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COLLAPSE MECHANISMS OF MASONRY BUILDINGS DERIVED BY THE DISTINCT ELEMENT METHOD

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SUMMARY

The present paper investigates the crack patterns and the collapse mechanisms of stone masonry structures subjected to severe earthquake excitations, using a distinct element representation of the structural walls. The investigation was focused on typical structures of traditional Cypriot architecture. It was found that, for typical houses, the most common mode of collapse is the out-of-plane failure of the façade walls, while other failure mechanisms were related to the excessive shearing of the stiffer walls and the collapse of window or door lintels. For this reason, three-dimensional analysis is usually necessary, in order to capture the response realistically. However, the response of the vaults and arches can be estimated quite realistically by two-dimensional modeling. A number of possible intervention techniques were also studied, as the addition of concrete ties or a concrete slab at the roof level and the construction of additional structural walls. Such techniques result in decreasing the out-of-plane deformation of the walls, diminishing, in a great extend, the possibility of collapse. The addition of steel ties to arched walls also enhances the seismic resistance. It is recognized that further research is needed in the field, especially in the simulation of the progressive damage and degradation of the masonry fabric during subsequent shaking cycles.

INTRODUCTION

Non-engineered stone masonry structures, which in many cases constitute the architectural heritage of a country, are the most vulnerable class of buildings to earthquakes. Damaging earthquakes of the last decades in Italy (Umbria-Marche, 1997), Greece (Grevena-Kozani, 1995, Aigio, 1995, Athens 1999), Cyprus (Paphos, 1995, Lemesos, 1996) and Turkey (Izmit, 1999), to confine the presentation to the eastern Mediterranean only, resulted in great losses of building stock in historical centers. Although damage statistics and empirical relations are important tools in order to quantify vulnerability and to estimate future losses, they do not provide any help or guidance on how to reduce this vulnerability and, more importantly, how to assess the effectiveness of various intervention techniques, commonly used by engineers in practice.

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On the other hand, a typical structural analysis of stone masonry buildings provides, in most cases, only a rough estimation of the building's actual performance. Usually, a lot of empiricism has to be used, in order to overcome the uncertainties in the model parameters and interpret the results. For the earthquake response, quite sophisticated analytical methods have to be employed, in order to capture the essential characteristics of the motion. Such methods usually include elastic or inelastic finite element analyses, which can provide the stress distribution over the structure and can be used to identify possible failure mechanisms, especially the in-plane shear failure. Recently, more advanced finite element constitutive models, oriented to the study of the out-of-plane rocking response of masonry walls, have been presented by Casolo et al. [1]. The importance of this, so far 'neglected', failure mode of old masonry buildings has been pointed out by Griffith et al. [2] and Doherty et al. [3], who used simplified rocking block models for its evaluation.

For large structural deformations, however, the anisotropy of the walls, caused by the discontinuous and blocky nature of the masonry, plays an important role and reduces the effectiveness of methods, which are based on continuum mechanics, to predict the failure mechanisms. In the contrary, the distinct (or discrete) element representation of the structure seems to be a promising and quite capable method for the estimation of the response, even after the initiation of cracking, as it has been shown by Brookes et al. [4], Sincraian and Azevedo [5] and Azevedo et al. [6] among others. The distinct element method, formulated by Cundall [7], allows the explicit modeling of the stones and the mortar joints and can handle large displacements and rotations of the individual blocks, including their complete detachment. In that respect it is ideal for the investigation of the failure mechanisms of stone masonry buildings. In this study the commercially available codes by Itasca Co. UDEC [8] (two-dimensional) and 3DEC [9],[10] ((three-dimensional) were used.

Previous studies, which can be found in the literature, are usually confined to two-dimensional models of specific buildings (e.g. bell towers, stone bridges). In this paper, the investigation of the more prominent failure mechanisms of typical, low-rise stone masonry structures is presented. Also, the effectiveness of various intervention techniques, which are common during repair or strengthening works, is examined. Two characteristic examples were selected, for which both two- and three-dimensional models were analyzed, in order to assess differences and similarities. Although the analysis is based on traditional houses of Cyprus, the results can be extended to similar structures of southern Greece and southern Turkey, which are built with many similarities to the Cypriot traditional architecture.

CALIBRATION OF THE DISTINCT ELEMENT MODELING

The calibration of the distinct element modeling of the stone masonry concerns the parameters, which should be assigned to the joints. It was performed by comparison of the numerical prediction of the response with experimental results from the literature.

The experimental data were obtained from a series of shear tests on stout stone masonry walls, performed at the National Technical University of Athens (Chronopoulos [11]). The walls were 1.0×1.0×0.4 m in size and were made of rubble stones and medium strength mortar (UCS~5MPa) representing typical traditional Greek stone masonry. Those walls were tested in plane shear loading for various levels of the axial load. The experiments allowed the study of the post cracking behavior of the masonry, since displacements as large as 20 to 80 mm were achieved during the tests. The typical wall and the testing setup are presented in figure 1.

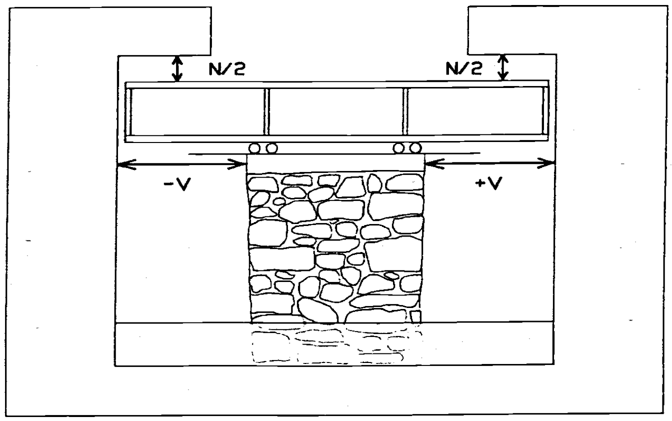


Figure 1. Experimental setup for testing a stone masonry wall (Chronopoulos, 1995).

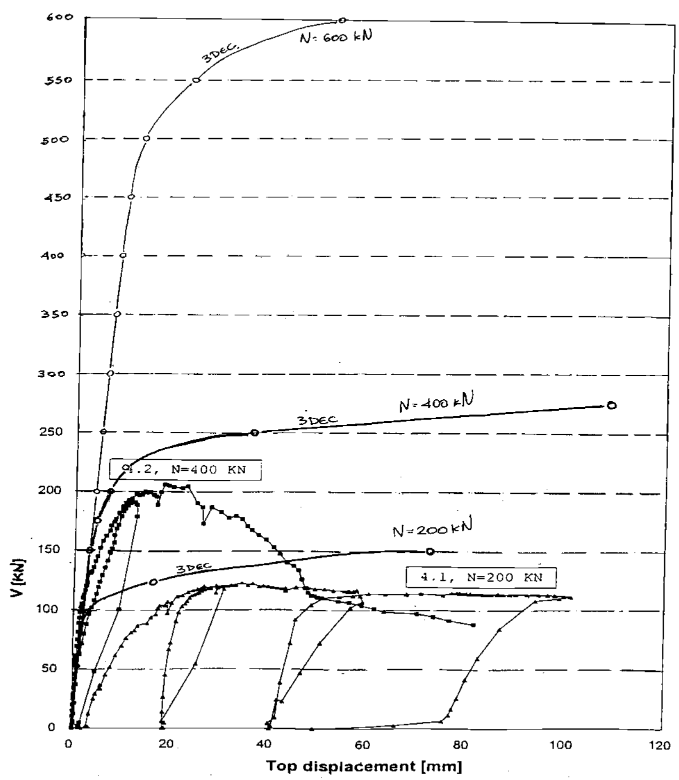
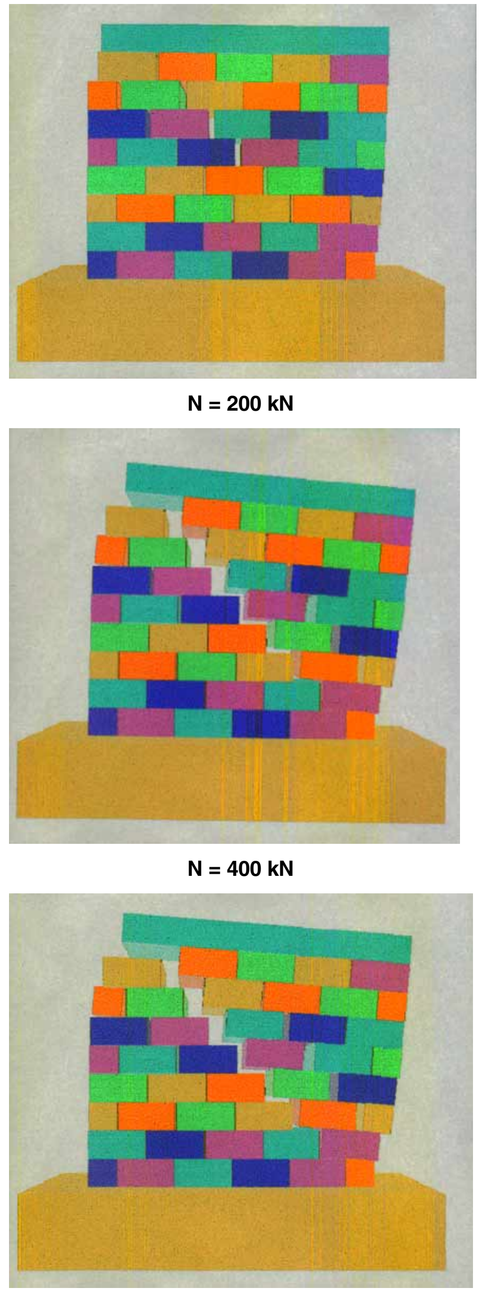


Figure 2. Experimentally obtained and numerically calculated Force–Displacement curves for the stone masonry wall of figure 1 (c=1000 KPa, φ =30°, σ_t = 125 kPa).



N = 200 kN

N = 400 kN

N = 1200 kN

Figure 3. Failure modes of the distinct element model of the wall of figure 1 for three values of the normal force N (c=1000 kPa, φ =30°, σ_t = 125 kPa).

The corresponding distinct element model of the walls consisted of prismatic brick elements with dimensions 0.20×0.125×0.40 m, as shown in figure 3. The Mohr-Coulomb elastic-fully plastic failure criterion was assigned to the joints and the parameters considered were: the cohesion, c, the angle of internal friction, φ, the normal stiffness, J_{kn}, the shear stiffness, J_{ks} and the tensile strength, σ_t. The joint deformability parameters, J_{kn} and J_{ks}, control the initial loading branch and the joint strength, σ_t, controls the ultimate force level. It is emphasized that these parameters depend on the joint pattern and the block size. In particular, the joint shear stiffness is an inverse function of the joint spacing. For this reason, the block size and pattern were selected to be the same with the ones used for the modeling of the masonry structures analyzed in the following sections.

The calibration of these joint parameters was based on the force-deformation curves for monotonic loading. Since the test results were obtained for cyclic loading, only the envelope curve, neglecting the hysteretic loops, was used in the comparison with the results obtained by the numerical analysis. A sufficiently good match was obtained using the following parameters: c=1000 KPa, φ =30°, σ_t = 125 KPa, J_{kn}=30×10^4 MPa/m and J_{ks}=30 MPa/m. The numerical and experimental load-displacement curves are compared in figure 2. It is apparent that a good matching was achieved for the loading branch and the ultimate load. However, the strength degradation after yielding was not reproduced by the simple Mohr-Coulomb failure criterion assigned to the joints. The wall failure modes for three different axial loads are presented in figure 3. It is well demonstrated in this figure that, for a low axial load the response is dominated by a sliding mechanism, while for larger axial loads a gradual transition to an overturning mode takes place.

ARCHITECTURAL TYPES MODELED

The Cypriot rural architecture has many similarities with the rural architecture of Crete and other islands of the Aegean Sea and some regions of mainland Greece. The architectural typology is based on the repetition of a basic rectangular housing unit with flat roofing. As shown in figure 4, the basic unit can be used to form a long single room house (called 'makrynari') or it can be used to form a twin room house with a vaulted wall separating the two rooms and subsequently supporting the roof. This typology introduces, in some extend, uniformity to the forms and the dimensions of the rural Cypriot architecture, thus facilitating its structural study with a limited number of representative models.

In the present study, two characteristic examples were selected; they sample sufficiently well a large number of Cypriot traditional houses. The first models a house in the village 'Neo Chorio' and it belongs to the type of the elongated structures presented in the upper row of figure 4; the second is a model of a structure in the village 'Arodes" and it is a typical example of a housing unit with a vaulted separating wall, like the one presented in the lower row of figure 4.

THE HOUSE IN 'NEO CHORIO'

The first series of analyses of this study considered a two-story house in the village 'Neo Chorio' in the prefecture of Paphos in Cyprus. This building, which is presented in figure 5, is a typical example of a developed single-room house. The ground floor consists of two elongated rooms with dimensions 4×10 m, and the first floor consists of a single room, which covers only half of the ground floor area. The house was built by medium strength stone masonry, with sandstone stones and lime-sand mortar. The roof is almost flat and was originally supported by wooden beams, spanning the short dimension of the structure, and it was covered by earth to provide insulation. At the present, the house is in ruinous condition (see figure 6) with its roof collapsed and some of the walls severely damaged by the earthquake that struck Paphos area in 1995.

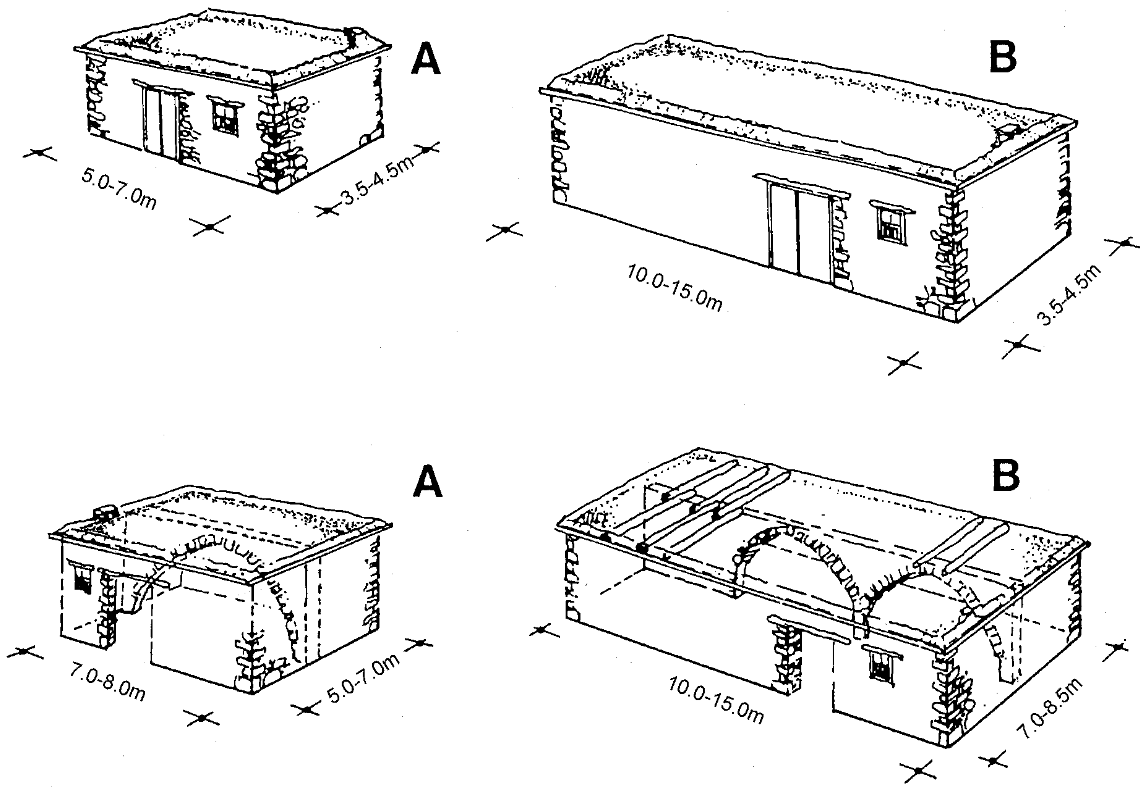


Figure 4. Typology and evolution of traditional houses of Cyprus. Upper row: Single room houses. Lower row: Twin room houses with vaulted partitions (after Pitta, 2000).

In all the analyses presented herewith, the Kalamata, Greece, 1986 earthquake record was used as the base excitation (Ambraseys et al., [13]), appropriately scaled each time to various shaking levels, necessary to study the structural damage and the collapse mechanisms. The earthquake was of magnitude 6.2 and caused considerable damage not only to stone masonry houses, but also to modern, reinforced concrete frame structures in the city of Kalamata, which was located directly above the seismic source. The record of this earthquake bears the characteristics of a near-source seismic motion: it contains a couple of long-period pulses and it has a rather high value of peak ground velocity. This earthquake record was selected because we believe that small size earthquakes at short epicentral distances are the main threat to Cypriot traditional building stocks.

A two-dimensional model of the long façade of the house was analysed using the code UDEC. In spite of the fact that the block size was almost double than the stone size used in the real structure, a significant number of blocks were necessary for a realistic modelling of the wall. The short walls, which run perpendicular to the plane of the analysis, were taken into account by appropriately scaling the density of the brick elements at the model edges. This technique was also used by De Felice and Giannini [14] and it gives satisfactory results when the out-of-plane response of the normal wall affects the overall behaviour.

The cracking pattern and the collapse mechanism, obtained for two levels of the seismic motion, are presented in figure 7. It can be seen that cracks originated from the corners of the window and the door openings and propagated diagonally. The separation of the normal walls at the model edges is also apparent in the lower figure, where the shaking intensity is higher. The collapse originated from the shear failure of the weakest pier, which, in this case, was the pier formed between the left window and the door.

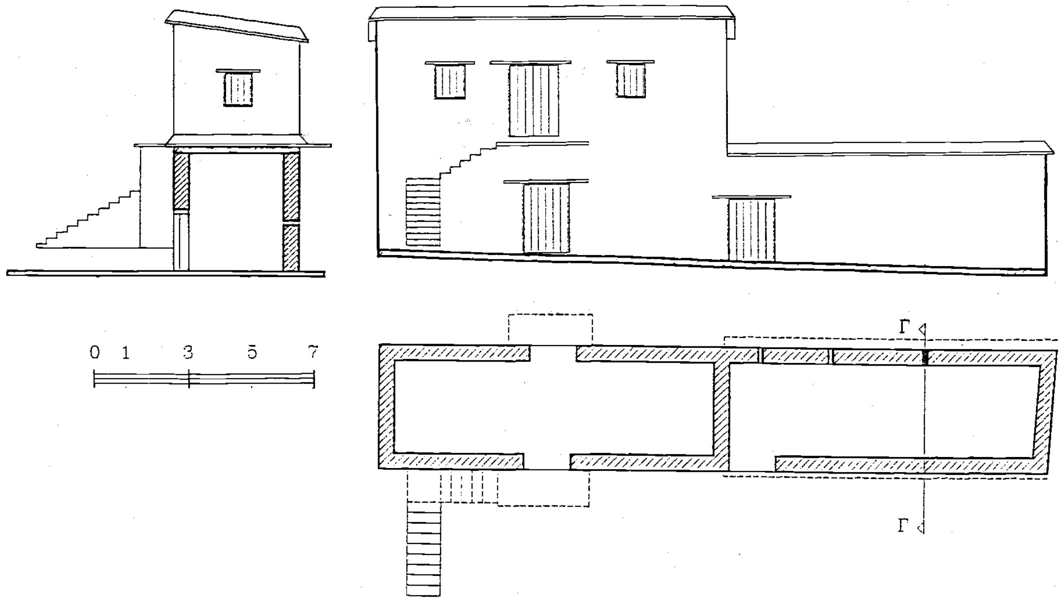


Figure 5. The typical two-story house considered in the analyses.

Figure 6. Photograph of the two-story house at 'Neo Chorio'.

The model house was also analysed by the three-dimensional code 3DEC. In this case, the two-story part of the structure was modelled. The results are presented in figure 8 for the scaled Kalamata record. In the left plot, the cracking pattern for a base excitation with pga=0.54g (twice the original record) is shown, while in the right plot, the collapse mechanism for the seismic motion scaled to pga=0.80g is given. For the lower shaking level, the cracking pattern observed is similar to the one obtained by the two-dimensional analysis (top of figure 7). It should be mentioned, however, that in the 3-D analysis the base excitation was double in amplitude than the one in the 2-D runs. As far as the collapse mechanism is concerned, the mechanism obtained by the 3-D analysis is completely different than the corresponding one from the 2-D analysis. In the three-dimensional model, the out-of-plane rocking of the long walls dominates and the collapse is reached when these walls topple.

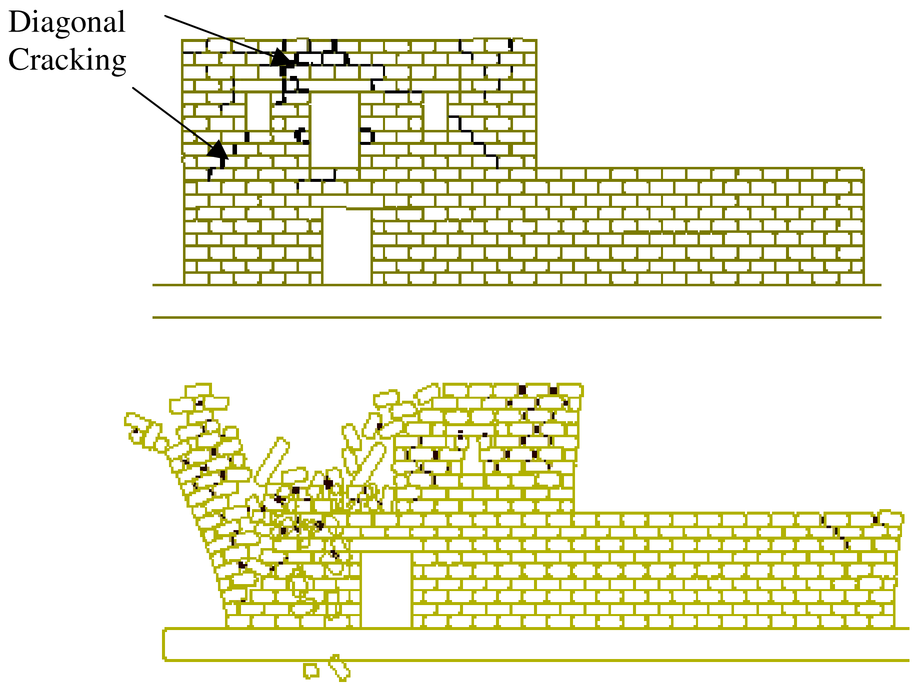


Figure 7. Cracking pattern and collapse mechanism of the long façade for the scaled Kalamata record, according to the 2-dimensional analyses. Upper: no scaling (pga=0.27 g).
Lower: excitation scaled to pga=0.80 g.

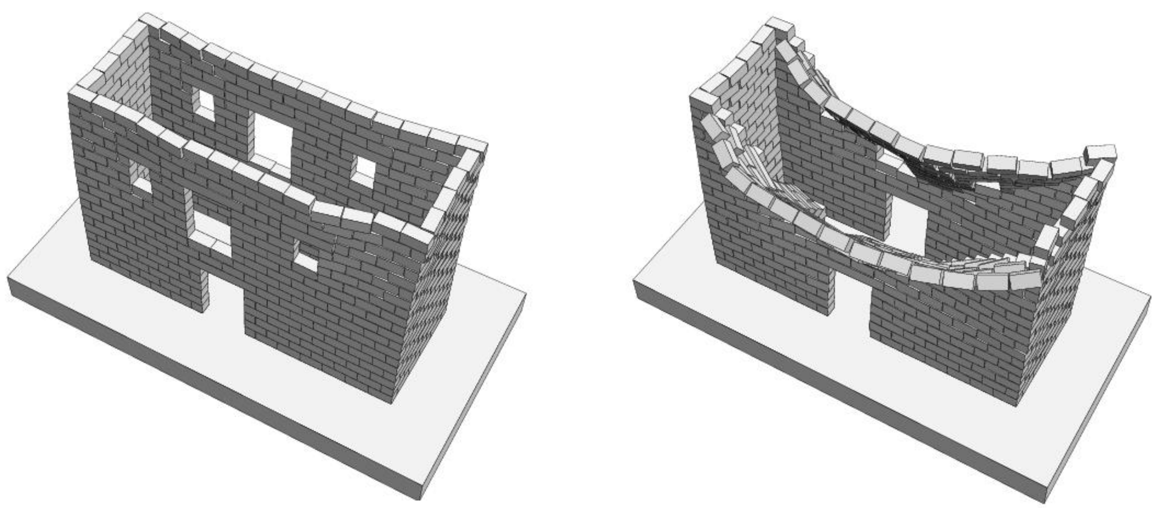


Figure 8. Cracking pattern and collapse mechanism of the model house for the scaled Kalamata record, according to the 3-dimensional analyses. Left: excitation scaled to pga=0.54 g.
Right: excitation scaled to pga=0.80 g.

It should be noted that the cracking patterns derived by the distinct element analyses (top of figure 7, left of figure 8) are in good agreement with the damage observed in similar structures after the 1995 Paphos earthquake. The most common damage is summarized in figure 9 and it consists of diagonals shear cracks, which were more often concentrated to weak piers between openings, vertical cracks at the building corners, caused by wall separation, and horizontal cracks at roof level, due to roof and wall separation. Wall buckling, caused by the out-of plane bending, was also frequent especially at the longer façades.

The shaking level, necessary to reach the collapse threshold, is considerably high according to the numerical analyses and it does not seem realistic, compared to the damage observed after the earthquake. This does not happen only for the Kalamata record, but it is also the case for a number of other records tested. It should be attributed to the perfectly shaped brick elements used in the masonry representation and the inability of the model to simulate contact breakage and rounding of the corners during the rocking. However, the collapse mechanisms are quite realistic.

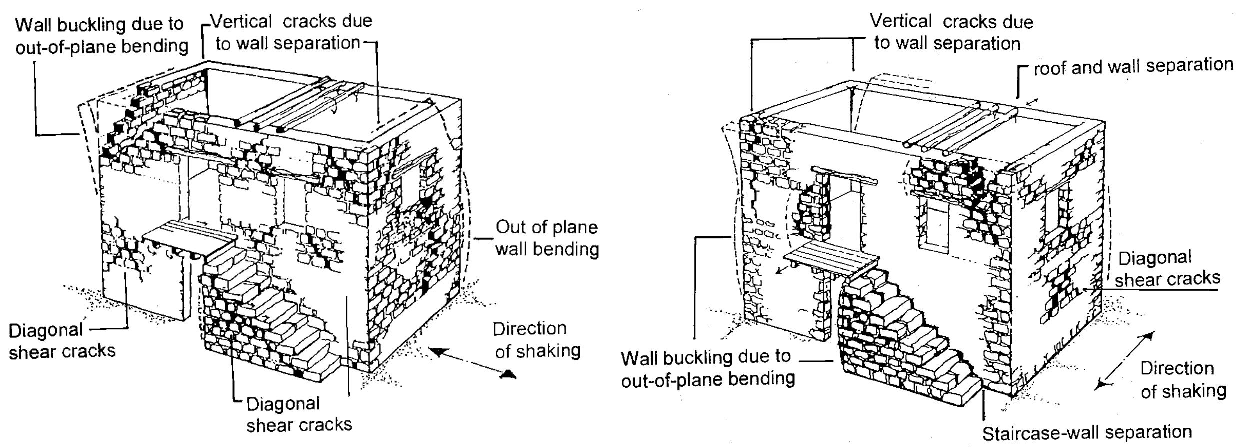


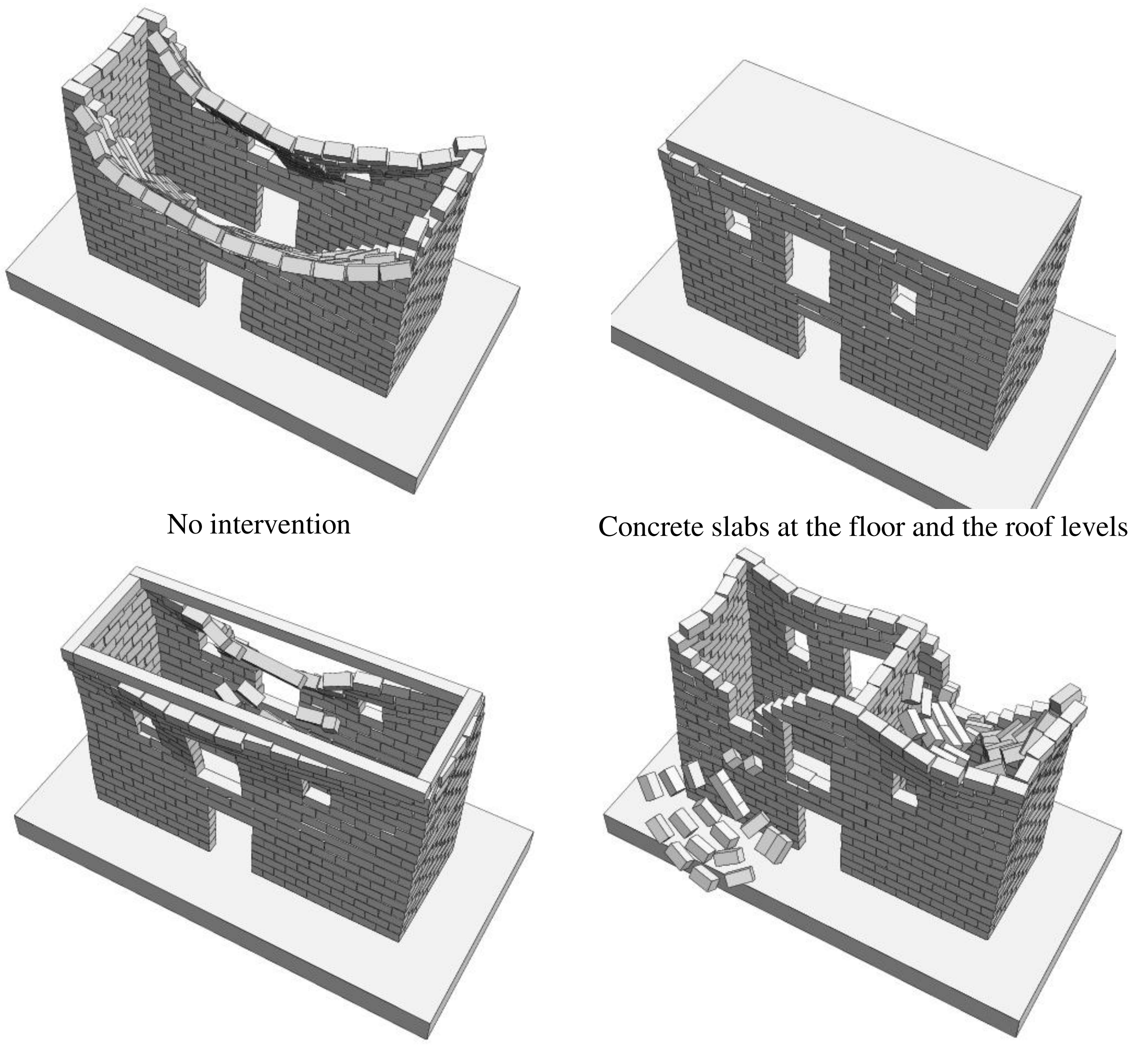
Figure 9. Typical earthquake damage of traditional houses (after Pitta, [12]).

Three intervention techniques, commonly used in practice, were also modeled and their effect to the dynamic behavior and the collapse mechanism of the building was explored. First, the addition of rigid diaphragms at the floor and the roof levels was considered. This is usually performed in practice by casting concrete slabs when the wooden floors have to be replaced. The second intervention concerned the addition of a rigid ring beam, which represents the casting of concrete on top of the masonry walls, in order to tie up the entire structure and to improve, in that way, its seismic resistance. In the third intervention, an internal wall was added. This is done in practice mainly for functional reasons and less for the improvement of the seismic resistance of the structure.

The response of the four models to the Kalamata record, scaled to pga=0.80g, are compared in figure 10. The addition of the concrete slabs clearly ties up the structure and reduces the danger against collapse. In that case, we had to increase the base acceleration to 1.0 g for collapse to occur. The construction of the ring beam on top of the walls clearly plays a beneficial role, but it is not sufficient to prevent collapse at pga=0.80g. It is interesting to notice that the collapse mechanism is similar to the one of the intact structure. Finally, the introduction of the internal wall changes the collapse mechanism but it does not prevent failure.

THE STUCTURE IN 'ARODES'

A traditional building with the typical Cypriot vault, located at the village of 'Arodes' of the prefecture of Paphos in Cyprus, was selected for the second set of analyses. The building is shown in figures 11 and 12. It has an almost square shape in plan (8.50×8.50 m) and it is separated in two rooms by a vaulted wall, which is also used to half the span of the wooden beams of the roof (figure 12). The building was built by medium quality rubble stone masonry, with small openings and a wooden roof with tiles for roofing. For the last few years, the building is used as a traditional coffee shop. It suffered significant structural damage during the 1995 Paphos earthquake and it had to be repaired and strengthened.



No intervention | Concrete slabs at the floor and the roof levels

Ring beam at the roof level | Addition of an internal wall

Figure 10. Collapse mechanisms considering various intervention techniques.
In all four models the excitation was the Kalamata record scaled at 0.80 g.

First, the two-dimensional model of the vaulted wall was analysed. The scaled Kalamata record was again used as the base excitation. The results are presented in figure 13 for various time instances; in the left column, snapshots of the vibrating structure for the original record, without scaling, is shown; in the right column, the collapse of the arch, which occurred when the record was scaled to 0.40g, is presented. It is important to acknowledge that the response is controlled by the development of a three-hinge arch mechanism, which results in extensive cracking at the crown. The building under consideration suffered this particular kind of damage during the 1995 earthquake.

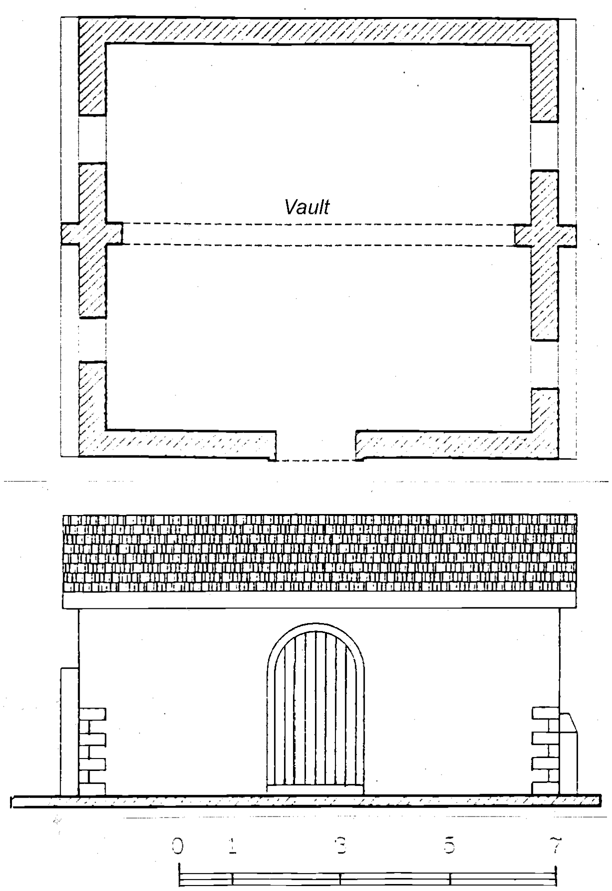


Figure 11. The typical vaulted building in Arodes

Figure 12. Photograph of the interior of the building. Notice the vault.

The response of the two-dimensional model was compared to the response of a simplified three-dimensional one, in which the openings of the external walls were omitted. The results are shown in figure 14. It is interesting to note the similarity of the collapse mechanism derived in both cases. However, in the three-dimensional model a larger excitation was needed in order collapse to occur. This increased resistance is attributed to the presence of the external walls.

A common strengthening practice for such arched structures is the introduction of a horizontal steel tie along the wall at the roof level. Sometimes a similar rod is placed at the ground floor level. The effect of such an intervention was studied using the two-dimensional model. In figure 15, the response of the unstrengthened and the strengthened arches to the Kalamata record, scaled to 0.40 g, is presented. It is observed that the introduction of the steel rod decreases significantly the damage and prevents collapse at this shaking level. The cracking at the crown area is reduced, but some cracking occurs at the area of the tie anchorage.

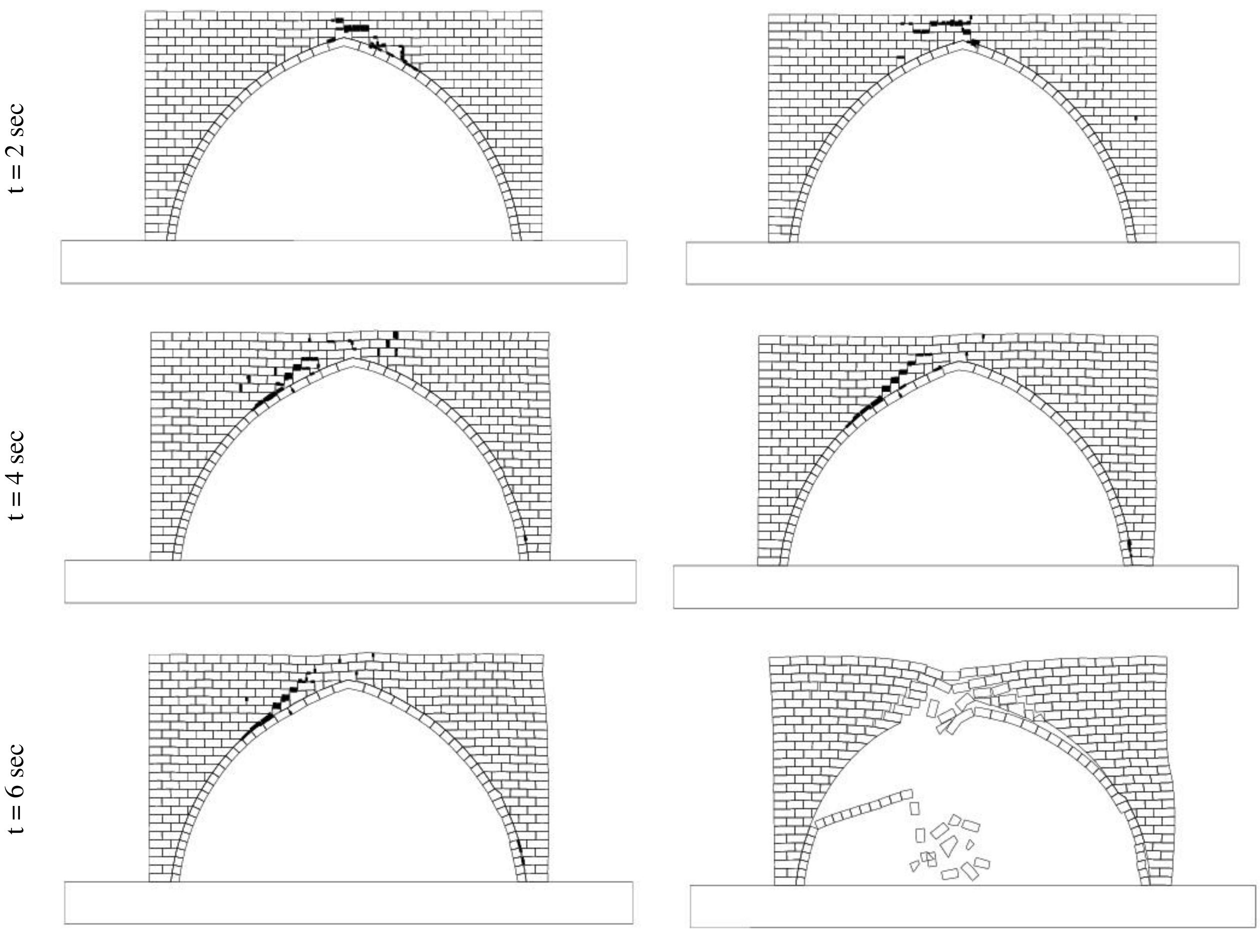


Figure 13. Cracking pattern and collapse mechanism of the vaulted wall for the scaled Kalamata record, according to the 2-dimensional analyses. Left column: no scaling (pga=0.27 g). Right column: excitation scaled to pga=0.40 g.

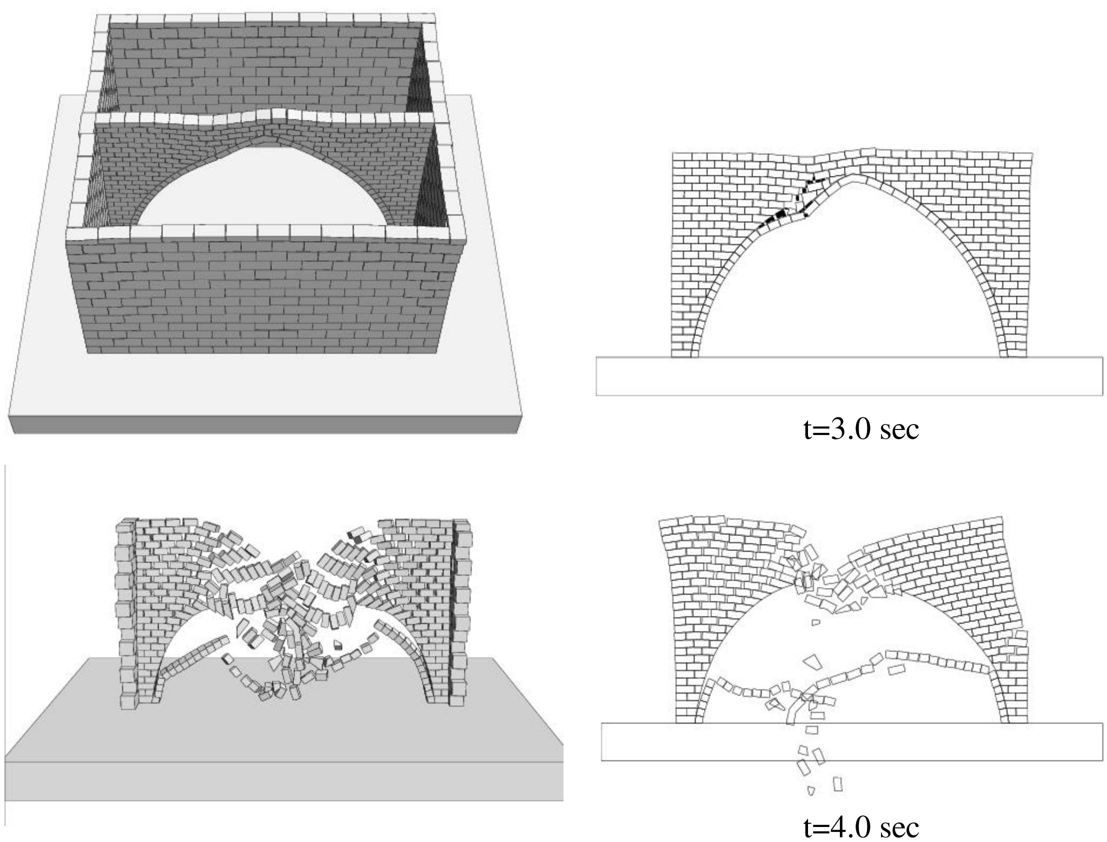
CONCLUSIONS AND DISCUSSION

In this paper, the earthquake response of masonry houses was examined by numerical analyses based on the distinct element method. Analyses were performed for two types of houses, common in the Cypriot architecture.

The analysis showed that the distinct element method can be a valuable tool in identifying damage patterns and collapse mechanisms of stone masonry buildings, subjected to severe earthquake excitations. In order to perform a successful and realistic modeling, however, the calibration of the parameters of the numerical model is required. The calibration can be done using experimental data. In this study, the joint parameters were determined from the force-displacement relation of simple wallets under monotonic loading. It was found, however, that the numerical model could not reproduce the stiffness degradation after yielding. More research is needed in order to be able to capture the full cyclic stress-strain response of the masonry.

The out-of-plane rocking response of long walls was found to be the dominant mode of the failure mechanism of masonry buildings. In that respect, two-dimensional analyses fail to realistically simulate the response. Only the response of vaulted walls is realistically captured by the two-dimensional modeling. Three-dimensional analysis seems to be able to capture the collapse mechanism quite well. In the analyses performed in this study, however, unrealistically high levels of the base excitation were required for collapse to occur. This was attributed to the fact that the masonry was represented by perfectly cut brick-shaped stones of infinite strength. In reality, the breakage of the stone corners after a few rocking cycles reduces significantly the contact area between adjacent blocks and result in a reduced strength against collapse. It is believed that the use of stone blocks with cut or rounded corners and the development of more advanced constitutive models for the representation of the joint parameters will significantly increase the accuracy of the method.

Several intervention techniques for masonry buildings were also examined. For typical houses, it was found that the best results were obtained with the replacement of the wooden floors by concrete slabs. Concrete beams, forming a ring at the top of the walls, also have a beneficial effect. In the contrary, the addition of an internal wall, normal to the long direction, changes the collapse mechanism, but it does not seem to decrease the possibility of failure. For vaulted walls, the addition of a steel tie at the top level decreases the danger of collapse significantly.



Kalamata record scaled to 0.54 g. Kalamata record scaled to 0.40 g.

Figure 14. Comparison of the cracking pattern and the collapse mechanism, derived by a three-dimensional analysis (left) and a two-dimensional analysis (right).

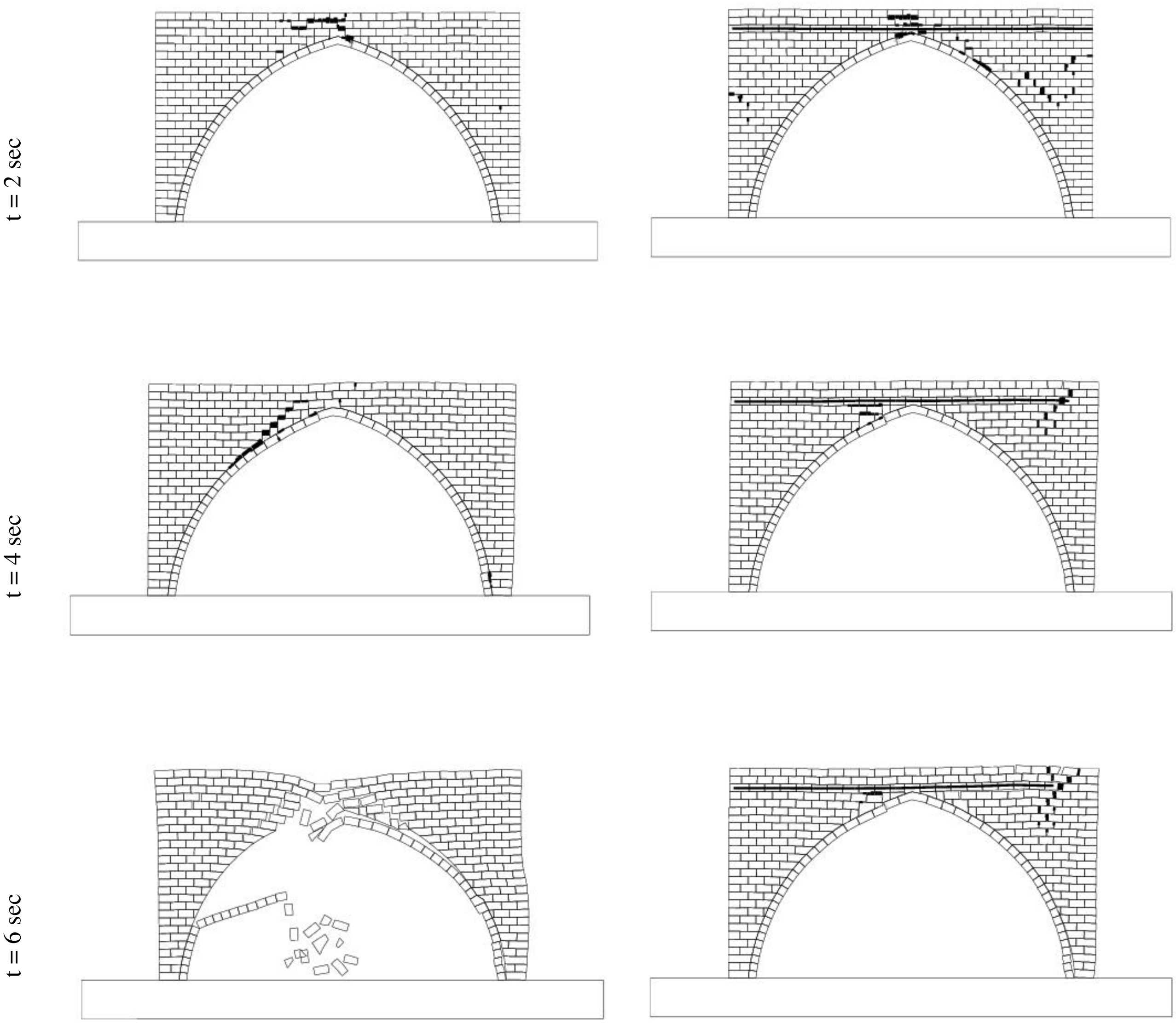


Figure 15. Cracking pattern and collapse mechanism of the vaulted wall. Left: without reinforcement. Right: reinforced with a steel tie. (Excitation: Kalamata record scaled to 0.40 g).

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