

SYNTHETIC AND OTHER NON-METALLIC FIBER REINFORCEMENT OF CONCRETE



Compilation 28

American Concrete Institute

Synthetic and Other Non-Metallic Fiber Reinforcement of Concrete

ACI Compilation 28

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The Material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

James I. Daniel
Chairman, ACI Committee 544
Fiber Reinforced Concrete

On The Cover: When this residence, made of fiber reinforced concrete was built, a major goal was to maximize the lakefront view. The rear wall of the structure was constructed of both precast and site-cast post-tensioned concrete panels and contains 880 ft² of windows. The house was designed so that the living room, dining room, and bedrooms have a view of the water. All panels, I-beams, bar joists, etc., were erected at one time from one location. (See story on p. 74.)



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Flexural and Shear Behavior of Concrete Beams Reinforced with 3-D Continuous Carbon Fiber Fabric

by Paul Zia, Shuaib H. Ahmad, Rakesh K. Garg, and Kristina Hanes

Corrosion of steel reinforcement is a major cause of deterioration of reinforced concrete structures. There is an increasing interest in the use of non-metallic high performance materials as alternative reinforcement for concrete structures. Many different types of fibers have been studied and used as reinforcement for cement and mortar composites and a few have been used for concrete.^(6,7,11)

Most of the research on fiber reinforced concrete reported in the literature has been concerned with randomly dispersed short fibers in one, two, or three dimensions.^(3,5) Some investigations involving one or two-dimensional contin-

uous fibers have also been reported. Only a few studies have been conducted on the use of three-dimensional continuous fiber fabric as reinforcement for cement mortar.^(1,2,4,10) This paper presents the results of a study, examining the flexural and shear behavior of concrete beams reinforced with three-dimensional continuous carbon fiber fabric.^(8,9)

3-D continuous carbon fiber fabric

The 3-D continuous carbon fiber fabric used in this study is a product developed recently in Japan. It is made from Polyacrylonitrile-type continuous fibers

(PAN-type) having a diameter of 275.59×10^{-6} in. (7×10^{-6} m), a specific gravity of 1.79, tensile strength of 497.37 ksi (3430 MPa), elastic modulus of 34,105 ksi (235,200 MPa), and 1.5 percent elongation. The fabric is constructed from the fibers by three-dimensional weaving, with each element encased in epoxy coating, thus forming a rigid fabric (Fig. 1). Each element of the fabric contains 48,000 fibers with a total cross sectional area of 0.00286 in.² (1.85 mm²). The nominal cross sectional area of the element including the epoxy coating is 0.0065 in.² (4.19 mm²).

To determine the stress-strain relationship of the fabric, an 8-ft long element consisting of 48,000 individual fibers was cut from the fabric from which three 16-in. long specimens were prepared for tension test. As shown in Fig. 2, an epoxy block of square prism ($\frac{1}{4} \times \frac{1}{4} \times 2\frac{1}{2}$ in. [19 x 19 x 64 mm]) was cast at each end of a specimen so that the specimen could be gripped by a universal testing machine. Between the two epoxy blocks, two small plexiglass disks were glued by epoxy 8 in. apart on the specimen.

An extensometer of 8-in. gauge length was attached to the plexiglass disks to measure the extension of the specimen in tension test as shown in Fig. 3. At failure, the specimens all fractured within the 8-in. gauge length. There was no evidence that the joints formed from the transverse elements through weaving had any adverse effect on the tests.

The test results are shown as the stress-strain curves for the three specimens in Fig. 4. The stress is computed on the basis of the nominal area of the fabric element which includes the epoxy coating rather than the net area of the carbon fibers. It is noted that, for the second specimen, the stress-strain curve became non-linear when some of the fibers broke at the stress level of about 100,000 psi (690 MPa). The average value of the *apparent* modulus of elasticity of the three specimens is about

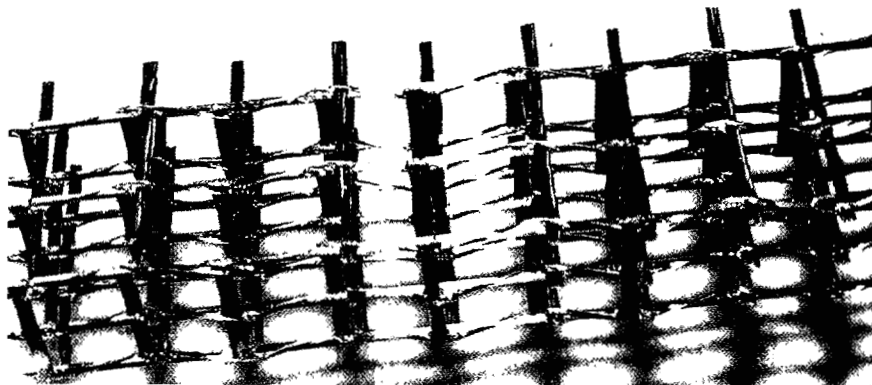


Fig. 1 — 3-D continuous carbon fiber fabric reinforcement.

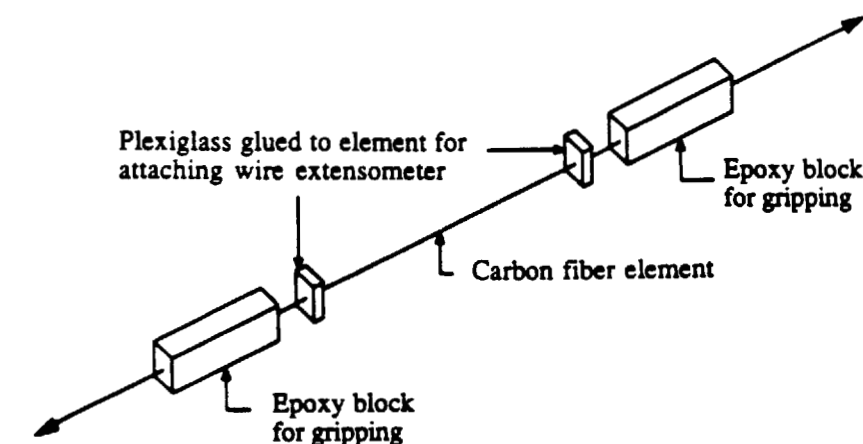


Fig. 2 — A tensile specimen of fiber fabric element.

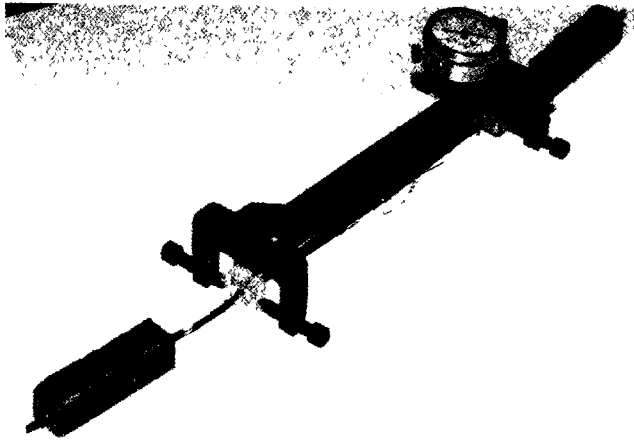


Fig. 3 — Extensometer and tensile specimen after failure.

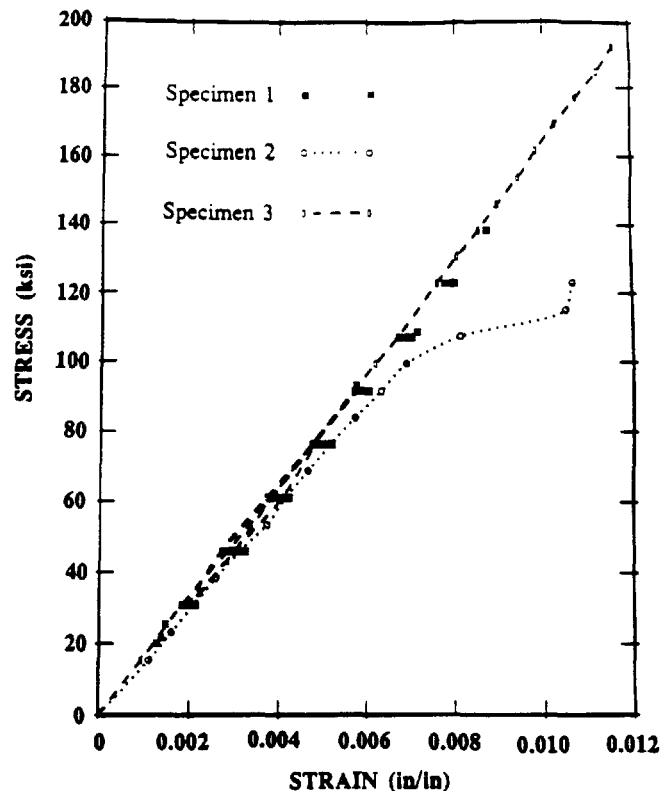


Fig. 4 — Stress-strain curves of fabric element.

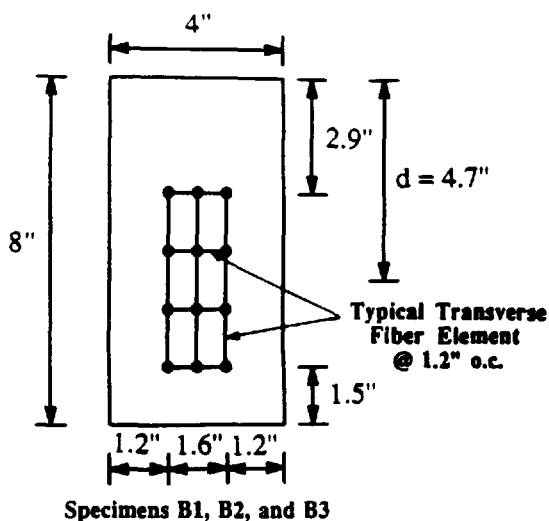


Fig. 5a — Details of carbon fiber fabric reinforced beam.

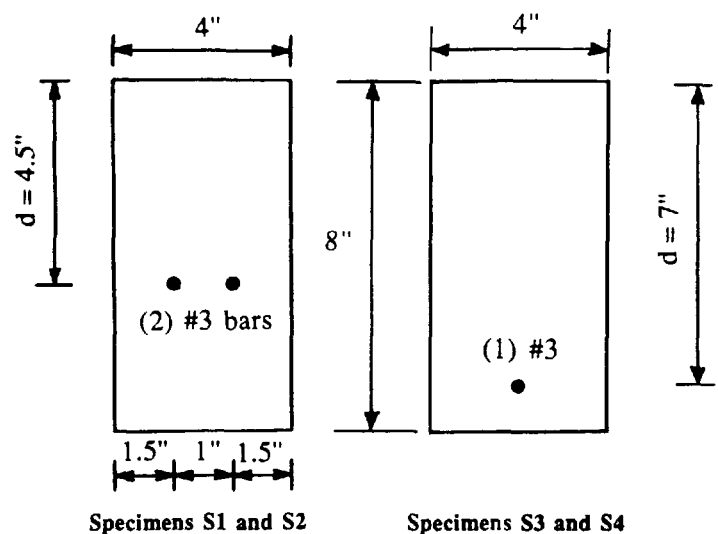


Fig. 5b — Details of steel reinforced beam.

16,400 ksi (113,000 MPa) with their fracturing strain varying between 0.008 and 0.0118.

Flexural tests

Three concrete beams were cast using the 3-D fabric as reinforcement. In addition, four beams were cast with the conventional #3 bars. The details of the test specimens are shown in Fig. 5 and the loading arrangement is shown in

Fig. 6. The three sets of beams were designed to have a comparable bending strength and they were fabricated in separate castings. It is noted that the center of gravity of the reinforcement in the fiber reinforced beams is located nearly at the same position as in beams S1 and S2. The compressive strength of concrete for the various specimens is shown in Table 1. The carbon fiber reinforced beams were tested at 28 days while the

steel reinforced beams were tested at 3 days (S1 and S2) and 7 days (S3 and S4). Since all of the beams were under-reinforced, the effect of the variations in concrete compressive strength on their flexural capacities would not be significant.

The load-deflection curves shown in Fig. 7 indicate that the cracking load for the carbon fiber reinforced beams was slightly over 500 lb (2,224 kN). On the

Table 1 — Compressive strength of concrete

Specimen No.	Age at testing (days)	Compressive strength at testing (psi)
CFRC-B1	28	2350
CFRC-B2	28	2820
CFRC-B3	28	2950
S1	3	2880
S2	3	3000
S3	7	4930
S4	7	4680

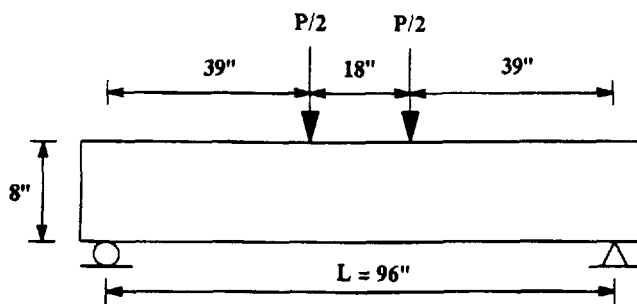


Fig. 6 — Loading arrangement for flexural test.

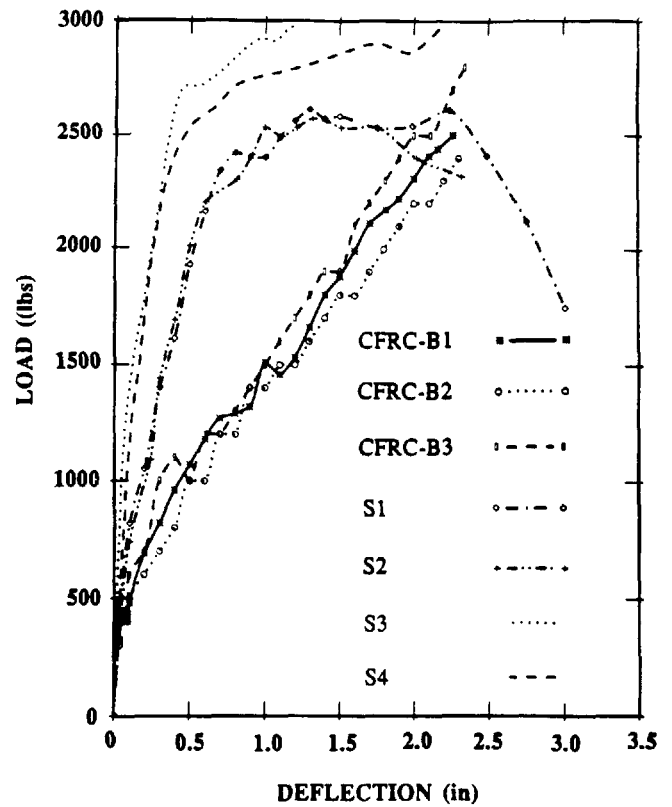


Fig. 7 — Load-deflection curves

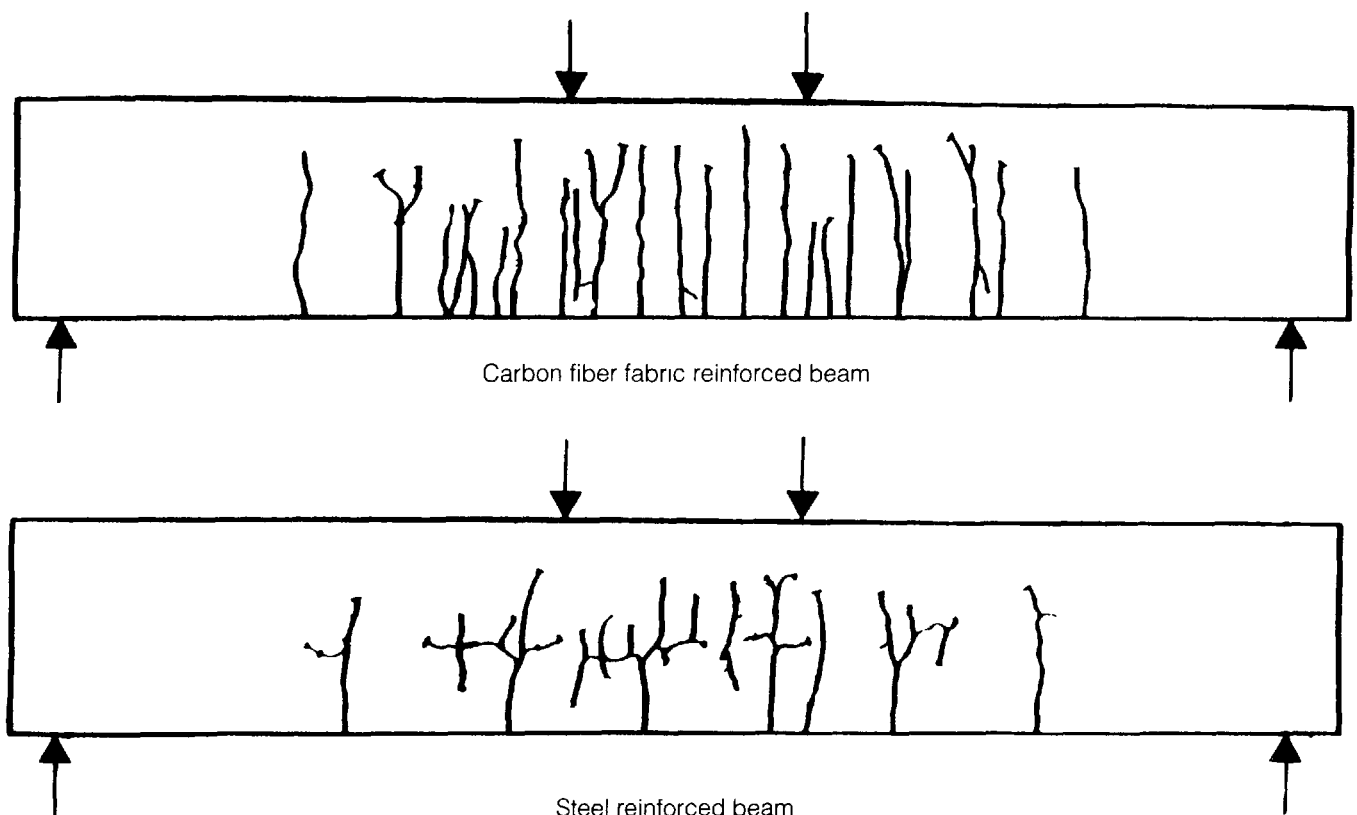


Fig. 8 — Crack pattern of test specimens.

other hand, the cracking load for beams S1 and S2 was about 800 lb (3.558 kN) and it was about 1100 lb (4.893 kN) for beams S3 and S4. These differences reflect the variations in concrete strength. However, all three sets of beams achieved comparable load carrying capacity as designed. It is significant to note that, after initial cracking and under increasing loading, the carbon fiber reinforced beams developed many closely spaced vertical small cracks since the small size reinforcements were well distributed, whereas the steel reinforced beams developed fewer widely spaced vertical as well as inclined large cracks, see Fig. 8. The carbon fiber reinforced beams also remained elastic whereas the steel reinforced beams were elastic only until the steel reached its yield strength.

Since the total gross area of the twelve longitudinal carbon fiber elements (0.078 in.² [50.32 mm²]) is only 71 percent of the area of one #3 steel bar (0.11 in.² [70.97 mm²]), and the apparent elastic modulus of the carbon fiber element is about 57 percent of that of the steel, the transformed area of the carbon fiber reinforcement (0.43 in.² [277.42 mm²]) is only 21 percent of that of the steel reinforcement in S1 and S2 (2.06 in.² [1329.03 mm²]), and 53 percent of that of the steel reinforcement in S3 and S4 (0.81 in.² [522.58 mm²]). Therefore, the moment of inertia of cracked section for the carbon fiber reinforced beam (7.18 in.⁴ [298.85 cm⁴]) is only 32 percent of that of S1 and S2 (22.74 in.⁴ [946.51 cm⁴]) and 25 percent of that of S3 and S4 (28.93 in.⁴ [1204.16 cm⁴]). Accordingly, the post-cracking deflections of the carbon fiber reinforced beams were proportionally larger than those of the steel reinforced beams. At the ultimate load, the carbon fiber elements fractured successively from the lowest layer upward whereas the steel bars were yielding.

Shear tests

To maximize the benefit of this study, flexural shear tests were conducted by

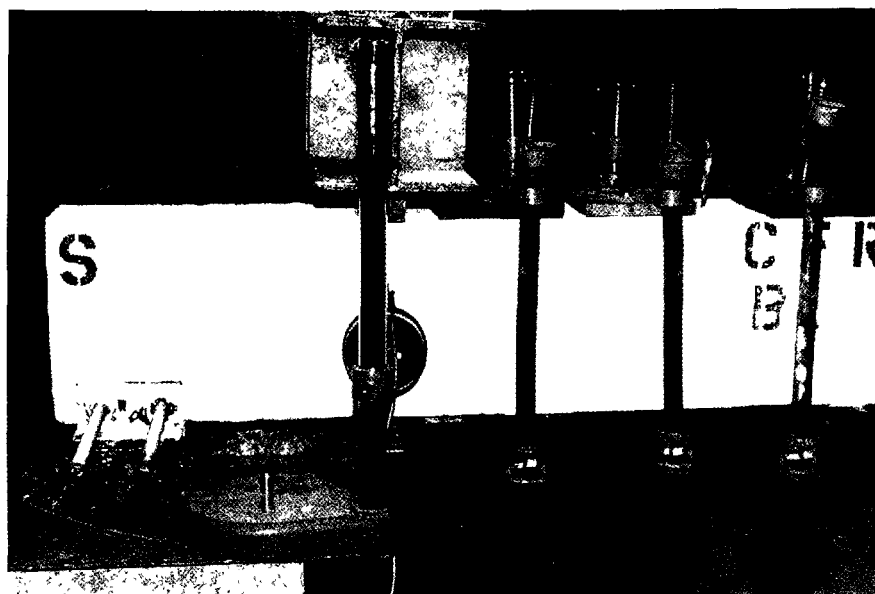


Fig. 9 — Setup for shear test.

Table 2 — Results of shear tests.

Specimen No.	a/d	V_u (kips)	v_u (psi)	$v_u/(f'_c)^{1/2}$	M_f (in-k)	M_u (in-k)	Tension in tie (kips)
<i>Carbon Fiber Fabric Reinforced Beams</i>							
B1-A	2.13	4.81	256	5.28	48.1	48.75	11.36
B1-B	2.13	5.08	270	5.57	50.8	48.75	12.00
B2-A	3.62	3.96	211	3.97	67.3	46.80	15.90
B2-B	3.62	3.49	186	3.50	59.3	46.80	14.00
B3-A	2.55	5.74	305	5.62	68.9	54.60	16.28
B3-B	2.55	5.94	316	5.82	71.3	54.60	16.84
<i>Steel Reinforced Beams</i>							
S3-A	2.00	2.50	89.3	1.27	35.0	62.4	5.56
S3-B	2.00	2.25	80.4	1.15	31.5	62.4	5.00
S4-A	2.29	2.25	80.4	1.18	36.0	60.8	5.71
S4-B	2.29	2.00	71.4	1.04	32.0	60.8	5.08

a/d = Shear span/depth ratio; b = Beam width; d = Effective depth; V_u = Ultimate shear force; v_u = Nominal ultimate shear stress = V_u/bd ; M_f = Moment at failure; M_u = Moment capacity of beam section

making use of the two end portions of each of the three fractured carbon fiber reinforced beams to examine how the 3-D fabric reinforcement would affect the shear strength of the specimens. A total of six shear tests were performed. For the portion of the beam where flexural cracks had developed under previous flexural tests, vertical external bar clamps were used to strengthen the beam before a single point test load was applied (Fig. 9). In all cases, no shear failure developed. Instead, failure was due to flexure with a major crack being developed directly below the load point and followed by fracture of the fabric reinforcement. It clearly demonstrated the effectiveness of the closely-spaced vertical carbon fiber elements as shear reinforcement.

Similar tests were also performed on the end portions of steel reinforced beams S3 and S4. These tests resulted in typical shear failure as expected. The results of the shear tests are summarized in Table 2.

For each of the carbon fiber fabric reinforced beams the nominal ultimate shear stress is considerably higher than that for each of the steel reinforced beams. One can also observe that the bending moment carried by each individual carbon fiber fabric reinforced beam at failure is considerably more than its moment capacity (except for specimen B1-A) which appears to be an anomaly. However, if one analyzes the beam resistance by the strut-tie model, one finds that the tension carried by the fiber fabric reinforcement as a tie ranges

from 11.36 to 16.84 kip (50.53 to 74.90 kN), which compares reasonably well with the potential tensile capacity of 17.07 kip (75.93 kN) for the fiber fabric reinforcement based on the tensile strength of the PAN-type fiber.

For the steel reinforced beams, shear failure due to diagonal tension and loss of anchorage and bond occurred when the moment carried by the beams was only about one-half of the moment capacity of the beams. Based on the strut-tie model, the tension carried by the #3 steel bar ranged from 5.56 to 5.71 kip (24.73 to 25.40 kN), which also compares quite well with the tensile capacity of 6.6 kip (29.36 kN) for the steel bar.

Conclusions

Based on the results of this pilot study, several conclusions can be drawn:

1. The 3-D carbon fiber fabric reinforcement, being made by weaving together many closely-spaced small elements, provided excellent bond and anchorage to concrete.

2. In the post-cracking stage, beams reinforced with such a fabric showed many closely-spaced small vertical cracks whereas beams reinforced with conventional steel bars developed fewer widely spaced vertical and inclined large cracks.

3. Since the *apparent* modulus of elasticity of the fabric element was lower than that of steel, and the reinforcement ratio of the fabric reinforced beam was also much smaller than that of the steel reinforced beam, the post-cracking deflection of the fabric reinforced beam was much larger than that of the steel reinforced beam.

4. Since the carbon fiber remained elastic up to failure, the beam reinforced with the fiber fabric also behaved elastically and failed abruptly in a brittle manner. The steel reinforced beam remained elastic only until the steel began to yield but failed with more ductility. However, both types of beam developed the same ultimate strength as designed and reached the same deflection at failure.

5. The closely-spaced vertical carbon fiber elements served very effectively as shear reinforcement to prevent shear failure even when the beams were loaded with shear span to depth ratio from 2.1 to 3.6.

Acknowledgments

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Punching Shear Tests of Slabs Reinforced with 3-D Carbon Fiber Fabric

by S.H. Ahmad, P. Zia, T.J. Yu, and Y. Xie

Use of non-metallic reinforcement is increasingly becoming an attractive alternative to the conventional mild steel bars for use in reinforced concrete applications. There is very little information on behavior of reinforced concrete members utilizing carbon fiber reinforcement, and practically no information on use of three-dimensional carbon fiber fabric in structural concrete.

Introduction

A major cause of the deterioration of concrete structures is the corrosion of steel reinforcing bars in concrete in aggressive environments; especially in the environment of underground, coastal and offshore structures, bridge decks, and areas of serious pollution. The traditional approach to overcoming this problem is to use larger covers, and relatively higher cement contents, to limit crack widths, and to use epoxy-coated steel reinforcing bars. However, the traditional systems have not shown the expected results, and costs to repair bridges in the United States currently are assessed at 20 billion dollars.¹

There is an increasing interest in the use of non-metallic, high-performance materials as alternative rein-

forcement for concrete structures.² Many different types of fibers have been studied and used as reinforcement for cement and mortar composites and only a few have been used for concrete.^{2,3-7}

Most of the research on the use of non-metallic fibers for reinforcement has been concerned with randomly dispersed short fibers in one, two, or three dimensions.⁸⁻⁹ A recent study reported the use of continuous fibers for reinforcing concrete.¹⁰ Other studies have reported the use of three-dimensional continuous fiber fabric as reinforcement for cement mortar,¹¹⁻¹⁴ and recently a study reported the use of three-dimensional continuous fiber fabric as reinforcement for beams.¹⁵

The problem of punching shear strength in slabs has received added emphasis due to its importance in connection with flat-slab floor systems and certain types of bridge deck slabs. This paper presents the results of a pilot study, examining the punching shear behavior of concrete slabs reinforced with three-dimensional continuous carbon fiber grids. The results are compared with slabs reinforced with conventional mild reinforcing steel bars.

3-D continuous carbon fiber fabric

The 3-D continuous carbon fiber fabric used in this study is a product developed recently in Japan. It is made of polyacrylo-

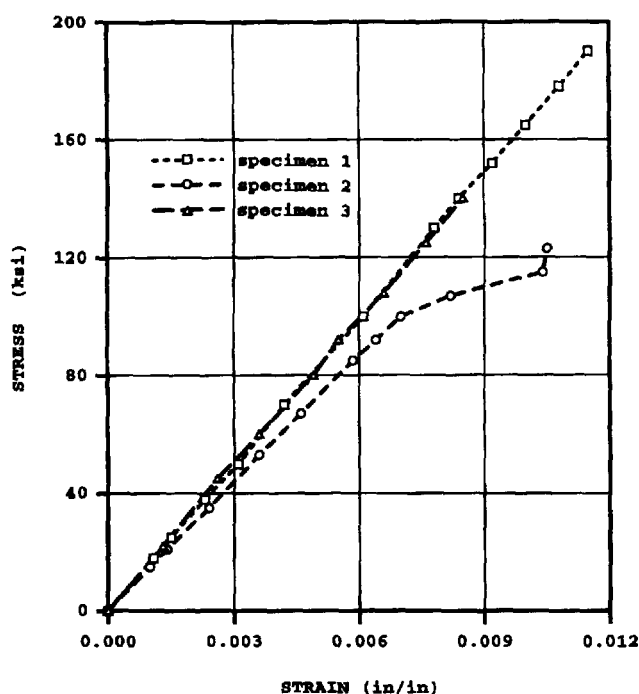


Fig. 1 — Stress-strain curve of the 3-D carbon fiber fabric element.

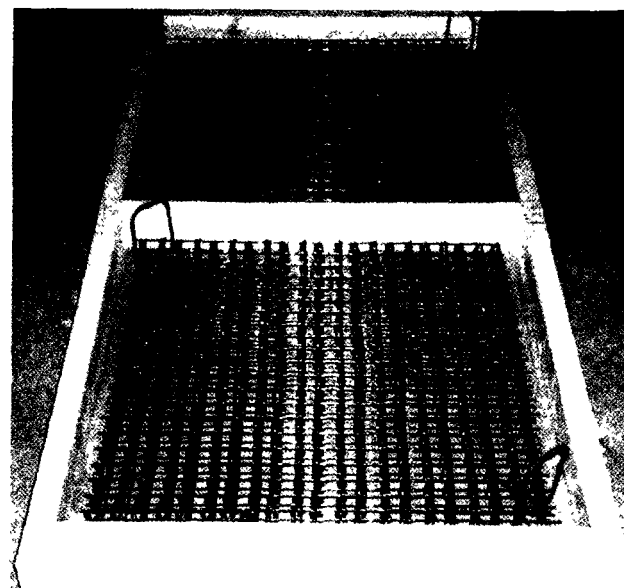


Fig. 2 — Three dimensional carbon fiber fabric reinforcement for CFRC slabs.

Table 1 — Test program for punching shear tests of slabs reinforced with either 3-D carbon fiber fabric or reinforcing bars.

Specimen type	Dimension $l_1 \times l_2 \times h$ (in.)	Effective depth (d) (d_1, d_2, d_3) (in.)	Area of one element x no. of elements x row of elements or bar details	Reinforcement ratio ρ (A_s/l_d) (%)	Strength of carbon fiber or steel (ksi)	Size of load head or stub c (in.)	Remarks
CFRC-S-N-1	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3*	0.95	195	3x3	CFRC without column stub
CFRC-S-N-2	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3*	0.95	195	3x3	CFRC without column stub
CFRC-S-3	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3*	0.95	195	4x4	CFRC slab without column stub
CFRC-S-4	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3*	0.95	195	4x4	CFRC slab without column stub
RC-S-1	27x27x3	2.4	7#3**	1.18	60	4x4	RC slab with column stub
RC-S-2	27x27x3	2.4	8#3**	1.35	60	4x4	RC slab with column stub

* Reinforcement in x, y, and z direction
 ** Reinforcement in x and y direction
 d_1 = effective depth of carbon strands in layer 1
 d_2 = effective depth of carbon strands in layer 2
 d_3 = effective depth of carbon strands in layer 3

nitrile-type continuous fibers (PAN-type) having a diameter of 275.59×10^{-6} in. (7000.0×10^{-6} m), a specific gravity of 1.79, tensile strength of 497.57 ksi (3431.7 MPa), elastic modulus of 34,105 ksi (235,210 MPa) and 1.5 percent elongation. The fabric is constructed from the fibers by three-dimensionally weaving each epoxy-coated element to form a rigid fabric. Each element of the fabric contains 48,000 fibers with a total cross-sectional area of 0.00286 in^2 (1.85 mm^2). The nominal cross-sectional area of the element including the epoxy coating is 0.0065 in^2 (4.2 mm^2).

The tensile stress-strain curves of the carbon fabric element are shown in Fig. 1. Details of the specimen size and tension testing are given in a recent paper by Zia, Ahmad, Garg and Hanes.¹⁵ The stress referred to in Fig. 2 is computed on the basis of the nominal area of the fabric element, including the epoxy coating.

The average value for the apparent modulus of elasticity of the three specimens is about 16,400 ksi (113,000 MPa) with a fracture strain varying between 8,000 and 11,800 microstrains. Premature failure due to slipping of the end grips occurred for specimen 3, and therefore the observed ultimate strength was smaller than the other two specimens.

Experimental investigation

The experimental investigation consisted of punching shear tests on four carbon fiber reinforced concrete (CFRC) slabs and two mild steel reinforced concrete (MSRC) slabs.

All the slabs were tested to failure by monotonically increasing the load. A summary of the test program and properties of the test specimens is presented in Table 1.

Specimen details

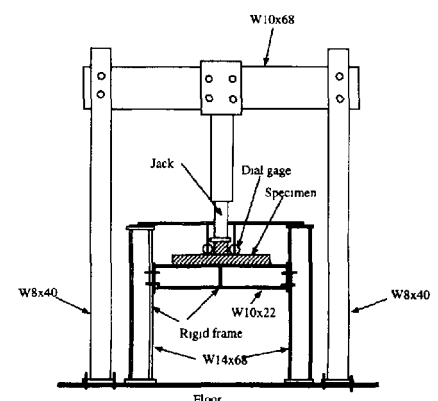
A total of six square concrete slab specimens were fabricated. All concrete slabs were $27 \times 27 \times 3$ in. ($690 \times 690 \times 80$ mm). Four of the slabs were reinforced with 3-D continuous carbon fiber fabric, and two of the slab specimens were reinforced with conventional mild steel reinforcement. Of the four CFRC slabs, two specimens were fabricated with a column stub and the other two specimens without the column stubs.

The reinforcement ratios in the three directions (x, y, and z) for the CFRC slabs were 0.95 percent. The steel bars used for reinforcing the MSRC slabs were #3 deformed bars with an average yield strength of 60 ksi (400 MPa). The reinforcement ratios in the x and y directions were 1.18 and 1.35 percent, respectively. Dimensions and details of the slab specimens are presented in Table 1.

Concrete with a 28-day design strength of 5 ksi (30 MPa) was used. The concrete mixture proportion is shown in Table 2. A view of the CFRC slab before casting is shown in

Table 2 — Concrete mix proportions for punching shear tests of slabs.

Material	Quantity
Cement (Type 1)	26.4 lb/cf
Sand (Linington, NC)	39.0 lb/cf
Course aggregate (crushed granite, max. size < 0.5 in.)	63.6 lb/cf
Retarder (PSI-400R)	15.6 ml/cf
Air entraining agent (Daravair -R)	6.4 ml/cf
Water	10.8 lb/cf
Conversion: 1 lb/cf = 16.03 kg/m ³ 1 ml/cf = 35.31 ml/m ³	

**Fig 3 — Test set-up for punching shear tests.**

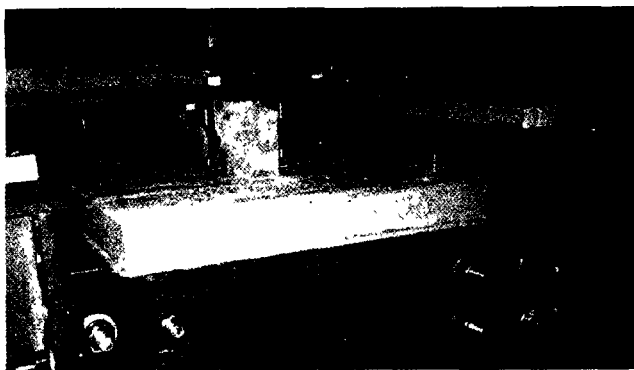


Fig. 4 — A view of a slab specimen for testing.

Fig. 3. Three 4 x 8 in. (100 x 200 mm) control cylinders were also cast with each slab specimen. After the slabs and the control cylinders were cast, they were covered with polyethylene sheets to minimize moisture loss. Twenty-four hours after casting, the slab specimens and the control cylinders were stripped from their molds and kept under laboratory conditions until the time of testing.

Testing

The slabs were tested to failure in a test frame with a 50 kip (200 kN) capacity. Prior to testing the slabs, the control cylinders were tested for concrete strength. The test set-up is shown in Fig. 4. The slabs were placed with their four edges supported on steel beams. A cement mortar bed was placed between the slab specimen and the supporting steel beams. A dial gage was placed at each side of the stub or loading head to monitor the slab deflection along the central line of the slab specimens.

The slabs were loaded at their geometric center by a loading jack with a capacity of 50 kips (200 kN) through a 4 in. (100 mm) square column stub or a 3 in. (70 mm) square steel block, simulating a concentrated load. Before starting the test, the specimens were loaded up to 1 kip (4 kN), and unloaded a few times to insure proper seating of the test specimens. The load was applied in increments of 1 kip (4 kN) initially, but after the load reached 10 kips (40 kN), loading increments were reduced to 0.5 kip (2 kN). For the CFRC slabs, near the maximum load stage when the load-deflection relationship became non-linear, the loading increments were reduced to 0.1 kip (0.45 kN). Loads and deflections were monitored and recorded during the test. A view of the slab specimen in the test frame is shown in Fig. 5.

Results

The results of the slab tests include the crack patterns and the load-deflections curves. The observed ultimate loads were also compared with the predictions according to the relevant equations recommended by the ACI 318-89 Code¹⁶ and the BS 8110 Code.¹⁷

Crack patterns

All the slab specimens carried a concentrated downward load at the center and an upward line reaction force along the support beams. In the corner zone of the slabs, curling-

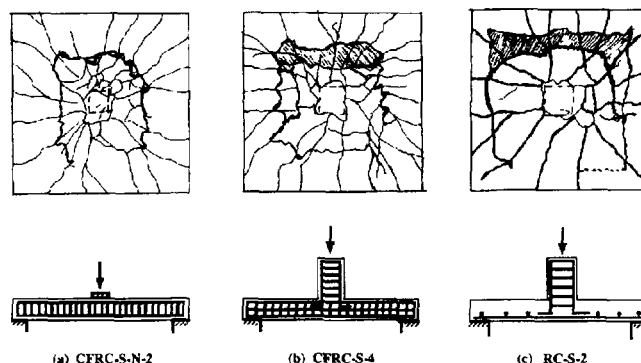


Fig. 5 — Schematic crack patterns after the completion of the testing.

up was observed during the tests since the slabs were not clamped down at the supports. As the load was increased, the underside of the slabs developed a series of cracks radiating outward to the edges from the centrally loaded area. The visible damage prior to failure was dominated by a series of wide radial cracks from the loaded area to the corners of the test slab.

Although the crack widths were not monitored, the cracks became wider as the load increased. Near the ultimate stage, a major crack at a distance of about twice the slab depth away from the perimeter of the loaded area surrounded the load area. Also there were minor cracks radiating outward from the loaded area to the outside edges of the slab.

The crack patterns for all the slab specimens after failure indicated that slabs failed in punching shear modes. Crack patterns on the bottom surface of the CFRC slabs (CFRC-S-2, CFRC-S-4) and MSRC slab (RC-S-4) after the completion of the tests are shown in Fig. 6. Comparison of the crack patterns for the CFRC and MSRC slabs showed that larger number of fine cracks developed in the CFRC slabs. The results indicate that the total area of the perimeter crack surrounding the loaded area is smaller for the CFRC slabs as compared to the MSRC slabs.

For the CFRC as well as for MSRC slabs, the first **radial (circumferential)** crack occurred at about 40 percent of the ultimate load at the bottom of the slabs near the loading area. With the increase in load, a series of radial cracks developed and some of them extended to the outer faces of the slabs. For the CFRC slabs, at about 90 percent of the ultimate load, a critical radial crack around the loading area was observed which was accompanied by an increase in deflection and a breaking sound due to the rupturing of some of the individual carbon fibers.

For the CFRC slabs, after the maximum load stage, the growth of cracking around the loading area increased rapidly so as to cause a punching shear failure. Although the tests were conducted under load control, the CFRC slabs were stable in the post-peak region of the load-deflection curve and for a constant load, the cracks did not propagate. For the MSRC slabs, as the load approached the maximum, the cracks propagated rapidly to cause punching shear failure.

Load-deflection response

The load mid-span deflection curves for the CFRC and MSRC slabs are shown in Fig. 6. From

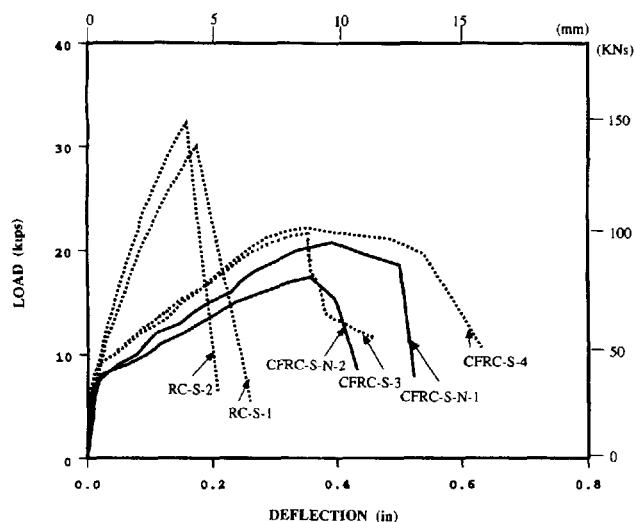


Fig. 6 — Load-deflection curves of slabs reinforced with either carbon fiber fabric, or mild reinforcing steel bars.

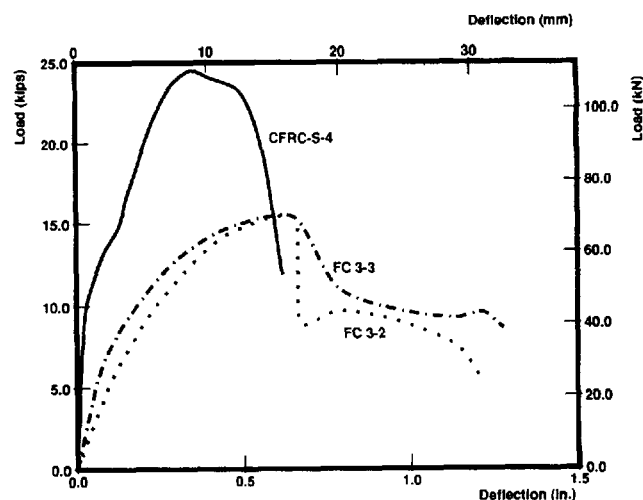


Fig. 7 — Comparison of load-deflection curves of slabs reinforced with 3-D carbon fiber fabric, and slabs reinforced with galvanized wire mesh (ferrocement slabs).

seen that the pre-cracking behavior and the initial stiffness of the CFRC and the MSRC slabs were similar; however, the post-cracking behavior of the CFRC slabs was drastically different from the MSRC slabs. After initial cracking, the stiffness of the CFRC slabs substantially reduced, whereas for the MSRC slabs the reduction in the stiffness was small. The greater reduction in the post-cracking stiffness of the CFRC slabs as compared to the MSRC slabs can be attributed to the lower apparent elastic modulus and the lower reinforcement ratio of the carbon fabric impregnated with epoxy resin.

The MSRC slabs exhibited brittle shear failure immediately upon reaching the maximum load stage. The CFRC slabs exhibited significant non-linear behavior before the maximum load and a deformation softening behavior after the maximum load. The post-maximum load deformation softening behavior is a relative measure of the ductility and indicates redistribution of stresses after the maximum load. During the testing of the CFRC slabs, sounds of carbon fibers breaking could be heard as the load approached maximum. From this, it appears that as the load approached the maximum load, individual carbon fiber elements were rup-

turing and transferring the loads to adjacent carbon fiber elements. In the post-maximum deformation softening portion of the response, a pyramidal-shaped block, which included the column stub, formed and punched downward through the thickness of the slab causing the slab to fail.

Since the 3-D carbon fabric is layered reinforcement, a comparison was made (Fig. 7) between the observed load-deflection responses of the CFRC slab tested in this study and ferrocement slabs tested, Paramasivam and Tan.¹⁸ The ferrocement slabs were 36x36 in. (910 x 910 mm). The depth of the FC3-2 and FC3-3 slabs were 2.3 x 2.8 in. (58 and 71 mm) respectively. The volume fraction of the fine mesh (P_f) was 1.07 percent. The volume fraction of the skeleton steel (P_s) was 0.87 percent. The tension reinforcement ratio (P) was 1.94 percent. The width of the loading plate was 4 in. (100 mm), and the spacing of the skeletal steel was 6 in. (150 mm).

The reinforcement consisted of two layers of galvanized wire mesh, 0.05 in. (1 mm) in diameter with a 0.5 in. (10 mm) square grid. They were separated by a layer of 6 in. (150 mm) square grid and a bar diameter of 0.22 in. (5.6 mm). The 28-day design strength of the cube mortar was 7.25 ksi (50 MPa). Ordinary portland cement and natural sand were used in the ratio of 1:2:3, and the water-cement ratio was 0.5.

It appeared that the initial stiffness of the CFRC slab was much greater than the ferrocement slabs. The comparison also showed that, although the load carrying capacity of the CFRC-S-4 slab with R of 0.95 percent was 66 percent more than the ferrocement slabs, the deformation softening behavior was less ductile. The deformation capacity of the CFRC slab is about half that of the ferrocement slabs.

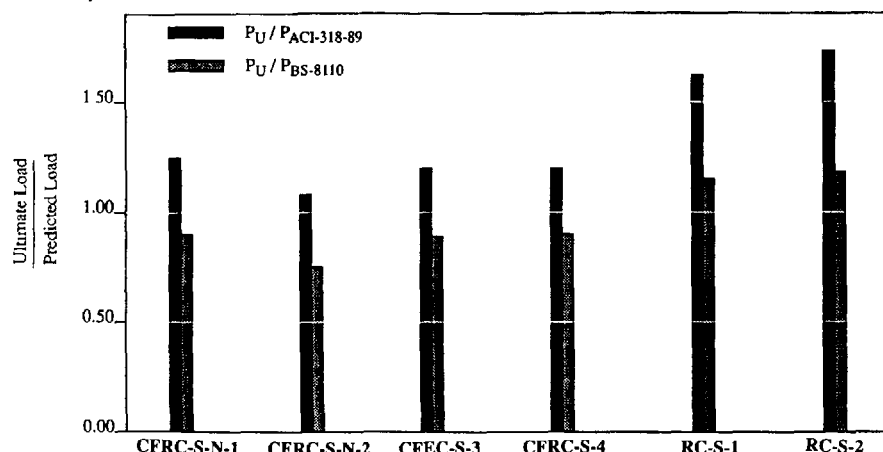


Fig. 8 — Comparison of experimental ultimate loads of CFRC and MSRC slabs with predicted values per ACI 318-89, and BS 8110 codes.

Ultimate load capacity

The average ultimate loads for the CFRC slabs with P of 0.95 percent was

42 percent lower than for MSRC slabs with $P = 1.18$ percent and 1.35 percent. The comparison of the experimental failure loads and the values predicted by the equations recommended by the ACI 318-89, and BS 8110 codes^{16,17} is shown in Table 3.

The values predicted by the Codes do not include the capacity reduction factor ($\phi = 0.85$ in ACI 318-89) and the partial safety factor ($\gamma_m = 1.25$ in BS 8110).

The expressions provided by the ACI 318-89 Code and the BS 8110 Code for the slabs reinforced with conventional reinforcement are:

$$P_{ACI} = 16 \sqrt{f'_c} (c + d) d \quad (\text{psi units})$$

$$P_{ACI} = 1.33 \sqrt{f'_c} (c + d) d \quad (\text{SI units})$$

(Eq. 1)

$$P_{BS\ 8110} = 38.9 \sqrt[3]{100 p} \times \sqrt[3]{\frac{f_{cu}}{25}} \times \sqrt[4]{\frac{400}{d}} (c + 3d) d \quad (\text{psi units})$$

$$P_{BS\ 8110} = 3.16 \sqrt[3]{100 p} \times \sqrt[3]{\frac{f_{cu}}{25}} \times \sqrt[4]{\frac{400}{d}} (c + 3d) d \quad (\text{SI units})$$

(Eq. 2)

From Table 3 and Fig. 8 it can be seen that for the CFRC slabs, the experimental ultimate load values were 8 to 27 percent higher than predicted by the ACI Code equations, and 10 to 25 percent lower than predicted by the BS-8110 Code.

Although the ACI code 318-89 equations were developed for conventional reinforcement, the predictions of the ultimate load capacities are in reasonable agreement also with experimental values for the CFRC slabs.

For the MSRC slabs the observed ultimate loads were 62 to 73 percent higher than predicted by the ACI code equations, and 15 to 18 percent higher than predicted by the BS 8110 code. For the MRSC slabs, a similar conclusion were also reached by Kuang and Morley.¹⁹

Summary

Results of a pilot study on the punching shear behavior of concrete slabs reinforced with three dimensional continuous carbon fiber fabric are presented.

The test results provide experimental information on the behavior of CFRC slabs subjected to a central concentrated load. The test results indicate that the punching shear behavior of the CFRC slabs are different from the MSRC slabs.

The CFRC slabs exhibit a significant non-linear behavior and reduction in the stiffness in the post-cracking stage, and also exhibit a post-maximum load deformation softening response, which is a relative measure of ductility.

Table 3 — Test results of slabs reinforced with either 3-D carbon fiber fabric or mild steel reinforcing bars.

Specimen type*	Cylinder strength f'_c (ksi)	Test age (days)	Ultimate load of slab P_u (kip)	$P_{ACI\ 318}$ (kip)	$P_{BS\ 8110}$ (kip)	P_u / P_{ACI}	P_u / P_{ACI}
CFRC-S-N-1	6.15	55	20.80	16.26	23.09	1.25	0.90
CFRC-S-N-2	6.47	53	17.53	16.68	23.48	1.08	0.75
CFRC-S-3	5.66	33	21.50	18.49	24.14	1.20	0.89
CFRC-S-4	5.31	37	22.18	17.91	24.46	1.20	0.90
RC-S-1	5.65	32	30.00	18.47	26.05	1.62	1.15
RC-S-2	5.78	38	32.33	18.68	27.45	1.73	1.18

*For CFRC slabs, effective depth in computing P_{ACI} was effective depth "d."

Acknowledgments

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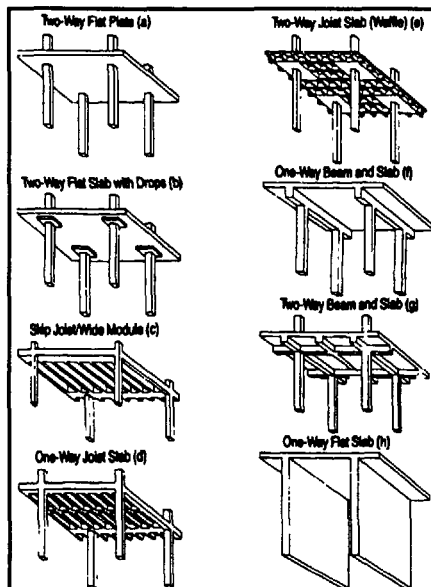
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Use of Continuous Fibers for Reinforcing

by Takayuki Hirai

In Japan continuous fibers are being used both in actual practice and experimentally as prestressing tendons, reinforcing bars, and mesh, for both new construction and retrofitting. The materials being used have high performance and durability characteristics not available with conventional steels. In his article in the August 1992 issue of *Concrete International*, Koichi Minosaku described the use of fiber reinforced materials in prestressed concrete. This article focuses on the use of such materials for non-prestressed reinforcement.

Reinforcing bars

Continuous fibers are being used for reinforcing concrete where long-term

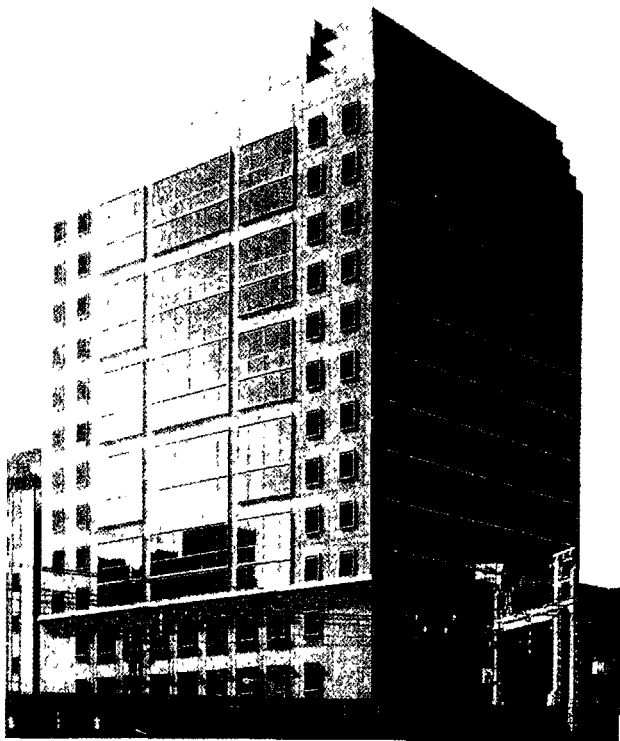
durability against corrosive solutions and gases is demanded, where non-magnetic properties are desired, and where ease of transporting, cutting, fabricating, and placing of reinforcement is needed.

Continuous fibers can be made into preformed reinforcement assemblies that can be cut into complex shapes. Because of its light weight, such reinforcement can be placed in formwork without the use of machinery such as winches. This is extremely useful where mechanical facilities are limited and where there is a shortage of skilled workers, since even in these situations construction time can be reduced and quality improved.

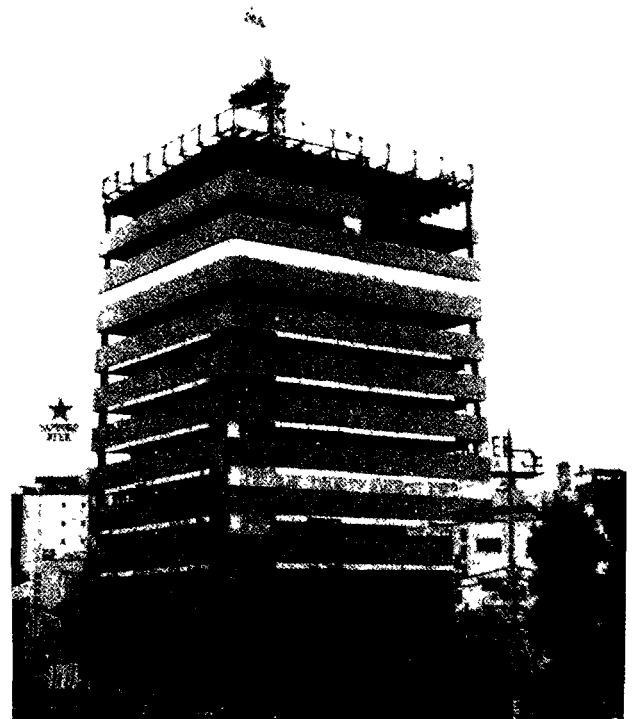
The photographs accompanying this article show a variety of uses of various

types of continuous fiber reinforcement in concrete in Japan. Also shown is use of the material in the three-story main administration building at the Showa Base in Antarctica.

Rods made of vinylon fibers and polyarylate fibers bonded together with epoxy resin are being used to reinforce concrete walls and floors. Some rods have plain surfaces, others have twilled, deformed surfaces. Other products are available such as accordion shapes for use as stirrups and twisted wires. Spiral filament-wound fiber reinforced plastic rods made of carbon, glass, or aramid continuous fibers bonded together with vinyl ester resin and having the same variety of fiber wound around and bonded to the rod to produce a deformed surface have also been developed.

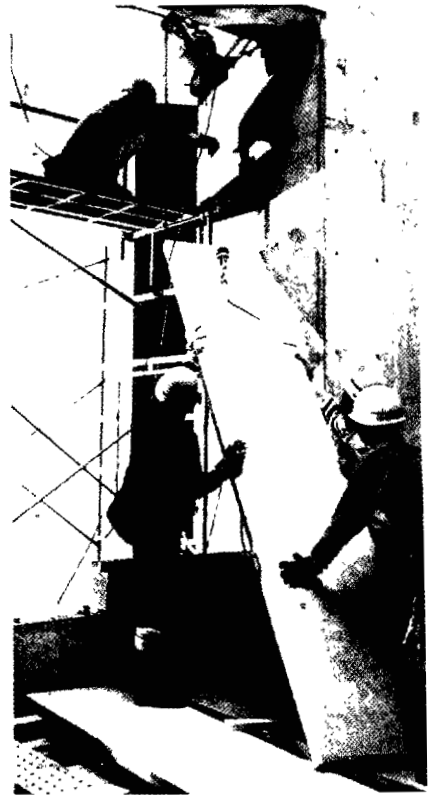


The parapet panels on the Tokyo Dental College Suidobashi Building are reinforced with bars containing continuous aramid fibers.

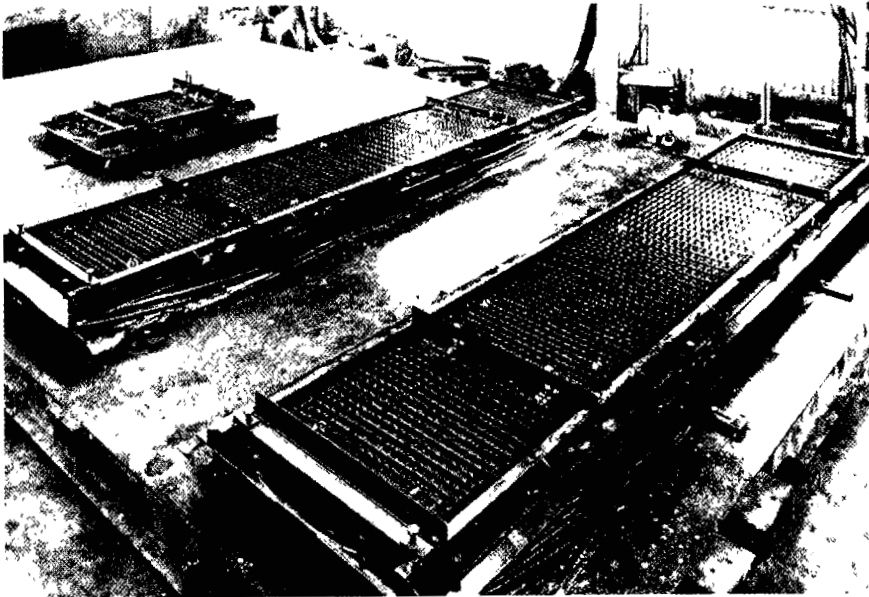


The lightweight concrete curtain wall panels of this building are reinforced with grids of carbon and glass fibers bonded together using vinyl ester resin.

Concrete



The photograph at left shows concrete panels being prefabricated using continuous fiber reinforcement. The photo above shows these panels being installed in a partition wall for a chlorine storage room at Higashi Murayama Water Purification Plant.



Mesh

Wire mesh is being made from continuous fibers of aramid, carbon, and glass braided into tape and impregnated with resin. Vinylon mesh is being used for reinforcement in both concrete and cement secondary products.

Retrofitting

Structures can be retrofitted with continuous fiber reinforcement by bonding fibers directly to the surface with resin, or by attaching a fiber-reinforced prepreg product.

Continuous fibers are being used in

this way to improve flexural strength, shear strength, ductility, and fatigue resistance, reduce deflections due to cracking, and prevent cracking due to thermal stresses. They are also being used to prevent wall finish materials such as tiles from falling, by covering



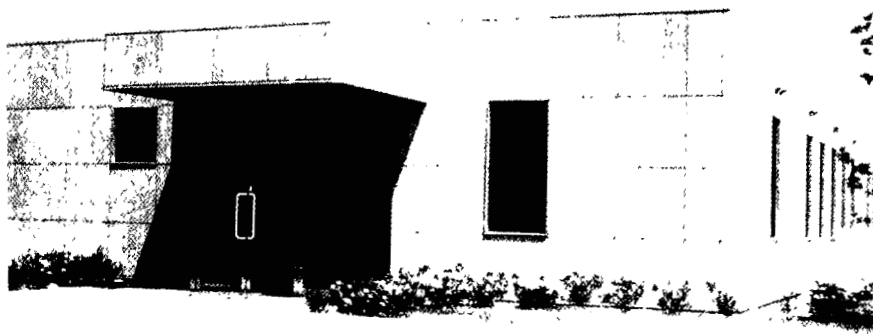
Lattice reinforcement of continuous carbon fibers bonded together with resin was used in the wall footings and floor slabs at Showa Base in Antarctica. The wall footing reinforcement was cut to fit the irregular shape of the rock surface on which the building was constructed.



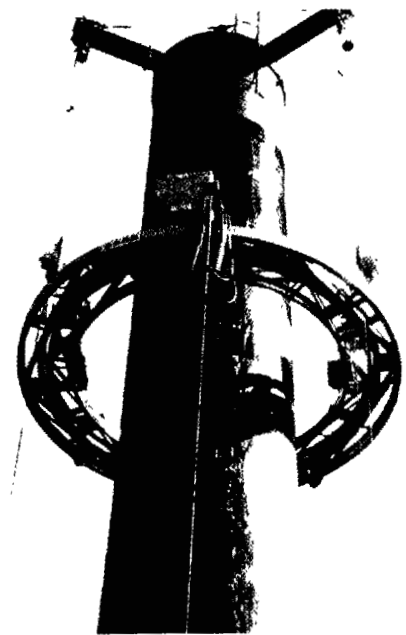
the finish materials with a fiber mesh fixed to the substrate by means such as anchor pins and fixing the mesh to the finish materials using resin mortar.

Selected for reader interest by the editors.

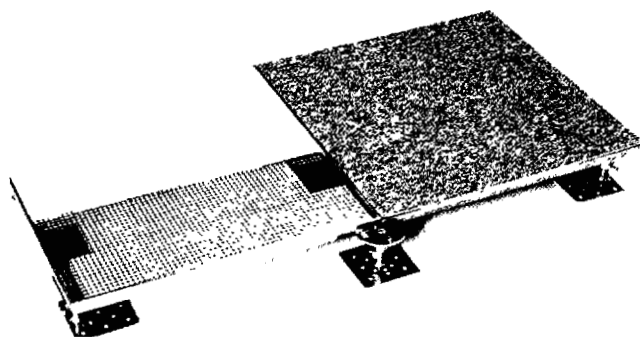
Takayuki Hirai is a professor in the Faculty of Engineering, Oita University, Japan. He is specializing in the development of cement matrix composite materials and FEM and BEM computer analysis of the mechanical behavior of composite materials.



The exterior wall panels of the Taisei Biotechnology Research Center in Chiba Prefecture were made by laminating prepreg sheets of cement-filled continuous carbon fiber bundles.



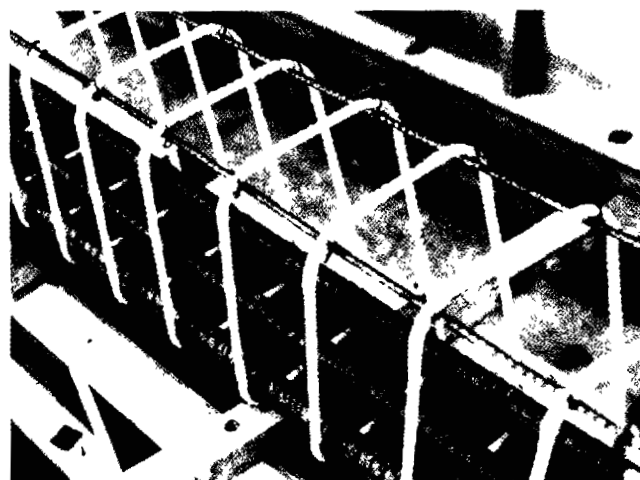
Uni-directional carbon fiber prepreg was resin-bonded to this chimney with the fibers aligned vertically to improve flexural strength, and carbon fiber strands were wound circumferentially to prevent cracking due to thermal stresses.



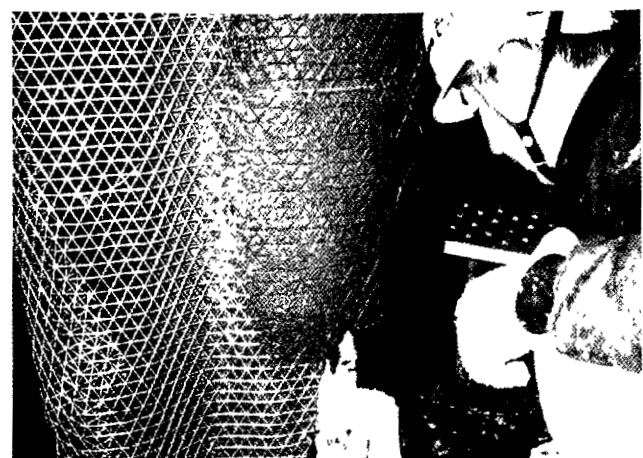
A free access floor panel of concrete reinforced with a mesh of continuous carbon fibers bonded together epoxy resin.



A uni-directional prepreg of carbon fiber bonded with resin to the underside of this reinforced concrete slab to improve flexural strength and reduce deflection due to cracking.



Accordion-type stirrups with no splices.



A triaxial screen being used to reinforce finish mortar on a column.

Using FRP Materials in Prestressed Concrete Structures

by Koichi Minosaku

The demand for fiber reinforced plastic (FRP) materials has been increasing for various uses, such as in aircraft, automobiles, and sporting goods. But recently, the use of FRP products as alternatives for such rod materials as reinforcing bars and prestressed concrete tendons has attracted special interest.

At the same time that fibers consisting of various new base materials have been developed, a need for corrosion-resistant materials has arisen as the problem of damage due to chloride ions has become spotlighted and the need for structural materials for offshore structures and waterfront facilities has increased. Also, nonmagnetic materials are required for developing magnetic levitation trains.

These circumstances led the Japan Society of Civil Engineers (JSCE) to establish the Study Committee on FRP Materials for the purpose of preparing guidelines for using FRP materials. The Japanese Ministry of Construction also has been studying the use of FRP products as tendons for prestressed concrete bridges in a comprehensive engineering development project called Development of Utilization Techniques for New Base Materials and New Materials in the Construction Field.

Furthermore, the Japanese Ministry of Transportation and the Japanese Railways Company are studying the application of FRP materials to offshore structures and magnetic levitation trains. Materials manufacturers and construction companies also are engaged in various activities promoting the use of FRP products.

Studies of FRP product use are already under way in Europe and the Americas, but Japan is moving more quickly toward practical application of the materials. Although

glass FRP rods were first used in a prestressed concrete bridge in Germany in 1980, FRP glass fibers had already been used in large quantities in Japan as a reinforcing material for shotcreting of tunnels. The Japanese also were the first to use carbon FRP strands as tendons in a prestressed concrete bridge.

Types of FRP products and component materials

Compared with conventional reinforcing bars and prestressing steels, FRP materials strengthened in one direction generally have a small modulus of elasticity and linear coefficient of expansion. However, these materials have high strength and excellent corrosion resistance. They are also lightweight and nonmagnetic.

Because of these features, and particularly because FRP materials possess many characteristics sought for tendons for prestressed concrete members, the properties of FRP materials are being studied.

Various fibers have been developed, but FRP products using carbon, aramid, and glass fibers are attracting interest in Japan. Carbon fibers are manufactured from rayon, petroleum, coal tar, and polyacrylonitrile (PAN), and exhibit special characteristics depending on the respective raw materials.

Carbon fibers consisting of petroleum or coal tar are divided into two types — having low and high elastic modulus. The latter generally is used for FRP products.

Aramid is an abbreviated term for aromatic polyamide. Para-type fibers with straight-chain benzene nuclei are generally used for FRP products. Aramid fibers are presently available commercially in Japan, the United States, and the Netherlands.

Glass fiber is inorganic, consisting of a noncrystalline high-siliceous fiber. Measures to avoid alkali contact are needed in developing FRP products using this fiber.

Matrix materials generally include thermosetting resins such as epoxy resins and vinyl ester resins, polyester resins, and thermoplastic resins. FRP materials already developed have matrix contents of about 40 to 70 percent.

Surface configurations of FRP products can be smooth or deformed with spirals perpendicular to the axial direction or meshes at the surface to prevent damage during handling. Cross-sectional configurations include strands and braids in addition to rods.

FRP tendons in prestressed concrete bridges

CFRP strands in pretensioned prestressed concrete highway bridges

Carbon FRP (CFRP) strands were first used as tendons in Shinmiya Bridge, a pretensioned prestressed concrete slab highway bridge constructed at Fuki Town, Hakui County, Ishikawa Prefecture in 1988. The 5.76 m (18.9 ft) span is 7.0 m (22.9 ft) wide CFRP strands used were composed of seven wires, with total diameter of 12.5 mm (0.5 in.). Six strands were provided at the bottom flange and two at the upper flange. Epoxy-coated reinforcing bars were used for stirrups. The manufacturer's guaranteed strength P_u of this material is 137 kN (15.4 tonf). The allowable tensile force was set at 0.60, 0.55, and 0.45 P_u for during prestressing, immediately after prestressing, and under the design load, respectively.

Ishikawa Prefecture then erected a cycling road bridge in 1991 using CFRP strands to prevent corrosion due to chloride ions from the sea

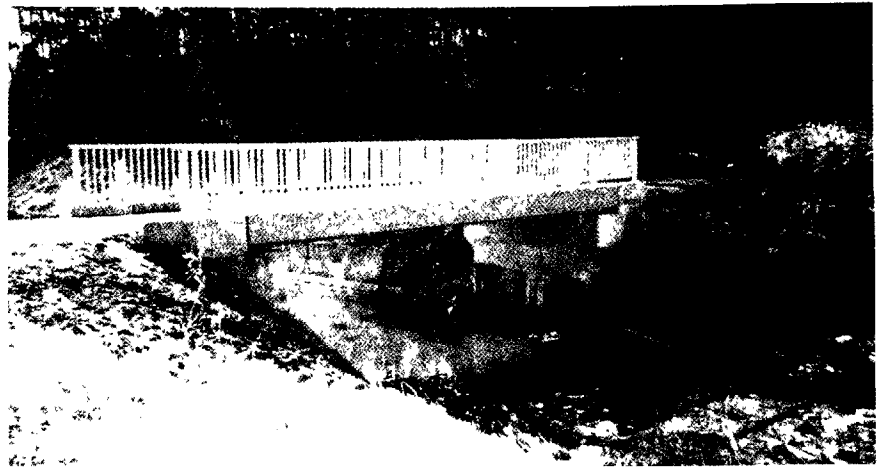


Fig. 1 — CFRP strands were used in this pretensioned cycling road bridge.

(Fig. 1). This bridge is a pretensioned hollow slab type in which straight CFRP wires are provided as substitutes for stirrups in half of the outside girders.

CFRP strands in a pretensioned prestressed concrete pedestrian overbridge

CFRP strands were used in the Nagatsugawa pedestrian overbridge, a pretensioned simple-slab bridge 2.5 m (8.2 ft) wide and 8.0 m (26.2 ft) long, built at Nagatsugawa, Funabashi City, Chiba Prefecture in 1989 (Fig. 2). The anchorages used in fabrication were surface-threaded steel pipes. CFRP strands were inserted into the anchorages before the injection of epoxy resin. This bridge was fabricated in the factory as a one-piece nonmetallic structure. CFRP rods with ribs and lugs on their surfaces were used as distributing bars and stirrups. The manufacturer's guaranteed strength P_u of the strands was 137 kN (15.4 tonf). The allowable tensile forces during prestressing, immediately after prestressing, and under the design load were set at 0.60, 0.55, and 0.50 P_u , respectively.

CFRP strands in a post-tensioned prestressed concrete highway bridge

CFRP strands developed in Japan were used for part of the tendons in a post-tensioned prestressed concrete highway bridge erected in 1991 on an autobahn crossing over a road in a German factory area. The girder bridge is approximately 80 m (260 ft) long and 11.2 m (36.7 ft) wide. Four large-capacity multicables

of 19 CFRP strands of 12.5 mm (0.5 in.) diameter were used and anchored by a wedge system. The allowable tensile force of the CFRP strands used was set at 0.50 P_u (manufacturer's guaranteed strength) under design load. This bridge was not grouted so that future inspections and data collection could be done.

CFRP rod in a post-tensioned prestressed concrete bridge

CFRP rods were first used as tendons in a simple two-span prestressed concrete highway bridge erected in 1989 as part of an improvement project on the Bachi River in Kitakyusyu City. The bridge is 12.3 m (40.5 ft) wide and 35.8 m (117.5 ft) long. The structure consists of a pretensioned girder section spanning 18.25 m (59.9 ft) and a post-tensioned girder section spanning 17.55 m (57.6 ft). CFRP rods were used in one of the two post-tensioned girders, which had a solid rectangular cross section. This girder was made integral with the other girders by lateral prestressing.

The tendons consisted of eight multicables bundled with eight CFRP rods of 8 mm (0.3 in.) diameter. The material manufacturer's guaranteed strength P_u per multicable was 709 kN (79.7 tonf), and the allowable tensile force under the design load was set at 392 kN (40.1 tonf), corresponding to 55 percent of P_u . A wedge-type steel anchorage was used for the multicables. The prestressing operation was divided into three stages to reduce scatter in prestressing forces.

Braided AFRP in a pretensioned prestressed concrete highway bridge

Braided Aramid FRP (AFRP) tendons were used in the construction of a slab bridge completed in 1990 on a golf course in Nasu County, Tochigi Prefecture (Fig. 3). The bridge, of three spans of 11.98 m (39.3 ft), is 2.40 m (7.9 ft), in accordance with pretensioned girders by JIS specifications. Braided AFRP tendons of 14.0 mm (0.55 in.) diameter were used in three of the 21 girders. The allowable cable tensile force under the design load was set at 94.1 kN (10.6 tonf), corresponding to 0.5 P_u (the manufacturer's guaranteed strength).

AFRP rods in a pretensioned and post-tensioned prestressed concrete bridge

A prestressed concrete road bridge using AFRP rods for tendons was erected at the freight entrance to a concrete products plant in 1990 and 1991. The bridge consisted of a pretensioned composite slab spanning 12.5 m (41.0 ft) and a post-tensioned box girder spanning 25.0 m (82.0 ft). AFRP rods were used for all tendons in both sections, and as stirrups and reinforcing bars in the pretensioned section and as external cables in the post-tensioned section as well. Specifications for the AFRP rods used were 3 $\varnothing$ 6 for the pretensioned section, 19 $\varnothing$ 6 for internal cables, and 7 $\varnothing$ 6 for external cables in the post-tensioned section. The manufacturer's guaranteed strengths P_u for the respective tendons are 150, 784, and 350 kN, (16.7, 88.1, and 39.3 tonf). The



Fig. 2 — CFRP strands were used in this pretensioned pedestrian bridge.

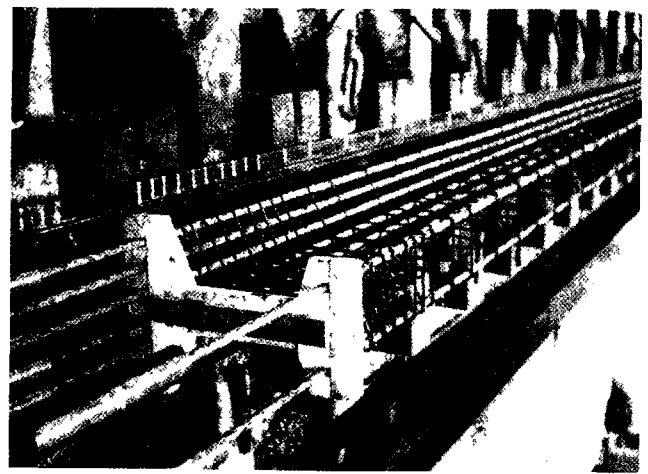


Fig. 3 — Pretensioned girder using braided AFRP tendons.

anchorage was secured by bonding with an inorganic mortar in cylinders of metal or FRP; the cylinders were fixed to the members by the threaded anchorage. The allowable tensile forces of tendons were set at 0.8, 0.7, and 0.6 P_u during prestressing, immediately after prestressing, and under the design load, respectively. These values are the same as those for ordinary prestressing steels.

AFRP bands in a post-tensioned prestressed concrete pedestrian bridge

AFRP bands were used as tendons in a post-tensioned prestressed concrete suspended slab bridge, 2.1 m (6.9 ft) wide and 54.5 m (178.8 ft) long pedestrian bridge built 1990 at a golf course in Mito City, Ibaragi Prefecture. The cross section of AFRP bands is 4.86 x 19.5 mm (0.19 x 0.77 in.) and the guaranteed strength P_u is 118 kN (13.2 tonf). Eight of the bands were combined to make a single cable, and 16 cables were used. The allowable tensile forces of the cables were set at 0.5 P_u under maximum tensile force and 0.33 P_u under static load. Cables were anchored by inserting eight AFRP bands into a sleeve (material: SCM 435) and filling it with expansive mortar. AFRP bands were also embedded in the slab as tensile reinforcement. Cable bundling nine CFRP rods of 8 mm (0.3 in.) diameter and tensile strength of 18 kgf/mm² (177 MPa) was used for ground anchors.

Application of GFRP Products

In Japan, application of glass FRP (GFRP) products as tendons

for prestressed concrete bridges lags behind that of CFRP and AFRP materials. However, in Germany, three bridges have been constructed: the Lunensach-Gasse Bridge, a highway bridge spanning 6.5 m (21.3 ft) erected on a trial basis in the outskirts of Dusseldorf in 1980; the Ulenberg-Strasse Bridge, a 2-span continuous-girder highway bridge built in Dusseldorf in 1986; and the Adolf-Kiepert pedestrian bridge, a 2-span continuous double-T-slab girder bridge erected in Berlin in 1989.

The Ulenberg-Strasse Bridge uses a post-tensioning system with 59 multicables consisting of 19 GFRP rods with a diameter of 7.5 mm (0.3 in.). The GFRP rods, called Polystal, contain 68 percent glass fibers by volume, and were formed by pultrusion using polyester resin. The allowable tendon tensile force was set at 0.47 P_u (manufacturer's guaranteed strength) under design load. Fig. 4 shows how the tendons are anchored by adhesion to a mortar combining quartz and polyester resin, which was injected to avoid alkali reaction of the glass fibers. Similar GFRP rods were used in the Adolf-Kiepert Pedestrian Bridge, however, this bridge was designed with partially prestressed members and the tendons were used as external cables.

In Japan, GFRP has been applied experimentally in lattice-form in a pretensioned prestressed concrete pedestrian overbridge. The lattice-form GFRP had 15-cm (5.9-in.) spacing grids formed of glass fiber bundles impregnated with vinyl ester resin.

FRP concrete in port and harbor structures

Because of excellent corrosion resistance, the application of FRP products as reinforcing bars, prestressing tendons, and tie materials in port and harbor structures is beginning to receive attention. FRP products are being considered for use as reinforcing material for fenders, tensioning material for precast members, lateral tie materials for precast members, tie materials for offshore-joined prestressed concrete floating breakwaters, and polygonal floating structures (HMS).

In one case, the material has been applied as reinforcing material for fenders attached to an existing pier. Lattice-form CFRP was used as an alternative to epoxy-coated reinforcing bars in this case. The polygonal floating structure was developed by the Japanese Ministry of Transport Second Port and Harbor Construction Bureau for marina facilities such as floating piers and floating breakwater. Structurally, the HMS is a hexagonal floating structure with six reinforced concrete or prestressed concrete pontoons that are trapezoidal in plane configuration and hollow inside joined together by tensioning materials (Fig. 5). The tensioning materials were nine multicables consisting of eight CFRP rods, each with a diameter of 8 mm (0.3 in.) The manufacturer's guaranteed strength per one multicable is 709 kN (79.7 tonf), and tensioning force of 294 kN (33.1 tonf) per cable was transferred during construction.

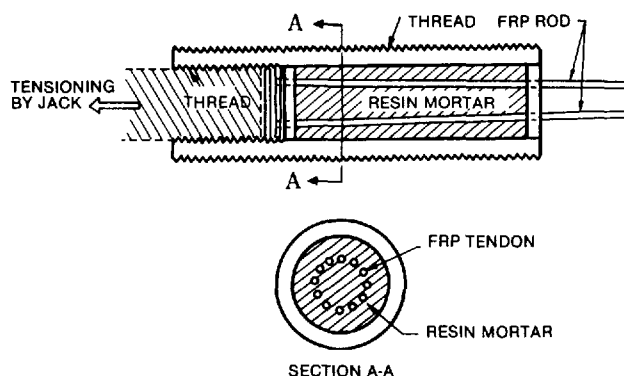


Fig. 4 — Anchorage used in Ulenberg-Strasse Bridge.

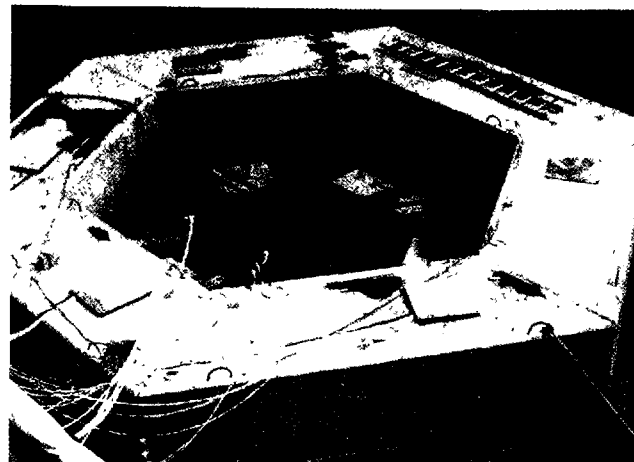


Fig. 5 — Polygonal floating structure using CFRP rods for tendons.

FRP as reinforcing material for concrete structures and soil structures

Reinforcing net for shotcrete

FRP products of glass fibers are now being used comparatively often as substitutes for welded-wire mesh for shotcrete. For such applications, FRP is lightweight and easy to handle, has excellent corrosion resistance, and is easier to mold to the shape of the shotcrete surface than welded wire mesh. FRP has been used in retaining wall concrete at an underground LPG storage tank.

Rock bolts

GFRP and AFRP materials are being applied as rock bolts in mountain tunnelling projects. Rock bolts

are driven into the ground before tunnelling to preventing cave-ins. FRP is more expensive than conventional steel bolts.

Embankment reinforcement

Concrete grids are often used on slopes of highway cuts with the grid openings used for vegetation growth. FRP materials are particularly effective in this application when corrosion of wire mesh is of concern in places such as hot spring areas. FRP products are also beginning to be used as soil reinforcing material, making tall and steeply-sloped embankments possible.

Conclusions

When assessing the performance of such concrete structures as bridges, there are comparatively few cases of deterioration occurring due to act-

ing loads; there are more cases of corrosion of reinforcing bars and prestressing steels. Considering the excellent corrosion resistance properties of FRP products, it may be expected that FRP products will see increased use in concrete structures.

However, even when the problem of cost is set aside, several technical problems remain to be solved before FRP products will see widespread use in civil engineering structures. These include the method of anchoring FRP products in post-tensioned prestressed concrete members; the fatigue strength of FRP products when used as unbonded tendons to take advantage of their corrosion resistance properties; design techniques for concrete members considering the stress-strain relationship of FRP; a unified materials testing method that considers FRP characteristics; and a method for evaluating durability of structural materials which can be reflected in actual design.

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Selected for reader interest by the editors.

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Bridges Constructed Using Fiber Reinforced Plastics

by Magdi A. Khalifa, Sharon S. B. Kuska, and James Krieger

In recent years, the deterioration and structural safety of bridges and the cost of their rehabilitation and repair have become a major concern. Most concrete deterioration can be attributed to the corrosion of reinforcing steel resulting from exposure to highly humid climates and salt water environments. Many procedures have been suggested to overcome this critical problem. One of the most promising solutions is the use of new materials. Composite materials such as fiber reinforced plastic (FRP) are being developed and tested for the possible replacement of conventional prestressing and reinforcing steel.

The development of advanced composite materials began more than four decades ago. FRP consists of high strength fibers bonded together in a polymer matrix to create composite action. Maximum strength is achieved by orienting the fibers in the longitudinal direction of the member. The aerospace industry was the leader in the use of composite materials such as glass fiber reinforced plastic (GFRP) as a substitute for expensive metals and alloys in some structural parts. The automobile and sports industries were also among the leading developers.

GFRP products were first used for prestressing experimental bridges about ten years ago in West Germany. Other FRP products using aramid fiber reinforced plastic (AFRP) and carbon fiber reinforced plastic (CFRP) were also developed.

Due to their light weight, high tensile strength, excellent corrosion resistance, and flexibility, many structural applications have been suggested for FRP. These applications cover a wide range of structures and structural components. All marine and water exposed structures such as piers, docks, artificial islands, and suspension and cable-stayed bridges can benefit from FRP's corrosion resistance. Radar stations, telecommunication facilities and metro-rail superstructures can benefit from the electromagnetic resistance of FRP.

Bridge applications

Pedestrian and road bridges using GFRP, AFRP, and CFRP have been constructed around the world. Bridge types range from simply-supported slabs to the most sophisticated systems, such as cable-stayed bridges.

Pedestrian bridges

The Westminster Cathedral GRP (glass reinforced polyester) Footbridge has two spans with a total length of 24.4 m (80 ft) and a U-shaped cross-section. It was constructed in London, England in the 1970s.¹

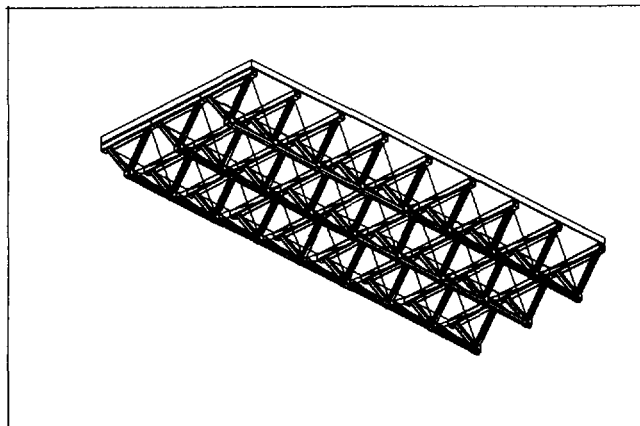


Fig. 1 — The Virginia Pedestrian Bridge.

The Virginia Pedestrian Bridge, erected in 1978, has three GFRP triangular trussed girders 4.9 m (16 ft) long (Fig. 1) connected by a 2.1 m (6.9 ft) wide GFRP cover plate. The web is 40 cm (16 in) high and the whole superstructure weighs approximately 3.5 kN (0.8 kip). The concrete deck slab acts as a portion of the top flange of the girder. Tension members in the web are made of GFRP.²

The Adolf Kiepert Pedestrian Bridge, built in the late 1980s in Berlin, Germany, has two spans of 27.61 and 22.96 m (90.6 and 75.4 ft) (Fig. 2). The superstructure has partial external prestressing without bond. The bridge's Double-T cross-section is 1.1 m (3.6 ft) high with a 4.8 m (15.75 ft) slab width. Each of the seven external prestressing tendons is made of 19 GFRP rods. Each rod is 7.5 mm (0.30 in) in diameter and consists of 60,000 single fibers oriented longitudinally. The tendons are exposed to allow for direct accessibility for checking and replacement. Optical fiber sensors and copper wire sensors are used to monitor the behavior of the prestressing tendons and the entire bridge.³

In 1980 GFRP tendons were used in Europe in the Lünen'sche Gasse experimental foot bridge in Düsseldorf, Germany. The post-tensioned concrete bridge has a single span of 6.55 m (21.5 ft). Each of the 12 7.5 mm (0.3 in) diameter GFRP tendons used have an ultimate load of 70 kN (16 kip). The tendons were installed without bond using four different anchorage systems.^{9,10}

The Chongqing Pedestrian Cable-Stayed Bridge in China, completed in 1986, contains a GFRP box girder. The system is asymmetrical, having one 11 m (36 ft) high tower and a

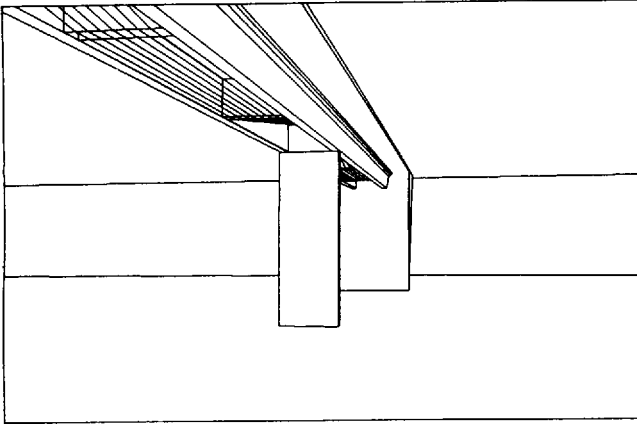


Fig. 2 — The Adolf Kiepert Pedestrian Bridge.

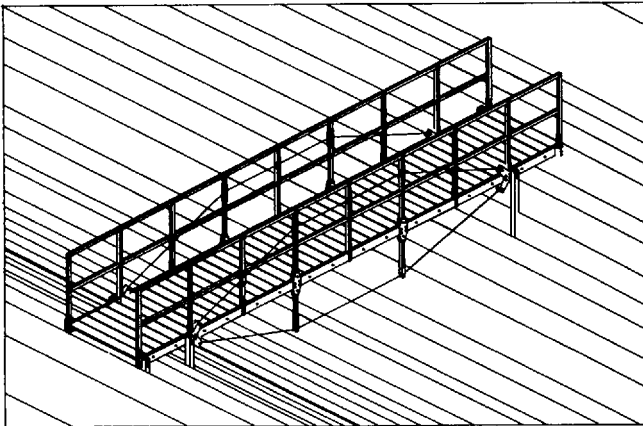


Fig. 3 — The DuPont Country Club Pedestrian Bridge.

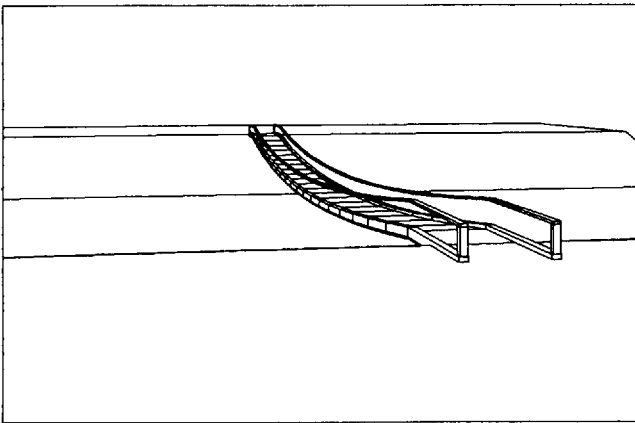


Fig. 4 — The Birdie Pedestrian Bridge.

one-side array of seven cables assembled in a harp shape. The stay cables are made of 19 steel wires encased in polyethylene. The main span is a 27.4 m (90 ft) long, 4.3 m (14 ft) wide, GFRP box girder with cover plates. The box girder was transported and erected in one piece. The side spans and the tower are made of reinforced concrete.⁴

The Guanyinqiao Pedestrian Bridge in Chongqing, China, completed in 1988, has eight GFRP deck girders suspended from reinforced concrete rigid frames by high strength wires. The overall bridge length is 157 m (515 ft) and the rigid

frames span 70 m (230 ft). All eight girders are 4.3 m (14.1 ft) wide and 0.9 m (3 ft) deep; four with 19 m (62.3 ft) spans and four with 9 m (29.5 ft) spans.⁴

Chiba Prefecture built the Nagatsugawa pedestrian bridge in 1989 in Funabashi City, Japan. It is a simple-slab 2.5 m (8.2 ft) wide and 8.0 m (26.2 ft) long, pretensioned with CFRP strands and fabricated as one piece and moved to the site. CFRP rods with ribs and lugs on their surfaces were used for distributing bars and stirrups.⁵

The DuPont Country Club Pedestrian Bridge in Wilmington, Delaware, was built in 1990 (Fig. 3). The bridge span is 9.14 m (30 ft) with a clear span of 7.0 m (23 ft) and a total weight after construction of less than 4.44 kN (1.0 kip). The bridge deck is made of pressure-treated lumber, the girders are made of pultruded fiberglass, and the supporting cables are made of aramid fiber.⁶

The Birdie Pedestrian Bridge was built in Mito City, Japan, in 1990 (Fig. 4). The post-tensioned concrete suspended-slab bridge is 54.5 m (178.8 ft) long with a clear span of 46.5 m (152.6 ft) and is 2.1 m (6.9 ft) wide. The sag of the bridge is 1.25 m (4.1 ft). Bundles of 8 mm (0.3 in.) diameter CFRP rods were used for ground anchors for the abutments. Sixteen bundles of flat rectangular AFRP cables with a cross-section of 4.86 x 19.5 mm (0.19 x 0.77 in) were used for the prestressing, eight for pretensioning and eight for post-tensioning.^{5,7}

Ishikawa Prefecture, Japan, erected three hollow-slab cycling bridges pretensioned with advanced materials. The 15th, 16th, and 17th Hakui Bicycle Bridges were constructed in March 1989, February 1992, and April 1992, respectively (Fig. 5). CFRP prestressing tendons were used to prevent corrosion due to chloride ions from the sea. Straight CFRP wires were used for stirrups in half of the outside girders.^{5,7}

The Aberfeldy Golf Club Footbridge over the River Tay in Scotland was completed in 1992. The cable-stayed bridge has two A-shaped towers and is the first bridge that was completely built out of fiber composites (Fig. 6). The cables are made of Parafil (kevlar fibers). The longest is 31 m (101.6 ft) and the shortest is 13 m (42.6 ft). The towers are 17.5 m (57.36 ft) high. The main span is 63.0 m (206.5 ft) long with an overall bridge length of 113.0 m (370.4 ft). The deck width is 2.23 m (7.3 ft).⁸

Road bridges

The first paper reinforced plastic bridge was built in the United States before 1971. Made with plastic materials reinforced with laminated paper, it has a single span of 10 m (32.8 ft). The bridge has five longitudinal flat girders and eight similar transverse girders, is approximately 1.3 m (4.3 ft) high, and can carry a 300 kN (67 kip) truck.⁹

The Ginizi Highway Bridge in Bulgaria was erected in 1982. It has one 12 m (39.4 ft) long simply-supported span. The cross section consists of parallel GFRP I-beams.¹¹

The simply-supported single span Miyun GFRP Highway Bridge was completed in 1982 in China. The 20.6 m (67.6 ft) long, 11.38 m (37.33 ft) wide, bridge has six box girders with a honeycomb core. The girders are 1.6 m (5.3 ft) wide, 1.67 m (5.5 ft) high, and weigh 52 kN (11.7 kip). The bridge with-

stood a test load of four trucks totalling 822 kN (185 kip), deflecting only 27 mm (1.06 in), or $\frac{1}{56}$ of the span.^{9,10}

The Ulenbergstrasse Bridge, Germany's first roadbridge built using GFRP prestressing tendons, was completed by 1986 in Düsseldorf (Fig. 7). It is a two-lane, two-span bridge. One span is 21.3 m (69.9 ft) long and the other is 25.6 m (84.0 ft) long, both are 15 m (49.2 ft) wide. The solid slabs were cast-in-place and are 1.75 m (5.7 ft) deep. Steel reinforcement was also used. Fifty-nine composite prestressing tendons were used in the longitudinal direction in each span. Each tendon consists of 19 7.5 mm (0.3 ft) diameter bars to provide a tensile working force of 600 kN (135 kip) per post-tensioned unit. This bridge is furnished with optical fiber sensors in addition to the sensors embedded directly in the concrete to measure changes in the bridge structure.^{12,13}

Eight 8 mm (0.3 in) diameter CFRP rods were used to prestress a simple two-span prestressed concrete highway bridge erected in 1989 over the Bachi River in Kitakyusyu City, Japan (Fig. 8). The bridge is 35.8 m (117.5 ft) long and 12.3 m (40.4 ft) wide. In one span the 17.55 m (57.6 ft) long girders are post-tensioned and in the other span the 18.25 m (59.9 ft) girders are pretensioned.⁵

The 85 m (278.9 ft) long Oststrasse bridge, built in Ludwigshafen, Germany, has space for two lanes. It is composed of four 20 m (65.6 ft) long spans, two straight and two curved with a radius of 62.8 m (206 ft). The superstructure was modelled as a bi-framed slab-and-beam bridge 1.12 m (3.7 ft) high and 11.2 m (36.8 ft) wide. Sixteen steel tendons and four carbon fiber reinforced composite cable tendons each consisting of 19 12.5 mm (0.5 in) diameter strands were used for the prestressing. Each strand was tensioned to 70 kN (15.7 kip) to use 50 percent of the cable strength. The post-tensioning tendons were not grouted to allow for future inspection and data collection.¹⁴

A prestressed, prefabricated concrete slab-beam system designed to withstand heavy road traffic and to cover an underground tank was erected in Dormagen, Germany in 1987 using GFRP tendons for prestressing. The system is composed of two spans, each 10.0 m (32.8 ft) long, and replaced a structure that had deteriorated because of highly corrosive chloride vapors. The HLV-prestressing tendons have a working load of 600 kN (135 kip).¹³

The Shinmiya Bridge, a pretensioned concrete slab highway bridge, at Fuki Town, Hakui County, Japan was constructed in 1988. It has 24 pretensioned concrete I-girders, 325 mm (12.8 in) high with a 200 mm (7.9 in) wide top flange and a 320 mm (12.6 in) wide bottom flange. The span is 5.76 m (18.9 ft) and the bridge is 7.0 m (22.9 ft) wide. Eight CFRP strands were used in each girder, six in the bottom flange and two in the upper flange. Each strand is composed of seven wires, with a total diameter of 12.5 mm (0.5 in). Epoxy-coated reinforcing bars were used for stirrups.^{7,14}

The A19 Tees-Viaduct in Middlesbrough, England has an enclosure made of GFRP for its longest span, which is 117 m (383.8 ft). The enclosure was built in 1989 to reduce the corrosion rate of the plate girders and to provide a structural platform for inspection and maintenance. The structural floor

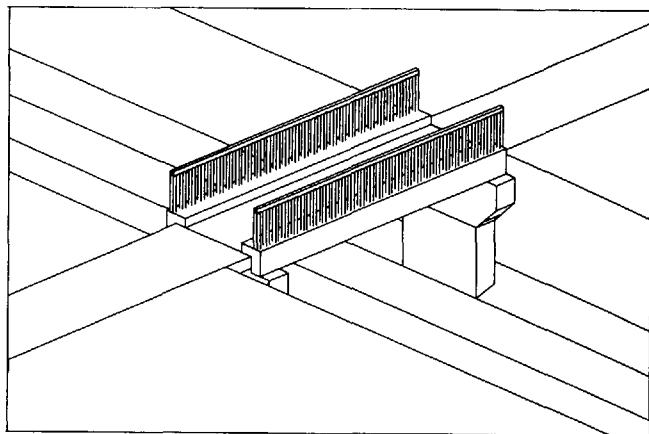


Fig. 5 — The Hakui Bicycle Bridge.

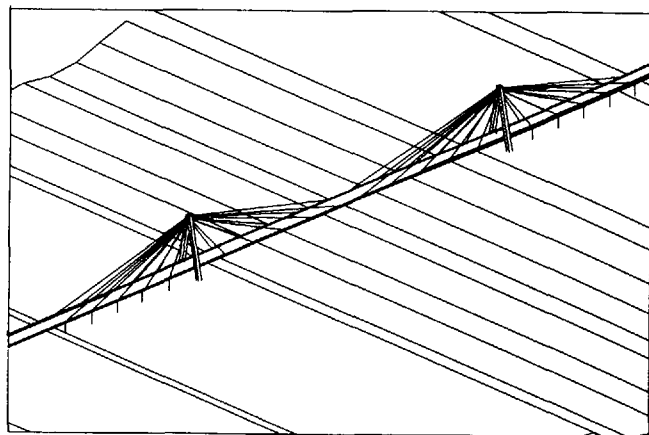


Fig. 6 — The Aberfeldy Golf Club Footbridge.

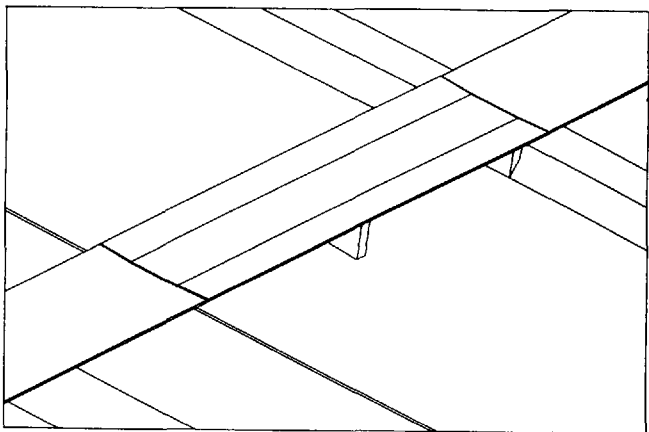


Fig. 7 — The Ulenbergstrasse Bridge.

of the enclosure is composed of pultruded GFRP panels, which are hollow with internal braces.¹⁰

Aramid fiber reinforced plastic (AFRP) tendons were used in a pretensioned concrete slab bridge in Nasu County, Japan. The bridge was completed in 1990 and consists of three spans, each 11.98 m (39.3 ft) long and 2.4 m (7.9 ft) wide. The AFRP tendons are 14.0 mm (0.55 in) in diameter and they were used in only three of the 21 girders. The design load was 94.1 kN (21 kip), which was 50 percent of the ultimate capacity.⁵

The Schiessbergstrasse Bridge in Lovelock, Germany

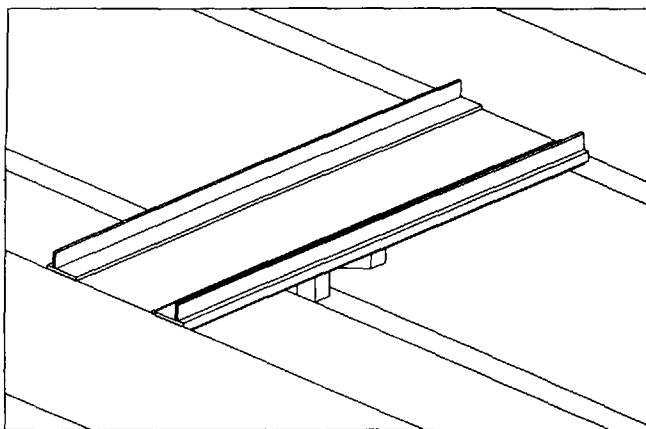


Fig. 8 — The Bachi River Bridge.

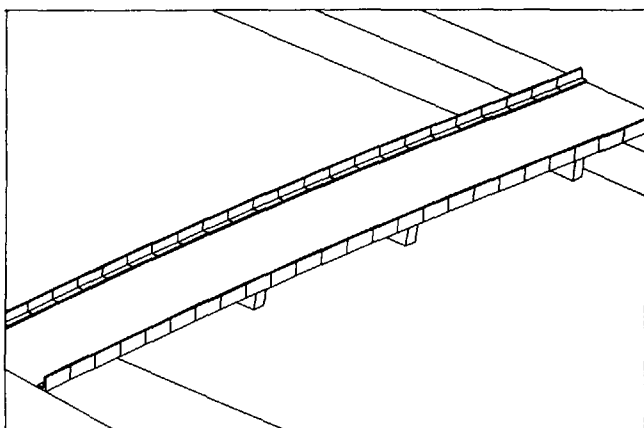


Fig. 9 — The Schiessbergstrasse Bridge.

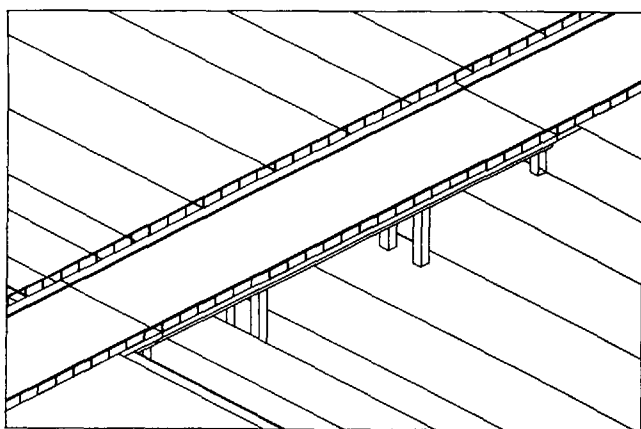


Fig. 10 — The Nötsch Road Bridge.

was built in 1990 (Fig. 9). It is a three-span, solid concrete slab bridge with two side spans of 16.3 m (53.5 ft) each and a 20.4 m (66.9 ft) middle span. The slab is 9.7 m (31.8 ft) wide and the deck is 1.12 m (3.67 ft) thick. Twenty-seven glass-fiber-prestressing tendons with a working load of 600 kN (135 kip) were used in a post-bonding process. Optical fiber sensors were also integrated for remote observation of the stresses.¹³

Construction of the Nötsch Road Bridge in Kärnten, Austria began in 1990 (Fig. 10). The bridge is very similar to the

Schiessbergstrasse Bridge, but has different span lengths and was designed to be partially prestressed using 27 glass-fiber tendons. The side spans are 13 m (42.65 ft), the main span is 18 m (59.1 ft), and the slab thickness is 0.75 m (2.5 ft). The bridge is equipped with optical fiber sensors as well as chemical sensors.¹³

Meier¹ suggested using CFRP as the most economical solution for a super-long span (more than 4,000 m [13,000 ft]) cable-stayed bridge across the Strait of Gibraltar. He provided graphs showing the difference in the limiting span (spans at which a structural system will just support itself) for suspension bridges versus the specific design load for steel and CFRP as well as GFRP. He also showed the price differences between composite materials and steel for different spans, concluding that for spans more than 4,000 m (13,000 ft) long carbon composites are more economical. However, the dynamics of these long span bridges when constructed using the lightweight FRP will need more attention and consideration.¹⁵

Conclusion

In the last two decades new materials such as FRP had been used in many civil engineering applications. Particularly, the use of fiber reinforced plastic, such as glass fiber reinforced plastics (GFRP), Aramid fiber reinforced plastic (AFRP), and carbon fiber reinforced plastics (CFRP), in highway bridges has increased significantly. Most of the GFRP and AFRP structures were built in Europe, China and the United States while CFRPs have been studied extensively in Japan and more recently in Germany.

Although FRPs are expensive compared to steel they provide a sound solution to the deterioration of concrete bridges, preventing the steel corrosion problem. In addition to their corrosion resistance, FRP have a high tensile strength that can provide high prestressing forces. Another advantage is their excellent electromagnetic resistance, which can be utilized in other applications.

It seems that the transfer of the technology from the aerospace, automobile, and sports industries to the civil engineering applications in bridges will be much faster than anticipated and by the turn of the next century FRPs will be well represented in highway bridge applications.

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Selected for reader interest by the editors.

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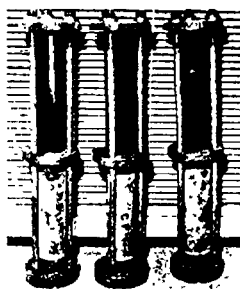
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