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Advantages and limitations of retrofitting masonry infilled RC Frames by Ferro-cement based on experimental observations

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Abstract

Seismic strengthening of RC buildings is one of the most important concern for structural engineers, especially in developing countries which has already been proved by the severe damage and large number of injuries during the past earthquakes, such as Nepal Earthquake 2015. RC buildings with masonry infill are one of the most popular structures in developing countries. The masonry infill walls are used as partition walls and commonly considered nonstructural elements. In this context, strengthening of existing non-structural component of RC frame, i.e. infill masonry, and use it as structural element would be a feasible and low cost solution. Among several retrofitting techniques, Ferro-cement lamination is low cost, can be easily applied and low labor intensive.

In general, ferro-cement retrofitting of masonry refers to the application of an initial mortar layer on the both faces of masonry wall which is followed by the placement of steel wire mesh and a second mortar layer. Some anchorages are also being used to attach wire-mesh to masonry and RC frame. Though, ferro-cement has been studied for decades as a construction material, there is no design specification e.g. amount of mesh reinforcement, mortar thickness etc. for using as a shear strengthening material on unreinforced infilled masonry. In addition, the estimation of improvement of seismic capacity of Ferro-cement is still unclear. On other words, the main problem is lack of understanding of influencing parameters and construction details, which greatly affect the performance of FC (such ratio of steel, type of connection to URM and mortar strength). This caused contradiction of conclusions in the past studies. Therefore, the objective of this study is first to investigate the in-plane seismic capacity and failure modes of masonry infilled RC frame when retrofitted with Ferro-cement. This study focuses on FC performance if applied as retrofitting after a damaging earthquake, where, many buildings need urgent repair and retrofitting in order to prevent further severe damage or collapse due to after-shocks and future major earthquake. This study is divided on two main parts: First, presents an experimental program and main results of two 1/2 scaled specimens of masonry infilled. The test specimens were first tested under in-plane static cyclic loading until the failure of masonry infill and then retrofitted by Ferro-cement was applied and reloaded again. The results showed the beneficial increase of 1.6 times of strength for the retrofitted specimen, but the failure modes and ductility of RC frame completely changed from flexural failure to punching shear failure which are overlooked in the past studies. The main results showed that failure mode of Ferro-cement does not depend only on wire mesh steel ratio and masonry but also the lateral capacity of surrounding RC frame and connections between frame and infill plays important role to control the failure mechanisms. Second part investigates an evaluation method of observed failure mode and lateral seismic capacity and proposed evaluation which showed a conservative estimation for the observed failure mode. Finally, a review of observations and limitations regarding FC retrofit are discussed.

Keywords: Masonry infill; Seismic retrofit; Ferro-cement lamination; Reinforced concrete frame

1. Introduction

Reinforced concrete (RC) buildings having masonry infills as partition walls are one of the most common structures in the world, especially in Europe and developing countries. Masonry infill is considered as a non-structural element but its damage as a structural element has been repeatedly observed in many earthquakes such as: the 2009 L'Aquila earthquake in Italy, 2008 Sichuan earthquake in China and 2015 Nepal earthquake. There are many techniques developed to retrofit buildings such as adding steel bracing or post installed walls. However, retrofitting a large stock of vulnerable exiting buildings is economically unfeasible for developing countries because of high costs and expertise needed. The masonry infill walls are used as partition walls and commonly considered nonstructural elements. In this context, strengthening of existing non-structural component of RC frame, i.e. infill masonry, and use it as structural element would be a feasible and low cost solution, the main concept is shown in Fig.1. Among all other retrofitting techniques, Ferro-cement lamination on masonry infill is also easy to apply and less labor intensive. Although FC can increase both in-plane and out of plane strength of masonry wall, this study will focus only on the in-plane capacity.

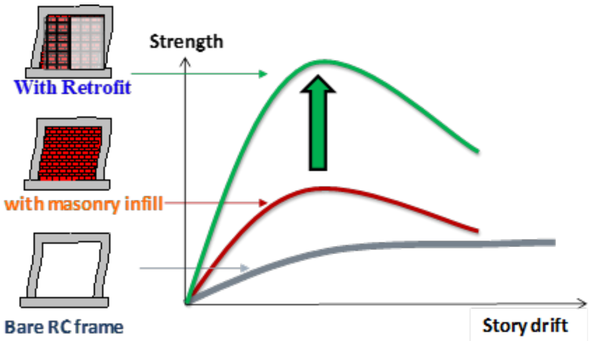


Fig. 1- Concept of in-plane retrofit of masonry infilled RC frame

In general, ferro-cement (FC) retrofitting of masonry refers to the application of an initial mortar layer on both faces of the masonry wall which is followed by the placement of steel wire mesh and a second mortar layer, as illustrated in Fig.2. In some cases, anchorage is used to attach the wire-mesh to masonry and RC frame. Though, Ferro-cement has been studied for decades as a construction material, there is no design guideline and no design specification e.g. amount of mesh reinforcement, mortar thickness for using as a retrofitting material on unreinforced infilled masonry. ACI-549 [1] also acknowledges the lacking of study on the Ferro-cement under lateral force.

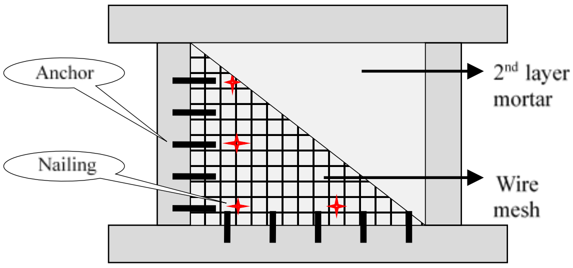


Fig. 2- Illustration of Ferro-cement laminated on masonry infilled RC frame

In this regard, several experiments and studies are ongoing to test retrofitting with FC as part of a wider scope ongoing experimental program of a Japanese project called SATREPS [8], which intended to upgrade seismic evaluation methods of reinforced concrete buildings in Bangladesh. A companion paper by Sen et al. [9] investigating FC regarding the influence of increasing the wire mesh area ratio. This paper investigates FC performance if applied as retrofitting after a damaging earthquake, where, many buildings need urgent repair and retrofitting in order to prevent further severe damage or collapse due to after-shocks and future major earthquake. In this case, FC is a good candidate for retrofit since it can be easily applied on damaged building with less labor requirement. Even though there are several experiments related to retrofitting with FC [2-7], there is a lack of experiments or past studies on seismic performance of retrofitting of previously damaged masonry infilled RC frames with FC.

The objective of this study is to experimentally investigate the in-plane seismic capacity and failure mode of a masonry infilled RC frame which has been previously damaged and then retrofitted with Ferro-cement. This study presents an experimental program and results of half scaled specimen of masonry infilled RC frame specimen before and after retrofitting with FC. Initially, the test specimen has been subjected to in-plane static cyclic loading until the failure of masonry infill and moderate damage on RC frame. This is followed by insertion of new masonry infill inside the damaged RC frame, along with ferro-cement lamination and reloaded.

2. Test program

2.1 Specimen Before Retrofit (BR)

Several parameters might greatly influence the seismic performance of masonry infill such as the masonry type, panel aspect ratio, mortar characteristics and strength, frame strength and vertical load. The specimen of masonry infilled RC frame in this study is part of an experimental program investigating the influence of ratio strength of RC frame to masonry infill strength ratio, considering variances in existing RC buildings in Bangladesh. The case study of Bangladesh is considered, since this experimental study is a part of a wider scope ongoing experimental program of a Japanese project called SATREPS [8], which intended to upgrade seismic evaluation methods of reinforced concrete buildings in Bangladesh.

The specimen in this study considers the parameter of relatively strong frame and weak infill. To classify the frame into weak and strong ones, the β index is used, which is defined in this study, as shown in Eq. (1).

$$\beta = V_f / V_{inf} \tag{1}$$

Where V_f is the boundary frame lateral strength which is calculated to be the ultimate flexural capacity of a bare frame with plastic hinges at top and bottom of columns. V_{inf} is the masonry infill lateral strength calculated based on Eq. (2) which is a simplified empirical equation showing good agreement with previous experimental database of 22 specimens studied by the author [10].

$$V_{inf} = 0.05 f_m \cdot t_{inf} \cdot l_{inf} \tag{2}$$

Where f_m is the masonry prism compressive strength, t_{inf} is the thickness of infill, and l_{inf} is the length of infill.

The β index of the specimen (before retrofit) is 1.5 which represent a relatively strong frame. The detailed calculation of β index of this specimen and with comparison with other specimens in the previous experimental program are published in [11].

The specimen dimensions are shown in Fig.3. The infill panels are constructed using 60x100x210 mm (height x thickness x length) solid bricks. A professional mason built the infill after the frame construction, where its thickness is 100mm and mortar head and bed joint thickness is about 10mm.

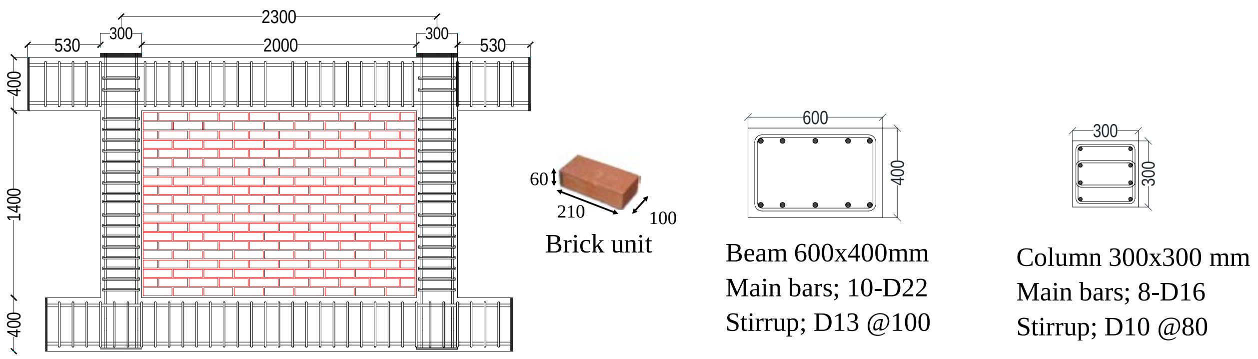


Fig. 3- Dimensions and reinforcement of specimen BR; units in mm

Table 1, Table 2 and Table 3, show the material mechanical properties of concrete, masonry and reinforcing steel, respectively. The masonry prism compressive strength was tested as per the ASTM [12]. The proportion of cement and sand for the mortar used for masonry infill is 1:2.5 (mass proportion).

Table.1- Concrete properties of RC frame

Compressive strength (MPa)	28.3
Elastic modulus (MPa)	27100
Split Tensile strength (MPa)	2.4

Table.2- Masonry properties

Prism compressive strength (MPa)	18.6
Elastic modulus (MPa)	8140
Brick unit compressive strength	38.1
Joint mortar compressive strength	34.9

Table.3- Reinforcement mechanical properties

Bar	Nominal strength	Yield strength (MPa)	Ultimate tensile strength (MPa)
D10	SD345	384	547
D13	SD345	356	555
D16	SD345	370	556
D22	SD390	447	619

2.2 Specimen After Retrofit (AR)

After loading the specimen (BR) with masonry infill, the damaged masonry infill was removed and a new masonry infill with same masonry materials was built inside the RC frame. Afterward, masonry has been retrofitted with FC. One of the most important parameters for FC is the wire mesh steel area ratio. Until now, there is no standard or guidelines for the application of FC on masonry. In order to understand common practice, past experimental results in literature [2-7] were investigated. The FC laminated masonry walls contain square wire mesh applied on masonry infill of solid or hollow bricks. The wire mesh steel area ratio used in past studies and its relation with the shear stress on FC lamination is shown in Fig. 4. Where wire mesh steel area ratio is represented by the ratio of horizontal mesh reinforcement area to masonry cross sectional area (A_{hs}/A_{mas}), where A_{hs} = total area of horizontal mesh reinforcement and A_{mas} = horizontal cross sectional area of masonry (length x thickness). The stress on FC lamination (τ_{FC}), has been determined from the difference in lateral capacity of retrofitted and without retrofitted specimens and then divided by cross sectional area of FC laminate (thickness of FC x length of FC).

As shown in Fig. 4, the previous studies had horizontal mesh reinforcement between 0.05~0.35% of the horizontal masonry area. The shear stress on FC layer varied greatly between specimens. This large variation in past experimental results could be due to varying materials types, wire mesh properties and connections of Ferro-cement layer with the surrounding RC frame. The specimen in this study is designed to have wire mesh ratio of 0.16% which is about lower boundary for wire mesh commonly used in literature. The wire mesh was applied to both back and front surfaces of the masonry panel. The wire mesh properties and mortar layers are shown in Table 4.

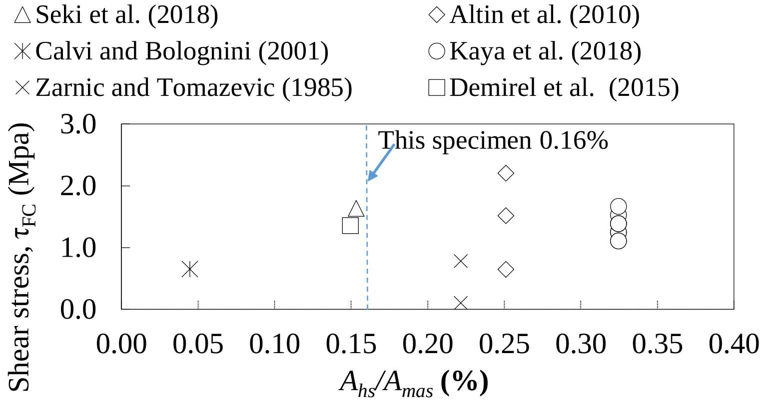


Fig. 4- Shear strength of FC layer as a function of mesh reinforcement ratio in past studies [2-7]

The construction procedure was as follows: first the masonry wall was rebuilt using materials similar to the original wall described previously in Table 2. Then, first layer of mortar of 10mm thickness was applied to both faces of the masonry wall. This was followed by the attachment of square wire mesh to the RC frame and masonry wall as shown in Fig. 5 and Fig. 6. The wire mesh has been connected to surrounding RC frame (both column and beam) with bolt (inserted thread) and steel plate at interval of about 100mm. It should be noted that connection of wire mesh with RC frame is a commonly overlooked practice in past studies, as only two studies [3-4] had some anchorage of FC with RC frame. In addition, the wire mesh has been connected with masonry infill by 32mm long nails to hold the wire mesh in place during application of second layer mortar as illustrated in Fig 5. The nails have been placed in drilled holes at a horizontal and vertical center to center distance of 250mm and 500mm, respectively. Epoxy was used to attach the nail with masonry. Then, a second layer of mortar with thickness of 15mm was applied on the wire mesh. The Ferro-cement mortar each layers had total thickness of 25mm and consisted of a rapid setting cement and sand ratio of 1:3. Rapid setting cement, which was used just to accelerate the retrofitting process.

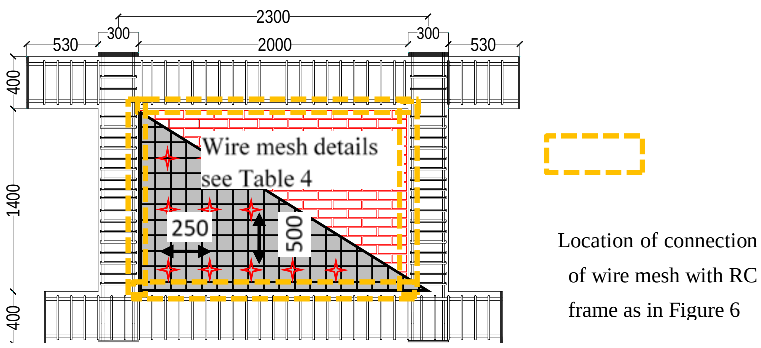
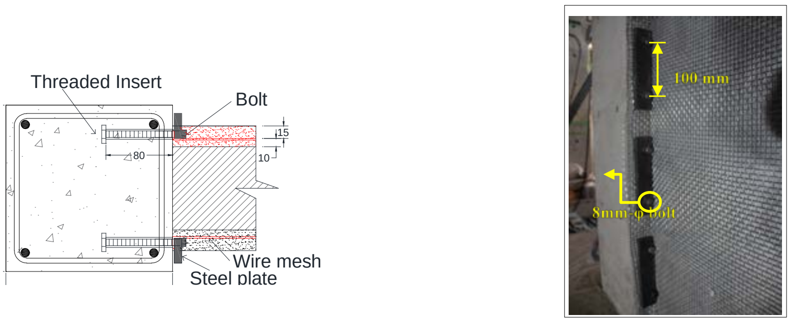


Fig. 5- Specimen after retrofit with FC retrofit



a) Top view b) Side view showing connection with column

Fig. 6- Connection of Wire mesh with RC frame

Table. 4- Ferro-cement properties

Wire mesh spacing (mm)	5.45
Wire diameter (mm)	0.9
Wire ultimate tensile strength σ_{wu} (MPa)	378
Wire yield tensile strength σ_{wy} (MPa)*	350
Mortar compressive strength (MPa)	37.9

* *σ_{wy} is not tested but taken 0.925 of ultimate tensile strength*

2.3 Test setup and loading protocol

The loading system is shown schematically in Fig.7. The vertical load applied on RC columns by two vertical hydraulic jacks and was maintained to be 200kN on each column. The common construction practice is that masonry infill is inserted (infilled) after the construction of RC frames, in that case the gravity load already taken by columns. Therefore, vertical load is applied directly to the columns. Two pantographs, attached with the vertical jacks, restricted any torsional and out-of-plane displacement. Two horizontal jacks simultaneously applied incremental cyclic loading, were attached at the beam level and were controlled by a drift angle of R%, defined as the ratio of lateral story deformation to the story height measured at the mid-depth of the beam (h=1600mm). The lateral loading program consisted of 2 cycles for each peak drift angle of 0.05%, 0.1%, 0.2%, 0.4%, 0.6%, 0.8%, 1%, 1.5% and 2%. After loading the un-strengthened masonry infilled specimen (BR), masonry infill was removed and the bare RC frame was reloaded till 2.2%. Then FC laminated masonry wall has been inserted, as discussed in the earlier section, and loaded in similar way to the main loading as shown in Fig.8.

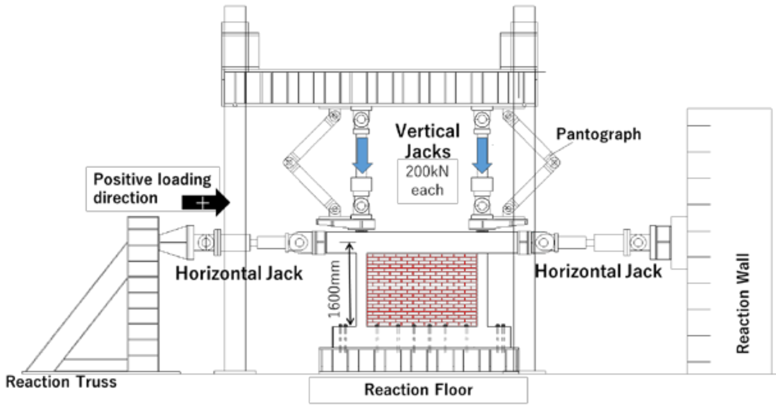


Fig. 7- Loading system

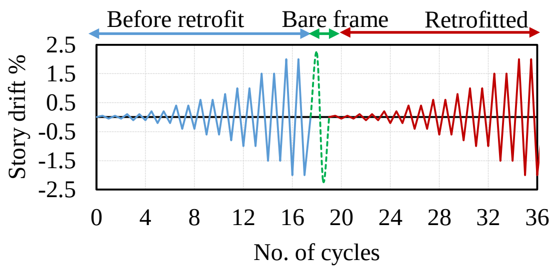


Fig. 8- Loading protocol

3. Experimental results

3.1 Specimen before retrofit (BR)

The lateral load versus story drift angle of masonry infilled specimen is shown in Fig.9. Crack and failure patterns after 2.0% are shown in Fig. 10.

The damage progression is as follows: very small cracks on mortar bed joint and diagonal cracks on bricks near loading corner of infill panel with width less than 0.3mm started at early stages of loading just when the drift angle was 0.05%. At drift angles between 0.6% ~ 0.7%, both columns yielded similar at ends of columns similar to RC bare frame (with strong beam and weak column). As it reached its maximum strength the lateral load gradually degraded with the drift angle increase until the drift angle of 1.5%, where there was a slight drop of the lateral load, after that sliding failure could be observed clearly. Crushing of masonry infill as diagonal compression failure is thought to be the dominant failure mechanism. At a drift angle of 2% the lateral load reached 355kN which is about 0.6 of maximum strength and the loading stopped as planned.

Even though the masonry is severely damaged as shown in Fig. 10, but the RC frame was moderately damaged with maximum residual cracks of width less than 2mm at hinge locations as shown in Fig. 11 and there was no spalling of concrete cover.

The damaged masonry infill wall is then removed and the RC frame is then reloaded till it reaches 2.2% and lateral load versus story drift angle for that cycle is shown in Fig. 9. The maximum lateral load of RC frame was 281 kN which is similar to the calculated maximum strength which is discussed in later section. It should be noted that the cracks in RC frame was not repaired.

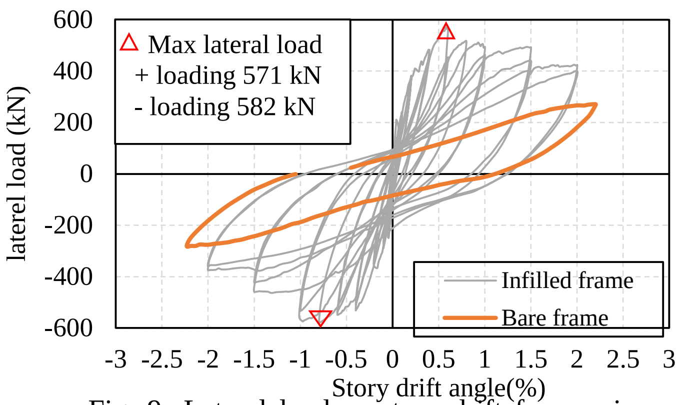


Fig. 9- Lateral load vs story drift for specimen

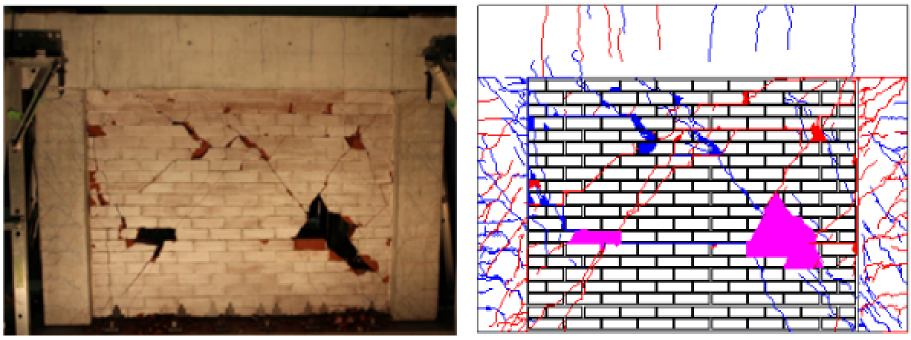


Fig.10- Failure of Specimen (BR) at story drift 2%

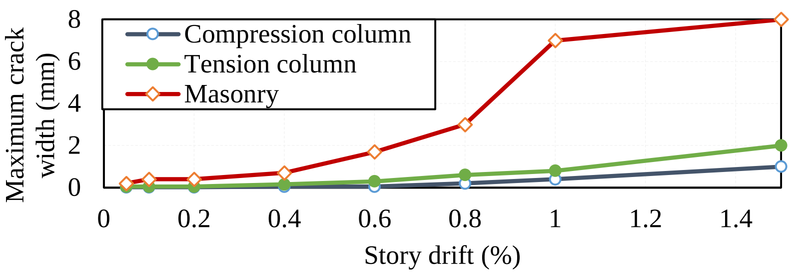


Fig.11- Residual maximum crack width at each cycle for specimen (BR)3.2 Specimen after retrofit (AR)

The lateral load versus story drift angle of the specimen after retrofit (AR) is shown in Fig.12. The Failure patterns after final drift cycle of 2.0% is shown in Fig.13.

The damage progression is as follows: one small inclined crack of width 0.35mm start to appear at the Ferro cement layer near the compression column. No other cracks were visible. The one diagonal crack started to extend gradually at each loading cycle. At story drift of 0.8%, shear cracks started to appear at the top of tension column, and sliding at the top of wall. The maximum strength of 942kN was also reached at story drift of 0.8%. From 1%~1.5%, the shear damage/crack at the top of column extended greatly. The punching shear failure following extended shear cracking in previous cycles has been obvious at the top of column (as shown in the close-up in Fig.13) and the lateral strength gradually decreased until test was ended at a story drift of 2 %.

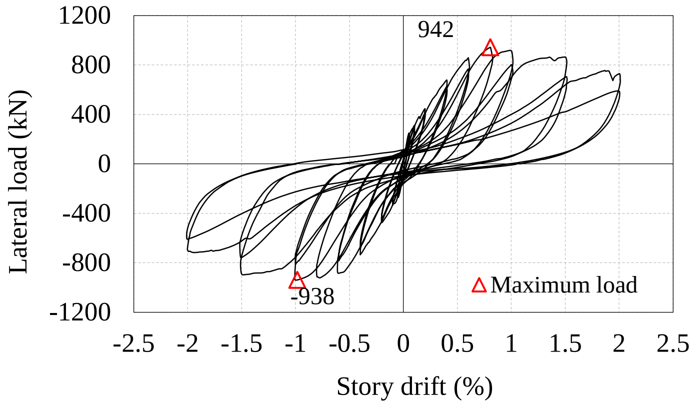


Fig. 12- Lateral load vs. story drift for specimen (AR)

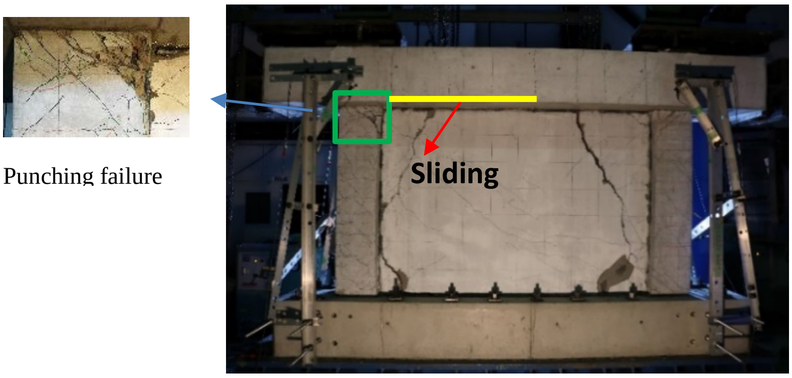


Fig. 13- Damage of specimen (AR) at story drift 2%

4. Discussion

4.1 Evaluation of experimental results:

The lateral strength of the specimen before retrofit (BR) can be estimated as the summation of the strength contribution of masonry infill and RC frame. It should be noted that this method is used for simplification, since obtaining the actual maximum lateral strength of the infill can be more complicated due to the complicated frame-panel interaction, variation of hinge locations, and internal varying axial load on columns, which are very challenging to pre-identify.

The bare frame strength here is calculated using Eq. (3).

$$V_f=4M_u/h_o \quad (3)$$

Where M_u is the minimum plastic moment of the column according to JBDPA [13] and h_o is the clear height of column (taken here as infill height). The moment capacity of columns was calculated considering axial load (200kN) applied by vertical jacks. The calculated strength of bare frame is 280kN, which is similar to the value of frame strength obtained after reloading the bare frame as shown in Fig.9. The masonry infill lateral strength capacity is calculated to be as 180kN using simplified Eq. 1. Thus, the lateral capacity estimate of specimen before retrofit is 460kN which is good estimate since Eq. (1) is thought to give conservative estimate considering the large variations of masonry material as stated in [10].

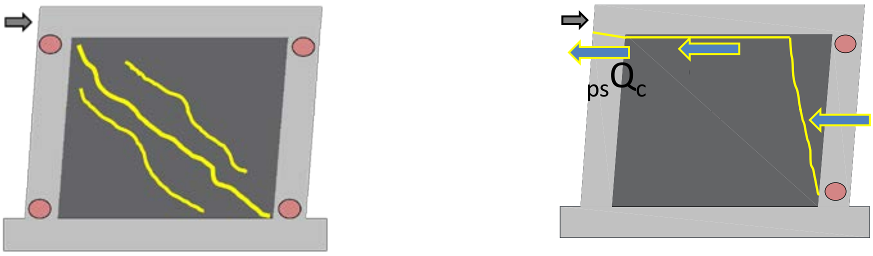
The maximum lateral strength of retrofitted specimen is 942 kN which is about 1. 6 times the original specimen, which proves that FC is effective in increasing the lateral strength. However, the failure mode is different from the original specimen and also the past research studies. In previous research studies such as Seki et al. [2], the FC is thought to fail in diagonal shear cracking as illustrated in Fig.12a). In that case, the FC will be effective all over the wall. In study by Sen et al. [14], a simplified equation is proposed assuming the FC layer fails in diagonal shear failure as shown in Eq. (4).

$$V_s = \alpha . n_s n_L \frac{A_s}{S_v} . f_y . h_{mas} \tag{4}$$

Where, V_s is the lateral load of FC, A_s is the cross sectional area of wire mesh of horizontal reinforcement, S_v is the spacing of horizontal mesh reinforcement, f_y is yield strength of mesh reinforcement, h_{mas} is height of masonry infill, α is an empirical reduction factor proposed as 0.7 based on empirical results, n_s is the number of surface retrofitted with FC, and n_L is the number of wire mesh layers in each FC layer.

Using Eq. (4) the expected shear strength of FC is only 80kN, which if applied here underestimate the strength of FC. One reason of this underestimation of strength and change of failure mode is the contribution of mortar strength applied in FC which is neglected in Eq. (4), which is relatively high in this study, 37.9 MPa, which greatly improved the diagonal compression capacity.

In addition, the failure mode observed in this study is completely different from previous studies, and thus application of Eq. (4) is not applicable here.



a) Diagonal failure in previous studies

b) Failure of the specimen AR after retrofit

Fig. 14- illustration of damage of masonry infilled retrofitted by FC

The failure mode of specimen retrofitted by FC in this study is similar to retrofitting with post installed RC wall, but with weak connections with frame. Therefore, the lateral strength is thought to be carried by summation of three main components: Punching shear at the top column (${}_{ps}Q_c$), bond strength of the wall with the top beam (Q_j), and lateral strength of compression column (${}_{f}Q_c$) as shown as Fig.14b) and Eq. (5):

$$Q_{sr} = {}_{ps}Q_c + Q_j + {}_{f}Q_c \tag{5}$$

The punching shear failure is calculated according to the according to JBDPA [12] (${}_{ps}Q_c$) as per Eq. (6):

$$ {}_{ps}Q_c = K_{\min} \tau_o b D \tag{6}$$

Where, K_{min} = 0.34/(0.52+a/D), a= shear span = D/3, τ_o is shear strength of tension column, b and D = width and depth of column, respectively.

As for compression column, it fails as flexural column and thus is calculated as Eq. (7).

$$ {}_{f}Q_c = \frac{2M_u}{h_o} \tag{7}$$

Where M_u is the flexural capacity of RC column, h_o is the clear height of RC column.

The bond strength of the wall with the top beam (Q_j) for FC is unclear at present with no previous studies. Several factors can influence the capacity of bond strength, such as connections strength, mortar cohesion capacity between masonry and RC frame. In this study, the bond strength (Q_j) is proposed to be taken as three main components: as the bond strength (cohesion strength) of mortar (${}_{w}Q_{mor}$) between masonry wall and top beam, the cohesion bond strength of mortar of FC (${}_{FC}Q_{mor}$) with top beam, and dowel action of wire mesh connected to the beam ${}_{FC}Q_{dowel}$, as shown in Eqs. (8) – (11).

$$Q_j = {}_{w}Q_{mor} + {}_{FC}Q_{mor} + {}_{FC}Q_{dowel} \tag{8}$$

$${}_{w}Q_{mor} = \tau_{mor} \cdot t_{inf} \cdot l_{inf} = 0.17\sqrt{f_{mor}} \cdot t_{inf} \cdot l_{inf} \tag{9}$$

$${}_{FC}Q_{mo} = \tau_{mor} \cdot t_{fc} \cdot l_{inf} \cdot ns = 0.17\sqrt{f_{mor}} \cdot t_{fc} \cdot l_{inf} \cdot n_s \tag{10}$$

$${}_{FC}Q_{dowel} = 0.25 \sum a_{wm} \cdot \sigma_{wy} \tag{11}$$

Where, τ_{mor} is the shear strength of mortar, f_{mor} is the compressive strength of mortar, t_{inf} and t_{fc} is the thickness of masonry infill and FC layer, respectively. l_{inf} is the length of infill. n_s is the number of surface retrofitted with FC. a_{wm} is cross sectional area of wire mesh in vertical direction. σ_{wy} is the yield strength of wire mesh. The factor 0.25 for dowel action used in Eq.11 is used as proposed by Pauley and Priestley [15]. The values for each component of Eqs. (5) – (11) is shown in Table 5.

The calculated values of both specimens after and before retrofit is shown in Fig. 15. The proposed evaluation of strength after FC retrofitting gave a conservative estimate of lateral strength of 857kN which is about 90% of the observed maximum strength. As shown in Table 5, about 40% of lateral strength is provided by the punching shear failure of top column. This is because the RC frame is relatively strong frame and well detailed reinforcement. However, in case of weak column with poor detailing, which is the common case in many developing countries, this punching failure in unfavorable brittle failure and could cause catastrophic collapse of a building. This failure mode is overlooked in previous studies of FC and needs further evaluation.

It should be noted that, Eq. (9) and Eq. (10) takes the mortar cohesion strength (bond strength) as general shear strength of mortar. This assumption gave good agreement with the experimental data, but need further verification, since bond strength depends on other factors such as quality of mortar, surface roughness, quality of construction, which can greatly affect the bond strength. In addition, in order to improve the performance of FC and to avoid punching failure, better connections of RC frame with FC need to be revaluated, this area of research lack experimental studies.

Table. 5- Calculated strength of specimen AR

${}_{ps}Q_c$ (kN) Eq.5	${}_{f}Q_c$ (kN) Eq.5	Q_j Eq.8			Total calcucated (KN)
		${}_{w}Q_{mor}$ (kN)	${}_{FC}Q_{mor}$ (kN)	${}_{FC}Q_{dowel}$ (kN)	
382	140	198	105	32	857

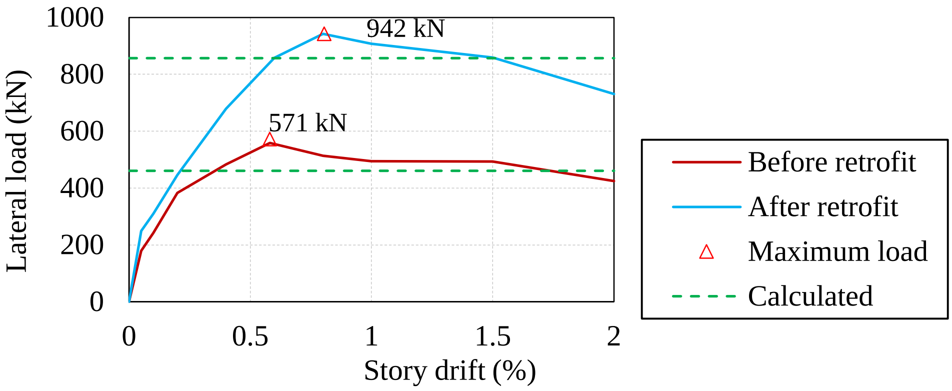


Fig. 15- comparison backbone curve of specimen after retrofit, with the calculated values.

4.2 Discussion of advantages and limitations of retrofitting by FC

From the observation of the tests of this study as well as reviewing several papers of FC in the literature, the following points regarding FC are noticed:

1- FC retrofitting on masonry infilled RC frame, improved the lateral strength by about 1.6 times, such increase was also observed in other companion experimental studies within the SATREPS project as Sen et al. [9], this increase could help to retrofit existing buildings at a relatively low cost.
2- However, the failure mode of RC frame, assumed to fail by flexural hinge formation for masonry infilled RC frame, could turn over to fail in shear failure or punching shear failure as observed in this study due to the large forces exerted on the columns. Shear failure is could be brittle and reduce the ductility of structure. Thus, if a moment resisting RC frame building is designed to have large ductility to resist seismic demand, the ductility could be affected after the retrofitting of infill walls using FC lamination. In other words, the FC retrofitting is a good candidate to improve seismic capacity of buildings lacking both strength and ductility such as old existing buildings. However, such retrofit scheme need to be carefully evaluated in case of application for newly designed buildings assumed to have large ductility factors. In other words, when FC is needed to be applied for the design of new building the response reduction factor, *R* should be adopted carefully to accommodate the possible non ductile behavior resulting from shear failure of surrounding RC column.
3- There are several failure modes of masonry infilled RC frame retrofitted by FC, such as failing as flexural walls similar to RC walls failing in flexural, such failure is not observed in this study, and are discussed in a study by Sen et al. [9]. In such case, the FC retrofit can show a relatively higher ductile performance. In order to have such failure the connections of infill and RC frame should be strong enough not to fail by pullout or sliding. There is a lack of studies regarding connection of masonry infill with RC frame and their performance, which needs further studies.
4- An evaluation methods of four possible failure modes of FC retrofitted techniques is discussed in a companion paper by Sen et al. [9]. Such evaluation might help to fill research gap on methods to evaluate lateral capacity of masonry infilled RC frame after retrofitting with FC. The number of tests is still limited, and needs further verification for other possible failure modes.
5- The out of plane performance of masonry infilled RC frame retrofitted by FC is thought to be greatly improved by application FC since masonry infill would act as one strong unit. In addition, the FC layer will decrease the slenderness ratio (height to thickness ratio of masonry wall), which will improve the out of plane performance. There is a lack of studies to evaluate such improvement of out of plane strength, and also interaction of in-plane and out of plane behavior of FC retrofit.
6- It should be noted that the above mentioned discussion points are for masonry infilled RC frame (RC frame is the main structure system and masonry are just partition walls) retrofitted by FC. Therefore, such observations do not necessarily apply for other structure types, such as masonry buildings or confined masonry structures.

5. CONCLUSION

This study experimentally investigated the in-plane seismic capacity of masonry infilled RC frame when retrofitted with FC. The study focused on FC retrofit performance if applied as retrofitting for a RC frame with prior damage. This represents the case of urgent retrofitting for RC buildings after a damaging earthquake, in order to prevent further severe damage due to after-shocks and future major earthquake. The following are main findings:

- Retrofitting with FC proved to be an effective retrofit scheme which showed an increase of strength to 1.6 times the original specimen. FC retrofitted is a good candidate to improve seismic capacity of buildings lacking both strength and ductility such as old existing buildings.
- The punching shear failure mode observed was different from the commonly assumed failure in past studies as diagonal cracking or compression failure. The punching shear failure occurred at top column, and masonry infill panel behaved similar to post installed RC wall having weak connections. One possible reason for the change of failure mode, is due to the large contribution of mortar strength applied in FC

with relatively high strength in this study, 37.9 MPa, which greatly improved the diagonal compression capacity and thus this mode was not observed.

- The proposed approach to evaluate the strength of specimen retrofitted with FC showed a conservative estimation for the observed failure mode.
- The bond (connection) between retrofitted panel with FC and RC frame, as well as FC with infill, is a commonly ignored parameter and lack past studies. This point needs further research in order to improve the estimation of seismic capacity of FC and to control the preferred failure mode.

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