Seismic Performance Evaluation of Masonry Infilled Reinforced Concrete Buildings Utilizing Verified Masonry Properties in Applied Element Method

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Abstract
Present study aims on evaluating the seismic performances of existing Masonry Infilled Reinforced Concrete (MIRC) buildings commonly found in many South and Southeast Asian countries utilizing appropriate masonry properties in Applied Element Method (AEM) models. First, masonry constituents and masonry composite properties were determined for different masonry meshes through extensive parametric studies verified through the experimental results of masonry prisms under uniaxial compression and half scaled masonry infilled RC frames under in-plane cyclic load. Next, the established infill properties were utilized for time history dynamic analysis of an existing 8-storied MIRC building for different AEM models including soft story, retrofitted soft story, infills in all floors and bare RC frames neglecting stiffness contribution of infills. The analytical results revealed: 1) the unpredicted soft story column failure compared to the similar bare RC frame, 2) the inability of infills to improve the seismic performance of the surrounding RC frames, 3) the effectiveness of steel plate jacketing for preventing soft story failure and, 4) the effect of the existence of overhead water tanks on the alteration of seismic behavior of RC buildings.

1. Introduction
Multistoried masonry infilled reinforced concrete (MIRC) buildings with soft ground stories are the popular construction practice in many South and Southeast Asian countries e.g. Bangladesh, India, Pakistan, Nepal, Indonesia as well as in China. Generally, in all such framed structures, burnt clay bricks are used in partition walls serving as infill to the reinforced concrete framing system. Lack of knowledge of local masonry properties and unavailability of a practical and easy to use analysis and design methodology discourages structural engineers from considering infill walls as structural components. As a consequence, it has become a common practice to exclude the stiffness and strength contributions of infill in structural analysis and design of MIRC buildings subjected to seismic and other lateral loads. According to Asteris \textit{et al.} (2013), the presence of infill provides a local as well as global increase in strength and stiffness depending on their extent and their position in the frames, affecting the distribution and intensity of the inertia forces generated in seismic excitation. This may initiate stress concentrations in certain regions of structures, causing localized concrete cracking, steel stresses as well as unexpected brittle failures detrimental to overall performance of MIRC frames. Hence, it is essential for professional structural engineers to understand the effect of local masonry infill on the seismic performance of MIRC buildings.

In this context, the present research aimed to determine the material properties of locally available masonry components, i.e. clay brick units and mortars in Applied Element Method (AEM) and evaluation of seismic performances of existing brick infilled RC framed buildings after validation of the determined masonry properties by comparing the analytical results for masonry prisms under uniaxial compression loads and brick infilled RC frame under in-plane cyclic loads with the corresponding experimental results (Zerin and Amanat 2015). The study presented in this paper is a continuation of the past studies (Zerin and Amanat 2015; Zerin \textit{et al.} 2015, 2016a, 2016b) carried out by the authors earlier.

2. Literature review

2.1 Seismic performances of infilled frames
Seismic performance evaluation of MIRC frames by means of numerical analysis requires utilizing reliable laboratory test results in order to formulate and verify the respective constitutive models denoting the material behavior and response (Asteris \textit{et al.} 2013). Over the last five decades, a large number of laboratory investigations (Dhanasekhar and Page 1986; Holmes 1963; Smith 1962, 1966) have been carried out to identify the effect of infill on overall response of RC frames. Researches from
1950s to 1960s on masonry infill had revealed that the walls could be modeled as a strut formulation (Holmes 1961, 1963; Stafford Smith 1962, 1966). The assumed ‘diagonal strut’ bears a part of applied seismic loads and transmit them to other regions of the structure providing relief to certain structural elements of the RC frames (Asteris 2003). Although this redistribution increases overall stiffness and load carrying capacity of the frame, it may develop stress concentrations in specific areas of joints, beams and columns which may result in localized cracking and even unpredictable failures (Asteris et al. 2013).

2.2 Material properties of masonry wall

2.2.1 Compressive behavior of masonry

Masonry is characterized as an anisotropic and inhomogeneous material composed of two materials of somewhat different properties: stiffer bricks and relatively soft mortar distributed at regular intervals. In this context, Hossain et al. (1997) conducted non-conventional experimental investigations of burnt clay brick masonry samples locally made in Bangladesh. The experimental results presented that burnt bricks exhibited brittle behavior approximately up to failure, whereas mortar represented non-linear stress-strain characteristics with relatively large deformations under compression. According to several compression tests of masonry prisms (McNary and Abraham 1985; Atkinson and Noland 1983), in the case of stronger and stiffer bricks with relatively softer mortar composition, the mortar in the bed joint exhibits a tendency to expand laterally more than the bricks because of its lower stiffness. Then, the mortar is confined laterally at the brick-mortar interface by the surrounding bricks. Consequently, the failure of masonry prisms initiates with lateral tensile splitting of bricks and splitting strengths of bricks, revealing that mortar does not initiate failure of the prism due to the lateral confinement provided by the bricks. Consequently, the failure of masonry prisms initiates with lateral tensile splitting of bricks induced by the mortar. According to Hendry (1990), the wall strength can be predicted from mortar and brick strengths using the following equation:

\[ f_{mw} = 1.242 \cdot f_b^{0.531} \cdot f_m^{0.208} \]  

where, \( f_{mw} \) = wall strength [N/mm²], \( f_b \) = strength of masonry unit [N/mm²] and \( f_m \) = strength of mortar [N/mm²].

2.2.3 Elastic modulus of bricks, mortars and masonry

Micro and macro modeling of brick infill requires inputting the material properties and constitutive relationships of masonry constituents. Kaushik et al. (2007) suggested simple relations for computation of the elastic modulus of bricks, \( E_b \), mortar, \( E_m \) and masonry, \( E_{mw} \) from their respective compressive strengths, \( f_b, f_m \) and \( f_{mw} \) based on their experimental results and analyses, as follows:

\[ E_b = 300 f_b \]  
\[ E_m = 200 f_m \]  
\[ E_{mw} = 550 f_{mw} \]

where all units are in MPa.

3. Applied element method

In the present study, a non-linear structural analysis software ‘Extreme Loading for Structure (ELS)’ based on Applied Element Method (AEM) (Applied Science International 2010) has been utilized for numerical modeling and analysis of MIRC specimen frames and MIRC full scale building structures. AEM proposed by Meguro and Tagel-Din (2002) is a stiffness-based numerical method that follows the concept of discrete cracking where the structures are modelled as an assembly of relatively small elements created by dividing up the structure virtually, as shown in Fig. 2(a). The elements are connected along their surfaces through a group of springs responsible for transferring normal and shear stresses between adjacent elements. Each spring represents the stresses and deformations of a certain volume of the material (Fig. 2(a)). In AEM the overall stiffness matrix is formulated and the equilibrium equations, including those for stiffness, mass and damping matrices, are solved non-linearly for the structural deformations. The solution for equilibrium equations is an implicit one that adopts a dynamic step-by-step Newmark-beta time integration procedure (Bathe 1995; Chopra 1995). In AEM two adjacent elements may separate from each other if the matrix springs connecting

Fig. 1 State of stresses in a masonry prism subjected to vertical compression. (Mosalam et al. 2009)
Elements can automatically separate, may come into contact again or make contacts with other elements. If two elements make contacts with each other, contact springs are created at the contact points as shown in Fig. 2(b). As stated in Fig. 2(a), non-linear, path-dependent constitutive models are used in AEM. The elastoplastic and fracture model of Maekawa and Okamura (1983) is adopted for concrete in compression. The elasto-plastic and fracture model proposed by Maekawa and Okamura described by stress and strain tensor field was degenerated into one directional field of stress to match the fiber model by Tsuchiya et al. 2001 which was also followed for the one dimensional normal spring in AEM. In shear, the elasto-plastic and fracture model is not used, the shear stress-strain relation used is linear up to cracking (Fig. 2(a)). Further, for concrete in tension, a linear stress–strain relationship is adopted up to cracking, where the stresses are relieved to zero. Since this is a discrete crack approach, the reinforcing bars are modelled as bare bars for the envelope (Okamura and Maekawa 1991), whereas the model of Ristic et al. (1986) is used for the interior cyclic loops. It is to be noted that Poisson’s ratio of elements is not accounted in AEM (Meguro and Tagel-Din 2002).

Moreover, for masonry wall both mortar and bricks are modeled in the same way as concrete and generation of brick springs and brick-mortar interaction springs are included in AEM as in Fig. 2(c) (Karbassi and Nollet 2013).

![Fig. 2(a) Modelling a structure with AEM (Helmy et al. 2012).](image1.png)

![Fig. 2(b) Different types of element contacts in AEM models (Helmy et al. 2012).](image2.png)

![Fig. 2(c) Stiffness of brick spring and brick-mortar interaction spring in AEM. (Karbassi and Nollet 2013)](image3.png)
2013) where $K_b$ and $K_m$ are the stiffness of bricks and mortar respectively, whereas $K_{eq}$ is the equivalent stiffness of brick-mortar interaction springs. According to Karbassi and Nollet (2013), normal stiffness, $K_n$, and shear stiffness, $K_s$, of brick springs are calculated considering the element geometry illustrated in Fig. 2(c) and incorporating elastic modulus of brick, $E_b$, and shear modulus of brick, $G_b$, as follows:

$$K_n = E_b \frac{a \, t}{d}$$
$$K_s = G_b \frac{a \, t}{d} \quad (5)$$

Additionally, for brick-mortar interface springs, equivalent normal stiffness, $K_{seq}$, and equivalent shear stiffness, $K_{seq}$, are obtained considering element geometry (Fig. 2), stiffness moduli of mortar ($E_m$ and $G_m$) from the following equations:

$$\frac{1}{K_{seq}} = (d - t_m) / (E_m \, a \, t) + t_m / (E_m \, a \, t)$$
$$\frac{1}{K_{seq}} = (d - t_m) / (G_m \, a \, t) + t_m / (G_m \, a \, t) \quad (6)$$

Normal and shear springs of brick elements are affected in case of diagonal tension failure of masonry through brick units, whereas bond failure of masonry affects brick-mortar interaction springs (Karbassi and Nollet 2013). It is to be noted that shear-compression behavior is not currently included in the masonry wall model in AEM (Applied Science International, 2010).

AEM is validated for capable of analyzing and visualizing structural behaviors from the elastic stage to non-linear small and large displacement along with geometric and material changes, elements separation and elements collision to total collapse of the structures without user interventions. Hence, in the present study, AEM based software ELS has been chosen as the most appropriate numerical tool for modeling and analysis of masonry prisms, MIRC frame specimens and MIRC full scale building structures under seismic excitation predicting structural responses from initial loading stages, crack initiation, crack propagation to complete collapse in complex 3D structural analysis.

4. Methodology and verification process in the present study

The methodology of the current analytical research followed the three steps as below:

**Step-1:** Determination and verification of the masonry constituent properties considering fine masonry mesh discretization

Considering stiff bricks - soft mortars composition in the brick masonry (Hossain et al. 1997), tri-axial stress conditions within the masonry (McNary and Abrahams 1985) have been explicitly accounted for the AEM masonry prism model. The constituent material (bricks and mortars) properties have been determined from extensive parametric studies for masonry prism models utilizing fine masonry mesh discretization where each brick units (size: 115mm×70mm×45mm) were divided into 64 (4×4×4) elements to ensure propagation of cracks through brick units along with brick-mortar interfaces (Fig. 3(a)). The material level verification of the proposed constituent properties was successfully carried out (Zerin et al. 2015) comparing the corresponding test results under uniaxial compression (Zerin and Amanat 2015).

The component level determination and verification of the masonry constituent properties have also been performed simultaneously along with the masonry prism model by simulating the in-plane cyclic behavior of brick infilled RC frame specimens in which brick units (size: 115mm×70mm×45mm) were divided into 16 (4×4×1) elements as shown in Fig. 3(b) to confirm the propagation of cracks both in bricks and mortars. Comparative hysteresis behaviors and ultimate shear capacities between the bare RC frame and the infilled RC frame were also revealed from the AEM analyses (Zerin et al. 2015).

**Step-2:** Determination and verification of the composite masonry properties for masonry mesh and coarse masonry mesh discretization

Since the fine masonry mesh discretization needs enormous number of elements in AEM models, two kinds of coarser meshing are investigated. In the masonry mesh (Fig. 3(c)), brick units were undivided and considered as single elements (size: 115mm×70mm×45mm), thus the cracks were allowed to propagate only through the brick-mortar interfaces. Hence, the composite masonry properties were proposed to be applied in the brick-mortar interfaces instead of mortar properties to obtain the overall behavior of masonry (Zerin et al. 2016a, 2016b).

Coarse masonry mesh in Fig. 3(d) (element size: 380mm×240mm×120mm) was also proposed to facilitate analyzing full scale MIRC buildings providing feasible element numbers for non-costly analysis. The proposed composite masonry properties both for masonry mesh and coarse masonry mesh were successfully verified through the same experimental results of brick infilled RC frame under in-plane cyclic loading, thus proved to be applicable for full scale models (Zerin et al. 2016a, 2016b).

**Step-3:** Seismic performance evaluation of existing brick infilled RC framed buildings

Time-history dynamic analysis for MIRC framed building models including soft ground story, infills in all floors, without infills in any floor and retrofitted soft story were performed to evaluate their seismic performances and the failure modes. Fine masonry mesh discretization with corresponding constituent material properties were used for the infills in the ground and first floors. Consecutively, masonry and coarse masonry meshes along with their corresponding composite masonry properties were applied in other floors to optimize the total element numbers into allowable analysis limit (Zerin et al. 2016a, 2016b).
5. Determination and verification of the masonry constituent properties for fine masonry mesh

5.1 Determination and material level verification of masonry constituent properties

5.1.1 Laboratory test for brick prisms under uniaxial compression

A total of twelve masonry prism specimens were subjected to monotonically increasing uniaxial compression load to determine the ultimate compressive strength of masonry made of available constituent materials in Bangladesh (Zerin and Amanat 2015). The commonly used size of brick units in Bangladesh is 240mm×115mm×70mm. For stretcher bond masonry prisms, five layers of half-scaled bricks with dimensions of 115mm×70mm×45mm and 1:4 mortars (Portland cement:local river sand) were used. The geometry of the masonry prisms was 230mm×230mm×70mm. The maximum compression loads and failure modes of the masonry prisms were investigated under two different loading conditions: six prisms out of twelve were subjected to a compression load perpendicular to the bed joints. The other six prisms were subjected to a compression load parallel to the bed joints.

5.1.2 Basic considerations in the masonry prism models

Poisson’s effect and tri-axial confining stress conditions in stiff brick-soft mortar composition stated in section 2.2.1 and in Fig. 1 (Mosalam et al. 2009) have not been considered in the constitutive models of masonry in AEM. Therefore, in the present study, some basic considerations along with parametric studies were adopted to determine the constituent material properties to artificially generate the tri-axial stress condition in the masonry considering stiff brick-soft mortar composition. For the masonry prism model, the compressive strengths of bricks and mortars were determined considering the local constituent material properties in general (Table 1) whereas, elastic modulus of bricks (5170 MPa) and that of mortar (2207 MPa) have been derived from Eq. 2 and Eq. 3 (Kaushik et al. 2007). Additionally, shear moduli of bricks and mortars were considered as around 40% of the respective elastic moduli. Thus, the adopted masonry constituent properties reproduced the stiff brick-soft mortar composition in masonry generating tri-axial compression-tension-tension stress condition under uniaxial compression load. Moreover, extensive parametric studies were conducted (by changing tensile strength of bricks and mortars systematically) in masonry prism models as well as in brick infilled RC frame model (described later in Section 5.2) to confirm the

![Fig. 3(a) 2D view of masonry prism model showing brick elements, brick springs and brick-mortar interfaces with mortar springs.](image1)

![Fig. 3(b) Brick infilled RC frame model with fine infill mesh (1 brick=16 elements).](image2)

![Fig. 3(c) Brick infilled RC frame model with masonry mesh (1 brick=1 element).](image3)

![Fig. 3(d) Brick infilled RC frame model with coarse masonry mesh (1 brick=1 element).](image4)

### Table 1 Input constituent masonry properties.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Brick</th>
<th>Mortar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, MPa</td>
<td>17</td>
<td>11</td>
</tr>
<tr>
<td>Tensile Strength, MPa</td>
<td>1.6</td>
<td>1.9</td>
</tr>
<tr>
<td>Elastic Modulus, MPa</td>
<td>5170</td>
<td>2207</td>
</tr>
<tr>
<td>Shear Modulus, MPa</td>
<td>2414</td>
<td>883</td>
</tr>
</tbody>
</table>

...
effect of splitting tensile strength of bricks and bond tensile strength of mortars on stress distribution and crack propagation in masonry and their failure loads. Based on such parametric studies, the bond tensile strength of mortar (at the brick-mortar interface) is intentionally magnified (1.9 MPa) over the tensile splitting strength of bricks (1.6 MPa) considering the failure initiated by the splitting of bricks within the masonry and thus the confining stress conditions are artificially generated in mortars in the analytical masonry models as explained in Fig. 1 in section 2.2.1. Moreover, the brick units in masonry models were divided into 64 (4×4×4) elements to ensure the propagation of cracks through the bricks in addition to mortars. The established masonry constituent properties are illustrated in Table 1.

5.1.3. Verification considering compression load perpendicular to bed joints of masonry prisms
Under uniaxial compression load perpendicular to the bed joints, the test specimens exhibited three basic failure modes, i.e. shear compression failure of masonry, compression failure of mortar and bricks and tensile splitting of bricks as in Figs. 4(a), 4(b) and 4(c). The mean failure load was 131 kN. The test results varied due to inherent flaws existing in bricks, whereas, the analytical model failed at 177 kN load. Since, Poisson’s effect is not included in the AEM model; it exhibited only compression failure of bricks and mortar at interfaces as shown in Fig. 5(b). Alternatively, principal normal strain distributions in Figs. 6(a), 6(b) and 6(c) within the model clearly identified the tri-axial compression-tension-tension stress condition along brick-mortar interface due to different elastic and shear moduli of the constituents, along with artificially magnified bond tensile strength of mortars over the splitting tensile strength of bricks, consequently generating the confining stress condition in mortars as shown in Fig. 1 in Section 2.2.1.

5.1.4 Verification considering compression load parallel to bed joints of masonry prisms
The masonry prism test specimens exhibited tensile

![Fig. 4 Failure mechanism for test specimens under load perpendicular to bed joints (a) Shear-compression failure, (b) Compression failure of bricks and mortars (c) Tensile splitting.](image)

![Fig. 5(a) Analytical model: load perpendicular to bed joint, (b) Principal normal strain in z-direction before ultimate compression failure under 11.1 MPa vertical compression.](image)

![Fig. 6(a) Principal normal strain in vertical (z) direction before compression failure under 11.1 MPa vertical compression.](image)

![Fig. 6(b) Principal normal strain representing tensile strain along brick surfaces in x-direction induced by soft mortars.](image)

![Fig. 6(c) Principal normal strain representing tensile strain along brick surfaces in y-direction induced by soft mortars.](image)
splitting of bricks along the shorter geometry direction as in Fig. 7(a) and 7(b) under average uniaxial compression load of 173 kN applied parallel to the bed joints. In addition, minor cracks along brick-mortar interfaces were observed as (Fig. 7c) due to uneven deformation characteristics of the constituent materials. The AEM model failed at 174 kN load with vertical tensile splitting of bricks (Fig. 8(b)). Simultaneously, principal stresses and global strain distribution (Figs. 9(a) and 9(b)) within the masonry model clearly identified the stress trajectory after splitting of units. However, brick-mortar interface cracks and tensile splitting along the shorter direction did not occur in analysis as Poisson’s effect is not currently included in AEM.

5.1.5 Maximum compressive strength of brick prism

Table 2 compares the highest compressive strength of masonry prisms obtained from the laboratory investigation, AEM analysis and the Hendry (1990) equation (Eq.1). Input constituent properties depicted in Table 1 were utilized to estimate the masonry wall strength from Eq.1 which seems to have good alignment with both the test and the analytical results.

5.2 Component level verification for masonry constituent properties

5.2.1 AEM structural models

Analytical models for a single-bay single-story bare RC frame and a locally available brick-infilled RC frame were developed to evaluate their seismic performances and failure modes. Laboratory test results of bare RC frames and brick infilled RC frames under in-plane increasing reversed cyclic loads obtained by Zerin and

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**Table 2: Highest compressive strength of brick prisms.**

<table>
<thead>
<tr>
<th>Description</th>
<th>Compression Loading Types</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Perpendicular to Bed Joint</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>Parallel to Bed Joint</td>
<td>10.8</td>
</tr>
<tr>
<td>AEM Analysis</td>
<td>Perpendicular to Bed Joint</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td>Parallel to Bed Joint</td>
<td>10.9</td>
</tr>
<tr>
<td>Hendry Equation (1990)</td>
<td>Wall high strength</td>
<td>9.3</td>
</tr>
</tbody>
</table>

*M=Mean, **Sd= Standard Deviation

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**Table 3(a): Average material properties for RC frame.**

<table>
<thead>
<tr>
<th>Properties (MPa)</th>
<th>Concrete</th>
<th>Normal Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>32</td>
<td>-</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>1.9</td>
<td>-</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>26897</td>
<td>200000</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>11034</td>
<td>80000</td>
</tr>
<tr>
<td>Tensile yield strength</td>
<td>-</td>
<td>388</td>
</tr>
<tr>
<td>Ultimate strength</td>
<td>-</td>
<td>537</td>
</tr>
</tbody>
</table>

**Table 3(b): Material parameters in AEM models.**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Brick</th>
<th>Mortar</th>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Separation Strain</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
<td>0.30</td>
</tr>
<tr>
<td>Friction Coefficient</td>
<td>0.60</td>
<td>0.65</td>
<td>0.65</td>
<td>0.78</td>
</tr>
<tr>
<td>Specific Weight (kN/m³)</td>
<td>18.85</td>
<td>18.85</td>
<td>18.85</td>
<td>76.884</td>
</tr>
<tr>
<td>Ultimate Strain</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.1</td>
</tr>
<tr>
<td>Ultimate Strength/</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.29</td>
</tr>
<tr>
<td>Tensile yield stress</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.01</td>
</tr>
<tr>
<td>Post yield stiffness ratio</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.01</td>
</tr>
</tbody>
</table>

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Fig. 7 Failure mechanism of test specimens under load parallel to bed joints: (a) Tensile splitting and compression failure (b) Shear-compression failure and (c) Bond tensile failure.

Fig. 8 (a) Analytical model: load parallel to bed joint, (b) Tensile splitting in brick units.

Fig. 9 (a) Minor principal stresses at failure, (b) Global strain distribution in transverse direction.
Amanat (2015) were utilized to verify the analytical models. The dimensions along with the reinforcement details incorporated in both the test and analytical models are illustrated in Fig. 10. For laboratory investigation of the infilled frame, half-scaled local clay brick units (size=115mm×70mm×45 mm) were used to construct an infill wall of 1525 mm high, 1525 mm wide and 100 mm thick. The analytical model was checked for mesh sensitivity and each brick unit was divided into 16 elements (4×4×1 elements) to ensure the possibility of crack propagation through bricks. The RC frame and the masonry infill were connected by mortar springs. The average material properties of concrete and steel obtained from the laboratory tests are listed in Table 3(a). Additionally, AEM material parameters adopted in the present study have been summarized in Table 3(b).

The unique combination of constituent masonry properties illustrated in Table 1 were further verified to utilize in the masonry infill wall based on extensive parametric studies regarding overall stiffness, hysteresis behavior, crack propagation, and failure load of the AEM brick infilled RC frame model.

In-plane increasing reverse cyclic loads (load controlled) were applied to the AEM models at the end elements of loading beam, which was the same as in test model (Fig. 11(a)).

There were four pairs of anchor bolts to fix the test frames. Each pair of anchor bolts were prestressed with 178 kN tension forces by a hydraulic jack producing the same amount of compression forces on the specimen bases to fix them with the strong base (Fig. 11(a)). Accordingly, the boundary conditions of the AEM models were kept same as test models where 178 kN compression loads were applied as lump masses on each fixing blocks created on the bases of the infilled and bare RC frame AEM models.

### 5.2.2 Brick infilled RC frame

Hysteretic behavior, failure load, initiation and propagation of cracks through RC frames and infills in the analytical model showed substantially good agreement with the test results (Figs. 11(a), 11(b) and 11(c)). Both the experimental and analytical models exhibited the same maximum shear capacity of 178 kN. Under in-plane reverse cyclic increasing loads, the first cracking initiated in columns due to flexure at 89 kN load in the analytical as well as in the test models. Brick infills experienced first cracking nearly at 133 kN horizontal load in both cases. Later, either of the models failed principally due to diagonal tension cracks in infills along with tensile yielding of longitudinal reinforcement in frames at 178 kN lateral load. Some shear cracks were also generated due to inadequate shear reinforcement in beam-column joints. Diagonal tensile splitting of wall penetrating bricks proved the concept of higher bond tensile strength of mortar due to the confining effect of the surrounding masonry and RC frames both in the test and AEM models.

### 5.2.3 Bare RC frame

The principal normal strain distribution in AEM bare frame model (Fig. 12(b)) represents the crack propagation in the test frame (Fig. 12(a)). Further load-displacement behavior of the AEM model also nearly reproduces the test results (12(c)). Flexural cracks initiated in columns both in the test specimen and analytical model at approximately 9 kN lateral load. As
the load increased, tensile yielding of the steel embedded in the columns and beam was noted along with flexural cracks. Diagonal shear cracks at the beam-column joints due to inadequate shear reinforcement in joints also observed. The loading was stopped at 42 kN for test specimen to prevent total collapse. Whereas, the maximum shear capacity observed for the analytical model was about 50 kN. Alternatively, both the experimental and analytical models for infilled RC frame revealed the maximum shear capacity as 178 kN which established the contribution of brick infills in enhancing the lateral stiffness of the overall frame by about 250% compared to the bare RC frames under in-plane cyclic loading.

5.2.4 Effect of bond tensile strength of mortar
The effect of lower bond tensile strength of mortars regarding fine masonry mesh was evaluated with an additional AEM model. The tensile strength of mortar was considered as 1.1 MPa (10% of mortar’s compressive strength and 58% of its tensile strength in Table 1), while the other constituent properties were kept unchanged from those of the previous model, which are listed in Table 1 and Table 3. Relatively lower bond tensile strength of mortars in the AEM model initiated early tensile cracks in masonry propagated through bed and head joints (Fig. 13(a)) exhibiting sudden energy dissipation in hysteresis behavior (Fig. 13(b)) unlike in the test specimen which justifies the use of higher bond tensile strength of mortar in AEM brick infilled RC framed models for fine masonry mesh discretization.

6. Determination and verification of composite masonry properties for masonry mesh and coarse masonry mesh
Since fine masonry mesh produces too large number of elements and costly calculations, coarse meshing with composite masonry properties has been proposed to keep the element numbers into the allowable analysis limit. In the masonry mesh, brick units were undivided and thus the cracks were allowed to propagate only through the brick-mortar interfaces. The composite masonry properties were applied at the brick-mortar interfaces instead of mortar properties to obtain the masonry composite behavior. The compressive strength of the composite masonry was calculated according to the Hendry (1990) equation (Eq.1, Section 2.2.2) from compressive strengths of bricks and mortars determined for fine masonry mesh. The elastic modulus of the masonry has been derived from Eq.4 in section 2.2.3. The tensile strength of the masonry composite (0.97 MPa) was determined from extensive parametric study. The obtained analytical results utilizing the proposed composite masonry properties for masonry mesh (Table 4) were compared with the experimental results of the same brick infilled RC frame (Zerin and Amanat 2015). Although the initial and unloading stiffness of the model differed from the test results, the crack propagation and the maximum shear capacity obtained from the AEM model illustrated in Figs. 14(a) and 14(b) seemed to represent the test results approximately. Further, coarse masonry mesh (element size: 380mmx250mmx120mm) has also been proposed.

Fig. 12 Cyclic behavior of bare RC frames: (a) Crack patterns in test specimen (b) Principal normal strain in AEM model under (-)44 kN lateral load, (c) Comparative hysteresis behaviors for test and model bare frames.

Fig. 13(a) Effect of lower bond tensile strength of mortar on crack propagation in infilled RC frame model considering fine mesh for infills.

Fig. 13(b) Effect of lower bond tensile strength of mortar on hysteresis behavior of infilled RC framed model.
for analyzing full scale MIRC buildings to keep the element numbers into analysis limit. In the coarse masonry mesh model, tensile strength of composite masonry has been determined from parametric studies while other properties were kept constant as masonry mesh model (Table 4). The maximum shear strength and displacement of the analytical model was also verified with the test results of the same infilled RC test frame (Figs. 15(a) and 15(b)). Thus the proposed composite masonry properties for masonry mesh and coarse masonry mesh are established for the full scale masonry infilled RC frame building models.

7. Seismic performance evaluation of mirc framed building

7.1 MIRC building under study
The building under study is a typical beam-column 8-storied RC frame structure with open ground floor and infilled with masonry wall in upper floors which is commonly constructed in the big cities in Bangladesh. The building was designed as the ordinary moment resisting frame according to BNBC 2006 (HBRI, 2006).

<table>
<thead>
<tr>
<th>Properties (unit: MPa)</th>
<th>Fine masonry mesh</th>
<th>Masonry mesh</th>
<th>Coarse masonry mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick</td>
<td>17</td>
<td>9.3</td>
<td>9.3</td>
</tr>
<tr>
<td>Mortar</td>
<td>11</td>
<td>1.9</td>
<td>0.97</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>1.6</td>
<td>0.97</td>
<td>0.38</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>5170</td>
<td>5106</td>
<td>5106</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>2414</td>
<td>2042</td>
<td>2042</td>
</tr>
</tbody>
</table>

7.2 AEM structural models
The structural models studied are illustrated in Fig. 16 and demonstrated in Table 5. All of the five models possessed the same building geometry in plan, structural design and reinforcement details according to the existing building under study. In all cases, the foundation is assumed fixed to the ground. Determined and verified material properties in Table 3 (Section 5.2.1) and Table 4 (Section 6) have been applied. The normalized PGAs as depicted in Table 6 were applied from 6 to 18 seconds duration of JMA Kobe 1995 ground motion (Fig. 17) to optimize the analyses time. Table 7 summarizes the mesh discretization corresponding to different stories, different building components and the reinforcement ratio of the structural members of the models.

7.4 Analytical results and discussions
Table 8 summarizes the overall seismic performances of the models. Moreover, failure mechanisms of different models have been illustrated in Fig. 18.
Table 5 AEM Models.

<table>
<thead>
<tr>
<th>AEM Models</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. RC frame (WT*)</td>
<td>RC frame building with overhead water tank and without brick wall (wall load is included in the unit weight of concrete beams)</td>
</tr>
<tr>
<td>b. RC frame (no WT)</td>
<td>RC frame building without water tank and brick walls (wall load is included in the unit weight of concrete beams)</td>
</tr>
<tr>
<td>c. Soft story</td>
<td>MIRC frame building with open ground floor (with overhead water tank)</td>
</tr>
<tr>
<td>d. Infilled</td>
<td>MIRC frame building (infilled with brick partition walls in all floors without overhead water tank)</td>
</tr>
<tr>
<td>e. Retrofitted</td>
<td>MIRC framed open ground floor building where ground floor columns were retrofitted with 9.5 mm thick steel plate jacketing. In AEM model, the steel plates are modeled as solid steel sections assumed fully bonded to the RC columns, which in practice could be carried out by both adhesive materials and bolts. Longitudinal reinforcements from connecting beams are extended into the column concrete penetrating the steel plates (Fig16(f)) which is quite inconvenient for practical retrofitting. Hence, there is a future scope for analyzing different retrofitting techniques suitable for practical applications.</td>
</tr>
</tbody>
</table>

*Water Tank

Table 6 Input ground motions.

<table>
<thead>
<tr>
<th>Models</th>
<th>Applied PGA (g) of Kobe Ground Motion</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. RC frame (WT)</td>
<td>0.20; 0.28; 0.36; 0.50</td>
</tr>
<tr>
<td>b. RC frame (no WT)</td>
<td>0.20; 0.30; 0.36;</td>
</tr>
<tr>
<td>c. Soft story</td>
<td>0.20; 0.25; 0.28</td>
</tr>
<tr>
<td>d. Infilled</td>
<td>0.20; 0.30; 0.36;</td>
</tr>
<tr>
<td>e. Retrofitted</td>
<td>0.30; 0.36;</td>
</tr>
</tbody>
</table>

(a) RC frame  (b) RC frame (no WT)  (c) Soft story  (d) Infilled  (e) Retrofitted

(f) Details of retrofitting for ground floor (first story)column in Retrofitted model

Fig. 16 AEM building models.
Table 7 Steel ratio ($\rho$) and mesh discretization for structural and building components.

<table>
<thead>
<tr>
<th>Structural/building components</th>
<th>Element number</th>
<th>Spring Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns ($\rho_{\text{longitudinal}}=3-4%$, $\rho_{\text{transverse}}=0.3-0.6%$)</td>
<td>6×3×4 per meter</td>
<td>Concrete and Steel</td>
</tr>
<tr>
<td>Beams ($\rho_{\text{longitudinal}}=1.5%$, $\rho_{\text{transverse}}=0.4-0.8%$)</td>
<td>7×1×5 per meter</td>
<td>Concrete and Steel</td>
</tr>
<tr>
<td>Slabs ($\rho=0.6%$)</td>
<td>1×1×1 per m²</td>
<td>Concrete and Steel</td>
</tr>
<tr>
<td>Masonry</td>
<td>Fine Brick Mesh (ground-1st floor)</td>
<td>1 brick = 16 elements (brick size: 240×120×70 mm)</td>
</tr>
<tr>
<td></td>
<td>Masonry Mesh (2nd-3rd floor)</td>
<td>1 brick = 1 elements (brick size: 240×120×70 mm)</td>
</tr>
<tr>
<td></td>
<td>Coarse Masonry Mesh (4th-7th floor, WT)</td>
<td>1 brick = 1 elements (brick size: 380×240×120 mm)</td>
</tr>
</tbody>
</table>

Table 8 Overall seismic performances of the MIRC building models.

<table>
<thead>
<tr>
<th>Models</th>
<th>Applied Scaled Down Kobe Earthquake PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.2g</td>
</tr>
<tr>
<td>a. RC frame(WT)</td>
<td>Sustained</td>
</tr>
<tr>
<td>b. RC frame(no WT)</td>
<td>Sustained</td>
</tr>
<tr>
<td>c. Soft story</td>
<td>Sustained</td>
</tr>
<tr>
<td>d. Infilled</td>
<td>Sustained</td>
</tr>
<tr>
<td>e. Retrofitted</td>
<td>Sustained</td>
</tr>
</tbody>
</table>

Fig. 17 JMA Kobe 1995 ground motion (0.8g PGA).

Fig. 18 Failure mechanism of different AEM models.
7.4.1 Effect of the soft story on the seismic performances MIRC building
The analytical results revealed that soft story model sustained with partial column damages up to 0.25g PGA ground motion, whereas, it initiated failure at ground floor columns under 0.28g PGA due to huge shear and moment development caused by vertical stiffness irregularity between ground and upper floors having differential infill distributions and consequent collapses were observed both at 0.28g and 0.36g PGA (Figs. 19(a), 19(b), 19(c) and 19(d)). Figure 20(a) illustrates the top acceleration at Y-direction for soft story model respect-
tively for different PGA. Figure 20(b) reveals the excessive top displacement of the model due to collapses under 0.28g and 0.36g PGA. Figure 20(c) and 20(d) illustrate that soft story model experiences 170% higher base shear and nearly two times less top acceleration than the bare RC frame with overhead water tank model which experienced partial column damages at 0.36g PGA.

7.4.2 Seismic performances of the MIRC building infilled in all floors
The infilled frame model sustained with column and infill damages in ground floor at 0.30g PGA while it collapsed at 0.36g PGA (Figs. 21(a) and 21(b)). Figs. 22(a) and 22(b) illustrate the comparative top acceleration and top displacement of infilled model in Y-direction.

In the failure PGA, infilled model exhibits maximum top acceleration as around 600 gal while corresponding RC frame (no WT) model showed nearly 6000 gal maximum acceleration (Figs. 22(d)). The infilled model experienced maximum base shear (4200 kN in Y direction) which is 300% larger than that of similar RC frame (no WT) model (Fig. 22(c)). This explains that, the existence of infills in RC frame increase the structural rigidity which causes a reduction in its natural period of vibration leading to increased seismic loads to the structure.

7.4.3 Seismic performances of retrofitted model.
Brick infilled RC framed soft story building retrofitted with steel plate (9.5 mm thick) jacketing in ground floor columns sustained seismic loading at 0.3g PGA (Fig.
23(a)). The retrofitted model exhibited first floor column failures under 0.36g PGA due to the vertical stiffness irregularity further developed between retrofitted ground floor and un-retrofitted first floor columns (Fig. 23(b)). Fig. 24(a) confirms the tremendous increment in top acceleration towards z-direction and Fig. 24(b) reveals consequent top displacements at the event of collapse under 0.36g PGA. The Retrofitted model exhibited maximum base shear as 4400 kN (Fig. 24(c)) which is 180% of that of un-retrofitted soft story model. Moreover, it exhibits top acceleration 20% higher that Softy Story model at failure PGA (0.36g) in Fig. 24(d).

7.4.4 Effect of overhead water tank on seismic performances of RC framed buildings
RC frame with water tank sustained 0.36g PGA with partial column damages, while it showed total collapse under 0.50 PGA initiating failure at 5th floor (Fig.s 25(a) and 25(b)). On the other hand, RC frame without water tank collapsed at 0.36g PGA due to total failures of ground floor columns (Fig. 25(c)). Moreover, Figs. 26(a) and 26(b) reveal that RC frame with overhead water tank sustained 0.36g PGA due to its increased time period induced by the water mass at the top while RC frame without water tank collapsed due to alteration in time period and natural frequency of the frame affecting its dynamic behavior.

8. Conclusions
The findings are summarized according to the three steps of the present research as follows.

Step-1: Determination and verification of the masonry constituent properties considering fine masonry mesh
(a) Material and component level verifications of the proposed masonry constituent properties for fine ma-

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![Fig. 23(a) Retrofitted Model sustained with infill damages in 1st and 2nd floor under 0.3g PGA.](image)

![Fig. 23(b) Retrofitted Model collapsed at 0.36g PGA.](image)

![Fig. 24(a) Top accelerations (Z-axis) of retrofitted model under different PGA.](image)

![Fig. 24(b) Top displacement of retrofitted model under different PGA.](image)

![Fig. 24(c) Base shear (Y-axis) variation between Soft Story and retrofitted model at 0.36g PGA.](image)

![Fig. 24(d) Variation of top accelerations (Y-axis) between Soft Story and Retrofitted model under at 0.36g PGA.](image)
Masonry mesh were successfully accomplished utilizing experimental results of masonry prisms under uniaxial compression (in terms of failure load and crack patterns) and the brick infilled RC frame under in-plane cyclic load (in terms of strength and hysteretic deformation characteristics) respectively. Thus, the assumption of higher bond tensile strength at the brick-mortar interfaces due to the confining effect of masonry in stiff brick-soft mortar composition has been successfully verified by AEM fine masonry mesh models.

(b) Experimental and analytical investigation of bare and masonry infilled RC frame applying masonry constituent properties confirms the contribution of brick infills in enhancing the lateral stiffness of the overall frame by about 250% compared to the bare RC frame under in-plane cyclic loading.

Step-2: Determination and verification of the composite masonry properties for masonry mesh and coarse masonry mesh

Masonry composite properties estimated from the constituent masonry properties for fine masonry mesh utilizing Hendry equation (1990) were applied in AEM masonry mesh and coarse masonry mesh models at the brick-mortar interface instead of mere mortar properties and consequently, component level verification was performed for the same experimental results for brick infilled RC frame under in-plane cyclic load.

Step-3: Seismic performance evaluation of existing brick infilled RC framed buildings

(a) Established masonry constituent and masonry composite properties in Step-1 and Step-2 were applied to the MIRC full scale AEM building models according to their mesh discretization along the story heights to evaluate their seismic performances by time history dynamic analysis.

(b) Time history dynamic analysis confirmed the column failure of soft story MIRC model at 0.28g PGA. Whereas, the similar RC frame model sustained up to 0.36 g PGA with partial damages revealing that conventional design method ignoring stress concentration induced by infills in RC frames overestimates the seismic performance of soft story buildings.

(c) MIRC model with infills in all floors sustained 0.30g PGA while failed at 0.36g, similar to the bare RC framed building model neglecting stiffness contribution of infills. Based on the limited case investigations in hand, it appears that existence of infills is not capable of improving the overall performance of surrounding RC frames under seismic load, despite under in-plane cyclic loading infills enhance the cyclic behavior and base shear capacity of the RC frame. This explains that, the existence of infills in RC frame increases structural rigidity by enhancing lateral stiffness which causes a reduction in its natural period of vibration leading to increased seismic demands to the structure.

(d) Steel plate jacketing of ground floor columns enhanced the seismic performance of the soft story model up to 0.30g PGA, whereas, it collapsed at 0.36g with initiation of first floor column failures due to the vertical stiffness irregularity further developed between the ret-
rofitted ground floor and the un-retrofitted upper floors.

(e) RC frame with overhead water tank sustained 0.36g PGA, a higher seismic action compared to RC frame without water tanks, due to its increased time period induced by the water mass at the top.

References


