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16. Abstract The Texas Department of Transportation (TxDOT) is currently designing typical highway bridge structures as simply supported using standard precast, pretensioned girders. TxDOT is interested in developing additional economical design alternatives for longer span bridges, through the use of the continuous precast, pretensioned concrete bridge structures that use spliced girder technology. The objectives of this portion of the study are to evaluate the current state-of-the-art and practice relevant to continuous precast concrete girder bridges and recommend suitable continuity connections for use with typical Texas bridge girders. A wide variety of design and construction approaches are possible when making these precast concrete bridges continuous with longer spans. Continuity connection details used for precast, prestressed concrete girder bridges across the United States were investigated. Several methods were reviewed that have been used in the past to provide continuity and increase the span length of slab-on-girder prestressed concrete bridges. Construction issues that should be considered during the concept development and design stage are highlighted. Splice connections are categorized into distinct types. Advantages and disadvantages of each approach are discussed with a focus on construction and long-term serviceability. A preliminary design study was conducted to explore potential span lengths for continuous bridges using the current TxDOT precast girder sections, standard girder spacings and material properties. The revised provisions for spliced precast girders in the <i>AASHTO LRFD Bridge Design Specifications</i> (2010) were used in the study. The results obtained from the literature review and preliminary designs, along with precaster and contractor input, are summarized in this report.					
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**CONTINUOUS PRESTRESSED CONCRETE GIRDER BRIDGES
VOLUME 1: LITERATURE REVIEW AND PRELIMINARY DESIGNS**

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permits purposes. The engineer in charge was Mary Beth D. Hueste, Ph.D., P.E. (TX 89660).

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1. INTRODUCTION

1.1 BACKGROUND

Significant traffic and congestion across urban areas, as well as waterways, creates a demand for long-span bridges. The construction of these longer spans plays a critical role in the development of modern infrastructure due to safety, environmental, and economic reasons. A variety of bridge construction practices have been observed over the years. Planning, design and construction techniques are revised and refined to satisfy several parameters including feasibility, ease of construction, safety, maintainability, and economy. For over 60 years, precast, prestressed concrete girders have been used effectively in different states across the nation because of their durability, low life-cycle cost, and modularity, among other advantages. These girders are most commonly used for full length, simply supported bridges. However, there has been a growing need in the transportation sector to build longer spans with the readily available standard precast, prestressed concrete girder shapes.

The methods used in different states for extending span ranges with incremental variations in the materials and conventional design procedures often result in relatively small increases in span range for precast, prestressed concrete girders. Splicing technology facilitates construction of longer spans using standard length girder segments. A spliced girder system can provide a number of constructible design options by altering parameters such as span and segment lengths, depth of superstructure, and number and location of piers.

Most prestressed concrete slab-on-girder bridges are simply supported with precast, pretensioned girders and a cast-in-place (CIP) deck. Spans are limited to about 150 ft due to weight and length restrictions on transporting the precast girder units from the prestressing plant to the bridge site. Such bridge construction, while economical from an initial cost point-of-view, may become somewhat limiting when longer spans are needed. According to the available literature, a variety of methods have been used to extend the span range of concrete slab-on-girder bridges. These include the use of high performance materials and modified girder sections (Abdel-Karim and Tadros 1995). However, to significantly increase the span length, it is necessary to modify the layout and provide continuity connections between the spans.

Spliced girder bridge construction can provide a less complex solution compared to segmental concrete bridge girder construction by reducing the number of girder segments.

Spliced precast, prestressed concrete girders were recently found to be the preferred solutions of contractors, as observed in performance-based bids of projects in several states (Castrodale and White 2004). For these longer spans, continuity between the girder segments has the advantage of eliminating bridge deck joints, which leads to reduced maintenance costs and improved durability.

The performance and cost-effectiveness of a spliced girder system depends on the design and construction details. This involves a combination of the different design enhancements instead of applying them individually. The main challenges for designers, contractors, and fabricators are: (i) how to best provide prestressing considering transportation, erection and service loads, and (ii) how to best splice girders together to provide continuity. Naturally, these three facets of design, fabrication, and construction are inextricably connected. So, the challenge becomes: how to best extend bridge spans from, say, 150 ft to as much as 300 ft.

This report:

- Reviews some of the key techniques that have been used for spliced, continuous, precast concrete bridge girder systems.
- Discusses a number of construction considerations.
- Summarizes preliminary designs.
- Proposes a general framework for categorizing connection splice types.
- Reviews input from precasters and contractors.
- Provides some potential connection details.

1.2 SIGNIFICANCE

Bridges are a critical element of the transportation system and provide a link over urban congestion, waterways, valleys, etc. The capacity of individual bridges controls the volume and the weight of the traffic carried by the transportation system, and is also expensive at the same time. Therefore, it becomes necessary to achieve a balance between handling future traffic volume and load and the cost of a heavier and wider bridge structure. Economic, aesthetic, and environmental demands often result in the need for a longer span range, fewer girder lines and a minimum number of substructure units in the bridge system. Designers, fabricators, and contractors, upon successful collaboration, can take advantage of applying continuous construction to the standard precast, pretensioned girders developed by different states.

Continuity in precast, prestressed concrete girders provides another cost-effective, constructible and high performance alternative that can be used for longer spans that are often constructed with custom steel plate girders, steel box girders, and post-tensioned segmental girders. This research study will identify and investigate effective and economical options for continuity details for continuous precast concrete girder bridges. The long-term goal of this project is to develop and recommend standard design procedures for this type of bridge system to be used throughout Texas for any prospective long-span bridge projects.

1.3 OBJECTIVES AND SCOPE

The major goal of this research project is to review, validate, and recommend details for the design of durable and constructible details to achieve structural continuity between the standard precast, prestressed concrete girder sections used in Texas. Additional goals are to obtain longer span-to-depth ratios and greater economy with the consideration of superimposed dead loads and live loads. The objectives of this study are:

- Review and document the various alternatives for the design and construction of continuous precast, prestressed concrete bridge girders.
- Identify the continuity connection technology that has the potential to extend span lengths providing a simple, constructible, and cost-effective solution.
- Validate the most appropriate splicing details and suitable construction procedure.
- Perform preliminary design for initial evaluation of benefits of continuous bridge girders.
- Recommend continuity splice details and specifications and identify limitations.

This study focuses on Tx70 and Texas U54 prestressed concrete bridge girders, which are precast sections widely used in Texas.

1.4 RESEARCH PLAN

The outcome of this research study will support TxDOT's implementation of continuous precast, prestressed concrete bridge girders to achieve longer span-to-depth ratios with greater economy than currently possible with simple spans. The following tasks were performed to accomplish the objectives of Phase 1 of this research study.

1.4.1 Review Literature and State-of-the-Practice

The research team compiled a comprehensive literature review of the state-of-the-art and state-of-the-practice related to continuous precast, prestressed concrete girders using the standard girder shapes developed by different state DOTs. Many states have used different techniques and approaches to extend span ranges with variations in the design enhancements and material properties. From review of the state-of-the-practice, it was found that the girder segment size is controlled by the hauling limitations and type of lifting equipment available. The current state-of-the-art and practice illustrated that in-span spliced girder technology has the greatest potential to extend the span range of simple spans. This technology facilitated wider spacing between girder lines, minimum number of substructure units, and adoption of conventional construction procedures on site. Application of continuous construction using splicing of standard precast, prestressed girders presented a cost-competitive, constructible, and high-performance alternative to steel plate or steel box girder solutions for longer spans up to 280 ft. Selection of the construction method and type of splice detail depended on the terrain, available equipment, and experience of the local contractors. Findings from the review indicated that designers, fabricators, and contractors with successful collaboration from the planning stages of bridge details can take the advantage of the most cost-effective use of personnel, equipment, and materials.

1.4.2 Preliminary Designs

Preliminary designs were developed to carry out an initial evaluation of the design details with regard to construction and implementation for use with the continuous precast, pretensioned girders. The research team considered the most promising options reviewed in Task 1.1. The focus of this study was Tx70 and Texas U54 prestressed girder bridges. The research team gathered input and suggestions from TxDOT related to consideration of the girder type and sizes, girder spacing, material properties, etc. to ensure that they are representative of typical bridges in Texas. The concrete strengths at service and at release were limited to values commonly available from Texas precasters. The girder segment length and girder spacing are dictated by TxDOT practice. The research team evaluated different design considerations to determine their impact on the final design loads and thermal effects. The potential key design constraints evaluated were deflection, shear demand on thin webs considering post-tensioning ducts,

moment demand and ultimate strength, flexure-shear interaction at supports, and serviceability stresses under live load and thermal gradient effects. The results of the preliminary designs helped to determine the maximum feasible spans that can be achieved using the standard TxDOT girders. Several design issues were identified and resolved using suitable recommendations that the research team provided. The results indicated that based on the above considerations, it may be possible to nearly double the span length of the standard Texas prestressed concrete girder bridges using drop-in and over-pier girder segments with in-span splice connections.

The research team proposed preliminary details for the splice connections. Results of the review indicated that the use of in-span splices to make precast, prestressed concrete bridge girders continuous presents a cost-competitive alternative for increasing span lengths using standard precast girder sections. This system was found to fill the gap between 150 ft precast, pretensioned concrete bridges made continuous at the pier for live loads and the 300 ft continuous, post-tensioned concrete segmental box girder bridges. Based on the review of different splice connection details used in the past to provide continuity, the splice details can be classified as fully prestressed, partially prestressed, and fully reinforced connections. The research team has discussed the advantages and disadvantages of each approach in this report, with focus on construction and long-term serviceability.

1.4.3 Focus Group Meetings

The research team held focus group meetings to present findings from Tasks 1.1 and 1.2 and solicited input regarding potential implementation of various continuity details. Three separate meetings were held with TxDOT engineers, precasters, and contractors. The research team developed questionnaires for Texas precasters and contractors, with input from the TxDOT Project Monitoring Committee (PMC), to collect feedback on the preliminary design and details developed in Task 1.2. In addition, information related to the preliminary details of the proposed splice connections was distributed to the precasters and contractors. The information and questionnaires included four connection styles for in-span splices of standard TX girders and specific feedback was requested on the connection types, as well as other considerations related to design, precasting, shipping, and construction. The precasters provided guidance related to the most economical and reliable details for precasting and hauling operations. The contractors

provided input that helped to integrate the construction considerations with the preliminary continuity design details and identify potential issues along with suggestions for improvement.

1.4.4 Prepare Phase 1 Research Report

The results of the above tasks are summarized in this report. Several areas requiring further study were also identified based on the detailed preliminary designs. The research team held focus group meetings with TxDOT engineers, as well as the precasters and contractors from the industry, to discuss the results and suggestions related to the design and construction benefits and issues of the proposed preliminary continuity details. This helped to narrow down the specific requirements of the different organizations such as design, fabrication, transportation, and erection and construction on the site. Recommendations from Phase 1 of this project will focus on specific pretensioned girder shapes and continuity splice details to be investigated in the experimental study that will be a part of Phase 2 of the project. A summary of the spliced prestressed concrete girder bridges, continuity designs using standard TX girder sections, and critical design issues and recommendations for Phase 2 are documented in this report.

1.5 OUTLINE

Chapter 1 provides an introduction to this research project. Chapter 2 includes a comprehensive literature review of continuous precast, prestressed concrete girder bridges built in the United States. It also highlights issues in the widespread use of spliced girder technology. Chapter 3 outlines the preliminary designs developed for continuous spliced precast, prestressed concrete girders. Chapters 4 and 5 present the results and findings from the preliminary designs conducted for Tx70 and Texas U54 girders, respectively. Chapter 6 discusses several design issues that were identified in the preliminary design stage of continuous prestressed concrete girders and recommendations provided by the research team. Chapter 7 presents the preliminary continuity splice connection details used for precast, prestressed concrete girder bridges along with the advantages and disadvantages of each splice connection type and approach. Chapter 8 gives the industry feedback from the precasters and contractors on the preliminary design and details with focus on potential implementation of the promising continuity details for precast, pretensioned girders made continuous. Chapter 9 provides the summary of Phase 1 of the project with conclusions and recommendations to be considered in finalizing the work plan for Phase 2.

2. LITERATURE REVIEW

2.1 BACKGROUND

Splicing technology facilitates construction of longer spans using standard length girder segments. A spliced girder system can provide a number of constructible design options by altering parameters such as span and segment lengths, depth of superstructure, and number and location of piers. The standard I-shape and bulb-tee precast concrete girder sections designed and fabricated in lengths up to 160 ft constitute approximately one-third of the bridges built in the United States (Castrodale and White 2004). The use of precast, prestressed concrete girders has facilitated the use of long-span girder segments that can be efficiently hauled and constructed, and presents a cost-effective solution with good serviceability and minimal maintenance. The application of prestressing to bridges has grown rapidly and steadily, beginning in 1949 with high-strength steel wires in the Walnut Lane Bridge in Philadelphia, Pennsylvania. From 1950 to the early 1990s, the count of prestressed concrete bridges surpassed 50 percent of all bridges built in the United States. Prestressing has facilitated the span capability of concrete bridges. By the late 1990s, spliced-girder spans reached a record 320 ft.

Over the years, the development of materials, section properties and fabrication technology coupled with improved methods for transportation and erection have helped to increase the span of single girders extending over the whole span up to 160 ft. Where it became necessary to eliminate intermediate substructure units, special techniques were used to extend spans up to 300 ft. The post-tensioning method of prestressing is one of the commonly used methods for bridge structures with long spans and unusual layouts. Investigation of the different methodologies for providing continuity employing standard precast, prestressed concrete girders is necessary to construct an economical and structurally efficient bridge system. A combination of post-tensioning with splicing of girders presents attributes of high performance and feasible construction. Implementation of splicing technology has the potential to extend the simple spans by approximately 50 percent and at the same time presents a simple and cost-effective solution (Castrodale and White 2004).

The proposed research will aid in sharing knowledge of the current state-of-the-art and practices for the use of precast, pretensioned girders made continuous. This study will help to

draw attention to the benefits, as well as the shortcomings, of various connection details that can be used to achieve continuity.

2.2 ON-PIER SPLICING WITH CONTINUITY DIAPHRAGM

Table 2.1 provides a summary of on-pier splicing details, which have been used for continuous precast, prestressed concrete girders. Additional details are provided below.

2.2.1 Non-Prestressed Design Options

2.2.1.1 Conventional Deck Reinforcement

Kaar et al. (1960) investigated the development of continuity in precast, prestressed concrete bridge girders used in conventional designs for extending span lengths. The conventional design used deformed reinforcement in the CIP deck slab over the girders to provide continuity designed for resisting the live loads. Kaar et al. (1960) carried out tests on the connection detail where the deformed rebar in the deck slab is made continuous over the supports and resists the negative bending moment. This detail also included the use of a diaphragm over the piers extending laterally between the girders on either side. The width of the diaphragms was greater than the spacing between the ends of the girders, which helped to provide lateral restraint to strengthen the concrete in compression. The results from this study found that this continuity connection detail was desirable as it permits sufficient redistribution of moment and is simple to construct and relatively economical.

Mattock and Kaar (1960) carried out additional tests on the continuity connection for precast, prestressed concrete bridge concrete girders with introduction of details for resisting the positive moments resulting from creep and shrinkage. They conducted static and dynamic load tests on half-scale component specimens of a two-span continuous connection between girders with CIP deck and diaphragm. The results from the static tests confirmed the results determined by Kaar et al. (1960). From the dynamic test using repeated pulsating loads applied to the free ends of the girders, the researchers found that the connection can potentially resist an indefinite number of applications of design loads without failure. However, the width of the cracks and the resulting flexibility of the connection were found to increase. They tested two connection details for positive moment resistance: (i) fillet welding the projecting ends of the reinforcement bars to a structural steel angle, and (ii) bending the projecting ends of the reinforcement to form right

angle hooks and lapping them with the longitudinal diaphragm reinforcement. Results from this test showed that the performance of the welded detail was satisfactory compared to the hooked detail both at service load and ultimate strength with careful attention to the welding. Brittle fractures in the reinforcing bars were observed in the hooked detail. It was suggested to use an inside radius of the hook larger than the bar diameter and a minimum distance of 12 times bar diameter from the edge of the precast member to the inside face of the hook to develop the yield strength of the reinforcement bars.

2.2.1.2 Positive Moment Connections

Oesterle et al. (1989) presented a research study through NCHRP Report 322 on the development of procedures to compute design moments in precast, prestressed bridge girders made continuous through the continuity connection in the CIP deck slabs and diaphragms at bridge piers. Experimental investigations of concrete creep and shrinkage for the continuous bridges were included to evaluate time-dependent material behavior as a part of the analytical study. The test results indicated that it is difficult to overcome the positive moment cracking without the presence of pre-compression of the splice due to positive thermal gradients. The uncertainties in the design of the continuity connections that were addressed in this research study include the prediction of elastic, inelastic, time-dependent, and ultimate positive and negative moments at the location of the connection. For this study, information on the current state-of-the-practice was extracted from literature review and a survey of state DOTs, bridge designers, and precasters. Some of the results of the questionnaire indicated that the decision to reduce the midspan moments due to the negative moment continuity effects does not appear to be related to whether or not the positive moment reinforcement is present at the pier connection. The positive moment reinforcement detail typically included either embedded bent bars or extended prestressed strands. Common problems associated with continuous precast, prestressed concrete girder bridges discovered from this survey include:

- Poor fit of the positive moment reinforcement requiring field adjustment.
- Incorrect placement of reinforcement and prestressing strands.
- Transverse cracking of the deck in the negative moment region.
- Excessive girder camber leading to adjustment of the profile grade.
- Incorrect construction sequence.

- Cracking of the diaphragms at support due to long-term creep and shrinkage.
- Cracking and spalling of diaphragms in cases where diaphragms were cast before the deck.
- Spalling of the piers and abutments caused by improper girder location or inadequate details for the girder seats.
- Movement of the girders when deck concrete was poured before the diaphragms.

In addition to these common problems, individual respondents listed issues such as brittle fracture of the bent reinforcement bars during placement of the girders, corrosion of the deck reinforcement after cracking, long-term girder movements leading to opening of expansion joints, and difficulty in replacement of these girders.

Mirmiran et al. (2001b) conducted a research study on positive moment cracking in the diaphragms of simple-span prestressed girders made continuous. This study was aimed at investigating precast bridge girders that can be made continuous for live loads by providing a moment connection over the supports. The researchers achieved this by placing negative moment reinforcement in a CIP deck over the support and by placing a diaphragm between the girder ends. The study also recommended that “a minimum amount of positive moment reinforcement equivalent to $1.2M_{cr}$ ” should be used to limit the crack width in the diaphragm and to avoid significant loss of continuity, where M_{cr} is the cracking moment of the diaphragm section.

Mirmiran et al. (2001b) found that bridges made continuous for live load can be successfully built using either bent strand or bent bar positive moment connections. Bent strand connections were easy to construct as the strand was flexible enough to move during assembly. However, these connections were found to fail by gradual pullout of the strand. Bent bar connections were more difficult to construct than bent strand connections. Embedding the bar in the end of the girders caused additional congestion in an already congested area. Embedding the girder ends in the diaphragm seemed to improve the connection capacity, but the effect was difficult to quantify. Placing additional stirrups in the diaphragm just outside of the bottom flange of the girder did not increase connection strength but did increase ductility. Use of horizontal bars through the web increased the connection strength, but at failure the girder webs cracked. Expansion and contraction of the deck caused by heat of hydration significantly affected the reactions and stresses in the girders.

Miller et al. (2004) presented a research study through NCHRP Report 519 on the connection of simple span precast concrete girders for continuity. This project report conducted a survey of the commonly used continuity connections for prestressed girders in different states. This survey was carried out to investigate the type of negative and positive moment connection at the support, the age at which continuity is established, design techniques, and construction sequence and issues. Six positive moment connection details were selected and developed for the experimental tests (see Figure 2.1). The connections details included:

- Extended mild steel bars.
- Extended prestressing strand.
- Extended bar with the girder ends embedded into the diaphragm.
- Extended strand with the girder ends embedded into the diaphragm.
- Extended bars with the girder ends embedded into the diaphragm with additional stirrups near the bottom of the girder.
- Extended strand with girder ends embedded into the diaphragm with horizontal bars placed through the web of the girder.

All six details were designed for $1.2 M_{cr}$ (composite girder cracking moment). The results of the test showed that all the details achieved the design cracking moment, and the last two details listed displayed additional ductility. The crack width due to positive moment loading in the prestressed strand connection was seven times larger than that in the bent bar connection. Also, the continuity loading showed that the bent strand connection was only 70 percent effective for continuity after positive moment loading and the resulting cracking had occurred at the connection. In general, the bent bar connection detail had sound structural performance over the strand connection. The important conclusion of this study was that even though the thermal loading did not reduce the strength of the continuity connection in the laboratory tests, repeated thermal effects in real conditions could create serviceability issues over a longer period of time.

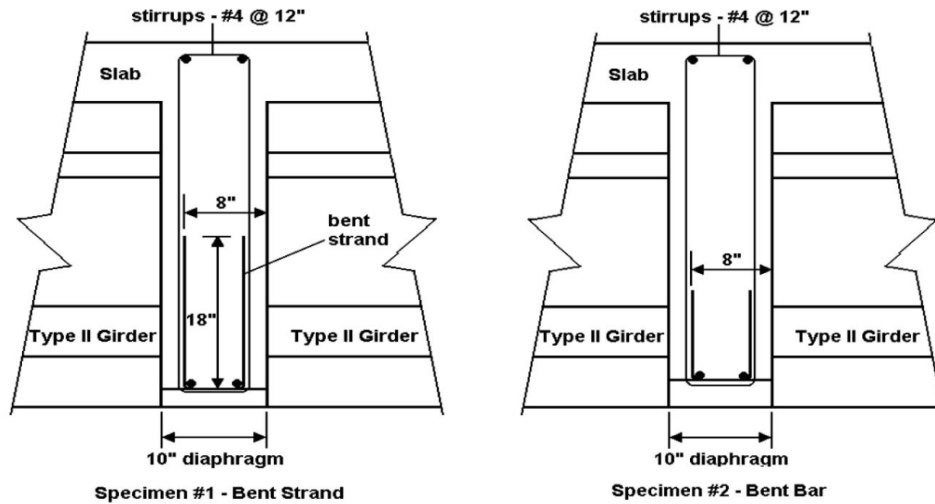


Figure 2.1. Positive Moment Connection Details for Prestressed Girders (Miller et al. 2004).

Newhouse et al. (2005) carried out a study on continuity connections over the support at Virginia Polytechnic and State University. The goal of this research was to recommend appropriate continuity details for the precast concrete bulb-tee (PCBT) girder sections. They developed and tested three continuity details using PCBT-45 girder sections. The first two continuity details consisted of a full continuity diaphragm with a CIP deck. Test 1 was carried out on specimens with prestressing strands extending out from the ends of the girders and bent to form a 90-degree hook. Test 2 involved specimens with #6 U bars bent into a 180-degree hook extending out from the bottom of the girders (see Figure 2.2). Test 3 was carried out on a third continuity connection detail that consisted of the slab only, which was cast continuous over the girders. The spacing between end faces of the adjacent girders was 12 in., 13 in., and 3 in. for Tests 1, 2, and 3, respectively.

Newhouse et al. (2005) found that the Test 2 specimen with 180-degree bent U bars was slightly stiffer with very small crack openings at the bottom interface as compared to the Test 1 specimen under static and dynamic loads. The results from this investigation showed that the thermal restraint moments were more significant than the restraint moments due to creep and shrinkage. Based on this study, it was suggested to design the girders as simple spans for dead and live loads for service conditions, and to assume a fully continuous system for ultimate strength conditions.

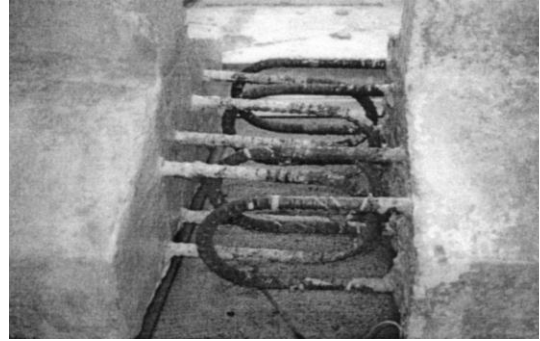
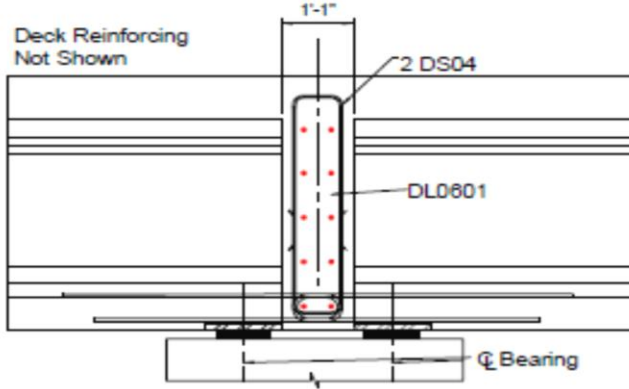


Figure 2.2. U Bars Bent into a 180-Degree Hook Extending out from the Face of Girders (Newhouse et al. 2005).

2.2.1.3 High Strength Threaded Rods

At the University of Nebraska, Tadros (2007) developed a threaded rod continuity system for precast concrete I-girders that was based on further refinement of his research study in 1998. This continuity detail used 1-3/8 in. high strength (150 ksi) threaded bars embedded in the top flange of the girder and connected using steel block and nuts. After the continuity diaphragm is cast, the bolts are tightened into position. The author noted that a major advantage of this system is that it can achieve continuity not only for live load and superimposed dead load, but also for the dead load of the slab. This added continuity can reduce the number of strands in the girders. Moreover, this connection was promoted as being relatively simple to construct. A notable span-to-depth ratio of 36 from this threaded rod spliced system can be achieved by using it in combination with a splice haunch block on the piers. The longest spans achieved using these arrangements were 148 ft and 151 ft on a four span unit employing 50 in. deep NU 1100 I-girders. No post-tensioning is required for this system. One possible problem with this design is that the bulky steel hardware may aggravate the reinforcement congestion in the diaphragm.

Sun (2004) further refined and investigated the threaded rod system first developed at the University of Nebraska. The high strength threaded rod system used in this study is shown in Figure 2.3. Two systems were tested under this study: (i) using high strength bars in line and cross-connecting with high strength threaded rods or transverse rebar, and (ii) using high strength bars in line and welding transverse bars to longitudinal 50 ksi straps in the form of an open box

member. The major advantage of this system is that the high strength bars are connected before casting of the deck slab and therefore are subjected to permanent negative moment at the support on application of the deck load. This eliminates the cracking of the bottom flange of the girders due to the positive thermal gradient effects.

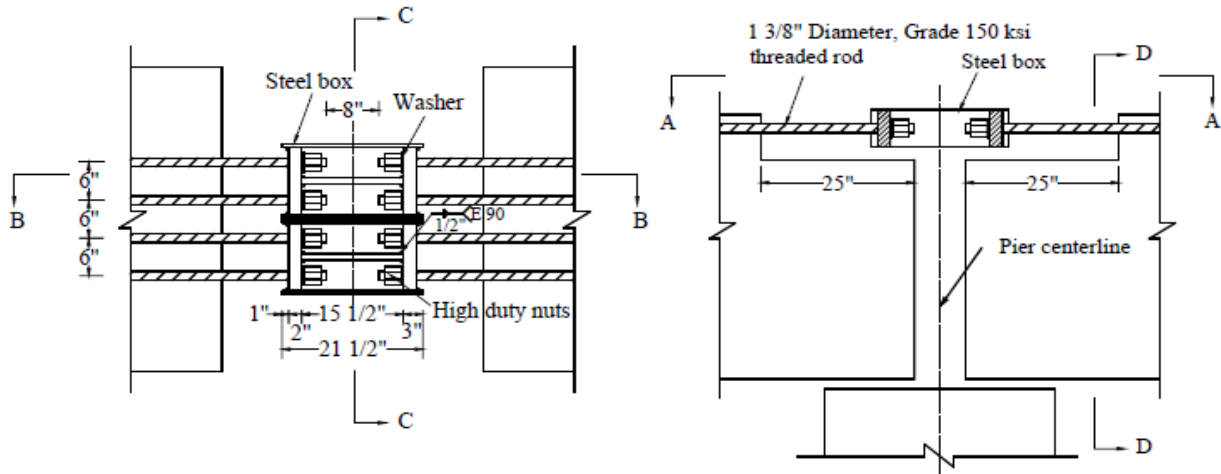


Figure 2.3. High Strength Threaded Rods (Sun 2004).

2.2.1.4 Bolted Steel Plate Splicing

Bishop (1962) proposed the plate connection in Figure 2.4. In this type of connection, the beams were first erected as simple spans. The end of one beam was jacked upward at the first support, and the beams were connected at the second support by welding together plates cast into the ends of the top and bottom flanges. The raised end was lowered to the final position, thus developing a bending moment at the support equal to that caused by the self-weight of the continuous beam. Though this appeared to be an innovative solution, there were some drawbacks. First, this method changed the loading conditions under beam self-weight from simply supported to a cantilever. This required additional reinforcement in the upper part of the beams. Second, it was difficult to construct. The steel plates, especially the bottom ones, were not easy to weld because of the limited space, and the welded plates could affect the diaphragm concrete casting.

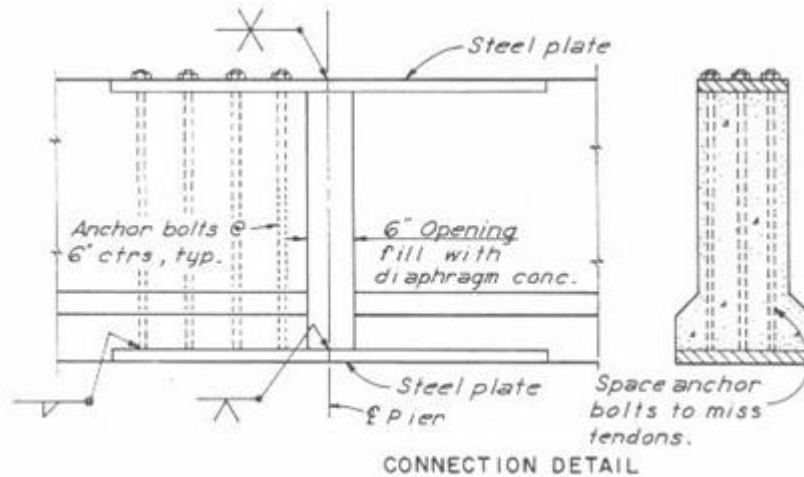


Figure 2.4. Bolted Steel Plate Connection (Bishop 1962).

2.2.2 Prestressed Design Options

2.2.2.1 Partial Length Post-Tensioning

Ficenec et al. (1993) described the project phases and implementation of new girder continuity technology for two bridge structures in Nebraska. The continuous spliced, prestressed concrete I-girder option was selected with an estimated cost of \$30,000 less than the steel plate girder. In this new girder continuity system, the girder segments were made continuous by splicing, coupling, and tensioning the pre-tensioning strand extensions at the adjacent ends of the girder segments. Full-length post-tensioning for continuity was also considered as an option but was ruled out because the structure lacked the post-tensioning volume necessary to render the use cost effective. The pedestrian/bicycle overpass bridge consisted of five spans with 90 ft exterior spans and 125 ft interior spans using 4 ft 6 in. deep Nebraska Type 4-A girders. The main viaduct bridge consisted of six spans with 86 ft and 114 ft exterior spans employing 4 ft 6 in. deep Nebraska Type 4-A girders and 172 ft interior spans employing 6 ft 3 in. deep Nebraska Type BT-1A girders. A combination of straight and harped strands was used for the pretensioned girders. The pretensioned strands were extended and positioned, and then spliced and stressed to fully withstand the service stresses and ultimate strength conditions providing the same structural benefits as full-length post-tensioning. For the design of the main viaduct in this project, the spliced, prestressed concrete girder bid augmented with full-length post-tensioning was found to be \$30,000 less than the alternate structural steel unit bid.

2.2.2.2 Full Length Post-tensioning

Lounis et al. (1997) investigated a variety of standard I-girder sections commonly used for continuous and segmental bridges. Three structural systems included in this study were:

- Two-span continuous girders with full length post-tensioning.
- Two-span conventional continuous pretensioned girders with non-prestressed reinforcement in the deck at the interior pier.
- Conventional simply supported pretensioned girders.

An optimization program was used considering different parameters such as span length, spacing between the girders, weight of the superstructure per unit surface area of the deck, durability, maintainability, life cycle costs, etc. Optimal sections were developed, which facilitated use of fewer girder lines and reduced the weight of superstructure. The span lengths of the girders considered for this study ranged from 115 ft to 200 ft. The authors made a few recommendations to modify the existing sections to enhance their strength and serviceability.

- Setting the width of the top flanges to 45 in. with a thickness of 4 in. was suggested as optimum to balance the structural efficiency and keep the girder weight to a minimum.
- For the bottom flanges, a width of 33 in. and a thickness equal to 6 in. was suggested as optimum when considering the fit of prestressing steel
- Webs that were 7 in. wide were adopted for the optimized sections to fit the required shear reinforcement and the prestressing steel with adequate cover to concrete.
- In general, it was recommended to keep the width of the bottom flange of the girder equivalent to the top flange, resulting in a symmetrical section that is beneficial for lateral stability.

Table 2.1. On-Pier Splicing Details.

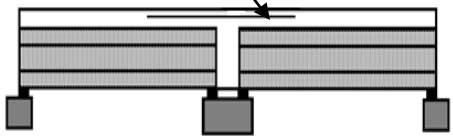
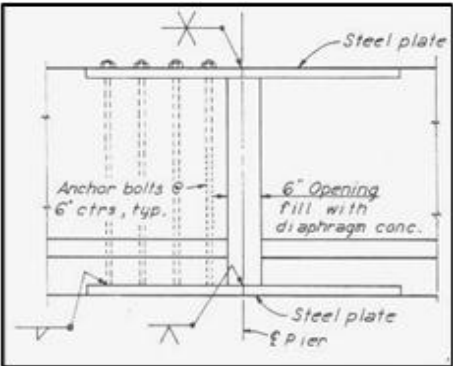
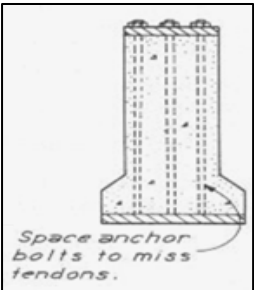
Splice Type	Advantages	Disadvantages
Non-prestressed Reinforcement in Deck (Kaar et al. 1960, and Mattock and Kaar 1960)		
<p>Maximum Span length = 140 ft</p> <p>Deck reinforcement for Superimposed D.L and L.L</p>  <p>(Kaar et al. 1960)</p>	<ul style="list-style-type: none"> ▪ Was found to be simple to construct and relatively economical. ▪ Could develop adequate resistant moments if designed for a static ultimate strength 2.5 times the design moment including impact effects. 	<ul style="list-style-type: none"> ▪ Maximum span length was restricted as a result of maximum transportable span length and weight. ▪ Simple span girders with single girder segment for whole span were found to be heavy in weight. <p>Cracks developed at the bottom of diaphragm due to positive restraint moment over the piers resulting from creep.</p>
Bolted Steel Plate Splicing (Bishop 1962)		
<p>Maximum Span length = 140 ft</p>  	<ul style="list-style-type: none"> ▪ Found to be a simple non-prestressed connection detail. ▪ This connection detail avoided the need for professional post-tensioning contractors. 	<ul style="list-style-type: none"> ▪ This method changed the loading conditions under beam self-weight from simply supported to a cantilever, which required additional reinforcement in the upper part of the beams. ▪ Found to be difficult to construct. The steel plates, especially the bottom ones, were not easy to weld because of the limited space, and the welded plates could affect the diaphragm concrete casting.

Table 2.1. On-Pier Splicing Details (continued).

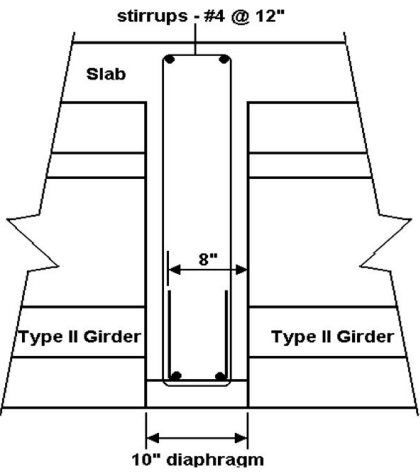
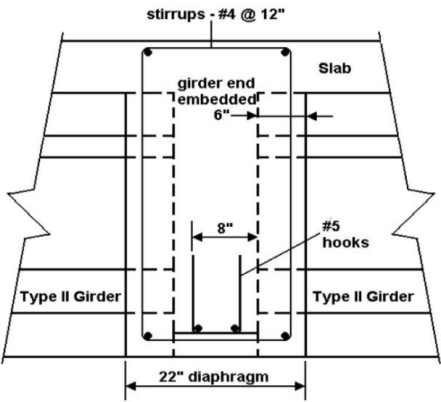
Splice Type	Advantages	Disadvantages
<p>Bent Bars to Resist Positive Moment at Support with Negative Moment Reinforcement in the Deck for Continuity (Dimmerling et al. 2005, Miller et al. 2004, and Mirmiran et al. 2001b)</p>		
<div style="text-align: center;">  <p>Bent bar connection (Dimmerling et al. 2005)</p>  <p>Bent bar connection with girder ends embedded in the Diaphragm (Dimmerling et al. 2005)</p> <p>Mild steel bars were embedded in the ends of the girders and bent into a 90-degree hook and extended in the diaphragm.</p> </div>	<ul style="list-style-type: none"> ▪ Controlled cracking found in the diaphragm due to positive moments ▪ Structure deemed safe even after cracking at the girder-diaphragm interface but at the expense of elimination of continuity action. ▪ Ductility of the connection could be improved by providing additional stirrups in the diaphragm close to the outside edge of the bottom flange of the girder. These stirrups could replace some of the extended bent bars and minimize congestion. ▪ Proposed alternative to these stirrups was horizontal bars in the diaphragm passing through the web of the beams. This connection proved to be stiffer than the stirrups and is more resistant to fatigue. 	<ul style="list-style-type: none"> ▪ Found to be costly with no structural benefit. ▪ Spalling of the diaphragm concrete was observed when girder end was embedded into the diaphragm. ▪ Greater amount of positive moment reinforcement could add to positive restraint moment, which needs to be accounted for in the design. ▪ Bars need to be bent in the field due to closure of forms for beams, and it was difficult to bend them consistently. ▪ For the connection detail using web bars, cracking in the beams at failure was noted, which might be undesirable.

Table 2.1. On-Pier Splicing Details (continued).

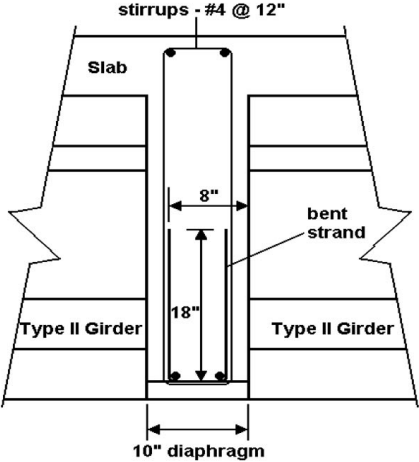
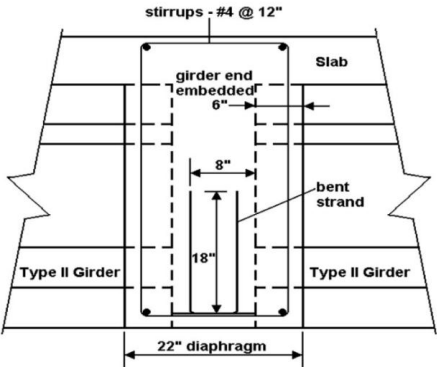
Splice Type	Advantages	Disadvantages
<p>Bent Strands to Resist Positive Moment at Support with Negative Moment Reinforcement in the Deck for Continuity (Dimmerling et al. 2005, Miller et al. 2004, and Mirmiran et al. 2001b)</p>		
<div style="text-align: center;">  <p>Bent Strand connection (Dimmerling et al. 2005)</p>  <p>Bent Strand connection with girder ends embedded in the Diaphragm (Dimmerling et al. 2005)</p> <p>Pre-determined length of prestressing strands was left protruding from the ends of the girders and bent into a 90-degree hook in the diaphragm.</p> </div>	<ul style="list-style-type: none"> ▪ Embedment of girder into the diaphragm was found to be beneficial for this type of connection. This reduced the stress in the connection. ▪ This connection was easy to fabricate and erect. Strands were flexible and easy to place. ▪ Structure was safe even after cracking at the girder-diaphragm interface but at the expense of elimination of continuity action. ▪ Reduced congestion in the diaphragm compared to bent bar connection detail. 	<ul style="list-style-type: none"> ▪ No accepted design method for determining the number and embedment length of the prestressing strands. ▪ Vibrating the concrete in casting the diaphragm, displaced the strands from position. ▪ Crack widths in the diaphragm were significantly large under full service and cyclic loads. ▪ Spalling of the diaphragm concrete was observed when girder end was embedded into the diaphragm. ▪ Inadequate development length for the bent strand could reduce the capacity of the connection.

Table 2.1. On-Pier Splicing Details (continued).

Splice Type	Advantages	Disadvantages
<p>Conventionally Reinforced with Mild Steel Bent Bars at Bottom at Support (Koch 2008, and Newhouse et al. 2005)</p>		
<div data-bbox="191 520 649 798" data-label="Image"> </div> <ul style="list-style-type: none"> ▪ Continuity connection provided at the bottom of the ends of girders by extending 180-degree mild steel bent bars into the diaphragm ▪ Negative moment continuity provided by reinforcement in the deck 	<ul style="list-style-type: none"> ▪ Girders were designed as simple spans for dead and live loads. Thermal, shrinkage, and creep effects were not considered in design. ▪ Continuity diaphragm was cast in flush with the ends of the girders. No embedment of girders in the diaphragm. ▪ Extended bars remained stiff during cyclic loading. ▪ Diaphragms were designed for thermal restraint moments. ▪ Connection was able to transfer service loads effectively. Bent bars were designed for maximum factored anticipated service load. ▪ Bent bar connection was efficient compared to the extended prestressing strands bent at 90 degrees in the diaphragm in relation to the crack openings under service and cyclic loads. <p>Cracking at girder-diaphragm interface could be controlled by providing additional reinforcement.</p>	<ul style="list-style-type: none"> ▪ Cracking was expected at the girder-diaphragm interface. Interface edges were required to be sealed during initial construction phase. ▪ Initial cracking occurred at a tensile stress lower than the modulus of rupture of concrete at the diaphragm-girder interface. ▪ Girders were recommended to be stored for 90 days before continuity was established. ▪ Noticeable increase was observed in the initial cost of construction of the detail.

Table 2.1. On-Pier Splicing Details (continued).

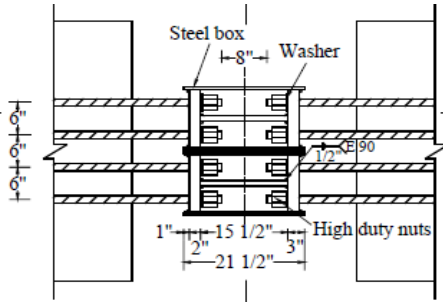
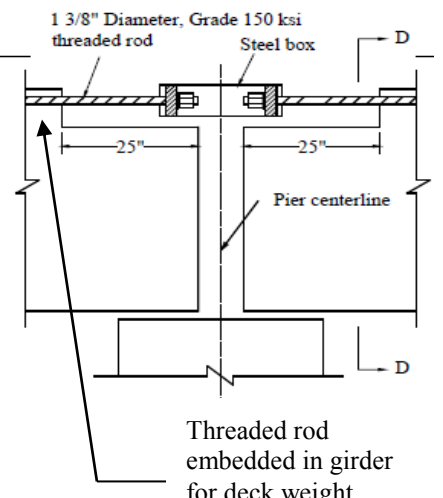
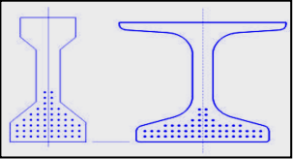
Splice Type	Advantages	Disadvantages
<p>Prestressed for Simple Span and Made Continuous with Threaded Rods over Support (Tadros and Sun 2003, Sun 2004, and Tadros 2007)</p> <p>Maximum Span Length = 200 ft</p>  <p><i>Plan View</i></p>  <p><i>Elevation</i></p> <p>Threaded Rod Detail (Sun 2004)</p> <ul style="list-style-type: none"> ▪ Embedding TR in girder ends ▪ Coupling girders over piers ▪ Pouring the diaphragm ▪ Placing the deck with the continuity deck reinforcement 	<ul style="list-style-type: none"> ▪ NU I-Girder had wide top and bottom flanges that improved strand capacity at both positive and negative moment locations. ▪ These girders facilitated shorter deck slab spans and served as better working platforms. ▪ Beam shared some of the negative moment. Diaphragm bottom was pre-compressed to balance the tension at top of the beam ends and it also mitigated the tension due to time-dependent positive moments. ▪ Haunched girder shape provided an increase in depth of 3.3 ft over a distance of 16.4 ft. ▪ Span lengths were extended beyond the practical limits of standard precast shapes.  <p>Standard I-Girder NU-I Girder</p>	<ul style="list-style-type: none"> ▪ Intermediate diaphragms were used, which added dead weight to the superstructure. ▪ New cross-section for the girders was used, which was found to add to the initial cost of the superstructure. ▪ Transportation of the heavy haunched section to the construction site was found to be difficult.

Table 2.1. On-Pier Splicing Details (continued).

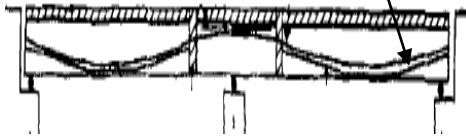
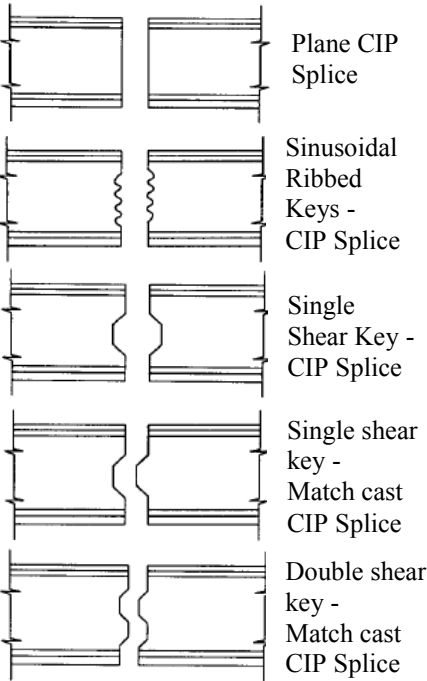
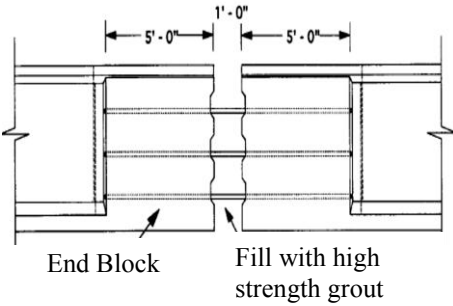
Splice Type	Advantages	Disadvantages
Post-tensioning for Splicing over Support (Castrodale and White 2004, and Lounis et al. 1997)		
<p>Maximum Span length = 160 ft</p> <p>Post-tensioning for continuity</p>  <p>(Lounis et al. 1997)</p>	<ul style="list-style-type: none"> ▪ This detail was found to overcome the problems of transportation and erection of long and heavy precast girders. ▪ Provided a precast I-girder system that was far more competitive with the steel plate girders and box girder alternatives for long spans. ▪ This detail eliminated end anchorage zone and congestion of reinforcement at ends in the girder section. ▪ Better serviceability and durability of the deck was observed by elimination of cracking. ▪ Though expensive, found to be an appropriate and efficient design detail. 	<ul style="list-style-type: none"> ▪ Post-tensioning operation was found to be expensive, but this was balanced with fewer substructure units and wider spacing between girders. ▪ This detail required anchorage of tendons in the diaphragms.

Table 2.1. On-Pier Splicing Details (continued).

Splice Type	Advantages	Disadvantages
Conventionally Reinforced/Post-tensioned Special End Diaphragm (Abdel-Karim and Tadros 1995)		
<p>Maximum Span Length = 160 ft</p>   <p>Plane CIP Splice</p> <p>Sinusoidal Ribbed Keys - CIP Splice</p> <p>Single Shear Key - CIP Splice</p> <p>Single shear key - Match cast CIP Splice</p> <p>Double shear key - Match cast CIP Splice</p> <p>End Block</p> <p>Fill with high strength grout</p> <p>Stitched Splice</p>	<ul style="list-style-type: none"> ▪ Simple span girders were post-tensioned for superimposed DL and LL. ▪ End blocks in girders were replaced with special end diaphragms that effectively distributed concentrated anchorage forces. ▪ This helped in simplifying adaptation to curved alignment. ▪ Sinusoidal shear keys reduced stress concentrations and distributed shear stresses effectively. ▪ A stitched splice combined merits of both post-tensioned and conventionally reinforced splices. Pretensioned segments were post-tensioned across the splice using short tendons or threaded bars. 	<ul style="list-style-type: none"> ▪ Splice was expected to crack at the top surface under full service loads. ▪ Shear keys in general were found to be aesthetically undesirable and structurally troublesome due to potential stress concentrations. ▪ In a stitched splice, if precise alignment of the post-tensioned ducts was not achieved, considerable frictional losses occurred, which undermined the effect of post-tensioning. ▪ Temporary support piers were required during construction.

2.3 IN-SPAN SPLICING WITH CONTINUITY DIAPHRAGM

Table 2.2 provides a summary of in-span splicing details that have been used for continuous precast, prestressed concrete girders. More details are provided below.

2.3.1 Partial Length Post-Tensioning

Caroland et al. (1992) presented the design of a 1000 ft long Shelby Creek bridge in eastern Kentucky using spliced prestressed concrete I-girders. An alternate competitive bid for a steel delta frame girder bridge was found to be \$2 million higher than the bid for spliced prestressed concrete I-girder bridge. The bridge consisted of five spans with end spans of 162 ft 3 in. and three equal interior spans of 218 ft 6 in. This continuous prestressed concrete I-girder option used seven lines of the I-girders spaced at 12 ft 6.5 in. supporting an 8.5 in. thick and 85 ft 3.5 in. wide deck slab. Each line of the girders was divided into nine equal length segments measuring 108 ft 3 in. Figure 2.5 presents the layout of the post-tensioning tendons used for the girders, pier cap, and girder splices and diaphragms.

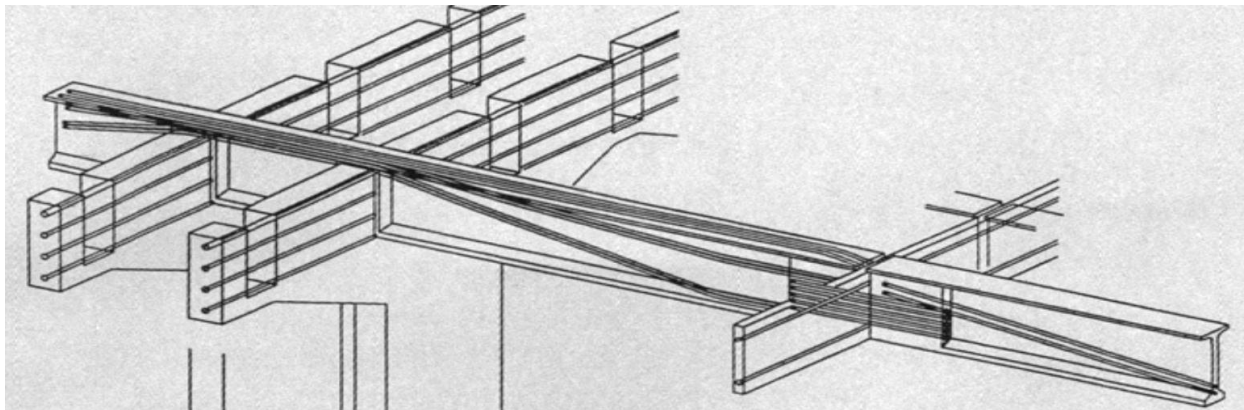


Figure 2.5. Layout of Post-Tensioning Tendons for Girders, Pier Cap, and Girder Splices/Diaphragms (Caroland et al. 1992).

The girder segments were pretensioned with temporary pre-tensioning strands in the pier segments for transportation and handling and augmented tendons for the drop-in segments to be post-tensioned before lifting on site. The piers consisted of four slender columns with heights ranging from 133 ft to 195 ft having a pier cap with deep slots to accommodate the 8 ft 6 in. constant depth I-girders. For each pier, the columns and caps were spaced 15 ft on centers

longitudinally with the pier segments grouted into the caps, resulting in a stable set of cantilevers supporting the drop-in segments. The precast concrete deck panels were set on the pier segments and then the post-tensioning tendons in the pier girder segments were stressed. The drop-in segments were erected using a Cazaly hanger and held in position while the temporary strands in the pier segments were released, and the precast concrete diaphragms and CIP closures were placed and the post-tensioning tendons through the girder segments and diaphragms were stressed. There were no continuity tendons running through the length of the bridge. The girder segments were individually stressed and then spliced with post-tensioned strands through the end blocks. The ducts through the girders and caps were spliced and grouted, and once this grout reached the specified strength, the post-tensioning tendons in the pier cap were installed and stressed.

2.3.2 Full Length Post-Tensioning

The types of methods used in different states for extending span ranges using incremental variations in the materials and conventional design procedures often result in relatively small increases in span range for the precast, prestressed concrete girders. One of the techniques adopted in the current state-of-the-art and practice is spliced girder technology, which has the potential to extend the simple spans by approximately 50 percent. In this technique, precast, prestressed concrete girders are fabricated in several relatively long segments and are assembled into the final bridge structure. Post-tensioning is generally used to provide continuity between the girder segments.

Constructed in the early 1990s, the bridge along US 231 over the White River, Indiana, is a multi-span spliced concrete girder bridge with constant depth, full span girders spliced at interior piers, and post-tensioned for continuity (Castrodale and White 2004). This spliced girder design was bid as an alternative to steel plate girder option. The bridge had three continuous spans. The provision of semi-lightweight concrete reduced the dead weight of the structure, and continuity allowed for a very wide girder spacing resulting in an economic solution.

The use of spliced-girder technology was successfully applied to increase span lengths and transverse spacing of the standard precast, prestressed concrete girders for the Highland View Bridge in Florida (Janssen and Spaans 1994). Figure 2.6 presents the layout of the bridge and girder cross-sections. This is a three-span continuous bridge with a main span of 250 ft,

which was a record for this type of structure at the time of its construction. Haunched girders 10 ft in depth were used over the piers, and constant depth drop-in girder segments had depth of 6 ft. Two falsework towers were erected 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 into position. Strong-backs were attached to the drop-in segments to support them from the ends of the pier segments.

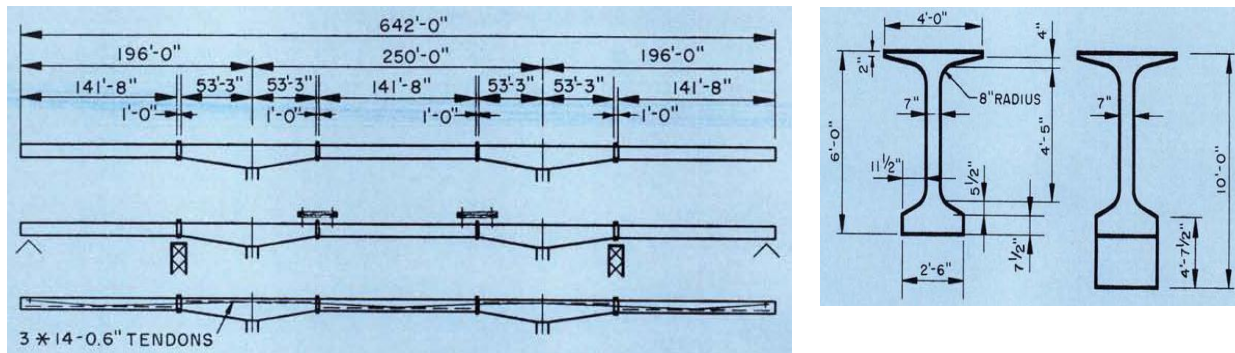


Figure 2.6. Use of Spliced Girders for Highland View Bridge, Florida (Janssen and Spaans 1994).

The Main Street Viaduct in downtown Pueblo, Colorado (Fitzgerald and Stelmack 1996) crosses 12 railroad tracks, the Arkansas River and its dike, and a city street. This resulted in many obstacles beneath the bridge for locating the piers. Full-span prestressed concrete girders were not used because close girder spacing was required for the long spans. The bridge has five spans ranging from 88 to 174 ft. Haunched girders and thickened webs were used at piers to satisfy vertical under-clearance and structural requirements. The contractors used two falsework towers for the erection of haunched girder segments over the piers, and strong-backs attached to the pier segments for the erection of the drop-in girder segments.

The Rock Cut Bridge in the Stevens and Ferry County, Washington (Endicott 1996), is a replacement over the deteriorating bridge spanning the Kettle River. Nicholls Engineering in Spokane, Washington, was assigned the task of redesigning the bridge. The designers focused on setting four three-piece precast concrete girders, each measuring 190 ft 6 in. long, that were post-tensioned and then lifted into place with a bridge launcher. Central Pre-Mix, a PCI Certified Plant, was able to provide innovative large bulb-tees 7 ft 5 in. deep with a 6.5 in. wide web. The

designers combined these dimensions with a 6 ft 1.5 in. wide top flange and a 2 ft wide bottom flange to span the 190 ft of the gorge. To facilitate the transportation of these long span girders, each 190 ft length girder was cast in three 63 ft long segments. On site, these girder segments were spliced and post-tensioned together at a nearby staging area before being launched across the gorge. Each of the girders had a final weight of nearly 250,000 lb. The specially designed launching equipment, working with the help of two large cranes, successfully placed these girders across the gorge. The use of post-tensioning and girder-launching system for erection eliminated the need for placing a pier in the water, which would have required an environmental impact study due to fish runs through the river. After the girders were launched into place, intermediate diaphragms were cast, and keyways on adjacent bulb-tees were welded and grouted to complete the bridge. This precast, prestressed concrete girder option lead to tremendous savings in the time of construction and overall maintenance costs of the bridge.

Ronald (2001) highlighted the use of a post-tensioning splicing system coupled with high performance concrete to build longer spans ranging up to 320 ft in Florida. This article focused on the various factors to be accounted for in the analysis, design, and construction of prestressed, post-tensioned bulb-tee girders. In this design approach, the bulb-tee girders were precast, pretensioned, and then spliced using post-tensioning performed in two stages on the construction site. Two types of spliced post-tensioned systems using haunched girder segments over the piers were discussed in this article (see Figure 2.7). The precast, prestressed bulb-tee girders fabricated in short segment lengths were spliced on the construction site. Stage 1 post-tensioning allowed for girders to become continuous before casting of the deck. Stage 2 post-tensioning resulted in residual compression in the deck for serviceability and deflection control. The two-stage system of post-tensioning allowed for wider spacing between the girders, and the higher cost of post-tensioning was compensated for by a reduction in the number of piers. The proposed system did not use intermediate diaphragms. Because lateral stability became an important issue for long and slender girders, it was recommended to use sections with wide top and bottom flanges. Creep and shrinkage significantly affect the stress and deflection in continuous composite prestressed concrete members; therefore, the use of the ultimate creep and shrinkage coefficients in the analysis was found to be critical. It was recommended to use the coefficients obtained from previous projects or mix design testing and adjust the girder fabrication and construction schedules to alleviate the time-dependent effects. The construction process for this spliced

structural system was found to be simple and cost-effective compared to span-by-span and balanced cantilever construction.

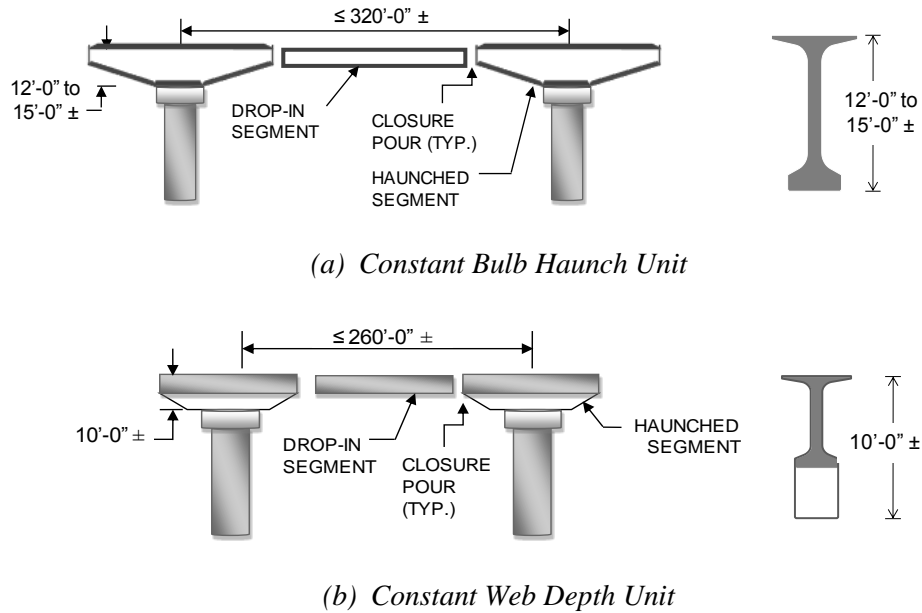


Figure 2.7. Splicing of Continuous Post-Tensioned Girders (Adapted from Ronald 2001).

Tadros and Sun (2003) developed haunched concrete girders using standard Nebraska NU sections to increase the span length up to 300 ft. The haunched block alternative helped to increase the span length of constant section depth standard girders that could be fabricated and shipped in small sections. However, in the case of a single piece pier segment, the shipping limits were more likely to be exceeded. For continuous prestressed bridges, the pier segment section was found to be critical due to high shear and negative moment demands. Using a deeper section on the pier was one approach to resist those forces. One alternative proposed was the use of a haunched section that consists of either a single piece or a composite haunched block (see Figure 2.8). Haunch block dimensions of $0.50L$ (length) and $0.9h$ (depth) were found to be the most efficient haunch block size (where h is the girder height and L is the span length). For three-span bridges, the span ratio $0.80L-L-0.80L$ was found to be the most efficient ratio. When the capacities between the two alternatives are far apart, the lower capacity controlled the design, leading to underutilized capacities. The system with the suggested haunch block dimensions

($0.5L$ and $0.9h$), and the modifications to the web thickness, eliminated the gaps between the capacities that existed in other systems.

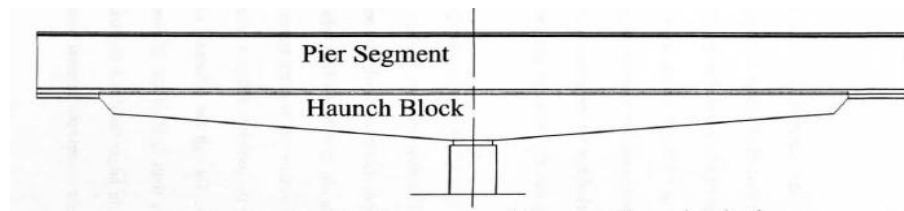


Figure 2.8. Composite Pier Segment and Precast Haunch Block (Tadros and Sun 2003).

Through the research study presented in NCHRP Report 517, Castrodale and White (2004) developed AASHTO LRFD design procedures, standard details, and design examples for long-span continuous precast, prestressed concrete bridge girders. This study noted that the precast, prestressed concrete bridge girders were rarely used for spans exceeding 160 ft due to material limitations, hauling size and weight limitations, and lack of design aids for the design of long span prestressed concrete girders. NCHRP Report 517 identified around 250 proven, spliced, precast, prestressed concrete girder bridges built around the nation but the experience and information on these job specific projects was not available widely for use on similar proposed bridge projects. This report provided the needed documentation on all the known technologies for extending the span lengths of the prestressed concrete girders to 300 ft. From the assessment of all these methodologies, this study concluded that the splicing of precast, prestressed concrete girders has the potential to significantly increase the span lengths to achieve the desired span range. The researchers identified the use of splicing with multiple means and locations within the span, and provided a list of similarities and differences between the spliced girder construction and the segmental bridge construction. NCHRP Report 517 summarized both material-related options and design enhancements for extending the span lengths. The material-related options included:

- High strength concrete.
- Specified density concrete.
- Increased strand size.
- Increased strand strength.
- Decks of composite materials.

The alternatives for design enhancements included:

- Modifying standard girder sections.
- Creating new standard girder sections.
- Modifying strand pattern or utilization.
- Enhancing structural systems.
- Enhancing analysis and design methods.

The multiple design examples presented in NCHRP Report 517 provide guidance for comparing the potential alternatives to extend span lengths. Cost comparisons with alternate bids were provided for some of the projects. For the US 231 bridge over the White River, Indiana, the low bid for the concrete alternate was more than 10 percent below the low bid for the steel alternate. Steel plate girder alternate bids were eliminated because of higher initial costs and the requirement for long-term maintenance over continuous spliced prestressed concrete I-girder bid for the Main Street Viaduct, Pueblo, Colorado. A spliced prestressed concrete I-girder option saved \$621,000 over the steel alternate bid for the Moore Haven Bridge, Florida. A segmental alternate bid for the Edison Bridge NB, Florida was 25 percent higher than the spliced precast girder bid. From the assessment of all these methodologies and examples of previous projects, it was concluded that the spliced precast, prestressed concrete girders post-tensioned for continuity have proven to be one of the most cost-effective structural systems with the potential to significantly increase span lengths up to and above 300-ft.

Nikzad et al. (2006) presented an article describing the design and construction of the Old 99 (Riverside) Bridge using spliced girder technology. Winner of the PCI Design Award in 2004, this bridge is an 850 ft long, 72 ft wide, five-span, post-tensioned prestressed concrete structure. Out of the five spans, the three interior spans are 180 ft long and the two end spans are 150 ft each. The superstructure consisting of the Washington State Department of Transportation W95PTG “supergirder” sections is semi-integral at the abutments and hinged longitudinally at the interior piers. The environmental constraints at the construction site completely eliminated the use of the temporary intermediate falsework supports and resulted in the development and design of a single piece long span girder. However, the major limitations for using this long, single piece girder were the maximum transportable weight and dimensions. As a result, spliced-girder technology was applied to increase span lengths and transverse spacing of the girders beyond the customary values. These precast concrete girder sections were transported to a

staging area close to the site, where they were spliced into single pieces that produced maximum spans of 180 ft (55 m). Then these girders were erected on top of the piers with no intermediate temporary supports. High-performance concrete (HPC), with design strengths of 7.5 ksi and 10 ksi, was used for the CIP splices and precast super-girder segments, respectively. After erection, placement and hardening of the deck and the diaphragms over the piers, post-tensioning was applied to further strengthen the girders and develop a continuous composite behavior. Special attention was paid to the accidental torsional buckling and lateral stability of the girders during erection and handling.

The Interstate 25 (I25) flyover in Denver, Colorado (Endicott 2005) is a continuous spliced precast, prestressed girder bridge employing trapezoidal sections (see Figure 2.9). The Colorado Department of Transportation used 7 ft deep open top trapezoidal sections (U-girder), both pretensioned and post-tensioned, for spans ranging between 156 ft to 200 ft. The two-lane, eight-span structure consists of two girders spanned by 17 ft 8 in. long and 8 ft wide deck panels that vary from 4.5 in. to 6 in. in depth. Of these panels, 131 were pie-shaped to accommodate the structure's curve. The approximate girder segment length was 100 ft with a maximum of 104 ft weighing 254,000 lb. Out of the total 28 U-girder segments, six were straight and the remaining 22 were cast with a superelevation of 5.6 percent and a radius of 962 ft along the curve. The girders featured 10 in. wide webs.

The precaster transported the heavy girder segments to the site by adding two hauling units. The contractor employed three large cranes on the site to handle the heavy lifting chores. Spliced girder technology was used in constructing the 200 ft long span of the bridge using the 100 ft long girder segments. The girder segments were first balanced on piers and temporarily supported on falsework. The drop-in segments were added between the pier segments and the entire system was post-tensioned for continuity. This spliced precast, prestressed concrete U-girder option was found to be approximately \$200,000 lower than the next-lowest bid, which used steel girders. Unlike other curved structures, which have been built using precast concrete, the girders themselves were straight segments forming a curve; this bridge employed curved U-girders cast with a radius and superelevation.



(a) Erection of U girders on site



(b) Cross-section of U girders

Figure 2.9. Spliced U Girders, I25 Flyover Denver, Colorado (PCI 2005).

Table 2.2. In-Span Splicing Details.

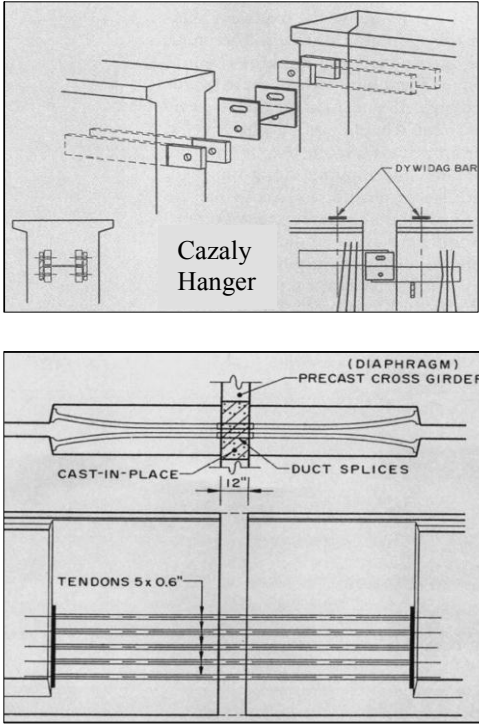
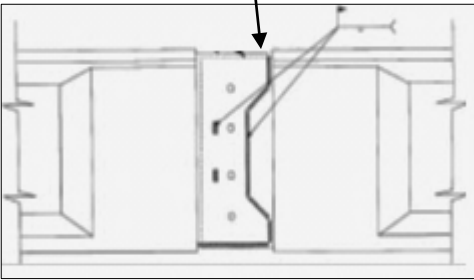
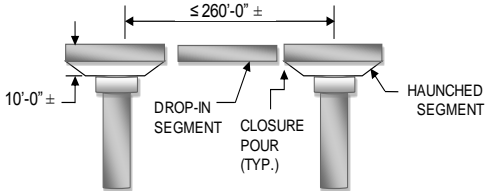
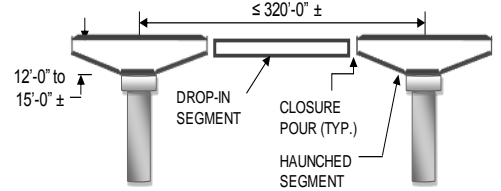
Splice Type	Advantages	Disadvantages
Prestressed for Simple Span and Partially Post-tensioned for Continuity (Caroland et al. 1992)		
<p>Maximum Span length = 250 ft</p> 	<ul style="list-style-type: none"> ▪ Girder segments were made continuous by stressing partial (short) length post-tensioned strands between the adjacent ends of the girder segments. ▪ The partial length post-tensioned strands were found to fully withstand the service stresses and ultimate strength conditions. ▪ Economical solution compared to steel plate girder alternatives in span range of 130 ft to 250 ft. 	<ul style="list-style-type: none"> ▪ No continuity tendons were provided throughout the length of the bridge. Therefore, complete load balancing was not achieved. ▪ Special attention was required in construction of the partially post-tensioned splice connection. ▪ End blocks were needed in the girder segments to anchor the partial post-tensioned strands.
Structural Steel Splice (Abdel-Karim and Tadros 1995)		
<p>Maximum Span Length = 160 ft</p> <p>Embedded steel plate</p>  <ul style="list-style-type: none"> ▪ Embedded structural steel plates welded on-site. ▪ One beam has outside plates, and matching beam has inner plates embedded in the section. 	<ul style="list-style-type: none"> ▪ Easy to erect beams into place and splice them. ▪ Use of connecting bolts facilitated welding operation to be carried out carefully. ▪ Light in weight and simple connection. 	<ul style="list-style-type: none"> ▪ High maintenance cost for the steel plates. Corrosion of steel may severely affect performance of the continuity joint. ▪ Careful alignment of the adjacent beams was required. ▪ Welding details needed special attention and care.

Table 2.2. In-Span Splicing Details (continued).

Splice Type	Advantages	Disadvantages
Prestressed for Simple Span and Post-tensioned for Continuity (Ronald 2001)		
<p>Maximum Span Length = 260 ft</p>  <ul style="list-style-type: none"> ▪ Girder section is Florida bulb-tee with 78 in. depth ▪ Girder spacing = 11 ft 6 in. ▪ Closure pour width = 1 ft 6 in. ▪ Web thickness of bulb-tee = 9 in. ▪ Depth of Haunched segment = 10 ft ▪ Length of Haunched segment = 110 ft 	<ul style="list-style-type: none"> ▪ Stage 1 post-tensioning: Allowed girders to be made continuous. ▪ Stage 2 post-tensioning: Provided residual compression in the deck for serviceability and deflection control. ▪ Cost of post-tensioning was offset by use of few girder lines and greater spacing between girders. ▪ Span lengths were extended beyond the practical limits of standard precast shapes. 	<ul style="list-style-type: none"> ▪ Cost of superstructure increased with longer spans. ▪ The deeper the haunch, the greater was the negative moment drawn toward interior piers. ▪ Long, slender bulb-tee girders deflected and twisted during handling and erection. ▪ Restriction in the length of the haunched segment based on the amount of prestress that can be provided in the top flange of the girder to resist cantilever bending before post-tensioning.
<p>Maximum Span Length = 320 ft</p>  <ul style="list-style-type: none"> ▪ Girder section is Florida bulb-tee with 78 in. depth ▪ Girder spacing = 9 ft 6 in. ▪ Closure pour width = 1 ft 8.5 in. ▪ Web thickness of bulb-tee = 9 in. ▪ Depth of Haunched segment = 12 ft ▪ Length of Haunched segment = 115 ft ▪ For Girders and Closure pours: $f'_c = 8500$ psi ▪ For Deck: $f'_c = 6500$ psi ▪ Strands: 0.6 in. diameter, ASTM A416, Grade 270 low relaxation 	<ul style="list-style-type: none"> ▪ No intermediate diaphragms were used. ▪ Fewer massive piers were used for longer spans. ▪ Wide web thickness of 9 in. to accommodate tendons with 16 strands. ▪ Shear key was provided in webs for interlocking. ▪ Blisters were used at closure points to overlap tendons. ▪ Minimum impact on surrounding environment and traffic during construction. 	<ul style="list-style-type: none"> ▪ Difficult to transport heavy haunched girder segments.

2.4 MATERIALS AND SECTION PROPERTIES

Abdel-Karim and Tadros (1992 and 1995) presented the state-of-the-art and practice of the design and construction of over 40 bridge projects with spliced I-girder bridges in the United States and Canada. The most popular trend was to use high strength concrete ranging from 6 ksi to 10 ksi along with slender and lightweight sections for the girders. The authors noted that the standard AASHTO and PCI I-girders and bulb-tee shapes were modified for their efficient utilization in the negative and positive moment regions with elimination of the end blocks and use of special end diaphragms.

Several advantages of spliced girders were noted. One of the major advantages was to construct an efficient and acceptable curved alignment. The use of haunched girder segments over the piers allowed for shallow and lighter drop-in segments with increased vertical clearance and effective use of materials than that for the simple spans. This type of continuous structural system exhibited an aesthetically pleasing appearance, greater structural capacity, dynamic response, elimination of the deck joints, and their potential for long-term maintenance. The number of piers required was reduced and thus substantially reduced the cost of substructure. Abdel-Karim and Tadros presented a cost estimate per girder line of a 350 ft long two span continuous spliced I-girder bridge system compared with that of a steel alternative of comparable span length. The results of this cost analysis showed that the cost of the steel alternative was more than twice the cost of the prestressed concrete spliced I-girder option.

2.5 ISSUES IN ADOPTING SPLICED GIRDER TECHNOLOGY

Spliced girder construction was found to be popular in some states for extending spans of their bridges. At the same time, other states have not used the benefits of this system. Bridges with intermediate diaphragms over piers providing continuity for live loads were found to be common in different states. However, in this form of construction the span length was governed by the length of the girder segments that can be easily fabricated and transported to the construction sites. Thus, this method of providing continuity was not found to produce a significant increase in the span length. The girders were designed as simple spans and made continuous for live load through continuity in the deck and diaphragm to eliminate expansion joints in the deck slab. In order to control cracking at the bottom of the diaphragm due to extreme events, unanticipated loadings and time-dependent effects, positive moment connections

were required to improve structural integrity of the system. The positive moment connection reinforcement was observed to have a negligible effect on reducing the resultant midspan moments. Mirmiran et al. (2001b) suggested that positive moment reinforcement between $0.6 M_{cr}$ to $1.2 M_{cr}$ should be used to limit the crack width in the diaphragm and to avoid significant loss of continuity, where M_{cr} is the cracking moment of the diaphragm section. Therefore, this type of continuity was disputed for efficiency with increase in the overall cost and time of construction.

The spliced girder system concept is midway between the conventional on-pier continuity system and segmental construction in terms of the achievable span range and complexity of construction. Like the on-pier system, this system uses girder segments. These girder segments are spliced at a CIP joint within the span utilizing a simpler form of construction and consuming less time than required for segmental box girders. This helps to fill the gap between the abovementioned conventional systems to construct cost-competitive bridges with spans ranging between 140 ft to 300 ft. Such bridges have been constructed successfully in some states like Florida. If the local contractors and designers in other states gain familiarity with the design concept and construction for this system, the state DOTs can benefit from this spliced girder technology. This technology uses precast bridge elements, which not only produces significant increase in the span but also reduces time of construction, disruption of existing traffic, and environmental impact. Other merits include the potential for wider girder spacing and a lower superstructure weight per unit surface area of the bridge deck. This facilitates use of fewer girder lines and a lower superstructure cost without penalizing the weight on the substructure.

Design concepts for spliced precast girders have recently been included in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2010). Specifications for spliced girder bridges have been separated from the segmental bridges to provide guidance to the designers. These new specifications, along with a review of the current state-of-the-art and state-of-the-practice, can assist bridge engineers in developing appropriate design strategies for spliced precast, prestressed concrete girder bridges.

2.6 RESEARCH NEEDS

Precast, prestressed concrete girders are the predominant element used in Texas bridges. This is a reflection of the durability, low cost, and adaptability of prestressed concrete. A key

factor in TxDOT's widespread use of precast, prestressed concrete components is cross-section standardization, facilitating economical mass production of these bridge elements. Splicing technologies that has been constructed and proven successful by different state DOTs can be considered as potential options to construct new long-span continuous bridge structures in Texas. Different types of techniques and approaches are used in many states for extending span ranges including variations in the material properties and design enhancements for precast, prestressed concrete girders.

The current state-of-the-art and practice and the NCHRP reports 517 (Castrodale and White 2004) and 519 (Miller et al. 2004) illustrate additional concepts and advantages of spliced girder bridges where multiple continuous spans are required. It was found that spliced girder technology has the greatest potential to extend the span range of simple spans. Use of spliced girders in these projects facilitated wider spacing between girder lines, minimized the number of substructure units, improved aesthetics with slender superstructures, and provided ease of fabrication, transportation, and erection and adoption of conventional construction procedure on site. This demonstrated that spliced concrete girders provide speed of fabrication and erection resulting in economical solutions when bid in competition with other solutions.

Although many spliced girder bridges have been constructed, the use of this technique is not widespread. Systematic investigation is required in this area to adopt this technology in new projects in states or regions where they have not been previously used. Also, the use of this technology requires addressing various issues related to design, construction, and cost of spliced girders. Designers, fabricators, and contractors, upon successful collaboration, can take advantage of applying continuous construction to standard precast, prestressed girders to present cost-effective, easily constructible, and high performance alternatives for longer spans.

3. PRELIMINARY DESIGN OUTLINE

3.1 OBJECTIVE

The research team developed preliminary designs to carry out an initial evaluation of the design details with regard to construction and implementation of continuous precast, pretensioned girders. The main purpose of the preliminary design study was to investigate maximum span limits for continuous bridges when using standard precast shapes regularly used by TxDOT for prestressed concrete bridge construction. The maximum span length was set based on transportation limitations. The focus of this study was on Tx70 and Texas U54 prestressed girder bridges. The design parameters such as the girder type, strand sizes, girder spacing, material properties, etc. were selected such that they are representative of typical bridges in Texas. The concrete strengths at service and at release were limited to values commonly available from Texas precasters. Different design methodologies based on assumptions were evaluated to determine their impact on the final design loads and thermal effects. The potential key design constraints evaluated were as follows.

- Deflection.
- Shear demand on thin webs plus duct placement.
- Moment demand and ultimate strength.
- Flexure-shear interaction at supports.
- Serviceability stresses under live load and thermal gradient.

The following sections describe in-depth analysis of the critical parameters in adopting the standard Tx70 and U54 girder sections in transportable lengths for constructing longer spans using splicing technology. The splices between girder segments are located within the span of the bridge.

3.2 BRIDGE GEOMETRY AND GIRDER SECTION

A spliced girder bridge system facilitates fabrication of the girder segments in transportable lengths to achieve a longer span range with a relatively simple form of construction. A four-span continuous spliced precast, prestressed concrete bridge system using the Tx70 and Texas U54 girders is considered for the preliminary designs. Figure 3.1 shows the layout of the bridge. The span lengths chosen for the bridges in the preliminary designs are such

that they fall within the range of existing structures and will provide reasonable examples of structures that are realistically constructible. The bridge length is taken to be on a tangent alignment with no horizontal curvature. The length of each girder segment is shown as ‘ L ’ ft. The length of the main span is twice the length of the girder segment. The end spans are 0.75 times the main span length. The lengths of the main span considered for the Tx70 and Texas U54 girder bridges are 280 ft and 240 ft, respectively. The 280 ft maximum span for the deeper Tx70 girder was selected based on a maximum transportable length of 140 ft. The bridge section consists of two 12 ft wide travel lanes, one in each direction with a 10 ft wide shoulder on each side and a 1 ft standard traffic barrier on each side. The bridge is designed for a total of two traffic lanes in accordance with the design criteria specified in the AASHTO LRFD Specifications. Intermediate diaphragms are not considered for the preliminary designs.

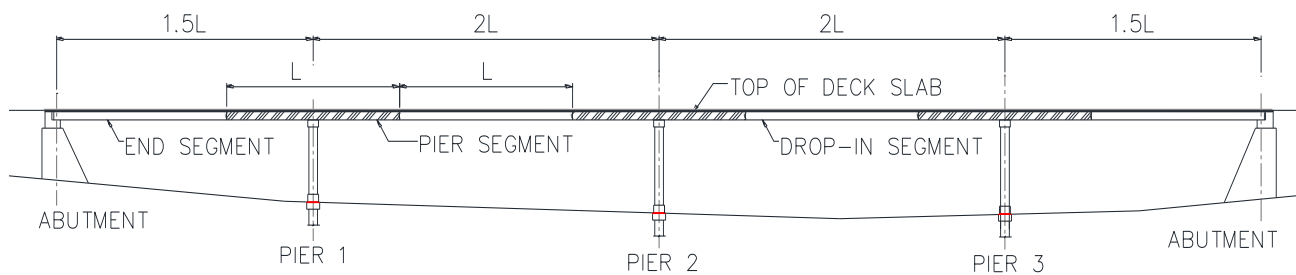


Figure 3.1. Continuous Spliced Precast, Prestressed Concrete Bridge Layout for Preliminary Designs.

The research team considered the most promising standard girder sections for continuous precast, prestressed concrete highway bridges in this study: Tx70 and Texas U54 girders. At the time of this study, the Tx70 girder section was the deepest section implemented by TxDOT. Figure 3.2 shows the details of the modified Tx70 girder cross-section used for the preliminary designs. Table 3.1 presents the non-composite properties of the modified Tx70 girder section. The girder is 70 in. deep with top flange 42 in. wide, bottom flange 32 in. wide and web thickness of 7 in. The web width of the standard Tx70 girder section was increased to 9 in. by spreading the girder side forms. This web width is required to accommodate 4 in. diameter post-

tensioning ducts. The width of the top and bottom flanges is also increased by 2 in.: the top flange to 44 in. and the bottom flange, to 34 in., respectively.

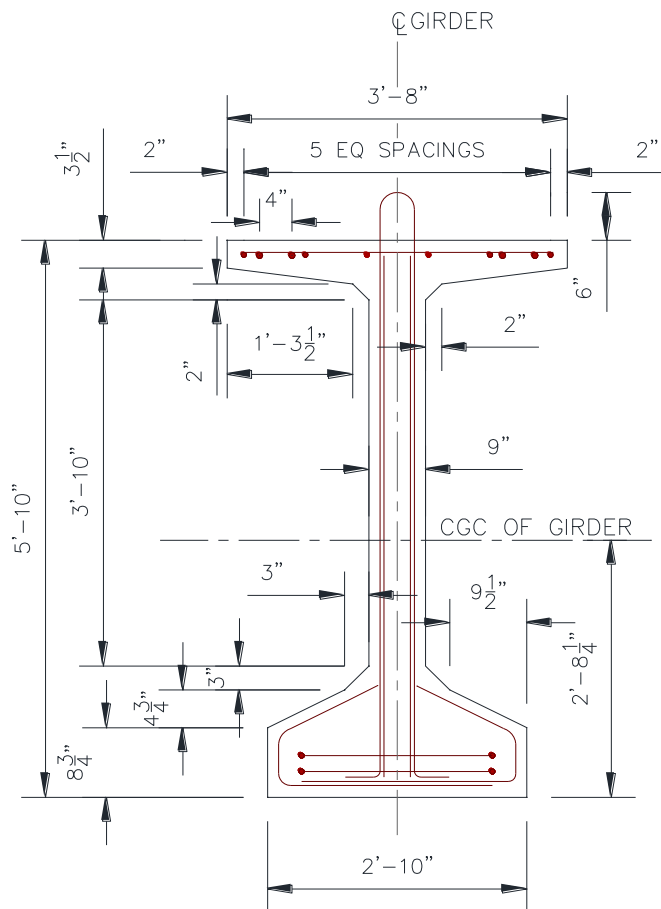


Figure 3.2. Typical Section Geometry of Modified Tx70 Girder with Widened Web (Adapted from TxDOT 2010).

Table 3.1. Section Properties for Modified Tx70 Girder with Widened Web.

Girder Type	Depth of N.A. from top, y_t (in.)	Depth of N.A. from bottom, y_b (in.)	Area, A (in. ²)	Moment of Inertia, I_x (in. ⁴)	Weight (plf)
Modified Tx70	37.70	32.30	1106	687,081	1,152

The Texas U54 girders have an open-top trapezoidal section that is aesthetically pleasing and an efficient cross-section that has torsional rigidity. Figure 3.3 shows the details of the Texas U54 girder cross-section. Table 3.2 presents the major dimensions of Texas U54 girder section.

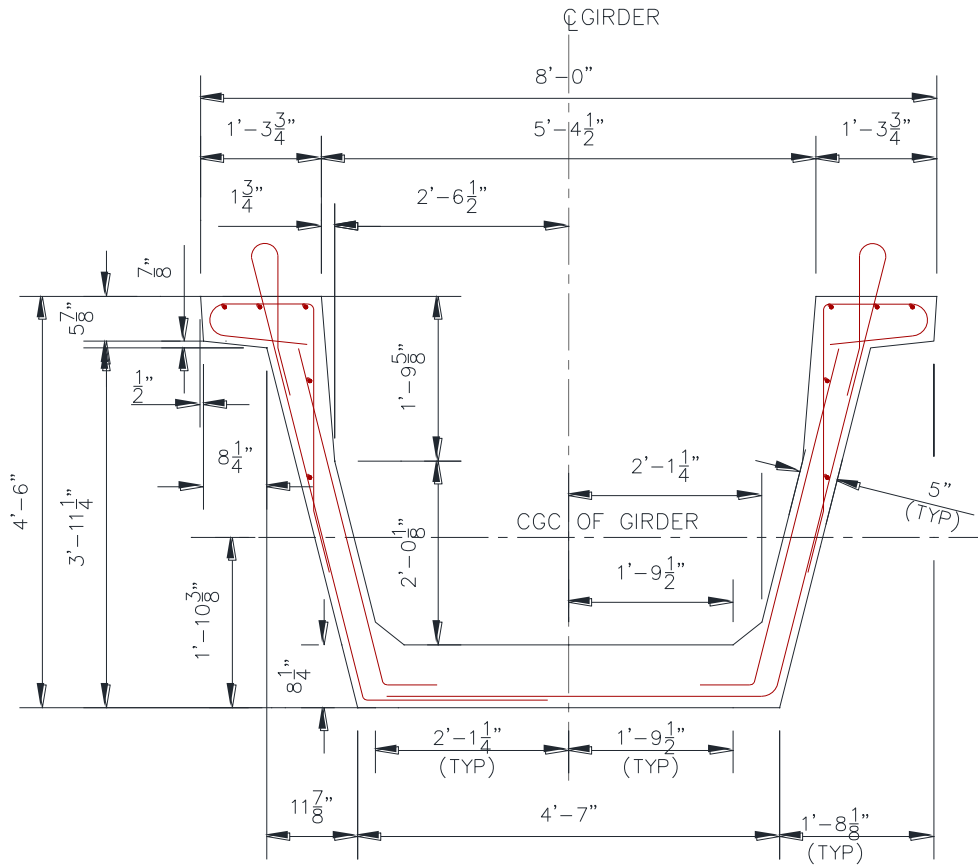


Figure 3.3. Typical Section Geometry of Standard Texas U54 Girder (Adapted from TxDOT 2010).

Table 3.2. Section Properties for Texas U54 Girder.

Girder Type	Depth of N.A. from top, y_t (in.)	Depth of N.A. from bottom, y_b (in.)	Area, A (in.²)	Moment of Inertia, I_x (in.⁴)	Weight (plf)
U54	31.58	22.36	1120	403,020	1,167

3.3 DESIGN PARAMETERS

The most promising options considered for preliminary designs are Tx70 and Texas U54 prestressed concrete girder bridges. The design parameters were selected based on the current state-of-practice of TxDOT. The girder type and sizes, girder spacing, material properties, etc. considered in this study are representative of the typical bridges in Texas. The concrete strengths at service and at release are limited to values commonly available from Texas precasters. TxDOT practice dictates the girder segment length and girder spacing. Table 3.3 presents the design parameters for the Tx70 and U54 girder bridge design used for this research study.

Table 3.3. Design Parameters for Preliminary Designs.

Parameter		Description/Selected Values
Total bridge width		46 ft
Deck slab thickness		8 in.
Unit weight of concrete (CIP), w_{cip}		0.150 kcf
Unit weight of concrete (Precast), w_c		0.150 kcf
Unit weight of 2 in. asphalt wearing surface, w_s		0.140 kcf
Weight of Type T501 rail		326 plf
Specified Concrete Strength at service for deck slab (CIP), f'_c		4 ksi
Maximum Specified Concrete Strength at service (Precast), f'_c		8.5 ksi
Maximum Specified Concrete Strength at release (Precast), f'_{ci}		6.5 ksi
Modulus of Elasticity, E_{cip}		$33,000 w_{cip}^{1.5} \sqrt{f'_c}$
Modulus of Elasticity, E_c		$33,000 w_c^{1.5} \sqrt{f'_c}$
Modular Ratio, n		E_c/E_{cip}
Coefficient of thermal expansion of concrete		$6 \times 10^{-6}/^\circ\text{F}$
Mild steel (ASTM A615 Grade 604)	Yield strength, f_y	60 ksi
	Modulus of Elasticity, E_s	29,000 ksi
Prestressing steel	Strand diameter	0.6 in.
	Ultimate tensile strength, f_{pu}	270 ksi (low relaxation)
	Yield strength, f_{py}	$0.9 f_{pu}$
	Stress limit at transfer, f_{pi}	$f_{pi} \geq 0.75 f_{pu}$

Table 3.3. Design Parameters for Preliminary Designs (continued).

Parameter		Description/Selected Values
Prestressing steel	Stress limit at service, f_{pe}	$f_{pe} \geq 0.8 f_{py}$
	Modulus of Elasticity, E_p	28,500 ksi
	Wobble Coefficient, K	0.0002/ft
	Coefficient of friction, μ	0.25

3.4 DESIGN ASSUMPTIONS

The preliminary designs developed and presented in this report are representative of the state-of-the-art and practice of spliced girder bridges. The following basic assumptions are used for the preliminary designs.

- The bridge length is taken to be on a tangent alignment with no horizontal curvature.
- Girder segments are erected on permanent piers using temporary shoring towers. (Note that the use of strongbacks can be considered when shoring towers are not feasible. This option will be explored in Phase 2 of this study.)
- Permanent intermediate diaphragms are not considered for the preliminary designs. Temporary intermediate diaphragms of structural steel shapes are used during construction and casting of the deck slab. The weight of these temporary components is minor and neglected in the calculations.
- Girder segments are pretensioned for stresses from handling and erection. The pretensioning force is selected to resist the girder self-weight with a 20 percent impact factor for transportation.
- Post-tensioning tendons running internally through the web of a Tx70 girder are placed in a single vertical plane. Ducts of 4 in. diameter are used within the 9 in. widened web of the Tx70 girder for the post-tensioning tendons. Post-tensioning tendons may contain up to 19 0.6 in. diameter strands for this duct size. The ducts are grouted after the tendons are stressed for protection against corrosion.
- Post-tensioning tendons run externally through the void space of the Texas U54 girder. These unbonded, monostrand tendons are placed in a casing (rigid steel pipe casing as specified in AASHTO LRFD Table 5.9.5.2.2b-1) only locally through the

ribs or blisters. Such casings of 4 in. diameter are used that may contain up to 19 0.6 in. diameter strands. Polyethylene ducts could be used within the void space of the U54 girders.

- Post-tensioning tendons are stressed from both the ends during Stage 1 and during Stage 2 to provide symmetry of stresses in the structure.
- After Stage 1 post-tensioning and splicing of the girders, the temporary shoring towers are detached and left in place. Stage 2 post-tensioning tendons are stressed after the deck slab is poured and has achieved the desired strength. This approach is taken in this initial design evaluation to maximize compression in the deck slab.
- Post-tensioning girders will be spliced with a separate splice connection between the ends of the ducts projecting from the girder segments.
- A standard reinforced concrete deck is used with an 8 in. thickness. A 2 in. thick asphalt wearing surface is used, but is not considered part of the structural composite section and is treated as additional superimposed dead load. A 2 in. thick concrete buildup is used between the top of girders and bottom of the deck slab to accommodate construction tolerances and variation in camber. For simplicity, full thickness of the buildup is applied to the entire length of the girders for dead load computations.
- The entire deck is cast in a single operation as traditionally preferred by contractors for significant savings in time, labor, and cost. This may require the contractor to develop a concrete mix with set-retarding admixtures to delay the initial set time of concrete. It is necessary to take into consideration the pouring of the entire deck slab prior to initial set time of concrete, to account for any backup equipment in case of a failure, and to demonstrate the availability of an adequate supply of concrete from the mixing plant.
- 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 deck slab.
- The sign convention used for the preliminary designs considers tension as positive and compression as negative.

3.5 DETAILED DESIGN EXAMPLES

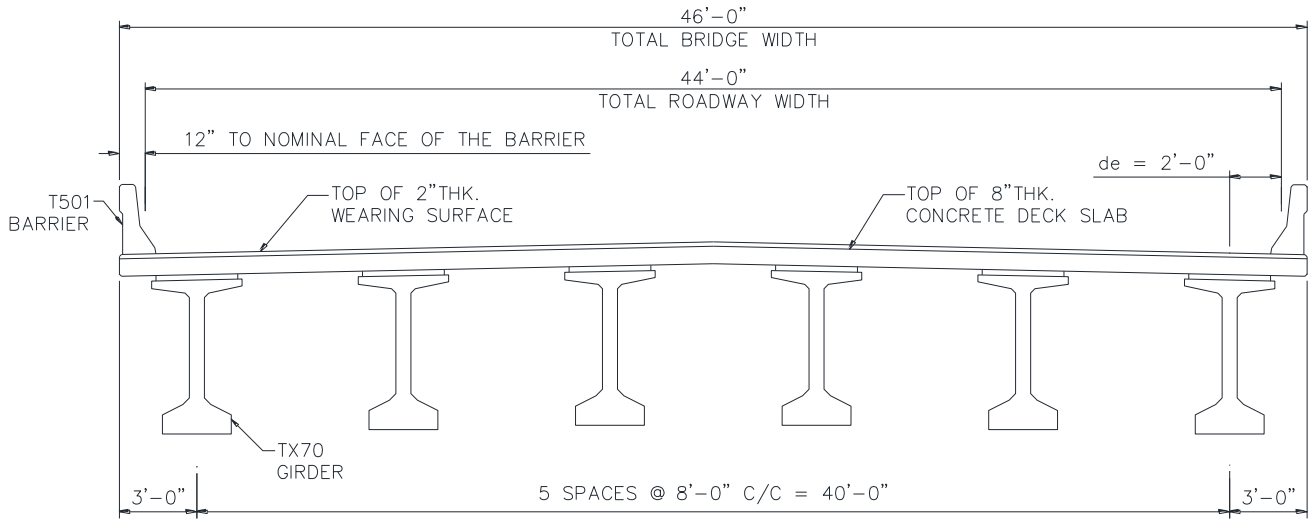
Two sets of detailed design examples were developed for the Tx70 and Texas U54 girders, respectively, to illustrate the design of continuous precast, prestressed concrete girders spliced in-span. The examples follow the revised provisions for ‘Spliced Precast Girders’ in the *AASHTO LRFD Bridge Design Specifications* and are intended to serve as a reference for bridge engineers in the transition from simply supported prestressed girder design to continuous spliced prestressed girder design. Figure 3.4 shows the cross-sections of the two bridge types.

The spacing and overhang configuration for both Tx70 and Texas U54 girder bridges result in optimum distribution of dead load and live load to the interior and exterior girders. Therefore, all girders have the same design requirements.

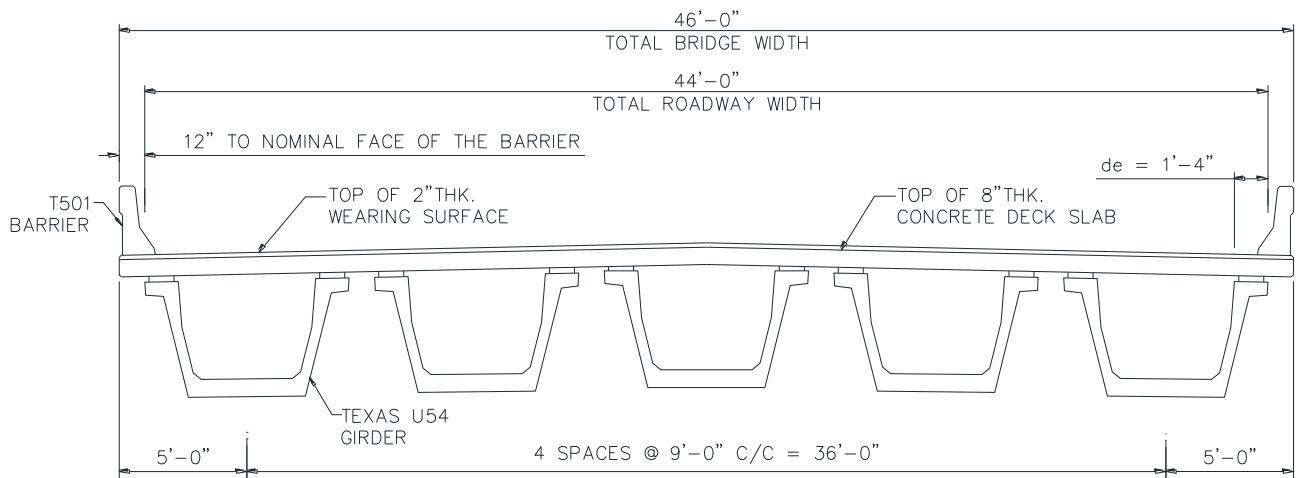
Table 3.4 provides additional parameters specific to the detailed design examples of Tx70 and Texas U54 girders, respectively.

Table 3.4. Additional Design Parameters for Detailed Design Examples.

Parameter	Tx70 Girder	Texas U54 Girder
Span of the bridge	280 ft	240 ft
Girder segment length	140 ft	120 ft
Total bridge width	46 ft	46 ft
Girder spacing	8 ft	9 ft
Total number of girders	6	5
Overhang	3 ft	5 ft



(a) Tx70 Bridge



(b) Texas U54 Bridge

Figure 3.4. Typical Bridge Section for Preliminary Designs.

3.6 DESIGN PROPOSAL FOR PRELIMINARY STUDY

Figure 3.5 shows the design proposal for a four-span continuous precast, prestressed concrete highway bridge using Tx70 and Texas U54 girders. This bridge is made continuous using in-span splice connections between pier and drop-in girder segments.

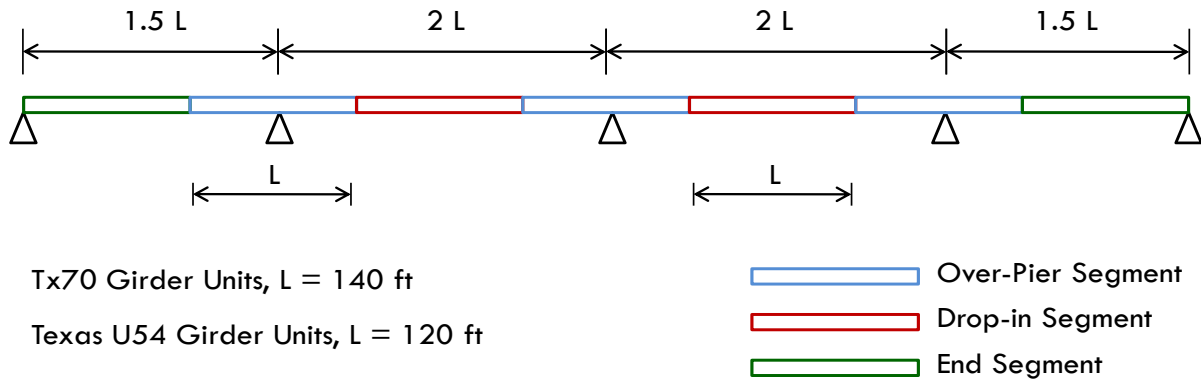


Figure 3.5. Design Proposal for a Continuous Spliced Girder Bridge Using Standard Tx70 and Texas U54 Girders.

The length of an individual girder segment is 140 ft for the Tx70 girder and 120 ft for the U54 girder. For the Tx70 girder, the end spans are 210 ft and central spans are 280 ft in length; and for the U54 girder, the end spans are 180 ft and central spans are 240 ft in length. The bridge is designed according to the *AASHTO LRFD Bridge Design Specifications*. Span lengths of 140 ft and 120 ft for the Tx70 and U54 girders, respectively, have been chosen as this length is considered to be a practical maximum for transportable units. This leads to a lower number of transportation units of the girders and thereby the number of joints. A T501 traffic barrier is used as presented in the Standard Drawings of the *TxDOT Bridge Design Manual*. The nominal face of the rail is 1 ft. The asphalt wearing surface is considered to be 2 in. thick.

3.7 LIMIT STATES AND LOAD COMBINATIONS

The *AASHTO LRFD Bridge Design Specifications* specifies Service III and Fatigue load combinations in addition to Service I and Strength I load combinations for prestressed concrete members. Service III load combination is exclusively applicable to prestressed concrete members to check tensile stresses at the bottom fiber of the girder to prevent cracking. Fatigue load combination is used to check the fatigue of prestressing strands due to repetitive vehicular live load. Fatigue loads and extreme events, such as earthquake loads and vehicle collision loads, are not accounted for in the preliminary designs. The wind load is also not considered as this does not generally govern the design of bridges in Texas.

The applicable load combinations including dead, superimposed and live loads specified in Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications* are outlined as follows.

Service I – checks compressive stresses in prestressed concrete components:

$$Q = 1.00(DC + DW) + 1.00(LL + IM) \quad (3.1)$$

where:

Q = Total load effect.

DC = Self-weight of girder and attachment (slab and barrier) load effect.

DW = Wearing surface load effect.

LL = Live load effect.

IM = Dynamic load effect.

Service III – checks tensile stresses in prestressed concrete components:

$$Q = 1.00(DC + DW) + 0.80(LL + IM) \quad (3.2)$$

Strength I – checks ultimate strength for both flexure and shear:

$$\text{Maximum } Q = 1.25(DC) + 1.5(DW) + 1.75(LL + IM) \quad (3.3)$$

$$\text{Minimum } Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM) \quad (3.4)$$

The maximum load factors generally govern the design. However, the minimum load factors are used for dead load (DC) and wearing surface load (DW) when dead load and wearing surface stresses are opposite to those of the live load.

3.8 ALLOWABLE STRESS LIMITS

The allowable stress limits from the *AASHTO LRFD Bridge Design Specifications* used for the preliminary designs are provided in Table 3.5. The reduction factor ϕ_w , for the compressive stress limit at the final loading stage is taken as equal to 1.0 when the web or flange slenderness ratio, calculated according to the AASHTO LRFD Art. 5.7.4.7.1, is less than or equal to 15. When either the web or flange slenderness ratio is greater than 15, the provisions of the AASHTO LRFD Art. 5.7.4.7.2 are used to calculate the value for the reduction factor ϕ_w (see AASHTO LRFD Art. 5.9.4.2). For a trapezoidal box section such as the Texas U54 girder, which has a variable thickness across the flanges and webs, the *AASHTO LRFD Bridge Design Specifications* outline a general guideline to determine the approximate slenderness ratios for

webs and flanges. For the final loading stage, Case I is applicable for all load combinations, and Case II is applicable for load combinations of ‘Live load + 0.5 x (Effective pretension force + Dead load).’

Table 3.5. Summary of Allowable Stress Limits.

Stage of Loading	Type of Stress	Allowable Stress Limits	
		f'_c or f'_{ci} (ksi)	f'_c or f'_{ci} (psi)
Initial Loading Stage at Transfer	Compressive	$-0.6 f'_{ci}$	$-0.6 f'_{ci}$
	Tensile	$0.24 \sqrt{f'_{ci}}$	$7.59 \sqrt{f'_{ci}}$
Intermediate Loading Stage at Service	Compressive	$-0.45 f'_c$	$-0.45 f'_c$
	Tensile	$0.19 \sqrt{f'_c}$	$6 \sqrt{f'_c}$
Final Loading Stage at Service	Compressive: Case I	$-0.6 \phi_w f'_c$	$-0.6 \phi_w f'_c$
	Compressive: Case II	$-0.4 f'_c$	$-0.4 f'_c$
	Tensile	$0.19 \sqrt{f'_c}$	$6 \sqrt{f'_c}$

3.9 LOADS

Dead load analysis of the girder is performed for the self-weight of the girder, self-weight of the deck slab, weight of the haunch, barrier, and the asphalt wearing surface. The *AASHTO LRFD Bridge Design Specifications* HL93 load model is used for live load analysis of the girder. The maximum moment and shear demand for the continuous girders under live loads is computed considering different load placement schemes. Figure 3.6 presents the critical live load placement on alternate spans of the bridge to determine the maximum moment demand. Figure 3.7 shows the critical load placement for the vehicular live loads to produce maximum shear.

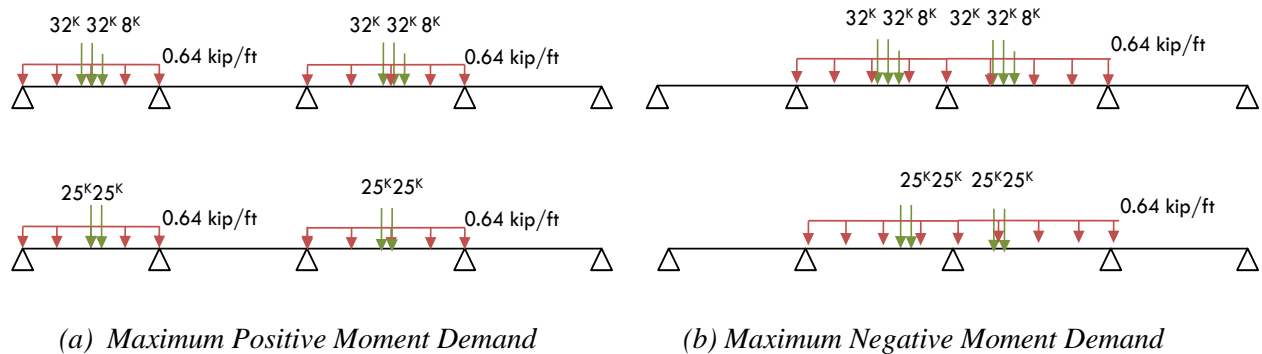


Figure 3.6. Critical Load Placement of HL93 Vehicular Live Load over Continuous Span for Maximum Moment Demand.

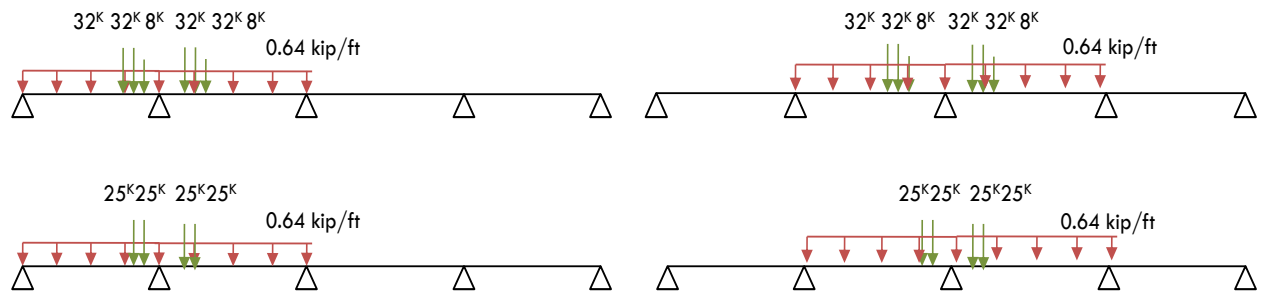


Figure 3.7. Critical Load Placement of HL93 Vehicular Live Load over Continuous Span for Maximum Shear Demand.

3.10 DESIGN PHILOSOPHY ADAPTED

A modified design approach involving the load balancing technique has been used for the preliminary design of the continuous precast, prestressed concrete girders. The girders are designed for service loads and then checked for their ultimate capacity and stresses under live load and impact and temperature stresses. Based on the total bridge width and girder properties, the number of girder lines is determined.

Table 3.6 shows the weights of the girder segments. The availability of the equipment for transporting and lifting the girder segments is considered before selecting the segment dimensions.

Table 3.6. Weights of Girder Segments.

Girder Segments	Tx70 Girder		Texas U54 Girder	
	Length	Weight	Length	Weight
End segment, Over-pier segment, and Drop-in segment	140 ft	161 kips	120 ft	140 kips

Figure 3.8 shows the design moments for pretensioning of the girder segments. The girder segments are pretensioned for a total load of 1.2 times the unfactored self-weight of the girder to:

- Provide a factor of safety of 20 percent for the additional flexural stresses due to transportation and erection.
- Provide allowance for construction loads.

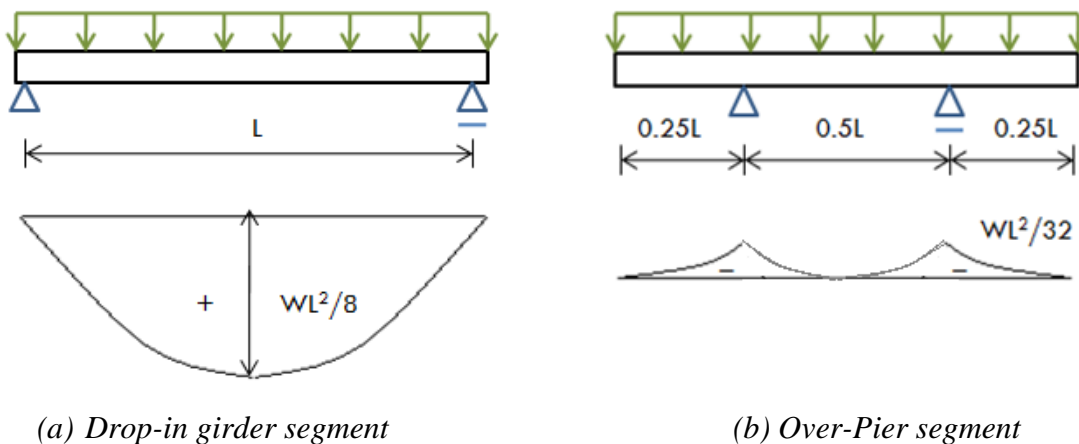


Figure 3.8. Design Moment for Pretensioning of Girders.

Once the girders reach the construction site, it is proposed that post-tensioning operations are carried out in two stages. In the two-stage post-tensioning approach, Stage 1 involves initial post-tensioning to balance the girder self-weight and the construction loads and Stage 2 involves post-tensioning to carry the dead weight of the deck and superimposed dead loads after the placement of the deck. Time-dependent losses and friction losses are considered for calculation of the final forces at transfer to determine the optimum number of tendons required for post-tensioning.

The tendon profiles for the second stage post-tensioning used for the preliminary designs are set to counteract the dead load moments due to the deck slab and superimposed dead loads in the continuous unit. The method is essentially a load-balancing procedure. It reduces the magnitude of the secondary moments to negligible values by providing tendon profiles that are very close to the concordant tendon profile. For the preliminary designs, the low point in the exterior spans is at a distance of 0.4 times the span length from the exterior support and the high point is located directly above the piers (see Figure 3.9). For the Tx70 girder bridge considered in the preliminary designs, the span length ' L_1 ' is equal to 210 ft, and the span length ' L_2 ' is equal to 280 ft. The inflection point in the tendon profile is located at 0.1 times the span length away from the centerline of the interior pier.

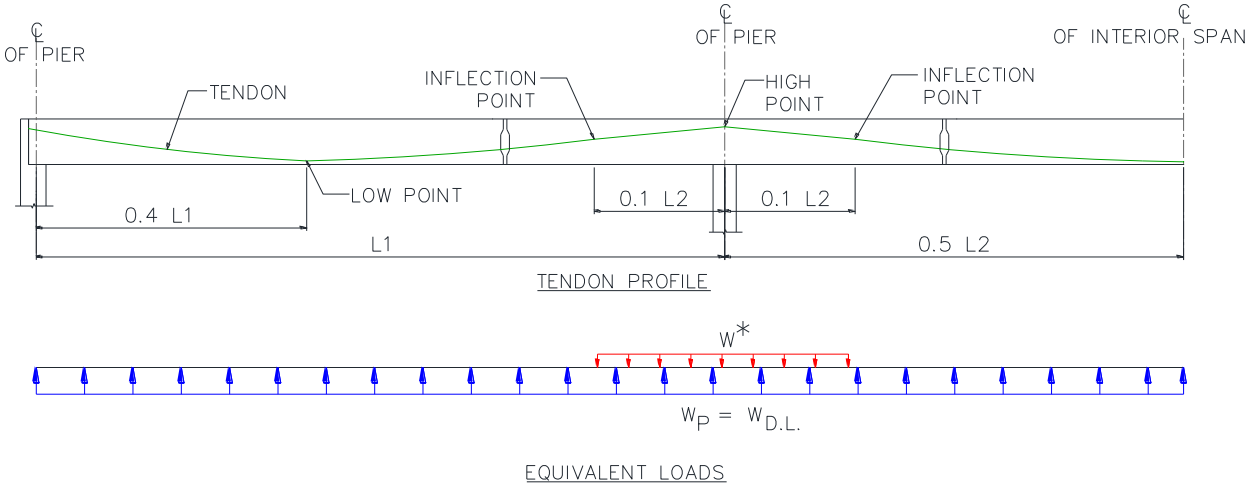


Figure 3.9. Tendon Profile and Secondary Moment Effect.

The tendon profile considered for the Tx70 girder is smooth and parabolic without any sharp curvature to facilitate the tendon placement in the girder formwork and to provide efficient load-balancing. The practical tendon profile for the Tx70 girder over the interior support is draped with a curvature. For a distance equal to 0.1 times the span length away from the centerline of the interior pier, the tendon profile does not follow the theoretical optimal profile scaled to the negative bending moment over the pier. This gives rise to secondary moments producing a downward acting load, W^* (see Figure 3.9). The secondary moment for the Tx70 girder is computed as the force in the tendon multiplied by the difference between the

eccentricity of the draped tendon of the theoretical optimal profile scaled to the negative bending moment and the eccentricity of the practical tendon profile. This moment is considered for checking stresses under service limit state and ultimate strength at the interior support.

The strength limit state is checked to ensure safety at the ultimate load conditions. The flexural strength limit state design requires the reduced nominal moment capacity of the member to be greater than the factored ultimate design moment. The service limit state design of prestressed concrete members typically governs the flexural design. Service stress analysis is carried out for the continuous girders under the total dead loads, prestress force, live loads with impact, and temperature gradient effects.

The prestressed concrete flexural members are reinforced for shear and diagonal tension stresses. Transverse shear design in the *AASHTO LRFD Bridge Design Specifications* is based on a variable angle truss analogy with modified compression strength of concrete commonly known as “Modified Compression Field Theory (MCFT).” The interface shear design in the *AASHTO LRFD Bridge Design Specifications* is based on shear friction theory. Shear displacement along an interface plane is considered to be resisted by cohesion and friction, maintained by the shear friction reinforcement crossing the crack.

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs that could impair serviceability and durability, even if self-limiting and not a potential source of collapse. The maximum deflection in the continuous girders under the vehicular live loads is checked using the *AASHTO LRFD Bridge Design Specifications* Article 2.5.2.6.2 criterion for deflection control.

4. PRELIMINARY DESIGN – TX70 GIRDERS

4.1 INTRODUCTION

The research team developed a preliminary design proposal for the Tx70 prestressed concrete bridge girder. The procedure outlined in Chapter 3 was used to develop the design for the continuous prestressed concrete Tx70 girder bridge. This chapter provides a summary of results of the preliminary designs for the Tx70 bridge girders. Table 4.1 presents the design parameters for the Tx70 bridge girder design used for preliminary designs.

Table 4.1. Design Parameters for Preliminary Designs.

Parameter		Description/Selected Values
Total bridge width		46 ft
Deck slab thickness		8 in.
Unit weight of concrete (CIP), w_{cip} , and (Precast), w_c		0.150 kcf
Unit weight of 2" asphalt wearing surface, w_s		0.140 kcf
Weight of Type T501 rail		326 plf
Specified Concrete Strength at service for deck slab (CIP), f'_c		4 ksi
Maximum Specified Concrete Strength at service (Precast), f'_c		8.5 ksi
Maximum Specified Concrete Strength at release (Precast), f'_{ci}		6.5 ksi
Modulus of Elasticity, E_{cip}		$33,000 w_{cip}^{1.5} \sqrt{f'_c}$
Modulus of Elasticity, E_c		$33,000 w_c^{1.5} \sqrt{f'_c}$
Coefficient of thermal expansion of concrete		$6 \times 10^{-6}/^\circ\text{F}$
Mild steel (ASTM A615 Grade 604)	Yield strength, f_y	60 ksi
	Modulus of Elasticity, E_s	29,000 ksi
Prestressing steel	Strand diameter	0.6 in.
	Ultimate tensile strength, f_{pu}	270 ksi (low relaxation)
	Yield strength, f_{py}	$0.9 f_{pu}$
	Stress limit at transfer, f_{pi}	$f_{pi} \geq 0.75 f_{pu}$
	Stress limit at service, f_{pe}	$f_{pe} \geq 0.8 f_{py}$
	Modulus of Elasticity, E_p	28,500 ksi
	Coefficient of friction, μ	0.25
	Wobble friction coefficient, K	0.0002 /ft of tendon

The requirements for service limit state design, flexural strength limit state design, and shear design are evaluated in this study.

4.2 MOMENT AND SHEAR DEMAND

4.2.1 Dead Load

The dead load analysis of the girder is performed for the self-weight of the girder, self-weight of the deck slab, weight of the haunch, barrier, and the asphalt wearing surface. Table 4.2 presents the dead loads considered for design in this study.

Table 4.2. Dead Loads for Modified Tx70 Girder.

Dead Loads	Tx70 Girder (modified)
Girder Self-weight	1.152 kip/ft
Deck	0.800 kip/ft
Haunch	0.079 kip/ft
Barrier	0.109 kip/ft
Wearing Surface	0.187 kip/ft
Total Weight DC (Structural Components and Non-structural Attachments)	2.140 kip/ft
Total Weight DW (Wearing Surface and Utilities)	0.187 kip/ft

Table 4.3 presents the unfactored dead load moment and shear demand at critical locations for the Tx70 girder bridge.

Table 4.3. Dead Load Moment and Shear Demand for Modified Tx70 Girder.

Critical Location	Moment Demand	Shear Demand
At 0.4 Times Length of End Span	6,950 kip-ft	-
At Interior Support	10,816 kip-ft	331 kips
At Midspan of Interior Span	7,722 kip-ft	-

4.2.2 Live Load

The AASHTO LRFD Specifications HL93 load model is used for the live load analysis of the girder. The live load is to be taken as one of the following combinations, whichever yields maximum stresses at the section considered.

- Design Truck and Design Lane load.
- Design Tandem and Design Lane load.

The maximum moment and shear demand for the continuous girders under live loads is computed considering different load placement schemes. The critical live load placement on the bridge to determine the maximum moment demand and maximum shear demand are as described in Section 3.9 of this report. According to AASHTO LRFD Section 3.6.1.3.1, the maximum shear under the vehicular live load is calculated as the larger of:

- 90 percent of the effect of (Two Design Trucks + Design Lane Load).
- 100 percent of the effect of (Two Design Tandems + Design Lane Load).

The two design trucks or tandems are spaced a minimum of 50 ft between the lead axle of one truck/tandem and the rear axle of the other truck/tandem on either side of the interior support. The two design trucks/tandems shall be placed in adjacent spans to produce maximum force effects. The AASHTO LRFD Article 3.6.2 specifies the dynamic allowance to be taken as 33 percent of the live load effects for all limit states, except the fatigue limit state for which the impact factor is specified as 15 percent of the fatigue load moment. The impact factor is applicable to truck and tandem loads only. The lane load is not increased for any dynamic effects.

Table 4.4 presents the unfactored distributed (live + impact) load moment and shear demand at critical locations for the Tx70 girder bridge.

Table 4.4. Live Load Moment and Shear Demand for Modified Tx70 Girder.

Critical Location	Moment Demand	Shear Demand
At 0.4 Times Length of End Span	4,917 kip-ft	-
At Interior Support	6,253 kip-ft	139 kips
At Midspan of Interior Span	5,121 kip-ft	-

4.2.3 Thermal Gradient

Thermal load analysis is performed for computing the primary and secondary thermal stresses in the girders. The primary thermal stresses are computed using the AASHTO LRFD Section 3.12.3 temperature distribution parameters. Figure 4.1 shows the temperature distribution for the Tx70 girder.

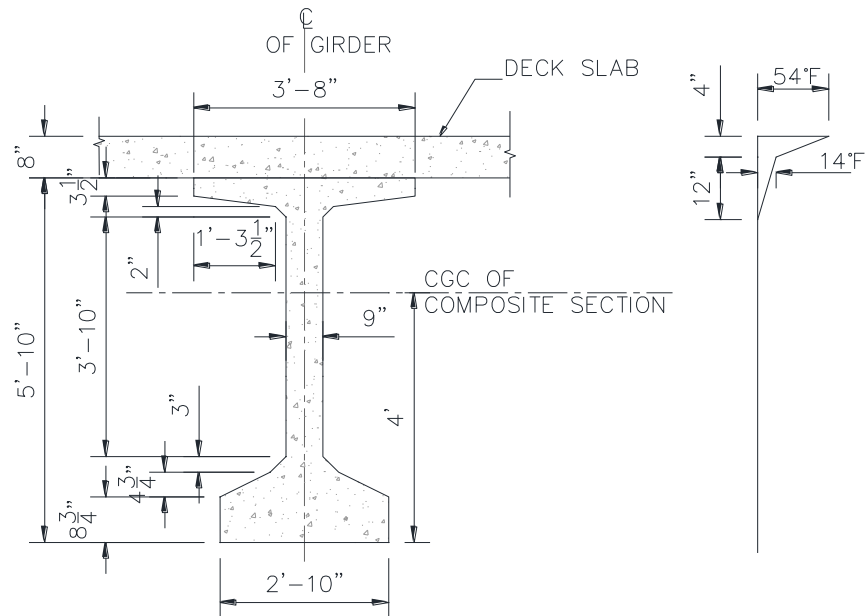


Figure 4.1. Vertical Temperature Gradient for Composite Tx70 Girder (AASHTO LRFD 2010).

From the temperature distribution for the Tx70 girder, the strain at the centroid of the composite section is calculated as follows.

$$\epsilon_0 = \frac{\alpha}{A} \int bT \, dy \quad (4.1)$$

where,

α = Coefficient of thermal expansion of concrete, °F.

A = Area of composite girder section, in.²

b = Width of the section, in.

T = Temperature distribution in the composite section, °F.

Curvature in the section is computed as follows.

$$\phi = \frac{\alpha}{I} \int bTy \, dy \quad (4.2)$$

where,

I = Moment of Inertia of the composite girder section, in.⁴

y = Distance of every layer from top of the composite girder section, in.

Bending strain in the section can then be computed as follows.

$$\varepsilon_b = \varepsilon_0 + \phi y \quad (4.3)$$

The strain due to thermal expansion is given as follows.

$$\varepsilon_t = \alpha T \quad (4.4)$$

The total strain in the composite section can then be calculated as follows.

$$\varepsilon_s = \varepsilon_b - \varepsilon_t \quad (4.5)$$

The primary thermal stress, f_t , for the composite section can then be calculated as

$$f_t = \varepsilon_s E_c \quad (4.6)$$

where,

E_c = Modulus of elasticity of concrete, ksi.

Figure 4.2 shows the primary thermal stresses for the composite Tx70 girder.

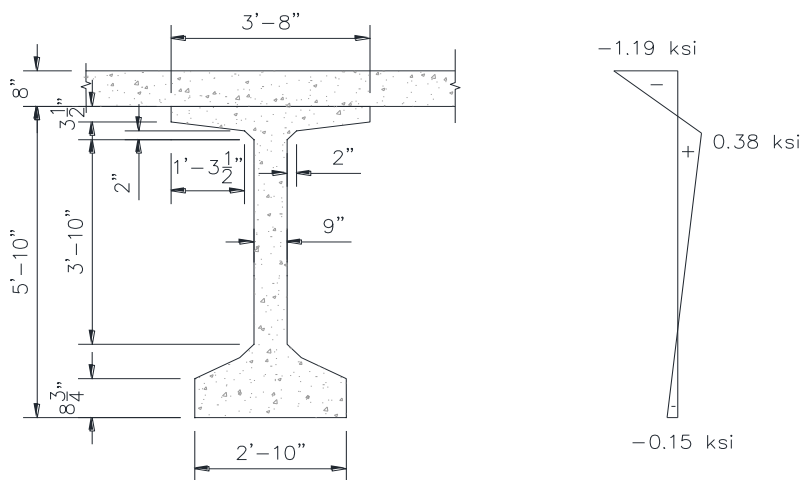


Figure 4.2. Primary Thermal Stresses in the Tx70 Girder Bridge.

Moments are developed in continuous bridges as a result of restraint to the bending caused by primary thermal stresses. These secondary thermal stresses are critical in continuous bridges. Secondary temperature stress analysis is done by applying unit moments at the interior supports (see Figure 4.3).

Moments due to secondary thermal stresses, M_{tsec} , for the Tx70 girder bridge can be computed as follows.

$$M_{tsec} = \eta E_c I \phi \quad (4.7)$$

where,

M_{tsec} = Moment due to secondary thermal stresses, kip-in.

η = Factor determined from analysis, shown in Figure 4.3.

E_c = Modulus of elasticity of concrete, ksi.

I = Moment of Inertia of the composite girder section, in.⁴

ϕ = Curvature in the composite girder section.

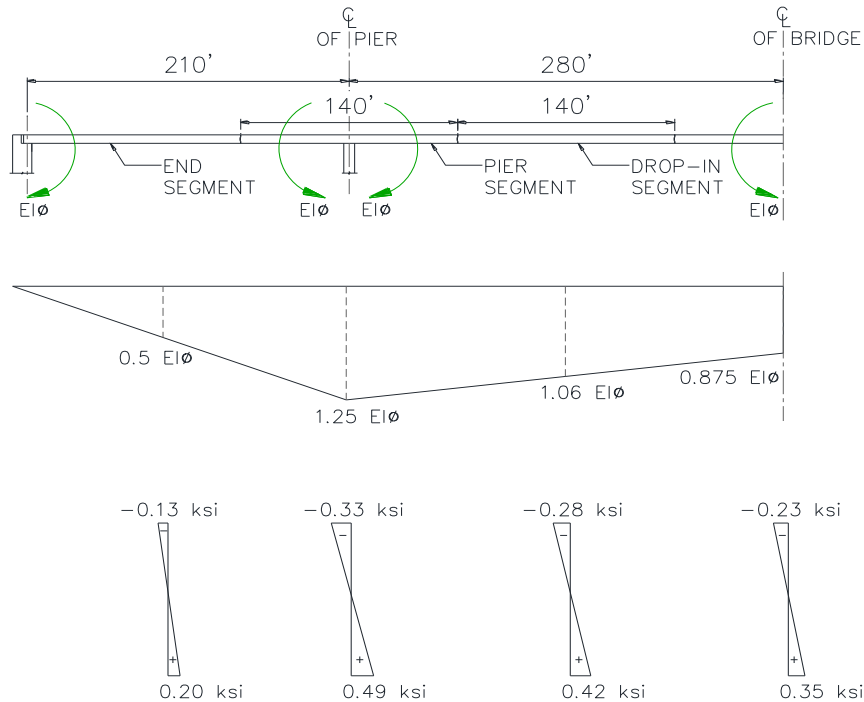


Figure 4.3. Secondary Thermal Stresses in the Tx70 Girder Bridge.

Secondary thermal stress, f_{tsec} , for the composite section can then be calculated as

$$f_{tsec} = \frac{M_{tsec}}{S_x} \quad (4.8)$$

where,

S_x = Modulus of the composite girder section, in.³

Figure 4.4 shows the total thermal stress, f_{tTotal} , at the critical locations in the continuous Tx70 girder bridge. The total thermal stresses, f_{tTotal} , are obtained by the summation of the primary thermal stresses and the secondary thermal stresses at that section, as shown in Figure 4.3, and are calculated as follows.

$$f_{tTotal} = f_t + f_{tsec} \quad (4.9)$$

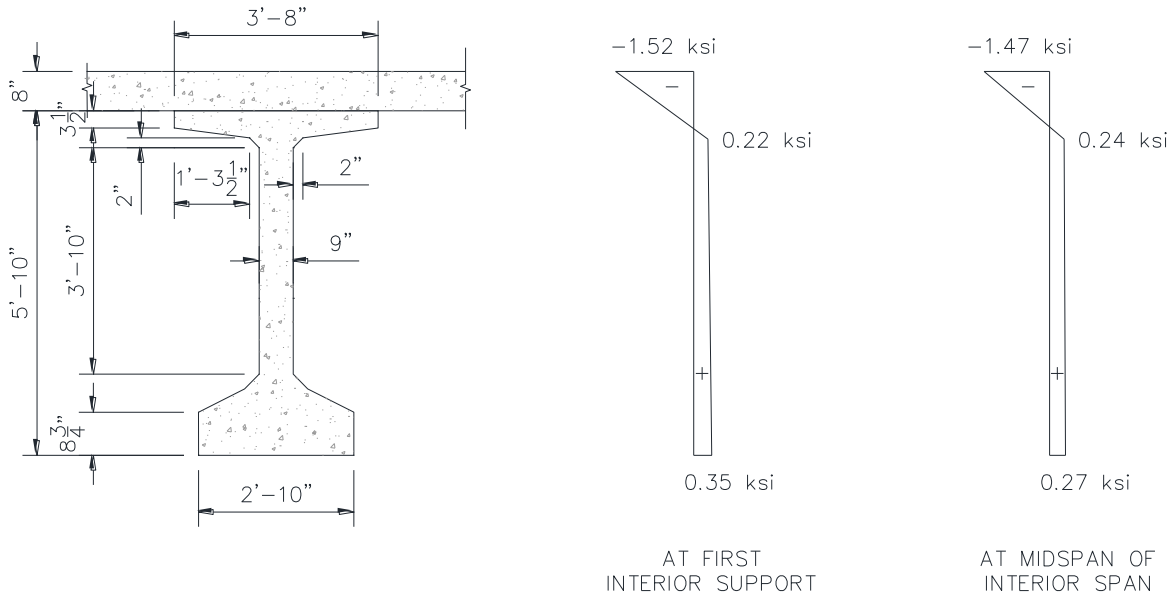


Figure 4.4. Total Thermal Stresses at Critical Locations in the Tx70 Girder Bridge.

4.3 LOAD BALANCING DESIGN

A modified design approach involving the load balancing technique has been used for the preliminary design of the continuous Tx70 prestressed concrete girders. The girders are designed for service loads and then checked for their ultimate capacity and stresses under live load and impact and temperature stresses.

The girder segments are pre-tensioned for a total load of 1.2 times the unfactored self-weight of the girder to provide a factor of safety of 20 percent for the additional flexural stresses due to transportation and erection and to provide allowance for construction loads. For pretensioning of the girder, 0.6 in. dia. low relaxation strands with f_{pu} of 270 ksi are considered. The initial stress in pretensioning strands at transfer f_{pi} is considered to be $0.75 f_{pu}$ (AASHTO LRFD Table 5.9.3-1), which is equal to 202.5 ksi. Time-dependent losses are considered at the final stages of pretensioning. The force at transfer is calculated after taking the losses into account to determine the optimum number of tendons required for pre-tensioning. Figure 4.5 and Table 4.5 presents the pretensioning design for the girder segments.

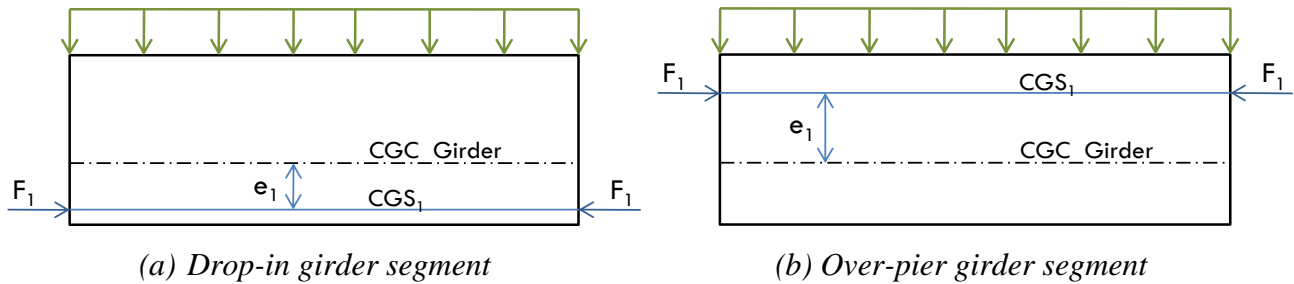


Figure 4.5. Pretensioning Steel Profile for Tx70 Girder Segments.

Table 4.5. Pretensioning Steel Design for Tx70 Girder.

Pretensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	30	26
Force at Transfer, F_{1i} (kips)	1230	1066
Force Final, F_1 (kips)	984	853
Eccentricity, e_1 (in.)	26	34

For transportation and handling purposes of the pier segments, four temporary unbonded Dywidag threadbars of 1.25 in. diameter are provided in the bottom flange of the pier segments. Various corrosion protection systems are available for these threadbars, none of which bond with the structure. Once the pier segment is erected on site, it behaves as a cantilever. These unbonded threadbars can then be released after the segment is safely installed into place. Dywidag

threadbars of 1.25 in. diameter and f_{pu} equal to 150 ksi, are considered for the preliminary designs. Dywidag bars may be stressed to the allowable limits of ACI 318 (2008). The maximum jacking stress shall not exceed $0.80 f_{pu}$, and the transfer stress shall not exceed $0.70 f_{pu}$. The final effective prestress level depends on the specific application. In the absence of a detailed analysis of the structural system, $0.60 f_{pu}$ may be used as an approximation of the effective prestress level as used for this design.

Once the girders reach the construction site, post-tensioning operations are carried out in two stages. In the two stage post-tensioning approach, the girder is initially post-tensioned to balance the girder self-weight and the construction loads and then post-tensioned for continuity to carry the dead weight of the deck and superimposed dead loads after the placement of the deck. Time-dependent losses and friction losses are considered for calculation of the final forces at transfer to determine the optimum number of tendons required for post-tensioning. Figure 4.6 and Table 4.6 present the design for the girder segments after Stage 1 post-tensioning. The stresses in the girder section are checked after Stage 1 post-tensioning so that there is no moment due to eccentricity developed at the ends of the girder.

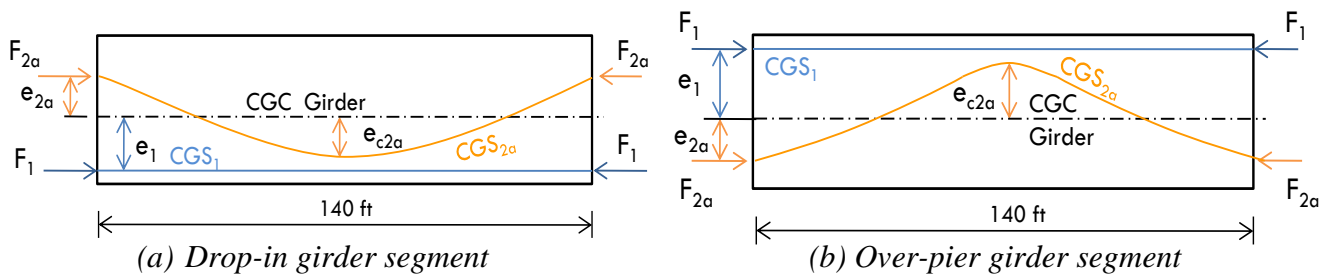


Figure 4.6. Prestress Layout for Tx70 Girder Segments after Stage 1 Post-Tensioning.

Table 4.6. Stage 1 Post-Tensioning Design for Tx70 Girder.

Stage 1 Post-tensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	38	38
Force at Transfer, F_{2ai} (kips)	1559	1559
Force Final, F_{2a} (kips)	1309	1309
Eccentricity, $(e_{c2a} + e_{2a})$ (in.)	26.8	37

Figure 4.7 and Table 4.7 present the design for the girder segments after Stage 2 post-tensioning. The tendon profile considered for the Tx70 girder is smooth and parabolic without any sharp curvature to facilitate the tendon placement in the girder formwork and provide efficient load-balancing. The eccentricity for Stage 2 continuous tendons is defined as the eccentricity of the whole parabolic drape of the tendon profile.

The practical tendon profile for the Tx70 girder over the interior support is draped with a curvature. The secondary moment due to reverse curvature of the tendon profile over the interior pier for the Tx70 girder as described in Section 3.10 of this report is computed as the force in the tendon multiplied by the difference between the eccentricity of the draped tendon of the theoretical optimal profile scaled to the negative bending moment and the eccentricity of the practical tendon profile and is found to be equal to 1473 kip-ft. This moment is considered for checking stresses under service limit state and ultimate strength at the interior support.

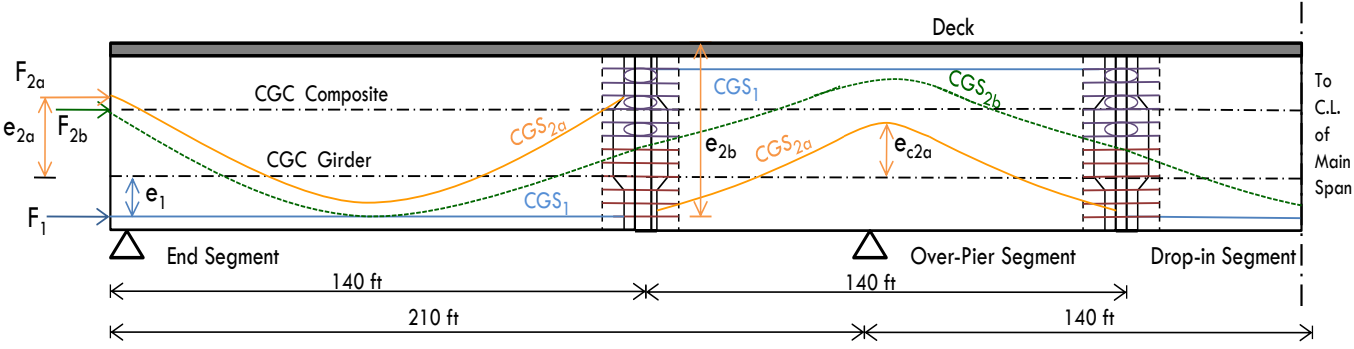


Figure 4.7. Prestress Layout for Tx70 Girder Segments after Stage 2 Post-Tensioning.

Table 4.7. Stage 2 Post-Tensioning Design for Tx70 Girder.

Stage 2 Post-tensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	57	57
Force at Transfer, F_{2bi} (kips)	2338	2338
Force Final, F_{2b} (kips)	1964	1964
Eccentricity, e_{2b} (in.)	66	66

The pier segments are also designed for additional moments due to the removal of temporary support towers used in shored construction. The temporary support removal corresponds to introducing a force on the pier segments that is equal and opposite to the reaction from the drop-in and end segments.

4.4 PRESTRESS LOSSES

The prestress losses are categorized as immediate losses and time-dependent losses. The prestress loss due to initial steel relaxation and elastic shortening are grouped into immediate losses. The prestress loss due to concrete creep, concrete shrinkage, and steel relaxation after transfer are grouped into time-dependent losses.

4.4.1 Elastic Shortening

Elastic shortening occurs only in pretensioned systems at the time of transfer. For the preliminary designs, the following computation is used.

$$\text{For } f'_{ci} = 6.5 \text{ ksi, } E_{ci} = 33,000 (0.150)^{1.5} \sqrt{6.5} = 4887.73 \text{ ksi}$$

$$\text{If } f_{cgp} = 1 \text{ ksi, } \Delta f_{pES} = \frac{28500}{4887.73} (1) = 5.83 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{5.83}{180} \times 100 \approx 3.2\%$$

4.4.2 Steel Relaxation

The relaxation of prestressing strands depends on the type and weaving of strands within a tendon. For the preliminary designs, the following computation is used. The instantaneous loss in prestress due to initial relaxation of steel is particularly prevalent in stressing systems using wedges. As per AASHTO LRFD Specifications, Article 5.9.5.4.2c, for low-relaxation strands, Δf_{pR1} may be assumed as equal to 1.2 ksi.

$$\% \Delta f_{pR1} = \frac{1.20}{180} \times 100 \approx 1\%$$

For low relaxation strands, the prestress loss due to relaxation over a period of time ($T \rightarrow \infty$) typically ranges from 1 percent to 7 percent. Therefore, considering average loss,

$$\% \Delta f_{pR2} \approx 4\%$$

4.4.3 Concrete Creep

The loss in prestress due to creep depends on the girder creep coefficient at the time of deck placement is due to loading introduced at transfer. For the preliminary designs, the following computation is used.

$$\text{For } f'_{ci} = 6.5 \text{ ksi, } E_{ci} = 33,000 (0.150)^{1.5} \sqrt{6.5} = 4887.73 \text{ ksi}$$

C_T = Time-dependent creep coefficient

$$C_{T \rightarrow \infty} = 2$$

$$E_{eff} = \frac{E_{ci}}{1+C_T} = \frac{4887.73}{1+2} = 1629.24 \text{ ksi}$$

Assuming $f_{cgp} = 1 \text{ ksi}$ and $\Delta f_{cdp} = 1 \text{ ksi}$,

$$\Delta f_{pCR} = \frac{E_p}{E_{eff}} f_{cgp} - \frac{E_p}{E_{ci}} \Delta f_{cdp} = 11.66 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{11.66}{180} \times 100 \approx 6\%$$

4.4.4 Concrete Shrinkage

The loss in prestress due to concrete shrinkage depends on the average humidity at the bridge site, W/C ratio, aggregate characteristics and proportions, and duration of drying period.

For the preliminary designs, the following computation is used.

$$H = 65 \%$$

$$\Delta f_{pCR} = 17 - 0.15(65) = 7.25 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{7.25}{180} \times 100 \approx 4\%$$

4.4.5 Instantaneous Losses

The instantaneous losses include the loss of prestress due to elastic shortening and initial relaxation of steel.

$$\% \Delta f_{pi} = \frac{(5.83+1.2)}{180} \times 100 \approx 4\%$$

4.4.6 Time-Dependent Losses

The time-dependent losses include the loss of prestress due to concrete creep, concrete shrinkage, and steel relaxation after transfer.

$$\% \Delta f_{pLT} = (4 + 6 + 4) \approx 15\%$$

4.4.7 Friction Losses

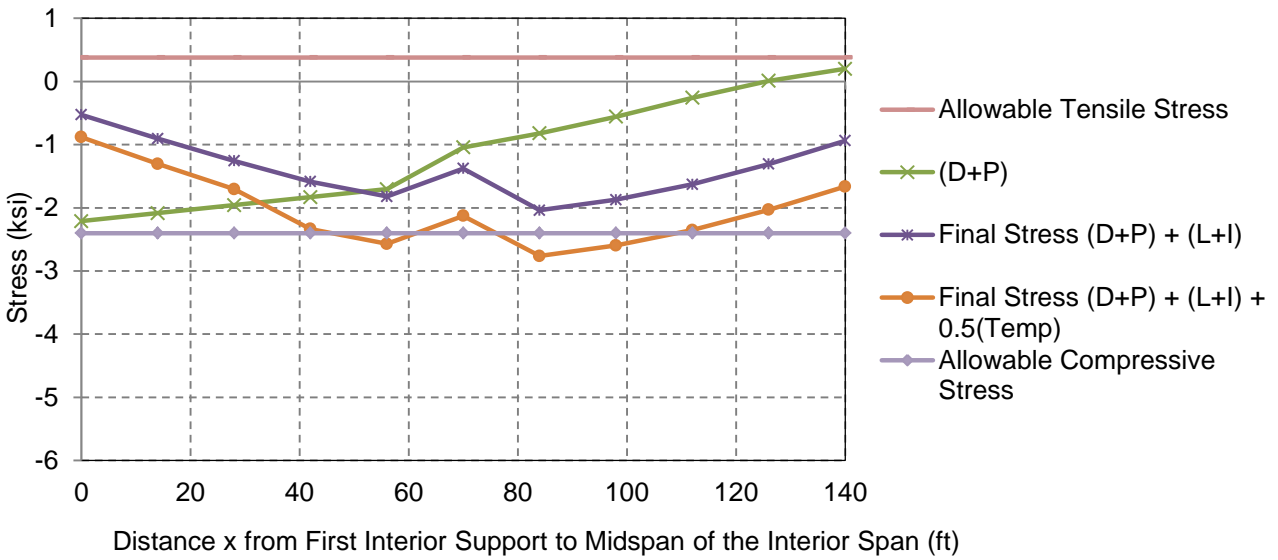
The loss in prestress due to friction between internal post-tensioning tendons and the duct (Δf_{pF}) depends on the wobble and curvature loss of the tendons in the duct.

$$\% \Delta f_{pF} = 15\% \text{ for Stage 1 Post-tensioning and } 30\% \text{ for Stage 2 Post-tensioning}$$

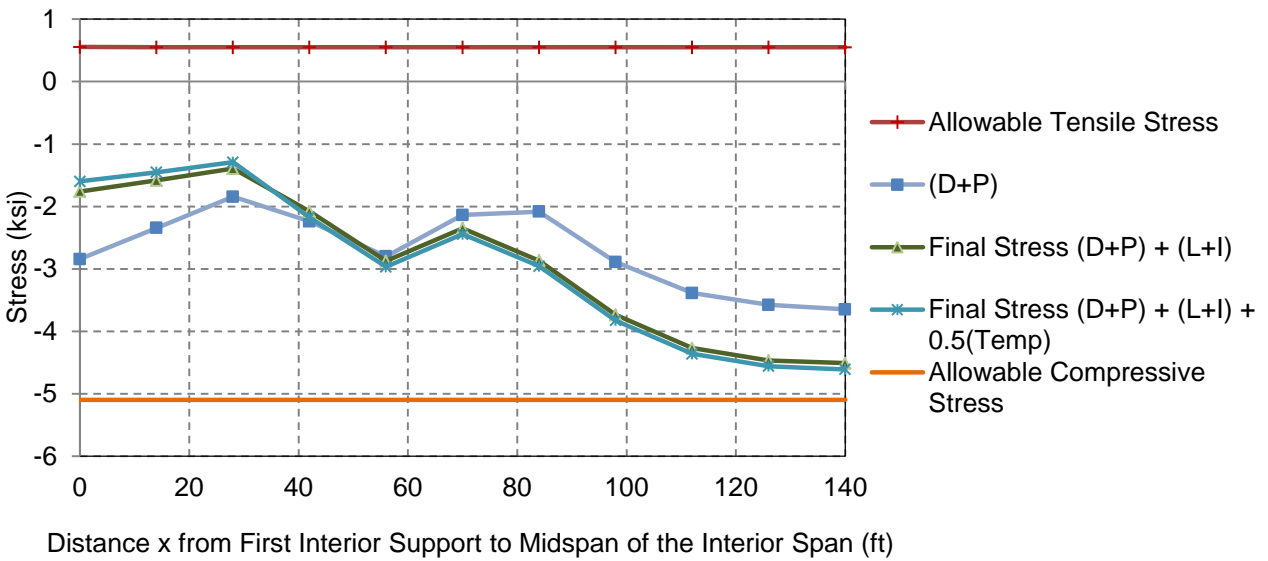
The losses due to friction can be mitigated by prestressing operations such as jacking from both the ends and by adopting understress/overstress operations.

4.5 SERVICE STRESS ANALYSIS

Service stress analysis is carried out for the continuous girders under the total dead loads, prestress force, live loads with impact and temperature gradient effects. The effect of the stresses due to the secondary moment arising from the curvature of the tendon profile over the support is also considered. The stresses are checked at every 20 ft along the span locations and especially at critical locations such as the interior support and the midspan of the interior span. The stresses are checked against the permissible values for the service limit state after losses as specified in AASHTO LRFD Article 5.9.4.2. The allowable compressive and tensile stress limits are specified for the three loading stages provided in Table 3.5 of this report. Compression in prestressed concrete girders is evaluated through the Service I limit state, and tension in the prestressed concrete girders is evaluated through the Service III limit state with the objective of crack control. Figure 4.8 shows the service stresses for the continuous Tx70 girder bridge under different service load combinations.

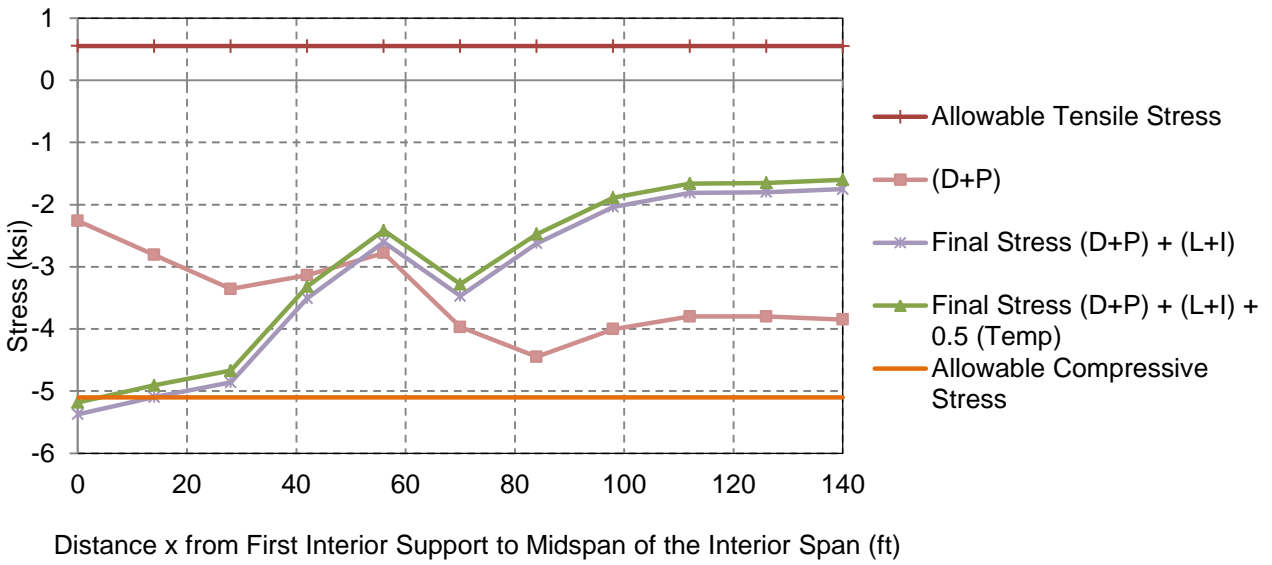


(a) Stresses at Top of CIP Deck

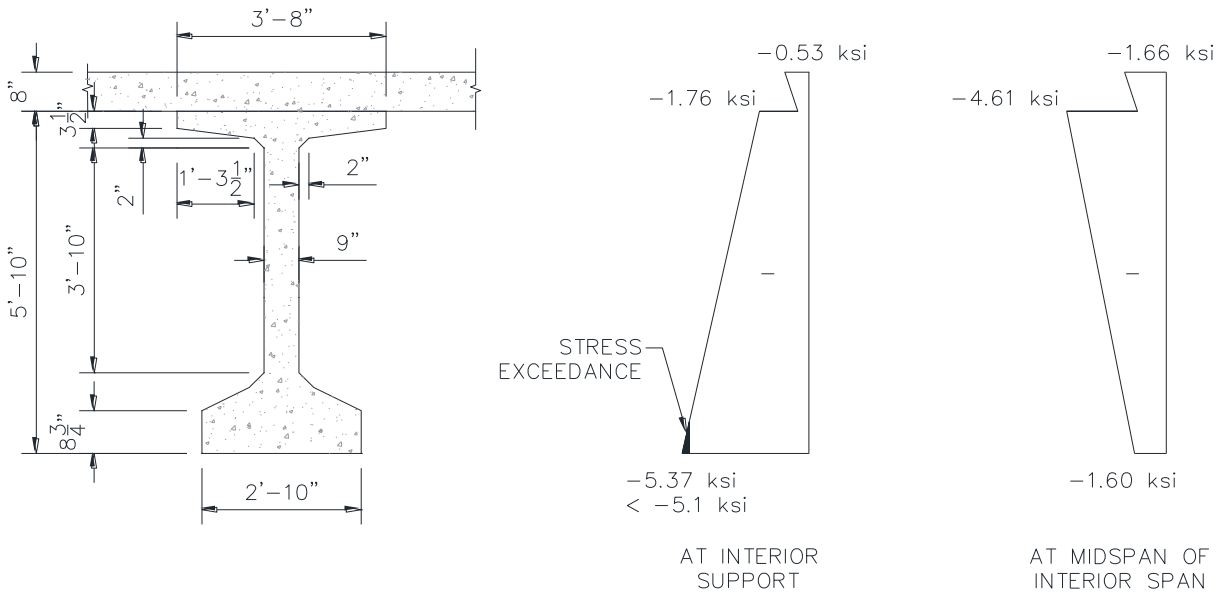


(b) Stresses at Top of Precast Girder

Figure 4.8. Service Stress Analysis for Continuous Prestressed Tx70 Girder Bridge.



(c) Stresses at Bottom of Precast Girder



(d) Final Stresses at Interior Support and Midspan

Figure 4.8. Service Stress Analysis for Continuous Prestressed Tx70 Girder Bridge (continued).

The compressive stresses in the girder soffit at the interior support in the negative moment region were exceeded due to the large amount of post-tensioning tendons in the section (see Fig. 4.8[c]). This stress exceedance may be addressed by increasing the specified concrete compressive strength to stay within the allowable compressive stress limit. Another option that is sometimes employed is to provide additional mild steel reinforcement in the compression zone. The amount of mild steel reinforcement is determined based on the force corresponding to the stress exceedance, shown in Figure 4.8(d). For this design, four #9 bars are added in the bottom flange of the girder to improve the nominal capacity of the section as specified in the ultimate strength check. This additional mild steel reinforcement is also adequate to serve as reinforcement in the girder soffit at the interior support over the pier for the computed stress exceedance at service load conditions.

4.6 ULTIMATE STRENGTH CHECK

The flexural strength limit state design requires the reduced nominal moment capacity of the member to be greater than the factored ultimate design moment. The effect of the secondary moment that occurs due to the curvature of the tendon profile over the support is also considered. This secondary moment is unfactored and is added to the factored load combination in the strength limit state to determine the maximum moment demand at the negative moment region over the support. The moment capacity of the girder is calculated based on the number, location, and stress in the tendons. The design capacity of the girders is calculated at three locations:

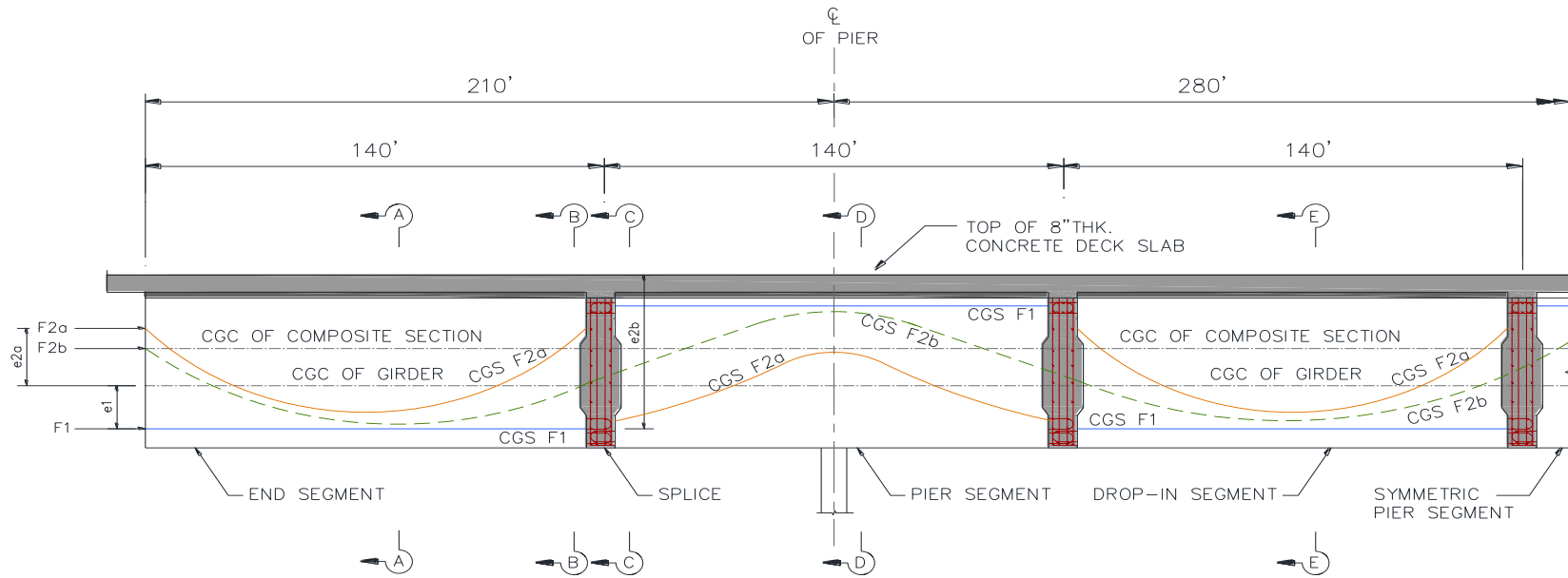
- At $0.4L$ for the maximum positive moment in the end span.
- At the face of the diaphragm at the support.
- At the midspan of the interior span.

The design moment capacity of the girders is calculated considering a rectangular section behavior if the depth of the neutral axis of the composite section lies within the depth of the deck slab. Flanged section behavior is considered when the depth of the neutral axis greater than the depth of the deck slab. The final step is to ensure that the capacity is greater than the demand. Table 4.8 shows the ultimate demand and capacity of the continuous Tx70 girder. If the flexural strength limit state is not satisfied, the capacity of the section is strengthened by providing additional mild steel.

Table 4.8. Ultimate Demand and Capacity for Tx70 Girder.

Capacity and Demand	At Maximum Positive Moment Location		At Maximum Negative Moment Location
	Exterior Span	Interior Span	
Applied Demand, M_u (kip-ft)	16,753	18,614	25,937
Available Capacity, ϕM_n (kip-ft)	20,983	20,983	27,423

The moment capacity that the pretensioning and post-tensioning tendons provide in the maximum negative moment region at the interior support is supplemented by adding mild steel reinforcement. In this case, mild tension steel reinforcement is provided in the deck slab and girder flanges to ensure capacity greater than demand at that location. For this design, eight #9 bars are added in the deck slab and four #9 bars are added in the bottom flange of the girder to provide the additional capacity and balance the moment demand at the interior support over the pier. The mild steel reinforcement provided in the bottom flange acts as compression steel. Figure 4.9 shows the final detailed design of the continuous spliced precast, prestressed concrete Tx70 girder bridge.



LONGITUDINAL SECTION SHOWING THE TENDON PROFILE FOR PRE-TENSIONING AS WELL AS POST-TENSIONING STAGES

NOTE: - HORIZONTAL AND VERTICAL SCALES ARE DIFFERENT. VERTICAL SCALE IS EXAGGERATED FOR CLARITY.

Figure 4.9. Design Details for Continuous Prestressed Tx70 Girder.

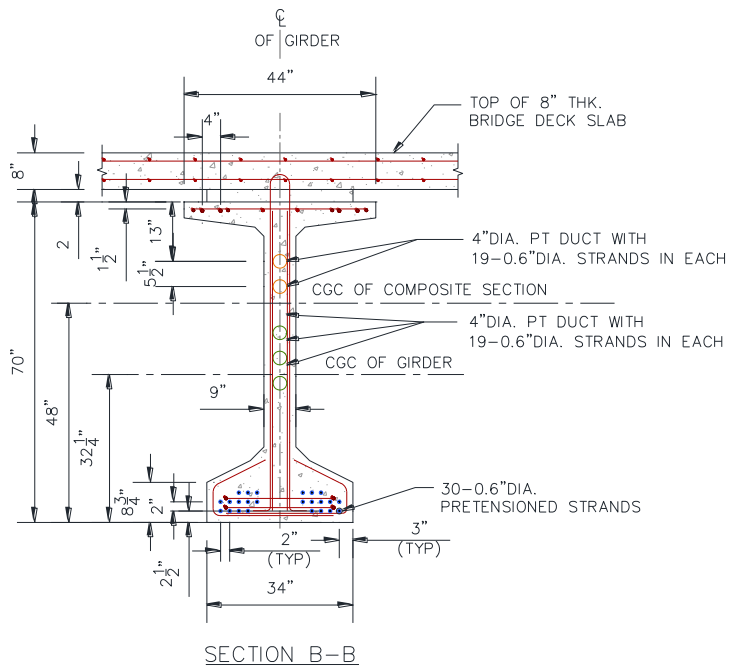
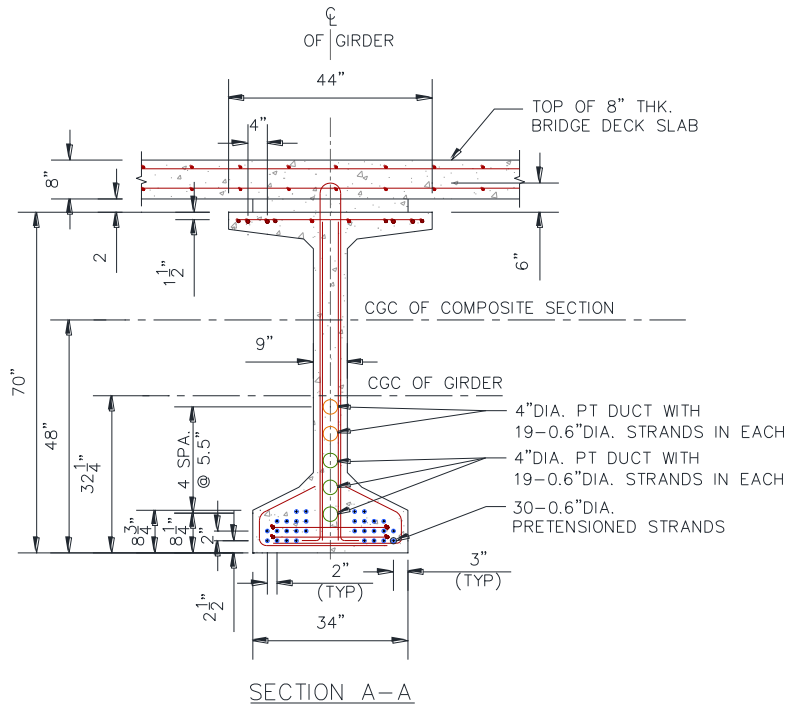


Figure 4.9. Design Details for Continuous Prestressed Tx70 Girder (continued).

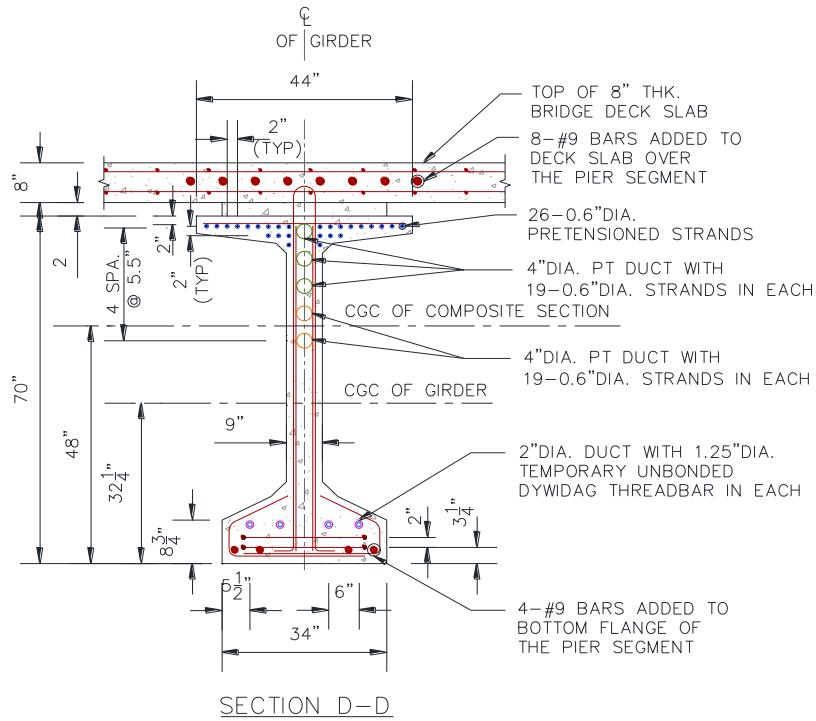
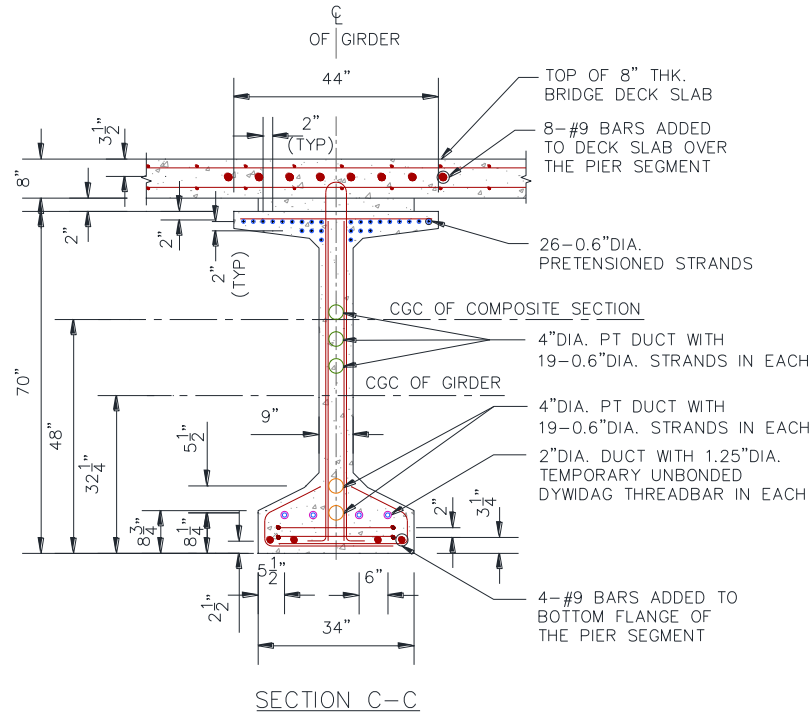


Figure 4.9. Design Details for Continuous Prestressed Tx70 Girder (continued).

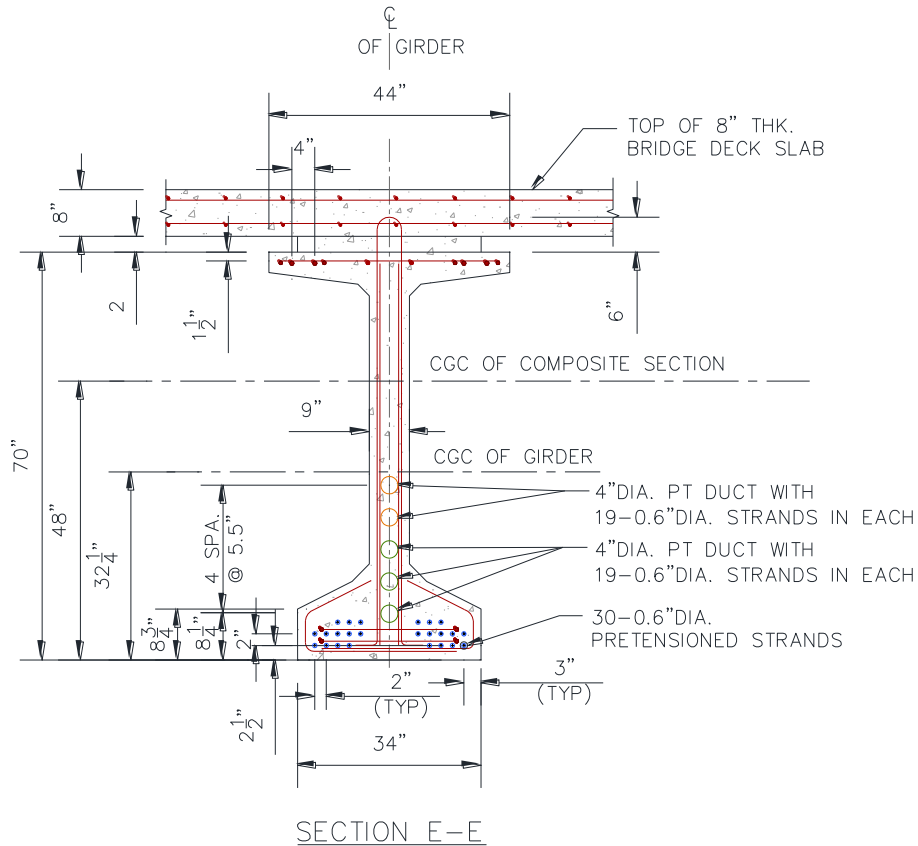


Figure 4.9. Design Details for Continuous Prestressed Tx70 Girder (continued).

4.7 SHEAR DESIGN

4.7.1 Transverse Shear Design

The Modified Compression Field Theory (MCFT) as specified in the *AASHTO LRFD Bridge Design Specifications* is used for the transverse shear design. The MCFT takes into account different factors such as strain condition of the section and shear stress in the concrete to predict the shear strength of the section. The shear strength of concrete is approximated based on a parameter β . The critical section for shear is calculated based on the angle of inclination of the diagonal compressive stress, θ . The critical section for shear near the supports is taken as the larger value of $0.5d_v \cot \theta$ or d_v , measured from the face of the support. The effective shear depth, d_v is calculated as minimum of the distance of the resultants of tensile and compressive forces, 0.9 times the effective depth and 0.72 times the depth of the composite section.

Figure 4.10 shows the transverse shear demand and design for the Tx70 girder using the *AASHTO LRFD Bridge Design Specifications*. Each listed stirrup size and dimension refers to the bar size in a double legged configuration (two transverse reinforcing bars through the web in the cross-section of the girder as shown in Figure 4.9), along with the on-center (o.c.) distance between stirrups along the girder length. The graph shows that:

- #5 double legged stirrups at 4 in. o.c. in each girder web will provide adequate shear strength for a distance of 30 ft on either side of the support.
- #5 double legged stirrups at 6 in. o.c. in each girder web will provide adequate shear strength for a distance of 30 ft to 80 ft from the supports.
- #5 double legged stirrups at 12 in. o.c. in each girder web will provide adequate shear strength for the remainder of the span.

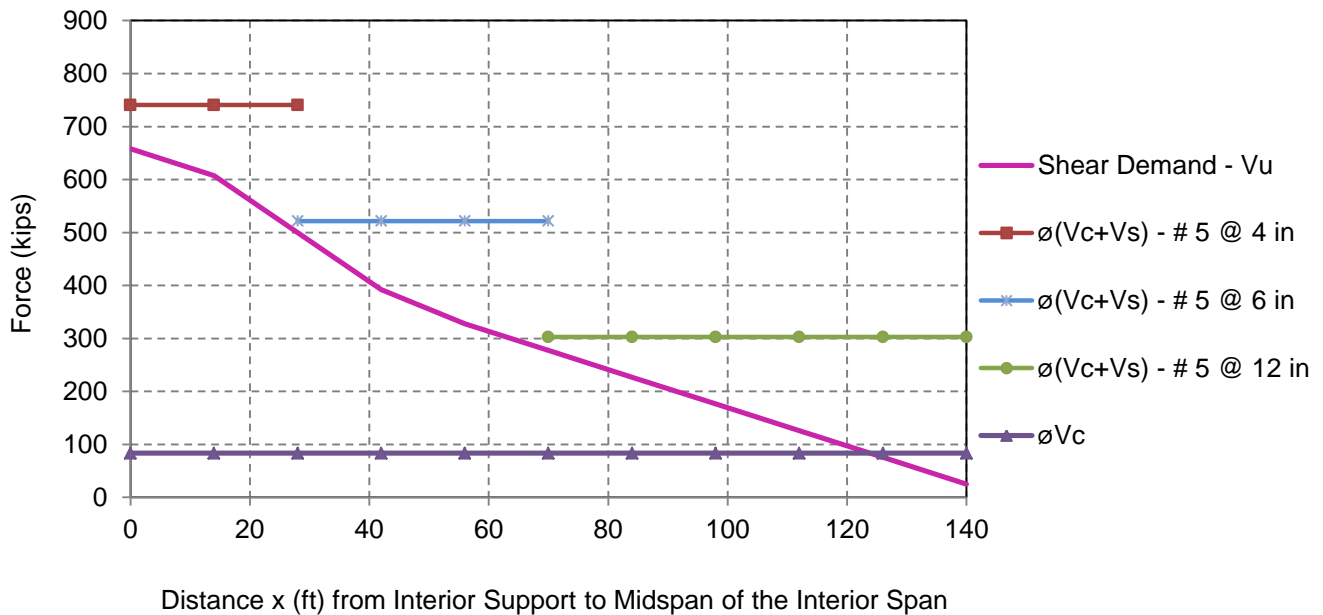


Figure 4.10. Transverse Shear Demand and Design for Tx70 Girder.

4.7.2 Interface Shear Design

The interface shear design as specified in AASHTO LRFD Article 5.8.4. is based on shear friction theory. The nominal shear resistance of the interface plane is based on the cohesion factor, c , friction factor, μ , and the area of concrete engaged in interface shear transfer, A_{cv} . For

preliminary designs, the case of normal-weight concrete placed against a clean concrete surface, free of laitance, with the surface intentionally roughened to an amplitude of 0.25 in. is used. The values of parameters c , μ , K_1 , and K_2 are as follows.

$$c = 0.24 \text{ ksi}$$

$$\mu = 1.0$$

$$K_1 = 0.25$$

$$K_2 = 1.5 \text{ ksi}$$

According to AASHTO LRFD Specifications Article 5.8.4.4, the minimum interface shear reinforcement may be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. In this case, the factored interface shear stress should be less than 0.21 ksi, and all transverse (vertical) shear reinforcement extended across the interface and adequately anchored in the slab. With respect to the girder/slab interface, the transverse shear reinforcement extended into the deck slab also serves as interface shear reinforcement.

For preliminary designs, the transverse shear design reinforcement as shown in Figure 4.10 when extended into the deck slab and bent to 180 degrees serves as interface shear reinforcement and is found to be adequate to resist the horizontal shear demand. Figure 4.11 shows the interface shear demand and design for the Tx70 girder using the *AASHTO LRFD Bridge Design Specifications*.

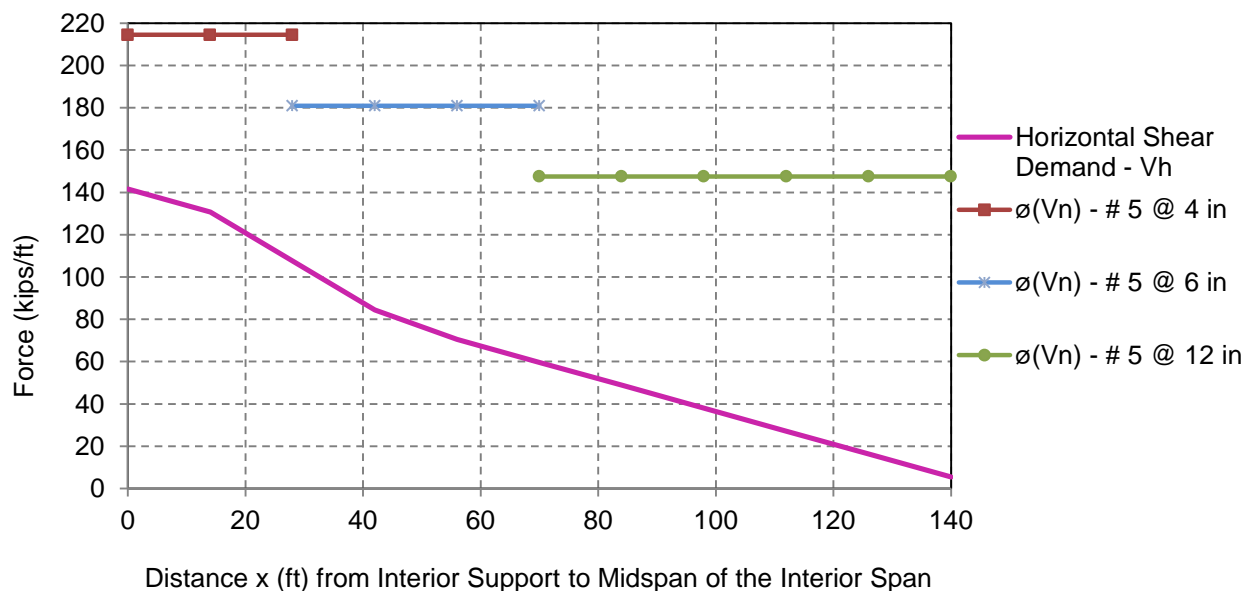


Figure 4.11. Interface Shear Demand and Design for Tx70 Girder.

Figure 4.12 shows the detailed drawing of shear reinforcement for the Tx70 girder.

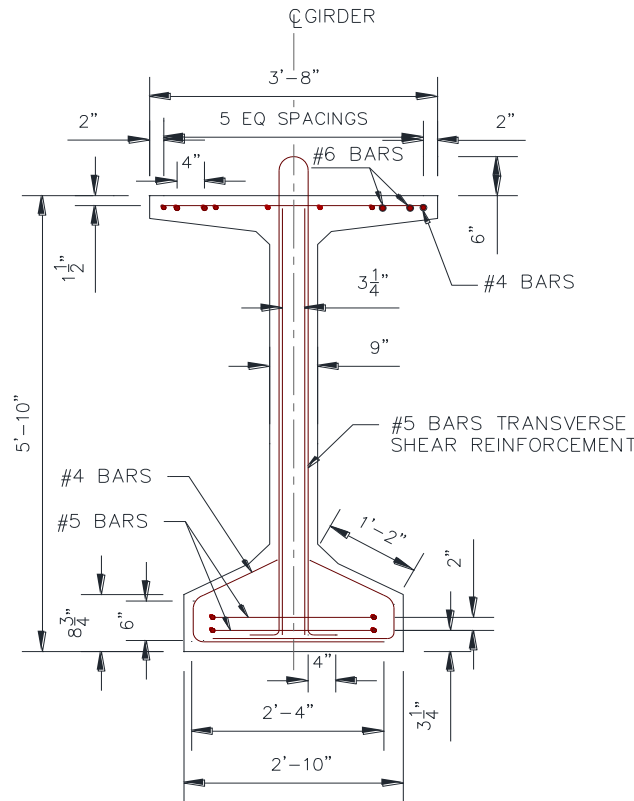


Figure 4.12. Shear Reinforcement Detail for Tx70 Girder (Adapted from TxDOT 2010).

4.8 DEFLECTION CHECK

For preliminary designs, the four-span continuous bridge is designed for a total of two traffic lanes in accordance with the design criteria specified in the *AASHTO LRFD Bridge Design Specifications*. The Tx70 girder bridge has a main span of 280 ft and end spans on either side of 210 ft. According to AASHTO LRFD Specifications Article 2.5.2.6.2, the composite bending stiffness of the girders is considered and all supporting components are assumed to deflect equally. Deflection is calculated under the larger of (i) Design Truck Load alone, or (ii) 25 percent of Design Truck Load and full Design Lane Load according to AASHTO LRFD Specifications Article 3.6.1.3.2. For the preliminary designs, case (ii) causes maximum deflection.

Figure 4.13 shows the critical load arrangement for the vehicular live loads and the lane load to produce maximum deflection in the four span continuous girders. The dynamic load allowance factor is multiplied to the vehicular live loads. The resultant of the truck point loads, ‘R’ is placed at $0.4L_{ext}$ of the exterior span length and at $0.5L_{int}$ of the interior span length.

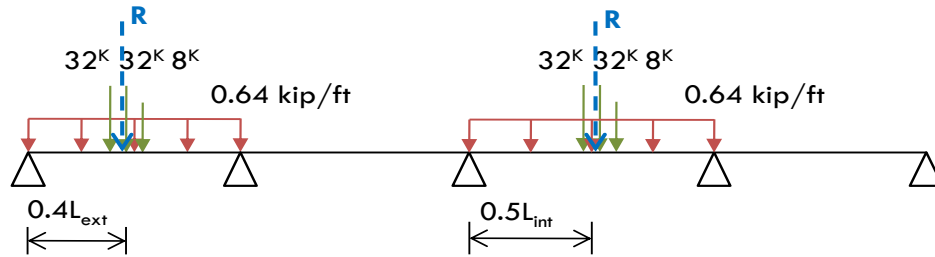


Figure 4.13. Critical Live Load Arrangement for Maximum Deflection of the Tx70 Girder Bridge.

Table 4.9 presents the allowable and actual values of maximum deflection for the four span continuous bridge girders considering two traffic lanes loaded. It can be observed that the deflection is within the allowable limits.

Table 4.9. Maximum Deflection for Tx70 Girder Bridge.

Deflection	Exterior Span	Interior Span
Allowable (in.)	3.15	4.20
Actual (in.)	1.60	2.80

5. PRELIMINARY DESIGN – TEXAS U54 GIRDERS

5.1 INTRODUCTION

The research team developed a preliminary design proposal for the Texas U54 prestressed concrete bridge girder. The procedure outlined in Chapter 3 was used to develop the continuous prestressed concrete girder design for the Texas U54 girder bridge. This chapter provides a summary of results of the preliminary designs for the Texas U54 bridge girders. Table 5.1 presents the design parameters for the Texas U54 bridge girder design used for preliminary designs.

Table 5.1. Design Parameters for Preliminary Designs.

Parameter		Description/Selected Values
Total bridge width		46 ft
Deck slab thickness		8 in.
Unit weight of concrete (CIP), w_{cip} , and (Precast), w_c		0.150 kcf
Unit weight of 2" asphalt wearing surface, w_s		0.140 kcf
Weight of Type T501 rail		326 plf
Specified Concrete Strength at service for deck slab (CIP), f'_c		4 ksi
Maximum Specified Concrete Strength at service (Precast), f'_c		8.5 ksi
Maximum Specified Concrete Strength at release (Precast), f'_{ci}		6.5 ksi
Modulus of Elasticity, E_{cip}		$33,000 w_{cip}^{1.5} \sqrt{f'_c}$
Modulus of Elasticity, E_c		$33,000 w_c^{1.5} \sqrt{f'_c}$
Modular Ratio, n		E_c/E_{cip}
Coefficient of thermal expansion of concrete		$6 \times 10^{-6}/^{\circ}\text{F}$
Mild steel (ASTM A615 Grade 604)	Yield strength, f_y	60 ksi
	Modulus of Elasticity, E_s	29,000 ksi
Prestressing steel	Strand diameter	0.6 in.
	Ultimate tensile strength, f_{pu}	270 ksi (low relaxation)
	Yield strength, f_{py}	$0.9 f_{pu}$
	Stress limit at transfer, f_{pi}	$f_{pi} \geq 0.75 f_{pu}$
	Stress limit at service, f_{pe}	$f_{pe} \geq 0.8 f_{py}$
	Modulus of Elasticity, E_p	28,500 ksi
Coefficient of friction, μ		0.25

This study evaluates the requirements for service limit state design, flexural strength limit state design and shear design.

5.2 MOMENT AND SHEAR DEMAND

5.2.1 Dead Load

The dead load analysis of the girder is performed for the self-weight of the girder, self-weight of the deck slab, weight of the haunch, barrier, and the asphalt wearing surface. Table 5.2 presents the dead loads considered for design in this study.

Table 5.2. Dead Loads for Texas U54 Girder.

Dead Loads	Texas U54 Girder
Girder Self-weight	1.167 kip/ft
Deck	0.920 kip/ft
Haunch	0.049 kip/ft
Barrier	0.109 kip/ft
Wearing Surface	0.215 kip/ft
Total Weight DC (Structural Components and Non-structural Attachments)	2.245 kip/ft
Total Weight DW (Wearing Surface and Utilities)	0.215 kip/ft

Table 5.3 presents the unfactored dead load moment and shear demand at critical locations for the Texas U54 girder bridge.

Table 5.3. Dead Load Moment and Shear Demand for Texas U54 Girder.

Critical Location	Moment Demand	Shear Demand
At 0.4 Times Length of End Span	5,189 kip-ft	-
At Interior Support	9,510 kip-ft	334 kips
At Midspan of Interior Span	5,766 kip-ft	-

5.2.2 Live Load

The AASHTO LRFD Specifications HL93 load model is used for the live load analysis of the girder. The live load is to be taken as one of the following combinations, whichever yields maximum stresses at the section considered.

- Design Truck and Design Lane load.
- Design Tandem and Design Lane load.

The maximum moment and shear demand for the continuous girders under live loads is computed considering different load placement schemes. The critical live load placement on the bridge to determine the maximum moment demand and maximum shear demand are as described in Section 3.9 of this report. According to AASHTO LRFD Section 3.6.1.3.1, the maximum shear under the vehicular live load is calculated as the larger of:

- 90 percent of the effect of (Two Design Trucks + Design Lane Load).
- 100 percent of the effect of (Two Design Tandems + Design Lane Load).

The two design trucks or tandems are spaced a minimum of 50 ft between the lead axle of one truck/tandem and the rear axle of the other truck/tandem on either side of the interior support. The two design trucks/tandems shall be placed in adjacent spans to produce maximum force effects. The AASHTO LRFD Article 3.6.2 specifies the dynamic allowance to be taken as 33 percent of the live load effects for all limit states, except the fatigue limit state for which the impact factor is specified as 15 percent of the fatigue load moment. The impact factor is applicable to truck and tandem loads only. The lane load is not increased for any dynamic effects.

Table 5.4 presents the unfactored distributed (live + impact) load moment and shear demand at critical locations for the Texas U54 girder bridge.

Table 5.4. Live Load Moment and Shear Demand for Texas U54 Girder.

Critical Location	Moment Demand	Shear Demand
At 0.4 Times Length of End Span	3,205 kip-ft	-
At Interior Support	5,428 kip-ft	134 kips
At Midspan of Interior Span	3,561 kip-ft	-

5.2.3 Thermal Gradient

Thermal load analysis is performed for computing the primary and secondary thermal stresses in the girders. The primary thermal stresses are computed using the AASHTO LRFD Section 3.12.3 temperature distribution parameters. Figure 5.1 shows the temperature distribution for the Texas U54 girder.

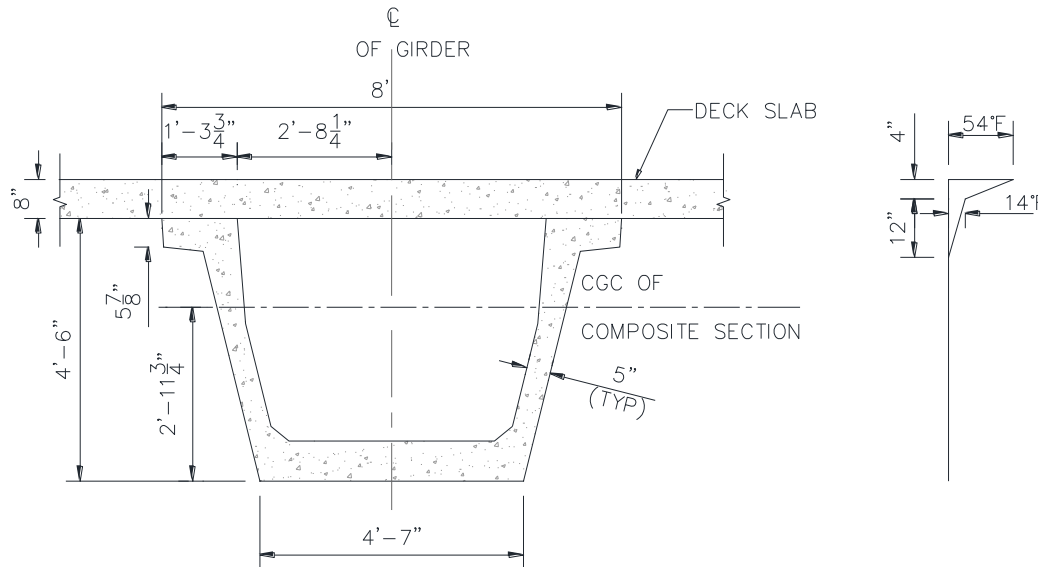


Figure 5.1. Vertical Temperature Gradient for Composite Texas U54 Girder (AASHTO LRFD 2010).

Figure 5.2 shows the primary thermal stresses for the composite Texas U54 girder. Section 4.2.3 of this report presents the equations for calculating the primary thermal stresses. Moments are developed in continuous bridges as a result of restraint to the bending caused by primary thermal stresses. These secondary thermal stresses are critical in continuous bridges. Figure 5.3 shows the secondary temperature stress analysis is done by applying unit moments at the interior supports. Section 4.2.3 of this report presents the equations for calculating the secondary thermal stresses.

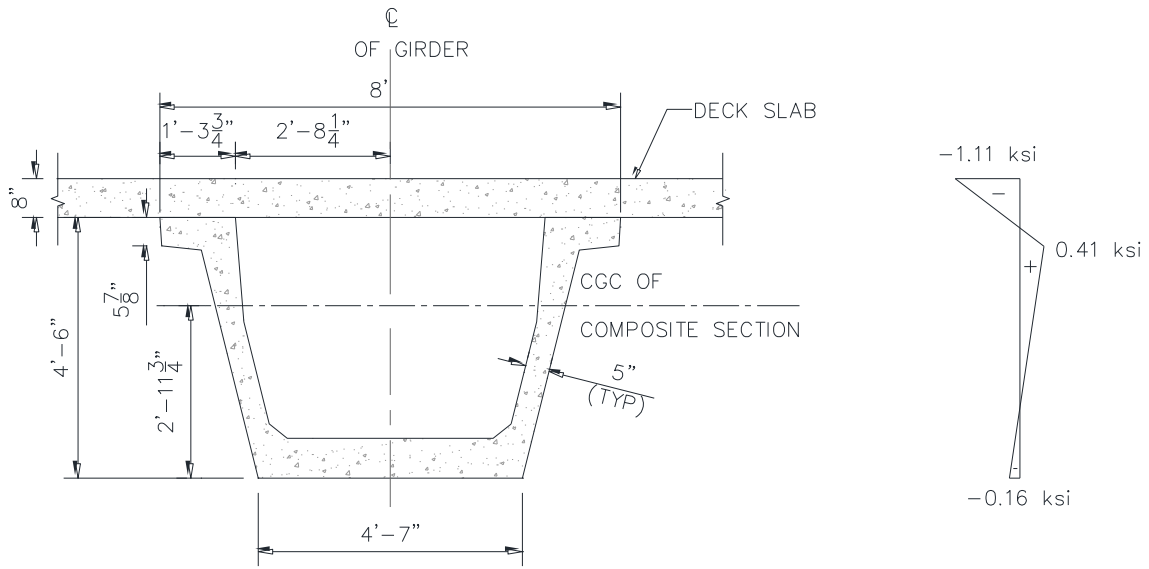


Figure 5.2. Primary Thermal Stresses in the Texas U54 Girder Bridge.

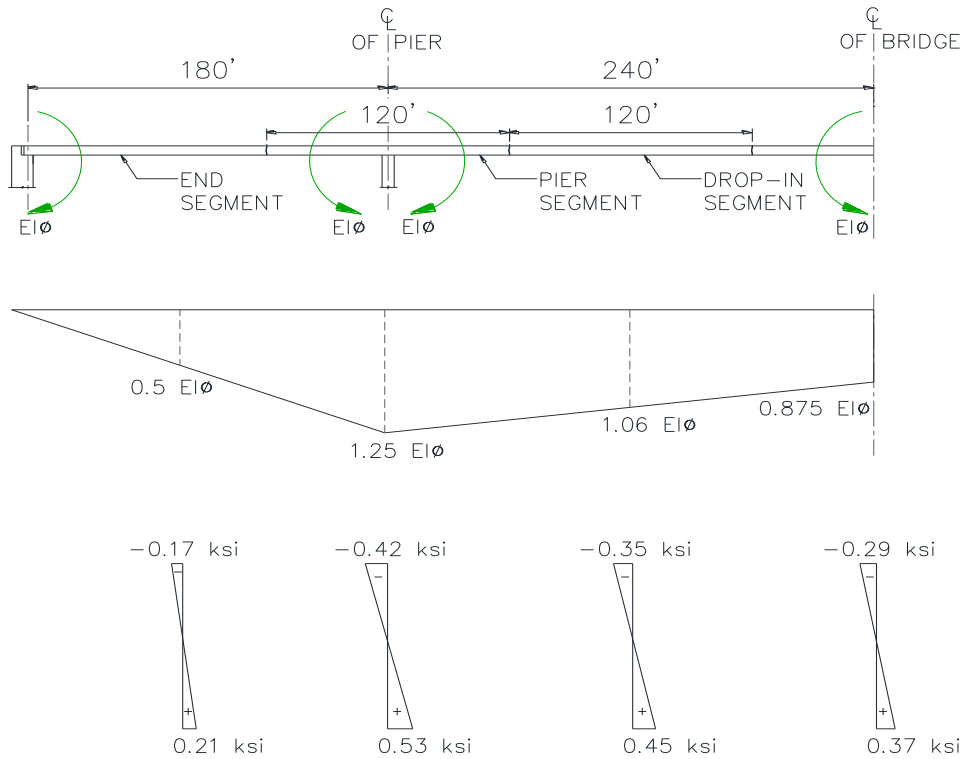


Figure 5.3. Secondary Thermal Stresses in the Texas U54 Girder Bridge.

Figure 5.4 shows the total thermal stress at the critical locations in the continuous Texas U54 girder bridge. The total thermal stresses are obtained by the summation of the primary thermal stresses and the secondary thermal stresses at that section and are calculated as presented in Section 4.2.3 of this report.

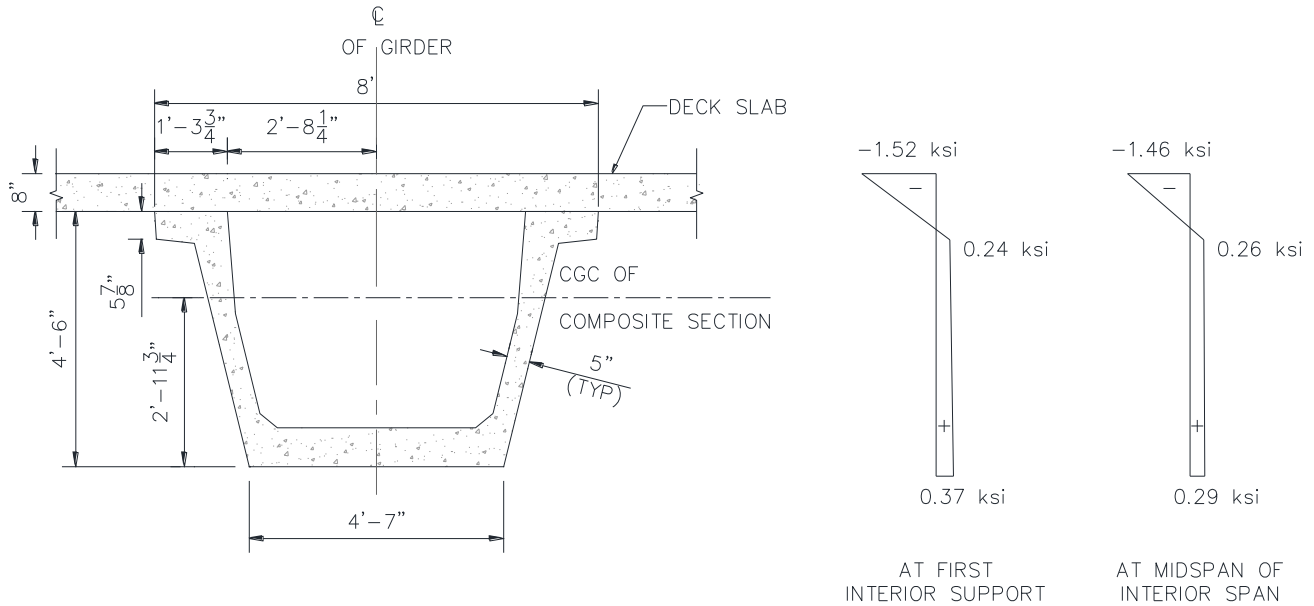


Figure 5.4. Total Thermal Stresses at Critical Locations in the Texas U54 Girder Bridge.

5.3 LOAD BALANCING DESIGN

A modified design approach involving the load balancing technique has been used for the preliminary design of the continuous Texas U54 prestressed concrete girders. The girders are designed for service loads and then checked for their ultimate capacity and stresses under live load and impact and temperature stresses.

The girder segments are pretensioned for a total load of 1.2 times the unfactored self-weight of the girder to provide a 20 percent safety factor for the additional flexural stresses due to transportation and erection, and to provide allowance for construction loads. For pretensioning of the girder, 0.6 in. dia. low-relaxation strands with f_{pu} of 270 ksi are considered. The initial stress in pretensioning strands at transfer f_{pi} is considered to be $0.75 f_{pu}$ (AASHTO LRFD Table 5.9.3-1), which is equal to 202.5 ksi. Time-dependent losses are considered at the final stages of pretensioning. The force at transfer is calculated after taking the losses into

account to determine the optimum number of tendons required for pre-tensioning. Figure 5.5 and Table 5.5 present the pretensioning design for the girder segments.

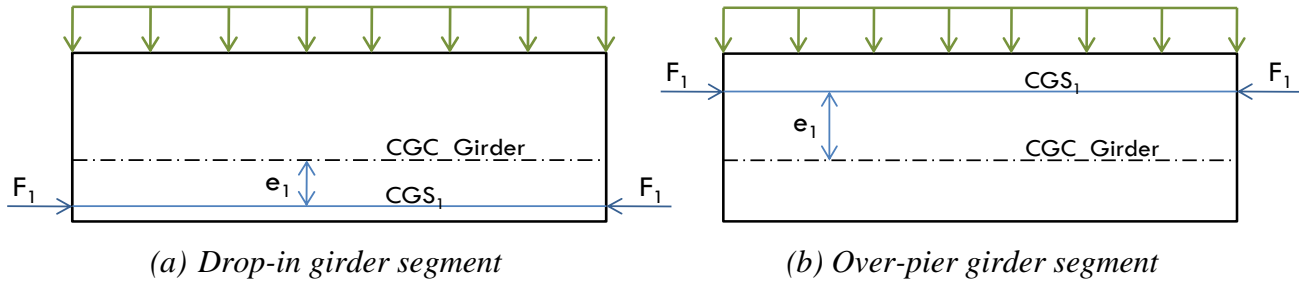


Figure 5.5. Pretensioning Steel Profile for Texas U54 Girder Segments.

Table 5.5. Pretensioning Steel Design for Texas U54 Girder.

Pretensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	28	26
Force at Transfer, F_{Ti} (kips)	1148	1066
Force Final, F_l (kips)	919	853
Eccentricity, e_l (in.)	16	24

For transportation and handling purposes of the pier segments, four temporary unbonded Dywidag threadbars of 1.25 in. diameter are provided in the bottom flange of the pier segments. Various corrosion protection systems are available for these threadbars, none of which bond with the structure. Once the pier segment is erected on site, it behaves as a cantilever. These unbonded threadbars can then be released after the segment is safely installed into place. Dywidag threadbars of 1.25 in. diameter and f_{pu} equal to 150 ksi, are considered for the preliminary designs. Dywidag bars may be stressed to the allowable limits of ACI 318 (2008). The maximum jacking stress shall not exceed $0.80 f_{pu}$, and the transfer stress shall not exceed $0.70 f_{pu}$. The final effective prestress level depends on the specific application. In the absence of a detailed analysis of the structural system, $0.60 f_{pu}$ may be used as an approximation of the effective prestress level as used for this design.

Once the girders reach the construction site, post-tensioning operations are carried out in two stages. In the two-stage post-tensioning approach, the girder is initially post-tensioned to balance the girder self-weight and the construction loads. Then, it is post-tensioned to carry the dead weight of the deck and superimposed dead loads after the placement of the deck. Time-dependent losses and friction losses are considered for calculation of the final forces at transfer to determine the optimum number of tendons required for post-tensioning. Figure 5.6 and Table 5.6 present the design for the girder segments after Stage 1 post-tensioning. The stresses in the girder section are checked after Stage 1 post-tensioning so that there is no moment due to eccentricity developed at the ends of the girder. Figure 5.7 and Table 5.7 present the design for the girder segments after Stage 2 post-tensioning. Eccentricity for Stage 2 continuous tendons is defined as eccentricity of the whole drape of the tendon profile (see Figure 5.7).

The pier segments are also designed for additional moments due to the removal of temporary support towers used in shored construction. The temporary support removal corresponds to introducing a force on the pier segments that is equal and opposite of the reaction from the drop-in and end segments.

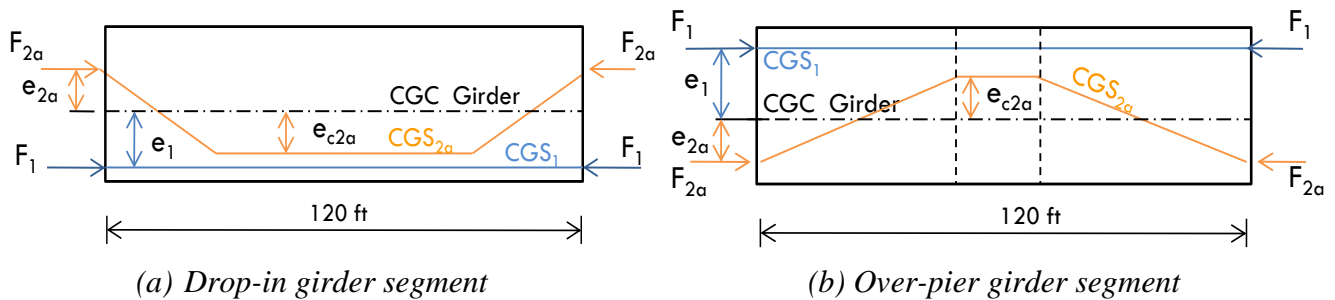


Figure 5.6. Prestress Layout for Texas U54 Girder Segments after Stage 1 Post-Tensioning.

Table 5.6. Stage 1 Post-Tensioning Design for Texas U54 Girder.

Stage 1 Post-tensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	28	38
Force at Transfer, F_{2ai} (kips)	1148	1559
Force Final, F_{2a} (kips)	965	1,309
Eccentricity, $(e_{c2a} + e_{2a})$ (in.)	25.6	26.6

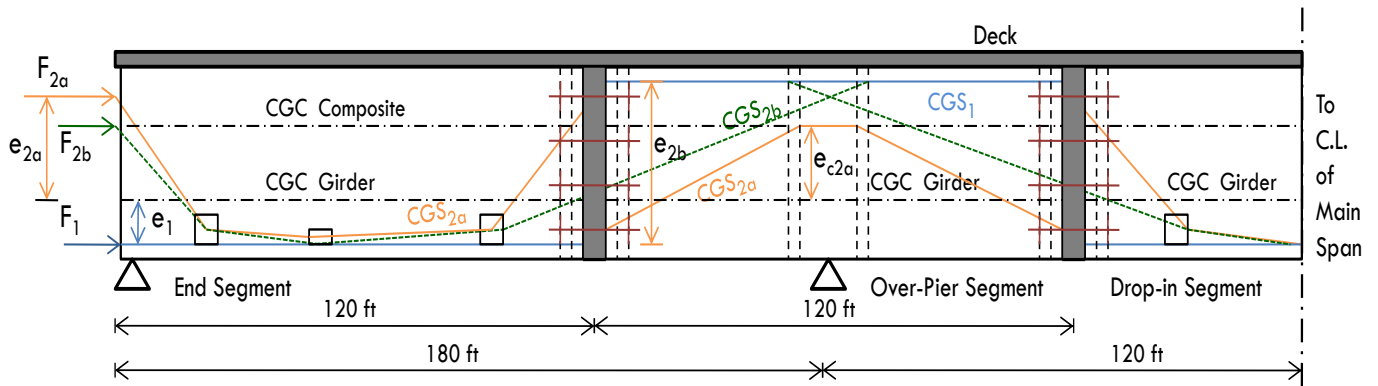


Figure 5.7. Prestress Layout for Texas U54 Girder Segments after Stage 2 Post-Tensioning.

Table 5.7. Stage 2 Post-Tensioning Design for Texas U54 Girder.

Stage 2 Post-tensioning	Drop-in Segment	Pier Segment
Strands (0.6 in. diameter)	76	76
Force at Transfer, F_{2bi} (kips)	3117	3117
Force Final, F_{2b} (kips)	2618	2618
Eccentricity, e_{2b} (in.)	38.3	38.3

5.4 PRESTRESS LOSSES

The prestress losses are categorized as immediate losses and time-dependent losses. The prestress loss due to initial steel relaxation and elastic shortening are grouped into immediate losses. The prestress loss due to concrete creep, concrete shrinkage, and steel relaxation after transfer are grouped into time-dependent losses.

5.4.1 Elastic Shortening

Elastic shortening occurs only in pretensioned systems at the time of transfer. For the preliminary designs, the following computation is used.

$$\text{For } f'_{ci} = 6.5 \text{ ksi, } E_{ci} = 33,000 (0.150)^{1.5} \sqrt{6.5} = 4887.73 \text{ ksi}$$

$$\text{If } f_{cgp} = 1 \text{ ksi, } \Delta f_{pES} = \frac{28500}{4887.73} (1) = 5.83 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{5.83}{180} \times 100 \approx 3.2\%$$

5.4.2 Steel Relaxation

The relaxation of prestressing strands depends on the type and weaving of strands within a tendon. For the preliminary designs, the following computation is used. The instantaneous loss in prestress due to initial relaxation of steel is particularly prevalent in stressing systems using wedges. As per AASHTO LRFD Specifications, Article 5.9.5.4.2c, for low-relaxation strands, Δf_{pR1} may be assumed as equal to 1.2 ksi.

$$\% \Delta f_{pR1} = \frac{1.20}{180} \times 100 \approx 1\%$$

For low relaxation strands, the prestress loss due to relaxation over a period of time ($T \rightarrow \infty$) typically ranges from 1 percent to 7 percent. Therefore, considering average loss,

$$\% \Delta f_{pR2} \approx 4\%$$

5.4.3 Concrete Creep

The loss in prestress due to creep depends on the girder creep coefficient at the time of deck placement due to loading introduced at transfer. For the preliminary designs, the following computation is used.

$$\text{For } f'_{ci} = 6.5 \text{ ksi, } E_{ci} = 33,000 (0.150)^{1.5} \sqrt{6.5} = 4887.73 \text{ ksi}$$

C_T = Time-dependent creep coefficient

$$C_{T \rightarrow \infty} = 2$$

$$E_{eff} = \frac{E_{ci}}{1+C_T} = \frac{4887.73}{1+2} = 1629.24 \text{ ksi}$$

Assuming $f_{cgp} = 1$ ksi and $\Delta f_{cdp} = 1$ ksi,

$$\Delta f_{pCR} = \frac{E_p}{E_{eff}} f_{cgp} - \frac{E_p}{E_{ci}} \Delta f_{cdp} = 11.66 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{11.66}{180} \times 100 \approx 6\%$$

5.4.4 Concrete Shrinkage

The loss in prestress due to concrete shrinkage depends on the average humidity at the bridge site, W/C ratio, aggregate characteristics and proportions and duration of drying period.

For the preliminary designs, the following computation is used.

$$H = 65 \%$$

$$\Delta f_{pCR} = 17 - 0.15(65) = 7.25 \text{ ksi}$$

$$\% \Delta f_{pES} = \frac{7.25}{180} \times 100 \approx 4\%$$

5.4.5 Instantaneous Losses

The instantaneous losses include the loss of prestress due to elastic shortening and initial relaxation of steel.

$$\% \Delta f_{pi} = \frac{(5.83+1.2)}{180} \times 100 \approx 4\%$$

5.4.6 Time-Dependent Losses

The time-dependent losses include the loss of prestress due to concrete creep, concrete shrinkage and steel relaxation after transfer.

$$\% \Delta f_{pLT} = (4 + 6 + 4) \approx 15\%$$

5.4.7 Friction Losses

The loss in prestress due to friction between the external tendon across a single deviator pipe, (Δf_{pF}) is found to be as follows.

$$\% \Delta f_{pF} = 4\% \text{ for Stage 1 Post-tensioning, and}$$

$$5\% \text{ for Stage 2 Post-tensioning}$$

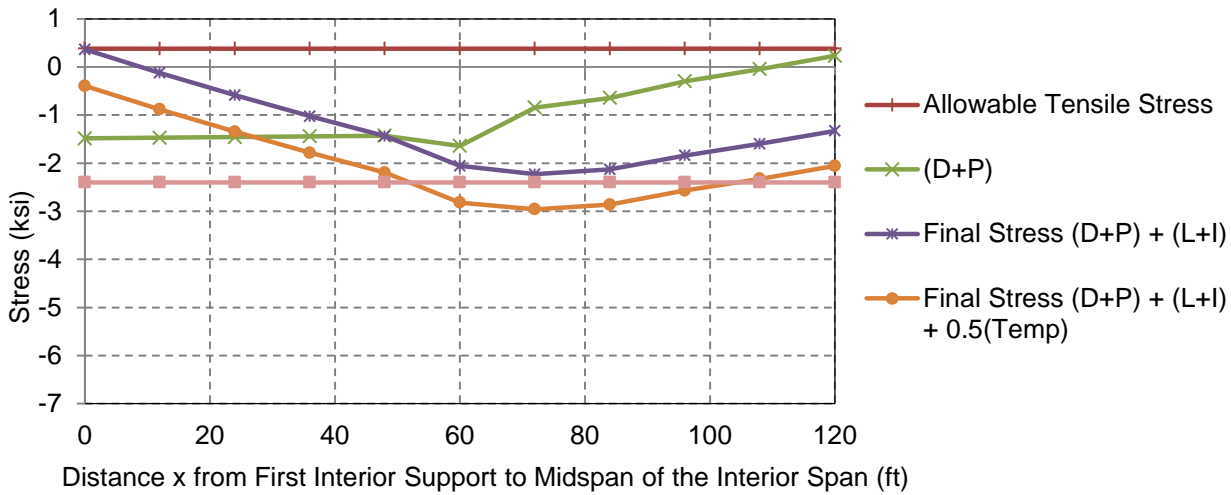
The losses due to friction can be mitigated by adopting appropriate prestressing operations.

5.5 SERVICE STRESS ANALYSIS

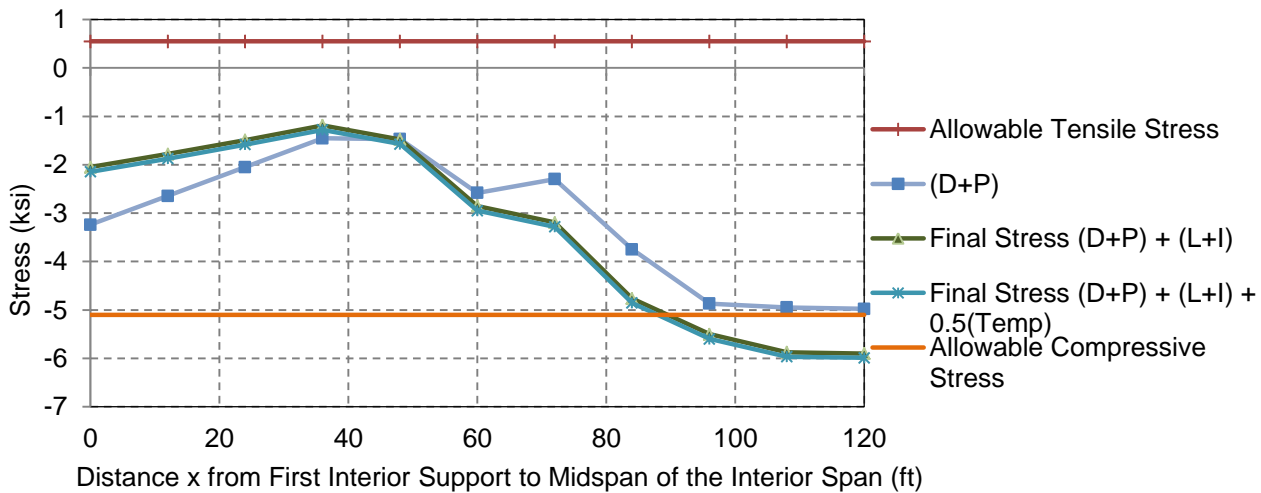
Service stress analysis is carried out for the continuous girders under the total dead loads, prestress force, live loads with impact and temperature gradient effects. The stresses are checked every 20 ft along the span locations and especially at critical locations such as the interior support and the midspan of the interior span. These stresses are also checked against the

permissible values for the service limit state after losses as specified in AASHTO LRFD Article 5.9.4.2. The allowable compressive and tensile stress limits are specified for the three loading stages specified in Table 3.5 of this report. Compression in prestressed concrete girders is evaluated through the Service I limit state, and tension in the prestressed concrete girders is evaluated through the Service III limit state with the objective of crack control.

Figure 5.8 shows the service stresses for the continuous Texas U54 girder bridge under different service load combinations.

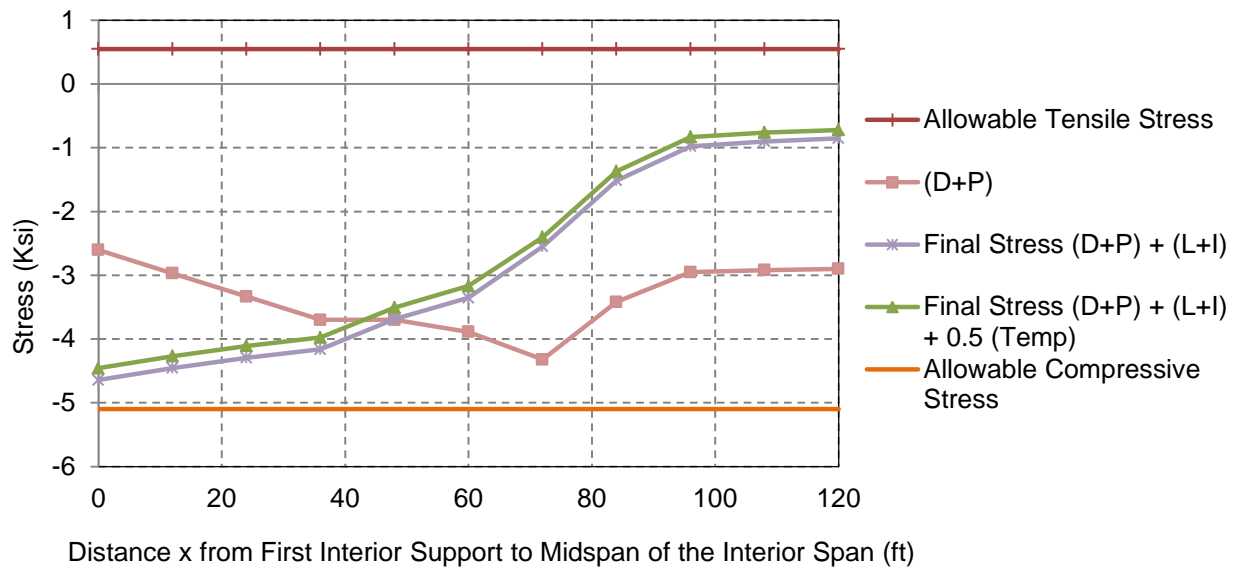


(a) Stresses at Top of CIP Deck

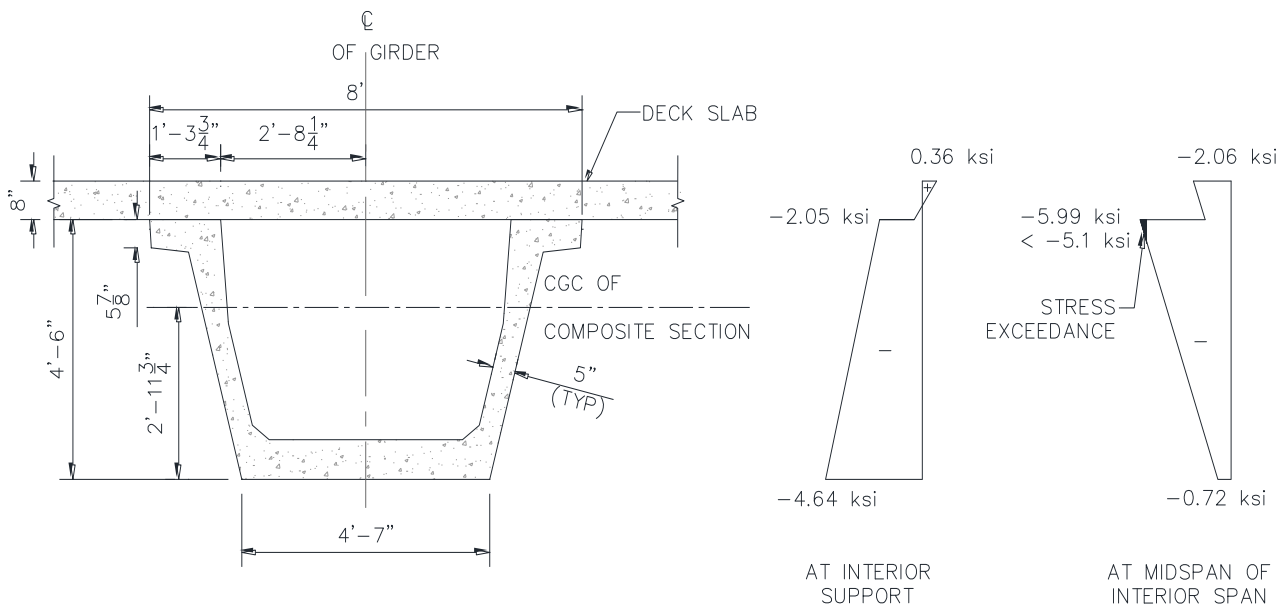


(b) Stresses at Top of Precast Girder

Figure 5.8. Service Stress Analysis for Continuous Prestressed Texas U54 Girder Bridge.



(c) Stresses at Bottom of Precast Girder



(d) Final Stresses at Interior Support and Midspan

Figure 5.8. Service Stress Analysis for Continuous Prestressed Texas U54 Girder Bridge (continued).

The compressive stresses in the top of the girder at midspan of the interior span in the positive moment region were exceeded due to a large amount of post-tensioning tendons in the section (see Figure 5.8[b]). This stress exceedance may be addressed by increasing the specified

concrete compressive strength to stay within the allowable compressive stress limit. Another option that is sometimes employed is to provide additional mild steel reinforcement in the compression zone. The amount of mild steel reinforcement is determined based on the force corresponding to the stress exceedance, shown in Figure 5.8(d). To limit the compressive stress in the top flange of the girder at the midspan of the exterior and interior spans, 2-#9 bars are provided in this design.

5.6 ULTIMATE STRENGTH CHECK

The flexural strength limit state design requires the reduced nominal moment capacity of the member to be greater than the factored ultimate design moment. The moment capacity of the girder is calculated based on the number, location, and stress in the tendons. The design capacity of the girders is calculated at three locations:

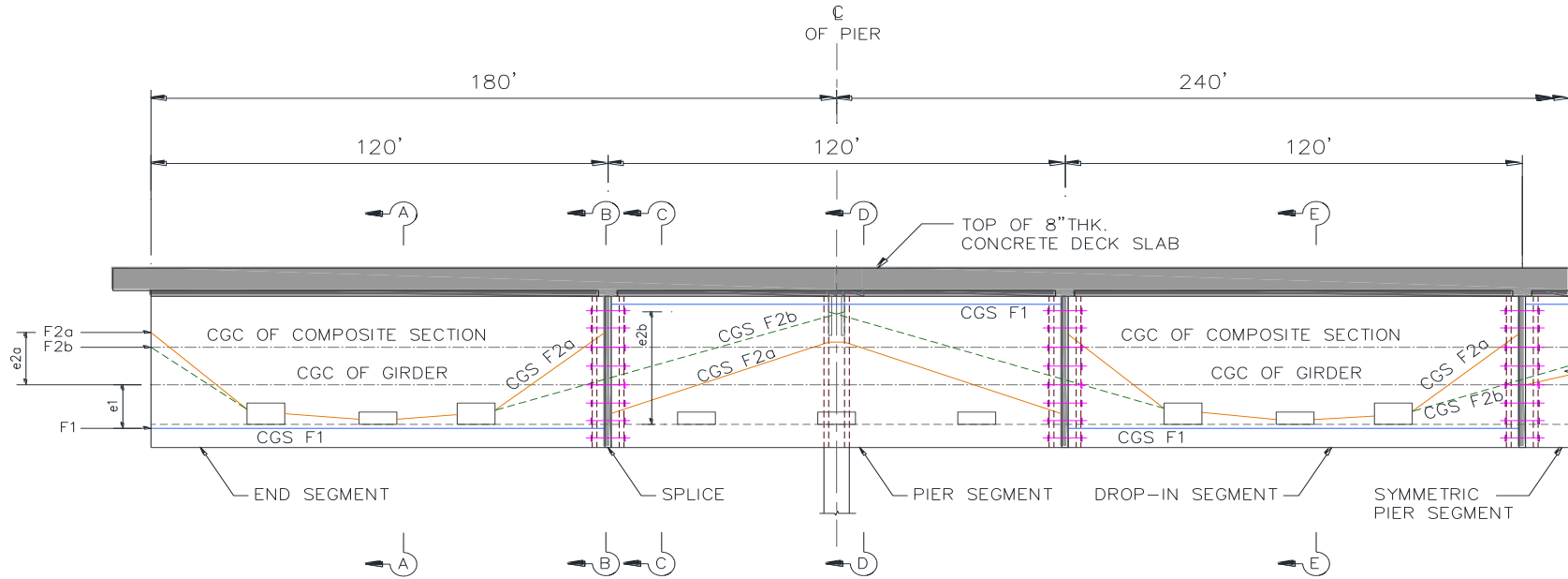
- At $0.4L$ for the maximum positive moment in the end span.
- At the face of the diaphragm at the support.
- At the midspan of the interior span.

The design moment capacity of the girders is calculated considering a rectangular section behavior if the depth of the neutral axis of the composite section lies within the depth of the deck slab. Flanged section behavior is considered when the neutral axis depth is greater than the deck slab depth. The final step is to ensure that the capacity is greater than the demand. Table 5.8 shows the ultimate moment demand and capacity of the continuous Texas U54 girder. If the flexural strength limit state is not satisfied, the capacity of the section is strengthened by providing additional mild steel.

Table 5.8. Ultimate Demand and Capacity for Texas U54 Girder.

Capacity and Demand	At Maximum Positive Moment Location		At Maximum Negative Moment Location
	Exterior Span	Interior Span	
Applied Demand, M_u (kip-ft)	12,094	13,438	21,387
Available Capacity, ϕM_n (kip-ft)	15,964	15,964	22,987

The moment capacity provided by the pretensioning and post-tensioning tendons in the maximum negative moment region at the interior support is supplemented by adding mild steel reinforcement. In this case, mild tension steel reinforcement is provided in the deck slab and girder flanges to ensure capacity greater than demand at that location. For this design, 10-#9 bars are added in the deck slab, and 4-#9 bars are added in the bottom flange of the girder to provide the additional capacity and balance the moment demand at the interior support over the pier. The mild steel reinforcement provided in the bottom flange acts as compression steel. Figure 5.9 shows the final detailed design of the continuous spliced precast, prestressed concrete Texas U54 girder bridge.



LONGITUDINAL SECTION SHOWING THE TENDON PROFILE FOR PRE-TENSIONING AS WELL AS POST-TENSIONING STAGES

NOTE: - HORIZONTAL AND VERTICAL SCALES ARE DIFFERENT. VERTICAL SCALE IS EXAGGERATED FOR CLARITY.

Figure 5.9. Design Details for Continuous Prestressed Texas U54 Girder.

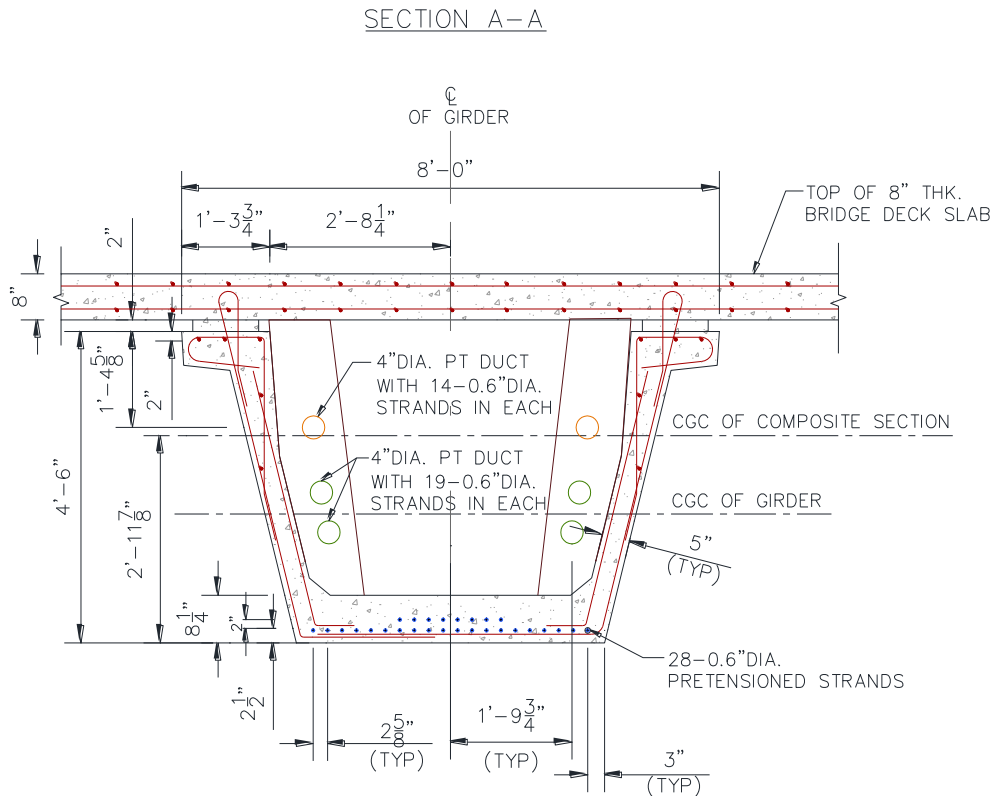
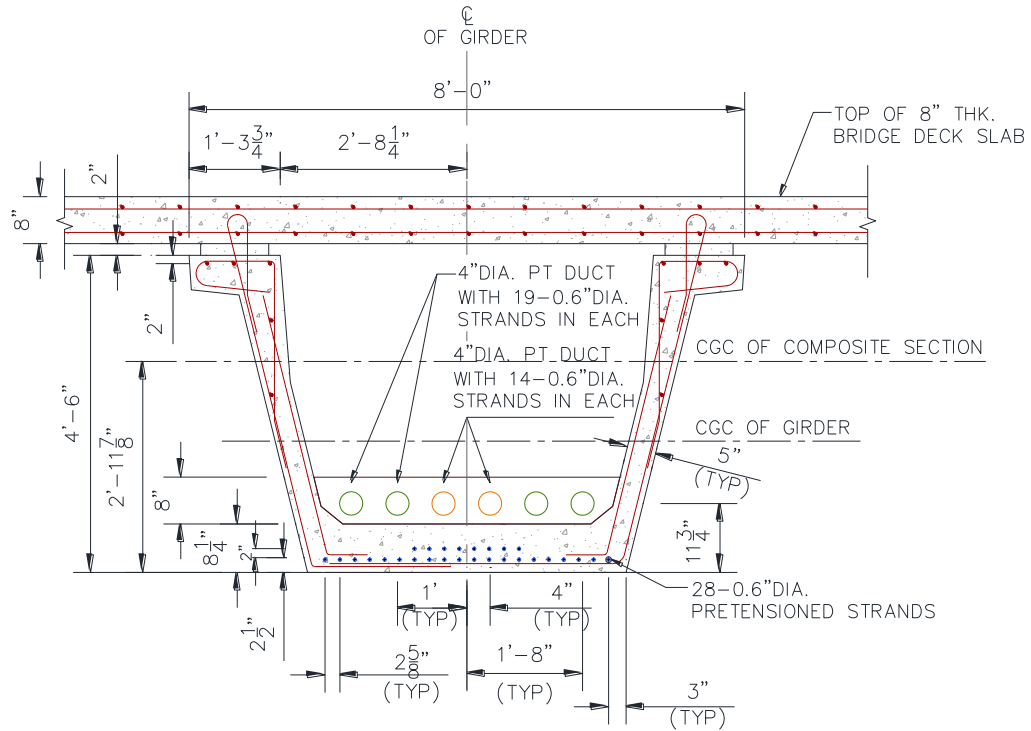
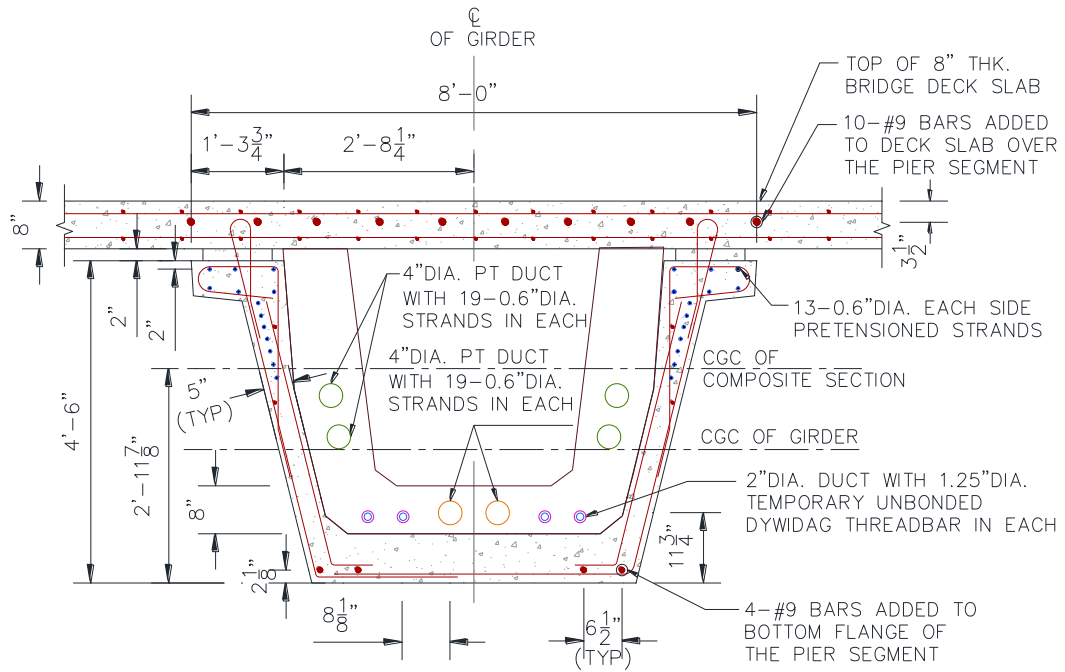
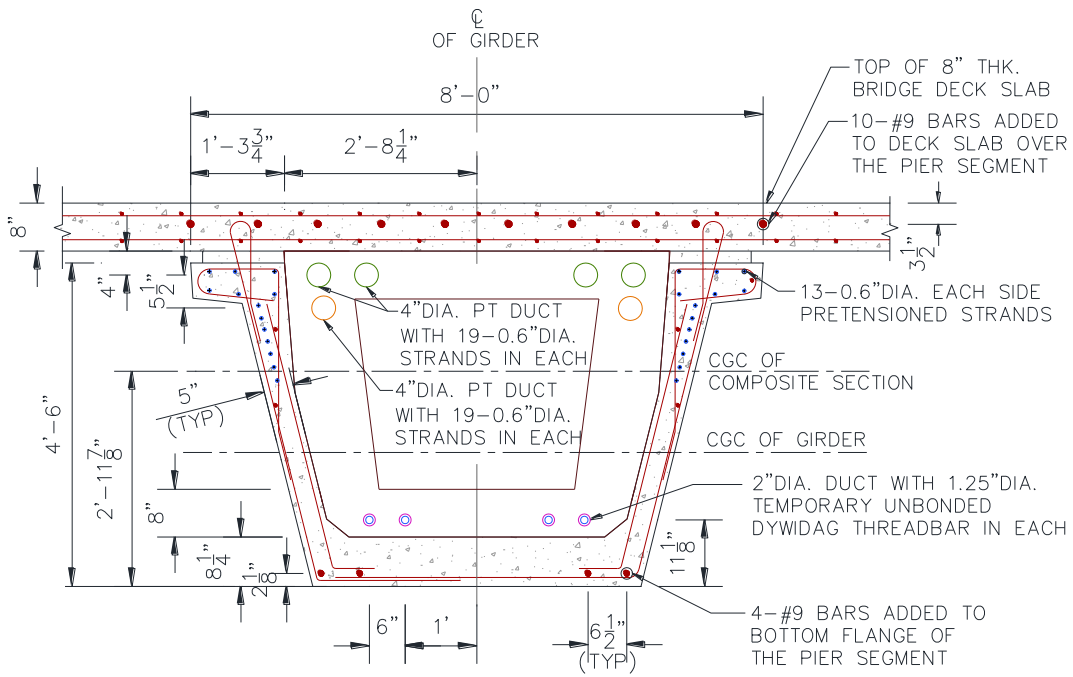


Figure 5.9. Design Details for Continuous Prestressed Texas U54 Girder (continued).



SECTION C-C



SECTION D-D

Figure 5.9. Design Details for Continuous Prestressed Texas U54 Girder (continued).

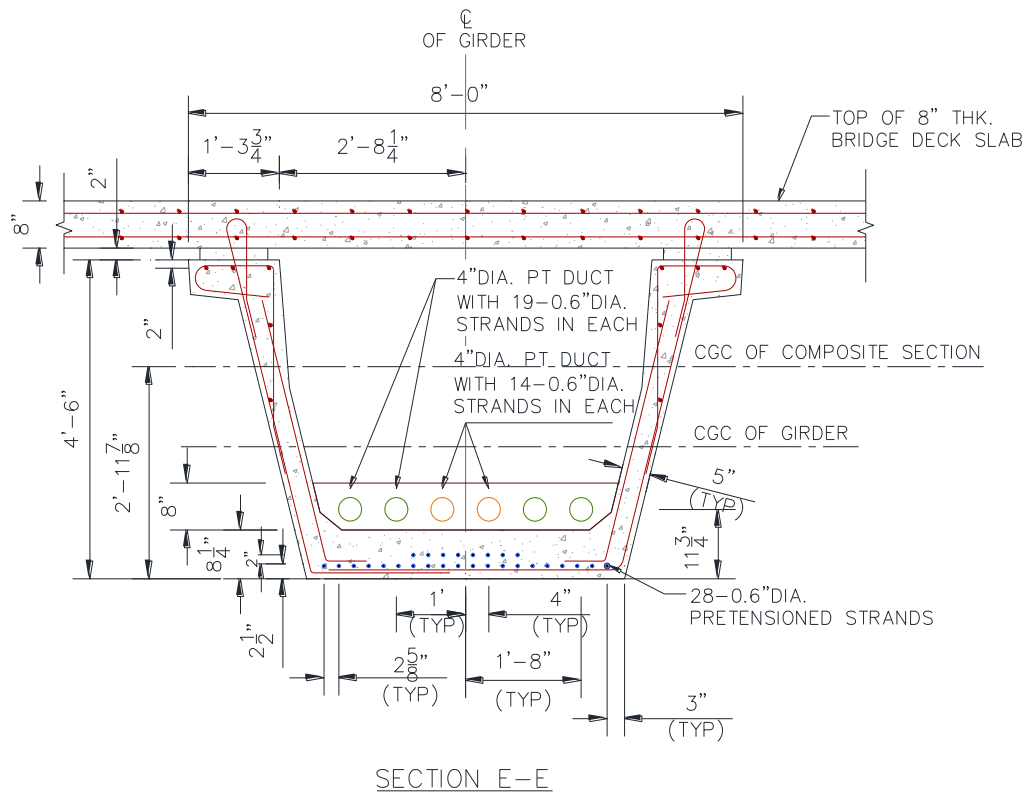


Figure 5.9. Design Details for Continuous Prestressed Texas U54 Girder (continued).

5.7 SHEAR DESIGN

5.7.1 Transverse Shear Design

Modified Compression Field Theory (MCFT) as specified in the AASHTO LRFD Specifications is used for the transverse shear design. The MCFT takes into account different factors such as strain condition of the section and shear stress in the concrete to predict the shear strength of the section. The shear strength of concrete is approximated based on a parameter β . The critical section for shear is calculated based on the angle of inclination of the diagonal compressive stress, θ . The critical section for shear near the supports is taken as the larger value of $0.5d_v \cot \theta$ or d_v , measured from the face of the support. The effective shear depth, d_v , is calculated as minimum of the distance of the resultants of tensile and compressive forces, 0.9 times the effective depth and 0.72 times the depth of the composite section.

Figure 5.10 shows the transverse shear demand and design for the Texas U54 girder using the AASHTO LRFD Bridge Design Specifications. From the graph, it is found that #4 double legged stirrups at 6 in. o.c. per web (see Figure 5.9) will provide adequate shear strength for a distance of 30 ft on either side of the support and #4 double legged stirrups at 12 in. o.c. per web will provide adequate shear strength for the remainder of the span.

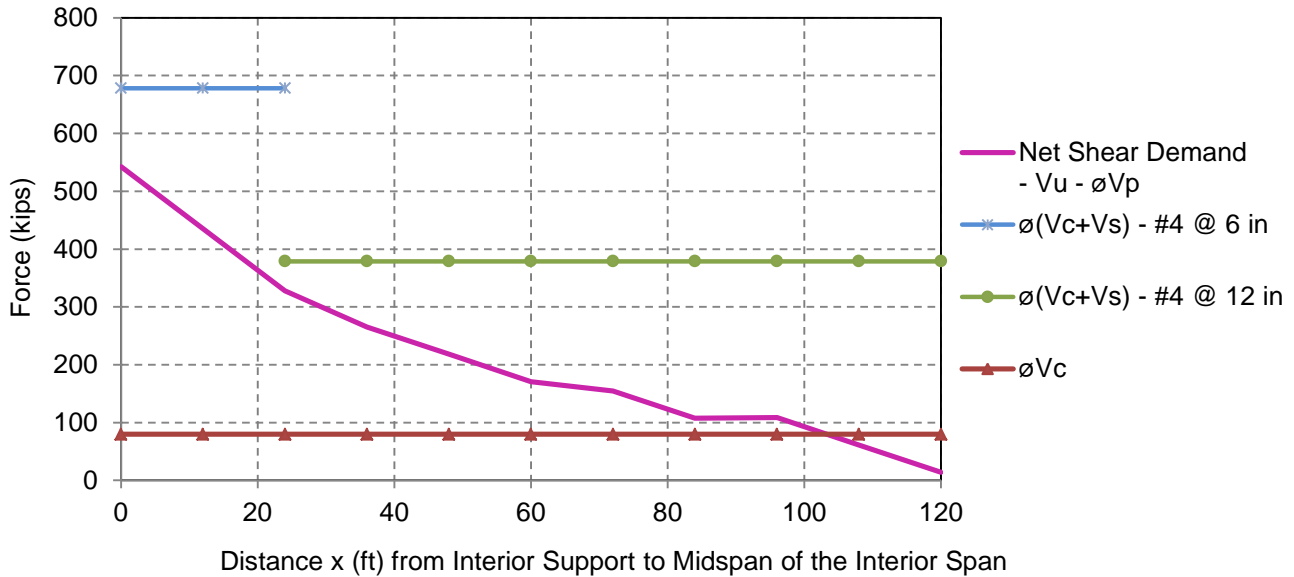


Figure 5.10. Transverse Shear Demand and Design for Texas U54 Girder.

5.7.2 Interface Shear Design

The interface shear design as specified in AASHTO LRFD Article 5.8.4. is based on shear friction theory. The nominal shear resistance of the interface plane is based on the cohesion factor, c , friction factor, μ , and the area of concrete engaged in interface shear transfer, A_{cv} . For preliminary designs, the case of normal-weight concrete placed against a clean concrete surface, free of laitance, with the surface intentionally roughened to an amplitude of 0.25 in. is used. The values of parameters c , μ , K_1 , and K_2 are as follows.

$$c = 0.24 \text{ ksi}$$

$$\mu = 1.0$$

$$K_1 = 0.25$$

$$K_2 = 1.5 \text{ ksi}$$

According to AASHTO LRFD Specifications Article 5.8.4.4, the minimum interface shear reinforcement may be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress is less than 0.21 ksi, and all transverse (vertical) shear reinforcement is extended across the interface and adequately anchored in the slab. With respect to the girder/slab interface, the transverse shear reinforcement extended into the deck slab also serves as interface shear reinforcement.

For preliminary designs, the transverse shear design reinforcement (shown in Figure 5.10) when extended into the deck slab and bent to 180 degrees serves as interface shear reinforcement and is found to be adequate to resist the horizontal shear demand. Figure 5.11 shows the interface shear demand and design for the Texas U54 girder using the *AASHTO LRFD Bridge Design Specifications*.

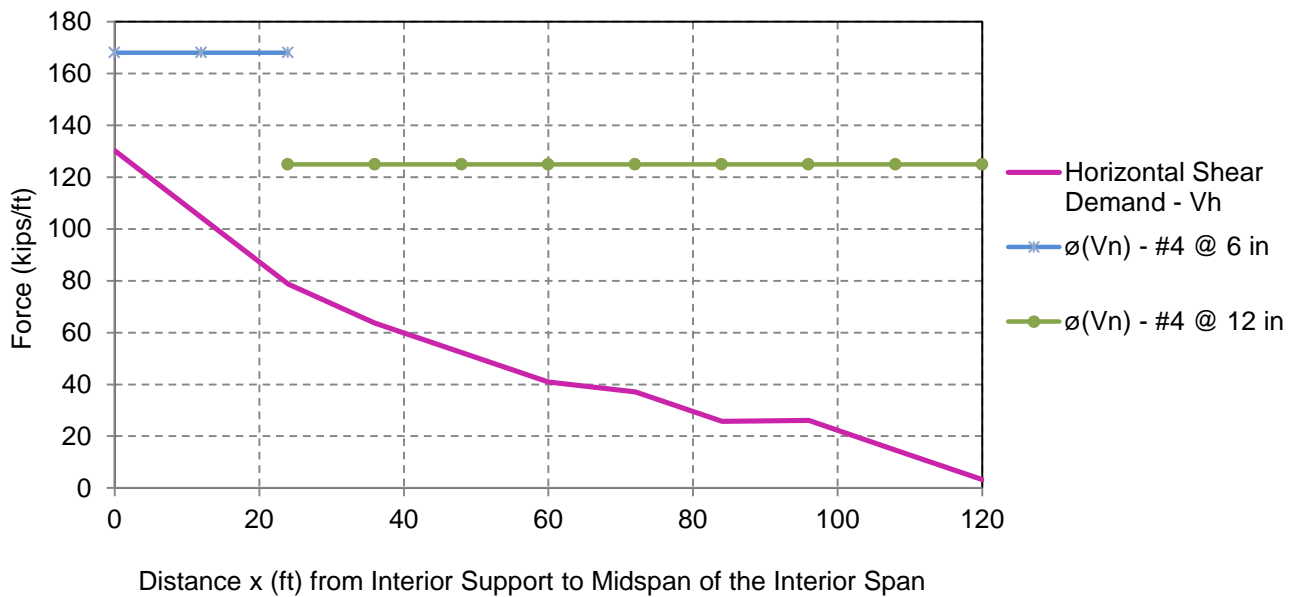


Figure 5.11. Interface Shear Demand and Design for Texas U54 Girder.

Figure 5.12 shows the detailed drawing of shear reinforcement for the Texas U54 girder.

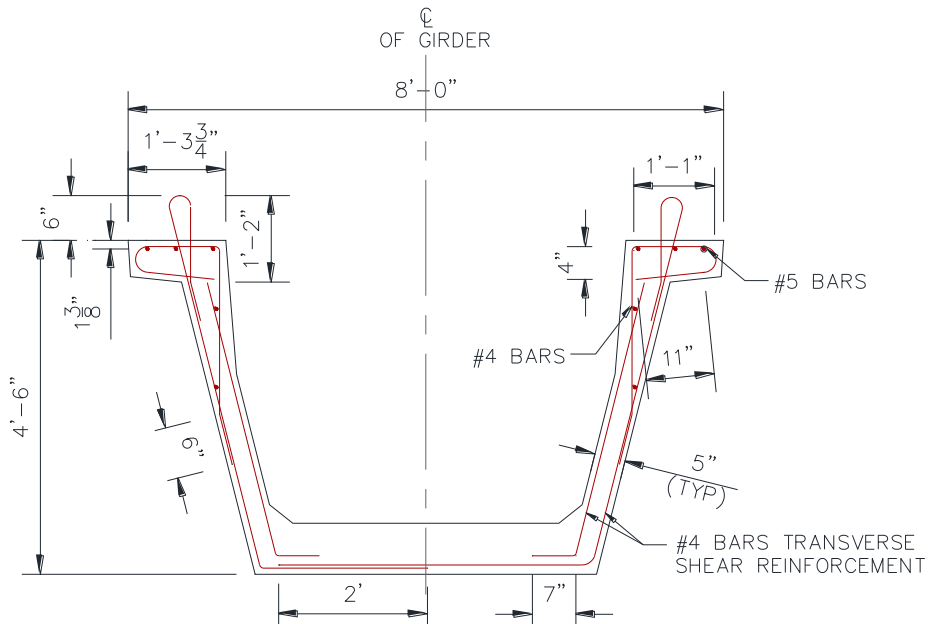


Figure 5.12. Shear Reinforcement Detail for Texas U54 Girder (Adapted from TxDOT 2010).

5.8 DEFLECTION CHECK

For preliminary designs, the four-span continuous bridge is designed for a total of two traffic lanes in accordance with the design criteria specified in the *AASHTO LRFD Bridge Design Specifications*. The Texas U54 girder bridge has a main span of 240 ft and end spans on either side of 180 ft. According to AASHTO LRFD Specifications Article 2.5.2.6.2, the composite bending stiffness of the girders is considered, and all supporting components are assumed to deflect equally. Deflection is calculated under the larger of (i) Design Truck Load alone, or (ii) 25 percent of Design Truck Load and full Design Lane Load according to AASHTO LRFD Specifications Article 3.6.1.3.2. For the preliminary designs, case (ii) causes maximum deflection.

Figure 5.13 shows the critical load arrangement for the vehicular live loads, and the lane load to produce maximum deflection in the four span continuous girders. The dynamic load

allowance factor is multiplied to the vehicular live loads. The resultant of the truck point loads, ‘ R ’ is placed at $0.4L_{ext}$ of the exterior span length and at $0.5L_{int}$ of the interior span length.

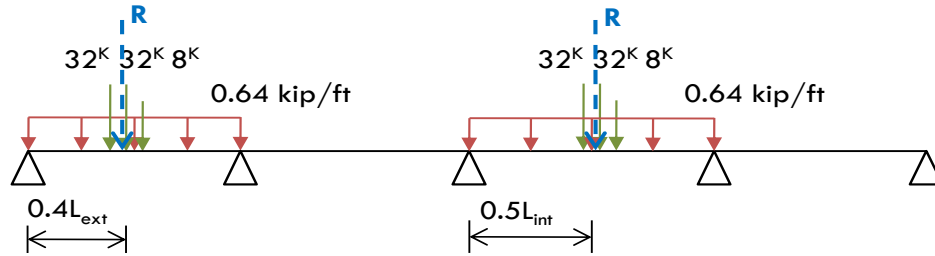


Figure 5.13. Critical Live Load Arrangement for Maximum Deflection of the Texas U54 Girder Bridge.

Table 5.9 presents the allowable and actual values of maximum deflection for the four span continuous bridge girders considering two traffic lanes loaded. It can be observed that the deflection is within the allowable limits.

Table 5.9. Maximum Deflection for Texas U54 Girder Bridge.

Deflection	Exterior Span	Interior Span
Allowable (in.)	2.7	3.6
Actual (in.)	1.4	2.5

6. DESIGN ISSUES AND RECOMMENDATIONS IDENTIFIED BY PRELIMINARY DESIGNS

6.1 GENERAL

The research team has identified several design issues in the preliminary design stage of continuous precast prestressed concrete bridges using standard Tx70 and Texas U54 girders, which are discussed in the following sections. The research team has also provided recommendations for resolving these issues.

6.2 GIRDER SECTIONS

The research team used an increased web width of 9 in. for the standard Tx70 girder section by spreading the girder side forms. This web width is required to accommodate 4 in. diameter post-tensioning ducts and shear reinforcement. Spreading the girder side forms also increased the width of the top and bottom flanges by 2 in. The width of the top flange is increased to 44 in. and that of the bottom flange to 34 in., respectively.

It is noted that the Texas U54 girder section was used in the preliminary designs because it was the deepest available tub section for typical Texas precast, pretensioned concrete bridges. However, the limited depth available in the void space of the beam would lead to difficulties for the Stage 2 post-tensioning that would take place after installation of the deck slab. Access to the external tendons to inspect grouting would also be an issue. Therefore, it would be beneficial to use a deeper tub section to allow for ease of installation and inspection of the external tendons provided in the void space of the beam. Increasing the girder depth would add to the weight and make these sections heavier. Therefore, a potential design proposal is to use smaller lengths of deeper tub girder segments (approximately 50 ft in length) with more than two splices per span of the bridge.

6.3 GIRDER DESIGN

For transportation and handling purposes of the pier segments of both Tx70 and Texas U54 girder bridges, temporary unbonded Dywidag threadbars of 1.25 in. diameter are provided in the bottom flange of the pier segments. Primary corrosion protection is provided by the alkalinity of the grout and concrete. Various corrosion protection systems are available for these

threadbars, none of which bond with the structure. Improved corrosion protection can be provided by using corrugated galvanized metal ducts, polyethylene or polypropylene ducts. Once the pier segment is erected on site, it behaves as a cantilever. These threadbars can then be destressed after the pier segment is safely installed into place.

The removal of temporary support towers used in shored construction adds moments at the support over the pier. The temporary support removal corresponds to introducing a force on the pier segments, which is equal and opposite to the reaction from the drop-in and end segments. The pier segments are designed for these additional moments.

The preliminary designs for Tx70 and Texas U54 girder bridges assumed shored construction. From the results of preliminary designs of the continuous spliced precast, prestressed concrete bridge design proposal using the Tx70 girder section, the research team found that although it may be technically feasible to construct 300 ft spans using the Tx70 girders, higher strength concrete and a large number of tendons are needed. A span length of 280 ft is possible using the Tx70 girders, but not easily obtainable. However, a span length of 240 ft can be achieved meeting all the design criteria. The results of the preliminary designs of the continuous spliced precast, prestressed concrete bridge using Texas U54 girder showed that a span length of 240 ft is viable for the U54 girders, providing a construction alternative.

From the preliminary designs, it was noted that the span lengths of 280 ft and 240 ft for the continuous prestressed concrete bridges using the standard Tx70 and Texas U54 girders, respectively are achieved using shoring towers (shored construction) and by making the girder sections work up to their limits. For increasing the span lengths beyond these values or considering use of unshored method of construction, the research team recommends the use of haunched girder segments over the piers and notes that higher strength concrete than that being currently used may be required.

6.4 SPLICE LOCATION

Splices are located at the joints between the girder segments. Splice locations vary for different projects built to date. It is important to determine the best possible location specifically for each project. The design approach for the continuous prestressed concrete girders is based on load balancing. The location of inflection points under total dead loads is important in selecting splice locations.

6.5 SEQUENCE OF CONSTRUCTION

The sequence of construction has a significant effect on the design and behavior of the bridge. Figure 6.1 shows the stages of shored construction of a continuous prestressed concrete girder bridge as considered for the preliminary designs. The over pier precast girders are erected on piers and temporary falsework. The precast girders are made continuous by casting a splice between the ends of the girder segments. Post-tensioning is performed in two stages. Stage 1 post-tensioning is carried out prior to casting of the deck slab and Stage 2 post-tensioning after casting the deck slab. The traffic barriers are then cast, and the future wearing surface is added. Therefore, the girders are considered continuous for the deck slab dead load and for all loads applied to the composite section. The bridge is made open to traffic loads (HL93 with Dynamic Allowance). The final condition of the bridge under service and after losses is checked. 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 from girder erection and post-tensioning up to the final stage.

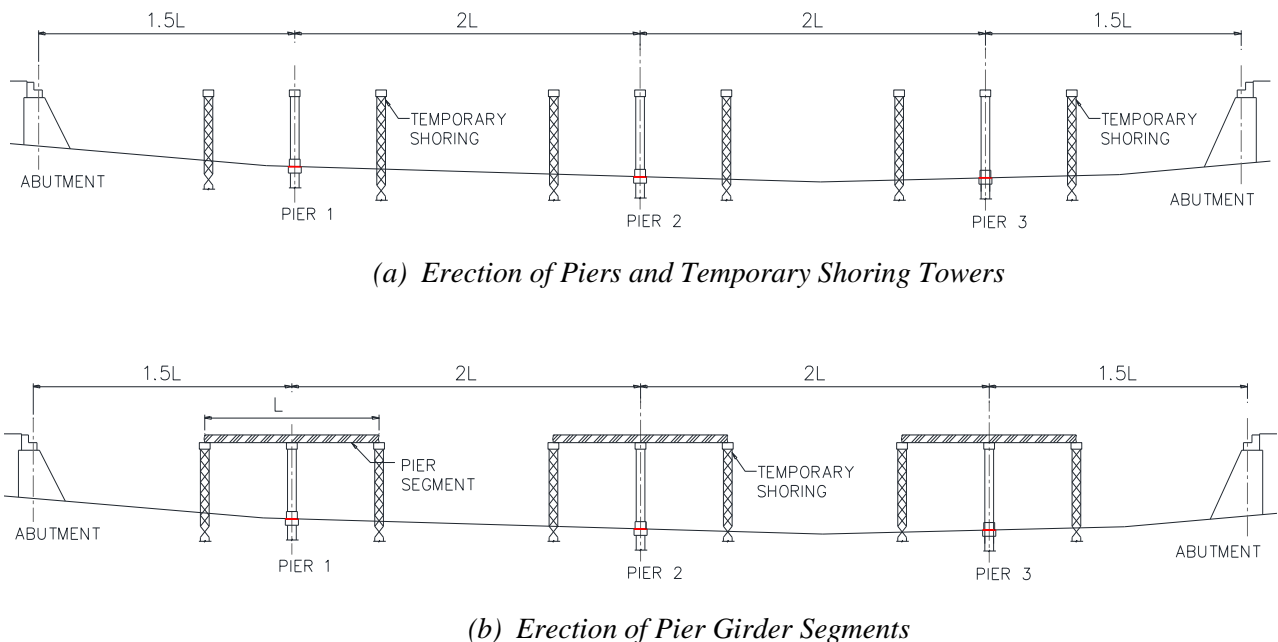
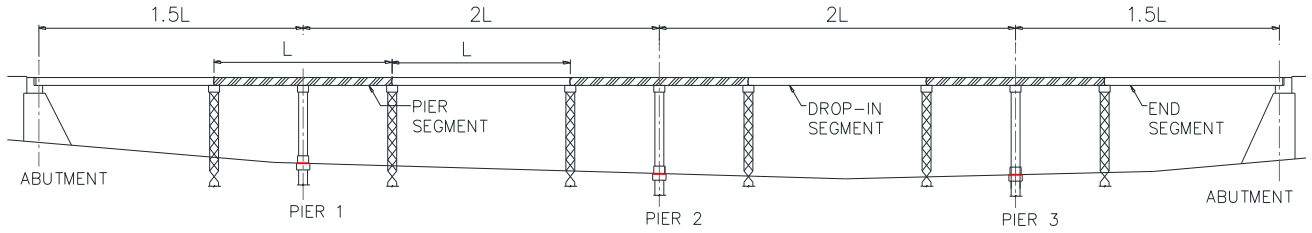
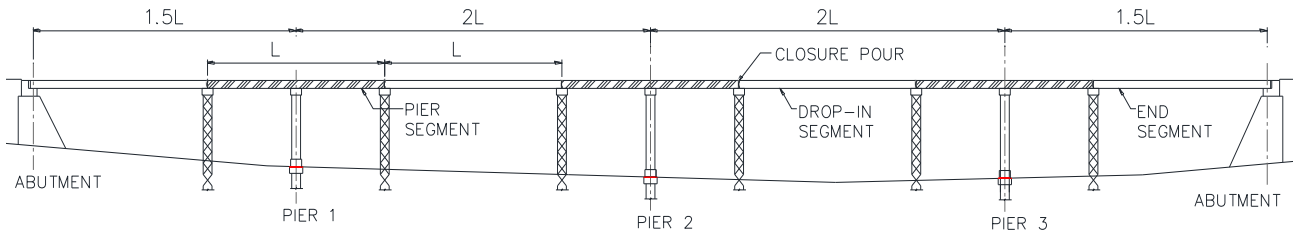


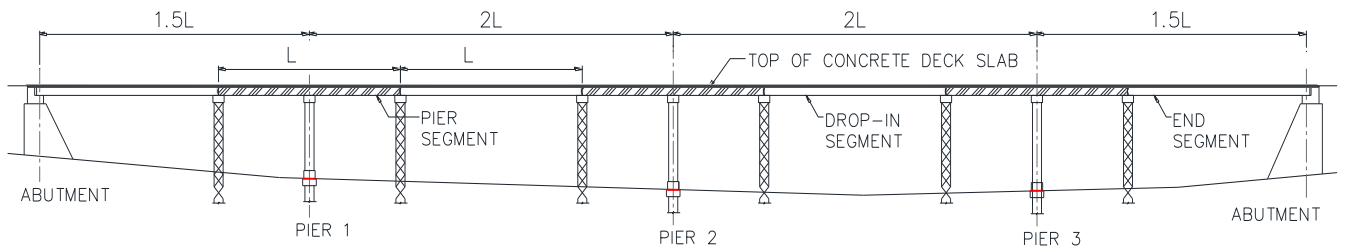
Figure 6.1. Stages of Shored Construction for a Continuous Prestressed Girder Bridge.



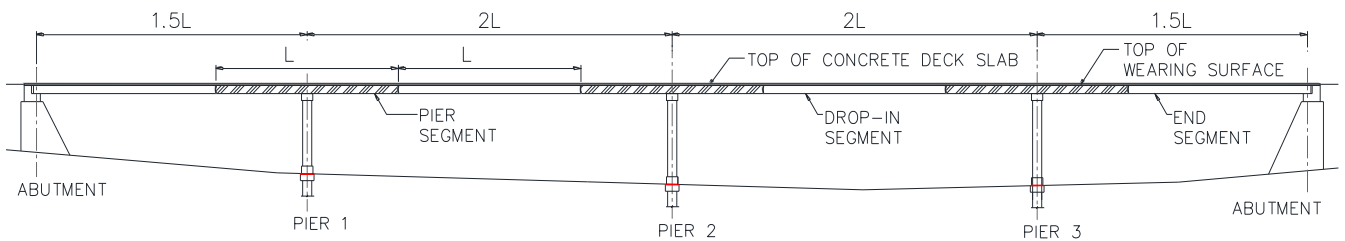
(c) Erection of End and Drop-in Girder Segments and Stressing Stage 1 Post-tensioning Tendons



(d) Casting Splices



(e) Casting Deck Slab and Stressing Stage 2 Post-tensioning Tendons



(f) Removal of Temporary Shoring Towers, Casting Barriers, Laying Wearing Surface and Opening Bridge to Traffic

Figure 6.1. Stages of Shored Construction for a Continuous Prestressed Girder Bridge (continued).

The temporary supporting of the end and drop-in girder segments on the ends of the pier girder segments has a significant effect on the demands on pier girder segments and negative moment region in general. In shored construction, the girder segments are supported on the temporary towers at the splice locations to resist any reaction forces during erection.

The research team recommends stressing the post-tensioning tendons from both ends sequentially in stages in order to provide equal conditions in both end spans. If this is not feasible, then the other option is to stress the first set of tendons from one end and the second set from the other end of the girders for uniform stresses in the tendons of the bridge. However, it is still recommended to stress all tendons from both ends, if possible.

6.6 STRENGTH LIMIT STATE

The researchers checked the girder at critical sections for flexural capacity under factored loads for the flexural strength limit state. The factored moment capacity of the girder section is compared with the factored moment demand under full dead load and full live load effects. The final step is to ensure that the capacity is greater than the demand. The researchers recommend adding mild steel reinforcement to supplement the moment capacity provided by the pretensioning and post-tensioning tendons, if necessary.

6.7 STRESSES UNDER SERVICE LOADS

The researchers computed stresses in the girders and the deck slab along the length of the bridge from the first interior pier to the midspan of the interior span. These stresses are computed at different loading stages as follows.

1. Dead Load + Prestress
2. Dead Load + Prestress + (Live + Impact Load)
3. Dead Load + Prestress + (Live + Impact Load) + 0.5 (Temperature)

The calculated stresses are compared with the allowable stress limits specified in the *AASHTO LRFD Bridge Design Specifications*. The midspan of the interior span typically controls the design of the post-tensioning force for tensile stresses in the positive moment region. The allowable tensile stresses in the top of the deck slab often controls the design of the post-tensioning force in the negative moment region over the piers. Due to the load-balancing

approach and after all the prestress losses have occurred in the final service stage, the deck is in compression to close any potential cracks when the bridge is open to traffic.

6.8 DEFORMATIONS

6.8.1 General

AASHTO LRFD Specifications Article 2.5.2.6 states that bridges should be designed to avoid undesirable structural or psychological effects due to their deformations. These deformations include the optional criteria for the live load deflection and span-to-depth ratio limitations.

The commentary on the AASHTO LRFD Specifications Article 2.5.2.6 on ‘Deformations’ explains the inclusion of the optional criteria for deflection under live load and the span-to-depth ratios. It was found that these criteria were adopted to limit the service load deformation effects such as deterioration of wearing surfaces and local cracking of concrete deck slabs. The span-to-depth ratio criterion was used since the early 1900s and the live load deflection limits were used since the 1930s.

The *AASHTO LRFD Bridge Design Specifications (2010)* mention that the ASCE Committee (1958) conducted a study on deflection limitations on bridges and found numerous shortcomings in the traditional approaches of using the span-to-depth ratios and live load deflection limits. As a part of this study, this Committee carried out a limited survey and revealed no evidence of serious structural damage that could be attributed to excessive deflection. They found that few examples of damaged stringer connections or cracked concrete floors could probably be corrected more effectively by changes in design than by more restrictive limitations on deflection. The results of the survey and review of historical studies indicated clearly that unfavorable psychological reaction to bridge deflection is probably the most frequent and important source of concern regarding the flexibility of bridges. However, those characteristics of bridge vibration, which pedestrians or passengers in vehicles considered objectionable, cannot yet be defined.

Extensive research has been conducted on the human response to motion, and it is now generally agreed that the primary factor affecting human sensitivity is acceleration, rather than deflection, velocity, or the rate of change of acceleration for bridge structures. However, the

problem is a difficult and subjective one. Thus, there are as yet no simple definitive guidelines for the limits of tolerable static deflection or dynamic motion.

6.8.2 Deflection

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs that could impair serviceability and durability, even if self-limiting and not a potential source of collapse. The AASHTO LRFD Specifications Article 2.5.2.6.2 provides an optional criterion for deflection control. In order to investigate the maximum absolute deflection for straight girder systems, all the design lanes are loaded, and all supporting components are assumed to deflect equally. In case of a composite design of slab-on-girder bridges, the stiffness of the design cross-section used for the determination of deflection includes the entire width of the roadway and the structurally continuous portions of the railings. The composite bending stiffness of an individual girder can be taken as the stiffness of the design cross-section, divided by the number of girders.

The limits for maximum deflection as specified in AASHTO LRFD Specifications Article 2.5.2.6.2 for concrete construction are as follows.

- Vehicular load, general = $\text{Span}/800$.
- Vehicular and/or pedestrian loads = $\text{Span}/1000$.

The critical position of the loads is important in calculating the maximum deflections. The live load portion of Load Combination Service I of AASHTO LRFD Specifications Table 3.4.1-1 is used, including the dynamic load allowance. The live load is considered as specified in AASHTO LRFD Article 3.6.1.3.2, according to which, the deflection is calculated under the larger of the following:

- Design truck alone.
- 25 percent of Design Truck Load and full Design Lane Load.

The *AASHTO LRFD Bridge Design Specifications* articles and commentary do not provide detailed explanations or justification for these limits. The available information indicates that they initiated as a method of controlling undesirable bridge vibration. Bridge vibration concerns are largely based on human perception. Human perception of vibration depends on a combination of maximum deflection, maximum acceleration, and frequency of response. Field measurements of bridges show that the actual bridge live-load deflections are often smaller than

computed values for a given truck weight. Increasing the stiffness would help reduce the bridge deflection and vibrations. There are alternative techniques that can be applied to reduce vibrations (e.g., using mechanical dampers).

From the deflection results for the continuous prestressed Tx70 and Texas U54 girder bridges considered for the preliminary designs, the maximum deflection in the continuous girders is below the allowable limits as specified in AASHTO LRFD Specifications Article 2.5.2.6.2.

6.8.3 Span-to-Depth Ratio

AASHTO LRFD Specifications Article 2.5.2.6.3 states that “Unless otherwise specified herein, if an Owner chooses to invoke controls on span-to-depth ratios, the limits in Table 2.5.2.6.3-1, in which S is the slab span length and L is the span length, both in ft., may be considered in the absence of other criteria. Where used, the limits in Table 2.5.2.6.3-1 shall be taken to apply to overall depth unless noted.” Table 6.1 is adapted from AASHTO LRFD Specifications Art. 2.5.2.6.3.

**Table 6.1. Traditional Minimum Depths for Constant Depth Superstructures
(Adapted from AASHTO LRFD 2010).**

Superstructure		Minimum Depth (Including Deck)	
		When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced Concrete	Slabs with main reinforcement parallel to traffic	$\frac{1.2(S + 10)}{30}$	$\frac{S + 10}{30} \geq 0.54 \text{ ft}$
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 6.5 \text{ in.}$	$0.027L \geq 6.5 \text{ in.}$
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$

**Table 6.1. Traditional Minimum Depths for Constant Depth Superstructures
(Adapted from AASHTO LRFD 2010) (continued).**

Superstructure		Minimum Depth (Including Deck)	
		When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Steel	Overall Depth of Composite I-Beam	0.040L	0.032L
	Depth of I-Beam Portion of Composite I-Beam	0.033L	0.027L
	Trusses	0.100L	0.100L

Leonhardt (1982) specified that the span-to-depth ratio is usually chosen based on past experience and on conventional values. It is particularly important for girder-type bridges as it affects the material cost and construction of the superstructure. Using a high ratio (i.e., slender girder) reduces the concrete volume, increases the prestressing requirement and simplifies the construction due to a lighter superstructure. Moreover, slenderness ratio has significant aesthetic impact, because the overall appearance of a girder-type bridge is highly dependent on the proportion of the superstructure.

Barker and Puckett (2007) noted that the AASHTO LRFD Specifications Table 2.5.2.6.3-1 was developed from the recommendations of ACI-ASCE Committee 343 (1988). The span-to-depth ratios specified in this table are traditional ratios provided in an attempt to ensure that vibration and deflection would not be a problem. These are not absolute maximums but are only guidelines. These values compare well with the span-to-depth ratios that are desirable for a pleasing appearance.

Poon (2009) conducted a research study at the University of Toronto to find the effect of span-to-depth ratio on cast-in-place box girders, solid slabs, and segmental box girders (not particularly on I-girders or U girders), and these studies indicate that the values given in the specifications give the optimal solution in terms of cost efficiency and aesthetic sense. For adjacent box beams the ratio of span-to-depth ratio is 1:40. This means that longer spans can be attained with shallower depths. As the depth of the cross section reduces, a larger amount of prestress is required. The additional webs that are present in this cross section help accommodate the higher number of prestressing tendons associated with slender girders without sacrificing the

efficiency of the tendon layout (i.e., lowering the tendon eccentricity by placing tendons in vertical layers within the webs). The same reasoning can be applied to U girders, and so a ratio of 1:25 may be too strict for U girders. In addition, external post-tensioning can be applied so additional prestress can be accommodated without reducing the eccentricity.

The span-to-depth ratio is an important bridge design parameter that affects the structural behavior, cost efficiency, and aesthetics of the structure. Prestressed concrete girder bridges are intended to be a competitive alternative to steel bridges. Steel bridges tend to be less stiff compared to prestressed concrete girder bridges of the same depth. There does not seem to be clear justification for allowing steel bridges to have longer span-to-depth ratios as compared to prestressed concrete girder bridges.

The continuous prestressed Tx70 and Texas U54 girder bridges considered for the preliminary designs have a total depth of 62 in. (U54 with 8 in. deck) and 78 in. (Tx70 with 8 in. deck). These depths are significantly smaller than the minimum suggested depths of 115 in. and 134 in. for the 240 ft (U54) and 280 ft (Tx70) spans, respectively, based on the traditional minimum depth of $0.04L$ for continuous precast, prestressed I-beams noted in Table 6.1. It should be mentioned, however, that the deflection criteria discussed in Section 6.8.2 were met for both preliminary designs considering two lanes across the transverse width of the bridge. In addition, the maximum number of lanes (three lanes) was also checked and the deflection criteria were satisfied for both the U54 and Tx70 bridges. Nevertheless, it is acknowledged that the preliminary designs represent aggressive span-to-depth ratios and require special additional measures to pass all the ultimate moment strength and stress checks. While the preliminary designs provide insight into upper bound limits of span-to-depth ratios for these particular cases, future bridges with these girder sections would likely be designed with smaller span-to-depth ratios. At the same time, the suggested AASHTO span-to-depth ratio limits appear to be quite conservative for continuous prestressed concrete bridges. Relaxing these recommendations appears to be reasonable as long as all other limit states are satisfied and appropriate stability checks for construction conditions are conducted.

7. PRELIMINARY DETAILS OF SPLICE CONNECTIONS

7.1 INTRODUCTION

Most prestressed concrete slab-on-girder bridges are simply supported with precast, pretensioned girders, and a cast-in-place (CIP) deck. Spans are limited to about 150 ft due to weight and length restrictions on transporting the precast girder units from the prestressing plant to the bridge site. While economical from an initial cost point-of-view, such bridge construction may become somewhat limiting when longer spans are needed. According to the available literature, various methods have been used to extend the span range of concrete slab-on-girder bridges. These include the use of high performance materials and modified girder sections (Abdel-Karim and Tadros 1995). However, to significantly increase the span length, it is necessary to modify the layout and provide continuity connections between the spans.

Spliced girder bridge construction can provide a less complex solution compared to segmental concrete bridge girder construction by reducing the number of girder segments. Spliced precast, prestressed concrete girders were recently found to be the preferred solutions of contractors, as observed in performance-based bids of projects in several states (Castrodale and White 2004). For these longer spans, continuity between the girder segments has the advantage of eliminating bridge deck joints, which leads to reduced maintenance costs and improved durability. The performance and cost-effectiveness of a spliced girder system depends on the design and construction details. This involves a combination of the different design enhancements instead of applying them individually.

This chapter reviews and outlines some of the key techniques that have been used for spliced, continuous, bridge girder systems, discusses a number of construction considerations, proposes a general framework for categorizing connection splice types, and provides some potential connection details.

7.2 SPLICED GIRDER SYSTEMS IN PRACTICE

A variety of bridge construction practices have been observed over the years. The methods used in different states for extending span ranges with incremental variations in the materials and conventional design procedures often result in relatively small increases in span range for the precast, prestressed concrete girders. Splicing technology facilitates construction of

longer spans using standard length girder segments. A spliced girder system can provide a number of constructible design options by altering parameters such as span and segment lengths, depth of superstructure, and number and location of piers. The focus of the research presented in NCHRP Report 517 (Castrodale and White 2004) was to develop AASHTO LRFD design procedures, standard details, and design examples for long span, continuous, precast, prestressed concrete bridge girders. The details of this study are presented in Chapter 2 of this report.

7.2.1 On-Pier Splicing with Continuity Diaphragms

The simplest, most economical, and constructible type of continuous bridge construction used in many states involves erection of the girders as simple spans supporting their self-weight and the weight of the deck. The diaphragm with continuity splicing is cast after pouring of the deck. When the deck and the diaphragm are fully cast and hardened, the structure acts as a continuous system to carry the live loads. Advantages of this design include minimization of expansion joint maintenance in the bridge deck, reduced midspan moments, and additional reserve capacity due to moment redistribution, improved appearance, and ride quality. However, areas of concern include creep effects, which cause the concrete in compression to compress further at the bottom of the girder due to the prestress. This builds up large positive moments in the continuous connection. Negative moments in the connection are amplified due to the rate of shrinkage of the deck exceeding that of the girder. Therefore, this type of continuity is questioned in terms of its efficiency given these issues and the increase in the overall cost and time of construction.

Chapter 2 of this report describes a variety of on-pier splicing techniques. From the review of all the methodologies involving on-pier splicing techniques, it was found that although on-pier splicing options can reduce the complexity of construction in the field, the span range of such bridge structures is controlled by the fabrication, hauling, and erection limitations for full-span girders. Other design and construction options should be considered to splice girder segments together to achieve longer spans.

7.2.2 In-Span Splicing with Cantilevered Pier Segments

Lengths of girder sections greater than 150 ft are not feasible due to the transportation constraints. However, this issue can be resolved by transporting the fabricated maximum available length of girder sections and splicing them together on the construction site. The

provision of continuity further improves the economy for this type of construction and extends the span range of the bridge. The use of spliced-girder technology has been successfully applied to increase span lengths and transverse spacing of the standard precast, prestressed concrete girders beyond the customary values in some states in the United States.

Chapter 2 of this report also describes the different types of in-span splicing techniques for prestressed concrete girders. The span increase is typically approximately 50 percent for prismatic sections adopting in-span splicing technology. But if the over-pier segment is haunched, greater span lengths have been achieved with an increase on the order of 100 percent (Castrodale and White 2004). In this technique, precast, prestressed concrete girders are fabricated in several relatively long segments that are connected on-site into the final bridge structure. Post-tensioning is generally used to provide continuity between the girder segments.

7.3 CONSTRUCTION CONSIDERATIONS

Large infrastructure projects are characterized by mass construction and potentially long project durations. The choice of the method of construction plays a very important role in the overall cost of the bridge structure. The technological aspects of construction combined with the design concepts help to determine the economic viability of any project. Inappropriate methods of bridge construction cause further traffic delays and congestion. Hence, the development of rapid methods of bridge construction is essential. One of the most efficient methods of bridge construction is the use of precast systems that are fabricated at the precasting plant and then brought to the job site and assembled. This section discusses several issues related to continuous bridge construction using precast girder elements.

7.3.1 Construction Techniques

Different construction methods are adopted for spliced girder connections, depending on the topography, available equipment, labor, and local contractors. These construction methods can be classified as either shored or unshored. The most common types of construction techniques used for spliced bridge structures are temporary shoring, structural steel strong backs, and structural steel hangers.

7.3.1.1 Temporary Shoring

Temporary shoring towers installed on the construction site are used for in-place splicing of girder segments. These towers facilitate the adjustments required to achieve final elevation of the bridge. However, setting up these towers requires on-site labor and consumes space. The drop-in and over-pier segments are handled only once using smaller lifting equipment. These towers can be reused again and again for mass progressive construction. This technique is the simplest and most economical for construction sites with a flat terrain.

7.3.1.2 Strong Backs

Construction using strong backs is commonly used for long span bridges crossing waterways. The strong backs are structural steel sections connected to the top flange of the drop-in girder segments with threaded rods. When the drop-in segments are placed in position, the projecting end of these strong backs rests on the cantilever end of the pier segments. The pier segments support the weight of the drop-in spans until the splice is cast and the girders are post-tensioned for continuity. The strong backs can be uninstalled and reused again. They prove to be economical in areas where it is difficult to provide temporary falsework. However, the use of this method requires precise design and construction procedures to be followed.

7.3.1.3 Steel Hangers

The use of steel hangers is not very common. This method involves embedment of structural steel members as hangers into the girder segments. A structural steel H-section known as a guide shoe is placed over the hangers projecting from the pier segment and bolted. This piece is used to provide careful alignment of both the hangers. The hangers from the drop-in segment are then seated on the guide shoe and bolted again. These steel hangers are permanently embedded into the girder and cannot be reused again. Careful design and erection of the hangers is necessary for this method. It is advantageous as an unshored method of construction in case of deep valleys or water crossings. However, the embedment of shear connectors and threaded bars in the girder section may add to the initial cost of the bridge.

7.3.2 Continuous Girder Splicing Techniques

The most commonly used continuity details involve on-pier splices that join girder segments at the intermediate diaphragms over the piers. However, this does not produce a

significant increase in the span of the bridge due to lifting and hauling limitations. Spliced girder bridges using in-span splices present a cost-competitive alternative and help to fill the gap between 150 ft continuous, precast, pretensioned, concrete bridges and the 300 ft continuous, post-tensioned, concrete, segmental box girder bridges. The use of in-span splices allows for spans long enough to minimize the impact on the areas of alignment of the bridge. This approach also helps to achieve significantly longer spans than other methods involving single precast, prestressed girder segments for a whole span.

Spliced bridge girders can include connection splices that are in-span, on-pier, or both. Figure 7.1 shows schematics of two alternative construction procedures for continuous spliced bridge girders:

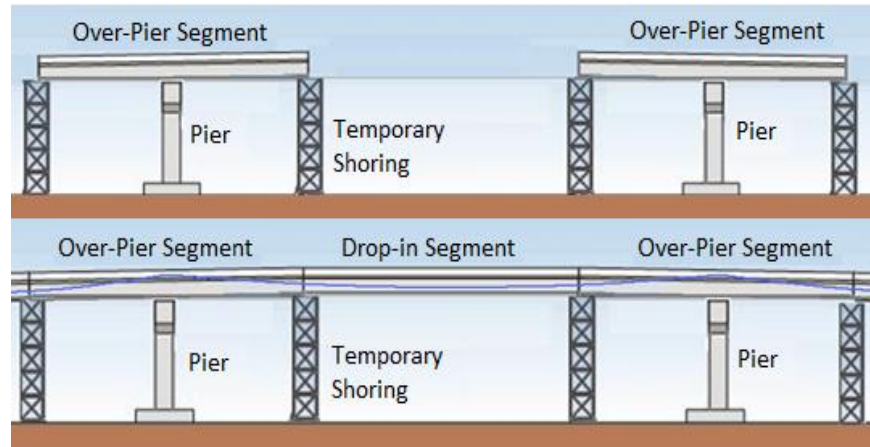
- Option A uses in-span splices only, with a single continuous girder segment over the pier and temporary supports. This approach has been used for many continuous spliced girder bridges with three to five spans.
- Option B includes splice connections at the pier and in-span. This approach has the advantage of conducting span-by-span post-tensioning with anchorage more easily provided at a pier support diaphragm. It also avoids the reverse curvature of continuity post-tensioning tendons at the support, which maximizes the drapes and minimizes losses. However, Option B requires twice as many splices compared to Option A. Option B also provides the possibility of constructing the in-span splice connections at ground level. However, this requires the ability to lift girders that are approximately twice as heavy and longer compared to Option A.

Selection of the construction method and the connection splice details depends on the terrain, available equipment, and experience of the local contractors. Successful teamwork of the designers, fabricators, and contractors from the planning stages can take advantage of the economical use of materials, equipment, and on-site personnel.

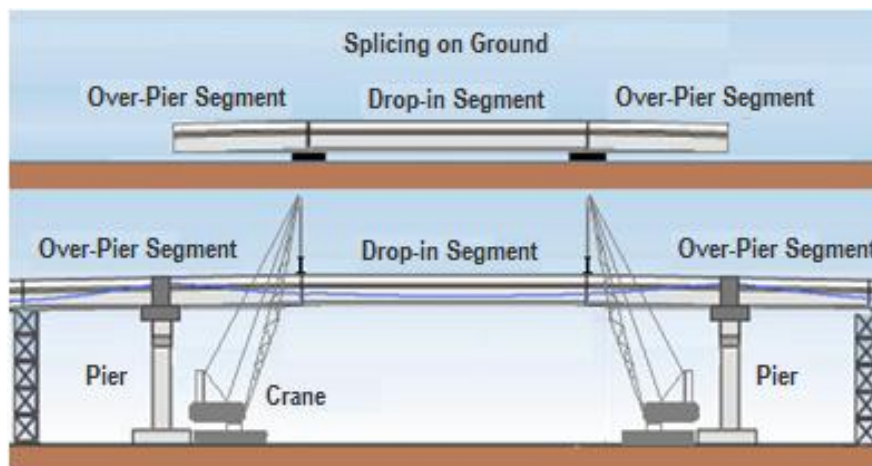
7.3.3 Transportation and Erection

A spliced girder bridge system allows the precaster to fabricate the girder segments in easy transportable lengths to achieve a new span range with a relatively simple form of construction. The transportation limitations on land imposed by different states control the length of the girder segments to be considered in the design.

The girder segments are typically pretensioned for transportation and erection on site. Figure 7.2 elaborates further on the transportation phase for the drop-in and over-pier girder segments. The way in which the girder segments are supported in both the phases should be considered in the design development stage.



(a) Option A – Single girder segment over pier



(b) Option B – Spliced girder segment over pier

Figure 7.1. Schematic of Two Different Construction Options for Continuous Spliced Girders.

Haunched girder segments over the piers have been used on specific projects. These heavyweight and deep girder segments present improved structural efficiency for longer spans.

Due to the transportation limitations on land, bridges using these sections are most commonly found crossing waterways where they can be easily transported on a barge.

Lifting of the girders on the construction site may be an issue for heavy sections such as a U-girder. The upper limit for standard lifting and hauling equipment is approximately 80 tons (160 kips) (Castrodale and White 2004). Lifting operation of heavier girders may delay the construction schedule and increase the costs substantially. Lifting equipment such as a crane or temporary supports are required to hold the girder segments when the splice connection is cast and gains adequate strength. Temporary shoring towers are installed on the construction site to support the girder segments placed in position. These towers do not apply upward force on the girders.

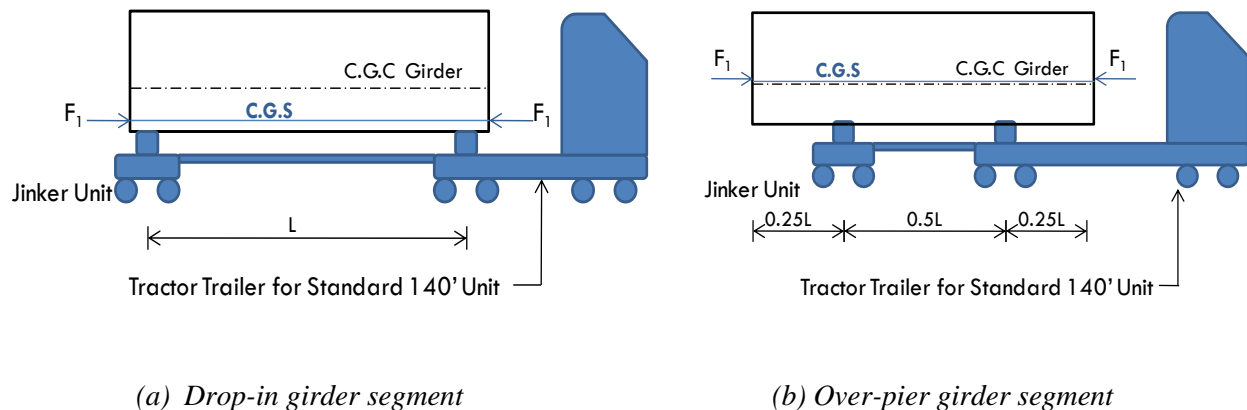


Figure 7.2. Transportation of Girder Segments.

7.3.4 Post-Tensioning

The use of full-length post-tensioning has the potential to provide superior long-term serviceability. This method can improve the structural efficiency of the bridge by virtually eliminating the cracking of the deck over the supports and load balancing the entire deadweight.

Both single-stage and two-stage post-tensioning has been used in practice for continuous bridge construction. For two-stage post-tensioning, the first stage post-tensioning is applied to balance the girder self-weight and obtain a straight profile of all the girder segments to be joined together. This creates an ideal condition for casting the concrete bridge deck and the CIP concrete for the splice connections. The splice connections between the ends of the precast, prestressed concrete girder segments are cast along with the deck closure locations after coupling

the post-tensioning tendons that provide continuity. After the CIP deck concrete has cured and reached sufficient strength, second stage post-tensioning is carried out to balance the deck weight as a composite girder section. The temporary shoring is then removed and can be reused for the next span.

When the prestressed concrete girders are post-tensioned for the first stage before casting the deck, the variation in camber of the girder profile along the alignment of the bridge is fixed by adjusting the deck forms and elevation. After casting the deck and second stage post-tensioning, any elevation adjustments can be made with suitable details that should be considered in the design. These details should allow for relative movement between the girder segments during the tendon stressing operations. The post-tensioning design should accommodate any increase or decrease in the stressing force to adjust the camber. Sometimes it might be necessary to provide additional slab or wearing surface thickness to match the final grade, which should be accounted for in the design.

In the case of curved alignments formed by short, straight girder segments, the splice should be detailed accordingly to accommodate curved post-tensioning ducts. The curvature in plan will contribute to frictional losses. The tendon force due to this curvature will have a significant effect on the interior-most girder segment. Proper design and detailing at these splice locations will prevent splice concrete from spalling off.

The potential for voids and corrosion in grouted post-tensioning tendons is a concern. Improvements in design details and materials can decrease the occurrences of corrosion. The Specification for Grouting of Post-tensioned Structures (Post-tensioning Institute 2003) provides requirements for proper grouting and inspection practices.

7.4 SPLICE CONNECTIONS

Mass construction and potentially long project durations characterize large infrastructure projects. The choice of the method of construction plays a very important role in the overall cost of the bridge.

Although the most commonly used continuity details adopted in different states are single, full-span girder segments joined at the intermediate diaphragms over the piers, these do not strictly produce any significant increase in the span length of the bridge due to practical limitations such as size and weight for transportation and erection. Therefore, to provide a

significant increase in the span length, in-span splices can be introduced using one of the two methods shown in Figure 7.1. In either case, longer spans can only be achieved if there are drop-in span units with splices located at either the quarter point (which could double the span length) or near the inflection point. Different types of splice connections have been used on specific projects. An important construction issue is the detailing of the splice. The splice details should be simple and easy for the contractor to construct under difficult site conditions.

Based on the state-of-the-practice review, the splice connection details can be broadly categorized into four major types as discussed in the following sections. Table 7.1 presents a comparison of the different types of splice connection details with respect to construction, serviceability, and advantages and disadvantages of each approach.

Table 7.1. Types of Splice Connection Details.

Construction Approach	Serviceability	Advantages	Disadvantages
Fully Prestressed (Caroland et al. 1992 and Abdel-Karim and Tadros 1995)			
<ul style="list-style-type: none"> ▪ Shored construction. ▪ Multiple trades and processes and difficult to construct. ▪ Girder segments are made continuous by splicing and coupling the post-tensioning tendons. 	<ul style="list-style-type: none"> ▪ Better serviceability and durability of the deck by elimination of cracking. 	<ul style="list-style-type: none"> ▪ Structural efficiency and long term performance is improved. ▪ Cost of post-tensioning is offset by use of few girder lines and greater spacing between girders. 	<ul style="list-style-type: none"> ▪ Difficulty in installation and stressing of the crossing prestressing tendons at high elevations. ▪ Thickened blocks at girder ends are required to anchor the partial post-tensioned tendons. ▪ Concerns about losses in the partial post-tensioned tendons.
Partially Prestressed – Option 1 (Abdel-Karim and Tadros 1995, Miller et al. 2004, and Newhouse et al. 2005)			
<ul style="list-style-type: none"> ▪ Shored Construction. ▪ Continuity connection provided between the ends of girders by post-tensioning and extending 180 degree mild steel bent bars. ▪ Vertical rebars are provided for resisting shear. 	<ul style="list-style-type: none"> ▪ Crack width is controlled by rebar. Cracking at the interface can be controlled by providing additional reinforcement. 	<ul style="list-style-type: none"> ▪ Connection is able to transfer service loads effectively. ▪ Combination of post-tensioning and mild steel is expected to provide better durability and performance. ▪ No thickened ends of girders required at the splice connection. 	<ul style="list-style-type: none"> ▪ Slight initial cracking may occur at the diaphragm-girder interface during construction. ▪ Embedment of 180 degree mild steel bent bars in the girders may add to the initial cost.
Partially Prestressed – Option 2 (Caroland et al. 1992 and Tadros and Sun 2003)			
<ul style="list-style-type: none"> ▪ Shored as well as unshored construction. ▪ Shear connectors are embedded in the adjacent girder segments and connected using a steel shoe on construction site. Threaded bars are provided in the top flanges of girder segments and coupled at the joint. 	<ul style="list-style-type: none"> ▪ Fully load-balanced except at joint. ▪ Continuity post-tensioning and threaded bars permit some load-balancing at the joint. 	<ul style="list-style-type: none"> ▪ Residual compression in deck for serviceability and deflection control. ▪ Use of shear connectors facilitates unshored construction, if necessary. ▪ Elimination of congestion of reinforcement at the joint. 	<ul style="list-style-type: none"> ▪ High maintenance cost for the shear connectors. ▪ Careful alignment of the adjacent beams and high control of fabrication and erection tolerances. ▪ Embedment of shear connectors and threaded bars in the girders may add to the initial cost.

Table 7.2. Types of Splice Connection Details (continued).

Construction Approach	Serviceability	Advantages	Disadvantages
Fully Reinforced (Abdel-Karim and Tadros 1995, Miller et al. 2004, and Newhouse et al. 2005)			
<ul style="list-style-type: none"> ▪ Shored construction. ▪ Most simple for field applications. ▪ This splice connection is typically provided over the pier. ▪ Diaphragm is cast over the pier. 	<ul style="list-style-type: none"> ▪ Cracking in the deck and bottom of diaphragm at the joint. ▪ Needs inspection and repair over time. 	<ul style="list-style-type: none"> ▪ Simple to construct and relatively economical. ▪ Could develop adequate resistant moments. ▪ This connection detail avoids the need for professional post-tensioning contractors. 	<ul style="list-style-type: none"> ▪ Potential for cracking due to lack of prestressing through the connection. ▪ Congestion of reinforcement in the joint.

7.4.1 Fully Prestressed Splice Connection

The fully prestressed splice connection detail, shown in Figure 7.3, can be used for both on-pier and in-span splices. This connection involves multiple trades and processes and can be more difficult to construct. Pretensioned girder segments are post-tensioned across the splice using short length tendons or thread-bars. The gap between girder segments is filled with high strength concrete or grout. The partial length post-tensioned tendons, along with continuity post-tensioning, resist the service stresses and are designed to meet the ultimate strength requirement at the connection. Closely spaced stirrups are provided to resist shear at the splice location. Precast girders are placed on falsework or temporary end supports, usually located near the inflection points.

This type of connection imparts superior serviceability and durability to the deck by eliminating cracking, which enhances the structural efficiency and long-term performance of the whole bridge system. Precise alignment of the post-tensioned ducts is needed; otherwise, considerable frictional losses can result, which may undermine the effect of the post-tensioning. End blocks are required at the splicing ends of the girders to anchor the post-tensioned tendons used in the connection and provide for end zone reinforcement to resist the concentrated stresses due to post-tensioning forces.

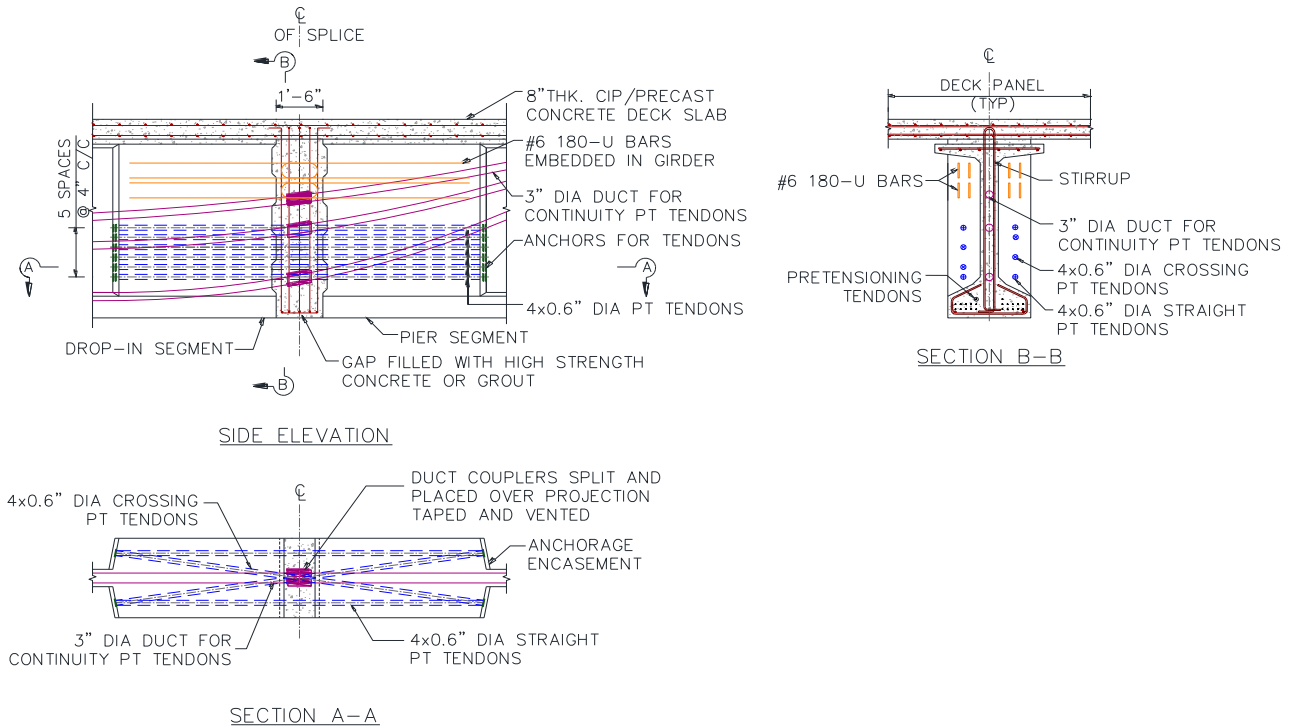


Figure 7.3. Fully Prestressed Spliced Connection Detail.

7.4.2 Partially Prestressed Splice Connection

Partially prestressed splice connections combine the merits of both reinforced and post-tensioned splice connections. These details can be used for both on-pier and in-span splicing methods. Two partially prestressed connection details are discussed below. The first partially prestressed connection detail, shown in Figure 7.4, uses mild steel reinforcement in addition to continuity post-tensioning running through the connection.

The mild reinforcement consists of 180-degree hooked bars anchored into the adjacent girder flanges and extending into the joint. The gap between girder segments is filled with high-strength concrete or grout. This connection is able to transfer service loads effectively. The mild steel bent bars are also designed for the maximum factored design loads. Vertical reinforcement is provided to strengthen the splice connection for shear. Cracking at the interfaces of the CIP connection and precast girders can be controlled by providing additional reinforcement.

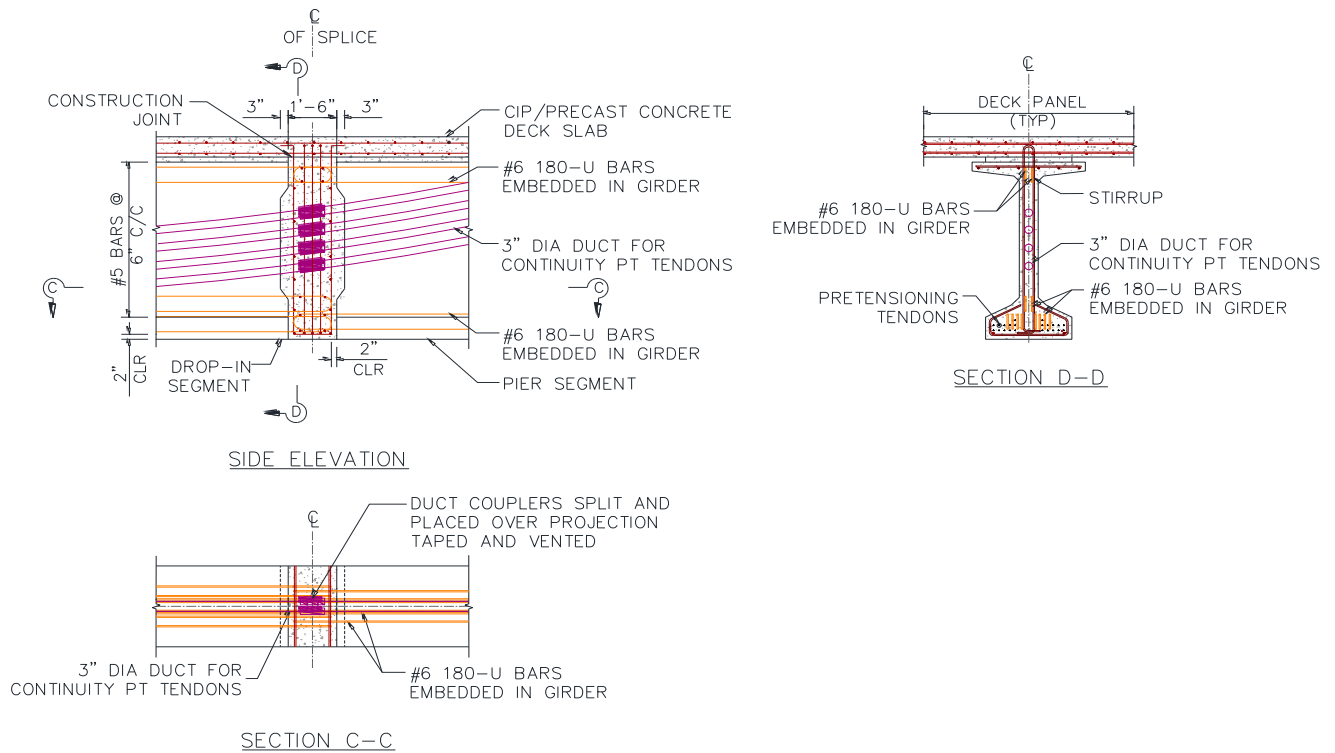


Figure 7.4. Partially Prestressed Spliced Connection Detail: Option 1.

The second partially prestressed connection detail, shown in Figure 7.5, uses shear connectors in the form of rectangular steel sections that are embedded in the adjacent girder segments. These steel hanger sections are properly aligned onsite and a steel shoe is then fitted and pinned onto the bars of the adjacent pier segments. The drop-in spans are placed in position, and their projecting bars are seated and pinned on to the shoe. Threaded bars are provided in the top flanges of girder segments and coupled at the joint. The gap between girder segments is filled with high strength concrete or grout.

This connection detail is fully load-balanced, except at the joint location. The residual compression in the deck improves serviceability and deflection control. The use of shear connectors facilitates unshored construction, if necessary, and eliminates congestion of reinforcement at the joint. However, the maintenance cost for the shear connectors is high because corrosion may severely affect performance of the continuity joint. Careful alignment of the adjacent beams and high control of fabrication and erection tolerances is required. Additional supporting members are required on the webs of girder segments for stability during construction.

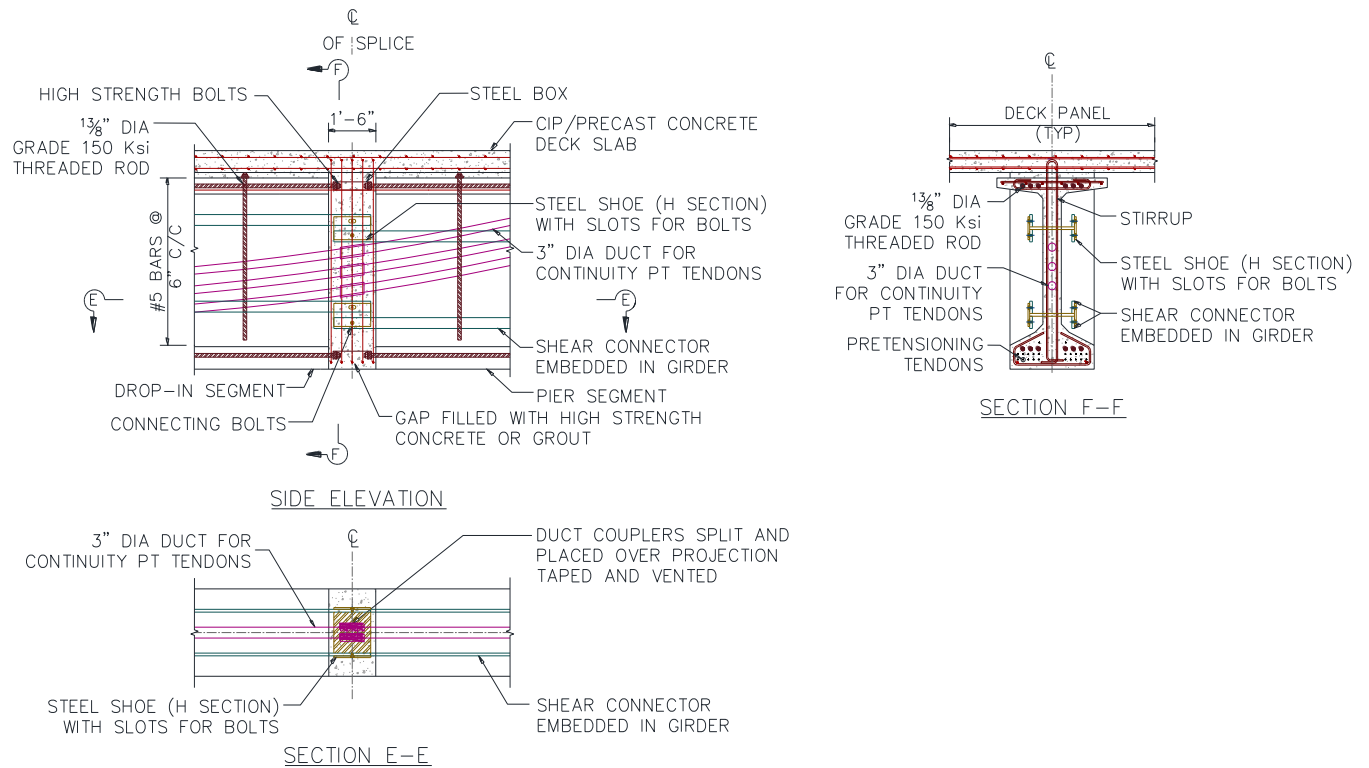


Figure 7.5. Partially Prestressed Spliced Connection Detail: Option 2.

7.4.3 Fully Reinforced Splice Connection

The fully reinforced splice connection, shown in Figure 7.6, is typically employed for on-pier spliced systems. Mild steel reinforcement projects from the ends of the precast girder segments with sufficient development length and is lap spliced within the CIP closure pour. Using stainless steel reinforcement could be beneficial in this type of connection detail due to its enhanced corrosion resistance as compared to standard mild steel reinforcement. For in-span splices, the precast girder segments are positioned end-to-end on a temporary support, usually near the dead load inflection point, and concrete is CIP at the splice.

The deck slab and joint concrete are cast together, providing continuity. This detail is the most simple for field applications. However, as the splice is non-prestressed, there is a possibility of cracking at the top surface under full service loads and development of cracks at the bottom of the diaphragm due to positive restraint moment over the piers resulting from creep. Regular inspection for corrosion should be conducted, and in severe conditions, repair may be required for this detail. If necessary, adequate reinforcement should be provided at the top and in the deck

slab for protection against possible corrosion. A major advantage of this connection is that it avoids the need for an onsite post-tensioning subcontractor, but this may be at the expense of congestion of reinforcement in the joint.

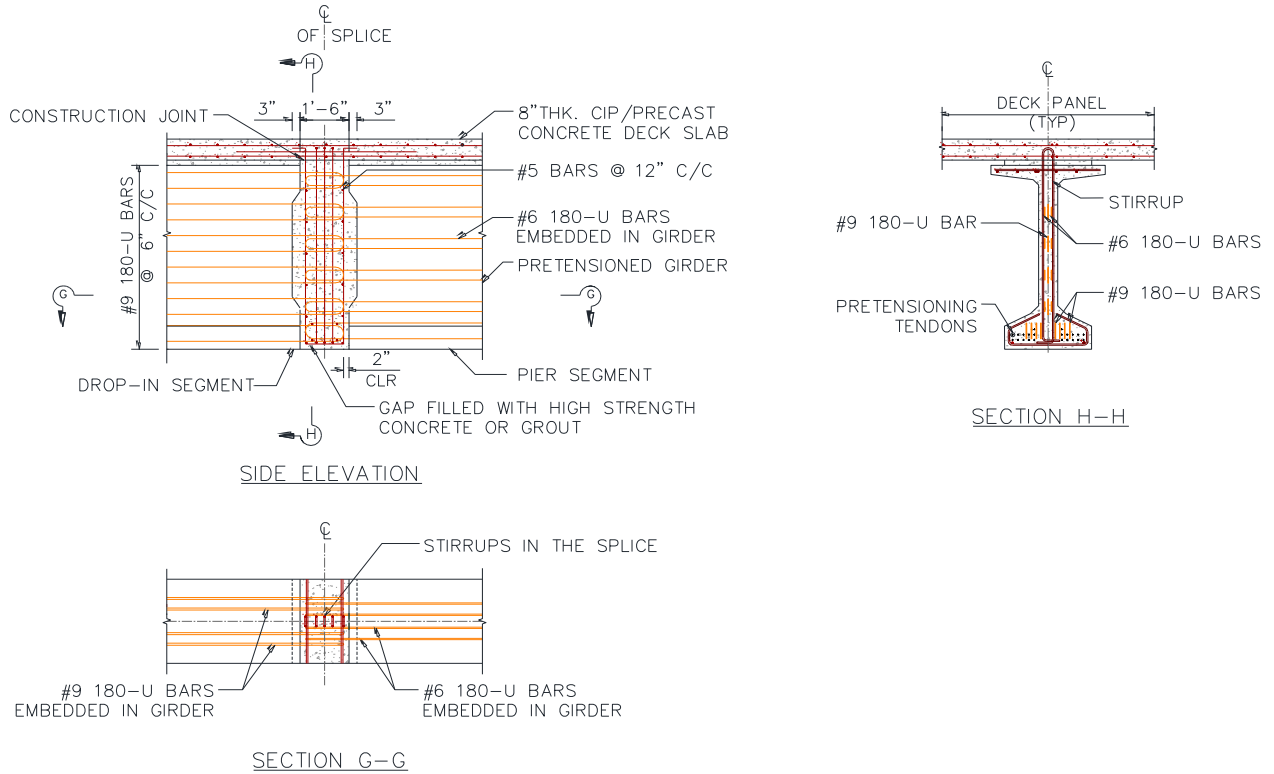


Figure 7.6. Fully Reinforced Spliced Connection Detail.

8. INDUSTRY FEEDBACK TO PRELIMINARY DESIGN AND DETAILS

8.1 INTRODUCTION

Input from the stakeholders who will be involved in implementing the research results is critical in evaluating the feasibility of this project. These include TxDOT engineers, precasters, and contractors. The research team held focus group meetings to present findings from the literature review and state-of-the-practice and preliminary designs, and to solicit input regarding the potential implementation of promising continuity details for precast, pretensioned girders made continuous.

The TxDOT Project Monitoring Committee (PMC) provided input to narrow down specific options of interest. One outcome of the discussion with TxDOT PMC was the decision to focus on in-span splice connections, which will provide the greatest potential for increasing span lengths using standard precast girder sections. A plan to involve precasters and contractors was identified for follow-up meetings.

A letter of invitation was developed and sent to precasters and contractors from the industry to attend the focus group meetings. The research team developed questionnaires for precasters and contractors from the industry, with input from the PMC, to collect feedback on the preliminary design and details that the team developed. In addition, information related to the preliminary details of the proposed splice connections was distributed to the precasters and contractors. The information and questionnaires included four connection styles for in-span splices of standard TX girders; specific feedback was requested on the connection types, as well as other considerations related to design, precasting, shipping, and construction. This chapter summarizes the information and industry feedback from the precasters and contractors.

8.2 PRECASTER INPUT

The research team, available TxDOT PMC members, and precasters met to discuss the findings from the literature review and state-of-the-practice and preliminary designs, along with potential continuity details. The Precast Concrete Manufacturers' Association (PCMA) of Texas was contacted to help extend the invitation to precasters in Texas to attend the meeting.

This section outlines the input from the precasters regarding the potential implementation of promising continuity details for precast, pretensioned girders made continuous. The responses

from precasters helped to identify potential issues with respect to precasting and shipping, along with economical and reliable details in terms of the precasting operation for the girder segments of the spliced bridge system.

The questions of the research team and responses from the precasters are as discussed below.

Tell us about your past experience in fabricating and/or constructing continuous, spliced, precast, prestressed I-girder bridges for span lengths exceeding 180 ft.

- Fabricated and/or constructed about 150 ft long spans using 8 ft 6 in. deep I-girders for a railroad bridge project.
- Texas Concrete Company fabricated 96 ft long haunched I-girders for a bridge (not constructed yet) with a main span of about 220 ft. The depth of the haunched girders varied from that of standard AASHTO Type VI section at the ends to about 15 ft at the center over the pier. The bottom flange is deepened to achieve the increase in depth.
- Fabricated and shipped precast I-girder segments up to 165 ft long.
- Increasing the span length results in an increase in the weight of precast elements. The standard weight limit is 200 kips. Otherwise, there is a significant cost increase. Precautions should be taken so that the weight does not exceed 200 kips because of transportation limitations.
- The standard transportation length limit is 140 ft. Increasing the span length makes transportation of precast elements difficult, thereby increasing the transportation costs.
- Texas Concrete Company fabricated a 180 ft precast, pretensioned girder for a railroad in Ft. Worth on I-35. It was also mentioned that a 160 ft length is still okay to work with, but a 140 ft length is better for transportation. Working with lengths of 180–200 ft lengths causes delivery prices to go up.
- TexPro software was noted as a good tool for determining transportation limitations in terms of weights and vertical clearances for a given route.

1. Is your precasting plant adaptable to accommodate fabricating a variety of over-pier, end, and drop-in girder segments? Note that the supports for segments during transportation differ for each type, as shown in Figure 7.2.
 - In general all the precasting plants are well equipped to handle fabricating a variety of over-pier, end, and drop-in segments.
 - The desirable maximum length for all girder segments is around 140 ft, with maximum weight around 180–200 kips, and maximum depth of 10 ft.
 - If necessary, harping of the pretensioned strands can be done for the drop-in and over-pier segments without any issues.
 - Standardizing the precast elements will help reduce the overall cost.
2. How well placed is your plant in adapting to a haunched girder for the over-pier segments, as shown in Figure 2.7 of this report? What issues could be anticipated in forming, lifting, and transporting these modified sections, as shown in Figure 8.1? How much additional cost, relative to standard girder fabrication, would be anticipated? This could be an estimated percent increase, for example.

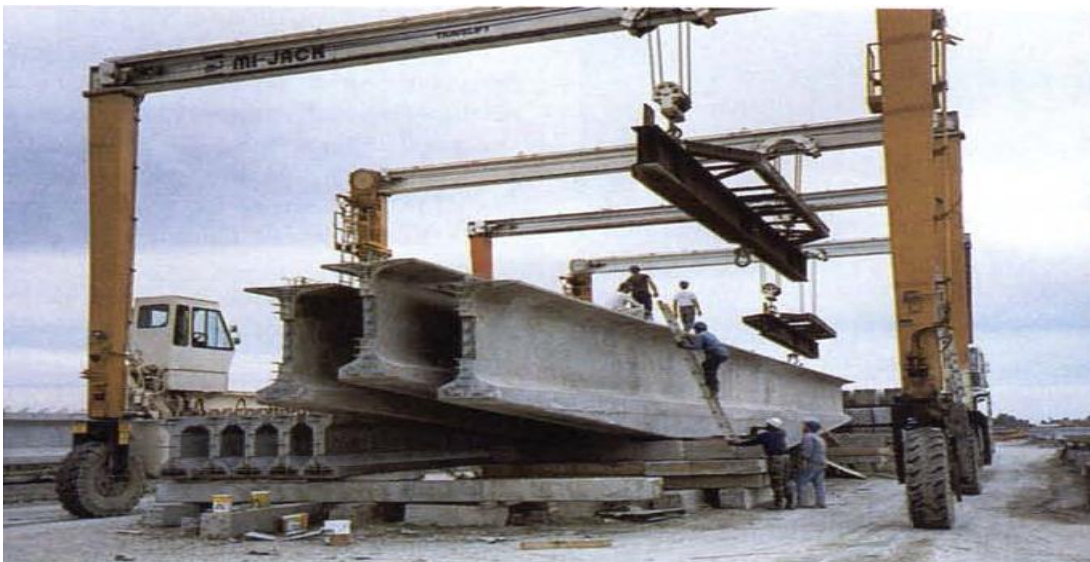


Figure 8.1. Transportation of Haunched Girder Segment (Janssen and Spaans 1994).

- Although the plants are well placed with regard to adapting to haunched girder elements, there can be difficulties with respect to storage of haunched girder elements.
 - Transportation problems are anticipated for I-girder segments with lengths exceeding 160 ft and depths exceeding 10 ft especially over narrow roads and in hilly areas. Good limits for shipping are 200 kips, 10 ft maximum haunch height, and 140 ft maximum girder length.
 - Approximately two cranes, each with 100 ton capacity, are required to lift the haunched girders onsite. Stability issues can be anticipated.
 - The forms required for haunched girders result in a higher initial cost. This cost is almost two to three times more compared to using the standard girder section depth.
 - Use of a constant standard girder section depth for over-pier segments is preferred over the haunched girders.
3. Are there any issues related to lateral instability or buckling of the deep I-girder sections during transit in Texas? Does this require additional pretensioned strands or a different support arrangement for stiffening the top flange?
- Issues related to instability of I-girder shapes could arise for drop-in segments with spans in excess of 160 ft. However, there have been no such issues for spans varying from 140 ft to 160 ft.
 - Not many projects have used the Tx70 sections to anticipate other concerns.
 - Stability issues depend in part on the shipping surface.
 - The use of a wider flange would be helpful for stability in cases where the web and flange are widened.
 - For the U-girder shapes, it is recommended to limit the segment length to 130 ft considering weight limit for transportation.
 - The recommended maximum span length for a spliced girder bridge is around 260 ft considering the stability issues of long span drop-in segments and deep haunched over-pier segments.

4. Are there any concerns about potentially widening the girder web to accommodate larger post-tensioning tendon ducts and shear reinforcement in the standard TX girder sections, as shown in Figure 8.2? How much additional cost, relative to standard girder fabrication, would be anticipated? This could be an estimated percent increase, for example.

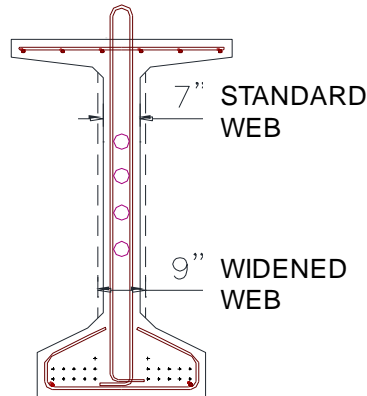


Figure 8.2. Tx70 Girder Section with Widened Web.

- The main concern related to an increase in web thickness is that it will result in an increase in self-weight. Care should be taken so that the weight of the precast element does not exceed 200 kips.
 - There are no concerns with widening the webs. The webs can be widened by increasing the space between the forms, which will result in widened top and bottom flanges of the girder section. It is a one-time cost to purchase new soffit. Standardizing the precast elements will help reduce the overall cost.
 - In order to resolve the issue of maximum shear demand at the supports, it is acceptable to have widened webs for over-pier segments only. The drop-in segments can have the standard web width. However, this may not look aesthetically pleasing at the connection where the transition is made.
 - The TxDOT PMC noted that the Tx82 girder will have 10 in. wide webs.
5. Are there any concerns about potentially thickening the girder ends (relative to the web width) to accommodate post-tensioning anchorages, placement of anchorages, tendon ducts, and shear reinforcement in the standard TX girder sections, as shown in

Figure 8.3? How much additional cost, relative to standard girder fabrication, would be anticipated? This could be an estimated percent increase, for example.

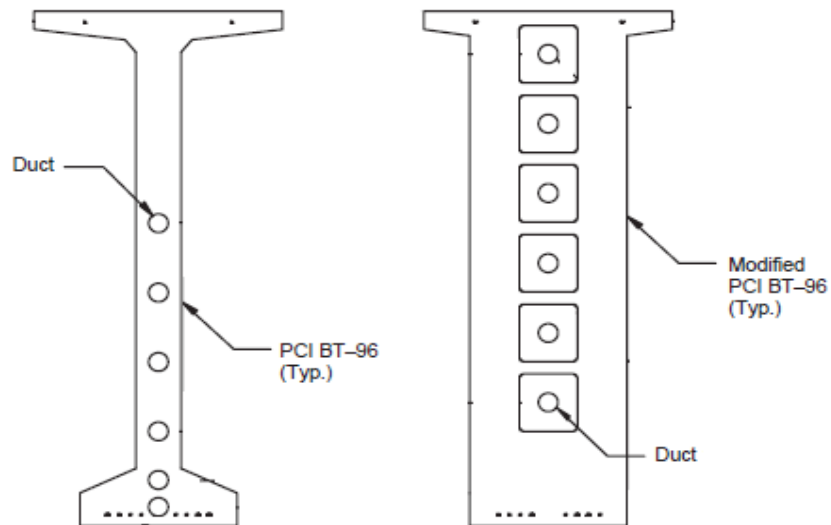


Figure 8.3. Thickened End of Girder (Castrodale and White 2004).

- Fabricating end segments with thickened ends is not an issue. The length of an end block is typically 10–15 ft. End blocks were used for a number of years, but are not currently standard.
 - The main concern related to thickening the girder ends is the increase in the weight of concrete and increase in the cost of formwork. However, it is a one-time cost, and standardizing the precast elements will help reduce the overall cost.
6. What other issues are anticipated in building the different types of spliced connection details as described in Chapter 7 of this report and shown in Figures 7.3 to 7.6? How much additional cost, relative to standard girder fabrication, would be anticipated by the details? This could be an estimated percent increase, for example.
- Connection 1: Fully prestressed spliced connection.
- Connection 2: Partially prestressed spliced connection – option 1.
- Connection 3: Partially prestressed spliced connection – option 2.
- Connection 4: Fully reinforced spliced connection.

- Connection 1
 - For this connection the main concern was related to post-tensioning losses in the short PT strands. TxDOT engineers anticipated about a 75 percent loss in prestress in the shorter tendons that cross one another. It was suggested to use full length PT to achieve typical stress levels.
 - There were also issues raised regarding the aesthetics of the connection because the ends of the girders would be thickened.

- Connection 2
 - This connection was the most preferred with respect to on-site construction.
 - This detail was thought to be better in terms of aesthetics, because the web is not required to be thickened at the splice connection.
 - A combination of post-tensioning and mild steel is expected to provide better durability and performance.
 - Precasters had no issues with fabricating the girder segments with the embedded 180-degree bars. They preferred this connection as compared to Connection 1.

- Connection 3
 - This connection was found to be complex for construction. Some amount of shoring would still be preferred in spite of this detail facilitating unshored construction (embedded steel H section provided similar to the Cazaly hanger in the Shelby Creek Bridge, Kentucky).
 - There was no strong preference for using bolted, threaded rods instead of the mild steel reinforcement provided in Connection 2. Precasters did not prefer this connection as compared to Connection 2.

- Connection 4
 - The precasters liked this detail in terms of fabrication and felt it would be less expensive to fabricate.
 - However, the potential for cracking was noted as a disadvantage due to the lack of prestressing through the connection.

- It was also noted that not using continuous PT through the connection could limit the span length.
 - It was recommended to consider the potential for deck cracking at this connection.
7. What are your perspectives and/or suggestions regarding methods to make spliced girder bridges more constructible in the future?
- Increasing the span length results in an increase in the weight of precast elements. Precautions should be taken so that the weight does not exceed 200 kips considering transportation limits.
 - The desirable length for girder segments is around 140 ft, weight around 180 to 200 kips and depth around 10 ft.
 - The recommended maximum span length for a spliced girder bridge is around 260 ft considering the stability issues of long span drop-in segments and deep haunched over-pier segments.
 - Use of a constant standard girder section depth for over-pier segments is preferred over the haunched girders to avoid issues related to high initial cost of fabrication and stability issues during transportation.
 - Some discussion was held about using longer precast panels over the supports with longitudinal prestressing. The precasters indicated that this should be no problem.
 - The TxDOT PMC indicated a preference for Connections 2 and 3 and noted that it would be simpler to complete all post-tensioning before casting the deck.

8.3 CONTRACTOR INPUT

The research team, available TxDOT PMC members, and contractors met to discuss the findings from the literature review and state-of-the-practice and preliminary designs, along with potential continuity details. The preferences and concerns of the precasters were also discussed. The Association of General Contractors (AGC) of Texas was contacted to help extend the invitation to the meeting.

This section outlines the input from the contractors regarding the potential implementation of promising continuity details for precast, pretensioned girders made

continuous. The responses from contractors helped to identify potential issues with respect to the construction, along with the preferred details and methods that will ensure safe, reliable, and efficient construction of continuous spliced precast, prestressed concrete bridge systems.

The questions of the research team and responses from the contractors are as discussed below.

1. Tell us about your past experience in constructing continuous, spliced, precast, prestressed I-girder bridges for span lengths exceeding 180 ft?
 - The contractors had no past experience in constructing continuous, spliced, precast, prestressed I-girder bridges for span lengths exceeding 180 ft.
 - There is a project under way that involves construction of haunched girders segments having a span of 220 ft.

2. What is the preferred construction technique for long span bridges: shored or unshored construction?
 - The most preferred technique of construction is unshored construction. It saves significant time during construction and also reduces the cost of construction.
 - Shored construction should be the last resort. It increases the cost of construction significantly and also reduces the speed of construction.
 - The speed of construction is also affected by the amount of available capital money for the project. It is preferable to have two design options: with and without time constraints.

3. How well placed is your firm in adapting to a haunched girder, as shown in Figure 2.7 of this report, for the over-pier segments? If you were to construct precast, prestressed concrete haunched girder segments, would your preference be to construct a constant bulb haunch or a constant web depth? What issues could be anticipated in lifting and placing these modified sections on site? How much additional cost, relative to standard girder erection, would be anticipated? This could be an estimated percent increase, for example.

- Use of a haunched girder should be more of a concern for precasters rather than contractors.
 - The contractors have no limits for lifting loads by cranes and have lifted up to 250 tons of precast segments in the past. They can lift whatever is brought to the job site. They noted that 250 kips and above requires a super heavy permit (permit is ~\$800 and engineering is ~\$2000).
 - Contractors prefer the constant web depth option because it is easy to fabricate and has more stability.
 - Contractors do not anticipate any issues in lifting and placing the modified sections. However, they had questions regarding the width of the bent cap to support the huge haunched girder segments.
 - From a construction point of view, the use of shore towers would increase the cost of construction relative to standard girder erection.
4. Would you prefer two separate girder segments with a connection at pier or a single segment continuous over the pier, as shown in Figure 8.4?

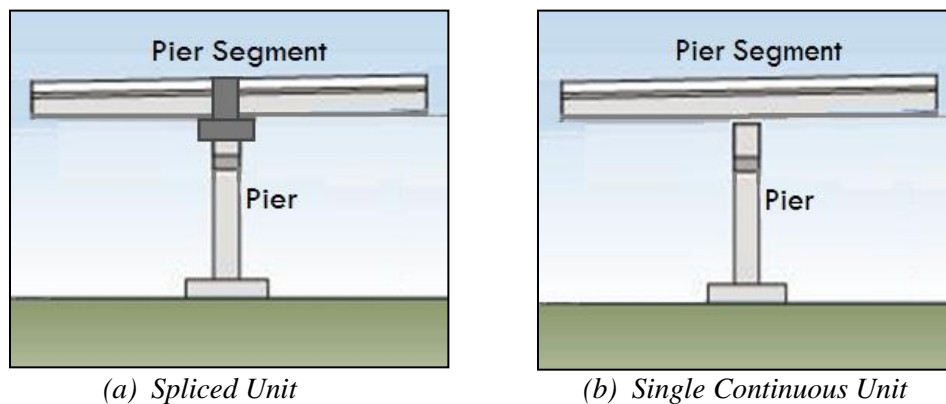


Figure 8.4. Over-Pier Girder Segments.

- Contractors are fine with either configuration for the over-pier segments.
- However, the option of two separate girder segments spliced over the pier provides flexibility of splicing the girder segments within span on ground before lifting them into place onsite.

5. What issues do you anticipate if the girder segments for a long span are spliced on ground and then lifted into place, as shown in Figure 7.1.(b)?
 - Contractors do not anticipate any issues in lifting and placing the spliced girder segments. This is a preferred option because no temporary shoring is required on-site.
 - Other concerns related to the spliced, long span of the bridges are access, traffic control, transportation of girders, and use of cranes from outside sources (larger tonnage rating).

6. Does your firm have experience with combined pre-tensioning and staged post-tensioning for continuity operations in spliced girders? Are there any concerns with this type of construction?
 - Contractors do not have experience with respect to combined pre-tensioning and staged post-tensioning for continuity operations in spliced girders. There are no special concerns in this regard.
 - The additional information needed is with respect to the construction sequence of the cast-in-slab.
 - The contractors suggested testing the behavior of the continuous bridge under vibrations due to construction and live loads.
 - Suggestions were made to the contractors to have a consulting engineer to monitor the effects of creep, shrinkage, and temperature on the continuous bridge.

7. Do you prefer to use your own forces or a specialty sub-contractor to carry out post-tensioning operations? How does this affect the construction schedule?
 - Some of the contractors do have their own forces to carry out the post-tensioning operations. However, most contractors will engage a subcontractor to handle the post-tensioning operations.
 - Appointing a subcontractor does not affect the construction schedule.

8. What measures do you prefer to ensure correct vertical alignment of girders onsite? What are other issues that may be encountered during erection of the girder segments?
- Contractors have no past experience in this regard but can adopt techniques such as cross-bracings in addition to usual construction measures to ensure correct vertical alignment of girders onsite.
 - The TxDOT PMC noted that the main issue during erection of the girders is the lateral stability of the over-pier segments, especially over the negative moment region.
 - Other concerns during erection include extended freeway closures and transportation of the girder segments.
9. An important issue is to ensure the integrity of the splice between girder segments. What are your concerns related to the construction of quite long continuous tendons, coupling of the ducts, and the site casting at the splice?
- Contractors have no special concerns related to the construction of long continuous tendons, coupling of the ducts, and the site casting at the splice.
10. What are the minimum preferred dimensions to achieve a quality splice between girder segments?
- Contractors have no past experience with casting splices between girder segments. Therefore, they are not able to specify the minimum preferred dimensions to achieve a quality splice between the girder segments.
11. Do you have any preferred method of grouting tendons?
- Contractors do not have any preferred method of grouting tendons.
12. Do you have experience with external post-tensioning in box-girders (going through the void space)? If yes, do you sense that this is less or more costly than internal post-tensioned tendons (running through the webs and flanges of the section)?

- Contractors do not have any past experience with external post-tensioning in the box-girders. Therefore, they were unable to cite any concerns related to the cost and construction of such post-tensioning systems.

13. What other issues are anticipated in building the different types of spliced connection details as described in Chapter 7 of this report and shown in Figures 7.3 to 7.6? How much additional cost, relative to standard girder erection, would be anticipated by the details? This could be an estimated percent increase, for example.

Connection 1: Fully prestressed spliced connection.

Connection 2: Partially prestressed spliced connection – option 1.

Connection 3: Partially prestressed spliced connection – option 2.

Connection 4: Fully reinforced spliced connection.

The contractors noted that selection of a connection detail depends on the life-span of the bridge structure and whether the connection details will provide the durability desired over the life-span of the structure. Because of this, selection of the connection details tends to be a design decision. However, it is good to have more options when it comes to construction.

- Connection 1
 - For this connection the main concern was related to installation and stressing of the crossing tendons at high elevations.
 - There were also issues raised regarding the loss of prestress in this connection and potentially the cost.
 - One cost estimate provided was \$7500 per splice.
- Connection 2
 - This connection was the most preferred with respect to onsite construction due to its relative constructability.
 - It was noted that Connection 2 was preferred over Connection 1.

- Fabricating the girder segments with the embedded 180-degree bars is more of a concern for precasters rather than contractors. Precasters had no issues with the fabrication.
- One cost estimate provided was \$5000 per splice.
- Connection 3
 - This connection was found to be preferable as an unshored option because of the embedded steel H section (similar to the Cazaly hanger in the Shelby Creek Bridge, Kentucky), although it is complex for construction.
 - This connection was found to have the benefit of constructability as suggested by the contractors.
 - Connection 3 was also thought to have some advantages as a good solution when a longer design life is required.
 - One cost estimate provided was \$5000 per splice.
- Connection 4
 - This is a preferred solution with respect to ease of construction.
 - There were questions raised regarding the durability of this connection and serviceability of the deck slab. Therefore, the required design life of the structure may dictate whether this is a good option. This might be considered for a shorter design life.
 - One cost estimate provided was \$5000 per splice.

14. What are your perspectives and/or suggestions regarding methods to make spliced girder bridges more constructible in the future?

General Comments

- The proposed bridge system would be an ideal substitute for steel bridges especially in coastal areas where corrosion of steel bridges is a major problem.
- The TxDOT PMC noted that this bridge type would compete well with shorter span segmental bridges. It was noted that TxDOT is not using steel girder bridges along

the coast. It was also pointed out that the proposed bridge type would not compete with just steel girder bridges.

- It was noted that the Webber Company is constructing the Sylvan Avenue Bridge for TxDOT. The bridge was designed by HNTB and uses Tx82 shapes with continuity with in-span splices (both haunched and prismatic sections over the pier are used within the bridge).

Cost Concerns

- The contractors suggested that there should be a cost analysis taking into account the life-cycle cost of the bridge, along with a cost comparison between a steel bridge and a precast concrete bridge.
- TxDOT is just now starting to use life-cycle cost analysis. Traditionally, initial cost has been used.
- Statements in the literature that the proposed system is cost competitive with steel should be reviewed in terms of costs for Texas bridges. It was noted that AGC can provide cost information upon request.
- It was noted that using fewer girders increases cost competitiveness of bridges.
- Contractors prefer having two design options for bid: one with a standard precast concrete girder shape and one with a steel plate girder. The TxDOT PMC noted that this would be done to get cost competitive bids.

Construction Concerns

- The quality control process is more complex for the proposed bridge system.
- Sequencing of the CIP concrete and PT operations are needed up front.
- Unshored construction is preferred because it saves a lot of time during construction and reduces the construction costs. Often the required footprint is not available to place shore towers. It was estimated that each tower would add \$50k–\$200k.
- Contractors look at both schedule and economy to determine best option.
- Time-dependent issues onsite need to be considered.

Performance Concerns

- There was a concern raised regarding the larger span-to-depth ratio for the presented precast continuous bridges and the effect it would have on the overall design. TxDOT noted that the span-to-depth ratio for the proposed bridges exceeds suggested values in AASHTO LRFD, and further review should be conducted.
- The contractors suggested considering vibrations characteristics for longer span bridges and the impact it would have on the bridge, both during construction and in-service. They suggested looking at whether any issues have been reported for similar longer span bridges.
- There was a concern about how the bearings would change for continuous construction and whether this would increase costs.

8.4 INPUT FROM A FLORIDA CONTRACTOR

In addition to the Texas contractors who attended the focus group meeting, input was received from a Florida contractor with experience in design and construction engineering of continuous, spliced, precast, prestressed I-girder bridges. Their response is provided separately below.

1. Tell us about your past experience in constructing continuous, spliced, precast, prestressed I-girder bridges for span lengths exceeding 180 ft?
 - The contractor has been involved in the design and construction engineering of about 20 continuous, spliced, precast, prestressed I-girder bridges for span lengths exceeding 180 ft.
2. What is the preferred construction technique for long span bridges: shored or unshored construction?
 - The most preferred technique of construction is unshored construction. It saves significant time during construction and also reduces the cost of construction and is found to be easy with experience.
3. How well placed is your firm in adapting to a haunched girder, as shown in Figure 2.7 of this report, for the over-pier segments? If you were to construct precast, prestressed

concrete haunched girder segments, would your preference be to construct a constant bulb haunch or a constant web depth? What issues could be anticipated in lifting and placing these modified sections on-site? How much additional cost, relative to standard girder erection, would be anticipated? This could be estimated percent increase, for example.

- Haunched girders have been used by the contractor successfully for several projects.
 - The contractor prefers the constant web depth option but this option has a span limitation of ± 225 ft. However, they do not anticipate any issues in using the constant bulb haunched girder segments.
 - A ± 15 percent additional cost is anticipated for the girders, shoring, and/or temporary works and post-tensioning.
4. Would you prefer two separate girder segments with a connection at pier or a single segment continuous over the pier, as shown in Figure 8.4?
- The contractor prefers a single continuous unit for the over-pier segments.
5. What issues do you anticipate if the girder segments for a long span are spliced on ground and then lifted into place, as shown in Figure 7.1.(b) ?
- The contractor anticipates issues related to the weight of the whole assembly and the size of the equipment in lifting and placing the spliced girder segments.
6. Does your firm have experience with combined pre-tensioning and staged post-tensioning for continuity operations in spliced girders? Are there any concerns with this type of construction?
- The contractor has experience with respect to combined pretensioning and staged post-tensioning for continuity operations in spliced girders. Pretensioning is needed for erection of the girders and post-tensioning is required for service. There are no special concerns in this regard.

7. Do you prefer to use your own forces or a specialty sub-contractor to carry out post-tensioning operations? How does this affect the construction schedule?
 - The contractor prefers to engage a subcontractor to handle the post-tensioning operations due to the amount and duration of post-tensioning operations and grouting.

8. What measures do you prefer to ensure correct vertical alignment of girders on-site? What are other issues that may be encountered during erection of the girder segments?
 - The contractor has used jacks and strong backs to ensure correct vertical alignment of girders on-site.
 - The main issue noted during erection of the girders is the lateral stability of the girder segments due to wind.

9. An important issue is to ensure the integrity of the splice between girder segments. What are your concerns related to the construction of quite long continuous tendons, coupling of the ducts, and the site casting at the splice?
 - The main concerns for the contractor are lifting weights and the stability of the girders.

10. What are the minimum preferred dimensions to achieve a quality splice between girder segments?
 - The contractor noted a minimum dimension of $\pm 3'-6"$ necessary for jacking operations and a minimum dimension of $\pm 1'-6"$ for splice connections only.

11. Do you have any preferred method of grouting tendons?
 - The contractor follows the Post-tensioning Institute (PTI) and/or American Segmental Bridge Institute (ASBI) specifications for grouting tendons.

12. Do you have experience with external post-tensioning in box-girders (going through the void space)? If yes, do you sense that this is less or more costly than internal post-tensioned tendons (running through the webs and flanges of the section)?

- The contractor has past experience with external post-tensioning in box-girders.
- It was noted that it is less costly to precast and erect the box-girders. However, a greater amount of post-tensioning is required to reduce the moment arm in capacity calculations for the strength limit state.
- For cantilever bridges, it was noted that the post-tensioning runs internally through the top slab and is draped externally.
- The contractor recommended use of diabolos instead of using pre-bent pipes for tendon deviations in case of external post-tensioning. Diabolos are curved pipes in deviators that flare open at each end (Florida Department of Transportation 2002). Tendons may enter and exit in a range of geometric angles in plan and elevation. Diabolos offer an alternative solution to calculating and detailing individual tendon pipe bends. However, structurally they are different from individual pre-bent pipes that bear evenly over a uniform radius. The actual contact length in diabolos varies, and may be quite short, depending on the minimum radius of the deviator flare.

13. What other issues are anticipated in building the different types of spliced connection details as described in Chapter 7 of this report and shown in Figures 7.3 to 7.6? How much additional cost, relative to standard girder erection, would be anticipated by the details? This could be an estimated percent increase, for example.

Connection 1: Fully prestressed spliced connection.

Connection 2: Partially prestressed spliced connection – option 1.

Connection 3: Partially prestressed spliced connection – option 2.

Connection 4: Fully reinforced spliced connection.

The contractor noted that selection of a connection detail depends on the situation and the span range adopted for the bridge. The selection of the connection details tends to be a design decision.

- Connection 1
 - For this connection the main concern was related to the large amount of post-tensioning.
 - This connection was not a preferred option.
- Connection 2
 - This connection was preferred with respect to onsite construction due to its relative constructability.
- Connection 3
 - This connection was found to be preferable over Connection 1.
- Connection 4
 - This is not a preferred solution because of the anticipated cracking in the deck slab.

14. What are your perspectives and/or suggestions regarding methods to make spliced girder bridges more constructible in the future?

Cost Concerns

- The contractor suggested that keeping the girder weights as low as possible and adopting repetitive girder details will aid in better pricing by the precasters.

Construction Concerns

- The contractor prefers to limit the span range for the continuous spliced girders to approximately ± 250 ft to 270 ft.

9. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 SUMMARY

This report summarizes the results of the first phase of a TxDOT-sponsored research project conducted to review and evaluate some of the key techniques that have been used for spliced, continuous, bridge girder systems, construction considerations, a general framework for categorizing connection splice types, and some potential connection details. The outcome of this research project will support TxDOT's implementation of continuous precast, prestressed concrete bridge girders to achieve longer span-to-depth ratios with greater economy than currently possible with simple spans.

The project is divided into two phases in order to arrive at the final research objectives. Phase 1 of the project evaluated the current state-of-the-art and practice of continuous precast concrete girder bridges and provided recommendations for suitable continuity connections for typical Texas bridge girders. This is achieved through:

- A comprehensive literature review of the current state-of-the-art and practice of continuous precast, prestressed concrete bridges in the country.
- Preliminary designs and identification of potential benefits and issues.
- Focus group meetings to seek input and suggestions from TxDOT, precasters and contractors for implementation of the proposed design into practice.

A wide variety of design and construction approaches are possible when making precast concrete bridges continuous with longer spans. The research team investigated different types of continuity connection details used for precast, prestressed concrete girder bridges across the United States that allow span lengths beyond 150 ft. Construction issues that should be considered during the concept development and design stage are highlighted. This research project categorized the splice connections into distinct types and discussed the advantages and disadvantages of each approach with a focus on construction and long-term serviceability. The research team developed preliminary designs using the current TxDOT practice for values of span length, girder spacing, and material properties. The revised provisions for spliced precast girders in the *AASHTO LRFD Bridge Design Specifications* (2010) were used in this project. This chapter summarizes the results obtained from the literature review and preliminary designs,

along with recommendations provided by the research team. The research team identified several areas requiring further study based on the detailed preliminary designs.

The research team held focus group meetings with the TxDOT engineers as well as the precasters and contractors from the industry to discuss the results and suggestions related to the design and construction benefits and issues of the proposed preliminary continuity details. This helped to narrow down the specific requirements of the different sectors such as design, fabrication, transportation, and erection and construction on the site. Recommendations from Phase 1 of this project focus on specific pretensioned girder shapes and continuity splice details to be investigated in the experimental study, which will be a part of Phase 2 of the project.

9.2 CONCLUSIONS

This section outlines the conclusions derived from the literature review, preliminary designs of continuous prestressed concrete girders using standard Tx70 and Texas U54 girders, preliminary details of splice connections and focus group meetings with TxDOT, precasters, and contractors for implementation of the proposed design into practice.

9.2.1 Review Literature and State-of-the-Practice

Many states have used different techniques and approaches to extend span ranges with variations in the design enhancements and material properties. The current state-of-the-art and practice and the NCHRP reports 517 (Castrodale and White 2004) and 519 (Miller et al. 2004) illustrate additional concepts and advantages of spliced girder bridges where multiple continuous spans are required. From review of the state-of-the-practice, the researchers found that the girder segment size is controlled by the hauling limitations and type of lifting equipment available. The use of on-pier splicing has limited potential because hauling limitations restrict the length of individual girders.

The use of in-span splices to make precast, prestressed concrete bridge girders continuous presents a cost-competitive alternative for increasing span lengths using standard precast girder sections. This system helps to fill the gap between 150-ft precast, pretensioned concrete bridges made continuous at the pier for live loads and the 300-ft continuous, post-tensioned concrete segmental box girder bridges. The precaster can fabricate the spliced girder segments in transportable lengths to achieve a new span range with a form of construction that is less

complex than span-by-span segmental and balanced cantilever systems. The spliced girder systems have fewer joints compared to segmental systems. Not only does this economize construction but minimizes the joint locations that may impair long term serviceability.

Based on a comprehensive review, the research team noted that the in-span spliced girder technology has the greatest potential to extend the span range of simple spans. This technology facilitates wider spacing between girder lines, minimum number of substructure units, and adoption of conventional construction procedures on site. Application of continuous construction using splicing of standard precast, prestressed girders presents a cost-competitive, constructible, and high performance alternative to steel plate or steel box girder solutions for longer spans up to 280 ft.

The research team noted that the selection of the construction method and type of splice detail depends on the terrain, available equipment, and experience of the local contractors. Designers, fabricators, and contractors with successful collaboration from the planning stages of bridge details can take the advantage of the most cost-effective use of personnel, equipment, and materials.

9.2.2 Preliminary Designs

The preliminary designs developed for this research project provided an initial evaluation of the potential benefits that can be realized through the use of continuous bridge design using precast, pretensioned girders. The researchers gathered input from TxDOT input to ensure that the girder types and sizes, girder spacings, material properties, etc. are consistent with the parameters of interest to TxDOT. The research team focused on the Texas Tx70 and U54 girder sections. The preliminary designs were carried out following the AASHTO LRFD Specifications for Highway Bridges and TxDOT standard design practices to ensure that the findings can be compared to typical span limits for standard TxDOT girders.

Findings from the preliminary design tasks have implications in terms of potential modifications that may be needed for existing girder sections. Widening the web of the Tx70 girder may be needed to better accommodate post-tensioning ducts and shear reinforcement in the standard TX girder sections. For transportation and handling purposes of the pier segments of both Tx70 and Texas U54 girder bridges, temporary unbonded Dywidag threadbars of 1.25 in. diameter in the bottom flange of the pier segments were included in the designs.

The preliminary designs assumed shored construction for the design of continuous prestressed bridge girders. For unshored construction, the span lengths may be increased through the use of haunched girder segments over the piers and use of higher strength concrete than that being currently used.

The sequence of construction has a significant effect on the design and behavior of the bridge. The researchers recommend stressing the post-tensioning tendons from both ends sequentially in stages in order to provide equal conditions and uniform stresses in both end spans of the bridge. Temporarily supporting the end and drop-in girder segments on the ends of the over-pier girder segments has a significant effect on the over-pier girder segments and negative moment region in general. In shored construction, the girder segments are supported on the temporary support towers at the splice locations to resist any reaction forces during erection. The removal of temporary support towers used in shored construction adds moments at the support over the pier. The pier segments are designed for the additional moments due to the removal of temporary support towers.

The researchers checked the girder sections at critical sections for flexural capacity under factored loads for the strength limit state. Mild steel reinforcement is added to supplement the moment capacity provided by the post-tensioning tendons, if necessary.

The researchers checked the stresses in the girders and the deck slab along the length of the bridge for the service limit states. Some regions of the beam experienced compressive stress levels that exceeded the allowable compressive stress at service conditions. This stress exceedance may be addressed by increasing the specified concrete compressive strength to stay within the allowable compressive stress limit. Another option that is sometimes employed is to provide additional mild steel reinforcement in the compression zone. The amount of mild steel reinforcement is determined based on the force corresponding to the stress exceedance. For the preliminary designs that necessary amount of additional mild steel was determined.

The researchers calculated deflections for the continuous prestressed Tx70 and Texas U54 girder bridges considered for the preliminary designs, and found that the maximum deflection in the continuous girders is below the allowable limits as specified in AASHTO LRFD Specifications Art. 2.5.2.6.2.

The span-to-depth ratio is an important bridge design parameter that affects the structural behavior, cost efficiency, and aesthetics of the structure. The suggested AASHTO span-to-depth

ratio limits were not applied when conducting the preliminary designs in order to push the span limits for the Texas U54 and Tx70 girder sections. Prestressed concrete girder bridges are intended to be a competitive alternative to steel bridges. Steel bridges tend to be less stiff compared to prestressed concrete girder bridges of the same depth. Therefore, the researchers noted that there does not seem to be a clear justification for allowing steel bridges to have longer span-to-depth ratios as compared to prestressed concrete girder bridges.

The researchers concluded the following from the preliminary designs of the continuous spliced precast, prestressed concrete bridge using the Tx70 girder section:

- Although it may be technically feasible to construct 300 ft spans using the Tx70 girders, higher strength concrete and a large number of tendons is needed.
- A span length of 280 ft is possible using the Tx70 girders, but not easily obtainable.
- A span length up to 240 ft can be more comfortably achieved.

For the preliminary design of a continuous spliced precast, prestressed concrete bridge using Texas U54 girders, the researchers concluded that a span length of 240 ft is viable for the U54 girders, providing a construction alternative.

Overall, the researchers noted that the span lengths of 280 ft and 240 ft for the continuous prestressed concrete bridges using the standard Tx70 and Texas U54 girders, respectively, can be achieved using shored method of construction and by making the girder sections work up to their limits. For increasing the span lengths beyond these values or considering the use of unshored construction, the research team recommends the use of haunched girder segments over the piers and notes the need for higher strength concrete than currently being used.

9.2.3 Preliminary Details of Splice Connections

Different splice connection details may be used. They can be generally classified as full prestressed, partially prestressed, and full reinforced connections. Several possible details have been used in the past. Advantages and disadvantages of each approach have been discussed with a particular emphasis on constructability and long-term serviceability. While all systems have their merits, a mixed solution is perhaps the most desirable, specifically a partially prestressed solution.

The performance and cost-effectiveness of a spliced girder system depends on the design and construction details. This involves a combination of the different design enhancements

instead of applying them individually. Selection of the construction method and type of splice detail depends on the terrain, available equipment, and experience of the local contractors. Designers, fabricators, and contractors can collaborate from the concept and design stages of the bridge system to make efficient use of materials, equipment, and onsite personnel.

9.2.4 Focus Group Meetings

The research team held focus group meetings with the TxDOT Project Monitoring Committee (PMC), precasters and contractors to solicit input regarding the potential implementation of promising continuity details for precast, pretensioned girders made continuous based on the findings from the literature review and state-of-the-practice and preliminary designs.

The TxDOT Project Monitoring Committee (PMC) provided their input to narrow down specific options of interest. One outcome of the discussion with the TxDOT PMC was the decision to focus on in-span splice connections, which will provide the greatest potential for increasing span lengths using standard precast girder sections.

The precasters presented their perspective regarding the potential implementation of potential continuity details for precast, pretensioned girders made continuous. The responses from precasters helped to identify possible issues with respect to precasting and shipping, along with economical and reliable details in terms of the precasting operation for the girder segments of the spliced bridge system. The main suggestions from the precasters regarding methods to make spliced girder bridges more constructible are as follows:

- In general all the precasting plants are well equipped to handle fabricating a variety of over-pier, end, and drop-in segments.
- Increasing the span length results in an increase in the weight of precast elements. Precautions should be taken so that the weight does not exceed 200 kips considering transportation limits.
- The desirable limits for I-girder segments is length around 140 ft, weight around 200 kips, and depth around 10 ft. For the U-girder shapes, it is recommended to limit the segment length to 130 ft considering weight limits for transportation.

- The recommended maximum span length for a spliced girder bridge is around 260 ft considering the stability issues of long-span drop-in segments and deep haunched over-pier segments.
- Use of a constant standard girder section depth for over-pier segments is preferred over the haunched girders to avoid issues related to high initial cost of fabrication, stability issues during transportation, and lifting weight issues onsite.
- There are no concerns with widening the webs to resolve the issue of maximum shear demand at the supports. The webs can be widened by increasing the space between the forms, which will result in widened top and bottom flanges of the girder section. It is a one-time cost to purchase a new soffit. Standardizing the precast elements will help reduce the overall cost.
- Fabricating end segments with thickened ends is not an issue. The length of an end block is typically 10–15 ft.
- The precasters preferred partially prestressed spliced connection details.
- Some discussion was held about using longer precast panels over the supports with longitudinal prestressing. The precasters indicated that this should be no problem.

The contractors presented their perspective regarding the potential implementation of promising continuity details for precast, pretensioned girders made continuous. The responses from contractors helped to identify potential issues with respect to the construction, along with the preferred details and methods that will ensure safe, reliable, and efficient construction of continuous spliced precast, prestressed concrete bridge systems. The main suggestions from the meeting with contractors regarding methods to make spliced girder bridges more constructible are as follows:

- The proposed bridge system provides another alternative to steel girder bridges, especially in coastal areas where corrosion of steel bridges is an issue.
- TxDOT engineers noted that this bridge type would compete well with shorter span segmental bridges. They also indicated that they are not using steel girder bridges along the coast, and the proposed bridge type would not compete with just steel girder bridges.
- Experienced contractors prefer to limit the span range for the continuous spliced girders to approximately \pm 250 ft to 270 ft.

- Unshored construction (no shoring towers) is preferred because it saves significant time during construction and reduces the construction costs. Often the required footprint is not available to place shore towers.
- It was noted that using fewer girders increases cost competitiveness of bridges.
- The contractor suggested that keeping the girder weights as low as possible and adopting repetitive girder details will aid in better pricing by the precasters.
- Contractors prefer the constant web depth option for the haunched girders because it is easy to fabricate and has more stability.
- Contractors noted that the option of two separate girder segments spliced over the pier provides flexibility of splicing the girder segments within span on ground before lifting them into place on site. This is a preferred option because no temporary shoring is required onsite. However, issues related to the weight of the whole assembly and the size of the equipment in lifting and placing the spliced girder segments are anticipated.
- The main issue noted during erection of the girders is the lateral stability of the girder segments due to wind.
- The partially prestressed connection detail was the most preferred with respect to on-site construction due to its relative constructability.
- Contractors prefer having two design options for bid: one with a standard precast concrete girder shape and one with a steel plate girder.
- It would be useful to consider various design options using life-cycle cost analysis. TxDOT is just now starting to use life-cycle cost analysis. Traditionally, initial cost has been used to evaluate design options.
- The quality control process is more complex for the proposed bridge system.
- Sequencing of the CIP concrete and PT operations are needed up front.
- Contractors look at both schedule and economy to determine the best option.

9.3 RECOMMENDATIONS

Based on the findings from Phase 1 of this research project, the following recommendations are made for use in finalizing the work plan for Phase 2.

1. The preliminary designs presented in this report are intended to push the limits of design. TxDOT input will be considered in finalizing the parameters for additional design studies in Phase 2.
2. The precasters preferred the use of a constant, standard girder section depth over the haunched girders for over-pier segments to avoid issues related to high initial cost of fabrication and stability issues during transportation. The preliminary designs assumed shored construction for the design of continuous prismatic prestressed bridge girders. For unshored construction, the span lengths may be increased through the use of haunched girder segments over the piers and/or use of higher strength concrete than that currently used. The contractors preferred unshored construction because it saves a lot of time during construction and reduces the construction costs.
3. Precautions should be taken so that the weight of the girder segments does not exceed 200 kips, considering transportation limits in Texas. The desirable limits for girder segments are length around 140 ft, weight around 200 kips, and depth around 10 ft. The recommended maximum span length for a spliced girder bridge is around 260 ft considering the stability issues of long span drop-in segments and deep haunched over-pier segments.
4. Sequencing of the CIP concrete and PT operations are important construction considerations and should be included with the future designs. Time-dependent issues onsite need to be considered.
5. With respect to the suggested span-to-depth ratio limits in AASHTO, there does not seem to be a clear justification for the larger value for steel bridges as compared to prestressed concrete girder bridges. However, vibration characteristics for longer span bridges and the impact on the bridge, both during construction and in-service, should be considered. Stability issues during construction are also an important consideration.
6. Different splice connection details have been proposed with advantages and disadvantages of each approach and with a particular emphasis on constructability

and long-term serviceability. While all systems have their merits, a mixed solution is perhaps the most desirable and should be considered, specifically a partially prestressed solution. TxDOT input will be taken into full consideration when finalizing the connection details for experimental testing in Phase 2.

7. Splice locations vary for different projects built to date. It is important to determine the best possible location specifically for each project. The whole design approach for the continuous prestressed concrete girders is based on load-balancing. Location of inflection points under total dead loads should be considered important for determining the splice locations.
8. There should be a cost analysis taking into account the life-cycle cost of the bridge, along with a cost comparison between a steel bridge, segmental bridge, and a standard precast concrete bridge.

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