FINITE ELEMENT MODEL OF MASONRY-INFILLED RC FRAME

Christiana A. FILIPPOU¹, Nicholas C. KYRIAKIDES², Christis Z. CHrysostomou³, Elpida S. GEORGIOU⁴

ABSTRACT

This paper presents a numerical study of the behavior of existing masonry-infilled reinforced concrete (RC) frame that was studied experimentally at the University of Patra (Greece) for a PhD study. It is a 2:3 scale three-storey structure with non-seismic design and detailing, subjected to in-plane cyclic loading. Two infilled frames were designed and built with and without strengthening material Textile Reinforced Mortar (TRM). This experimental case study was simulated and analyzed in DIANA finite element analysis (FEA) software.

In DIANA FEA software, a 2D masonry-infilled RC frame was simulated and a structural linear, eigenvalue and nonlinear cyclic analysis were performed to simulate the experimental results. The objective of this study was to identify suitable numerical constitutive models of each component of the structural system in order to create a numerical tool to represent the masonry infill’s in-plane behaviour in DIANA FEA.

The model results were compared and represented an agreed correlation to the experimental ones through a static nonlinear cyclic analysis. The capabilities of FE models that simulated the experimental nonlinear cyclic behavior of the tested masonry-infilled RC frame are illustrated. Finally, conclusions are drawn regarding the effectiveness of the analytical model to capture the behavior of the proposed structural system.

Keywords: Cyclic loading; Masonry; Reinforced concrete; Nonlinear cyclic analysis; Constitutive model

1. INTRODUCTION

Masonry-infilled RC frame structures are a construction typology widely dispersed around the world. The response of masonry-infilled RC frame structures during an earthquake has attracted major attention since the early 1950s. Past studies have shown that the in-plane strength and stiffness of the infill walls have a significant influence on the global behavior of the structure, subjected to seismic loads. The existence of infill walls in a RC frame can increase the structural strength, stiffness (relative to a bare frame) (Kappos and Ellul 2000 and Stavridis et al. 2012) and lateral capacity of the building (Mehrabi et al. 1996, Fardis and Panagiotakos 1997 and Decanini et al. 2004) and at the same time it can introduce brittle shear failure mechanisms associated with the wall failure and wall-frame interaction. However, the infill walls provide most of the earthquake resistance and prevent collapse of relatively flexible and weak RC structures (Fardis 2000). Fardis (2000) and Kappos and Ellul (2000) presented a very useful global picture of the seismic performance of the studied masonry-infilled RC frames by referring to the energy dissipated by each component of the structural system as a function of the considered earthquake intensity. They revealed that at the serviceability level of over 95% of the energy dissipation takes place in the infill walls.

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The failure mechanism and load resistance of a masonry-infilled RC frame depends on the strength and stiffness of an infill with respect to those of the bounding frame. It is known that masonry structures are vulnerable to both in-plane and out-of-plane movements under the action of lateral loads. Five in-plane failure mechanisms of masonry-infilled RC frames are typically identified according to ATC 43 (1998), Asteris et al. (2011) and Shing and Mehrabi (2002). The infill wall itself fails in a variety of modes, most often involving some combination of bed joint sliding, corner crushing, diagonal cracking and diagonal compression. The exact mode of failure depends upon material properties, such as compressive strength, shear strength and coefficient of friction, geometric constraints, such as frame-wall interface gaps or window openings and other characteristics, such as workmanship. Figure 1 shows the possible five in-plane failure mechanisms of masonry-infilled RC frames.

![Figure 1. Failure mechanisms of masonry-infilled RC frame: (a) Corner crushing and diagonal compression failure modes and (b) Diagonal tension, sliding shear and frame failure modes](image)

In this paper, a numerical model has been developed to study the overall response of masonry-infilled RC frame, using nonlinear cyclic analysis, in DIANA FEA software. The main objective of this work is to present a numerical tool to represent the masonry infill’s in-plane behaviour in DIANA using meso-level approach for modeling the infill wall. The in-plane calibration was performed based on the experimental test performed by Koutas et al. (2014) at his PhD study at University of Patra (Greece).

2. BACKGROUND OF MASONRY INFILL MODELING

In the literature, different modeling techniques that simulate the behaviour of the infill’s panel can be found and are divided into three categories namely micro-modeling, meso-modeling and macro-modeling (Lourenco 1996). In micro-modeling approach, the masonry panel is divided into numerous elements considering the local effects in detail. On the other hand, macro-models are simplified models based on the physical understanding of the behaviour of the infill’s panel. In these models, masonry infill panel is replaced by equivalent strut member along the loading direction. For large structures, it is more reasonable to use meso-model which is between micro and macro-modeling approach. This study employs meso-modeling approach to model the masonry infill. Figure 2 presents the three modeling strategies for masonry infill.

![Figure 2. Modeling strategies for masonry infill (Chrysostomou 1991, Crisafulli 2007 and Lourenco 1996).](image)
2.1 Macro- modeling

The most popular method for modeling the masonry infill is based on the concept of replacing the infill with equivalent diagonal strut (Polyakov 1960, Holmes 1961, Stafford-Smith 1962 and Mainstone 1974). Although the fact that the single-strut model constitutes a sufficient tool for the prediction of the overall response, the multi-strut model is superior in precision (Chrysostomou 1991 and Crisafulli and Carr 2007). In all strut models, the nonlinear strut behavior is described by constitutive monotonic or cyclic law (Panagiotakos and Fardis 1996). In the case of multiple strut configurations (Chrysostomou et al. 2002), the assessment of a constitutive monotonic or cyclic law is needed for each strut. The representation of the non-linear cyclic behavior of infill’s masonry with equivalent diagonal strut increases not only the complexity but also the uncertainties of the problem.

2.2 Micro- modeling

The micro-modeling approach considers the effect of the mortar joints as discrete element in the model. According to Lourenço (2002) and Tzmatzis and Asteris (2003), in micro-modeling the brick units and the mortar are represented by continuum elements and the brick units–mortar interaction is represented by different interface element, which leads to accurate results and intensive computational requirement (Asteris et al. 2013). Vertical interfaces can be introduced in the brick middle to reproduce its possible tensile failure. Past studies, (Mehrabi 1997, Lotfi and Shing 1991 and Lourenço and Rots 1997) have investigated the use of a smeared crack approach for modeling the brick and mortar (quasi-brittle materials). Lourenco (1996) developed an interface failure criterion for interface characterized by a tension cut-off, Coulomb friction law and a compression cap model. The infill-frame contact is also modelled with interface element.

2.3 Meso- modeling

The main problem in micro-scale analysis is the significant computational demand that will be required to model large scale structures. Therefore, for large structures, it is more reasonable to use meso-model. For the meso-modeling approach the same element types, material properties and constitutive models are used as with the micro-modeling approach as described before. The exception that occurs in meso-model compared to micro-model is: the mortar is not explicitly modeled and the approach for modeling the mortar-brick interfaces differs as described here. In this approach, bricks are modeled by continuum elements, but the mortar joint and its interface with bricks is modeled together in an interface element. In the meso-modeling approach, the material properties and the constitutive law of the interface elements are modified to incorporate those of the mortar and the brick-mortar joints.

3. DESCRIPTION OF EXPERIMENTAL CASE STUDY

The experimental case study was carried out by Koutas et al. (2014) in which the effectiveness of seismic retrofitting of existing masonry-infilled RC frames with TRM was studied. It is a 2:3 scale three-storey model with non-seismic design and detailing subjected to in-plane cyclic loading. Two infilled frames were designed and built with and without TRM. The scope of this design effort was the representing a full-height internal bay of an existing non-ductile building built in southern Europe in the 1960s. In this section detailed description of the experimental case study regarding the masonry-infilled RC frame without the strengthening material TRM is presented, since the main scope of the article is to present a numerical tool to represent the masonry infill’s in-plane behavior. Further details about the experimental case study can be found in Koutas et al (2014) and (Koutas et al. 2014b).

3.1 The masonry-infilled RC frame geometry

The geometry of the bare frame and the masonry-infilled frame is shown in Figure 3. The test specimens had a total height of 6.0 m and each story was 2.0 m height and between the edges of the column had span of 2.73m. The columns were rectangular cross section 170 × 230 mm (Figure 4(a)) and the beams were T-section (Figure 4(a)). The column longitudinal reinforcement (Figure 4(a)) consisted of Y12 deformed bars and lap-
splined only at the base of the first story (connection to the foundation). The transverse reinforcement for all concrete members consisted of Y6 plain bars with 90° hooks at the ends. The thickness of the cover concrete to stirrups was 10 mm.

![Figure 3](image)

Figure 3. Geometry of the bare and masonry infill RC frame: (a) 3D view of bare frame, (b) 3D view, (c) front view and (d) side view (Koutas et al. 2014) of masonry-infilled RC frame.

The dimension of the infill wall was 2.27x1.67x0.17 m and it had length-to-height aspect ratio 1.36. The infills wall were constructed by perforated, fired clay bricks (185 × 85 × 55 mm). The perforations of the bricks running parallel to the unit’s length, x- direction. The infill wall was composed of two individual wythes separated by a 60-mm gap (Figure 4(b)). Lime mortar was used between the bricks. The thickness of the bed and the head mortar joints was approximately 10 mm. The wall was supported rigidly by the RC foundation beam plate with dimensions 0.4 x 0.9 x 4.0 m at the bottom the wall. The strong foundation beam was fixed to the laboratory floor via 16 prestressing rods to provide specimen full clamping.

![Figure 4](image)

Figure 4. Detailing of study frame: (a) Rectangular section of RC columns and T-shaped RC beams (details of reinforcement) (Koutas et al. 2014) and (b) geometry details of infill wall with perforated fired clay bricks.

### 3.2 Material properties of masonry-infilled RC frame

For the construction of the RC frame, C25/30 class of concrete was used with the compressive strength of concrete on the day of testing was equal to 27.8 MPa for control specimen and 27.2 MPa for retrofitted. The modulus of elasticity of the concrete was 24.1 GPa. The reinforcement that was used is steel reinforcement class of B500C as longitudinal reinforcement in the beams and columns and plain steel stirrups class of S220. The mean value of reinforcing steel yield stress was equal to 270 and 550 MPa for the plain steel stirrups and for the deformed reinforcement, respectively.
The mean compressive strength of the bricks perpendicular to the perforations was 11.3 MPa. The cement:lime:sand proportion in the mortar used to bind the bricks was 1:1:5. The average compressive strength of the mortar was 12.95 MPa and the flexural strength was 2.6 MPa.

Compression tests on masonry wallets with dimensions of 500 × 500 mm and a thickness of 55 mm (equal to the thickness of the masonry units) were performed. The mean value of compressive strength was 5.1 MPa and elastic modulus of the masonry perpendicular to the bed joints was 3.37 GPa for unretrofitted specimen. In addition, diagonal compression tests on masonry wallets were performed to determine the diagonal cracking strength and the shear modulus of the masonry. The mean value of diagonal cracking stress was 0.39 MPa and the shear modulus was 1.38 GPa for unretrofitted specimen.

### 2.3 Experimental campaign

The masonry-infilled RC frame without TRM were subjected to a sequence of quasi-static cycles of a predefined force pattern. A history of imposed cycles of displacements was defined to be applied at the top, while maintaining an inverted-triangular distribution of forces to all three floor levels until failure (in terms of global response) occurred. The displacement history of all stories is shown in Figure 6(a). A total of five cycles were finally applied to the un-retrofitted specimen. A general view of the test setup is shown in figure below (Figure 6(b)). Three servohydraulic actuators were mounted on the specimen, one per story. The strong foundation beam was fixed to the strong laboratory floor via 16 prestressing rods to provide specimen full clamping. Gravity loading of 80 kN per story was considered to represent the fraction of permanent loads concurrent to the lateral loading action.

![Figure 5. (a) History of imposed cycles of displacements for all stories and (b) test set up (Koutas et al. 2014a).](image)

### 2.4 Experimental Results

Free vibration test was performed in RC bare frame and in masonry-infilled RC frame in order to identify the experimental fundamental period of structure in each phase of the construction. In order to perform free vibration test the specimens subjected to a static displacement at the top of the specimen. For all free vibration test the gravity loading of 80 KN per story was not considered. The results of free vibration tests are shown in the Table 1.

<table>
<thead>
<tr>
<th>Dynamic characteristics</th>
<th>Bare frame</th>
<th>Masonry-infilled RC frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fundamental period (second)</td>
<td>0.24</td>
<td>0.062</td>
</tr>
<tr>
<td>Fundamental frequency (Hz)</td>
<td>4.15</td>
<td>16</td>
</tr>
</tbody>
</table>
During early loading in masonry-infilled RC frame two step-type cracks formed at the first story, running in parallel to the diagonal and accompanied by horizontal sliding-type cracking. During the last cycle of loading, the cracking pattern was completed with the formation of two sliding cracks, one at top of the infill (soffit of inclined bricks) and the other slightly lower than mid-height that joined the tips of the step-type cracks of the previous cycle. Figure 6(a) shows the damage in the masonry infill at the first story at the end of the fifth loading cycle. The maximum base shear force was attained during the third cycle of loading: for the two directions of loading a maximum base shear of 264 KN and −252 KN was recorded at corresponding top displacement of 25 mm and −24 mm. Figure 6(b) shows the base shear force vs. displacement at the top storey.

The frame-infill separation occurred at the very early stages of cyclic loading. The interfaces between the columns and the infill exhibited large gap opening. The peak values of the measured gap opening were: 2.0 mm for the first story (at the column-infill panel interface), 1.5 mm for the second story (at the bottom slab-infill panel interface) and 0.7 mm for the third story (at the bottom slab infill panel interface).

4. FINITE ELEMENT MODEL SIMULATION

This study used DIANA FEA software to model the masonry-infilled RC frame. The proposed meso-model for masonry-infilled RC frame was implemented in DIANA FEA using available materials, sections and elements commands. The DIANA FEA was selected for the analysis, since it is one of the most appropriate software for RC structures (Johnson 2006).

4.1 Constitutive law-Material model

In DIANA FEA software there are different available material models to simulate the masonry-infilled RC frame. In this study most of the material properties are taken from experimental case study and other material parameters were taken from the DIANA FEA software manual or from the literature. The numerical results were compared to the experimental results and some parameters were adjusted to achieve reasonable matches to the experiment.

The concrete material model that was chosen is the Total Crack Strain model. The Total Crack Strain based model describes the tensile and compressive behavior of concrete. Beside the definition of basic properties like young modulus, the total crack strain model requires input of the material behavior in tension and compression. The Total Crack Strain concrete model requires only a small number of engineering parameters such as the tensile and compressive strength and the fracture energy as shown in Table 2. The basic concept of the Total Strain crack model is that the stress is evaluated in the directions which are given by the crack directions. In this study the approach which is used is the Rotating crack model (Rots 1991) which is one commonly used approach in which the stress–strain relations are evaluated in the principal directions of the strain vector.

Cyclic performance of RC elements is highly depended on nonlinear response of reinforcing bars under cyclic
loading. The Menegotto-Pinto model is a special plasticity model for the cyclic behavior of steel. The Menegotto-Pinto model is available for embedded reinforcements. The Menegotto-Pinto model consists of a finite stress-strain relationship for branches between two subsequent reversal points and the parameters involved are updated after each load reversal. The model is defined in DIANA FEA with the parameters as shown in Table 3.

The masonry infill wall material model that was chosen, is the Engineering Masonry model which is a smeared failure model and it is a total-strain based continuum model that covers tensile, shear and compression failure modes. The Engineering Masonry model describes the unloading behaviour assuming a linear unloading for compressive stresses with initial elastic stiffness. In addition, a shear failure mechanism based on the standard Coulomb friction failure criterion is included in the model. The Engineering Masonry model requires large number of engineering parameters and most of these parameters were not measured in the experimental case study. These material parameters were taken from the literature. The direct input that is necessary to apply the Engineering Masonry model as implemented in the DIANA FEA, is the Young’s modulus (E) of masonry in x direction (Ex=7GPa) and y direction (Ey=3.37GPa), the shear modulus (Gxy) was taken as 1.38 GPa based on the experiment, the mass density (ρ) was equal to 800 Kg/m³. The new Engineering Masonry model have four options for cracking behaviour for the head joint failure. In this case study the Head joint failure not considered option was selected due to the fact that in case of selected diagonal crack option the results are less realistic with overestimation of capacity and energy absorption. The tensile strength normal to the bed joint (fty) was taken as 0.5 MPa according to Lourenço and Rots (1997b) and Lourenço et al. (1995) and the residual tensile strength was obtained 40% of fty. The tensile strength of the joint was a subject of research and therefore the tensile behavior parameters have been assumed according to the information provided by the respective experimental testing reports or related references. The compressive strength (fc) normal to the bed joint was equal to 5.1MPa, the factor to the strain at maximum compressive stress (n) was equal to 1 and the compressive unloading factor (λ) was 0.2. The compressive fracture energy (Gfc) and the crack energy (Gft) was obtained 4000 N/m and 35 N/m respectively according to the relation proposed by Rots et al. (2017). The cohesion (c) was obtained 1.5 ft according to the relation that was proposed by CUR (1994). The shear fracture energy (GIII) was equal to ten times smaller of the cohesion (0.1 c) as proposed by Lourenço (1996). The friction coefficient (φ) was chosen so that the ratio between the specimen compressive and tensile strengths was about ten, which a ratio is often found for masonry units.

The gap between the frame and masonry infill could significantly influence the overall behavior of the masonry infill RC frame as described elsewhere. An interface cap model, plasticity based and proposed by Lourenço,

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**Table 2: Mechanical properties of total strain model**

<table>
<thead>
<tr>
<th>Total Crack Strain Model</th>
<th>Elastic parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity (E)*</td>
<td>9.1 GPa</td>
</tr>
<tr>
<td>Poison ratio (ν)</td>
<td>0.2</td>
</tr>
<tr>
<td>Mass density (ρ)</td>
<td>2547 kg/m³</td>
</tr>
<tr>
<td>Crack orientation</td>
<td>Rotating</td>
</tr>
<tr>
<td>Crack band width</td>
<td>Rots</td>
</tr>
<tr>
<td>Tensile behavior</td>
<td>CEB-FIP model code 2010</td>
</tr>
<tr>
<td>Tensile strength (f₀)</td>
<td>2.151 MPa</td>
</tr>
<tr>
<td>Fracture energy (G₀) **</td>
<td>130 N/m</td>
</tr>
<tr>
<td>Compressive behavior: Maeakawa concrete model</td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength (fₐ)</td>
<td>27.2 MPa</td>
</tr>
</tbody>
</table>

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**Table 3: Mechanical properties of Menegotto-Pinto model**

<table>
<thead>
<tr>
<th>Menegotto-Pinto Reinforcement Steel Model</th>
<th>Elastic parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity (E)</td>
<td>206 GPa</td>
</tr>
<tr>
<td>Initial yield stress (σ₀)</td>
<td>Longitudinal bar 549MPa Stirrups 295MPa</td>
</tr>
<tr>
<td>Initial tangent slope (b₀)</td>
<td>0.05</td>
</tr>
<tr>
<td>Initial curvature parameter (R₀)</td>
<td>20</td>
</tr>
<tr>
<td>Material constant</td>
<td></td>
</tr>
<tr>
<td>A1</td>
<td>18.5</td>
</tr>
<tr>
<td>A2</td>
<td>0.01</td>
</tr>
<tr>
<td>A3</td>
<td>0.2</td>
</tr>
<tr>
<td>A4</td>
<td>3</td>
</tr>
</tbody>
</table>

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* This model has no ability to reduce the stiffness due to early cracking of the concrete section and therefore the modulus of elasticity was reduced.

** The tensile strength and fracture energy were obtained by the empirical equations according to CEB-FIP (2010)model code. 
Rots (1997c) was used for the interface elements describing the connection between the masonry infill wall and the enclosure RC frame. The model includes a tension cut-off for tensile failure (mode I), a Coulomb friction envelope for shear failure (mode II) and a cap mode for compressive failure. As shown in Figure 7, fracture of the interface is controlled by its tension mode, shear behaviour by Coulomb friction behaviour and crushing by the cap in compression mode.

Table 4: Mechanical properties of interface model

<table>
<thead>
<tr>
<th>Coulomb friction model</th>
<th>Y direction</th>
<th>X direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness(Kn)</td>
<td>6e+13N/m³</td>
<td>6e+13N/m³</td>
</tr>
<tr>
<td>Shear stiffness (Ks)</td>
<td>1e+13N/m³</td>
<td>1e+12N/m³</td>
</tr>
<tr>
<td>Friction angle (φ)</td>
<td>30 degree</td>
<td>30 degree</td>
</tr>
<tr>
<td>Dilatancy (ψ)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Model for gap appearance</td>
<td>Brittle</td>
<td>Brittle</td>
</tr>
</tbody>
</table>

One drawback regarding the use of this interface model is the lack of material properties, as no data were available regarding the behaviour of the interface between the masonry infill and the frame. Therefore, it was decided to define the material properties of the interface by fitting the numerical results to the experimental results obtained for the experimental case study. The interfaces normal modulus and shear modulus are given in Table 4 and were calculated according to CUR (1994).

4.2 Type of elements and mesh

Diana offers a broad range of element types for modeling structures made of brittle and quasi-brittle materials. The concrete frame and masonry infill wall were modelled with plane stress element and especially with eight-node quadrilateral isoperimetric plane stress elements (CQ16M). The steel reinforcement in the frame was modeled with two-node bar elements and they were connected to the eight-node concrete elements at the two external nodes. Figure 8(a) shows both elements.

![Figure 8. Elements used in the model: (a) CQ16 element and (b) Position of nodes of CQ16M and CL12I element.](image)

The nonlinearity was introduced in the interface masonry infill –RC frame zone with 2D line interface element. The interface between an infill wall and the frame was modeled by the 3-point line interface element (CL12I) capable of modeling cohesion, separation and cyclic behavior. The CL12I (Figure 8(b)) element is an interface element between two lines in a two-dimensional configuration.

The squared mesh is preferred in FE models (Ho-Le and An 1988) and therefore in this case study the shape of the 2D elements were kept rectangular with nearly equal sides. The size of the finite elements was about 110 mm for the plane stress element.
4.3 Type of loading and constrains

The model was loaded with a constant axial load on the top of the columns to simulate the dead load and with imposed cyclic horizontal displacement as shown in Figure 6(a). The loading process was done as similar as possible to the experimental case using point prescribed deformation load. All nodes at the bottom of the lower of masonry-infill RC frame were restrained by preventing any translation in the x and y directions to simulate the strong foundation beam that was used in experimental case study. All displacement-loaded nodes were restrained in horizontal direction assuming a perfect bond between loading cap and beam elements in the model.

4.4 Type of analysis and convergence criteria

Three types of analysis were performed Eigen value analysis, Structural linear analysis and nonlinear analysis. To perform nonlinear cyclic analysis two phased analysis is selected. At the first phase the self-weight and the additional dead load of the structure is considering. At the second phase a quasi-static implicit non-linear analysis is performed with secant iteration scheme, taking physical nonlinear (Total Lagrange). Automatic incrementation procedure is used in which both the number of steps and the corresponding step sizes are automatically computed. The energy based convergence criterion is applied with standard DIANA FEA tolerance values (0.0001). The continuation option was activated. For some tests, which produce not a good agreement with the reference experiment, some parameter variations were performed for the engineering masonry model and for interface model.

5. FINITE ELEMENT MODEL RESULTS

In this part the results of three types of analysis Eigen value analysis, Structural linear analysis and cyclic nonlinear analysis are presented. Natural frequencies and mode shapes analyses for the bare frame and for the masonry-infilled RC frame are shown in Figure 9. The fundamental frequency of 4.2 Hz was calculated for the bare frame and 16.01 Hz for the masonry-infilled RC frame. The computed values of the fundamental frequencies are compared and represented an agreed correlation to the experimental ones (Table 1).

![Figure 9. Eigen value analysis results for: (a) bare frame and (b) masonry-infilled RC frame.](image)

The results obtained from masonry-infilled RC frame model, subjected to non-linear analysis, described in Figure 10(a) which shows the base shear–displacement curve. In addition, the base shear in relation to the load step and the top storey displacement vs. load step are presented in Figure 10(b) and Figure 10(c) respectively. Simulation results and experimental data of the masonry-infilled RC frame have been compared and represented an agreed correlation to the experimental ones on the following points:

- Initial stiffness
Figure 10. Model results: (a) Base shear force versus top storey displacement, (b) base shear versus load step and (c) displacement versus load step.

The shear-force capacity for the last cycle of loading is overestimated and the energy absorption in the last two cycles of loading is severely overestimated too. This might be depended on the analysis convergence. Therefore, more test must be performed with variation of some parameter for the engineering masonry model and for the interface model and for the convergence criteria to achieve reasonable matches to the experimental results.

Figure 11(a) shows the main failures that occurred at the first floor in the extreme maximum principal strains at the third and fifth cycles of loading. In addition, a shear failure at the top of the first story east-bound column occurred. From the shear stress distribution (Figure 11(b)) in the numerical model, it is shown that the specific failures were captured. Figure 11(b) shows clear diagonal cracks of the infill as same location as observed in the experiment. The results from the analysis shows a shear failure at the top of the first story east-bound column is the same damage that was observed in the experiment upon test completion.

Figure 11. (a) Damage of infill at the third and fifth cycle of loading in the experiment and (b) shear stress distribution in model with diagonal strut of the infill at the third and fifth cycle of loading and shear failure of the column.
6. CONCLUSION

This paper presents a numerical model that simulates the nonlinear behavior of masonry-infilled RC frame subjected to in-plane actions and the respective application in the computer program DIANA FEA. A simplified procedure to determine the required parameters in order to define the model was presented followed by the validation of the proposed method with available experimental data. Simulation results and experimental data of the masonry-infilled RC frame have been compared and represented an agreed correlation to the experimental ones. The fundamental period of the bare frame and of the masonry-infilled RC frame were well compared with the fundamental period that was obtained from the analysis of the same specimens in experiment. The model results have proven that the stiffness of the masonry infill and therefore the presence of the infill are crucial parameters influencing the fundamental period of RC buildings. It has been observed that the lateral strength of the infill frame is significantly higher compared to bare frame. In general, good agreement of the simulation results with experimental results was observed for the initial stiffness, the ultimate stiffness and for the maximum shear-force capacity in nonlinear cyclic analysis. The crack-patterns show in general good agreement with experiment, with respect to location and orientation of the cracks.

It can be concluded that this model is a reliable model of the behavior of masonry-infilled RC frame, although the energy absorption and maximum shear-force capacity in the last cycle of loading is not very close to reality. In the future, this proposed numerical model which simulates the nonlinear behavior of masonry-infilled RC frame will be used to perform numerical experiments through a parametric study to investigate the influence of the mechanical properties of infill wall in the strength, stiffness and lateral capacity of these structural systems. These results will contribute to the investigation of a general model for the application and the design of masonry infills in existing RC frames. The model will be then further expanded to study the behavior of infilled frames reinforced with TRM.

7. References


