DESIGN OF CONTINUOUS PRESTRESSED CONCRETE SPLICED GIRDER BRIDGES

A Thesis

by

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ABSTRACT

Traditionally, prestressed concrete girder bridges are limited to 150 ft span lengths in Texas due to restrictions on handling and transportation. An effective way of increasing span lengths of precast, prestressed concrete girder bridges is demonstrated using splicing technique. In spliced girder bridges, precast girder segments are transported in shorter segments for handling and transportation and then spliced together to form long-span continuous bridges. Different methods are explored for construction of spliced girder bridges. Two application examples are developed to demonstrate the design of continuous prestressed concrete spliced girder bridges for both shored and partially shored methods of construction. A three-span bridge having a span configuration of 190-240-190 ft is considered for both examples. Advantages and dis-advantages of each method of construction are discussed. Construction issues that should be considered in the design are highlighted. The results of this study indicate that span lengths up to 240 ft are achievable using standard Tx70 girders with the help of splicing techniques. A parametric study is performed to further explore the design space of spliced girder bridges. The results of the parametric study, along with critical design issues that were identified, are highlighted and related recommendations are provided. The results of this study will be of significant interest to bridge engineers and researchers for guidance in implementing spliced girder bridges in Texas and other states.

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1. INTRODUCTION

1.1 BACKGROUND

Prestressed concrete bridges have been constructed in the US since the 1950s. However, most of them were short-span bridges having a maximum span length up to 160 ft. Waterways and obstructions in roadways drive construction toward long-span bridges. Long-span bridges reduce the number of piers and can make the structure more cost effective. For many years long-span bridges were associated with steel girder bridges. However, concrete is a versatile, economical, and weather-resistant material and is considered an attractive and cost effective option for bridge construction. Also, in coastal environments there is a need for alternatives to steel bridges due to problems related to corrosion. Recent advancements in concrete technology have resulted in high strength and more durable concrete. This coupled with advantages of prestressing has made longer span bridges using prestressed concrete a viable option. Prestressed concrete can also result in lower initial cost, reduced vibration, reduced noise in construction and greater aesthetic sense.

Simply supported bridges turn out to be a favorable option for span lengths up to 150 ft. For span lengths exceeding these limits and in the range of 200-300 ft, it becomes necessary to make the bridges continuous when using standard girder sections. In continuous bridges the bending moment anywhere in the span is considerably less than that of a simply supported bridge. This results in reduced stresses throughout the section as compared to simply supported bridges, which ultimately results in an economic section for the bridge. Thus, continuous bridges can have considerable savings compared to simply supported bridges. By adding continuity, redundancy is added to the structure which is valuable in cases of extreme events such as earthquakes, floods and vehicle impact. Thus, for span lengths in the range of 200-300 ft, continuous bridges seem to be the most favorable option.

As the span lengths of bridges increase, the handling and transportation of the girder segments becomes increasingly difficult. The maximum length of girder segments that can be hauled and transported is restricted to 160 ft in length and up to 200 kips in weight based on input from precasters and contractors (Hueste et al. 2012). One of the options to overcome this issue is to transport shorter length girder segments and splice them on site. The girders are fabricated in a precasting plant in shorter segments and then transported to the job site where they are spliced together to form long-span continuous bridges. Thus, splicing techniques provide an attractive option for extending span lengths.

Different methods have been used in the construction of spliced girder bridges. Shoring towers were predominantly used in the construction of spliced girder bridges when they were first implemented into practice. However, topographical constraints, construction over rivers, and construction across railway intersections may prevent the use of shoring towers. Under such circumstances, an unshored method of construction is preferred. A partially shored method of construction has become popular where the shoring towers are used in the back span, but no shoring towers are used in the center span. The method of construction has a significant effect on the design and behavior of spliced girder bridges.

Spliced precast concrete girder bridges have become the most preferred method of construction for medium span bridges. This bridge type has become popular in the last decade due to various advantages. Some of the advantages were highlighted by Castrodale and White (2004) as follows:

1. Increasing span lengths helps to reduce the number of piers. This could be of supreme importance in projects that involve placing the piers across waterways. Fewer piers help reduce the environmental impact associated with construction in water bodies.
2. With the help of spliced girder bridges, the depth of the superstructure is reduced. This could be beneficial in areas where vertical clearance is required for traffic and waterways.

3. Haunched segments over piers improve the efficiency of the structure and make the structure aesthetically pleasing.
4. Reducing the number of joints in the deck helps improve the long term service life of the structure and reduce the overall maintenance cost.

Additionally, in the construction of spliced girder bridges, precasting the girder segments can be done simultaneously with construction of foundation and cast-in-place portions of the structure. This reduces the overall time required for construction.

1.2 RESEARCH OBJECTIVES

This research study focuses on continuous precast prestressed concrete spliced girder bridges. The four major research objectives are as follows.

1. Although splicing of precast, pre-tensioned concrete girders is not a brand new concept, it is not commonly used in Texas. There is limited information regarding the design of spliced girder bridges and the various issues that need to be considered in the design. An overarching objective of this study is aimed at helping engineers to become familiar with the design and construction procedures involved in the design of spliced girder bridges.
2. The use of temporary shoring in the form of strong backs, tie downs and shoring towers is typical for spliced girder bridges. The topography of the bridge crossing dictates the type of temporary shoring. Based on the type of temporary shoring, spliced girders can be categorized into three subcategories: (1) shored, (2) unshored, and (3) partially shored. This study helps distinguish between different methods of construction highlighting the advantages and disadvantages of each and recommend the most preferable method(s) of construction.
3. Texas I-girder shapes for pretensioned girders have been optimized for simple spans. As the trend for long-span continuous bridges continues, there is a need to investigate the behavior of these girders shapes for continuous bridges. To explore the design of continuous prestressed concrete girders,

application examples are developed for shored and partially shored method of construction using the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

4. The design space for continuous prestressed girders is further explored through a parametric design study. The parametric study is performed by varying the cross-section and construction approach while keeping the span length of the bridge constant. The results from the parametric study are used to make recommendations to optimize the girder cross-section and method of construction for the selected span length.

1.3 METHODOLOGY

The following four major tasks have been identified to accomplish the objectives of this research study, as follows. Each of the tasks is described below.

- Task 1: Investigate the Integration of Design and Construction for Continuous Bridges
- Task 2: Develop Application Examples
- Task 3: Conduct Parametric Study
- Task 4: Develop Conclusions and Recommendations

1.3.1 Task 1: Investigate the Integration of Design and Construction for Continuous Bridges

The implementation of spliced girder bridges involves two main features: design and construction. These two features are interdependent and are necessary for implementation of spliced girder bridges. Beginning with fabrication and erection, to the final stage when the bridge is opened to traffic, all the relevant construction and design factors need to be carefully studied. The goal of this task is to identify all the important factors in the design and construction of spliced precast prestressed concrete girder bridges and to determine their impact on implementation of these structures.

1.3.2 Task 2: Develop Application Examples

Application examples using a typical Texas pretensioned concrete girder section will be developed using the *AASHTO LRFD Bridge Design Specifications* (2012) for both shored and partially shored construction. A three-span bridge having a span length of 190-240-190 ft is considered to represent a typical spliced girder bridge for the application examples. This is based on TxDOT's recommendation for the typical number of spans expected in practice. Tx70 girder shapes are used in the application examples. The applications examples helped determine the efficiency of Tx70 girders when used for continuous prestressed concrete spliced girder bridges.

1.3.3 Task 3: Conduct Parametric Study

A parametric study is performed to allow consideration of several additional prestressed concrete continuous bridge systems. For the parametric study, both Tx70 and Tx82 girders are considered. Span lengths of 190-240-190 ft are used. Also, the web of the girders is varied to determine the effect of increase in web thickness on the shear capacity of the girders. The results of the parametric study are checked for girder stresses at service and for live load deflections. Also, the ultimate moment and shear limit states are checked.

1.3.4 Task 4: Develop Conclusions and Recommendations

The first three tasks are followed by discussion and synthesis of results. Based on the design examples, issues pertaining to design, adoption and implementation of spliced girder bridges are identified. The significant factors governing the design are highlighted. Major differences between shored and partially shored construction are determined. Maximum span lengths that are readily achievable using the existing Tx70 girders are identified. Measures that could be taken for further increasing the span lengths are specified.

1.4 ORGANIZATION OF THESIS

Section 1 provides an introduction to this thesis and outlines the objectives of the thesis. Section 2 provides a comprehensive literature review of spliced girder bridges that have currently been built in the United States. Section 3 provides an outline to the application examples highlighting the design parameters, design assumptions and limit states to be considered in the design. Sections 4 and 5 provide details for the designs examples developed using shored and partially shored methods of construction, respectively. All the steps that need to be considered in the design and construction of spliced girder bridges will be highlighted. Section 6 presents the results of the parametric study and identifies the impact of different parameters on design of spliced girder bridges. Section 7 summarizes the results and findings of the application examples and the parametric study. It further provides conclusions based on these findings and provides recommendations for future work.

2. LITERATURE REVIEW

2.1 INTRODUCTION

Splicing is not a new concept. Several examples of spliced girder bridges can be found dating back to the 1950s. Since the introduction of prestressed concrete, almost one-third of the bridges built in the United States are made of prestressed concrete. The standard I-girder and bulb tee girder have become very common for simple span bridges for span lengths up to 150 ft. As the advantage of using prestressed concrete for bridges became more evident, there arose a need for finding alternative methods for increasing span lengths of prestressed concrete bridges. Higher strength concrete, larger diameter prestressing strands and other methods were identified for increasing the span lengths. Splicing combined with these methods was found to have the maximum advantage. Splicing was initially used for simple spans and then extended to continuous spans, thereby further increasing span lengths. In the early 21st century spliced girder bridges have achieved span lengths in the range of 320 ft. Spliced girder bridges have been very economical as compared to segmental and steel bridges in these span lengths (Caroland et al. 1992, Mumber et al. 2003, PCI 2004).

Lin et al. (1968) demonstrated methods using precast, prestressed concrete to enable construction of long-span bridges. The report stressed that along with aesthetic sense, prestressed concrete offers low initial cost of construction, less maintenance, safety against fire, less vibration and traffic noise reduction. The report provided basic information on applying prestressing to achieve long-span bridges. Design examples were provided for simple span spliced girder bridges and two-span continuous spliced girder bridges for span lengths up to 150 ft. The author highlighted that the length of spans was not governed by allowable stresses alone and the behavior of the entire bridge must be considered giving due importance to deflection, camber, crack limitation, vibration control, shrinkage, temperature and secondary stresses. Haunched girders over piers were recommended for reducing stresses. Also, tie rods could be used for unbalanced loading to remove temporary shoring and prevent traffic obstruction. Further advantage can be obtained by rigidly connecting the cap beam and the pier head. Using inclined piers was recommended for increasing the span lengths as it eliminates the shoring towers and provides stability during construction.

Castrodale and White (2004) presented various options for increasing bridge span lengths as part of NCHRP project 517. Along with high strength concrete, many other techniques have been identified. These include material related options, design enhancements, post-tensioning and spliced girder construction. Some of the recommendations include increased strand size, increased strand strength, increasing section properties, and combined pre-tensioning and post-tensioning. It was recommended that spliced girder construction combined with the above enhancements provided the maximum benefits. The report provided information on the available software resources that are applicable to the analysis and design of spliced girder bridges. The report also recommends modifications to the *AASHTO LRFD Bridge Design Specifications* with regard to implementation of spliced girder bridges that were incorporated into the AASHTO 2007 Specifications.

2.2 SELECTED SPLICED GIRDER BRIDGES

Table 2.1 summarizes selected spliced girder bridges that have been constructed in the U.S. Design parameters like span lengths, the depth of girder segments used and the strength of concrete used for the girders are provided. The girder shapes used are bulb tees or standard I shaped girders.

Table 2.1. Selected Spliced Girder Bridges.

Bridge Name, State Location	Year Built	Span Lengths (ft)	Girder Depth (in.)	Haunch Depth (in.)	Girder Strength (ksi)	Reference
Shelby Creek, Kentucky	1992	162-218-218-218-162	102	102	7-8	Caroland et al. (1992)
Highland View Bridge, Florida	1994	196-250-196	72	120	6.5	Janssen and Spaans (1992)
Bow River Bridge Alberta, Canada	2000	174-213-213-174	110	110	Not reported	PCI (2004)
Moore Haven, Florida	2000	215-320-200	78	180	Not reported	PCI (2004)
Palm Valley Bridge, Florida	2002	210-290-210	81	180	Not reported	Castrodale and White (2004)
Ocean-City Longport Bridge, New Jersey	2002	184-222-184	90	90	Not reported	Mumber et al. (2003)
Wonderwood Connector, Florida	2003	195-250-195	78	144	8.5	Ronald (2001)
St. George Island, Florida	2004	207-257-250-257-200	78	144	8.5	Ronald (2001)
Route 123 Bridge, Virginia	2006	180-240-180	77	150	8	Saunders (2005)
Route 33 Bridge, Virginia	2007	200-240-240-200	96	126	8	Saunders (2005)
Sylvan Avenue, Texas	Under Construction	200-250-200	82	130	8.5	Marin (2011)
Sylvan Avenue, Texas	Under Construction	170-200-170	82	82	8.5	Marin (2011)

2.3 SIMPLE SPAN CONSTRUCTION WITH SPLICES

Nicholls and Prussack (1997) provided details for the design of the Rock Cut Bridge in Washington where splicing was used for simple span girder bridges. The bridge was constructed over Kettle River. In order to avoid the environmental impact associated with constructing the pier or temporary shore towers in the river, an innovative method was used. Three girder segments 63 ft long, each weighing 40 tons and having a depth of 90 in. were pre-tensioned during handling and transportation and then carried to the site. The girder segments were then spliced using post-tensioning very close to the site resulting in a single 190.5 ft long girder weighing 121 tons. Decked bulb-tees were used to prevent any concreting on site. Four lines of girders were used at a spacing of 6 ft-1.5 in. Concrete with an in-service strength of 6 ksi was specified for the precast girders. A launching truss was used and the girders were pushed across until they reached the other end and then a crane was used to drop the girders into their final position. The entire construction was carried out in three and half months thereby saving a significant amount of time.

The I-15 reconstruction project in Salt Lake City involved construction of sixteen simple span bridges having a maximum span length up to 220 ft. Modified girders were developed by Washington State Department of Transportation having a depth of 94.5 in. Three segments of girders were used and were supported by shoring towers. Diaphragms were provided at the pier as well as at the splices. The deck was cast and then the girders were post-tensioned.

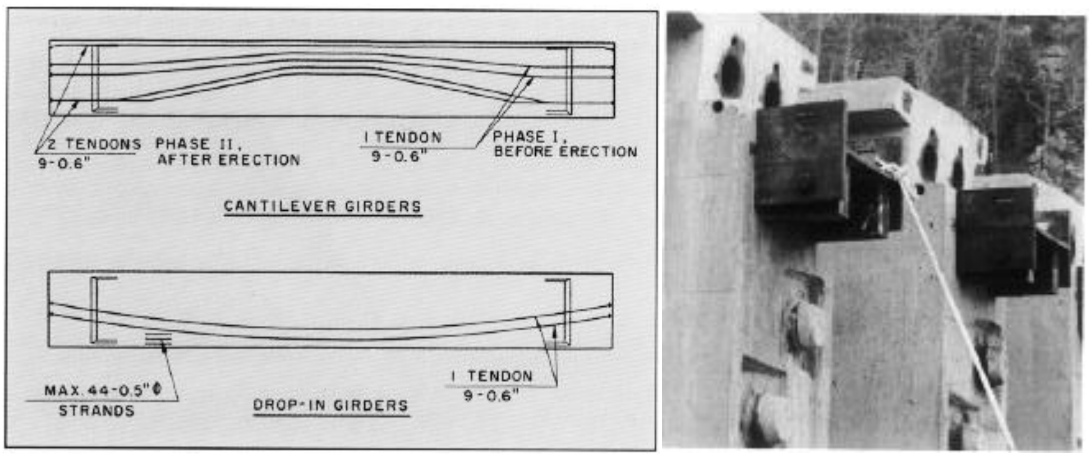
2.4 CONTINUOUS SHORED CONSTRUCTION WITH SPLICES

Lin et al. (1968) provided design details for a two-span symmetric bridge having a span length of 150 ft. The pier segments and end segments were 100 ft. AASHTO Type VI girders were used for design purposes. The girders were pre-tensioned for self-weight and post-tensioned for continuity. Temporary shoring was provided for construction purposes. A specified in-service concrete strength of 6 ksi was used for the girder segments. The report concluded that improved methods of construction combined with prestressing and new equipment will help in further extending the span lengths.

Abdel-Karim and Tadros (1995) compiled information on some of the spliced girder bridges that have been built in the US from 1960-1990. The report describes some of the existing spliced girder bridges and provides steps for design of two-span continuous spliced girder bridges. Design details are provided for fully shored construction of a two-span continuous bridge with equal span lengths of 175 ft. The end segments are 135 ft and the pier segments are 80 ft. Six lines of prismatic girders were used with a spacing of 7 ft-2 in. and a depth of 72 in. Pre-tensioning was provided for the girder self-weight and deck weight. Single stage post-tensioning was provided after the deck was cast to apply compression in the deck. A cast-in-place post-tensioned splice was used. The in-service concrete compressive strength requirement was 7 ksi for girders and 4 ksi for the desk. The preliminary design was performed for allowable stresses and strength criteria. The author emphasized giving due consideration to shear design, deflection calculations and prestress losses. The concept of external post-tensioning for bulb tee girders was introduced for consideration in the future.

2.5 CONTINUOUS UNSHORED CONSTRUCTION WITH SPLICES

Caroland et al. (1992) described the design of an unshored continuous prestressed concrete girder bridge over Shelby Creek. The five-span continuous bridge has a total length of 985 ft with three interior spans of 218 ft and two end spans of 162 ft. The girder segments are 102 in. deep constant bulb depth I-girders having equal lengths of 108 ft. Seven lines of girders were used at a spacing of 12 ft-6.5 in. Lightweight concrete with in-service design strength of 7 ksi was specified for the girders. Precast deck panels having a thickness of 3.5 in. were used to speed up the construction. The bridge is 175 ft above ground, which made the use of shoring towers impractical and the designers used unshored construction. A unique method of construction was used where the girder segments are prestressed individually as shown in Figure 2.1. No continuity post-tensioning was provided. Because the segments were individually post-tensioned, thickened end blocks were required. Temporary pre-tensioning was provided in the on-pier segment for transportation. Pier segments were post-tensioned transversely to the cap to stabilize the cantilevered on-pier segments. The piers were designed to create moment fixity between the piers and the pier table girders. A Cazaly hanger was used to connect the drop-in segments to the pier segments. Longitudinal prestress was provided through the splice with the help of five-0.6 in. diameter strands. The cost of constructing the spliced girder bridge design was $417,000 less than the alternate steel bids.



(a) Layout of Prestressing in Girders *b) Cazaly Hanger system*

Figure 2.1. Details of Shelby Creek Bridge (Caroland et al. 1992).

Mumber et al. (2003) presented the design of the Ocean City-Longport Bridge that has a three-span spliced girder bridge with a total length of 590 ft. The bridge was built across the Atlantic Ocean. In order to avoid corrosion problems and long term maintenance issues, a steel bridge was ruled out and preference was given to a prestressed concrete spliced girder bridge. The end spans are 184 ft with a center span of 222 ft. The system used modified AASHTO Type VI I-girders that were 90 in. deep. Deep waters and other site constraints made use of falsework towers impractical, forcing the designers to select unshored construction. A unique construction sequence was adopted where the drop-in girder segments were erected on the on-pier girder segments. When the drop-in-segments were erected significant unbalanced moments were imposed on the pier. An innovative approach was used where tie downs created a temporary moment connection between the pier and the pier table girders and the unbalanced moments were transferred directly to the piers. Sand jacks were used for temporary blocking of the girder segments. Once the end segments were erected the moments were balanced and the temporary post-tensioning was removed. Figure 2.2 provides details of the temporary moment connection provided for the girder segments.

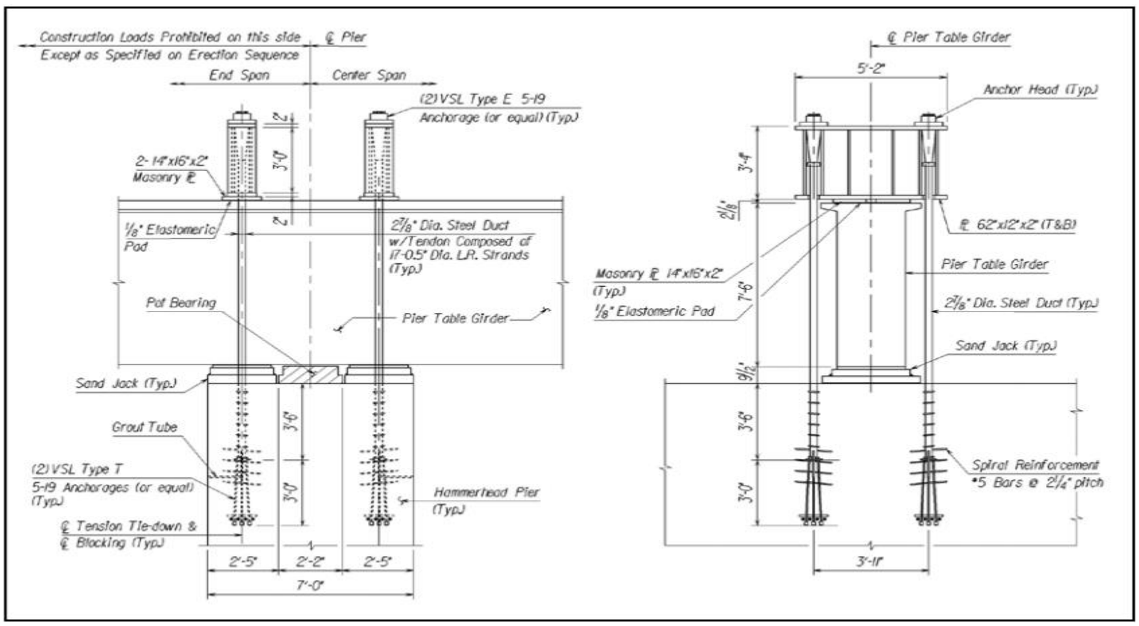


Figure 2.2. Temporary Moment Connection (Mumber et al. 2003).

Nikzad et al. (2006) described the construction of 850 ft long, five-span post-tensioned spliced girder bridge. The bridge had span lengths of 150-180-180-180-150 ft. The girder segments are 94 in. deep constant bulb depth I-girders. Concrete with in-service design strength of 10 ksi was specified for the girder and 7.5 ksi for the splice. The use of high strength concrete allowed for higher amount of post-tensioning to be applied to the girder. Eight lines of girders were used at a spacing of 9 ft. The girders were transported in shorter segments and spliced near the site to form 180 ft long girder segments. No temporary shore towers were used in the design.

2.6 CONTINUOUS PARTIALLY SHORED CONSTRUCTION WITH SPLICES

Janssen and Spaans (1992) provided details for a bridge on U.S. 98 over the Gulf Intercostal Waterway. The 2600 ft bridge was the longest bridge in the US having a three-span spliced girder system with a record center span of 250 ft. The spliced girder bridge had a span length of 196-250-196 ft providing a total length of 642 ft. The drop-in girder segments were 141 ft-8 in. and the on-pier segments were 106 ft-6 in. AASHTO Type VI girders were used for the drop-in girder segments. For the on-pier segment, constant web depth haunched girders were used having a depth of 10 ft. Concrete with an in-service strength of 6.5 ksi was specified for the girders and 5 ksi for the deck. Five lines of girders were used at a spacing of 9 ft-6 in. The splice was located at the inflection point having a width of 12 in. A partially shored method of construction was used where temporary shore towers were used in the end spans and strong backs were used in the center spans as shown in Figure 2.3. Temporary bracing was provided during the construction stage until permanent concrete cross girders were provided at the splice and the pier. Cross girders were transversely post-tensioned with 1.25 in. diameter strands.

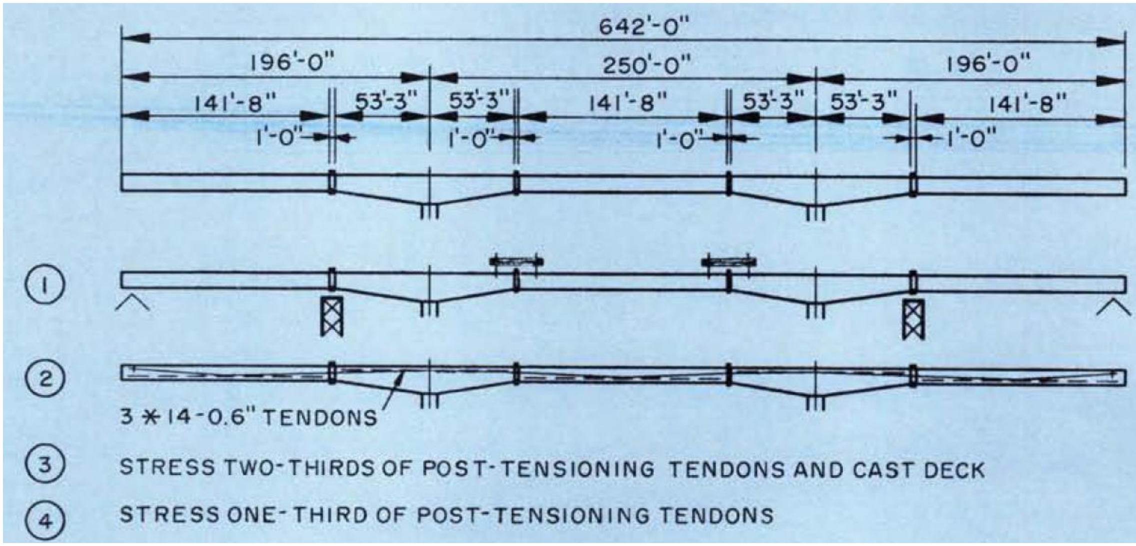


Figure 2.3. Highland View Bridge, Florida (Janssen and Spaans 1994).

Ronald (2001) summarized many of the important issues in the construction of spliced girder bridges. The article focused on a partially shored system of construction and provided design examples for three-span and five-span units. The Wonderwood Connector has a three-span main unit that consists of a spliced girder bridge with span lengths of 195-250-195 ft. The girder segments were 78 in. deep Florida bulb tees. The drop-in girder segments and end-segments are 140 ft long and the haunch girder segments are 110 ft long and 12 ft deep. Eight lines of girders are used at a spacing of 11 ft-3 in. The St. George Island Bridge is a five-span spliced girder bridge having a span length of 207.5-257.5-250.5-257.5-207.5 ft. The haunch girder segments are 12 ft deep and 115 ft long. Five lines of girders are used at a spacing of 9 ft-6 in. Both bridges had an in-service specified concrete strength of 8.5 ksi for the girders and 6.5 ksi for the deck. A partially shored method of construction was used where shore towers were used in the back span but no shore towers were provided in the center span. Tie downs and strong backs were used for the purpose of stability and to drop the girder segments on the pier segments. Ronald highlighted the effect of differential shrinkage and the effect of casting schedule on the design. Ronald correlated span length with the depth of haunch.

Ronald recommended increasing the haunch depth as the span length of the bridge increases. A haunch depth of 10 ft was recommended for span lengths up to 260 ft while a maximum haunch depth of 15 ft was recommended for span lengths up to 320 ft as shown in Figure 2.4.

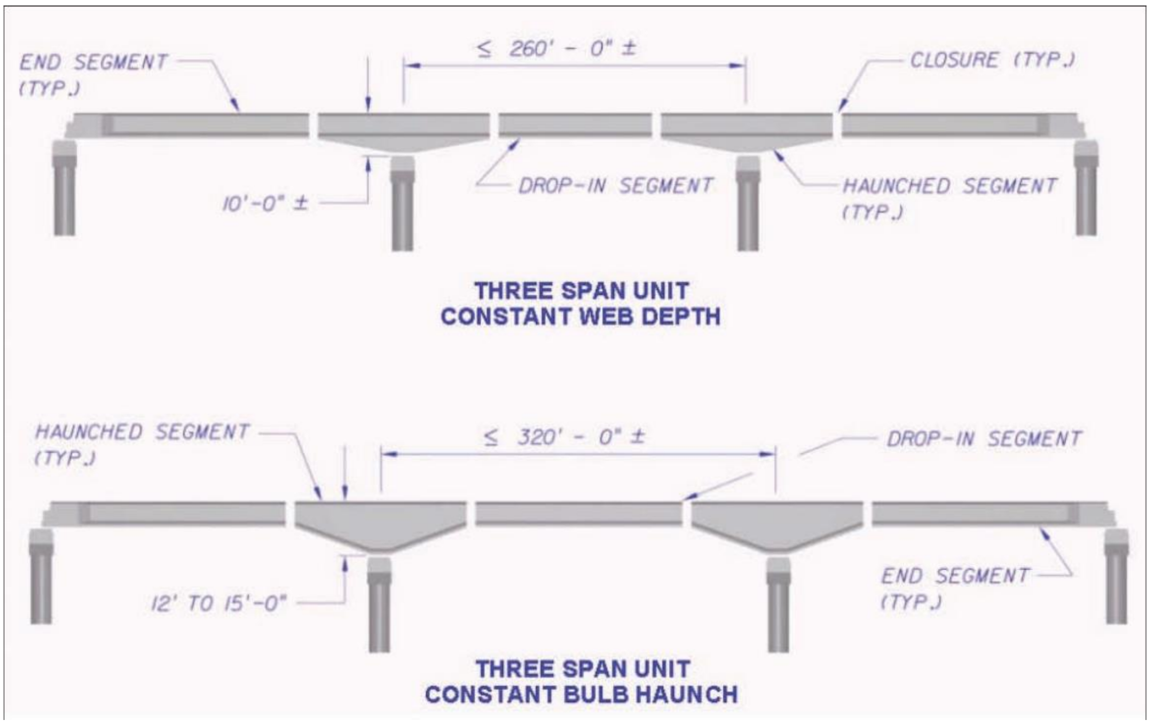


Figure 2.4. Recommended Span Lengths for Spliced Girder Bridges (Ronald 2001).

Castrodale and White (2004) as part of NCHRP Project 517 highlighted many of the important factors that must be considered in the design of spliced girder bridges. Design examples were presented in the report for simple span structures, continuous spans and for use of spliced girders in seismic regions. Design details were given for a typical three-span spliced girder bridge having a span length of 210-280-210 ft and total length of 700 ft. Five lines of girders were used at a spacing of 9 ft-6 in. The drop-in girder segment was 146 ft long and 78 in. deep and the end segment was 152 ft long and 96 in. deep. The on-pier segment was 124 ft long and 15 ft deep. The specified concrete strength was 8.5 ksi for the girders and 4.5 ksi for the deck. A partially shored method of construction was used. The girders were pre-tensioned for handling and transportation and post-tensioning was carried out in two stages. Stage I post-tensioning was used to make the girders continuous and Stage II post-tensioning put compression in the deck.

2.7 GIRDER SPACING AND SPAN LENGTHS

Ronald and Theobald (2008) correlated the relation between the girder spacing and span lengths with the help of a parametric study. A three-span continuous spliced girder system was chosen for the parametric study because of its wide use in the industry. The center span length was varied from 250 ft to 295 ft and the end span length was varied from 200 ft to 236 ft, respectively. The pier segment length was kept constant at 115 ft. Haunched segments were used for the on-pier segments with a depth of 12 ft. The drop-in girder segments and end segments were 78 in. deep Florida bulb-tees. The girder spacing was varied from 12 ft-9 in. for the 250 ft center span to 9 ft-6 in. for the 295 ft main span. Other parameters like creep and shrinkage parameters, friction coefficient, section properties and concrete strengths were kept constant. Ronald pointed out that the same amount of prestress can satisfy the ultimate strength requirement for the 250 ft main span and the 290 ft main span. Also the shear and service stresses were satisfied in all the cases. However, the amount of prestress required to provide camber in the bridge to prevent the bridge from sagging at dead load had to be increased for the 290 ft span. The author recommended using a maximum spacing of 12 ft-9 in. The article highlighted the advantage of using existing forms versus designing new forms specific to each system. Ronald pointed out that maximum efficiency is achieved when the existing girder forms are used in the design. He concluded that using existing forms would be much more economical than building forms specific to each project.

2.8 PRESTRESS LOSSES

Ronald (2001) laid emphasis on the effect of creep and shrinkage on the design of spliced girder bridges. Ronald pointed out that the amount of prestress required to satisfy the stress limits depends on the creep and shrinkage parameters. An extremely conservative value of creep and shrinkage parameters would make the satisfaction of allowable stresses extremely difficult while a less conservative value would yield stresses which are unrealistic. An ultimate creep coefficient of 2.0 and an ultimate shrinkage strain of 0.004 were specified for design purposes. The two regions where the effect of creep and shrinkage would be detrimental are the deck region near the pier top and the mid-span of the center segment. Ronald highlighted the effect of differential shrinkage and the effect of casting schedule on the design. A concrete mix with low water-cement ratio and shrinkage reducing admixtures is recommended to reduce the effect of tension stresses. Also, reducing the age difference between casting the deck and girder was recommended to help reduce the effect of differential shrinkage.

Wollmann et al. (2003) presented a method for simplifying the creep and shrinkage loss calculation for simple span spliced girder bridges. The complex creep and shrinkage laws were linearized with the help of age adjusted modulus of elasticity. The results indicated that the effect of differential shrinkage between deck and girder was negligible. By taking advantage of higher strength of concrete at the time of post-tensioning, the prestress losses were reduced and it provided better accuracy with respect to camber calculations.

Seguirant et al. (2004) emphasized the need for accurately predicting the time dependent material properties in spliced girder bridges. Different methods as specified in AASHTO LRFD, NCHRP Project 496 and WSDOT BDM were used for estimating the time dependent properties for simple spans. The important time dependent properties of concrete which affect the prestress losses were identified as modulus of elasticity, creep and shrinkage. The losses in pre-tensioned members were distinguished into losses due to elastic shortening, long term loss due to creep and shrinkage and loss due to steel relaxation. For post-tensioned members, it was recommended to compute losses due to elastic shortening, long term creep and shrinkage losses and relaxation due to steel before and after post-tensioning in addition to friction and anchor set losses. It was pointed out that computation of losses in continuous bridges is very complicated as compared to simple spans because of the effect of continuity.

Pantelides et al. (2007) monitored the post-tensioning losses in the simple span spliced girder bridges that were constructed during I-15 reconstruction in the Salt Lake City, Utah. The spliced girder bridges were instrumented and the data was recorded which included concrete strains at selected locations, girder post-tensioning losses and girder deflection for one of the girders. The girders were monitored for 3 years and 8 months. At the end of monitoring period the actual loss in the mid-span was found to be 14.5 percent of the initial post-tensioning force and the deflection was found out to be 0.15 percent. Field measurements were compared with the analytical results to compare the results. The AASHTO LRFD Bridge Design Specification was used to analyze and compare the results with the field measurements. It was found out that the time dependent method accurately predicts the losses at the mid-span and at the abutment. Concrete shrinkage and creep tests were performed on the girders to obtain ultimate creep coefficient and shrinkage strain. It was found out that the ultimate creep coefficient and shrinkage strain reached an asymptotic relation in about 8 months.

2.9 END BLOCK DETAILS

Ronald (2001) identified different end block types that can be used in the construction of spliced girder bridges depending on the sequence of construction. In the first end block type, all the post-tensioning tendons are anchored on the vertical face of the girder. The advantage of such a type of end block is that it is very simple in design and the length of end block required is short. However, the main disadvantage of this is that it governs the erection sequence since all the post-tensioning must be done prior to casting the deck. In the second end block type, the post-tensioning tendons terminate at the top of the anchor block. Although this end block allowed greater flexibility with regards to erection sequence, it resulted in complex designs and a longer length of end block is required. In the third type of end block, the Stage I post-tensioning tendons are anchored at the vertical face of the end block while the Stage II post-tensioning tendons are anchored at the top of the end block. Although the end block details are complicated, it allows for Stage II post-tensioning to be done after the deck is poured. Figure 2.5 shows the different end block types recommended by Ronald.

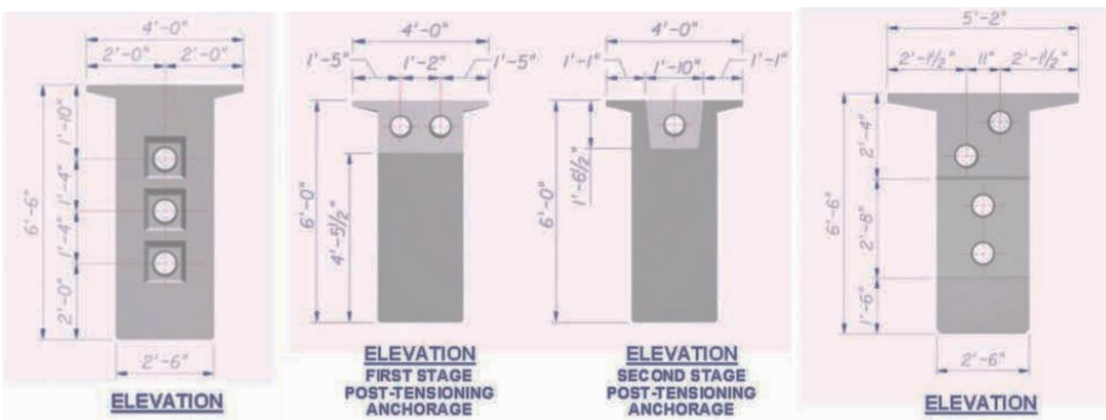


Figure 2.5. Different End Block Types for Spliced Girder Bridges (Ronald 2001).

2.10 SPLICE CONNECTIONS

2.10.1 Overview

Lin et al (1968) singled out connections as the most important components of spliced girder bridges. The report provided design details for splices provided near inflection point, splices provided for negative moment and splices provide for positive moment connection. Post-tensioning is required to carry the moment across the splice for the negative moment and positive moment splice. When the splice is located at the inflection point, non-prestressed reinforcement can be adequate but there can be localized cracking since there is no prestress across the joint. Lin pointed out that for splices at inflection point even a dry joint could be provided, however, more research was warranted with regards to this connection. Lin pointed out that if post-tensioning was used to provide continuity, the location of the splice was not critical as most of the shear is carried out by the vertical component of the post-tensioning and the rest can be taken by the friction between matching surfaces or by providing shear key. Lin also mentioned that the pre-compression provided by the post-tensioning could be useful in resisting shear.

Abdel-Karim and Tadros (1995) described some of the spliced girder connections which are typical of spliced girder bridges. These include conventionally reinforced splice, cast-in-place post-tensioned splice, stitched splice, epoxy filled post-tensioned splice, drop-in splice and steel splice. The report highlighted the advantage and dis-advantage of each of the splice.

2.10.2 Conventionally Reinforced Splice

Conventionally reinforced splices are usually provided near the inflection point of the dead load moments. Also, the live load moments near the splice are relatively small. Conventionally reinforced splices are also used for on-pier splicing when continuity is provided for live loads. The concrete for the splice and the deck needs to be poured at the same time to provide continuity for the superimposed dead loads. Sufficient length of splice is needed to develop the splice. However, since the splice is not prestressed, the splice is expected to crack under full service loads. Although, this splice turns out to be economical, regular inspection is required and there could be congestion of reinforcement in the joint. A reinforced splice as shown in Figure 2.6 is usually provided for shored construction.

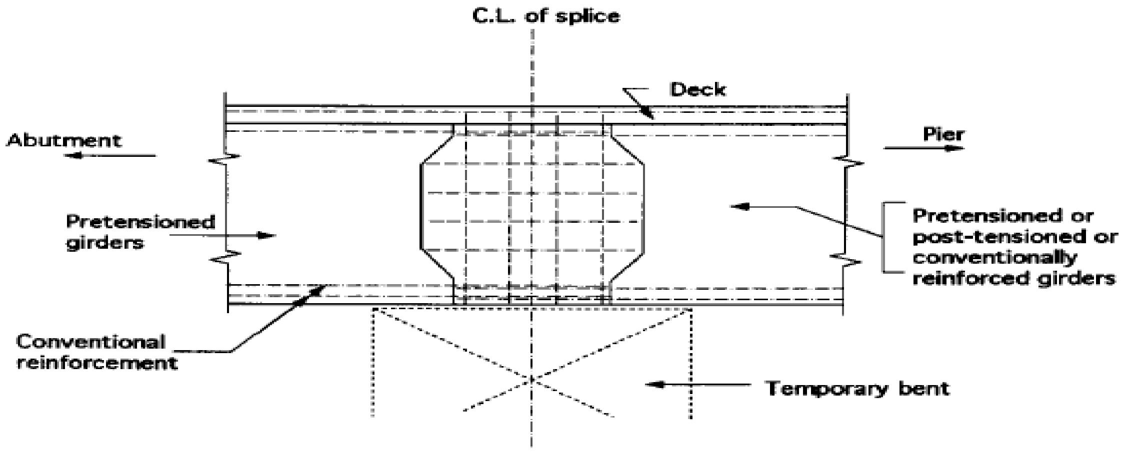


Figure 2.6. Fully Reinforced Splice (Abdel-Karim and Tadros 1995).

2.10.3 Post-Tensioned Splice

Cast-in place post-tensioned splice could be used with conventionally reinforced or pre-cast girder sections. Concrete for the deck slab can be placed after the post-tensioning or before post-tensioning. In such a type of connection continuity post-tensioning runs through the splice. Since post-tensioning carries the moment across the splice the location of the splice is not critical. A cast-in-place post-tensioned splice, even though found out to be expensive as compared to other splices is considered to be efficient as compared to other splices. Since post-tensioning is carried out after the deck is cast, a net compression can be obtained on the splice. This improves the serviceability of the splice. Also, mild reinforcement can be added across the joint to increase the ultimate strength of the joint. A cast in place post-tensioned splice as shown in the Figure 2.7 is widely used in post-tensioned spliced girder bridges and can be provided for both shored and unshored construction.

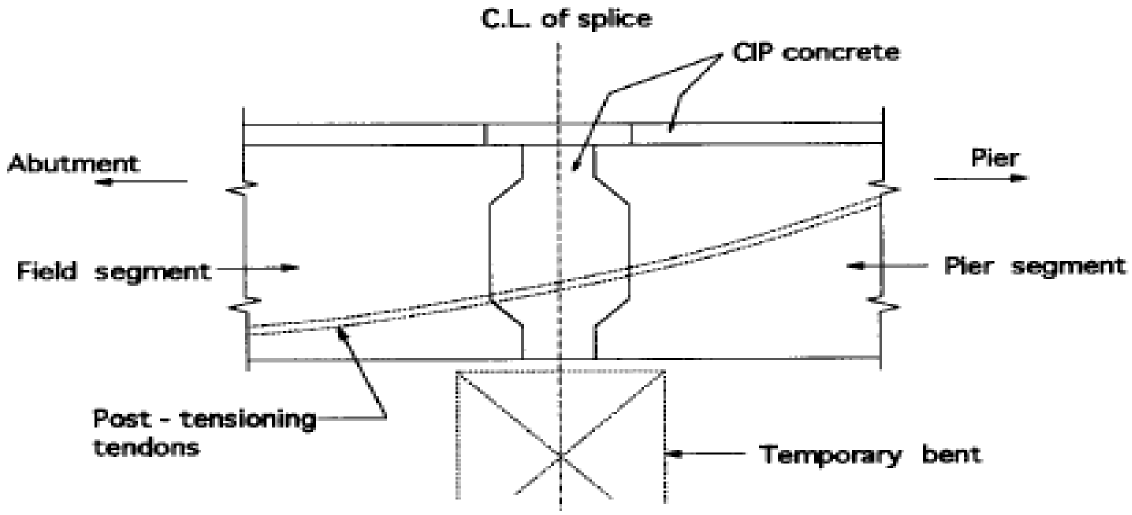


Figure 2.7. Cast-in-Place Post-Tensioned Splice (Abdel-Karim and Tadros 1995).

2.10.4 Stitched Splice

A stitched splice combines the advantage of fully reinforced splice and cast-in place post-tensioned and cancels out the dis-advantage of both. In a stitched splice, post-tensioning is carried out across the splice in short longitudinal tendons or threaded bars. Thickened ends are required at the splice to anchor the post-tensioning tendons. Also, higher reinforcement is required in that region for post-tensioning anchorages. Such a type of splice could be provided for both on-pier and in-span splices. This splice provides better serviceability as compared to reinforced concrete splice. A stitched splice was used in the Shelby Creek Bridge as shown in the Figure 2.8.

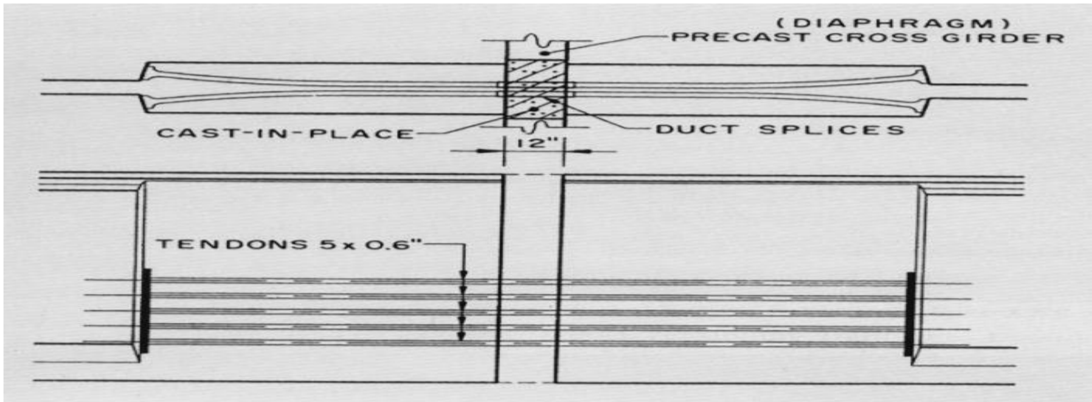


Figure 2.8. Stitched Splice Used in Shelby Creek Bridge (Caroland et al. 1992).

2.11 LATERAL STABILITY

Mast (1989) pointed out that lateral stability of the composite structure after the deck is cast is not the most critical case. The most critical condition for lateral stability occurs during the transportation. Concrete being torsionally stiff as compared to steel, twisting of middle part relative to beam ends was not considered to be a problem. The problem with lateral stability arises when the supports have roll flexibility and supports roll sideways which causes lateral bending of the beams. The condition when the beam hangs from the lifting point was identified as the most critical case. Equations were developed for determining factor of safety against buckling for hanging beam. The author recommended moving the lifting point from the end by small amount in order to improve the lateral stability.

Stratford and Burgoyne (1999) identified the three important stages in lateral stability analysis of girders as during lifting, transportation, placement of structure in storage. Three different support conditions were identified based on the various conditions as simply supported beam, transport-supported beam and the hanging beam. Owing to complexity of the stability analysis, a finite element analysis was performed and formulas are developed for buckling loads for three different conditions. It was shown that the hanging beam was the most critical case since no restraint is provided against rigid body rotation.

Nikzad et al. (2006) laid emphasis on lateral stability of girders during transportation and erection of girder segments in spliced girder bridges. The construction tolerances in the manufacturing of the girders in the location of prestress and lifting loops results in lateral bending of the top flange of the prestressed concrete girders. Also, the soft torsional stiffness of the trucks and dollies results in lateral bending of the precast concrete members. The article stated that all the safety factors associated with the transportation of girder segments are satisfied if the sum of transportation dolly rotational stiffness exceeds 55,000 k-in. /rad. Also, to increase the lateral stability of the girders, it was recommended to provide unbonded temporary strands in the top flange of the girder.

Ronald (2001) noted that intermediate diaphragms are typically provided at the closure pour locations. The author highlighted that intermediate diaphragms have been usually used at closure pours that have kinks at splices in horizontal curved alignment. Diaphragms help distribute the effect of wind forces and live load. Also, diaphragms add inertial mass to the structure which increases the inertial response of the system to seismic acceleration. However, Ronald suggested that diaphragm reinforcement adds addition level of congestion at the closure pour. Ronald pointed out that temporary cross bracings are provided at critical locations like the splices and the pier for lateral stability of the girder till the deck is cast and attains composite action.

3. DESIGN OUTLINE

3.1 INTRODUCTION

Two sets of application examples are developed to demonstrate the design of continuous precast prestressed concrete spliced girder bridges considering both a shored and a partially shored method of construction. A three-span bridge is considered to represent a typical spliced girder bridge configuration for the application examples. This is based on TxDOT's recommendation for the typical number of spans expected in practice. In shored construction, shoring towers are provided in both the end span and the center span. In partially shored construction, shoring towers are provided in the end span, but no shoring towers are provided in the center span. The design is carried out in accordance with the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012). The length of an individual girder segment is selected based on the length and weight limitations during handling at the precast plant and transportation. The girder spacing is based on typical practice followed by TxDOT. The design parameters such as material properties, strand diameter and concrete strength are representative of typical values used in Texas. Figure 3.1 provides an elevation view of the bridge. The following parameters are selected for the design examples.

- A span configuration of 190-240-190 ft is used for both the shored and partially shored cases.
- The length of the drop-in and the end girder segments is 140 ft, while that of the on-pier segment is 96 ft. A 2 ft splice connection length is assumed.
- For the shored case, prismatic modified Tx70 girder sections are used for all girder segments where the modified section uses a 9 in. web rather than the standard 7 in. web.
- For the partially shored case, prismatic modified Tx70 girders are used for the end and drop-in girder segments. Constant web depth haunched girders are used for the on-pier segments. The depth of these haunched girders varies

from 70 in. at the ends to a maximum depth of 108 in. at the centerline of the pier.

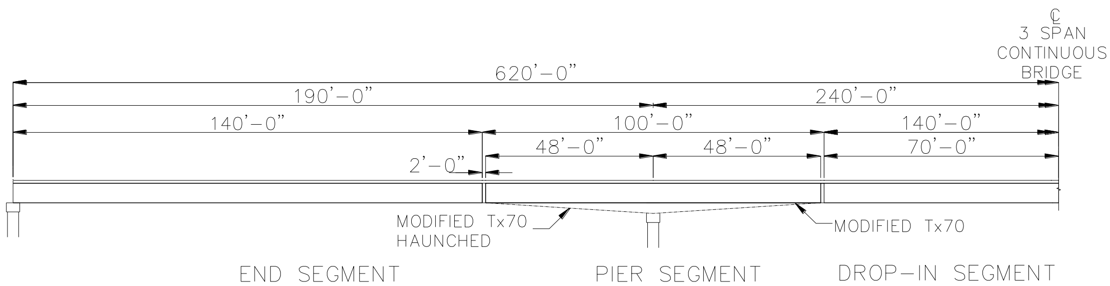


Figure 3.1. Elevation of Three-Span Continuous Bridge.

The load balancing technique is used for the design of prestressed concrete spliced girder bridges. The girders are designed for service loads and then checked for their ultimate capacity under live load and impact. The limit states considered for the application examples are as follows:

- Service stress under live loads and thermal gradients.
- Live load deflections.
- Shear demand and capacity at ultimate.
- Moment demand and capacity at ultimate.

This section provides an outline of all the critical design parameters for spliced girder bridges. Details of the selected design parameters, design assumptions, limit states and prestress losses for spliced girder bridges are provided.

3.2 DESIGN PARAMETERS

Table 3.1 gives the design parameters selected for the application examples. The design parameters such as concrete strength are based on standard practices that are followed throughout the state of Texas. A relative humidity of 65 percent is assumed based on the average value in Texas as specified in AASHTO LRFD Article 5.4.2.3. The other parameters, which include prestressing steel and mild steel, are based on the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

Table 3.1. Design Parameters.

Parameter		Selected Value
Concrete strength at service for deck slab, f'_c		4 ksi
Precast Concrete strength at release, f'_{ci}		6.5 ksi
Precast Concrete strength at service, f'_c		8.5 ksi
Coefficient of thermal expansion of concrete		6×10^{-6}/° F
Relative humidity		65%
Mild steel	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}
	Modulus of elasticity, E_p	28,500 ksi
Pre-Tensioning	Stress limit at transfer, f_{pi}	f_{pi} = 0.75 f_{pu}
	Stress limit at service, f_{pe}	f_{pe} = 0.8 f_{py}
Post-Tensioning	Prior to seating	f_{pi} = 0.90 f_{py}
	Stress limit at service	f_{pe} = 0.8 f_{py}
	Coeff. of friction, μ	0.25
	Wobble coefficient	0.0002/ ft
	Anchor set	0.375 in.

3.3 DESIGN ASSUMPTIONS

Design assumptions used for the application examples and parametric study in this thesis are based on the Phase 1 report for the TxDOT project on continuous prestressed concrete girder bridges (Hueste et al. 2012). The following assumptions are made for the application examples:

1. Post-tensioning tendons are stressed from both the ends during both Stage I and Stage II to minimize friction losses and to provide symmetry of stresses in the structure.
2. Post-tensioning tendons used for the modified Tx70 girder are internal and bonded. The post-tensioning tendons are encased in a 4 in. diameter metal duct. A maximum of 19-0.6 in. diameter strands can be encased in a 4 in. diameter duct. All the post-tensioning tendons are located in a single vertical plane.
3. For the design under consideration, the entire deck is cast in a single operation.
4. A reinforced concrete deck of 8 in. thickness is used. A 2 in. thick haunch is assumed between the girders and the deck to accommodate construction tolerances and variation in camber. A 2 in. thick asphalt wearing surface is used but is not considered a part of structural composite section and is treated as additional superimposed dead load.
5. The weights of deck forms, strongbacks, temporary diaphragms and other temporary components are minor and neglected in the design.
6. Permanent intermediate diaphragms are not considered in the design. Temporary intermediate diaphragms can be provided at critical locations like the splices and piers for lateral stability of the girder until the deck slab attains composite action. (Note that permanent diaphragms can be considered when desirable for the purpose of lateral stability. This option will be discussed in Section 7.)

7. The composite section properties are based on the transformed effective width of the composite deck slab considering the specific modulus of elasticity for the girder and deck, respectively.
8. The sign convention for the design considers tension as positive and compression as negative.

3.4 DEAD LOADS

Dead loads considered in the design are girder self-weight, and weight of the haunch, slab, barrier and wearing surface. For the haunch segment, self-weight varies linearly with increasing depth from the prismatic section at the splice to the centerline of pier. The load due to deck is distributed to the individual girder based on center-to-center spacing between the girders. The loads due to wearing surface and barrier loads act on the composite section and are distributed equally to all the girders. Table 3.2 gives the dead loads acting on each individual girder.

Table 3.2. Dead Loads for Modified Tx70 Girder.

Load Type	Value (kip/ft)	Applied to
Self-weight prismatic	1.152	Girder Section
Self-weight haunch (for pier segment-partially shored case)	1.152-2.488	Girder Section
Deck weight	0.800	Girder Section
Haunch weight (between girder and deck)	0.079	Girder Section
Barrier weight	0.109	Composite Section
Wearing surface	0.187	Composite Section

3.5 LIVE LOADS

The AASHTO LRFD Specifications HL-93 load model is used for the live load analysis of the girder. Three traffic lanes are considered for the design in accordance with the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012). The live load is to be taken as one of the following combinations, whichever yields maximum stresses at the section considered.

1. Design Truck and Design Lane load.

   The design truck load consists of one front axle weighing 8 kips and two rear axles weighing 32 kips each, spaced 14 ft apart. A dynamic load allowance factor of 33 percent is considered for the design truck. The design lane load consists of 0.64 klf uniformly distributed in the longitudinal direction and is not subjected to a dynamic load allowance. Figure 3.2 shows the details for design truck and design lane load.

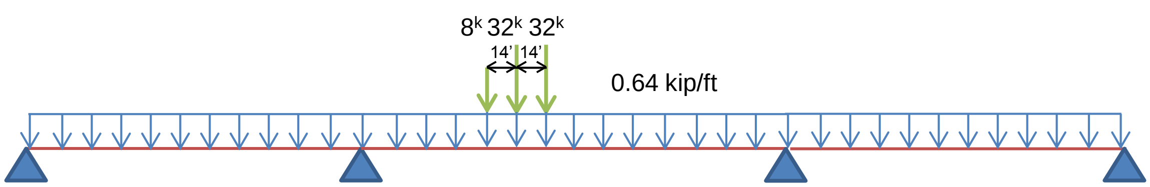


Figure 3.2. Design Truck and Design Lane Load.

2. Design Tandem and Design Lane load.

   The design tandem load consists of a pair of 25 kip axles spaced 4 ft apart and is subjected to a dynamic load allowance. The design lane load consists of 0.64 klf uniformly distributed in the longitudinal direction and is not subjected to a dynamic load allowance. Figure 3.3 shows the details for design tandem and design lane load.

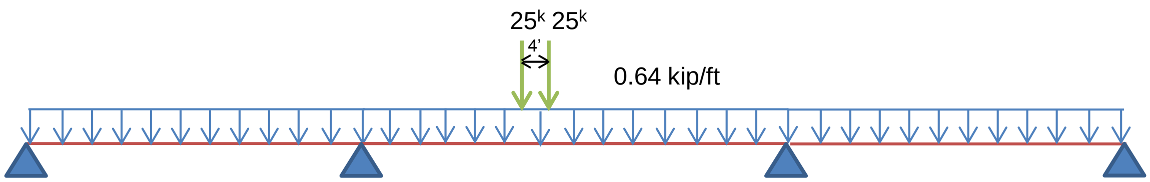


Figure 3.3. Design Tandem and Design Lane Load.

The live load moments and shear forces including the dynamic load effect are distributed to the individual girders using distribution factors (DFs). AASHTO LRFD Tables 4.6.2.2.2 and 4.6.2.2.3 specify the distribution factors for moment and shear for I-shaped girder sections. The use of these DFs is allowed for prestressed concrete girders having an I-shaped cross-section with composite slab, if the conditions outlined below are satisfied. For bridge configurations not satisfying the limits below, refined analysis is required to estimate the moment and shear DFs.

1. Width of slab is constant
2. Number of girders (N_b) is not less than four
3. Girders are parallel and of the same stiffness
4. The roadway part of the overhang, $d_e \leq 3.0$ ft
5. Curvature in plan is less than 4 degrees
6. Cross-section of the bridge girder is consistent with one of the cross-sections given in AASHTO LRFD Table 4.6.2.2.1-1.
7. $3.5 \leq S \leq 16.0$
8. $4.5 \leq t_s \leq 12.0$
9. $20 \leq L \leq 240$
10. $10{,}000 \leq K_g \leq 7{,}000{,}000$

where:

K_g $= n(I + Ae_g^2)$

n = Modular ratio between the girder and slab concrete

A = Area of the girder cross-section, in.^{2}

e_g^2 = Distance between the centroid of the girder and the slab, in.

S = Beam Spacing, ft

L = Span Length, ft

N_b = Number of beams

d_e = Distance from exterior web of exterior beam to the interior edge of curb or traffic barrier, in.

t_s = Thickness of slab, in.

The live load DF formulas for precast prestressed concrete I-shaped girders are given in Table 3.3. These formulas are valid within their range of applicability.

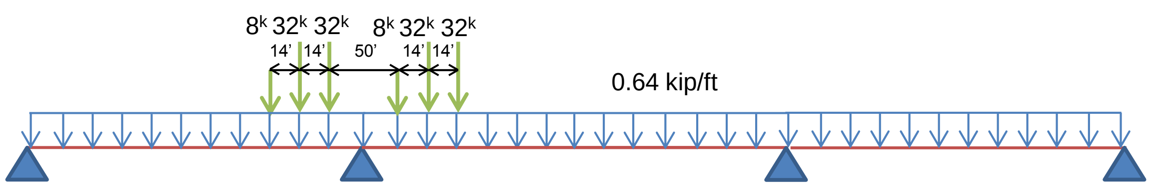
Table 3.3. LRFD Live Load DFs for Concrete Deck on Modified Tx70 Girder.

Category	DF Formulas	Range of Applicability
Live Load Distribution per Lane for Moment in Interior Beam	One Design Lane Loaded: $$0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$ Two or More Design Lanes Loaded: $$0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$ $10{,}000 \le K_g$ $\le 7{,}000{,}000$
Live Load Distribution per Lane for Moment in Interior Beam	One Design Lane Loaded: Lever Rule Two or More Design Lanes Loaded: $$g = eg_{interior}$$ $$e = 0.77+\frac{d_e}{9.1}$$	$-1.0 \le d_e$ ≤ 5.5
Live Load Distribution per Lane for Shear in Interior Beam	One Design Lane Loaded: $$0.36+\frac{S}{25}$$ Two or More Design Lanes Loaded: $$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$
Live Load Distribution per Lane for Shear in Interior Beam	One Design Lane Loaded: Lever Rule Two or More Design Lanes Loaded: $$g = eg_{interior}$$ $$e = 0.6+\frac{d_e}{10}$$	$-1.0 \le d_e$ ≤ 5.5

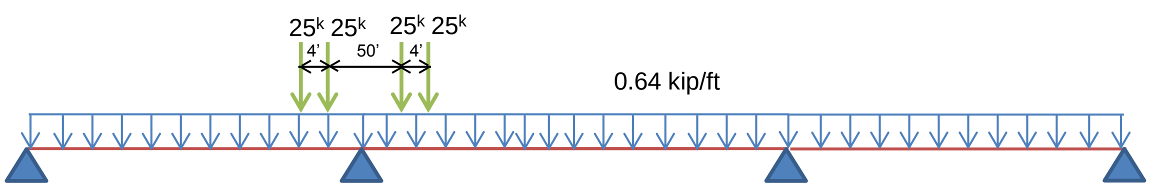
According to AASHTO LRFD Article 3.6.1.3.1, the maximum shear and negative moment under the vehicular live load is calculated as the larger of:

1. 90 percent of the effect of (Two Design Trucks + Design Lane Load).
2. 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 to produce the maximum negative moment demand and shear demand as shown in Figure 3.4.The loads are symmetric over the support. The two design trucks/tandems shall be placed in adjacent spans to produce maximum force effects.



(a) Design Truck and Design Lane Load



(b) Design Tandem and Design Lane Load

Figure 3.4. Critical Load Placement of HL-93 Vehicular Live Load over Continuous Span for Maximum Shear Demand.

3.6 ALLOWABLE STRESS LIMITS

The design of spliced girder bridges involves various stages. It is necessary to ensure that the girder stresses are within limits during all the stages of construction. Tables 3.4 and Table 3.5 summarize the allowable stress limits as given in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012). The allowable stress limits have been computed for the girder for a specified concrete compressive strength at service (f'_c) of 8.5 ksi and a specified concrete compressive strength at transfer (f'_{ci}) of 6.5 ksi. For the deck, a specified concrete compressive strength (f'_c) of 4 ksi is used. The reduction factor ϕ_w, for the compressive stress limit at the final loading stage is taken 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).

Table 3.4. Summary of Allowable Stress Limits in Girder.

Stage of Loading	**Type of Stress**	**Allowable Stress Limits**	
		f'_c or f'_{ci} (ksi)	**Limiting Value (ksi)**
Initial Loading Stage at Transfer	Compressive	$-0.60\ f'_{ci}$	-3.825
	Tensile	$0.24\sqrt{f'_{ci}}$	0.611
Intermediate Loading Stage at Service	Compressive	$-0.45\ f'_c$	-3.825
	Tensile	$0.19\sqrt{f'_c}$	0.550
Final Loading Stage at Service	Compressive: Case I	$-0.60\Phi_w\ f'_c$	-5.100
	Compressive: Case II	$-0.40\ f'_c$	-3.400
	Tensile	$0.19\sqrt{f'_c}$	0.550

Table 3.5. Summary of Allowable Stress Limits in Deck.

Stage of Loading	Type of Stress	Allowable Stress Limits	
		f'_c (ksi)	**Limiting Value (ksi)**
Final Loading Stage	Compressive	$-0.60\ f'_c$	-2.400
	Tensile	$0.19\sqrt{f'_c}$	0.380

3.7 LIMIT STATES

3.7.1 Service Limit State

For prestressed concrete members, the service load design typically governs, and the design satisfying service load criteria usually satisfies the strength limit state. Service load stresses are checked during various stages of construction based on the limits given in Table 3.4 and Table 3.5. Tension in prestressed concrete members is checked considering the Service III limit state while compression is checked using the Service I limit state as specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

Service I – checks compressive stresses in prestressed concrete components:

$$Q = 1.00(DC + DW) + 1.00(LL + IM) \tag{3.1}$$

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:

$$Q = 1.00(DC + DW) + 0.80(LL + IM) \tag{3.2}$$

3.7.2 Flexural Strength Limit State

The flexural strength limit state needs to be 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, expressed as follows.

$$\phi\, M_n \geq M_u \tag{3.3}$$

where:

M_u = Factored ultimate moment at a section, kip-ft

M_n = Nominal moment strength at a section, kip-ft

ϕ = Resistance factor

= 1.0 for flexure and tension of prestressed concrete members.

The total ultimate bending moment for Strength I limit state, according to the *AASHTO LRFD Specifications* is as follows.

$$M_u = 1.25\,(M_{DC}) + 1.5\,(M_{DW}) + 1.75(M_{LL+IM}) \tag{3.4}$$

where:

M_{DC} = Bending moment due to all dead loads except wearing surface, kip-ft

M_{DW} = Bending moment due to wearing surface load, kip-ft

M_{LL+IM}= Bending moment due to live load and impact, kip-ft

3.7.3 Shear Limit State

The *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012) specifies using the Modified Compression Field Theory (MCFT) for transverse shear reinforcement. MCFT takes into account the combined effect of axial load, flexure and prestressing when designing for shear. Shear in prestressed concrete members is checked using the Strength I limit state as specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012). The shear strength of concrete is based on parameters β and θ. The transverse reinforcement is based on demands of both transverse and interface shear. The interface shear design is based on shear friction theory where the total resistance is based on the cohesion and friction maintained by shear friction reinforcement crossing the crack.

The AASHTO LRFD Specifications require that transverse reinforcement is provided at sections with the following condition.

$$V_u > 0.5\phi(V_c + V_p) \tag{3.5}$$

where:

V_u = Factored shear force at the section, kips

$= 1.25(DC) + 1.5(DW) + 1.75(LL + IM)$

DC = Shear force at the section due to dead loads except wearing surface load, kips

DW = Shear force at the section due to wearing surface load, kips

$LL + IM$= Shear force at the section due to live load including impact, kips

V_c = Nominal shear strength provided by concrete, kips

V_p = Component of prestressing force in the direction of shear force, kips

ϕ = Strength reduction factor specified as 0.9 for shear in prestressed concrete members

The nominal shear resistance at a section is the lesser of the following two values:

$$V_n = V_c + V_s + V_p \text{ and} \tag{3.6}$$

$$V_n = 0.25f_c'b_v d_v + V_p \tag{3.7}$$

Shear resistance provided by the concrete, V_c, is given as:

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v \tag{3.8}$$

Shear resistance provided by transverse steel reinforcement, V_s, is given as:

$$V_s = \frac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{3.9}$$

where:

d_v = Effective shear depth, in.

b_v = Girder web width, in.

f_c' = Girder concrete strength at service, ksi

V_p = Component of prestressing force in the direction of shear force, kips

β = Factor indicating ability of diagonally cracked concrete to transfer tension

θ = Angle of inclination of diagonal compressive stresses (slope of compression field), radians.

A_v = Area of shear reinforcement within a distance s, in.^{2}

s = Spacing of stirrups, in.

f_y = Yield strength of shear reinforcement, ksi

α = Angle of inclination of diagonal transverse reinforcement to longitudinal axis, taken as 90 degrees for vertical stirrups

3.7.4 Deflection

As a final check for service conditions, the girders are checked for allowable deflection at live load and impact as specified in the AASHTO LRFD Specifications Article 2.5.2.6.2. The deflection limit state ensures that there are no undue vibrations in the bridge and also limits the cracking in members. In order to investigate maximum deflections for straight girder systems, all the design lanes are loaded and all the supporting components are assumed to deflect equally. 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.

1. Vehicular load, general = Span/800
2. Vehicular and/or pedestrian loads = Span/1000

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

Figure 3.5 shows the critical load arrangement for vehicular live loads to produce maximum deflections in the continuous girders. For maximum deflection in the center span, the resultant of reaction from point loads should be placed at the midspan. For maximum deflection in end span, the resultant of reaction from point loads should be located at the maximum positive moment location in the end span.

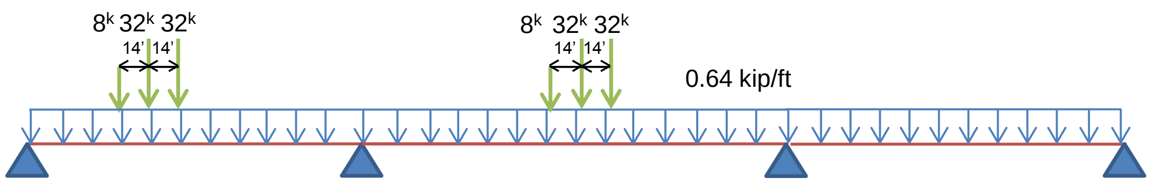


Figure 3.5. Critical Load Placement of HL-93 Vehicular Live Load over Continuous Span for Maximum Deflection.

3.8 PRESTRESS LOSSES

Prestressing operations are accompanied with losses that result in a reduction of the total prestressing force with time. The prestress losses are classified into instantaneous losses and long-term losses. The losses due to elastic shortening and initial steel relaxation are grouped into instantaneous losses. The losses due to creep, shrinkage and steel relaxation after transfer are long-term losses. The losses due to creep and shrinkage are time dependent. For post-tensioned members, along with these losses, friction and anchor set losses also need to be included. Based on previous research, empirical formulas are provided for computation of prestress losses. An approximate method can be used for computation of prestress losses for preliminary design. The general equations for an approximate estimate of prestress losses in prestressed concrete members are given below.

3.8.1 Approximate Estimation of Losses

3.8.1.1 Elastic Shortening

The AASHTO LRFD Specifications (AASHTO 2012) specify the following expression to calculate loss in prestress due to elastic shortening.

For pretensioned members:

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \tag{3.10}$$

For post-tensioned members:

$$\Delta f_{pES} = \left(\frac{N-1}{2N}\right) \frac{E_p}{E_{ci}} f_{cgp} \tag{3.11}$$

where:

Δf_{pES} = Prestress loss due to elastic shortening, ksi

E_p = Modulus of elasticity of prestressing reinforcement, ksi

E_{ci} = Modulus of elasticity of girder concrete at release, ksi

= $33{,}000\, {w_c}^{1.5}\sqrt{f'_{ci}}$

w_c = Unit weight of girder concrete, kcf

f'_{ci} = Girder concrete strength at transfer, ksi

f_{cgp} = Sum of concrete stresses at the center-of-gravity of the prestressing steel due to the prestressing force at transfer and self-weight of the member at section of maximum moment, ksi

N = Number of identical prestressing tendons

3.8.1.2 Steel Relaxation

The AASHTO LRFD Specifications provide the following expressions to estimate the loss in prestress due to relaxation of steel.

At transfer – low-relaxation strands initially stressed in excess of $0.5f_{pu}$:

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj} \tag{3.12}$$

where:

Δf_{pR1} = Prestress loss due to steel relaxation at transfer, ksi

t = Time estimated in days from stressing to transfer

f_{pj} = Initial stress in tendon at the end of stressing, ksi

f_{py} = Specified yield strength of prestressing steel, ksi

After transfer – low-relaxation strands:

$$\Delta f_{pR2} = 0.3\left[20.0 - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right)\right] \quad (3.13)$$

where:

Δf_{pR2} = Prestress loss due to steel relaxation after transfer, ksi

Δf_{pES} = Prestress loss due to elastic shortening, ksi

Δf_{pSR} = Prestress loss due to concrete shrinkage, ksi

Δf_{pCR} = Prestress loss due to concrete creep, ksi

3.8.1.3 Concrete Creep

The AASHTO LRFD Specifications provide the following expression to estimate the loss in prestress due to creep of concrete.

$$\Delta f_{pCR} = 12 f_{cgp} - 7\Delta f_{cdp} \geq 0 \quad (3.14)$$

where:

Δf_{pCR} = Prestress loss due to concrete creep, ksi

f_{cgp} = Sum of concrete stresses at the center-of-gravity of the prestressing steel due to prestressing force at transfer and self-weight of the member at section of maximum moment, ksi

Δf_{cdp} = Change in concrete stresses at the center-of-gravity of the prestressing steel due to permanent loads, except the dead load present at the time the prestress force is applied, calculated at the same section as f_{cgp}, ksi

3.8.1.4 Concrete Shrinkage

The AASHTO LRFD Specifications provide the following expression to estimate the loss in prestress due to concrete shrinkage.

$$\Delta f_{pSR} = 17 - 0.15H \quad (3.15)$$

where:

Δf_{pSR} = Prestress loss due to concrete shrinkage, ksi

H = Mean annual ambient relative humidity in percent, taken as 65 percent for this preliminary study.

3.8.1.5 Losses due to Friction

The AASHTO LRFD Specifications Article 5.9.5.2.2 provides the following expression to estimate the loss in prestress due to friction between internal post-tensioning tendons and the duct.

$$\Delta f_{pF} = f_{pj}(1 - e^{-(Kx+\mu\alpha)}) \tag{3.16}$$

where:

Δf_{pF} = Prestress loss due to friction, ksi

f_{pj} = Stress in the post-tensioning tendons at jacking, ksi

x = Length of a tendon from the jacking end to any point under consideration, ft

K = Wobble friction coefficient, per ft of tendon

μ = Coefficient of friction

α = Sum of the absolute values of angular change of the tendon path from the jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation, rad.

3.8.2 Refined Estimate of Time Dependent Losses

For complex prestressed concrete bridges, exact evaluation of prestress losses is desired. A more exact estimate of prestress losses can be made using the time step method. An approximate method can be used for computation of prestress losses for preliminary design. However, for final design, AASHTO LRFD Specifications Article 5.9.5.4.1 specifies a time step method for computation of prestress losses for spliced girder bridges. For a refined estimate of time dependent losses, prestress losses are calculated at different stages of load application. The general equation for computing time dependent prestress losses is as follows:

$$\Delta f_{pLT} = \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df} \quad (3.17)$$

where:

Δf_{pSR} = Prestress loss due to shrinkage of girder concrete between transfer and deck placement, ksi

Δf_{pCR} = Prestress loss due to creep of girder concrete between transfer and deck placement, ksi

Δf_{pR1} = Prestress loss due to relaxation of prestressing strands between time of transfer and deck placement, ksi

Δf_{pR2} = Prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time, ksi

Δf_{pSD} = Prestress loss due to shrinkage of girder concrete between time of deck placement and final time, ksi

Δf_{pCD} = Prestress loss due to creep of girder concrete between time of deck placement and final time, ksi

Δf_{pSS} = Prestress gain due to shrinkage of deck in composite section, ksi

$\left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id}$ = Sum of time dependent prestress losses between transfer and deck placement, ksi

$(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$ = Sum of time dependent prestress losses after deck placement, ksi

However, the exact computation of prestress losses is cumbersome for spliced girder bridges because of multiple stages of pre-stressing and combined pre-tensioning and post-tensioning. According AASHTO LRFD Specifications Article 5.9.5.2.3, whenever combined pre-tensioning and post-tensioning are involved and when post-tensioning is not applied in identical increments, the effect of subsequent post-tensioning on previously stressed members should be considered. Accordingly, multiple stages of prestressing will have an effect on creep and elastic shortening of members which needs to be included in the losses. A time step analysis that includes the effects of multiple stages of prestressing will provide an accurate evaluation of prestress losses. The following expressions show the effect of multiple stages of prestressing on prestress losses.

Losses in Pretensioning:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pR} + \Delta f_{pCR} + \Delta f_{pSR} + \left(\Delta f_{pES} + \Delta f_{pCR}\right)_{\mathrm{PT1}} + \left(\Delta f_{pES} + \Delta f_{pCR}\right)_{\mathrm{PT2}} \quad (3.18)$$

where,

Δf_{pT} = Total loss in prestress, ksi

Δf_{pES} = Loss due to elastic shortening, ksi

Δf_{pR} = Loss due to relaxation, ksi

Δf_{pCR} = Loss due to creep, ksi

Δf_{pSR} = Loss due to shrinkage, ksi

$(\Delta f_{pES} + \Delta f_{pCR})_{PT1}$ = Elastic shortening and creep loss due to Stage I post-tensioning, ksi

$(\Delta f_{pES} + \Delta f_{pCR})_{PT2}$ = Elastic shortening and creep loss due to Stage II post-tensioning, ksi

Losses in Stage I Post-tensioning:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pR} + \Delta f_{pF} + \Delta f_{pCR} + \Delta f_{pSR} + \left(\Delta f_{pES} + \Delta f_{pCR}\right)_{\mathrm{PT2}} \quad (3.19)$$

where,

Δf_{pF} = Loss due to friction, ksi

The remaining variables are same as defined above.

Losses in Stage II Post-tensioning:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pR} + \Delta f_{pF} + \Delta f_{pCR} + \Delta f_{pSR} \quad (3.20)$$

The variables are same as defined above.

A software analysis can be performed to compute prestress losses for spliced girder bridges. An input of all the time-dependent material properties is required along with section properties, prestressing tendons, construction stages and applied loads for the software analysis. Time intervals between various stages of construction are required. An exact estimation of prestress losses is unwarranted during preliminary design stage. However, for detailed design, an exact evaluation of prestress losses is required.

3.9 TIME DEPENDENT PROPERTIES

Time dependent material properties of concrete are important in analysis and design of spliced girder bridges. The time dependent material properties have an effect on deflection, stresses and prestress losses. The important time dependent properties that need to be considered are creep, shrinkage, modulus of elasticity and compressive strength of concrete. Accurate estimation of modulus of elasticity helps determine camber and elastic gains and losses. Creep and shrinkage of concrete has a significant effect on deflections and stresses. The effect of creep and shrinkage is more pronounced in the deck region over the piers. Shrinkage of concrete results in tensile stresses in the deck. Because of creep, the compression in the deck reduces. The values of creep coefficient and shrinkage strain should be selected based on mix specific data or prior experience. In absence of specific data, an average values for the creep coefficient and shrinkage strains can be used. According to AASHTO LRFD Specifications Article 5.4.2.3, when mix specific data is not available, estimates of creep and shrinkage can be made by:

- Articles 5.4.2.3.2 and 5.4.2.3.3
- CEB-FIP Model code
- ACI 209

The general equations to determine creep coefficient, shrinkage strain, and modulus of elasticity of concrete, as specified in AASHTO 5.4.2.3, are as follows:

3.9.1 Creep

The AASHTO LRFD Specifications (AASHTO 2012) provide the following expression to determine the creep coefficient in concrete.

$$\psi(t, t_i) = 1.9 k_s k_{hc} k_f k_{td} t_i^{-0.118} \tag{3.21}$$

in which:

k_s $= 1.45 - 0.13(V/S) \geq 1.0$

k_{hc} $= 1.56 - 0.008H$

k_f $= \left(\frac{5}{1+f'_{ci}}\right)$

k_{td} $= \left(\frac{t}{61-4f'_{ci}+t}\right)$

where,

H = Relative humidity (%). In the absence of better information H may be taken from AASHTO LRFD Specifications Figure 5.4.3.3-1

k_s = Factor for the effect of the volume to surface ratio of the component

k_{hc} = Humidity development factor

k_f = Factor for the effect of concrete strength

k_{td} = Time development factor

t_i = Age of concrete at the time of load application

(V/S) = Volume to surface ratio (in.)

f'_{ci} = Specified compressive strength of concrete at the time of prestressing for pre-tensioned members and at time of initial loading for non-prestressed members. If concrete age at time of initial loading is unknown at design time, f'_{ci} may be taken as $0.8f'_c$ (ksi).

3.9.2 Shrinkage

The AASHTO LRFD Specifications (AASHTO 2012) provide the following expression to determine the shrinkage strain in concrete.

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} (0.48 \times 10^{-3}) \tag{3.22}$$

in which:

k_{hs} = Humidity factor for shrinkage

= $(2.00 - 0.014H)$

The remaining variables are the same as defined previously.

3.9.3 Modulus of Elasticity

The AASHTO LRFD Specifications (AASHTO 2012) provide the following expression to estimate the modulus of elasticity in concrete.

$$E_c = 33{,}000 k_1 w_c^{1.5} \sqrt{f_c'} \tag{3.23}$$

where,

k_1 = Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction.

w_c = Unit weight of concrete.

f_c' = Specified compressive strength of concrete.

4. CASE STUDY 1 - SHORED CONSTRUCTION

4.1 INTRODUCTION

The following example gives the details for design of a three-span continuous precast prestressed concrete girder bridge using shored construction. A modified Tx70 girder section has been used for this bridge. The design is based on the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

4.2 BRIDGE DESCRIPTION

The bridge shown in Figure 4.1 represents a typical three-span continuous prestressed concrete bridge. The length of the drop-in and end girder segments is 140 ft and that of the on-pier segments is 96 ft. The end spans are 190 ft and the center span is 240 ft in length. The ratio of end span to center span is 0.8. The width of the splice is 2 ft. Prismatic modified Tx70 girders with a 9 in. web width are used for all girder segments.

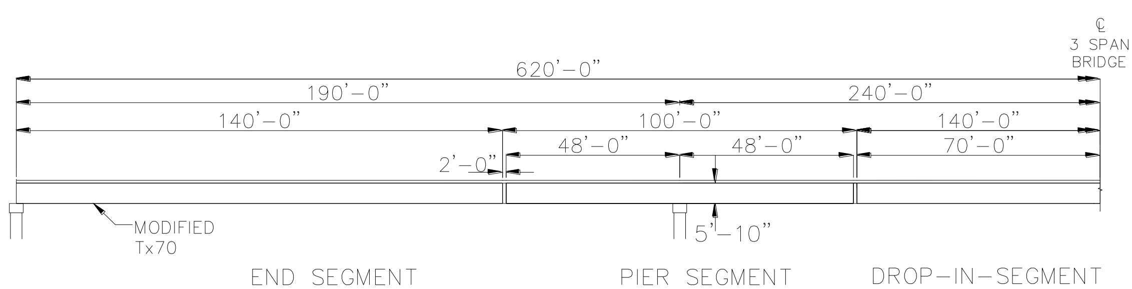


Figure 4.1. Elevation View of Three-Span Continuous Bridge for Shored Construction.

4.3 BRIDGE GEOMETRY AND GIRDER CROSS-SECTION

The bridge cross-section at midspan is shown in Figure 4.2. The bridge has a total width of 46 ft and total roadway width of 44 ft. The bridge superstructure consists of six Tx70 girders spaced 8 ft center-to-center, with 3 ft overhangs on each side designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 2 in. TxDOT standard T501 type rails are considered in the design. Three design lanes are considered for the purpose of design in accordance with the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

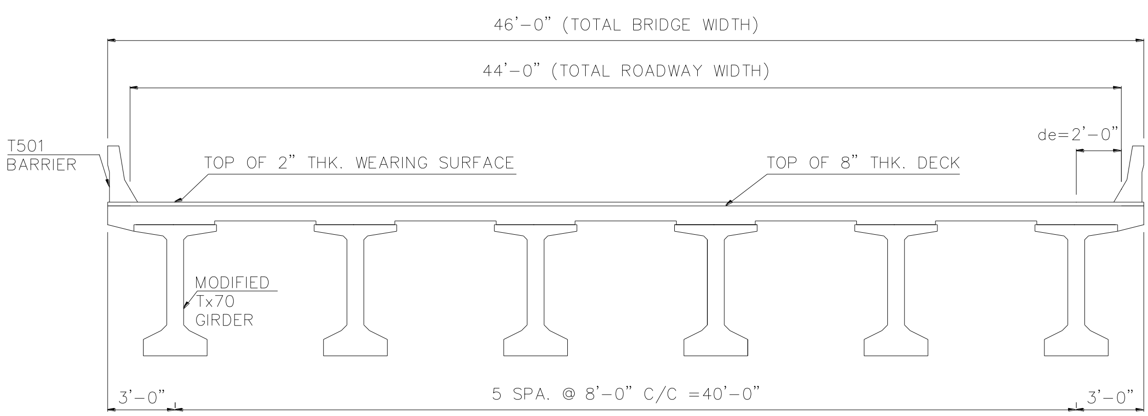
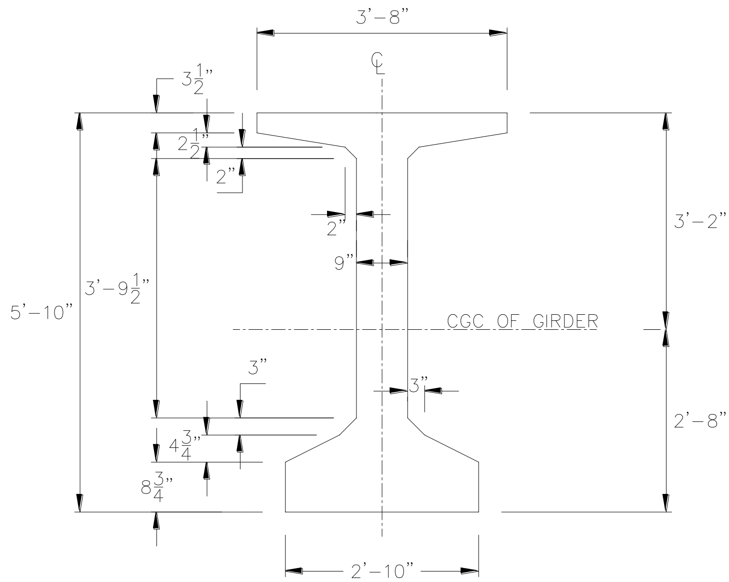


Figure 4.2. Transverse Bridge Section at Midspan for Shored Construction.

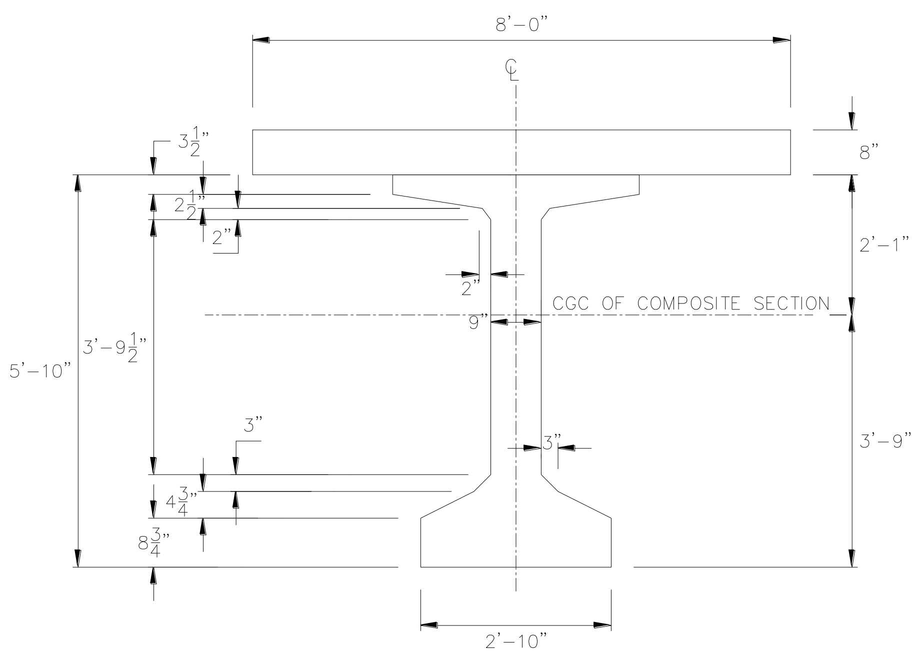
A modified Tx70 girder has been considered for the design. The web width of the standard Tx70 girder has been increased to 9 in. to allow placement of post-tensioning ducts. This results in an increase in the width of the top flange to 44 in. and of the bottom flange to 34 in. Table 4.1 provides the composite and non-composite section properties for the modified Tx70. Figure 4.3 shows the details of the non-composite and composite section for the prismatic modified Tx70 girder, respectively.

Table 4.1. Section Properties for Prismatic Modified Tx70 Girder for Shored Construction.

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})
Tx70 Modified	37.7	32.3	1106	687,111
Tx70 Modified Composite	32.7	45.3	1607	1,287,145



(a) Non-composite Section



(b) Composite Section

Figure 4.3. Prismatic Modified Tx70 Girder for Shored Construction.

4.4 DESIGN PHILOSOPHY

4.4.1 General

The load balancing technique has been used for the design of the continuous Tx70 prestressed concrete bridge girders. A two stage post-tensioning approach is used. Stage I post-tensioning is applied individually to girders to balance the self-weight. Then, Stage II post-tensioning is carried out to balance the deck weight and superimposed dead load.

4.4.2 Handling and Transportation

4.4.2.1 Overview

The drop-in and end segments are transported from the precast plant to the construction site while supporting their ends. Pre-tensioning and Stage I post-tensioning is applied to balance the self-weight of the girders. Figure 4.4 shows the support configuration during transportation of the drop-in and end segments.

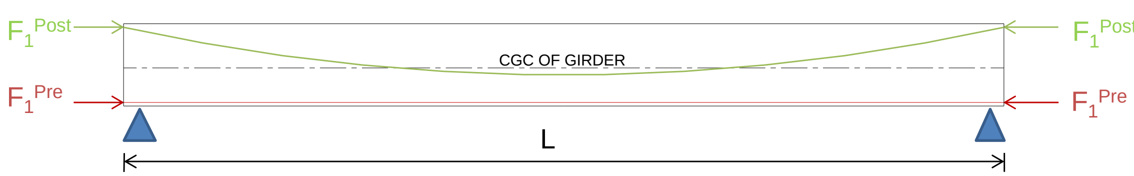


Figure 4.4. Support Arrangement During Transportation of Drop-in and End Segments for Shored Construction.

The on-pier segment is transported by supporting it at the quarter span points near the ends of the girder. A large amount of the prestress force is required in the top flange of the on-pier segment because these segments cantilever from the piers and eventually support the ends of the drop-in and end segments. Pre-tensioning and Stage I post-tensioning is applied in the precasting plant to balance the self-weight and the reaction from drop-in segment. Until the time that the pier segment supports the drop-in girder segment, the tension stresses in the bottom flange are high. This is offset by providing temporarily prestressed Dywidag bars in the bottom flange. Four temporary unbonded Dywidag threadbars of 1.25 in. diameter and f_{pu} equal to 150 ksi are provided in the bottom flange of the pier segments. Once the pier segment is erected on site, it behaves as a cantilever, and the Dywidag bars are released and grouted to act as non-prestressed compression reinforcement.

Figure 4.5 shows the details of support configuration during transportation of the on-pier girder segment.

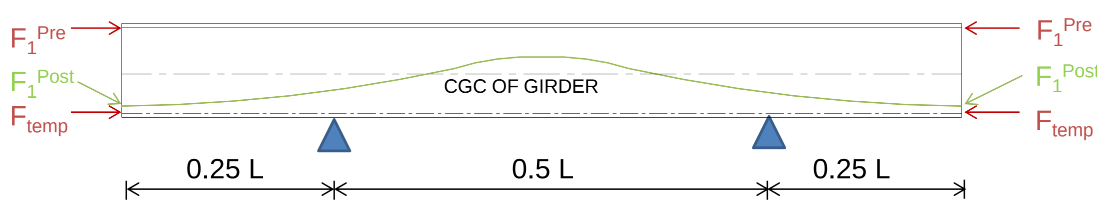


Figure 4.5. Support Arrangement During Transportation of On-Pier Segment for Shored Construction.

The span lengths and weights of girder segments are taken into consideration during handling and transportation. In the state of Texas, precasters recommend a maximum transportable segment length of 160 ft, a maximum weight of 200 kips and a maximum depth of 10 ft (Hueste et al. 2012). Table 4.2 provides the span lengths and weights for the girder segments. It is observed that the segment lengths and weights are within transportation limits.

Table 4.2. Segment Lengths and Girder Weights for Shored Construction.

Girder Segments	Length (ft)	Weight (kips)
End Segment	140	161
Drop-in-Girder Segment	140	161
On-Pier Segment	96	111
Limits in Texas	160	200

4.4.2.2. Pre-tensioning

For pre-tensioning of the girder segments, 0.6 in. diameter Grade 270 low relaxation strands with an ultimate tensile strength f_{pu} of 270 ksi are considered. The initial stress in pre-tensioning strands prior to transfer, f_{pi}, is taken as 0.75 f_{pu} which is equal to 202.5 ksi. The force at transfer is calculated after taking the initial losses due to steel relaxation and elastic shortening into account. Prestress losses of 20 percent are assumed in the pre-tensioned strands at service. Table 4.3 presents the pre-tensioning strands design summary for the girder segments.

Table 4.3. Pre-tensioning Strands Design Summary for Shored Construction.

Description	End Segment	On-Pier Segment	Drop-in Segment
	Bottom Flange	Top Flange	Bottom Flange
No. of Strands (0.6 in. dia.)	32	26	30
Prestress Force at Transfer (kips)	1312	1066	1230
Prestress Force at Service (kips)	1125	913	1054

4.4.2.3 Stage I Post-tensioning

Stage I post-tensioning is provided individually to each girder segment. The details of parameters used for post-tensioning are outlined in Table 3.1. The force at transfer is calculated after taking the initial losses due to elastic shortening, anchor set and friction into account. Long term prestress losses due to steel relaxation, creep and shrinkage of 25 percent are assumed for the Stage I post-tensioning at service. Table 4.4 presents the post-tensioning design summary for the girder segments.

An initial estimate of the amount of post-tensioning required can be obtained by

$$(F * \delta) = \frac{W * L^2}{8} \tag{4.1}$$

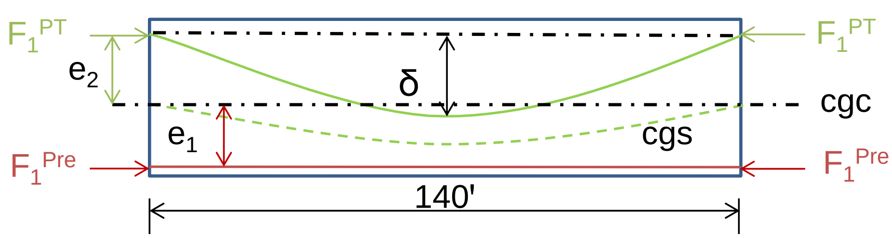
Where:

F = Required post-tensioning force

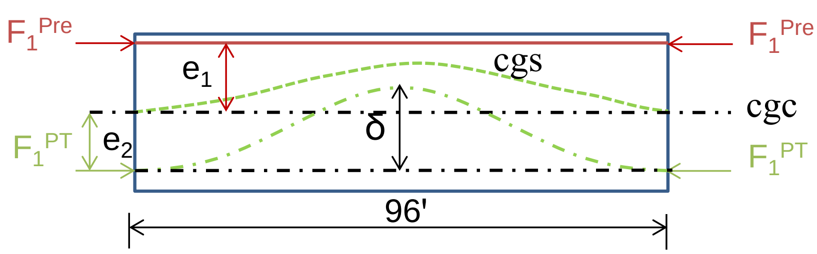
W = Total dead load (girder self-weight)

L = Span length

δ = Eccentricity of tendons



(a) Drop-in girder segment



(b) Over-pier girder segment

Figure 4.6. Load Balancing for Tx70 Girder Segments.

Table 4.4. Stage I Post-Tensioning Design Summary for Shored Construction.

Description	End Segment	On-pier Segment	Drop-in Segment
No. of Strands (0.6 in. dia.)	19 (1 duct)	38 (2 ducts)	19 (1 duct)
Prestress Force at Transfer (kips)	779	1558	779
Prestress Force at Service (kips)	584	1168	584

4.4.3 Construction on Site

4.4.3.1 Construction Sequence

After the girders are transported to the job site, the girders are lifted and placed on piers and temporary shoring towers. Then splice is cast, deck is poured and Stage II post-tensioning is carried out to balance the weight of the deck and to provide compression in the deck. Figure 4.7 shows the details of various stages of construction. The step-by-step construction procedure is as follows.

(a) Erect piers, temporary supports and abutments. Set on-pier girder segments on the piers and secure the girders to the temporary shoring towers located at A and D in the end spans. The shoring towers at B and C in the center span should be lowered.

(b) Attach strongbacks to the ends of the end segments at ground level. Erect the end girder segment on the abutment and shoring towers. Connect the strongbacks to the on-pier girder segment. The shoring towers should be capable of transferring the reaction from the end girder segment to the foundation.

(c) Attach the strongbacks to the ends of the drop-in girder segment at ground level. Erect the drop-in-girder segment by connecting the strongbacks to the on-pier girder segment. It is necessary to ensure that the end girder segments are installed prior to this step. This minimizes uplift caused by the erection of the drop-in-girder segment. Tie-downs could also be used to prevent the uplift.

(d) After all the segments have been placed, check the vertical alignment of the girder ends. Strongbacks help in maintaining the vertical alignment of the adjacent girders prior to threading the post-tensioning strands through the ducts. Then, thread the post-tensioning tendons through the ducts in the web of the girders. Cast the splice in between the girder segments. Once the splice has cured and gained sufficient strength, remove the strongbacks. Raise the shoring towers located at B and C in the center span

(e) Construct the formwork for the deck and place the precast deck panels and deck reinforcement. Pour the concrete for the deck.

(f) After the deck has cured and gained sufficient strength, stress the Stage II post-tensioning and grout the tendons. Remove the temporary shoring towers located at A, B, C and D.

(g) Cast the barriers and wearing surface. After a suitable time interval, the bridge is opened to traffic.

An alternate sequence of construction can be considered where the end segments can be erected first which would put a downward reaction in the shoring towers and the pier segments can be erected later. This would prevent the uplift in the shoring towers which is expected during the erection of pier segments.

4.4.3.2 Stage II Post-tensioning

Stage II post-tensioning is designed to act continuously to balance the deck and superimposed dead load and is to be carried out on site after the girders are erected on temporary supports and piers. The details of parameters used for post-tensioning are outlined in Table 3.1. The force at transfer is calculated after taking the initial losses due to elastic shortening, anchor set and friction into account. Long-term prestress losses due to steel relaxation, creep and shrinkage of 15 percent are assumed for the Stage II post-tensioning at service. It is observed that service stresses may control the amount of post-tensioning provided. Stage II post-tensioning is carried out after the deck is cast which helps to provide compression in the deck at service. This reduces the amount of cracking in the deck in the pier region due to the effect of live loads. Table 4.5 shows the details for post-tensioning.

Table 4.5. Stage II Post-Tensioning Design Summary for Shored Construction.

Description	End Segment	On-pier Segment	Drop-in Segment
No. of Strands (0.6 in. dia.)	57 (3 ducts)	57 (3 ducts)	57 (3 ducts)
Prestress Force at Transfer (kips)	2337	2337	2337
Prestress Force at Service (kips)	1987	1987	1987

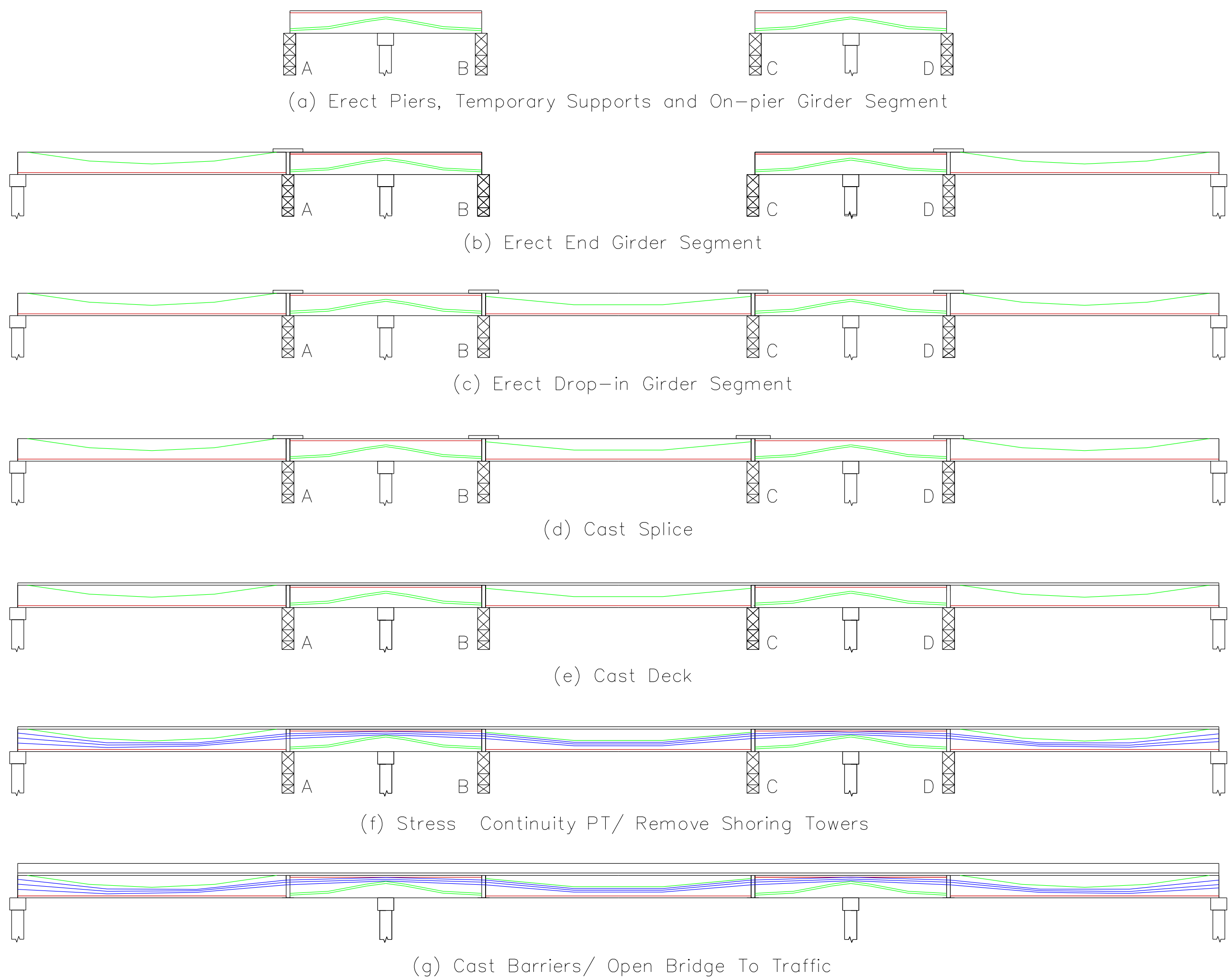
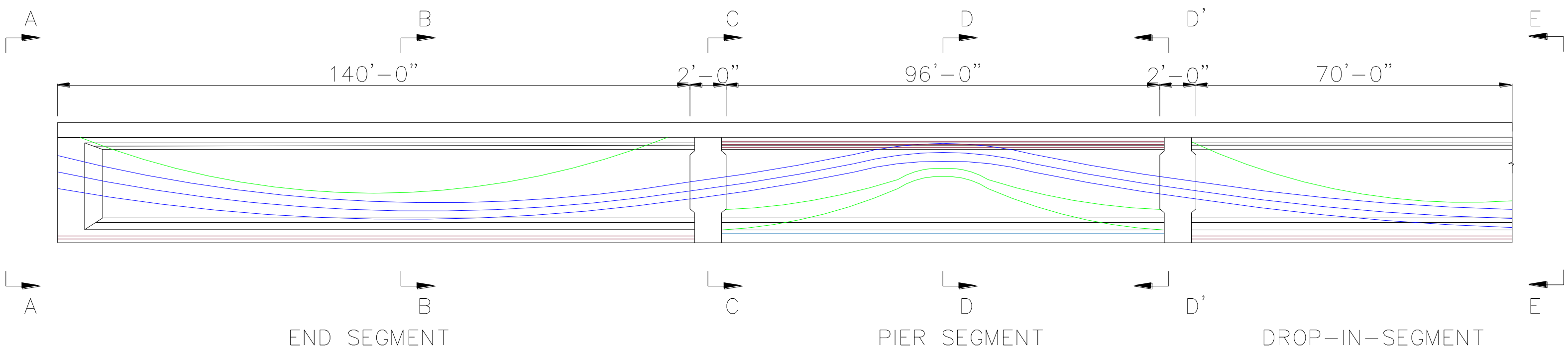


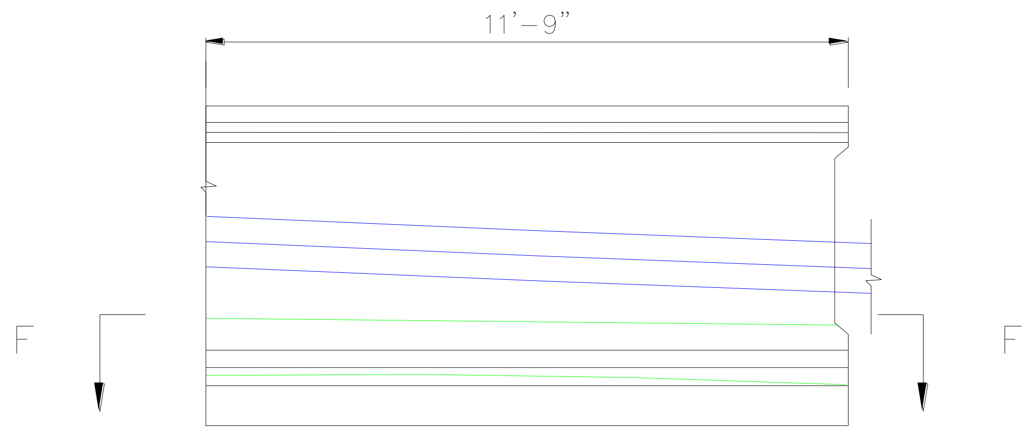
Figure 4.7. Stages of Construction for Shored Construction.

4.5 PRESTRESSING LAYOUT

Figure 4.8 shows the prestressing details for the girder segments at the anchorage end in the end span (Section A-A), at 0.4*L* from the abutment support of the end span (Section B-B), at the end span splice (Section C-C), at the face of the pier (Section D-D), at the interior span splice (Section D'-D'), at the midspan of interior span (Section E-E) and at the anchor zone of the interior span (Section F-F). The Stage I post-tensioning tendons are provided individually to all the girder segments and are anchored at the ends of the girder. An option of anchoring the post-tensioning ducts in the interior span is shown in Section D'-D'. Thickened ends are required for anchoring the post-tensioning ducts at the ends of the girders. However, for aesthetic purposes, the web of the girder could be thickened only on the interior face of the girder. Thickened end blocks are provided for a distance equal to depth of the girder and then gradually tapered to the thickness of the web as shown in the plan view in Section F-F. In order to anchor the post-tensioning ducts on the vertical face of the girder, the post-tensioning ducts are staggered which deviates them from the vertical plane as shown in the plan view in Section F-F. Figure 4.9 shows details of the post-tensioning layout for the three-span continuous bridge.

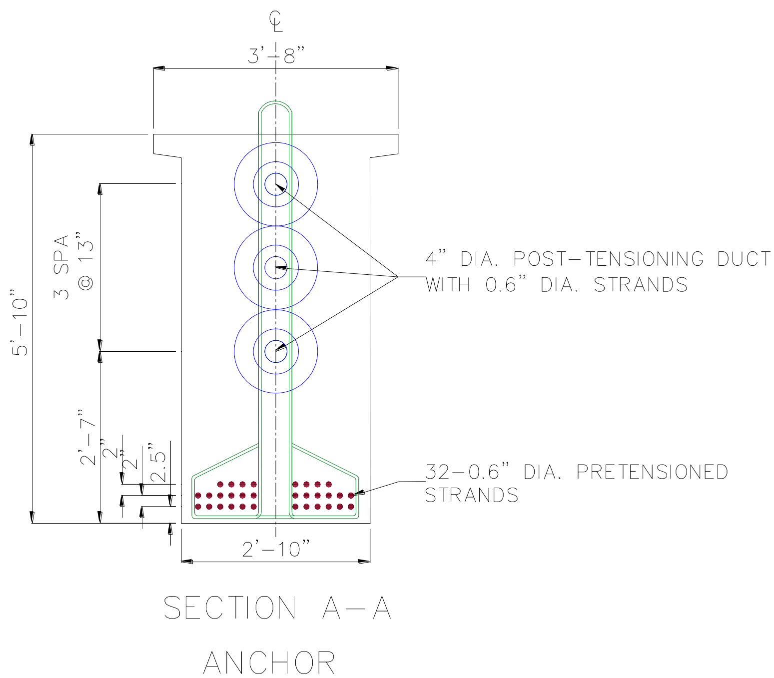


(a) Elevation View

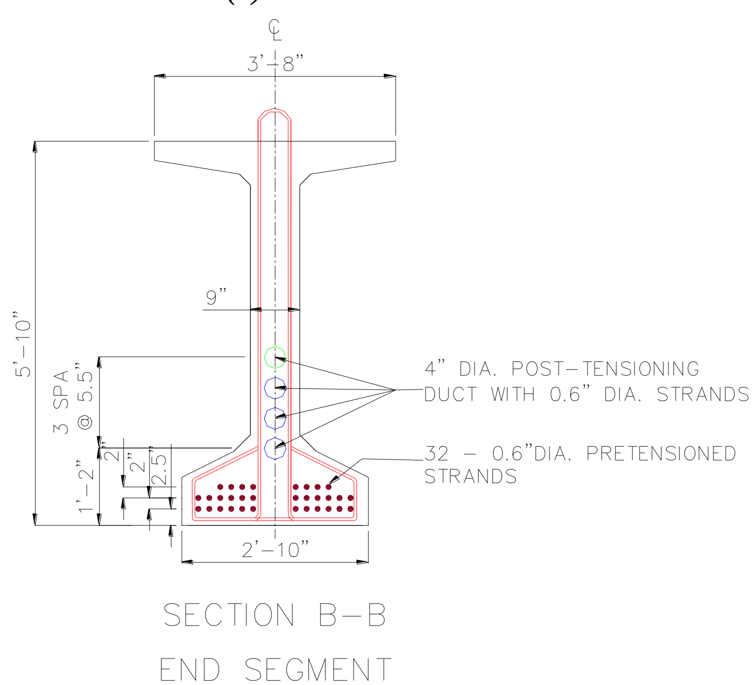


(b) Elevation View at Anchor Zone at Splice

Figure 4.8. Prestressing Details for Continuous Prestressed Concrete Modified Tx70 Girder Bridge Using Shored Construction.

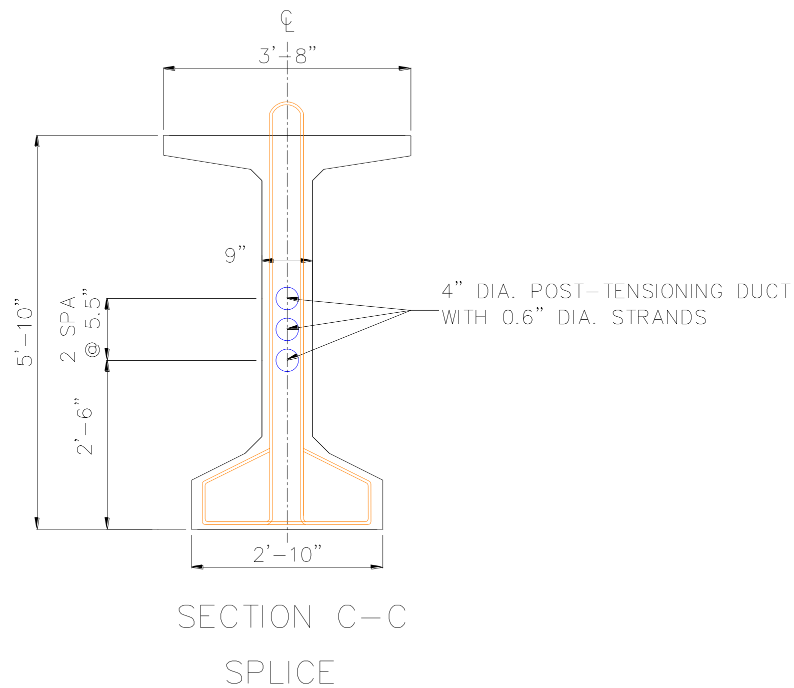


(c) Section A-A at Anchor

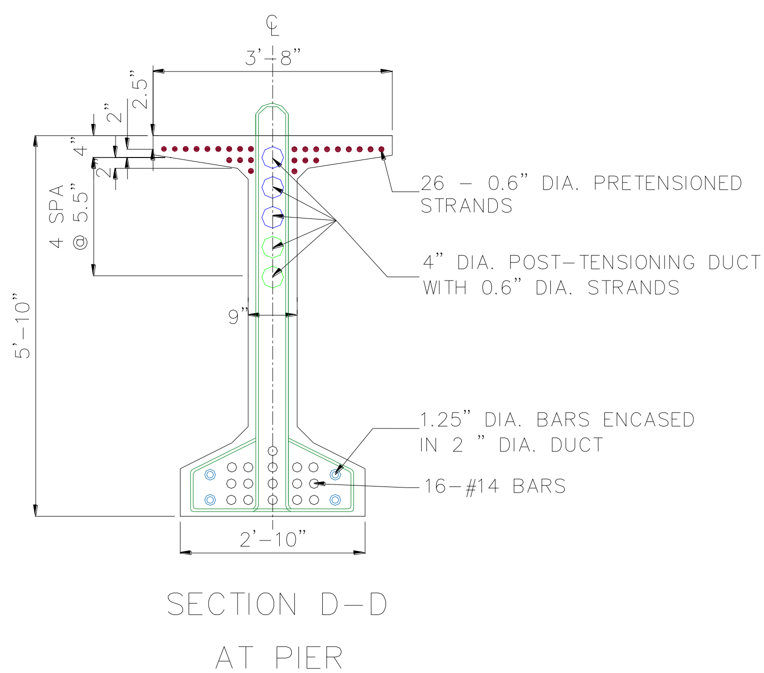


(d) Section B-B at End Segment

Figure 4.8. Continued.

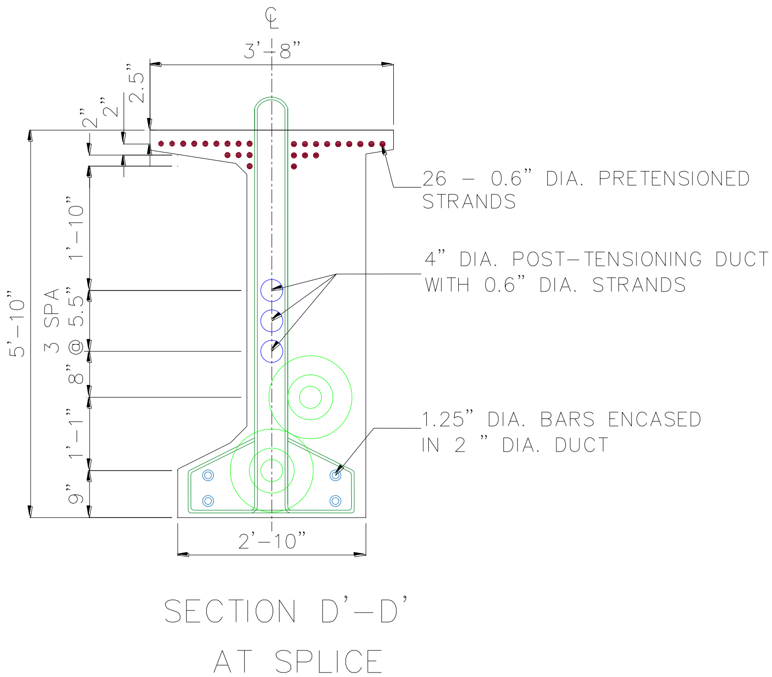


(e) Section C-C at Splice

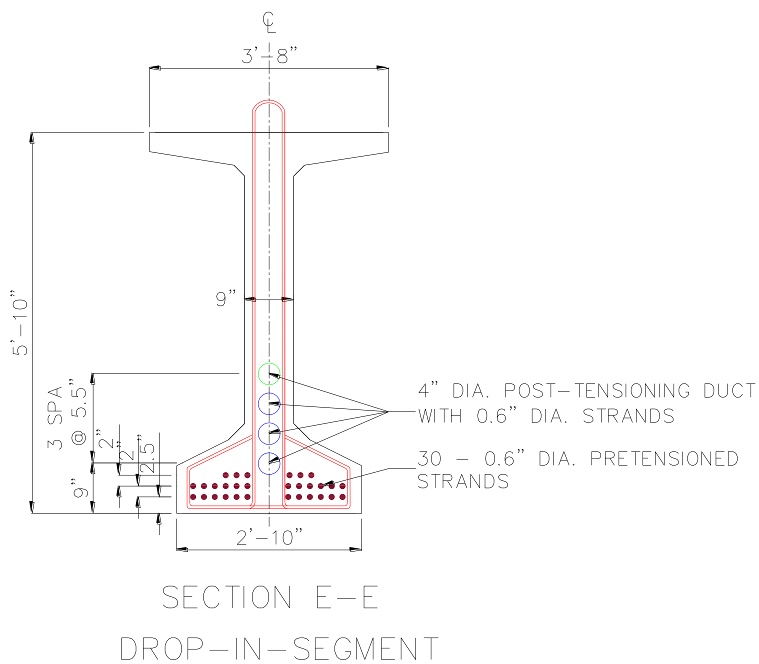


(f) Section D-D at Pier

Figure 4.8. Continued.

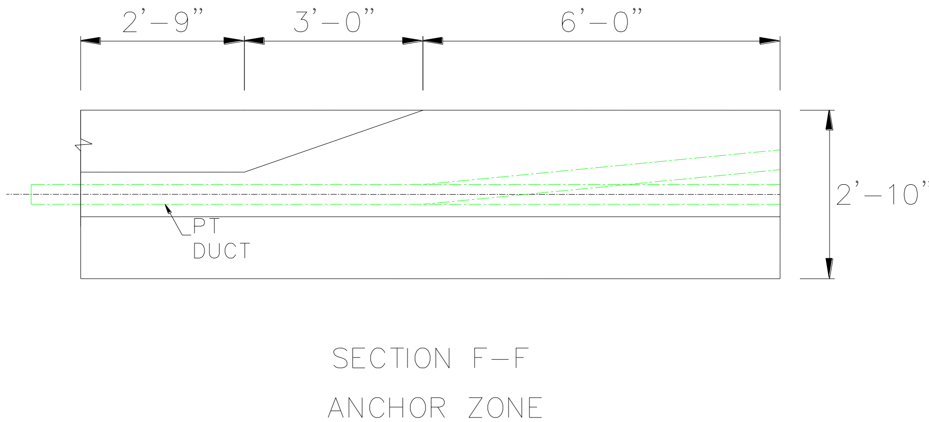


(g) Section D'-D' at Splice



(h) Section E-E at Drop-in Segment

Figure 4.8. Continued.



(i) Section F-F at Anchor End of On-Pier Segment

Figure 4.8. Continued.

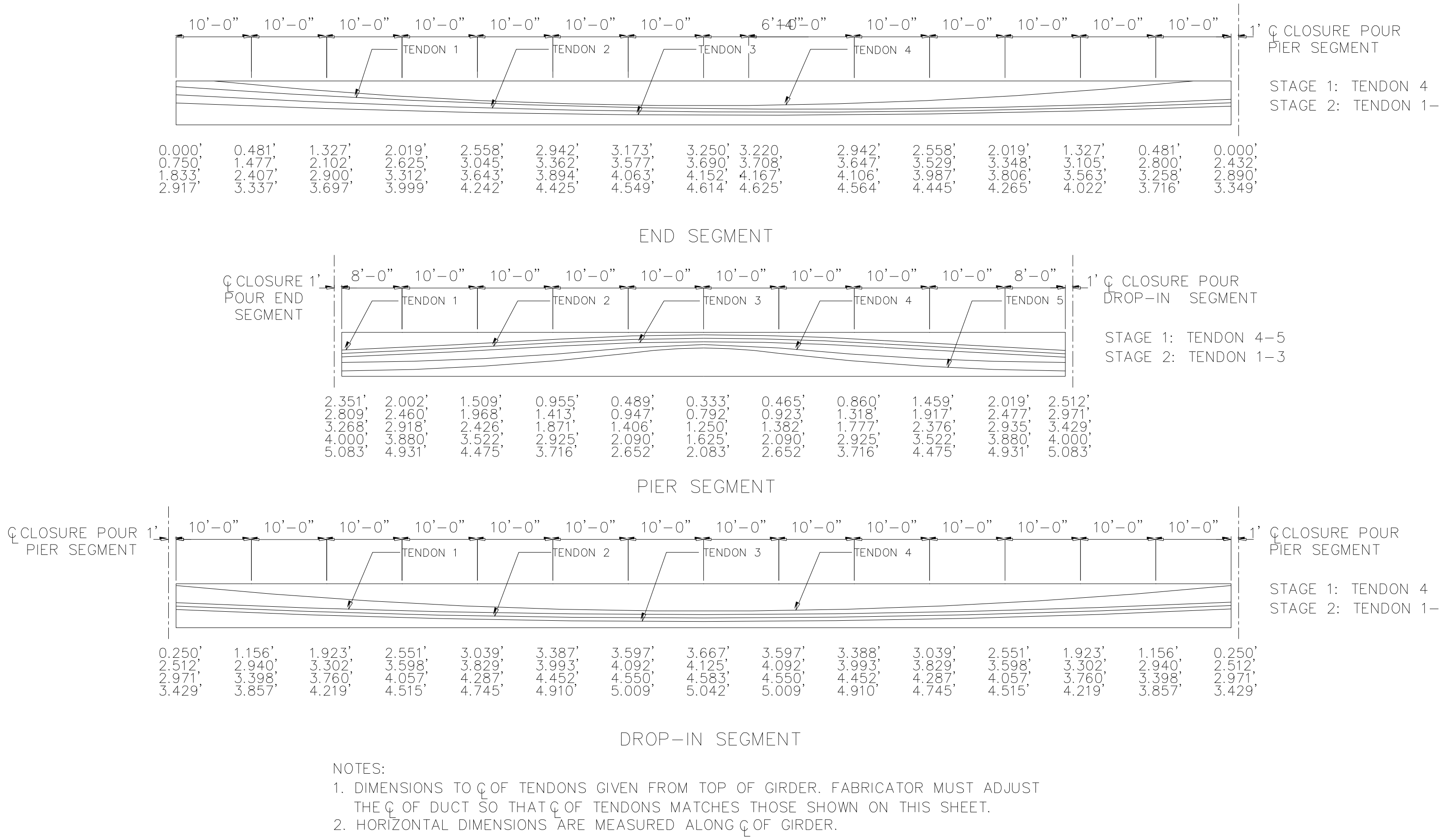


Figure 4.9. Post-Tensioning Layout for Continuous Prestressed Concrete Modified Tx70 Girder Bridge Using Shored Construction.

4.6 MOMENTS DURING VARIOUS STAGES OF CONSTRUCTION

The moments during various stages of construction are computed at selected locations along the structure. The moments are computed at 0.4*L* from the abutment support within the end span (Section A-A), at the splice in the end span (Section B-B), at the face of the pier (Section C-C), at the interior span splice (Section D-D), and at midspan of the interior span (Section E-E), as shown in the Figure 4.10. The moments due to self-weight, pre-tensioning, Stage I post-tensioning and the wet CIP deck act on the non-composite girder section. The moments due to removal of shoring towers, superimposed dead loads and Stage II post-tensioning act on the composite girder section. The moments due to prestressing are computed before losses. The girder moments at each section are summarized in Table 4.6. Figures 4.11 and 4.12 show the moments acting on the non-composite girder section and the composite girder section, respectively.

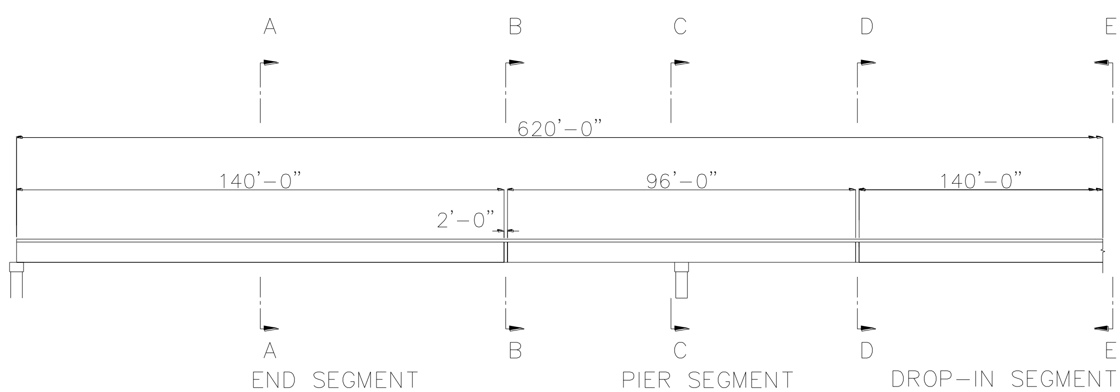
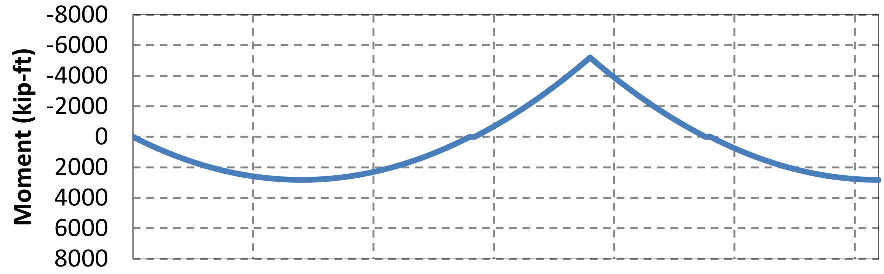


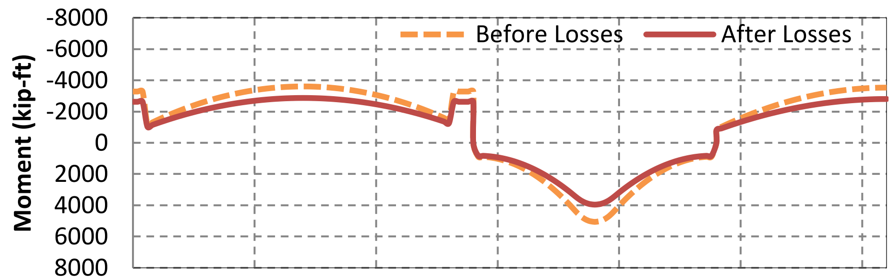
Figure 4.10. Section Locations for Moments for Three-Span Continuous Bridge Using Shored Construction.

Table 4.6. Girder Moments at Various Sections for Shored Construction.

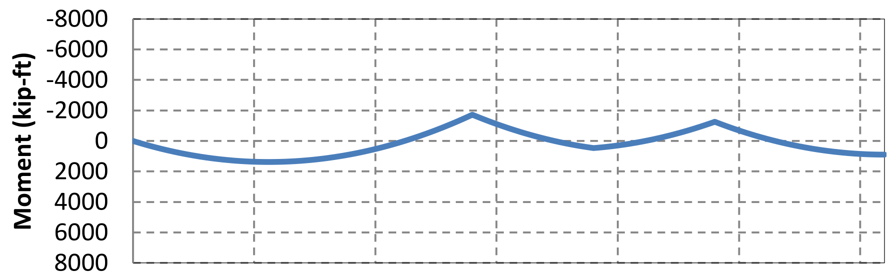
Loading	Section				
	A-A (End Segment)	B-B (Splice Exterior)	C-C (Pier)	D-D (Splice Interior)	E-E (Drop-in Segment)
Girder Self-Weight	2822	-	-1383	-	2822
Pre-tensioning and Stage I Post-tensioning	-3281	-	5185	-	-3511
Reaction From Drop-in segment	-	-	-3871	-	-
Haunch and Deck	1293	-1719	467	-1256	896
Stage II Post Tensioning	-4161	195	5436	-327	-3344
Shoring Support Removal	942	1763	-4593	1306	1306
Superimposed Dead Load	725	11	-1391	15	739
Live Load	5736	3660	-5391	2371	6109



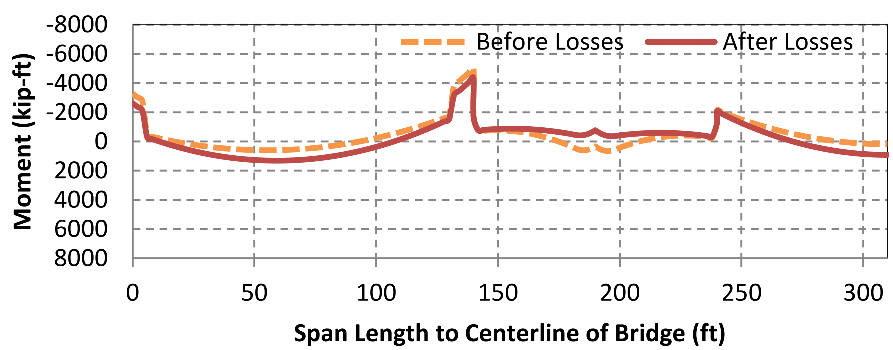
(a) Self-weight and Girder Reaction



(b) Pre-Tensioning and Stage I Post-Tensioning

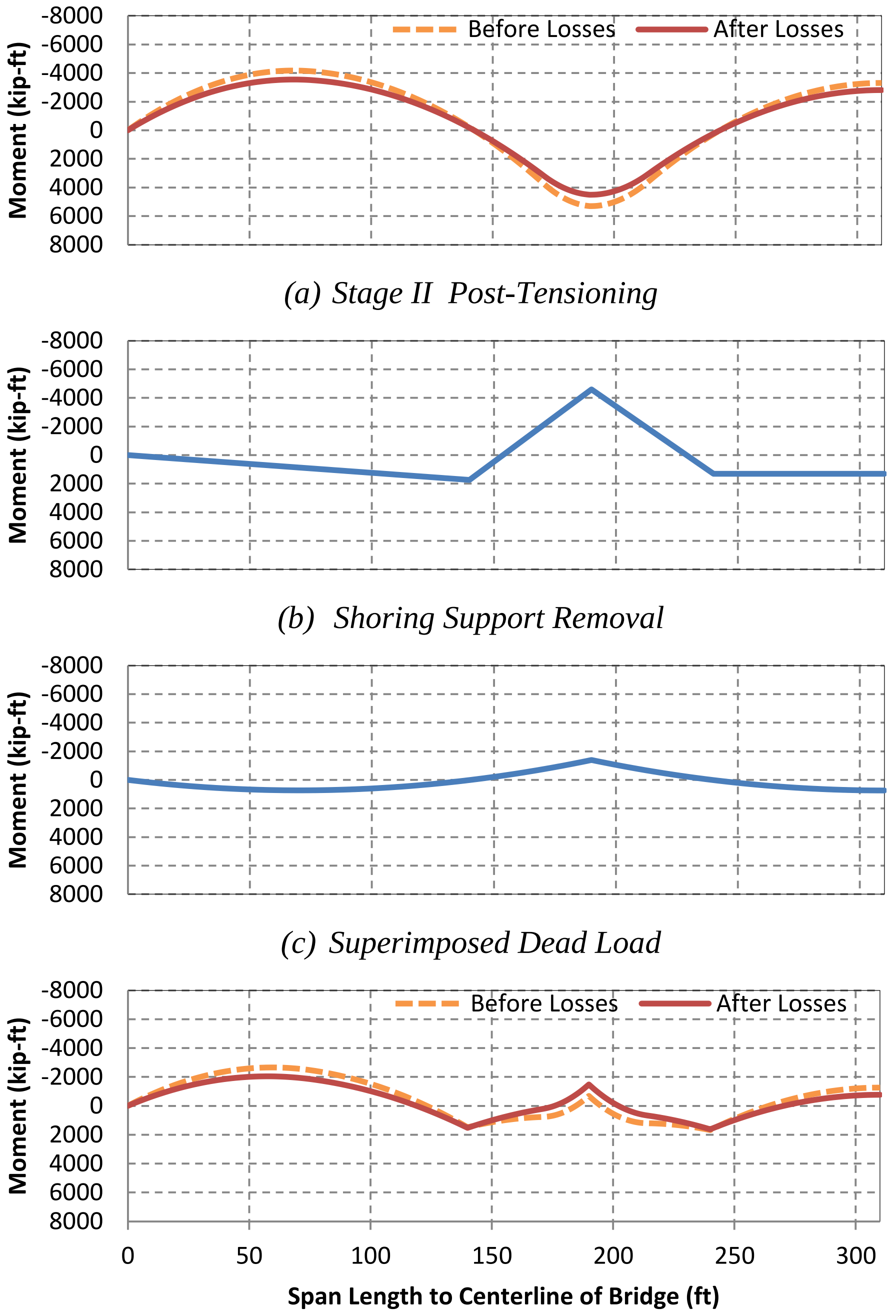


(c) Wet Deck Weight



(d) Girder Moments with Wet Deck

Figure 4.11. Moments Acting on Non-Composite Girder for Shored Construction.



(a) Stage II Post-Tensioning

(b) Shoring Support Removal

(c) Superimposed Dead Load

(d) Total Composite Section Moments

Figure 4.12. Moments Acting on Composite Girder for Shored Construction.

4.7 SERVICE STRESS ANALYSIS

Service stress analysis is carried out under the effect of dead loads, prestress, live loads and temperature and thermal gradient. The stresses are checked at various steps of construction. The important construction steps for checking girder stresses are identified as follows:

- Step I: Girder segments supported on piers and temporary supports.
- Step II: Girders supporting weight of wet CIP deck.
- Step III: Application of Stage II post-tensioning, removing of shoring towers and casting of barriers.
- Step IV: Bridge in Service.

For the various stages of construction, stress checks are provided at the following points: (1) at 0.4*L* of the end span, (2) at the splice in the end span, (3) at the face of pier, (4) at the splice in the center span, and (5) at the midspan of center span (see Figure 4.10). The allowable tension and compression limits at various stages of construction are provided in Table 3.4 and Table 3.5. Compression in prestressed concrete girders is evaluated through the Service I limit state while tension in prestressed concrete girders is evaluated through the Service III limit state.

Figures 4.13 through 4.17 present the stress blocks at each of these five sections. Table 4.7 summarizes the stresses at each section.

The stress blocks are obtained by adding the stress values due to the effect of various loads during the different steps of construction. The stress blocks are divided into two parts ‘Part a’ and ‘Part b’. ‘Part a’ shows the stresses during construction and ‘Part b’ shows the stresses during the service life of the bridge. ‘Part a’ and ‘Part b’ are further divided into two halves. The top half shows the stress values due to individual loads. The bottom half shows the cumulative effect of the stress values due to the corresponding individual load. The cumulative effect is obtained by adding the preceding cumulative value to the stress value due to individual loads. The final stress value corresponding to cumulative effect of ‘Part a’ is carried over to ‘Part b’.

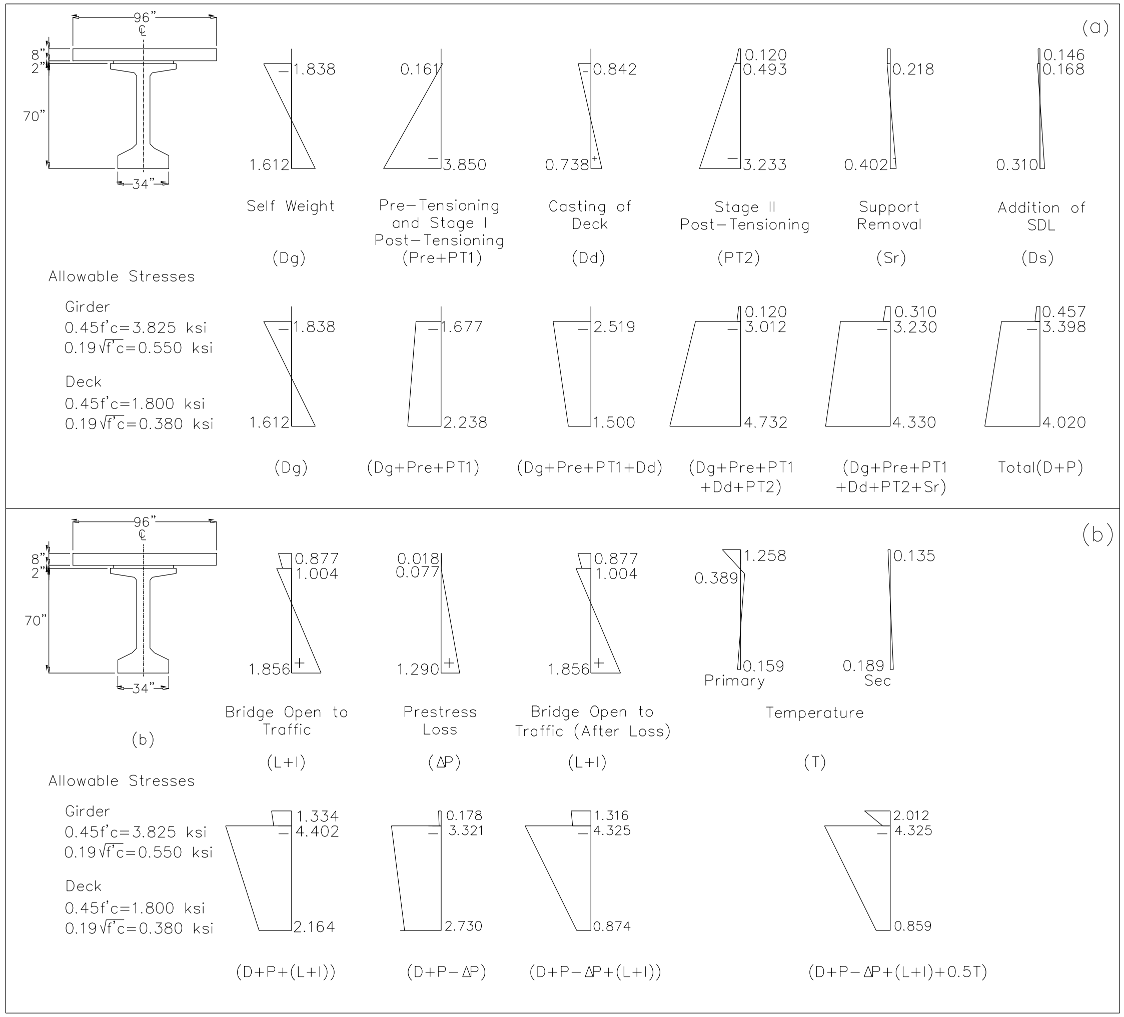


Figure 4.13. Stress Check at Section A-A for (a) Construction and (b) In-service Before and After Losses for Shored Construction.

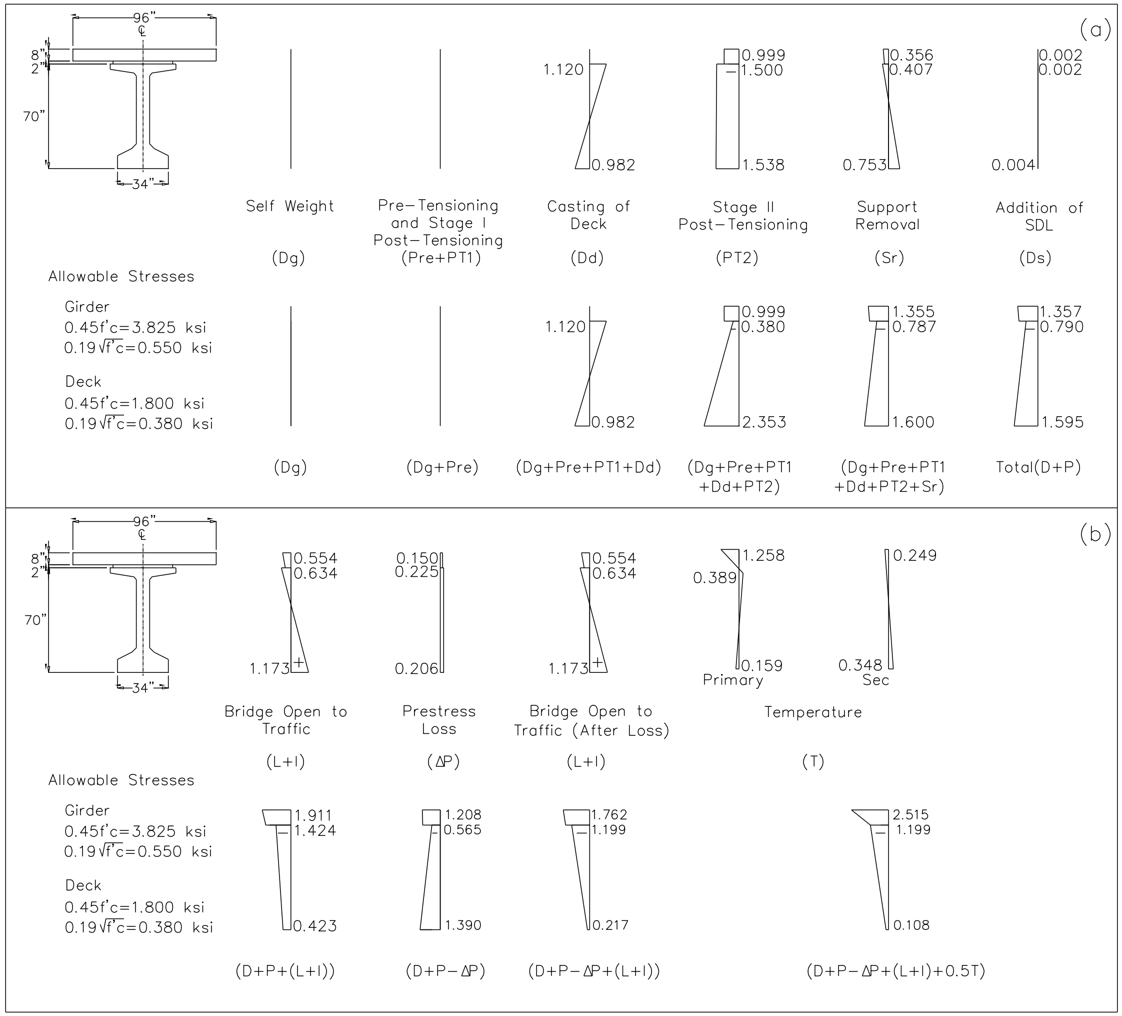


Figure 4.14. Stress Check at Section B-B for (a) Construction and (b) In-service Before and After Losses for Shored Construction.

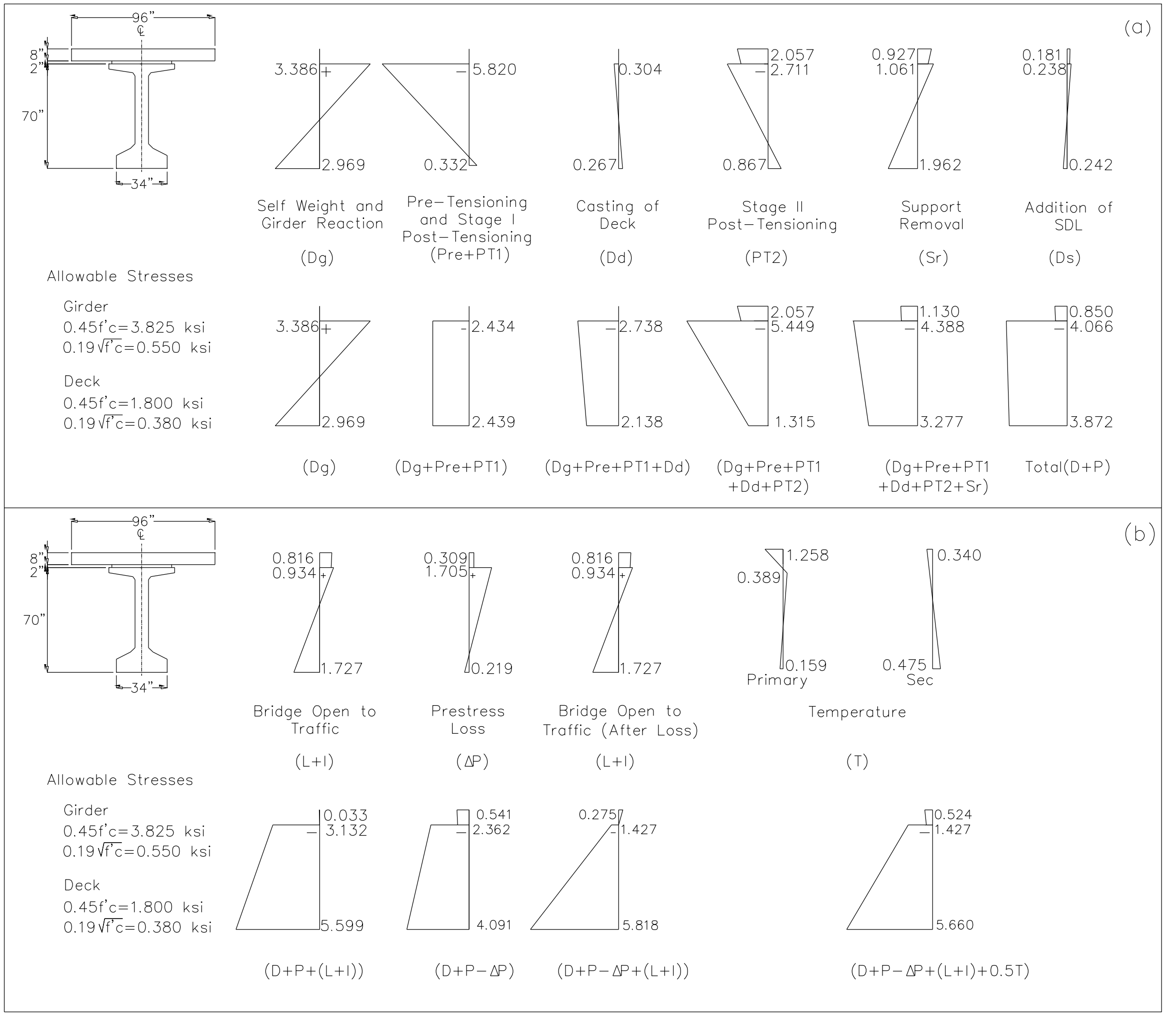


Figure 4.15. Stress Check at Section C-C for (a) Construction and (b) In-service Before and After Losses for Shored Construction.

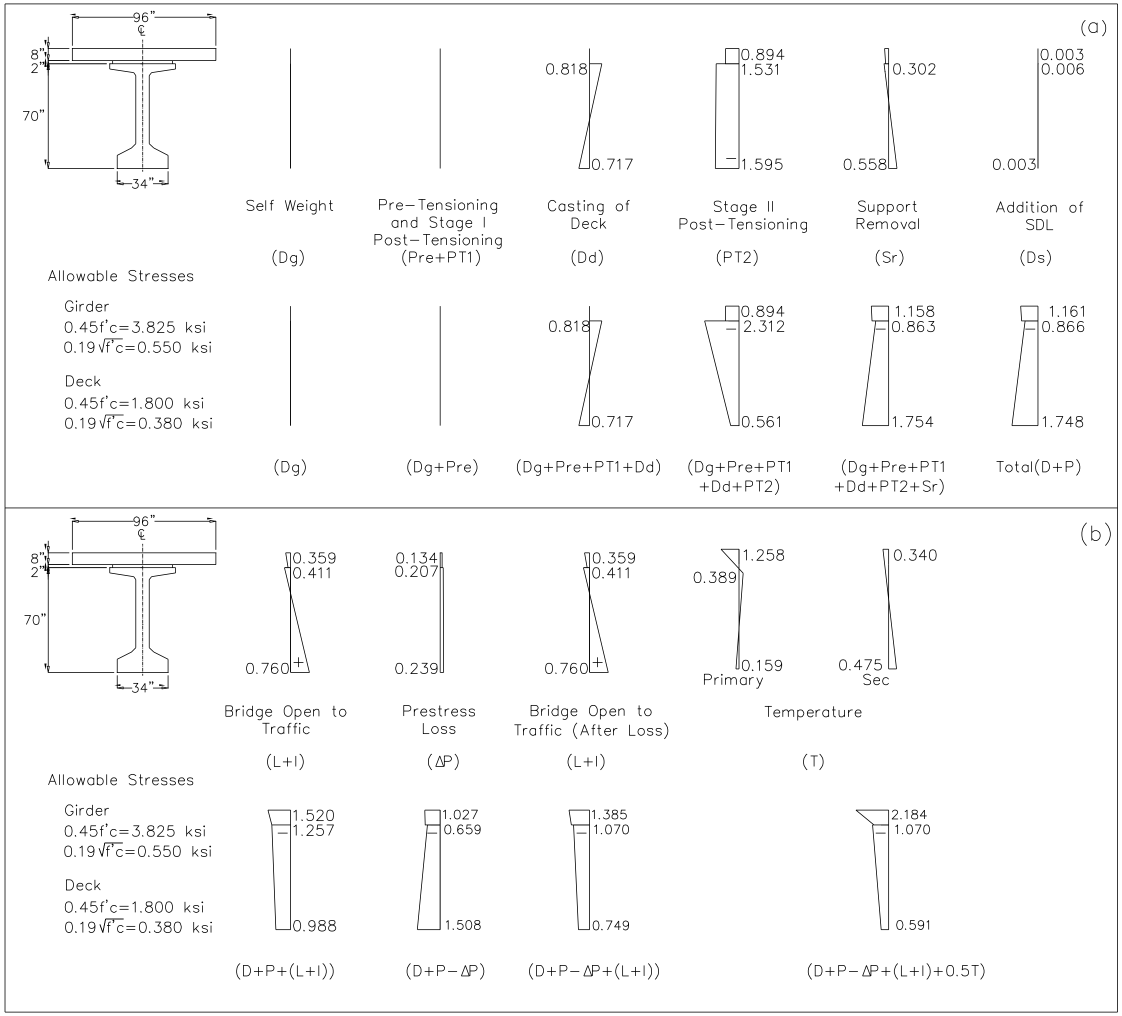


Figure 4.16. Stress Check at Section D-D for (a) Construction and (b) In-service Before and After Losses for Shored Construction.

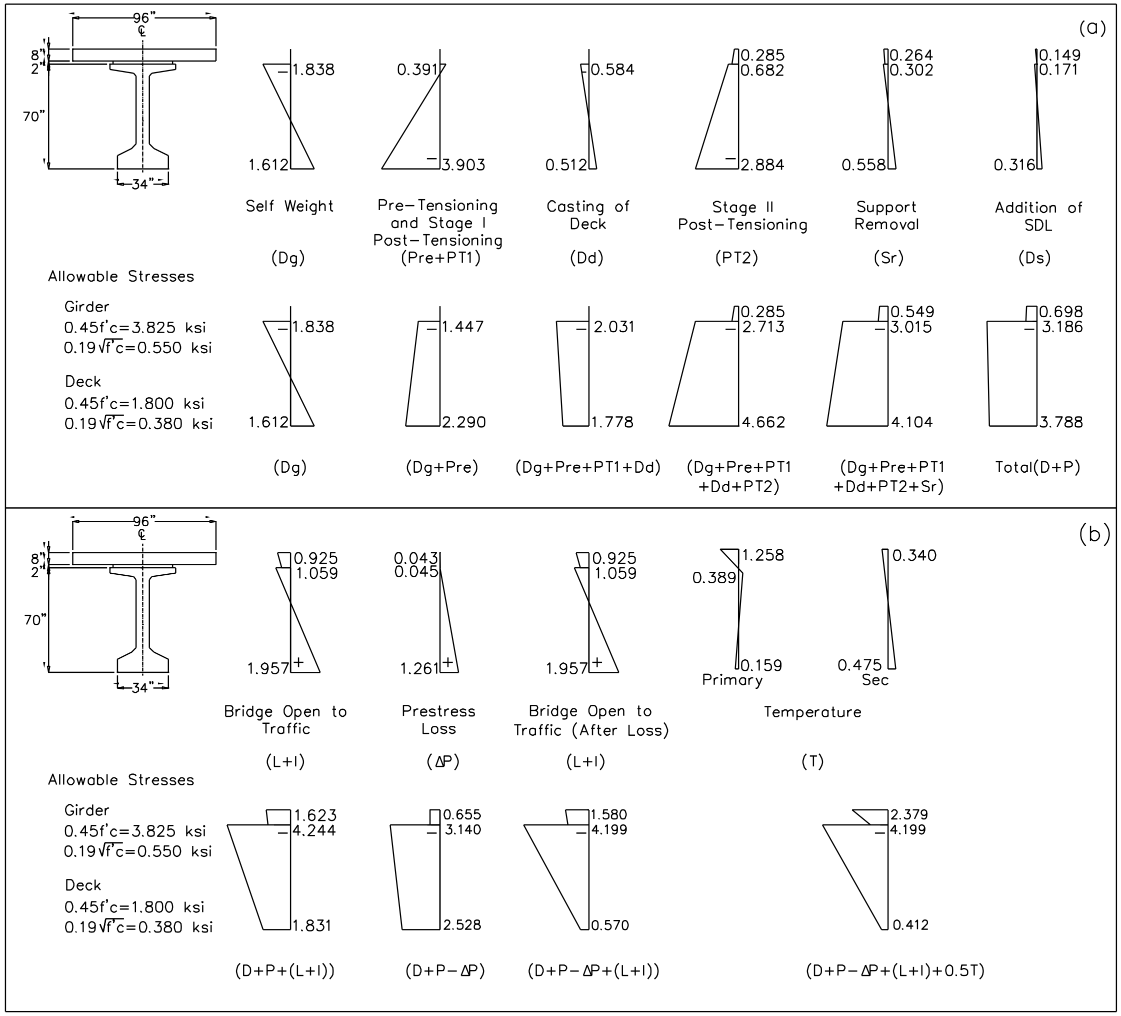


Figure 4.17. Stress Check at Section E-E for (a) Construction and (b) In-service Before and After Losses for Shored Construction.

Table 4.7. Girder Stresses at Various Sections for Shored Construction.

Loading	Component	Location	Section					Allowable Stress Limits	
			A-A (End Segment)	B-B (Splice Exterior)	C-C (Pier)	D-D (Splice Interior)	E-E (Drop-in segment)	Compression	Tension
Step I (Before Loss)	Girder	Top	-1.677	-	-2.434	-	-1.447	-3.825	+0.550
		Bot	-2.238	-	-2.449	-	-2.290		
Step II (Before Loss)	Girder	Top	-2.519	**+1.120**	-2.738	**+0.818**	-2.031	-3.825	+0.550
		Bot	-1.500	-0.982	-2.183	-0.717	-1.778		
Step III (After Loss)	Girder	Top	-3.321	-0.565	-2.362	-0.659	-3.140	-3.825	+0.550
		Bot	-2.730	-1.390	-4.091	-1.508	-2.528		
	Deck	Top	-0.439	-1.208	-0.541	-1.027	-0.655	-2.400	+0.380
		Bot	-0.531	-1.112	-0.608	-0.975	-0.694		
Service (After Loss)	Girder	Top	-4.325	-1.199	-1.427	-1.070	-4.199	-5.100	+0.550
		Bot	-0.874	-0.217	**-5.818**	-0.481	-0.570		
	Deck	Top	-1.316	-1.762	+0.275	-1.385	-1.580	-2.400	+0.380
		Bot	-1.194	-1.531	+0.009	-1.246	-1.393		

Note: Bold values indicate allowable stress limit is exceeded.

The splice region of the beam experienced tensile stresses that exceeded the allowable tensile stresses at service condition during the stage when the deck is poured. This stress exceedance is addressed by providing supplemental mild steel reinforcement. However, the stresses are brought within limits when the Stage II post-tensioning operation is carried out.

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. This stress exceedance is addressed by providing supplemental mild steel reinforcement in the compression zone. For this design, 16-#14 bars and 4 Dywidag 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 compression reinforcement in the girder soffit at the interior support over the pier for the computed stress exceedance at service load conditions.

The pier region of the deck experienced tensile stresses at service condition. However, these tensile stresses are within the allowable tensile stress limits.

4.8 DEFLECTION CHECK

The girders are to be checked for allowable deflection under live load and impact as specified in AASHTO LRFD Specifications Article 2.5.2.6.2 (AASHTO 2012). Composite section properties are used in computing these deflections that occur in service. According to AASHTO LRFD Specifications Article 3.6.1.3.2 (AASHTO 2012), the deflection is calculated as the larger of:

1. Design Truck alone, or
2. 25 percent of Design Truck Load and full Design Lane Load.

The design truck load is multiplied by the dynamic load amplification factor to compute deflections. The limit for maximum deflection, as specified in the AASHTO LRFD Specifications (2012) Article 2.5.2.6.2, is given by $L/800$. Table 4.8 gives the allowable and actual deflection results for the three-span bridge. The deflections are observed to be within limits.

Table 4.8. Live Load Deflections for Three-span Continuous Bridge Using Shored Construction.

Deflection	**Exterior Span**	**Interior Span**
Allowable (in.)	2.85	3.60
Actual (in.)	1.21	1.34

4.9 ULTIMATE STRENGTH CHECK

The strength limit state needs to be checked to ensure safety at 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, expressed as follows.

$$M_u \leq \Phi M_n \tag{4.2}$$

where,

M_u = Factored ultimate moment at a section, kip-ft.

ΦM_n = Nominal moment strength at a section, kip-ft.

Φ = 1.0 for flexure and tension of prestressed concrete members.

The total factored moment at ultimate according to AASHTO LRFD Bridge Design Specification is given by,

$$M_u = 1.25\ (M_{DC}) + 1.5\ (M_{DW}) + 1.75(M_{LL+IM}) \tag{4.3}$$

where,

M_{dc} = Bending moment due to all dead loads, kip-ft.

M_{dw} = Bending moment due to wearing surface load, kip-ft.

M_{LL+IM}= Bending moment due to live load and impact, kip-ft.

The moment capacity and demand is checked at the following points: (1) at 0.35 L of the end span, (2) at the face of pier, and (3) at the midspan of center span. The moment capacity at ultimate depends on the number of strands, diameter of strands, stress in the stands, design strength of concrete, deck reinforcement and the cross-section properties of the girder. The deck reinforcement consists of 11-#5 bars provided in the top of the deck and 11-#4 bars provided in the bottom of the deck. This is based on recommendation by TxDOT which is the typical deck reinforcement provided for steel bridges. Table 4.9 gives the moment demand and capacity for the three-span bridge. It is observed that the capacity is greater than demand.

Table 4.9. Ultimate Demand and Capacity for Three-Span Bridge Using Shored Construction.

Capacity and Demand	End Span	Over Pier	Interior Span
Demand, M_u (kip-ft)	14,950	20,690	15,340
Capacity, ϕM_n (kip-ft)	22,780	24,180	24,430

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. For this design, 16-#14 bars and 4 Dywidag bars 1.25 in. diameter are added in the bottom flange of the girder to provide the additional capacity and meet the moment demand at the interior support over the pier. The mild steel reinforcement provided in the bottom flange acts as compression steel.

4.10 SHEAR DESIGN

Modified compression field theory (MCFT) is used for transverse shear design as specified in the *AASHTO LRFD Bridge Design Specifications* (2012). The MCFT takes into consideration the combined effect of axial load, flexure and prestressing when designing for shear. Figure 4.18 shows the maximum factored shear demand and the reduced nominal shear capacity to resist the maximum demand. Figure 4.19 shows the details of the shear reinforcement selected to meet the design requirements, which includes the following.

- # 5 double legged stirrups at a spacing of 4 in. are provided for a distance of 10 ft from the anchorage end for the end-segment. # 5 double legged stirrups at a spacing of 6 in. are provided for the next 10 ft, and # 5 double legged stirrups at a spacing of 12 in. are provided in the remaining portion.
- # 5 double legged stirrups at a spacing of 6 in. are provided for a distance of 29 ft and 24 ft from the ends of the pier segment. # 5 double legged stirrups at a spacing of 4 in. are provided in the remaining portion.

- # 5 double legged stirrups at a spacing of 6 in. are provided for a distance of 20 ft from the ends of drop-in segment and # 5 double legged stirrups at a spacing of 12 in. are provided in the remaining portion.

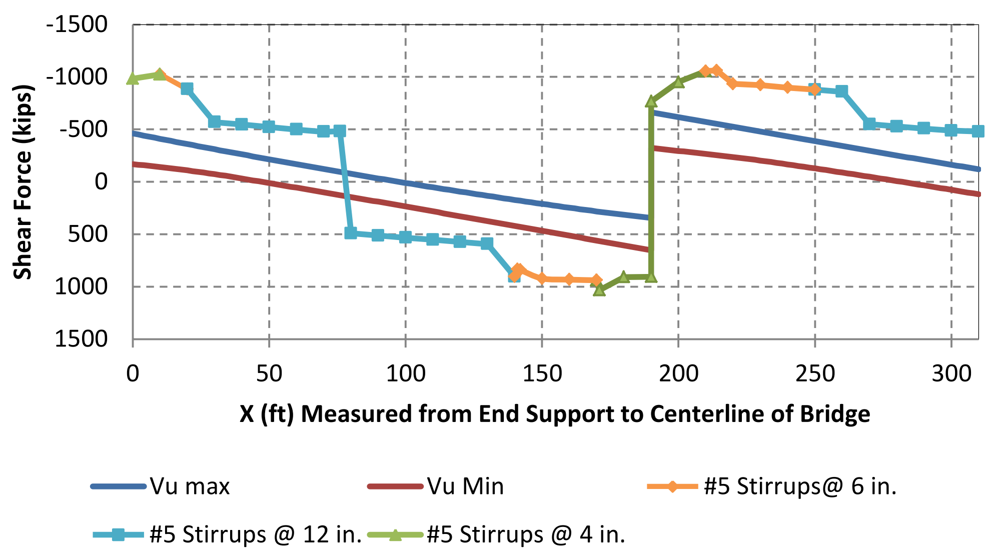


Figure 4.18. Transverse Shear Demand and Capacity for Three-Span Continuous Bridge Using Shored Construction.

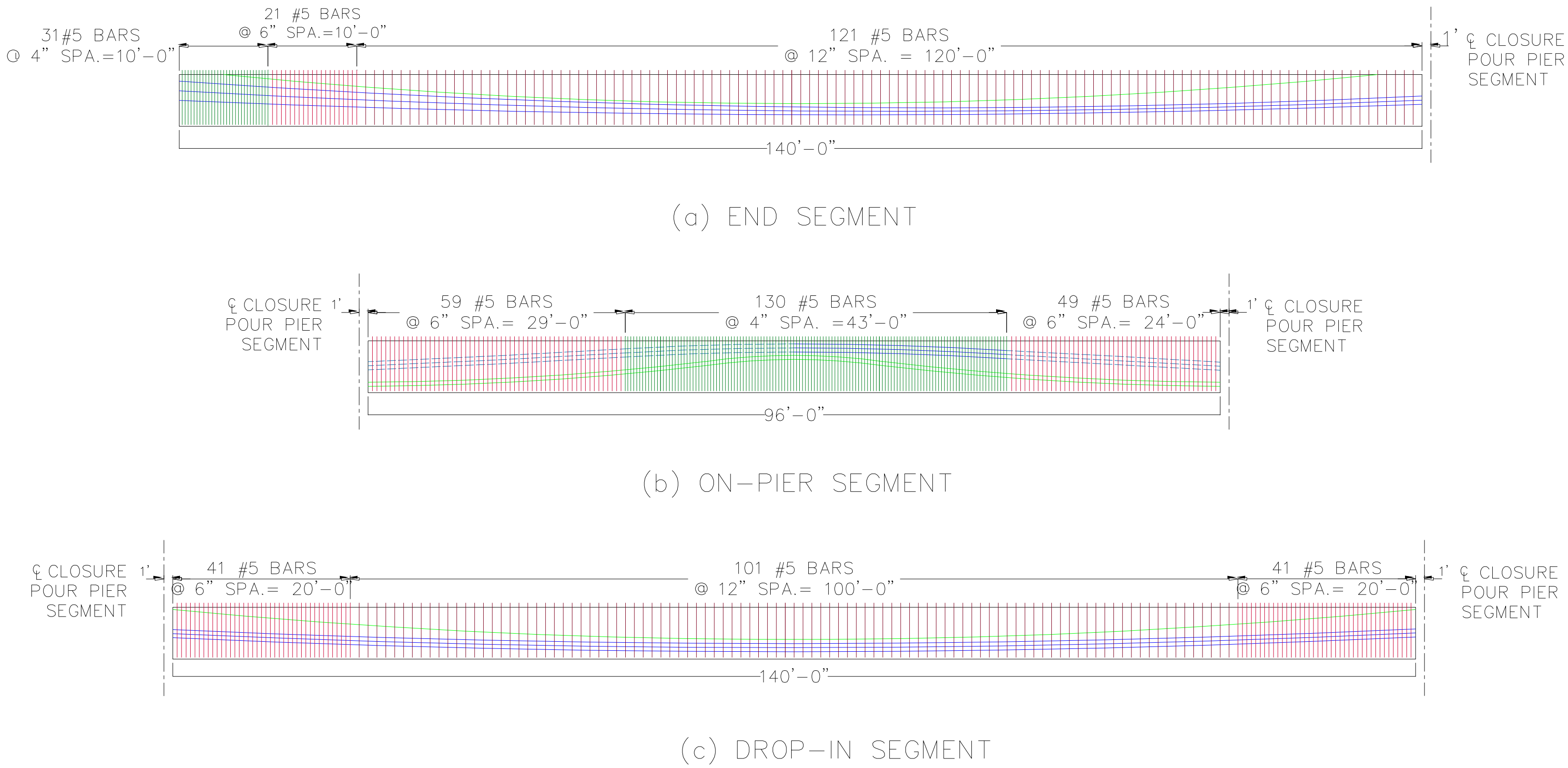


Figure 4.19. Shear Design Details - Elevation View for Three-Span Continuous Bridge Using Shored Construction.

5. CASE STUDY 2 - PARTIALLY SHORED CONSTRUCTION

5.1 INTRODUCTION

The following example gives the details for design of a three-span continuous prestressed concrete girder bridge using partially shored construction. A modified Tx70 girder section has been used for this bridge. The design is based on the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

5.2 BRIDGE DESCRIPTION

The bridge shown in Figure 5.1 represents a typical three-span continuous prestressed concrete bridge. The length of the drop-in and end girder segments is 140 ft and that of the on-pier segments is 96 ft. The end spans are 190 ft and the center span is 240 ft in length. The ratio of end span to center span is 0.8. The width of the splice is 2 ft. Prismatic modified Tx70 girders with a 9 in. web width are used for the end and drop-in girder segments. Constant web depth haunched girders are used for the on-pier segments. The depth of these haunched girders varies from 70 in. at the ends to a maximum depth of 108 in. at the centerline of the pier.

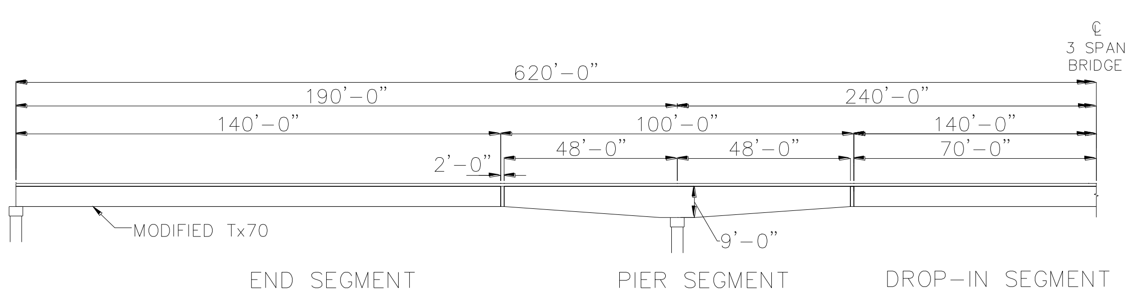


Figure 5.1. Elevation View of Three-Span Continuous Bridge for Partially Shored Construction.

5.3 BRIDGE GEOMETRY AND GIRDER CROSS-SECTION

The bridge cross-sections at midspan and at centerline of the pier are shown in Figure 5.2 and Figure 5.3, respectively. The bridge has a total width of 46 ft and a total roadway width of 44 ft. The bridge superstructure consists of six modified Tx70 girders spaced 8 ft center-to-center, with 3 ft overhangs on each side designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete deck. The wearing surface thickness is 2 in., which includes the thickness of any future wearing surface. TxDOT standard T501 type rails are considered in the design. Three design lanes are considered for the purpose of design in accordance with the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

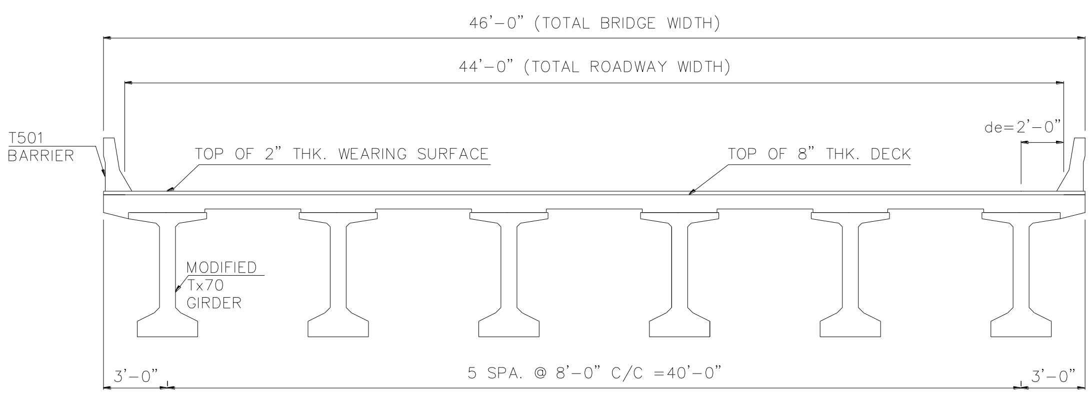


Figure 5.2. Transverse Bridge Section at Midspan for Partially Shored Construction.

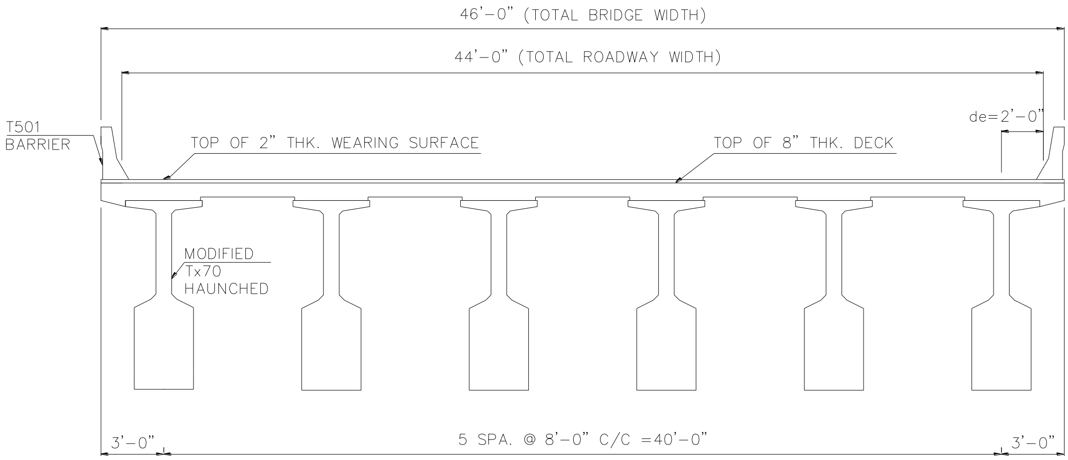


Figure 5.3. Transverse Bridge Section at Centerline of Pier for Partially Shored Construction.

A modified Tx70 girder has been considered for the design. The web width of the standard Tx70 girder has been increased to 9 in. to allow placement of post-tensioning ducts. This results in an increase in width of the top flange to 44 in. and that of the bottom flange to 34 in. Table 5.1 provides the composite and non-composite section properties for the prismatic modified Tx70.

For the haunched girder, a constant web depth haunch is provided below the prismatic modified Tx70 girder. The thickness of the bottom flange of the modified Tx70 girder is increased by 38 in. Table 5.2 gives the composite and non-composite section properties for the haunched modified Tx70.

Table 5.1. Section Properties for Prismatic Modified Tx70 Girder for Partially Shored Construction.

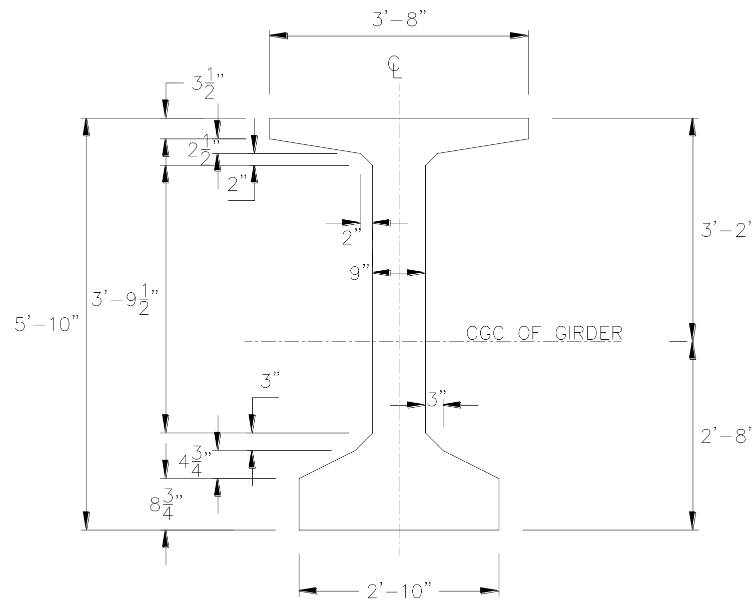
Girder Type	Depth of N.A. from top, y_t (in.)	Depth of N.A. from bottom, y_b (in.)	Area, A (in.^{2})	Moment of Inertia, I_x (in.^{4})
Modified Tx70	37.7	32.3	1106	687,110
Modified Tx70 Composite	32.7	47.3	1607	1,285,140

Table 5.2. Section Properties for Haunched Modified Tx70 Girder for Partially Shored Construction.

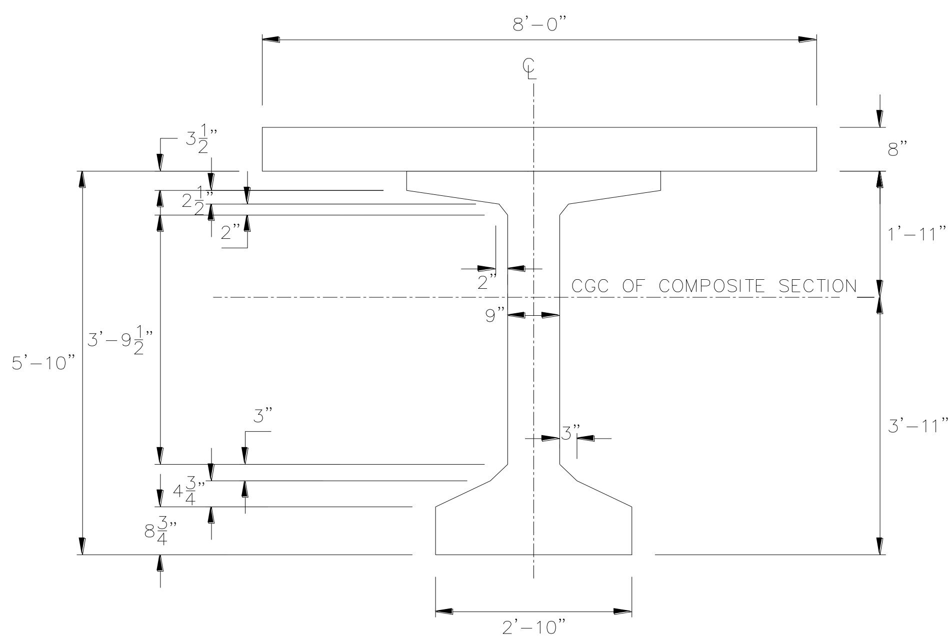
Girder Type	Depth of N.A. from top, y_t (in.)	Depth of N.A. from bottom, y_b (in.)	Area, A (in.^{2})	Moment of Inertia, I_x (in.^{4})
Modified Tx70 Haunched	65.6	42.4	2398	2,435,340
Modified Tx70 Haunched Composite	61.5	54.5	2899	4,420,290

Figure 5.4 shows the details of the non-composite and composite section for the prismatic modified Tx70 girder, respectively.

Figure 5.5 shows the details of the non-composite and composite section for the haunched modified Tx70 girder, respectively.

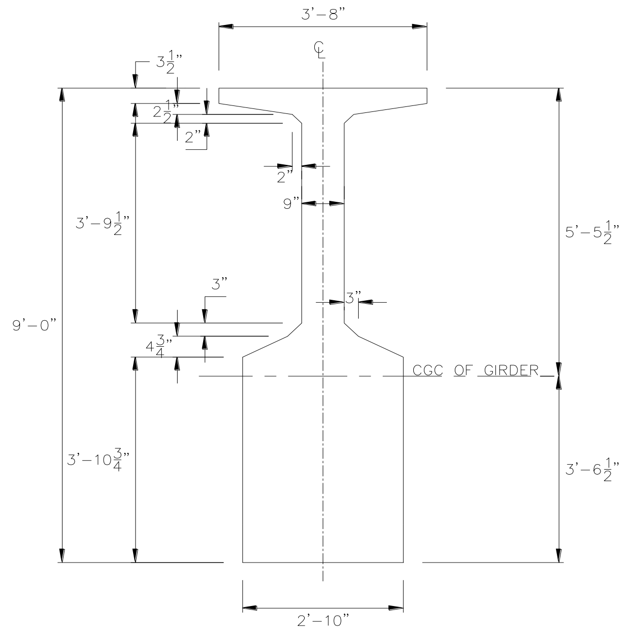


(a) Non-composite Section

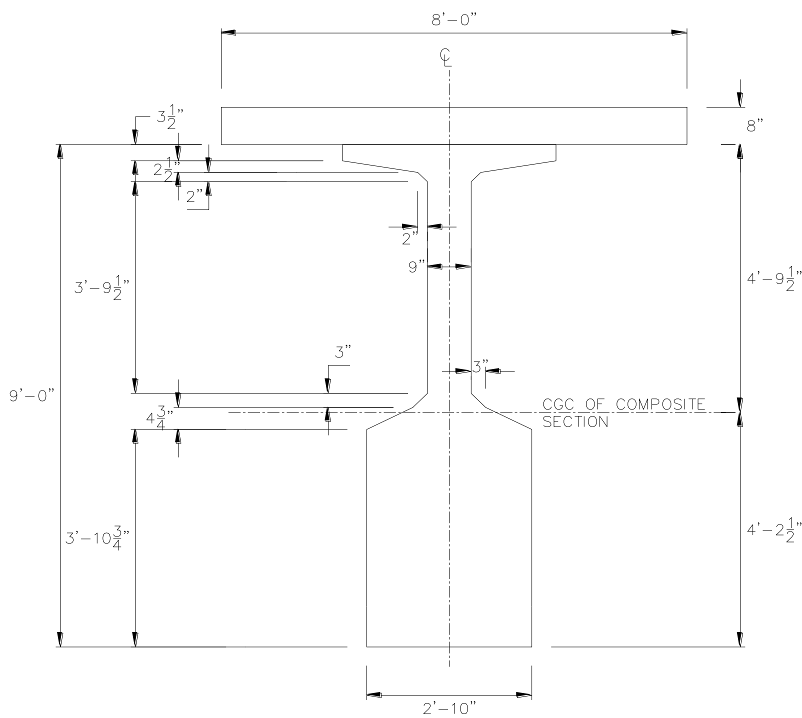


(b) Composite Section

Figure 5.4. Prismatic Modified Tx70 Girder for Partially Shored Construction.



(a) Non-Composite Section



(b) Composite Section.

Figure 5.5. Haunched Modified Tx70 Girder for Partially Shored Construction.

5.4 DESIGN PHILOSOPHY

5.4.1 General

The load balancing technique has been used for the design of the continuous Tx70 prestressed concrete bridge girders. A two stage post-tensioning approach is used. Stage I post-tensioning is applied to girders to balance the self-weight. Then, Stage II post-tensioning is carried out to balance the deck weight and superimposed dead load.

5.4.2 Handling and Transportation

5.4.2.1 Overview

The drop-in and end segments are transported from the precast plant to the construction site while supporting their ends. The girder segments are pre-tensioned for self-weight during handling and transportation. Figure 5.6 shows the support configuration during transportation of the drop-in and end segments.

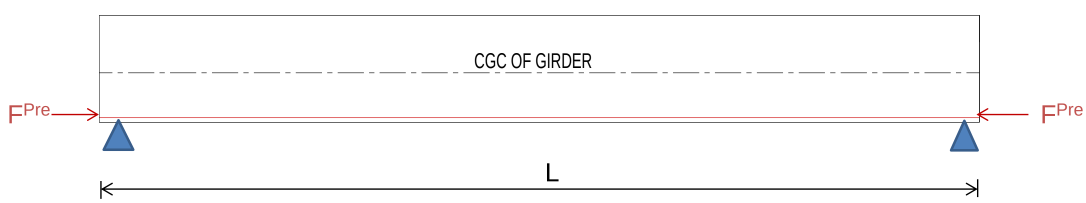


Figure 5.6. Support Arrangement During Transportation of Drop-in and End Segments for Partially Shored Construction.

The on-pier segment is transported by supporting at the quarter span points near the ends of the girder. A large amount of prestress force is required in the top flange of the on-pier segment because these segments cantilever from the piers and eventually support the ends of the drop-in and end segments before Stage I post-tensioning. The on-pier girder segment is pre-tensioned for self-weight and the girder reactions from the drop-in and end segments. Until the time that the pier segment supports the drop-in girder segment, the tension stresses in the bottom flange are high. This is offset by providing pre-tensioning in the bottom flange.

Figure 5.7 shows the details of transportation for the haunched girder segment.

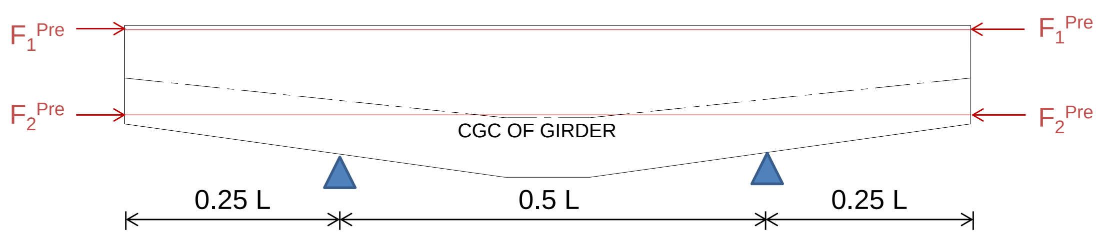


Figure 5.7. Support Arrangement During Transportation of On-Pier Segment for Partially Shored Construction.

The span length and weight limitations of girder segments are taken into consideration during handling and transportation. In the state of Texas, precasters recommend a maximum transportable segment length of 160 ft, a maximum weight of 200 kips and a maximum depth of 10 ft (Hueste et al. 2012). Table 5.3 gives the span lengths and weights for the girder segments considered in the design. It is observed that the segment lengths and weights are within transportation limits.

Table 5.3. Segment Lengths and Girder Weights for Partially Shored Construction.

Girder Segments	Length (ft)	Weight (kips)
End Segment	140	161
Drop-in Segment	140	161
On-Pier Segment	96	182
Limits in Texas	160	200

5.4.2.2 Pre-tensioning

For pre-tensioning of the girder segments, 0.6 in. diameter Grade 270 low relaxation strands with an ultimate tensile strength f_{pu} of 270 ksi are considered. The initial stress in pre-tensioning strands at transfer, f_{pi}, is considered to be 0.75 f_{pu} which is equal to 202.5 ksi. The force at transfer is calculated after taking the initial losses due to initial steel relaxation and elastic shortening into account. Prestress losses of 20 percent are assumed in the pre-tensioned strands at service. Table 5.4 presents the pre-tensioning strands design summary for the girder segments.

Table 5.4. Pre-tensioning Strand Design Summary for Partially Shored Construction.

Description	End Segment	On-Pier Segment		Drop-in Segment
	Bottom Flange	Top Flange	Bottom Flange	Bottom Flange
No. of Strands (0.6 in. dia.)	24	24	20	24
Prestress Force at Transfer (kips)	984	984	820	984
Prestress Force at Service (kips)	843	843	703	843

5.4.3 Construction on Site

5.4.3.1 Construction Sequence

After the girders are transported to the job site, the girders are lifted and placed on the piers and temporary shoring towers. A two stage post-tensioning approach is used. Stage I post-tensioning ensures that the girders are balanced for self-weight and provides continuity. Then Stage II post-tensioning is carried out after the deck is cast to balance the weight of the deck and to provide compression in the deck. Figure 5.8 shows the details of various stages of construction. The step-by-step construction procedure is as follows.

(a) Erect piers, temporary supports and abutments. Set on-pier girder segments on the piers and secure the girder segments to the temporary shoring towers located at A and D in the end spans.

(b) Attach strongbacks to the ends of the end segments at the ground level. Erect the end girder segment on the abutment and shoring tower. The shoring tower should be capable of transferring the reaction from the end girder segment to the foundation below. Connect the strongbacks from the end segment to the on-pier girder segment.

(c) Attach the strongbacks to the ends of the drop-in girder segment at the ground level. Erect the drop-in-girder segment by connecting the strongbacks to the on-pier girder segment. It is necessary to ensure that the end girder segments are installed prior to this step. This ensures minimizes uplift caused by the erection of the drop-in-girder segment. Tie-downs could also be used to prevent the uplift. After all the girder segments have been placed and connected, check the vertical alignment of the girder ends. Strongbacks help in maintaining the vertical alignment of the adjacent girders prior to threading the post-tensioning strands through the ducts. Then, thread the post-tensioning strands through the ducts in the web of the girders.

(d) Cast the splice in between the girder segments. Once the splice has cured and gained sufficient strength, stress the Stage I post-tensioning tendons to provide continuity between the girder segments and then grout the tendons. Remove the strongbacks and the temporary shoring towers located at A and D.

(e) Construct the formwork for the deck and place the precast deck panels and deck reinforcement. Pour the concrete for the deck.

(f) After the deck has cured and gained sufficient strength, stress the Stage II post-tensioning and grout the tendons.

(g) Cast the barriers and wearing surface. After a suitable time interval, the bridge is opened to traffic.

5.4.3.1 Stage I and Stage II Post-Tensioning

The Stage I and Stage II of post-tensioning is provided continuously to the entire bridge. The details of parameters used for post-tensioning are outlined in Table 3.1. The force at transfer is calculated after taking the initial losses due to elastic shortening, anchor set and friction into account. Long term prestress losses due to steel relaxation, creep and shrinkage are assumed equal to 20 percent for the Stage I post-tensioning and 15 percent for the Stage II post-tensioning at service, respectively. It is observed from the design that service stresses during various stages of construction may control the amount of post-tensioning provided. A higher amount of stage II post-tensioning is desirable to apply compression in the deck for service. Stage II post-tensioning is carried out after the deck is cast and helps to provide compression in the deck at service. This reduces the amount of cracking in the deck in the pier region due to the effect of live loads. Table 5.5 presents the post-tensioning design summary for the girder segments.

Table 5.5. Post-tensioning Design Summary for Partially Shored Construction.

Description	**Stage I Post-Tensioning**	**Stage II Post-Tensioning**
No. of Strands (0.6 in. dia.)	32 (2 ducts)	30 (2 ducts)
Prestress Force at Transfer (kips)	1312	1230
Prestress Force at Service (kips)	1049	1045

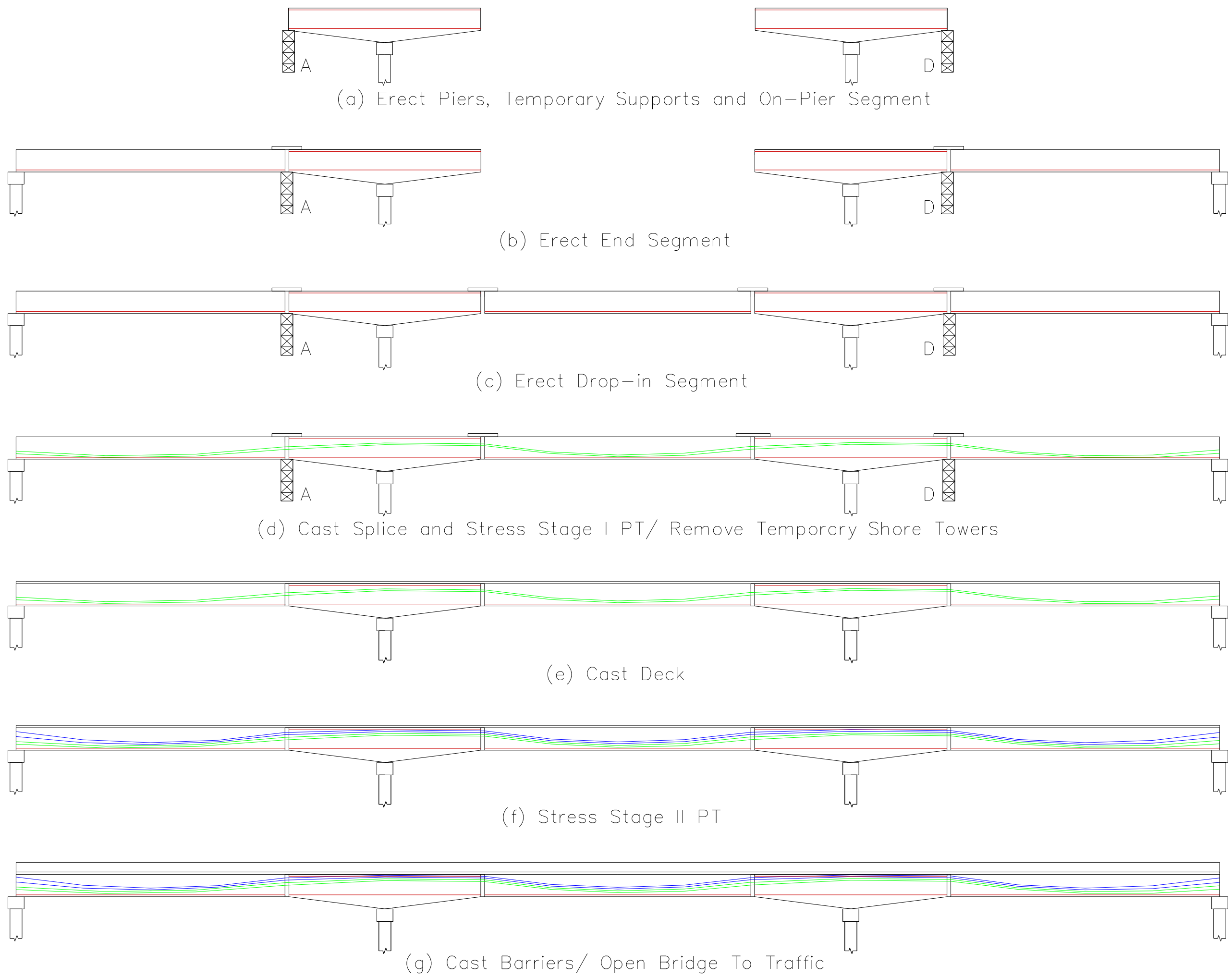
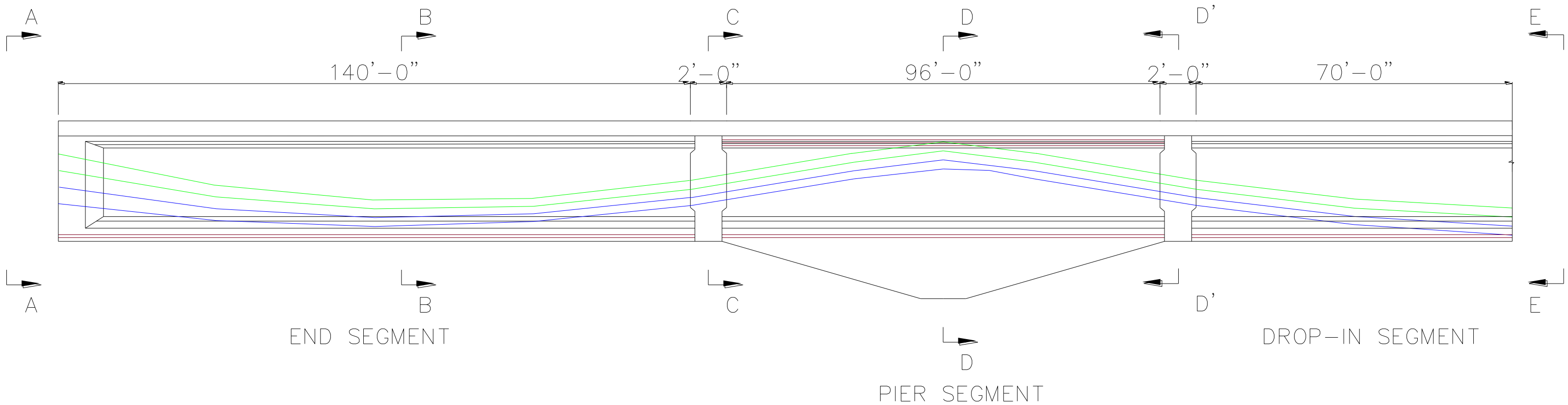


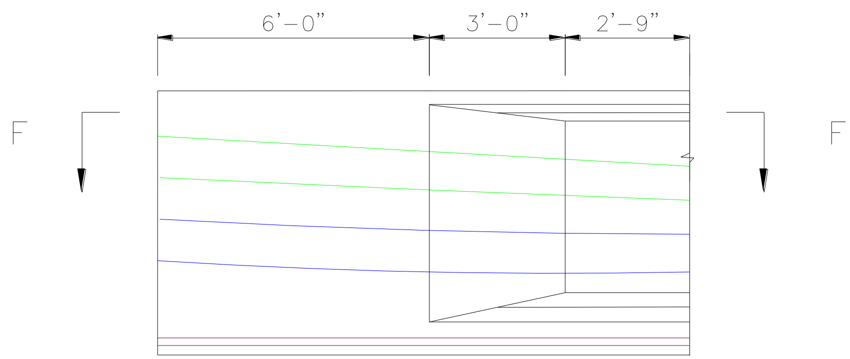
Figure 5.8. Stages of Construction for Partially Shored Construction.

5.5 PRESTRESSING LAYOUT

Figure 5.9 shows the prestressing details for the girder segments at the anchorage end in the end span (Section A-A), at 0.35*L* from the abutment support of the end span (Section B-B), at the end span splice (Section C-C), at the face of the pier (Section D-D), at the interior span splice (Section D'-D'), at the midspan of interior span (Section E-E) and at the anchor zone of the end span (Section F-F). An option of anchoring the post-tensioning ducts in the end span is shown in Section A-A. Thickened ends are required for anchoring the post-tensioning ducts at the ends of the girder segments. Thickened end blocks are provided for a distance equal to depth of the girder and then gradually tapered to the thickness of the web as shown in the plan view in Section F-F. In order to anchor all the post-tensioning ducts on the vertical face of the girder, the post-tensioning ducts are staggered which deviates them from the vertical plane as shown in the plan view in Section F-F. Thickened ends are provided for a distance of 6 ft and a transition zone of 3 ft is provided where the thickness gradually decreases to the web width of the girder as shown in the plan view in Section F-F. Figure 5.10 shows details of the post-tensioning layout for the three-span bridge.

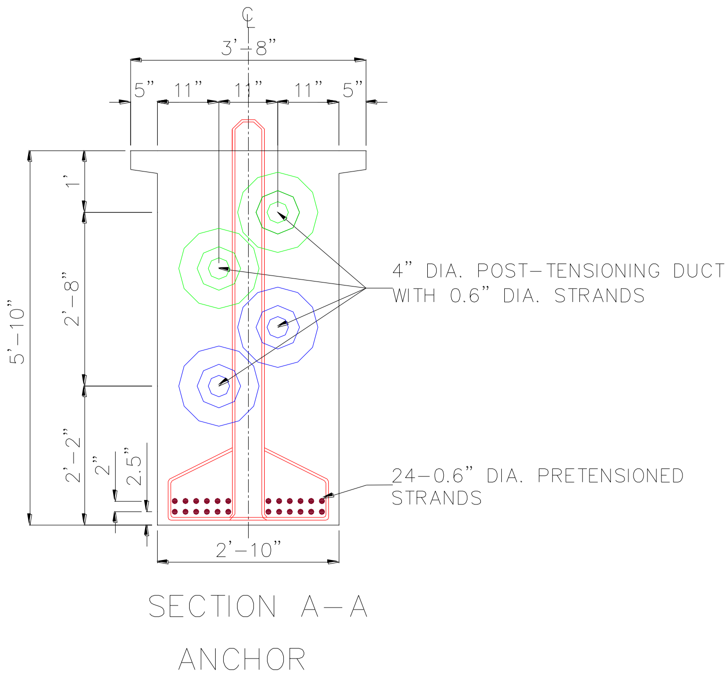


(a) Elevation View

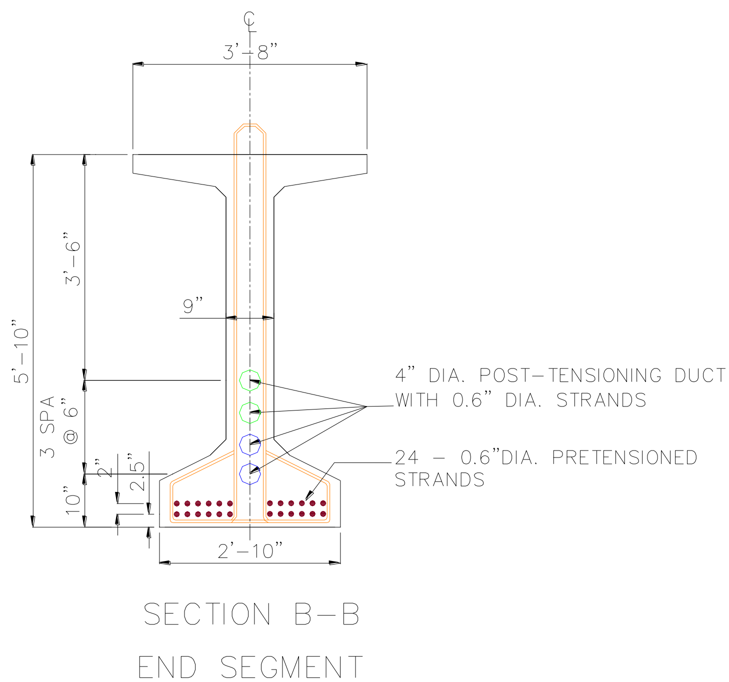


(b) Elevation View at Anchor Zone in End Span

Figure 5.9. Prestressing Details for Continuous Prestressed Concrete Modified Tx70 Girder Bridge Using Partially Shored Construction.

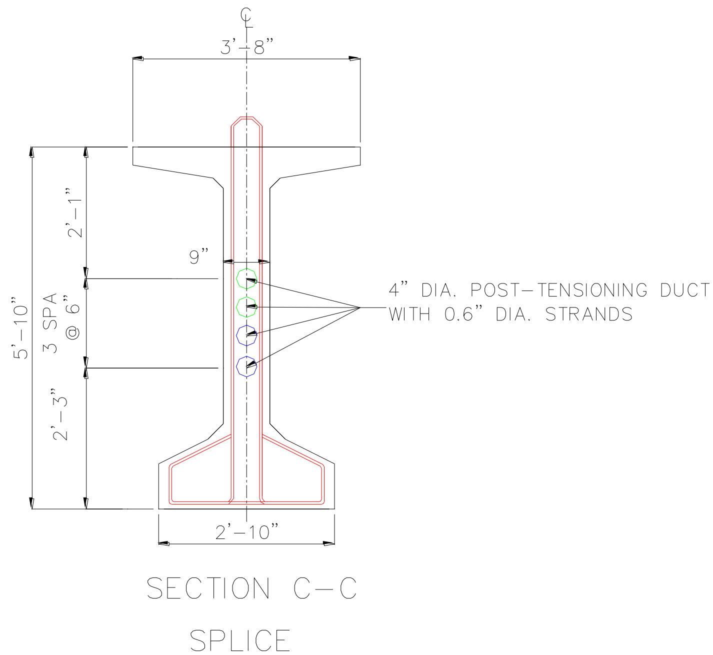


(c) Section B-B at Anchor

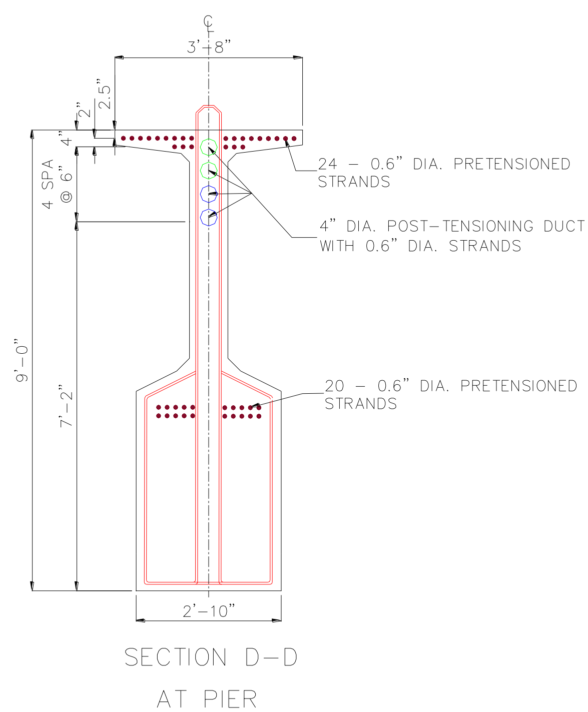


(d) Section B-B at End Segment

Figure 5.9. Continued.

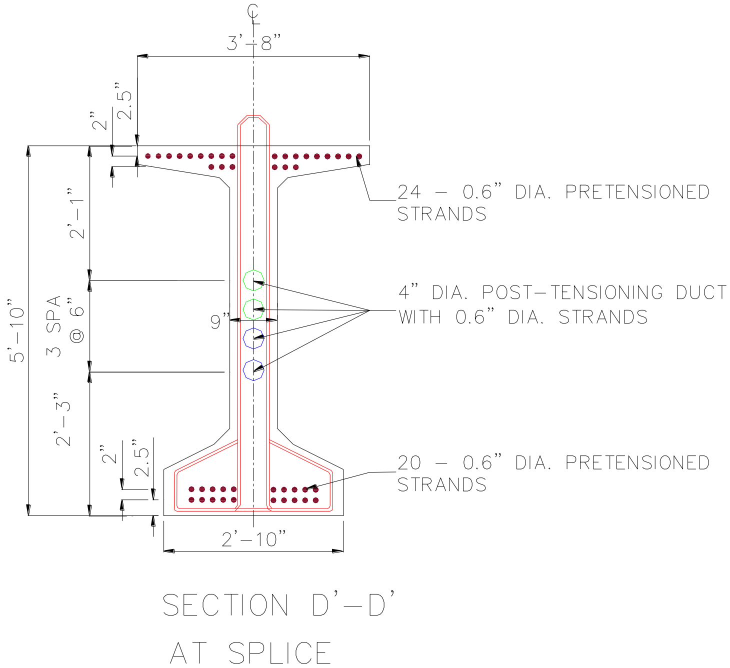


(e) Section C-C at Splice

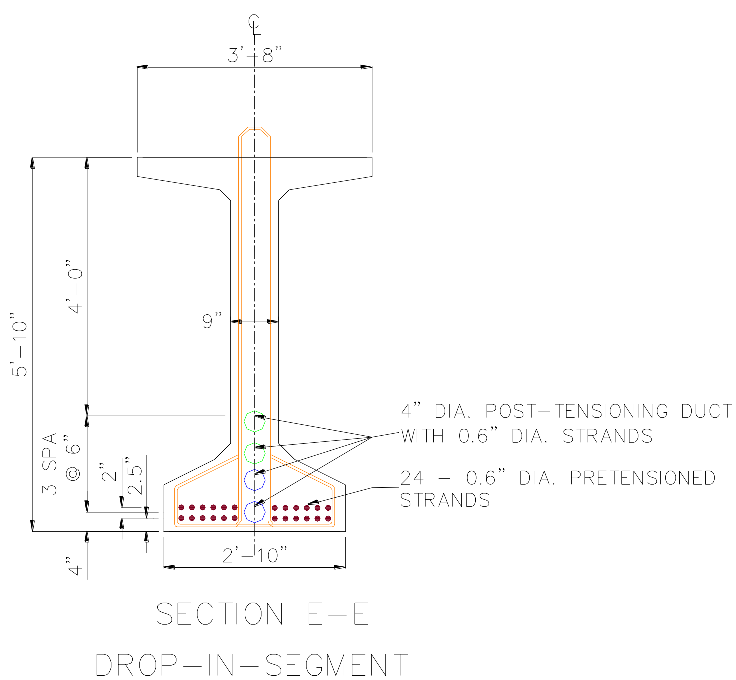


(f) Section D-D at Pier

Figure 5.9. Continued.

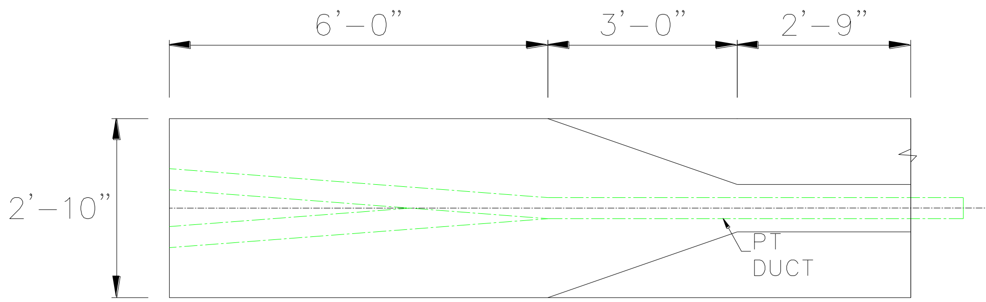


(g) Section D'-D' at Splice



(h) *Section E-E at Drop-In Segment*

Figure 5.9. Continued.



SECTION F–F

ANCHOR ZONE

(i) Plan View

Figure 5.9. Continued.

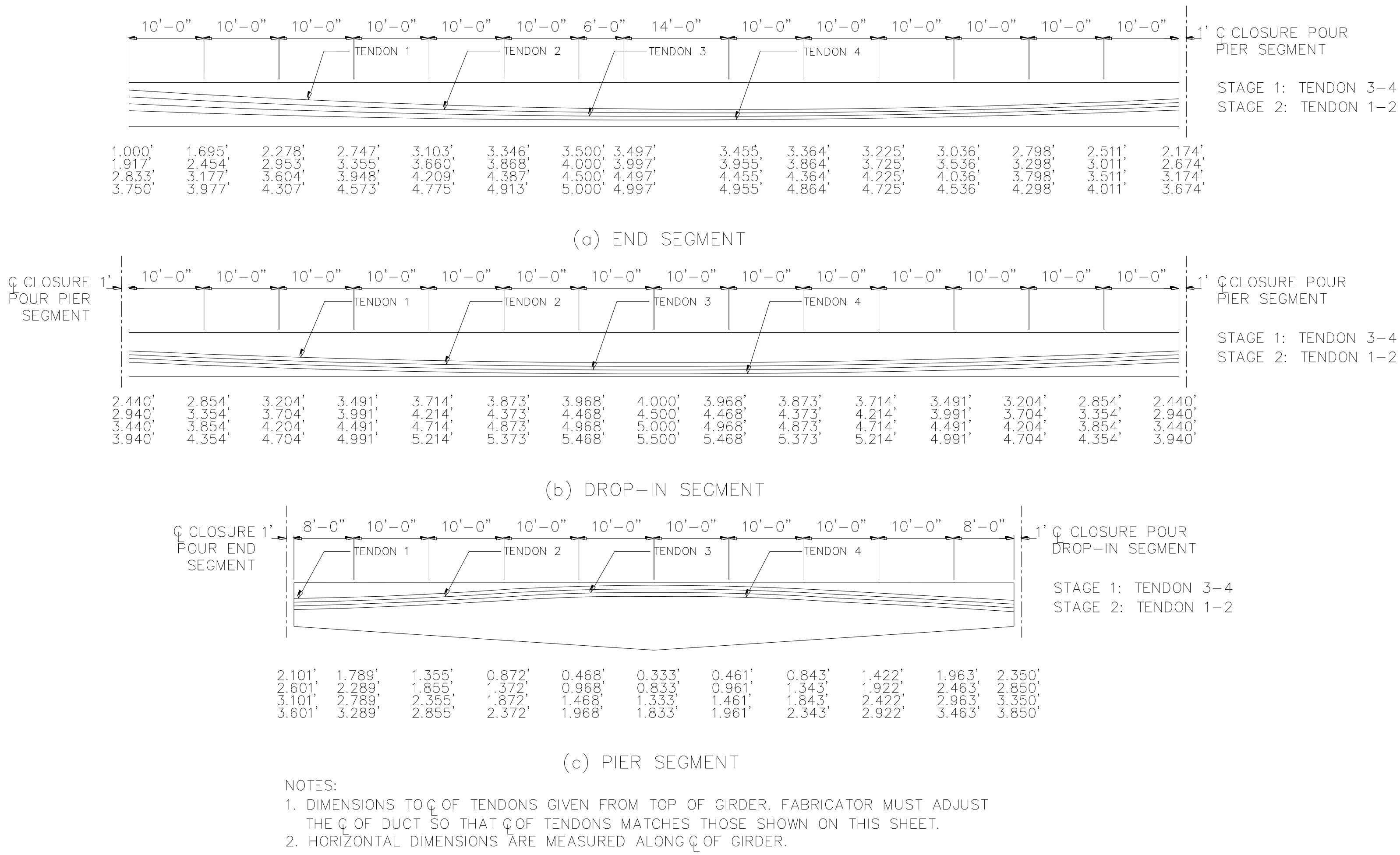


Figure 5.10. Post-Tensioning Layout for Continuous Prestressed Concrete Modified Tx70 Girder Bridge Using Partially Shored Construction.

5.6 MOMENTS DURING VARIOUS STAGES OF CONSTRUCTION

The moments during various stages of construction are computed at selected locations along the structure. The moments are computed at $0.35L$ from the abutment support at the end span (Section A-A), at the splice in the end span (Section B-B), at the face of the pier (Section C-C), at the interior span splice (Section D-D), and at the midspan of the interior span (Section E-E), as shown in Figure 5.11. The moments due to self-weight, pre-tensioning, Stage I post-tensioning and the wet CIP deck act on the non-composite girder section. The moments due to superimposed dead load and stage II post-tensioning act on the composite girder section. The moments due to prestressing are computed before losses. The girder moments at each section are summarized in Table 5.6. Figures 5.12 and 5.13 show the moments acting on the non-composite girder section and the composite girder section, respectively.

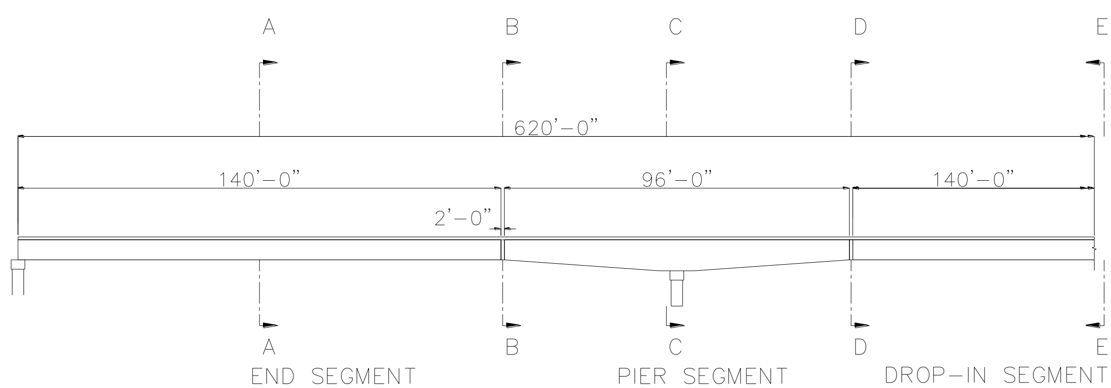
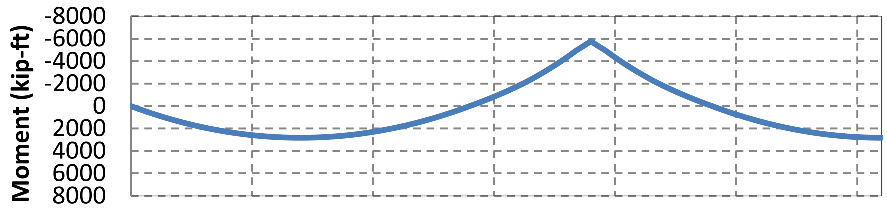


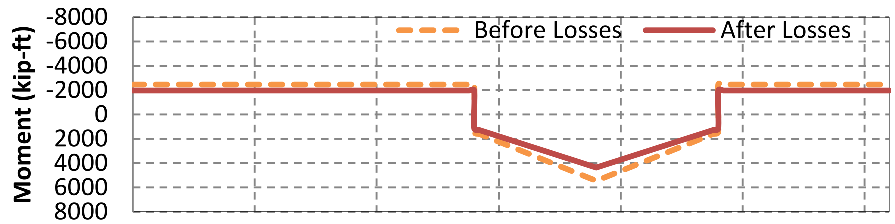
Figure 5.11. Section Locations for Moments for Three-Span Continuous Bridge Using Partially Shored Construction.

Table 5.6. Girder Moments at Various Sections for Partially Shored Construction.

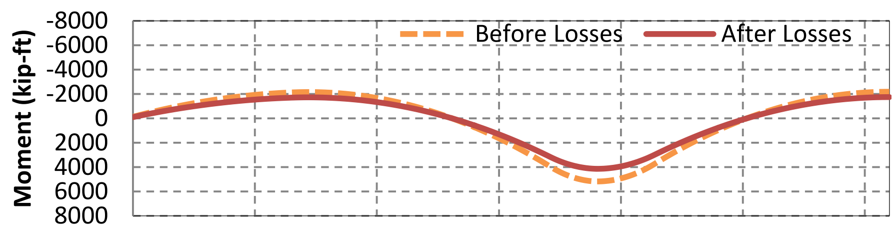
Loading	Section				
	A-A (End Segment)	B-B (Splice Exterior)	C-C (Pier)	D-D (Splice Interior)	E-E (Drop-in Segment)
Girder Self-Weight	2822	-	-1904	-	2822
Prestressing	-2460	-	5457	-	-2460
Reaction From Drop-in segment	-	-	-3871	-	-
Stage I Post-Tensioning	-2145	726	5176	952	-2181
Haunch and Deck	1905	-510	-4867	-692	1461
Stage II Post-Tensioning	-2557	863	6153	1119	-2607
Superimposed Dead Load	642	-172	-1639	-233	492
Live Load	5736	3228	-7488	2371	5627



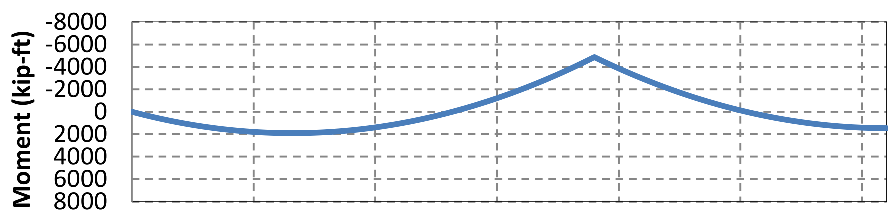
(a) Self-weight and Girder Reaction



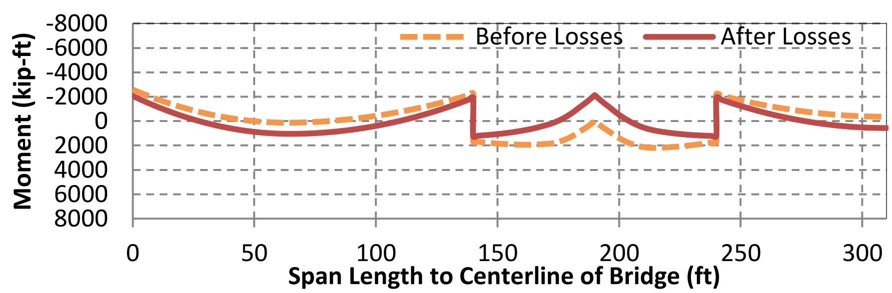
(b) Pre-Tensioning



(c) Stage I Post-Tensioning

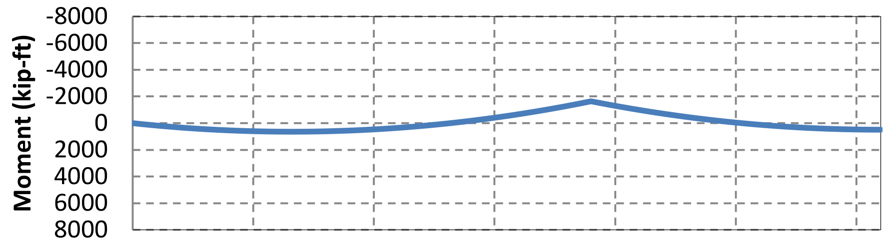


(d) Deck Weight

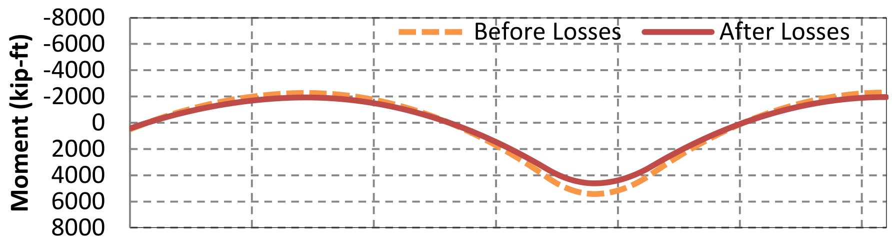


(e) Total Girder Moments

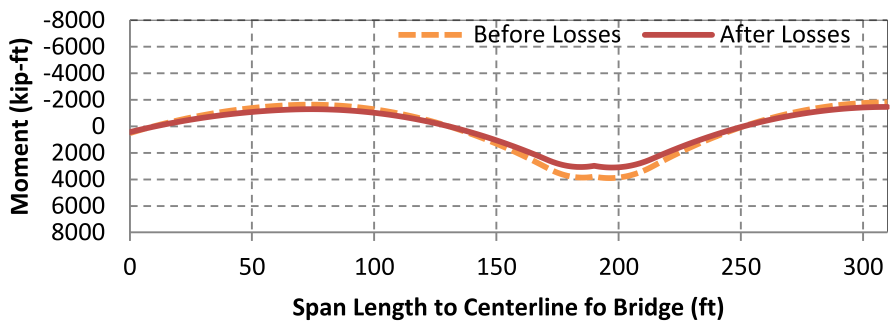
Figure 5.12. Moments Acting on Non-Composite Girder for Partially Shored Construction.



(a) Superimposed Dead Load



(b) Stage II Post-Tensioning



(c) Total Composite Moments

Figure 5.13. Moments Acting on Composite Girder for Partially Shored Construction.

5.7 SERVICE STRESS ANALYSIS

Service stress analysis is carried out under the effect of dead loads, prestress, live loads and temperature and thermal gradient. The stresses are checked at various steps of construction. The important construction steps for checking girder stresses are identified as follows:

- Step I: Girder segments supported on piers and temporary supports.
- Step II: Application of Stage I post-tensioning and casting deck.
- Step III: Application of Stage II Post-tensioning and casting barriers.
- Step IV: Bridge in service.

For the various stages of construction, stress checks are provided at the following points: (1) at 0.35*L* of the end span, (2) at the splice in the end span (3) at the face of the pier, (4) at the splice in the center span, and (5) at the midspan of the center span (see Figure 5.11). The allowable tension and compression limits at various stages of construction are provided in Table 3.4 and Table 3.5. Compression in prestressed concrete girders is evaluated through the Service I limit state while tension in prestressed concrete girders is evaluated through the Service III limit state.

Figures 5.14 through 5.18, present the stress blocks at each of these five locations. The stress blocks are obtained by adding the stress values due to the effect of various loads during the different steps of construction. The stress blocks are divided into two parts ‘Part a’ and ‘Part b’. ‘Part a’ shows the stresses during construction and ‘Part b’ shows the stresses during the service life of the bridge. ‘Part a’ and ‘Part b’ are further divided into two halves. The top half shows the stress values due to individual loads. The bottom half shows the cumulative effect of the stress values due to individual loads. The cumulative effect is obtained by adding the preceding cumulative value to the stress value due to the corresponding individual loads. The final stress value corresponding to cumulative effect of ‘Part a’ is carried over to ‘Part b’. Table 5.7 summarizes the stresses at each section. It is observed that the stresses are within allowable limits during all stages of construction and there is minimal tension stress in deck at service.

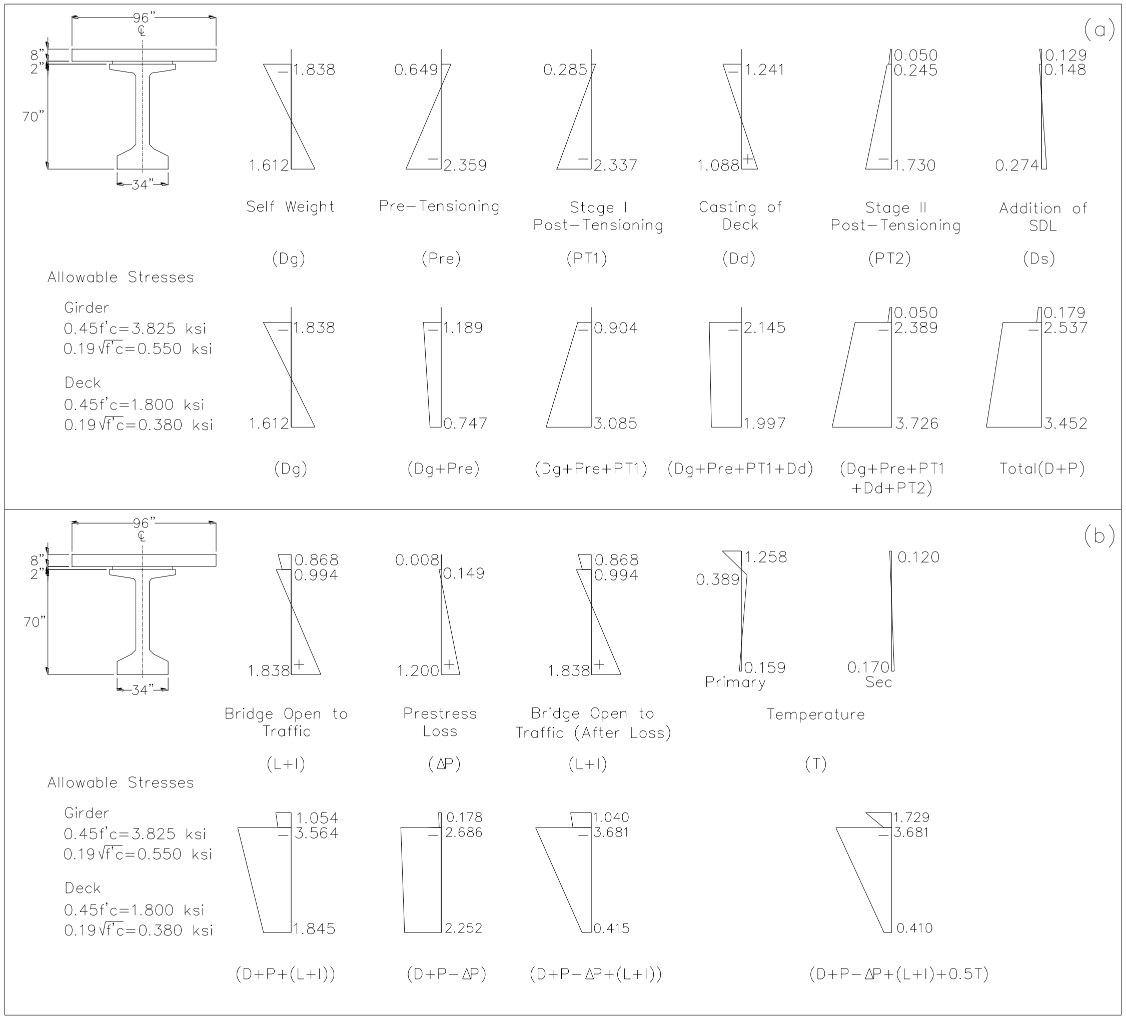


Figure 5.14. Stress Check at Section A-A for (a) Construction and (b) In-service Before and After Losses for Partially Shored Construction.

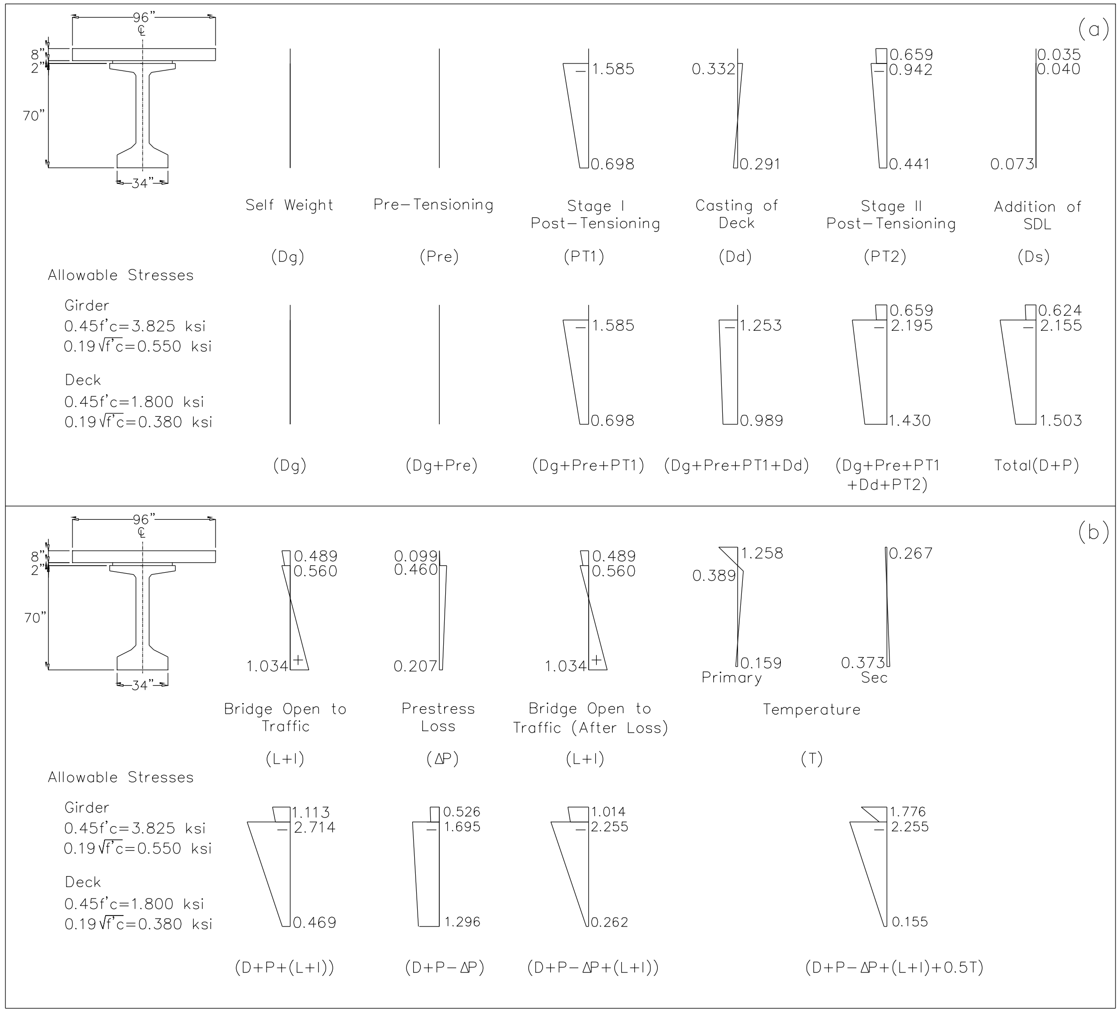


Figure 5.15. Stress Check at Section B-B for (a) Construction and (b) In-service Before and After Losses for Partially Shored Construction.

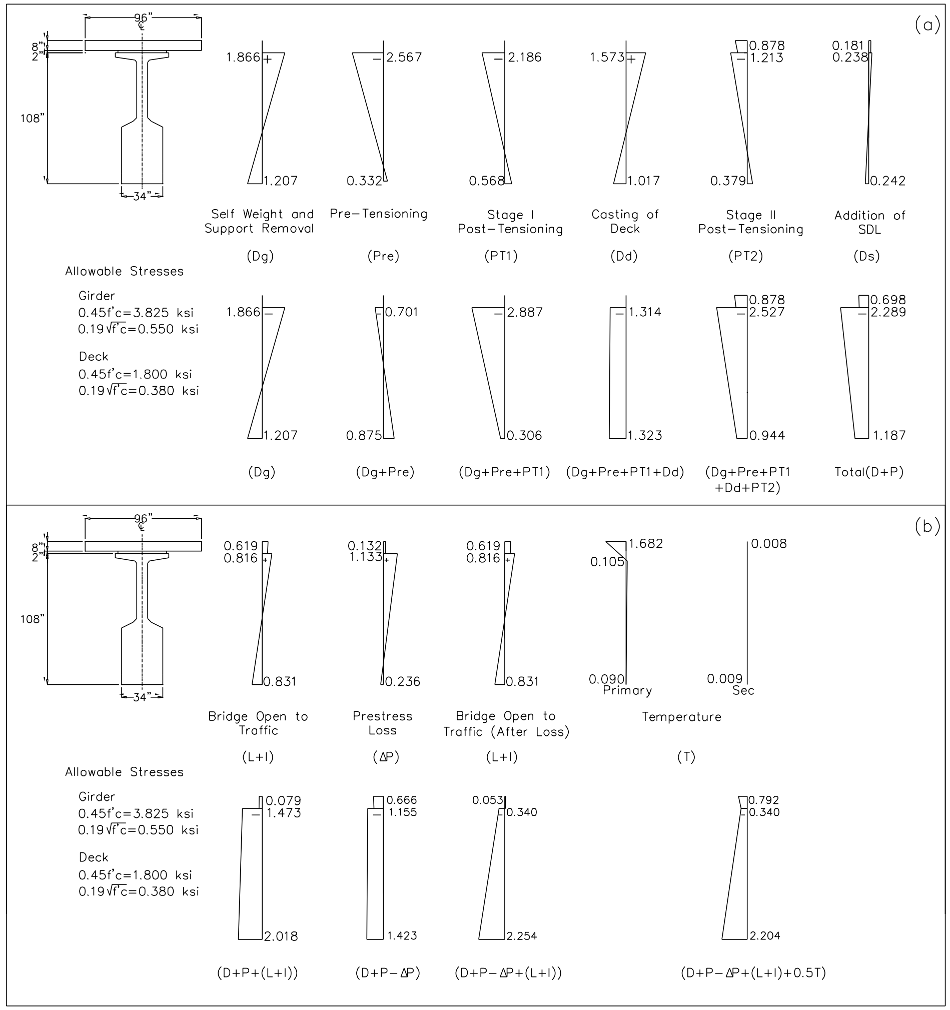


Figure 5.16. Stress Check at Section C-C for (a) Construction and (b) In-service Before and After Losses for Partially Shored Construction.

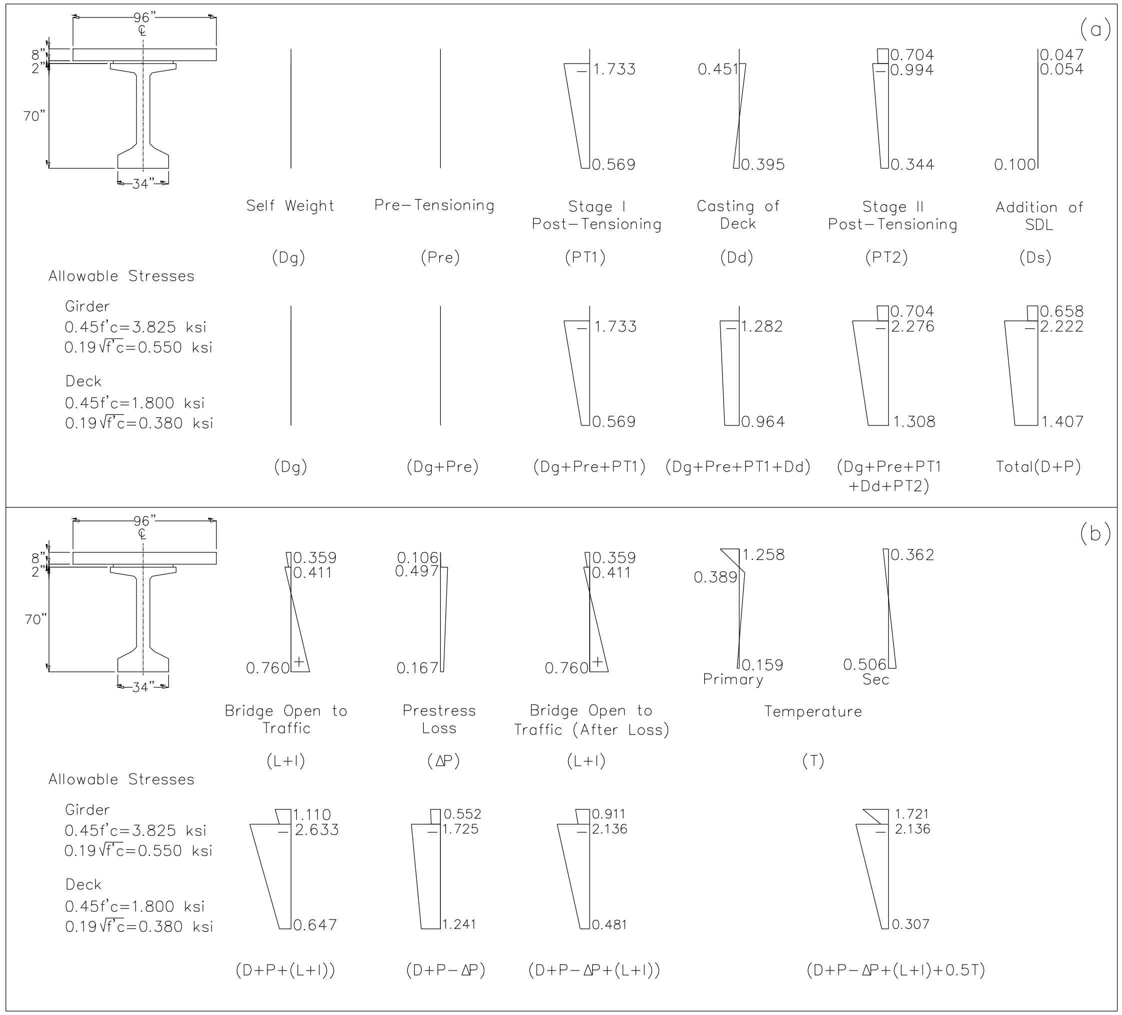


Figure 5.17. Stress Check at Section D-D for (a) Construction and (b) In-service Before and After Losses for Partially Shored Construction.

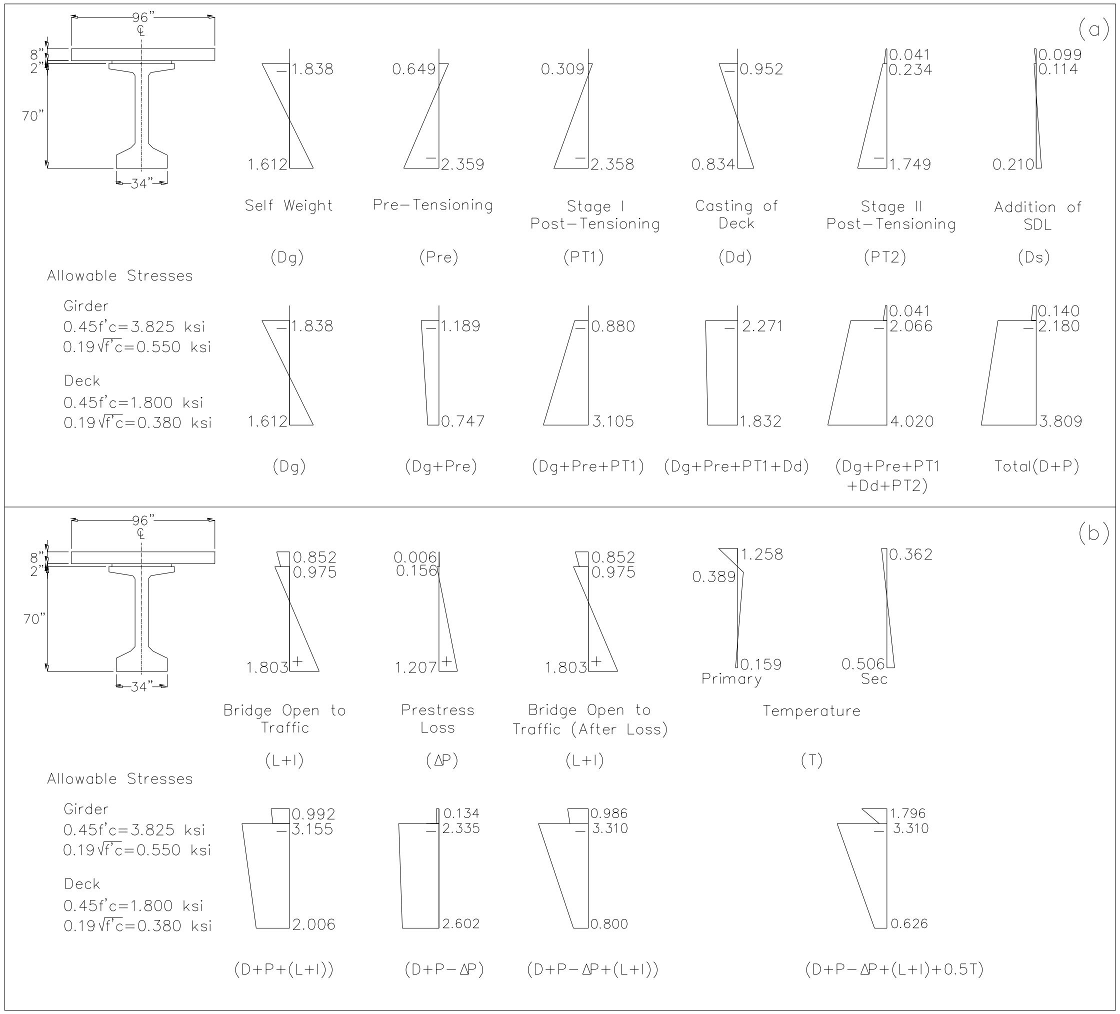


Figure 5.18. Stress Check at Section E-E for (a) Construction and (b) In-service Before and After Losses for Partially Shored Construction.

Table 5.7. Girder Stresses at Various Sections for Partially Shored Construction.

Loading	Component	Location	Section					Limit	
			A-A (End Segment)	B-B (Splice Exterior)	C-C (Pier)	D-D (Splice Interior)	E-E (Drop-in Segment)	Compression	Tension
Step I (Before Loss)	Girder	Top	-1.189	-	-0.701	-	-1.189	-3.825	+0.550
		Bot	-0.747	-	-0.874	-	-0.747		
Step II (Before Loss)	Girder	Top	-2.145	-1.253	-1.314	-1.282	-1.832	-3.825	+0.550
		Bot	-1.997	-0.989	-1.323	-0.964	-2.271		
Step III (After Loss)	Girder	Top	-2.686	-1.695	-1.155	-1.725	-2.335	-3.825	+0.550
		Bot	-2.252	-1.296	-1.423	-1.241	-2.602		
	Deck	Top	-0.172	-0.526	-0.566	-0.552	-0.134	-2.400	+0.380
		Bot	-0.235	-0.502	-0.523	-0.522	-0.206		
Step IV (After Loss)	Girder	Top	-3.681	-2.255	-0.340	-2.136	-3.310	-5.100	+0.550
		Bot	-0.415	-0.262	-2.254	-0.481	-0.800		
	Deck	Top	-1.040	-1.041	+0.053	-0.911	-0.986	-2.400	+0.380
		Bot	-0.891	-0.871	+0.015	-0.793	-0.850		

5.8 DEFLECTION CHECK

The girders are to be checked for allowable deflection under live load and impact as specified in AASHTO LRFD Specifications Article 2.5.2.6.2 (AASHTO 2012). Composite section properties are used in computing these deflections that occur in service. According to AASHTO LRFD Specifications Article 3.6.1.3.2, the deflection is calculated as the larger of:

1. Design Truck alone, or
2. 25 percent of Design Truck Load and full Design Lane Load.

The design truck load is multiplied by the dynamic amplification factor to compute deflections. The limit for maximum deflection as specified in AASHTO LRFD Specifications (2012) Article 2.5.2.6.2, is given by $L/800$. Table 5.6 gives the allowable and actual deflection results for the three-span bridge. The deflections are observed to be within limits.

Table 5.8. Live Load Deflections for Three-Span Continuous Bridge Using Partially Shored Construction.

Deflection	Exterior Span	Interior Span
Allowable (in.)	2.85	3.60
Actual (in.)	1.15	1.06

5.9 ULTIMATE STRENGTH CHECK

The strength limit state needs to be checked to ensure safety at 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, expressed as follows.

$$M_u \leq \Phi M_n \tag{5.1}$$

where,

M_u = Factored ultimate moment at a section, kip-ft.

ΦM_n = Nominal moment strength at a section, kip-ft.

Φ = 1.0 for flexure and tension of prestressed concrete members.

The total factored moment at ultimate according to the *AASHTO LRFD Bridge Design Specifications (*AASHTO 2012*)* is given by,

$$M_u = 1.25\ (M_{dc}) + 1.5\ (M_{dw}) + 1.75\ (M_{LL+IM}) \tag{5.2}$$

where,

M_{dc} = Bending moment due to all dead loads, kip-ft.

M_{dw} = Bending moment due to wearing surface load, kip-ft.

M_{LL+IM}= Bending moment due to live load and impact, kip-ft.

The moment capacity and demand is checked at the following points: 1) at 0.35 *L* of the end span, 2) at the face of pier, and 3) at the midspan of center span. The moment capacity at ultimate depends on the number of strands, diameter of strands, stress in the stands, design strength of concrete, deck reinforcement and the cross-section properties of the girder. The deck reinforcement in the effective flange width consists of 11-#5 bars provided in the top of the deck and 11-#4 bars provided in the bottom of the deck. This is based on recommendation by TxDOT which is the typical deck reinforcement provided for continuous steel bridges. Table 5.9 gives the moment demand and capacity for the three-span bridge. The capacity is found to be greater than demand.

Table 5.9. Ultimate Demand and Capacity for Three-Span Continuous Bridge Using Partially Shored Construction.

Capacity and Demand	Interior Span	Pier	End Span
Demand, M_u (kip-ft)	13,430	25,430	14,340
Capacity, ϕM_n (kip-ft)	26,000	39,360	24,590

5.10 SHEAR DESIGN

Modified compression field theory (MCFT) is used for transverse shear design as specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012). The MCFT takes into consideration the combined effect of axial load, flexure and prestressing when designing for shear. Figure 5.19 shows the maximum factored shear demand and the reduced nominal shear capacity to resist the maximum demand. Figure 5.20 shows the details of the shear reinforcement selected to meet the design requirements, which include the following.

- #4 double legged stirrups at a spacing of 6 in. are provided for a distance of 20 ft from the anchorage end for the end-segment and #4 double legged stirrups at a spacing of 12 in. are provided in the remaining portion.
- #4 double legged stirrups at a spacing of 6 in. are provided for the on-pier segment.
- #4 double legged stirrups at a spacing of 6 in. are provided for a distance of 20 ft from both the ends of drop-in segment and #4 double legged stirrups at a spacing of 12 in. are provided in the remaining portion.

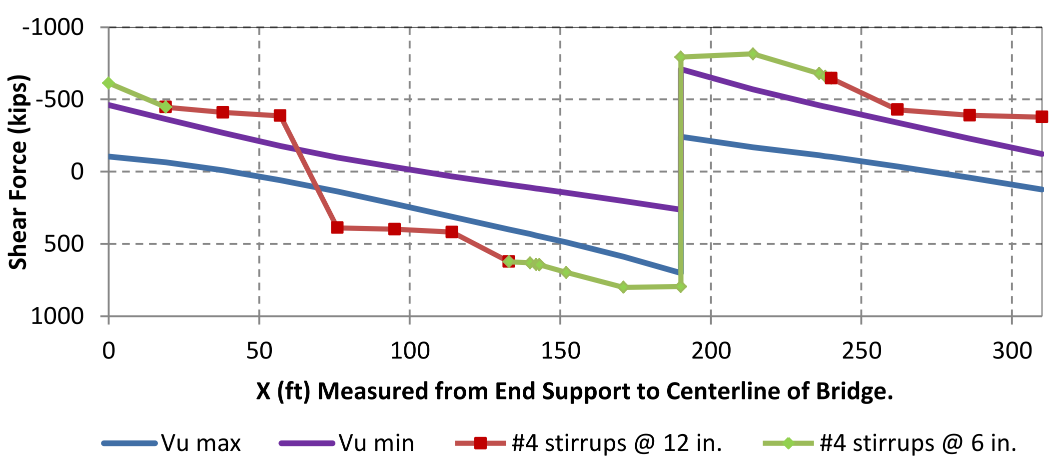


Figure 5.19. Transverse Shear Demand and Capacity for Three-Span Continuous Bridge Using Partially Shored Construction.

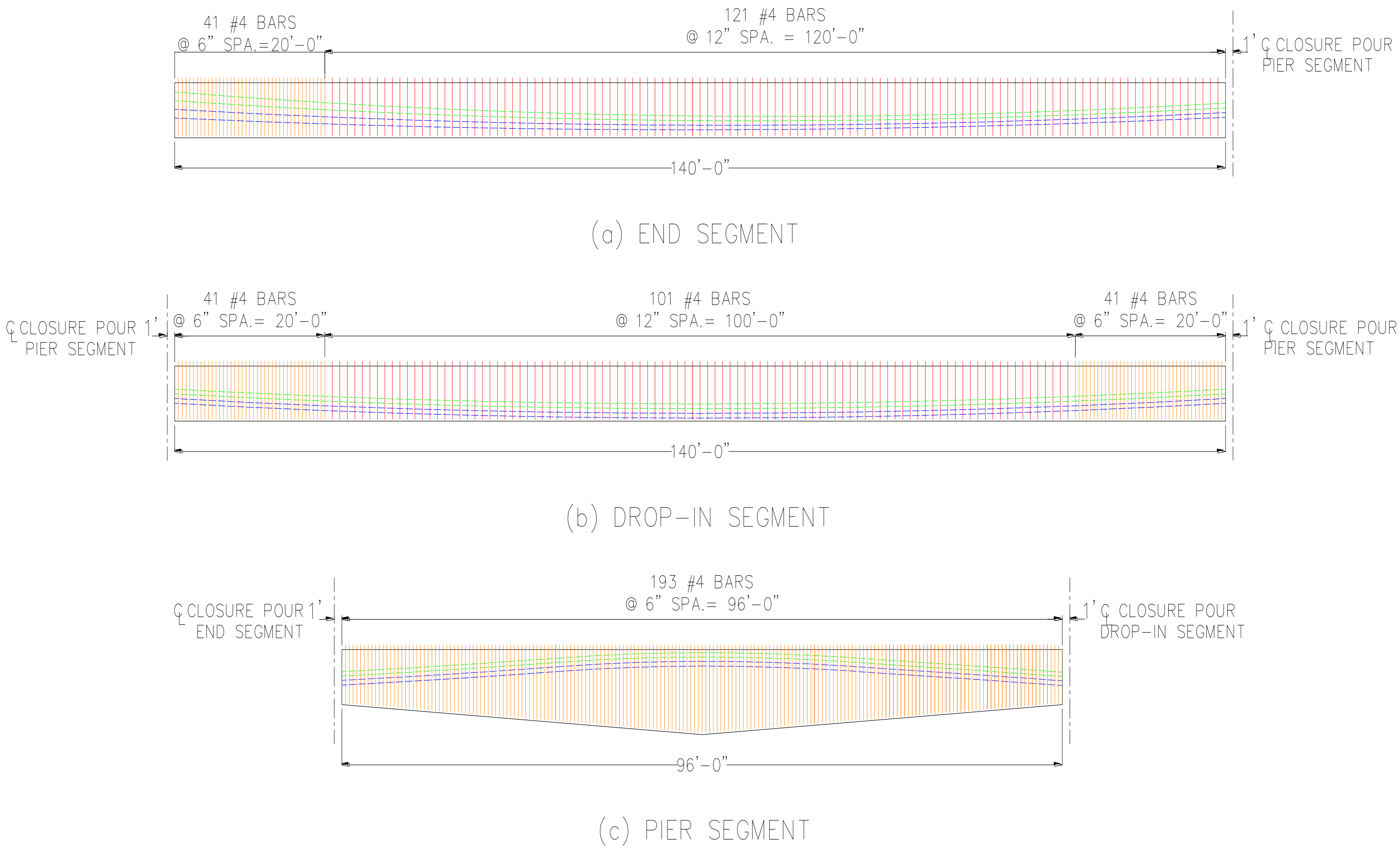


Figure 5.20. Shear Design Details - Elevation View for Three-Span Continuous Bridge Using Partially Shored Construction.

6. PARAMETRIC STUDY

6.1 INTRODUCTION

A parametric study is performed to further explore the design space of spliced girder bridges. For the parametric study the Tx70 and Tx82 girder cross-sections are considered. The design procedure outlined in Section 4 is employed for the parametric study. The requirements for service load limit state design, flexural strength limit state design, and transverse shear design are evaluated in the parametric study. A comparative study is carried out between the design cases in Sections 4 and 5 and the additional cases considered for the parametric study. Table 6.1 outlines the cases that are included in the comparative study.

Table 6.1. Design Cases.

Design Case	Girder Type			Shored Prismatic	Partially Shored Haunched	Span Configuration (ft)
	Tx70 (9 in. web)	Tx82 (9 in. web)	Tx82 (10 in. web)			
1	×			×		190-240-190
2		×		×		190-240-190
3			×	×		190-240-190
4	×				×	190-240-190

The following section provides a summary of differences observed in the parallel designs based the results of this study. The differentiating factors considered for the study are as follows:

- Section properties
- Girder weights
- Prestressing details

- Service stress analysis
- Transverse shear reinforcement
- Ultimate strength consideration and ductility.
- Deflections

6.2 SECTION PROPERTIES

Table 6.2 summarizes the composite and non-composite section properties for the modified Tx70, Tx70 haunched, Tx82 (9 in. web) and Tx82 (10 in. web). For Tx82 (9 in. web), the web height of the modified Tx70 girder is increased by 12 in. The girder is 82 in. deep with a top flange width of 44 in. and a bottom flange width of 34 in. For Tx82 (10 in. web) girder, the web of Tx82 (9 in. web) is increased by an additional 1 in. This results in increases in the width of the top flange and bottom flange. Figure 6.1 and Figure 6.2 shows the details for the prismatic Tx82 (9 in. web) girder and prismatic Tx82 (10 in. web) girder, respectively. The transformed width of slab equal to 62.4 in. is considered to determine the composite section properties.

Table 6.2. Section Properties for Girders.

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})**
Tx70	37.7	32.3	1106	687,111
Tx70 Composite	32.7	45.3	1607	1,287,145
Tx82 (9 in. web)	44.0	38.0	1214	1,088,079
Tx82 Composite (9 in. web)	40.0	52.0	1715	1,902,522
Tx82 (10 in. web)	44.0	38.0	1296	1,106,011
Tx82 Composite (10 in.)	40.0	52.0	1797	1,920,067
Tx70 Haunched	65.6	42.4	2398	2,435,339
Tx70 Haunched Composite	61.5	54.5	2899	4,420,288

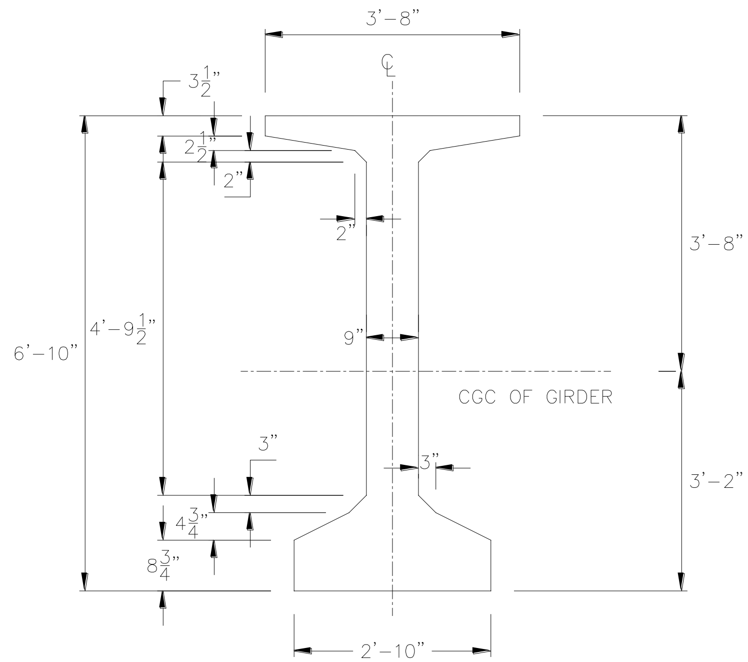


Figure 6.1. Prismatic Tx82 (9 in. Web) Girder.

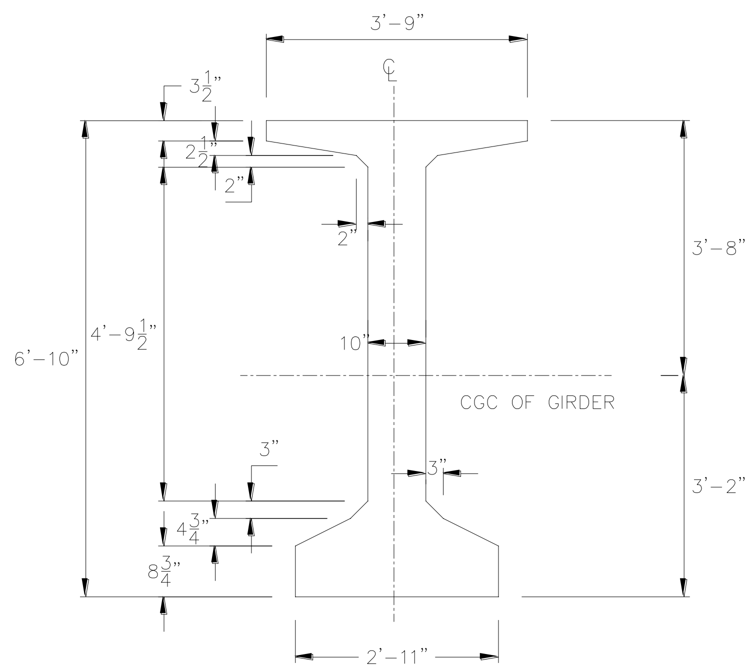


Figure 6.2. Prismatic Tx82 (10 in. Web) Girder.

6.3 GIRDER WEIGHTS

Table 6.3 provides the segment lengths and weights of the girder segments considered for the parametric study. For the haunched segment, a constant web depth haunched girder is considered where the girder weight varies linearly from the girder end to the girder midspan. An increase in web depth of the girders results in an increase in the self-weight of the girders. Also, an increase in web thickness results in an increase in the self-weight of the girders. An increase in web thickness beyond the 9 in. begins to be counterproductive from the design point of view as the additional dead load of the girder may limit the use of longer spans due to handling transportation and erection considerations.

Table 6.3. Segment Lengths and Girder Weights.

Girder Segments	Length (ft)	Weight (kips)	Weight (kip/ft)
Tx70 (9 in. web) Prismatic	140	161	1.152
Tx82 (9 in. web) Prismatic	140	176	1.264
Tx82 (10 in. web) Prismatic	140	189	1.350
Tx70 (9 in. web) Prismatic	96	110	1.152
Tx82 (9 in. web) Prismatic	96	121	1.264
Tx82 (10 in. web) Prismatic	96	130	1.350
Tx70 (9 in. web) Haunched	96	182	1.152-2.488

6.4 PRESTRESSING

The pre-tensioning for the end segment and drop-in-segment is based on handling and transportation requirements. For the on-pier segment pre-tensioning is designed to balance the self-weight and the reactions from the drop-in segment. The pre-tensioning for the on-pier segment is limited by the amount of pre-tensioning strands that can be provided in the top flange of the girder. For the haunched on-pier segment pre-tensioning is provided in the top and bottom flange. Table 6.4 provides information on the amount of pre-tensioning provided for the girder segments.

Table 6.4. Summary of Pre-tensioning.

Girder Section	**End Segment**	**On-Pier Segment**	**Drop-in Segment**
Tx70 (9 in. web) Shored	32	26	30
Tx82 (9 in. web) Shored	22	26	20
Tx82 (10 in. web) Shored	26	26	24
Tx70 (9 in. web) Partially Shored	24	24 top 20 bottom	24

For the shored case, Stage I post-tensioning is designed to balance the self-weight of the girders for the drop-in segment and end segment. The Stage I post-tensioning is provided individually to each girder for the shored case. The Stage I post-tensioning is same for all the cases and is limited by the number of strands that can be provided in a single duct. For the on-pier segment of the shored case, Stage I post-tensioning is designed to balance the self-weight and the reactions from the drop-in segment. The Stage I post-tensioning in the on-pier segment is same for all the shored cases and is limited by the number of strands that can be provided in two ducts. For the partially shored case, Stage I Post-Tensioning is provided continuously and the total strands required are reduced to 32 versus 38. Table 6.5 provides summary of the Stage I post-tensioning.

Table 6.5. Summary of Stage I Post-tensioning.

Girder Section	**End Segment**	**On-Pier Segment**	**Drop-in Segment**
Tx70 (9 in. web) Shored	19 (1 duct)	38 (2 ducts)	19 (1 duct)
Tx82 (9 in. web) Shored	19 (1 duct)	38 (2 ducts)	19 (1 duct)
Tx82 (10 in. web) Shored	19 (1 duct)	38 (2 ducts)	19 (1 duct)
Tx70 (9 in. web) Partially Shored	32 (2 ducts)	32 (2 ducts)	32 (2 ducts)

Stage II post-tensioning is provided to balance the deck weight and super-imposed dead load and is provided continuously for both the shored and partially shored cases. Because the Stage II post-Tensioning balances the deck and superimposed dead load, the Stage II post-tensioning is the same for the Tx82 (9 in. web) and Tx82 (10 in. web). The increase in depth results in a decrease in the amount of post-tensioning required. Thus, the Stage II post-tensioning required is less for the Tx82 girder as compared to the Tx70 girder. Table 6.6 summarizes the Stage II post-tensioning.

Table 6.6. Summary of Stage II Post-tensioning.

Girder Section	**Continuous Bridge**
Tx70 (9 in. web) Shored	57 (3 ducts of 19)
Tx82 (9 in. web) Shored	34 (2 ducts of 17)
Tx82 (10 in. web) Shored	34 (2 ducts of 17)
Tx70 (9 in. web) Partially Shored	30 (2 ducts of 15)

Table 6.7 and Table 6.8 provide results for areas and weights of prestressing steel (pre-tensioning and post-tensioning) for the four cases considered for this study. The area and weight of steel required is the highest for the shored case using the Tx70 girder. The thicker bottom flange in the partially shored case for the on-pier segment reduces the area and the weight of the prestressing steel required. The area and weight of the steel is also reduced as the depth of the girder increases.

Table 6.7. Summary of Prestressing Steel Area.

Girder Section	End Segment A_{ps} (in.^{2})	On-Pier Segment A_{ps} (in.^{2})	Drop-in Segment A_{ps} (in.^{2})	Total A_{ps} (in.^{2})
Tx70 (9 in. web) Shored	23.4	26.2	23.0	72.6
Tx82 (9 in. web) Shored	16.2	21.2	15.8	53.2
Tx82 (10 in. web) Shored	17.1	21.2	16.7	55.0
Tx70 (9 in. web) Partially Shored	18.6	23.0	18.6	60.2

Table 6.8. Summary of Prestressing Steel Weight.

Girder Section	End Segment (lbs)	On-Pier Segment (lbs)	Drop-in Segment (lbs)	Total Weight (lbs)
Tx70 (9 in. web) Shored	11,164	8577	10,973	30,714
Tx82 (9 in. web) Shored	7753	6946	7546	22,245
Tx82 (10 in. web) Shored	8166	6946	7959	23,071
Tx70 (9 in. web) Partially Shored	8890	7513	8890	25,293

6.5 SERVICE STRESS

This section provides a summary of stresses in the girder and the deck at selected locations during different steps of construction for different cases considered for the parametric study. Table 6.9 summarizes the allowable stress limits for the girder and deck which are specific to this study.

Table 6.9. Summary of Allowable Stress Limits in Girder and Deck.

Description	Type of Stress	Initial Loading Stage at Transfer (ksi)	Intermediate Loading Stage at Service (ksi)	Final Loading Stage at Service (ksi)
Girder	Compression	-3.825	-3.825	-5.100
	Tension	+0.611	+0.550	+0.550
Deck	Compression	-	-	-2.400
	Tension	-	-	+0.380

The important construction steps for checking girder stresses for the shored cases are identified as follows:

- Step I: Girder segments supported on piers and temporary supports.
- Step II: Girders supporting weight of wet CIP deck.
- Step III: Application of Stage II post-tensioning, removing of shoring towers and casting of barriers.
- Step IV: Bridge in Service.

The important construction steps for checking girder stresses for the partially shored cases are identified as follows:

- Step I: Girder segments supported on piers and temporary supports.
- Step II: Application of Stage I post-tensioning and casting deck.
- Step III: Application of Stage II Post-tensioning and casting barriers.
- Step IV: Bridge in service.

Table 6.10. Stresses (ksi) at the Location of Maximum Positive Moment in End Segment (Section A-A).

Loading	Component	Location	Tx70 (9 in. web) Shored	Tx82 (9 in. web) Shored	Tx82 (10 in. web) Shored	Tx70 (9 in. web) Partially Shored
Step I (Before Loss)	Girder	Top	-1.677	-1.089	-1.007	-1.189
		Bot	-2.238	-1.740	-1.892	-0.747
Step II (Before Loss)	Girder	Top	-2.519	-1.752	-1.641	-2.145
		Bot	-1.500	-1.167	-1.344	-1.997
Step III (After Loss)	Girder	Top	-3.321	-2.278	-2.187	-2.686
		Bot	-2.730	-1.766	-1.832	-2.252
	Deck	Top	-0.439	-0.229	-0.213	-0.172
		Bot	-0.531	-0.277	-0.259	-0.235
Service (After Loss)	Girder	Top	-4.325	-3.131	-3.014	-3.681
		Bot	-0.874	-0.306	-0.417	-0.415
	Deck	Top	-1.316	-0.941	-0.903	-1.040
		Bot	-1.194	-0.840	-0.805	-0.891

Table 6.11. Stresses (ksi) at Midspan of Drop-in Segment (Section E-E).

Loading	Component	Location	Tx70 (9 in. web) Shored	Tx82 (9 in. web) Shored	Tx82 (10 in. web) Shored	Tx70 (9 in. web) Partially Shored
Step I (Before Loss)	Girder	Top	-1.447	-1.011	-0.934	-1.189
		Bot	-2.290	-1.673	-1.829	-0.747
Step II (Before Loss)	Girder	Top	-2.031	-1.470	-1.373	-1.832
		Bot	-1.778	-1.275	-1.449	-2.271
Step III (After Loss)	Girder	Top	-3.140	-2.195	-2.118	-2.335
		Bot	-2.528	-1.567	-1.633	-2.602
	Deck	Top	-0.655	-0.388	-0.367	-0.134
		Bot	-0.694	-0.403	-0.381	-0.206
Service (After Loss)	Girder	Top	-4.199	-3.095	-2.990	-3.310
		Bot	-0.570	-0.028	-0.141	-0.800
	Deck	Top	-1.580	-1.139	-1.095	-0.986
		Bot	-1.393	-0.997	-0.957	-0.850

Table 6.10 and Table 6.11 provide results for stresses at the midspan of the drop-in segment (section E-E in Figure 5.11) and at the location of maximum positive moment in the end segment (Sectioin A-A in Figure 5.11) during various stages of construction for the different load cases considered for the parametric study. The stresses are within the allowable stress limit during all the stages of construction for both the shored and partially shored case.

Table 6.12 and Table 6.13 provide results for stresses at the end span and interior span splice (Section B-B and Section D-D in Figure 5.11) during various stages of construction for the different loading considered for the parametric study. The stresses in bold font exceed the limiting stresses for the corresponding load stage. For the shored construction, the splice exceeds the prestressed tension stress limit and some cracking is anticipated during the stage when deck is poured. However, Stage II post-tensioning puts the splice in compression at service. A partially prestressed splice is used and mild steel needs to be provided for serviceability and strength. It is observed that the splice in the end span is more critical as compared to the splice in the interior span. Also, tensile stresses are observed at the bottom of the splice at service.

For the partially shored Tx70 case, since the Stage I post-tensioning is carried out continuously, the splice is uncracked during construction and at service because the Stage I post-tensioning is carried out continuously. A cast-in-place post-tensioned splice is used. Mild steel reinforcement can be provided to meet strength requirements.

Table 6.12. Stresses (ksi) at End Span Splice (Section B-B).

Loading	**Component**	**Location**	**Tx70 (9 in. web) Shored**	**Tx82 (9 in. web) Shored**	**Tx82 (10 in. web) Shored**	**Tx70 (9 in. web) Partially Shored**
Step I (Before Loss)	Girder	Top	-	-	-	-
		Bot	-	-	-	-
Step II (Before Loss)	Girder	Top	**+1.120**	**+0.882**	**+0.843**	-1.253
		Bot	-0.982	-0.762	-0.728	-0.989
Step III (After Loss)	Girder	Top	-0.565	-0.205	-0.197	-1.695
		Bot	-1.390	-0.786	-0.737	-1.296
	Deck	Top	-1.208	-0.786	-0.753	-0.526
		Bot	-1.112	-0.718	-0.686	-0.502
Service (After Loss)	Girder	Top	-1.199	-0.744	-0.719	-2.255
		Bot	-0.217	+0.136	+0.156	-0.262
	Deck	Top	-1.762	-1.236	-1.189	-1.041
		Bot	-1.531	-1.073	-1.031	-0.871

Table 6.13. Stresses (ksi) at Interior Span Splice (Section D-D).

Loading	**Component**	**Location**	**Tx70 (9 in. web) Shored**	**Tx82 (9 in. web) Shored**	**Tx82 (10 in. web) Shored**	**Tx70 (9 in. web) Partially Shored**
Step I (Before Loss)	Girder	Top	-	-	-	-
		Bot	-	-	-	-
Step II (Before Loss)	Girder	Top	**+0.818**	**+0.645**	**+0.616**	-1.282
		Bot	-0.717	-0.557	-0.532	-0.964
Step III (After Loss)	Girder	Top	-0.659	-0.256	-0.243	-1.725
		Bot	-1.508	-0.900	-0.850	-1.241
	Deck	Top	-1.027	-0.631	-0.602	-0.552
		Bot	-0.975	-0.595	-0.567	-0.522
Service (After Loss)	Girder	Top	-1.070	-0.606	-0.582	-2.136
		Bot	-0.481	-0.302	-0.271	-0.481
	Deck	Top	-1.385	-0.922	-0.884	-0.911
		Bot	-1.246	-0.825	-0.790	-0.793

Table 6.14 provides results for stresses at the pier (Section C-C in Figure 5.11) during various stages of construction for the different load cases considered for the parametric study. The bold font indicates a stress exceeds the limiting stress value. For the shored case, the pier region of the girder experienced compressive stress levels that exceeded the allowable compressive stress at service conditions. This stress exceedance is addressed by providing supplemental mild steel reinforcement in the compression zone. The pier region of the deck also experienced tensile stresses that exceed the allowable stress limits. However, these stresses are only 0.15 ksi over the tensile stress limit of 0.380 ksi. Mild steel is used in the deck and will help to limit crack widths.

For the partially shored case, the stresses are within limits during all the stages of construction and service. The pier region of the beam experienced tensile stresses but are within the allowable stress limits.

Table 6.14. Stresses (ksi) at Pier (Section C-C).

Loading	**Component**	**Location**	**Tx70 (9 in. web) Shored**	**Tx82 (9 in. web) Shored**	**Tx82 (10 in. web) Shored**	**Tx70 (9 in. web) Partially Shored**
Step I (Before Loss)	Girder	Top	-2.434	-2.919	-2.555	-0.701
		Bot	-2.449	-1.626	-1.674	-0.874
Step II (Before Loss)	Girder	Top	-2.738	-3.158	-2.784	-1.314
		Bot	-2.183	-1.419	-1.476	-1.323
Step III (After Loss)	Girder	Top	-2.362	-2.430	-2.014	-1.155
		Bot	-4.091	-2.960	-3.030	-1.423
	Deck	Top	-0.541	-0.130	-0.117	-0.566
		Bot	-0.608	-0.199	-0.184	-0.523
Service (After Loss)	Girder	Top	-1.427	-1.644	-1.244	-0.340
		Bot	**-5.818**	-4.318	-4.346	-2.254
	Deck	Top	+0.275	**+0.532**	**+0.525**	+0.053
		Bot	+0.009	**+0.325**	**+0.324**	+0.015

6.6 DEFLECTIONS

Table 6.15 provides results for maximum live load deflections in the end span and center span for the cases considered for the parametric study. It is observed that the deflections are within the limit (*L*/800) for all the design cases.

Table 6.15. Maximum Live Load Deflections.

Girder Section	End Span	Center Span
Tx70 (9 in. web) Shored	1.21	1.34
Tx82 (9 in. web) Shored	0.81	0.91
Tx82 (10 in. web) Shored	0.80	0.90
Tx70 (9 in. web) Partially Shored	1.15	1.06
Limit (in.)	2.85	3.60

6.7 ULTIMATE FLEXURAL STRENGTH REQUIREMENT AND DUCTILITY

Table 6.16 provides results for moment capacity and demand at ultimate. Ductility requirements for the girder at the pier section are a limiting factor in setting the maximum span lengths of the girder segments. For the shored case, mild steel reinforcement is added in the bottom flange of the on-pier girder segment, which acts as compression steel to improve ductility. Also, Dywidag bars that are provided during handling and transportation of girder segments are included as compression steel for the shored case. The amount of compression steel required reduces as the depth of the girder increases. Also, the increase in web thickness results in a reduction in the required compression steel. However, an increase in web thickness has a minimum effect on the amount of mild steel. The thicker bottom flange for the on-pier segment in the partially shored case helps in providing higher moment capacity at ultimate. Table 6.17 provides results for the amount of mild steel added for ductility.

Table 6.16. Summary of Moment Capacity and Demand at Ultimate.

Girder Section	Description	End Segment	On-Pier Segment	Drop-in Segment
Tx70 (9 in. web) Shored	Demand, M_u (kip-ft)	14,940	20,680	15,330
	Capacity, ϕM_n (kip-ft)	22,780	24,180	24,430
Tx82 (9 in. web) Shored	Demand, M_u (kip-ft)	15,280	21,320	15,680
	Capacity, ϕM_n (kip-ft)	25,420	28,530	25,580
Tx82 (10 in. web) Shored	Demand, M_u (kip-ft)	15,550	21,800	15,9400
	Capacity, ϕM_n (kip-ft)	26,280	28,280	26,450
Tx70 (9 in. web) Partially Shored	Demand, M_u (kip-ft)	14,340	25,430	13,430
	Capacity, ϕM_n (kip-ft)	24,590	39,360	26,000

Table 6.17. Summary of Compression Steel for Ductility.

Girder Section	Compression Steel
Tx70 (9 in. web) Shored	16-#14 and 4 Dywidag
Tx82 (9 in. web) Shored	12-#14 and 4 Dywidag
Tx82 (10 in. web) Shored	10-#14 and 4 Dywidag
Tx70 (9 in. web) Partially Shored	-

Note: Dywidag bars are 1.25 in. diameter.

6.8 SHEAR DESIGN

Table 6.18 provides details for shear design for the four design cases. It is observed that an increase in depth results in an increase in shear capacity of the girders. Also, an increase in web thickness results in an increase in shear capacity of the girders. However, the increase in web thickness considered has a very minimal effect on increase in shear capacity of the girders. The deeper bottom flange provides higher shear capacity for the on-pier segment for the partially shored case.

Table 6.18. Summary of Shear Design Details.

Girder Section	End Segment	On-Pier Segment	Drop-in Segment
Tx70 (9 in. web) Shored	#5@4 in. (0-10 ft) #5@6 in. (10-20 ft) #5@12 in. (20-140 ft)	#5@6 in. (0-20 ft) #5@12 in. (20-120 ft) #5@6 in. (120-140 ft)	#5@6 in. (0-29 ft) #5@4 in. (29-72 ft) #5@6 in. (72-96 ft)
Tx82 (9 in. web) Shored	#4@12 in. (0-140 ft)	#4@12 in. (0-140 ft)	#4@6 in. (0-38 ft) #4@4 in. (38-58 ft) #4@6 in. (58-96 ft)
Tx82 (10 in. web) Shored	#4@12 in. (0-140 ft)	#4@12 in. (0-140 ft)	#4@6 in. (0-38 ft) #4@4 in. (38-58 ft) #4@6 in. (58-96 ft)
Tx70 (9 in. web) Partially Shored	#4@6 in. (0- 20 ft) #4@12 in. (20-140 ft)	#4@6 in. (0-20 ft) #4@12 in. (20-120 ft) #4@6 in. (120-140 ft)	#4@6 in. (0-96 ft)

Note: All shear reinforcement consists of double legged stirrups.

7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 SUMMARY

This report summarizes the results of a study that has been conducted to develop guidelines for design of spliced girder bridges in Texas. First, a review of literature on design and construction techniques employed for existing spliced girder bridges was carried out. Second, detailed application examples were prepared for both the shored and the partially shored method of construction. Third, a parametric study is carried out by varying the construction approach and the girder cross-sections. Based on the results of the design examples and the parametric study, critical design issues are highlighted. Additional information and recommendations for these critical design issues have been provided to assist in the implementation of spliced girder bridges in Texas. Several areas requiring further study are identified based on the detailed design examples.

7.2 CONCLUSIONS

7.2.1 General

The use of in-span splices to make precast, prestressed concrete bridge girders continuous, presents a viable alternative for increasing span lengths using standard precast girder sections. This system helps to bridge the gap between simply supported precast pre-tensioned concrete girder bridges and post-tensioned concrete segmental box or steel girder bridges. Different methods are available for the construction of spliced girder bridges, which are categorized into shored, unshored and partially shored. The selection of method of construction depends on the site conditions, availability of equipment and the experience of the local contractor. Spliced girder bridges present a competitive, economical and high performance alternative to steel plate or segmental bridges for longer spans up to 300 ft. The load balancing technique has been effectively used for design of spliced girder bridges. One advantage of using load balancing is that are no or only minimal creep deflections. This section outlines the conclusions derived from the application examples and the parametric study.

7.2.2 Shored Design

The following conclusions were developed based on the designs using shored construction.

1. A span length of 240 ft is possible using shored construction using prismatic Tx70 girders (with 9 in. web), but not easily obtainable. A large numbers of tendons are required and mild steel is required in the pier region for ductility.
2. For transportation and handling purposes of the pier segments of the prismatic girder bridges, temporary unbonded Dywidag threadbars of 1.25 in. diameter were included in the designs for shored construction.
3. Tensile strain limits over the pier are a critical factor in setting the maximum span lengths of the girder segments. Mild steel reinforcement is added in the bottom flange of the on-pier girder segment as compression steel to improve ductility and the moment capacity of the girder section in the negative moment region.
4. The shoring towers are provided both in the end span and center span and are removed after pouring the deck and Stage II post-tensioning. The removal of shoring towers results in support removal moments that need to be considered in the design.
5. The newly cast splice is cracked during the stage when deck is poured. A partially prestressed splice is used and mild steel is provided for serviceability and strength. The splice is uncracked after Stage II post-tensioning is applied and at service conditions.
6. The stresses in the girders and the deck were checked at critical locations along the length of the bridge for the service limit states. The pier region of the beam experienced compressive stress levels that exceeded the allowable compressive stress at service conditions. This stress exceedance is addressed by providing supplemental mild steel reinforcement in the compression zone.

7. A span length of 240 ft is possible using shored construction and prismatic Tx82 (9 in. or 10 in. web) girders. The compressive stresses at the different load stages are within limits but relatively small tensile stresses are observed in the pier region of the deck.
8. For the same span length, girder section and method of construction, the advantage of using Tx82 over Tx70 include reduction in total amount of prestressing steel, increased shear and moment capacities and reduction in mild steel requirements for ductility in the pier region.

7.2.3 Partially Shored Design

The following conclusions were developed based on the designs using partially shored construction.

1. A span length of 240 ft is attainable using partially shored construction using prismatic Tx70 girders for drop-in and end segments and a haunched on-pier segment.
2. For transportation and handling purposes of the haunched on-pier segments, pre-tensioning strands are provided in the bottom flange.
3. The thicker bottom flange for the haunched on-pier segment allows for higher moment and shear capacities at ultimate.
4. The backspan shoring towers are removed after Stage I post-tensioning and before pouring the deck. This prevents any residual stresses due to removal of shoring towers to be transmitted to the deck.
5. The splice is uncracked during construction and at service. A cast-in-place post-tensioned splice is used. Mild steel reinforcement should be provided to meet strength requirements.
6. The design for unshored construction can be carried out similarly to partially shored design. A temporary connection (tie downs) can be provided at the pier instead of providing back span shoring towers. The tie downs would be removed after Stage I post-tensioning and before pouring the deck. However,

wider piers are required for stability and overturning and the details for the connection are more complicated.

7.3 RECOMMENDATIONS

7.3.1 Handling and Transportation

Based on previous input from precasters and contractors (Hueste et al. 2012) it is recommended to limit the maximum span length to 160 ft, the maximum weight to 200 kips and maximum depth to 10 ft due to handling, transportation and erection considerations.

7.3.2 Splice Considerations

1. In-span splice locations vary for different projects built to date. The location of a splice at the inflection point is ideal in terms of serviceability and to limit demands on the splice. However, it is important to determine the best possible splice locations specifically for each project.
2. The length of splice should be large enough so as to allow splicing of tendons, but not too large since there is no pre-tensioning through the joint and minimum mild steel reinforcement before stressing of continuity post-tensioning occurs. A 2 ft splice length was assumed for this study.
3. For shored construction design cases, cracking is expected in the splice region during the stage when deck is poured. A fully prestressed splice can be used whereby cracking can be prevented by providing prestressing as short tendons across the splice. However, thickening the girder ends will be required and may not be desirable from the aesthetic point of view.

7.3.3 Web Thickness

AASHTO LRFD Article 5.4.6.2 states that the size of duct shall not exceed 0.4 times the least gross concrete thickness at the duct. A thicker web is desirable in terms of strength and serviceability and to better accommodate the stirrups. Also, the web thickness should be sufficient to provide cover to mild steel reinforcement. However, some of the earlier post-tensioned bridge girders have used a 7.87 in. web thickness for a 4 in diameter duct (PCI 2004). However, based on the literature review a web thickness of 9 in. can be considered adequate for a 4 in. diameter duct. The parametric study indicated that 9 in. web is sufficient to meet design requirements. It is noted that an increase in web thickness beyond 9 in. results in increase in weight of the girders which becomes detrimental as compared to increase in the shear capacity of the girders (NCHRP 517). However, it is generally desirable to have a thicker web in terms of girder stability and concrete placement. In addition, a thicker web can allow the use of harped pre-tensioning to avoid the need for Stage I post-tensioning for the shored case.

7.3.4 Limitation of Tx70 and Tx82 Cross-section with Regard to Continuous Girders

1. The thickness of the top flange of the Tx70 and Tx82 girder for the on-pier segment should be increased to allow placing of two rows of pre-tensioned strands.
2. Proper coordination between the precaster and the designer is required for efficient design. For the haunched on-pier segment, if the precasting plant is not equipped to provide pre-tensioning in the top flange, it can be replaced with post-tensioning. However, thickened ends are required which may not be desirable from the aesthetic point of view.
3. Because ductility of the girders over the pier is one limiting parameter for selecting maximum span lengths, a girder with a wider bottom flange can be considered to improve ductility. Also, a bottom slab can be added to provide additional moment resistance at the interior support.

7.3.5 Sequence of Construction

An alternate sequence of construction can be considered for both the shored and the partially shored methods of construction. The end segments can be erected first which would put a downward reaction in the shoring towers and the pier segments can be erected later. This would prevent the uplift in the shoring towers which is expected in the sequence of construction considered in the design examples during the erection of pier segments. The location of the shoring towers needs to be considered prior to selecting an appropriate sequence of construction.

7.4 SCOPE FOR FUTURE WORK

1) Handling and Transportation
    - The maximum transportable length of girder segments is influenced by the weights of girder segments. Using lightweight concrete can be considered to reduce the weights of girder segments.
    - An on-pier splice can be combined with an in-span splice. This will help reduce the weight of the on-pier segment, which primarily limits the maximum transportable length of the girder segments, especially in cases of haunched on-pier girder segments. This will help in further increasing the span lengths of spliced girder bridges.
2) Deck Pouring
    - For the designs under consideration, the entire deck is assumed to be poured in single stage. However, as the span lengths of the bridge increases, the pouring of the concrete for the deck in a single phase becomes difficult. Sequencing of the CIP deck concrete is an important design consideration and should be included with future designs.
3) Ductility
    - The maximum span lengths that can be easily achieved using prismatic girders are greatly limited by ductility in the pier region. A partially

prestressed solution has been considered where mild steel is added in the bottom flange of the on-pier segment to increase ductility. However, the effect of mild steel needs to be considered in composite section properties and further study is required.

4) Prestress Losses and Time Dependent Parameters

- Time dependent material properties of concrete like creep and shrinkage are important in analysis and design of spliced girder bridges. Creep and shrinkage of concrete have an effect on deflection and stresses. Selecting a conservative value for creep and shrinkage may make satisfaction of allowable stresses difficult while underestimating the values that may result in cracking in the deck. A detailed time dependent study needs to be performed taking into consideration the effect of creep and shrinkage.
- For design purposes, prestress losses for pre-tensioning and for post-tensioning are assumed. However, proper estimation of prestress losses is critical in the design of spliced girder bridges. Overestimation of loss would result in higher prestress than expected which will result in higher camber. Underestimation of loss would result in less prestress and could lead to unexpected cracking. A more accurate prediction of prestress loss taking into consideration the time dependent effect of creep and shrinkage is recommended in the future designs.

5) Lateral Stability

- Lateral stability of the girders needs to be checked during handling, transportation and erection of girder segments. It is recommended to proportion the width of the top flange of the girder as a function of span length for the purpose of lateral stability. Temporary diaphragms or cross bracings can be provided to ensure lateral stability of the girders during transportation and erection. Also, permanent diaphragms can be provided for lateral stability. The advantages and disadvantages of using diaphragms need further review.

6) Unshored Construction

   - An unshored design can be considered where a permanent connection can be created between the on-pier segments and the pier. The moments due to the drop-in segment and end-segment can be directly transferred to the pier. However, wider piers will be required and this option requires further study.

7) Girder Spacing

   - One of the advantages of spliced girder bridges is that they facilitate use of wider spacing of girders. Reducing the number of lines of girders will aid in economical construction of spliced girder bridges. A comparative study between the girder spacing and span length will help in optimizing the design of spliced girder bridges.

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