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Structural Behavior of Precast Foamed Concrete Sandwich Panel Subjected to Vertical In-Plane Shear Loading

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***Abstract*—**Experimental and analytical studies were accomplished to examine the structural behavior of precast foamed concrete sandwich panel (PFCSP) under vertical in-plane shear load. PFCSP full-scale specimens with total number of six were developed with varying heights to study an important parameter slenderness ratio (H/t). The production technique of PFCSP and the procedure of test setup were described. The results obtained from the experimental tests were analysed in the context of in-plane shear strength capacity, load-deflection profile, load-strain relationship, slenderness ratio, shear cracking patterns and mode of failure. Analytical study of finite element analysis was implemented and the theoretical calculations of the ultimate in-plane shear strengths using the adopted ACI318 equation for reinforced concrete wall were determined aimed at predicting the in-plane shear strength of PFCSP. The decrease in slenderness ratio from 24 to 14 showed an increase of 26.51% and 21.91% on the ultimate in-plane shear strength capacity as obtained experimentally and in FEA models, respectively. The experimental test results, FEA models data and theoretical calculation values were compared and provided a significant agreement with high degree of accuracy. Therefore, on the basis of the results obtained, PFCSP wall has the potential use as an alternative to the conventional load-bearing wall system.

***Keywords*—**Deflection profiles, foamed concrete, load-strain relationships, precast foamed concrete sandwich panel, slenderness ratio, vertical in-plane shear strength capacity.

I. Introduction

PRECAST concrete sandwich panels (PCSP) are energy efficient products used to construct exterior and interior bearing, non-bearing walls, and floors for all building types in the construction industry [1]-[6]. This typical system consists of two concrete wythes that are separated by an internal insulation layer of various materials (e.g., polystyrene). The concrete wythes are combined using shear connectors that can transfer the longitudinal interface shear among the wythes to ensure a fully composite or semi-composite behaviour of the sandwich panel [6]. A sandwich panel can flexibly and economically meet structural, thermal, and architectural requirements [7], [8]. The thermal insulation property controlled by the insulation value and panel buckling is restricted by the strength of both concrete wythes [9]-[12]. The structural behaviour of PCSP depends on the strength and stiffness of the mechanical shear connector, which provides rigidity and shear terms on the full composite behaviour [13]. PCSPs possess efficient functions that are much similar to those of precast solid wall panels, which differ only in terms of build-up. Furthermore, sandwich panel construction lacks information because of the costly materials used to conduct full-scale experimental testing rather than small-scale testing models [3]; many sandwich panel applications are available in Europe and North America, and are proprietary [1], [14]. The majority of current PCSP components are fabricated using heavy normal concrete rather than lightweight ones, and the complaint and inability of foundation engineers provide enough bearing capacity foundation to carry the self-weight of superstructures, causing structural engineers to contribute in reducing the self-weight of upcoming sandwich panel system application generation [15]. Besides, a lack of information available about concrete sandwich applications studies subjected to shear loading [16], [17].

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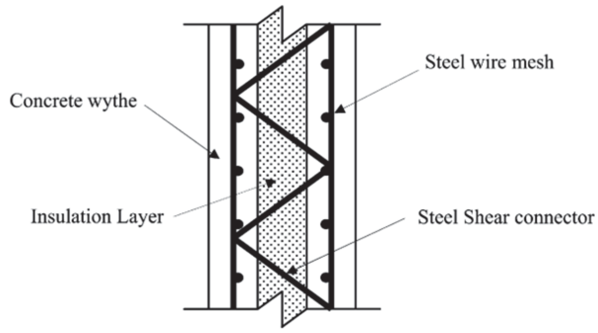


Fig. 1 A 2D PCSP [2]

II. Applications of Foamed Concrete

Foamed concrete (FC) has distinctive properties including density reduction, self-compacting concrete, high flowability, and low thermal conductivity, therefore, it gives the ease of producers and it is relatively cost-effectiveness. FC has found applications in many civil and structural engineering areas [18]-[20]. For instance, the low density FC has been used for cavity filling and insulation while the high densities were used in structural applications and the other applications are production of lightweight pre-cast panels and blocks, fire insulation, road sub-base, thermal and acoustic insulation, trench reinstatement, shock absorbing barriers for airports and regular traffic and soil stabilization [18]-[22]. Also, due to flowability features, it is an exaggerated material for voids, such as old sewers, storage tanks, basements, ducts and voids under roadways occurred by cliff of heavy rains [23].

Application of FC has become popular worldwide, especially at the regions suffering from housing shortages or subjected to adverse weather, hurricanes and earthquakes [19], [24]. In North America, the overall demand was more from the southern US and to be equal to the actual production. But, in Canada, cement based foam has been widespread used for tunnel annulus grouting, flowable fills and in geotechnical applications. This growing interest seems to be partially due to a significant increase in the costs of other lightweight building materials such as dry wall and wood and in part to the environmental issues [24], [25]. Besides, an additional feature of foamed concrete encouraged it as to be appropriate for large volumes of supplementary cementing admixtures because of the manufacturing and environmental cost associated with cement production [23]. The FC material is also beneficial when used in concrete paving works to prevent frost heave on roads, to insulate shallow foundation systems and placements, to prevent frost heave under pile caps and frost jacking of shallow piles. FC is also used as a grout to fill abandoned pipes and as backfill under buried oil field modules, to decrease the temperature under hot oil tanks and the tank support and to fill voids under slabs and to reduce the thermal stress and the thermal gradient in hot concrete pits and thus insulate shallow [23], [19].

III. Precast Foamed Concrete Sandwich Panel

The PFCSP is a structure with an insulated and layered system. PFCSPs are composed of external structural FC wythes that sandwich a polystyrene layer with a high thermal insulation of 0.07 K/m.w and a low density of 1.05 g/cm³. The three layers act integrally via a proper connection established by continuous steel truss-shaped shear connectors. PFCSPs are cast and cured in a place other than their final location in the building skeleton. FC is chosen as an alternative lightweight material to heavy and normal concrete [26]. Structural FC could aid in achieving the desired strength and reducing the total dead load relative to traditional concrete. It also exhibits ideal strength–weight characteristics. The reduced total dead load encourages a decrease in foundation size and facilitates transportation and operation. The use of cranes in assembling is also minimized. Hence, the overall construction cost can be reduced by up to 10%. In this regard, PFCSPs are considered as one of the valuable contributions of structural researchers to the resolution of the problem faced by foundation engineers in providing enough bearing capacity foundation that can carry the overall dead load of building superstructures in grounds made unstable by peat soil or mud. Recently, the development of lightweight concrete sandwich panels has increased in construction industries worldwide [27]. PFCSPs are effective in certain aspects, such as in terms of the weight–strength ratio based on full-scale architectural and structural considerations.

IV. Experimental Study

PFCSP specimens of total number six and varying in heights were produced in timber formwork and tested. The PFCSPs were produced using FC. The mix composition of the FC mortar used was 541:1080:243 for cement: sand: water: respectively per meter cube. In addition, chemical protein foam agent of volume of 82 kg/m^3 was used to reduce a certain percentage of the dry density. The control concrete mortar mix ratio of 0.16: 0.267: 1 was used for water, cement and sand respectively. A square welded mild steel BRC mesh of 6 mm diameter deformed bars with 100 mm × 100 mm openings were used as longitudinal and transverse reinforcements for the outer and inner concrete wythes of PFCSP. Five steel truss-shaped shear connectors made of 6 mm diameter round mild steel bars with 45° inclination angle ran along the heights of PFCSP. Expanded polystyrene (EPS) foam was used as a sufficient thermal insulation material between the two concrete wythes.

A. Material Properties

The FC compressive was 24.83 N/mm^2 at 28 days, elastic modulus E_c 17.74 kN/mm^2 and splitting strength f_{ct} 1.89 N/mm^2. The steel bar and shear connector were having yield strength f_y 490 N/mm^2 and 300 N/mm^2, elastic modulus E_s 152 kN/ mm^2 and 112 kN/ mm^2, respectively. Moreover, two types of electrical strain gauges (*ESG*s, with 120.2 ± 0.2 mm accuracy values), namely, 2 and 67 mm lengths, were used to measure strains on steel and surface of foamed concrete, respectively.

B. Details of the Design Formulation of PFCSP

The PFCSP specimens were varied in heights between 1750 and 3000 mm with 250 mm constant increment between former and subsequent specimen. The PFCSP overall thickness is 125 mm as such divided into; 50 mm thickness of outer and inner concrete wythes each and 25 mm thickness of insulated layer in between. In addition, a 15 mm concrete cover. Further, the details of the typical design properties of PFCSP including positions of strain gauges are shown in Fig. 2.

C. Procedure of Preparation, Fabrication, and Casting

The fabrication formwork is made from a timber of 18 mm thickness. All panels were designed as a load-bearing wall with different heights. The FC was poured in to form the bottom concrete wythe and was self-compacted without any external aids. Then, the BRC was inserted into the concrete to the bottom wythe with a cover of 15 mm. The polystyrene insulation sheets were implanted between the shear connectors, after which the BRC of the top wythe was then laid and tied to shear connectors to hold both wythes together. The surface was trowelled to a smooth finish. Therefore, the overall process of preparation, fabrication, and casting of PFCSP specimens is shown in Fig. 3.

D. The Setup and Testing Procedures

PFCSP slab specimens were tested under in-plane shear load. All PFCSP specimens were placed horizontally inside Magnus frame plus special steel frame shaped as U-steel channel attachment plates to achieve a full fixed support at one end and the other end remains free. The vertical in-plane shear tests were performed using a universal testing machine of 2000 kN capacity, as shown in Fig. 4. The panels were assumed to be installed in a single-story building.

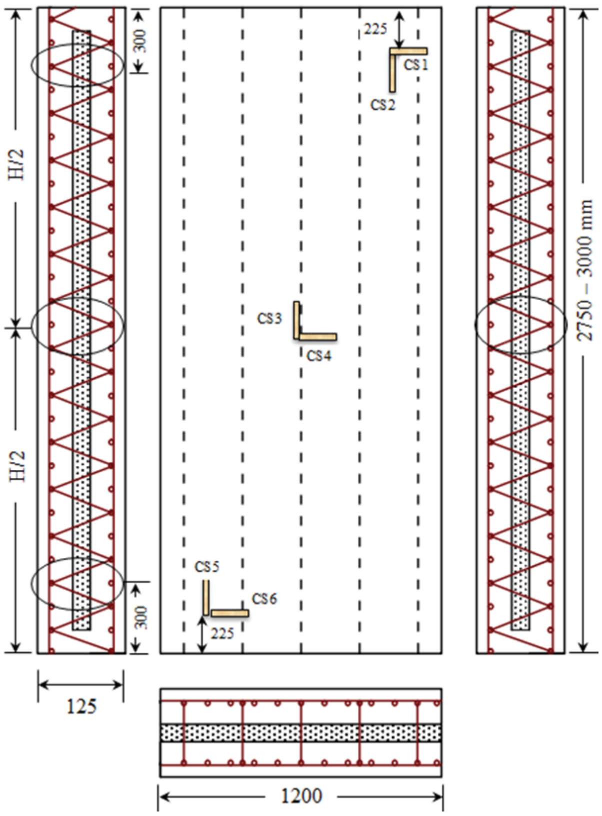


Fig. 2 Details of the PFCSP wall

All the specimens were also subjected to vertical in-plane shear force near the free-end edge supported with a pinned support 250–300 mm away. The shear force was loaded as a point covering the full thickness of the tested panel. The load was applied in gradual increments prior to activation by a manually-operated pump. All the specimens were cleaned with a sponge, and were painted white to clearly observe the cracking patterns and for easy reading and marking. However, 3 kN was the first load applied to ensure that all the instruments were properly working followed by 40 kN as gradual load increments until failure. At every stage of loading, strains in steel (reinforcement bars, shear connectors) and concrete were automatically recorded using a Data Logger UCAM-70A|KYOWA and scanner USB-70A-10. The shear cracking patterns were observed, marked on the surface of the specimen at each increasing on the applied load, with the corresponding load indicated. However, in this full-scale laboratory study, the structural behaviour of the test specimens was investigated during the time of applying the in-plane shear load. Moreover, the linear variable displacement transformers (LVDTs) were positioned at three different locations along the full height of the test wall specimen subjected to in-plane shear loads as described in Fig. 4. The three LVDTs were labeled as SD1, SD2, and SD3, which were used to measure the deflection of the full thickness of the concrete wythes. The LVDTs were functioned to measure and record deflection automatically.

Fig. 3 Preparation, fabrication, and casting of PFCSP specimens

The requirements for setting up the testing frame are:

- It must have the capacity to withstand the applied load of the tested panel.
- It must be capable of accommodating the size of tested panel.
- The ground screw bolts should be strong and stiff enough to inhibit its deformation from considerably influencing the measurements of specimen testing.

V. Results, Analysis and Discussions

The experimental tests and the adopted FEA model results obtained were analyzed and discussed in terms of ultimate vertical in-plane shear strength capacity, load-deflection profile, slenderness ratio, load-strain relationship, load failure mode, and shear cracking patterns and propagations.

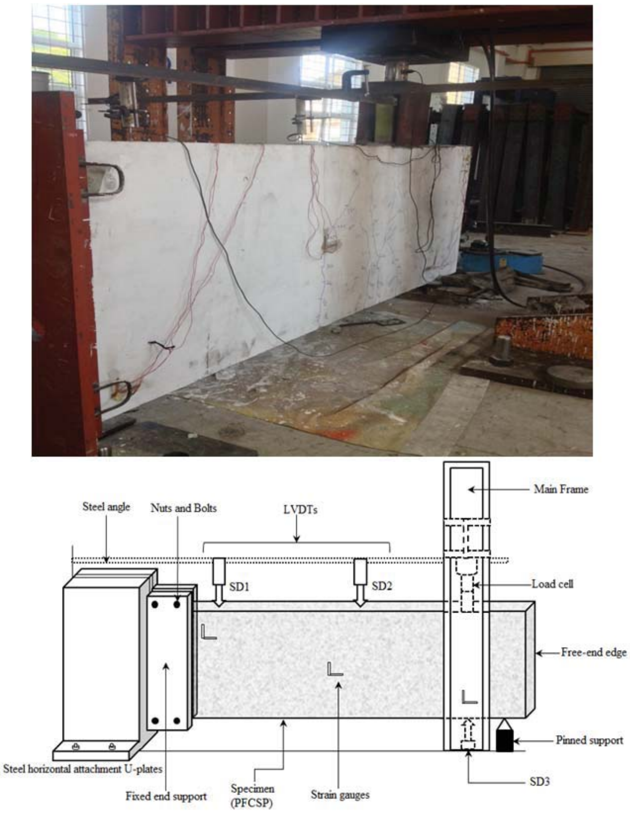


Fig. 4 Vertical in-plane test setup and procedures

A. The Load-Deflection Profile

The shear load–deflection curves for panels GS1 to GS6 at SD1 (SD1 = 350 mm from the free end of panel GS1) are shown in Fig. 5. By increasing the load at the tension side, the PFCSPs deflected elastically even after the first cracks occurred in the concrete when approximately 40% of the ultimate load was applied. The load–deflection relationship was almost linear until approximately 78% to 94% of the final failure load. The panel behaved like a cantilever deep beam. However, the load–deflection relationship became non-linear after concrete surface cracking patterns became clearly visible. The deflection curves also approximately changed proportionally to the increase in load. The crack length reached almost two-thirds of the specimen walls, especially panels GS1 and GS2, and the panel became unstable because the sliding of the top portion onto the bottom side initiated a collapse. The overall behavior of the PFCSP walls was in agreement with the tests performed under direct shear loading as reported by Kabir [28].

The typical deflections along the height of the wall panels of GS1 through GS6 at different load stages are depicted in Figs. 6 (a) and (b). The deflections of the three LVDTs were very small in the early loading stages, specifically at point SD1. Such deflections of the LVDTs eventually increased just before failure in all the panels, especially in panel GS6. The deflection mode of GS6 of H/t = 24 was 76.51%, which was significantly greater than that of GS1 of H/t = 14. This result could be attributed to GS6 having a 71.43% higher slenderness ratio of 14 to 24 in comparison with GS1. The deflection values of the panels did not differ much because of the influence of the width-to-height ratio of the specimens.

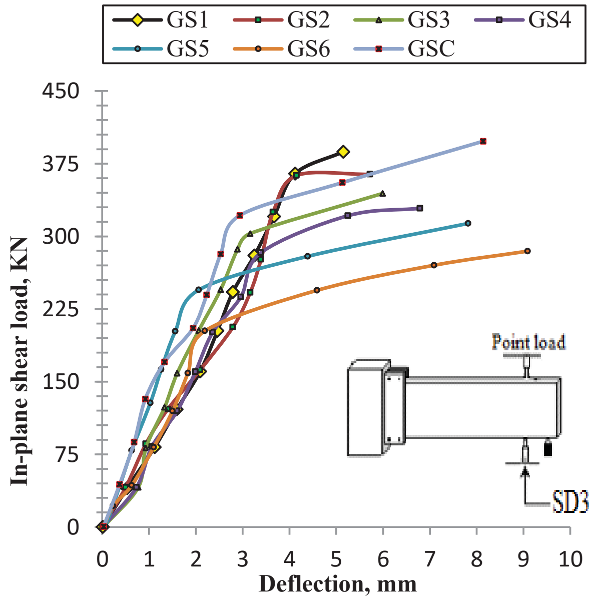
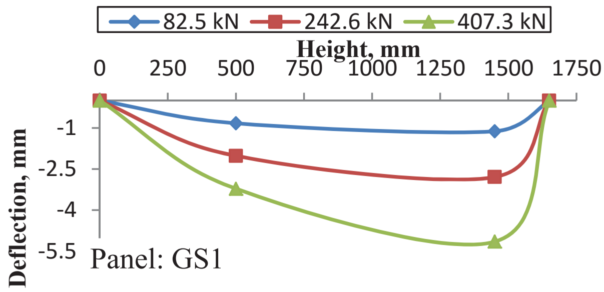
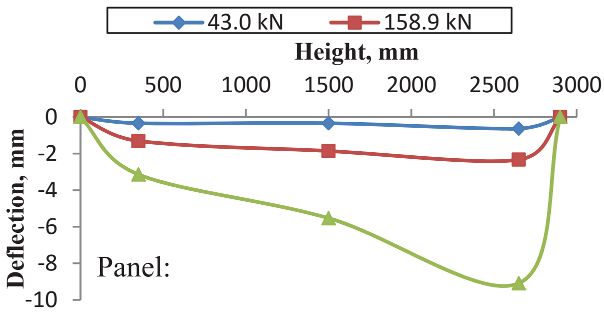


Fig. 5 Load against deflection, at SD3 from the top end for panels GS1-GS6



(a) Deflection against span for panel SF1



(b) Deflection against span for panel SF6

Fig. 6 Deflection along the height of panels SF1and SF6 at different load stages

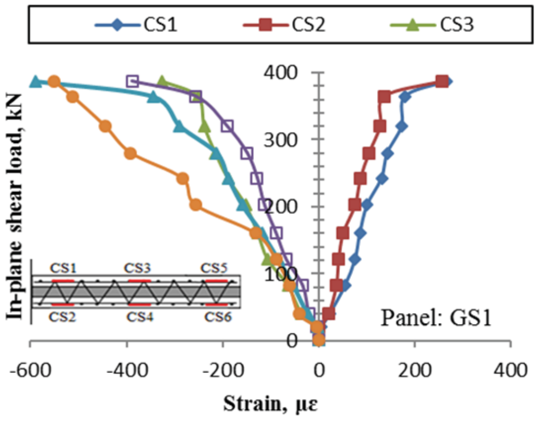
The shear failure occurred near the support after massive cracks extended diagonally to the point of shear load. The FC wythes were likely to deflect in ductile mode rather than in brittle mode in all the tested panels. The influence of the slenderness ratio on ultimate strength in resisting in-plane shear loads was effectively studied, and the deflection curves were drawn. In conclusion, all the panels functioned as solid wall system components in a composite manner.

B. Strain Characteristics of the Composite Panel

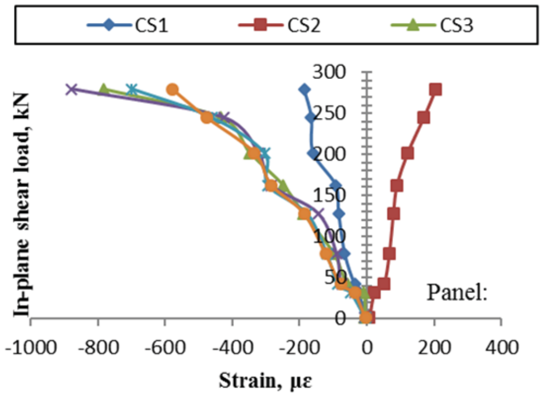
A typical strain distribution across the thickness of the wall at a distance of 350 mm from the free end edge at different load stages is shown in Fig. 8. The strains varied linearly and with almost negligible discontinuity across the insulation layer even after the appearance of the first crack at 120 kN load. This discontinuity remained small until the application of 89% of the failure load following the occurrence of serious shear cracking, especially at the bottom tension zone. The same figure shows that the strain discontinuity became large just before the ultimate specimen load. With the increment in applied load, the recorded maximum compressive strain was 698 με. However, this strain value exceeded the ultimate compressive strain of concrete (350 με) given by BS8110-1-92. The wythes also showed small tensile strains at failure load as expected. In conclusion, the same behavior was observed in both concrete wythes through the compressive strain measurements. This result indicated that the tested panels behaved like an RC solid wall panel reported by Riva [29].

C. The Load-Strains on Concrete

The load–strain relationship observed in the concrete wythe surface for panels GS1 to GS6 is depicted in Figs. 7 (a) and (b). The load–strain behavior was elastic until the first transmittance to the plastic zone at 42.2% load following the appearance of the initial concrete crack such that the strains were less than 200 με.



(a) Load against strain on concrete for panel SF1



(b) Load against strain on concrete for panel SF5

Fig. 7 In-plane shear load vs. surface strain in concrete wythes at different locations for panels GS1and GS5

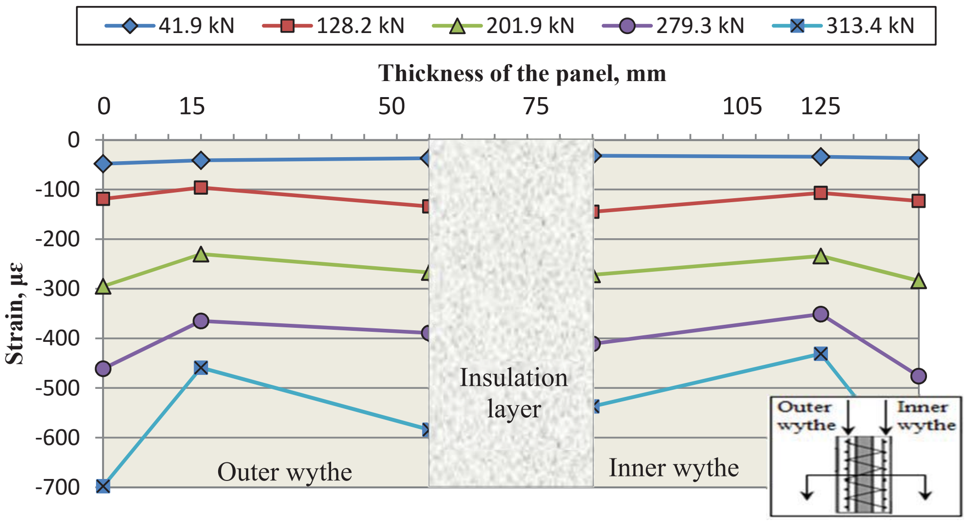
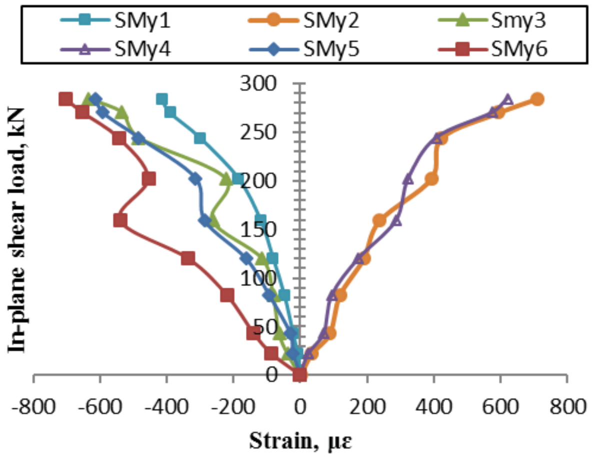


Fig. 8 Typical strain variation across the mid-height of a PFCSP at different load stages

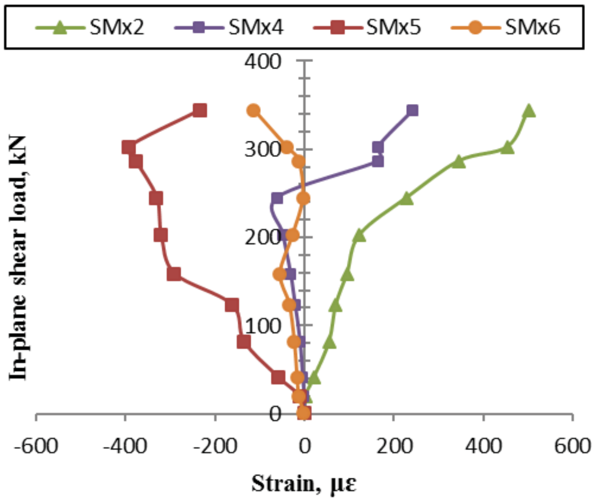
The increase in concrete strains was still near proportional as the load increased. The maximum compressive strain of panel GS5 was 877 με at the mid-height at CS4. This concrete compressive strain value exceeded the specified ultimate strain of concrete (350 με) cited in BS8110-1-1992 [30]. All the panels reportedly failed because of the significant shear cracking of the concrete wythes diagonally positioned from the support to the shear loading point, followed by the final crushing of concrete near the support. The compressive concrete strains of the panels at CS3, CS4, CS5, and CS6 labeled in the vertical direction exhibited more plastic characteristics in the compressive zone compared with those at CS1 and CS2, which were closer to the fixed support and behaved more elastically in the tension zone. CS1 and SC2 did not take show much compressive strains compared with those at the mid-height and/or the other locations near the free end support of the tested specimen. The strains of (CS1, CS2) mostly reacted in the tension zone, whereas panel GS5 only behaved in both compression and tension zones. The strain distribution at the two concrete wythes indicated that the recorded compressive strains were very close to symmetric at either the linear elastic zone or the non-linear plastic portion. In conclusion, all samples tested precisely behaved in a fully composite action even after the occurrence of major shear cracking.

D. The Load-Strains on Steel

The developed strains in steel were very small and did not exceed the value of 575 με at SC3. The typical strains of the steel reinforcement bars at the vertical and horizontal axes for different panels at three different locations are depicted in Figs. 9 (a) and (b). The maximum steel strain value recorded was 711 με in panel GS6 at SM5 in the y_{axes} and 501 με in panel GS3 at SM3 in the x_{axes}. The same figure shows that steel deformation was elastically and linearly deflected during the initial time of loading. However, the natural behavior of the steel reinforcement bars became plastic when initial concrete cracking appeared. Therefore, the deformations were noticeably proportional to the load increments. The yielding of steel reinforcement probably occurred because of the small cross-sectional area of the steel bars with a 6 mm diameter and the concrete cracking that appeared along the specimen concrete wythes, which resulted in more stresses that tended to cause bending. However, the positive and negative signs were assigned to indicate the tension and compression behaviors of the steel, respectively. The strain behavior of all the panel shear connectors under shear loadings was evidently very small and was lower than the yield strain. The typical load–strain profiles were generally the only ones plotted in both the compression and tension zones along the specimen height. A similar behavior was observed for the steel reinforcement bars that exhibited either compression or tension, or sometimes both, especially in the horizontal axes.



(a) Load against strain on steel at Y_{axes} for panel SF6



(a) Load against strain on steel at X_{axes} for panel SF3

Fig. 9 Load against strains on steel bars

E. The Influence of Slenderness Ratio (H/t)

The influence of slenderness ratio on the ultimate in-plane shear strength capacity of the panels was determined through a full-scale experimental investigation, as illustrated in Fig. 10. The slenderness ratios (H/t) for panels GS1, GS2, GS3, GS4, GS5, and GS6 were 14, 16, 18, 20, 22, and 24, respectively. The load increases were 5.6, 5.5, 4.5, 4.7, and 9.2% for panels GS1 to GS6. The decrease in the ultimate shear strength capacity of the tested specimens was non-linearly proportional to the increasing slenderness ratio. The reported increasing was roughly 26.51% for a reduction of H/t from 24 to 14. Therefore, the reduction in the in-plane shear strength was regarded as proportional to the increase in the slenderness ratio. The reduction effect can be maintained by increasing the overall thickness of the concrete wythes.

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F. Cracking Patterns and Failure Loads

The vertical in-plane shear load increased in constant increments until failure. The primal load applied was 3 kN to determine the accuracy and performance of the instruments and the measurement preparation. The gradual load increment was set to 40 kN. The cracking patterns were observed and marked corresponding to the increasing loads along the whole panel height as shown in Fig. 11. All six PFCSPs plus one PCSP were experimentally tested under vertical in-plane shear loading through the same procedures.

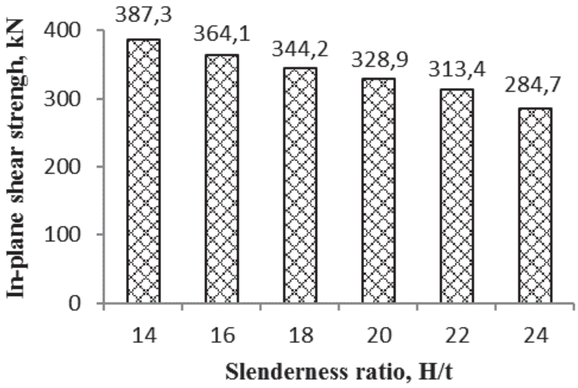


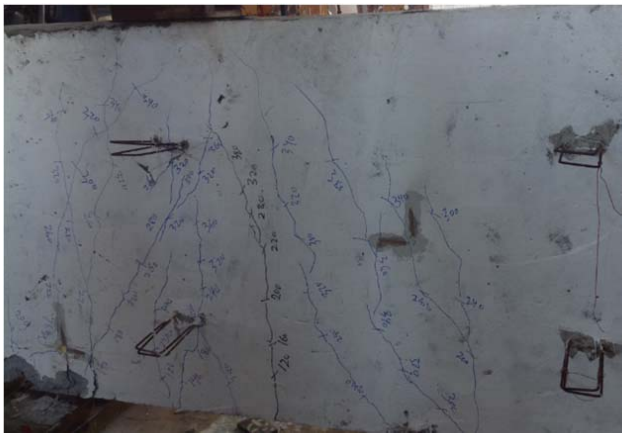
Fig. 10 Load against slenderness ratio (H/t)

The appearance of the first crack and the failure load were recorded corresponding to the appearance of cracking patterns and shear loads, respectively. The recorded appearance of the first crack was at loads of 140, 140, 120, 140, 120, and 120 kN for panels GS1, GS2, GS3, GS4, GS5, and GS6, respectively. Cracking patterns was reported occurring simultaneously in both concrete wythes in an inclined shape directed diagonally toward the applied load

The crack pattern records of the tested panels indicated that the cracks primarily occurred as flexural cracking and shifted to an inclined shape directed diagonally from the support and near the mid-span of the panels toward the applied point load, followed by concrete crushing in all cases at the bottom close to the panel support. Violent concrete cracking also occurred near the support of GS5 of H/t = 22 and GS6 of H/t = 24. However, the first cracks appeared at 35-42% load of the ultimate failure load, as shown in Table I. In addition, the cracking patterns mostly occurred in the inclined direction. The cracking pattern widths were very small at most because of the naturally lightweight and ductile concrete materials used to cast the wythes.

TABLE I
SHEAR CRACKING PATTERNS AND ULTIMATE IN-PLANE SHEAR STRENGTH CAPACITY FOR PANELS GS1 TO GS6

Panel	H/t	1st shear crack, kN	Ultimate in-plane shear load, kN
GS1	14	140	387.3
GS2	16	140	364.1
GS3	18	120	344.2
GF4	20	140	328.9
GS5	22	120	313.4
GS6	24	120	284.7



(a) Typical shear crack patterns for panel GS5 along the height of the concrete wythe

(b) Typical shear crack patterns for panel GS1 along the height of the concrete wythe

Fig. 11 Typical shear cracking patterns of PFCSP as wall

VI. ANALYTICAL STUDY

The developments in construction and civil engineering in recent decades have attracted the attention of international research schools and centers. These developments have strongly alarmed structural engineers and architects, thereby leading them to create sound constitutive models and dynamic numerical techniques using computerized software to resolve engineering problems and validate the actual behavior of developed applications. A non-linear finite element analysis (FEA) is an essential technique to simulate the structural behavior of RC structures for sustainable and continuous development. The FEA attributes are used to verify the degree of accuracy of structural applications experimentally used for engineering practice. The method is also utilized to analyze and design complicated problems. Several commercial software is now available in the market (e.g., LUSAS - 2000). The present study performed an analytical study of non-linear models to separately simulate the structural behavior of PFCSPs under vertical in-plane shear load. Due to that the shear connectors has no much contribution in resisting the in-plane shear loads because of its layout in perpendicular direction to the applied load as found in the experimental tests, Thus, in the adopted FEA models was ignored the design of shear connectors and the simulation was accomplished as a 2D solid wall. The model was subjected to concentric load acting vertically in-plane of the panel design geometry as shown in Fig. 13. FC and steel (with main reinforcement bars and shear connectors) were modelled by assigning a four noded 2-D isoparametric plane stress element and a 2-D isoparametric bar element, respectively. The areas of steel reinforcement bars and compressive strength values of foamed concrete were inserted and the adopted dimensions were similar to the actual specimen's details.

The PFCSP was simulated as a 2D FEA model by taking a horizontal section along its full height. The PFCSP wall was discretized through a number of four-node plane stress elements (Fig. 13). A non-linear FEA models were used by considering concrete non-linear elastic to plastic material. The model was subjected to concentric load acting vertically in plane of the panel design geometry. The boundary conditions and applied load were similar to the experimental test setup (Fig. 4). The influence of slenderness ratio was investigated by varying an important parameter, such as height. The stress distribution in the panels was also investigated.

TABLE II
COMPARISON OF RESULTS BETWEEN THE 2D FEA MODELS AND EXPERIMENT

Name of panel	H/t	Ultimate In-plane strength, kN		$\frac{F_u^{Exp} - F_u^{FEA}}{Fu_u^{Exp}}$ (%)
		FEA	Experiment	
GS1	14	366	387	5.40
GS2	16	355	364	2.38
GS3	18	317	344	7.90
GS4	20	298	329	9.33
GS5	22	290	313	7.42
GS6	24	286	285	0.47

The load-deflection profile of panel GS4 as obtained experimentally and verified using the 2D FEA models is shown in Fig. 12. During the initial level of load increments, the first results were noticeably well-correlated until the primal crack load appeared. But at the ultimate state, the experimental deflection value declined by only 17.87% in comparison with the value obtained from the FEA models. The difference could have been caused by the unintended eccentricity or the setup of the actual boundary conditions in the experimental tests. The same figure shows that the experimental shear strength of GS4 was decreased by 9.33%, compared the value obtained via the FEA models (Table II). The experimental testing model behaved in a less stiff manner than the FEA models did after the load increased and specimen cracking appeared, especially at the non-linear level. Minimal changes were also evident, especially after a massive diagonal shear cracking occurred.

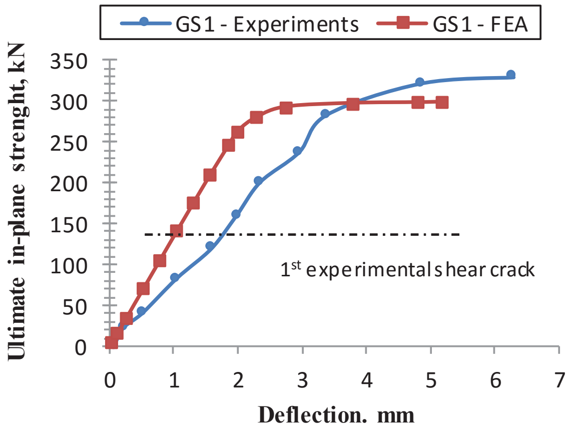


Fig. 12 Load-deflection profile for panel GS4 at 350 mm far from the free end edge

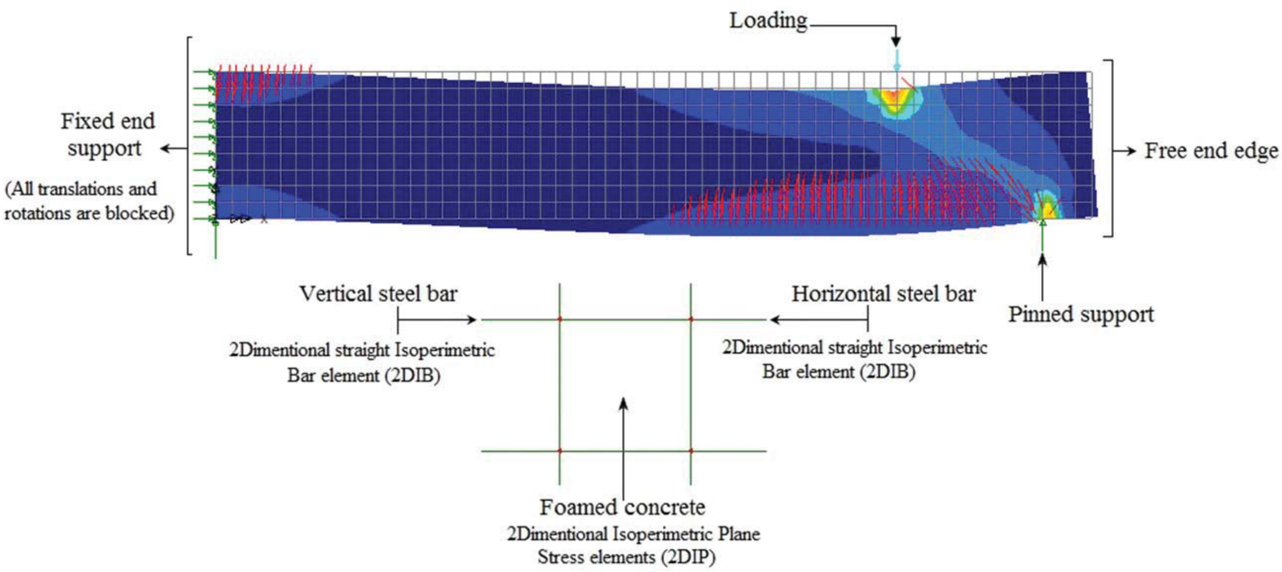


Fig. 13 The FEA model idealisation of PFCSP, loads and boundary conditions

VII. Theoretical Study

The computed ultimate strength values were obtained using the ACI318M-5 Equation (1). However, in the experimental ultimate in-plane shear strength for panels GS1 (H/t = 14), GS2 (H/t = 16), GS3 (H/t = 18), GS4 (H/t = 20), GS5 (H/t = 22), and GS6 (H/t = 24); the differences decreased to 12.3% and 4.5% and increased to 1.8%, 7.4%, 11.7%, and 10.9% compared with the ACI318M-5 Equation (1), respectively, as shown in Fig. 14.

The recommended equation by ACI318M-5 [31]:

$$V_n = 0.83 \times \sqrt{f_{cu}}\,(tb) + \left(\frac{A_v \times f_y \times d}{s_2}\right) \quad (1)$$

where; A_v: area of vertical shear reinforcement, mm^2; s_2: spacing of horizontal reinforcement, mm; b: width of the wall, mm; t: thickness of concrete wall, mm; f_{cu}: characteristic strength of concrete, Mpa; f_y: characteristic strength of yield steel reinforcement, Mpa.

Based on the results obtained, using the expression of the recommended ACI318M-5 Equation (1) was very conservative and significantly closer to the experimental test results obtained. Moreover, ACI318M-5 Equation (1) is considerably increased the slenderness function to improve the performance of the developed PFCSP application.

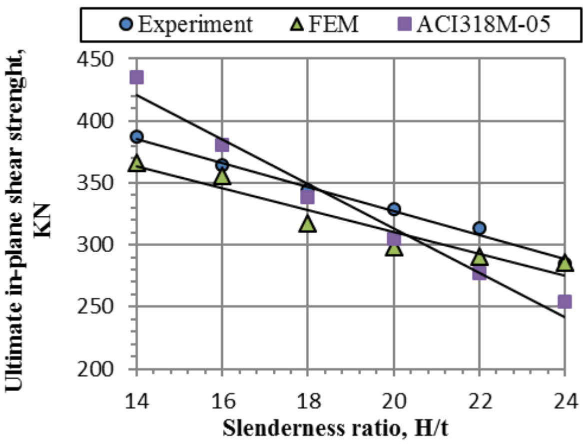


Fig. 14 Comparison of the designed ultimate in-plane shear strength of wall panels

VIII. Conclusion

The design performance of the six PFCSP specimens with slenderness ratios ranging from 14 to 24 and subjected to vertical in-plane shear loads was studied analytically via FEA models, theoretically and experimentally. In this paper, the conclusion was made on the basis of the results obtained via these three studies approaches:

a. The ultimate in-plane shear strength capacity of the tested specimens increased by 26.51 and 21.91% as H/t decreased from 24 to 14, as obtained experimentally and analytically, respectively.

b. It was found that the panels behaved in fully-composite manner even before the load of failure as integrated by steel truss-shaped shear connectors of 6 mm diameter round bar.

c. In the experimental tests, it shown that the shear cracking appeared at 35% to 42% of the ultimate load at failure. The shear crack patterns of the tested panels indicated that the cracks primarily occurred as flexural cracking and shifted to an inclined shape direct from the support and near the mid-span of the panel diagonally toward the applied point load.

d. All the tested PFCSP panels under vertical in-plane shear loads failed because of concrete crushing motivated by the width-to-height and further by its lightweight material origin and brittle fracture.

e. The analytical results obtained using the FEA models, the experimental test data, the theoretical calculation values were compared, and a significant agreement was observed, along with a high degree of accuracy; the slenderness ratio function also increased.

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