3-D APPLIED ELEMENT METHOD FOR STATIC NON-LINEAR SIMULATION OF PP-BAND RETROFITTED MASONRY

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ABSTRACT: Masonry, through its long history, is widespread used around the world and still remains a main building material in many places especially developing countries. However a poorly designed masonry is known as brittle and susceptible to the earthquake. To improve their seismic capacity, polypropylene band retrofitting technique method was purposed base on economic point of view and local availability of material and skilled labor.   In this study, we proposed 3-D Applied Element Method as analysis tools to help understanding the polypropylene band retrofitted masonry behavior which will be benefit in the future design process.   Unlike the previous version, 3-D Applied Element Method elements can be any rectangular prism which helps reducing the number of elements. Brick and mortar springs are represented by using different spring properties.   Nonlinear constitutive law of the mortar spring employed the Gambarotta model which considers the material softening.   Polypropylene band is modeled as beam element using conventional plastic constitutive law connected together with the masonry by elastic spring representing the polypropylene band to brick connector.   The numerical simulation of non-retrofitted and retrofitted out of plane wallets shows that with the suitable selected parameter the behavior of masonry can be closely reproduced.

Key Words: Applied Element Method, 3-Dimension, Masonry, Retrofit, Polypropylene band

INTRODUCTION

Masonry along with timber structures are among the oldest structures that are still used nowadays. Masonry structures history can be tracked back as early as 8000-9000 B.C. near Lake Hullen, Israel (Lourenço, 1996).   With its long history, abundance of material, ease of construction and the advantages in thermal property, masonry is widespread used around the world.   This type of structure still remains a main building material in many places especially in developing countries (Paola, 2003). Despite its advantages as residential structure, masonry is known as brittle and unsuitable for construction of buildings in seismic zones (Tomaževič, 1999).   The 1997 Umbria-Marche, 1999 Bhuj and 2003 Bam earthquakes shows that masonry is rather susceptible and a huge number of masonry collapses is found especially in the region where the poorly designed masonries are concentrated.   Moreover, masonry collapse also results in high casualty because masonry material tends to break into the small debris and left insufficient void which reduce the chance of survival.

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In order to improve the current situation, the proper retrofitting method must be invented. Several proposed retrofitting method in the literatures includes grout injection and internal reinforcing, ferrocement coatings, FRP composites and adding of steel elements. These methods were reporting successfully for increasing the seismic capacity of the building. Besides, the recent retrofitting scheme considering economic and availability of material and skilled labor is Polypropylene band (referred later as PP-band) retrofitting technique. The method was first proposed by Paola in 2003. Unlike the former methods, a main objective of this retrofitting technique is to hold the disintegrated elements together and preventing the collapse. In attempt to understand the behavior and to rationalize the design of these retrofitting methods, she carried out the PP-retrofitted masonry wall testing. It was found that the behavior of the retrofitted walls was more stable and larger deformations could be sustained compared to unretrofitted one. Moreover, Paola also proposed the 2-D numerical model based on Applied Element Method (AEM) for simulating PP-band retrofitted wall using constitutive laws provided by Lorenço (1996). In 2005, Sathiparan conducted series of shaking table tests of the ¼ scaled single unit masonry models with opening. He found that the retrofitted test models can resist up to 88 mm compared to 7 mm in an unretrofitted case for top displacement without collapse. Later, Guragain (2006) adopted the constitutive laws from Gambarotta (1997) into AEM and succeeded in proposing the model for simulating the in-plane cyclic behavior of PP-band retrofitted masonry.

By considering the damage mechanisms in the masonry structures, the typical damage mechanism in the masonry structure can be identified as 1. cracks between walls and floors 2. cracks at the corners and at wall intersections 3. out-of-plane collapse of perimetral walls 4. cracks in spandrel beams and/or parapets and 5. diagonal cracks in structural walls (Tomaževič, 1999). Except the diagonal cracks, most masonry damage behaviors inevitably relate with three dimensional behaviors. With the proposed PP-band retrofitting, the interaction between each wall will be higher and therefore the seismic resisting mechanism tends to become more important. Therefore, three dimensional behaviors have to be investigated before developing the less complicated two dimension analysis and design process. Moreover, because PP-band stiffness is rather low compared to masonry, the effect of retrofitted PP-band will play more significant role when a structure largely deforms. This is inevitably required the good understanding of three dimensional seismic behavior of masonry structure in the large deformation state. Despite a number of numerical models for structural analysis, few are suitable to simulate masonry in the large deformation range and AEM is one among these. In this study, we proposed the 3-D AEM, based on previous 2-D AEM for masonry and 3-D AEM for concrete, to simulate three dimensional behaviors for PP-band retrofitted masonry structure.

3-DIMENSIONAL APPLIED ELEMENT METHOD

In AEM, the structure is divided in rigid elements, carrying only the system's mass and damping, connected with normal and shear springs representing the material properties (**Figure 1**). The stress and strain fields are calculated from the spring deformations. 3-D AEM rigid elements with 6 degrees of freedom each are connected through sets of one normal and two shear springs.
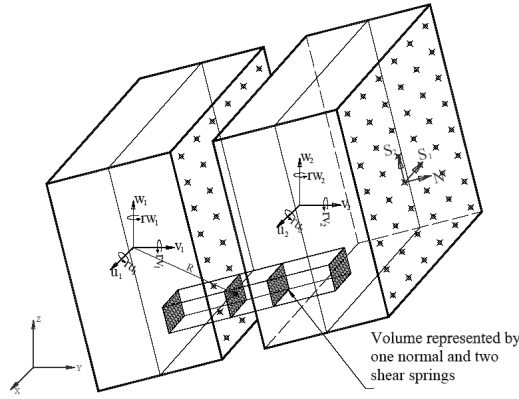
Figure 1. 3-D AEM

3-D AEM used an explicit scheme to solve structural problems therefore it is required to assemble the system stiffness matrix. For this purpose, it is necessary to sum up the contributions of all the springs around one element to the relevant degrees of freedom. Because each element has six degrees of freedom, the stiffness matrix of each spring is a 12 by 12 matrix which its components were generated by direct stiffness method.

Masonry is constituted by two phases: brick and mortar. Therefore, there are two types of springs: one inside brick units, brick spring, and the other at the joint interface, brick-mortar spring, are defined in 3D-AEM. In this paper, the brick spring stiffness is assumed to be elastic and its stiffness can be calculated as:

$$K_n = \frac{E \times b \times c}{a} \quad \text{and} \quad K_s = \frac{G \times b \times c}{a} \tag{1}$$

where E and G are Young's and shear modulus of brick, respectively. Other variables are shown in **Figure 2.** For the brick-mortar springs, an equivalent normal and shear stiffness is calculated by assuming that these springs represent a system of brick and mortar springs arranged in series as shown in **Figure 2**.
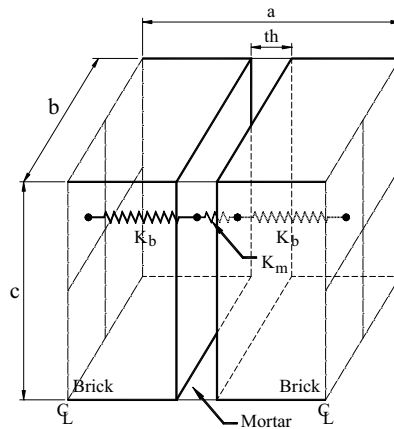

Figure 2. Concept of equivalent brick-mortar spring

The corresponding equivalent stiffness is:

$$\frac{1}{Kn_{eq}} = \frac{a-th}{E_b \times b \times c} + \frac{th}{E_m \times b \times c} \tag{2}$$

$$\frac{1}{K1s_{eq}} = \frac{a-th}{G_b \times b \times c} + \frac{th}{G_m \times b \times c} \tag{3}$$

$$\frac{1}{K2s_{eq}} = \frac{a-th}{G_b \times b \times c} + \frac{th}{G_m \times b \times c} \tag{4}$$

where E_b and G_b are the Young's and shear modulus of brick and E_m and G_m for the mortar. Eigenvalue analysis is also possible with the current version of 3D-AEM. The vector iteration with shifts technique is adopted. This technique is chosen as it provides a practical tool for computing as many pairs of natural vibration frequencies and modes of the structure as desired.

Masonry Modeling
The material constitutive relation requires spring level stress/strain updating for each loading step in the 3-D AEM. The stiffness changes in accordance to damage that material sustained in local level is required to monitor throughout the loading history. As mentioned earlier, the constitutive law needed to be modified in order to take into account this phenomenon. Such a model should be able to reflect the highly nonlinear behavior of masonry with the fewest number of parameters so that it results in a simple and stable numerical model. Considering these criteria, the damage model of brick masonry proposed by Gambarotta et al (1996) has been chosen to implement in the 3D-AEM for cyclic behavior of the masonry. This constitutive law is able to reflect the important physical phenomena exhibited by masonry under cyclic loading.
The chosen constitutive model is based on damage mechanics and takes into account both the mortar damage and brick-mortar de-cohesion which is considered to take place when opening and frictional sliding are activated. Constitutive property of joint springs is based on two damage variables representing frictional sliding and mortar joint damage. Those variables are obtained from Mohr-Coulomb's friction surface and damage condition based on fracture mechanics. The damage evolutions in tensile and compression zones as well as the frictional limit criteria are given in **Figure 3**.
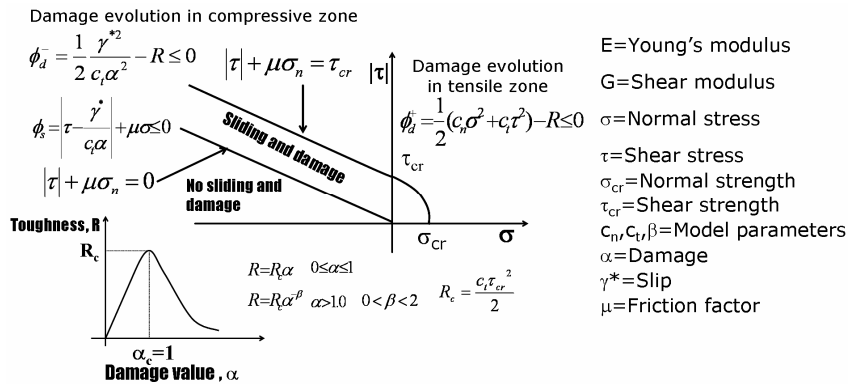

Figure 3. Constitutive relation to model the cyclic behavior of masonry

Stress at any springs can be defined as:

$$\{\sigma\} = [D]\ \{\varepsilon\} \tag{5}$$

Strain in springs can be split into elastic and plastic strain as:

$$\{\varepsilon\} = [K_e]\ \{\sigma\} + \{\varepsilon_p\} \tag{6}$$

where, $[K_e]$ is the elastic compliance matrix of the spring. The plastic strain is $\{\varepsilon_p\} = \{\varepsilon_{np},\ \gamma_p\}^t$ and is the result of inelastic normal and tangential displacement. Strain components are defined as functions of the damage variable α,

$$\varepsilon_{np} = h(\alpha)\ H(\sigma_n)\ \sigma_n \tag{7}$$

$$\gamma_p \quad = k(\alpha)\ (\tau\text{-}f) \tag{8}$$

where $h(\alpha)$ and $k(\alpha)$ are positive functions representing inelastic compliances in normal and tangential direction. $H(\sigma_n)$ is the heaviside function used to make the expression unilateral in tensile straining action and f is the friction developed at the interface between mortar and brick. Evaluation of the variable α and friction f is made through the limiting conditions of damage and friction sliding.

The damage condition based on R-curve approach of fracture mechanics is defined on the basis of energy release rate of damage, Y and toughness, R as:

$$\phi_d = Y\text{-}\ R \leq 0$$
$$Y = (1/2)\ h'(\alpha)\ H(\sigma_n)\ \sigma_n^{\,2} + (1/2)\ k'(\alpha)\ (\tau\text{-}f)^2 \tag{9}$$

and the compliance functions h(α) and k(α) are assumed linear function of inelastic compliance constants as follows:

$$h(\alpha) = c_n\ \alpha$$
$$k(\alpha) = c_t\ \alpha \tag{10}$$

The toughness function, R is related to damage evolution and is defined in terms of damage index α setting to attain its peak value R_c at critical value of $\alpha = \alpha_c (=1)$ and vanishing to zero as α increasing. The friction sliding condition is obtained from Mohr-Coulumb friction condition as:

$$\phi_s = |f| + \mu\ \sigma_n \ \leq \ \ 0 \tag{11}$$

where, μ is friction coefficient and $|f|$ represents the shear component in surplus to cohesion.

Pp-Band Mesh Modeling

The PP-band mesh is modeled through beam elements spanning between band intersections points as shown in **Figure 4**. These ends were then connected to the masonry structure through a set of three springs: normal, shear, and rotational. By appropriately setting the properties of these springs, it is possible to consider all possible connecting conditions between mesh and structure. For instance, if there is a wire connector at that particular location, all three springs have values proportional to the connector properties. On the other hand, if there is no connector and no mortar overlay, the normal spring only works in compression, i.e. when the mesh and the structure are in contact. As for the shear and rotation springs, there values are almost zero. This would not be case if there was mortar overlay.
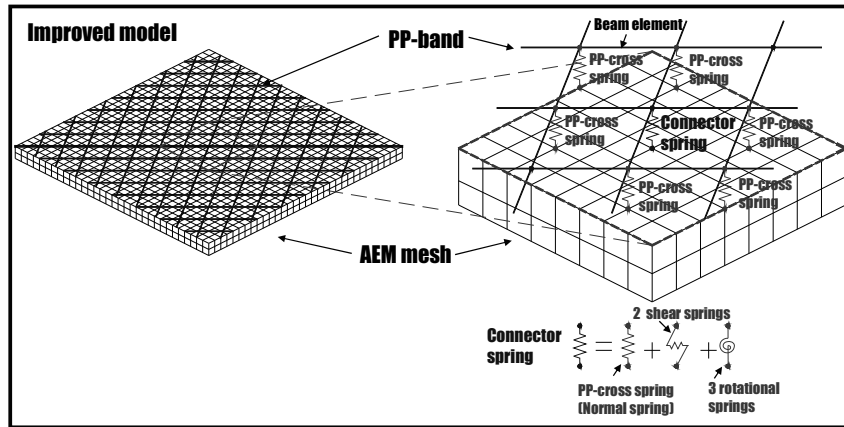
Figure 4. 3D-AEM mesh modeling

The direct implication of a model as the one described above is the considerable increase of degrees of freedom of the system because each intersection point is associated with six more degrees of freedom. A 3-dimensional analysis by itself also involves the solution of systems with large number of degrees of freedom. Therefore, it was absolutely necessary to optimize the algorithms used to solve the equations of motion. This step was successfully implemented.

The material model used for each PP-band beam element was elastic in tension as shown in **Figure 5**. No compression forces were taken by the beam element. The beam elements were defined so as to have almost no moment resistance at their ends.
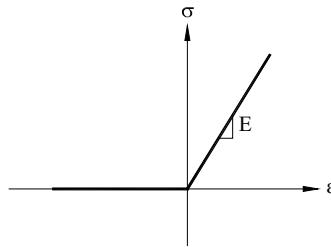
Figure 5. Material model for PP-band beam element

MODEL VERIFICATION

The 3D-AEM model was verified using the experimental data obtained by Sathiparan (2005). The non-retrofitted and retrofitted wallettes, shown in **Figure 6.** and **Figure 7.**, were 475x235x50mm3 and consisted of 6 rows of 6 bricks each. The PP-band mesh was made of 6mm-width, 0.32mm-thick PP-bands placed at 40mm pitch. A total of 6 wire connectors were used to attach the meshes to the wallettes. The wallettes were simply supported by high strength steel rods in both ends. The masonry wallettes were tested under line load using another steel rod of 200mm diameter in the mid span.
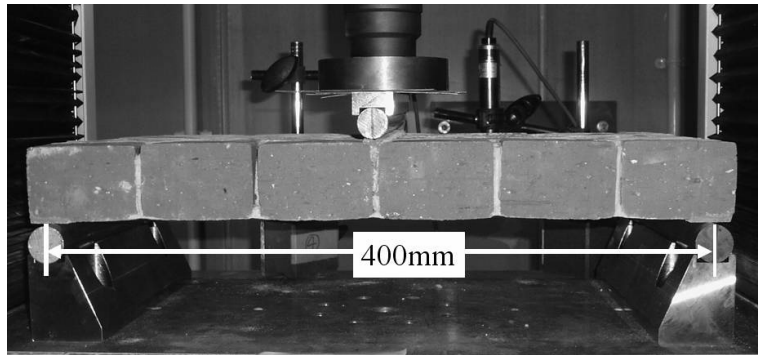
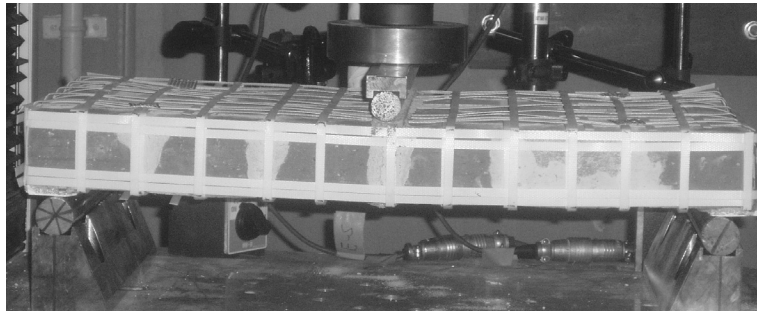**Figure 6.** Retrofitted masonry wallet tested for out of plane

Figure 7. Boundary condition and loading of the masonry wallet for out of plane test

The material properties used for the masonry are summarized in **Table 1**. The PP-band mesh stiffness was set equal to 9.375 MPa.

Table 1 Material properties used for the modeling of out-of-plane masonry wallettes

	Young's modulus E (kN/mm^2)	Shear modulus G (kN/mm^2)	Tensile strength σ_{cr} (kN/mm^2)	Shear strength τ_{cr} (kN/mm^2)	Friction coeff. μ	β	$1/C_{mt}$ (kN/mm^2)
Mortar	0.5	0.25	0.16e^{-3}	0.22e^{-3}	0.6	0.9	1/30
Brick	15.0	7.5	NA	NA	NA	NA	NA

NA: Not applicable

The models used for non-retrofitted and retrofitted models are shown in **Figure** .

(a) Non-retrofitted wallette   (b) Retrofitted wallette

(c) Boundary and loading condition (top dark elements: loading point,
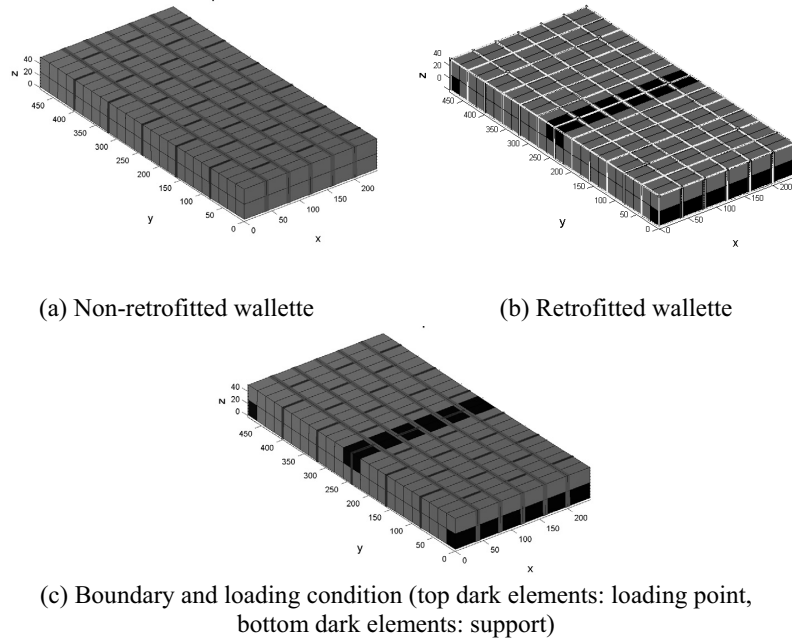bottom dark elements: support)

Figure 8. Boundary condition and loading of the masonry wallette
for out of plane test

Figures 9. and 10. show the comparison of numerical and experimental simulations for non-retrofitted and retrofitted wallettes, respectively. It can be seen that in both cases, the model could accurately capture the force-deformation relationships.
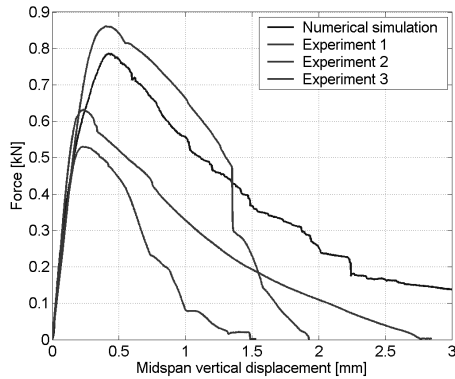
Figure 9. Numerical and experimental force-deformation curves for non-retrofitted wallettes. (Experiments by Sathiparan, 2005)
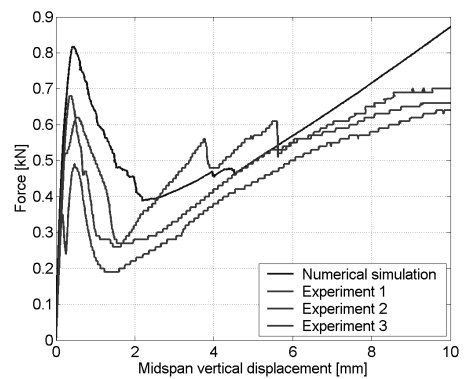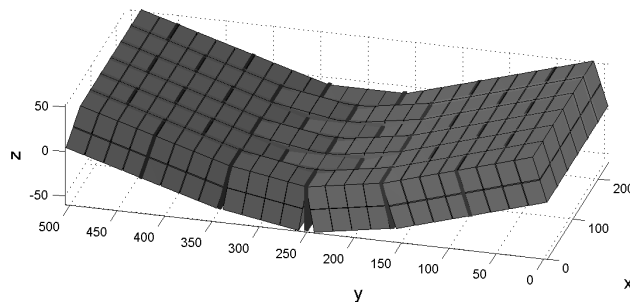

Figure 10. Numerical and experimental force-deformation curves for retrofitted wallettes. (Experiments by Sathiparan, 2005)

The agreement between experiments and numerical simulation can also be observed in the crack patterns and deformed shapes (**Figure 11** and **12**).

(a) Experiment

(b) Numerical simulation (scale factor: 20; midspan vertical displacement=3mm)

Figure 11. Comparison of numerical simulation and experimental deformed shapes for non-retrofitted masonry.

CONCLUSION

The 3-D AEM for simulating static behavior of PP-band retrofitted masonry was developed. The main improvements in this version are the rectangular prism AEM element which helps reducing the element number and the additional beam element and connected spring allowing AEM for simulating PP-band. The verification for 3-D AEM for PP-band retrofitted masonry for the out of plane test was carried out. The verification result shows that with the suitable selected parameter, the behavior of masonry can be closely reproduced.

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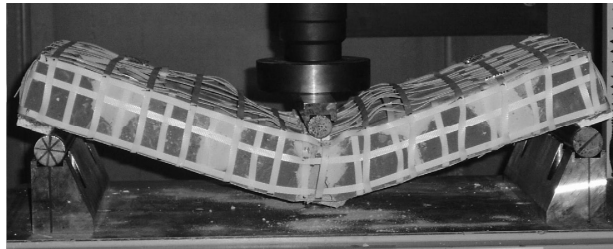
(a) Experiment
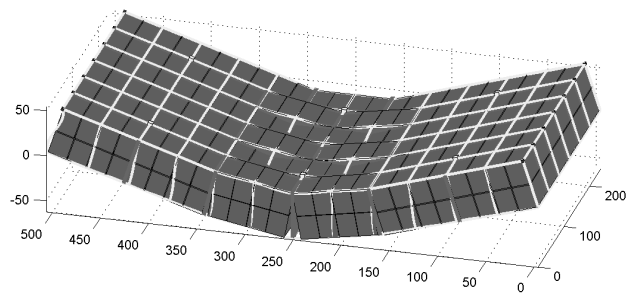
(b) Numerical simulation (scale factor: 20; midspan vertical displacement=3mm)
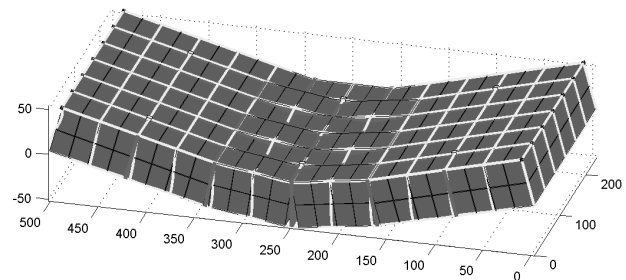
(c) Numerical simulation (scale factor: 5; midspan vertical displacement=10mm)

Figure 12. Comparison of numerical simulation and experimental deformed shapes for non-retrofitted masonry