Posttensioning of segmental bridges using carbon-fiber-composite cables

Xiong Yang, Pedram Zohrevand, Amir Mirmiran, M. Arockiasamy, and William Potter

Posttensioning is a prevalent and cost-effective construction method for cast-in-place or precast concrete. Together with prefabricated elements, posttensioning is an ideal technique for accelerated bridge construction, reducing on-site construction time and labor. Given the importance of tendons to the structural integrity of posttensioned concrete, they are typically protected in plastic or galvanized steel ducts, which are then filled with grout to prevent corrosion. The ducts, however, may not be completely filled during construction, and if they crack or corrode while in service, moisture and air may reach the tendons and initiate their corrosion.

Corrosion of steel tendons is a major problem for posttensioned concrete, especially because corrosion is often hard to detect inside grouted ducts. While research continues to develop better means to protect steel tendons against corrosion, considerable effort is devoted to finding suitable nonmetallic tendons for posttensioning applications. Fiber-reinforced-polymer (FRP) composite offers a viable alternative to steel tendons. In addition to their superior durability, FRP tendons may result in lower relaxation losses compared with steel.

This paper has been modified since its original publication in the May–June 2015 issue of the PCI Journal. The editors have removed Nabil Grace as an author at his request.

This paper describes the construction and testing of a 3½:1 scale model of the Long Key segmental box girder bridge model posttensioned with carbon-fiber-composite cables (CFCCs) or steel strands at different prestress and for different loading configurations.

Although the study shows the feasibility of CFCC for segmental bridges, end anchorages are factory-made together with the strands, and therefore, strands must be ordered at predetermined lengths.

Whereas the initial stiffness was about the same for both CFCC and steel tendons, the CFCC tendons led to a much softer response after decompression and joint opening, suggesting that a stiffness-based approach may be more appropriate for CFCC.

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Corrosion of steel tendons is a major problem for posttensioned concrete, especially because corrosion is often hard to detect inside grouted ducts. While research continues to develop better means to protect steel tendons against corrosion, considerable effort is devoted to finding suitable nonmetallic tendons for posttensioning applications. Fiber-reinforced-polymer (FRP) composite offers a viable alternative to steel tendons. In addition to their superior durability, FRP tendons may result in lower relaxation losses compared with steel. FRP tendons may be made with different types of fibers, such as carbon, glass, aramid, or basalt. Due to their higher strength, elastic modulus, and excellent durability, carbon FRP (CFRP) tendons are the most practical option. There are a number of commercially available CFRP tendons, including carbon-fiber-composite cable (CFCC) and CFRP tendons.

CFCC has been used in a number of applications, ranging from reinforcement in concrete structures to the stay or main cable in bridges and ground anchors. It has also been used in three bridges in Michigan and in prestressed piles in Florida and Virginia. To date, however, no study has focused on posttensioning of CFCC in segmental bridges. The objective of this study was to develop a realistic test bed for assessing constructability, design, and inspection of CFCC for posttensioning in segmental bridges.
Review of literature

A number of studies have focused on posttensioning applications of CFCC. Two half-scale precast, prestressed concrete box-beam bridge superstructure specimens, one with CFCC tendons and the other with conventional steel strands, were tested by Grace et al. to assess their performance in flexure. Both models exhibited similar behavior irrespective of the type of strand used. Grace et al. studied the flexural behavior of a full-scale, double-tee beam prestressed using bonded pretensioned CFRP tendons and unbonded posttensioned CFCC tendons. Strain distribution along the beam section, deflection, cracking load, tensioning, ultimate load-carrying capacity, and failure mode were investigated. The specimens showed considerable strength beyond service load. The results of this study were applied to the design of the double-tee beams used in the construction of the Bridge Street Bridge in Southfield, Mich.

Grace et al. tested three bridge models in flexure up to failure. The three models included unbonded, posttensioned CFCC tendons; unbonded CFCC tendons with zero posttensioning force; and no unbonded CFCC tendons. The same number of bonded pretensioned and bonded non-posttensioned CFCC tendons was used for each model. Test results showed that the specimen with the higher prestressing force would fail in a sudden and brittle mode, whereas specimens with lower prestressing force would fail gradually. The additional unbonded CFCC posttensioning force delayed the development of cracks and reduced the number and size of cracks, as well as the residual deflections.

Fatigue behavior of concrete structures prestressed with CFRP has also been studied. Dolan et al. reported that cracked CFRP prestressed concrete beams did not show any sign of fatigue failure after 3 million cycles of flexural loading, though cracking was observed after the first million cycles. The beams did not lose any strength due to fatigue even though a gradual softening was observed. Grace et al. investigated the performance of a continuous CFRP prestressed concrete bridge under 15 million cycles of repeated loads. In this study, there was no significant effect on the prestressing force of externally draped posttensioning tendons. Similarly, no sign of damage in CFRP tendons was observed at the deviators.

Long-term performance assessment of FRP tendons in bridges is necessary. A comprehensive structural-health monitoring of highway bridges consists of four steps:

- sensor placement and measurement
- structural identification and modeling
- damage detection and degradation assessment
- decision-making on rehabilitation and maintenance

Limited data are available on the implementation of an integrated monitoring system for FRP tendons. Available nondestructive methods with potentials for inspection of FRP prestressed concrete structures include visual, reflective, imaging, and load testing.

Visual inspection of CFCC tendons could be easily implemented using a borescope but may only offer limited insight as to their performance. Reflective methods rely on transmitting various forms of sonic waves and investigating their return response. Most reflective methods are suited for investigating local and small areas and are generally time-consuming if conducted on a point-by-point basis. The acoustic-emission technique, which effectively listens for wire breaks, has been used for monitoring posttensioned and prestressed concrete bridges. Nondestructive techniques in the imaging category allow for a more detailed visual inspection. They include radiographic imaging or tomographic systems. In general, these techniques are difficult and expensive to implement in field conditions and are often applicable to small areas. Finally, structural-element characteristics can be determined via global load testing. Installation of fiber-optic sensors (at the time of construction) or load cells to monitor stress along CFCC tendons is perhaps the most reliable technique for inspection and monitoring of CFCC.

Even though segmental bridge construction is popular in practice, only a few experiments have been carried out on these types of bridges, and none with CFCC tendons. A comprehensive study investigated the behavior of a three-span external posttensioned concrete box-girder bridge model with different types of joint connections, including dry joints and epoxy joints. The bridge model was tested under service loads, factored loads, and ultimate loads for different loading configurations, all of which could result in maximum flexure and maximum shear. Test results demonstrated that the epoxy joints could help prevent joint opening and limit bridge deflections. Also, the performance of a scaled single-cell precast concrete posttensioned segmental box-girder bridge model with dry joints subjected to cyclic loading and temperature changes was investigated. Joint opening and cracking were found negligible up to 2 million load cycles. Temperatures did not seem to make much difference across the section either.

Experimental program

Specimen preparation and erection process

A simple-span 1:3½-scale superstructure model of the Long Key segmental box-girder bridge in the Florida Keys was constructed as a test bed for a series of experiments to compare posttensioning with CFCC and steel strands. Consistent with the prototype bridge, the model was designed based on the American Association of State Highway and...
Transportation Officials’ (AASHTO’s) *Standard Specifications for Highway Bridges.* In the 1990s, Arockiasamy et al. had used a similarly scaled model of the same prototype, as described earlier. Figure 1 shows the bridge model consisting of seven trapezoidal box-girder segments and two solid end blocks with a rectangular section. The tendons were harped at a 5˚ angle with contacts limited to the end blocks, and the two deviators in the two segments were placed adjacent to the center segment. Each deviator was designed as a beam to resist uplift at the harping point. The solid end blocks were designed to resist the prestressing force. The segments were connected as dry joints with multiple shear keys along both flanges and webs. Each segment was reinforced using ¼ in. (6 mm) diameter steel bars with a yield strength of 60 ksi (410 MPa) spaced at 3½ in. (89 mm) on center in both longitudinal and transverse directions.

Figure 2 shows the fabrication process for the bridge model. Wooden formwork was assembled for the entire bridge model to match cast the segments. Styrofoam was used to create hollow-cores of box-girder sections and the joint shear keys on both flanges and webs of each segment. Foam blocks of each pair of segments were matched with the divider foam, interlocking the tongue and grooves that were precut on their surfaces. The pieces of foam were bonded using foam glue. Spacers were made using a circular plate tack welded at one end of a screw to support the foam and to control the thickness of the bottom flange and the webs (Fig. 2). Formwork and the template were designed and built for each anchorage zone to accurately and firmly support the posttensioning ducts and steel plates.
at the exact 5° harping angle. The posttensioning ducts within the end blocks were made using a cardboard tube that is supported between the anchorage block and the first foam divider (Fig. 2 upper right). A boom pump facilitated casting of the entire bridge model and its supports using a self-consolidating concrete with a compressive strength of 8630 psi (59.5 MPa) as measured at the time of testing from at least three companion cylinders. The segments were de-molded a week after casting. The foams were carefully removed from within each segment using acetone. The segments were then erected side by side with temporary wooden stands before tensioning.

**Posttensioning**

Table 1 lists the geometric and material properties of the two types of strands used for posttensioning the bridge model, as reported by their manufacturers. CFCC tendons are made as seven-wire twisted cables similar to typical steel strands and are available in diameters up to 1.57 in. (40 mm). They are guaranteed in strengths up to 241 kip (1070 kN). For the bridge model, 0.49 in. (12.5 mm) diameter CFCC tendons were used to closely match the ½ in. (13 mm) diameter, seven-wire low-relaxation steel strands.

CFCC tendons are shipped as coiled strands with prefabricated special end anchorages. Although regular chuck anchoring devices for steel tendons are widely available, cost-effective, and reliable, they cannot be applied directly to CFCC tendons because of the brittleness of the cables under transverse-gripping pressure. The anchorage system for CFCC tendons is a factory-made steel sleeve filled with a proprietary resin. The sleeve has internal threads to facilitate posttensioning using a threaded-steel rod and external threads using a circular nut to help lock the prestressing force. This system does not easily accommodate deviations from the preordered lengths, whether due to construction tolerances or miscalculation in the elongation of the cable. Any such deviation may require abandoning the cable entirely or having to develop a build-up at the jacking end to make up for the difference.

Cables in the bridge model were protected at both ends and at the two deviators by using flexible reinforced braided-PVC tubes as jackets. A ½ in. (13 mm) thick neoprene pad was placed atop the cables at each harping point in the deviators to avoid potential damage at sharp corners. The four cables on each side of the model were passed through the segments before placing the end block on the south side of the model, which effectively closed off the system.

From the constructability perspective, the most feasible option for simultaneous posttensioning of multiple cables is to develop a super coupler that transfers the forces from the sleeves of multiple CFCC tendons on one side to regular steel strands on the other side that are pulled by a hydraulic jack. This option allows the contractor to use its current jacking tools to stress CFCC tendons. For the bridge model, however, a second option was developed (Fig. 3). It included two 14 7⁄8 × 12 in. (378 × 305 mm), 2 in. (50 mm) thick steel plates with four 1½ in. (38 mm) diameter holes at 3 in. (75 mm) on center to allow passing the end sleeves through the holes and turning the locknuts or placing the load cells. A pair of hydraulic jacks was sandwiched between the two steel plates at each anchorage. A similar approach was applied to steel strands, except for the use of chucks instead of sleeves and locknuts (Fig. 3). Both CFCC and steel strands were posttensioned alternatively between the east and west sides of the model in increments of 20 kip (89 kN), with an average force of 5 kip (22 kN) in each cable, to reach the target prestressing force, while the load in each cable was continuously monitored using a load cell.

The cables were stressed up to 63%, 65%, and 70% of their guaranteed capacity. The 63% prestress was designed to match the prestress force that was tested by Arockiasamy et al. Sixty-five percent is the maximum stress permitted for carbon cables per ACI 440.4R-04. Seventy percent was chosen to evaluate the performance of the bridge model under an overdesigned prestress force. The relaxation loss of CFCC was recorded at 63% and 70% prestress for different durations before load testing the model. After monitoring of stresses in CFCC for almost two months, the 63% prestress had dropped to 62% before load testing and is noted as such in the following sections.

**Test setup**

Figure 4 shows the test setup. The test frame included 16 high-strength threaded rods tied down to the strong floor, two W sections supported by the threaded rods on the two sides of the model in the longitudinal direction, one long W section in the lateral direction, and two hollow structural sections as spreader beams. Under each hollow structural

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**Table 1. Geometric and material properties CFCC and steel strands**

<table>
<thead>
<tr>
<th>Strand type</th>
<th>CFCC</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal diameter, in.</td>
<td>0.492</td>
<td>½</td>
</tr>
<tr>
<td>Effective area, in.²</td>
<td>0.118</td>
<td>0.153</td>
</tr>
<tr>
<td>Guaranteed strength, ksi</td>
<td>351</td>
<td>270</td>
</tr>
<tr>
<td>Guaranteed capacity, kip</td>
<td>41.4</td>
<td>41.3</td>
</tr>
<tr>
<td>Elastic modulus, ksi</td>
<td>22,300</td>
<td>29,000</td>
</tr>
<tr>
<td>Mass density, lb/ft</td>
<td>0.10</td>
<td>0.53</td>
</tr>
</tbody>
</table>

Note: Steel strands were of seven-wire, low-relaxation type. CFCC = carbon-fiber-composite cable. 1 in. = 25.4 mm; 1 lb = 4.448 N; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.
section, there were two loading points, together simulating a single truck. At each loading point, a steel hinge was placed under the hollow structural section and on top of a 2 in. (50 mm) thick steel plate and a 1 in. (25 mm) thick 12 × 6 in. (300 × 150 mm) neoprene pad, the size of which was scaled down to simulate the tire of a standard AASHTO HS truck on the top flange of the bridge segments.

Instrumentation

Eight doughnut load cells were used at the dead end of the model to continuously monitor prestress force in each cable. A total of 13 string pots were attached to the segments to measure joint deflections. Six linear potentiometers were mounted on the bottom flange to monitor the opening at critical joints in each load test. Six strain gauges were attached on the top and bottom flanges to record concrete strains near critical joints. Two calibrated pressure transducers were connected to the two hydraulic jacks to monitor the applied loads. A high-speed data acquisition system was used to record the data at a high frequency. Last, four web cameras and floodlights were placed inside the two segments with deviators. The cameras were mounted right on top of the deviators to visually monitor the conditions of the tendons during the tensioning process and the load testing. Figure 5 shows the images for the cameras.

Loading protocol

Because the prototype bridge was designed for two lanes of traffic, two standard HS20 trucks were considered as the design live load on the model. Each truck was simulated with two patch loads in the longitudinal direction of the bridge (Fig. 6). Each patch was scaled down from the
tire of a standard truck, as described earlier. Figure 6 also shows the three critical positions of the two trucks based on a detailed analysis to develop maximum shear or flexure in the model. Position 1 simulated the most critical flexural and shear stresses at the joint closest to the midspan of the bridge model, while position 2 represented maximum flexural stresses in the center segment. Last, position 3 simulated maximum shear at the support joint. The model was tested up to service loads at each of the three load positions, but it was tested up to factored loads only at position 1. Tests were repeated for each strand type and level of prestressing.

The load in each case was calculated so as to create a stress resultant in the bridge model that was similar to the stress experienced in the prototype bridge. An impact factor of 20.6% was calculated based on the 118 ft (36.0 m) span length of the prototype bridge. Therefore, the sum of live load and impact was calculated as 9.66 kip (43.0 kN) for the front axle and 38.7 kip (172 kN) for each of the two rear axles of each HS20 truck on the prototype bridge.

The three axles for an HS20 truck were simulated as two equivalent axles on the bridge model at a scale factor of 3½:1, leading to 7.1 kip (32 kN) for each axle. Also, a uniform dead-load compensation of 2.5 times the self-weight of the bridge model was considered to account for the scale factor of 3.5:1. The dead-load compensation was replaced with point loads to result in the same maximum moments at critical points. The magnitude of the dead-load compensation was 20.7, 20.3, and 19.1 kip (92.1, 90.3, and 85.0 kN), for positions 1, 2, and 3, respectively. Therefore, the total service load on the model was 34.9, 34.5, and 33.3 kip (155, 153, and 148 kN) for positions 1, 2, and 3, respectively. The factored load was 51.4 kip (229 kN) using appropriate load factors.11

Because the purpose of the experiments was to assess the behavior of posttensioning strands and because the expected mode of failure was the crushing of concrete (Arockiasamy et al.10), tests were stopped at the target factored loads due to safety concerns and to allow for repeated loading of the model as a test bed.

Test results and discussions

Physical observations

No major crack or failure was observed in the concrete segments during any of the experiments. The images from the interior cameras did not reveal any damage in the strands. However, when CFCC tendons were removed from the bridge model at the conclusion of the experiments, both the
ings for position 1 were measured at the joint between the two axles. For position 2, the deflections were measured at the midspan of the bridge model and the data for the joint opening was the average reading of four potentiometers attached at the two adjacent joints. For position 3, the displacements and openings are displayed for the joint between the first segment and the end block. The bridge cables and the neoprene pads used at the deviators showed minor friction damage (Fig. 5). Although no stiffness or strength degradation was observed throughout the experiments, the observed damage implied that CFCC tendons may require a more rigid jacket to protect them from abrasion.

**Relaxation losses of CFCC tendons**

Figure 7 shows the prestress relaxation losses for CFCC tendons recorded at 63% and 70% of the guaranteed cable strength for different durations. The stress loss for the 63% prestress after 57 days was about 0.7 kip (3 kN), or 2.7% of the initial stress, which made the effective prestress at the time of testing 62% of the guaranteed strength. The stress loss after 15 days was 0.2% for the 70% stress. The relaxation loss for CFCC tendons was found to be comparable to that of low-relaxation steel tendons.

**Performance under service loads**

Figures 8 and 9 show the load displacements and joint openings, respectively, for the bridge model posttensioned with CFCC tendons under service loads at three different load positions and at different prestress. Prestress was measured at the time of testing. The deflections and openings for position 1 were measured at the joint between the two axles. For position 2, the deflections were measured at the midspan of the bridge model and the data for the joint opening was the average reading of four potentiometers attached at the two adjacent joints. For position 3, the displacements and openings are displayed for the joint between the first segment and the end block. The bridge
model deflected linearly for the most part at positions 1 and 2, with some stiffness degradation at about service load level. Joint openings, on the other hand, were more curvilinear. Both the displacements and joint openings at position 3 were found to be linear and yet insignificant.

Table 2 summarizes test results at service loads for both CFCC and steel strands. The deflection limit of \( \frac{L}{800} \) (where \( L \) = span length), which translates to 0.51 in. (13 mm) for the bridge model, is found acceptable for both steel and CFCC at the minimum stress of 65%. As noted, displacements and joint openings at position 3 were found to be insignificant.

**Performance under factored loads**

Table 3 summarizes test results at factored loads for the two types of tendons at different prestress values. These tests were conducted only at position 1. Figures 10 and 11 show the complete load displacement and joint opening responses, respectively. Due to the limited stroke of the hydraulic jacks used for testing and safety concerns, the maximum factored load was not achieved at 62% and 65% of strength for CFCC tendons, given the higher deformation of the bridge model when posttensioned with CFCC.

The bridge model showed a clear bilinear response for both types of strands and at all prestress values. Before joint opening, the initial stiffness of the bridge model depended on the moment of inertia of the entire section, including concrete and the tendons. As such, the different elastic moduli of the two types of strands did not significantly affect the initial stiffness or the first slope of the response because the tendons contributed only a small portion (3% for steel and 2% for CFCC) to the entire stiffness of the section. Therefore, the initial stiffness of the bridge model posttensioned with CFCC tendons was only 1.3% lower than that with steel strands, even though the elastic

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**Figure 8.** Load displacements of bridge model with carbon-fiber-composite cable tendons at service load positions. Note: 1 kip = 4.448 kN.

**Figure 9.** Load-joint openings of bridge model with carbon-fiber-composite cable tendons at service load positions. Note: 1 kip = 4.448 kN.

**Table 2. Summary of test results at service loads**

<table>
<thead>
<tr>
<th>Strand type</th>
<th>CFCC</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective prestress force, kip</td>
<td>25.6</td>
<td>27.1</td>
</tr>
<tr>
<td>Effective prestress, % of guaranteed strength or ultimate strength</td>
<td>62</td>
<td>65</td>
</tr>
<tr>
<td>Loading position 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Displacement, in.</td>
<td>0.484</td>
<td>0.408</td>
</tr>
<tr>
<td>Joint opening, in.</td>
<td>0.015</td>
<td>0.015</td>
</tr>
<tr>
<td>Loading position 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Displacement, in.</td>
<td>0.575</td>
<td>n/a</td>
</tr>
<tr>
<td>Joint opening, in.</td>
<td>0.019</td>
<td>n/a</td>
</tr>
<tr>
<td>Loading position 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Displacement, in.</td>
<td>0.065</td>
<td>n/a</td>
</tr>
<tr>
<td>Joint opening, in.</td>
<td>0.003</td>
<td>n/a</td>
</tr>
</tbody>
</table>

Note: CFCC = carbon-fiber-composite cable; n/a = not applicable. 1 in. = 25.4 mm; 1 kip = 4.448 kN.
portion of the section and the corresponding joint openings between the segments. For each type of strand, the higher prestress force helps extend the initial slope of the response and delays the decompression and joint opening. The higher prestress force also slightly increases the initial stiffness of the bridge model, in other words, the first slope of the bilinear response.

The transition zone that connects the two linear portions of the response correlates to the decompression of the bottom modulus of CFCC is about 23% lower than that of steel.

On the other hand, the second slope of the response corresponded to the fully opened joints, in which the tendons played a much more important role by contributing to about 99% of the stiffness of the section. Therefore, the secondary stiffness of the bridge model correlated directly with the stiffness of the tendons, which explains the much higher stiffness of the bridge model when posttensioned with steel strands.

The transition zone that connects the two linear portions of the response correlates to the decompression of the bottom

<table>
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<tbody>
<tr>
<td><strong>Strand type</strong></td>
</tr>
<tr>
<td>Effective prestress force, kip</td>
</tr>
<tr>
<td>Effective prestress, % of guaranteed strength or ultimate strength</td>
</tr>
<tr>
<td>Initial stiffness, kip/in.</td>
</tr>
<tr>
<td>Secondary stiffness, kip/in.</td>
</tr>
<tr>
<td>Deflection at factored load, in.</td>
</tr>
<tr>
<td>Joint opening at factored load, in.</td>
</tr>
</tbody>
</table>

Note: CFCC = carbon-fiber-composite cable; n/a = not applicable. 1 in. = 25.4 mm; 1 kip = 4.448 kN.
force and average strain increase in the strands. The bridge model has a slightly higher initial stiffness with steel strands compared with CFCC. On the other hand, it takes a greater load to develop the same force and strain increase in CFCC strands compared with steel strands.

As described earlier, the bridge model was posttensioned with CFCC to develop the same prestressing force as that of steel strands (Eq. [1]).

$$ A_{ps}f_{ps} = A_{pf}f_{pf} $$

(Eq. [1]).

$A_{ps}$ = area of prestressing carbon-fiber tendons

$A_{pf}$ = area of prestressing steel strands

$f_{ps}$ = stress in prestressed carbon-fiber tendons

$f_{pf}$ = stress in prestressed steel strands

This approach, however, leads to much higher deformations and joint openings for segmental bridges with CFCC tendons, as observed in the experiments. Therefore, one may consider a stiffness-based approach to make the responses of the two types of tendons more comparable (Eq. [2]).

$$ A_{ps}E_{ps} = A_{pf}E_{pf} $$

(Eq. [2]).

$E_{ps}$ = modulus of elasticity of carbon fiber

$E_{pf}$ = modulus of elasticity of steel

Such a stiffness-based approach could, however, reduce the stress in CFCC to compensate for its lower elastic modulus (Eq. [3]).

$$ f_{pf} = \frac{E_{ps}}{E_{pf}} f_{ps} $$

(Eq. [3]).

For instance, the maximum allowable jacking stress for steel is typically 80% of its ultimate strength, compared with the 65% limit recommended for CFCC by ACI 440.4R-04. Although CFCC could physically be stressed as high as 65% of its guaranteed strength without any concern for stress rupture, using this stiffness-based equivalency approach could potentially reduce the stress to 47% of its guaranteed strength to provide a similar response to steel tendons.

Figure 11. Load-joint openings of bridge model with CFCC and steel strands at factored loads. Note: CFCC = carbon-fiber-composite cable. 1 kip = 4.448 kN.
stiffness seemed to be approximately the same for both types of tendons, CFCC tendons led to a much softer response after decompression and joint opening. Higher prestress in both CFCC and steel tendons can delay joint openings and reduce deflections in the segmental bridge model. Due to the higher flexibility of the segmental bridge model posttensioned with CFCC, a stiffness-based equivalency approach may provide a more comparable performance with the same bridge model posttensioned with steel. Such an approach, however, may lower the stress in CFCC commensurate with its lower elastic modulus.

Further research is needed on constructability, inspection, and maintenance of CFCC. It may also be of great benefit to replace factory-made anchorages for CFCC tendons with an on-site assemblage.

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References


Notation

$A_{pf}$ = area of prestressing carbon-fiber tendons

$A_{ps}$ = area of prestressing steel strands

$E_{pf}$ = modulus of elasticity of carbon fiber

$E_{ps}$ = modulus of elasticity of steel

$f_{pf}$ = stress in prestressed carbon-fiber tendons

$f_{ps}$ = stress in prestressed steel strands

$L$ = span length
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Abstract

Carbon-fiber-composite cables (CFCC) offer a corrosion-free alternative to steel strands. This paper describes the construction and testing of a 3 1/2:1 scale model of the Long Key segmental box girder bridge model posttensioned with CFCC or steel strands at different prestress and for different loading configurations. The study shows the feasibility of CFCC for segmental bridges. The main concern, however, is that end anchorages are factory-made together with the strands, and therefore, strands must be ordered at predetermined lengths, considering the elongation of stressed tendons. The model behaved bilinearly with both types of strands. Whereas initial stiffness was about the same for both types of strands, CFCC tendons led to a much softer response after decompression and joint opening. Given the higher flexibility of the segmental bridge model posttensioned with CFCC, a stiffness-based equivalency approach may provide a more comparable performance with the same bridge model posttensioned with steel. Such an approach, however, may lower the stress in CFCC commensurate with its lower elastic modulus.

Keywords

Bridge, carbon-fiber-composite cable, carbon-fiber-reinforced polymers, CFCC, CFRP, posttensioning, segmental.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

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