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Behaviour of Innovative Concrete Sandwich Panels with Pervious Concrete Core under Flexural Bending

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Research studies investigating the possibility of using 5 new core material (other than polystyrene) in concrete sandwich panels are being reported in the literature. Though several alternate core materials are suggested in the literature, not many would improve the fire-resistance property of the sandwich panels. Towards addressing this issue, in this paper, a lightweight pervious concrete core is proposed as a new alternative. Experimental studies were conducted to determine the flexural behavior of sandwich panels with pervious concrete core. The flexural behaviour was investigated under three-point and four-point bending conditions. The geometrical variables considered were core thickness and a number of shear connector lines. The variables considered were found to influence the initial stiffness, cracking moment, ultimate moment, load-deflection behaviour and load-strain behaviour of the panels. Test results showed that, sandwich panels without shear connector trusses failed due to the separation of wythes, and the panels with shear connector trusses achieved composite action. An analytical study indicated that the equation given in ACI 318 underestimated the strength of the tested panels. The equation has to be modified by explicitly including the shear connector details in the equation for predicting the flexural strength with reasonable accuracy. Further experimental and analytical studies on prototype concrete sandwich panels with pervious concrete core produced using lightweight aggregates are required in view of developing design guidelines for practical applications.

Keywords: concrete sandwich panels, pervious concrete, lightweight, composite, flexure, panels.

1. INTRODUCTION

Concrete sandwich panels are load bearing structural members consisting of a lightweight core sandwiched between two reinforced concrete skins or wythes. These panels can be precast, and hence have combined advantages of lightweight members and precast technology. A typical concrete sandwich panel is shown in Fig. 1.

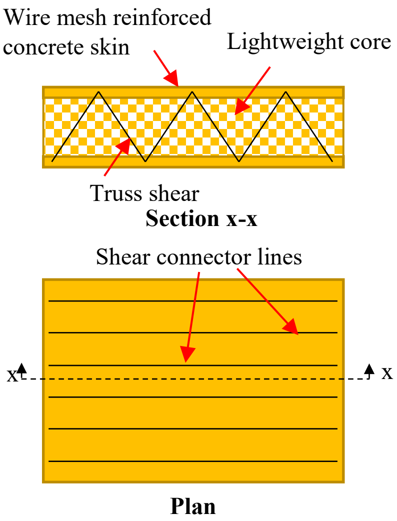

Fig. 1. Concrete sandwich panel

The main aim of using a lightweight core is to reduce the dead load and obtain thermal resistance and sound insulation properties for the panel. Several researchers [1 – 21] have proposed different types of sandwich panels based on materials used for skins, core and shear connectors, reinforcement used for wythes (wire mesh or rebar mesh) and shear connector configuration (discrete or continuous), and have experimentally determined their strength and behaviour under axial compression, flexure or combined loading conditions. Kim and You [1] used extruded polystyrene (XPS) and expanded polystyrene (EPS) core in sandwich panels. The authors found that the use of the XPS core improved the composite action of the panel as the number of shear connectors was increased. While Zhi and Guo [2], Amran et al. [3], Benayoune et al. [4] and Daniel Ronald Joseph [5, 6, 7] studied the behaviour of sandwich panels with even-surfaced EPS core, Gara et al. [8] and Carbonari et al. [9] studied the effect of undulated EPS core. Kinnane et al. [10] have used rigid board EPS in contrast to other studies that used lightweight EPS. Jiang et al. [11] and Tomlinsons and Fam [12] studied the behaviour of sandwich panels with XPS core. Choi et al. [13] showed that the surface roughness of the EPS core improved the bending resistance. The authors [14] also showed that the bonding action can improve the composite action. It is generally observed from the literature that EPS or XPS has been the most used material for acting as a core in concrete sandwich

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panels. Research studies are also found in the literature that investigated the possibilities of using other materials as the core. Kandil et al. [15] have tried using palm bark as a core in concrete sandwich panels. Hartoni et al. [16] have used bamboo core in sandwich panels. Srivaro [17] have found that sandwich panels with a density lower than 500 kg/m^3 could be produced by using oil palm core and bamboo faces. Marani et al. [18] used phase changing material as a core and evaluated the thermal performance. Awan and Shaikh [19] have tried using a crumb rubber core in sandwich panels and found that the crumb rubber core improved the ultimate strength. Castillo-Laro et al. [20] have studied the performance of sandwich panel with reinforced foam concrete core. Salgado [21] used autoclaved aerated concrete core and observed adhesion properties at wythe-to-core interfaces. Recently Kontoleon et al. [22] observed that research on the evolution of fire through sandwich panels with polystyrene core has not been exhaustively carried out. Very importantly, the authors observed that the insulation stability was maintained only for non-load bearing walls and not for load-bearing walls. Hulin et al. [23] found that loss of bonding action occurred at the interface due to differential thermal expansion. Huang et al. [24] found that the axial load capacity decreased by 88.5 % after exposure to one-side fire exposure for 4-hour. Having made these observations, it can be stated that under extreme fire conditions, in particular on both sides of the panels, the behaviour of sandwich panels with polystyrene core has to be studied in detail for practical applications. To overcome this shortcoming, in this paper, it is proposed to use pervious concrete core in sandwich panels. Pervious concrete is a no-fines (no-sand) concrete or concrete containing only less quantity of fines. Pervious concrete is porous and allows water to penetrate through it (Fig. 2). The mixed design of pervious concrete requires a balance between the strength and the required hydraulic property. Hitherto, researchers have not explored the effect of using pervious concrete core in concrete sandwich panels. Some studies on pervious concrete can be found elsewhere [25 – 29].

Fig. 2. Pervious concrete

It is also important to note that, in comparison with a polystyrene core, the pervious concrete core would not reduce a dead load of sandwich panels. However, the dead load of the pervious concrete core can be kept minimum by using lightweight aggregates. As per ASTM C330 [30], the density of lightweight coarse aggregates should be 880 kg/m^3. The literature review indicated that lightweight aggregates with density in the range of 231 – 443 kg/m^3 can be produced by using expanded clay or pumice [31 – 33]. ACI 533 [34] has mentioned that concrete with a density in the range of 256 – 800 kg/m^3 can be used as insulating material. Utilization of lightweight aggregate in pervious concrete can lead to core density in the range recommended by the ACI 533 [34]. Studies that investigated the influence of stainless steel wire mesh and the type of faces on the behaviour of sandwich panels produced using polymer faces can be found elsewhere [35, 36].

This paper is the first of its kind to investigate the effect of using pervious concrete as a core in concrete sandwich panels consisting of thin mortar wythes and truss shear connectors. This paper experimentally investigates the flexural behaviour of concrete sandwich panels with a pervious concrete core. The variables considered are the type of loading conditions, core thickness and a number of shear connector lines. Four different shear span to depth ratios are considered to determine they influenced the behaviour of the tested panels. Twelve specimens are cast and tested. First crack load, ultimate load, load-displacement behaviour, load-strain behaviour of wythes and load-strain behaviour of wire mesh are obtained from the tests. The ultimate strength of the tested panels with shear connectors is predicted using the equation given in ACI 318 [37] and compared with the test results.

2. EXPERIMENTAL DETAILS

2.1. Materials and casting procedure

In the experimental program, 12 small-scale sandwich panel specimens with two mortar skins of average thickness 25 mm and one pervious concrete core were tested till failure. The test variables were core thickness, number of shear connector lines and type of loading conditions. Two core thicknesses of 50 mm and 80 mm, two numbers of shear connector lines 3# and 6#, and two types of loading conditions (three-point and four-point bending) were considered. The shear span to depth ratios considered were in the range of 2.50 – 4.75. The mix proportion of mortar was 1:6 in the order of Ordinary Portland Cement (OPC) and manufactured sand (M-sand) with water-cement ratio of 0.45. The average compressive strength of the mortar was 12.5 MPa. Steel wire mesh with a grid size of 50 × 50 mm was used as reinforcement in the skins. Truss type shear connectors were used to connect the wythes. The trusses were formed by bending a single steel wire of diameter 3 mm in a zig-zag pattern (Fig. 3).

Fig. 3. Shear connector trusses tied with bottom wire mesh

The trusses were oriented along the spanning direction of the panels. The truss members were inclined at an angle of 45 °C and tied to the top and bottom wire meshes using a binding wire. The average tensile strength of the wire was 615 MPa. The mix proportion for pervious concrete was

1:6:0.45 in the order of OPC, coarse aggregate (gravel) and water-cement ratio. The size of coarse aggregate particles was in the range of 7 – 10 mm. The mass density was 1600 kg/m^3 and the compressive strength was 8.8 MPa. A steel mold of inner dimension 1000×300×100/130 (length × breadth × depth) was taken. First, the bottom wire mesh tied to nodes of inclined members of the trusses was placed inside the mold and cement mortar was poured for a thickness of 25 mm to form the bottom wythe. The cover for the wire mesh was 15 mm. The mortar layer was allowed to be set. The pervious concrete was poured on top of the bottom layer for a thickness of 50/80 mm. On the top of the pervious concrete core, the top wire mesh was placed and the nodes of inclined members of trusses were tied with the top wire mesh. Mortar was then poured on top of the core to a thickness of 25 mm to form a top wythe. The panel specimens were cured for 28 days. The details of panel specimens are given in Table 1. In Table 1, the specimens are identified as PXSY-Z, where, the letter P stands for the panel, X denotes the core thickness, the letter S stands for shear connector, Y denotes the number of shear connector lines and Z indicates the type of loading conditions. For example, P50S3-I indicates a panel with a 50 mm thick core, three numbers of shear connector lines and subjected to three-point bending.

2.2. Panel geometry and test setup

The specimen dimension was 1000×300×100/130 mm (length×width×thickness). The schematic of the test setup for four-point bending is shown in Fig. 4.

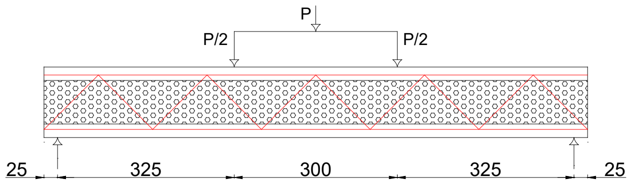


Fig. 4. Test setup for four-point bending

Simply supported boundary conditions were simulated. The loading was applied using a hydraulic jack at a loading rate of 0.1 kN/min. In three-point bending tests, the loading was distributed across the width of the specimen using a solid steel round bar of diameter 30 mm. In four-point bending tests, the point loading was transferred to the specimens using a rigid steel girder and distributed through two solid steel round bars across the width of the panels to produce two line loads. Strain gauges were affixed on the surface of the top and bottom wythes at the mid span. The strain in the bottom wire mesh (along the spanning direction) was also measured at the mid span. The deflection was monitored at the mid span. The typical panel in the test setup for the three-point bending test is shown in Fig. 5.

3. TEST RESULTS AND DISCUSSION

3.1. Failure modes

Test results showed that the failure pattern of panels is influenced by the number of shear connector lines and core thickness. An earlier study [38] on the shear transferring ability between the wythes of sandwich panels indicated that the number of shear connector lines and core thickness influence the through-thickness shear behaviour which ultimately affect the composite action of the panels. Typical failed panels are shown in Fig. 6.

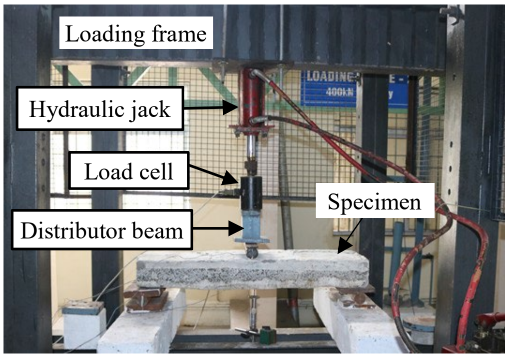
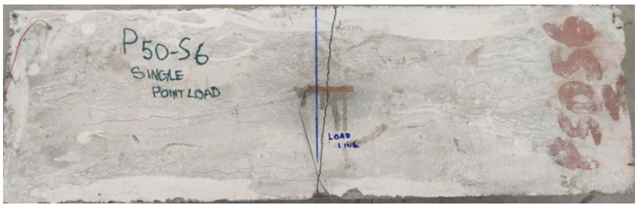


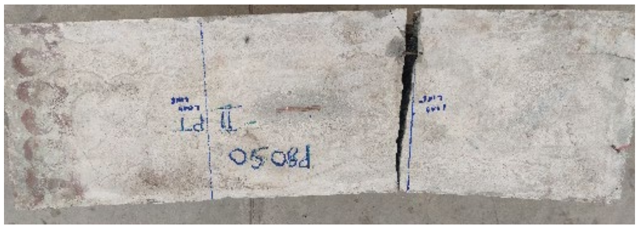
Fig. 5. Specimen in the test setup



bottom skin

side view

a



bottom skin

side view

b

Fig. 6. Typical failed panels: a – P50S6-I; b – P80S0-II

The specimen was considered to have failed when it could not resist further increase in the magnitude of loading. For all the tested specimens, only a few cracks formed at failure.

3.1.1. Failure under three-point bending

Under three-point bending, the specimens without shear connectors (P50S0-I and P80S0-I) failed due to bottom wythe separation from the core. Bottom wythe separation occurred first and then a flexural crack initiated in the core and extended till the top wythe and caused the failure. The panels without shear connectors did not achieve composite action and the failure was brittle. The reason for this type of failure could be explained as follows.

Table 1. Specimens details

ID	Core thickness, mm	Total thickness, mm	# of shear connector lines	Reinf. %	Loading type	ID	Core thickness, mm	Total thickness, mm	# of shear connector lines	Reinf. %
P50S0-I	50	100	0	0.12	Three-point bending	P50S0-II	50	100	0	0.12
P50S3-I			3			P50S3-II			3	
P50S6-I			6			P50S6-II			6	
P80S0-I	80	130	0	0.09		P80S0-II	80	130	0	0.09
P80S3-I			3			P80S3-II			3	
P80S6-I			6			P80S6-II			6	

Since the shear transfer between the two wythes is provided by the connectors and the bonding at the wythe-to-core interfaces, in these panels due to the absence of the shear connectors, only the bonding force would have initially provided composite action. However, after the bonding force was exceeded, a slip occurred at the interface. This failure type can be compared with that reported by Gara et al. [39] where the study was conducted on panels with no-shear connectors. The tested panels failed by rupture of the bottom wythe and the core. However, the was no significant wythe separation observed. In the present study, the wythe separation was therefore attributed due to poor bonding of pervious concrete with the wythe when compared with polystyrene bonding with the wythe. This observation clearly indicated the non-reliability of the bonding action in achieving the composite action in pervious core sandwich panels.

The specimens with shear connectors (P50S3-I, P50S6-I, P80S3-I and P80S6-I) failed by forming cracks in the bottom wythe. In specimen P50S3-I, the flexural crack initiated in the bottom wythe and extended only up to the bottom wythe-to-core interface. The flexural crack did not extend into the core. However, it was noted that, in specimens P50S6-I, P80S3-I and P80S6-I the flexural crack extended into the core to a depth of more than half the thickness of the core. These observations indicate that as the number of shear connector lines was increased and with increased thickness of the core, the flexural crack that initiated in the bottom wythe propagated into the core also. Vertical extension of flexural crack toward the extreme compressive fiber can be considered to indicate an increased level of composite action. Therefore, it can be stated that the degree of composite action of the sandwich panel with pervious concrete core improved with the increased number of shear connector lines and increased core thickness of the panel. For all the specimens with shear connector trusses, the wythe and core interfaces were intact until failure, and the failure was in flexural mode. Comparison with the earlier studies [4, 5] showed that concrete sandwich panels with wire mesh and truss shear connectors behaved similarly to the panels tested in this study. However, in their study, the flexural cracks were distributed along the span due to good bonding between the wythes and the EPS core. The present results therefore indicate that more shear connector lines are required for enhanced composite action when the pervious core was used.

3.1.2. Failure under four-point bending

The failure pattern of panels (with and without shear connectors) under four-point bending was found to be almost similar to that under three-point bending. The specimens without shear connectors failed due to wythe separation from the core. In the specimens with shear connectors (P50S3-II, P50S6-II, P80S3-II and P80S6-II), the wythe-to-core interfaces were found to be intact until failure and the panels failed in flexural mode. For all the panels with shear connectors, flexural cracks initiated in the bottom wythe under one of the load lines and extended in to the core for a depth of more than half the thickness of the core, similar to most of the panels under three-point bending. All the panels with truss shear connectors achieved composite action as evidenced by intact wythe-to-core interfaces. The observations clearly indicated that the truss shear connectors are required for achieving composite action in sandwich panels with a pervious concrete core. The bonding action at wythe-to-core interfaces, however, cannot be relied upon completely to transfer the shear stresses across the thickness of the panel. This behaviour is found to be similar to that of sandwich panels with polystyrene core [40] and FRP connectors.

3.2. First crack load/moment

The first crack load and cracking moment of the tested panel specimens are given in Table 2. For discussion, the specimens provided with truss shear connectors are only considered.

Table 2. Details of sandwich panel specimens

ID	First crack load, kN		Cracking moment, kNm		Ultimate load, kN		Max bending moment, kNm		Shear span/thickness	
	3-point bending	4-point bending	3-point bending	4-point bending	3-point bending	4-point bending	3-point bending	4-point bending	3-point bending	4-point bending
P50S0	0.3	1.0	0.07	0.2	2.3	3.2	0.5	0.5	4.75	3.25
P50S3	2.3	6.5	0.5	1.1	7.3	15.5	1.7	2.5		
P50S6	5.6	9.9	1.3	1.6	11.3	18.0	2.7	2.9		
P80S0	9.6	3.0	2.3	0.5	13.3	4.5	3.2	0.7	3.65	2.50
P80S3	11.1	13.5	2.6	2.2	15.4	23.8	3.7	3.9		
P80S6	13.5	16.5	3.2	2.7	17.3	24.8	4.1	4.0		

The first crack occurred under one of the loading lines under both types of loading conditions considered.

3.2.1. Influence of loading conditions

Theoretically, the cracking moment of the panel with a given core thickness and a number of shear connector lines is constant (tensile reinforcement percentage being constant). However, results showed that, for specimens with a 50 mm thick core, the cracking moment was higher under four-point bending than under three-point bending, and for specimens with an 80 mm thick core, the cracking moment was higher under three-point bending than under four-point bending. The reason for this was attributed due to the influence of shear span to depth ratio and the thickness of the core. Reducing the shear span to depth ratio decreased the cracking moment capacity of the panels.

In the study reported by Daniel Ronald Joseph et al. [6] under punching and bending, the results showed that the cracking moment was more in panel subjected to punching than under bending. The reason was attributed due to different magnitudes of principal stresses and the combined effect of flexural and shear stresses. The span to depth ratio was 6.0 in the case of bending, and it was 9.3 in the case of punching. Both the ratios were higher in magnitude indicating susceptibility to flexural failure mode. However, in the present study, the ratios were in the range of 2.5 – 4.75. The ratios were less for four-point bending tests. 80mm core specimens (with span to depth ratio in the range of 2.5 – 3.65) behaved similarly to that reported by [6]. But the specimens with span to depth ratio in the range of 3.25 – 4.75 behaved the opposite. The reason for this observation was attributed to the fact that earlier study [6] did not simulate exactly the three point bending; instead, punching or point load was considered.

3.2.2. Influence of the number of shear connector lines

Test results showed that the cracking moment increased with an increase in the number of shear connector lines. Under four-point bending, for 50 mm thick core panels, as the number of shear connector lines was increased from three to six, the cracking moment increased by 45 %. However, with an 80 mm thick core, increasing the shear connector lines from three to six increased the cracking moment by only 23 %. Under three-point bending, for 50 mm thick core panels, as the number of shear connector lines was increased from three to six, the cracking moment increased by 260 %. However, with an 80 mm thick core, increasing the shear connector lines from three to six increased the cracking moment by only 23 %. It is clear from these observations that, under both types of loading conditions considered, as the thickness of the core is increased, the cracking moment dependency on the number of shear connector lines is reduced. However, with EPS core, test results [6] showed that the dependency on the number of shear connector lines decreased with increased core thickness. These observations lead to conclude that, for achieving higher cracking moment capacity, either the number of shear connector lines or the thickness of the core has to be increased, and the cracking moment dependency on the number of shear connector lines is influenced by the core material (based on bonding characteristics with the wythes).

3.2.3. Influence of core thickness

Test results showed that, for specimens with three shear connector lines under four-point bending, increasing the core thickness from 50 mm to 80 mm, almost doubled the cracking moment capacity. With six shear connector lines, however, the cracking moment capacity increased by only 69 %. Under both types of loading conditions, with an increased number of shear connector lines, the cracking moment dependency on the core thickness is reduced.

3.3. Ultimate load/moment

3.3.1. Influence of loading conditions

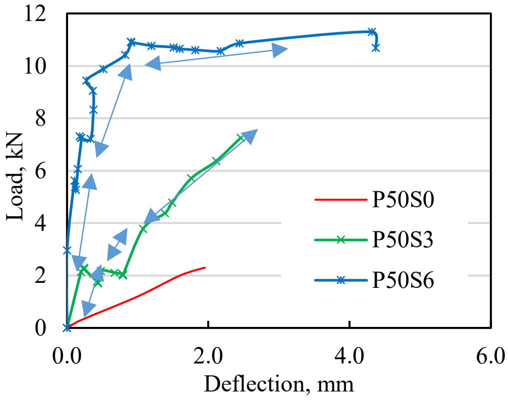
It was noted that, for a given core thickness and number of shear connector lines, the ultimate bending moment resisted under four-point bending was almost similar to that under three-point bending (except for P50S3). The maximum variation in the ultimate bending moment under different types of loading conditions was within ± 7 %, and this variation could be due to the statistical characteristics. The maximum shear force resisted by the panels varied based on three-point bending or four-point bending. Higher the bending moment resisted, the higher the shear force resisted by the panels. Also, increasing the core thickness and the number of shear connector lines improved the maximum shear force resisted by the panel.

3.3.2. Influence of the number of shear connector lines and core thickness

For both types of loading conditions, increasing the number of connector lines and core thickness improved the ultimate bending moment. Under four-point bending, for 50 mm thick core panels, as the number of shear connector lines was increased from three to six, the cracking moment increased by 16 %. However, with an 80 mm thick core, increasing the shear connector lines from three to six increased the cracking moment by only 3 %. Under three-point bending, for 50 mm thick core panels, as the number of shear connector lines was increased from three to six, the cracking moment increased by 59 %. However, with an 80 mm thick core, increasing the shear connector lines from three to six increased the cracking moment by only 11 %. The test results showed that the dependency of the ultimate moment capacity of the panel on the number of shear connector lines decreased as the core thickness increased, and the dependency of ultimate moment capacity on the core thickness decreased as the number of shear connector lines is increased. Studies [5, 40] in the literature have been conducted on wire mesh-reinforced sandwich panels. However, the effect of core thickness was examined only in Ref. [5] and hence the present results are compared with the study reported in Ref. [5]. The literature indicated that increased thickness improved the ultimate bending moment resisted. Similar observations are made in the present study also, and it is expected due to the higher lever arm available.

3.4. Load-deflection behaviour

The load-deflection curves are shown in Fig. 8 and Fig. 9 respectively. It was observed from Fig. 8 and Fig. 9 that the stiffness of panels increased with an increase in the number of shear connector lines and core thickness.



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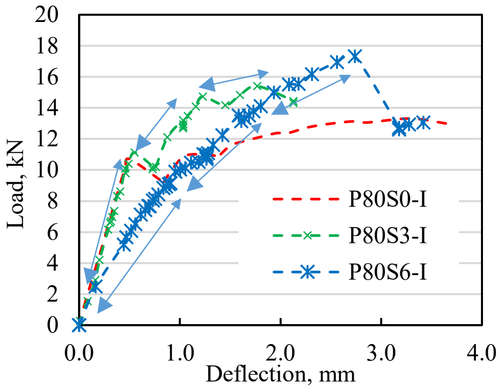
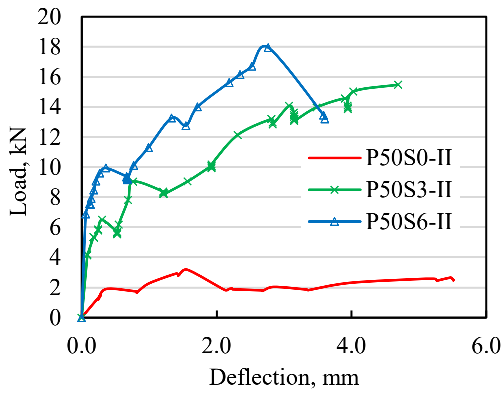


b

Fig. 8. Load-deflection curves under three-point bending: a – 50 mm core; b – 80 mm core



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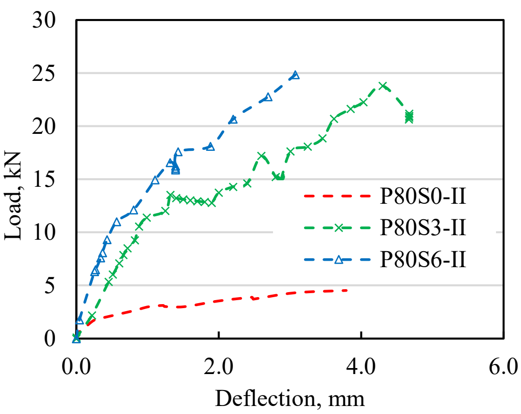

b

Fig. 9. Load-deflection curves under four-point bending: a – 50 mm core; b – 80 mm core

The load-deflection curves of the specimens provided with connector trusses exhibited tri-linear behaviour for both loading conditions, and can be assumed to be linear till cracking. The results are in concurrence with that observed in the reported studies [4, 5, 39]. Therefore, it was concluded that panels with pervious concrete core behave similarly to the panels with polystyrene core. This enables practical applications of this type of panels.

To examine the influence of a number of shear connector lines and core thickness on the stiffness, the initial stiffness of panels with shear connector trusses was determined from the four-point bending test results (Table 3).

Table 3. Stiffness of tested panels

ID	Stiffness, kN/mm
P50S3-II	10.9
P50S6-II	16.2
P80S3-II	11.9
P80S6-II	19.5

Only four-point bending tests were considered for initial stiffness determination because they provided constant bending moment between the load lines. The initial stiffness was determined based on a linear (approximate) portion of the curves. For panels with a 50 mm thick core, increasing the number of shear connector lines from three to six improved the initial stiffness of the panels by 49 %, and for panels with an 80 mm thick core, increasing the number of shear connector lines from three to six improved the initial stiffness of the panels by 64 %. This observation indicated that the dependency of initial stiffness on the number of shear connector lines increased with an increase in the core thickness of the panels. For panels with three shear connector lines, it was observed that increasing the core thickness from 50 mm to 80 mm increased the initial stiffness by 9 %, and for panels with six shear connector lines, it was observed that increasing the core thickness from 50 mm to 80 mm increased the initial stiffness by 20 %. The dependency of initial stiffness on the core thickness was found to be increasing with an increase in the number of shear connector lines. The observations also indicated that the initial stiffness of the panels was predominantly dependent on the number of shear connector lines than on the core thickness.

3.5. Load-strain behaviour of wythes and bottom wire mesh

Typical load-strain curves obtained for the top and bottom wythes of the tested specimens are shown in Fig. 10 and Fig. 11. In these figures, tensile strain (bottom wythe) is a negative and compressive strain (top wythe) is positive. It was observed from Fig. 10 that, for a specific applied load magnitude, there was no significant difference seen in the bottom wythe strain variation for both core thicknesses considered. Increasing the core thickness, however, decreased the compressive strain value achieved in the top wythe (Fig. 10). Similar observation was made when the number of shear connector lines was increased from three to six (Fig. 11). Since the tensile stresses in the bottom wythe is expected to be resisted by the mesh reinforcement the strain variation in the bottom wythe is not of our interest.

The curves for the bottom wythe are thus not considered for discussion; nevertheless, they are shown in the figures only for completeness. Test results showed that increasing the number of shear connector lines or the thickness of the core decreased the compressive strain achieved in the top wythe. Decrease in the compressive strain in the top wythe clearly indicated improved composite action and stiffness of the panel. The typical load-strain behaviour of the bottom wire mesh is shown in Fig. 12.

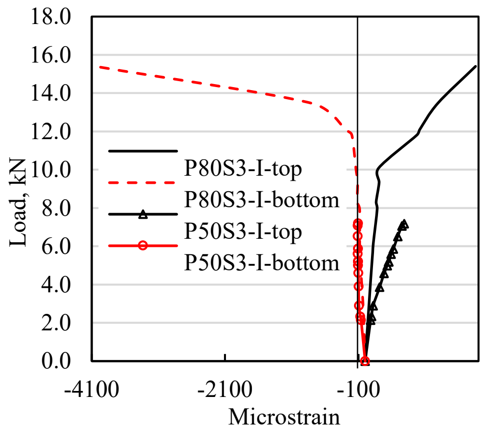

Fig. 10. Effect of panel thickness on the wythe strain

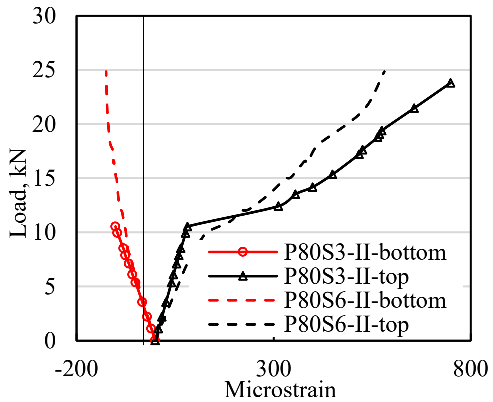

Fig. 11. Effect of the number of shear connector lines on wythe strain

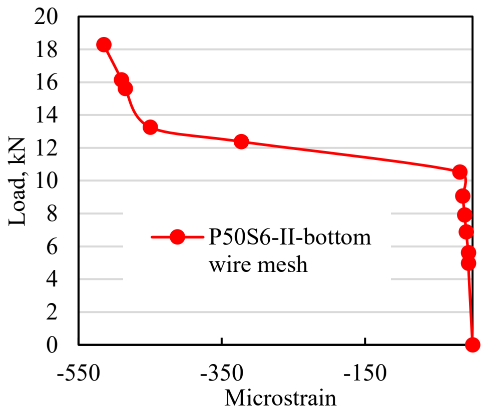

Fig. 12. Typical load-strain behaviour of bottom wire mesh of P50S6-II

The strain value achieved in the bottom wire mesh was tensile, as expected, and was found to be effective until failure. This indicated that the wire mesh bonded well with the surrounding mortar and effectively resisted the applied loading until failure. An increase in the applied loading linearly increased the strain value achieved in the bottom wire mesh until cracking. Beyond the first crack load, the strain value increased by a large value clearly indicating the effective participation of the wire mesh in resisting the tensile stresses after cracking of the bottom wythe. The wire mesh did not yield as the maximum strain observed was found to be well below the yielding strain (3250μ). This observation indicated that the ultimate failure of the panel was due to the strength limitation of the mortar used for casting the wythes.

4. ANALYTICAL PREDICTION

In this section, the flexural strength of the tested specimens with shear connector trusses was predicted based on the equation given in ACI 318 [37] for predicting the flexural strength of reinforced concrete beams or one-way slabs. The theoretical bending moment equation as given in ACI 318 [38] is given below. Theoretical bending moment capacity (see APPENDIX for nomenclature):

$$(M_n) = f_y A_{st} \times \left(d - \frac{\beta_1 c}{2}\right). \tag{1}$$

The specimens subjected to four-point bending were only considered for prediction. In the calculation, the tensile strength of mortar was neglected (assuming a cracked section) and the bottom mesh reinforcement was alone assumed to resist the tensile stress. Since the bonding action at wythe-to-core interfaces cannot be relied upon to resist the applied flexural loading, the stiffness of the pervious concrete core was not considered to contribute to the stiffness of the panel. The top wire mesh and top wythe mortar resisted the compressive stress.

The equation given in ACI 318 [37] does not consider the effect of a number of shear connector lines. Therefore, for both numbers of shear connector lines considered in the present study, the predicted flexural strength was the same. The predicted flexural strength of the specimens was compared with the experimental strength in Table 4.

Table 4. Experimental and predicted flexural strength

ID	Ultimate flexural strength, kN		Strength ratio
	Experiment	Predicted	
P50S3-II	15.5	13.35	0.86
P50S6-II	18.0		0.74
P80S3-II	23.8	18.22	0.77
P80S6-II	24.8		0.73

It was observed from Table 4 that the equation given in ACI 318 [37], in general, underestimated the flexural strength of the tested specimens by 23 % (average). Concerning the predictability of ACI 318 [37], the following points were noted. First, the number of shear connector lines influenced the predicted flexural strength, and observations showed that an increase in a number of shear connector lines decreased the strength ratio indicating poor predictability. Increasing the number of shear connector lines should improve the degree of composite action, and therefore, the sandwich panel would tend to behave more like a solid panel leading to strength ratio values closer to 1.0. However, a contradictory observation is made for both core thicknesses considered. Secondly, for a given number of shear connector lines, the predictability of ACI 318 [37] decreased (as indicated by decreased strength ratio) with an increase in thickness of the core. The analytical study in general showed that the influence of a number of shear connector lines has to be explicitly included in the equation for better predictability. Since the present analytical study involved only a limited number of specimens, several experimental and analytical studies are required to make a conclusive statement.

5. SUMMARY AND CONCLUSIONS

This paper presents the experimental studies conducted on innovative small-scale concrete sandwich panel specimens with pervious concrete core. The conclusions arrived based on the experimental studies are summarized below.

1. Pervious concrete can be used as a core in concrete sandwich panels.
2. Truss shaped shear connectors are required to achieve the composite action of the sandwich panels with the pervious concrete core.
3. Bonding action at a wythe-to-core interface cannot be relied upon.
4. Increased thickness of the core and increased number of shear connector lines provided higher ultimate strength of the panels.
5. Towards developing design guidelines for practical application, further experimental and analytical studies are required on the behaviour of prototype concrete sandwich panels with pervious concrete core produced using lightweight aggregates under different loading conditions, and for determining the effect of using different mix proportions of the pervious concrete core. These can be considered as a future area of research.

APPENDIX. Typical calculation for strength prediction using ACI [38], Panel ID: P80S3/P80S6

Breadth of panel	300 mm
Thickness of wythe	25 mm
Thickness of core	80 mm
Total thickness of panel	130 mm
Effective cover	12.5 mm
Effective depth (*d*)	117.5 mm
Grid size of wire mesh	50×50 mm
Yield strength of wire mesh (f_y)	615 MPa
Diameter of wires	3 mm
Number of wires	6
Area of tensile reinforcement	$A_{st} = 6 \times \frac{3.14\times3^2}{4} = 42.39$ mm^2
Compressive strength of mortar (f_c)	12.5 MPa
$\beta_1 c = \frac{A_{st} f_y}{0.85 f_c b} = \frac{42.39\times615}{0.85\times12.5\times300} = 8.18$ mm	
Theoretical bending moment capacity	$M_n = f_y A_{st} \times \left(d - \frac{\beta_1 c}{2}\right) = 615 \times 42.39\left(117.5 - \frac{8.18}{2}\right) =$ 2956581.69 Nmm = 2.96 kNm
Theoretical ultimate flexural load	$P_{th} = \frac{2\times2.96}{0.325} \approx 18.22$ kN

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