Performance of an AASHTO Beam Bridge Prestressed with CFRP Tendons

Nabil Grace¹; Elin Jensen²; Vasant Matsagar³; and Prasadu Penjendra⁴

Abstract: Corrosion-induced deterioration of steel RC highway bridges is one of the major distress types that can render a structurally deficient bridge before reaching the design life. One feasible solution to the problem is to replace the conventional steel reinforcement with noncorrosive carbon fiber—reinforced polymer (CFRP) reinforcements. However, the CFRP reinforcement as an internal reinforcement has not been explored in AASHTO-type prestressed concrete beam bridges. AASHTO-type beams have an I-type cross section with a bottom flange, and on integration of the deck slab, the final shape is a bulb-T section. This paper discusses the experimental investigation of a precast prestressed AASHTO control beam and a bridge model. A 12.5-m-long one-third scale AASHTO-type control beam was experimentally investigated for its flexural behavior when reinforced and prestressed with CFRP. Subsequently, a one-third scale bridge model made of five such beams was constructed, instrumented, and tested under both service and ultimate load conditions. As anticipated, the control beam and the bridge model failures were initiated by rupturing of the prestressing CFRP tendons at the bottom layer. The observed flexural response of the bridge model was in close agreement with that of the control beam. As expected, the failure mode was progressive, with extensive cracking of the bridge model, which gives significant warning prior to the ultimate collapse, overcoming issues related to the otherwise brittle behavior of the CFRP-reinforced structures. It is therefore highly recommended to provide the CFRP tendons in different layers along the depth of the beams to effectively address the issues related to brittle failure exhibited by CFRP reinforcements. DOI: 10.1061/(ASCE)BE.1943-5592.0000339. © 2013 American Society of Civil Engineers.

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Introduction

In this paper, the flexural behavior of an AASHTO-type beam bridge reinforced and prestressed with carbon fiber—reinforced polymers (CFRPs) is presented. The high strength-to-weight ratio, superior fatigue resistance, ease of handling, low thermal expansion, and low relaxations are some of the key advantages of the composite materials compared with conventional steel reinforcement. These characteristics make the FRPs an emerging potential construction material in the bridge construction industry. The importance and use of AASHTO-type concrete beam bridges have gained momentum recently. For a wide range of practical spans, AASHTO beams are often selected because they have a simple cross section, have higher flexural capacities, and are less expensive to fabricate. In addition, the noncorrosive CFRP reinforcement provided in the bridges helps sustain the bridge for a longer lifespan with minimum maintenance costs. Apparently, there has been very limited research on the application of the FRP in AASHTO-type beams.

Achieving ductile failure of the FRP-reinforced structures in flexure by various approaches has remained the focus of research in recent years. Fam et al. (1997) concluded that the flexural behavior of the beams prestressed with FRP tendons exhibited similar stiffness as that of the beam prestressed with steel reinforcement. Abdelrahman and Rizkalla (1999) confirmed that the flexural behavior of concrete beams prestressed with CFRP tendons under repeated load was similar to that for the beams prestressed with steel strands, if the steel is in the elastic stage before unloading. A full-scale test of an overreinforced FRP prestressed double-tee beam was presented by Grace et al. (2003). The results provided from this full-scale experimental study coupled with the design equations developed by Grace and Singh (2003) were directly used in the construction of the United States’ first precast CFRP prestressed concrete bridge called the Bridge Street Bridge in Michigan.

Dolan and Swanson (2002) developed a strength design approach to maximize the use of vertically distributed CFRP tendons located at various depths. Naaman et al. (1993) experimentally and theoretically analyzed the partially prestressed concrete T-beams with carbon fiber composite cable (CFCC) strands. Progressive failure was achieved in the T-beams, and the postpeak load-deflection behavior was characterized by an incremental steplike descent corresponding to the rupture of CFCC strands. Morais and Burgoyne (2003) proposed the step layering of the FRPs to have a progressive failure and to improve ductility.

Attempts were made in developing design approaches for the structures reinforced with FRP materials. In the unified design approach for the design of CFRP prestressed concrete beams proposed by Grace and Singh (2003), a strain compatibility-based approach was suggested. This approach was validated by...
experimental results conducted on double-tube beam bridge models (Grace et al. 2003). Furthermore, a compression controlled failure mode was recommended as the design failure mode for the CFRP prestressed concrete beams. This recommendation was based on the better ductility characteristics of the overreinforced sections. ACI 440.1R-06 [American Concrete Institute (ACI) Committee 440 2006] general guidelines suggest using a higher reserve strength of the FRP-reinforced members to compensate for the lack of ductility. Further, the failure mode involving concrete crushing has been preferred compared with rupture of the FRP reinforcement. Moreover, it concludes that the member would not exhibit ductility in any case as commonly observed in steel underreinforced members. Despite these ACI 440.1R-06 (ACI Committee 440 2006) guidelines to prefer overreinforced beams, for an underreinforced beam, a careful design of the vertical distribution of the longitudinal FRP tendons is essential to enforce a slow progressive failure of tendons, which simulates a ductile failure of the section reinforced with steel. In addition, strength equations and a strain compatibility approach of steel prestressed beams could be directly applied to calculate the flexural capacity of the underreinforced CFRP prestressed beams.

The objective of this study is to investigate the flexural behavior of the AASHTO-type beam and a bridge model in demonstrating progressive failure. To assess the flexural behavior, the AASHTO-type control beam and a bridge model with CFRP tendons as internal reinforcement were constructed, instrumented, and tested. The flexural behavior was investigated through the load-deflection responses, ultimate load-carrying capacities, modes of failures, load-strain responses, and energy ratios of the control beam and the bridge model.

**Design Details**

The issue of brittle behavior in CFRP-reinforced and prestressed concrete structures is common, and this could be overcome by achieving a progressive type of failure in deep beams. A mathematical model based on the strain compatibility approach was developed to test different levels of prestressing forces at various layers to achieve progressive failure of prestressing tendons. The model showed a pyramid-type prestressing pattern, with a reinforcement ratio of 0.35% as optimum given the model scale and laboratory limitations. The unified design approach proposed by Grace and Singh (2003) and ACI 440.4R-04 design guidelines (ACI Committee 440 2004) were also used in the development of the laboratory-scale bridge model and control beam.

**Construction of AASHTO Beams**

Six prestressed AASHTO-type I-beams were constructed, each 502 mm deep with top and bottom flange widths of 203 mm and web thickness of 95 mm. The horizontal shear transfer/composite action between the beam and deck slab was ensured through 57-mm protruding stirrups above the top of the beam (Fig. 1). Each AASHTO beam reinforcing cage consisted of three longitudinal CFRP prestressing tendons and eight longitudinal nonprestressing CFRP rods, both having a diameter of 10 mm as flexural reinforcement distributed vertically along the depth of the beam in layers to achieve a controlled progressive type of failure with successive rupturing of the CFRP tendons. Steel stirrups, with a diameter of 10 mm spaced at a center-to-center distance of 102 mm, were used as shear reinforcement. The steel stirrups were used in this investigation because the main focus was on the flexural behavior and not shear. An arrangement to accommodate the transverse reinforcement for the diaphragms was provided in the beam with the help of 13-mm-diameter polyvinylchloride pipes at designated places. The material properties of the CFRP tendons and steel stirrups used in the construction of the beams are provided in Table 1. A rectangular end-block with 532 mm length, 203 mm width, and 502 mm depth was provided at each end of the beams to resist bursting stresses generated during the transfer of the prestressing forces. Moreover, confinement in the end-block regions was provided by the rectangular stirrups spaced at a reduced center-to-center spacing of 51 mm. The arrangement of the longitudinal reinforcement and stirrups spacing is shown in Fig. 2.

On completion of constructing the formwork, a reinforcement cage of corresponding materials was assembled and placed inside the formwork. The CFRP prestressing tendons were passed through the reinforcement cage at designated layers and positioned between the two bulkheads. Calibrated load cells were mounted on the prestressing tendons at the dead-end and connected to a data acquisition system to monitor and record the level of pretensioning.
forces. The pretensioning force was applied through a 305-mm center-hole hydraulic jack situated at the live end. Each prestressing CFRP tendon was stressed to an average jacking force of 90 kN (1,146 MPa) for achieving a total pretensioning force of 270 kN on each AASHTO beam. After the prestressing operation, the ready-mix concrete in accordance with the Michigan Department of Transportation (MDOT) standard specifications for precast pre-stressed beams was placed in the formwork. The average 28-day compressive strength of the concrete in the AASHTO beams was 48 MPa.

Table 1. Properties of CFRP Tendons and Steel Stirrups Used in Control Beam and Bridge Models

<table>
<thead>
<tr>
<th>Material characteristics</th>
<th>CFRP tendons</th>
<th>Steel stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Diversified Composites Inc. (DCI 2000)]</td>
<td>[Nawy 2003]</td>
<td></td>
</tr>
<tr>
<td>Nominal diameter, (d) (mm)</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Cross-sectional area (mm(^2))</td>
<td>71.3</td>
<td>71.3</td>
</tr>
<tr>
<td>Ultimate tensile strength (MPa)</td>
<td>2,344</td>
<td>414</td>
</tr>
<tr>
<td>Elastic modulus (GPa)</td>
<td>157</td>
<td>200</td>
</tr>
<tr>
<td>Ultimate tensile strain (%)</td>
<td>1.49</td>
<td>9.00</td>
</tr>
<tr>
<td>Average measured breaking load (kN)</td>
<td>164.6</td>
<td>44.1</td>
</tr>
<tr>
<td>Yield strain —</td>
<td>0.002</td>
<td></td>
</tr>
</tbody>
</table>

Construction of the Control Beam

One of the six precast prestressed AASHTO beams was used as a control beam for the bridge model. A 64-mm-thick and 502-mm-wide deck slab was topped on this individual beam to simulate the deck slab of the bridge model. For the deck slab reinforcement, four longitudinal nonprestressing CFRP rods of 10 mm diameter and transverse nonprestressing CFRP tendons of the same diameter spaced at a center-to-center distance of 203 mm were used.

Construction of the Bridge Model

A three-dimensional (3D) view of the bridge model is shown in Fig. 3, and its cross-section details at midspan are shown in Fig. 4. The bridge model was 12.5 m in length and 2.5 m in width, constructed of five precast prestressed concrete AASHTO beams. The AASHTO

Fig. 2. Details of longitudinal flexural reinforcement

Fig. 3. 3D view of the bridge model
beams of the bridge model were arranged at a center-to-center distance of 502 mm, joined with five equally spaced CFRP reinforced transverse diaphragms, and topped with a 64 mm thick CFRP-reinforced deck slab. The AASHTO beams were placed side by side at a center-to-center distance of 502 mm. The CFRP rods of 2.5 m length were passed through the previously made transverse holes in the beams and epoxy grouted to brace the beams transversely (Fig. 4) to serve as the transverse reinforcement of the diaphragms. The vertical reinforcement of the diaphragms was attached to the transverse reinforcement, and formwork was provided around the reinforcement. Prior to the placement of the deck slab reinforcement, the formwork for the deck slab was attached to the beams and diaphragm formwork and supported as typically practiced in shored construction. The placing of concrete for the diaphragms and deck slab was carried out as mentioned earlier for the beams, achieving 28-day compressive strength of 48 MPa. The completed bridge model is shown in Fig. 5.

**Instrumentation and Test Setup**

Both the control beam and the bridge model were simply supported with an effective span length of 12.2 m. The test program included an ultimate load test on the control beam, effective prestress force, ultimate load, and residual strength tests on the bridge model. Linear motion transducers were used to measure the deflections, and electrical resistance strain gauges were used to measure the strains at predetermined locations.

**Test Setup: Control Beam**

Flexural load was applied to the control beam through a load-spreader, which was designed to distribute a two-point load symmetrically to achieve flexural failure. The test setup for the control beam is shown in Fig. 6. The length between the two loading points of the load-spreader was 1.2 m. The control beam was subjected to several loading and unloading cycles to separate the elastic and inelastic energies under the load-deflection curve; the ductility was evaluated based on the energy concept (Grace et al. 1998). The loading cycles applied on the control beam were 27, 53, 80, 93, 107, and 120 kN and a final load cycle up to failure.

**Test Setup: Bridge Model**

The load applied to the bridge model was through a four-point load-spreader. The length between two loading points of the load-spreader was 1.2 m, and the width of each loading point was 508 mm, which were obtained by scaling down (one-third) the axle distance of the AASHTO HL-93 truckload. The bridge model was also subjected to several loading and unloading cycles to separate the elastic and inelastic energies. The loading cycles applied were 89, 178, 267, 356, 445, and 534 kN and a final load cycle up to failure. The first and second loading cycles additionally helped in determining the effective prestress forces in the bridge model.

The bridge model was further subjected to a residual strength test, as only Beam B-3 failed partially during the ultimate test. A transversely placed loading steel I-beam of 2.54 m length was used to
Results and Discussion

The flexural behavior of the control beam and the bridge model is discussed in this section in terms of the modes of failure: load-deflection responses; strains in the prestressing and nonprestressing tendons and the extreme concrete fibers; energy-based ductility indices; and the ultimate load-carrying capacities. To assess the performance of the CFRP tendons during progressive failure, the tension-controlled design approach was used.

Load-Deflection Response

The midspan deflections of the simply supported control beam and the bridge model were used to evaluate the load-deflection response. The load-deflection response of the control beam during the loading and unloading cycles is shown in Fig. 8. The load-deflection response is bilinear until reaching the ultimate load with two distinct slopes. The first slope (OA) is from the beginning of the loading until the cracking load, and the second slope (AB) is from cracking until the ultimate load. The change in slope corresponds to the reduction of flexural rigidity to 75% because of the developed flexural cracks in the tension zone of the concrete. The beam cracked in the second loading cycle at 20 kN, with a deflection of 10 mm. The ultimate load-carrying capacity of the control beam was 127 kN, with a deflection of 215 mm.

At the ultimate load, the prestressing CFRP tendons in the seventh layer (bottommost layer) ruptured. Because of this rupture, a sudden drop of load from 127 to 71 kN was observed, and then an increase in the applied load started again. The slope of the load-deflection curve (CD) at this stage was almost parallel to the slope of the curve from cracking load to the ultimate load (AB). The prestressing CFRP tendons in the sixth layer ruptured at a load of 97 kN, with a deflection of 283 mm. This rupture caused another drop in the load from 97 to 63 kN. A further increase in the load resulted in a load-deflection response, with the slope (EF) almost parallel to the initial postcrack slope (AB). After reaching a load of 103 kN, failure of the nonprestressing CFRP rods in the bottom layer and in the successive upper layers was experienced, which exhibited a progressive type of failure, with an ultimate deflection of 502 mm. The estimated ultimate load from the unified design approach proposed by Grace and Singh (2003) was 131 kN, showing a difference of about 3% between the predicted and experimental results.

The load-deflection response of the bridge model during the conducted loading and unloading cycles is shown in Fig. 9. The failure sequence of the bridge model included (1) rupture of the prestressing CFRP tendons in the bottom layer of central Beam B-3, (2) local shear failure of the deck slab around the load-spreader, (3) rupture of the nonprestressing CFRP rods in the midspan diaphragm, and (4) separation of loaded Beam B-3 from the deck slab and other diaphragms. The rupture of the prestressing CFRP tendons was observed at 605 kN, and the experienced ultimate load was 650 kN, with a deflection of 264 mm. At the separation of loaded Beam B-3 (shear delamination failure), an ultimate deflection of 342 mm was experienced.

Although different postultimate failure modes were observed for the control beam and the bridge model, the ultimate load-carrying capacity of the bridge model was 5.08 times higher than that of the control beam. This signifies the important role of the transverse diaphragms in load distribution among the beams in the bridge model.

Strain in CFRP Reinforcement

The load versus strain in the CFRP reinforcement of the control beam is presented in Fig. 10. The strain responses up to the ultimate load of all the nonprestressing and prestressing CFRP tendons were bilinear, with a change in slope at the cracking load.

Load-Strain Response of Prestressing CFRP Tendons

Two of the prestressing tendons were located at the bottom layer, i.e., seventh layer, and the other prestressing tendon was located at the sixth layer. The rupture strain observed in the prestressing tendons in the seventh layer at the ultimate load was 1.57%, which is a little over the guaranteed ultimate strain of 1.49% as specified by the manufacturer (see Table 1). At the ultimate load of 127 kN, the tensile strain in the prestressing tendon at the sixth layer was 1.34%. Because of the rupture of the prestressing tendons in the bottom layer, the load dropped suddenly from 127 to 71 kN. At a load of 97 kN, the prestressing tendon in the sixth layer ruptured, and the recorded tensile strain was 1.51%, which was in close agreement with the manufacturer’s specifications (see Table 1).

Load-Strain Response of Nonprestressing CFRP Rods

The strain of the nonprestressing rods in the second layer was compressive at the beginning of the loading because the position of the neutral axis was below the level of the second layer of reinforcements. This reinforcement later experienced tensile strain while increasing the load, which is attributed to an upward shifting of the location of the neutral axis. The maximum tensile strain observed was 0.45% at the ultimate load. At the ultimate load, the developed tensile strain of the nonprestressing rods in the fourth and sixth layers were 0.53 and 0.70%, respectively. These experienced strains clearly indicate that the prestressing tendons only in the bottommost layer ruptured at the ultimate load. Even after the rupture of the prestressing tendons, the control beam could withstand the applied load with the help of the nonprestressing CFRP rods.

The load-strain response of the CFRP reinforcement in the bridge model is shown in Fig. 11. The prestressing tendon in the bottom...
Fig. 7. Ultimate load and residual strength test setups for the bridge model
layer of Beam B-3 experienced strain of 1.47%, including the initial prestress effects. However, because of the stiffening effect of the transverse diaphragms and loading points directly resting on the deck slab, the bridge model experienced local shear failure instead of controlled progressive failure. The maximum tensile strain experienced by the nonprestressing CFRP rods in the seventh layer of Beam B-3 was 1.38%, which is close to the guaranteed ultimate strain. The difference in strain of the prestressing tendons and nonprestressing rods in the first, second, sixth, and seventh layers at the beginning of the loading is a result of the effect of the prestressing force.

**Ductility of Control Beam Bridge Model**

Because the CFRP materials have a linear stress-strain relationship up to rupture, the conventional approach to evaluate the ductility of the steel-reinforced beams is inapplicable. Hence, the energy-based approach proposed by Grace et al. (1998) is used to quantify ductility of the CFRP-reinforced and prestressed structures. The ductility index determined from the ratio of absorbed inelastic energy to the total energy as

\[
\frac{E_{\text{inelastic}}}{E_{\text{total}}} = \frac{E_{\text{inelastic}}}{E_{\text{elastic}} + E_{\text{inelastic}} + E_{\text{inelastic, additional}}}
\]

where \( E_{\text{inelastic}} \) = inelastic energy absorbed; \( E_{\text{inelastic, additional}} \) = additional inelastic energy absorbed after the ultimate load; and \( E_{\text{elastic}} \) = elastic energy absorbed.

For the control beam, the estimated inelastic energy was 3,758 kN-mm, the elastic energy was 14,535 kN-mm, and the additional
inelastic energy was 20,559 kN-mm, as shown in Fig. 12. The total energy developed in the control beam was 38,852 kN-mm, which resulted in an energy ratio of 62.6%. It is observed that the progressive failure of the control beam caused significant inelastic energy to be absorbed. Although theoretically, the control beam falls into the brittle failure category, it still provided a sufficient and significant warning in terms of excessive cracking as in the case of well-known conventional prestressed steel-reinforced concrete beams and had a significantly large deflection of 502 mm at failure.

Similar to the control beam, the energy ratio is calculated based on the load-deflection curve of loaded Beam B-3 of the bridge model, as shown in Fig. 13. The inelastic energy experienced by the bridge model prior to the ultimate load was 31,543 kN-mm, and the elastic energy was 70,129 kN-mm. The additional inelastic energy after the postpeak response was 32,668 kN-mm. The total energy absorbed by the bridge model was 134,340 kN-mm, which resulted in an energy ratio of 48%.

Because all the CFRP tendons in the bridge model were not completely ruptured, as Beam B-3 only failed partially during the ultimate load test, a significant amount of additional inelastic energy of the remaining beams was available in the bridge model. To release this additional inelastic energy, a residual strength test was conducted on the bridge model, and the obtained additional inelastic energy was added to the energy ratio calculated from the ultimate load test.

The energy ratio of the bridge model developed from this residual strength test is presented in Fig. 14. The elastic energy for the residual strength test was 71,339 kN-mm. This energy was in close agreement with the elastic energy observed in the ultimate load test.
which was 70,129 kN-mm. Because no load cycles were conducted in the residual strength test, there was no inelastic energy measured in the bridge system. However, successive rupture of the prestressing CFRP tendons after the ultimate load created additional inelastic energy. The measured additional residual inelastic energy after postpeak response was 18,945 kN-mm. Thus, total energy experienced by the bridge model was 153,214 kN-mm, and the summation of inelastic energy, additional inelastic energy from the ultimate load test and the additional residual inelastic energy from the residual strength test, was 83,156 kN-mm. The calculated energy ratio for the bridge model from the ultimate load and residual strength tests was 54.3%.

Although the energy ratios of the control beam (62.6%) and the bridge model (54.3%) are different, the elastic energy ratio of the bridge model was similar to the energy ratio of the control beam. Hence, the postpeak behavior governed the energy ratios of the control beam and the bridge model, i.e., the ductility of the two models during failure.

**Strain in Extreme Concrete Fiber**

The longitudinal compressive strain of the extreme concrete fiber was monitored through the readings of the strain gauges installed at the top of the deck slab at midspan. The concrete strains were expected to be less than 0.003, as the control beam and the bridge model failures were governed by rupture of the prestressing CFRP tendons. The compressive strain in the control beam is shown in Fig. 15, and the maximum strain observed was 0.002 at the ultimate load and a strain of 0.0025 at complete collapse. The compressive strain in the middle three beams of the bridge model is shown in Fig. 16, and the maximum strain observed was 0.0017 at the ultimate load.

**Failure Mode of the Bridge Model**

For the bridge model, a flexural tension failure of central Beam B-3 in the form of rupture of the prestressing CFRP tendons was
experienced. It was followed by the punching shear of the deck slab at the centrally loaded beam, Beam B-3, and then failure of the transverse diaphragms. The diaphragms at the midspan and quarter-spans completely failed by crushing of the concrete and either rupture of the transverse CFRP rods or its debonding from the beams, i.e., failure of the epoxy grout. Although the bridge model was designed to achieve a flexural type of failure through rupturing of all the prestressing CFRP tendons, only the prestressing tendons in the bottom layer of loaded Beam B-3 were ruptured at the ultimate loading condition. This was a result of the higher stiffness contribution of the diaphragms through the arching/tension-stiffening action and use of a smaller loading area on the deck slab by the load-spreader. The ruptured prestressing CFRP tendons were expected to initiate progressive type of failure in the other tendons before complete structural collapse as experienced in the control beam failure.

**Effective Prestress Force**

A decompression test was conducted on the bridge model to determine the effective prestressing force level. The bridge model was loaded until the initiation of the first crack at the bottom surface of the beams at the midspan section. Strain gauges were attached at both sides of the first crack, and the bridge model was loaded again. The strain induced and load applied were monitored and recorded as shown in the load-strain curve (Fig. 17). The decompression load observed from the plot is 31.1 kN, which is used to derive the effective prestressing force level and the prestress losses of 6.2%. The prestress loss estimated in conventional stress-relieved tendon is approximately 15% (Naaman 2004). This result suggests that the use of CFRP tendons coupled with its developed anchorage systems proved to be an efficient system in producing less prestress losses compared with the conventional steel-reinforced/prestressed members.
Conclusions

The conclusions arrived at from this investigation are as follows:

1. The failure mode of the control beam was a sequential progressive rupture of the prestressing CFRP tendons in the bottom layers, followed by the rupture of nonprestressing CFRP rods. The sequence of failure of the bridge model consisted of rupture of the prestressing tendons, local punching shear failure of the deck slab, rupture of the CFRP rods in the transverse diaphragm, and separation of the loaded beam.

2. The failure modes of the control beam and the bridge model were governed by the rupture of the prestressing CFRP tendons.

3. The controlled progressive failure yielded a significant amount of inelastic energy released during postpeak response and resulted in a higher energy ratio of the control beam. After the ultimate load test and residual strength test, the bridge model experienced an energy ratio of 54.3%, which was less than the energy ratio (62.6%) of the control beam.

4. Although theoretically, the energy ratios of the control beam and the bridge model fall in the category of brittle failures, significant and substantial warning was experienced by extensive cracking before failure. The combined rupture of the prestressed and nonprestressing CFRP tendons yielded large deflection up to failure, whereas the concrete in the compression zone did not fail.

5. The effective prestress loss measured from the experiment in the tested bridge model was 6.2%. The use of the CFRP tendons coupled with their specialized anchorage systems proved to be an efficient system resulting in less prestress.

Fig. 16. Concrete compressive strain response at the midspan in longitudinal direction

Fig. 17. Load-strain response to estimate the decompression load
losses compared with the conventional steel prestressed members.

6. The theoretical ultimate load capacity of the control beam based on the unified design approach (Grace and Singh 2003) was 131 kN, whereas the experimentally obtained ultimate load was 127 kN. Hence, the unified design approach proposed by Grace and Singh (2003) is recommended for the design and analysis of the multilayered CFRP-reinforced and prestressed AASHTO beams.

7. The cracking load, ultimate load, deflection at ultimate load, and elastic energy of the bridge model were in agreement with the control beam. This suggests that testing a control beam for any large-scale bridge model will help in predicting the behavior of the bridge model.

Acknowledgments

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References


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