SHEAR TESTS OF HIGH PERFORMANCE CONCRETE BULB-TEE GIRDERS

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ABSTRACT

Three 96-ft (29.3-m) long, 72-in. (1.83-m) deep, pretensioned bulb-tee girders were tested to evaluate behavior under static shear loadings. The three girders had a design concrete compressive strength of 10,000 psi (69.0 MPa) and incorporated 0.6-in. (15.2-mm) diameter, Grade 270, low relaxation prestressing strands. The shear reinforcement was designed to evaluate the applicability of the shear strength design provisions of the *AASHTO Standard Specifications for Highway Bridges* and the *AASHTO LRFD Bridge Design Specifications.* Shear reinforcement consisted of conventional bars or deformed welded wire reinforcement.

Prior to testing, a 10-ft (3.05-m) wide reinforced concrete deck slab was added to each girder. The six girder ends were tested to evaluate static shear strength. Measured strengths consistently exceeded the design strengths calculated by both AASHTO design approaches using both design and measured material properties.

Keywords: High-Performance Concrete, High-Strength Concrete, Bridge Girders, Prestressed Concrete, Shear Strength, Welded Wire Reinforcement

INTRODUCTION AND OBJECTIVES

The Louisiana Department of Transportation and Development (LADOTD) has been gradually introducing high performance concrete into its bridge construction program. At the same time, the Louisiana Transportation Research Center (LTRC) has been sponsoring research work to address design and construction issues related to the utilization of high performance concrete.

In 1988, a bridge project was used as an experiment to determine if a concrete compressive strength of 8,000 psi (55 MPa) could be obtained on a production project. The experiment was only partially successful as the contractor was penalized on 68 percent of the project's 2,370 ft (723 m) of prestressed concrete girder. In 1992, a 130-ft (39.6-m) long, square prestressed concrete pile with a compressive strength of 10,453 psi (72.1 MPa) was produced, shipped, and successfully driven without damage as part of the State Route 415 bridge over the Missouri Pacific Railroad. In 1993, two bridges on the Inner Loop Expressway near Shreveport were built using AASHTO Type IV girders with a specified compressive strength of 8,500 psi (59 MPa) at 28 days.

A 1994 LTRC report recommended that the LADOTD consider the implementation of concrete with compressive strengths up to 10,000 psi (69 MPa) in a bridge and the bridge should be instrumented to measure long-term behavior[1]. This was implemented with the design and construction of the Charenton Canal Bridge, which was opened to traffic in November 1999[2]. The successful construction of the Charenton Canal Bridge demonstrated that a high performance concrete bridge could be designed and built in the state of Louisiana using locally available materials.

Prior to the start of this research project, LADOTD was considering the use of 72-in. (1.83-m) deep bulb-tee girders for a future bridge project. The girders were expected to require the use of concrete with a specified compressive strength of 10,000 psi (69 MPa) and 0.6-in. (15.2-mm) diameter prestressing strands. During the course of the project, several other bridges with a specified strength of 10,000 psi (69 MPa) for the prestressed concrete girders were also designed. To obtain test data that will provide assurance that these girders will perform satisfactorily, a research program was initiated to evaluate the structural performance of bulb-tee girders under flexural fatigue and shear loading conditions.

The objectives of the proposed research were as follows:

- Provide assurances that 72-in. (1.83-m) deep prestressed concrete bulb-tee girders made with 10,000 psi (69 MPa) compressive strength concrete will perform satisfactorily under flexural fatigue and static shear loading conditions.
- Determine if a higher level of concrete tensile stress can be used in flexural design of high-strength prestressed concrete girders.
- Investigate the use of welded-wire deformed reinforcement as an alternative to

deformed bars for shear reinforcement.

This paper addresses the shear tests of the HPC bulb-tee girders.

METHODOLOGY

PROTOTYPE BRIDGE DESIGN

Two bridge superstructure designs utilizing 72-in. (1.83-m) deep bulb-tee girders on a 95-ft (28.96-m) long span were prepared by the LADOTD for the purpose of determining representative test specimen design details. The 72-in. (1.83-m) deep bulb-tee girders were selected to be representative of the girders to be used on an upcoming bridge project for LADOTD. The span length was selected by the research team based on transportation and laboratory handling limitations. One of the bridge designs was based on the *AASHTO Standard Specifications for Highway Bridges, 16th Edition, 1996*[3]. The second design was based on the *AASHTO LRFD Design Specifications, 2nd Edition, 1998*[4]. The prototype bridge designs were performed using CONSPAN V6.0 for the *Standard Specifications* design and CONSPAN LRFD V1.1 for the *LRFD Specifications* design[5,6].

Both designs were based on an overall bridge width of 46 ft 10 in. (14.27 m), with a curb-to-curb width of 44 ft (13.41 m) consisting of two 12-ft (3.66-m) wide travel lanes and two 10-ft (3.05-m) wide shoulders. A girder spacing of 13 ft 6 in. (4.11 m) was selected to minimize the number of girders and still utilize an 8-in. (203-mm) thick cast-in-place reinforced concrete deck. Concrete compressive strengths used in the design of the girders were 7,000 psi (48 MPa) at release of the strands and 10,000 psi (69 MPa) at 56 days. The cast-in-place concrete deck design compressive strength was taken as 4,200 psi (29 MPa). Both girder designs utilized 0.6-in. (15.2-mm) diameter, low-relaxation Grade 270 prestressing strands conforming to ASTM Designation: A416[7].

BRIDGE DESIGN LOADS

Dead loads used in the design of each bridge were based on a concrete unit weight for both the girder and deck of 150 lb/cu ft (2,403 kg/cu m). Design dead loads did not include superimposed loads from barrier rails or future wearing surface. Dead loads were assumed to be supported entirely by the non-composite bridge girders.

Live load classification used for the designs by the *Standard Specifications* and *LRFD Specifications* were HS 20 and HL-93, respectively. Calculated impact factors by the two specifications were 1.227 and 1.333.

SECTION PROPERTIES

Section properties for the bridges designed using both the *AASHTO Standard Specifications* and the *LRFD Specifications* are shown in table 1. In computing section properties for both designs, a 2-1/2-in. (64-mm) deep haunch is included. The section properties of the bulb-tee section for the two designs are slightly different because different dimensions were used for the cross section in the analyses. Based on the section dimensions, the calculated eccentricities of the strands at midspan are 33.13 and 33.10 in. (842 and 841 mm), for the designs by the *Standard Specifications* and *LRFD Specifications*, respectively.

Table 1
Bridge section properties

Section Property	Bulb-Tee Section		Composite Section	
	Standard Specifications	LRFD Specifications	Standard Specifications	LRFD Specifications
Effective compressive flange width, in.	—	—	138.0	117.0
Cross-sectional area, in.^{2}	767	767	1,551	1,442
Moment of inertia, in.^{4}	545,850	545,894	1,217,131	1,165,169
Height of center of gravity, in.	36.61	36.60	57.55	55.96
Section modulus-girder bottom, in.^{3}	14,910	14,915	21,148	20,821
Section modulus-girder top, in.^{3}	15,424	15,421	84,189	72,642
Section modulus-deck slab top, in.^{3}	—	—	48,769	43,902

A dash indicates that the property is not applicable.

As shown in table 1, the composite section properties for the design based on the *Standard Specifications* are greater than those based on the *LRFD Specifications*. The difference between the composite section properties for the two designs involves the calculation of the effective width of the compressive flange (deck slab). Using the provisions of the *Standard Specifications*, an effective compressive flange width of 138 in. (3.50 m) is calculated. Provisions of the *LRFD Specifications* produce an effective compressive flange width of 117 in. (2.97 m).

ALLOWABLE STRESSES AND STRESS LIMITS

Allowable stresses and stress limits are per the *Standard Specifications* and the *LRFD Specifications* respectively. For both designs, the girder tensile stress in the precompressed tensile zone controlled the design. For both the *Standard Specifications* and *LRFD Specifications*, the maximum allowable tensile stress in the precompressed tensile zone is $6\sqrt{f'_c}$. However, a value of $7.5\sqrt{f'_c}$ was used in the LRFD design to take advantage of the higher tensile strength of the high-strength concrete.

COMMENTS ON THE DESIGNS

For flexure, both designs resulted in girders requiring twenty-four 0.6-in. (15.2-mm) diameter Grade 270 low-relaxation strands. For both designs, six strands were required to be debonded at each end of the girders. For the *Standard Specifications* design, the strands were debonded in pairs for lengths of 21, 24, and 30 ft (6.4, 7.3, and 9.1 m). For the *LRFD Specifications* design, the six strands were all debonded for a length of 9 ft (2.7 m). Calculated prestress losses at release were 15.70 ksi (108 MPa) and 14.54 ksi (100 MPa) for the *Standard Specifications* design and *LRFD Specifications* design, respectively. Corresponding calculated final losses were 43.57 ksi (300 MPa) and 45.59 ksi (314 MPa). In calculating prestress loss due to concrete shrinkage, a relative humidity of 70 percent was assumed.

Shear design in the *LRFD Specifications* utilizes a different approach from the shear design in the *Standard Specifications*. Consequently, the requirements for shear reinforcement were different even though the factored shear forces were approximately the same. In the *Standard Specifications* design, the critical section for shear is taken at a distance of one half the overall depth of the composite section from the support. This was 3.44 ft (1.05 m). In the *LRFD Specifications* design, the location of the critical section is dependent on the angle of the inclined compressive stresses and was calculated to be 6.52 ft (1.99 m) from the support.

At the critical section in the *Standard Specifications* design, the required shear reinforcement was 0.47 sq in./ft (1.0 sq mm/mm). This is equivalent to two No. 4 (13-mm diameter) stirrups at 10-in. (254-mm) spacing. At the critical section in the *LRFD Specifications* design, the required shear reinforcement was 0.65 sq in./ft (1.4 sq mm/mm). This is equivalent to two No. 4 (13-mm diameter) stirrups at 7-in. (178-mm) spacing.

TEST SPECIMENS

The three test specimens were designated BT6, BT7, and BT8, to follow the numbering sequence established from the previous feasibility study[1]. The ends of each specimen were designated “live” and “dead” corresponding to their locations in the precasting bed. Design of Test Specimen BT6 was based on the prototype bridge design using the *Standard Specifications*. Designs of Specimens BT7 and BT8 were based on the prototype bridge design using the *LRFD Specifications*.

SHEAR DESIGN

The major differences between the test specimens and the prototype bridge girders occur in the shear reinforcement details for the following reasons:

1. The shear reinforcement in the prototype bridge girders was calculated to support factored dead and live loads on a girder span of 95 ft (29.3 m). For the shear tests, approximately one-half of the girder was supported on a span of 46 ft 8 in.

(14.2 m) with concentrated test loads applied near the as-cast ends of the girder. The shorter span length and concentrated loads were used to increase the likelihood of a shear failure at the as-cast end of the girder before a flexural failure.

2. The prototype bridge designs were based on factored dead loads and live loads. The dead loads were generally uniformly distributed along the span. The live loads were either concentrated loads or a combination of uniformly distributed loads and concentrated loads. In the test specimens, the majority of the shear force was produced by the concentrated test loads.

3. The prototype bridge design using the *LRFD Specifications* was made using CONSPAN LRFD V1.1[6]. This version of the program did not include a revision to the shear design provisions that was introduced into the 2000 Interim Revisions to the *LRFD Specifications*[8]. However, this revision was used in the shear design of the test specimens.

Shear design in the *LRFD Specifications,* involves a term A_{ps}, defined as the area of prestressing steel on the flexural tension side of the member, reduced for any lack of full development at the section under investigation. No guidance is provided on how to calculate the lack of full development. In the commentary to the section dealing with longitudinal reinforcement, it states that in calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the development length or the transfer length may be assumed. Since the Federal Highway Administration requires a multiplier of 1.6 on the basic development length, the transfer length and development length are significantly different. For the prototype bridge, the transfer and development lengths are 3 ft and 13.635 ft (914 mm and 4.156 m), respectively. The design of the prototype bridge using the *LRFD Specifications* utilized a length of 13.635 ft (4.156 m). Since the required amount of shear reinforcement can vary significantly depending on the value of A_{ps}, it was decided that the assumed value of A_{ps} should be a primary variable in designing the shear reinforcement for opposite ends of both BT7 and BT8. Girder BT7 had individual bars as shear reinforcement. Girder BT8 used welded wire reinforcement.

Shear reinforcement in each test girder was divided into three regions:

The first or end region consisted of reinforcement at the end of the girder that is a standard LADOTD detail. This consisted of two No. 5 bars or two D20 wires at 4-in. centers for a length of 2 ft-8 in. (16-mm diameter bars at 203-mm centers for 813 mm).

The second region consisted of shear reinforcement between the end of the first region and the concentrated load points. This is the region in which the shear failure was expected to occur during testing and is referred to as the test region. The shear reinforcement in this region was that required at the critical section in the corresponding prototype bridge and was maintained constant throughout the region.

The third or midspan region consisted of shear reinforcement from the concentrated load points to midspan to prevent shear failure in this region during the shear test.

Details of the shear reinforcement in each region are shown in table 2. For Girder BT6, the shear reinforcement in the test region at the live end was that calculated for the critical section in the prototype bridge using the *Standard Specifications*. This reinforcement consisted of two No. 4 (13-mm diameter) bars at 10-in. (254-mm) centers. At the dead end of Girder BT6, the shear reinforcement in the test region consisted of an equivalent quantity of welded wire reinforcement. In calculating the area of the welded wire reinforcement, a strength of 70 ksi (483 MPa) was used instead of the 60 ksi (414 MPa) that was used for the bars. This resulted in pairs of D20 (13-mm diameter) welded wire reinforcement at 12-in. (305-mm) centers.

Table 2
Specimen details

Specimen	Design Specification	Deck Concrete Cementitious Materials	Girder End	Shear Reinforcement Details		
				End Region	Test Region	Midspan Region
BT6	Standard	Cement and ground granulated blast-furnace slag (50%)	Live	No. 5 stirrups at 4 in.	No. 4 stirrups at 10 in.	No. 4 stirrups at 16 in.
			Dead	D31 welded wire reinf. at 4 in.	D20 welded wire reinf. at 12 in.	D20 welded wire reinf. at 16 in.
BT7	LRFD	Cement and silica fume (5%)	Live	No. 5 stirrups at 4 in.	No. 4 stirrups at 6-1/2 in.	No. 4 stirrups at 16 in.
			Dead	No. 5 stirrups at 4 in.	No. 4 stirrups at 15 in.	No. 4 stirrups at 16 in.
BT8	LRFD	Cement and fly ash (20%)	Live	D31 welded wire reinf. at 4 in.	D20 welded wire reinf. at 8 in.	D20 welded wire reinf. at 16 in.
			Dead	D31 welded wire reinf. at 4 in.	D20 welded wire reinf. at 18 in.	D20 welded wire reinf. at 16 in.

The shear reinforcement in the test region of Girders BT7 and BT8 was based on the design of the prototype bridge using the *LRFD Specifications* but including the revisions published in the 2000 Interim Revisions[8]. As discussed previously, the assumed value of the effective area of the prestressing steel on the flexural tension side of the member has a significant effect on the required amount of shear reinforcement. The effective area of the prestressing steel depends on the assumed variation of resistance over the transfer and development length of the strand. Consequently, it was decided to design one end of Girder BT7 based on a linear variation of resistance over the transfer length of 60 in.

(1.52 m) followed by a parabolic variation from the end of the transfer length to the end of the development length of 8.52 ft (2.60 m) without the multiplier of 1.6. The other end of Girder BT7 was based on a linear variation of resistance over the development length including the multiplier of 1.6 for a total length of 13.64 ft (4.16 m). The design resulted in two No. 4 (13-mm diameter) bars at 6.5-in. (165-mm) centers at the live end and two No. 4 (13-mm diameter) bars at 15-in. (381-mm) centers at the dead end.

Shear reinforcement in Girder BT8 consisted of welded wire reinforcement with an equivalent quantity to that of the bars in Girder BT7. A strength of 70 ksi (483 MPa) for the welded wire reinforcement was used when determining the shear reinforcement. This resulted in pairs of D20 (13-mm diameter) welded wire reinforcement at 8-in. (203-mm) centers at the live end and pairs of D20 (13-mm diameter) welded wire reinforcement at 18-in. (457-mm) centers at the dead end.

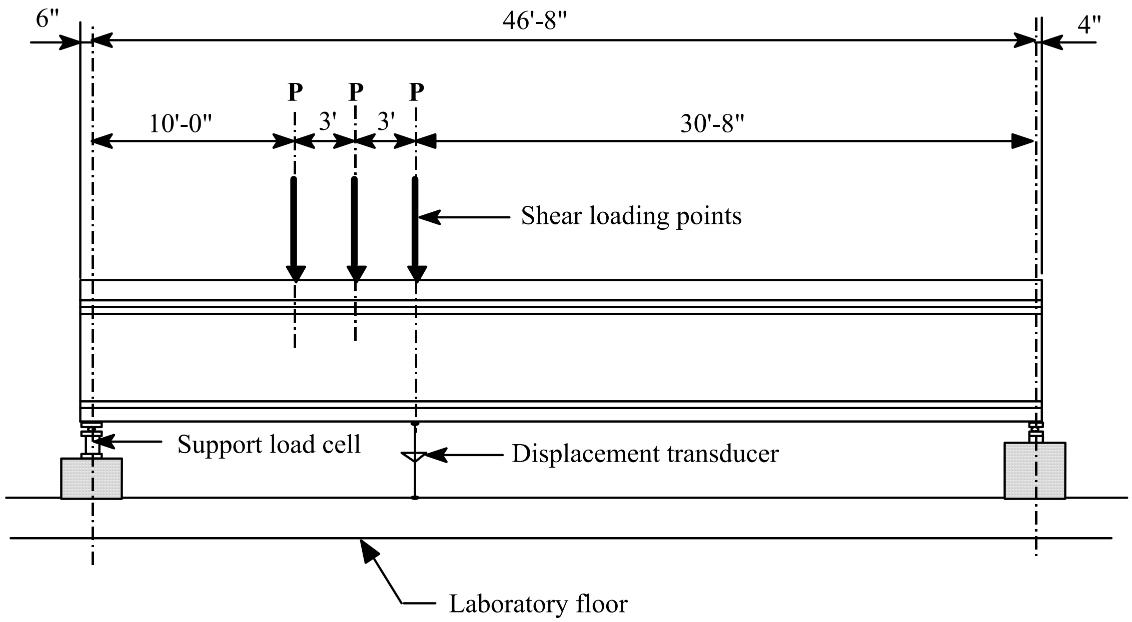
The *LRFD Specifications* also require a check of the internal longitudinal force at the end of the girder. This is to ensure that there is adequate reinforcement to resist the horizontal component of force along the diagonal compression strut caused by shear. Based on this check, additional reinforcement, consisting of 8 No. 6 (19-mm diameter) bars, was provided at both ends of Girders BT7 and BT8. Four bars had a length of 7 ft (2.13 m) and four bars had a length of 19 ft (5.79 m).

SHEAR TEST SETUP AND PROCEDURE

After completion of the fatigue loading test, the two ends of each specimen were tested to evaluate static shear strength performance. Each specimen was cut into two and each specimen half placed on supports creating a simply-supported span. One support was centered on the sole plate location at the as-cast end of the girder. Location of the second support was selected with the objective of inducing a shear failure at the as-cast end of the girder prior to exceeding the flexural strength near the midspan region. The test configuration for the shear tests is shown in figure 1. For BT8-Dead, the span length was reduced from 46 ft 8 in. (11.85 m) to 43 ft 0 in. (13.10 m) because of damage at midspan during the fatigue test.

Load was applied to each specimen using three concentrated load points. The first loading point was located 10 ft (3.05 m) from the end reaction. Two additional loading points were provided at 3-ft (914-mm) intervals. Equal loads were applied at each loading point using two hydraulic jacks. Load cells were used to monitor the applied load at the six loading points and the support reactions at the as-cast end of the girder. Potentiometers were used to monitor specimen displacements at the location of maximum applied bending moment. Two prestressing strands protruding from the as-cast end of the girder were instrumented with displacement transducers to detect any strand slippage relative to the concrete.

Load was applied incrementally to each specimen. Output from all instrumentation was monitored continuously using a DDAS and computer. At selected intervals (load stages), data were stored on disk to provide a permanent record of test specimen behavior. Tests were terminated when the specimen could no longer sustain additional load or the capacity of the test equipment was reached.



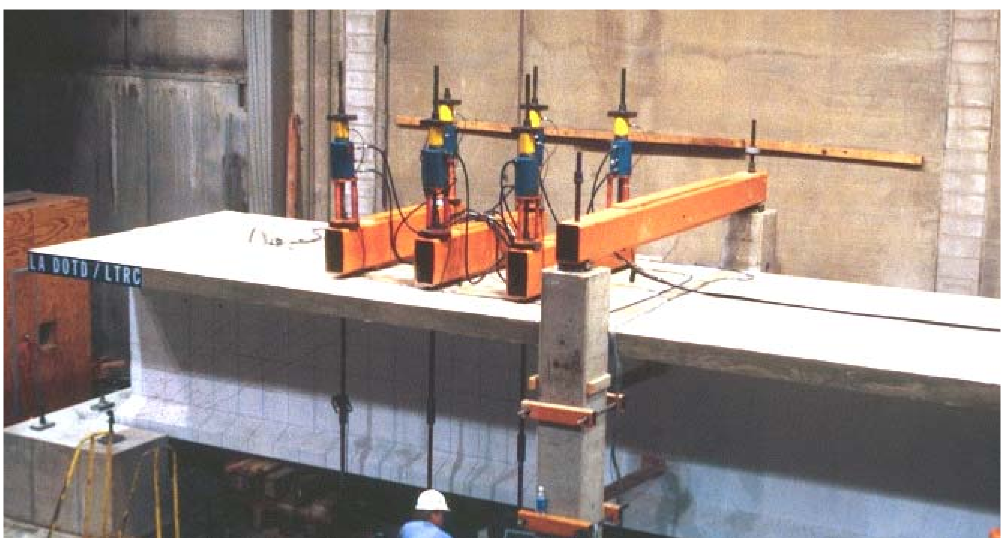


Figure 1
Shear test configuration

DISCUSSION OF RESULTS

SHEAR TESTS

The shear tests were conducted by incrementally loading each specimen until the specimen could no longer sustain additional load or the capacity of the test equipment was reached. The first diagonal crack in each specimen occurred at an applied shear that ranged from 270 to 302 kips (1.20 to 1.34 MN) as reported in table 3. Applied shear is the shear force produced in the test region from the hydraulic rams and is calculated from the load cells at each loading point. The applied shear does not include the self weight of the specimen or the weight of the loading equipment. Measured strengths of the concrete, prestressing strand, and nonprestressed reinforcement are given in table 4.

Table 3
Summary of shear test results

Specimen	BT6		BT7		BT8	
End	Live	Dead	Live	Dead	Live	Dead
Applied Shear, kips						
First Crack	270	275	299	295	302	291
Maximum	592	557	614[a]	605	599[a]	564
Angle of Diagonal Crack from the Horizontal						
First Crack	44	45	38	34	39-43	41
Range	30-44	30-45	30-46	29-43	32-44	31-46

a Test stopped at the load capacity of the test equipment.

Table 4
Specified and measured material properties

Property	Specified Value	Measured Value					
		BT6		BT7		BT8	
		Live	Dead	Live	Dead	Live	Dead
Concrete Strength, psi							
Girder	10,000	11,780	11,590	12,400	12,730	11,850	11,310
Deck	4,200	5,780	4,860	7,330	7,950	7,340	6,850
Steel Strength, ksi							
Prestressing Strand	270	284.0	284.0	284.0	284.0	284.0	284.0
No. 4 Bar	60	62.5	62.5	62.5	62.5	62.5	62.5
D20 Wire	70	85.0	85.0	85.0	85.0	85.0	85.0
No. 6 Bar	60	65.5	65.5	65.5	65.5	65.5	65.5

When the first diagonal crack formed, a large increase in strain occurred in the stirrups intercepted by the crack as shown in figure 2 for Stirrups 5 and 6 of Specimen BT8-Live. A similar pattern of behavior occurred in the other specimens. After the first diagonal crack formed, further increases in the applied shear caused more additional diagonal cracks to form and existing cracks to extend. As a diagonal crack crossed each instrumented stirrup, a large increase in stirrup strain was measured. This behavior continued until the end of the test. The maximum applied shears for each specimen are shown in table 3 together with the angle of the diagonal cracks as measured from the horizontal. A photograph of the diagonal crack pattern in Specimen BT8-Dead is given in figure 3. A description of the behavior of each specimen as it relates to the maximum shear is given in the following sections.

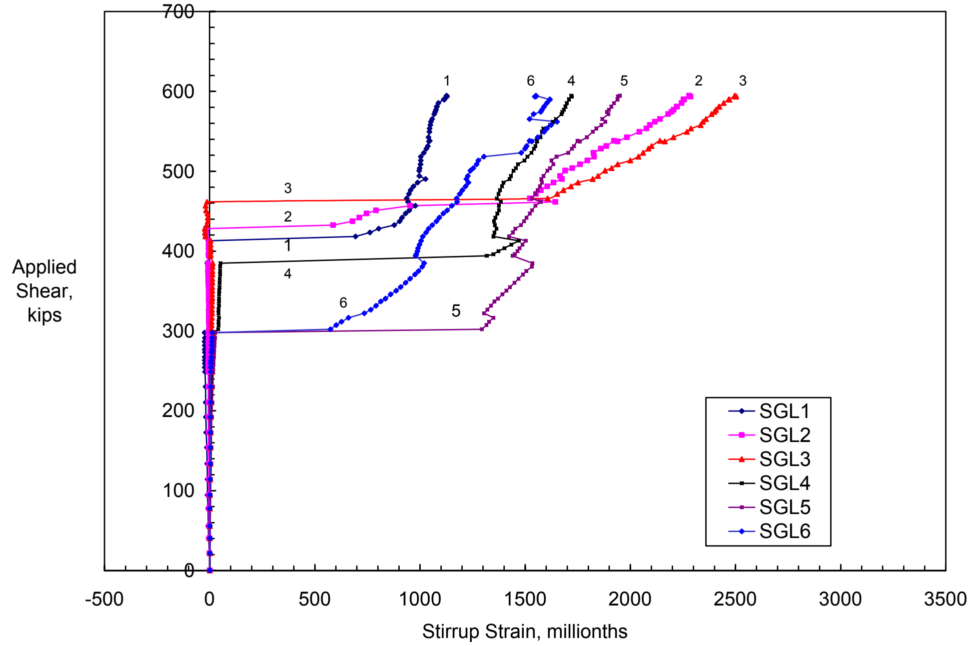


Figure 2
Stirrup strains in Specimen BT8-Live

Figure 3
Specimen BT8-Dead after the shear test

BT6-Live End

At an applied shear of about 430 kips (1.91 MN), a gradual increase in strand slip was measured on the two dial gages attached to the prestressing strands. The strand slip increased until the maximum shear of 592 kips (2.63 MN) was reached. By that time, a slip of approximately 0.5 in. (13 mm) was measured. Based on this information, the maximum shear applied to BT6-Live was limited by strand slip. However, three of the instrumented stirrups had exceeded their yield strength but were well below the ultimate elongation and stirrup strength.

BT6-Dead End

The first large increase in strand slip on BT6-Dead occurred at an applied shear of 502 kips (2.23 MN). Further increases in applied shear caused additional slip. Based on this information and observation of the test specimen at the end of the test, it was concluded that the maximum shear applied to BT6-Dead was limited by strand slip. At the maximum load, one instrumented stirrup had strains of about 0.005 and most stirrups had strains of about 0.002, although two gages had ceased to provide reliable data.

BT7-Live End

Loading of BT7-Live was stopped when the capacity of the test equipment was reached at an applied shear of 614 kips (2.73 MN). Prior to reaching the end of the test, one strand had a measured slip of 0.34 in. (8.6 mm) while the other instrumented strand showed no slip. Measured strains in the longitudinal nonprestressed reinforcement indicated that it had not reached the yield point. Measured strain in one stirrup was almost at the yield point. Based on these data, it is likely that BT7-Live could have sustained additional shear before reaching its capacity.

BT7-Dead End

Strand slip on BT7-Dead began at an applied shear of about 332 kips (1.14 MN) in one strand only and then steadily increased. However, the other instrumented strand showed no slip throughout the whole test. At the maximum shear on BT7-Dead, a combination of web crushing at the lower end of the diagonal strut and horizontal shear along the web-bottom flange interface occurred. At that time, measured strains indicated that the longitudinal reinforcement had not exceeded its yield strength and three instrumented stirrups had stresses at or greater than their yield strength.

BT8-Live End

Loading of BT8-Live was stopped when the capacity of the test equipment was reached at an applied shear of 599 kips (2.66 MN). Prior to and after the end of the test, there was no consistent evidence of strand slip. Measured strains in the longitudinal nonprestressed reinforcement indicated that the stress had not reached the yield strength. Based on these data, it is likely that BT8-Live would have sustained additional shear before reaching its capacity.

BT8-Dead End

Throughout the testing of BT8-Dead, there was no consistent evidence of strand slip. At the maximum applied shear of 564 kips (2.50 MN), a combination of concrete spalling in the webs, horizontal shear along the web-bottom flange interface, and vertical longitudinal splitting in the bottom flange directly below the faces of the web occurred. A photograph of the end of BT8-Dead after testing is shown in figure 3.

COMPARISON OF RESULTS

The measured applied shear forces at first cracking and maximum load are compared with the specimens arranged in pairs and are shown in figure 4. Each pair of specimens represents one with individual bars and the one with welded wire reinforcement designed for the same shear strength. The spacing of the stirrups in the specimens with individual bars was based on a yield strength of 60 ksi (414 MPa), whereas, 70 ksi (483 MPa) was used for the specimens with welded wire reinforcement. For the first four specimens, the two specimens with the welded wire reinforcement have measured strengths slightly lower than the corresponding specimens with individual bars. However, the strength difference is not that significant. A comparison of strengths for the last two specimens cannot be made because the maximum applied load was limited by the strength of the test equipment and not by the strength of the specimens.

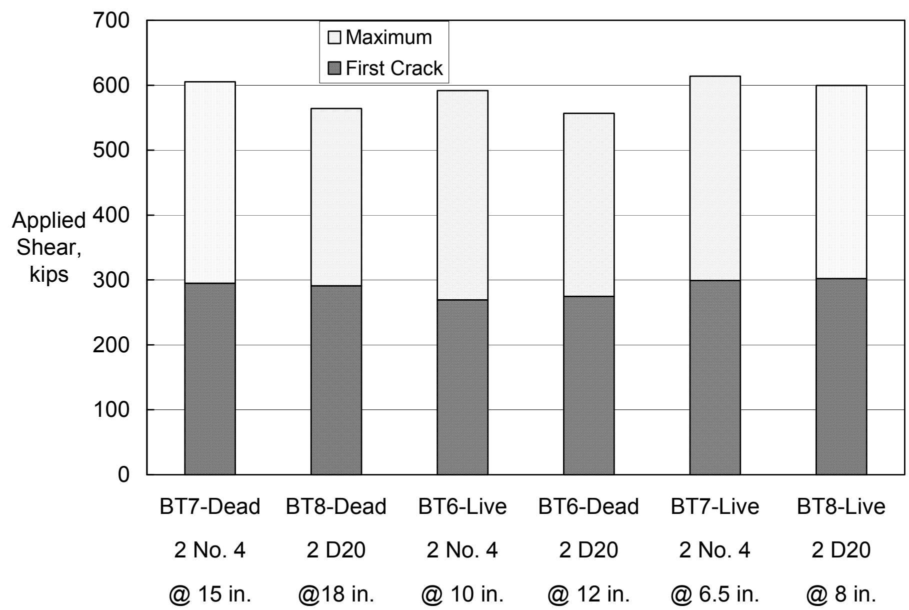


Figure 4
Comparison of applied shear forces

The pairs of specimens are also arranged by decreasing stirrup spacing. A decrease in stirrup spacing would normally result in an increase in shear strength. However, this did not occur for the BT6 specimens because the nonprestressed longitudinal reinforcement was not included in the bottom flange. This reinforcement consisted of 4 No. 6 bars 7 ft

(2.13 m) long and 4 No. 6 bars 19 ft (5.79 m) long. The lack of this reinforcement changed the performance of the specimen to one in which the strength was controlled by slip of the strand. The presence of the longitudinal reinforcement in BT7-Dead and BT8-Dead was beneficial in preventing slip of the strand and enhancing the strength of the test specimens.

COMPARISON OF MEASURED STRENGTHS WITH THE AASHTO SPECIFICATIONS

The shear strength design of the specimens was based on the prototype bridge design and its loadings. However, if the test specimens had been loaded with a configuration corresponding to the design loading, they would probably have failed in flexure and nothing would have been learned about the shear strength. Consequently, they were tested with a shorter span to reduce the bending moment and the loads were placed closer to the as-cast end of the girder to induce the shear failure at that end. This necessitated that the girders be analyzed based on the actual loading configuration.

The shear strength of each specimen was calculated for the following four different sets of assumptions:

1. Using the provisions of the *AASHTO Standard Specifications*[3] with specified material properties and nominal section dimensions corresponding to a design situation

2. Using the provision of the *AASHTO Standard Specifications*[3] with measured material properties, measured self weights, and measured cross-sectional dimensions corresponding to an analysis of an as-built structure

3. Using the provisions of the sectional design model of the *AASHTO LRFD Specifications*[4] with specified material properties and nominal section dimensions corresponding to a design situation

4. Using the provisions of the structural design model of the *AASHTO LRFD Specifications*[4] with measured material properties, measured self weights, and measured cross-sectional dimensions corresponding to an analysis of an as-built structure

Material properties used in the analyses are listed in table 4.

Analyses using the provisions of the *AASHTO Standard Specifications* were straight forward since the provisions can be used easily for both design and analysis. Analyses using the *AASHTO LRFD Specifications* were considerably more complex because the provisions provide a procedure for design and not for analysis. To use the provisions for analysis, it is necessary to assume the applied loads and angle of the diagonal compressive stresses. Analyses are then made to calculate the load and angle. In hand calculations, many iterations were needed to arrive at a solution where the assumed load and angle matched the calculated values.

Once the angle was obtained, a check was made to determine if the capacity was limited by the longitudinal reinforcement at the end of the member. The commentary to the *AASHTO LRFD Specifications* states that a linear variation of resistance over the development length or the transfer length may be assumed in determining the tensile resistance of the longitudinal reinforcement at the end of the girder. In performing the calculations, a linear variation of strand stress was assumed along the transfer length starting from zero stress at the end of the girder to the value after losses at the end of the transfer length. A parabolic distribution of stress was assumed from the end of the transfer length to the end of the development length, which was calculated with a K factor of 1.6.

Values of shear strength calculated using the four methods of analyses are shown in tables 5 and 6 for calculations using the *AASHTO Standard Specifications* and the *AASHTO LRFD Specifications*, respectively. Values for the nominal shear strength provided by the concrete, V_c,, and the nominal shear strength, V_n, are included for comparison with the measured shear strengths. The calculated strengths in tables 5 and 6 use a φ factor of 1.0. The measured strengths include the self weight shear, loading equipment shear, and applied shear. Some variation in the measured strengths occur because the calculated critical section for shear is not the same for all the analyses and this affects the contribution of the self weight to the measured shear strength.

Table 5
Comparison of measured strengths calculated using the *AASHTO Standard Specifications*

Girder	Shear Reinforcement	Specified Properties		Measured Properties	
		Measured Strengths	Calculated Strengths	Measured Strengths	Calculated Strengths
Nominal Shear Strength Provided by the Concrete, V_c, kips					
BT6-Live	No. 4 at 10 in.	309	187	308	203
BT6-Dead	D20 at 12 in.	314	187	313	204
BT7-Live	No. 4 at 6-1/2 in.	339	187	339	203
BT8-Live	D20 at 8 in.	342	187	341	196
BT7-Dead	No. 4 at 15 in.	335	187	334	198
BT8-Dead	D20 at 18 in.	327	186	327	192
Nominal Shear Strength, V_n, kips					
BT6-Live	No. 4 at 10 in.	630	371	630	395
BT6-Dead	D20 at 12 in.	596	366	595	422
BT7-Live	No. 4 at 6-1/2 in.	654[a]	471	653[a]	499
BT8-Live	D20 at 8 in.	639[a]	456	638[a]	523
BT7-Dead	No. 4 at 15 in.	645	310	644	327
BT8-Dead	D20 at 18 in.	600	306	600	338

a Test stopped at the load capacity of the test equipment.

In the case of the nominal shear strength calculations using the *AASHTO LRFD Specifications*, two values are reported in table 6. The first value is the nominal shear strength on the basis that it is not controlled by the amount of longitudinal reinforcement. The second value corresponds to the limit based on the strength being controlled by the longitudinal reinforcement. This had a big effect on the calculated strengths of BT6 because no nonprestressed reinforcement was provided at the ends of the girder. Nevertheless, the measured strengths were still in excess of the calculated strengths even when the limitation was not included.

Table 6
Comparison of measured strengths calculated using the *AASHTO LRFD Specifications*

Girder	Shear Reinforcement	Specified Properties		Measured Properties	
		Measured Strengths	Calculated Strengths	Measured Strengths	Calculated Strengths
Nominal Shear Strength Provided by the Concrete, V_c, kips					
BT6-Live	No. 4 at 10 in.	303	118	303	132
BT6-Dead	D20 at 12 in.	308	119	308	127
BT7-Live	No. 4 at 6-1/2 in.	334	106	334	123
BT8-Live	D20 at 8 in.	336	108	336	117
BT7-Dead	No. 4 at 15 in.	329	135	329	156
BT8-Dead	D20 at 18 in.	321	136	322	143
Nominal Shear Strength, V_n, kips					
BT6-Live	No. 4 at 10 in.	625	440 278[a]	625	454 326[a]
BT6-Dead	D20 at 12 in.	590	435 275[a]	590	474 344[a]
BT7-Live	No. 4 at 6-1/2 in.	648[b]	534 478[a]	648[b]	556 549[a]
BT8-Live	D20 at 8 in.	634[b]	522 467[a]	634[b]	578 573[a]
BT7-Dead	No. 4 at 15 in.	639	385 336[a]	639	406 390[a]
BT8-Dead	D20 at 18 in.	594	381 333[a]	595	419 408[a]

a Strength limited by longitudinal reinforcement.

b Test stopped at the load capacity of the test equipment.

A graphical comparison of measured and calculated strengths using the *AASHTO Standard Specifications* is given in figure 5. As expected, the strengths calculated using the measured material properties are slightly higher than the strengths calculated using the specified properties. In all cases, the measured strengths are greater than the calculated strengths even for the BT6 specimens, where the strength was limited by strand slip. The calculated strengths in figure 5 also exhibit the expected result that shear

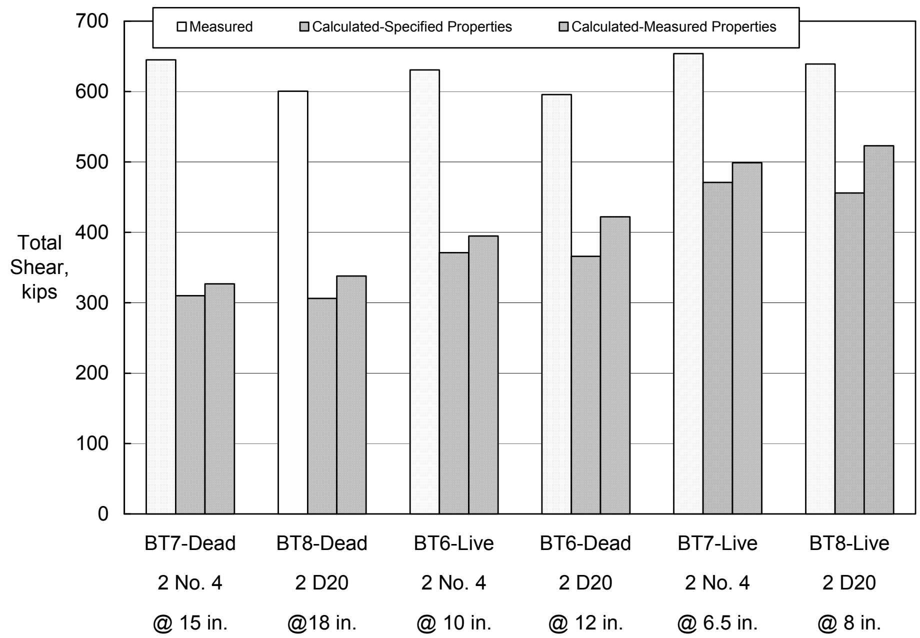


Figure 5
Comparison of measured and calculated strengths using the *AASHTO Standard Specifications*

strength increases as the shear reinforcement spacing decreases. In all tests, the measured strengths were greater than the calculated strengths.

A graphical comparison of measured and calculated strengths using the *AASHTO LRFD Specifications* is given in figure 6. In all tests, the measured strengths were greater than the calculated strengths. The calculated strengths are the lower values given in table 6, which correspond to the strengths being limited by the longitudinal reinforcement capacity. Strengths calculated using the measured material properties are again higher than those calculated using the specified properties. The calculated strengths for the BT6 specimens are lower than for the BT7-Dead and BT8-Dead specimens because the BT6 specimens were designed by the *AASHTO Standard Specifications* and did not have the additional nonprestressed reinforcement in the bottom flange at the ends of the girders.

The large difference between the measured and calculated shear strengths may be partially attributed to the short distance of 10 ft (3.05 m) between the end reaction and the first concentrated load point. In this case, loads may be transferred directly to the support by compressive arch action. Accordingly, a strut-and-tie analysis as permitted by the *AASHTO LRFD Specifications* may provide a closer estimate of the measured strengths.

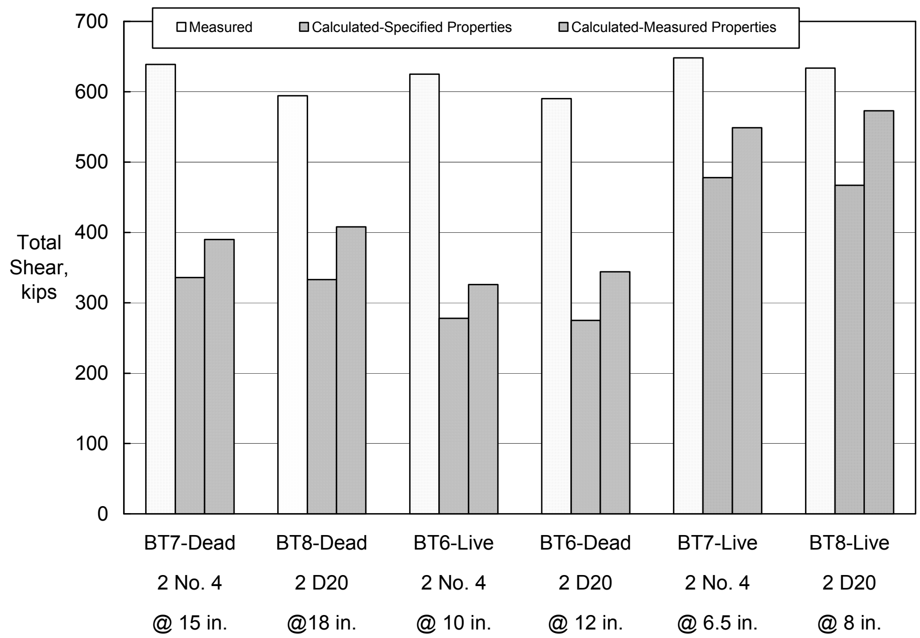


Figure 6
Comparison of measured and calculated strengths using the *AASHTO LRFD Specifications*

CONCLUSIONS FROM THE SHEAR TESTS

The following conclusions are based on the results of the six shear tests conducted in this project:

- All measured shear strengths were greater than the strengths calculated using the *AASHTO Standard Specifications* and the *AASHTO LRFD Specifications* using both specified and measured material properties.

- The shear design approach of the *AASHTO Standard Specifications* is applicable to precast, prestressed concrete beams with concrete compressive strengths up to 13,000 psi (90 MPa).

- The sectional design model of the *AASHTO LRFD Specifications* is applicable to precast, prestressed concrete beams with concrete compressive strengths up to 13,000 psi (90 MPa).

- The use of deformed welded wire reinforcement with a specified yield strength of 70,000 psi (483 MPa) provided an acceptable alternative to conventional deformed bars with a specified yield strength of 60,000 psi (414 MPa).

- Reinforcement with yield strengths greater than 60,000 psi (414 MPa) may be successfully used in the design of shear reinforcement in precast, prestressed concrete beams.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on the test program and test results described in this report:

- Six 72-in. (1.83-m) deep prestressed concrete bulb-tee girders with concrete compressive strengths greater than 10,000 psi (69 MPa) had measured shear strengths greater than the shear strengths calculated using the procedures of the *AASHTO Standard Specifications* and the Sectional Design Model of the *AASHTO LRFD Specifications* when either specified or measured material properties were used.

- Two 72-in. (1.83-m) deep prestressed concrete bulb-tee girders with concrete compressive strengths greater than 10,000 psi (69 MPa) and containing welded wire deformed reinforcement had measured shear strengths greater than the shear strengths calculated using the procedures of the *AASHTO Standard Specifications* and the Sectional Design Model of the *AASHTO LRFD Specifications* when either specified or measured material properties were used.

- The existing limitation of 60,000 psi (414 MPa) for the design yield strength of transverse reinforcement in both the *AASHTO Standard Specifications* and the *AASHTO LRFD Specifications* is conservative and higher reinforcement yield strengths can be utilized in the design of prestressed concrete beams.

- A maximum design strength of 75 KSI (517 MPa) may be conservatively used in the design of transverse reinforcement using welded wire deformed reinforcement.

The following recommendations are based on the conclusions listed above:

- 72-in. (1.83-m) deep prestressed concrete bulb-tee girders made with 10,000 psi (69 MPa) compressive strength concrete will perform satisfactorily under static shear loading conditions when designed by either the *AASHTO Standard Specifications* or the Sectional Design Model of the *AASHTO LRFD Specifications*.

- Welded wire deformed reinforcement with a yield strength of 75 KSI (517 MPa) may be used as an alternative to deformed bars for shear reinforcement in prestressed concrete beams.

- LADOTD may implement the use of 72-in. (1.83-m) deep prestressed concrete bulb-tee girders with 10,000 psi (69 MPa) compressive strength concrete designed by the existing provisions of either the *AASHTO Standard Specifications* or the *AASHTO LRFD Specifications* with the knowledge that the girder performance will be satisfactory.

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