Concrete Beams Reinforced with CFRP

Investigating the shear performance of box beams strengthened with carbon fiber-reinforced polymers

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Fiber reinforced polymers (FRPs), particularly those incorporating carbon fibers (CFRPs), can be highly resistant to corrosion. Because they can also have high strength and high stiffness-to-weight ratios,[1] they show potential for use as reinforcing elements in concrete structures. FRP elements loaded in tension, however, tend to be linearly elastic up to fracture, and FRP elements loaded in shear can have low stiffness and strength. Concrete beams reinforced with FRP elements can therefore display brittle failure modes when the FRP elements are subjected to tension[2] or to dowel action.[3] The goal of the present investigation is to examine the shear responses of box beams with CFRP reinforcing. The shear cracking load and ultimate load-carrying capacity of the tested box beams are presented herein.

PREVIOUS FINDINGS

Yonekura, Tazawa, and Nakayama[4] determined that increasing the prestressing force is an effective way to increase the shear strength of FRP reinforced beams. Regarding prestressing tendons, Tottori and Wakui[5] concluded that the tendon type does not affect the shear capacity of beams. Fam et al.[6] observed that concrete beams with CFRP prestressing bars and stirrups failed in shear due to spalling of concrete cover at the bends in the stirrups. Further, they deduced that the maximum developed strength for a stirrup could be only 45% of its uniaxial tensile strength.

Using data from Yonekura, Tazawa, and Nakayama,[4] Dowden and Dolan[7] reasoned that the ACI 318[8] and AASHTO[9] code requirements are not conservative for evaluation of shear capacities of beams incorporating FRP tendons. Kato and Hyashida[10] showed that beams prestressed using bonded CFRP tendons failed in a brittle mode, while beams prestressed with unbonded CFRP tendons had roughly the same degree of ductility as beams reinforced with steel strands.

Investigations have also been conducted to evaluate the use of bonded and unbonded CFRP tendons in prestressed concrete bridges. Researchers developed and tested a system for multi-span concrete bridges comprising bonded, prestressed CFRP tendons with external, draped, and continuous post-tensioning tendons.[11-13] Most recently, Grace and Singh[14,15] have proposed an approach for the flexural design of concrete bridge girders with CFRP reinforcing comprising bonded pretensioned tendons, unbonded post-tensioned tendons, and bonded non-pretensioned rods arranged in multiple, vertically distributed layers.

The design approach[14] has been validated by comparing the analytical and experimental results obtained for a full-scale double-tee (DT) beam[12] similar to those used in the Bridge Street Bridge[13] as well as for three box beams.[16] While the DT beam failed by crushing of the concrete followed by rupture of bonded prestressing tendons,[12] all box beams failed due to rupture of bonded pretensioning tendons followed by crushing of the concrete.[16] Energy ratios[17] for box beams with post-tensioned tendons were found to be roughly 32%.

CONSTRUCTION DETAILS

In this study, a total of six box beams were evaluated for shear behavior. The construction details and testing methods were similar to those used for beams evaluated for flexural behavior.[16] However, the number of non-prestressing rods located near the bottom (Fig. 1) of the beams was increased from two to four to help eliminate flexural failure modes, the arrangement and types of the stirrups were varied as test parameters, and the loading points were shifted to the third points in the span to ensure shear failure of the beam.

Each box beam was 4.9 m (16 ft) long, 965 mm (38 in.) wide, and 305 mm (12 in.) deep, and was reinforced with pairs of steel stirrups at a uniform spacing of 75 mm (3 in.) in one shear span and with single steel stirrups at a uniform spacing of 230 mm (9 in.) through its midspan (Fig. 2). The remaining shear span was the test zone. One beam was a control beam, with no stirrups in the test zone. Another beam was reinforced with 9.5 mm (0.37 in.) diameter steel stirrups at a uniform spacing of 125 mm (5 in.) in the test zone. Using uniform spacings of 75 or 125 mm (3 or 5 in.) in the test zone, two beams were reinforced with 9.5-mm-diameter (0.37 in.) Type S_A stirrups, and two beams were reinforced with 10.5 mm (0.4 in.) Type S_B stirrups (Table 1 and 2).

All beams were reinforced longitudinally with a total of eleven, 9.5-mm-diameter (0.37 in.) non-prestressing, Type T_A rods (Fig. 2). Four of these rods were located in the bottom flange and seven were provided at the top flange of the beam (Fig. 1). Seven bonded pretensioning

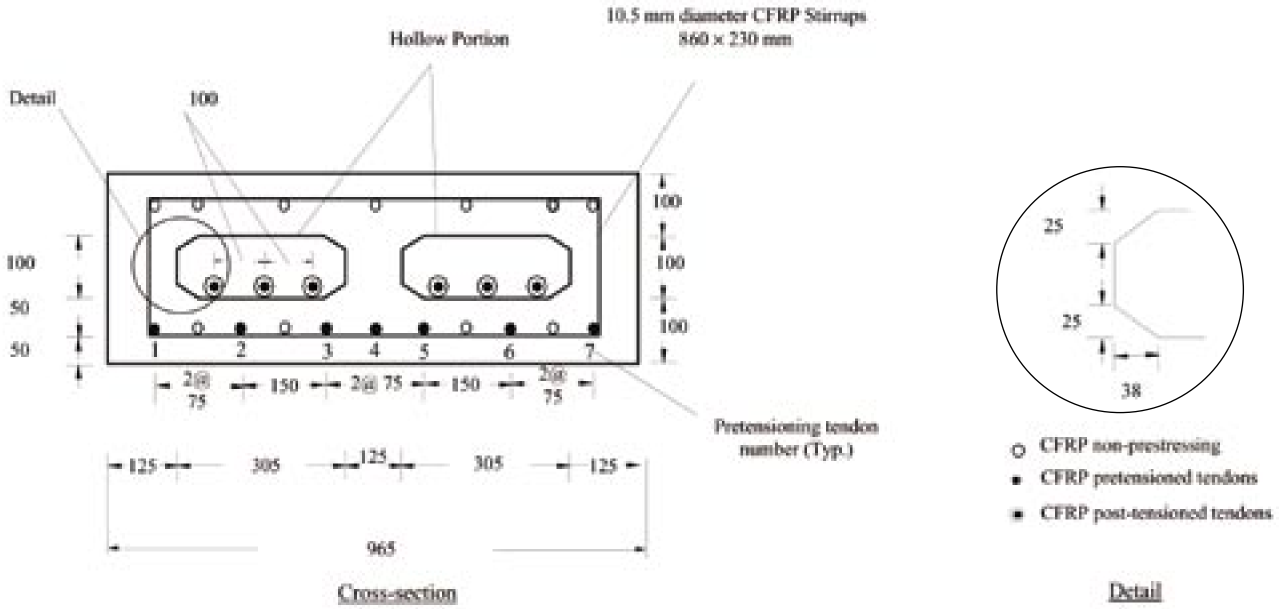

Fig. 1: Typical section through box beams evaluated for shear capacity. Note: 1 in. = 25.4 mm

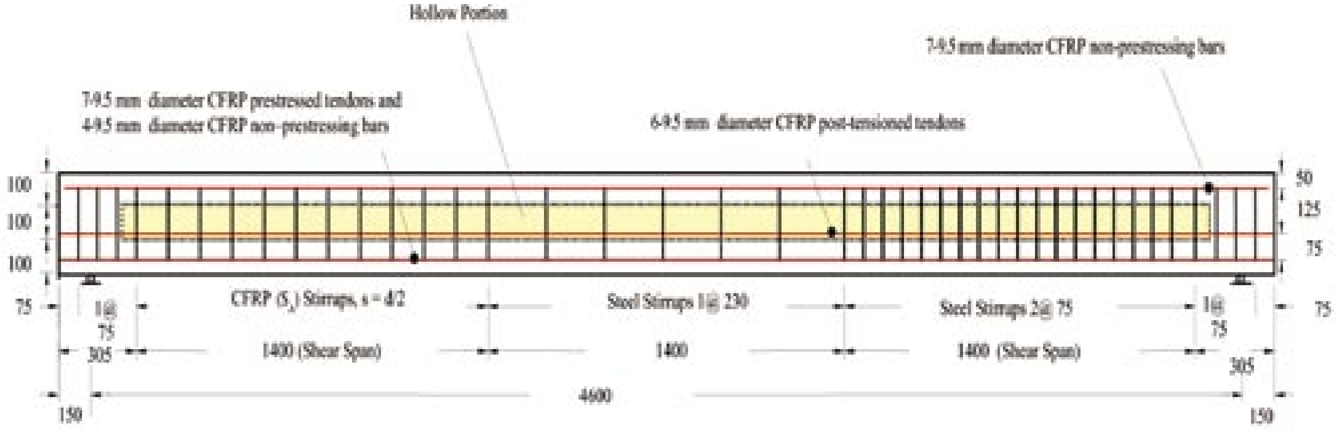

Fig. 2: Reinforcement details of box beam M2 with Type S_A shear stirrups. Note: 1 in. = 25.4 mm

tendons were installed in the bottom flange and six unbonded post-tensioning tendons were installed in the hollow portions of the box beam (Fig. 1 and 2). All prestressing rods were 9.5-mm-diameter (0.37 in.) Type T_A tendons (Table 1) tensioned to a force of 92.6 kN (20.8 kip). Load cells at the dead end of the beams monitored the forces in the prestressing tendons. Concrete with specified 28-day compressive strength of 50 MPa (7 ksi) was placed immediately after tensioning of the prestressing tendons. Seven days after the concrete placement, the bonded tendons were released (by cutting with a saw) and the post-tensioning tendons were stressed.

INSTRUMENTATION

Strain gauges were installed on the stirrups before the placement of concrete. In addition, 15 strain gauges (five gauges on the top surface and five gauges on each side surface) were installed on the midspan concrete surface to monitor the concrete strain distribution. Demountable mechanical strain gauge (DEMEC) point rosettes were located on each side of the test beam in the shear critical zones. They were used to monitor the propagation of shear cracks by measuring horizontal, diagonal, and vertical strains at each station.

Beam deflection was measured using string pots attached at midspan and at the quarterspan points. Sensors were connected to a data acquisition system to monitor the readings throughout the tests.

TESTING SET-UP

All beams were simply supported with a roller at one end and a hinged support at the other end of their 4.6 m (15 ft) span. An 890 kN (200 kip) jack and a hydraulic

TABLE 1:

PROPERTIES OF CFRP AND STEEL STIRRUPS, BARS, AND TENDONS

	Tendons	Stirrups		
	T_A[18]	S_A[20]	S_B[21]	Steel
Nominal diameter *d*, mm (in.)	9.5 (0.37)	9.5 (0.37)	10.5 (0.4)	9.5(0.37)
Cross sectional area, mm^2 (in.^{2})	71 (0.11)	71 (0.11)	55.7* (0.09)	9.5 (0.37)
Guaranteed strength, MPa (ksi)	1524 (220)	1580 (230)	1867 (270)	414 (60†)
Ultimate tensile strength, MPa (ksi)	1930 (280)	1896 (275)	2103 (305)	414 (60†)
Elastic modulus,GPa (ksi)	131 (19,000)	110 (16,000)	137 (19,900)	200 (29,000)
Maximum elongation, %	1.47	1.7	1.5	0.2‡

Note: *Effective cross-sectional area, †Yield strength; ‡Yield strain.

TABLE 2:

DETAILS OF CRACKING AND FAILURE LOADS, STIRRUP STRAINS, AND ENERGY RATIOS FOR THE BOX BEAMS TESTED FOR SHEAR

Beam notation	Stirrup type	Spacing,* mm (in.)	Shear cracking force, kN (kip)	Angle of major crack, deg.(θ)	Failure load, kN (kip)	Average stirrup strain at failure, %	Energy ratio, %
M2	S_A[20]	125 (5)	111 (25)	45	257 (58)	0.40	24
M3		75 (3)	156 (35)	47	267 (60)	0.30	23
T2	S_B[21]	125 (5)	111 (25)	46	226 (51)	0.38	17
T3		75 (3)	156 (35)	45	291 (66)	0.25	17
S2	STEEL	125 (5)	133 (30)	45	223 (50)	0.19	25
No	—	—	80 (20)	46	177 (40)	—	15

*Stirrup spacing corresponds to *d*/2 or *d*/3, where *d* is the distance from the extreme compression fiber to the centroid of the bonded prestressing rods.

pump were used to apply load through a four-point loading system. The longitudinal distance between the two pairs of loading points was 1400 mm (56 in.). The beams were first loaded to a load slightly less than the cracking load, and then unloaded. Five more loading and unloading cycles were applied to the beams before they were loaded to failure. DEMEC rosette readings were taken at 90 kN (20 kip) increments and the measured gauge data were later used to compute the shear crack width.

SHEAR RESPONSE

Table 2 presents shear strengths, average stirrup strains, shear cracking loads, and energy ratios for the beams. The energy ratios represent the ratio of the total inelastic energy absorbed in the box beam to the total energy (elastic energy plus inelastic energy) and are determined from the load-deflection curves using loading and unloading cycles.[17] Although the control beam (N0 with no shear stirrups) had the lowest ratio compared with the beams evaluated for flexural failure,[16] all shear-critical beams had low energy ratios, including Beam S2 (with steel stirrups). It is also noted that the energy ratio was apparently not affected by the stirrup spacing.

Stirrups with greater center-to-center spacing experienced higher strains at beam failure. The maximum strains developed in the CFRP stirrups of Beams M2, M3, T2, and T3 are 0.004, 0.003, 0.0038, and 0.0025 (23.5%, 17.6%, 25%, and 16.7% of the corresponding specified ultimate strain of the stirrups), respectively, whereas steel stirrups of Beam S2 experienced strain of 0.0019 (95% of the yield strain). These strains are lower than strains reported by Fam et al.[6] and confirm that the strength of CFRP stirrups must be evaluated using realistic tests.

Shear cracks were inclined at 45 to 47 degrees relative to the longitudinal axis of the beams, and each beam failed in shear in the designated test zone. It is interesting to note that the shear cracking force for the beam with steel stirrups was significantly higher than the cracking forces for beams with CFRP stirrups at the same spacing. The shear failure of each box beam was accompanied by widening of the shear cracks, followed by rupture of the pretensioning tendons due to the dowel action. The ultimate loads measured for Beams M2, M3, T2, T3, S2, and N0 were 257, 267, 226, 291, 223, and 177 kN (57.8, 60.0, 50.9, 65.5, 50.1, and 40 kip), respectively.

The research team observed that the unbonded post-tensioning tendons of the box beams remained intact even after beam failure. It should be noted, however, that the tendons were free to move within the beam core. Therefore, it is unlikely that the tendons' strains increased significantly during beam loading. Figure 3 shows the shear failure of Beam M2 with Type S_A stirrups.

Fig. 3: Shear failure of box beam M2 with S_A stirrups

SHEAR STRENGTH INCREASED

All box beams tested for shear failed due to widening of shear cracks followed by the rupture of pretensioning tendons due to the dowel action and rupture of CFRP stirrups. The unbonded post-tensioning tendons, however did not rupture even after the failure of box beams. The shear strength of box beams reinforced with CFRP stirrups was slightly higher than that reinforced with steel stirrups due to higher values of induced effective stresses in the CFRP stirrups. It is important to note, however, that shear cracking occurred at higher loads with the steel stirrups.

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References

1. ACI Committee 440, "State-of-the-Art Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures (ACI 440R-96)," American Concrete Institute, Farmington Hills, MI, 1996, 65 pp.

2. Naaman, A. E., and Jeong, S. M., "Structural Ductility of Concrete Beams Prestressed with FRP Tendons," *Non-Metallic (FRP) Reinforcement for Concrete Structures*, Proceedings of the Second International RILEM Symposium (FRPRCS), Ghent, Belgium, Aug. 23-25, 1995, pp. 379-386.

3. Naaman, A. E., and Park, S. Y., “Shear Behavior of Beams Prestressed with CFRP Tendons: Preliminary Test Evaluations,” *Proceedings of the Third International Symposium on Non-Metallic (FRP) Reinforcement for Concrete Structures*, V. 2, Sapporo, Japan, Oct. 14-16, 1997, pp. 679-686.

4. Yonekura, A.; Tazawa, E.; and Nakayama, H., “Flexural and Shear Behavior of Prestressed Concrete Beams Using FRP Rods as Prestressing Tendons,” *Fiber Reinforced Plastic Reinforcement for Concrete Structures,* SP-138, A. Nanni and C. W. Dolan, eds., American Concrete Institute, Farmington Hills, MI, 1993, pp. 525-548.

5. Tottori, S., and Wakui, H., “Shear Reinforcement of RC and PC Beams Using FRP,” *Fiber-Reinforced-Plastic Reinforcement for Concrete Structures—International Symposium,* SP-138, A. Nanni and C. W. Dolan, eds., American Concrete Institute, Farmington Hills, MI, 1993, pp. 615-631.

6. Fam, A. Z.; Abdelrahaman, A.; Rizkalla, S. H.; and Saltzberg, W., “FRP Flexural and Shear Reinforcements for Highway Bridges in Manitoba,” *Proceedings of the Second International RILEM Symposium* (FRPRCS-2), Ghent, Belgium, Aug. 23-25, 1995, pp. 395-402.

7. Dowden, D. M., and Dolan, C. W., “Comparison of Experimental Shear Data with Code Predictions for FRP Prestressed Beams,” *Proceedings of the Third International Symposium on Non-Metallic (FRP) Reinforcement for Concrete Structures*, V. 2, Sapporo, Japan, Oct. 14-16, 1997, pp. 687-694.

8. ACI Committee 318, “Building Code Requirements for Structural Concrete (318-02) and Commentary (318R-02),” American Concrete Institute, Farmington Hills, MI, 2002, 443 pp.

9. *LRFD Bridge Design Specifications*, American Association of State Highway and Bridge Officials (AASHTO), 1998.

10. Kato, T., and Hayashida, N., “Flexural Characteristics of Prestressed Concrete Beams with CFRP Tendons,” *Fiber Reinforced Plastic Reinforcement for Concrete Structures,* SP-138, A. Nanni and C. W. Dolan, eds., American Concrete Institute, Farmington Hills, MI, 1993, pp. 419-440.

11. Grace, N. F.; Enomoto, T.; and Yagi, K., “Behavior of CFCC and CFRP Leadline™ Prestressing Systems in Bridge Construction,” *PCI Journal*, V. 47, No. 3, May-June 2002, pp. 90-103.

12. Grace, N. F.; Enomoto, T.; Abdel-Sayed, G.; Yagi, K.; and Collavino, L., “Full-Scale Evaluation of CFRP/CFCC Prestressed DT-Beam: Experimental Study and Analysis,” *PCI Journal*, V. 48, No. 4, Jul.-Aug. 2003, pp. 120-139.

13. Grace, N. F.; Navarre, F.; Nacey, R. B.; Bonus, W.; and Collavino, L., “Design-Construction of Bridge Street Bridge—First CFRP Bridge in the United States,” *PCI Journal,* V. 47, No. 5, Sept.-Oct. 2002, pp. 20-35.

14. Grace, N. F., and Singh, S. B., “Design Approach for CFRP Prestressed Concrete Bridge Beams,” *ACI Structural Journal*, V. 100, No. 3, May-June 2003, pp. 365-376.

15. Grace, N. F., and Singh, S. B., “Unified Analysis and Design Approach for CFRP Prestressed Concrete Bridge Girders,” *Proceedings of the 10th U.S.-Japan Conference on Composite Materials*, Stanford, CA, Sept. 16-18, 2002, pp. 328-335.

16. Grace, N. F.; Singh, S. B.; Shinouda, M. M.; and Mathew, S. S., “Flexural Response of CFRP Prestressed Concrete Box Beams for Highway Bridges,” *PCI Journal*, V. 49, No. 1, Jan.-Feb. 2004, pp. 92-103.

17. Grace, N. F., “Response of Continuous CFRP Prestressed Concrete Bridges under Static and Repeated Loadings,” *PCI Journal*, V. 45, No. 6, Nov.-Dec. 2000, pp. 84-102.

18. Diversified Composites, Inc., “Product Manual of Diversified Composites, Inc.,” 2002.

19. Mitsubishi Chemical Corp., “Leadline™ Fiber Tendons/Bars,” Product Manual, Tokyo, Japan, 1994, 14 pp.

20. Marshall Industries Composites, Inc., “Product Manual of Marshall Industries Composites, Inc.,” 2002.

21. Tokyo Rope Mfg. Co., Ltd., “Technical Data on CFCC Strands,” Product Manual, Tokyo, Japan, 1993, 100 pp.

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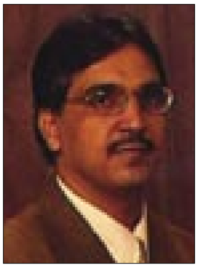

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