

Structural Behavior of Insulated Prestressed Concrete Sandwich Panels Reinforced with FRP Grid

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ABSTRACT: This paper presents an experimental program conducted at North Carolina State University to investigate the behavior of concrete sandwich wall panels reinforced with carbon-fiber shear connection grid. The study included testing of full-size precast sandwich panels consisting of two wythes of prestressed concrete and an inner layer of rigid foam. Carbon fiber reinforced polymer (CFRP) grid, commercially known as C-GRID, was used at selected locations to achieve composite action between the two concrete wythes. Panels were tested vertically in a frame that allowed simultaneous application of gravity and reverse cyclic lateral loads. Strain measurements taken through the thickness of each panel were used to evaluate the level of composite action achieved. A total of four specimens were tested to evaluate parameters including the wythe and core thicknesses, type of foam core, amount and configuration of CFRP shear reinforcement, and inclusion of solid concrete zones.

1 INTRODUCTION

Precast insulated sandwich wall panels, commonly known as concrete sandwich panels, are typically used for the construction of building envelopes. Such panels consist of two outer layers of concrete separated by an inner layer of rigid foam insulation. Panels can serve to carry gravity loads from floors or roofs, to resist normal or transverse lateral loads caused by wind, to insulate a structure, and to provide the interior and exterior finished wall surfaces.

Insulated concrete sandwich panels may be designed as: non-composite, partially composite or fully composite. The degree of composite action depends on the nature of the connection between the concrete wythes. Connections between wythes have traditionally been made using solid zones of concrete, bent reinforcing bars, or various specially-designed steel shear connectors. Increasing the degree of composite action between the two concrete wythes using any of these types of shear connections increases the structural capacity of the panel, making it more structurally efficient. However, traditional composite shear connections have the negative consequence of thermally bridging the two concrete wythes, thus decreasing insulating efficiency.

In order to achieve the greater structural efficiency provided by a composite panel while avoiding the thermal bridges created by traditional means of shear transfer, a group of precast producers have recently begun utilizing a carbon-fiber shear connection grid. Since carbon fiber has a relatively low thermal conductivity along with beneficial strength and stiffness characteristics, connecting concrete wythes with CFRP allows a panel to develop composite structural action without developing thermal bridges, thus maintaining the insulating value.

2 BACKGROUND

Precast concrete insulated sandwich panels continue to gain in popularity as the demand for energy efficient structures increases. A fully composite panel is designed to allow utilization of the

two concrete wythes acting together as a single unit to resist the applied loads. Composite behavior is evidenced by a single neutral axis for the through-thickness strain profile at any cross-section along the height and width of the panel. Fully non-composite behavior occurs when each wythe acts independently to resist the applied loads. Thus, a fully non-composite panel will have an independent strain profile for each of the concrete wythes. The term “partially composite” has been introduced by several researchers including Pessiki & Mylnarczyk (2003), Lee & Pessiki (2007) and Bush & Stine (1994). Defining and designing for a partial degree of composite action can significantly increase the structural efficiency and reduce both initial and life-cycle costs of a panel, compared to a fully non-composite case.

In order to achieve composite behavior, several horizontal shear transfer mechanisms have been developed. Some of the commonly used mechanisms are wire truss connectors, bent wire connectors, and solid zones of concrete penetrating the foam core, as detailed in Figure 1 a, b, and c, respectively. Wire truss connectors provide diagonal wires to transmit longitudinal shear forces from one wythe to another while bent wire connectors rely on ties perpendicular to the panel faces to transfer forces. Connectors with diagonally oriented members tend to be more efficient than bent wire connectors in transferring longitudinal shear; however, solid zones of concrete (Figure 1c) often provide the simplest and most structurally efficient longitudinal shear transfer mechanism. The major drawback to using these traditional methods is that all provide paths for heat to bridge the foam core which reduces the thermal efficiency of a panel.

Recently, a more efficient shear transfer mechanism has been developed using Carbon Fiber Reinforced Polymer (CFRP) grid. The CFRP grid used in this study, commercially known as C-GRID, consists of carbon fiber polymer strips approximately 6 mm ($\frac{1}{4}$ in) wide by 1.6 mm ($\frac{1}{16}$ in) thick arranged in an orthogonal pattern. The grid is oriented diagonally between the concrete wythes, normal to the wall surface, allowing for a truss mechanism to develop. The CFRP grid referenced in this paper is shown alone in Figure 1d, and shown again in Figure 1e as placed in a cut wall panel section.

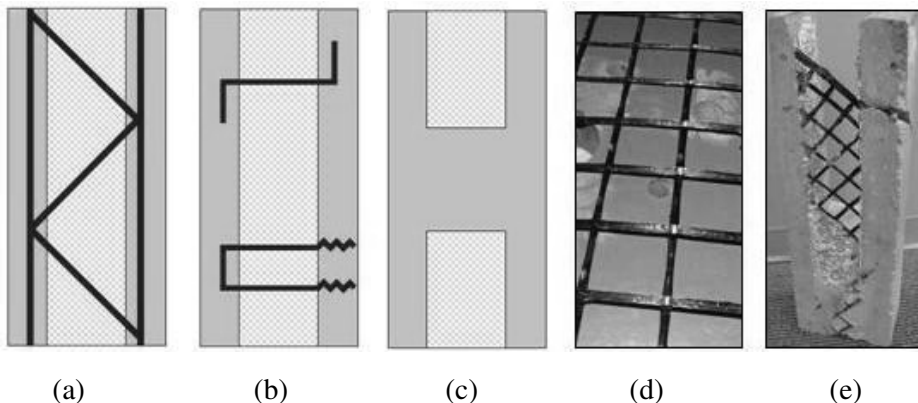


Figure 1. (a) Wire truss connector (b) Bent wire connectors (c) Solid concrete zone (d) CFRP grid material sample (e) CFRP grid shear transfer mechanism in section cut from test panel (foam removed)

3 EXPERIMENTAL PROGRAM

The experimental program conducted at the Constructed Facilities Laboratory at North Carolina State University consisted of four panels, each measuring 6.1 m (20 ft) tall by 3.7 m (12 ft) wide. All panels were 203 mm (8”) thick and were comprised of three layers through their thickness. The first three panels consisted of a 51 mm (2 in) layer of concrete followed by a 102 mm (4 in) layer of foam and a second 51 mm (2 in) layer of concrete. This arrangement is designated as a 2-4-2 panel configuration. The inner wythe for the 2-4-2 panels included two internal pilasters 51 mm (2 in) thick by 610 mm (24 in) wide along the full height of each panel at the quarter and three-quarter widths. The pilasters are provided to carry axial loads from two corbels located at the top of the inner panel face. The fourth panel consisted of a 102 mm (4 in) thick inner concrete wythe followed by a 51 mm (2 in) thick foam core and an outer 51 mm (2

in) thick concrete wythe. This configuration was designated as 4-2-2 with two corbels located at the top of the 102 mm (4 in) wythe. The 4-2-2 panel carried axial load through its thicker inner wythe, and did not use internal pilasters. It is important to note that all axial loads were eccentric, applied on the corbel projections 152 mm (6 in) from the surface of the inner wythe

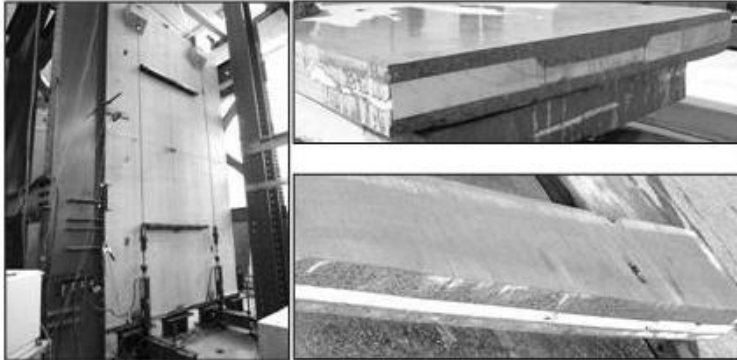


Figure 2. Panel during testing (left); Section cut from 2-4-2 panel showing internal pilaster (top right); Section cut from 4-2-2 panel (bottom right)

Each concrete wythe was reinforced with a sheet of welded wire reinforcement in the plane of the wythe and prestressed in the longitudinal direction by five 9.5 mm ($\frac{3}{8}$ in) diameter 1860 MPa (270 ksi) low-relaxation prestressing strands. Shear transfer between the wythes was provided by two continuous C-GRID strips running along the height of the panel at roughly the quarter-width points. An additional 4 strips of C-GRID were provided for the first 1.5 m (5 ft) at the upper and lower ends of the panel. Two more 1.5 m (5 ft) strips were provided at the panel mid-height. In addition to the C-GRID, panel 3 contained discretely located solid concrete zones throughout the height and width of the panel. Panels 1 and 2 were fabricated using an expanded polystyrene foam (EPS) while panels 3 and 4 were fabricated using an extruded polystyrene foam (XPS). Table 1 summarizes the configurations of the tested panels.

Table 1. Summary of experimental tests and results

Panel	Foam	Configuration	Solid Zones	Service Load Deflection D+L _r +W	Failure Load
1	EPS	2-4-2	No	h/460	1.2D+0.5L _r +2.8W ₁₂₀
2	EPS	2-4-2	No	h/500	1.2D+0.5L _r +1.8W ₁₅₀
3	XPS	2-4-2	Yes	h/1480	1.2D+0.5L _r +1.6W ₁₂₀
4	XPS	4-2-2	No	h/755	1.2D+0.5L _r +3.2W ₁₂₀

3.1 Test Setup

All panels were tested in the laboratory using a steel testing frame that allowed simultaneous application of gravity and lateral loads. Reverse cyclic lateral loads were applied to simulate the effects of wind pressure. The testing frame consisted of one braced frame on each side of the panel to support an upper cross-beam. This cross-beam in turn provided the upper lateral panel reaction. The entire setup was anchored to the laboratory strong floor. A closed-loop MTS hydraulic actuator, supported by a strong reaction wall, was used to apply lateral load.

Each panel was simply supported in the testing frames at the top and bottom edge. The bottom of the panel was supported by a hinge which restrained horizontal and vertical movements while allowing rotation. The top of the panel was supported by a specially designed connection that restrained horizontal panel motion while allowing for vertical movement and rotation. Vertical loads were applied to the top of each corbel by a hydraulic jack and cable, as shown in Figure 2. These vertical loads were provided to simulate the effects of a double-tee roof system. Lateral loads were applied by the actuator which in turn was connected to a spreader beam system, allowing loading on the push and pull strokes to simulate wind pressure and suction. Two

loading tubes were provided at each quarter-height of each panel, one on each wythe, to distribute the lateral load across the surface of the panel. The lateral loading mechanism included a vertical spreader beam that could shorten and elongate as the panel deformed laterally to prevent the transfer of any unintended forces to the panel. Each panel was run through several thousand reverse cyclic high-level cycles (around 80% of service load) that were rationally selected by the use of a Wiebull distribution.

All panels were instrumented to measure lateral deflection, relative displacement between the two concrete wythes, surface strain of the concrete and the applied axial and lateral loads. The strain profile across the thickness of each panel was measured using four electrical-resistance strain gauges across the panel section at three locations along the height.

Each panel was subjected to 3710 fully-reversed lateral load cycles at 45% of the factored lateral wind load, equivalent to $1.6W$, with a factored axial load of $1.2D+0.5L_r$. The initial cycles were followed by 177 cycles at 50% of the factored lateral wind load with the factored axial load in place. Subsequent individual cycles were applied at 60%, 80%, and 100% of the factored lateral wind load, all with axial load in place. After the factored lateral load cycle was completed, incremental static load cycles were continued in one direction only until failure.

3.2 Test Results

The panel behavior observed from experimental data indicates that the stiffness of a panel is largely influenced by the magnitude and type of the shear transfer mechanism. All three panels having C-Grid as the sole shear transfer mechanism exhibited the same panel stiffness, regardless of configuration or foam (2-4-2, 4-2-2, EPS, or XPS). However, panels having solid zones of concrete in addition to C-Grid showed increased panel stiffness. Test results indicate that the cracking load for all of the tested panels was higher than the design ultimate load. All panels remained substantially uncracked up to failure.

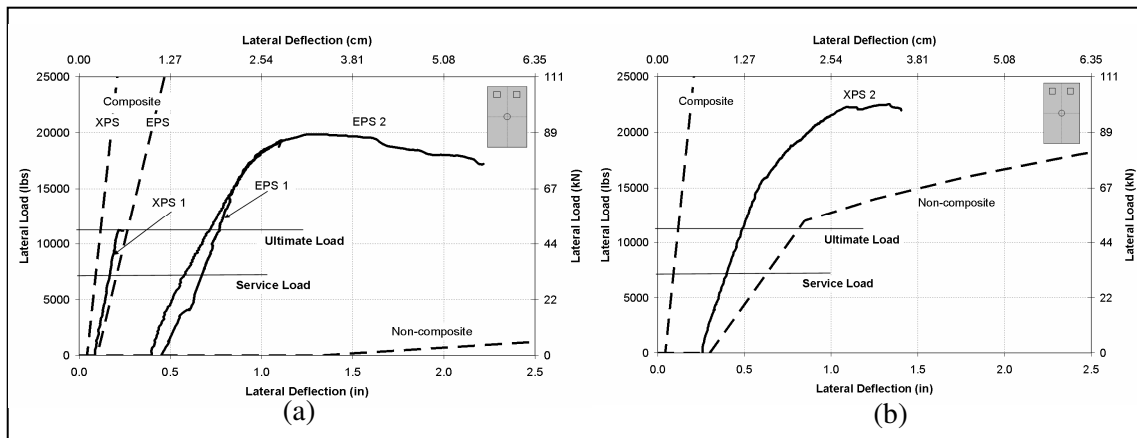


Figure 3. Load deflection of experimental data (a) 2-4-2 panels (b) 4-2-2 panels

The measured lateral deflection due to the applied axial load only was found to be dependant on the configurations and type of shear transfer mechanism used for each panel, as represented in Figure 3 by the offset in deflection at zero lateral load. This offset is due to the eccentric axial load applied prior to initiating lateral loading. The 2-4-2 XPS panel with solid concrete zones experienced an initial deflection close to a theoretical fully composite behavior, while the panels with C-GRID only experienced higher initial deflections prior to application of lateral load.

Typical panel strain profiles for theoretical fully composite and fully non-composite behaviors are shown in Figure 4a and b, respectively. The experimental strain profile for a 2-4-2 EPS panel, shown in Figure 4c, closely matches the fully composite profile, while panels with XPS foam exhibited a strain profile closer to the fully non-composite case, as shown in Figure 4d.

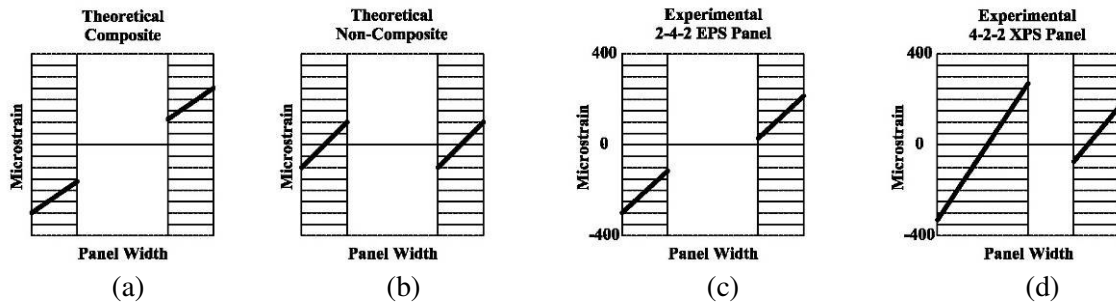


Figure 4. (a) Theoretical fully composite; (b) Theoretical fully non-composite; (c) Experimental 2-4-2 EPS panel; and (d) Experimental 4-2-2 XPS panel

3.3 Failure Modes

The observed failure modes for the first 2-4-2 EPS panel and both XPS panels were localized around the corbels. Failure was characterized by prying shear failure around the corbels in a radial pattern surrounding the supporting weld plate directly above each corbel. The 2-4-2 XPS panel with solid zones exhibited an abrupt failure due to localized failure of the corbels as shown in Figure 5a. The second 2-4-2 EPS panel exhibited a flexural-shear failure across the width of the panel at approximately 7/8 panel height as shown in Figure 5b. All panels exhibited deflections well below the limiting value of $h/360$ specified by ACI 533R (2004). Measured failure loads for all tested panels exceeded their factored design loads. It is important to note that uniform design pressures for panels 1,3, and 4 were assumed to be 1.4 kPa (29 psf), corresponding to a design wind speed of 54 m/s (120 mph). Panel 2 was tested to a design pressure of 2.1 kPa (44 psf), corresponding to a design wind speed of 67 m/s (150 mph). Thus, the total lateral force exerted on panel 2 at any given cycle was higher than the force exerted on the other panels at that same cycle. Table 1 summarizes failure loads and deflections for all tested panels.

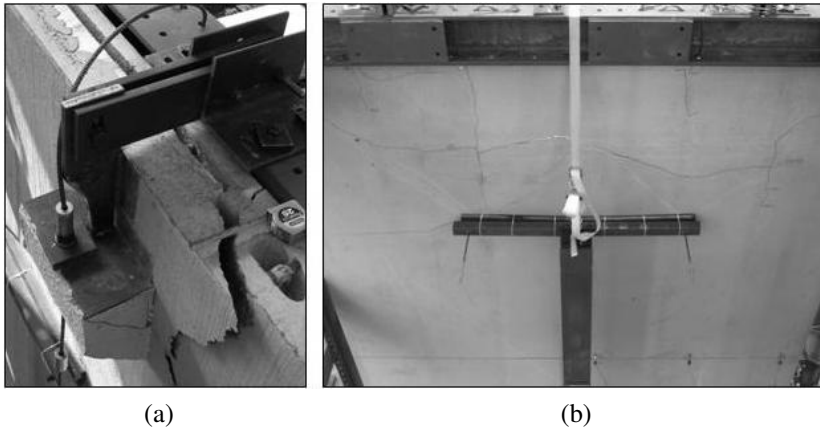


Figure 5. (a) Failure of 2-4-2 XPS panel with solid zones; (b) Failure of 2-4-2 EPS panel

4 ANALYSIS

To examine the composite action, Pessiki & Mlynarczyk (2003) defined the degree of composite action based on the measured deflection to determine the experimental moment of inertia. The experimental moment of inertia was then compared to the theoretical fully composite and non-composite moments of inertia. All tested panels considered in the Pessiki study were subjected to pure bending with no applied axial load.

Applied axial load complicates the analysis with second-order effects. These effects change panel behavior and make accurate determination of the experimental moment of inertia impractical. To evaluate the degree of composite action for the panels in this research, a deflection relationship was used to determine the percentage of composite action, κ , as follows:

$$\kappa = \frac{\Delta_{\text{exp}} - \Delta_{\text{nc}}}{\Delta_c - \Delta_{\text{nc}}} \quad (1)$$

Where Δ_{exp} is the measured deflection at a given load within the service load level, Δ_c and Δ_{nc} are the corresponding deflection of the same panel for the same load level based on theoretical fully composite and fully non-composite behaviors. To determine theoretical panel deflections beyond cracking, the effective moment of inertia was calculated in accordance with ACI 318-05 equation 9-5 (ACI 318, 2005). For the theoretical fully composite case, applied loads and moments act on the full composite section. For the theoretical non-composite case, applied axial load is resisted by the inner corbelled wythe alone. Applied moments are resisted by the stiffness of the two wythes acting individually, as determined from their individual uncracked moments of inertia.

The method followed for determining the theoretical deflection at any load level is summarized in *State-of-the-Art of Precast/Prestressed Sandwich Wall Panels* (PCI, 1997), including the effect of axial load. With each panel being subjected to a series of fatigue cycles, percent composite action was determined for the first service load cycle using the experimental, fully composite and fully non-composite behavior shown in Figure 3. It is important to note that the EPS panels were cast by one precast producer and the XPS panels by another. Thus, differences in theoretical behavior between XPS panels and EPS panels can be attributed to variations in the modulus of elasticity of the concrete.

Evaluating each panel at service load resulted in a percent composite action ranging from 40% to nearly 100%. The estimated composite action for panels 1 and 2 (EPS foam) are 98% and 99%, respectively. Panel 3, with XPS foam and discretely located solid concrete zones exhibited approximately 98% composite action while panel 4 with XPS foam and a 4-2-2 configuration exhibit only 39% composite action.

5 CONCLUSIONS

Behavior of four full-scale insulated precast sandwich panels tested under fatigue loading and monotonic lateral loading up to failure indicated that panel stiffness and deflection is significantly affected by the shear transfer mechanism. Test results indicate that solid concrete zones provide higher percent composite action than does the use of C-GRID alone. However, appropriate use of C-GRID as the sole shear transfer mechanism in a precast sandwich panel can provide significant composite action while also allowing for a highly thermally-efficient structure. Observed behavior indicates that, for a given shear transfer mechanism, a higher percent of composite action can be achieved at service load by EPS foam compared to XPS foam. The research is ongoing, and aims to optimize the use of CFRP grid as a shear transfer mechanism in precast concrete sandwich wall panels.

6 REFERENCES

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