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SEISMIC LOAD ASSESSMENT FOR MASONRY MONUMENTAL BUILDINGS

Thomas N. Salonikios, Konstantinos E. Morfidis, and Vasilios A. Lekidis

Earthquake Planning and Protection Organization (EPPO)
5 Agiou Georgiou Str., Patriarchika Pylaia, 55535 Thessaloniki, Greece
e-mails: salonikios@itsak.gr, morfidis@itsak.gr, lekidis@itsak.gr

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Abstract. *In present work, selected experimental data of the last 25 years are used together with the inelastic analysis of masonry frames and the postprocessing of the records of strong earthquakes in Northern Greece, for the estimation of rational seismic loads that are appropriate for the seismic evaluation and/or upgrade of existing masonry buildings. The estimated rational seismic loads have significantly lower values than the seismic loads that are suggested by modern seismic codes. It is commonly accepted that modern codes usually suggest very high seismic loads. This happens even in the case that in these codes are foreseen provisions appropriate for masonry structures. These provisions usually refer to modern masonry buildings where is possible to provide high strength. Through the literature review was found that in existing unreinforced masonry buildings is possible to be developed an "equivalent" damping around 20%. Also was found that the eigenfrequency of masonry buildings is reduced at 2/3 - 1/2 of the elastic one and the corresponding reduction of the stiffness is at least 50% of the elastic one. These reductions happen close after the "yielding" point of the "equivalent" bilinear elastoplastic diagram of the structure. The performance points of such structures are located after that "yielding" point. Through the nonlinear analyses of masonry frames, resulted the base shear that these frames is possible to resist and its value is 25% of the total vertical loads of the frames. This value was found for the models with nonlinear frame elements and for the models with discrete brick – mortar joint elements. The models, where bricks and mortar joints were simulated by continuous elements, had higher strength than two aforementioned models. Through the postprocessing of recorded accelerograms of strong earthquakes, resulted spectral accelerations with values around 70%g. The corresponding normalized accelerations resulted 50%g and correspond to damping 5%. By considering "equivalent" damping 20% (as was found above) the normalized spectral accelerations resulted around to 25%g that are close to the value noticed above as the resisted accelerations of the examined frames. These spectral accelerations were found for the records of Achaia – Ilia earthquake. For this earthquake were not observed any collapses of monumental masonry structures at the stricken area with the exception of some local damages and local failures. This way, many of the conclusions - suggestions of the present work are confirmed. It is also concluded that additional experimental and analytical research effort is necessary in the aforementioned fields for the legislation of the conclusion of the present work.*

1 INTRODUCTION

The estimation of the seismic loads that should be used for the seismic evaluation of masonry monumental buildings is the field of extensive research, last decades, in countries with high seismic activity. These loads are strongly affected by parameters such as the "equivalent" damping, the variation of the eigenproperties of the building and the available capacity of the masonry monumental buildings for inelastic deformation. According Beucke and Kelly [1], the total damping of masonry walls is composed by two parts, the viscous and the friction parts. The viscous part of the damping is divided to "constant" and to "kinetic" Coulomb damping. All aforementioned parts give the "equivalent" damping "ξ_{eq}". This consideration is a simplified expression of the damping and according parametric analyses the exact solution is appropriately approached. Calvi, Kingsley and Magenes [2], utilize laboratory tests of masonry specimens, pseudo-dynamic loading as well as seismic table tests on building specimens to examine the type of the response of the specimens (shear or flexural rocking) in relation with the capacity for seismic energy damping. Through specimens' tests was found that the response of the specimens is strongly affected by the aspect ratio. For high aspect ratios the flexural rocking dominates while for low aspect ratios dominates the shear response. In the second case (shear response) the dissipated energy is higher than the energy that is dissipated by flexural rocking response. During rocking the unloading path of the load – response curve is parallel with the loading path with slightly lower values. This way is indicated that an important amount of the seismic energy that is needed for the maximum deformation of the tested walls return to the specimen during the unloading procedure. During diagonal shear response the unloading path is almost a vertical line resulting thus to higher area of the hysteresis loops than in the case of flexural response. In the case of sliding shear response the absorbed and dissipated energy is higher than in the aforementioned two cases. The main disadvantage in this case is the high permanent deformations along the base sliding cracks. In some cases the shear response of masonry walls is ductile. This is observed in masonry piers with low aspect ratio ($\leq$1.0) that are subjected to shear loads. In this case the failure cracks are formed in mortar joints. After this type of cracking, remain significant strength reserves to gravity and live loads. Also through the development of friction resistance at the cracks there is a capacity for resistance to shear loads. According Magenes and Calvi [3], masonry walls respond in flexural, diagonal shear or sliding shear ways and for each case of response are given the corresponding shear forces. Also in this work is extensively studied the damping capacity during each considered way of response. During flexural – rocking response the hysteretic damping is 10% and the viscous damping 5% should be added to this value. During diagonal shear response the "equivalent" damping results 10%. In the case of sliding shear response, resulted "equivalent" damping 64% for high level of deformation. In every test, in this study, was observed a combined response of flexural, diagonal shear and sliding shear response. For this reason the value of 64% is very high and for the definition of reliable damping factors all possible responses should be taken into account. Before 1997 there were not adequate experimental data for the consideration of the contribution of the sliding shear mechanism to the damping during combined shear – flexural response. In this work is suggested that in the case of flexural – rocking response should be considered "equivalent" damping 15% and ultimate drift 1.0%, in the case of diagonal shear response the corresponding values are suggested as 15% and 0.5% while in the case of sliding shear response are suggested "equivalent" damping 20% (considered as conservative) and ultimate drift 1.0%. The definition of the type of the response (flexural, diagonal shear, sliding shear) is achieved by simplified equations that are given by Magenes and Calvi in [3]. The design response spectrums that are given by Eurocode 8 are reduced by 1.8 times when "equivalent" damping is considered. This reduction is higher, when are used response spectrums that resulted from the records of real earthquakes. It is concluded that the proposed behavior factor, 1.5, is very conservative and higher values should be used when are taken into account the complete model of the masonry building as well as the activation of inelastic mechanisms, the “equivalent” damping and the variation of the eigenperiods of the building. In the study by Tomazevic and Weiss [4], are given results from the test of a 1:5 scale building model at the seismic table. During the tests resulted reduction at the eigenfrequencies of the tested building model about 40% and the resulted behavior factor was found q=2.84. The Eurocode value q=1.5 is conservative but seems reasonable in the case that are not desired any damages at the masonry buildings. In the study by Bothara et al [5], are described the tests at the seismic table of a 1:2 scale masonry building model. The seismic loads were applied in two directions. A part of the study was related with the estimation of the dissipated energy for various levels of deformation and vulnerability. From the postprocessing of the recorded response resulted “equivalent” damping factors up to 36%. This way is shown a significant capacity of the masonry buildings for a steady nonlinear response under high seismic loading. This nonlinear response is attributed to the flexural – rocking response and to the reduction of the eigenfrequencies of the building (softening) due to the development of cracks at the shear mechanisms. Ahmad et al. [6], suggest that the energy that is dissipated by masonry structures that are subjected to seismic loads should be defined in relation with the achieved ductility: $\xi=0.05+c.(\mu-1)/(\pi.\mu)$. The coefficient “c” resulted from experiments on masonry piers and was found c=0.32. In this work the hysteretic damping varied between 9% and 14%. These values vary due to the influence of aspect ratio and the level of axial compression. In the study by Michel et al [7], was found that the main eigenfrequency of the masonry buildings is reduced at 60% when are applied accelerations about 0.20g. Also the drop to the eigenfrequency is about 50% for inter storey drifts 0.5% - 1.0%. These values resulted from pseudo-dynamic tests at two masonry buildings that were constructed in natural scale.

Through the literature review is concluded that there are many reasons for the consideration of an “equivalent” damping that represents the viscous and hysteretic response of the masonry walls that are subjected to seismic loading, with value over 5%. In the case of the seismic evaluation of masonry buildings for a predefined level of damage, is concluded that these buildings have adequate capacity for seismic energy dissipation. This capacity is attributed to the “equivalent” damping that was found during the literature review, about 20%. Another important conclusion from the literature review is that the main eigenfrequency of the masonry buildings is reduced by 40% to 50% for drift values 0.1% - 1.0%. From the aforementioned conclusion, the stiffness of the masonry buildings seems to be reduced by 50% of the initial one (elastic) at the yield point of the bilinearized diagram of the response of the building. This way are developed lower stresses at the masonry structural elements than in the case of consideration of the elastic stiffness.

2 PARAMETRIC ANALYSES

In this section are given results from the inelastic analysis of masonry frames that are subjected to seismic loading that is applied by horizontal step by step increased forces [8]. Two types of frames are considered. One frame is constructed with two piers (1BAY) and the other is constructed with eight piers (7BAY), both frames are two-storey. The frames were selected, by taking into account that masonry buildings are generally short in height while is possible to form two-pier to multi-pier frames. The frames were modeled with inelastic properties and were analyzed by the CAST3M Code (Visual CAST3M, 2000) [8]. The frames were modelled by two dimension finite elements with two different property types. In “Continuous” models, masonry is considered as a two-dimensional homogeneous material under the plane stress assumption (homogenized media) so that bricks and mortar joints are not represented separately. The numerical model chosen is the model of Mazars existing in the code CAST3M: it is an isotropic scalar damage model suitable for concrete structures under monotonic loadings. The damage variable varies from 0 (virgin material) to 1 (fully destroyed material). The elastic characteristics are Young modulus 1650MPa and Poisson coefficient 0.2 and tension/compression strengths 0.1MPa and 3.0 MPa respectively. The parameters governing the post-peak behaviour of the model have been chosen so as to obtain a brittle softening in tension and a more ductile softening in compression (1BAY, 7BAY frames). In “Discontinuous” models, masonry is considered as a two-dimensional heterogeneous material under the plane stress assumption: the bricks are modeled separately (elastic material) and the mortar joints are represented by elasto-plastic interface elements [8]. The plastic yielding of the interface is governed by a composite yield locus featuring a usual Mohr-Coulomb criterion, a tension cut-off and a compression cut-off. The post-peak regime is controlled by three uniaxial evolutions featuring respectively the behaviors in pure shear, pure tension and pure compression. The elastic characteristics of the brick and mortar joint are usually very different (the former are much stiffer than the latter). The Young moduli of the two materials (2400MPa for brick and 400MPa for mortar) have been chosen so as to give in average (owing to the bond pattern, masonry is orthotropic) a value around 1650MPa. The friction angle of the interface was 30^o and the cohesion 0.15MPa. The tension/compression strengths of the masonry (0.1MPa and 3.0MPa) have been affected to the tension/compression cut-offs of the interface [8]. The post-peak behaviour was again chosen very brittle in tension and more ductile in compression and shear (1BAY, 7BAY frames). The loading is imposed on the structures in a two step sequence. In the first step the vertical uniform distributed loads are applied and in the next step the lateral loads are imposed. The values of the latter are then increased monotonically step by step [8]. The analysis is carried out until a prescribed displacement value at a prescribed (control) node is developed. In the parametric solutions, three different lateral load distribution were used:

- Loadcase ACC: A lateral force is applied at each node, that is proportional to the mass tributary to that node.
- Loadcase MODE: Application of a load pattern in lateral direction that is proportional to the product of a specified mode shape times the mass tributary to that node. For this case the first mode is used.
- Loadcase LOAD: A triangle distribution of lateral loads proportional to the mass and height of each floor.

From the different analysis cases, resulted the load – displacement curves for the examined masonry frames. By the use of these strengths and the existing axial loads are defined the “equivalent” seismic load coefficients that can be withstood by the considered frames (Table 1).

From table 1, is concluded that by modeling the masonry frames by continuous elements results higher strength than in the case that are used discrete elements or frame elements. By the use of seismic loads according “ACC” load case, results increased strength of the frames in comparison with the distribution of seismic loads according “ LOAD” or “MODE” load cases. By the use of inelastic analyses and table 1, is concluded that the considered frames, with properties similar to the ones of ordinary masonry buildings, can resist an “equivalent” seismic load coefficient around 0.25g.

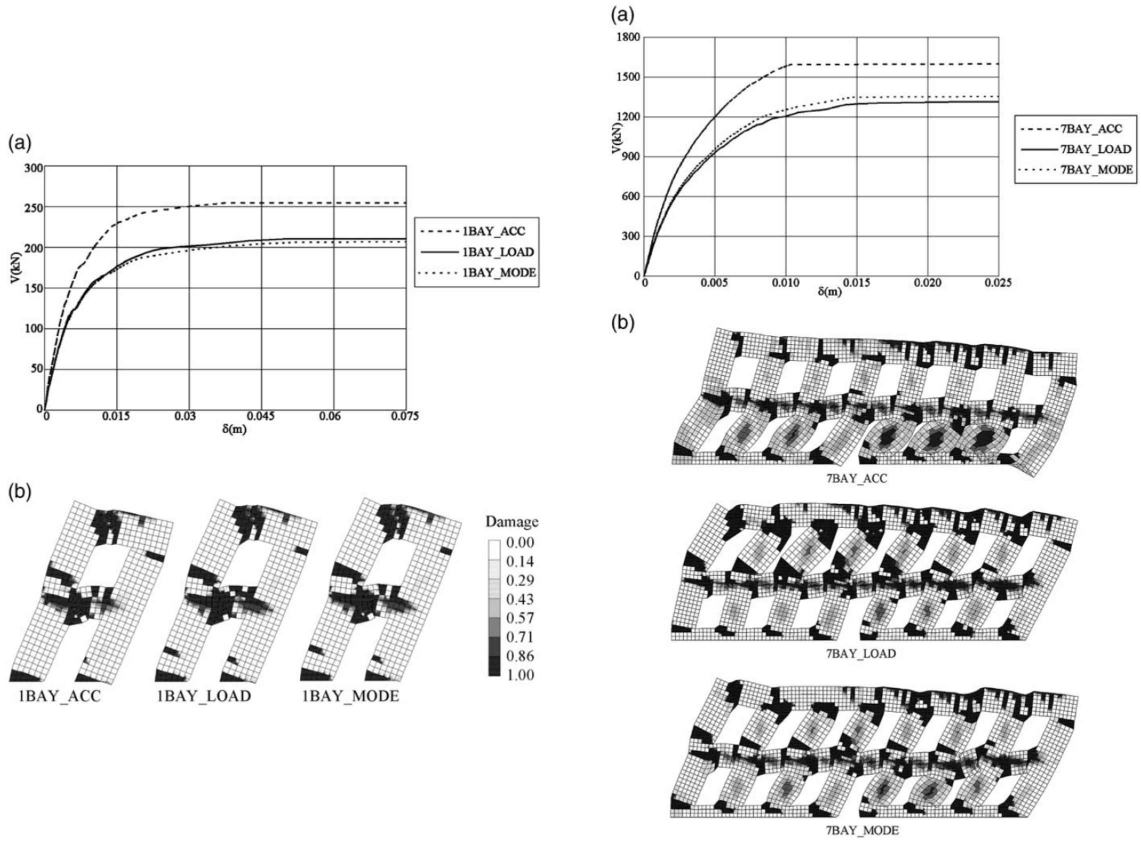


Figure 1: (a) Load – displacement curves (b) Failure types of considered frames. Continuous models.

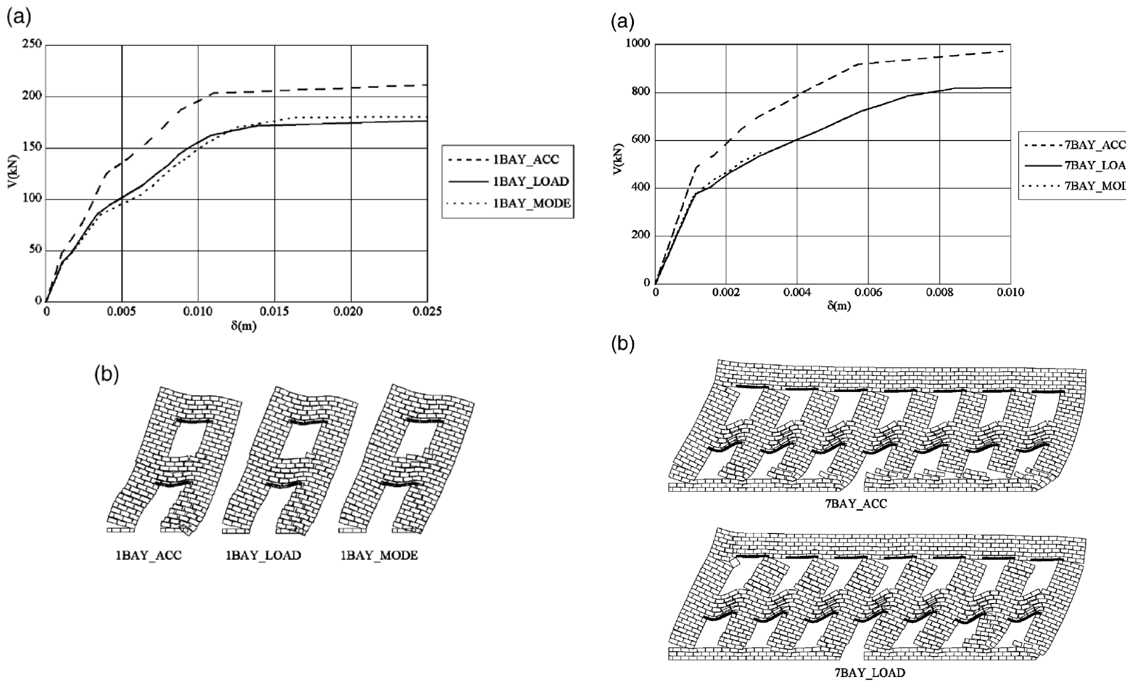


Figure 2: (a) Load – displacement curves (b) Failure types of considered frames. Discrete models.

“Equivalent” seismic load coefficients				
FRAME TYPE	1 BAY		7 BAY	
LOAD TYPE	LOAD, MODE	ACC	LOAD, MODE	ACC
SAP2000 Nonlinear Frame Elements	0.26g		0.24g	
Continuous Model	0.30g	0.36g	0.47g	0.55g
Discrete Model	0.25g	0.31g	0.29g	0.33g

Table 1. “Equivalent” seismic load coefficients for the various model types and various loading schemes.

3 RESULTS FROM THE POSTPROCESSING OF RECENT EARTHQUAKES

In this paragraph are postprocessed the records of four strong earthquakes that happened in Peloponnesus (in southern Greece), during last years, and were recorded by the Strong Motion Network of EPPO. The epicenters of these earthquakes and the records are given in figures 3-8. The response spectrums of these earthquakes are given in figures 9-11. The response spectrum of Achaia – Ilia earthquake has the highest ordinates. The spectral accelerations for these earthquakes are elevated for periods between 0.18sec to 0.30sec. Through simplified considerations the main eigenperiod of this earthquake is close to the main eigenperiod of two storey masonry buildings that are located close to epicentral area. For the rest earthquakes the response spectrum accelerations are lower with Koroni earthquake having the lowest values. In table 2, the maximum observed response spectrum accelerations (column 2) are compared with the corresponding code accelerations (column 4). The highest different result for Achaia – Ilia earthquake. The normalized spectral accelerations that are given at column 5, denote that many monumental masonry buildings should been collapsed especially for Achaia – Ilia earthquake with normalized Sa_{max}=0.57g. From the in-situ inspections resulted that for Koroni earthquake were not observed any damages to monumental masonry buildings (MMB), for Kithira earthquake were observed slight damages while for Achaia – Ilia earthquake were observed extended damages to some MMBs. This is attributed to the development of high “equivalent” damping coefficient by the MMBs in contrast with the norms of many Seismic Codes. It is obvious that the suggested values of damping coefficient, 5% is very low. The same value is suggested by similar codes for the damping of reinforced concrete walls. It is obvious that a masonry wall of 0.60m thick is not possible to have the same damping coefficient as a reinforced concrete wall of the same thickness. In two records of Achaia – Ilia earthquake are calculated the response spectrums for damping 5%, 10% and 20%. These spectrums are normalized at 66% of the maximum ordinate, Figures 12-13.

EQ	Sa_{omax}	Sa_{max}	T	2.5*Ao/ q	(2/3)*Sa_{max}
	(1)	(2)	(3)	(4)=2.5*0.24/1.5	(5)=2/3*(2)
Koroni	0.07g	0.31g	0.28sec	0.6g/1.5=0.40g	0.20g
Kythera	0.15g	0.65g 0.46g	0.55sec 0.32sec	0.6g/1.5=0.40g	0.43g 0.31g
Achaia - Ilia	0.21g	0.87g	0.18sec	0.6g/1.5=0.40g	0.57g

Table 2. Calculation of Spectral Accelerations

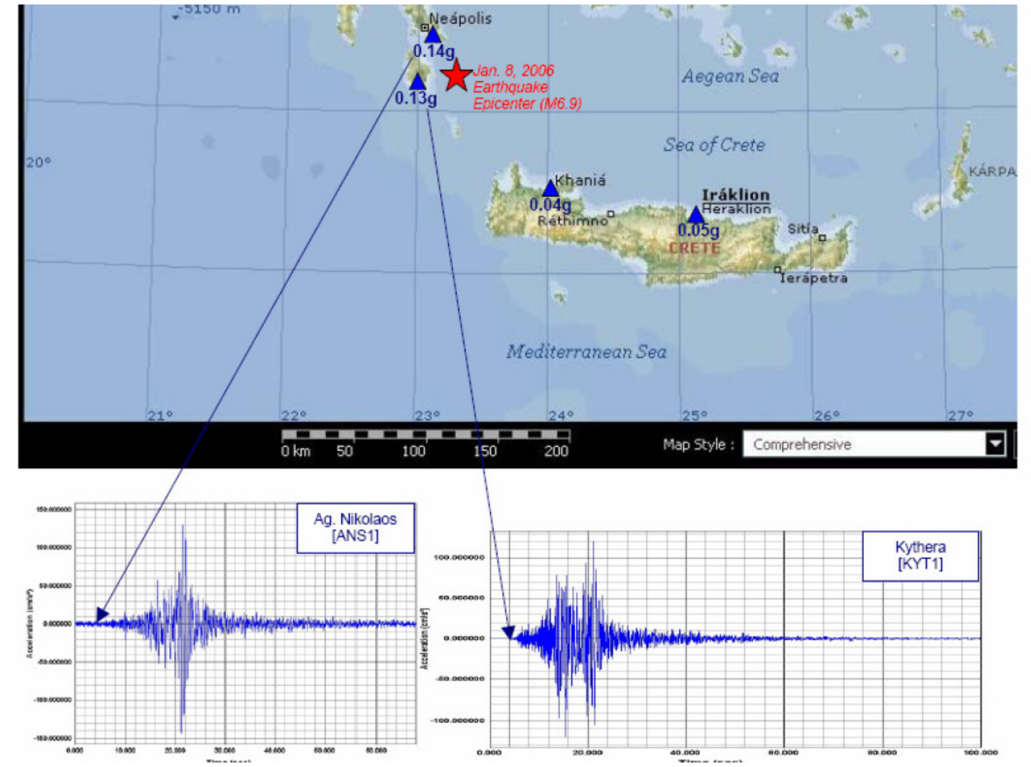


Figure 3. Kithira Earthquake 8/1/2006, (M6.9). Epicenter, Accelerometric Network and records.

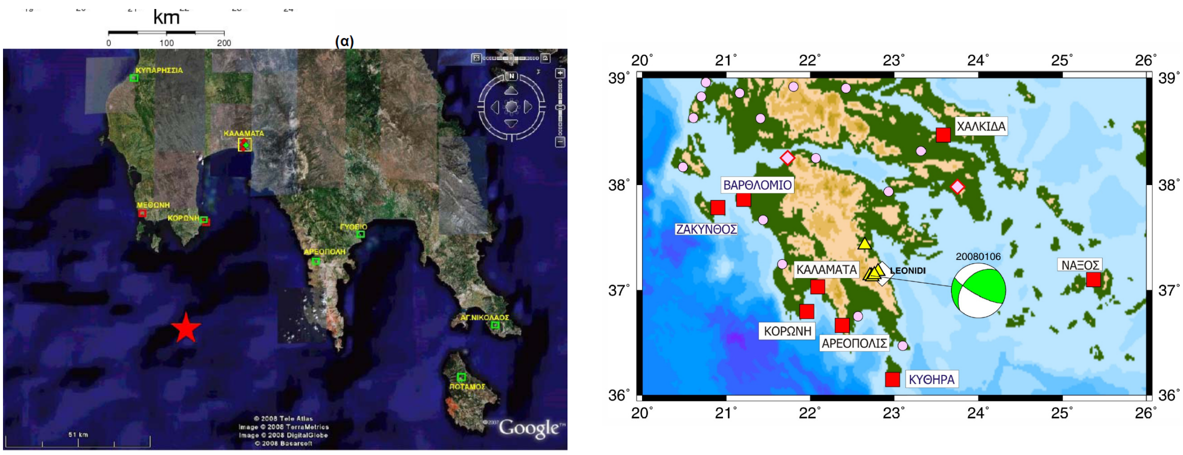


Figure 4. Left: Koroni Earthquake 14/2/2008, (M6.7). Right: Leonidio Earthquake 6/1/2008 (M6.5).

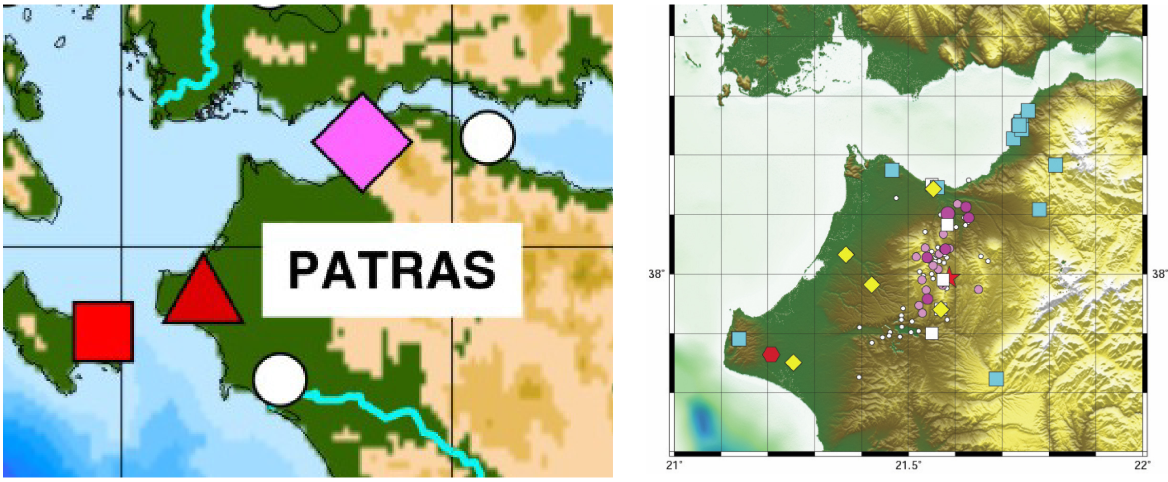


Figure 5. Achaia – Ilia Earthquake 8/6/2008 (M6.5). Left: Accelerometric Network, Right: Epicenters of the seismic activity.

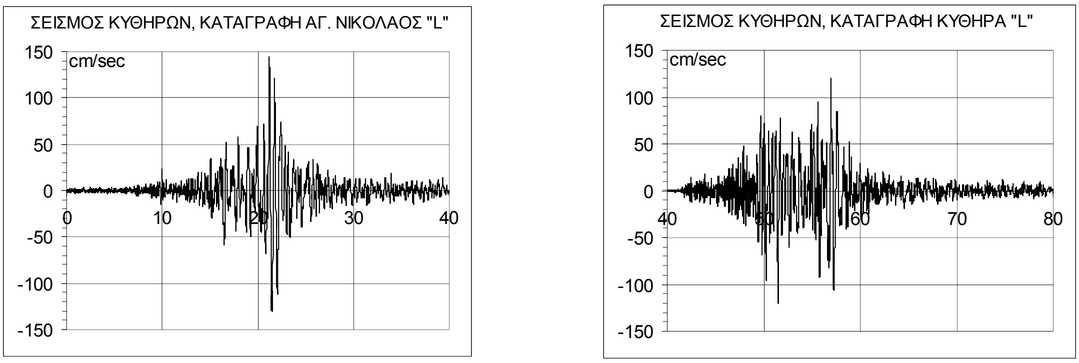


Figure 6. Accelerograms of Kithira earthquake. Left: records at Agios Nikolaos, Lakonia. Right:Records at Kithira.

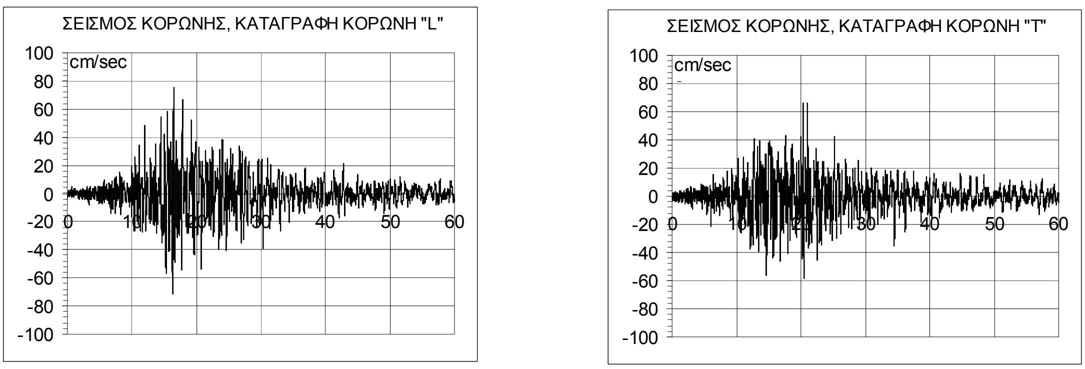
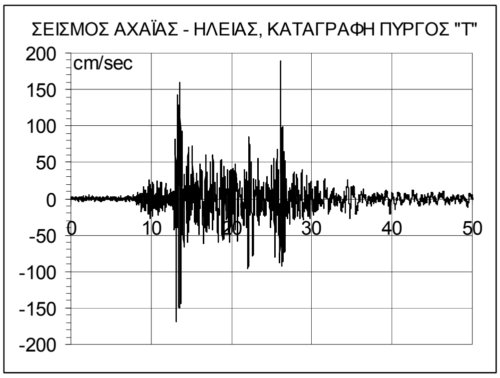


Figure 7. Accelerograms of Koroni earthquake. Left: Longitudinal (L) direction. Right: Transverse (T) direction.



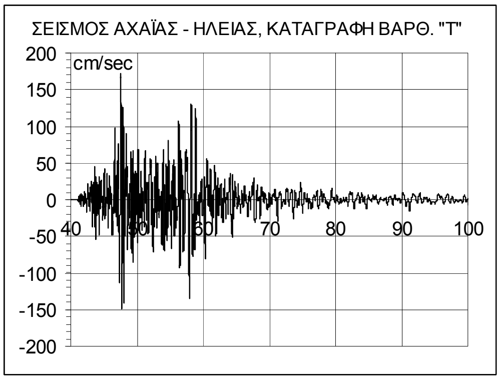


Figure 8. Accelerograms of Achaia – Ilia earthquake. Left: "T" Record at Pyrgos. Right: "T" Record at Vartholomio

Figure 9. Response spectrums for Kithira earthquake. Bold line "L", "T" records at Agios Nikolaos, Lakonia. Smooth line "L", "T" records at Kithira. (ζ=0.05)

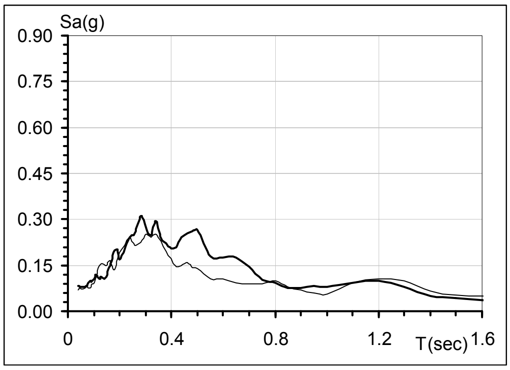


Figure 10. Response spectrums for Koroni earthquake. (ζ=0.05)

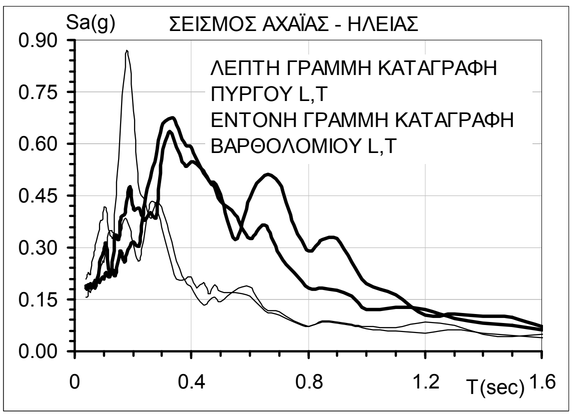


Figure 11. Response spectrums for Achaia - Ilia earthquake. Bold line "L", "T" records at Vartholomio. Smooth line "L", "T" records at Pyrgos. (ζ=0.05)

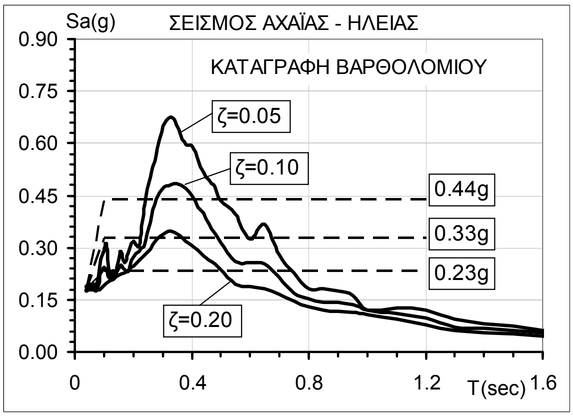


Figure 12. Response spectrums of Achaia – Ilia earthquake from records at Vartholomio (continuous lines) for 5%, 10%, 20% damping. Normalization of response spectrums at 66% of the maximum value (discontinuous lines).

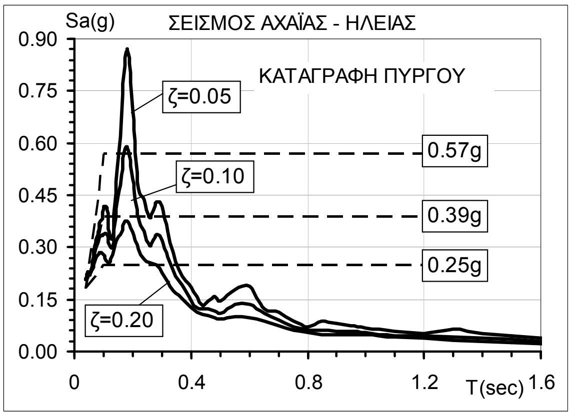


Figure 13. Response spectrums of Achaia – Ilia earthquake from records at Pyrgos (continuous lines) for 5%, 10%, 20% damping. Normalization of response spectrums at 66% of the maximum value (discontinuous lines).

4 CONCLUSIONS

Through present studies, result conclusions about the seismic loads that should be used for the seismic evaluation of monumental masonry buildings. The seismic loads that are suggested by modern seismic codes have usually very high values. This happen even in the cases that in these codes are foreseen provisions for existing masonry buildings (e.g. Eurocode 8, Part 3). The proposed provisions were instituted for new constructed buildings, where can be easily designed to withstand such loads.

- Through the literature review is concluded that existing masonry monumental buildings can develop an “equivalent” damping (both viscous and “hysteretic”) about 20%. Also, was found that the actual initial eigenfrequency of these buildings is reduced at 2/3 to 1/2 of calculated analytical one. The resulted reduction of the actual stiffness is at least 50% of the analytically calculated one. These reductions happen close to the bending point of the bilinear elastoplastic diagram that represents the response of the building to horizontal loading and in generally is located before the performance point for the check of the deformations of the piers and spandrels.
- Through the nonlinear analysis of masonry frames, resulted that these frames can resist horizontal loading about the 25%.g of their total mass. This value was verified for the models with frame elements as well as for the models with discrete elements (brick – masonry). For the frames with continuous elements the aforementioned value was higher. Also higher value was found, when the seismic loads were calculated as a percentage of the mass at each node (ACC) in comparison with the other two loadcases (MODE, LOAD).
- From the postpossessing of recorded earthquakes resulted spectral acceleration about 70%g for 5% damping. The normalized value was 50%g. By considering “equivalent” damping 20% (as defined after literature review) the normalized spectral accelerations resulted 25%g that are close to the value of previous conclusion. These values are considered rational and were resulted by the postprocessing of Achaia – Ilia recorded earthquake, where only damages and no collapses of monuments were observed.

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