PROTOTYPE DESIGN AND EXPERIMENTAL ABSTRACTION FOR VERIFYING SPLICED GIRDER PERFORMANCE OF CONTINUOUS PRESTRESSED CONCRETE BRIDGES

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ABSTRACT

Traditionally, precast prestressed concrete girder bridges have had limited span lengths due to handling and transportation restrictions on individual girder segments. So, how can one increase the span length of these bridges by 50 to 100 percent? An effective way is to splice the individual girder segments within the span of the continuous bridge. In order to demonstrate the use of in-span splicing techniques, a design for a three-span continuous prototype bridge has been developed using Tx70 precast concrete girders. The span-to-depth ratio adopted for this bridge has been maximized based on design limitations.

The design and analysis results suggest that spliced girders can provide a feasible approach for constructing more economic and efficient longer span bridges. A full-scale experimental study is underway to investigate the performance of the prototype bridge details in the splice region under service and ultimate loads. This paper discusses the prototype design and the development of the full-scale laboratory specimen.

Keywords: Prestressed Concrete, Post-tensioning, Continuity, Spliced Girder, Connection, Design

INTRODUCTION

Continuous precast prestressed concrete bridges using standard precast girder sections and in-span splices are becoming more popular for medium span bridges. Spliced concrete girder bridges help to bridge the gap between precast, pretensioned concrete bridges made continuous at the pier for live loads (limited to approximately 150 ft spans) and continuous, post-tensioned concrete segmental box girder bridges (limited to approximately 300 ft spans). These bridges also present a cost-competitive, constructible, and high performance alternative to steel plate or steel box girder solutions for longer spans up to 280 ft.

In-span spliced girder technology facilitates wider spacing between girder lines and a minimum number of substructure units. The provision of continuity and absence of joints over the pier region improves the serviceability of the deck, resistance to corrosion and cracks and provides a smooth ride. The precaster can fabricate the girder segments in transportable lengths to achieve a new span range with a contractor preferred form of construction. This design and construction approach is less complex than segmental construction and results in fewer joints, which can improve long-term serviceability. The performance and cost-effectiveness of a spliced girder system depends on design and construction details. This involves a combination of different design enhancements instead of applying them individually.[1]

This paper outlines some of the strategies that have been used for spliced, continuous, bridge girder systems, discusses a number of construction considerations, presents a continuous spliced girder bridge design, proposes a potential splice connection detail, and provides an outline of the testing program to investigate the performance of the girder design as well as connection detail under service and strength limit states.

CONTINUOUS SPLICED GIRDER BRIDGE SYSTEMS

An investigation of the design and performance of spliced girder bridges is being conducted at Texas A&M University and is supported by the Texas Department of Transportation (TxDOT) and FHWA through the Texas A&M Transportation Institute as part of ongoing project 0-6651, "Continuous Prestressed Concrete Girder Bridges." The outcome of this research project will support TxDOT's implementation of continuous precast, prestressed concrete bridge girders to achieve longer span-to-depth ratios with greater economy than currently possible with simple spans.[2]

Preliminary designs were conducted in the first phase of this research study as an initial evaluation intended to push the limits of span-to-depth ratios and design of continuous bridges using standard TxDOT girder sections. The preliminary designs and details helped to identify the maximum span limits and design and construction limitations with regard to implementation of standard precast shapes currently used by TxDOT for simply supported prestressed concrete bridges. The proposed splice connection types varied between a full prestressed connection to a full reinforced connection with each type having their own advantages and disadvantages. A partially prestressed solution having a balance of constructability and long-term serviceability was selected to be the most desirable based on input from TxDOT engineers, contractors and precasters.[2]

The main goals of this study are to evaluate the parameters and factors that influence the design of continuous spliced girder bridges. Along with the usual limit states considered for the girder design, special consideration is given to the connections at the splice locations. Sequencing of the precast prestressed concrete girder fabrication, cast-in-place (CIP) concrete on site and post-tensioning (PT) operations are also important factors in the design. The following sections outline the prototype bridge design and the abstraction of the specimen that will be tested in the laboratory.

PROTOTYPE BRIDGE - DESIGN AND CONSTRUCTION INTEGRATION

LAYOUT

A prototype three-span continuous spliced precast, prestressed concrete bridge system was investigated. The design follows the revised provisions for spliced precast girders in the AASHTO LRFD Bridge Design Specifications.[3] Fig. 1 provides an elevation view of the bridge superstructure. The length of the main span is 240 ft and that of the end spans is 190 ft with 2 ft wide connection splices. Prismatic Tx70 girders are used for all the girder segments. The research team considered a modified Tx70 shape with a 9 in. web (versus the standard 7 in. web) to accommodate 4 in. diameter PT ducts and to help ensure sufficient shear strength. The web width is increased by spreading the girder side forms. This increases the width of the top and bottom flanges by 2 in., resulting in a top flange width of 44 in. and a bottom flange width of 34 in. The length of the drop-in and end girder segment is 140 ft and that of the on-pier segment is 96 ft. The lengths of the drop-in, end and on-pier girder segments are chosen to satisfy the transportation limitations in Texas. It is recommended to limit the maximum span length of the girder segments to 160 ft, the maximum weight to 200 kips and the maximum depth to 10 ft based on handling, transportation and erection considerations.[2] Table 1 summarizes the span lengths and weights for the girder segments.

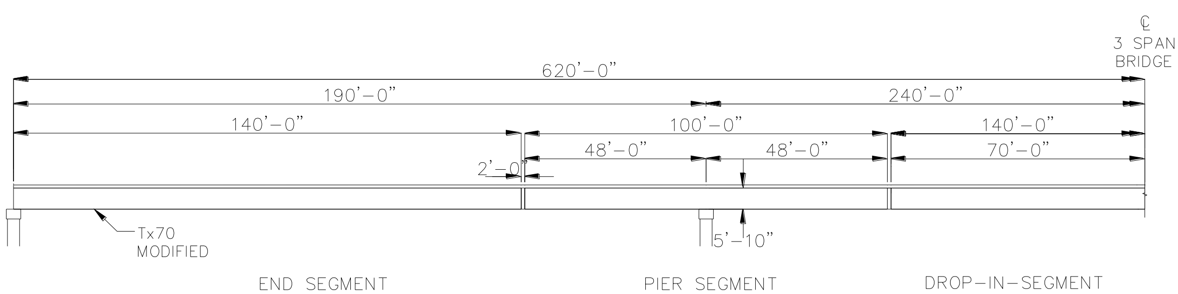


Fig. 1. Elevation of three-span continuous prototype bridge

Table 1. Girder Segment Lengths and Weights

Girder Segment Description	Length (ft)	Weight (kips)
End Segment and Drop-in-Girder Segment	140	161
On-Pier Segment	96	111

Fig. 2 shows the cross-section of the bridge. A standard reinforced concrete deck is used with an 8 in. thickness. The bridge is designed for a total of three traffic lanes in accordance with the design criteria specified in the AASHTO LRFD Bridge Design Specifications.[3] A T501 traffic barrier is used as presented in the standard drawings of the TxDOT Bridge Design Manual.[4] The nominal face of the rail is 1 ft. The asphalt wearing surface is considered to be 2 in. thick. The spacing and overhang configuration considered for this Tx70 girder bridge results in optimum distribution of dead load and live load to the interior and exterior girders. Therefore, all girders have the same design requirements. The HL-93 design live load is used.

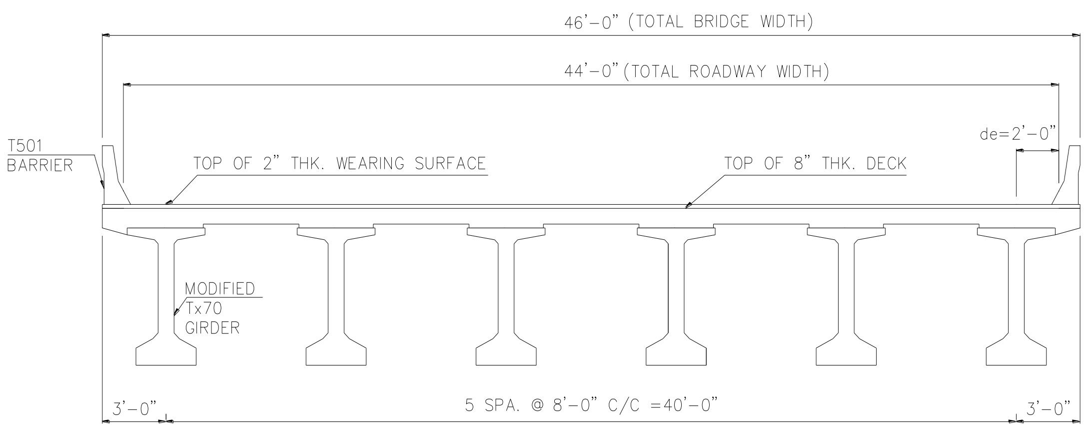


Fig. 2. Transverse section of prototype bridge

MATERIAL PROPERTIES

Design parameters for the prototype bridge are based on standard practices followed by TxDOT. The specified concrete compressive strength at service f'_c is 8.5 ksi for precast girders and 4.0 ksi for the deck. The specified concrete compressive strength for precast girders at release f'_{ci} is 6.5 ksi. The prestressing steel consists of 0.6 in. diameter low relaxation strands. Additional design parameters are based on the AASHTO LRFD Bridge Design Specifications.[3]

DESIGN PHILOSOPHY

Load balancing with two-stage PT was used for the design of the continuous prestressed concrete bridge girders. The Stage I PT is applied individually to girders to balance the self-weight. Then, Stage II PT is carried out to balance the deck weight and superimposed dead load. The Stage II PT is applied after the deck is cast to provide compression in the deck and limit cracking due to the effect of live loads. Prestress losses are assumed to be 20 percent for pre-tensioning and 15 percent for post-tensioning. Important design considerations include handling, transportation, and erection. The construction sequence is also influenced by a number of factors and can vary from bridge to bridge.

CONSTRUCTION SEQUENCE

The sequence of construction is critical for the design of spliced girder bridges. Fig. 3 shows the details for the construction sequence assumed for the prototype bridge. The step-by-step construction sequence 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.
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.
d) 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.

HANDLING, TRANSPORTATION AND ERECTION

Handling, transportation and erection impose demands on the girders that must be considered in designing the continuous girder system. The design approach for the prototype bridge uses the following strategies and assumptions.

- The drop-in and end segments are transported from the precast plant to the construction site as simply supported girders. Pre-tensioning and Stage I PT are applied to balance the self-weight of the girders.
- The on-pier segment is transported with supports at the two quarter-span locations from the girder ends.
- For the on-pier segment, pre-tensioning and Stage I PT is applied to balance the self-weight and the reaction from the drop-in segment. Note that a significant amount of prestress force is required in the top flange of the on-pier segment because these segments are cantilevered from the piers and support the ends of the drop-in and end segments before Stage II post-tensioning is applied.
- Until the stage when the pier segment supports the drop-in girder segment, the stresses in the bottom flange are high. This is offset by providing temporary Dywidag bars in the bottom flange. Once the pier segment is installed on site, it behaves as a cantilever and the Dywidag bars can be released.

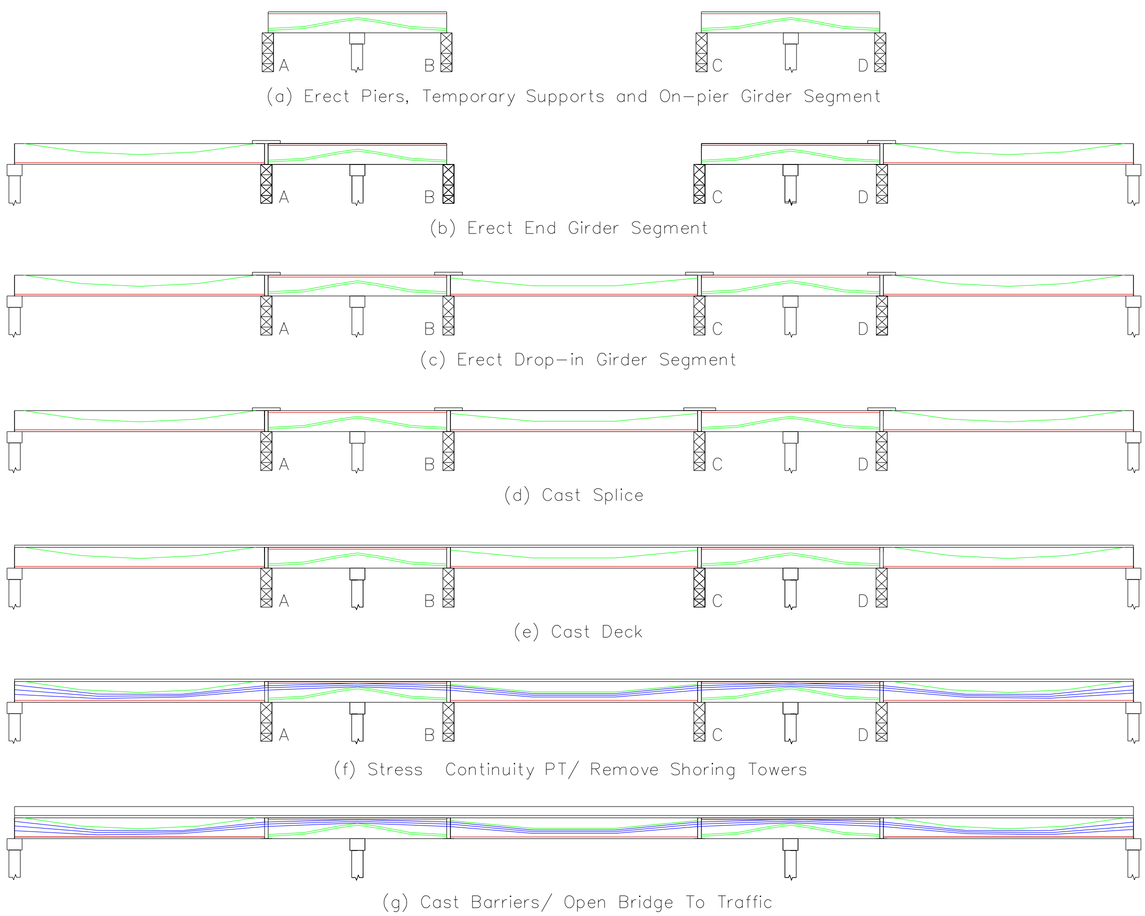


Fig. 3. Construction Sequence

ALLOWABLE STRESS ANALYSIS

Prestressed concrete bridges are designed to satisfy allowable stresses during fabrication, construction and at service. This prevents or limits cracking during various stages of construction. Service stress analysis is carried out under the effect of prestress, dead loads, live loads, and temperature and thermal gradients. For spliced girder bridges, stresses are checked during various stages of construction. Using the above considerations for the handling, transportation and construction; prestressing was selected for the three-span continuous girders with modified Tx70 shapes. Tables 2-4 summarize the prestressing details.

Table 2. Pre-tensioning Summary

Pre-Tensioning	End Segment	On-Pier Segment	Drop-in Segment
Strands (0.6 in. dia.)	32	26	24
Force at Transfer (kips)	1406	1142	1054
Force Final (kips)	1125	913	843

Table 3. Stage I Post-tensioning Summary

Post –Tensioning	End Segment	On-pier Segment	Drop-in Segment
Tendons (19-0.6 in. dia. strands per duct)	19 (1 duct)	38 (2 ducts)	19 (1 duct)
Force at Transfer (kips)	779	1558	779
Force Final (kips)	662	1324	662

Table 4. Stage II Post-tensioning Summary

Post –Tensioning	End Segment	On-pier Segment	Drop-in Segment
Tendons (19-0.6 in. dia. strands per duct)	57 (3 ducts)	57 (3 ducts)	57 (3 ducts)
Force at Transfer (kips)	2337	2337	2337
Force Final (kips)	1987	1987	1987

Several additional stress checks were made and are discussed in the following points.

- The spliced connection region of the girder reaches tensile stresses that exceed the allowable tensile stresses at service when carrying the weight of the CIP deck during the concrete pour. This stress exceedance is addressed by providing supplemental mild steel reinforcement in the connection.
- The compressive stresses in the girder soffit at the interior support were exceeded due to the large amount of PT tendons in the section. This stress exceedance is addressed by providing mild steel reinforcement in the compression zone. For this design, 16-#14 bars are added in the bottom flange of the girder to improve the nominal capacity of the section as specified in the ultimate strength check. This additional mild steel reinforcement is also adequate to serve as reinforcement in the girder soffit at the interior support over the pier for the computed stress exceedance at service load conditions.

- The deck over the pier experienced tensile stresses for the service condition. While it is desirable to avoid tensile stresses in the deck, the maximum stress is below the allowable tensile stress limit.

ULTIMATE STRENGTH DESIGN

The flexural strength limit state must be checked to ensure safety at ultimate load conditions. The moment capacity that the pretensioning and PT 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 are added in the bottom flange of the girder to provide the additional capacity and balance the negative moment demand at the interior support over the pier. The mild steel reinforcement provided in the bottom flange acts as compression steel. Table 5 summarizes the moment capacity and demand for the three-span bridge.

Table 5. Ultimate Demand and Capacity

Demand and Capacity	End Span	Over Pier	Interior Span
Demand, M_u (kip-ft)	14,950	20,690	15,340
Capacity, ϕM_n (kip-ft)	25,170	24,190	26,320

DESCRIPTION OF FUTURE EXPERIMENTAL PROGRAM

SIGNIFICANCE

The performance and cost-effectiveness of a continuous spliced precast prestressed concrete girder system depends largely on the design and construction details of the girders and splice connections. Based on literature review and preliminary design examples, it was noted that sequencing of the CIP concrete, PT operations and time-dependent issues are important construction considerations to be included in the design phase.[2] A partially prestressed solution having a balance of constructability and long-term serviceability was selected to be the most desirable alternative for the splice connections.[2] In addition, the design of the anchorage zone is complicated due to the disturbed region ahead of the anchorage device.

This project includes a significant experimental program that will provide important insight into the aforementioned design, construction and performance limiting criteria. A full-scale girder specimen will be tested. This experiment will highlight the design, detailing and construction issues related to the splice connection and post-tensioned anchorage zones of the specimen at service levels and then to ultimate strength. It will also allow an evaluation of the girder performance under negative bending over the support and to determine whether failure will occur at the splice or outside the splice. The results of this test will inform designers as to whether the splice connection details provide adequate performance for serviceability and ultimate strength.

SPECIMEN EXTRACTION

The experimental setup to evaluate the performance of the splice connection details is extracted from the prototype bridge. Fig. 4 shows the extraction of the specimen from the prototype bridge. The splice in the end span of the prototype bridge will be tested. Because the specimen is load-balanced for dead load, live load with impact is the only load producing maximum actions at the service limit state.

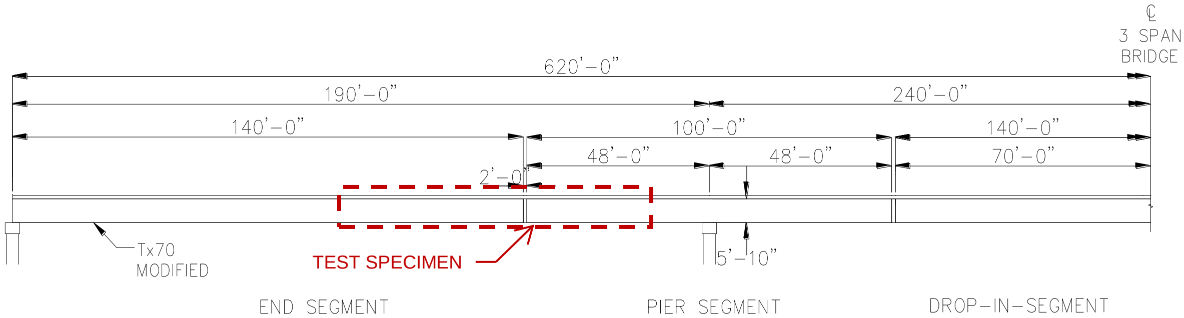


Fig. 4. Side View of Prototype Bridge Girder Showing the Test Specimen

Different options were developed and assessed by the research team to arrive at the final specimen design. Fig. 5 shows the elevation of the final test-setup with the location of tie downs, pedestals, and splices. An interior splice is provided corresponding to the in-span splice in the prototype bridge. However, due to the lifting weight limitation of the crane in the laboratory, two additional splices were provided connecting the prismatic girder segments to the thickened girder ends. The research team will take advantage of the additional splices by investigating different combinations of shear and moment at all three connections.

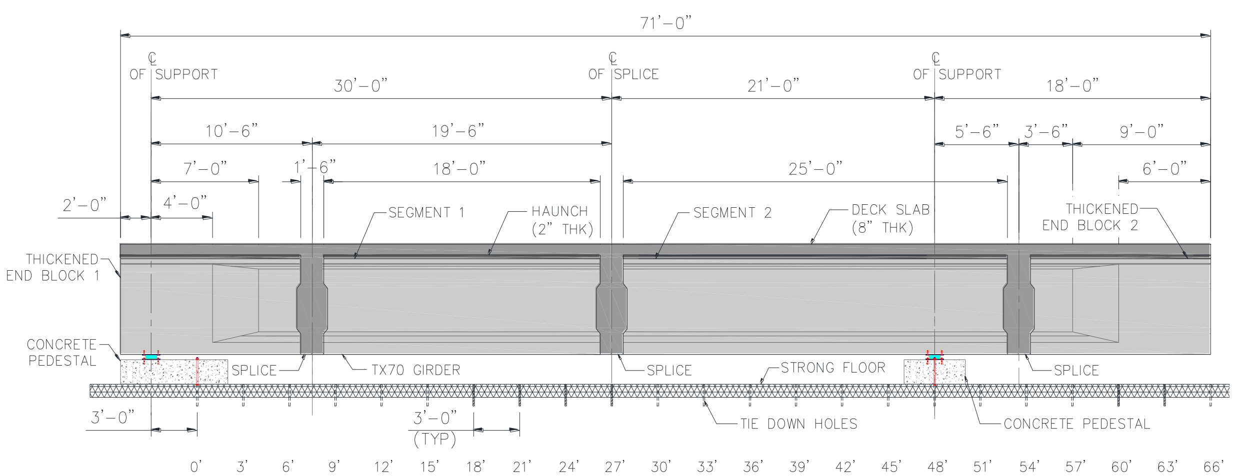
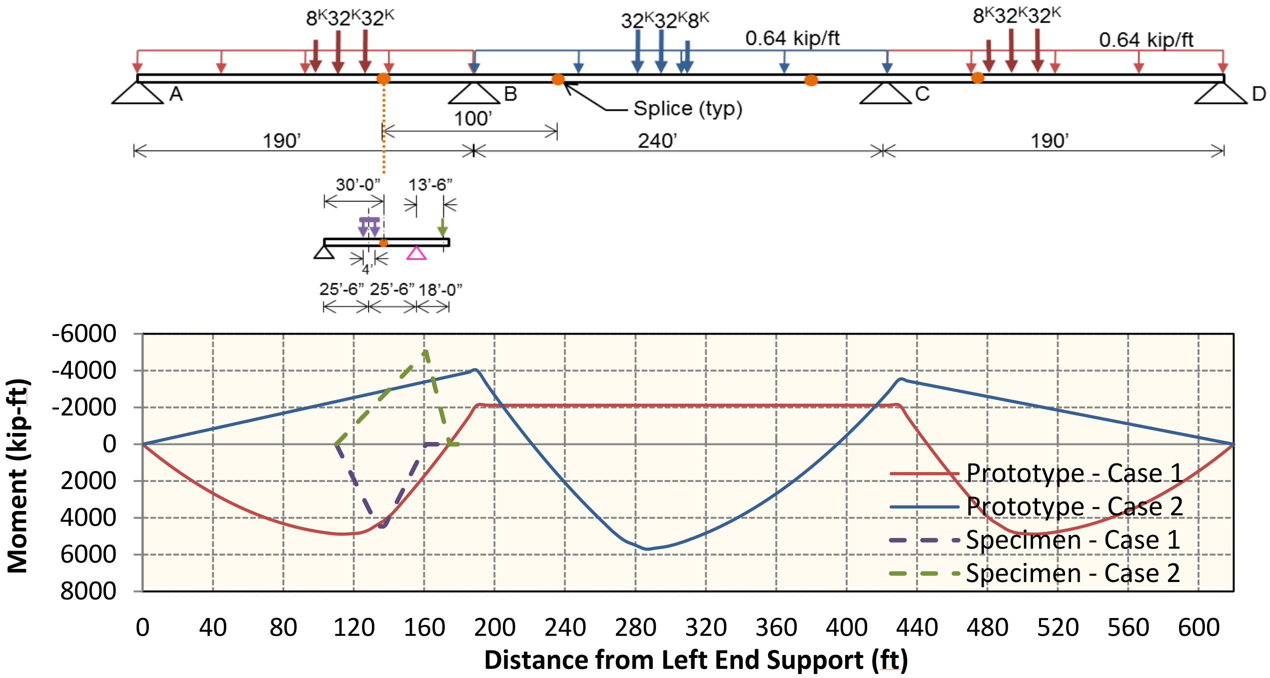


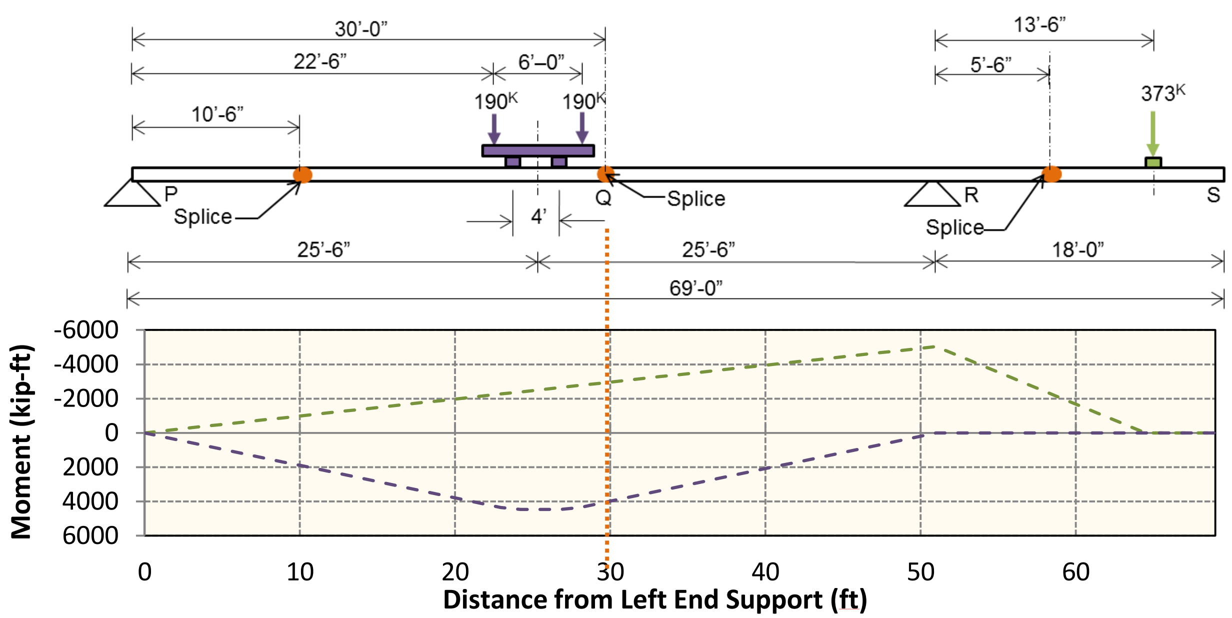
Fig. 5. Side View of Test Specimen

The moment and shear demand under live load with impact at service corresponding to the splice location of the prototype bridge are replicated in the specimen as shown in Fig. 6 and

Fig. 7. The corresponding loads to be applied on the actuator in the experimental setup are determined from this analysis.

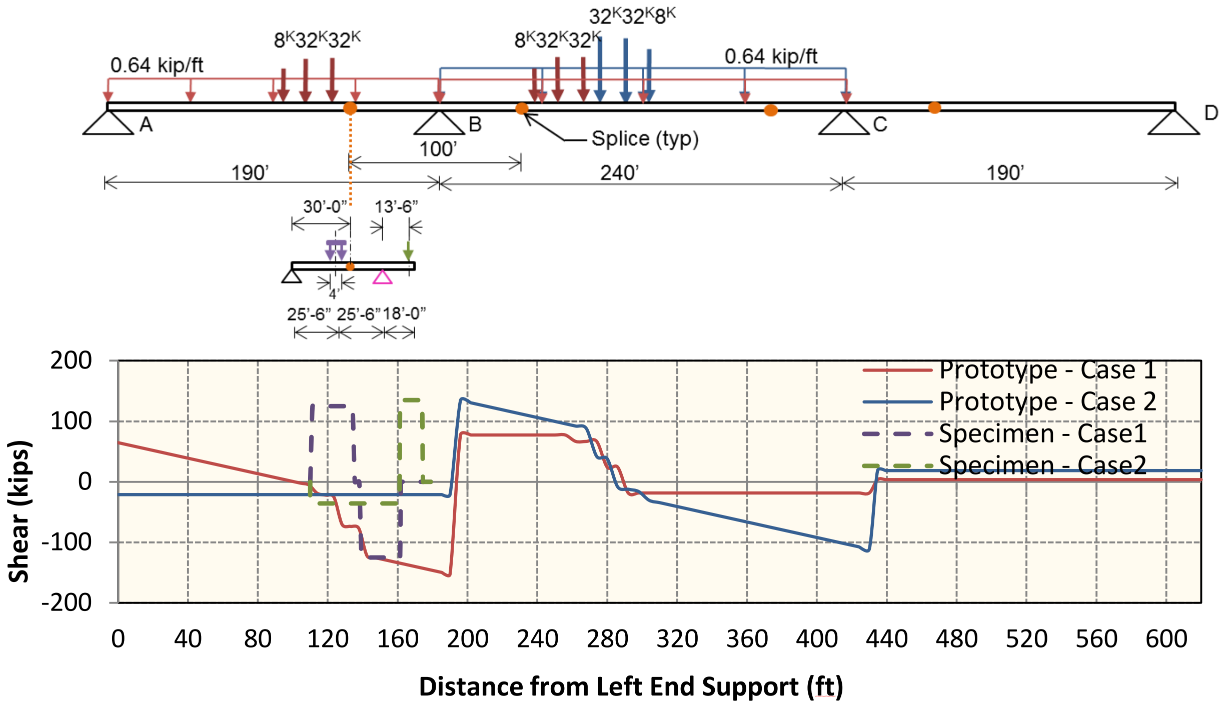


(a) Maximum Service Moment under Live Load with Impact from Prototype Bridge

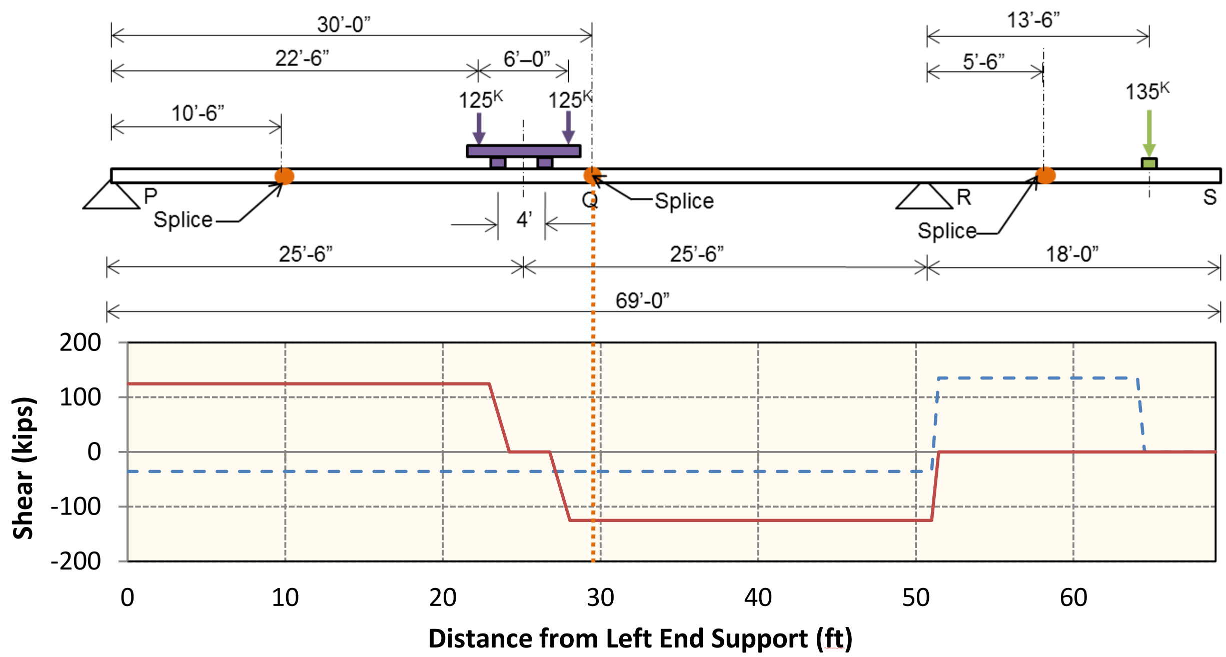


(b) Experimental Setup

Fig. 6. Replicating Maximum Moment at Service at the Interior Splice from Prototype Bridge



(a) Maximum Service Shear under Live Load with Impact from Prototype Bridge



(b) Experimental Setup

Fig. 7. Replicating Maximum Shear at Service at the Interior Splice from Prototype Bridge

The basic characteristics of the specimen are as follows.

- A modified Tx70 girder cross-section will be used for the specimen to match the prototype bridge at full-scale.
- Thickened end blocks that gradually taper to the girder cross-section are provided at both ends of the specimen to accommodate the necessary PT anchorage systems. The girder segments and end blocks will be fabricated at a precast plant and transported to the laboratory.
- The concrete used for prestressed concrete girder segments is specified as TxDOT Class H self-consolidating concrete with a required initial compressive strength at release f'_{ci} of 6.5 ksi and required compressive strength f'_c of 8.5 ksi at service.
- A partially prestressed splice connection detail is used at all three splice locations. Mild steel reinforcement is provided in addition to continuity PT running through the connection. The mild steel reinforcement consists of 180-degree bent hooked bars anchored into the adjacent girder flanges and extending into the joint.
- The concrete used for the splice connections is the same as that used for the precast girder segments. TxDOT Class H self-consolidating concrete with a required initial compressive strength at release f'_{ci} of 6.5 ksi and required compressive strength f'_c of 8.5 ksi at service will be used.
- The continuity PT tendons will be installed, stressed and grouted in the laboratory by a post-tensioning contractor.
- A reinforced concrete CIP deck slab, 8 ft wide and 8 in. thick, will be cast in the laboratory. Conventional concrete with a specified 28-day f'c of 4 ksi will be used for the deck slab. Deck reinforcement details for the specimen are provided in accordance with TxDOT construction practice. Typical clear cover provided is 2 in. and 1.25 in. for top and bottom reinforcement, respectively.

SPECIMEN COMPONENT DESIGN AND DETAILS

GIRDER SEGMENT DESIGN

Design by Load Balancing

The girder specimen is load-balanced for all the dead loads. The amount of steel provided in the girder segments is similar to that provided in the prototype bridge. However, the Stage I PT provided in the prototype is replaced with pre-tensioning in the specimen to avoid thickened girder segment ends at the splice location. This provides a more critical section through the splice connection because only the web thickness is available. Table 6 presents the design summary for the specimen.

The load-balancing design moments at each stage and service stresses are checked. In the load-balancing approach, the girder segments are designed such that the prestress moments in the girders are balanced with the loading stages throughout the loading history of the specimen construction. The different loading and design stages to which the girder is subjected are as follows.

- Girder Section
  - Self-weight + Pretensioning
  - Self-weight + Pretensioning + Non-composite Deck Weight
- Composite Girder and Deck Section
  - Stage 2 PT + Deck weight
  - Stage 2 PT + Deck weight + Removal of Temporary Shoring Towers

Table 6. Design Summary for Specimen

Design Parameters		Segment 1 and Thickened End Block 1	Segment 2 and Thickened End Block 2
Pre-tensioning	Strands (0.6 in. dia.)	34	26
	Force at Transfer, F_{1i}	1494 kips	1143 kips
	Eccentricity, e_1	27 in.	34.6 in.
Stage 1 PT (Replaced with pre-tensioning in specimen)	Tendons (19-0.6 in. dia. strands per duct)	26	34
	Force at Transfer, F_{2ai}	1143 kips	1494 kips
	Eccentricity, e_{2a}	34.6 in.	27 in.
Stage 2 PT	Tendons (19-0.6 in. dia. strands per duct)	57 (3 ducts)	57 (3 ducts)
	Force at Transfer, F_{2bi}	2337 kips	2337 kips

Flexure Considerations

Service load evaluations will be carried out for the Tx70 girder specimen at various stages of construction and under the total live loads with impact effects simulated by hydraulic actuators in the laboratory. The specimen will also be loaded to ultimate conditions to verify adequacy for the flexural strength limit state.

The factored moment capacity of the girder section and the splice is compared with the factored moment demand under full dead load and full live load effects. Ductility in the on-pier region of the specimen was also assessed. For the critical section in the on-pier region of the specimen, the tensile strain ε_t in the extreme tension steel at nominal strength is equal to 0.005. According to AASHTO LRFD Article 5.5.4.2.1, this is a tension-controlled section and the resistance factor ϕ is taken equal to 1.[3]

Shear Considerations

Transverse and interface shear design for the specimen are the same as that for the prototype bridge. Modified compression field theory is used for the transverse shear design as specified in the AASHTO LRFD Bridge Design Specifications.[3] Table 7 presents shear design details for the specimen. Due to the high amount of shear reinforcement required in the Tx70 girder specimen, the principal tensile stresses in the web were reviewed. The AASHTO LRFD Bridge Design Specifications Article 5.8.5 requires checking the principal tension stress to verify the adequacy of the webs of segmental concrete bridges for longitudinal shear and torsion.[3] This article states that when investigating principal stresses for the Service III limit state at all stages during the life of the structure and during construction, the tensile stress limits of Table 5.14.2.3.3-1 ($0.11\sqrt{f_c'}$) shall apply. The principal tensile stress using classical beam theory and the principles of Mohr's Circle is checked at the critical sections of the girder specimen over the pier and at the three splice locations as shown in Table 8.

Table 7. Shear Design Details for Specimen.

Girder Segment	Shear Reinforcement
Thickened End Block 1	34 - #5 stirrups @ 4 in. spacing
Splice near End Support	5 - #5 stirrups @ 4 in. spacing
Segment 1	9 - #5 stirrups @ 4 in. spacing 29 - #5 stirrups @ 6 in. spacing
Interior Splice	4 - #5 stirrups @ 6 in. spacing
Segment 2	19 - #5 stirrups @ 6 in. spacing 46 - #5 stirrups @ 4 in. spacing
Splice in Overhang	5 - #5 stirrups @ 4 in. spacing
Thickened End Block 2	34 - #5 stirrups @ 4 in. spacing

Note: All shear reinforcement consists of double legged stirrups.

Table 8. Principal Tensile Stress Calculation for Specimen.

Critical Location	Principal Tensile Stress at Service (considering V_p) (ksi)	Principal Tensile Stress Limit ($0.11\sqrt{f_c'}$) (ksi)
Interior Support	0.039	0.321
Splice near End Support	0.015	0.321
Interior Splice	0.066	0.321
Splice in Overhang	0.095	0.321

The AASHTO LRFD Bridge Design Specifications Article 5.8.5 specifies that the vertical force component of draped longitudinal tendons, $\boldsymbol{V_p}$ shall be considered as a reduction in the shear force due to the applied loads.[3] Also, from a load-balancing approach, the total dead load of the girder and deck slab is balanced by the prestressing and PT tendon profiles. It was found that the principle tension stress values considering $\boldsymbol{V_p}$ were below the allowable limit of $\mathbf{0.11}\sqrt{f'_c}$.

SPLICE DESIGN

The interior splice in the specimen replicates the splice located in the end span of the prototype bridge. This splice location corresponds to the dead load point of contraflexure in the prototype bridge so as to minimize the load demands. The width of the splice connection should be kept as small as possible since there is no pre-tensioning in this region and a minimal amount of mild steel reinforcement is provided. However, the splice width should be large enough to splice the continuity PT tendon ducts and allow for proper vibration of concrete. The width of the splice connection detail was kept equal to 24 in. (2 ft). Fig. 8 shows the splice connection detail used for the test specimen.

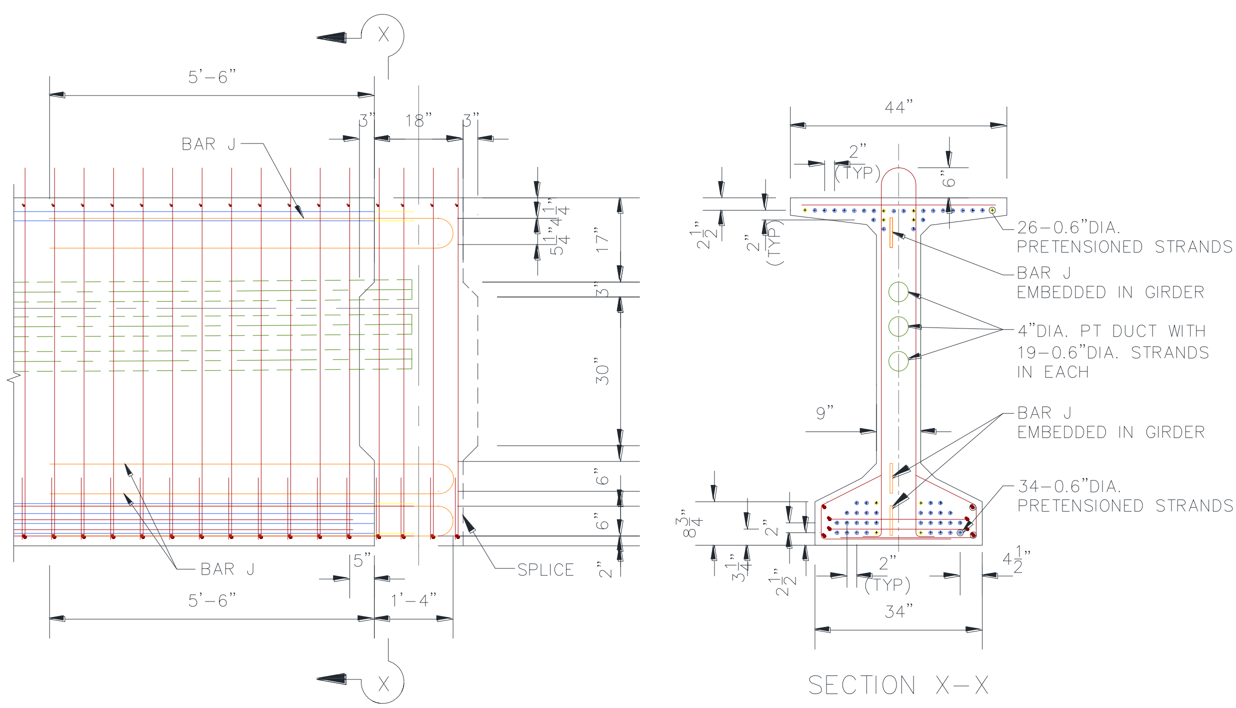


Fig. 8. Splice Detail Used for Test Specimen

A partially prestressed splice connection detail is used at all three splice locations. Mild steel reinforcement is provided in addition to continuity PT running through the connection. The mild steel reinforcement consists of 180-degree bent hooked bars anchored into the adjacent girder flanges and extending into the joint. The mild steel bent bars are designed for the maximum factored design loads. The combination of PT and mild steel is expected to provide better durability and performance. Vertical reinforcement is provided to strengthen the splice connection for shear as presented in Table 7. The integrity of the splice connection largely depends on the shear transfer mechanism at the interface of the precast girder and closure pour. This shear transfer mechanism is mainly provided by the lapped 180-degree bent hooked bars in the connection and roughened edges of the precast girder at the interface.

SUMMARY AND CONCLUSIONS

Spliced precast concrete girder bridges provide a cost-competitive alternative to steel and segmental concrete bridges for span lengths between 150 ft to 300 ft. The design of a prototype bridge presented in this paper helps to identify the maximum span limits, design and construction limitations and the effect of construction sequence with respect to standard precast shapes currently used by TxDOT for prestressed concrete bridges. In-span spliced girder technology has helped to extend the span length of the prototype bridge beyond the typical span to depth ratios without having to exceed the typical transportable lengths of girder segments. The splice within the span of this bridge is critical because there is no pre-tensioning across the joint and minimum PT. The full scale experimental testing will help to highlight the design, detailing and construction issues related to this partially prestressed splice connection and PT anchorage zones of the specimen. The results of this test will determine whether failure will occur at the splice or outside the splice and evaluate as to whether the selected splice connection detail provides adequate structural performance for serviceability and ultimate strength.

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