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DAMAGE ASSESSMENT OF MASONRY INFILLED FRAMES

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ABSTRACT

There are many different parameters that affect the behavior of masonry-infilled reinforced concrete frames under earthquake loadings and it is difficult to predict. Selection of the parameters, that have the greatest impact on the response system, was carried out on the EDIF database of 113 one- storey one- bay infilled frame tests processed by statistical sensitivity analysis. Those parameters were height to length ratio, longitudinal reinforcement ratio of column and compressive strengths of concrete and masonry infill. A simplified and reliable analytical model, based on the equivalent diagonal and validated on different experimental tests, has been adopted for extensive studies on seismic vulnerability of infilled frame buildings. We investigated the seismic response and the influence of the masonry infills on the global structural response in terms of inter-storey drift ratio and damage states by incremental dynamic analysis. Results are presented in order to determine the relationship between infill's type and damage state for low-rise infilled framed buildings.

INTRODUCTION

The buildings with reinforced-concrete frames infilled with masonry ("framed-masonry") under earthquake excitation behave neither as frames nor as confined masonry walls. Their behaviour depends on many different parameters and on the expected drift. The masonry infill wall within the reinforced-concrete frame increases system stiffness and strength, at small and medium drifts. It also could cause irregularities, in both geometrical and mechanical distribution of infills, and could lead to undesirable failure mechanisms that compromise the bearing capacity of the structure. Due to the incomplete understanding of such composite structures there is a lack of code guidance. That limits the engineer's ability to design these structures taking into account both, the beneficial and detrimental properties.

In order to increase the understanding of the „framed-masonry"structural elements we have collected the experimental database of infilled frames –EDIF. It has been organized in a way that could be used for the performance evaluation and for calibration of the numerical model. Due to the high variability of mechanical properties of masonry and their uncertainty, their properties were divided into three groups that represented a wide number of possible situations that could happen in practice. A lower bound was represented by weak infill, the upper bound with strong infill and medium infill panel had a range of compressive strengths between them. Further, we parametrically investigated the seismic response "framed-masonry" structures, with the numerical model that was simple enough in the computation and elaboration of results, and sufficiently refined for adequate simulation of the main behavioural characteristics. Such reliable and calibrated numerical model was used for the parametric analysis and damage assessment of the "framed-masonry" structures.

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On the basis of EDIF database and parametric analysis done with the calibrated numerical model a practical displacement-based damage evaluation methodology has been suggested.

EXPERIMENTAL DATABASE OF INFILLED FRAMES

The collected experimental database (Experimental Database of Infilled Frames – EDIF) contains data of 113 published tests of the basic ‘‘framed-masonry’’ composite system. It contains all available input and output data systematized in a way applicable for performance evaluation. Input data were selected considering their availability and uniformity. Primary geometrical properties of the frames were span length (L) and story height (H). Dimensions of the frame elements and longitudinal reinforcement in the columns were taken as secondary parameters expressed as moments of inertia and reinforcement ratios. All parameters were expressed as dimensionless in all possible cases (a= height to length ratio, b= ratio of moments of inertia of beam to column, g= ratio of column width to the thickness of masonry infill, r_b = longitudinal reinforcement ratio of beam and r_c = longitudinal reinforcement ratio of column). The material properties of the frames consisted of the concrete compressive strength (f_{ck}) and its modulus of elasticity (E_c) as well as steel yield strength (f_y).The properties of the masonry infill were compressive strength (f_k) modulus of elasticity (E_i)and thickness of the masonry infill wall (t).The applied lateral loading sequence in the tests was either static–monotonic, cyclic or seismic excitation and the chosen parameters in the EDIF database were horizontal loading(monotonic or cyclic) and axial vertical load (N) applied to the columns.

For representing the behavior of the framed-masonry composite measured resistance envelope curve was replaced by bilinear curve obtained by the equal energy rule (Figure 1.). Two points on the idealized bilinear primary curve, important for description of the system behavior, could be obtained from the EDIF database by using the Artificial Neural Network algorithms trained on it, that is yield (V_y and d_y) and capping point (V_u and d_u) as well as the Failure mode (FM).

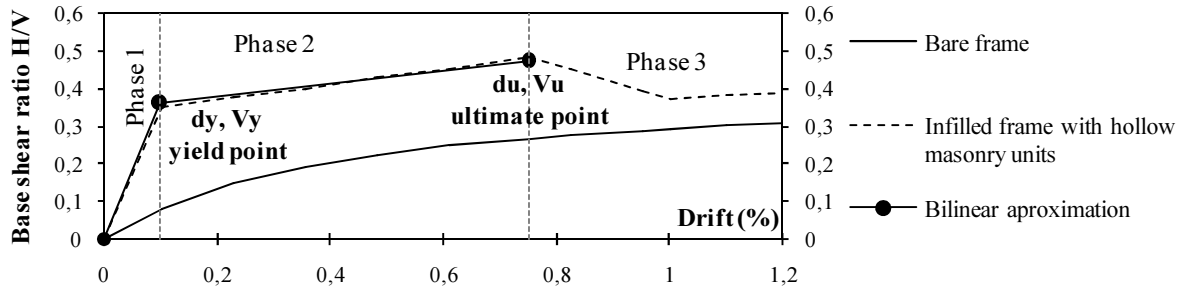

Figure1. Bilinear approximation of the primary curve in the EDIF database

A statistical sensitivity analysis, based on the forward stepwise method, was used for elimination of the unimportant parameters in the learning process of the ANN (Kalman Šipoš at all, 2013). Forward stepwise method assesses the change in the mean squared error of the network by sequentially adding input neurons to the neural network (rebuilding the neural network at each step). According to the results of the ANN, the most important input parameters were: a= height to length ratio, r_c = longitudinal reinforcement ratio of column and compressive strengths of concrete (f_{ck}) and masonry infill (f_k).

CALIBRATION AND VALIDATION OF THE NUMERICAL MODEL

The behaviour of the “framed-masonry” structural elements has been analysed using the existing nonlinear element models used for modelling the frame and infill-panel elements. Its calibration was done on three samples taken from the EDIF database, and its validation was performed on the model of multi-storey multi-bay “framed-masonry” building structures tested in the European

laboratories. It has been proven that calibrated model could be applied with sufficient reliability, for the parametric analysis of the real "framed-masonry" buildings.

All numerical analyses were performed by the Seismostruct v6.5 (2013). RC frame elements were modeled as force-based elements with concentrated plastic hinges at the ends. The nonlinear confined concrete model and the nonlinear model for reinforcing steel were used. The masonry infill wall was modeled as the infill panel model with calibrated parameters of hysteretic behavior. The initial diagonal width, w_1, was determined according to proposal of Stafford Smith and Carter (1969). It is based on the parameter λ_h which presents a measure of the relative stiffness of the frame to infill. Reduced area of the compressed diagonal depends on the stiffness coefficient λ_h, according to recommendations of Decanini (1986). Corresponding deformations were determined according to the limit states: start of reduction of the initial area A_{ms1} corresponds to deformation at the end of linear elastic behaviour ($\varepsilon_m/3$) while A_{ms2} secondary area is reached at 70% of the maximum compressive stress and the associated strain corresponds to the $1.5\times\varepsilon_m$ (Figure 2).

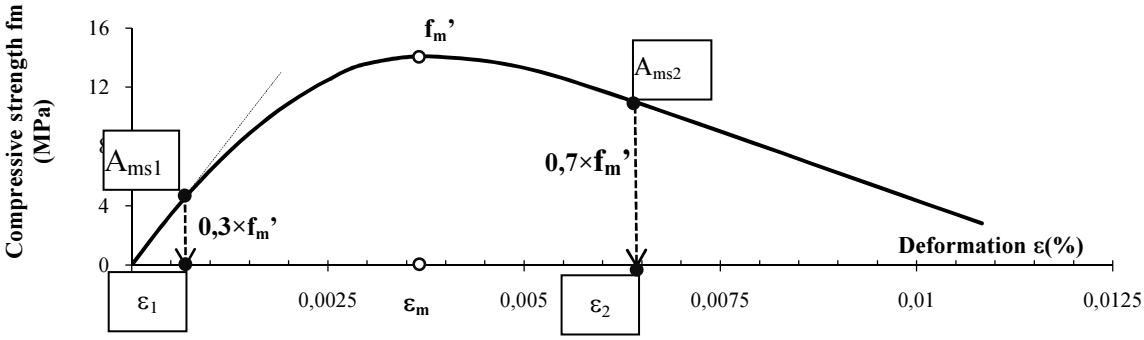

Figure 2. Variability of the areas and associated deformations

Table 1. Material and geometric properties of the experimental samples

	Parameter	Legend	**N2**	**C1**	**M2**
	l (m)	frame span	2.5	1.9	2.3
	h(m)	frame height	1.425	1.425	1.61
	b/h_{stup} (m)	column dimensions	0.2/0.2	0.2/0.2	0.22/0.22
	I_{stup} (m^4)	moment of inertia of column	1.3E-04	1.3E-04	2E-04
	E_c(kN/m^2)	concrete modulus of elasticity	3.56E+07	3.53E+07	1.26E+07
	h' (m)	height of the infill	1.3	1.3	1.5
	l' (m)	length of the infill	2.3	1.7	2.08
	Q (°)	angle of diagonal	29.48	37.41	35.80
	t(m)	thickness of infill	0.16	0.12	0.12
	d(m)	length of diagonal	2.64	2.14	2.56
	f_k(kN/m^2)	masonry compressive strength	2740	5100	14400
	E_i (kN/m^2)	masonry modulus of elasticity	1.21E+06	4.23E+06	5.48E+06
SSC (1969)	λ_h	stiffness parameter	2.30	3.01	4.11
	w	initial width of diagonal	0.46	0.56	0.77
	A_{ms1}	initial area of diagonal	0.46	0.56	0.77
Decanini (1986)	A_{ms2}=%A_{ms1}	secondary area of diagonal	77.4	73.4	68.2
Kaushik (2007)	ε_m	deformation that corresponds to the maximum value of the compressive strength	0.0024	0.0027	0.0039
Figure 2.	ε_1	deformation that corresponds to the initial area	0.00082	0.00089	0.0013
	ε_2	deformation that corresponds to the secondary area	0.0037	0.004	0.0059

Three samples from the EDIF database were selected according to the compressive strength of the masonry infill as the samples with weak, medium and strong infill type. From the experiments conducted by Colangelo (2005) taken was one sample as frame with weak infill (N2) and the other as frame with medium infill type (C1). The sample with strong infill type (M2) was tested by Žarnić (1985). All three samples were produced and tested in a scale 1:2.

Experimental results are presented as primary load-displacement curve characterized by two points: yielding and ultimate capacity of the framed-masonry element. The quality of the calculated results, obtained by described method, is evaluated by their relative error to the experimentally obtained values for the characteristic values (displacements and forces) in Table 2. According to the results relative average error for all four parameters was 8%, while largest error was 12% for the yield drift.

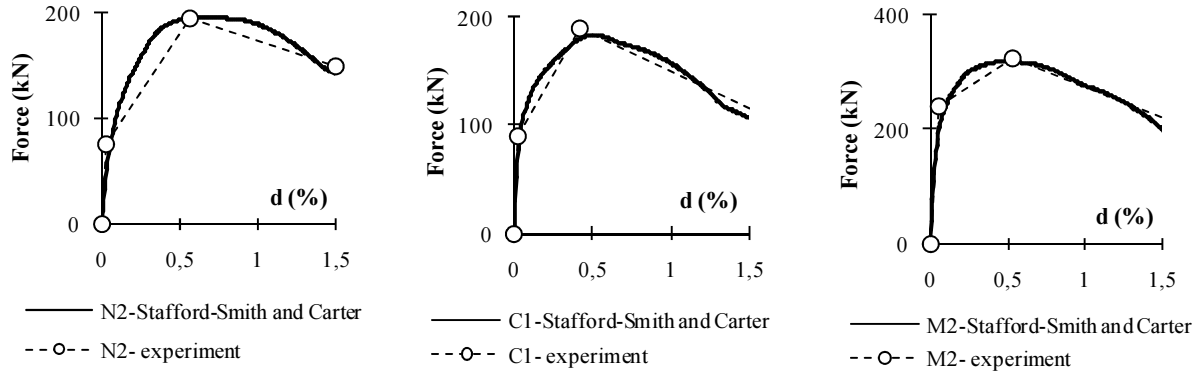

Figure 3.Calculated and measured results of the samples

Table 2. Evaluation of the calibration for samples

		N2		C1		M2	
		exp	SSC	exp	SSC	exp	SSC
d_y (%)	yield drift	0.035	0.04{14}[1]	0.029	0.032 {10}[1]	0.042	0.047{12}[1]
d_u (%)	ultimate drift	0.561	0.6{7}	0.42	0.45{7}	0.53	0.49{8}
V_y (kN)	yield force	75	67{11}	90	87{3}	240	202{16}
V_u (kN)	ultimate force	195	194{1}	190	183 {4}	324	319{2}

1 - Values in brackets {} represent the relative errors expressed in percentage of values determined numerically in relation to the experimental ones

After calibration of the numerical model, its validation was done on a three story building named PATRAS (Fardis, 1997.). The building was designed according to Eurocodes for intermediate level of ductility (DCM) for the design ground acceleration of 0.3g with class C25/30 concrete and reinforcement S500.It had two spansof 4m and 6m, first floor storey height was 3.5 meters and other floors were 3 meters high. The side columns were 40/40 cm and the central one was 45/45 cm. The beams were 30/45 cm and the slab was 15 cm thick. Compressive strength of concrete ranged from 24.1 to 51.8 MPa, the yield strength of reinforcement was 55.5 MPa, masonry infill units were UNIBRICK 11.2 cm with vertical holes and the compressive strength of the infill was 2.4 MPa with elastic modulus of 2.5 GPa .The building was tested in 1:1 scale under pseudo-dynamic excitation (Verzeletti et al, 1997) that represented the Eurocode 8 response spectrum with a maximum ground acceleration of 0.45g.

The numerical model of the Patras building is shown in Figure 4 and the analysis was performed in Seismostruct v6,5 (2013) by use of the previously calibrated data, as explained (Table 3).

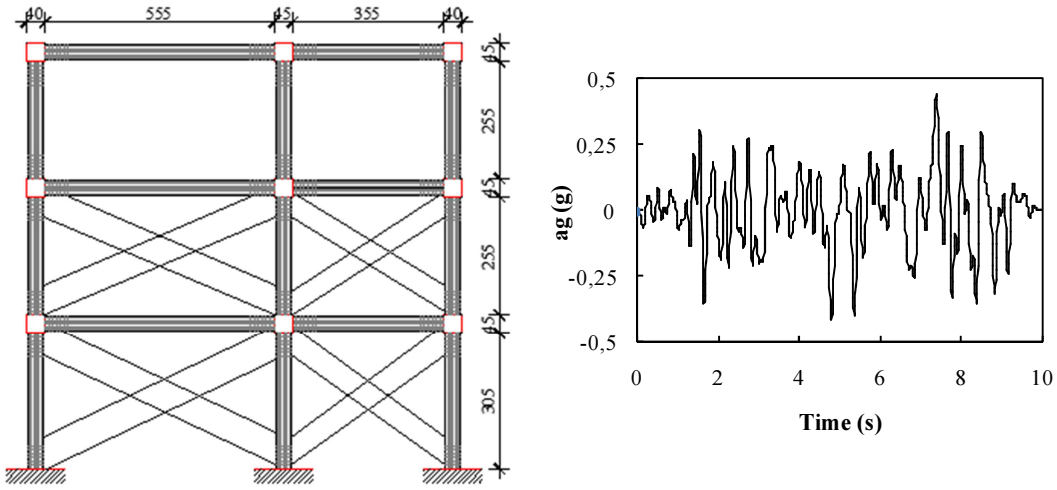

Figure 4. Numerical model of the PATRAS building with the excitation time history

Table 3.Material and geometric properties of the masonry infill

PATRAS	f_k (MPa)	E_i (MPa)	ε_m	ε_u	ε_1	ε_2	λ_h	$f_{m\theta}$* (MPa)	A_{ms1} (m^2)	A_{ms2} (%A_{ms1})
ground (4m)	2.4	2500	0.0032	0.0088	0.00107	0.0048	2.51	0.273	0.216	76.17
ground (6m)							2.33	0.283	0.281	76.64
1. floor(4m)							2.22	0.258	0.199	77.83
1. floor(6m)							2.12	0.263	0.229	78.47

The calculated and experimental results are compared by the relative errors of the maximum and minimum floor displacement and base shear and by their correlation values. Based on the relationship shown in the Figure 5 to 7 the maximum relative error was 8% and the mean relative error was 4% for floor displacements and 2% for the base shear. Correlation values in all cases showed values higher than 0.9.

It could be concluded that the adopted numerical model was capable of predicting the behaviour of the real structure exposed to earthquake excitation.

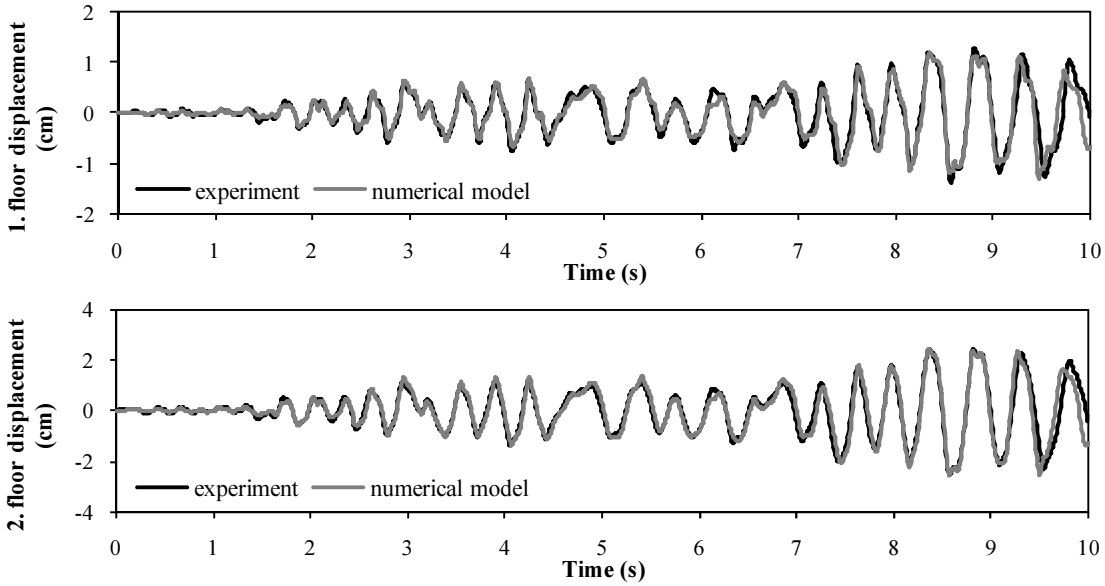

Figure 5. Measured and calculated displacements of the first and second floor

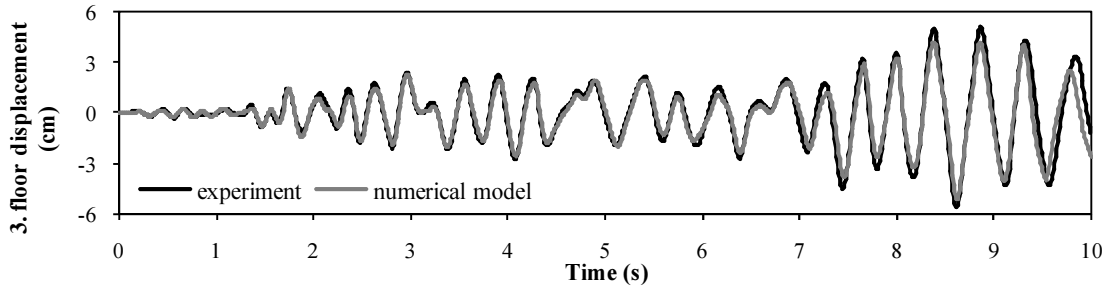

Figure 6. Measured and calculated displacements of the third floor

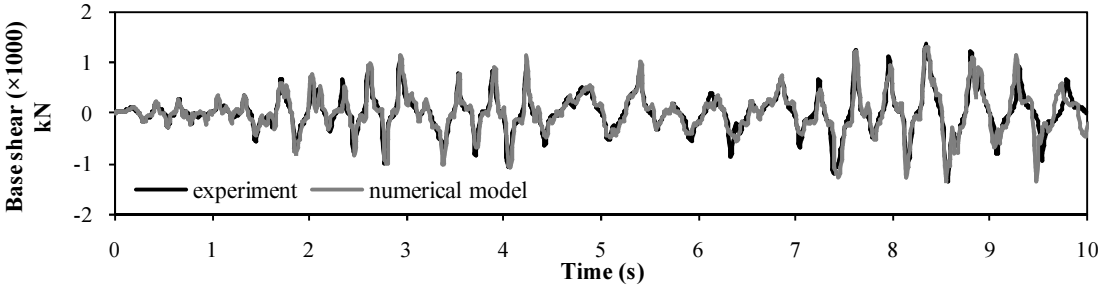

Figure 7. Measured and calculated base shears

PARAMETRIC ANALYSIS

The previously explained calibration process turned out the reliable numerical tool and valuable support for further extensive parametric investigations of the framed-masonry buildings with an aim of developing a methodology for damage assessment of the framed-masonry structures.

Model building was a three-storey RC framed-masonry building with parameters of the masonry infill type according to the results of the forward stepwise method. Ground floor was 3.75 m, and other floors were 3m high (Figure 7). Bay span was 4 m, columns were 40/40cm and the beams 30/50. The masonry infill type was varied according to its compressive strength as weak, medium and strong infill (1.17 MPa, 2.92 MPa and 5.01 MPa).

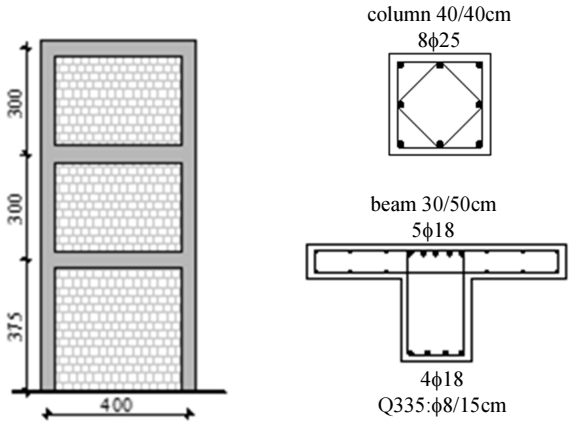

Figure 8. The model building with element dimensions

In order to simplify the design of framed-masonry buildings, we correlated the required capacity and area of framed-masonry in relation to the area of floors by the surface ratio (Equation (1).

Analyzed infilled frame(framed-masonry) area (A_{IF}= A_I+A_{FR}; A_I=infill's area; A_{FR}=frame area) in the direction of the earthquake and the corresponding floor area (A_F), were expressed in percentage (ρ = 3%, 4%, 5%) ratios.

$$\rho = \frac{A_{IF}}{A_F} \tag{1}$$

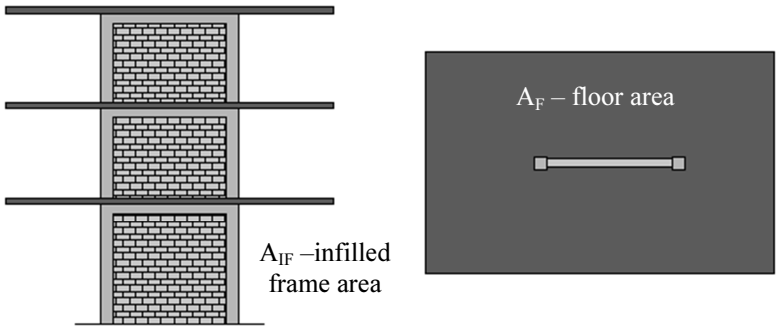

Figure 9. Surface ratio

For the seismic analysis we used real earthquake records sets selected from the European Strong-Motion Database. For their selection the Rexel v3.5. software(Iervolino, 2010.) was used. It gave all possible combinations of seven earthquake records and found the set compatible with the design spectrum of Eurocode 8 with the smallest possible deviations for three different peak ground accelerations (three seismic zones, namely 0.1g, 0.2g, 0.3g). These deviations were 0.053, 0.043 and 0.038 .

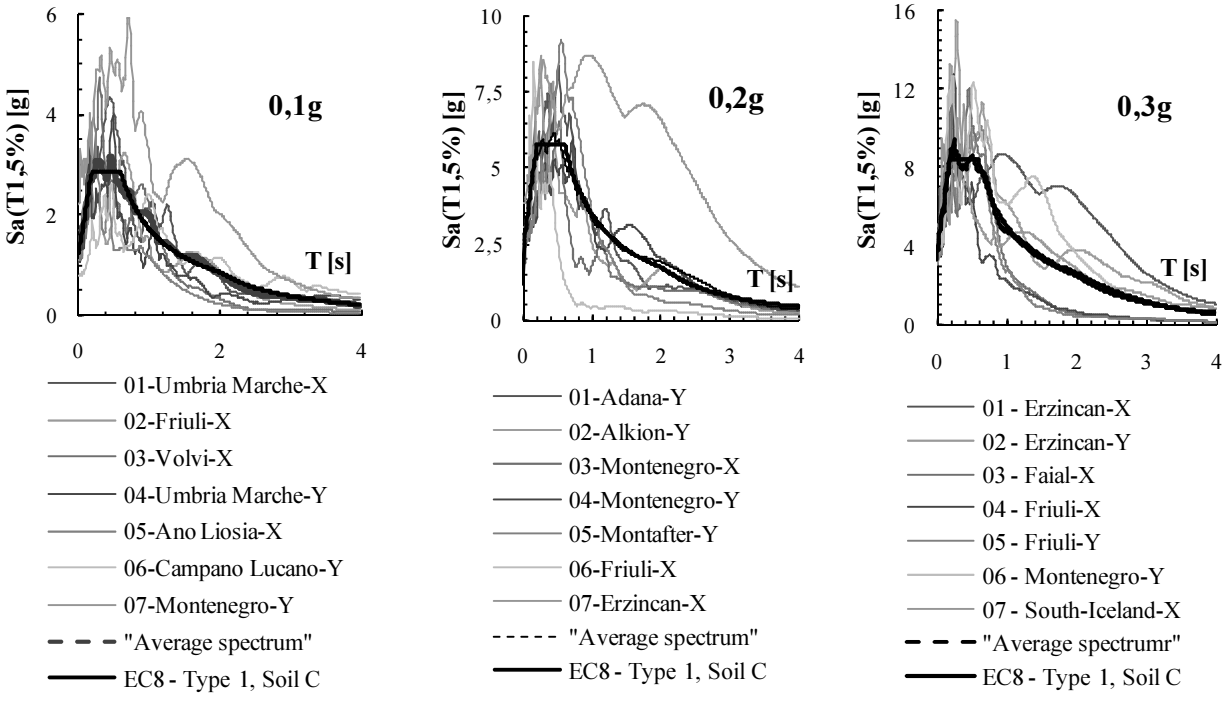

Figure 10. Sets of earthquake records for three seismic zones

DAMAGE ASSESSMENT METHODOLOGY

In order to evaluate the calculated structural behavior of framed-masonry buildings four damage states, “Slight”, “Moderate”, “Extensive”, and “Near collapse” were used (Table 4). The limit values in terms of inter–storey drift ratio (IDR) for each limit state were determined on the basis of data from the EDIF experimental database.

Table 4. Damage states and corresponding inter–storey drift ratio range, IDR (%)

Damage state	Description	IDR (%)
Slight damage	Fine cracks in infill	IDR<0,10
Moderate damage	Cracking at infill–frame interfaces	0.10≤IDR<0.30
Extensive damage	Extensive cracking of infills, falling masonry units, out–of–plane bulging	0.30≤IDR<0.75
Near collapse	Partial failure of many infills, damage in frame members, some fail in shear	IDR≥0.75

By definition of the damage states and incremental dynamic analysis of the nonlinear numerical models we obtained the results presented thru the fragility curves. They represent discrete limit state probabilities with consideration of a level of structural damage for assessment of the seismic fragility.

For each model building the incremental dynamic analysis curves were calculated according to the variation of inter-story drift ratio in relation to the intensity measure and spectral acceleration (T_1,5%) for the set of seven earthquake records corresponding to the particular earthquake intensity. For each limit state and for each earthquake values from IDA curves, a cumulative vulnerability curve were presented as lognormal function. The probable level of structural damage, for a given period and spectral acceleration of medium response spectrum for specific earthquake seismic zone, is the condition with the highest probability of occurrence (Figure 11).

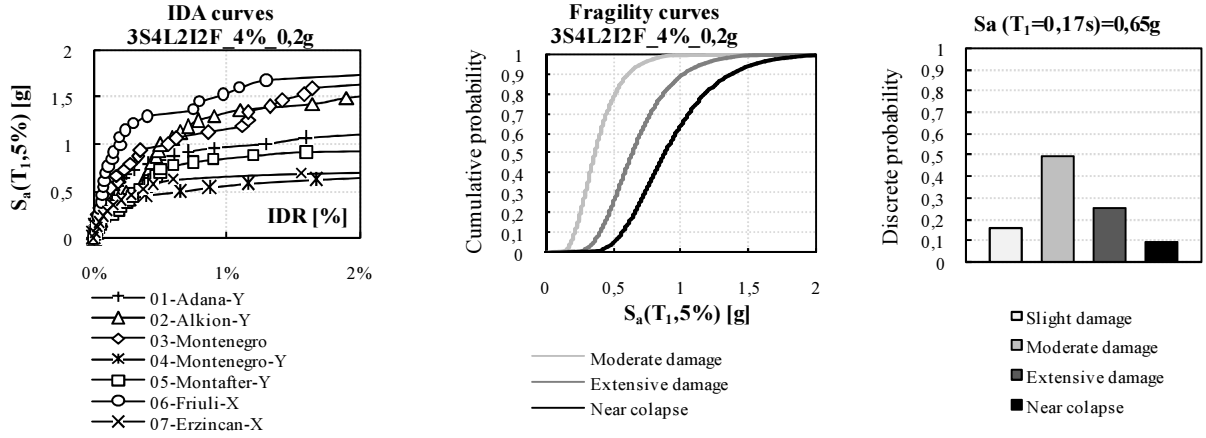

Figure 11. IDA curves, fragility curves and definition of relevant damage state

The results of 27 model buildings obtained from the incremental dynamic analysis are summarized and presented according to the level of structural damage in Figure 12. They indicate an expected trend of damage increase in the direction of a stronger seismic load. Damage could be reduced by increasing the ρ ratio and/or by using the higher strength of the masonry infill wall. Three-story framed-wall building, with no or little damage, could be located in the area of the peak seismic acceleration of up to 0.1 g with a compulsory use of the medium and strong infill with compressive strength higher than 2.5 MPa.

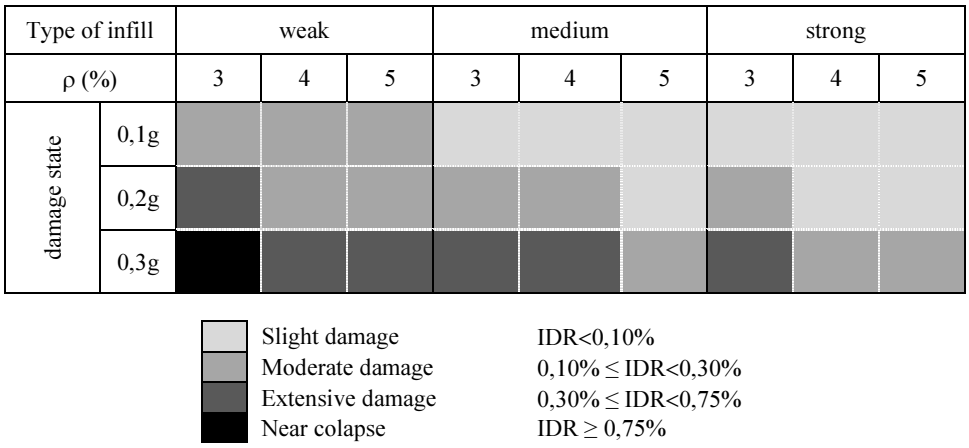

Figure 12. IDA results of the model building with three different masonry strengths and three peak ground acceleration levels expressed by damage levels

In order to further observe the trend of damage according to the building's height and variation of inter-story drift ratios by floors, the IDA results have been presented for every earthquake record for particular seismic zone (with normalized scale factor) in Figure 13. The damage was expressed by the mean inter-story drift ratio per floor. Higher values of IDR (%) were in the lower half of the building. The range of values for the lowest seismic zone was 0.03-0.19 %, from 0.05 to 0.61 % for the second, and from 0.08 to 1.88 % for the third seismic zone. It depended on the combination of the applied masonry types with surface ratio, ρ. This also revealed the importance of applying the surface ratio, ρ, in the numerical model. Its lower value led to the increased damage. The comparison of the damage levels from the incremental dynamic analysis (Figure 12) and damages according to the maximum value of the individual building inter-story drift ratio from time history analysis (Figure 13) presents a very good compatibility. It differed only in 18% of buildings with small deviations from the limit values of IDR (%).

frame	infill	floor	ρ=3%			ρ=4%			ρ=5%		
			0.1g	0.2g	0.3g	0.1g	0.2g	0.3g	0.1g	0.2g	0.3g
Inter-story drift ratio (%)	weak	3	0.14	0.29	0.34	0.08	0.17	0.24	0.07	0.13	0.22
		2	0.17	0.48	1.15	0.13	0.31	0.69	0.11	0.23	0.41
		1	0.13	0.36	0.76	0.09	0.25	0.45	0.08	0.18	0.29
	medium	3	0.07	0.22	0.34	0.05	0.12	0.3	0.04	0.05	0.18
		2	0.11	0.52	0.73	0.08	0.33	0.71	0.06	0.12	0.43
		1	0.10	0.33	0.67	0.07	0.22	0.51	0.04	0.14	0.33
	strong	3	0.06	0.19	0.32	0.04	0.11	0.31	0.03	0.05	0.08
		2	0.07	0.34	0.70	0.08	0.27	0.42	0.05	0.08	0.16
		1	0.08	0.24	0.48	0.07	0.21	0.39	0.04	0.08	0.14

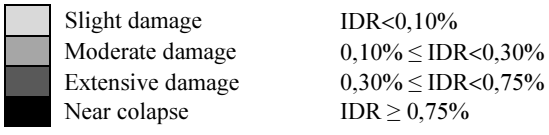

Figure 13. IDR (%) values for normalized factor of IDA analysis

The results are presented in a way to establish the relationship between the infill type and damage state for the analyzed three story (low-rise) framed-masonry buildings.

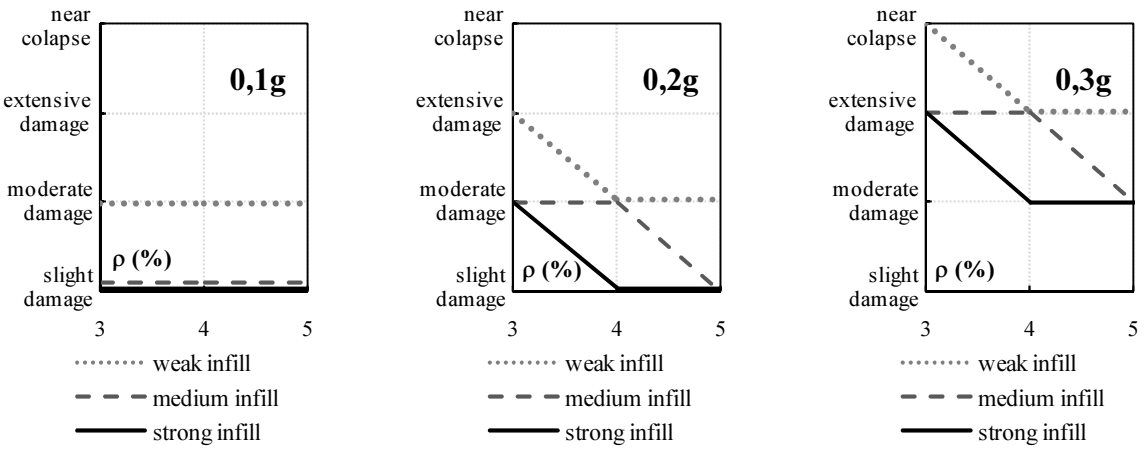

Figure 14. Damage states in relation to the infill's strength

According to the results shown in the Figure 14 it is obvious that the behavior of low-rise framed-masonry building is completely defined by the type of masonry infill. Correspondence of results for 0.2g and 0.3g peak ground acceleration indicated that damage states could be reliably predicted with the difference in the first higher damage level.

CONCLUSIONS

The assessment methodology for low rise framed-masonry buildings has been outlined. It was arrived at by use of the calibrated numerical model. It has been calibrated on the experimental database of tested framed-masonry elements and further validated on the tested full-scale building. The numerical model was used for parametric analysis of these structural elements with particular attention focused on the effect of mechanical properties of infill panels and their influence to the global structural response. Three different strengths of masonry infill walls were: weak, medium and strong. This choice was also made with the purpose of obtaining an upper and lower limit of the results, thus giving a reasonable range for the evaluation of their seismic response.

The results of analyses performed on the infilled frame showed that the typology of masonry panels, whether weak or strong, has a moderate influence on the global structural response. The strength of the masonry infills gives a contribution to the maximum resistance of the building but, also the level of deformation experienced by the frame are moderately influenced by the characteristics of the infill panels. According to the damage states achieved in parametric analysis it can be concluded that the damage of infilled frames increases with increasing seismic load until the failure of infill, if the weak infill type is used. The use of medium and strong type of infill will reduce damage to the first lower level of damage states in relation to the application of a weak type. An importance of applying the surface ratio ρ in the numerical model was observed. Its lower value led to the increased damage (to the first higher level of damage states).

It is important to emphasise that an analysis has been performed for a fully and uniformly infilled frame, which structural response is strongly influenced by the presence of the infill panels.

A more exhaustive parametric analysis on masonry infilled structures, considering the effects of number of stories, types of reinforced concrete frames according to reinforcement ratios, infill dimensions, relative stiffness frame/infill, etc., would help to extend the analysis of infilled framed building model here proposed for a more wide range of conditions. This could lead to practical

methodology valid for a broad range of situations, which could be considered in future seismic structural assessment standards.

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