Project Review Committee

Each research project has an advisory committee appointed by the LTRC Director. The Project Review Committee (PRC) is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

LTRC appreciates the dedication of the following Project Review Committee members in guiding this research study to fruition.

LTRC Manager

Walid R. Alaywan
Senior Structures Research Engineer

Members

Doug Hood, DOTD
Paul Fossier, DOTD
Masood Rasoulian, DOTD
Mike Boudreaux, LTRC
John Eggers, DOTD
Arturo Aguirre, FHWA

Directorate Implementation Sponsor

William T. Temple, DOTD Chief Engineer

TECHNICAL REPORT STANDARD PAGE

Report No. **FHWA/LA.05/395**

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
FATIGUE AND SHEAR BEHAVIOR OF HPC BULB-TEE GIRDERS FINAL REPORT

5. Report Date **February 2005**

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
Final Report
PERIOD OCTOBER 2003 – DECEMBER 04

14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract
Five 96-ft. (29.3-m) long, 72-in. (1.83-m) deep, precast, pretensioned bulb-tee girders were tested to evaluate their behavior under flexural fatigue. Three of the girders were also tested to measure their static shear strength. One girder was tested after the fatigue test to measure its flexural strength. The five 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 three girders 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 five 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. Three girders were intentionally cracked at midspan before the start of the fatigue test. Two girders were uncracked. After completion of fatigue testing, the three intentionally cracked girders were cut in half and the six girder ends tested to evaluate static shear strength.

The cracked bulb-tee girders performed satisfactorily under 5 million 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 concrete tensile stress was 750 psi (5.17 MPa) or larger, fatigue fractures of the prestressing strand in the cracked girders occurred, and the fatigue life of the girder was reduced. However, the uncracked girders performed satisfactorily under 5 million cycles of flexural fatigue loading when the tensile stress was 600 and 750 psi (4.14 and 5.17 MPa).

The measured flexural strength of one girder, after being subjected to 5 million cycles of flexural fatigue loading, 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.

Measured shear strengths of six bulb-tee girder ends 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. Based on the results of the shear tests, the existing limitation of 60,000 psi (414 MPa) for the design yield stress of transverse reinforcement cited 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, Flexural Strength, 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**	**75**	**N/A**

FATIGUE AND SHEAR BEHAVIOR OF HPC BULB-TEE GIRDERS

FINAL 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

February 2005

ABSTRACT

Five 96-ft. (29.3-m) long, 72-in. (1.83-m) deep, precast, pretensioned bulb-tee girders were tested to evaluate their behavior under flexural fatigue. Three of the girders were also tested to measure their static shear strength. One girder was tested after the fatigue test to measure its flexural strength. The five 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 three girders 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 five 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. Three girders were intentionally cracked at midspan before the start of the fatigue test. Two girders were uncracked. After completion of fatigue testing, the three intentionally cracked girders were cut in half and the six girder ends tested to evaluate static shear strength.

The cracked bulb-tee girders performed satisfactorily under 5 million 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 concrete tensile stress was 750 psi (5.17 MPa) or larger, fatigue fractures of the prestressing strand in the cracked girders occurred, and the fatigue life of the girder was reduced. However, the uncracked girders performed satisfactorily under 5 million cycles of flexural fatigue loading when the tensile stress was 600 and 750 psi (4.14 and 5.17 MPa).

The measured flexural strength of one girder, after being subjected to 5 million cycles of flexural fatigue loading, 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.

Measured shear strengths of six bulb-tee girder ends 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. Based on the results of the shear tests, the existing limitation of 60,000 psi (414 MPa) for the design yield stress of transverse reinforcement cited 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 were utilized and implemented in the original design of the proposed Rigolets Pass Bridge on Highway U.S. 90 east of New Orleans (SP No. 006-05-0045). The original design used 130-ft. (40-m) long, 72-in. (1.83-m) deep, high performance concrete bulb-tee girders spaced at 7.87 ft. (2.40 m). A redesign of the bridge uses 130-ft. (40-m) long, 78-in. (1.98-m) deep bulb-tee girders spaced at 12.6 ft. (3.83 m). Construction of the bridge began 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

LIST OF TABLES

LIST OF FIGURES

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 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 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, static shear, and static flexural 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, static shear, and static flexural loading conditions.

- Determine if a higher allowable concrete tensile stress can be used in the 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 utilizing 72-in. (1.83-m) deep prestressed concrete bulb-tee girders using a design concrete compressive strength of 10,000 psi (69 MPa).
- Design five full-scale test specimens based on the prototype bridge designs.
- Instrument, fabricate, and ship five test girders.
- Cast a high performance concrete deck slab on each girder to complete the test specimen.
- Test each specimen under flexural fatigue loading.
- Test both ends of three specimens under static shear loading.
- Test one specimen to determine static flexural strength.
- 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 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 in 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). The girder haunch load is based on a 3-in. (76-mm) deep haunch. Design dead loads did not include superimposed loads from barrier rails or a 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 classifications 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 *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. This is 0.5 in. (12.7 mm) less than the haunch depth used for calculation of dead load. 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.

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. It is anticipated that the use of $7.5\sqrt{f'_c}$ compared to $6\sqrt{f'_c}$ can result in a small reduction in the number of prestressing strands.

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

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
Eccentricity to center of gravity of strands, in.	33.13	33.10	—	—

A dash indicates that the property is not applicable.

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	—
	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.

In calculating prestress loss due to concrete shrinkage, a relative humidity of 75 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 five test specimens were designated BT6, BT7, BT8, BT11, and BT12 to follow the numbering sequence established from the previous feasibility study *[1]*. Girders for Specimens BT9 and BT10 were also cast but measured concrete cylinder compressive strengths were less than the specified values and the girders were rejected and recast as BT11 and BT12. The ends of each specimen were designated “live” or “dead” corresponding to their locations in the precasting bed. The “live” end is the end of the bed at which the strands are tensioned. The “dead” end is the end at which the strands are anchored before tensioning. The designs of Specimens BT6, BT11, and BT12 were 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 Specifications* and *LRFD Specifications*) prepared by the LADOTD required 24 0.6-in. (15.2-mm) diameter Grade 270 prestressing strands for a typical interior girder. Therefore, the 5 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. Strand debonding lengths in the test specimens were also the same as calculated for the prototype bridge. For BT6, BT11, and BT12, pairs of strands in each girder were debonded over lengths of 21, 24, and 30 ft. (6.40, 7.32, and 9.14 m) from the ends of the girders. For

BT7 and BT8, six strands in each girder were debonded over a 9 ft. (2.74 m) length. 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 developing a flexural failure elsewhere.

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 standard truck load used in the design with the *Standard Specifications* or a combination of uniformly distributed lane load and truck or tandem load used in the design with 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 in 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 was provided on how to account for 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 as follows:

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 102-mm centers for 813 mm). This is a standard LADOTD detail.

The test region extended from the end of the first region to a point beyond the first concentrated load. This is the region in which the shear failure was expected to occur during testing. The size and spacing of the shear reinforcement in the test region was the same as that required at the critical section in the corresponding bridge design and was maintained constant throughout the test region.

The midspan region incorporated the length from the end of the test region 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 design yield 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 included 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

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.
BT11 BT12	Standard	Cement and ground granulated blast-furnace slag (50%)	Live and 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.

prestressing steel depends on the assumed variation of resistance over the transfer and development length of the strand. Consequently, the design of the live end of Girder BT7 was based on a linear variation of resistance over the transfer length of 60 in. (1.52 m) followed by a parabolic variation from the end of the transfer length to the end of the development length 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 FHWA multiplier. The dead end of Girder BT7 was based on a linear variation of resistance over the development length including the FHWA 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 used in Girder BT7. A yield strength of 70 ksi

(483 MPa) for the welded wire reinforcement was used to determine the quantity of 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.

Shear reinforcement at both ends of Girders BT11 and BT12 was the same as that at the dead end of Girder BT6.

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, longitudinal nonprestressed reinforcement, consisting of eight No. 6 (19 mm diameter) bars, was required and provided at both ends of Girders BT7 and BT8. Four bars had a length of 7 ft. (2.13 m) and four bars had a length of 19 ft. (5.79 m).

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 uniform 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 *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). The deck concrete cementitious materials for each of the five specimens are 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

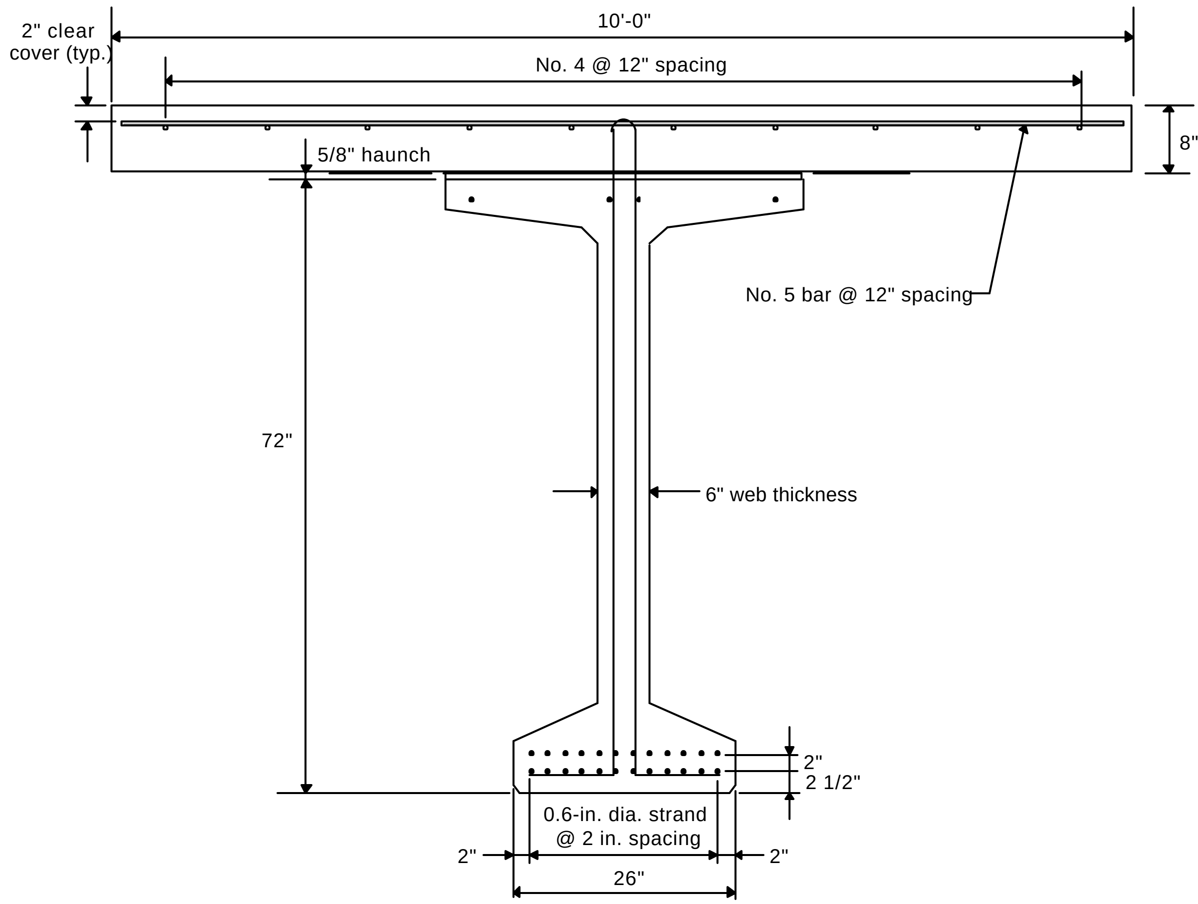


Figure 1
Test specimen cross section

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 five 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. Girders BT6, BT7, and BT8 were fabricated at the same time on a single casting bed, as shown in figure 2. Girders BT11 and BT12 were cast later on a single casting bed. The concrete mix proportions used for each set of girders are given in table 5. The mix proportions used for Girders BT6, BT7, and BT8 were essentially the same as the mix proportions used in the girders for the Charenton Canal Bridge project *[2]*. During the course of the project, LADOTD changed its specifications to allow the use of ground granulated blast-furnace slag. Consequently, this material was used in Girders BT11 and BT12.

Figure 2
Girder fabrication

Table 5
Mix proportions for girder concrete

Material	Quantities			
	BT6, BT7, BT8		BT11, BT12	
	per yd^3	per m^3	per yd^3	per m^3
Portland Cement – Type III	691 lb	410 kg	740 lb	439 kg
Fly Ash – Class C	296 lb	176 kg	—	—
GGBFS	—	—	247 lb	147 kg
Fine Aggregate	1,135 lb	673 kg	1,128 lb	669 kg
Course Aggregate – Limestone	1,803 lb	1,070 kg	1,800 lb	1,068 kg
Water	247 lb	147 kg	247 lb	147 kg
Water Reducer, ASTM C 494 – Type D	80 fl oz	3.094 L	60 fl oz	2.320 L
High-Range Water Reducer, ASTM C 494 – Type F	160 fl oz	6.189 L	270 fl oz	10.44 L
Air Entrainment	None	None	None	None
Water-Cementitious Materials Ratio	0.25	0.25	0.25	0.25

During casting, GCP produced match-cured cylinders for determination of concrete compressive strength development. In addition, the research team also prepared concrete cylinders and beam 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, 6 of the 24 prestressing strands in each girder 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).

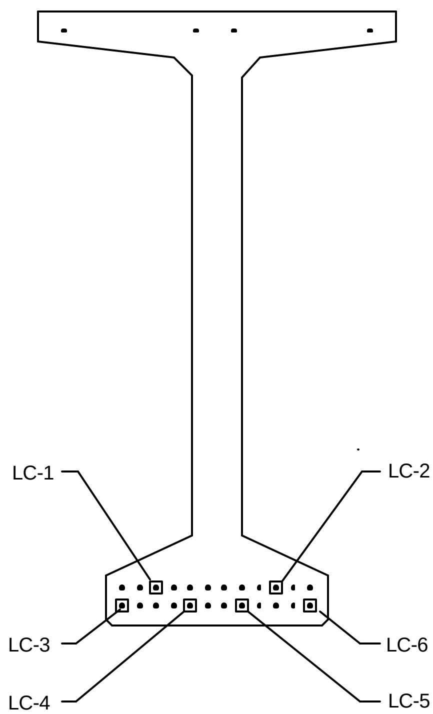


Figure 3
Location of load cells

Internal strain gauges. After pretensioning the strands and prior to casting the concrete, three vibrating wire concrete strain gauges and four “sister bars” instrumented with electrical resistance strain gauges were installed in each girder specimen at midspan. The three vibrating wire concrete strain gauges 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 gauges were also installed in the top flange of each girder.

Girder camber measurements. Immediately after casting and while the concrete was still plastic, steel bolts were embedded in the top surface of each girder at midspan and 6 in. (150 mm) from each girder end to provide permanent fixed points for camber measurements. Camber measurements were made using a level to sight elevations at each point. Midspan camber relative to the ends of each girder was 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 Girders BT6, BT7, and BT8, weldable electrical resistance strain gauges 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 gauge was installed at approximately midheight of the girder cross section. In addition, two weldable strain gauges 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 five girders. However, as indicated in table 4, the deck slab concrete for different specimens incorporated different combinations of cementitious materials. The concrete mix proportions used for the deck concretes are shown in table 6.

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.

Table 6
Mix proportions for deck concrete

Material	BT6, BT11, BT12	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

Fatigue test setup and procedure

The configuration for the fatigue tests is shown in figure 4. Specimens were simply supported near the ends, 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 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 1.9 hertz for Specimens BT6, BT7, and BT8 and 1.0 hertz for Specimens BT11 and BT12. The upper- and lower-bound loads applied by the actuators were determined from the support reactions while loading at 1.9 or 1.0 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 the fatigue tests for BT6, BT7, and BT8, 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 gauges were installed immediately

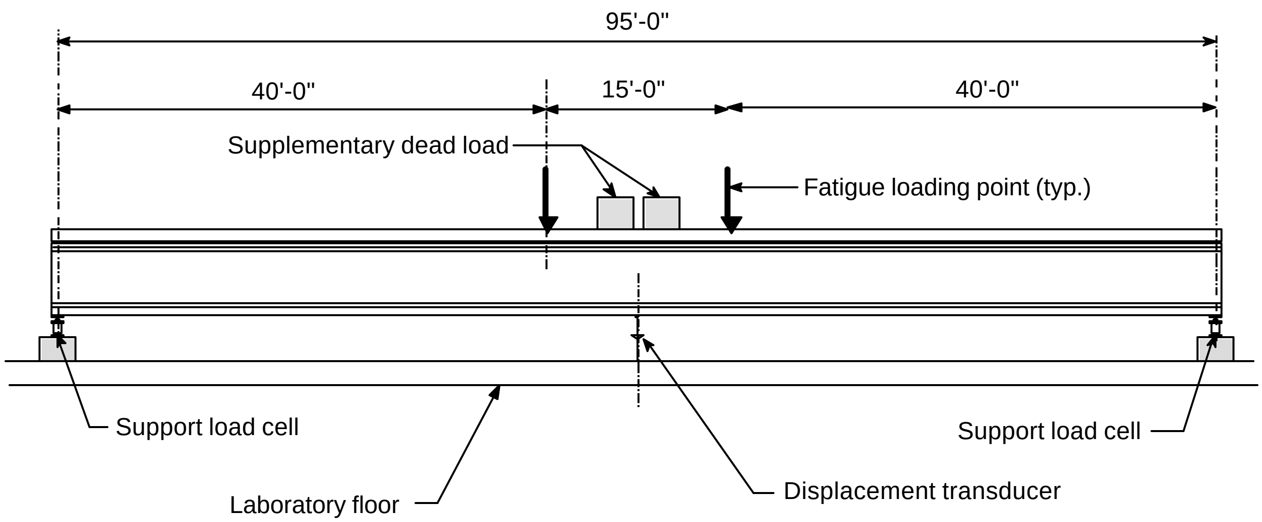


Figure 4
Fatigue test configuration

adjacent to the crack on the bottom concrete surface of the lower flange. After the gauges 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. Specimens BT11 and BT12 were not cracked before the start of the fatigue tests.

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

The fatigue load range was selected to produce a midspan bending 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 *Standard Specifications*. For Girders BT7, BT8, BT11, and BT12, the moment range corresponded with Service Load III of the *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

Table 7
Levels of maximum tensile stress for fatigue tests

Girder	Condition	Tensile Stress	f'_c (psi)	Tensile Stress (psi)
BT6	Precracked	$6.0\sqrt{f'_c}$	Specified Strength = 10,000	610[a]
BT7	Precracked	$7.5\sqrt{f'_c}$	Measured Strength = 13,050	857
BT8	Precracked	$7.5\sqrt{f'_c}$	Specified Strength = 10,000	750
BT11	Uncracked	$6.0\sqrt{f'_c}$	Specified Strength = 10,000	600
BT12	Uncracked	$7.5\sqrt{f'_c}$	Specified Strength = 10,000	750

a Stress was 10 psi higher than intended.

static load test to measure specimen response at the full design service load condition. Prior to each static test, midspan camber and prestress losses were measured. For precracked girders, the bottom flange decompression load was 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.

During the static load tests, output from 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 were 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 tests of Specimens BT6, BT7, and BT8, the girders were cut in half and the two ends were tested to evaluate static shear strength performance. Each specimen half was placed on supports creating a simply-supported span. One support was located at the as-cast end of the girder. The other support was located near the cut end of the girder. 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.

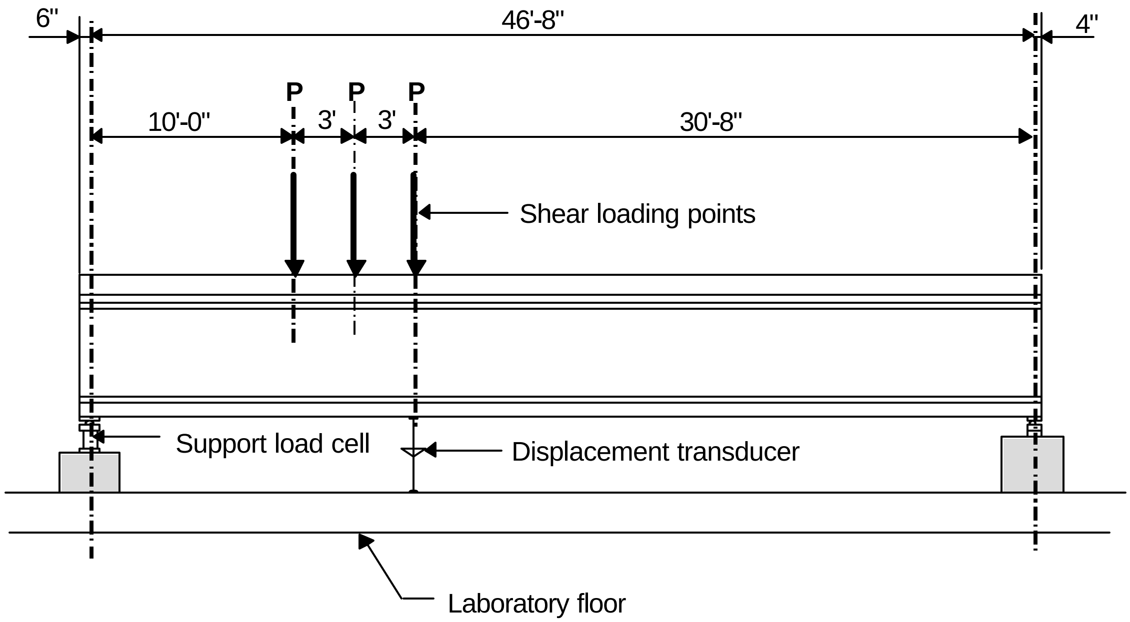


Figure 5
Shear test configuration

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 jacking 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 strand slippage relative to the concrete. Strain gauges on the stirrups were used to monitor strains in the shear reinforcement.

Load was applied incrementally to each specimen. Output from the 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.

Flexural strength test setup

After completion of the fatigue loading test of Specimen BT12, a flexural test to destruction was performed. The test configuration for the flexural strength test was similar to that for the fatigue test shown in figure 4 except that the support load cells were replaced with a rigid concrete support and the supplementary dead load was removed. The flexural strength test consisted of two stages. In the first stage, the test specimen was statically loaded in a series of increments until a flexural crack developed in the constant moment region. The applied load was then decreased to zero and strain gauges installed on the bottom surface of the lower flange immediately adjacent to a crack.

After the gauges were installed, the specimen was statically loaded again in a series of increments until a flexural failure occurred. During the test, it was necessary to restroke the jacks several times to accommodate the large specimen deflection. Output from the 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. The test was terminated when the prestressing strands fractured. Following the test, the bottom flange of the girder was dissected to inspect the fractured strands.

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 test ages corresponded to release of the strands, 56 days, and start of the fatigue, shear, or flexural tests. Coefficient of thermal expansion was determined at a concrete age of 84 days for the concrete used in Girders BT6, BT7, and BT8 and at 76 days for the concrete used in Girders BT11 and BT12. In addition, petrographic examination of core samples extracted from the midspan region of Girders BT6, BT7, and BT8 after completion of fatigue testing was conducted. A complete matrix of the planned tests for the girder concrete is given in tables 8a and 8b.

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 tables 8a and 8b were 4x8-in. (102x203-mm) cylinders that were match cured until the time the girder prestress force was released. Specimens designated as “field” in the same column of tables 8a and 8b were 6x12-in. (152x305-mm) cylinders initially cured on the side of the steel forms under the tarpaulin that covered the girders after casting. After release, all

Table 8a
Girder concrete material property testing program for Specimens BT6, BT7, and BT8

Concrete Age	Material Property[a]	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	—
56 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[b]	Core	CTL	—	1	—	—	1	—	—	1	—

A dash indicates that the property was not measured.
a f'_c, E_c = compressive strength and modulus of elasticity, respectively.
" = coefficient of thermal expansion.
MOR = modulus of rupture.
b 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 8b
Girder concrete material property testing program for Specimens BT11 and BT12

Concrete Age	Material Property[a]	Curing Method	Testing Responsibility	BT11 Midspan	BT12 Midspan
Release	f'_c, E_c, w	Match	LTRC	3	3
7 Days	f'_c, E_c, w	Match	LTRC	3	—
28 Days	f'_c, E_c, w	Match	LTRC	3	3
28 Days	MOR	Field	LTRC	3	3
56 Days	f'_c, E_c, w	Match	LTRC	3	3
56 Days	MOR	Field	LTRC	3	3
56 Days	"	Field	CTL	3	3
Fatigue Test	f'_c, E_c	Match	LTRC	3	3
Fatigue Test	f'_c, E_c	—	CTL	3[b]	3[b]
Fatigue Test	MOR	Field	LTRC	3	3
Flexural Test BT12	f'_c, E_c, w	Match	LTRC	2	1
90 Days	f'_c, E_c, w	Match	LTRC	3	3

A dash indicates that the property was not measured.
a f'_c, E_c, w = compressive strength, modulus of elasticity, and unit weight, respectively.
MOR = modulus of rupture.
" = coefficient of thermal expansion.
b Concrete cores extracted from midspan region of each girder.
Summary:
36 match-cured 4x8-in. cylinders.
6 field-cured 6x12-in. cylinders.
18 field-cured 6x6x20-in. MOR beams.
6 3x6-in. concrete cores.

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 deck slab of each specimen. 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 testing. 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 both end regions of BT6, BT7, and BT8 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 of BT6, BT7, and BT8 was conducted. A complete test matrix of the planned tests for the deck slab concrete is given in tables 9a and 9b.

Table 9a
Deck slab concrete material property testing program for Specimens BT6, BT7, and BT8

Concrete Age	Material Property[a]	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[b]	Core	CTL	—	2	—	—	2	—	—	2	—

A dash indicates that the property was not measured.
a f'_c, E_c = compressive strength and modulus of elasticity, respectively.
b 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.

Table 9b
Deck slab concrete material property testing program for Specimens BT11 and BT12

Concrete Age	Material Property[a]	Curing Method	Testing Responsibility	BT11	BT12
7 Days	f'_c	Laboratory	CTL	3	3
28 Days	f'_c	Laboratory	CTL	3	3
Fatigue Test	f'_c, E_c	Laboratory	CTL	3	3

A dash indicates that the property was not measured.
a f'_c, E_c, = compressive strength and modulus of elasticity, respectively.
Summary:
18 laboratory-cured 6x12-in. cylinders.

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 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 the girders. Measured values of compressive strength, modulus of elasticity, and unit weight for various test ages are given in table 10. Test ages other than 1 and 56 days correspond to the start of the fatigue, shear, or flexural 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 tables 8a and 8b. 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. Modulus of rupture specimens were made from concrete representing the midspan region of each girder. Measured values of modulus of rupture for various test ages are given in table 11. The measured values always exceeded the normally assumed value of $7.5\sqrt{f_c'}$. The average measured coefficient of thermal expansion was 5.54 millionths/° F (9.97 millionths/° C) at 84 days for the concrete used in Girders BT6, BT7, and BT8 and 6.68 millionths/° F (12.1 millionths/° C) at 76 days for the concrete used for Girders BT11 and BT12.

Measurements of compressive strength and modulus of elasticity were made on test specimens representing concrete used in the deck slab of the test specimens. Measured properties are shown in table 12. For the concrete used in the deck slab of BT12, inconsistent results were obtained from the 6x12-in. (152x305-mm) cylinders and the results were discarded. The data reported in table 12 for BT12 are for concrete cores extracted from the deck.

A petrographic examination of one core from the webs of Girders BT7, BT8, and BT9 was made at a concrete age of approximately 550 days. The examination revealed that the concrete was non air-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

Table 10
Measured compressive strength, modulus of elasticity, and unit weight of girder concrete[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	11,220	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	5,000	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
BT11 Midspan	1	Match	4x8	10,490	6,050	149.5
	7	Match	4x8	11,440	5,400	149.0
	28	Match	4x8	12,910	5,600	149.8
	56	Match	4x8	12,970	5,600	149.0
	118	Match	4x8	13,580	5,800	149.3
	118	Core	3x6	12,770	5,800	150.1
	180	Match	4x8	13,780	5,700	149.0
	483	Match	4x8	14,520	5,600	150.1
BT12 Midspan	1	Match	4x8	10,790	6,150	150.3
	28	Match	4x8	13,230	5,850	149.8
	56	Match	4x8	14,000	5,800	149.5
	90	Match	4x8	13,900	5,850	149.4
	357	Match	4x8	14,580	5,800	150.3
	359	Core	3x6	12,690	5,950	151.8
	483	Match	4x8	14,920	5,600	149.4

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.

Table 11
Measured modulus of rupture of girder concrete

Test Specimen[a]	Test Age (days)	Modulus of Rupture (psi)	Comp. Strength[b] (psi)	$\frac{MOR}{\sqrt{f'_c}}$ [c]
BT6	131	1,065	9,290	11.05
BT7	252	1,045	13,050	9.15
BT8	369	1,080	11,850	9.92
BT11	28	1,120	12,910	9.86
	56	885[d]	12,970	7.77
	118	1,075	13,580	9.58
BT12	28	1,115	13,230	9.69
	56	980[d]	14,000	8.28
	357	1,225	14,580	10.15

a All modulus of rupture specimens were 6x6x20-in. beams cured alongside the girder until covers were removed and then stored in a similar environment as the girders.
b See Table 10 for curing condition and specimen size.
c MOR = modulus of rupture.
f'_c = compressive strength.
d Values appear to be low.

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

Concrete cores taken from the deck slabs of BT6, BT7, and BT8 before and after the fatigue test were also examined petrographically to assess whether or not the fatigue loading had caused any 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.

Table 12
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
BT11	Midspan	7	4,330	—
		28	5,550	—
		47	5,940	5,000
BT12	Midspan	72	6,340[b]	—
		252	7,810[b]	5,300[b]

A dash indicates that the property was not measured.
a Most values are the average of three specimens.
b Measured on 3x6-in. cores.

Reinforcement. Measured properties of the 0.6-in. (15.2-mm) diameter prestressing strand and the nonprestressed reinforcement are shown in tables 13 and 14, 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 a specific level of tensile stress that would be produced in an uncracked section. The fatigue load range was selected to produce a midspan bending moment range equal to that resulting from the design live load plus impact in the prototype bridge.

Table 13
Measured properties of 0.6-in. (15.2-mm) diameter prestressing strand[a]

Specimen	1% Elongation Load (lb)	Breaking Load (lb)	Total Elongation (%)	Modulus of Elasticity (ksi)
BT6, BT7, BT8	54,170	61,550	6.2	28,720
BT11, BT12	55,070	59,670	5.9	29,090

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

Table 14
Measured properties of nonprestressed reinforcement[a]

Specimen	Bar Size	Yield[b] Strength (psi)	Ultimate Strength (psi)	Elongation	Modulus of Elasticity (ksi)
BT6, BT7, BT8	No. 4	62,500	103,000	12.0	25,700
	No. 5	62,000	103,000	13.4	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 release was calculated from the average force measured by the load cells shortly after casting the girders and prior to initial set of the concrete.

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 concrete strain measured at the centroid of the strand group with the vibrating wire strain gauges.

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 measured 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 measurements from 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. These values represented the anticipated loads 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 15.

For Specimens BT6, BT7, and BT8, static load was intentionally applied to produce a crack near midspan prior to start of the fatigue test. For Specimen BT11, static load was intentionally applied to crack the girder during the static test after 5 million cycles of fatigue loading. For Specimen BT12, this was accomplished during the first part of the flexural strength test. These tests provided an opportunity to experimentally determine the loads

Table 15
Loads and stresses for fatigue tests

	Girder				
	BT6	BT7	BT8	BT11	BT12
Loads					
Girder Weight, lb/ft	858.4	877.4	861.5	842.7	852.6
Deck and Haunch Weight, lb/ft	1,011.6	1,036.8	1021.9	1,074.0	1,074.5
Supplementary Total Dead Load, kips/block	11.4	11.5[a]	11.5	11.5	11.5[a]
Max. Actuator Load, kips/ram	64.5	72.9	67.9	66.6	70.2
Min. Actuator Load, kips/ram	10.8	17.6	12.5	11.3	14.8
Calculated Decompression Load, kips/ram	38.3	35.6	35.4	40.5	37.9
Calculated Cracking Load, kips/ram	84.1	94.2[a]	82.0	87.3	103.8[a]
Measured Cracking Load, kips/ram	84.2	89.1[a]	74.0	87.5	101.6[a,b]
Calculated Girder Stresses,[c] psi					
Initial Prestress	3,119	3,116	3,096	3,199	3,240
Girder Weight	-772	-789	-774	-761	-772
Prestress Losses	-251	-280	-286	-236	-313
Deck and Haunch	-909	-932	-919	-969	-972
Supplementary Dead Load	-299	-300	-302	-301	-304
Max. Actuator Load	-1,498	-1,672	-1,565	-1,532	-1,629
Net at Maximum Stress	-610	-857	-750	-600	-750
Min. Actuator Load	-251	-403	-289	-259	-344
Net at Minimum Stress	637	412	526	673	535
Concrete Stress Range	1,247	1,269	1,276	1,273	1,285

a Supplementary dead loads were not in place when girder was cracked.
b Determined after the fatigue test.
c Calculated at the extreme bottom fiber of the girder at midspan. Compressive stresses are positive.

required to overcome the prestressing force and to crack the girder. These measured loads were then compared with the calculated values.

Measured loads corresponding to the first observed crack are included in table 15 for comparison with the calculated loads. The calculated loads are based on the measured prestress losses and values of the girder concrete modulus of rupture measured before the start of the fatigue test. For Specimens BT6, BT11, and BT12, 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 or 1.0 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 matched the target static load based on calculations. A photograph of the fatigue test setup is shown in figure 6.

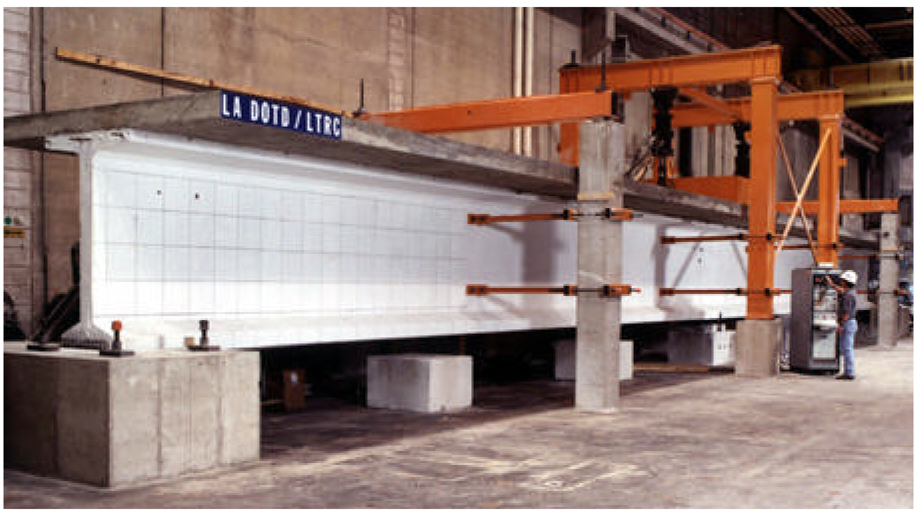


Figure 6
Fatigue test setup

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. 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.

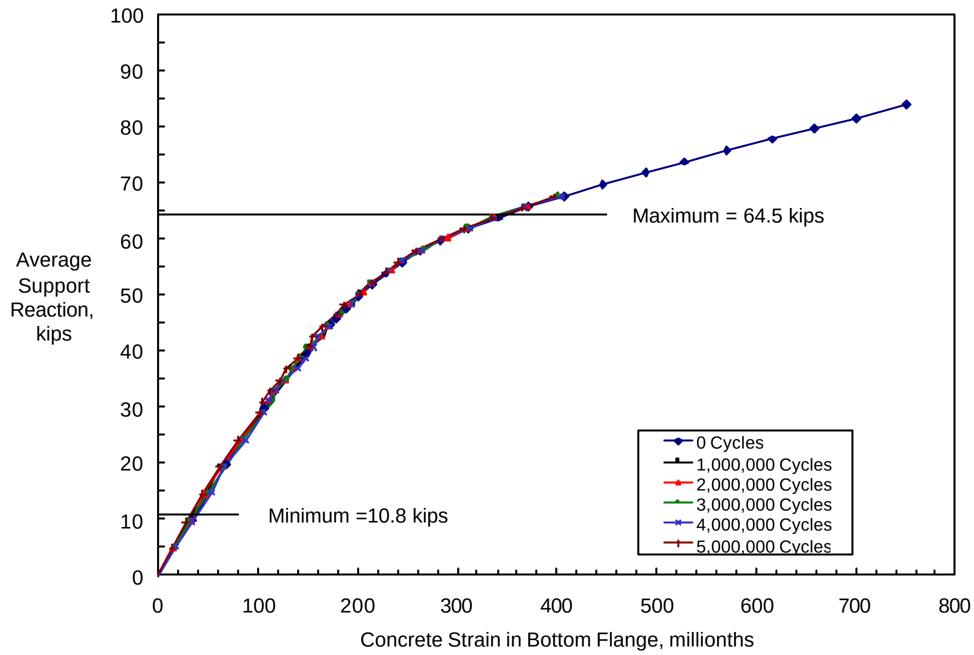


Figure 7
Support reaction versus concrete strain for fatigue test of Specimen BT6

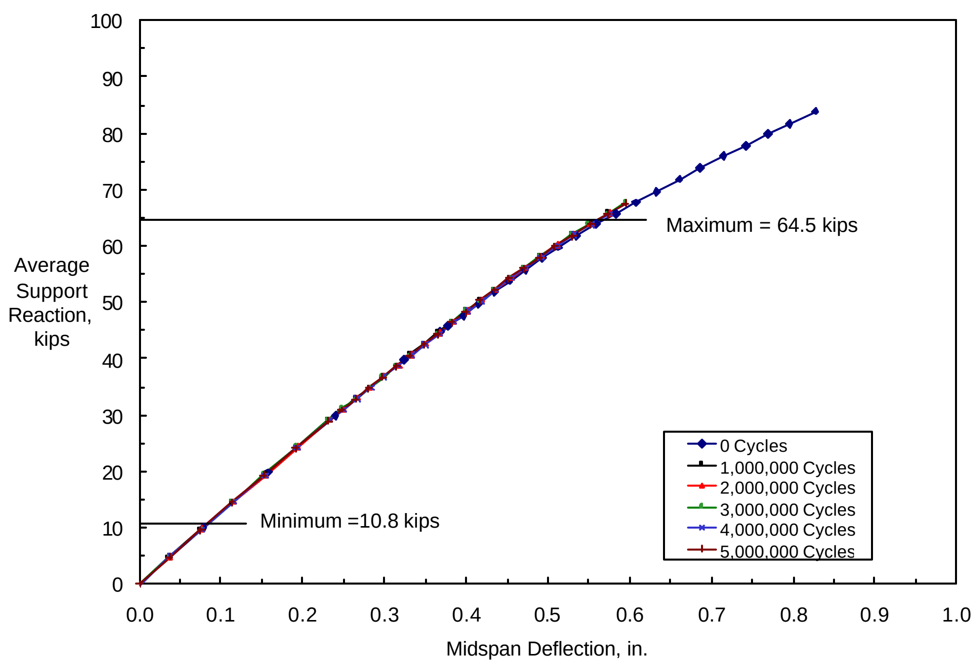


Figure 8
Support reaction versus midspan deflection for fatigue test of Specimen BT6

Therefore, Specimen BT6 withstood 5 million cycles of loading without any apparent wire or strand breakage.

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 strain range between minimum and maximum loads measured in the bottom row of strands during the initial static test was 315 millionths. This corresponds to a steel stress range 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. The fatigue 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 gauges. Nevertheless, the fatigue test was discontinued. The strain range between minimum and maximum loads measured in the bottom row of strands during the initial static test was 410 millionths. This corresponds to a steel 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 bottom flange was dissected over a length of 5 ft. (1.52 m) on either side of the midspan center line to check for fatigue fractures. Three of the 24 strands in the bottom flange had completely fractured and 8 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. The fatigue 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,

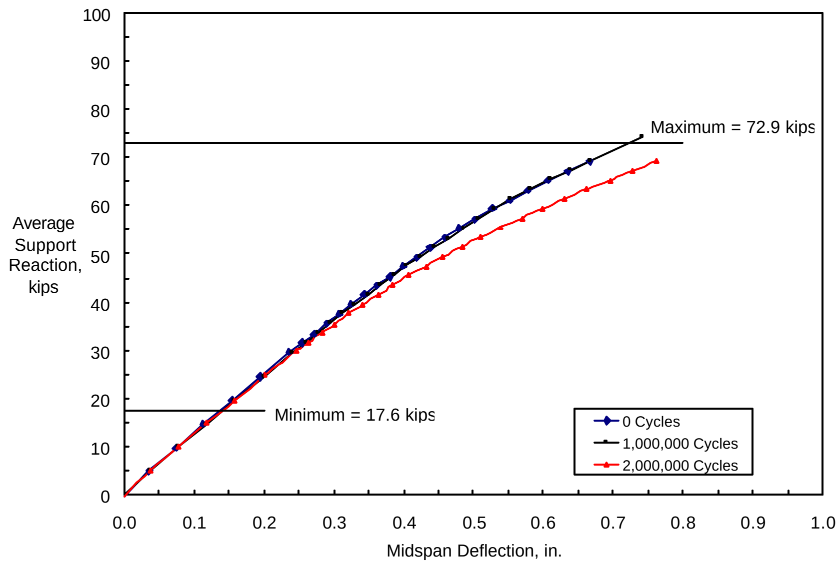
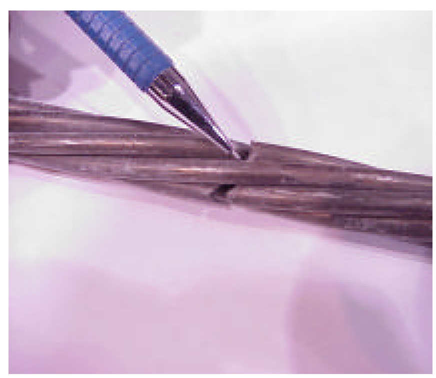
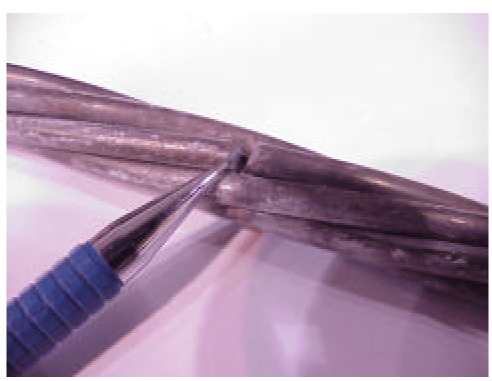


Figure 9
Support reaction versus midspan deflection for fatigue test of Specimen BT7

Strand break



Multiple wire break



Single wire break

Figure 10
Fatigue fractures

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 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 final static test. The strain range between the minimum and maximum loads measured in the bottom row of Specimen BT8 strands during the initial static test, was 335 millionths. This corresponds to a steel 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 bottom flange was dissected over a length of 5 ft. (1.52 m) on either side of the midspan centerline to check for fatigue fractures. One of the 24 strands in the bottom flange had all 7 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.

Specimen BT11. The fatigue loads applied to Specimen BT11 were based on a calculated maximum tensile stress in the bottom surface of the girder of 600 psi (4.14 MPa). The measured relationships between average support reaction and concrete strain in the bottom flange measured with the sister bar gauges are shown in figure 13 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. Similar behavior was observed in the midspan deflection shown in figure 14. Based on the data obtained during each static test, no adjustments to the dynamic loads were made throughout the fatigue test. It was, therefore, concluded that Specimen BT11 withstood 5 million cycles of loading without any apparent wire strand breakage.

Figure 13 also shows the minimum and maximum reaction of 11.3 and 66.6 kips (50.3 and 296.3 kN) used throughout the fatigue test. The strain range between minimum and maximum loads measured at the level of the bottom row of strands during the initial static test was 212 millionths. This corresponds to a steel stress range of 6,170 psi (42.5 MPa) based on a measured modulus of elasticity of 29,090 ksi (200.6 GPa). The low steel stress range in BT11 results from the girder remaining uncracked throughout the fatigue test.

Specimen BT12. The fatigue loads applied to Specimen BT12 were based on a calculated maximum tensile stress in the bottom surface of the girder of 750 psi (5.17 MPa). The measured relationships between average support reaction and concrete strain in the

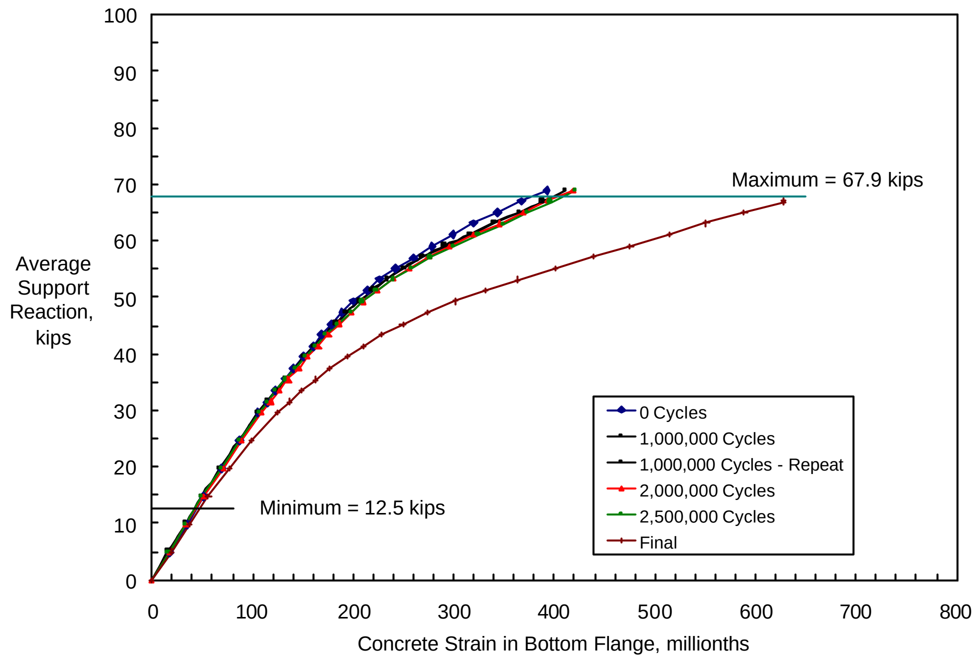


Figure 11
Support reaction versus concrete strain for fatigue test of Specimen BT8

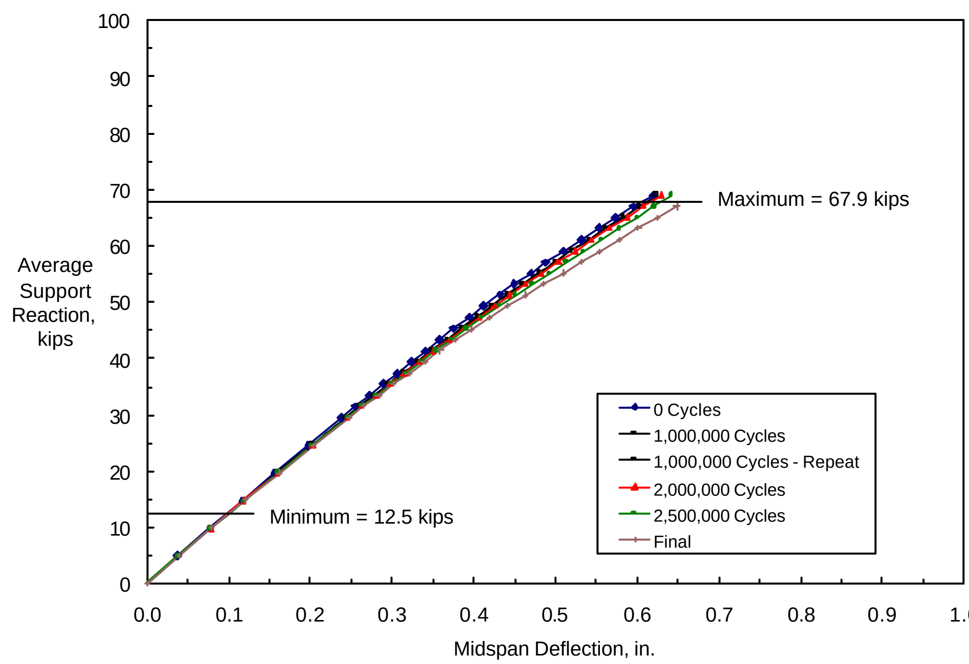


Figure 12
Support reaction versus midspan deflection for fatigue test of Specimen BT8

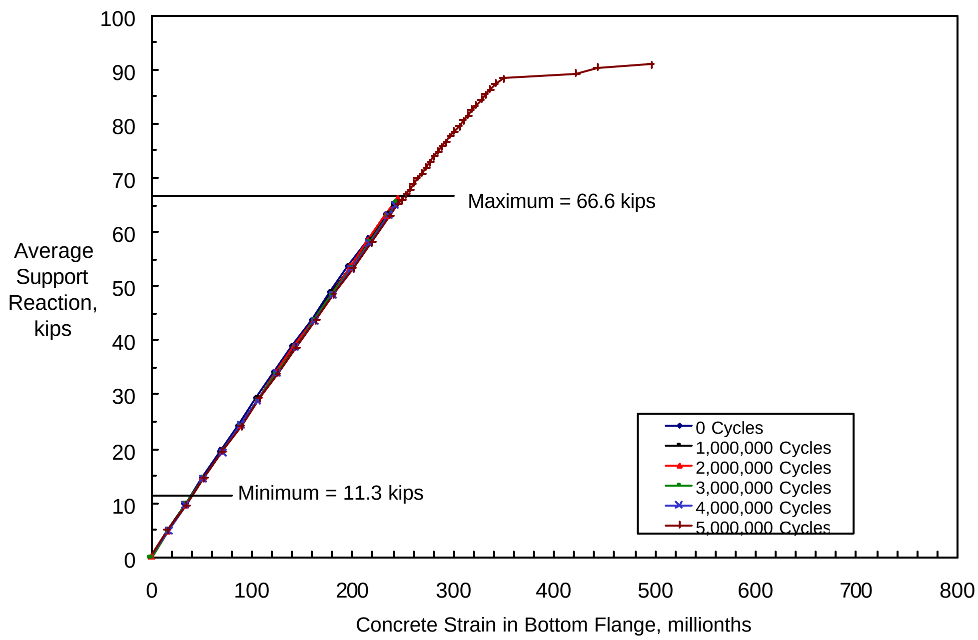


Figure 13
Support reaction versus concrete strain for fatigue test of Specimen BT11

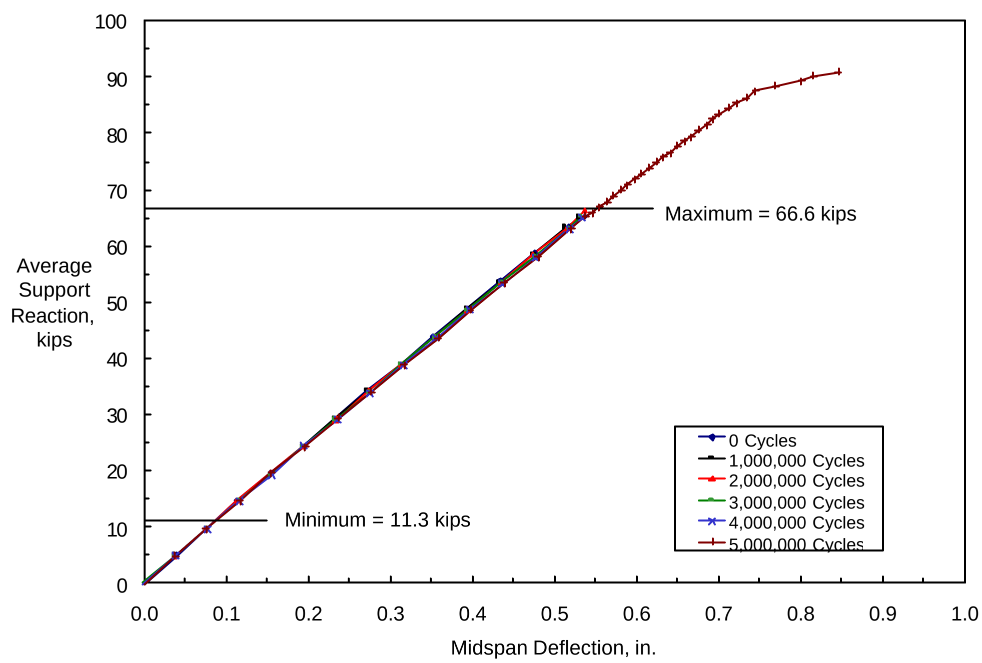


Figure 14
Support reaction versus midspan deflection for fatigue test of Specimen BT11

bottom flange measured with the sister bar gauges are shown in figure 15 for the static tests conducted before the fatigue test and after each 1 million cycles of fatigue loading. After the first 1 million cycles of fatigue loading, a slight change in stiffness was observed. After the static test at 1 million cycles, there was no further discernable change in the load versus strain relationship for each of the subsequent static tests. Similar behavior was observed in the midspan deflection shown in figure 16. Based on the data obtained during each static test, no adjustments to the dynamic loads were made throughout the fatigue test. It was, therefore, concluded that Specimen BT12 withstood 5 million cycles of loading without any apparent wire strand breakage.

Figure 15 also shows the minimum and maximum reactions of 14.8 and 70.2 kips (65.8 and 312.3 kN) used throughout the fatigue test. The strain range between minimum and maximum loads measured at the level of the bottom row of strands during the initial static test was 200 millionths. This corresponds to a steel stress range of 5,820 psi (41.1 MPa) based on a measured modulus of elasticity of 29,090 ksi (200.6 GPa). The low steel stress range in BT12 results from the girder remaining uncracked throughout the fatigue test. The measured steel stress ranges for BT11 and BT12 were similar because both specimens had similar material properties and similar section properties and were subjected to the same fatigue load range.

Summary of fatigue test results. Results of the five fatigue tests are summarized in table 16. 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.

A comparison of the results of BT6 with BT11 indicates that the higher rate of loading for BT6, which also had a higher strand stress range, did not appear to adversely affect the test results since both specimens survived 5 million cycles of fatigue loading. However, a comparison of BT8 and BT12 indicates that the higher rate of loading of BT8 may have contributed to the reduced number of loading cycles. Consequently, the results are inconclusive about the effect of loading rate on fatigue behavior.

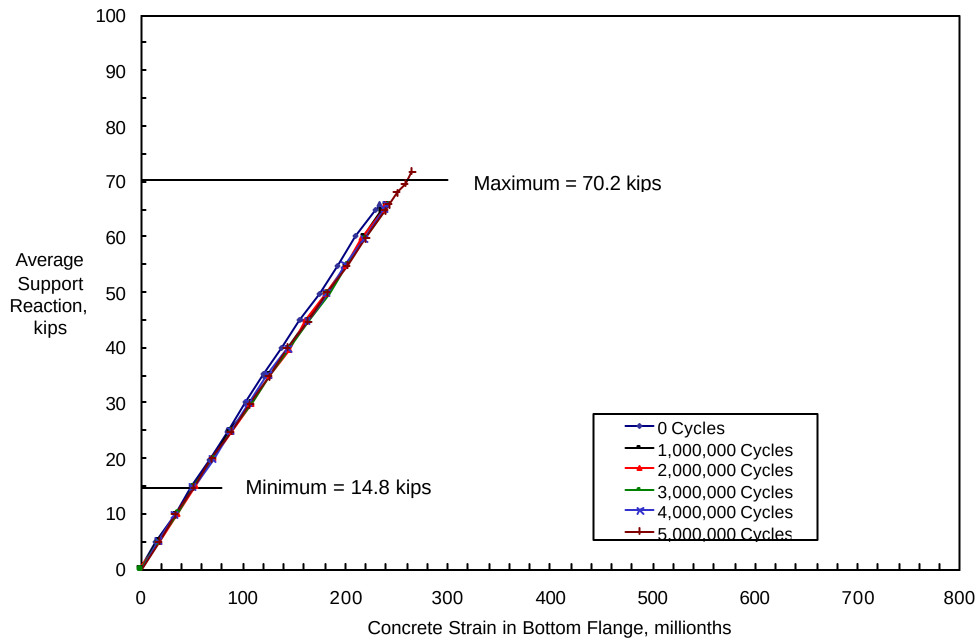


Figure 15
Support reaction versus concrete strain for fatigue test of Specimen BT12

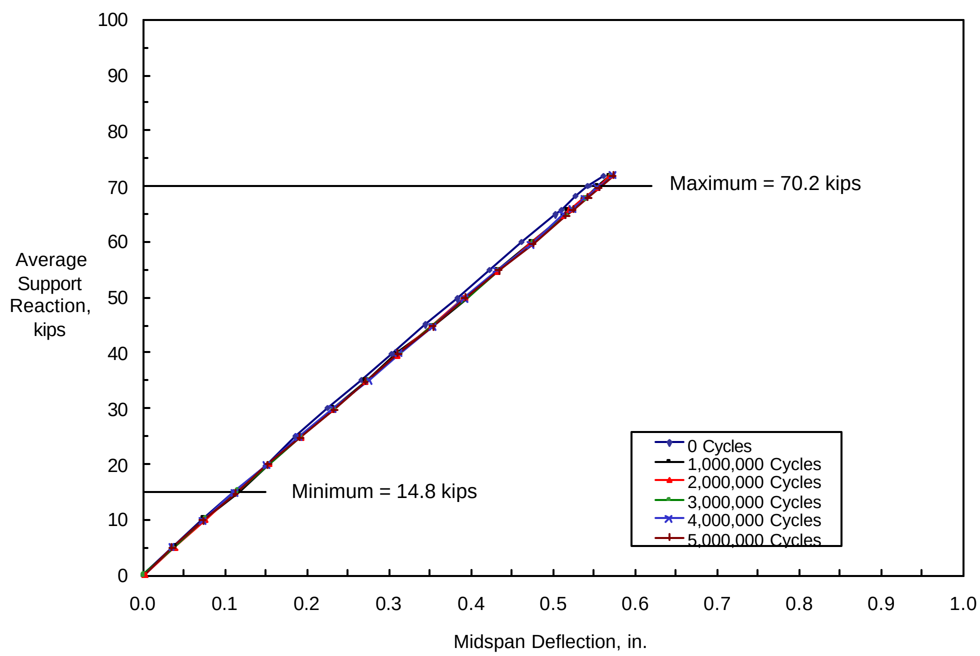


Figure 16
Support reaction versus midspan deflection for fatigue test of Specimen BT12

Table 16
Fatigue test results

Test Specimen	Loading Rate (Hertz)	Calculated Concrete Stresses (psi)		First Detected Wire Fracture	Loading Cycles Achieved	Strand Stress (psi)	
		Maximum	Range			Range	Minimum
BT6	1.9	610	1,247	None	5,000,000	9,050	167,000
BT7	1.9	857	1,269	1,600,000	1,910,000	11,780	165,000
BT8	1.9	750	1,276	2,250,000	2,500,000	9,620	162,000
BT11	1.0	600	1,273	None	5,000,000	6,170	169,000
BT12	1.0	750	1,285	None	5,000,000	5,820	167,000

Flexural strength test

A photograph of the flexural strength test setup of Specimen BT12 is shown in figure 17. The test was conducted by incrementally loading the specimen in two stages. In the first stage, the specimen was statically loaded until a flexural crack developed in the constant moment region. Additional load was then applied to ensure that a well-defined crack was developed across the full width of the bottom flange. The applied load was then decreased to

Figure 17
Flexural strength test setup

zero. Following the application of strain gauges to the bottom surface of the lower flange, the specimen was again statically loaded in increments until the maximum load was reached.

The measured relationship between total applied load and midspan deflection during the first stage of the test is shown in figure 18. The first flexural crack occurred at a total load of 203.3 kips (904.3 kN). This applied load, together with a self-weight of 1,927 lb/ft (28.12 kN/m), produced a total cracking moment of 6,239 kip-ft. (8.459 MN·m). The corresponding cracking moments calculated using the procedures of the *Standard Specifications* and the *LRFD Specifications* with the section and material properties used in design were 5,336 and 5,237 kip-ft. (7.235 and 7.100 MN·m), respectively. These values are approximately 15 percent less than the measured value. The calculated cracking moment based on the measured section dimensions and measured material properties with a modulus of rupture of 1,225 psi (8.45 MPa) was 6,324 kip-ft. (8.574 MN·m) showing very good agreement with the measured value. Most of the difference between the cracking moment calculated using the design properties and the measured properties can be attributed to the measured modulus of rupture being higher than the design values.

The measured relationship between total applied load and midspan deflection during the second stage of the test is shown in figure 19. As additional load beyond that required to open the first flexural crack was applied, more flexural cracks formed in the constant moment region followed by flexural cracks outside the constant moment region. These cracks developed into inclined flexural-shear cracks. Finally, inclined web-shear cracks developed near the ends of the specimen. A composite picture of the crack patterns is shown in figure 20.

During the test, it was necessary to restroke the hydraulic rams three times as noted in figure 19. At a deflection of about 28-3/4 in. (730 mm) and a total applied load of 349.8 kips (1.556 MN), the first strands fractured and the load resistance of the specimen began to decrease. Successive strand fractures occurred until all strands were broken and the test stopped. The maximum total applied load was 350.0 kips (1.557 MN), which was the load prior to the third restroking of the rams. A photograph of the specimen just prior to the end of the test is shown in figure 21.

The maximum applied load of 350.0 kips (1.556 MN) together with a self-weight of 1,927 lb/ft. (28.12 kN/m) produced a bending moment of 9,170 kip-ft. (12.43 MN·m). The flexural strength of the midspan cross section calculated using the procedures of the *Standard Specifications* and the *LRFD Specifications* with both design material properties and

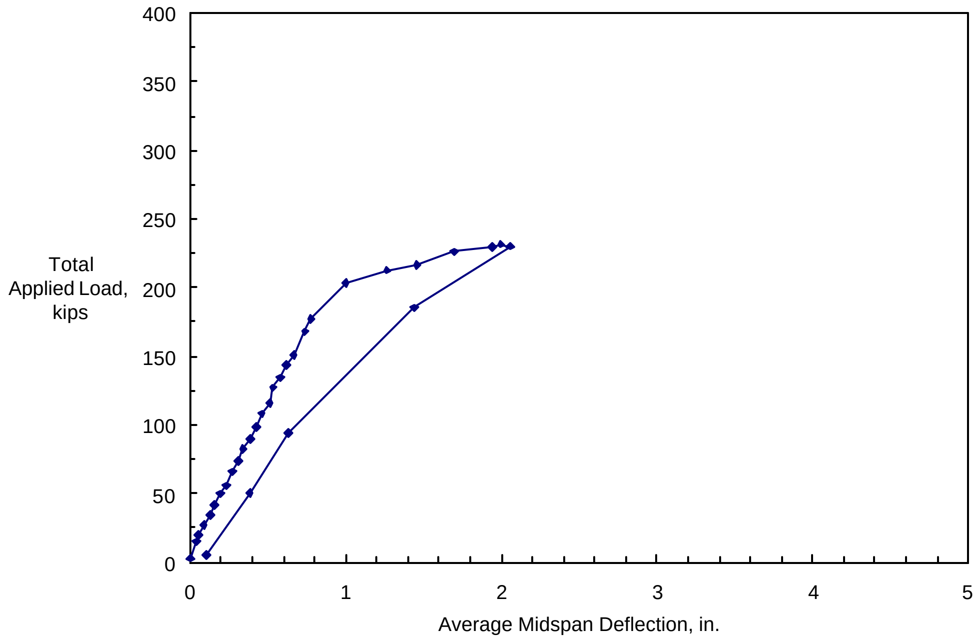


Figure 18
Total applied load versus midspan deflection for the first stage of the flexural strength test of Specimen BT12

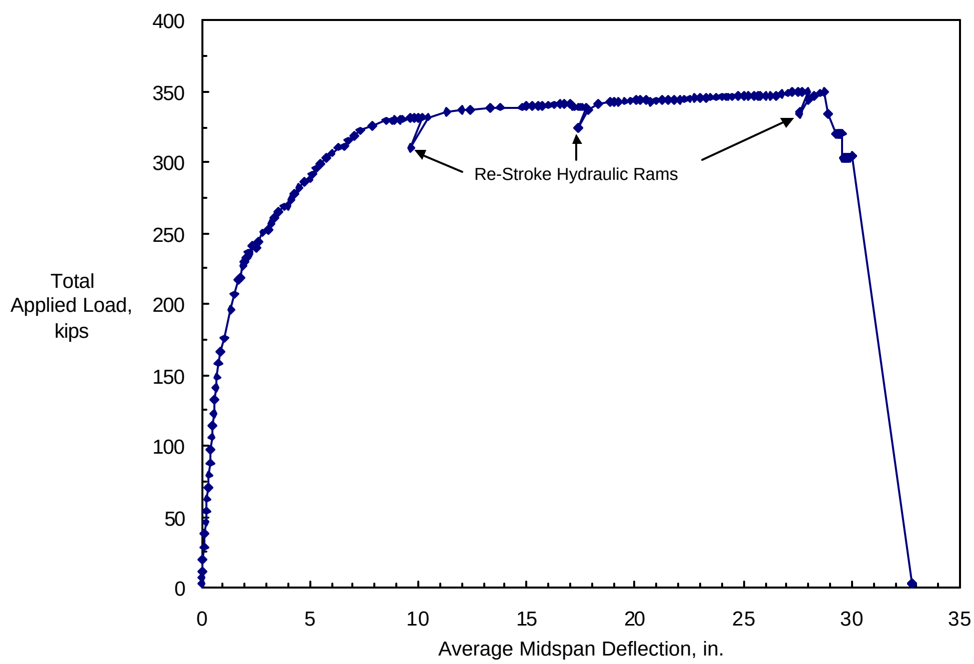


Figure 19
Total applied load versus midspan deflection for the second stage of the flexural strength test of Specimen BT12

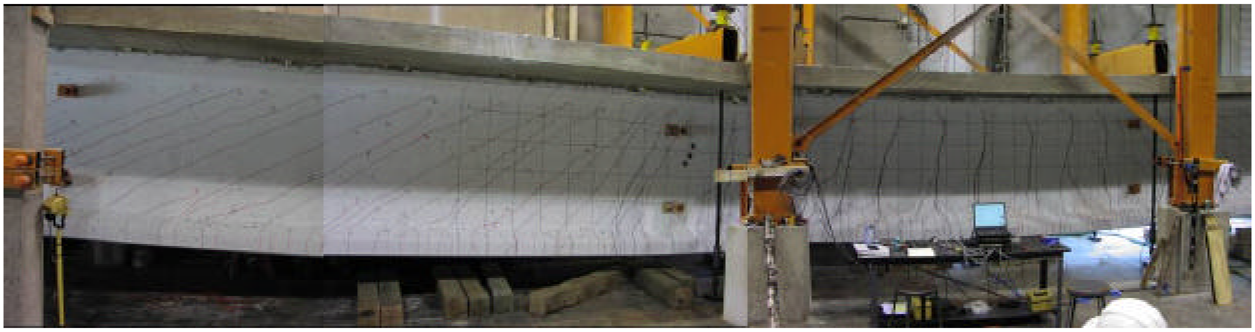


Figure 20
Crack patterns in Specimen BT12

Figure 21
Specimen BT12 prior to completion of the flexural strength test

measured material properties are shown in table 17. All calculated values are about the same since the procedures in both specifications are very similar for a rectangular section and the average measured strength of the prestressing strand of 275,000 psi (1.896 GPa) was only slightly greater than the design value of 270,000 psi (1.862 GPa). In calculating the flexural strength, the stress in all strands was assumed to be 275,000 psi (1.896 GPa). For all combinations, the measured flexural strength was slightly greater than the calculated strengths.

Table 17
Flexural strengths of Specimen BT12

Specifications	Material Properties	
	Design[a]	Measured
Standard	9,000 kip-ft	9,100 kip-ft
LRFD	8,940 kip-ft	9,100 kip-ft
Measured	9,170 kip-ft	

a. Calculations made using CONSPAN programs.

After the ultimate strength test, the bottom flange of BT12 was dissected over a length of 1 to 2 ft. (300 to 600 mm) either side of the main flexural crack to inspect the strand fractures. No fatigue fractures were detected. Individual wires of each strand exhibited ductile fractures.

Shear tests

The shear tests were conducted by incrementally loading each specimen until it 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 22. The first diagonal crack in each specimen occurred at an applied shear that ranged from 270 to 302 kips (1.20 to 1.34 MN) as reported in table 18. Applied shear is the shear force produced in the test region from the

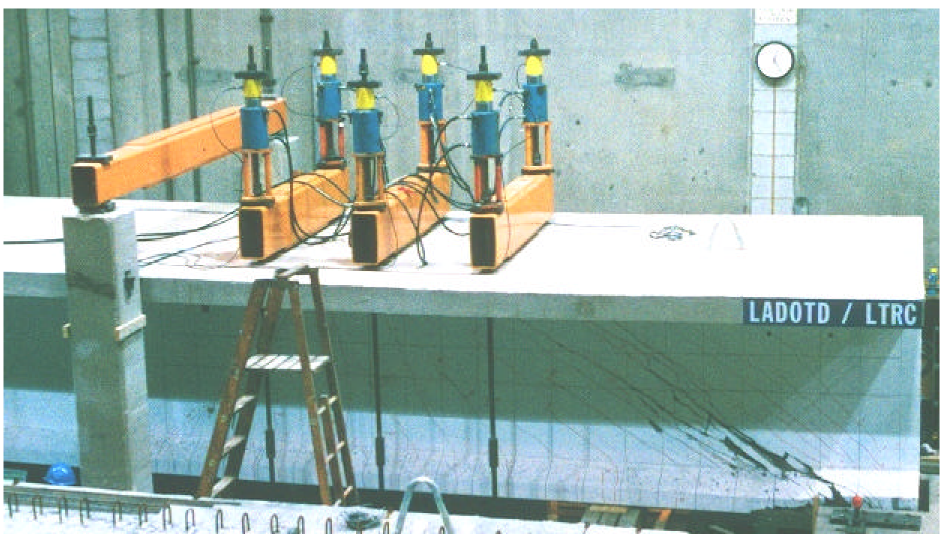


Figure 22
Shear test setup

Table 18
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.

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. A detailed description of the shear tests of each specimen, and analysis of the results, using the procedures of the *Standard Specifications* and the *LRFD Specifications*, is provided in the Interim Report *[9]*.

Conclusions from the shear tests. The following conclusions were developed in the Interim Report and are based on the results of the six shear tests conducted during this project *[9]*:

- 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 and the Interim Report *[9]*:

- A 72-in. (1.83-m) deep prestressed concrete bulb-tee girder made with 10,000 psi (69 MPa) compressive strength concrete and cracked in flexure before fatigue testing endured 5 million cycles of flexural fatigue loading when the maximum concrete tensile stress used in the test specimen design was limited to a maximum value of 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) and cracked in flexure before fatigue testing endured 2.5 and 1.9 million cycles of flexural fatigue loading when the maximum concrete tensile stresses used in the test specimen design were limited to maximum values of 750 and 857 psi, (5.17 and 5.91 MPa), respectively.

- Two 72-in. (1.83-m) deep prestressed concrete bulb-tee girders with concrete compressive strengths greater than 10,000 psi (69 MPa) and uncracked before fatigue testing endured 5 million cycles of flexural fatigue loading when the maximum concrete tensile stresses used in the test specimen design were limited to maximum values of 600 psi (4.14 MPa) in one girder and 750 psi (5.17 MPa) in the other girder.

- A 72-in. (1.83-m) deep prestressed concrete bulb-tee girder made with a concrete having a compressive strength greater than 10,000 psi (69 MPa) had a measured flexural strength after being subjected to 5 million cycles of fatigue loading that was greater than the flexural strength calculated using the procedures of the *AASHTO Standard Specifications* and the *AASHTO LRFD Specifications* when either specified or measured material properties were used.

- The fatigue test results are inconclusive about the effect of loading rate on fatigue behavior.

- 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:

- The maximum level of concrete tensile stress used in flexural design of high-strength prestressed concrete girders should be limited to $6\sqrt{f_c'}$ when cracking of the girders is anticipated and $7.5\sqrt{f_c'}$ when cracking is not anticipated. The cracks may occur as flexural cracks or temperature cracks. Flexural cracks occur as a result of bending moments applied to the beam. The cracks extend from the tension face of the member towards the compression zone and are clearly visible to the naked eye. Vertical temperature cracks can occur during the fabrication process and subsequently close and become invisible when the strands are released.

- 72-in. (1.83-m) deep prestressed concrete bulb-tee girders made with 10,000 psi (69 MPa) compressive strength concrete and cracked in flexure can be expected to 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 and uncracked can be expected to perform satisfactorily under flexural fatigue when the concrete design tensile stress has a maximum value of $7.5\sqrt{f_c'}$ = 750 psi (5.17 MPa).

- 72-in. (1.83-m) deep prestressed concrete bulb-tee girders made with 10,000 psi (69 MPa) compressive strength concrete can be expected to perform satisfactorily under static flexural loading conditions when designed by either the *AASHTO Standard Specifications* or the *AASHTO LRFD Specifications*. Therefore, both design approaches may be used by LADOTD.

- 72-in. (1.83-m) deep prestressed concrete bulb-tee girders made with 10,000 psi (69 MPa) compressive strength concrete can be expected to 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*. Therefore, both design approaches may be used by LADOTD.

- 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.

- In order to achieve a larger data base, additional research is needed to determine the effect of loading rate on fatigue performance. The additional testing could utilize smaller specimens.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO = American Association of State Highway and Transportation Officials
A_{ps} = area of prestressing steel
ASTM = American Society for Testing and Materials
BT = bulb-tee
° 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
° F = degree Fahrenheit
fl oz = fluid ounce
ft. = foot
f'_c = concrete compressive strength
GCP = Gulf Coast Pre-Stress, Inc.
GGBFS = ground granulated blast-furnace slag
GPa = gigapascal
in. = inch
kg = kilogram
kN = kilonewton
ksi = kip per square inch
LADOTD = Louisiana Department of Transportation and Development
L = liter
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
mm = millimeter
No. = number
P = concentrated load or prestressing force

Petro	=	petrographic examination
pp	=	pages
psi	=	pound per square inch
sq	=	square
w	=	unit weight of concrete
yd	=	yard
a	=	coefficient of thermal expansion
@	=	at

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.

9. Bruce, R. N.; Russell, H. G.; and Roller, J. J., “Fatigue and Shear Behavior of HPC Bulb-Tee Girders,” Interim Report - Louisiana Transportation Research Center, Research Report No. FHWA/LA. 03/382, Baton Rouge, LA, 2003, 58 pp.