An experimental investigation of high strength concrete bridge girders was conducted. The investigation included fabrication and testing of 70 ft (21.3 m) long, 54 in. (1372 mm) deep, pretensioned, prestressed high strength concrete bulb-tee girders. The design concrete compressive strength for the girders was 10,000 psi (69 MPa). Two girders were used to evaluate long-term static load and fatigue performance. Results from the girder used to determine long-term static load behavior indicated prestress losses that were significantly less than those predicted by the AASHTO Standard. The girder used to determine fatigue performance withstood more than 5 million cycles of fatigue loading and satisfied all serviceability requirements. Upon completion of the long-term static load and fatigue test, measured flexural properties for both girders were adequate with respect to both design and specification requirements. Based on results from this investigation, high strength concrete bridge girders can be expected to perform adequately over the long-term when designed and fabricated in accordance with current AASHTO provisions.
Economic benefits are made possible by utilizing high strength concrete in highway bridge structures. Various studies have indicated the advantages of using high strength concrete in precast, prestressed bridge girders. Utilization of high strength concrete in precast, prestressed concrete members has been the subject of much recent research and steps have already been taken to implement its use in actual highway bridge structures on an experimental basis.

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 dead load of the superstructure, and increased durability. However, designers have been reluctant to specify higher strength concrete due to uncertainties regarding applicability of current design provisions and fabrication constraints.

In an effort to answer some of the questions regarding the feasibility of using high strength concrete in highway bridge structures, the Louisiana Transportation Research Center (LTRC), in cooperation with the Federal Highway Administration (FHWA) and the Louisiana Department of Transportation and Development (LaDOTD), sponsored a research program. The research program was conducted jointly by Construction Technology Laboratories, Inc. (CTL) and Tulane University.

This research program included the manufacture and testing of full-size, prestressed, high strength concrete bridge girders. As part of this research, long-term performance of girders subjected to both static and fatigue loading was evaluated. This paper describes the details and results of two girders tested to evaluate long-term performance.

Prior to testing, a deck slab was added to the girders. One of the girders (Girder 3) was subjected to a static load, approximating the in-service design dead load, for a duration of 18 months to evaluate time-dependent properties of the girder. The other girder (Girder 4) was subjected to 5 million cycles of fatigue loading to evaluate the fatigue performance of the girder. Upon completion of the long-term performance tests, both girders were tested in flexure to failure.

**GIRDER DESIGN**

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. Therefore, the bulb-tee cross section was selected for this research program. A bridge design utilizing 70 ft (21.3 m) long, 54 in. (1372 mm) 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. A 28-day concrete compressive strength of 10,000 psi (69 MPa) and a release com-
The girders were fabricated at two different precasting plants in Alabama. 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. These strands were pretensioned to a stress level corresponding to approximately 75 percent of guaranteed ultimate strength. Shear reinforcement and various inserts for handling purposes were installed after pretensioning the strands. Shear reinforcement in the girder specimens consisted of No. 4 bars at spacings ranging from 4 in. (102 mm) near the ends to 12 in. (305 mm) at midspan.

Three concrete batches were used for each girder. During casting, several 6 x 12 in. (152 x 305 mm) cylinders were made from each of the concrete batches. Measured concrete slump during casting ranged from 4 to 7 in. (100 to 180 mm). After the concrete had achieved initial set, Girder 3 was 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 forms were removed from the girder. Girder 4 was cured under a waterproof tarpaulin for 10 hours and the steel forms were removed approximately 12 hours after casting.

After form removal, a large crack was observed near midspan through the full depth of both girders. The prestressing strands were released approximately 39 hours after initial set for Girder 3 and approximately 18 hours after casting for Girder 4. The existing crack near midspan of each girder was virtually invisible after transferring the prestress force.

After fabrication, the girders were loaded on semi-trailers and transported to CTL for testing. Upon arrival, the girders were off-loaded and placed on supports inside the CTL structural laboratory. A deck slab was cast on Girders 3 and 4 approximately 28 and 41 days after fabrication, respectively. The existing crack near midspan of each girder was virtually invisible after transferring the prestress force.
the girder while the remaining 50 percent was supported by the laboratory floor. This distribution of the deck slab dead load was taken into consideration when evaluating the test results. Girders and concrete test cylinders (for both the girder and deck slab concrete) were stored in the laboratory and maintained at 73°F (23°C) and 50 percent relative humidity throughout the testing.

**GIRDER INSTRUMENTATION**

During the fabrication process, the girders were instrumented to measure concrete strains and camber. Internal strain indicators were installed at midspan in both the upper and lower flanges 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 Fig. 2.

**MATERIAL PROPERTIES**

The concrete mixes used for the girder specimens were designed to have a 28-day compressive strength of 10,000 psi (69 MPa). The mix designs used for the girders are listed in Table 1. It should be noted that these mix designs were not intended to be state-of-the-art high strength concrete mix designs. With the exception of the silica fume and admixtures, the high strength concrete mix designs made use of materials that were readily available at the fabricator's plant.

Average concrete material properties for the girder specimens are shown in Table 2. Each value in Table 2 represents the average of three test results (one for each concrete batch). In addition to these tests, creep and shrinkage were measured on cylinders from Girder 3. Results from these tests were used to aid interpretation of the long-term static load test results.

Prestressing strand used in the girder specimens was specified to be 1/4 in. (13 mm) diameter, 7-wire, Grade 270, low-relaxation strand, conforming to ASTM A 416. Strand samples were tested in accordance with ASTM A 370, Supplement VII. All tested strands met the requirements of ASTM A 416 for Grade 270, low-relaxation strand. Shear and confinement reinforcing steel used in the girders conformed to ASTM A 615, Grade 40. Reinforcing steel used in the deck slabs conformed to ASTM A 615, Grade 60.

<table>
<thead>
<tr>
<th>Girder</th>
<th>Concrete age</th>
<th>Compressive strength (ASTM C39), psi</th>
<th>Modulus of elasticity (ASTM C469), ksi</th>
<th>Splitting tensile strength (ASTM C496), psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Release</td>
<td>6920</td>
<td>5600</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>28 days</td>
<td>9930</td>
<td>6050</td>
<td>770</td>
</tr>
<tr>
<td></td>
<td>660 days</td>
<td>9660</td>
<td>—</td>
<td>660</td>
</tr>
<tr>
<td>4</td>
<td>Release</td>
<td>6960</td>
<td>4900</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>28 days</td>
<td>8830</td>
<td>5400</td>
<td>520</td>
</tr>
<tr>
<td></td>
<td>86 days</td>
<td>8770</td>
<td>5300</td>
<td>600</td>
</tr>
</tbody>
</table>

Note: 1 ksi = 1000 psi = 6.895 MPa.

**LONG-TERM STATIC LOAD TEST**

**Test Setup**

The setup for the long-term static load test is shown in Fig. 3. Girder 3 was simply supported at the centerline of the sole plates, creating a total span length of 69 ft (21.03 m). The high strength concrete bridge design prepared by LTRC resulted in a girder spacing of 13.3 ft (4.1 m). Because Girder 3 had only 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 arranged to produce a series of four concentrated loads, each of approximately 6800 lbs (30.3 kN). The concentrated loads were positioned to produce bending moments similar to those resulting from the uniform design dead load.

The purpose of the long-term static

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*Fig. 3. Long-term test setup.*
Table 3. Measured long-term properties for Girder 3.

<table>
<thead>
<tr>
<th>Age after loading</th>
<th>Measured strain, in./in.</th>
<th>Prestress loss, psi</th>
<th>Camber, in.*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial*</td>
<td>600 x 10^-6</td>
<td>18,000</td>
<td>-0.44</td>
</tr>
<tr>
<td>6 months</td>
<td>658 x 10^-6</td>
<td>19,700</td>
<td>-0.27</td>
</tr>
<tr>
<td>12 months</td>
<td>727 x 10^-6</td>
<td>21,800</td>
<td>-0.30</td>
</tr>
<tr>
<td>18 months</td>
<td>766 x 10^-6</td>
<td>23,000</td>
<td>-0.29</td>
</tr>
</tbody>
</table>

* Age of girder at initial loading was 69 days.
† Negative value indicates an upward camber.

Note: 1 ksi = 1000 psi = 6.895 MPa; 1 in. = 25.4 mm.

load test was to monitor the behavior of Girder 3 under full design dead load for a period of 18 months. During the 18-month test period, internal concrete strains and girder camber were measured at ages of 6, 12, and 18 months after applying the load.

Prestress Losses

Concrete strains, measured with internal Carlson strain meters, were used to calculate prestress loss by multiplying the average measured strains by the average measured strand modulus of elasticity of 30,000 ksi (206.9 GPa). As indicated in Table 3, after 12 months of being subjected to full design dead load, measured average total prestress loss in Girder 3 was 21,800 psi (150.3 MPa).

Recommendations by the PCI Committee on Prestress Losses suggest that approximately 74 percent of the ultimate creep and 86 percent of the ultimate shrinkage will occur within the first year. Based on these assumptions, along with the AASHTO provisions for calculating prestress losses and measured concrete and steel material properties, prestress losses of approximately 44,000 psi (303.1 MPa) would be expected at 12 months.

Based on the AASHTO provisions for calculating prestress losses and the design material properties, total prestress losses of approximately 53,200 psi (366.5 MPa) would be expected for the girder. The actual measured prestress loss in Girder 3 at 18 months was approximately 50 percent less than the expected value. However, there were indications that the steam curing employed for Girder 3 had an effect on the extraordinarily low measured prestress loss. Specifically, early-age prestress loss in Girder 4, which was not steam cured, was significantly higher than the prestress loss measured in Girder 3 after 18 months.

Measured creep deformations in 6 x 12 in. (152 x 305 mm) cylinders representing the concrete in Girder 3 were consistent with the findings regarding the measured prestress losses. Measured cylinder creep deformations after 1 year for three different ages (7, 28 and 56 days) of loading were consistent and averaged $426 \times 10^{-6}$ 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. The creep coefficient after 1 year reflected by the cylinder data fell below the expected range. Even if only 74 percent 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.

Measured cylinder shrinkage deformations after 1 year of measurements averaged $262 \times 10^{-6}$ in./in. According to ACI Committee 209, the ultimate shrinkage strain will normally fall into the range of $415 \times 10^{-6}$ to $1070 \times 10^{-6}$ in./in. According to the PCI Committee on Prestress Losses, approximately 22 and 86 percent of the ultimate shrinkage can be expected to occur within 7 days and 1 year after completion of the initial curing, respectively.

Assuming that the shrinkage deformations measured in the cylinders represent only 64 percent of the ultimate shrinkage (86 – 22 percent), the expected ultimate shrinkage implied by the cylinder shrinkage deformations would be $409 \times 10^{-6}$ in./in. This implied ultimate shrinkage falls on the low end of the expected range developed by ACI Committee 209.

While the measured prestress losses in Girder 3 and the measured cylinder creep and shrinkage deformations both support the conclusion that the total prestress losses are significantly less than those predicted by the AASHTO provisions, further research into the creep and shrinkage characteristics of high-strength concrete is required to determine if and how the AASHTO provisions for creep and shrinkage prestress losses can be modified for high-strength concrete. Such research would not only have to address various mix designs and materials, but various curing regimes as well.

Camber

Girder camber was calculated using measured elevations at the girder ends and midspan. The measured elevation at midspan was subtracted from the average of the measured elevations at the two ends to determine midspan camber. Girder 3 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 3 had an upward camber of 0.44 in. (11.2 mm).

Camber measurements taken at various stages of the long-term static load test are listed in Table 3. 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 3 had an upward camber of 0.34 in. (8.6 mm).

FATIGUE LOAD TEST

Test Setup

The test setup for the flexural fatigue test is shown in Fig. 4. A photograph of the fatigue test setup is shown in Fig. 5. 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 4 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 that 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 that 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 midspan and using dial gauges at each end of the girder.

In addition to the internal strain gauges installed during girder fabrication, several concrete surface gauges were added prior to the fatigue test. During fabrication, a midspan crack was observed in the girder prior to release of prestress. Prior to starting 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 existing crack. The two concrete surface strain gauges were placed adjacent to the crack at approximately mid-width of the lower flange.

An accelerometer was also installed on the bottom surface of the lower flange near midspan. The purpose of the accelerometer was to detect any longitudinal energy releases, such as concrete cracks or strand wire fractures.

Test Procedure

During the fatigue test, Girder 4 was subjected to cyclic flexural loading. The upper bound load was the load required to produce a midspan tensile stress at the extreme fiber of the lower flange equal to \(6\sqrt{f_{ct}}\). 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.

The fatigue test comprised 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 existing midspan crack. Decompression was judged to have occurred when the load vs. strain or 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 \(6\sqrt{f_{ct}}\) to the decompression load.
Each of the five remaining parts of the fatigue test included the application of 1 million loading cycles followed by an 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), midspan 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 two cycles per second. The effects of dynamic load amplification due to the bouncing mass of the girder specimen were accounted for 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 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 midspan 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 high speed DDAS to take several successive readings of all the instrumentation.

**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 cycles. A 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 million cycle segment in Fig. 6. Initial and intermittent camber and prestress loss measurements at
As indicated in Fig. 6, gradual "softening" of the girder was observed during fatigue testing. The measured midspan deflection at a load corresponding to the full design service load moment (Dead Load + Live Load + 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. As indicated in Table 4, the midspan camber decreased during the fatigue test but the prestress loss did not change significantly.

The decompression load determined from strain gauge data during the static tests did not change significantly for the first 4 million cycles. Therefore, the upper and lower bound fatigue test loads were kept the same for the first 4 million cycles. Based on the data from the static test conducted after achieving 4 million cycles, it was determined that the upper and lower bound fatigue test loads should be reduced slightly, because the decompression load had apparently decreased slightly. However, the load range was kept constant throughout the entire fatigue test.

A summary of the fatigue test parameters used for Girder 4 is given in Table 5. At the start of the fatigue test, the average stress in the prestressing strand, based on measured strains, was approximately 178,260 psi (1229 MPa). Based on this initial stress, the calculated steel stress at the upper bound test load was approximately 10.0 ksi (69 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 midspan, approximately 6 in. (152 mm) from the 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 stress ranges 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.

During the fatigue test of Girder 4, several transverse cracks developed in the deck slab outside 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 midspan 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

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### Table 4. Camber and prestress loss at various stages of the fatigue test.

<table>
<thead>
<tr>
<th>Stage of test</th>
<th>Midspan camber, in.</th>
<th>Prestress loss, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>-0.98</td>
<td>27,200</td>
</tr>
<tr>
<td>1,000,000 cycles</td>
<td>-0.90</td>
<td>26,500</td>
</tr>
<tr>
<td>2,000,000 cycles</td>
<td>-0.87</td>
<td>26,400</td>
</tr>
<tr>
<td>3,000,000 cycles</td>
<td>-0.89</td>
<td>26,300</td>
</tr>
<tr>
<td>4,000,000 cycles</td>
<td>-0.87</td>
<td>26,300</td>
</tr>
<tr>
<td>5,000,000 cycles</td>
<td>-0.84</td>
<td>26,300</td>
</tr>
</tbody>
</table>

* Negative value indicates an upward camber.

**Note:** 1 ksi = 1000 psi = 6.895 MPa; 1 in. = 25.4 mm.

### Table 5. Summary of fatigue test parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>0 to 4 million cycles</th>
<th>4 to 5 million cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper bound load</td>
<td>179.6 kips</td>
<td>174.6 kips</td>
</tr>
<tr>
<td>Decompression load</td>
<td>130.0 kips</td>
<td>125.0 kips</td>
</tr>
<tr>
<td>Lower bound load</td>
<td>50.9 kips</td>
<td>45.9 kips</td>
</tr>
<tr>
<td>Measured steel stress range</td>
<td>10.0 ksi</td>
<td>10.0 ksi</td>
</tr>
</tbody>
</table>

**Note:** 1 ksi = 1000 psi = 6.895 MPa; 1 kip = 4.448 kN.

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Fig. 7. Flexural test setup.

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of the deck slab cracks. Therefore, it is believed that the variable stiffness of the girder may have been a contributing factor to the deck slab cracking.

**FLEXURAL STRENGTH TESTS**

After completion of the long-term static load test and fatigue load test, Girders 3 and 4 were tested to determine the ultimate flexural strength. Both girders were tested in flexure to failure approximately 660 and 86 days after fabrication, respectively. The test setup used for the flexural tests is shown in Fig. 7.

In addition to the internal instrumentation installed during fabrication, four concrete surface strain gauges were added at midspan on the top surface of the deck slab prior to the flexural tests. Pairs of potentiometers were used to monitor vertical girder displacements at both ends and at midspan. Crack width gauges were also placed across the existing midspan cracks that were noted prior to release.

Using hydraulic jacks, load was applied to the girders in increments of approximately 2000 lbs (8.9 kN). Instrumentation was monitored continuously throughout each test using a 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 was tested to determine flexural cracking moment and ultimate moment capacity (failure).

**Discussion of Test Results**

Cracking of the girder specimens during flexural tests progressed as follows. The first flexural crack in Girder 3 occurred prior to the opening of the existing crack that was noted near midspan after form removal. In Girder 4, the existing crack near midspan opened prior to the development of any new cracks. Therefore, the cracking moment in Girder 4 was taken as the moment corresponding to the opening of the existing crack.

Initially, several flexural cracks developed within the constant moment region. Spacing of these cracks coincided approximately with the locations of the stirrups used for shear reinforcement. Additional cracks developed along the shear spans as the load was increased. These cracks gradually became inclined after propagating through the bottom flange. Diagonal cracks not initiated by

<table>
<thead>
<tr>
<th>Girder</th>
<th>Cracking moment, kip-ft</th>
<th>Ultimate moment, kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
<td>AASHTO</td>
</tr>
<tr>
<td>3</td>
<td>3031</td>
<td>3190</td>
</tr>
<tr>
<td>4</td>
<td>2730</td>
<td>3020</td>
</tr>
</tbody>
</table>

Note: 1 kip-ft = 1.356 kN-m.
flexural cracks were not observed during the flexural tests of Girders 3 and 4. Flexural cracks and flexural shear cracks in Girder 3 propagated well into the deck slab at ultimate load.

Measured flexural properties for Girders 3 and 4 are shown in Table 6. Load vs. midspan deflection plots for Girders 3 and 4 are shown in Figs. 8 and 9, respectively. No open cracks in Girders 3 or 4 were noted at the full design service load moment. As indicated in Table 6, measured cracking moments for Girders 3 and 4 were consistent with the values calculated using the provisions from the AASHTO Standard and measured material properties and prestress losses.

Measured ultimate moments for Girders 3 and 4 were greater than the corresponding calculated values. Data from the deck slab concrete surface strain gauges indicated that the full width of the deck slab was effective in both girders. Girders 3 and 4 exhibited approximately 22.4 and 17.4 in. (569 and 442 mm) of deflection at maximum load, respectively. Audible indications of strand fractures were noted at maximum load.

Upon dissection of the failure region of Girder 4, it was determined that four of the 30 prestressing strands in the lower flange had completely fractured. The fractured strands all occurred on one side of the flange (the same side that exhibited slightly higher strains and crack width during the fatigue test), in the bottom row of strands, and at the location of the existing midspan crack. Fatigue fractures (fractures with no apparent reduction of area) of individual wires were detected on two of the four strands that fractured. One of these strands had three wire fatigue fractures and the other had one wire fatigue fracture.

**CONCLUSIONS**

The conclusions presented below pertain only to the research presented in this paper. See Refs. 5 and 6 for complete conclusions for the entire research program.

1. High strength concrete with compressive strengths up to 10,000 psi (69 MPa) can be produced using materials available in the southeastern United States. However, quality control procedures more stringent than fabricators are presently accustomed to using may be required.

2. AASHTO provisions for calculating creep and shrinkage prestress losses may be overly conservative for high strength concrete. After 18 months of being subjected to the design dead load, the prestress losses measured in Girder 3 were significantly less than the total long-term prestress losses predicted using the provisions in the AASHTO Standard. Measured creep and shrinkage deformations of cylinders representing the concrete in Girder 3 were consistent with the findings regarding the measured prestress losses. Long-term camber measurements were also consistent with the measured prestress losses in that little change was noted over the duration of the test period.
3. Measured early-age and long-term prestress losses in Girder 3 were significantly less than the early-age prestress losses in Girder 4. Girder 3 was steam cured for 24 hours after the concrete had reached initial set. Girder 4 was cured under a waterproof tarpaulin for 10 hours. Given that the concrete mixes for the two girders were similar, the very low measured creep and shrinkage characteristics of the concrete in Girder 3 and prestress losses may be partially attributable to the steam curing.

4. Although the steam curing of Girder 3 may have been beneficial from the standpoint of reducing creep and shrinkage, as indicated in Table 2, it also appears that it could be detrimental with respect to development of 28-day compressive strength.

5. Based on the results from this investigation, high strength concrete bridge girders can be expected to perform adequately over the long-term when designed and fabricated in accordance with current AASHTO provisions. Measured long-term prestress losses in Girder 3 were approximately 50 percent less than the expected value. Girder 4 withstood 5 million cycles of fatigue loading without any significant increase in prestress loss or cracking in the girder concrete. After the long-term static load test and the fatigue load test, Girders 3 and 4 both still fulfilled the strength and serviceability requirements of the AASHTO Standard.

6. Deck slab strains measured during flexural strength tests of Girders 3 and 4 indicated that the full width of the slab was effective, as suggested by the AASHTO Standard.

RECOMMENDATIONS

1. Further research into the creep and shrinkage characteristics of high strength concrete is required to determine if and how the AASHTO provisions for creep and shrinkage prestress losses can be modified for high strength concrete. Such research would not only have to address various mix designs and materials, but various curing regimes as well.

2. Research to investigate and determine appropriate strand spacing limitations for high strength concrete should be conducted. For many design conditions, existing girder cross sections cannot make efficient use of high strength concrete using current AASHTO provisions and restrictions imposed by the FHWA on federally funded projects. If strand spacing requirements cannot be modified for high strength concrete, optimized girder cross sections will need to be developed.

3. An investigation into the lateral stability of long slender high strength concrete girders should be performed. One of the potential economic advantages of high strength concrete is the ability to utilize longer spans than conventional prestressed concrete. The longer spans and deeper cross sections result in a decrease in lateral stability. This issue should be considered when evaluating the feasibility of using existing girder cross sections or developing optimized sections for high strength concrete.

4. High strength concrete should be considered as a viable and economically beneficial alternative for future highway bridge projects. Results of research efforts conducted nationwide have been very positive. Bridge projects that demonstrate the benefits of high strength concrete are needed to establish this technology in the industry.

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