

An Innovative Hybrid FRP-Concrete Bridge System

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Synopsis: An innovative corrosion-free system for short- and medium-span bridges consisting of precast prestressed concrete truss girders and cast-*in-situ* concrete deck has been developed. Advantages of the new system include reduced self-weight and enhanced durability. The girders consist of top and bottom concrete flanges connected by precast vertical and diagonal members made of fiber reinforced polymer (FRP) tubes filled with concrete. Glass FRP dowels and corrosion-resistant steel stud reinforcement are used, respectively, to connect the vertical and diagonal members to the concrete flanges. The flanges are pretensioned with carbon FRP tendons. The deck slab is reinforced with corrosion-resistant steel bars in the bottom transverse layer and with glass FRP bars in the bottom longitudinal and the top layers. The girders may be post-tensioned with external carbon FRP tendons to balance the slab weight and to provide continuity in multi-span bridges. The general details of the system and an experimental evaluation of its critical components, namely, the FRP tubes and the truss connection, are presented. Three types of FRP tube and four types of connection are investigated. The results of testing eight connection specimens under static loading are presented. The tests have shown superior performance of the connection when filament wound tubes and continuous double-headed studs are used.

Keywords: bridges; concrete-filled tubes; corrosion-free; deck slab; durability; fiber-reinforced polymers; headed studs; innovative; prestressing; truss girder

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INTRODUCTION

Over the past decade, there has been a significant increase in the application of fiber reinforced polymers (FRPs) for strengthening and rehabilitation of aging and deteriorated infrastructure. However, the use of these innovative materials in new construction has not reached its full potential, mainly because of their high cost in comparison with other conventional construction materials such as concrete and steel. The advantages of FRPs, such as the high strength, light weight, easy handling, enhanced durability and low maintenance, can be more realized when FRPs are used as load-carrying structural elements. Reduction of construction costs can be achieved when the design offers reduced amounts of materials, simplified fabrication and construction procedures, lighter weight and shorter construction time. The use of FRP in new structures, particularly bridges, can be limited if all the structural components are made of FRP, mainly due to its relatively small rigidity. This limitation can be eliminated when the structural system combines FRPs with other conventional materials in a hybrid form.

A few researchers around the world have attempted to develop new bridge systems that employ FRP structural components as load-carrying elements. In Japan, for example, a new composite concrete-FRP prestressed box-girder bridge system has been proposed (Gossila and Yoshioka 2000; Niitani et al. 2001). In this system, the heavy concrete web of conventional box girders is replaced with pultruded glass FRP panels. In the United States, two modular bridge systems have been developed: one is made of FRP box sections supporting conventional concrete slab cast on stay-in-place FRP deck panels (Seible et al. 1998; Cheng et al. 2005), and one consists of carbon fiber shells filled with concrete and used as primary flexural members connected in the span direction along their length to a conventional reinforced concrete slab by means of steel or special FRP dowel connectors

(Zhao et al. 2001a). This latter system was used in construction of the 20 m long Kings Stormwater Channel Bridge on California State Route 86 (Zhao et al. 2001b).

The concrete-filled FRP tube system utilizes the best characteristics of both the FRP and the concrete. Under compression, the tube provides confinement to the concrete core and, hence, increases its compressive load carrying capacity. The concrete on the other hand provides local stability to the thin-walled tube and, hence, prevents premature local buckling (Fam 2000). Under tension, when properly bonded to concrete, the FRP provides corrosion-resistant tensile reinforcement. Use of concrete-filled tubes primarily as flexural members is, however, less efficient than their use as axially-loaded tension or compression members. Fam (2000) has shown that for concrete-filled FRP tubes used as flexural members, the shear strength provided by the tube is small, unless filament-wound tubes with fibers mainly oriented in the circumferential direction are used. Also, the confining effect of the tube on concrete in the compression zone of flexural members is insignificant as compared to that of similar members under pure axial compression. The FRP tubes, however, serve as stay-in-place formwork for the concrete, protect it from the environment, and enhance its durability.

This paper is concerned with the development of a system for short- and medium-span bridges. In the proposed system, the superstructure is built entirely from materials that are not vulnerable to corrosion. The system consists of precast prestressed concrete truss girders and cast-*in-situ* concrete slab (Fig. 1). Each girder has top and bottom concrete bulbs (flanges) connected by precast vertical and diagonal truss members. In addition to concrete, the materials used are FRP and stainless or any other type of corrosion-resistant steel, which are utilized as described in the following section. In addition to being immune to corrosion, the new system is light in weight and durable. The light weight reduces the load on the supports and allows for longer spans, resulting in reduction in the size of the substructure and in the number of supporting piers in multi-span bridges and, hence, reduction in the initial cost. The improved durability reduces the maintenance cost and extends the life span of the structure.

DESCRIPTION OF THE BRIDGE SYSTEM

The Truss Girders

Each truss girder consists of top and bottom concrete bulbs (flanges) connected by precast vertical and diagonal members (Fig. 2a). The concrete bulbs are pretensioned with carbon FRP tendons and provide the flexural resistance of the girder. The bulbs are provided with stainless steel stirrups and non-prestressed glass FRP longitudinal bars at the stirrup corners (Figs. 2b and c). The vertical and diagonal truss members, resisting the shear forces, are made of hollow glass FRP tubes filled with high-strength concrete. The verticals are predominantly in compression and the diagonals are mainly in tension. The diagonals are optimally set at 45 degrees. Both the verticals and the diagonals are produced prior to the bulbs. Glass FRP dowels protrude from the ends of the verticals to connect them to the bulbs (Figs. 2a and b). Stainless steel headed studs (straight bars with anchor heads at one or both ends) connect the diagonals to the bulbs (Figs. 2a and c). Alternatively, the diagonals can be pretensioned with FRP flexible tendons (e.g., Carbon Fiber Composite Cables, known as CFCC, produced by Tokyo Rope, Japan). The pretensioning will provide the diagonals with a reserve tensile capacity in case the FRP

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tubes are damaged, for example, by fire. The tendons protrude from the ends of the diagonals and are bent to serve as dowels connecting the diagonals to the bulbs.

Single or double-headed studs are selected for this project because of their superior anchorage properties. With an area of the head 9 to 10 times the cross-sectional area of the stem, the full yield strength of the studs can develop immediately behind the head, without the need for development length as required in conventional reinforcement (Ghali and Dilger 1998).

For ease in production, it is preferable to cast the bulbs in a rotated position, while the verticals and the diagonals lie on a horizontal surface. In case of damage by fire, the FRP tubes can be easily replaced by wrapping the concrete diagonals and verticals with FRP sheets or jackets. Stainless steel double-headed studs are used to connect the deck slab to the girders (Fig. 2b).

After casting the deck slab the precast truss girders can be post-tensioned by external carbon FRP tendons (CFCC) to counterbalance the slab weight and to provide continuity in multi-span bridges. An external tendon can be harped (held down) to the bottom bulb at one or two points within the span and held up to the top bulb at one point near the intermediate supports in continuous bridges. No deviators are required at the harping points. The horizontal parts of the tendons between the harping points pass through ducts placed inside the bottom bulb, in a single-span bridge, or inside both the top and bottom bulbs in a continuous bridge. Deviation of the tendons from the horizontal is done at the location of the truss joints. The ducts can be left ungrouted for easy replacement of the tendons, or can be grouted to achieve bond between the horizontal parts of the tendons and the concrete bulb(s). Research has shown that partially bonded external tendons enhance both the strength and ductility of externally prestressed concrete members (Hindi et al. 1996).

The Deck Slab

Particular attention is given to the bridge deck slab as it represents an important component that considerably affects the overall cost and quality of the structure. Reinforcement of the deck slab can all be glass FRP bars. Alternatively, the FRP bars can be used in both the transverse and longitudinal directions for the top reinforcement, whereas the bottom reinforcement can be composed of stainless steel bars in the transverse direction (to act as ties in the arch action) and glass FRP in the longitudinal direction (Figs. 2b and c). The glass FRP reinforcement at the bottom longitudinal and top layers is only for crack control and should be of minimum amount.

EXPERIMENTAL PROGRAM

The initial phase of this research involved computer modeling and analysis of the proposed bridge system in order to determine the optimum design of the truss girder in terms of its depth for different ranges of span and loading, dimensions of the top and bottom bulbs, amount and placement of prestressing, size of the truss elements, and number and size of the stud reinforcement required. An extensive experimental program is currently in progress at

the University of Calgary to investigate performance of the various components of the bridge system and to verify the optimum design. The experimental program includes the following:

1. Axial compression tests on samples of different types of concrete-filled FRP tube.
2. Tests on bond between the FRP tube and the concrete.
3. Static and fatigue loading tests on different types of connection between the truss elements and the top and bottom bulbs.
4. Tests on large-scale truss girder specimens covered with or without a concrete slab under static and fatigue loading and long-term effects.

Details of the tests of items 1 to 3 above (except for fatigue loading) are given below.

Compression Tests on Concrete-Filled FRP Tubes

As previously mentioned, under compression, an FRP tube confines the concrete core and increases its load carrying capacity. Tests were conducted on three different types of available glass FRP tubes to determine the enhancement of the compressive resistance that could be achieved and, hence, the most suitable type for the vertical (compression) members of the proposed truss girder. One type of pultruded glass FRP tube and two types of filament-wound tubes were used. The pultruded tube contained approximately 80% of the fibers oriented in the longitudinal direction and 20% in the circumferential direction. One type of the filament-wound tube contained approximately 70% circumferential fibers and 30% longitudinal fibers. The other type contained fibers wound in the diagonal direction at approximately 45 degrees.

The pultruded tube specimens were of 80, 100, and 150 mm nominal inner diameters. All filament-wound tubes were of 85 mm nominal inner diameter. The height of all specimens was set at twice the concrete core diameter. Two groups of specimens were tested. In Group 1, the FRP tube was 10 mm shorter in height than the concrete core. This was achieved by cutting a 5 mm ring from each end of the tube. In this manner, only the concrete core would be in direct contact with the loading plate and the pure confining effect of the tube could be assessed. In Group 2, both the tube and the concrete core had the same height and the compression load was applied on both the concrete and FRP. Table 1 gives nominal dimensions of all the tested specimens. Some specimens of the filament-wound tube were also tested with a rough inner surface consisting of sand mixed with resin adhesive to obtain bond between the concrete and the tube. Plain concrete specimens, with target compressive strength between 30-35 MPa, were also tested for comparison. All specimens were instrumented with strain gauges attached in both the vertical and transverse (circumferential) directions. Figure 3 shows elevations and cross-sections of typical specimens of Groups 1 and 2.

Bond Tests on Concrete-Filled FRP Tubes

Three groups of concrete-filled pultruded tubes were tested to examine means of better bond between the concrete and the tube. In the first group (Type S), the inner surface of the tube was left without treatment for bond and was used as control specimens. In the second group (Type A), the inner surface was coated with a 50 mm wide circumferential strip of sand mixed with resin adhesive. In the third group (Type R) shear connectors were provided in the form of two 60 x 150 mm GFRP strips glued to the inner surface and fixed in position by means of rivets. Figure 4 gives dimensions of both the tubes and the concrete core. All specimens were tested under vertical load applied to the top of the concrete core.

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Connection Tests

The most important component of the truss girder is the connection of the vertical and diagonal elements to the concrete bulbs. Tests were therefore carried out to examine the behavior and efficiency of different types of connection and to determine the best type to be used. Specimens of a single section of truss consisting of one vertical and one diagonal member connected to portions of the top and bottom bulbs were fabricated (Fig. 5). The specimens were sized based on a prototype design for a pedestrian bridge spanning 45 m. The bottom bulb was fixed to the strong floor and a horizontal load was applied to the top bulb to produce a compressive and tensile force in the vertical and diagonal elements, respectively.

Four different reinforcing types were used to connect the diagonal to the concrete bulbs:

1. Unbonded post-tensioned tendon passing through the centre of the diagonal and anchored outside the top and bottom bulbs (Fig. 6a).
2. Long double-headed studs with the heads embedded in the top and bottom bulbs and the stems extending through the length of the diagonal (Fig. 6b).
3. Short double-headed studs at the upper and lower ends of the diagonal, with one head embedded in the top or bottom bulb and the other head embedded in the concrete inside the FRP tube (Fig. 6c).
4. Pairs of single-headed studs, one at each end of the diagonal, with the head embedded in the top or bottom bulb and the stems spliced inside the FRP tube with deformed steel bar of the same diameter as the stem of the stud (Fig. 6d).

A total of eight specimens, two for each connection type, were fabricated and tested under static loading. The first four specimens were built using pultruded FRP tubes of 3 in. and 5 in. nominal diameters for the vertical and diagonal elements, respectively. The inner diameter and wall thickness were, respectively, 80 mm and 4.5 mm for the vertical, and 126 mm and 3.5 mm for the diagonal element. The tensile strength and modulus of elasticity of the FRP tube were, respectively, 400.5 MPa and 25.3 GPa in the longitudinal direction, and 40.8 MPa and 10.15 GPa in the circumferential direction.

In the second four specimens, filament-wound FRP tubes with 70% circumferential fibers and 30% longitudinal fibers were used for the truss elements. Tubes of 3 in. and 4 in. nominal diameters were used for the vertical and diagonal elements, respectively. The inner diameter and wall thickness were, respectively, 84 mm and 1.9 mm for the vertical, and 110 mm and 1.9 mm for the diagonal element. Nominal diameter of 5 in. was not available in this type of tube. Adequate concrete consolidation was achieved and tensile resistance was not affected with the 4-in. tube. The tensile strength and modulus of elasticity of the tube were, respectively, 240 MPa and 20.6 GPa in the longitudinal direction, and 480 MPa and 29.0 GPa in the circumferential direction.

Figure 7 shows the stud reinforcement used in this project. The double-headed studs had plain stem, whereas the single-headed studs had deformed stems. Four studs of 12.7 mm diameter were used in the diagonal of each specimen. Deformed 10M (11.3 mm diameter) were used to splice the single-headed studs. Material properties of the studs and the splice bars are given in Table 2. A single 15 mm 7-wire strand ($A_{ps} = 140 \text{ mm}^2$) was prestressed to $0.6 F_{pu}$ in the specimens with post-tensioned diagonal element.

The compressive strength of concrete used in all specimens was 55 MPa, with 14 mm maximum aggregate size in the flanges, and 10 mm maximum aggregate size in the truss elements in order to obtain complete consolidation of concrete in the tubes. Figures 8 and 9 summarize the fabrication process. The studs were placed in position inside the diagonal tube, with one end of the studs extending from either end of the tube. The truss elements were then precast and placed in position in the connection specimen formwork. The flanges were then cast embedding the studs to connect the truss elements to the flanges. The bottom flange was then post-tensioned with four dywidag threadbars.

During the test, the specimen was bolted to the laboratory strong floor within the loading frame shown in Fig. 10. A horizontal load was applied through an MTS system with two 1.5 MN rams in displacement control. Displacement and rotation transverse to the load direction were prevented by means of lateral supports bearing against Teflon sheets on each side of the top flange allowing free movement only in the load direction (Fig. 10). Loading was applied until failure occurred or the stroke limit of the rams was reached. Loading the specimens with post-tensioned diagonal continued until the tendon reached 85% of its ultimate tensile strength

TEST RESULTS

Compression Tests on Concrete-Filled FRP Tubes

Table 1 summarizes all compression test results. Figure 11 depicts the failure modes of the three types of concrete-filled FRP tubes. In Table 1, the strength of confined concrete, f'_{cc} , is defined as the load carried by the concrete core divided by its cross-sectional area. For Group 2 specimens, this load is calculated from the total measured load less the contribution of the FRP tube in the axial direction. The load carried by the tube is calculated from the measured axial strain in the FRP times its modulus of elasticity in the load direction multiplied by the tube cross-sectional area. Effective confinement is defined as the ratio f'_{cc}/f'_c of the strength of confined concrete to the strength of unconfined plain concrete cylinder.

As can be seen from Table 1, the pultruded FRP tubes provide low effective confinement to the concrete core varying from 15 to 40%. This is mainly due to the small percentage of fibers oriented in the circumferential direction. The confinement is less effective in larger diameter specimens since the tube circumferential stiffness is proportional to $1/R$, where R is the tube radius (Fam 2000). However, the concrete-filled pultruded tubes can be more efficient in carrying compression load when both the concrete and the tube are in direct contact with the load (Group 2). In this case, the longitudinal fibers share the load with the concrete. Because of the small percentage of the circumferential fibers, specimens Group 1 failed by splitting of the tube over its height, whereas specimens Group 2, with both the core and the tube directly loaded, failed by buckling combined with splitting.

The two types of filament wound tubes (FWC and FWD) offer a significant increase in the strength of confined concrete as opposed to the pultruded tubes, even though the tube thickness is much smaller, raising the effective confinement ratio to approximately 3-4.5. This is due to the higher percentage of the circumferential or diagonal fibers. The results also indicate greater increase in strength in specimens with just the concrete core

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loaded (Group 1) than in specimens with both the concrete and the tube loaded (Group 2). This confirms the findings of Fam (2000) and Mirmiran and Shahawy (1997).

Specimens made of the filament-wound tubes, FWC and FWD, exhibited similar modes of failure (Figs. 11b and c), with fibers splitting along a diagonal line over the tube height. Failure of the specimens of FWD type, with diagonal fibers, was drastic and very brittle, with all fibers along the tube length rupturing, blowing out the crushed concrete core and allowing no residual load to be taken. On the other hand, failure of the FWC specimens, containing 30% longitudinal fibers, was less drastic (Fig. 11b) and the tubes continued to offer some confinement and load carrying capacity after the ultimate load was reached. Therefore, of the two types of filament-wound tubes, the FWC type was selected to build the connection specimens of the truss as described above.

Bond Tests on Concrete-Filled FRP Tubes

Table 3 shows results of the bond test carried out on the concrete-filled pultruded FRP tubes. As expected, the shear connectors made of FRP strips fixed to the inner surface by epoxy and rivets provided bond strength between the concrete and the tube higher than the sand/adhesive coating applied to the tube surface. Figure 12 shows failure of the two types of specimens. The increase in strength, however, was only 11%. It was therefore decided to use adhesive coating in the tubes of the truss connection specimens described above.

Connection Tests

Figure 13 compares the load-displacement behavior of the four different connections when pultruded tubes were used. Figure 14 shows the comparison for specimens built with filament-wound tubes. The displacement was measured in the direction of the applied load, at the centre of the top flange, by two transducers located on each side of the flange at the intersection of the truss elements. Failure modes of the connections are shown in Figs. 15-19.

As can be seen, the specimens built with filament-wound tubes showed better performance, particularly in terms of ductility. The maximum load that each connection type sustained was comparable for specimens of each tube type. However, the filament wound tube specimens continued to sustain load under larger horizontal displacement. This is attributed mainly to the increase in concrete strength due to better confinement.

The post-tensioned specimens performed quite similarly. The tendon in both specimens yielded at approximately the same load and displacement. The specimen with filament-wound tubes experienced more displacement after the tendon yielded, and reached a slightly higher applied load under the same tensile force in the tendon.

The connection with long double-headed studs showed the best performance in both types of specimens, with much higher ductility than the other three types of connection. The specimen with pultruded tube ultimately failed due to buckling and premature rupture of the tube of the vertical compression member (Fig. 17). The studs in the diagonal member did not reach their ultimate tensile strength. The specimen with filament-wound tube showed much better performance and continued to sustain load at displacements well beyond the pultruded tube specimen (Fig. 14). In fact, the test was stopped because the stroke limit of

the rams was reached when the horizontal displacement of the specimen was approximately 85 mm. Rupture at the bottom of the vertical tube under compression was also observed.

The connection with spliced single-headed studs sustained a slightly higher load, but exhibited much less ductility and failure was caused by fracture of the studs (Fig. 18). This is possibly because the single-headed studs have deformed stems, whereas the double-headed studs have plain stems. The single-headed studs could develop bond with the concrete and share part of the load with the tube resulting in localized yielding and failure at the connection to the flange. The double-headed studs, on the other hand, developed little bond and were free to yield over a longer length (Fig. 16).

The short double-headed stud connections failed well below their yield strength. In the pultruded specimens, the FRP tube cracked longitudinally in the diagonal member, most likely due to circumferential stress induced from the transfer of force from the stud head. In the filament-wound tube specimens, the tube was able to withstand the circumferential forces and no failure in the tube took place. However, the concrete inside the tube cracked at the end of the head and the core ultimately slipped from the tube (Fig. 19). The series of peaks and drops in the load-displacement curve (Fig. 14) was due to successive failure of bond between the concrete and the tube at the locations of the sand-adhesive coating of the tube inner surface.

SUMMARY AND CONCLUSIONS

An innovative corrosion-free bridge system that combines precast prestressed hybrid FRP-concrete truss girders and a cast-*in-situ* concrete deck has been developed. The web members of the truss girders are made of concrete-filled FRP tubes connected to the prestressed concrete top and bottom flanges of the truss by means of single- or double-headed studs. Compression tests were performed on small FRP tubes filled with concrete in order to examine the effect of confinement on the truss compression members. Tests were also carried out on a single section of the truss in order to investigate the performance of different types of connections. It has been shown that filament wound GFRP tubes with fibers oriented in the circumferential direction perform extremely well in confining concrete in the truss compression members, and create a ductile connection that continues to take load well beyond yielding. Because of their excellent anchorage properties, continuous double-headed studs or spliced single-headed studs can be used efficiently to connect the truss tension members to the flanges. The studs have shown better performance than post-tensioned tendons. The studs can resist the required tensile forces and provide much more ductile connections. It is concluded that the proposed bridge system can be an efficient and more economical superstructure for short- and medium-span bridges.

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Table 1 – Results of compression tests on concrete-filled FRP tubes

FRP tube type	Specimen Designation*	FRP tube size (mm)	Area of FRP (mm ²)	Concrete size (mm)	Area of concrete (mm ²)	P_{max} (kN)	f'_c or f'_{cc} (MPa)	$\frac{f'_{cc}}{f'_c}$
Pultruded (80 percent longitudinal fibers)	PC-80	—	—	75x150	4418	131.00	29.7	1.00
	PT1-80	80x150x4.5	1195	80x160	5027	210.46	41.9	1.41
	PT2-80	80x160x4.5	1195	80x160	5027	386.87	62.2	2.10
	PC-100	—	—	100x200	7854	261.95	33.4	1.00
	PT1-100	102x200x3.5	1160	102x210	8171	341.46	41.8	1.25
	PC-150	—	—	150x300	17671	583.60	33.0	1.00
	PT1-150	150x300x4.5	2184	150x310	17671	673.25	38.1	1.15
Filament wound (70 percent circumferential fibers)	PC-100	—	—	100x200	8107	278.60	34.4	1.00
	FWC1-85S	84x150x1.8	485	84x160	5542	671.54	121.2	3.53
	FWC2-85S	84x160x1.8	485	84x160	5542	667.34	111.9	3.26
	FWC1-85R	84x150x1.8	485	84x160	5542	646.47	116.7	3.39
Filament wound (diagonal fibers)	FWC2-85R	84x160x1.8	485	84x160	5542	576.41	96.7	2.81
	PC-100	—	—	100x200	8107	237.80	29.3	1.00
	FWD1-85S	85x160x2.0	547	85x170	5674	676.21	119.2	4.06
	FWD2-85S	85x170x2.0	547	85x170	5674	657.73	113.5	3.87
	FWD1-85R	85x160x2.0	547	85x170	5674	731.54	128.9	4.40
	FWD2-85R	85x170x2.0	547	85x170	5674	657.42	113.5	3.87

* The specimen designations PC, PT, FWC and FWD refer, respectively to plain concrete, pultruded tube, filament-wound tube with 70% circumferential fibers, and filament-wound tubes with diagonal fibers. The indices 1 and 2 refer to specimens Group 1 (only concrete core loaded) and Group 2 (both tube and concrete core loaded), and S and R refer, respectively, to smooth and rough conditions of the tube inner surface.

Table 2 – Properties of stud reinforcement and splice bars

Reinforcement Type	Modulus of Elasticity (MPa)	Yield Strength (MPa)	Ultimate Strength (MPa)
Long double-headed stud	192 200	500	589
Short double-headed stud	199 000	507	602
Single-Headed Stud	184 400	504	570
10M deformed bar	201 800	445	509

Table 3 – Results of bond tests on concrete-filled FRP tubes

Specimen Designation*	Tube inner surface condition	Contact area (mm ²)	Maximum Load (kN)	Bond Stress (MPa)
S-100	Smooth	63668	4.98	0.08
A-100	Adhesive	16022	75.21	4.69
R-100	Rivets	18000	93.89	5.22

* Specimen dimensions are given in Fig. 4.

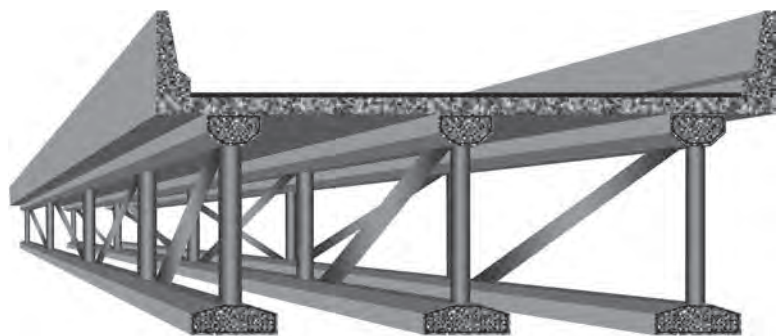


Figure 1 – Perspective view of the proposed hybrid FRP-concrete bridge system

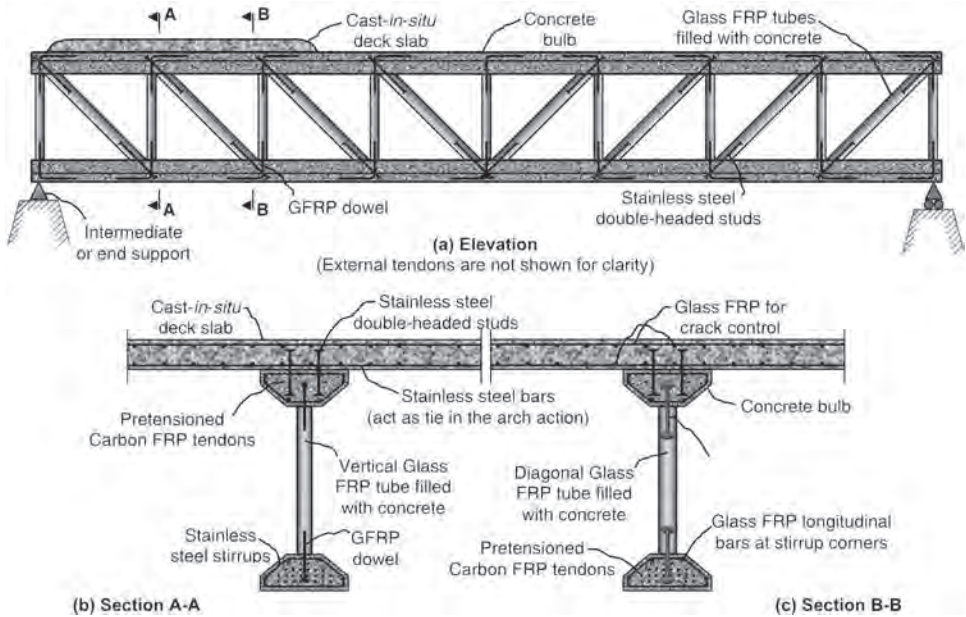


Figure 2 – Details of the proposed hybrid FRP-concrete bridge system

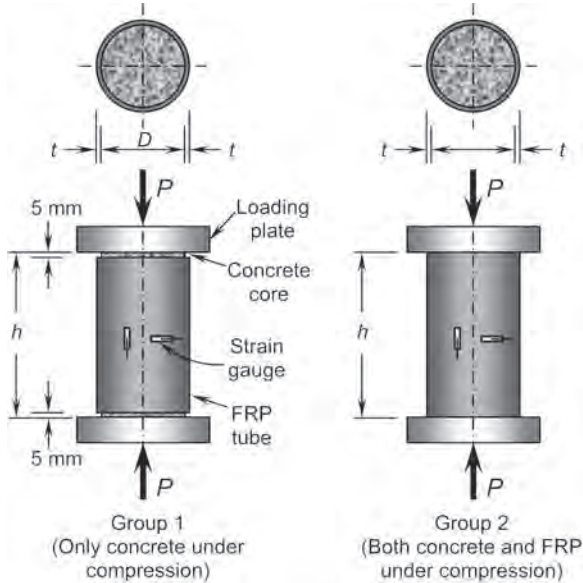


Figure 3 – Concrete-filled FRP tube compression test

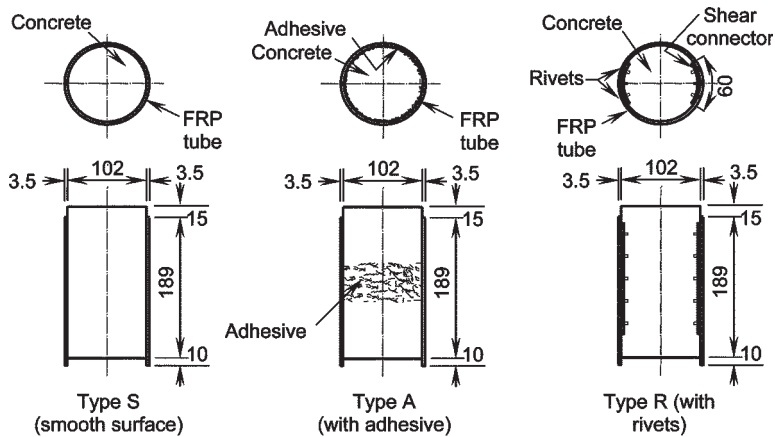


Figure 4 – Concrete-filled FRP tube bond test specimens

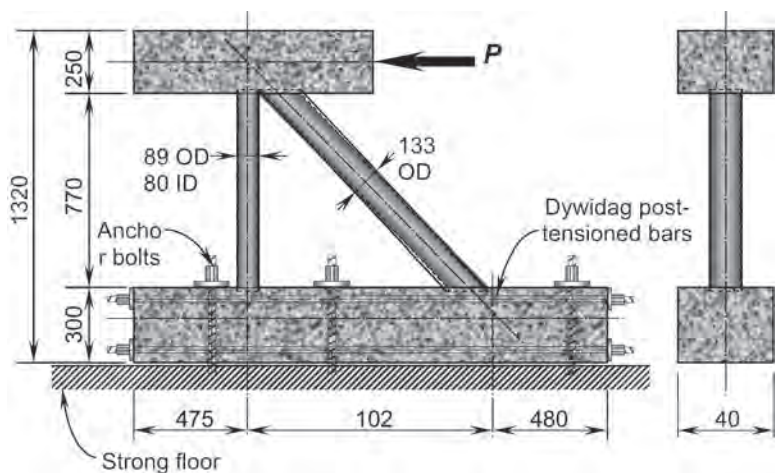


Figure 5 – Connection specimen

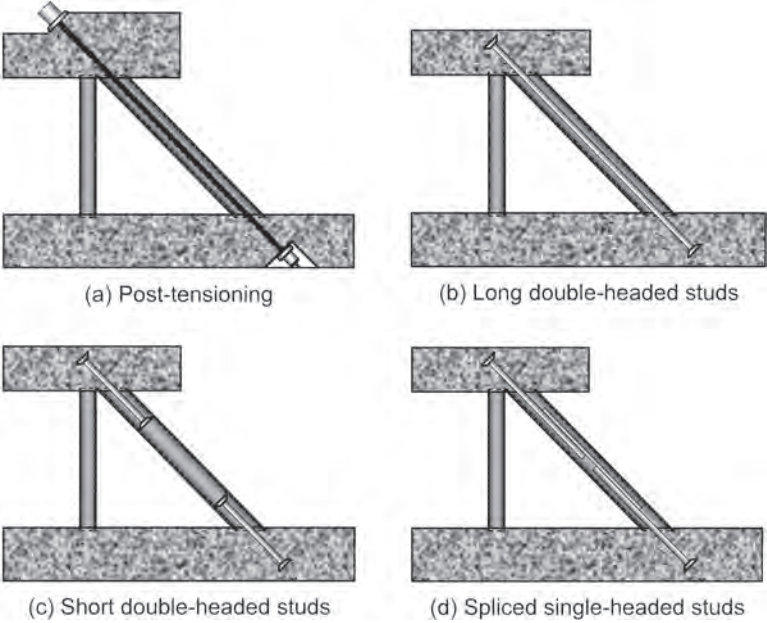


Figure 6 – Different types of connection for diagonal members

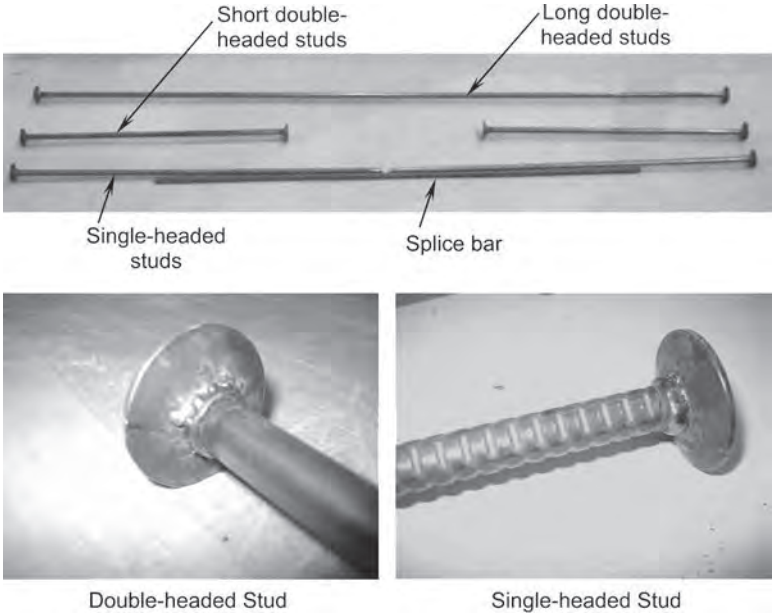


Figure 7 – Stud reinforcement used in different connections



Figure 8 – Preparation of the stud reinforcement and placement inside the FRP tube

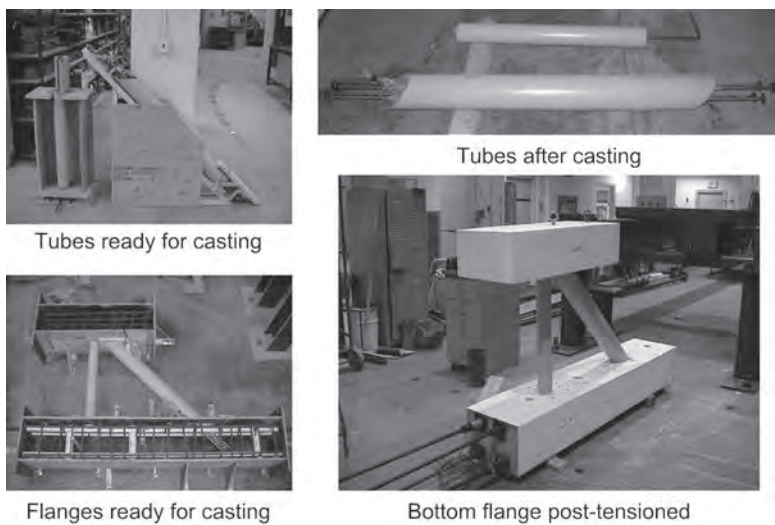


Figure 9 – Casting and prestressing of the connection specimen

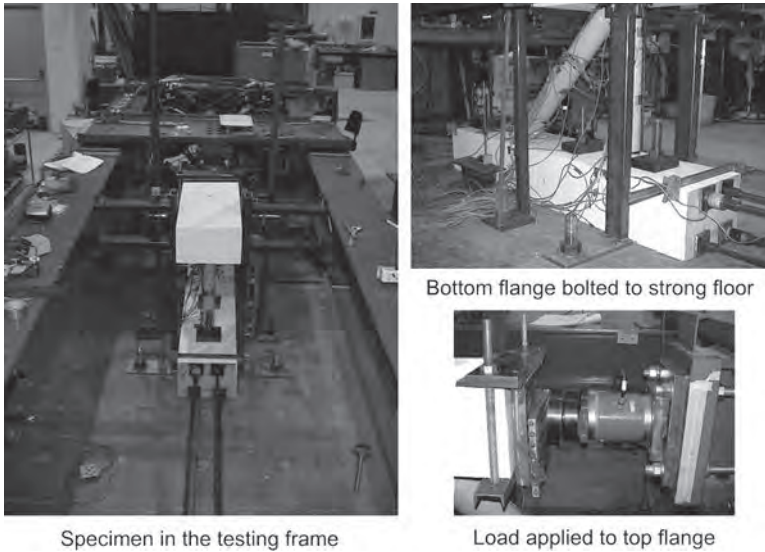


Figure 10 – Connection specimen during static loading

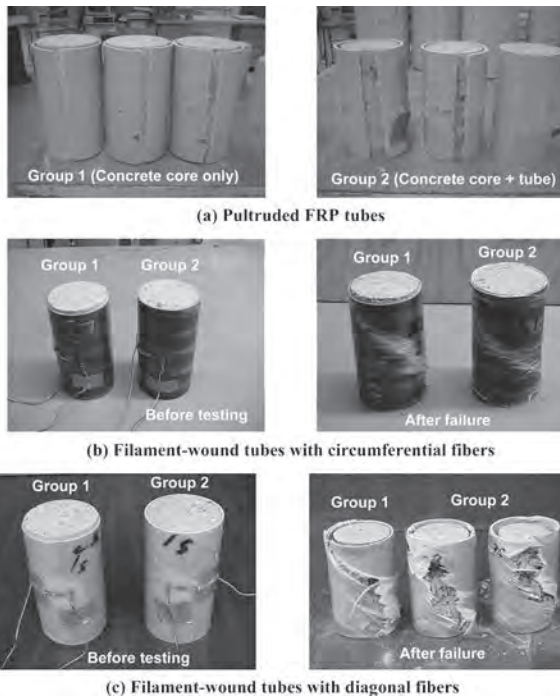


Figure 11 – Failure of different types of concrete-filled FRP tubes in compression test

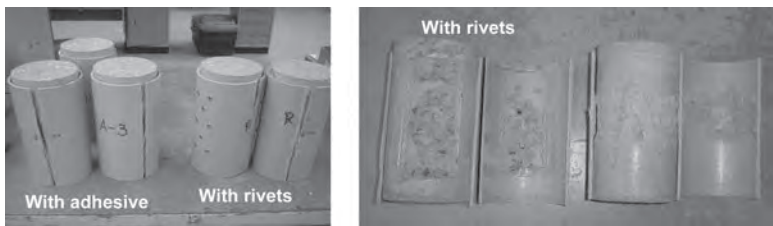


Figure 12 – Failure of concrete-filled pultruded FRP tubes in bond test

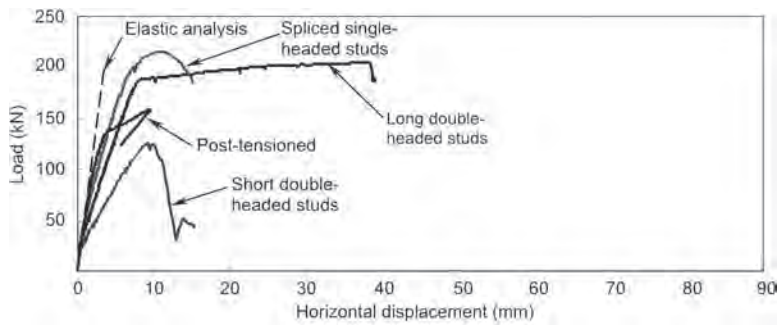


Figure 13 – Load-displacement behavior of pultruded tube connection specimens

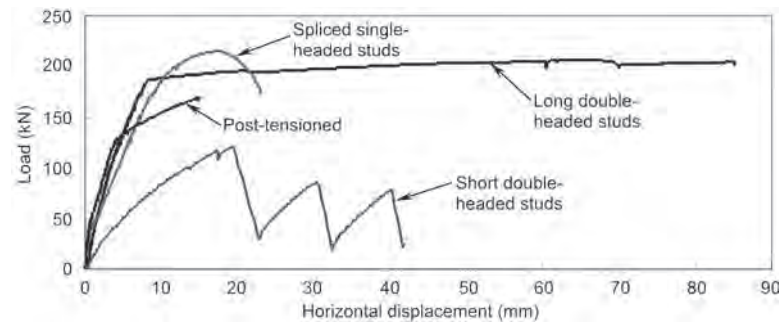


Figure 14 – Load-displacement behavior of filament-wound tube connection specimens



Figure 15 – Failure of connection specimens reinforced with long double-headed studs

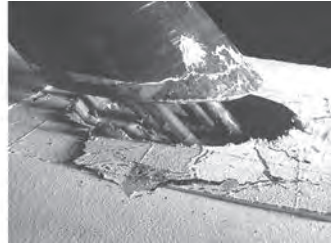


Figure 16 – Failure of diagonals in connections reinforced with long double-headed studs



Figure 17 – Failure of verticals in connections reinforced with long double-headed studs

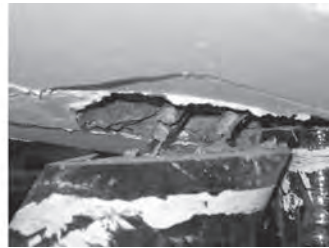


Figure 18 – Fracture of spliced single-headed studs at top or bottom of diagonals



Figure 19 – Failure of connections reinforced with short double-headed studs

Proposed GFRP Connectors in Sandwich Panels

by W.A. Pong, A.F. Morgan Girgis, and M.K. Tadros

Synopsis: In this paper, a precast concrete sandwich panel system with high thermal resistance and optimum structural performance was developed. Fiber-reinforced polymer was used to provide the connector in this panel system. Each connector was prefabricated in a V-shape with both end hooks providing anchorage in the reinforced concrete wythes. The developed shear connecting system is described, along with its performance and advantages. An experimental investigation of the connecting system was conducted. The experimental program included pure-shear testing of push-off specimens with a variety of wythe connectors, flexure testing of full-scale specimens with FRP connectors, and testing of FRP material. The developed shear transfer and degree of composite were evaluated and the observed behavior of the members is presented. Design criteria are given for use by engineers and others to apply with appropriate engineering judgment.

Keywords: composite action; GFRP; horizontal shear; precast; sandwich panels

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INTRODUCTION

Typical precast concrete sandwich panel (PCSP) systems¹ are composed of two concrete wythes with insulation placed at the middle, used for exterior walls. Concrete wythes are connected by metal or plastic connectors. PCSP systems can be constructed to be fully composite structural members depending on the capacity of the connectors to transfer horizontal shear generated by flexure. PCSP systems can be divided into three major categories:²

1. Fully composite panel
2. Non-composite panel
3. Partially composite panel

PCSP systems are considered to be structurally and thermally efficient elements; however, the effective thermal resistance can be reduced by heat transfer through metal connectors.³ The objective of this research to study the use of glass fiber-reinforced polymer (GFRP) as a shear transfer connector. GFRP is well-known as a high tensile strength and low-conductive material.

EXPERIMENTAL PROGRAM

Testing Program

The testing program consisted of push-off and full scale flexural tests. A push-off test was designed to investigate the shear transfer capacity of the various types of shear connectors on the sandwich panel. A full-scale test was designed to investigate the composite behaviors and the degree of composite action provided by the shear connector.

The various parameters considered were concrete strength, thickness of concrete wythes, insulation, reinforcement ratio, and the effect of transverse reinforcement provided in the hook of the GFRP connector.

Push-off Specimens

The size of each push-off specimen was 2 by 4 ft comprising three concrete wythes and two insulation wythes. To compare shear transfer capacity, three material types of connectors: GFRP, welded wire and GFRP in alkali resin were used. Thirty specimens were produced, six specimens with 21 in. thickness and twenty four specimens with 13.5 in. thickness.

Full-Scale Specimens

Six specimens were produced. Each full-scale specimen was 12 in. thick, comprising 3 in. reinforcement concrete wythes and a 6 in. insulation wythe connected by a GFRP shear connector. Each concrete wythe contained a No.3 bar, Grade 60, with 4 inch spacing in both the longitudinal and transversal directions.

Three types of full-scale specimens were constructed, with two specimens of each type. Specimen serial I was horizontally constructed. Concrete wythes were connected by two rows of 2 ft. spacing with a single shear connector placed in a longitudinal direction. Specimen serial II was also horizontally constructed. Concrete wythes were connected by two rows of 2 ft. spacing with double connectors. Specimen serial III was vertically constructed with a single shear connector.

Connector Details

Three types of shear connectors were used. A GFRP-connector is a new type of fiberglass connector (see Figs. 5 through 7) that transfers both horizontal shear force and tension force between the concrete wythes. The other two commercial types of connectors are 1) GFRP embedded in an alkali resistant resin (Fig. 8), and 2) welded wire girder (Fig. 9).

PUSH-OFF TESTING

Test Setup

A test frame was installed at the structural laboratory of the University of Nebraska, Omaha. The test frame was anchored to a 500-kip tie-down accessory on the structural testing floor by two high-strength steel threaded rods. The test set up for the push-off specimen is shown in Fig. 10. The specimen was placed vertically in the test frame. Steel tubes, 4 by 4 in., were placed underneath the outer concrete wythes and another steel tube was placed on the middle concrete wythe. Rubber sheets were inserted between the steel tubes and concrete wythes.

The specimens were tested using a 400 kip hydraulic jack. The attached load cell was used to measure the force applied to the specimens.

The specimens were subjected to a concentrically vertical load applied at the middle concrete wythe, which was incrementally increased until failure occurred. The specimens failed when the applied load was reduced from its peak point.

FULL-SCALE FLEXURAL TESTING

Test Setup

The test set-up for the full-scale specimens is shown in Fig. 11. Each specimen was placed horizontally in the test frame. A deflection-measuring device was attached at the center of the bottom concrete wythe.

The specimens were tested by using a 400-kip hydraulic jack placed at the center of the panel. The attached load cell was used to measure the force applied to the specimens.

The specimens were then subjected to a vertical load. The load was incrementally increased until failure occurred. Failure of the specimens was regarded when the applied load was reduced from its peak point. The specimens were inspected and observed at specified intervals during testing.

MATERIAL PROPERTIES

Compressive strength and Modulus of Elasticity

The final compressive strength was obtained from compressive tests of concrete cylinders at the same age as their corresponding specimens. The modulus of elasticity was computed as $E_c = 57,000\sqrt{f'_c}$

TENSILE TESTING OF GFRP RODS

Tensile testing of GFRP rods was performed according to the Guide Test Methods for Fiber Reinforced Polymers (FRP) for Reinforcing or Strengthening Concrete Structures prepared by ACI Subcommittee 440.⁴

A 48-in. long GFRP rod was attached to the anchorage device to facilitate the gripping dimension, as shown in Fig. 12. An Extensometer, a strain measuring device, was mounted at the middle of the specimen, as shown in Fig. 13. Six specimens were tested under a loading rate of 6000 lbs/min. The test results are shown in Table 5. Tensile strength, ultimate strain and modulus of elasticity were calculated as follows:

$$\text{Tensile strength} \quad f_u = \frac{F_u}{A}$$

$$\begin{aligned} \text{where} \quad f_u &= \text{tensile strength (psi)} \\ F_u &= \text{load failure (lbs)} \\ A &= \text{average cross section area (in}^2\text{)} \end{aligned}$$

$$\text{Tensile modulus of elasticity} \quad E_L = \frac{F_1 - F_2}{(\varepsilon_1 - \varepsilon_2)A}$$

$$\begin{aligned} \text{where} \quad E_L &= \text{axial modulus of elasticity (psi)} \\ A &= \text{cross section area (in}^2\text{)} \end{aligned}$$

F_1 and ε_1 = load and corresponding strain, respectively, at approximately 50% of the ultimate tensile capacity or guaranteed tensile capacity

F_2 and ε_2 = load and corresponding strain, respectively, at approximately 20% of the ultimate tensile capacity or guaranteed tensile capacity

$$\text{Ultimate strain} \quad \varepsilon_u = \frac{F_u}{E_L A}$$

where ε_u = ultimate strain of FRP bar

Ductility is the ratio of the strain at failure to the strain at yielding. Fig. 14 shows the stress-strain relationship obtained from the tensile test. Stress and strain increased in a linear proportion to failure with no yield point. The test results may be different since the tensile strength and modulus of elasticity of FRP depend on the fiber content and manufacturing process, as well as quality control.

PUSH-OFF TEST RESULTS

The failure behavior of specimens serial NU-R, NU-C, NU-CX was very similar. Failure of GFRP connector was brittle failure due to shear and flexure, as seen in Figs. 15 (a) and (b). When loading was in process, the applied load was abruptly dropped and then increased after each connector failure until ultimate load was reached.

The test results of the push-off specimens are summarized in Table 6. The relationship between ultimate load and total area of connector was computed as the ultimate load divided by area (wire or GFRP).

FULL-SCALE TEST RESULTS

The failure behavior of specimens was consistent. Visual cracks formed at the bottom fiber when the applied load was in the range of 12-15 kips. Horizontal shear failure occurred when the outermost connector failed. Crushing of concrete due to flexure failure occurred in the top concrete wythe before the second row of connectors failed. Flexure failure took place in the bottom concrete wythe when the ultimate load was reached.

Observations revealed that loading did not continuously increase. Similar to the push-off test, the applied load abruptly dropped and then increased at every single connector failure until ultimate load was reached.

The test results of the full-scale specimens in terms of ultimate load and deflection are given in Table 7.

DISCUSSION

Deflections

The load and mid-span deflection of the six specimens is shown in Fig. 16. Also shown in Fig. 16 are theoretical predictions which were calculated based on the assumption of fully composite behavior and non-composite behavior. Our predicted deflections, based on the assumption of fully composite using Equation 4 defined by ACI 318-02,⁵ were underestimated. This is attributed to the fact that behavior of specimens is

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partially composite. Another reason for the larger deflection in the sandwich panels with the GFRP is that the low tensile modulus of elasticity of the GFRP connectors could lead to more shear deformation that was not included in the deflection equation. To predict deflection of sandwich panels using GFRP connector, more accurate analysis procedure is required. An average modulus of elasticity is 6.548×10^6 psi obtained from GFRP tensile test results as shown in Table 5.

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \quad I_e \leq I_g \quad (1)$$

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (2)$$

$$f_r = 7.5 \sqrt{f'_c} \quad (3)$$

$$\Delta_i = \frac{5 M_a L^2}{48 E_c I_e} \quad (4)$$

where

E_c = modulus of elasticity of concrete, psi.

f'_c = specified compressive strength of concrete, psi.

f_r = modulus of rupture of concrete, psi.

I_{cr} = moment of inertia of cracked section transformed to concrete, in.⁴

I_e = effective moment of inertia for computation of deflection, in.⁴

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.⁴

L = span length of panel

M_{cr} = cracking moment

M_a = maximum moment in member at stage deflection is computed

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension, in.

Δ_i = deflection at mid span, in.

Development of GFRP reinforcement

The test results show that no direct pull-out failure occurred. The average ultimate load of specimens serial NU-C and NU-CX was 34,526 lbs and 33,631 lbs respectively. Transverse reinforcement provided in the hook, specimen serial number NU-C, did not affect the bond characteristics of the GFRP connector. It can be concluded from the test results that 85 degree connector hooks provide sufficient strength to resist force that may cause pull-out failure in concrete wythes. However, precaution in quality control should be taken to prevent direct failure when the hook is not fully embedded in concrete.

Shear transfer

The ultimate loads obtained from the push-off and full-scale tests were much lower than those computed using Equation (5) or (6) defined by ACI 318-02. The test results showed that the GFRP connector did not fail by tension. That can be explained by the

fact that in the sandwich panel there is no direct contact between the layers. Direct contact can take some of the shear stresses, as can be deduced from the first term of the ACI empirical equation 6. In addition, the contribution of steel reinforcement to shear strength is represented in the second term of the equation. The presence of the contact surface at higher level of stresses can cause separation perpendicular to the layers, which leads to yielding in steel reinforcement as in equation 5.⁶ On the other hand, in the sandwich panel case, there are moments and shear forces created in the connectors in addition to the axial forces that are created from the formed truss action shown in Fig. 17. Because of that significant difference, which was confirmed by testing results, it is not recommended to use the ACI horizontal shear equation.

In the formed truss action, one link of the connector carries tensile force and the second takes the compression force as shown in Fig. 17. The compression strength of the GFRP is limited because GFRP has high tensile strength but low compressive and shear strength. Tensile strength in the range of 80-130 ksi and compressive strength in the range of 46-68 ksi have been reported for GFRP reinforcing bars. Higher compressive strengths are expected for bars with higher tensile strength.⁷

Shear friction equation:

$$V_n = A_{vf} f_y (\mu \sin \alpha_f + \cos \alpha_f) \quad (5)$$

Horizontal shear strength equation:

$$V_{nh} = (260 + 0.6 \rho_v f_y) \lambda b_v d < 500 b_v d \quad (6)$$

Where

A_{vf} = area of shear-friction reinforcement, in.²

b_v = width of cross section at contact surface being investigated for horizontal shear, in.

d = distance from extreme compression fiber to centroid of tension reinforcement for entire composite section, in.

V_n = nominal shear strength, lb

V_{nh} = nominal horizontal shear strength, lb

μ = coefficient of friction

α_f = angle between shear-friction reinforcement and shear plane

ρ_v = ratio of tie reinforcement area to area of contract surface

λ = correction factor related to unit weight of concrete

Composite behavior

As shown in Table 6, the average stress of GFRP reinforcement is 31,482 psi and 38,727 psi for specimens with 6 in. and 3 in. insulation, respectively. This average stress is much lower than that of the average tensile stress of 141,212 psi, as shown in Table 5. Therefore, the GFRP connection did not fail in tension but rather in flexure-shear or compression.

Similar to the push-off test results, the average ultimate load obtained from full scale flexure tests is lower than the predicted load, calculated based on the assumption of fully composite behavior. This again can be explained by fact that the failure happened in the

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connectors due to moment and shear rather than tensile stresses. The current equation for predicting sandwich panel strength assumes that connectors carry tensile stresses, which results in overestimating the overall panel strength.

For example, consider the sandwich panel subjected to a vertical load, as shown in Fig 17. The applied load would create tension in the bottom fiber and compression in the top fiber, thereby causing slip in the bottom concrete,⁸ insulation and top concrete wythe. The separation would create flexure and tensile stress in the connector which, in turn, would create compressive stress in another leg of the connector. The tensile stress, compressive stress and flexure occur at the same time; therefore the lowest strength governs the strength of the connector.

More experimental data are needed to verify flexure and compressive stress under the above-mentioned assumption. With lack of flexural and horizontal strength test data, designers can only use the lower bound of these test results to predict the capacity of the sandwich panel, which is constructed of the same material. Precaution should be taken in designing the thickness of the panel.

CONCLUSION AND RECOMMENDATION

The results of the presented experimental program indicate that about 75% of the full composite strength was achieved by using NU connectors every 2x2 ft. It is suggested that the number of NU connectors per square footage should be increased to achieve full composite action.

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Table 1. Push-off specimen dimensions

	Serial No				
	NU-R	NU-C	NU-CX	DT-C	MC-C
Concrete wythe thickness (in.)	3.0	2.5	2.5	2.5	2.5
Insulation thickness (in.)	6.0	3.0	3.0	3.0	3.0
Total thickness (in.)	21.0	13.5	13.5	13.5	13.5
No. of connectors	4	4	4	4	2
Type of connectors	GFRP	GFRP	GFRP	GFRP	WW
No. of specimens	6	6	6	6	6

Table 2. Full-scale specimen dimensions

	Serial No		
	I	II	III
Specimen size 4 by 10 ft			
Concrete wythe thickness (in.)	3.0	3.0	3.0
Insulation thickness (in.)	6.0	6.0	6.0
Total thickness (in.)	12.0	12.0	12.0
No. of connectors	10	20	10
No. of specimens	2	2	2
Construction method	Horizontally	Horizontally	Vertically

Table 3. Final compressive strength and modulus of elasticity of push-off specimens

Specimen	NU-R		NU-C		NU-CX		MC-C		DT-C	
	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)
Concrete wythes										
Layer 1	5,954	4,398	5,954	4,398	5,952	4,398	5,954	4,398	4,890	3,986
Layer 2	9,052	5,423	9,052	5,423	5,662	4,289	9,052	5,423	5,952	4,398
Layer 3	8,409	5,227	8,409	5,227	5,662	4,289	8,409	5,227	5,952	4,398

Table 4. Final compressive strength and modulus of elasticity of full-size specimens

Specimen	NU-R-I		NU-R-II		NU-R-III	
	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)	f'_c (psi)	E_c (ksi)
Concrete wythes						
Layer 1	4,890	3,986	6,063	4,438	6,531	4,606
Layer 2	5,662	4,289	6,063	4,438	6,531	4,606

Table 5. Tensile test results

Sample No	Average Gross section area (in ²)	Load Failure (lbs)	Tensile Strength (psi)	Ultimate Strain (in/in)	Modulus of Elasticity (psi)
1	0.11	15,610	141,909	0.0222	6.736x10 ⁶
2	0.11	15,440	140,364	0.0145	6.202x10 ⁶
3	0.11	15,500	140,909	0.0142	6.514x10 ⁶
4	0.11	15,310	139,182	0.0284	6.628x10 ⁶
5	0.11	15,950	145,000	0.0466	6.483x10 ⁶
6	0.11	15,390	139,909	0.0194	6.726x10 ⁶
Ave.		15,533	141,212	0.0242	6.548x10 ⁶

Table 6. Push-off test results

Specimen					Connector		Test results	
Serial No.	No.	Thickness		f'_c (psi)	Type	Area (in ²)	Ultimate Load (lbs.)	$\frac{P_u}{A_s}$ (psi.)
		Con.	Ins.					
NU-R	1	3	6	5,954	GFRP	0.88	25,417	28,883
	2	3	6	5,954	GFRP	0.88	26,233	29,810
	3	3	6	5,954	GFRP	0.88	24,910	28,307
	4	3	6	5,954	GFRP	0.88	33,923	38,549
	5	3	6	5,954	GFRP	0.88	25,130	28,557
	6	3	6	5,954	GFRP	0.88	30,613	34,788
							27,704	31,482
NU-C	1	2.5	3	5,954	GFRP	0.88	37,148	42,214
	2	2.5	3	5,954	GFRP	0.88	31,790	36,125
	3	2.5	3	5,954	GFRP	0.88	40,191	45,672
	4	2.5	3	5,954	GFRP	0.88	36,524	41,505
	5	2.5	3	5,954	GFRP	0.88	28,306	32,166
	6	2.5	3	5,954	GFRP	0.88	33,201	37,728
							34,526	39,235
NU-CX	1	2.5	3	5,662	GFRP	0.88	27,798	31,589
	2	2.5	3	5,662	GFRP	0.88	30,709	34,897
	3	2.5	3	5,662	GFRP	0.88	36,405	41,369
	4	2.5	3	5,662	GFRP	0.88	34,406	39,098
	5	2.5	3	5,662	GFRP	0.88	37,058	42,111
	6	2.5	3	5,662	GFRP	0.88	35,413	40,242
							33,631	38,218
DT-C	1	2.5	3	4,890	GFRP	1.25	25,123	20,098
	2	2.5	3	4,890	GFRP	1.25	26,196	20,957
	3	2.5	3	4,890	GFRP	1.25	28,257	22,606
	4	2.5	3	4,890	GFRP	1.25	22,727	18,182
	5	2.5	3	4,890	GFRP	1.25	23,932	19,146
	6	2.5	3	4,890	GFRP	1.25	24,079	19,263
							25,052	20,040
MC-C	1	2.5	3	5,954	WW	0.64	37,089	57,952
	2	2.5	3	5,954	WW	0.64	37,971	59,330
	3	2.5	3	5,954	WW	0.64	39,669	61,983
	4	2.5	3	5,954	WW	0.64	40,410	63,141
	5	2.5	3	5,954	WW	0.64	38,081	59,502
	6	2.5	3	5,954	WW	0.64	35,803	55,942
							38,170	59,641

Area = Total area of diagonal welded wire or GFRP

NU Area of GFRP = $(2 \times 4 \times 0.11) = 0.88 \text{ in}^2$ (0.375 in Dia.)

MC Area of Welded wire = $(2 \times 8 \times 0.04) = 0.64 \text{ in}^2$ (0.225 in Dia.)

DT Area of GFRP = $(4 \times 2.5 \times 1.25) = 1.25 \text{ in}^2$ (0.125 in Thickness)

Table 7. Full-scale flexure test results

Specimen				Connector		Predicted Load (lbs)	Test results		
Serial No.	No.	Thickness		f'_c (ksi)	Type		Area (in ²)	Ultimate Load (lbs.)	Deflection (in)
		Con.	Ins.						
I	1	3	6	4,890	GFRP	1.1	27,200	19,399	2.81
	2	3	6	4,890	GFRP	1.1	27,200	20,931	2.28
II	3	3	6	6,063	GFRP	2.2	27,300	23,891	2.55
	4	3	6	6,063	GFRP	2.2	27,300	20,681	2.95
III	5	3	6	6,531	GFRP	1.1	27,330	16,095	3.12
	6	3	6	6,531	GFRP	1.1	27,330	20,956	2.00
Ave.							27,276	20,325	2.13

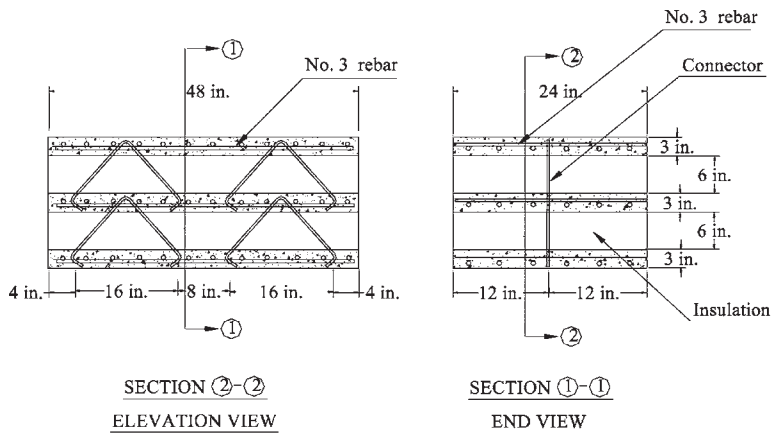


Fig. 1: Details of 2 ft. by 4 ft. by 21-inch push-off specimens, FRP connectors

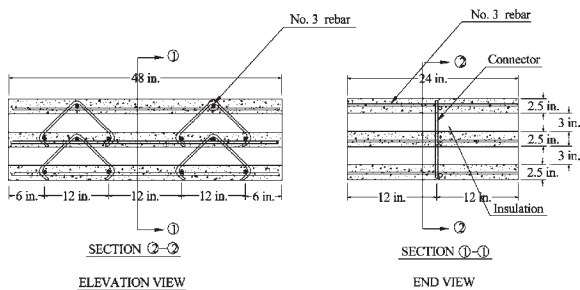


Fig. 2: Details of 2 ft. by 4 ft. by 13.5-inch push-off specimens, FRP connectors

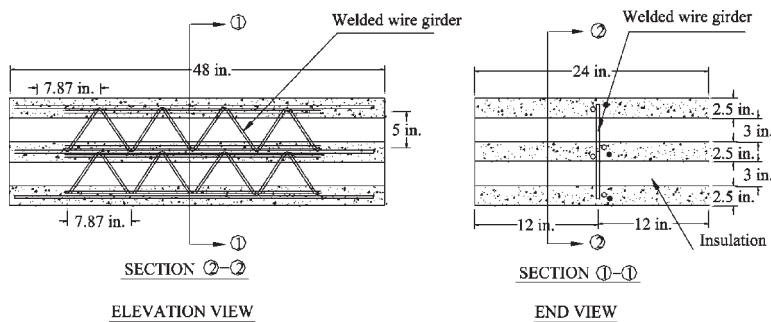


Fig. 3: Details of 2 ft. by 4 ft. by 13.5-inch push-off specimens, welded wire girder connectors

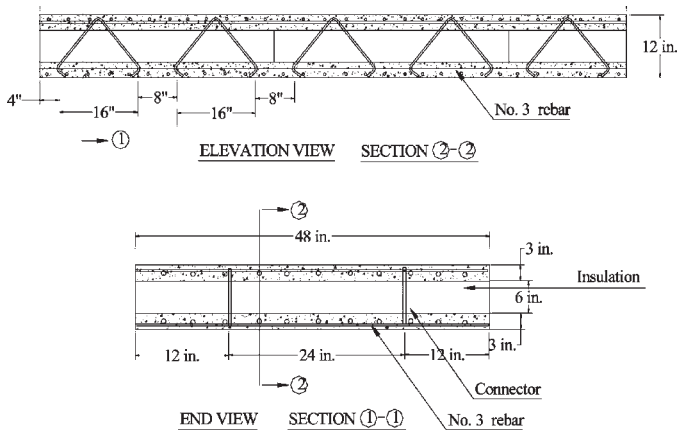


Fig. 4: Details of full-scale specimen

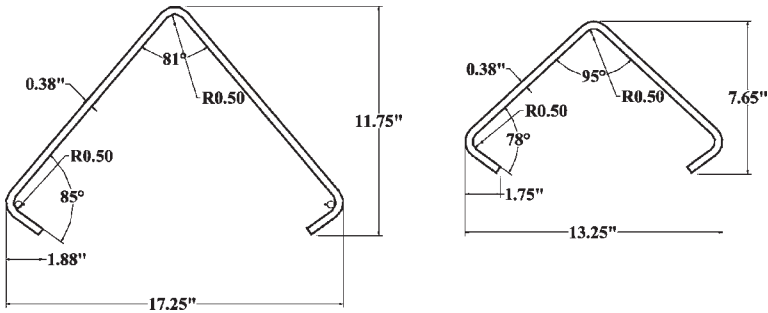


Fig. 5: Details of FRP connectors



Fig. 6: FRP connector 3-6-3



Fig. 7: FRP connector 2.5-3-2.5

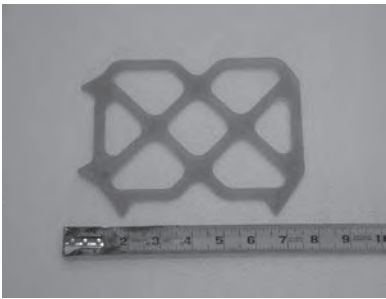


Fig. 8: FRP tie connector



Fig. 9: Welded wire girder connector

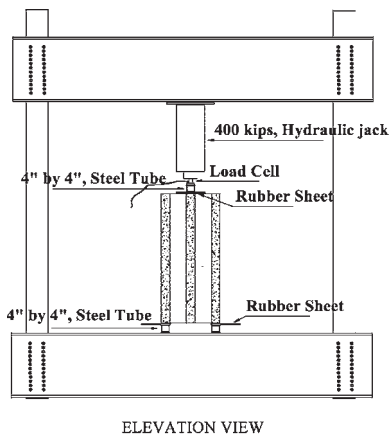


Fig. 10: Push-off test set-up

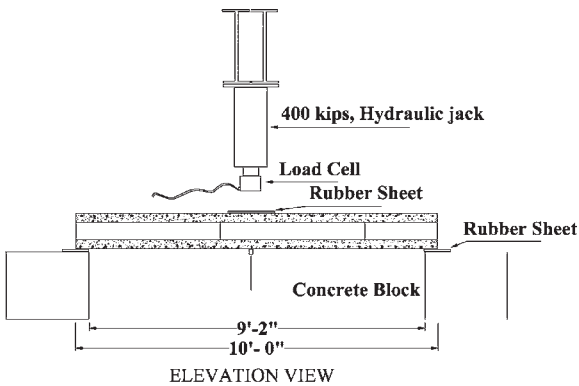


Fig. 11: Full-scale flexure test set-up



Fig. 12: Facilitate gripping



Fig. 13: Extensometer

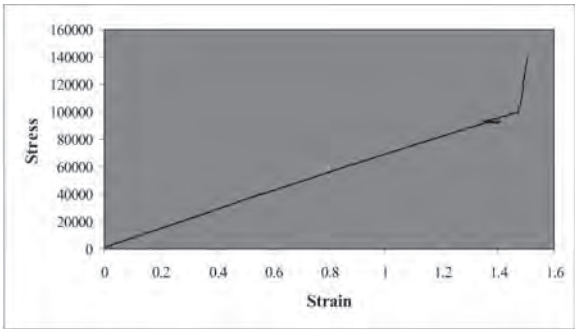


Fig. 14: FRP stress-strain relationship

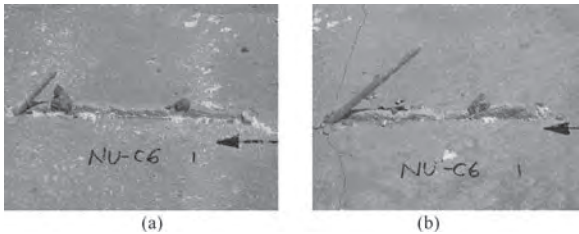


Fig. 15 (a), (b): Failure of GFRP connectors

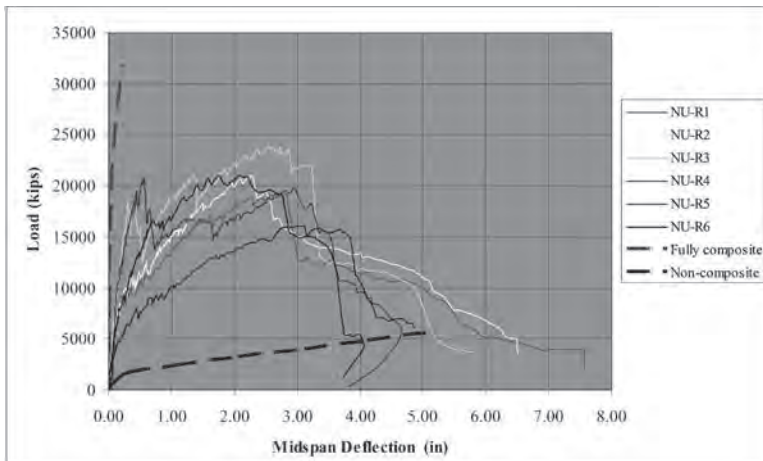


Fig. 16: Mid-span deflection

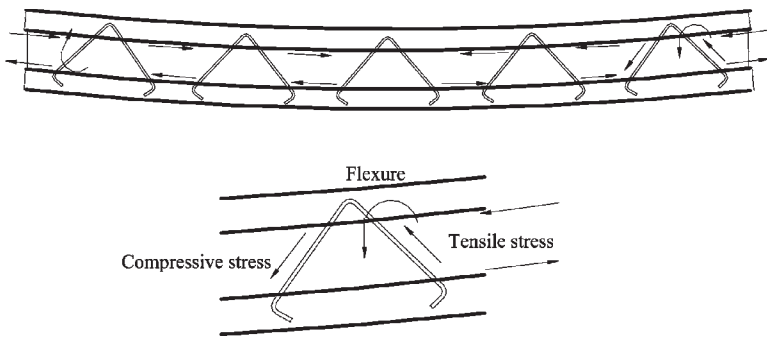


Fig. 17: Forces in the connector

Tensile Capacities of CFRP Anchors

by G. Ozdemir and U. Akyuz

Synopsis: During major earthquakes, buildings which have inadequate lateral strength may be subjected to severe damage or total collapse. To prevent such failures, buildings that are vulnerable against earthquakes must be strengthened. In reinforced concrete buildings inserting adequate amount of reinforced concrete infills is an effective strengthening technique. Another promising technique is to strengthen the existing hollow clay tile infill with diagonally placed CFRP sheets. In this technique, CFRP sheets are extended to the frame members. The connection between CFRP sheets and frame members is provided by CFRP anchors. In this strengthening technique, the effectiveness is dictated by the CFRP anchors. In this study, by means of the prepared test setup, the pull-out strength capacities of CFRP anchors are measured. The effects of concrete compressive strength, anchor embedment depth, anchor hole diameter, and number of fibers (CFRP sheet width) on the tensile strength capacity of CFRP anchors were studied.

Keywords: adhesive anchor; carbon fiber-reinforced polymer (CFRP); CFRP anchor; strengthening

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INTRODUCTION

Structures should be constructed in such a way that they have enough capacity to carry all possible loads. At the design stage, some of the loads can inadvertently be underestimated. Unintentionally created deficiencies during design and/or construction stages may cause catastrophic results when the structure is subjected to unpredicted loads. Due to these deficiencies, existing structures may require strengthening or stiffening to increase their ultimate flexural and/or shear capacity. One of the strengthening techniques is to insert enough amounts of reinforced concrete (RC) infills to meet the lateral demands of the structure. However, for the existing undamaged buildings this technique is not an occupant friendly technique.

With the improvements in technology, a promising technique was developed in the strengthening of structures aiming fewer disturbances and less time consumption. For this purpose, fiber reinforced polymers (FRP), made of high-modulus fibers bonded with a resin matrix, have been increasingly used because of its superior properties. In comparison with steel, FRP possess many advantages such as high corrosion resistance, high strength to weight ratio, electromagnetic neutrality and ease of handling. The first applications of FRP in strengthening are seen in the retrofitting of damaged columns and beams, which provide member improvements only. In recent studies, knowing that strengthening with FRP is successful for member repair, it was used in rehabilitation of undamaged structures with a new technique. In this new technique, the goal was the system improvement rather than member rehabilitation. This technique is called as seismic retrofit by carbon fiber sheets (SR-CF system).

In SR-CF system, the surface of the existing hollow clay tile infill walls are covered with diagonally glued carbon fiber sheets providing that edges of the sheet are connected to the peripheral column, beam and floor using special connections. In this method, the carbon fiber sheet behaves like a tensile bracing and increases the shear resistance of the wall. The key issue in SR-CF application is the performance of the connection between the carbon fiber reinforced polymer (CFRP) sheet and the peripheral structural members. This can be possible with proper connection details. For this purpose, special devices namely, CFRP anchors, were developed. The effectiveness of SR-CF system is highly depending on the capacity of CF anchors. When this connection is lost,

load-transmitting process is failed and this causes a sudden increase in internal forces in peripheral frame members, which may cause a failure of the structure. Therefore, the behavior of these special connections, namely CFRP anchors need to be investigated for the success of this strengthening technique. In this study, only the direct tensile load capacities of CFRP anchors were determined experimentally. During the tests, the effects of concrete strength, anchor embedment depth, anchor hole diameter, and CFRP sheet width on the tensile capacity of CFRP anchors were studied.

STRENGTHENING WITH CFRP

To investigate the effect of SR-CF method on the behavior of the rehabilitated structure, an experimental study was performed in structural mechanics laboratory of Middle East Technical University (METU) (Erdem 2003). The test specimen was two story three bay 1/3 scale reinforced concrete frame. The width of the middle bay was 1000 mm while the others were 1600 mm. The first story height was 1500 mm and the second story height was 1000 mm. All the columns were 110×110 mm and all the beams were 110×150 mm. This frame had common deficiencies of the buildings in Turkey. To strengthen this frame, first the middle bay of the frame was infilled with hollow clay tiles and plastered. Then CFRP sheets were glued on both surfaces of the infill diagonally (Figure 1). The connection of the CFRP sheet with the frame members were provided by CFRP anchors (Figure 2). The specimen was subjected to quasi-static reversed cyclic lateral load at the second story level with an axial load of 9 kN applied to the columns during the test. The results of this experiment was compared with the results of the tests conducted with exactly the same frames in which the frame was a bare frame without any strengthening and a frame that was strengthened with RC infill wall inserted on the middle bay only.

In (Erdem 2003), by using the bare frame test data also, which was provided by (Canbay, Ersoy, and Ozcebe 2003), two different strengthening techniques were compared. These are namely, inserting a RC infill and inserting a hollow clay tile infill strengthened with CFRP. Both strengthening techniques provide significant increase in lateral load carrying capacity. The lateral load capacity of the frames strengthened with RC infill wall and CFRP was about 5 times greater than that of the bare frame. Test results also revealed that the increase in the initial stiffness of the frames was considerable due to application of both strengthening techniques. The significant difference observed between these two successive strengthening techniques was the failure types. As expected, strengthening with RC infill failed in flexure in a very ductile manner. However, the frame strengthened with CFRP failed due to premature failure of the CFRP anchors providing the connection at the foundation level. To give an idea about the effectiveness of the strengthening techniques the drift ratio envelopes are presented in Figures 3 and 4 (Erdem 2003) for the first and second stories, respectively. Drift ratio of a story is defined as relative displacement of that story divided by its height. As it is seen, the lateral deformations in both stories of the frame strengthened with CFRP were smaller than that of the frame with RC infill. This is because the test conducted with CFRP strengthened frame could not be continued to produce enough shear displacement

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due to the failure of the CFRP anchors. It is obvious that the behavior could be superior unless the CFRP anchors were ruptured. The CFRP anchors used in this study were of Type-1 anchors which will be discussed in detail in proceeding sections.

The experimental study conducted by (Erdem 2003) resulted with a very important conclusion, which emphasizes the importance of the connections, namely CFRP anchors in the effectiveness of the strengthening application with CFRP. The observed results show that the key issue in a SR-CF application is the performance of connection between the CFRP sheet and the peripheral structural members.

EXPERIMENTAL PROGRAM

Previous studies reveal that in strengthening with CFRP, connections between the frame members and CFRP sheet are the crucial points. The effectiveness of the rehabilitation depends on a proper connection. Provided that the connection has sufficient strength, then this strengthening method is satisfactory. The load transfer from hollow clay tile infill strengthened with CFRP to the RC frame members is provided by the anchors. Therefore, the capacity of the anchors needs to be known for a successful strengthening.

In the experimental study performed by Erdem, the anchors which were used in the infill were subjected to both shear and axial forces while anchors which were used on the frame were subjected to tensile force only. Due to complexity of the combined shear and axial force problem, as an initial study it is decided to investigate the uniaxial tensile capacity of the embedded CFRP anchors. Tensile capacity of a single CFRP anchor was measured throughout the experiments under the effect of aforementioned parameters. These experiments were conducted in two series of tests depending on the preparation of the CFRP anchors.

Test specimens

In order to obtain the uniaxial tensile capacities of CFRP anchors, concrete blocks with different compressive strengths were prepared as test specimens. Dimensions of these concrete blocks were 300×400×4000 mm, and holes in the desired diameters were drilled by means of a rotator hammer on it. Five concrete beams were cast with three different compressive strengths. Desired and actual 28-day compressive strengths of these blocks are given in Table 1.

CFRP anchors

In this study, MBrace produce C1-30 unidirectional fiber sheets were used. CFRP anchors were composed of an epoxy based matrix and C1-30 carbon fibers. A two component, room temperature cure epoxy resin adhesive, namely saturant, is used as the component of the CFRP. The properties of C1-30 fiber sheets and epoxy resin are given in Tables 2 and 3, respectively.

It is known that adhesive anchors transfer the load through the adhesive layer along the bond surface. Therefore, the important points for an adhesive anchor are the quality and type of the adhesive. An epoxy adhesive is a synthetic compound consisting of an epoxy resin cross-linked with a curing agent. The epoxy resin is designated as compound A and the curing agent as compound B by the manufacturer. Epoxy adhesives are thermosetting polymers; that is, they require heat to cure. This heat is generated during the exothermic reaction between the epoxy resin and the curing agent (Cook and Klingner 1992; Cook, Doerr, and Klingner 1993; Cook 1993; Cook et al. 1998).

In the literature, there are different application techniques of adhesively bonded anchors. In this experimental study, an injection type adhesive anchor application was chosen. Before the installation of the anchor, a catalyzed resin has been injected into the hole and then the anchor was pushed into the hole.

Type 1—These anchors are exactly identical to the one used in two story three bay 1/3 scale reinforced concrete frame test. While preparing these anchors, first, CFRP sheets were cut into two layers of equal width and equal height. Then, these soft CFRP sheets (Figure 5a) were rolled to have a cylindrical form. To prevent the CFRP sheet bundle deviating from its cylindrical shape, rolled sheets were tied with CFRP fibers. CFRP sheets became ready to be used as an anchor after 1 cm portion from the bottom of the rolled CFRP sheet was embedded into the epoxy resin and waited for one day. The aim of the epoxy coated bottom part of the CFRP anchor is to have a stiff part to maintain ease in installation. During the installation procedure, CFRP anchors were pushed from the hardened end into the drilled and carefully cleaned holes by using a steel wire. All of the holes were cleaned by an air pump. After pouring enough epoxy resin into the clean, dry anchor hole, epoxy coated end of the CFRP anchor was embedded into the concrete as seen in Figure 5b. The bond free part of the CFRP anchor was also perfectly bonded to a steel rod to apply a tensile force. The steel rod is used to transfer the applied load to the CFRP anchor and it is not prolonged into the hole.

In Type-1 series, three different sheet width (w) (80 mm, 120 mm, and 160 mm), three different embedment depth (h) (70 mm, 100 mm, and 150 mm), three different concrete compressive strength (f_c) (10 MPa, 16 MPa, and 20 MPa), and three different anchor hole diameter (d) (12 mm, 14 mm, and 16 mm) were chosen as the study parameters.

Type 2—In the second series of tests, the CFRP anchors were prepared completely out of the hole and then installed into the concrete. This technique was developed after the first series of tests. In this technique, first the desired width of CFRP sheet was cut and coated with epoxy resin. Then, the epoxy coated CFRP sheet is rolled over a silicon rod, which is 10 mm longer than the embedment depth, and the steel rod through which the force is applied (Figures 6a-6b). The diameters of the silicon rod and the steel rod were the same. This technique was developed to have straight anchors in which the fibers of the CFRP sheets are oriented in the same alignment. The part of the anchor rolled around the silicon rod was embedded into the drilled hole and bonded there.

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The extra part of the silicon rod was standing over the concrete level to prevent a discontinuity at the critical section.

The parameters studied in this series were four different embedment depths (50 mm, 70 mm, 100 mm, and 150 mm), and two different concrete compressive strengths (10 MPa, and 16 MPa). The CFRP sheet width and anchor hole diameter was chosen as 120 mm and 20 mm, respectively.

Testing apparatus and procedure

The steel frame, which was used to apply the tensile force to the embedded anchor, is shown in Figure 7. The load was applied by means of a center hole hydraulic jack and the load was measured by means of a load cell. Applied load was transferred to the CFRP anchors through the steel rod extending along the loading equipments. Load readings were taken at a rate of five samples per second.

TENSILE CAPACITY OF CFRP SHEETS

While presenting the experimental results, the measured ultimate tensile loads were divided into the ultimate capacity of a CFRP sheet with equivalent sheet width. It is well known that the tensile capacity of carbon fiber sheets can be calculated using the following equation

$$P_{FRP} = w \times t \times f_u \quad (\text{N}) \quad (1)$$

where w is the CFRP sheet width in mm, t is the thickness of the CFRP sheet in mm, and f_u is the characteristic tensile strength of the carbon fibers in terms of MPa. t and f_u are declared by the manufacturer as 0.165 mm and 3,430 MPa, respectively. Thus, for different w , the rupture capacity of CFRP sheet can easily be determined.

TEST RESULTS

The test results are going to be presented in two parts composed of the results obtained from Type-1 and Type-2 series of tests. Complete results are provided in (Ozdemir 2005). All tensile capacities observed during the tests were normalized with the capacity of carbon fibers (determined from Equation (1)) that have the same sheet width (w) with the anchors.

Type-1 test results

In Type-1, 124 tests were conducted. Results of these numerous tests are going to be discussed under the name of each parameter. Before the discussion of the results, it should be stated clearly that all anchors failed as rupture of the CFRP sheet covered with epoxy resin (Figure 8).

Effect of anchor hole diameter (d)--Load transfer mechanism of adhesive anchors is different from that of mechanic or headed anchors by the way how they transmit the applied tensile load. The applied load is transferred from an adhesive anchor by means of the bonding between the surface and the adhesive. That is why the bond strength is important for an adhesive anchor. The main idea to choose the hole diameter as a parameter was to investigate the effect of the free space between the anchor and concrete surface on the tensile capacity of the anchors. The procedure throughout the experiments for investigating the effect of the anchor hole diameter was to change the hole diameter while the other parameters remain unchanged. Effect of hole diameter (d) on 10 MPa concrete block is discussed in Figure 9 for anchors having sheet width of 80 mm, and embedment depth of 150 mm. As it is seen the normalized tensile capacity of the CFRP anchors in different hole diameters is almost constant, and it is equal to 45–48% of the capacity of the CFRP sheet calculated by Equation (1). All anchors failed in such a way that the effect of hole diameter could not be observed. CFRP sheets have reached their ultimate tensile capacity under the applied load before any bond failure occurred. So, for this series of tests it can be said that a minimum of 2-3 mm free space, which is also the suggested necessary spacing by the manufacturers, is adequate for proper bonding.

Effect of concrete compressive strength (f_c)--In previous studies (Cook et al. 1998; Makitani, Irisawa, and Nishiura 1995; Hattori et al. 1995), compressive strength of the medium, where adhesive anchors were installed, has been chosen as a parameter influencing the capacity of the anchors. Some of those studies indicate that the compressive strength does not have any significant effect on the capacity of the anchors (Makitani, Irisawa, and Nishiura 1995; Hattori et al. 1995) while some of them designate that compressive strength is influential on the capacity of the adhesive anchors (Cook et al. 1998). Figure 10 represents test results for a group of anchor which have 160 mm sheet width, 100 mm embedment depth, and 14 mm hole diameter. In Type-I tests, for concrete compressive strengths between 10 to 20 MPa, it is not seen any significant difference between the normalized tensile capacities of CFRP anchors. The ratio of the rupture load of CFRP anchor to the tensile capacity of CFRP sheet obtained from Equation (1) is approximately 0.31 for all f_c between 10 to 20 MPa. Similar to the case in hole diameter, CFRP anchors in this category failed before the effect of concrete compressive strength can be observed. The dominant failure type is the rupture of the CFRP sheet. The load transfer mechanism at the sections very close to hole seems to be very important. The orientation of the CFRP fibers becomes very crucial since any deviation from the straight alignment of the fibers lead to premature rupture of the CFRP sheet.

Effect of CFRP sheet width (w)--Tensile capacity of a CFRP anchor is related with the number of fibers in that anchor. Thus, CFRP sheet width is a crucial parameter. A wider anchor contains more carbon fibers to resist applied tensile load. The ultimate tensile capacities versus CFRP sheet width are plotted in Figure 11. All anchors presented in this graph have 100 mm embedment depth, 12 mm hole diameter. They are installed into a concrete beam, which has a compressive strength of 20 MPa. As expected, test results show that load capacity increases with an increase in sheet width. However, the

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behavior is not linearly proportional to the sheet width. When ultimate loads, P_u , are normalized according to the capacity of a carbon fiber sheet with the same width, there is a decrease as width becomes wider (Figure 12). This shows that the tensile capacity does not increase in the same rate with the ratio of CFRP sheet widths. The normalized tensile capacity of an anchor with 80 mm sheet width is obtained in average as 0.52 while this ratio is approximately equal to 0.35 for an anchor having 160 mm sheet width.

Effect of embedment depth (h)--Most of the previous studies that investigate the behavior of the anchors were mostly interested in the effect of the embedment depth of the anchors (Cook and Klingner 1992; Cook, Doerr, and Klingner 1993; Cook 1993; Cook et al. 1998). The data given in Figure 13 corresponds to the anchors with 160 mm sheet width, 14 mm hole diameter, and 20 MPa concrete compressive strength. As it is seen, the normalized tensile capacity of a CFRP anchor increases as embedment depth increases from 70 mm to 100 mm. However, there is a decrease or no change in the CFRP anchor capacity after 100 mm embedment depth. Among the embedment depths studied, maximum normalized value is obtained when the embedment depth of 100 mm.

Type-2 test results

To investigate the behavior of CFRP anchors under the effect of concrete compressive strength and embedment depth of the anchor in detail, this second series of tests were carried out for only one CFRP sheet width and one hole diameter, while the embedment depth changes from 50 mm to 150 mm, including 70 mm and 100 mm. This series was studied in two concrete blocks with 10 MPa and 16 MPa compressive strengths. In this series, total of 23 tests were conducted. The normalization of the ultimate tensile capacities was also performed by dividing them to the tensile capacity of the CFRP sheet determined by Equation (1). All of the anchors tested in this part have 120 mm sheet width and installed into a hole with 20 mm diameter.

The main difference observed during this series of tests was in the failure types. A concrete cone observed for anchors with 50 mm embedment, while a combined cone-bond failure observed for 70 mm and 100 mm embedment depths. The failure of the CFRP anchors with 150 mm embedment was due to rupture of CFRP sheet.

Effect of concrete compressive strength (f_c)--Normalized tensile capacities for all embedment depths are given in Figure 14. It is seen that the highest normalized tensile capacity of a CFRP anchor is reached for an embedment depth of 100 mm. As it is also seen, normalized data for 50 mm embedment depth are so close to each other that the difference in tensile capacities of anchors installed into 10 MPa and 16 MPa concrete beams is negligible. However, as the embedment depth increases, the significance of the concrete compressive strength increases. For the highest normalized tensile capacity, i.e. for $h=100$ mm, the effect of concrete compressive strength is more significant.

Effect of embedment depth (h)--Figure 15 gives the normalized test results of anchors installed into a concrete beam of 10 MPa. As expected, the normalized tensile capacity of a CFRP anchor increases as the embedment depth increases. However, there is an effective depth beyond which the capacity becomes almost constant. For the

parameters studied, effective embedment depth is 100 mm. The capacities of CFRP anchors that are embedded to 150 mm are almost equal to those of 100 mm embedment depth. Figure 16 shows the change in normalized tensile capacities of anchors installed into a concrete beam of 16 MPa. Similar behavior was obtained. Tensile capacities of CFRP anchors increase up to 100 mm embedment depth. Beyond this depth, the capacity becomes constant and equal to that of 100 mm. This is due to high bonding effect resulting in high load transfer to the concrete at the top of the anchor. The bond stress is no longer uniform, and if the tensile load is sufficiently high, the failure initiates with a concrete failure in the upper portion of the concrete and then the bond failure occurs in the remaining part of the embedment depth.

CONCLUSIONS

During the last decade, fiber reinforced polymers (FRP) has been widely used to strengthen bridge girders, piers, columns of structures and masonry walls in wall bearing. Usage of carbon fiber reinforced polymers (CFRP) to strengthen the existing structures is relatively new and promising technique. While strengthening the existing structures CFRP sheet are applied on the hollow clay tile infill walls diagonally. In strengthening the hollow clay tile infill by CFRP, CFRP anchorages are used both in masonry and in reinforced concrete members to provide a sufficient bond between the CFRP sheet and the masonry and the reinforced concrete member. Better connections lead higher energy dissipation and higher ductility. Thus, the capacity increase in the existing structure mostly depends on the load transfer through the CFRP anchors, or simply the increase in capacity of the structure depends on CFRP anchor capacity. In this experimental study, direct tensile capacities of CFRP anchors were investigated for different parameters. Effect of CFRP sheet width, embedment depth of adhesive anchor, hole diameter of anchor and compressive strength of concrete on the uniaxial tensile capacity of the CFRP anchors were determined. With the help of the test results, the followings are concluded.

For the embedment depth of 50 mm, uniaxial tension tests ended with a concrete cone failure. However, the anchors with 70 mm and 100 mm embedment depths formed pullout failure with a shallow concrete cone at the top. On the other hand, CFRP rupture was observed for the anchors embedded into 150 mm.

For the studied parameters, the maximum tensile load capacities are obtained for CFRP anchors which have 100 mm embedment depth. This indicates that there is an effective bond length beyond which load capacity does not increase (Cook, Doerr, and Klingner 1993; Cook 1993; Cook et al. 1998). The increase in tensile load capacities can be assumed linear up to 100 mm embedment depth.

For the shallow embedment depths, i.e. 50 mm, the effect of concrete compressive strength, in the range of 10 MPa to 16 MPa, on the tensile capacity of CFRP anchor is not significant. However, as the embedment depth increases, the effect of concrete compressive strength becomes more significant.

ACKNOWLEDGMENT

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Table 1— Desired and actual 28-day compressive strengths of these blocks

Specimen	28 day f_c (MPa)	Test day f_c (MPa)
1 st beam	10.7	10.7-11.3
2 nd beam	15.8	15.8-16.4
3 rd beam	19.4	19.4-20.1
4 th beam	10.2	10.2-10.5
5 th beam	16.3	16.3-16.5

Table 2— Properties of C1-30 fiber sheets

Property	Amount	Unit
Unit Weight	0.300	kg/mm ²
Effective Thickness	0.165	mm
Characteristic Tensile Strength	3,430	MPa
Characteristic Elasticity Modulus	230,000	MPa
Ultimate Strain	0.015	mm/mm

Table 3— Mechanical properties of epoxy resin (provided by MBrace)

Property	Amount
Compressive Strength	>80 MPa
Direct Tensile Strength	>50 MPa
Flexural Tensile Strength	>120 MPa
Elasticity Modulus	>3000 MPa
Ultimate Strain	>0.025

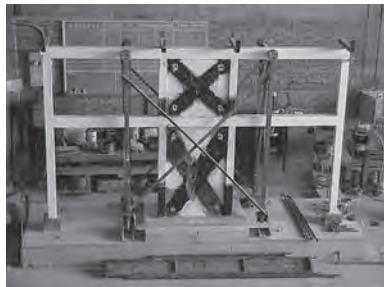


Figure 1—Test frame (taken from (Erdem 2003))

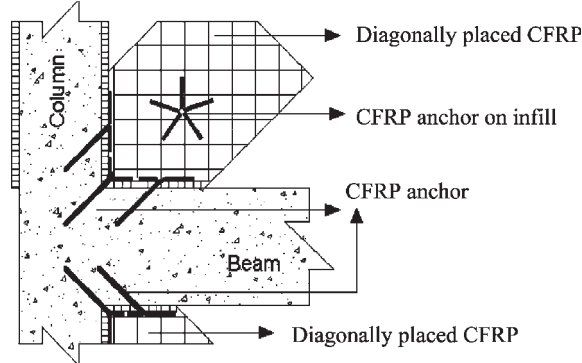


Figure 2—Connection detail (Erdem 2003)

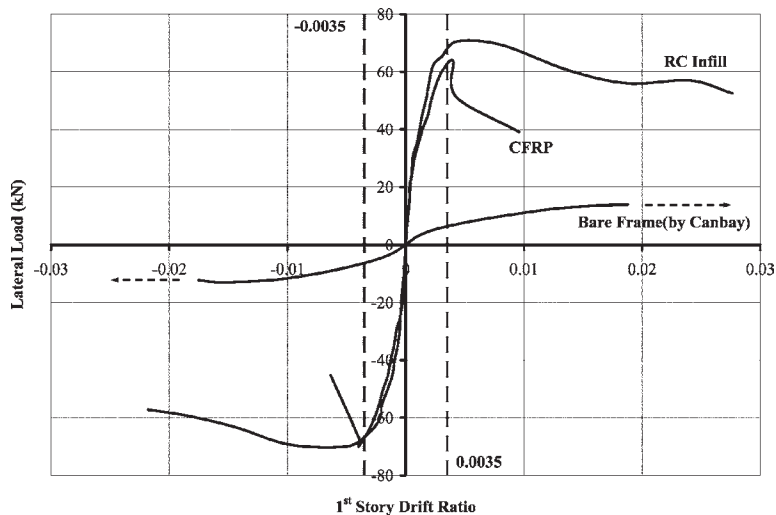


Figure 3— First story envelope curve of test frame (taken from (Erdem 2003))