INVESTIGATING COMPOSITE ACTION AT ULTIMATE FOR COMMERCIAL SANDWICH PANEL COMPOSITE CONNECTORS

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ABSTRACT

To achieve full or partial-composite action in prestressed concrete sandwich panel walls, the engineer must obtain a percent composite action from a connector manufacturer, making some engineers uncomfortable. Engineers are dependent upon the recommendations given by the connector manufacturers to establish their designs. This project tested six full scale sandwich panel walls to evaluate the percent composite action of various connectors and compare the results to those provided by the composite connector manufacturers. This study concluded that the reported degrees of composite action from each manufacturer are considered conservative in all instances for the connectors tested in this paper. Additionally, the intensity and type of connectors are important factors in determining the degree of partial composite action in a panel.

Keywords: Precast Insulated Wall Panels, Composite Shear Connectors, Reinforced Concrete, Prestressed Concrete

INTRODUCTION

The earliest documented project completed using sandwich panel construction in the United States of America occurred in 1906[1]. At that time, the new sandwich panel system was a novelty to designers and contractors. The panels were constructed by pouring a 2-in. layer of concrete while embedding steel ties into the concrete wythes (the steel tie configuration used is unknown). After the concrete cured, a 2-in. layer of sand was poured across the panel on top of which a second 2-in. layer of concrete was poured. After an unspecified amount of time, the panels were tilted on an angle, at which point the sand was washed out of the panel with a fire hose leaving an air gap between the inside and outside wythes. This air gap created a simple thermal barrier. After the sand was washed out of the panel, it was turned upright and fixed into place. Between 1906 and 1951, modern machinery enabled the invention of precast sandwich panel walls. Sandwich panel walls became much more efficient and led the way for the precast sandwich panel walls used today.

Research performed by F. Thomas Collins in 1954 relative to precast concrete sandwich wall panel (PCSWP) construction paved the way for the methods used currently[1]. This research began to explore important aspects of PCSWP design including different insulations, different shear connectors, and rational design. Shear connectors were limited to steel at the time, allowing for a composite system. Collins pointed out advantages of early sandwich panel walls including thermal efficiency, extended fire rating, and reduced dead weight[1]. These benefits are all similar to contemporary PCSWPs.

In 1971, ACI commissioned a committee of 23 people to develop standard design procedures for PCSWPs[2]. The design approach in the 1971 edition utilized an "effective section" approach. This standard recommended that "shearing stress should not be transferred through the nonstructural insulation core," as had been the common practice prior to that time. Instead, "compressive stress and bending stress should be carried by the concrete sections only"[2]. The new standards recommended insulating material be limited to cellular or mineral based aggregate in lightweight concrete. These design procedures assumed wythes of the PCSWPs do not act compositely. Though very conservative, this made design of PCSWPs very simple and enabled many engineers to design with them.

In the early 1990's it seemed the idea that panels could have a percent of composite action between 0% and 100% began to become a concern among engineers. Composite action is defined as the two concrete wythes acting together as if a single unit; non-composite action is defined as the two concrete wythes acting independently; and partial composite action is somewhere in between (see Figure 1). The amount of composite action of PCSWPs (and that panels can have a certain degree of composite action other than fully composite or non-composite) was explored by Einea et al.[3]. This study introduced a new proposed type of shear connector made from fiber-reinforced plastic (polymer), or FRP. Four shapes of connectors were created with only one connector (the FRP bent bar) selected for further study. The geometry of this bar was such that the shear capacity of the panels was heavily dependent upon the axial capacity of the FRP connectors. Conclusions of this report indicated that FRP connectors were structurally sufficient and thermally superior to their steel predecessors[4]. Though FRP connectors are brittle, ductile behavior was observed during failure.

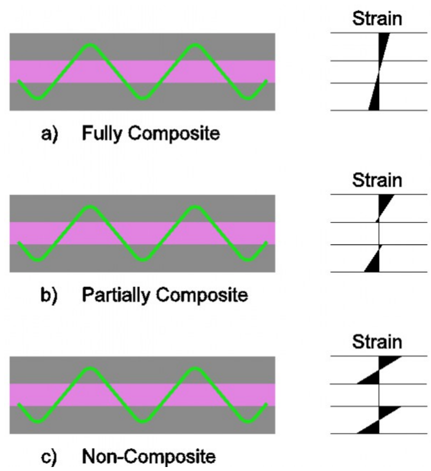


Figure 1 – Strain profiles for a) fully composite, b) non-composite, and c) partially composite

Engineers currently assume a certain percent of composite action based solely upon recommendations from connector companies. Designing PCSWPs to be 100% composite is challenging, but possible, while remaining thermally efficient. There is no general, industry accepted method to determine how to generate full composite action or a certain level of partially composite action. Connector sizing, selection and placement patterns are empirically determined by the manufacturers of an individual connector. The purpose of this paper is to evaluate several proprietary composite connectors and patterns recommended by their manufacturers and compare the results to the manufacturer recommendation for percent composite action at ultimate.

EXPERIMENTAL PROGRAM

Two 16-ft long and four 15-ft long concrete sandwich wall panels were tested to evaluate their flexural strength and the composite action provided different shear connectors and configurations. Three different connector configurations were investigated as presented in Figure 2. Two panels were tested with THiN Wall Tie 3/8 in. diameter connectors, two with HK composite ties, and two with a combination of Thermomass CC and X connectors. All connectors are a type of glass fiber reinforced polymer (GFRP). The THiN Wall connectors are similar to a zig-zag patterned 3/8 in. diameter rebar with longitudinally aligned fibers, manufactured by a pultrusion process. The Thermomass connectors are also an aligned fiber flat bar of GFRP that is either oriented in an X shape or orthogonal to the concrete wythes. The HK connectors are a mold-injected product with randomly aligned fibers. The manufacturing process and alignment of the fiber significantly changes the failure mode and ductility of the connectors[5].

Figure 2 – Shear Connectors Tested, Left to Right: THiN Wall NU-Tie, HK Composite Tie, Thermomass CC and Thermomass X

All panels were fabricated with XPS insulation and utilized shear connectors to attain a certain degree of composite action by transferring the shear flow between the wythes through the insulation. The design of the panels was performed in conjunction with representatives from Forterra Structural Precast (Salt Lake City, Utah) and Concrete Industries (Lincoln, Nebraska).

The THiN Wall panels had a 3-4-3 in. configuration with prestressed reinforcement in the longitudinal direction and shear connectors as shown in Figure 3 and Figure 4. The prestressing consisted of three low-relaxation 270 ksi strands with a 3/8 inches diameter. The panels were designated 343-2 (Figure 3) and 343-4 (Figure 4) with the 2 and 4 designating the number of THiN Wall shear connectors in each row. Shear connectors were distributed uniformly with a total of eight in the THiN Wall 343-2 panel and sixteen in the THiN Wall 343-4 panel. The difference in the number of connectors was intended to demonstrate the dependence of panel performance on the number of connectors contained within the panel. At the authors' request, THiN Wall panel 343-2 uses connectors at a lower level than typically used by THiN Wall for this panel configuration.

The HK and Thermomass panels had mild reinforcement and a 4-3-4 in. configuration. The reinforcement of these panels included four Grade 60 #3 bars in the longitudinal direction for each wythe and three shear connectors in each row. The HK shear connectors were distributed uniformly at sixteen-inch spacing for a total of 33 in the each panel (see Figure 5). In the Thermomass panels, 33 type CC-series shear connectors were uniformly distributed with an additional six X-series shear connectors spread throughout the panel (see Figure 6).

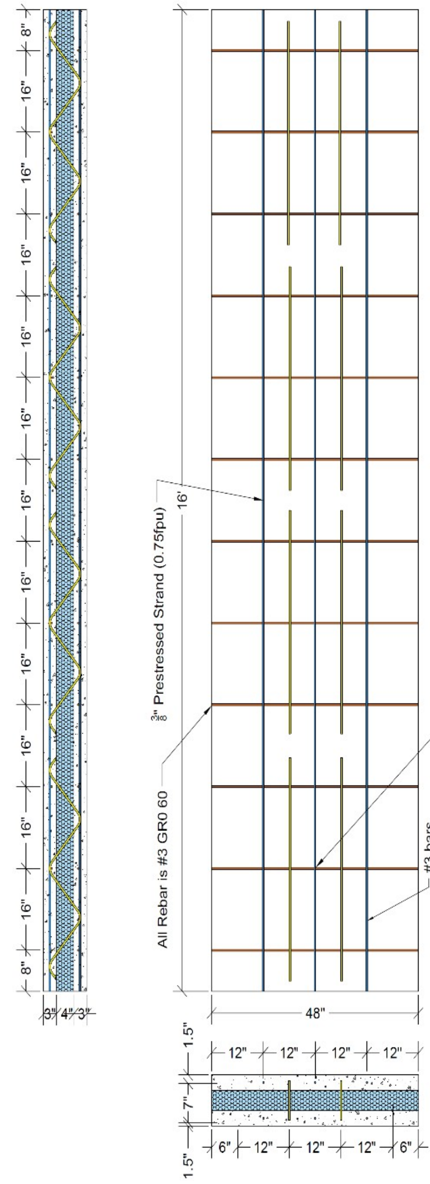


Figure 3- THiN Wall 343-2 panel details

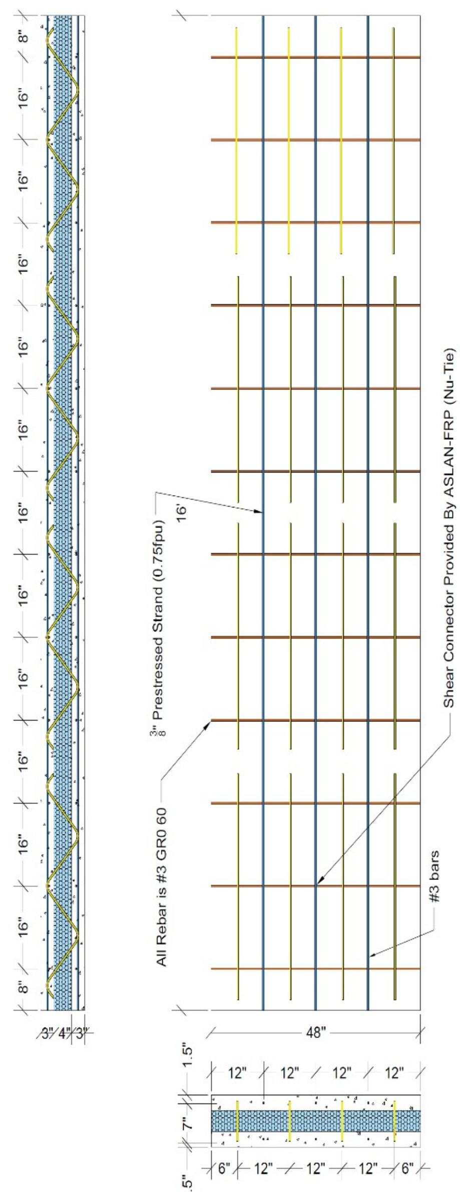


Figure 4- THiN Wall 343-4 panel details

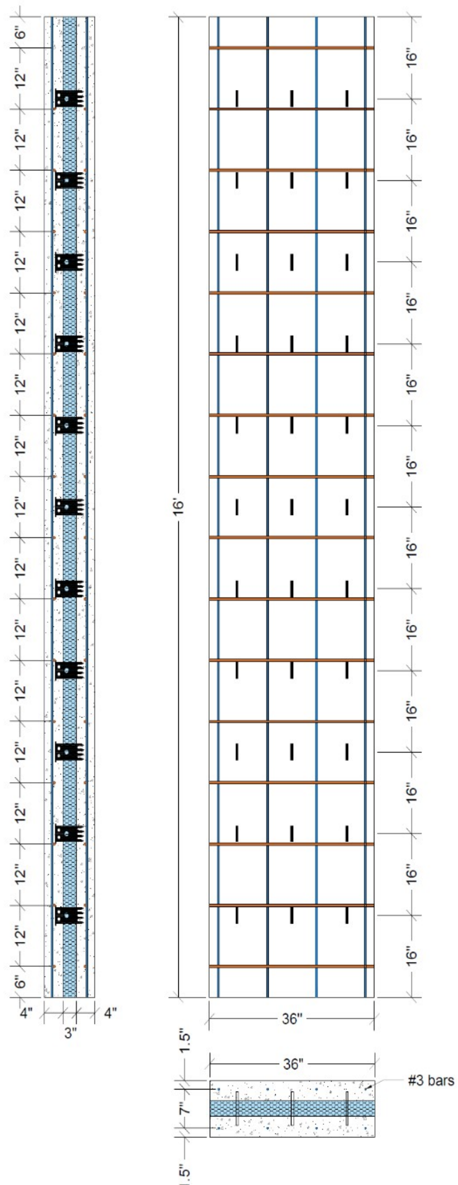


Figure 5- HK panel details

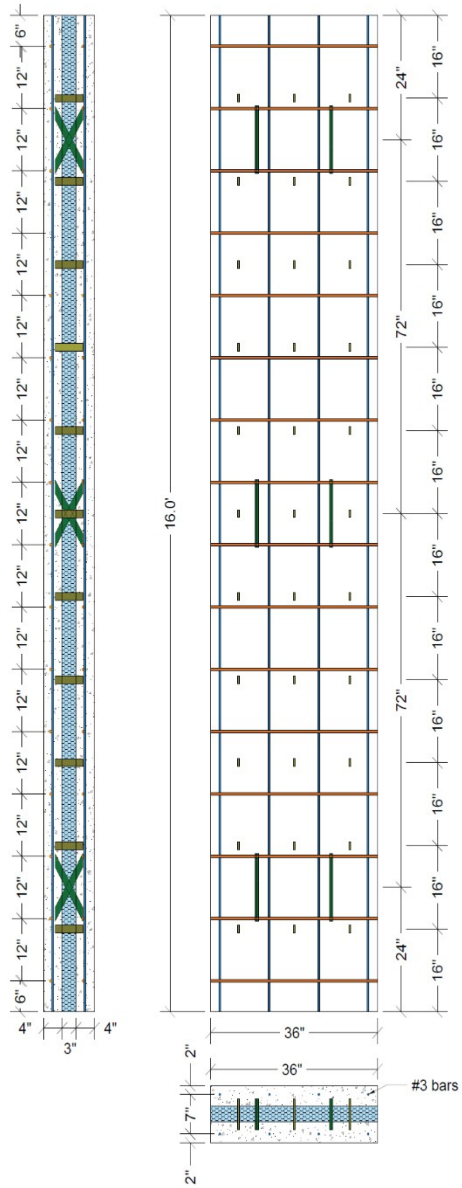


Figure 6- Thermomass A panel details

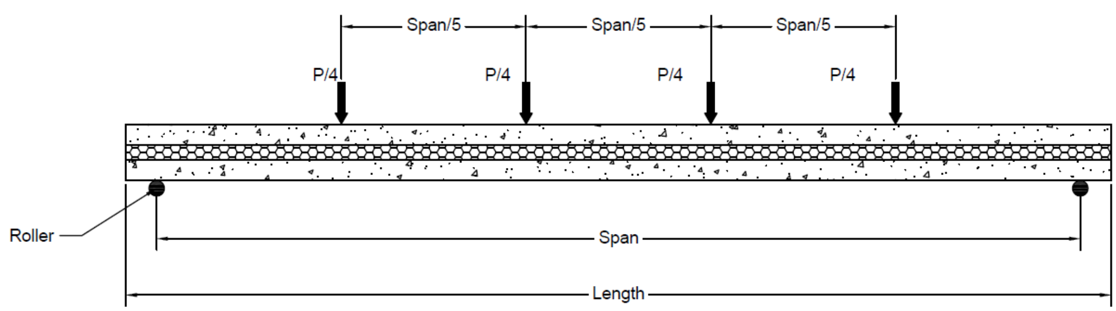
TEST SETUP

Each 16-ft long panel was placed on simple supports with a 15-ft span for THiN Wall 343-2 and THiN Wall 343-4 panels, and each 15-ft long panel had a 14-ft span for the HK and Thermomass panels. A single hydraulic actuator applied four point loads with a spreader beam assembly to simulate a distributed load, as shown in Figure 7.

Deflection was measured at midspan on both edges (north and south) of the panel. Relative slip between concrete wythes was measured using linear variable differential transducers at each panel corner (northeast, southeast, northwest and southwest). Prior to testing, dead load deflection was measured at midspan with a total station and high accuracy steel ruler by finding the elevations of the supports and at midspan. This procedure provided a dead load midspan deflection with an accuracy of 1/32 in. (0.031 in.)

Concrete compression strengths were measured using ASTM C39 procedures from 4 in. x 8 in. concrete cylinders sampled and provided by the precasters. Rebar and prestressing steel samples were obtained from each panel after testing by breaking out the concrete from the ends, where there was no plasticity.

Rebar were tested according to ASTM A370 and the full stress strain curved developed using a 2-in. extensometer. Because of gripping limitations of the tensile testing machine available, standard reusable chucks were used to test the 3/8-in. prestressing strand. Using chucks during tensile testing is known to limit both elongation and provide slightly lower ultimate stresses[6,7]. Only ultimate tensile stress was recorded for the prestressing strand because a proper (24-in. gauge length, rotation capable) extensometer was not available.



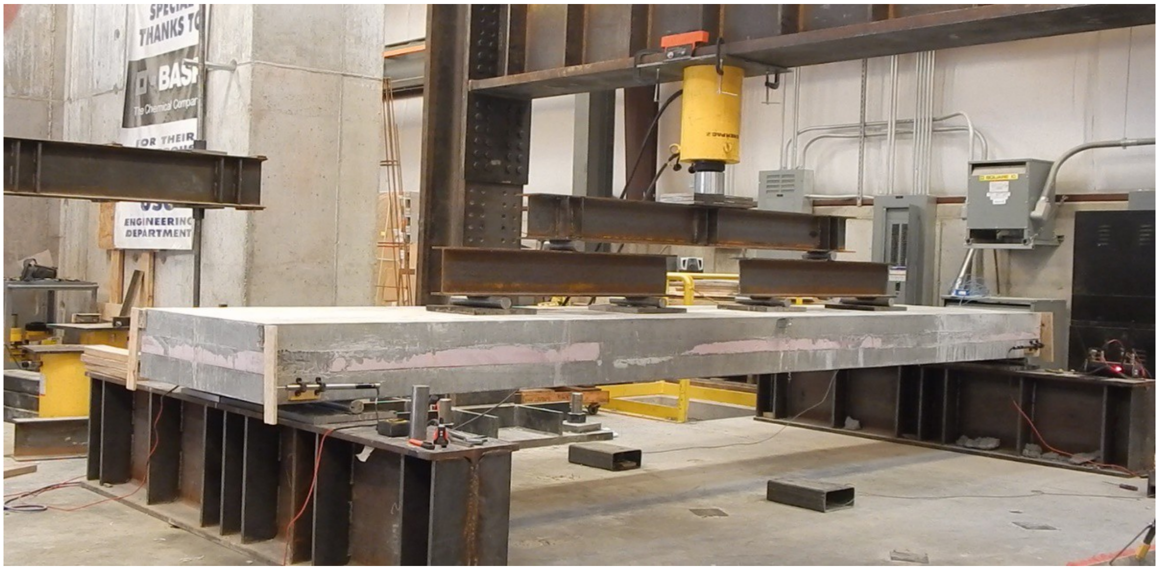


Figure 7 – Test setup

RESULTS AND DISCUSSION

Material Testing

The results of the ASTM C39 compression testing is presented in Table 1. Each value presented in Table 1 is the average of three cylinders from the compression wythe taken on the day of testing.

Table 1 – Concrete Compression Strength

Specimen	Average f'c (psi)
THiN Wall 343-2	10,400
THiN Wall 343-4	10,400
HK Composites 1	9,230
HK Composites 2	8,000
Thermomass 1	9,230
Thermomass 2	8,000

Figure 8 presents the stress vs strain curves for the rebar in the HK and Thermomass sandwich panels. The average yield stress was 72,200 psi and the ultimate stress was 110,000 psi. The average ultimate capacity for the prestressing strands was 259 ksi. It is likely the testing method described above affected the ultimate capacity of the strands.

Figure 8- Stress vs Strain for rebar in HK and Thermomass A panels

Full scale Panel Testing

All loads shown herein include self-weight, and all deflections include deflection due to self-weight as measured by a total station. Figure 9 presents the Load versus Deflection plot for THiN Wall 343-2 and THiN Wall 343-4 panels. The maximum loads attained by the two panels were considerably different. The maximum loads attained were 39% different (compare 463 psf to 333 psf in Figure 9). Observed slip at the maximum load in the 343-4 panel was 0.167 inches, whereas the slip at maximum load observed in the 343-2 panel was 0.24 inches at failure. It is clear that the shear tie intensity at the level tested in these two panels had a large effect on maximum load and slip.

Figure 9- Load vs Deflection for THiN Wall 343-2 (left) and 343-4 (right)

Figure 10 presents the Load versus Deflection plots for the HK 1 and HK 2 panels. The maximum loads attained by the two panels were similar. The maximum loads attained had only a 6% difference (comparing 529.5 psf to 498.8 psf in Figure 10). The amount of slip measured in the panels at maximum capacity was 0.08 in. in both panels.

Figure 10- Load vs Deflection for HK1 (left) and HK2 (right)

Load vs. deflection of Thermomass 1 and Thermomass 2 panels are presented in Figure 11. The maximum loads for these panels are also very similar with a difference of only 8% (compare 528 psf for Thermomass 1 and 485 psf for Thermomass 2 in Figure 11). The amount of slip measured in the panels at maximum capacity was 0.05 in.

Figure 11- Load vs Deflection for Thermomass A1 (left) and A2 (right)

Table 2 summarizes the maximum loads and slips measured for all tested panels.

Table 2- Panel test results

Specimen	Wythe configuration ***(in)***	Span length ***(ft)***	Maximum Load ***(psf)***	Slip at maximum Load ***(in)***
THiN Wall 343-2	3-4-3	15.0	334	0.26
THiN Wall 343-4	3-4-3	15.0	463	0.18
HK Composites 1	4-3-4	14.0	530	0.08
HK Composites 2	4-3-4	14.0	499	0.08
Thermomass 1	4-3-4	14.0	528	0.11
Thermomass 2	4-3-4	14.0	485	0.05

ANALYSIS OF SANDWICH WALL PANELS

Utilizing the theoretical fully composite moment, theoretical non-composite moment, and the actual measured moment from the test results, the degree of composite action, K_{Mu}, can be determined as shown in for different panels using Eq. (1).

(1)

Where

$M_{n,test}$ = experimental maximum moment of the sandwich panel
$M_{n,NC}$ = theoretical maximum moment of the non-composite sandwich panel
$M_{n,FC}$ = theoretical maximum moment of the fully composite sandwich panel

Figure 12 graphically demonstrates the relationship between moment and degree of composite action shown in Eq. (1).

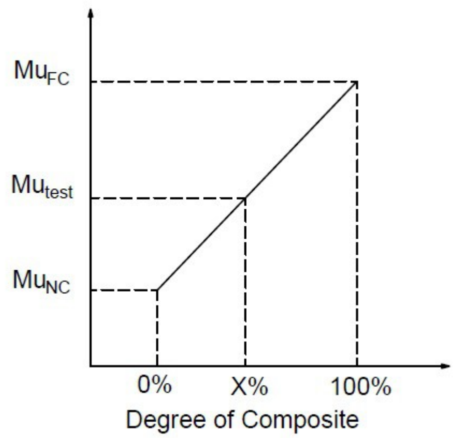


Figure 12- Degree of composite concept

Table 3 presents the midspan moment comparisons for the THiN Wall, HK Composites and Thermomass full scale panels. The measured maximum moments of the sandwich panels were used to evaluate the composite action achieved. The measured maximum moment was calculated at midspan, using the self-weight of the panel (a distributed load) and the four point loads. The fully composite nominal moment was calculated using strain compatibility and actual material properties for the concrete and steel as presented above, assuming the entire cross section was active. The non-composite moment strength was calculated in the same manner using only the properties of a single wythe and multiplying by two.

Table 3 – Composite Action Comparison

Specimen	M_{nFC} (lb*ft)	M_{nNC} (lb*ft)	Test % Composite	Manufacturers Reported % Composite
THiN Wall 343-2	55,000	15,800	70%	-*
THiN Wall 343-4	55,000	15,800	115%	100%
HK Composites 1	44,100	12,800	104%	80%
HK Composites 2	43,400	12,200	97%	80%
Thermomass 1	44,100	12,800	103%	70%
Thermomass 2	43,400	12,200	93%	70%

* Purposely reinforced lower than usual – not a typical panel

The THiN Wall 343-4 panel resulted in 115% composite action. Other programs have noticed over 100% in the past, which is likely due to material variability as it would be impossible for a panel to be stronger than theoretically composite. This panel, had it been designed by THiN Wall, would have been designed at 100% composite. The 343-2 panel would not have been a design coming from THiN wall, but was prepared to demonstrate what would come from under detailing such a panel. Doubling the number of connectors resulted in a 30% increase in composite action at ultimate.

The HK Composites connectors at the as-built 16-in. spacing would have resulted in panel designed as 80% composite per HK Composites guidelines. Both panels achieved far more than 80% composite (see 104% and 97% in Table 3).

The Thermomass panels resulted in a similar amount of composite action as those presented by HK Composites connectors (see 103% and 93% in Table 3). However, Thermomass would recommend only 70% composite action at nominal strength for these connectors.

From the panel configurations tested (length, reinforcing, etc.), with the recommended connectors and connector patterns, it is clear that the manufacturer recommended empirically based composite actions can be accurate and conservative.

In this study, the concrete was allowed to bond to the foam, possibly affecting the apparent composite action, which is a justified concern[5]. This may or may not be a serious design consideration considering shear testing performed by Olsen and Maguire[5] and cyclic testing performed by Frankl et al.[8], respectively, but was not investigated in this study.

CONCLUSIONS

Six concrete sandwich panels were tested to failure at the Utah State University Structures Lab. The purpose of the testing was to evaluate the percent composite action for the connector configurations and compare the results to those reported by composite connector manufacturers. The following conclusions can be made from the experimental program:

1. The type and intensity of shear connectors significantly affect the degree of composite action achieved in a concrete sandwich wall panel. Doubling the number of shear connectors in the THiN Wall panels resulted in a large gain in percent composite action. Note that the THiN Wall 343-2 panel is reinforced much lighter than would be detailed for an actual building
2. The manufacturer-reported degree of composite action can be considered conservative for the panel configurations and connectors and connector patterns tested in this paper.

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