TECHNICAL REPORT STANDARD PAGE

1. Report No. FHWA/LA-FHWA/LA-94-282	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Feasibility Evaluation of Utilizing High-Strength Concrete in Design and Construction of Highway Bridge Structures	5. Report Date January 1994 6. Performing Organization Code	
7. Author(s) R.N. Bruce, H.G. Russell, J.J. Roller, B.T. Martin	8. Performing Organization Report No. 282	
9. Performing Organization Name and Address Tulane University New Orleans, LA	10. Work Unit No. 11. Contract or Grant No. LA HPR Study No. 90-4C	
12. Sponsoring Agency Name and Address Louisiana Transportation Research Center 4101 Gourrier Avenue Baton Rouge, LA 70808	13. Type of Report and Period Covered Final Report - October 1990 - December 1993 14. Sponsoring Agency Code	

15. Supplementary Notes
Conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. Abstract

The objective of this investigation was to evaluate the feasibility of using high-strength concrete in the design and construction of highway bridge structures. A literature search was conducted; a survey of five regional fabrication plants was performed; concrete mix designs were studied in the laboratory and in the field; tests of nine full-scale specimens were conducted.

Three concentrically prestressed pile specimens were fabricated and tested in flexure as part of this program. Each of the pile specimens had a 24-in. (610-mm) square cross section with a concentric 12-in. (305-mm) diameter void. All pile specimens were 24-ft. (7.3-m) long. These three pile specimens had a design 28-day concrete compressive strength of 8,500 psi (58.6 MPa).

Four full-size prestressed bulb-tee girders were fabricated and tested as part of this program. Early-age flexure and shear strength tests were conducted on two bulb-tee flexural strength test. The four specimen was subjected to a fatigue test followed by a flexural strength test.

The specimens were 54-in. (1.37-m) deep bulb-tee sections with a 6-in. (152-mm) thick web. Each girder specimen was 70 ft. (21.3-m) long. Each bulb-tee specimen had a design 28-day concrete compressive strength of 10,000 psi (69.0 MPa). Prior to testing, three of the four bulb-tee specimens had a 9-1/2 in. (240-mm) thick and 10 ft. (3.05-m) wide deck slab added. One bulb-tee specimen used for early age testing did not have a deck.

Fabrication and driving of a single 130-ft (39.6-m) long prestressed pile specimen was also included as part of this program. The pile specimen had the same cross-sectional configuration and design 28-day concrete compressive strength as the first three pile specimens tested in flexure. Performance of this pile specimen during handling and driving was evaluated in the field.

The high-strength concrete piles and girders tested as part of this research program performed adequately with respect to both design requirements and the requirements of the AASHTO Standard Specifications for Highway Bridges. It is concluded that the provisions of the AASHTO Standard are conservatively applicable for members with concrete compressive strengths up to 10,000 psi (69.0 MPa). Results of this program clearly demonstrated several potential benefits of utilizing high-strength concrete for highway bridge structures including (1) larger girder spacing, (2) longer pile lengths, and (3) lower prestress losses. Therefore, use of high-strength concrete for highway bridge structures is recommended. However, based on the difficulties encountered during this project with respect to achieving 10,000 psi (69.0 MPa) compressive strength concrete, it appears that precast fabricators will need to make some changes to the normal production regime in order to consistently produce high-strength concrete.

17. Key Words high strength concrete - full size prestressed bulb-tee girder testing pile driving		18. Distribution Statement Unrestricted. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 168	22. Price

Form DOT F1700.7 (1-92)

FEASIBILITY EVALUATION OF UTILIZING HIGH-STRENGTH CONCRETE IN DESIGN AND CONSTRUCTION OF HIGHWAY BRIDGE STRUCTURES

FINAL REPORT

by

R. N. BRUCE
CATHERINE AND HENRY BOH CHAIR IN CIVIL ENGINEERING,
TULANE UNIVERSITY

H. G. RUSSELL
VICE-PRESIDENT, CONSTRUCTION TECHNOLOGY LABORATORIES, INC.

J. J. ROLLER
SENIOR ENGINEER, CONSTRUCTION TECHNOLOGY LABORATORIES, INC.

B. T. MARTIN
BOH FELLOW, TULANE UNIVERSITY

CONDUCTED FOR

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
LOUISIANA TRANSPORTATION RESEARCH CENTER
in cooperation with
U. S. Department of Transportation
FEDERAL HIGHWAY ADMINISTRATION

JANUARY 1, 1994

ABSTRACT

The objective of this investigation was to evaluate the feasibility of using high-strength concrete in the design and construction of highway bridge structures. A literature search was conducted; a survey of five regional fabrication plants was performed; concrete mix designs were studied in the laboratory and in the field; tests of nine full-scale specimens were conducted.

Three concentrically prestressed pile specimens were fabricated and tested in flexure as part of this program. Each of the pile specimens had a 24-in. (610-mm) square cross section with a concentric 12-in. (305-mm) diameter void. All pile specimens were 24-ft (7.3-m) long. These three pile specimens had a design 28-day concrete compressive strength of 8,500 psi (58.6 MPa).

Four full-size prestressed bulb-tee girders were fabricated and tested as part of this program. Early-age flexure and shear strength tests were conducted on two bulb-tee specimens. The third specimen was used for evaluation of long-term static-load behavior followed by a late-age flexural strength test. The fourth specimen was subjected to a fatigue test followed by a flexural strength test.

The specimens were 54-in. (1.37-m) deep bulb-tee sections with a 6-in. (152-mm) thick web. Each girder specimen was 70-ft (21.3-m) long. Each bulb-tee specimen had a design 28-day concrete compressive strength of 10,000 psi (69.0 MPa). Prior to testing, three of the four bulb-tee specimens had a 9-1/2-in. (240-mm) thick and 10-ft (3.05-m) wide deck slab added. One bulb-tee specimen used for early age testing did not have a deck.

Fabrication and driving of a single 130-ft (39.6-m) long prestressed pile specimen was also included as part of this program. The pile specimen had the same cross-sectional

configuration and design 28-day concrete compressive strength as the first three pile specimens tested in flexure. Performance of this pile specimen during handling and driving was evaluated in the field.

The high-strength concrete piles and girders tested as part of this research program performed adequately with respect to both design requirements and the requirements of the AASHTO Standard Specifications for Highway Bridges. It is concluded that the provisions of the AASHTO standard are conservatively applicable for members with concrete compressive strengths up to 10,000 psi (69.0 MPa). Results of this program clearly demonstrated several potential benefits of utilizing high-strength concrete for highway bridge structures including (1) larger girder spacing, (2) longer pile lengths, and (3) lower prestress losses. Therefore, use of high-strength concrete for highway bridge structures is recommended. However, based on the difficulties encountered during this project with respect to achieving 10,000 psi (69.0 MPa) compressive strength concrete, it appears that precast fabricators will need to make some changes to the normal production regime in order to consistently produce high-strength concrete.

IMPLEMENTATION STATEMENT

The overall research constituted a comprehensive examination of high-strength concrete as a material and its use in structural elements. The scope of the overall project included the following:

1. A survey of regional fabrication plant capabilities.
2. A determination of the suitability of local materials.
3. A study of mix designs in the laboratory and in the field.
4. The fabrication of full-size structural members in the field.
5. The testing of full-size structural members.
6. The analysis of full-size structural members with respect to production feasibility, strength and behavior, and compliance with design codes.

A survey of regional plant capabilities indicated that there were at least two regional plants, both in Alabama, that are capable of immediate implementation of the fabrication of high-strength concrete structural members having a 28-day compressive strength in the range of 9,000 to 10,000 psi (62.1 to 69.0 MPa). It was concluded that high-strength concrete in this range of compressive strength can be consistently produced using local materials.

A study of mix designs in the laboratory indicated that 28-day compressive strengths in excess of 10,000 psi (69.0 MPa) can be achieved consistently using local materials. These mix designs can be duplicated in the field while maintaining a 28-day compressive strength in the range of 9,000 psi (62.1 MPa) to 10,000 psi (69.0 MPa).

The fabrication of full-size structural members, having a nominal 28-day compressive strength in the range of 9,000 psi (62.1 MPa) to 10,000 psi (69.0 MPa), can be implemented immediately in at least two regional fabrication facilities. In order to achieve this level of compressive strength, a high level of quality control is demanded.

The piles tested in the laboratory behaved in a manner that would be conservatively predicted using the provisions of the AASHTO Standard Specifications for Highway Bridges. The field driving test of a single 130-ft (39.6-m) long pile demonstrated that high-strength concrete can be used effectively in long piles. The 130-ft (39.6-m) pile performed well during transportation, handling, and driving. The high-strength concrete and corresponding high level of precompression were particularly valuable in soft driving conditions where tensile driving stresses are high. Normally, a single test should not form the basis for a blanket recommendation. However, it is the opinion of the project team that this single test went so well that immediate implementation of the utilization of high-strength concrete in long piles should be considered. Such implementation could increase the resistance of the pile to tensile driving stresses and offer the added advantage of serving to eliminate pile splices in long piles.

The results from static and fatigue tests conducted on girder specimens indicated adequate performance with respect to provisions of the AASHTO standard. Long-term tests of concrete cylinders and a corresponding girder specimen indicated creep and shrinkage losses that were substantially less than those predicted by the AASHTO standard. Measured camber in the girder used for long-term loading correlated well with deflections calculated using conventional procedures. There are strong indications of potential economic benefits of high-strength concrete in highway bridge girders. Indications are that a wider transverse spacing of the girders may be used effectively and that the wider spacing can reduce the number of girders. Increased material costs for high-strength concrete can be compared and balanced against the advantages of longer spans, fewer girders in a typical bridge cross section, decreased weight with corresponding foundation efficiency, and increased durability. It is the opinion of the project team that immediate utilization of high-strength concrete in bridge girders can be implemented.

Concrete with 28-day compressive strengths up to 10,000 psi (69.0 MPa) should be considered for use by the Louisiana DOTD. Structural members of 10,000 psi (69.0 MPa) concrete can be designed conservatively using the AASHTO Standard Specifications. Based on findings from this investigation, utilization of high-strength concrete for highway bridge structures is recommended.

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

LIST OF TABLES

LIST OF TABLES (cont'd.)

LIST OF TABLES (cont'd.)

LIST OF FIGURES

LIST OF FIGURES (cont'd.)

1. INTRODUCTION

1.1 OBJECTIVE AND SCOPE

The objective of this investigation is to evaluate the feasibility of using high-strength concrete in the design and construction of highway bridge structures. This investigation includes a literature search, a survey of regional fabrication plant capabilities, a study of mix designs in the laboratory and in the field, the fabrication and testing of full size high-strength concrete test specimens, and the analysis of those specimens with respect to production feasibility, strength, and compliance with design codes. On the basis of the findings of this investigation, recommendations are made regarding the use of high-strength concrete in the design and construction of highway bridge structures. While mechanical properties of high-strength concrete are addressed as part of the investigation, the major emphasis of the research is the determination of the behavior of structural elements made of high-strength concrete.

1.2 OUTLINE OF INVESTIGATION

This investigation began with a literature search concerning high-strength concrete. This selective literature search was conducted focusing primarily on recent research performed in Louisiana, Texas, North Carolina and by the Portland Cement Association/Construction Technology Laboratories, Inc. (CTL). A detailed examination of the Standard Specifications of the American Association of State Highway Transportation Officials (1) and the Building Requirements for Reinforced Concrete of the American Concrete Institute (2) was also performed to determine compliance with design requirements.

Concurrent with the literature search, a survey of five regional fabrication plants was conducted to ascertain plant capabilities with respect to fabrication of high-strength concrete members. Using locally available materials, high-strength concrete mix designs were developed for use in the production of test specimens. Input from fabricators was encouraged at all stages of the project in an effort to familiarize the fabricator with the special requirements of high-strength concrete. Nine high-strength concrete test specimens were produced for this research program. The high-strength concrete test specimens were fabricated at three different precast plants using two different fabricators. Eight of the nine test specimens were then shipped to Construction Technology Laboratories, Inc. (CTL) in Skokie, Illinois, for testing. The remaining specimen was shipped to a job site and incorporated in a bridge structure.

The test program involved four separate series of tests. The first series included flexural strength tests of three 24-ft (7.3-m) long concentrically prestressed pile specimens. Each pile specimen had a 24-in. (610-mm) square cross-section with a concentric 12-in. (305-mm) diameter void. These three pile specimens had a design 28-day compressive strength of 8,500 psi (58.6 MPa). Details of the fabrication and testing of the pile specimens are discussed in Section 5.

The second series of the program included tests of three 70-ft (21.3-m) long, 54-in. (1.37-m) deep prestressed bulb-tee girder specimens. These three girder specimens had a design 28-day compressive strength of 10,000 psi (69.0 MPa). Early-age flexure and shear strength tests were conducted on two of the three bulb-tee specimens. The third specimen was used for evaluation of long-term static-load behavior followed by a late-age flexural strength test. Prior to testing, one of the two bulb-tee specimens used for early-age testing and the specimen used for long-term evaluation had a 9-1/2-in. (240-mm) thick and 10-ft (3.05-m) wide deck slab added. Details of the fabrication and testing of the girder specimens are discussed in Section 6.

The third series of the test program involved the fabrication and field driving of a 130-ft (39.6-m) long, concentrically prestressed pile. This pile had the same cross-sectional configuration and design 28-day compressive strength as the first three pile specimens. Details of the pile fabrication and field testing are discussed in Section 7.0.

The fourth series of tests was intended to include fabrication and fatigue testing of two full-size bulb-tee specimens with details identical to those of the first three girder specimens. However, the concrete for one of these two girders did not achieve an acceptable 28-day compressive strength and was discarded. The remaining girder had a 9-1/2-in. (240-mm) thick and 10-ft (3.05-m) wide deck slab added and was subsequently subjected to a fatigue test. Fabrication details for the two girders and fatigue testing details for one girder are discussed in Section 8.

1.3 ACKNOWLEDGEMENTS

The work on this project was conducted jointly by the Tulane University Department of Civil and Environmental Engineering and Construction Technology Laboratories, Inc., under sponsorship of the Louisiana Transportation Research Center, and in cooperation with the Louisiana Department of Transportation and Development and the Federal Highway Administration.

On the part of the Louisiana Transportation Research Center, the work was performed under the administrative direction of Mr. Harold R. Paul, Materials Research Administrator; and Mr. Masood Rasoulian, Materials Research Engineer Supervisor. Mr. Norval P. Knapp, Bridge Design Engineer, and Mr. Paul Fossier, Senior Design Engineer, of the Louisiana Department of Transportation and Development, contributed greatly to the investigation.

Appreciation is expressed to the following individuals who have served on the Project Advisory Committee established for this project by the Principal Investigator and have contributed technical background information and data to this investigation:

Dr. Ned Burns	University of Texas at Austin
Mr. Sherrell Helm	Gulf South Prestressed Concrete Association
Mr. Norval Knapp	Louisiana Department of Transportation and Development
Dr. John Kulicki	Modjeski and Masters Consulting Engineers
Mr. Harold Paul	Louisiana Transportation Research Center
Mr. Ken Rear	W. R. Grace & Co.
Dr. Paul Zia	North Carolina State University

On the part of Tulane University, the investigation was directed by Dr. Robert N. Bruce, Jr., Catherine and Henry Boh Chair in Civil Engineering, as Principal Researcher and Mr. Barney T. Martin, Jr., Boh Fellow in Civil Engineering.

On the part of Construction Technology Laboratories, Inc., the investigation was directed by Dr. Henry G. Russell, Vice-President, and Mr. John J. Roller, Senior Engineer.

On the part of Sherman Prestressed Concrete, the fabrication was directed by Mr. Jim Glass, Prestressed Concrete Sales Engineer.

1.4 NOTATION

a = length of the shear span, in.

A_s = area of prestressing steel in tension zone, in.^{2}

b = width of compression face of member, in.

b_w = width of the web, in.

d = distance from the extreme compressive fiber of a member to the centroid of the prestressing strand, in.

d_b = nominal diameter of prestressing strand, in.

E_c = modulus of elasticity of the concrete, psi

f'_c = compressive strength of the concrete, psi

f_d = stress at extreme fiber of specimen due to self weight, psi

f_{pc} = compressive stress in concrete at centroid of section, psi

f_{pe} = stress at extreme bottom fiber of specimen due to effective prestressing force, psi

f_r = modulus of rupture of the concrete, psi

f_{se} = effective stress in prestressing strand after losses, ksi

f_{su} = average stress in prestressing steel at ultimate load, psi

I = moment of inertia of the specimen, in.^{4}

M_{DL} = dead load moment, in.-lb

M_{cr} = bending moment at flexural cracking due to total applied load, in.-lb

M_{max} = maximum moment at section under consideration, in.-lb

M_n = nominal flexural strength of a section, in.-lb

P = total applied load, not including member self-weight, lb

V_{ci} = flexural shear strength provided by the concrete, lb

V_{cw} = shear strength provided by the concrete, lb

V_d = dead load shear at the section under consideration, lb

V_i = shear force at section under consideration, occurring simultaneously with M_{max}, lb

V_n = nominal shear strength of a section, lb

V_p = vertical component of effective prestress force at section, lb

V_s = shear strength provided by the reinforcement, lb

Y_t = distance from the centroid of the section to the bottom fiber of the specimen, in.

w_c = unit weight of concrete, lb/ft^3

ϕ = strength reduction factor. A value of 1 was used for experimental investigations

ρ = $\frac{A_s}{bd}$ = ratio of prestressing steel

2. REVIEW OF PREVIOUS INVESTIGATIONS OF HIGH-STRENGTH CONCRETE

2.1 INTRODUCTORY REMARKS

This section contains a brief summary of selected recent experimental research regarding high-strength concrete and the use of high-strength concrete in prestressed highway structures. A description and the results of each investigation are summarized. Only recent investigations providing the background necessary for this study are reported.

2.2 RESEARCH AT THE LOUISIANA TRANSPORTATION RESEARCH CENTER

To date there have been two research efforts performed by, or in cooperation with, the Louisiana Transportation Research Center. The first, performed by Law & Rasoulian (3), concluded that concrete strengths of 6,500 psi (44.8 MPa) and higher could be fabricated using regionally available materials and such strengths could best be achieved through the use of crushed limestone as the coarse aggregate.

The second research effort was performed by Adelman and Cousins (4). The objective of this project was to investigate the cost effectiveness and material properties of high-strength concrete made with Louisiana available materials for use in prestressed, pretensioned bridge girders. A summary of the findings of this research is as follows:

1. A mix design was developed that consistently yielded 28-day concrete compressive strengths in excess of 10,000 psi (69.0 MPa). The mix consisted of crushed limestone with 3/4-in. (19-mm) maximum size, Type I portland cement, Class C fly ash, and a superplasticizer. A mix yielding 10,000 psi (69 MPa) without a superplasticizer proved unworkable.

2. The measured concrete modulus of rupture compared well with results reported in other literature. The measured concrete modulus of elasticity, 6,800 ksi (47.9 GPa), represented the upper bound for the range of values reported in the literature.

3. An increase in concrete design compressive strength from 6,000 psi (41.4 MPa) to 10,000 psi (69.0 MPa) resulted in an approximate 10 percent increase in span length, with an overall decrease in superstructure cost of five percent for a typical three-span LaDOTD bridge.

2.3 RESEARCH PERFORMED AT THE UNIVERSITY OF TEXAS AT AUSTIN

The University of Texas at Austin has performed a significant amount of research into the use of high-strength concrete in highway structures. Most of this research has been part of Research Project 3-5-84-381, "Optimum Design of Bridge Girders Made Using High-Strength Concretes and Deflection of Long-Span Prestressed Concrete Beams." This research was performed for the Texas State Department of Highways and Public Transportation. From this research project, four reports were written. Each of these reports are discussed below.

2.3.1 Kelly, Bradberry, and Breen - Report 381-1

In this investigation by Kelly, Bradberry, and Breen (5), field instrumentation of eight 127-ft (38.7-m) long pretensioned AASHTO Type IV bridge girders was reported. Four of the girders were in one bridge, while the remaining four were in a second bridge. The girders were instrumented at the time they were cast and were monitored for three years. Two of the girders in each bridge were instrumented to measure concrete surface strains, prestressing strand strains, internal concrete temperatures, and camber or deflection. The remaining two girders in each bridge were instrumented to measure only concrete surface strains and camber.

The concrete mix used in the girders consisted of Type III portland cement, sand, crushed limestone aggregate, and a superplasticizer admixture. The mix had an average water/cement ratio of 0.4. The average 28-day concrete compressive strength for the mix was 8,620 psi (59.4 MPa). The objectives of this research were to:

1. Measure the elastic and time-dependent deformations of the girders during field construction and early service life and determine the sensitivity of time-dependent behavior to variations in material properties and the construction time schedule.

2. Test material samples from the instrumented girders to determine the long- and short-term material properties.

3. Evaluate the accuracy of current analytical techniques for predicting time-dependent behavior and revise and improve those techniques where required.

Problems were encountered in this research project that complicated the achievement of the stated objectives. The concrete surface strain measuring system did not work properly due to a failure in the adhesive system used to attach the Demec points. The readings were considered totally unreliable within 100 days. The strand strain measuring system basically failed, with most gauges no longer providing reasonable readings one month after installation. The system for measuring internal temperature gradient and camber measurements, however, performed well.

The material tests on samples taken from the concrete batches used for the girders were successful. Tests were made to determine the concrete compressive strength and the modulus of elasticity at various ages. In addition, measurements were made to determine the creep and shrinkage of the concrete.

Some of the conclusions of this study were as follows:

1. The AASHTO formula for predicting the elastic modulus of concrete should not be used for predicting deflections of high-strength concrete members ($f'_c > 9{,}000$ psi (62.1 MPa)). It was recommended that the following formula be used in lieu of the one presently proposed by AASHTO:

$$E_c = 40{,}000 \sqrt{f'_c} + 1.5 \times 10^6 \text{ psi}$$

2. Elastic camber and deflection of high-strength concrete girders can be accurately predicted using moment area equations if actual concrete strengths are known and used.

3. The time-dependent camber or deflection of a girder is significantly affected by the rate of concrete strength gain, the concrete creep coefficient, the relative humidity, the age of concrete at release and the construction schedule.

4. Additional research is needed to develop a reliable system for measuring long-term strain of strand in pretensioned, prestressed concrete girders.

2.3.2 Hartman, Breen and Kreger - Report 381-2

In this research report, Hartman, Breen and Kreger (6) performed studies on the shear capacity of high-strength, prestressed concrete girders. Ten pretensioned girder specimens, made utilizing concretes with compressive strengths ranging from 10,800 psi (74.5 MPa) to 13,160 psi (90.7 MPa), were tested. Six 16-ft (4.9-m) long girder specimens were fabricated specifically for this program. The remaining four girder specimens were 17.3-ft (5.3-m) long end sections of flexural specimens previously tested by Castrodale, Kreger, and Burns (7). All girders were one-third scale models of AASHTO Type IV girders.

The six new girder specimens had a shear span-to-depth ratio, a/d, of 3.0, while the remaining four specimens had an a/d ratio of 3.2. The primary variable in these tests was the amount of shear reinforcement. Shear strength provided by web reinforcement ranged from 0 to 15 $\sqrt{f'_c} b_w d$.

The concrete mix design used for the six new girders consisted of Type I portland cement, very hard 3/8-in. (9.5-mm) maximum size crushed limestone coarse aggregate, and natural river sand fine aggregate. The water-cement ratio for the mix was 0.25. High-range water reducer was used in the mix.

In addition to the laboratory tests, a comprehensive review of shear tests in high-strength concrete girders reported in American literature was carried out. All test results were evaluated in comparison with current AASHTO/ACI provisions (1,2), the compression field theory recommended by the Canadian code (7), and the variable inclination truss models.

The results of the tests indicated that current maximum shear reinforcement limits could be substantially increased. It was also discovered that all three methods of shear evaluation presently in general use gave conservative results for prestressed high-strength concrete members. Any of the three design methods evaluated would be acceptable for concrete strengths up to 12,000 psi (82.7 MPa). It was also discovered that all three design procedures showed little variation in conservatism as concrete compressive strength increased.

2.3.3 Castrodale, Kreger and Burns - Report 381-3

In this research project, Castrodale, Burns, and Kreger (8) examined the stress transfer characteristics of 1/2-in. (13-mm) diameter low-relaxation strand in normal- and high-strength concrete. In addition, two one-third scale Type IV girder specimens were fabricated with 12,000 psi (82.7 MPa) concrete and tested in flexure to evaluate current design provisions and analysis techniques.

The findings of this investigation revealed the following:

1. The current AASHTO expression for estimating transfer length is conservative for high-strength concrete.

2. The maximum usable concrete strain to failure was lower for high-strength concrete than for normal-strength concrete.

3. Placement of high-strength concrete in narrow, congested sections is possible through the use of high-range water reducers (superplasticizers).

2.3.4 Castrodale, Kreger, and Burns - Report 381-4F

In research report 381-4F, Castrodale, Kreger, and Burns (9) performed a review of the AASHTO and ACI Codes, current practices, previous tests, and recent literature to determine the safety and efficiency of using high-strength concrete in pretensioned bridge girders. Selected girder cross sections were reviewed to determine the sensitivity of different design parameters and to determine the effectiveness of those cross sections with the use of high-strength concrete. Some of the findings of this research were as follows:

1. The AASHTO and ACI simplified flexural design approach for strength appears to provide good, conservative estimates of the capacity.

2. Current expressions for determining strand stress at ultimate appear to be adequate.

3. Further study should be performed to determine the strain in the top fiber that leads to crushing in a member.

4. Further study should be conducted to determine the effects that concrete strength has on all aspects of design, including lateral stability, fatigue, cracking, and deflections.

2.4 RESEARCH PERFORMED AT NORTH CAROLINA STATE UNIVERSITY

Zia, Schemmel, and Tallman (10) performed parametric investigations to determine the feasibility of using high-strength concrete for bridge projects. Included in the investigation were prestressed concrete hollow core slabs, prestressed concrete girders, and reinforced concrete piers. The current AASHTO design procedures were used for the design of the flexural and compression members; however, the modulus of rupture equation recommended by ACI Committee 363 was used in lieu of the one recommended by AASHTO.

The investigation recommended that 10,000 psi (69.0 MPa) concrete be used for bridge projects and that a demonstration bridge be built. Properties of high-strength concrete used in the studies were based on previous work performed by Leming (11) on high-strength concrete made using local North Carolina materials.

2.5 RECENT RESEARCH PERFORMED BY THE PORTLAND CEMENT ASSOCIATION AND CONSTRUCTION TECHNOLOGY LABORATORIES, INC.

2.5.1 Shin, Kamara, and Ghosh

Shin, Kamara, and Ghosh (12) tested 36 specimens, each 6-in.x12-in.x10-ft (152-mmx305-mmx3-m). The specimens were manufactured using concrete strengths of 4,000 psi (27.6 MPa), 12,000 psi (82.7 MPa) and 15,000 psi (103.4 MPa). The specimens were reinforced as columns, though they were tested in flexure. It was determined that the equivalent rectangular compression stress block of the ACI code was valid for the flexural strength computations of members with concrete strengths ranging up to 15,000 psi (103.4 MPa).

2.5.2 Roller and Russell

The purpose of this investigation by Roller and Russell (13) was to evaluate the shear strength performance of high-strength concrete beams with web reinforcement relative to the current ACI code. The investigation consisted of two test series with each series containing five beams. The beams of the first series ranged in depth from 25 in. (635 mm) to 29-1/4 in. (743 mm) and were fabricated using concrete with a compressive strength of 17,420 psi (120.1 MPa). All beams in the first series were 14 in. (356 mm) wide but had different quantities of shear reinforcement ranging from the minimum amount required by ACI to the maximum amount that can be assumed when calculating shear capacity. In the second series, all beams were 34-1/4 in. (870 mm) deep with two of the specimens fabricated from concrete with a compressive strength of 10,500 psi (72.4 MPa) and the remaining three with a compressive strength of 18,170 psi (125.3 MPa). The second series of beams had a web width of 18 in. (457 mm) and had amounts of shear reinforcement varying from the minimum amount prescribed by ACI to three times the minimum.

The high-strength concrete in eight of the beams was made using Type I portland cement, Class C fly ash, silica fume, coarse aggregate with a top size of 1/2 in. (13 mm), sand, and a water-cement ratio of 0.26. A high-range water reducer and a water-reducing retarder were used to improve workability. The concrete in the remaining two beams contained all of the same constituents listed above except the silica fume and high-range water reducer. The water-cement ratio for these two beams was 0.31.

It was concluded from this study that the present ACI code provisions overestimate the nominal shear strength provided by the concrete when the compressive strength of the concrete exceeds 17,000 psi (117.2 MPa). A recommendation was also made that the minimum quantity of shear reinforcement specified in the ACI code should be increased as the concrete compressive strength increases.

2.6 OTHER RECENT PERTINENT RESEARCH

2.6.1 Peterman and Carrasquillo

This report by Peterman and Carrasquillo (14) demonstrated that high-strength concrete could be produced using conventional batching procedures. Some of the other findings of this investigation were as follows:

1. The compressive strength of concrete increases as the amount of superplasticizer increases, up to the dosage which causes the concrete mix to become segregated and unworkable. Too much superplasticizer can result in the retardation of concrete hardening. There was a definite correlation between the brand of superplasticizer and the workability and compressive strength of the high-strength concrete.

2. High-strength concrete can be produced using natural gravel or crushed stone, though crushed stone will yield higher strengths.

3. Higher compressive strength can be attained using a Class C fly ash rather than using an equal amount of portland cement, if the ratio of the weight of fly ash to the combined weights of the fly ash and Portland cement is in the range from 20 to 30 percent.

4. The one-day strength of high-strength concrete is reduced slightly by the addition of fly ash; however, the addition of superplasticizer more than compensates for this reduction.

5. The 28-day compressive strength of high-strength concrete that has been cured under ideal conditions for seven days is not seriously affected by curing under hot, dry conditions from seven to 28 days after casting.

6. The modulus of rupture of high-strength concrete falls between $8\sqrt{f'_c}$ and $12\sqrt{f'_c}$.

7. The compressive strength of high-strength concrete specimens cast in steel molds is generally about 10 percent higher than that of specimens cast in cardboard molds.

2.6.2 Ahmad and Shah (1985)

In this report, Ahmad and Shah (15) investigated the structural properties of high-strength concrete and the implications that these properties may have for precast, prestressed concrete. Experimental data generated by the authors, as well as that generated by other investigators, was used to develop empirical expressions to substitute for those currently being used. Some of the most significant findings of this report are as follows:

1. The stress-strain curves of normal- and high-strength concrete are significantly different. The high-strength concrete stress-strain curve is much more linear to a higher fraction of the compressive strength. It was also noted that the slope of the post maximum stress increases as the strength increases.

2. The present ACI code expression for modulus of rupture, f_r, should be changed from $7.5\sqrt{f'_c}$ to $2\,(f'_c)^{2/3}$.

3. The present ACI equation for estimating the secant modulus of elasticity predicts values as much as 20 percent too high for concrete with a compressive strength near 12,000 psi (82.7 MPa).

4. In general, high-strength concrete can reduce construction time. Since required release strength is attained earlier with high-strength concrete, stress transferring operations can be performed earlier.

2.6.3 Carrasquillo, Nilson, and Slate

Carrasquillo, Nilson, and Slate (16) studied concretes in three strength ranges in this investigation: high-strength concrete with compressive strength of at least 9,000 psi (62.1 MPa), medium-strength concrete with compressive strength from 6,000 psi (41.4 MPa) to 9,000 psi (62.1 MPa), and normal-strength concrete with compressive strength from 3,000 psi (20.7 MPa) to 6,000 psi (41.4 MPa). All concrete mixes were made using Type I portland cement. The course aggregate used was either a crushed limestone or a crushed gravel. The purpose of the investigation was to determine the properties of high-strength concrete subject to short term loads. Compression tests were conducted on both 4 x 8-in. (102 x 203-mm) and 6 x 12-in. (152 x 305-mm) cylinders. Plain concrete 4 x 4 x 14-in. (102 x 102 x 356-mm) beams were used for modulus of rupture tests.

Some of the findings of interest from this study are summarized as follows:

1. When moist cured for seven days and then allowed to dry at about 50 percent relative humidity until testing at 28 days, high-strength concrete showed a larger reduction in compressive strength than did normal-strength concrete. High-strength concrete had an average reduction of 10 percent, while normal-strength concrete showed only a four percent reduction. Significantly greater reductions in modulus of rupture occurred with high-strength concrete, showing a 26 percent reduction compared to an 8 percent reduction for normal-strength concrete.

2. The following equations were recommended to replace those presently being used by ACI and AASHTO:

   Modulus of elasticity

   $$E_c = 40{,}000\sqrt{f'_c} + 1.0 \times 10^6 \text{ psi}$$

Modulus of rupture

$$f_r = 11.7 \sqrt{f'_c} \text{ psi}$$

3. Poisson's ratio is close to 0.20 regardless of the compressive strength or the testing age.

3. FEASIBILITY OF PRODUCING HIGH-STRENGTH CONCRETE

3.1 INTRODUCTORY REMARKS

One of the objectives of this research effort is to establish the feasibility of producing high-strength concrete using fabricators and materials indigenous to the southeastern region of the United States. To accomplish this objective, a survey of local fabricator capabilities was performed; a study of trial mix designs was made, both in the laboratory and the field; and test specimens were fabricated using an established mix design and standard production techniques and procedures. The purpose of this chapter is to present the findings of the production feasibility portion of the study. Section 3.2 presents the results of the fabricator survey. Section 3.3 presents the mix design studies that were performed and the results of using those mix designs under plant production conditions. Section 3.4 presents the conclusions drawn from this portion of the study.

3.2 SURVEYS OF REGIONAL FABRICATORS

During the first phase of the research effort, a total of five fabrication companies in four different states were contacted to determine their interest in, and their capability of, producing concrete with a compressive strength of 10,000 psi (69.0 MPa). Of the five firms contacted, three stated a reluctance to participate in the project citing concerns about consistently producing 10,000 psi (69.0 MPa) concrete using local materials under production conditions. The other two firms felt they could consistently produce 10,000 psi (69.0 MPa) compressive strength concrete.

3.3 MIX DESIGN AND PRODUCTION

Two approaches were used to establish the mix design for this research project. For the 24-ft (7.3-m) long pile specimens, the fabricator submitted a mix design to the research team which was intended to yield concrete with a 28-day compressive strength of 10,000 psi (69.0 MPa). The proportions of that mix design by weight are shown in Section 5. Prior to the pile production, the fabricator had performed tests of several trial batches in the plant using Type III cement and had achieved the desired results. However, one change was made in the established mix design for the production of the pile specimens. At the request of LaDOTD, Type III cement used in the trial batches was changed to Type I. The change was made because Type I cement is more representative of the material normally supplied in Louisiana. No additional trial batches were made after the cement type was changed. Twenty-eight days after casting the pile specimens, the highest strength achieved by the control cylinders was 8,410 psi (58.0 MPa). At 56 days, the highest control specimen strength was 8,290 psi (57.2 MPa). At 169 days, 4 x 8 in. (101 x 203 mm) concrete cores taken from the pile specimens were tested. The average strength of these cores was 9,780 psi (67.4 MPa). These cores indicated that the concrete mix design had the potential of eventually achieving 10,000 psi (69.0 MPa).

In the case of the mix design used for the first three bulb-tee specimens, a different approach was used. The Louisiana Transportation Research Center (LTRC) performed extensive laboratory evaluations of several trial concrete mixes to determine a mix design most likely to achieve concrete with a 28-day compressive strength in excess of 10,000 psi (69.0 MPa). All laboratory trial batches were prepared in a 3 ft^3 (0.09 m^3) mixer. In addition to batch proportions, batch sequencing and mixing times were developed. The mix design developed by LTRC resulted in concrete with an average 28-day compressive strength of 12,395 psi (85.5 MPa).

Following the development of a mix design in the laboratory, field tests were made by the fabricator using 3 yd^3 (2.3 m^3) batches. These trial batches resulted in concrete with an average 28-day compressive strength of 10,854 psi (74.8 MPa). This correlated well with the findings of Cook (17) which indicated that an approximate 10 percent reduction in compressive strength will occur from the laboratory to the field tests. In both the laboratory and field tests, compressive strengths in excess of 10,000 psi (69.0 MPa) were achieved. The mix design developed by LTRC is shown in Section 6.

With a mix design developed in the laboratory and tested in the field, production of the first three bulb-tee specimens was approved. A total of nine 4-1/4 yd^3 (3.25 m^3) batches were required to cast the three bulb-tee specimens. The mix proportions for the 4-1/4 yd^3 (3.25 m^3) batches, as well as the mixing procedures, are given in Appendix B. As indicated by the individual batch quantities, water, water reducer and air entrainment admixture quantities were varied slightly from batch to batch to maintain slump and air content within an acceptable range. Mixing times were also modified where judged necessary. Several concrete cylinders were made for each of the nine batches used in the girders. After the concrete in the girders had reached initial set, the girders and the test cylinders were steam-cured for 24 hours. Although steam curing was not part of the initial trial laboratory and field tests, it was felt that steam curing would give a little insurance with respect to the early-age strength gain, since Type III cement was not used. Compressive strength data for the nine batches used in the girders are given in Section 6. As indicated by these data, the average 28-day concrete compressive strength of 9,750 psi (67.2 MPa) for the nine batches did not reach the desired 10,000 psi (69.0 MPa). It is also interesting to note that there was very little strength gain after release.

For the fabrication of the 130-ft (39.6-m) long pile, the LTRC mix design was modified slightly at the discretion of the fabricator. The 130-ft (39.6-m) long pile was made by the same fabricator as the first three girders, but at a different precast plant. The supply sources of all mix constituents were changed from those used for the first three girders. Five 4 yd^3 (3.06 m^3) batches were used for the pile production. After achieving initial set, the pile specimen was subjected to 18 hours of steam curing followed by ambient air curing. Cylinders prepared from the concrete batches used in the pile were cured under three different environments. One curing environment was the same as that experienced by the pile specimen. The second curing environment was ambient air only. The third environment was a continuous moist cure. These three curing environments resulted in 28-day strengths of 10,450 psi (72.1 MPa) for the steam/air cured specimens, 11,870 psi (81.8 MPa) for the air-cured specimens and 11,640 psi (80.3 MPa) for moist-cured specimens. Based on these results, it was concluded that the steam curing was unnecessary and most probably hindered the compressive strength development. A complete discussion of the fabrication of the 130-ft (39.6-m) pile specimen can be found in Section 7.

For the fabrication of the two girders to be used for the fatigue tests, the mix design used for the 130-ft (39.6-m) pile was again used without modification. However, this time a decision was made not to steam cure. Six 4 yd^3 (3.1 m^3) batches were used for the girders. The girder specimens and cylinders were subjected to ambient air curing only. The average 28-day concrete compressive strengths for the two girders were 8,800 psi (60.7 MPa) and 8,830 psi (60.9 MPa), respectively. Extreme variations in strength between batches used for each girder were noted. One of the two girders was discarded as a result of unacceptable concrete strength at the mid-span region. Complete discussion of fabrication details for these two girders can be found in Section 8.

3.4 CONCLUSIONS

Although laboratory and field trial batches produced the desired strengths, only the production batches used in the 130-ft (39.6-m) pile specimen achieved the desired strength. To determine probable causes for the shortfall in production concrete strength, petrographic analyses were performed on cores from the 24-ft (7.3-m) pile specimens and the bulb-tee specimens. These petrographic analyses revealed no explanation for the failure to achieve the desired strength. In addition to the petrographic analyses, samples of the coarse aggregate used in the bulb-tee specimen were tested by LTRC. These tests revealed that the coarse aggregate had sufficient compressive strength to reach the desired ultimate strength of the concrete, but with little reserve.

High-strength concrete, unlike lower strength concrete, requires a high degree of quality control. A mix design must be developed and then the standards used for measuring batch weights, measuring water content of aggregate, and maintaining quality of materials must be significantly "tighter" than those presently used in the industry in the southeastern United States. Any change in material source, material specifications, or batching procedures can have a dramatic effect on both the plastic consistency of the mix and the resultant concrete strength. To consistently produce high-strength concrete using regionally available materials, fabricators will have to undergo a period of transition wherein the standard procedures used for producing "normal" strength concrete (5,000 to 8,000 psi (34.5 to 55.2 MPa) are replaced by the more exacting requirements of high-strength concrete. With this transition, consistent production of high-strength concrete - 10,000 psi (69.0 MPa) and higher - can be achieved.

4. DESIGN OF TEST SPECIMENS

4.1 INTRODUCTORY REMARKS

The specimens fabricated for use in this research program were intended to be representative of members presently in use by state departments of transportation. It was further intended to design these members to make efficient use of the properties of high-strength concrete. A discussion of the design considerations for the specimens used in each series of tests is presented below.

4.2 DESIGN OF THE PILE SPECIMENS

Concrete piles in Louisiana are often limited to lengths that can be picked up and placed in the leads without excessive bending stresses or driven without excessive tensile stresses. When long piles are used, there is increased potential for pile damage or failure due to excessive flexural stresses during handling, or if driven in soft soils, due to excessive tensile driving stresses. In an effort to alleviate this problem, the Louisiana Department of Transportation and Development, like many states, has long sought a dependable pile splice for long piles. Because LaDOTD has not yet found a satisfactory splice, the decision was made to investigate the possibility of taking advantage of the higher tensile strength and higher level of precompression of high-strength concrete to increase the length of piles.

The LaDOTD uses 24-in. (610-mm) square piles with an 12-in. (305-mm) diameter concentric void for many applications. Four 24-in. (610-mm) square pile specimens were designed, fabricated and tested as part of this research program. These pile specimens were designed assuming a length of 130 ft (39.6 m) and two-point pickup for handling. Flexural stresses resulting from handling (rather then tensile driving stresses) were assumed to control the pile design. Twenty-four, 1/2-in. (12-mm) diameter, Grade 270, low-relaxation strands were used, resulting in an initial precompressive stress of 1,606 psi (11.1 MPa). A 28-day concrete compressive strength of 8,500 psi (58.6 MPa) was assumed for the design of the piles. The initial precompressive stress in long piles is typically below the maximum allowed by the AASHTO standard in order to allow for additional compressive stresses due to flexure during handling. In a similar pile designed assuming 5,000 psi (34.5 MPa) compressive strength concrete, 14 strands would have been required and the resulting initial precompressive stress would have been 937 psi (6.5 MPa). Therefore, the benefits of using high-strength concrete for piles are apparent regardless of whether tension or flexure controls design.

Although pile specimen design was based on a 130-ft (39.6-m) long pile, three of the four pile specimens were only 24-ft (7.3-m) long. These three specimens were fabricated to a length sufficient to evaluate strength properties through laboratory flexural tests. The remaining pile specimen was fabricated to the full 130-ft (39.6-m) design length and was used for field evaluation of handling and driving stresses.

4.3 DESIGN OF THE BULB-TEE SPECIMENS

Various studies have indicated the advantage of using high-strength concrete in precast, prestressed bridge girders (8, 10). An evaluation conducted by Rabbat and Russell concluded that the structural efficiency of the bulb-tee cross section was superior to that of other widely used bridge girder cross sections (18, 19). The bulb-tee girder cross section was selected for this research program in order to make the most efficient use of high-strength concrete.

A bridge design utilizing 70-ft (21.3-m) long, 54-in. (1,37-m) deep bulb-tee girders with draped prestressing strands was prepared by the LaDOTD for the purpose of developing a representative girder design for the test specimens. The length and depth of the bridge girders were selected by the project team to facilitate transportation to the testing laboratory. A 28-day concrete compressive strength of 10;000 psi (69.0 MPa) and a release compressive strength of 6,000 psi (41.4 MPa) were assumed in the design of the bridge girders.

Bridge design was based on the AASHTO standard as well as requirements of the state of Louisiana. The LaDOTD made use of a computer program (Span Version 5.1) developed by LEAP software in the design of the girder specimens. Typical allowable stresses and material property relationships prescribed by the AASHTO standard were used in the girder design. Although most of the AASHTO-prescribed values were established for lower strength concretes, previous high-strength concrete studies suggested that the use of most typical values for the girder design would be appropriate. Data from the material property tests conducted on concrete from the 24-ft (7.3-m) long pile specimens also confirmed the appropriateness of using typical values.

The bridge design resulted in a girder spacing of 13.3 ft (4.1 m). Thirty 1/2-in. (12-mm) diameter, Grade 270, low relaxation strands were required in each girder. Six of the strands were draped. Based on the girder spacing, a deck slab thickness of 9-1/2 in. (241 mm) was established. The live load stresses were governed by the LaDOTD's HST-18 design vehicle rather than the HS-20 truck or lane load. Dead load stresses were determined using an assumed concrete unit weight of 150 lb/ft^3 (2,435 kg/m^3) for both the girder and deck slab concrete.

Initial (release) and final prestress losses used in the LEAP design program were based on the provisions of the AASHTO standard. Prestress losses for high-strength concrete are expected to be less than those calculated using the AASHTO provisions, as a result of the lower creep and shrinkage characteristics of high-strength concretes. Measured prestress losses in the 24-ft (9.3-m) long pile specimens supported this theory. Therefore, use of the AASHTO provisions for calculating prestress losses may be overly conservative for high-strength concrete.

The LEAP program was used to calculate girder and composite stresses at various stages of loading. Initial (release) stresses due to prestress and self weight calculated by the LEAP program for the top and bottom extreme fibers of the girder were 161 psi (1.115 MPa) tension and 2,813 psi (19.4 MPa) compression, respectively. Based on the design concrete compressive strength at release of f_{ci} = 6,000 psi (41.4 MPa), the initial calculated girder concrete stresses approached the prescribed AASHTO allowable stresses of 0.6 f_{ci} compression and $3\sqrt{f'_c} \leq 200$ psi (1.379 MPa) tension.

Final stresses calculated by the LEAP program for the top extreme fiber of the deck slab, top extreme fiber of the girder and bottom extreme fiber of the girder were 440 psi (3.03 MPa) compression, 1,455 psi (10.0 MPa) compression and 550 (3.79 MPa) tension, respectively. Based on the design concrete compressive strength at 28-days of f'_c = 10,000 psi (69.0 MPa), the final calculated concrete stresses in the girder were within the prescribed AASHTO allowable stresses of 0.40 f'_c compression and approached the AASHTO allowable stress of $6\sqrt{f'_c}$ tension. Stresses in the composite section were calculated using the transformed effective deck slab width. The effective slab width was calculated using Article 8.10.1.1 of the AASHTO standard. Moduli of elasticity of the girder and deck slab concrete were calculated from the equation $E_c = 33\,w^{3/2}\sqrt{f'_c}$.

Three of the four girder specimens tested as part of this research incorporated a deck slab. Based on a design girder spacing of 13.3 ft (4.1 m), the total allowable width of deck slab effective as a T-girder flange is 10 ft (3.1 m) per AASHTO 8.10.1.1 (1). Therefore a 10-ft (3.1-m) wide deck slab was used for the girder specimens. Deck slab reinforcement details for the girder specimens were representative of the requirements of both the AASHTO standard and the state of Louisiana. The bottom layer of slab reinforcement (distribution steel) was only included in the girder used for the long-term test. It was determined analytically that this layer would not have any significant effect on girder flexural strength and therefore was not included in the other girder specimens with deck slabs.

The 28-day concrete compressive strength for the deck slab assumed in the bridge design was 3,200 psi (22.1 MPa). The state of Louisiana typically specifies a minimum 28-day concrete compressive strength of 4,200 psi (29.0 MPa) for bridge deck slabs. The minimum concrete compressive strength specified for the girder specimen deck slabs was 6,000 psi (41.4 MPa) in order to (1) provide a more compatible relationship between material properties of the deck slab and girder concrete, and (2) help avoid a compression failure mode during the flexural strength tests.

5. FLEXURAL TESTS OF PILE SPECIMENS

5.1 FABRICATION OF PILE SPECIMENS

Three pile specimens, designated P1, P2, and P3, were fabricated for flexural tests. These piles were fabricated at Southern Prestressed, Inc. in Pensacola, Florida. Specific details of the pile specimens are shown in Figure 1. Supplemental materials and fabrication details are discussed in Appendix A.

The cross sections of the pile specimens were nominally 24-in.(610-mm) square with a centrally-located 12-in. (305-mm)-diameter circular void. The pile specimens were 24-ft (7.3-m) long. Twenty-four uncoated, 1/2-in (13-mm) diameter, Grade 270, low-relaxation, 7-wire strands were used to prestress the pile specimens. The strands were placed symmetrically around the perimeter of the cross-section of each pile specimen. Following placement, the strands were pretensioned to a stress level corresponding to 75% of guaranteed ultimate strength. Transverse reinforcement in the pile specimens consisted of W4.5 spiral wire.

The three pile specimens were pretensioned simultaneously (in series) within a single prestressing bed. The strands across the bottom of the piles were tensioned first, followed by the two sides, and finally the top. Each pile specimen was cast separately using a single concrete batch. The average ambient temperature during casting was approximately 65°F (18°C). Approximately one hour after casting, the pile specimens were covered with a tarpaulin consisting of a thin layer of foam insulation sandwiched between two layers of waterproof canvas.

Approximately 24 hours after casting, the prestress was released by simultaneously torch cutting the strands at the far ends of the prestressing bed. After torch cutting the strands at the ends, the strands were cut within the space between the pile specimens. The pile specimens were removed from the forms following release of prestress.

5.2 PILE SPECIMEN INSTRUMENTATION

During the fabrication process, the piles were instrumented to measure concrete strains. Internal strain meters were installed at mid-length of each pile. External mechanical strain gauge reference points were installed at the ends of each pile for transfer length determination. Pile instrumentation details are shown in Figure 2.

5.2.1 Internal Strain Meters

Subsequent to pretensioning and prior to casting the concrete, four Carlson concrete strain meters were installed in each pile specimen. The strain meters were placed at mid-length, adjacent to each of the four sides of the pile. Readings were taken after installation to verify function of the strain meters. Several readings were taken after casting the concrete to monitor both early concrete shrinkage strains and temperature. Readings were taken immediately before and after release of prestress to determine the initial concrete strains. The initial concrete strains were used to determine the magnitude of prestress losses due to elastic shortening. Concrete strains were monitored periodically until an age of 28 days for the purpose of determining the magnitude of short-term prestress losses. The Carlson strain meters were also used to monitor concrete strains during the flexural strength tests of the pile specimens.

5.2.2 Transfer Length Instrumentation

Just prior to release of the prestress, a longitudinal line of Whittemore mechanical strain gauge points were glued to the top surface of each pile specimen at both ends. The gauge points were spaced exactly 5 in. (127 mm) apart, between 5 in. (127 mm) and 60 in. (1.52 m) from the ends of the piles. Using the various pairs of Whittemore points, concrete surface strains were measured within the first 5 ft (1.52 m) from each pile end using a 10-in. (254-mm) Whittemore gauge. Concrete surface strain readings were used to determine the strand transfer length required for full development of the prestress in the concrete. The longitudinal distance from the ends of the piles at which the concrete surface strains became somewhat uniform provided an indication of the transfer length.

5.3 PILE SPECIMEN MATERIAL PROPERTIES

5.3.1 Concrete Material Properties

The concrete mix for the pile specimens was developed by the fabricator and was designed to produce concrete with a minimum 28-day compressive strength of 8,500 psi (58.6 MPa). Specific mix design details are given in Table 1. Each pile specimen was fabricated using a separate concrete batch. The mix proportions of each batch were exactly the same and were consistent with those shown in Table 1. Six 6 x 12 in. (152 x 305 mm) concrete cylinders and two 6x6x20 in. (152x152x508 mm) concrete beams were made from each batch for material property testing. Concrete material property tests were conducted at release and at concrete ages of 28 and 56 days. In addition to the standard cylinder specimens, four 4 x 8 in. (102 x 203 mm) cores were taken from each pile specimen. Three of the cores from each pile were tested in compression at concrete ages of 71, 85, and 186 days.

Concrete compressive strength tests of the cylinder specimens were conducted in accordance with ASTM Designation: C39-86 (20). Compressive strength tests of the concrete cores were conducted in accordance with ASTM Designation: C42-90 (21). Modulus of elasticity tests were conducted in accordance with ASTM Designation: C469-87 (22). Splitting tensile strength tests were conducted in accordance with ASTM Designation: C496-90 (23). Modulus of rupture tests were conducted in accordance with

TABLE 1

CONCRETE MIX DETAILS - P1, P2 AND P3

Materials	Quantity/Cubic Yard
Cement	752 lb
Fly Ash	70 lb
Silica Fume	50 lb
Fine Aggregate	928 lb
Coarse Aggregate	1,912 lb
Water	31.0 gal.
Air Entrainment Admixture	17.4 oz.
Superplasticizer	157.0 oz.

Metric Equivalents:

1 lb = 0.454 kg
1 gal. = 3.785 l
1 oz. = 29.574 cc

ASTM Designation: C78-84 (24). Results of the concrete material property tests are given in Table 2 for the three pile specimens. Each value in Table 2 represents a single test result.

In addition to the tests reported in Table 2, concrete unit weight was determined from two core samples taken from each pile specimen. Average unit weight for piles P1, P2, and P3 was 151.5 , 150.0, and 147.1 lb/ft^3 (2,427, 2,403, and 2,356 kg/m^3), respectively.

As indicated in Table 2, the concrete in each of the three piles failed to achieve the required 28-day compressive strength of 8,500 psi (58.6 MPa). However, results from the core compressive strength tests indicated that the pile concrete had the potential for achieving strengths approaching 10,000 psi (69.0 MPa). Petrographic (microscopic) examination of a single core sample from each pile was conducted in an effort to determine the cause of the low

TABLE 2

CONCRETE MATERIAL PROPERTIES - P1, P2, and P3

Pile No.	Concrete Age, days	Compressive Strength, psi	Modulus of Elasticity, ksi	Splitting Tensile Strength, psi	Modulus of Rupture, psi
P1	Release	5,210	4,900	No Test	No Test
	28	7,400	5,550	575	720*
	56	7,660	5,350	494	733
	71	8,200	No Test	No Test	No Test
	85	9,290**	No Test	No Test	No Test
	169	9,790***	No Test	No Test	No Test
P2	Release	5,480	5,950	No Test	No Test
	28	7,900	6,000	670	750*
	56	8,290	5,950	694	883*
	71	9,740	No Test	No Test	No Test
	85	9,490**	No Test	No Test	No Test
	169	9,730***	No Test	No Test	No Test
P3	Release	5,855	6,200	No Test	No Test
	28	8,410	5,750	690	665
	56	8,250	5,530	592	817*
	71	10,180	No Test	No Test	No Test
	85	9,260**	No Test	No Test	No Test
	169	9,810***	No Test	No Test	No Test

* Test specimen fractured below loading point.

** Moist cured for 14 days after coring.

*** Moist cured for 98 days after coring.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

28-day strength. Petrographic examinations of the cores were conducted in accordance with ASTM Designation C856-83 (25). Results of the petrographic examinations did not identify any specific cause for the strength shortfall. The observed concrete void system from the air entrainment and paste characteristics was as expected for the mix design used. Concrete aggregates were uniformly distributed and the paste-aggregate bond was tight.

5.3.2 Prestressing Strand Material Properties

Prestressing strand used in the pile specimens was 1/2-in.(13-mm) diameter, Grade 270, low-relaxation, 7-wire strand, conforming to ASTM Designation: A416-88 (26). All prestressing strand used in the pile specimens came from the same coil. Samples of the prestressing strand were taken from the beginning and end of the coil for material property testing. Strand samples were tested in accordance with ASTM Designation: A370-90a, Supplement VII (27). Results of the prestressing strand material property tests are given in Table 3. All strands tested met the requirements of ASTM A416-88 (26).

TABLE 3

PRESTRESSING STRAND MATERIAL PROPERTIES - P1, P2, AND P3

Sample	Area, sq in.	Load @ 1% Extension, lb	Breaking Strength, lb	Total Elongation, %	Modulus of Elasticity, ksi
Beginning #1	0.151	38,800	42,400	5.9	29,150
Beginning #2	0.151	39,000	42,600	6.5	29,950
Beginning #3	0.151	38,800	42,600	7.2	29,150
Beginning #4	0.151	39,000	42,700	6.1	29,650
End #1	0.151	39,000	42,400	6.1	29,650
End #2	0.151	38,400	42,200	6.0	29,900
End #3	0.151	38,800	42,600	6.6	28,900
End #4	0.151	39,000	42,600	5.9	28,000

Metric Equivalents:

1 sq in. = 645.2 sq mm
1 lb = 0.454 kg
1 ksi = 6.895 MPa

5.4 DISCUSSION OF MATERIAL PROPERTIES AND EARLY-AGE BEHAVIOR

5.4.1 Material Properties

The AASHTO Standard Specification for Highway Bridges (1) and ACI 318-89 (2) include specific equations for calculating concrete modulus of elasticity and modulus of rupture in lieu of actual test data. These equations make use of established relationships between concrete compressive strength and other concrete properties developed based on test results from lower strength concretes. For this reason, ACI published ACI 363R-84, State-of-the-Art Report on High-Strength Concrete (28). This report introduces alternate equations proposed by Carrasquillo (16) for both modulus of elasticity and modulus of rupture for use with high-strength concrete. Table 4 presents a comparison of the AASHTO/ACI 318 equations and the alternate equations of ACI 363.

TABLE 4

COMPARISON OF MATERIAL PROPERTY EQUATIONS
AASHTO/ACI 318-89 VERSUS ACI 363-84

Property	AASHTO ACI 318-89	ACI 363
Modulus of Elasticity	$E_c = w_c^{1.5}\, 33 \sqrt{f'_c}$ psi	$E_c = 40{,}000 \sqrt{f'_c} + 1 \times 10^6$ psi
Modulus of Rupture	$f_r = 7.5 \sqrt{f'_c}$ psi	$f_r = 11.7 \sqrt{f'_c}$ psi

Using the above equations with the measured concrete compressive strength and unit weight, the mechanical properties of the concrete for each pile specimen were calculated. In comparing the measure properties with computed properties, it is concluded that the measured results agree more closely with the AASHTO/ACI 318 values than with the ACI 363 values. The ACI 363 values tended to overestimate modulus of rupture and underestimate modulus of elasticity. The AASHTO/ACI 318-values tended to slightly underestimate both modulus of rupture and modulus of elasticity.

5.4.2 Transfer Length

As previously described, transfer length was experimentally evaluated for each pile specimen by measuring the change in concrete surface strains over a 5 ft (1.52 m) distance from the ends of each pile specimen. Concrete surface strains were measured using a 10-in. (254-mm) Whittemore gauge. Whittemore gauge readings were taken just prior to release (reference readings), just after release, after removing the piles from the forms, and 14 days after release. Transfer lengths interpreted from the Whittemore gauge readings are presented in Table 5. Documented values represent an average based on measurements from both ends of each pile specimen.

TABLE 5

PRESTRESSING STRAND TRANSFER LENGTHS - P1, P2, AND P3

Pile Specimen	Concrete Age	Average Transfer Length, in.
P1	Release	30
	After Stripping	30
	14 Days	31
P2	Release	18
	After Stripping	23
	14 Days	31
P3	Release	25
	After Stripping	30
	14 Days	31

Metric Equivalent:

1 in. = 25.4 mm

A mathematical expression for calculating the required development length for prestressed strand is provided in both the AASHTO standard (1) and ACI 318-89 (2). According to the ACI 318-89 Commentary, the required development length is equal to the sum of the transfer length plus an additional length over which the strand must be bonded to ensure that bond failure does not occur at nominal strength of the member. An expression for the transfer length alone can be written as:

$$\text{Transfer Length} = (f_{se}/3)d_b$$

where

f_{se} = effective stress in prestressing strand after losses

d_b = nominal diameter of prestressing strand

Using the above expression and assuming prestress losses based on readings from the Carlson strain meters, a transfer length of approximately 32 in. (813 mm) would be expected. As indicated in Table 5, transfer lengths interpreted from Whittemore readings were in reasonable agreement with the expected value.

5.4.3 Prestress Losses

The AASHTO Standard Specification for Highway Bridges contains provisions for calculating total prestress losses due to concrete shrinkage, elastic shortening, concrete creep, and steel relaxation. Based on these provisions and the design material properties, total prestress losses of approximately 37,530 psi (258.8 MPa) are expected for the piles. However, only approximately 35-40% of the total ultimate concrete creep and shrinkage losses are expected to occur within the first 28 days (29). Therefore, prestress losses of approximately 21,348 psi (147.2 MPa) are expected at a concrete age of 28 days.

Internal Carlson strain meters were installed in each pile specimen and monitored up until a concrete age of 28 days. Measured concrete strains were used to calculate prestress losses by multiplying the average measured strains by the average measured strand modulus of elasticity of 29,300 ksi (202 GPa). Concrete strains measured just after release and at an age of 28 days, and corresponding calculated prestress losses, are shown in Table 6.

Concrete strains measured immediately after release were used to provide an indication of prestress losses due to elastic shortening. Based on these concrete strains, losses due to elastic shortening averaged 9,239 (63.7 MPa) between the three pile specimens. This value is in reasonable agreement with the average elastic shortening losses of 7,402 psi (51.0 MPa) calculated using provisions from the AASHTO standard and actual measured concrete and steel material properties.

TABLE 6

MEASURED PRESTRESS LOSSES AT RELEASE AND 28 DAYS - P1, P2, AND P3

Pile No.	Specimen Age	Measured Strain in./in.	Loss in Prestress psi
P1	Release 28 days	306×10^{-6} 462×10^{-6}	8,966 13,537
P2	Release 28 days	298×10^{-6} 446×10^{-6}	8,731 13,068
P3	Release 28 days	342×10^{-6} 473×10^{-6}	10,021 13,859

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

Concrete strains measured at a concrete age of 28 days indicated prestress losses averaging approximately 13,488 psi (93.0 MPa) among the three pile specimens. This value is significantly less than the 21,470 psi (148.0 MPa) calculated using provisions from the AASHTO standard, actual measured concrete and steel material properties and recommendations of the PCI Committee on Prestress Losses (29). Based on this information, it appears that the actual total prestress losses in the three pile specimens can be expected to be significantly less than the total losses predicted using the AASHTO standard. However, calculated values for prestress losses were based on the assumption that approximately 35% of the ultimate creep and 42% of the ultimate shrinkage will occur within the first 28 days. These percentages, although shown to be reasonable for conventional concretes, may not be applicable for higher strength concretes. This may explain the difference between measured and calculated prestress losses at 28 days.

5.5 PILE SPECIMEN FLEXURAL TEST DETAILS

5.5.1 Flexural Test Setup

The test setup for the pile flexural tests is shown in Figures 3 and 4. Pile specimens were simply supported at their ends, creating a total span length of 22 ft (6.71 m). Pile specimens were tested in flexure using two point loads spaced 3 ft (914 mm) apart at mid-span. Load was applied to the specimens using hydraulic rams. Load cells were used to monitor the applied load. Pile deflection under load was measured using a pair of linear potentiometers located at mid-span. Load cells and potentiometers are calibrated annually.

Internal strain meters were used to monitor concrete strains continuously throughout the test. In addition to the internal meters, a concrete surface strain gauge was installed at mid-span on the bottom surface of each pile specimen. This gauge was also monitored continuously throughout the test.

5.5.2 Flexural Test Procedure

Load was applied to the pile specimens in increments of approximately 4000 lb (17.8 kN). Load cells, potentiometers, Carlson strain meters and the concrete surface strain gauge were monitored continuously throughout each test using a digital data acquisition system (DDAS) and computer. At selected levels of applied load, data were stored on floppy disk, providing a permanent record of test specimen behavior. Each pile was tested to determine flexural cracking moment, flexural shear cracking load, and ultimate moment capacity (failure). The tests were terminated when the girder could no longer sustain additional load.

5.6 ANALYSIS OF FLEXURAL TEST RESULTS

The three pile specimens were tested in flexure 56 days after fabrication. Total applied load (not including pile weight) versus mid-span deflection is plotted in Figures 5, 6, and 7 for pile specimens P1, P2, and P3, respectively. As indicated in these plots, behavior of the pile specimens was very consistent. All pile specimens failed in flexural compression. As indicated in Figure 8, cracking patterns observed in all three pile specimens were very similar.

5.6.1 Pile Specimen Behavior

In the following discussions, cracks are classified into three categories: flexural cracks, flexure-shear cracks, and web shear cracks. This terminology, as applied to prestressed concrete members, is defined by MacGregor (30) as follows:

> Flexural cracks are cracks that form when the normal applied stress exceeds the compression due to prestress and the modulus of rupture of the concrete in the vicinity of the crack.
>
> Flexure-shear cracks are cracks that originate as flexure cracks in the shear spans of a member and become inclined toward the load point.

Web-shear cracks are inclined cracks that originate in the web of I-shaped members. The cracks result from excessive principal tensile stresses in the web.

Flexural cracks occurred in the constant moment region and along the shear span as the applied load was increased. The observed total applied load corresponding to the formation of the initial flexural crack was 91.7, 95.7, and 86.8 kips (408, 426, and 386 kN) for pile specimens P1, P2, and P3, respectively. Further load increase caused additional flexural cracks to appear in the constant moment region, and caused flexural cracks to appear along the shear spans.

The cracks in the shear spans propagated upward to the bottom of the void and then became inclined toward the load points. These cracks are termed flexure-shear cracks, since they originate as flexural cracks in the shear span. The total applied load at which the flexure-shear cracking occurs was interpreted to be the load at which the inclined (angle of approximately 45 degrees) portion of the flexure-shear crack first reaches mid depth of the section. According to previous research (30), the flexure-shear crack which leads to failure of a specimen typically originates in the shear span at a distance from the load point corresponding approximately to the effective depth, d, of the member. The effective depth of the pile specimens, according to the requirements of the AASHTO specifications and the ACI code, is 80 percent of the total overall depth, or about 19 in. (480 mm). The crack identified as the first flexure-shear crack for each pile specimen was traced back to its origin on the tension face of the member. Consistent with previous research, the origin of these cracks occurred at a distance approximately equal to "d" from the load point. The total applied load corresponding to the formation of initial flexure-shear cracks was 135.4, 130.9, and 130.7 kips (602, 582, and 581 kN) for pile specimens P1, P2, and P3, respectively.

As the test progressed, additional flexural cracks and flexure-shear cracks developed and existing cracks continued to propagate. Web shear cracks did not occur in any of the pile specimens during the flexural tests. The load-deflection curves peaked at total applied loads of 161.2, 155.2, and 156.0 kips (717, 690, and 694 kN) for pile specimens P1, P2, and P3, respectively. The deflections coincident with the peak load were 1.53, 1.30, and 1.50 in. (39, 33, and 38 mm) for pile specimens P1, P2, and P3, respectively. Flexural compressive failure within the constant moment region marked the limit of the load capacity of the pile specimens.

5.6.2 Evaluation of Flexural Cracking Moment

The observed flexural cracking moment for each pile specimen was evaluated by comparison with computed values. For this evaluation, the flexural cracking moment was computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine the calculated flexural cracking moment. For the second comparison, actual measured material properties and prestress losses were used to calculate the cracking moment. Observed and computed flexural cracking moments for all three pile specimens are shown in Table 7.

TABLE 7

OBSERVED AND COMPUTED FLEXURAL CRACKING MOMENT - P1, P2, AND P3

Specimen No.	Observed Cracking Moment		Computed Cracking Moment - Design Condition		Computed Cracking Moment - Actual Condition	
	kip-ft	kN-m	kip-ft	kN-m	kip-ft	kN-m
P1	436	591	315	428	345	468
P2	455	617	315	428	350	475
P3	412	559	315	428	349	473

Observed cracking moment for the three pile specimens was determined using the observed cracking load and the following equation:

$$M_{cr} = a\,(P/2)$$

where

M_{cr} = observed cracking moment, in.-lb

a = length of shear span, in.

P = total applied load coincident with the first flexural crack, lb

The computed cracking moment, based on both design and actual conditions, was determined using Equation 9-28 of the AASHTO standard (1).

$$M_{cr} = (I/Y_t)\,(6\sqrt{f'_c} + f_{pe} - f_d)$$

where

M_{cr} = calculated cracking moment, in.-lb

I = moment of inertia of the uncracked section, in.^{4}

Y_t = distance from the centroid of the section to the bottom fiber, in.

f'_c = compressive strength of concrete, psi

f_{pe} = stress at extreme fiber due to effective prestressing force, psi

f_d = stress at extreme fiber due to self weight, psi

As indicated in Table 7, the observed cracking moment for each pile specimen exceeded the cracking moment computed using design values. Observed cracking moments also exceeded cracking moments calculated using actual measured material properties and prestress losses, indicating the AASHTO provisions are conservative with respect to determination of first cracking.

5.6.3 Evaluation of Flexure-Shear Cracking

The observed shear load resulting in flexure-shear cracking for each pile specimen was evaluated by comparison with computed values for the flexural shear strength provided by the concrete. For this evaluation, the flexural shear strength provided by the concrete, V_{ci}, was computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine V_{ci}. For the second comparison, actual measured material properties and prestress losses were used to calculate V_{ci}. Computed values for concrete flexural shear strength were determined for the section located at a distance "d" from the load point. Observed and computed concrete flexural shear strength for all three pile specimens are shown in Table 8.

TABLE 8

OBSERVED AND COMPUTED CONCRETE FLEXURAL SHEAR STRENGTH - P1, P2, AND P3

Specimen No.	Observed Concrete Flexural Shear Strength		Computed Concrete Flexural Shear Strength - Design		Computed Concrete Flexural Shear - Actual	
	kips	kN	kips	kN	kips	kN
P1	69.2	308	54.5	242	57.6	256
P2	67.0	298	54.5	242	58.7	261
P3	66.9	298	54.5	242	58.5	260

Observed shear load resulting in flexure-shear cracking for the three pile specimens was determined using the following equation:

$$V_{ci} = P/2 + V_d$$

where:

V_{ci} = observed shear load resulting in flexure-shear cracking, kips

P	=	total applied load coincident with the first flexure-shear crack, kips
V_d	=	dead load shear at section, kips

The computed concrete flexural shear strength, based on both design and actual conditions, was determined using Equation 9-27 of the AASHTO standard (1).

$$V_{ci} = 0.6 \sqrt{f'_c}\, b'\, d + V_d + (V_i\, M_{cr})/M_{max}$$

where:

V_{ci}	=	flexural shear strength provided by concrete, lb
f'_c	=	compressive strength of the concrete, psi
b'	=	width of the web (12 in.)
d	=	effective depth for shear, 0.8 times member depth, in.
V_d	=	dead load shear at the section under consideration, lb
M_{cr}	=	moment required to cause flexural cracking at the section under consideration, in.-lb
V_i	=	shear force at section under consideration, occurring simultaneously with M_{max}, lb
M_{max}	=	maximum moment at section under consideration, in.-lb

As indicated in Table 8, the observed shear load resulting in flexure-shear cracking for each pile specimen exceeded the concrete flexural shear strength computed using design values with Equation 9-27 of the AASHTO standard. The observed shear load resulting in flexure-shear cracking also exceeded the values calculated using actual measured material properties and prestress losses with Equation 9-27, indicating the AASHTO provisions are conservative with respect to determination of flexural shear strength provided by the concrete.

5.6.4 Evaluation of Flexural Strength

The observed flexural strength for each pile specimen was evaluated by comparison with computed values. For this evaluation, the calculated flexural strength for the pile specimens was computed using two methods. In one comparison, the design material properties and design total prestress losses were used to determine the flexural strength. For the second comparison, the flexural strength was computed using actual measured material properties and prestress losses in a strain compatibility analysis. Observed and computed flexural strengths for all three pile specimens are shown in Table 9.

TABLE 9

OBSERVED AND COMPUTED FLEXURAL STRENGTH - P1, P2, AND P3

Specimen No.	Observed Flexural Strength		Computed Flexural Strength - Design Conditions		Computed Flexural Strength - Strain Compatibility Analysis	
	kip-ft	kN-m	kip-ft	kN-m	kip-ft	kN-m
P1	794	1,077	636	863	771	1,047
P2	766	1,038	636	863	787	1,068
P3	769	1,043	636	863	786	1,067

Observed flexural strength for each pile specimen was determined using the following equation:

$$M_n = a\,(P/2) + M_{DL}$$

where:

M_n = observed flexural strength, in.-lb

P = total applied load at failure, lb

a = length of shear span, in

M_{DL} = dead load moment at location of failure, in.-lb

The computed flexural strength based on design conditions was determined using Equation 9-13 of the AASHTO standard (1) and assuming a strength reduction factor, ϕ, equal to 1.0.

$$M_n = A_s f_{su} d (1 - 0.6 \rho f_{su} / f'_c)$$

where:

M_n = nominal moment strength of section, in.-lb

A_s = area of prestressing steel in tension zone, in.^{2}

f_{su} = average stress in prestressing steel at ultimate load, psi

d = distance from extreme compression fiber to centroid of prestressing steel, in.

b = width of compression face of member, in.

ρ = A_s/bd, ratio of prestressing steel

f'_c = compressive strength of concrete, psi

As indicated in Table 9, the strain compatibility approach yielded values significantly closer to the observed values for the pile specimens than the more general AASHTO provisions. The AASHTO provisions for flexural strength yielded conservative results when used with design assumptions.

5.7 SUMMARY OF RESULTS

The concrete in piles P1, P2, and P3 failed to achieve the required minimum pile design compressive strength of 8,500 psi (58.6 MPa). The average 28-day compressive strength of piles P1, P2, and P3 was 7,400 psi (51.0 MPa), 7,900 psi (54.5 MPa), and 8,410 psi (58.0 MPa), respectively. As discussed in Section 3.3, trial high-strength concrete batches and tests conducted by the fabricator were successful. However, for the production of the piles, the type of cement was changed from that used in the trials. The concrete strength results from the pile specimens clearly demonstrated that the production of high-strength concrete requires a higher level of control than conventional concretes.

It is concluded that the pile specimens behaved in a manner that would be conservatively predicted using the provisions of the AASHTO Standard Specifications for Highway Bridges. It appears that the general equations given in the AASHTO specifications for modulus of elasticity and modulus of rupture as a function of compressive strength are acceptably conservative for the concrete strengths used in the pile specimens. The equations proposed in ACI 363-84 (28) are conservative for the modulus of elasticity and unconservative for the modulus of rupture. It is noted, however, that a limited number of concrete property specimens were tested and the conclusions drawn from those tests should be judged in light of the number of tests. The equations in the AASHTO specifications for moment capacity are conservative for members, such as the pile specimens, with prestressing strand in the compressive region. The use of a strain compatibility method provided a more precise method than the AASHTO provisions for prediction of flexural capacity. The equations in the AASHTO specifications for cracking moment and concrete flexural shear strength are also conservative. The present strand transfer length implied by the AASHTO provisions for development length gives reasonably good results for concrete of the strength tested in the piles. It appears that the actual prestress losses in the piles can be expected to be less than the total losses predicted using the AASHTO standard.

6. STATIC TESTS OF GIRDER SPECIMENS

6.1 FABRICATION OF GIRDER SPECIMENS

Three bulb-tee girder specimens, designated BT1, BT2, and BT3, were fabricated for the static tests. These girders were fabricated at Sherman Prestressed Concrete in Pelham, Alabama. Specific details of the girder specimen are shown in Figure 9. Supplemental materials and fabrication details are discussed in Appendix B.

The three bulb-tee specimens were 54-in. (1.37-m) deep and 70-ft (21.3-m) long. Each girder was prestressed using 30 uncoated, 1/2-in. (13-mm) diameter, Grade 270, low-relaxation 7-wire strands. Six of these strands were draped. Each prestressing strand was pretensioned to a stress level corresponding to 75% of guaranteed ultimate strength. During pretensioning, strand loads were verified through both measured elongations and the use of load cells on selected strands. Measured strand load correlated well with theoretical strand load determined based on measured elongation and strand modulus of elasticity.

The three girder specimens were pretensioned simultaneously (in series) within a single prestressing bed. The strand tensioning sequence started with the bottom row of strands and proceeded upward. The six draped strands were the last to be tensioned. Shear reinforcement and various inserts were installed after pretensioning the strands. Shear reinforcement in the girder specimens consisted of No. 4, Grade 40 bars at spacings ranging from 4 in. (102 mm) near the ends to 12 in. (305 mm) at mid-span. After all the steel elements were in place, the side forms were installed and secured.

Three concrete batches were used for each girder. The average ambient temperature during casting was approximately 40°F (14°C). After casting, the three girders were covered with a waterproof tarpaulin. After the concrete had achieved initial set, the girders were steam-cured at a temperature of approximately 140°F (60°C) for 24 hours. Approximately 10 hours after completion of the steam curing period, or 34 hours after initial set, the steel forms were removed from the girders. After form removal, a large crack was observed near mid-span through the full depth of each of the three girders. Crack widths were widest at the top flange and gradually diminished towards the bottom flange. According to plant personnel, the cracks are a typical occurrence in some of the deeper sections, such as the bulb-tee. Approximately 39 hours after initial set, the prestress was released by simultaneously torch cutting the strands at the far ends of the prestressing bed. After torch cutting the strands at the ends, the strands were cut within the spaces between the girders. After release, the crack observed near mid-span of each girder was virtually invisible.

Approximately nine days after fabrication, the three girders were loaded on semi-trailers and transported to CTL for testing. Upon arrival, the girders were off-loaded and placed on supports inside CTL's structural laboratory, which is maintained at a constant 73°F (23°C) and 50% relative humidity.

A deck slab was cast on girders BT1 and BT3 approximately 28 days and 65 days after fabrication, respectively. Deck slab dimensions and reinforcement details are shown in Figure 10. The deck slabs were cast using partially shored construction. Approximately 50% of the deck slab weight was supported by the girder while the remaining 50% was supported by the laboratory floor. This distribution of the deck slab dead load was taken into consideration when evaluating girder/composite section stresses.

6.2 GIRDER SPECIMEN INSTRUMENTATION

During the fabrication process, the girders were instrumented to measure concrete strains and camber. Internal strain indicators were installed at mid-span in both the upper and lower flanges of each girder to monitor prestress losses and concrete strains. External mechanical strain gauge reference points were installed at the ends of each girder for transfer length determination. For the camber measurements, permanent elevation reference points were embedded in the top surface of the top flange at the middle and ends of each girder. Girder instrumentation details are shown in Figure 11.

6.2.1 Internal Strain Gauges

Subsequent to pretensioning and prior to casting the concrete, three Carlson concrete strain meters were installed in each girder specimen at mid-span. As indicated in Figure 11, Carlson meters were installed in the lower flange, at the level of the centroid of the strand group. In addition to the Carlson meters, two 1/4-in. (6-mm) diameter reinforcing bars instrumented with weldable strain gauges were installed near the top and bottom surfaces of each girder, at mid-span. These gauges were used to provide an indication of concrete strains near these surfaces. Locations of the weldable strain gauges are also shown in Figure 11. Carlson meter and weldable bar strain gauge readings for girders BT1, BT2, and BT3 were taken prior to release, after release, at a concrete age of 28 days, before and after deck casting (where applicable), and prior to static testing. In addition to these readings, Carlson meter and weldable strain gauge readings for girder BT3 were also taken at various ages of loading during the long-term test.

6.2.2 Transfer Length Instrumentation

Prior to casting the girder concrete, rows of Whittemore gauge points were attached to the inside surface of the formwork. Whittemore points were installed along one side of the lower flange, at both ends of each of the three girders. Additional rows of Whittemore points were installed along one side of the web, coincident with the slope of the draped strands, at both ends of girder BT3 (long-term test specimen). After casting the concrete, the embedded rows of Whittemore points were detached from the formwork prior to form stripping and release of prestress. Locations of Whittemore points at the girder ends are shown in Figure 11.

Whittemore gauge points were spaced exactly 5 in. (127 mm) apart over a distance of 60 in. (1.52 m) from the ends of the girders. Using the various pairs of Whittemore points, concrete surface strains were measured using a 10-in. (254-mm) Whittemore gauge. Measured concrete surface strains were used to determine the strand transfer length required for full development of the prestress in the concrete. The longitudinal distance from the ends of the girders at which the concrete surface strains became somewhat uniform provided an indication of the transfer length.

Whittemore readings for girders BT1, BT2, and BT3 were taken prior to release (zero reading), after release, at a concrete age of 28 days, before and after deck casting (where applicable), and prior to static testing. In addition to these readings, Whittemore readings for girder BT3 were also taken at various ages of loading during the long-term test.

6.2.3 Girder Camber Instrumentation

Immediately after casting (while the concrete was still plastic) large steel bolts were embedded in the top surface of each girder at mid-span and near both ends to provide a permanent fixed reference point for future camber measurements. The embedded bolts near each end were centered above the sole plate. Camber measurements were made using a level to sight elevations at each reference point. Mid-span camber measurements relative to the ends for girders BT1, BT2, and BT3 were measured prior to release, after release, at a concrete age of 28 days, before and after deck casting (where applicable), and prior to static testing. Camber measurements for girder BT3 were also taken at various ages of loading during the long-term test.

6.3 GIRDER SPECIMEN MATERIAL PROPERTIES

6.3.1 Girder Concrete Material Properties

The concrete mix design developed by LTRC was used for the fabrication of girders BT1, BT2, and BT3. Specific mix design details are given in Table 10. As previously discussed in Chapter 3, the mix design was intended to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa).

TABLE 10

CONCRETE MIX DETAILS - BT1, BT2, AND BT3

Materials	Quantity/Cubic Yard
Cement	752 lb
Silica Fume	82 lb
Fine Aggregate	1,185 lb
Coarse Aggregate	2,030 lb
Water	27.1 gal.
Air Entrainment Admixture	7.1 oz.
Water Reducer	35.3 oz
Superplasticizer	209.4 oz.

Metric Equivalents:

1 lb = 0.454 kg
1 gal. = 3.785 l
1 oz. = 29.574 cc

Each girder specimen was fabricated using three separate concrete batches. The mix quantities of water, water reducer, and air entrainment admixtures varied slightly from batch to batch to maintain slump and air content within an acceptable range. Specific details for the individual concrete batches are given in Appendix B. Ten 6 x 12 in. (152 x 305 mm) concrete cylinders were made from each concrete batch in girders BT1 and BT2 for material property testing. Fifteen 6 x 12 in. (152 x 305 mm) concrete cylinders were made from each concrete batch in girder BT3 for material property testing. Two 6 x 6 x 20 in. (152 x 152 x 508 mm) concrete beams were made from the concrete batch placed in the mid-length region of each of the three girder specimens. Concrete strength material property tests were conducted at release and at concrete ages of 7, 28, 40, 56, and 660 days. In addition to the standard cylinder specimens, three 3 x 6 in. (76 x 152 mm) cores were taken from girders BT1 and BT2 and tested in compression at a concrete age of 40 days, and three 3 x 6 in. (76 x 152 mm) cores were taken from girder BT3 and tested in compression at a concrete age of 660 days.

Concrete compressive strength tests of the cylinder specimens were conducted in accordance with ASTM Designation: C39-86 (20). Compressive strength tests of the concrete cores were conducted in accordance with ASTM Designation: C42-90 (21). Modulus of elasticity tests were conducted in accordance with ASTM Designation: C469-87 (22). Splitting tensile strength tests were conducted in accordance with ASTM Designation: C496-90 (23). Modulus of rupture tests were conducted in accordance with ASTM Designation: C78-84 (24). Results of the concrete material property tests are given in Table 11 for the three girder specimens. Each value in Table 11 represents a single test result unless indicated otherwise.

Concrete unit weight was determined from one concrete core taken from girders BT1, BT2, and BT3. Unit weights for the concrete in girders BT1, BT2, and BT3 were 151.4, 146.3, and 147.5 lb/ft^3 (2425, 2343, and 2362 kg/m^3), respectively.

As indicated in Table 11, the average 28-day concrete compressive strength of each girder specimen failed to reach the required 28-day compressive strength of 10,000 psi (69.0 MPa). In addition, there was not much concrete strength gain since release. Petrographic (microscopic examination) of a single core sample from each girder was conducted in an effort to determine the cause of the slightly substandard 28-day strength. The

TABLE 11

CONCRETE MATERIAL PROPERTIES - BT1, BT2, AND BT3

Girder No.	Concrete Batch	Concrete Age, days	Compressive Strength, psi	Modulus of Elasticity, ksi	Splitting Tensile Strength, psi	Modulus of Rupture, psi
BT1	1-1	Release	10,200	6,450	No Test	No Test
		7	10,030	6,500	720	No Test
		28	10,240	6,400	680	No Test
		40 (Cyl.)*	10,300	No Test	650	No Test
		40 (Core)*	10,000	No Test	No Test	No Test
		56	10,300	6,200	670	No Test
	1-2	Release	9,090	6,900	No Test	No Test
		7	Bad Break	6,150	700	No Test
		28	9,710	6,100	670	No Test
		40 (Cyl.)*	9,570	No Test	670	810**
		40 (Core)*	9,720	No Test	No Test	No Test
		56	9,630	5,900	630	No Test
	1-3	Release	9,080	5,750	No Test	No Test
		7	9,220	5,800	690	No Test
		28	9,460	5,800	650	No Test
		40 (Cyl.)*	9,260	No Test	610	No Test
		40 (Core)*	8,560	No Test	No Test	No Test
		56	9,890	5,700	670	No Test
BT2	2-1	Release	9,550	5,850	No Test	No Test
		7	9,320	6,200	690	No Test
		28	9,600	5,900	740	No Test
		40 (Cyl.)*	9,650	No Test	770	No Test
		40 (Core)*	9,960	No Test	No Test	No Test
		56	9,850	5,950	760	No Test
	2-2	Release	9,430	6,000	No Test	No Test
		7	9,130	5,850	700	No Test
		28	9,750	6,150	690	No Test
		40 (Cyl.)*	9,900	No Test	790	840**
		40 (Core)*	9,630	No Test	No Test	No Test
		56	9,430	6,050	730	No Test
	2-3	Release	8,960	6,150	No Test	No Test
		7	8,990	5,800	650	No Test
		28	9,570	5,900	730	No Test
		40 (Cyl.)*	9,770	No Test	640	No Test
		40 (Core)*	10,230	No Test	No Test	No Test
		56	9,700	5,850	680	No Test

* Girder concrete age at planned start of flexural test for BT1 and BT2.

** Average of two beam tests.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

TABLE 11 (cont'd.)

CONCRETE MATERIAL PROPERTIES - BT1, BT2, AND BT3

Girder No.	Concrete Batch	Concrete Age, psi	Compressive Strength, psi	Modulus of Elasticity, ksi	Splitting Tensile Strength, psi	Modulus of Rupture, psi
BT3	3-1	Release	8,600	6,100	No Test	No Test
		7	9,370	6,150	680	No Test
		28	9,770	6,050	800	No Test
		56	9,730	5,900	730	No Test
		660 (Cyl.)*	9,370	No Test	650	No Test
		660 (Core)*	9,770	No Test	No Test	No Test
	3-2	Release	9,460	5,450	No Test	No Test
		7	9,720	6,150	700	No Test
		28	10,200	6,150	780	No Test
		56	10,110	6,150	740	No Test
		660 (Cyl.)*	10,180	No Test	660	890
		660 (Core)*	10,720	No Test	No Test	No Test
	3-3	Release	8,700	5,250	No Test	No Test
		7	9,520	6,100	570	No Test
		28	9,810	6,000	720	No Test
		56	10,080	6,150	640	No Test
		660 (Cyl.)*	9,430	No Test	660	No Test
		660 (Core)*	11,390	No Test	No Test	No Test

* Girder concrete age at planned start of flexural test for BT1 and BT2.

** Average of two beam tests.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

examination of the core from girder BT1 (from Batch 1-3) was made immediately following the static tests. The examination of the core from girder BT2 (from Batch 2-3) was made 12 months after completion of the static test. The examination of the core from girder BT3 (from Batch 3-3) was made immediately following the long-term test and flexural strength test. Petrographic examinations of the cores were conducted in accordance with ASTM Designation C856-83 (25). Results of the petrographic examinations did not identify any specific cause for the strength shortfall. Observed concrete void system from the air entrainment and paste characteristics were as expected for the mix design used. Concrete aggregates were uniformly distributed and the paste-aggregate bond was tight.

In addition to the concrete strength material property tests reported in Table 11, concrete cylinders from girder BT3 were also tested to determine creep, shrinkage, coefficient of thermal expansion and freeze-thaw properties. Creep and shrinkage tests were conducted in accordance with ASTM Designation: C512-87 (31). Coefficient of thermal expansion tests were conducted in accordance with a test procedure outlined in U.S. Army Corps of Engineers' Handbook for Concrete and Cement, Vol. 1 (32). Freeze-thaw tests were conducted in accordance with ASTM Designation: C666-90, Procedure A (33).

Six 6 x 12 in. (152 x 305 mm) cylinders were used for the creep and shrinkage tests. Creep and shrinkage tests were conducted using three different ages of loading (7, 28 and 56 days) in an environment maintained at 73°F (23°C) and 50% relative humidity. The test duration was 1 year. Two cylinders (one from Batch 3-1 and one from Batch 3-2) were used for each of the three test ages. The compression loads used for the tests were determined based on the average measured compressive strength of the two concrete batches at the age of loading. Average combined creep and shrinkage deformation and average shrinkage deformation alone are plotted versus time for loading ages of 7, 28, and 56 days in Figures 12, 13, and 14, respectively. Further discussion of the creep and shrinkage test results is given in

Section 6.10.2 which deals with the evaluation of long-term prestress losses in girder specimen BT3.

Three 6 x 12 in. (152 x 305 mm) cylinders were used for the tests to determine coefficient of thermal expansion. These cylinders were tested at three different concrete ages (7, 28, and 56 days). The averaged measured coefficients of thermal expansion at concrete ages of 7, 28, and 56 days were 5.9, 5.8, and 4.6 millionths/°F (10.6, 10.4, and 8.4 millionths/°C), respectively.

Three 3 x 3 x 11 in. (51 x 51 x 279 mm) concrete prisms were prepared from 6 x 12 in. (152 x 305 mm) cylinders and were used for freeze-thaw tests. Results of the freeze-thaw tests indicated an average durability factor of 1.0 for the three samples tested.

6.3.2 Prestressing Strand Material Properties

Prestressing strand used in the girder specimens was 1/2-in.-diameter, 7-wire, Grade 270, low-relaxation strand, conforming to ASTM Designation: A416-88 (26). Prestressing strand used in the girder specimens came from two different coils. Four samples of the prestressing strand were taken from the beginning and end of each coil for material property testing. Strand samples were tested in accordance with ASTM Designation: A370-90a, Supplement VII (27). Results of the prestressing strand material property tests are given in Table 12. All tested strands met the requirements of ASTM A416-88 (26).

6.3.3 Deck Slab Concrete Material Properties

As discussed in Section 4.3, the concrete mix design for the girder deck slabs was intended to yield concrete with a minimum 28-day compressive strength of 6,000 psi (41.4 MPa). Specific details regarding the deck slab concrete mix design are given in Appendix B. Three 8 yd^3 (6.1 m^3) batches were used to cast the deck slab for each of girders

TABLE 12

PRESTRESSING STRAND MATERIAL PROPERTIES - BT1, BT2, AND BT3

Coil No.	Sample	Area, sq in.	Load @ 1% Extension, lb	Breaking Strength, lb	Total Elongation, %	Modulus of Elasticity, ksi
78976	Beginning-1	0.153	39,600	43,100	6.7	28,700
	Beginning-2	0.153	40,500	44,100	6.4	28,900
	Beginning-3	0.153	40,200	43,800	7.5	30,950
	Beginning-4	0.153	40,500	43,600	6.5	30,800
	End-1	0.153	40,400	43,600	6.5	30,400
	End-2	0.153	-	43,600	7.4	29,950
	End-3	0.153	40,500	44,000	7.1	30,550
	End-4	0.153	40,000	43,200	7.2	28,450
78979	Beginning-1	0.153	40,400	43,800	7.7	30,200
	Beginning-2	0.153	40,600	43,800	6.3	31,950
	Beginning-3	0.153	40,200	43,800	6.1	28,850
	Beginning-4	0.153	40,400	43,900	7.3	30,550
	End-1	0.153	40,500	43,800	7.0	31,150
	End-2	0.153	40,200	43,800	6.7	29,600
	End-3	0.153	39,600	43,600	6.9	29,350
	End-4	0.153	40,100	43,700	6.6	29,800

Metric Equivalents:

1 sq in. = 645.2 sq mm
1 lb = 0.454 kg
1 ksi = 6.895 MPa

BT1 and BT3. Since the girders were oriented in the east-west direction for testing, the three deck slab concrete batches were designated as east batch, middle batch and west batch. Three 6 x 12 in. (152 x 305 m) concrete cylinders were made from each of the three concrete batches for material property testing.

Deck slab concrete material property tests for girders BT1 and BT3 were conducted at an age of 10 days and 603 days, respectively. Concrete compressive strength tests of the cylinder specimens were conducted in accordance with ASTM Designation: C39-86 (20). Modulus of elasticity tests were conducted in accordance with ASTM Designation: C469-87 (22). Results of the deck slab concrete material property tests are given in Table 13. Each value in Table 13 represents a single test result.

TABLE 13

DECK SLAB CONCRETE MATERIAL PROPERTIES - BT1 AND BT3

Girder No	Concrete Batch	Concrete Age	Compressive Strength, psi	Modulus of Elasticity, ksi
BT1	East	10 Days*	6,440 5,970 5,920	4,950 4,700 4,500
	Middle	10 Days*	7,370 7,560 7,640	4,700 4,800 4,700
	West	10 Days*	7,520 7,400 7,510	4,650 4,600 4,650
BT3	East	603 Days**	9,700 9,280 9,280	No Test 6,200 6,050
	Middle	603 Days**	10,150 10,030 8,820	No Test 5,800 4,850
	West	603 Days**	9,500 10,120 9,600	No Test 6,150 6,200

* Deck slab concrete age at start of flexural strength test for BT1.

** Deck slab concrete age at end of long-term test and at start of flexural strength test for BT3.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

6.4 DISCUSSION OF MATERIAL PROPERTIES AND EARLY-AGE BEHAVIOR

6.4.1 Material Properties

The AASHTO Standard Specification for Highway Bridges (1) and ACI 318-89 (2) include specific equations for calculating concrete modulus of elasticity and modulus of rupture in lieu of actual test data. These equations make use of established relationships between concrete compressive strength and other concrete properties developed based on test results from lower strength concretes. For this reason, ACI published ACI 363R-84, State-of-the-Art Report on High-Strength Concrete (28). This report introduces alternate equations for both modulus of elasticity and modulus of rupture proposed by Carrasquillo (16) for use with high-strength concrete. Table 4 (shown in Section 5.4.1) presents a comparison of the AASHTO/ACI 318 equations and the alternate equations of ACI 363.

Using the equations in Table 4 with the measured concrete compressive strength and unit weight, the mechanical properties of the concrete for girders BT1, BT2, and BT3 were calculated. In comparing the measure properties with computed properties, it is concluded that the measured results agree more closely with the AASHTO/ACI 318 values than with the ACI 363 values. The ACI 363 values tended to overestimate concrete modulus of rupture and underestimate concrete modulus of elasticity. The AASHTO/ACI 318 values tended to slightly underestimate both modulus of rupture and modulus of elasticity. The same conclusions were drawn for the three pile specimens (P1, P2, and P3), previously discussed in Section 5.4.1.

6.4.2 Transfer Length

As previously described, transfer length was experimentally evaluated for each girder specimen by measuring the change in concrete surface strains over a 5 ft (1.52 m) distance from the ends of each girder specimen. Concrete surface strains were measured using a 10-in.

(254-mm) Whittemore gauge. Whittemore gauge readings were taken just prior to release (reference reading), just after release, and at 28 days after release. Transfer lengths interpreted from the Whittemore gauge readings are presented in Table 14. Documented values represent an average based on measurements from both ends of each girder specimen. The average transfer lengths reported for girder BT3 were based on surface stain measurements along the bottom flanges as well as along the web (coincident with the draped strands).

TABLE 14

PRESTRESSING STRAND TRANSFER LENGTH - BT1, BT2, AND BT3

Girder Specimen	Concrete Age	Average Transfer Length, in.
BT1	Release 28 days	21.0 22.0
BT2	Release 28 days	21.0 23.0
BT3	Release (Flange) Release (Web) 28 Days (Flange) 28 Days (Web)	21.0 15.0 23.0 20.0

Metric Equivalent:
1 in. = 25.4 mm

A mathematical expression for calculating the required development length for prestressed strand is provided in both the AASHTO standard (1) and ACI 318-89 (2). According to the ACI 318-89 Commentary, the required development length is equal to the sum of the transfer length plus an addition length over which the strand must be bonded to ensure that bond failure does not occur at nominal strength of the member. An expression for the transfer length alone can be written as:

$$\text{Transfer Length} = (f_{se}/3)d_b$$

where

f_{se} = effective stress in prestressing strand after losses

d_b = nominal diameter of prestressing strand

Using the above expression and assuming prestress losses based on average readings from the Carlson strain meters, a transfer length of approximately 31 in. (787 mm) would be expected. As indicated in Table 14, transfer lengths interpreted from Whittemore readings were significantly less than the expected value.

6.4.3 Prestress Losses

The AASHTO Standard Specification for Highway Bridges contains provisions for calculating total prestress losses due to concrete shrinkage, elastic shortening, concrete creep, and steel relaxation. Based on these provisions and the design material properties, total prestress losses of approximately 53,154 psi (366.5 MPa) would be expected for the girders. However, only approximately 35-40% of the total ultimate concrete creep and shrinkage losses are expected to occur within the first 28 days (29). Therefore, prestress losses of approximately 33,422 psi (230.5 MPa) would be expected at a concrete age of 28 days.

Internal Carlson strain meters were installed in each girder specimen and monitored until a concrete age of 28 days. Measured concrete strains were used to calculate prestress losses by multiplying the average measured strains by the average measured strand modulus of elasticity of 30,000 ksi (207 GPa). Concrete strains measured just after release and at an age of 28 days, and corresponding calculated prestress losses, are shown in Table 15.

TABLE 15

MEASURED PRESTRESS LOSS AT RELEASE AND 28 DAYS - BT1, BT2, AND BT3

Girder Specimen	Specimen Age	Measured Strain, in./in.	Prestress Loss, psi
BT1	Release	473×10^{-6}	14,190
	28 days	666×10^{-6}	19,980
BT2	Release	484×10^{-6}	14,520
	28 days	733×10^{-6}	21,990
BT3	Release	346×10^{-6}	10,380
	28 days	548×10^{-6}	16,440

Metric Equivalent:
1 ksi = 1000 psi = 6.895 MPa

Concrete strains measured immediately after release were used to provide an indication of prestress losses due to elastic shortening. Based on these concrete strains, losses due to elastic shortening averaged 13,030 psi (89.8 MPa) between the three girder specimens. This value is in agreement with the average elastic shortening losses of 13,236 psi (91.3 MPa) calculated using provisions from the AASHTO standard and actual measured concrete and steel material properties.

Concrete strains measured at a concrete age of 28 days indicated prestress losses averaging approximately 19,470 psi (134.3 MPa) among the three girder specimens. This value is slightly less than the 31,209 psi (215.2 MPa) calculated using provisions from the AASHTO standard, actual measured concrete and steel material properties, and recommendations of the PCI Committee on Prestress Losses (29). Based on this information, it appears that the actual total prestress losses in the three girder specimens can be expected to be significantly less than the total losses predicted using the AASHTO standard. However, calculated values for prestress losses were based on the assumption that approximately 35% of the ultimate creep and 42% of the ultimate shrinkage will occur within the first 28 days. These percentages, although shown to be reasonable for conventional concretes, may not be applicable for higher strength concretes. This may explain the noted difference between measured and calculated prestress losses at 28 days.

6.4.4 Girder Camber

As discussed in Section 6.2, permanent fixed reference points were embedded in the top flange of the girders for camber measurements. Mid-span camber measurements relative to the ends of each girder were measured prior to release, after release, at a concrete age of 28 days, and after deck casting (where applicable). Previous research by Kelly, Bradberry, and Breen (5) found that camber and deflection of high-strength girders could be accurately predicted using moment area equations and known concrete properties. To verify this, camber was calculated for girders BT1, BT2, and BT3 using the moment-area method. Average measured material properties and prestress losses were used in the calculations. Camber calculations did not account for camber growth deflections due to concrete creep. Camber measurements and corresponding calculated deflections for girders BT1, BT2, and BT3 are documented in Table 16. Additional camber measurements made at various stages of the long-term test for girder BT3 are reported in Section 6.10.

As indicated in Table 16, measured camber at release correlated well with calculated values. However, due to early-age camber growth, measured camber at 28 days was significantly greater than the corresponding calculated deflection. Measured camber in girders BT1 and BT3 after adding the deck slab also correlated well with calculated values. Deflection due to the addition of the deck slab was calculated assuming that the deck dead load was distributed equally between the girder section and the composite section.

TABLE 16

MID-SPAN CAMBER MEASUREMENTS AND DEFLECTION CALCULATIONS - BT1, BT2, AND BT3*

Event	Girder BT1 Measured Camber	Girder BT2 Measured Camber	Girder BT3 Measured Camber	Calculated Deflection
Release	-0.69 in.	-0.69 in.	-0.67 in.	-0.83 in.
28 Days	-0.98 in.	-1.02 in.	-1.00 in.	-0.79 in.
With Deck	-0.44 in.	N/A	-0.55 in.	-0.44 in.

* Negative value indicates an upward camber.
Metric Equivalent:
1 in. = 25.4 mm

6.5 GIRDER SPECIMEN FLEXURAL TEST DETAILS

6.5.1 Flexural Test Setup

The test setup used for the flexural tests is shown in Figures 15 and 16. Girders were simply supported at the centerline of the end sole plates, creating a total span length of 69 ft (21.0 m). Girder specimens were tested in flexure using two point loads spaced 12 ft (3.66 m) apart at mid-span. Load was applied to the girders using hydraulic jacks. Load cells were used to monitor the applied load. Pairs of potentiometers were used to monitor girder displacements at both ends and at mid-span. Potentiometers were mounted on the two outer edges of the lower flange at each location. Load cells and potentiometers are calibrated annually. Crack width gauges were placed across the existing mid-span cracks that were noted prior to release. Dial gauges were placed on selected strands at each end of the girders to monitor any strand slippage relative to the concrete during the flexural test.

Internal strain gauges were used to monitor concrete strains continuously throughout the test. In addition to the internal gauges, concrete surface strain gauges were installed at mid-span on the top surface of the deck slab (girders BT1 and BT3) or upper flange (girder BT2) prior to flexural tests. Four concrete surface gauges were installed across the deck width of girders BT1 and BT3. Two concrete surface gauges were installed across the flange width of girder BT2. These gauges were also monitored continuously throughout the test.

6.5.2 Flexural Test Procedure

Load was applied to the girders in increments of approximately 2,000 lb (8.9 kN). Load cells, potentiometers and strain gauges were monitored continuously throughout each test using a digital data acquisition system (DDAS) and computer. At selected levels of applied load (load stages) data were stored on disk, providing a permanent record of test specimen behavior. Girder specimens BT1 and BT3 were initially loaded to produce a mid-span moment corresponding to the full design service load moment (D.L. + L.L. + Impact). Following this proof of design test, girders BT1 and BT3 were unloaded. Girders BT1, BT2, and BT3 were tested to determine flexural cracking moment, flexural shear cracking load, and ultimate moment capacity (failure). The tests were terminated when the girder could no longer sustain additional load.

6.6 ANALYSIS OF FLEXURAL TEST RESULTS

Girder specimens BT1 and BT2 were tested in flexure approximately 40 days after fabrication. Girder specimen BT3 was tested in flexure after the completion of the long-term test and 660 days after fabrication. Total applied load (not including girder weight) versus mid-span deflection is plotted in Figures 17, 18, and 19 for girder specimens BT1, BT2, and BT3, respectively. For all specimens, the deflections were measured from an initial upward cambered position of 0.44 in., 1.02 in., and 0.38 in. (11.2, 25. 9, and 9.7 mm) for girders BT1, BT2, and BT3, respectively.

6.6.1 Girder Specimen Behavior

Girder specimens BT1 and BT3 were initially subjected to a proof-of-design test where these girders were loaded to produce a mid-span moment corresponding to the full design service load moment (D.L. + L.L. + Impact). At the full design moment, girders BT1 and BT3 did not exhibit any new concrete cracking and the pre-existing crack noted during fabrication did not open. Girder BT1 deflected 0.48 in. (12.2 mm) at full design moment, thus the pre-test camber of 0.44 in. (11.2 mm) was completely negated at this loading. Girder BT3 deflected 0.41 in (10.4 mm) at full design moment, thus the pre-test camber of 0.38 in. (9.7 mm) was also completely negated at this loading. Measured deflections in girders BT1 and BT3 correlated well with calculated values for loads up to the full design service load moment. After unloading, both girders BT1 and BT3 returned to the original pre-test camber. Girders BT1 and BT3 were both judged to have performed adequately in the proof-of-design test.

After the proof-of-design test, girders BT1 and BT3 were tested to destruction. Girder BT2 was also tested to destruction, but was not used for an initial proof-of-design test since this girder did not have a deck slab. In the following discussions, cracks are classified into three categories: flexural cracks, flexure-shear cracks, and web-shear cracks. This terminology, as applied to prestressed concrete members, is defined by MacGregor (30) as follows:

> Flexural cracks are cracks that form when the normal applied stress exceeds the compression due to prestress and the modulus of rupture of the concrete in the vicinity of the crack.

Flexure-shear cracks are cracks that originate as flexure cracks in the shear spans of a member and become inclined toward the load point.

Web-shear cracks are inclined cracks that originate in the web of I-shaped members. The cracks result from excessive principal tensile stresses in the web.

The first flexural crack in girders BT1 and BT3 occurred prior to the opening of the pre-existing crack that was noted near mid-span prior to release. In girder BT2, the pre-existing crack near mid-span opened prior to the development of any new cracks. Therefore, cracking moment in girder BT2 was taken as the moment corresponding to the opening of the pre-existing crack. The observed total applied loads corresponding to the formation of the initial flexural crack were 224.0, 189.3, and 212.7 kips (996, 842, and 946 kN) for girder specimens BT1, BT2, and BT3, respectively. Initially, several flexural cracks developed within the constant moment region. Spacing of these cracks approximately coincided with the locations of the stirrups used for shear reinforcement. Additional cracks developed along the shear spans as the load was increased.

The cracks in the shear spans propagated upward into the web and then became inclined toward the load points. These cracks are termed flexure-shear cracks, since they originated as flexural cracks in the shear span. The total applied load at which flexure-shear cracking occurred was interpreted to be the load at which the inclined (angle of approximately 45 degrees) portion of the flexure-shear crack first reached mid-depth of the section. According to previous research (20), the flexure-shear crack which leads to failure of a specimen typically originates in the shear span at a distance from the load point corresponding approximately to the effective depth, d, of the member. The effective depth at mid-span for girders BT1, BT2, and BT3 was 60.9, 49.9, and 60.9 in. (1.547, 1.267, and 1.547 m), respectively. The crack identified as the first flexure-shear crack for each girder specimen was traced back to its origin on the tension face of the girder. Consistent with previous research, the origin of these cracks occurred at a distance approximately equal to "d" from the load point. The total applied load corresponding to the formation of initial flexure-shear cracks was 279.6, 274.5, and 274.7 kips (1,244, 1,221, and 1,222 kN) for girder specimens BT1, BT2, and BT3, respectively.

As the test progressed, additional flexure cracks and flexural shear cracks developed and existing cracks continued to propagate. Typical extent of flexural cracking in the constant moment region of girders with and without a deck slab is shown in Figure 20. Typical extent of flexure-shear cracking in the shear spans of girders with and without a deck slab is shown in Figure 21. Web shear cracks did not occur in any of the girder specimens during the flexural tests. The load-deflection curve peaked at a total applied load of 408.6, 294.2, and 410.0 kips (1,818, 1,309, and 1,824 kN) for girder specimens BT1, BT2, and BT3, respectively. The deflection coincident with the peak load was 24.00, 4.95, and 22.40 in. (610, 126, and 569 mm) for girder specimens BT1, BT2, and BT3, respectively.

Flexural tension failure within the constant moment region, evidenced by audible indications of strand breakage, marked the limit of the load capacity of girder specimens BT1 and BT3 (girder specimens with deck slabs). A photograph of girder BT1 at ultimate load is shown in Figure 22. Flexural compression failure of the top flange marked the limit of the load capacity for girder BT2. Girder BT2 buckled laterally coincident with achieving the ultimate moment. A photograph of the failure zone is shown in Figure 23.

The three Carlson strain meters and two weldable strain gauges placed in the lower flange of girders BT1, BT2, and BT3 performed well until flexural cracking occurred. Once the girders cracked, all of the strain indicators in the lower flange ceased to provide meaningful data. Prior to cracking, there was good correlation between all the strain indicators in the lower flange. At a given load, strains indicated by the weldable gauges were slightly greater than those indicated by the Carlson meters. This behavior was expected since the weldable gauges were located slightly closer to the extreme fiber of the lower flange.

The two weldable strain gauges in the upper flange of girders BT1, BT2, and BT3 provided meaningful data throughout the flexural test. At the start of the flexural test, measured strains in the top flange of all three girders indicated a state of slight compression relative to the initial strain readings taken prior to release of prestress. During the flexural test, existing compressive strains in the upper flange of girders BT1, BT2, and BT3 increased as the load increased. However, during the later stages of the tests of girders BT1 and BT3 (girders with deck slab) the strains in the top flange gradually reversed from compression to tension as the neutral axis of the composite section moved up into the deck slab.

Data from the four deck slab concrete surface strain gauges on girders BT1 and BT3 indicated that the full width of the deck slab was effective with no apparent shear lag. Load versus deck slab surface strain plots for girders BT1 and BT3 are shown in Figures 24 and 25, respectively. The two concrete surface strain gauges on the extreme fiber of the upper flange of girder BT2 indicated uniformly distributed strain across the top flange, as expected. However, the concrete strains coincident with failure were not as high as would be expected for a flexural compression failure (average strain of approximately 1350 millionths). Therefore, it appears that failure occurred as a result of the lateral buckling rather than flexural compression.

6.6.2 Evaluation of Flexural Cracking Moment

The observed flexural cracking moment for each girder specimen was evaluated by comparison with computed values. For this evaluation, the flexural cracking moment was computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine the calculated flexural cracking moment. For the second comparison, actual measured material properties and prestress losses were used to calculate the cracking moment. Observed and computed flexural cracking moments for all three girder specimens are shown in Table 17.

TABLE 17

OBSERVED AND COMPUTED FLEXURAL CRACKING MOMENT - BT1, BT 2, AND BT3

Specimen No.	Observed Cracking Moment		Computed Cracking Moment - Design Condition		Computed Cracking Moment - Actual Condition	
	kip-ft	kN-m	kip-ft	kN-m	kip-ft	kN-m
BT1	3,192	4,332	2,384	3,235	3,066	4,161
BT2	2,698	3,661	2,189	2,970	2,712	3,680
BT3	3,031	4,114	2,384	3,235	3,190	4,329

Observed cracking moment for the three girder specimens was determined using the observed cracking load and the following equation:

$$M_{cr} = a\,(P/2)$$

where

M_{cr} = observed cracking moment, in.-lb

a = length of shear span, in.

P = total applied load coincident with the first flexural crack, lb

The computed cracking moment, based on both design and actual conditions, was determined using Equation 9-28 of the AASHTO standard (1).

$$M_{cr} = (I/Y_t)\,(6\sqrt{f'_c} + f_{pe} - f_d)$$

where

M_{cr} = calculated cracking moment, in.-lb

I = moment of inertia of the uncracked section, in.^{4}

Y_t = distance from the centroid of the section to the bottom fiber, in.

f'_c = compressive strength of concrete, psi

f_{pe} = stress at extreme fiber due to effective prestressing force, psi

f_d = stress at extreme fiber due to self weight, psi

As indicated in Table 17, the observed cracking moment for each girder specimen exceeded the cracking moment computed using design values. Observed cracking moments correlated well with cracking moments calculated using actual measured material properties and prestress losses, indicating the use of AASHTO provisions are appropriate with respect to determination of first cracking.

6.6.3 Evaluation of Flexure-Shear Cracking

The observed shear load resulting in flexure-shear cracking for each girder specimen was evaluated by comparison with computed values for the flexural shear strength provided by the concrete. For this evaluation, the flexural shear strength provided by the concrete, V_{ci}, was computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine V_{ci}. For the second comparison, actual measured material properties and prestress losses were used to calculate V_{ci}. Computed values for concrete flexural shear strength were determined for the section located at a distance "d" from the load point. Observed and computed concrete flexural shear strength for all three girder specimens are shown in Table 18.

TABLE 18

OBSERVED AND COMPUTED CONCRETE FLEXURAL SHEAR STRENGTH - BT1, BT2, AND BT3

Specimen No.	Observed Concrete Flexural Shear Strength		Computed Concrete Flexural Shear Strength - Design		Computed Concrete Flexural Shear Strength - Actual	
	kips	kN	kips	kN	kips	kN
BT1	160.2	713	143.6	639	159.6	710
BT2	144.5	643	114.0	507	128.7	573
BT3	157.7	702	143.6	639	164.0	730

Observed shear load resulting in flexure-shear cracking for the three girder specimens was determined using the following equation:

$$V_{ci} = P/2 + V_d$$

where:

V_{ci} = observed shear load resulting in flexure-shear cracking, lb

P = total applied load coincident with the first flexure-shear crack, lb

V_d = dead load shear at section, lb

The computed concrete flexural shear strength, based on both design and actual conditions, was determined using Equation 9-27 of the AASHTO standard (1).

$$V_{ci} = 0.6 \sqrt{f'_c}\, b' d + V_d + (V_i M_{cr})/M_{max}$$

where:

V_{ci} = flexural shear strength provided by concrete, lb

f'_c = compressive strength of the concrete, psi

b' = width of the web (6 in.)

d = effective depth, in.

V_d = dead load shear at the section under consideration, lb

M_{cr} = moment required to cause flexural cracking at the section under consideration, in.-lb

V_i = shear force at section under consideration, occurring simultaneously with M_{max}, lb

M_{max} = maximum moment at section under consideration, in.-lb

As indicated in Table 18, the observed shear load resulting in flexure-shear cracking for each girder specimen exceeded the concrete flexural shear strength computed using design values with Equation 9-27 of the AASHTO standard. The observed shear load resulting in flexure-shear cracking correlated well with the values calculated using actual measured material properties and prestress losses with Equation 9-27, indicating the AASHTO provisions are appropriate with respect to determination of flexural shear strength provided by the concrete.

6.6.4 Evaluation of Flexural Strength

The observed flexural strength for each girder specimen was evaluated by comparison with computed values. For this evaluation, the calculated flexural strength for the girder specimens was computed using two methods. In one comparison, the design material properties and design total prestress losses were used to determine the flexural strength. For the second comparison, the flexural strength was computed using actual measured material properties and prestress losses in a strain compatibility analysis. Observed and computed flexural strengths for all three girder specimens are shown in Table 19.

TABLE 19

OBSERVED AND COMPUTED FLEXURAL STRENGTH - BT1, BT2, AND BT3

Specimen No.	Observed Flexural Strength		Computed Flexural Strength - Design Conditions		Computed Flexural Strength - Strain Compatibility Analysis	
	kip-ft	kN-m	kip-ft	kN-m	kip-ft	kN-m
BT1	6,977	9,461	6,086	8,252	6,575	8,915
BT2	4,601	6,238	4,829	6,548	5,073	6,878
BT3	6,996	9,495	6,086	8,252	6,596	8,943

Observed flexural strength for each girder specimen was determined using the following equation:

$$M_n = a\,(P/2) + M_{DL}$$

where:

M_n = observed flexural strength, in.-lb

P = total applied load at failure, lb

a = length of shear span, in.

M_{DL} = dead load moment at location of failure, in.-lb

The computed flexural strength based on design conditions was determined using Equation 9-13 of the AASHTO standard (1) and assuming a strength reduction factor, ϕ, equal to 1.0:

$$M_n = A_s f_{su} d (1 - 0.6 \rho f_{su} / f_c')$$

where:

M_n = nominal moment strength of section, in.-lb

A_s = area of prestressing steel in tension zone, in.^{2}

f_{su} = average stress in prestressing steel at ultimate load, psi

d = distance from extreme compression fiber to centroid of prestressing steel, in.

b = width of compression face of member, in.

ρ = A_s/bd, ratio of prestressing steel

f_c' = compressive strength of concrete, psi

As indicated in Table 19, the strain compatibility approach provided a slightly more precise prediction of flexural strength for girder specimens BT1 and BT3 than the more general AASHTO provisions. The AASHTO provisions for flexural strength yielded conservative results for girders BT1 and BT3 when used with design assumptions. Comparison of the observed flexural strength of girder specimen BT2 with the computed strengths confirms that girder BT2 did not experience a true flexural failure.

6.7 GIRDER SPECIMEN SHEAR TEST DETAILS

6.7.1 Shear Test Setup

Following the flexural test, girders BT1 and BT2 were cut in half and the ends were used for shear tests. The test setup for the shear tests was designed to cause web shear cracking and a shear failure at the natural end (not the cut end) of the girder halves. The test setup for the shear tests is shown in Figures 26 and 27. Each half girder was simply supported over a span of 27 ft (8.2 m). Four concentrated loads were used. Load was applied to the girders using hydraulic jacks. Load cells were used to monitor the applied load. Pairs of potentiometers were used to monitor the girder displacements at both ends and at mid-span. Potentiometers were mounted on the two outer edges of the lower flange at each location. Load cells and potentiometers are calibrated annually. Dial gauges were placed on selected strands at the loaded end of the girder to monitor any strand slippage relative to the concrete during the shear test.

6.7.2 Shear Test Procedure

Load was applied to the girders in increments of approximately 5,000 lb (22.2 kN). Load cells and potentiometers were monitored continuously throughout each test using a digital data acquisition system (DDAS) and computer. At selected levels of applied load (load stages), data were stored on disk, providing a permanent record of test specimen behavior. Each girder half was tested to determine web shear cracking load and maximum shear load capacity. The tests were terminated when the girder could no longer sustain additional load.

6.8 ANALYSIS OF SHEAR TEST RESULTS

As previously mentioned, girders BT1 and BT2 were cut in half and the ends were used for shear tests. The shear test specimens were given the designations BT1-D, BT1-L, BT2-D, and BT2-L. The "L" in the designation denotes the girder end closest to the live or jacking end of the prestressing bed as established during fabrication. The "D" in the designation denotes the girder end closest to the dead end of the prestressing bed as established during fabrication. One of the halves of girder BT2 (BT2-D) sustained excessive damage during the flexural test and was subsequently not used for shear testing. Therefore, three girder ends were tested in shear, two with a deck slab and one without a deck slab.

6.8.1 Girder Specimen Behavior

During the shear tests, web shear cracks developed between the first point load and the natural end of each specimen. Observed web shear cracking load was interpreted to be the load at which a diagonal web crack, not initiated as a flexural crack, was first observed. The total shears between the point load and the natural end corresponding to the formation of the initial web shear cracks were 422.7, 416.0, and 327.6 kips (1,880, 1,850, and 1,457 kN) for specimens BT1-L, BT1-D, and BT2-L, respectively.

As the tests of specimens BT1-L and BT1-D (girder halves with a deck slab) progressed, existing diagonal cracks at the cut end, which initiated during the flexural test of girder BT1, propagated further and eventually led to failure at the cut end of the specimen. As shown in Figure 28, the shear-compression failure mode of specimens BT1-L and BT1-D was most likely influenced by damage that had occurred during the flexural test. Therefore, ultimate loads obtained during these tests did not represent the true shear strength. As a result of this, only the web shear cracking load was evaluated for specimens BT1-L and BT1-D.

As the test of specimen BT2-L (girder half without a deck slab) progressed, several additional web shear cracks developed near the natural end. Cracks did not develop near the cut end of the specimen during the test. As shown in Figure 29, web shear cracks that developed at the natural end eventually led to shear-compression failure at that end. The total shear between the first point load and the natural end coincident with the failure was 455.1 kips (2024 kN). However, the failure mode of BT2-L was influenced by bond failure of prestressing strand as evidenced by observed slippage of the prestressing strands. Since the failure mode of specimen BT2-L was influenced by bond failure, measured ultimate shear load must be considered as an indication of minimum shear strength.

6.8.2 Evaluation of Web Shear Cracking

The observed shear load resulting in web shear cracking for each specimen was evaluated by comparison with computed values for the shear strength provided by the concrete. For this evaluation, the shear strength provided by the concrete, V_{cw}, was computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine V_{cw}. For the second comparison, actual measured material properties and prestress losses were used to calculate V_{cw}. Observed and computed concrete shear strength for all three specimens tested are shown in Table 20.

TABLE 20

OBSERVED AND COMPUTED CONCRETE SHEAR STRENGTH - BT1 AND BT2

Specimen	Observed Concrete Shear Strength		Computed Concrete Shear Strength - Design Condition		Computed Concrete Shear Strength - Actual Condition	
	kips	kN	kips	kN	kips	kN
BT1-D	422.7	1,880	232.7	1,035	260.9	1,161
BT1-L	416.0	1,851	232.7	1,035	260.9	1,161
BT2-L	327.6	1,457	189.7	844	212.0	943

Observed shear load resulting in web shear cracking for the three girder specimens was determined using the following equation:

$$V_{cw} = P/2 + V_d$$

where:

V_{cw} = observed shear load resulting in web shear cracking, lb

P = total applied load coincident with the first web shear crack, lb

V_d = dead load shear at section, lb

The computed concrete shear strength, based on both design and actual conditions, was determined using Equation 9-29 of the AASHTO standard (1).

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3 f_{pc}) b'd + V_p$$

where:

V_{cw} = shear strength provided by concrete, lb

f'_c = compressive strength of the concrete, psi

f_{pc} = compressive stress in concrete at centroid of section, psi

b' = width of the web, in.

d = effective depth, in.

V_p = vertical component of effective prestress force at section, lb

As indicated in Table 20, observed web shear cracking loads were greater than values computed using the AASHTO Equation 9-29 for both design conditions and actual conditions. Therefore, the AASHTO equation yielded a conservative prediction of the shear strength provided by the concrete.

6.8.3 Evaluation of Shear Strength

As discussed in Section 6.8.1, shear strength was documented for specimen BT2-L only. However, since there was evidence of bond failure during the test of BT2-L, the shear load at the time of failure must be considered as an indication of minimum shear strength. The laboratory measured value of applied shear at the time of failure was 455.1 kips (2,024 kN).

The observed shear strength for specimen BT2-L was evaluated by comparison with computed values. For this evaluation, the shear strength provided by the concrete, V_{cw}, and the shear strength provided by the reinforcement, V_s, were computed two different ways. In one comparison, the design material properties and design total prestress losses were used to determine V_{cw} and V_s. For the second comparison, actual material properties and prestress losses were used to calculate V_{cw} and V_s

The shear strength at the location of failure was determined using the following equation from both the AASHTO specification and ACI standard:

$$V_n = V_c + V_s$$

where:

V_n = nominal shear strength of a section, lb

V_c = shear strength provided by the concrete, lb

V_s = shear strength provided by the reinforcement, lb

The calculated value of shear strength based on design conditions was 372.5 kips (1,657 kN). The calculated value of shear strength based on actual conditions was 452.0 kips (2,011 kN). These test results indicate that even though the member experienced bond failure at the end, the AASHTO/ACI equations yielded a conservative answer.

6.9 GIRDER SPECIMEN LONG-TERM TEST DETAILS

6.9.1 Long-Term Test Setup

The test setup for the long-term test is shown in Figures 30 and 31. Girder BT3 was simply supported at the centerline of the sole plates, creating a total span length of 69 ft (21.0 m). The high-strength concrete bridge design prepared by LTRC resulted in a girder spacing of 13.3 ft (4.1 m). Since girder BT3 only had a 10-ft (3.1-m) wide deck slab, additional load representing an additional 3.3-ft (1.0-m) width of deck slab was required in order to simulate the full design dead load conditions. Additional dead load was applied to the girders using concrete blocks to produce a series of concentrated loads. Locations and weights of the concrete blocks were selected to produce bending moments similar to those resulting from the uniform design dead load. The deck slab was cast on girder BT3 65 days after fabrication. The additional dead load was applied to the girder 69 days after fabrication.

6.9.2 Long-Term Test Procedure

The purpose of the long-term test was to monitor the behavior of girder BT3 under full design dead over an 18-month time period. During the 18-month test period, internal concrete strains, transfer length based on concrete surface strain, and girder camber were measured at ages of 6 months, 12 months, and 18 months after applying the load.

6.10 ANALYSIS OF LONG-TERM TEST RESULTS

6.10.1 Evaluation of Long-Term Transfer Length

The procedure for determining transfer length is discussed in Section 6.4.2. Transfer lengths were determined from concrete surface strain measurements taken along both the bottom flange and the web (coincident with the draped strands) at each end of girder BT3.

Average transfer length determined at release and at concrete age of 28 days are reported in Table 14. Average transfer lengths determined at various stages of the long-term test are reported in Table 21. As previously discussed in Section 6.4.2, a transfer length of approximately 31 in. would be expected. As indicated in Table 21, transfer lengths interpreted from Whittemore readings during the course of the long-term test were less than the expected value and did not change significantly from the values reported at release and 28 days.

TABLE 21

PRESTRESSING STRAND LONG-TERM TRANSFER LENGTHS - BT3

Girder Specimen	Age After Loading	Average Transfer Length, in.
BT3	Initial* (Flange)	23
	Initial* (Web)	25
	6 months (Flange)	23
	6 months (Web)	25
	12 months (Flange)	23
	12 months (Web)	25
	18 months (Flange)	23
	18 months (Web)	25

* Age of girder at initial loading was 69 days.

Metric Equivalent:
1 in. = 25.4 mm

6.10.2 Evaluation of Long-Term Prestress Losses

As discussed in Section 6.4.3, actual prestress losses were determined by multiplying the average measured concrete strains by the average measured strand modulus of elasticity of 30,000 ksi (207 GPa). Measured concrete strains were used to determine early-age prestress losses in girders BT1, BT2, and BT3, and also to determine long-term prestress losses in girder BT3. As reported in Table 15, measured average early-age prestress loss for girder BT3 was 16,440 psi (113.4 MPa) at 28 days. Measured prestress losses at various stages of the long-term test are reported in Table 22. As indicated by the data in Tables 15 and 22, measured prestress loss during the long-term test did not change substantially from the value reported at 28 days.

TABLE 22

MEASURED LONG-TERM PRESTRESS LOSS - BT3

Girder Specimen	Age After Loading	Measured Strain, in./in.	Prestress Loss, psi
BT3	Initial*	600×10^{-6}	18,015
	6 Months	658×10^{-6}	19,745
	12 Months	727×10^{-6}	21,805
	18 Months	766×10^{-6}	22,986

* Age of girder at initial loading was 69 days.

Metric Equivalent:

1 ksi =1000 psi = 6.895 MPa

After 18 months of being subjected to full design dead load, measured average total prestress loss in girder BT3 was 22,986 psi (158.5 MPa). Based on the AASHTO provisions for calculating prestress losses and the design material properties, total prestress losses of approximately 53,154 psi (366.5 MPa) would be expected for the girder. Therefore, actual measured prestress losses in girder BT3 were significantly less than those predicted using the provisions of the AASHTO standard. Since measured elastic shortening prestress losses in girder BT1, BT2, and BT3 were in reasonable agreement with those calculated using the AASHTO provisions, and strand relaxation loss represents only a very small portion of the total prestress loss, it appears reasonable to conclude that the AASHTO provision for calculating creep and shrinkage prestress losses are overly conservative for high-strength concrete.

Recommendations of the PCI Committee on Prestress Losses suggest that approximately 74% of the ultimate creep and 86% of the ultimate shrinkage will occur within the first year (29). Based on this assumption, the AASHTO provisions for calculating prestress losses, and the actual measured concrete and steel material properties, prestress losses of approximately 43,953 psi (303.1 MPa) would be expected at 12 months. Actual measured prestress losses in girder BT3 at both 12 months and 18 months were approximately 50% less than the expected value.

Measured creep deformations in 6 x 12. in. (152 x 305 mm) cylinders representing the concrete in girder BT3 are consistent with the findings regarding the measured prestress losses in girder BT3. As indicated in Figures 12, 13, and 14, measured cylinder creep deformations after 1 year for the three ages of loading were consistent and averaged 426 x 10^{-6} in./in. This average value reflects a creep coefficient, defined as the ratio of creep strain to initial strain, of approximately 1.11. According to ACI Committee 209, the ultimate creep coefficient of concrete normally falls into the range of 1.30 to 4.15 (34). The creep coefficient after 1 year reflected by the cylinder data fell below the expected range. Even if only 74% of the ultimate creep actually occurred in the first year, as suggested by the PCI Committee on Prestress Losses, the cylinder data still suggest that the ultimate creep will be at the low end of the expected range.

As further indicated in Figures 12, 13, and 14, measured cylinder shrinkage deformations after 1 year of measurements averaged 262 x 10^{-6} in./in. According to ACI Committee 209, the ultimate shrinkage strain will normally fall into the range of 415 x 10^{-6} in./in. to 1070 x 10^{-6} in./in. (34). According to the PCI Committee on Prestress Losses, approximately 22% and 86% of the ultimate shrinkage can be expected to occur within 7 days and one year after completion of initial curing, respectively. Assuming that the shrinkage deformations measured in the cylinders represents only 64% of the ultimate shrinkage (86% - 22%), the expected ultimate shrinkage implied by the cylinder shrinkage deformations would be 409 x 10^{-6} in./in. This implied ultimate shrinkage falls on the low end of expected range developed by ACI Committee 209.

While the measured prestress losses in girder BT3 and the measured cylinder creep and shrinkage deformations both support the conclusion that the total prestress loses in girder BT3 will be significantly less than those predicted by the AASHTO provisions, further research into the creep and shrinkage characteristics of high-strength concrete would be required to determine how the AASHTO provisions for creep and shrinkage prestress losses should be modified for high-strength concrete.

6.10.3 Evaluation of Long-Term Camber

The procedure for determining girder camber is discussed in Section 6.4.4. Girder BT3 had an upward camber of 0.55 in. (14.0 mm) after casting the deck slab. Prior to starting the long-term test, and after applying the additional dead load, girder BT3 had an upward camber of 0.44 in. (11.2 mm). Camber measurements taken at various stages of the long-term test are reported in Table 23. As indicated by the camber data, measured camber changed slightly during the first 6 months of the long-term test. However, no substantial change in the measured camber occurred over the final 12 months of the test. After the long-term test was completed and the additional dead load was removed, girder BT3 had an upward camber of 0.34 in. (8.6 mm).

TABLE 23

MEASURED LONG-TERM MID-SPAN CAMBER - BT3

Girder Specimen	Age After Loading	Measured Camber*
BT3	Initial* 6 Months 12 Months 18 Months	-0.44 in. -0.27 in. -0.30 in. -0.29 in.

Age of girder at initial loading was 69 days

Metric Equivalent:

1 in. = 25.4 mm

6.11 SUMMARY OF RESULTS

The concrete in girders BT1, BT2, and BT3 failed to achieve the required minimum girder design 28-day compressive strength of 10,000 psi (69.0 MPa). The average 28-day compressive strengths of girders BT1, BT2, and BT3 were 9,800, 9,640, and 9,930 psi (67.6, 66.5, and 68.5 MPa), respectively. The concrete in girders BT1, BT2, and BT3 was steam-cured for 24 hours after initial set. As previously discussed in Section 3.3, use of steam-curing was not part of the development of the mix design used for BT1, BT2, and BT3. Based on the strength results from girders BT1, BT2, and BT3, it is concluded that the steam-curing may have hindered the 28-day strength of the concrete used for girders BT1, BT2, and BT3.

It is concluded that the girder specimens behaved in a manner that would be conservatively predicted using the provisions of the AASHTO Standard Specifications for Highway Bridges as they relate to the following:

Concrete Modulus of Elasticity

Concrete Modulus of Rupture

Flexural Strength

Flexural Cracking Moment

Concrete Flexural Shear Strength

Concrete Web Shear Strength

Shear Strength

Strand Transfer Length

Estimation of Prestress Losses

It appears that the general equations given in the AASHTO specifications for concrete modulus of elasticity and modulus of rupture as a function of compressive strength are acceptably conservative for the concrete strengths used in the girder specimens. The equations proposed in ACI 363R-84 (28) are conservative for the concrete modulus of elasticity and not conservative for the concrete modulus of rupture. It is noted, however, that a limited number of concrete property specimens were tested and the conclusions drawn from those tests should be judged in light of the number of tests.

The equations in the AASHTO specifications for moment capacity are conservative for members, such as the girder specimens. The use of a strain compatibility method provided a slightly more precise prediction of flexural capacity than the AASHTO provisions. The equations in the AASHTO specifications for cracking moment, concrete flexural shear strength, concrete web shear strength, and total shear strength were also conservative.

The present strand transfer length implied by the AASHTO provisions for development length were conservative and reasonably consistent with the measured transfer length in the girders. It appears that the actual prestress losses in the girders can be expected to be significantly less than the total losses predicted using the AASHTO standard. Measured mid-span girder camber generally correlated reasonably well with deflections calculated using conventional techniques.

7. FIELD EVALUATION OF 130-FT (39.6-M) PILE SPECIMEN

7.1 FABRICATION OF PILE SPECIMEN

One pile specimen, designated P4, was fabricated for field evaluation. This pile was fabricated at Sherman Prestressed Concrete in Mobile, Alabama. Specific details of the pile specimen are shown in Figure 1. Supplemental materials and fabrication details are discussed in Appendix C.

The cross section of the pile specimen was nominally 24-in. (610-mm) square with a centrally-located 12-in-diameter circular void. The pile specimen was 130-ft (39.6-m) long. Twenty-four uncoated, 1/2-in. (13-mm) diameter, Grade 270, low-relaxation 7-wire strands were used to prestress the pile specimen. The strands were placed symmetrically around the perimeter of the cross-section of the pile specimen. Following placement, the strands were pretensioned to a stress level corresponding to 75% of guaranteed ultimate strength. The strands across the bottom of the pile were tensioned first, followed by the two sides, and finally the top. Transverse reinforcement in the pile specimen consisted of W4.5 spiral wire. Three lifting inserts were placed in the pile for handling.

The pile specimen was cast using five 4 cu. yd. (3.1 m^3) concrete batches. The average ambient temperature during casting was approximately 80°F (27°C). After casting, the pile specimen was covered with a waterproof tarpaulin. After the concrete had achieved initial set, the pile specimen was steam-cured for 18 hours at a temperature of approximately 140°F (60°C). Approximately 20 hours after casting (2 hours after completion of steam curing period), the prestress was released by simultaneously torch cutting the strands at each end of the prestressing bed. Immediately following release of the strands, the pile was lifted out of the form using three point pick-up and transported to another location in the casting yard.

7.2 PILE SPECIMEN MATERIAL PROPERTIES

7.2.1 Concrete Material Properties

For the fabrication of the 130-ft (39.6-m) pile specimen, the concrete mix design developed by LTRC was modified slightly at the discretion of the fabricator. Specific mix design details are given in Table 24. As previously discussed in Chapter 3, the mix design was intended to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69 MPa).

TABLE 24

CONCRETE MIX DETAILS - P4

Materials	Quantity/Cubic Yard
Cement	750 lb
Silica Fume	95 lb
Fine Aggregate	1,030 lb
Coarse Aggregate	1,875 lb
Water	27.6 gal.
Air Entrainment Admixture	3.0 oz.
Water Reducer	10.0 oz
Superplasticizer	85.0 oz.

Metric Equivalents:

1 lb = 0.454 kg

1 gal. = 3.785 l

1 oz. = 29.574 cc

The pile specimen was fabricated using five separate concrete batches. The mix proportions varied slightly among the five concrete batches to maintain slump and air content within an acceptable range. Specific details for the individual concrete batches are given in Appendix C. Seven 6 x 12 in. (152 x 305 mm) concrete cylinders were made from each batch for compressive strength testing. Five of the seven cylinders were steam cured with the pile. The remaining two cylinders from each batch were air-cured during the same 18 hour period. After the initial 18 hour curing period, one of the two cylinders from each batch, not previously subjected to steam, was moist-cured until an age of 28 days. The remaining six cylinders were air-cured until time of testing. Concrete material property tests were conducted at release and at concrete ages of 24 hours, 7 days, 14 days, and 28 days. These concrete cylinders were tested by LTRC in their laboratory in Baton Rouge, Louisiana.

Compressive strength tests were conducted in accordance with ASTM Designation: C39-86 (20). Results of compressive strength tests are given in Table 25. Each value in Table 25 represents the average of five test results, one for each concrete batch. Test results at 28 days represented three different curing regimes. As indicated in Table 25, cylinders that were steam-cured had the lowest 28-day compressive strength. Based on this data, it appears that steam-curing may not be advantageous for high-strength concrete.

TABLE 25

CONCRETE COMPRESSIVE STRENGTH - P4

Concrete Age	Compressive Strength, psi (Steam/Air Cure)	Compressive Strength, psi (Air/Air Cure)	Compressive Strength, psi (Air/Moist Cure)
18 hours	8,449	No Test	No Test
24 hours	9,071	No Test	No Test
7 days	9,596	No Test	No Test
14 days	10,264	No Test	No Test
28 days	10,453	11,874	11,637

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

7.2.2 Prestressing Strand Material Properties

Prestressing strand used in the pile specimen was 1/2-in. (13-mm) diameter, 7-wire, Grade 270, low-relaxation strand, conforming to ASTM Designation: A416-88 (26). All the prestressing strand used in the pile specimen came from one coil. Material properties of the strand were supplied by the strand manufacturer and are presented in Appendix C.

7.3 TRANSPORTATION

The site selected for driving the pile was on Route La 415 at the new bridges located over the Missouri Pacific Railroad near Port Allen, Louisiana. The selection of this site resulted in a trucking distance of approximately 180 miles, with most of this distance on interstate highways. Thirteen days after release, the pile was loaded onto a truck and transported to a bridge under construction on Route La 415 over the Missouri Pacific Railroad in West Baton Rouge Parish. Three-point pick-up was used to load the pile on the truck for transportation to the construction site. As shown in Figure 32, the pile was supported in two locations during transportation. The pile arrived at the construction site with no visible signs of damage.

7.4 HANDLING AT THE CONSTRUCTION SITE

At the construction site, the contractor first tried to lift the pile from the truck using a two-point pick-up. This attempt was made using a short lifting cable running between the two lifting points, resulting in a very "flat" angle and very high tensile forces in the lifting loops. Because of this high tensile force, one of the lifting loops failed in tension but did not pull out of the pile. To compensate for the lack of a lifting loop, a choker was placed around the pile at the location of the damaged lifting loop, and the two remaining loops were used in a three-point pick-up to get the pile off the truck. Other than the broken lifting loop, no damage was sustained by the pile during unloading operations.

Three-point pick-up was used to lift the pile into the pile leads. Again, no visible signs of damage were noticed during this operation.

7.5 DRIVING THE PILE

During driving, the pile performance was evaluated using a Pile Driving Analyzer™ (PDA). Prior to driving the pile, two strain gauges and two accelerometers were installed on the surface of the pile approximately four feet below the top end. The PDA was used to convert measured strains and accelerations for each hammer blow to force and velocity data. The recorded measurements were used to estimate the pile capacity, monitor hammer performance and energy transfer, measure pile driving stress, and evaluate pile damage. Further details of the pile evaluation are described in the PDA report included in Appendix C.

The pile was driven using a Vulcan 020 single acting external combustion hammer with a 20,000 lb ram operating at a maximum stroke of 3-ft (0.91-m) and a maximum energy rating of 60.0 kip-ft (81.5 kN-m). The hammer cushion consisted of an 18-in. (457-mm) thick pad of alternating one inch layers of micarta and aluminum. The pile cushion consisted of 6-in. (127-mm) thick compressed oak. A 2,600 lb (11.6 kN) helmet was used between the hammer cushion and the pile cushion. During driving, the maximum measured energy transferred to the pile varied from 12.0 kip-ft (16.3 kN-m) in the first 22 ft (6.7 m) of penetration to 15.0 kip-ft (20.4 kN-m) for the remainder of the driving. This translates into an efficiency of 20 to 25 percent of maximum rated hammer energy.

7.6 EVALUATION OF PILE PERFORMANCE DURING DRIVING

The maximum allowable driving stresses for precast prestressed concrete piles recommended by the Federal Highway Administration are based on the following formulas:

$$\text{Maximum Compressive Driving Stress} = 0.85\ f_c' - \text{Effective Prestress}$$

$$\text{Maximum Tension Driving Stress} = 3\sqrt{f_c'} + \text{Effective Prestress}$$

Based on 14-day and 28-day strength tests, the compressive strength of the concrete at the time of driving was 10,400 psi (71.7 MPa) throughout pile driving. The effective prestress was assumed to be 1,454 psi (10.0 MPa). Based on the above formulas, the concrete compressive strength, and the assumed effective prestress, the maximum allowable compressive driving stress was 7,390 psi (51.0 MPa) and the maximum allowable tension driving stress was 1,760 psi (12.1 MPa).

The compressive driving stresses reached a maximum of 1,830 psi (12.6 MPa) and then leveled off to an average of 1,700 psi (11.7 MPa) throughout the remainder of the pile driving. The maximum tensile driving stresses increased from 860 psi (5.9 MPa) at the beginning of driving to 1,310 psi (9.0 MPa) at a pile tip penetration of 55 ft (16.8 m). The maximum tensile stress gradually reduced to 1,000 psi (6.9 MPa) at a tip elevation of 86 ft (26.2 m) then dropped off sharply as driving resistance increased in the dense sand. The driving stresses were well below the maximum allowable compressive stress of 7,390 psi (51.0 MPa) and the maximum allowable tensile stresses of 1,760 psi (12.1 MPa) throughout pile driving. A profile of the driving stresses is shown in the PDA Monitoring report included in Appendix C.

There was no evidence of pile distress or damage during driving. According to the Pile Driving Analyzer, the pile had an integrity factor of 1.0 throughout the entire driving operation. Therefore, it is concluded that the pile was not damaged during transportation, handling, or driving. It should be noted that had the pile been fabricated using concrete with a compressive strength of either 5,000 or 6,000 psi (34.5 or 41.4 MPa), damage to the pile would have likely resulted. The maximum allowable tensile driving stresses for 5,000 and 6,000 psi concretes are 1,000 and 1,170 psi (6.9 and 8.1 MPa), respectively.

7.7 SUMMARY OF RESULTS

Pile P4 achieved both the required pile minimum design 28-day compressive strength of 8,500 psi (58.6 MPa) and the concrete mix design compressive strength of 10,000 psi (69.0 MPa). The average 28-day compressive strength of the concrete for pile P4 was 10,453 psi (72.1 MPa). This concrete compressive strength was determined for cylinders that were steam-cured for 18 hours after initial set and air cured for the remainder of the 28-day period. Other cylinders of the same concrete that were not steam-cured yielded compressive strengths that were more than 1,000 psi (6.9 MPa) greater than the steam-cured concrete. Therefore, it appears that steam-curing of high-strength concrete may not be beneficial.

On the basis of this one test, it appears that high-strength concrete can be used effectively in long piles. The 130-ft (39.6-m) long pile performed well during transportation, handling, and driving. The higher tensile strength and higher precompression is particularly valuable in soft driving conditions where tensile driving stresses are the highest.

8. FATIGUE TESTS OF GIRDER SPECIMENS

8.1 FABRICATION OF GIRDER SPECIMENS

Two bulb-tee girder specimens, designated BT4 and BT5, were fabricated for the fatigue tests. These girders were fabricated at Sherman Prestressed concrete in Mobile, Alabama. Specific details of the girder specimens are shown in Figure 9. Supplemental materials and fabrication details are discussed in Appendix D.

The two bulb-tee specimens were 54-in. (1.37-m) deep and 70-ft (21.3-m) long. Each girder was prestressed using 30 uncoated, 1/2-in. (13-mm) diameter, Grade 270, low-relaxation 7-wire strands. Six of these strands were draped. Each prestressing strand was pretensioned to a stress level corresponding to 75% of guaranteed ultimate strength. During pretensioning, strand loads were verified through both measured elongations and the use of load cells on selected strands. Measured strand loads correlated well with theoretical strand load determined based on measured elongation and strand modulus of elasticity.

The two girder specimens were pretensioned simultaneously (in series) within a single prestressing bed. The strand tensioning sequence started with the bottom row of strands and proceeded upward. The six draped strands were the last to be tensioned. Shear reinforcement and various inserts were installed after pretensioning the strands. Shear reinforcement in the girder specimens consisted of No. 4, Grade 60 bars at spacings ranging from 4 in. (102 mm) near the ends to 12 in. (305 mm) at mid-span. After all the steel elements were in place, the side forms were installed and secured.

Three concrete batches were used for each girder. The average ambient temperature during casting was approximately 90°F (32°C). After casting, the two girders were covered with a waterproof tarpaulin. The steel forms were removed approximately 12 hours after casting. After form removal, a large crack was observed near mid-span through the full depth of each of the two girders. Crack widths were widest at the top flange and gradually diminished towards the bottom flange. According to plant personnel, the cracks are a typical occurrence in some of the deeper sections such as the bulb-tee. Approximately 18 hours after casting, the prestress was released by simultaneously torch cutting the strands at the far ends of the prestressing bed. After torch cutting the strands at the ends, the strands were cut within the space between the girders. After release, the crack observed near mid-span of each girder was virtually invisible.

Approximately twelve days after fabrication, the two girders were loaded on semi-trailers and transported to CTL for testing. Upon arrival, the girders were off-loaded and placed on supports inside CTL's structural laboratory, which is maintained at a constant 73°F (23°C) and 50% relative humidity.

A deck slab was cast on girder BT5 approximately 41 days after fabrication. Deck slab dimensions and reinforcement details are shown in Figure 10. The deck slab was cast using partially shored construction. Approximately 50% of the deck slab weight was supported by the girder while the remaining 50% was supported by the laboratory floor. This distribution of the deck slab dead load was taken into consideration when evaluating girder/composite section stresses.

8.2 GIRDER SPECIMEN INSTRUMENTATION

During the fabrication process, the girders were instrumented to measure concrete strains and camber. Internal strain indicators were installed at mid-span in the lower flange of each girder to monitor prestress losses and concrete strains. For the camber measurements, permanent elevation reference points were embedded in the top surface of the top flange at the middle and ends of each girder. Girder instrumentation details are shown in Figure 11.

8.2.1 Internal Strain Gauges

Subsequent to pretensioning and prior to casting the concrete, three Carlson concrete strain meters were installed in each girder specimen at mid-span. As indicated in Figure 11, Carlson meters were installed in the lower flange, at the level of the centroid of the strand group. In addition to the Carlson meters, two 1/4-in. (6-mm) diameter reinforcing bars instrumented with weldable strain gauges were installed near the bottom surface of each girder, at mid-span. These gauges were used to provide an indication of concrete strains near the extreme fiber of the lower flange. Locations of the weldable strain gauges are also shown in Figure 11. Carlson meter and weldable bar strain gauge readings for girders BT4 and BT5 were taken prior to release, after release, and at a concrete age of 28 days. Strain gauge readings for girder BT5 were also taken before and after deck casting, and during both the fatigue and static flexural testing.

8.2.2 Girder Camber Measurement

Immediately after casting (while the concrete was still plastic) large steel bolts were embedded in the top surface of each girder at mid-span and near both ends to provide a permanent fixed reference point for future camber measurements. The embedded bolts near each end were centered above the sole plate. Camber measurements were made using a level to sight elevations at each reference point. Mid-span camber measurements relative to the ends for girders BT4 and BT5 were measured prior to release, after release, and at a concrete age of 28 days. Camber measurements for girder BT5 were also taken before and after deck casting and at various stages of flexural fatigue testing.

8.3 GIRDER SPECIMEN MATERIAL PROPERTIES

8.3.1 Girder Concrete Material Properties

For the fabrication of girders BT4 and BT5, the concrete mix design developed by LTRC was modified slightly at the discretion of the fabricator. The resulting mix design was the same as that used for the 130-ft (39.6-m) long pile specimen P4. Specific mix design details are given in Table 24. As previously discussed in Chapter 3, the mix design was intended to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa).

Each girder specimen was fabricated using three separate concrete batches. The mix proportions varied slightly among the six concrete batches to maintain slump and air content within an acceptable range. Specific details for the individual concrete batches are given in Appendix D. Twelve 6 x 12 in. (152 x 305 mm) concrete cylinders were made from each batch for material property testing. One 6 x 6 x 20 in. (152 x 152 x 508 mm) concrete beam was made from each of the three batches in girder BT5 for material property testing. Concrete material property tests were conducted at release and at concrete ages of 7, 14, 21, 28, 86, and 191 days. In addition to the standard cylinder specimens, two 3 x 6 in. (76 x 152 mm) cores were taken from girder BT5 and tested in compression at a concrete age of 191 days.

Concrete compressive strength tests of the cylinder specimens were conducted in accordance with ASTM Designation: C39-86 (20). Compressive strength tests of the concrete cores were conducted in accordance with ASTM Designation: C42-90 (21). Modulus of elasticity tests were conducted in accordance with ASTM Designation: C469-87 (22). Splitting tensile strength tests were conducted in accordance with ASTM Designation: C496-90 (23). Modulus of rupture tests were conducted in accordance with

ASTM Designation C78-84 (24). Results of the concrete material property tests are given in Table 26 for the two girder specimens. Each value in Table 26 represents a single test result unless indicated otherwise.

In addition to the tests reported in Table 26, concrete unit weight was determined from three cylinders from each concrete batch used in girder BT5. Average unit weights for concrete batches 5-1, 5-2, and 5-3 were 141.9, 145.5, and 143.1 lb/ft^3 (2,273, 2,331, and 2,292 kg/m^3), respectively. It is interesting to note that the concrete batch with the lowest average unit weight also had the lowest concrete compressive strength.

As indicated in Table 26, the concrete in both girders BT4 and BT5 failed to achieve the required 28-day compressive strength of 10,000 psi (69.0 MPa). Since these girders were to be used for flexural fatigue tests, the concrete properties at mid-length were particularly important. Concrete batches 4-2 (BT4) and 5-2 (BT5) were placed in the mid-length region. It was decided by the project team that 8,500 psi (58.6 MPa) would be the absolute minimum acceptable concrete compressive strength for the mid-length concrete batches. Since Batch 4-2 failed to meet the minimum acceptable strength, girder BT4 was returned to the fabricator and not used for a fatigue test.

8.3.2 Prestressing Strand Material Properties

Prestressing strand used in the girder specimens was 1/2-in. (13-mm) diameter, 7-wire, Grade 270, low-relaxation strand, conforming to ASTM Designation: A416-88 (26). Prestressing strand used in the girder specimens came from four different coils. Samples of the prestressing strand were taken from the beginning and end of each coil for material property testing. Strand samples were tested in accordance with ASTM Designation: A370-90a, Supplement VII (27).

TABLE 26

CONCRETE MATERIAL PROPERTIES - BT4 AND BT5

Girder No.	Concrete Batch	Concrete Age, days	Compressive Strength, psi	Modulus of Elasticity, ksi	Splitting Tensile Strength, psi	Modulus of Rupture, psi
BT4	4-1	Release	7,710	5,450	No Test	No Test
		7	8,290	No Test	No Test	No Test
		14	8,460	No Test	No Test	No Test
		21	8,380	No Test	No Test	No Test
		28	8,170	5,100	600	No Test
	4-2	Release	7,200	5,300	No Test	No Test
		7	6,300	No Test	No Test	No Test
		14	8,040	No Test	No Test	No Test
		21	7,980	No Test	No Test	No Test
		28	7,880	5,600	480	No Test
	4-3	Release	8,100	5,850	No Test	No Test
		7	9,080	No Test	No Test	No Test
		14	9,750	No Test	No Test	No Test
		21	10,190	No Test	No Test	No Test
		28	10,360	5,950	580	No Test
BT5	5-1	Release	6,100	4,650	No Test	No Test
		7	7,250	No Test	No Test	No Test
		14	7,420	No Test	No Test	No Test
		21	7,270	No Test	No Test	No Test
		28	7,680	5,500	470	750
		86*	7,640**	5,150**	530	No Test
		191 (Cyl.)	7,490	No Test	No Test	No Test
		191 (Core)	7,450	No Test	No Test	No Test
	5-2	Release	7,780	5,100	No Test	No Test
		7	8,840	No Test	No Test	No Test
		14	9,170	No Test	No Test	No Test
		21	9,540	No Test	No Test	No Test
		28	9,250	5,400	560	580
		86*	9,260**	5,350**	660	No Test
		191 (Cyl.)	8,800	No Test	No Test	No Test
		191 (Core)	8,270	No Test	No Test	No Test
	5-3	Release	7,000	5,000	No Test	No Test
		7	8,720	No Test	No Test	No Test
		14	9,320	No Test	No Test	No Test
		21	9,750	No Test	No Test	No Test
		28	9,550	5,350	530	820
		86*	9,400**	5,400**	600	No Test

* Girder concrete age at planned start of fatigue test for BT5.

** Average of three cylinder tests.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

Results of the prestressing strand material property tests are given in Table 27. All tested strands met the requirements of ASTM A416-88 (26).

TABLE 27

PRESTRESSING STRAND MATERIAL PROPERTIES - BT4 AND BT5

Coil No.	Sample	Area, sq in.	Load @ 1% Extension, lb	Breaking Strength, lb	Total Elongation, %	Modulus of Elasticity, ksi
983141	Beginning	0.153	39,600	43,400	6.1	30,250
	End	0.153	39,200	43,300	6.6	28,300
984541	Beginning	0.153	39,500	43,100	6.6	29,700
	End	0.153	39,100	43,200	7.1	29,100
988032	Beginning	0.153	38,800	43,600	6.5	28,100
	End	0.153	40,000	43,600	6.1	29,250
988101	Beginning	0.153	40,100	43,500	6.2	28,700
	End	0.153	40,200	43,600	7.0	29,100

Metric Equivalents:

1 sq in. = 645.2 sq mm

1 lb = 0.454 kg

1 ksi = 6.895 MPa

8.3.3 Deck Slab Concrete Material Properties

As discussed in Section 4.3, the concrete mix design for the girder deck slab was intended to yield concrete with a minimum 28-day compressive strength of 6,000 psi (41.4 MPa). Specific details regarding the deck slab concrete mix design are given in Appendix D. Three 8 yd^3 (6.1 m^3) batches were used to cast the deck slab for girder BT5. Since the girder was oriented in the east-west direction for testing and the deck was poured from east to west, the three deck slab concrete batches were designated as east batch, middle batch and west batch. Three 6 x 12 in. (152 x 305 m) concrete cylinders were made from each of the three concrete batches for material property testing.

Deck slab concrete material property tests were conducted at an age of 45 days. Concrete compressive strength tests of the cylinder specimens were conducted in accordance with ASTM Designation: C39-86 (20). Modulus of elasticity tests were conducted in accordance with ASTM Designation: C469-87 (22). Results of the deck slab concrete material property tests are given in Table 28. Each value in Table 28 represents a single test result.

In addition to the tests reported in Table 28, concrete unit weight was also determined from three cylinders from each concrete batch used in the deck slab for girder BT5. Average unit weight for east, middle, and west concrete batches was 141.4, 143.4, and 143.2 lb/ft^3 (2,265, 2,297, and 2,294 kg/m^3), respectively.

TABLE 28

DECK SLAB CONCRETE MATERIAL PROPERTIES - BT5

Girder No	Concrete Batch	Concrete Age	Compressive Strength, psi	Modulus of Elasticity, ksi
BT5	East	45 Days*	6,140	4,000
			6,040	4,200
			5,920	4,200
	Middle	45 Days*	7,380	4,500
			7,510	4,500
			7,380	4,500
	West	45 Days*	7,650	4,500
			7,780	4,450
			7,720	4,500

* Deck slab concrete age at planned start of fatigue test for BT5.

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

8.4 DISCUSSION OF MATERIAL PROPERTIES AND EARLY-AGE BEHAVIOR

8.4.1 Material Properties

The AASHTO Standard Specification for Highway Bridges (1) and ACI 318-89 (2) include specific equations for calculating concrete modulus of elasticity and modulus of rupture in lieu of actual test data. These equations make use of established relationships between concrete compressive strength and other concrete properties developed based on test results from lower strength concretes. For this reason, ACI published ACI 363R-84, State-of-the-Art Report on High-Strength Concrete (28). This report introduces alternate equations for both modulus of elasticity and modulus of rupture proposed by Carrasquillo (16) for use with high-strength concrete. Table 4 (shown in Section 5.4.1) presents a comparison of the AASHTO/ACI 318 equations and the alternate equations of ACI 363.

Using the equations in Table 4 with the measured concrete compressive strength and unit weight, the mechanical properties of the concrete for girders BT4 and BT5 were calculated. In comparing the measure properties with computed properties, it is concluded that the measured results agree more closely with the AASHTO/ACI 318 values than with the ACI 363 values. The ACI 363 values tended to overestimate concrete modulus of rupture and underestimate concrete modulus of elasticity. The AASHTO/ACI 318 values tended to slightly underestimate both modulus of rupture and modulus of elasticity. The same conclusions were drawn for the three pile specimens (P1, P2, and P3) and three girder specimens (BT1, BT2, and BT3), as previously discussed in Sections 5.4.1 and 6.4.1, respectively.

8.4.2 Prestress Losses

The AASHTO Standard Specification for Highway Bridges contains provisions for calculating total prestress losses due to concrete shrinkage, elastic shortening, concrete creep, and steel relaxation. Based on these provisions and the design material properties, total prestress losses of approximately 53,154 psi (366.5 MPa) would be expected for the girders. However, only approximately 35-40% of the total ultimate concrete creep and shrinkage losses are expected to occur within the first 28 days (29). Therefore, prestress losses of approximately 33,422 psi (230.4 MPa) would be expected at a concrete age of 28 days.

Internal Carlson strain meters were installed in each girder specimen and monitored until a concrete age of 28 days. Measured concrete strains were used to calculate prestress losses by multiplying the average measured strains by the average measured strand modulus of elasticity of 29,050 ksi (200 GPa). Concrete strains measured just after release and at an age of 28 days, and corresponding calculated prestress losses, are shown in Table 29. Additional prestress loss measurements made at various stages of the fatigue test for girder BT5 are reported in Section 8.6.1.

TABLE 29

MEASURED PRESTRESS LOSS AT RELEASE AND 28 DAYS - BT4 AND BT5

Girder No.	Specimen Age	Measured Strain, in./in.	Prestress Loss, psi
BT4	Release	691×10^{-6}	20,073
	28 days	917×10^{-6}	26,639
BT5	Release	635×10^{-6}	18,447
	28 days	933×10^{-6}	27,104

Metric Equivalents:

1 ksi = 1000 psi = 6.895 MPa

Concrete strains measured immediately after release were used to provide an indication of prestress losses due to elastic shortening. Based on these concrete strains, losses due to elastic shortening averaged 19,260 psi (132.8 MPa) between the two girder specimens. This value is in reasonable agreement with the average elastic shortening losses of 14,696 psi

(101.3 MPa) calculated using provisions of the AASHTO standard and actual measured concrete and steel material properties.

Concrete strains measured at a concrete age of 28 days indicated prestress losses averaging approximately 26,872 psi (185.3 MPa) among the two girder specimens. This value is slightly less than the 32,523 psi (224.3 MPa) calculated using provisions of the AASHTO standard, actual measured concrete and steel material properties, and recommendations of the PCI Committee on Prestress Losses (29). Based on this information, it appears that the actual total prestress losses in the two girder specimens can be expected to be significantly less than the total losses predicted using the AASHTO standard. However, calculated values for prestress losses were based on the assumption that approximately 35% of the ultimate creep and 42% of the ultimate shrinkage will occur within the first 28 days. These percentages, although shown to be reasonable for conventional concretes, may not be applicable for higher strength concretes. This may explain the noted difference between measured and calculated prestress losses at 28 days.

8.4.3 Girder Camber

As discussed in Section 8.2, permanent fixed reference points were embedded in top flange of the girders for camber measurements. Mid-span camber measurements relative to the ends of each girder were measured prior to release, after release, at a concrete age of 28 days, and after deck casting (where applicable). Previous research by Kelly, Bradberry, and Breen (5) found that camber and deflection of high-strength girders could be accurately predicted using moment area equations and known concrete properties. To verify this, camber was calculated for girders BT4 and BT5 using the moment-area method. Average measured material properties and prestress losses were used in the calculations. Camber calculations did not account for camber growth due to concrete creep. Camber measurements and corresponding calculated deflections for girders BT4 and BT5 are documented in Table 30.

Additional camber measurements made at various stages of the fatigue test for girder BT5 are reported in Section 8.6.1.

TABLE 30

MID-SPAN CAMBER MEASUREMENTS AND DEFLECTION CALCULATIONS - BT4 AND BT5*

Event	Girder BT4 Measured Camber	Girder BT5 Measured Camber	Calculated Deflection
Release	-0.90 in.	-1.14 in.	-0.93 in.
28 Days	-1.32 in.	-1.60 in.	-0.83 in.
With Deck	N/A	-1.09 in.	-0.45 in.

* Negative value indicates an upward camber.

Metric Equivalent:
1 in. = 25.4 mm

As indicated in Table 30, measured camber at release correlated well with calculated values. However, due to early-age camber growth, measured camber at 28 days was significantly greater than the corresponding calculated deflection. This observation is consistent with the behavior observed in girders BT1, BT2, and BT3; however, the difference between calculated and measured values was significantly greater in girders BT4 and BT5. Measured camber in girder BT5 after adding the deck slab was also significantly greater than the corresponding calculated value; however, the net difference between measured and calculated values remained approximately the same as that noted at 28 days. Deflection due to the addition of the deck slab was calculated assuming that the deck dead load was distributed equally between the girder section and the composite section. The fact that the difference between the measured and calculated values did not change from that observed at 28 days confirmed that the assumption regarding the distribution of the deck slab load was reasonable.

8.5 GIRDER SPECIMEN FATIGUE TEST DETAILS

8.5.1 Fatigue Test Setup

The test setup for the flexural fatigue test is shown in Figures 33 and 34. The girder specimen was simply supported at the centerline of the end sole plates, creating a total span length of 69 ft (21.0 m). Girder BT5 was subjected to cyclic (fatigue) flexural loading using two point loads spaced 12 ft (3.66 m) apart at mid-span. Load was applied using servo-controlled hydraulic actuators which were connected to the bottom flange of the girder at one end and anchored to the laboratory floor at the opposite end. Swivels were attached to each end of the actuators to allow both rotation and translation within the longitudinal plane of the girder.

Each hydraulic actuator incorporated a load cell which was used to monitor the applied loads. Additional load cells were placed at each support to monitor support reactions. Girder deflections and displacements were monitored using a pair of linear variable differential transformers (LVDTs) located at mid-span and using dial gauges at each end of the girder. Load cells and LVDTs are calibrated annually.

In addition to the internal strain gauges installed during girder fabrication, several concrete surface gauges were added prior to the fatigue test. During girder fabrication, a mid-span crack was observed in girder BT5 prior to release of prestress. Prior to the fatigue test, two crack width gauges and two concrete surface strain gauges were installed on the bottom surface of the lower flange. The crack width gauges were located near each outside edge of the lower flange, spanning the pre-existing crack. The two concrete surface strain gauges were placed adjacent to the crack at approximately mid-width of the lower flange. Crack width gauges and concrete surface gauges are shown in Figure 35. An accelerometer was also installed on the bottom surface of the lower flange near mid-span. The purpose of the accelerometer was to detect any longitudinal energy releases such as cracks or strand wire fractures.

8.5.2 Fatigue Test Procedure

During the fatigue test, girder BT5 was subjected to cyclic flexural loading. The upper bound load was the load required to produce a mid-span tensile stress at the extreme fiber of the lower flange equal to $6\sqrt{f'_c}$. The fatigue load range was selected to produce a stress range in the extreme fiber of the lower flange equal to that caused by the design live load plus impact. The lower bound load was subsequently determined based on the stress range. Initially, it was planned that the fatigue test would be performed on an uncracked girder. However, during the first static loading of the girder to full design load, the crack that had formed at casting reopened. Consequently, the test was performed on a cracked girder and the applied loads were based on the load to cause decompression of the bottom fiber.

The fatigue test consisted of six individual parts. The first part consisted of an initial static flexural test to determine the load corresponding to complete decompression of the bottom flange. This was necessary in order to accurately establish the upper-bound fatigue load. The decompression load was determined based on data from the concrete surface strain gauges and the crack width gauges located at the pre-existing mid-span crack. Decompression was judged to have occurred when the load versus strain/crack width relationship first deviated from linearity. Once the decompression load was determined, the upper-bound load was established by adding the load required to produce an additional tensile stress of $6\sqrt{f'_c}$ to the decompression load.

Each of the five remaining parts of the fatigue test included the application of 1 million loading cycles followed by a intermittent static test to verify or re-establish the decompression load, and subsequently the upper- and lower-bound loads, for the next test segment. Fatigue performance was evaluated based on observed change in response to the static loading occurring as a result of increasing fatigue exposure. Prior to each static test (including the initial static test) mid-span camber and prestress losses were also measured and documented.

The fatigue test was conducted as a load-controlled test. The fatigue loading was applied at a frequency of approximately 2 cycles per second. The effects of dynamic load amplification due to the bouncing mass of the girder specimen were accounted for by using the support reactions to establish the applied loads. The loading function applied by each actuator was established to produce upper- and lower-bound support reactions that were consistent with the required static test loads.

During the initial and intermittent static tests, all load cells (actuator and support), internal strain gauges, external strain gauges, crack width gauges, and LVDTs were monitored continuously using a digital data acquisition system (DDAS) and computer. At selected levels of applied load, data were stored on floppy disk, providing a permanent record of test specimen behavior. The mid-span accelerometer was also monitored continuously during the static tests. The static test control software was set up so that any significant energy release detected by the accelerometer would immediately trigger the DDAS to take several complete readings of all instrumentation.

During fatigue testing, data from all electronic instrumentation were taken at least twice per day using a high speed digital data acquisition system (DDAS) and a computer. Each data collection typically consisted of three hundred load stage readings over a 1 second time period. This allowed us to examine data for two complete load cycles. The mid-span accelerometer was monitored continuously during fatigue loading. The fatigue test control software was set up so that any significant energy release detected by the accelerometer would immediately trigger the high-speed DDAS to take several complete readings of all instrumentation.

8.6 ANALYSIS OF FATIGUE TEST RESULTS

8.6.1 Evaluation of Fatigue Performance

As previously mentioned, fatigue performance was evaluated based on observed change in response to the static loading occurring as a result of increasing fatigue exposure. Load-deflection relationship under static loading was used for the evaluation. The initial static load-deflection curve is plotted along with each of the static load-deflection curves for the girders after each 1,000,000 cycle segment in Figure 36. Initial and intermittent camber and prestress loss measurements are documented in Table 31.

As indicated in Figure 36, gradual "softening" of the girder was observed during fatigue testing. The measured mid-span deflection at a load corresponding to the full design service load moment (D.L. + L.L. + Impact) was 0.49 in. (12.5 mm) at the beginning of the fatigue test and 0.53 in. (13.5 mm) after completion of 5 million cycles. The decompression load determined from strain gauge data during the static tests did not change significantly for the first four million cycles. Therefore, the upper- and lower-bound fatigue test loads were kept the same for the first four million cycles. Based on the data from the static test conducted after achieving four million cycles, it was determined that the upper- and lower-bound fatigue test loads should be reduced slightly, since the decompression load had apparently decreased slightly. However, the load range was kept constant throughout the entire fatigue test. As indicated in Table 31, the mid-span camber decreased during the fatigue test but the prestress loss did not change significantly.

TABLE 31

CAMBER AND PRESTRESS LOSS AT VARIOUS STAGES OF FATIGUE TEST - BT5

Stage of Test	Mid-Span Camber*, in.	Prestress Loss, psi
Initial	-0.98	27,229
1,000,000 Cycles	-0.90	26,484
2,000,000 Cycles	-0.87	26,434
3,000,000 Cycles	-0.89	26,329
4,000,000 Cycles	-0.87	26,309
5,000,000 Cycles	-0.84	26,329

* Negative value indicates an upward camber.

Metric Equivalents:

1 in. = 25.4 mm

1 ksi = 1000 psi = 6.895 MPa

A summary of the fatigue test parameters used for girder BT5 is given in Table 32. At the start of the fatigue test, the average stress in the prestressing strand, based on measured strains, was approximately 178,260 psi (1,229 MPa). Based on this initial stress, the calculated steel stress at the lower bound test load was approximately 180,630 psi (1,245 MPa). Although the test loads were reduced slightly for the final 1 million cycles, the measured steel stress range remained at approximately 10.0 ksi (69.0 MPa) throughout the fatigue test. The steel stress range at the level of the lowest row of prestressing strands was determined based on strain gauge data from the weldable strain gauges. These strain gauges were located at mid-span, approximately 6 in. (152 mm) from the pre-existing crack. For comparison purposes, steel stresses at the level of the lowest strand row were also calculated using a strain compatibility analysis. Calculated steel stresses based on both uncracked and cracked sections were 6.5 and 9.5 ksi (44.8 and 65.5 MPa), respectively. The measured stress range correlated well with the calculated stress range for a cracked section.

TABLE 32

SUMMARY OF FATIGUE TEST PARAMETERS FOR GIRDER BT5

Parameter	0-4 Million Cycles	4-5 Million Cycles
Upper Bound Load	179.6 kips	174.6 kips
Decompression Load	130.0 kips	125.0 kips
Lower Bound Load	50.9 kips	45.9 kips
Measured Steel Stress Range	10.0 ksi	10.0 ksi

Metric Equivalents:

1 kip = 4.448 kN

1 ksi = 1000 psi = 6.895 MPa

During the fatigue test of girder BT5, several transverse cracks developed in the deck slab outside of the constant moment region. These cracks were spaced approximately 10 ft (3.1 m) apart and extended from the edge of the slab to approximately mid-width. On one half of the girder the cracks originated on the north side of the slab, while on the other half the cracks originated on the south side. This cracking behavior suggests that the applied flexural load may have produced a torsional moment in the girder. The cause of this torsion is not completely clear. However, measured mid-span strains and crack widths on one side of the lower flange were slightly greater than those noted on the opposite side, indicating that the stiffness of the girder may have varied slightly across the width. This observation was noted prior to the development of the deck slab cracks. Therefore, it is believed that variable stiffness of the girder may have been a contributing factor to the deck slab cracking.

8.6.2 Evaluation of Post-Fatigue Test Flexural Strength

After completion of the fatigue test and 199 days after fabrication, girder BT5 was subjected to a flexural strength test. Prior to the start of the flexural strength test, four concrete surface strain gauges were installed across the top surface of the deck slab. The flexural strength test setup and procedure were the same as those previously described for girders BT1, BT2, and BT3 in Sections 6.5.1 and 6.5.2. Behavior of girder BT5 during the test was similar to that previously described in Section 6.6.1 for girder BT1 and BT3.

Total applied load (not including girder weight) versus mid-span deflection is plotted in Figure 37. Deflections were measured from an initial upward cambered position of 0.84 in. (21.3 mm). During the flexural strength test, girder BT5 was loaded to produce a mid-span moment corresponding to the full design service load moment (D.L. + L.L. + Impact). At the full design moment, girder BT5 did not exhibit any new concrete cracking and the pre-existing crack did not open. Girder BT5 deflected 0.53 in. (13.5 mm) at full design moment.

In girder BT5, the pre-existing crack near mid-span opened prior to the development of any new cracks. Therefore, the cracking moment in girder BT5 was taken as the moment corresponding to the opening of the pre-existing crack. The observed total applied load corresponding to the opening of the pre-existing crack was 191.6 kips (852 kN).

The total load corresponding to the formation of initial flexure-shear crack for girder BT5 was 285.1 kips (1,268 kN). Consistent with findings from previous research (30) and girders BT1, BT2, and BT3, the initial flexure-shear crack originated at a distance from the load point that was approximately equal to the effective depth of the girder. Unlike the behavior observed from girders BT1, BT2, and BT3, a large web shear crack occurred near one end of girder BT5 during the flexural test. As indicated in Table 26, the compressive strength of concrete from Batch 5-1 was particularly low. The web shear crack noted during the flexural test occurred within the end of the girder where Batch 5-1 was placed.

The load-deflection curve for girder BT5 peaked at a total applied load of 383.3 kips (1,705 kN). The deflection coincident with the peak load was 17.4 in. (442 mm). Flexural tension failure within the constant moment region, evidenced by audible indications of strand breakage, marked the limit of the load capacity of girder BT5. Upon dissection of the failure region, it was determined that 4 of the 30 strands in the lower flange had completely fractured. The fractured strands all occurred on one side of the bottom flange, in the bottom row of strands, and at the location of the initial (pre-existing) mid-span crack. Fatigue fractures (fractures with no apparent reduction of area) of individual wires were detected on 2 of the 4 strands that fractured. One of these strands had 3 wire fatigue fractures and the other had 1 wire fatigue fracture.

Internal Carlson strain meters and weldable strain gauges in girder BT5 indicated flexural behavior similar to that previously described for girder BT1 and BT3 in Section 6.6.1. Data from the four deck slab concrete surface strain gauges on girder BT5 indicated that the full width of the deck slab was effective with no apparent shear lag. Load versus deck slab surface strain for girder BT5 is shown in Figure 38.

Observed cracking moment, flexure-shear cracking, and flexural strength for girder BT5 were evaluated by comparison with computed values. For this evaluation, the computed values were determined using two methods. In one comparison, the design material properties and design total prestress losses were used to determine the cracking moment, flexure-shear cracking load, and flexural strength. For the second comparison, actual measured material properties and prestress losses were used in the computations. Details of the computations were the same as those previously described for girders BT1, BT2, and BT3 in Sections 6.6.2, 6.6.3, and 6.6.4. Observed and computed values are shown in Table 33.

TABLE 33

OBSERVED AND COMPUTED FLEXURAL PROPERTIES - BT5

Cracking Moment, kip-ft			Concrete Flexural Shear Strength, kips			Flexural Strength, kip-ft		
Observed	Design	Actual	Observed	Design	Actual	Observed	Design	Actual
2,730	2,384	3,016	162.9	143.6	157.2	6,594	6,086	6,491

Metric Equivalents:

1 kip = 4.448 kN

1 kip-ft = 1.357 kN-m

As indicated in Table 33, the observed cracking moment, concrete flexural shear strength, and flexural strength exceeded the values calculated using design conditions. However, the observed cracking moment was approximately 9.5% less than the cracking moment computed using actual measured material properties and prestress losses. Observed concrete flexural shear strength correlated well with the corresponding value calculated using actual measured material properties and prestress losses. The observed flexural strength exceeded the value calculated using actual conditions by approximately 4%. However, the observed flexural strength of girder BT5 was approximately 5% less than the flexural strength of girders BT1 and BT3.

8.7 SUMMARY OF RESULTS

The concrete in both girders BT4 and BT5 failed to achieve the required minimum girder design 28-day compressive strength of 10,000 psi (69.0 MPa). The average 28-day compressive strengths of girders BT4 and BT5 were 8,800 psi (60.7 MPa) and 8,820 psi (60.8 MPa), respectively. Based on these strength results, as well as the strength results from the other specimens fabricated as part of the research program, it is apparent that precast fabricators will need to make some changes to the normal production regime in order to consistently produce high-strength concrete.

With respect to concrete material properties, it is concluded that the concrete in girders BT4 and BT5 behaved in a manner that would be conservatively predicted using the provisions of the AASHTO Standard Specification for Highway Bridges. Early-age prestress losses in girders BT4 and BT5 were less than expected values calculated using the AASHTO provisions, but significantly greater than those measured in girders BT1, BT2, and BT3. Early-age camber of girders BT4 and BT5 was also significantly greater than that noted in girders BT1, BT2, and BT3. The significant difference between the early-age behavior of girders BT4 and BT5 and the other three girders was probably primarily due to the fact that the first three girders were steam-cured.

Girder BT5 withstood 5 million cycles of fatigue loading without failure of serviceability requirements. Upon completion of the fatigue test, girder BT5 was loaded to produce a mid-span moment corresponding to the full design service load moment (D.L. + L.L. + Impact). At the full design moment, girder BT5 did not exhibit any new concrete cracking and the pre-existing crack did not open. The measured mid-span deflection at full design moment was 0.49 in. (12.5 mm) at the beginning of the fatigue test and 0.53 in. (13.5 mm) after completion of 5 million cycles. Therefore, girder BT5 was judged to have fulfilled the serviceability requirements even after 5 million cycles of flexural fatigue loading.

The AASHTO provision for computing cracking moment, concrete flexural shear strength, and flexural strength using design material properties and prestress losses were shown to be conservative for the fatigue tested girder BT5. However, the observed post-fatigue test flexural strength of girder BT5 was approximately 5% less than the observed flexural strength of girders BT1 and BT3.

9. SUMMARY

9.1 DETAILS OF INVESTIGATION

The objective of this investigation was to evaluate the feasibility of using high-strength concrete in the design and construction of highway bridge structures. A literature search was conducted; a survey of five regional fabrication plants was performed; mix designs were studied in the laboratory and in the field; and a total of nine full-size specimens were fabricated and tested.

Three concentrically prestressed pile specimens were fabricated and tested in flexure as part of this program. Each of the pile specimens had a 24-in. (610-mm) square cross section with a concentric 12-in. (305-mm) diameter void. All pile specimens were 24-ft (7.3-m) long. These three pile specimens had a design 28-day concrete compressive strength of 8,500 psi (59 MPa).

Five full-size prestressed bulb-tee girders were fabricated as part of this program. Early-age flexure and shear strength tests were conducted on two bulb-tee specimen. The third specimen was used for evaluation of long-term static-load behavior followed by a late-age flexural strength test. The fourth specimen was subjected to a fatigue test followed by a flexural strength test. The fifth specimen was not tested.

The specimens were 54-in. (1.37-m) deep bulb-tee sections with a 6-in. (152-mm) thick web. Each girder specimen was 70-ft (21.3-m) long. Each bulb-tee specimen had a design 28-day concrete compressive strength of 10,000 psi (69.0 MPa). Prior to testing, three of the bulb-tee specimens (BT1, BT3, and BT5) had a 9-1/2-in. (241-mm) thick and 10-ft (3.05-m) wide deck slab added. One bulb-tee specimen used for early-age testing did not have a deck. A comparison of major results from girders BT1, BT3, and BT5 is given in Table 34.

TABLE 34

SUMMARY OF RESULTS FOR GIRDERS BT1, BT3, AND BT5

Parameter	Girder BT1	Girder BT3	Girder BT5
Cracking Moment, kip-ft			
Observed	3,192	3,031	2,730
Calculated-Design Conditions	2,384	2,384	2,384
Calculated-Actual Conditions	3,066	3,190	3,016
Concrete Flexural Shear, kips			
Observed	160.2	157.7	162.9
Calculated-Design Conditions	143.6	143.6	143.6
Calculated-Actual Conditions	159.6	164.0	157.2
Flexural Strength, kip-ft			
Observed	6,977	6,996	6,594
Calculated-Design Conditions	6,086	6,086	6,086
Calculated-Actual Conditions	6,575	6,596	6,491
Deflection at Design Moment, in.	0.48	0.41	0.49-0.53
Camber, in.			
Release	-0.69	-0.67	-1.14
28 Days	-0.98	-1.00	-1.60
Prestress Losses, psi			
Release	14,190	10,380	18,447
28 Days	19,980	16,440	27,104

Metric Equivalents:

1 kip = 4.448 kN

1 kip-ft = 1.357 kN-m

1 in. = 25.4 mm

Fabrication and driving of a single 130-ft (39.6-m) long prestressed pile specimen was also included as part of this program. The pile specimen had the same cross-sectional configuration and design 28-day concrete compressive strength as the first three pile specimens tested in flexure. Performance of this pile specimen during handling and driving was evaluated in the field.

9.2 CONCLUSIONS

Based on the test results and analyses described in this report, the following conclusions are made:

1. High-strength concrete with compressive strengths of 10,000 psi (69.0 MPa) can be produced using regionally available material. However, special attention to both material selection and production quality control are required. Production of concrete with 10,000 psi (69.0 MPa) compressive strength requires more stringent quality control procedures than regional fabricators are presently accustomed to using.

2. Results of this study indicate the use of steam-curing with high-strength concrete may not be beneficial and could be detrimental with respect to development of 28-day compressive strength.

3. Measured concrete material properties for the pile and girder specimens correlated well with properties computed used the AASHTO standard. In addition, it was concluded that the measured properties agreed more closely with the values calculated using AASHTO and ACI 318 provisions than with the values calculated using ACI 363 provisions. The ACI 363 values tended to overestimate concrete modulus of rupture and underestimate concrete modulus of elasticity. The AASHTO/ACI 318 values tended to slightly underestimate both modulus of rupture and modulus of elasticity, thus providing a conservative value.

4. Transfer lengths interpreted from strain readings from pile and girder specimens were consistently less than the transfer length implied by the AASHTO and ACI provisions for development length.

5. Measured early-age prestress losses in the pile and girder specimens due to elastic shortening were in reasonable agreement with the values calculated using AASHTO provisions and actual concrete and steel material properties. Measured prestressed losses at 28 days were significantly less than values calculated using AASHTO provisions and recommendations of the PCI Committee on Prestress Losses (29). Based on these findings, AASHTO provisions and PCI Committee recommendations for predicting prestress losses may be too conservative for higher-strength concretes.

6. Girder camber/deflection measurements, made under various loading conditions up to and including the design service load, were reasonably consistent with deflections calculated using "conventional" methods.

7. The equations in the AASHTO specifications for calculating flexural cracking moment, concrete flexural shear strength, and flexural strength provide a conservative prediction of the pile specimen flexural properties. The use of a strain compatibility method provides a more precise method than the AASHTO provisions for prediction of flexural strength.

8. The equations in the AASHTO specifications for calculating cracking moment, concrete flexural shear strength, flexural strength, concrete web shear strength, and total shear strength provided a conservative prediction of the girder specimen flexural properties. The use of a strain compatibility method provided a slightly more precise method than the AASHTO provisions for prediction of flexural strength.

9. Strain gauges placed across the top of the deck slab on girder BT1, BT3, and BT5 indicated that the full 120-in (3.05-m) width of the slab was effective as suggested by the AASHTO provisions.

10. Measured long-term prestress losses in girder BT3 were approximately 50% less than the total prestress loss calculated using the AASHTO provisions and design material properties. It is concluded that the AASHTO provisions for creep and shrinkage losses are overly conservative for high-strength concrete such as that used in girder BT3. Measured creep and shrinkage deformations from concrete cylinders from girder BT3 supported this conclusion and correlated well with the portion of the measured prestress loss in girder BT3 attributed to creep and shrinkage.

11. Girder specimen BT5 withstood 5 million cycles of fatigue loading without failure of serviceability requirements. The AASHTO provision for computing cracking moment, concrete flexural shear strength, and flexural strength using design material properties and prestress losses were shown to be conservative for the fatigue tested girder BT5. However, the observed post-fatigue test flexural strength of girder BT5 was approximately 5% less than the observed flexural strength of girders BT1 and BT3.

12. Behavior of pile specimen P4, the 130-ft (39.6-m) pile, during transportation, handling, and driving was satisfactory. Driving stresses were well below levels that would cause damage to the pile. If the pile specimen had been fabricated using concrete with a compressive strength of 5,000 psi (344 MPa) or 6,000 psi (41.3 MPa), damage to the pile would have likely resulted.

9.3 RECOMMENDATIONS

Based on the findings of this investigation, the following recommendations are made:

1. Concrete with strengths up to 10,000 psi (69.0 MPa) should be considered for use by the Louisiana Department of Transportation and Development.

2. A quality control training program should be developed to train local fabricators. The importance of the tighter quality control required by high-strength concrete must be understood by fabricators if successful production of high-strength concrete is to occur.

3. High-strength concrete, up to 10,000 psi (69.0 MPa), should be implemented in a bridge. This bridge should be instrumented to determine long term behavior and consideration given to a modal analysis of the structure.

4. An investigation into the effects of steam-curing on the properties of high-strength concrete should be performed.

5. An investigation into the lateral stability of long slender high-strength concrete members should be performed.

10. LIST OF REFERENCES

1. American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, Washington, American Association of State Highway and Transportation Officials, 1992.

2. American Concrete Institute Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary-ACI 318R-89, Detroit, American Concrete Institute, 1989.

3. Law, S. M. and Rasoulian, M., Design and Evaluation of High Strength Concrete for Girders, FHWA Report No. FHWA-LA-80-138, McLean, Virginia, Federal Highway Administration, 1980.

4. Adelman, D. and Cousins, T. E., "Evaluations of the Use of High Strength Concrete Bridge Girders in Louisiana," Journal of the Prestressed Concrete Institute 35, No. 5, Sept.-Oct. 1990, pp. 70-78.

5. Kelly, D. J., Bradberry, T. E. and Breen, J. E., Time Dependent Deflections of Pretensioned Beams, Research Report 381-1, Austin Center for Transportation Research, 1987.

6. Hartman, D. L., Breen, J. E. and Kreger, M. E., Shear Capacity of High Strength Prestressed Concrete Girders, Research Report 381-2, Austin Center for Transportation Research, 1988.

7. Canadian Standards Association, Design of Concrete Structures for Buildings (CAN3-A23.3-M84), Rexdale, 1984.

8. Castrodale, R. W., Kreger, M. E. and Burns, N. E., A Study of Pretensioned High Strength Concrete Girders in Composite Highway Bridges - Laboratory Tests, Research Report 381-3, Austin Center for Transportation Research, 1988.

9. Castrodale, R. W., Kreger, M. E. and Burns, N. E., A study of Pretensioned High Strength Concrete Girders in Composite Highway Bridges - Design Considerations. Research Report 381-4, Austin Center for Transportation Research, 1988.

10. Zia, P., Schemmel, J. J. and Tällman, T. E., Structural Applications of High Strength Concrete, Research Report 23241-87-3, Raleigh Center for Transportation Studies, 1989.

11. Leming, M. L., "Comparison of Mechanical Properties of High Strength Concrete made with Different Raw Materials," presented at the 69th Annual Meeting of the Transportation Research Board, Washington, D. C., 1990.

12. Shin, S. W., Kamara, M. and Ghosh, S. K., "Flexural Ductility, Strength Predictions, and Hysteretic Behavior of Ultra-High-Strength Concrete Members," High Strength Concrete - Second International Symposium, SP 121, Detroit, American Concrete Institute, 1990, pp. 239-263.

13. Roller, J. J. and Russell, H. G., "Shear Strength of High Strength Concrete Beams with Web Reinforcement," Journal of the American Concrete Institute 87, No. 2, Mar.-Apr. 1990, pp. 191-198.

14. Peterman, M. B. and Carrasquillo R. L., Production of High Strength Concrete, New Jersey, Noyes Publications, 1986.

15. Ahmad, S. H. and Shah, S. P., "Structural Properties of High Strength Concrete and its Implications for Precast Prestressed Concrete," Journal of the Prestressed Concrete Institute 30, No. 6, Nov.-Dec. 1985, pp. 93-119.

16. Carrasquillo, R. L., Nilson, A. H. and Slate, F. O., "Properties of High Strength Concrete Subject to Short Term Loads," Journal of the American Concrete Institute 78, No. 3 May-June 1981, pp. 171-178.

17. Cook, J. E., "10,000 psi Concrete," Concrete International: Design & Construction 11, No. 10, Oct. 1989, pp. 69-75.

18. Rabbat, B. G. and Russell, H. G., "Optimized Sections for Precast Prestressed Bridge Girders," Journal of the Prestressed Concrete Institute 30, No. 4, Jul.-Aug. 1982, pp. 88-104.

19. Rabbat, B. G. and Russell, H. G., "Proposed Replacement of AASHTO Girders with New Optimized Sections," Transportation Research Record 950, Vol. 2, 1982, pp. 85-92.

20. American Society for Testing and Materials, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, ASTM C-39-86, Philadelphia, 1986.

21. American Society for Testing and Materials, Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, ASTM C42-90, Philadelphia, 1986.

22. American Society for Testing and Materials, Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression, ASTM C469-87, Philadelphia, 1987.

23. American Society for Testing and Materials, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, ASTM C496-90, Philadelphia, 1990.

24. American Society for Testing and Materials, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading, ASTM C78-84, Philadelphia, 1984.

25. American Society for Testing and Materials, Standard Practice for Petrographic Examination of Hardened Concrete, ASTM C856-83, Philadelphia, 1983.

26. American Society for Testing and Materials, Standard Specifications for Steel Strand, Uncoated Seven-Wire Stress Relieved for Prestressed Concrete, ASTM A416-88, Philadelphia, 1988.

27. American Society for Testing and Materials, Standard Test Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370-90a, Philadelphia, 1990.

28. American Concrete Institute Committee 363, "State of the Art Report on High-Strength Concrete," Journal of the American Concrete Institute 81, No. 4, July-Aug. 1984, pp. 364-411.

29. PCI Committee on Prestress Losses, "Recommendations for Estimating Prestress Losses," Journal of the Prestressed Concrete Institute 20, No. 4, July-Aug. 1975, pp. 44-75.

30. MacGregor, J. G., Sozen, M. A. and Siess, C. P., "Strength and Behavior of Prestressed Concrete Beams with Web Reinforcement," Civil Engineering Studies, Structural Research Series 201, Urbana, University of Illinois, 1960.

31. American Society for Testing and Materials, Standard Test Method for Creep of Concrete in Compression, ASTM C512-87, Philadelphia, 1987.

32. U. S. Army Corps of Engineers, "Test Method for Coefficient of Linear Thermal Expansion of Concrete," Handbook for Concrete and Cement, Vol. 1, Vicksburg, Waterways Experiment Station, 1981.

33. American Society for Testing and Materials, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, ASTM C 666-90, Philadelphia, 1990.

34. ACI Committee 209, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," (ACI 209R-82) (Reapproved 1986), American Concrete Institute, Detroit, 1982, 108 pp.

35. Louisiana Department of Transportation and Development, Louisiana Standard Specifications for Roads and Bridges, Baton Rouge, privately printed, 1982.

36. American Society for Testing and Materials, Standard Specification for Portland Cement, ASTM C150-89, Philadelphia, 1989.

37. American Society for Testing and Materials, Standard Specifications for Chemical Admixtures for Concrete, ASTM C494-90, Philadelphia, 1990.

38. American Society for Testing and Materials, Standard Specification for Air-Entraining Admixtures for Concrete, ASTM C260-86, Philadelphia, 1986.

39. American Society for Testing and Materials, Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete, ASTM C618-91, Philadelphia, 1986.

40. American Society for Testing and Materials, Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance, ASTM C403-90, Philadelphia, 1990.

Metric Equivalents:
1 ft = 0.305 m
1 in = 25.4 mm

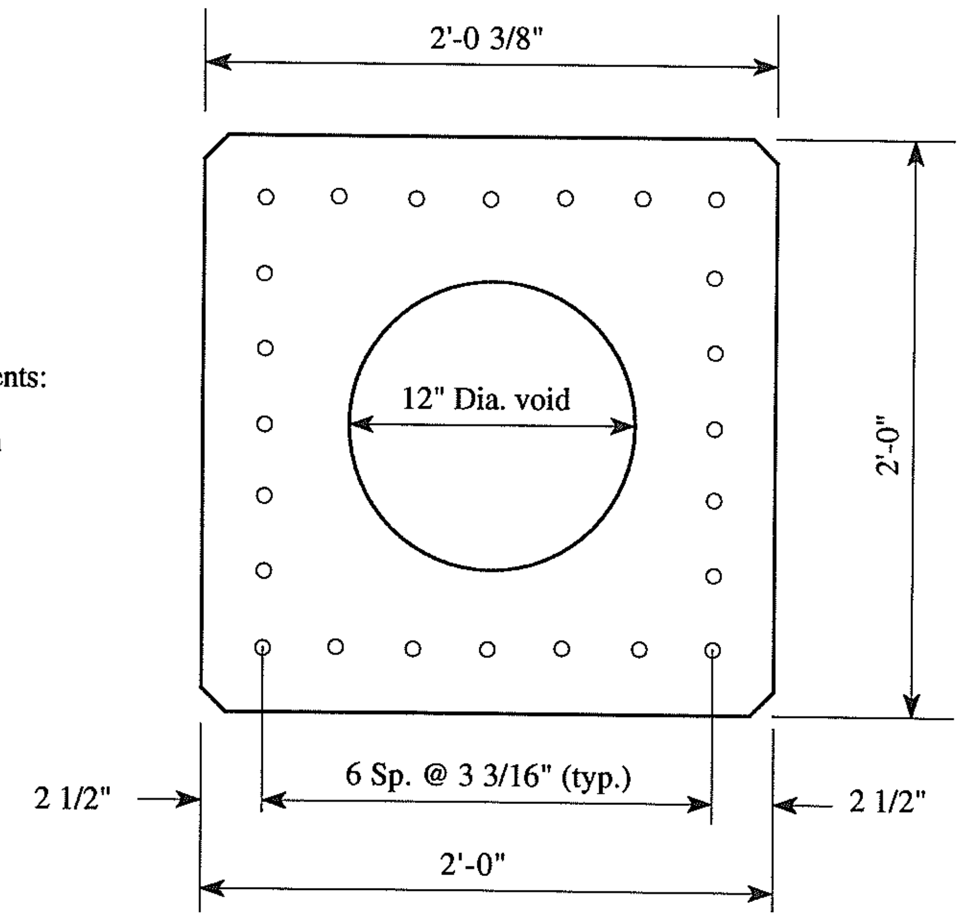
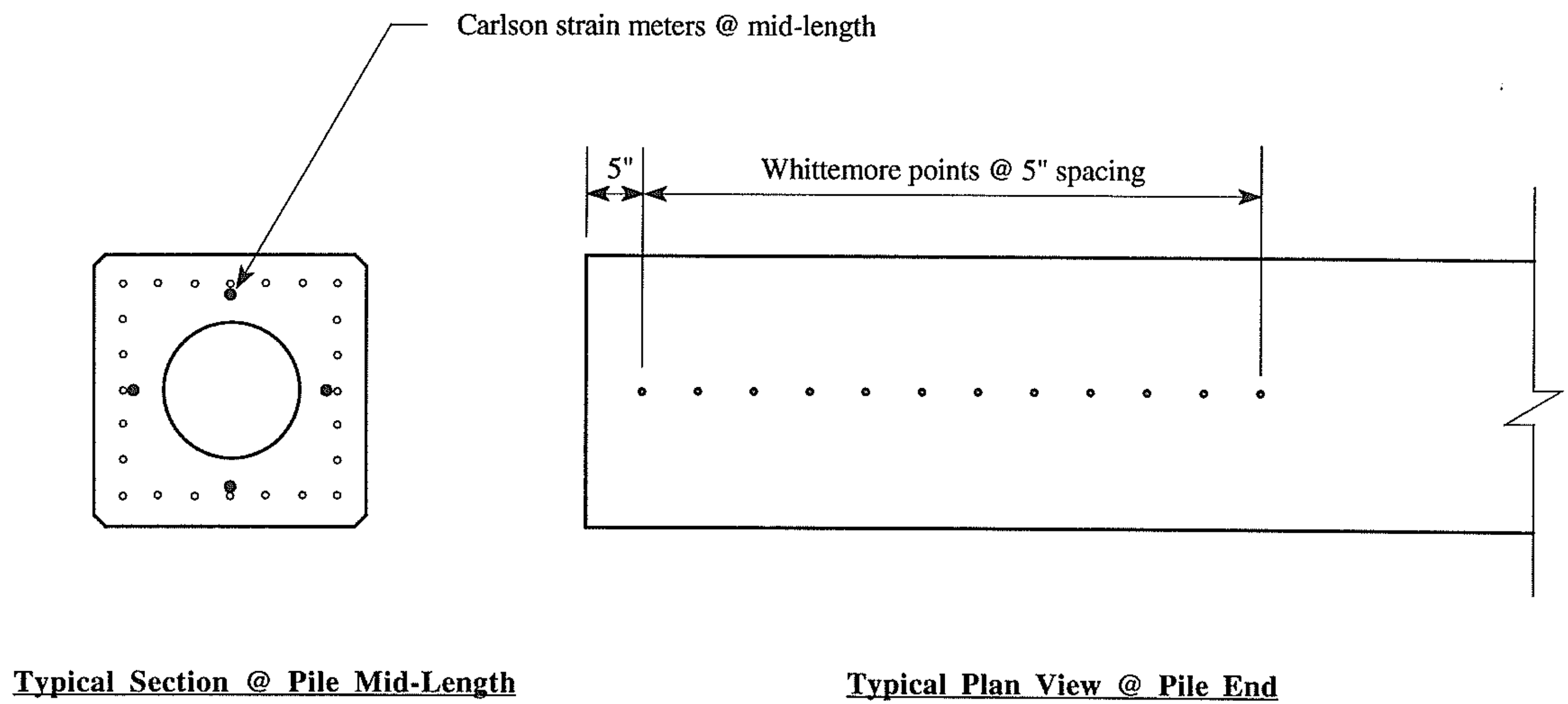


Figure 1. Cross section of pile specimens



Metric Equivalents:
1 ft = 0.305 m
1 in = 25.4 mm

Figure 2. Instrumentation for pile specimens P1, P2, and P3

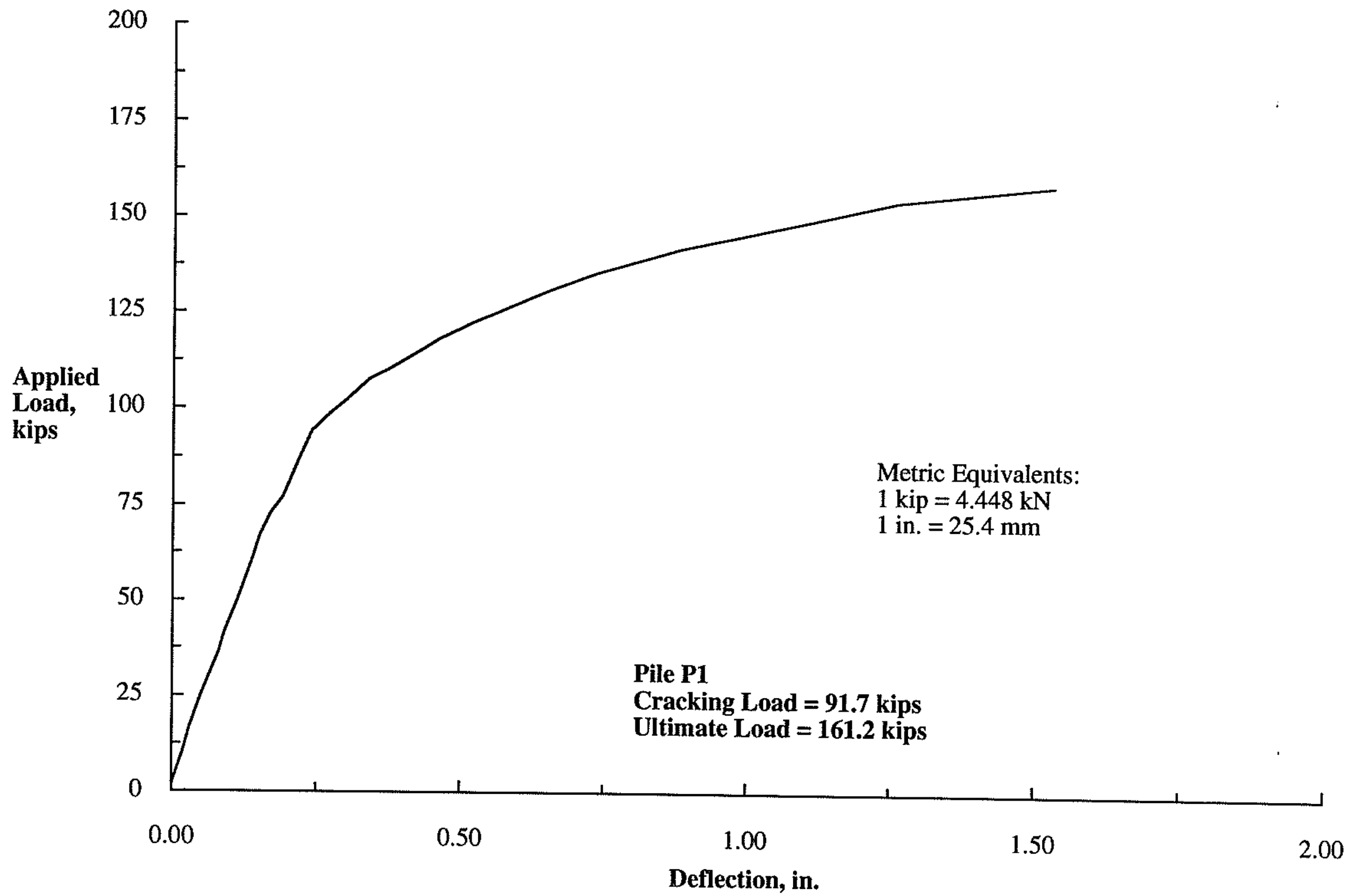


Figure 5. Load-deflection curve for pile specimen P1

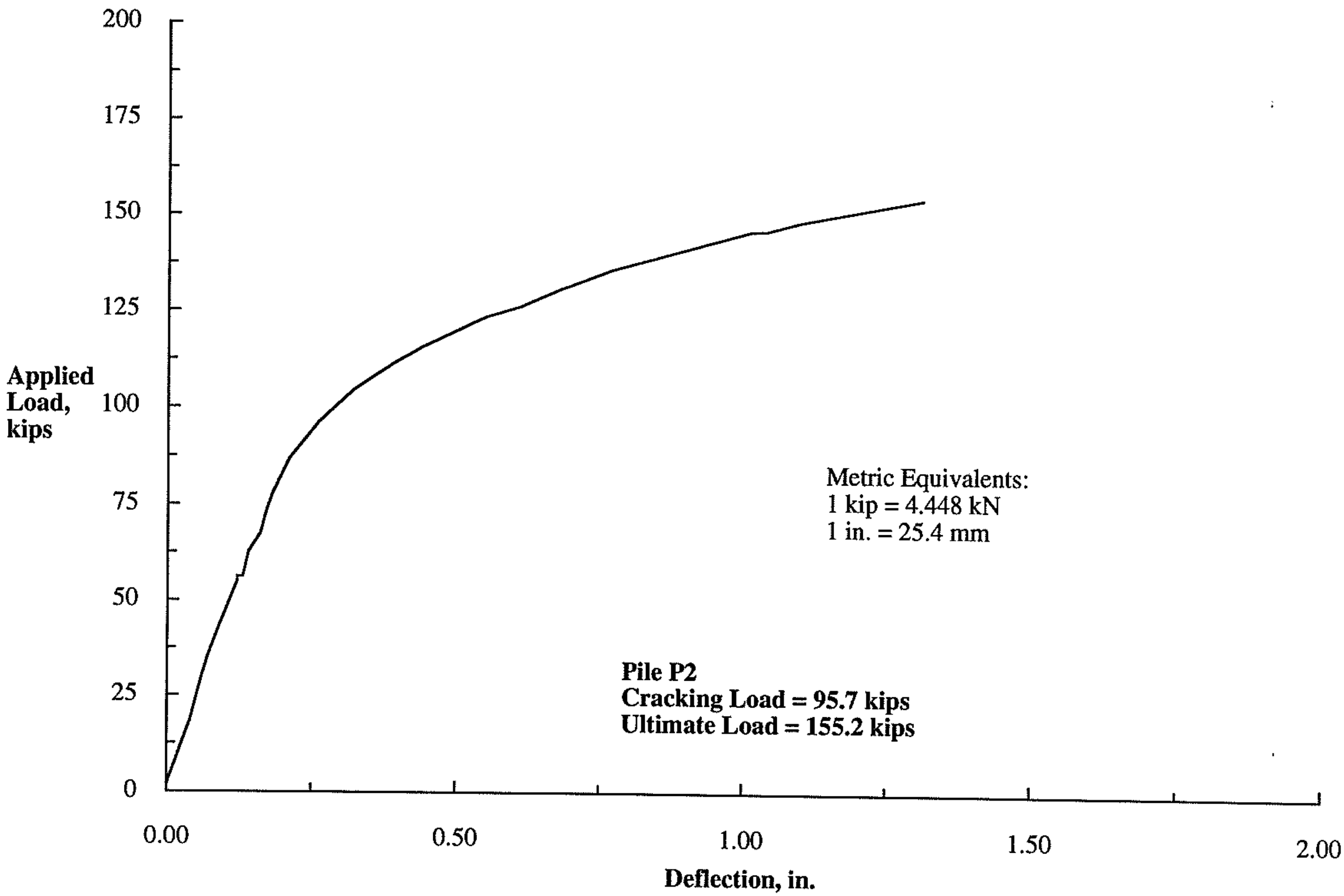


Figure 6. Load-deflection curve for pile specimen P2

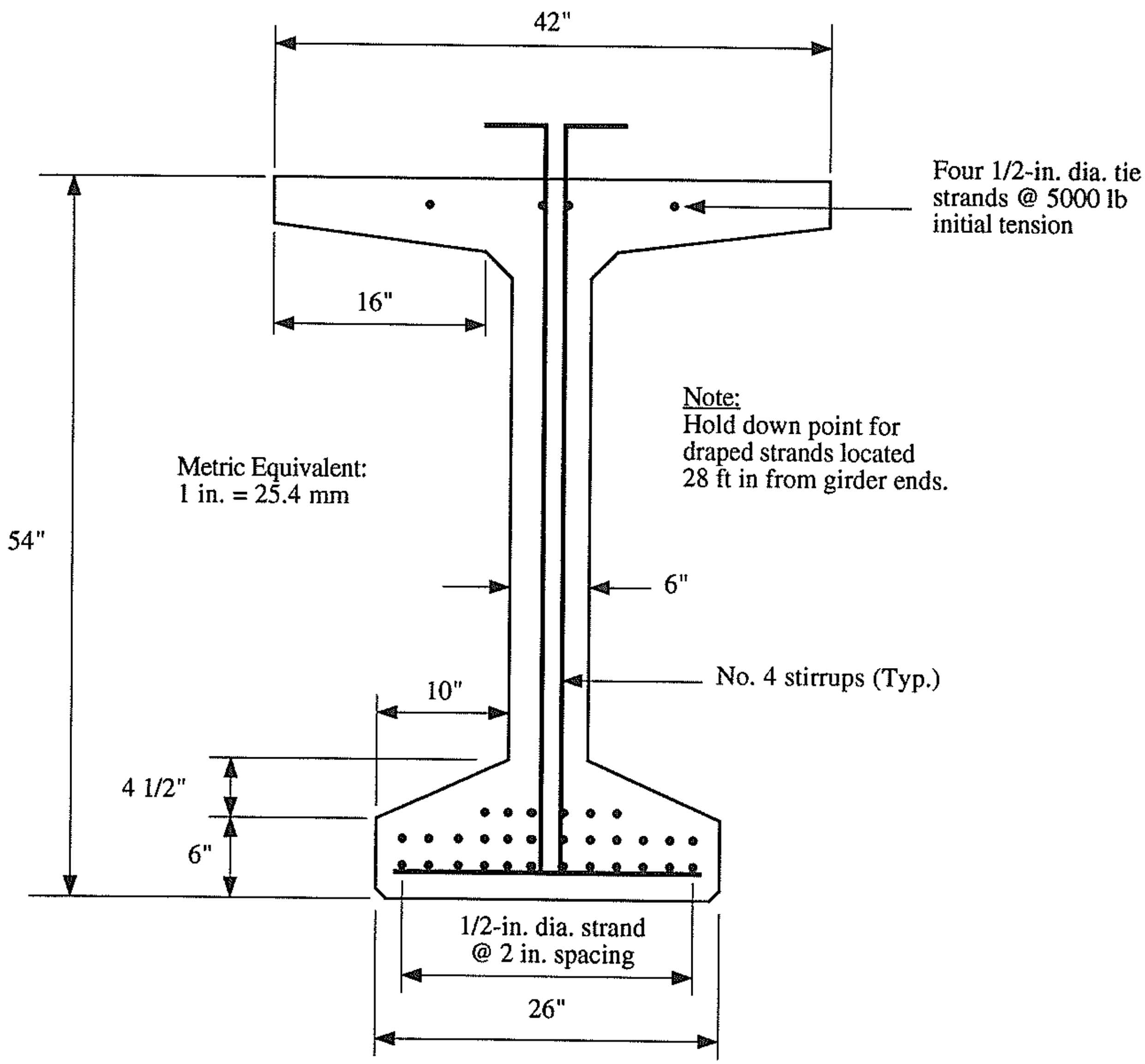


Figure 9. Cross section at mid-span for girder specimens

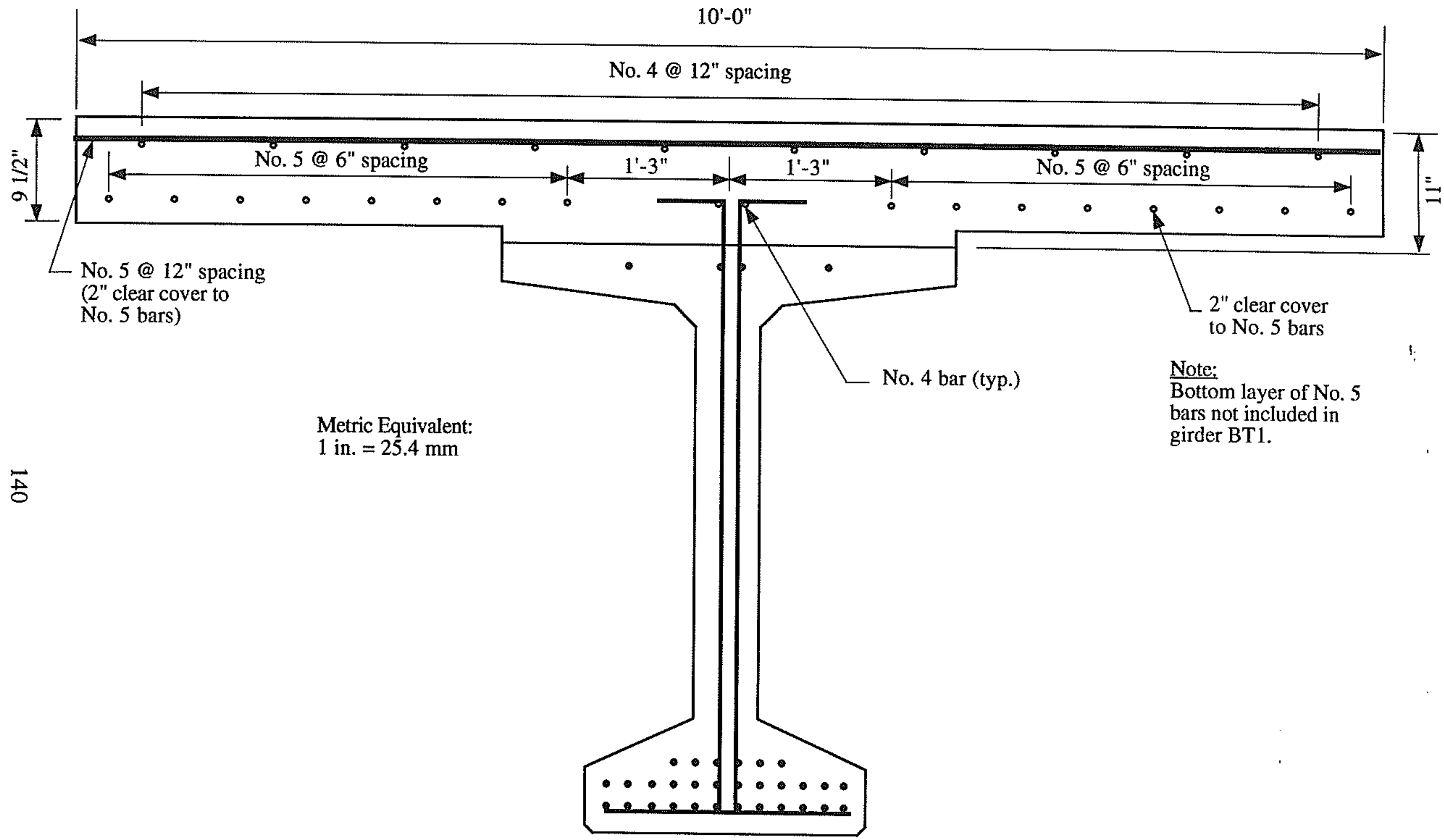


Figure 10. Deck slab dimensions and reinforcement details for girder specimens

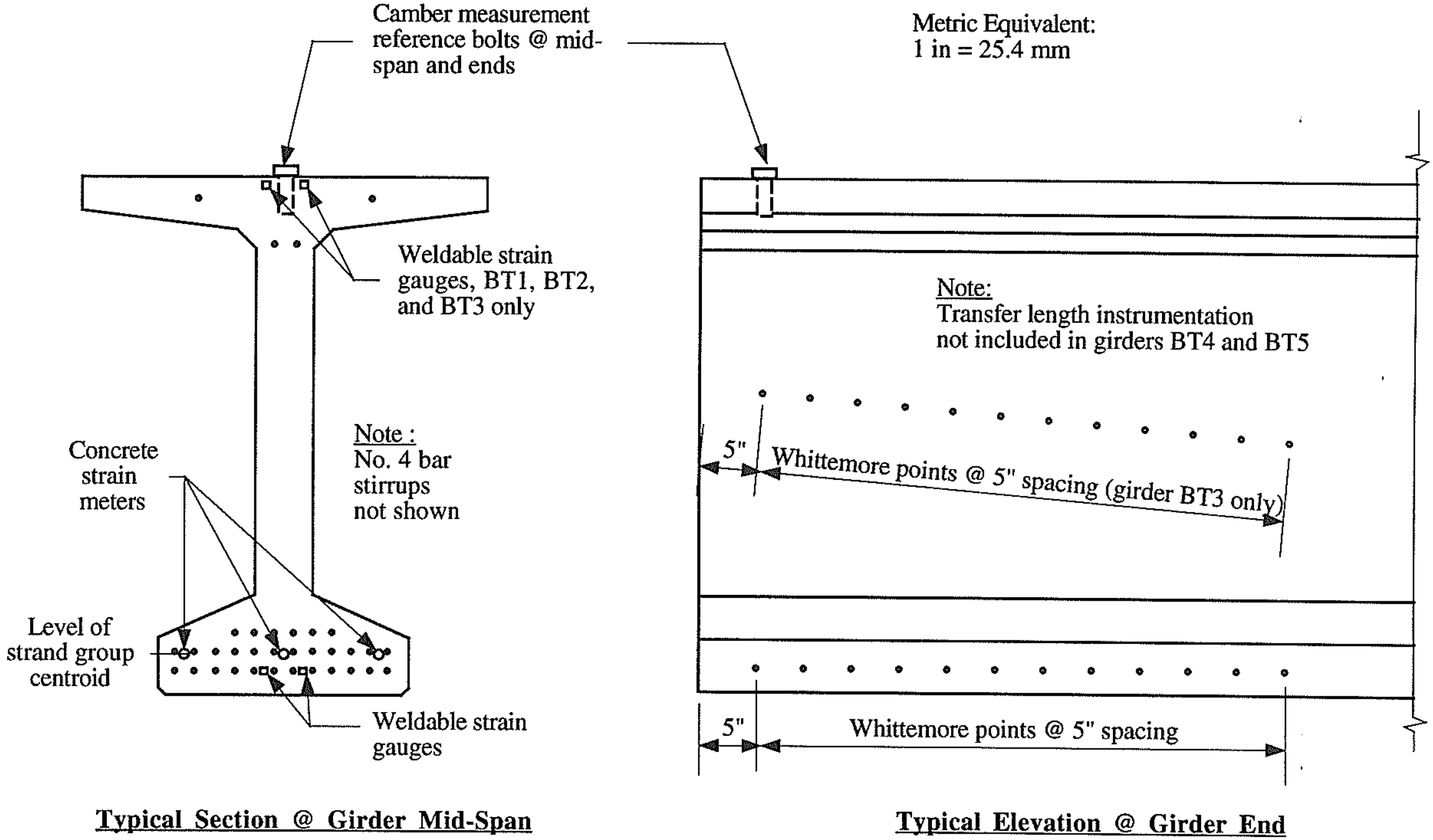


Figure 11. Instrumentation for girder specimens

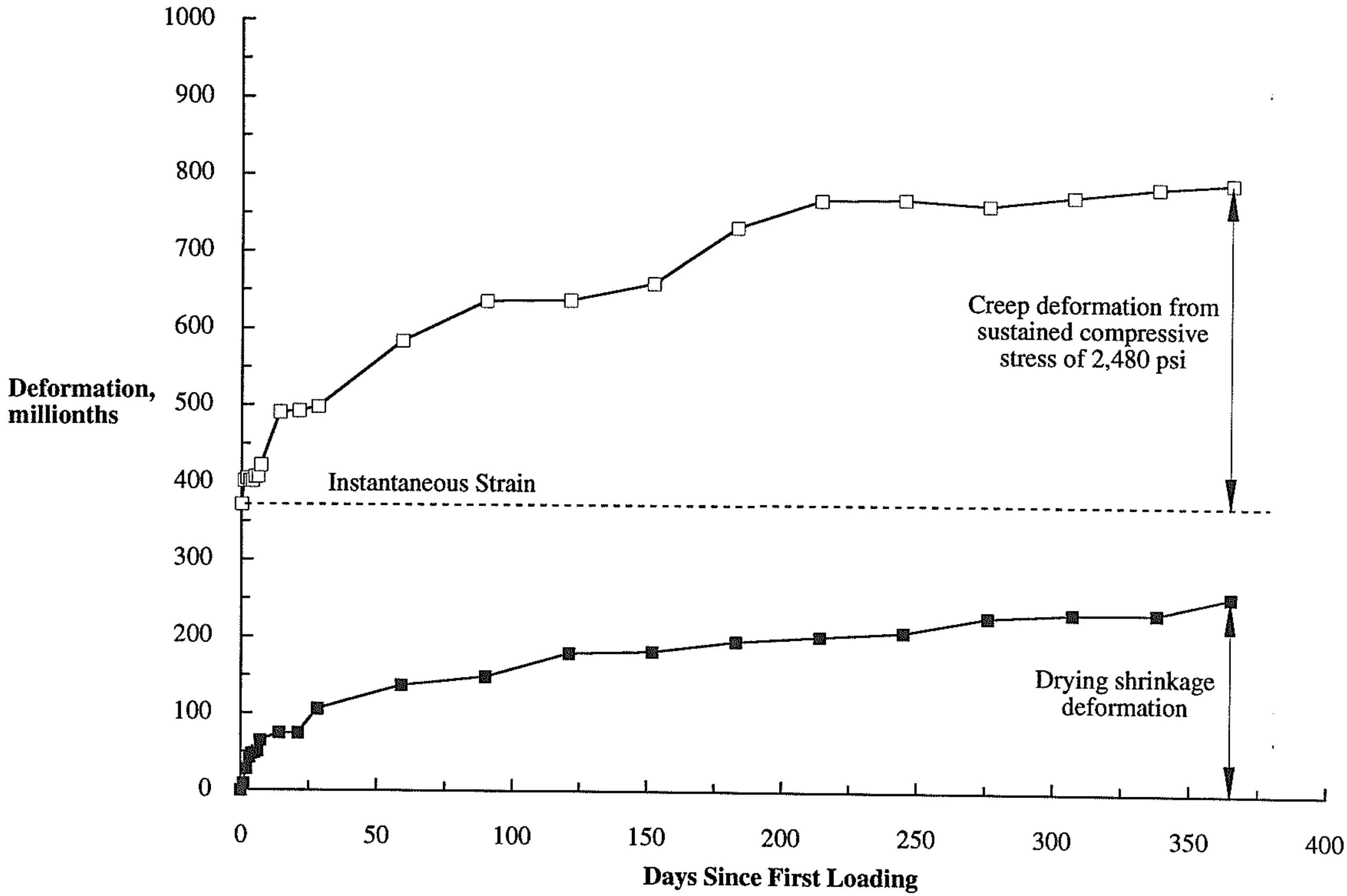


Figure 12. Measured creep and shrinkage deformations for cylinders loaded at 7 days

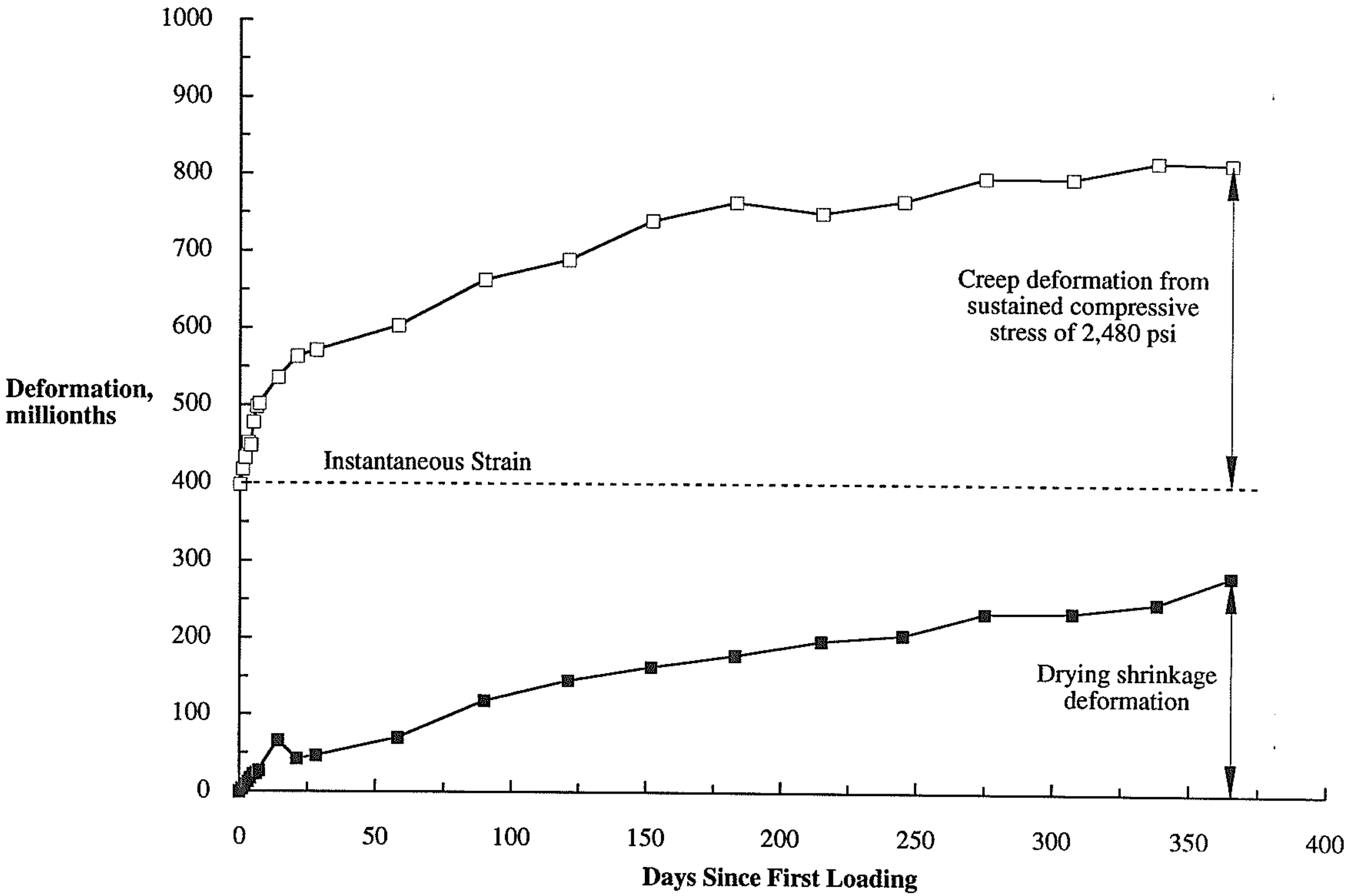


Figure 13. Measured creep and shrinkage deformations for cylinders loaded at 28 days

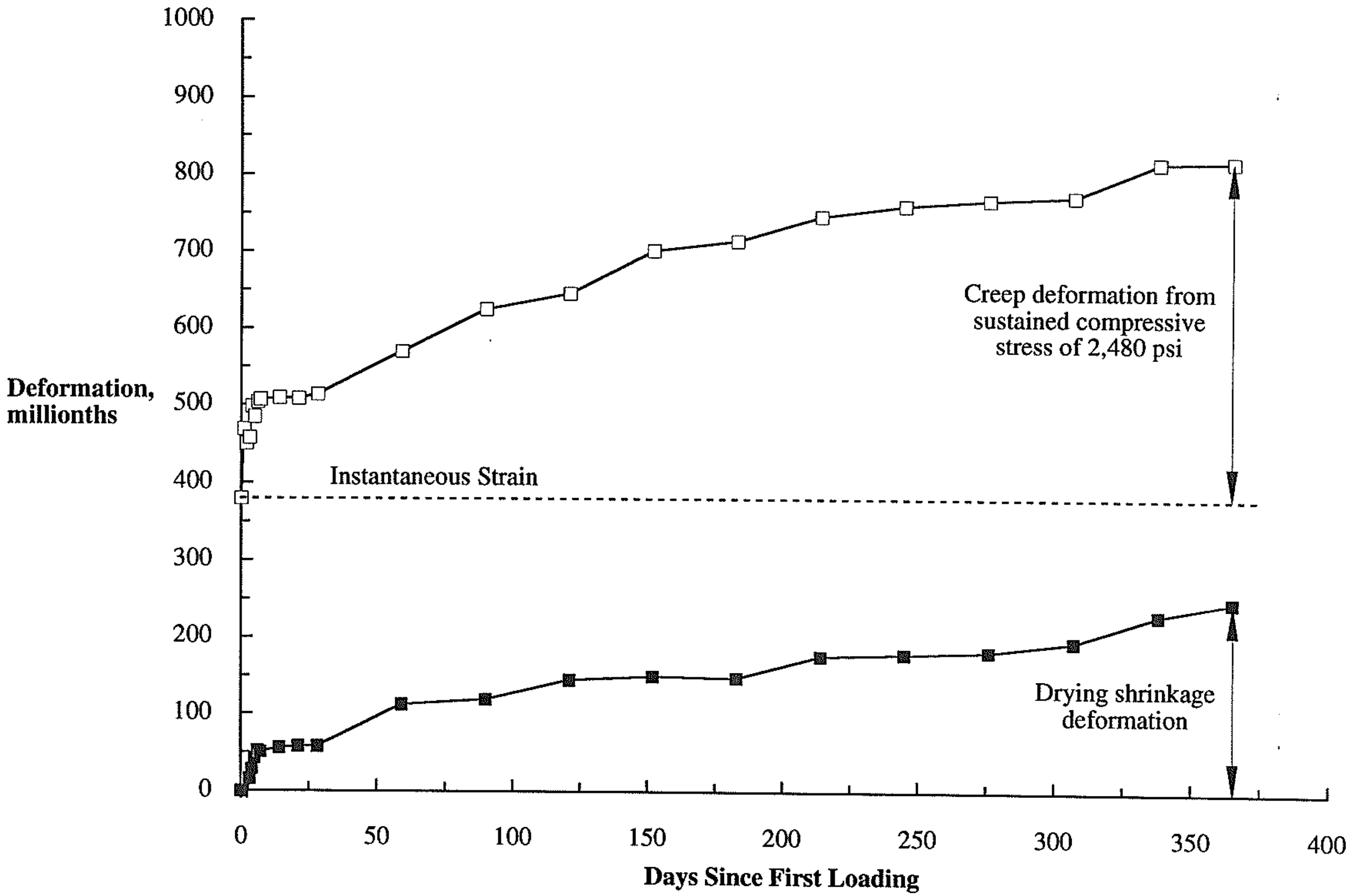


Figure 14. Measured creep and shrinkage deformations for cylinders loaded at 56 days

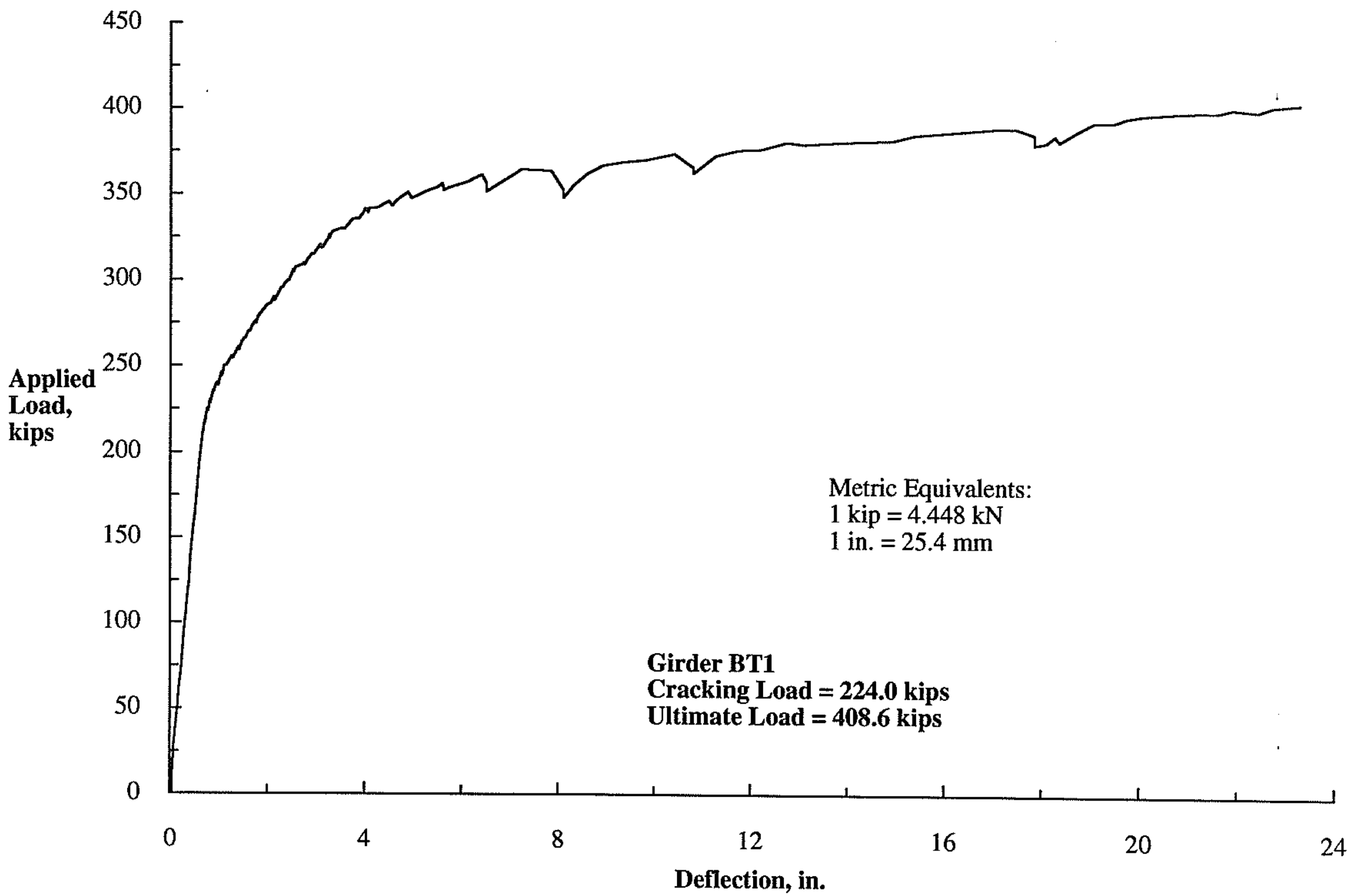


Figure 17. Load-deflection curve for girder specimen BT1

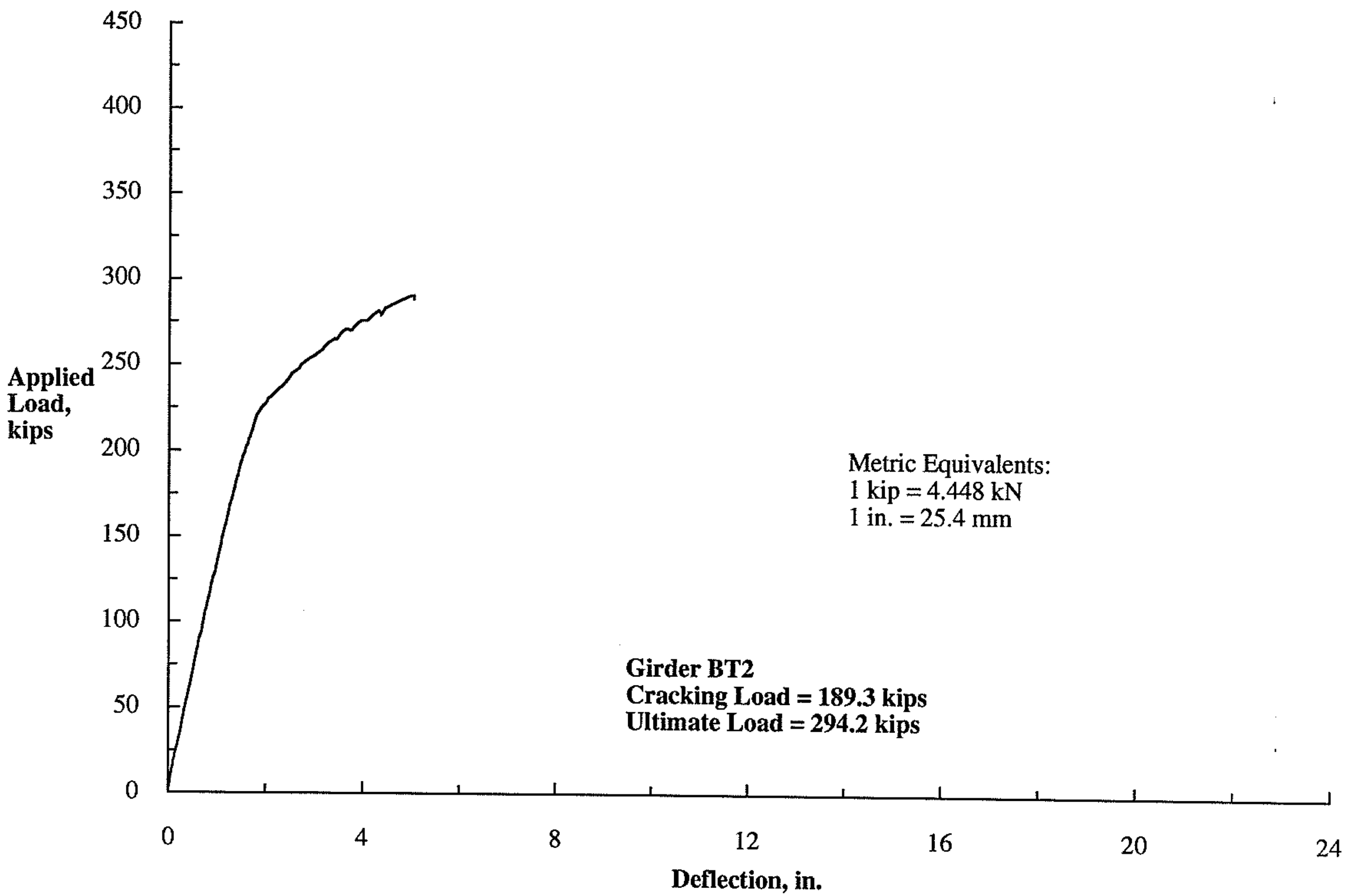


Figure 18. Load-deflection curve for girder specimen BT2

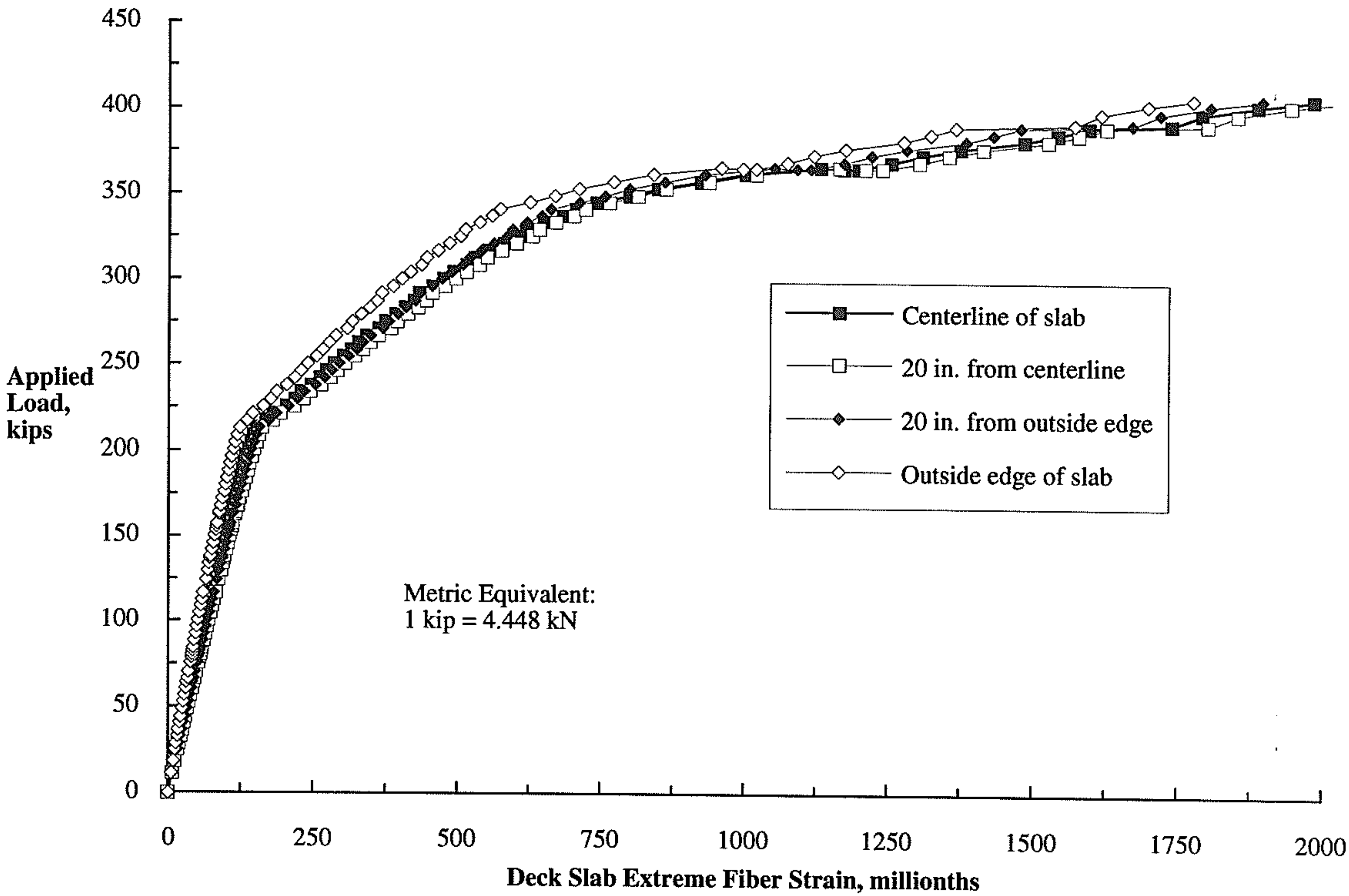
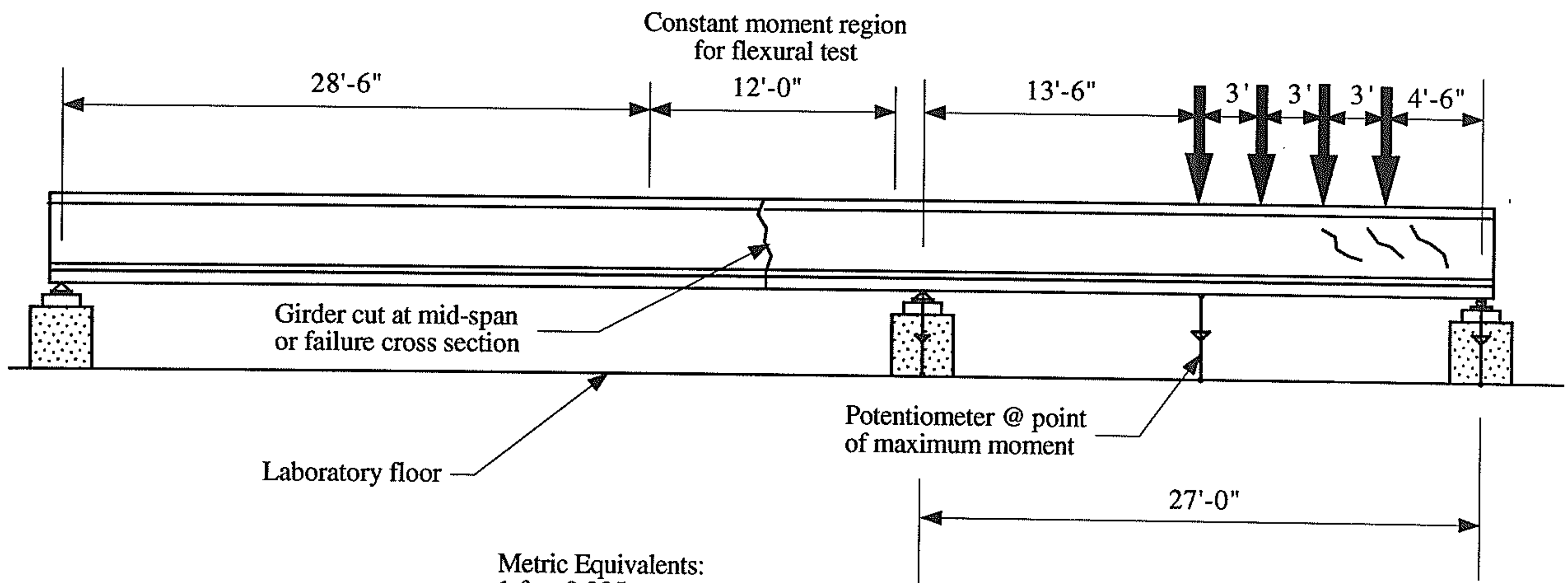


Figure 25. Load-deck slab surface strain for girder specimen BT3



Metric Equivalents:
1 ft = 0.305 m
1 in. = 25.4 mm

Figure 26. Girder specimen shear test setup

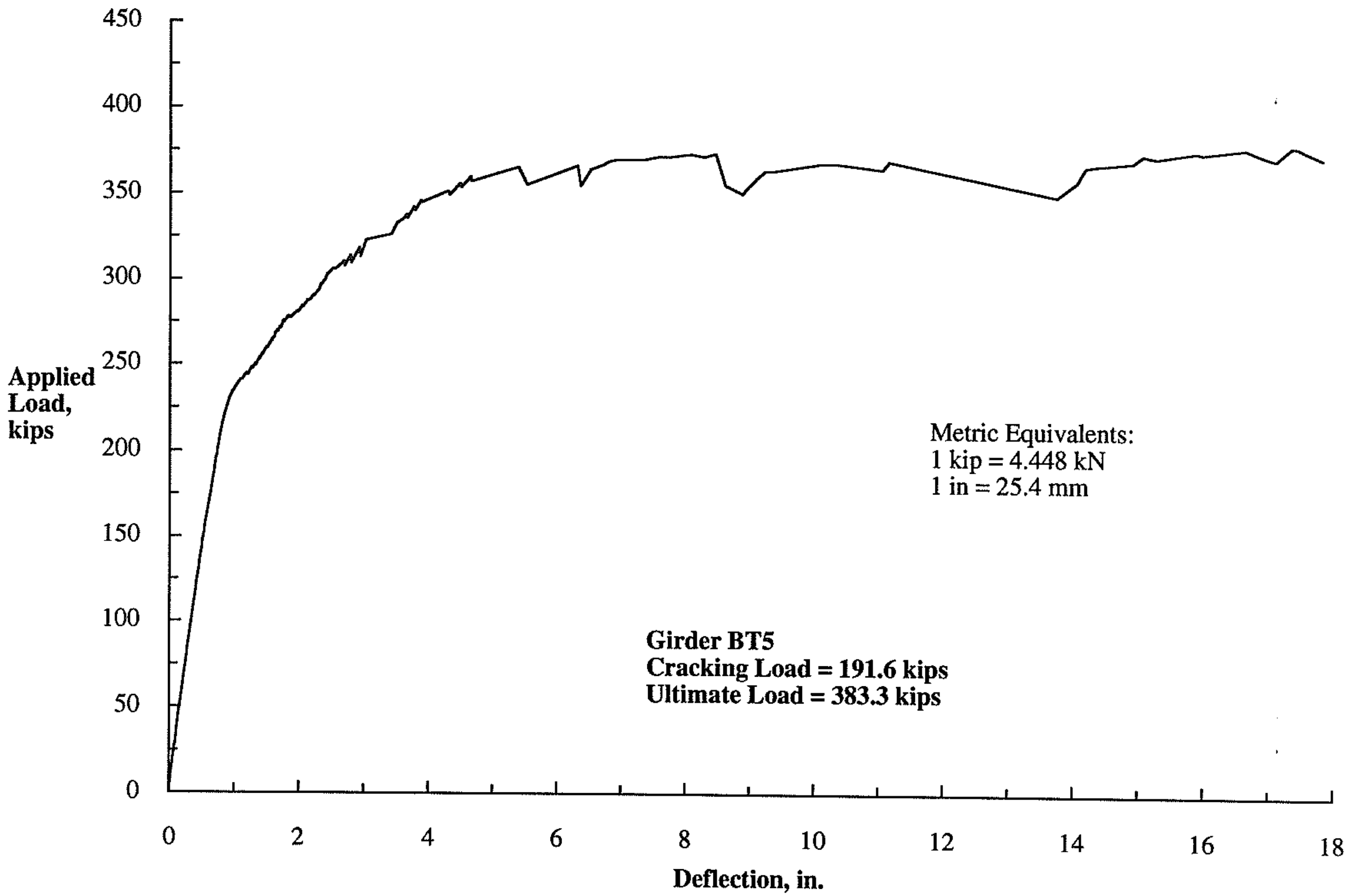


Figure 37. Load-deflection curve for girder specimen BT5

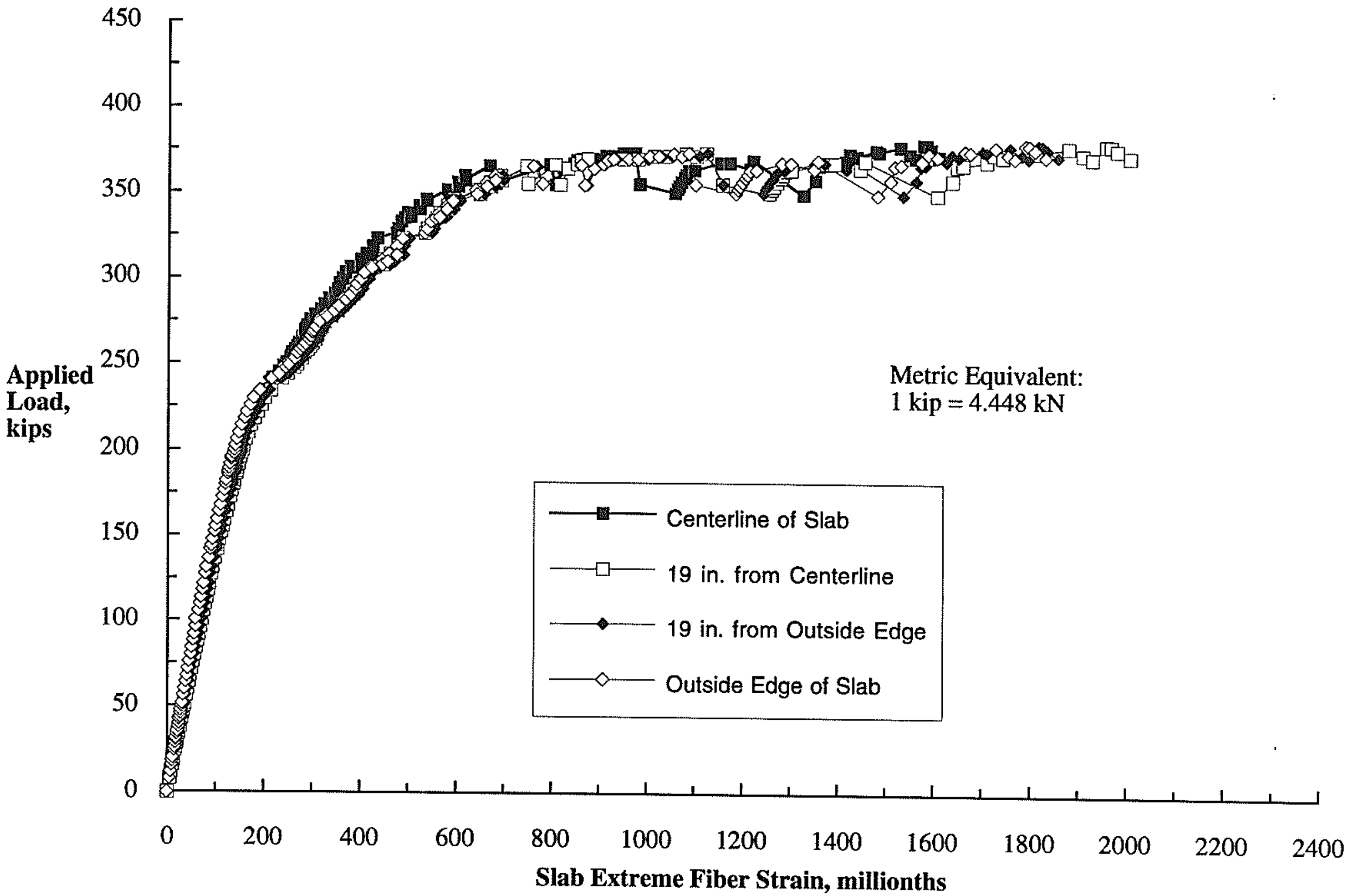


Figure 38. Load-deck slab surface strain for girder specimen BT5

APPENDIX A

MATERIALS AND FABRICATION DETAILS OF PILE SPECIMENS USED FOR FLEXURAL TESTS

A.1 MATERIALS

A.1.1 Aggregate

The sand for the concrete mix was obtained from the Campbell Pit located off U.S. Highway 29, near Flowington, Alabama. The coarse aggregate was obtained from the Calera Quarry located just west of the I-65/U.S. 31 Interchange, north of Calera, Alabama. The coarse aggregate was from the Newala limestone producing formation and was considered high calcium and dolomitic limestone. Both aggregates were tested and found to be in conformance with the Louisiana Department of Transportation and Development (LaDOTD) Standard Specifications for aggregates (35). Sieve analysis of the aggregates used is reported in Table A.1.

A.1.2 Cement

Type I portland cement was used for all pile test specimens. The cement was manufactured by the Citadel Corporation and was certified to conform to ASTM Standard C150-89, Specifications for Portland Cement (36).

A.1.3 Water

The water used in the concrete was obtained from the Southern Prestressed, Inc. underground well. The water conformed to LaDOTD Standard Specifications (35) for water used in portland cement concrete.

TABLE A.1

GRADATION ANALYSIS OF FINE AND COARSE AGGREGATE

Fine Aggregate - Campbell Sand

Sieve #	Weight Retained	Percent Retained	Percent Passing
3/8 in.	0	0	100
4	15	3	97
8	61	11	89
16	110	21	79
30	183	34	66
50	326	61	39
100	482	91	9
Pan	532		

Coarse Aggregate - #67 Stone from Calera Quarry

Sieve #	Weight Retained	Percent Retained	Percent Passing
1-1/2 in.	0	0	100
1 in.	0	0	100
3/4 in.	101	4	96
1/2 in.	783	33	67
3/8 in.	1,363	58	42
4	2,193	93	7
8	2,301	98	2
Pan	2,357		

A.1.4 Admixtures

Four admixtures were used in the concrete mix. These were as follows:

1. MONEX MIGHTY 150 high-range water reducer conforming to ASTM C494-90 (37) Type A & F. This item is on the LaDOTD Qualified Products List as #5864.

2. MONEX SEPTAIR air entraining admixture conforming to ASTM C260-86 (38). This item is on the LaDOTD Qualified Products List as #5855.

3. EMSAC DA silica fume - densified 100 percent solids. Since silica fume is still considered experimental by the LaDOTD, this product is not on the Qualified Products List.

4. DUNDEE Class C fly ash conforming to ASTM C 618-91 (39). This item does not have a Louisiana Department of Transportation and Development product source code.

A.1.5 Prestressing Strand

The low-relaxation prestressing strand used in the pile specimens was domestically fabricated using domestically manufactured steel and was obtained from Florida Wire and Cable Company of Jacksonville, Florida. The nominal diameter of the strand was 1/2 in.(13 mm). The strand conformed to ASTM A-416-88, Specifications for Steel Strand, Uncoated Seven Wire Stress Relieved for Prestressed Concrete (26). All prestressing strand used in the pile specimens came from a single coil (858732). A test report obtained from the manufacturer indicated the following material properties:

Coil No.: 858732

Breaking Load	43,505 lb (193.5 kN)

Load @ 1% Elongation	40,447 lb (179.9 kN)
Elongation	5.1%
Modulus of Elasticity	28,600,000 psi (197.2 GPa)

A.1.6 Web Reinforcement

The spiral reinforcement used in the pile specimens was domestically manufactured by Ivy Steel Products Corporation of Houston, Texas. The W4.5 wire had a nominal diameter of 1/4 in. (6.4 mm). The average tensile strength of four samples tested by the manufacturer was 96,664 psi (666 MPa).

A.2 FABRICATION

Three pile specimens, designated as piles P1, P2, and P3, were fabricated by Southern Prestressed, Inc. at their plant in Pensacola, Florida, in accordance with the shop drawings shown in Figure A.1. The three pile specimens were fabricated in a single 120 ft 10 in. (36.8 m) casting bed. All work was performed by the fabricator except for installation of the instrumentation. All phases of the pile specimen fabrication and preparation of the control specimens were observed and supervised by the research personnel to insure all quantities and measurements were known accurately.

A.2.1 Placement of the Longitudinal Strand

The seven wire prestressing strands were drawn from a single reel located in an outdoor storage area adjacent to the casting bed. Each strand was taken from the reel and threaded through a template at each end of the casting bed. The strands were tensioned using a single strand jack manufactured by G.T. Bynum. The jack had been calibrated within the past six months to have an average gauge reading error of less than one percent.

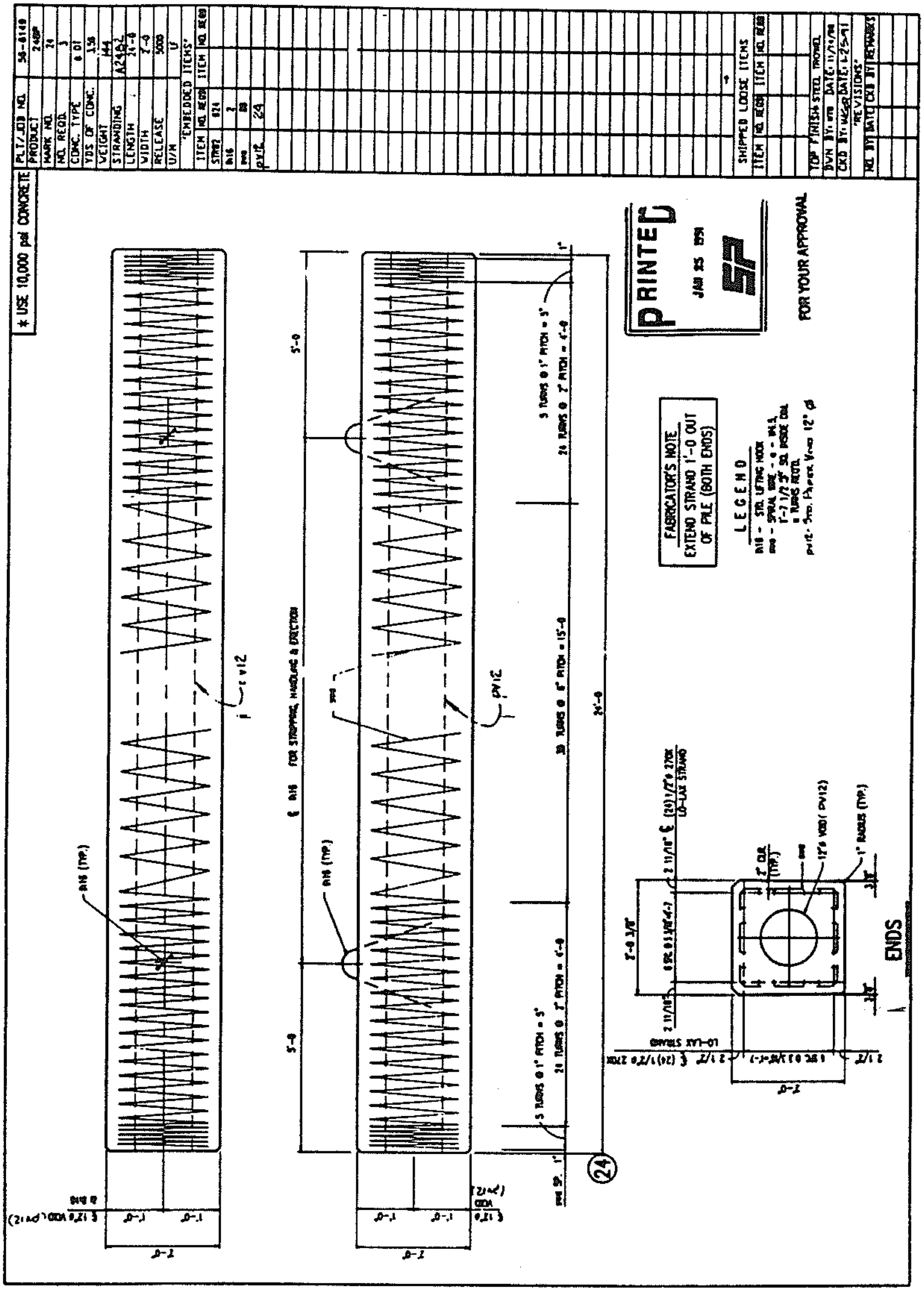


Figure A.1. Shop drawing for pile specimens P1, P2, and P3.

A.2.2 Prestressing of the Strand

In accordance with the present AASHTO code, each of the strands was initially stressed to a value of 0.75 f_{pu} or 30,980 lb (138 kN). In order to accomplish this, the strands were initially tensioned to a proof load of 2,000 lb (8.9 kN) based on gauge readings. After being subjected to the proof load, the strands were marked and stressed to an average elongation of 9.6 in. (244 mm). The load corresponding to this elongation can be obtained from the following expression:

$$e = \frac{PL}{AE}$$

where:

e = elongation of strand, in.

P = load per strand, lb

L = nominal length between stressing ends, 120 ft-10 in. (36.8 m)

A = nominal steel area of strand, 0.153 in.^{2} (98.7 mm^2)

E = modulus of elasticity of the strand, 28,600,000 psi (197.2 GPa)

Using the above expression, the load in the strand beyond the proof load is calculated to be 28,970 lb (129 kN). When the proof load is added to this value, a total force of 30,970 lb (138 kN) per strand results. The total load applied to each strand was confirmed using the pressure gauge readings of the jack. The jack gauges were calibrated within six months of the date of member fabrication and were found to have an average gauge reading error of less than one percent.

A.2.3 Placing of Spiral Reinforcing and Form Erection

The spiral reinforcing was arranged into its final position after the full prestress force had been applied to the longitudinal strand. The 12-in. (305-mm) diameter void in the center of the pile was formed using a cardboard tube. The cardboard tubes were hung from short lengths of steel angles that were bolted to the top of the forms. These steel angles also served to hold the cardboard tube down during the concrete pour. The exterior sides of piles were formed by a steel pan with an open top. The forms could not be broken apart, hence the piles were removed by lifting them out of the forms.

A.2.4 Installation of Instrumentation

After the final tensioning of the strands, Carlson strain meters were installed adjacent to the strands at mid-length in four locations. The location of the strain gauges are shown in Figure 2. The strain meters on the lower face of the pile were Carlson model A-10 gauges, while those on the sides and top face were Carlson model M-8. The model A-10 was used on the lower face because it is a heavier, more rugged strain meter than the M-8 and was better able to withstand fabrication procedures. Each of the strain gauges was fixed to the inside side of the strand using duct tape.

The Carlson strain meter has the capacity to measure both longitudinal strains and temperature. The meter contains two coils of steel wire, one of which increases in length and electrical resistance when a deformation occurs, while the other decreases with deformation. The ratio of the two resistances is independent of temperature, except for thermal expansion, and therefore the change in resistance ratio is proportional to strain. The total resistance is independent of strain, since one coil increases the same amount as the other decreases due to change in length of the body of the meter. Thus, the total resistance is proportional to temperature.

After installation, each strain gauge was tested to insure it was functioning correctly. Each of the strain meters were read after casting to verify that no damage resulted from the fabrication procedures.

Just before the prestressing strands were cut, a longitudinal line of Whittemore mechanical strain gauge points were glued to the top surface of each pile at both ends, as shown in Figure 2. Sika Dur epoxy manufactured by Sika Corporation was used to glue the gauge points. The gauge points were spaced exactly 5 in. (127 mm) apart, between 5 in. (127 mm) and 60 in. (1.52 m) from the ends of the piles. Using various pairs of Whittemore points, concrete surface strains were measured using a 10-in. (254-mm) Whittemore gauge. Concrete surface strain readings were used to determine the strand transfer length required for full development of the prestress strand in the concrete. The longitudinal distance from the ends of the pile at which the concrete surface strains became somewhat uniform provided an indication of the transfer length.

A.2.5 Concrete Mixing, Placement and Curing

Casting of the pile specimens began at 9:40 a.m. on March 5, 1991, and was completed at 10:45 a.m. on the same day. The daytime temperature on the day of casting ranged from 60°F (16°C) to 75°F (24°C). The concrete mix used for the piles was designed to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa). The concrete was mixed in the fabricator's batch plant located adjacent to the pile casting bed. The batch plant had a single four cubic yard (3.1 m^3) Helzel pan type central mixer.

The fine aggregate was initially weighed to the required amount with adjustments made for the free moisture content. The coarse aggregate was then cumulatively weighed with the fine aggregate in the scales located above the central mixer. The cement was weighed in another set of scales also located above the central mixer. The fly ash and silica fume were added using weighed bags. Water and admixtures were added and controlled by an automatic dispenser/meter. The scales and automatic dispenser had been calibrated less than three months before the date of fabrication. The weighing dials on the batching equipment were graduated in the following increments: cement, 5 lb (2.27 kg); aggregate, 20 lb (9.07 kg); water, 1 lb (0.45 kg). The concrete was mixed in four yard batches. Specific proportions of the five batches used for pile fabrication are given in Table A.2.

After mixing, the concrete was then placed into a screw type transporter, taken to the casting bed and placed in the forms. Each pile segment required slightly less than four cubic yards of concrete, hence each member was cast from a separate batch of concrete. During casting, internal vibration of the concrete, using hand held vibrators, was performed carefully to insure proper consolidation and to avoid damage to any of the internal instrumentation. Six standard 6 x 12 in. (152 x 305 mm) cylinders and two 6x6x20 in. (152x508 mm) beams were made from each batch. After casting, the pile specimens were cured for approximately 24 hours under a tarpaulin consisting of 1/4-in. (6.4 mm) foam insulation sandwiched between two layers of waterproof canvas.

A.2.6 Release of Prestress

Approximately 21 hours after casting, one control cylinder representing each of the three concrete batches/pile specimens was removed from the casting bed and tested to determine compressive strength. A minimum of 5,000 psi (34.5 MPa) was required for release of the prestress. The average compressive strength of the three cylinders tested was 5,515 psi (38 MPa), thus the strength was satisfactory for release of the prestress.

The prestress was released by simultaneously cutting the strands with an acetylene torch at both ends of the casting bed. Then the strands between each pile specimen were cut.

TABLE A.2

CONCRETE MIX PROPORTIONS (per cubic yard)

	Mix Designation (same as pile designation)		
Components	P1	P2	P3
Cement	755 lb	756 lb	754 lb
Fly Ash	79 lb	70 lb	70 lb
Silica Fume	50 lb	50 lb	50 lb
Water	175 lb	175 lb	206 lb
Sand	974 lb	976 lb	976 lb
Stone	1,915 lb	1,915 lb	1,913 lb
Air Entrainment	17.5 oz.	17.5 oz.	17.5 oz.
High-Range Water Reducer	157 oz.	157 oz.	157 oz.

* Fabricator recorded 5% free moisture on the sand and 0.0% free moisture on the stone.

Metric Equivalents:

1 lb = 0.454 kg

1 oz. = 29.574 cc

Instrumentation readings were taken and recorded just before release and after all strands were cut.

A.2.7 Transporting and Handling of the Piles

Immediately following release of the strands, the pile segments were lifted out of the forms and transported to another location in the casting yard. The piles were lifted using the lifting inserts shown on the fabrication drawing in Figure A.1. The control specimens remained with the pile segments until the time of shipment. Seven days after release, the pile segments were loaded onto a flatbed truck and transported to Construction Technology Laboratories, Inc. (CTL) in Skokie, Illinois. The control specimens were shipped on the same vehicle, packed in a wooden crate containing Styrofoam packing material. The pile segments and the control specimens were stored indoors at CTL until time of testing.

APPENDIX B

MATERIALS AND FABRICATION DETAILS OF GIRDER SPECIMENS USED FOR STATIC TESTS

B.1 MATERIALS

B.1.1 Aggregate

The sand for the concrete mix used in the bulb-tee girders was obtained from Superior Sand in Jemison, Alabama. The coarse aggregate was obtained from Vulcan Stone in Helena, Alabama. Both aggregates were tested and found to be in conformance with the Louisiana Department of Transportation and Development (LaDOTD) Standard Specifications for aggregates (35). Sieve analyses of the aggregate used is reported in Table B.1.

B.1.2 Cement

Type III portland cement was used for all bulb-tee test specimens. The cement was manufactured by the Citadel Corporation and was certified to conform to ASTM Standard C150-89 , Specifications for Portland Cement (36).

B.1.3 Water

The water used in the concrete was obtained from the city water supply of Pelham, Alabama. The water conformed to LaDOTD Standard Specifications (35) for water used in portland cement concrete.

TABLE B.1

GRADATION ANALYSIS OF FINE AND COARSE AGGREGATE

Fine Aggregate - Superior Sand

Sieve #	Weight Retained	Percent Retained	Percent Passing
3/8 in.	0	0	100
4	10	2	98
8	45	9	91
16	81	16	84
30	198	40	60
50	448	90	10
100	495	99	1
Pan	500		

Coarse Aggregate - #67 Stone from Helena Quarry

Sieve #	Weight Retained	Percent Retained	Percent Passing
1-1/2 in.	0	0	100
1 in.	0	0	100
3/4 in.	250	5	95
1/2 in.	-	-	-
3/8 in.	3,751	72	28
4	5,085	98	2
8	5,175	99	1
Pan	5,227		

B.1.4 Admixtures

Four admixtures manufactured by W. R. Grace & Co. were used in the concrete mix. These were as follows:

1. W.R.D.A. 79 water reducer conforming to ASTM C494-90 (37) Type A & D. This item is on the LaDOTD Qualified Products List as #5832.

2. DARAVAIR air entraining admixture conforming to ASTM C260-86 (38). This item is on the LaDOTD Qualified Products List as #5850.

3. W.R.D.A. 19 Superplasticizer conforming to ASTM C-494-90 as a Type A, or as a Type F admixture. This item is on the LaDOTD Qualified Products List as #5866.

4. FORCE 10,000D silica fume - densified 100 percent solids. Since silica fume is still considered experimental by the LaDOTD, this product is not on the Qualified Products List.

B.1.5 Prestressing Strand

The low-relaxation prestressing strand used in the bulb-tee specimens was domestically manufactured and was obtained from American Spring Wire Corp. of Bedford Heights, Ohio. The nominal diameter of the strand was 1/2 in. (12.7 mm). The strand conformed to ASTM A-416-88, Specifications for Uncoated Stress Relieved Wire for Prestressed Concrete (26). The prestressing strand used in the girder specimens came from two different coils (78979 and 78976). A test report was obtained from the manufacturer for coil 78979 only. This report indicated the following material properties:

Coil No.: 78979

Breaking Load	42,550 lb (189.3 kN)
Load @ 1% Elongation	38,900 lb (173.0 kN)
Elongation	5.0%
Modulus of Elasticity	29,230,000 psi (201.5 GPa)

B.1.6 Web Reinforcement

The No. 4, Grade 40, web reinforcement used in the bulb-tee specimens was domestically manufactured by Birmingham Steel Corporation of Flowood, Mississippi. The average yield strength and tensile strength, as determined by the manufacturer, were 52,500 psi (362.1 MPa) and 77,500 psi (534 MPa), respectively.

B.2 FABRICATION

Three bulb-tee specimens, designated as girders BT1, BT2, and BT3, were fabricated by Sherman Prestressed Concrete at their plant in Pelham, Alabama, in accordance with the shop drawings shown in Figure B.1. The three bulb-tee specimens were fabricated in a single 326 ft 8 in. (99.63 m) casting bed. All work was performed by the fabricator except for installation of the instrumentation. All phases of the bulb-tee specimen fabrication and preparation of the control specimens were observed and supervised by the research personnel to ensure all quantities and measurements were known accurately.

B.2.1 Placement of the Longitudinal Strand

The seven wire prestressing strands were drawn from two reels located in an outdoor storage area adjacent to the casting bed. Each strand was taken from the reel and threaded

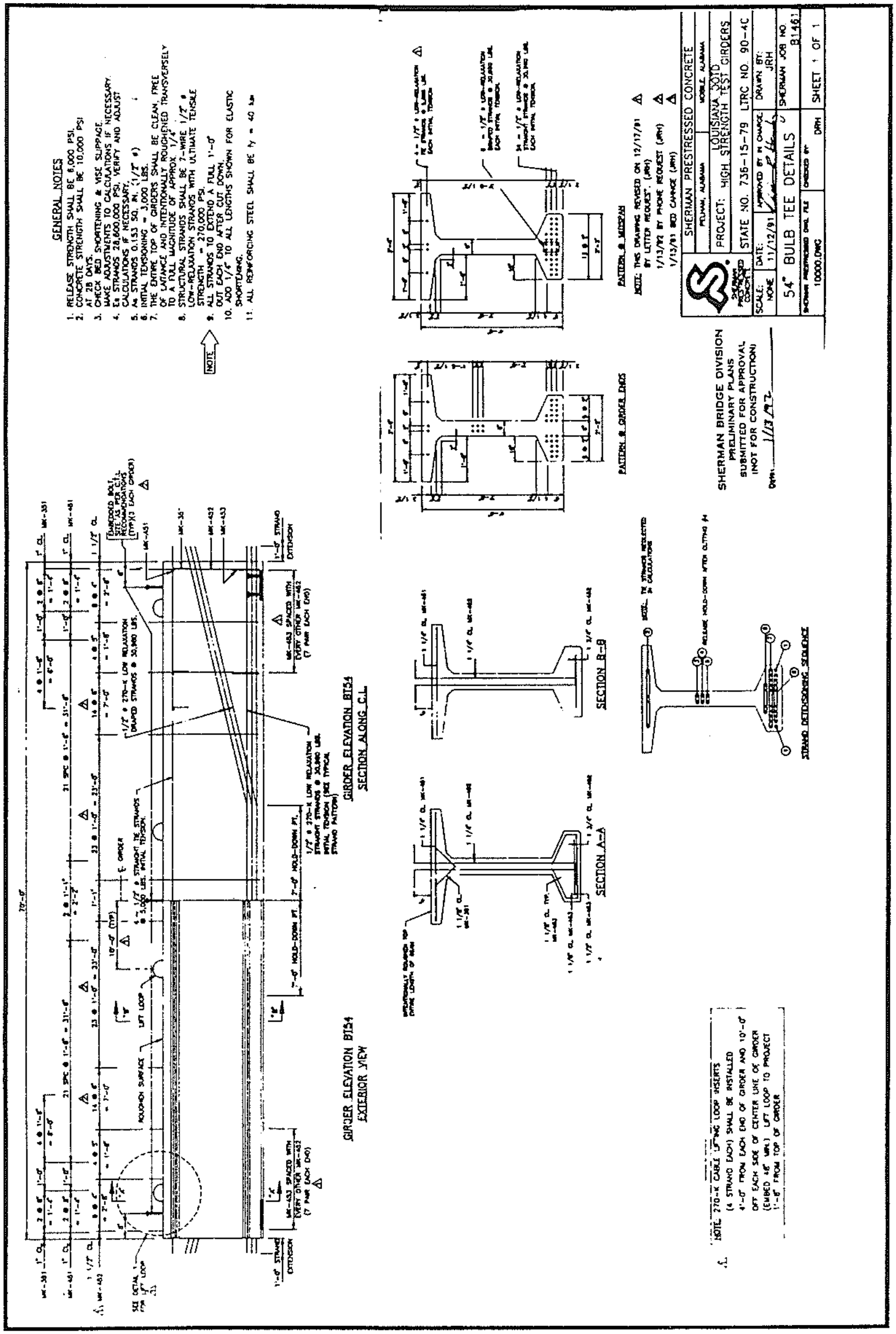


Figure B.1. Shop drawing for girder specimens.

through a template at each end of the casting bed. Intermittent pick-up and hold-down points were positioned to establish the desired strand drape pattern for the six strands coincident with the girder web. The strands were tensioned using a single strand jack manufactured by G. T. Bynum. Calibration tests were not available for the jack, however, the accuracy was verified from load cells placed on selected strands.

B.2.2 Prestressing of the Strand

In accordance with the present AASHTO code, each strand was initially stressed to a value of 0.75 f_{pu} or 30,980 lb (138 kN). In order to accomplish this, the strands were initially tensioned to a proof load of 3,000 lb (13.4 kN). Once this was done, the straight strands were marked and stressed to an average elongation of 25.56 in. (649 mm). Of this elongation, 0.5 in. (12.7 mm) was assumed to be lost in slippage at the dead end and in seating losses at the live end. In the case of the draped strands, the elongation beyond proof load was calculated to be 20.59 in. (523 mm) with 0.5 in. (12.7 mm) lost in slippage and seating losses, and 4.99 in. (127 mm) gained due to pulling the strand up at the drape-points. The load corresponding to the strand elongation can be obtained from the following expression:

$$e = \frac{PL}{AE}$$

where:

e = elongation of strand, in.

P = load per strand, lb

L = nominal length between stressing ends, in.

A = nominal steel area of strand, 0.153 in.^{2} (98.7 mm^2)

E = modulus of elasticity of the strand, 28,600,000 psi (197.2 GPa)

Using the above expression, the load in the straight strand beyond the proof load is calculated to be 27,970 lb (125 kN). When the proof load is added to this value, a total force of 30,970 lb (138 kN) per strand results. Load cells were placed on selected strands to verify jacking loads.

B.2.3 Placement of Web Reinforcement and Form Erection

After the full prestress force had been applied to the straight and draped strands, the web reinforcing was placed in accordance with the shop drawings. The bottoms of the steel exterior side forms for the bulb-tee specimens were anchored to the casting bed. Adjacent sides were tied together across the top.

B.2.4 Installation of Instrumentation

After the final tensioning of the strands, three Carlson strain meters were installed at mid-length of each girder, adjacent to the strands at the level of the strand pattern centroid. In addition to the Carlson meters, two 1/4-in (6.4-mm) diameter reinforcing bars instrumented with weldable wire strain gauges were installed at mid-length near the top and bottom surfaces of each girder. The location of the strain gauges and strain meters are shown in Figure 11. The strain meters were Carlson model A-10. The welded wire strain gauges were model AWC-8b, manufactured by Tokyo Sokki Kenkyujo Company, Ltd. After installation and casting, each strain gauge and strain meter was tested to ensure it was functioning correctly.

Just prior to placing the steel side forms in position, eight aluminum channel strips were attached to the inside surface of the forms. These aluminum channels had attached Whittemore points spaced at five inches (127 mm) on center over a length of 60 in. (1.52 m) and were installed along one side of the lower flange, at both ends of each of the three girders. Aluminum channels were also installed along one side of the web, coincident with the slope of the draped strands, at both ends of girder BT3 only. After casting, and prior to removing the forms, the aluminum channels were detached from the formwork. After form removal, the aluminum channels and Whittemore points remained embedded in the concrete. The channels were then disconnected and removed, leaving only the brass Whittemore points remaining embedded in the concrete. Using the various pairs of Whittemore points, concrete surface strains were measured using a 10-in. (254-mm) Whittemore gauge within the first 60 in. (1.52 m) of each girder end. Concrete surface strain readings were used to determine the strand transfer length required for full development of the prestressing strand in the concrete. The longitudinal distance from the ends of the girder at which the concrete surface strains became somewhat uniform provided an indication of the transfer length.

In order to facilitate measurement of girder camber, three stainless steel bolts were pressed into the top flange concrete of each girder during casting. One bolt was placed at mid-length, while the other two were centered above the sole plate at each end. Girder camber was determined by using a level to sight elevations of the mid-length reference bolt relative to the two end reference bolts.

B.2.5 Concrete Mixing, Placement, and Curing

Casting of the bulb-tee specimens began at 10:30 a.m. on January 22, 1992, and was completed at 1:20 p.m. of the same day. The daytime temperature during the pour ranged from 45°F (7°C) to 55°F (13°C). The concrete mix used for the girders was designed to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa). The concrete was mixed in the fabricator's batch plant located adjacent to the girder casting bed. The batch plant had a single 8 yd^3 (6.1 m^3) drum type mixer.

The mixer was initially charged with the coarse aggregate, the silica fume, the air-entrained agent, 66 percent of the superplasticizer, and about 90 percent of the water. The drum was then turned for about two minutes. Next, the cement, sand, water reducer, and the remainder of the high range water reducer and water were added and mixed for approximately one minute. Aggregate was weighed to the required amount with adjustments made for the free moisture content. The silica fume was added using weighed bags. Water and admixtures were added and controlled by an automatic dispenser/meter. The scales and automatic dispenser had been calibrated within the year prior to the date of fabrication.

On the day of casting, a total of eleven 4-1/4 yd^3 (3.25 m^3) batches were made. The first two batches were rejected due to unacceptable slump levels. The last nine batches were used to cast the bulb-tee specimens. Each bulb-tee specimen required slightly less than 12 yd^3 (9.17 m^3) of concrete, hence each girder was cast from three separate batches of concrete. Specific proportions of the nine batches used for the girder fabrication are given in Table B.2.

After mixing, each batch was placed into a screw type transporter, taken to the casting bed, and placed into the forms. During casting, vibration of the concrete using both hand held vibrators and form vibrators was performed carefully to ensure proper consolidation and to avoid damage to any of the internal instrumentation. Several standard 6 x 12 in. (152 x 305 mm) cylinders and 6x6x20 in. (152x152x508 mm) beams were made from the nine concrete batches. After casting, the bulb-tee specimens were steam cured for 24 hours under a tarpaulin consisting of 1/4-in. (6.4-mm) foam insulation sandwiched between two layers of waterproof canvas. The control specimens were also placed under the tarpaulin to undergo the same curing process as the girders. The steam was applied after the concrete had reached initial set. Initial set was defined in accordance with ASTM C 403-90 (40), Time of Setting of Concrete Mixture by Penetration Resistance. Initial set occurred at 8:10 p.m. on January 22, 1992. Steam was turned on immediately after initial set. Temperature probes at two locations along the casting bed indicated a curing temperature ranging from 120°F (49°C) to 140°F (60°C) during the 24 hours of steam application. Steam was cut off at 8:15 p.m. on January 23, 1992, and the specimens were allowed to cool down slowly over the next 12 hours.

TABLE B.2

CONCRETE MIX PROPORTIONS (per cubic yard)

	Mix Designation								
	Specimen BT1			Specimen BT2			Specimen BT3		
Item	Batch 1-1	Batch 1-2	Batch 1-3	Batch 2-1	Batch 2-2	Batch 2-3	Batch 3-1	Batch 3-2	Batch 3-3
Cement	753 lb	753 lb	753 lb	753 lb	753 lb	753 lb	753 lb	753 lb	753 lb
Silica Fume	82.3 lb	82.3 lb	82.3 lb	82.3 lb	82.3 lb	82.3 lb	82.3 lb	82.3 lb	82.3 lb
Water*	21.6 gal	21.6 gal	21.6 gal	21.2 gal	20.5 gal	20.0 gal	19.8 gal	19.5 gal	19.5 gal
Sand	1,233 lb	1,232 lb	1,233 lb	1,233 lb	1,233 lb	1,233 lb	1,233 lb	1,233 lb	1,233 lb
Stone	2,049 lb	2,215 lb	2,049 lb	2,049 lb	2,049 lb	2,049 lb	2,049 lb	2,049 lb	2,049 lb
Water Reducer	42.4 oz	56.5 oz	56.5 oz	56.5 oz	56.5 oz	56.5 oz	56.5 oz	56.5 oz	56.5 oz
Air	7.0 oz	7.0 oz	7.0 oz	7.0 oz	7.0 oz	6.1 oz	6.1 oz	6.1 oz	6.1 oz
Superplasticizer	209.0 oz	209.0 oz	209.0 oz	209.0 oz	209.0 oz	209.0 oz	209.0 oz	209.0 oz	209.0 oz

* Fabricator recorded 3.9% free moisture on the sand and 1.0% free moisture on the stone.

Metric Equivalents:

1 lb = 0.454 kg

1 gal = 3.785 1

1 oz. = 29.574 cc

B.2.6 Release of Prestress

Approximately 39 hours after casting, one control cylinder from each batch used in the fabrication of the bulb-tee specimens was removed from the casting bed and tested to determine compressive strength. A minimum of 6,000 psi (41.4 MPa) was required before release of the prestress. The average compressive strength of the nine cylinders was 9,250 psi (63.8 MPa), thus the strength was satisfactory for release of the prestress.

Upon removal of the forms, a full-depth vertical crack was discovered near mid-span of each of the girder specimens. The location of the crack was marked. The prestress was released by simultaneously cutting the strands at each end of the casting bed with an acetylene torch. Then, the strands between each girder specimen were cut. After release of prestress, the mid-span crack in each girder was virtually invisible. Instrumentation readings were taken and recorded just before and after release of all strands.

B.2.7 Transporting and Handling of the Girders

Following release of the strands, the bulb-tee specimens were lifted from the casting bed and transported to another location in the casting yard. The girders were lifted using the lifting inserts shown on the fabrication drawing in Figure B.1. The control specimens remained with the girder specimens until the time of shipment. Twelve days after release, the bulb-tee specimens were loaded onto tractor-trailer trucks and transported to Construction Technology Laboratories, Inc. (CTL) in Skokie, Illinois. The control specimens were shipped on the same vehicle, packed in a wooden crate containing plywood support racks for the cylinders. The bulb-tee specimens and the control specimens were stored indoors at CTL until time of testing.

B.2.8 Casting the Deck Slab on Girders BT1 and BT3

Once the bulb-tee specimens had been delivered to CTL, a deck slab was cast on girders BT1 and BT3. The slabs were cast on girders BT1 and BT3 approximately 28 days and 65 days after fabrication, respectively. The deck slabs were cast using partially shored construction. Approximately 50% of the deck slab weight was supported by the girder, while the remaining 50% was supported by the laboratory floor.

Each deck slab required approximately 24 yd^4 (18.4 m^3) of concrete. The concrete mix design for the deck slab is shown in Table B.3. Three 8 yd^3 (6.1 m^3) batches were used for each girder deck slab. The concrete for the deck slabs was supplied by a Chicago ready-mix supplier, and was designed to have a minimum strength of 6,000 psi (41.4 MPa) by the time of testing. The concrete was taken from the ready-mix trucks and placed in hoppers that were lifted by the overhead crane in the laboratory. Vibration was used throughout the pouring operation to ensure good consolidation of the concrete. During casting, three standard 6 x 12-in. (152 x 305 mm) cylinders were made from each concrete batch, representing the two ends and middle regions of the slab. Following casting, the slab was covered with plastic for three days. The forms and shoring were removed three days after casting.

TABLE B.3

CONCRETE MIX DESIGN FOR DECK SLAB

Materials	Quantity/Cubic Yard
Cement	705 lb
Fly Ash	75 lb
Fine Aggregate	1,160 lb
Coarse Aggregate (#67)	1,750 lb
Water	32.4 gal.
Air Entrainment Admixture	5.0 oz.
Superplasticizer	21.2 oz

Metric Equivalents:

1 lb = 0.454 kg

1 gal. = 3.785 l

1 oz. = 29.574 cc

APPENDIX C

MATERIALS AND FABRICATION DETAILS OF PILE SPECIMEN USED FOR FIELD EVALUATION

C.1 MATERIALS

C.1.1 Aggregate

The sand for the concrete mix was obtained from a pit near Atmore, Alabama. The course aggregate was obtained from the Reed Quarry located near Gilbertsville, Kentucky. The course aggregate was a crushed limestone. Both aggregates were tested and found to be in conformance with the Louisiana Department of Transportation and Development (LaDOTD) Standard Specifications for aggregates (35). Sieve analysis of the aggregate is reported in Table C.1.

C.1.2 Cement

Type I-II FG portland cement was used for the pile test specimen. The cement was manufactured by the Holnam Corporation and was certified to conform to ASTM Standard C150-89 , Specifications for Portland Cement (36).

C.1.3 Water

The water used in the concrete was obtained from the Sherman Prestress, Inc. underground well. The water conformed to LaDOTD Standard Specifications (35) for water used in portland cement concrete.

TABLE C.1

GRADATION ANALYSIS OF FINE AND COARSE AGGREGATE

Fine Aggregate - Atmore Sand

Sieve #	Weight Retained	Percent Retained	Percent Passing
3/8 in.	0	0	100
4	22	4	96
8	59	10	90
16	96	16	84
30	284	49	51
50	433	74	26
100	566	97	3
Pan	584		

Coarse Aggregate - #57 Stone from Reed Quarry

Sieve #	Weight Retained	Percent Retained	Percent Passing
1-1/2 in.	0.0	0	100.0
1 in.	0.74	3	97.0
3/4 in.	0.0	0	0.0
1/2 in.	11.02	42	58.0
3/8 in.	0.0	0	0.0
4	24.20	93	7.0
8	25.75	99	1.0
Pan	26.12		

C.1.4 Admixtures

Four admixtures were used in the concrete mix. None of the admixtures used in the mix were on the LaDOTD Qualified Products List. The admixtures were as follows:

1. FRITZ FR-2 water reducer conforming to ASTM C494-90 (37), Type A, B & D.
2. FRITZ SUPER 6 high-range water reducer conforming to ASTM C-494-90, Type F (37).
3. FRITZ AIR PLUS air entraining admixture conforming to ASTM C260-86 (38).
4. FRITZ PAK silica fume - 100 percent solids.

C.1.5 Prestressing Strand

The low-relaxation prestressing strand used in the pile specimens was domestically fabricated using domestically manufactured steel and was obtained from Wiremill, Inc. of Jacksonville, Florida. The nominal diameter of the strand was 1/2 inch. The strand conformed to ASTM A-416-88, Specifications for Steel Strand, Uncoated Seven Wire Stress Relieved for Prestressed Concrete (26). All prestressing strand used in the pile came from a single coil. A test report obtained from the manufacturer indicated the following material properties:

Coil No.: Unknown

Breaking Load	43,505 lb (193.5 kN)
Load @ 1% Elongation	40,447 lb (179.9 kN)
Elongation	5.1%
Modulus of Elasticity	28,600,000 psi (197.2 GPa)

C.1.6 Web Reinforcement

The spiral reinforcement used in the pile specimens was domestically manufactured by Florida Wire and Cable Company of Jacksonville, Florida. The W4.5 wire had a diameter of 0.211 in. (5.35 mm). The average tensile strength of samples tested by the manufacturer was 98,588 psi (680 MPa).

C.2 FABRICATION

One pile specimen, designated as pile P4, was fabricated by Sherman Prestressed Concrete at their plant in Mobile, Alabama, in accordance with the shop drawings shown in Figure C.1. The pile specimen was fabricated in a 240 ft 8 in. (73.3 m) casting bed. All work was performed by the fabricator. All phases of the pile specimen fabrication and preparation of the control specimens were observed and supervised by the research personnel to insure all quantities and measurements were known accurately.

C.2.1 Placement of the Longitudinal Strand

The seven wire prestressing strands were drawn from a single reel located in an outdoor storage area adjacent to the casting bed. Each strand was taken from a single reel and threaded through a template at each end of the casting bed. The strands were tensioned using a single strand jack manufactured by G.T. Bynum. The jack had been calibrated within the past three months to have an average gauge reading error of less than 0.86 percent.

C.2.2 Prestressing of the Strand

In accordance with the present AASHTO code, each of the strands was initially stressed to a value of 0.75 f_{pu} or 30,980 lb (138 kN). In order to accomplish this, the strands were

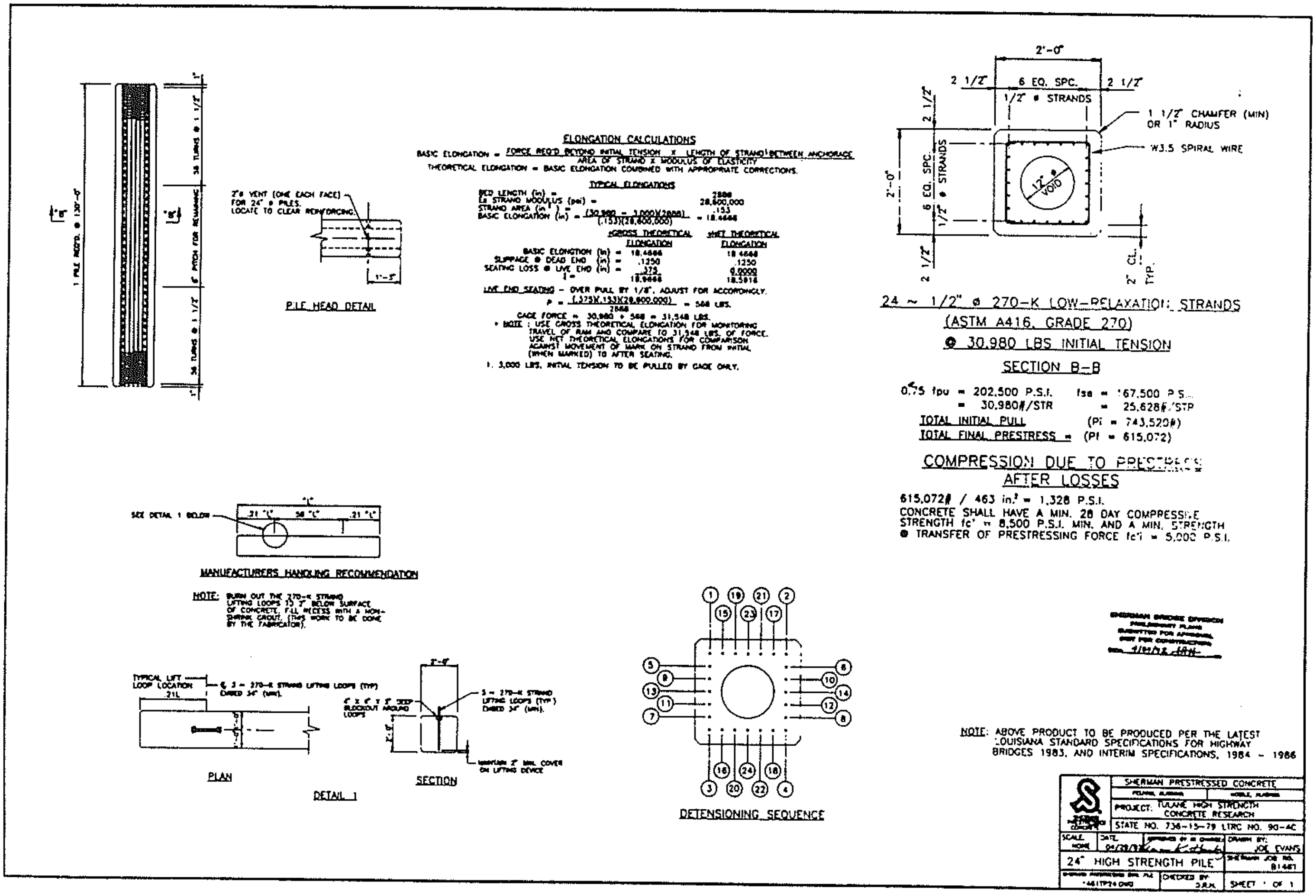


Figure C.1. Shop drawing for 130 ft (39.6 m) pile.

initially tensioned to a proof load of 3000 lb (13.4 kN) based on gauge readings. After being subjected to the proof load, the strands were marked and stressed to an average elongation of 18.5 in. (470 mm). The load corresponding to this elongation can be obtained from the following expression:

$$e = \frac{PL}{AE}$$

where:

e = elongation of strand, in.

P = load per strand, lb

L = nominal length between stressing ends, 120 ft 10 in. (36.8 m)

A = nominal steel area of strand, 0.153 in.^{2} (98.7 mm^2)

E = modulus of elasticity of the strand, 28,600,000 psi (197.2 GPa)

Using the above expression, the load in the strand beyond the proof load is calculated to be 27,970 lb (129 kN). If the proof load is added to this value, a force of 30,970 lb (138 kN) per strand results. This was confirmed using the gauge readings of the jack. The jack gauges were found have an average gauge reading error of less than one percent for the load range used in recent calibration tests.

C.2.3 Placement of Spiral Reinforcing and Form Erection

The spiral reinforcing was arranged into its final position after the full prestress force had been applied to the longitudinal strand. The 12-in. (305-mm) void in the center of the pile was formed using a cardboard tube. The cardboard tube was hung from short lengths of steel angles that were bolted to the top of the forms. These steel angles also served to hold the cardboard tube down during the concrete pour. The exterior sides of the pile were formed by a steel pan with an open top. The form could not be broken apart, hence the pile was removed by lifting it out of the forms.

C.2.4 Concrete Mixing, Placement and Curing

Casting of the pile specimen began at 10:45 a.m. on May 29, 1992, and was completed at 12:30 p.m. of the same day. The daytime temperature on the day of the pour ranged from 78°F (26°C) to 80°F (27°C). The concrete mix used for the pile was designed to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa). The concrete was mixed in the fabricator's batch plant located adjacent to the pile casting bed. The batch plant had a single four cubic yard (3.1 m^3) drum type central mixer.

The mixer was initially charged with the stone, sand, silica fume, cement, and 80 percent of the water. Next, the air entraining agent was added, followed 30 seconds later by the water reducer, followed 30 seconds later by the superplasticizer. Following this charging of the mixer, the batch was mixed for 200 seconds. Water was added as required to obtain a slump of 5 in. (127 mm) with mixing continuing for at least 90 seconds after the last water addition. Aggregate was weighed to the required amount with adjustments made for the free moisture content on the scales located above the central mixer. The free moisture in the sand was measured at 7.5 percent while the free moisture of the stone was 0.9 percent. The cement was weighed in another set of scales also located above the central mixer. Water was added and controlled by an automatic dispenser/meter. Admixtures were added manually. The concrete was mixed in four cubic yard batches. Specific proportions for the five batches used for pile fabrication are given in Table C.2.

TABLE C.2

CONCRETE MIX PROPORTIONS (per cubic yard)

	Mix Designations				
Components, lb	#2	#3	#4	#5	#6
Cement	752 lb	753 lb	755 lb	753 lb	757 lb
Silica Fume	96 lb	95 lb	97 lb	95 lb	95 lb
Water	136 lb	127 lb	125 lb	125 lb	142 lb
Sand	1,105 lb	1,105 lb	1,110 lb	1,105 lb	1,107 lb
Stone	1,890 lb	1,890 lb	1,890 lb	1,890 lb	1,893 lb
Water Reducer	10 oz.	10 oz.	10 oz.	10 oz.	10 oz.
Air Entrainment	3 oz.	3 oz.	3 oz.	3 oz.	3 oz.
High-Range Water Reducer	78 oz.	78 oz.	78 oz.	78 oz.	78 oz.

* Fabricator recorded a "free" moisture on the sand of 7.5% and 0.9% on the stone.

Metric Equivalents:

1 lb = 0.454 kg

1 oz. = 29.574 cc

After mixing, the concrete was placed into a screw type transporter, taken to the casting bed, and placed in the forms. The pile specimen required slightly less than 18 cubic yards of concrete. During casting, internal vibration of the concrete using hand held vibrators was performed carefully to insure proper concrete consolidation. Seven standard 6 x 12 in. (152 x 305 mm) cylinders were made from each batch.

After casting, the pile specimen was covered with a tarpaulin consisting of 1/4-in. (6.4-mm) foam insulation sandwiched between two layers of waterproof canvas. After initial set, the pile specimen was steam-cured for 18 hours at an average temperature of 140°F (60°C).

C.2.5 Release of Prestress

Approximately 20 hours after completion of casting, five control cylinders (one for each concrete batch) were removed from the casting bed, and tested to determine their compressive strength. A minimum of 6,000 psi (41.4 MPa) was required for release of the prestress. The average compressive strength of the five cylinders was 9,071 psi (62.4 MPa), thus satisfactory for release of the prestress.

The prestress was released by cutting the strands simultaneously at each end of the pile with an acetylene torch.

C.2.6 Transporting and Handling of the Pile

Immediately following release of the strands, the pile was lifted out of the form and transported to another location in the casting yard. The pile was lifted using the lifting inserts shown on the fabrication drawing in Figure C.1. The control cylinder specimens remained with the pile segments until the time of shipment. Thirteen days after release, the pile was loaded onto a truck and transported to a bridge under construction on Route La 415 over the Missouri Pacific Railroad in West Baton Rouge Parish. The control specimens were picked up by LTRC and transported to their laboratory in Baton Rouge, Louisiana, for testing.

C.2.7 Louisiana DOTD PDA Monitoring Report

The following report prepared by J. B. Esnard and E. Tavera of the Louisiana Department of Transportation summarizes findings from dynamic monitoring of the pile.

STATE OF LOUISIANA
DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT

PDA MONITORING

HIGH STRENGTH PRECAST PRESTRESSED CONCRETE PILE

S.P. 736-15-0079/LTRC NO. 90-4C
FEASIBILITY EVALUATION OF HIGH STRENGTH CONCRETE

S.P. 13-01-0024
BRIDGES OVER MISSOURI PACIFIC RAILROAD
ROUTE LA 415
WEST BATON ROUGE PARISH

PAVEMENT AND GEOTECHNICAL DESIGN SECTION

July 16, 1992

STATE OF LOUISIANA
DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
P. O. Box 94245
Baton Rouge, Louisiana 70804-9245

EDWIN W. EDWARDS
GOVERNOR

JUDE W. P. PATIN
SECRETARY

July 16, 1992

Department of Civil Engineering
206 Civil Engineering Building
Tulane University
New Orleans, Louisiana 708118-5698

Re: **PDA Monitoring**
S.P. 736-15-0079/LTRC No. 90-4C
Feasibility Evaluation of High Strength Concrete
S.P. 13-01-0024
Bridges over Missouri Pacific Railroad
Route LA 415
West Baton Rouge Parish

Attention: Dr. Robert N. Bruce, Jr.
Principal Investigator

This report summarizes our findings from dynamic monitoring of a research pile driven on the above referenced project.

BACKGROUND

The LA DOTD Bridge Design section requested that we monitor a 24" high strength precast prestressed concrete pile during pile driving with the Pile Dynamic Analyzer™ (PDA). This pile is part of a research study, *Feasibility Evaluation of High Strength Concrete*, being performed by Tulane University. The pile was driven at the LA 415 bridge construction project (S.P. 13-01-0024). Our general requirements to perform the PDA testing were transmitted to the Project Engineer in our letter dated April 2, 1992. We received the completed *Pile Driving And Equipment Data Form* on June 6, 1992. Related correspondence is enclosed in the appendix.

On June 16, 1992, a field trip was made to prepare the research pile for dynamic monitoring with the PDA. Several dynamic wave speed measurements were taken for PDA input during dynamic monitoring. In order to expedite PDA monitoring, several concrete anchors were set into the pile surface for future attachment of strain gauges and accelerometer transducers.

The high strength concrete research pile was monitored during driving with the Pile Driving Analyzer™ on June 22, 1992. The pile was driven on the northbound bridge Bent 10 (pile no. 3) at station 217+74.43. The PDA is a computerized data acquisition system which uses two strain gauges and two accelerometer transducers to record the strain and acceleration at the gauge location (4' below pile top). The PDA converts the strain and acceleration signals for each hammer blow to force and velocity data versus time. The recorded measurements are used to estimate the pile capacity, monitor hammer performance and energy transfer, measure pile driving stresses, and evaluate pile damage. Pile capacity computations are based on the Case Method of analysis.

A review of our records indicates that soil core borings, electronic cone penetration test (ECPT) probings, and test pile reports are available in the vicinity of the test site for this research project. A soil boring was taken at station 218+65, 20' Lt. C.L. The ECPT probing was taken at Bent 10 of the southbound roadway. Test Pile 3 and four piles in Bent 14 (Sta. 218+85, 50'Rt. C.L.) were monitored with the PDA by the FHWA in March of 1990 as part of the FHWA Demonstration Project 66. Test pile 3 was driven to tip elevation -63 feet, which is above the dense sand. The four piles tested in Bent 14 had an ABB splice and were driven into the dense sand to a pile tip elevation of -69 feet. PDA test results of the FHWA Demonstration Project 66 are included in the appendix.

TEST DETAILS

High Strength Precast Prestressed Concrete Pile

The tested pile is a 130 ft long 24" prestressed precast high strength concrete pile. The high strength concrete has been prepared for a minimum 28 day concrete strength of 8500 psi and a release strength of 5000 psi. The pile was driven 24 days after casting it on May 29, 1992. The LTRC concrete strength test results for different curing times are as follows:

AGE	AVERAGE COMPRESSIVE STRENGTH - f'c
7-days	9,596 psi
14-days	10,264 psi
28-days	10,453 psi

The following effective prestresses were computed by LA DOTD Bridge Design section.

AGE	EFFECTIVE PRESTRESS
Initial	1606 psi
18-hours	1512 psi
10-days	1463 psi
90-days	1410 psi

Based on the above concrete information, we estimate that at the time the pile was driven, the concrete strength was 10,400 psi and the effective prestress was 1454 psi. The unit weight was assumed to be 150 pcf. The average measured material stress wave speed is 14,700 ft/sec which corresponds to a dynamic elastic modulus of 6990 ksi.

The maximum allowable driving stresses for precast prestressed concrete piles that are recommended by the FHWA are based on the following formulas:

$$\textit{Maximum Compressive Driving Stress} = 0.85f'c - \textit{Effective Prestress}$$

$$\textit{Maximum Tension Driving Stress} = 3\sqrt{f'c} + \textit{Effective Prestress}$$

Based on these formulas, the maximum allowable compressive driving stress is 7.39 ksi and the maximum allowable tensile driving stress is 1.76 ksi.

Pile Driving Hammer
Pile driving was done with a Vulcan 020 single acting external combustion hammer, SECH, with a 3 foot stroke. This hammer has a manufacturer's maximum energy rating of 60.0 kip-ft at 3.0 ft length of stroke. The pile cushion used was a 5 inch thick compressed oak cushion. The oak cushion was originally 6 inches thick. The *Pile And Driving Equipment Form* is in the appendix.

Soils
The ground surface is at approximately elevation 26 feet, M.S.L. Prior to stabbing the pile, the pile location was pre-drilled to 20 feet below natural ground. The pile was driven to tip elevation -68.57 feet. The soil profile consists of soft to medium clays and silty clays down to elevation -64 feet underlain by a layer of medium to very dense sand. The soil boring log is shown in Figure 2 and the ECPT probing is shown in Figure 3.

CONCLUSIONS

Dynamic pile monitoring and subsequent data analysis support the following conclusions:

Hammer and Driving System Performance
The maximum transferred energy averaged 12.0 kip-ft in the first 22 feet of pile penetration, which translates into an efficiency of 20% of the maximum rated hammer energy. The maximum transferred energy averaged 15.0 kip-ft for the remainder of the driving, which is 25% efficiency of the maximum rated hammer energy. The total measured energy transfer efficiency for the same hammer in the Federal Demonstration Project was 27% to 28%. A relative evaluation of this energy transfer is possible by statistical comparison with a database of results under similar end of drive conditions. See Figure 4 for statistical energy transfer comparison.

Pile Performance

The compressive driving stresses measured at the gauge location reached a maximum of 1.83 ksi and then leveled off to an average of 1.70 ksi for the remainder of the pile driving. The maximum pile driving tension stresses increased from 0.86 ksi at the beginning of driving to 1.31 ksi at a pile tip penetration of 55 feet. The maximum tensile stress gradually reduced to 1.0 ksi at a pile tip penetration of 86 feet then sharply dropped off as the driving resistance increased in the dense sand. These driving stresses are well below the maximum allowable compressive driving stress of 7.39 ksi and maximum allowable tensile driving stress of 1.76 ksi. A profile of the driving stresses is shown in Figure 1.

The maximum measured tension driving stress of 1.31 ksi would have been considered damaging if 5,000 psi or 6,000 psi concrete strengths would have been used. The maximum allowable tensile driving stress for 5,000 psi and 6,000 psi concrete are 1.00 ksi to 1.17 ksi, respectively.

No structural damage was detected by the pile top dynamic data. Figure 5 is a PDA screen printout that shows a graph of the Force (solid line) and Velocity (dashed line) measurements versus time for one blow. This record shows the maximum tensile stress of 1.31 ksi recorded during pile driving. The Force and Velocity traces are proportional and do not indicate any change in impedance. The PDA computes a pile integrity factor, BTA, which is a ratio of the reduced pile cross-section to original pile cross-section. Pile damage guidelines based on BTA factor are shown in Table 1.

Table 1 - PILE DAMAGE GUIDELINES (Rauche & Goble, 1979)

BTA	SEVERITY OF DAMAGE
1.0	Undamaged
0.8 - 1.0	Slightly Damaged
0.6 - 0.8	Damaged
Below 0.6	Broken

The PDA recorded a pile integrity factor, BTA, of 1.0 for the entire pile driving operation. Based on the pile damage guidelines shown above, the pile is undamaged.

SUMMARY

A 24" high strength precast prestressed concrete pile was monitored with the Pile Dynamic Analyzer™. The PDA computed both compressive and tensile driving stresses which were within FHWA maximum allowable driving stresses. The dynamic data, Force and Velocity wave traces, showed no pile damage.

If we can be of further assistance, please advise.

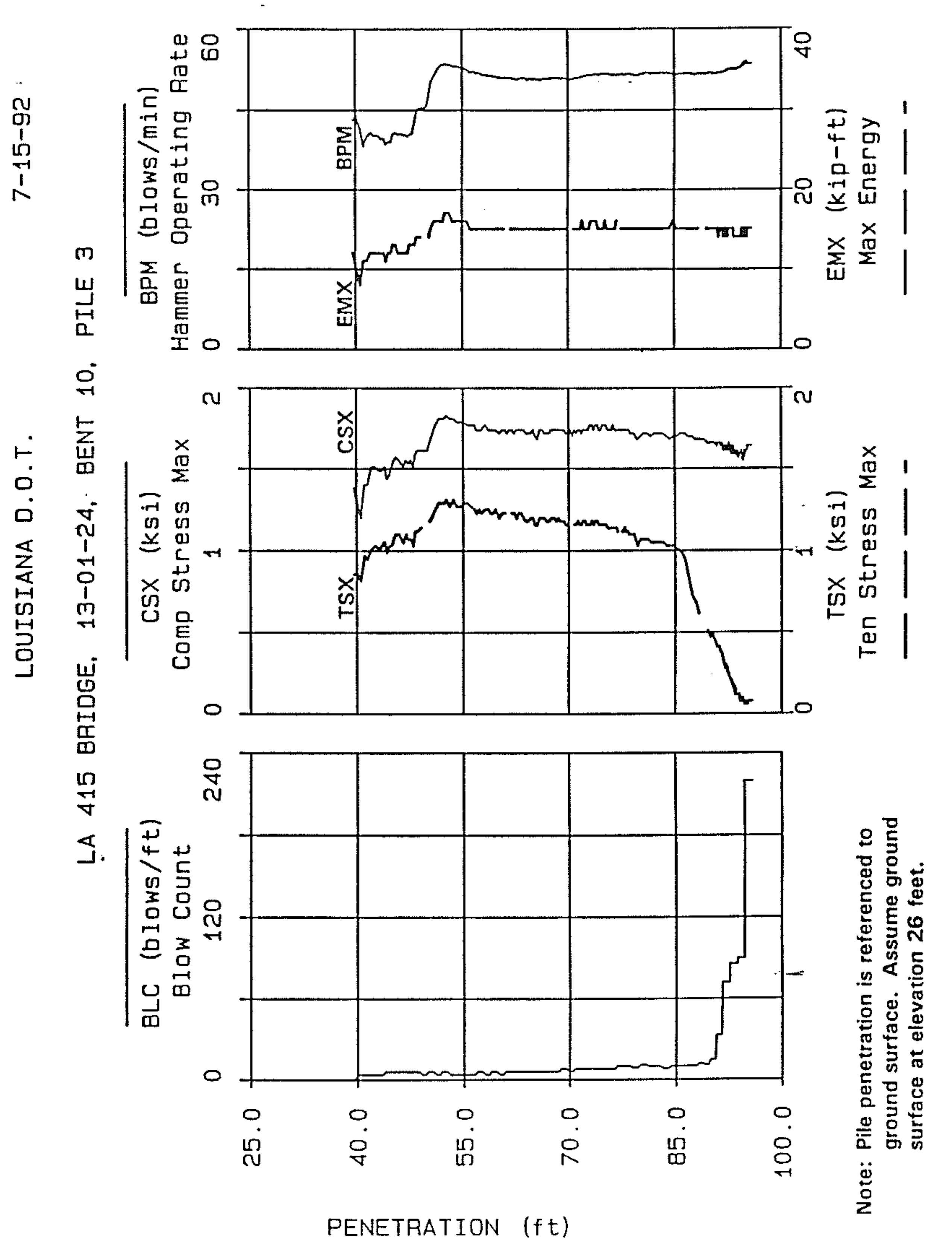
J. B. Esnard
Pavement and Geotechnical
Design Engineer

Ed Tavera

Ed Tavera, P.E.
Geotechnical Design Engineer

Attachments:

- Figure 1, PDA Test Data Plot
- Figure 2, Soil Boring Log - Sta. 218+65, 20' LT. C.L.
- Figure 3, ECPT Probing - Southbound Bent No. 10
- Figure 4, GRL Hammer Performance - SECH Concrete/Timber Piles
- Figure 5, PDA Screen Printout
- Appendix:
  - Bridge Design Section Letter - March 24, 1992
  - Pavement & Geotechnical Design Sec. Letter - April 2, 1992
  - *Pile And Driving Equipment Form* - June 6, 1992
  - High Strength Precast Prestressed Concrete Pile Plan Sheet
  - LTRC Concrete Strength Letter - July 15, 1992
  - FHWA Demonstration Project 66 - PDA Test Results


LOUISIANA D.O.T.
7-15-92
LA 415 BRIDGE, 13-01-24, BENT 10, PILE 3
BLC (blows/ft)
Blow Count
CSX (ksi)
Comp Stress Max
BPM (blows/min)
Hammer Operating Rate
TSX (ksi)
Ten Stress Max
EMX (kip-ft)
Max Energy
BPM
EMX
CSX
TSX
0
120
240
1
2
30
60
20
40
25.0
40.0
55.0
70.0
85.0
100.0
PENETRATION (ft)
Note: Pile penetration is referenced to ground surface. Assume ground surface at elevation 26 feet.

APPENDIX D

MATERIALS AND FABRICATION DETAILS OF GIRDER SPECIMENS USED FOR FATIGUE TESTS

D.1 MATERIALS

D.1.1 Aggregate

The sand for the concrete mix used in the bulb-tee girders was obtained from a pit near Atmore, Alabama. The coarse aggregate was obtained from the Reed Quarry located near Gilbertsville, Kentucky. Both aggregates were tested and found to be in conformance with the Louisiana Department of Transportation and Development (LaDOTD) Standard Specifications for aggregates (35). Sieve analyses of the aggregate used is reported in Table D.1.

D.1.2 Cement

Type I portland cement was used for the bulb-tee test specimens. The cement was manufactured Holnam Inc. and was certified to conform to ASTM Standard C150-89, Specifications for Portland Cement (36).

D.1.3 Water

The water used in the concrete was obtained from the Sherman Prestress, Inc. underground well. The water conformed to LaDOTD Standard Specifications (35) for water used in portland cement concrete.

TABLE D.1

GRADATION ANALYSIS OF FINE AND COARSE AGGREGATE

Fine Aggregate - Atmore Sand

Sieve #	Weight Retained	Percent Retained	Percent Passing
3/8 in.	0	0	100
4	18	4	96
8	59	12	88
16	103	20	80
30	262	52	48
50	402	80	20
100	490	97	3
Pan	503		

Coarse Aggregate - #57 Stone from Reed Quarry

Sieve #	Weight Retained	Percent Retained	Percent Passing
1-1/2 in.	0	0	100
1 in.	0.41	2	98
3/4 in.	-	-	-
1/2 in.	14.43	71	29
3/8 in.	-	-	-
4	20.15	99	1
8	20.21	99	1
Pan	20.35		

D.1.4 Admixtures

Four admixtures were used in the concrete mix. None of these admixtures were on the LaDOTD Qualified Products List. The admixtures were as follows:

1. FRITZ FR-2 water reducer conforming to ASTM C494-90 (37), Type A, B & D .
2. FRITZ SUPER 6 high-range water reducer conforming to ASTM C-494-90, Type F (37).
3. FRITZ AIR PLUS air entraining admixture conforming to ASTM C260-86 (38).
4. FRITZ PAK silica fume - 100 percent solids.

D.1.5 Prestressing Strand

The low-relaxation prestressing strand used in the bulb-tee specimens was domestically manufactured and was obtained from Wiremill, Inc. of Jacksonville, Florida. The nominal diameter of the strand was 1/2 in. (12.7 mm). The strand conformed to ASTM A-416-88, Specifications for Uncoated Stress Relieved Wire for Prestressed Concrete (26). The prestressing strand used in the girder specimens came from four different coils (983141, 984541, 988032 and 988101). Test reports obtained from the manufacturer indicated the following material properties:

Coil Nos.: 983141, 988032 and 988101

Breaking Load	42,840 lb (190.6 kN)
Load @ 1% Elongation	40,200 lb (178.8 kN)
Elongation	5.5%
Modulus of Elasticity	28,200,000 psi (194.4 GPa)

Coil No.: 984541

Breaking Load	43,108 lb (191.8 kN)
Load @ 1% Elongation	40,439 lb (179.9 kN)
Elongation	5.2%
Modulus of Elasticity	28,400,000 psi (195.8 GPa)

D.1.6 Web Reinforcement

The No. 4, Grade 60, web reinforcement used in the bulb-tee specimens was domestically manufactured by Magnolia Steel Company, Inc. of Meridian, Mississippi. The average yield strength and tensile strength, as determined by the manufacturer, were 71,500 psi (493 MPa) and 110,000 psi (759 MPa), respectively.

D.2 FABRICATION

The two bulb-tee specimens, designated as girders BT4 and BT5, were fabricated by Sherman Prestressed Concrete at their plant in Mobile, Alabama, in accordance with the shop drawings shown in Figure B.1. The two bulb-tee specimens were fabricated in a single 328 ft 1-1/2 in. (100.0 m) casting bed. All work was performed by the fabricator except for installation of the instrumentation. All phases of the bulb-tee fabrication and preparation of the control specimens were observed and supervised by the research personnel to ensure all quantities and measurements were known accurately.

D.2.1 Placement of the Longitudinal Strand

The seven wire prestressing strands were drawn from four reels located in an outdoor storage area adjacent to the casting bed. Each strand was taken from the reel and threaded through a template at each end of the casting bed. Intermittent pick-up and hold-down points were positioned to establish the desired strand drape pattern for the six strands coincident with the girder web. The strands were tensioned using a single strand jack manufactured by G. T. Bynum. Calibration tests were not available for the jack, however, the accuracy was verified from load cells placed on selected strands.

D.2.2 Prestressing of the Strand

In accordance with the present AASHTO code, each strand was initially stressed to a value of 0.75 f_{pu} or 30,980 lb (138 kN). In order to accomplish this, the strands were initially tensioned to a proof load of 3,000 lb (13.4 kN). Once this was done, the straight strands were marked and stressed to an average elongation of 25.67 in.(652 mm). Of this elongation, 0.5 in. (12.7 mm) is assumed to be lost in slippage at the dead end and in seating losses at the live end. In the case of the draped strands, the elongation beyond proof load was calculated to be 23.16 in. (588 mm) with 0.5 in. (12.7 mm) lost in slippage and seating losses, and 2.54 in. (65 mm) gained due to pulling the strand up at the drape-points. The load corresponding to this elongation can be obtained from the following expression:

$$e = \frac{PL}{AE}$$

where:

e = elongation of strand, in.

P = load per strand, lb

L = nominal length between stressing ends, in.

A = nominal steel area of strand, 0.153 in.^{2} (98.71 mm^2)

E = modulus of elasticity of the strand, 28,600,000 psi (197.2 GPa)

Using the above expression, the load in the straight strand beyond the proof load is calculated to be 27,970 lb (125 kN). When the proof load is added to this value, a total force of 30,970 lb (138 kN) per strand results. Load cells were placed on selected strands to verify jacking loads.

D.2.3 Placement of Web Reinforcement and Form Erection

The web reinforcing was placed in accordance with the shop drawings after the full prestress force had been applied to the straight and draped strands. The bottom of the steel exterior side forms for the bulb-tee specimens were anchored to the casting bed with adjacent sides tied together across the top.

D.2.4 Installation of Instrumentation

After the final tensioning of the strands, three Carlson strain meters were installed at mid-length of each girder, adjacent to the strands, at the level of the strand pattern centroid. In addition to the Carlson meters, two 1/4-in (6.4-mm) diameter reinforcing bars instrumented with weldable wire strain gauges were installed at mid-length near the top and bottom surfaces of each girder. The location of the strain gauges and strain meters are shown in Figure 11. The strain meters were Carlson model A-10. The welded wire strain gauges were model AWC-8b manufactured by Tokyo Sokki Kenkyujo Company, Ltd. After installation and casting, each strain gauge and strain meter was monitored to ensure it was functioning correctly.

In order to facilitate measurements of girder camber, three stainless steel bolts were pressed into the top flange concrete of each girder during casting. One bolt was placed at mid-length while the other two were centered above the sole plate at each end. Girder camber was determined using a level to sight elevations of the mid-length reference bolt relative to the two end reference bolts.

D.2.5 Concrete Mixing, Placement and Curing

Casting of the bulb-tee specimens began at 10:30 a.m. on January 22, 1992, and was completed at 1:20 p.m. of the same day. The daytime temperature during the pour ranged from 45°F (7°C) to 55°F (13°C). The concrete mix used for the girders was designed to yield concrete with a minimum 28-day compressive strength of 10,000 psi (69.0 MPa). The concrete was mixed in the fabricator's batch plant located adjacent to the girder casting bed. The batch plant had a single 8 yd^3 (6.1 m^3) drum type mixer.

The mixer was initially charged with the coarse aggregate, sand, 50% of the mix water, and the air entraining agent. After waiting approximately 30 seconds, the high-range water reducer and silica fume were added. The drum was then turned for about 45 seconds. Next, the cement and 50% of the superplasticizer were added. After mixing for approximately 1 minute, the remainder of the superplasticizer and water were added and mixed for approximately two more minutes. Aggregate was weighed to the required amount with adjustments made for the free moisture content. The silica fume was added using weighed bags. Water and admixtures were added and controlled by an automatic dispenser/meter. The scales and automatic dispenser had been calibrated within the year prior to the date of fabrication.

On the day of casting, a total of six principal, 4 yd^3 (3.06 m^3) batches were made. Two small additional "filler" batches were made to supplement the six principal batches. Each bulb-tee specimen required slightly less than 12 yd^3 (9.17 m^3) of concrete, hence each member was cast from three separate batches of concrete. Specific proportions of the six principal batches used for the girder fabrication are given in Table D.2. After mixing, each batch was placed into a screw type transporter, taken to the casting bed, and placed into the forms.

During casting, vibration of the concrete using both hand held vibrators and form vibrators was performed carefully to ensure proper consolidation and to avoid damage to any

TABLE D.2

CONCRETE MIX PROPORTIONS (per cubic yard)

	Mix Designation					
	Specimen BT1			Specimen BT2		
Item	Batch 4-1	Batch 4-2	Batch 4-3	Batch 5-1	Batch 5-2	Batch 5-3
Cement	753 lb	752 lb	751 lb	751 lb	750 lb	750 lb
Silica Fume	97.0 lb	96.5 lb	95.0 lb	95.0 lb	95.5 lb	101.5 lb
Water*	22.2 gal	22.2 gal	21.7 gal	21.5 gal	21.5 gal	20.7 gal
Sand	1,075 lb.	1,085 lb	1,090 lb	1,080 lb	1,075 lb	1,080 lb
Stone	1,885 lb.	1,885 lb	1,885 lb	1,890 lb	1,885 lb	1,885 lb
Water Reducer	16.0 oz	16.0 oz	16.0 oz	16.0 oz	16.0 oz	16.0 oz
Air	1.0 oz	1.5 oz	1.5 oz	2.5 oz	2.0 oz	2.0 oz
Superplasticizer	85.0 oz	85.0 oz	90.0 oz	90.0 oz	90.0 oz	90.0 oz

* Fabricator recorded 4.0% free moisture on the sand and 0.5% free moisture on the stone.

Metric Equivalents:

1 lb = 0.454 kg

1 gal = 3.785 1

1 oz. = 29.574 cc

of the internal instrumentation. Twelve standard 6 x 12 in. (152 x 305 mm) cylinders and three 6x6x20 in. (152x152 x 508 mm) beams were made from the six concrete batches. After casting, the bulb-tee specimens were covered with a tarpaulin consisting of 1/4-in. (6.4-mm) insulation sandwiched between two layers of waterproof canvas. The control specimens were also placed under the tarpaulin to undergo the same curing process as the girders.

D.2.6 Release of Prestress

Approximately eighteen hours after casting, one control cylinder from each batch used in the fabrication of the bulb-tee specimens was removed from the casting bed and tested to determine compressive strength. A minimum of 6,000 psi (41.4 MPa) was required before release of the prestress. The average compressive strength of the six cylinders was 7,320 psi (50.5 MPa), thus the strength was satisfactory for release of the prestress.

Upon removal of the forms, a full-depth vertical crack was discovered near mid-span of each of the girder specimens. The location of the crack was marked. The prestress was released by simultaneously cutting the strands with an acetylene torch at each end of the casting bed, and then in between each girder specimen. After release of prestress, the mid-span crack in each girder was virtually invisible. Instrumentation readings were taken and recorded just before and after release.

D.2.7 Transporting and Handling of the Girders

Following release of the strands, the bulb-tee specimens were lifted from the casting bed and transported to another location in the casting yard. The girders were lifted using the lifting inserts shown on the fabrication drawing in Figure B.1. The control specimens remained with the girder specimens until the time of shipment. Twelve days after release, the bulb-tee specimens were loaded onto tractor-trailer trucks and transported to Construction Technology Laboratories, Inc. (CTL) in Skokie, Illinois. The control specimens were shipped on the same vehicle, packed in a wooden crate containing plywood support racks for the cylinders. The bulb-tee specimens and the control specimens were stored indoors at CTL until time of testing.

D.2.8 Casting the Deck Slab on Girder Specimen BT5

Once the bulb-tee specimens had been delivered to the CTL, a deck slab was cast on girder BT5. The slab was cast on girder BT5 approximately 41 days after fabrication. The deck slab was cast using partially shored construction. Approximately 50% of the deck slab weight was supported by the girder while the remaining 50% was supported by the laboratory floor.

The deck slab required approximately 24 yd^3 (18.4 m^3) of concrete. The concrete mix design for the deck slab is shown in Table B.3. Three 8 yd^3 (6.1 m^4) batches were used for each girder. The concrete for the deck slab was supplied by a Chicago ready-mix supplier and was designed to have a minimum strength of 6,000 psi (41.4 MPa) by the time of testing. The concrete was taken from the ready-mix trucks and placed in hoppers that were lifted by the overhead crane in the laboratory. Vibration was used throughout the pouring operation to ensure good consolidation of the slab concrete. During casting, three standard 6 x 12 in. (152 x 305 mm) cylinders were made from each concrete batch, representing the two ends and middle regions of the slab. Following casting, the slab was covered with plastic for three days. The forms and shoring were removed three days after casting.

This public document is published at a total cost of $4420.00. Three hundred and fifteen copies of this public document were published in this first printing at a cost of $3410.00. The total cost of all printings of this document including reprints is $4420.00. This document was published by Louisiana State University, Graphic Services, 3555 River Road, Baton Rouge, Louisiana 70802, to report and publish research findings of the Louisiana Transportation Research Center as required in R.S.48:105. This material was printed in accordance with standards for printing by State Agencies established pursuant to R.S.43:31. Printing of this material was purchased in accordance with the provisions of Title 43 of the Louisiana Revised Statutes.