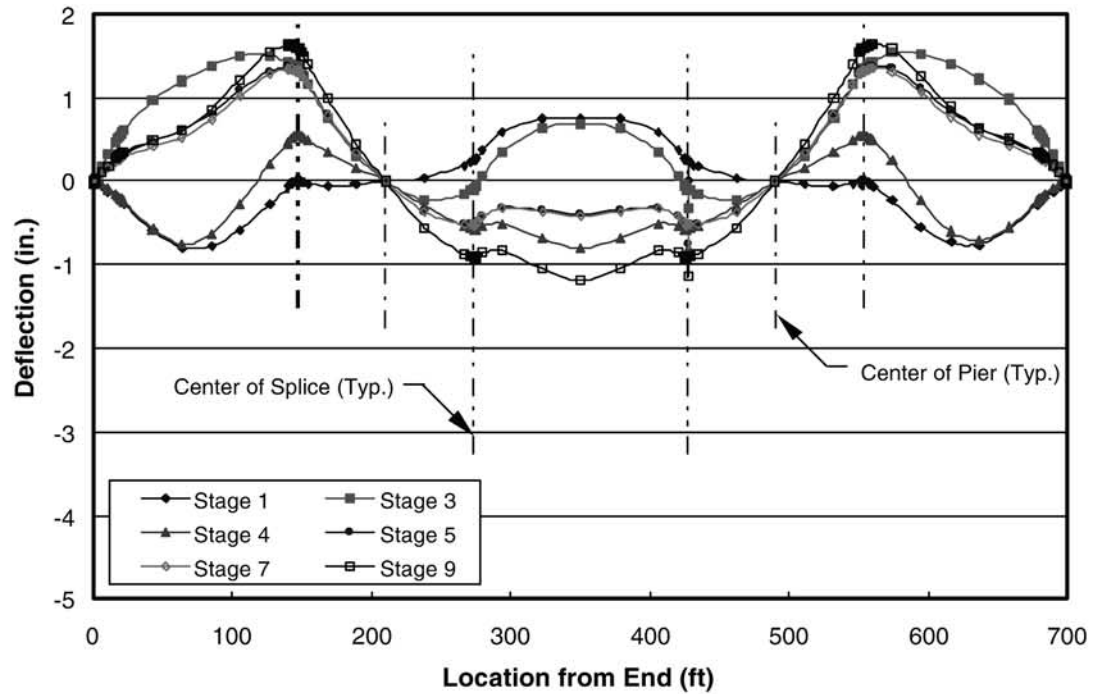
Figure 11.2.1.1-2 Final Design – Dead Load Deflection of Spliced Girder (Unadjusted)

11.2.1.2   **Adjusted Dead Load Deflection**

In order to reduce the final downward sag of the bridge and to minimize the required build-up, the elevations of the bearings at some supports were adjusted. To achieve this, the bearing elevations at the interior permanent piers were raised 0.5 in. and the bearing elevations at the temporary supports were lowered 0.5 in. with respect to the grade line. The results of this adjustment are shown in **Figures 11.2.1.2-1** through **11.2.1.2-3**.

As shown in **Figure 11.2.1.2-1**, the initial upward camber of the cantilevered tip of the pier girder segment is increased from 1.2 in., as shown in **Figure 11.2.1.1-1**, to 2.2 in. in **Figure 11.2.1.2-1**. This initial adjustment increases the camber in the center span and reduces the camber in the exterior spans. Overall, this results in a modified final deflection as shown in **Figure 11.2.1.2-3**. The final deflection of the bridge at the end of service life is 1.2 in. upward in the exterior spans and 0.2 in. downward at midspan of the interior span. When compared to the deflection of the unadjusted final design shown in **Figure 11.2.1.1-2**, which had a final upward camber of 1.6 in. in the exterior spans and a final downward sag of 1.2 in. in the interior span, it is clear that adjusting the supports elevations provide a better final grade profile.

Continuous Three Span Girder   **Design Example 3**
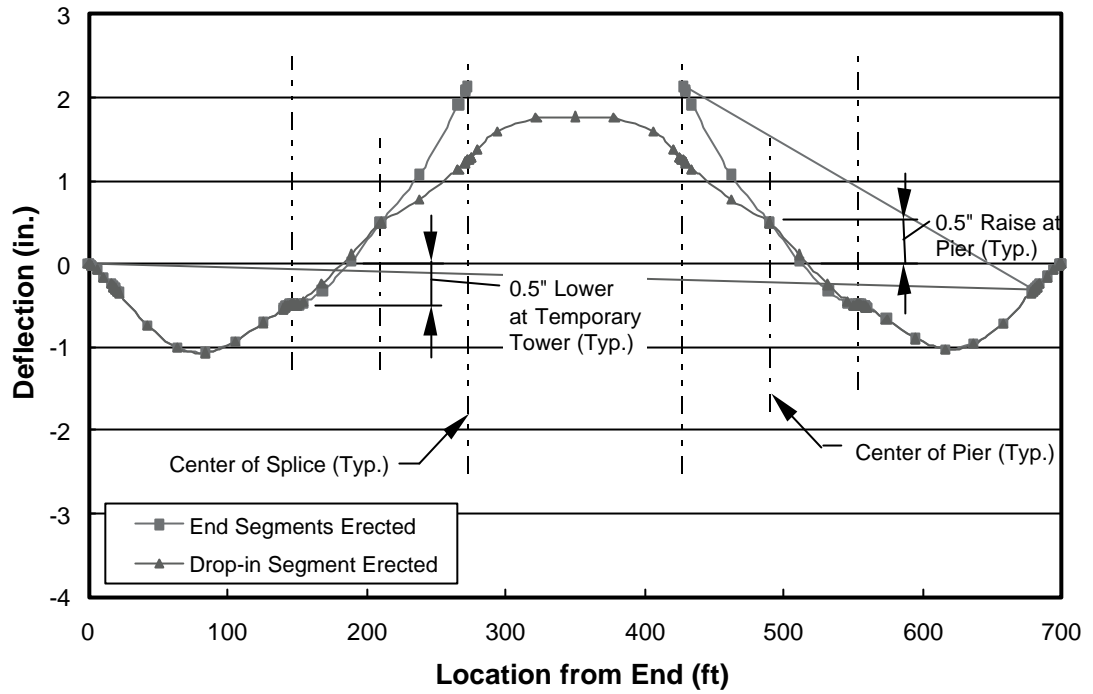
Figure 11.2.1.2-1 Final Design - Deflection of Girder Segments at Erection (Adjusted)
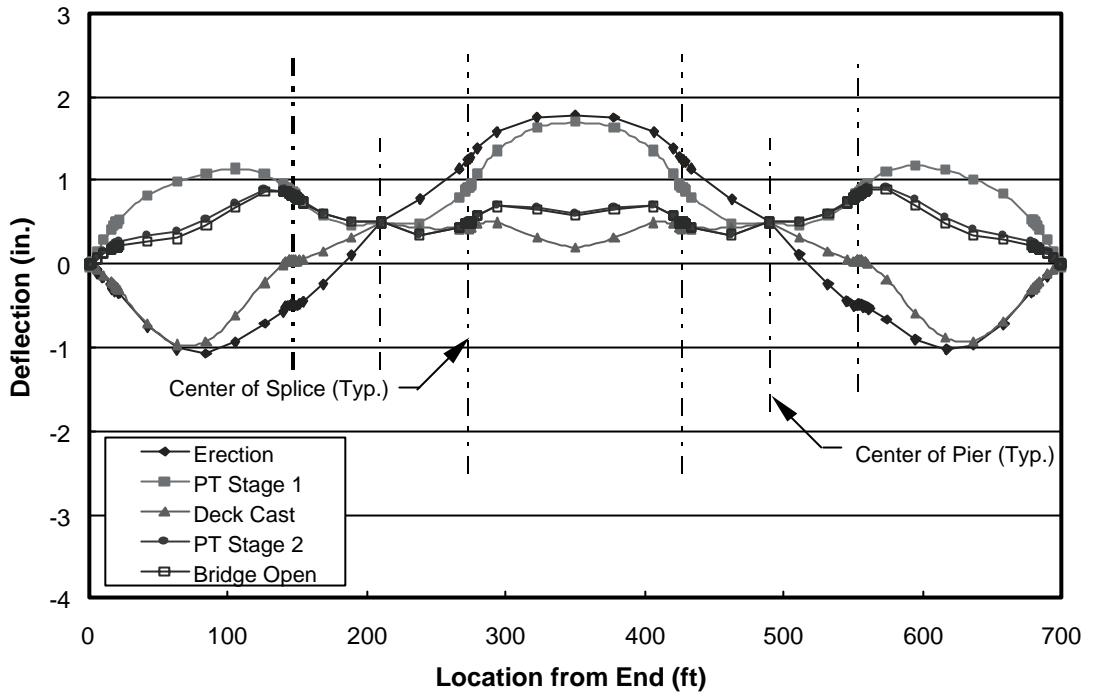
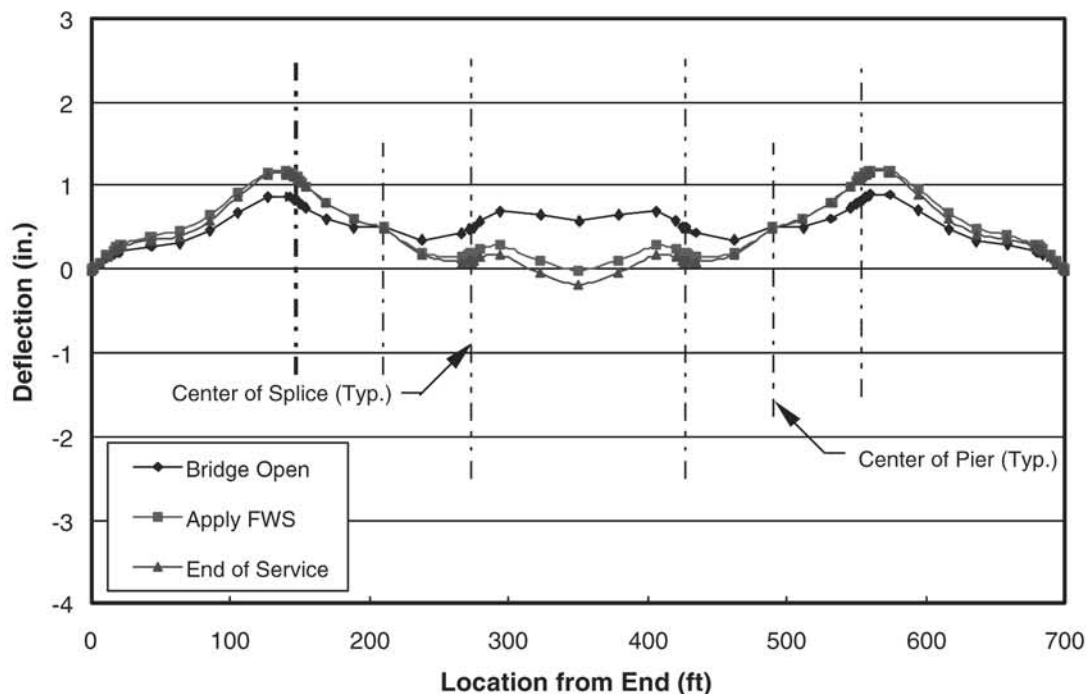**Figure 11.2.1.2-2 Final Design - Deflection of Spliced Girder at Intermediate Load Stages (Adjusted)**

D3-50

Figure 11.2.1.2-3 Final Design – Dead Load Deflection of Spliced Girder in Service (Adjusted)

11.2.2 Live Load Deflections

The maximum and minimum live load deflections computed by the design program are shown in **Figure 11.2.2-1**. The adjusted elevation bearing elevations do not have any affect on this figure.

The maximum live load deflection for this bridge was 3.16 in. for the exterior spans and 4.00 in. for the interior span. The maximum deflection of the interior span is well within the allowable limit as specified in LRFD Art. 2.5.2.6.2. The exterior span slightly exceeds the limit of 3.15 in. Since the exterior span deflection only exceeds the limit by 0.2%, it is considered acceptable. The maximum and allowable deflections are summarized in **Table 11.2.2-1**.

Table 11.2.2-1   Final Design Live Load Deflections

Span	Maximum Live Load Deflection (in.)	Allowable Live Load Deflection (in.)
Exterior Span	3.16	3.15
Interior Span	4.00	4.20
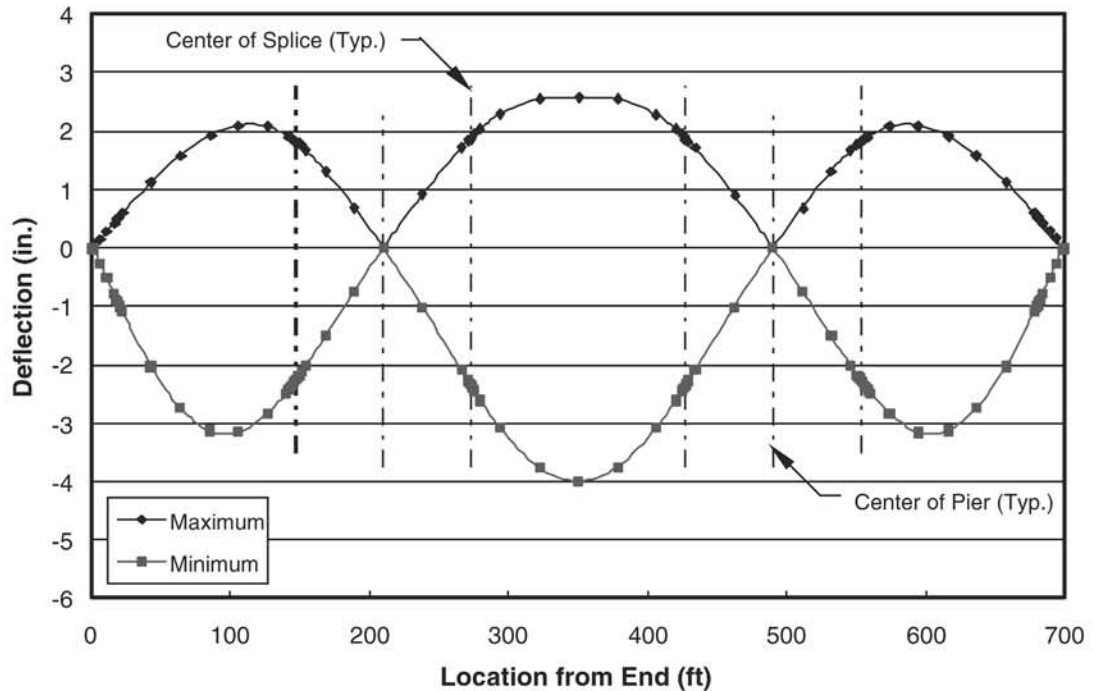
Figure 11.2.2-1   Live Load Deflection of Spliced Girder

12   ADDITIONAL DESIGN CONSIDERATIONS

12.1   SUPPORT REACTIONS

Support reactions during girder segment erection and other early stages of construction are shown in **Table 12.1-1**. The reactions are based on the construction sequence shown in **Figure 3.1-1**. The erection sequence and means of support for each girder segment has a significant effect on the reactions and design moments. Therefore, the designer must examine the effects of the erection sequence when designing the girder segments and the temporary support structures. The end girder segments are erected using strongbacks resting on the pier segments to help balance the weight of the drop-in girder segment. The pier segment must be tied down to the temporary tower to resist the uplift forces shown in **Table 12.1-1**.

Table 12.1-1   Support Reactions During Initial Stages

Action/Event	Initial Design			Final Design		
	End Bent (kips)	Temp. Support (kips)	Pier (kips)	End Bent (kips)	Temp. Support (kips)	Pier (kips)
Erect Pier Segments	0.0	-0.3	210.5	0.0	-4.0	245.1
Erect End Segments	121.6	101.2	207.3	121.6	97.5	241.9
Erect Drop-in Segment	121.6	-2.0	410.2	121.6	-14.4	462.2

Cast Splice	121.6	-1.9	415.6	121.6	-14.8	468.2
Cure Splice	121.4	-1.3	415.1	121.4	-13.3	467
PT Stage 1	121.1	----	414.7	124.4	----	450.7
Cast Deck	196.2	----	687.6	213.8	----	826.4

12.2 WEIGHT OF GIRDER SEGMENTS

The weights of the girder segments are shown in **Table 12.2-1**. It is important for a designer to investigate the availability of equipment for transporting and lifting the girder segments.

Table 12.2-1   Weights of Girder Segments

Girder Segments	Length (ft)	Weight (kips)
End Girder Segment	146	220
Pier Girder Segment	124	241
Drop-in Girder Segment	152	217

12.3 STRESSING OF POST-TENSIONING FROM ONE END OR BOTH ENDS

The post-tensioning tendons are stressed from both ends in this design example. This section considers the effect of stressing the post-tensioning tendons from only one end.

The difference in tendon force between tendons stressed from one end versus stressing from both ends is shown in **Figure 12.3-1**. The results show about a 180 kips difference at the end that is not stressed at the time of tensioning. This amounts to a 17% reduction in the force in the end span where the tendons are not stressed. When both stages of post-tensioning are considered, this difference increases to roughly 400 kips at the time of the second post-tensioning and a final difference of 350 kips. It is highly recommended that both ends be stressed sequentially in order to provide equal conditions in both end spans.

Another option available to the designer is to stress the first set of tendons from one end and the second set from the other end of the girder as shown in **Figures 12.3-2** and **12.3-3**. In this case, the difference at the time of the second post-tensioning is only about 190 kips initially and 160 kips at the end of the service life of the bridge. While this improves conditions, it is still recommended to stress all tendons from both ends if possible. However, if this is not feasible, stressing the first set of tendons from one end and the second set from the other end is a better option compared to stressing all tendons from one end only, since it results in more uniform stresses in the tendons for this bridge.

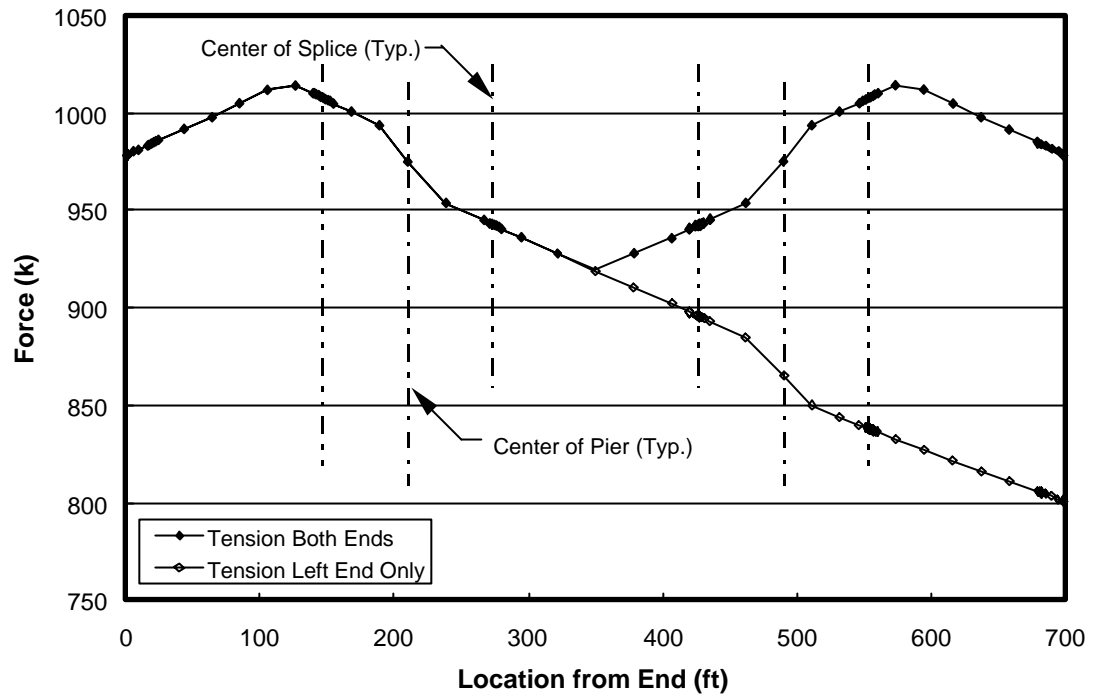## Continuous Three Span Girder

Design Example 3

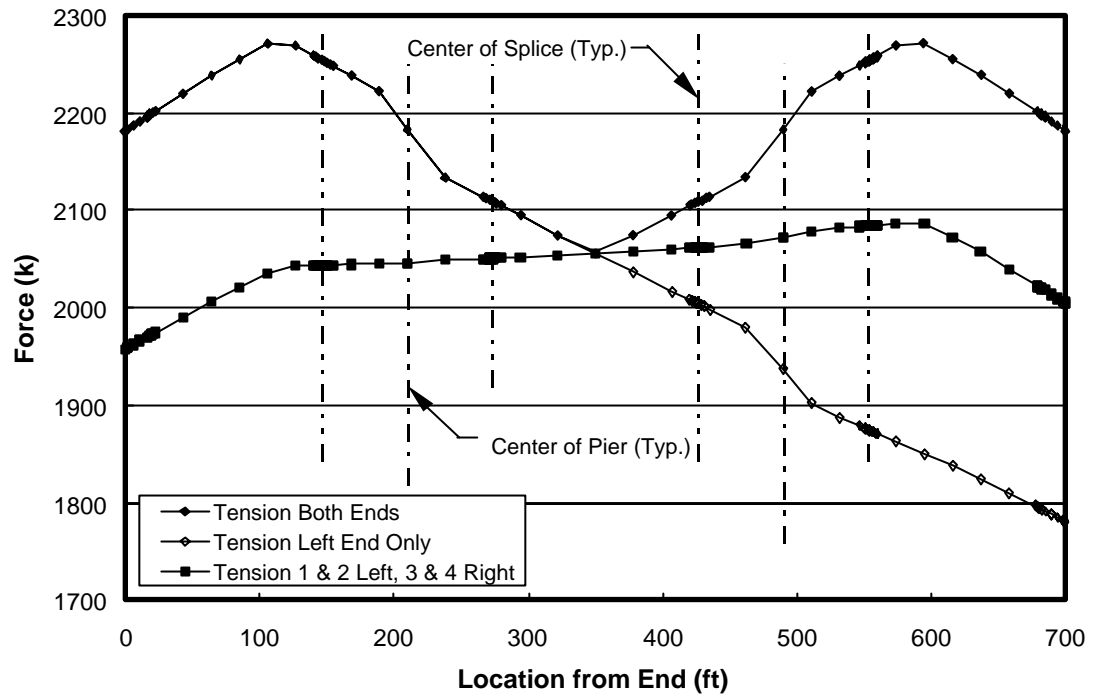**Figure 12.3-1   Tensioning Both Ends vs. Left End Only (Stage 3)**

Figure 12.3-2   Stressing at Both Ends vs. Left End Only vs. Left and Right Ends (Stage 5)
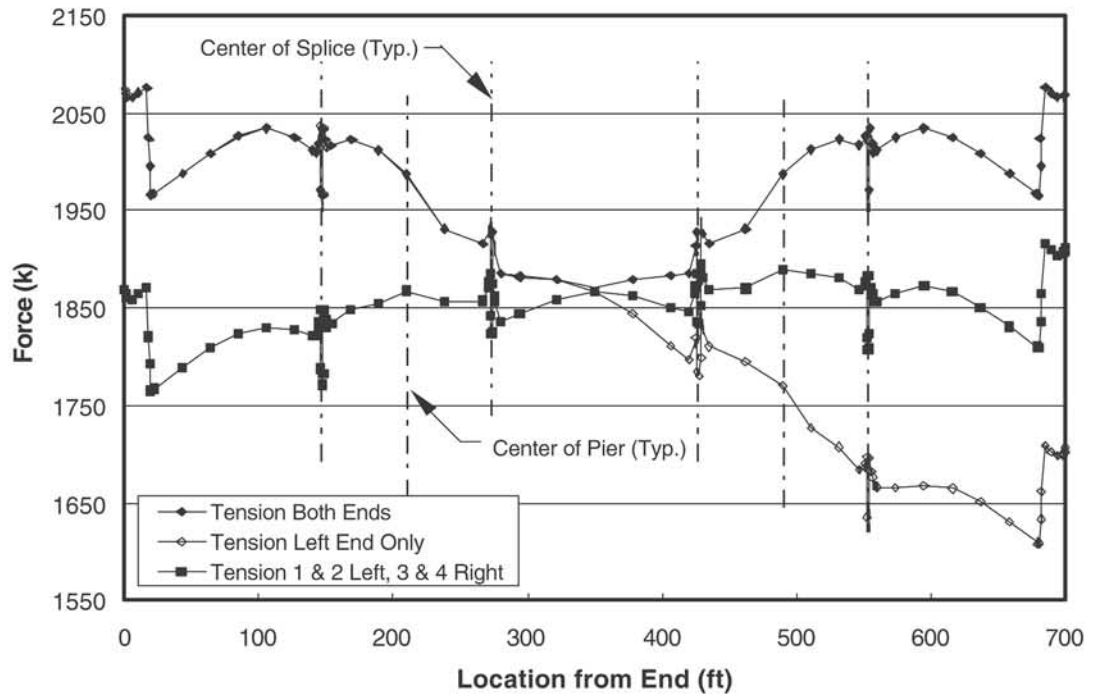
D3-54

Figure 12.3-3   **Stressing at Both Ends vs. Left End Only vs. Left and Right Ends (Stage 9)**

13   CONSTRUCTION METHODS AND IMPORTANCE IN DESIGN STAGES

13.1   TEMPORARY SUPPORTS AND STRONGBACKS

As noted in **Sections 6.1** and **12.1**, the method of erection has a significant effect on design considerations mainly for the haunched girders. The most popular method of erection is the use of temporary support towers at the splices in the end spans and of "strongback" hanger systems at the splices in the main span.

Temporary support towers may be constructed using scaffolding on footings or using steel piling driven to required capacity and then connected together with lateral bracing to form a tower. Another method used for support towers is to utilize steel wide flange sections as diagonal struts from the splice location to the top edge of the large concrete waterline footings in waterways. These footings are typically massive in size due to ship impact requirements and provide adequate capacity for use during construction.

It is recommended that designers owners and contact local contractors for preferred methods of erection. The section of this report entitled "History of Florida's Spliced Girder Bridges" (**Appendix C2**) also contains description of construction means and methods.

13.2　　**DECK SLAB CASTING SEQUENCE**

Traditionally, design plans have shown a casting sequence of the deck slab such that the positive moment regions are cast first, then the negative moment regions are cast after the initial concrete placements cure. This results in a "checkerboard" pouring sequence similar to continuous span structural steel girder construction. However, this is an expensive operation for the contractor due to the time and labor involved in waiting for days between pours and moving equipment back and forth over the continuous unit.

Experience has shown that contractors prefer, at significant cost savings, to cast the entire deck slab in a single operation. The contractor is required to develop the following: a concrete mix with set retarding admixtures in order to delay the initial set time of the concrete; to establish procedures for backup equipment if a failure were to occur; and to demonstrate the availability of an adequate supply of concrete from the mixing plant. In addition, the means and methods must be demonstrated to pour the entire deck slab unit prior to initial set of the concrete.

14　　**REFERENCES**

1. *AASHTO LRFD Bridge Design Specifications*, 2nd Edition with Interims, American Association of State Highway and Transportation Officials. Washington, DC. (1998-2003).

Continuous Three Span Girder   **Design Example 3**

APPENDIX A

The initial design of the bridge was very similar to that of the final design. The span arrangement and dimensions were unchanged between the two designs. The differences between the initial design and the final design include:

1. The drop-in segment of the initial design was a Florida Modified 78" BT.

2. The pier segments were 12'-6" deep with a constant bottom flange depth and equal cross-section at the splices.

3. The concrete strength of the deck and splice were 0.5 ksi lower than in the final design.

4. The drop-in segment had 20 pretensioned strands instead of 30.

5. The build-up height was initially assumed to be 1.5 in.

The figures in this appendix demonstrate these differences when compared to the corresponding figures in the body of the design example as indicated in **Table A-1**.

Table A-1   Figure Reference

Final Design		Initial Design	
Figure	Page #	Figure	Page #
2-1	D3-2	A-1	D3-57
2-2	D3-3	A-2	D3-58
2-3	D3-3	A-3	D3-58
5.4.1-2	D3-15	A-4	D3-59
6.1-3	D3-22	A-5	D3-59
6.1-4	D3-23	A-6	D3-60
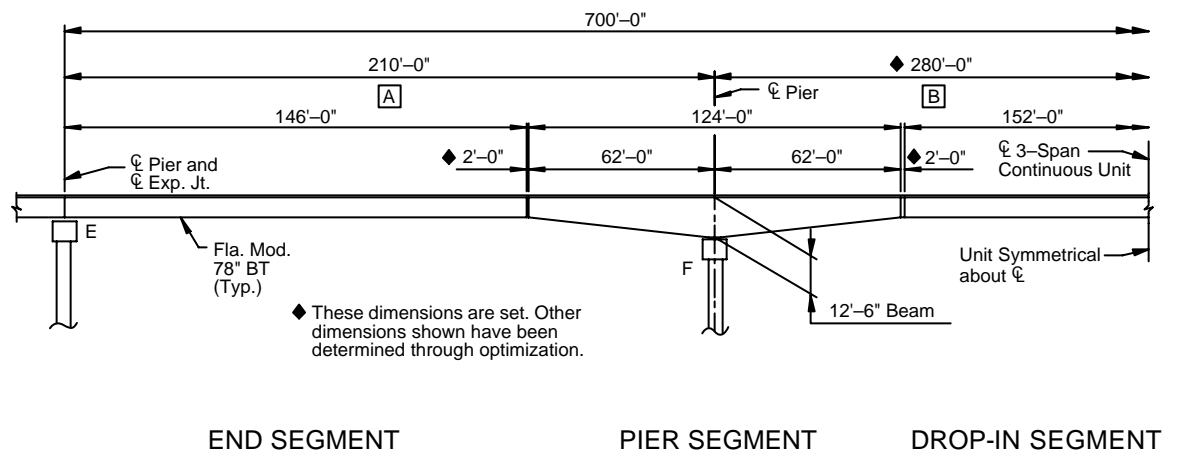
Figure A-1   Initial Design – Half Elevation of Bridge Girder Layout

D3-57

Continuous Three Span Girder   Design Example 3
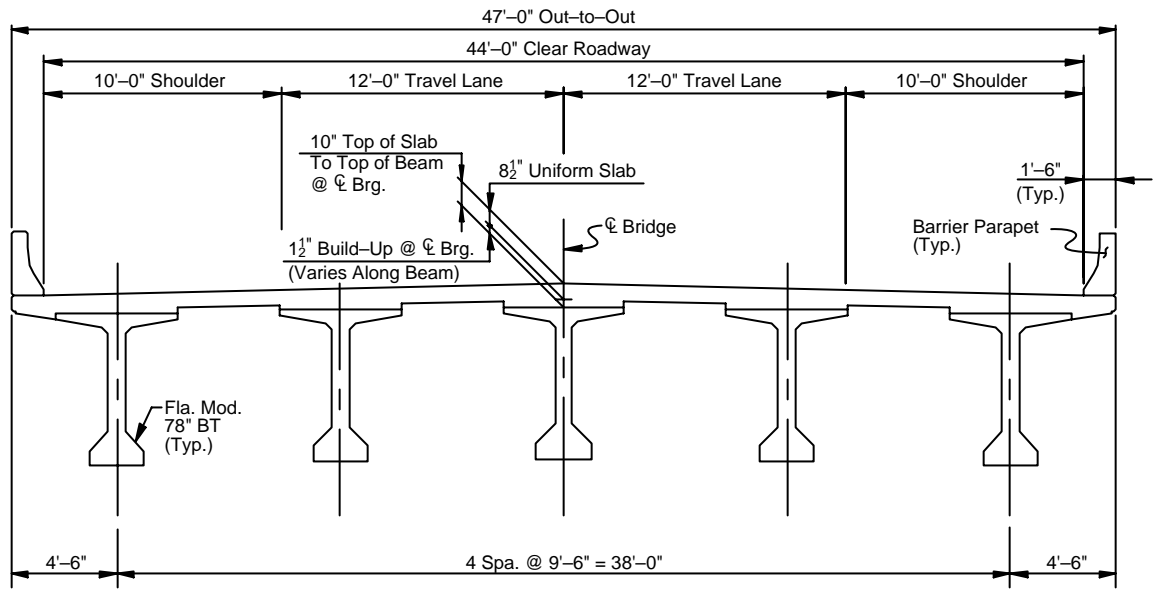

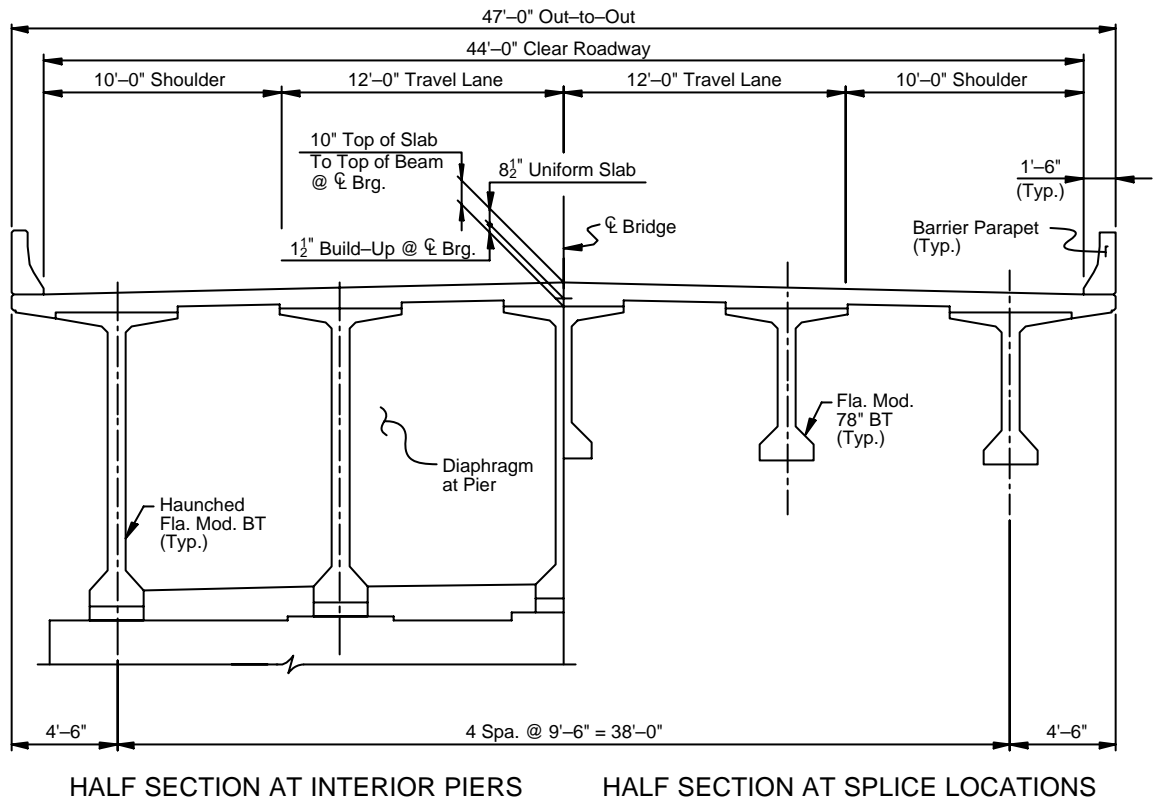**Figure A-2   Initial Design – Typical Section for Constant Depth Girders**

HALF SECTION AT INTERIOR PIERS   HALF SECTION AT SPLICE LOCATIONS

Figure A-3   Initial Design – Sections at Critical Locations

D3-58

Continuous Three Span Girder

Design Example 3

AT SPLICE

AT PIER
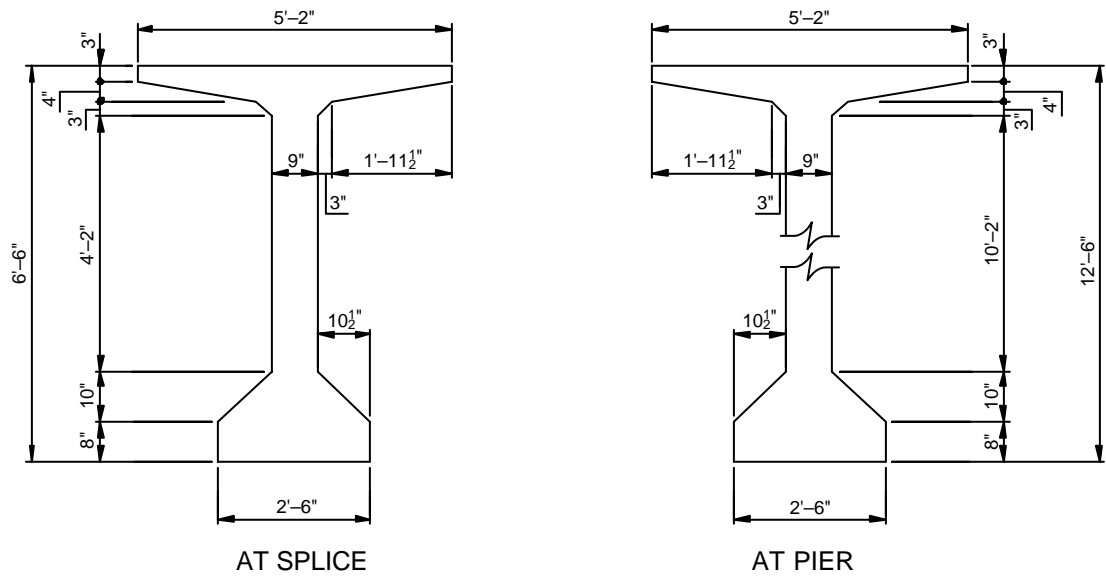
Figure A-4   Initial Design – Cross Sections of Pier Girder Segments

28–0.6" Dia.
LR Strands Stressed
To 43,950# Ea.

TENDON 4
TENDON 3
TENDON 2
TENDON 1

10–0.6" Dia. LR Strands
Stressed To 43,950# Ea.

AT PIER

AT SPLICE IN SPAN 2
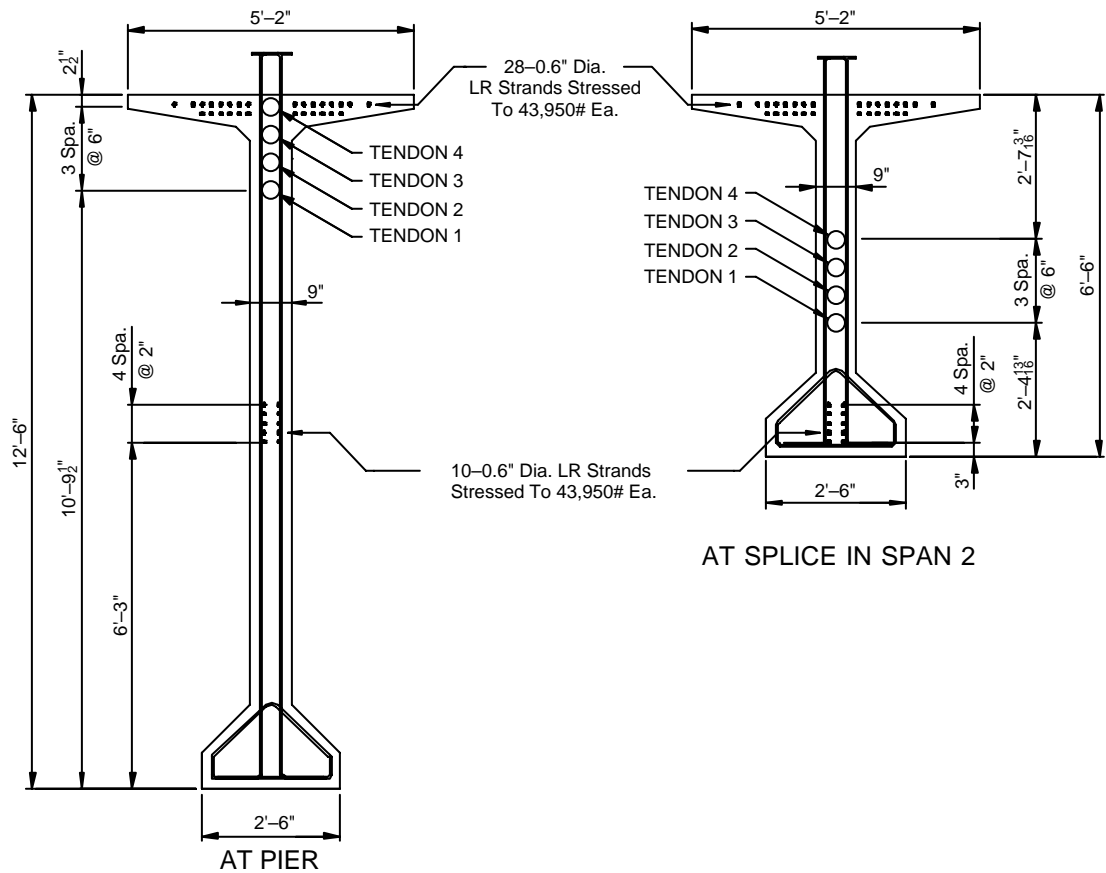
Figure A-5   Initial Design – Prestress Layout for Pier Girder Segments

D3-59

Continuous Three Span Girder   **Design Example 3**

AT SPLICE   MIDSPAN
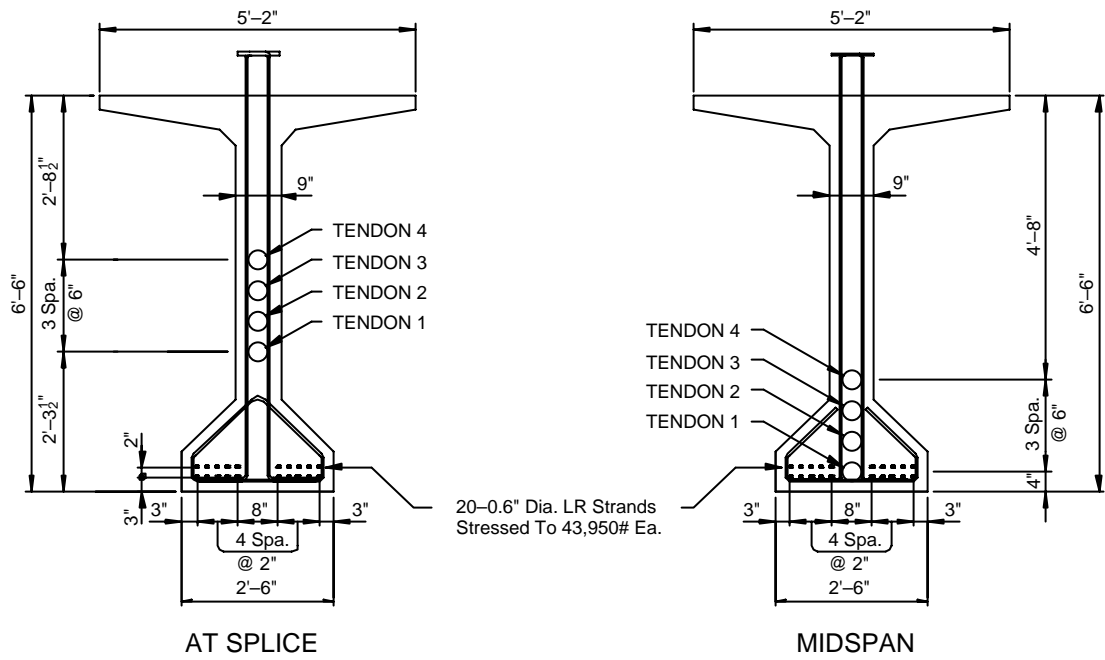
Figure A-6   Initial Design – Prestress Layout for Drop-in Girder Segments

APPENDIX E

Design Specifications – Proposed Revisions and Review

Table of Contents

APPENDIX E
DESIGN SPECIFICATIONS – PROPOSED REVISIONS AND REVIEW

A thorough review of the *AASHTO LRFD Bridge Design Specifications* was conducted as one of the initial activities of this study. This review identified issues that needed action to allow full use of the options available for extending the span ranges for precast prestressed concrete girders. Design examples were also prepared using the current version of the *LRFD Specifications*. Based on these activities, proposed revisions to the *LRFD Specifications* were developed to address spliced girder construction.

Two products have been prepared to address issues in the *LRFD Specifications* to improve them for use with extending span and spliced bridge girders. These products are contained in this appendix and are introduced briefly below.

In this appendix, the list of potential issues follows the proposed revisions because the proposed revisions are seen as the more significant and rapidly implementable contribution of the project to the *LRFD Specifications*. The list of potential issues, which was completed prior to the proposed revisions and served as the foundation for them, simply identifies issues for which changes could be developed and proposed.

Appendix E1 Proposed Revisions to the LRFD Specifications for Spliced Girder Construction

Both the review of the *Specifications* and the development of the design examples revealed a need for the reorganization of the provisions addressing spliced girder construction. Therefore, proposed revisions were developed.sss Most of the revisions address the reorganization of the existing content of the *Specifications*, although some new provisions have been added. A brief introduction and summary precedes the proposed revisions, which are presented in the two-column format of the *Specifications*.

Appendix E2 List of Potential Issues

A list of potential issues in the *LRFD Specifications* that may require revision was compiled during the early stages of the project following a thorough review of the *Specifications* and discussions with design engineers. The list contains items that are related to or impact methods for extending span

ranges of precast prestressed concrete girders or using spliced concrete girders. Therefore, the scope of this list is broader than the proposed revisions for spliced girder construction. A brief introduction that describes the format of the list and summarizes some of the major items found precedes the list of potential issues.

APPENDIX E1

PROPOSED REVISIONS TO THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

APPENDIX E1

PROPOSED REVISIONS TO THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

1 INTRODUCTION

A major goal for this research project was to develop proposed revisions to the *AASHTO LRFD Bridge Design Specifications* where needed to allow, facilitate or clarify the use of extended spans for precast prestressed concrete bridge girders. To fulfill this project goal, the *Specifications* were reviewed to identify provisions that need revision. The review revealed that the most significant issues were related to the refinement of the *Specifications* for the design of spliced girders. Major revisions do not appear necessary to implement strategies for extending span ranges other than the splicing of girders. Therefore, the proposed revisions to the *Specifications* address only issues related to spliced girder design.

The most important issue for clarifying the application of the Specifications to spliced girder design is the need to distinguish between spliced girder and segmental construction. Revision of the *Specifications* in this area is essential so that the *Specifications* may clearly state, and that designers may properly interpret, the requirements that apply to spliced girder construction. The most reasonable approach to making the distinction was to treat spliced girder bridges in the *Specifications* as conventional prestressed concrete girder bridges with additional design requirements for the splice locations where some provisions related to segmental construction are applicable. This distinction will prevent the unnecessary and inappropriate application of requirements for segmental construction, which appear to be intended for short-segment cast-in-place or match-cast box girder bridges, to spliced girder bridges. Revisions have been worded to preserve as much flexibility in design options as possible to allow cases in which the boundaries between spliced girder and segmental construction, and other concepts, may be blurred.

2 BRIEF OVERVIEW OF REVISIONS

Draft proposed revisions to the *Specifications* have been developed to implement clarification discussed above and to address other issues related to spliced girders. The major revisions include:

- Article 5.14.1.3 Spliced Precast Girders has been added to specifically address the detailing and design of spliced girder bridges. This new article contains provisions from Article 5.14.1.2.6 Transverse Construction Joints and some appropriate provisions related to spliced girder construction from Article 5.14.2 Segmental Construction. Additional information from other articles in the *Specifications* and other sources has also been added.

- Article 5.14.2.4.7 Precast Segmental Beam Bridges has been deleted, with its contents distributed to the new Article 5.14.1.3 as appropriate.

- A definition of "Segmental Construction" is proposed to replace the definition of "Segmental Components" currently appearing in Article 5.2. A definition of "Spliced Precast Girders" is also proposed to emphasize the distinction between the two types of construction.

- The provisions of Article 5.14.1.2.7, which address precast girders made continuous, have been moved to a new Article 5.14.1.4 Simple Span Precast Girders Made Continuous. The introduction to the article has been revised to more clearly reflect the intent of this type of construction, limiting it to simple span construction with closure joints at interior piers, and to coordinate its requirements with the new section on spliced precast girders.

- Article 5.14.1.2.8 Longitudinal Construction Joints has been moved to Article 5.14.4.3.3d which is a subarticle related to precast slab bridges.

- With the addition of Articles 5.14.1.3 and 5.14.1.4, current Article 5.14.1.3 Cast-in-Place Girders and Box and T-Beams must be renumbered as Article 5.14.1.5.

- Several other articles have also been revised to address additional issues and concerns. Some others have been included for comment or to note ongoing revision activity by AASHTO Committee T-10.

3   VERSION OF SPECIFICATIONS USED AS BASIS FOR REVISIONS

The proposed revisions are based on the *AASHTO LRFD Bridge Design Specifications, 2nd Edition*, with interims through 2003.

4 PRESENTATION OF REVISIONS

The proposed revisions are presented in a format as similar to the actual *Specifications* (including the commentary) as possible. Because of the extensive nature of the revisions, comments and justification for revisions are included in the text, as noted below.

To assist in the evaluation of the proposed *Specifications*, they are presented in two forms. In the first presentation, revisions and comments are indicated in the text using the following conventions:

- <u>Underlined</u> - changes to the *Specifications*.

- ~~Strike though~~ - deletions from the current *Specifications*.

- *Italics with shading* - comments and references to source articles in the *Specifications*

In the second presentation, the *Specifications* are shown in their final revised form. Most comments are also included. The same articles are included in both presentations of the proposed *Specifications*. Pages for the two sets of *Specifications* are numbered separately.

While most articles that are shown have been revised, a few articles are shown for information because they are referenced in other articles, or have recent revisions that have significance for spliced girders. In some cases, only a portion of an article is shown. Those portions not shown have not been revised. Article titles are shown for prior sections to clarify the position of each article shown.

Notes have been added in some cases to describe and give reasons for the changes. Wherever possible, wording is taken from existing articles to maintain consistency in the *Specifications*. In most cases, the authors have attempted to avoid proposing changes to existing text in the *Specifications* where it was not required for the purposes of this research project.

The proposed revisions are presented in the order in which they appear in the *Specifications*.

PROPOSED REVISIONS TO THE
AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

WITH REVISIONS MARKED

2.5.2.3 MAINTAINABILITY

Structural systems whose maintenance is expected to be difficult should be avoided. Where the climatic and/or traffic environment is such that ~~the~~ a bridge deck may need to be replaced before the required service life, ~~either~~ provisions shall be shown on the contract plans for the addition of a protective overlay (see Article 5.14.2.3.10e), the future replacement of the deck, or the provision of additional structural resistance ~~shall be provided~~.

Areas around bearing seats and under deck joints should be designed to facilitate jacking, cleaning, repair, and replacement of bearings and joints.

Jacking points shall be indicated on the plans, and the structure shall be designed for jacking forces specified in Article 3.4.3. Inaccessible cavities and corners should be avoided. Cavities that may invite human or animal inhabitants shall either be avoided or made secure.

THESE REVISIONS REFLECT THE POSITION THAT DECK DURABILITY CAN BE ADDRESSED USING AN OVERLAY RATHER THAN REQUIRING DECK REMOVAL. THIS POSITION HAS BEEN RECOGNIZED IN THE SPECIFICATIONS IN ARTICLE 5.14.2.3.10e (INCLUDED BELOW FOR INFORMATION), WHICH WAS ADDED IN THE MOST RECENT REVISIONS.

C2.5.2.3

Maintenance of traffic during replacement should be provided either by partial width staging of replacement or by the utilization of an adjacent parallel structure.

Measures for increasing the durability of concrete and wood decks include epoxy coating of reinforcing bars, post-tensioning ducts, and prestressing strand in the deck. Microsilica and/or calcium nitrite additives in the deck concrete, waterproofing membranes, and overlays may be used to protect black steel.

SECTION 5 – TABLE OF CONTENTS

5.2 DEFINITIONS

~~**Segmental Component** – A component made up of individual elements, either precast or cast-in-place, and posttensioned together to act as a monolithic unit under loads.~~

REPLACE CURRENT DEFINITION WITH:

Segmental Construction – The fabrication and erection of a structural element (superstructure and/or substructure) using individual elements, which may be either precast or cast-in-place. The completed structural element acts as a monolithic unit under some or all design loads. Posttensioning is typically used to connect the individual elements. For superstructures, the individual elements are typically short (with respect to the span length) box-shaped segments with monolithic flanges that comprise the full width of the structure. (See Article 5.14.2)

REASON: The terms "segmental construction" and "segmentally constructed" are used frequently in the Specifications (e.q., Articles 5.4.2.3.1, 5.5.4.2.2, 5.9.4.1.2, 5.9.4.2.2 and 5.9.5.4.1), while "segmental component" is not. Therefore, it is reasonable to provide a definition which addresses this utilization. The proposed definition is also necessary to clarify the typical type of superstructure for which these and other articles are intended to apply. A reference to the main article that addresses segmental construction is given to provide further assistance in the clarification of the definition (an article reference has not been provided for any other current definition).
Note: The American Segmental Bridge Institute (ASBI) has indicated that they do not have a definition for segmental construction that goes beyond what is currently stated in the LRFD Specifications. They consider spliced girders to be segmental bridges.

ADD DEFINITION:

Spliced Precast Girder – A type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using posttensioning, to form the completed girder. The bridge cross-section is typically a conventional structure in which multiple precast girders support a cast-in-place concrete composite deck. This type of construction is not considered to be segmental construction for the purposes of these Specifications. (See Article 5.14.1.3)

REASON: This term is defined here to further attempt to clarify the difference between typical segmental construction and spliced girder construction. A reference to the main article addressing spliced girder design is also given.

5.4 MATERIAL PROPERTIES

5.4.6 Ducts

5.4.6.2 SIZE OF DUCTS

The inside diameter of ducts shall be at least 0.25-IN larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct shall be at least 2.0 times the net area of the prestressing steel with one exception where tendons are to be placed by the pull-through method, the duct area shall be at least 2.5 times the net area of the prestressing steel.

The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct.

C5.4.6.2

The pull-through method of tendon placement is usually employed by contractors where tendons exceed 400 FT in length.

THIS ARTICLE IS CURRENTLY BEING CONSIDERED BY AASHTO T-10 FOR REVISION. WHILE PROVISIONS OF THIS ARTICLE MAY REDUCE THE EFFICIENCY OF SPLICED GIRDER DESIGNS, REVISION OF THESE PROVISIONS IS NOT PROPOSED IN THIS STUDY.

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.5 Loss of Prestress

5.9.5.2 INSTANTANEOUS LOSSES

5.9.5.2.2 Friction

5.9.5.2.2b Posttensioned Construction

THE INITIAL PORTION OF THIS ARTICLE IS NOT REPEATED HERE.

Values of K and μ should be based on experimental data for the material specified and shall be shown in the contract documents. In the absence of such data, a value within the ranges of K and μ as specified in Table 1 may be used.

THE REMAINDER OF THIS ARTICLE IS NOT REPEATED HERE, EXCEPT AS INCLUDED BELOW.

C5.9.5.2.2b

For slender members, the value of x may be taken as the projection of the tendon on the longitudinal axis of the member. A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes. See also Article C5.14.2.3.7. THIS REFERENCE WAS ADDED TO PROVIDE ADDITIONAL GUIDANCE FOR DESIGNERS.

THE REMAINDER OF THIS ARTICLE IS NOT REPEATED HERE, EXCEPT AS INCLUDED BELOW.

5.9.5.2.3 Elastic Shortening

ADD – NO TEXT CURRENTLY UNDER THIS HEADING:

The following equations apply for pretensioned or posttensioned members when the prestress is applied in a single stage. For components with combined pretensioning and posttensioning, and where posttensioning is applied in more than one stage, the effects of subsequent posttensioning on the elastic shortening of previously stressed prestressing tendons shall be considered.

C5.9.5.2.3

ADD COMMENTARY:

See Castrodale, et al. 200x for information on computing the effect of subsequent posttensioning on the elastic shortening of previously stressed prestressing tendons.

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT

LOSSES

5.9.5.4.3 Creep

The following equation applies for pretensioned or posttensioned members when the prestress is applied in a single stage. For components with combined pretensioning and posttensioning, and where posttensioning is applied in more than one stage, the effects of other prestressed tendons on the creep loss for each prestressing tendon shall be considered.

Prestress loss due to creep may be taken as:

C5.9.5.4.3

See Castrodale, et al. 200x for information on computing the interaction of creep effects for prestressing applied at different times.

The " to 0.0" in Equation 1 is needed because a negative value could result in some cases of partial prestressing, but Δf_{pCR} should not be taken as less than 0.0.

5.14 PROVISIONS FOR STRUCTURE TYPES

5.14.1 Beams and Girders

5.14.1.1 GENERAL

The provisions specified herein shall be applied to the design of cast-in-place and precast beams as well as girders with rectangular, I, T, bulb-T, and open- and closed-box sections.

Precast beams may resist transient loads with or without a superimposed deck. Where a structurally separate concrete deck is applied, it shall be made composite with the precast beams.

The flange width considered to be effective in flexure shall be that specified in Section 4.6.2. 6 or 5.7.3.4.

5.14.1.2 PRECAST BEAMS

5.14.1.2.2 Extreme Dimensions

The thickness of any part of precast concrete beams shall not be less than:

top flange:	2.0 IN
web, non posttensioned:	5.0 IN
web, posttensioned:	6.5 IN
bottom flange:	5.0 IN

The maximum dimensions and weight of precast

C5.14.1.1

These provisions supplement the appropriate provisions of other articles of these Specifications.

This article applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.14.2. There is a large variety of possible concrete superstructure systems, some of which may fall into either category. Precast deck bridges, which utilize girder sections with integral decks, are covered in Article 5.14.4.3.

Components that directly carry live loads, i.e., incorporated elements of the deck, should be designed for the applicable provisions of Section 9 and with particular reference to minimum dimension requirements and the way the components are to be joined to provide a continuous deck.

C5.14.1.2.2

The 2.0-IN minimum dimension relates to bulb-T and double-T types of girders on which cast-in-place decks are used. The 5.0-IN and 6.5-IN web thicknesses have been successfully used by contractors experienced in working to close tolerances. The 5.0-IN limit for bottom flange thickness normally relates to box-type sections.

For highway transportation, the permissible load size

members beaves beams manufactured at an offsite casting yard shall conform to local hauling restrictions. *MOVED WITHIN ARTICLE, BUT NOT REVISED*

~~Field splices may be used where precast members exceed transportable lengths. Such splices shall conform to the provisions specified in either Article 5.14.1.2.6 or Article 5.14.2.4.2.~~ *MOVED TO COMMENTARY WITH REVISIONS*

INSERT A NEW SECTION:

THE NEW SECTION IS PROVIDED TO EMPHASIZE THE SPECIAL CONSIDERATIONS FOR SPLICED PRECAST GIRDERS. FURTHERMORE, THE DESIRED NUMBER OF LEVELS OF SUBDIVISIONS UNDER THE SPLICED PRECAST GIRDERS HEADING CANNOT BE ACCOMMODATED USING CURRENT PRACTICE IN THE LRFD SPECIFICATIONS UNLESS THE MAIN HEADING IS PLACED AT THIS LEVEL.

5.14.1.3 SPLICED PRECAST GIRDERS

The provisions herein apply to precast girders fabricated in segments that are joined or spliced longitudinally to form the girders in the final structure. The cross-section for bridges utilizing segmented precast girders is typically comprised of several girders with a composite deck. The final structure may be a simple span or a continuous span unit.

The requirements specified herein shall supplement the requirements of other sections of these Specifications for other than segmentally constructed bridges. *[FROM 5.4.2.3.1 AND 5.14.2.1]* Therefore, spliced precast girder bridges shall not be considered as segmental construction for the purposes of design. For special design cases, additional provisions for segmental construction found in Article 5.14.2 and other articles in these Specifications may be used where appropriate.

THE DISTINCTION BETWEEN SPLICED PRECAST GIRDERS AND SEGMENTAL CONSTRUCTION IS PROVIDED TO CLARIFY THE APPLICATION OF ARTICLES SUCH AS 5.4.2.3.1, 5.5.4.2 AND 5.9.4.

and weight limits are constantly being revised. For large members beams, an investigation should be made prior to design to ensure transportability. *MOVED WITHIN ARTICLE, BUT NOT REVISED* Investigations may include driving the route or surveying route portions with known vertical or horizontal clearance problems. Contract documents should alert the contractor to weight and permitting complications as well the possibility of law enforcement escort requirements. *ADDED*

When the weight or dimensions of a precast member beam exceed local hauling restrictions, field splices may be used conforming to the requirements of Article 5.14.1.3 for spliced girders. *MOVED FROM BODY OF SPECIFICATIONS AND REVISED*

ADD A NEW SECTION:

C5.14.1.3

Bridges consisting of spliced precast girder segments have been constructed in a variety of locations for many different reasons. An extensive database of spliced girder bridge projects has been compiled by Castrodale, et al. (200x) *[THIS REPLACES CURRENT ART. C5.14.2.4.7a WHICH ADDRESSES POTENTIAL PROJECTS RATHER THAN EXISTING PROJECTS]*

Splicing of girder segments is generally performed in place, but may be performed prior to erection.

In previous editions of these Specifications, spliced precast girder bridges were considered as a special case of both conventional precast girders and segmental construction. However, it is more appropriate to classify this type of structure as a conventional bridge with additional requirements at the splice locations that are based on provisions developed for segmental construction.

Spliced precast girder bridges may be distinguished from what is referred to as "segmental construction" elsewhere in these Specifications by several features which typically include:

The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span. In some cases, the segment may be the full span length.

Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.

Wet-cast closure joints are usually used to join girder segments rather than match-cast joints.

The bridge cross-section is comprised of several

The method of construction assumed for the design shall be shown in the contract documents. All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents. *[FIRST SENTENCE FROM 5.14.2.1; SECOND SENTENCE IS REVISED]*

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor to the construction method or to the design shall comply with the requirements of Article 5.14.2.5. *[FROM 5.14.2.1]*

Stresses due to changes in the statical system, in particular, the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude. *[FROM 5.14.2.4.5]*

Spliced girder superstructures which satisfy all service limit state requirements of this article may be designed as fully continuous at all limit states for loads applied after the girder segments are joined. *[FROM 5.14.1.2.7c]*

Prestress losses in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.9.5. The effects of combined pretensioning and posttensioning and staged posttensioning shall be considered.

When required, the effects of creep and shrinkage in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.4.2.3.

Precast deck girder bridges, for which some or all of the deck is cast integrally with a girder, may be spliced. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of Article 5.14.4.3.

Spliced precast girders may be made continuous for some permanent loads using details for simple span precast girders made continuous. In such cases, design shall conform to the applicable requirements of Article

individual girders with a cast-in-place concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be divided into pieces that are integrally cast with each girder. A bridge of this type is completed by connecting the girders across the longitudinal joints.

Girder sections are used, such as bulb tee or open-topped trapezoidal boxes, rather than closed cell boxes with wide monolithic flanges.

Provisional ducts are required for segmental construction (Article 5.14.2.3.8a) to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

The method of construction and any required temporary support is of paramount importance in the design of spliced precast girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing. *[FROM C5.14.2.1]*

Deck girder bridges are often spliced because the significant weight of the cross-section, which is comprised of both a girder and deck, may exceed usual limits for handling and transportation.

5.14.1.4

~~5.14.1.2.6~~ 5.14.1.3.1 ~~Transverse Construction~~ Joints Between Segments

~~5.14.1.2.6a~~ 5.14.1.3.1a *General*

~~In-span construction joints~~ Joints between girder segments shall be either ~~match-cast or~~ cast-in-place closure joints or match-cast joints. ~~type. Joints at internal piers in continuous construction shall be of the closure type.~~ Match-cast joints shall satisfy the requirements of Article 5.14.2.4.2. ~~For prestressed beams, in-span construction joints shall be posttensioned.~~

THE REQUIREMENT THAT SPLICES MUST BE POSTTENSIONED HAS BEEN REMOVED BECAUSE CONVENTIONALLY REINFORCED SPLICES HAVE BEEN USED SUCCESSFULLY IN SEVERAL BRIDGES.

~~If the closure joint exceeds 6.0 IN, its compressive chord section shall be reinforced for confinement.~~ *[MOVED TO 5.14.1.3.1b]* The sequence of placing concrete for the closure joints and ~~the slab~~ deck shall be specified in the contract documents.

~~5.14.1.2.6b~~ 5.14.1.3.1b ~~Fully Effective Construction Details of Closure~~ *Joints*

Precast concrete ~~beam~~ girder segments, with or without a cast-in-place slab, may be made longitudinally continuous for both permanent and transient loads with combinations of posttensioning and/or reinforcement crossing the closure joints.

The width of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of posttensioning ducts. ~~, but it~~ The width of a closure joint shall not be less than 12.0 IN except for joints located within a diaphragm, for which the width shall not be less than 4.0 IN. ~~when the joint is located in the span and 4.0 IN at an internal pier.~~ *[ALSO FROM 5.14.2.4.7c WITH REVISIONS. MINIMUM JOINT WIDTHS ARE UNCHANGED]*

If the width of the closure joint exceeds 6.0 IN, its compressive chord section shall be reinforced for confinement. *[FROM 5.14.1.2.6a]*

If the joint is located in the span, its web reinforcement, A_s/s, shall be the larger of that in the adjacent girders.

~~In the case of multistage posttensioning, length of draped ducts for tendons to be tensioned before the compressive strength of the slab concrete attains f'_c shall not be located in the slab.~~ *[MOVED TO 5.14.1.3.3]*

~~C5.14.1.2.6a~~ C5.14.1.3.1a

This article codifies current best practice, which allows the Designer considerable latitude to formulate new structural systems. The great majority of in-span construction joints have been posttensioned. Conventionally reinforced joints have been used in a limited number of bridges.

Cast-in-place closure joints are typically used in spliced girder construction. Machined bulkheads have been used successfully to emulate match-cast joints for spliced girders. Prestress, dead load and creep effects may cause rotation of the faces of the match-cast joints prior to splicing. Procedures for splicing the girder segments that overcome this rotation to close the match-cast joint should be shown on the contract plans.

~~The bottom flange in the internal pier area acts nearly as a column, hence the requirement for confinement steel.~~ *[MOVED TO C5.14.1.3.1b]*

~~C5.14.1.2.6b~~ C5.14.1.3.1b

When diaphragms are provided at closure joint locations, designers should consider extending the closure joint at the exterior girder beyond the outside face of the girder. This detail creates a distinct and interesting visual element that obscures differences in concrete color between the closure and the segment. Such a detail may also smooth the appearance of geometric discontinuities between segments from vertical and horizontal curves. Extending the closure joint beyond the face of the exterior girder also provides improved development of diaphragm reinforcement for bridges subject to extreme events.

The intent of the joint width requirement is to allow proper compaction of concrete in the cast-in-place closure joint. In some cases, narrower joints have been used successfully. ~~At internal piers,~~ Consolidation of concrete in a closure joint is enhanced when the joint is contained within a diaphragm. ~~the diaphragms may incorporate the joint, thus facilitating compaction of concrete in a narrower space.~~ A wider closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

The bottom flange ~~in the internal pier area~~ near an interior support acts nearly as a column, hence the requirement for confinement steel. *[FROM C5.14.1.2.6a]*

~~This provision is to ensure that ducts as yet unsecured by concrete will not be used for active posttensioning.~~ *[MOVED TO C5.14.1.3.3]*

The face of the precast segments at closure joints shall be specified as either intentionally roughened to expose coarse aggregate, or having shear keys in accordance with Article 5.14.2.4.2. *[FROM 5.14.2.4.3 & 5.14.2.4.7c]*

~~5.14.2.4.7c~~ 5.14.1.3.1c ~~Fully Effective Construction Details of Match-Cast~~ *Joints*

~~Epoxied~~ Match-cast joints for ~~segmental beam~~ spliced precast girder bridges shall be ~~designed~~ detailed in accordance with Article 5.14.2.4.2.

5.14.1.3.1d Joint Design

Stress limits for temporary concrete stresses in joints before losses specified in Article 5.9.4.1 for "Segmentally Constructed Bridges" shall apply at each stage of prestressing (pretensioning or posttensioning). The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in Article 5.9.4.2 for "Segmentally Constructed Bridges" shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Resistance factors for joints specified in Article 5.5.4.2.2 for segmental construction shall apply.

The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations. *[FROM 5.14.2.4.7c]* ~~Where cast-in-place closure joints are contained within a diaphragm, the design at the closure joint may be based on the strength of the concrete in the precast elements.~~ *[FROM 5.14.1.2.7a]*

~~5.14.2.4.7b~~ 5.14.1.3.2 *Girder* Segment ~~Reinforcement Design~~

Stress limits for temporary concrete stresses in girder segments before losses specified in Article 5.9.4.1 for "Other Than Segmentally Constructed Bridges" shall apply at each stage of prestressing (pretensioning or posttensioning) with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in girder segments at the service limit state after losses specified in Article

AASHTO LRFD Bridge Construction Specifications requires vertical joints to be keyed. However, proper attention to roughened joint preparation is expected to ensure bond between the segments, providing better shear strength than shear keys. *[FROM C5.14.2.4.3]*

~~C5.14.2.4.7c~~ C5.14.1.3.1c

~~Cast-in-place joints eliminate the need for match-casting of segments.~~ One or more large shear keys may be used with spliced girders rather than the multiple small amplitude shear keys indicated in Article 5.14.2.4.2. The shear key proportions specified in Article 5.14.2.4.2 should be used.

C5.14.1.3.2

Segments of ~~segmental beam~~ spliced precast girders ~~bridges~~ shall preferably be pretensioned for dead load and all applicable construction loadings to satisfy temporary stress limits ~~the tensile stress~~ in the concrete. *MOVED FROM 5.14.2.4.7b*

Temporary construction loads must be considered where these loads may contribute to critical stresses in girder segments at an intermediate stage of construction, such as when the deck slab is placed when only a portion of the total prestress has been applied. Temporary construction loads are specified in the AASHTO *Guide*

5.9.4.2 for "Other Than Segmentally Constructed Bridges" shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'$_c$ in the stress limits.

Where girder segments are precast without prestressed reinforcement are used, the provisions of Article 5.7.3.4 shall apply until posttensioning is applied.

Where variable depth girder segments are used, the effect of inclined compression on the variable depth members can generally be neglected when calculating shear resistance. shall be considered.

The potential for buckling of tall thin web sections shall be considered.

The designer shall give consideration to requirements for bracing of the girder segments once they have been erected on the substructure. Any requirements for temporary or permanent bracing during subsequent stages of construction, along with the contractor's responsibilities for designing and placing them, shall be specified in the contract documents.

MOVED FROM 5.14.2.4.7d

5.14.2.4.7d 5.14.1.3.3 Posttensioning

Posttensioning may be applied either before and/or after placement of deck concrete. Part of the posttensioning may be applied to provide beam girder continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement.

All posttensioning tendons shall be fully grouted after stressing.

Prior to grouting of posttensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers. The use of couplers shall conform to Article 5.10.3.5, with special attention given to the effect of voids for the couplers on the section properties at the time of posttensioning.

Posttensioning shall be shown on the contract documents according to the requirements of Article 5.14.2.3.9.

Where tendons terminate at the top of the beam section a girder segment, the contract documents shall require that the duct openings be protected during construction to prevent debris accumulation and that. Ddrains shall be provided at tendon low points.

Design Specifications for Bridge Temporary Works.

Because gravity loads induce compression in the bottom flange of girders at support locations, the vertical force component from inclined flexural stresses in a haunched girder segment generally acts to reduce the applied shear. Its effect can be accounted for in the same manner as the vertical component of the longitudinal prestressing force, V$_p$. However, it is usual to neglect the reduction of the vertical shear force from this effect is usually neglected.

ADD:

C5.14.1.3.3

Where some or all posttensioning is applied after the deck concrete is placed, fewer posttensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the posttensioning is applied before the deck concrete is placed, a greater number of posttensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary. See Castrodale, et al. 200x.

See Article 5.10.3.5.

Where tendons terminate at the top of the girder, blockouts and pourbacks in the deck slab are required for access to the tendons and anchorages. While this arrangement has been used, it is preferable to anchor all tendons at the ends of girders. Minimizing or eliminating deck slab blockouts by placing anchorages at ends of girders reduces the potential for water seepage and corrosion at the posttensioning tendon anchors.

In the case of multistage posttensioning, ~~length of draped ducts for tendons to be tensioned before the compressive strength of the~~ slab concrete is placed and attains the minimum specified compressive strength f'_{ci} shall not be located in the slab. *[FROM 5.14.1.2.6b]*

Where some or all posttensioning tendons are stressed after the deck concrete is placed, provisions shall be shown on the contract plans satisfying the provisions of Article 2.5.2.3 on maintainability of the deck.

See Article 5.14.2.3.10e.

PROMOTE THIS ARTICLE HEADING AND RENUMBER SUBARTICLES

THIS ARTICLE IS PROMOTED BECAUSE OF ITS SIMILAR STATURE TO SPLICED GIRDERS.

THIS ARTICLE HAS BEEN REVISED TO LIMIT ITS APPLICATION TO FULL-SPAN PRECAST GIRDERS MADE CONTINUOUS AT PIER LOCATIONS. THE NEW ARTICLE "SPLICED PRECAST GIRDERS" WILL ADDRESS SIMILAR CONNECTIONS AT LOCATIONS OTHER THAN AT PIERS. IT REFERS TO THE PROVISIONS OF THIS ARTICLE WHERE SPLICED GIRDER BRIDGES USE CLOSURE JOINTS AT PIERS.

~~5.14.1.2.7 Bridges Composed of Simple Span Precast Girders Made Continuous~~
SIMPLE SPAN PRECAST GIRDERS MADE CONTINUOUS

~~5.14.1.2.7a~~ 5.14.1.4.1 *General*

Bridges consisting of simple span precast concrete girders and cast-in-place concrete slabs may be made continuous for transient and/or permanent loads by using a cast-in-place closure ~~placement~~ joint at piers ~~. with tensile reinforcement located in the slab or by using closure pours at other locations. At interior piers where the~~

Where the ~~diaphragms contain~~ cast-in-place closure ~~placement~~ joint is contained within a diaphragms ~~contain~~, the design at the closure joint may be based on the strength of the concrete in the precast elements.

THE REMAINDER OF FORMER ARTICLE 5.14.1.2.7 IS NOT REPEATED HERE. CURRENT PROVISIONS, WITH PROPOSED REVISIONS AND ADDITIONS BEING DEVELOPED BY NCHRP PROJECT 12-53, WOULD BE RETAINED IN THIS ARTICLE AFTER RENUMBERING.

MOVE THIS ARTICLE IN ITS ENTIRETY TO REPLACE ARTICLE 5.14.4.3.3D IN ITS ENTIRETY.

MOVE THIS ARTICLE IN ITS ENTIRETY TO ARTICLE C5.14.4.3.3D

THE REASON FOR THIS CHANGE IS THAT ALL OF THE REQUIREMENTS OF ARTICLE 5.14.1.2.8 APPLY ONLY TO PRECAST DECK BRIDGES, AS IS CLEARLY STATED IN THE COMMENTARY TO THIS ARTICLE. THEREFORE, IT APPEARS APPROPRIATE TO CONSOLIDATE ALL REQUIREMENTS FOR THAT TYPE OF STRUCTURE IN ONE LOCATION IN THE SPECIFICATIONS.

~~5.14.1.2.8 Longitudinal Construction Joints~~

~~Longitudinal construction joints between precast concrete flexural components shall consist of a key filled~~

~~C5.14.1.2.8~~

~~This article relates to deck systems composed entirely of precast beams of box, T- and double-T sections, laid~~

with a nonshrinkage mortar attaining a compressive strength of 5.0 KSI within 24 hours. The depth of the key should not be less than 6.5 IN.

If the components are posttensioned together transversely, the top flanges may be assumed to act as a monolithic slab. However, the empirical slab design specified in Article 9.7.2 is not applicable.

The amount of transverse prestress may be determined by either the strip method or two-dimensional analysis. The transverse prestress, after all losses, shall not be less than 0.25 KSI through the key. In the last 3.0 FT at a free end, the required transverse prestress shall be doubled. The transverse posttensioning tendons should be located at the centerline of the key.

side-by-side and, preferably, joined together by transverse posttensioning.

The preferred joint is a simple v-joint.

Grinding of grout and concrete in the vicinity of the joint may be expected and specified for construction.

RENUMBER SECTION 5.14.1.3 AND ITS SUBSECTIONS BECAUSE OF THE CREATION OF SECTIONS 5.14.1.3 AND 5.14.1.4.

THESE NEW SECTIONS ARE RELATED TO PRECAST BEAMS (ARTICLE 5.14.1.2), SO IT APPEARS REASONABLE TO KEEP THESE THREE ARTICLES TOGETHER AND RENUMBER SECTION 5.14.1.3.

THE SPECIFICATIONS SHOULD BE SEARCHED FOR ANY REFERENCES TO THIS ARTICLE WHICH SHOULD BE CHANGED.

5.14.1.3 5.14.1.5 CAST-IN-PLACE GIRDERS AND BOX AND T-BEAMS

5.14.2 Segmental Construction

5.14.2.1 GENERAL

The requirements specified herein shall supplement the requirements of other sections of these Specifications for concrete structures designed to be constructed by the segmental method.

C5.14.2.1

For segmental construction, superstructures of single or multiple box sections are generally used, but beam-type sections may also be utilized. Segmental construction includes construction by free cantilever, span-by-span, or incremental launching methods using either precast or cast-in-place concrete segments which are joined to produce either continuous or simple spans.

Bridges utilizing beam-type sections may also be constructed using segmental construction techniques. Such bridges, which are referred to as spliced precast girder bridges in these Specifications, are considered to be a special case of conventional concrete bridges. The design of such bridges is covered in Article 5.14.1.3.

The span length of bridges considered by these specifications ranges to 800 FT. Bridges supported by stay cables are not specifically covered in this article, although

The provisions herein shall apply only to segmental construction using normal weight concrete.

The method of construction assumed for the design shall be shown in the contract documents. Temporary supports required prior to the time the structure, or component thereof, is capable of supporting itself and subsequently applied loads, shall also be shown in the contract documents.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor in the construction method or in the design shall comply with the requirements of Article 5.14.2.5.

5.14.2.3 DESIGN

5.14.2.3.10 Box Girder Cross-Section Dimensions and Details

5.14.2.3.10e Overlays

Overlays shall be considered for all bridge decks exposed to freeze thaw cycles and application of deicing chemicals. The governing authority should consider providing additional protection against penetration of chlorides. For all types of segmental bridges (precast and cast-in-place), it is recommended that this additional protection be provided by the addition of a minimum of 1.5 IN of concrete cover, added as an overlay or alternatively a waterproof membrane with bituminous overlay. The governing authority may require specific materials and placement techniques stipulated by local practices.

many of the specification provisions are also applicable to them.

Lightweight concrete has been infrequently used for segmental bridge construction. Provision for the use of lightweight aggregates represents a significant complication of both design and construction specifications. Given this complication and questions concerning economic benefit, use of lightweight aggregates for segmental bridges is not explicitly covered.

The method of construction and any required temporary support is of paramount importance in the design of segmental concrete bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

For segmentally constructed bridges, designs should and generally do allow the Contractor some latitude in choice of construction methods. To ensure that the design features and details to be used are compatible with the proposed construction method, it is essential that the Contractor be required to prepare working drawings and calculations based on his choice of methods for review and approval by the Engineer before work begins.

C5.14.2.3.10e

Overlays are encouraged instead of the inclusion of monolithic concrete additional because an the overlay will add protection at the critical segment joint. Delamination of overlays is generally due to poor installation practices or material selection and can be resolved. It is not recommended that the additional cover be obtained by merely increasing concrete covers. The added cover will not add protection across the segment joint which is the area of most concern due to the ability of the water to migrate to the tendon and reinforcement.

Careful attention to detail is required when using overlays to assure the proper railing heights are obtained. All railings next to deck areas to be overlayed should be detailed from the top of the overlay.

The need to remove and replace the overlay can be based on measurement of chloride penetration into the overlay. Use of high performance concrete is an effective means of minimizing chloride penetration into concrete.

Bridges located in other corrosive environments, such as coastal bridges over salt water, should be evaluated for the need for additional protection.

5.14.2.4 TYPES OF SEGMENTAL BRIDGES

5.14.2.4.1 General

Bridges designed for segmentally placed superstructures shall conform to the requirements specified herein, based on the concrete placement method and the erection methods to be used.

C5.14.2.4.1

Precast segmental bridges are normally erected by balanced cantilever, use of erection trusses, or progressive placement.

Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons. Bridges built with erection trusses may utilize internal tendons, external tendons, or combinations thereof. Due to considerations of segment weight, span lengths for precast segmental box girder bridges, except for cable-stayed bridges, rarely exceed 400 FT.

5.14.2.4.2 Details for Precast Construction

The compressive strength of precast concrete segments shall not be less than 2.5 KSI prior to removal from the forms and shall have a maturity equivalent to 14 days at 70°F prior to assembly into the structure.

Multiple small-amplitude shear keys at match-cast joints in webs of precast segmental bridges shall extend over as much of the web as is compatible with other details. Details of shear keys in webs should be similar to those shown in Figure 1. Shear keys shall also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single-element keys.

Figure 5.14.2.4.2-1 NOT SHOWN

Joints in precast segmental bridges shall be either cast-in-place closures or match cast epoxied joints.

Precast segmental bridges using internal posttensioning tendons and bridges located in areas subject to freezing temperatures or deicing chemicals shall employ bonded joints.

C5.14.2.4.2

This provision intends to limit the magnitude of construction deflections and to prevent erratic construction deflections and creep.

Small-amplitude shear keys in the webs are less susceptible to construction damage, which will result in loss of geometry control, than larger single-element keys. Shear keys in the top and bottom flanges are less susceptible to such damage

Match casting is necessary to ensure control of the geometry upon reassembly of the segments.

Epoxy on both faces serves as a lubricant during placement of the segments, prevents water intrusion, provides a seal to prevent cross-over during grouting, and provides some tensile strength across the joint.

The use of dry joints (identified as Type B in past versions of these specifications) was eliminated with the adoption of the 2003 revision due to the critical nature of posttensioning reinforcing and the need for a multiple layer protection system. Failures of some posttensioning reinforcing in Florida and Europe due to corrosion have resulted in a review of the effectiveness of previous multiple layer protection systems. The most rigorous review was performed by the British Concrete Society and the recommendations are contained in the report titled "Durable Post-tensioned Concrete Bridges." This European report codifies the need for a three-level protection system and suggested details to achieve the required results. Improved grout and duct materials and methods are also discussed. As a result of this European Report and studies by Dr. John Breen of the University of Texas, Austin, the multiple level protection system for posttensioning has been universally accepted.

A temporary prestressing system shall provide a minimum compressive stress of 0.030 KSI and an average stress of 0.040 KSI across the joint until the epoxy has cured.

AASHTO LRFD Bridge Construction Specifications requires this temporary stress to ensure full bond and to prevent uneven epoxy thickness. Such variations could lead to a systematic accumulation of geometric error. Large stress changes on epoxy joints should be avoided during the initial curing period.

DELETE THIS ARTICLE, MOVING ITS CONTENTS INTO VARIOUS SECTIONS OF ARTICLE 5.14.1.3. NO FURTHER RENUMBERING IS REQUIRED.

5.14.2.4.7 Precast Segmental Beam Bridges

5.14.2.4.7a General

Precast segmental beam bridges shall be designed in accordance with the provisions of these specifications and in accordance with the additional provisions of this section.

MOVED TO 5.14.1.3.2

5.14.2.4.7b Segment Reinforcement

Segments of segmental beam bridges shall preferably be pretensioned for dead load and all construction loadings to limit the tensile stress in the concrete to 0.0948 √f′c.

Where segments without prestressed reinforcement are used, the provisions of Article 5.7.3.4 shall apply.

PORTIONS MOVED TO 5.14.1.3 AS INDICATED

C5.14.2.4.7a

AASHTO Type III and Type IV segmental beam designs have been proposed for spans up to 180 FT (Anderson 1973). Longer segmental beam bridge spans are feasible with deeper beam sections. *[MOVED TO C5.14.1.3]*

5.14.2.4.7c Joints

The width of cast-in-place joints shall permit access for coupling of conduits and thorough vibration of the concrete. *[MOVED TO 5.14.1.3.1b]* The compressive strength of the joint concrete at a specified age shall be compatible with design stress limitations. *[MOVED TO 5.14.1.3.1d]* The face of the precast segments shall be intentionally roughened to expose coarse aggregate, or shear keys shall be utilized in accordance with Article 5.14.2.4.2. *[MOVED TO 5.14.1.3.1b]*

Epoxied match-cast joints for segmental beam bridges shall be designed in accordance with Article 5.14.2.4.2. *[MOVED TO 5.14.1.3.1c]* A minimum of 0.040 KSI compression shall be applied to the joint during the "open time" of the epoxy. *[THIS SENTENCE NOT MOVED TO 5.14.1.3.1c, BECAUSE IT DUPLICATES REQUIREMENTS IN REFERENCED ARTICLE 5.14.2.4.2]*

MOVED TO 5.14.1.3.3

5.14.2.4.7d Posttensioning

C5.14.2.4.7c

Cast-in-place joints eliminate the need for match-casting of segments. *[MOVED TO 5.14.1.3.1c]*

Posttensioning may be applied either before and/or after placement of deck concrete. Part of the posttensioning may be applied to provide beam continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement. The use of couplers shall conform to Article 5.10.3.1, with special attention given to the effect of voids for the couplers on the section properties at the time of posttensioning.

Where tendons terminate at the top of the beam section, the contract document shall require that the duct opening be protected during construction to prevent debris accumulation. Drains shall be provided at tendon low points.

5.14.4 Slab Superstructures

5.14.4.3 PRECAST DECK BRIDGES

5.14.4.3.3 Shear-Flexure Transfer Joints

REPLACE THE CONTENTS OF ARTICLE 5.14.4.3.3d WITH THE CONTENTS OF ARTICLE 5.14.1.2.8 AS SHOWN.

MOVE ARTICLE C5.14.1.2.8 TO THIS ARTICLE.

C5.14.4.3.3d

5.14.4.3.3d Longitudinal Construction Joints

The provisions of Article 5.14.1.2.8 shall apply.

Longitudinal construction joints between precast concrete flexural components shall consist of a key filled with a nonshrinkage mortar attaining a compressive strength of 5.0 KSI within 24 hours. The depth of the key should not be less than 6.5 IN.

If the components are posttensioned together transversely, the top flanges may be assumed to act as a monolithic slab. However, the empirical slab design specified in Article 9.7.2 is not applicable.

The amount of transverse prestress may be determined by either the strip method or two-dimensional analysis. The transverse prestress, after all losses, shall not be less than 0.25 KSI through the key. In the last 3.0 FT at a free end, the required transverse prestress shall be doubled. The transverse posttensioning tendons should be located at the centerline of the key.

This article relates to deck systems composed entirely of precast beams of box, T- and double-T sections, laid side-by-side and, preferably, joined together by transverse posttensioning.

The preferred joint is a simple v-joint.

Grinding of grout and concrete in the vicinity of the joint may be expected and specified for construction.

THE SENTENCE REGARDING JOINT TYPE IS DELETED BECAUSE IT IS NOT GENERAL, BUT APPLIES TO A SPECIFIC SECTION TYPE.

REVISIONS TO THIS ARTICLE (FORMERLY 5.14.1.2.8), WHILE DESIRABLE, HAVE NOT BEEN PROPOSED AS PART OF THIS STUDY BECAUSE THEY ARE NOT RELATED TO THIS STUDY.

REFERENCES

Add the following to the list of references currently appearing at the end of Section 5:

Castrodale, R.W., White, C.D., "Extending Span Ranges of Precast, Prestressed Concrete Girders," Project No. 12-57, National Cooperative Highway Research Program, NCHRP, National Research Council, Washington DC, submitted September 2003.

PROPOSED REVISIONS TO THE
AASHTO LRFD BRIDGE DESIGN
SPECIFICATIONS

WITH REVISIONS INCORPORATED

2.5.2.3 MAINTAINABILITY

Structural systems whose maintenance is expected to be difficult should be avoided. Where the climatic and/or traffic environment is such that a bridge deck may need to be replaced before the required service life, provisions shall be shown on the contract plans for the addition of a protective overlay (see Article 5.14.2.3.10e), the future replacement of the deck, or the provision of additional structural resistance.

Areas around bearing seats and under deck joints should be designed to facilitate jacking, cleaning, repair, and replacement of bearings and joints.

Jacking points shall be indicated on the plans, and the structure shall be designed for jacking forces specified in Article 3.4.3. Inaccessible cavities and corners should be avoided. Cavities that may invite human or animal inhabitants shall either be avoided or made secure.

THESE REVISIONS REFLECT THE POSITION THAT DECK DURABILITY CAN BE ADDRESSED USING AN OVERLAY RATHER THAN REQUIRING DECK REMOVAL. THIS POSITION HAS BEEN RECOGNIZED IN THE SPECIFICATIONS IN ARTICLE 5.14.2.3.10e (INCLUDED BELOW FOR INFORMATION), WHICH WAS ADDED IN THE MOST RECENT REVISIONS.

C2.5.2.3

Maintenance of traffic during replacement should be provided either by partial width staging of replacement or by the utilization of an adjacent parallel structure.

Measures for increasing the durability of concrete and wood decks include epoxy coating of reinforcing bars, post-tensioning ducts, and prestressing strand in the deck. Microsilica and/or calcium nitrite additives in the deck concrete, waterproofing membranes, and overlays may be used to protect black steel.

SECTION 5 – TABLE OF CONTENTS

5.2 DEFINITIONS

REPLACE CURRENT DEFINITION WITH:

Segmental Construction – The fabrication and erection of a structural element (superstructure and/or substructure) using individual elements, which may be either precast or cast-in-place. The completed structural element acts as a monolithic unit under some or all design loads. Posttensioning is typically used to connect the individual elements. For superstructures, the individual elements are typically short (with respect to the span length) box-shaped segments with monolithic flanges that comprise the full width of the structure. (See Article 5.14.2)

REASON: The terms "segmental construction" and "segmentally constructed" are used frequently in the Specifications (e.g., Articles 5.4.2.3.1, 5.5.4.2.2, 5.9.4.1.2, 5.9.4.2.2 and 5.9.5.4.1), while "segmental component" is not. Therefore, it is reasonable to provide a definition which addresses this utilization. The proposed definition is also necessary to clarify the typical type of superstructure for which these and other articles are intended to apply. A reference to the main article that addresses segmental construction is given to provide further assistance in the clarification of the definition (an article reference has not been provided for any other current definition).
Note: The American Segmental Bridge Institute (ASBI) has indicated that they do not have a definition for segmental construction that goes beyond what is currently stated in the LRFD Specifications. They consider spliced girders to be segmental bridges.

ADD DEFINITION:

Spliced Precast Girder – A type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using posttensioning, to form the completed girder. The bridge cross-section is typically a conventional structure in which multiple precast girders support a cast-in-place concrete composite deck. This type of construction is not considered to be segmental construction for the purposes of these Specifications. (See Article 5.14.1.3)

REASON: This term is defined here to further attempt to clarify the difference between typical segmental construction and spliced girder construction. A reference to the main article addressing spliced girder design is also given.

5.4 MATERIAL PROPERTIES

5.4.6 Ducts

5.4.6.2 SIZE OF DUCTS

The inside diameter of ducts shall be at least 0.25-IN larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct shall be at least 2.0 times the net area of the prestressing steel with one exception where tendons are to be placed by the pull-through method, the duct area shall be at least 2.5 times the net area of the prestressing steel.

The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct.

C5.4.6.2

The pull-through method of tendon placement is usually employed by contractors where tendons exceed 400 FT in length.

THIS ARTICLE IS CURRENTLY BEING CONSIDERED BY AASHTO T-10 FOR REVISION. WHILE PROVISIONS OF THIS ARTICLE MAY REDUCE THE EFFICIENCY OF SPLICED GIRDER DESIGNS, REVISION OF THESE PROVISIONS IS NOT PROPOSED IN THIS STUDY.

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.5 Loss of Prestress

5.9.5.2 INSTANTANEOUS LOSSES

5.9.5.2.2 Friction

5.9.5.2.2b Posttensioned Construction

THE INITIAL PORTION OF THIS ARTICLE IS NOT REPEATED HERE.

Values of K and μ should be based on experimental data for the material specified and shall be shown in the contract documents. In the absence of such data, a value within the ranges of K and μ as specified in Table 1 may be used.

THE REMAINDER OF THIS ARTICLE IS NOT REPEATED HERE, EXCEPT AS INCLUDED BELOW.

C5.9.5.2.2b

For slender members, the value of x may be taken as the projection of the tendon on the longitudinal axis of the member. A friction coefficient of 0.25 is appropriate for 12 strand tendons. A lower coefficient may be used for larger tendon and duct sizes. See also Article C5.14.2.3.7. *THIS REFERENCE WAS ADDED TO PROVIDE ADDITIONAL GUIDANCE FOR DESIGNERS.*

THE REMAINDER OF THIS ARTICLE IS NOT REPEATED HERE, EXCEPT AS INCLUDED BELOW.

5.9.5.2.3 Elastic Shortening

ADD – NO TEXT CURRENTLY UNDER THIS HEADING:
The following equations apply for pretensioned or posttensioned members when the prestress is applied in a single stage. For components with combined pretensioning and posttensioning, and where posttensioning is applied in more than one stage, the effects of subsequent posttensioning on the elastic shortening of previously stressed prestressing tendons shall be considered.

C5.9.5.2.3

ADD COMMENTARY:
See Castrodale, et al. 200x for information on computing the effect of subsequent posttensioning on the elastic shortening of previously stressed prestressing tendons.

5.9.5.4 REFINED ESTIMATES OF TIME-DEPENDENT LOSSES

5.9.5.4.3 Creep

The following equation applies for pretensioned or posttensioned members when the prestress is applied in a single stage. For components with combined pretensioning and posttensioning, and where posttensioning is applied in more than one stage, the effects of other prestressed tendons on the creep loss for each prestressing tendon shall be considered.

Prestress loss due to creep may be taken as:

C5.9.5.4.3

See Castrodale, et al. 200x for information on computing the interaction of creep effects for prestressing applied at different times.

The " to 0.0" in Equation 1 is needed because a negative value could result in some cases of partial prestressing, but Δf_{pCR} should not be taken as less than 0.0.

5.14 PROVISIONS FOR STRUCTURE TYPES

5.14.1 Beams and Girders

5.14.1.1 GENERAL

The provisions specified herein shall be applied to the design of cast-in-place and precast beams as well as girders with rectangular, I, T, bulb-T, and open- and closed-box sections.

Precast beams may resist transient loads with or without a superimposed deck. Where a structurally separate concrete deck is applied, it shall be made composite with the precast beams.

The flange width considered to be effective in flexure shall be that specified in Section 4.6.2. 6 or 5.7.3.4.

C5.14.1.1

These provisions supplement the appropriate provisions of other articles of these Specifications.

This article applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.14.2. There is a large variety of possible concrete superstructure systems, some of which may fall into either category. Precast deck bridges, which utilize girder sections with integral decks, are covered in Article 5.14.4.3.

Components that directly carry live loads, i.e., incorporated elements of the deck, should be designed for the applicable provisions of Section 9 and with particular reference to minimum dimension requirements and the way the components are to be joined to provide a continuous deck.

5.14.1.2 PRECAST BEAMS

5.14.1.2.2 Extreme Dimensions

The thickness of any part of precast concrete beams shall not be less than:

top flange:	2.0 IN
web, non posttensioned:	5.0 IN
web, posttensioned:	6.5 IN
bottom flange:	5.0 IN

The maximum dimensions and weight of precast

C5.14.1.2.2

The 2.0-IN minimum dimension relates to bulb-T and double-T types of girders on which cast-in-place decks are used. The 5.0-IN and 6.5-IN web thicknesses have been successfully used by contractors experienced in working to close tolerances. The 5.0-IN limit for bottom flange thickness normally relates to box-type sections.

For highway transportation, the permissible load size

beams manufactured at an offsite casting yard shall conform to local hauling restrictions. *MOVED WITHIN ARTICLE, BUT NOT REVISED*

THE NEW SECTION IS PROVIDED TO EMPHASIZE THE SPECIAL CONSIDERATIONS FOR SPLICED PRECAST GIRDERS. FURTHERMORE, THE DESIRED NUMBER OF LEVELS OF SUBDIVISIONS UNDER THE SPLICED PRECAST GIRDERS HEADING CANNOT BE ACCOMMODATED USING CURRENT PRACTICE IN THE LRFD SPECIFICATIONS UNLESS THE MAIN HEADING IS PLACED AT THIS LEVEL.

5.14.1.3 SPLICED PRECAST GIRDERS

The provisions herein apply to precast girders fabricated in segments that are joined or spliced longitudinally to form the girders in the final structure. The cross-section for bridges utilizing segmented precast girders is typically comprised of several girders with a composite deck. The final structure may be a simple span or a continuous span unit.

The requirements specified herein shall supplement the requirements of other sections of these Specifications for other than segmentally constructed bridges. *[FROM 5.4.2.3.1 AND 5.14.2.1]* Therefore, spliced precast girder bridges shall not be considered as segmental construction for the purposes of design. For special design cases, additional provisions for segmental construction found in Article 5.14.2 and other articles in these Specifications may be used where appropriate.

THE DISTINCTION BETWEEN SPLICED PRECAST GIRDERS AND SEGMENTAL CONSTRUCTION IS PROVIDED TO CLARIFY THE APPLICATION OF ARTICLES SUCH AS 5.4.2.3.1, 5.5.4.2 AND 5.9.4.

and weight limits are constantly being revised. For large beams, an investigation should be made prior to design to ensure transportability. *MOVED WITHIN ARTICLE, BUT NOT REVISED* Investigations may include driving the route or surveying route portions with known vertical or horizontal clearance problems. Contract documents should alert the contractor to weight and permitting complications as well the possibility of law enforcement escort requirements. *ADDED*

When the weight or dimensions of a precast member beam exceed local hauling restrictions, field splices may be used conforming to the requirements of Article 5.14.1.3 for spliced girders. *MOVED FROM BODY OF SPECIFICATIONS AND REVISED*

C5.14.1.3

Bridges consisting of spliced precast girder segments have been constructed in a variety of locations for many different reasons. An extensive database of spliced girder bridge projects has been compiled by Castrodale, et al. (200x) *[THIS REPLACES CURRENT ART. C5.14.2.4.7a WHICH ADDRESSES POTENTIAL PROJECTS RATHER THAN EXISTING PROJECTS]*

Splicing of girder segments is generally performed in place, but may be performed prior to erection.

In previous editions of these Specifications, spliced precast girder bridges were considered as a special case of both conventional precast girders and segmental construction. However, it is more appropriate to classify this type of structure as a conventional bridge with additional requirements at the splice locations that are based on provisions developed for segmental construction.

Spliced precast girder bridges may be distinguished from what is referred to as "segmental construction" elsewhere in these Specifications by several features which typically include:

The lengths of some or all segments in a bridge are a significant fraction of the span length rather than having a number of segments in each span. In some cases, the segment may be the full span length.

Design of joints between girder segments at the service limit state does not typically govern the design for the entire length of the bridge for either construction or for the completed structure.

Wet-cast closure joints are usually used to join girder segments rather than match-cast joints.

The bridge cross-section is comprised of several

The method of construction assumed for the design shall be shown in the contract documents. All supports required prior to the splicing of the girder shall be shown on the contract documents, including elevations and reactions. The stage of construction during which the temporary supports are removed shall also be shown on the contract documents. *[FIRST SENTENCE FROM 5.14.2.1; SECOND SENTENCE IS REVISED]*

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor to the construction method or to the design shall comply with the requirements of Article 5.14.2.5. *[FROM 5.14.2.1]*

Stresses due to changes in the statical system, in particular, the effects of the application of load to one structural system and its removal from a different structural system, shall be accounted for. Redistribution of such stresses by creep shall be taken into account and allowance shall be made for possible variations in the creep rate and magnitude. *[FROM 5.14.2.4.5]*

Spliced girder superstructures which satisfy all service limit state requirements of this article may be designed as fully continuous at all limit states for loads applied after the girder segments are joined. *[FROM 5.14.1.27c]*

Prestress losses in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.9.5. The effects of combined pretensioning and posttensioning and staged posttensioning shall be considered.

When required, the effects of creep and shrinkage in spliced precast girder bridges may be estimated using the provisions for other than segmentally constructed bridges in Article 5.4.2.3.

Precast deck girder bridges, for which some or all of the deck is cast integrally with a girder, may be spliced. Spliced structures of this type, which have longitudinal joints in the deck between each deck girder, shall comply with the additional requirements of Article 5.14.4.3.

Spliced precast girders may be made continuous for some permanent loads using details for simple span precast girders made continuous. In such cases, design shall conform to the applicable requirements of Article

individual girders with a cast-in-place concrete composite deck rather than precasting the full width and depth of the superstructure as one piece. In some cases, the deck may be divided into pieces that are integrally cast with each girder. A bridge of this type is completed by connecting the girders across the longitudinal joints.

Girder sections are used, such as bulb tee or open-topped trapezoidal boxes, rather than closed cell boxes with wide monolithic flanges.

Provisional ducts are required for segmental construction (Article 5.14.2.3.8a) to provide for possible adjustment of prestress force during construction. Similar requirements are not given for spliced girder bridges because of the redundancy provided by a greater number of webs and tendons, and typically lower friction losses because of fewer joint locations.

The method of construction and any required temporary support is of paramount importance in the design of spliced precast girder bridges. Such considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing. *[FROM C5.14.2.1]*

Deck girder bridges are often spliced because the significant weight of the cross-section, which is comprised of both a girder and deck, may exceed usual limits for handling and transportation.

5.14.1.4.

5.14.1.3.1 Joints Between Segments

5.14.1.3.1a General

Joints between girder segments shall be either cast-in-place closure joints or match-cast joints. Match-cast joints shall satisfy the requirements of Article 5.14.2.4.2. *THE REQUIREMENT THAT SPLICES MUST BE POSTTENSIONED HAS BEEN REMOVED BECAUSE CONVENTIONALLY REINFORCED SPLICES HAVE BEEN USED SUCCESSFULLY IN SEVERAL BRIDGES.*

The sequence of placing concrete for the closure joints and deck shall be specified in the contract documents.

5.14.1.3.1b Details of Closure Joints

Precast concrete girder segments, with or without a cast-in-place slab, may be made longitudinally continuous for both permanent and transient loads with combinations of posttensioning and/or reinforcement crossing the closure joints.

The width of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations and the accommodation of the splicing of posttensioning ducts The width of a closure joint shall not be less than 12.0 IN except for joints located within a diaphragm, for which the width shall not be less than 4.0 IN. *[ALSO FROM 5.14.2.4.7c WITH REVISIONS. MINIMUM JOINT WIDTHS ARE UNCHANGED]*

If the width of the closure joint exceeds 6.0 IN, its compressive chord section shall be reinforced for confinement. *[FROM 5.14.1.2.6a]*

If the joint is located in the span, its web reinforcement, A_s/s, shall be the larger of that in the adjacent girders.

The face of the precast segments at closure joints shall be specified as either intentionally roughened to expose coarse aggregate, or having shear keys in accordance with Article 5.14.2.4.2. *[FROM 5.14.2.4.3 & 5.14.2.4.7c]*

MOVED FROM 5.14.2.4.7c
5.14.1.3.1c Details of Match-Cast Joints

Match-cast joints for spliced precast girder bridges shall be detailed in accordance with Article 5.14.2.4.2.

C5.14.1.3.1a

This article codifies current best practice, which allows the Designer considerable latitude to formulate new structural systems. The great majority of in-span construction joints have been posttensioned. Conventionally reinforced joints have been used in a limited number of bridges.

Cast-in-place closure joints are typically used in spliced girder construction. Machined bulkheads have been used successfully to emulate match-cast joints for spliced girders. Prestress, dead load and creep effects may cause rotation of the faces of the match-cast joints prior to splicing. Procedures for splicing the girder segments that overcome this rotation to close the match-cast joint should be shown on the contract plans.

C5.14.1.3.1b

When diaphragms are provided at closure joint locations, designers should consider extending the closure joint at the exterior girder beyond the outside face of the girder. This detail creates a distinct and interesting visual element that obscures differences in concrete color between the closure and the segment. Such a detail may also smooth the appearance of geometric discontinuities between segments from vertical and horizontal curves. Extending the closure joint beyond the face of the exterior girder also provides improved development of diaphragm reinforcement for bridges subject to extreme events.

The intent of the joint width requirement is to allow proper compaction of concrete in the cast-in-place closure joint. In some cases, narrower joints have been used successfully. Consolidation of concrete in a closure joint is enhanced when the joint is contained within a diaphragm. A wider closure joint may be used to provide more room to accommodate tolerances for potential misalignment of ducts within girder segments and misalignment of girder segments at erection.

The bottom flange near an interior support acts nearly as a column, hence the requirement for confinement steel. *[FROM C5.14.1.2.6a]*

AASHTO LRFD Bridge Construction Specifications requires vertical joints to be keyed. However, proper attention to roughened joint preparation is expected to ensure bond between the segments, providing better shear strength than shear keys. *[FROM C5.14.2.4.3]*

MOVED FROM C5.14.2.4.7c
C5.14.1.3.1c

One or more large shear keys may be used with spliced girders rather than the multiple small amplitude

ADD:

5.14.1.3.1d Joint Design

Stress limits for temporary concrete stresses in joints before losses specified in Article 5.9.4.1 for "Segmentally Constructed Bridges" shall apply at each stage of prestressing (pretensioning or posttensioning). The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in joints at the service limit state after losses specified in Article 5.9.4.2 for "Segmentally Constructed Bridges" shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Resistance factors for joints specified in Article 5.5.4.2.2 for segmental construction shall apply.

The compressive strength of the closure joint concrete at a specified age shall be compatible with design stress limitations. *[FROM 5.14.2.4.7c]*

MOVED FROM 5.14.2.4.7b AND REORGANIZED:

5.14.1.3.2 Girder Segment Design

Stress limits for temporary concrete stresses in girder segments before losses specified in Article 5.9.4.1 for "Other Than Segmentally Constructed Bridges" shall apply at each stage of prestressing (pretensioning or posttensioning) with due consideration for all applicable loads during construction. The concrete strength at the time the stage of prestressing is applied shall be substituted for f'_{ci} in the stress limits.

Stress limits for concrete stresses in girder segments at the service limit state after losses specified in Article 5.9.4.2 for "Other Than Segmentally Constructed Bridges" shall apply. These stress limits shall also apply for intermediate load stages, with the concrete strength at the time of loading substituted for f'_c in the stress limits.

Where girder segments are precast without prestressed reinforcement, the provisions of Article 5.7.3.4 shall apply until posttensioning is applied.

Where variable depth girder segments are used, the effect of inclined compression shall be considered.

The potential for buckling of tall thin web sections shall be considered.

The designer shall consider requirements for bracing of the girder segments once they have been erected on the substructure. Any requirements for temporary or permanent bracing during subsequent stages of construction, along

ADD:

C5.14.1.3.2

Segments of spliced precast girders shall preferably be pretensioned for dead load and all applicable construction loadings to satisfy temporary stress limits in the concrete. *MOVED FROM 5.14.2.4.7b*

Temporary construction loads must be considered where these loads may contribute to critical stresses in girder segments at an intermediate stage of construction, such as when the deck slab is placed when only a portion of the total prestress has been applied. Temporary construction loads are specified in the AASHTO *Guide Design Specifications for Bridge Temporary Works*.

Because gravity loads induce compression in the bottom flange of girders at support locations, the vertical force component from inclined flexural stresses in a haunched girder segment generally acts to reduce the applied shear. Its effect can be accounted for in the same manner as the vertical component of the longitudinal prestressing force, V_p. However, the reduction of the vertical shear force from this effect is usually neglected.

with the contractors responsibilities for designing and placing them, shall be specified in the contract documents.

5.14.1.3.3 Posttensioning

Posttensioning may be applied either before and/or after placement of deck concrete. Part of the posttensioning may be applied to provide girder continuity prior to placement of the deck concrete, with the remainder placed after deck concrete placement.

All posttensioning tendons shall be fully grouted after stressing.

Prior to grouting of posttensioning ducts, gross cross-section properties shall be reduced by deducting the area of ducts and void areas around tendon couplers.

Posttensioning shall be shown on the contract documents according to the requirements of Article 5.14.2.3.9.

Where tendons terminate at the top of a girder segment, the contract documents shall require that duct openings be protected during construction to prevent debris accumulation and that drains be provided at tendon low points.

In the case of multistage posttensioning, draped ducts for tendons to be tensioned before the slab concrete is placed and attains the minimum specified compressive strength f'_{ci} shall not be located in the slab.

Where some or all posttensioning tendons are stressed after the deck concrete is placed, provisions shall be shown on the contract plans satisfying the provisions of Article 2.5.2.3 on maintainability of the deck.

5.14.1.4 SIMPLE SPAN PRECAST GIRDERS MADE CONTINUOUS

5.14.1.4.1 General

Bridges consisting of simple span precast concrete girders and cast-in-place concrete slabs may be made continuous for transient and/or permanent loads by using a cast-in-place closure joint at piers.

Where the cast-in-place closure joint is contained within a diaphragm, the design at the closure joint may be based on the strength of the concrete in the precast

C5.14.1.3.3

Where some or all posttensioning is applied after the deck concrete is placed, fewer posttensioning tendons and a lower concrete strength in the closure joint may be required. However, deck replacement, if necessary, is difficult to accommodate with this construction sequence. Where all of the posttensioning is applied before the deck concrete is placed, a greater number of posttensioning tendons and a higher concrete strength in the closure joint may be required. However, in this case, the deck can be replaced if necessary. See Castrodale, et al. 200x.

See Article 5.10.3.5.

Where tendons terminate at the top of the girder, blockouts and pourbacks in the deck slab are required for access to the tendons and anchorages. While this arrangement has been used, it is preferable to anchor all tendons at the ends of girders. Minimizing or eliminating deck slab blockouts by placing anchorages at ends of girders reduces the potential for water seepage and corrosion at the posttensioning tendon anchors.

This provision is to ensure that ducts as yet unsecured by concrete will not be used for active posttensioning.

See Article 5.14.2.3.10e.

elements.

5.14.1.5 CAST-IN-PLACE GIRDERS AND BOX AND T-BEAMS

5.14.2 Segmental Construction

5.14.2.1 GENERAL

The requirements specified herein shall supplement the requirements of other sections of these Specifications for concrete structures designed to be constructed by the segmental method.

The provisions herein shall apply only to segmental construction using normal weight concrete.

The method of construction assumed for the design shall be shown in the contract documents. Temporary supports required prior to the time the structure, or

C5.14.2.1

For segmental construction, superstructures of single or multiple box sections are generally used. Segmental construction includes construction by free cantilever, span-by-span, or incremental launching methods using either precast or cast-in-place concrete segments which are joined to produce either continuous or simple spans.

Bridges utilizing beam-type sections may also be constructed using segmental construction techniques. Such bridges, which are referred to as spliced precast girder bridges in these Specifications, are considered to be a special case of conventional concrete bridges. The design of such bridges is covered in Article 5.14.1.3.

The span length of bridges considered by these specifications ranges to 800 FT. Bridges supported by stay cables are not specifically covered in this article, although many of the specification provisions are also applicable to them.

Lightweight concrete has been infrequently used for segmental bridge construction. Provision for the use of lightweight aggregates represents a significant complication of both design and construction specifications. Given this complication and questions concerning economic benefit, use of lightweight aggregates for segmental bridges is not explicitly covered.

The method of construction and any required temporary support is of paramount importance in the design of segmental concrete bridges. Such

component thereof, is capable of supporting itself and subsequently applied loads, shall also be shown in the contract documents.

The contract documents shall indicate alternative methods of construction permitted and the Contractor's responsibilities if such methods are chosen. Any changes by the Contractor in the construction method or in the design shall comply with the requirements of Article 5.14.2.5.

considerations often govern final conditions in the selection of section dimensions and reinforcing and/or prestressing.

For segmentally constructed bridges, designs should and generally do allow the Contractor some latitude in choice of construction methods. To ensure that the design features and details to be used are compatible with the proposed construction method, it is essential that the Contractor be required to prepare working drawings and calculations based on his choice of methods for review and approval by the Engineer before work begins.

5.14.2.3 DESIGN

5.14.2.3.10 Box Girder Cross-Section Dimensions and Details

5.14.2.3.10e Overlays

Overlays shall be considered for all bridge decks exposed to freeze thaw cycles and application of deicing chemicals. The governing authority should consider providing additional protection against penetration of chlorides. For all types of segmental bridges (precast and cast-in-place), it is recommended that this additional protection be provided by the addition of a minimum of 1.5 IN of concrete cover, added as an overlay or alternatively a waterproof membrane with bituminous overlay. The governing authority may require specific materials and placement techniques stipulated by local practices.

C5.14.2.3.10e

Overlays are encouraged instead of the inclusion of monolithic concrete additional because an overlay will add protection at the critical segment joint. Delamination of overlays is generally due to poor installation practices or material selection and can be resolved. It is not recommended that the additional cover be obtained by merely increasing concrete covers. The added cover will not add protection across the segment joint which is the area of most concern due to the ability of the water to migrate to the tendon and reinforcement.

Careful attention to detail is required when using overlays to assure the proper railing heights are obtained. All railings next to deck areas to be overlayed should be detailed from the top of the overlay.

The need to remove and replace the overlay can be based on measurement of chloride penetration into the overlay. Use of high performance concrete is an effective means of minimizing chloride penetration into concrete.

Bridges located in other corrosive environments, such as coastal bridges over salt water, should be evaluated for the need for additional protection.

5.14.2.4 TYPES OF SEGMENTAL BRIDGES

5.14.2.4.1 General

Bridges designed for segmentally placed superstructures shall conform to the requirements specified herein, based on the concrete placement method and the erection methods to be used.

C5.14.2.4.1

Precast segmental bridges are normally erected by balanced cantilever, use of erection trusses, or progressive placement.

Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons. Bridges built with erection trusses may utilize internal tendons, external tendons, or combinations thereof. Due to considerations of segment weight, span lengths for precast segmental box

5.14.2.4.2 Details for Precast Construction

The compressive strength of precast concrete segments shall not be less than 2.5 KSI prior to removal from the forms and shall have a maturity equivalent to 14 days at 70ºF prior to assembly into the structure.

Multiple small-amplitude shear keys at match-cast joints in webs of precast segmental bridges shall extend over as much of the web as is compatible with other details. Details of shear keys in webs should be similar to those shown in Figure 1. Shear keys shall also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single-element keys.

Figure 5.14.2.4.2-1 NOT SHOWN

Joints in precast segmental bridges shall be either cast-in-place closures or match cast epoxied joints.

Precast segmental bridges using internal posttensioning tendons and bridges located in areas subject to freezing temperatures or deicing chemicals shall employ bonded joints.

A temporary prestressing system shall provide a minimum compressive stress of 0.030 KSI and an average stress of 0.040 KSI across the joint until the epoxy has cured.

5.14.4 Slab Superstructures

5.14.4.3 PRECAST DECK BRIDGES

5.14.4.3.3 Shear-Flexure Transfer Joints

girder bridges, except for cable-stayed bridges, rarely exceed 400 FT.

C5.14.2.4.2

This provision intends to limit the magnitude of construction deflections and to prevent erratic construction deflections and creep.

Small-amplitude shear keys in the webs are less susceptible to construction damage, which will result in loss of geometry control, than larger single-element keys. Shear keys in the top and bottom flanges are less susceptible to such damage

Match casting is necessary to ensure control of the geometry upon reassembly of the segments.

Epoxy on both faces serves as a lubricant during placement of the segments, prevents water intrusion, provides a seal to prevent cross-over during grouting, and provides some tensile strength across the joint.

The use of dry joints (identified as Type B in past versions of these specifications) was eliminated with the adoption of the 2003 revision due to the critical nature of posttensioning reinforcing and the need for a multiple layer protection system. Failures of some posttensioning reinforcing in Florida and Europe due to corrosion have resulted in a review of the effectiveness of previous multiple layer protection systems. The most rigorous review was performed by the British Concrete Society and the recommendations are contained in the report titled "Durable Post-tensioned Concrete Bridges." This European report codifies the need for a three-level protection system and suggested details to achieve the required results. Improved grout and duct materials and methods are also discussed. As a result of this European Report and studies by Dr. John Breen of the University of Texas, Austin, the multiple level protection system for posttensioning has been universally accepted.

THIS PARAGRAPH WAS APPROVED TO BE ADDED IN 2002. SEVERAL OTHER MINOR REVISIONS WERE ALSO MADE TO THIS ARTICLE AT THE SAME TIME.

AASHTO LRFD Bridge Construction Specifications requires this temporary stress to ensure full bond and to prevent uneven epoxy thickness. Such variations could lead to a systematic accumulation of geometric error. Large stress changes on epoxy joints should be avoided during the initial curing period.

5.14.4.3.3d Longitudinal Construction Joints

Longitudinal construction joints between precast concrete flexural components shall consist of a key filled with a nonshrinkage mortar attaining a compressive strength of 5.0 KSI within 24 hours. The depth of the key should not be less than 6.5 IN.

If the components are posttensioned together transversely, the top flanges may be assumed to act as a monolithic slab. However, the empirical slab design specified in Article 9.7.2 is not applicable.

The amount of transverse prestress may be determined by either the strip method or two-dimensional analysis. The transverse prestress, after all losses, shall not be less than 0.25 KSI through the key. In the last 3.0 FT at a free end, the required transverse prestress shall be doubled. The transverse posttensioning tendons should be located at the centerline of the key.

C5.14.4.3.3d

This article relates to deck systems composed entirely of precast beams of box, T- and double-T sections, laid side-by-side and, preferably, joined together by transverse posttensioning.

Grinding of grout and concrete in the vicinity of the joint may be expected and specified for construction.

REFERENCES

Add the following to the list of references currently appearing at the end of Section 5:

Castrodale, R.W., White, C.D., "Extending Span Ranges of Precast, Prestressed Concrete Girders," Project No. 12-57, National Cooperative Highway Research Program, NCHRP, National Research Council, Washington DC, submitted September 2003.

APPENDIX E2

LIST OF POTENTIAL ISSUES IN AASHTO LRFD SPECIFICATIONS

APPENDIX E2
LIST OF POTENTIAL ISSUES IN AASHTO LRFD SPECIFICATIONS

1 INTRODUCTION

Based on a careful review of the *LRFD Specifications* and discussions with design engineers, over 60 items related to methods for extending span ranges of precast, prestressed concrete girders have been identified. This list contains items related to conventional construction and post-tensioned spliced girders. Therefore, the scope of this list is broader than the proposed revisions for spliced girder construction. The actions required to resolve these items vary widely from editorial corrections, to investigation of the background and justification of current provisions, to review of the current practice, to in-depth design comparisons, and to additional research involving physical testing.

Items that are general, affecting conventional construction as well as extended span girders, have not been included even though they may have some effect on the design of girders with extended spans. The majority of the general items have already been noted in an earlier comprehensive review of the *LRFD Specifications* conducted for PCI *(1)*.

In most cases, items of a typographical or style nature have not been included in the current list. Many of these have also been identified in the PCI study.

Identified items have been placed in the same tabular format used for the PCI study mentioned above. This format has been selected because it has been found to provide the information in a useful format. The members of the AASHTO Technical Committee on Concrete Design (T-10), who may eventually review the results of this research project, are also familiar with this format since they have been using the PCI report to identify possible revisions to the *Specifications*.

The comments contained in this list will in many cases be different than those presented in the PCI study, even when addressing the same articles and issues. This difference exists because the client and objective of the study are different than for the earlier study.

In most cases, each item in the list applies to a single article or part of an article. The following information is provided for each item:

- Index number (each item is numbered sequentially for reference)
- Article reference in the *LRFD Specifications*

- Corresponding article in the *Standard Specifications*. Not all comments have a corresponding article listed.

- Description of item

- Comment or description of concern

- Proposed action or revision

- Reason for revision

- A subjective evaluation of the priority of the issue (High/Medium/Low)

- A subjective evaluation of the level of effort required to address the issue (High/Medium/Low) or an evaluation of the type of the required revision (Editorial/ Clarification).

Comments are also added where the *Specifications* do not provide provisions that the research team feels are necessary. Some editorial revisions are made or missing words supplied when quoting provisions from the *Specifications*.

2   SUMMARY OF MAJOR ISSUES

Several issues have been identified that are of special significance. These are highlighted in the following and may also be included in the list of comments where appropriate. Several items in the list may be associated with each of the following issues.

2.1   Integration of Provisions for Segmentally and Other Than Segmentally Constructed Bridges

It is not clear how spliced concrete girders are to be considered in the current *Specifications*. It appears that spliced girder construction could be classified as "Segmentally Constructed" or "Segmental Components," which would appear to make them subject to all the requirements of segmental bridges. However, many of the articles in the *Specifications* that apply to segmentally constructed bridges appear to be intended for bridges where the segments are relatively short and for which match-cast joints are typically used. Many provisions for this type of construction are not appropriate, necessary or applicable for spliced concrete girder bridges.

A particular area of concern is the limiting stresses, which are given in Article 5.9.4. Limiting stresses are provided for two categories of bridges: "Other than Segmentally Constructed Bridges" and "Segmentally Constructed Bridges." By definition, spliced girders appear to fall into the category of "Segmentally Constructed Bridges." However, the only limiting tensile stresses given for this type of construction are for the joints. This is appropriate for bridges with short segments where

stresses at splice locations will govern designs, but is not appropriate for bridges with long segments where limiting stresses for "Other than Segmentally Constructed Bridges" should apply at locations away from the splices.

A second particular area of concern is Article 5.14 that presents provisions for specific structure types. Spliced girder construction is specifically mentioned in the *Specifications*, but the designer is left to choose which of the articles apply. There is an article for "Beams and Girders" (5.14.1) and an article for "Segmental Construction" (5.14.2). Both articles contain provisions that can be applied to spliced girder construction. The revision of these sections of Article 5.14 could require significant effort because it would be so comprehensive. An initial attempt to accomplish this revision has been presented in Appendix E1.

The integration of differing design provisions for limiting stresses and other issues for the various types of construction was a major focus of the effort in developing the proposed revisions for the *Specifications*.

2.2 Definitions of Lightweight Concrete

The *Specifications* are not consistent or clear in their treatment of lightweight concrete. This includes definitions of the different types of lightweight concrete and corrections for different quantities, such as shear capacity and resistance factors, that depend on the type of lightweight concrete.

This lack of clarity and consistency makes it difficult to use specified density concrete, which may have a range of unit weights and compositions. The *Specifications* need to be revised so that any special considerations required for using reduced density concrete to extend span ranges will be simple, clear and rational.

2.3 Partial Prestressing

Partial prestressing has been introduced into United States bridge design practice for the first time in the *LRFD Specifications*. However, complete direction for its implementation has not been provided. The authors are not aware of any problems to date concerning this issue because owners and designers presently do not have applications where the concept can be applied.

One possible application where the implementation of partial prestressing may be beneficial is the design of bridge decks in the negative moment region over interior supports. Using this concept could clarify the issue concerning allowable tensile stresses in the deck at these locations for continuous bridges (see next item). However, since the concept of partial prestressing has not been well developed

in the *Specifications*, considerable effort may be required to identify design methods and understand the advantages and disadvantages of this approach.

2.4 Deck Design for Continuous Members

In continuous members that are post-tensioned after the deck has been added, the deck may be subject to tensile stresses due to continuity moments at interior pier locations. As a result, tensile stress limits may be applied to the deck. This may lead to additional prestress (post-tensioning) being provided to satisfy the tensile stress limits in the deck. However, tensile stresses in the deck are not subject to limiting stresses in other types of continuous bridges, such as those constructed on steel girders.

Further study is needed on this issue. Its resolution may not be simple and it will likely involve several of the AASHTO Technical Committees.

3 REFERENCES

1. Castrodale, Reid W., "Review of LRFD Specifications." Special Study, PCI, Chicago, IL (2000).

REVIEW OF THE
AASHTO LRFD BRIDGE DESIGN
SPECIFICATIONS

Index No	LRFD Specs Article No. *Std. Specs Article No.*	1. Description of Item 2. Comments or Description of Concern 3. Proposed Action or Revision 4. Reason for Revision	Priority	Level of Effort or Type of Revision
1	2	3	4	5
1	2.5.2.3	1. The 1st paragraph states: "Where the climatic and / or traffic environment is such that the bridge deck may need [to] be replaced prior to the [end of the] required service life, either provisions shall be shown on the contract plans for the replacement of the deck, or additional structural resistance shall be provided." 2. Requiring design of a bridge to accommodate removal of the concrete deck during the service life of the structure may eliminate several economical design options from consideration. This is a critical issue for concepts where a final stage of post-tensioning is applied after the deck concrete has been placed and cured. In this case, the girders may not be capable of resisting the post-tensioning force remaining when the deck is removed. Other options are available for increasing the service life for concrete decks. The beneficial effect of the precompression of the deck from the final stage post-tensioning should also be considered where this is available. The intent of the last phrase of the referenced paragraph is unclear regarding the purpose of the additional resistance. 3. Evaluate and propose other options available for improving performance and durability of concrete decks to relax or eliminate the requirement for deck removal. Revise article to reflect findings of evaluation. Provide additional information in the *Specifications* or commentary to clarify intent. 4. Clarify intent of the provisions and make available other options for improvement of deck durability so use of bridges where decks are prestressed with final stage post-tensioning would not be eliminated unnecessarily.	High	Medium
2	2.5.2.6.3 C2.5.2.6.3 *8.9.2*	1. The *Specifications* provide optional minimum depth criteria for limiting deflections. The commentary states: "Traditional minimum span-to-depth ratios for constant depth superstructures, contained in previous editions of the *Standard Specifications*, are given in Table 1 with some modifications." 2. No such limits have been provided for prestressed concrete bridges in the *Standard Specifications*. Therefore, the limits appearing in the *LRFD Specifications* are new. The source for the new limits is not given. 3. Request additional information from the authors of the *LRFD Specifications* regarding the source of the minimum depth criteria for prestressed concrete bridges. Prepare proposed revision for the commentary to clarify the *Specifications* and indicate that, for prestressed concrete bridges, these are not traditional limits that have been used for many years, but are new limits. 4. Correct an inaccurate statement in the *Specifications*.	Low	Clarification
3	2.5.2.6.3 C2.5.2.6.3 *8.9.2*	1. The *Specifications* provide optional minimum depth criteria for limiting deflections. The commentary states: "Traditional minimum span-to-depth ratios for constant depth superstructures, contained in previous editions of the *Standard Specifications*, are given in Table 1 with some modifications." 2. Because these limits are optional, this does not appear to be an urgent issue. However, some owners and designers rely on such information to develop and evaluate alternative designs. Therefore, improved limits based on current practice should be developed. Additional categories may be useful to indicate the reduced depth possible with some types of precast, prestressed concrete girder construction. 3. Review collected data to determine current practice regarding span-to-depth ratios. Prepare proposed revisions based on this	Medium	Medium

Index No	LRFD Specs Article No. *Std. Specs Article No.*	1. Description of Item 2. Comments or Description of Concern 3. Proposed Action or Revision 4. Reason for Revision	Priority	Level of Effort or Type of Revision
1	*2*	*3*	*4*	*5*
		review, possibly including one or more additional categories. 4. Provide current criteria for span-to-depth ratios for prestressed concrete members.		
4	2.5.5	1. This article addresses bridge aesthetics. The article provides a list of "guidelines" that "should be considered" to produce aesthetically pleasing bridges. The final bullet item in the list states: "For spanning deep ravines, arch-type structures should be preferred." 2. The final bullet item is of concern since it indicates a preferred structure type (arch) for a given application (deep ravines). Many attractive constant depth or haunched girder structures have been constructed over deep ravines. Since the list of issues is preceded with permissive language ("should be considered"), it appears that the bullet list in this article may be more appropriately located in the Commentary. Some owners and designer tend to take even permissive information found in the *Specifications* as required. An additional concern is that this list appears to make aesthetics the first issue to be considered rather than one of several important issues that must be considered when selecting a structure type. 3. Propose the following (or similar) revision to the final bullet item "… arch-type structures should be ~~preferred~~ considered." Consider proposing the movement of the bullet list in this article to the Commentary. 4. Relax the statement of a subjective preference that could limit or inhibit consideration of some accepted and innovative design alternatives.	Low	Editorial
5	3.3.2 3.4.1 Tables 3.4.1-1 3.4.1-2	1. The load combination tables define a load effect EL as "Locked in Erection Stresses". These are defined in Article 3.3.2 as "accumulated locked in force effects resulting from the construction process, including the secondary forces from post-tensioning." These EL forces use a permanent load factor, γ_P, of 1.0. 2. While it is good that the *LRFD Specifications* acknowledge the existence of secondary prestressing effects and provide guidance concerning their use at the Strength Limit State (ultimate), the concept of Locked in Erection Stresses (Forces) is redundant and confusing. No additional erection forces exist that have not already been accounted for by other permanent load effects (i.e., DC, DW, etc.) or time-dependent effects (i.e., CR, SH, etc.) In fact, the Commentary to Article 3.4.1 refers to " locked-in force effects due to TU, TG, CR, SH and SE". However, these load effects and the permanent dead load effects, have their own load factors prescribed in Table 3.4.1-1, leaving the designer to ponder what other EL effects should be separated out with a factor of 1.0. 3. Propose a revision to change the EL load category to SP, "Secondary Prestress Effects." 4. Eliminate the misconception that "locked in erections forces" (other than secondary prestressing) differ from or can be separated out from other permanent and/or time-dependent load effects.	Low	Editorial
6	3.4.1	1. The Strength IV Limit State applies for structures with high dead to live load ratios. According to the commentary, this limit state was introduced to allow use of a single set of resistance factors for small-, medium- and long-span bridges. 2. It has been reported that this limit state will have a significant negative impact on long-span concrete bridge designs. The effect will be to reduce the efficiency of the long-span designs. The	High	High

Index No	LRFD Specs Article No. / Std. Specs Article No.	1. Description of Item / 2. Comments or Description of Concern / 3. Proposed Action or Revision / 4. Reason for Revision	Priority	Level of Effort or Type of Revision
1	2	3	4	5
		effect of this limit state is not known for the extended span concepts considered in this research.		
		The situation is similar to what was found when trial designs were performed for prestressed concrete girder bridges (see discussion of Service III Limit State below). It is more difficult to demonstrate problems with the *Specifications* for long-span bridges because of the increased complexity of the design of this type of structure and the reduced number of designers able to perform the required computations.		
		3. The effect of this limit state on long-span precast, prestressed concrete girder designs should be reviewed by generating comparative designs for the *LRFD* and *Standard Specifications* in the range of spans anticipated for the extended span concepts considered in this research. Based on this review and further study of the calibration process, revisions to the *Specifications* may be proposed to AASHTO.		
		4. Ensure that the economy of long-span concrete bridges considered in this research project is not unnecessarily affected by this provision.		
7	3.4.1 C3.4.1	1. Service III Limit State applies only to limiting " … tension in prestressed concrete structures with the objective of crack control." .	Medium	High
		2. It seems reasonable that the reduced load factor for live load used in this service limit state should be applied to both tension and compression in prestressed concrete members. It appears that it currently only applies to tension because, in trial designs, the tensile stress governed a significant number of designs, which resulted in reduced span capabilities (or increased strand requirements) when designs using the *LRFD Specifications* were compared to designs using the *Standard Specifications*. This reduction in span capability for designs using the *LRFD Specifications* is caused by the increased vehicular live load with unchanged load factors and limiting stresses for the Service Limit State.		
		It is felt that, if compression had also governed in some of the trial designs conducted during the development of the *LRFD Specifications*, perhaps Service III Limit State would have been extended to apply to compression as well. Compressive stresses often become critical in long-span continuous bridges.		
		3. Evaluate the impact of revising the Specification for the live load reduction in Service III to apply to compression as well as tension. Based on the results of the evaluation, develop a proposed revision to the *Specifications* to allow application of Service III Limit State to both tension and compression in prestressed concrete members.		
		4. Applying the reduced live load factor to both tension and compression simplifies design and ensures that designs governed by compression are not penalized by the higher vehicular live load used in the *LRFD Specifications*.		
8	3.4.1 3.12.1 3.12.3 C3.12.3	1. Article 4.3.1 states: "The load factor for temperature gradient, γ_{TG}, and settlement, γ_{TG}, should be considered on a project-specific basis". The commentary for this article states "Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e., $\gamma_{TG} = 0$".	High	Clarification
		Article 3.12.1 states: "the effect of temperature gradient should be included where appropriate."		
		Article 3.12.3 gives details for consideration of temperature		

Index No	LRFD Specs Article No. Std. Specs Article No.	1. Description of Item 2. Comments or Description of Concern 3. Proposed Action or Revision 4. Reason for Revision	Priority	Level of Effort or Type of Revision
1	2	3	4	5
		gradients. The first paragraph of the commentary for Article 3.12.3 reads: "Temperature gradient is included in various load combinations in Table 3.4.1-1. This does not mean that it need be investigated for all types of structures. If experience has shown that neglecting temperature gradient in the design of a given type of structure has not led to structural distress, the Owner may choose to exclude temperature gradient. Multi-beam bridges are an example of a type of structure for which judgment and past experience should be considered".		
		2. Neither article specifies with certainty conditions for which temperature gradient should be considered. In the past this provision has been selectively applied to segmental concrete bridges alone, thereby penalizing these types of structures when compared to other alternatives. This practice has been extended to post-tensioned spliced girder bridges on a number of past and recent projects. It seems that this inequity between structure types has arisen out of differences between working stress design methods applied to prestressed concrete structures and strength design methods applied to steel and other structures with reinforced concrete decks. Despite the improved performance of bridge decks that are precompressed longitudinally due to prestressing and concrete structures when compared to steel, these types of concrete structures are handicapped with regard to temperature gradient. Despite the indication in the commentary that multi-beam bridges (e.g., spliced girder bridges) have not suffered from neglecting temperature gradient in design in the past, many owners are still requiring that temperature gradient be considered in the design of spliced girder bridges.		
		3. Request additional information from the authors of the *Specifications* to provide justification for why temperature gradient criteria are not applied equally to all types of structures. Prepare proposed revisions to the *Specifications* to either (1) require that temperature gradient be included for all structure types, or (2) to allow temperature gradient to be neglected for all structure types, or (3) some combination of (1) and (2) depending on the size and importance of the structure, while still giving the owner agency the option of applying their own criteria. It appears that any proposed revisions for this item would have to be considered by several of the AASHTO Technical Committees.		
		4. Ensure that temperature gradient effects are considered equally and appropriately for bridges of all materials and structure types.		
9	3.12.4	1. The article and commentary address differential shrinkage. The article states "Where appropriate, differential shrinkage strains ..." shall be considered.		
		2. The *Specifications* do not indicate where consideration of differential shrinkage is required. No equation or methodology for computing the magnitude or effect of differential shrinkage is provided in the *Specifications*. References are available, but are not given.		
		3. Review existing literature on estimating the effect of differential shrinkage. Develop revised procedures if required. Evaluate the effect of differential shrinkage on extended span concrete bridges. Based on this evaluation, prepare proposed revisions to the *Specifications* to include references for computation of differential shrinkage effects and to indicate when differential shrinkage should be considered.	Medium	High
		4. Provide necessary information to designers to satisfy requirements of the *Specifications* and clarify when differential shrinkage should be considered in design.		

Index No	LRFD Specs Article No. Std. Specs Article No.	1. Description of Item 2. Comments or Description of Concern 3. Proposed Action or Revision 4. Reason for Revision	Priority	Level of Effort or Type of Revision
1	2	3	4	5
10	4.2	1. The definition for "Monolithic Construction" indicates that adjacent deck elements are "effectively tied together by transverse posttensioning." 2. Article C4.6.2.2.1 and elsewhere indicates that a structural reinforced concrete overlay is an option for establishing monolithic behavior for adjacent elements. Other options may also exist, such as diaphragms and welded flange connections in decked bulb tees. This research project only has interest in this item as it relates to decked bulb-tee girders. 3. Evaluate options available for establishing monolithic behavior in adjacent unit bridge decks. Revise definition as required. Effort on this item for this research project would be limited to decked bulb-tee girder bridges. 4. Provide consistency with the commentary to Article 4.6.2.2.1 and expand options to indicate current practice.	Low	Editorial
11	4.5.2.2 C4.5.2.2	1. The 2nd paragraph of the article states: "The stiffness properties of concrete and composite members shall be based upon cracked and/or uncracked sections consistent with the anticipated behavior. Stiffness characteristics of beam-slab-type bridges may be based on full participation of concrete decks." The commentary states: " ... cracking of concrete seems to have little effect on the global behavior of bridge structures. This effect can, therefore, be safely neglected by modeling the concrete as uncracked for the purposes of structural analysis." 2. Cracking of decks is difficult to avoid in most bridges. Continuous structures are generally more susceptible to deck cracking than simple spans. However, bridges made continuous by post-tensioning over interior piers are frequently designed for the deck to remain in compression, and therefore uncracked, at the service limit state. Computed service limit state stresses in continuous structures will be affected by whether the deck is assumed to be cracked in negative moment regions. The assumed condition of the deck for service limit state analysis should be consistent for both concrete and steel superstructures and should reflect the beneficial effect of prestress on the deck, where appropriate. It is not clear whether "full participation of concrete decks" mentioned in the last sentence of the article is intended to indicate an uncracked deck or simply full composite action between the girder and deck. 3. Evaluate the effect of considering potential deck cracking in the design of continuous bridges, especially those made continuous with post-tensioning. The impact of using cracked deck properties in service limit state analysis should be reviewed. Based on these studies, propose reasonable assumptions for use in analysis, especially for continuous structures. If the deck is assumed to crack, guidance should be provided regarding the effect and extent of the cracking. The scope of the investigation of this item may need to be limited for the purposes of this research study. 4. Ensure that structural modeling of bridges reflects actual behavior and provides conservative results.	Medium	High
12	4.6.2.2.1	1. This article defines the term S as "spacing of beams or webs." The term is used in equations to compute approximate live load distribution factors. 2. Application of this definition is not clear for open-topped trapezoidal box beams. 3. Request clarification from authors of the Specifications regarding the definition of "S" for open-topped trapezoidal box beams.	Medium	Clarification

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		Obtain or develop proposed revision to definition, or add commentary to clarify definition for open-topped box girders. 4. Avoid improper use of approximate live load distribution factors.		
13	C4.6.2.2.1	1. The commentary states: "The transverse post-tensioning shown for some cross-sections herein is intended to make the units act together. A minimum 0.25 KSI prestress is recommended." 2. This level of transverse prestress is very high compared to many details in current use. For a 6 IN deep key, this would require 0.5 IN diameter strands spaced at less than 2 FT to achieve the specified level of prestress. This requirement would reduce the economy of construction methods that utilize this type of joint. Additional methods for establishing monolithic behavior are available and should be listed. Weld plates and diaphragms are two options used for decked bulb-tee construction. This research project only has interest in this item as it relates to decked bulb-tee girders. 3. A complete study of this issue is beyond the scope of this current research project. This issue has been discussed among practitioners for several years and has been proposed for funded research, but no solution has been developed. Effort on this item for this research project would be limited to consideration of decked bulb-tee girder bridges. Review the current practice for decked bulb tees. Based on this review, develop a proposed revision addressing the limited case of decked bulb tees. 4. Provide reasonable requirements for establishing monolithic behavior using transverse post-tensioning and other methods for decked bulb tees.	Low	High
14	4.6.2.2.2b	1. The 3rd paragraph states: "For concrete beams, other than box beams, used in multi-beam decks with shear keys", the second bullet requires: "If the stem spacing of stemmed beams is less than 4.0 FT or more than 10.0 FT, a refined analysis complying with Article 4.6.3 shall be used." 2. This research project only has interest in this item as it relates to decked bulb-tee girders. Decked bulb tees could possibly be used with stem spacings greater than 10 FT. It seems unnecessary for these members to be designed using refined methods. 3. Review background documents to determine origin of limit on stem spacing. Evaluate simplified methods to determine if any of the other cross-section types is applicable to this type of member, or identify or develop a simplified method applicable for these members. Develop proposed revision addressing the limited case of decked bulb tees. 4. Allow simplified designs using widely spaced decked bulb tees.	Low	Medium
15	Tables 4.6.2.2.2b-1 4.6.2.2.2d-1 4.6.2.2.3a-1 4.6.2.2.3c-1	1. The listed tables provide equations for computing approximate live load distribution factors. For some of the equations in the tables, a "Range of Applicability" is defined. The maximum span length in the "Range of Applicability" for concrete decks on precast I-girders (type "k") is 240 ft, and 140 ft for concrete decks on open-topped trapezoidal (tub) girders (type "c"). 2. Post-tensioned, spliced girder bridges have been built with spans in excess of 240 ft on several occasions. Similarly, precast tub girders have been used for spans exceeding 140 ft. Design by the *LRFD Specifications* requires spans in excess of the stated limits to be designed using refined methods. It seems unnecessary for these members to be designed using refined methods. 3. Review background documents to determine origin of limit on span	Medium	Medium

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		length. Evaluate simplified methods to determine if the span limits can be increased, or identify or develop a simplified method applicable for these members. Based on this evaluation, develop proposed revision to the *Specifications*. 4. Make simplified design method for live load distribution applicable to longer spans than currently shown.		
16	4.6.2.9	1. Article 4.6.2.9 is titled, "Analysis of Segmental Concrete Bridges". 2. Based on later definitions of segmental construction in Section 5, it is unclear whether this article is intended to include spliced multi-girder bridges or only box sections with full width segments. 3. Review the *Specifications* and develop proposed clarifications that will establish when the deck-beam bridge provisions are applicable and when the segmental provisions are applicable. 4. Avoid confusion concerning provisions applicable to post-tensioned, spliced girder bridges.	Medium	Medium
17	4.6.2.9.1	1. The 2nd paragraph states: "For spans in excess of 250 ft, results of elastic analysis should be evaluated with consideration of possible variations in the modulus of elasticity of the concrete, variations in the concrete creep and shrinkage properties, and the impact of variations in the construction schedule on these and other design parameters." 2. This applies to segmental concrete bridges (Article 4.6.2.9). Assuming that this paragraph applies to multi-girder systems as well as conventional segmental bridges, the arbitrary span length limit of 250 ft appears to have no rational basis. Bridges constructed in such a manner require a more detailed analysis and special consideration no matter what their span length. The completeness and correctness of the design should not be vastly different depending on whether the span length is 249 ft or 251 ft. 3. Review the applicability of time-dependent parameters on spliced multi-girder bridge design as compared to segmental design. Determine whether more simplified design and analysis methods are possible. Based on this review, develop proposed revisions. 4. Eliminate arbitrary span limit in lieu of more rational guidelines for determining whether more detailed analysis and design procedures are required.	Medium	Medium
18	5.2	1. Definitions are given for "Lightweight Concrete", "Normal-Weight Concrete" and "Sand-Lightweight Concrete". • Lightweight Concrete is defined as concrete "having an air-dry unit weight not exceeding 0.120 KCF." • Normal-Weight Concrete is defined as concrete "having a [unit] weight between 0.135 and 0.155 KCF". • Sand-Lightweight Concrete is defined only in terms of the type of aggregates used. No range of unit weights is given. 2. Definitions for types of concrete, related to unit weight, are necessary because later provisions in the *Specifications* utilize these terms. Definitions used in the past may need revision to properly address current practice. The structure and content of the definitions also varies as noted above. 3. Review current practice and develop proposed revisions for the definitions for different types of concrete. 4. In some parts of the country, the use of lightweight concrete and reduced density concrete is increasing. Therefore, the *Specifications* must be clarified to avoid penalizing or discouraging the use of these materials.	Medium	Low

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19	5.2	1. "Partially Prestressed Concrete" is defined as "Concrete with a combination of prestressing strands and reinforcing bars." 2. This definition appears too vague. All prestressed concrete members contain mild reinforcement in addition to prestressing steel. It is assumed that the definition is intended to mean "Concrete with flexural tensile reinforcement comprised of a combination of prestressing tendons and mild reinforcement." Furthermore, even if the definition is revised as proposed above, this definition does not appear to capture what is generally intended when the concept of partial prestressing is used, and that is that concrete is allowed to crack at service loads, resulting in reduced section properties and member stiffness. 3. Review current practice and develop proposed revisions for the definition of Partially Prestressed Concrete. Evaluate, as mentioned below, the current definition of partially prestressed concrete, and develop a proposed revision if necessary. 4. Clarify the intent of the *Specifications* so that subsequent provisions related to partially prestressed concrete can be correctly applied.	Low	Low
20	5.2	1. "Segmental Component" is defined as "A component made up of individual elements, either precast or cast-in-place, and posttensioned together to act as a monolithic unit under loads." 2. This is a general definition. However, it is clear that the intended concept of segmental construction throughout the *Specifications* generally refers to short segment box girder structures. Also, while this definition is intended to be general, the use of post-tensioning is assumed. In a few cases, segmentally constructed (spliced girder) bridges have been spliced without post-tensioning. 3. Consider refinement of the definition. This will be part of the larger reassessment of the concept of segmental construction that needs to be done with the *LRFD Specifications*. 4. Clarify applicability of sections of the *Specifications* related to segmental construction.	High	Low
21	5.4.2.1	1. This article states: "Concrete strengths above 10.0 KSI shall be used only when physical test are made to establish the relationships between the concrete strength and other properties." 2. The use of concrete at or above 10 KSI is becoming much more commonplace. Research is underway to summarize the results of many of the projects for which high performance/high strength concrete has been used for bridges. This requirement appears to be more restrictive than necessary and may need updating. 3. Review definition in light of current practice and recent studies. The maximum concrete strength for which special examination of the material is required prior to use in a structure may need to be increased. 4. Update *Specifications* to allow more common use of high strength concrete where appropriate.	High	Medium
22	5.4.2.1	1. This article states: "The sum of Portland cement and other cementitious materials shall be specified not to exceed 800 PCY." 2. Some high performance concrete mixes used in recent bridge projects have successfully used cementitious contents well in excess of the 800 PCY limit. With proper attention to material selection and proportioning, placement, and curing, it has been demonstrated that it is possible to successfully use concrete mixes with higher cementitious contents. 3. Review current studies of high performance concrete to	High	Low

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		demonstrate that this limit is restrictive. Revise or eliminate limit to reflect current practice. 4. Remove obstacles to the use of high strength concrete for more economical bridges.		
23	5.4.2.3.1 8.5.4	1. The first paragraph of the article states: "Values of shrinkage and creep, specified herein and in Articles 5.9.5.3 and 5.9.5.4 [the approximate lump sum and refined loss estimates of time-dependent losses] shall be used to determine the effects of shrinkage and creep on the loss of prestressing force in bridges other than segmentally constructed ones." The 4th paragraph of the article states: "For segmentally constructed bridges, a more precise estimate shall be made, including the effect of specific materials, structural dimensions, site conditions, and construction methods." 2. This article indicates that the loss methods of Articles 5.9.5.3 and 5.9.5.4 cannot be used for segmentally constructed bridges. A more refined analysis is required, including knowledge of the specific materials being used. However, it would be expected that the estimated loss methods would still be appropriate for segmentally constructed girder bridges. Knowledge of actual materials, while possibly necessary in very large bridges, should not be required. 3. Evaluate whether the methods for estimating prestress losses mentioned above are still applicable for spliced girder bridges. Evaluate the sensitivity of spliced girder bridges to variation in material properties. Based on the results of the evaluation, develop proposed revisions to this article as needed. 4. Allow use of simple and familiar loss computation methods for spliced girder bridges, if appropriate.	Medium	High
24	5.4.2.3.1 8.5.4	1. The 2nd paragraph of the article states: "In the absence of more accurate data, the shrinkage coefficients strains may be assumed to be 0.0002 after 28 days and 0.0005 after one year of drying." 2. The final shrinkage strain differs significantly from the total value of 0.0002 given in Article 8.5.4 of the Standard Specifications. This may have a significant, possibly detrimental, effect on designs. No justification for these values is given. An editorial change is also suggested as noted above. 3. Evaluate the significance of this change. Develop proposed revision to article based on the results of the evaluation. Propose editorial revision as noted. 4. Provide reasonable estimates of shrinkage for designers.	Medium	Medium
25	5.4.4.2	1. The last paragraph of this article states: "Where complete prestressing details are included in the contract documents, the size and grade or type of steel shall be shown. If the plans indicate only the prestressing forces and location of application, the choice of size and type of steel shall be left to the Contractor, subject to the Engineer's approval." 2. While the specification of only the forces and locations of prestressing has been widely used in the past, designers should be cautioned that integrated shop drawings should be prepared to avoid potential conflicts of post-tensioning hardware and associated reinforcement with other reinforcement in the member. 3. Review current practice. Provide commentary and/or proposed revision to the Specifications. 4. Update Specifications to current practice and warn designers of potential problems with clearances and conflicts between post-tensioning hardware and other reinforcement.	Low	Low

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26	5.4.6.2	1. This article states: "… the cross-sectional area of the duct shall be at least 2.0 times the net area of the prestressing steel with one exception where tendons are to be placed by the pull-through method, the duct area shall be at least 2.5 times the net area of the prestressing steel." The commentary for this article states: "The pull-through method of tendon placement is usually employed by contractors where tendons exceed 400 FT in length." 2. These statements reflect common practice. However, the designer of a longer bridge is left with uncertainty regarding the size of duct that must be detailed, because the duct size may depend on the preferences of the contractor, rather than on a specific requirement. 3. Review current practice to determine whether revision is necessary. If so, develop proposed revision and/or commentary for the *Specifications*. 4. Provide more definite guidance for designers regarding size of duct.	Medium	Low
27	5.4.6.2	1. The 2nd paragraph of this article states: "The size of ducts shall not exceed 0.4 times the least gross concrete thickness at the duct." 2. While the 0.4 limit has been used in the past, ducts with a size equal to 0.5 times the least gross concrete thickness have been successfully used, provided that adequate cover is provided over reinforcement and ducts. Use of the current provisions will lead to thicker webs than may be necessary in some situations. This could reduce the economy of post-tensioned bridges. 3. Review current practice. If warranted based on the review of current practice, develop proposed revision to allow use of ducts with an area not exceeding 0.5 times the least gross concrete thickness of the member, with the following conditions: (1) In such instances it is extremely important to consider the effects during construction of high web shears and stresses resulting from post-tensioning tendon curvature prior to the ducts being grouted; (2) The web width must also be adequate to provide room for mild reinforcement to pass the duct and provide at least minimum cover. 4. Update the *Specifications* to current practice and relax a potentially restrictive requirement that may lead to less economical designs.	Medium	Low
28	5.5.1	1. The 2nd paragraph states: "Prestressed and partially prestressed concrete structural components shall be investigated for stresses and deformations for each stage that may be critical …" 2. While it is customary to consider all stages of prestressed concrete members, it may also be necessary to consider different stages of construction for reinforced concrete members, such as piers that may experience unbalanced loads or moments during erection of segments. The need to check temporary stresses and forces during construction is not so much a function of the type of member (e.g., prestressed, reinforced, etc.) as it is a function of the type of bridge and method of construction. Bridges that involve staged construction and intermediate support conditions that differ from the final structure configuration require additional scrutiny in the design process. 3. Develop proposed revision to make provision more general, applying to reinforced as well as prestressed concrete members, where necessary. 4. Make provisions more general and inclusive of conditions that may	Low	Low

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		need to be considered in design.		
29	5.5.4.2.1 5.5.4.2.2 8.16.1.2.2 9.14	1. These articles provide resistance factors for "Conventional Construction [Other than Segmentally Constructed Bridges]" and for "Segmental Construction". For conventional construction, resistance factors are reduced for shear and torsion, and for compression in anchorage zones if lightweight concrete is used. However the factor for flexure is not reduced. No values are given for sand-light weight concrete. For segmental construction, all factors for flexure, shear and joints are reduced when sand-lightweight concrete is used. No values are given for lightweight concrete. 2. The provisions in the two articles address lightweight concrete differently. Factors are reduced for conventional construction when *lightweight* concrete is used, but for segmental construction, factors are reduced when *sand-lightweight* concrete is used. No provisions are made for sand-light weight concrete with conventional construction or for lightweight concrete for segmental construction. The reduction factor for flexure for segmental construction is also reduced when sand-lightweight concrete is used, while there is no reduction in the factor for flexure for conventional construction. The potential for confusion also exists because of the definitions of the different types of concrete (see discussion under Article 5.2). In the *Standard Specifications*, Article 8.16.1.2.2 for reinforced concrete, no strength-reduction factors are modified for lightweight concrete. In Article 9.14 for prestressed concrete, only the strength-reduction factor for anchorage zones is reduced for use of lightweight concrete. For shear, the capacity equations are modified for lightweight concrete (Articles 8.16.6.2.4 and 9.20.2.5). 3. Evaluate the provisions for the different types of construction to make them consistent in their treatment of lightweight and sand-lightweight concrete. Request clarification from authors of the *Specifications* regarding justification for using reduced factors for lightweight concrete, especially where reductions were not made in the *Standard Specifications*. Based on the evaluation, develop proposed revisions. If a reduction is necessary, a sliding resistance factor based on unit weight, similar in concept to the factor used for the current definition of PPR, could be considered. 4. Clarify the use of resistance factors for reduced density concrete. Eliminate excessively restrictive provisions that may limit or discourage the use of lightweight concretes.	Medium	Medium
30	5.5.4.2.1 5.8.2.2 8.16.1.2.2 9.14	1. Article 5.5.4.2.1 provides resistance factors for "Conventional Construction [Other than Segmentally Constructed Bridges]". Article 5.8.2.2 provides modifications to shear capacity for lightweight concrete components. 2. As mentioned in the item above, when lightweight concrete is used, the resistance factors for shear for reinforced or prestressed concrete are not modified in the *Standard Specifications*. However, in the *LRFD Specifications*, the resistance factors for shear are significantly reduced (from 0.90 to 0.70) when lightweight concrete is used. The effect of combining the reduced reduction factors with the modifications to equations for computing shear capacity for lightweight concrete is not known. 3. Evaluate the impact of the combined modifications to resistance factors and shear resistance on designs for members using lightweight concrete. Compare results to *Standard Specifications* designs. Based on the evaluations, develop proposed revisions. 4. Prevent introduction of potentially restrictive provisions for	Medium	High

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		lightweight concrete as a result of decreasing reduction factors for lightweight concrete.		
31	5.5.4.2.1	1. These articles provide resistance factors for "Conventional Construction [Other than Segmentally Constructed Bridges]" and for "Segmental Construction". 2. The terms "Conventional Construction" and "Segmental Construction" are not defined. Therefore, there it is unclear which resistance factors should be used for the design of spliced multi-girder bridges. It appears that values for "Conventional Construction" should apply to superstructure sections between the joint/splice locations and the values for "Segmental Construction" should apply at joint/splice locations. 3. Develop proposed definitions for "Conventional Construction [Other than Segmental Construction]" and "Segmental Construction" to be added to Article 5.2. Evaluate the application of resistance factors for spliced girder bridges. This will be part of the larger reassessment of the concept of segmental construction that needs to be done with the *LRFD Specifications*. 4. Clarify applicability of sections of the *Specifications* related to spliced girder and segmental construction.	High	Low
32	5.5.4.2.1	1. The partial prestress ratio, PPR, is defined by Equation 2 as the ratio of total yield force of prestressed reinforcement to the total yield force of all tension reinforcement. 2. Using this definition, spliced girder segments that have mild tension reinforcement for handling would be considered as partially prestressed even though, in the completed structure, they would be designed to satisfy all allowable stress requirements for fully prestressed components. Additionally, negative moment regions of a bridge, in which mild reinforcement may also be in tension at service loads, would be partially prestressed. Technically, segmental structures (using the current definition where short segments are used) would also be partially prestressed except at the joints. This points out the misleading nature of the definition of partially prestressed concrete used in the *LRFD Specifications*. 3. Develop proposed revision for PPR to be based on a ratio of stiffnesses rather than forces, taking into account the reduction in stiffness if the partially prestressed member were to crack at service loads. 4. Improve the rational basis for the partial prestress ratio.	Low	Medium
33	5.7.3.5	1. This article contains provisions regarding moment redistribution "... at the internal supports of continuous reinforced concrete beams ..." when certain conditions are satisfied. It does not appear that there is an equivalent section in either Section 8 or 9 of the *Standard Specifications*. Article 4.6.4.1 indicates that the owner may permit moment redistribution. 2. The article as stated applies to reinforced concrete members. This appears to exclude its application to prestressed concrete members. However, it is possible that the intent of the provision is to consider bonded reinforcement provided at internal supports of "<u>continuously</u> reinforced concrete beams" instead of "<u>continuous</u> reinforced concrete beams." The intent would then be to include beams that are continuously reinforced (mild steel or prestressing) as opposed to just reinforced concrete beams that are continuous. Equivalent equations appear in ACI 318 (Articles 8.4.1 and 18.10.4.1), where moment redistribution is permitted for both reinforced and prestressed concrete, respectively. The articles in ACI only require that minimum reinforcement requirements be	Low	Low

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		satisfied for moment redistribution to be permitted.		
		3. Review ACI requirements and develop proposed revisions to this article to include prestressed concrete as appropriate. Consider adding portions of commentary from ACI 318 Articles 8.4 and 18.10.4.		
		4. Extend the application of moment redistribution to prestressed concrete members. Clarify application of the requirement by providing commentary.		
34	5.9.1.1	1. The 3rd paragraph restricts use of compressive stress limits to load combinations other than Service III load combination.	Low	
		2. It seems reasonable to use Service III load combination for computing compressive stresses. This is especially important for continuous bridges where compression at interior support may be the governing stress condition. *See discussion for Item 6, LRFD Article 3.4.1 above.*		
35	Tables 5.9.4.1.2-1 5.9.4.2.2-1	1. These tables define tensile stress limits for "Other than Segmentally Constructed Bridges" and "Segmentally Constructed Bridges".	High	Medium
		2. It is clear that the intended concept of segmental construction throughout the *Specifications* generally refers to short segment box girder structures. However, it appears that tensile stress limits from both categories may be applicable to the design of spliced girder bridges, depending on whether the design section is at or between splice locations.		
		3. Develop proposed revisions to clarify application of tensile stress limits to spliced girder construction. This will be part of the larger reassessment of the concept of segmental construction that needs to be done with the *LRFD Specifications*.		
		4. Clarify applicability of sections of the *Specifications* related to spliced girder construction.		
36	Table 5.9.4.2.1-1	1. Table 1 in this article specifies the concrete compressive stress limits at service limit state, after all losses. The 3rd bullet is a limiting stress for the condition of live load and one-half the sum of effective prestress and permanent load for other than segmentally constructed bridges. A limiting stress for the same condition is not given for segmentally constructed bridges.	High	Medium
		2. The limiting stress mentioned is intended to prevent fatigue in the concrete in compression. It would seem appropriate for the same condition to apply to all types of prestressed concrete construction, including segmentally constructed bridges. If the condition is not appropriate for segmental bridges at a particular location, for instance near a pier that was built in balanced cantilever, then the provision may not be applicable to a spliced girder bridge at a similar locations. It appears that all three conditions given in the table should be either applicable or not applicable to both types of construction.		
		3. Evaluate the effect of the tensile stress limit at the 3rd bullet for spliced girder bridges. Verify that the limiting stress given at the 3rd bullet is appropriate for spliced girder and segmental construction. Based on the results of the evaluation, develop proposed revisions to the Table.		
		4. Revise *Specifications* to include appropriate tensile limiting stresses for all types of prestressed concrete construction and simplify the presentation of limiting stresses.		
37	5.9.4.2.2	1. Table 1 in this article specifies the concrete tensile stress limits at service limit state, after all losses. The 4th bullet (1st for segmentally constructed bridges) ends with "... $0.5 f_y$; internal	Medium	Editorial

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		tendons". The 6th bullet (3rd for segmentally constructed bridges) is "Type B joints; external tendons". 2. The intended meaning of two bullet items mentioned above is unclear regarding internal and external tendons. Comparison with the similar Table 1 in Article 5.9.4.2.1 reveals a clearer description. The 4th bullet (1st for segmentally constructed bridges) of that table ends with "... 0.5 f_y with internal tendons or external tendons". The 6th bullet (3rd for segmentally constructed bridges) of that table is "Type B joints with external tendons". 3. Propose editorial revisions to make descriptions in Table 5.9.4.2.2-1 consistent with those in Table 5.9.4.2.1-1 for the bullets discussed above. 4. Clarify the intent of the *Specifications*.		
38	5.9.4.3	1. This article states that cracking in the precompressed tensile zone "may be permitted" for partially prestressed components. Limiting stresses for concrete in compression shall be as specified for fully prestressed components. 2. As discussed above in the item addressing the definition of the partial prestress ratio (PPR) in Article 5.5.4.2.1, some spliced girders will be classified as partially prestressed components if they have mild tension reinforcement. Therefore, under a strict interpretation of the *Specifications*, these members could be designed to crack under service loads. This is generally not desirable for prestressed concrete bridge members. 3. Consider revision of the definition of partial prestress, as discussed in the item for Article 5.5.4.2.1. 4. Clarify intent of the *Specifications* for members with both prestressing and mild tension reinforcement.	Low	Medium
39	5.9.5.3	1. The 4th paragraph states: "For members made from structural lightweight concrete, the values specified in Table 1 shall be increased by 5.0 KSI." 2. The term "structural lightweight concrete" has not been defined. It is therefore unclear whether this requirement applies to lightweight concrete (unit weight $\leq$ 0.120 KCF) and/or sand-lightweight concrete (no unit weight range currently defined - see discussion item for LRFD Article 5.2 regarding definitions). 3. Request clarification from authors of the *Specifications* regarding the type(s) of lightweight concrete to which this article applies. Base revisions on definitions previously established in the *Specifications*. 4. Clarify use of *Specifications*.	Low	Clarification
40	5.10.3.3 / 9.26.2&3	1. This section provides minimum spacing requirements for strands and tendons. New requirements are given for bundled strands and tendons (Articles 5.10.3.3.1 and 5.10.3.3.2). 2. There are several subarticles that contain provisions that are new or are somewhat different from the *Standard Specifications*. Articles may not cover all issues necessary for spliced girder construction, especially regarding vertically stacked tendons. The use of the terms "not curved" and "straight" in the articles does not appear consistent. A revision to Article 5.10.3.3.1 is already being considered by AASHTO Technical Committee T-10. 3. Review article to identify the changes from the *Standard Specifications*. Also review current practice and specifications. Based on reviews, develop proposed revisions, including clarification of terminology.	Medium	Medium

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1	*2*	*3*	*4*	*5*
		4. Ensure that the *Specifications* are clearly stated and consistent with current practice.		
41	5.10.3.3.2	1. The 3rd paragraph of this article gives minimum clear horizontal spacing between bundled post-tensioning ducts for "construction other than segmental". 2. There is not justification provided as to why this applies only to nonsegmental construction nor is there a similar spacing requirement for segmental construction, though the last paragraph gives allowable spacing for "precast construction". 3. Review article to identify the changes from the *Standard Specifications*. Also review current practice and specifications. Based on reviews, develop proposed revisions, including clarification of terminology. 4. Ensure that the *Specifications* are clearly stated and consistent with current practice.	Low	Low
42	5.10.3.5 9.27	1. This article contains provisions regarding post-tensioning tendon couplers. Requirements for minimum longitudinal spacing of coupler locations is given in terms of segment length and depth dimensions. 2. The *Standard Specifications* provide much more information than is given in the *LRFD Specifications*. Segment dimensions are related to conventional segmental construction, which means that no guidance is given for other types of bridges using tendon couplers. 3. Review the *LRFD* and *Standard Specifications* provisions. Review current practice. Determine whether any of the *Standard Specifications* provisions should be added to the *LRFD Specifications*. Develop proposed revisions as needed. 4. Include pertinent information from the *Standard Specifications* and current practice in the *LRFD Specifications*.	Low	Low
43	5.10.4	1. This article contains provisions regarding tendon confinement. 2. These provisions appear to be intended for use with box girders. It appears that they will be difficult to apply to spliced girders. It is also difficult to understand the details of the provision as worded. 3. Review articles and current practice. Develop proposed revisions that will apply to both box girders and spliced girders. Obtain or develop sketch to illustrate requirements of this article. 4. Clarify provisions so they can be applied to spliced girders.	Medium	Medium
44	5.13.2.2	1. This article specifies locations where diaphragms are required, such as at piers, abutments and hinge joints, and for box girder shapes on curved alignments. 2. Diaphragms are often used at spliced locations for spliced girder bridges. 3. Review current practice to determine whether diaphragms should be required, or at least recommended, at splice locations. Based on this review, develop proposed revisions. 4. Make designers of spliced girder bridges aware of the requirements for transverse diaphragms at splice locations.	Low	Low
45	C5.14.1.1	1. The commentary states: "This article [5.14.1 Beams and Girders] applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.14.2. There is a large variety of possible concrete superstructure systems, some of which may fall into either category." 2. While the *Specifications* recognize that there is "a large variety of	High	High

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		possible concrete superstructure systems," it offers no guidance to designers concerning what criteria should be applied to determine which provisions will govern a given structure.		
		3. Review Articles 5.14.1 and 5.14.2 regarding spliced girder construction. Develop proposed revisions to articles as required to clarify which provisions apply to spliced girder construction while preserving provisions that apply to either conventional girder construction or segmental construction.		
		4. Clarify applicable provisions for spliced girder construction.		
46	5.14.1.2.1 C5.14.1.2.1	1. This article indicates that the full spectrum of loadings and conditions needs to be considered in the design of precast beams. The 2nd paragraph states: "If critical to the safety of the component ... the locations of temporary braces and support points and their minimu m resistance and stiffness shall be indicated in the contract documents." The commentary indicates that the Contractor is generally responsible "... to provided adequate devices and methods for the safe storage, handling, erection, and temporary bracing of precast members. If ... limits need to be placed on the locations or magnitude of temporary supports or lateral restraints, these limits should be shown in the contract documents." 2. This article appears to place on the designer some responsibility for providing detailed information to the contractor in areas that have generally been left to the contractor. The concern here is how this should be interpreted by designers who do not know about the equipment or methods that will be used by the contractor or precast fabricator. Furthermore, the designer is directed to indicate the minimum resistance and stiffness of the required bracing on the contract documents. This would imply that the designer has knowledge of the contractor's means and methods that is generally unavailable during the design process. The designer's responsibility concerning such matters is to ensure the integrity of the permanent structure and protect the safety of the traveling public. While it may be necessary for him to specify a maximum reaction and location for the brace point on the permanent structure, it is not his responsibility to actually design such equipment. 3. Review article to identify changes from the Standard Specifications. Also review current practice and specifications. Based on reviews, develop proposed revisions, including clarification of terminology. 4. Ensure that the Specifications are clearly stated and consistent with current practice.	Medium	Low
47	5.14.1.2.6a	1. The 1st paragraph states: "Joints at internal piers in continuous construction shall be of the closure type." 2. This will generally be the case. However, there has been discussion of producing match-cast haunched pier segments in order to reduce shipping size and weight. This would not be permitted by the Specifications as currently worded. It seems unnecessary to require this type of joint for this specific detail. 3. Prepare proposed revision to move this statement to the commentary with permissive language. 4. Revise provisions that may restrict use of other viable girder splicing options.	Low	Editorial
48	5.14.1.2.6a	1. The 2nd paragraph states: "If the closure joint exceeds 6.0 IN, its compressive chord section shall be reinforced for confinement." The Commentary states: "The bottom flange in the internal pier	Low	Clarification

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		area acts nearly as a column, hence the requirement for confinement steel."		
		2. Closure joints are used at other than interior supports. At in-span splice locations, the top flange may be in compression. However, the compression stress at the splice may not be high compared to the limiting stress. Therefore, confinement reinforcement may not be necessary, and providing effective confinement reinforcement in the top flange of a girder may be difficult.		
		No guidance is given for the amount of reinforcement required. Significantly different quantities of reinforcement may be provided if confinement is determined using column requirements (Article 5.7.4) or prestressed girder confinement requirements (Article 5.10.10.2).		
		3. Request clarification from authors of the *Specifications* regarding the intended quantity of reinforcement to be provided to satisfy this requirement. Develop proposed revision to include appropriate provisions for spliced girders.		
		4. Provide information to enable designers to properly satisfy the requirements of the *Specifications* for spliced girders.		
49	5.14.1.2.6b	1. The 1st paragraph states: "Precast concrete beam segments ... may be made longitudinally continuous for both permanent and transient loads with combinations of posttensioning and reinforcement."		
		2. It is not clear whether the intent of this statement is that both post-tensioning and reinforcement are required to obtain continuity or if they are both required only if continuity is desired for both permanent and transient loads.	Low	Clarification
		3. Request clarification from authors of the *Specifications* regarding the intent of the article. Develop proposed revision to clearly state the intent of the provision.		
		4. Clarify application of the *Specifications*.		
50	5.14.1.2.6b C5.14.1.2.6b	1. The 2nd paragraph specifies the minimum widths of closure joints to be "... 12.0 IN when the joint is located in the span and 4.0 IN at an internal pier."		
		The commentary states: "The intent of the joint width requirement is to allow proper compaction of concrete in the joint. At internal piers, the diaphragms may incorporate the joint, thus facilitating compaction of concrete in a narrower space."		
		2. The setting of minimum joint widths seems to be unnecessarily prescriptive. The 12.0 IN minimum length requirement seems to contradict the 6.0 IN length and the discussion of prestressed beams in the previous article. The assertion that concrete may be properly consolidated in a 4 IN joint because it may be incorporated in a diaphragm is also questionable, especially for deep members. Diaphragms may also be used at in-span closure joints which, following the reasoning of the commentary, would make possible 4 IN in-span closure joints as well.	Low	Low
		3. Review current practice. Consider moving these to the commentary with permissive language. The mention of a 4 IN joint at interior piers should be reconsidered.		
		4. Eliminate potentially restrictive requirements and possibly inaccurate statements.		
51	5.14.1.2.8	1. This article contains requirements for longitudinal construction joints.	Low	Editorial
		2. This article appears redundant. Article 5.14.4.3 "Precast Deck Bridges" covers many of the same topics, although in some cases		

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		(see below) the requirements are not consistent. 3. Consider the combination of Articles 5.14.1.2.8 and 5.14.4.3. Alternatively, one article could refer to the other to eliminate duplication. 4. Eliminate unnecessary duplication in the *Specifications*.		
52	5.14.1.2.8	1. The 1st paragraph states: "Longitudinal construction joints between precast concrete flexural components shall consist of a key filled with a nonshrinkage mortar grout attaining a compressive strength of 5.0 KSI within 24 hours." 2. There are other concepts or materials that may be equally successful in achieving monolithic behavior. Other types of grout are also being used in shear keys, so it is questionable whether the type of grout should be specified. Furthermore, it appears unnecessary to specify the grout strength and the period of time in which the strength must be achieved. These quantities are generally determined by project requirements. 3. Eliminate this requirement or move it to the commentary with permissive language. 4. Remove unnecessarily restrictive requirements from the *Specifications*.	Low	Editorial
53	5.14.1.2.8	1. The 1st paragraph states the shear key "should not be less than 6.5 IN". 2. Justification is not given for requiring this minimum depth nor is a method given for determining a suitable depth. Some decked bulb-tee members have been successfully used in which the total flange thickness is only 6 IN. This article does not appear to be consistent with Articles 5.14.4.3.2 and 5.14.4.3.3c, which appear to apply to a similar condition, and state that the shear key should not be less than 7.0 IN. The article also uses permissive language "should" rather than "shall". 3. Review current practice. Resolve inconsistency between articles. Consider moving this requirement to the commentary and providing information regarding determination of required depth of shear key. 4. Resolve apparent inconsistency within the *Specifications* and provide guidance for designers.	Low	Clarification
54	5.14.1.2.8	1. The 2nd paragraph states: "If the components are posttensioned together transversely, the top flanges may be assumed to act as a monolithic slab". 2. This appears to require post-tensioning for monolithic behavior, while other articles in the *Specifications* indicate that a structural deck can also provide monolithic behavior. Other connection details not listed may also provide monolithic action. Details of construction must be carefully considered for the transverse post-tensioning to be effective. If the bearings and beams are relatively stiff, much of the transverse post-tensioning may go directly into the bearings and will not be effective in the deck. Other factors may also affect the amount of prestress that will actually cross the joint. It should also be noted that some states prefer to pull the beams together with post-tensioning prior to grouting the keyways. However, there is debate as to whether this is an effective procedure. 3. Develop proposed revision expanding options for monolithic action. A brief discussion of these items could be prepared for addition to the commentary. As mentioned in other items, the development of a complete solution to this issue, which includes	Low	Low

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		types of members not being considered in this study, is beyond the scope of this research project. 4. Provide additional information to designers to assist in properly designing transverse post-tensioning.		
55	5.14	1. The 3rd paragraph identifies two methods for determining the required amount of transverse prestress. It then states: "The transverse prestress after losses shall not be less than 0.250 KSI through the [shear] key." 2. The minimum required transverse prestress (0.250 KSI) is very high, corresponding roughly to one fully tensioned 0.5 IN diameter strand every two feet for a 6 IN deep shear key. This type of connection could be used for decked bulb tees. No justification is provided in the *Specifications* for this limit, nor is a reference provided for where it was developed. This article specifies that the transverse prestress shall be 0.250 KSI, while Article C4.6.2.2.1 only recommends this level of prestress. This conflict should be resolved. 3. Review current practice for decked bulb tees. Propose commentary statement to indicate other options. As mentioned in other items, the development of a complete solution to this issue, which includes types of members not being considered in this study, is beyond the scope of this research project. 4. Correct severe requirements for transverse prestress for adjacent members, including decked bulb tees.	Low	Low
56	C5.14.1.2.8	1. The 1st paragraph lists "precast box beams, T- and double-T sections" as the types of members to which the article applies. The article describes longitudinal construction joints in deck systems. 2. Decked bulb-tees are commonly used in some regions of the country as adjacent members but are not included in the list. 3. Revise list to include decked bulb-tees and any other sections possibly used for this type of construction. 4. Expand coverage of requirements to members in current use.	Low	Editorial
57	C5.14.1.2.8	1. The 2nd paragraph states: "The preferred joint is a simple v-joint." 2. The simple v-joint, indicated as the preferred joint type, is rarely used for this type of connection. It appears that other joint configurations may be more effective. The most commonly used types of shear keys are the ones detailed on the AASHTO box beam and voided slab standards and a diamond-shaped key that is used in the northwestern U.S. for decked bulb-tees. Joining of adjacent sections using weld plates is also a viable option for making the transverse connection between members. 3. Review current practice. Develop proposed revision considering current practice, including other options for making the transverse connection between members. 4. Revise commentary to reflect current practice.	Medium	Low
58	5.14.2.1 C5.14.2.1	1. The 2nd paragraph states the provisions "apply only to segmental construction using normal weight concrete." The commentary indicates that lightweight concrete is "infrequently used for segmental bridge construction" and that using it "represents a significant complication of both design and construction specifications." Therefore, it is not addressed in this article because of "this complication and questions concerning economic benefit ..." 2. Lightweight concrete is now being used for at least one significant	Medium	Medium

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		segmental bridge. While its use is not common for precast segmental bridges, it has been used on a number of cast-in-place segmental structures where the use of longer cantilevers necessitates a reduction in dead load moments at the piers. Reduced density concrete has also been used in a number of spliced girder bridges. The potential uncertainties and complications in using lightweight concrete can be overcome by appropriate design in order to realize the benefits of lightweight concrete. Limiting this section to only normal weight concrete appears to be in conflict with the different resistance factors for segmental construction given in Table 5.5.4.2.2-1 for both normal weight and sand-light weight concrete. This research project only has interest in this item as it relates to spliced girder bridges. 3. Review current practice. Review Article 5.14.2 to determine where modifications may be required to remove limitations on the use of lightweight concrete for segmental bridges. Develop proposed revisions to this article to reflect current practice. Effort on this item for this research project would be limited to spliced girder bridges. 4. Revise article to better reflect current practice regarding design of bridges using lightweight concrete so designers will not be discouraged from using it where appropriate.		
59	5.14.2.3.2 5.14.2.3.3 5.14.2.3.4	1. These articles give guidance concerning loads and load combinations to be applied during the construction of segmental concrete bridges. 2. The article addresses conventional segmental construction. Some or all of the provisions may not be relevant to spliced multi-girder bridges. 3. Verify the applicability of the referenced articles to the design of spliced multi-girder bridges. Develop proposed revisions to increase relevance for spliced girder bridges. 4. Ensure that proper guidance is provided concerning loads and load combinations to be considered during design of spliced girder bridges.	Medium	Low
60	5.14.2.3.8	1. This article addresses provisional ducts in segmental construction. 2. It does not appear that these requirements are intended to apply to post-tensioned girder bridges. Multi-girder bridges constructed from long prestressed concrete members offer a number of advantages over conventional segmental construction with regard to post-tensioning. The multiple beams provide redundancy and the post-tensioning forces are not concentrated in only two webs. Because tendon ducts are not made up of short (segment length) pieces connected at segment joints, tendon threading and stressing operations in multi-girder bridges generally have less friction losses. Finally, though an extreme measure, individual girders offer the potential to be replaced in the event of an emergency. Therefore, it appears possible that provisional tendons may not be necessary for post-tensioned girder bridges. 3. Review current practice. Develop proposed revision to address the issue of requiring additional strands or tendons in post-tensioned girder bridges. 4. Provide provisions that reflect current practice for post-tensioned girder bridges.	High	Medium
61	5.14.4.3	1. This article contains requirements for precast deck bridges. 2. It appears that this article needs significant revision. It is not clear how the different types of joints (Articles 5.14.4.3.2 and 5.14.4.3.3)	Low	Clarification

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		are to be used, where they apply or what they affect.		
		This section may have only limited application to extending span ranges of precast prestressed concrete bridges. Therefore, revision of article in this section may be outside the intended scope of this current research.		
		3. Review the article and its relationship to other articles. Develop proposed revisions as needed.		
		4. Clarify application of these requirements.		
62	5.14.4.3.1	1. The 2nd paragraph states: "Where structural concrete overlay is not provided, the minimum thickness of concrete shall be 5.5 IN at the top of round voided components and 7.0 IN for all other components."	Low	Low
		2. The specification of a minimum deck thickness may limit girder designs for decked bulb tee bridges. The minimum deck thickness should be governed by structural and cover requirements.		
		A revision to Article 5.14.4.3.1 is already being considered by AASHTO Technical Committee T-10. The revision addresses allowing the use of standard AASHTO-PCI voided slab and box beams, which do not satisfy these requirements.		
		3. Evaluate current practice and the article to determine the effect on decked bulb tees. Based on results of evaluation, develop proposed revision, if needed.		
		4. Allow use of appropriately designed and detailed bridge components.		
63	5.14.4.3.2	1. The 1st paragraph specifies a minimum shear key depth of 7.0 IN.	Low	Clarification
		2. This appears to be in conflict with Article 5.14.1.2.8 that specifies a minimum shear key depth of 6.5 IN for apparently the same conditions. In both cases the minimum shear key may be thicker than the deck thickness in decked bulb tee girders that have been used successfully with 6 IN decks.		
		3. Propose revisions to resolve the inconsistency between articles. Consider moving the statement on shear key depth to the commentary, with additional information if required.		
		4. Resolve apparent inconsistency within the *Specifications* and provide guidance for designers.		
64	5.14.4.3.2	1. The 2nd paragraph specifies the type of grout, its strength and time to gain the specified strength.	Low	Editorial
		See discussion for LRFD Specification Article 5.14.1.2.8 above.		
65	5.14.4.3c	1. This article specifies a minimum shear key depth of 7.0 IN	Low	Clarification
		See discussion for LRFD Specification Article 5.14.4.3.2 above.		
66	5.14.4.3c	1. This article specifies a minimum prestress of 0.250 KSI across the shear key.	Low	Medium
		See discussion for LRFD Specifications Article 5.14.1.2.8 above.		
67	5.14.4.3e	1. The article applies to precast deck bridges and states: "... The width of the longitudinal closure joint shall be large enough to accommodate development of reinforcement in the joint, but in no case shall the width of the joint be less than 12.0 IN."	Low	Editorial
		2. In most cases, this provision is reasonable. However, this requirement may restrict innovation in the use of closure joint connections by requiring the width of the joint width to be at least 12.0 IN.		
		3. Consider moving this statement to the commentary.		
		4. Eliminate a possibly restrictive requirement.		

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1	*2*	*3*	*4*	*5*
68	5.14.4.3f C5.14.4.3f	5. This article applies to precast deck bridges and states: "Where a structural overlay is used to qualify for improved load distribution …, the thickness of structural concrete overlay shall be 4.5 IN. An isotropic layer of reinforcement shall be provided in accordance with the requirements of Article 5.10.8 *[shrinkage and temperature reinforcement]*". The commentary states: "The composite structural overlay should be regarded as a structural component and should be designed and detailed accordingly." 6. No background is given for a 4.5 IN overlay being adequate to provide monolithic behavior in a bridge with adjacent members. No guidance is given for how to design the overlay as a structural component. It appears that the structural concrete overlay is intended to provide monolithic action in the absence of any other type of connection between units, such as transverse post-tensioning. 7. Review current practice. If information is available, develop proposed revision addressing the design of structural overlays to serve as a connection between adjacent members to provide monolithic action. Further study of this issue may be beyond the scope of this project. 8. Provide appropriate design requirements and procedures for structural concrete overlays to provide monolithic behavior.	Low	Medium
69	9.5.3	1. This article exempts "concrete decks … in multigirder applications" from investigation for fatigue. 2. It is not clear whether decked bulb tees would be included in this exemption for "multigirder applications." Since decks are not continuous for typical decked bulb tee construction and the decks are often relatively thin, it would appear that this exemption may not be appropriate for decked bulb tee girders. 3. Evaluate current practice regarding deck design for decked bulb tees to determine whether this may be a significant issue for this type of structure. 4. Clarify correct approach for consideration of fatigue in concrete decks on decked bulb tees.	Medium	Medium

APPENDIX F
Issues Requiring Additional Action

APPENDIX F
ISSUES REQUIRING ADDITIONAL ACTION

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ISSUES REQUIRING ADDITIONAL ACTION

1 Introduction

One of the tasks of this study was to identify other actions that are needed to maximize the potential for extending the span ranges of precast prestressed concrete girders. These include issues that currently impede the implementation and use of the design alternatives identified elsewhere in this study, both for spliced girder construction and for more conventional approaches for extending span ranges of precast prestressed concrete girders.

This appendix contains brief discussions on a number of issues that have been identified as requiring additional action to resolve. Where possible, recommendations on implementation are also given.

The issues are grouped in the following categories:

- General
- Specifications
- Design and Detailing
- Fabrication
- Transportation
- Construction

2 General

2.1 Clarify Definition of Segmental Construction

Segmental construction has generally become understood in the bridge engineering community to indicate box girder sections cast in relatively short segments that are then assembled into a completed structure with post-tensioning. Typically, joints between segments are match-cast.

Spliced girder construction, which is the focus of this study, may also be considered segmental construction according to the *LRFD Specifications*. This is demonstrated by the general definition of "Segmental Component" found in Article 5.2 of the *LRFD Specifications*:

A component made up of individual elements, either precast or cast-in-place, and posttensioned together to act as a monolithic unit under loads.

Provisions applicable for the design of spliced girder bridges are found in both Article 5.14.1 "Beams and Girders" and 5.14.2 "Segmental Construction." The *Specifications* recognize the possible application of provisions to both types of construction in the Commentary to Article 5.14.1.1, which states:

> This article [5.14.1 "Beams and Girders"] applies to linear elements, either partial or full span and either longitudinal or transverse. Segmental construction is covered in Article 5.14.2. There is a large variety of possible concrete superstructure system, some of which may fall into either category.

The first sentence of the above quote from the Commentary indicates that provisions of Article 5.14.1 "Beams and Girders" apply to spliced girders. However, the final sentence indicates that some members, including spliced girders, "... may fall into either category." Finally, Article 5.14.2.4.7 "Precast Segmental Beam Bridges," which clearly addresses spliced girder construction, is part of Article 5.14.2 on "Segmental Construction."

Design information for extended-span post-tensioned structures, spliced multi-girder bridges or precast segmental beam bridges is reasonably complete in the *LRFD Specifications* – in the hands of a knowledgeable and experienced designer. Unfortunately, because clear guidance about which provisions should be used for spliced girder construction is lacking, the applicable provisions must be sifted from articles addressing segmental bridges, beam bridges and other topics. Many of the segmental construction provisions clearly pertain only to conventional segmental box girder structures. Many of the provisions for the design of beam bridges are limited to conventional simple-span prestressed structures. A great deal of extrapolation and interpretation is required to apply all of these provisions to design of spliced girder bridges. The result is that many decisions are left to the designer's own discretion. Therefore, uniformity in design from one bridge to the next cannot be expected using the current *Specifications*.

It is interesting to note that in the 1970s, box girder and I-girder bridges assembled from pieces, whatever the length, were both considered segmental construction *(1)*. With the introduction of precast segmental box girder construction from Europe into this country around that same time, the term "segmental" became associated with short, match-cast box girder segments.

The question of terminology and nomenclature has been discussed in **Chapter 1** of this study. This issue must be addressed in both the *LRFD Specifications* and in the bridge community at large. Proposed revisions to the *LRFD Specifications* contained in **Appendix E1** of this report have been developed in an effort to ensure that the design of spliced girder bridges is adequately and clearly addressed by the *Specifications*. During the preparation of the proposed revisions, an effort has also been made to ensure that provisions addressing the design of conventional segmental construction has not been compromised.

2.2 Familiarize Owners, Designers and Contractors with Systems and Concepts

Whenever new concepts are introduced involving the design or construction of concrete bridges, time is required before the concept becomes generally accepted by owners, designers and contractors. While limited areas of the country have already used many of the methods for extending spans that are presented in this report, others are unfamiliar with the concepts and the benefits that may be available by implementing them. In areas not using particular methods, those involved in design and construction may also not be aware of their use in other parts of the country and may also be unaware of the resources that are available to assist them in implementing these concepts for design and construction. In some areas, equipment or support infrastructure may not be readily available to facilitate the implementation of the new concepts.

Because of the lack of familiarity and experience, owners, designers and contractors may be reluctant to proceed with new concepts. This has been reported in one state where an excellent opportunity for a spliced concrete girder bridge was lost because contractors were not comfortable with the construction method. In other states, post-tensioned alternates have been disallowed because the owner did not feel comfortable reviewing bridge designs using unfamiliar concepts. Finally, in most locations, new concepts must overcome the familiarity and comfort that those involved in the process have with the older, proven methods.

In certain parts of the country contractors are more experienced and familiar with complex methods of construction that utilize post-tensioning and joining of precast concrete members. An example is Florida, which leads the nation in the number of segmental box girder bridges that have been built and in the number of spliced girder bridges. As a result, owners, designers and contractors may prefer a spliced girder bridge over continuous steel bridges, or even segmental box girder bridges if the length of the long span unit is relatively short. Continued use of spliced girder construction leads to improvements in the technology and an increased level of comfort for all parties involved. As a result, the frequency of use may increase further. In other areas of the country, however, local contractors are

not experienced with spliced girder concepts and may not feel comfortable bidding a spliced girder alternative. Because the vast majority of bridges built using spliced girder methods tend to be relatively short, one to five spans, the project size is often not sufficient to attract larger contractors from outside the area even though they may already be experienced with construction involving post-tensioning and related operations.

Therefore, following completion of this research project, it is recommended that significant effort be invested by industry organizations and government agencies to promote the use of precast, prestressed concrete members for extended span bridges. It will be necessary to educate and expose owners, designers and contractors to the benefits of this method of construction, to demonstrate that other projects have been successfully completed using the methods, and to make them aware that design guidance and examples are available to assist in the implementation of these new concepts. It is also recommended that a resource contact or organization be established by the industry or by a government agency to provide information, review assistance, or guidance concerning extended span methods using precast, prestressed concrete girders. Standard design, construction and materials specifications from agencies with experience with the more complex designs should be used to relieve the concerns of owners whose staff may not be familiar with proposed alternate design concepts.

2.3   Promote Innovative Contracting Methods

In recent years, design/build projects have been used in several areas of the United States. Some states allow contractors to submit value engineering or contractor alternate proposals after receiving a project award. This may involve a proposal for an alternate design concept that will reduce the cost of the project or provide some other significant benefit. States may also consider contractor alternate designs at the time of bid. In many instances where these relatively new contracting methods are employed, innovative designs are developed to give the contractor a competitive edge. Many of the projects in which innovative methods were employed for extending span ranges that are identified in this report are the result of such methods. This is evidence that many contractors prefer prestressed concrete construction to other types of materials or construction methods, even when specialized methods and equipment are involved. It also points out the importance of design and construction specifications that encourage innovation for all project delivery methods: design/bid/build, design/build or other contractor-driven alternatives.

Therefore, it is recommended that bridge owners continue to pursue the use of innovative contracting methods. To encourage contractors to pursue these types of projects in the future, they must be educated regarding available construction methods and considerations for extending span ranges

using prestressed concrete girders. In states where local contractors are not experienced in construction of post-tensioned concrete structures and may not be familiar with the concepts involved, owners could offer incentives to contractors selecting the spliced girder bridge alternative. In the design/build selection process, owners could also reward teams that pair a local bridge contractor with a contractor proficient in complex construction methods. Fabricators of spliced girders should also be active in educating the contractors in their local area about the concepts and advantages of spliced girder construction.

2.4   Delineate Responsibility between Designer of Record and Construction Engineer

The design of conventional prestressed concrete girder bridges that began in this country in the 1950s generally required little consideration for the contractor's means or methods of construction. That changed with the introduction of precast segmental construction. With this type of construction, the final design is greatly influenced by the method and sequence of construction, so the increased complexity requires contractors to scrutinize the construction process in light of their available means and equipment. Designers of such bridges must not only design a finished structure to resist the design loads at service conditions but must also anticipate and propose a reasonable method for constructing the bridge. This is necessary so that contractors on design/bid/build projects may bid the project with reasonable expectations concerning its construction and so that it will behave as intended by the designer when construction is completed.

With increased complexity and more options concerning how a bridge is built, contractors have taken on an increasing level of design and detailing responsibility. This level of responsibility is increased even further if the successful contractor proposes an alternate design to better utilize his equipment and experience. As a result, it has become necessary for a contractor to hire a Specialty Engineer to perform redesigns or to produce design calculations and details to justify his proposed method of construction. This has become standard practice over the past thirty years on segmental bridge projects, so the delineation of responsibilities between the Engineer of Record and the Specialty Engineer for these projects implies reasonable expectations from either party and is generally spelled out in detailed project specifications.

For spliced girder bridge projects, however, the level of complexity is somewhere between that for segmental box girder bridges and that for conventional beam bridges. Because of this and because the method of construction has not yet been refined and standardized to the extent it has been for segmental box girder bridges, questions routinely arise concerning the responsibilities of the Engineer of Record and the Specialty Engineer for spliced girder bridges contracted out through the

design/bid/build process. There are also questions concerning how much design information and what level of details should be shown in the contract drawings.

These issues are significant and need further study. It is recommended that owners utilize the experience of segmental construction to clearly define the responsibilities between the Engineer of Record and the Specialty Engineer.

2.5   Promote Alternative Strategies for Increased Service Life of Concrete Decks

In many areas of the country, bridge decks have deteriorated as a result of corrosion of the reinforcement in the deck. Because this deterioration is so widespread in some areas, some agencies require that bridges be designed so that the deck can be removed and replaced prior to the end of the service life of the girders. For conventional bridges, this can be accomplished relatively easily but at a significant cost. However, for concrete girders that are post-tensioned after the deck has been placed, removal and replacement of the deck may not be possible without overstressing the girders.

Other acceptable approaches to extending the design life of the concrete deck have been proposed and used on a number of projects. Some of these approaches have been developed for use with conventional segmental construction, where deck removal is also not generally feasible.

Therefore, it is recommended that the viability of alternative approaches to improving deck durability be assessed and the performance of such strategies on past and current projects be monitored. This information should then be disseminated so that other agencies will be able to implement strategies to encourage construction of bridges using these alternatives, which may include wearing surfaces, integral decks or prestressed decks. This topic is discussed further in **Chapter 2** of this study.

3   Specifications

3.1   Revise the *LRFD Specifications*

While proposed revisions to the *LRFD Specifications* are included in this report, there remain some complex issues in the *Specifications* that cannot be resolved as part of this current research project. A more comprehensive revision of the *Specifications* is needed to provide a consistent framework that clearly addresses the design of conventional precast prestressed girders, spliced girders and segmental construction.

It appears that it may be necessary for industry or AASHTO to engage consultants to expedite the development of such a revision. Cooperation would be necessary between different interested industry parties to ensure appropriate treatment of design issues for all types of construction.

3.2 Produce Standard Specifications for Spliced Girder Construction

Specifications for segmental construction, post-tensioning, grouting and even explicitly for spliced girder construction have been developed for individual projects and as part of the construction standards in many states. To facilitate design and construction involving extended span concrete girder methods in regions where they are not presently being used, relevant specifications have been and will be collected that are currently available from a number of state highway departments and that have been used on past projects.

The availability of many specifications on the web has greatly enhanced the opportunity for sharing resources of this type between agencies. Therefore, this resource and contacts between agencies should be utilized as specifications are developed in states where none have previously existed.

3.3 Revise Deck Design Procedures for Continuous Bridges

In continuous members that are post-tensioned after the deck has been added, the composite cross section may be considered a prestressed member. As a result, tensile stress limits are generally applied to the deck at service limit state conditions as required by the *LRFD Specifications* because the member is considered fully prestressed (i.e., not partially prestressed). This may result in the requirement of additional prestress to satisfy the tensile stress limit in the deck. However, with other types of continuous bridges, such as steel plate girder structures, the stress in the deck is not limited at service limit state conditions. .

It appears that the *LRFD Specifications* may permit the deck at an interior support to be considered partially prestressed, since a significant fraction of the tension reinforcement is composed of the mild deck reinforcement. In this case, the deck may be allowed to crack at the service limit state and the limiting tensile stresses would not apply. Owners must carefully consider whether allowing cracking in the deck is desirable. However, requiring uncracked decks in prestressed concrete bridges while not requiring it in other types of bridges may significantly affect the comparative economy of different types of construction, so this issue should be considered when comparing design alternates.

An issue also exists regarding whether the analysis upon which the design is based should consider cracking in the deck at either or both the service and strength limit states. The current Specifications allow the effect of cracking in the deck to be neglected.

It is recommended that these issues be clarified in the *Specifications*. Further study is required to assess the use of the concept of partial prestressing for the design of decks of continuous girder bridges. It is also recommended that the inequality in deck design requirements should be reviewed by the appropriate AASHTO Technical Committees. Additional funded research or studies by owner agencies and industry may be required to reach a final resolution of these issues.

4   Design and Detailing

4.1   Make Available Preliminary Design Cost Data

A critical factor in the successful implementation of extended span girder methods in areas where there is no experience with this type of construction is the availability of relevant cost data. An attempt has been made during the preparation of this study to acquire and compile cost data in a meaningful manner. The results, which have been limited, are reported in **Chapter 2** of this report. Issues have been identified that have an impact on costs and guidance has been provided guidance in assessing their qualitative effect on the overall project cost.

It is recommended that comparative cost data continue to be collected after completion of this study and that it be made available to designers, owners and contractors interested in implementing extended span methods. This will be especially valuable in areas where spliced girders are just beginning to be used. Data for each project must be complete and contain adequate detail to be useful. Data on other conventional projects constructed in the same area at the same time should also be collected to provide a benchmark for comparing project costs so that the information can be better utilized for other areas of the country. The cost data for new projects will enable designers to make better construction cost estimates for subsequent projects.

4.2   Make Designers Aware of Local Hauling Restrictions for Girders

Designers need to be aware of hauling limitations in the state where the bridge is being designed, as well as in neighboring states in which the girders may be fabricated or through which the girders may be transported. This knowledge would allow designers to consider these limitations when laying out and designing the bridge, which may improve the economy of long-span bridges in some cases. Designers should also be aware that, when very large loads are being hauled, the availability of

equipment may be limited, which may have a significant impact on the cost and schedule for any movements. (See also **Section 6.1** "Hauling Limits on Highways").

By making designers and owners aware of this issue, it is hoped that designers will obtain this information from appropriate agencies when beginning a design utilizing extended span range bridges with long or heavy members. Fabricators are also a source of this type of information.

4.3 Streamline and Standardize Design Procedures and Details

In today's competitive economy, engineers must be able to quickly and accurately produce details for economical bridge designs. This is especially true because many spliced concrete girder bridges are the direct result of value engineering or design/build processes, which require speed, efficiency and economy in design as well as construction. This report is a resource to address this issue by providing design procedures, examples and details. It can also be used to identify additional resources for guidance and direction concerning issues and details not addressed in the report.

4.4 Inform Designers of Available Software and Design Aids

For designs utilizing enhanced or modified material properties or cross-sections, conventional design software can generally be utilized. However, for special structures, such as those involving post-tensioning, splicing and staged construction, more advanced software with special features is needed to automate the analysis and design processes. The availability and successful application of such software has a positive impact on designers and may encourage the use of similar construction methods on future design projects. For example, the availability of a simple DOT-produced program may have been one of the factors that led to the widespread use of cast-in-place, post-tensioned box girder bridges in California.

There has been progress in the development of software for the design of special concrete bridges. Several programs are now available with varying degrees of sophistication, specialization and cost, as described elsewhere in this report. However, there is currently no industry-preferred software product for design of spliced girder bridges. The software industry, owner agencies and the precast industry need to continue to pursue the development of easy-to-use and reliable software for the design of bridges utilizing specialized methods of extending span ranges. Designers also need to be very cautious as they evaluate new software products, especially when they are using it to design a more complex bridge type for the first time. In this case, designers must carefully review results and perform hand computations to verify the accuracy of the design.

4.5 Investigate Connections between Adjacent Decked Bulb Tees

There appears to be a significant potential for using decked bulb-tee girders, in many cases in conjunction with splicing, to significantly extend span ranges of precast, prestressed concrete girders. A major benefit of this type of bridge is speed of construction, since the deck is complete once the connection is made between adjacent girders. However, acceptance of this type of construction by owners and designers is essential for the expanded use of this type of girder. These parties must be convinced of the structural effectiveness and long-term durability of the longitudinal joint between adjacent girders. While such girders are currently used in several states, they are generally not used for main-line bridges. The cost and complexity of making an effective joint, both during girder fabrication and placement in the field, will strongly affect the viability of this concept.

Further study is required to study and promote the use of this type of girder section. This has been recognized by AASHTO which has provided funding for NCHRP Project 12-69 (FY 2004) "Implementation of Precast, Prestressed Concrete Long Span Decked Girder Bridges". It is anticipated that this research will clearly demonstrate the behavior of this type of bridge and will provide guidance in the design of the connection between adjacent decked bulb-tee girders.

5 Fabrication

5.1 Review Existing Tolerances

All construction requires tolerances on dimensions and other quantities in order to deal with the normal variations that occur during construction. Tolerances currently used for precast, prestressed concrete girders have been in place for a number of years. Since the time that they were established, girder lengths have increased, and the intent of this current study is to encourage the trend to continue toward longer members. As a result, there is concern that the current tolerances may not be adequate for future construction.

Therefore, it is recommended that owners and fabricators review existing tolerances for use with higher performance materials and establish limits that are reasonable for the new materials and span ranges. Design issues related to tolerances, such as minimum build-up requirements, distances between ends of girders, and locations of bearings from ends of beams, should also be reviewed.

5.2 Promote Understanding of Fabrication Issues

To promote wider use of extended span girder methods in other areas of the country, it would be beneficial to promote dialogue and information exchange between precasters in these areas

and precasters in areas where the extended span girder methods have already been used. This can be accomplished through interaction at PCI and other trade association meetings or technical association meetings. Fabricators can then be aware of the requirements and details for implementing these specialized methods and the potential benefits that can be realized. They can then be facilitators of the transition process to the implementation of the extended span girder methods.

However, for significant advances to be made in implementation of the extended span girder methods, designers must be made aware of the issues that affect precasters and thus have an impact on cost and practicality of particular construction practices. For example, a better understanding of how girders are fabricated and how forms can be modified to enable the casting of special girder sections could lead to better design decisions. Close cooperation between fabricators, owner agencies and designers should be promoted to facilitate the exchange of this information.

In the move to increase the span lengths of precast prestressed concrete girders, prestress forces and material strengths are increasing. This involves large forces being placed on limited areas at the ends of concrete members, resulting in high stress concentrations that are not easily analyzed. Industry and owners should be alert to any new conditions that may arise during fabrication of these girders that may indicate that girders are being overstressed, even though they may fully satisfy the requirements of the design specifications.

Wherever there is interest in use of extended span girder methods, steps should be taken to encourage the exchange of information between fabricators, owner agencies and designers, to encourage a better understanding of how the design and fabrication processes can best accommodate each other to decrease costs and improve the efficiency of extended span girder methods.

6 Transportation

6.1 Address Hauling Limits on Highways

This study has revealed that shipping weight and length restrictions can seriously affect the economy of precast, prestressed concrete girder bridges. Through informal communications during the course of this study, it was discovered that there is a large variation in hauling weight limits, with some states allowing girders up to 200,000 lbs to be shipped on the highways, while other states may limit the weight of a girder to approximately 130,000 lbs. Furthermore, additional requirements may complicate shipments, such as axle configurations (see **Chapter 2**). The limited availability of equipment for handling very large loads may have a significant impact on the cost and schedule.

Therefore, an initiative to encourage greater uniformity of hauling limits on highways is recommended, with relaxation of requirements wherever possible and reasonable. It is recognized that agencies have reasons for the limits that are currently in place, and that owner agency departments responsible for the weight limits are generally not the same department responsible for design. However, industry should demonstrate to agencies the benefits that may be available with long-span concrete girders. Industry should also make the agencies aware of the hauling equipment that is currently available for transporting long-span girders, since some restrictions may be based on older equipment.

7 Construction

7.1 Standardized Details for Various Spliced Girder Construction Components

As demonstrated by the data collected and presented in **Appendix A**, a large number of extended span prestressed concrete girder bridges have been designed and constructed in the past. A significant number of new structures are currently in the design or construction phase. This collection of bridges represents a wide array of precast girder types, structure configurations, construction details and erection methods. While all these bridges represent successful application of methods to expand span ranges, each job had its share of shortcomings and successes with regard to individual details and procedures.

As part of the data collection effort, an effort was made to obtain details featuring different aspects of construction used on past jobs in order to provide designers, owners and contractors the benefit of learning from other's past successes and shortcomings. This effort met with only limited success. Based on conversations with fabricators, field personnel, owner's representatives and designers during this study, some recommended details for spliced girders are provided in this report. It is recommended that industry and owner agencies continue to pursue the development and distribution of standard details for this type of construction.

7.2 Consider Access and Availability of Erection Equipment for Spliced Girders

Special equipment may be required for the erection of spliced girders because of the large size or weight of the girder segments. These include, but may not be limited to, large capacity cranes and erection trusses, depending on project parameters. A contractor's ability to pursue a spliced girder project may be influenced by his experience with different erection methods and the availability of required equipment. In some cases, the equipment may be available, but site constraints may prohibit use of the equipment without modifications to the site, the methods or the equipment. Designers must

be aware of available equipment and limitations of its use in order to properly consider constructability issues in the design.

This report cannot directly address issues regarding equipment and its availability for local contractors at a particular project site. However, all parties to a design must cooperate to properly assess equipment availability and limitations for a design to be successful. If designers are aware of the availability of specialized equipment, they may be able to incorporate locally available equipment into their design concepts. Fabricators and designers may also want to make contractors aware of specialized equipment that can be used or may be available from other areas of the country in order to facilitate a particular project.

7.3 Address Concerns About Quality Control and Durability of Post-Tensioned Tendons

Recent developments have revealed that a failure to follow proper procedures for detailing, grouting and inspecting post-tensioning tendons can lead to corrosion (2). These concerns have resulted in the reluctance or refusal to use post-tensioning by some bridge owners and designers.

This issue must be clearly and adequately addressed in order to allay these fears. A significant effort is underway in Florida to address these concerns (3). PTI has recently issued a guide specification for grouting of post-tensioned structures (4) and ASBI has also issued an "Interim Statement On Grouting Practices" (5). ASBI has also instituted a "Grouting Certification Program" for certification of grouting contractors and inspectors. New materials and details are also being developed for use in post-tensioning tendons, anchorages, ducts and grouts to enhance the corrosion resistance of the systems.

Significant progress has been made on this topic. To expand the use of post-tensioning, the results of studies and the availability of new procedures, standards and certifications must be widely distributed by owner agencies and industry. This topic is discussed further in **Chapter 2** of this study.

7.4 Supplier for Installation, Stressing and Grouting of Post-Tensioning Tendons

In some areas of the country, post-tensioned tendons are installed, stressed and grouted by the post-tensioning suppliers. In other areas, the fabricator of the precast concrete girders may provide the post-tensioning services. In yet other areas, contractors perform these activities, or they may hire independent specialty subcontractors to perform the services. In some cases, the provider of the post-tensioning materials and services may be a combination of firms with different responsibilities.

There are several issues associated with this situation. The first is how the owner can be assured of the qualifications of the party or parties that perform the post-tensioning operations. Another is how the owner can be assured of qualified inspection of the post-tensioning operations. If the owner cannot become comfortable with these issues, post-tensioned construction is not likely to proceed, and without the use of post-tensioning, significant increases in span ranges of precast, prestressed concrete bridges will be limited. Related to these questions is the issue of methods for testing ducts to determine whether grout-tight duct splices have been made.

Contractors must also be comfortable with the availability and responsiveness of the party providing post-tensioning services if the potential economy of extended span designs is to be realized.

These concerns must be addressed in several different ways. The qualification of those performing and inspecting post-tensioning operations is currently being addressed with the certification program instituted by ASBI. Other issues of procedures and inspection should also be identified and addressed. These issues may vary between agencies involved.

It is recommended that precast fabricators consider acquiring the equipment and expertise necessary to perform the post-tensioning operations for the contractor. This concept has been successfully implemented in some areas and may simplify the relationships involved. However, liability issues appear to limit the interest of fabricators in taking on this work. In areas where post-tensioning is not widely used, designers should investigate the local availability of qualified suppliers for post-tensioning services for the project.

8 References

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(3) Nickas, William N., "Temporary Design Bulletins C00-5 and C01-01 (Post-Tensioning Issues)." Memorandum dated November 6, 2001, Florida Department of Transportation < http://www11.myflorida.com/structures/posttensioning.htm> "Extension to C00-5 & C01-1: Midbay Bridge Post Tensioning Report (Nov 6, 2201)." (December 18, 2001)

(4) Post-Tensioning Institute, "Specification for Grouting of Post-Tensioned Structures." Phoenix, AZ (February 2001) 69 pp.

(5) American Segmental Bridge Institute, "Interim Statement on Grouting Practices." Phoenix, AZ (December 2000) 6 pp.

APPENDIX G
DESIGN SOFTWARE AND OTHER RESOURCES

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APPENDIX G
Design Software and Other Resources

It is recognized that owners and designers do not usually venture into new designs without having the proper design resources. For the design of more complex structures, these resources include design software, design examples, and other resources. Therefore, one of the tasks of this study was to identify existing software and other resources for the design of extended span bridges.

Appendix G1   Design Software and Other Resources

This appendix contains the three products from this part of the research study. They include a report on design software, a brief summary of other available design resources, and a summary of the bibliography for the entire study.

Appendix G2   Bibliography

This appendix contains the bibliography for the entire report including the appendices.

APPENDIX G1
DESIGN SOFTWARE AND OTHER RESOURCES

APPENDIX G1
DESIGN SOFTWARE AND OTHER RESOURCES

1   INTRODUCTION

It is recognized that owners and designers do not usually venture into new designs without having the proper design resources. For the design of more complex structures, these resources include design software, design examples and other resources. Therefore, one of the tasks of this study was to identify existing software and other resources for the design of extended span bridges.

This appendix contains the three products from this part of the research study. They include a report on design software, a brief summary of other available design resources, and a summary bibliography for the entire study.

2   DESIGN SOFTWARE

The following describes the factors that influence the selection of design software, a summary of information on software use that was collected along with the spliced girder project information that is reported in Appendix A, and a list of questions intended to assist designers when considering or evaluating the purchase of design software for more complex bridge designs.

2.1   Factors Influencing Software Selection

The availability and sophistication of computer programs to facilitate the design of post-tensioned, spliced girder bridges have increased significantly over the past ten to fifteen years, as evidenced by the wide variety of programs identified during the data collection phase of the project. Programs vary greatly in sophistication, specialization and cost, with cost generally being inversely proportional to specialization and sophistication.

The selection of appropriate analysis or design software is dependent on a number of factors, including:

- Degree of complexity of the structure being designed (i.e., simple span, continuous beam, integral frame, etc.)

- Construction staging (i.e., the number of post-tensioning stages; the number and degree of changes to static support conditions during construction; the degree of horizontal/rotational restraint in the final structure; etc.)

- Software presently available or licensed to the designer

- Potential for re-use of a particular software program

- Program capitalization costs versus design labor costs

2.2 Historical Usage

An inventory of software used to date for the design of spliced girder bridges indicates a long and varied list. There is no clear favorite indicated from the tally of respondents to the Data Collection Form. Earlier spliced girder bridge designers had less choice with regard to software selection since most of the time-dependent, stage construction programs were not commercially available before the early 1980s and many have only trickled into the market over the past decade.

Table G1-1 **Software usage frequency for post-tensioned, spliced girder bridge applications identified to date.**

Time-Dependent, Stage Construction		*FEM / Continuous Beam / Frame Analysis*	
IDS BD2 (PRESTO)	17	TDV RM	1
BRUCO	6	SAP90	3
ADAPT	8	SAP2000	4
TANGO	11	GT-STRUDL/STRUDL	4
LEAP CONSPLICE PT	5	wFRAME	1
SFRAME	1	P-FRAME	1
BC	1	LARSA	2
STDS	1		
Bridge Girder Design		*Spreadsheet / Manual Calculations*	
LEAP CONBEAM / BRIDGE	1	Excel, 1-2-3, others	18
LEAP SPAN	1	MathCAD	1
PS-GIRDER	2	Hand calculations	14
BDS	7		
FDOT Prestress Girder	2	*Unclassified*	
LEAP CONSPAN	2	CT	1
		T187	1
		BVN/STS	2
		PC-BRIDGE	2

The list of software used by design respondents is listed in **Table G1-1** along with the incidence of use reported. It is interesting to note that the majority of responses regarding software indicated the use of multiple programs, with spreadsheets being the most common supplemental program.

2.3 Software Considerations

Available tools for the design of post-tensioned, spliced girder bridges can be grouped into one of four categories, ranging from general, multi-purpose calculation hardware/software to design programs created specifically for the design of post-tensioned, spliced girder bridges. The four categories, in order of increasing specialization, include:

- Spreadsheet / Manual Calculations

- FEM, Continuous Beam or Frame Analysis Programs

- Bridge Girder Design Programs

- Time-Dependent, Stage Construction Programs

Programs in each category that were identified by respondents during the data collection process are given in **Table G1-1**. Other software may be appropriate, but in an effort to maintain objectivity, no attempt has been made to include a list of such programs.

When purchasing software for spliced girder bridge design, it is important to consider the flexibility of the program for future design applications. In general, capital costs increase as one moves from general purpose software (spreadsheets, beam analysis, etc.) to specialized design programs that account for post-tensioning, stage construction and time-dependent behavior. However, depending on the complexity of the structure, the increased capital costs can result in reduced labor costs during the design process. The less sophisticated the analysis and design tools used, the more intensive the labor process. Also, if the available analysis/design program does not have all the necessary features to perform the bridge design in its entirety, the designer may need to compensate for certain effects when modeling or perform additional calculations. With general purpose analysis software, it becomes necessary to produce supplemental computer, manual or spreadsheet calculations to perform the same level of analysis or design as the specialized design software.

Whether the reduced labor costs from using specialized software are sufficient to offset the cost of the software is dependent on the complexity of the structure being designed and the potential for using the software on more than one project. For a simple-span spliced girder bridge with a conventional construction sequence and post-tensioning only on the final composite section, purchase of

a specialized design program may not be warranted. The analysis and design procedures required for such a structure are within the capabilities of most designers armed with a calculator, spreadsheet software and a basic beam analysis program.

For a two-span continuous bridge with a constant superstructure depth and single-stage post-tensioning, the labor effort becomes somewhat more intensive. The post-tensioning tendon geometry becomes more complex, and secondary forces must be considered. It may still be cost-effective to analyze and design the bridge using conventional methods and software, but other factors may influence the final decision. The requirement for considering a nonlinear temperature gradient will increase the complexity of the analysis process further. (It should be recognized that a nonlinear temperature gradient will cause secondary forces even in a simple span structure.) The availability of an alternative structural analysis/design program already available in-house or potential for imminent selection on another spliced girder bridge job may also sway judgment concerning design tools or methods chosen for a particular project.

As the type of structure becomes even more complex, such as one involving multiple spans, haunched pier segments, temporary supports during construction or staged post-tensioning, it becomes necessary to consider more sophisticated software to account for stage construction, secondary effects and time-dependent behavior.

Some of the things that should be considered when purchasing software for the analysis and design of post-tensioned spliced girder bridges include:

1. Can the bridge be designed with presently owned/licensed software, or is it necessary to purchase new software?

    a. Is the software selected adequate to perform the analysis/design without requiring an excessive amount of labor for preprocessing, post-processing or supplemental calculations?

    b. Does the selected software only perform analysis or does it perform design as well? If the program performs design, what specifications does it use (e.g., *AASHTO Standard Specifications* (AASHTO STD), *AASHTO LRFD Specifications* (AASHTO LRFD))? Does it check service load stresses against allowable stresses and factored loads against ultimate capacities?

    c. If the cost of new software cannot be fully justified based on present needs alone, is there sufficient potential for future work to further justify its purchase?

2. Is the program input convenient and the output useful?

a. Does it automatically generate node and element numbering? Can the automatic numbering system be overridden by the user?

b. Does it allow: interactive input? graphical input? file-based input?

c. Does it have a library of common structural shapes (e.g., AASHTO girders, bulb tees, etc.) or allow user-defined shapes to be stored for future use?

d. Does it show a graphical representation of the model with node numbering, element numbering, support locations, load locations, etc.?

e. Does it output forces, stresses and displacements at all relevant construction and service stages?

f. Does it permit user-specified output results and formats?

g. Does it present both tabular and graphical results?

h. Can the output be exported to spreadsheets or other post-processing programs?

i. Is the program compatible with the user's Windows operating system platform (98/2000/NT/XP/2003)?

j. Is there sufficient documentation to use the program correctly and does the supplier have a past history of adequate technical support?

3. How does the program handle prestressing (pretensioning and post-tensioning)?

a. How is tendon geometry input? Is the process practical and convenient?

b. What tendon profiles are permitted (circular drape, parabolic drape, harp, general)?

c. Does the program allow different treatment of pretensioning strands and post-tensioning tendons? Does it print separate results for each?

d. How does the program output prestressing forces? Does it separate primary prestressing forces from secondary forces in the output? (This is necessary for strength limit states design where secondary effects are considered on the load side of the equation with a load factor of 1.0.)

4. What types of loads will the program analyze?

a. Does the program separate noncomposite and composite dead loads?

b. Does the program generate live load envelopes for both HS20 (AASHTO STD) and HL-93 (AASHTO LRFD) loads?

c. Are prestressing loads input using tendon geometry and force data or using equivalent loads? (See Question 3.)

d. Does the program analyze creep, shrinkage and temperature induced loads? (See Question 7.)

e. Does the program perform load combinations (e.g., AASHTO STD, AASHTO LRFD, user generated)?

5. How does the program model composite sections?

a. Does it accurately transfer forces and stresses between the noncomposite and composite sections?

b. Does it present stresses for top of slab, bottom of slab, top of girder and bottom of girder for all stages of construction and service?

c. Does it give girder deflections for all stages of construction?

6. What types of bridge configurations can be modeled with the program (i.e., limitations on structure types)?

a. Can the program be used for other han post-tensioned, spliced girder bridges (e.g., continuous prestressed girders, segmental bridges, continuous slabs, etc.)

b. Does the program only model superstructure elements (i.e., pin or roller connection to substructure; no modeling of piers)? If so, what boundary conditions are permitted at supports (e.g., partial fixity, springs w/only diagonal terms, springs w/diagonal and off-diagonal terms)? (This is an important consideration for bridges with integral substructure connections, as may be necessary in seismic regions, or with a double row of neoprene bearings at piers.)

c. Can the program model frame structures, such as those with integral substructure connections?

d. Can temporary supports be added to and removed from the structure?

7. What types of advanced analysis capabilities does the program have?

a. Does it permit stage construction with varying support conditions?

b. Can it model structure behavior resulting from creep, shrinkage, steel relaxation and prestress losses? What time-dependent models can be used (e.g., ACI 209, FIP-CEB, AASHTO LRFD)?

c. Will it readily accommodate effects due to temperature change (uniform, linear, nonlinear)?

8. Are there any additional features of the program that may be beneficial for present or future projects?

a. Does it interface with any other structural design, foundation design, geometry or CAD software?

b. Does it have nonlinear capabilities?

c. Does it perform any special analyses, such as those required for girder stability during shipping or deck replacement?

3   DESIGN RESOURCES

The following is a brief description of some of the reference material available to assist designers of spliced girder bridges. This information was collected during the early stages of the research and helped direct the development of the design resources included in this report.

3.1   Design Examples

During data collection efforts, the following existing design examples were identified that are related to extending spans using spliced girders:

- Article by Anderson (1973) *(1)*
- Concrete Technology Associates Bulletin (1973) *(2)*
- "Design Guide for Prestressed Concrete Long-Segment Bridge Girders" by Gamble (1979) *(3)*
- PCI "State of the Art Report" by Abdel-Karim and Tadros (1992) *(4)*
- Seminar at the International Bridge Conference in Pittsburgh, PA by Tadros (1996) (5)
- PCMAC Seminar by LoBuono Armstrong and Associates (1996) *(6)*
- Florida DOT Structures Design Conference by Aalami (1998) *(7)*

The fact that this list is relatively short, that several items are dated, and that several are focused on the design of spliced girders in a seismic region highlights the necessity for the design examples developed in this study.

3.2 Design Manuals, Standard Details and Specifications

Most states have bridge design manuals, standard details and specifications. Some have addressed the design and detailing of spliced girders. This information is very helpful, and is updated frequently. Therefore, the designer is encouraged to use such documents, several of which are now available on the Internet. These web-based resources are very useful for getting the most up-to-date information for design practices and details. It should be noted that some states use practices, details or procedures that are not widely accepted, so any of these resources must be used with caution when being applied to bridges in other states.

States and other organizations have developed reference material helpful for design, including design manuals, standard details and specifications which address spliced girders or related topics. Web sites for these states and organizations include:

- California   www.dot.ca.gov/hq/esc/techpubs/manual/bridgemanuals/ bridgemanuals.html *(8)*

- Florida   http://www.dot.state.fl.us/structures *(9)*

- New York State   www.dot.state.ny.us/structures/manuals.html *(10)*

- Oregon   www.odot.state.or.us/tsbbridgepub/ *(11)*

- Washington   www.wsdot.wa.gov/eesc/bridge/index.cfm *(12)*

- PCI New England   www.pcine.org/designtools.html *(13)*

3.3 Bibliography

A list of references that may be of value for the designer of spliced girder bridges is presented in the Bibliography, located in **Appendix G2**. These references include a wide range of publications from project descriptions to design aids and technical references, and includes a number of significant publications have been developed by industry organizations. The Bibliography includes all references cited in this report, plus many that were not cited, but which contain useful information related to extending span ranges of precast prestressed concrete girder bridges.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation