FLEXURAL BEHAVIOR OF CONTINUOUS NON-LOADBEARING INSULATED WALL PANELS

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ABSTRACT

Currently, there are no codified methods nor models to analyze multi-span insulated panels under any type of load; thus engineers analyze them differently within the industry. This paper aims to provide a better understanding of non-load bearing multi-span panel behavior by comparing simple-span to multi-span panel out-of-plane behavior while performing a parametric study employing multiple finite element models. This study found connector forces, and wythe internal forces and stresses in simple-span panels differ substantially from those in multi-span panels for panels of any given dimension both in location and magnitude, being the tensile stresses the most affected. It was concluded that an effective section approach – which is common for contemporary sandwich wall panel design – can predict tensile stress maxima and deflections if properly calibrated.

Keywords: Precast Concrete Sandwich Wall Panels, Architectural Precast Concrete, Multiple Span Sandwich Panels, Finite Element Analysis.

INTRODUCTION

There are many advantages of using precast concrete sandwich wall panels (PCSWPs) as non-load bearing elements rather than conventional solid panels. Unlike solid panels, PCSWPs are more suited for energy conservation, lighter, and provide fire and impact protection to the insulation layer[1,2]. Contemporary concrete insulated wall panels consist of one insulation layer sandwiched between two concrete wythes, with ties connecting these layers to act as compositely or non-compositely[3,4], also known as the degree of composite action (DCA). Sandwich panels have been generally studied as simple span structures throughout the literature. This type of support condition generates more significant flexural moments at midspan, which are easier to predict with modern and classical structural analysis methods, in both the elastic and inelastic range[5–7]. Unfortunately, few test programs have focused on the behavior of continuous concrete sandwich panels and their failure mechanism. In the only experimental research in this area, two-span continuous panels tend to fail at the middle support due to multi-directional compression of the connection region, and flexural cracking due to the distributed load[8]. Other researchers also investigated full scale profiled steel sandwich beams and derived equations to predict their behavior[9]. Although profiled sandwich panels are made of steel and other types of foam core, these researchers found out that continuous panels fail at the support showing a similar mechanism presented in reference 8.

In addition to the experiments developed to study the behavior of sandwich structures, multiple sandwich beam theories have been developed. The earliest attempts can be attributed to Granholm[10], who developed the nailed sandwich beam theory for timber, and Newmark, who developed a similar theory for composite steel beams[11]. Both theories are highly similar, but the latter includes elements of different materials, while the former only consider one type of material for the layers. Granholm's theory was later extended to concrete sandwich panels by Holmberg and Plem in 1965 but was not developed for wythes of different materials and thicknesses[12]. In 1969 Allen also developed a new theory for sandwich panels, but unlike the others, it incorporated the deformation of the core to the assupmtions[13]. Bai & Davidson verified these theories, and a general solution was developed for the Holmberg & Plem analytical solution[14,15]. Unfortunately, only the solutions proposed for profiled steel panels and wood beams have been expanded for a support condition other than the simply supported beam[9,16]; therefore it is critical to investigate whether the classical structural analysis solutions for stresses and deformations hold for this type of structure. In this paper, a finite element model constructed with a widely accepted framework is used to investigate simple, two-span, and three-span continuous sandwich wall panels.

PARAMETRIC STUDY

In this paper, the finite element method was used as a means of analyzing continuous sandwich panels response when subjected to uniform pressure, see reference structure in Fig. 1a. The primary objective of this study was to evaluate the variables that influence the behavior of multi-span sandwich panels. The model used in this study was the Beam-Spring Model, see Fig. 1b, variations of which are common in the industry and has been verified using full-scale test and hand methods[4,14,17–19]. This model was developed assuming that the materials employed are isotropic and linearly elastic for both wythes and the connector, the displacement and rotations are small, and the connecting medium can rotate about an axis perpendicular to the analysis plane and displace perpendicular to its vertical axis but not deform along its length. Since the shear connector can be modeled at any spacing, it is up the user to consider the additional strength that the insulation layer could provide. In this analysis, the foam participation is lumped into a connector for simplification. The parameters chosen were the length of the span (12 and 16 feet), stiffness of connectors per area (10-100 kip/in/ft^2 selected arbitrarily so as not to use any single connector type), and single, two-span and three-span continuous panels with a 4ft width. A compressive strength of 5 ksi was assumed for all analyses. All panels were subjected to a 100 plf load over their length.

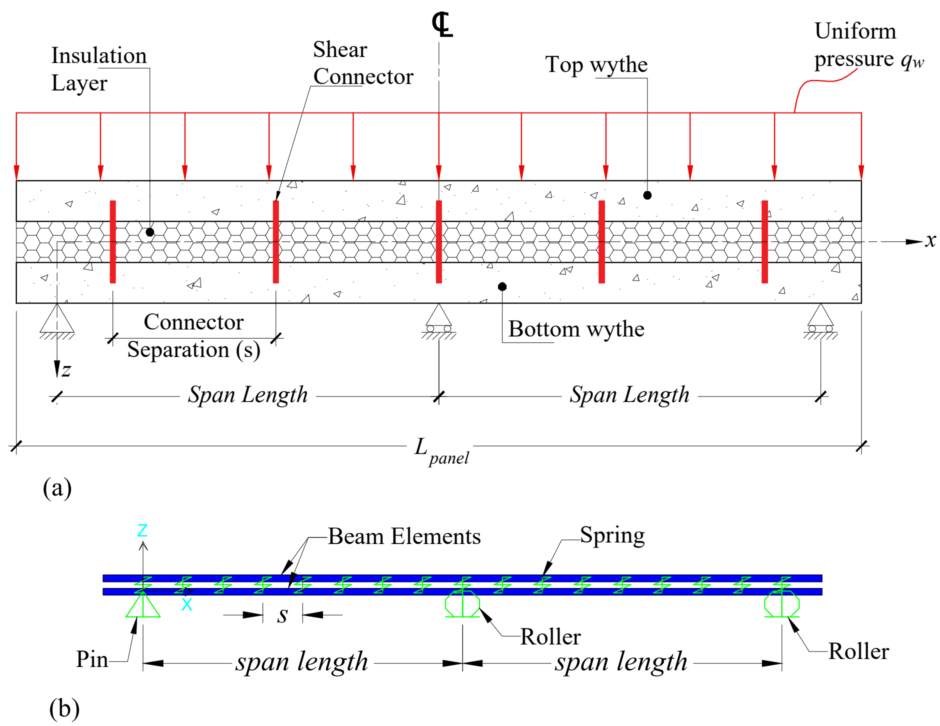

Fig. 1 Diagram of the (a) reference structure, (b), finite element model

RESULTS

Thirty-six models were analyzed linearly using a commercial finite element software; thus, all results belong to the pre-cracking behavior of the panels. A uniform load of q_w = 100 plf was applied to the top wythe (see reference structure orientation) of all panels. The panels were labeled according to the number of spans and span length; hence, a twelve-feet simple supported panel corresponds to a P1L12, where P1 stands for simple span and L12 is the span length in feet. The insulation layer and wythe thicknesses were all three inches thick for all models. The variation of panel stiffness per area for each of the panel lengths, or the number of connectors multiplied by their stiffness and divided by the area of the panel, resulted on a non-linear relationship on all variables studied; however, only deflections and internal moments decrease with an increase in panel stiffness. The deflection of the 12 ft long panels was significantly smaller than the 16 ft panel for low connector shear stiffness, or low composite action, but the difference decreased when the panels approached the 100 kip/in/ft² stiffness, see Fig. 2. Connectors are often reported as having a shear stiffness (kip/in.), the value of stiffness reported here is the amount of stiffness per square foot of panel (i.e., kip/in/ft2).

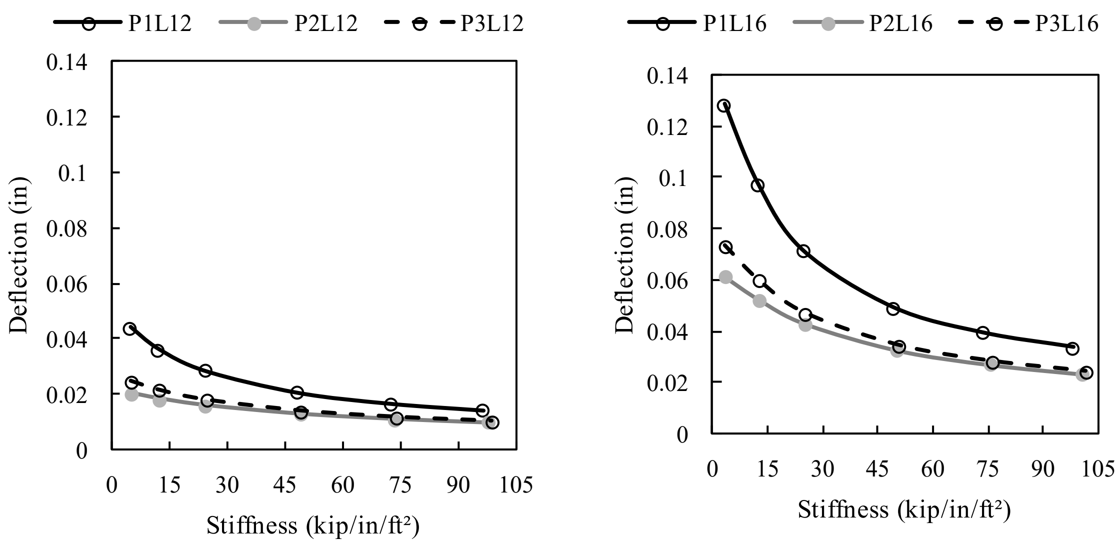

Fig. 2 Influence of panel stiffness on the deflection of 12 ft (a) and 16 ft (b) panels under a 100 plf distributed load

Conversely, the shear force in the connectors and internal axial force on the wythes increase with an increase in panel stiffness for both panel length, which is a result of the higher degree of composite action, as shown in Fig. 3. This is also accompanied by a reduction in the internal moments in both wythes for any panel length, as shown in Fig. 4 and Fig. 5. These internal moments are also affected by the number of spans used in the model. Hence, the three-span panels developed lower moments than the simple span and two span panels.

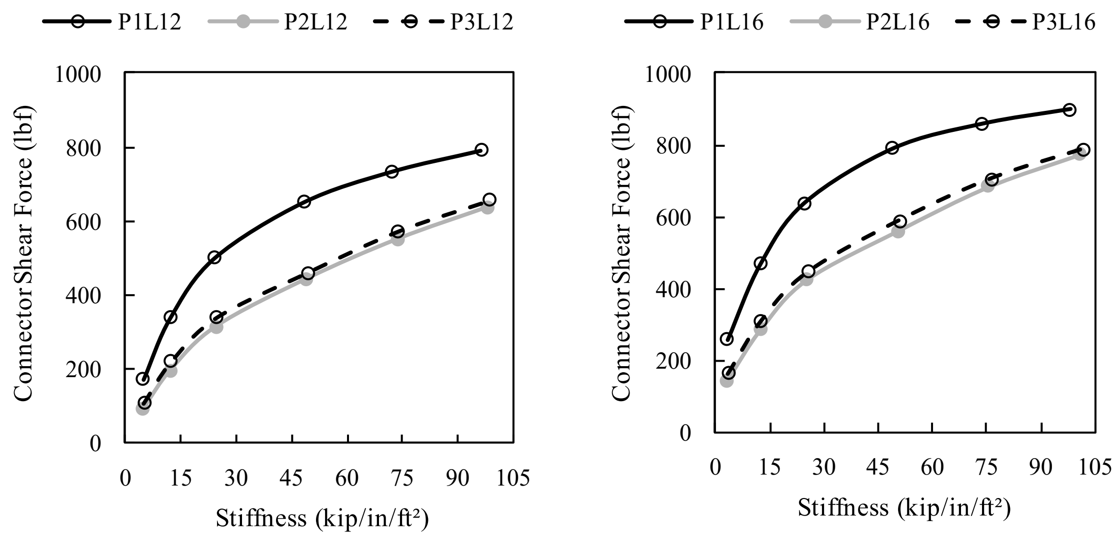

Fig. 3 Influence of panel stiffness on connector the deflection of 12 ft (a) and 16 ft (b) panels under a 100 plf distributed load

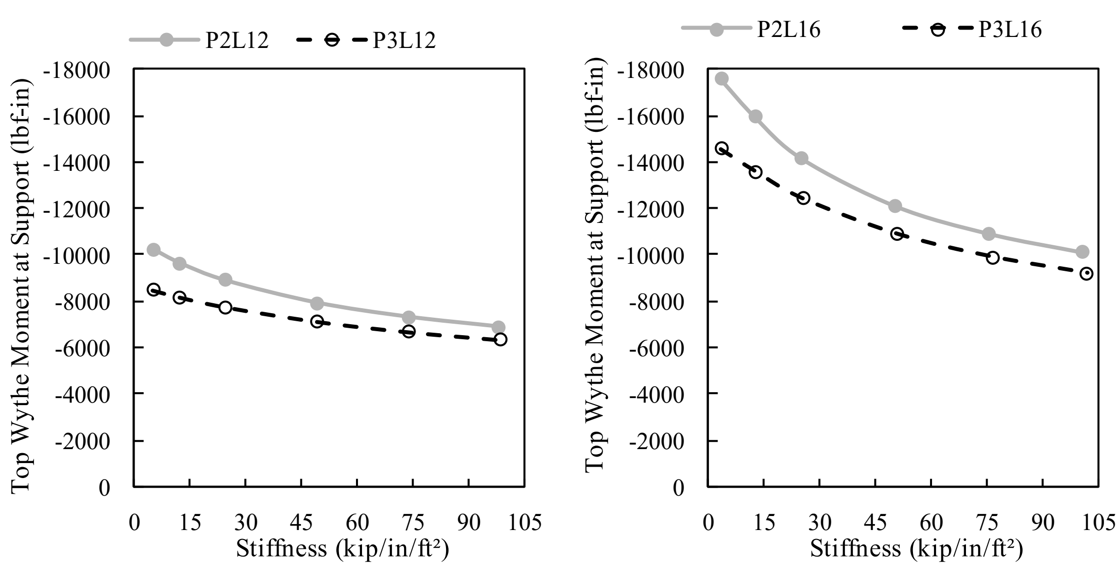

Fig. 4 Influence of panel stiffness on top wythe internal moment at the support of 12 ft (a) and 16 ft (b) panels under a 100 plf distributed load.

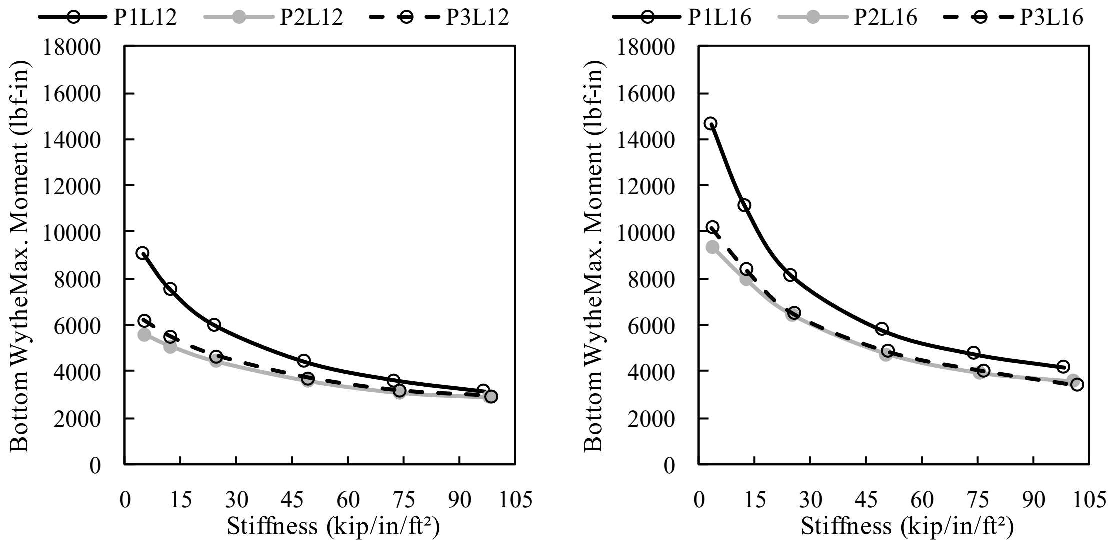

Fig. 5 Influence of panel stiffness on bottom wythe internal moment at a critical location of 12 ft (a) and 16 ft (b) panels under a 100 plf distributed load.

The internal moment variation of simply supported panels also exhibited different trends when compared to two-span and three-span continuous panels. The ratio of the maximum internal moment of a single-span panel (M1S) was compared to the maximum absolute internal moment of continuous panels (M2S and M3S) to display this behavior clearly. As Fig. 6 shows, the internal moment for simple support can be up to 50% smaller than the one at the support of a continuous panel regardless of which length is being analyzed. All results are displayed in Table 1 and Table 2.

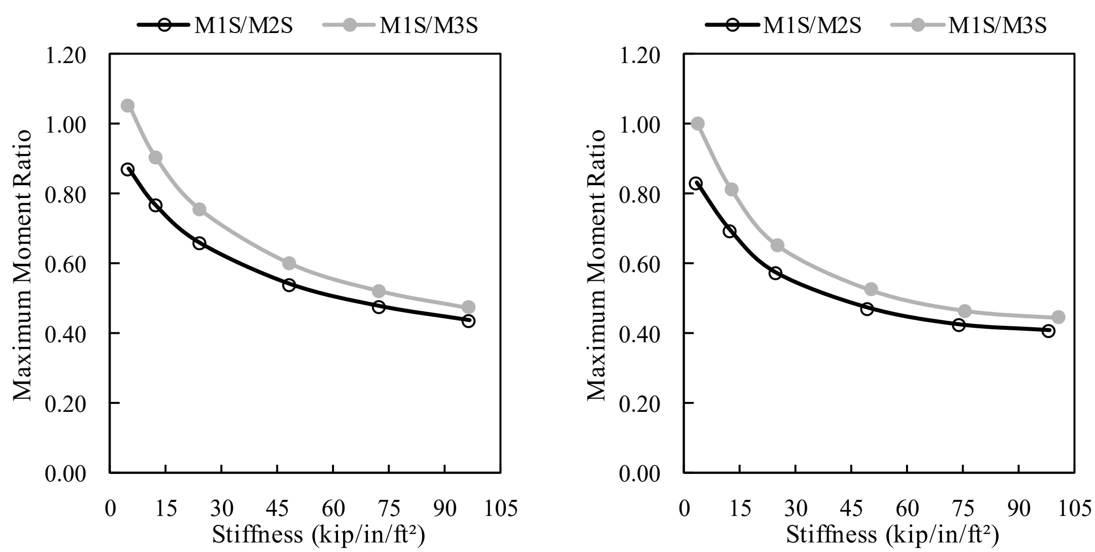

Fig. 6 Influence of panel stiffness on maximum moment ratios (simply supported to 2-3 span continuous) of 12 ft (a) and 16 ft (b) panels under a 100 plf distributed load.

Table 1 Summary of parametric study results for the 12-feet long panels

Panel	Stiffness (kip/in/ft²)	Deflection (in)	Connector Shear Force (lbf)	Connector Slip (in)	Wythe Axial Force (lbf)		Top Wythe Moment (lbf-in)		Bottom Wythe Moment (lbf-in)		Tensile Stress (psi)	
					At support	Max Span Value	At support	At mid span	At support	Max Span Value	At support	Max Span Value
P1L12	5	0.04	170	0.00567	170	512	-1111	8927	-511	9078	-14	-130
	12	0.04	337	0.00449	337	1025	-1612	7387	-1012	7539	-20	-112
	24	0.03	500	0.00333	500	1540	-2099	5842	-1499	5993	-26	-94
	48	0.02	651	0.00217	651	2060	-2552	4282	-1952	4433	-31	-76
	72	0.02	735	0.00163	718	2324	-2753	3491	-2153	3642	-33	-67
	96	0.01	792	0.00132	752	2485	-2856	3009	-2256	3160	-34	-61
P2L12	5	0.02	92	0.00307	44	240	-10250	5469	-10025	5614	-142	-80
	12	0.02	196	0.00261	69	521	-9639	4866	-9414	5085	-133	-74
	25	0.02	313	0.00209	51	842	-8887	4217	-8662	4436	-123	-67
	49	0.01	442	0.00147	74	1209	-7921	3391	-7696	3612	-110	-59
	74	0.01	549	0.00122	223	1406	-7311	2887	-7086	3106	-100	-53
	98	0.01	636	0.00106	365	1526	-6883	2680	-6658	2907	-93	-51
P3L12	5	0.02	106	0.00353	28	291	-8461	6091	-8236	6185	-117	-88
	12	0.02	218	0.00291	23	598	-8158	5237	-7933	5472	-113	-80
	25	0.02	338	0.00225	39	926	-7742	4448	7517	4667	-107	-71
	49	0.01	460	0.00153	92	1279	-7126	3505	-6901	3724	-98	-61
	74	0.01	572	0.00127	242	1467	-6685	2959	-6460	3178	-91	-54
	99	0.01	655	0.00109	504	1585	-6351	2730	-6126	2956	-89	-52

Table 2 Summary of parametric study results for the 16-feet long panels

Panel	Stiffness (kip/in/ft²)	Deflection (in)	Connector Shear Force (lbf)	Connector Slip (in)	Wythe Axial Force (lbf)		Top Wythe Moment (lbf-in)		Bottom Wythe Moment (lbf-in)		Tensile Stress (psi)	
					At support	Max Span Value	At support	At mid span	At support	Max Span Value	At support	Max Span Value
P1L16	3	0.13	258	0.01242	258	1425	-1373	14574	-773	14674	-17	-214
	12	0.10	467	0.00900	467	2602	-2000	11045	-1400	11145	-25	-173
	25	0.07	638	0.00205	637	3589	-2511	8082	-1911	8182	-39	-139
	49	0.05	789	0.00127	771	4432	-2914	5702	-2314	5808	-46	-111
	74	0.04	857	0.00092	823	4809	-3068	4614	-2468	4784	-48	-100
	98	0.03	898	0.00073	846	5023	-3137	4103	-2539	4204	-49	-93
P2L16	3	0.06	144	0.0069333	37	677	-17566	9273	-17466	9373	-244	-135
	13	0.05	286	0.0055106	32	1353	-15926	7826	-15826	7943	-221	-120
	25	0.04	423	0.0013577	271	2013	-14133	6313	14033	6413	-198	-103
	50	0.03	560	0.0008988	803	2630	-12088	4663	-11988	4763	-173	-84
	76	0.03	684	0.0007318	1255	2905	-10899	3915	-10799	3941	-160	-75
	101	0.02	774	0.0006277	1619	3050	-10094	3435	-9994	3574	-151	-71
P3L16	3	0.07	163	0.0078482	52	794	-14550	10162	-14450	10204	-202	-147
	13	0.06	311	0.0059923	124	1507	-13597	8300	-13497	8394	-190	-127
	25	0.05	445	0.0014283	418	2156	-12420	6532	-12320	6494	-175	-105
	51	0.03	589	0.0009453	947	2755	-10886	4782	-10786	4861	-158	-87
	76	0.03	705	0.0007543	1340	3037	-9903	3976	-9803	4034	-147	-77
	102	0.02	787	0.0006383	1636	3200	-9203	3474	-9103	3411	-139	-70

Additionally, a set of 12 ft panels was analyzed to understand the distribution of forces and deformations along the length of the panel. An average panel stiffness of 45 kip/in/ft² was chosen, which corresponds to an approximate mid value of the analyzed stiffness range. The maximum deflection point occurred at mid-span (0.5 Ls) for a simply supported panel, while it occurred at 0.4375 the length of the span, starting from the first support, for the continuous panels. The supports were located at 0, 12, 24, and 36 feet, as shown in Fig. 7.

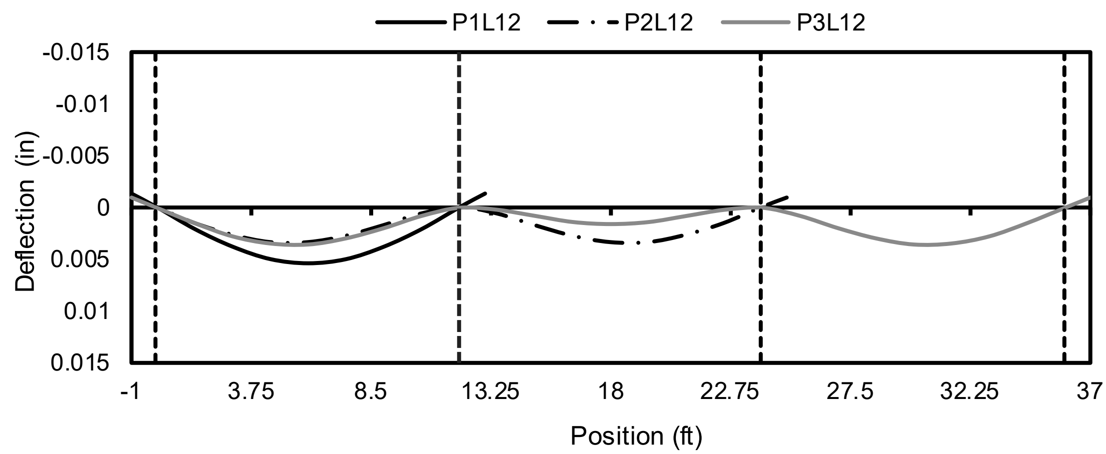

Fig. 7 Comparison of deflection values for simple span and continuous panels under a 100 plf distributed load.

Equally important, the connector shear force distribution was examined to visualize the trends in its distribution along to the panel. As Fig. 8 shows, connector forces have an antisymmetric distribution with respect of an axis passing halfway through the total panel length. For example, the force values for a two-span continuous panel will be reversed at 12 ft. The internal axial force in the wythes also shown a similar trend, see Fig. 9.

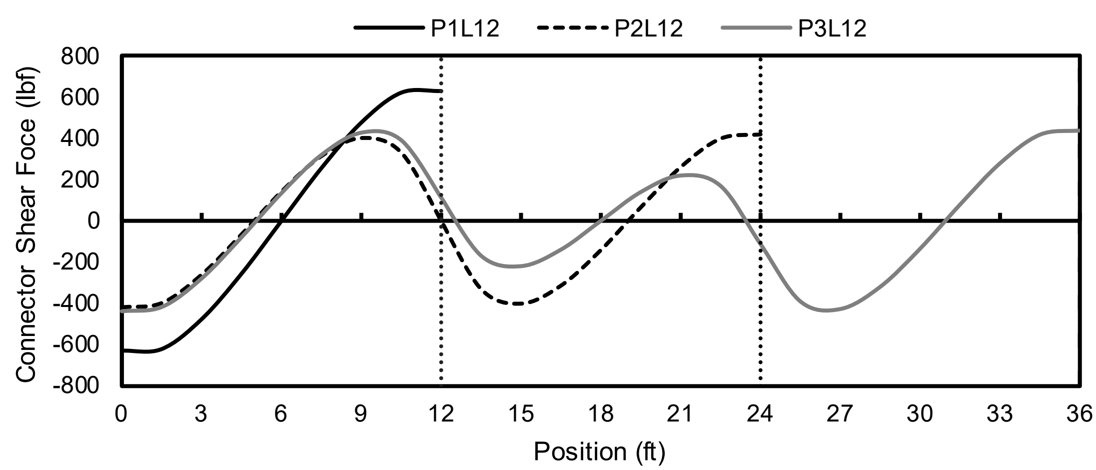

Fig. 8 Comparison of connector shear force for simple span and continuous panels

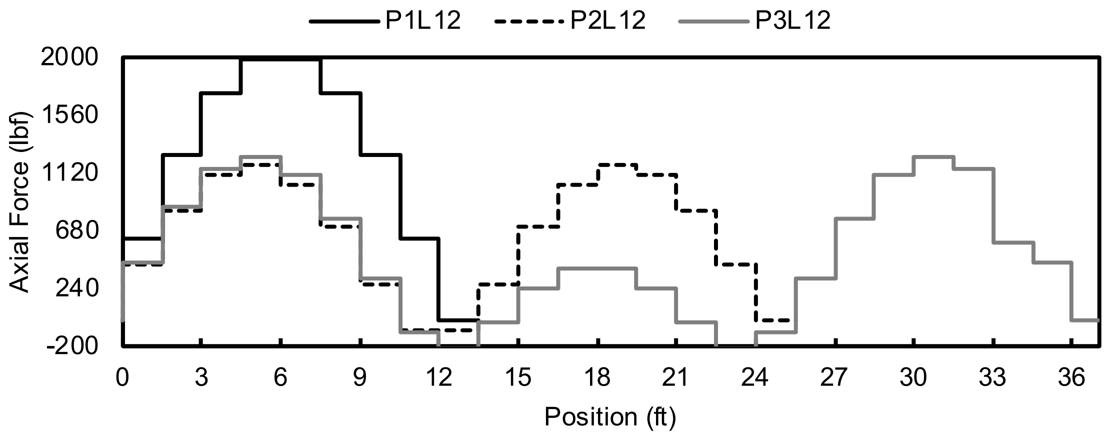

Fig. 9 Comparison of wythe internal axial force for simple span and continuous panels

Additionally, the internal moments in the bottom wythe were plotted along the length of the panels, see Fig. 10. It can be noted that the moment at the support for the continuous panels is 100% larger than the moment at mid-span of a simply supported panel, and the moments for a two-span panel are slightly larger than the three-span one. Each peak in the internal moment diagram is located over the shear connector that is modeled as an elastic spring element.

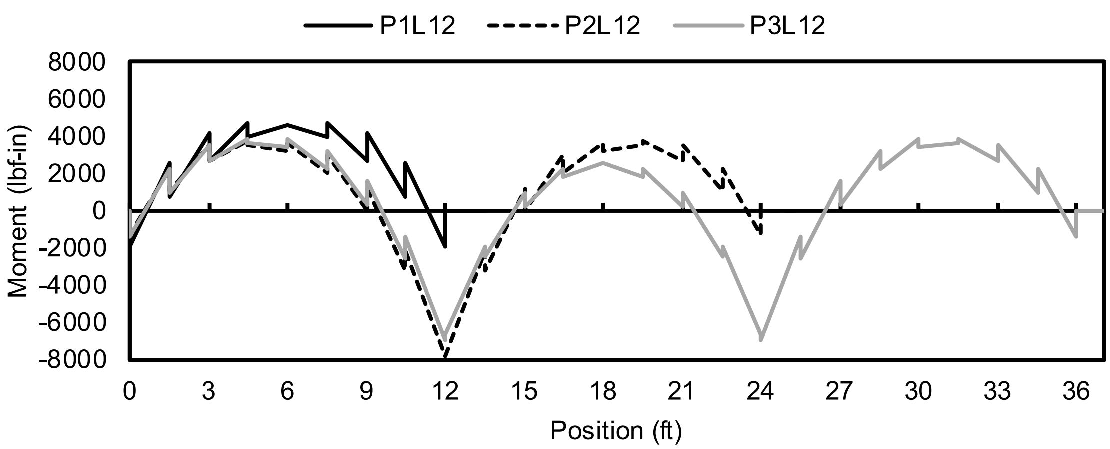

Fig. 10 Comparison of the bottom wythe internal moment for simple span and continuous panels

Ultimately, the slip and stresses in the bottom wythe were computed and plotted along the length of the panel to illustrate the differences among the different configurations. Both continuous panels had tensile stresses 25% higher at the supports than the simple span panel had, which is opposite to what the structural analysis fundamentals and engineering judgment would anticipate, as seen in Fig. 11. The connector slip was significantly smaller for multi-span panels versus simple spam, as shown in Fig. 12.

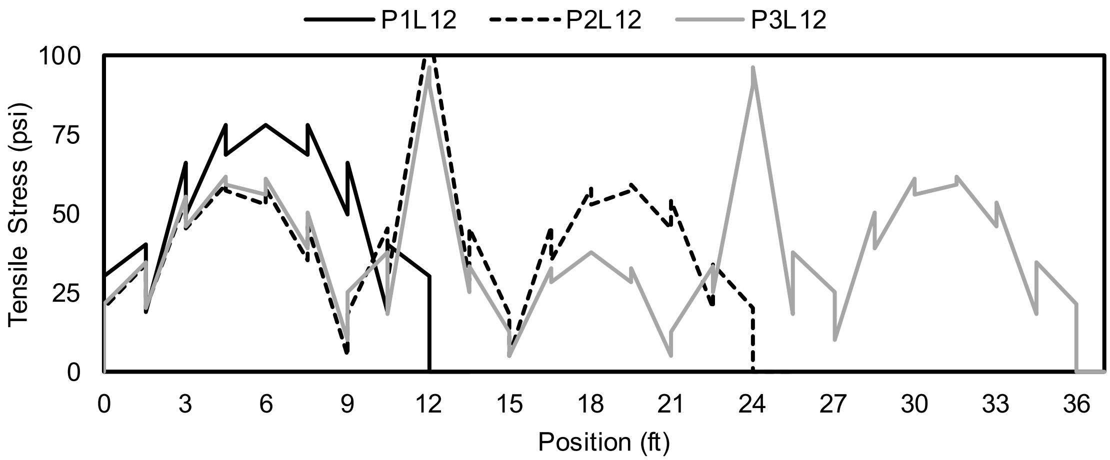

Fig. 11 Comparison of the bottom wythe tensile stresses for simple span and continuous panels

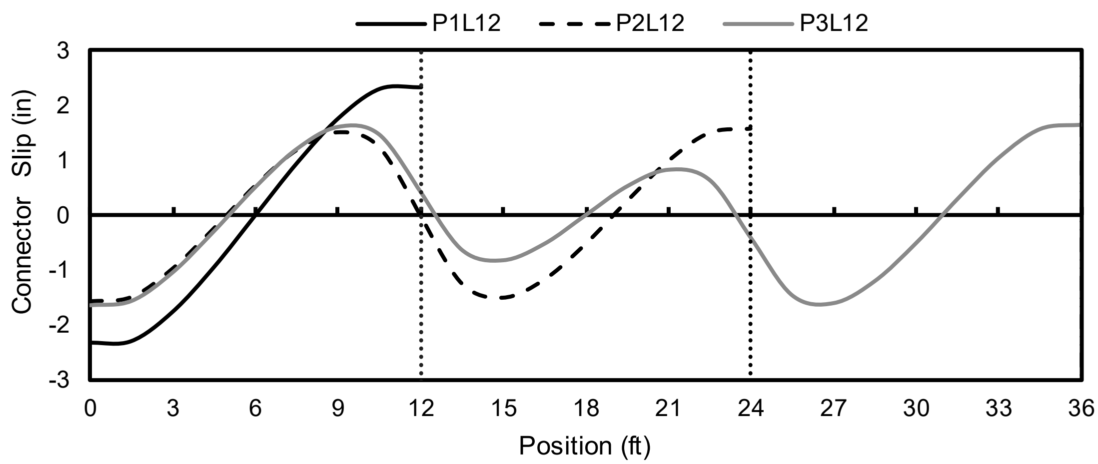

Fig. 12 Comparison of connector slip for simple span and continuous panels

DISCUSSION

Tensile stresses are critical in the design of precast concrete sandwich panels, as they constitute the main concern for durability, performance, and aesthetics. However, the design process differs among engineers and the connector shape, material, and performance varies across the whole industry. As it was shown in Fig. 2-Fig. 6, the overall stiffness of the panel that connectors provide to the panel plays an important role in the prediction of the forces and deformations; thus, caution and engineering judgment should be exercised when providing or using connector properties.

As illustrated in Fig. 11 and shown in Table 3, tensile stresses can be substantially higher in multi-span panels than they are in the simply supported panels, which can sound inconsistent to the extent of the structural analysis fundamentals. The reason for this is that the connector slip is distributed differently than might be expected in continuous panels. The connectors in a continuous panel do not slip as much along the length of the panel when compared to the simply supported panel (see Fig. 8), resulting in reduced resultant axial forces generated by the connectors (compare maximum values in Fig. 9), increasing the observed tensile stresses in the continuous panel over the support. From a design perspective, one may need more total connector stiffness per area – or a different connector distribution – in a continuous panel compared to a single span panel when trying to design for tensile stresses. The reason for this is the differences in slip magnitude and distribution between the two-panel types.

Table 3 Effect of panel stiffness per are on maximum tensile stress

Stiffness ($kip/in/ft^2$)	Tensile Stress (psi)*					
	P1L12	P2L12	P3L12	P1L16	P2L16	P3L16
4.8	-130	-142	-117	-214	-244	-202
12.1	-112	-133	-113	-173	-221	-190
24.1	-94	-123	-107	-139	-198	-175
48.2	-76	-110	-98	-111	-173	-158
72.3	-67	-100	-91	-100	-160	-147
96.4	-61	-93	-89	-93	-151	-139

* Negative for tension

Designers often use an effective section approach, also known as the percent composite approach. In practice, an effective section modulus (S) is used to calculate stresses and an effective flexural stiffness (EI) is used to calculate deflections. Connector manufacturers often provide the effective S and EI for the designer based on the panel geometry and often use a similar approach to that in this paper. The percentage of fully composite for the effective S and EI is rarely the same. To illustrate this for a continuous panel, consider two case scenarios: a nine-inch thick precast concrete solid panel and a 3-3-3 (concrete-insulation-concrete) precast concrete sandwich panel with stiffness per area of 45 $kip/in/ft^2$. Both panels are 4 ft wide, 12 ft two-span continuous, and are subject to a 25 psf load. The stresses in the solid section are much smaller than in the sandwich panel for the given shear stiffness, but approximately the same when 17.6% of S effective section is used, see Fig. 13. The equivalent section approach does not predict the behavior with accuracy at all locations due to the stress concentrations near discrete shear connectors, but it satisfies the tensile stress criterion where positive and negative moment maxima occur. On the other hand, if the same comparison is made for the deflections, the approach yields 12.6% EI of a fully composite 3-3-3 panel, as shown in Fig. 14.

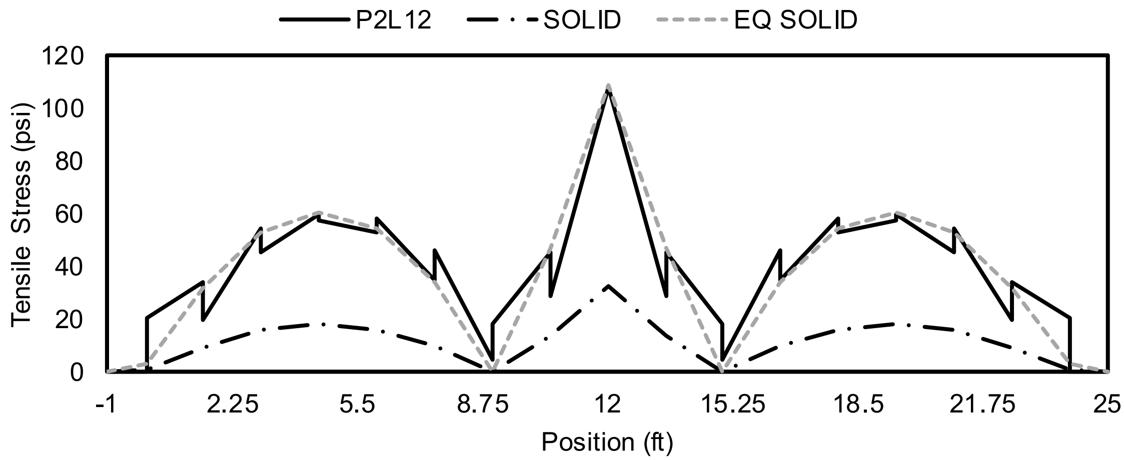

Fig. 13 Comparison of the tensile stresses' envelope in a sandwich panel versus solid panels for two-span continuous arrangement

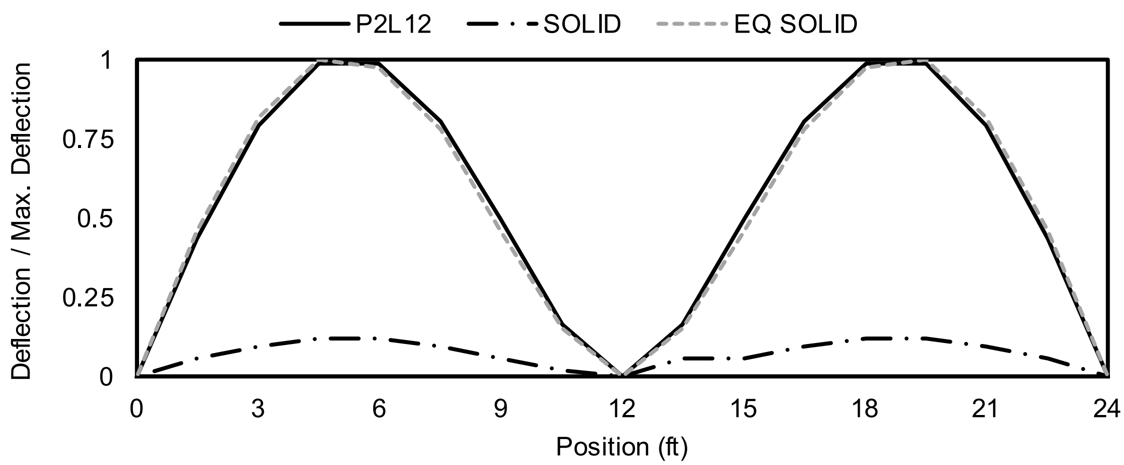

Fig. 14 Comparison of deflections in a sandwich panel versus solid panels for a two-span continuous arrangement

CONCLUSIONS

With the growing interest in the use of concrete sandwich wall panels due to their inherent thermal efficiency, resistance to impact and fire, and economy, the challenges in their design have also increased. This paper dealt with the implications of building non-load bearing continuous sandwich wall panels and the effects of the connecting medium stiffness in their performance. The following conclusions can be made from the parametric study:

- The stiffness per area of the panel (the number of connectors multiplied by the stiffness and divided by the area of the panel), influenced all design parameters. As the stiffness per area increased, the deflections and internal moments decreased, whereas the wythe internal axial force and connector shear force increased.

- Continuous sandwich panels develop more significant flexural moments, and tensile stress than in similar simply supported panels. These flexural moments and stresses were similar for low stiffness per area but were substantially different for high stiffness per area values.
- An effective section approach predicts stresses at tensile stress maxima and deflections if properly calibrated for the situation.

REFERENCES

1. PCI. *Architectural Precast Concrete*. 3rd ed. (Freedman S, ed.). Chicago, IL: Precast/Prestressed Concrete Institute; 2007.
2. Sorensen T, Dorafshan S, Maguire M, Thomas RJ. Thermal Bridging in Concrete Sandwich Walls. *Concr Int*. 2019;40(10).
3. Olsen J, Al-Rubaye S, Sorensen T, Maguire M. *Developing a General Methodology for Evaluating Composite Action in Insulation Wall Panels*.; 2017.
4. Al-Rubaye S, Sorensen T, Maguire M. Investigating Composite Action at Ultimate for Commercial Sandwich Panel Composite Connectors. *Civ Environ Eng Fac Publ Pap 3532*. 2017. https://digitalcommons.usu.edu/cee_facpub/3532.
5. Al-Rubaye S, Sorensen T, Maguire M. Full Scale Testing of Concrete Sandwich Wall Panels. In: *Proceedings of the PCI National Bridge Conference,*. Cleveland, OH; 2017.
6. Einea A, Salmon DC, Tadros MK, Culp TD. Full Scale Testing of Precast Concrete Sandwich Panels. *Struct J*. 1997;94(4):354-362. doi:10.14359/486
7. Tomlinson D, Fam A. Flexural behavior of precast concrete sandwich wall panels with basalt FRP and steel reinforcement. *PCI J*. 2015. doi:10.15554/pcij.11012015.51.71
8. Naito CJ, Hoemann JM, Shull JS, et al. *Precast/Prestressed Concrete Experiments Performance on Non-Load Bearing Sandwich Wall Panels*. BLACK AND VEATCH OVERLAND PARK KS; 2011.
9. Gosowski B, Gosowski M. Distributional solutions of bending problems for continuous sandwich panels with thin facings. *Arch Civ Mech Eng*. 2012;12(1):13-22. doi:10.1016/j.acme.2012.03.003
10. Granholm H. *Om Sammansatta Balkar Och Pelare Med Särskild Hänsyn till Spikade Träkonstruktioner: On Composite Beams and Columns with Particular Regard to Nailed Timber Structures*. Elanders boktryckeri aktiebolag; 1949.
11. Newmark NM. Test and analysis of composite beams with incomplete interaction. *Proc Soc Exp Stress Anal*. 1951;9(1):75-92.
12. Holmberg A, Plem E. *Behaviour of Load-Bearing Sandwich-Type Structures*.

Byggforskningen; 1965.

13. Howard GA. Analysis and design of structural sandwich panels. 1969.
14. Bai F, Davidson JS. Analysis of partially composite foam insulated concrete sandwich structures. *Eng Struct*. 2015;91:197-209. doi:10.1016/j.engstruct.2015.02.033
15. Bai F, Davidson JS. Theory for composite sandwich structures with unsymmetrical wythes and transverse interaction. *Eng Struct*. 2016;116:178-191.
16. Girhammar UA. A simplified analysis method for composite beams with interlayer slip. *Int J Mech Sci*. 2009;51(7):515-530. doi:10.1016/j.ijmecsci.2009.05.003
17. Al-Rubaye S, Sorensen T, Olsen J, Maguire M. Evaluating Elastic Behavior for Partially Composite Precast Concrete Sandwich Wall Panels. *PCI J*. 2018;63(5):71-88.
18. Sorensen T, Dorafshan S, Maguire M. Matrix Model Accuracy of Partially Composite Concrete Sandwich Panels. In: *Proceedings of the PCI National Bridge Conference*. Denver, CO; 2018.
19. Al-Rubaye S, Sorensen T, Thomas RJ, Maguire M. Generalized beam–spring model for predicting elastic behavior of partially composite concrete sandwich wall panels. *Eng Struct*. 2019;198:109533. doi:10.1016/j.engstruct.2019.109533