Load-carrying capacity of composite precast concrete sandwich panels with diagonal fiber-reinforced-polymer bar connectors

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- The failure modes of composite precast concrete sandwich panels made with diagonal shear connectors made of fiber-reinforced-polymer (FRP) bar are investigated in this paper.
- A mathematical model that was previously developed by the author is further enhanced and used for the analysis, which is also compared with test results from the literature.
- A parametric study that investigates key parameters in the design of precast concrete sandwich panels is presented, including the diameter of the diagonal FRP bars, the diameter of the steel reinforcement in the wythes, and the grade of concrete.

The use of precast concrete sandwich panels has increased significantly in the past few years due to their superior thermal and acoustic insulation properties. They are composed of two reinforced concrete layers (wythes) separated by a layer of rigid foam insulation. Unlike traditional noncomposite panels that rely only on the interior wythe to resist the load, composite sandwich panels rely on the composite action between the wythes, which can be achieved by using shear connectors that are mainly made from nonmetallic, diagonal fiber-reinforced-polymer (FRP) materials. FRP connectors are preferred by many manufacturers over traditional steel connectors because of their excellent thermal insulation properties and their corrosion resistance. The result of such a combination of materials is an energy-efficient sandwich panel that has a thinner inner wythe than a noncomposite wall, which can be used in many more applications than its counterpart noncomposite wall.[1]

In the past, the full advantages of precast concrete sandwich panels have not been realized due to the lack of appropriate design guidelines. This was highlighted in the PCI committee report[1] in 2011, which indicated the need for future research in this area. Although many studies have focused on the partial composite behavior of precast concrete sandwich panels at the serviceability limit state, few studies have investigated their load-carrying capacity. Einea et al.[2] tested panels made with FRP bent bars as the shear connectors. Results of this research indicated a ductile behavior of the panels though the FRP material was linear. Salmon et al.[3] presented test results of panels under a uniformly distributed load. A finite element analysis was conducted, and it was shown that the partial composite behavior can be described by a linear analysis that accounts for the elastic response of the diagonal shear connectors and the flexural response of the reinforced concrete wythes as beams.

Benayoune et al.[4] presented an experimental investigation of six eccentrically loaded precast concrete sandwich panels with different slenderness ratios. The results exhibited a large scatter in terms of strength, stiffness, and ductility, which made it difficult to establish a behavior pattern or trend. Hassan and Rizkalla[5] developed simplified design procedures for precast concrete sandwich panels made with carbon-fiber-reinforced polymer (CFRP) grids as shear connectors. Naito et al.[6] tested precast concrete sandwich panels with different types of continuous and discretely placed shear ties under direct shear. The results showed that the different strengths and stiffnesses of the ties can significantly influence the postcracking stiffness of the entire precast concrete sandwich panel but have only a small influence on its ultimate strength. Cuypers and Wastiels[7] highlighted the importance of carefully considering the assumptions made for analyzing precast concrete sandwich panels because concrete cracking leads to shifting of the global neutral axis; hence, classical sandwich theories cannot be used. Comprehensive double lap-shear testing of precast concrete sandwich panels were conducted by Choi et al.[8] and Jang et al.[9]

In this paper, a theoretical model that was previously developed by the author[10] is further enhanced and used to calculate the load-carrying capacity of precast concrete sandwich panels made with diagonal FRP bar connectors. The model in Hamed[10] accounts for cracking and tension stiffening only, but it is improved here to account for the material nonlinearity of the concrete in compression, as well as yielding of the steel reinforcement. A numerical example is presented and followed by a parametric study and a comparison of the model with test results from the literature.

Model, constitutive laws, and failure criteria

For brevity, the details of the mathematical model used for the analysis are not shown here, but they can be found in detail in Hamed.[10] **Figure 1** shows the sign convention of the model. The reinforced concrete wythes are modeled as Euler-Bernoulli beams. The insulation layer is assumed to possess shear and through-the-thickness normal stiffness

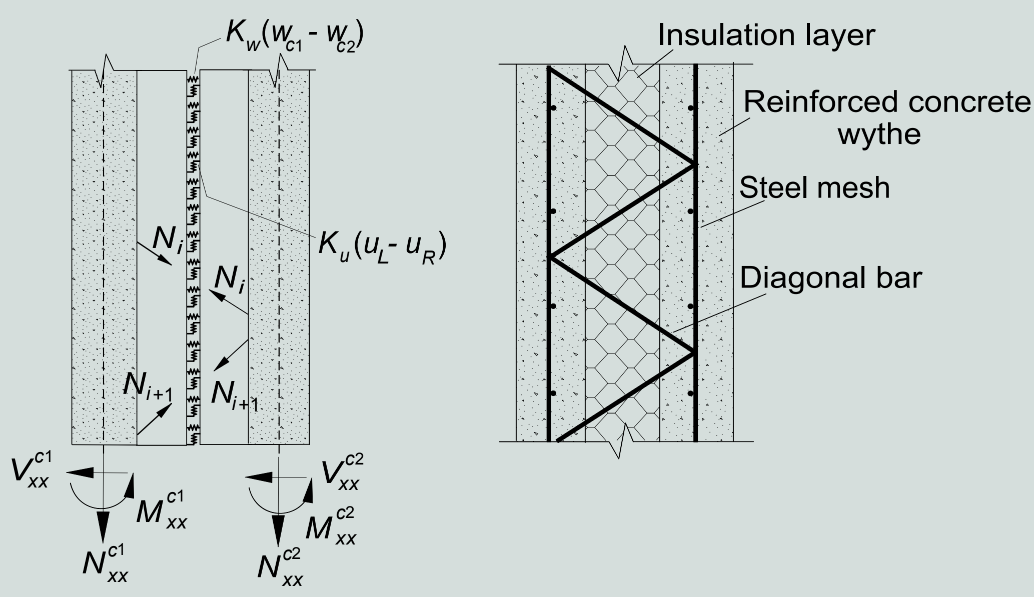

Figure 1. Mathematical model and sign conventions of precast concrete sandwich panels. Note: K_u = spring stiffness in height direction; K_w = spring stiffness in out-of-plane direction; M_{xx}^{c1} = internal bending moment in the inner reinforced concrete wythe; M_{xx}^{c2} = internal bending moment in the outer reinforced concrete wythe; N_i = axial force in the ith diagonal bar; N_{i+1} = axial force in the $i + 1$ diagonal bar; N_{xx}^{c1} = internal axial force in the inner reinforced concrete wythe; N_{xx}^{c2} = internal axial force in the outer reinforced concrete wythe; u_L = longitudinal displacement at the left interface of the middle line of the insulation layer; u_R = longitudinal displacement at the right interface of the middle line of the insulation layer; V_{xx}^{c1} = internal shear force in the inner reinforced concrete wythe; V_{xx}^{c2} = internal shear force in the outer reinforced concrete wythe; w_{c1} = out-of-plane displacement of the inner reinforced concrete wythe; w_{c2} = out-of-plane displacement of the outer reinforced concrete wythe.

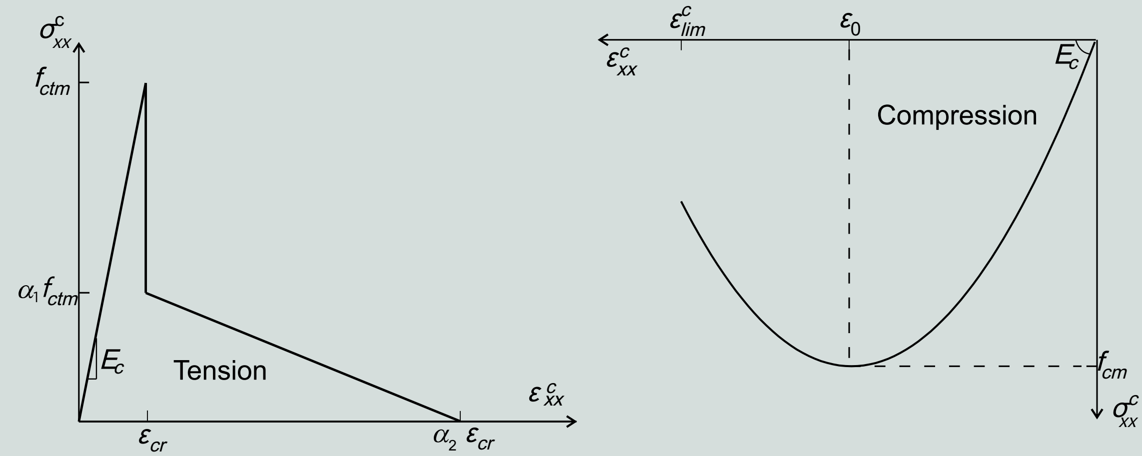

Figure 2. Constitutive laws of concrete. Note: E_c = modulus of elasticity of concrete; f_{cm} = mean compressive strength of concrete; f_{ctm} = mean tensile strength of concrete; α_1= parameter for tension stiffening; α_2 = parameter for tension stiffening that depends on reinforcement ratio and dimensions; ε_0 = strain at peak compressive stress in concrete; ε_{cr} = cracking strain in concrete; ε^c_{lim} = ultimate compression strain in concrete; ε^c_{xx}= concrete normal strain; σ^c_{xx} = concrete normal stress.

with negligible in-plane rigidity. The insulation layer is considered to be two halves that are each connected to the adjacent reinforced concrete wythe with linear strain distribution through the thickness. In this sense, its rigidities are introduced through springs located at the middle of its thickness (Fig. 1). The connection of the two reinforced concrete wythes via the diagonal FRP bar connector creates a truss mechanism. The reinforced concrete wythes carry the bending moment by a force couple, while the shear force is carried by axial forces in the diagonal bars. Hence, for simplicity, the diagonal bars are assumed to transfer axial forces N_i only.

The following failure modes are considered in the analysis:

- flexural failure (either by concrete crushing or yielding of the steel)
- buckling of the diagonal FRP bars
- rupture or crushing of the diagonal FRP bars

To account for the first mode of failure, a nonlinear iterative analysis that is based on the secant modulus approach was conducted.[10] For this, each reinforced concrete wythe was divided into a number of layers through its thickness and the stresses were examined at each point through the height of the panel for the determination of cracking, tension stiffening, and material softening in compression. The constitutive relation in Eq. (1) for the concrete in compression follows Euro-International Concrete Committee (CEB)–International Federation for Prestressing (FIP)[11] and adopts the model proposed by Torres et al.[12] to account for the tension-stiffening effect (**Fig. 2**).

$$\sigma^c_{xx} = \begin{cases} \dfrac{E_c\varepsilon^c_{xx} + f_{cm}\left(\dfrac{\varepsilon^c_{xx}}{\varepsilon_0}\right)^2}{1-\left(\dfrac{E_c\varepsilon^c_{xx}}{f_{cm}}+2\right)\dfrac{\varepsilon^c_{xx}}{\varepsilon_0}} & \text{for} \quad \varepsilon^c_{lim} \le \varepsilon^c_{xx} \le 0 \\ E_c\varepsilon^c_{xx} & \text{for} \quad 0 < \varepsilon^c_{xx} \le \varepsilon_{cr} \\ \dfrac{\alpha_2\varepsilon_{cr} - \varepsilon^c_{xx}}{(\alpha_2 - 1)\varepsilon_{cr}}\alpha_1 f_{ctm} & \text{for} \quad \varepsilon_{cr} < \varepsilon^c_{xx} \le \alpha_2\varepsilon_{cr} \\ 0 & \text{otherwise} \end{cases} \tag{1}$$

where

σ^c_{xx} = concrete normal stress

E_c = modulus of elasticity of concrete

ε^c_{xx} = concrete normal strain

f_{cm} = mean compressive strength of concrete

ε_0 = strain at peak compressive stress in concrete

ε^c_{lim} = ultimate compressive strain in concrete

α_2 = parameter for tension stiffening that depends on reinforcement ratio and dimensions

ε_{cr} = cracking strain in concrete (determined based on the mean tensile strength as f_{ctm}/E_c)

f_{ctm} = mean tensile strength of concrete

α_1 = parameter for tension stiffening = 0.4

Following CEB-FIP,[11] all concrete mechanical properties (that is, E_c, f_{cm}, ε_0, ε^c_{lim}, and f_{ctm}) can be determined based on the concrete grade (characteristic strength).

The constitutive relation of the steel reinforcement under both tension and compression assuming an elastic–perfectly plastic behavior is given by Eq. (2)

$$\sigma_s = \begin{cases} E_s\varepsilon_s & \text{for} \quad |\varepsilon_s| \le \varepsilon_y \\ E_s\varepsilon_y & \text{for} \quad \varepsilon_y < \varepsilon_s \\ -E_s\varepsilon_y & \text{for} \quad \varepsilon_s < -\varepsilon_y \end{cases} \tag{2}$$

where

σ_s = stress of steel reinforcement

E_s = elastic modulus of steel reinforcement

ε_s = strain of steel reinforcement

ε_y = yielding strain of steel reinforcement

It was shown by Zong et al.,[13] who investigated the buckling response of steel bars embedded in concrete, that their stress-strain curve is linear up to the yield stress, whereas buckling actually occurs at the inelastic range and is characterized by a gradual softening or hardening that depends on a number of parameters. For simplicity, the postbuckling response is not accounted for here, and only an elastic–perfectly plastic response is considered.

Buckling of the diagonal FRP bars of the shear connector is assumed to occur once the axial compression load exceeds the Euler buckling load of the bars N_{cr}, which is calculated by Eq. (3).

$$N_{cr} = \frac{\pi^2 E_b I_b}{L_b^2} \tag{3}$$

where

E_b = modulus of elasticity of the diagonal bars

I_b = geometrical moment of inertia of the diagonal bars

L_b = buckling length of the diagonal bars

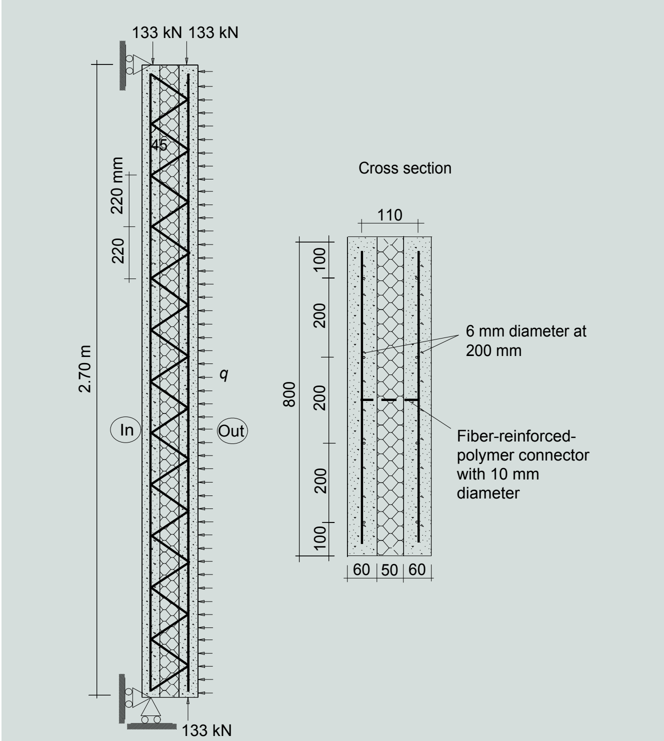

Figure 3. Geometry and loading of investigated precast concrete sandwich panel. Note: All dimensions are in millimeters unless otherwise indicated. q = lateral load on the panel. 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip.

The buckling length is taken as the distance between the connection points of the bars to the reinforced concrete wythes at their interfaces assuming fixed connections. This follows the experimental observations in Bush and Stine,[14] where push-out tests indicated no deformation of the truss diagonals within the length embedded in the concrete.

Rupture or crushing of the diagonal bars is assumed to occur once the stress exceeds the tensile or compressive strength as follows:

$$\frac{N_i}{A_b} > f_u \tag{4}$$

where

A_b = cross-sectional area of diagonal bars

f_u = tensile or compressive strength of diagonal bars

Numerical example

A precast concrete sandwich wall that is subjected to uniformly distributed lateral loading was investigated (**Fig. 3**). The spacing between the truss connectors was 800 mm (31 in.), and therefore the analysis was conducted on a representative 800 mm width of the panel with one truss connector only. The panel was assumed to be simply supported at the top and bottom edges of the inner reinforced concrete wythe. Deformed steel bars of 6.0 mm (0.24 in.)

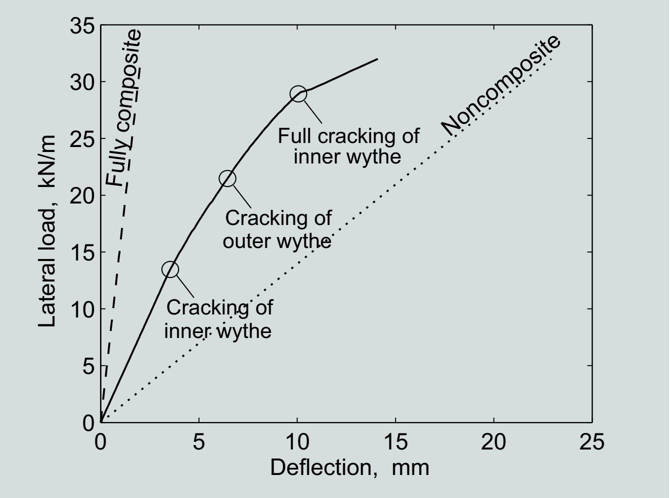

Figure 4. Load-deflection curve of precast concrete sandwich panel. Note: 1 mm = 0.0394 in.; 1 kN/m = 0.0685 kip/ft.

diameter with spacing of 200 mm (8 in.) that were located at the middle of the thickness of each reinforced concrete wythe were used. The elastic modulus of steel was taken as 200 GPa (29,000 ksi), and the yielding strain was 0.25%. The diagonal bars were made from FRP with a bar diameter of 10.0 mm (0.394 in.) and an inclination angle of 45 degrees. The elastic modulus of the FRP was taken as 45 GPa (6500 ksi), and the tensile strength was 970 MPa (140 ksi), following Maximos et al.[15] The insulation layer was taken as expanded polystyrene with an elastic modulus of 5 MPa (0.7 ksi) and a shear modulus of 2.27 MPa (0.329 ksi). The concrete had a mean compressive strength of 38 MPa (5500 psi) and a tensile strength of 2.9 MPa (420 psi). The concrete properties that are used in Eq. (1) are as follows:

E_c = 33.6 GPa (4870 ksi)

ε_0 = 0.23%

ε^c_{lim} = 0.35%

α_1 = 0.4

α_2 = 40.5

To simulate real applications of load-bearing composite wall panels, the wall was subjected to a combined axial compression force and lateral loading. The axial force describes the loads that are transferred from the superstructure, such as floors or load-carrying beams connected to the wall. The lateral load may result from wind, hydrostatic pressure, or other action. For simulating the results, the axial load of 266 kN (59.8 kip) that was shared between the two wythes is kept constant, while the lateral load was assumed to be applied quasi-statically and to gradually increase until failure.

Figure 4 shows the load-deflection curve of the sandwich panel along with predictions that are based on elastic fully composite and noncomposite actions. The results reveal that first cracking of the inner wythe occurs at a load level

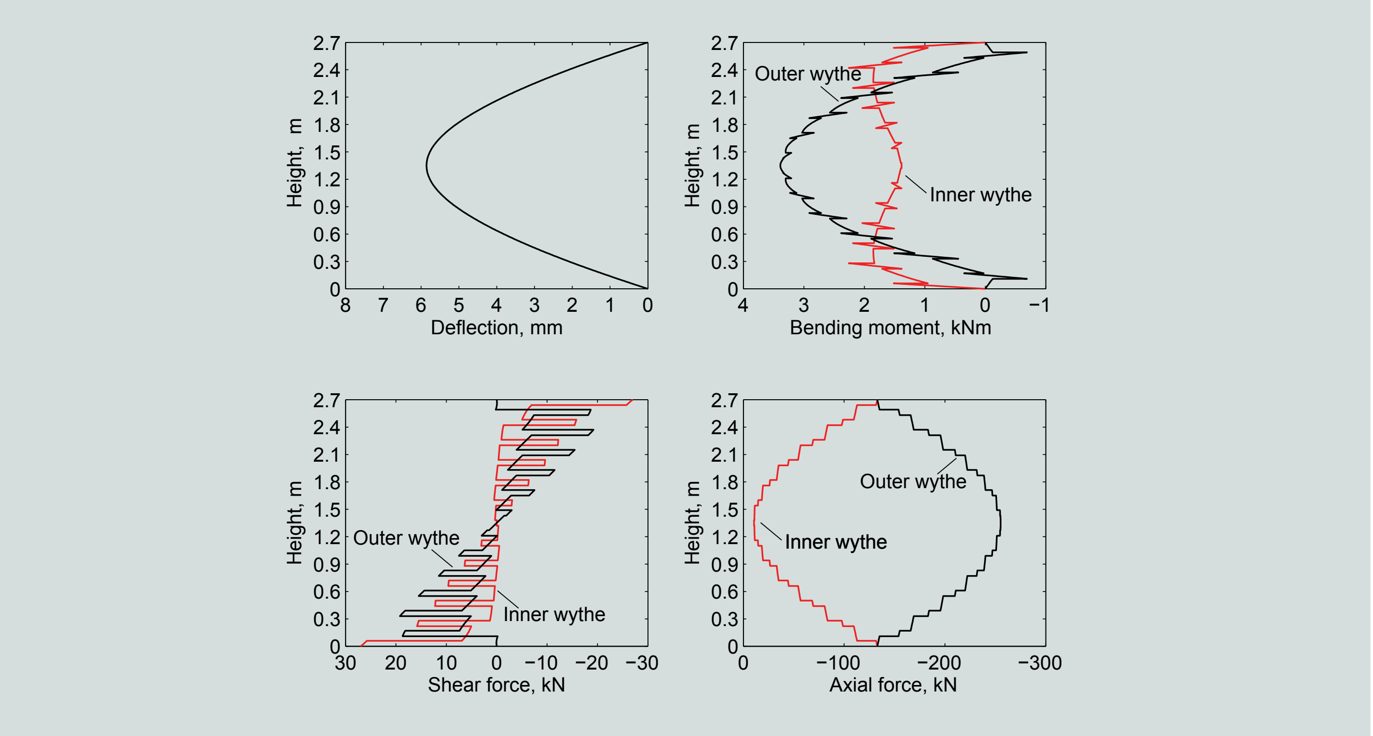

Figure 5. Structural response under a load level of 20 kN/m (1370 lb/ft). Note: 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip; 1 kN-m = 0.738 kip-ft.

of 13.1 kN/m (0.897 kip/ft), followed by cracking of the outer wythe at a load of 22.1 kN/m (1.51 kip/ft). The drastic reduction in stiffness that is observed at a load level of 29.3 kN/m (2.01 kip/ft) is attributed to full cracking of the inner wythe, which leads to a significant loss of the composite action in the panel. The panel loses its moment-carrying capacity shortly after full cracking of the inner wythe by yielding of the steel reinforcement at a load level of 32 kN/m (2.2 kip/ft), indicating a typical flexural failure. Due to convergence issues governed by the load-control approach of the analysis, the analysis was stopped at the yield point of the reinforcement. Because of the flexibility of the shear connectors, the response of the panel cannot be predicted with simple models that are based on fully composite or noncomposite actions. These aspects highlight the importance of using refined models that properly account for the layered structural configuration of sandwich panels and the interaction between the layers through the shear connectors.

The failure load predicted by assuming full composite action using typical flexural-strength design methods, which consists of a rectangular stress block in the concrete and yielding of the steel and an approximated lever arm of 0.95 times the effective depth, equals 27.4 kN/m (1.8 kip/ft). This is slightly less than the predicted failure load using the proposed detailed model. Hence, as was also shown by Salmon et al.,[3] the strength of sandwich panels can be approximately and conservatively computed assuming a full composite panel when the failure mode is dominated by flexure.

Figure 5 shows the distribution of the deformations and internal forces through the height of the panel at a load level of 20 kN/m (1.37 kip/ft). The distributions of the forces and moments show the sharp changes at the locations of the diagonal bar connections, which also lead to the development of negative moments in the outer reinforced concrete wythe near the edges. The results quantitatively show the portion of the moment that is carried by local bending in the reinforced concrete wythes, and the portion of the moment that is carried in terms of a force couple between the reinforced concrete wythes. The total bending moment equals $20 \times 2.7^2/8$, which equals 18 kN-m (13 kip-ft). The local bending moment in the inner and outer reinforced concrete wythes (Fig. 5) equals 1.4 kN-m (1.0 kip-ft) and 3.4 kN-m (2.5 kip-ft), respectively, which in sum are only about 26% of the total bending moment. The remaining 74% of the moment is carried in terms of a force couple between the reinforced concrete wythes with a lever-arm length that is equal to the thickness of the insulation layer plus half the thickness of each wythe. Figure 5 shows that the reinforced concrete wythes are subjected to different axial forces due to the combination of the applied axial compression force and the overall bending due to lateral loading. As mentioned, the inner reinforced concrete wythe is already cracked under this load level due to bending, which leads to a significant reduction in its ability to carry bending moments. Figure 5 also shows that because the inner wythe is the one that is supported laterally, the shear force in the outer reinforced concrete wythe drops to zero at the edges, while all of the shear forces are transferred to the supports via the inner wythe. These aspects of behavior cannot be obtained using simple equivalent beam analysis that assumes full or no composite action.

Figure 6 shows the distribution of stresses at midheight through the thickness of the inner and outer reinforced concrete wythes at three different load levels that correspond to first cracking of the inner wythe, cracking of the outer wythe, and failure. The nonlinear stress distributions ob-

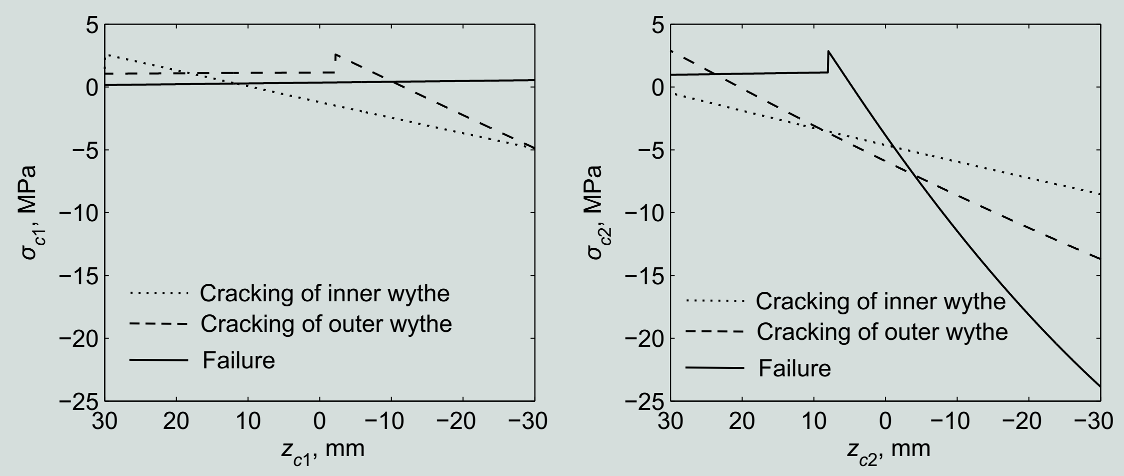

Figure 6. Stress distributions through the thickness of the reinforced concrete wythes at different stages. Note: z_{c1} = horizontal distance from the middle thickness of the inner wythe (positive to the left); z_{c2} = horizontal distance from the middle thickness of the outer wythe (positive to the left); σ_{c1} = stress in the inner reinforced concrete wythe; σ_{c2} = stress in the outer reinforced concrete wythe. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

tained by the model show that the maximum compression stress at the outer wythe is still less than the compressive strength. Cracking propagates through the entire depth of the inner wythe at the maximum load level and the cracks extend to almost half the depth of the outer wythe. These observations indicate that simplified models, which assume cracking of the inner wythe only, can lead to inaccurate results. The main reason for cracking of both wythes is the combined axial and bending actions of the wythes, where the external moment is carried by a force couple and local bending of each wythe.

Parametric study

Three main parameters that may govern the load-carrying capacity of precast concrete sandwich panels are investigated here: diameter of the fiber-reinforced polymer diagonal bars, diameter of the steel reinforcement, and concrete grade. The sandwich panel investigated in the numerical study is used as a reference, and when one of the parameters is changed, the other two are kept the same as in the reference panel.

Figure 7 shows the influence of changing the diameter of the diagonal bars on the load-deflection curve of the panel. The results are also compared with the response obtained assuming full and noncomposite actions of the panel. The sandwich panel that contained 6 mm (0.2 in.) diameter diagonal bars failed by buckling of the critical bars. Nevertheless, increasing the diameter of these diagonal bars from 8 to 12 mm (0.3 to 0.47 in.) increases the panel stiffness but does not have an influence on its failure load, which is a typical flexural one in this case. This was also noticed in the test results reported in Naito et al.,[6] who tested different types of shear ties and indicated only a small influence of the type of shear tie on the flexural strength of the sandwich panel but with significant influence on the post-cracking response. This observation is important because it indicates that as long as diagonal-bar shear connectors are used and a reasonable diameter of the diagonal bars is chosen so that no buckling occurs. In most cases, engineers can conservatively use simplified models that are based on full composite action for the strength design of precast concrete sandwich panels. Nevertheless, the precracking and postcracking response at the serviceability limit state is always characterized by a partial shear interaction (Fig. 7), which requires the use of advanced models for its characterization.

Figure 8 shows the influence of changing the diameter of the internal steel reinforcement from 5 to 12 mm (0.2 to 0.47 in.) on the load-deflection curve of the panel. All panels failed by typical flexural failure. Unlike the influence of the diameter of the diagonal bars (Fig. 7), increasing the diameter of the steel mesh reinforcement does not influence the stiffness of the sandwich panel but signifi-

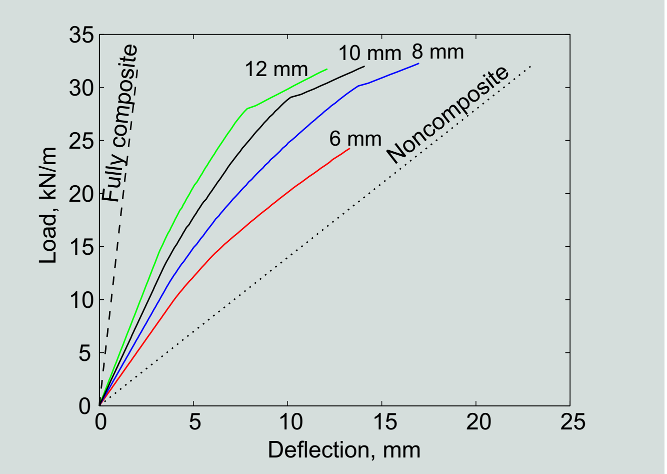

Figure 7. Influence of the diameter of the fiber-reinforced polymer diagonal bars on the load-carrying capacity of precast concrete sandwich panels. Note: 1 mm = 0.0394 in.; 1 kN/m = 0.0685 kip/ft.

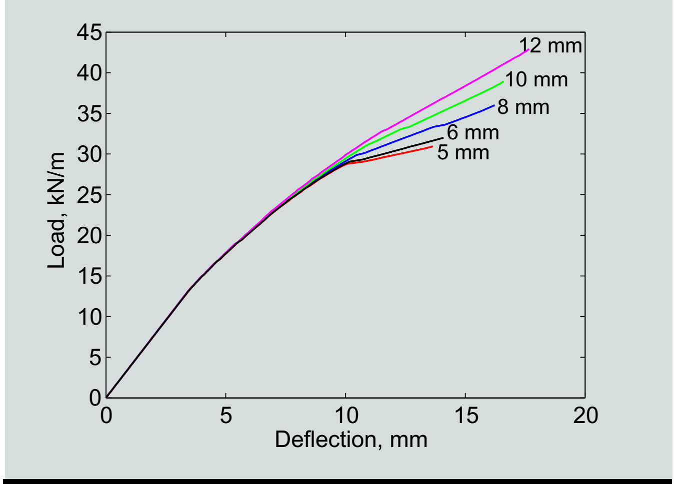

Figure 8. Influence of the diameter of the steel reinforcement on the load-carrying capacity of precast concrete sandwich panels. Note: 1 mm = 0.0394 in.; 1 kN/m = 0.0685 kip/ft.

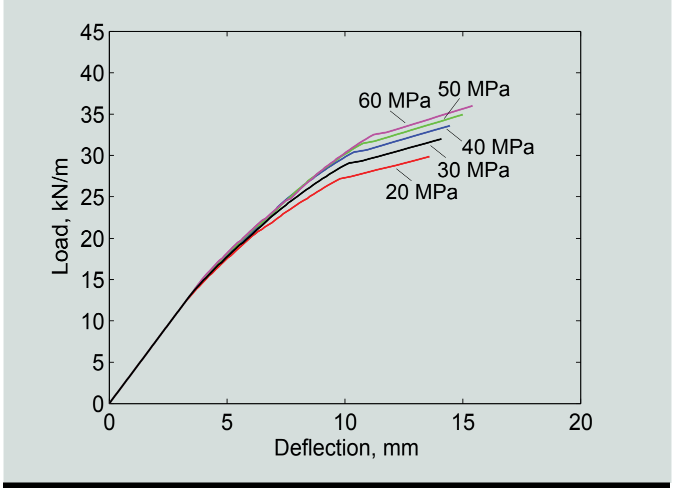

Figure 9. Influence of the grade of concrete on the load-carrying capacity of precast concrete sandwich panels. Note: 1 mm = 0.0394 in.; 1 kN/m = 0.0685 kip/ft; 1 MPa = 0.145 ksi.

cantly increases the failure load. An increase of about 40% is expected by increasing the diameter of the steel mesh from 5 to 12 mm, thereby increasing the reinforcement ratio from 0.16% to 0.92%. As expected, the stiffness of the panel after full cracking of the inner wythe at a load level of about 30 kN/m (2.1 kip/ft) is also influenced by the reinforcement ratio.

Figure 9 shows the influence of changing the design concrete compressive strength from 20 to 60 MPa (2900 to 8700 psi) on the load-deflection curve of the panel. All panels failed by typical flexural failure. Increasing the concrete strength slightly increases the stiffness after first cracking of the inner wythe at a load level that ranges from 12 to 15 kN/m (0.82 to 1.0 kip/ft). Increasing the concrete strength can also increase the ultimate load, yet the increase is less significant compared with changing the reinforcement ratio.

Comparison with test results

A comparison with the test results of Salmon et al.[3] includes the testing of two identical panels that were simply supported and tested vertically under a uniformly distributed lateral load. Due to the lack of other well-reported test results, only these test results are used for the validation of the model. The height of the sandwich panels was 9140 mm (360 in.), and their width was 2440 mm (96.1 in.). The thickness of the reinforced concrete wythes was 63.5 mm (2.50 in.), and that of the insulation layer was 76.2 mm (3.0 in.). Each reinforced concrete wythe included a 6 × 6 (W4 × W4) welded-wire steel reinforcement mesh of 5.7 mm (0.22 in.) diameter with spacing of 152 mm (5.98 in.) located at the middle of the thickness. Five 9.5 mm (0.37 in.) nominal diameter prestressing strands were used in each wythe. Prestressing was introduced in the model as an axial load, and the area of the strands was accounted for as additional reinforcement with initial strain due to prestressing. The ultimate tensile strength of prestressing strands f_{pu} was 1860 MPa (270 ksi), with a jacking stress of $0.8f_{pu}$ and a reported total prestress loss of 30%.

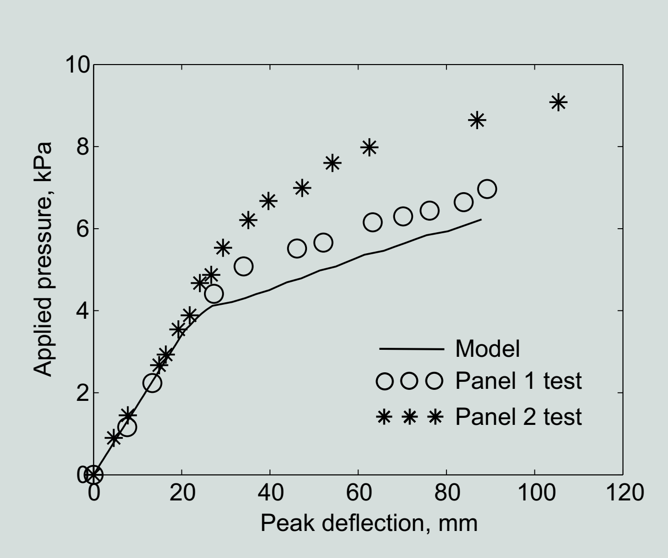

Figure 10. Theoretical and experimental load-deflection curves. Note: 1 mm = 0.0394 in.; 1 kPa = 0.145 psi. Source: Data from Salmon et al. (1997).

The elastic modulus of steel was taken as 200 GPa (29,000 ksi), and the yield strain was 0.26%. The shear connectors were made from FRP with a diameter of 9.5 mm (0.37 in.) and an inclination angle of 50 degrees. The modulus of elasticity of FRP was 48.263 GPa (7000 ksi), and its tensile strength was 401 MPa (58.2 ksi). Six rows of shear connectors were used in the third of the span near the supports and three rows in the central third. The insulation layer was taken as expanded polystyrene with an elastic modulus of 5 MPa (0.7 ksi) and a shear modulus of 2.27 MPa (0.329 ksi). The concrete had a compressive strength of 34.5 MPa (5000 psi), a tensile strength of 3.3 MPa (0.48 ksi), and an elastic modulus of 30.544 GPa (4429.9 ksi). The strain at peak compressive stress ε_0 was 0.23%, and the ultimate strain used for the constitutive law ε^c_{lim} was 0.35%.[11]

Figure 10 shows the predicted and measured load-deflection curves of the panels. The actual behavior of the two identical sandwich panels that were tested under the same conditions differed significantly after cracking, which shows the sensitivity of these structures to uncertain imperfections and to other parameters that depend on the construction procedure and that can be different between the two panels. The predicted response agreed well with the response of panel 1 at the precracking and postcracking stages, as well as the ultimate load. Cracking of the inner wythe was predicted at an applied pressure of 3.5 kPa (0.51 psi), which propagated rapidly through the entire depth of the inner wythe and led to its full cracking at a pressure of 4.21 kPa (0.611 psi) with significant reduction in stiffness. Cracking of the outer wythe occurred at an applied pressure of 5.46 kPa (0.792 psi). The predicted flexural failure pressure by steel yielding was 6.22 kPa (0.902 psi), which agreed well with that of panel 1, which is 6.96 kPa (1.01 psi). A similar comparison to these test results was conducted in Hamed[10] but with less correlation because the model in Hamed did not account for the material nonlinearity of the concrete in compression nor for the yielding of the steel.

Conclusion

This study shows that typical composite precast concrete sandwich panels that are made with diagonal FRP bar connectors are dominated by a flexural mode of failure that is governed by yielding of the steel reinforcement. No FRP rupture was observed, and only one case out of the thirteen examined cases exhibited buckling of the diagonal bars. As such, the diameter of the diagonal bars of the shear connector has a minor influence on the load-carrying capacity of the sandwich panel but significantly influences its stiffness and the degree of shear interaction at the serviceability limit state. Alternatively, it was shown that the reinforcement ratio and the concrete strength could significantly increase the load-carrying capacity. It was also shown that simple strength design approaches can be effectively and conservatively used to predict the failure load of composite sandwich panels.

Based on this study, only the thermal and corrosion characteristics of the shear connector should be a concern for choosing the appropriate material (steel or FRP). It appears that ductility of the shear connector is not needed because the panel will eventually fail by complete cracking and yielding of the steel reinforcement before yielding or rupture of the shear connector occurs. Hence, it is recommended that FRP connectors be used for these applications because of their superior thermal and corrosion resistance despite being a linear elastic material. The overall behavior of precast concrete sandwich panels would still exhibit some level of nonlinear behavior that is mainly attributed to cracking, tension stiffening, and yielding of the steel reinforcement.

The model presented in this study was solved by a computationally efficient in-house computer code developed by the author. It can be further used to investigate additional parameters and to present design aids in terms of charts and simple equations in the future.

Acknowledgments

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Notation

A_b = cross-sectional area of diagonal truss bars

E_b = modulus of elasticity of diagonal truss bars

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

f_{cm} = mean compressive strength of concrete

f_{ctm} = mean tensile strength of concrete

f_{pu} = ultimate tensile strength of prestressing strands

f_u = tensile strength of diagonal truss bars

I_b = geometrical moment of inertia of diagonal truss bars

K_u = spring stiffness in height direction

K_w = spring stiffness in out-of-plane direction

L_b = buckling length of diagonal truss bars

M_{xx}^{c1} = internal bending moment in the inner reinforced concrete wythe

M_{xx}^{c2} = internal bending moment in the outer reinforced concrete wythe

N_{cr} = Euler buckling load of the diagonal bars

N_i = axial force in the ith diagonal bar

N_{i+1} = axial force in the $i + 1$ diagonal bar

N_{xx}^{c1} = internal axial force in the inner reinforced concrete wythe

N_{xx}^{c2} = internal axial force in the outer reinforced concrete wythe

q = lateral load on the panel

u_L = longitudinal displacement at the left interface of the middle line of the insulation layer

u_R = longitudinal displacement at the right interface of the middle line of the insulation layer

V_{xx}^{c1} = internal shear force in the inner reinforced concrete wythe

V_{xx}^{c2} = internal shear force in the outer reinforced concrete wythe

w_{c1} = out-of-plane displacement of the inner reinforced concrete wythe

w_{c2} = out-of-plane displacement of the outer reinforced concrete wythe

z_{c1} = horizontal distance from the middle thickness of the inner wythe (positive to the left)

z_{c2} = horizontal distance from the middle thickness of the outer wythe (positive to the left)

α_1 = parameter for tension stiffening

α_2 = parameter for tension stiffening that depends on reinforcement ratio and dimensions

ε_0 = strain at peak compressive stress in concrete

ε_{cr} = cracking strain in concrete

ε_s = strain of steel reinforcement

ε_y = yielding strain of steel reinforcement

ε_{lim}^c = ultimate compression strain in concrete

ε_{xx}^c = concrete normal strain

σ_{c1} = stress in the inner reinforced concrete wythe

σ_{c2} = stress in the outer reinforced concrete wythe

σ_s = stress in steel reinforcement

σ_{xx}^c = concrete normal stress

About the author

Ehab Hamed, PhD, is a senior lecturer in the School of Civil and Environmental Engineering at the University of New South Wales and a member of its Centre for Infrastructure Engineering and Safety in Sydney, Australia. He received his PhD from the Technion – Israel Institute of Technology. His research interests include modeling and analysis of concrete structures, creep, sandwich structures, and the retrofitting of steel and concrete structures with composite materials.

Abstract

The failure modes of composite precast concrete sandwich panels made with diagonal shear connectors made of fiber-reinforced-polymer (FRP) bar are investigated in this paper. A mathematical model that was previously developed by the author is further enhanced and used for the analysis, which is also compared with test results from the literature. The model accounts for cracking; tension stiffening; nonlinear softening of the concrete in compression; yielding of the steel reinforcement; and rupture, crushing, and buckling of the FRP diagonal bars. A parametric study investigates key parameters in the design of concrete sandwich panels, including the diameter of the diagonal FRP bars, the reinforcement ratio in the wythes, and the grade of concrete. The results explain the structural behavior of concrete sandwich panels and provide recommendations and a basis for their design.

Keywords

Composite action, failure, FRP, fiber-reinforced polymer, partial interaction, sandwich panel.

Review policy

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

Reader comments

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