MATRIX MODEL ACCURACY OF PARTIALLY COMPOSITE CONCRETE SANDWICH PANELS

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ABSTRACT

Concrete sandwich wall panels have been used for decades in the precast concrete construction industry because of their thermal efficiency. Predicting concrete sandwich panel elastic stresses and deformations is paramount for design in order to prevent cracking and to limit second order effects. This paper addresses the accuracy of a matrix analysis approach that is a generalized variation of the current standard practice for most sandwich panel composite connector manufacturers. A generalized modeling approach is presented that eliminates dependence on the connector type and geometry, and is termed the Beam Spring Model (BSM). The BSM uses only beam and spring elements to predict the elastic limit of the panel, but can capture effects such as panel geometry, boundary conditions, and loads. The stiffness of the spring elements is obtained from push-off tests that are available in the literature, or in most cases directly from a composite connector manufacturer. To verify its accuracy, the BSM was used to model six full-scale sandwich wall panels that were tested at Utah State University, which consisted of various connectors, connector patterns, geometry, and insulation. A parametric study was performed using different parameters to better understand partial composite action in sandwich wall panels and different design situations.

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

INTRODUCTION

Concrete sandwich wall panels (SWPs) are increasing in popularity due to their thermal and structural efficiency and an increasing demand in society for energy-efficient buildings. In nearly all cases, a sandwich panel consists of three layers: two concrete layers (known as wythes) with a layer of insulation in between. SWPs are designed to act non-composite, fully-composite, or partially composite, depending on the shear connector design used to transfer the shear force between the concrete layers. Although steel connectors have historically been quite common, fiber reinforced polymer (FRP) connectors have become more common due to their significantly superior thermal efficiency. Figure 1 shows a thermal image of two sandwich panel buildings, one with steel connectors (left) and one with FRP connectors (right). See Sorensen et al.[1] for additional heat loss in sandwich panels discussion.

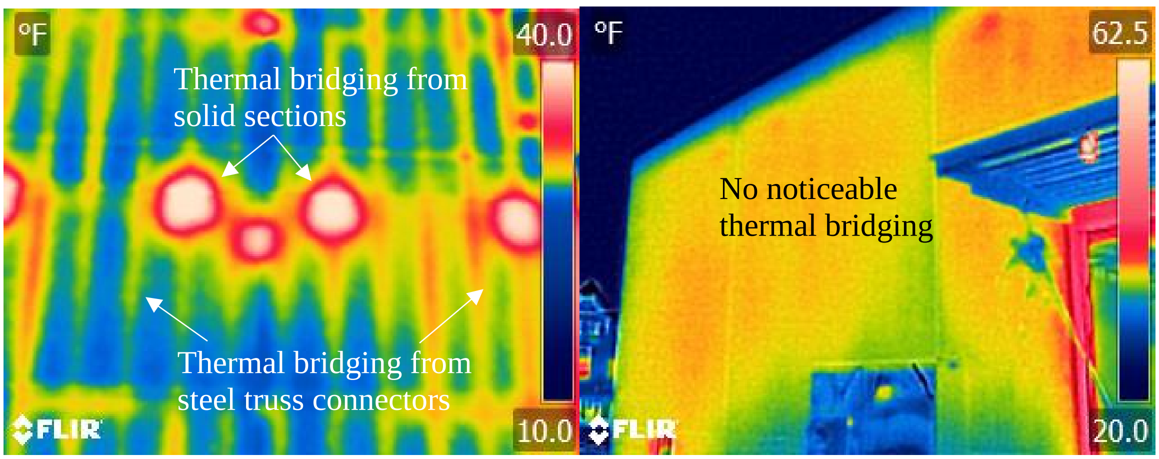

Figure 1 Thermal Images of SWP (left) with steel connectors (right) using composite connectors.

PREDICTING STRESSES IN THE LITERATURE

Predicting concrete sandwich panel elastic stresses and deformations is paramount for design. Doing so prevents cracking and limits second order effects. Several researchers have developed techniques to predict sandwich panel deformations. Prediction methods vary significantly in complexity and accuracy.

Hassan and Rizkalla modified Newmark et al.'s approach to composite steel beams to be suitable to predict the flexural behavior of partially composite concrete sandwich panels.[2,3] Hassan and Rizkalla's method focuses on concrete sandwich panels reinforced with continuous Carbon FRP grid connectors. Naito et al. found that connector stiffness affects flexural sandwich panel behavior.[4] In addition, connector stiffness greatly affects behavior after the sandwich panel has cracked. Naito et al. proposed a numerical method to estimate SPW behavior under uniform static load by using the degree of composite action and moment curvature. Tomlinson used a numerical model similar to the one proposed by Naito et al. [4,5] However, Tomlinson used an analytical approach to estimate shear in the connector and the foam. In addition, the Tomlinson model was more complicated and involved integrating the strain in the panel in order to estimate slip.

Teixeira, Tomlinson, and Fam used a two-dimensional finite element program that consists of two parts, a beam element and a link element, to predict the flexural behavior of a partially composite sandwich panel.[7] Their model accounts for nonlinear behavior of the materials. The model's results were promising, however, it is highly variable in predicting the ultimate load.

Olsen and Maguire introduced the beam spring model (BSM) using a commercial matrix analysis program to predict the elastic behavior of SWPs with various concrete strengths and shear distributions.[6] The motivation for creating this model was that several shear connector manufacturer companies use variations of this model (i.e. normal and Vierendeel trusses) to predict percent composite action for the situation, which is then given to the engineer. In some cases, this has been somewhat of a "black box" process and highly connector specific, which has made engineers uncomfortable. This easy to construct model will allow the precast engineer, rather than the connector manufacturer to determine percent composite action for a given panel. Beam elements were used to represent the concrete wythes and spring elements were used to emulate the discrete shear connectors with the stiffness attained from double shear push-off tests. Olsen and Maguire found that the model was highly accurate when predicting cracking and elastic deflections.

Building upon Olsen and Maguire, the purpose of this paper is to evaluate the beam spring model with respect to elastic behavior of a partially composite, reinforced, pre-stressed sandwich panel and how model results compare to experimental results. A parametric study was also performed to explore the effect of the shear connectors' distribution on elastic composite action in terms of cracking and deflection.

BEAM SPRING MODEL

The BSM was created using a commercial matrix analysis software package, which is a more general variation of what many connector manufacturers currently use. This model could easily be replicated using any commercial or personally written matrix analysis software and could easily be implemented in commercial wall panel analysis and design software. This model was shown to work for any connector type. This approach modeled the SWP using only beam and spring elements (Figure 2) combined with the appropriate material values, boundary conditions, and shear connector stiffness from push-off specimens. Other research programs[7,8] have described similar methods using matrix software. Surprisingly, this analysis concept has been around for decades when analyzing multi-wythe masonry.[9] Many connector manufacturers use a truss analysis with matrix software, usually a Vierendeel truss, but some angled connectors, like those in the A-series panels below use angled truss elements.[10] The purpose of developing a simple model that relies on only spring and beam elements is to be able to model a panel with any connector type repetitively, with minor change between analyses. Another purpose is that this type of model relies only on shear testing data, which some connector companies already have from ICC-ES acceptance criteria, specifically ICC-ES AC320[11] and ASTM E488-96[12] or other in-house testing.

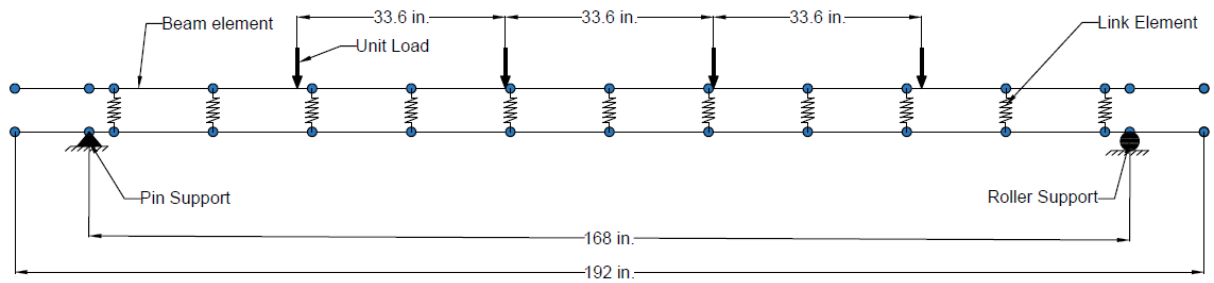

Figure 2 Example of a Full-scale specimen modeled using the Beam Spring Model

The proposed two-dimensional model consists of two beam elements with cross-sectional areas and moments of inertia equal to the gross cross sectional areas of their representative wythes. Each beam element can be assigned the properties of their representative wythe and separated by a distance equal to the distance between the centroids of the wythes. Spring elements are assigned both shear and axial stiffness corresponding to the actual stiffness of the connectors as measured by the connector manufacturer or reported in the literature.[4,5,6] The shear and axial spring elements are placed at the locations of the discrete connectors or are lumped at regular intervals for continuous connectors (not discussed in this paper) and are then used to model the transfer of force between wythes. For the purposes of modeling the SWPs in this paper, support conditions are modeled as pin (translation fixed, rotation free) and roller (longitudinal translation free, transverse translation fixed, rotation free). However, any support condition can be modeled and should be placed at the appropriate locations on the panel. Similarly, any loading can be applied, but for the purposes of the experimental program a combination of distributed load for dead load and four point loads, mimicking the experimental loading, are applied (see Figure 2)

EXPERIMENT SETUP

Full-scale panels were tested at Utah State University to verify the efficacy of the BSM. Six full-scale panels were tested with various commercial connectors, configurations and panel geometry. A-series panels used NU-Tie connectors, BC-series panels used Thermomass X connectors and D-series panels used HK CA Ties for shear connectors, all shown in Figure 3. All panels used Extruded Polystyrene (XPS) insulation. The panels were intentionally varied (e.g. prestressed versus reinforced and a high versus low number of connectors). Panels A-2 and A-4 had two equal 3 in. concrete wythes and a 4 in. insulating layer (often called a 3-4-3 panel) configuration and used prestressed reinforcement in the longitudinal direction and shear connectors. Panels BC, and D were a 4-3-4 configuration and reinforced with four

Grade 60 #3 steel bars running in the longitudinal direction and centered in each wythe and three shear connectors in each row. Detailed SWP drawings including shear connector locations are shown in Figure 5.

Figure 3 Shear connectors used in this study, NU-Tie, HK CA, Thermomass CC, and Thermomass X (left to right).

The panels were placed on simulated pin and roller supports. The A-2 and A-4 panels were 16 ft long with a 15 ft span and the BC, and D panels were 16 ft long with a 14 ft span. Four point loads were used to simulate distributed loads by using a spreader beam assembly and a single hydraulic jack with a load cell, as shown in Figure 4. The deflection on both edges of the panels were measured as well as the relative slip between the concrete wythes in each corner of the panel. Dead load deflection was measured using a total station and high accuracy steel ruler prior to applying the external load.

Figure 4 Test Setup

Concrete cylinders were tested according to ASTM C39[13] for compression strength, and the modulus of rupture and elasticity were estimated based on ACI 318-14[14] from the compression strength. Mild steel and prestressing steel were also tested according to ASTM A370[15] with slight modification,[12,13] but are presented elsewhere[10] because steel strength information is not relevant to the elastic behavior of the SWPs discussed in this paper. Table 1 summarizes the panel details, including concrete testing results and connector stiffnesses, as tested in Olsen et al.[20]

Table 1 Panel details and connectors properties

Panel	**Width**	**Configuration**	**Span**	**Concrete Modulus of Elasticity**	**Concrete Tensile Strength**	**Connector Stiffness (K_E)**
	in.	*in.*	*in.*	*psi*	*psi*	*kips/in*
A-2	48	3-4-3	180	6,191,000	766	118
A-4	48	3-4-3	180	6,191,000	766	118
BC-1	36	4-3-4	168	5,824,000	691	17.9[b]
						205[c]
BC-2	36	4-3-4	168	5,986,000	699	17.9[b]
						205[c]
D-1	36	4-3-4	168	5,824,000	691	94.8
D-2	36	4-3-4	168	5,986,000	699	94.8

[b] – connector B

[c] – connector C

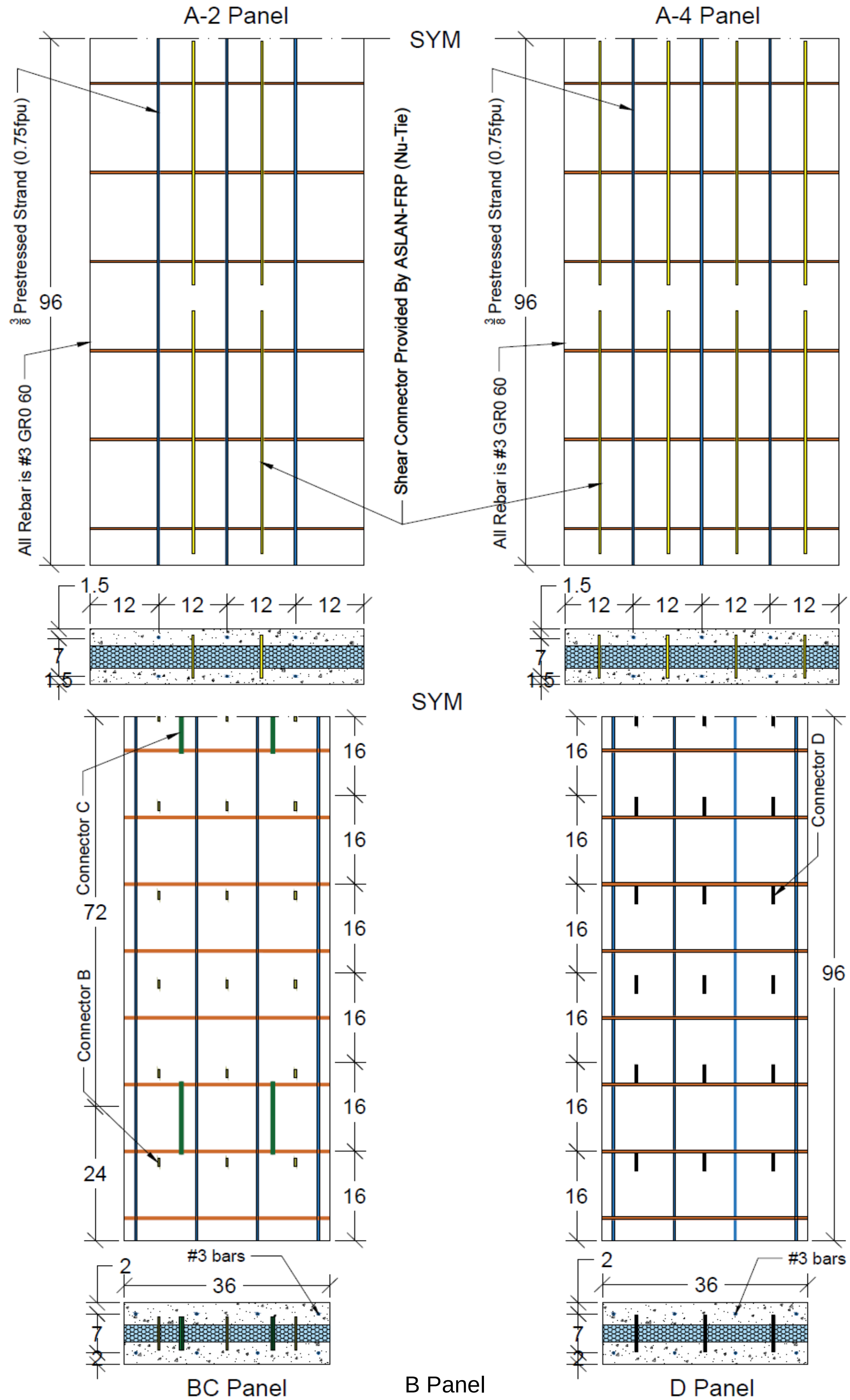

Figure 5 Half symmetry Panel details.

VALIDATION OF THE BEAM-SPRING MODEL

To verify that the BSM is accurate, each test specimen was modeled as described above and the elastic deflections and stresses of the models were compared to the test results. Because each test specimen had a different connector configuration and spacing, links connecting the beam elements were placed at locations corresponding to each of the shear connectors. The values of shear stiffness, K_E, used in each model are shown in Table 1. The shear stiffness selected were the reported bonded values because the panels were tested early in their lifetime and were never severely loaded and the bond was considered intact.

The model included four point loads, which were applied to the top face of the model. This was done to imitate the full-scale testing. In addition, self-weight was added to the total load. Links were also assigned a longitudinal stiffness based on the tributary geometry and the assumption that the elastic modulus was equal to that of XPS insulation (670 psi).[21] Axial stiffness values for the connectors were not readily available in the literature, but since the model was not sensitive to this value it was not a concern. With this model, the deformations and deflections were easily predicted along with axial forces and bending moments in the concrete wythes, which were then resolved into stresses.

The BSM was used to make predictions of cracking moment, deflection, and slip of the eight full-scale test panels. The predictions were then compared to the actual values to validate these predictions. Figure 6 presents the actual results and predicted results of the model for the full-scale A-2 sandwich panel. Note, a theoretically fully composite (FC) line and non-composite (NC) line are plotted for reference. In this figure and in the figures similar to this one that follow, the BSM load versus deflection prediction is plotted up until it reached the concrete tensile strength. In the plots, a slightly bi-linear relationship for the Beam-Spring model can be observed, which is counterintuitive for an elastic method. This is because the method was applied using the uniform load to simulate the dead load after this point. four point loads were also applied as in the full scale test, creating the slight stiffness (i.e. slope) change.

The BSM results show excellent agreement with the observed behavior. The cracking moment differs by only 0.5 percent and deflection at the cracking moment differs by only 14 percent, in the BSM. The actual slip of the A-2 panel was measured at 0.05 in., with the Beam-Spring Model predicting 0.045 in. Furthermore, in the figures below, it is easy to see the experimental load deformation plots and the slip plots become non-linear (i.e. crack) just as the beam spring model predicted.

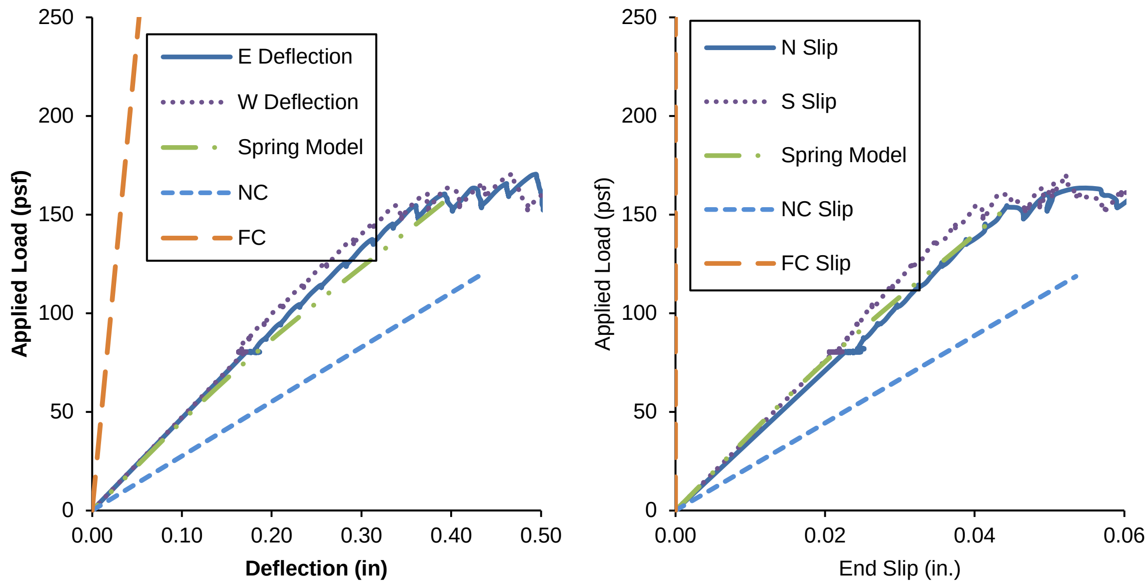

Figure 6 Load versus Deflection (left) and Load versus End Slip (right) for A-2 Panel

The Beam-Spring Model underpredicted the cracking moment of the A-4 panel by 5 percent. Figure 7 shows that the applied load at cracking was around 200 psf, which differed slightly from the Beam-Spring Model prediction. The model overpredicted the slip at cracking in this specimen by 11 percent.

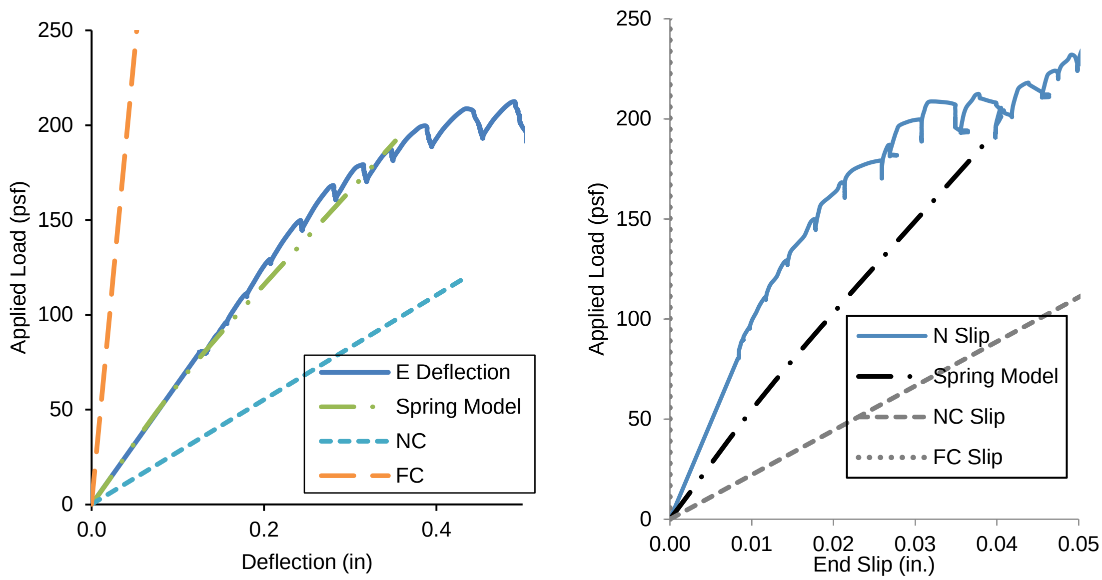

Figure 7 Load versus Deflection (left) and Load versus End Slip (right) for the A-4 Panel

The model over predicted both the cracking load and the Elastic Hand Method by 10 percent in the BC1 and BC2 panels, as shown in Figure 8 and Figure 9. The model under predicted slip for the BC specimens by 55 percent on average.

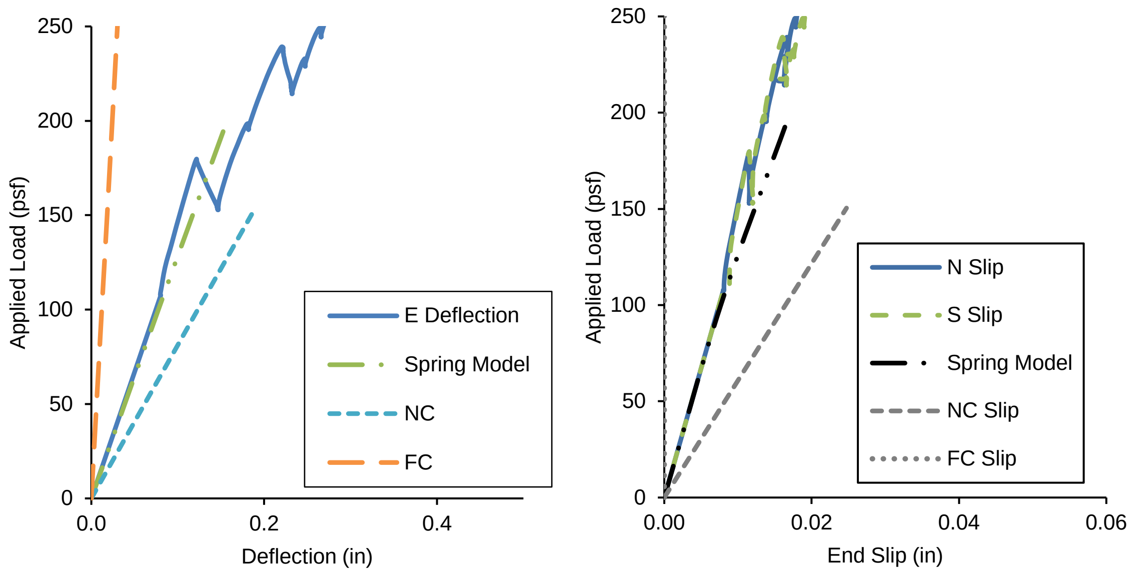

Figure 8 Load versus Deflection (left) and Load versus End Slip (right) for the BC-1 Panel

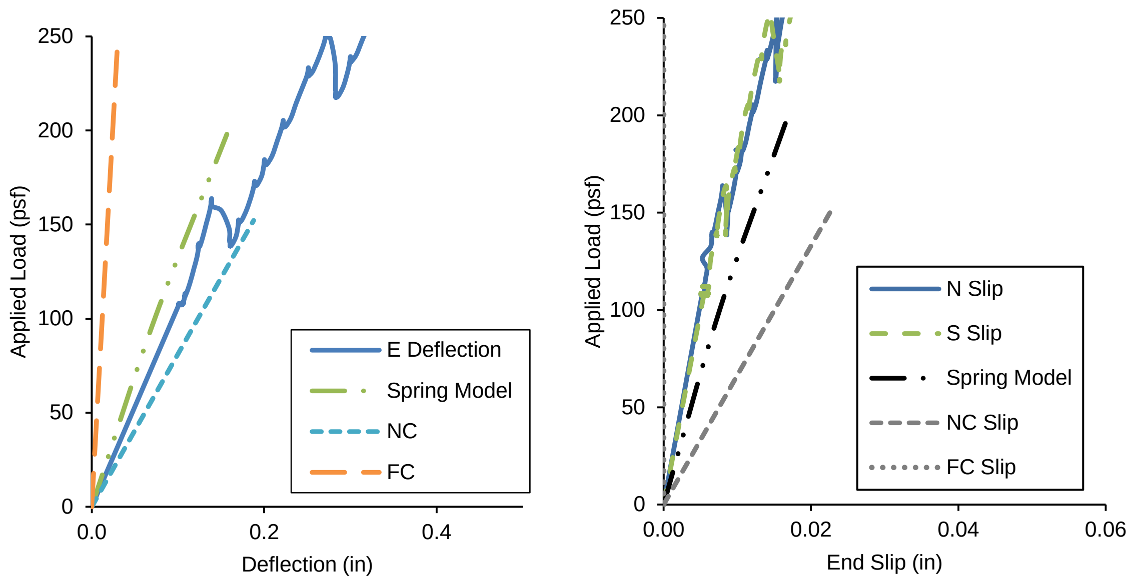

Figure 9 Load versus Deflection (left) and Load versus End Slip (right) for the BC-2 Panel

Figure 10 and Figure 11 display the measured and predicted elastic behavior for the D-1 and D-2 specimens. The cracking load predicted by the BSM matched the average result of the full-scale D-series specimens. The BSM under predicted the slip by 15 percent on average.

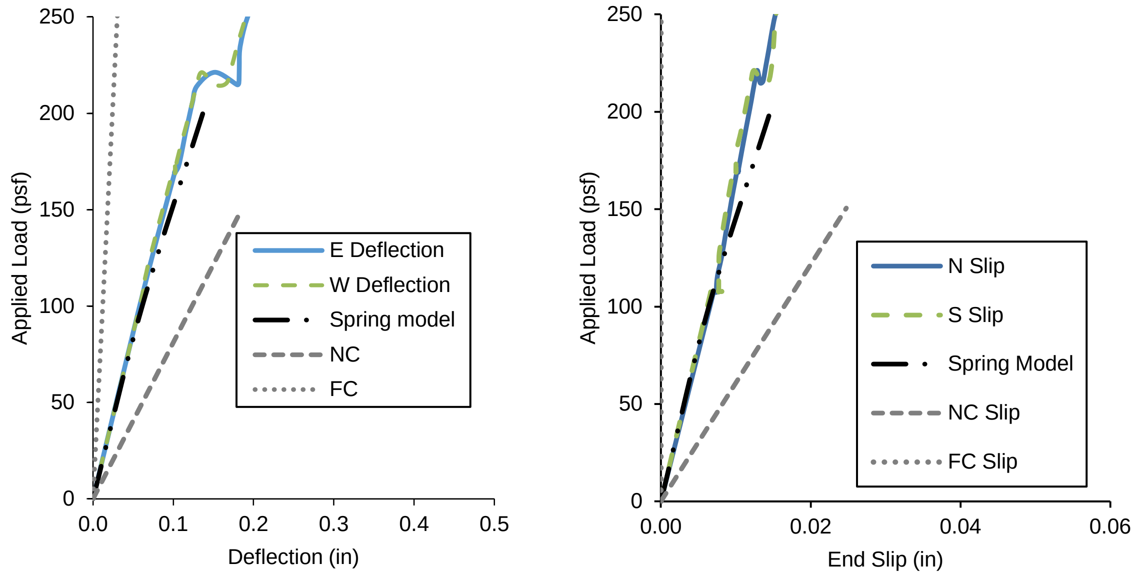

Figure 10 Load versus Deflection (left) and Load versus End Slip (right) for the D-1 Panel

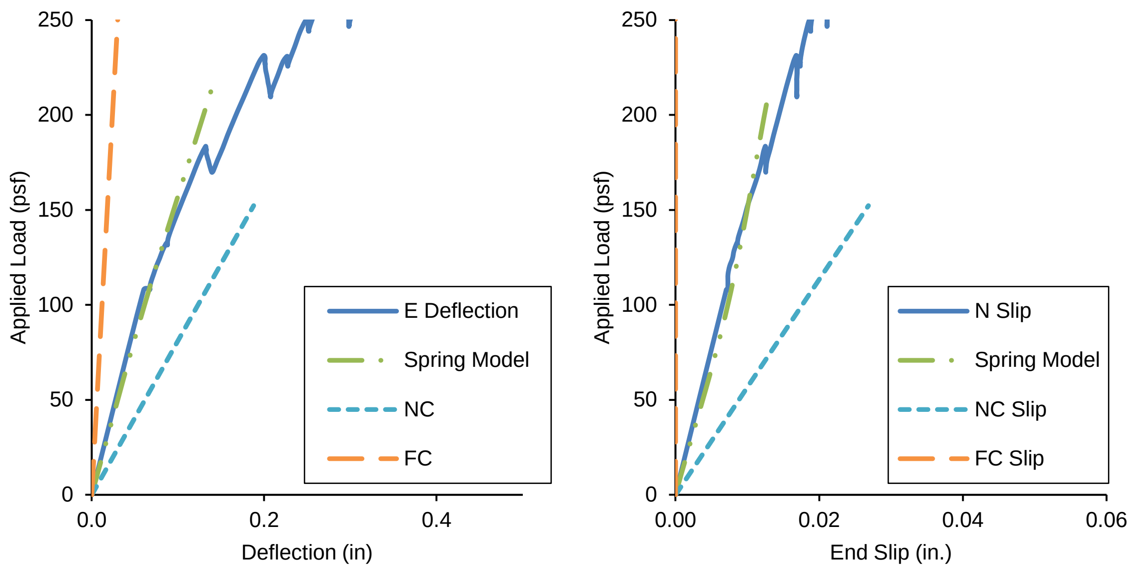

Figure 11 Load versus Deflection (left) and Load versus End Slip (right) for the D-2 Panel

The measured cracking load and deflection at cracking for each full-scale test and the BSM are presented in Table 2. The BSM was very accurate except in the case of the D-2 and BC-2 specimens. The reason for these inaccuracies is unclear, but may be due to measurement error. However, it might be because of the split tension test BC-2 and D-2 showed highly variable results (572 psi, 468 psi, 1057 psi for the individual cylinders) which was on average 699 psi. That may also explain the over-prediction of the cracking load of the beam-spring model for BC-2 and D-2 panels because the split of tension 699 psi was used. If the average of the other two cylinders is used (520 psi) the prediction becomes 162 psf and 164 psf for the cracking load for BC-2 and D-2 panels, respectively, resulting in a much better prediction. Table 3 contains the measured-to-predicted ratios for the BSM. As shown in the table, the average predictions are very good: 0.95 for BSM cracking predictions and 0.97 for deflection at cracking. These accuracies are similar to those of other analysis methods for structures like reinforced and prestressed concrete beams and steel members.[17] If the BC-2 and D-2 panels are not included, the measured-to-predicted ratios are nearly 1.0. End slips were predicted relatively well, with exception of the BC-series panels. The reason for the poor predictions on these panels is unknown, but could have to do with the mixing of connector types, built in modeling assumptions, or testing error and deserves additional study.

Table 2 Summary of measured and predicted cracking and deflections

Panel	**Measured**		**Beam-Spring Model**	
	Cracking Load	**Deflection**	**Cracking Load**	**Deflection**
	(psf)	*(in)*	*(psf)*	*(in)*
A-2	155	0.34	156	0.39
A-4	202	0.44	192	0.352
BC-1	180	0.12	198	0.155
BC-2	164	0.15	197	0.157
D-1	221	0.14	209	0.144
D-2	184	0.13	208	0.138

Table 3 Beam-Spring Model Measured-to-Predicted Ratios

Panel	**Cracking Load**	**Deflection**
A-2	0.99	0.87
A-4	1.05	1.25
BC-1	0.91	0.79
BC-2	0.83	0.96
D-1	1.06	1.00
D-2	0.88	0.96
Average	**0.95**	**0.97**

DEGREE OF COMPOSITE ACTION FOR ELASTIC PROPERTIES

Utilizing the theoretical fully-composite load, theoretical non-composite load, and the actual predicted load from BSM, the degree of composite action can be calculated for the cracking load and deflection for different panels.

The degree of composite action with respect to the cracking load can be found using the following equation:

$$K_{Lcr} = \frac{L_{cr,BSM} - L_{cr,NC}}{L_{cr,FC} - L_{cr,NC}} \tag{1}$$

Where Lcr_{BSM} = cracking load of the sandwich panel from BSM results
Lcr_{NC} = theoretical cracking load of the non-composite sandwich panel
Lcr_{FC} = theoretical cracking load of the fully composite sandwich panel

The degree of composite action with respect to deflection can be found using the following equation:

$$K_d = \frac{I_{BSM} - I_{NC}}{I_{FC} - I_{NC}} \qquad (2)$$

Where I_{BSM} = equivalent moment of inertia the sandwich panel from BSM results
I_{NC} = theoretical moment of inertia of the non-composite sandwich panel
I_{FC} = theoretical moment of inertia of the fully composite sandwich panel

Figure 12 graphically demonstrates the degree of composite action shown in Eq. (1) and (2). Percent composite action is often used as a design parameter in partially composite SWP design. However, this term is a very design dependant and often convenient analogy, but may give engineers a misguided view of how SWP behave. Typically, a SWP design will require the use of three different percent composite actions for elastic deflection, first crack and nominal strength. The parametric study below examines percent composite action for different connector design scenarios.

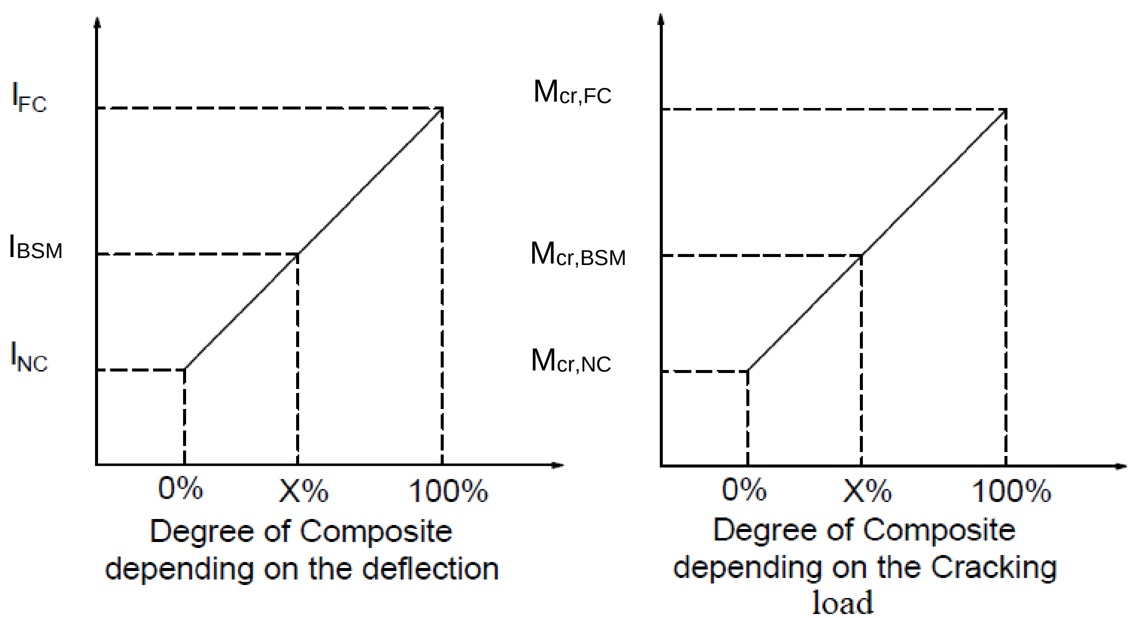

Figure 12 Visual demonstration of degree of composite action

PARAMETRIC STUDY

Two cases were investigated to examine the behavior of concrete sandwich panels. The use of combined connectors in sandwich panels was investigated in Case 1. The second case, Case 2, investigated how composite action was affected by the use of different distributions of connectors. The prototype panel for this investigation has a length of 24-ft and a width 4-ft with a 3-3-3 wythe configuration using XPS insulation.

PARAMETRIC STUDY: CASE 1

Case 1 compared composite action across four sandwich panel models, Case 1-1 through Case 1-4, presented as quarter symmetric drawings in Figure 13. Model Case 1-1 had a total of 54 type B connectors spaced at 16 inches. Model Case 1-2 had 12 type C shear connectors, concentrated in the first 40 in. of the panel. Model Case 1-3 is a combination of the shear connector from Case 1-1 and Case 1-2 and has 54 type B connectors and 12 type C connectors, oriented as shown in Figure 13. Model Case 1-4 has 18 type C connectors, i.e., one more column than Case 1-2. Note that these connector configurations, especially Case 1-2 and Case 1-4, are not likely a suitable design for delamination failure, but are merely used for demonstration.

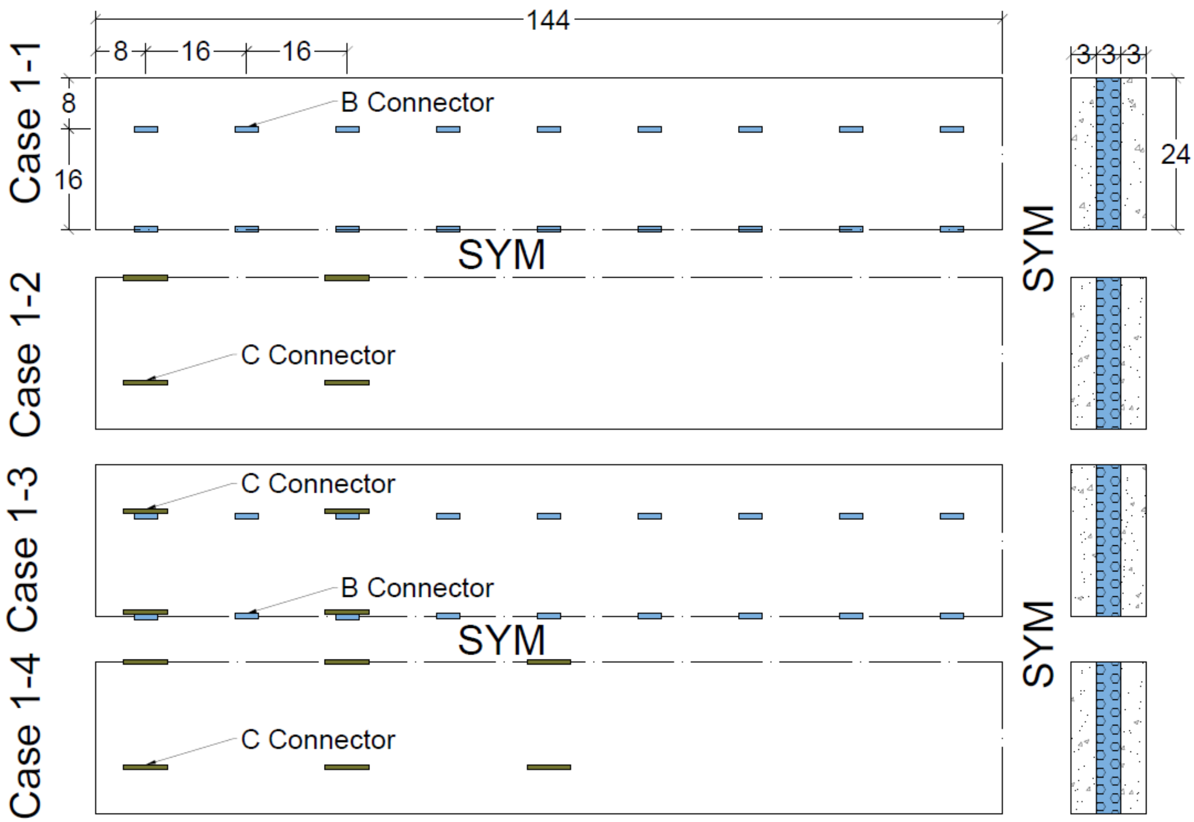

Figure 13 Quarter symmetry of models that used parametric study in Case 1 (All units are inches)

Figure 14 plots the connector forces at cracking versus location from the end of the panel for a half-length of panel. Comparing Case 1-1 to Case 1-2, the connectors have very different forces due to their different stiffness. When the two patterns are combined in Case 1-3, forces in the B connectors decrease by approximately 50 percent and the C connectors have nearly the same shear force. This occurs because the vastly different stiffness between the B and C connectors. Case 1-4 indicates that adding additional columns of connectors to Case 1-2 will result in additional forces imparted to the panel; however, as will be shown below, the effect on the percent composite action may not be significant.

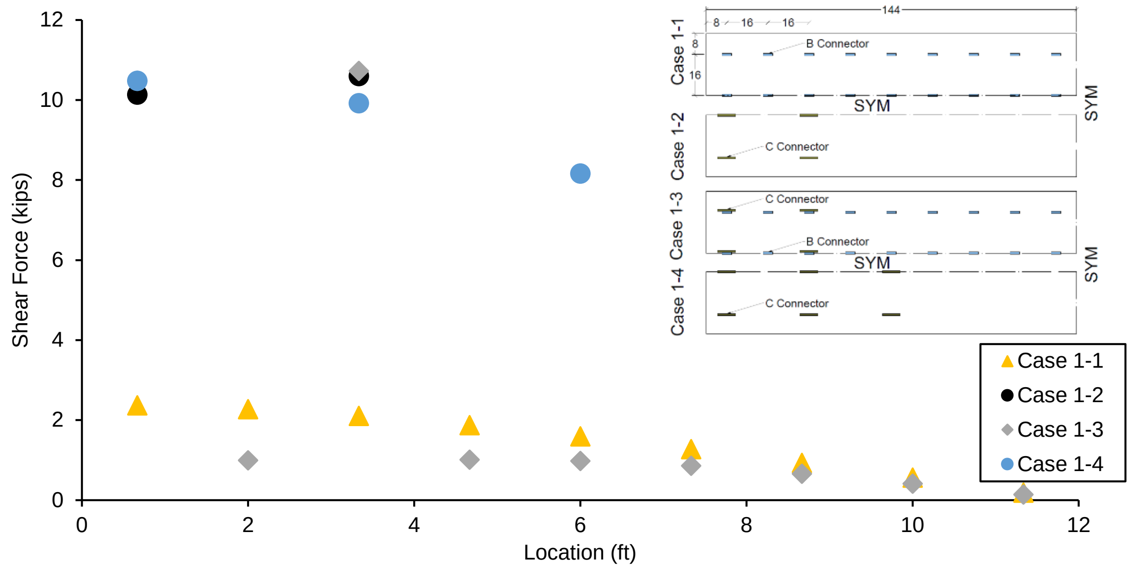

Figure 14 Case 1 Connector Force vs Location (half length)

The composite action of the Case 1 models are presented in Figure 15 and were calculated using Eq. (1) and Eq. (2). Interestingly, when comparing Case 1-1 to Case 1-2, Case 1-2 had over double the deflection percent composite action, but only 60 percent more for cracking. This illustrates the well-known fact that even though both behaviors are elastic, they often have different percent composite actions. Additionally, concentrating the connectors at the ends of the member allows them to be used more effectively. However, this effect is not directly additive to the percent composite action, as illustrated by Case 1-3 and Case 1-4. Case 1-3 shows that the percent composite behaviors from Case 1-1 and Case 1-2 are not added together, indicating as more connectors are added there are diminishing returns and they are not directly additive. Case 1-4 shows that adding an additional row to Case 1-2 does not result in a large jump in percent composite action, only adding 3 percent for both deflections and cracking. However, this knowledge may allow certain designs to become successful with only the purchase of a small number of additional connectors per panel, rather than needing to add larger wythes or stronger concrete.

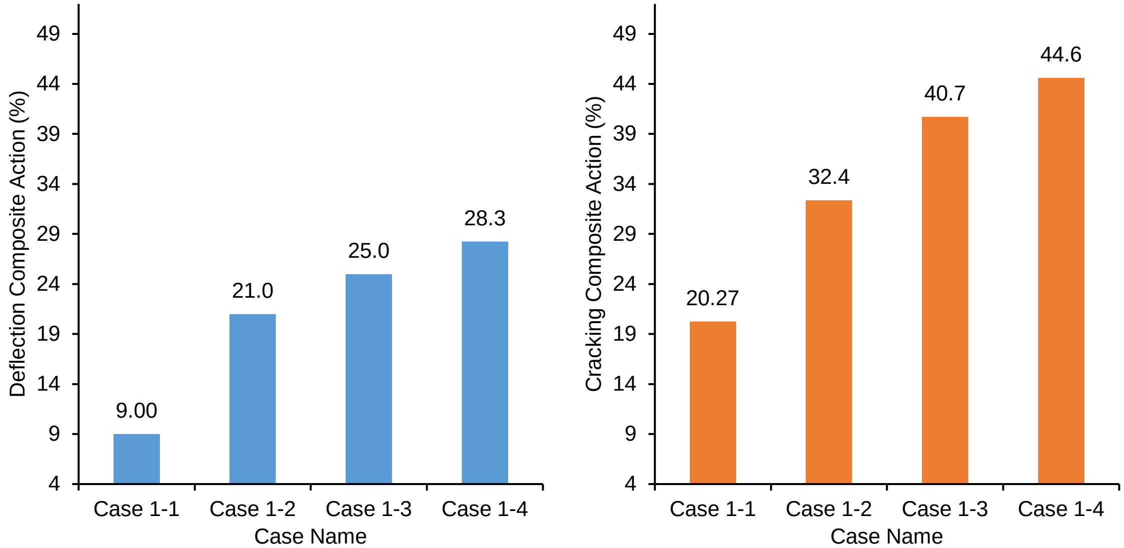

Figure 15 Composite action per case for (a) elastic deflection (b) first cracking

PARAMETRIC STUDY: CASE 2

In Case 2, a total of eight models were used. Two basic connector patterns, Case 2-U with uniformly distributed connectors and Case 2-T, with a linear distribution of connectors, more at the ends, less in the middle (i.e., triangular), are investigated. Each of these cases were investigated for connector type A and D in addition to bond condition (B or UB). For the uniform and triangular distributions, each sandwich panel was divided into three regions as shown in Figure 16, where the number of connectors was changed. The Case 2-U model has a total of 18 type A connectors and 54 connectors with type D connectors due to their size and minimum recommended spacing differences. Comparatively, the Case 2-T has 12 type A connectors and 36 type D connectors, both exactly 1/3 fewer connectors than Case 2-U. Note that Figure 16 shows three connector columns in each region. This is true for connector D, but because of connector A's size, only one column of connectors will fit in each region. Once again, these connector patterns are intended for demonstration purposes. Case 2-T may not be an ideal candidate for a real panel design due to delamination failure. Most connector manufacturers will recommend a uniform spacing for this purpose.

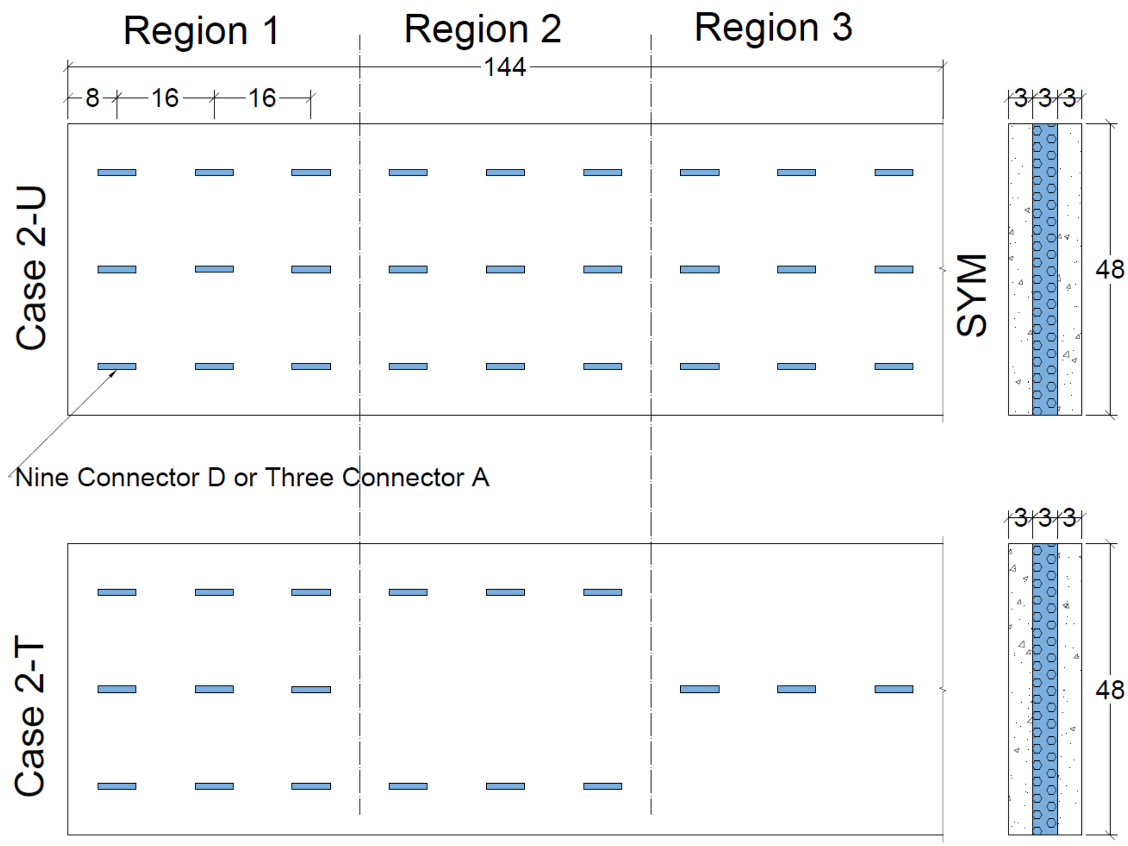

Figure 16 half symmetry of models that used parametric study in Case 2 (All units are inches)

Figure 17 and Figure 18 show the deflection and cracking composite action vs cracking load for each of the eight model permutations. The difference between the uniform connector distribution and the triangular distribution is a 33 percent reduction in connectors, but in the worst-case reduction, the cracking percent composite action was only 18 percent for the bonded D connectors. The results from the previous case study indicate that the removed 33 percent of connectors could be added back in with the triangular pattern to get a boost in percent composite action. Engineers would probably prefer to design without counting on the bond of the foam to the concrete. Neglecting this bond results in a very significant percent composite action drop, with the worst case up to 20% lower when comparing uniformly distributed connector D for deflections (compare 28.7 percent to 22.6 percent in Case 2-U, Figure 17).

The stiffness of connectors A and D are 221 k/in. (for 3 in insulation) and 95 k/in. respectively. However, since connector D is much smaller, three usually fit within the same footprint as one A connector, making the stiffness per square ft of a panel similar in the D connector panel – nearly 1.3 times higher for the uniformly spaced panels. However, even though the distributed stiffness of the D connector panel is greater (nearly 30 percent), the panels only exhibit a 20 percent increase in percent deflection composite action (compare 23.9 percent to 28.7 percent in Figure 17 and Figure 18, respectively). Note that this discussion revolves around stiffness, not strength. See Olsen and Maguire[6] and Al-Rubaye et al.[16] for additional connector strength and stiffness discussion.

Selection and use of an individual connector is dependent upon more than strength, stiffness and percent composite. Functionality, fabrication costs, customer service and many other factors should play a role in selecting a composite connector. The results in this paper do not constitute an endorsement, grading, comparison or judgment of different connectors or manufacturers and only focus on one limited aspect of the mechanical behavior.

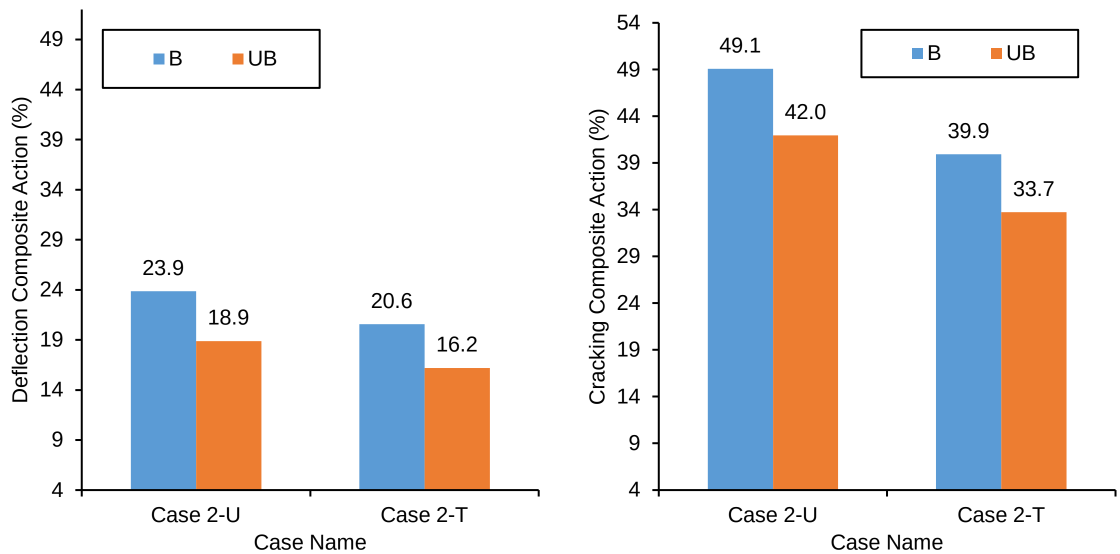

Figure 17 Connector A (right) Deflection composite action vs Load (left) Cracking composite action

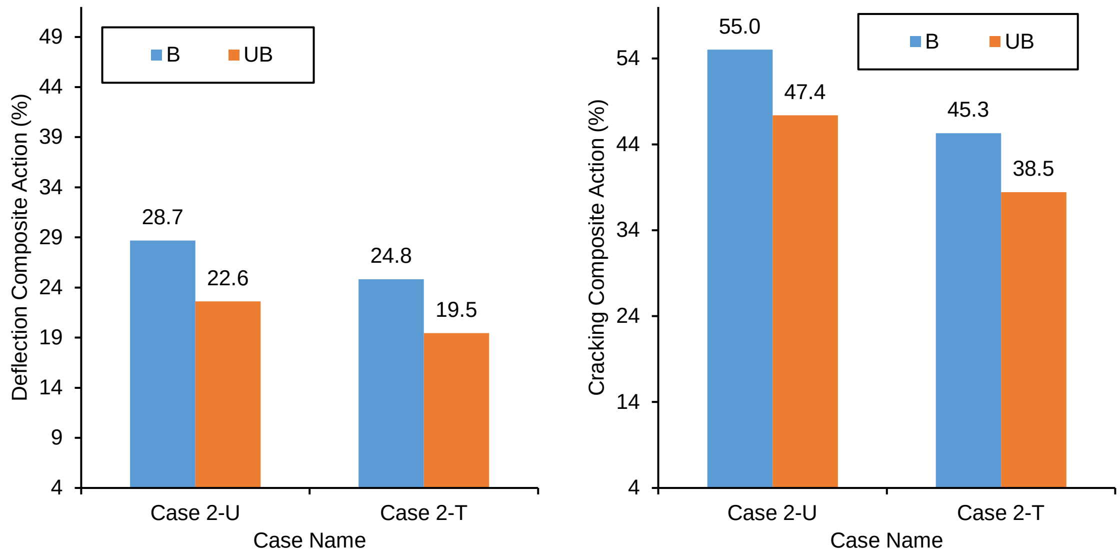

Figure 18 Connector D (right) Deflection composite action vs Load (left) Cracking composite action

CONCLUSIONS

The BSM is a simple, general, and versatile matrix analysis framework that allows for accurate prediction of sandwich panel behavior. The BSM was used to predict elastic deformations and cracking. The BSM is limited to elastic behavior, although if inelasticity (non-linear springs and beam elements) were introduced, it is likely that ultimate deflections and ultimate strength could be determined.

The elastic BSM predictions were compared to the elastic portions of the full-scale tests. The BSM had measured-to-predicted ratios of 0.97 for cracking load and 0.96 for deflections. Additional validation on more varied panels should be performed, but the results are very promising. The Beam-Spring Model is a promising option for elastic analysis of precast concrete sandwich panel walls using composite shear connector systems that can be used in a very general manner and to understand various configurations of SWPs.

A parametric study was performed to investigate the effects of using a combination of connectors and different patterns of connectors on the BSM percent composite action. The study demonstrated that panels with combined connectors have a significant difference in force distribution that can result in less contribution of connectors with lower stiffness. Additionally, grouping shear connectors near the support can increase deflection and cracking composite action or can allow connectors to be used more efficiently. Finally, using more connectors in a panel can increase the percent composite action, but there are diminishing returns with adding more connector stiffness.

ACKNOWLEDGEMENTS

The authors are very grateful to the Precast/Prestressed Concrete Institute and the Daniel P. Jenny Fellowship that funded this work. All connectors and foam was donated by the respective companies, THiN Wall, Thermomass and HK Composites. Mark Lafferty, Maher Tadros (THiN Wall) Jordan Keith, Dave Keith (HK Composites) and Venkatesh Sheshappa (Thermomass) were very helpful. This paper is not an endorsement by the authors or Utah State University of any manufacturer or system.

Forterra Structural Precast in Salt Lake City, Utah fabricated four of the four panels and Concrete Industries in Lincoln, Nebraska provided the other two. The support from these precast producers was integral to project completion. Several undergraduate and graduate students deserve thanks for their volunteer work: Parker Syndergaard, Ethan Pickett, Hunter Buxton, Tyson Glover, Mohamed Shwani and Sattar Dorafshan. Special thanks is due to our tireless editor Hannah Young.

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