Retraction

Retraction: Cyclic behaviour of expanded polystyrene (Eps) sandwich concrete walls (IOP Conf. Ser.: Mater. Sci. Eng. 620 012060)

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Cyclic behaviour of expanded polystyrene (Eps) sandwich concrete walls

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Abstract. Precast concrete walls become increasingly utilized due to the rapid needs of inexpensive fabricated house especially as traditional construction cost continue to climb. Moreover, particularly at damaged area due to natural disaster when the requirement of a lot of fast-constructed and cost-efficient houses are paramount. However, the performance of precast walls under lateral load such as earthquake or strong wind is still not comprehensively understood due to various type of reinforcements and connections. Additionally, the massive and solid wall elements also increase the total weight of building and hence increase the earthquake base shear force significantly. Therefore, the precast polystyrene reinforced concrete walls which offers light weight and easy installment became the focus of this investigation. The experimental test on two RC wall specimens using EPS (Expanded Polystyrene) panel and wire mesh reinforcement have been conducted. Quasi-static load in the form of displacement controlled cyclic tests were undertaken until reaching failure. At each discrete loading step, lateral load-deflection behavior, crack propagation and collapse mechanism were measured which then were compared with theoretical analysis. The findings showed that precast polystyrene reinforced concrete walls gave sufficient seismic performance reaching up to 1% drift at failure point against lateral.

1. Introduction

Tall buildings particularly with irregularities are prone to behave poorly and collapse when subjected to lateral loads such as earthquake excitation or strong wind. To overcome this problem, shear walls are commonly preferable to increase the lateral strength of structures significantly. However, the added massive and solid shear walls results in increasing the total weight of building which in turn increase the base shear force due to earthquake load which might reduce the effectiveness of the shear wall use in the structures. The effort to reduce the weight of shear walls without losing lateral strength capacity is necessary.

There were many studies investigated lightweight concrete shear walls with various techniques to reduce the element weight such as the use of lightweight aggregates, applying porous concrete system, or inserting lightweight panel into the wall.

There were various light-weight aggregates used in shear wall elements investigated by researches. Mousavi et.al. [1] studied the effectiveness of JK system wall, composed of EPS concrete (mortar with EPS beads as fine aggregates) and galvanized steel reinforcement, in sustaining lateral load. It was observed that JK walls had high ductility capacity, but still need
further observation for the application in tall and medium buildings. Zuang [2] investigated that the use of gangue as an aggregate in concrete shear wall provided larger energy dissipation compared with normal concrete shear wall. Furthermore, Tao et.al. [3] focused on ash ceramsite as alternative for lightweight aggregate concrete shear wall which gave similar load-deflection behaviour and collapse mechanism to those on normal concrete ones. Whereas, Chai and Anderson [4] found that the performance of perforated lightweight aggregate concrete wall panels in low rise buildings subjected to lateral forces were generally satisfactory. Cavaleri et.al [5] investigated pumice stone in comparison with expanded clay and normal stone as aggregates in concrete shear wall which showed the benefit of the use pumice stones.

On the other side, reducing the weight of structural elements can be achieved using sandwich system by inserting lightweight panel inside the concrete element. This panel system is usually applied for insulation purpose as well. The lightweight wall system investigated in this paper focused on the use of EPS panel as a filler and galvanized wire mesh for reinforcing bar as shown in Figure 1.

Figure 1. Typical EPS sandwich concrete wall panel [6]

2. Methodology

The specimens were designed as structural walls composing low-rise building which were commonly found in house or school precast buildings. The squat walls are generally dominated with shear behaviour which comparably differ to tall walls commonly found in high-rise building. Concrete tall walls have been considerably well researched and understood [7-10]. Whereas, concrete squat walls have been increasingly investigated recently [11-14]. However, innovation studies on sandwich squat walls with EPS panel were rarely found [15], and hence became the main focus of this study.

The experimental tests on two sandwich RC wall specimens RCW4 and RCW8 have been conducted. All specimens had wall height and width of 90 cm and 60 cm respectively, which corresponds to aspect ratio of 1.5. RCW4 wall used 4cm thick EPS panel compared to 8cm EPS panel installed in the RCW8 wall. The RC wall specimens were reinforced with φ2.5 mm galvanized steel wires at 75 mm spacing vertically and horizontally which connected to each other by φ3.0mm wire connectors. The yield and ultimate tensile strengths were 600 MPa and 680 MPa respectively. The wall specimens used 35mm thick shotcrete on each outer side of walls with the concrete strength of 15 MPa. Fig 1 shows the typical RC walls specimens.

Quasi-static cyclic load procedure was applied at the tip of the wall specimens to obtain representative hysteretic curves of lateral load versus displacement (refer Figures 2-3). The drift controlled loading sequence consisted of drift increments of 0.42% until reaching 0.167%, then drift increments of 0.167% until reaching 0.66%, and followed by drift increment of 0.33% until failure. Three cycles of loading were used in each drift ratio to ensure that hysteretic behaviour could be maintained.

During the testing process, discrete displacement stages were defined where lateral loading was held constant whilst LVDT and dial gauge measurements were taken, crack patterns
recorded, and visual inspections made. The test stopped when the peak lateral strength of the specimen reduced by 20% (lateral load failure).

![Figure 2: Geometry and reinforcement details of RC wall specimens](image1)

**Figure 2** Geometry and reinforcement details of RC wall specimens

![Figure 3: Schematic of Wall Loading Test Setup](image2)

**Figure 3.** Schematic of Wall Loading Test Setup

![Figure 4: Quasi-static lateral loading history](image3)

**Figure 4.** Quasi-static lateral loading history

3. Result and discussion

Hysteretic curves of lateral load-drift relationship and the crack propagation pattern of all specimens are presented in Figure 5. Both specimens RCW4 and RCW8 had similar peak lateral load of about 25 KN with different behaviour characteristic. RCW4 (EPS panel thickness of
40mm) developed more classical flexural mechanism, whilst RCW8 (EPS panel thickness of 80mm) was more dominated with yield penetration behaviour due to the thinner concrete cover of beam foundation. As shown that specimen RCW4 managed to complete all three cycles of quasi-static cyclic load at 1.0% drift, then failed at the first cycle of load at drift of 1.33%. Whereas specimen RCW8 produced shorter maximum drift capacity with failure at first cycle of lateral load at 1.0% drift. A comparison of lateral strengths and drifts between the experimental results and the theoretical predictions is presented in Table 1.

![Image](image1.png)

**Figure 5.** Crack Patterns and hysteric curves for of all specimens

The total lateral deformation consists of flexural, shear and yield penetration components which were measured using LVDTs and dial gauges as shown in Figure 6.

The flexural displacement component at the column top for each LVDT segment was obtained using:

\[
\Delta f_j = \int \phi(x) dx = \frac{L}{L_f} (\delta_2 - \delta_1)
\]

(1)

### Table 1 Strength and deformation properties of specimen RCW4

|    | Strength, kN | Drift (%) |
|----|--------------|-----------|
|    | $F_{cr}$ | $F_y$ | $F_u$ | $\delta_{cr}$ | $\delta_y$ | $\delta_u$ | $\delta_{lf}$ |
| RCW4 Exp. | 2.8 | 18 | 23.5 | 0.17 | 0.47 | 1.00 | 1.33 |
| Theoretical | 4.0 | 16 | 23 | 0.1 | 0.42 | 0.75 | n.a |
| RCW8 Exp. | 2.3 | 20 | 24.5 | 0.17 | 0.55 | 0.67 | 1.00 |
| Theoretical | 4.3 | 18 | 23.5 | 0.1 | 0.43 | 0.8 | n.a |

The total lateral deformation consists of flexural, shear and yield penetration components which were measured using LVDTs and dial gauges as shown in Figure 6.
Note: the theoretical values were taken from moment-curvature analysis (flexural component only)

Whilst, the displacement of the upper segment without a LVDT transducer was calculated assuming uncracked section properties:

\[
\Delta_{hi,A} = \frac{\varphi L_{i}^{2}}{3} = \frac{\nu t_{i}^{3}}{3E_{c}I}
\]  

(2)

Figure 6. Typical measurement obtained from transducers and dial gauges: (a) flexure, (b) shear

The measured shear deformation \( \Delta_{sh} \) was estimated from the diagonal LVDT (refer Figure 5.14b), such that:

\[
\Delta_{sh} = \frac{(\delta_{23} - \delta_{22})}{2} \theta_{EC} \xi = \frac{(\delta_{23} - \delta_{22})}{2} \sqrt{\frac{L_{v}^{2} + D^{2}}{L_{v}}}
\]  

(3)

where \( \delta_{i} \) = diagonal LVDT measurement and \( D \) = cross section depth (parallel to lateral loading direction).

The displacement for flexural, shear and yield penetration component are shown in Table 2.

| Specimen | \( \Delta_{fl} \) (mm) | \( \Delta_{sh} \) (mm) | \( \Delta_{yb} \) (mm) |
|----------|----------------|----------------|----------------|
| RCW4     | 2.17           | 0.59           |                |
| RCW8     | 1.1            | 0.21           |                |

4. Backbone curve models

4.1. Model 1 – detailed

The backbone model for sandwich concrete walls is developed based on the model developed for lightly reinforced concrete walls [16].
A detailed curve model is developed based on displacement-based design methodology for predicting the lateral load-drift behaviour (comprising four stages: cracking, yield, peak, and lateral load failure) is shown conceptually in Figure 7.

![Figure 7. The backbone curve model of lateral load-drift capacity](image)

4.1.1. Point A (Cracking)
The cracked lateral strength and drift are calculated as follows:

\[ F_{cr} = \frac{M_{cr}}{L} \; ; \; \gamma_{cr} = \frac{M_{cr}L}{3EI/e} \]  

(4)

where the flexural tensile strength \( f_t \) is taken as \( 0.6f_{ck} \).

4.1.2. Point B (Yield)
The yield drift is calculated using the effective second moment of area as follows:

\[ F_y = \frac{M_{y}}{L} \]  

\[ \gamma_y = \frac{M_{y}L}{3EI/e} \]  

(5)

(6)

The Paulay and Priestley model for effective moment of inertia [8] is used as follow:

**Flexure-dominated Walls:**

\[ I_e = \left( \frac{100}{f_y} + \frac{P_y}{f_{ck}A_g} \right)A_g \]  

(7)

**Shear-dominated Walls:**

\[ I_w = \frac{I_e}{1.2 + C} \; ; \; C = \frac{30I_e}{L^2fD} \]  

(8)

where:
- \( P_y \) = nominal axial load
- \( A_g \) = gross cross section area of walls
- \( t \) = wall thickness
4.1.3. Point C (peak strength)
The model was developed by investigating the curvature within the plastic hinge region using the force equilibrium equation \( N = C_c + C_s - T \) with the spalling strain \( \varepsilon_{cu} = 0.003 \) is used as a limit state for concrete strain. For low rise building, the presence of gravity axial load is reasonably small, and hence for simplicity, the compression steel area is eliminated from equilibrium equation. The peak flexural lateral load \( F_u \) and the drift at concrete fracture \( \gamma_u \) can then be obtained as follows:

\[
F_u = \frac{M_u}{L} \quad (9)
\]

\[
\gamma_{peak} = \gamma_y + \gamma_{pl,p} \quad (10)
\]

where:

\[
\gamma_{pl,p} = \left( \phi_{peak} - \phi_y \right) L \quad (11)
\]

\[
\phi_{peak} = \frac{\varepsilon_{cu}}{k_d} \quad (12)
\]

\[
k_u = \frac{N + A_{st}f_y}{0.85\gamma_{db}} \quad (13)
\]

\[
\gamma = 0.85 - 0.007(f_{y,c}^{-1/28}) \quad (14)
\]

\[A_{st} = \text{tensile steel area}\]

\[\varepsilon_{sm} = \text{steel strain-hardening strain}\]

The plastic hinge length \( L_p \) can be estimated as follows:

\[
L_p = 0.054L + 0.022d_f \quad (15)
\]

4.1.4. Point D (ultimate displacement)
Lateral load-displacement relationship of squat walls is dominated by shear behaviour; however, for lightly reinforced squat walls, the flexure behaviour still provides large influence on lateral load-drift behaviour. The failure mechanism which is influenced by shear strength degradation is needed; and hence, the lateral load failure model developed for lightly reinforced concrete columns [17] is modified for this model due to the similarity of lateral load-displacement behaviour between lightly reinforced concrete walls and columns.

Shear strength \( (V_u) \) of RC walls consists of concrete strength \( (V_c) \) and steel strength \( (V_s) \) components, as follows:

\[
V_u = V_c + V_s \quad (16)
\]

In this model, the concrete shear strength uses the formula developed based on principal tensile strength [18], whilst the steel strength proposed by Wesley and Hashimoto [19] is used as follows:

\[
V_c = \frac{2}{3}A_{cr} \sqrt{(\gamma_f)^2 + \frac{f_y P}{A_{cr}}} \quad (17)
\]

\[
V_s = (c_f \rho_n + c_v \rho_v) \gamma_{db} \quad (18)
\]

where:

\[
A_{cr} = 0.85(n_{c} \rho_{sf})^{0.36} \quad (19)
\]
\[ d = \text{effective depth of RC walls, can be assumed as } 0.8D \]

\[ c_1 = 1 - c_v \]
\[ c_v = 1 \quad \text{for } a < 0.5 \]
\[ = 2(1 - a) \quad \text{for } 0.5 < a < 1 \]
\[ = 0 \quad \text{for } a > 1 \]

\[ n_c = E_s/E_c \]

\[ \rho_v = \text{the transverse reinforcement ratio} \]

\[ \rho_c = \text{the total longitudinal reinforcement ratio} \]

\[ \rho_s = \text{the tension reinforcement ratio} \]

As a note, for moderate and slender walls \((a > 1, \text{ and hence } c_v = 0)\) the steel strength component \((\text{Eq. 18})\) can be re-written as a common shear strength formula:

\[ V_s = \rho_h f_y d t = (A_0 f_y c) \\frac{1}{\gamma} \]  \(\text{(20)}\)

The ultimate drift can be obtained as follows:

\[ \gamma_u = \gamma_y \left[ \frac{1 + k \alpha}{(1 + k \alpha) - 0.8} \frac{f_u}{V_u} \right] \]  \(\text{(21)}\)

where:

\[ k = \frac{0.3 \lambda}{9 - \chi} \]  \(\text{(22)}\)

\[ \alpha = \text{the drift ductility when the shear strength commences to decline} \]

4.2. Model 2 - Simplified

The simplified model comprises three stages: cracking, yield and ultimate as shown in Figure 8. This model is aimed to provide simple procedures for checking lateral load-drift behaviour of non-ductile concrete walls.

**Figure 8.** The Simplified model of lateral load-drift capacity

4.2.1. Point A (Cracking)

The cracked lateral strength and drift are calculated by assuming cracking drift \(\gamma_{cr} = 0.05\%).
4.2.2. Point B (Yield)
The ultimate yield strength are calculated using factored ultimate strength,

\[ F_y = \phi F_u \]  

(23)

whilst the yield drift (\( \gamma_y \)) is determined using the smallest values of the following alternatives:
- Approximate value of \( \gamma_y = 0.2\% - 0.3\% \).
- Apply \( I_{eff} = 0.5I_e \) (refer [20])

4.2.3. Point C (peak strength)
The ultimate drift (\( \gamma_m \)) can be calculated as a sum of the yield drift (\( \gamma_y \)) and the plastic drift (\( \gamma_{pl} \)) as follows (refer Figure 9)

\[ \gamma_m = \gamma_y + \gamma_{pl} \]  

(24)

The plastic drift can be estimated by assuming a maximum acceptable strain in the steel bar at single crack at the wall base in the order of \( \varepsilon_s = 5.0\% \), and taking a more conservative approach to Priestley and Paulay [8] strain penetration length of \( l_{yp} = 4400 \varepsilon_y d_b \approx 15d_b \). Hence, the following models can be obtained (refer Figure 10):

Crack width,

\[ W_c = \varepsilon_s l_{yp} = 0.05 \times 15d_b = 0.75d_b \]  

(25)

Plastic drift,

\[ \gamma_{pl} = \theta_{pl} = \frac{W_c}{D} = 0.75 \frac{d_b}{D} \]  

(26)

\[ \gamma_{pl} = (\varphi_u - \varphi_y)l_p \]

Figure 9 Plastic Drift of Simplified Model

\[ \theta_{pl} \]

Figure 10. Plastic rotation at wall base
The result of applying the detailed and simplified models on the wall specimens is shown in Figure 11. The lateral load-drift relationship between experimental data and proposed models are in considerably good agreement, with the simplified model showed better prediction due to the dominant combination of flexural and yield penetration behaviour as shown in Figures 12-13.

**Figure 11.** Lateral load-drift curves of detailed and simplified models

**Figure 12.** Comparison between experimental data and theoretical models for specimen RCW4

**Figure 13.** Comparison between experimental data and theoretical models for specimen RCW8
Hysteretic curves of lateral load-drift relationship and the crack propagation pattern of all specimens are presented in Figure 5. Both specimens RCW4 and RCW8 had similar peak lateral load of about 25 KN with different behaviour characteristic.

5. Conclusions
Two specimens of sandwich concrete walls have been tested in order to investigate the lateral load-drift behaviour and collapse mechanism. Specimen RCW4 with thinner EPS panel developed more classic flexural behaviour with drift capacity maxed at about 1.3%, whilst specimen RCW8 only managed to reached 1.0% with dominant yield penetration behaviour due to thinner concrete cover at beam foundation. However, the results can still be considered satisfactory since it can fulfil the required maximum drift suggested by the code for low-to-moderate seismic region. The higher seismic region will require double-panel wall type for this purpose.

Two models comprising detailed and simplified approach for predicting the load-displacement behaviour of sandwich concrete wall subjected to lateral load have been developed. The experimental data and the proposed models are in good agreement, particularly the simplified model due to the dominant behaviour of flexural and yield penetration

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