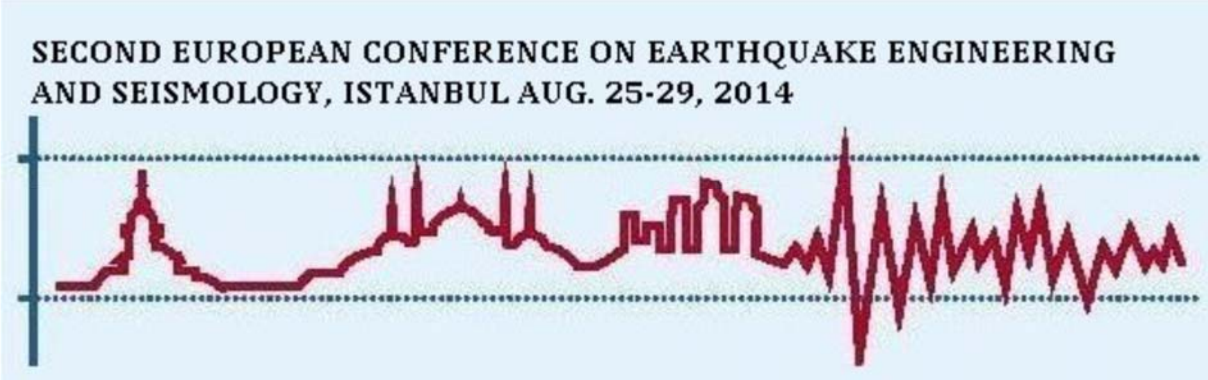

CYCLIC LOAD TESTS OF RC FRAME WITH COLUMN-ISOLATED MASONRY INFILLS

J S KUANG[1] and Zhe WANG[2]

ABSTRACT

Although a great improvement has been acheived for the seismic design of masonry-infilled reinforced concrete frame buildings, it is still far from satisfactory especially when consensus on some complicated issues are yet to be reached, in particular the frame-infill interaction problem. In this paper, two large-scale, single-storey and single-bay masonry-infilled reinforced concrete frame specimens were tested under in-plane reversed-cyclic loading. The two frame specimens have different infill configurations, one being infilled with full contact (standard) and the other column-isolated with infills. From the test observations, it is apparent that the standard test was sufferred much severe damage than the isolated one. In standard specimen, major shear and tensile cracks of infill wall occurred along the diagonal corners and local collapse appeared at lower mid-height regions, whereas the masonry panel was almost intact with only small minor cracks in the column-isolated one. In subesequnt analysis, hysteresis curves and evenlop curves show that the column-isolated infills condition can significantly preserve the integrity of infill panel and exhibit the improved seismic performance than the standard one in terms of ductility, degradation of stiffness and strength, and energy dissipation capability.

INTRODUCTION

Reinforced concrete framed buildings infilled with masonry walls are the most popular structure system in the world. However, masonry infills are generally treated as the non-structural elements in analysis and design and are only considered for the involvement in global behaviour of structures in the natural period or the reduction factor in most current seismic design codes of practice. The complex interaction between the surrounding frame and masonry walls have not been systematically studied by many.

Among various seismic design codes in the world, there is a lack of consensus on the approach for the seismic design of RC frame system with consideration of the infill effect. Conventionally, two distinct alternatives are being used, where the RC frame either integrates infills or directly isolates from them, respectively. Most seismic design codes of practice, such as Eurocode 8, have mandated that masonry infills shall be in full contact with the bounding frame. Consequently, the remarkable structural interaction between the frame and infill panels could be induced, thus dramatically changing the ductile behaviour of frame and resulting in the poor seismic performance of overall structure during earthquakes. Although the updated American masonry structures building code ACI 530.1-11 and Canadian masonry structure standard CSA S304.1-04 have introduced an equivalent diagonal strut model to analyse the masonry infill effect, there is still a long way to go before the complex failure modes of the structures are thoroughly considered.

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On the other hand, to weaken the frame-to-infill interaction, the initial gaps between infill walls and frame were introduced by the New Zealand masonry design standard, NZS 4230:2004, aiming at preserving the favourable ductile structural behaviour of frame through application of capacity design procedures. Currently, American masonry code ACI 530.1-11 has already specified the "non-participating infills" as the preferable alternative by applying the in-plane isolation joints with at least 9.5 mm wide in the plane of infills. In addition, in Chinese seismic code GB50011-2010, one level of inclined bricks has been required to lie beneath the beam bottom in order to achieve a weak interaction. The isolation alternative with consideration of frame-to-infill interaction is very likely to develop into a preferred design approach for infilled frame in seismic codes of practice. Nevertheless, it should be emphasised that to avoid the possible instability from out-of-plane failure by setting the initial gaps, connectors shall be provided as detailing to attach the infill panels to the frame, as required in both the US and Chinese codes.

Although masonry infills are commonly considered as the non-structural elements in most of the seismic design codes, the frame-infill interaction mechanism has been extensively studied during the past decades. The comparison between the gravity load-designed and seismic load-designed reinforced concrete frames, different material types of masonry wall, the significant influence of masonry infills on the contribution of global lateral stiffness and strength, and the overall modification of ductility and energy dissipation capacity, have all been discussed in published literatures (Crisafulli et al., 2000; Mehrabi et al., 1996; Mosalam et al., 1997; Murty and Jain, 2000). In addition, the stability analysis of masonry walls with the in-plane damage under out-of-plane loads, the enhancement of seismic response of masonry walls by slight reinforcements, and the analysis comparing ductile performance of infill panels with and without openings have been experimentally studied (Dawe and Seah, 1989; Kakaletsis and Karayannis, 2009; Pereira et al., 2011; Leite et al., 2011; Tasnimi and Mohebkhah, 2011).

At the same time, a few innovative techniques for improving the seismic performance of infilled RC frames are developed. The seismic infill wall isolator subframe (SIWIS) system, designed to act as a "structural fuse", reduces building drift under minor-to-moderate earthquakes while disengages columns and infill walls in order to protect the overall structure during severe damaging events (Aliaari and Memari, 2005). The concept of horizontal sliding joints is to preinstall several sliding frictional joints at different height levels within masonry infill panel to create sliding frictional actions to replace diagonal strut actions, so as to improve the deformation capacity of infills and eventually reduce the infill-frame interaction (Preti et al., 2012). The "hybrid masonry infills" enhances lateral stiffness and strength within the frame by utilising the isolated infills while resists the out-of-plane failure through steel connector plates (Biggs and Throop, 2010; Abrams et al., 2010).

To find an effective solution, the discrete method of analysis with a damage-based cohesive crack modelling technique has been proposed to investigate the seismic performance of infilled RC frame under the combined in-plane and out-of-plane seismic excitations (Kuang and Yuen, 2013; Yuen and Kuang, 2014). Simultaneously, several experienments on reinforced concrete frame specimens with different configurations of maosnry-infilled walls have been progressively carried out to thoroughly investigate the possible failure modes of the masonry walls as well as damages to the surrounding frame, particularly in relation to the weak frame-to-infill interaction (Wang and Kuang, 2013).

The primary objective of the study is to continue to mitigate the frame-to-infill interaction by introducing the "column-isolated gaps", whilst avoiding out-of-plane failure of masonry infill by keeping infill panel tightly fitted with bounding beam ("Arching Action") and utilising the steel connectors between columns and infill panel. Hysteretic characteristics and backbone curves of the two specimens are extracted and compared with each other in initial stiffness, strength, ductility, and energy dissipation capacity behaviours. It is shown that the introduction of the column-isolated gaps alternative can significantly sustain the ductile behaviour of reinforced concrete frame, which may be able to provide some meaningful design recommendations to future seismic codes.

EXPERIMENTAL PROGRAMME

Test specimens

Two large-scale masonry-infilled RC frame specimens with different frame-to-infill connection configurations were tested under the reversed cyclic loading. The bounding frame was designed to Eurocode 8 (EC8) with ductility class medium (DCM). Details of reinforcement layout are illustrated in Fig. 1. The cross-section of columns is 250 mm × 250 mm with the reinforcement of 8T16 in a steel ratio of 2.57 %, while the beam is 200 mm × 300 mm reinforced by 3T16 with a steel ratio of 1.16 % to guarantee the sufficient ductility demand. The critical region of beam and columns is both 300 mm in length with stirrups T10 spaced in 90 mm. The solid-clay brick units were chosen with the compatible size suitable to bouding frame as 205 mm × 98 mm × 50 mm to build the masonry infill walls. The concrete strength of specimen C1 and C2 are obtained from the mean value of 150mm cubes with 46.9 N/mm^2 and 41.4 N/mm^2, respectively. The strength of mortar is experimentally different by 25.2 N/mm^2 for Specimen C1 and only 4.7 N/mm^2 for Specimen C2, while the strength of bricks are range from 10 N/mm^2 to 25 N/mm^2. The yield strength of the steel is 500 N/mm^2 for T16/T10.

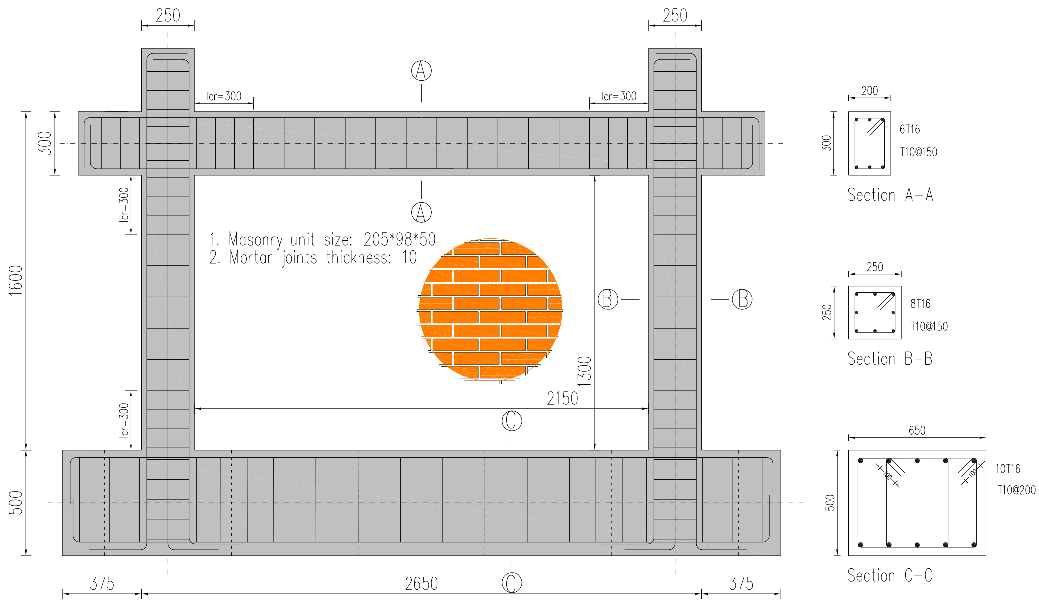


Figure 1. Geometry and reinforcement layout of test specimens (unit: mm)

Experimental set-up and procedure

The test set-up and loading arrangement are shown in Fig. 2. It is seen that the specimen is subjected to both the vertical and horizontal loads. The vertical load is first applied to the two column ends by the loading frames with the axial load ratio of about 10 % of the column squash capacity, to simulate as a practical axial load ratio in laboratory testing as well as in real buildings (Park and Keong, 1979). Lateral load reversal is then applied to the end of the top beam by a 440-kN servo actuator. In the test, the displacement controls are adopted through the whole loading stages. The test specimens are subjected to two complete cycles of reversed loading gradually to achieve the indicated drift ratios as shown in Fig. 3. Both the two tests are continued until the specimen experiences a significant loss of capacity, which is generally regarded as 0.8 of maximum applied lateral load.

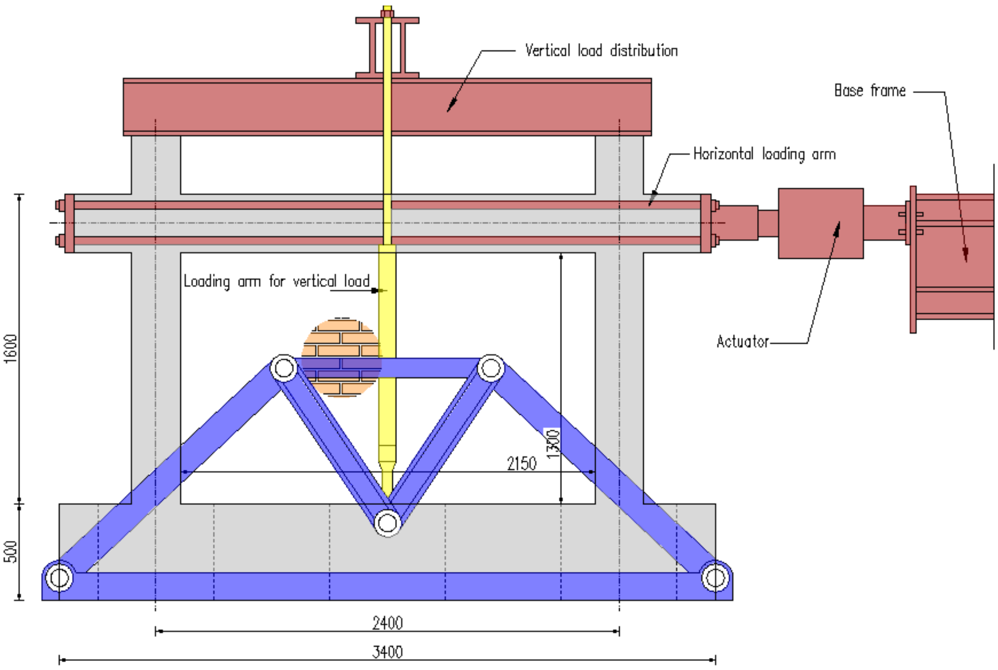


Figure 2. Test set-up and loading arrangement (unit: mm)

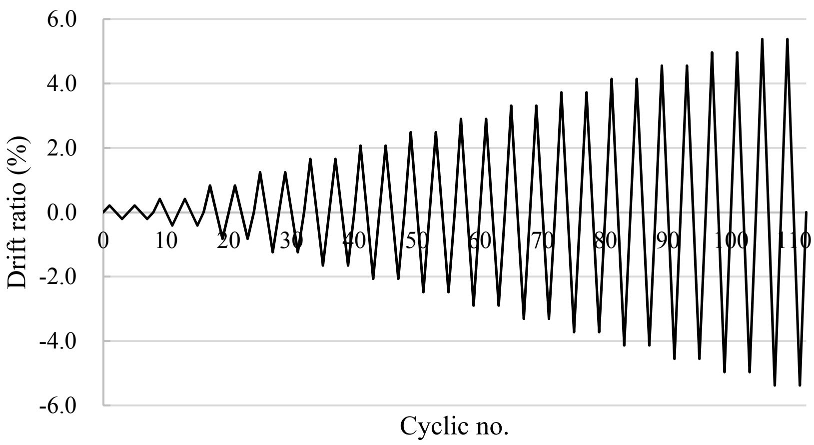
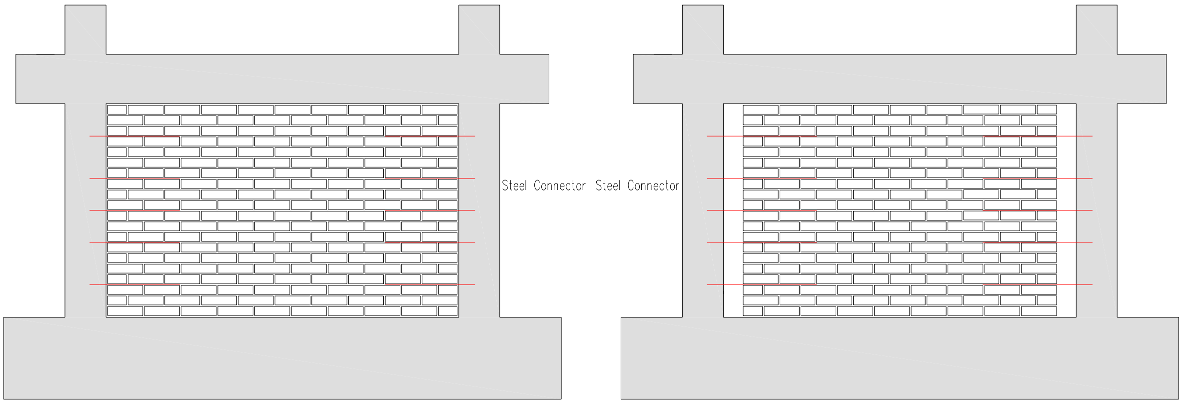


Figure 3. Loading histories applied to the test specimen

Description of test

As aforementioned, the major purpose of the study is to experimentally invesitgate the perferable improvement of seimisc behaviour of reinforced concrete frame under the column-isolated infill condition. The two comparable specimens with different infill configurations are clearly illustrated in Fig. 4. Specifically, Specimen C1 is presented here as a benchmark test based on the Eurocode 8 to be fulfilled with masonry infills full contacting with bounding frame. Comparatively, Specimen C2 is shown as an column-isolated infills, which the masonry panel is only isolated with columns but still tightly fit to the bounding beam. In fact, this design is different from the past published references and also the American masonry code ACI 530.1-11, which are all specified that the masonry infills shall be isolated with both bounding beam and columns. It is reasonable in the design of Specimen C2 because the fully isolated infills will tremendously increase the risk of out-of-plane failure, while keeping the contacting with beam can effectively guarantee the development of so-called “Arching Action”. Meanwhile, it should be emphesised that both the specimens are introduced the “steel connectors” to effectively resist the instability of masonry infills under the out-of-plane loads. Those connectors are designed to the Chinese seismic code GB50011-2010 by detailing as 2 R6 mild bars at each level with the effective anchorage length in masonry panel as 450 mm and 200 ~250 mm for spacing length.



(a) Specimen C1 (b) Specimen C2

Figure 4. Infills configurations of specimens

GENERAL OBSERVATION OF TESTS

The major failure modes of test specimens corresponding lateral drift ratios and force levels are summarised at Table 1. The apparent state of damage at drifts of 0.4 %, 1.2 %, and 3.6 % are illustrated in Fig. 5. From test observations, it was found that the bounding frame of both specimens experienced the two distinct failure stages at the early stage: flexural failure and shear failure. The flexural failure stage was initiated by few small flexural cracks occurred at the hogging moment regions of columns or beam under the relatively low lateral drift around 0.4 %, while the shear failure stage progressively occurred when several obvious shear cracks suddenly developed at column ends and beam-column joint regions at the drift of over 1.2 %. Under the relative large lateral drift at around 2.5 ~ 3.0 %, the structure system was almost reached the ultimate strength. After that, the spalling of concrete occurred at critical regions of columns accompanied by the much severe shear cracks at larger drifts (> 3.0 %). These kind of failure revealed that the structure system begun to fail under the increase of deformation levels.

Table 1. Major damage stages of bounding frame

Specimen	First flexural cracks		First shear cracks		Peak lateral load		Concrete spalling	
	Drift [%]	Load [kN]	Drift [%]	Load [kN]	Drift [%]	Load [kN]	Drift [%]	Load [kN]
C1	0.4	248	1.2	343	2.6	432	3.7	258
C2	0.4	135	1.2	209	3.3	259	4.2	249

Although the specimens were subjected to the significant different loading level with respect to the different frame-infill systems, the two specimens experienced the two failure stages at almost the same lateral drifts. Therefore, it may be deduced under the limited records that the lateral drift ratio can be regarded as a major parameter to reflect the failure modes of reinforced concrete frame structure.

The corresponding damage of masonry-infill walls were significantly different due to different frame-infills configurations (Fig. 5). It is observed that even at low lateral drift (0.4 %), the standard infill wall (Specimen C1) was appearred some minor cracks at masonry units along the diagonal direction and wall-to-beam interface, while the isolated wall (Specimen C2) was intact and only displayed local buckling of steel connectors at higher height of the wall. At the lateral drift of 1.2 %, it is particularly apparent that the standard infill wall (Specimen C1) was sufferred the large lateral load and developed several major diagonal tensile cracks parallel to the columns' shear crack direction. On the contrary, the isolated infills (Specimen C2) was almost intact and only few cracks emerged along the mortar joints, meanwhile, the steel connectors experienced larger deformation and most of them were buckled under the reversed cyclic loads.

Spalling and crushing of concrete were appeared at column ends accompanied with severe shear cracks at drift ratios of 2.0~2.5 %. These behaviour might imply that the structure would approach the maximum lateral load and the lateral strength of structure will then be progressively turned down with the increasing of lateral displacements. It should be indicated that the peak load for isolated infills condition (Specimen C2) was only around 60 % of standard one (Specimen C1), which were consistent with the much severe damage in Specimen C1 due to stronger frame-infill interactions.

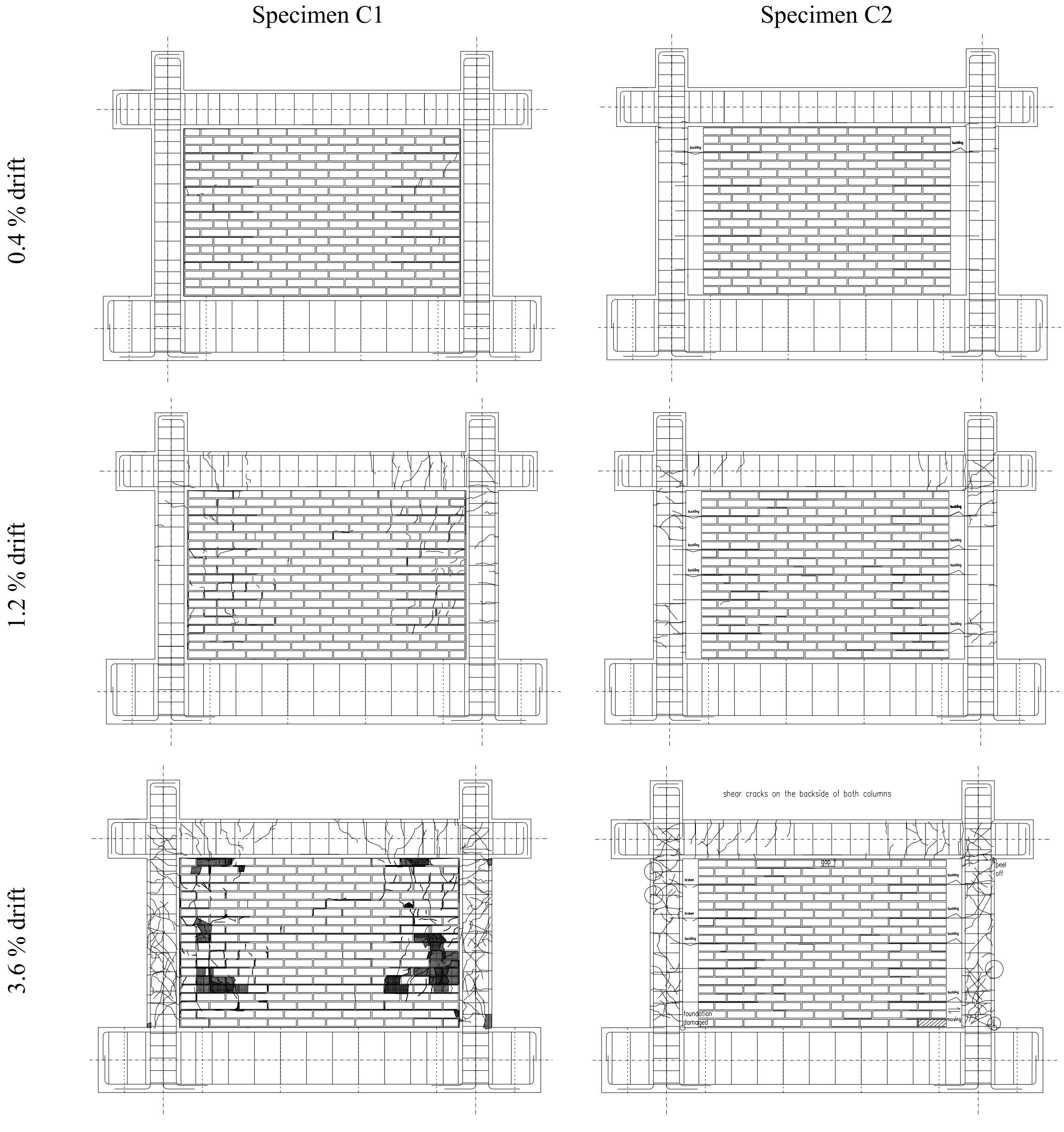


Figure 5. Failure modes of specimens under different lateral drifts

(a) Specimen C1

(b) Specimen C2

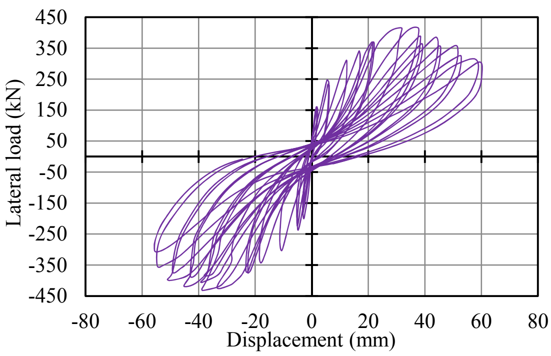
Figure 6. Damage pattern of test specimens

The failure patterns of two specimens at drift of 3.6 % and final stage are shown in Fig. 5 and Fig. 6 respectively. It can be seen that the standard specimen (Specimen C1) was obviously sufferred much severe damage than the column-isolated one (Specimen C2). In Specimen C1, the seriously local crushing and collapse of infills at lower mid-height region near columns, particularly at right column, finally restrain the column deformation only at the lower-mid height regions, and thus causing the short column failure finally concentrate at the right column end. Comparably, in Specimen C2, the masonry infill wall was almost intact and only developed minor cracks at masonry units and mortar joints. However, it should be indicated that more than half of the steel connectors were eventually fractured and resulted in the potential risk of the instability of maosnry wall in the out-of-plane direction.

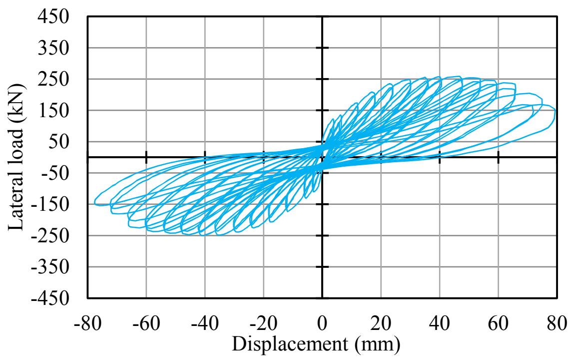
ANALYSIS OF TEST RESULTS

Hysteresis behaviour

The hysteretic responses of two specimens are illustrated in the form of displacement plotted against corresponding horizontal applied load (Fig. 7). It is observed that the standard test (Specimen C1) displayed the poor seismic performance as compared with the column-isolated one in terms of hysteretic behaviour under reversed cyclic loading. It is experimentally recorded that the undesirable earlier deterioration of concrete after the maximum strength give rise to the rapid drop of stiffness and strength, and eventually decrease in global ductility. At the same time, the remarkable pinching effect is observed particularly in Specimen C1 due to the strong frame-infills interaction. The further exhibited hysteresis behaviour with only the first cycle clearly compared the different hysteretic behaviour of the two tests (Fig. 8).



(a) Specimen C1



(b) Specimen C2

Figure 7. Load-displacement hysteresis curves

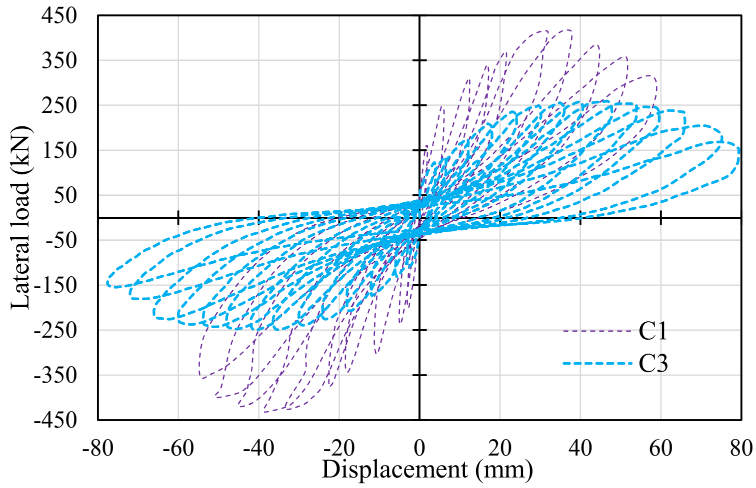


Figure 8. Load-displacement hysteresis loops (first cycle)

Ductility and stiffness

Ductility is considered as a major parameter to investigate the seismic performance of reinforced concrete structures. In this paper, the displacement ductility factor μ is defined as a ratio of displacement at 80 % of the maximum resistance Δ_u to the displacement of longitudinal reinforcement bar yielding Δ_y. The displacement ductility factor are summarized in Table 3. It is shown that the ductility factor can be achieved to nearly 4.2 in the column-isolated test (Specimen C2), which is more than 20 % enhancement of the standard test (Specimen C1). The load-displacement envelope curves are plotted in Fig. 10. It is particularly apparent that the standard curve (Specimen C1) has much higher peak strength, however, experiences a relatively rapid drop of strength after reaching the maximum strength. On the other hand, the column-isolated curve (Specimen C2) only reaches the peak strength around 260kN and behaves in a typical ductile manner on a comparatively flat plateau without a substantial reduction in strength even undergoing large inelastic deformation.

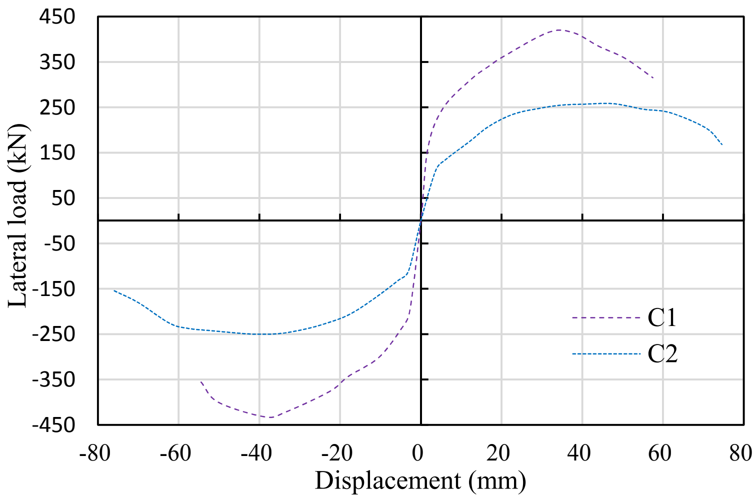
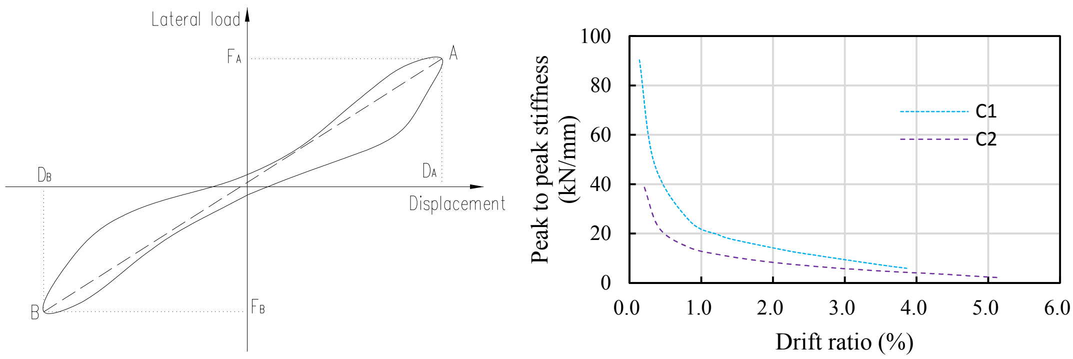


Figure 10. Load-displacement envelope curves

Table 3. Displacement ductility factor μ

	Positive		Negative	
	Specimen C1	Specimen C2	Specimen C1	Specimen C2
Peak (kN)	415.61	258.84	-432.24	-249.88
0.8Peak (kN)	332.49	207.07	-345.79	-199.9
Ductility	3.56	4.17	3.17	4.16

The peak-to-peak lateral stiffness is employed to evaluate the lateral stiffness of the frame load-displacement relationship. It is illustrated in Fig. 10(a) that the peak-to-peak stiffness can be obtained from the slop of line connecting the positive and negative peak point at A and B in a loading cycle. As shown in Fig. 10(b), it can be seen that the the standard test (Specimen C1) has much higher stiffness than the column-isolated one (Specimen C2), which can be explained by the strong frame-infill interaction actions developed in the standard specimen. It is observed that the peak-to-peak stiffness of both specimens at the early stage are high but degrade very rapidly until drfit ratios reaches to 1 %. This sudden drop of the test at the early drift of less than 1 % is consistent with the serviceability and ultimate limit states as mentioned above. After about 2.0 % drift, similar stiffness can be abtained from two specimens which is implied that the stiffening effect of infills is relatively small, then the structure progressively fails to collapse with the increasing deformations.



(a) Definition (b) Degradation of stiffness

Figure 10. Peak-to-peak stiffness of two specimens

Energy dissipation capacity

The energy dissipation capacity of two specimens are calculated in terms of the area enclosed within the hysteretic loops in each cycle. The cumulative energy dissipation plotted against the lateral drift ratios are shown in Fig. 11. It is observed that the energy dissipation for standard test (Specimen C1) is much larger than for column-isolated test (Specimen C2) under the same latearl drift. This is mainly because the masonry infills greatly interact with surrounding frame and give rise to the pronounced increase of lateral resistance, finally increasing the total energy in the standard test. Meanwhile, it is found that both the specimens underwent a effective increase of dissipated energy at the drift raios around 2~3 %. At the same time, the relative energy dissipation ratio β is introduced according to ACI 374.1-05. As shown in Fig. 12, it is indicated that the energy dissipation capacity in isolated test (Specimen C2) is much more effective than the standard test (Specimen C1).

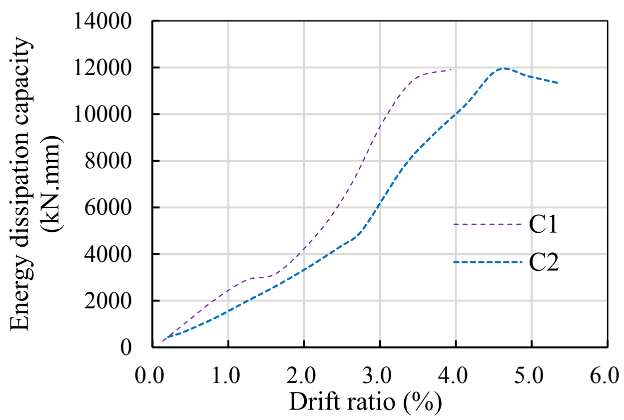


Figure 11. Calculated dissipated energy versus drift ratios

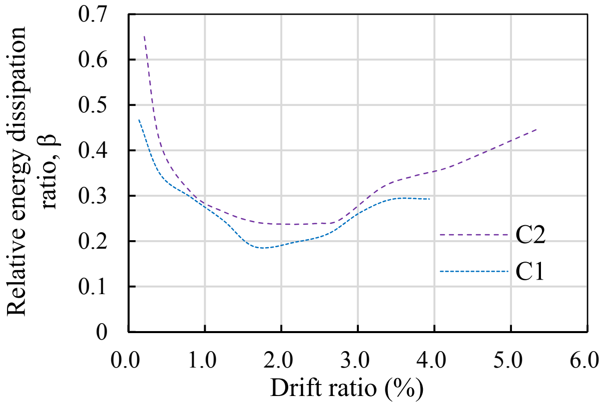


Figure 12. Relative energy dissipation ratio β versus drift ratios

CONCLUSION

The experimental study has been directed to systematically investigate masonry-infilled reinforced concrete frame structure with and without the column-isolated gap. It is noticed that the "non-participating infills" have already been introduced in the updated American masonry code ACI 530.1-11, and treated as a current and future trend of seismic design approach in terms of the complicated frame-infill interaction issue.

In this paper, two large-scale, single-storeyand single-bay masonry-infilled reinforced concrete frame specimens were tested and recorded under reversed in-plane cyclic loading. Configurations of the full contact (standard) and column-isolated infills are thoroughly studied. Test results in hysteresis curves and corresponding failure modes of surounding frame show that the column-isolated infills condition can effectively preserve the integrity of infill panel and exhibits the improved seismic performance than the standard one in terms of ductility, degradation of stiffness and strength, and energy dissipation capability. It should be mentioned that an certain amount of steel connectors are strongly required to prevent the possible instability from out-of-plane failure.

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