Synopsis: This paper describes the structural behavior of precast prestressed concrete sandwich wall panels reinforced with carbon fiber reinforced polymer (CFRP) shear grid to achieve composite action. The study included testing of six full-scale sandwich wall panels, each measuring 20 ft (6.1 m) by 12 ft (3.7 m). The panels consisted of two outer prestressed concrete wythes and an inner foam core. The study included two types of foams and several shear transfer mechanisms with different CFRP reinforcement ratios to examine the degree of composite action developed between the two concrete wythes. All wall panels were simultaneously subjected to applied gravity and lateral loads. The paper also presents a general methodology to determine the behavior of fully and partial composite wall panels. The effects of imperfect connection between the two concrete wythes are considered by varying the total shear force transmitted through the shear connectors at the interface. The shear flow capacity of the insulating materials as well as the CFRP shear grid is determined using the proposed approach. The influence of the degree of the composite interaction on the induced curvature and slip-strain behavior is presented. A simple design chart for estimating the flexural capacity of the wall panels with different shear reinforcement ratios is proposed.

Keywords: CFRP grid, composite, concrete, precast, prestressed, sandwich panels, wythes.
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INTRODUCTION

Precast concrete sandwich panels (PCSP) are typically used for building envelopes. Such panels consist of two outer layers of prestressed concrete separated by an inner foam core. The panels can serve to support 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. Typical panels are fabricated with heights up to 45 ft (13.7 m) and with widths up to 12 ft (3.7 m). Concrete wythe thickness ranges from 2 to 6 in. (50 to 150 mm) with overall panel thicknesses ranging from 5 to 12 in. (127 to 300 mm). Insulated concrete sandwich panels may be designed as non-composite, partially composite, or fully composite. Defining and designing for a partial degree of composite action can significantly increase the structural efficiency and reduce both initial and lifecycle costs of a panel, compared to the fully non-composite case. The degree of composite action depends on the nature of the connections between the two concrete wythes. Commonly used shear transfer mechanisms include wire truss connectors, bent wire connectors, and solid zones of concrete penetrating the foam core, as detailed in Figure 1. Increasing the degree of composite action between wythes increases the structural capacity of a given panel, making it more structurally efficient. However, traditional composite shear connections have the negative consequence of thermally bridging the two concrete wythes, thus decreasing the thermal efficiency.

Wall panels were first introduced during the 1960’s as double tee sandwich panels. Solid concrete zones were used between the double tees in order to develop the composite action. Double tees sandwich panels provided a robust structural wall, but sacrificed the potential thermal savings. Flat concrete slabs were soon used in place of double tees to reduce concrete material, to optimize the structural performance, and to reduce overall costs. More recently, steel ties connecting the two concrete wythes were used to replace solid concrete zones in an attempt to enhance the thermal performance of wall panels. It was found that the use of steel ties significantly improved the thermal efficiency in comparison to solid concrete zones; however, such ties also created thermal bridges. Non-composite panels were introduced in the 1980’s and aimed to address the thermal deficiencies created by the steel ties. Non-composite panels contained minimal shear connectors for handling loads only, but the lack of shear transfer compromised the structural integrity of the system. Despite the lower structural capacity, non-composite panels became popular due to their thermal savings and architectural characteristics. It should be noted that the typical design method for precast sandwich panels assumes non-composite action. Recent tests by Lee and Pessiki showed that a panel with staggered solid concrete zones exhibits behavior similar to that of a fully composite panel. It was also observed that transfer of the prestressing force induced cracks in the concrete wythes parallel to the prestressing strands. A finite element analysis was conducted to investigate the prestressing forces during release. Results of the analysis showed that modeling the concrete and the foam insulation with solid block elements provided close results to the measured values. The use of fiber reinforced polymer (FRP) bars formed in a truss orientation in place of metal wire trusses was introduced by Salmon et al. Test results showed that the use of FRP achieved a high level of composite action and provided thermal benefits similar to non-composite insulated sandwich wall panels. Following the same concept, carbon fiber reinforced polymer (CFRP) shear connection grid was introduced for use the construction of sandwich wall panels in 2003. Since carbon fibers have a thermal conductivity of approximately 14 percent that of steel, connecting concrete wythes with carbon grid allows a panel to develop composite structural action without developing thermal bridges; therefore, maintaining the insulating value of the panel. The grid was oriented diagonally between the concrete wythes, normal to the wall surface, allowing for a truss mechanism to develop.
This paper describes the behavior of six full-scale precast prestressed concrete sandwich panels. The panels comprised two prestressed outer concrete wythes, an internal layer of foam insulation, and shear grid reinforcement placed through the foam core into each concrete wythe. The various parameters considered in the current study included the type of foam, presence of solid concrete zones, panel configuration, and shear grid reinforcement ratio. The loading sequence for each panel was selected to simulate the effect of gravity and wind loads for a 50 year lifespan. Load and support conditions were designed to mimic field conditions. The paper also presents design guidelines for PCSP reinforced with CFRP shear grid. The analytical approach provides a general methodology to determine the behavior of fully and partial composite wall panels at any given curvature. The approach is calibrated with the test results. A sensitivity analysis was conducted using test results to estimate the shear flow capacity of the insulating materials as well as the CFRP connectors. The influence of the degree of composite interaction on the induced curvature and slip-strain behavior is also enumerated.

**EXPERIMENTAL PROGRAM**

Six precast prestressed insulated wall panels were designed and tested to evaluate their flexural response under combined vertical and lateral loads. The study included panels fabricated with two different foam types. The first foam type was expanded polystyrene foam (EPS), which consisted of a beaded material with a high void ratio. The second foam used was extruded polystyrene foam (XPS) which consisted of a dense hard foam material with a low void ratio. The panels were 20 ft (6.1 m) tall by 12 ft (3.7 m) wide, as shown in Figures 2a and 2b. All panels were 8 in. (203 mm) thick and consisted of three layers through their thickness. Table 1 summarizes the configurations of the tested panels.

Panels EPS1, EPS2, XPS1, XPS3 and XPS4 consisted of a 2 in. (51 mm) layer of concrete followed by a 4 in. (102 mm) layer of foam and a second 2 in. (51 mm) 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 2 in. (51 mm) thick by 24 in. (610 mm) wide along the full height of each panel at the quarter and three-quarter widths as shown in Figure 3. The two pilasters are provided to carry axial loads from the two corbels located at the top of the inner panel face. Panel XPS2 consisted of a 4 in. (102 mm) thick inner concrete wythe followed by a 2 in. (51 mm) thick foam core and an outer 2 in. (51 mm) thick concrete wythe. This configuration, as shown in Figure 4, is designated as 4-2-2 with two corbels located at the top of the 4 in. (102 mm) wythe. The 4-2-2 panel was designed to carry the axial load through its thicker inner wythe, and therefore, did not have internal pilasters. 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 270 ksi (1860 MPa) low-relaxation prestressing strands. The diameter of the prestressing strands in the 2 in. (51 mm) and 4 in. (102 mm) thick concrete wythes was 3/8 in. (9.5 mm) and 1/2 in. (12.7 mm), respectively. CFRP shear grid was placed between the two concrete wythes to transfer the shear stresses across the foam and to develop a composite action between the wythes. The carbon grid was provided in strips running parallel to the longitudinal axis of each panel at the locations shown in Figures 5a and 5b. All panels except XPS4 contained the same grid layout as shown in Figure 5a. Panel XPS4 contained an additional 30 ft (9.1 m) of shear grid as shown in Figure 5b. In addition to the CFRP grid, panel XPS1 contained ten discretely located solid concrete zones throughout the height and width of the panel.

**Test Setup**

All panels were tested in the laboratory using a steel testing frame that allowed for 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 frame in turn provided the upper lateral support to the panel. 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 the 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 for rotation. The top of the panel was supported using a specially designed connection that restrained horizontal 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 Figures 2a and 2b. These vertical loads were provided to simulate the effects of a double-tee roof system. Lateral loads were applied by the actuator connected to a spreader beam system, which was used to push and pull the panel 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 width of the panel. The lateral loading mechanism included a vertical spreader beam that could shorten and elongate as the panel deformed to prevent the transfer of any unintended forces to the panel. Each panel was subjected to reverse-cyclic loading beginning at a level equivalent to 70% of the service load. The loading regime was selected by a Weibull distribution to simulate wind loads over a 50-year service life.

All panels were instrumented to measure lateral deflection, relative displacement between the two concrete wythes, surface strain of the concrete, and 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 0.7$W$, with a factored axial load of $1.2D+0.5L_r$, in place, where $W$ is the wind load; $D$ and $L_r$ are the dead and roof live loads, respectively. The initial cycles were followed by 177 cycles at 50% of the factored lateral wind load (0.8$W$) with the factored axial load in place. Subsequent individual cycles were applied at 60%, 80%, and 100% of the factored lateral wind load (1.0$W$, 1.3$W$, 1.6$W$), all with axial load in place. After completion of all lateral cycle loads in the presence of gravity load, the lateral load was increased in one direction only until failure.

RESULTS AND DISCUSSION

Lateral Displacements

Figures 6a to 6f depict the measured lateral displacement at mid-height for the different panels. In general, the measured lateral deflections due to the applied axial load only were found to be dependent on the through thickness panel configuration (2-4-2 or 4-2-2), and also on the type and configuration of shear transfer mechanism used. Lateral deflection due to axial load alone is shown as the offset deflection at zero lateral load level. The allowable displacement of $h/360$ at service load level, as per the ACI 533R, is compared to the measured values for the tested panels. The two EPS foam-core panels, (EPS1 and EPS2), behaved almost identically throughout the loading cycles. The panels’ stiffness remained constant up to a lateral load of 15 kips (67 kN), beyond which concrete cracking occurred, causing a considerable reduction of the stiffness as shown in Figures 6a and 6b. Such a behavior was also observed for the other panels except XPS3, which failed at a lateral load level of 5 kips (22 kN). The maximum measured displacements at service load level for EPS1 and EPS2 were equivalent to $h/460$ and $h/500$, respectively. These displacements were well within the ACI 533R limit. Among the XPS foam-core panels, XPS1 experienced the least stiffness degradation with increased load cycles due the presence of solid concrete zones connecting the inner and outer wythes. The maximum lateral displacement at service load level for XPS1 was equivalent to $h/1480$. Panels XPS2 and XPS4 exhibited minimal lateral load degradation. The measured lateral displacements for both panels did not increase noticeably throughout the fatigue cycles. The maximum lateral displacement at service load level for XPS2 and XPS4 were equivalent to $h/755$ and $h/700$, respectively. Failure of panel XPS3 occurred before reaching the design service load level. Test results suggest that the accumulated degradation for XPS3 was substantial compared to other panels.

Strain Profiles

The strain profiles across the thickness of the panels were measured to determine the degree of composite action between the two concrete wythes. Typical results recorded during the testing of the panels at ultimate load level are shown in Figures 7a to 7f. For all panels, the inner wythe experienced compressive strains while the outer wythe experienced tensile strains under the effect of applied factored gravity loads. Test results showed that EPS foam-core panels (EPS1 and EPS2) as well as XPS1 with solid concrete zones exhibited and maintained a high level of composite action up to failure. The strain profile at ultimate indicates that the neutral axes of these panels are located closer to the elastic centroid of the composite cross-section rather than the elastic centroid of each individual wythe. The measured strains for XPS2 (4-2-2 configuration), indicated that each wythe acted independently in carrying the applied loads with the neutral axis located within the thickness of each wythe. This behavior indicates a non-composite action. Panels XPS3 and XPS4 showed a clear reduction in composite action with increasing lateral load. A significant discontinuity in the strain at ultimate was observed for these panels indicating a partial composite behavior at ultimate.

Failure Modes

The observed failure modes for EPS1, XPS1, and XPS2 were localized at the top of the panel in the corbel zones. Failure was characterized by a shear failure around the corbels extending down by approximately 2 ft (610 mm) as
shown in Figure 8a. This failure was accompanied by separation of the top of the panel. Panel EPS2 exhibited a flexural-shear failure across the width of the panel at approximately 7/8 the panel’s height as shown in Figure 8b. Panels XPS3 and XPS4 exhibited a flexural-shear failure across the width of the panel at approximately 7/8 the panel’s height as shown in Figure 8c, along with a simultaneous top of panel separation as shown in Figure 8d. All panels with sufficient shear transfer mechanisms exhibited deflections well below the limiting value specified by ACI 533R, and sustained loads prior to failure in excess of their factored design loads, as given in Table 1. However, panel XPS3, with reduced shear reinforcement, failed prematurely prior to the service load with high deflections. It is important to note that uniform design pressures for panels EPS1, XPS1,2,3, and 4 were assumed to be 29 psf (1.4 kPa), corresponding to a design wind speed of 120 mph (54 m/s). EPS2 was tested to a design pressure of 44 psf (2.1 kPa), corresponding to a design wind speed of 150 mph (67 m/s). Thus, the total lateral fatigue loading on EPS2 was higher than the load used for the other panels.

**PARTIAL INTERACTION THEORY FOR PCSP**

The assessment approach developed in this paper is based on the partial interaction theory that was originally developed by Newmark et al.\textsuperscript{10} for composite steel beams with incomplete interaction. The approach has been modified to account for the non-linear behavior of the concrete and the wall panel configuration. The approach is primarily based on iteration procedures that can be easily programmed, as will be demonstrated in the following sections.

**Theory and Assumptions**

PCSP are typically subjected to axial gravity loads acting on corbels extending from the inner wythe in addition to lateral wind or seismic loads. For any given bending moment, $M_u$, and axial force, $P_u$, the corresponding strains at the inner and outer wythes can be estimated assuming a fully composite interaction using strain compatibility and equilibrium of the wall panel section as shown in Figure 9a. Typically, a situation of full-interaction arises when there is no slip at the interface; therefore, the entire section has a continuous strain distribution with only one neutral axis at the centroid of the composite section. Conversely, the non-composite interaction occurs when there is no shear connection and the two concrete wythes act independently. In this case, there are two neutral axes at the centroid of the inner and outer wythes as shown in Figure 9b. The presence of the shear forces at the interface of the wythes and the insulating materials provides the mechanism for partial interaction.

Under the action of the applied loads for the simply supported PCSP shown in Figure 10a, the outer fibers tend to lengthen, whereas, the inner fibers tend to shorten. The shear connectors, which comprise CFRP shear grid and insulating foam, counteract these tendencies by exerting forces that produce compression in the inner wythe and tension in the outer wythe as shown in Figure 10b. These forces typically act at the interface and can be replaced by a couple and a force acting at the centroid of the inner and outer wythe as shown in Figure 10b.

The analysis presented in this paper is based on the following assumptions:

- The shear connectors between the concrete wythes are assumed to be continuous along the length of the wall panel;
- The distribution of strains along the depth of the inner and outer wythes is linear;
- Both inner and outer wythes are assumed to displace equal amounts at all points along their length. Therefore, the curvature of the inner and outer wythes are equal at all loading stages, as expressed in Eq. (1). Such an assumption is reasonable based on the experimental results.

$$\phi_I = \phi_O$$

(1)

where $\phi_I$ and $\phi_O$ are the curvature of the inner and outer wythes, respectively. The total applied moment $M_u$ is resisted by three components as given by Eq. (2):

$$M_u = M_I + M_O + FZ$$

(2)
where \( M_I \) and \( M_O \) are the moments in the inner and outer wythes, respectively; \( F \) is the applied force at the interface and \( Z \) is the distance between the centroids of the inner and outer wythes. The last term in Eq. (2) represents the composite interaction between the inner and outer concrete wythes.

**Full Composite Interaction**

At any applied lateral and axial load level, the maximum force required at the interface to develop the full composite interaction, \( F_c \), can be estimated by plotting the moment-curvature relationship of the inner and outer wythes independently for an assumed value of the force \( F_c \). The summation of internal forces of the inner and outer wythes at each point on the moment-curvature relationship can be expressed by:

\[
\text{Inner wythe:} \quad \sum C - \sum T = P_I + F_c
\]

\[
\text{Outer wythe:} \quad \sum T - \sum C = F_c
\]

where \( \sum C \) and \( \sum T \) are the summation of all compressive and tensile forces acting on the section, respectively.

For a given curvature of the fully composite section under the action of the applied moment and axial load, the moments carried by the inner and outer wythes can be estimated as shown in Figure 11. The analysis can be repeated for different values of the force \( F_c \) until Eq. (2) is satisfied. The internal forces as well as the strains at the top and bottom layers of the inner and outer wythes can be extracted from the moment-curvature analysis at the final selected value of the force \( F_c \) that satisfies the equilibrium.

**Partial Composite Interaction**

For any value of the interaction force, \( F \), less than \( F_c \), partial composite interaction takes place. The degree of composite interaction, \( k \), can be expressed as:

\[
\text{Partial Composite Interaction:} \quad k = \frac{F - F_c}{100}
\]

At any value of the interaction force \( F \), the unknowns are \( M_I, M_O, \phi_I \), and \( \phi_O \). These unknowns can be easily determined knowing the moment-curvature relationship of the inner and outer wythes and satisfying both Eqs. (1) and (2), as shown in Figure 11. The analysis can be repeated at different levels of composite interaction by varying the force \( F \) at the interface, re-establishing the moment-curvature relationships for the inner and outer wythes and finding the curvature that satisfies equilibrium.

**COMPARISON WITH EXPERIMENTAL RESULTS**

**Validation of the Analytical Approach**

To validate the proposed approach for PCSP, three different panels EPS2, XPS3, and XPS4 were selected from the experimental program. The panels were analyzed at different lateral load levels. At every load increment, the following procedures were carried out:

a. The moment was calculated at mid-span of the panel based on the applied axial and lateral loads;
b. The curvature of the fully composite section was evaluated based on strain compatibility and equilibrium of the composite section;
c. The maximum force required at the interface, \( F_c \), to develop the full composite action was estimated using the procedures outlined in the previous section of this paper;
d. Different degrees of composite interaction were considered by reducing the interaction force at the interface and calculating the corresponding curvature and strains at the top and bottom surfaces of the inner and outer wythes from the moment-curvature analysis;
e. The measured curvature was determined from the experimental results;
f. The predicted strains at the same curvature were compared to the measured values and the degree of composite interaction was evaluated at that load level.
Figures 12a to 12f show the predicted strain distribution for the three panels used in the current study at different load levels. Results of the analysis indicated that the proposed approach is consistent with the actual behavior of the panels, as the predicted strains compared well with the measured values at all load levels for the different panels. The approach is beneficial to determine the degree of the composite interaction at different load levels for different panels at any given curvature. Results of the analyses showed that the percent of composite interaction for both EPS and XPS foam-core panels were around 95-100 percent under the applied axial load only (lateral load = zero). As the lateral load increases, the percent of composite interaction decreases. At ultimate load level, the percent of composite interaction for EPS foam-core panel was around 93 percent, whereas, for XPS foam-core panels, the percent of composite interaction was around 82-85 percent depending on the reinforcement ratio of the CFRP shear grid. Such a behavior was also observed experimentally but it was not quantified. It should be noted that the configuration and layout of the CFRP shear grid were identical for panels EPS2 and XPS3. In XPS4 panel, the amount of the CFRP grid was increased by 33 percent as shown in Figure 5.

Shear Flow Capacity of CFRP Shear Grid and Foam Insulations

In this section, the proposed approach is extended to determine the shear flow capacity of the CFRP shear grid, EPS and XPS foam materials based on test results. The panels were reanalyzed at the critical section (section of maximum bending moment) at the ultimate load level. Steps a. through d. in the previous section were followed, and the maximum force at the interface, $F$, required to develop the specified percent of composite interaction at ultimate was evaluated for different panels. Results of the analysis are summarized in Figure 13. The combined shear flow capacity of the CFRP shear grid in addition to the foam, $q$, can be expressed by:

$$ q = \frac{F}{L} $$

where $L$ is the total length of the CFRP grid along the width of the panel up to the critical section. It should be noted that $L$ is equal to 360 in (9.1 m) for the EPS2 and XPS3 panels and 480 in (12.2 m) for the XPS4 panel. Test results revealed a very weak bond between the XPS foam and the concrete. Inspection of the panels after testing showed that the XPS foam was completely separated from the concrete and could be pulled up easily by hand. Therefore, the shear flow capacity of the XPS foam-core panels can be assumed to represent the capacity of the CFRP grid alone. Results of the analysis showed that the maximum force developed at 82 and 85 percent of composite interaction for panels XPS3 and XPS4 were 63 kips (280 kN) and 98 kips (436 kN), respectively. Consequently, the nominal shear flow capacity of the CFRP grid for XPS3 and XPS4 are 63/360=0.18 kips/in (32 kN/m) and 98/480=0.20 kips/in (35 kN/m), respectively, with an average value of 0.19 kips/in (34 kN/m). For the EPS foam-core panel, the maximum shear force developed at the interface at 93 percent of composite interaction is 144 kips (641 kN), which reveals a combined nominal shear flow capacity of the CFRP grid and EPS foam of 144/360=0.40 kips/in (70 kN/m). It should be noted that these estimated shear flow capacities for EPS and XPS foam-core panels are nominal values and should not be used in design without a suitable strength reduction factor. It is also interesting to note that the durability of the EPS foam has not been investigated experimentally. Therefore, the proposed shear flow capacity for the EPS foam-core panels is preliminary until further test data is available.

Simplified Design Chart for PCSP Reinforced with CFRP Shear Grid

The analytical approach proposed in this paper is computationally intensive to be used in everyday design of wall panels with EPS or XPS foam materials. Therefore, a simplified procedure is required to calculate the moment capacity of these panels at different degrees of composite interaction. Figure 14 shows a proposed design chart to calculate the nominal moment capacity of EPS or XPS wall panels as a function of the maximum shear force developed at the interface. The chart will vary by varying the cross-sectional dimensions and/or the reinforcement configuration/layout of the inner and outer wythes. The chart was developed by varying the applied moment and finding the corresponding shear force at the interface at different degrees of composite interaction. The degree of composite interaction was varied from 60 to 100 percent. Reducing the degree of composite interaction below 60 percent increases the curvature of the panel significantly and induces severe cracking prior to failure. This behavior is not recommended in practical applications as the panels are typically designed to remain uncracked up to the ultimate load level. The chart demonstrates that for any required moment capacity, there is a range for the shear force at the interface that the designer can select from depending on the desired degree of composite interaction. However, the lower the degree of composite action, the higher is the curvature and consequently the deflections as shown in Figure 15.
The minimum nominal moment corresponds to the fully non-composite panel, which is the summation of the moment capacities of the inner and outer wythes. Conversely, the maximum nominal moment is the capacity of the fully composite section of the wall panel. The thick solid line shown in Figure 14 is proposed to simplify the calculation and to optimize the selection of the shear force needed at the interface for any required moment capacity. Knowing the required force, \( F \), at the interface, the total length of the CFRP grid up to the critical section can be estimated using Eq. (7). The predicted capacities for the different wall panels used in the current study are also shown to illustrate the adequacy of the proposed simplified approach.

\[
L = \frac{F}{q}
\]

where \( q = 0.19 \text{ kips/in (34 kN/m)} \) for XPS panels and 0.40 kips/in (70 kN/m) for EPS panels.

CONCLUSIONS

The flexural behavior of six full-scale insulated precast prestressed concrete sandwich panels was investigated. The panels were subjected to monotonic axial and reverse-cyclic lateral loading to simulate gravity and wind pressure loads, respectively. Based on the findings of this study, the following conclusions could be made.

1. Panel stiffness, and consequently, deflections are significantly affected by the type and the configuration of the shear transfer mechanism.
2. Solid concrete zones provide higher values for percent composite action in comparison to other types of shear mechanisms.
3. Appropriate use of CFRP shear grid can provide an effective shear transfer mechanism in precast prestressed concrete sandwich panels. CFRP grid can be used to provide required composite action while allowing for a structure with high thermal-efficiency.
4. Appropriate selection of the carbon shear grid quantity and configuration is critical to achieve optimal structural performance of a panel.
5. Both EPS and XPS foam-core panels do not exhibit plane section behavior at ultimate loads. The percentage of composite interaction at ultimate for EPS foam-core panels is superior compared to that of the XPS foam-core panels.
6. An analytical approach for PCSP has been developed based on the interaction theory originally developed for composite steel beams. The approach can be used to determine the percentage of composite interaction for PCSP at different load levels at any given curvature of the panel. The approach has been validated with the experimental results and the predicted strains compared well with the measured values. The approach is applicable to PCSP of different configurations and can be applied to quantify the efficiency of various shear transfer mechanisms.
7. XPS foam does not contribute considerably to the shear transfer mechanism between the inner and outer wythes and can be completely ignored in the analysis of XPS foam-core panels.
8. The combined shear flow capacity of the EPS foam and CFRP shear grid used in the current study is estimated to be 0.40 kips/in (70 kN/m). Results of the analysis showed that the average corresponding value for XPS foam-core panels is 0.19 kips/in (34 kN/m).
9. A simplified design chart is proposed to calculate the nominal moment capacity of EPS and XPS foam-core panels at different degrees of composite interaction. The chart is valid only for the panel configuration, geometry, materials and reinforcement used in the current study. However, it can be easily produced for different panels. The chart demonstrates the effect of composite interaction on the induced curvature.

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REFERENCES


<table>
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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 a tested panel (foam removed)

Figure 2a—Inner panel view during testing

Figure 2b—Outer panel view during testing
Figure 3—Configuration and dimensions of 2-4-2 panels

Figure 4—Configuration and dimensions of 4-2-2 panels
Figure 5 — Layout of CFRP shear grids

- a- Layout - 1-
  Panels EPS1, EPS2, XPS1, XPS2, XPS3

- b- Layout - 2-
  Panel XPS4
Figure 6a— Load-displacement behavior of EPS1

Figure 6b— Load-displacement behavior of EPS2

Figure 6c— Load-displacement behavior of XPS1

Figure 6d— Load-displacement behavior of XPS2

Figure 6e— Load-displacement behavior of XPS3

Figure 6f— Load-displacement behavior of XPS4
Figure 7— Strain profile distribution for different panels at ultimate loads

Figure 8a— Typical failure of EPS1, XPS1 and XPS2 panels solid zones

Figure 8b— Failure of EPS2 panel
Figure 8c—Typical flexural-shear failure of XPS3 and XPS4 panels

Figure 8d—Separation at the top of XPS3 and XPS4 panels
Figure 9a— Strain distribution in fully-composite PCSP

Figure 9b— Strain distribution in non-composite PCSP

Figure 10a— Applied vertical and lateral forces on PCSP

Figure 10b— Forces and strain distribution on PCSP with partial composite interaction

Figure 11— Moment-curvature relationship for both inner and outer wythes
Figure 12a—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 0 for EPS 2

Figure 12b—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 5 kips (22 kN) for EPS 2

Figure 12c—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 11 kips (49 kN) for EPS 2

Figure 12d—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 20 kips (89 kN) for EPS 2

Figure 12e—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 5 kips (22 kN) for XPS 3

Figure 12f—Strain distribution under axial load of 37.8 kips (168 kN), lateral load = 12.6 kips (56 kN) for XPS 4
Figure 13 — Strain distribution at ultimate for EPS2, XPS3 and XPS4 panels at critical section

Figure 14 — Simplified design chart for partial composite interaction of PCSP

Figure 15 — Influence of partial composite interaction of PCSP on curvature at ultimate