A New Structurally and Thermally Efficient Precast Sandwich Panel System

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A new precast concrete sandwich panel system with a high thermal resistance and optimum structural performance has been developed. A hybrid truss provides the connector in this panel system — the diagonals are fiber-reinforced plastic bars and the chords are prestressed steel strands. Each connector consists of a fiber-reinforced plastic bar fabricated in a deformed spiral shape through which a pair of prestressing strands is threaded to provide anchorage in the concrete wythes. The developed shear connecting system is described together with its advantages. An experimental and analytical investigation of the connecting system was conducted. The experimental program included testing of small scale specimens by push-off (pure shear) loading, small scale specimens by flexural loading, and full scale panels by flexural loading. The analytical investigation included finite element modeling of the tested small scale specimens and comparisons with theory of elasticity solutions. Experimental and analytical results from finite element modeling and from theory of elasticity equations correlated well and showed that the developed panel system meets the objectives of the research and is expected to have a promising future.

Precast concrete sandwich panels (PCSPs) are structurally and thermally efficient elements used for exterior walls in multi-unit residential, commercial, and warehouse buildings throughout North America. A typical PCSP consists of two precast reinforced concrete layers (called wythes) separated by a layer of insulation and joined with connectors that penetrate the insulation layer.

The connectors used in the majority of available PCSP systems consist of concrete webs or blocks, steel elements, or a combination of these components. Because of their low thermal resistance, steel and concrete connectors can significantly reduce the effective thermal resistance of the panel through thermal bridging.

PCSPs outperform other construction materials because of their superior thermal and structural efficiency. For example, PCSP walls require lower peak loads by about 13 percent for heating and 30 percent for cooling than insulated metal or wood-framed walls having the same U-value under the same heat gradient conditions.[1]

Like all other precast concrete products, PCSP walls possess several beneficial features such as high quality, proven durability, fast erection, and attractive architectural appearance.

Several PCSP systems are available in European and American markets.[2] They can be divided into three major categories:

1. Composite panels with concrete and steel connectors — These panels are used as both architectural and bearing walls because of their composite behavior (see Fig. 1a).

2. Non-composite panels with steel connectors — These panels are used only as architectural walls (see Fig. 1b).

3. Non-composite panels with non-metallic connectors (see Fig. 1c).

The first category is structurally efficient because of its composite behavior. However, these systems typically have two disadvantages: steel and concrete connectors cause significant reduction in the effective thermal resistance of the panel through thermal bridging,[3] and excessive undesirable panel bowing (thermal bowing) may occur due to different temperatures at their interior and exterior surfaces.

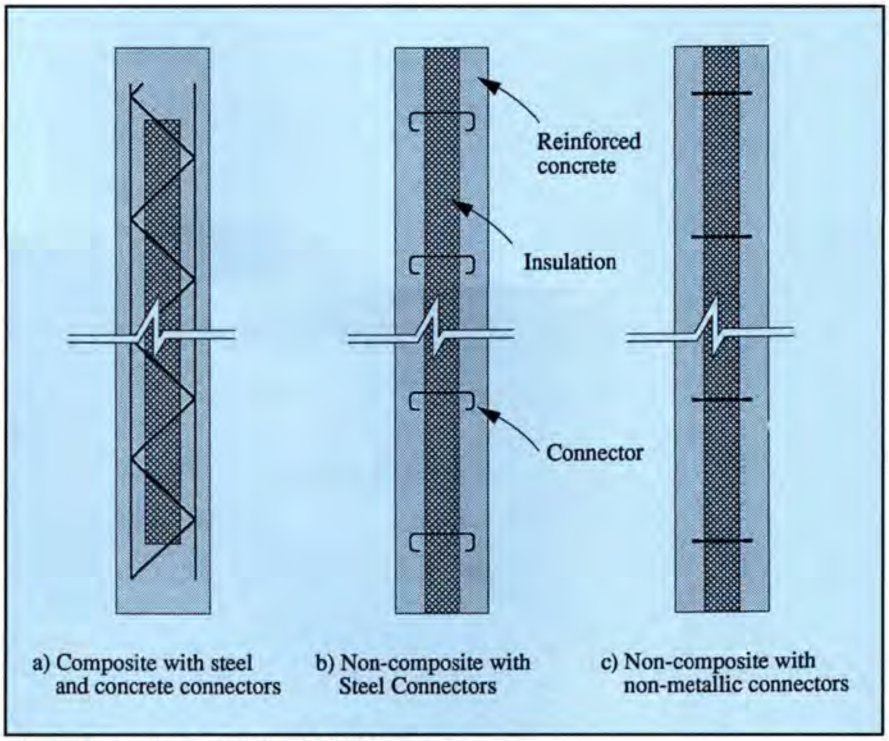

Fig. 1. Examples of existing PCSP systems.

The second and third panel categories do not experience thermal bowing, but are not structurally efficient with the third being only thermally efficient.

The objective of the research summarized in this paper is to devise a sandwich panel system that is both structurally and thermally efficient. This can be achieved by devising a shear connecting system that has the highest possible thermal resistance and provides enough shear capacity for optimum composite action between the exterior reinforced concrete wythes.

To fulfill this objective, the authors developed a new PCSP system that provides both composite strength and thermal efficiency. This new system uses fiber-reinforced plastic (FRP) connectors that have high thermal resistance and can develop the composite strength of the panel. Although the initial material cost is marginally higher than steel or concrete connectors at the present time, their use can reduce considerably the long term building heating/cooling costs.

This paper describes a new PCSP system developed at the University of Nebraska including experimental evaluation through small scale and full scale testing, analytical modeling, and a discussion of the results. The results demonstrate the structural efficiency of the new system. Evaluation of the system's thermal efficiency is in progress with results expected in the near future.

DESCRIPTION OF THE NEW PCSP SYSTEM

The primary difference between the new PCSP system and other existing systems is the method of connection through the insulation wythe. A thermally and structurally efficient PCSP system requires the following features:

1. The connectors must be strong and stiff enough to develop composite behavior of the panels.

2. The connectors must have a high thermal resistance.

3. No concrete penetrations through the insulation layer should exist.

The use of FRP connectors, which are strong and thermally resistant, satisfies the first two criteria while the third requires special precautions during panel manufacturing to prevent concrete penetrations through the insulation layer.

A preliminary evaluation of the candidate connectors shown in Fig. 2 led to the selection of the FRP bent bar (FRPBB) connector (see Fig. 2d) for further evaluation. The evaluation

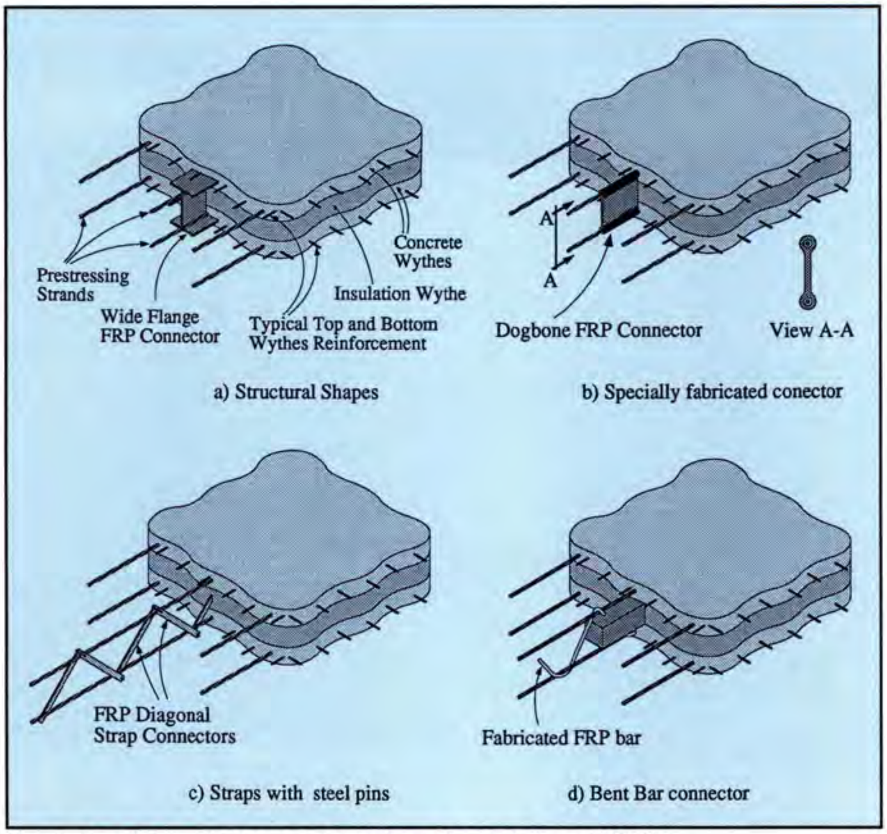

Fig. 2. Candidate FRP connectors shown in partially cut PCSPs.

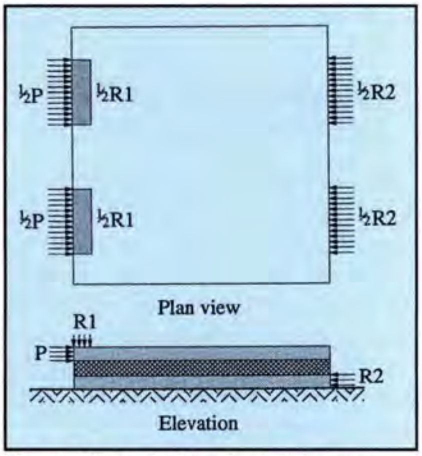

Fig. 3. Shear testing load diagram.

consisted of tests similar to the shear tests described below. The tested specimens in this evaluation step included various FRP connectors made from sections readily available, such as channels and wide flanges (see Fig. 2a). Manufacturing such specimens was laborious and their performance was inferior due to weak anchorage capability.

The connector shown in Fig. 2b was estimated to be more expensive to produce, cut, and place in the panels. In addition, possible air voids around the strand, inside the connectors, rendered this type of connector unsuitable. The third type of connector, shown in Fig. 2c, was also expected to be expensive due to the need for FRP straps and steel pins. Also, the various components are difficult to physically fit into the panels.

The FRPBB connector consists of an FRP bar fabricated in a deformed helical shape with approximate inclination angles of 45 degrees. The cross section of the bar is circular with a diameter as required by design. Two reinforcing bars or prestressing strands are threaded into the FRPBB during construction of the panel to form a truss with steel chords and FRP diagonals. Installation of the FRPBB connectors requires only modest labor and no special skills other than threading the top and bottom strands (chords) into the FRP bents. Embedding these chords into the wythes provides good anchorage for the FRP bars.

The FRPBB connectors are encased in foam insulation blocks before casting to prevent concrete penetrations during construction of the panels.

TESTING

Two small scale testing programs were conducted to assess the behavior of the FRPBB connectors, to obtain their strength and stiffness, and to provide information for the development of full scale specimens.

Shear Testing

Initial evaluation of the structural performance of the FRP bent bar connectors in shear was performed using a sequence of pure shear (push-off) tests. This type of test has been used by previous researchers for similar purposes.[3,4] These tests are conducted by placing each specimen in a horizontal position and pushing the top wythe relative to the bottom one in a specially designed steel frame (see Fig. 3).

Eight specimens with identical overall dimensions were constructed for shear testing. The insulation wythe consists of a 3 in. (76 mm) expanded polystyrene (EPS) insulation board. Normal weight concrete with f'_c = 5000 psi (34.5 MPa) is used for the top and bottom wythes. Along with the #3 (10 mm diameter) steel reinforcing bar threaded into the FRPBB connector, each wythe is reinforced with a layer of Type 4x4-W4.0xW4.0 (102x102-MW25.8xMW25.8) welded wire fabric to resist stresses due to stripping and handling.

The fabrication process and quality control were similar to those used in producing commercial PCSPs. Fig. 4 contains a summary of specimen differences. The insulation used in Specimens 1, 2, 3, and 4 was faced with bond breaking sheets, while the insulation used in Specimens 1A, 2A, 3A, and 4A was not.

Summary of Results — Einea provides a full discussion of the results obtained from the shear tests in Ref. 5. The significant observations from this report are:

1. The axial strength of the connectors governs the shear strength of all specimens. No failure occurs in the wythes.

2. The majority of the FRPBB connectors failed at the portions of the diagonals falling within the insulation layer due to axial compression, flexure combined with axial compression, and flexure combined with axial tension.

3. The apparent axial ultimate strength of the connectors used varied between 25.2 and 60 ksi (173.7 and 413.7 MPa) with that of Specimen 4 being 32.5 ksi (224.1 MPa). (The strength of the connectors in Specimen 4 is specifically mentioned here for comparison with analytical results presented below for the same specimen.)

4. Concrete bonds to unfaced insulation and causes the insulation to contribute as much as 10 percent of the shear capacity of the specimen. This contribution was obtained by comparing the capacities of the specimens containing insulation faced with bond breaking sheet with those of the specimens containing unfaced insulation.

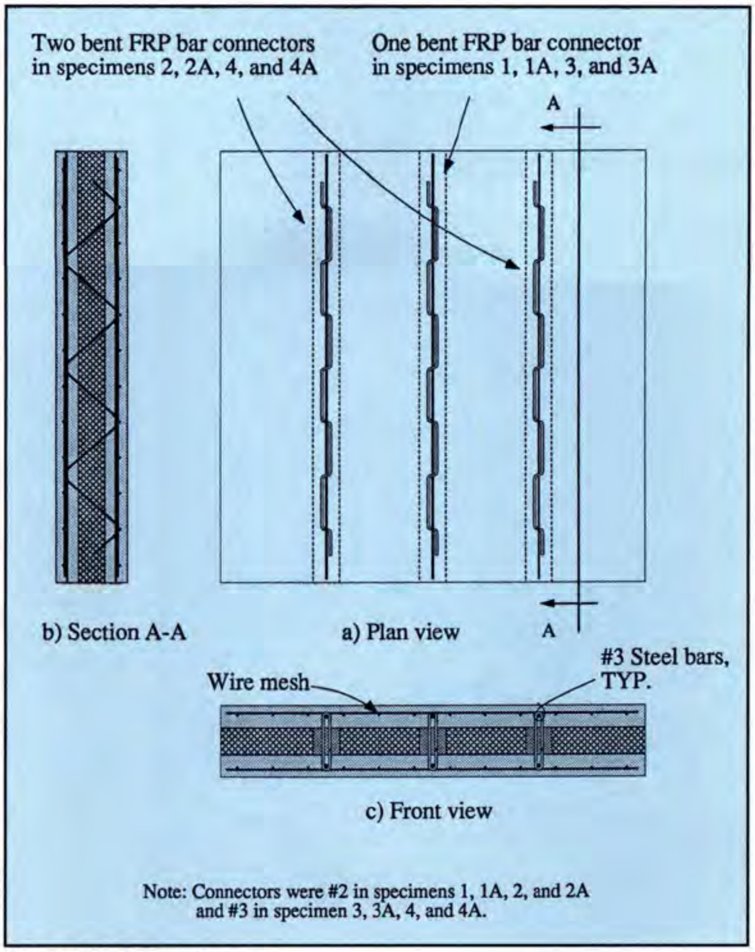

Fig. 4. Shear specimens.

Small Scale Flexural Testing

The purpose of these tests was to explore the behavior of the FRPBB connectors in flexure, a primary load condition of PCSPs in the field. Einea provides detailed descriptions of these tests in Ref. 5.

Testing Setup and Procedure — Fig. 5 shows a typical specimen, placed horizontally in a heavy steel frame, supported by a steel roller at each end, and loaded from the top with two concentrated loads. A hydraulic ram with a load capacity of 100 kips (444.8 kN) and a maximum stroke of 3 in. (76 mm) applies the load through a steel bracket at a rate that allows readings at 500 lb (2.2 kN) increments. A load cell and a displacement dial gauge measure the load and midspan deflection, respectively. Strain gauges mounted on all FRP diagonals measure their axial strains during the tests.

Specimens — Two specimens were constructed with one FRPBB connector in each. The FRP diagonals have a circular cross section with a nominal diameter of $^3/_8$ in. (10 mm). Specimen 1 contains expanded polystyrene insulation faced with bond breaker sheets and Specimen 1A contains unfaced insulation. The compressive strength of concrete at the time of testing was 6200 psi (42.7 MPa). Each bearing location contains two wood blocks to help transfer the applied loads and reactions into both wythes while allowing relative slip between the wythes (see Fig. 5).

The total area of the wood blocks was kept small, compared with the total area of the specimens, to minimize the frictional effect on the horizontal shear capacity of the specimen. Welded wire fabric of Type 4x4-W4.0xW4.0 (102x102-MW25.8xMW25.8) reinforces each concrete wythe along with the #3 (10 mm diameter) deformed steel bar that anchors the FRPBB connector.

Results and Discussion — The details of the testing procedure and results are slightly different for each of the tested specimens and are described separately below. Measurements were initiated with self weight, the weight of the steel bracket, and the weight of the loading ram all applied to the specimen. Specimen 1 was loaded to 5600 lbs (24.9 kN), unloaded, and reloaded to failure at 6340 lbs (28.2. kN).

The load-deflection curve for Specimen 1 is nonlinear during the loading stage and linear during the reloading stage up to approximately the maximum load recorded in the initial loading. Figs. 6 and 7 show this behavior in the load-deflection curve and the FRP bar strain curves, respectively. Nonlinearity during loading is clearly due to cracking of the wythes. After cracking the wythes, reloading is linear until new cracks develop beyond the maximum load previously applied.

The first crack was observed at the bottom surface of the bottom wythe at a load of 2000 lbs (8.9 kN). Onset of cracking in the top wythe occurred at a load of 3900 lbs (17.3 kN). The majority of cracks are concentrated at the location of the peak moment in each wythe, in the regions under the applied concentrated loads.

Specimen 1A was loaded directly to failure at a load of 8000 lbs (35.6 kN). In this specimen, strain gauges on Bars 3 and 4 did not function. Fig. 6 shows the load-deflection curve and Fig. 8 shows the strain in the FRP bars. The shear strength contribution of the unfaced insulation (through bonding with both concrete wythes) increases the composite behavior of Specimen 1A, and causes its higher stiffness and load capacity.

Failure of this specimen is similar to

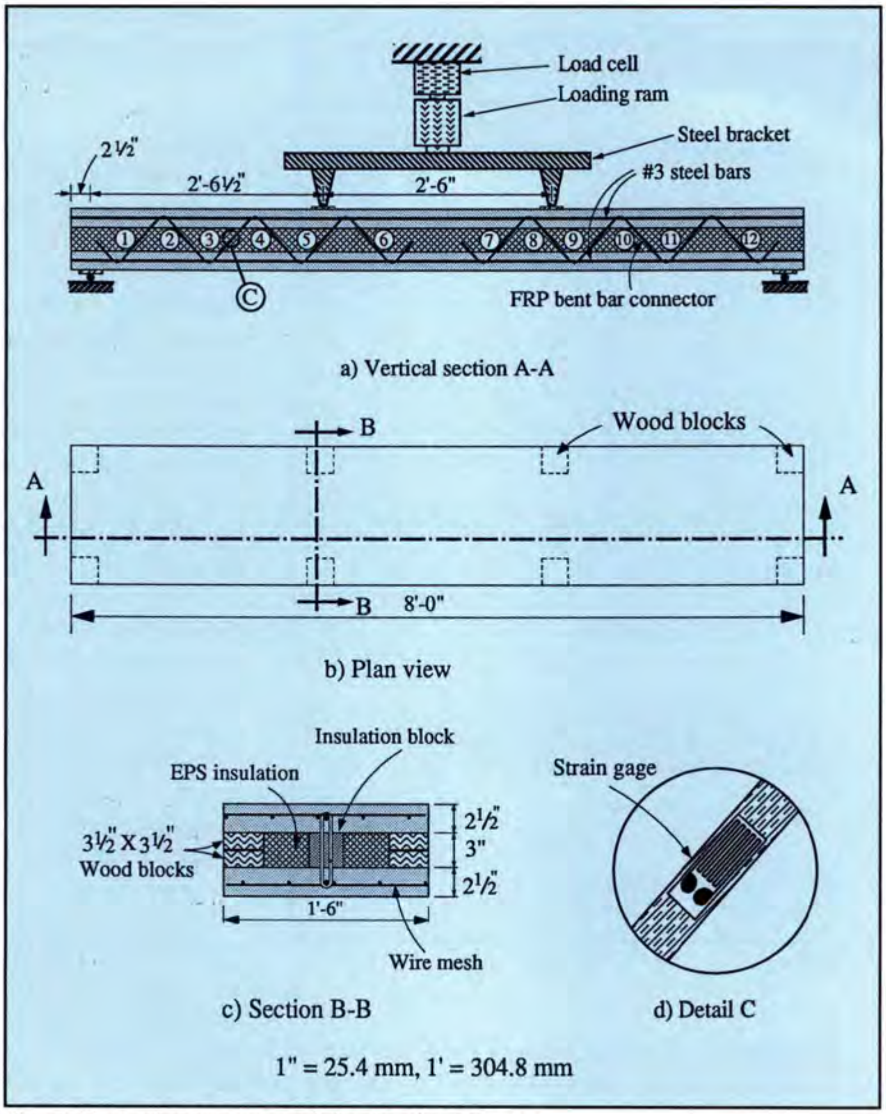

Fig. 5. Flexural test setup and details of specimens.

that of Specimen 1. Cracks initiated in the bottom wythe at a load of 3500 lbs (15.6 kN), and in the top wythe at a load of 5300 lbs (23.6 kN). Fig. 9 shows the failed specimen and the distribution of cracks.

The ultimate moments at the locations of the concentrated loads, including self weight and the weight of the steel bracket, are 107 and 133 kip-in. (12.1 and 15.0 kN-m) for Specimens 1 and 1A, respectively.

Assuming no composite action at ultimate strength (zero percent composite), the calculated elastic stiffness is $EI = 2.11 \times 10^5$ kip-in.^{2} (605.5 kN-m^2), and the ultimate flexural capacity of the specimen is $M_n = 55.4$ kip-in. (6.3 kN-m). For a fully composite panel at ultimate strength (100 percent composite), the calculated elastic stiffness is $EI = 3.278 \times 10^5$ kip-in.^{2} (9407 kN-m^2), and the ultimate flexural capacity of the specimen is $M_n = 163.5$ kip-in. (18.5 kN-m).

The elastic stiffness of Specimen 1 is $EI = 2.07 \times 10^5$ kip-in.^{2} (594 kN-m^2) and that of Specimen 1A is $EI = 3.22 \times 10^5$ kip-in.^{2} (924 kN-m^2), as calculated from the load-deflection relationship in the elastic range. Defining the percentage of composite action at ultimate strength as the ratio between the measured ultimate strength and the 100 percent composite ultimate strength, Specimen 1 is computed to be 65 percent composite and Specimen 1A is 81 percent composite.

From the above comparison of elastic and ultimate strength behavior, the two specimens behaved non-compositely at the elastic stress level although their ultimate strength was close to the full composite ultimate strength. This behavior indicates that the FRP bars slip inside the concrete at early stages of load. The forces increase gradually in these bars, as the load is increased, until their strength is reached at the ultimate strength of the specimen.

This behavior can be beneficial in reducing the bowing due to the differential temperature conditions sandwich panels can be subjected to during their service life. However, the panel system has a higher degree of composite action at ultimate strength.

According to beam flexure theory, the magnitude of the axial forces should be nearly equal in the FRP diagonals located between each support and the closest applied load since these regions are subject to a nearly constant shear. However, the observed absolute strain values in those bars varied due to such factors as the variable shear component added due to dead load of the specimen itself, manufacturing imperfections of the specimens and their components, and inaccuracy in measuring the actual strains in the FRP bars using strain gauges.

Full Scale Flexural Testing

Full scale tests of the new PCSP system are important for two main reasons: to show the practicality and simplicity of commercially manufactured panels of this type and to investigate the structural behavior of the new panel system under conditions similar to those being used in commercial applications. Culp presents a detailed account of these tests in Ref. 6.

Testing Setup and Procedure — A special free-standing loading frame was designed and constructed for these tests. This setup simulates an actual PCSP subjected to uniformly distributed wind or seismic loads (see Fig. 10). Pressure is applied to the specimens by pumping air into neoprene bags. Air pressure in the bags is measured at intervals using an electric/dial air pressure gauge. A linear variable displacement transducer (LVDT) measures out-of-plane deflection of the specimen at mid-height, and strain gauges measure the axial strain in the FRPBB connectors.

Specimens — Two identical specimens, 8 ft (2.44 m) wide by 30 ft

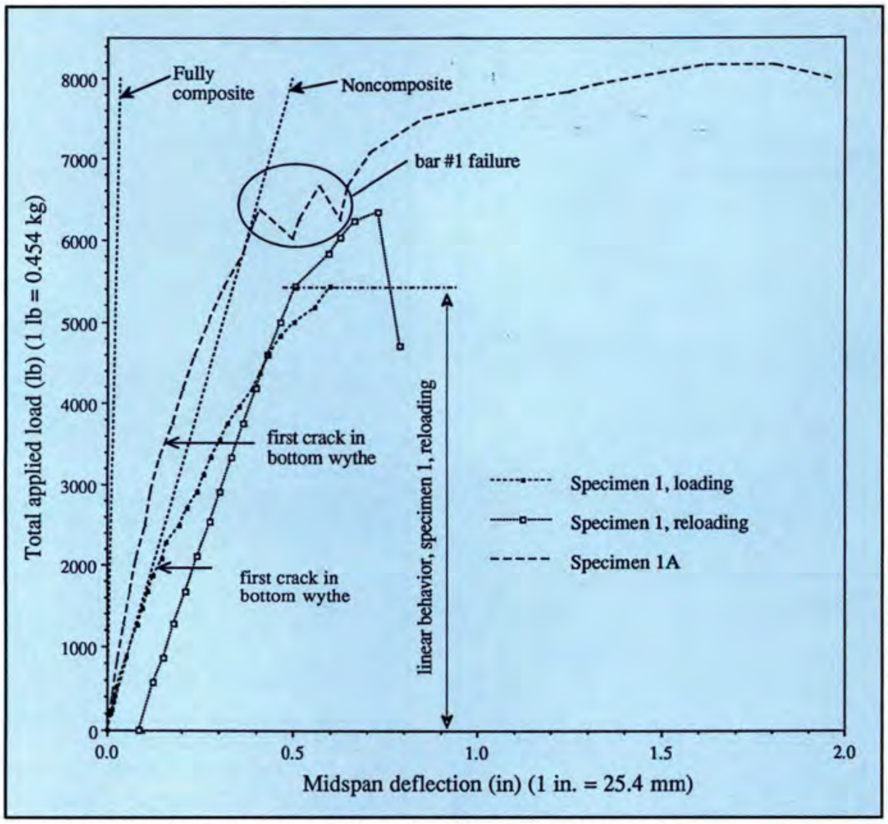

Fig. 6. Load-deflection curves for bending Specimens 1 and 1A.

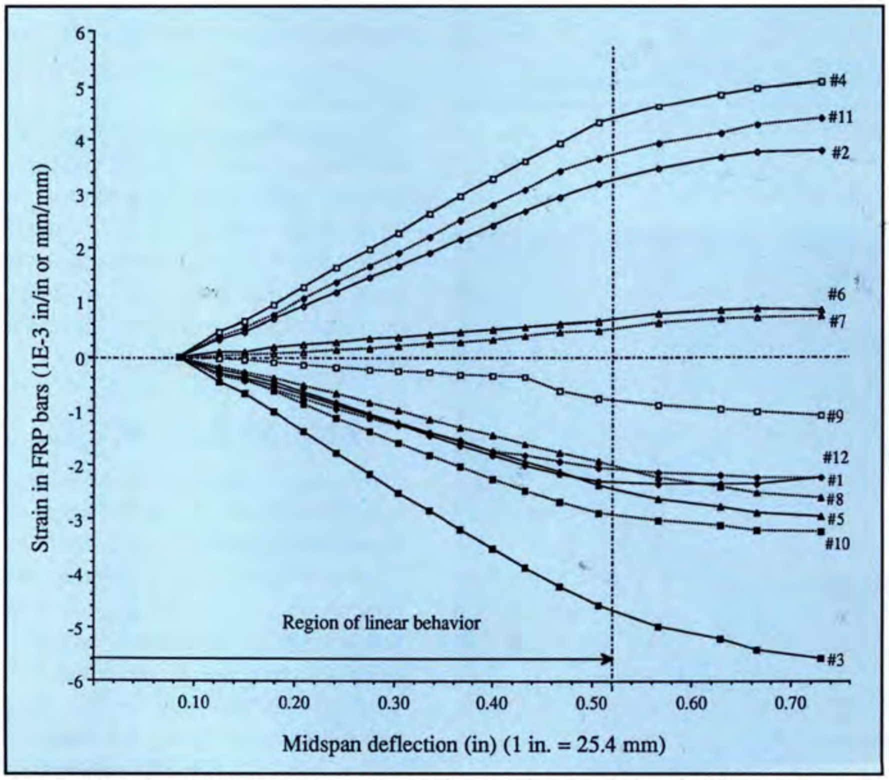

Fig. 7. Strain in FRP bars vs. midspan deflection for Specimen 1, reloading.

(9.14 m) high, were constructed. Wythe thickness in all specimens is 2.5 in. (64 mm) concrete and 3 in. (76 mm) EPS insulation with no bond breaking sheets. Concrete wythes are reinforced with Type 4x4W4.0xW4.0(102x102MW25.8x MW25.8) welded wire fabric and five ½ in. (12.5 mm) diameter, low-relaxation, Grade 270 ksi (1861 MPa) strands prestressed to an effective stress of 111 ksi (765 MPa).

The specimens were designed to support their own weight during erection. This load, 63 psf (3 kPa), is equivalent to an inertial load resulting from an earthquake acceleration of 1 g = 32.2 ft per sec^2 (9.81 m/sec^2) or a wind pressure corresponding to a UBC-1991 basic wind speed of 120 miles per hr (193 km/hr) in Exposure C.

Three of the five strands in each wythe were used as chords for the FRPBB connectors. The horizontal shear strength design of these panels was performed assuming full composite action, and the size and spacing of the connectors were selected to resist the shear as calculated standard beam flexure theory. FRPBB connectors with ⅜ in. (10 mm) nominal diameter diagonals were used in each specimen. Shear requirements dictate the use of six connectors from the panel ends to the quarter points, while three connectors were placed in the central length of the panel.

Results and Discussion — The maximum load applied to the specimens was 185 psf (8.9 kPa) for Panel 2; loading was discontinued at this stage for personnel safety precautions. This load is three times the design load. Fig. 11 shows the midspan deflection vs. applied pressure curves for the two specimens.

The panels behaved linearly up to a pressure of 60 psf (2.9 kPa). The high initial stiffness of the panel corresponds to the stiffness computed assuming the diagonals to be rigidly embedded in the concrete wythes. The decrease in stiffness that occurs at approximately 60 psf (2.9 kPa) corresponds to the computed panel stiffness if the connectors are assumed to be pinned at the point where they loop around the prestressing strands. At pressures above 80 to 100 psf (3.8 to 4.8 kPa), cracking of the wythes further reduces the stiffness of the panels.

In comparison with the small scale flexure test results, shown in Figs. 6 and 11, the full scale panels are much stiffer than the small scale specimens, although the number and size of shear connectors per unit width of panel

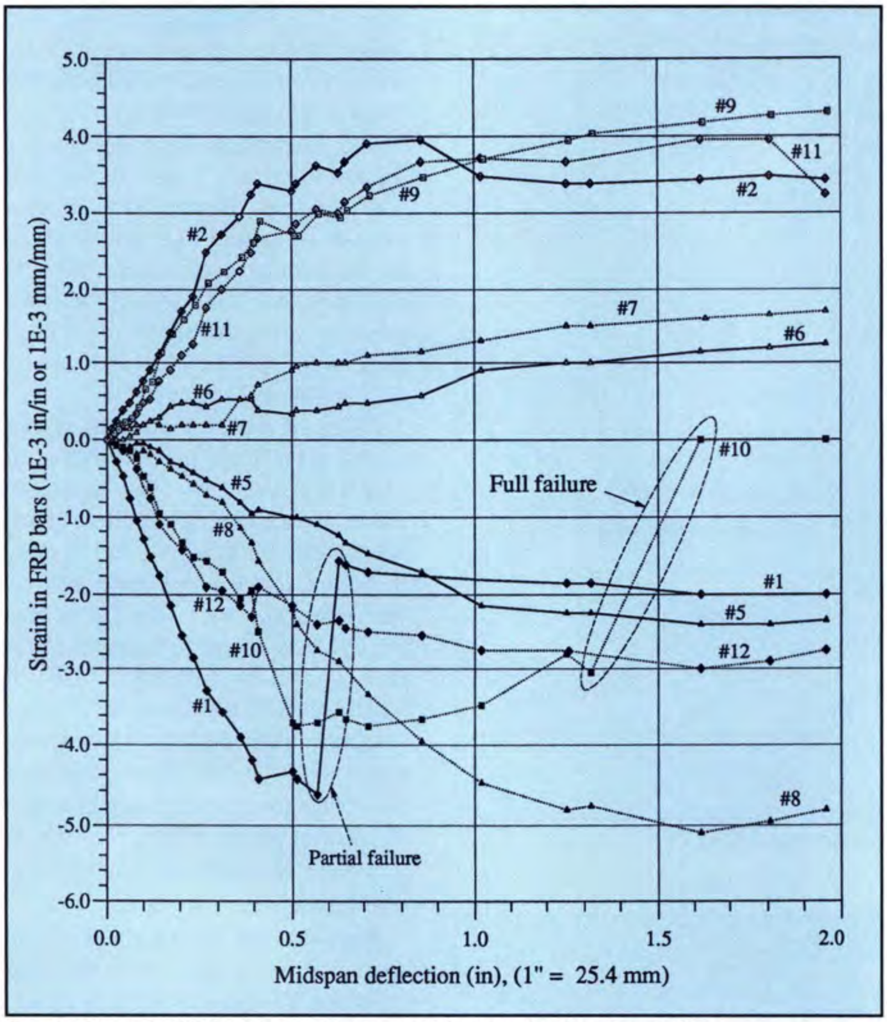

Fig. 8. Strain in FRP bars vs. midspan deflection for Specimen 1A.

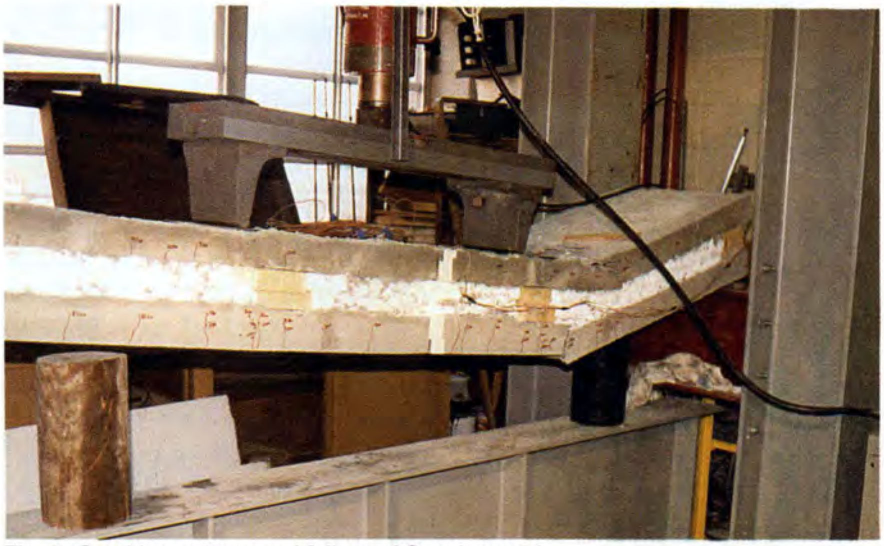

Fig. 9. Cracking pattern and failure of Specimen 1A.

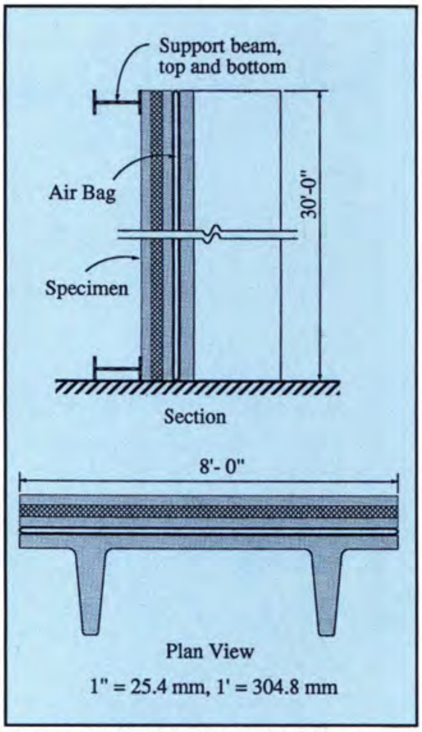

Fig. 10. Full scale flexural testing setup. (Connections of support beams to double tee not shown for clarity.)

were almost identical. Two main reasons may best explain this difference:

1. The small scale specimens were not prestressed with the connectors' chords being #3 (10 mm diameter) reinforcing bars, while the full scale panels were prestressed with the connectors' chords being 1/2 in. (12.5 mm) diameter strands. Prestressing the concrete wythes provides much more efficient anchorage to the bent FRP bars and nearly eliminates their slippage around the chords.

2. Analytical investigation indicates that longer panels with the same stiffness of shear connecting system behave more composite than shorter ones.[7] Since the full scale specimens were more than three times as long as the small scale specimens, the former should behave more composite.

ANALYTICAL MODEL

Because of the different dimensions, loading, and behavior of the shear and flexural specimens, separate finite element models are used to simulate each type of test. Each component of the sandwich panel specimens is modeled using a general finite element analysis program (ANSYS) and the appropriate two-dimensional library elements.

For stresses below the proportional limit, isoparametric plane stress elements (Q4) are used to model the concrete and insulation wythes, and elastic beam elements (EB2) are used to model the FRP bent bar connectors and wythe steel reinforcement. Steel

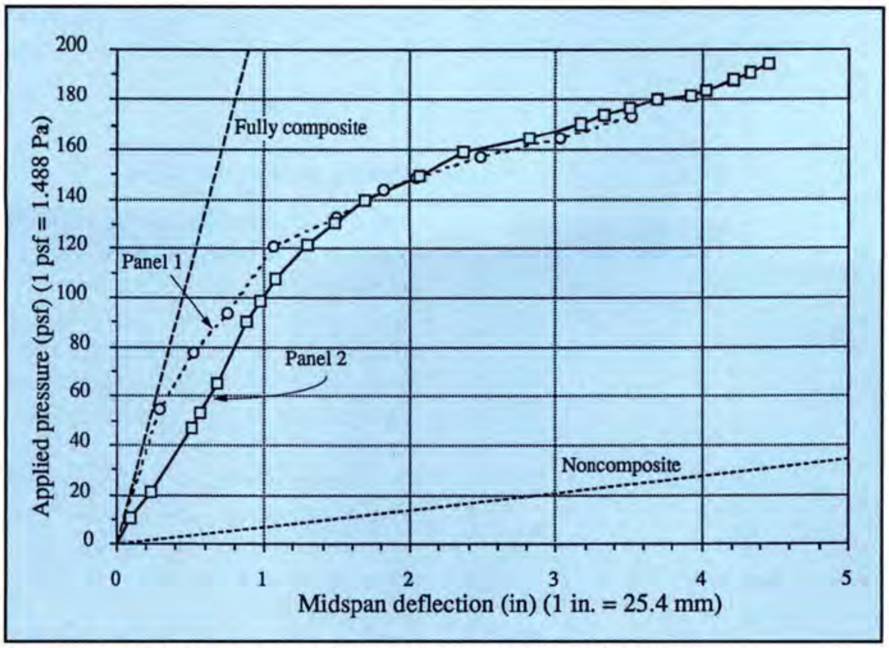

Fig. 11. Applied pressure vs. midspan displacement relationship.

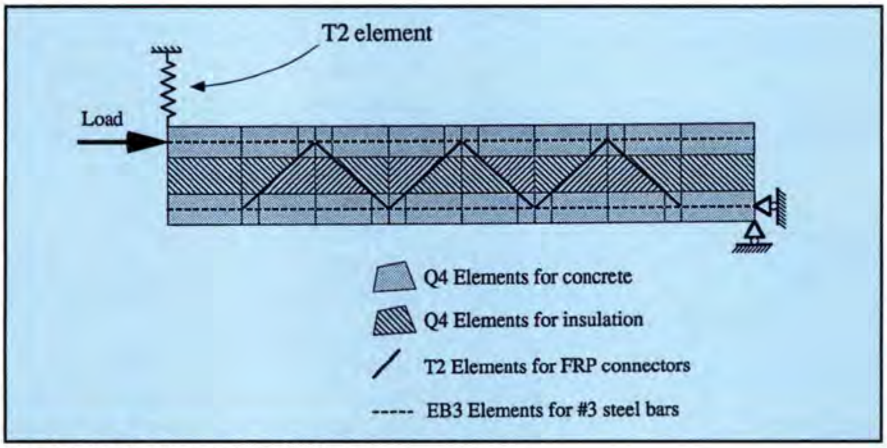

Fig. 12. Finite element model for shear specimens.

reinforcement is modeled with plastic beam elements (PB2) whenever yielding is possible. Truss elements (T2) model some boundary conditions and a combination of interface (I2) and control (C2) elements simulate concrete cracking.

Linear Material Model

Because no cracking was observed during shear testing, the material behavior of these specimens is taken to be linear. A plane stress model is used because the variation in stress over the panel width is negligible (see Fig. 12). The geometry of Specimen 4 is used as representative of all shear specimens.

Two layers of Q4 elements are used to model each concrete wythe. Q4 elements with a negligible shear modulus are also used to model the insulation layer and eliminate its contribution to the shear resistance of the panel. The FRP bent bar connectors and the #3 (10 mm diameter) steel bars that form the connector's chords consist of EB2 elements. Properties of concrete, FRP, and steel correspond to laboratory test results of representative samples. The modulus of elasticity of the insulation wythe is taken as 2 x 10^5 psi (1379 MPa).[8]

Boundary conditions of the model simulate the actual boundary conditions and the observed behavior during the test. A single T2 element models the vertical support at one end of the test frame.

The relationship of the FRPBB connectors to concrete wythes and #3 (10 mm diameter) steel reinforcing bars (chords) deserves special attention. Negligible bond exists between the concrete and FRPBB connectors. Therefore, the concrete provides lateral support for the embedded portions of the FRPBB. Axially stressed connectors, however, slip relative to the concrete in these regions. The finite element model simulates this behavior. The FRP diagonals are modeled as explained in the nonlinear model below.

The model was loaded in increments to simulate the tests and to allow detection of failure. The results are summarized as follows:

1. The load-displacement curve shows that the finite element model is significantly stiffer than the actual tested specimen (see Fig. 13). The reason for this difference is that the diagonals are assumed to be pinned at mid-thickness of the concrete wythes and the #3 (10 mm diameter) steel chord. In the test specimens, however, the FRP bars loop around the steel chords and thus slip when the specimen is loaded.

2. Maximum tension and compression stresses in the FRPBB at a load equal to the experimental ultimate load are 31 and 34 ksi (213 and 234 MPa), respectively. These stresses consisted of a combination of axial and flexural stresses and correlate closely with the experimental results of Specimen 4, as mentioned above. This analytical model shows sufficient accuracy in predicting the stress distribution among the connectors' diagonals, but is stiffer than the specimen because it does not simulate the slippage of the FRP bar inside the concrete wythes.

3. Flexural stresses in the FRP bars contribute up to 15 percent of total stress.

4. Buckling of the compression legs of the connectors could account for the difference between the ultimate strength of the connectors within the panels and that measured in separate laboratory tests of the connectors. The critical buckling stress of the connectors is between 15 and 60 ksi (103 and 414 MPa), depending on the boundary conditions assumed for the connector. Further research is necessary to estab-

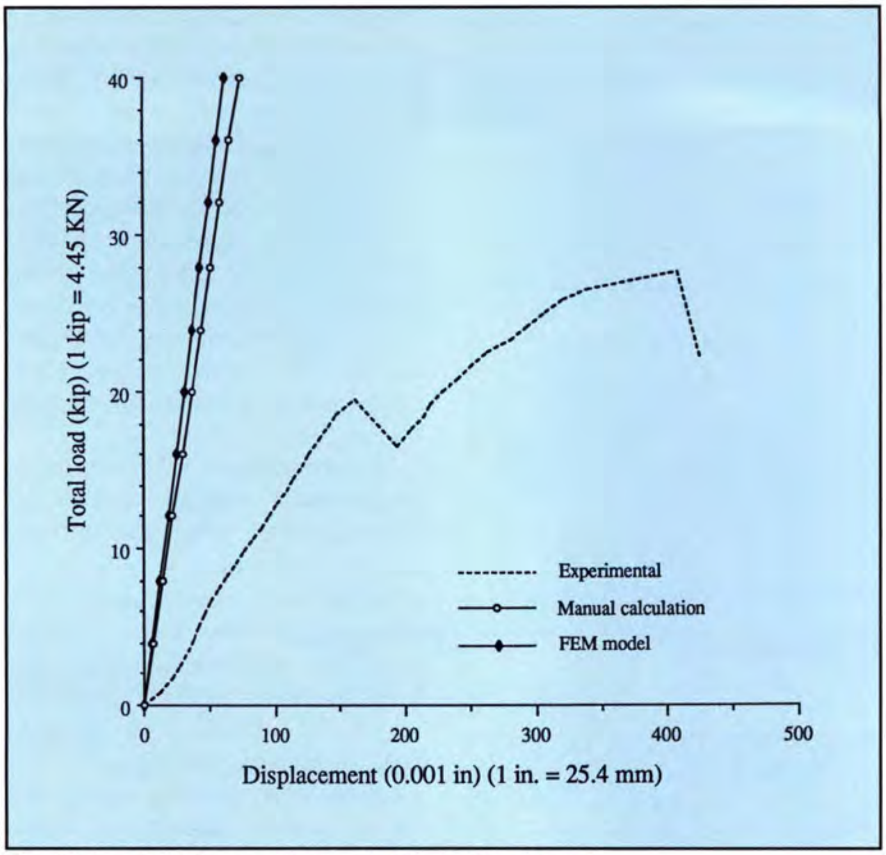

Fig. 13. Load-displacement curve for Shear Specimen 4.

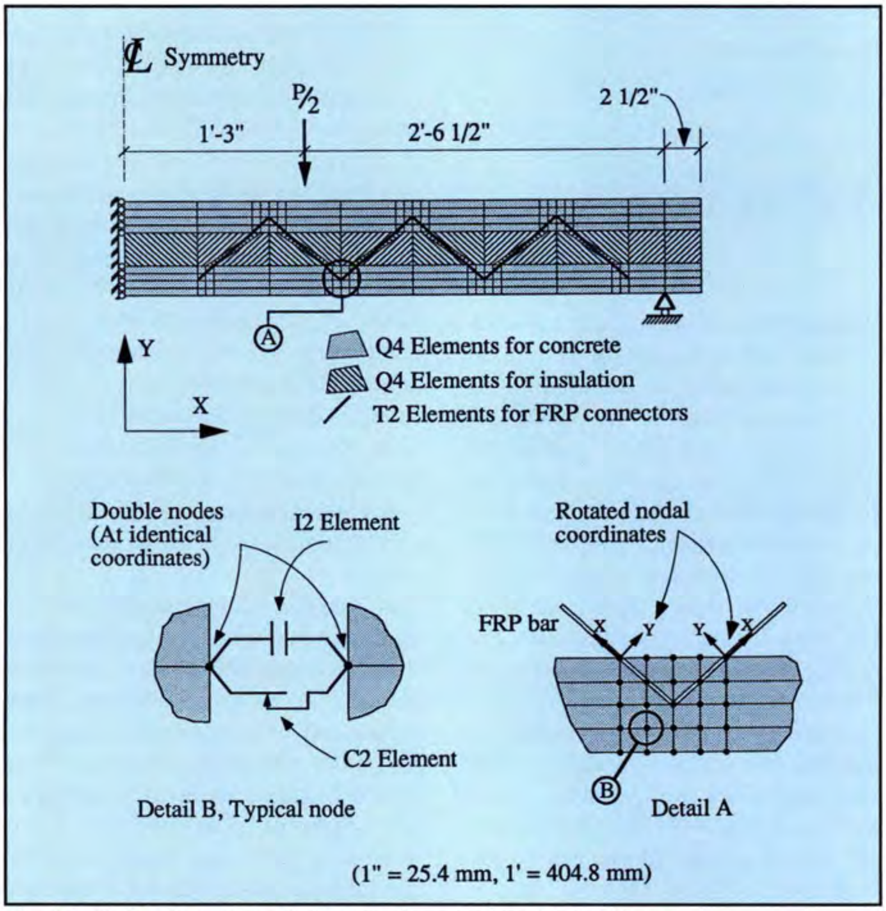

Fig. 14. Finite element model for small-scale flexural specimens.

lish the true behavior of the compression legs of the connectors within the panels.

Nonlinear Material Model

Four layers of Q4 elements are used to model each concrete wythe to provide higher accuracy (see Fig. 14). The modeling of the FRP bars and the insulation wythe is similar to that used in the linear model. The welded wire fabric and the #3 (10 mm diameter) reinforcing bars in each wythe consist of PB2 elements. Material properties of elements are derived from tests of representative samples and boundary conditions for the model simulate those used in the experiment.

To model concrete cracking, double nodes are defined at each nodal location. One node connects to the Q4 elements on its left and the other to the Q4 elements on its right. One I2 element and one C2 element simultaneously connect the two nodes (see Fig. 14, Detail B). The Y degrees of freedom at each double node are coupled to maintain continuity and stability in the Y direction.

To model this mechanism, the FRP elements are directly connected, along with the steel chord elements, to the nodes at the center of wythes. At the insulation-to-concrete interface, however, two duplicate nodes with identical coordinates are defined. One of the nodes is connected to the Q4 elements that model the concrete and insulation wythes while the other is connected to the EB2 elements that model the FRP bars.

Nodal coordinates of the doubled nodes are rotated such that the X coordinates are parallel to the axis of the FRP bar and the Y coordinates are perpendicular to it (see Fig. 14, Detail A). The Y direction degrees of freedom are coupled. This arrangement provides support to the FRP elements perpendicular to their axis throughout the wythe, but axial support only at their connection to the chord members.

Large displacement analysis is used to model the nonlinear behavior. The load is applied in increasing increments to monitor cracking and to detect buckling of the FRPBB, if it occurs. The results of the analysis are:

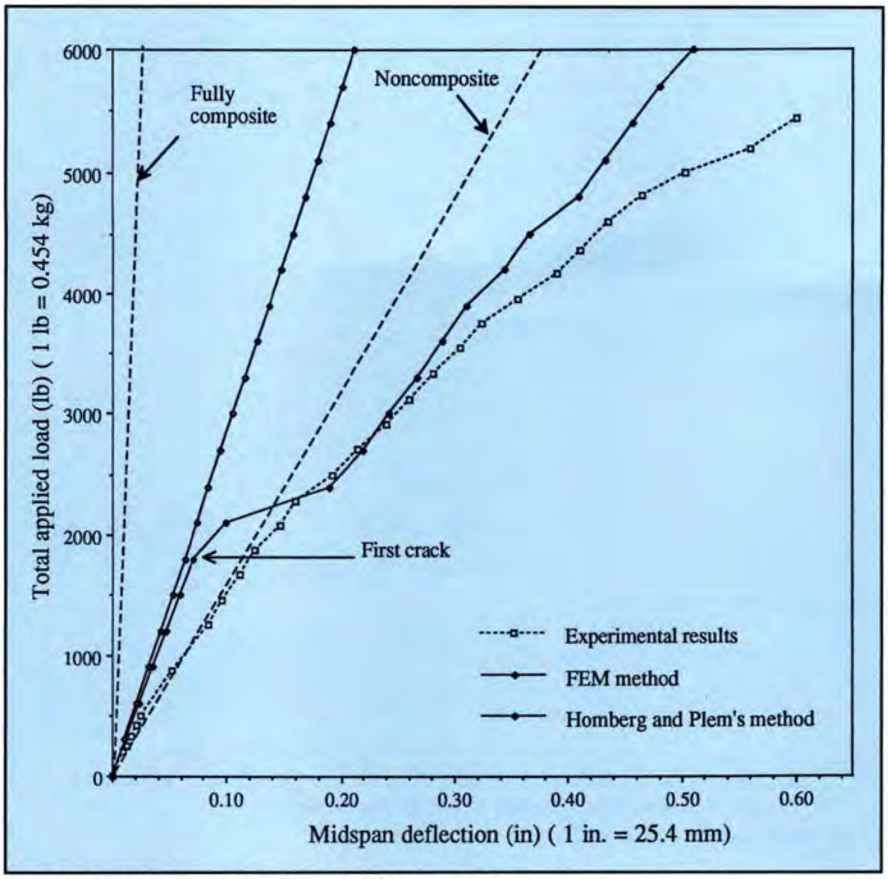

Fig. 15. Load-deflection curves for Specimen 1.

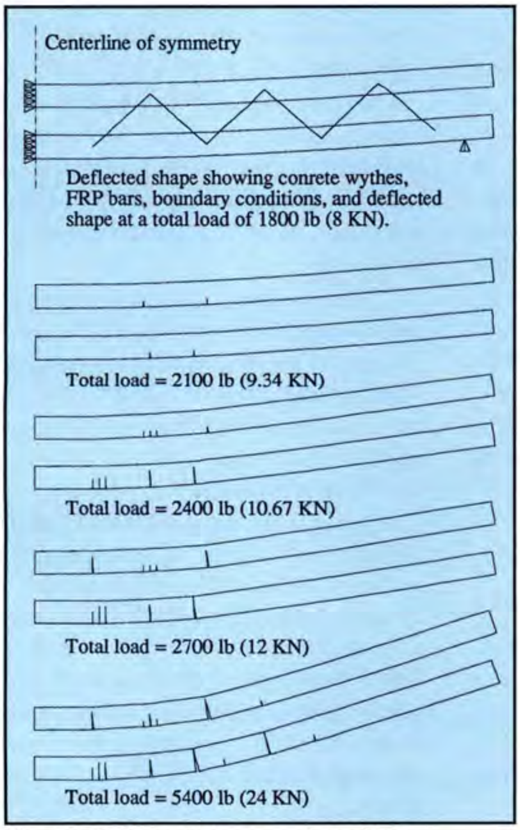

Fig. 16. Deflected shape and cracking development in Specimen 1. (Deflection values are 20 times the actual value and element divisions are removed for clarity.)

1. The load-displacement curve obtained from this model is very similar to the experimentally obtained curve (see Fig. 15). The curve is initially linear until the tensile stress in the lowest row of C2 elements exceeds the tensile strength assigned to these elements at which point their stiffness falls to zero, simulating a crack. This causes a sudden loss of stiffness reflected by the softening in the load-deflection curve. Further loading causes the next row of C2 elements to "crack." As the C2 elements simulate cracking, the PB2 elements, representing the steel reinforcement, take on the tensile stress. This process continues until the bottom wythe cracks completely. At this stage, the slope of the load-deflection curve stabilizes to the reduced model stiffness. The high early stiffness, compared with that observed during the experiment, is due to the slip of the FRP bar, inside the concrete wythes around the #3 (10 mm diameter) steel bars, that was not included in this analytical model.

2. The analytical crack pattern and its progressive development (see Fig. 16) correlate well with the observed experimental crack pattern (see Fig. 9).

3. Maximum tension and compression stresses in the FRPBB connectors at a load equal to the experimental ultimate load are 37 and 40 ksi (255 and 276 MPa), respectively. These stresses are a combination of axial and flexural stresses, up to 16 percent of which are flexural stress.

Holmberg and Plem's Method

The equations presented by Holmberg and Plem,[9] a brief background of which is presented by Salmon and Einea in Ref. 7, are useful in studying the behavior of the flexural specimens. Although the equations are based on linear behavior, they provide additional verification of the finite element model described above. The equations assume a two-dimensional stress state, a continuous truss connector between the concrete wythes, and connector anchorage at mid-thickness of the concrete wythes.

Fig. 16 shows a comparison between the load-deflection relationship obtained using this method and that obtained from FEM. The correlation between the methods is good up to concrete cracking stress. If cracked wythe properties are used in these equations, the resulting midspan deflection is 0.63 in. (16 mm) at ultimate load, which is in close agreement with the experimental midspan deflection.

Fig. 17 compares the maximum tensile stress distribution in the bottom concrete wythe calculated using Holmberg and Plem's equations and that obtained from the nonlinear FEM, at the first crack load during the experiment — 2000 lbs (8.9 kN). Both methods result in a maximum tensile stress close to the modulus of rupture of the concrete, f_r = 590 psi (4.1 MPa). In addition, the maximum concrete tensile stress occurs directly below the concentrated load, which explains the initiation of cracking at that location.

Holmberg and Plem's method clearly demonstrates this because it neglects the stiffness of the insulation, whereas the FEM model includes the transfer of forces perpendicular to the axis of the specimen by the insulation wythe. In reality, the insulation wythe provides a continuous cushion that helps transfer stresses perpendicular to the panel axis, thereby reducing the stress concentration.

It is clear from the previous comparisons between the analytical and the experimental results that both analytical methods, finite element and Holmberg and Plem's, agree well with each other, and both show an increase in stiffness over that of the shear and small flexural test specimens. As mentioned earlier, this is likely due to the slippage of the FRP bar at its anchorage, which is not modeled in either analytical method. If this behavior is incorporated into the analytical models accurately, improved agreement between the analytical and experimental initial stiffnesses is expected.

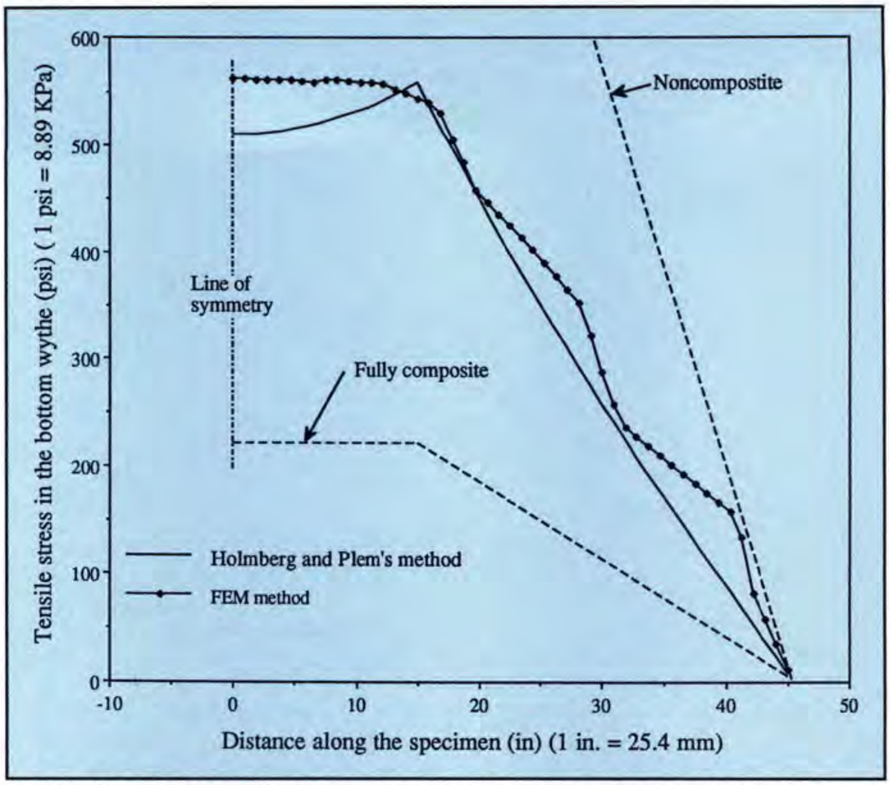

Fig. 17. Variation of maximum tensile stress in the bottom wythe of Specimen 1, at a total load of 2000 lbs (8.9 kN).

CONCLUSIONS

The testing and analytical work conducted in this research project elicit the following conclusions about the suitability and structural performance of FRPBB connectors for PCSPs:

1. The proposed FRPBB bar connectors fulfill the objective of this research. They have significant thermal resistance, and their structural performance in PCSPs is satisfactory. Their structural strength is large enough to develop a high percentage of composite action when constructed to do so. The full scale specimens resisted three times the design load.

2. Although FRP is a linear material with no yield point and fails at ultimate strength with no prior warning, very ductile behavior was observed in all tests performed. This ductile behavior is likely caused by cracking in the connections between the bent bar connector and the concrete that leads to a gradual loss of composite action and hence larger deflections.

3. The initial linear stiffness of the proposed sandwich panel system is less than predicted by analysis due to slippage of the FRP bent bars at their anchorages. If no further modification is done to the connecting system, this stiffness should be considered in design for serviceability calculations for the panels.

4. In general, some form of mechanical anchorage should be provided when FRP elements need to be anchored into concrete. In the proposed system, mechanical anchorage is provided by threading some of the longitudinal panel reinforcement through the FRP connectors. This anchorage probably provides some, if not all, of the observed ductility of the system.

SUGGESTED FUTURE RESEARCH

The following topics concerning FRPBB bar connectors and PCSP systems are in need of further research:

1. The effect of long term loading on the proposed system.

2. Cyclic load testing to investigate the ductility and energy dissipation characteristics of the panels for use in high seismic risk areas.

3. Development of lifting and connection inserts to maintain the thermal and structural efficiency of panels. Research is required to develop, test, and obtain design parameters for such accessories.

4. Determination of the fire rating of the proposed panel system. FRP material loses a large portion of its strength when exposed to fire or a high temperature environment. Investigation of concrete cover or other means to prolong the fire rating of the system is needed.

5. Determination if lateral support provided by insulation and concrete wythes is sufficient to prevent instability of the connectors when small bars are used.

6. Experiments to determine the nature of load-slip behavior of the connectors inside the wythes to more accurately predict the stiffness of the panels.

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REFERENCES

1. *PCI Design Handbook — Precast and Prestressed Concrete,* Third Edition, Precast/Prestressed Concrete Institute, Chicago, IL, 1985, Chapter 9, pp. 2-22, 55-61.
2. Al-Einea, A., Salmon, D. C., Tadros, M. K., Culp T., and Fogarasi, G. J., "State-of-the-Art of Precast Concrete Sandwich Panels," PCI JOURNAL, V. 36, No. 6, November-December 1991, pp. 78-98.
3. Wade, T. G., Porter, M. L., and Jacobs, D. R., "Glass-Fiber Composite Connectors for Insulated Concrete Sandwich Walls," Report, Engineering Research Institute, Iowa State University, Ames, IA, 1988.
4. Jokela, J., and Sarja, A., "Development of Reinforcement of Sandwich Facade Element," Research Note No. 19, VTT-Technical Research Center of Finland, Espoo, Finland, 1981.
5. Einea, A., "Structural and Thermal Efficiency of Precast Concrete Sandwich Panel Systems," Ph.D. Dissertation, Department of Civil Engineering, University of Nebraska-Lincoln, Omaha, NE, 1992.
6. Culp, Todd, M.S. Thesis, University of Nebraska-Lincoln, Omaha, NE, December 1994.
7. Salmon, D. C., and Einea, A., "Partially Composite Sandwich Panel Deflections," Approved for publication in the ASCE *Journal of Structural Engineering.*
8. Strzepek, W. R., "Overview of Physical Properties of Cellular Thermal Insulations," *Insulation Materials, Testing, and Applications,* ASTM STP 1030, American Society for Testing and Materials, Philadelphia, PA, 1990, pp. 121-140.
9. Holmberg, A., and Plem, E., "Behavior of Load Bearing Sandwich-Type Structures," Handout No. 49, State Institute for Construction Research, Lund, Sweden, 1965.