- Technical Paper-

EXPERIMENTAL STUDY OF RETROFITTING MASONRY INFILLED RC FRAMES BY FERRO-CEMENT: AN OVERLOOKED FAILURE MECHANISM

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ABSTRACT

After a major earthquake, many buildings need an urgent repairing and retrofit in order to prevent further severe damage or collapse. Reinforced concrete (RC) buildings having masonry infills as partition walls are one of the common structures in many countries. Retrofitting masonry infill with ferro-cement, FC, is a good candidate for retrofit since its can be easily applied and low labor intensive. This study presents an experimental study of 1/2 scaled specimen retrofitted with FC and the evaluation of lateral seismic capacity of a failure mechanism that is not presented in previous research.

Keywords： RC buildings, Masonry infill, Seismic retrofit, Ferro-cement

1. INTRODUCTION

Reinforced concrete (RC) buildings having masonry infills as partition walls are one of the most common structures in the world, with the exception of Japan. This structure type is very common 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 labour intensive.,

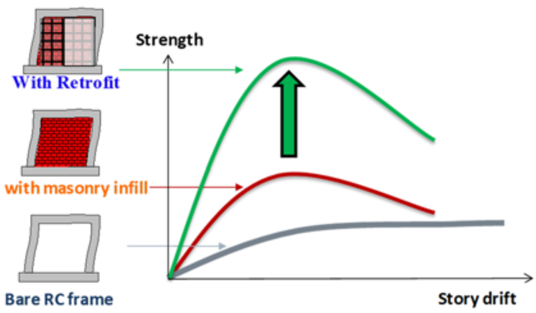

Fig. 1 Concept of in-plane retrofit of masonry infilled RC frame.

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.

Previous experimental research [2-7] have shown that FC can improve the in-plane strength of masonry infilled RC frame. However, the estimation of improvement of in-plane seismic capacity of Ferro-cement is still unclear. In other words, FC is still applied as non-engineering practice and there is a lack of knowledge about FC strengthening of infilled masonry.

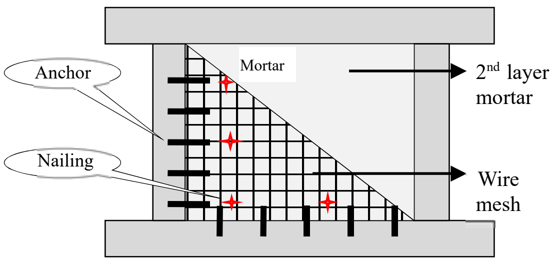

Fig. 2 Illustration of Ferro-cement laminated on masonry infilled RC frame .

In addition, after a damaging earthquake, 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

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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. 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. 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 1/2 scaled specimen of masonry infilled RC frame specimen before and after retrofitting with FC. The test specimen has been first tested under 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 in the damaged RC frame, along with ferro-cement lamination and reloaded again.

2. TESTS 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.

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.

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

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 in [9].

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

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 comparison with other specimens in the previous experimental program are published in [10].

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.

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 [11]. The proportion of cement and sand for the mortar used for masonry infill is 1:2.5 (mass proportion). The masonry infill strength is relatively high than masonry commonly used in developing countries. However, this experiment, the specimens were designed to reflect the parameter of ratio of frame strength of masonry strength.

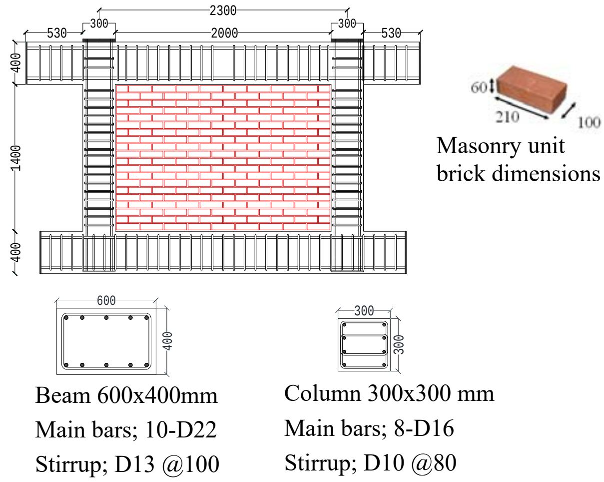


Fig. 3. Dimensions and reinforcement of specimen BR; units in mm

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 (MPa)	38.1
Joint mortar compressive strength (MPa)	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 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.

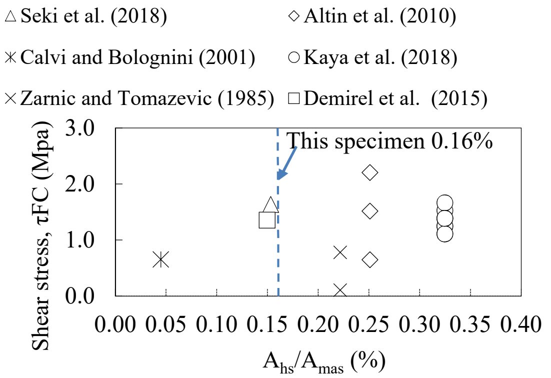


Fig. 4 Shear strength of FC layer as a function of mesh reinforcement ratio in past studies [2-7]

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 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.

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*

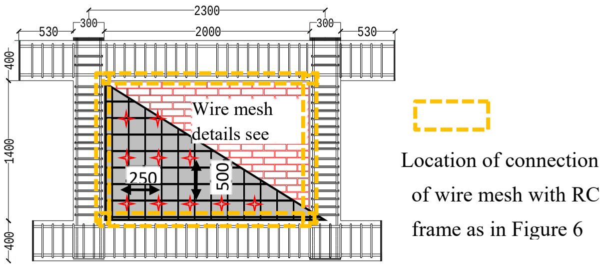
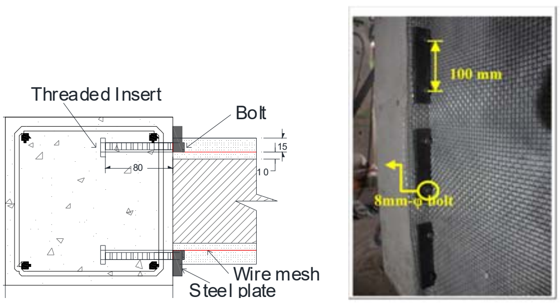


Fig. 5 Specimen after retrofit with FC retrofit



a) Top view

b) Side view showing connection with column

Fig. 6 Connection of Wire mesh with RC frame

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=1,600mm). 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%. Then, masonry infill was removed and 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.

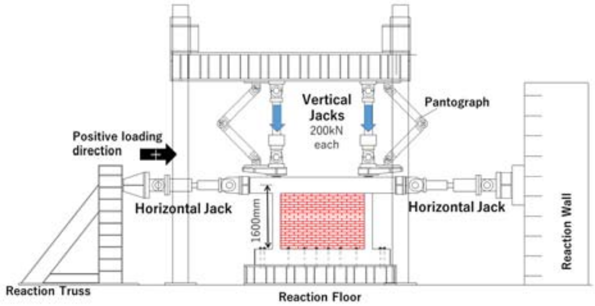


Fig. 7 Loading system

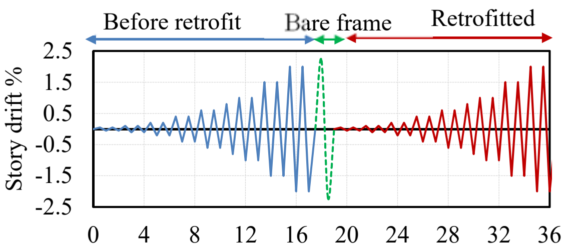


Fig. 8 Loading protocol

3. EXPERMENTAL 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 Figure 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.

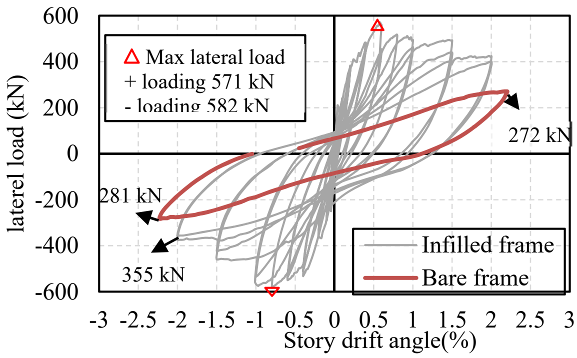


Fig. 9 Lateral load vs story drift for specimen (BR)

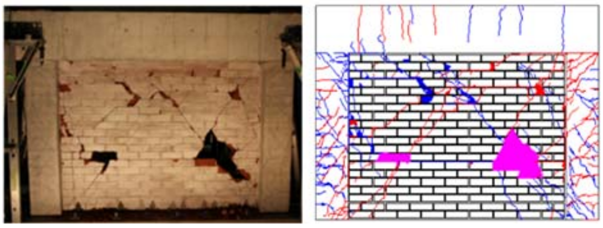


Fig.10 Failure of Specimen (BR) at story drift 2%

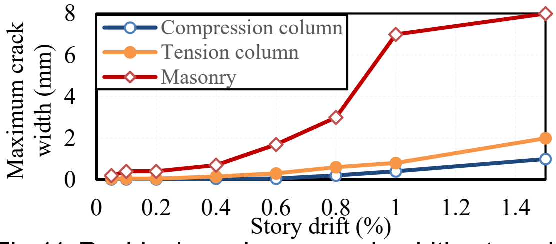


Fig.11 Residual maximum crack width at each cycle for specimen (BR)

3.2 Specimen after retrofit (AR)

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

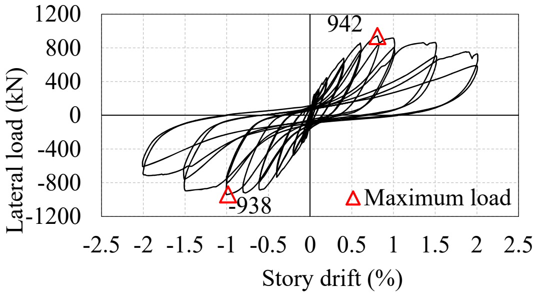


Fig. 12 Lateral load vs. story drift for specimen

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 terminated at a story drift of 2 %.

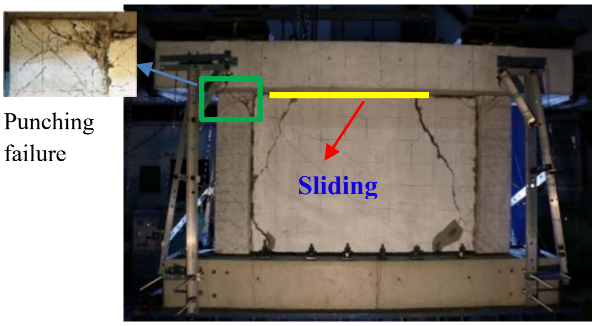


Fig. 13 Damage of specimen (AR) at story drift 2%

4. DISCUSSION OF 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.

$$V_f = 4M_u/h_o \tag{3}$$

Where M_u is the minimum plastic moment of the column according to [12] 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.2. Thus, the lateral capacity estimate of specimen before retrofit is 460kN which is good estimate since Eq.2 is thought to give conservative estimate considering the large variations of masonry material as stated in [9].

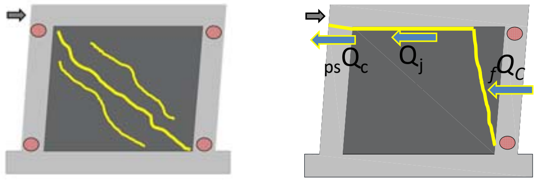
The maximum lateral strength of retrofitted specimen is 942 kN which is about 1. 6 times the original specimen, which proves that FC is effective in increasing the lateral strength. However, the failure mode is different from the original specimen and also the past research studies. In previous research studies such as [2], the FC is thought to fail in diagonal shear cracking as illustrated in Fig.14a). In that case the FC will be effective all over the wall. In study by Sen et al. [13], 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 cross sectional area of wire mesh of horizontal reinforcement, S_v is spacing of horizontal mesh reinforcement, f_y = yield strength of mesh reinf., h_{mas} = height of masonry infill, α= empirical reduction factor proposed as 0.7 based on empirical results, n_s = number of surface retrofitted with FC, and n_L=number of wire mesh layer 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 in Eq.3. could be the neglect of the contribution of mortar strength applied in FC.

In addition, the failure mode observed in this study is completely different from previous studies, and thus application of Eq.3 is not applicable here.



a) Diagonal failure in previous studies

b) Failure of the specimen 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 shown in Eq.6:

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

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 = flexural capacity of RC column, h_o = 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 Eq. (8-11).

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

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

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

$${}_{FC}Q_{dowel} = 0.\cdot 25 \sum a_{wm} \cdot \sigma_{wy} \tag{11}$$

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 [14]. The values for each component of Eq. (5-11) is shown in Table 5. It should be noted that Eq.8 takes the assumption of the three components reach their maximum strength together.

The calculated values of both specimens after and before retrofit is shown in Fig.15. The proposed evaluation of strength a conservative estimate of lateral strength of 857kN which is about 9% of 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 completely overlooked in previous studies.

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	$_fQ_c$ (kN) Eq.5	Q_j Eq.8			*Total calucated (KN)*
		$_wQ_{mor}$ (kN)	$_{FC}Q_{mor}$ (kN)	$_{FC}Q_{dowel}$ (kN)	
382	140	198	105	32	857

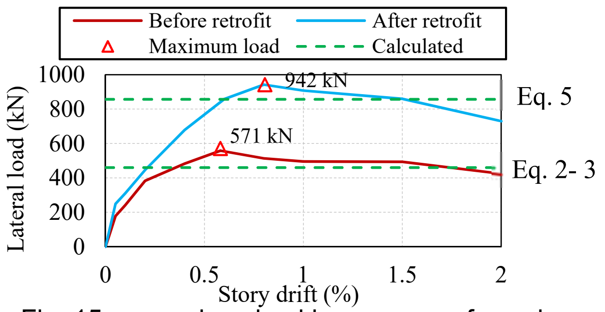


Fig. 15 comparison backbone curve of specimen after retrofit, with the calculated values.

5. CONCLUSION

This paper presented an experimental study of ½ scaled infilled masonry RC specimen before and after retrofit with FC and the following are main findings:

1- Retrofitting with FC proved to be an effective retrofit scheme which showed an increase of strength to 1.6 times the original specimen.
2- The punching shear failure mode observed was different from the commonly assumed failure in past studies. The punching shear failure occurred at top column, and masonry infill panel behaved similar to post installed RC wall having weak connections.
3- The proposed approach to evaluate the strength of specimen retrofitted with FC showed a conservative estimation for the observed failure mode.
4- The bond (connection) between retrofitted panel with FC and RC frame, is a commonly ignored parameter and lack past studies. This point needs further research in order to improve the evaluation and to control the preferred failure mode.

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