SHEAR TESTING OF PRECAST CONCRETE SANDWICH WALL PANEL COMPOSITE SHEAR CONNECTORS

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ABSTRACT

As energy codes become more stringent, thermal efficiency of Precast Concrete Sandwich Panel Walls has become more important. This paper addresses the problem of predicting the behavior of full scale precast concrete sandwich panel walls, using data collected from small, inexpensive push-off specimens. Several fiber reinforced polymer (FRP) connectors being used today underwent shear testing performed on component scale push-off specimens. Each specimen contained several of the FRP connectors and the variables studied were wythe thickness, insulation type and insulation bond. A simplified beam spring model was created which uses beams to represent the concrete wythes and shear springs to model the shear deformation behavior created by the foam and shear connectors. This model was found to be accurate as compared to results from the literature. A short parametric study was performed using the beam spring model to investigate the effects of connector strength, pattern and intensity. It was found a triangular distribution of shear connectors – more lumped near the ends – is more structurally efficient. Further validation of this model is required and economizing and simplifying this procedure is key to more widespread implementation of thermally efficient, structurally composite, precast concrete sandwich panel walls.

Keywords: Precast Sandwich Wall Panels, Shear Connectors, Composite Action, Partially Composite

INTRODUCTION

Precast Concrete Sandwich Panel Walls (PCSPWs) have been in use for over 60 years. They provide a very efficient building envelope for many structures. Sandwich panel walls combine structural and thermal efficiencies into one simplistic design. This system is also advantageous over conventional methods because it eliminates many delays caused by field work as well as the need for several sub-contractors. Characteristic PCSPWs comprise an outer and inner layer (or wythe) of concrete separated by an insulating material. To achieve maximum structural efficiency, the wythes are connected by shear connectors that penetrate through the insulating layer which can provide various levels of composite action. More stringent energy building codes have demanded greater thermal efficiency so these shear connections are often made of various composites to eliminate thermal bridging.

Historically, Sandwich Panel Walls, as structural envelopes, have been produced in the United States for more than 100 years. One of the earliest examples of sandwich panel walls was built in 1906 (Bunn 2011)[1]. This *tilt-up* wall was produced by casting a 2-inch layer of concrete, covered by a 2-inch layer of sand, and then casting a second 2-inch layer of concrete. The concrete panels were connected using steel ties with an unknown design. The sand was washed out with a fire hose as it was put into place. Some of the earliest PCSPW's were built in 1951 in New York City, New York. The production lines used to build these precast insulated wall panels were 200 feet long. Each panel was six feet high and ten feet wide and was transported to British Columbia, Canada and used for a pulp mill. "[The panels] consist of a 2-in. thick layer of cellular glass insulation and two wire-mesh reinforced slabs of 3000-psi concrete, tied together with channel-shaped strips of expanded metal. These ties also serve as shear reinforcing" (Roberts 1951)[2]. These panels had an overall thickness of 5.5 inches.

The majority of precast sandwich panel walls between 1951 and the mid 1990's have had nearly identical components with varying insulation types, dimensions and wythe connection design. Reinforced concrete wythes, foam insulation, and steel connectors were components of every panel. With the huge push for Leadership in Energy and Environmental Design (LEED) Certified buildings, there is increased demand for these thermally and cost efficient structural elements. There has been a great deal of research done on PCSPW's in the last two decades, specifically with respect to thermal efficiency. Thermal bridging is still a significant challenge for PCSPWs, particularly in structurally composite panels. There have been many proposed solutions to enable structural composite action without thermal bridging and many have been implemented and are currently in use across the United States. Fiber Reinforced Polymer (FRP) connectors are a growing segment of todays wythe connectors, enabling composite action with limited thermal bridging. A major challenge associated with designing with FRP connectors is determining the percentage of composite action. Many FRP connectors tout 100% composite action at failure, however, designing for cracking and P-delta effects is much less clear.

The research presented in this paper is aimed at developing general tools for PCSPW designers to use in everyday practice, specifically through component level testing and simplified modeling. Currently in design, the level of composite action is typically based on limited testing performed by the connector companies themselves. Composite percentages are then given to the design engineers for the specific connector so that the engineer can design the panel. This must be done at three design stages: cracking, elastic deflections and nominal strength. This paper presents a design tool to predict the behavior at cracking and elastic deformations using connector shear testing.

EXPERIMENTAL PROGRAM

The experimental portion of this research was to test several different proprietary and non-proprietary FRP shear connector systems by fabricating and testing 40 small scale "push-off" specimens to apply direct shear to the connectors. By determining the shear load versus shear deformation behavior of each system at the component level, engineers can make more informed decisions about the full scale behavior.

DESCRIPTION OF TEST SPECIMEN

The push-off test specimens were all the same dimensions, with only connector spacing number changing per manufacturer recommendations. Each specimen was 3 feet wide by 4 feet tall and contained a variety of connectors and configurations from four companies (A, B, C and D). Each specimen was made up of three concrete wythes and two foam wythes. Wythe dimensions were either 3"x3"x6"x3"x3" or 4"x4"x8"x4"x4". Foam types that were used include: Extruded Polystyrene (XPS), Polyisocyanurate (ISO), and Expanded Polystyrene (EPS). The concrete was reinforced with No. 3 rebar spaced every 6 inches (exact spacing of rebar was contingent upon the accommodation of connectors).

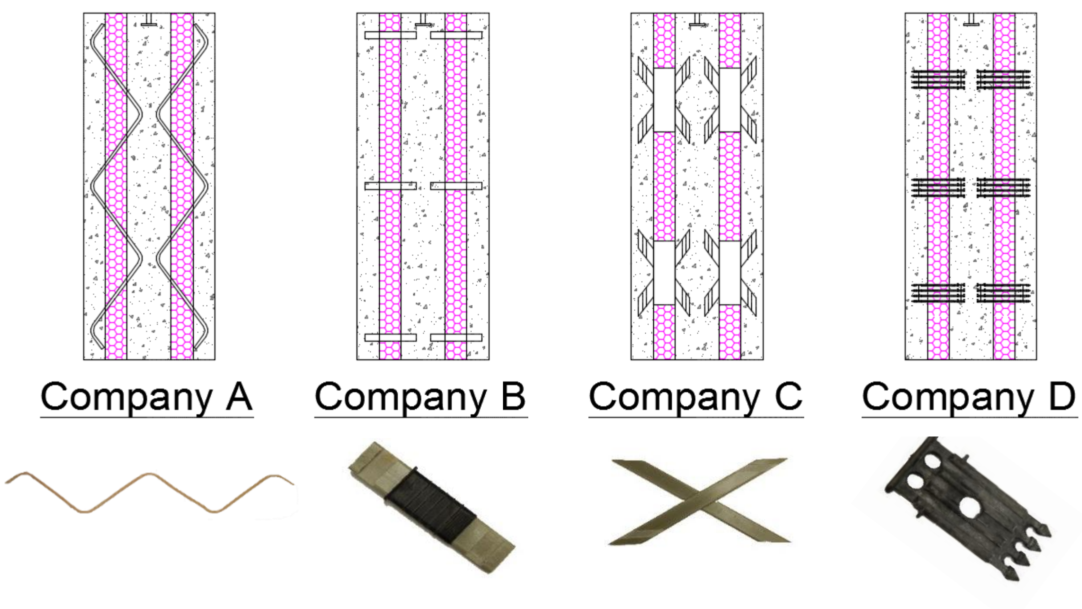

Figure 1 – Concrete Push-off Specimens

Each connector group tested was manufactured using Glass Fiber Reinforced Polymer (GFRP). However, not all companies used the same manufacturing process. Companies B and C use an extrusion process in which the fibers are all aligned, giving the connector a definitive grain. Company D connectors are created using a very economical mold injection process that results in random fiber alignment. Company A aligns fibers in the shape of a “zig-zag”, or truss, prior to setting them in the polymer. Once hardened, fibers are aligned to this truss shape giving it a multi-directional grain. When the fibers are completely aligned and oriented perpendicular to the load, larger deformations may occur, when compared to a random fiber orientation.

TEST PARAMETERS

A test matrix was created to provide information on each of the specimens that needed to be constructed. This was based on three variables: 1) connector type, 2) foam type, and 3) concrete/foam interface bond. The blank spaces in the table below exist only because companies B and C do not supply expanded polystyrene foam with their connectors.

Table 1 – Test Matrix for Five-Wythe Push-Off Specimens

TEST MATRIX FOR FIVE-WYTHE PUSH-OFF SPECIMENS						
			Connector A	Connector B	Connector C	Connector D
Foam Type	Wythe Thickness	Bond	CA	CB	CC	CD
Expanded Polystyrene (EPS)	3"	B	EPS.3.B.CA	-*	-*	EPS.3.B.CD
		UB	EPS.3.UB.CA	-*	-*	EPS.3.UB.CD
	4"	B	EPS.4.B.CA	-*	-*	EPS.4.B.CD
		UB	EPS.4.UB.CA	-*	-*	EPS.4.UB.CD
Extruded Polystyrene (XPS)	3"	B	XPS.3.B.CA	XPS.3.B.CB	XPS.3.B.CC	XPS.3.B.CD
		UB	XPS.3.UB.CA	XPS.3.UB.CB	XPS.3.UB.CC	XPS.3.UB.CD
	4"	B	XPS.4.B.CA	XPS.4.B.CB	XPS.4.B.CC	XPS.4.B.CD
		UB	XPS.4.UB.CA	XPS.4.UB.CB	XPS.4.UB.CC	XPS.4.UB.CD
Polyisocyanurate (ISO)	3"	B	ISO.3.B.CA	ISO.3.B.CB	ISO.3.B.CC	ISO.3.B.CD
		UB	ISO.3.UB.CA	ISO.3.UB.CB	ISO.3.UB.CC	ISO.3.UB.CD
	4"	B	ISO.4.B.CA	ISO.4.B.CB	ISO.4.B.CC	ISO.4.B.CD
		UB	ISO.4.UB.CA	ISO.4.UB.CB	ISO.4.UB.CC	ISO.4.UB.CD

*: Company does not use EPS with their system

SPECIMEN FABRICATION

Specimens were cast horizontally, one layer at a time. Forms were built out of HDO (high-density overly) plywood. The first wythe would be poured immediately followed by the insertion and vibration of the connectors and foam. The forms would be stripped and taller forms constructed in their place. Once taller forms were in place, the second wythe would be poured and immediately followed by the insertion and vibration of the connectors and foam. Forms would be stripped and the tallest forms would be constructed, after which the final concrete wythe would be poured. The unbonded specimens used a plastic sheet between the foam and concrete surfaces to eliminate the bond. Form work was removed completely 7 days after the final pour. After at least 28 days and once the concrete strength was achieved (>4000 psi), the specimens were prepared for testing. The purpose of the concrete in this experiment is to provide fixity for the connectors. By ensuring concrete strength greater than 4000 psi, failure of the specimen was forced in the connector rather than in the concrete. As long as connector pullout is not achieved, the concrete strength is very unlikely to affect the pure shear testing for the connectors due to the vast difference in strength and stiffness of the connectors compared to the concrete in the tested position.

TEST SETUP & INSTRUMENTATION

Push-off specimens were loaded by placing a ram and load cell on the wide center wythe. The load was transferred to the specimen through a spreader beam which in turn passed the load into the specimen directly in line with the connectors. The specimen was supported only on the outer wythes at the bottom. Extra care was taken to ensure the specimen was flush on the supports. Relative displacement of the inner wythe to the outer wythes was measured in four places and averaged to determine the reported displacements. The Linear Variable Differential Transformers (LVDTs) were attached to the outer wythes using a custom built bracket. Displacements were measured by fixing a small piece of mild steel to the center wythe, providing a reference point for LVDT's to measure from. A load cell was placed at the ram-to-spreader beam interface to measure the overall applied load.

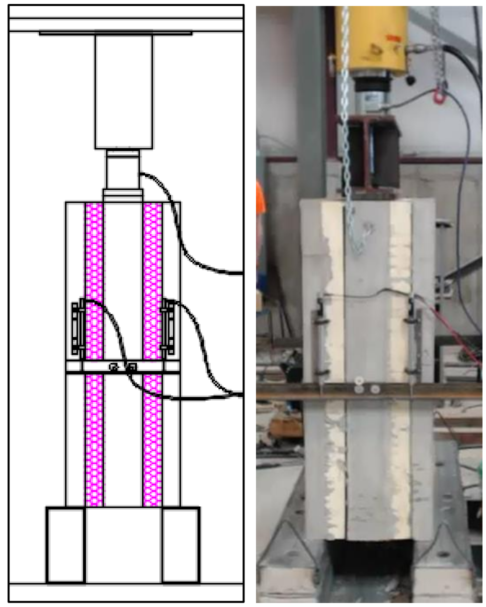

Figure 2 – Push-off Test Setup

The LVDTs used for this testing were newly purchased with NIST traceable calibration in February 2015. Load cell calibration was verified in February using a Tinius Olsen testing machine with NIST traceable calibration, last calibrated March 2014. The equipment used to collect data was the Bridge Diagnostics Inc.-Structural Testing System.

RESULTS OF THE EXPERIMENTAL PROGRAM

Each push-off specimen was loaded through failure and results are presented in Figure 3. Many of the connectors maintained significant load while continuing to deform, whereas, others failed soon after they reached peak load. Foam type and bond between the concrete and foam interface did not produce vast differences in strength or ductility performance, though unbonded specimens show a consistent reduction in capacity. Due to the inherent variability associated with concrete bond, it is not recommended that designers use the fully bonded values for long term strength without long term testing.

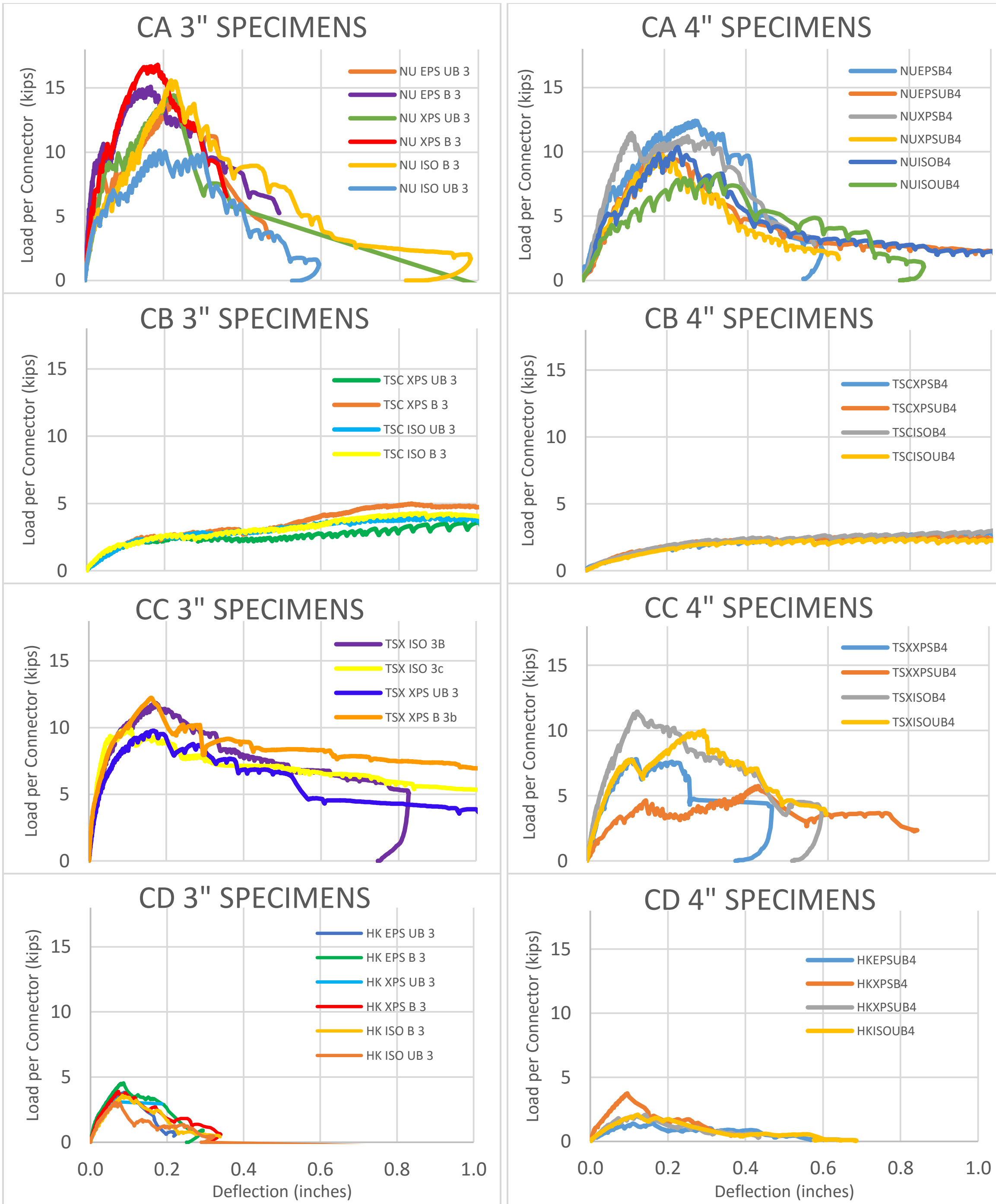

Figure 3 – Load versus Deformation for all shear connectors in this study

Figure 4 presents an ultimate strength (F_{max}) comparison for the specimens tested in Figure 3. Company A using XPS insulation produced the strongest individual shear connection (16.8 kips each), while Company D with 4 in. unbonded ISO insulation produced the smallest (1.39 kips each), although in this instance there may have been a fabrication issue. There was a consistent reduction in strength between 3 in. and 4 in. wythe specimens, although connector C with ISO and Connector D with XPS experience no or only a small reduction in strength.

Overall the unbonded specimens produced a reduction in ultimate strength across all connectors, although it was not consistent. For example Connector A with EPS produced a reduction of approximately 10% when unbonded, but Connector D with EPS produced an approximately 70% difference when unbonded.

Foam types did contribute to the ultimate strength, however, the results were inconsistent, especially with the ISO. The ISO surfaces were not consistent between connector manufacturers because the ISO form selected for each was part of the manufacturer's system and therefore what a precast producer would receive upon purchase. Some ISO surfaces were smooth plastic, others had a paper surface so bonded and unbonded behavior is inconsistent for the ISO. Ultimate strengths were typically higher with XPS, but Connector D experienced higher loads with EPS.

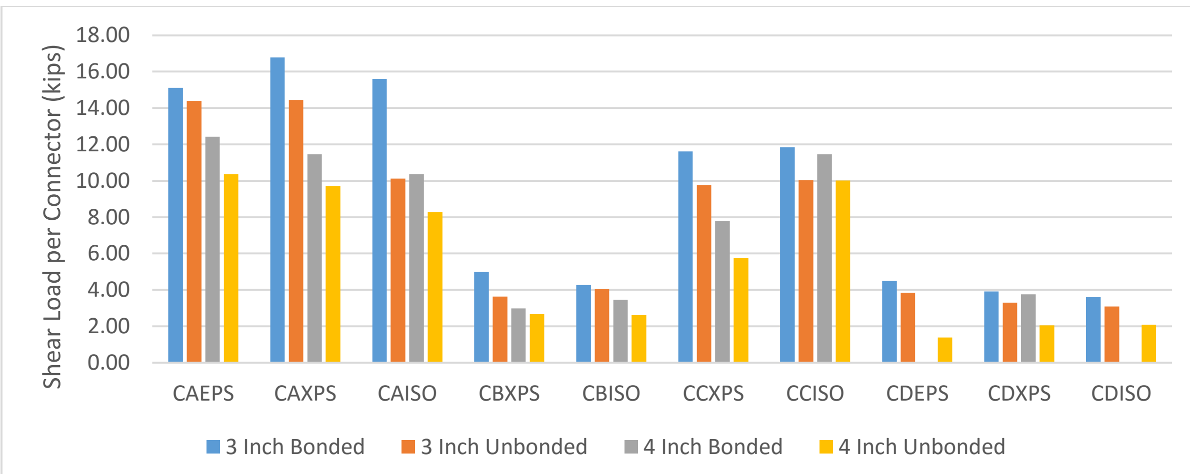

Figure 4 Ultimate Strength Comparison

An "elastic limit" load (F_E) and "elastic" stiffness (K_E) were identified from each push-off specimen's load deformation curve. This was done by visually identifying the yield point as shown in Figure 5. Figure 6 also includes the maximum force (F_{max}) that was observed during testing. Although fatigue testing was not performed, it is assumed that F_E should be the maximum force allowed in the connector during service loading scenarios as damage may accumulate at higher loads. Figure 6 presents a visual comparison of elastic load limits *for all push-off specimens in this paper. The connectors that exhibited a high ultimate strength in*

Figure 4 Ultimate Strength Comparison

also presented with a similar F_E, relative to the other connectors. Connector A with XPS had the highest F_E value (10.6 kips), but Connector A with ISO was significantly lower than the EPS and XPS combinations. This is likely due to the difference in ISO surface treatment used with the Connector A system as previously discussed, which might cause inconsistent bond and results. There was relatively little difference between the Connector A ISO bonded and unbonded. Similar relationships between insulation, wythe thickness and bond performance are observed with respect to F_E.

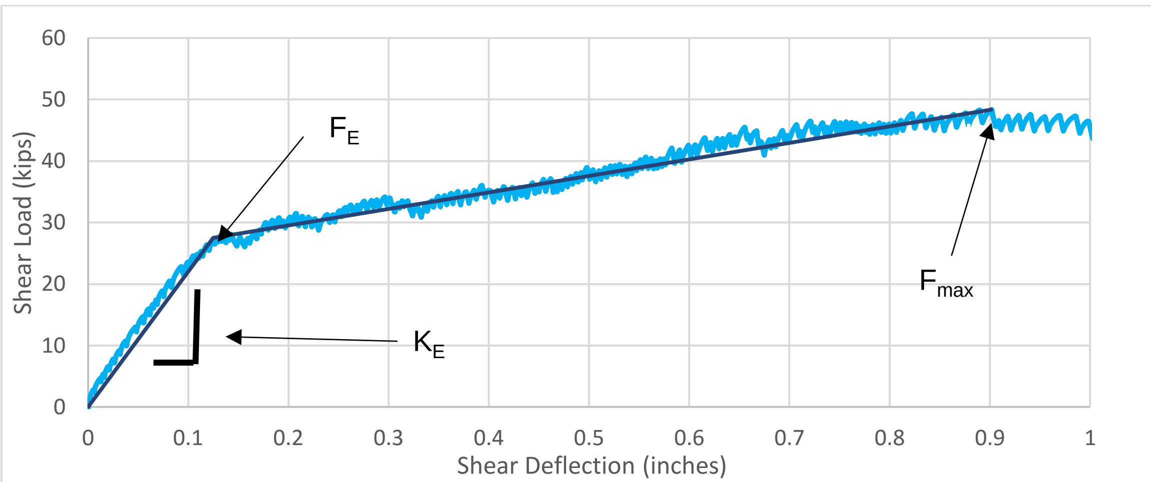

Figure 5 Determination of Elastic Load and Stiffness

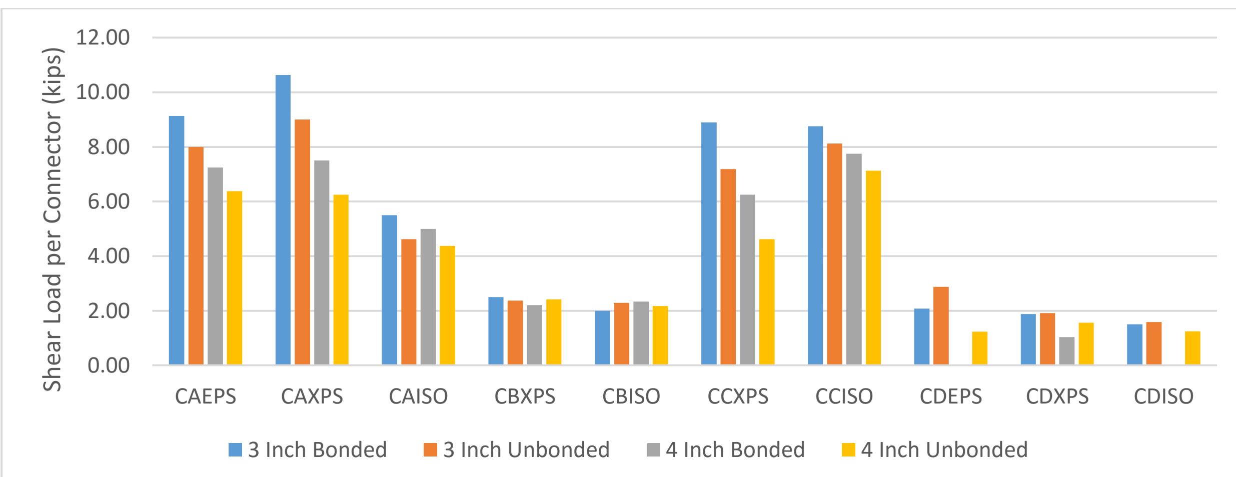

Figure 6 – Elastic Limit (F_E) Comparison (per connector)

Figure 7 presents elastic stiffness values for the push-off specimens tested in this program. Connector B presented with the lowest K_E values with as low as 6 kips/in. with the

4 in. unbonded specimens, whereas several Connector A specimens exceeded 150 kips/in. Connector D specimens had been testing with lower relative strengths when compared to the others, but is of similar stiffness to the other connectors in many instances. Connector A and Connector C showed significantly higher stiffness and strength, this is likely due to their truss like fiber orientation, allowing more efficient horizontal load transfer, as opposed to the Connector B and Connector D load transfer mechanism, which is more similar to dowel action or pure shear .

The Connector A with unbonded ISO as well as Connector C with unbonded ISO presented with higher stiffness than their bonded counterparts. This was unexpected and may be evidence of highly variable bond behavior and/or insulation behavior. Generally, 4 in. wythes, bonded and unbonded, exhibit significantly lower stiffness, much lower than the observed reductions in strengths in

Figure 4 Ultimate Strength Comparison

and Figure 6.

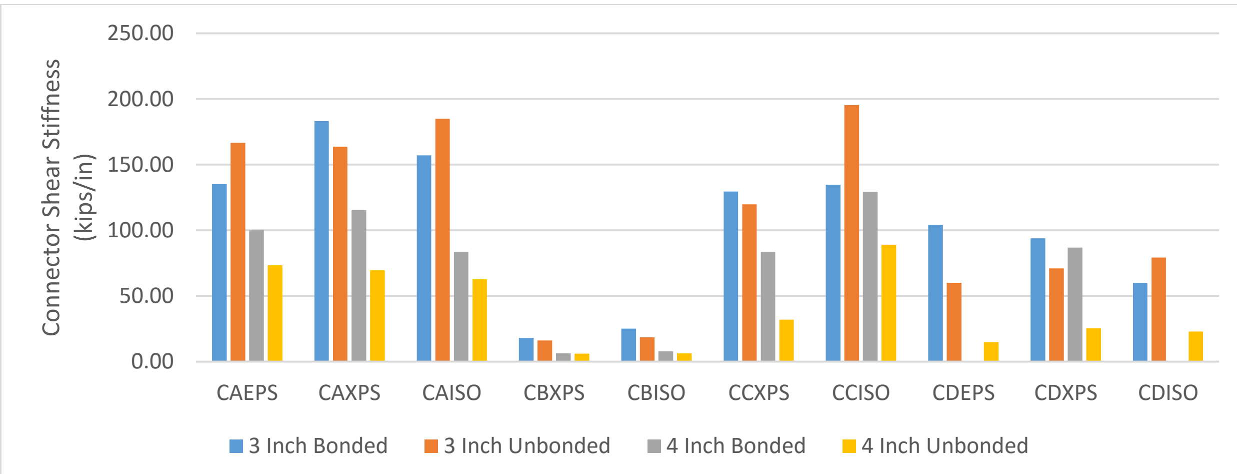

Figure 7 - Elastic Stiffness (K_E) across Specimens (per connector)

The reader should keep in mind that the differences in strength and stiffness should not be the sole factor in selecting a shear connector component. Cost, durability, ease of fabrication and customer support should also be considered when selecting a system. Also, as will be discussed below, connector configuration is also important to performance.

Only a single specimen was tested under each configuration. The results are undoubtedly affected by random variations and the presented results should not be considered exact, average, minimum or maximum examples of the strength provided by these composite connectors. Rather, the produced results should be considered comparative to one another to identify general trends. Further study should be performed to assess the variability of the individual connectors considering the significant differences in manufacturing, shape and material, especially with inherently variable insulation properties, quality and bond performance.

SIMPLE MODEL TO PREDICT ELASTIC FULL SCALE BEHAVIOR

Predicting sandwich panel elastic stresses and deformations is paramount for design. Several researchers have developed techniques to predict sandwich panel deformations[1,3,4]. Prediction methods vary significantly in complexity and accuracy[4,5].

Full scale test data from Naito et al. (2011)[6] for a precast concrete sandwich panel was compared to a complex mechanics based model created by Bai and Davidson[5] and the simplified beam and spring element model below. The precast panels tested by Naito et al. (2011) were 3 in. x 3 in. x 3 in. wythe panels 32 in. wide, 12 ft. long, loaded with a 10 ft. span. Connector B shear connecters were placed at 16 in. on center starting 8 in. from the end of the panel, using extruded expanded polystyrene. Concrete was 8,800 psi concrete with an estimated elastic modulus of 5,350 ksi. The Connector B shear connectors are the only connectors that overlapped in this study and the Naito et al.[6] study.

The analytical model created used commercial matrix analysis software package and any commercial or personal matrix analysis software could produce an identical model and could also be easily built into commercial wall panel analysis and design software. The very simple model, shown in **Error! Reference source not found.**, uses only beam and spring elements combined with the appropriate material values, boundary conditions and the results for the shear connector testing presented in this paper. Beam elements are assigned the individual gross properties of each wythe, separated by the distance between the wythe centroids. Link elements assigned Connector B shear stiffness link the wythes, in this case at 8 in. on-center along the panel length. The test specimen had shear connectors placed at 16 in. centers, starting at 8 in. from the end of the beam. Spring elements corresponding to the location of the shear connectors were assigned a shear stiffness equal to K_E for bonded XPS Connector B connectors in Figure 7. The remaining links, which represent a lumped insulation stiffness between the links representing the composite connectors, were assigned a shear stiffness equivalent to 17 kip/in based on the shear modulus (estimated at 200 psi) and the tributary geometry of the insulation wythe (32 in. wide x 8 in. tributary length x 3 in. thick) and a rigid longitudinal stiffness. Point loads were assigned at each node on one face corresponding to the pressure, multiplied by the tributary width between nodes. All links were assigned a longitudinal stiffness of 45 kip/in based on the tributary geometry and an assumed Young's modulus of XPS insulation (estimated at 500 ksi).

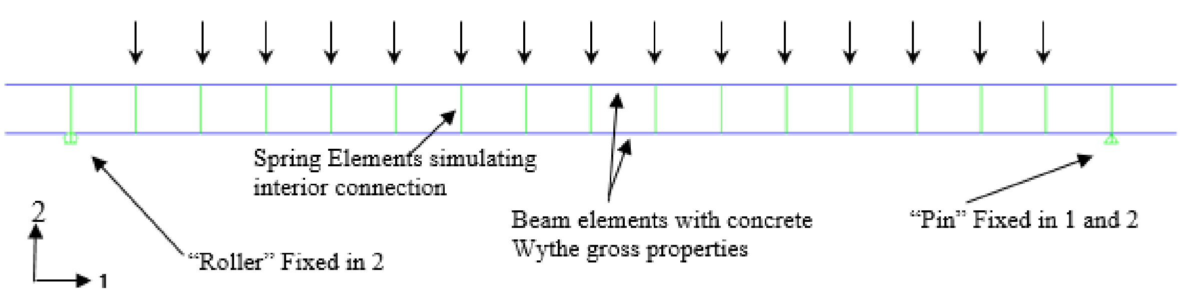

Figure 5 Beam and Spring Model used to model Naito 2009 Full Scale Specimens

Figure 6 presents the comparison between the three identical test specimens (denoted PCS5 A, B and C) from Naito (2011)[6]. The beam and spring model shows very good agreement with the observed test data and the complex mechanical model presented by Bai and Davidson (2015). The beam and spring model is limited to elastic deflections, although if inelasticity were introduced (non-linear springs and beam elements) ultimate deflections and strength can likely be determined, however this may not be necessary for most designs.

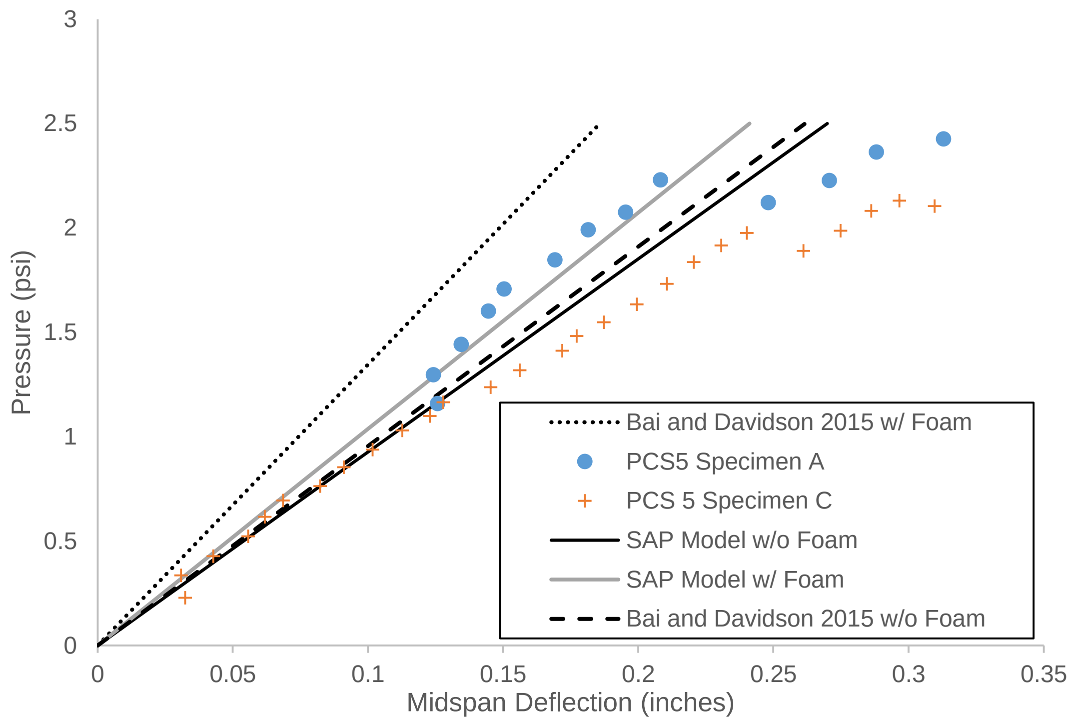

Figure 6 Deflection and Resistance Comparisons in Elastic Range for PCS5 Specimens from Naito et al (2011)

The beam and spring model has only been validated using Connector B shear connectors, for a single wall panel configuration. The authors are in the process of testing full scale specimens for all connectors in the present study and will be able to determine how valid the beam and spring model is in all situations. Regardless, the beam and spring model presented here is a promising option for elastic analysis of precast sandwich wall panels with composite shear connector systems, including those with unsymmetrical wythes and irregular connector patterns, inclusion of P-δ and P-Δ effects. Based on preliminary evaluation, using this model, it should be possible to tailor percent composite action at cracking checks, deflection checks by distributing connectors over the wall panel, while maintaining elastic behavior within the connectors.

For instance, in an example 8 ft. wide, 30 ft. long, with a 30 ft. span, under 50 psf lateral load, with concrete compression strength of 8000 psi and elastic modulus 5100 ksi, ignoring P-δ and P-Δ, can be simulated with various connector patterns. For a generic connector with individual unbonded stiffness of K_{EL} = 50 kip/in, Figure 7 presents the difference between adding connectors in a uniformly distributed fashion or triangularly distributed with connectors concentrated near the panel ends. With the same number of connectors (~75) deflection could be reduced by 10% by changing connector distribution. Deflection for the uniformly distributed connectors was matched with 16% fewer connectors (74 connectors versus 62 connectors, see Figure 7) when using a triangular connector distribution and locating more near the ends.

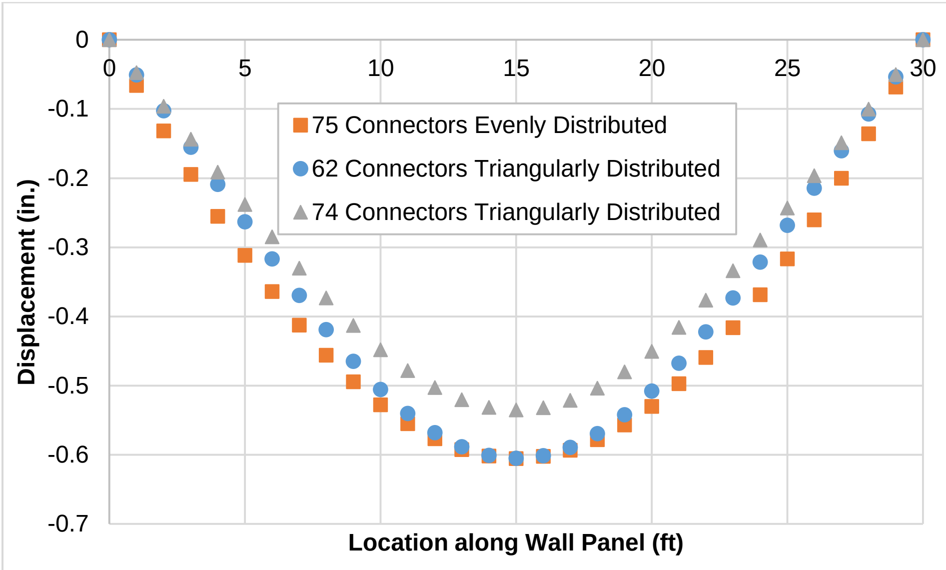

Figure 7 – Deflection Comparison with Different Connector Distributions

Distribution of connector force did change for these different connector patterns. Figure 11 presents a plot of connector force along the length. For the uniformly distributed connectors, the maximum connector force is located 4 ft. from the end, while for both triangularly distributed models, maximum connector force occurred 8 ft. from the end. Furthermore, the uniformly distributed connectors exhibited a higher maximum connector force. These results indicate designers should be aware of where connectors are highly loaded, especially at service limit states where connector forces should remain elastic.

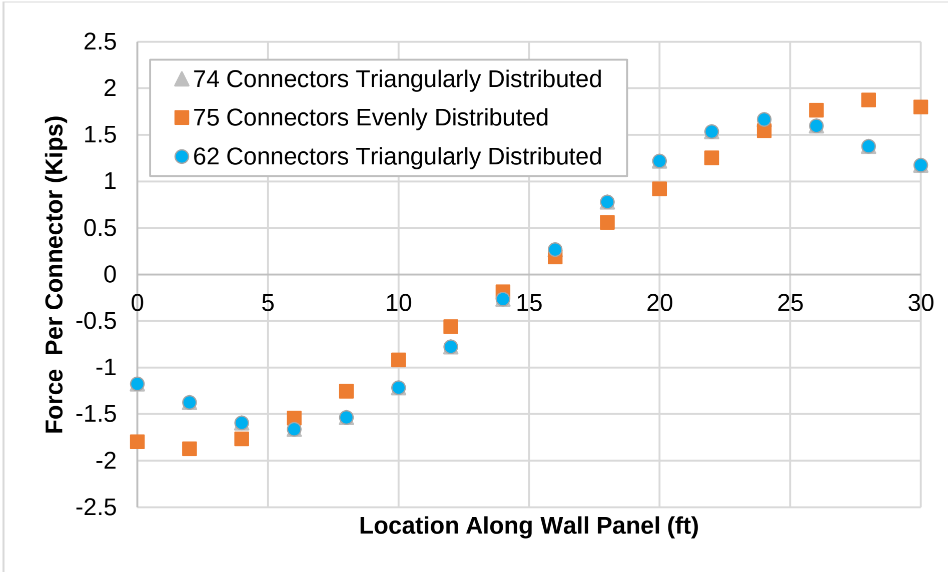

Figure 11 – Force per Connector for Different Connector Distributions

Currently, when a designer uses commercial wall panel software, they are asked to input a degree of composite action (in percent) for evaluation of cracking, elastic deflections and ultimate load. Most connector systems are considered to have a standard degree of composite action for each design limit state, but this is not necessarily the case. Figure 8 presents the same panel as described above, with varying levels of uniformly distributed shear connectors, as the number of shear connectors increases, the panel becomes stiffer and approaches the fully composite line. This implies that using additional connectors of the same stiffness will provide different levels of composite action. These results indicate that the degree of composite action for a given system, deflections, cracking and even ultimate is not a single number and is directly related to the stiffness provided by the shear connectors. Adding more connectors, or redistributing connectors towards the panel ends, as described above, will present an apparent increase in composite behavior, regardless of the manufacturer's connector system. There is likely a practical limit to the amount of composite action available to a given system due to differences in strength, stiffness and the total number of connectors that can practically be fabricated in a wall panel for a given system.

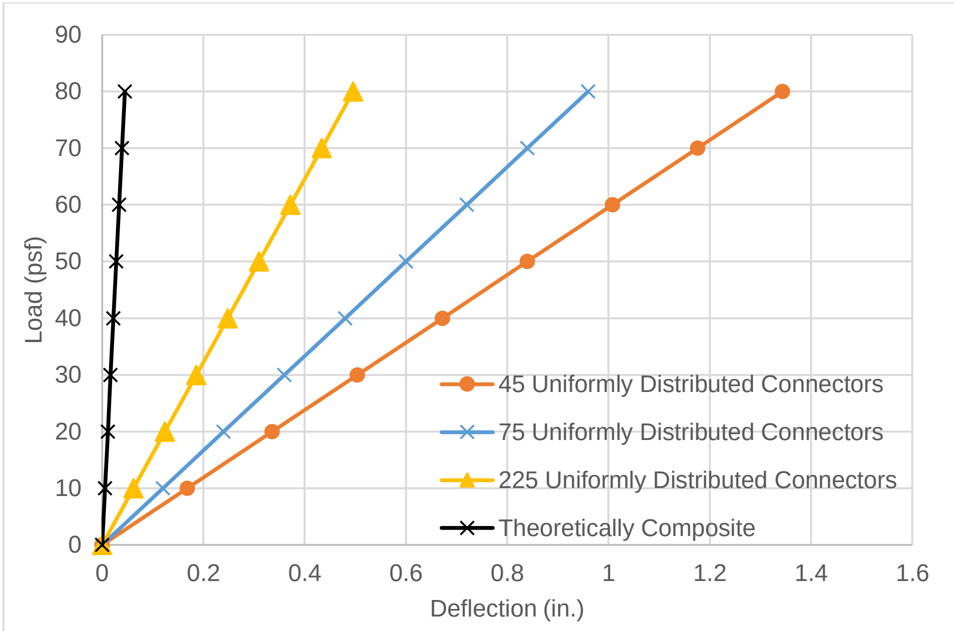

Figure 8- Example Elastic Load versus Deformation Relationship for Various levels of Shear Connector Intensities

CONCLUSIONS

The above paper describes experiments on various composite sandwich wall panel connector systems in pure shear using push-off specimens. Each specimen contained several FRP connectors and the variables studied were wythe thickness, insulation type and insulation bond. In general, connectors provided less strength and stiffness with larger wythe thicknesses and when debonded. Stiffness and strength were found to be unrelated and likely due more to the orientation of the connectors.

A simplified beam spring model was created which uses beams to represent the concrete wythes and shear springs to model the shear deformation behavior created by the foam and shear connectors. This model was found to be accurate as compared to results from the literature. A short parametric study was performed using the beam spring model to investigate the effects of connector strength, pattern and intensity. It was found a triangular distribution of shear connectors – more lumped near the ends – is more structurally efficient. Additionally, composite action was shown to increase with the increase of shear connectors, rather than be a single set value for a given shear connector system as has been used in the past for design. Further validation of this model is required, but it shows considerable versatility and promise as a design tool.

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