Seismic assessment and rehabilitation of a historical masonry mosque

Ayman Trad[1,*], *Tarek Sobhieh*[2], *Hassan Ghanem*[1], *and Yehya Temsah*[1]

[1]Beirut Arab University, Faculty of Engineering, Beirut, Lebanon
[2]American University of Beirut, Faculty of Engineering, Beirut, Lebanon

Abstract. In order to assess the structural behaviour and to evaluate the seismic vulnerability of old masonry structures located in Lebanon, a historical masonry mosque was analysed under earthquake loading. A numerical model developed by the finite element method using Abaqus software was elaborated on the basis of previously published experimental studies. It was concluded that the numerical model can predict maximum stresses with reasonable accuracy, allowing control of a full scale wall model. This analysis shows that the stresses generated in the joints between the blocks exceed the ultimate shear stress of the mortar, resulting in cracks in the joints. The choice of an adequate structural rehabilitation method was limited because the mosque is of archaeological importance and its original appearance should not be modified. Therefore, a seismic retrofit solution using internal or external post tensioned tendons was recommended.

1 Introduction

The preservation and restoration of historical and archaeological buildings in Lebanon is becoming an urgent issue that requires the attention of the Lebanese engineering community as a whole. The studied mosque considered in this study, “Al-Muaallak”, is one of the mosques built up by Mahmud ibn-Lutfi, governor of Tripoli during the Ottoman period (Fig. 1). It is located near the south end of Tripoli’s main street of the souks at Al Haddadeen district.

The studied structure was built using limestone masonry units assembled by lime mortar joints. These joints result in weakness points since mortar material breaks the continuity and uniformity of stones. The mechanical characteristics of similar blocks and mortar were previously determined [1]. These properties were used in order to develop and validate a numerical model at the scale of an assembly then at the scale of a wall. A structural analysis of the studied building under gravity and earthquake loads was carried out based on the UBC 97 code [2]. This analysis yields to determine the lateral and gravitational forces applied to the masonry walls. The objective was to verify whether the effect of these loads was greater than the capacity of the masonry shear walls of the mosque.

* Corresponding author: a.trad@bau.edu.lb

Fig. 1. External and internal views of the mosque

2 Structural analysis of the mosque

Before starting identifying and calculating the loads of the mosque, it is to know that modelling the entire mosque seems a complicated mission encountered by several difficulties on technical level. To simplify the task, and because of the moderate height of the mosque (10.2 m), the equivalent lateral force method of the UBC 97 [2] was applied. This method allows the determination of the seismic force applied to each masonry shear wall of the mosque. However, the masonry shear wall absorbing the higher portion of the lateral force was chosen as the one to be modelled.

Firstly, the mosque weight was determined by making a summation of all mosque elements weights (slabs, columns, walls, arches …). Each element weight is obtained by multiplying its volume by the natural limestone unit weight (25.15 kN/m^3, [1]). The volume was calculated through architectural plans supported by site visits. In addition to mosque self-weight, a minimum percentage equal to 25% of live load was taken into consideration. For this case, the live load was taken equal to 5 kN/m^2 for the prayer hall while it was taken equal to 1 kN/m^2 for the roof with a 50% probability of occurrence. The total self-weight for roof and first floor are then estimated equal to 14 360 kN.

The location of the mosque is characterized by soil profile type class "C". Referring to the Lebanese standard NL 135 [3] and to the universal building code UBC 97 [2], the seismic zone factor is Z = 0.25 and the seismic coefficients are C_a = 0.29 and C_v = 0.4. The architectural plans of the mosque show a dominance of walls in both directions compared to columns (Fig. 2). Hence, the lateral load resisting system is considered as a masonry shear wall, and the over-strength factor is taken R = 4.5. Moreover, the importance factor is considered equal to 1. The total design base shear is then calculated and found equal to 2416 kN. Finally the story shear at the roof level is obtained equal to 1397 kN. This force will be transferred through the slab to the resisting walls and columns according to their stiffness.

A manual calculation of the inertial contribution of the walls and columns was carried out in X and Y directions. This calculation demonstrated that X axis is the weaker one, thus it will be considered in the seismic analysis. Based on stiffness contribution, the wall located at the north facade participates by 56.4% of the story shear and the wall located at the south facade by 37.2%, thus the wall of the north will be selected for this study. Due to the symmetry, only half of the wall will be presented in the following analysis.

The vertical load applied to the wall was estimated equal to 459 kN taking into consideration all the following overlying elements (Fig. 3):
- Upper wall having a thickness of 0.5 m and including three openings of medium size 0.475 m × 0.61 m × 0.5 m and 0.435 m × 0.435 m × 0.5 m.
- Roof slab with a span of 6 m.
- Arches at the corners having irregular shapes.

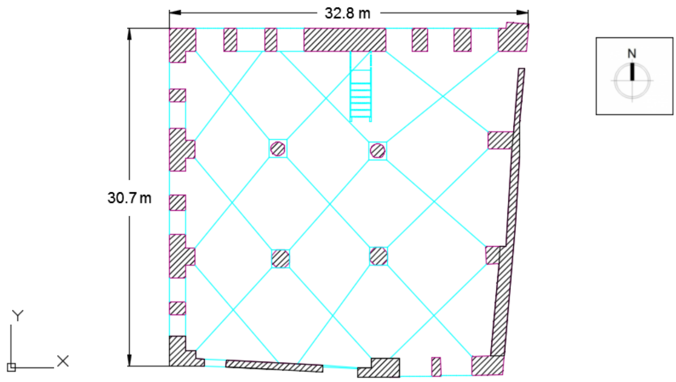

Fig. 2. Planar view of the mosque

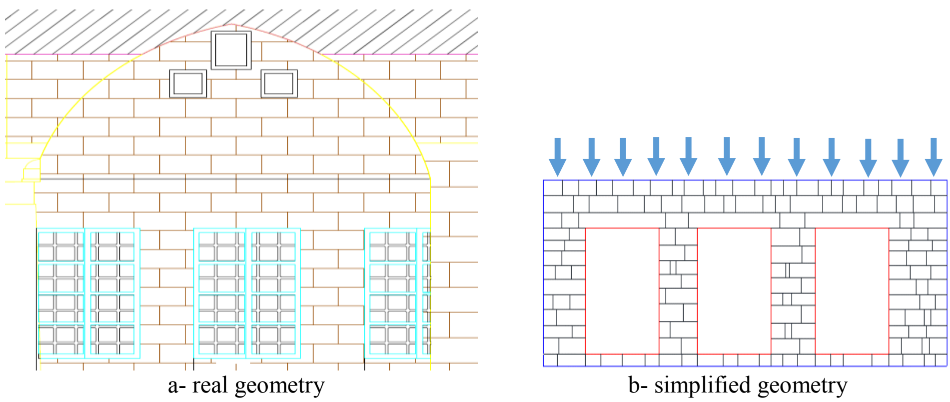

Fig. 3. Modelled wall and overlying loads (arches, upper wall and roof).

3 Numerical modelling

3.1 Model at the scale of an assembly

The mechanical characteristics of the limestone masonry units and lime mortar in addition to the shear strength of mortar joints were previously investigated [1]. The limestone blocks unit weight was found equal to 25.15 kN/m^3, the modulus of elasticity E = 17000 MPa and the Poisson ratio $\mu = 0.2$. The relationship between the mortar shear stress in function of the displacement under different levels of normal stress was also determined (Fig. 4). This test was made on an assembly of two masonry units with one mortar joint between them.

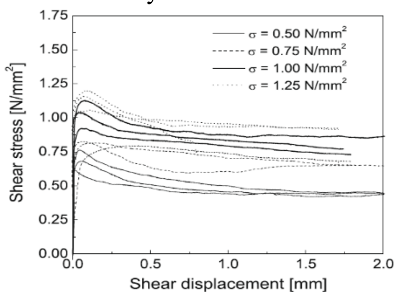

Fig. 4. Shear stress in function of the displacement for a 10 mm mortar joint [1]

The general shape of the shear stress–shear displacement is characterized by a sharp initial linear stretch. For very small shear displacement, the peak load is rapidly attained. Nonlinear deformations develop in the pre-peak regime. After peak load is reached, there is a softening branch corresponding to progressive reduction of the cohesion until reaching a constant dry friction value.

The experimental results obtained on the two blocks assembly scale will be compared to the numerical model in order to check the applicability of the used finite element techniques for the analysis of the studied structure. A couplet specimens having same dimensions was considered for this model. The model was analysed for direct shear test case with a pre-compression stress level of 0.5 MPa. All degrees of freedom at the bottom of the block were restrained.

The method developed by Lorenço and Rots (1997) [4] to model the masonry-mortar interface has been adopted in this study. The 10 mm thickness lime mortar joint between blocks is replaced by zero thickness mortar interface controlled by a shear stress and friction coefficient derived from experimental results. The interaction at the interface is then defined as surface to surface standard contact where two main behaviors are assumed:

- Normal behaviour where the surface contact is assumed to be “Hard contact”. This contact relationship minimizes the penetration of the slave surface into the master surface.
- Tangential behaviour where the shear strength and friction coefficient of lime mortar joint are considered as 0.65 MPa and 0.45 respectively.

A real representation of the experiment loading order and timing was taken into consideration by affecting different loading steps. The self-weight of the couplet specimens is applied firstly followed by a pre-compression stress level equal to 0.5 MPa added at the top surface. The last loading step consists of a shear pressure distributed along the front surface of the top block with a magnitude equal to 1 MPa. Three dimensional solid elements (Continuum C3D-8 Nodes) were used in the model to provide the highest degree of accuracy allowing the detection of any possible displacement between blocks. The assembly model is shown in Fig 5.

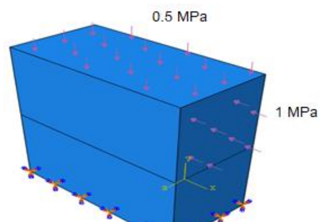

Fig. 5. Assembly model showing loadings and boundary conditions

The shear stress and shear displacement variation as a function of time are investigated in the elastic phase for the shear loading step only (Fig 6). The results of the numerical model show a perfect match with the experimental results (Fig 7). Therefore, this modelling technique can be used to estimate the shear capacity at a larger scale.

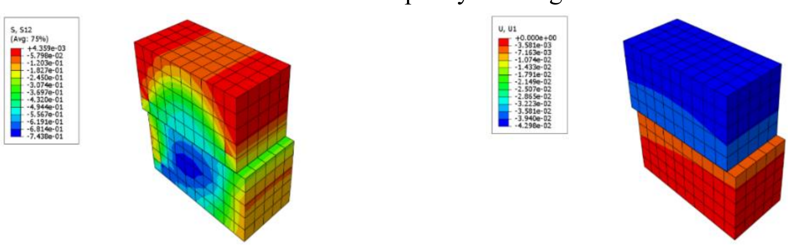

Fig. 6. Assembly shear stress distribution in N/mm^2 (left) and shear displacement in mm (right)

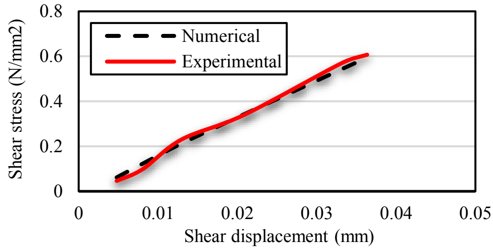

Fig. 7. Comparison between numerical and experimental results

3.2 Model at the scale of a wall

The wall to be modelled has the dimensions of 14 m in length, 3.1 m in height and 1m in depth. Its geometrical parameters were determined by hand measurements in order to achieve the most possible realistic simulation of the wall's behaviour. It is noteworthy that only half of the real wall was modelled (see Fig. 8) in order to reduce the computational time. However, an X-symmetry boundary condition is used for this case to reproduce the behaviour of the entire wall. This option can be used since it fulfils the required conditions regarding symmetric geometry and boundary conditions. The lateral and vertical forces are converted to pressures in order to distribute uniformly the load. The vertical pressure is found equal to 0.065 MPa (vertical load of 459 kN divided by the wall section corresponding to a length of 7 m and depth of 1 m). The horizontal pressure is calculated by dividing the horizontal load of 394 kN by the vertical cross section of the last layer of blocks.

The failure criterion used for the mortar joint is based on the experimental behaviour described in Fig. 4. The failure stress of mortar observed experimentally, even with a normal stress value higher than 0.065 MPa, is lower than the stress generated in joints by the numerical model (up to 0.73 MPa – Fig. 9a). The shear stress values observed in the blocks (up to 1.11 MPa – Fig. 9a) are low enough to say that the failure mechanism is governed by the interaction between the blocks.

The proposed strengthening method aims to apply a sustained normal stress on the masonry assembly by external or internal post tension. Two plates at the extreme top and bottom of the walls are installed to tie the tendons and transfer the compression forces resulting from tensioned tendons as pressure over the area of the wall. The lower end of the tendons can be fixed by various techniques (e.g. channel steels embedded at the bottom of the foundation, or chemical anchor for concrete foundations).

To demonstrate the potential of this method, a trial design is adopted using strands of 12.7 mm diameter with an ultimate strength of 1860 MPa. Strands are installed as shown in Fig. 10 and are stretched to 60 % of their ultimate capacity. The generated post tension force allows the vertical pressure applied to the wall to be increased to 0.5 MPa. Results show that stresses in mortar are decreased to around 0.40 MPa (Fig. 9b) which falls in the elastic range of the mortar behaviour under the same level of normal stress. Block stresses (up to 1.56 MPa – Fig. 9-b) are still too small. Therefore, cracks in joints are well prevented.

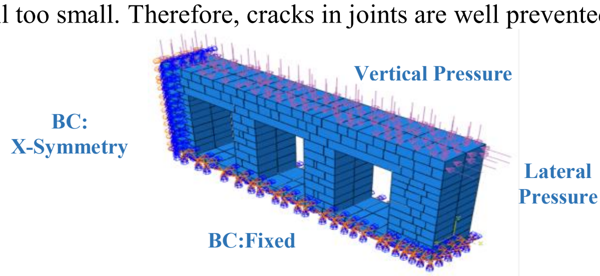

Fig. 8. Modelled wall showing loads and boundary conditions.

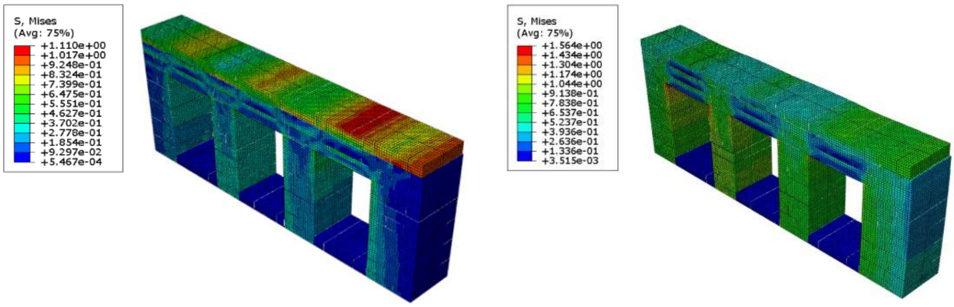

Fig. 9. Distribution of stresses in the wall. a) At left: without post tension. b) At right: with post tension

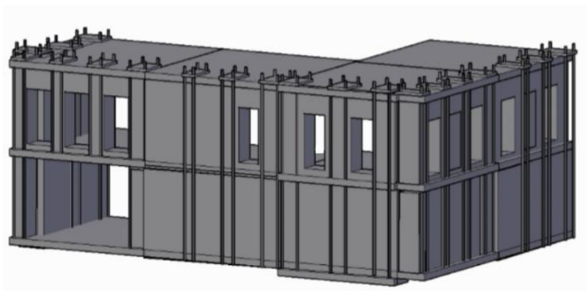

Fig. 10. 3D schematic view of tendons distribution along the walls of the mosque

4 Conclusion

The numerical simulation has shown that stresses in joints exceed the mortar ultimate shear stress. Cracks in joints are then expected in case of earthquake. These cracks will not cause the collapse of the walls, and thus no real danger is threatening the worshiper's life.

Due to the historical value of the studied mosque, it is then important to enhance its seismic capacity in order to keep it away from any risk of minor or major damages.

A rehabilitation method based on the increase of the shear friction capacity of the mortar joint by increasing the applied normal stress is recommended. Internal or external post tension could be an adequate solution.

A further improvement of the results would be to take into consideration the entire geometry of the mosque in the numerical model. This will not be possible using the volumetric finite elements due to the required computational time. The use of shell finite element will be an adequate solution.

References

1. G. Vasconcelos, P. B. Lourenco, Experimental characterization of stone masonry in shear and compression, Construction and Building Materials 23, pp. 3337–3345, (2009).
2. UBC-97, Uniform Building Code, International Code Council, 1997.
3. NL 135, Lebanese standard, Protection from Earthquakes: General Rules, 2012.
4. P. B. Lourenço, and J. G. Rots, Multi-surface interface model for the analysis of masonry structures, J. Engrg. Mech., 123(7), pp. 660-668, (1997).