END-REGION SERVICEABILITY AND SHEAR STRENGTH OF PRECAST, PRETENSIONED I-GIRDERS EMPLOYING 0.7-IN. DIAMETER STRANDS

Hyun su Kim, The University of Texas at Austin, Austin, TX
Alex Katz, The University of Texas at Austin, Austin, TX
Jessica Salazar, The University of Texas at Austin, Austin, TX
Roya Abyaneh, The University of Texas at Austin, Austin, TX
Hossein Yousefpour, Ph.D., The University of Texas at Austin, Austin, TX
Trevor Hrynyk, Ph.D., The University of Texas at Austin, Austin, TX
Oguzhan Bayrak, Ph.D., P.E., The University of Texas at Austin, Austin, TX

ABSTRACT

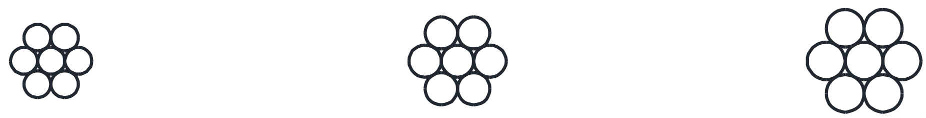
The use of 0.7-in. strands in place of 0.5- or 0.6-in. strands is believed to reduce the number of required strands and increase the span capability of pretensioned girders. However, greater end-region stresses in girders employing these larger-diameter strands may lead to undesirable cracking at release and atypical failure modes. The work presented in this paper aims to investigate the serviceability and shear strength of Tx-girders that are fabricated using 0.7-in. strands on a 2- by 2-in. grid. A series of full-scale specimens was fabricated at the Ferguson Structural Engineering Laboratory. The specimens were subjected to prestress transfer at concrete compressive strengths commonly used in practice, and end-region stresses, end-region cracking, transfer length, and strand slip were carefully monitored. To evaluate the load-resisting performance, shear-critical loading was also applied to the specimens until failure. The testing program was carried out in conjunction with parametric studies to identify and maximize the benefits of using 0.7-in. strands as well as nonlinear finite element analyses in ATENA-3D to evaluate the mechanical performance of the girders. The results presented in this paper provide valuable insight into the influences of 0.7-in. strands on the behavior of pretensioned girders at prestress transfer and under applied shear.

Keywords: 0.7-in. Strands, Prestressed Concrete, I-Girders, Horizontal Shear, Anchorage Failure, Prestress Release

INTRODUCTION

Currently, 0.5- and 0.6-in. diameter prestressing strands are used in the precast industry for fabricating bridge girders. However, in recent years, there has been interest in using 0.7-in. strands in pretensioning applications. As shown in Fig.1, the cross-sectional area of a 0.7-in. strand is approximately twice that of a 0.5-in. strand and is about 35 percent greater than that of a 0.6-in. strand. As a result, considerably fewer 0.7-in. strands would be needed to provide the same steel area for the girder compared to 0.5- or 0.6-in. strands. Therefore, the time and cost of the fabrication process for the pretensioned girders could be reduced.

With the use of 0.7-in. strands, it is also possible to concentrate a greater steel area near the bottom of the precast cross sections, leading to a potential increase in the internal lever arm and therefore the flexural capacity of the cross sections. Depending on the design objectives, such an increase in the flexural capacity of girders can be used to increase the span capability of existing precast cross sections, increase the transverse spacing between the girders, or reduce the depth of bridge superstructure. These benefits could improve the capabilities of pretensioned bridge girders in terms of fabrication time and cost, competitiveness with steel bridge girders, and aesthetics.



$A_{ps} = 0.153\ in^2$ $A_{ps} = 0.217\ in^2$ $A_{ps} = 0.294\ in^2$

(a) 0.5-in. strand (b) 0.6-in. strand (c) 0.7-in. strand

Fig. 1 Cross section of different strand sizes

Despite the benefits associated with 0.7-in. diameter strands, the only real-world applications of pretensioned girders with these larger-diameter strands in the United States have been limited to the Pacific Street Bridge and the Oxford South Bridge in Nebraska, which were opened to traffic in 2008 and 2013, respectively. In fact, there has been a general apprehension toward the use of the larger-diameter strands because of the potential need for upgrading the prestressing facilities, concerns related to the availability of materials and accessories, and concerns regarding the structural behavior of girders that employ 0.7-in. strands.

The impacts of using 0.7-in. strands on the fabrication plants will be minimized if these larger-diameter strands are used on the standard 2- by 2-in. grid. However, current reinforcement detailing used for precast, pretensioned girders has been developed based on the use of 0.5- and 0.6-in. strands, and its suitability for 0.7-in. strands has yet to be verified.

The most crucial concerns regarding the structural behavior of girders that employ 0.7-in. strands on a 2- by 2-in. grid include: (1) possibility of undesirable cracking within the girder end regions, which might negatively affect the serviceability of girders, and (2) possibility of

diminished shear strength due to atypical failure modes.
At prestress transfer, the end regions of pretensioned girders are subjected to transverse stresses that are categorized as bursting, spalling, and splitting stresses. These stresses might result in cracking near the end-faces of the girders after the prestressing strands are released. Fig. 2 shows the primary locations of end-region stresses and the potential cracking that might happen as a result of each type of stress.

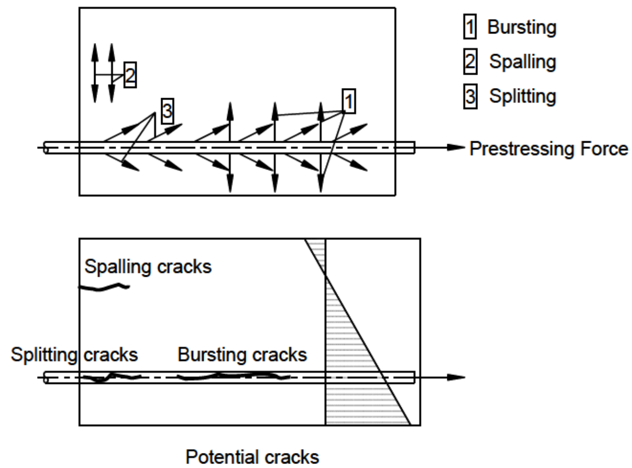


Fig. 2 End-region stresses and potential cracks formed after prestress transfer[1]

Bursting cracks are primarily related to the magnitude of force in the bottom flange, and as shown in Fig. 2, form along the strands. Spalling cracks are associated with the eccentricity of the strands or the distance between the centroid of the strands and the geometric centroid of the cross section. These cracks are usually localized near the end face of the beam at some distance from the strands within the cross section. Splitting cracks form near the end face and are a result of the radial compressive stresses that are generated as the prestressed strand returns to its original diameter by compressing the surrounding concrete, a mechanism commonly referred to as Hoyer's effect[2]. To ensure the serviceability and safety of precast pretensioned girders, it is essential to use appropriate detailing within the end regions of girders so that the width of aforementioned cracks is controlled.

Atypical shear failure modes are another important concern regarding the use of 0.7-in. strands. The web-flange interface regions of pretensioned concrete girders are prone to high stress concentrations. This situation can lead to horizontal shear failure, especially in pretensioned sections with relatively thin webs. A horizontal shear failure occurs when there is a loss of strain compatibility along a horizontal plane within the beam. Current design specifications for shear strength are not developed to take this atypical failure mode into account and therefore, there is a potential for unconservative shear strength estimates when this mode controls. Increased web-flange interface stresses, which could potentially develop in girders employing large-diameter strands, increase the likelihood of this failure mode and therefore need to be considered in different pretensioned cross sections.

End regions of prestressed concrete girders constructed with larger-diameter strands are also more likely to experience anchorage-induced shear failures. When a shear crack first initiates near the edge of the bearing pad, it is prevented from opening further by the restraining force provided by the prestressing strand. If the steel crossing the crack does not provide an adequate amount of restraining force (e.g., when proper development of the strands is impeded by anchorage zone distress) further progression of the diagonal shear cracking in this region will occur. The use of 0.7-in. diameter strands is expected to result in increased transfer lengths, hence reducing the restraining force available near the supports of pretensioned girders. Unless proper end-region reinforcement detailing is provided in girders employing 0.7-in. diameter strands, end-region stress development attributed to the large-diameter strands could potentially result in anchorage zone distress and yield shear strengths that fall below code estimates.

The structural behavior of full-scale girders with 0.7-in. strands has been investigated in relatively few studies. Vadivelu[3] studied the transfer length and end-region cracking in an AASHTO Type I girder that was fabricated using 0.7-in. strands. The prestress transfer was conducted at a concrete compressive strength of 10 ksi. The transfer length was found to be shorter than that predicted by the AASHTO LRFD 2008[4] and ACI 318-08[5] provisions. Cracks were observed in the bottom flange of the girder within a 2-in. distance from the end face. To reduce end-region cracking, it was suggested that greater ratios of confining steel be used within the regions near girder end faces.

Tadros and Morcous[6] investigated the transfer and development lengths, and flexural and shear performance of NU1100 girders and 24-in. deep T-girders that were fabricated using concrete release strengths of 8 and 9 ksi, respectively. The results showed that the development length predicted by AASHTO LRFD equations was sufficient for the beams to reach their flexural capacity. However, the transfer length was reported to be considerably less than that predicted using AASHTO LRFD equation. The shear failure load of the specimens exceeded the capacity predicted using AASHTO LRFD equations. Similar observations were made from a bridge double tee section and a NU900 girder that were fabricated using ultra high performance concrete and a compressive release strength of 12 ksi.

Morcous et al.[7-8] investigated the performance of pretensioned girders in the Pacific Street Bridge and the Oxford South Bridge in Nebraska. NU900 girders were used in the Pacific Street Bridge, with a 2- by 2.5- in. spacing grid between the 0.7-in. strands. The Oxford South Bridge included NU1350 girders with 0.7-in. strands that were used on the standard 2- by 2-in. grid. The compressive release strength was 7 ksi for the Pacific Street Bridge and between 6 and 10 ksi for the Oxford Street Bridge. In both projects, satisfactory performance was reported for the girders at release, indicating satisfactory performance of AASHTO LRFD provisions for bursting and confinement reinforcement in girders using 0.7-in. diameter strands.

In addition to the studies mentioned above, a series of studies has been performed by Dang et al.[9-10] on the transfer and development lengths of 0.7-in. diameter strands using small (6.5- by 12- in.) specimens that were reinforced with only one strand. The transfer length was found to be shorter than that recommended by the provisions of ACI 318-14[11] and the AASHTO LRFD specifications[13]. The development length was also found to be considerably shorter than the lengths estimated using ACI 318-14 provisions.

While these past efforts represent valuable contributions toward improving the current understanding of the behavior of precast pretensioned girders with 0.7-in. diameter strands, significant gaps remain in the knowledge regarding the performance of such members. Small-scale test specimens possess unrealistic strand spacing and boundary conditions, and the applicability of the results from such specimens to pretensioned girders is questionable. Further, the full-scale beams that have been investigated experimentally were subjected to prestress transfer at concrete release strengths considerably greater than what is used in common practice. Due to these discrepancies, the behavior of the specimens and the observed parameters of interest (e.g. transfer length) may not be indicative of the performance of actual pretensioned girders used in the field. Additionally, impacts of using 0.7-in. diameter strands on the shear strength, which is one of the primary concerns associated with the use of these large-diameter strands, have not been sufficiently investigated in the studies noted above.

This paper provides an overview of perhaps the most comprehensive study on the serviceability and shear strength of girders that employ 0.7-in. strands, which is currently underway at The University of Texas at Austin. The research program includes fabrication, release, and structural testing of full-scale Tx-girder specimens to evaluate end-region serviceability and shear strength, nonlinear finite element analysis (FEA) to provide insight into the mechanics of pretensioned girders employing 0.7-in. strands, and parametric studies to identify the potential benefits and limitations associated with the use of these larger-diameter strands.

PARAMETRIC INVESTIGATION

A comprehensive parametric study was conducted on a variety of I-, bulb-tee, U-, and box-sections to investigate the benefits and limitations of using 0.7-in. strands within precast, pretensioned girders. Thousands of design cases were evaluated using a parametric study tool that was originally developed by Garber et al.[12] at the University of Texas at Austin.

The parametric study tool was designed such that typical design parameters could be easily varied to produce a range of full girder designs. The tool accounted for the design flexural and shear strengths at ultimate conditions, as well as other limits defined within AASHTO LRFD 2014[13] on stresses at the times of prestress transfer, deck placement, and under live load. The stresses are checked at three locations along the girder: midspan, forty percent of the length, and at the estimated transfer length.

The tool was modified to satisfy the requirements of the current study and also to reflect the most current design provisions from 2016 interim revisions to AASHTO LRFD 2014 Bridge Design Specifications. The results obtained from this parametric study tool have also been

verified using PGSuper[14].
A variety of parameters and precast concrete cross sections have been investigated in this parametric study. For brevity, only the number of strands that are required for different span lengths of Tx46 and Tx70 cross sections are presented in the results section of this paper.

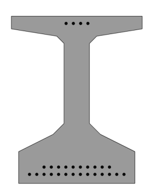
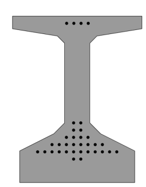
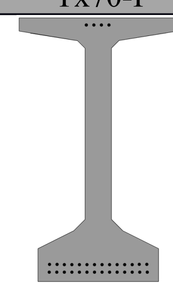
EXPERIMENTAL PROGRAM

As part of the research program at the University of Texas at Austin, six full-scale specimens with 0.7-in. strands are to be fabricated and tested to evaluate their performance at the time of prestress transfer and under shear-critical loading. The first three specimens in this test program, which have been fabricated and tested to date, are presented in this section.

SPECIMEN DESIGN

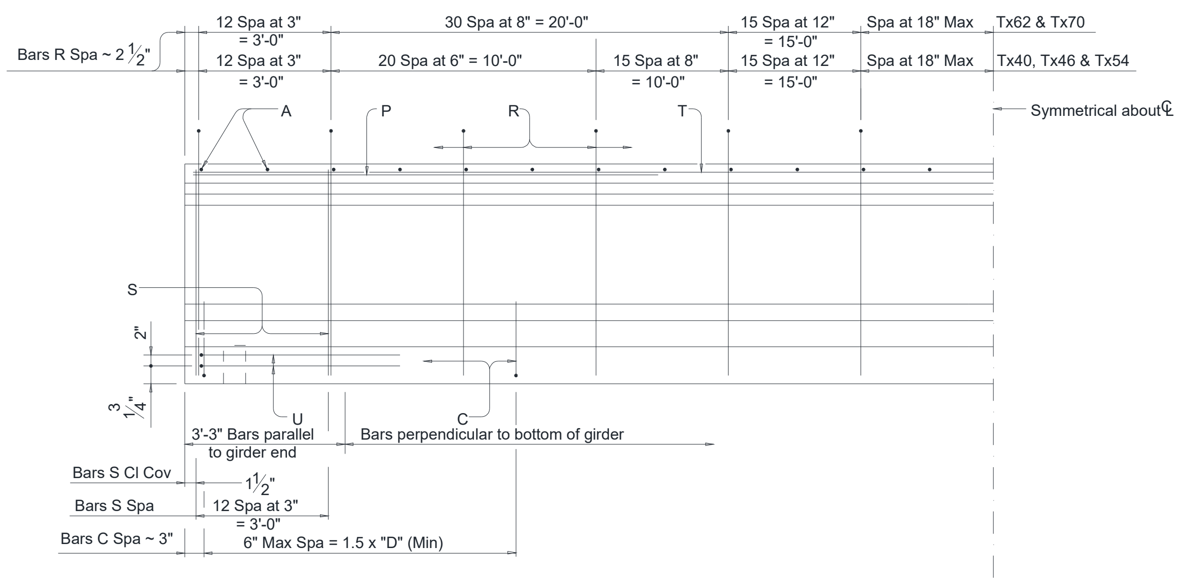
A summary of the design parameters for the first three specimens within the research program is shown in Table 1. All three specimens were designed using the current detailing standard for Tx-girders to evaluate the performance of existing end-region detailing when it is used for the larger-diameter 0.7-in. strands on a 2- by 2-in. grid. The specimens had a total length of 30 ft and were fabricated using the prestressing facility at the Ferguson Structural Engineering Laboratory (FSEL).

Table 1 Summary of the design parameters for the specimens

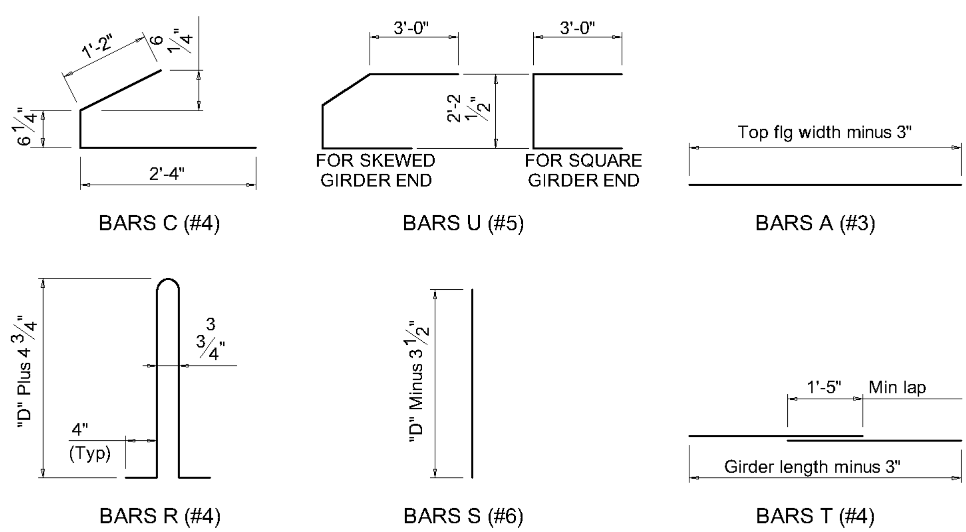
Design properties	Tx46-I	Tx46-II	Tx70-I
Strand pattern			
Design objective	Maximizing the eccentricity	Maximizing the prestressing force	Maximizing the eccentricity
Design f'_{ci} [ksi]	5.5	5.5	5.5
Effective depth from top* [in.]	44.7	37.6	66.5
Number of top strands	4	4	4
Number of bottom strands	24	30	28
Design [ksi]	270 (245)	270 (245)	270 (245)
Jacking stress for bottom strands [ksi]	202.5	202.5	202.5
Jacking stress for top strands [ksi]	157.5	202.5	110.0

* Deck not included

The specimens were intended to represent the critical conditions with regard to bursting and spalling stresses. Tx46-I was designed to maximize the spalling stresses by maximizing the number of strands in the bottommost rows of the section and therefore producing the greatest possible eccentricity in a Tx46 section. Tx46-II was designed to maximize the bursting stresses by placing the maximum number of strands in the bottom flange without exceeding the allowable stress limits at release. Specimen Tx70-I was designed to investigate the maximum spalling stresses for the deepest Tx-girder and therefore the greatest eccentricity possible in the whole family of Tx-girders. Bridge Design Manual[15] by the Texas Department of Transportation (TxDOT) was used for designing all of the specimens



(a) Details of reinforcing bars



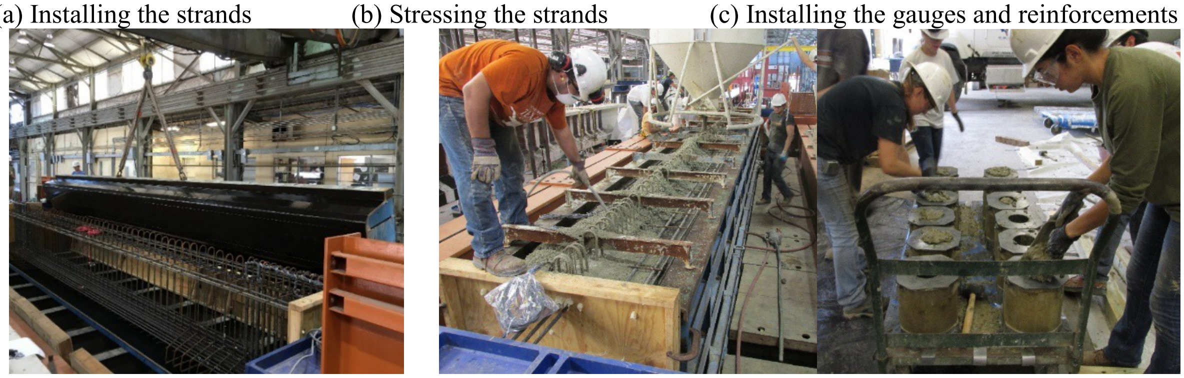
(b) Reinforcing bars schedule

Fig. 3 Standard TxDOT detailing for end-region reinforcement in Tx-girders[16]

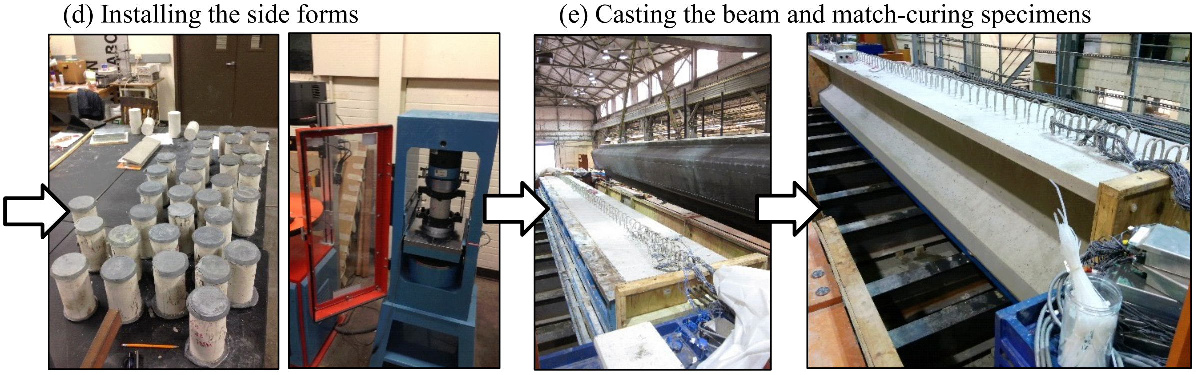
SPECIMEN FABRICATION AND PRESTRESS TRANSFER

Fig. 4 shows the sequence through which the pretensioned girders with 0.7-in. strands were fabricated. The entire construction process, which included installation and stressing of strands, installation and instrumentation of the transverse reinforcement, and concrete placement, was conducted by the research team at FSEL. In-house fabrication of the pretensioned girders provided unlimited access for instrumentation and firsthand assessment of constructability issues and concerns.

(a) Installing the strands (b) Stressing the strands (c) Installing the gauges and reinforcements



(d) Installing the side forms (e) Casting the beam and match-curing specimens



(f) Conduction compressive strength tests on match-curing specimens (g) Removing the side forms (h) Prestress release

Fig. 4 Fabrication sequence for precast, pretensioned girders with 0.7-in. strands at FSEL

To prestress 0.7-in. strands, modifications were made to the prestressing facility, including drilling larger holes in stressing plates and fabricating a new frame for prestressing the top strands. The chucks that were used for 0.7-in. strands were also considerably larger than those used for 0.6-in. strands. However, the strands could still be used on the standard 2- by 2-in. grid without any constructability issues. No unusual difficulties were observed when handling, unreeling, and stressing of the 0.7-in. strands.

Concrete mixtures with Type III cement were used for all specimens for consistency with the common practice in the pretensioning industry. The concrete for the first two specimens was batched and mixed in house whereas the third specimen was cast using the concrete batched and mixed by Coreslab Structures in Cedar park, Texas.

A series of match-curing specimens was cast and cured at temperatures measured from the beam using thermocouples. The compressive strength of these specimens was carefully monitored in the hours following the concrete placement. Once the desired compressive strength was achieved, the prestressing strands were released by retracting the hydraulic rams applying the prestressing force. Specimen strains were extensively monitored in the end-region reinforcement, and end-region cracking was documented. Further, extensive material tests were conducted to obtain the compressive strength and modulus of elasticity of the concretes comprising the specimens at the time of prestress transfer.

Table 2 shows a summary of the concrete material properties for the first three specimens within the test program. As can be seen in this table, the actual release strength for Tx46-I and Tx70-I was greater than 5.5 ksi that was assumed for their design.

Table 2 Summary of concrete material properties at prestress transfer

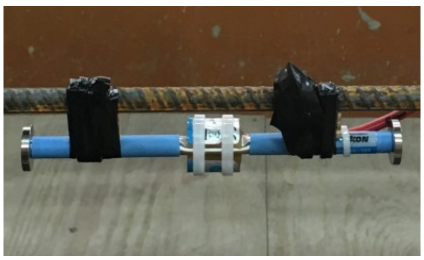
Variables	Tx46-I	Tx46-II	Tx70-I
Concrete Release Strength, f'_{ci}	5.7 ksi	5.2 ksi	6.5 ksi
Modulus of Elasticity of Concrete (E_c)	-	4,940 ksi	4,900 ksi

A variety of instruments were used to monitor the specimen performance at release. Typically, 90 strain gauges were installed on the stirrups within the end region to measure the strains due to bursting, spalling and splitting stresses (Fig. 5(b)). Another 90 strain gauges were installed on the strands to estimate the transfer length (Fig. 5(a)). In addition, three vibrating wire strain gauges (Fig. 5(c)) were embedded in the concrete at the midspan locations of the girders to estimate prestress losses throughout the life of the specimens.

At the time of prestress transfer, a series of linear potentiometers (LPOTs) at midspan and each end of the specimen was used to measure camber. Linear strain conversion transducers (LSCTs), shown in Fig. 5(d), were also installed on some of the strands to measure strand slip. Lastly, an optical tracking system, consisting of two 3-D vision cameras and a set of LED targets that were attached to the specimen, was used to provide a secondary measurement of transfer length.

(a) Strain gauges on strands

(b) Strain gauges on transverse steel



(c) Vibrating wire gauge

(d) LSCTs at end faces

Fig. 5 Instrumentation used at the time of prestress transfer

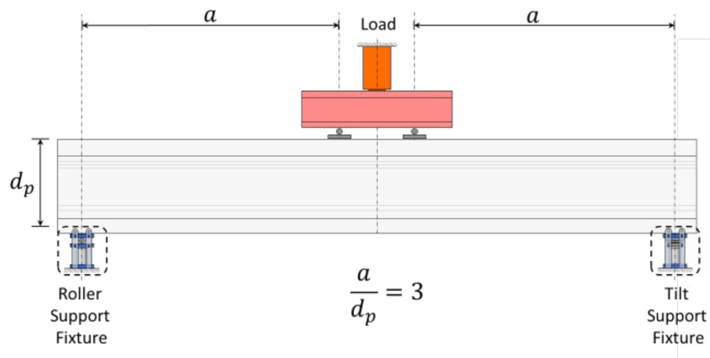
SHEAR TEST

The shear tests were carried out at least 28 days after casting. Prior to shear test, an 8-in. reinforced concrete slab was cast on each specimen to distribute the loads, simulate the composite bridge deck, and increase the flexural capacity of the specimens. A summary of the material properties at the time of shear test is shown in Table 3. The Tx46 specimens were tested at a span-to-depth ratio of 3.0 in a four-point loading configuration, which made it possible to load both of the live and dead ends simultaneously. The Tx70 girder was tested using a three-point loading configuration and a shear span-to-depth ratio of 2.3.

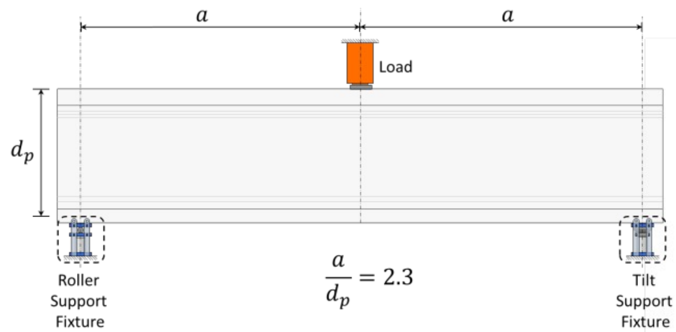
Loading configurations for both cases are shown in Fig. 6. Both live and dead spans were subjected to the same shear force by a symmetric load configuration. This configuration was used to investigate the potential difference in behavior between the two ends and to ensure a consistent shear span-to-depth ratio regardless of the failed span.

Table 3 Summary of material properties at the time of shear test

Variables	Tx46-I	Tx46-II	Tx70-I
Concrete strength, f'_c	7.6 ksi	6.9 ksi	10.7 ksi
Modulus of elasticity of concrete (E_c)	4,920 ksi	5,220 ksi	7,000 ksi
Yield strength of shear reinforcement (f_y)	60.7 ksi	60.7 ksi	72.2 ksi

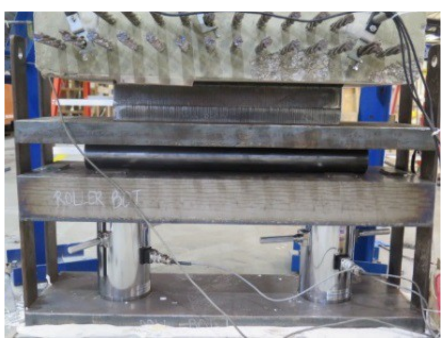


(a) Tx46-I and Tx46-II



(b) Tx70

Fig. 6 Loading configuration for shear test



(a) Roller Support

(b) Tilt Support

Fig. 7 Supports for shear test

As shown in Fig.7, carefully designed support fixtures were used in the setup, which provided roller and pin support conditions at the reaction points. The supports also contained load cells, which made it possible to measure the reaction forces due to applied loads as well as self-weight. Moreover, a series of displacement transducers were used to monitor the deformations of the specimen and strand slip during the test. A 2,000-kip hydraulic ram, which was pressurized using a pneumatically powered hydraulic pump was used to apply the load to the beams. Load was applied at increments of 50 or 100 kips to mark the cracks until considerable diagonal cracking occurred, after which the applied load increased continuously until failure.

FINITE ELEMENT ANALYSIS

A series of nonlinear finite element (FE) analyses was conducted to improve the understanding of the mechanics of prestress transfer and shear failure in prestressed concrete I-girders that employ 0.7-in. strands. FE analyses supplemented the experiments by providing insight into the complicated stress states developed in the end regions, the failure mechanisms of the girders, and the efficacy of the reinforcement detailing.

The FE analyses were conducted using ATENA-3D[17], which is a commercial software developed specifically for modeling reinforced concrete structures. A validation study was conducted to compare the software predictions with experimental results from previous research programs. As a result, a uniform combination of suitable material models and solution methods was selected, which yielded good agreement with experimental results.

The only user-input variable entered for the concrete mechanical properties was the measured compressive strength at the times of prestress transfer and shear test. Default parameters calculated by ATENA-3D were used for all other concrete mechanical properties. The reinforcement material properties were based on measured stress-strain data whereas standard material properties for Grade 270 seven-wire low-relaxation strands were assumed for prestressing steel. The Bigaj bond model[18] was used to model the bond between the prestressing strands and concrete. All other steel reinforcement was modeled discretely and was assumed to be perfectly bonded with the concrete.

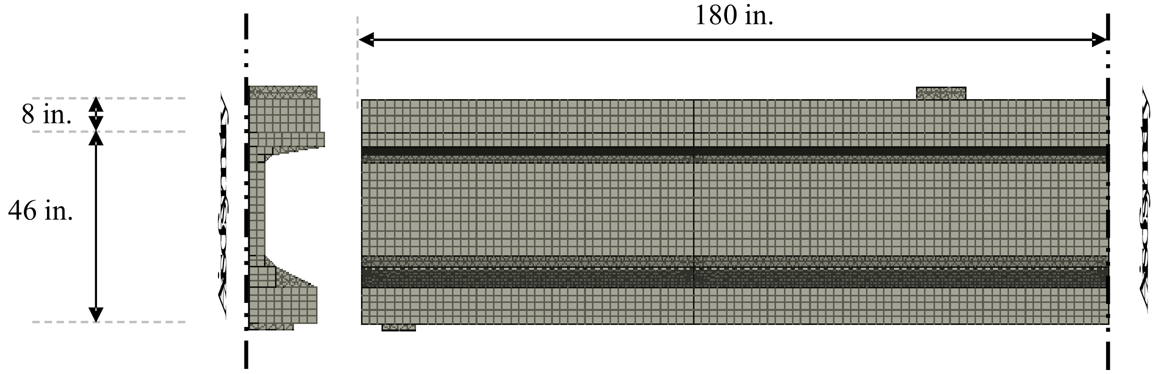


Fig. 8 Typical mesh density for Tx46 girders

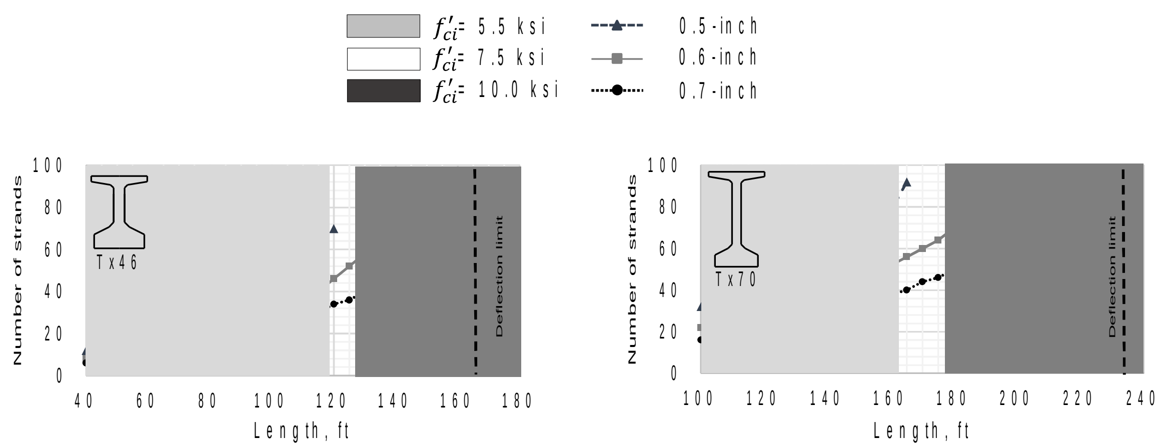
Fig. 8 shows the typical quarter-beam (half-span and half-cross section) mesh that was used in the modeling of the pretensioned girders. A consistent element size of 50 mm (1.97 in.) was chosen based on a mesh sensitivity analysis for the behavior at prestress transfer. The prestressing force was modeled by the application of an equivalent prestrain in the strands. Key parameters at prestress transfer, including crack patterns and widths and the transfer lengths of the prestressing strands, were estimated numerically. Subsequent shear loading tests were simulated using a displacement-controlled loading procedure, with imposed displacements applied to a single node within the load-bearing plate.

RESULTS AND DISCUSSION

PARAMETRIC STUDY

Fig. 9 shows the number of strands required at different span lengths for Tx46 and Tx70 girders. In each plot, three zones (5.5, 7.5, and 10 ksi) are shown, which represent the concrete release strengths that are required to achieve each span length. Release strengths beyond 10 ksi were not considered because they were deemed impractical. The designs presented in this figure are based on the assumption of an 8 ft spacing between girders.

Fig. 9 goes on to show that using larger-diameter strands results in a considerable reduction in the number of required strands for both cross sections. This observation comes as no surprise, since fewer larger-diameter strands would be needed to provide the same area of prestressing steel. At the maximum span that can be achieved with all strand diameters for each cross section, the use of 0.7-in. strands makes it possible to use 12 fewer strands in Tx46 and 16 fewer strands in Tx70. Such a noticeable reduction in the number of strands could potentially correspond to a considerable decrease in the required time and effort, and therefore costs, for the fabrication process.



(a) Tx46 (b) Tx70

Fig. 9 Number of strands required at different span lengths for each strand diameter and concrete release strength

The plots in Fig. 9 also show that for each section, different span lengths could be achieved with different strand sizes. With 0.5-in. strands, the maximum span length was governed by the number of strands that could be used within the girder cross section. The design of girders with larger-diameter strands, however, was governed by the stress limits at the time of prestress transfer. Using 0.6-in. strands instead of 0.5-in. strands is estimated to increase the span capabilities of Tx46 and Tx70 girders by 20 and 25 ft, respectively. Using 0.7-in. strands results in further increase in span capabilities of 5 ft for Tx46 and 10 ft for Tx70. However, in these cases, a release strength of 10 ksi is required.

A variety of other parameters and precast concrete cross sections were investigated in the parametric study. Detailed results of this study will be made available in future publications.

END-REGION BEHAVIOR AT PRESTRESS TRANSFER

During the release operation, strains were measured in end-region reinforcement to evaluate end-region stresses and at different locations along the strands to obtain the transfer length. After prestress transfer, the specimens were also carefully inspected, and the width and patterns of end-region cracks were documented. Camber was also measured for two specimens and was found as 0.07 in. for Tx46-II and 0.26 in. for Tx70-I.

Transfer Length

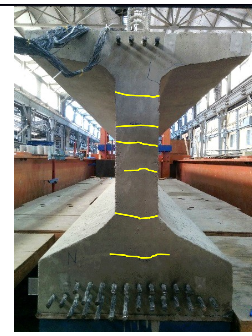
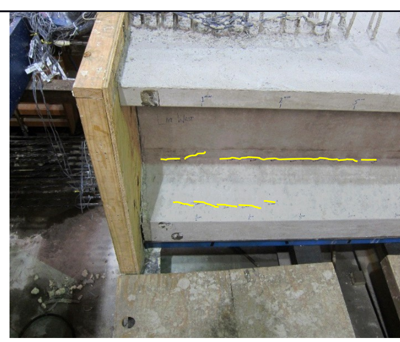
The transfer length was determined using a modified version of the 95 percent average maximum strain (AMS) method by Russell and Burns[19], based on the data obtained from the strain gauges. The transfer length immediately after release was measured between 29 in. and 47 in., which was generally less than those predicted by ACI 318-14[11] and AASHTO LRFD[13] provisions. However, in all specimens, the transfer length grew considerably over time. At 24 hours after release, the transfer length was measured between 31 in. and 52 in. among the specimens, with almost all instrumented strands in Tx46-I and Tx46-II exceeding the transfer length estimates by AASHTO LRFD and ACI 318-14. Due to the gradual release operation that was used, the difference between the transfer lengths at live and dead ends of the specimens was found to be generally small.

It is expected that the transfer length grows even further over time. However, monitoring the transfer length after 24 hours was not considered for the three specimens presented in this paper. Investigating the transfer length in greater detail is an ongoing effort in this research program, which includes evaluating changes in transfer length over time and studying the potential differences between transfer lengths obtained from measurements of surface strains (used by many researchers) and those presented above. The results of these investigations will be discussed in a later publication.

Cracking Pattern

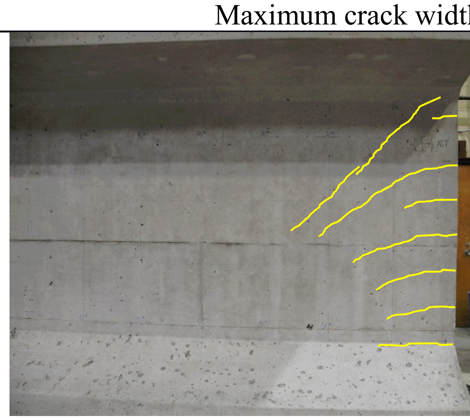
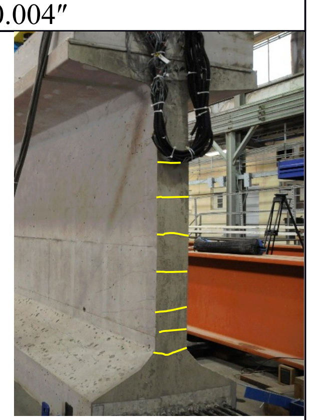
Tables 4 and 5 show the observed end-region cracking immediately after prestress transfer in the first three specimens of the testing program. As can be seen from these tables, Tx46-I and Tx70-I showed clear patterns of spalling cracks, whereas bursting cracks were observed within the end regions of Tx46-II. These cracking patterns were compatible with what was expected based on the selected prestressing layout within each specimen. No unusual cracking, i.e. cracking that is different from that in Tx-girders with 0.6-in. strands, was observed in any of the specimens immediately after prestress transfer.

Table 4 End-region cracking in Tx46 specimens immediately after prestress transfer

Specimen	End region	Side view	End face
Tx46-I	Dead end		
		Maximum crack width*: 0.008″	
	Live end		
		Maximum crack width: 0.006″	
Tx46-II	Dead end		
		Maximum crack width: 0.004″	
	Live end		
		Maximum crack width: 0.004″	

* In Tx46-I, the 0.008″ crack width was observed in an isolated 1-in. length. In other locations, the maximum crack width was 0.006″.

Table 5 Observed end-region cracking in Tx70-I immediately after prestress transfer

Specimen	End region	Side view	End face
Tx70-I	Dead end		
		Maximum crack width: 0.004″	
	Live end		
		Maximum crack width: 0.006″	

End-Region Stresses in Transverse Reinforcement

The strains in the transverse steel within the end region of the girders were carefully examined to estimate the stresses developed in this reinforcement due to prestress transfer. The stresses inferred from strain gauge measurements within the end regions of the girders are presented in Tables 6 and 7. The plots in these tables show that in the end-region reinforcement of Tx46-I and Tx70-I, stresses up to 20 and 25 ksi were observed, respectively. However, these stresses quickly diminish with increasing distance from the end face of the girder.

Section 5.10.10.1 in AASHTO LRFD Bridge Specification[13] requires the average stress in the transverse reinforcements that is located within *h/4* from the end face of pretensioned girders to be limited to 20 ksi, where *h* is the overall height of the member. While stresses up to approximately 25 ksi were observed in Specimen Tx70-I, the average stresses within the last *h/4* length of all three specimens were below the 20-ksi limit.

Table 6 Transverse steel stresses in the Tx46 specimens immediately after prestress transfer

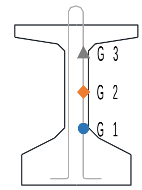
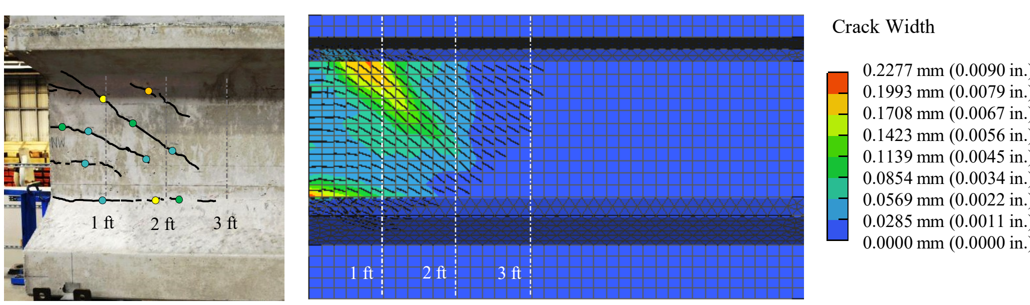
Specimen	End region	Stirrup stresses
Tx46-I	Dead end	
	Live end	
Tx46-II	Dead end	
	Live end	

Table 7 Transverse steel stresses in Tx70-I immediately after prestress transfer

Specime n	End region	Stirrup stresses
Tx70-I	Dead end	
	Live end	

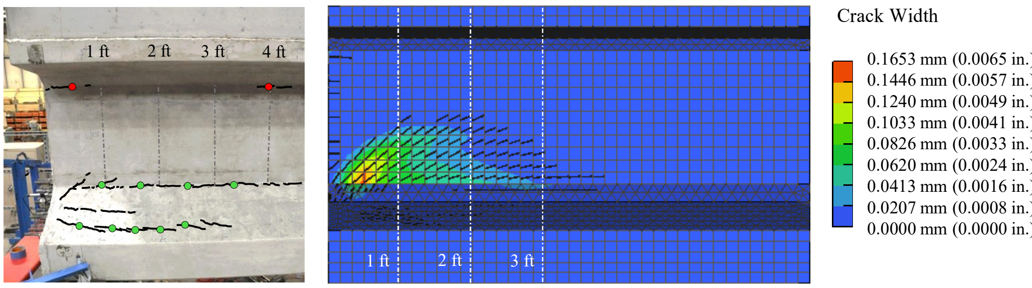
Fig. 10 through 12 demonstrate the estimated cracking distributions in the end regions of the girders developed from nonlinear FE simulations. As can be seen in these figures, the computed cracking patterns and crack widths were in general agreement with the observations from the specimens.

As part of FE studies, a series of reinforcement alterations was considered through the addition of vertical or horizontal reinforcement to control the width of cracks. Details of this investigation will be presented in a later publication.



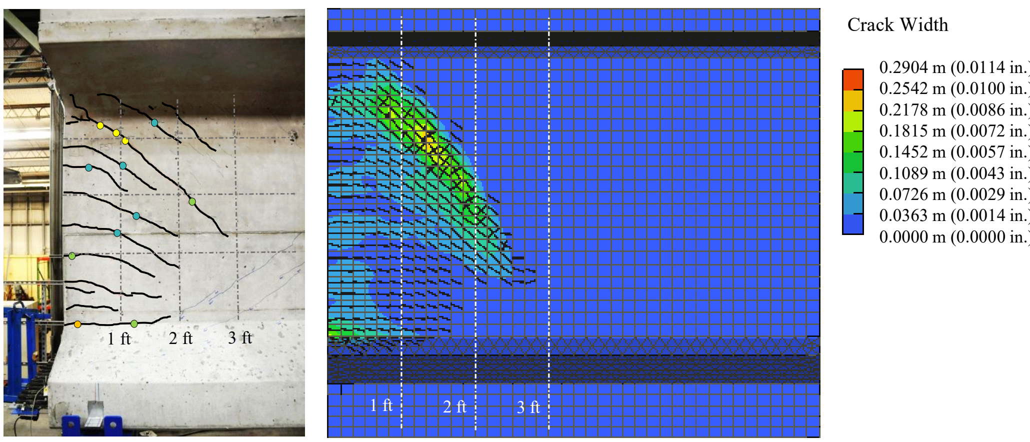
(a) Measured (b) ATENA-3D prediction

Fig. 10 Tx46-I release cracks (live end)



(a) Measured (b) ATENA-3D prediction

Fig. 11 Tx46-II release cracks (live end)



(a) Measured (b) ATENA-3D prediction

Fig. 12 Tx70-I release cracks (live end)

SHEAR TEST RESULTS

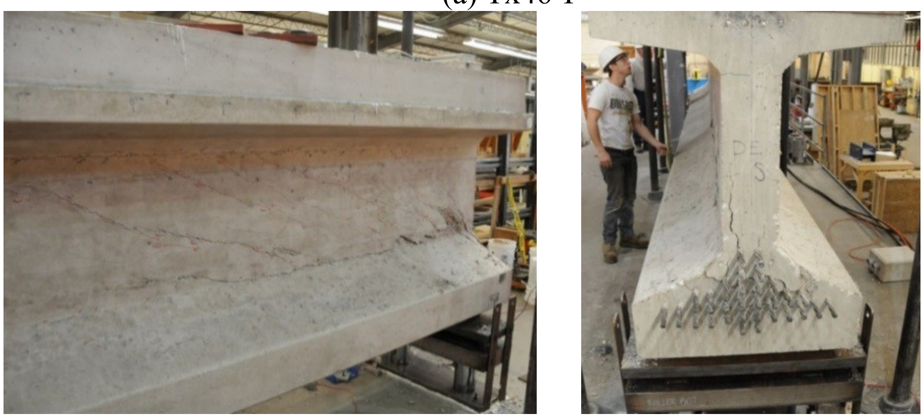
Fig. 13 shows the patterns of cracking and damage in the specimens after failure. All three specimens failed at their dead ends. No signs of flexural distress, i.e. cracking or crushing that was associated with flexure, were observed in any of the specimens. Yielding was observed in stirrups in all three specimens before failure, indicating a shear-tension behavior. However, all specimens showed clear signs of atypical failure modes.

Specimen Tx46-I showed noticeable horizontal cracking at the web-bottom flange interface before the occurrence of the first diagonal web crack. This specimen failed after considerable strand slip happened, indicating anchorage distress. Moreover, failure of the specimen was accompanied by a significant horizontal crack at the web-bottom flange interface and displacement of the web with respect to the bottom flange.

The first noticeable event during shear testing of Specimen Tx46-II was the occurrence of diagonal cracking in the web. Similar to Specimen Tx46-I, failure of this specimen showed clear signs of anchorage distress, i.e. strand slip. The post-failure damage of the specimen also showed horizontal cracking at the web-bottom flange interface, although not as

distinctive as in Tx46-I.
The Tx70 specimen did not show signs of horizontal shear cracking. However, considerable strand slip was observed in this specimen prior to failure.

(a) Tx46-I



(b) Tx46-II

(c) Tx70-I

Fig. 13 Observed damage in the specimens after failure in shear test

Fig. 14 shows the load-deflection plots from the three specimens. Also shown in this figure are the load-deflection responses developed using ATENA-3D and the capacities calculated on the basis of the general method (Section 5.8.3.4.2) in AASHTO LRFD specifications[13]. As can be seen in the figure, all specimens failed at a load that was conservatively estimated by the AASHTO general method. It should be noted that if the demands on the longitudinal reinforcement, according to section 5.8.3.5 in AASHTO LRFD, are also considered, a more conservative estimate of the shear capacity will be found for the specimens.

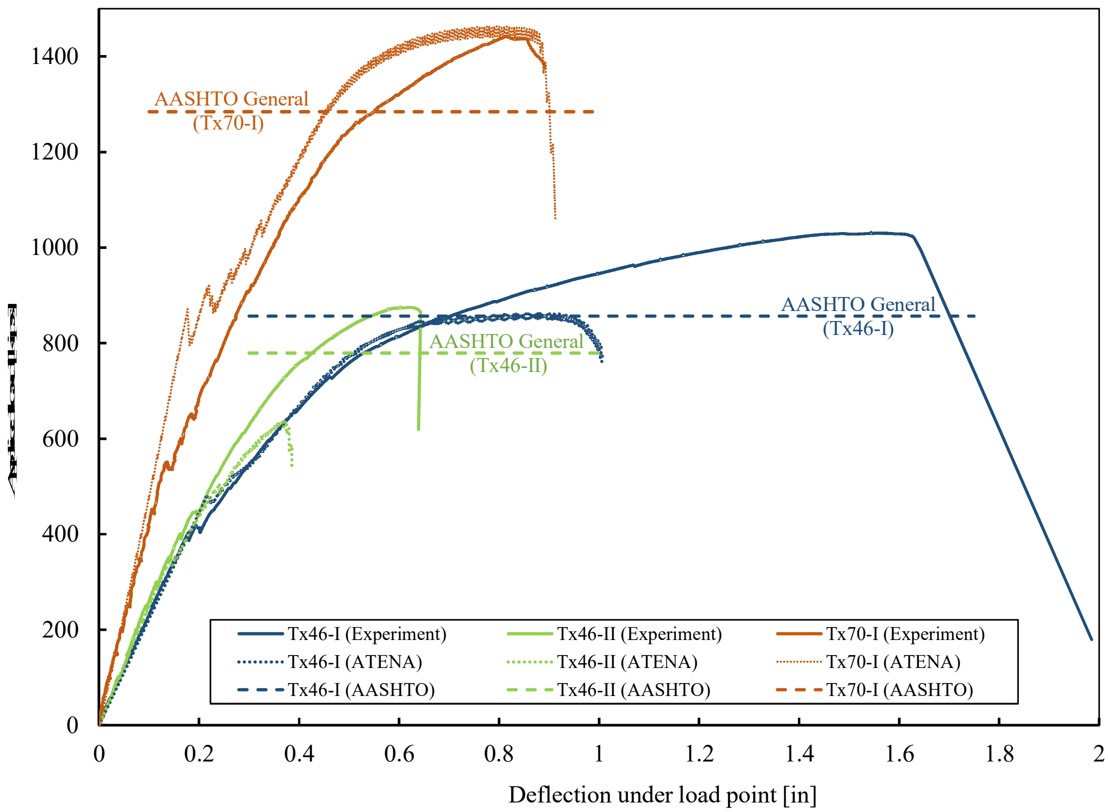


Fig. 14 Load-deflection plots for shear tests

Fig. 14 also shows that the FE simulations of the specimens were generally successful in capturing the load-displacement behavior, especially for Tx46-I and Tx70-I. In general, the initial stiffnesses of the load-displacement plots obtained from ATENA were also in good agreement with the measured stiffnesses. The load-carrying capacity estimated using ATENA-3D was less than the measured capacity for Tx46-I and Tx46-II, but slightly greater than the measured capacity for Tx70-I.

CONCLUSION

This paper has introduced an ongoing research program at the University of Texas at Austin on the end-region serviceability and shear strength of Tx-girders that employ 0.7-in. diameter strands. This comprehensive research program involves full-scale experimental studies and finite element simulations of pretensioned girders with larger-diameter strands as well as a parametric study. While the research is still underway, the results from the first three specimens in the test program have been briefly summarized in this paper.

The specimens comprising the program were carefully designed to represent the critical conditions at the time of prestress transfer. End-region detailing consistent with current TxDOT standard drawings was used. The specimens were fabricated at Ferguson Structural Engineering Laboratory and were extensively instrumented and monitored for end-region stresses, strand slip, and end-region cracking at the time of prestress transfer. The specimens were also loaded in a shear-critical loading configuration until failure. To fabricate the specimens, modifications were made to the prestressing facility at Ferguson Structural Engineering Laboratory so that 0.7-in. strands could be accommodated. However, the strands could be used on the 2- by 2in. grid without any constructability issues.

The results from the prestress transfer for the first three specimens showed that girders with 0.7-in. diameter strands that were placed on a standard 2- by 2-in. grid did not exhibit cracking patterns atypical from that generally reported for Tx-girders employing 0.6-in. strands. The greatest crack width observed in the first three specimens immediately after prestress transfer was 0.008 in. However, this width was localized over a 1-in. length of a crack developed in Tx46-I. All other cracks in the first three specimens had a maximum width of 0.006 in. immediately after prestress transfer. Despite the considerable number of 0.7-in. strands in Tx46-II, bursting and spalling stresses were not severe enough to cause considerable cracking in this specimen. The magnitude of cracking in Tx70-I immediately after prestress transfer was also less than that in Tx46-I, although greater stresses were observed in the end-region reinforcement of the Tx70 specimen. The stress in the transverse reinforcement in the Tx70 specimen reached a maximum of approximately 25 ksi. However, large stresses were observed only in the vicinity of the end face of the girder and quickly diminished with distance from the end face.

All three specimens failed in shear-tension failure modes that were accompanied by signs of anchorage distress (strand slip). The Tx46 specimens also showed signs of horizontal shear distress. However, yielding of the stirrups was confirmed in all specimens, and no evidence of premature failure due to anchorage or horizontal shear distress was found. Despite the atypical damage that was observed, all three specimens failed at a load that was conservatively estimated using the general method in AASHTO LRFD specifications.

The observations from the first three specimens yielded no indication for a need to modify the current end-region detailing of Tx-girders, to accommodate 0.7-in. strands on a 2- by 2-in. grid. However, additional specimens will be investigated, in which greater release strengths, different strand patterns, and different precast cross sections will be evaluated. Compiled results from all current and future specimens, together with findings from the FE simulations will be used to determine the potential need for end-region modifications or a reevaluation of shear strength calculations for Tx-girders with 0.7-in. strands.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the Texas Department of Transportation (TxDOT) for providing financial support for this study. Moreover, the authors would like to thank Coreslab Structures for donating the concrete used for one of the specimens presented in this paper. The findings, opinions, and recommendations presented in this article are those of the authors and do not necessarily reflect the views of TxDOT.

REFERENCES

1. Comité Euro-International Du Beton (CEB), "Anchorage Zones of Prestressed Concrete Members," State-of-the-Art Report, CEB Bulletin No. 181, May 1987.
2. Hoyer, E., and Friedrich, E., "Beitrag zur Frage der Haftspannung in Eisenbetonbauteilen" [Contribution to the Question of Bond in Concrete Elements], *Beton und Eisen*, V. 30, No. 6, Mar. 1939, pp. 107-110.
3. Vadivelu, J., "Impact of Larger Diameter Strands on AASHTO/PCI Bulb-Tees," University of Tennessee, Knoxville, 2009.
4. AASHTO (American Association of State Highway and Transportation Officials), "AASHTO LRFD Bridge Design Specification," 4th Edition, Washington D.C., 2007.
5. ACI Committee 318, "Building Code Requirements for Structural Concrete" (ACI 318-08), and Commentary (ACI 318R-08)," American Concrete Institute, Farmington Hills, Michigan, 2008.
6. Tadros, M.K., and Morcous, G., "Impact of 0.7 Inch Diameter Strands on NU I-Girders. Technical Report," Lincoln, Nebreska: Nebraska Department of Roads, 2011.
7. Morcous, G. Hanna, K., and Tadros, M.K., "Use of 0.7-in.-diameter strands in pretensioned bridge girders," *PCI Journal*, V.56, No. 4, Fall 2011, pp. 65-82.
8. Morcous, G., Assad, S., and Hatami, A., "Implementation of 0.7 in. Diameter Strands in Prestressed Concrete Girders," University of Nebraska-Lincoln, 2013.
9. Dang, C.N., Floyd R.W., Hale, M., and Marti-Vargas, J.R., "Measured Transfer Lengths of 0.7 in. (17.8 mm) Strands for Pretensioned Beams," *ACI Structural Journal*, V.113, No. 1, Jan.-Feb. 2016, pp. 85-94.
10. Dang, C.N., Floyd R.W., Hale, M., and Marti-Vargas, J.R., "Measured Development Lengths of 0.7 in. (17.8 mm) Strands for Pretensioned Beams," *ACI Structural Journal*, V.113, No. 3, May-June 2016, pp. 525-535.
11. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)," American Concrete Institute, Farmington Hills, Michigan, 2014.
12. Garber, D. B., Gallardo, J. M., Deschenes, D. J., and Bayrak, O., "Effect of New Prestress Loss Estimates on Pretensioned Concrete Bridge Girder Design,", Austin, Texas: Texas Department of Transportation, 2012.
13. AASHTO (American Association of State Highway and Transportation Officials), "AASHTO LRFD Bridge Design Specifications, 7th Edition – with 2016 Interim

Revisions," Washington, D.C., 2016.

14. Bridgesight Inc., "PGSuper," Software for design, analysis, and load rating of precast, prestressed girder bridges, South Lake Tahoe, California, 2014.
15. Texas Department of Transportation (TxDOT), "Bridge Design Manual – LRFD," Austin, Texas, October 2015.
16. Texas Department of Transportation (TxDOT), "Concrete I-Girder Details," Standard Drawing by the Bridge Division, Austin, Texas, 2015.
17. Červenka, J., and Procházková, Z., "ATENA Program Documentation- Part 4-2: Tutorial for Program ATENA 3D," Červenka Consulting s.r.o., Prague, Czech Republic., 2013.
18. Bigaj, A.J., "Structural Dependence of Rotations Capacity of Plastic Hinges in RC Beams and Slabs," Delft University of Technology, the Netherlands, 1999.
19. Russell, B.R., Burns, N.H., "Measured Transfer Lengths of 0.5 and 0.6 in. Strands in Pretensioned Concrete," *PCI Journal,* V.41. No.5, Spt.-Oct.1996, pp.44-65.