Alexandria Engineering Journal (2019) 58, 901–908

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Alexandria Engineering Journal

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ORIGINAL ARTICLE

Experimental study on the flexural behavior of insulated concrete sandwich panels with wires as shear connectors

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Received 14 July 2016; revised 8 June 2017; accepted 19 August 2019
Available online 4 September 2019

KEYWORDS

Precast;
Sandwich panels;
Insulated structural panels;
EPS;
Composites;
Experiment

Abstract Composite structural elements can serve dual purposes of transferring the load and insulating the buildings. This paper presents and discusses the results of experimental study carried out to understand the flexural behavior of prototype precast insulated concrete sandwich panels using truss-shaped continuous shear connectors. Experimental study consisted of four prototype concrete sandwich panels tested under four-point bending simulating one-way slab action. Panel thickness and size of wire mesh used as reinforcement in concrete wythes are the major parameters considered. Test results indicate that the truss-shaped shear connectors are effective to achieve composite action of the panels until failure. Test results also indicate that the panel thickness affects the flexural load carrying capacity, and size of wire mesh affects the ductility. Experimental and analytical studies are required in this area towards developing guidelines for design of concrete sandwich panels for field applications.

1. Introduction

Precast concrete structural elements are manufactured under controlled factory conditions, and hence concrete structural elements with good precision in geometry and finishing may be manufactured. Background information on precast technology may be found in the literature [1–3]. Precast concrete elements besides being structurally and economically efficient [4], also have social and environmental benefits [5]. Precast structural elements if light-weighted may also have additional advantages such as (i) less attraction of seismic forces, (ii) ease of handling and transportation, and (iii) cost effective. Light-weight concrete sandwich panels produced by replacing core concrete using less dense material may consist of two skins of concrete called as wythe, one on either side of the core. The core is made of materials such as Expanded Polystyrene (EPS) or Extruded Polystyrene (XPS) that normally provides significant thermal and sound insulation. In order to achieve composite action of the panels, shear transfer between the wythes is ensured by using shear connectors that connect the wythes. The shear connectors may be discrete that are pro-

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Peer review under responsibility of Faculty of Engineering, Alexandria University.

https://doi.org/10.1016/j.aej.2019.08.005
1110-0168

vided at specific predefined locations or may be continuous that are oriented along the longitudinal (spanning) direction. Wire mesh or conventional steel rebars may be used to reinforce the wythes. Shear connectors may be made of materials such as steel, Glass Fiber Reinforced Polymer (GFRP) or Carbon Fiber Reinforced Polymer (CFRP).

Bush and Stine [6] have reported that under flexural load 100% composite action may be achieved in concrete sandwich panels. Composite and non-composite action of sandwich panels using different numbers of truss connectors are explored in their study. They reported that the panels with shear connectors exhibit composite action. They also observed that stripping and handling inserts offer significant shear transfer between the wythes. Einea and Salmon [7] have carried out experimental and analytical studies on the flexural behavior of concrete sandwich panels with hybrid shear connectors using FRP and prestressed steel strands. They found that the panels exhibit thermal and structural efficiency. They recommend that mechanical anchorages be provided in shear connectors made of FRP rebars. They also reported that the axial strength of the shear connectors have profound effect on the shear strength of the panels, and found that full-scale size panels achieve more composite action than the small-scale size specimens. Salmon and Einea [8] proposed equations to determine the out-of-plane deflections of partially composite insulated sandwich panels due to temperature difference between the wythes. The authors show that the deflections are insensitive to stiffness of the connectors, and hence, partially composite panel and fully composite panel have almost same deflection due to temperature difference.

Salmon et al. [9] have tested concrete sandwich panels under lateral loads. Two different types of shear connectors such as FRP connectors and steel connectors are used in their study. The authors reported that the panels achieve semi-composite action under service loads, and the panels with FRP shear connectors are efficient in thermal insulation compared to steel shear connectors. Benayoune et al. [10] have carried out experimental and theoretical investigations on the behavior of concrete sandwich panels under flexural load. The authors reported that the mode of failure of the panels is similar to conventional solid concrete panel behavior. Also, they reported that the shear connectors significantly affect the load carrying capacity and composite action of panels. Gara et al. [11] carried out experimental and numerical analysis of sandwich panels subjected to four-point bending. Totally six numbers of full-scale panels were tested in their study. Flexural tests indicate that the panel failure occurs due to the rupture of the bottom wythe. They report that the equivalent loads at which failure occur is lesser than the design loads on floors and hence, these types of panels may be used for practical applications. The authors also report that increase in the load carrying capacity is achieved by increasing the panel thickness.

Hopkins et al. [12] have carried out studies to investigate the creep effect in concrete sandwich panels and the behavior was compared with conventional solid RC panel. They reported that the concrete sandwich panels showed better performance as compared with the conventional solid panel. Daniel Ronald Joseph et al. [13] have carried out experimental and analytical studies on the behavior of concrete sandwich panels under different flexural loading conditions like bending and punching. They reported that type of loading conditions have significant effect on the panel behavior. They also noted that the behavior of concrete sandwich panels subjected to punching load is similar to conventional solid RC slabs. Amran et al. [14] have studied the structural behavior of foamed concrete sandwich panels. They found considerable weight reduction by using foamed concrete for casting the wythes. Numerical simulations results and experimental results were found to be in good agreement. They also proposed a formula to predict the ultimate load carrying capacity of these panels. However, they conclude that further studies are required in this area. Daniel Ronald Joseph et al. [15] have carried out experimental studies on the behavior of prototype concrete sandwich panels with wire mesh and conventional rebars as reinforcing elements in bottom wythe. They reported that the panels behave as composite elements and their cracking behavior is similar to ferrocement cracking behavior.

Literature survey indicates that only limited experimental and numerical studies are available to study the behavior of prototype concrete sandwich panels under flexural loads. Note that the experimental studies on the behavior of prototype concrete sandwich panels are important because prototype panels achieve more composite action as compared to small-scale panels [7]. Design recommendations (strength and serviceability requirements) for the design of concrete sandwich panels are also not readily available for practical applications. There exists an urgent need to determine the load carrying capacity and study the behavior of concrete sandwich panels under out-of-plane flexural loading, and develop guidelines for practical applications which is the motivation for the present study. Studies on flexural behavior of prototype concrete sandwich panels with continuous truss-shaped shear connectors made of wires are also not available in the literature. This paper presents and discusses experimental and numerical studies carried out to understand the flexural behavior of prototype concrete sandwich panels with continuous truss-shaped shear connectors under four-point bending. The paper is organized as follows. Section 2 presents experimental details such as materials, casting method, test set-up and the instrumentation details, Section 3 presents results and discussions, and Section 4 presents summary and conclusions.

2. Experimental study

2.1. Materials used

Four prototype concrete sandwich panels are tested in the present experimental study. The major parameters considered are the thickness and wire mesh size used as wythe reinforcement. The size of the prototype panels tested is $3000 \times 1200 \times 150/100$ mm (length × width × thickness). The schematic sketch of the components of panels is shown in Fig. 1. The thickness is varied by varying EPS thickness. The wire mesh sizes considered are 100×100 mm and 50×50 mm. The two meshes are connected using continuous truss-shaped shear connectors that are inclined at 70°. The wires of the shear connectors are welded to the wire mesh. The continuous truss-shaped shear connectors are oriented along the longitudinal (spanning) direction of the panels, and the spacing between the trusses is 100 mm. The wires of the mesh and the shear connectors are nearly 2.2 mm in dia. The average tensile strength of the wires as supplied by the manufacturer is 651.6 N/mm^2.

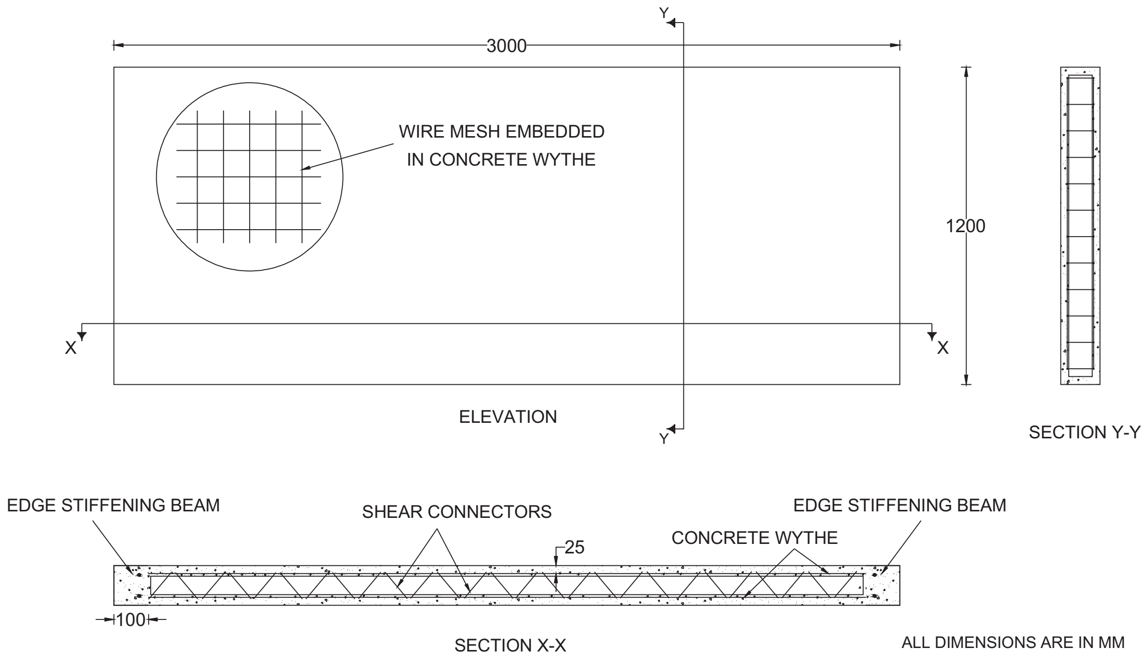

Fig. 1 Schematic sketch of sandwich panel.

Self-compacting concrete is used for casting the wythes. The concrete mix proportion is arrived based on the guidelines of ACI [16], and it is 1:1.89:2.34:0.3:0.41:0.6% in the order of Cement, Coarse aggregate, Fine aggregate, Ground Granulated Blast Furnace Slag (GGBFS), Water and Superplasticizer (by weight of binder content). In designing the concrete mix, target slump value of >650 mm is chosen (See Table 2.5 of ACI [16]) and, the total powder content used is 481-kg/m^3. The cement and the GGBFS content used are 370-kg/m^3 and 111-kg/m^3 respectively. As per ACI [16], filling ability and passing ability of the concrete are measured using slump flow test and L-box test respectively. The average slump value is 678 mm. In L-box test the ratio of the height of concrete in the horizontal section to that of vertical section determined is 1.3. T_{50} which gives an indication of the viscosity of the concrete is nearly 4-secs, and hence according to ACI [16] the SCC is found to have viscosity that lies between low viscosity and high viscosity. It is observed from these tests that SCC satisfies the minimum requirements [16]. Coarse aggregates passing through 10 mm sieve are used. The average cube compressive strength (f_{ck}) and flexural tensile strength of SCC are 45.97 MPa and 4.34 MPa respectively. The flexural tensile strength of concrete is determined using concrete prism specimens of size 150 × 150 × 700 mm as per Indian Standard [17]. Number of trusses as shear connectors present in a panel is 13. The thickness of top and bottom concrete wythes is 25 mm for all the panels. Table 1 gives the details of the panels considered in the experimental study.

2.2. Casting of panels

The sequence of casting a panel is shown in Fig. 2. A steel mould of inner dimension 3000 × 1200 mm is placed on a level surface and concrete is poured to a depth of 25 mm to form bottom wythe. EPS panel with wire mesh and shear connectors is placed on the concrete, and concrete is poured on the EPS to form top wythe of 25 mm thickness. Stiffening concrete beams are provided along the supporting edges (by dissolving 100 mm EPS) to avoid failure due to local crushing of concrete. The panels are cured for 28 days. This method of manufacturing does not require highly skilled labors and hence, may be suitable for mass production of the panels.

2.3. Test set-up and instrumentation

The panels manufactured are tested under four-point bending. This type of loading is chosen because of constant bending moment region being developed between the loading points. Displacement controlled loading is applied until the panels failed. One edge of the panel is supported on a hinge and the other is supported on a roller. Linear Voltage Displacement

Table 1 Details of prototype concrete sandwich panels.

Specimen	Mesh size (mm)	Thickness (mm)		
		Wythe	EPS	Total
F1	100 × 100	25	100	150
F2	100 × 100	25	50	100
F3	50 × 50	25	100	150
F4	50 × 50	25	50	100

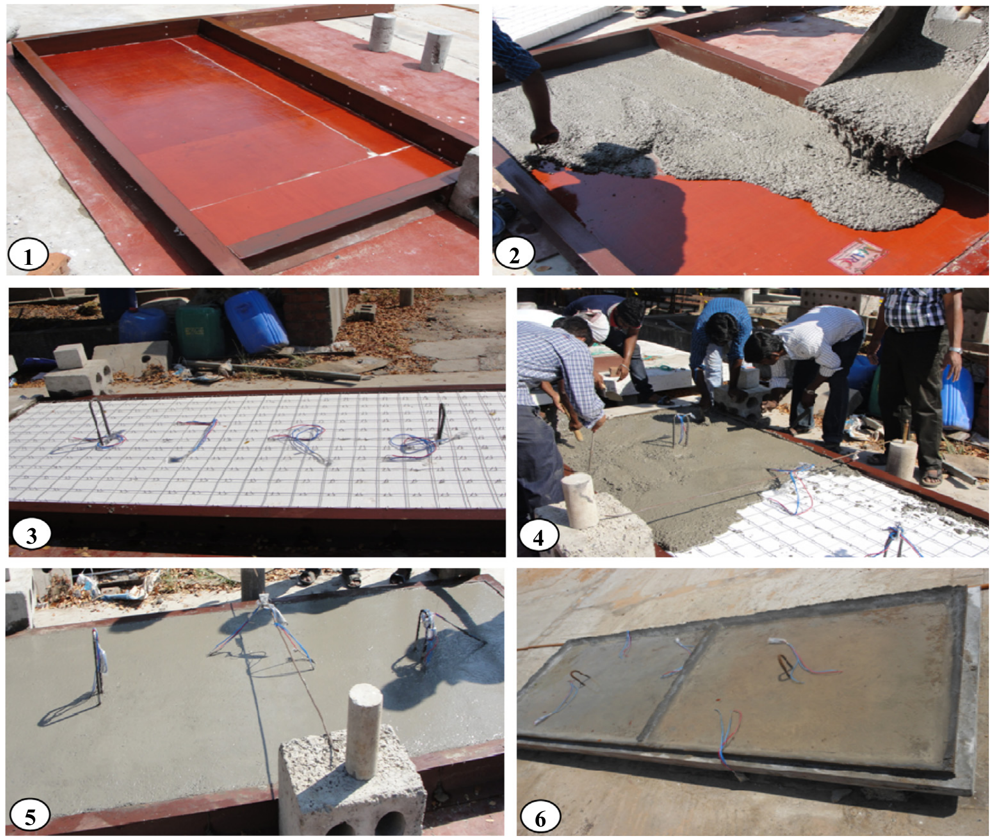

Fig. 2 Sequence of casting a panel.

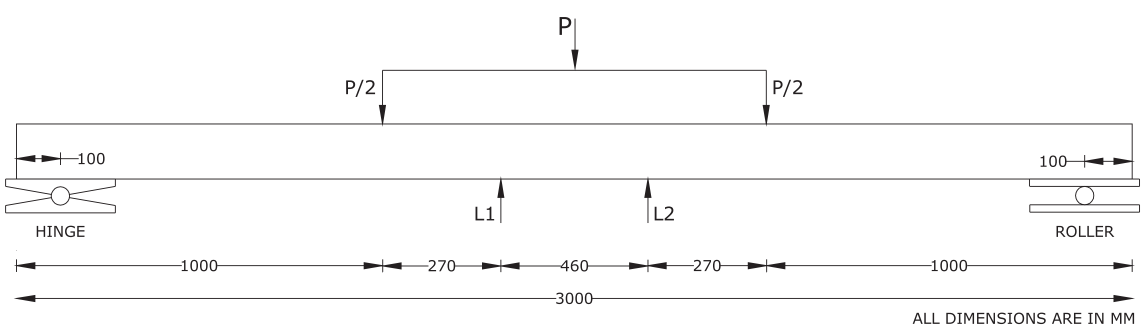

Fig. 3 Schematic layout of test setup and LVDT locations.

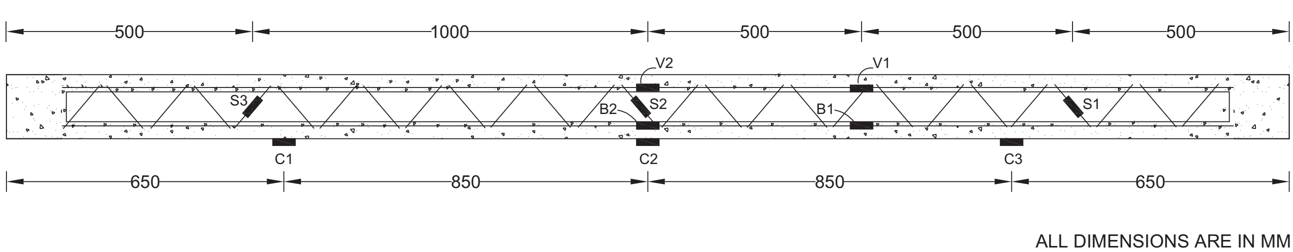

Fig. 4 Locations of strain gauges.

Fig. 5 Picture of a panel in test setup.

Transducers (LVDTs) with 50 mm range are used to measure the lateral deflections of the panels. The strains on the wires and concrete are measured using strain gauges with gauge lengths of 2 mm and 30 mm respectively. Schematic sketch of the test set-up and instrumentation are shown in Figs. 3 and 4. Picture of typical panel in test set-up is shown in Fig. 5. The deflection of F1 is measured at the mid-span.

3. Results and discussions

This section presents and discusses the results of the experimental program. The discussions are presented in the following order: with respect to cracks observed and failure, load-deflection behavior and load-strain behavior.

Fig. 6 Picture of failed panel F1.

Fig. 7 Picture of failed panel F4.

Pictures of typical failed panels are shown in Figs. 6 and 7. First crack load, cracking moment, ultimate load, and ultimate moment of the tested panels are given in Table 2. Cracking moment and ultimate moment are determined using first principles. Weight of distributor beams (5.52 kN) and self-weight of the panel are not included in the values of Table 2.

Sample calculation for F1:

$$\text{First crack load} = 12.6\ \text{kN}$$

$$\begin{aligned}\text{Cracking moment} &= 12.6 \times (1000 - 100) \times 0.5/1000 \\ &= 5.7\ \text{kNm}\end{aligned}$$

$$\begin{aligned}\text{Ultimate moment} &= 14.6 \times (1000 - 100) \times 0.5/1000 \\ &= 6.6\ \text{kNm}\end{aligned}$$

Due to larger size of panels and congested experimental set-up, locating exact first crack on panel soffit is difficult. The first crack loads reported in Table 2 are the loads at which first visible cracks are seen on sides of the panels. For all the panels first crack occurred in bottom wythe between one of the loading points and the nearest support (ie. In shear span), and no cracks occurred in top wythe until failure. No separation of wythes which would be witness by formation of horizontal crack along the EPS-concrete interface is observed in any panel, and hence this leads to conclude that all the panels behave as composite elements until failure.

The panels F1, F2 and F4 fail by widening of first crack which tend to reach the nearest loading point and, only very few cracks in the bottom wythe occurred between the loading points. Numbers of cracks in bottom wythe of panel F3 is relatively more than other panels. Figs. 8a and 8b show the cracks marked in bottom wythe of typical failed panels.

In all the panels, first crack occurred in shear span, and it is important to note that, it occurred at cross-section located approximately at effective depth distance from the loading point. Due to formation of cracks in the shear span at cross-section which is also near to the maximum bending moment region, the formation and propagation of crack that cause panel failure may be attributed due to the combined effect of flexural and shear stresses. More number of cracks in panel F3 is due to lower mesh size. The panels with high mesh size (and hence lower percentage of reinforcement) fail due to widening of single crack. The panels with lower mesh size with relatively high percentage of reinforcement achieve better energy dissipation by formation of number of cracks in bottom wythe of the panel. When panel thickness is less, lower size of mesh does not result in formation of number of flexural cracks as can be seen in panel F4 (see Fig. 7.). This is attributed due to lesser capacity of the panel due to less lever arm.

Fig. 9 shows the load-deflection curves obtained for the panels. Among the configurations of the panels considered, the ultimate load and deformability are more for panel F3. Comparison of these curves indicate that the thickness of concrete sandwich panel affects the flexural load capacity significantly, and the mesh size affects deformability of the panel significantly. Unlike conventional steel rebars, decreasing the mesh size, and thereby increasing the percentage of reinforcement, did not increase the load carrying capacity of the panel. However, as noted earlier, mesh size affects the deformability

Table 2 Test results of panels.

SI. No.	Panel ID	First crack load (kN)	Cracking moment (kNm)	Ultimate load (kN)	Ultimate moment (kNm)
1	F1	12.6	5.7	14.6	6.6
2	F2	5.2	2.3	10.5	4.7
3	F3	9.7	4.4	20.4	9.2
4	F4	5.5	2.5	9.8	4.4

Fig. 8a Crack pattern seen in the panel F1.

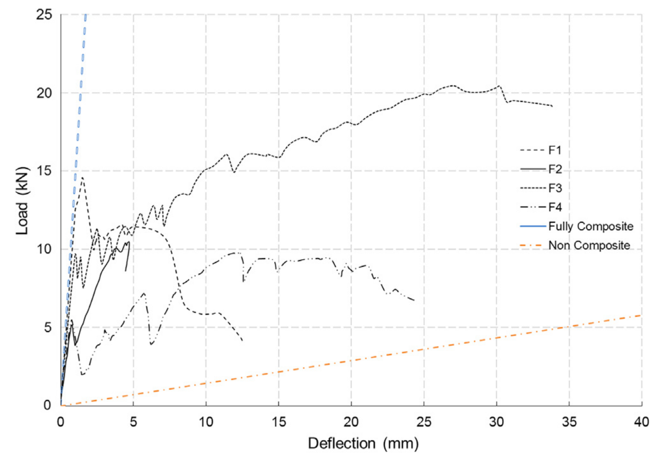

Fig. 9 Load-deflection curves of the panels.

Fig. 8b Crack pattern seen in the panel F3.

of the panels. Higher deformability of panels F3 and F4 is due to lower mesh size.

Fig. 9 shows that the panels F1, F2, F3 and F4 behave linearly up to loads 12.6 kN, 5.2 kN, 9.7 kN and 5.5 kN, respectively, beyond which they behave non-linearly until failure. Comparing these loads with first crack loads of the panels indicate that the load magnitudes are comparable, and hence the panels may be considered to behave linearly up to first crack load. The panel F3 resisted higher load compared to other panels. This may be attributed due to the reasons that, (i) panel thickness is higher, and hence moment carrying capacity is higher due to larger lever arm, and (ii) lower mesh size that results in formation of relatively more number of cracks precluding panel failure by widening of single crack.

In Fig. 9, a slight change in slope of load-deflection curve, indicating slight reduction in stiffness, for panel F1 at 12.6 kN is attributed to initiation and propagation of first crack in bottom wythe. The panels with same thickness behave linearly up to almost same magnitude of load. Also it is observed that, increase in the panel thickness increases the load up to which they behave linearly.

Number of drops in the load-deflection curves are seen for panels F3 and F4. This observation is attributed primarily to formation of new cracks in bottom wythe. Such drops are not seen for panels F1 and F2 due to very few numbers of cracks in bottom wythe. Very few numbers of cracks in panels F1 and F2 are attributed to larger mesh size and hence, lower percentage of reinforcement. Number of cracks in panels F3 and F4 occurred due to lower mesh size and hence, higher percentage of reinforcement. In general, the test results indicate that, lower mesh size resulted in formation of number of cracks in bottom wythe and hence increases the deformability of the panels. The load carrying capacity of concrete sandwich panels' primary depend on panel thickness rather than on the mesh size.

The load-deflection curves of the panel indicates that the panels with lower mesh size exhibit hardening behavior wherein the panels with higher mesh size exhibit softening behavior. This is attributed to the reason that the stiffness of panels with higher mesh size decreases significantly after crack formation. Due to higher mesh size, and hence lower percentage of reinforcement area, more number of cracks could not be formed which results in widening of the crack with increase in the load.

Table 3 Degree of composite action (K) of the panels.

ID	Deflection (mm)			K in %
	Experimental	Fully composite	Non-composite	
F1	1.5	2.1	207	100
F2	1.5	2.1	207	100
F3	1.0	2.1	207	100
F4	0.4	2.1	207	100

Fig. 9 also shows the ideal load-deflection curves of fully composite and non-composite panel. It is observed that the load-deflection curves of the panels tested in the present study lie between these limits. It is noted that the initial stiffness of all the panels are nearly same as that of fully composite panel. The decrease in stiffness of the panels with increase in load magnitude is attributed due to formation and growth of cracks in the panels. Non-composite action of the panel may be witnessed by formation of horizontal cracks along the EPS-concrete interface. Since no such cracks evidencing wythe separation is observed in the panels, it leads to conclude that the panels behave as composite elements, and the decrease in stiffness is primarily attributed due to material strength limitations. First crack loads are considered to determine the degree of composite action of the panels as proposed by Frankl et al. [18] and the degree of compositeness is presented in Table 3.

$$\text{Degree of composite action, K} = \frac{\Delta_{nc} - \Delta_{\exp}}{\Delta_{nc} - \Delta_c} \times 100$$

$\Delta_{\exp}$ – Measured displacement at a selected load level
Δ_c – Corresponding theoretical displacement assuming fully composite behavior
Δ_{nc} – Corresponding theoretical displacement assuming non-composite behavior

It is observed that all the panels achieve 100% composite action prior to formation of cracks. Typical tensile strain variations measured on the concrete surface until failure of the panels F2 and F3 are shown in Figs. 10a and 10b. In these figures, the strains are expressed in microstrain.

In panel F2 (see Fig. 10a), the strain at C1 increases with increase in load, while at C2 and C3 no/less increase in strain is observed. This is due to the failure of the panel by widening of first crack which causes the fibres at the cross-sections near the crack location (at C1) be stressed, and hence strained, relatively more, leaving the fibres at other cross-sections (at/near C2 and C3) with less/no strain. This observation is expected because, after the formation of plastic hinge resulting in a mechanism, increasing the load will tend to rotate the member about the hinge if no moment redistribution is possible. For the panel F3, the strains in concrete surface at all instrumented locations increases with increase in the load (see Fig. 10b). Thus, fibres at all cross-sections of the panel are stressed, and hence strained, and resisted the load applied. These observations indicate that alternate load transfer mechanisms after formation of cracks in bottom wythe of concrete sandwich panels are required to increase the load carrying capacity by avoiding failure of the panel due to widening of first crack.

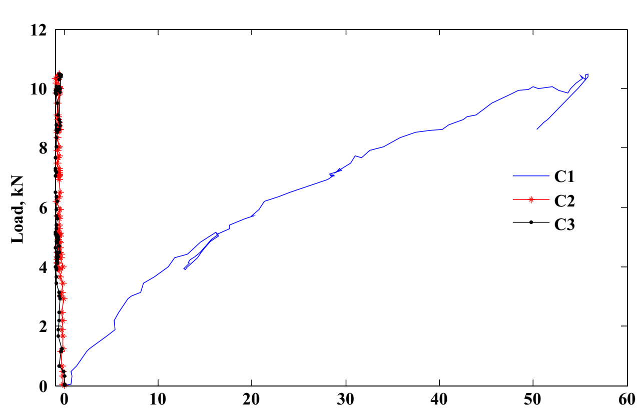

Fig. 10a Tensile strain variations in concrete for panel F2.

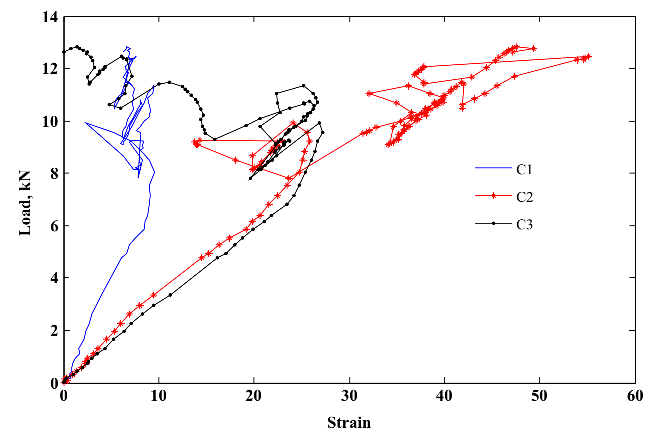

Fig. 10b Tensile strain variations in concrete for panel F3.

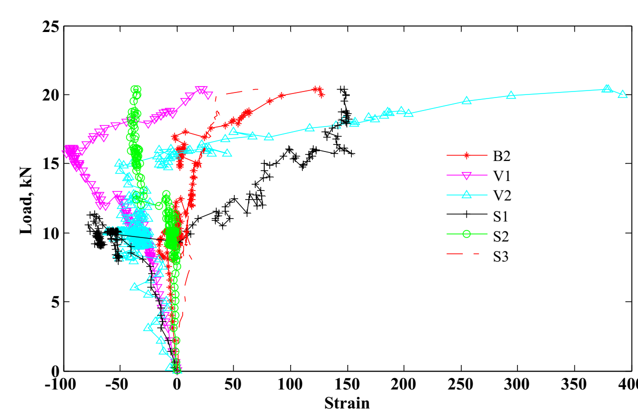

Fig. 11 Strain variations measured on wires of mesh for panel F3.

This alternate load transfer mechanism may therefore lead to maximum utilization of panel cross-sectional strength. Decreasing the mesh size and providing additional conventional rebars in bottom wythe thus may be expected to increase energy dissipation of the panel by formation of number of cracks.

Typical strain variations measured on the wires until failure of the panel F3 is shown in Fig. 11. In this figure, the strains are expressed in microstrain. Test results indicate that, for all panels, the strains measured at B1 and B2 are tensile in nature and at V1 and V2 are compressive in nature. The strains measured at S1 and S2 are either tensile or compressive. For all the panels, even at ultimate load, the strains of the wires of the mesh and the shear connectors are considerably less than the yielding strain of the wire. This clearly indicates that the wires are effective until failure of the panels and, in particular, it is evident that the truss-shaped shear connectors are effective to achieve composite action of panel until failure. Due to early failure of panels due to material failure of concrete in bottom wythe the strain in shear connectors are low. These observations in general indicates that there is large scope for increasing the load carrying capacity of these panels by providing additional conventional rebars together with wire mesh in bottom wythe. Addition of fibres in bottom wythe concrete to increase the tensile strength of concrete may also affect the flexural behavior of concrete sandwich panels.

4. Summary and conclusions

Results of the experimental studies carried out to explore flexural behavior of precast light-weight concrete sandwich panel are presented and discussed in this paper. Experimental study indicates that the wires in the form of continuous truss-shaped shear connectors may be used effectively to achieve 100% composite action of concrete sandwich panels. Experimental results also indicate that the panel thickness has significant effect on load carrying capacity, and wire mesh size affects deformability of the panel significantly. Decrease in panel stiffness with increase in the load magnitude is primarily attributed due to the material (concrete) strength limitations. More number of experiments together with numerical studies are required in this area towards developing design guidelines for practical applications of EPS insulated concrete sandwich panels.

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