Structural performance of FCS wall subjected to axial load

Noridah Mohamad a,⇑, Goh Wan Inn a, Redzuan Abdullah b, Abdul Aziz Abdul Samad a,c, Priyan Mendis c, Massoud Sofi c

⇑ Corresponding author at: Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn Malaysia, Malaysia.
E-mail addresses: noridah@uthm.edu.my, noridah.mohamad@unimelb.edu.au (N. Mohamad).

a Faculty of Civil and Environmental Engineering, Universiti Tun Hussein Onn Malaysia, Malaysia
b Faculty of Civil Engineering, Universiti Teknologi Malaysia, Malaysia
c Department of Infrastructure Engineering, The University of Melbourne, Australia

The primary purpose of this paper is to present the structural behavior of precast foamed concrete sandwich walls (FCS) strengthened with double steel shear connectors (DSC) subjected to axial load. The discussion of results addresses the effect of slenderness ratio (H/t) on the behavior of the walls. It also addresses the significant effects of using double shear connectors on these behaviors compared to single steel shear connector (SSC). The FCS wall was modeled using ABAQUS and validated by the results from experiment and previous research. Parametric finite element study (FEA) was conducted by simulating these walls with various H/t under axial load. The structural behavior recorded from FEA includes its ultimate load, failure mode, load-deflection profiles, strain distribution across the wall's thickness, and ultimate shear load. It was found that as H/t increased, the ultimate load decreased but the maximum horizontal deflection increased. FCS with H/t ≤ 25 failed from crushes and cracks within the top and bottom area while FCS with H/t ≥ 25 failed from buckling at its mid-height. It was also noticed that FCS with lower H/t experienced more uniform strain distribution across its thickness. FCS with DSC was able to sustain higher load but it deflected less compared to walls with SSC, caused by higher stiffness due to larger steel areas. It is interesting to find that FCS walls with DSC behaved in a more composite manner and was able to sustain higher ultimate shear force under push of loading test compared to FCS walls with SSC. These findings show that the FCS wall with DSC is suitable to be used as a load-bearing wall.

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1. Introduction

Precast prefabricated concrete is seen as an alternative to the traditional in-situ concrete because it satisfies the strength requirement needed for a building while offering new technology, which can speed up the construction. Previous researches have proved that this type of panel has a great potential to be used in building construction. It can provide solutions to shortening of affordable houses that is faced in many parts of the world now [1–3].

Precast sandwich panel is an industrialized building system, which designed to have higher strength to low weight ratio. It consists of three or more layers bonded together as a single unit. The most common sandwich panel only consists of three layers with two outer skin layers enclosing a core or inner layer. The outer skin layers are known as wythes while the inner core layer, usually lighter and used as the insulation material, is known as the core. There are various materials that can be used to cast a sandwich panel, either as the wythes or as the core layer [4–9].

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sandwich panels made from various lightweight materials have been studied to determine their mechanical and thermal insulation properties [10–12]. The positive remarks on the results on the lightweight materials' properties from these studies has led to further study on its structural behavior and its possibility to be used as a structural element [13–16]. All studies showed that sandwich panels could be designed to behave either as a non-composite, partially composite or fully composite system.

The degree of composite action achieved by a concrete sandwich panel system is dependent on the configuration and material used as the shear connection system. Shear connectors are mainly used to transfer the in-plane shear forces between the two wythes. The structural efficiency of a sandwich panel is dependent on the level to which interfacial shear is transferred between the wythes [17]. A sandwich panel that achieves complete interfacial shear transfer between the wythes is said to be fully composite, as both wythes work together as a single unit to resist applied load all the way to failure [18].

There have been many studies on the performance of various types of shear connectors in sandwich system. Kim and Young examined the use of GFRP shear grids in sandwich panels where it was found that the GFRP grid produced favorable degrees of composite action resulting in increased flexural strength [19]. Meanwhile, studies on behavior of high strength CFRP shear grids systems has concluded that CFRP shear grids produced nearly full composite action in addition to maintaining the thermal integrity of various panel systems [20–28].

Several previous studies have also been conducted on steel bar as shear connectors but the numbers are limited. Benayoune et al. and Mohamad both studied the performance of steel shear connectors in precast sandwich wall system [29,30]. Benayoune et al. studied the structural behavior of precast concrete sandwich walls (PCSP) using conventional concrete as the wythe and polystyrene as the insulation layer while Mohamad studied the performance of precast lightweight foamed concrete sandwich walls (PLFP) using foamed concrete as the outer wythe and polystyrene as the insulation layer. In both studies, single steel shear connector (SSC) was used to strengthen the wall. Results from both studies showed that the ultimate strength achieved in PLFP was slightly lower than the ultimate load achieved in the PCSP but both sandwich walls behaved in a partially composite manner. Slenderness ratio (H/t) was proven to have significant effects on the structural behavior of both walls. These studies have shown that sandwich wall from lightweight foamed concrete strengthened with efficient steel shear connector system has potential to be developed as a load bearing wall.

This paper discusses the effects of H/t and DSC on the structural behavior of lightweight FCS walls subjected to axial load by means of FEA. FCS wall consists of lightweight foamed concrete wythes which enclose a polystyrene layer. It is strengthened by steel bar reinforcement, which is embedded in the wythes. The different layers in the wall are held together by using DSC, which are inserted through the layers diagonally. Full-scale model of FCS walls (FCS-F) with DSC was first validated by full-scale model of PLFP walls with SSC from previous study by Mohamad [30] to confirm the material models and assemblage of various model parts in the wall. To confirm the steel material model in the FCS-F wall, it was further validated by experimental results conducted on half-scale FCS walls (FCS-H) strengthened with DSC. Since the experimental work was conducted on half-scale FCS walls due to limitation in the laboratory, this second validation is also to confirm that the results from half-scale FCS is able to predict the results of full-scale FCS according to scaling law from previous research [29–32].

After validation process, parametric study on full-scale FCS-F walls with various H/t under axial load was conducted. The aim of this study is to determine the wall’s structural performance in the context of its ultimate load, failure mode, load-deflection profile, and strain distribution across its thickness. Simulation of push off load test was also conducted on the wall to obtain its ultimate shear load. The discussion of results obtained will mainly look at the influence of H/t and efficiency of using DSC in the FCS walls. The results were studied and analyzed to see the improvement, if any, on the structural behavior of the FCS sandwich panels strengthened with DSC. The answers looked for from this study are its ability to sustain larger ultimate and shear loads and whether it behaves in a more composite manner compared to similar walls strengthened with SSC.

### 2. Methods and material

The methodology adopted in this study was finite element analysis validated by results from experiment and previous research. Other than to investigate the effects of H/t on the structural behavior of FCS walls, the main study in this research is to determine the influence of DSC on the overall structural behavior of the walls. Therefore, both FCS-F walls with DSC and SSC were modeled and simulated under axial load. As illustrated in Fig. 1(a), SSC is the truss shape steel connectors bent at angle 45 degrees. It is inserted through the wythes and core layers of the sandwich wall and tied to the steel reinforcement which is embedded in the wythes. Meanwhile, DSC consists of two (2) SSC, which are tied to each other using wire mesh as illustrated in Fig. 1(b).
2.1. Experimental programme

The experimental work involved casting of five (5) half-scale FCS-H walls strengthened with DSC tested under axial load. The materials used to fabricate the FCS-H wall are cement, water, fine sand and foam. The foam was produced with foam generator through foam agent dilution with water. In this process, a synthetic foam agent based on protein hydrolyzates was used. The mixing method was obtained from several trial mixtures completed in previous research by Mohamad [30]. The ratios for material’s composition are 2:1, 0.65, and 0.55 for sand:cement, foam:cement, and water:cement, respectively.

Material property tests were conducted on the foamed concrete cubes and cylinders to determine its compressive strength, tensile strength and modulus of elasticity. Three cubes were tested to determine the compressive strength at each 7, 14 and 28 days base on specifications in BS 1881: Part 116 [33]. Three cylinders were tested under split cylindrical test to determine its tensile strength at 28 days and three cylinders were tested under compressive test to determine its Young’s Modulus, $E$, and Poisson ration, $\nu$, at age 28 days, according to BS 1881: Part 117 [34] and BS 1881: Part 121 [35], respectively.

2.1.1. Components of FCS-H wall

The different components in FCS-H wall consist of foam concrete wythe as the outer layers which enclosed a polystyrene insulation layer. The wythes were strengthened by embedding 6 mm steel mesh with 75 mm x 75 mm openings as reinforcement in both wythes. It was further strengthened with the DSC from 6 mm diameter steel bar inserted across the height of the walls as shown in Fig. 2(a). Five (5) numbers of DSC were embedded along the height of wall at 75 mm spacing. Capping at both ends of wall was cast by using normal concrete from grade 25 with thickness of 50 mm. These caps functioned to prevent premature cracking within the area of top and bottom edge of foamed concrete as shown in Fig. 2(b).

2.1.2. Material properties

The characteristic properties for all the materials used in the FCS-H wall’s fabrication were obtained from the material tests conducted on foamed concrete and steel bars. Table 1 gives the properties of foamed concrete for the wythe, normal concrete for the capping and expanded polystyrene for the core layer. Table 2 gives the properties of steel bar with diameter 6 mm and 9 mm.

2.1.3. Designation and dimension of FCS walls

The designation and dimensions of all the half-scale FCS-H wall specimens are as listed in Table 3 and as illustrated in Fig. 3.

2.1.4. Experimental Set-up

The wall specimens were tested using Magnus Frame as shown in Fig. 4 according to ASTM E72-10 [37]. Two units of Linear Voltage Displacement Transducers (LVDT) were placed at the middle of both rear and front surface of the panel to record the horizontal displacement.

2.2. Finite element model

A three dimensional nonlinear finite element model for full-scaled FCS-F wall with DSC was developed using ABAQUS 6.13 [38] to study its structural behavior under axial load. Material properties of foamed concrete obtained from experiment were used to calculate the concrete damage plasticity (CDP) parameters. The parameters were calculated based on the relationship between the stress-strain tension and compression loading.

The concrete damaged plasticity model is the failure criterion required in the analytical process. The model is a continuum, plasticity-based, damage model for concrete. It assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material. Concrete damaged plasticity model uses stress/strain relationships to correlate parameters for relative concrete damage for both tension and compression. It provides a general capability for modelling concrete and other quasi-brittle materials in all types of structures. This model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. Under uniaxial tension the stress-strain response follows a linear elastic relationship until the value of the failure stress is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure. Under uniaxial compression the response is linear until the value of initial yield. In the plastic regime the
Table 1
Properties of foamed concrete, normal concrete for capping, and expanded polystyrene [36].

<table>
<thead>
<tr>
<th>Material</th>
<th>$P_y$ (MPa)</th>
<th>$F_t$ (MPa)</th>
<th>$E$ (GPa)</th>
<th>Density, $\rho$ (kg/m$^3$)</th>
<th>Poisson’s ratio, $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foamed concrete</td>
<td>7.5</td>
<td>0.8</td>
<td>12</td>
<td>1600</td>
<td>0.2</td>
</tr>
<tr>
<td>Normal concrete</td>
<td>25</td>
<td>–</td>
<td>2400</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Expanded Polystyrene</td>
<td>–</td>
<td>–</td>
<td>0.8963</td>
<td>16</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 2
Properties of steel used as reinforcement and shear connectors in the FEA.

<table>
<thead>
<tr>
<th>Diameter</th>
<th>$\sigma_y$ (MPa)</th>
<th>$P_u$ (MPa)</th>
<th>$\varepsilon$ failure strain</th>
<th>$E_s$ (GPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$\nu$ Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 mm</td>
<td>359</td>
<td>374</td>
<td>0.0049</td>
<td>200</td>
<td>7,700</td>
<td>0.3</td>
</tr>
<tr>
<td>9 mm</td>
<td>343</td>
<td>381</td>
<td>0.0061</td>
<td>200</td>
<td>7,800</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 3
Designation and details of dimensions for FCS-H wall specimens.

<table>
<thead>
<tr>
<th>Panel</th>
<th>$H \times W \times t$ (mm$^3$)</th>
<th>Slenderness ratio, $H/t$</th>
<th>$t_1$ (mm)</th>
<th>$t_2$ (mm)</th>
<th>$c$ (mm)</th>
<th>Reinforcement (vertical and horizontal)</th>
<th>Diameter of shear connectors</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCS-H1</td>
<td>1000 $\times$ 375 $\times$ 70</td>
<td>14</td>
<td>20</td>
<td>30</td>
<td>15</td>
<td>6 mm$\Phi$75 mm c/c</td>
<td>R6</td>
</tr>
<tr>
<td>FCS-H2</td>
<td>900 $\times$ 375 $\times$ 50</td>
<td>18</td>
<td>20</td>
<td>10</td>
<td>15</td>
<td>6 mm$\Phi$75 mm c/c</td>
<td>R6</td>
</tr>
<tr>
<td>FCS-H3</td>
<td>1000 $\times$ 375 $\times$ 50</td>
<td>20</td>
<td>20</td>
<td>10</td>
<td>15</td>
<td>6 mm$\Phi$75 mm c/c</td>
<td>R6</td>
</tr>
<tr>
<td>FCS-H4</td>
<td>1100 $\times$ 375 $\times$ 50</td>
<td>22</td>
<td>20</td>
<td>10</td>
<td>15</td>
<td>6 mm$\Phi$75 mm c/c</td>
<td>R6</td>
</tr>
<tr>
<td>FCS-H5</td>
<td>1170 $\times$ 375 $\times$ 50</td>
<td>23.4</td>
<td>20</td>
<td>10</td>
<td>15</td>
<td>6 mm$\Phi$75 mm c/c</td>
<td>R6</td>
</tr>
</tbody>
</table>

$H$ = wall’s height, $W$ = wall’s width, $t_1$ = thickness of wythe, $t_2$ = thickness of core layer, $c$ = concrete cover.
response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress [39–42].

2.2.1. Material model
Each part of the wall was modeled separately in part module by using different types of element based on the suitability of each element. The concrete damage plasticity was used as the parameter for modeling the foamed concrete. Material properties of foamed concrete were obtained from the experiment (Table 1) to calculate the concrete damage plasticity parameters as input to the model. Table 4 shows the constitutive parameters used in CDP model for both compressive and tensile behavior of foamed concrete material. Certain parameters that were not measurable from the experiment which are flow potential, yield surface and viscosity were taken from previous research on conventional concrete by Newberry et al.[43], Mokhatar and Abdullah [42], and default values from ABAQUS.

The properties of 9 mm and 6 mm diameter steel as the reinforcement and shear connector, respectively, were used to simulate FCS-F walls under axial load. The material properties used in the finite element for both steel bars included yield stress, \(\sigma_y\), ultimate stress, \(\sigma_t\), failure strain, \(e\), modulus of elasticity, \(E_s\), mass density, \(\rho\), and Poisson’s ratio, \(v\), as tabulated in Table 2 in Section 2.1.2.

Normal concrete capping was used to prevent premature cracking and transfer the axial load to the wall. Compressive strength, tensile strength, modulus of elasticity, mass density and Poisson’s ratio of normal concrete of grade 25 were tabulated in Table 1 in Section 2.1.2. Polystyrene was used as an insulation material in the core layer. The polystyrene sheet was inserted in between the steel mesh. It had a mass density of 16 kg/m\(^3\), a Young’s Modulus of 0.896 MPa and poison ratio of 0.4 as shown in the similar table.

2.2.2. Assemblage of different parts in the FCS-F model
The assemblage of different parts in the model was conducted by connecting the parts to each other. Tie contact technique (perfect bonding) was utilized to create proper interaction between surfaces of solid elements, which are foamed concrete wythe, normal concrete capping and polystyrene as shown in Fig. 5. This technique was used to prevent slippage between surfaces of the elements. The connection between the main reinforcement and shear connectors with solid elements was obtained using embedded technique to constrain the reinforcement into these solid elements in order to create a proper bonding action.

The boundary conditions are applied in such a way that displacement will occur in the \(y\) direction which is in the direction of the applied load. For this study, panel PLFP-PA5 from previous study by Mohamad [30], which size, properties and ultimate load achieved under axial load, as shown in Table 5, was used. Models with various mesh sizes were analyzed to determine the best mesh density that would give a result, which is closer to the experimental work. The convergence of the wall model was checked with mesh density study by changing it repetitively to obtain the final mesh size for FCS-F wall model. In the convergence study, the quasi-static analysis procedure was conducted with several element sizes to illustrate mesh sensitivity. Same material properties were used for all mesh sizes. Results of the analyses using different mesh density were plotted as shown in Fig. 6. The most suitable mesh size for the wall in this study was chosen from the mesh with the lowest percentage difference of ultimate load as obtained from FEA and from experiment.

![Fig. 4. Magnus frame.](image-url)

### Table 4
Concrete damaged plasticity of foamed concrete.

<table>
<thead>
<tr>
<th>Dilatation angle</th>
<th>Eccentricity</th>
<th>Initial biaxial/uniaxial ratio, (\sigma_{bc}/\sigma_{bu})</th>
<th>(K_c)</th>
<th>Viscosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>1</td>
<td>1.12</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material behavior</th>
<th>Inelastic strain</th>
<th>Damage parameter</th>
<th>Tensile behavior</th>
<th>Yield stress (MPa)</th>
<th>Cracking strain</th>
<th>Damage parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete damaged plasticity</td>
<td>6.3</td>
<td>0.0000</td>
<td>0.000</td>
<td>0.861</td>
<td>0.00000</td>
<td>0.000</td>
</tr>
<tr>
<td>7.1</td>
<td>0.0017</td>
<td>0.000</td>
<td>0.776</td>
<td>0.00159</td>
<td>0.204</td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>0.0033</td>
<td>0.000</td>
<td>0.605</td>
<td>0.00409</td>
<td>0.476</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>0.0041</td>
<td>0.215</td>
<td>0.518</td>
<td>0.00526</td>
<td>0.582</td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>0.0047</td>
<td>0.337</td>
<td>0.431</td>
<td>0.00638</td>
<td>0.673</td>
<td></td>
</tr>
<tr>
<td>6.7</td>
<td>0.0055</td>
<td>0.456</td>
<td>0.345</td>
<td>0.00746</td>
<td>0.752</td>
<td></td>
</tr>
<tr>
<td>6.3</td>
<td>0.0066</td>
<td>0.577</td>
<td>0.259</td>
<td>0.00854</td>
<td>0.824</td>
<td></td>
</tr>
<tr>
<td>5.6</td>
<td>0.0078</td>
<td>0.682</td>
<td>0.173</td>
<td>0.00966</td>
<td>0.889</td>
<td></td>
</tr>
<tr>
<td>3.9</td>
<td>0.0127</td>
<td>0.862</td>
<td>0.086</td>
<td>0.01082</td>
<td>0.947</td>
<td></td>
</tr>
<tr>
<td>2.9</td>
<td>0.0194</td>
<td>0.934</td>
<td>0.000</td>
<td>0.01202</td>
<td>1.000</td>
<td></td>
</tr>
</tbody>
</table>

* \(K_c\) = the ratio of the second stress invariant on the tensile meridian, \(q(TM)\) to that on the compressive meridian, \(q(CM)\).*
3. Finite element analysis

3.1. Validation of FCS-F wall

FCS-F studied using FEA is a full-scale wall whereas FCS-H tested experimentally is a half-scale wall, both strengthened with DSC. Therefore, two phases of validation are needed in this study; first, validation of the full-scale FCS-F wall with SSC modeled using ABAQUS with the full-scale PLFP wall also with SSC tested experimentally by Mohamad [30], and second, validation of full-scale FCS-F wall with DSC modeled using ABAQUS with the half-scale FCS-H wall, also with DSC, tested experimentally. The first validation was to confirm the material models and assemblage of various parts in the wall and the second validation was to confirm the steel material model as the DSC in the wall.

3.1.1. Validation of full-scale FCS-F with PLFP wall

PLFP-PA8 wall studied by Mohamad [30] with similar material and geometrical properties as FCS-F wall in this study were chosen as shown in Table 6. For the purpose of validation process, a full-scale FCS-FA wall with SSC was modeled and simulated under axial load. The validation was achieved by comparing the ultimate load and failure mode obtained from the ABAQUS simulation on FCS-F wall with the results obtained from the previous experiment on PLFP-PA8 wall.

The ultimate load recorded from the simulation on FCS-FA was compared with the ultimate load recorded from the previous experiment on PLFP-PA8 wall as shown in Table 7. From the table, it can be seen that both values recorded from FEA and previous experiment are within acceptable range, which is 3.78%.

Failure mode of PLFP-PA8 and FCS-FA walls from experiment and FEA was compared in Fig. 7. From the figure, it is shown that failure of concrete in the wall as obtained from FEA was similar to that obtained from the experiment, where crushing had occurred at the mid height of panel as shown in Fig. 7(a). DAMAGE-C in ABAQUS represents the material damage and its failure mechanism. Fig. 7(b) presents the stiffness degradation of foamed concrete in FCS-FA when it responded to compression. Contour of DAMAGE-C presents the damage and crack pattern of foamed concrete wythes right after the panel reached its ultimate load.

3.1.2. Validation of full-scale FCS-F model with experimental results of half-scale FCS-H

According to scaling laws by Knappet et al. [31], ultimate load value of half scale structural element is able to represent its full scale’s ultimate load after scaling adjustment, where the ratio of ultimate load of full-scale to half-scale structure is equal to 4. Thus, result of half-scale FCS-H1 with dimension of 1000 mm × 375 mm × 70 mm was used to predict the ultimate load of full scale FCS-FB wall with dimension of 2000 mm × 750 mm × 140 mm. Both walls have slenderness ratios equal to 14. The validation will again compare the ultimate load and mode of failure obtained in each wall.

In this section, the ultimate load obtained from the experiment on half-scale FCS-H1 wall is multiplied by a factor 4 to predict the ultimate load of full-scale FCS-F according to Knappet et al. The predicted ultimate value of FCS-F wall was then compared with the ultimate load of a full-scale model of FCS-FB from FEA. The ratio of the ultimate load from FEA to experiment is found to be equal to 1.01 as shown in Table 8. The prediction on ultimate load of full scale FCS-FB wall can further be confirmed from the calculation using the classical equation in ACI 318-11 [32], and from equations derived by Mohamad [30] and Benayoune et al. [29] for both half-scale and full-scale walls. Results form this calculation give ratio of ultimate load of full-scale to half-scale wall equal to 7, 5.8 and 5.8, for ACI 318-11, Mohamad, and Benayoune et al., respectively. The equations from ACI 318-11, Mohamad, and Benayoune et al., are as given in Eqs. (1)–(3).

\[
P_u = 0.550 f'_{c} A'_{k} \left[ 1 - \left( \frac{KH}{327} \right)^2 \right] \quad (1)
\]

\[
P_u = 0.4 f_{cu} A_{k} \left[ 1 - \left( \frac{KH}{40(\text{t} - 4f)} \right)^2 + 0.67f_{y} A_{c} \right] \quad (2)
\]

\[
P_u = 0.4 f_{cu} A_{k} \left[ 1 - \left( \frac{KH}{40t} \right)^2 + 0.67f_{y} A_{c} \right] \quad (3)
\]

Table 5
Dimension, properties and ultimate load of PLFP-PAS.[30]

<table>
<thead>
<tr>
<th>Panel</th>
<th>H × W × t (mm)</th>
<th>Density ρ (kg/m³)</th>
<th>Compressive strength Pc (N/mm²)</th>
<th>Reinforcement (vertical and horizontal)</th>
<th>Diameter of shear connectors</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLFP-PAS</td>
<td>2800 × 750 × 100</td>
<td>1780</td>
<td>17</td>
<td>9 mm @ ø150 mm c/c</td>
<td>R6</td>
<td>583</td>
</tr>
</tbody>
</table>
1.16, 1.05 and 1.02, respectively, as shown in Table 8. All the ratios are closed to 1. Therefore, the value of ultimate load obtained from FEA is acceptable.

Fig. 8(a) and (b) depict the mode of failure experienced by half-scale FCS-H and full-scale FCS-F wall, respectively. As can be seen from the figure, both walls crushed within the similar location at the top region near the capping area. This proves that half-scale FCS-H experienced similar failure mode with full-scale FCS-F. This also proves that the FEA model for FCS-F developed can be used to conduct the parametric study on the structural performance of FCS wall.

3.2. Parametric study on FCS-F wall with DSC under axial load

The parametric FEA was conducted once the FCS-F wall model was validated. Simulations of FCS-F walls with various H/t, ranged from 18 to 40, were carried out. The list of full-scale FCS-F walls modeled in this study is as tabulated in Table 9. All these walls were simulated under axial load.

4. Results and discussion

The results discussed in this section are mainly on the influence of various H/t on the structural behavior, which include ultimate load, failure mode, load-deflection profile, and strain distribution of FCS-F wall subjected to axial load. The results are further analyzed to study the effects of DSC in FCS-F wall on its structural behavior compared to similar walls with SSC. The effects of DSC are also discussed from the results obtained from push off test simulated using ABAQUS.

4.1. Ultimate load

Table 6 and 11 show the ultimate load of FCS-F and FCS-H walls as obtained from FEA and experiment, respectively. It is noticed from both FEA and experiment that the ultimate strength of the panels decreased nonlinearly with the increased in H/t. From FEA results in Table 9, it is noticed that the decrease in strength became more obvious for walls with H/t above 30, which was from 6.2% to 19.4% for an increase of H/t from 30 to 40.

Slenderness ratio (H/t) has proven to have measurable influence on the ultimate load obtained in a wall [29,30,41]. When a wall is slender, its length is greater than the critical buckling length and it fails by buckling. Generally the higher the H/t, the lower is the allowable stress. This will therefore reduce the ultimate load achieved. This is as expected for walls subjected to compressive axial load where they behave just like column. As such, walls subjected to axial compressive load can fail like column due to buckling with a sudden and large lateral deflection before the compressive stress in the wall reach its allowable yield value.

Fig. 9 shows the relationship of the ultimate load with the slenderness ratio for FCS-H1 to FCS-H5 walls as obtained from both experiment and FEM. Both the graphs of Pu vs H/t from experiment and FEM show a declining pattern of Pu as H/t increased. In general, the values of Pu for walls obtained from FEM are higher compared to values obtained from experiment. The Pu obtained from FEM are influenced by material model selected and the constraints. It is known that FEM model is always more ideal compared to the actual specimen from the experiment which normally experience imperfections and discrepancies during casting and set up of testing apparatus. This could be the factor to the lower ultimate value obtained from FEA simulations. The difference between the Pu from FEM and experiment is about 20% to 30%. This is as expected because the previous work by Mohamad [30] and Benayoune et al. [29] on full-scale panel specimens recorded maximum of about 20% with FEM results. The two previous researches used Lucas for FEM simulations.

The work done by Mohamad [30] and Benayoune et al. [29] had also reported the similar effects of H/t on the ultimate load. Both studied the structural behavior of PLFP and PCSP walls, respectively, with various H/t experimentally and by using FEA under axial load. It was found that H/t has a significant effect on the strength capacity of both PLFP and PCSP walls. Results from experiment and FEA showed the ultimate strength capacity decreased as the H/t increased in all walls tested. Study on sandwich or composite column strengthened by a different system of reinforcement also recorded similar finding. Karimi et al. [44] studied experimentally the influence of slenderness on the behavior of FRP-encased steel-concrete composite column under compressive load. It was found that the ultimate load achieved decreased with the increase of slenderness ratio. Saima et al. [45] studied the behavior of slender partially encased composite column, PEC, using Newmark’s iterative procedure to identify the potential variables that can significantly affect the behavior of slender column. It was also found that the eccentric axial capacity of the column has been reduced prominently as the overall slenderness ratio increased.

The effect of using DSC to strengthen the wall was examined by comparing the ultimate load values of FCS-F4 and PLFP-PA10 from previous study by Mohamad [30]. Both walls have similar dimension of 2500 mm x 750 mm x 100 mm and slenderness ratio, H/t, equal to 25. It is found that FCS-F4 was able to sustain higher load (587 kN) compared to PLFP-PA10 (441 kN) even though with much lesser compressive strength (45% less), and smaller size of steel bar used as DSC. This is due to higher stiffness of FCS with DSC from the larger cross section area of steel shear connectors used compared to FCS with SSC. This finding agrees to finding by Tomlinson [46] who studied the influence of GFRP connectors on behavior of partially composite precast concrete wall panels under flexure test.
The study concluded that as the total cross-sectional area of connectors to surface area of panel in shear span for GFRP shear connectors increased, the load in the composite panel increased.

### 4.2. Failure mode

Generally, FCS-F walls were bent and crushed at its mid-height and either at the top or bottom of the wall near the capping area. It is noticed that FCS-F walls have similar failure pattern even though their heights varied. Walls with \( \frac{H}{t} \leq 25 \) were crushed at the top half or at the bottom half with cracks near to the capping; however, walls with \( \frac{H}{t} \geq 25 \) failed from crack and crush at its mid-height. Cracks were also observed near the capping area. This indicates that walls with lower slenderness ratio failed from material failure while walls with higher slenderness ratio failed from buckling.

Failure modes of FCS-F1, FCS-F2 and FCS-F4 walls with DCS and PLFP-PA2, PLFP-PA3 and PLFP-PA10 with single shear connectors subjected to axial load are shown in Table 11. It is noticed that FCS-F and PLFP walls with similar \( \frac{H}{t} \) experienced different trend of failure mode. PLFP-PA2 and PLFP-PA3 with \( \frac{H}{t} = 18 \) and 20 experienced horizontal cracks along the mid height and crushing at the bottom while FCS-F1 and FCS-F2 with similar \( \frac{H}{t} \) experienced crush and crack at the bottom half of the panel and near capping area. There was no crack detected along the mid height of panel. This indicates that walls with similar \( \frac{H}{t} \) but strengthened with DSC seem to be able to manage the different layers in the sandwich walls together by efficiently transferring the shear force from one wythe to the other. However, for walls with higher \( \frac{H}{t} \), they tend to fail at mid-height. This is proven by comparing FCS-F4 and PLFP-PA10 both with \( \frac{H}{t} = 25 \) as shown in Table 12. Both walls experienced crack at the mid-height of panel (Fig. 10).

### 4.3. Load-deflection profile

The maximum horizontal displacement recorded from the FEM simulation on all FCS-F panels are found to increase with the increase of slenderness ratio. This is as expected for walls subjected to compressive axial load where they behave just like column as described previously in Section 4.1. Horizontal displacements of FCS-F1 to FCS-F11 walls at mid height are as shown in Fig. 11(a). As evident in the figure, trends for horizontal displacement increments at the mid-height of wall were similar for all FCS-F walls, where the horizontal displacement increased gradually with the increase of the load during the elastic stage. When the wall entered the plastic stage, the trend of curves becomes non-linear from first cracking until it reached the ultimate load. For FCS-F wall with lower slenderness ratio, cracking, yielding and crushing occurred when the panel reached the ultimate loading. This phenomenon was reduced when the \( \frac{H}{t} \) of walls increased. Buckling and out of plane bending occurred but walls tend to sustain the load longer before it failed. It was observed from the figure that the maximum horizontal deflection of each wall increased with the increased \( \frac{H}{t} \). The maximum horizontal deflection of 18.49 mm was recorded in FCS-F11, which is the most slender wall. General trend of horizontal displacement for FCS-F wall is presented in Fig. 11(b). It shows that the mid height section of wall experienced highest horizontal displacement due to bending. Previous study on effect of \( \frac{H}{t} \) towards deflection in a column with different system of reinforcement has also recorded similar finding. Saravanan et al. [47] studied the performance of glass fiber reinforced polymer (GFRP) wrapped high strength concrete (HSC) columns with various slenderness ratios under uni-axial compression. It was found that the axial deflection was recorded higher for more slender columns compared to less slender ones. The relationship of \( \frac{H}{t} \) with horizontal deflection of FCS-F walls was further illustrated in Fig. 12.

<table>
<thead>
<tr>
<th>Slenderness ratio ( \frac{H}{t} )</th>
<th>Ultimate Load, ( P_u ) (kN)</th>
<th>( \frac{P_{\text{EXP}}}{P_{\text{FEA}}} \times 100% )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experiment (PLFP-PA8) FEA (FCS-FA)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22.4</td>
<td>660</td>
<td>685</td>
</tr>
</tbody>
</table>

Table 7 Ultimate load of PLFP-PA8 and FCS-FA walls with SSC.
Table 8
Ratio of ultimate load of full-scaled to half-scale FCS walls.

<table>
<thead>
<tr>
<th></th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experiment</td>
</tr>
<tr>
<td>FCS-H1 (Half scale)</td>
<td>139</td>
</tr>
<tr>
<td>FCS-F (Full scale)</td>
<td>–</td>
</tr>
<tr>
<td>Ratio (F/F)</td>
<td>–</td>
</tr>
</tbody>
</table>

Fig. 12 shows the relationship between ultimate load with horizontal displacement for FCS-F with H/t 18, 20 and 24, as obtained from experiment and FEM. The load-deflection curves in the figure show that for both experiment and FEM, the wall with higher H/t
obtained higher maximum deflection. The deflection increased gradually as the load increased during the elastic stage.

Table 13 shows the ultimate load and maximum horizontal deflection achieved by PLFP-PA10 and FCS-F4, each with SSC and DSC, respectively. FCS-F4 with greater ultimate load deflected less compared with PLFP-PA10. The results indicate that double shear connectors with obviously more steel instilled in FSC-F wall had caused it to be more stiffer compared to PLFP-PA10 with less steel in it. The ratio of the area of steel to that of the concrete is the percentage of reinforcement, which for concrete sections such as slabs, beams, columns or walls could typically be 3–5%. High percentage of steel could encounter significant cracking with accompanied spalling and excess steel area could cause higher stiffness in the wall. For this study, the DSC in all FCS-F walls is less than 5% of the concrete cross section area.

4.4. Strain distribution across PLFP panel’s thickness

Strain distribution across the thickness was used to predict the extent of compositeness achieved by the walls. The estimation of composite behavior of the panel is characterized by strain across the cross section in the axial test as referred to Benayoune et al. [29]. Vertical strains across the thickness of FCS-F4 were plotted in Fig. 13. It is shown that at the early stage of loading, strains were distributed uniformly through the thickness of the wall. However, as the load increased, the foam concrete wythes and polystyrene layer began to experience irregular strain distribution. It is also seen that the strain variations in FCS-F4 recorded small discontinuity across the depth of the polystyrene layer. The rear wythe seem to experience larger strain compared to the front wythe. This is as expected because the cracks did not appear simultaneously on both wythes of concrete.

This trend of strain distribution across the panel’s thickness is similar to the finding by Mohamad [30] on PLFP and Benayoune et al. [29] on PCSP walls with SSC. PCSP recorded smaller discontinuity of the strain across the polystyrene layer under increasing load when compared to PLFP and FCS-F walls as illustrated in Figs. 14, 15 and 13, respectively. This could be due to stronger property of normal concrete as the PCSP’s wythes and larger overall thickness of the panel. Overall thickness of PCSP is 130 mm with 50 mm thick polystyrene layer compared to PLFP and FCS-F, both with 100 mm overall and 20 mm polystyrene thickness. The larger overall thickness will make larger depth of reinforcement. This will increase the compositeness because the ultimate moment capacity of a sandwich panel increase with the increase of depth of reinforcement as concluded in the report by Mohamad et al. [48]. In this study, four (4) PLFP slabs with double shear connectors were tested under flexure load. These PLFP slabs were with various compressive strengths and overall thicknesses. It was found that slab panels with higher compressive strength and overall thickness managed to obtain higher degree of compositeness, which is from 30% to 60% higher. Larger overall thickness of wall will also enlarge its cross sectional area. The cross section area has an important influence on the load carrying capacity of a column. A rectangular section will buckle about the axis with the least dimension. In these walls, the width is kept constant at 750 mm; therefore the

![Fig. 9. Ultimate load versus slenderness ratio for FCS-F walls.](image)

![Table 12](table)

Table 12

<table>
<thead>
<tr>
<th>Specimen</th>
<th>H/t</th>
<th>Crack pattern and failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCS-F1</td>
<td>18</td>
<td>Cracked and crushed near the bottom half of the panel and cracked near to capping</td>
</tr>
<tr>
<td>FCS-F2</td>
<td>20</td>
<td>Cracked and crushed near the bottom half of the panel and cracked near to capping</td>
</tr>
<tr>
<td>FCS-F4</td>
<td>25</td>
<td>Cracked and crushed near the middle of the panel and cracked near to capping</td>
</tr>
<tr>
<td>PLFP-PA2</td>
<td>18</td>
<td>Horizontal crack along the mid-height and crushing at the bottom part and along the thickness</td>
</tr>
<tr>
<td>PLFP-PA3</td>
<td>20</td>
<td>Horizontal crack along the mid-height and crushing at the bottom part</td>
</tr>
<tr>
<td>PLFP-PA10</td>
<td>25</td>
<td>Crack along mid-height of panel</td>
</tr>
</tbody>
</table>

![Fig. 10. Mode of failure of FCS-walls from FEA.](image)
wall will buckle about the minor axis (y-y) with dimension equal to its overall layers’ thickness as shown in Fig. 16. The critical load is determined by taking the least moment of inertia, I, for the rectangular section; in this case, $I_{y-y}$.

As such, as the overall thickness of the wall increased, the $I_{y-y}$ also increased, and so did the critical load due to buckling which is directly proportional to moment of inertia.

The strain distribution also predicts the effectiveness of shear truss connectors to sustain the applied axial load and transfer it in between the wythes. The effectiveness of DSC from the strain distribution across the thickness is obvious from the illustration in Figs. 13 and 14. Even though all the strain values recorded are small, the strains occurred in PLFP wall with SSC are larger compared to the strain values recorded in FCS-F wall with DSC. The difference are quite significant with maximum strain values recorded were 3000 micro strain and 1040 micro strain for PLFP and FCS-F, respectively. This smaller strains recorded indicates that FCS-F walls with DSC experienced more uniform strain distribution across its thickness compared to PLFP. The more uniform strain distribution indicates that FCS-F walls behaved in a more composite behavior compared to PLFP. However, the current
finding on strain distribution under axial load test did not provide enough out of plane bending to study the horizontal shear interaction. Therefore, to confirm the composite action in the sandwich panel, further investigation was conducted by simulating the FCS-F walls under push off load using FEA.

4.5. Behaviour of FCS-F walls under push off loading

Push off loading test using FEA was conducted to determine the effects of DSC on the shear strength of FCS-F wall. For this purpose, a wall with dimension 2800 mm x 750 mm x 100 mm (FCS-F5) was simulated to study the effects of both single and double shear truss connectors on the sandwich structural system in FCS-F wall. Fig. 17 illustrates the support at the top part of wall was applied at the front wythe with constraints in x and z directions, while the support at the bottom of wall was applied on the rear wythe with constraints in x, y and z directions. Load was applied on the top of front wythe until the wall failed.

Ultimate shear forces obtained from the analyses were recorded in Table 14. FCS-F wall with SSC sustained ultimate shear force up to 197 kN. Meanwhile, FCS-F wall with DSC sustained higher ultimate shear force up to 405 kN, which is about 106% higher. Comparison of shear capacity for FCS-F with single and double shear truss connectors shows that FCS-FD with DSC achieved much higher ultimate shear load compared to FCS-FS with SSC. Both walls behaved in a ductile manner with FCS-FD tend to sustain higher load. This ductile behavior is likely caused by slippage of the bent bar connector from the concrete that led to gradual loss of composite action and hence experienced larger deflection.

Mechanical anchorage which exist from the tie between the connector bar and longitudinal wythe’s reinforcement could also have contributed to the observed ductility. FCS-FD also recorded higher maximum slip at its ends, which is about 132% compared to FCS-FS. This indicates that FCS-FD has more strength, behaved in a more composite manner, and failed at much higher slip.

From the results of push off loading test, it is evidenced that the use of DSC in FCS-F walls has enabled it to become more ductile and sustain higher out of plane load. With DCS, the load was transferred between the wythes more efficiently with higher degree of compositeness achieved.

5. Conclusion

The results recorded from this study have been discussed with focusing on the influences of H/t and DCS on the overall behavior of the FCS walls. The increase in H/t has resulted with decrease in ultimate load but increase in deflection. The decrease in ultimate load achieved is more obvious for walls with H/t greater than 30, which was from 6.2% to 19.4% for an increase of H/t from 30 to 40. These results are as expected for slender walls subjected to axial load where the failure mode is often due to buckling and controlled by mainly the geometrical properties of the wall.

Failure load from buckling is generally smaller for wall with higher H/t, but it tend to sustain the load longer before it failed completely. It was found that the maximum horizontal deflection of each wall increased with the increased H/t with mid height section of the wall experienced highest horizontal displacement due to bending. FCS-F1 walls with lowest H/t recorded lowest
horizontal displacement about 6 mm whereas FCS-F11 with highest H/t recorded highest horizontal displacement about 20 mm. It is also observed that under axial load, the strain distribution across the thickness of wall is not directly influenced by H/t, but rather by its overall thickness. The larger overall thickness has resulted with higher degree of compositeness, measured from the more uniform strain distribution across its thickness.

The effectiveness of shear truss connectors plays very important role in transferring the load in-between the wythes. Load transferred efficiently between the wythe and throughout the whole wall system resulted with higher load sustained and the wall behave in a more composite manner. The effects of DSC on the structural behavior of FCS walls were analyzed by comparing the walls strengthened with DSC with walls strengthened with SSC, but with similar H/t. It is found that walls with DSC achieved higher ultimate load which is about 32% higher compared to walls with SSC. However, the larger cross section area of steel in the DSC has caused the wall to become stiffer and therefore deflected less, about 15% less, compared to the walls with SSC. It is also noticed that walls with DSC and SSC with similar H/t experienced different trend of failure mode. Walls with SSC tend to fail from buckling at lower H/t compared to wall with DSC.

The composite behavior of the walls was analyzed from the strain distribution recorded across the mid-height’s thickness under axial load and from the ultimate shear load and maximum slip value recorded under push off load. The strain distribution across the thickness of FCS walls with DSC in Fig. 13 was more uniform compared to walls with SSC in Figs. 14 and 15. This indicates that these walls behaved in a more composite behavior compared to walls with SSC. Meanwhile, its effect on ultimate shear strength and ductility as obtained from push off load were more prominent. Walls with DSC managed to sustain significantly higher load with high ductility and higher maximum slip recorded at its ends compared to walls with SSC.

The findings from this study have shown that double steel shear truss connectors, DSC, do have significant effects on the load-bearing capacity and composite behavior of the wall. The design of steel truss as the double shear connectors has proven to enhance the FCS wall’s strength, stability, and compositeness under axial load. Considering its lightweight and precast construction method, the FCS wall with DSC has a great potential as structural wall in a low to medium rise building.

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References