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SEISMIC VULNERABILITY OF EXISTING MASONRY BUILDINGS-PROJECT SEISMOWALL, RECENT RESULTS

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Abstract

A significant number of masonry buildings, built prior to existing seismic codes, nowadays are used and serve a function as public institutions, namely schools, administration offices, courts, museums, theatres, etc. The necessity for evaluation of the seismic risk of these existing buildings is of high priority. In the framework of the research project SeismoWall, sixteen representative masonry buildings were selected and studied. The investigated buildings with their architectural layout, structural system and materials are typical for the buildings built between the end of the nineteenth and the beginning of twentieth century, not only in the country, but in the wider region of the Balkan Peninsula.

The research activities in the project are divided in four main work packages: WP1-Selection of representative buildings and their static and seismic analysis, WP2-Experimental analysis of the mechanical properties of constituent components of the buildings and ambient vibration testing (AVT), WP3-correlation of numerical and experimental data and calibration of the dynamic characteristics of the buildings with the results from AVT and WP4-Determination of vulnerability curves for the selected masonry buildings.

The main aim of the project SesimoWall is to define a series of seismic vulnerability curves for four classes of masonry buildings (unreinforced masonry with rigid/flexible floors, regular/irregular plan layout) for five geographical regions in Republic of North Macedonia with distinctive severe seismic hazard.

Keywords: Masonry Structures; Ambient Vibration Tests; Seismic Hazard; Seismic Vulnerability

1. Introduction

The series of earthquakes that affected the city of Skopje and its surrounding in the autumn of year 2016 and the Ohrid region in the summer of year 2017 have strongly disturbed the community in our country. Having in mind the low to medium intensity of these events, the effects they produced have seriously imposed the question about the level of implemented risk management in the country.

In historical point of view, the devastating 1963 Skopje earthquake initiated a lot to be invested, with support of the world elite in the field of earthquake engineering, not only in the renewal of the city of Skopje but also in establishing a system for managing this type of natural hazards. The triggered public awareness for earthquake hazards in the proceeding days of the tragic event led to various experts' findings and discussions, which can be summarized in two general conclusions: i) the seismic risk management is dominated by prevention, or only limited to design and construction of new buildings and ii) the mitigation of seismic risk of existing structures is seriously neglected. The prevention aspect is provided by the regulations for design of seismic resistant structures which are dating from the 1980s. However, the number of existing buildings in the country, especially the ones built prior to any seismic regulation, is significant. Therefore, it is not a surprise that even during minor earthquakes, like the recent ones, exactly such type of buildings have suffered a certain amount of nonstructural and structural damage. These considerations have directed the scientific society in the country towards analysis of seismic vulnerability of existing structures. The first experiences in the field of seismic vulnerability assessment in Republic of North Macedonia date from the early 1990-ies in Petrovski J. et al. [1]. Following this, Nocevski [2] in his Doctoral thesis presented methodologies for the definition of empirical and analytical vulnerability functions. Dumova-Jovanoska [3] proposed an analytical method for the development of *earthquake intensity* - *damage* relations, specified as fragility curves and damage probability matrices. The proposed method was applied on reinforced concrete frame-wall structures. Milutinovic and Trendafiloski [4] in the frame of a Risk-UE European project have developed an integral approach for estimation of seismic behavior and vulnerability of RC structures.

In this context, within this study the authoring team has focused to existing buildings built at the end of the 19th until the mid-20th century (before the devastating 1963 Skopje earthquake), which nowadays represent public institutions in the field of education, culture and administration. The main goal of the research project Seismic Vulnerability of Existing Masonry Buildings (SEISMOWALL 2017-2019) is to provide geographically sensitive vulnerability curves for this type of buildings. The Seismological Observatory of Republic of North Macedonia and the Department of Mathematics and Geosciences at the University of Trieste joined the project activities, further contributing to the definition of the region specific seismic hazard. Having in mind that the focal point are buildings from the first half of the 20th century, when there was no awareness for taking into account earthquake loads in the design of buildings, the dominant structural material is unreinforced masonry in combination with a limited use of reinforced concrete (mainly in floor structures and tie beams). As a first step, and with the support of local authorities from several cities characterized with medium to serious seismic risk, namely Skopje, Bitola, Ohrid, Debar, Kavadarci and Gevgelija, representative buildings were selected. The locations are presented on the map of the country (Fig. 1). The selected buildings include a kindergarten, four primary schools, a high school, three museums, a film agency, a local court, a post office, a sport hall, a railway station, a town hall and a health care facility. Regarding their structural systems these buildings are categorized as M5 - U masonry (old bricks) and M6 - U masonry RC floors types according to the classification system adopted in Risc-UE project [5]. With only visual inspection of the layout and elevation of the buildings it becomes evident that most of them are irregular, however regularity control according to EN 1998-1 [6] recommendations verifies that. This comes as no surprise having in mind that most of them are located in the cities' old parts, where they had to be built in a limited space and rough terrain. Therefore, the selected structures are divided as regular and irregular, Tables 1 and 2 present this classification.

Fig. 1 - Map with locations of the representative buildings

Table 1 - Selected building type M5 -U masonry (old bricks) (H is horizontal, V is vertical)

Building	City	Year of build	Floors	Height (m)	Irregularity	Type
Sch. "St. Kliment Ohridski"	Ohrid	1910	3	11.95	No	A
Mental health center	Gevgelija	1910	2	7.25	No	A
Museum of Alban. alph.	Bitola	1921	3	12.16	No	B
Sports center "Partizani"	Debar	1930	2	9.80	H and V	/
Train station	Gevgelija	1880	3	10.30	H and V	B
Sch. "St. Sava"	Ohrid	1900	3	14.25	H and V	A
Museum	Ohrid	1929	4	10	H and V	B

Table 2 - Selected buildings type M6- U masonry RC floors (H is horizontal, V is vertical)

Building	City	Year of build	Floors	Height (m)	Irregularity	Type
Film agency of RM	Skopje	1950	2	9.70	No	B
Museum	Bitola	1928	3	11.25	No	/
Sch. "Strasho Pindzur"	Kavadartsi	1941	3	10.60	H and V	A
Sch. "Vojdan Cernodrinski"	Skopje	1955	4	11.70	H	A
Municipality building	Kavadartsi	1955	3	11.30	H	/
Kindergarten "Pepelashka"	Skopje	1920	2	10.08	H	/
Basic court	Kavadartsi	1945	2	14.28	H	B
Sch. "Dimkata Angelov G."	Kavadartsi	1928	3	15.70	H and V	A
Post office/Telekom	Debar	1935	3	10.00	H	B

2. Organization of the SEISMOWALL project

The research program is organized as 4 Working Packages (WP). Following sections provide an overview of the important goals and outputs for each package.

2.1 WP1: Selection and analysis of representative buildings

For most of the selected buildings there are no relevant design documents, often they were scarce and far from easily applicable. Therefore, the goal was to produce drawings of structural and non-structural elements of the buildings as detailed as possible. To accomplish this, in-situ geometry measurements, comparison with available documents, and communication with occupants to get information of possible interventions either for repair/strengthening or adaptation of the buildings were conducted.

The key point of this WP is the definition of three-dimensional FE models of the selected structures for further utilization for linear static and modal analysis, Fig. 3. SAP2000 [7] is used as a calculation tool. The mechanical properties of the masonry are assumed in line with previous experiences (Churilov and Dumova-Jovanoska, [8]) as well as recommendations proposed in Eurocode 6 [9] and Tomazevič [10]. The modeling of flexible floors was also a considerable challenge [11]. One of the project goals was to verify the seismic resistance of the structures, built before the Skopje earthquake, according to relevant actual codes. To this end, static and seismic analysis of the structures is performed according to current Regulation PIOVSP'81 [12].

It is important to note that the analysis and distribution of seismic forces in PIOVSP'81 [12] is provided for an assumed 2D model of a structure, where structural bearing walls are continuous in height, without openings. Therefore, an adjustment of the proposed methods for more detailed and up-to-date 3D FE models is necessary. More precisely, the current standards provide Equations 1 and 2 for calculating principal tensile stresses in structural walls, as well as control of the walls' seismic capacity:

$$\sigma_n = \sqrt{\sigma_0^2/4 + (1{,}5 \cdot \tau_0)^2} - \sigma_0/2 \geq \sigma_{n,doz} \tag{1}$$

$$\tau_{ult} = \sigma_{n,ult}/1{,}5 \cdot \sqrt{1 + \sigma_0/\sigma_{n,ult}} \tag{2}$$

where σ_n - principal tensile stresses, $\sigma_{n,doz}$ - allowed principal tensile stresses (predefined in the code), σ_0 - normal stresses due to gravity loads and τ_0 - shear stresses due to seismic forces, τ_{ult} - ultimate shear stresses, $\sigma_{n,ult}$ - ultimate principal tensile stresses (predefined in the code) and σ_0 - normal stresses due to gravity loads.

However, available contemporary software tools enable a relatively simple generation of more complex 3D FE models. These spatial and more detailed modeling approaches introduce the effects from adjacent in-plane walls (between openings) and from the walls distributed in the orthogonal direction, which in turn imposes a different verification approach of the seismic resistance (not as proposed in Eq. (1) and Eq. (2)). Namely, the maximum FEM calculated principal stresses to be compared with allowed principal tensile stresses, and FEM-based minimum principal stresses to be compared with allowed principal pressure stresses.

In Fig. 4 a selected 3D FE model (SAP2000) and maximum and minimum principal stresses in bearing walls for one plane of the model are presented. The calculated modal characteristics of all selected structures are a valuable input for further calibration of the FE models with ambient vibration tests as well, included in following steps.

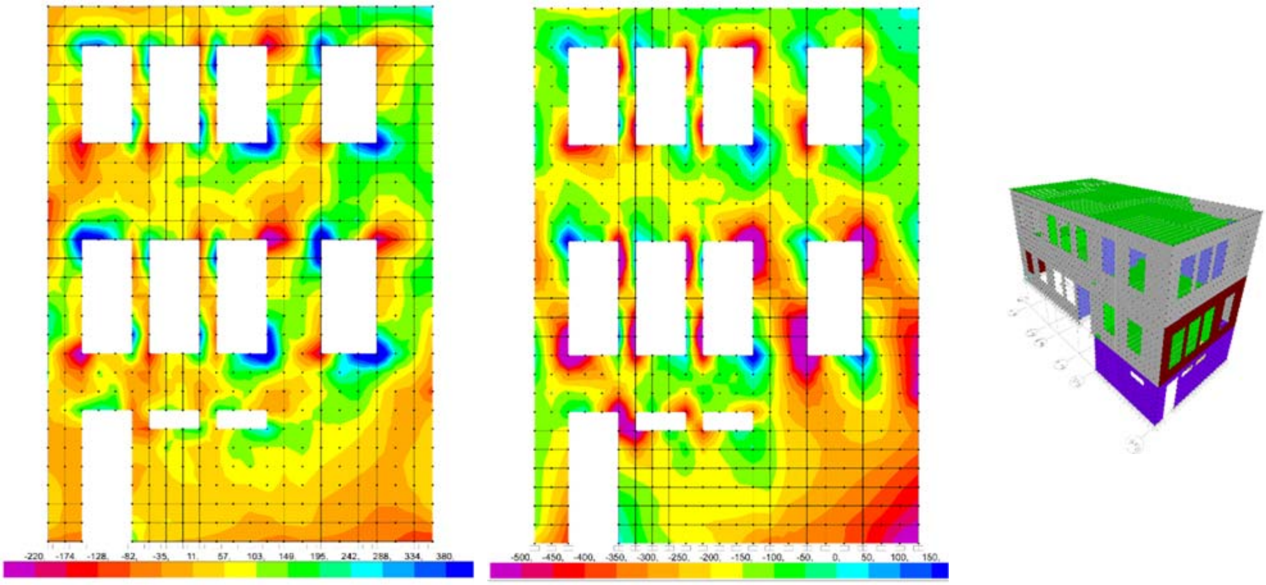


Fig. 2 - The maximum (left) and minimum (right) principal stresses from vertical loads and seismic forces of one selected building [KPa]

2.2 WP2: Experimental data analysis

The experimental analysis is performed as part of a second WP of the project framework. It encompasses two main activities: (1) laboratory testing of properties of inbuilt materials and (2) in-situ dynamic testing.

The investigated buildings are constructed from solid clay bricks and lime mortar, hence the aim of the laboratory testing was to obtain the physical and mechanical properties of the masonry components and the masonry itself, as a structural material. Brick units and mortar samples have been extracted from two of the twenty selected buildings, where actual permit was obtained. The laboratory testing revealed that the bricks have mean density of 1744.66 kg/m3 and mean compressive strength of 9.02 MPa. The type and quality of the mortar were estimated as lime mortar with additional admixture of fine crushed bricks, with specific density of 1700 kg/m3, compressive strength of 2.18 MPa and flexural tensile strength of 0.47 MPa.

As a second step, 6 wallets with dimensions 500x440x125 mm made from the original brick samples from the buildings and laboratory prepared lime mortar with similar characteristics as the original mortar samples, were tested. The mortar formulation mixture consists of river sand, lime paste and cement in density ratios of 2.25:1:0.083. The final measured strength properties of the laboratory prepared mortar, after 28 days of curing were: specific density of 1910 kg/m3, compressive strength of 2.28 MPa and flexural tensile strength of 0.48 MPa. The testing of the 6 wallets resulted in a mean compressive strength of 2.45 N/mm^2.

The dynamic in-situ tests were performed for all 16 buildings to determine their dynamic characteristics. Mobile equipment consisting of Digitexx PDAQ Premium portable system and uniaxial and tri-axial accelerometers was used, Fig. 3. The tests were conducted by measuring the accelerations on certain locations in each building from ambient vibration sources. Different number of measurement points was used for each structure, generally in the range of 3 to 12 per floor. The measured acceleration time series are 10 minute long, with a sampling frequency of 200 Hz. After some quality verification and preprocessing of the recorded signals, operational modal analysis using the well-known Frequency Domain Decomposition method was performed. The mode shapes and natural frequencies of the buildings were estimated from the preprocessed signals with ARTeMIS Modal 4.5 [13]. For each building at least one dominant mode shape and corresponding frequency was identified. For some of the buildings, the first natural mode shapes in each respective direction was obtained. In general, the identified frequencies are within a frequency range of 2.05-12.40 Hz. Most of the buildings show coupled translation and torsional modes, and only few clear translation or rotation modes.

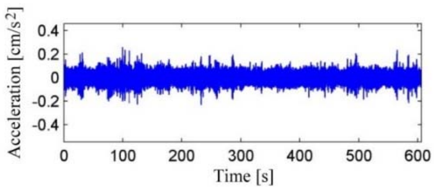


Fig. 3 - Testing of wallets compressive strength (left) and testing equipment and typical acceleration signal in selected building location (right)

Table 3 - Fundamental frequencies of the surveyed buildings.

	Building	Frequency (Hz)		
		Translation X	Translation Y	Rotation XY
1	Sch. “St. Kliment Ohridski”	3.906	2.539	2.832
2	Mental Health Centre	6.151	3.711	4.883
3	Museum of Alban. alph.	4.297	5.273	7.813
4	Sports center “Partizani”	12.402	7.324	NI
5	Train station	NI	NI	7.129
6	Sch. “St. Sava”	5.469	3.711	7.227
7	Museum in Ohrid	4.785	2.051	6.25
8	Film agency of RM	8.301	6.348	NI
9	Museum in Bitola	NI	NI	NI
10	Sch. “Strasho Pindzur”	NI	4.813	NI
11	Sch. “Vojdan Cernodrinski“	5.080	4.490	NI
12	Municipality building	NI	NI	NI
13	Kindergarten “Pepelashka”	6.455	7.422	8.203
14	Basic Court	4.102	4.199	4.213
15	Sch. “Dimkata Angelov G.”	5.957	NI	5.762
16	Post office/Telekom	6.055	6.934	11.523

The experimentally obtained mode shape frequencies are analyzed in function of building height and the cross-sectional area of walls in selected direction, [14]. The equation for seismic design of buildings given in Eurocode 8 [5], the period-height relation specified for structures with concrete or masonry shear walls is given with Eq. (3), Eq. (4) and Eq. (5).

$$T_1 = C_t \cdot H^{3/4} \tag{3}$$

$$C_t = 0.075/\sqrt{A_c} \tag{4}$$

$$A_c = \sum[A_i \cdot (0.2 + (l_{wi}/H)^2)] \tag{5}$$

where, T_1-fundamental period, H-height of the building, A_c-total effective area of the shear walls in the first story of the building (in m^2), A_i-effective cross-sectional area of shear wall i in the direction considered in the first story of the building (in m^2), l_{wi}-length of the shear wall i in the first story in the direction parallel to the applied forces (in m) with the restriction that $l_{wi}/H \leq 0.9$.

As a result of the inconsistency in compared results a modified expression for approximate calculation of the natural frequency is proposed, [14] Eq. (6). Fig. 4 gives a clear overview of estimated frequencies with the modified and original EC8 expression.

$$C_t = \frac{0.075}{\sqrt{A_c}} \cdot \frac{A_c}{6} = 0.0125 \cdot \sqrt{A_c} \tag{6}$$

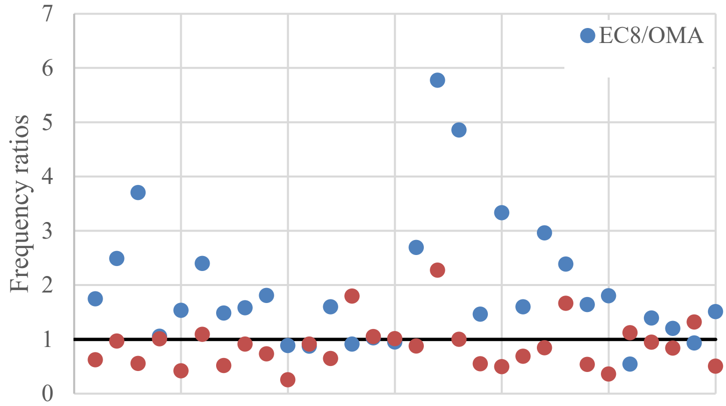


Fig. 4 - Comparison of frequency ratios with original and modified Eurocode 8 relations

2.3 WP3: Calibration of numerical models with experimental data

The third working package aims at providing efficient methodology for calibration of developed 3D finite element models with the experimentally obtained information, more precisely the estimated frequencies and mode shapes of the structures. For this purpose, the software tool FEMtools [15] offers automatized algorithms which will significantly reduce computational load in calibration of detailed mathematical models.

Three school buildings have been selected to test the software capabilities for calibration of the experimental results, St. Sava school, St. Kliment Ohridski school and Dimkata A. Gaberot school. In the beginning, a comparison of Finite Element Analysis (FEA) and Experimental Modal Analysis (EMA) results for all buildings was performed. A correlation analysis by using MAC (Modal Assurance Criterion) values above 60% was performed and mode shapes from FEA and EMA were paired. Then, in favor of selecting the most sensitive calibration parameters, sensitivity analysis was executed. Resonant frequencies and mode shapes MAC values were used as target responses and inspection of the sensitivity of the parameters such as: masonry specific weight, masonry Young's modulus and three wall thicknesses was checked.

As expected, the most sensitive parameters were masonry specific weight and masonry Young's modulus and therefore both of them were used in the calibration process with lower and upper bound limits of their values according to recommendation from literature. For St. Sava school and Dimkata A. Gaberot school only one resonant frequency was automatically selected, while the mode pairing of St. Kliment Ohridski school allowed using three resonant frequencies. Tables 4 and 5 present a comparison of the calibration results as well as the initial and calibrated parameter values.

Table 4 - Calibration results of the inspected buildings.

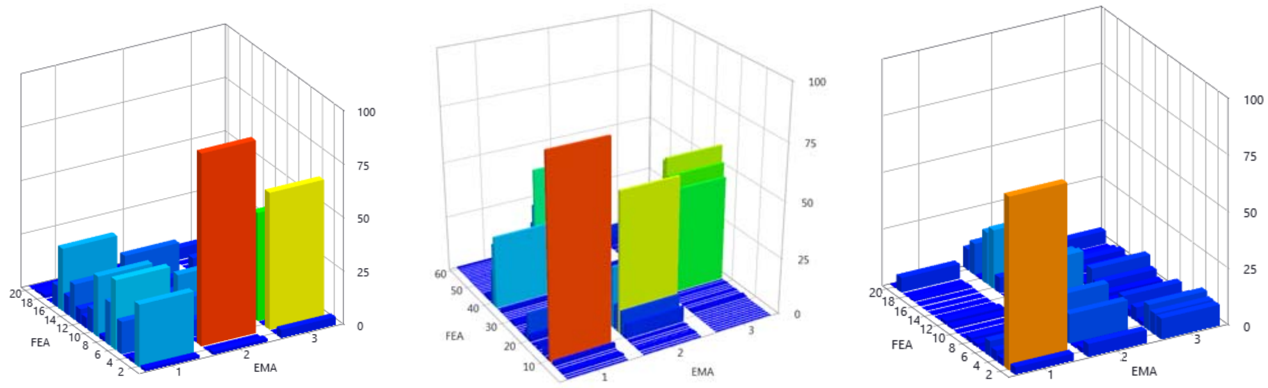
Building	**Before calibration**				**After calibration**		
	EMA (Hz)	FEA (Hz)	MAC (%)	Diff. (%)	FEA (Hz)	MAC (%)	Diff. (%)
St. Sava school	5.37	9.29	88.8	73.03	5.35	90.2	-0.46
St. Kliment Ohridski school	2.54	5.15	80.1	102.72	2.74	87.7	7.99
	2.83	5.48	67.7	93.66	2.89	62.0	2.07
	3.91	7.65	66.8	95.93	3.9	60.1	-0.18
Dimkata A. Gaberot school	5.76	9.80	78.4	70.1	5.78	76.6	0.45

Table 5 - Initial and calibrated values for the selected parameters.

Building	**Parameter**	**Initial value**	**Calibrated value**	**Percentage change**
St. Sava school	Specific weight	16 kN/m^3	19.01 kN/m^3	18.81%
	Young’s modulus	3232 Mpa	1219 Mpa	-62.3%
St. Kliment Ohridski school	Specific weight	16 kN/m^3	19.24 kN/m^3	20.25%
	Young’s modulus	3232 Mpa	922 Mpa	-71.47%
Dimkata A. Gaberot school	Specific weight	16 kN/m^3	16.94 kN/m^3	5.88%
	Young’s modulus	3232 Mpa	1140 Mpa	-64.71%

As a result of the automatic calibration, one can conclude that this process applied to unreinforced masonry buildings was successful with reasonable agreement of the mode shape pairing and good agreement of the dynamic properties between results from calibrated models and those experimentally obtained, with percentage difference of 1-8%. Having in mind the complexity of the material, structural irregularity and the general stiffness of the buildings, a limited improvement of the MAC values was obtained, Fig 5. All the other buildings were calibrated manually by modifying the selected parameters within reasonable limits.

Several useful remarks for application of FEMtools software for unreinforced masonry buildings with flexible floors are given. Namely, SAP2000 shell model imported through S2K file does not allow execution of modal analysis with the SAP2000 solver. The obtained results with its internal solver give notable differences from SAP2000 results. Calibration without SAP OAPI file yields results that are beyond any engineering interpretation. In many cases, mode pairing by automatic correlation analysis was not relevant, since local modes of vibration were paired with global experimental modes. Therefore, careful visual inspection and selection of relevant mode shape pairing is necessary.



St. Sava school St. Kliment Ohridski school Dimkata A. Gaberot school

Fig. 5 - Obtained MAC values after calibration.

2.4 WP4: Vulnerability curves for structural types

The central goal in the SEISMOWALL project is obtaining series of vulnerability curves for 4 structural types of masonry structures (unreinforced masonry with stiff/flexible floor, regular/irregular) typical for five geographical regions in the country which are characterized with severe seismic hazard. This objective revolves around two main points: a) the selection of suitable approach in representation of the local/ regional seismic hazard; b) the selection of the type of methodology for obtaining the vulnerability curves.

Within the SEISMOWALL project the seismic hazard is defined with a neo-deterministic approach proposed by the partner institution [16], and it is represented via acceleration spectra for five regions of the territory of Republic of North Macedonia. It is worth mentioning herein that the region seismicity estimation is based on available seismic data, gathered from a network of six active permanent seismological stations located in various parts of the country, provided by the Seismological observatory in Macedonia, faculty of Natural sciences University of Ss. Cyril and Methodius.

Table 6 - The procedure for obtaining vulnerability curves

Seismic hazard	- Defined with inelastic acceleration spectrum as proposed in Eurocode 8, - With three types of spectra relevant for ground type A, B and C, - With maximum ground acceleration varying a_g (PGA) = 0.05g to 0.25g.
Structural response	- Selected method is capacity spectrum (non-linear static analysis). - Force-deformation relationship is calculated for each single wall taking into account the geometry and the material characteristics as well as the applied vertical load. - The mathematical model works as cantilever shear beam (MINEA Software [17]), - Bearing walls with continuous height distribution are contributing to stiffness - Iterative procedure where the compressed length of the wall is calculated in every step is implemented in the nonlinear analysis - Three failure mechanisms are considered: flexural failure mode, brick structural failure mode and shear failure mode. - Performance point – the intersection between the capacity curve and the selected spectra is calculated.
Damage definition	- The selected damage indicator is spectrum displacement Sd, - Four levels of damage [18], where one is no damage and last is structure collapse are utilized. The deformation thresholds are based on the bilinear representation of the capacity curve. Yield point and the ultimate displacement are identified and the damage limit states are defined.
Vulnerability curves	- Functions for damage distribution are based on calculated structural responses, ground type and seismic intensity variations - All of the single results are assigned to a corresponding damage grade and the ratio of the number of realizations in the selected damage grade and the total number of realizations is calculated for every PGA. - Maximum likelihood estimation procedure is applied for the determination of the discrete points that describe the relationship between the earthquake intensity and the probability of damage - Vulnerability curves are generated for the two classes of structures: structures with bearing walls dominantly distributed in one direction and structures with bearing walls in cell-like pattern.

In these preliminary tests, the analysis is performed using the spectra defined in Eurocode 8, using three types of ground types and five levels of PGA.

On the other hand, the proposed methodology for obtaining vulnerability curves is preliminary tested for selection of structures with unreinforced masonry. More precisely, instead for four classes of buildings, all the structures are preliminary categorized in two types with similar behavior under seismic loads, namely structures with bearing walls dominantly distributed in one direction (marked as type A in table 1 and table 2) and structures with bearing walls in cell-like pattern (marked as type B in Table 1 and Table 2). The applied procedure is outlined in Table 6 and the results are shown on Fig. 6 and Fig. 7.

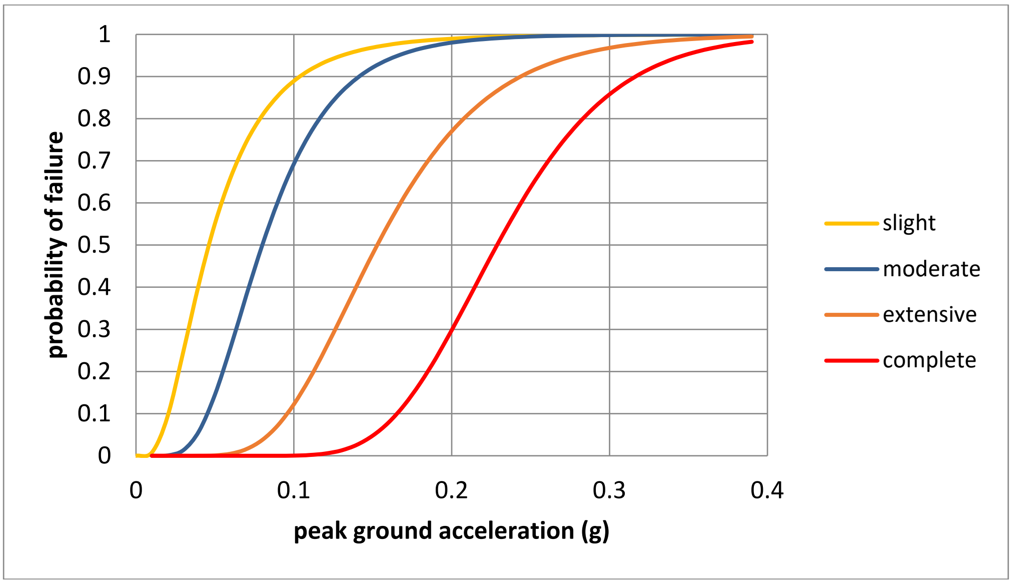


Fig. 6 - Fragility curves for structures with bearing walls in cell-like pattern (Type B).

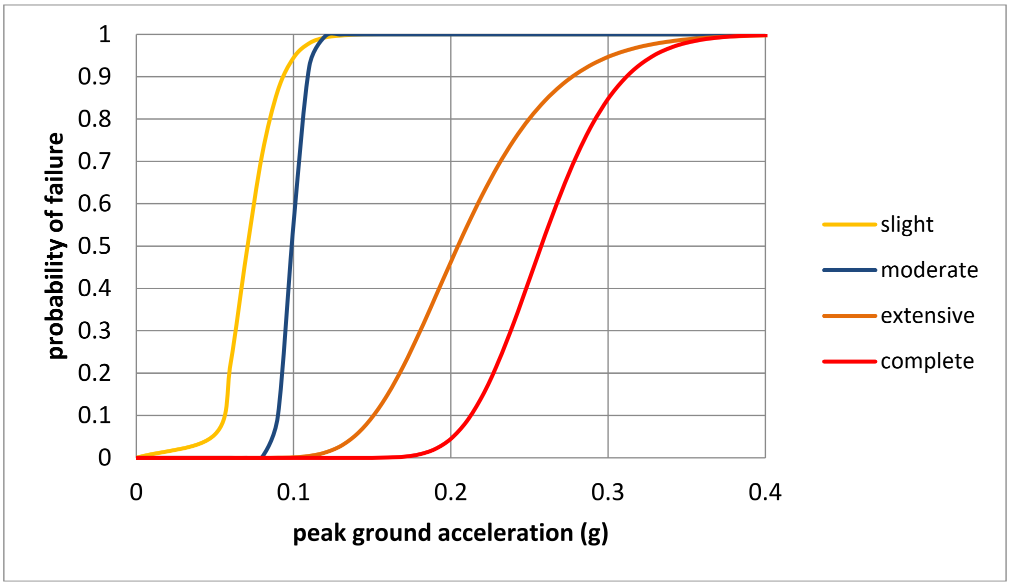


Fig. 7 - Fragility curves for structures with walls dominantly distributed in one direction (Type A)

According to the shape of the curves the following conclusions can be outlined: for earthquake intensity of 0.1g the probability of exceedance of moderate damage for structures Type A is 50% and for structures Type B is 70%; for earthquake intensity of 0.3g the probability of collapse for structures of both types is 85% and for earthquake intensity of 0.35g is 100%.

3. Conclusions

The main aim of the project SesimoWall is to obtain a series of seismic vulnerability curves for four classes of masonry buildings (unreinforced masonry with rigid/flexible floors, regular/irregular plan layout) corresponding to five geographical regions in Republic of North Macedonia with distinctive severe seismic hazard. The outputs and recommendations of the completed analysis within the four described working packages of the SEISMOWALL research project can be summarized as in the schematic overview presented in Fig.8.

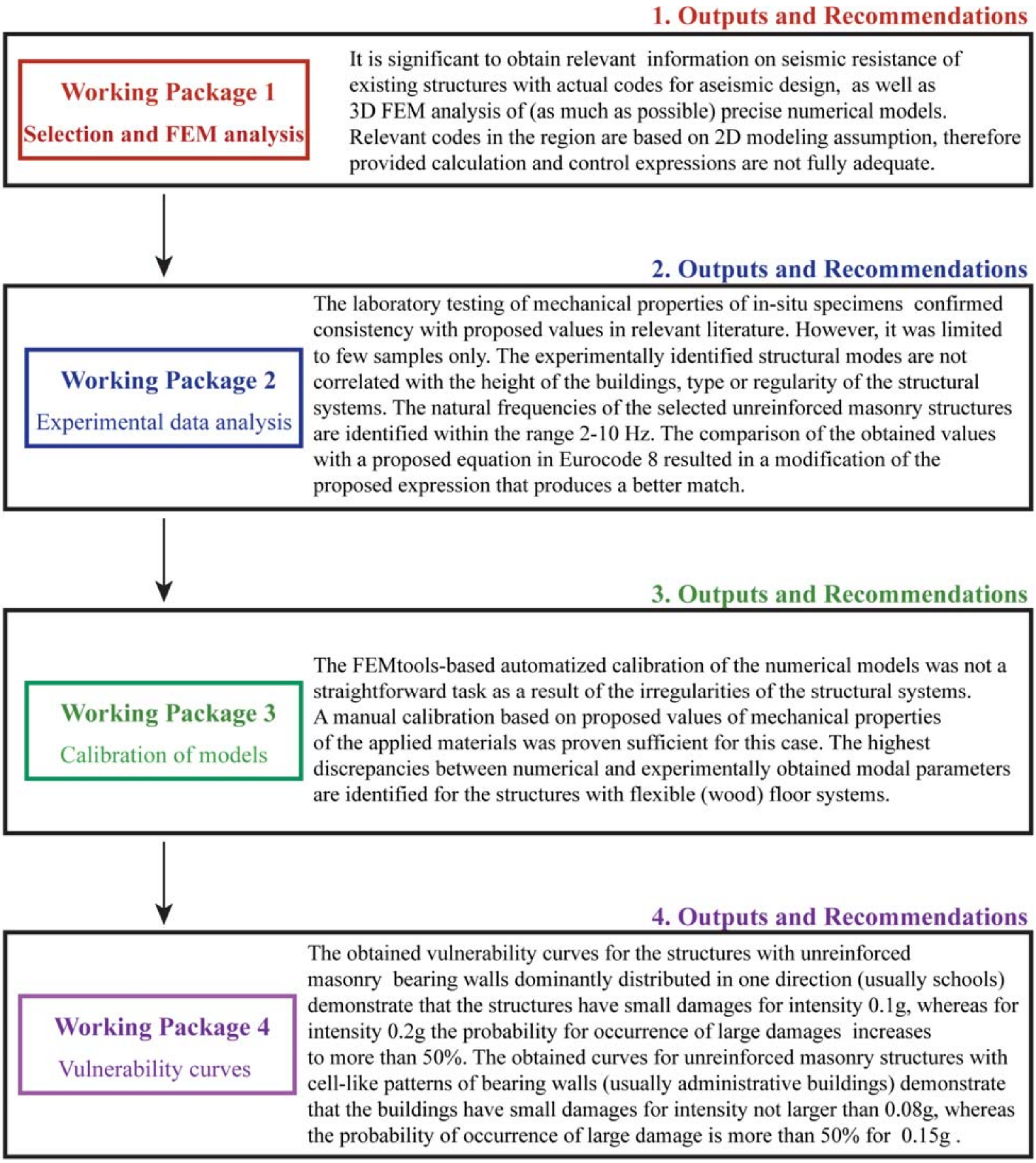


Fig. 8 - Schematic overview of SEISMOWALL project outputs

4. Acknowledgments

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