SHEAR PERFORMANCE OF POST-TENSIONED BULB-TEE GIRDERS

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ABSTRACT

AASHTO specifications assume a reduction in the shear strength of a girder if a post-tensioning duct is present in the web. This reduction is accounted for by taking an 'effective web width' which is dependent on the duct diameter, the gross web thickness, and if the duct has been grouted. Currently, this reduction in shear strength has been calibrated using small scale compressive tests designed to model the compressive strut in the web. The scarcity of shear tests on post-tensioned girders is of concern because the literature contains one series of panel tests indicating that plastic post-tensioning ducts may cause up to a 30% reduction in compressive strength when compared to galvanized steel ducts of similar size. To better understand the shear behavior of post-tensioned girders, six post-tensioned bulb-tee girders were tested to failure in shear. These girders contained pre-tensioning in the bottom flange and a stressed post-tensioning tendon in the web. This paper examines existing code provisions for the reduction in shear strength due to the presence of a post-tensioning duct, and specifically the usefulness of panel testing to evaluate the shear strength of full scale girders.

Keywords: Post-tensioned concrete, Prestressed concrete, Post-tensioning duct, Shear, Concrete shear, Spliced girders

INTRODUCTION

The experimental program, sponsored by the Texas Department of Transportation, was designed to provide data for the safe implementation of spliced bulb-tee girder bridges by better understanding the shear behavior of post-tensioned bulb-tee girders. In order to meet this objective, eleven shear tests were performed on seven full-scale post-tensioned concrete bridge girders. The data from these tests have led to a better understanding of the shear behavior of girders containing post-tensioning ducts within their webs. The current shear provisions for post-tensioned webs within the AASHTO LRFD Bridge Design Specifications[1] were calibrated based on small-scale panel tests. The behavior of these panels, however, does not adequately capture the complex stress transfer within full-scale girders. The usefulness of the full-scale tests cannot, therefore, be overemphasized. The results of the full-scale girder tests performed at Ferguson Structural Engineering Laboratory (FSEL) are outlined in the following paragraphs.

RESEARCH SIGNIFICANCE

The primary factors influencing the shear behavior of prestressed concrete girders are well understood, but several factors remain less studied. Among these is the reduction in strength as a result of a duct that may be present in the web of a post-tensioned girder. The majority of research into this phenomenon has been performed on small scale panel compression tests. These panel test results form the basis for the current web width reduction factors which reduce the “effective” width of the girder web for some percentage of the duct diameter. The relevance of this experimental approach is one of the primary topics of concern for this article.

PANEL TEST RESEARCH

Historically, research into the effect of post-tensioning ducts has been addressed by small scale panel tests that are meant to be representative of the inclined compressive strut formed during shear loading. As the compressive stresses flow around or through the post-tensioning duct tension is developed near the duct (as shown in Figure 1). This out-of-plane tension causes a reduction in shear strength compared to a cross-section without a duct. Therefore, it is assumed that the compressive strength of a panel with duct could be compared to the compressive strength of a solid (“control”) panel. This relative reduction in strength has formed the basis of the strength reduction factors found in all current code provisions and is discussed in the following section.

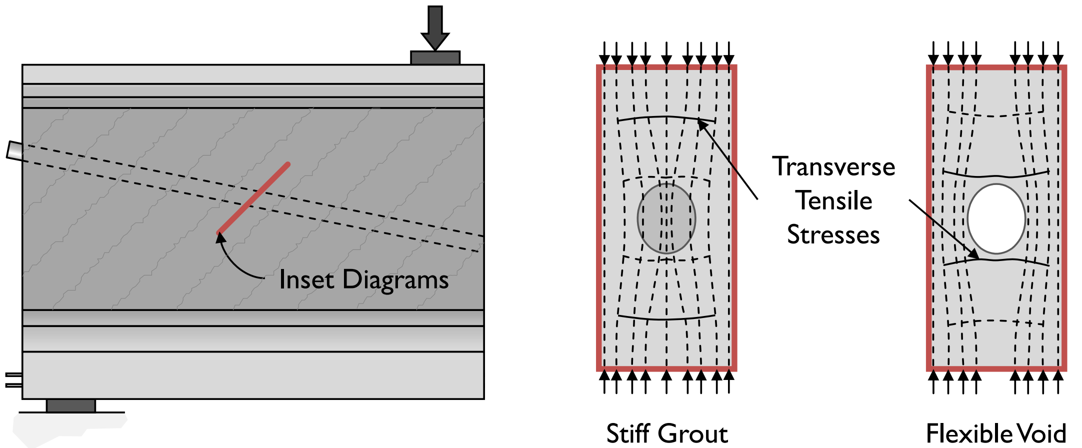

Figure 1: Compressive Strut - Panel Strength Analogy (from Muttoni et al. 2006)[2]

WEB WIDTH REDUCTION FACTORS

The effective web width reduction calculation has been calibrated through the use of panel test results that have shown the three primary variables to be the following: the duct diameter to web width ratio, whether or not, the tendon is grouted, and the duct material (plastic or steel). The effective web width is used in calculations within AASHTO LRFD (2013) for both the concrete contribution to shear strength (V_c) and the upper limit on shear stress ($0.25f_c'b_vd_v$). Although the precise terminology within each code may vary, the effective web width concept can be summarized as noted in the following four equations.

$$b_v = b_w \cdot \eta_D$$ ***Equation 1***

$$\eta_D = 1 - k \cdot \delta$$ ***Equation 2***

$$\delta = {}^{\phi}/_{b_w}$$ ***Equation 3***

OR MORE SIMPLY:

$$b_v = b_w - k \cdot \phi$$ ***Equation 4***

Where:

b_v = effective web width available to resist shear accounting for presence of ducts
b_w = gross web width
k = diameter correction factor, varied per code (see Table 1)
δ = duct diameter to web thickness ratio
ϕ = duct diameter

The diameter correction factor, k, is dependent on the design code being considered and has been calibrated using past panel test data. These k-values were calibrated by testing ducted panels and comparing the failure strength to a control specimen without a duct (refer to illustration in Figure 2).

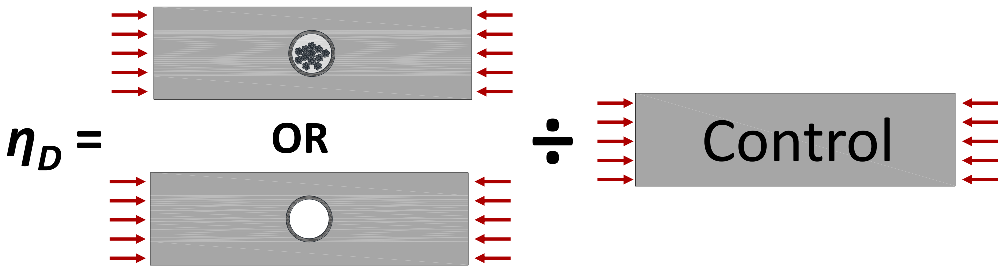

Figure 2: Description of η_D Calibration Calculations

The *k*-factors for the four design codes considered in this document are shown in Table 1, but it should be noted that the only design code that takes into consideration the duct material is the EuroCode2 (2004)[3].

Table 1: Diameter Correction Factors (k) for Design Codes Considered

Code Provision	*Code Reference*	*Empty Steel*	*Grouted Steel*	*Empty Plastic*	*Grouted Plastic*
ACI 318-11	*not addressed*				
AASHTO General Shear	*§5.8.2.9*	*0.25*	*0.5*	*0.25*	*0.5*
AASHTO Segmental Shear	*§5.8.6.1*	*1.0*	*0.5*	*1.0*	*0.5*
EuroCode2 2004*	*§6.2.3-5*	*1.2*	*0.5*	*1.2*	*1.2*

**EuroCode2 does not reduce effective web widths at Duct Diameter to Thickness values <0.125*

TX62 BULB-TEE GIRDERS

The Tx62 girders used in this experimental program were 50-ft. long with a 7.5-ft. long thickened end-region at each end to accommodate the post-tensioning anchorages. These test girders *did not have a spliced* region, as they were designed to model the behavior of spliced bulb-tee girders outside of the spliced regions. Aside from the end-blocks, the cross-section follows the Texas Department of Transportation (TxDOT) standard Tx62 cross-sectional dimensions shown in Figure 3[5]. Also shown in Figure 3 are the modified Tx62 cross-sectional dimensions, which were widened to accommodate a 9-in. web. This was accomplished by increasing the width of the girder as a whole, resulting in top and bottom flanges 2-in. wider than the standard Tx62. The reinforcement of these two girders was also modified and the details of and comparisons between these girders are discussed in more detail in the following sections.

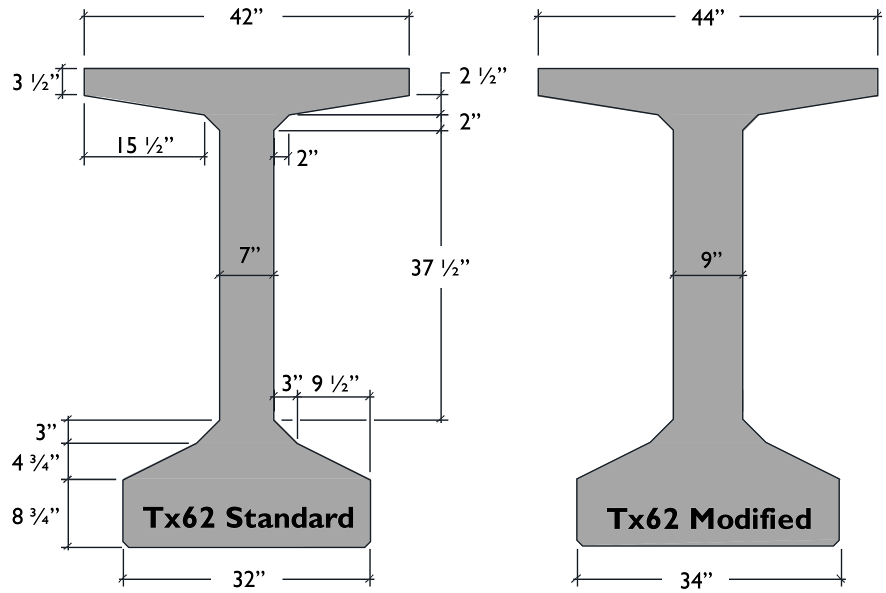

Figure 3: Tx62 Cross-Section 7 and 9 in. Web (consistent dimentions not shown)

End-Block Design

The existing standards for pretensioned Tx62 girders maintain a constant cross-section throughout their length (i.e. no thickened end-region or other change of cross-section throughout the length). All girders tested during this research program were modified to include a thickened end-block to accommodate the post-tensioning anchorages.

To aid in the design of the end-block, both cross-sections and end-block lengths from several states were reviewed and evaluated for their advantages and disadvantages. The geometry selected for the end-blocks of the Tx62 test girders most closely follows that of the Washington State Department of Transportation (WSDOT) standardized post-tensioned bulb-tee girder[6]. The total length of the end-block was 7.5 feet with a 3 foot length of constant cross-section and a 4.5 feet tapered section that transitions into the Tx62 cross-section shown previously. The locations of the cross-sectional transitions are shown in Figure 4.

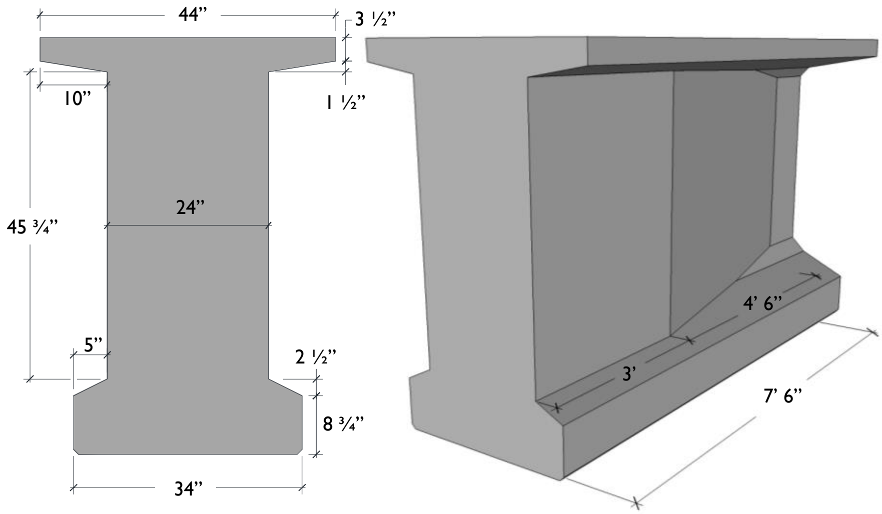

Figure 4: End-Block Details for Tx62 (standard cross-section with 7 in. web)

Although the overall geometry of the end-block was based on WSDOT standards, the reinforcement of the end-block was designed specifically to provide adequate resistance to the prestressing force seen during the experimental program. Although all post-tensioned girders all contained only one post-tensioning tendon, three tendons were used in later tests not addressed in this article. Therefore the end-block was designed to withstand the prestressing force resulting from three 12 – Ø 0.6-in. post-tensioning tendons as well as a maximum of 80 – Ø 0.5-in. pretensioning strands. A rendering of the end-block reinforcement is shown in Figure 5.

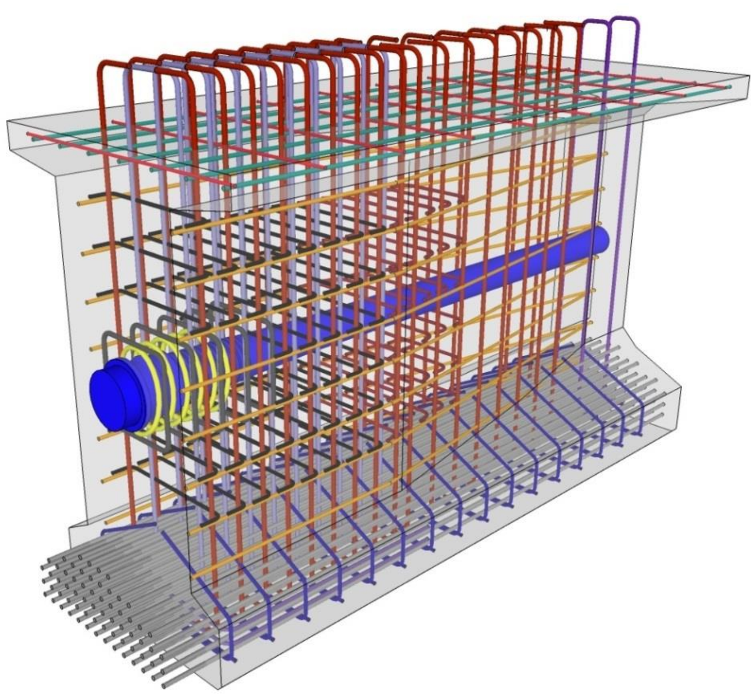

Figure 5: Rendering of End-Block Reinforcement Details

Post-Tensioning Anchorage and Tendon Layout

Each test girder contained one post-tensioning tendon made up of 12–0.6-in. diameter low-relaxation strands. The tendon had a straight profile throughout the length of the girder and was located at the mid-height of the web, or 35.25-in. from the bottom of the girder. The post-tensioning duct was secured by supports at a minimum of 2 foot intervals in accordance with §10.4.1.1 AASHTO LRFD Bridge Construction Specifications 2010[7]. This specification requires that polyethylene ducts be supported every two feet and steel ducts every 4 feet. The spacing between supports was maintained at 2 ft. for all girders in the interest of consistency among all tests.

Each tendon was anchored within the end-block by a multi-plane cast steel anchor heads shown in Figure 6. The tendons were housed in post-tensioning ducts that varied both in diameter and in material (either plastic or steel) depending on the test variable under consideration. All post-tensioning ducts were grouted after the tendons had been fully stressed.

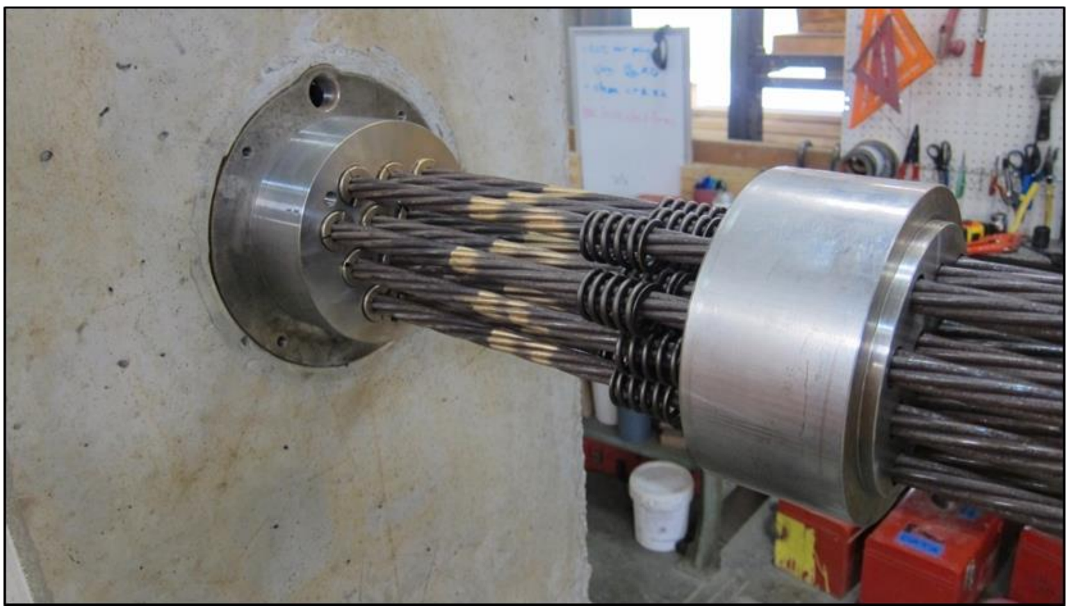

Figure 6: Post-Tensioning Anchorage

Deck Placement

After the post-tensioning duct was grouted, an 8-in. thick deck was placed on the girder to increase moment capacity and provide more realistic test conditions that match those in an actual bridge structure. This deck was 2-in. narrower than the top flange of the girder as shown in Figure 7 to allow the deck formwork to rest on the top flange during casting. The concrete used for the deck was sourced from a local ready-mix concrete supplier. The concrete material properties for the deck are shown in the following section listing the experimental variables.

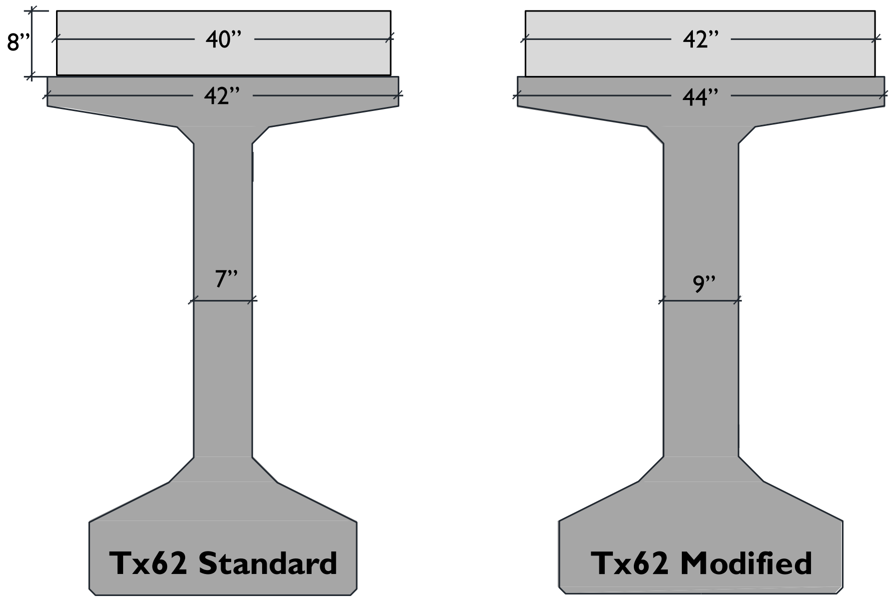

Figure 7: Deck Dimensions (dimensions consistent with earlier figures not shown)

Concrete Materials

The test girders were cast at a single precast concrete fabrication yard and used the fabricator's prestressed TxGirder self-consolidating mix design. The precaster had been using SCC exclusively for over two years at the time that the first test girder was cast, and experienced no issues with consolidation or bug-holes during the casting of the test girders. The mix designs are omitted here for brevity, but the course aggregated was comprised of ½ in. nominal maximum size pea gravel and the concrete strengths ranged from 10.6 to 13.9 ksi compressive strength at the time of testing.

EXPERIMENTAL VARIABLES

All girders were tested to failure in shear on the elevated slab strong floor at the Ferguson Structural Engineering Laboratory (FSEL). The test girders were designed to accommodate two tests per girder, but in several cases the damage proved too extensive to perform a subsequent test. Therefore, eleven shear tests were successfully performed on seven girder specimens over the course of this project.

To minimize scatter and facilitate a better understanding of the effect of a post-tensioning duct on the shear strength of a girder, the primary experimental variables were limited to five areas: duct presence, duct material, web width, the duct diameter to web width ratio, and transverse reinforcement. All test variables and material properties relevant to this experimental program are shown in the following five tables.

Table 2: Girder Geometry and Post-Tensioning Duct Information

Girder Geometry & Post-Tensioning Duct Properties						
Test	**Girder**		**Duct Material**	**Duct Diam. $\emptyset_D$**	**Web Width b_{gross}**	$\frac{\emptyset_D}{b_{gross}}$
1	**Tx62-1**	**S**	HDPE	3-in.	7-in.	0.43
2	**Tx62-2**	**S**	Steel			
3		**N**	Steel			
4	**Tx62-3**	**S**	No Duct			
5	**Tx62-4**	**S**	Steel			
6		**N**	HDPE			
7	**Tx62-5**	**S**	HDPE			
8		**N**	Steel			
9	**Tx62-6**	**S**	HDPE	4-in.	9-in.	0.44
10		**N**	Steel			
11	**Tx62-7**	**S**	Steel	3-in.		0.33

Table 3: Pretensioning Properties within Tension Zone

Prestressing Properties within Tension Zone**									
Test	**Girder**		**Prestressing Properties**			**Post-Tensioning Properties**			P_{total}
			Force [stress]	A_{ps}	y_p	**Force [stress]**	A_{ps}	y_p	
1	**Tx62-1**	**S**	1663 kips [165 ksi]	10.10-in.^{2}	6.44-in.	318 kips [122 ksi]	2.60-in.^{2}	35.25-in.	1981 kips
2	**Tx62-2**	**S**	1679 kips [166 ksi]			434 kips [167 ksi]			2113 kips
3		**N**							
4	**Tx62-3**	**S**	1699 kips [168 ksi]			--			1698 kips
5	**Tx62-4**	**S**	1691 kips [168 ksi]			490 kips [188 ksi]			2181 kips
6		**N**							
7	**Tx62-5**	**S**	1720 kips [170 ksi]			478 kips [183 ksi]			2197 kips
8		**N**							
9	**Tx62-6**	**S**	1887 kips [167 ksi]	11.32-in.^{2}	7.64-in.	488 kips [187 ksi]			2375 kips
10		**N**							
11	**Tx62-7**	**S**	1856 kips [164 ksi]			490 kips [188 ksi]			2346 kips

***** A_{ps} located below the mid-depth of member used in AASHTO general shear calc.***

Table 4: Pretensioning Properties within the Top Flange

Pretensioning Top Strands ***(not considered in AASHTO general shear A_{ps})***					
Test	**Girder**		**Force [stress]**	**Area**	**y_p**
1	**Tx62-1**	**S**	117 kips [192 ksi]	0.61-in.^{2}	57.5-in.
2	**Tx62-2**	**S**	118 kips [193 ksi]		
3		**N**			
4	**Tx62-3**	**S**	122 kips [199 ksi]		
5	**Tx62-4**	**S**	120 kips [195 ksi]		
6		**N**			
7	**Tx62-5**	**S**	119 kips [195 ksi]		
8		**N**			
9	**Tx62-6**	**S**	176 kips [192 ksi]	0.92-in.^{2}	56.5-in.
10		**N**			
11	**Tx62-7**	**S**	175 kips [190 ksi]		

Table 5: Transverse Reinforcement Properties

Transverse Reinforcement Properties**						
Test	**Girder**		$A_{vs} f_{vs} / s\, b_{gross}$	**Yield (f_{vs})**	**Spacing**	**Size**
1	**Tx62-1**	**S**	0.638 ksi	67.0 ksi	6-in.	No. 4
2	**Tx62-2**	**S**	0.650 ksi	68.3 ksi		
3		**N**				
4	**Tx62-3**	**S**	0.642 ksi	67.4 ksi		
5	**Tx62-4**	**S**	0.950 ksi	66.5 ksi	4-in.	
6		**N**				
7	**Tx62-5**	**S**	0.214 ksi	67.4 ksi	18-in.	
8		**N**				
9	**Tx62-6**	**S**	0.854 ksi	74.4 ksi	6-in.	No. 5
10		**N**				
11	**Tx62-7**	**S**	0.862 ksi	75.1 ksi		

****** This refers to the transverse reinforcement within the test region but outside of the thickened end region *(r-bars)*.

Table 6: Concrete and Grout Strengths

Concrete and Grout Strengths						
			Girder		Deck	Grout
Test	Girder		Compressive (f'_c)	Split Cyl. (f'_t)	Compressive (f'_c)	Compressive (f'_{cg})
1	**Tx62-1**	**S**	10.58 ksi	0.94 ksi	7.27 ksi	5.15 ksi
2	**Tx62-2**	**S**	11.97 ksi	0.89 ksi	11.43 ksi	5.66 ksi
3		**N**	11.97 ksi	0.89 ksi	9.39 ksi	4.28 ksi
4	**Tx62-3**	**S**	11.69 ksi	1.07 ksi	9.61 ksi	--
5	**Tx62-4**	**S**	13.92 ksi	1.15 ksi	12.70 ksi	9.92 ksi
6		**N**	13.61 ksi	1.00 ksi	11.11 ksi	9.38 ksi
7	**Tx62-5**	**S**	12.45 ksi	0.90 ksi	7.59 ksi	6.33 ksi
8		**N**	12.45 ksi	1.04 ksi	8.15 ksi	6.93 ksi
9	**Tx62-6**	**S**	12.35 ksi	0.94 ksi	8.16 ksi	7.92 ksi
10		**N**	13.16 ksi	1.01 ksi	9.77 ksi	8.43 ksi
11	**Tx62-7**	**S**	12.20 ksi	1.05 ksi	9.66 ksi	7.17 ksi

SHEAR TESTING

All girder end-regions contained two 7.5 ft. long thickened end-blocks. It was important that a significant portion of the shear span be outside of the thickened end-block region so that the capacity of the thinner (weaker) section of girder could be evaluated. Another factor influencing the length of the girder and shear span was the weight of the girder. Once the deck was placed on the girder the weight of the girder was so great that it could no longer be lifted. Since two tests were planned for each girder it was important that the girder configuration was such that it did not require lifting between tests. These considerations lead to the final configuration shown in Figure 8 with a shear span of 14.25 ft. and a back span of 20 ft. The 14.25 ft. shear span yields a span-to-depth ratio of 3.0 for all girders with the exception of the control specimen (Tx62-3) for which the ratio is 2.7 (the lack of a post-tensioning duct increased d_p). After testing the first girder, this layout was modified slightly by increasing the back span to 22 ft. but keeping the shear span at the original 14.25 ft. This configuration was maintained throughout the remaining 10 tests and allowed for the majority of the second test region to be overhung during the second test, thereby preventing damage to the second test region during the first test.

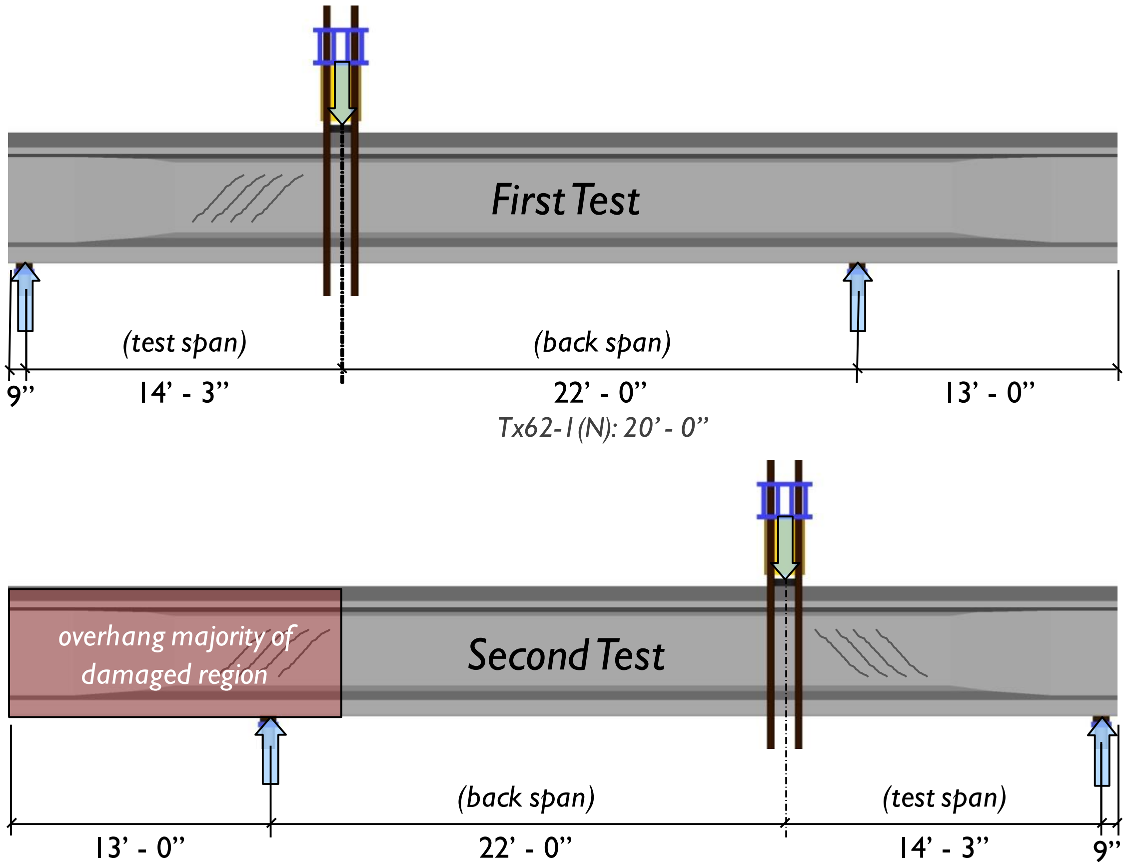

Figure 8: Shear Test Span Layout

Loading Procedure & Failure Shear Determination

All test girders were loaded in increments of approximately 75 kips shear until shear failure. This load was applied using the 2,000 kip load frame shown in Figure 9(A). Until the applied load level created safety concerns, all cracks were marked with felt-tipped permanent markers as shown in Figure 9(B).

Figure 9: (A) 2,000 kip Load Frame & (B) Crack Mapping between Load Steps

Reactions were measured by two 1,000-kip load cells at each support, and the shear at the critical section, taken to be at the end of the end-block transition, was calculated as shown in Figure 10. The shear failure load is defined here as the sum of the self-weight of the girder at the critical section, the weight of the load frame transmitted through the reaction at the test region, and the *maximum* applied load transmitted through the same reaction during testing.

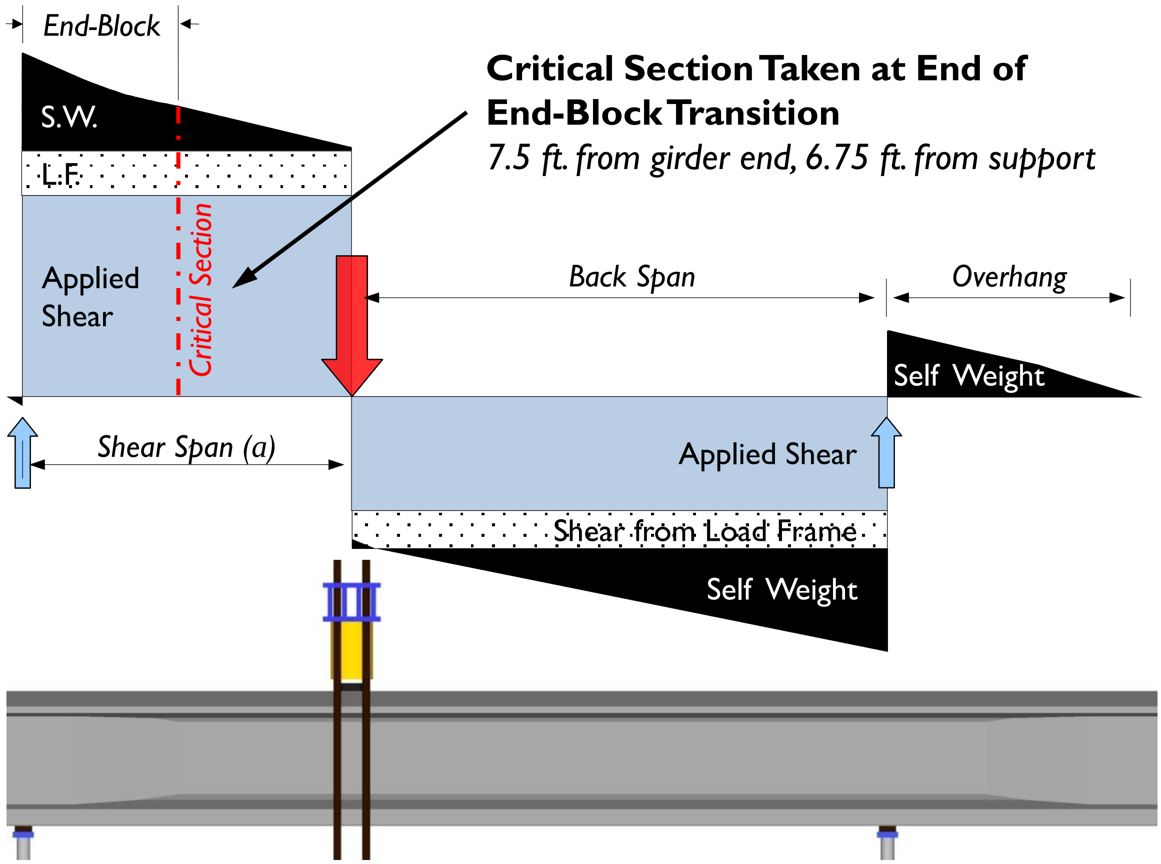

Figure 10: Shear Diagram and Illustration of Critical Section

Observed Failure Mechanisms

Each test consisted of loading the shear span of the test girder until failure occurred within that span and the applied load to the girder continuously dropped. The failure mechanism of all girders was characterized by the crushing of the compressive strut within the girder's web above the location of the post-tensioning duct. This was the controlling failure mechanism for all girders, including the control girder (Tx62-3 South which did not contain a post-tensioning duct). The primary difference between the failures of the post-tensioned girders and the control girder was the location within the strut which the concrete crushed. This topic will be covered in more detail in the following two sections.

Compressive Failure of Diagonal Strut in Control Girder

Tx62-3 (referred to as the "control") did not contain a post-tensioning duct but only pretensioning strands in the bottom and top flanges. This girder was designed to provide a comparison between the shear behavior of a post-tensioned girder (with a duct in the web region) and a prestressed girder without a duct. Although this girder did not contain a duct within the web it still failed due to crushing of the primary diagonal compressive strut. The primary difference between the failure mechanism observed in the control test and the post-tensioned shear tests was the location of this crushing failure. Figure 11 (A) shows the initial crushing of the web concrete at the base of the diagonal strut near the beginning of the end-block transition. Figure 11 (B) shows the moment after failure as the entire web of the girder crushes. It is important to understand that Figure 11 was captured with a pair of high speed cameras and the time between photos was approximately one second.

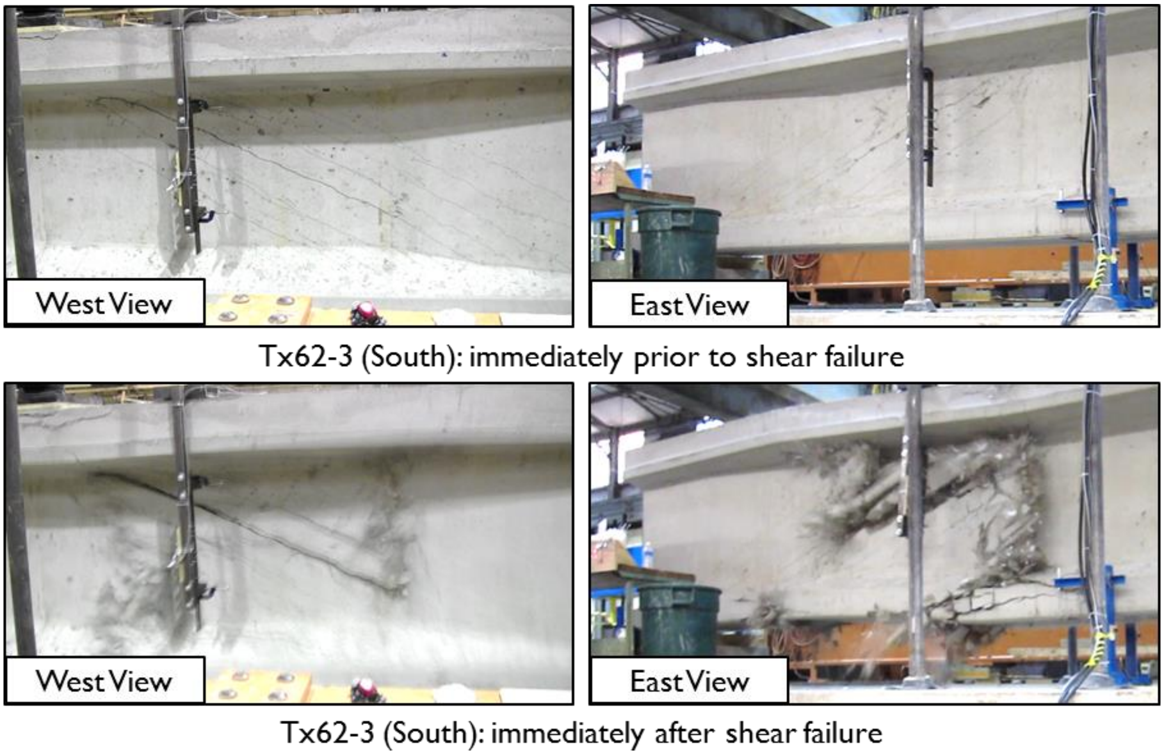

Figure 11: Crushing of Diagonal Strut in Control Specimen (Tx62-3 South) At the moment of failure & immediately after failure

Compressive Failure at Duct in Post-Tensioned Girders

The shear failure of the post-tensioned girders was controlled by the crushing of the web concrete just above the duct. This crushing failure initiated at the interface between the duct and the principal diagonal compressive strut as shown in Figure 12. After crushing of the diagonal strut, the shear load on the section dropped and although additional displacement was applied the girder never again reached the maximum shear obtained at the time of crushing. It is for this reason that the crushing at the location of the post-tensioning duct is referred to as the controlling shear failure mechanism for all test girders regardless of the amount of transverse reinforcement or duct size.

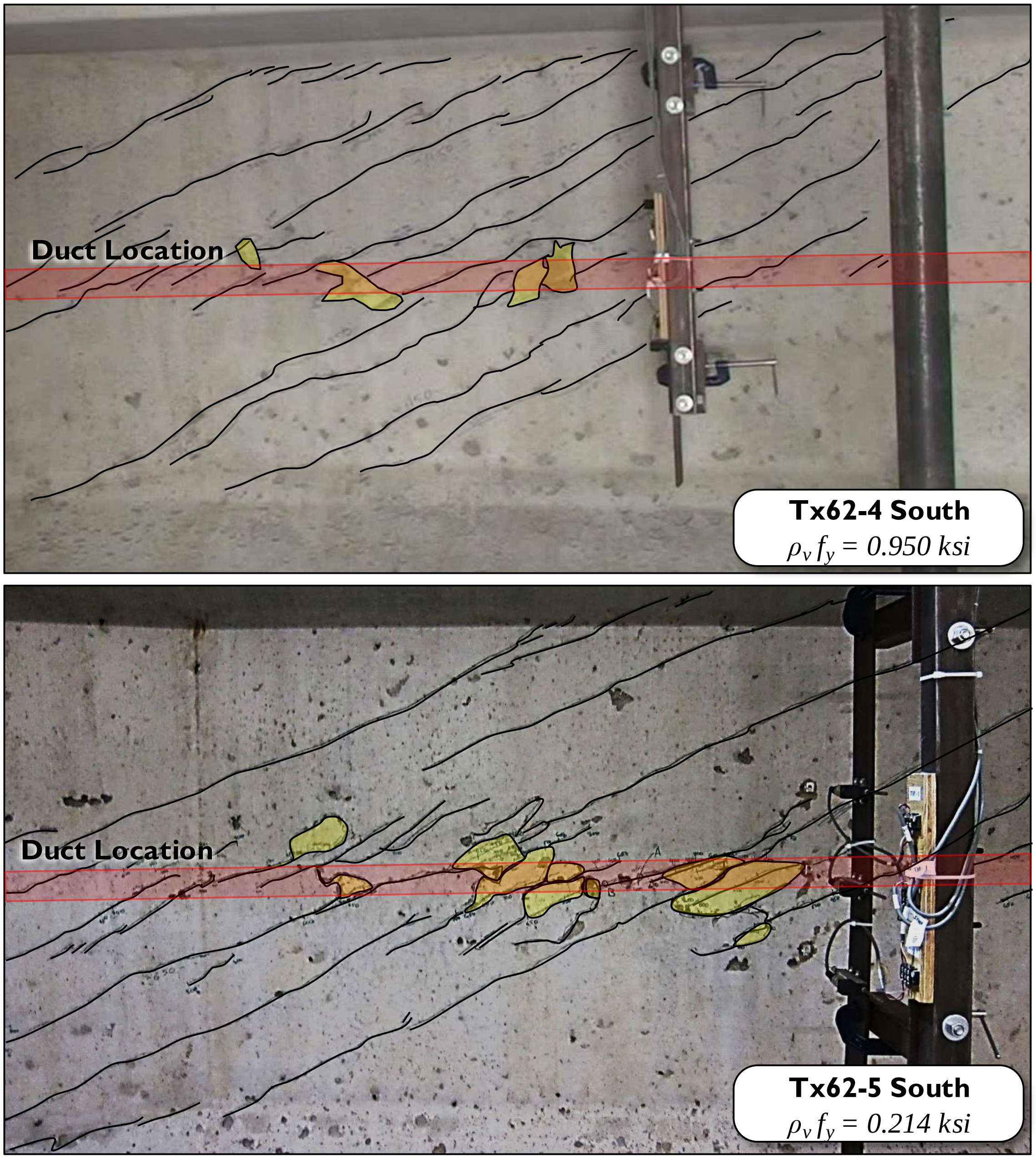

Figure 12: Crushing above the Duct Initiates Failure of Post-Tensioned Girders Regardless of Amount of Transverse Reinforcement Provided

SUMMARY OF SHEAR TEST DATA

Table 7 details the results of the Tx62 shear testing program. All AASHTO LRFD (2013) shear strength calculations were performed using the general shear provisions of AASHTO 2013 §5.8.3.1. The web widths used in these calculations accounted for the reduction in the effective web width as stipulated in AASHTO 2013 §5.8.2.9 that accounts for the presence of a post-tensioning duct. Because all post-tensioned girders contained grouted ducts the effective web width was taken as the gross thickness less one quarter of the duct diameter.

Table 7: Summary of Shear Test Results

Tx62 Shear Test Results									
Test	**Girder**		**Duct* Material [diam.]**	**Web Width** *(in.)*	$\frac{\emptyset_{duct}}{b_{gross}}$	$\frac{A_v f_{vy}}{s\, b_{gross}}$ *(psi)*	V_n *(kips)*	V_{test} *(kips)*	$\frac{V_{test}}{V_n}$
1	Tx62-1	S	3-in. (plastic)	7	0.43	638	609	687	1.13
2	Tx62-2	S	3-in. (steel)	7	0.43	650	652	816	1.25
3	Tx62-2	N	3-in. (steel)	7	0.43	650	643	749	1.17
4	Tx62-3	S	no duct	7	0.43	642	713	986	1.38
5	Tx62-4	S	3-in. (steel)	7	0.43	950	855	831	0.97
6	Tx62-4	N	3-in. (plastic)	7	0.43	950	845	832	0.98
7	Tx62-5	S	3-in. (plastic)	7	0.43	214	379	703	1.86
8	Tx62-5	N	3-in. (steel)	7	0.43	214	381	735	1.93
9	Tx62-6	S	4-in. (plastic)	9	0.44	854	946	930	0.98
10	Tx62-6	N	4-in. (steel)	9	0.44	854	967	1099	1.14
11	Tx62-7	S	3-in. (steel)	9	0.33	862	970	1166	1.20

***gray indicates steel post-tensioning duct; white indicates plastic post-tensioning duct*

The ratio of the test to calculated shear capacity of each girder is illustrated in Figure 13. Thirty percent of tests on post-tensioned girders were unconservative when compared to the shear capacity as calculated by the AASHTO LRFD (2013) general shear provisions. These three unconservative tests failed at shear loads within two to three percentage points of the calculated values. This indicates that the code provisions could be improved upon in terms of accuracy, but the shear capacity is close to the code values.

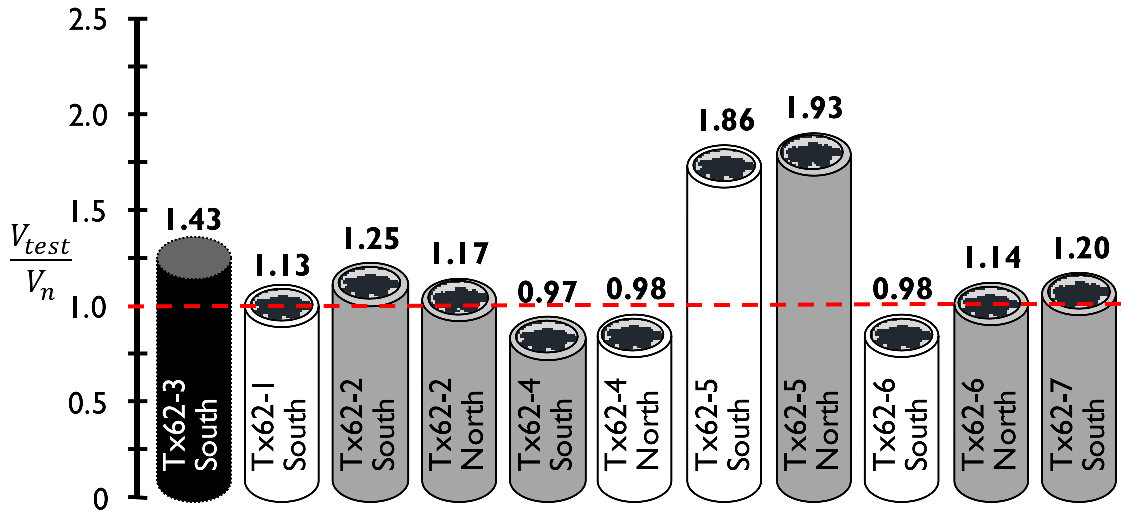

Figure 13: Comparison of AASHTO LRFD (2013) Calculated to Tested Strengths

Effect of Duct Material Type on Girder Strength

Unlike the results of small-scale panel testing programs,[2,8] the post-tensioning duct material had no discernable effect on the shear strength of the full scale girders. Three beams were cast which contained both plastic and steel post-tensioning ducts, which were coupled into a single tendon outside of the test region. These three beams allowed for a direct comparison of the effect of duct material on the shear strength of a post-tensioned girder because they isolated a number of other variables which could have influenced the shear behavior. The accuracy of the six tests on these three girders is illustrated in Figure 14. Although there is a slight variation between the tested and calculated capacities of each of these girders they are within the scatter of the data seen during this testing program as a whole. Moreover, these tests indicate that the severe reduction in strength seen in panel testing of plastic ducts does not hold true in full scale girder testing, and that the equations used for design should not follow calibrations from the small scale panel tests found in the literature[2,8].

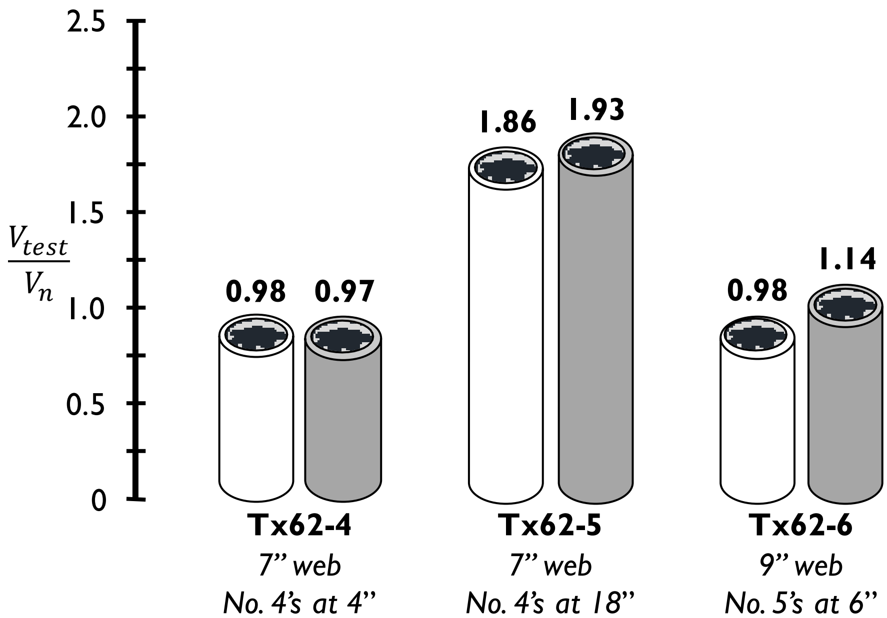

Figure 14: Effect of Duct Material on Girder Strength
(gray indicates steel post-tensioning duct; white indicates plastic post-tensioning duct)

CONCLUSIONS

The most recent change in a design code relating to the use of an effective web width occurred in the 2004 version of Eurocode2[4]. This code, for the first time, distinguished between plastic and steel post-tensioning duct materials and imposed a larger reduction in the effective web width of girders containing plastic post-tensioning ducts than those containing steel ducts. These changes resulted from small scale panel testing that showed a significant reduction in the crushing capacity for panels containing grouted plastic ducts compared to those containing grouted steel ducts[2]. An explanation of the differences in behavior between panel and full scale beam shear testing is beyond the scope of this article, but it has been shown through this beam testing program that grouted plastic ducts do not cause a significant reduction in strength when compared to those containing grouted steel ducts.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation.

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