Insulated precast concrete sandwich panels under punching and bending

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- Precast concrete sandwich panels are being considered as an alternative to conventional construction materials for use in housing structures in India because of beneficial factors, including thermal and acoustic properties, affordability, and sustainability.
- This article studies the effects of punching and bending testing on two prototype precast concrete sandwich panels and compares the results to an analytical study of predicted loading capacities.

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In developing countries such as India, providing housing for all citizens is an essential requirement, and it may be considered a factor in the human development index, which Wikipedia defines as a statistic composite index of life expectancy, education, and per capita income indicators, which are used to rank countries into four tiers of human development.

At present, housing systems in India are predominantly built with materials such as burned clay bricks, hollow blocks, and granite stones. Most of these housing systems consist of load-bearing brick masonry walls and reinforced concrete floors. They do not have adequate lateral-load-resisting elements and are, therefore, vulnerable to earthquakes. These shortcomings in current construction practices have led researchers to look for innovative techniques to construct earthquake-resistant housing systems that are both lightweight and economical to build.

In addition, to mitigate the possible increase in indoor temperatures due to climate change and to achieve thermal and acoustic comfort inside the building, researchers are also looking for ways to make the walls and floors serve the dual purpose of insulating and load-transferring elements. With this goal, housing systems using lightweight insulated concrete sandwich panels are an attractive alternative to conventional housing systems.

Concrete sandwich panels consist of two reinforced concrete layers, called wythes, that are separated by a less dense core made of materials such as expanded polystyrene or extruded polystyrene, which impart insulating properties and lightweight characteristics to the panel. These concrete sandwich panels, which are also called insulated structural panels, may be precast and, hence, have the advantages of precast concrete technology and lightweight structural panels. Precast concrete sandwich panels act as thermal insulators thereby reducing the energy required to maintain ambient conditions within the house and also provide acoustic comfort. Housing systems that use precast concrete sandwich panels have comfortable ambient conditions, are less vulnerable to earthquakes, and have structural, economic, social, and environmental benefits.[1,2] The use of industrial waste—such as fly ash, slag cement, or similar recycled materials—to partially replace cement in the concrete helps to achieve sustainability, thus making residential housing systems using precast concrete sandwich panels greener and more economical. Wire mesh (also known as welded-wire reinforcement) or reinforcing bars may be used as reinforcement in the wythes, and shear connectors are used to connect the top and bottom wythes. The degree of composite action of precast concrete sandwich panels depends on the type of shear connectors, which may be discrete or continuous. Discrete shear connectors are provided at predefined locations; truss-type continuous shear connectors are oriented along the longitudinal (spanning) direction of the panel.

In reinforced concrete slabs, punching load may arise due to the use of heavy machines, an overhead water tank with one corner placed on a pedestal that rests on the roof slab, the slabs being directly supported on columns without beams (flat plates), or accidental falling of heavy objects. Research studies on the punching behavior of conventional solid reinforced concrete slabs and design equations for practical applications are available in the literature.[3–5,6–10] Even though the available experimental and analytical studies on precast concrete sandwich panel behavior under axial or eccentric compression and out-of-plane flexural loading indicate that there is a large potential for using these panels as load-bearing walls and floors in housing systems, studies that investigate the punching behavior of precast concrete sandwich panels are scant.[11–32] Furthermore, most of the available research studies are concerned with the behavior of precast concrete sandwich panels with a wythe thickness of 40 mm (1.57 in.) or more; however, previous studies by the authors indicate that sandwich panels may still achieve high out-of-plane flexural load-carrying capacity with a wythe thickness of 25 mm (0.98 in.) by providing solid edges along the spanning direction of the panel.[20–22] These types of precast concrete sandwich panels using thin wythes are being used for the construction of low-cost residential houses in India. Andhra Pradesh State Housing Corp. has constructed nearly 500 houses using thin-wythe precast concrete sandwich panels for Hudhud cyclone victims (**Fig. A1**, for appendix figures, go to https://www.pci.org/2019Mar-Appx). It is also important to note that the thin wythes of precast concrete sandwich panels with wire mesh as reinforcement are structurally similar to ferrocement, and a previous study by the authors indicates that the cracking behavior of these precast concrete sandwich panels is also similar to that of ferrocement.[20] Another study also indicates that there is a large potential for using these panels as load-bearing walls in housing systems.[23] To facilitate the assembly of wall and roof panels, different types of jointing systems are also being developed at the Council of Scientific and Industrial Research (CSIR) Structural Engineering Research Centre in Chennai, India. It is proposed that these jointing systems be embedded in solid concrete regions provided near the end regions of precast concrete sandwich panels to provide efficient structural joints in housing systems. These solid regions may act as thermal bridges and may lower the thermal efficiency of the panels; however, they are required to facilitate assembling and jointing of the precast concrete sandwich panels. An experimental study indicated that in these types of panels, the composite action of the panels is improved due to the solid regions near the supporting/reaction edges, which increases the ultimate in-plane compressive loads.[23]

The aim of the present experimental and analytical studies is to compare the flexural behavior of prototype precast concrete sandwich panels with a thin wythe thickness of 25 mm (0.98 in.) subjected to four-point bending and punching. The experimental study consisted of flexural testing of two prototype precast concrete sandwich panels simply supported along their edges. The analytical study included examining the predictability of the strength equation specified in the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*[6] for conventional solid reinforced concrete slabs to determine the ultimate flexural load-carrying capacity of the panels.

Precast concrete sandwich panel behavior

The flexural behavior of precast concrete sandwich panels under one-way bending may be expected to be similar to that of composite beams. In the structural design of composite beams subjected to flexural load, it is common to assume that the wythes resist bending moment and the core resists shear. The only difference between ordinary beams that obey elastic bending theory and composite beams is that the deflection due to shear deformation of the core is not neglected in the latter. Based on the interaction between the wythes and the core, composite beams may be classified into three types: fully composite, semicomposite, and noncomposite.[33] **Figure 1** shows the bending stress distributions across the depth of the panel cross section based on linear elastic bending theory for the three types of composite beams. To achieve fully composite action, the shear transferring capacity between the top and bottom wythes should be ensured.

Experimental program

The experimental program consisted of two prototype precast concrete one-way sandwich panels tested under four-point bending and punching. Panel geometry, wire mesh size, wythe thickness, and expanded polystyrene thickness were the same for the tested panels. The size of the wire mesh was 50 mm (1.97 in.), and the mesh wires and shear connectors were made of high-strength steel with an average yield strength of 650 MPa (94 ksi), as supplied by the manufacturer. Tensile tests on the wires of the mesh were conducted to determine their stress-strain characteristics. The gauge length of the wire considered for the tensile test was 500 mm (19.69 in.), and the maximum tensile elongation of the wire at failure was 8.17 mm (0.32 in.). **Figure A2** shows a portion of the tested wire after failure, and **Fig. A3** shows the typical stress-strain curve of the tested wires. Figure A3 indicates that the yield stress (proof stress) corresponding to 0.2% proof strain is greater than the yield strength of the wires specified by the manufacturer and that, in general, the behavior of the wire material is similar to that of mild steel. However, the percentage elongation of the tested wires is less than that of mild steel, which achieves nearly 25% strain. Similar observations were made in a previous study. This reduction in strain capacity is attributed to the stretching of the material.[34]

The bottom wythe of the panel contained conventional steel reinforcing bars together with the wire mesh as tensile reinforcement. The reinforcing bars (five 8 mm [0.3 in.] diameter) used in the bottom wythe of the panels were made of Fe 415 (60 ksi) grade steel. Self-consolidating concrete with an average cube compressive strength of 45.9 N/mm^2 (6.7 ksi) and an average flexural tensile strength of 4.3 N/mm^2 (0.6 ksi) was used to cast the wythes. The average thickness of the top and bottom wythes and of the side cover along the longitudinal direction of the panels was 25 mm (0.98 in.). At the supported ends of the panels, expanded polystyrene was removed to form a 100 mm (3.9 in.) thick (at both longitudinal ends) section of solid concrete that were reinforced using two 8 mm diameter Fe 415 grade steel reinforcing bars. A previous study by the authors indicated that these solid regions improved the composite action of the panels.[23]

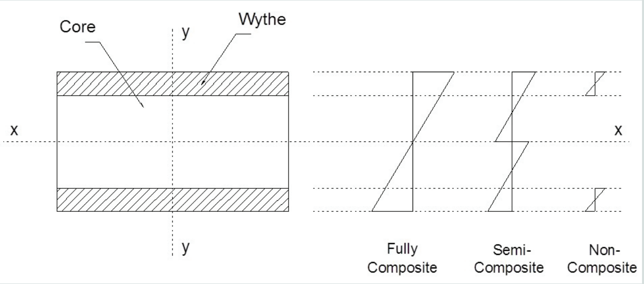

Figure 1. Bending stress distribution of composite beams.

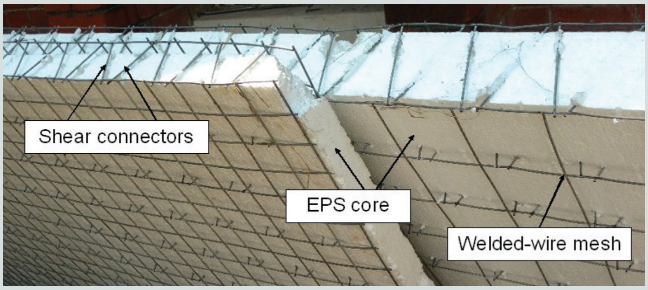

Figure 2. Typical panel used in the study. Note: EPS = expanded polystyrene.

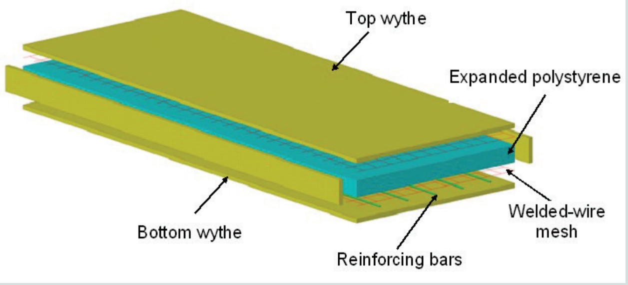

Figure 3. Exploded-view diagram of panel showing expanded polystyrene, wire mesh, and reinforcing bars.

Panel details and instrumentation

The dimensions (length × width × depth) of the tested panels were 3000 × 1220 × 150 mm (9.84 × 4.00 × 0.49 ft). **Figures A4** and **2** show a schematic sketch of the precast concrete sandwich panels considered and the typical expanded polystyrene panels used in this study. **Table 1** gives the details of the panels tested. **Figure 3** shows an exploded-view diagram of the panel with wire mesh and steel reinforcing bars in the bottom wythe (shear connectors are not shown).

The wire mesh was connected by truss-type continuous shear connectors that were oriented along the longitudinal (span-

Table 1. Details of panels tested

Specimen number	ID	Mesh size, mm	Dimensions (length × width × depth), mm	Thickness, mm: Wythe	Thickness, mm: Expanded polystyrene	Thickness, mm: Total	Reinforcing bars in bottom face	ρ_t	$\rho_{t,min}$
1	PB	50 × 50	3000 × 1220 × 150	25	100	150	Five 8 mm diameter	0.191	0.0011
2	PP	50 × 50							

Note: PB = panel tested in bending; PP = panel tested in punching; ρ_t = percentage of reinforcement provided; $\rho_{t,min}$ = required minimum percentage of reinforcement as per ACI 318 (2011). 1 mm = 0.039 in.

ning) direction of the panels; hence, the panel effectively resisted bending only in the spanning direction. Thirteen shear connector trusses were provided in each panel. The shear connector wires were inclined at an angle of 70 degrees to the horizontal and were welded to the wire mesh. Table 1 indicates that the tested panels had the recommended minimum percentage of tensile reinforcement.[6] The panels were simply supported at their edges and were subjected to four-point bending and punching. The loading was applied using a hydraulic jack, and, in the case of four-point bending, a rigid transfer girder was used to produce two line loads on the panel. The distance between the line loads was 900 mm (35.4 in.). In the punching load test, a 150 × 150 mm (5.9 × 5.9 in.) concrete cube was used to apply the punching load. **Figure A5** shows the test setup and the instrumentation details. Strain gauges with lengths of 30 and 5 mm (1.18 and 0.2 in.) were used to measure the strains on the concrete surfaces and the steel wires, respectively. A linear variable displacement transducer with a 50 mm (2 in.) range was used to measure the mid-deflection of the panels. **Figures A6** and **A7** show the panels in the test setup. The panel subjected to bending was labeled PB, and the panel subjected to punching was labeled PP.

Test results and discussion

First crack load, failure mode, and cracking pattern

Results of the panels tested under punching and bending are presented in this section. **Figures A8** and **4** through **6** show the cracks in the side web and the cracking pattern in the top and bottom wythes of the panels.

In the punching testing, the first crack in panel PP occurred in the bottom wythe at a load of 12.70 kN (2.86 kip) exactly under the loading point. Increasing the load resulted in the formation of new flexural and shear cracks in the sides and bottom wythes of the panel (Fig. A8 and 4). At a load of 14.0 kN (3.15 kip), a horizontal crack at the top wythe-expanded polystyrene interface occurred and its length increased as the applied load increased; however, it did not extend into the solid concrete regions provided near the supported edges. In the top wythe the cracks formed concentric circles, and in the bottom wythe the cracks formed radial lines. The concentric crack circles closed at one longitudinal edge of the panel but did not close at the other edge (Fig. 5). The horizontal crack at the wythe-expanded polystyrene interface occurred only in the side cover nearer to where the concentric crack circles closed. No such horizontal crack occurred in the other side cover of the panel. The panel failed by widening of the flexural cracks in the bottom wythe.

In the bending testing, the first crack in panel PB occurred at a load of 11.75 kN (2.64 kip) below one of the loading lines. With further increase in the applied load, new flexural cracks formed between the loading points (in the constant bending moment region) while the first crack increased in length and width. No cracks were seen in the shear span until loading reached 36.65 kN (8.24 kip). At this load, inclined shear cracks occurred nearer to one of the loading lines in the shear span. Increasing the applied load either widened the inclined shear cracks or resulted in the formation of new inclined shear cracks in the shear span. The panel failed by widening of the inclined shear cracks that ran between the loading point and the nearest support (Fig. A8).

The concrete side covers provided at the longitudinal edges of the panels transferred the shear (in conjunction with the truss-type shear connectors) across the depth of the panel by acting as elements to connect the top and bottom wythes. Two types of cracks occurred in the side cover: horizontal cracks at the wythe-concrete interface and flexural and shear cracks.

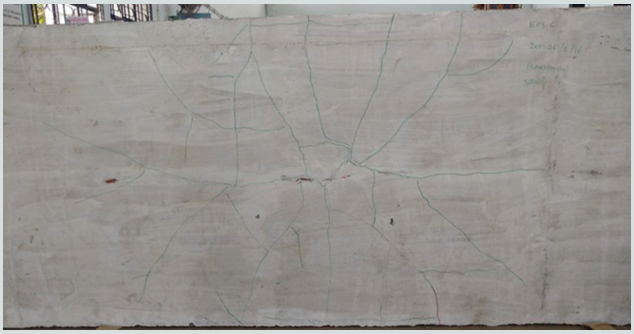

Figure 4. Cracks in bottom wythe of panel PP. Note: PP = panel tested in punching.

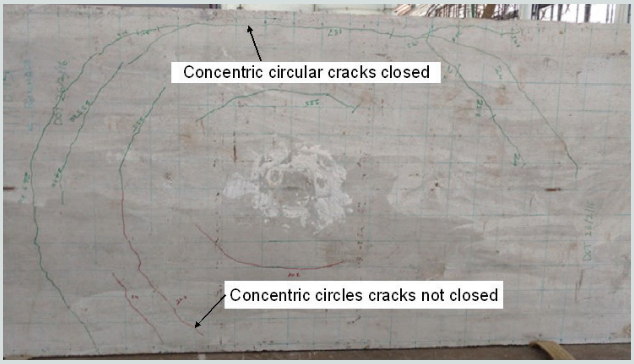

Figure 5. Cracks in top wythe of panel PP. Note: PP = panel tested in punching.

Figure 6. Cracks in bottom wythe of panel PB. Note: PB = panel tested in bending.

The formation of a horizontal crack at the wythe-expanded polystyrene interface occurred only during the punching load test. The formation of the horizontal crack may be due to loss of frictional resistance at the interface when it is exceeded by the shear stress caused by the applied load or by loss of moment of resistance provided by the side cover to the top wythe. Before the formation of the horizontal crack, the shear stress due to the applied loading was resisted by both the frictional resistance at the interface and the shear strength of the side cover. Because the side cover thickness is less than the thickness at the ends, its shear strength would be less compared with the frictional resistance at the interface. Therefore, it is logical to expect formation of horizontal cracks in the side covers at the wythe-expanded polystyrene interface as soon as the frictional resistance is overcome. However, the results of the punching load test show that the horizontal crack occurred only in one side cover. If cracking had been due to the loss of frictional resistance, it is expected to have occurred on both side covers. This fact, together with the observation that the horizontal crack occurred only near the side cover where the concentric crack circles closed, leads to the conclusion that the horizontal crack occurred due to loss of moment of resistance provided by the side cover to the top wythe. The formation of flexural and shear cracks in the side cover clearly indicates that the side cover behavior is similar to that of solid reinforced concrete beams subjected to flexural loading and therefore effectively transfers the shear stress across the depth of the panel. The cracks in the side covers of the panel tested under four-point bending also indicate that the side covers have a significant role in transferring the shear stresses between the top and bottom concrete wythes. These observations from the tested panels lead to the conclusion that the side covers form part of the shear-transferring elements and, hence, affect the degree of composite action of the panel. The effect on panel behavior for side covers with reinforcement similar to shear stirrups in conventional solid reinforced concrete beams may be found in the literature.[20] The formation of flexural cracks in the bottom wythe indicates that the shear connectors were effective to achieve composite action in the panels.

Both wythes showed specific crack formation patterns. In solid reinforced concrete slabs subjected to punching load, the critical yield line pattern consists of curved negative-moment lines on the compression side of the slab and radial positive-moment lines on the tension side of the slab.[35] In the top wythe of the panels (Fig. 5), cracks occurred in the shape of concentric circles with the application of the punching load. The radius at which these crack circles formed increased as the applied load increased. The maximum radius of the outer crack circle was nearly 1000 mm (39.37 in.). These cracks were due to tensile stresses; hence, it is clear that the top wythe was not subjected to only compressive stress, as in cylindrical bending of plates. Such cracks may occur due to local punching of the top wythe, which would result in relative deformation of the top wythe with respect to the bottom wythe. Local punching would have developed tensile stresses in the top wythe, causing these types of concentric crack circles. However, if local punching had occurred, further flexural cracks would not have formed in the bottom wythe of the panel. Local punching of the top wythe would also have resulted in a huge difference in the ultimate bending moment resisted by the panels, as discussed in the next section. Local punching would also have prevented further formation of concentric crack circles with larger diameters. The concentric crack circles that formed in the top wythe were exactly similar to the negative-moment yield lines that would be expected in solid reinforced concrete slabs subjected to punching. Therefore, the panel did not fail due to local punching failure of the top wythe and the shear connectors were efficient to transfer the shear forces to ensure composite action of the panels.

Cracks patterns in the bottom wythe of the panel were similar to circular fans, as would be expected in conventional solid reinforced concrete slabs subjected to punching load.[35] These observations indicate that the panel failed in flexure, similar to conventional solid reinforced concrete slabs under punching load, and that there may be an opportunity for using precast concrete sandwich panels with thin wythes as flat plates in housing systems. However, more experimental and numerical studies are required to make a conclusive statement.

The failure mode of the panel under four-point bending was similar to the classical shear failure of beams; however, solid reinforced concrete slabs are not normally governed by shear failure. No cracks occurred in the top wythe of the panel until failure. This is because, as indicated by the strain variations (see the Load-Strain Behavior section), the top wythe of panel PB was primarily subjected to compressive stresses similar to the cylindrical bending of plates and the top wythe effectively resisted the compressive stress. The cracking pattern in the bottom wythe of the panel (Fig. 6) indicates that the cracks primarily occurred in the shear span rather than in the constant bending moment region, meaning that the panel failed in shear mode. Further discussions on the failure mode of panel PB may be found in the literature.[20]

Comparison of the failure modes of the tested panels indicates that under punching the panel failed in flexure; however, under four-point bending the panel failed in shear. Note that the percentage of reinforcement was the same for both panels and satisfied the required minimum percentage.[6] However, the test results presented in this study indicate that the precast concrete sandwich panels with the same reinforcement percentage under different flexural loading conditions have different failure modes.

Load-deflection behavior

Figure 7 shows the load-deflection responses and the corresponding bending moment-deflection responses of the precast concrete sandwich panels tested in this study. It also shows the theoretical load-deflection responses of fully composite and noncomposite panels. Figure 7 indicates that the behavior of panels PB and PP was similar and linear up to an applied load of 7 kN (1.57 kip), which corresponds to an applied bending moment of 4.9 kN-m (3.6 kip-ft) in the case of punching and 3.15 kN-m (2.3 kip-ft) in the case of bending. Observations indicate that the load-deflection behavior of both panels may be considered to be linear up to the cracking load. The initial stiffnesses of the precast concrete sandwich panels tested under both types of flexural loading conditions were nearly the same as that of the theoretically fully composite panel. Therefore, in general, the type of the flexural loading conditions does not affect the linear behavior of the panels until formation of the first crack, at least for the panel geometry considered in the present experimental program. As the applied load is increased, the flexural stiffness of the panels reduces, which may be attributed to the formation and widening of cracks. However, the reduction in the flexural stiffness was greater for panel PP than for panel PB. This may be because panel PP could dissipate more energy by forming a number of flexural cracks in the bottom wythe. In panel PB, because the inclined shear connector wires were oriented nearly perpendicularly to the inclined shear cracks, they effectively resisted the applied load even though the concrete side cover failed. This may be the reason for the higher flexural stiffness (compared with panel PP) observed in the load deflection of panel PB beyond 20 kN (4.5 kip). The ultimate bending moments resisted by the panels subjected to different flexural loading conditions are nearly the same because the geometry and cross-sectional details of both panels are the same.

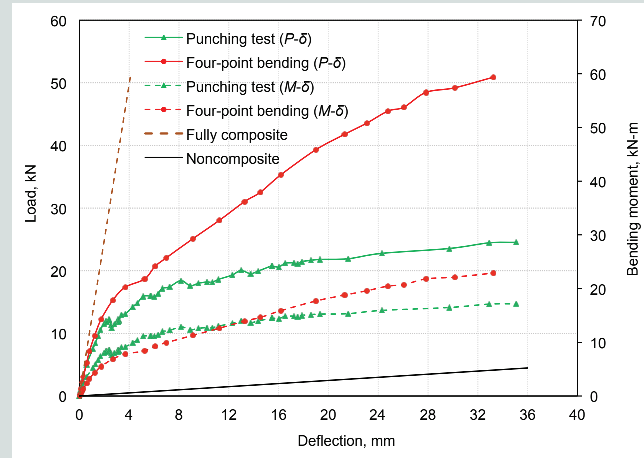

Figure 7. Load-deflection responses of precast concrete sandwich panels. Note: 1 mm = 0.039 in.; 1 kN = 0.225 kip; 1 kN-m = 0.737 kip-ft.

Load-strain behavior

Figures 8 and **9** show the strain variations obtained in the top wythes of the panels until failure. The tensile strain is positive, and the compressive strain is negative. Strain in the wythe surface of panel PP is marked as PC, and strain in the wythe surface of panel PB is marked as BC. The top wythe of panel PB was primarily subjected to compressive stress (except for lower values of tensile stress at BC1 and BC3) until failure. Therefore, the behavior of panel PB may be considered to be similar to that of beams subjected to cylindrical bending. To transfer some compressive stress into the top wythe, the shear connectors must be effective enough to create composite action of the panel. These observations indicate that panel PB achieved composite action due to the presence of shear connectors; however, as previously noted, the panel failed due to shear. The tensile strains at PC1/PC3 and compressive stress at PC2 show that the top wythe of panel PP was subjected to tensile and compressive stresses. At the loading point, the strain in the top wythe was compressive in nature. However, at cross sections located away from the loading point, tensile stresses occurred in the top wythe, which caused formation of tensile cracks in concentric circles. These cracks were curved negative-moment yield lines; thus, the cracks in the top wythe of the panel were similar to cracks in the compression side of a solid reinforced concrete slab subjected to punching.

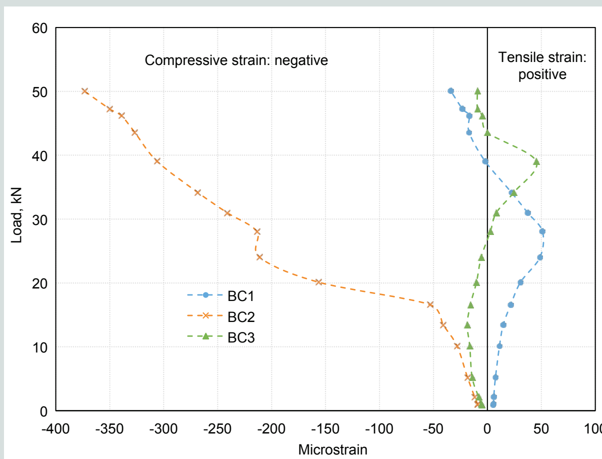

Figure 8. Strain variations in top wythe for bending test panel PB. Note: PB = panel tested in bending. 1 kN = 0.225 kip.

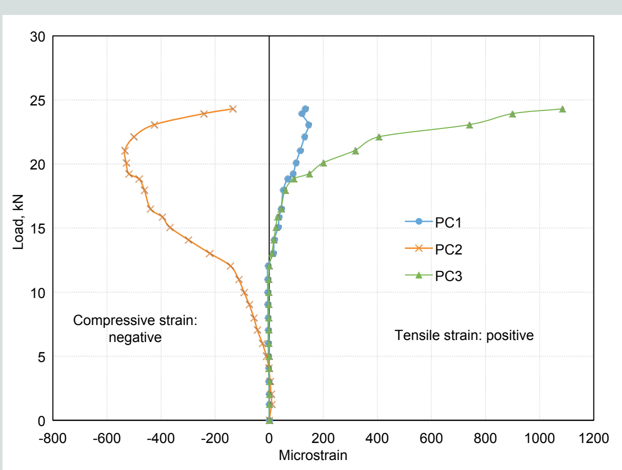

Figure 9. Strain variations in top wythe for punching test panel PP. Note: PP = panel tested in punching. 1 kN = 0.225 kip.

The tensile stress was greater at the outer rings (larger diameter) of the concentric circles than near the loading region, which may happen only when the shear connectors are effective. If the shear connectors are not effective, the strain values at cross sections located away from the loading region may not increase with an increase in the applied load because the top wythe might have failed due to local punching. The strain reversal observed at PC2 is attributed to the formation of new tensile cracks and widening of existing tensile cracks that relieved the stress—and, hence, the compressive strain—at PC2.

Figure 10 shows the strain variations measured in the wires and reinforcing bars present in the bottom wythes of the panels until failure. Strain measured in the wire mesh of panel PP is marked as PW, and strain measured in the reinforcement of panel PP is marked as PR. For panel PP, the load-strain behavior measured at PW and PR were similar until panel failure. For panel PB, the strain variation in the wire mesh (BW) and reinforcing bars (BR) were also similar, but the wire mesh achieved larger strain than the reinforcing bars. Furthermore, the strain in the wire mesh and the reinforcing bar increased rapidly once the load exceeded 16.5 kN (3.7 kip), followed by hardening behavior after 20 kN (4.5 kip).

This may be explained as follows. The axial tension (calculated with a lever arm of 133 mm [5.2 in.]) in one wire corresponded to a bending moment of 16.5 kN-m (12.2 kip-ft) is 2.23 kN (0.50 kip), which created an axial tensile stress of 586.9 MPa (85.1 ksi). At this stress, the strain in the wire (Fig. A3) was 0.015 mm/mm (0.015 in./in.), and observation from Fig. 10 showed that the measured strain in the wire due to the applied loading in the panel was nearly 0.015 mm/mm (0.015 in./in.) at this stress value. Beyond the applied load of 20 kN, there was no appreciable increase in the strain. This may be because the panel failed in shear in the shear span, but the strain was monitored at the midspan of the panel. Beyond 20 kN of applied load, the load-deflection response of panel PB was significantly different from panel PP, and a similar effect was seen in the load-strain variation measured in the wire mesh.

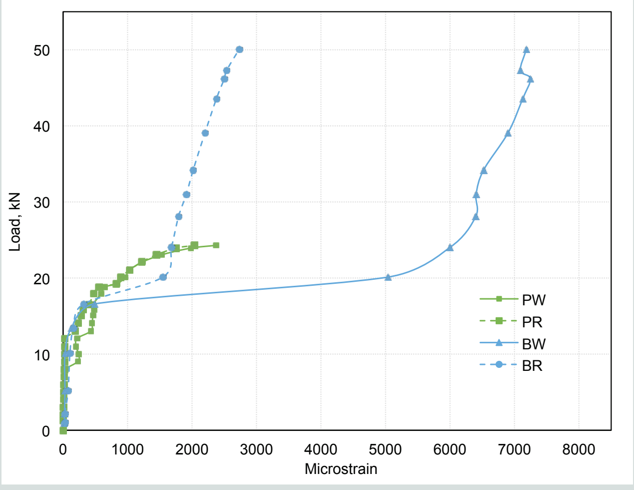

Figure 10. Strain variations in wire mesh and reinforcing bars of precast concrete sandwich panels. Note: BR = strain measured in the reinforcement of panel PB; BW = strain measured in the wire mesh of panel PB; PB = panel tested in bending; PP = panel tested in punching; PR = strain measured in the reinforcement of panel PP; PW = strain measured in the wire mesh of panel PP. 1 kN = 0.225 kip.

In general, for both loading conditions, the strain in the wire mesh and reinforcing bars increased after the first crack load, indicating that they effectively resisted the applied load after cracking of the bottom wythe. This also indicates that the bond between the bottom-wythe concrete and the reinforcement was satisfactory, and hence, both wire mesh and reinforcing bars effectively resisted the applied load. Therefore, in numerical simulations of flexural behavior of these types of precast concrete sandwich panels, the bond between the concrete and reinforcement may be modeled as perfect to predict the panel behavior under flexural loading conditions.

Analytical study

The principal tensile stresses at the extreme bottom fiber of the panels corresponding to the first crack load at the first crack location were determined using elastic cross-sectional properties. The calculated principal tensile stresses were expected to be equal to the flexural tensile strength of the concrete (4.3 N/mm^2 [0.6 ksi]). In the calculation, the concrete wythes were assumed to resist bending and the shear connectors were assumed to resist shear. Deflection due to shear deformation and the possible contribution of the expanded polystyrene for resisting bending and shear stresses were not considered in the calculations. The presence of wire mesh was not considered in the moment-of-inertia calculation. **Table 2** gives the elastic cross-sectional properties of the panel. **Table 3** gives the calculated cracking moment and principal tensile stresses. The predictability of the ACI 318 strength equation in determining the ultimate flexural load-carrying capacity of precast concrete sandwich panels is presented in this section. Table 3 shows that for both types of flexural loading conditions, the principal tensile stress was generally significantly lower than the flexural tensile strength of the concrete. This may be due to reasons such as the presence of shrinkage stresses, redistribution of shear stresses between flexural stresses, and local weakening of the cross section by transverse reinforcement.[36] It may also be due to statistical uncertainties in the material and geomet-

Table 2. Cross-sectional and material properties

Property	Magnitude
Width of panel b	1220 mm
Thickness of wythe	25 mm
Thickness of panel	150 mm
Center-to-center distance of wythes	125 mm
Moment of inertia (neglecting core) I	244.2 × 10^6 mm^4
Elastic section modulus Z	3.26 × 106 mm^3
Young's modulus E*	33,541 N/mm^2
Tensile strength of concrete f_t*	4.7 N/mm^2

Note: 1 mm = 0.039 in; 1 mm^3 = 5.93 x 10^{-5} in.^{3}; 1 mm^4 = 2.31 x 10^{-6} in.^{4}; 1 N/mm^2 = 0.145 ksi.

* According to IS 456:2000.

Table 3. Cracking moment and principal tensile stresses				
S number	Panel ID	Cracking load, kN	Cracking moment, kN-m	Principal tensile stress, MPa
1	PB	11.75	5.30	1.62
2	PP	12.70	8.89	2.72

Note: PB = panel tested in bending; PP = panel tested in punching. 1 kN = 0.225 kip; 1 kN-m = 0.737 kip-ft; 1 MPa = 0.145 ksi.

rical properties of the panel. Table 3 shows that the cracking moments, and hence the principal tensile stresses, were different for punching and bending loading conditions, even though the cross-sectional and geometric details of the two panels are the same. This may be attributed to the following reasons.

For panel PP, the first crack occurred exactly under the loading point (at the maximum bending moment) where the shear force was considered to be zero. The reasons stated earlier may explain the lower principal tensile stress for panel PP compared with the flexural tensile strength of concrete. With respect to panel PB, the principal tensile stress was lower than that of panel PP. This may be because the first crack occurred in panel PB at a cross section where both the bending moment and the shear force were maximum. Therefore, the first crack in panel PB may primarily be due to the combined effect of flexural and shear stresses causing mixed-mode fracture conditions, which may be the primary reason for the much lower principal tensile stress. Therefore, the occurrence of the first crack in these types of precast concrete sandwich panels also depends on the shear force distribution along the spanning direction. For the practical use of precast concrete sandwich panels as floor panels, structural designers should ensure that both shear and bending moment are not maximum at a specific cross section, as in the case of cantilever beams or slabs.

In the following, the theoretical ultimate flexural load-carrying capacities of precast concrete sandwich panels subjected to punching and bending are determined using the ACI 318 strength equation. The following assumptions were made:

- Dead load was not considered.
- The compressive force was resisted by the top wythe only, and the tensile force was resisted only by the wire mesh and reinforcing bars present in the bottom wythe.
- The tensile behavior of the wire mesh and reinforcing bars was considered to be elastic–perfectly plastic.
- The compressive stress distribution was uniform across the thickness of the top wythe.
- The panels achieved fully composite action.
- The neutral axis was at middepth of the panel cross section.

Figure A9 shows the assumed stress block. The effective cover of the tensile reinforcement was taken as 10 mm (0.4 in.), and the yield strength of the reinforcing bar is used conservatively for both reinforcing bars and wires.

To calculate the area of steel A_s for this example, the area of wire mesh and steel reinforcement are needed.

Number of longitudinal mesh wires = 26

Number of reinforcing bars = 5

Area of wires $= 26 \times \frac{\pi}{4}\left(2.2^2\right) = 98.8 \text{ mm}^2 \left(0.15 \text{ in.}^2\right)$

Area of steel reinforcing bars =

$5 \times \frac{\pi}{4}\left(8^2\right) = 251.2 \text{ mm}^2 \left(0.39 \text{ in.}^2\right)$

The ultimate bending moment capacity M_n as per ACI 318 is

$$M_n = A_s f_y \left(d - \frac{\beta_1 c}{2}\right)$$

$$\beta_1 c = \frac{A_s f_y}{0.85 f_c' b}$$

where

A_s = 350 mm² (0.54 in.²)

f_y = yield strength of reinforcing bar = 415 N/mm² (60,190.7 psi)

d = distance between extreme fiber in compression and tensile reinforcement = 135 mm (5.31 in.)

$\beta_1 c$ = depth of neutral axis

f_c' = compressive strength of concrete = 45 N/mm² (6526.7 psi)

b = width of the panel = 1220 mm (48.03 in.)

To calculate the depth of neutral axis $\beta_1 c$,

$$\beta_1 c = \frac{350 \times 415}{(0.85)(45)(1220)} = 3 \text{ mm } (0.12 \text{ in.})$$

$$M_n = (350)(415)\left(135 - \frac{3}{2}\right) = 19.29 \text{ kN-m } (14.2 \text{ kip-ft})$$

The ultimate load P_n corresponding to the ultimate bending moment was calculated as the following:

For bending, $P_{nb} = \frac{M_n \times 2}{0.9} = \frac{19.29 \times 2}{0.9} = 42.87$ kN (9.64 kip)

For punching, $P_{np} = \frac{M_n \times 4}{2.8} = \frac{19.29 \times 4}{2.8} = 27.56$ kN (6.19 kip)

The theoretical ultimate shear forces corresponding to theoretical ultimate bending moment capacity were

$V_{nb} = P_{nb}/2 = 21.44$ kN (4.82 kip) for bending

$V_{np} = P_{np}/2 = 13.78$ kN (3.09 kip) for punching

The calculations indicate that the predicted bending moment capacity and ultimate load based on the ACI 318[6] strength equation for panel PP (that failed in flexural failure mode) are comparable to the experimental results. However, the predictions are much greater than the experimental ultimate load and bending moment for panel PB. This may be because the ultimate failure of this panel was due to shear failure mode of the panel PB. Previous studies by the authors indicate that even if these types of precast concrete sandwich panels fail in flexure under bending, the predicted ultimate flexural load capacities based on the ACI 318 strength equation are too conservative, and hence the strength equations may have to be modified to develop semi-empirical formulas to predict the ultimate flexural load-carrying capacities of precast concrete sandwich panels.[20] However, the analytical study presented in this paper indicates that for punching load conditions, the predictions are comparable to the experimental ultimate loads when the panels fail in flexure. Further experimental and analytical studies in this area may be required to develop design guidelines.

Conclusion

This paper presents and discusses the results of experimental and analytical studies conducted on prototype precast concrete sandwich panels to determine the effects of different loading conditions, such as punching and bending, on the flexural behavior and failure modes of the panels. The following conclusions were reached:

- The type of flexural loading conditions, such as punching and bending, has a significant effect on the flexural behavior and failure modes of precast concrete sandwich panels; however, up to the theoretical cracking load, based on elastic cross-sectional properties, the behaviors of the panels under different loading conditions are found to be similar.

- The panel tested under punching load failed in flexure, and the panel tested under four-point bending failed in shear.

- The flexural behavior and yielding pattern of the top and bottom concrete wythes of precast concrete sandwich panels subjected to punching load were similar to those of a conventional solid reinforced concrete slab subjected to punching. The side cover around the four edges of the panel provided partial fixity conditions for the wythes, which may be the primary reason for two-directional bending and radial cracking.

- The predicted ultimate punching load capacity based on the ACI 318 strength equation is in good agreement with the experimental ultimate load; however, the predictability of the ACI equation is poor for the panel subjected to bending due to shear failure of the panel. Possibilities for including mixed-mode fracture conditions to restrict the stress in the concrete may be explored to identify options for modifying the ACI strength equation.

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Notation

A_s = area of tensile reinforcement

b = width of the panel

d = distance between extreme fiber in compression and tensile reinforcement

E = Young's modulus

f_c' = compressive strength of concrete

f_t = tensile strength of concrete

f_y = yield strength of reinforcing bar material

f_{yw} = yield strength of wire material

I = moment of inertia of the panel

M_n = ultimate bending moment capacity of panel cross section as per ACI 318

P_{nb} = theoretical ultimate bending load capacity

P_{np} = theoretical ultimate punching load capacity

V_{nb} = theoretical ultimate shear load capacity under bending

V_{np} = theoretical ultimate shear load capacity under punching

Z = elastic section modulus

$\beta_1 c$ = depth of neutral axis

ϱ_t = percentage of tensile reinforcement provided

$\varrho_{t,min}$ = required minimum percentage of tensile reinforcement

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Abstract

Precast concrete sandwich panels can serve dual purposes of transferring loads and insulating the structure. A survey of the literature indicates that feasibility studies on the use of precast concrete sandwich panels to resist punching load are not reported. This paper presents the results and discussions of experimental and analytical studies conducted on two prototype precast concrete sandwich panels to determine and compare the behavior of precast concrete sandwich panels under punching and bending loads.

The precast concrete sandwich panels consisted of top and bottom reinforced concrete wythes connected using continuous truss-type shear connectors and expanded polystyrene as the core. During the experiments, load-deflection curves and strains in concrete surfaces and wire mesh and reinforcing bars were monitored until panel failure. Test results indicate that the type of flexural loading conditions has a significant effect on the flexural behavior of these types of sandwich panels, in particular the failure mode, after a specific applied load magnitude and bending moment. Analytical studies indicate that the ultimate flexural load capacity predicted using the ACI 318-11 strength equation is comparable to that of experimental ultimate flexural load of the panel subjected to punching. Further studies are required in this area, in particular, developing design guidelines to use these panels as floor panels in housing systems.

Keywords

Bending, composite, expanded polystyrene, experiment, insulated panel, panel, punching, sandwich.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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