TECHNICAL REPORT STANDARD PAGE

1. Report No. 382	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle FATIGUE AND SHEAR BEHAVIOR OF HPC BULB-TEE GIRDERS INTERIM REPORT	5. Report Date October 2003 6. Performing Organization Code	
7. Author(s) Dr. Bob Bruce, Dr. Henry Russell, and Mr. John Roller	8. Performing Organization Report No.	
9. Performing Organization Name and Address Department of Civil and Environmental Engineering Tulane University New Orleans, Louisiana 70118	10. Work Unit No. 11. Contract or Grant No. State Project Number: 736-99-0821 LTRC Project Number: 00-6S	
12. Sponsoring Agency Name and Address Louisiana Transportation Research Center 4101 Gourrier Avenue Baton Rouge, LA 70809	13. Type of Report and Period Covered Interim Report Period February 2000 – October 2003 14. Sponsoring Agency Code	
15. Supplementary Notes		

16. Abstract

Three 96-ft (29.3-m) long, 72-in. (1.83-m) deep, precast, pretensioned bulb-tee girders were tested to evaluate behavior under flexural fatigue and 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 quantities at the ends of each girder were selected 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.

The three prestressed concrete girders were produced in a commercial plant. Prior to testing, a 10-ft (3.05-m) wide reinforced concrete deck slab was added to each girder. After completion of fatigue testing, each girder was cut in half and the six girder ends tested to evaluate static shear strength.

The bulb-tee girder performed satisfactorily under 5,000,000 cycles of flexural fatigue loading when the tensile stress in the extreme fiber of the bottom flange was limited to a maximum value of 610 psi (4.21 MPa). When the tensile stress was 750 psi (5.17 MPa) or larger, fatigue fractures of the prestressing strand occurred and the fatigue life of the girder was reduced.

Measured shear strengths consistently exceeded the strengths calculated according to the *AASHTO Standard Specifications for Highway Bridges* and the *AASHTO LRFD Bridge Design Specifications* using both design and measured material properties. The existing limitation of 60,000 psi (414 MPa) for the design yield stress of transverse reinforcement in both AASHTO specifications is conservative. Higher reinforcement yield strengths can be utilized in the design of prestressed concrete beams. Welded wire deformed reinforcement can be used as an equally effective alternate to deformed bars as shear reinforcement.

17. Key Words Fatigue, High Performance Concrete, High Strength Concrete, Bridge Girders, Prestressed Concrete, Shear Strength, Welded Wire Reinforcement.	18. Distribution Statement Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
N/A	**N/A**	**65**	

FATIGUE AND SHEAR BEHAVIOR OF HPC BULB-TEE GIRDERS

INTERIM REPORT

by

Robert N. Bruce, Jr.
Catherine and Henry Boh Chair in Civil Engineering, Tulane University
New Orleans, LA 70118

Henry G. Russell
Engineering Consultant, Henry G. Russell, Inc.
Glenview, IL 60025

John J. Roller
Principal Structural Engineer, Construction Technology Laboratories, Inc.
Skokie, IL 60077

LTRC Project No. 00-6SS
State Project No. 736-99-0821

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

October 2003

ABSTRACT

Three 96-ft (29.3-m) long, 72-in. (1.83-m) deep, precast, pretensioned bulb-tee girders were tested to evaluate behavior under flexural fatigue and 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 quantities at the ends of each girder were selected 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.

The three prestressed concrete girders were produced in a commercial plant. Prior to testing, a 10-ft (3.05-m) wide reinforced concrete deck slab was added to each girder. After completion of fatigue testing, each girder was cut in half and the six girder ends tested to evaluate static shear strength.

The bulb-tee girder performed satisfactorily under 5,000,000 cycles of flexural fatigue loading when the tensile stress in the extreme fiber of the bottom flange was limited to a maximum value of 610 psi (4.21 MPa). When the tensile stress was 750 psi (5.17 MPa) or larger, fatigue fractures of the prestressing strand occurred and the fatigue life of the girder was reduced.

Measured shear strengths consistently exceeded the strengths calculated according to the *AASHTO Standard Specifications for Highway Bridges* and the *AASHTO LRFD Bridge Design Specifications* using both design and measured material properties. The existing limitation of 60,000 psi (414 MPa) for the design yield stress of transverse reinforcement in both AASHTO specifications is conservative. Higher reinforcement yield strengths can be utilized in the design of prestressed concrete beams. Welded wire deformed reinforcement can be used as an equally effective alternate to deformed bars as shear reinforcement.

ACKNOWLEDGMENTS

Work on this project was performed jointly by Tulane University Department of Civil and Environmental Engineering, Henry G. Russell, Inc., and Construction Technology Laboratories, Inc. under the sponsorship of the Louisiana Transportation Research Center and in cooperation with the Louisiana Department of Transportation and Development. Paul B. Fossier, Bridge Engineer Administrator of the Louisiana Department of Transportation and Development performed the prototype bridge designs described in this report and provided technical guidance throughout the project. Walid Alaywan, Senior Structures Research Engineer of the Louisiana Transportation Research Center provided administrative management for the project. John Eggers and Randy Young were responsible for the concrete materials test program.

Appreciation is expressed to the staff of Gulf Coast Pre-Stress, Inc., of Pass Christian, MS, especially Don Theobald and Larry Nicely for their patience and perseverance during the fabrication, storage, and shipping of the girders.

The work of Katherine K. Raymond, Chris Tanner, and Cullen Ledet (graduate students at Tulane University) is also acknowledged.

IMPLEMENTATION STATEMENT

The results of the investigation described in this report will be utilized and implemented in the design of the proposed Rigolets Pass Bridge on Highway U.S. 90 east of New Orleans (SP No. 006-05-0076). The design will use 130-ft (40-m) long, 72-in. (1.83-m) deep, high performance concrete bulb-tee girders. Construction of the bridge is anticipated to begin in 2004.

Two other projects are scheduled to utilize high performance concrete girders in their design and construction. The Union Pacific Railroad Overpass on Highway U.S. 165 in Jefferson Davis Parish (SP No. 014-02-0018) will use AASHTO Type IV girders with a maximum span of 115 ft (35 m). The LA 27 Overpass in Calcasieu Parish (SP No. 450-91-0087) will use AASHTO Type IV girders with a maximum span of 112 ft (34 m).

TABLE OF CONTENTS

ABSTRACT ... iii
ACKNOWLEDGMENTS ... v
IMPLEMENTATION STATEMENT ... vii
TABLE OF CONTENTS ... ix
LIST OF TABLES ... xi
LIST OF FIGURES ... xiii
INTRODUCTION ... 1
OBJECTIVES ... 3
SCOPE ... 5
METHODOLOGY ... 7
- Prototype bridge design ... 7
  - Bridge design loads ... 7
  - Section properties ... 8
  - Allowable stresses and stress limits ... 9
  - Comments on the designs ... 9
- Test specimens ... 10
  - Flexural design ... 10
  - Shear design ... 11
  - Deck slab ... 14
  - Cross section ... 14
- Girder fabrication ... 14
- Girder instrumentation ... 16
  - Strand load measurement ... 16
  - Internal strain gages ... 16
  - Girder camber measurements ... 16
  - Steel stirrup and nonprestressed reinforcement strains ... 17
- Deck slab construction ... 17
- Fatigue test setup and procedure ... 18
- Shear test setup and procedure ... 21
- Material property tests ... 22

DISCUSSION OF RESULTS ... 27
- Material property tests ... 27
  - Concrete ... 27
  - Reinforcement ... 29

Fatigue tests 29
Determination of applied loads 30
Specimen BT6 33
Specimen BT7 35
Specimen BT8 35
Summary of fatigue test results 37
Shear tests 37
BT6-Live end 40
BT6-Dead end 41
BT7-Live end 41
BT7-Dead end 41
BT8-Live end 42
BT8-Dead end 42
Comparison of results 43
Comparison of measured strengths with the AASHTO specifications 44
Conclusions from the shear tests 49
CONCLUSIONS...51
RECOMMENDATIONS 53
ACRONYMS, ABBREVIATIONS, AND SYMBOLS 55
REFERENCES 57

LIST OF TABLES

Table 1 Design dead loads 7
Table 2 Bridge section properties 8
Table 3 Allowable concrete stresses and stress limits 9
Table 4 Specimen details 13
Table 5 Mix proportions for girder concrete..........16
Table 6 Mix proportions for deck concrete..........18
Table 7 Levels of maximum tensile stress for fatigue tests 20
Table 8 Girder concrete material property testing program 23
Table 9 Deck slab concrete material property testing program..........24
Table 10 Measured girder concrete material properties..........28
Table 11 Measured deck concrete material properties..........29
Table 12 Measured properties of 0.6-in. (15.2-mm) diameter prestressing strand 30
Table 13 Measured properties of nonprestressed reinforcement 30
Table 14 Loads and stresses for fatigue tests..........32
Table 15 Fatigue test results..........37
Table 16 Summary of shear test results 39
Table 17 Material properties used in analyses of shear test results 45
Table 18 Comparison of measured strengths calculated using the *AASHTO Standard Specifications* 46
Table 19 Comparison of measured strengths calculated using the *AASHTO LRFD Specifications* 47

LIST OF FIGURES

Figure 1 Test specimen cross section....................15
Figure 2 Girder fabrication....................15
Figure 3 Location of load cells....................17
Figure 4 Fatigue test configuration....................19
Figure 5 Shear test configuration....................21
Figure 6 Fatigue test setup....................33
Figure 7 Support reaction versus concrete strain for fatigue test of Specimen BT6....................34
Figure 8 Support reaction versus midspan deflection for fatigue test of Specimen BT6....................34
Figure 9 Support reaction versus midspan deflection for fatigue test of Specimen BT7....................36
Figure 10 Fatigue fractures....................36
Figure 11 Support reaction versus concrete strain for fatigue test of Specimen BT8....................38
Figure 12 Support reaction versus midspan deflection for fatigue test of Specimen BT8....................38
Figure 13 Shear test setup....................39
Figure 14 Stirrup strains in Specimen BT8-Live....................40
Figure 15 Diagonal crack pattern in Specimen BT8-Live....................41
Figure 16 Specimen BT8-Dead after the shear test....................42
Figure 17 Comparison of applied shear forces....................43
Figure 18 Comparison of measured and calculated strengths using the *AASHTO Standard Specifications*....................48
Figure 19 Comparison of measured and calculated strengths using the *AASHTO LRFD Specifications*....................49

INTRODUCTION

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 that the bridge should be instrumented to measure long-term behavior *[1]*. This recommendation 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, the 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 this 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 static shear loading conditions.

OBJECTIVES

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.

SCOPE

The following scope of activities was undertaken to accomplish the objectives of the project:

- Design two prototype bridge superstructures with 72-in. (1.83-m) deep prestressed concrete bulb-tee girders utilizing a design concrete compressive strength of 10,000 psi (69 MPa).
- Design three full-scale test specimens based on the prototype bridges.
- Instrument, fabricate, and ship three test girders.
- Cast a high performance concrete deck slab on each girder to form a test specimen.
- Test each specimen in flexural fatigue.
- Test each end of each specimen under static shear loads.
- Analyze the test results.
- Prepare a report.

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 4,200 psi (29 MPa). Both designs utilized 0.6-in. (15.2-mm) diameter, low-relaxation Grade 270 prestressing strands conforming to ASTM Designation: A 416 in the girders *[7]*.

Bridge design loads. Dead loads used in the design of each bridge are listed in table 1. Unit weight of the concrete for both the girder and deck was taken as 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 distributed equally to all girders and supported entirely by the non-composite bridge girders.

Table 1
Design dead loads

Girder Weight	Girder Haunch	Midspan Diaphragm	Deck Slab, 8-in. thick
799 lb/ft	131 lb/ft	5.2 kips	1,350 lb/ft

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 2. 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). In computing section properties for both designs, a 2-1/2-in. (64-mm) deep haunch is included. 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 2
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.

Allowable stresses and stress limits. Allowable stresses per the *Standard Specifications* and stress limits per the *LRFD Specifications* used in the prototype bridge designs are listed in table 3. 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 $6\sqrt{f'_c}$. This value was used in the design with the *Standard Specifications*. However, it was decided that a value of $7.5\sqrt{f'_c}$ would be used in the LRFD design to take advantage of the higher tensile strength of the high-strength concrete.

Table 3
Allowable concrete stresses and stress limits

Loading Condition[a]	Concrete Stress	Standard Specifications (psi)	LRFD Specifications (psi)
Release	Compression-girder	4,200	4,200
	Tension-girder	200	200
	Tension-girder w/bonded reinf.	627	586
P + DL + LL	Compression-girder	6,000	6,000
	Compression-deck	2,520	2,520
	Tension-girder top	300	194
	Tension-girder bottom	600	—
P + DL	Compression-girder	4,000	4,500
	Compression-deck	1,680	1,890
0.5 (P + DL) + LL	Compression-girder	4,000	4,000
	Compression-deck	1,680	1,680
P + DL + 0.80LL	Tension-girder	—	750
	Tension-deck	—	490

A dash indicates that the stress is not applicable.

a P = Prestressing force. DL = Dead load. LL = Live load including impact.

Comments on the designs. For flexure, both designs resulted in girders requiring 24 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 from the support equal to one half the overall depth of the composite section. Therefore, the critical section was 3.44 ft (1.05 m) from the support. 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*.

Flexural design. Both superstructure designs (Standard and LRFD) prepared by the LADOTD required twenty-four 0.6-in. (15.2-mm) diameter prestressing strands for a typical interior girder. Therefore, the three 96-ft (29.3-m) long, 72-in. (1.83-m) deep bulb-tee girder test specimens fabricated for this research also incorporated 24 prestressing strands, each initially stressed to 75 percent of the specified ultimate strength. Debonding of the strands was also the same as calculated for the prototype bridge. Specified compressive strengths for the girder concrete were 7,000 psi (48 MPa) at release of the strands and 10,000 psi (69 MPa) at an age no later than 56 days.

Shear design. Details of the shear reinforcement in the test specimens were different from those in the prototype bridge girders 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, each girder half 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 near the ends 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 a truck load in the design using the *Standard Specifications* or a combination of uniformly distributed lane load and truck or tandem load in the design using the *LRFD Specifications*. 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. The *LRFD Specifications'* commentary to the section dealing with longitudinal reinforcement 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 development 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 end region of each girder was reinforced with pairs of No. 5 bars or D31 wires at 4-in. centers for a length of 2 ft-8 in. (16-mm diameter bars at 203-mm centers for 813 mm). This is a standard LADOTD detail.

The test region extended from the end of the first region to the concentrated load points. This is the region in which the shear failure was expected to occur during testing. The shear reinforcement in the test region was the same as that required at the critical section in the corresponding prototype bridge. The quantity of reinforcement was maintained constant throughout the test region.

The midspan region incorporated the length from the concentrated load points to midspan. The shear reinforcement provided in the midspan region was selected to prevent shear failure in this region during the shear test.

Details of the shear reinforcement in each region are shown in table 4. The shear reinforcement in the live end test region of Girder BT6 was the same as 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 equivalent quantity 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.

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

Table 4
Specimen details

Specimen	Design Specif-ication	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 reinforce-ment at 4 in.	D20 welded wire reinforce-ment at 12 in.	D20 welded wire reinforce-ment 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 reinforce-ment at 4 in.	D20 welded wire reinforce-ment at 8 in.	D20 welded wire reinforce-ment at 16 in.
			Dead	D31 welded wire reinforce-ment at 4 in.	D20 welded wire reinforce-ment at 18 in.	D20 welded wire reinforce-ment at 16 in.

development length located 8.52 ft (2.60 m) from the end of the girder. The development length of 8.52 ft (2.60 m) did not include the 1.6 multiplier. 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. These quantities of reinforcement were used in the test regions.

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. These quantities of reinforcement were used in the test regions.

The *LRFD Specifications* also require a check of the internal longitudinal force at the end of the girder. This check is required to ensure that there is adequate reinforcement to resist the horizontal component of force along the diagonal compression strut caused by the shear force. As a result, horizontal nonprestressed reinforcement, consisting of 8 No. 6 (19-mm diameter) bars, was required 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).

Deck slab. Prior to testing each specimen, a 10-ft (3.05-m) wide, 8-in. (203-mm) thick deck slab was cast on each girder using unshored construction. A 5/8-in. (16-mm) thick haunch was provided along the entire length of each girder. The width of the deck slab was selected to represent the calculated design effective widths of the compressive flange of 117 in. (2.97 m) and 138 in. (3.51 m) for the *AASHTO LRFD* and *Standard Specifications*, respectively. In addition, the 10-ft (3.05-m) width represented the maximum practical deck width that could be cast on the girder without using partial shoring. The specified compressive strength of the deck slab concrete was 4,200 psi (30 MPa) at 28 days. In addition, the concrete for the deck slab of each girder incorporated a specific mineral admixture (ground granulated blast-furnace slag, silica fume, or fly ash). A summary of details associated with each of the three test specimens is shown in table 4.

Cross section. The cross-section of a test specimen with a deck slab is shown in figure 1. The top layer of longitudinal deck slab reinforcement, consisting of No. 4 bars at 12-in. (13-mm diameter bars at 305-mm) spacing, was representative of the quantity indicated in the LADOTD drawings resulting from the two bridge designs. The top layer of transverse reinforcement provided in the deck slab was designed to support the cantilevered portion of the deck, and was not intended to be representative of the quantities required per the bridge designs. The bottom layers of main reinforcing steel (perpendicular to girder) and distribution steel (parallel to girder) were intentionally omitted since these layers did not have a significant effect on the structural performance of the girder during fatigue or shear testing.

Girder fabrication

The three 72-in. (1.83-m) deep bulb-tee girders required for this research were fabricated by Gulf Coast Pre-Stress, Inc. (GCP) in Pass Christian, Mississippi. The concrete mix proportions used to fabricate the three girder test specimens were essentially the same as the mix proportions used in the girders for the Charenton Canal Bridge project *[2]*. These are given in table 5. All three test girders were fabricated at the same time, on a single casting bed, as shown in figure 2.

During casting, GCP produced match-cured cylinders for determination of concrete strength development. In addition, the research team also prepared concrete cylinders and beam

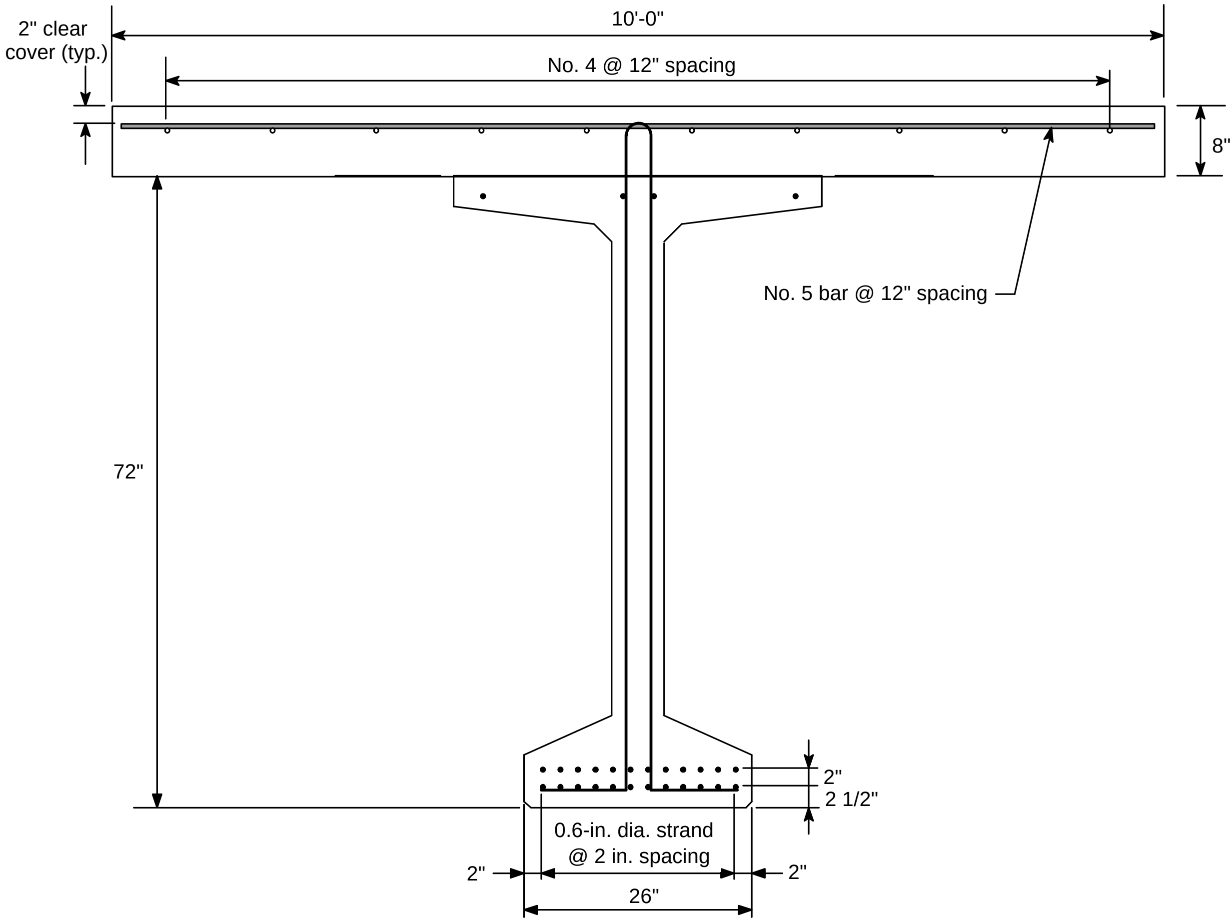


Figure 1
Test specimen cross section

Figure 2
Girder fabrication

Table 5
Mix proportions for girder concrete

Material	Quantities	
	per yd^3	per m^3
Portland Cement – Type III	691 lb	410 kg
Fly Ash – Class C	296 lb	176 kg
Fine Aggregate	1,135 lb	673 kg
Course Aggregate – Limestone	1,803 lb	1,070 kg
Water	247 lb	147 kg
Water Reducer, ASTM C 494 – Type D	80 oz	3,094 ml
High-Range Water Reducer, ASTM C 494 – Type F	160 oz	6,189 ml
Air Entrainment	None	None
Water-Cementitious Materials Ratio	0.25	0.25

specimens for independent evaluation of girder concrete material properties. After fabrication, the girders were stored at GCP until shipped individually to Construction Technology Laboratories, Inc. (CTL) for testing.

Girder instrumentation

Strand load measurement. During girder fabrication, six of the 24 prestressing strands were instrumented with load cells at the dead end (anchorage end) of the stressing bed to measure strand loads. The specific strands that were instrumented with load cells are indicated in figure 3. Loads in the selected strands were measured before stressing (zero reading), after all strands had been stressed, at selected intervals during girder fabrication, just prior to release, and after release (return to zero load).

Internal strain gages. After pretensioning the strands and prior to casting the concrete, three vibrating wire concrete strain gages and four “sister bars” instrumented with electrical resistance strain gages were installed in each girder specimen at midspan. The three vibrating wire concrete strain gages were installed in the lower flange, at the elevation of the strand group centroid. Two sister bar gages were installed in the lower flange at the level of the bottom strand row. Two sister bar gages were also installed in the top flange of each girder.

Girder camber measurements. Immediately after casting and while the concrete was still plastic, large steel bolts were embedded in the top surface of each girder at midspan and near both ends to provide a permanent fixed point for girder camber measurements. The

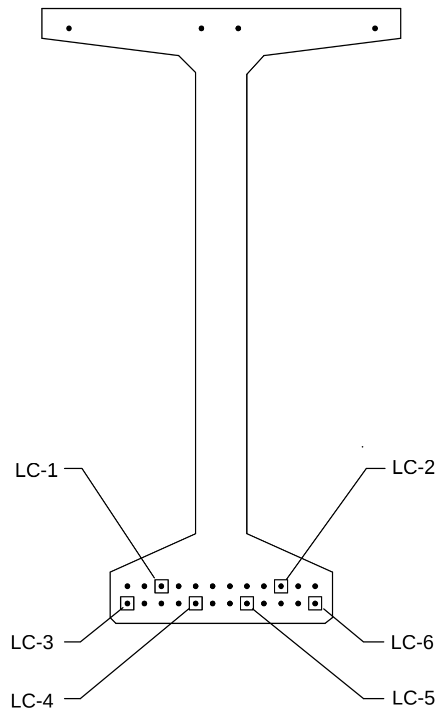


Figure 3
Location of load cells

embedded bolt near each end was centered 6 in. (150 mm) from the end of the girder. Camber measurements were made using a level to sight elevations at each point. Midspan camber measurements relative to the ends of each girder were measured at various girder ages including prior to release; after release; at a concrete age of 28 days; before and after shipping; after being placed on supports for testing; before and after deck casting; prior to the fatigue test; and after each 1 million cycles of fatigue loading.

Steel stirrup and nonprestressed reinforcement strains. Prior to casting each girder, weldable electrical resistance strain gages were installed on selected stirrups (bars or wires) located within a distance of 10 ft (3.05 m) from the ends of each girder. Each gage was installed at approximately midheight of the girder cross section. In addition, two weldable strain gages were installed on the outer two nonprestressed longitudinal reinforcing bars at a distance of 20 in. (510 mm) from each end of Girders BT7 and BT8.

Deck slab construction

Upon arrival at CTL, each girder was placed on supports, creating a span of 95 ft (28.96 m). In preparation for testing, load cells were installed beneath each end support for the purpose of measuring reaction forces. An 8-in. (203-mm) thick, 10-ft (3.05-m) wide reinforced concrete deck was cast on each girder using unshored construction methods. Support reaction forces were measured before and after casting the deck slab to obtain an accurate account of the total dead load carried by the girder section. Deck slab width, thickness, and reinforcement details were the same for all three girders. However, as indicated in table 4, the deck slab concrete for each of the three girders incorporated a different combination of cementitious materials. The concrete mix proportions used for the deck concretes are shown in table 6.

Table 6
Mix proportions for deck concrete

Material	BT6	BT7	BT8
Portland Cement	306 lb	491 lb	414 lb
Ground Granulated Blast-Furnace Slag	306 lb	—	—
Silica Fume	—	26 lb	—
Fly Ash – Class F	—	—	103 lb
Fine Aggregate	1,176 lb	1,315 lb	1,250 lb
Course Aggregate	1,900 lb	1,845 lb	1,875 lb
Water	238 lb	209 lb	207 lb
Water Reducer, ASTM C 494	—	21 fl oz	31 fl oz
High-Range Water Reducer, ASTM C 494	43 fl oz	41 fl oz	62 fl oz
Air Entrainment	5 $\pm$ 1%	5 $\pm$ 1%	5 $\pm$ 1%
Water-Cementitious Materials Ratio	0.39	0.40	0.40

Concrete for each deck slab was placed and finished using standard practices and procedures. After casting, the concrete was wet cured under burlap for a minimum of seven days. Once the concrete achieved a compressive strength of 3,200 psi (22.1 MPa), but no earlier than seven days, all formwork was removed, and test setup preparations started.

Fatigue test setup and procedure

The configuration for the fatigue tests is shown in figure 4. Specimens were simply supported at the centerline of the sole plates, creating a total span length of 95 ft (28.96 m). Load was applied to the specimens using a pair of hydraulic actuators spaced 15 ft (4.57 m) apart and centered about midspan. Supplementary dead load was added to reduce the loads required by the actuators. Load cells were used to monitor the applied load from the

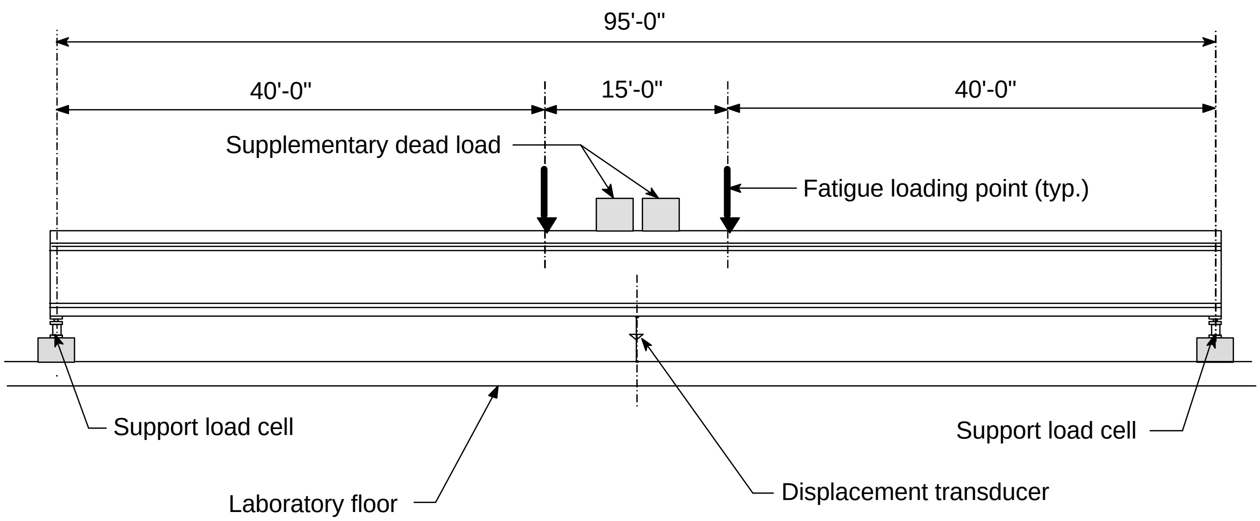


Figure 4
Fatigue test configuration

actuators and the support reactions at both ends of the specimen. Two potentiometers were used to monitor specimen displacement at midspan.

The specimens were tested using a closed-loop, load-controlled servo-hydraulic system. For dynamic loading, the test equipment was programmed to maintain a forced loading function at a frequency of approximately 1.9 hertz. The upper- and lower-bound loads applied by the actuators were determined from the support reactions while loading at 1.9 hertz. Because of the dynamic inertia of the specimen, the maximum actuator load was less than the maximum support reaction and the minimum actuator load was larger than the minimum support reaction.

Prior to the start of each fatigue test, the specimen was statically loaded until a flexural crack developed in the constant moment region. After cracking the girder, the applied load was decreased to zero and strain gages were installed immediately adjacent to the crack on the bottom concrete surface of the lower flange. After the gages were installed, the specimen was statically loaded again to determine: 1) the effective prestress based on measured concrete strain data, 2) the decompression load, and 3) the initial specimen deflection at service load levels prior to fatigue loading.

For each fatigue test, the upper-bound load was selected to correspond with different levels of maximum tensile stresses that would be produced in an uncracked section. The level of maximum tensile stress used for each of the three specimens is given in table 7.

Table 7
Levels of maximum tensile stress for fatigue tests

Girder	Tensile Stress	f_c' (psi)	Tensile Stress (psi)
BT6	$6.0\sqrt{f_c'}$	Specified Strength = 10,000	610[a]
BT7	$7.5\sqrt{f_c'}$	Measured Strength = 13,050	857
BT8	$7.5\sqrt{f_c'}$	Specified Strength = 10,000	750

a Stress was 10 psi higher than intended.

The fatigue load range was selected to produce a moment range equal to that resulting from the design live load plus impact in the prototype bridge design. For Girder BT6, the moment range corresponded to the full live load plus impact per the *AASHTO Standard Specifications*. For Girders BT7 and BT8, the moment range corresponded with Service Load III of the *AASHTO LRFD Specifications,* which uses 0.8 (live load plus impact). Service Load III is the load combination that governs tension in the bottom of the girders at midspan in design using the *LRFD Specifications [4]*. The lower bound load for each test was established based on the difference between the upper bound load and the load range.

Each fatigue test consisted of up to five individual parts unless it was necessary to terminate the test earlier. Each part included the application of 1 million loading cycles followed by a static load test to measure specimen response at the full design service load condition. The bottom flange decompression load was also verified or re-established during each static test and adjustments to the target upper- and lower-bound loads were made as necessary for the next dynamic loading part. Prior to each static test, midspan camber and prestress losses were also measured.

During the static load tests, output from all instrumentation was monitored continuously using a digital data acquisition system (DDAS) and computer. At selected intervals (load stages), data were stored on disk to provide a permanent record of test specimen behavior. During fatigue loading, data from selected instruments was read at least twice per day using a high-speed DDAS and a computer. Applied loads, support reactions, and midspan deflections for two full loading cycles were included in each daily data collection interval.

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 located 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 5. For BT8-Dead, the span length was reduced from 46 ft 8 in. to 43 ft 0 in. (14.23 m to 13.10 m) because of damage at midspan during the fatigue test.

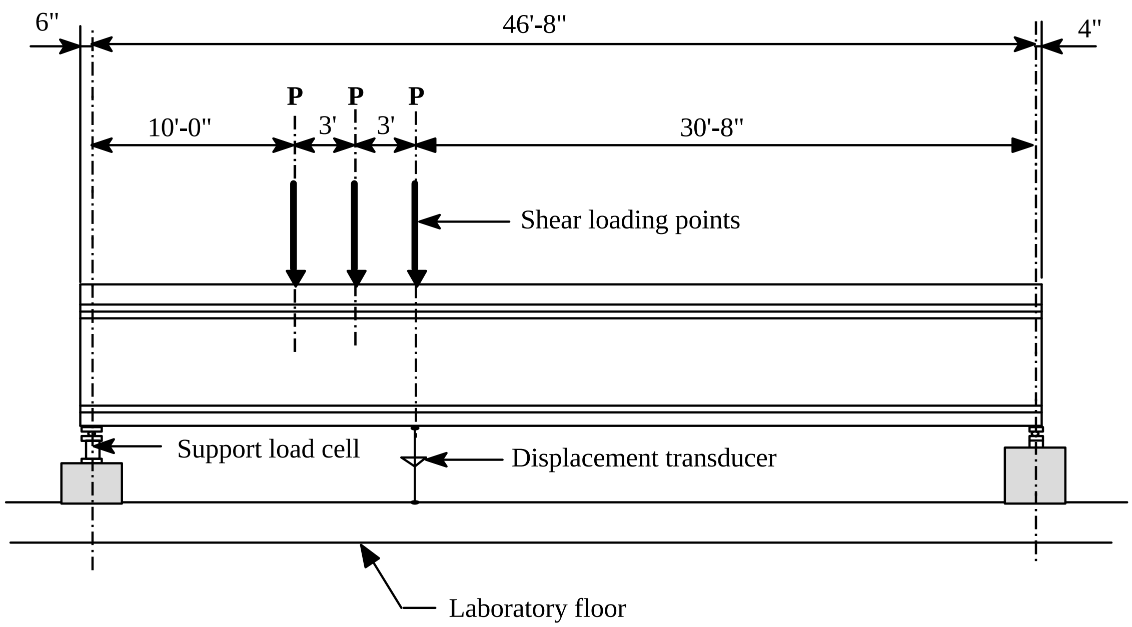


Figure 5
Shear test configuration

Load was applied to each specimen using three concentrated load points. The first loading point was located 10 ft (3.05 m) from the centerline of the support at the as-cast end of the girder. Two additional loading points were provided at 3-ft (914-mm) intervals. During the shear tests, 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.

Material property tests

Concrete compressive strength (ASTM C 39), modulus of elasticity (ASTM C 469), unit weight, modulus of rupture (ASTM C 78), and coefficient of thermal expansion (CRD C 39) were determined at various ages for concrete incorporated in each of the girders. In general, the ages corresponded to release of the strands, 56 days, and start of the fatigue and shear tests. Coefficient of thermal expansion was determined at a concrete age of 84 days. In addition, petrographic examination of core samples extracted from the midspan region of each girder after completion of fatigue testing was conducted. A complete matrix of the planned tests for the girder concrete is given in table 8.

The concrete materials testing program for the girders was a cooperative effort by the researchers and LTRC. Concrete specimens designated as "match" in the Curing Method column of table 8 were match cured until the time the girder prestress force was released. Specimens designated as "field" in the same column of table 8 were initially cured on the side of the steel forms under the tarpaulin that covered the girders. After release, all specimens were cured under similar conditions as the girders for as long as possible prior to testing.

Several 6x12-in. (152x305-mm) concrete cylinders were made from concrete representing the middle and ends of each deck slab. Concrete cylinders were tested to determine compressive strength (ASTM C 39) at concrete ages of 7 days, 28 days, and the age corresponding to the start of the girder fatigue or shear tests. Concrete modulus of elasticity (ASTM C 469) was determined for the midspan region of each deck slab prior to the start of the fatigue tests and for the end regions prior to the start of the shear tests. In addition, petrographic examination of one core sample extracted from the midspan region of each deck slab before and after the fatigue testing was conducted. A complete test matrix for the deck slab concrete is given in table 9.

Table 8
Girder concrete material property testing program

Concrete Age	Material Property	Curing Method	Testing Responsibility	BT6			BT7			BT8		
				Live End	Midspan	Dead End	Live End	Midspan	Dead End	Live End	Midspan	Dead End
Release	f'_c/E_c	Match	LTRC	—	2	—	—	2	—	—	2	—
56 Days	f'_c/E_c	Match	LTRC	—	3	—	—	3	—	—	3	—
84 days	"	Field	CTL	—	3	—	—	3	—	—	3	—
Fatigue Test	f'_c/E_c	Match	LTRC	—	3	—	—	3	—	—	3	—
Shear Test	f'_c/E_c	Match	LTRC	4	—	—	4	—	—	4	—	—
Shear Test	f'_c	Field	LTRC	3	—	3	3	—	3	3	—	3
Fatigue Test	MOR	Field	LTRC	—	3	—	—	3	—	—	3	—
Fatigue Test	Petro[a]	Core	CTL	—	1	—	—	1	—	—	1	—

A dash indicates that the property was not measured.

a Concrete core extracted from midspan region of each girder.

Summary:

36 match-cured 4x8-in. cylinders.

27 field-cured 6x12-in. cylinders.

9 field-cured 6x6x20-in. MOR beams.

3 cores for petrographic examination.

Table 9
Deck slab concrete material property testing program

Concrete Age	Material Property	Curing Method	Testing Responsibility	BT6			BT7			BT8		
				Live End	Midspan	Dead End	Live End	Midspan	Dead End	Live End	Midspan	Dead End
7 Days	f'_c	Laboratory	CTL	—	1	—	—	1	—	—	1	—
28 Days	f'_c	Laboratory	CTL	—	3	—	—	3	—	—	3	—
Fatigue Test	f'_c/E_c	Laboratory	CTL	—	3	—	—	3	—	—	3	—
Shear Test	f'_c/E_c	Laboratory	CTL	3	—	3	3	—	3	3	—	3
Fatigue Test	Petro[a]	Core	CTL	—	2	—	—	2	—	—	2	—

A dash indicates that the property was not measured.

a Concrete core extracted from top surface of each deck slab before and after completion of fatigue test.

Summary:

39 laboratory-cured 6x12-in. cylinders.

6 cores for petrographic examination.

Samples of prestressing strand were obtained from the same coils of strand used in the girders. Each strand sample was tested to determine 1 percent elongation load, breaking load, total elongation, and modulus of elasticity.

Samples of deformed bars and welded wire used for vertical stirrups and longitudinal reinforcement that were instrumented with strain gages were obtained and tested to determine actual properties. Three samples of each size and type (bar or wire) were tested in tension to determine yield strength, ultimate strength, total elongation, and modulus of elasticity.

DISCUSSION OF RESULTS

Material property tests

Concrete. Measurements of compressive strength, modulus of elasticity, unit weight, modulus of rupture, and coefficient of thermal expansion were made on test specimens representing concrete used in the midspan and end regions of each girder. Measured values of compressive strength, modulus of elasticity, and unit weight for various tests ages are given in table 10. Test ages other than 1 and 56 days correspond to the start of different tests performed on the girders. Due to technical difficulties with one set of match-cure molds and with testing machines, it was not possible to exactly adhere to the testing program shown in table 8. To overcome these difficulties, concrete cores were cut from the webs of the girders for determination of compressive strength, modulus of elasticity, and unit weight. The measured properties from the cores are included in table 10. Compressive strength was always measured on the same cylinders and cores used for the modulus of elasticity tests. The specimens were unloaded after conducting the modulus of elasticity tests to remove the compressometer before being tested for compressive strength.

The measured values of modulus of rupture for the girder concrete at the start of the fatigue tests were 1,065 psi (7.34 MPa) at 131 days, 1,045 psi (7.21 MPa) at 252 days, and 1,080 psi (7.45 MPa) at 396 days for Girders BT6, BT7, and BT8, respectively. The average measured coefficient of thermal expansion for the girder concrete was 5.54 millionths/^{o}F (9.97 millionths/^{o}C) at 84 days.

Measurements of compressive strength and modulus of elasticity were made on test specimens representing concrete used at the midspan and end regions of the deck slab of each test specimen. Measured properties are shown in table 11.

A petrographic examination of one core from the webs of each of the three girders was made at a concrete age of approximately 550 days. The examination revealed that the concrete was nonair-entrained and contained crushed carbonate rock coarse aggregate and siliceous sand fine aggregate uniformly dispersed in a cementitious paste matrix of portland cement and fly ash. The observed paste properties were judged to be of good quality and the distribution of concrete constituents relatively uniform within each core. All three cores contained a well developed, continuous network of microcracks. The microcracks in the core from BT6 appeared to be slightly more abundant and densely spaced in localized areas of cement paste compared to the other two core samples. The petrographic examination could not distinguish between microcracking caused by the fatigue testing and microcracking caused by restrained

Table 10
Measured girder concrete material properties[a]

Test Specimen and Location	Test Age (days)	Curing[b]	Specimen Size (in.)	Comp. Strength (psi)	Mod. of Elasticity (ksi)	Unit Weight (lb/ft^3)
BT6 Midspan	1	Match	4x8	8,640	3,950	—
	56	Match	4x8	10,340	5,600	150.3
	124	Match	4x8	9,290	5,500	151.0
BT6 Live End	193	Match	4x8	10,390	5,750	149.6
	193	Field	6x12	10,050	—	—
	203	Core	3x6	11,780	6,000	150.2
BT6 Dead End	193	Field	6x12	11,410	—	—
	203	Core	3x6	11,590	5,750	150.3
BT7 Midspan	1	Match	4x8	9,120	4,250	—
	56	Match	4x8	11,560	5,950	150.2
	263	Core	3x6	13,050	6,250	151.2
	403	Match	4x8	11,300	5,500	150.0
BT7 Live End	329	Core	3x6	12,400	5,950	150.2
	403	Match	4x8	11,630	4,975	151.0
	403	Field	6x12	11,120	—	—
BT7 Dead End	329	Core	3x6	12,730	5,550	150.2
	403	Field	6x12	11,010	—	—
BT8 Midspan	1	Match	4x8	8,840	3,800	—
	56	Match	4x8	10,400	5,600	149.9
	396	Core	3x6	11,850	5,950	151.0
BT8 Live End	462	Core	3x6	11,850	5,450	149.9
BT8 Dead End	462	Core	3x6	11,310	5,500	151.0

A dash indicates that the property was not measured.

a Most values are the average of three specimens.

b Match = match cured until release of strands and then stored in a similar environment as the girders.

Field = cured alongside the girder until covers were removed and then stored in a similar environment as the girders.

Core = core taken horizontally through web of the girder.

drying shrinkage. No evidence of deleterious reactions was observed in any of the examined concrete.

Concrete cores taken from each deck slab before and after each fatigue test were also examined petrographically to assess whether or not the fatigue loading had caused any

Table 11
Measured deck concrete material properties[a]

Test Specimen	Location	Test Age (days)	Comp. Strength (psi)	Modulus of Elasticity (ksi)
BT6	Midspan	7	3,740	—
		28	5,670	—
		35	5,950	4,650
	Live	105	5,780	4,450
	Dead	105	4,860	4,050
BT7	Midspan	7	5,740	—
		28	5,670	—
		32	6,630	5,350
	Live	78	7,330	4,850
	Dead	78	7,950	5,250
BT8	Midspan	7	4,840	—
		35	6,560	—
		35	6,570	4,750
	Live	95	7,340	4,700
	Dead	95	6,850	4,650

A dash indicates that the property was not measured.

a Most values are the average of three specimens.

apparent distress such as microcracking in the top surface of the deck slabs. Results of comparative microscopical examination of the top surface and longitudinal profile of the cores revealed no general differences in the extent of microcracking attributed to fatigue testing of BT6 and BT7. The core taken from BT8 after fatigue testing exhibited somewhat more microcracking than the pre-test core, but the findings were inconclusive as to whether cyclic fatigue loading was the primary or contributory cause of the increased cracking.

Reinforcement. Measured properties of the 0.6-in. (15.2-mm) diameter prestressing strand and the nonprestressed reinforcement are shown in tables 12 and 13, respectively.

Fatigue tests

Each fatigue test consisted of applying a maximum of 5 million cycles of loading or less if a fatigue fracture of the prestressing strands was detected. The upper bound test load was selected to correspond with different levels of tensile stress that would be produced in an uncracked section. The fatigue load range was selected to produce a moment range equal to that resulting from the design live load plus impact in the prototype bridge design.

Table 12
Measured properties of 0.6-in. (15.2-mm) diameter prestressing strand[a]

1% Elongation Load (lb)	Breaking Load (lb)	Total Elongation (%)	Modulus of Elasticity (ksi)
54,167	61,550	6.2	28,720

a All properties are based on the average of five or six samples.

Table 13
Measured properties of nonprestressed reinforcement[a]

Bar Size	Yield[b] Strength (psi)	Ultimate Strength (psi)	Elongation	Modulus of Elasticity (ksi)
No. 4	62,500	103,000	12.0	25,700
No. 5	62,000	103,000	12.4	25,350
No. 6	65,500	106,500	12.7	27,700
D20	85,000	99,500	8.2	27,500
D31	83,500	97,500	7.8	29,400

a All properties are based on the average of three samples except two samples were used for the No. 5 bar. All values are calculated using the nominal cross-sectional area.

b For Nos. 4, 5, and 6 bars, values were determined using the halt of the pointer method. For D20 and D31 bars, values are based on a strain of 0.005.

Determination of applied loads. All calculations to determine the loads to be applied during the fatigue tests were based on measured values of modulus of elasticity, measured prestress losses, measured test specimen dimensions, an uncracked section, static equilibrium, and calculated stresses at midspan. Determination of the loads involved the following steps:

1. The force in the strands prior to their release was calculated from the average force in the load cells shortly after casting the girders.

2. The elastic shortening at release, creep and shrinkage losses between release of the strands and deck casting, and creep and shrinkage losses occurring after deck casting were calculated from the average change in strain measured with the vibrating wire strain gages.

3. The prestress losses were subtracted from the initial strand force to obtain the force in the strands at the start of the fatigue test.

4. The strand force was used to calculate the concrete stresses in the girder caused by the prestressing force.

5. Concrete stresses caused by the self weight of the girder were calculated based on the weight of the girder measured by the support load cells.

6. Concrete stresses caused by the weight of the deck and haunch acting on the girder cross section were determined using the weights measured by the support load cells.

7. The concrete stresses caused by the supplementary dead load acting on the composite girder and deck cross section were calculated using the measured weight of the supplementary dead load.

8. The concrete stresses from the various loads were combined to determine the net stress in the extreme bottom fiber of the girder.

9. The additional loads from the actuators to be applied to the composite section to produce zero tensile stress and the target tensile stress in the extreme bottom fiber were calculated. This provided the anticipated load required to overcome precompression of the bottom flange and the maximum load to be applied during the fatigue test.

10. The load range was determined so that the moment range applied by the two actuators was equal to the live load plus impact moment determined from the prototype bridge design.

11. The minimum load was determined by subtracting the load range from the maximum load.

Loads and stresses determined using the above procedures are given in table 14.

Prior to start of the fatigue test, each girder was intentionally loaded to produce a crack near midspan. This provided an opportunity to experimentally determine the load required to crack the girder and the load required to subsequently decompress the girder. These loads were then compared with the calculated values.

Table 14
Loads and stresses for fatigue tests

	Girder		
	BT6	BT7	BT8
Loads			
Girder Weight, lb/ft	858.4	877.4	861.5
Deck and Haunch Weight, lb/ft	1,011.6	1,036.8	1021.9
Supplementary Total Dead Load, kips/block	11.4	11.5[a]	11.5
Max. Actuator Load, kips/ram	64.5	72.9	67.9
Min. Actuator Load, kips/ram	10.8	17.6	12.5
Decompression Load, kips/ram	38.3	35.6	35.4
Cracking Load, kips/ram	84.1	94.2[a]	82.0
Measured Cracking Load, kips/ram	84.1	89.1	74.0
Calculated Girder Stresses[b], psi			
Initial Prestress	3,119	3,116	3,096
Girder Weight	-772	-789	-774
Prestress losses	-251	-280	-286
Deck and Haunch	-909	-932	-919
Supplementary Dead Load	-299	-300	-302
Max. Actuator Load	-1,498	-1,672	-1,565
Net at Maximum Stress	-610	-857	-750
Min. Actuator Load	-251	-403	-289
Net at Minimum Stress	637	412	526
Concrete Stress Range	1,247	1,269	1,276

a Additional dead loads were not in place when girder was cracked.

b Calculated at the extreme bottom fiber of the girder at midspan. Compressive stresses are positive.

Measured loads corresponding to the first observed crack are included in table 14 for comparison with the calculated loads. The calculated loads are based on the measured values of the modulus of rupture. For Specimen BT6, excellent agreement was obtained between calculated and measured values. For Specimens BT7 and BT8, the measured loads were less than the calculated loads.

Determination of an exact decompression load was more difficult than expected because the load-strain relationships for the concrete adjacent to the cracks did not exhibit a well defined bend-over point to indicate when the cracks opened. However, the calculated decompression loads were in the range that was considered acceptable relative to the measured data.

The fatigue test was conducted as a load-controlled test. The loading was applied at a frequency of approximately 1.9 cycles per second. The effects of dynamic load amplification due to the mass of the specimen were accounted for by using the support reactions to establish the applied loads. The maximum and minimum loads applied by each actuator were determined experimentally such that the maximum and minimum support reactions were consistent with the static load calculations and static load tests. A photograph of the fatigue test setup is shown in figure 6.

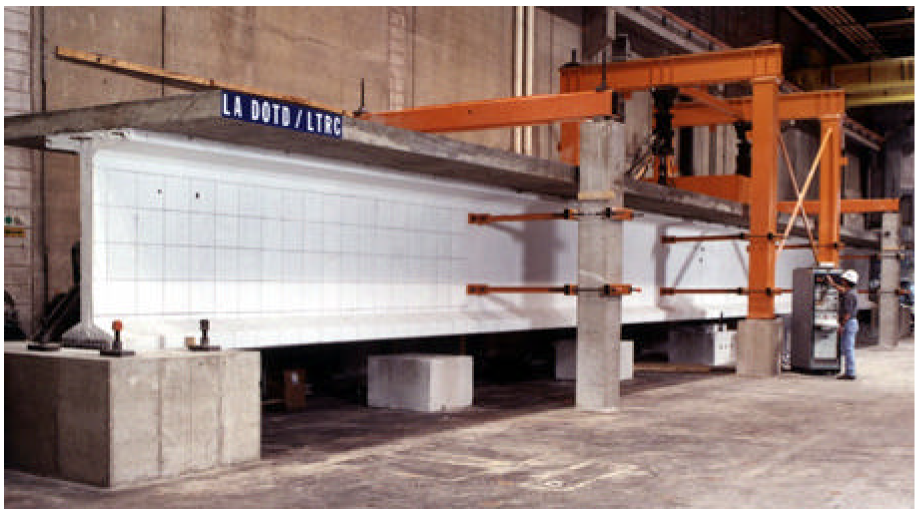


Figure 6
Fatigue test setup

Specimen BT6. The fatigue loads applied to Specimen BT6 were based on a calculated maximum tensile stress in the bottom surface of the girder of 610 psi (4.21 MPa). The measured relationships between average support reaction and concrete strain in the bottom flange measured with the sister bar gages are shown in figure 7 for the static tests conducted before the fatigue test and after each 1 million cycles of fatigue loading. There was no discernable change in the relationship for each of the tests. A similar behavior was observed in the midspan deflection shown in figure 8. Based on the data obtained during each static test, no adjustments to the dynamic loads were made throughout the fatigue test. Specimen BT6 was determined to have been subjected to 5 million cycles of loading without any apparent wire or strand breakage.

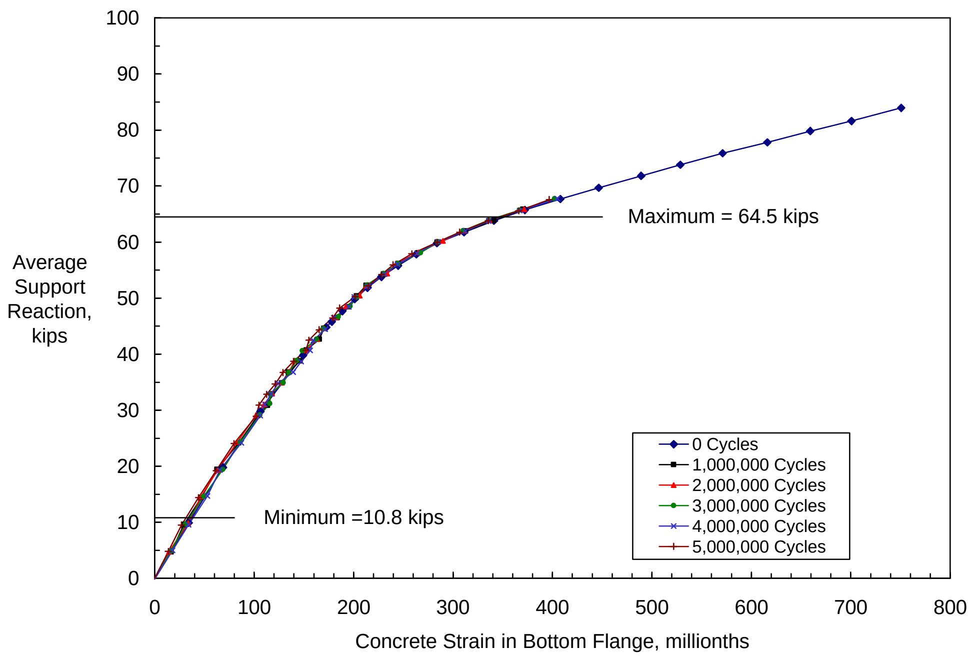


Figure 7
Support reaction versus concrete strain for fatigue test of Specimen BT6

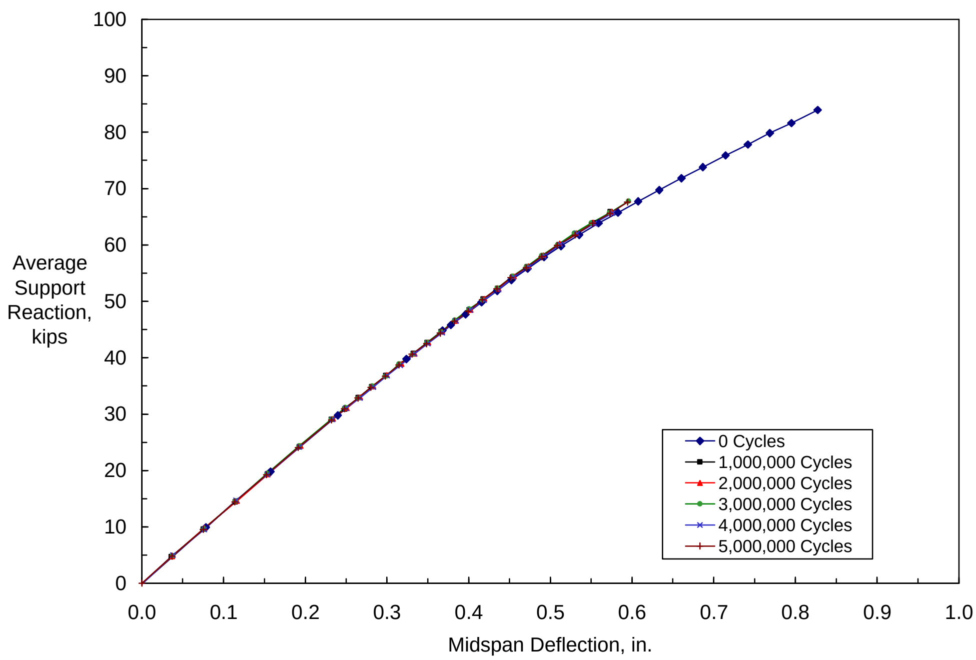


Figure 8
Support reaction versus midspan deflection for fatigue test of Specimen BT6

Figure 7 also shows the minimum and maximum support reactions of 10.8 and 64.5 kips (48.0 and 287 kN) used throughout the fatigue test. The measured strain range between minimum and maximum loads is 315 millionths, which corresponds to a stress range in the lower row of strands of 9,050 psi (62.4 MPa) based on the measured modulus of elasticity for the strand of 28,720 ksi (198.0 GPa).

Specimen BT7. Following the successful test of Specimen BT6, the loads applied to Specimen BT7 were based on a calculated maximum tensile stress in the bottom surface of the girder of 857 psi (5.91 MPa). After approximately 1.6 million cycles of fatigue loading, audible indications of wire breaks were noted. At the same time, it became more difficult to maintain the target loads overnight. After approximately 1.91 million cycles, additional vertical cracks and a horizontal crack in the bottom flange along one side of the girder were observed. The test was continued for a few more hours before being terminated. A static load test was conducted on the girder. As shown in figure 9, the vertical midspan deflection at the maximum load had increased by approximately 14 percent since the initial static test. The same increase was not evident in the bottom flange strains as measured by the sister bar gages. Nevertheless, it was decided to discontinue the fatigue test. Based on the strain ranges measured during the initial static tests, the stain range in the bottom row of strands was 410 millionths corresponding to a stress range of 11,780 psi (81.2 MPa) based on a measured modulus of elasticity of 28,720 ksi (198.0 GPa).

Prior to discarding Specimen BT7 after the shear tests, the midspan region was dissected over a length of 5 ft (1.52 m) either side of the center line to check for fatigue fractures. Three of the 24 strands in the cross section had completely fractured and eight other strands had at least one wire break. It is likely that the first fatigue fracture occurred at about 1.6 million cycles of loading. Photographs of fractures of the complete strand, multiple wires, and single wire are shown in figure 10.

Specimen BT8. Following the testing of Specimens BT6 and BT7, the loads applied to Specimen BT8 were based on a calculated maximum tensile stress in the bottom surface of the girder of 750 psi (5.17 MPa). After about 2.25 million cycles, audible indications of wire breaks were noted. At 2.5 million cycles, cracks on one side of the girder bottom flange were noticeably wider. The fatigue test was temporarily halted and an interim static test was conducted. The test did not indicate any significant reduction in prestress force or girder stiffness. The fatigue test was, therefore, resumed. Shortly thereafter, a piece of concrete from the bottom flange on one side of the girder fell off. The fatigue test was stopped and another static test conducted. Plots of average support reaction versus bottom flange strain

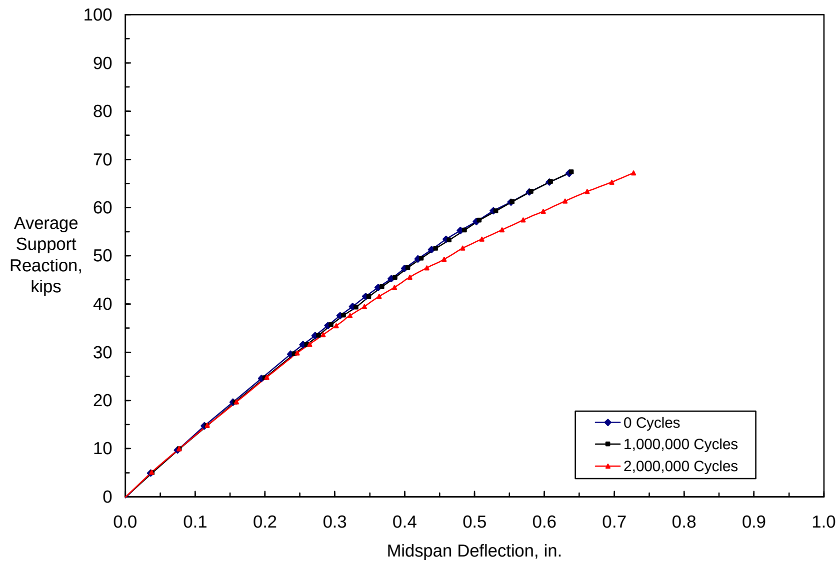
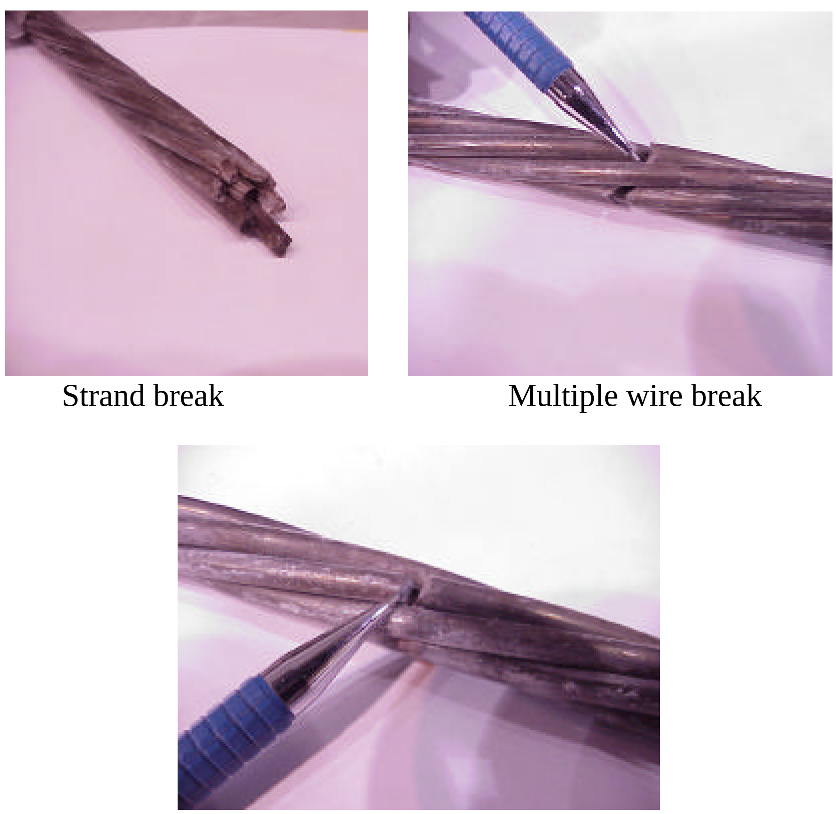


Figure 9
Support reaction versus midspan deflection for fatigue test of Specimen BT7



Strand break   Multiple wire break

Single wire break

Figure 10
Fatigue fractures

and midspan deflection are given in figures 11 and 12, respectively. A significant increase occurred in the bottom flange strain and a small increase in the overall deflection was evident during the 2.5 million cycle static test. Based on the strain ranges measured during the initial static tests, the strain range in the bottom row of strands was 335 millionths corresponding to a stress range of 9,620 psi (66.3 MPa) based on a measured modulus of elasticity of 28,720 ksi (198.0 GPa).

Prior to discarding Specimen BT8 after the shear tests, the midspan region was dissected over a length of 5 ft (1.52 m) either side of the centerline to check for fatigue fractures. One of the 24 strands in the cross section had all seven wires fractured. A second strand had four wires fractured and a third strand had one wire fractured. It is likely that the first fatigue fracture occurred at about 2.25 million cycles of loading.

Summary of fatigue test results. Results of the three fatigue tests are summarized in table 15. The calculated concrete stresses are based on an uncracked section as used to calculate the applied loads. The number of cycles to first wire break are based on audible detection and may not truly represent the first wire break. The strand stress ranges are based on the measured strains during the static tests and assume that the strain ranges measured on the sister bars are the same strain ranges that occurred in the strands. The minimum strand stress is based on the same assumptions used to determine the test loads.

Table 15
Fatigue test results

Test Specimen	Calculated Concrete Stresses (psi)		First Detected Wire Fracture	Loading Cycles Achieved	Strand Stress (psi)	
	Maximum	Range			Range	Minimum
BT6	610	1,247	None	5,000,000	9,050	167,000
BT7	857	1,269	1,600,000	1,910,000	11,780	166,000
BT8	750	1,276	2,250,000	2,500,000	9,620	162,000

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. A photograph of the shear test setup is shown in figure 13. The first diagonal crack in each

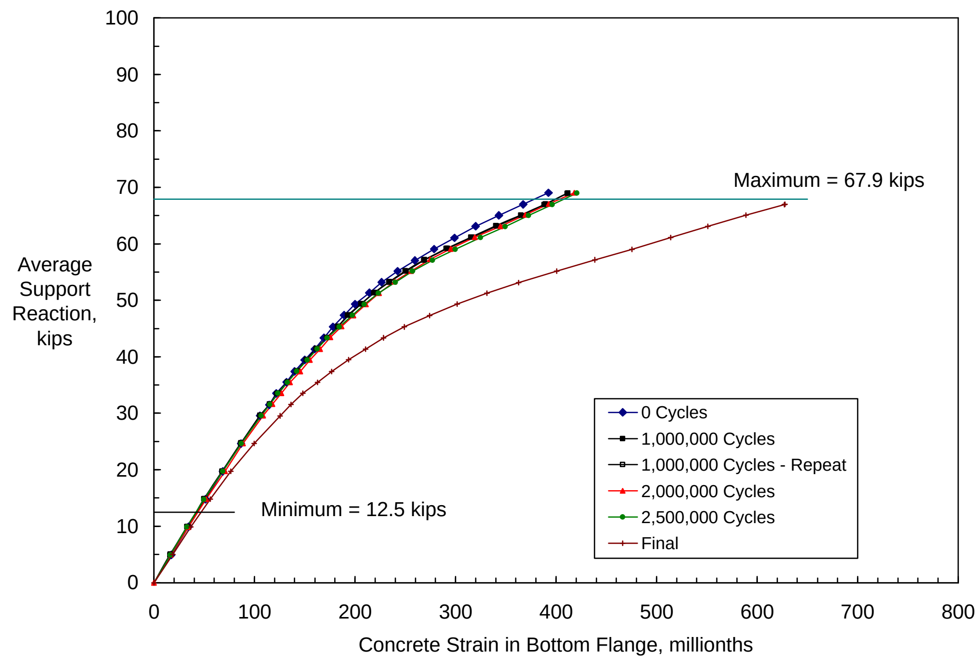


Figure 11
Support reaction versus concrete strain for fatigue test of Specimen BT8

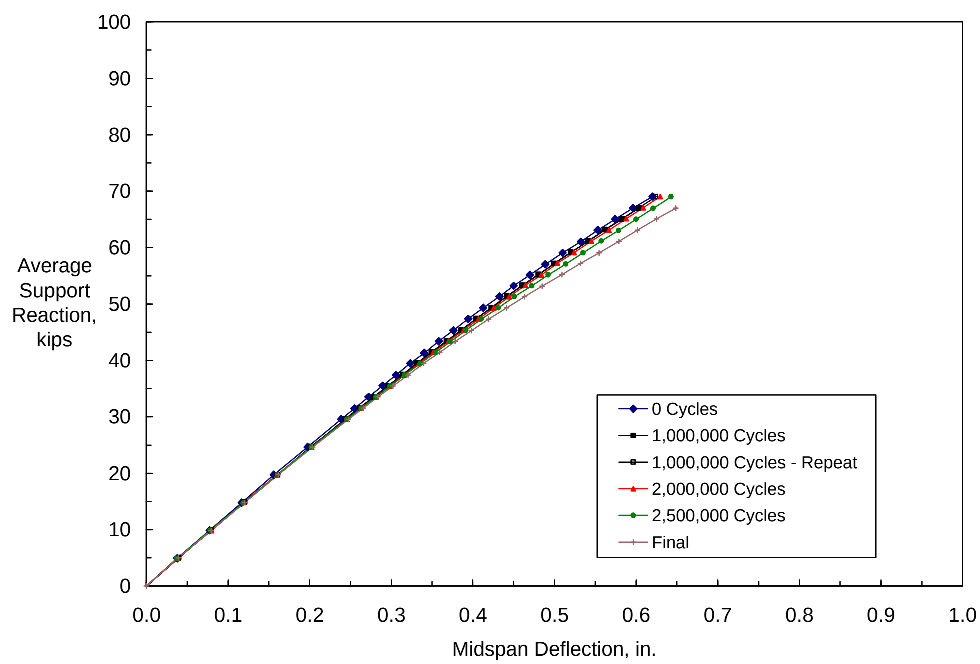


Figure 12
Support reaction versus midspan deflection for fatigue test of Specimen BT8

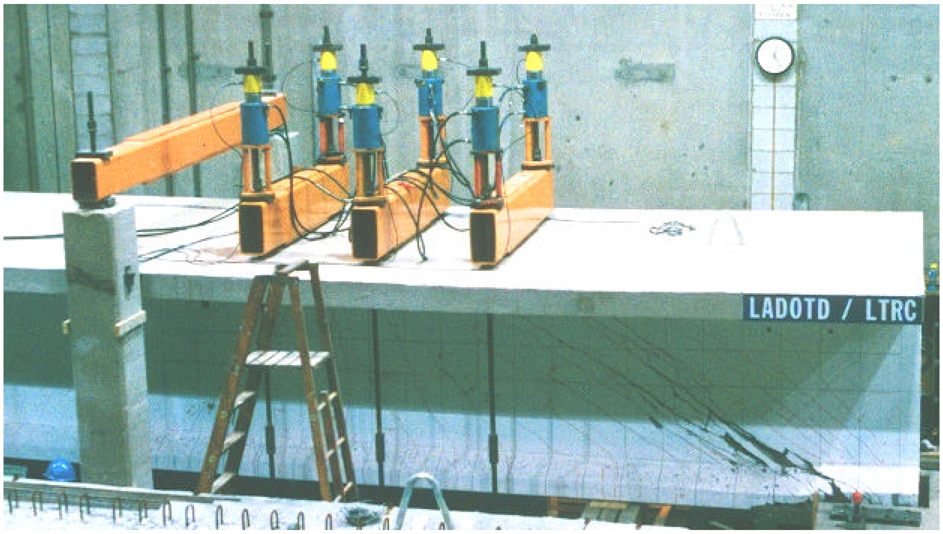


Figure 13
Shear test setup

specimen occurred at an applied shear that ranged from 270 to 302 kips (1.20 to 1.34 MN) as reported in table 16. 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.

Table 16
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.

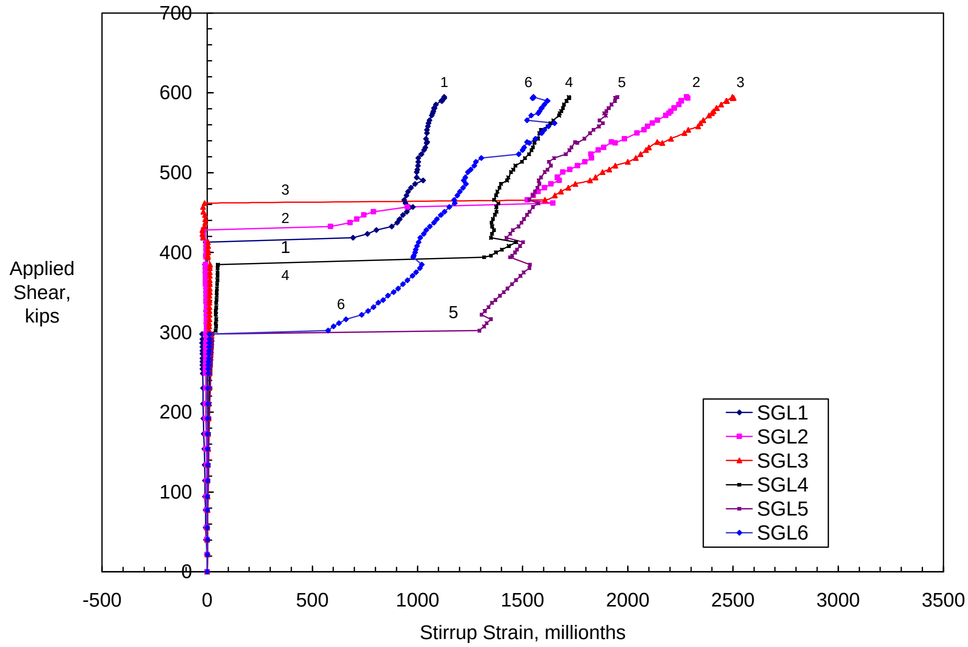


Figure 14
Stirrup strains in Specimen BT8-Live

When the first diagonal crack formed, a large increase in strain occurred in the stirrups intercepted by the crack as shown in figure 14 for stirrup strain gages SGL5 and SGL6 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 16 together with the angle of the diagonal cracks as measured from the horizontal. A photograph of the diagonal crack pattern in Specimen BT8-Live is given in figure 15. A description of the behavior of each specimen as it relates to the maximum shear is given in the following sections.

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

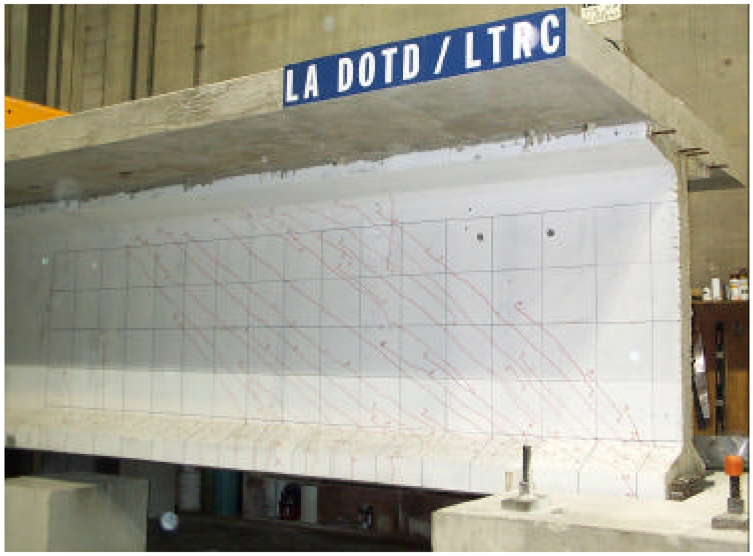


Figure 15
Diagonal crack pattern in Specimen BT8-Live

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 exhibited 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 16.

Figure 16
Specimen BT8-Dead after the shear test

Comparison of results. The measured applied shear forces at first cracking and maximum load are compared in figure 17 where the specimens are arranged in pairs. 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, it can be seen that 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.

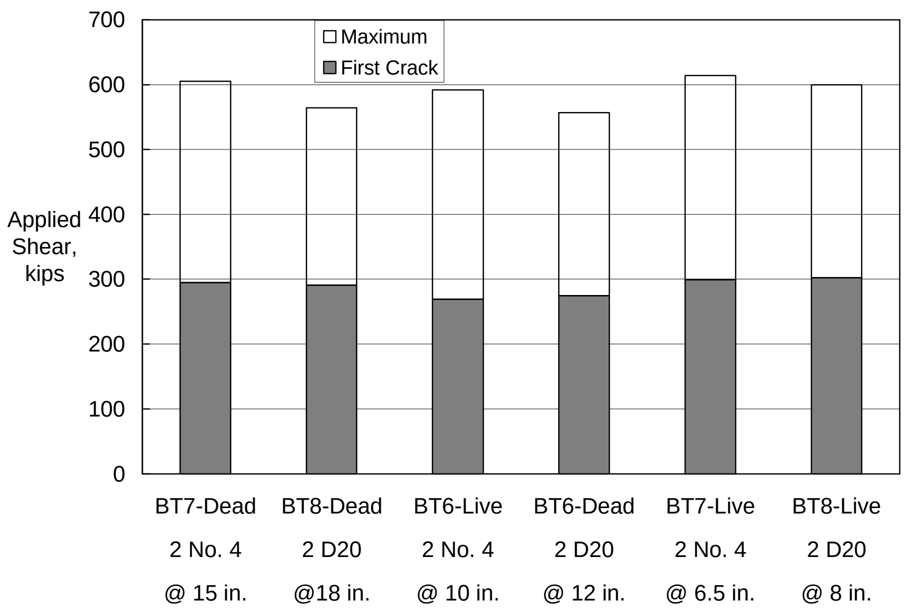


Figure 17
Comparison of applied shear forces

The pairs of specimens in figure 17 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 there was no nonprestressed longitudinal reinforcement included in the bottom flange. The lack of this reinforcement caused the performance of the specimen to be one in which the strength was controlled by slip of the strands. The presence of the longitudinal reinforcement in BT7-Dead and BT8-Dead was beneficial in preventing slip of the strands and enhancing the strength of the test specimens. This reinforcement consisted of 4 No. 6 (19-mm diameter) bars 7 ft (2.13 m) long and 4 No. 6 (19-mm diameter) bars 19 ft (5.79 m) long.

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 using a configuration that was comparable to the design loading, the specimens would probably have failed in flexure and nothing would have been learned about the shear strength. Consequently, the specimens 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

2. Using the provision of the *AASHTO Standard Specifications [3]* with measured material properties, measured self weights, and measured cross-sectional dimensions

3. Using the provisions of the Sectional Design Model of the *AASHTO LRFD Specifications [4]* with specified material properties and nominal section dimensions

4. Using the provisions of the Sectional Design Model of the *AASHTO LRFD Specifications [4]* with measured material properties, measured self weights, and measured cross-sectional dimensions

Analyses 1 and 3 correspond to design calculations. Analyses 2 and 4 correspond to calculations for analysis of an existing structure using as-built dimensions and measured material properties. In performing the analysis, the material properties given in table 17 were used. A total of 24 strength analyses were made.

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

Table 17
Material properties used in analyses of shear test results

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

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 18 and 19 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 18 and 19 use a f factor of 1.0. The measured

Table 18
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.

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.

In the case of the nominal shear strength calculations using the *AASHTO LRFD Specifications*, two values are reported in table 19. 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 in the bottom flange 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 19
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 18. 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 18 also exhibit the expected result that shear strength increases as the shear reinforcement spacing decreases. In all tests, the measured strengths were greater than the calculated strengths.

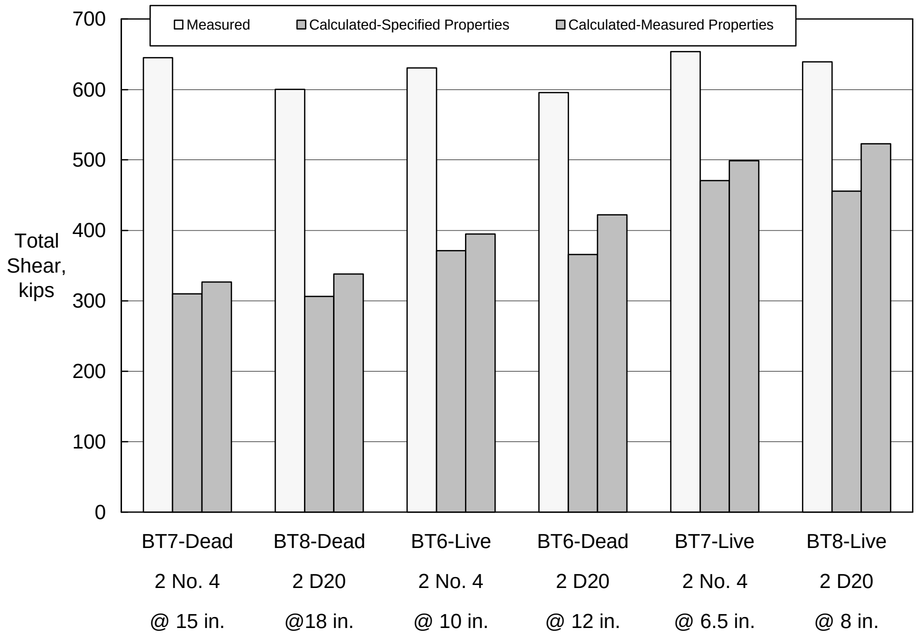


Figure 18
Comparison of measured and calculated strengths using the *AASHTO Standard Specifications*

A graphical comparison of measured and calculated strengths using the *AASHTO LRFD Specifications* is given in figure 19. In all tests, the measured strengths were greater than the calculated strengths. The calculated strengths are the lower values given in table 19, 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.

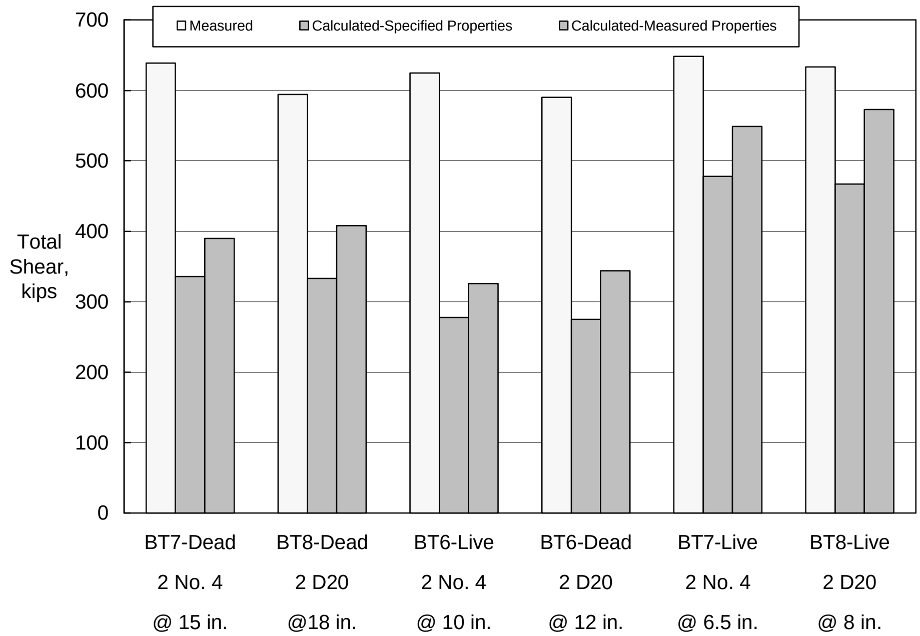


Figure 19
Comparison of measured and calculated strengths
using the *AASHTO LRFD Specifications*

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 equally effective 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

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

- A 72-in. (1.83-m) deep prestressed concrete bulb-tee girder made with 10,000 psi (69 MPa) compressive strength concrete endured 5 million of flexural loading when the maximum concrete tensile stress used in design was 610 psi (4.21 MPa).

- Two 72-in. (1.83-m) deep prestressed concrete bulb-tee girders with concrete compressive strengths greater than 10,000 psi (69 MPa) endured 2.5 million cycles of flexural fatigue loading when the maximum concrete tensile stresses used in design were 750 and 857 psi, (5.17 and 5.91 MPa), respectively.

- 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.

RECOMMENDATIONS

The following recommendations are based on the conclusions listed in the previous section:

- 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 provided the concrete design tensile stress is limited to a maximum value of $6\sqrt{f_c'}$ = 600 psi (4.14 MPa).

- 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*.

- The maximum level of concrete tensile stress used in flexural design of high-strength prestressed concrete girders should be limited to $6\sqrt{f_c'}$ until further testing indicates that fatigue fractures of the strand will not occur.

- 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.

- Additional fatigue tests should be made on uncracked girders using a slower rate of loading to determine if satisfactory performance at a higher level of concrete stress than $6\sqrt{f_c'}$ can be achieved.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO	=	America Association of State Highway and Transportation Officials
A_{ps}	=	area of prestressing steel
ASTM	=	American Society for Testing and Materials
BT	=	bulb-tee
^{o} C	=	degree Celsius
CRD	=	Concrete Research Division
CTL	=	Construction Technology Laboratories, Inc.
cu	=	cubic
DDAS	=	digital data acquisition system
DL	=	dead load
E_c	=	modulus of elasticity
^{o} F	=	degree Fahrenheit
ft	=	foot
f'_c	=	concrete compressive strength
GCP	=	Gulf Coast Pre-Stress, Inc.
GPa	=	gigapascal
in.	=	inch
K factor	=	multiplier for development length
kg	=	kilogram
kN	=	kilonewton
ksi	=	kip per square inch
LADOTD	=	Louisiana Department of Transportation and Development
LL	=	live load
LRFD	=	Load Resistance Factor Design
LTRC	=	Louisiana Transportation Research Center
lb	=	pound
MN	=	meganewton
MOR	=	modulus of rupture
MPa	=	megapascal
m	=	meter
ml	=	milliliter
mm	=	millimeter
No.	=	number
P	=	concentrated load or prestressing force

Petro = petrographic examination
pp = pages
psi = pound per square inch
sq = square
V_c = nominal shear resistance provided by the concrete
V_n = nominal shear resistance of the section
yd = yard
f = resistance factor
a = coefficient of thermal expansion

REFERENCES

1. Bruce, R. N.; Russell, H. G.; Roller, J. J.; and Martin, B. T., “Feasibility Evaluation of Utilizing High Strength Concrete in Design and Construction of Highway Bridge Structures,” Final Report - Louisiana Transportation Research Center, Research Report No. FHWA/LA-94-282, Baton Rouge, LA, 1994, 168 pp.

2. Bruce, R. N.; Russell, H. G.; Roller, J. J.; and Hassett, B. M., “Implementation of High Performance Concrete in Louisiana Bridges,” Final Report - Louisiana Transportation Research Center, Research Report No. 310, Baton Rouge, LA, 2001, 67 pp.

3. *Standard Specifications for Highway Bridges*, Sixteenth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996, 677 pp.

4. *AASHTO LRFD Bridge Design Specifications*, Second Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1998.

5. CONSPAN Version 6.0, Leap Software Inc., Tampa, FL.

6. CONSPAN LRFD Version 1.1, Leap Software Inc., Tampa, FL.

7. Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete, ASTM A 416, American Society for Testing and Materials, Volume 01.04, West Conshohocken, PA, 1999.

8. 2000 Interim *AASHTO LRFD Bridge Design Specifications*, Second Edition (1998), American Association of State Highway and Transportation Officials, Washington, DC, 2000.