Experimental Study on Seismic Behavior of RC Frames with Different Infilled Masonry

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Abstract

Six 1/2 scaled, single-storey, one-bay frame specimens were tested in this study to investigate the seismic behavior of masonry infilled reinforced concrete (RC) frames subjected to lateral loading. The parameters investigated include types of masonry and types of openings. The crack patterns, failure modes, load-displacement hysteretic loops, stiffness degradation, and energy dissipation capacity are presented and discussed. It is found that the infilled wall (with or without openings) could improve the behavior of RC frames significantly. Moreover, as expected, the infilled frame with higher strength masonry performed better than those with relatively low strength masonry. Furthermore, the openings may detriment the stability of the infilled walls. The concentric widow opening has worse effects than the eccentric door opening. The proposed analytical model could determine the load resisting capacity of bare frame and infilled frame with reasonable accuracy.

Keywords: reinforced concrete, frames, masonry, testing, structural analysis

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Introduction

The collapse of masonry infilled frames from previous earthquakes (Decanini et al. 2004, Zhao et al. 2009) indicated that it is necessary to carried out studies to understand the behavior of masonry infilled RC frames subjected to seismic loads. Actually, dozens studies including experimental and analytical investigations had been conducted since 1950s. It was first proposed the idea of using equivalent single strut to represent the in-plane stiffness of the infilled walls. Holmes (1961) provides suggestion to model the infill panels by an equivalent compression strut with width of $w = 1/3 r_{\inf}$; in which $r_{\inf}$ is the diagonal length of the infill panel. Smith (1966) recommended the width of the equivalent strut ranged from $0.1 r_{\inf}$ to $0.25 r_{\inf}$ base on the experimental data. In 1969, Smith and Carter (1969) adopted the idea of single-strut and proposed an analytical model to quantify the effective width of the strut. Based on the analytical model proposed by Simith and Carter (1969), Fiorato et al. (1970) indicated that infilled walls could enhance the lateral load resisting, strength, stiffness and energy dissipation capacity of multi-storey frames. Single-strut model could predict the stiffness of the infilled frame, but not the peak strength. Based on experimental and analytical results, Mainstone and Weeks (1970) gives an empirical equation to determine the equivalent width of the strut, which is adopted by FEMA-306 (1998). Mehrabi et al. (1996) tested twelve 1/2 scaled, single-storey, single-bay, frame specimens. It is indicated that infill panel could improve the performance of RC frames significantly. However, specimens with strong frames and strong panels perform superior than those with weak frames and weak panels. A method is proposed by Gulan and Sozon (1999) to estimate the vulnerability of RC infilled structures. It is indicated that the compressive and tensile strength of the mortar is important for estimation of the contribution of filled panels properly. Al-Chaar et al. (2002) tested five 1/2 scaled frame specimens to estimate the effects of the number of bays on seismic performance of infilled RC frames with non-ductile details. It is indicated that the number of bays appears to affect the peak and residual capacity, shear stress distribution, and failure mode of the frames significantly. Eight 1/3 scaled, single storey, single bay, frame specimens were tested by Kakaletsis and Karayannis (2007) to study the effects of eccentric openings on the seismic performance of infilled RC frames. Comparing with bare frames, the infilled frames even with eccentric opening could enhance the stiffness, strength, and general behavior. To achieve better performance, it is preferred to locate the eccentric opening as close to the edge of the infill as possible. Kakaletsis and Karayannis (2008) tested another series of seven 1/3 scaled, single-stroey, single-bay, frame specimens. The effects of opening shape and infill compressive strength are investigated. Based on collected test data, Mohammadi and Nikfar (2013) proposed a formula for predicting the strength and stiffness of the infilled frames with central openings. It is indicated that the reduction factor of the peak load resisting capacity (PLRC) due to openings depends highly on the material of the confining frame, but the reduction factor of stiffness is not. Eight 1/3 scaled RC infilled frame specimens were tested by Moretti et al. (2014). The design variables are aspect ratio and types of connections between the infill walls and the frame. It is found that the dowels should be installed along the horizontal interfaces of the frame to avoid early failure in the columns. Seven full-scale, single story, single bay, RC frame specimens are tested subjected to reversed cyclic loading. It is indicated that including the contribution of infill walls, the lateral strength, stiffness and energy-dissipation capacity of the frame will enhance significantly. However, the displacement-based ductility will decrease considerably. Niyompanitpattana and Warnitchai (2017) tested five one-half scaled RC frame specimens to study the effects of different openings on seismic behavior of gravity-load-designed long span frames. In the past two decades, researchers found that equivalent single-strut model may not be able to model the complex behavior of the infilled frames: such as bending moment or shear force in the frame components, although it simulates the general response (lateral strength or stiffness) not bad (Saneinejad and Hobbs 1995; Buonopane and White 1999). Therefore, multiple-strut models were proposed by researchers

(Thiruvengadam 1985, Syrmakezis and Vratsanou 1986, Chrysostomou 1991, and Chrysostomou et al. 2002, and EI-Dakhakhni 2000, EI-Dakhakhni et al. 2001, and Crisafulli and Carr 2007). Although extensive experimental and analytical studies had been conducted to estimate the impacts of infill walls on seismic behavior of RC structures, little studies had been carried out on interaction between the infills and the frames with various types of masonry. The relative strength and stiffness between the infills and frames may change the failure mode of the infilled frames significantly, (Kakaletsis and Karayannis ,2008). Therefore, to further quantify the effects of different types of masonry on failure modes and load resisting mechanism of infilled frames subjected to reverse cyclic loading, a series of six frame specimens with different types of masonries were tested in the present study.

Research Significance

Although extensive studies had been carried out on seismic behavior of infilled frame subjected to cyclic loading, the tests on quantification of infilled frame with different masonry are relatively few, especially considering the effects of different types of opening. Therefore, a series of six infilled frames with two types of masonry with various openings were tested in this study. For quantification of the effects of opening and masonries, analytical models were proposed based on the principle of superimpose.

Experimental program

Test specimens

Six single-storey, single-bay, 1/2 scaled frame specimens (BF, IF-S, IF-P, IFD-P, IFW-S, and IFW-P) were tested in this experimental program. The designation and properties of test specimens were tabulated in Table 1. As shown in Figure 1, the prototype frame was a six-storey, four-bay by four-bay, RC moment resisting frame, which was designed for seismic resistance in accordance with ACI 318-14 (2014) and it was located on a class D site with the parameters of response spectrum , S_{DS} and S_{D1}, taken as 0.43 and 0.28, respectively. The specimen for the testing was extracted from the bottom storey of the frame and was 1/2 scaled down. As shown in Figure 2, for bare frame BF, the height of the frame was 1400 mm while the span of the frame was 2250 mm. Thus, the aspect ratio is about 1/1.6. The cross section of the beam and column was 130 mm × 230 mm and 250 mm × 250 mm, respectively. More transverse reinforcements were placed at the beam and column ends (potential plastic hinge zones). Moreover, two transverse reinforcements were also placed at the joint zone. The infilled frames have identical dimensions and reinforcement details as the bare frame, except different configurations or types of masonry. For Specimens IF-S, and IFW-S, sintered shale hollow blocks (relatively higher strength) were utilized in construction. However, porous sintered bricks (lower strength) were used for Specimens IF-P, IFD-P and IFW-P. As shown in Figure 2, solid walls were built for Specimens IF-S and IF-P while door opening with size of 500 mm × 900 mm was constructed in IFD-P. The window opening with size of 300 mm × 500 mm was designed for Specimens IFW-S and IFW-P. Thus, the opening ratio in IFD-P and IFW-P were 17.5 % and 8.5 %, respectively. The clear cover of the RC beam and column was 15 mm.

Material properties

Ready-mix concrete, which had designed strength of 25 MPa, was used for casting. However, the measured average compressive strength from six cylinder tests was 26.8 MPa. The properties of reinforcements are tabulated in Table 2. It is worth emphasizing that R6 represents plain rebar with diameter of 6 mm while T12 and T16 mean deformed rebar with diameter of 12 and 16 mm, respectively. The compressive and shear strength of masonry type 1 (based on porous sintered brick) were 5.0 MPa and 0.55 MPa, respectively, while the compressive and shear strength of the masonry type 2 (based on sintered shale hollow blocks) were 5.5 MPa and 0.67 MPa. Moreover, based on a series of six 70.7 mm cubic tests, the measured average compressive strength of the mortar for type 1 and type 2 walls were 5.0 MPa and 5.6 MPa, respectively.

Test setup and instrumentation

The typical setup of test specimen is shown in Figure 3. As shown in the figure, a hydraulic actuator (Item 1 in Figure 3) was utilized to apply lateral displacement at the center of the top beam. Displacement-controlled loading procedure was used, as shown in Figure 4. In the initial four increments (0.1 % to 0.33 % drift ratio), the specimens were only subjected to one fully reversed loading cycle. After that, three fully reversed loading cycles were applied at each increment. To simulate the axial force applied on the column from the upper stories, a hydraulic jack (Item 2 in Figure 3) was installed above side columns to apply axial force with magnitude worked out as $0.2 f_c^{'} A_g$. A special designed assembly (Item 3 in Figure 3) was installed to prevent out-of-plane failure. The specimen was fixed to the strong floor by two compression beams (Item 4 in Figure 3). The compression beams were fixed to the floor by prestressed bolts with diameter of 50 mm. The applied load and corresponding displacement at the center of the top beam was measured by built-in load cell and displacement transducer. To measure the deformation shape of the panel and to monitor the translation of the foundation beam, a series of displacement transducers were also installed as illustrated in Figure 2b. Electric wire strain gauges (TML FLA-5-11-5LT) were installed in longitudinal reinforcements before casting, as shown in Figure 2a.

Results and discussion

Crack patterns and failure modes

Figure 5 presents the crack patterns of test specimen v.s. critical drift ratio (DR), which is defined as the ratio of lateral displacement at the loading point to the wall height. When the DR reached 0.14 %, crack with length of 40 mm was first formed at the bottom of the left column. However, the crack could close back once the lateral displacement was back to zero. As shown in Figure 5a, When DR reached 0.33 %, cracks in the columns kept developing and cracks were also observed at the beam ends. When the DR reached 0.4 %, the initial flexural cracks at the column bottom become inclined.

Moreover, flexural cracks also formed at the top column-beam interfaces. When the DR reached 1.0 %, the concrete at the beam ends and bottom of the column began to crush. At a DR of 1.3 %, the concrete crushing became more severe at the bottom of the column and concrete spalling occurred at the beam ends. At a DR of 2.0 %, the concrete spalling was observed in both beam ends as well as the horizontal cracks at the bottom of the column connected. Further increased the DR to 2.8 %, concrete spalling was also observed at the bottom of the columns. At the DR of 4.0 %, the reinforcement at the right beam end suddenly buckled due to severe concrete spalling. The failure mode of Specimen BF is shown in Figure 6. It can be seen that plastic hinges formed at the column bottom and beam ends. Concrete spalling and crushing was also observed at there. However, limited damage was observed at the beam-column joints.

For solid infilled frame IF-S, when DR reached 0.14 %, flexural crack was first observed in the column bottom. At a DR of 0.33 %, flexural cracks occurred in the beam ends. Slight sliding was observed between the top inclined course and the top beam. Cracks also formed in the corner of the infill walls. Further increase of the DR to 0.5 %, mortar spalling was observed at the interface between the infilled wall and the beam. Diagonal crack occurred at the compression corner. When DR reached 0.67 %, penetrated crack formed at the column base. Sliding was also formed at the mid-height of the wall. At a DR of 1.0 %, X-shaped crack was formed in the wall. Horizontal crack was observed at 1/3 height of the wall from the bottom. At DR of 1.3 %, brick crushing was observed at the right up corner. When DR reached 2.0 %, concrete spalling began to occur at the left beam end. The X-shaped crack became wider and brick crushing occurred not only at the corner, but also at the middle of the wall. Further increase of the DR to 3.3 %, concrete spalling became more severe in the plastic hinge zones of the beam. Brick crushing became more and more severe and some bricks fell off. The test was terminated as the wall may collapse if further applying displacements. The failure mode of the specimen is shown in Figure 7. As shown in the figure, severe concrete crushing occurred at the beam end. Some of the bricks had totally lost contact due to spalling. However, comparing with Specimen RC, the damage in the column base was milder. Similar to Specimen RC, no obvious damage occurred at the beam-column joints.

For solid infilled frame Specimen IF-P, which has relatively lower strength masonry, flexural cracks occurred at the column base at a DR of 0.2 %. Increasing the DR to 0.33 %, flexural cracks formed at the mid-height of the columns. At this DR stage, flexural cracks were also observed at the beam ends and diagonal stepped cracks were formed at the infilled walls. In general, the specimen only experienced elastic response with little residual deformation after force releasing. Further increasing the DR, more flexural cracks formed at the beam ends and mid-height of the columns. Two cracks were also observed at the beam-column joints. However, no new cracks occurred at the infills. When DR reached 1.0 %, the infills at the right upper corner began to crush and obvious gap was observed between the infills and surrounding frame. Diagonal cracks were suddenly formed at the right column tip at a DR of 1.3 %. Further increasing the DR, more bricks began to crush and the gap between the infills and frame became wider. At a DR of 2.8 %, shear failure occurred at the top of the right column. Similar failure modes were observed by Kakaletsis and Karayannis (2008) and Kim et al. (2010). The failure mode of this specimen is illustrated in Figure 8. Comparing with that of Specimen IF-S, the diagonal cracks in infills of IF-P was stepped while they were brick failure in IF-S. Moreover, the crushing of infills at the corner was much milder in IF-P. The failure in the frame of IF-P was shear failure of the column end while it was forming plastic hinges and concrete crushing at beam ends in IF-S.

For door punched infilled Specimen IFD-P, mortar crushing is observed at the interface between the beam and infills at a DR of 0.14 %. As shown in Figure 5d, X-shaped stepped cracks are appeared at the right panel of the infills at the DR of 0.33 %. Further increasing the DR to 0.5 %, more diagonal stepped cracks are formed at the right panel. Flexural cracks not only occurred at the beam ends, but also at the column base. At a DR of 0.67 %, the diagonal cracks in the infills become wider and crushing is occurred at the infills. When the DR reaches 1.0 %, more cracks were appeared at the mid-height of the columns. Concrete crushing occurred at the beam ends. At a DR of 1.3 %, partial of the bricks at the door edge began to crush. Concrete crushing also occurred at the column edge. Further increase of the DR to 2.8 %, the bricks at the right edge of the door began to collapse along the main diagonally stepped crack. When DR reaches 4.0 %, more and more bricks fell off. Due to the embedded tie bars along the column height, the infills did not collapse completely. The failure mode of this specimen is illustrated in Figure 9.

For window punched infilled Specimen IFW-S, when DR reached 0.14 %, flexural cracks occurred at the mid-height of the column. At a DR of 0.33 %, vertical crack was observed above the opening. At this stage, diagonally stepped cracks were observed at the bottom panels, as shown in Figure 5e. However, limited cracks formed at the frame, which indicated the load resisting capacity was mainly attributed to the infills. At a DR of 0.67 %, the diagonally stepped cracks became wider and flexural cracks also formed at the columns and beams. Slight sliding was observed at the right panel along the stepped crack. When the DR reached 1.0 %, more diagonal cracks occurred in the infills. Moreover, diagonal cracks were also observed at the beam-column joints. Concrete crushing was occurred at the beam ends. Some of the bricks were crushed at this stage. At a DR of 2.0 %, more cracks and severe brick crushing were occurred at the side panels of the opening. As shown in Figure 5e, the brick crushing became more severe and partial of the bricks were entirely collapsed. When the DR reached 4.0 %, the bricks above the opening were totally collapsed. The failure mode of IFW-S is shown in Figure 10.

For window punched infilled Specimen IFW-P, at a DR of 0.2 %, stepped diagonal crack was formed at the left upper corner of the opening. When the DR reaches 0.33 %, stepped diagonal crack formed at the left lower corner and right upper corner of the opening. However, the flexural cracks were be confined in the frame. As shown in Figure 5f, at a DR of 0.67 %, several flexural cracks were observed at the column and beam. More diagonally stepped cracks formed at the infills. Some of the diagonal cracks were connected and developed a sliding crack at the bottom of the opening. Further increasing the DR to 1.3 %, concrete crushing was occurred at the beam ends. The column flexural crack was extended into the joint zone. More flexural damage was observed at the columns. Brick crushing was also observed at this stage. At a DR of 2.8 %, the concrete crushing became more severe at beam ends. Moreover, concrete crushing was also occurred at the column base. The corner of the infill was observed crushed and some of the bricks at the opening edge were collapsed completely. When the DR reached 4.0 %, more bricks were collapsed completely and severe crushing was occurred at the beam and column ends.

Hysteretic behavior

The hysteretic behavior of the wall was summarized in a plot of lateral load vs. DR. Figure 12a shows the lateral load-displacement response of Specimen BF. It was found that the positive and negative PLRC were 175 kN and -166 kN, respectively. No obvious pinching was observed during the test. The resistance deterioration was quite slow, which agrees with the flexural critical failure mode well. The ultimate deformation capacity was 70 mm and corresponds to 5.0 % DR. The yield strength of the specimen was calculated to be 131.9 kN based on Eq. 1

$$F_y = \frac{4M_y}{h_c} \tag{1}$$

where M_y is the yield strength of the column section with including the effects of column axial force, h_c is the height of the column.

However, the measured average yield strength was 139.5 kN based on the energy equilibrium method, as shown in Figure 13. The yield displacement was 9.5 mm and thus, the displacement based ductility of the specimen is over 5.8. Figure 12b shows the load-displacement response of Specimen

IF-P. It can be seen that the positive and negative PLRC were 417 kN and -396 kN, respectively. The resistance deterioration was much faster than that of BF. The deformation capacity of the specimen was 28.0 mm and DR of 2.0 %, which is corresponding 15 % strength drop from the PLRC. It was much lower than that of BF. Similarly, based on energy equilibrium method, the average yield strength of IF-P was determined to be 341.0 kN in positive load, which was about 244.4 % of that of BF. The yield displacement was about 7.2 mm and thus, the displacement-based ductility was 3.9. The load-displacement hysteretic loop of Specimen IFD-P is shown in Figure 12c. The measured positive and negative PLRC was 251.0 kN and -275.0 kN, respectively. The slight difference between positive and negative PLRC was mainly due to the door opening was eccentric. The measured yield strength was 203.8 kN, which is only about 59.8 % of that of IF-P with solid walls. The average yield displacement and displacement-based ductility was 9.4 mm and 6.0, respectively. For Specimen IFW-P, which has window opening, it was measured positive and negative PLRC of 335.0 kN and -313.0 kN, respectively. The average yield displacement and yield strength of this specimen was 10.4 mm and 280.0 kN, respectively. Thus, the window opening decreased the yield strength by 15.0 %.

For Specimen IF-S with relatively higher strength masonry, the measured positive and negative PLRC was 452 kN and -447 kN, as shown in Figure 12e. The average yield strength was determined to be 374.8 kN, which is about 109.9 % of that of IF-P. Similar to Specimen IF-P, the slope of strength reduction is steeper. The measured yield displacement was 4.5 mm, which is only about 62.5 % of that of IF-P with porous sintered bricks. Thus, the ductility of the specimen was about 4.1. As shown in Figure 12f, the positive and negative PLRC of Specimen IFW-S was 362.0 kN and -352.0 kN, respectively. The average yield strength was about 315.3 kN in accordance with a displacement of 3.4 mm. Therefore, the ductility of the specimen is 6.8. In general, comparing to IF-P and IFW-P, pinching was more obvious in IF-S and IFW-P.

Stiffness degradation

Figure 14 illustrates the stiffness degradation of tested specimens. It can be seen that the initial stiffness of BF, IF-P, IFD-P, IFW-P, IF-S, and IFW-S were 31.4 kN/mm, 98.5 kN/mm, 65.7 kN/mm, , 81.2 kN/mm, 123.1 kN/mm, and 100.0 kN/mm, respectively. Thus, the infill walls even with drop openings could increase the initial stiffness of the frame significantly. Moreover, as expected, the initial stiffness of IF-S and IFW-S was much higher than that of IF-P and IFW-P due to relatively higher strength of the masonry. However, the slope of stiffness degradation of IF-S and IFW-S was much larger than that of IF-P and IFW-P. Thus, when the DR exceeded 1.0 %, IF-P achieved similar secant stiffness as that of IF-S. For IFW-P, similar secant stiffness as IFW-S was obtained after the DR beyond 1.3 %. Furthermore, for all specimens, the stiffness degradation becomes slower when the DR beyond 1.3 %.

Energy dissipation capacity

The energy dissipation capacity is a critical characteristic for evaluation the ability of a structure to survivean earthquake. The energy dissipation capacity was determined by the area enclosed by the lateral load-displacement loops. Figure 15 illustrates the comparison of the curves of cumulative energy dissipation capacity, which is calculated by the summation of energy dissipated in consecutive loops. It is found that the energy dissipation capacity of Specimen BF, IF-P, IFD-P, IFW-P, IF-S, and IFW-S were 3.3, 2.8, 3.0, 3.0, 2.6, and 2.9 kN·m, respectively. However, it should be noted that the lower energy dissipation capacity measured in the infilled frames was mainly because the tests were terminated when the load resisting capacity dropped over 15 % from the PLRC. If we only concern the energy dissipation capacity at DR of 2.8 %, the energy dissipation capacity of infilled frames was much larger than the bare frame, similar to Kakaletsis and Karayannis (2007). Similarly, the infilled frames with solid walls was achieved the larger value than that of the frame with punched walls. Moreover, as shown in the figure, at the beginning of the test, IF-S and

IFW-S achieved slightly larger energy than that of IF-P and IFW-P, respectively. However, when the DR reached 2.4 %, the dissipated energy capacity in IF-P will exceed that of IF-S. Similarly, the dissipated energy capacity in IFW-P became larger when the DR was beyond 3.3 %.

Discussion of the design variables

As aforementioned, a series of six specimens were tested in this study. The effects of the design variables on the load resisting capacity of frames are discussed.

Effects of infilled walls

Figure 16 shows the comparison of the envelope of hysteretic loops of the specimens with or without infill walls and Table 3 tabulated the key results. As shown in figure and table, the average peak resistance of BF, IF-P, IFD-P, and IFW-P are 170.5 kN, 406.5 kN, 263.0 kN, and 324.0 kN, respectively. Thus, the solid infill wall increased the PLRC by 138.4 %. The walls with door opening and window opening increase the PLRC of the bare frame by 54.3 % and 90.0 %, respectively. Similar conclusions were obtained from previous studies (Fiorato et al. 1970, Mehrabi et al. 1996). Moreover, the displacement-based ductility of BF, IF-P, IFD-P, and IFW-P is 5.8, 3.9, 6.0, and 5.4, respectively. As shown in Figure 16b, for infilled frame with higher strength of masonry, similarly, the solid infill walls increased the PLRC by 163.6 % while the infilled walls with opening could upgrade the PLRC by 109.4 %. The displacement-based ductility of IF-S and IFW-S was 4.1 and 6.8, respectively. Thus, the solid infilled walls may decrease the ductility, similar as Al-Chaar G and Sweeney (2002). However, the openings will increase the ductility of the infilled frame. Comparison of their failure modes, the infilled walls may result in shear failure of the column due to interaction between the walls and frames. Moreover, the openings may detriment the stability of the walls. The punched walls prone to out-of-plane collapse when they subjected to in-plane lateral loading. Although the infilled walls may increase the initial stiffness of the bare frame significantly, they may decrease its deformation capacity.

Effects of masonry types

Figure 17 compares the envelopes of hysteretic loops of specimens with different types of masonry. As shown in the figure, the average peak strength of IFW-P, IFW-S, IF-P, and IF-S were 324.0 kN, 357.0 kN, 406.5 kN and 449.5 kN, respectively. Thus, the specimen with higher strength masonry achieved higher peak strength comparing with their counterparts with relatively lower strength masonry. Meanwhile, the yield displacement of IF-S and IFW-S was 4.5 mm and 3.4 mm, respectively. Thus, IFW-S and IF-S achieved much larger initial stiffness than that of IFW-P and IF-P, respectively. However, the resistance deterioration in IFW-S and IF-S was faster than the corresponding specimens IFW-P and IF-P. The displacement-based ductility of IF-S, IF-P, IFW-S, and IFW-P was 4.1, 3.9, 6.8, and 5.4, respectively. Thus, the higher strength of masonry will not degrade the ductility of the frame, similar as the conclusions from Kakaletsis and Karayannis (2008). Comparing their failure modes, similar failure modes were observed in the specimens with higher or lower strength. This is mainly because the strength of the masonries was not so distinct. Thus, it is worth to carry out more tests on specimens with more distinct masonry strength in the future.

Analytical analysis

To deep understand the effects of infilled walls on behavior of RC frames subjected to lateral cyclic loads, a series of analytical analysis was carried out using the diagonal compressive struts model.

Specimen BF - As shown in Figure 18a, for bare frame, it is assumed plastic hinges were formed at the bottom of the column, which is actually observed in Specimen BF. Thus, the PLRC of BF could be determined by Eqs. 2 and 3:

$$F_c \cdot h_c + \Delta \cdot N_c = 2 \cdot M_{pc} \tag{2}$$

$$V_u = 2F_c \tag{3}$$

where F_c is the shear force in each column; M_{pc} is ultimate moment strength of the column considering axial force effects; N_c is the initial axial force of the column and Δ is the lateral displacement in accordance with PLRC.

The calculated PLRC is 164.5 kN, which is about 96.5 % of the measured average PLRC of Specimen BF.

Specimens IF-S and IF-P - As shown in Figure 18b, for infilled frame with solid walls, the infilled wall worked like a single diagonal compression strut could help to resist the lateral load, as recommended by FEMA 306 (1998). Thus, the PLRC of IF-S and IF-P could be determined as below:

$$F_c \cdot h_c + \Delta \cdot N_c = 2 \cdot M_{pc} \tag{4}$$

$$V_u = 2F_c + V_W \tag{5}$$

$$V_W = a t_{\inf} f'_{m90} \cos\theta \tag{6}$$

where V_W is the lateral resistance from the infill wall; $a = 0.175(\lambda_1 h_c)^{-0.4} r_{\inf}$ is the width of the strut; $\lambda_1 = \left[\dfrac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{\frac{1}{4}}$ is a factor; $t_{\inf}$ is the thickness of the infill panel and equivalent strut; $r_{\inf}$ is the diagonal length of the infill panel; θ is the angle whose tangent is the infill height-to-length aspect ratio; f'_{m90} is the compressive strength of the infill panel; E_{fe} is modulus of elasticity of frame material; E_{me} is modulus of elasticity of infill material; I_{col} is the moment inertial of column; $h_{\inf}$ is the height of infill panel.

The calculated PLRC of IF-S and IF-P are 376.5 kN and 344.3 kN, respectively. As the measured average PLRC of IF-S and IF-P are 449.5 kN and 406.5 kN, respectively. The calculated values are 83.8 % and 84.7 % of the measured one for IF-S and IF-P, respectively.

Specimen IFD-P - For punched infilled frame with door opening, the layout of the struts is shown in Figures 18c and d. It should be noted that the layout of the struts in positive and negative direction is different as the door opening is eccentric. Thus, similar to IF-P and IF-S, by using superposition principle, the negative and positive PLRC could be determined by Eqs. 8 and 9, respectively:

$$F_c \cdot h_c + \Delta \cdot N_c = 2 \cdot M_{pc} \tag{7}$$

$$V_u = 2F_c + V_{W1} + V_{W2} + V_{W3} \tag{8}$$

$$V_u = 2F_c + V_{W2} + V_{W3} \tag{9}$$

For V_{W1}, V_{W2},and V_{W3}, they could be determined similar as V_w and as suggested by FEMA 306 (1998). The calculated positive and negative PLRC of IFD-P is 302.0 kN and -318.9 kN, respectively. As the measured positive and negative PLRC of IFD-P is 251.0 kN and -275.0 kN, respectively. The analytical values are 120.3 % and 116.0 % of the measured ones, respectively.

Specimens IFW-S and IFW-S - For punched infilled frame with window opening, the layout of the struts is shown in Figures 18e. The PLRC of IFW-S and IFW-P could be determined by Eqs. 10 and 11.

$$F_c \cdot h_c + h_z \cdot V_{W4} + \Delta \cdot N_c = 2 \cdot M_{pc} \tag{10}$$

$$V_u = 2F_c + V_{W1} + V_{W2} + V_{W3} + V_{W4} \tag{11}$$

The calculated PLRC of IFW-S and IFW-P is 375.0 kN and 347.0 kN, respectively. As the measured average PLRC of IFW-S and IFW-P is 357.0 kN and 324.0 kN, respectively. The analytical values are 105.0 % and 107.1 % of the measured ones, respectively.

Conclusions

The experimental study in this research derived the following conclusions:

1. The infilled walls could enhance the load resisting capacity and initial stiffness of the frame significantly. However, the infilled walls may detriment the deformation capacity of the frame if assuming the specimen is failed when the load resistance dropped over 15 %. Thus, it was arguable to conclude that infilled walls could improve the seismic behavior of RC frames,

as the higher initial stiffness leads to larger seismic force. Moreover, although the solid walls may also decrease the ductility of the frame slightly, the openings do increase the deformation capacity and ductility.

2. Comparison of the failure mode of the specimens indicated that solid infilled wall may result in shear failure at the top of column. When opening presence in the infilled wall, more damage may concentrate at the mid-height of the column. Moreover, the presence of opening may detriment the stability of the infilled wall significantly. The concentric widow opening has great effects on the stability of the infills, comparing to the eccentric door opening, even the door opening has higher opening ratio. Furthermore, infilled walls may restraint the bending of the beam and prevent it to develop plastic hinges at the beam ends. However, the door or window openings may weak the restraints.
3. Relatively higher strength masonry will improve the behavior of the filled frame in terms of load resisting capacity, stiffness degradation, and energy dissipation capacity. However, higher strength masonry does not change the failure mode of the frames significantly as similar mortar is utilized for both types of masonry walls. Moreover, the specimens with higher strength masonry undergo faster load decreasing after they reached the peak load resisting capacity.
4. The analytical analysis indicated that considering the load resistance of the infilled walls by diagonal compressive struts could evaluate the lateral strength of infilled frames effectively. However, as simple superposition principle was utilized in this study, the accuracy still has potential to be improved. For more accurate evaluation, finite element model is a good alternative.

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NOTATION

a	width of the strut
E_{fe}	modulus of elasticity of frame material
E_{me}	modulus of elasticity of infill material
F_c	shear force in each column
F_y	yield strength of the specimen
f'_{m90}	compressive strength of the infill panel
h_c	height of the column
h_{inf}	height of infill panel
M_y	yield strength of the column section with including the effects of column axial force
M_{pc}	ultimate moment strength of the column considering axial force effects
N_c	initial axial force of the column
I_{col}	moment inertial of column
r_{inf}	diagonal length of the infill panel
t_{inf}	thickness of the infill panel and equivalent strut
V_W	lateral resistance from the infill wall

Δ lateral displacement in accordance with PLRC

λ_1 a factor

θ angle whose tangent is the infill height-to-length aspect ratio

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Figure caption list

Figure 1: Elevation view of the prototype frame

Figure 2: Dimensions and reinforcement details of tested specimens: (a) BF, (b) IF-S&IF-P, (c) IFD-P, and (d) IFW-S&IFW-P

Figure 3: Specimen IFW-S ready for test

Figure 4: Applied lateral displacement history

Figure 5: Crack pattern development of the specimens: (a) RC, (b) IF-S, (c) IF-P, (d) IFD-P, (e) IFW-S, and (f) IFW-P

Figure 6: Failure mode of Specimen BF

Figure 7: Failure mode of Specimen IF-S

Figure 8: Failure mode of Specimen IF-P

Figure 9: Failure mode of Specimen IFD-P

Figure 10: Failure mode of Specimen IFW-S

Figure 11: Failure mode of Specimen IFW-P

Figure 12: Lateral load versus displacement hysteresis loops: (a) BF, (b) IF-P, (c) IFD-P, (d) IFW-P, (e) IF-S, and (f) IFW-S

Figure 13: Schematic view for determining the yield strength of the specimens

Figure 14: Comparison of the stiffness degradation

Figure 15: Comparison of the energy dissipation capacity

Figure 16: Effects of infilled walls: (a) porous sintered bricks, (b) sintered shale hollow blocks

Figure 17: Effects of masonry types

Figure 18: Analytical models for tested specimens

Table 1. Property of test specimens

Test ID	Dimensions Beam (mm²)	Dimensions Column (mm²)	Joint Trans. Rebar	Infilled Walls	Wall Type	Types of Masonry
BF	130×230	250×250	0.2%	No	N/A	N/A
IF-P	130×230	250×250	0.2%	Yes	Solid	Porous Sintered
IFD-P	130×230	250×250	0.2%	Yes	Door Opening	Porous Sintered
IFW-P	130×230	250×250	0.2%	Yes	Window Opening	Porous Sintered
IF-S	130×230	250×250	0.2%	Yes	Solid	Sintered Shale Hollow
IFW-S	130×230	250×250	0.2%	Yes	Window Opening	Sintered Shale Hollow

Table 2. Properties of reinforcements

Types	Diameter	Yield Strength MPa	Ultimate Strength MPa	Elastic Modulus GPa	Elongation
R6	6	318	529	198	15.1%
T12	12	348	488	203	16.3%
T16	16	486	599	206	16.6%

Note: R and T represents plain rebar and deformed rebar, respectively.

Table 3. Comparison of the critical results and failure modes

Test ID	Positive Peak load (kN)	Negative Peak load (kN)	Total energy Dissipation (kN·m)	Initial Stiffness (kN/mm)	Yield Displacement (mm)	Yield Strength (kN)	Ductility
BF	175	-166	3.3	31.4	9.5	139.5	5.8
IF-P	417	-396	2.8	98.5	7.2	341.0	3.9
IFD-P	251	-275	3.0	65.7	9.4	203.8	6.0
IFW-P	335	-313	3.0	81.2	10.4	280.0	5.4
IF-S	452	-447	2.6	123.1	4.5	374.8	4.1
IFW-S	362	-352	2.9	100.0	3.4	315.3	6.8