Seismic Retrofit of Precast Concrete Panel Connections with Carbon Fiber Reinforced Polymer Composites

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A technique for the seismic retrofit and rehabilitation of existing welded steel plate edge connections between precast concrete shear wall panels using carbon fiber reinforced polymer (FRP) composite sheets is presented. Nine lateral load tests were performed on shear wall assemblies comprising three 4 x 12 ft x 8 in. (1.22 x 3.66 m x 203 mm) hollow-core precast concrete wall panels. Each wall assembly was loaded in the in-plane direction by quasi-static simulated seismic loads. The test variables were the covered area of FRP composite sheets, the number and orientation of FRP composite layers, and wall panel surface preparation. Concrete surface preparation using high-pressure water jet and subsequent application of a bonding agent resulted in the most effective bond between concrete and FRP composite. The tests demonstrated that FRP composite connections could transfer the applied load across the joint effectively and achieve a higher capacity compared to welded steel plate edge connectors.

Connections play a vital role in the design and construction of precast/prestressed concrete structures, especially in high seismic regions. Until recently, fiber reinforced polymer (FRP) composites had been used extensively in the aerospace, automotive, and chemical industries. Due to lack of research, design specifications, and a relatively high cost, their use in the construction industry has been limited. With production costs steadily decreasing, the use of FRP composites is gradually becoming economically feasible in a variety of engineered structures.

High ultimate strength, relative ease of application, and the ability to conform to irregular surfaces make FRP composites an attractive alternative for seismic retrofitting of existing structures. FRP composites exhibit resistance to environmental and chemical corrosion, which makes them an excellent material for use in aggressive environments.[1]

FRP composites have been used in a variety of bridge and building projects, including composite decks, composite beams, column wrapping, strengthening of reinforced concrete beams and joints with external FRP composite sheets, and external prestressing.

Many recent projects using FRP composite systems have been documented, including the seismic rehabilitation of a tilt-up concrete building,[2] the strengthening of column brackets,[3] the flexural reinforcement in prestressed concrete beams,[4] the flexural strengthening of prestressed concrete bridge slabs,[5] and the fabrication of pretensioned and post-tensioned girders.[6] These applications comprise seismic retrofitting, post-earthquake rehabilitation, flexural and shear strengthening of structural members, and new construction.

The bond and force transfer mechanism in FRP composite material plates bonded to concrete was investigated by Chajes et al.[7] The objective of that study was to develop rehabilitation procedures involving externally applied FRP composite plates for concrete structures. Direct bond tests were performed on joints consisting of FRP composite material plates bonded to concrete.

Two types of failure mechanisms were observed: failure by direct concrete shearing beneath the concrete surface and a cohesive type of failure. The failure modes depended on the surface preparation of the concrete and the type of bonding agent used.

The PCI PRESSS (Precast Seismic Structural Systems) investigators have taken the lead on research and design recommendations for precast concrete structures in areas of high seismicity.[8,9] One of the vulnerable structural elements observed in recent earthquakes is the connection between precast shear wall panels.[10] These connections may not have the ductility needed to develop the yield capacity of the members.

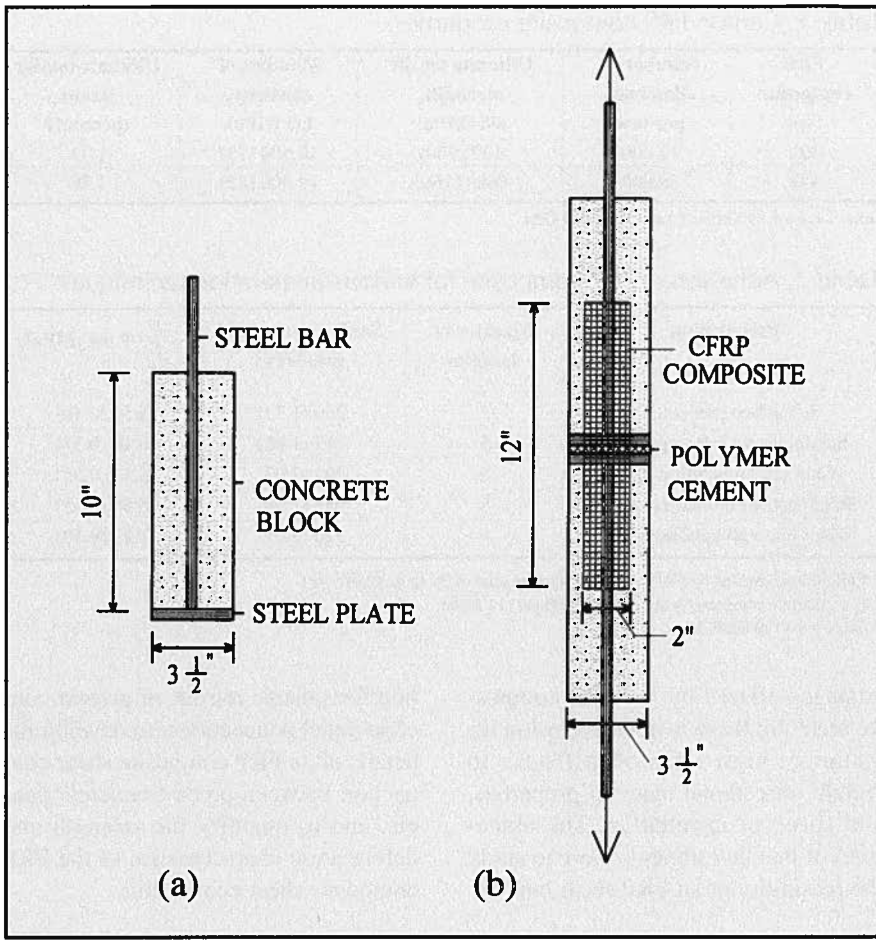

Fig. 1. Configuration of specimens and setup for pullout tests: (a) concrete block and (b) block assembly showing carbon FRP composite strips.

One suggestion for improving the capacity of these connections is the addition of steel plates across the joint.[11] When considering this method, however, care must be taken to ensure adequate anchor bolt capacity by providing adequate edge distances. Simple shear connections for wall panels or slabs have been tested and their capacity evaluated and compared to the PCI Design Handbook.[12] The shear connections tested that are relevant to the present study did not perform satisfactorily.

Cyclic testing of new edge connectors for wall panels has shown that such connectors can exhibit limited ductility;[13] however, they are still susceptible to concrete spalling and low cycle fatigue.

Structural engineers are increasingly faced with the problem of finding cost-effective solutions to strengthening of non-ductile structural elements, which are vulnerable to damage in strong earthquakes. Panel-to-panel connections made of steel inserts are non-ductile and may not be able to transfer overturning moments and forces between panels.[14]

One evaluation of welded loose-plate steel connectors located in the vertical joint between panels has shown that the connection possessed little ductile capacity.[15] The lack of seismic retrofit and rehabilitation techniques for welded shear connections in precast concrete wall panels led to the present investigation.

The overall goal of this research was to develop a cost-effective seismic retrofit and rehabilitation technique for shear connections between precast shear wall panels using FRP composites. The reasons for developing such shear connections are related to the ad-

Table 1. Carbon FRP composite properties.

FRP composite type	Number of filaments per tow	Ultimate tensile strength, ksi (MPa)	Modulus of elasticity, ksi (GPa)	Ultimate tensile strain (percent)
12k	12,000	139 (960)	10,600 (73)	1.33
48k	48,000	168 (1160)	12,300 (85)	1.36

Note: 1 ksi = 6.895 MPa; 1 ksi = 0.006895 GPa.

Table 2. Adhesion of FRP composite for various preparation techniques.

Preparation type	Number of samples	Surface stress, σ,* psi (MPa)	$\frac{\sigma}{\sqrt{f_c'}}$,† psi (MPa)
No surface preparation	3	250 (1.72)	5.59 (0.46)
Sandblast, no bonding agent	5	273 (1.88)	6.10 (0.51)
Water jet, no bonding agent	5	294 (2.03)	6.57 (0.55)
Sandblast, with bonding agent	5	288 (1.98)	6.44 (0.53)
Water jet, with bonding agent	4	326 (2.25)	7.29 (0.59)

* Pullout load divided by FRP composite surface area of 24 sq in. (0.015 m²).
† f_c' = concrete compressive strength of 2000 psi (14 MPa).
Note: 1 psi = 0.006895 MPa.

vantages offered by the FRP composite material; these include corrosion resistance, high ratio of stiffness to weight, directional material properties, and speed of installation. The objectives of this investigation were to study the feasibility of an FRP shear connection for seismic retrofit of precast concrete panel connections, to develop the details of an FRP composite shear connection between precast concrete panels, and to quantify the strength and deformation characteristics of the FRP composite shear connection.

PULLOUT TESTS

The bond properties of FRP composites to concrete were investigated using various preparation techniques as part of the research.[16] Concrete blocks with dimensions $3^1/_2$ x $3^1/_2$ x 10 in. (89 x 89 x 254 mm) were made with a 28-day compressive strength of 2000 psi (14 MPa).

A No. 5 (16 mm) Grade 60 (414 MPa) steel bar was used as the tab for the grips of the testing machine. Each steel bar was welded to a $3^1/_2$ x $3^1/_2$ x $^3/_8$ in. (89 x 89 x 10 mm) steel plate as shown in Fig. 1(a). Two concrete blocks were aligned as shown in Fig. 1(b), and the gap between the two blocks was sealed with polymer cement.

Two layers of carbon fibers were cut into 2 x 12 in. (51 x 305 mm) strips; the fibers were saturated with resin consisting of Shell Epon Resin #828 epoxy and Epicure 3234 hardener and applied to two opposite sides of the concrete blocks; the FRP composite was cured at room temperature for seven days.

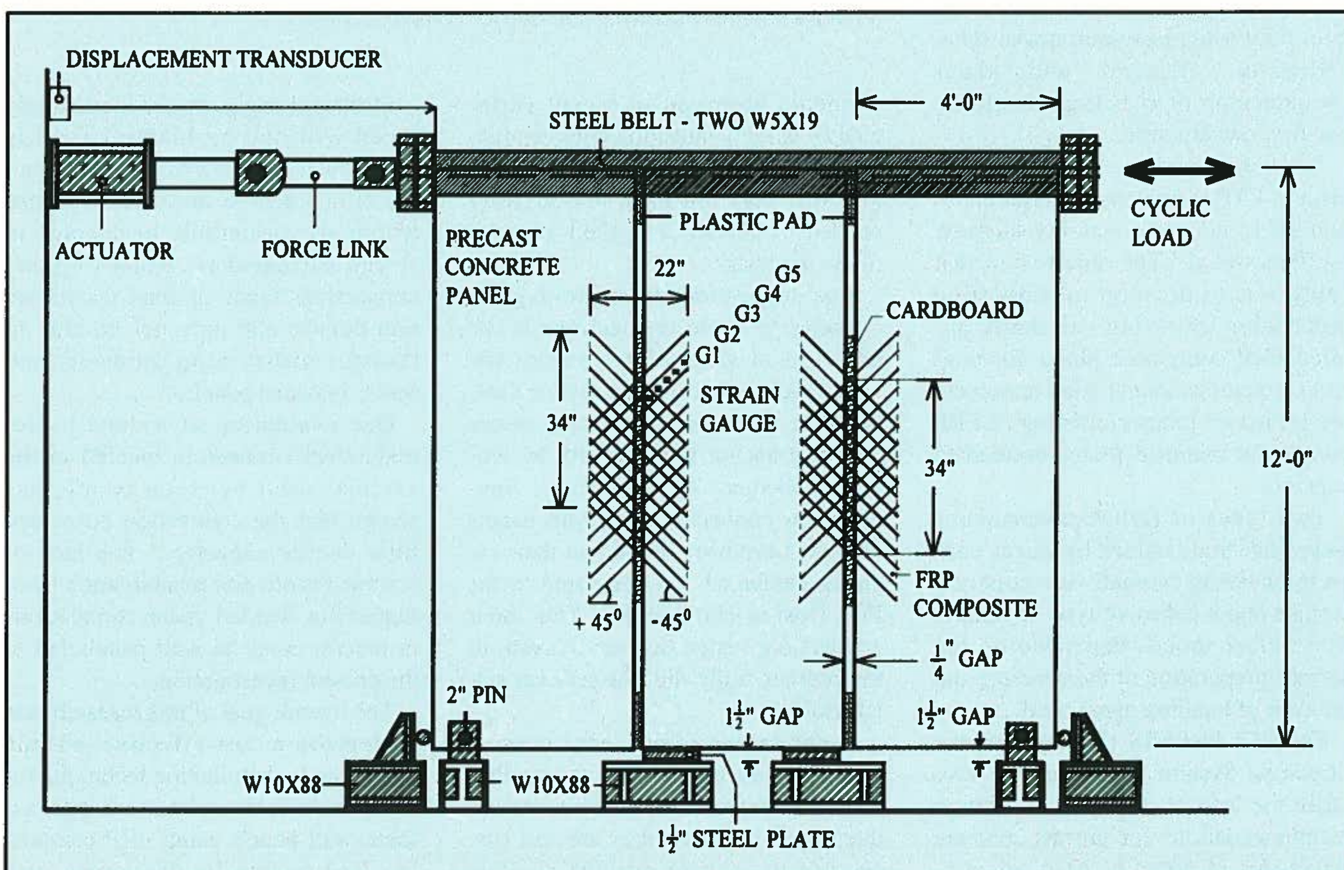

Fig. 2. Typical setup for in-plane test of wall assembly.

Surface preparation of the specimens included three treatments: sandblasting, high-pressure water jetting (with a water jet pressure of 40,000 psi, or 276 MPa, dispersed with a multiple head nozzle), and no surface preparation. For some of the specimens, a bonding agent was applied between the FRP composite and concrete, whereas for others no bonding agent was used. For these pullout tests, the bonding agent was Sikadur 32, a multi-purpose, two-component structural epoxy adhesive that conforms to ASTM C 881-90 Type I and Type II epoxy resin adhesive. The bonding agent was applied using hand tools.

The properties of the FRP composite are given in Table 1. The carbon fibers used in the pullout tests consisted of 48k (48,000 fibers per tow) woven unidirectional fabric. When the bond between carbon FRP composite and concrete was sufficiently strong, the concrete failed in a cohesive failure mode, which is essentially a substrate failure in the concrete. However, the carbon FRP composite lay-up, their dimensions, and the bonding agent were instrumental in achieving a stronger bond with the concrete.

The water-washed surface exposed the concrete aggregate better than sandblasting. As a consequence, for the water-jet-prepared concrete specimens, the FRP composite pulled out some concrete in a cohesive concrete failure mode, as opposed to predominant bond failure in the sandblasted specimens.

Table 2 shows the results of the study on surface preparation and application of bonding agent. Two layers of FRP composite were used for all specimens. Both sandblasted and water-jet specimens performed better than the specimens with no surface preparation. The adhesive bond stress can be described for different conditions in terms of $\sqrt{f_c'}$ times a constant, which for psi units varies from 5.59 for the case of no surface preparation to 7.29 for the case of water jet with a bonding agent (0.46 to 0.59 for MPa units).

The water-jet specimens with bonding agent had a surface stress an average of 13 percent better than the sandblasted specimens. The water-jet specimens without bonding agent had a surface stress an average of 8 percent better than the sandblasted specimens without the bonding agent. Finally, the water-jet specimens without bonding agent were 2 percent better than the sandblasted specimens with bonding agent. It is clear from these tests that water-jet preparation offers tangible technological and economical advantages.

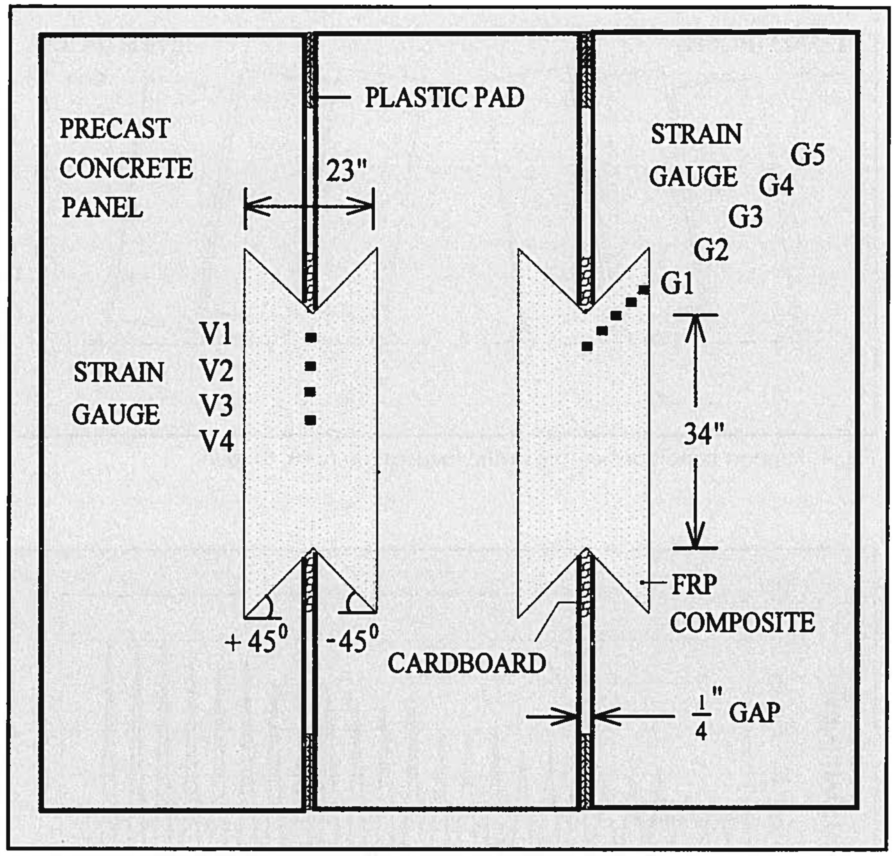

Fig. 3. Strain gauge locations for a typical assembly.

EXPERIMENTAL SETUP AND TEST PROCEDURE

Test Facility and Setup

Testing of the precast shear wall panel assemblies with FRP composite connections was performed at the University of Utah Structures Laboratory.[17] Three 4 x 12 ft (1.22 x 3.66 m) precast hollow-core concrete panels were used to create each wall assembly (see Fig. 2).

Each concrete panel had an overall thickness of 8 in. (203 mm) and contained two hollow cores and one solid concrete core. This configuration was essential in forming the base supports. The lateral load was provided through a 150 kip (667 kN) hydraulic actuator attached to one of the load frame columns.

A steel belt was constructed out of two W5x19 sections to hold the three wall panels together. This steel belt was connected to the hydraulic actuator so that the wall section could be pushed or pulled by way of the steel belt to simulate the effects of seismic loads (see Fig. 2).

To reduce friction and localized damage to the concrete walls, two 1/8 in. (3 mm) high-strength HDPE pads were placed between adjoining wall panels at top and bottom. This gave a standard $^1/_4$ in. (6 mm) space between panels (see Fig. 2). A set of five strain gauges — G1, G2, G3, G4, and G5 — were placed in the fiber direction and spaced 1 in. (25.4 mm) apart. Although the configuration shown in Fig. 2 was typical, other strain gauge configurations and FRP composite lay-ups were also used.

Several strain gauge rosettes were

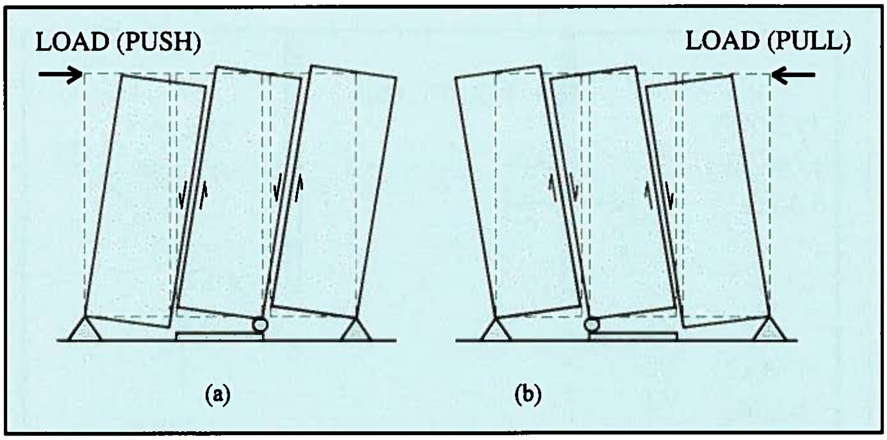

Fig. 4. Support conditions during cyclic loading: (a) push, (b) pull.

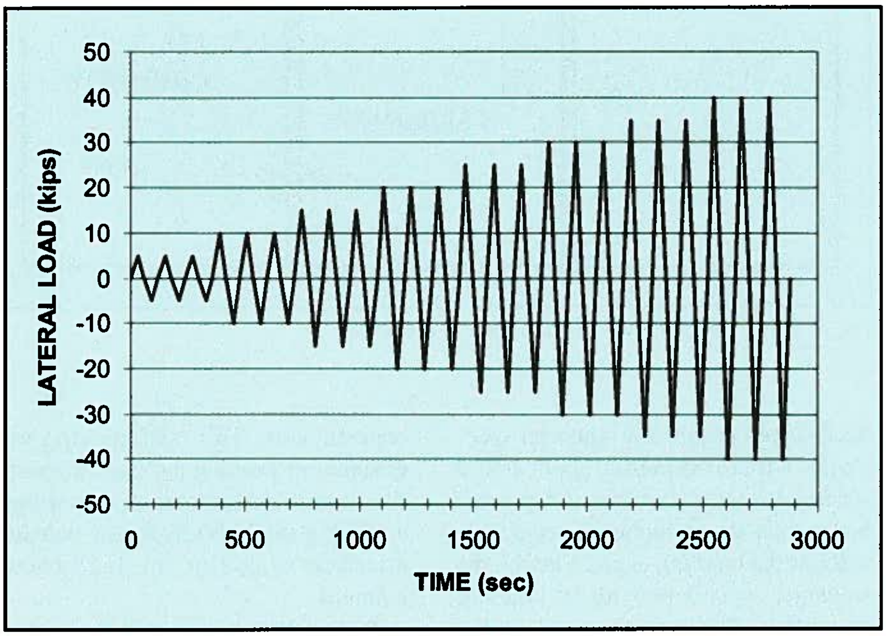

Fig. 5. Lateral load pattern used in the tests.

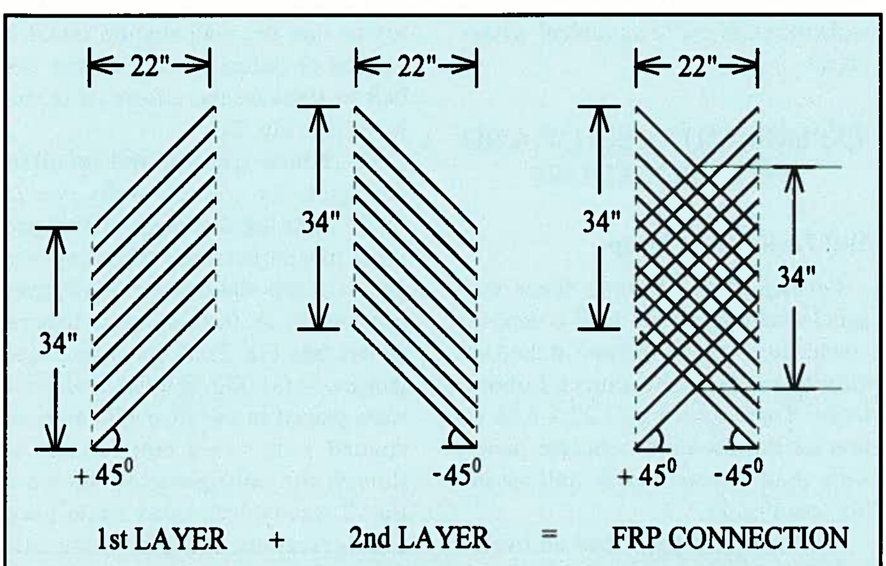

Fig. 6. Carbon FRP composite lay-up for Assembly 7.

also used in selected tests on the FRP composite connection. Four strain gauges — V1, V2, V3, and V4 — were also placed on the second FRP composite connection in a vertical configuration, as shown in Fig. 3.

Test Procedure

The concrete panels were tested in the in-plane direction using a quasi-static load in the configuration shown in Fig. 2. Pin supports at the bottom, one at the left wall panel and one at the right, allowed the wall assembly to rotate. The pin supports lifted the two outer panels 1.5 in. (38 mm) off the base. The center panel rested on steel plates that raised it up to the same level as the outer panels.

The support conditions changed depending on the load direction. When the assembly was pushed to the right, as shown in Fig. 4(a), the steel plate on the left of the middle concrete panel did not touch the concrete; when the assembly was pulled to the left, as shown in Fig. 4(b), the steel plate on the right of the middle concrete panel did not touch the concrete. As the hydraulic actuator applied the load at the top left of the wall section, the first panel rotated, the connecting force was transferred through the FRP composite connection to the middle panel, and, by the same mechanism, the force was transferred to the third panel.

The lateral load was applied through the steel belt in 5 kip (22.2 kN) increments until failure, with failure defined as the load beyond which the connection could not sustain further increase in the lateral load. Each time the load was increased, it was repeated quasi-statically three times to complete one load step (see Fig. 5). The load was applied very slowly in a quasi-static fashion to simulate damage resulting from seismic excitations and to allow time for detailed observations of the test specimens.

TEST SPECIMENS

The precast concrete walls had an average measured 28-day compressive strength of 7150 psi (49 MPa).[18] The wall panels were manufactured on a continuous cast in an 8 ft (2.44 m)

Table 3. Carbon FRP composite connection test results.

Wall assembly no.	Fiber type	Connection type	No. of layers per connection	Total carbon FRP thickness, in. (mm)	Area of connection, sq in. (m²)	Surface preparation	Failure mode	Failure load, kips (kN)	Lateral deflection, in. (mm)
1†	12k*	Rectangular	6@±60°	0.09 (2.3)	768 (0.49)	Water jet	Support	40.9 (182)	0.52 (13.2)
2	48k	Butterfly	2@±45°	0.10 (2.5)	512 (0.33)	Water jet	Adhesive in bonding agent	36.1 (161)	0.55 (14.0)
3	12k	Rectangular	4@±45°	0.06 (1.5)	330 (0.21)	Water jet	Cohesive	23.0 (102)	0.46 (11.7)
4†	48k	Butterfly	2@±45°	0.10 (2.5)	896 (0.58)	Water jet	Support	35.1 (156)	0.59 (15.0)
5‡	–	–	–	–	–	–	–	–	–
6	48k	Butterfly	2@±45°	0.10 (2.5)	1046 (0.68)	Wire brush	Adhesive in resin	15.5 (69)	0.66 (16.8)
7	48k	Butterfly	2@±45°	0.10 (2.5)	990 (0.64)	Water jet	Cohesive	44.4 (197)	0.71 (18.0)
8	48k	Butterfly	2@±45°	0.10 (2.5)	990 (0.64)	Water jet	Cohesive	34.8 (155)	0.59 (15.0)
9	48k	Butterfly	2@±45°	0.10 (2.5)	990 (0.64)	Water jet	Cohesive	35.3 (157)	0.63 (16.0)
10	48k	Butterfly	2@±45°	0.10 (2.5)	1283 (0.83)	Water jet	Cohesive	33.1 (147)	0.65 (16.5)

* 1k = 1000 fibers per tow
† Assembly not taken to ultimate because of failure of supports.
‡ Assembly not tested due to failure of supports in steel connection test.
Note: 1 in. = 25.4 mm; 1 sq in. = 0.000645 m²; 1 kip = 4.45 kN.

wide bed. The panels were prestressed in the direction in the 12 ft (3.66 m) dimension, and smooth reinforcing steel was placed at 3 ft (0.91 mm) on center perpendicular to the prestressing reinforcement. After curing, the continuous panel was cut in half longitudinally, which resulted in two 4 x 12 ft (1.22 x 3.66 m) specimens.

Surface Preparation of Wall Assemblies

An important factor in the design of FRP composites applied to concrete is the bond between the FRP composite and the concrete. The bond strength of resin and concrete depends on many parameters, among which the most important are roughness of the concrete surface, type and quality of the concrete, and curing conditions of the FRP composite.

Mechanical bond, adhesion, and cohesion are the three factors that affect the capacity of FRP composites externally attached to concrete. Surface preparation procedures were correlated with the pullout tests described earlier.

To increase the strength of the mechanical bond between the epoxy resin and concrete surface, the concrete aggregate was exposed by removing the thin outside layer of cement paste with a high-pressure water jet. This increased the surface roughness and bond strength of the concrete.

A high-pressure water jet was used to remove approximately $^1/_{16}$ in. (1.6 mm) of the concrete's surface from the hollow-core concrete panels. The water jet was operated at 40,000 psi (276 MPa) pressure with a 3 gallon per minute (189 mL/s) water supply; the water jet was dispersed with a multiple-head nozzle.

In one of the wall assemblies, all concrete surfaces were wire brushed only. After the water jet or wire brush treatment, the surface of the walls was vacuum cleaned to remove the dust and loose particles. The next stage of preparation was the application of a bonding agent, Sikadur 31, to the concrete surface.

The purpose of Sikadur 31 was to improve the bond between the FRP composite and concrete panel and to bring the surfaces of the two adjoining concrete wall panels to the same level. Sikadur 31 is a two-component structural epoxy resin adhesive very similar to Sikadur 32 used in the pullout tests.

The three wall panels were placed so that a $^1/_4$ in. (6 mm) gap existed between them (see Fig. 3). Underneath the FRP composite, these gaps were filled with cardboard material to prevent buckling of the FRP composite material (see Fig. 3).

FRP Composite Shear Connections

An FRP shear connection was implemented to connect a pair of precast concrete hollow-core wall panels. The FRP shear connection was applied on only one side of the wall assembly. An assembly consisting of three such wall panels with two FRP composite shear connections was used in each test.

In typical construction, these walls would be 8 ft wide x 24 ft high (2.44 x 7.62 m); due to space limitations, the panels in these tests were 4 x 12 ft (1.22 x 3.66 m).[17,18]

Nine tests were performed, with the FRP composite shear connections consisting of two different types of carbon fibers: 12k (12,000 fibers per tow) and 48k (48,000 fibers per tow). The same epoxy resin was used for both carbon fiber configurations. Seven of the tests used carbon FRP composite made with 48k tow fibers, and two tests used 12k tow fibers.

The lay-up of the FRP composite varied; however, the tests using 48k tow fibers used the lay-up shown in Figs. 3 and 6. The carbon fiber unidirectional sheets were 24 in. (610 mm) wide that were cut as shown in Fig. 6.

After the surface preparation and application of the bonding agent, the first layer of fibers was applied at an angle of +45 degrees from the horizontal (see Fig. 6) and was saturated with epoxy resin; this consisted of Shell epoxy resin 826 and Shell 3579 as the curing agent in a 2:1 ratio by weight.

The second layer of carbon fibers was then applied at –45 degrees from the horizontal (see Fig. 6) to form the FRP composite shear connection as shown in Fig. 6. The width of the connection was 22 in. (559 mm), and the height of the connection at the inter-

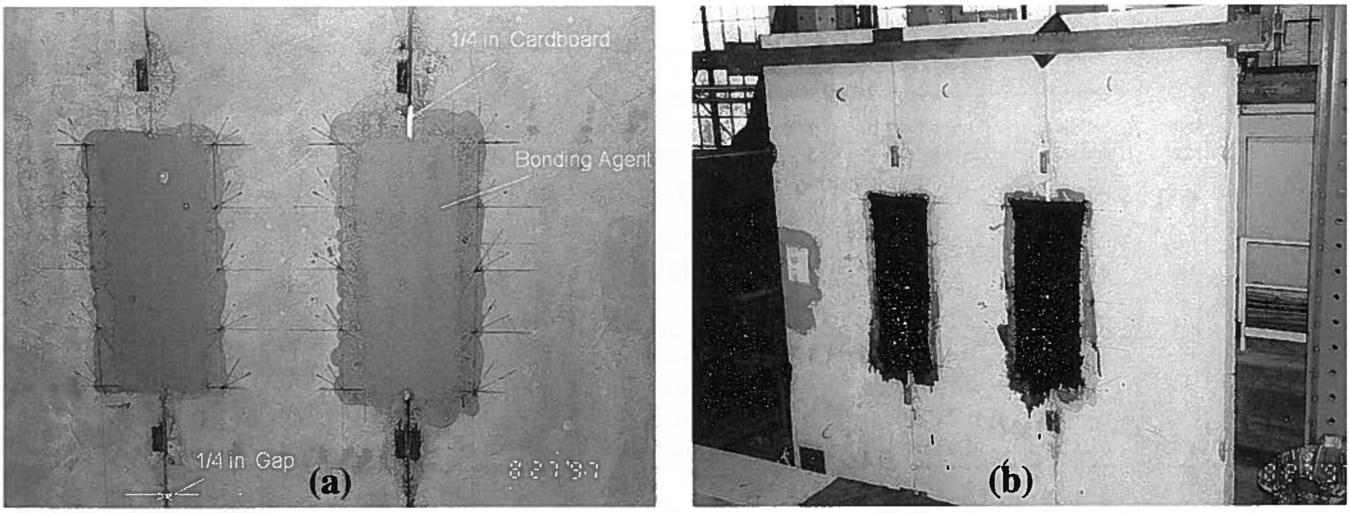

Fig. 7. Application of carbon FRP composite for Assembly 1: (a) Bonding agent application, (b) FRP composite sheet application.

face between the two concrete panels was 34 in. (864 mm).

The FRP composite connection design was evaluated using a computer program based on laminate theory,[19] in terms of the dimension of the connection, the angle of fibers in each ply with respect to the horizontal, and the thickness of the FRP composite. The FRP composite connections were cured at room temperature and allowed to cure for seven days before testing.

Typical properties of the FRP composite for the 12k and 48k tow fibers were obtained using ASTM D 3039 methods,[20] and these are shown in Table 1.

EXPERIMENTAL RESULTS

Each wall assembly was tested twice, first with the steel welded edge connectors and then with the FRP composite shear connections. The welded connections in the wall assembly were tested first, after which the welds were cut, the FRP composite unidirectional sheets were applied, and the wall assembly was tested again. The results for welded connectors are presented in Reference 15.

The carbon FRP composite shear connection details for the nine tests are given in Table 3. Note that the notation 6@±60° means that the first carbon fiber layer was applied at an angle of +60 degrees with respect to the horizontal, followed by fibers at an angle of –60 degrees; this was repeated two more times, resulting in a total of six layers for one connection.

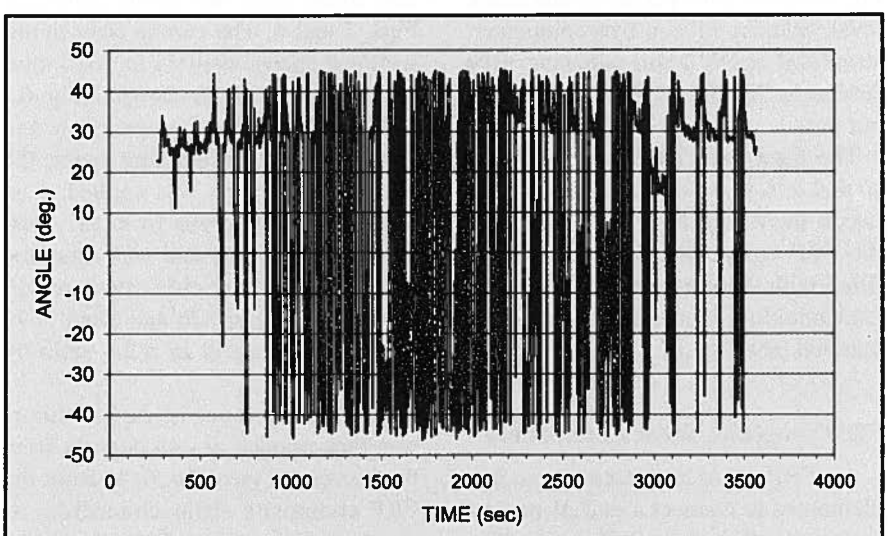

Fig. 8. Strain gauge rosette readings on FRP composite to determine optimal direction of fibers.

Assembly 1

In this test, six carbon fiber sheets, each 16 in. wide and 48 in. deep (406 x 1219 mm) in a rectangular shape, were placed at an angle of ±60 degrees. Fig. 7(a) shows the application of the bonding agent and 1/4 in. cardboard in the gap, and Fig. 7(b) shows the three precast concrete wall panels, the steel belt, and the two FRP composite shear connections. The fibers used were 12k tows. (Properties of the FRP composite are given in Table 1.)

Three-element strain gauge rosettes were placed on the surface of the FRP composites to determine their strain characteristics during testing. Tensile failure of the concrete wall support at the bottom right pin connection terminated this test.

This support failure occurred at a lateral load of 40.9 kips (182 kN). The carbon FRP composite shear connection remained in its original condition without any visible damage; therefore, the contact or bond area between carbon FRP composite and concrete was found to be more than sufficient. Strain gauge rosette measurements on the FRP composite confirmed that the trend for the maximum strain in the FRP composite was 45 degrees from the horizontal, as shown in Fig. 8.

Because of this behavior, fibers

were configured at ±45 degrees for all subsequent tests. In addition, the number of FRP composite layers was reduced to avoid any possible failure of the supports.

Assembly 2

Two 16 x 24 in. (406 x 610 mm) carbon fiber sheets, configured in a "butterfly" shape similar to that shown in Figs. 3 and 6, were placed at an angle of ±45 degrees. For this connection, 48k fibers were used.

At the 30 kip (133 kN) load step, the carbon fiber sheets on the left connection began to delaminate from the concrete surface and fail in compression. At a lateral load of 36.1 kips (160 kN), delamination of the connection occurred and continued until the whole left connection completely delaminated (see Fig. 9). An adhesive type of failure occurred at the interface between the bonding agent and the concrete surface.

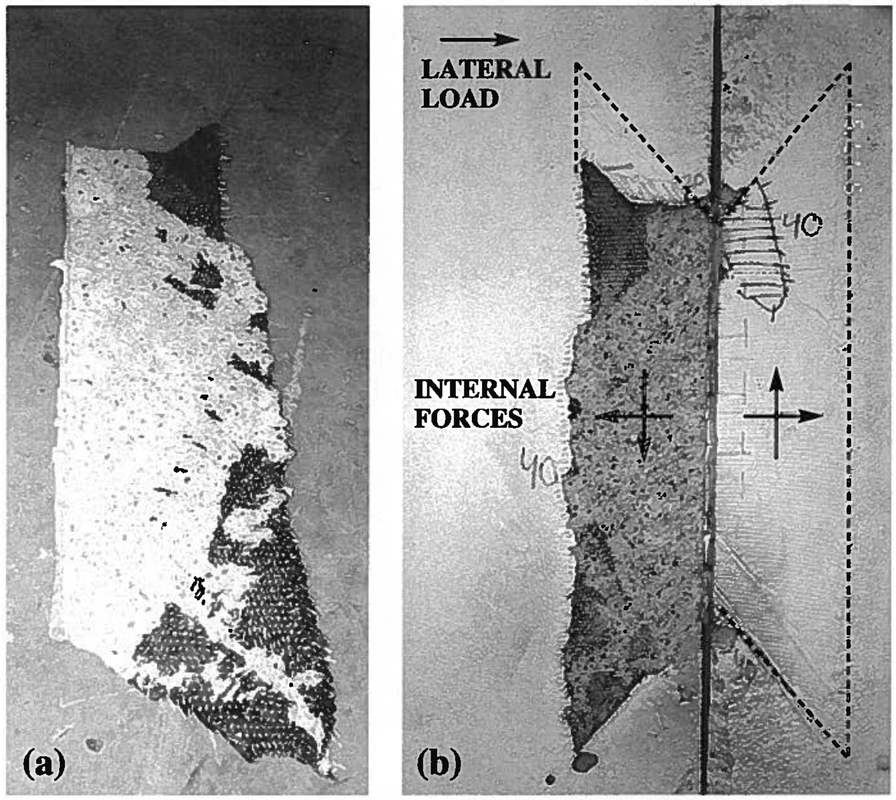

Fig. 9. Adhesive mode of failure in the bonding agent for Assembly 2.

This failure started at the corner of the V-intersection of the fibers. Fig. 9(a) shows the FRP composite plate after it was cut at the wall interface and turned over to expose the surface of the FRP composite. Fig. 9(b) shows the condition of the concrete underneath the FRP composite plate of Fig. 9(a); the outline of the FRP composite shear connection is shown by the dotted line in Fig. 9(b).

The direction of the lateral load imposed by the steel belt is shown in Fig. 9(b). Due to the rotation of the concrete panels, which was allowed by the test setup, both vertical shear and horizontal tension-compression were applied on the connection [see Fig. 9(b)]. Very little concrete was bonded to the FRP composite plate due to the adhesive failure of the bonding agent; this type of failure should be avoided.

Assembly 3

For this wall assembly, 12k carbon fiber fabric was used with rectangular connection dimensions of 15 in. wide and 22 in. high (381 x 559 mm), for a contact area of 330 sq in. (0.22 m^2). The four layers of carbon fiber sheets were rectangular in shape, similar to Assembly 1 shown in Fig. 7(b), and placed on the wall assembly at an angle of ±45 degrees from the horizontal.

At 20 kips (89 kN), the carbon fiber sheets on the left connection began to delaminate from the concrete surface. Delamination continued until complete failure occurred at the left connection at 23 kips (102 kN); the right connection failed shortly thereafter. The failure mode was cohesive concrete failure.

The FRP connection failed relatively early compared to other tests. This was expected because of the fewer number of layers used (two-thirds that of Assembly 1) and the smaller surface area of the connection (43 percent of Assembly 1).

Assembly 4

In this test, 48k tow carbon fiber was used for the FRP composite connections. Two layers of carbon fiber sheets were placed on the wall panel assembly at an angle of ±45 degrees in the "butterfly" shape (see Figs. 3 and 6).

The connection dimensions were 16 x 48 in. (406 x 1219 mm), for a contact area of 896 sq in. (0.58 m^2) per connection.

Tensile failure of the concrete at the lower right 2 in. (51 mm) pin support occurred at 35.1 kips (156 kN). Failure occurred due to insufficient reinforcement of the concrete at the pinned support. This failure mode was similar to that of Assembly 1.

Assembly 5

The fifth wall assembly was not tested with FRP composite shear connections because the concrete around the pin supports failed during testing of the welded steel edge connections. In subsequent tests for both welded steel connections and FRP composite shear connections, the outer wall panels were reinforced with FRP composite sheets at the lower left and right corner near the 2 in. (51 mm) pins, in an effort to prevent premature failure of the concrete at these locations and early termination of the test.

Assembly 6

The details of the FRP composite shear connection were similar to Assembly 4, in the "butterfly" shape, except that the connection width was 23 x 34 in. (584 x 864 mm), for a total contact area of 1046 sq in. (0.68 m^2) per

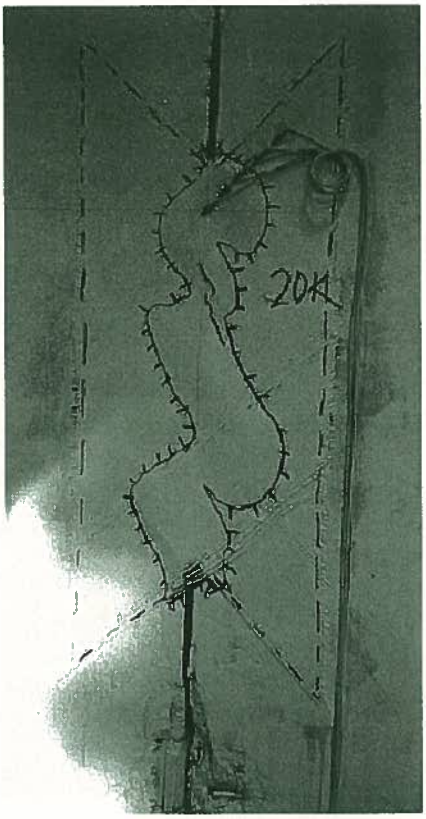

Fig. 10. FRP composite connection following adhesive resin failure of Assembly 6.

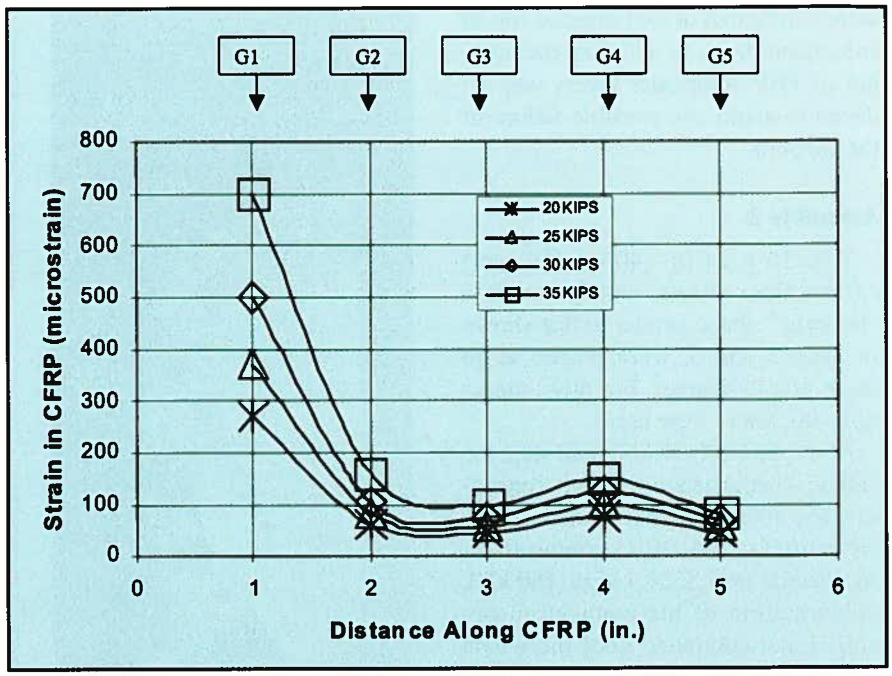

Fig. 11. FRP composite strain distribution of Assembly 7 as function of distance from the gap.

connection (see Fig. 3). The FRP composite was applied directly to the wall panel surface after it had been cleaned with a wire brush and vacuumed.

This assembly was the only one for which the concrete surface was not prepared with water jet; further, no bonding agent was used, but rather a layer of resin was applied to the concrete surface. This was done to evaluate the performance of the FRP composite connection with an inferior surface preparation.

Early in the loading steps, the FRP composite began to lift from the panel's concrete surface at the central area of the FRP shear connection in a zig-zag configuration, as shown by the dotted lines of Fig. 10; these lines were determined by tapping the FRP composite with a coin. When the FRP composite peeled off the concrete surface, a hollow sound could be heard.

The FRP composite eventually began to fail in compression due to debonding of the FRP composite from the concrete surface. This debonding action continued until the left connection failed completely, as shown in Fig. 10. The lateral load, 15.5 kips (69 kN), was the smallest of any wall assembly, which demonstrates the importance of surface preparation for the FRP composite connection. This failure mode was an adhesive failure in the resin.

Assembly 7

The details of the FRP composite shear connection were similar to Assembly 4. The configuration was a butterfly shape, with connection dimensions of 22 x 34 in. (559 x 864 mm), for a total contact area of 990 sq in. (0.64 m^2) per connection. Strain gauges were configured similarly to the gauges shown in Fig. 3.

The strain gauges were placed in the fiber direction and spaced 1 in. (25 mm) apart (see Fig. 3). The strain in the fibers was highest near the gap between the walls. Gauge G1 indicated a strain of 1500 microstrain, which gradually decreased to 100 microstrain in Gauge G5. This length of approximately 5 in. (127 mm) corresponds to the bond length required to develop the strength of the connection. The strain distribution was approximately exponential, as shown in Fig. 11.

The strains in the fiber direction measured by Gauges V2, V3, and V4, which were offset vertically, were approximately the same as the strain in Gauge V1 directly above them. This behavior suggests that the strength of the connection is proportional to the vertical height of the FRP composite in the connection, which was also substantiated from strain measurements from the other tests.

At 44.4 kips (197 kN), the FRP composite sheets on the right connection began to delaminate from the concrete surface, and the FRP composite shear connection failed. This was the first time a failure occurred in the FRP composite, and it occurred after a cohesive concrete failure had taken place.

Assembly 8

The details of the carbon FRP composite shear connection were identical to those of Assembly 7. At 34.8 kips (155 kN), the carbon FRP composite sheet on the right connection began to delaminate from the concrete surface. Delamination began at the top of the left side of the connection and propagated down toward the bottom. This behavior occurred because placement of the wall panels was not even, making stress concentrations inevitable. The failure mode was a cohesive concrete failure.

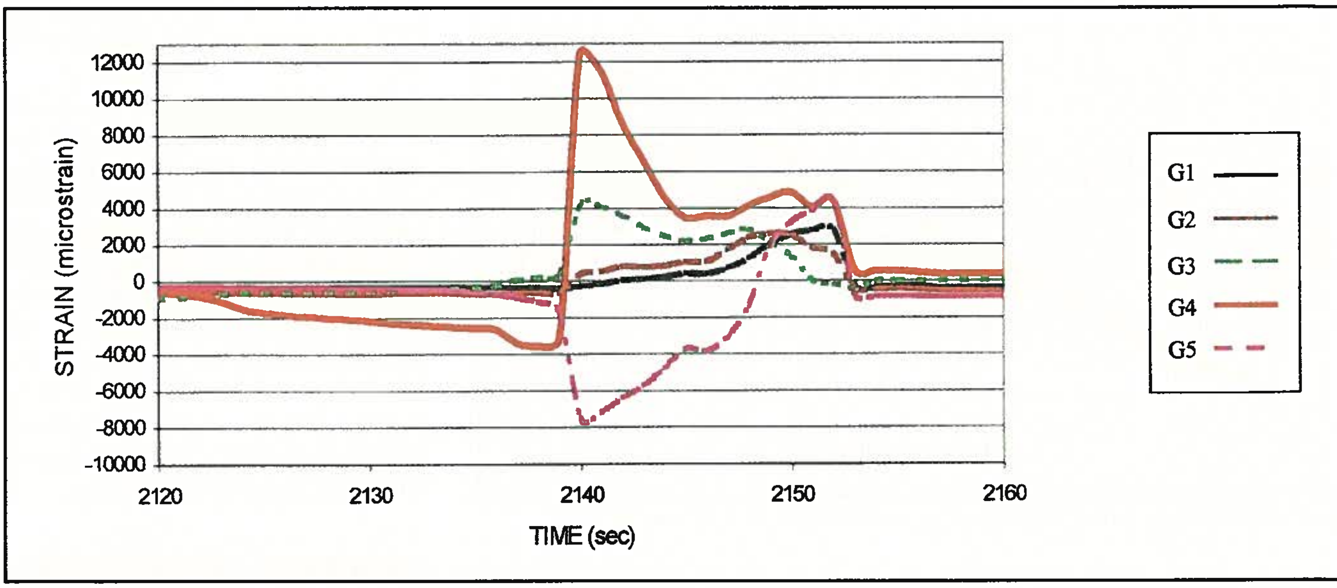

Fig. 12. Strain gauge readings for left connection of Assembly 9.

Assembly 9

The details of the surface preparation, the FRP composite shear connection, and the strain gauge orientation were identical to those of Assemblies 7 and 8. At 35.3 kips (157 kN), the carbon FRP composite sheets on the left connection began to delaminate from the concrete surface.

Delamination started at the left connection until the FRP composite sheet completely delaminated from the concrete surface. This failure mode was a cohesive concrete failure, similar to that of Assemblies 7 and 8. The strain gauge readings from the left connection are shown in Fig. 12. The notation for the strain gauges is identical to that shown in Fig. 2. The FRP composite at the left connection experienced a sudden strain increase up to 1.2 percent just before the cohesive concrete failure of the connection; this was probably due to localized stress concentrations.

The hysteresis loops for the ninth wall assembly, shown in Fig. 13, are stable but narrow, which indicates that the connection did not absorb sufficient energy to exhibit ductile behavior. The behavior shown in Fig. 13 is typical of the hysteresis loops for all wall assemblies that were tested.

Assembly 10

The details of the surface preparation, the FRP composite shear connection, and the strain gauge orientation were identical to those of Assemblies 7, 8, and 9, except that the connection width was 27 in. (686 mm), rather than 22 in. (559 mm). The vertical connection height remained 34 in. (864 mm), for a total contact area of 1283 sq in. (0.83 m^2).

At the 30 kips (133 kN) load step, the carbon FRP composite sheets on the left connection began to delaminate from the concrete surface. At 33.1 kips (147 kN), the FRP composite completely delaminated from the concrete surface at the left connection. This failure mode was a cohesive concrete failure.

GENERALIZED BEHAVIOR OF RETROFITTED SHEAR CONNECTIONS

As demonstrated by the poor behavior of Assembly 6, surface preparation is extremely important. That wall assembly surface was prepared only by wire brushing, and failure occurred in an adhesive failure mode in the resin.

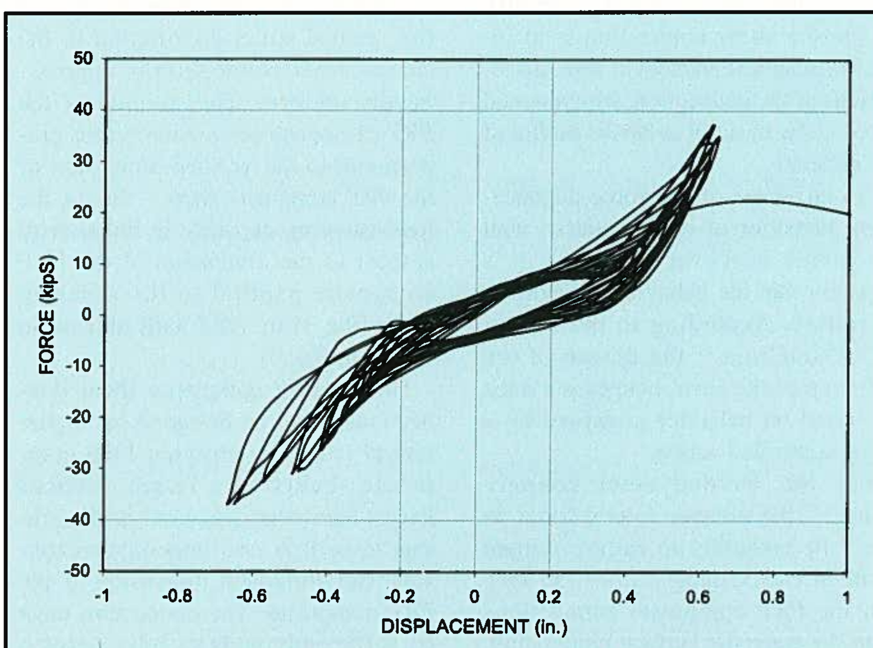

Fig. 13. Hysterisis curve for Assembly 9.

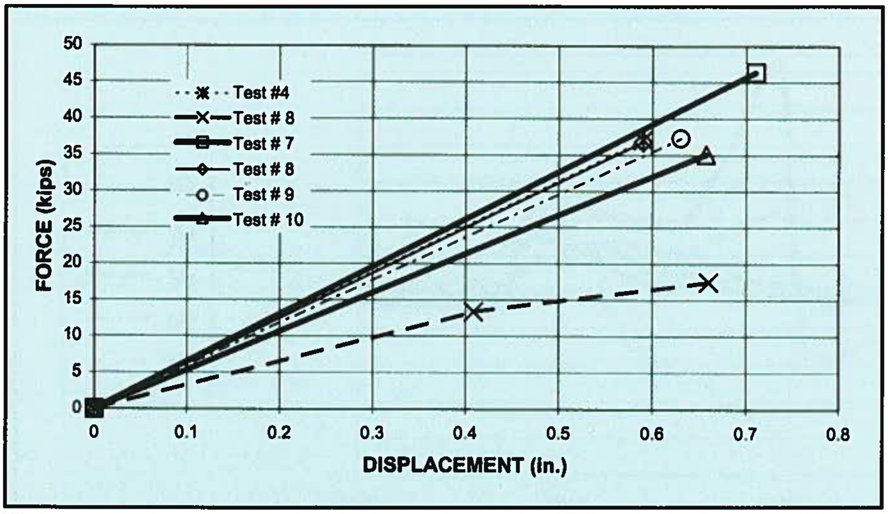

Fig. 14. Force-displacement behavior of Assemblies 4, 6, 7, 8, 9, and 10.

When the cohesive concrete failure mode occurred, after application of the water jet, the depth of the concrete that was peeled off the wall panel was much larger than the adhesive failure mode in the resin or bonding agent. The depth of the concrete that was peeled off the wall panel by the FRP composite plate ranged from $^1/_{12}$ to $^3/_4$ in. (2 to 19 mm) in the cohesive concrete failure mode.

The corresponding maximum strain in the FRP composite ranged from 0.24 to 0.78 percent, which is less than the ultimate tensile strain of the FRP composite, which ranged between 1.33 and 1.36 percent. The results for Assemblies 3, 7, 8, 9, and 10 suggest that one possible way to improve the FRP composite shear connection is to install mechanical anchors at selected locations of the connection, which would prolong the onset of cohesive failure of the concrete.

An envelope of the force-displacement behavior of representative wall assemblies is shown in Fig. 14. It is apparent that the behavior is linear up to failure. According to the FEMA 273 Guidelines,[11] the design of the FRP composite shear connection must be based on behavior governed by a force-controlled action.

For the welded steel connections,[15,17] the ultimate lateral force on the wall assembly to failure ranged from 28.1 to 35 kips (125 to 156 kN). For the FRP composite connections with the water-jet surface preparation, the ultimate lateral force to failure ranged from 23 to 44.4 kips (102 to 197 kN). Note that the capacity of the FRP composite connections was limited by the test setup configuration. In particular, the lateral load at failure for Assemblies 1 and 4 might have been considerably higher.

The ultimate lateral displacement of the wall assembly for welded connections ranged from 0.53 to 0.80 in. (13 to 20 mm), whereas for the FRP composite connections with the water jet surface preparation it ranged from 0.52 to 0.71 in. (13 to 18 mm), which indicates that both connections are nonductile and should be designed as force-controlled elements.

Strain measurements on the FRP composite connection have shown that the vertical stress distribution in the carbon fiber connection is approximately uniform. The strength of the FRP composite connection is thus proportional to the vertical dimension of the FRP composite sheet – that is, the load-carrying capacity is linear with respect to the dimension of the FRP composite parallel to the shearing plane [the 34 in. (864 mm) dimension shown in Fig. 3].

Hence, FRP composite shear connections could be designed for higher lateral loads by applying FRP composite sheets on a larger vertical length than what was used in the present tests. It is also important to consider the horizontal dimension of the FRP composite. The connection must be sufficiently wide to fully develop the strength of the fibers and depends on the effective bond length of the FRP composite and the required bond area.[21]

The results presented in this paper are the first of a series of studies on the subject of FRP composite shear connections. Subsequent studies have been carried out which have developed the FRP shear connection further by studying such effects as number of concrete panels, out-of-plane bending, use of adhesive anchors, and variations of the FRP composite shear connection size, number of layers, and location. These studies are currently being developed into design recommendations.[22]

PRACTICAL IMPLICATIONS

In this investigation, the FRP composite connections failed in three distinct modes: cohesive concrete failure, which is a substrate failure in the concrete and was the most frequent failure mode, adhesive failure of the bonding agent, and adhesive failure of the resin, which occurred only for the wire-brushed panels.

The strength of the FRP composite shear connection with the cohesive concrete failure mode depends on the vertical length of the FRP composite, the number of FRP composite layers, the area of the connection, the surface tension strength of the concrete, the effective bond length of the carbon fibers, and the concrete surface condition. The other two adhesive failure modes can be prevented if the concrete surface is prepared properly.

The water-jet surface preparation provides a superior bonding surface for both strength and economy. It is important that the on-site surface preparation be monitored and inspected. In this study, the butterfly-shaped connection showed a tendency to develop stress concentrations; therefore, the use of a rectangular-shaped connection is recommended.

The experimental behavior of the FRP composite connection is linear up to failure, which is sudden and brittle. However, one could design for this type of linear behavior in seismic retrofit by following the FEMA 273 Guidelines for a force-controlled action.

The use of mechanical anchors could improve the cohesion strength of the concrete and thus increase the strain at failure of the FRP composite, and the capacity of the connection. Both welded steel plate[15] and FRP composite connections are non-ductile and should be designed as force-controlled elements.

The significant difference between the welded steel plate and FRP composite connections is that the latter can be made sufficiently strong to withstand large earthquake loads. The capacity of the FRP composite connection could be increased significantly, compared to a currently used welded connection, by providing a longer vertical length of FRP composite connection.

Design guidelines for FRP composite shear connections are currently under development. The basic steps include: (1) determining the force demand in the connection, (2) providing sufficient bond length and bond area for the FRP composite connection, and (3) designing the connection dimensions based on the FRP composite allowable strain, which in turn is based on the FRP composite material properties.

The results of this experimental study can be extended to FRP composite shear connections between horizontal panels in floor and roof diaphragms. In addition, instead of the dry lay-up FRP composite used in this study, the use of pre-impregnated fiber reinforced polymer composite material should be considered.

CONCLUSIONS

A carbon FRP composite shear connection was investigated for the possible seismic retrofit and rehabilitation of precast concrete wall panel connections. Based on the results of this investigation, the following conclusions can be drawn:

1. The FRP composite laminate successfully restored the shear connection between damaged concrete wall panels. For certain FRP composite laminates and lay-ups, the original shear capacity provided by the welded loose plate connections was increased between 17 and 40 percent; however, it should be noted that this increase could be much higher in actual structures because in the experiments the limiting condition was support failure rather than failure of the FRP composite connection.

2. Provided that there is adequate surface preparation, such as high-pressure water jet with application of a bonding agent, a cohesive concrete failure mode can be achieved at relatively high FRP composite strain levels. Wire brushing is not an effective surface preparation procedure for this application.

3. The design of such an FRP shear connection to resist a target shear force can be achieved by using equations from classical lamination theory. Several commercial and educational software programs are available that can assist the designer in this task.

4. The capacity of the FRP composite shear connection depends on the FRP composite laminate – that is, the vertical length of the FRP connection, fiber type and orientation, lay-up, and number of plies – and on the concrete surface tension capacity and surface preparation. As expected, the heavier [+45, –45] carbon FRP composite laminate with a large surface connection area resulted in the highest lateral load resistance.

5. The butterfly-shaped connections developed higher stress concentrations than the rectangular-shaped connections, and hence the latter are preferred. Furthermore, for architectural reasons the FRP composite shear connection was applied to only one side of the wall (the same side as the welded connection). If there is a requirement for higher shear resistance, both sides could be considered in the design of the FRP composite shear connection. Finally, instead of the dry lay-up FRP composite used in this study, the use of a pre-impregnated fiber reinforced polymer composite material should be considered.

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