Historic buildings subjected to a risk with huge consequences and low probabilities

SEISMIC VULNERABILITY OF UNREINFOCED MASONRY HERITAGE BUILDINGS FROM ROMANIA SUMMARY

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REZUMAT

Admițând faptul că există consecințe uriașe pe care le for înfrunta clădirile istorice în cazul evenimentelor cu probabilitate redusă așa cum sunt cutremurele puternice, dezbaterile referitoare la protejarea fondului construit ar trebui să se refere la ce sunt de fapt dispuși să pierdem. Inginerii implicați în evaluarea construcțiilor existente și asigurarea unui anumit nivel de siguranță urmăresc de multe ori principii care contrazic teoriile de conservare și prezervare. Pentru a atinge obiectivul principal al reducerii riscului seismic prin reducerea vulnerabilității structurale este nevoie să se ajungă la un consens cu privire la nivelul acceptabil de risc pentru clădirile istorice. Așadar, alegerea unor metode de consolidare optime și eficiente pentru clădirile existente trebuie să fie bazată pe înțelegerea vulnerabilităților asociate fiecărei tipologii structurale și fiecărei practici de construire.

Urmărind să contribuie la cercetarea din domeniul vulnerabilității clădirilor de zidărie din România, prezenta lucrare propune modele numerice pentru două clădiri istorice, realizate pentru diferite etape de consolidare și calibrate în funcție de avariile înregistrare post cutremur și tiparele de avariere observate. Comparațiile realizate pentru aceste diferite stadii ale sistemelor structurale permit stabilirea unor concluzii referitoare la eficiența măsurilor de consolidare aplicate gradual. Sunt propuse în continuare funcții de fragilitate pentru cinci tipuri de clădiri de zidărie: zidărie portantă cu planșee flexibile, zidărie portantă cu planșee rigide, zidărie confinată, zidărie portantă consolidară prin cămășuire și zidărie confinată consolidată prin cămășuire.

Folosind aceste două studii de caz, reprezentative pentru fondul construit din România, teza prezintă o metodologie aplicabilă la nivel de tipologie pentru analize de vulnerabilitate și de risc seismic pentru clădirile de zidărie. Testarea aplicabilității metodei propuse s-a realizat utilizând o bază de date extinsă a întregului portofoliu de școli preuniversitare din România, care a fost analizată pentru a estima potențialele pierderi asociate clădirilor de zidărie existente.

Analizând doar clădirile de zidărie din sectorul educației, dar ținând cont de faptul că acestea reprezintă aproape jumătate din fondul construit, teza dorește să evidențieze pierderile potențiale uriașe asociate acestui tip de structuri, în particular celor istorice. Pentru a avea o imagine clară în legătură cu proporția de clădiri de patrimoniu folosite în sectorul public de educație pre-universitară, s-a realizat o analiză amplă, combinând lista oficială a monumentelor istorice cu baza de date din sectorul educației din România.

Este necesar să se realizeze lucrări de consolidare, admițând faptul că patrimoniul construit se confruntă cu riscul de dispariție în urma avariilor post-cutremur și că pierderile pentru clădiri cu funcțiuni destinate publicului, așa cum sunt școlile, sunt de neacceptat. Pentru a sublinia importanța reducerii riscului seismic în sectorul educației, s-au propus criterii de prioritizare a investițiilor și s-au realizat analize de cost-beneficiu pentru întreg eșantionul de clădiri de zidărie. În final, rezultatele obținute sunt prezentate sub formă de grafice și hărți pentru a ilustra probabilități anuale de depășire a stadiilor de avariere extinsă, date de expunere și scenarii referitoare la optimizarea beneficiilor generate de investiții în lucrări de consolidare. Metodologia propusă în cadrul tezei de doctorat poate reprezenta un instrument valoros de fundamentare a politicilor publice axate pe reducerea riscului seismic asociat în particular clădirilor existente cu structură de zidărie.

ABSTRACT

Starting by admitting that historic constructions are expected to face huge consequences when exposed to such low probability events as earthquakes are, discussions related to the protection of the built heritage should focus on what are we willing to lose. The engineering community involved in assessing existing constructions and insuring they grant a certain level of safety often follow principles that are in conflict with the theories of conservation and preservation. The ultimate goal of reducing the seismic risk by reducing the seismic vulnerability of existing construction implies first of all reaching an agreement related to what is to be considered an acceptable risk in case of heritage buildings. Therefore, effective and optimal seismic retrofitting measures applied to existing constructions should arise from a strong understanding of the vulnerabilities associated with each structural typology and construction practice.

Aiming to contribute to the research done for the seismic vulnerability of masonry structures in Romania, numerical models of two heritage buildings are created for different stages of structural upgrades, calibrated based on post-damage assessments and local damage patterns. Comparisons done for these different structural stages allow to draw important conclusions related to the effectiveness of retrofitting measures applied gradually. Subsequently, fragility functions are proposed for five types of masonry structures: unreinforced masonry with flexible floors, unreinforced masonry with rigid floors, confined masonry, unreinforced masonry retrofitted by means of reinforced concrete jacketing and confined masonry retrofitted by means of reinforced concrete jacketing.

Using these two particular case studies which are representative for the building stock in Romania, the thesis introduces a methodology applicable for typological assessments of the structural vulnerability and the seismic risk of masonry structures. Testing the applicability of the proposed method was done using a comprehensive database of the entire portfolio of preuniversity school buildings from Romania, which was analyzed in order to establish potential losses associated with the existing masonry structures in the sector.

Looking only at masonry constructions from the education sector but keeping in mind that they cover almost half of the existing building stock, the thesis aims to highlight the huge potential losses associated to this type of structures, in particular the historical ones. In order to have a clear picture of how large the share of listed heritage structures is used in the public pre-university education sector, an extensive analysis was done, combining the official historical monuments list with the database of the Romanian education sector.

Understanding that architectural heritage might face the risk of disappearance after suffering sever post-earthquake damage and losses might not be acceptable in the case of spaces used for public services, such as in the case of school buildings, retrofitting interventions are to be done. For emphasizing the outmost importance of seismic risk reduction in the education sector, prioritization criteria were proposed and cost-benefit analysis were performed for the entire database of school masonry structures. Finally, visual results presented in the forms of graphs and maps are used to illustrate annual probabilities of exceeding extensive damage states, exposure data and benefits optimization scenarios for investments in retrofitting. The methodology proposed in the thesis can represent a valuable tool for substantiating public policies aiming at reducing the seismic risk, in particular for existing masonry structures.

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1. INTRODUCTION

1.1. Motivation

The earthquake from Vrancea which happened on the 4th of March 1977 did not cause complete collapses for unreinforced masonry buildings built before WWI, even if they did suffer significant damage. Structural deficiencies such as flexible floors made of wooden beams not properly anchored, the lack of connections in between walls by means of interlocking bricks or the areas with not enough connections between vertical elements (attic walls, gable walls) and the horizontal ones generated most of the damages (Agent 1998).However, a better behavior was observed for buildings with rigid floors made of reinforced concrete which could ensure the rigid box behavior and the redistribution of forces. Massive public buildings constructed before 1920 (schools, museum, institutions) suffered especially due to flexible floor systems, and thus, one of the main retrofitting measures employed after the earthquake was the strengthening of slabs by adding reinforced concrete floors (Agent 1998). Steel ties were less used, since they were generally chosen only for flexible floors that could not be replaced with reinforced concrete floors. When possible, reinforced concrete tie beams along with the new floors and reinforced concrete lintels were added above openings. Retrofitting masonry walls by reinforced concrete jacketing on one or both sides is one of the most frequently used methods when shear capacity increase is needed, while for bending, it is recommended to add reinforced concrete columns and tie beams, thus transforming the unreinforced masonry structure into a confined masonry structure.

Assessments for the retrofitting interventions are a topic of interest, especially for European countries with a built environment consisting of many heritage buildings, such as Italy, Portugal, the United Kingdom, etc. The need for research in the field of seismic risk assessment and seismic risk reduction for existing buildings is highlighted in the literature, through numerous studies addressing this topic (Ravara et al. 2001), (Calvi et al. 2006), (Lourenço et al. 2013), (Ferreira, Maio, and Vicente 2017), (Anwar 2017), (Marques et al. 2018), (Gilani, Kit Miyamoto, and Nifuku 2018). The effectiveness of retrofitting works carried out in recent decades is questionable, given that they have often led to increased vulnerability, according to studies in the PERPETUATE project (Lagomarsino and Cattari 2015). In countries that experienced damage to masonry buildings in the 20th century, such as Italy and Portugal, it was recommended to use reinforced concrete elements for seismic upgrading works. Studies following the earthquake in the Molise area of Italy indicated that the most affected buildings were those with reinforced concrete floors, where the connections were not made properly (Decanini et al. 2004). The difficulty of intervening to carry out retrofitting works according to the international restoration principles for historic buildings located in seismic areas is a challenge highlighted by researchers in the field (Penazzi et al. 2001). The increase in weight generated by traditional reinforcement techniques with incompatible deformations between masonry and reinforced concrete has been found to be responsible for the increased vulnerability of buildings retrofitted and damaged by the 2009 earthquake in L'Aquila, Italy (Cimellaro et al. 2010).

Although seismic hazard provisions have been significantly improved in recent years, the vulnerability to earthquakes is still high for existing structures, built before the adoption of appropriate seismic design standards. The assessment of the vulnerability of existing buildings lays at the base of large-scale seismic risk analyses, which subsequently lead to decisions on retrofitting interventions. Moreover, the behavior of mixed structures, the results of the retrofitting of masonry buildings with reinforced concrete elements is far from being fully understood, still requiring complex studies (Lagomarsino and Magenes 2009).

Fragility functions are an important aspect of seismic risk and vulnerability studies. They create the link between seismic hazard and the effects of earthquakes on buildings. The availability of fragility functions for masonry buildings that are appropriate to the local specificity is a challenge. Although there are studies in the field, identifying or combining the different fragility functions previously obtained can prove to be a difficult task, given the regional structural features of the buildings. The complexity of the fragility functions for masonry buildings is generated by the variability of construction methods and numerous types of structural systems. Obtaining weighted fragility functions involves combining functions presented in the literature, to which different probabilities of relevance can be attributed, following a subjective analysis. The development of new fragility functions, adapted to particular situations, implies the existence of empirical data (observations of the damage states caused by earthquakes) or the use of analytical models. The only information available for the masonry buildings in Romania is that collected through post-earthquake investigations, conducted for approximately 15,000 buildings after the earthquake from the 4th of March, 1977. The results of field inspections are presented in the form of diagrams of mean damage degrees, aggregated at the typology level, expressed in terms of macroseismic intensities (Sandi 1986). In the absence of other information, analytical methods validated through damage patterns are an alternative for calibrating fragility analyses.

Following a series of earthquakes that destroyed important European historic centers such as Naples, Athens, Palermo or Lisbon, researchers pointed to the need to protect the built heritage located in areas with high seismic risk (Ayala et al. 1997). The importance of estimating losses for such scenarios is closely linked to the retrofitting strategies needed to be adopted in order to meet the structural performance criteria. Moreover, the need to consider techniques for estimating potential losses based on the assessment of existing damages was highlighted, given the variability of construction techniques, materials and age of the building stock. Given the growing interest on this topic, the need to adapt the methods of assessing and analyzing the vulnerability of historic buildings to the particularities of Romania is an argument for in-depth research on the subject.

This paper aims to analyze in particular the behavior of historic, massive buildings with masonry load-bearing walls, used in principle for public functions (museums or schools). The chosen case studies fall into this category of monumental buildings built in the early twentieth century, their characteristics and the retrofitting interventions to which they have been subjected over time can be considered representative for the historic masonry buildings in Romania.

Increasing disaster resilience through actions focused on seismic risk reduction in areas prone to such extreme events is a topic of interest worldwide. The National Institute of Building Sciences 2020 has produced a report on the benefits of investing in natural disaster risk reduction. In terms of earthquakes, the cost-benefit ratio is 13:1 for investments in retrofitting in the private sector and 6:1 for investments in the public sector. In order to apply locally the methods and to use the tools designed to reduce the seismic risk, the first step is to evaluate the built environment at national level. Given the lack of information on the vulnerability of masonry buildings in Romania, in contrast to the percentage they represent of the entire building stock and their increased importance in the context of buildings of historical value, the thesis aims to contribute to the progress of studies aiming to reduce seismic risk.

1.2. Objectives

The structural analyses of the monuments having masonry structures presented in the specialized literature, as well as the seismic vulnerability studies mentioned above represent a basis for the realization of the fragility studies for representative typologies in Romania. Starting from the taxonomies adopted in the previous programs aimed at reducing the seismic risk at European or international level (Pitilakis et al. 2014), (Ayala et al. 2014), common characteristics can be established, in relation to the analyzed building stock. Comparing the fragility models already validated by previous research with numerical analyses of heritage masonry buildings in Romania, one major objective is to propose some fragility functions representative for this type of structures.

To highlight the importance of such analyses, this paper aims to evaluate the expected losses in the event of an earthquake, as well as the benefits of a retrofitting program, using the database of buildings in the Romanian education system, where over 60% of buildings have unreinforced or confined masonry structures. The analysis of the portfolio of pre-university education units together with the list of historical monuments allowed the identification of heritage buildings in the education sector.

The structure of the thesis comprises 6 chapters and 3 annexes.

The first chapter presents a brief introduction to the topic of the importance of fragility and seismic risk analysis for historical masonry buildings, as well as the main objectives of the thesis.

The second chapter covers the current state of research in the field, presenting the existing proposals in the literature at national and international level for the taxonomy of masonry buildings, methods for assessing structural fragility and seismic risk.

Starting from the methods presented in the previous chapter, the third chapter describes the methodology proposed for seismic risk analyses of the heritage masonry structures from Romania.

The fourth chapter is divided into two sections, each of them analyzing in detail a heritage building, representative of the typologies considered. Using the available data on the initial conformation of the buildings, their behavior over time and the structural changes made or proposed so far, the numerical models for static nonlinear analyses were validated. The results and fragility functions resulting from the analyses performed are discussed, as well as the annual failure probabilities associated with each structural stage.

The sixth chapter proposes the extrapolation of the results from fragility analysis presented in the previous chapter for a set of structural typologies established according to the building stock from the pre-university education sector in Romania. The first part refers to buildings in the education sector around the world, in particular from areas exposed to seismic events and the ones that benefited from seismic risk reduction initiatives. In continuation, it is presented a sample of the buildings included in the analysis, as well as the proposed fragility functions for the established taxonomy. The seismic risk analysis of the masonry buildings in the portfolio allows the estimation of losses according to the expected damage degrees and the annual failure probabilities, aggregated at county level. The last part of the chapter includes the cost-benefit analysis for two scenarios: with or without a process of prioritizing investments in retrofitting works. The effectiveness of the school infrastructure rehabilitation projects carried out so far is also evaluated through a cost-benefit analysis applied to a small sample of retrofitted masonry buildings.

The last chapter is the one dedicated to the conclusions and perspectives of further research in the field, highlighting the personal contributions for each of the aspects included in the thesis.

Overall, this paper aims to address the issue of reducing the seismic risk for masonry buildings, in particular for heritage buildings. Starting from the fragility analyses of some historical buildings analyzed in the form of a case studies and comparing the results with similar studies from the specialized literature, it was possible to extrapolate them at the typological level. Combining the data obtained regarding the structural vulnerability with those of seismic hazard, a seismic risk analysis was performed for the building stock comprising masonry buildings in the education sector, throughout the country. Such an instrument can be used to estimate expected national losses in the event of an earthquake and to plan seismic risk reduction actions.

2. LITERATURE REVIEW

2.1. Masonry buildings: taxonomy

In order to establish similarities between the typologies proposed in the literature for masonry buildings and the building stock from Romania, a presentation of it is necessary, made based on the monograph of the 1977 Vrancea earthquake (Bălan et al. 1982). After the earthquake from the 4th of March 1977, the behavior of the damaged structures was analyzed, and the typologies were defined according to the construction methods, as follows:

- Very old buildings, with clay brick unreinforced masonry walls, with wooden floors, with ground floor and basement or partial floor
- Unreinforced masonry structures, with reduced height: single family dwellings (GF+1, GF+2) or multi-family units (up to GF+5)
- Mixed structures (GF+3…6): unreinforced masonry walls, perimetral columns on the exterior and reinforced concrete beams, usually with reinforced concrete slabs

The conclusion presented in the monograph of the 1977 earthquake regarding the main structural deficiencies of the old unreinforced masonry buildings was referring to the lack of transversal connections. The authors highlighted the beneficial influence of reinforced concrete elements (tie beams, slabs) to the detriment of flexible floors, insufficiently connected to the perimeter walls. On the other hand, the floors made of brick vaults, being much more rigid, were closer to the behavior of reinforced concrete slabs (Bălan et al. 1982). As for the old URM buildings, they performed better due to the quality of the construction work itself and the rigidity they have. Among them, the ones with reinforced concrete floors were observed, where the connections created between the walls helped in redistributing the lateral forces among elements in a unitary way.

For the newer masonry buildings (built after 1950), constructive measures have been imposed since 1962 to improve the behavior when subjected to seismic actions. These involved the installation of reinforced concrete columns, placed at the corners and intersections of orthogonal masonry walls, the use of reinforced concrete tie beams at the top of walls thus achieving a box behavior of the structure, the reinforcement of mortar bed joints and the use of materials with superior strength in general.

The building stock from Latin America can be considered similar to that in Romania, given that their structural systems with unreinforced masonry walls were gradually replaced by confined masonry, following the damage observed during earthquakes in the first part of the twentieth century. Buildings of confined masonry with low or medium height regimes, with regular layout, with dense walls and which comply with the construction provisions behaved favorably when subjected to seismic action (Brzev 2007). After the earthquakes in El Salvador in 2001, more than 90% of the confined masonry buildings were not damaged at all, the most significant problems being encountered in the unreinforced masonry buildings, made of local materials. In Peru, Mexico (Brzev 2007) and Chile (Moroni, Astroza, and Acevedo 2004) it has been observed that extensive damage to confined masonry buildings can be correlated with poor ground conditions.

Internationally, a uniform methodology for classifying the building stock has been proposed in the European SYNER-G project (Pitilakis et al. 2014). The taxonomy was used to group buildings with similar characteristics in terms of behavior when subjected to seismic actions. In order to define typologies for masonry buildings, a distinction must be made between traditional buildings, built only according to empirical principles, and engineering structures, designed to withstand seismic action (confined masonry, the presence of RC tie beams and reinforced concrete slabs).

In order to use the proposed taxonomy, a lot of information related to each particular structure is needed, which is not possible in the case of a vulnerability analysis at territorial level. Once a limited number of typological classes have been established that take into account the available information, static nonlinear analytical methods can be applied to create numerical models and subsequently fragility functions associated with each typology.

Starting from the macro-seismic method that refers to the classes of vulnerability, within the RISK-EU project a taxonomy adapted to monuments was created, where the heritage buildings were separated according to common structural characteristics. Thus, 13 types of historical buildings indicated Tab. 1 were proposed, together with the average values of the vulnerability indices (Vi *) and the factor β.

Tab. 1 Structural typologies for heritage buildings RISK-UE (Lagomarsino et al. 2003)

Typology	V_i^*	β
Palaces/buildings	0.616	2.3
Monasteries	0.736	2.3
Castles	0.456	2.3
Churches	0.89	3
Chapels	0.77	3
Mosques	0.73	2.65
Theatres	0.736	2.65
Towers/bells	0.776	2.3
Bridges	0.296	2.3
Defense walls	0.496	2.3
Arches of triumph	0.456	2.3
Obelisks	0.456	1.95
Statues/monumental fountains	0.296	1.95

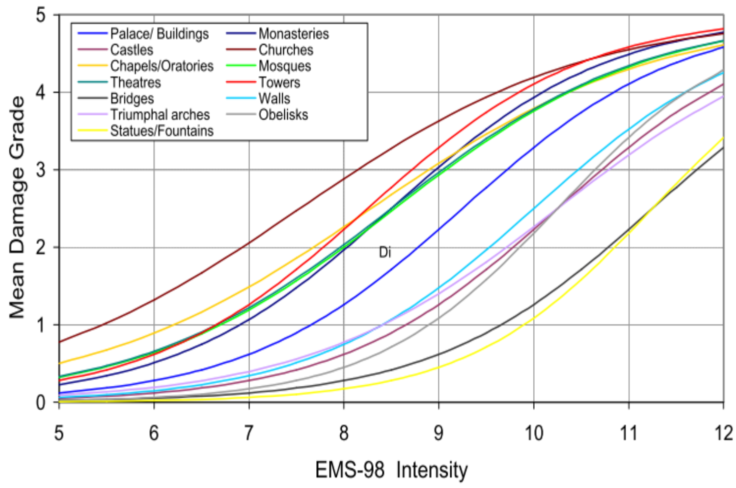


Fig. 1 Vulnerability curves for heritage buildings (Lagomarsino et al. 2003)

2.2. Structural fragility evaluation methods

Fragility analyses are performed to obtain the probability of reaching a certain level of damage, for a certain seismic scenario. Thus, the seismic vulnerability of a building can be defined in the form of a cause-effect law (earthquake-damage) (Sandi 1986).

To estimate the vulnerability of existing buildings, either statistical methods, based on observations of damage from post-earthquake investigations, or mechanical methods can be used, for which the damages are based on analytical models for estimating the seismic response. For vulnerability analyses at the territorial level, the statistical approach has the advantage of providing fast results, for which less information is needed, compared with the analytical calculations. A first step for conducting the large-scale vulnerability analyses is to establish the structural typologies according to the characteristics of the analyzed building stock. The next step is the choice of a seismic ground motion parameter that defines the damage level. It is also necessary to establish the limits for the transition from one damage state to another, in relation to the chosen parameter. Finally, the response to the seismic action is obtained and fragility functions are established for each of the analyzed typologies.

2.2.1. Analytical methods

Structural fragility can be expressed using capacity curves, which capture the response expressed in the form of displacements of a single degree of freedom equivalent system, subjected to seismic action. Capacity curves can be obtained by bilinearizing Pushover curves resulted from static nonlinear analyses for structural models with more degrees of freedom. There are various methods to estimate the displacement capacity and to reduce the seismic requirement in the nonlinear response of the structure, finally obtaining the performance point located at the intersection between the capacity curve and the corresponding reduced demand curve (Pitilakis et al. 2014).

Vulnerability or fragility curves can be obtained by processing statistical data from the results of non-linear mechanical analyses performed taking into account the characteristics of the analyzed building stock and the local seismicity. Monte Carlo simulations can be used to generate virtual buildings, built on randomly extracted mechanical parameters. To ensure reliable results for the existing building stock, calibrations are required in order to reproduce coherent combinations of structural parameters. In previous studies, databases with field investigations were used to estimate the distributions of the different random variables involved in the generation of virtual models and to determine the common probabilities for different combinations of characteristics. In order to take into account layout irregularities, amplification coefficients can be artificially introduced, as the proposed procedure only models orthogonally arranged facades and walls and not the entire building (Giulio Zuccaro 2015).

Another method of estimating the fragility of masonry buildings is presented in the literature (Lumantarna et al. 2006) starting from experimental campaigns that determined the hysteretic behavior of the walls subjected to quasi-static cyclic loads. For time-history analyses, equivalent systems with a single degree of freedom were used, subjected to seismic action by means of a set of accelerograms, used to determine the maximum displacement demand. The considered limit states are minor damage associated with the first crack (displacement at half the height of the wall = 5 mm), moderate damage (limit displacement equal to half the thickness of the wall, that is 55 mm) and collapse. Static and dynamic nonlinear analyses were performed in the Tremuri program, for the derivation of fragility functions and for a case study of a prototype building in southern Italy (Rota, Penna, and Magenes 2008). All mechanical characteristics of the structure were determined using Monte Carlo simulations, and then static nonlinear (Pushover) analyses were performed for the numerical model.

2.2.2. Empirical methods

Empirical methods for generating fragility functions involve matching a mathematical expression with post-earthquake observations or laboratory experiments. In order to more easily identify the factors that can influence the seismic response of buildings, various methods of calibrating vulnerability indices have been proposed in the literature, based on more information about the typology analyzed. The SAVE vulnerability analysis procedure uses the typological classifications defined in EMS'98 (Grunthal 1998), modified based on post-disaster investigations conducted since 1980, for which approximately 170.000 buildings in Italy were assessed (G Zuccaro and Cacace 2015). For each typology considered, an average damage parameter (SPD) is calculated in order to be able to compare the typological categories in terms of vulnerability.

Other methods of reducing uncertainty regarding vulnerability classes, based on the allocation of weights resulting from expert opinions, are difficult to apply if not all the need parameters are available. The SAVE methodology can be used for any level of knowledge available at the building level. On the other hand, its disadvantage is the consideration of macroseismic intensity, as the parameter for seismic motion.

Another method of using empirical data to obtain vulnerability curves for masonry buildings has been proposed by Lagomarsino and Giovinazzi (Lagomarsino and Giovinazzi 2006), using the vulnerability classes defined in the Macroseismic Intensity Scale (Grunthal 1998). In this case, the vulnerability is expressed by means of vulnerability curves that provide the average degree of damage ($0 \leq \mu_D \leq 5$) calculated according to the macroseismic intensity according to the formula:

$$\mu_D = 2.5 + 3tanh\left(\frac{I + 6.25V - 12.7}{Q}\right)$$

Parameters V (vulnerability index) and Q (ductility index = 3) define the behavior of the typologies taken into account for the vulnerability analysis. Limit values for the V vulnerability index were set for each vulnerability class, using fuzzy set theory. The fragility functions expressed according to macroseismic intensities are subsequently obtained using the binomial distribution. To convert the intensities into peak values of ground acceleration, the authors (Pitilakis et al. 2014) propose the following formula, some of the parameters used being those proposed by Murphy and O’Brien (Murphy and O’Brien 1977):

$$I = a_1 + a_2 \log(PGA)\,, where\ a_1 = 7\ și\ \ a_2 = 4$$

To apply the macroseismic method together with an empirical database of post-earthquake damage data, the average damage values associated with the different recorded intensities are used. These are then used to determine the values of the vulnerability index associated with the typology.

2.2.3. Hybrid methods

Hybrid methods involve the use of mechanical analyses together with data obtained from post-seismic investigations in the analyzed area or from subjective evaluations performed by experts in the field. For seismic risk analyses in Italy, such an approach has been proposed for determining vulnerability curves by generating a set of randomly created models based on geometric and mechanical properties extracted from post-earthquake observation databases (G Zuccaro and Cacace 2015).

This method of structural analysis can later be translated into seismic vulnerability levels, but it is difficult to establish a clear connection between the damage estimated in this way and the damage matrices built on the basis of post-earthquake investigations. At the same time, the level of damage corresponding to the triggering of the failure mechanisms under a certain seismic intensity is evaluated, but it is difficult to estimate the amplification of these damages under another level of seismicity.

2.2.4. Vulnerability assessment for heritage buildings

In order to assess the vulnerability of heritage buildings at the territorial level, a hybrid method has been proposed in the literature (Romeu et al. 2014), based on rapid visual inspections or data on damage from post-earthquake investigations. Following the level II assessment process used in Italy (GNDT II), the structure analyzed is characterized by 14 key parameters needed in order to calculate a global vulnerability index.

An analysis of vulnerability applied to heritage constructions was also proposed in the European RISK-EU project (Lagomarsino et al. 2003), starting from the level I methodology for assessing the vulnerability of the building stock function of structural typologies. In the case of heritage buildings, an extension of the structural typology defined in EMS-98 as a massive stone can be considered, taking into account the quality of the materials and the construction technique. The level I methodology is based on the estimation of the vulnerability according to the seismic intensity and the qualitative evaluation of some structural parameters. Level II methodology involves the use of mechanical methods to assess vulnerability, determining capacity curves by nonlinear analysis.

In the case of the level I methodology, certain parameters are used to change the vulnerability index depending on the state of conservation, the existing level of damage, architectural changes or existing interventions, the quality of the masonry or geometric irregularities.

For the building stock composed of unreinforced masonry buildings in Romania, fragility functions obtained from empirical data were proposed, collected in investigation forms filled in in Iași after the 1977 earthquake (Văcăreanu, Lungu, and Arion 2012). The authors of the study proposed an adapted formula for calculating the normalized degree of damage, established according to the macro seismic intensity and the vulnerability index Iv. The results presented both in the form of damage matrices and the fragility functions obtained using the Beta distribution for the damage stages represent an alternative to the method used previously, namely the estimation of damage degrees by binomial distributions (Sandi 1986). Empirical methods are difficult to use in fragility analyses for the building stock in Romania, given the limited availability of data, which includes only the degrees of damage expressed in macro-seismic intensities, in Bucharest and Iasi, following the 1977 earthquake (Bălan et al. 1982).

2.3. Methods for risk assessment

Seismic risk maps are a necessary tool in the process of optimizing resource allocation to reduce the impact of earthquakes, both in the post-disaster period and for prevention. Such maps can be obtained in an analytical manner, determining the amount of expected losses for the elements exposed to risk, for a certain level of hazard in a certain location (Gociman et al. 2016).

In the literature, risk is defined as the convolution between three probability functions: hazard, exposure, and fragility (Douglas et al. 2013). Hazard is the probability that in a certain location, a seismic event of a certain intensity occurs in a defined period of time. The exposure describes the elements affected by the event considered, from the analyzed area (people, buildings, infrastructure, etc.). Fragility is defined as the probability of reaching or exceeding a certain damage degree, for a certain intensity of the considered event. If fragility is the probability of damage to a structure in the event of an earthquake with certain characteristics, structural vulnerability refers to the potential losses associated with a seismic ground motion parameter (Porter 2015). In the present case, the seismic fragility of a building is the probability that the structural system or parts of it will be damaged in the event of an earthquake of a certain intensity.

In order to carry out analyses at territorial or national level, most of the time there is no complete database of the building stock in order to be able to obtain realistic estimates regarding the seismic risk. The methodology proposed by Zuccaro (Giulio Zuccaro 2015) aims at an adaptation to the situation in which the available information is partially found in the data from the census. Census data that is of interest for vulnerability analyses refer to the number of buildings grouped according to the vertical structural system (masonry or reinforced concrete) and the number of floors (1-2, 3-5, etc.). This information has been backed up by a database collected over more than 20 years, through field investigations or post-earthquake assessments, analyzing over 250000 buildings. The parameters considered for the territorial vulnerability analyses are the position of the building (isolated / marginal / middle), the material of the vertical structure (masonry/ reinforced concrete/ reinforced concrete with flexible ground floor/ others), construction period (<1919, 1919-1945, 1946 -1961, 1962-1971, 1972-1981, 1982-1991,> 1991) and the number of above-ground floors (1-2, 3-4, 5-6, 7-8). Two other parameters were available from the census (height and number of inhabitants of the municipality) that can indirectly influence the vulnerability, providing indications regarding the structural typologies and the quality of the constructions.

2.3.1. The evaluation of seismic retrofitting interventions

Comparative studies on the effectiveness of strengthening interventions for unreinforced masonry buildings have been conducted in Europe (Spencer et al. 1998) and New Zealand (Ingham and Griffith 2011). In Europe, most of the buildings in the historic centers are part of the architectural heritage, so they need to be protected, especially in areas with seismic activity. Concerns about improving the behavior of these buildings during earthquakes have existed since 150 years ago in areas frequently exposed to destructive earthquakes (Spencer et al. 1998). One of the measures adopted was the introduction of steel ties or wooden tie beams, in order to connect the exterior walls. Subsequently, this technique was replaced by the introduction of rigid reinforced concrete slabs instead of the flexible wooden ones, together with jacketing interventions for the load-bearing walls to increase their strength, sometimes also adding reinforced concrete tie beams.

The last part of the risk analysis for Lisbon (D. F. D'Ayala et al. 1997) focused on an intervention strategy for the Alfama district, taking into account the results of vulnerability analyses. Thus, the plan for the strengthening interventions and the distribution of shear capacities associated with the new situation was realized, both illustrated in Fig. 2. Considering the beneficial influence evaluated for the buildings with steel ties, as well as the low costs associated with this type of intervention (approximately $ 50/m^2), it was decided to use this technique to reduce the vulnerability of the building stock from Alfama.

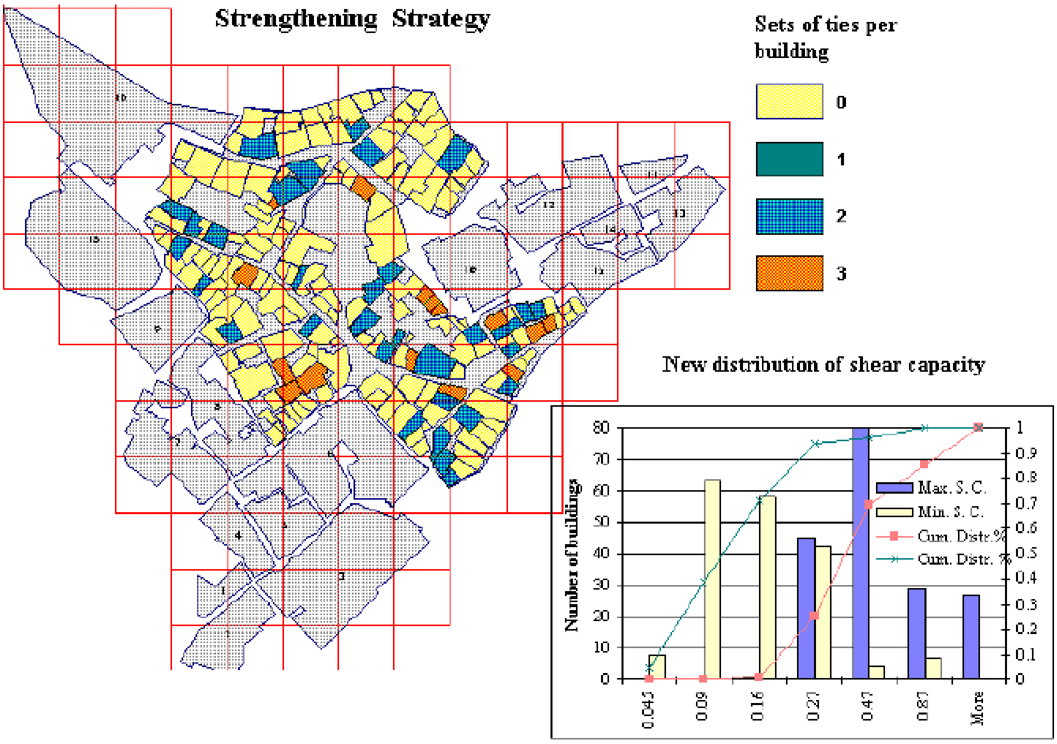


Fig. 2 Retrofitting strategy for Alfama neighborhood in Lisbon (D. F. D'Ayala et al. 1997)

Following the 1997 Umbria earthquake, numerous investigations have been carried out in Italy in order to better understand the behavior of historic masonry buildings, especially the retrofitted ones (Penazzi et al. 2001). Conclusions from the aftermath of the 1997 earthquake (Umbria, Italy) indicated insufficient knowledge of the materials and overall behavior of masonry buildings, so that strengthening interventions were either wrongly chosen or improperly implemented (Penazzi et al. 2001). Among the most common modern methods of retrofitting are the following:

- Mortar injections applied to fill in gaps and cracks in the masonry, thus increasing the load-bearing capacity of the elements. To apply this technique, it is necessary to know the internal composition of the wall, to analyze the compatibility between materials and the effectiveness of this intervention.
- The jacketing of the vertical elements consists in the application of a reinforcement mesh on both sides with the help of a cement-based mortar layer, used in order to increase the tensile strength and ductility (Penazzi et al. 2001). The difficulty of performing such an intervention, as well as the non-homogeneity of the walls led to many negative effects such as: lack of connections between the reinforcing meshes of the orthogonal walls or in connection with the floors, the absence of transverse connectors that led to the separation of layers, insufficient covering that caused the

corrosion of the steel, the lack of uniform repairs at the structural level that induced a torsional stress due to an uneven rigidity.

- Another option is adding reinforced concrete tie beams and replacing wooden floors and roofs in order to improve the connection between the vertical and horizontal elements, creating the "rigid box" type behavior against horizontal actions. The most common problems encountered were due to a lack of connections between the tie beams and the walls or due to partial eccentric loads on the walls.

Most of the problems caused by such interventions are caused by the peculiarities of the stone masonry with inhomogeneous section, consisting of several layers, which has a completely different behavior from that of regular brick masonry.

Following the earthquake in the Azores on the 9th of July 1998 with a maximum intensity of 7.9, the behavior of buildings retrofitted after previous earthquakes of 1926 and 1973 was analyzed (Spencer et al. 1998). From a typological point of view, the most affected buildings were those made of irregular stone masonry of 1-2 floors, located near the epicenter (5-10 km), in opposition to the moderate damage recorded in the old masonry buildings of 3- 4 floors. After the 1926 earthquake, steel ties were used for this type of medium-height buildings, but after the 1973 event, RC tie beams were chosen, but they were placed only for damaged walls, without creating continuous connections. The walls are made of low-quality volcanic tuff, with a thickness of about 66 cm, uniform over the entire height of the building, and the wooden floors are simply supported on the walls. Damage after 1998 did not lead to complete collapses in the Horta area, but considerable repairs were required for a number of buildings that suffered out-of-plane failures or moderate damage from shear cracks.

The 1980 Irpinia earthquake was followed by a large-scale assessment in 1986 to estimate the seismic vulnerability of buildings in the historic center of Naples, Italy. 11 typologies were considered for the vertical structures and 8 for the horizontal ones, taking into account the existing strengthening interventions, most of them consisting in steel tie rods. The EU-funded TOSQUA project aimed to estimate average damage levels for all buildings analyzed and to investigate the effectiveness of tie rods, in relation to reducing the degree of damage.

The FaMIVE procedure (D. D’Ayala and Speranza 2003) was applied to evaluate the effectiveness of strengthening interventions in four cities in the Marche area, Italy. For buildings retrofitted before 1983 by the addition of reinforced concrete slabs, no change in the failure mechanism was observed and only a 10% increase in the equivalent shear capacity was observed, which does not lead to a change in the vulnerability class. In cases where there is an additional vertical load and no effective connection between the vertical load-bearing elements and the floors, such structural changes may lead to collapse mechanisms for very low values of equivalent shear capacity. After the addition of supplementary connections (tie rods or tie beams), out-of-plane collapse mechanisms are avoided, the values of the equivalent shear capacity being approximately equal to the coefficient of friction (0.4) for the in-plane failure mechanisms. The results of the study showed that such interventions are especially useful for slender buildings, which are generally quite vulnerable. This method can provide through the load factors a quantification of the structural performance, in correlation with the parameters that define the seismic action.

The residential building stock from Portugal has similar characteristics to masonry buildings in Romania, in particular for mixed buildings with unreinforced masonry walls and reinforced concrete elements. A study for the Alvalade district of Lisbon examines the changes brought about by the new building principles of the 1930s, which aimed mainly at the introduction of reinforced concrete elements, in particular reinforced concrete tie beams and slabs, elements designed to connect masonry walls (Lamego et al. 2016) and reinforced concrete frames, for tall buildings. Five typologies are included in the study, most of which are mixed structures, called "Placa" and the rest are unreinforced masonry or reinforced concrete structures. The authors use numerical analyses of some representative buildings for each type of structure and height regime, in order to then determine the level of damage and the expected losses in a typological seismic risk analysis.

The evaluation of interventions carried out on existing buildings with masonry walls is also a topic of interest to researchers in Portugal. The authors of a paper examining the challenges associated with reinforced concrete interventions for masonry buildings state that such works have proven to be extremely vulnerable to seismic action and the subject requires further analysis to determine the characteristics of these mixed buildings, located in seismic zone (Correia Lopes et al. 2019). Disadvantages of this type of interventions include variations in stiffness and increased load-bearing capacity, leading to inappropriate force distributions. The simple addition of reinforced concrete slabs and the lack of vertical elements to help take over the lateral forces has been highlighted as one of the most important structural deficiencies of the structural conformation of the "Placa" buildings. Reinforced concrete columns were initially added only in the corners of the facades behind the buildings, being most frequently reinforced with 4Φ12/40 cm (Monteiro and Bento 2012). The influence of reinforced concrete floors depends very much on the execution procedures, since often there are not enough connections between the reinforced concrete slabs and the masonry walls (Pomba 2007). Alternative reinforcement methods for mixed-use buildings in Lisbon involve replacing reinforced concrete elements with either steel frames or composites materials made of fibers, which are considered more compatible with the original structure (Ravara et al. 2001).

Studies conducted after the 2009 L'Aquila earthquake indicate numerous buildings partially or even completely collapsed due to inappropriate or poorly performed interventions (Cimellaro et al. 2010). The authors present situations in which the choice of retrfottting methods was not made taking into account the initial structure, the major differences in rigidity between the elements and the insufficient connections generating significant damage. The conclusions of the study refer to the importance of choosing measures appropriate to the local specificity of materials and construction methods.

Although traditional methods of retrofitting unreinforced masonry buildings, namely the addition of reinforced concrete elements, are recognized as costly and invasive for the Latin American building stock, they are still a preferred option. The transformation of buildings with unreinforced masonry walls into jacket masonry structures has been proposed as an alternative solution, easier to apply, proving by laboratory tests that it can ensure an increased level of energy dissipation during seismic events (Casabonne 2000). Similar conclusions were presented following studies conducted in Romania for unreinforced masonry walls, confined masonry and reinforced masonry by jacketing with fiber-reinforced polymer composite materials (Lozincă et al. 2016).

On the other hand, in South America, the damage caused by the strong earthquake in Pisco, Peru (2007) demonstrated the effectiveness of interventions to strengthen existing buildings. The image in Fig. 3 illustrates the reinforced brick housing jacketed with reinforced concrete in the areas between the walls and the terrace, which did not suffer damage, unlike the building next to it which was not retrofitted, and it collapsed after the earthquake (Earthquake

Engineering Research Institute 2007). Much of Peru's residential constructions are made up of confined masonry buildings, which have been damaged only when having important structural deficiencies, such as severe irregularities or construction problems.

The authors of the study for residential buildings in Chile (Moroni, Astroza, and Acevedo 2004) present hybrid structural systems, characterized by combining masonry with reinforced concrete: the walls on the lower floors are made of reinforced concrete, the others are partially confined, with constructive details such as illustrated in Figs. 4, which in most cases does not comply with the minimum provisions of the building codes. This type of building has proven to be extremely vulnerable during earthquakes, especially for buildings with more than 3 floors. After the 1939 earthquake in Chile, only 16% of confined masonry buildings were completely or partially collapsed, while more than 50% of them were not damaged at all (Moroni, Astroza, and Acevedo 2004), but 60% of the unreinforced masonry ones collapsed. For this reason, the strengthening methods involved confining masonry panels with reinforced concrete elements (columns and tie beams) and jacketing them with reinforced concrete, shotcrete and anchors.

Fig. 3 Buildings from Guadalupe, after the 2007 Pisco earthquake, Peru (Earthquake Engineering Research Institute 2007)

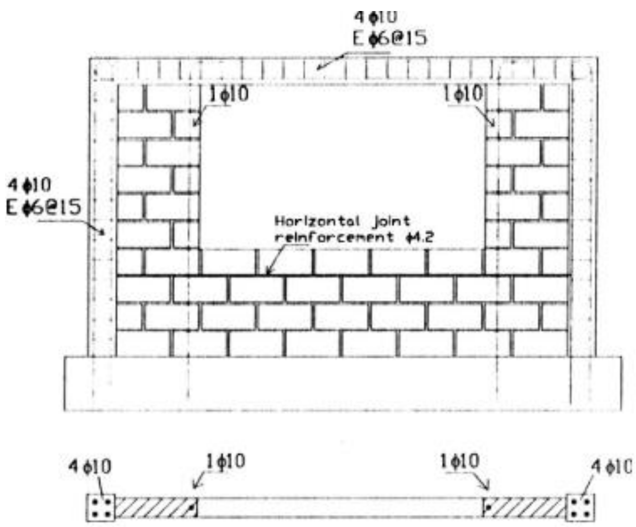


Fig. 4 Reinforcement detailing for mixed structures (masonry and RC) from Chile (Moroni, Astroza, and Acevedo 2004)

3. METHODOLOGY PROPOSED FOR SEISMIC RISK ASSMESNT OF MASONRY STRUCTURES

3.1. Structural capacity evaluation using an analytical method

Given that there is no information available on post-seismic investigations for buildings in Romania, the use of a method based on empirical data is not possible in the case of the building stock from Romania. Following the 1977 earthquake, over 18,000 buildings in Bucharest and about 2,000 in Iasi were inspected (Bălan et al. 1982). Over 70% of those in Bucharest and 90% of those in Iasi were masonry buildings. The report by a working group of the EAEE (Sandi 1986) on the analysis of post-earthquake investigations presents the diagrams of the degrees of damage (between 0 and 5), established according to the macroseismic intensity. The types of masonry buildings considered in the analysis from Bucharest are: A2 (masonry buildings with flexible floors, built before 1940), A3 (masonry buildings with flexible floors, built after 1940), A4 (masonry buildings with rigid floors, built before 1940) and A5 (masonry buildings with rigid floors, built after 1940). Because the data presented contains only the aggregated information at the typological level, without the individual inspection sheets and because they represent the data collected following a single seismic event, hybrid or purely empirical methods cannot be applied for a fragility analysis applied to the building stock of masonry buildings.

3.2. Numerical models and numerical analyses

The calculation programs used in the thesis for nonlinear static analyses of masonry buildings are Tremuri (Lagomarsino et al. 2013) and 3DMacro. For the analysis of buildings with unreinforced masonry walls, the Tremuri program was used, which allows the modeling of masonry walls in the simplified form of equivalent frames, divided into piers and spandrels, connected to each other by means of rigid nodes.

The failure modes considered in the analyses performed with the Tremuri program are based on two types of failure mechanisms observed in the masonry buildings damaged by earthquakes (Alexandra Scupin, Vacareanu, and Pavel 2021). The first refers to the yielding caused by shear, manifested by diagonal cracks in the shape of "X" which appeared mainly in the central area of the masonry spandrels, as can be seen in the image on the left in Fig. 5. The second type of failure is caused by compression and bending, illustrated in the image on the right in Fig. 5. Starting from these failure modes, specific to masonry buildings, the Tremuri program uses micro-elements whose central area works in shear, while the piers work in combined bending and compression.

Fig. 5 Damage patterns observed after the earthquake for masonry structures (S.T.A. DATA 2012)

Unlike Tremuri, the second program used, 3DMacro, uses two-dimensional elements discretized according to the scheme illustrated in Fig. 6. It uses springs at the interface between masonry panels (piers and spandrels), attributing to them mechanical characteristics that allow compression-bending or shear behavior (Formisano 2014). The masonry elements are also modeled by means of diagonal springs that describe the tensile and compressive behavior. Comparing the results of the experimental campaign with those obtained from numerical models, it was concluded that both software allow for a reasonable trust level in what concerns the evaluation of structural capacity through nonlinear static global analysis (Marques and Lourenço 2014).

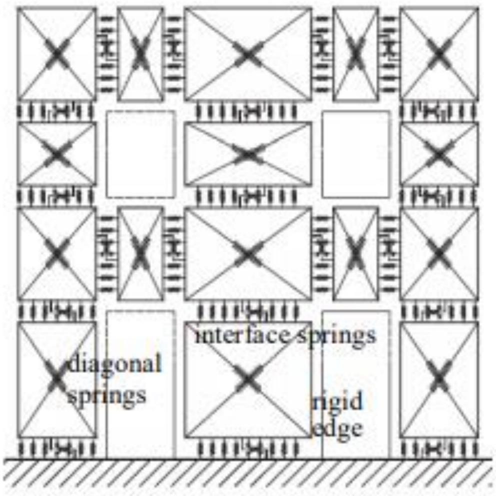


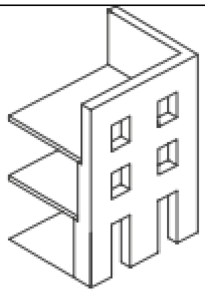
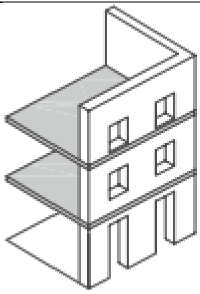
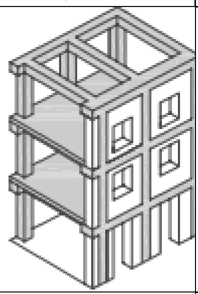
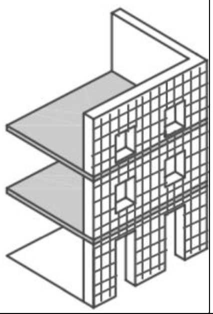
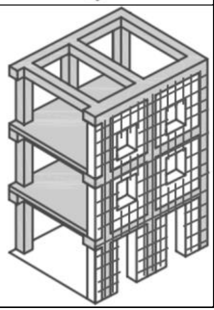
Fig. 6 Element modelling for masonry walls using 3DMacro software (Marques and Lourenço 2014)

3.3. Taxonomy

In order to perform a representative fragility analysis for the structural typologies most commonly encountered in the portfolio of pre-university education buildings, five numerical models were proposed, starting from the structure of a school in Bucharest. The analyzed building can be considered representative for the typology of historical monument buildings, with unreinforced masonry structure, built at the end of the 19th century - beginning of the 20th century. Given the age of the fund built by masonry buildings in the education sector, with a quarter of the sample of masonry buildings (15,106 school buildings) built before 1920, the characteristics of such a monumental building can be considered specific to historic masonry buildings in general.

Such old structures, located in areas with high seismicity were damaged by earthquakes and so over time various local strengthening interventions were needed. The stages of structural changes that the building went through represented the basis for numerical analysis that reproduced representative numerical models for several typologies of masonry buildings. Starting with a classification of mixed masonry and reinforced concrete structures specific to the building stock in Portugal (Correia Lopes et al. 2019), Tab. 2 presents the numerical models, starting with the initial situation of the buildings built at the end of the 19th century and ending with the structural layout of the currently retrofitted building, based on the seismic evaluation report.

Tab. 2 Numerical models and typologies used in the fragility (adapted after (Correia Lopes et al. 2019)*)*

Typology	Sketch	Structural system	Floor system	Software used
URM_FF		Unreinforced masonry	Flexibile	Tremuri
URM_RF		Unreinforced masonry	Rigid	Tremuri
CM		Confined masonry (RC beams and columns)	Rigid	3DMacro
RM		Retrofitted unreinforced masonry	Rigid	Tremuri
CM-RM		Retrofitted confined masonry	Rigid	3DMacro

3.4. Discussion of results: damage state evaluation

The damage conditions considered are those described in the HAZUS handbook (Federal Emergency Management Agency 2015): slight damage, moderate damage, extensive damage and complete damage. In the case of complete damage, only a portion of the building is assumed to be collapsing, depending on the structural type.

The level of damage resulting from static nonlinear analyses is compared to the damage caused by earthquakes, where such information exists. The global displacements recorded from the Pushover curves are still used to determine the limit state thresholds, but the maximum relative displacements are analyzed to study the local failures, specific to masonry buildings with irregular layouts. The regulations proposed in the American ASCE guide (ASCE 2014) use relative level drifts, not global drifts, to determine the level of structural performance. The verification involves comparing the relative level drifts in the numerical model with the values proposed in the literature (Derakhshan and Griffith 2018) for a performance level associated with the damage degree level of the elements.

3.5. Fragility functions

Structural damage can be estimated according to the intensity of the seismic ground motion and can be expressed through damage probability matrices or through fragility functions (G. M. Calvi et al. 2006). The expression of structural fragility through the lognormal distribution model involves the use of only two parameters: the median value of a characteristic parameter for structural behavior (such as spectral displacements or spectral accelerations) and the standard deviation, which represents the uncertainty associated with structural capacity and seismic demand. Thus, the probability of exceeding or reaching a certain damage state can be expressed, according to the formula below:

$$P[ds|SD] = \Phi\left[\frac{1}{\beta_{ds}} ln\left(\frac{SD}{\overline{SD}_{ds}}\right)\right]$$

where: $\overline{SD}_{ds}$ represents the median value of the spectral displacement, for which the structure reaches the threshold of the damage state ds, β_{ds} is the standard and ϕ is the standard normal distribution function.

The methodology further used in the fragility analysis followed the principles of the Level II procedure proposed in the RISK-EU project (Kappos et al. 2006a). This involves the generation of fragility functions expressed in terms of spectral displacements using the results of nonlinear static analyses of numerical models performed for the study case buildings. The ultimate ductility μ_u represents the ratio between the ultimate displacement and the corresponding displacement of the yielding point on the bilinearized capacity curves. This parameter is part of the following formulas, proposed for estimating the standard deviations corresponding to each damage state (Z V Milutinovic and Trendafiloski 2003):

$$\beta_{d1} = 0{,}25 + 0{,}07 \ln(\mu_u)$$
$$\beta_{d2} = 0{,}20 + 0{,}18 \ln(\mu_u)$$
$$\beta_{d3} = 0{,}10 + 0{,}40 \ln(\mu_u)$$
$$\beta_{d4} = 0{,}15 + 0{,}50 \ln(\mu_u)$$

The calculation formulas used to establish the thresholds from one state of damage to another, depending on the relative displacement values, characteristic of each structure are presented below (Lamego et al. 2016):

$$\overline{SD}_{d1} = 0{,}7\, S_{dy}$$

$$\overline{SD}_{d2} = S_{dy}$$
$$\overline{SD}_{d3} = S_{dy} + 0{,}25 \cdot (S_{du} - S_{dy})$$
$$\overline{SD}_{d4} = S_{du}$$

The fragility functions obtained from a small number of analyzed numerical models cannot be considered representative for an entire typology. Thus, in order to validate the fragility functions proposed for the five typologies included in the analysis, similar studies from the literature will be compared with the results obtained in the thesis. At the same time, the detailed analysis of the level of damage recorded in the numerical analyses is compared with the observations from the seismic evaluation report regarding the behavior of the structures during the earthquakes. In this way the degree of damage associated with a certain amplitude of seismic motion can be validated.

3.6. Seismic risk assessment

The latest results of probabilistic hazard analysis for Romania are included in the seismic risk analysis (Pavel et al. 2016), including the average annual rates for which a certain value of a seismic motion parameter is exceeded (Pavel and Vacareanu 2017).

Seismic risk analysis involves combining the notion of fragility and hazard to estimate expected losses or probabilities of failure. The method used to determine the annual probability of failure, understood as exceeding a certain level of structural damage, is calculated according to the approach of the integral convolution proposed by Kennedy (Kennedy 2011):

$$P_F = \int_0^{+\infty} H_A(a) \cdot \frac{dP_{F|a}}{da} da$$

where $P_{F|a}$ represents fragility understood as failure probability associated with a certain seismic action and $H_a(a)$ represents the seismic hazard, expressed by the annual rate of exceedance associated with an ground motion amplitude. Estimates regarding the annual probabilities of exceeding certain stages of damage were similarly made for residential buildings in Romania, with unreinforced masonry structure or confined masonry and low height regime (Alexandra Scupin, Văcăreanu, and Pavel 2020).

The estimations of the annual failure probabilities can be applied to both buildings analyzed in the case studies and to the types of masonry buildings defined for the entire sample of buildings in the education sector, aggregated at regional level.

3.7. Cost-benefit analysis

Cost-benefit analyses are tools that help making informed decisions about investment projects and public policies. Quantifying benefits involves setting values that people are willing to pay to reduce certain risks, in particular reducing seismic risk for the case of retrofitting masonry buildings in the education sector. Thus, the investment can be considered profitable if the estimated benefits of the investment outweigh the costs associated with the interventions.

For school infrastructure rehabilitation programs, the cost-benefit analysis was used to assess the effectiveness of the interventions, quantifying the losses avoided by retrofitting the buildings included in the project.

4. CASE STUDIES: HERITAGE MASONRY STRUCTURES

4.1. Geology Museum: numerical models, results, fragility and risk assessment

The building of the National Museum of Geology is analyzed in the form of a case study, being representative for the typology of palace-type heritage buildings, with masonry structure. The building was built between 1904 and 1906, initially hosting the headquarters of the National Institute of Geology and since 1975, the National Museum of Geology. It is listed as historical monument, being included in group B: historical monuments representative for the local cultural heritage. The building was designed by architect Ion Ștefănescu, who studied at the National School of Architecture and the École de Beau Art in Paris.

The building is composed of two bodies, with different heights and layouts, without being separated by expansion joints. The main body has a basement of 4.4 m, ground floor of 6.8 m and intermediate floors of 3 m height, first floor of 5.6 m. The rear façade belongs to the secondary body, where buttresses were placed shortly after completion of construction to strengthen the body affected by settlements. These elements may be based on inadequate ground (filling) or the connection areas to existing walls may not have been interlocked. Therefore, there is insufficient information about the buttresses, so as to justify their consideration as improving the behavior of the structure.

The floor above the basement in the area of the secondary body is raised by 1.60 m from the main body, by near the main staircase. Below the staircase, there is a basement of 7.85 m, and above it is the exhibition hall with a height of over 10 m. There are large differences in rigidity between the two building parts, insufficiently connected to each other, therefore their behavior is different.

From the point of view of regularity, there are continuous vertical routes for load transmission, but also discontinuities at the level of window openings, especially those from the main façade. One of the discontinuity areas is at the level of the main façade's portico, where stone columns and arches above them support the attic wall. At the same time, high level elevations favor the creation of strong horizontal coupling elements that compensate for the lack of continuous masonry piers up to the level of the infrastructure. In terms of the thickness of the masonry walls, the above-ground levels are significantly reduced in size compared to those in the basement. Reductions in wall thickness start at 25% and are accentuated with height, reaching up to about 55%.

According to the information on materials presented in the seismic evaluation report, there is fired clay brick masonry (C75 / C100 with dimensions of 14x29x6.5 cm) with lime mortar and cement addition (M4 / M10) of good quality and reinforced concrete with a mark equivalent to B100-170.

According to the seismic evaluation report, the most important structural deficiencies are the following:

- Increased floor height and reduced wall thickness
- Lack of seismic joints between the building parts with different characteristics
- The size of the openings on the facades
- Insufficiently anchored partition walls in the transverse direction at the attic level

Strengthening interventions carried out in the early 1980s contributed to the relatively low accumulation of structural damage, taking into account over 100 years of use. Climatesensitive materials (frame timber or mortar in masonry walls at the attic level) show significant degradation, amplified by mechanical actions caused by earthquakes (1940 and 1977) and bombings (1944).

As a result of these damages, the interventions completed in 1984 included the following works, highlighted in the scheme of retrofitting measures in Fig. 7:

- Casting in site reinforced concrete slabs over the wooden floor at the attic level;
- Casting in site reinforced concrete slabs over the ground floor and mezzanine, where only the steel beams were kept, not the wooden ones;
- Injecting cracks in the walls with cement paste and filling the cracks with mortar;
- Restoration of intersections by anchoring with reinforcements in perforated holes and reinforced concrete jacketing;
- Addition of steel ties for the anchorage of the portico and damaged and dislocated walls;
- Rebuilding of areas with partial collapses and crushed masonry;
- Jacketing walls with layers of reinforced concrete 10 cm thick, cast in or shotcrete mortar, reinforced with welded nets;
- Plating reinforcing meshes around new openings (longitudinally and transversely reinforced);
- Covering the staircase with a ceiling made of reinforced concrete plaster with thin steel mesh, anchored with reinforcements (OB 37 Φ12) to steel beams (profiles I 135mm high) supported on the longitudinal walls of the attic.

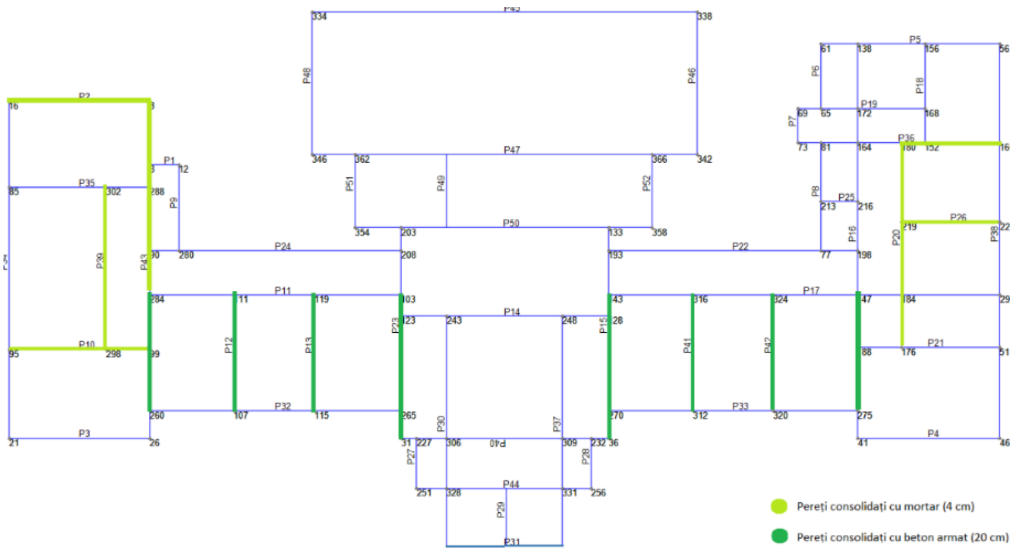


Fig. 7 Retrofitting interventions done in 1982

The earthquakes of 1986 and 1990 did not cause any damage to the retrofitted or rebuilt areas, thus demonstrating the effectiveness of the work carried out in 1984, in particular the box behavior as a result of the stiffening of the floors. From the point of view of the structure's behavior over time, no differential settlements were observed.

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4.1.1. Nonlinear static analyses: numerical model

The program used for nonlinear static analyses is Tremuri (Lagomarsino et al. 2013). For the realization of the numerical model illustrated in Fig. 8 in the Tremuri program a series of simplifications were made at the level of geometry, given the limitations of the program and the time allocated to modeling and subsequent running of the analyses. The buttresses present in the area of the exhibition hall were decided not to be modeled, taking into account the impossibility of creating a rigid box effect, given their position. The conclusion on how to build these buttresses also justifies the choice made.

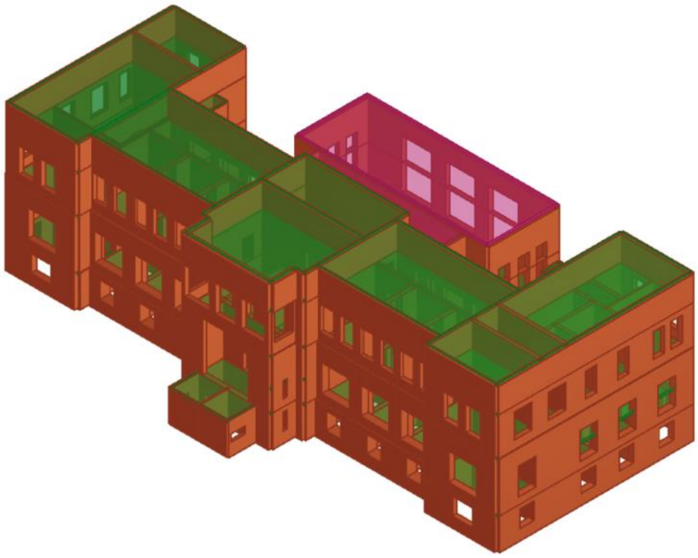


Fig. 8 3D model for the National Geology Museum (Tremuri)

4.1.2. Nonlinear static analyses: results

For the two structural models (the original building, respectively the building retrofitted in the 80's) separate analyses were performed, in order to highlight the contribution of strengthening interventions on the structural behavior of the building. Pushover analyses involve the static and monotonous application of forces, using a model of nonlinear behavior for the strength of materials, so that in the end capacity curves are obtained. These represent the envelope of hysteretic cycles produced by seismic loading and are used as an indicator of the post-elastic behavior of the structure (S.T.A. DATA 2012).

In order to perform the comparative analysis of the two variants of the Geological Museum, 4 Pushover analyses were performed for each model in Fig. 9, considering only the positive direction of action for seismic loading (X and Y direction) and the force applied in proportion to the height (Pushover Static) or evenly distributed (Pushover Uniform). For a more detailed analysis of the results obtained, only the “Pushover static” loading case will be considered.

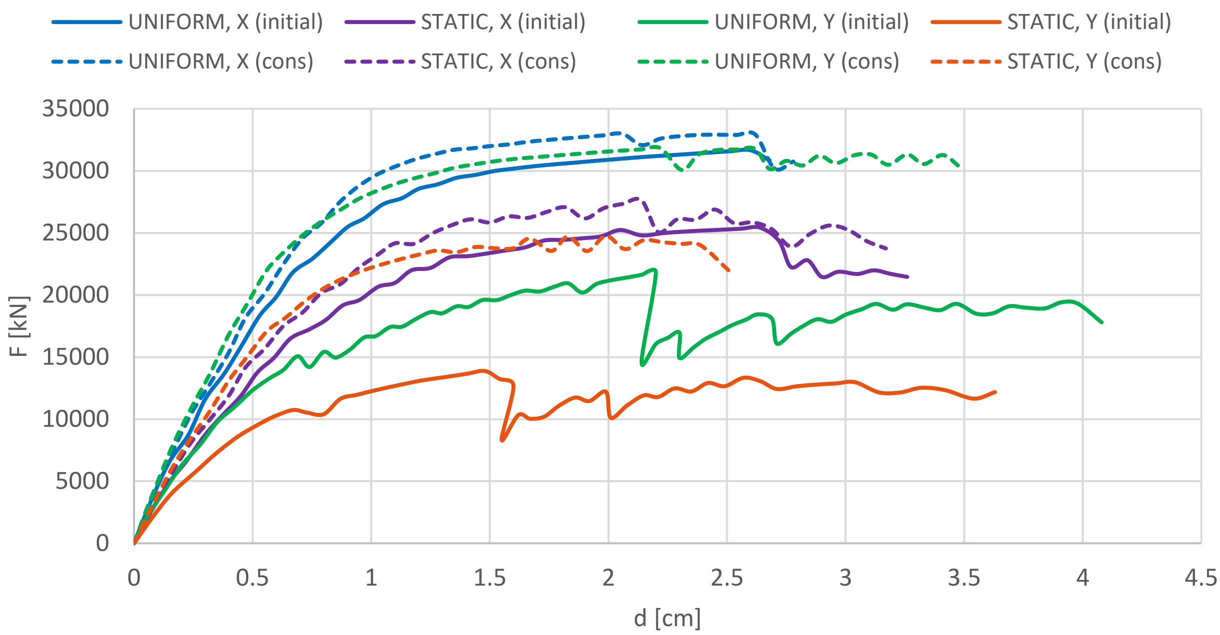


Fig. 9 Pushover curves for the initial and for the retrofitted model

4.1.3. Nonlinear static analyses: global displacements, drifts and damage states

The level of damage of the structural elements subjected to seismic actions can be established in correspondence with the relative lateral displacements, thus completing the fragility analyses used for seismic risk analyses.

All global relative displacement values corresponding to the displacements near the elastic limit on the bilinearized Pushover curves are below the D0 (immediate occupancy) limit of 0.06%. For the final values of displacements recorded by the Pushover curves, the damage control state is reached, denoted D2 with values between 0.1% and 0.2%. The only analysis that led to a relative displacement value greater than 0.2 is the Uniform Pushover (Y) for the initial model, in this case reaching the collapse prevention stage (D3). In studies conducted for buildings in New Zealand (Derakhshan and Griffith 2018) (Cattari et al. 2015), the value of 0.14% was also proposed as a boundary between D2 and D3, as opposed to 0.2%, the result being based on the damage recorded on the analyzed buildings after earthquakes. Using this low value would result in pre-collapse stages for all Pushover analyses of the original Geological Museum model.

Tab. 3 Drift limtis for URM

		Drift limits at wall level (ASCE 2014)(ASCE 2000)	Global drift limits((Derakhshan and Griffith 2018), (Cattari et al. 2015))
D0	Immediate Occupancy (**IO**)	0 – 0.3%	0 – 0.06%
D1	Damage Control (**DC**)	0.3% - 0.6%	0.06% - 0.1%
D2	Life safety (**LS**)	0.6% - 1%	0.1% - 0.2%
D3	Collapse Prevention (**CP**)	≥ 1%	≥ 0.2 %

In the case of the Geology Museum, the damage reported following the 1977 earthquake indicated areas with concentration of damage, especially for situations where there were gaps in the location of structural walls, for example those in the area of the portico on the main facade. The gradual decrease in the thickness of structural walls that contributes to significant differences in stiffness from one floor to another can also favor concentrated level drift that cannot be captured by global relative displacement analyses. In order to analyze these aspects, the relative displacement values of the walls are presented, recorded only for the static pushover analysis, both in the case of the initial model and in the case of the retrofitted one (A Scupin, Vacareanu, and Pavel 2021).

Strengthening works from 1982 focused on jacketing the transverse walls of the main building part. Comparing the damage stages associated with the load-bearing walls in the Y direction from the initial model with those in the retrofitted model, significant changes can be observed in the lateral forces and the distribution of forces in the walls (A. Scupin, Văcăreanu, and Pavel 2021). For the transverse direction, the walls with the highest relative displacements (marked in Fig. 10a in red and yellow) in the analysis of the initial model are the walls that were retrofitted in 1982. For the longitudinal direction, the areas having the largest drifts are found neat the portico and the front façade at ground floor level, with insignificant differences when compared to other walls from the same level. The changes recorded for the retrofitted model are minor in terms of the relative displacements of the longitudinal walls.

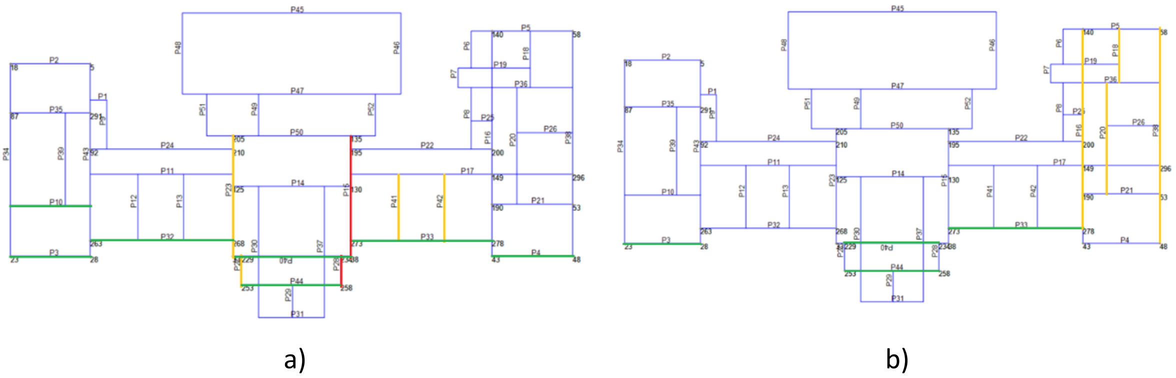


Fig. 10 Damage states for walls: initial model (a) and retrofitted model (b)

4.1.4. Nonlinear static analysis: fragility functions

In order to evaluate the strengthening interventions from 1980, fragility functions were determined for both variants of the Geology Museum building, starting from the bilinearized capacity curves. Once the SA-SD spectra were obtained, the procedure presented in Chapter 3 was followed to determine the limits of the damage limit states:

The centralized results for the parameters of the fragility functions are presented in Tab. 4, being obtained based on the capacity curves resulting from the static Pushover analyses, on both directions. Comparing the two directions X and Y for the initial model, it can be seen that the probabilities of exceeding the slight and moderate damage states are higher for the Y direction, while there are higher probabilities of exceeding the stages of extended and complete damage for the direction X. However, for spectral displacements below 1.5 cm, the probability of exceeding all damage states is higher in the Y direction. In the case of the retrofitted model, the probabilities of failure associated with all four damage states are higher for the Y direction.

Tab. 4 Fragility functions parameters

		Spectral displacements [cm]							
		Slight damage		Moderate damage		Extensive damage		Complete damage	
		Median	β	Median	β	Median	β	Median	β
Initial model	X	0,40	0,36	0,57	0,50	1,16	0,76	2,94	0,97
	Y	0,30	0,40	0,42	0,58	1,19	0,94	3,48	1,20
Retrofitted model	X	0,38	0,36	0,55	0,49	1,11	0,75	2,81	0,97
	Y	0,34	0,35	0,49	0,47	0,90	0,69	2,13	0,90

In order to evaluate the influence of the strengthening works made in 1982, Fig. 13 illustrates in parallel the fragility functions for the initial model versus the retrofitted one. Given that the interventions focused on jacketing the walls in the transverse direction, only the comparison for the Y direction is presented below, where the changes are significant. For the first two stages of damage, there are no noticeable changes between the two structural models, but for the extensive and complete damage state, the retrofitted model has much higher probabilities of exceedance than in the case of the initial model. The strengthened building by means of RC elements leads to a much more rigid behavior of the structure, reaching a 45% higher rigidity. The reinforcement solution increased the strength and rigidity, limiting the deformability of the structure. For this reason, seismic fragility must be judged in terms of both force and displacement.

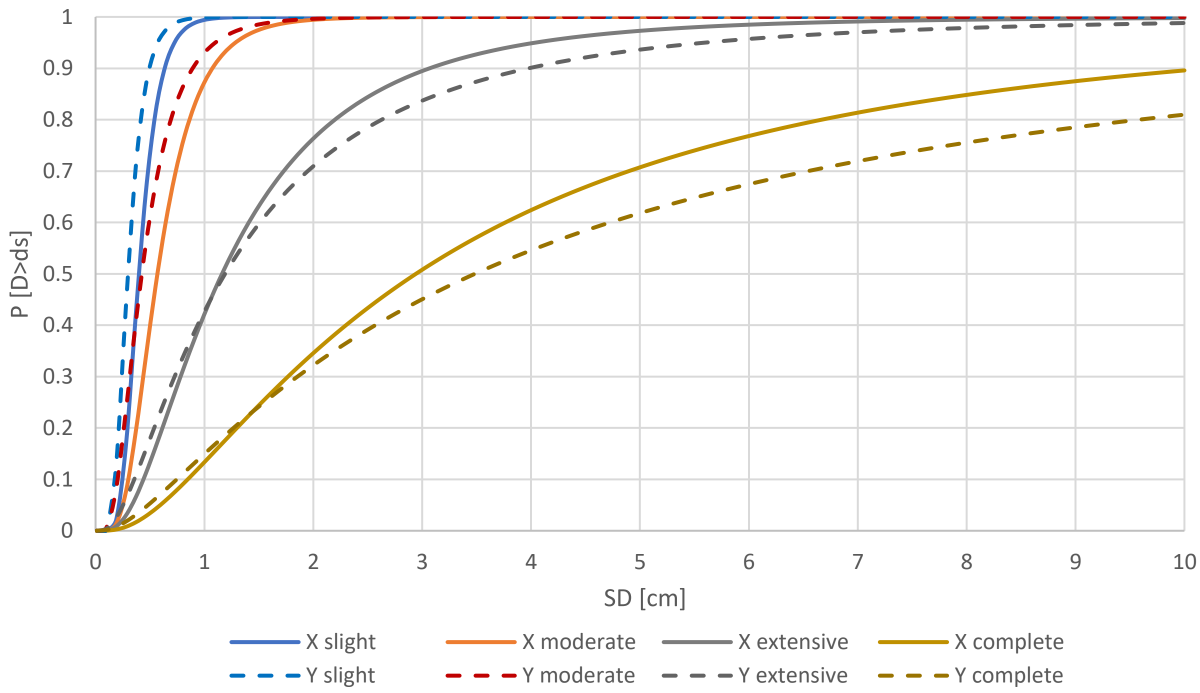


Fig. 11 Fragility functions for the initial model: comparison for X and Y direction

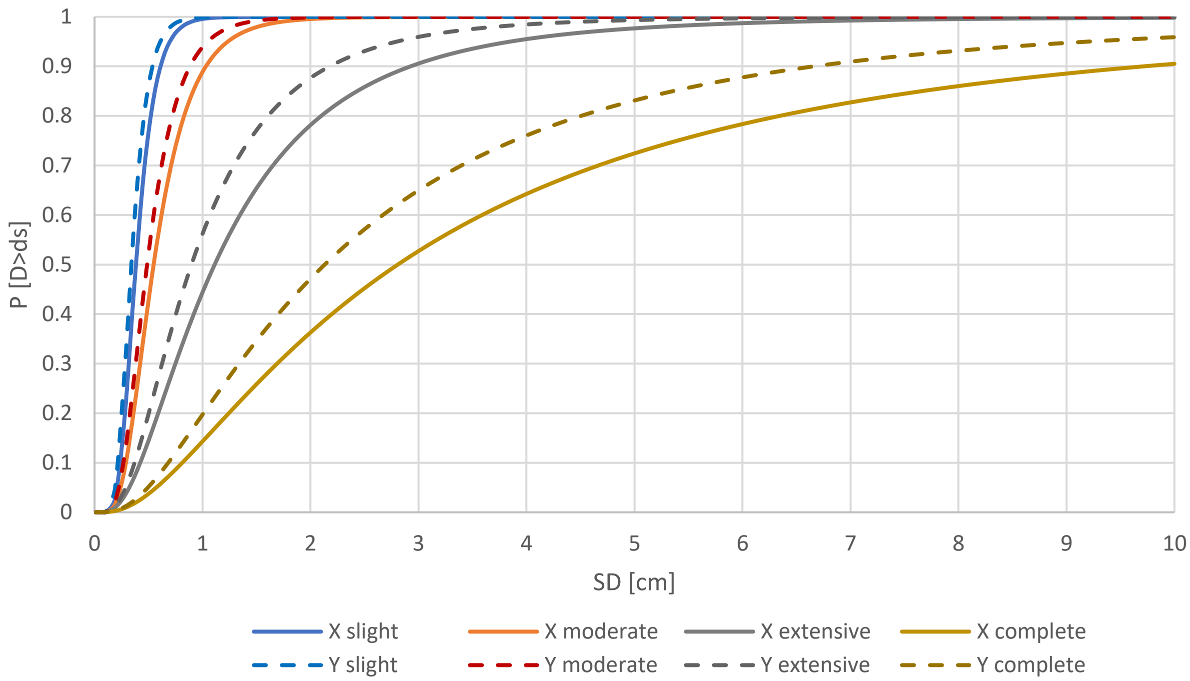


Fig. 12 Fragility functions for the retrofitted model: comparison for X and Y direction

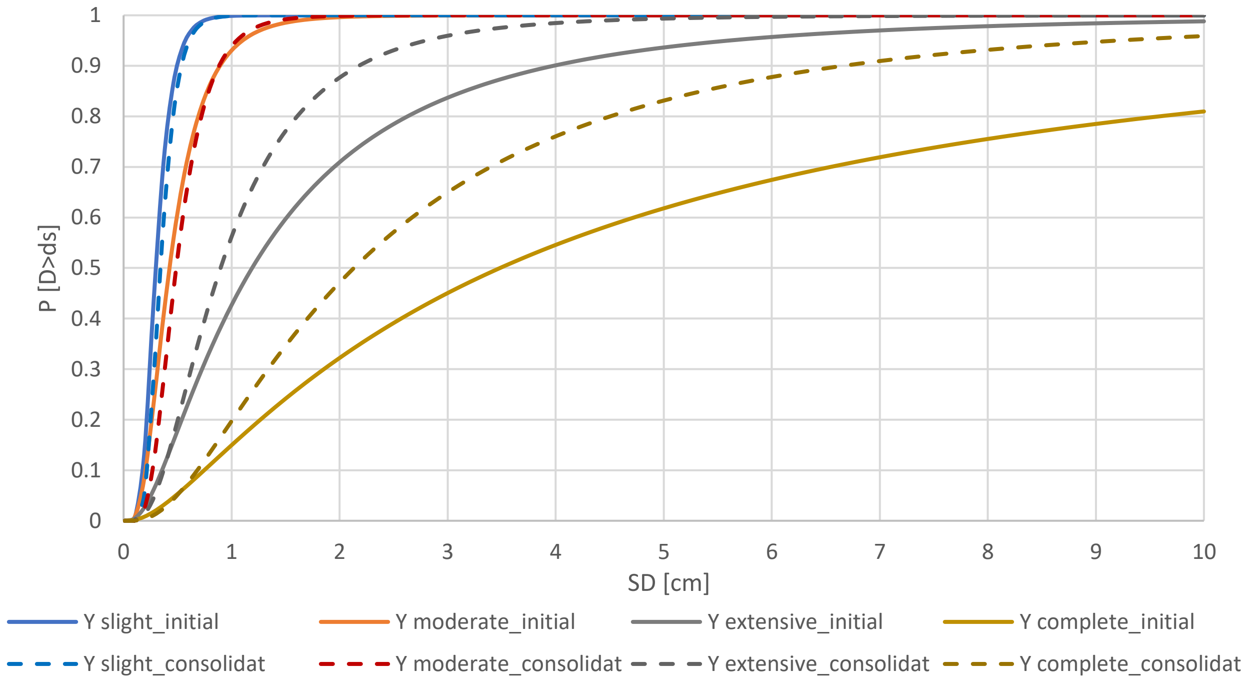


Fig. 13 Fragility functions comparison between the initial and the retrofitted model (Y direction)

Strengthening the masonry walls by RC jacketing involves substantial increases in strength, given that a new structural element is formed on the surface of the masonry wall, but does not ensure an increase in displacement. This method is almost as effective as a new reinforced concrete wall ((VTT) 2011). Such traditional methods of strengthening do not appear very often in research papers, given the growing interest in innovative and less invasive methods of intervention on existing buildings, especially for monuments. For a historic unreinforced masonry building located in Istanbul, the retrofitting proposal aimed at reinforcing the main load-bearing walls with reinforced concrete and applying FRP carbon fiber strips to the secondary walls (Ufuk Hancilar, Eser Durukal 2009). For the evaluation of the proposed solution, the verifications involved the evaluation of the ratio between the requirement and the load-bearing capacity at the wall level.

4.1.5. Alternative retrofitting solution: fragility function

The effectiveness of the retrofitting interventions carried out in 1982 could not be captured by the fragility functions. In order to present an alternative consolidation option that could lead to an increase in strength and at the same time an increase in deformation capacity, a new model of the Geology Museum has been developed where reinforced concrete jacketing interventions have been replaced with FRP carbon fiber reinforced polymer meshes.

Laboratory tests for carbon fiber-reinforced masonry walls have shown an increase in energy dissipation capacity of more than twice as much as the unreinforced masonry walls (Lozincă et al. 2016). For the material characteristics of the carbon fibers introduced in the Tremuri model, the data from the data sheet for SIKA 230C were used, material also used in the experimental campaign carried out within the Technical University of Constructions in Bucharest (Lozincă et al. 2016).

The results of the nonlinear static analysis are shown in Figs. 14 only for static analysis in the Y direction, by comparison with the initial model of the Geological Museum and the one retrofitted by jacketing. It can be seen that the level of maximum shear force reached by the FRP model reaches a level approximately equal to the maximum reached and reinforced by the jacket, but without the change of rigidity compared to the initial model and with an increase of the ultimate displacement capacity. In terms of energy dissipation capacity, the estimated increase for the area below the FRP chart is 2.65 times higher than the initial model.

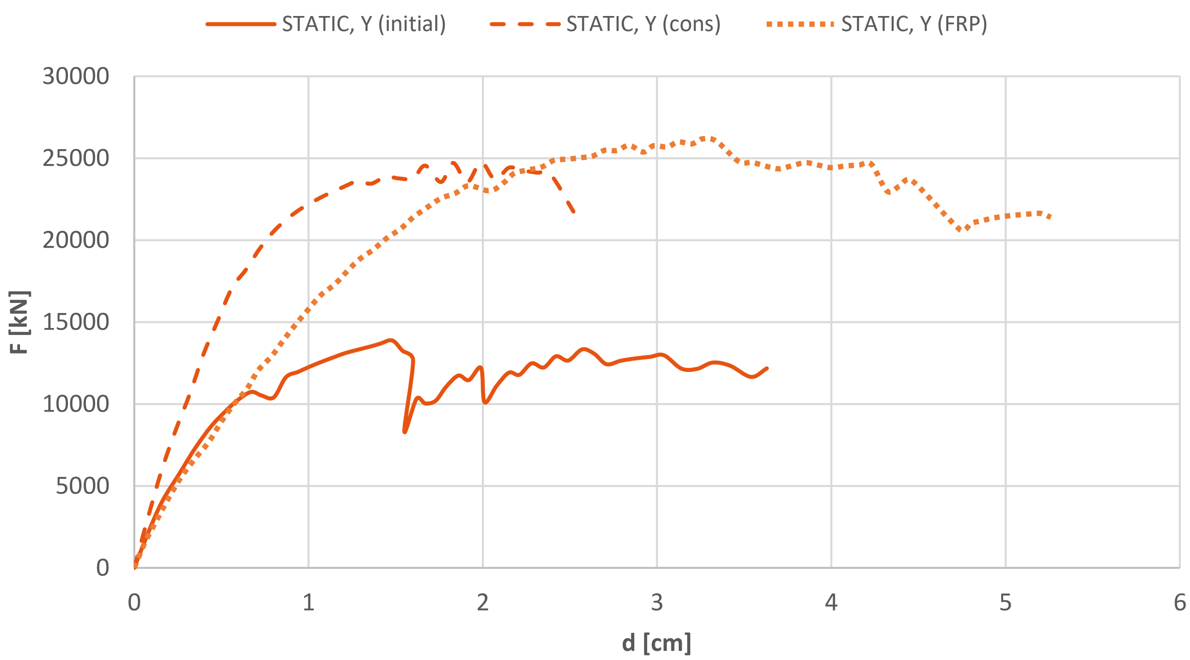


Fig. 14

Pushover curves for the initial model, the retrofitted model (jacketing) and the retrofitted model (FRP), transversal direction

The findings of the case study show that in order to highlight the effectiveness of local and non-uniform strengthening interventions, static nonlinear global analyses cannot provide relevant results, especially in the case of irregular structures. Reinforced concrete jacketing and slab strengthening were considered effective, given the behavior of the retrofitted structure subjected to earthquakes, although fragility analyses indicate otherwise. To compensate for the stiffness generated by the reinforcement method, an alternative method may be the used, which ensures both the increase in strength and the capacity for deformability. More detailed analyses should be made to optimize the use of the two methods, depending on the constraints generated by the monumental character of the building, as well as the structural performance criteria. In the process of assessing the behavior of an unreinforced masonry building subjected to seismic action, simplified calculation tools that involve modeling walls with macro-elements can capture areas with high potential for damage, taking into account assumptions about failure modes associated with each type of macro - elements. In order to extrapolate the conclusions of this case study at the typological level, several individual analyses are needed for similar buildings whose calculation models can be validated in a manner similar to the one proposed in this paper.

4.2. School building: numerical analyses, results, fragility and risk assessment

4.2.1. Brief presentation of the building

The school from Bucharest considered as case study, was built in 1884, according to the data from the seismic evaluation report. It was listed as a historical monument, being representative of the typology of heritage buildings in the education sector, built in the late nineteenth century - early twentieth century.

The structure in the initial version was made of unreinforced masonry with wooden floors, which were transformed in the 40's into rigid floors, made of reinforced concrete, 12 cm thick. After the 1977 earthquake, a small number of reinforced concrete frames (beams and columns) were added. Masonry walls have reduced thicknesses along the height (25% -30% reductions in wall areas, from one floor to another), but there are no major discontinuities in their location. The building has a basement, semi-basement, ground floor and first floor, with a wooden roof. Fig. 13 shows the three-dimensional model of the building, made in 3DMacro, where the basement was not modeled.

Currently, the school has a main building part (the old school) connected to the secondary building part by a connection part that houses the stairwell and toilets. The retrofitting project proposed in the seismic evaluation report involves strengthening interventions only for the main building part and the connecting one, as the other parts are to be rebuilt. The retrofitting interventions proposed in the seismic evaluation report aim at local repairs and the jacketing of walls with a layer of 10 cm thick shotcrete, reinforced with ø10/15x15cm (OB37) in both directions and M50T mortar, applied on one or both sides.

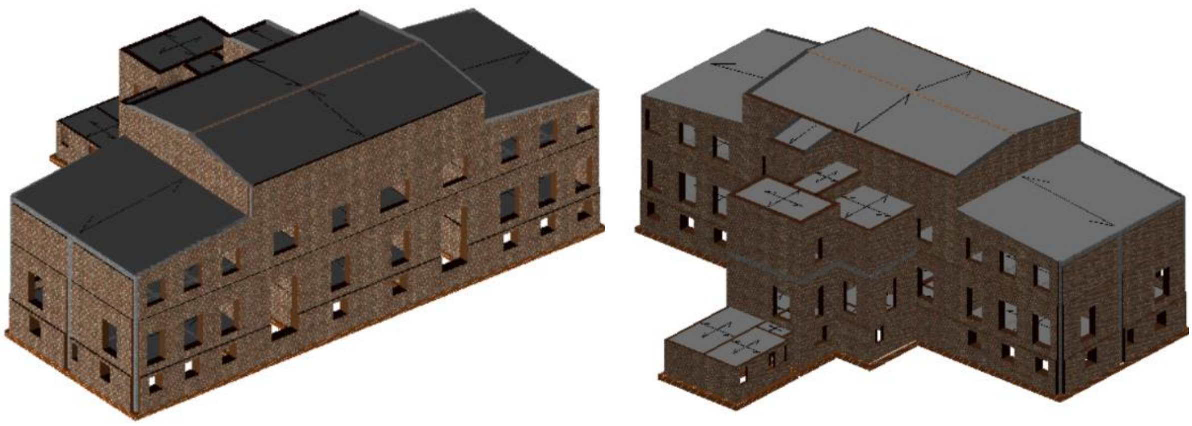


Fig. 15

3D model of the school building (3DMacro): the main building part facade (left) and the secondary building parts (right)

4.2.2. Nonlinear static analyses: numerical model

The proposed numerical models for the school building were made in the Tremuri program, according to the procedure presented in detail in section 4.1.1 and in the 3DMacro program. The latter was chosen because, unlike Tremuri, it allows the modeling of mixed structures, with the structure of masonry walls and confinement elements made of reinforced concrete (tie beams and columns). Confined masonry buildings can be considered a particular type of masonry building, although they also have characteristics similar to reinforced concrete frames. The transfer of forces between reinforced concrete frames consisting of columns and beams and masonry walls involves defining an interaction between the two materials (Marques and Lourenço 2014).

Starting from the data regarding the analyzed structure, four numerical models of the school were made, using the 3DMacro program for some of the nonlinear static analyses. The unreinforced masonry model, hereinafter referred to as URM, has a structural conformation close to the shape in which the building was built in 1884. It keeps the unreinforced masonry walls without columns, but also contains the reinforced concrete slabs that were added in year 1947, over the original floors with wooden beams and filling. Fig. 16 illustrates the areas in which the above strengthening measures have been applied.

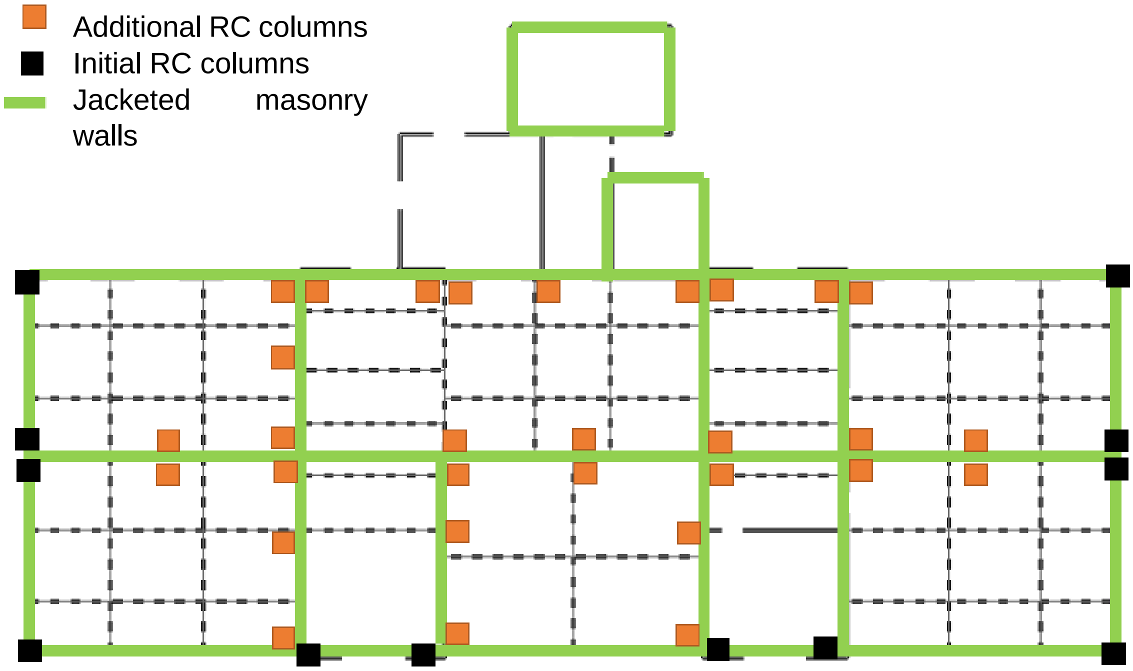
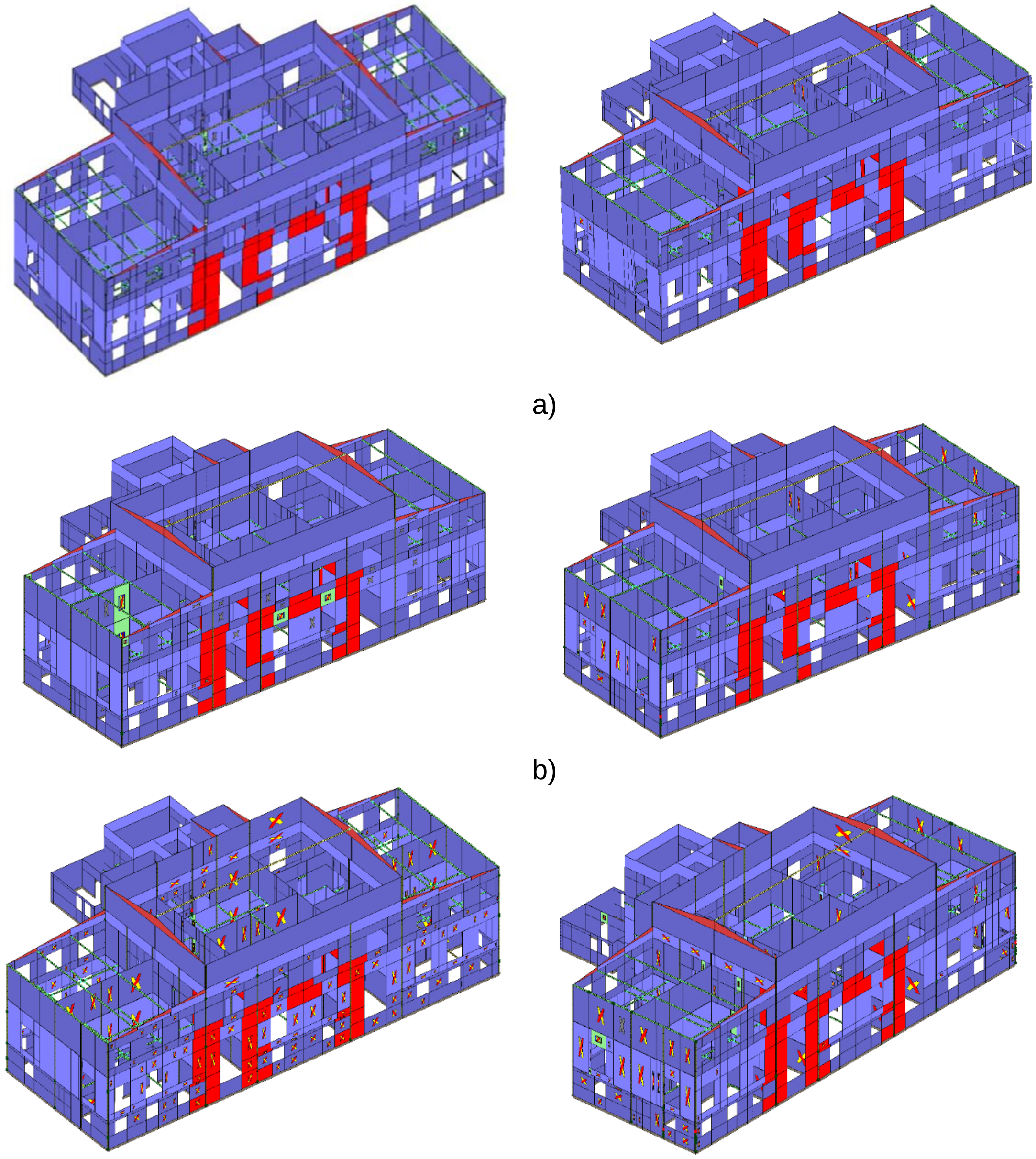


Fig. 16 Retrofitting measures (semi-basement level)

4.2.3. Nonlinear static analyses: results

The predominant failure modes are represented by inclined cracks in the masonry panels, which appeared in the case of the + X analysis from the first steps, at a relative displacement of 0.007%. Subsequently, for a relative displacement of 0.08% and PGA of 0.204 g, the first shear failure appears, in the wall no. 2 of the façade. As can be seen in Figs. 17, the masonry panels suffer extensive damage, but controlled in the case of the jacketed model, where the final displacement reaches values higher than those in the URM model. The structural conformation of the initial model, where reinforced concrete columns are present, contributes to the redistribution of efforts between the elements and thus prevents the local collapse, specific to the unreinforced masonry buildings. The presence of confinement elements generates failure mechanisms presenting diagonal cracks in the walls, subsequently contributing to the formation of plastic joints in the reinforced concrete elements. Similar analyses in the literature indicate similar behaviors of confined masonry buildings, where for 1 cm displacements the confined masonry has extensive but controlled damage (Marques and Lourenço 2014).



a)

b)

c)

Fig. 17 Damage states for structural elements for: a) URM model, b) Initial model, c) retrofitted model

However, both sets of results illustrate the major differences between the responses obtained for the two directions. For the transverse direction, the first cracks and the first yields appear at values 50% lower than those in the longitudinal direction, given the asymmetrical layout of the building. The jacketing interventions and the addition of RC columns are focused on the longitudinal direction of the building, the direction in which most of the masonry walls are located. Thus, the effectiveness of the interventions is better highlighted by the comparative results from the X direction.

4.2.4. Nonlinear static analyses: global and relative displacements

Using the global relative displacement limits presented in the literature, the color code indicated in Tab. 5 was overlapped with the Pushover curves of the URM model, to indicate the thresholds corresponding to each performance criteria (D0 - D3). The last recorded movements indicate that the safety limit (D2) has been exceeded for all cases, except for the analysis in the longitudinal direction, positive direction, for which the damage control state (D1) is reached.

Most masonry structures in Romania, especially those made of confined masonry and those with rigid reinforced concrete floors, are made of interlocked brick walls at intersections. For this reason, the results of the study by Vatteri and D'Ayala (Parammal and Dina 2021) on the limits of global relative displacement for confined masonry buildings yielding to the plan were used. The authors present a collection of studies in the field and finally ranges of values for the global relative displacements established on their basis. The three limit states considered correspond to the occurrence of the first crack (performance criterion: immediate occupation), significant damage and detachment of masonry walls from confinement elements (performance criterion: safety of life) and pre-collapse (performance criterion: prevention of collapse). The proposed average intervals are noted in Tab. 5 for the category of confined masonry buildings.

Both the initial and the retrofitted model reach final displacements that exceed the collapse prevention limit, thus having a higher capacity than the displacement requirement associated with the collapse prevention state. The unreinforced masonry model, on the other hand, reaches values even lower than the requirement for immediate occupation, which indicates the need for strengthening measures.

Tab. 5 Global drift limits for URM and CM structures

		URM ((Derakhshan and Griffith 2018), (Cattari et al. 2015))	CM (Parammal and Dina 2021)
D0	Immediate Occupancy (**IO**)	0% - 0.06%	0% – 0.125%
D1	Damage Control (**DC**)	0.06% - 0.1%	0.125% – 0.4%
D2	Life safety (**LS**)	0.1% - 0.2%	0.3% – 1.39%
D3	Collapse Prevention (**CP**)	≥ 0.2 %	1.54% – 4%

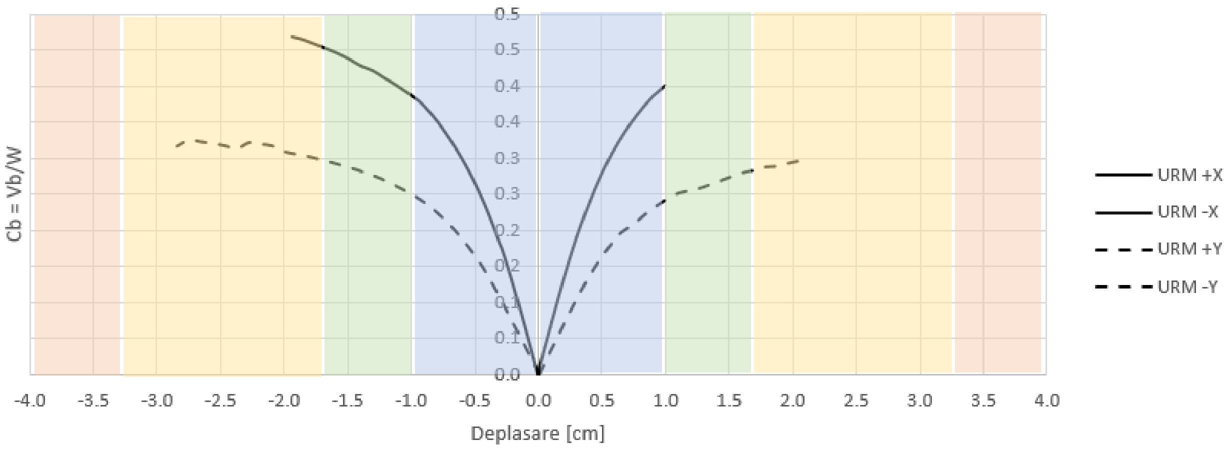


Fig. 18 Performance limits – URM model

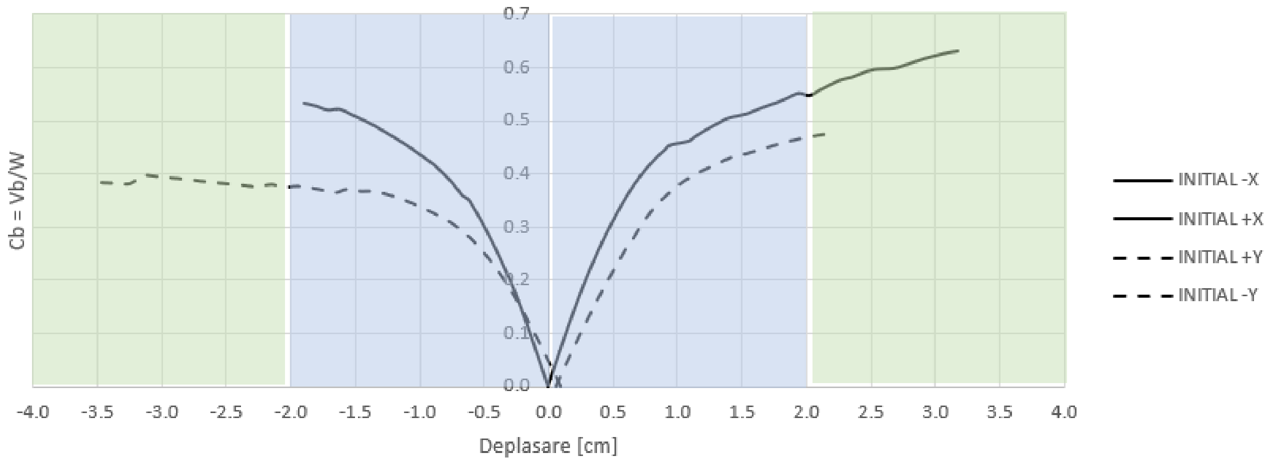
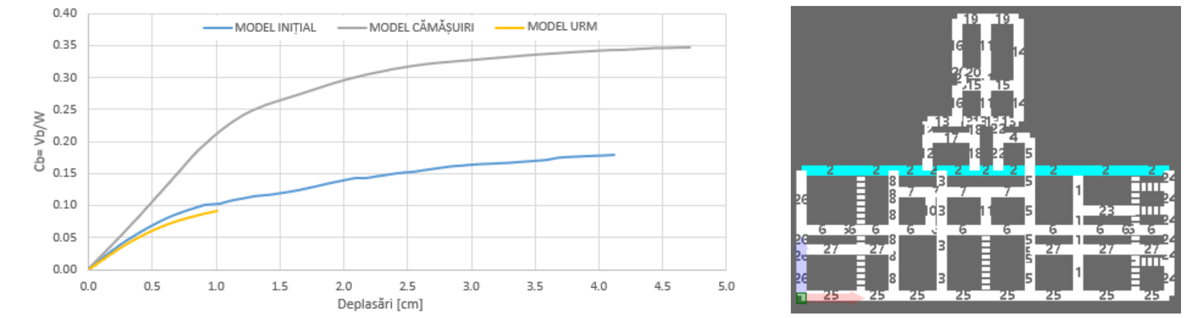


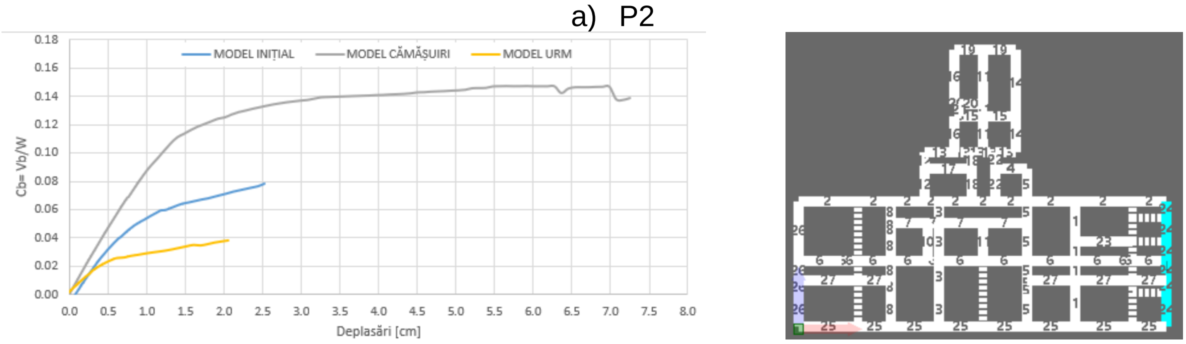
Fig. 19 Performance limits – initial model

In order to be able to estimate the efficiency of the partial retrofitting works (initial model) and final (jacketed model), four walls were chosen for a more detailed analysis. They were compared in terms of damage level, the direction in which they are placed and the differences in between jacketed and not jacketed walls. The two retrofitted walls considered below are: P2 (west facade, longitudinal direction) and P24 (north facade, transverse direction), and the not retrofitted ones are: P27 (inner wall, longitudinal direction) and P12 (transversal direction).

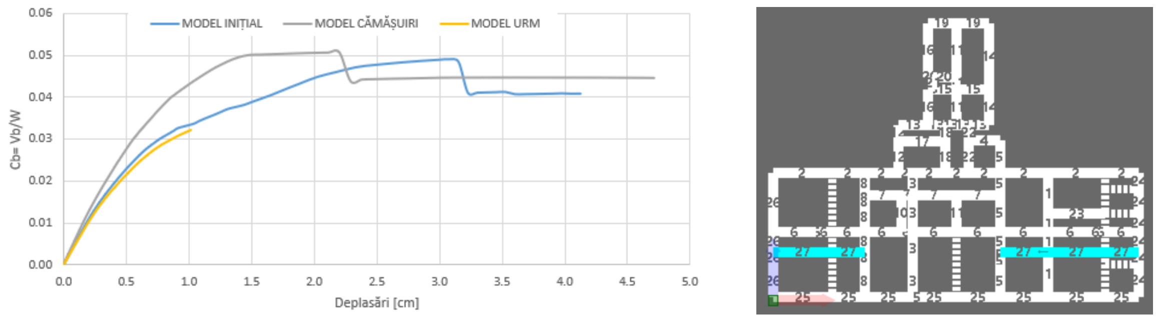
Comparing the results obtained for the jacketed walls with those that have not been retrofitted, major differences can be observed. Wall 27 and wall 12 show no increases in the lateral force capacity for the evolving models. Moreover, in the case of the retrofitted model, the P27 shows local failures marked on the Pushover curve by sudden decreases, which occur for smaller displacements than in the case of the initial model. The walls chosen to be retrofitted, such as P2 and P24, contribute significantly to the improvement of the structural capacity, but it should be emphasized that most of the walls have benefited from such strengthening interventions, as illustrated in Fig. 16.



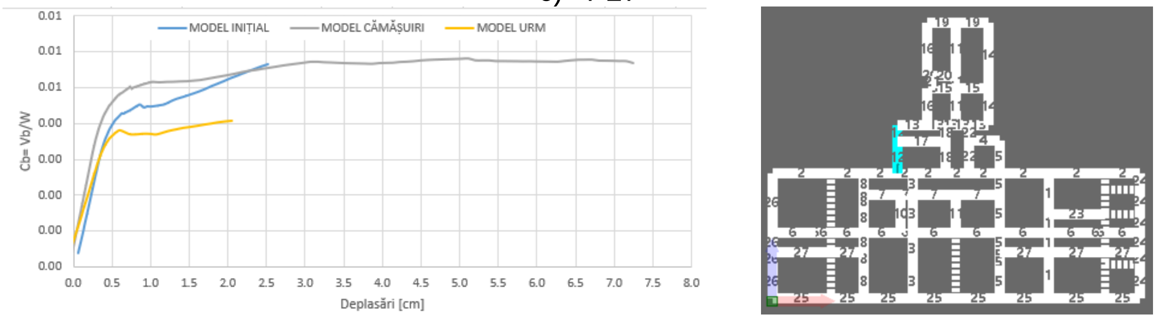
a) P2



b) P24



c) P27



d) P12

Fig. 20 Pushover curves for the retrofitted walls (a, b) and not retrofitted walls (c, d)

Performance criteria and associated damage also take into account the behavior of masonry walls in the out-of-plane. Numerical analyses performed in 3DMacro do not capture these aspects, but it can be considered that the reinforced concrete slabs ensure the rigid box behavior, which prevents out-of-plane failures. Comparing the damage to the walls of the initial model with those described for the performance levels of FEMA 356 for reinforced masonry, there is a clear exceedance of the immediate occupancy limit (minor cracks) and a possible compliance with the life safety requirements (extended cracks, distributed along the walls and some isolated failures of the main elements). The requirement for collapse presentation would involve extensive cracks and considerable damage around openings and at the corners of the building, were parts of the masonry should have collapsed. The limited amplitude of cracks and local failures does not indicate exceeding the criterion of life safety, hypothesis confirmed by the drift checks, which have values below 1.5%, the relative displacement indicated in the code as limit for collapse prevention.

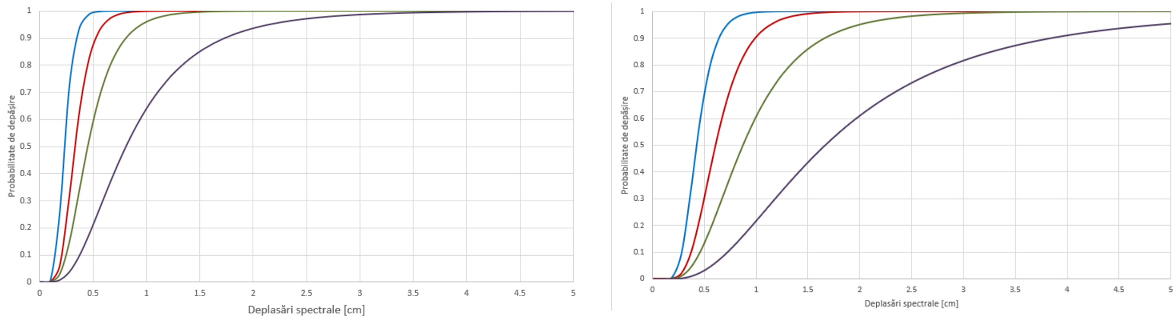
In conclusion, the relative maximum displacements in the models correspond to the values proposed for reference in FEMA 356 for the three types of vertical structural elements, taking into account the extent of the damage of the masonry panels. In what concerns the target displacement estimates, the unreinforced masonry model fails prematurely before the performance criterion corresponding to immediate occupation.

4.2.5. Nonlinear static analyses: fragility functions

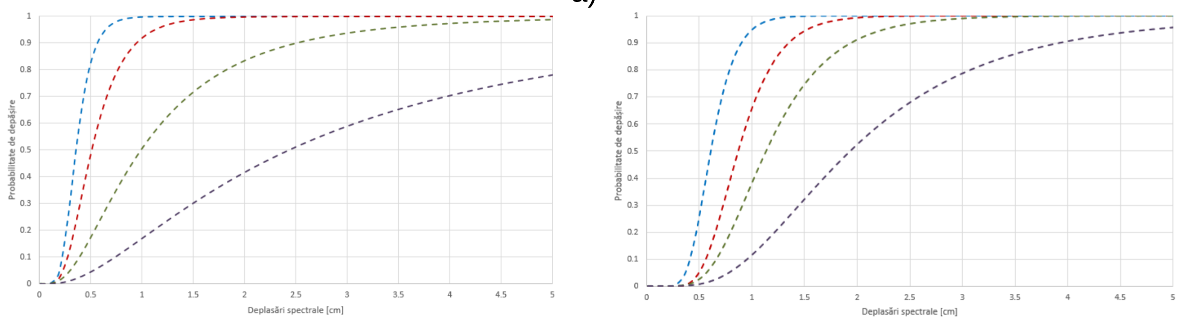
The fragility functions establish relationships between the intensity of the seismic action and the level of damage, through conditioned probabilities. The procedure applied to the first case study and described in Chapter 3 was also used for the school building.

The fragility functions expressed in spectral displacements for the four models analyzed are illustrated in the images below, for the X and Y direction, positive direction. For the hypothetical URM and the initial model, there are significant differences between the two directions, while for the retrofitted model, the results obtained for X and Y are closer.

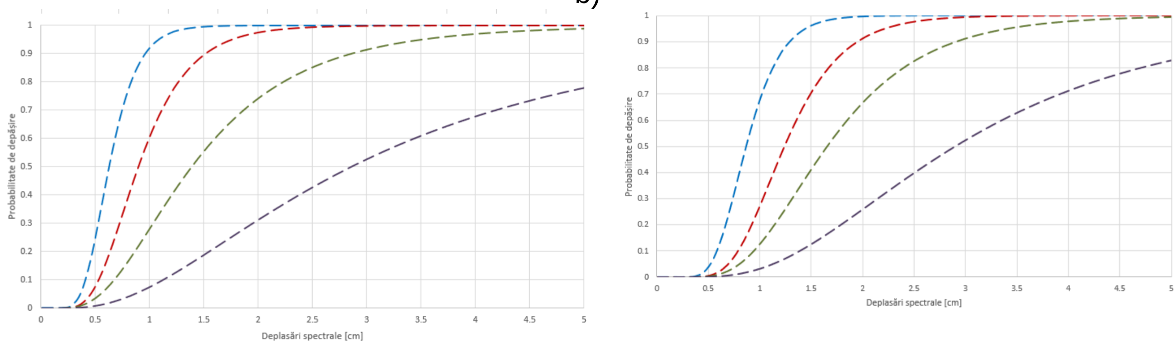
For the same level of spectral displacement, the graphs in Fig. 21 illustrate the influence of gradual strengthening, starting from high values for the probability of extensive/complete damage to the unreinforced masonry building (URM) and reaching values below 10% for the jacketed model. The most significant reductions are recorded for higher values of spectral displacements and for the X direction. In these cases, there are reductions of more than 50% of the probability of being in the complete damage state. Changes can also be observed between the URM model and the initial one, thus highlighting the contribution of the RC columns, even if they are not placed as densely as those from the final, strengthened model.



a)



b)



c)

Fig. 21 Fragility functions: +X direction (left) and +Y direction (right) for: a) URM model, b) initial model, c) retrofitted model

5. EDUCATION SECTION: SEISMIC RISK ANALYSIS

5.1. Introduction

Losses in the education sector caused by earthquakes around the world are a strong argument for promoting public policies to reduce seismic risk. The International Labor Office (CRISIS 2010) report on reducing the impact of earthquakes shows the losses caused by some of the most devastating earthquakes in the last 20 years. Examples include the tragedy caused by the Molise earthquake in Italy in 2002, which killed 26 of the 51 students in the building of a collapsed school in San Giuliano. The earthquake in Bingol, Turkey in 2003 caused the collapse of 4 schools and 9 were moderately damaged, causing 84 casualties. Even more devastating earthquakes occurred in Pisco, Peru (2007) where 18 buildings were completely destroyed and 118 were damaged by the earthquake, or in Haiti (2010) where 97% of schools in Port Principe were destroyed. More than 90,000 students in Sumatra were affected by the 2009 earthquake and 241 schools collapsed (International Labor Office / CRISIS 2010).

According to a study by Beraldo et al. (2009) between 1971 and 1998 in 19 countries, estimates of the impact of investment in education on gross domestic product can be quantified by a 0.03% increase in GDP, for an increase of 1% in investments in the education sector. Studies in this regard emphasize the importance of prioritizing investments, identifying potential losses and preparing retrofitting programs based on techniques appropriate to the needs of existing building stock.

Looking at the building stock in Romania, the oldest schools in the country were initially organized around the churches, and later, most of the school buildings built in the second half of the 19th century were organized similar to the mansions in the cities or even dwellings were transformed into educational buildings, most often with ground floor and first floor only (Tănăsoiu 1979).

The end of the century generates changes in the architecture of schools in Romania, bringing in the pattern of "academic schools", characterized by rigorous symmetries and large capacities compared to the buildings used until then. At the beginning of the 20th century, over 20,000 students attended 69 primary schools in Bucharest and another 6,000 were enrolled in secondary education (Tănăsoiu 1979). Since the 1920s, these secondary school buildings have been monumental constructions, especially those located in the big cities of the country. With the political and social changes of 1945 and the widespread use of reinforced concrete, the structures of school buildings have undergone drastic changes, adapting to the increased number of students. If at the end of the 40's there were about 1.600.000 students in elementary school and high school, in the 70's the numbers reached almost 3.400.000. In order to cover the need for educational units, the design institutes (I.P.C.T.) worked on standard projects, both for the urban and the rural environment (Tănăsoiu 1979).

5.2. Current situation

5.2.1. Description of the sample analyzed

The building stock from the education sector (kindergartens, schools, high schools, colleges, dormitories or children's clubs) consists of unreinforced or confined masonry buildings in a proportion of approximately 60%, according to data collected nationwide in the database from the Integrated Informatics of Education in Romania (SIIIR). Only about 20% are unreinforced masonry buildings, but more than half were built before 1920. Masonry structures with columns and reinforced concrete tie beams are the most common structural system and more than two-thirds of these buildings have been built. built before 1977.

These statistics justify the concern for a typological analysis of masonry buildings, while also considering the peculiarities of buildings classified as historical monuments. The approximately 550 buildings analyzed below are established based on the List of Historical Monuments (2015), where they were marked as having functions of kindergarten, school, high school or college. University buildings have been excluded from this analysis. A communiqué of ICOMOS Romania from 2013 (Nistor 2013) mentioned that there are 605 buildings belonging to the education sector, being included here also faculties, university headquarters and libraries.

The map in Fig. 22 illustrates the distribution of buildings classified as historical monuments, used in the education sector. The symbols represent the values normalized to the maximum number of buildings in Bacău County, so that comparisons between counties are possible. The counties with the most buildings are located mainly in the south (Teleorman, Dâmbovița), northeast (Vaslui, Iași, Bacău, Neamț) and northwest (Maramureș, Cluj).

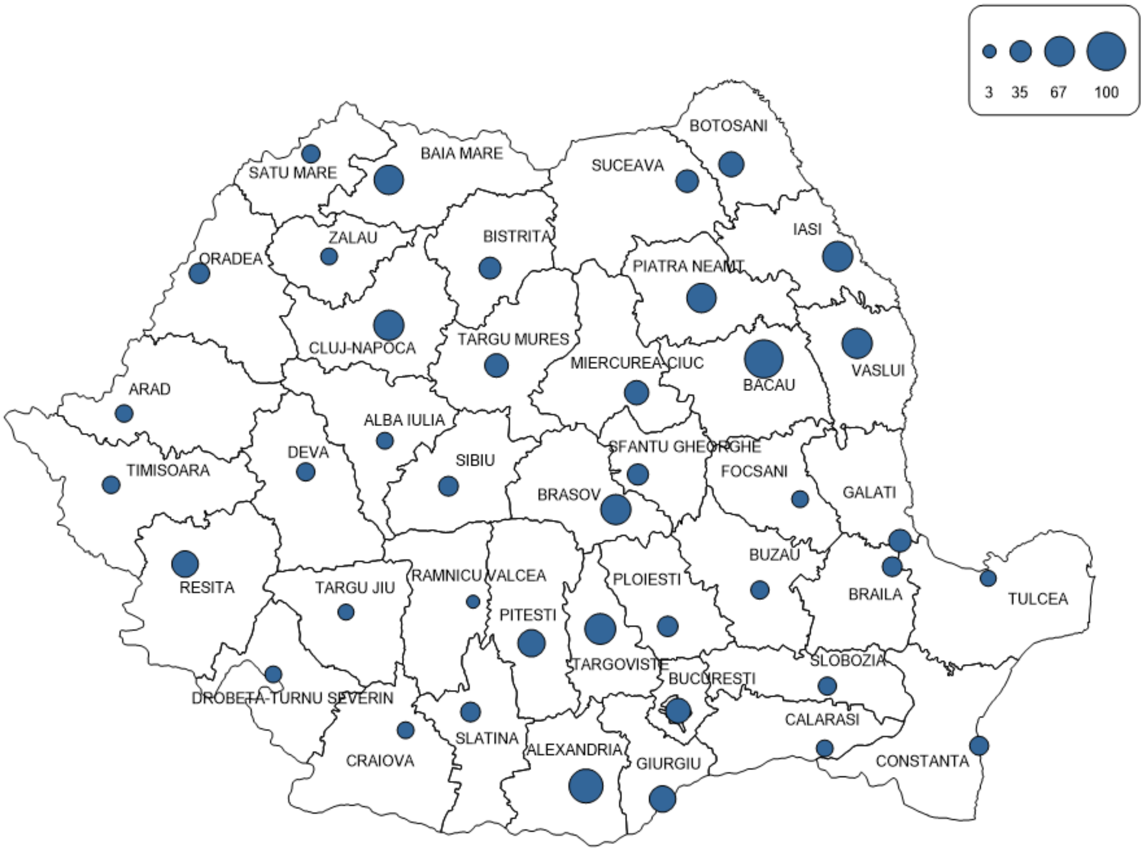


Fig. 22 The number of heritage buildings from the education sector

Considering the value of the buildings that are part of the education sector, given their historical and architectural importance, but also their age, it is necessary to analyze the seismic risk associated with them. The recommended approach involves establishing structural typologies that allow the initial filtering of a large number of buildings through objective criteria in order to estimate the expected structural response.

Measures to reduce seismic risk for monumental buildings in the education sector will be analyzed through a case study of a school whose structural upgrades are representative for retrofitting interventions on masonry buildings in Romania. The results will be compared with studies in the literature and later on, fragility characteristics will be established for three types of masonry buildings: unreinforced masonry, confined masonry and reinforced masonry (invasive retrofitting measures).

Seismic risk analyses for the entire building stock require information on exposure, in the form of inventories on buildings and people exposed. For the public schools in the pre-university education sector, an Integrated Information System of Education in Romania (SIIIR) has been implemented, which contains also data relevant for the analysis of the schools buildings. They were further used to assess the exposure and vulnerability of buildings.

The main attributes required for the analysis include: the structural system (as well as the type of floor systems for masonry buildings), the year of construction, the location, the built-up area, the height regime, the number of students enrolled and information on the existence of retrofitting works.

The analyzed sample for which sufficient attributes were available is 29.284 buildings from over 15.900 educational units, distributed throughout the country. Out of these, 2% are buildings listed as historical monuments (2015) (National Heritage Institute 2015) from 315 school units that could be correlated with the data from the initial list of 546 monuments. The difference is represented by buildings classified as historical monuments in the education sector, but later transformed into administrative buildings, museums or housing units (Commercial School, Giurgiu – currently used as Design Institute, Normal Boys School, Vaslui - today working as a hospital), schools that were demolished (Obedeanu School, Craiova) or which could not be identified in the SIIIR database (Bulbucata Kindergarten, Giurgiu).

Starting from the list of structural systems predefined in SIIIR, the following typologies were considered:

- CM – confined masonry
- URM – unreinforced masonry
- RC_F – reinforced concrete frames
- RC_W – reinforced concrete shear walls
- W – wood
- S – steel
- RC_LP – reinforced concrete large panels

Out of the total sample of buildings, over 50% of them have masonry structural systems (unreinforced or confined) and about 40% are reinforced concrete structures. Only 4% of all buildings are made of reinforced concrete shear walls or wood and less than 2% are made of steel frames or large prefabricated concrete panels.

The largest share is represented by buildings built to withstand only gravitational actions, constructed before the appearance of seismic calculation regulations (P13-63, P13-70). Only for 30% of the total buildings analyzed, the level of seismic action was estimated using the calculation codes developed after the 1977 earthquake (P100-78, P100-81, P100-90, P100-92, P100-1 / 2006, P100-1 / 2013). Between 1964 and 1992, about a quarter of the total schools analyzed were built, 85% being reinforced concrete buildings, buildings made according to standard plans.

5.2.2. Retrofitting interventions

The list of retrofitting works carried out within the P.R.I.S project were considered together with the analyzed sample from the SIIIR database to evaluate data on strengthening interventions at national level. From the data available for 1253 units (93% of the total educational units included in the program), seismic upgrading works were carried out for about 35% of them. Only 4% of the retrofitted schools have buildings listed as monuments, namely 17 units, out of which 9 are high schools or colleges and 8 are general schools. The cost values per m^2 evaluated for the monument buildings are between 70 €/sqm and 530 €/sqm, with an average of 240 €/sqm, below the estimated value for the retrofitting of the entire sample of investments through P.R.I.S.

5.3. Fragility functions

5.3.1. Taxonomy

The fragility functions used further on to characterize the structures in the portfolio of buildings in the education sector were established for the five representative typologies: unreinforced masonry structure with flexible (URM_FF) or rigid (URM_RF) floors, unreinforced masonry structure reinforced by RC jacketing (RM), confined masonry, with reinforced concrete columns and tie beams (CM) and confined masonry reinforced by RC jacketing (CM_RM).

The results presented below for the types of confined masonry (CM and CM-RM) were obtained from numerical models made using the 3DMacro program. In order to better illustrate the differences between unreinforced masonry buildings with flexible floors (URM_FF) and those with rigid floors (URM_RF), as well as those with retrofitted unreinforced masonry (RM), the Tremuri program was used for the numerical models.

Based on the information presented in section 4.2.1, the model of the building with flexible floors has masonry vaults and steel beams over the basement level and wooden beams for the rest of the floors. These were transformed into rigid diaphragms to capture the initial stage of strengthening the building by stiffening the floors and adding reinforced concrete beams. Starting from the layout of the model with rigid floors (URM_RF), for the walls reinforced by jacketing, a different material was defined, keeping the characteristics of the masonry and modifying only the allowable drifts for bending (2% compared to 0.8%) and flexure (1 % compared to 0.4%). These values are recommended in the regulations of Italy (Ministero delle Infrastrutture e dei Trasporti 2018), (Ministerio delle Infrastrutture e dei Transporti 2009) and they are used in simplified numerical analysis to quantify the increase in ductility of the strengthened masonry. Also, the steel reinforcement characteristics were added for the jacketed walls, according to the retrofitting design proposal included in the seismic evaluation report.

5.3.2. Fragility functions comparisons based on proposals from literature

The fragility functions establish relationships between the intensity of the seismic action and the level of damage, through conditioned probabilities. Fragility functions are defined using the median values of spectral accelerations/displacements and standard deviations, taking into account such uncertainties associated with each state of damage, according to the procedure presented in Chapter 3.

Based on the case study conducted for the school building in Bucharest, fragility functions were proposed for each type of masonry building: unreinforced masonry (URM) with flexible (FF) or rigid (RF) floors, confined masonry (CM) and reinforced masonry (RM) or reinforced confined masonry (CM-RM). In order to validate the fragility parameters proposed for the building stock in Romania, comparisons were made with similar studies in the literature.

Tab. 6 presents the centralized results for the parameters of the fragility functions with median values expressed in spectral displacements.

Tab. 6 Fragility functions parameters per typology (spectral displacement)

Spectral displacements [cm]			
Slight damage	Moderate damage	Extensive damage	Complete damage

	Median	β	Median	β	Median	β	Median	β
URM_FF	0.24	0.35	0.35	0.45	0.62	0.66	1.42	0.85
URM_RF	0.25	0.38	0.36	0.53	0.81	0.83	2.18	1.06
CM	0.36	0.36	0.51	0.48	0.99	0.72	2.43	0.93
RM_RF	0.47	0.37	0.68	0.52	1.51	0.81	4.00	1.04
CM_RM	0.63	0.33	0.90	0.41	1.39	0.56	2.86	0.73
CM_RM	0.89	0.33	1.28	0.41	1.98	0.56	4.07	0.73

Finally, in order to have a clearer picture of the differences regarding the behavior of the typologies proposed in this paper and the references in the literature, the graphs in Fig. 23 and Fig. 24 illustrate the average damage degrees expected for a set of spectral displacements of 1 cm, 2 cm and 3 cm. For small spectral displacements, the jacketed confined masonry structures reach minimum degrees of damage, but for spectral displacements of 2 or 3 cm, CM-RM and RM reach close values. The only typology for which the comparison with the cited studies does not indicate very close values is CM-RM. Given that the interventions included in the CM-RM numerical model are much more invasive than those presented in Marques et al. (2018), justifies the difference of approximately - 10% for the damage degrees from Fig. 23.

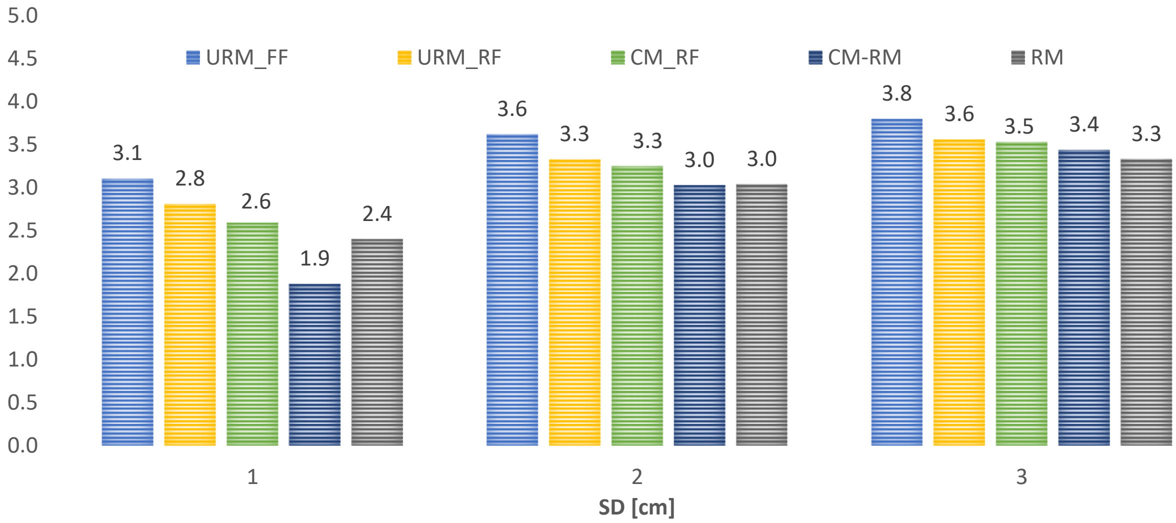


Fig. 23 Mean damage degree for masonry typologies in Romania (SD)

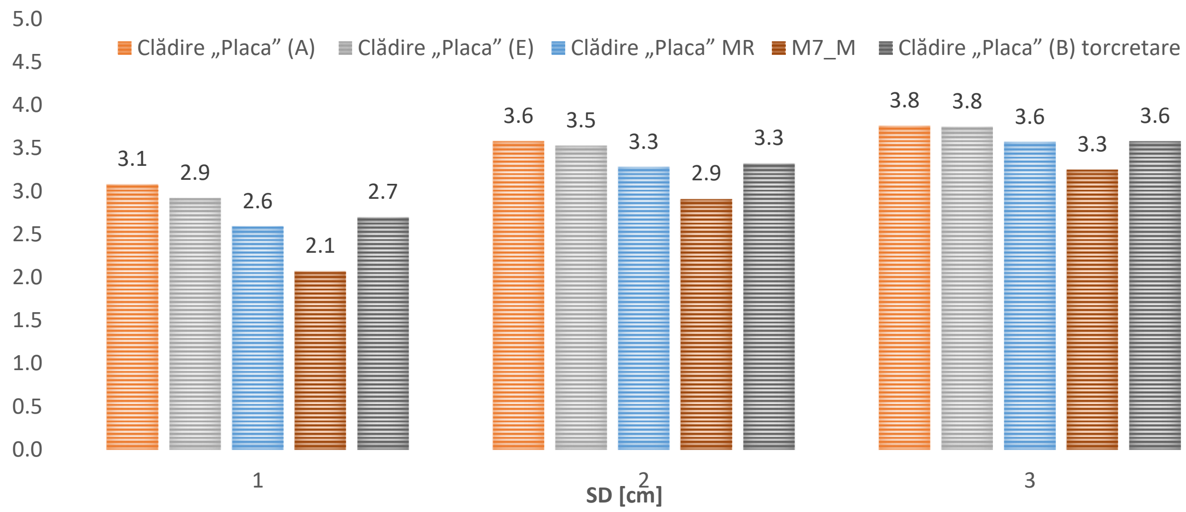


Fig. 24 Mean damage degrees for corresponding typologies from literature (SD)

Due to the need for fragility functions expressed in accelerations, that are needed for the convolution integral in the risk calculations, the median values of the spectral displacements from the fragility functions were transformed into spectral accelerations. Based on these, the average damage degrees were estimated for each of the five typologies, for two reference values of spectral accelerations: 0.5g and 1g. In Fig. 25 it can be observed that for the CM-RM typology the expected average degrees of damage are significantly lower than for the RM-typology. Also, for the CM typology, the damage degrees are significantly lower than for RM, a trend that was not so pronounced in the case of the average degrees of damage calculated for spectral displacements.

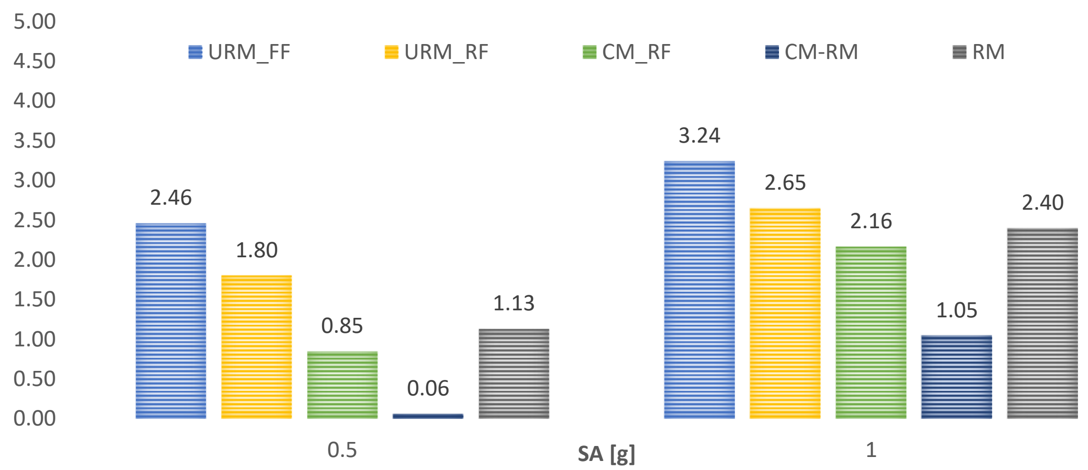


Fig. 25 Mean damage degree for masonry typologies in Romania (SA)

5.4. Seismic risk assessment

5.4.1. Brief presentation of the method used for the estimation of annual failure probabilities

Seismic risk analysis involves estimating annual probabilities of failure, based on fragility functions and seismic hazard, considering a permanent exposure. The amplitude of the seismic motion is estimated based on the mean return periods, magnitude, location and effects produced by earthquakes, together with the uncertainties associated with the analysis. The data on average annual exceedance rates which were developed in the national research project BIGSEES (Pavel et al. 2016) included 14 seismic sources, aiming to obtain aggregate seismic hazard curves for all localities in Romania. The exceedance probability P_F of a certain damage state is calculated following the procedure presented above in Chapter 3.

5.4.2. Representation of annual failure probabilities

In order to represent on the maps the annual exceedance probabilities for DS3 and DS4 damage stages, the absolute values were grouped in 5 intervals, ranging from 0.01 to 0.000001, according to the color legend in Fig. 26.

In Tab. 7 it can be observed which are the counties in which the seismic risk estimated by means of the annual probability of failure is maximum. In principle, the northwestern area has the lowest risk, taking into account the fact that the Vrancea seismic source mainly influences the southeast of the country. Analyzing the transition from DS43 to DS4, a decrease in the annual exceedance probabilities can be observed but overpassing an extensive damage level (DS3) for an unreinforced masonry building involves extensive cracking of walls, possible collapse of gable walls or parapets. To exceed the complete damage stage (DS4) means that there is a probability of approximately 15% of the complete collapse of the building (Federal Emergency Management Agency 2015).

The structural typology that records the highest values for the annual failure probabilities is the unreinforced masonry with flexible floors, at the opposite pole being the confined masonry buildings retrofitted by RC jacketing. It is important to note that for both DS3 and DS4, buildings with confined masonry present a lower risk than those with unreinforced masonry strengthened by RC jacketing. These results confirm the benefits of confinement elements such as RC tie beams and columns.

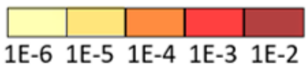
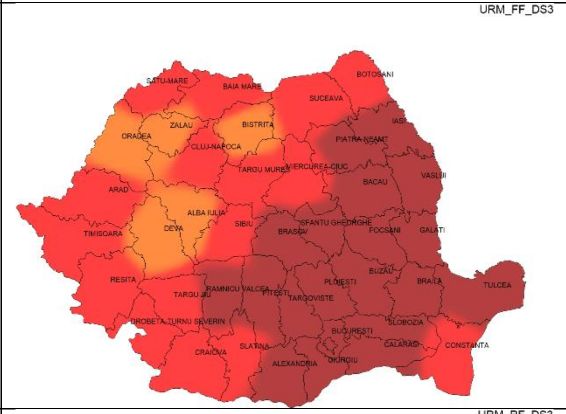
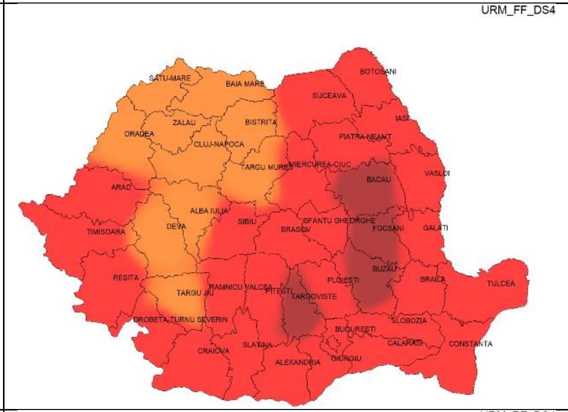
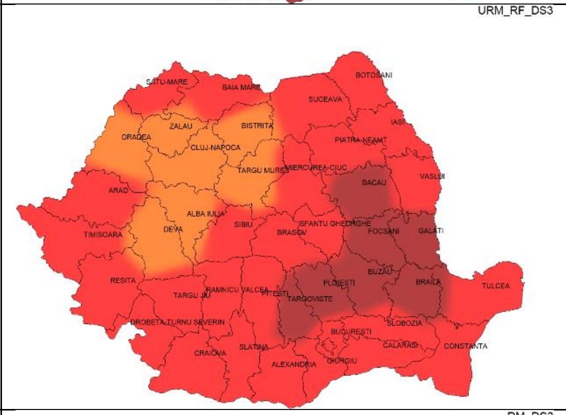
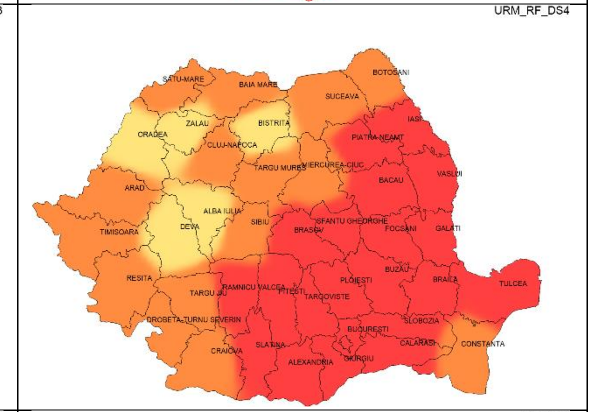
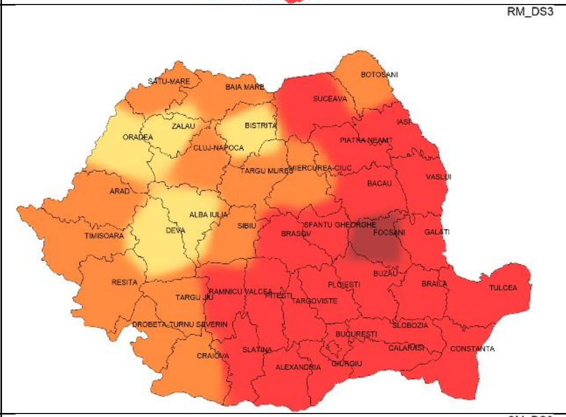
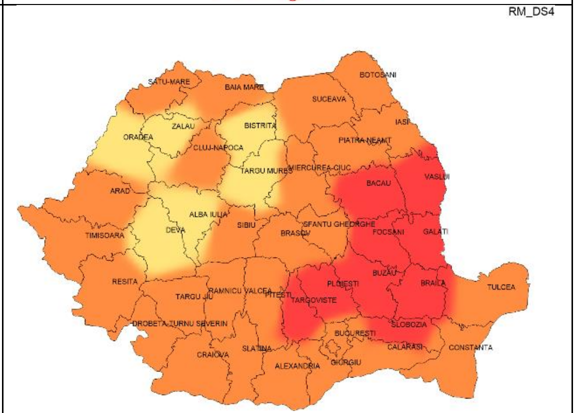
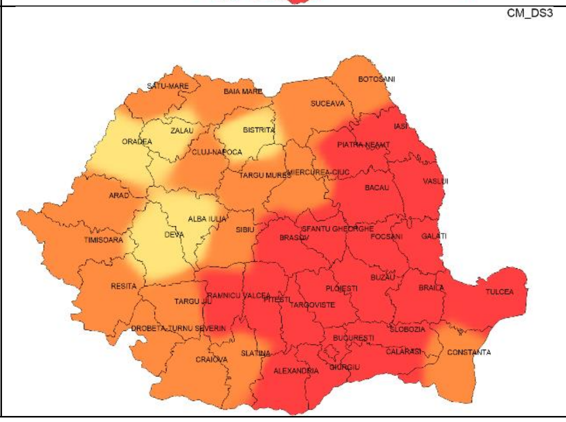
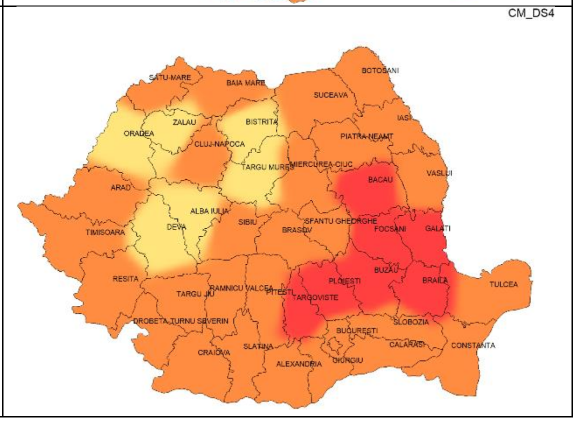
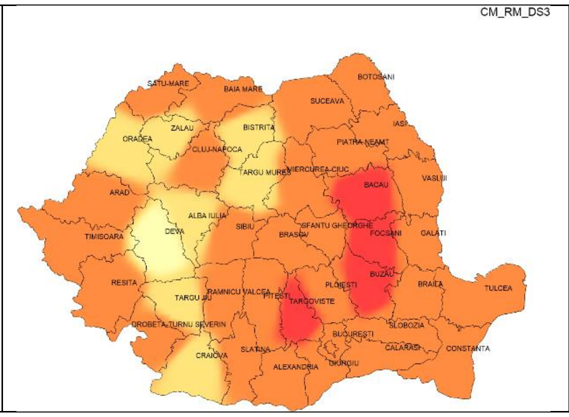
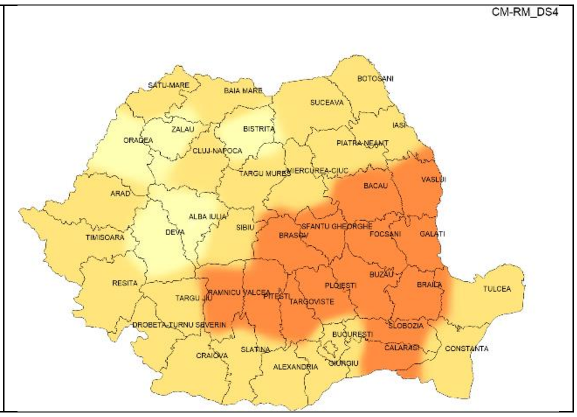


Fig. 26 Annual failure probabilities interval: color legend

Tab. 7 Seismic risk maps: annual failure probabilities for masonry structures

<table>
<tr><th>Structural typology</th><th>Exceedance probability DS3</th><th>Exceedance probability DS4</th></tr>
<tr><td>URM_FF</td><td>
</td><td>
</td></tr>
<tr><td>URM_RF</td><td>
</td><td>
</td></tr>
<tr><td>RM</td><td>
</td><td>
</td></tr>
<tr><td>CM</td><td>
</td><td>
</td></tr>
</table>

CM-RM		

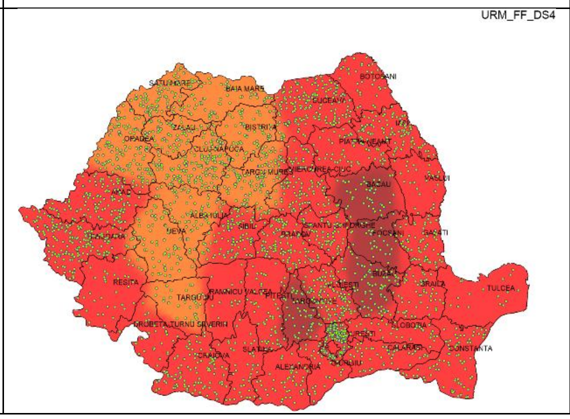
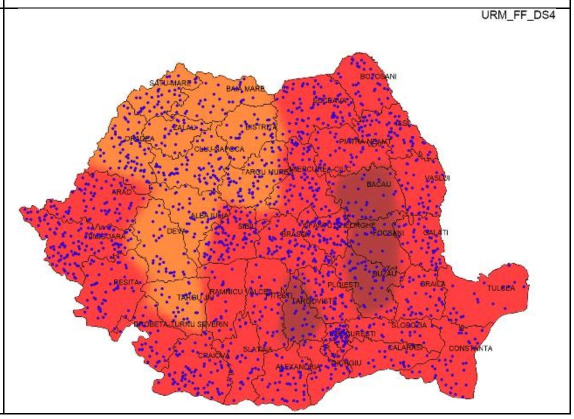
5.4.3. Overlapping exposure data with seismic risk data

Tab. 8 illustrates the exposure in the education sector, the data being centralized at county level. It shows the absolute number of people exposed, respectively the total number of students enrolled in each building and the number of buildings, included in each of the typologies analyzed. Since there is no information on the strengthening interventions carried out so far in the education sector, seismic risk maps obtained for the CM-RM typology were used, taking into account the fact that unreinforced masonry buildings have often been transformed at least partially into confined masonry in order to comply with current design requirements.

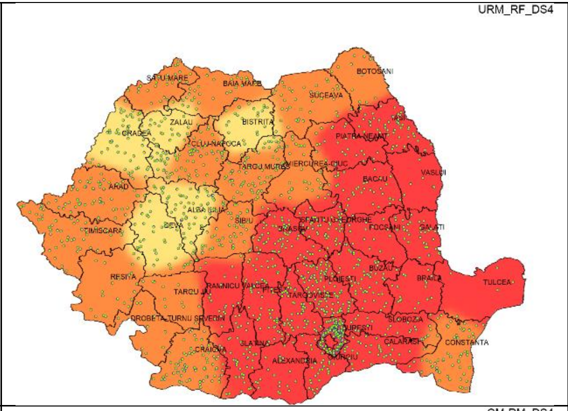
Most of the buildings included in the risk analysis are those of confined masonry (approximately 8700), in which approximately 700 000 students study. Out of these, only 18% (buildings and students) are exposed to a high seismic risk, with an annual probability of failure of approximately 0.001.

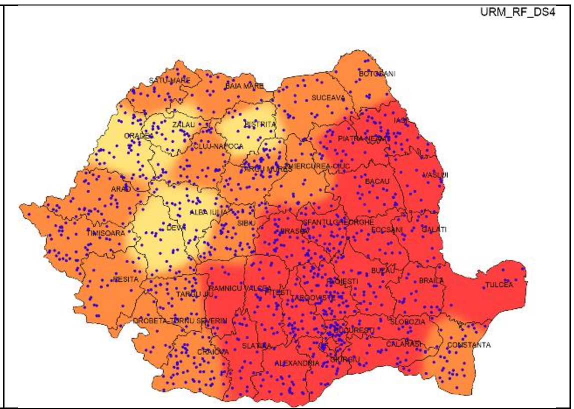
The situation of the most vulnerable structures, namely unreinforced masonry with flexible floors, affects to a greater extent the building stock in the education sector. The analysis includes almost 3500 such buildings, of which 70% have an exceedance probability for DS4 equal to 0.001 and another 7% have an exceedance probability of 0.01. Nearly 190 000 students studying in schools with unreinforced masonry structures are exposed to this level of risk associated with a potential partial collapse of the structure.

Tab. 8 Seismic risk and exposure maps for masonry structures in the pre-university education sector

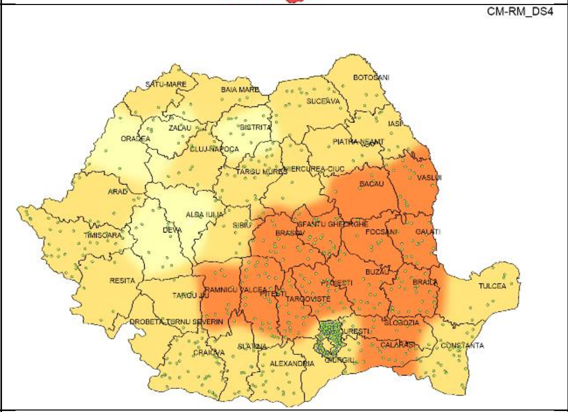
Structural typology	People exposed to seismic risk	Buildings exposed to seismic risk
URM_FF		

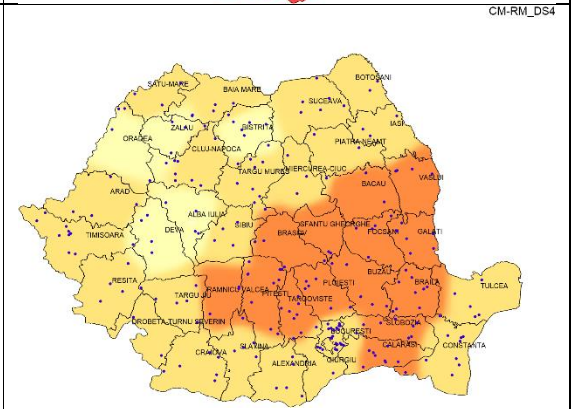
URM_RF



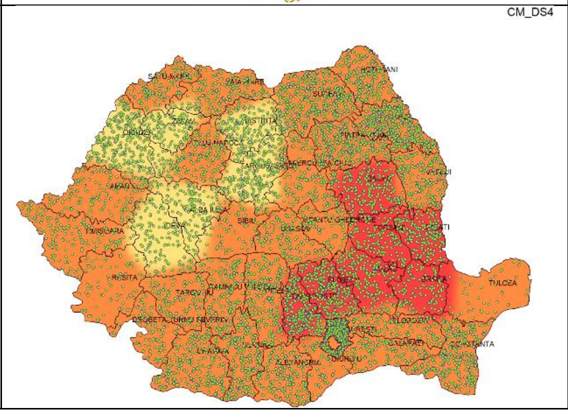


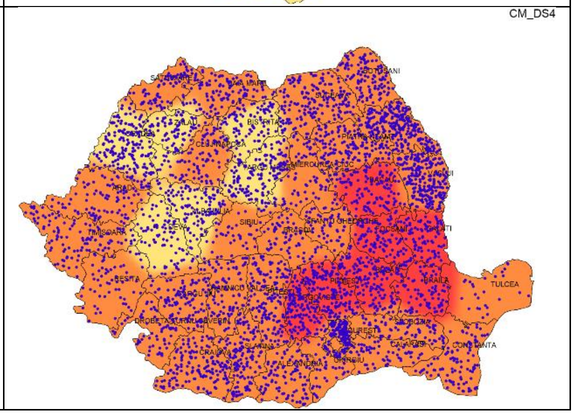
CM-RM





CM





5.5. Cost-benefit analysis

5.5.1. Prioritization methodology of retrofitting investments for the buildings belonging to the education sector

The input data for the buildings were either taken from the SIIIR database (building and school identification code, built-up area, year of construction, location), or processed according to the SIIIR data (structural typology, number exposed persons) and other databases containing information about previous seismic upgrading interventions implemented for pre-university education units.

In order to be able to establish a ranking according to the level of seismic risk, a prioritization matrix was proposed based on four key parameters: the hazard level established according to location, the level of vulnerability established according to the year of construction and the structural system of building, and the level of exposure expressed based on the number of students in the building. Four intervals have been set for each of the four parameters, so that the values included in these ranges can be associated with a score between 1 and 4, where 1 indicates a high priority and 4 a low priority.

Tab. 9 Prioritization matrix

Score	1	2	3	4	Weight
Seismic hazard: PGA value function of the location	≥0,30g	0,3g – **0,25**g	0,25g – **0.15**g	<0,15g	0,30
Year of construction	≤1920	1920-**1950**	1950-**1977**	>1977	0,20
Structural system	URM+FF	URM+RF	CM	RM	0,20
Importance (exposure)	4th quartile (75% - 100%)	3rd quartile (50% - 75%)	2nd quartile (25% - 50%)	1st quartile (0% - 25%)	0,30

Legend: URM+FF =unreinforced masonry with flexible floors, URM+RF = unreinforced masonry with rigid floors, CM = confined masonry and RM = reinforced masonry (RC jacketed walls)

The parameter regarding the importance of the school in the education system is evaluated according to the number of students enrolled in the 2019-2020 school year. Thus, the fourth quartile includes the schools in the country that have the most students, taking into account the national distribution, respectively over 1740 students per building. Estimates of the number of people exposed in each building are calculated based on the number of students enrolled in each school, distributed inside each of the buildings function of the area of the building.

The weights used for the four parameters are the following: a) seismic hazard - 0.3; b) year of construction - 0.2; c) structural typology - 0.2; d) the importance of the building - 0.3. Lower weights were assigned for the year of construction and for the type of structural system, considering that the two criteria are correlated in terms of construction techniques specific to each period. The final score for each building is the weighted average of the four parameters presented above.

5.5.2. Cost-benefit analysis for retrofitting interventions for the education section

Cost-benefit analysis was performed for the 15,084 masonry buildings (unreinforcedFload masonry with flexible floors: URM_FF or rigid: URM_RF, confined masonry: CM and reinforced masonry reinforced by lining: CM_RM) to compare economic benefits with potential losses in case of earthquakes with 63%, respectively 39% probability of overcoming in 50 years. The two earthquake scenarios are defined by seismic hazard parameters, in particular in the form of PGA values for each location (at the locality level), according to data from the BIGSEES project (Pavel et al. 2016). Considering the magnitude of the moment generated by a medium depth earthquake in Vrancea, the first scenario with a 50 year return period corresponds to a Mw = 7.2 ... 7.3, and the second scenario with a 100 year return period return corresponds to a Mw = 7.5 ... 7.6.

Using the proposed average values, the total consolidation investment of the 15 084 buildings included in the cost-benefit analysis resulted in € 4388 million. Assuming that during one year, retrofitting works can be carried out on 100 buildings, the implementation period of the project is 15 years. In order to estimate the current value of the buildings, an amount of 600 €/sqm was considered for the value of the building and assets inside it, whose potential destruction constitutes direct losses in the cost-benefit analysis.

The fragility functions used for cost-benefit analysis are those presented above in section 5.3.2.

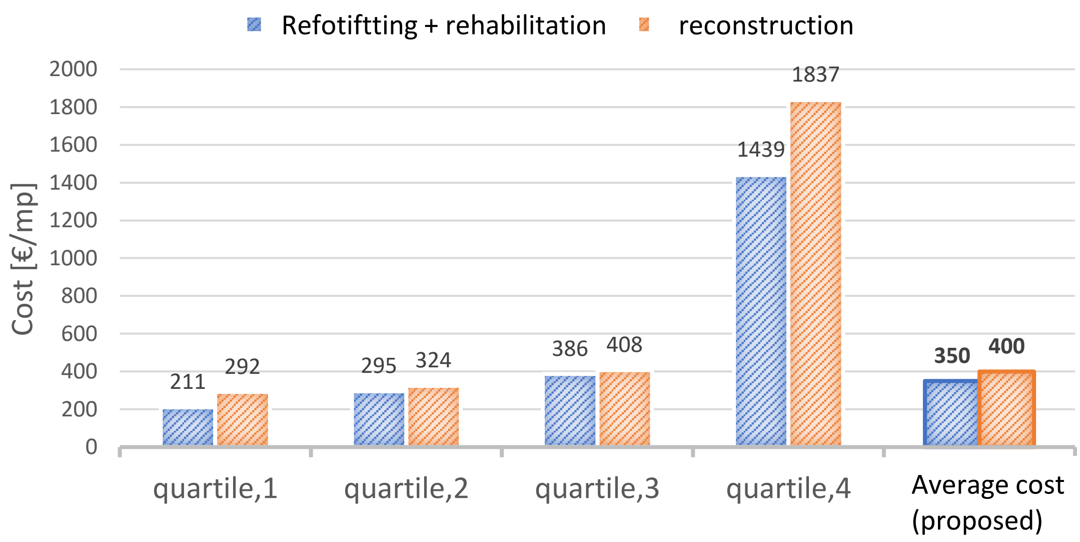


Fig. 27 Costs/sqm for investments in rehabilitating the school infrastructure

In order to estimate the value of the potential loss of life for the 1 285 275 people studying in the analyzed buildings, the concept of statistical value of life was used. The cost-benefit analysis for an investment project in retrofitting the building stock from the education sector involves estimating the benefits generated by saving potential casualties in case of an earthquake. There are no official estimates for the statistical value of life for Romanian citizens, but US values were used as a reference, based on the average values proposed by the US Environmental Protection Agency, namely $ 9.7 million in 2018. Calibration of the value used in the USA was based on the ratio between the Gross Domestic Product of the USA ($ 65118) and that of Romania ($ 12920), according to the data presented by the World Bank for 2019.

The losses expressed in potential loss of life and direct losses associated with the collapse of the buildings were calculated according to the methodology of HAZUS (Federal Emergency Management Agency 2015). The methodology involves a link between the level of damage to structural or non-structural elements and potential casualties. The types of injuries presented in HAZUS are numbered from 1 to 4, depending on the severity. For the cost-benefit analysis, only the proposed rates for severity 4 were considered, which means those who died or were fatally injured. Severity damage rates were used for each of the four damage states, considering that complete damage involves the collapse of the building, given that a partial collapse involves the replacement of the building rather than its retrofitting. Tab. 10 shows the percentage of life losses used for all types of masonry buildings included in the cost-benefit analysis.

Tab. 10 Casualty rates established function of the structural typology, according to HAZUS methodology (Federal Emergency Management Agency 2015)

Structural typology	Slight damage [%]	Moderate damage [%]	Extensive damage [%]	Complete damage [%]
URM_FF URM_RF CM	0	0,001	0,002	10
RM	0	0	0,001	10

Using the fragility functions proposed for each of the typologies and the level of seismic hazard for each location, the probabilities of exceeding a certain damage state were calculated. The probability that a building is in a certain state of damage is multiplied by the corresponding percentages in Tab. 10 to obtain the probability of having severity 4 casualties. The estimation of the number of casualties for each building is done by multiplying the number of people exposed in the building with the probability of having victims of severity 4.

Assessing the direct losses associated with the collapse involves estimating the total costs required to repair or replace the building affected by the earthquake. The average cost considered for the replacement of the building was 600 €/sqm, starting from the average cost of reconstruction in Fig. 27 equal to 400 €/sqm to which were added 200 €/sqm representing the value of the building assets (furniture, equipment, documents, etc.). The repair cost ratio, expressed as a percentage of the replacement value of the building, was established according to the HAZUS methodology, depending on the probability of being in one of the four stages of damage: slight, moderate, extensive and complete.

Estimating losses in financial terms involves transforming the probabilities of being in a certain damage state into equivalent values expressed in monetary units. Total direct losses include losses caused by damage to the structural elements of the building and those caused by damage to non-structural components (Federal Emergency Management Agency 2015), sensitive to acceleration (ceilings, technical equipment, piping, elevators, etc.) or displacement (partition walls, ornaments, etc.).

The benefits included in the cost-benefit analysis include the lives saved (the number of potential victims multiplied by the statistical value of life) and the direct losses avoided by retrofitting the buildings included in the analysis. The costs are calculated considering the built-up areas and the average retrofitting cost of 350 €/sqm.

To calculate the discount rate, a long-term growth rate of approximately 4.9 was considered. Once the social rate of time preference is calculated, both benefits and costs are reduced over the entire investment planning period, namely for 50 years.

The economic indicators resulting from the cost-benefit analysis are the cost-benefit ratio, the internal rate of return, the net present value and the payback period. The results of the cost-benefit analysis regarding the retrofitting of the building stock composed of masonry buildings in the education sector, as well as the input data used in the analysis, are presented in Tab. 11.

Tab. 11 Parameters used in the cost-benefit analysis and the resulted economic parameters

Description	Unit measure	Project implementation: retrofitting school infrastructure SCENARIO 1	Project implementation: retrofitting school infrastructure SCENARIO 2
Seismic hazard: PGA values	cm/s^2	Function of the location of each school unit	
Exceedance probability (seismic hazard)	% in 50 years	63% in 50 years	39% in 50 years
Investment value	Mil. Euro	4.388	
Total number of enrolled students	No. students	1.285.275	
Total number of buildings	No. buildings	12.084	
Total number of school units	No. school units	10.170	
Total building value	Mil. Euro	7.522	
Total built surface	Mil. m^2	12,5	
Potential casualties	No. persons	2.459	3.622
Value of statistical life	Euro[1]	700.018	
Value of avoided life losses	Mil. Euro	1.721	3.622
Value of avoided direct losses	Mil. Euro	388	544
Value of total avoided losses	Mil. Euro	2.109	2.536
Planning horizon	ani	50	
Discount rate	%	5	
Cost-benefit ratio	-	2,95	2,42
Net present value	Mil. Euro	6.356	4.612
Internal rate of return	%	13	10,8
Payback period	ani	18	22

The 1 285 275 students enrolled in kindergartens, schools and high schools in the public education system can benefit from the investment project in reducing the seismic risk associated with masonry buildings. From the total amount of people, 3611 could be potential victims in the event of an earthquake with a probability of exceeding 39% in 50 years (second scenario). Given that the total investment required to retrofit these buildings is 4388 mil. Euro, a value of approximately 1.2 mil. Euro is associated with saving a life, compared to the

[1] Curs valutar: 1 US$ = 0.82 Euro

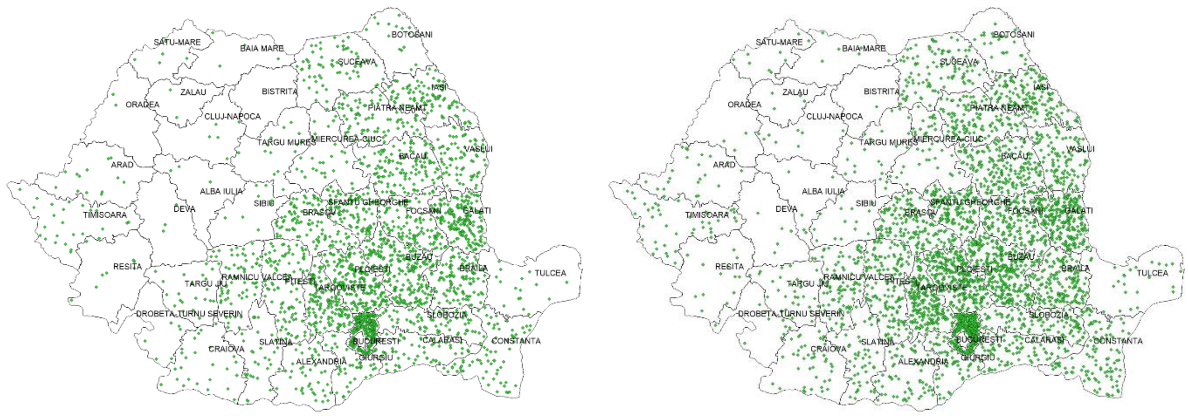
statistical value of life estimated at 0.7 mil. Euro. However, calculating the total investment in relation to the total number of beneficiaries, namely 1.28 million students, it shows an average cost of 3400 Euro/person.

The economic performance indicator called net present value is the difference between discounted benefits and costs. Consequently, the discount rate for which the net present value is zero at the end of the planning horizon is called the internal economic rate of return ((Direcția Generală Politică Regională 2008). At the end of the planning horizon, the cost-benefit ratio is calculated as the ratio between the present value of the benefits and the costs.

Fig. 28 illustrates the main results of the cost-benefit analysis for the two earthquake scenarios. The cost-benefit ratio for the 50-year planning horizon is in both scenarios larger than 1, for a payback period in between 18 and 22 years. Thus, the investment can be considered profitable from an economic point of view, without including in the analysis the costs of thermal rehabilitation or modernization, but also the social benefits associated with reducing the seismic risk for the pre-university education system.

Fig. 28 Expected losses for the two earthquake scenarios and economic indicators for the cost-benefit analysis

In order to have a clearer picture of the spatial distribution of the expected number of victims in the case of the two proposed earthquake scenarios, Fig. 29 presents in parallel the results obtained, each green dot representing a potential victim. The cumulative number of points on the map on the left (scenario 1) is equal to 2459, and for the map on the right (scenario 2) is 3622, concentrated in both cases in the southern and eastern part of the country. Most of the expected victims are in the Bucharest-Ilfov area, Buzau, Prahova, Galati, counties exposed to high seismic hazard. The ratio of the number of casualties obtained for an earthquake with an average return period of 50 years (scenario 1) to that of 100 years (scenario 2) is between 1.4 (counties with maximum PGA values) and 1.8 (counties with values PGA minimum).



Scenario 1 (63% in 50 years) Scenario 2 (39% in 50 years)

Fig. 29 Distribution of potential casualties in case of the two earthquake scenarios

5.5.3. Cost-benefit analysis results

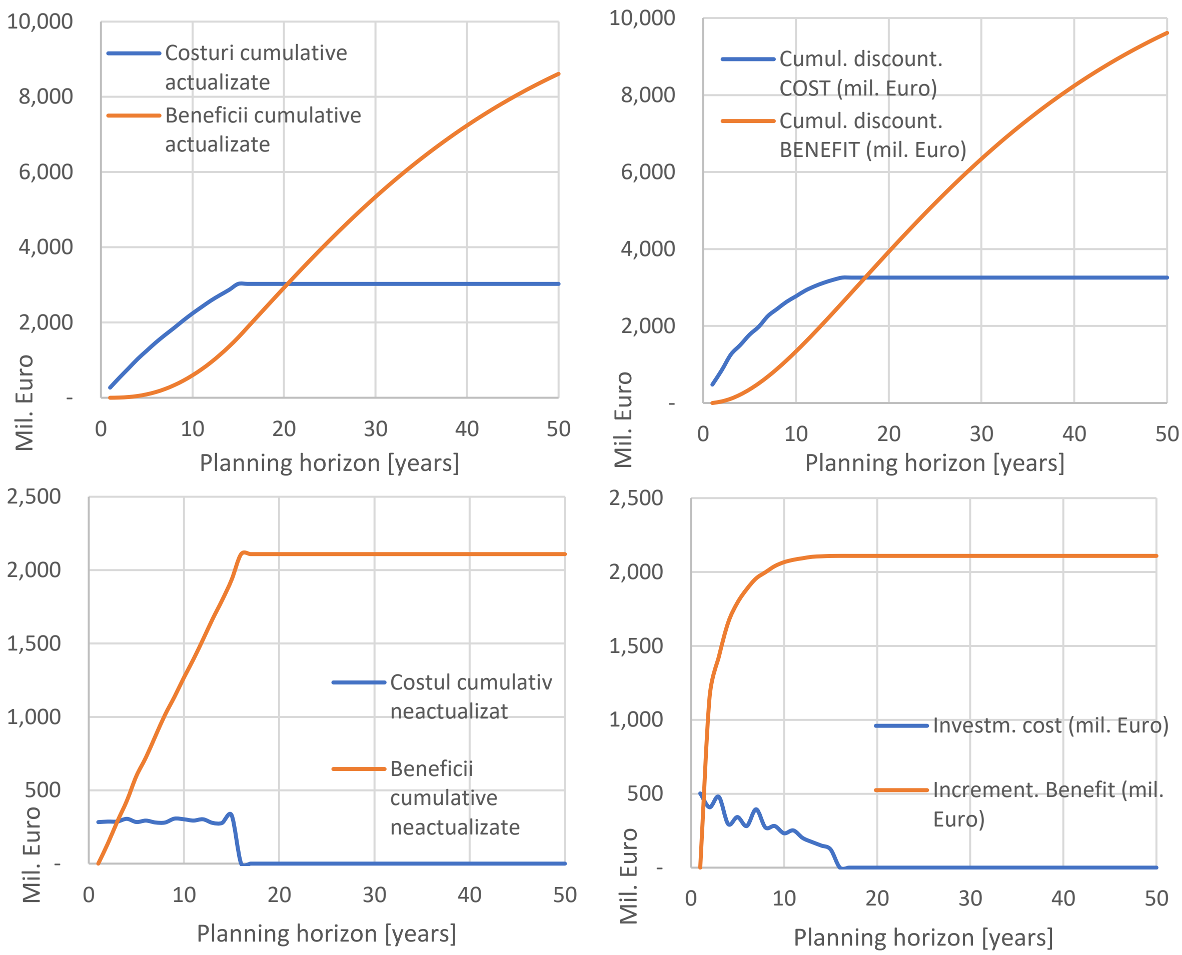
The results presented above are based on the allocation of funds according to the priorities set by the prioritization methodology. Depending on the ranking established based on the final scores obtained from the prioritization stage, masonry buildings are progressively included in the list of investments. Thus, buildings with a minimum final score occupy a leading position in the list of priorities set to reduce seismic risk in education. If the allocated funds cannot cover the need for the entire portfolio of educational institutions, a threshold can be set in the established hierarchy, so as to ensure that the investment is directed towards the objectives that maximize the resulting benefits.

In order to highlight the importance of directing funds to schools having the highest seismic risk, two different alternatives have been proposed in the cost-benefit analysis:

a. Random investment planning (no prioritization)
b. Prioritized investment planning (based on the ranking resulted following the methodology presented in section 5.5.1.)

The absolute total values for the number of potential casualties or losses associated with the damage are equal in both variants. The final economic parameters do not differ significantly either, as can be seen in Tab. 11 for the scenario corresponding to an earthquake with a 50-year average return period. The first set of graphs shows the evolution of retrofitting costs and cumulative benefits over the 50-year planning period. The transition from higher costs to benefits marks the end of the payback period, namely the year in which the cost-benefit ratio reaches 1. Thus, in the case of a prioritization process, the payback period is reduced from 21 to 18 years.

The second set of graphs also allows the comparison of the cumulative benefit values year by year, with a much faster growth recorded in the first ten years of implementation for the prioritized investment option. On the other hand, costs are also rising sharply at the beginning of the 15 years needed to retrofit the portfolio of masonry buildings.



a. Without prioritization

b. With prioritization

Fig. 30 Economic indicators of the cost-benefit analysis: without prioritization (left) and with prioritization (right)

The importance of a prioritization process is better reflected for such a retrofitting program when measured in terms avoided losses. Given the fact that the most significant losses are those caused by potential victims, the graph in Fig. 31 illustrates the evolution of the number of potential victims saved by retrofitting buildings in the first 20 years of planning. After the 15th year, the projects are completed and thus the plateau reached represents the maximum expected number of potential life losses saved for each of the two seismic scenarios.

It can be seen that the differences between the two implementation variants (with or without the application of the prioritization methodology) are significant from the first years. By directing funds to schools according to the score obtained in the prioritization stage, the number of lives saved is maximized from the beginning. After only two years of implementation in the case of the seismic scenario with a mean return period of 100 years, following a prioritization process could be saved about 2500 lives (70% of the total potential victims), as opposed to only 500 (14% of the total potential victims) when funds are randomly allocated.

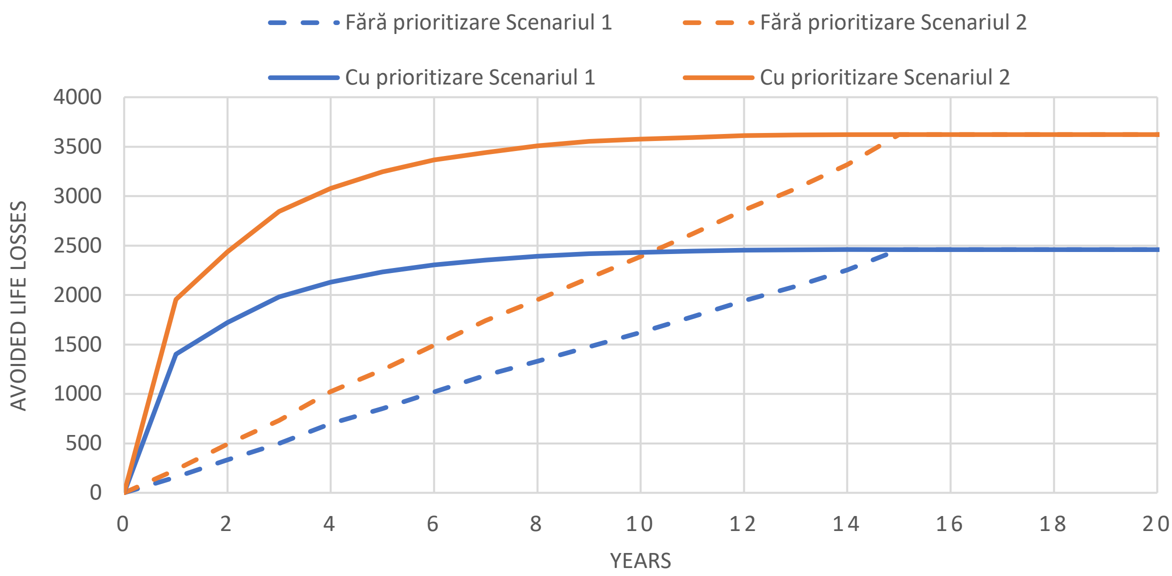


Fig. 31 Cumulative life losses avoided in case of retrofitting: with and without prioritizing investments

5.5.4. Evaluating retrofitting programs for retrofitting school infrastructure by means of cost-benefit analysis

Based on the results of the retrofitted school building case study in Section 4.2.3, the proposed fragility features for the reinforced masonry buildings were considered to be representative of the strengthening works carried out so far. Thus, another cost-benefit analysis was performed for the sample of 443 masonry buildings (unreinforced or confined) retrofitted within the School Infrastructure Rehabilitation Project (P.R.I.S.).

This analysis was performed only to compare the expected losses in the initial version of the masonry buildings with those resulting from retrofitting. Thus, for the 443 retrofitted buildings, the structural system was changed into RM (jacketed UMR). The transition from unreinforced/ confined masonry to retrofitted masonry implies significant differences in terms of expected losses, as can be seen in Fig. 32. For scenario 1, the expected losses from the strengthening of the 443 buildings are about 30 times lower than for the initial structures. For the second scenario, the implemented strengthening projects indicate 20 timer larger reductions in terms of expected losses. These results confirm the economic viability of the retrofitting investment dedicated to reducing the seismic risk for masonry buildings with educational function.

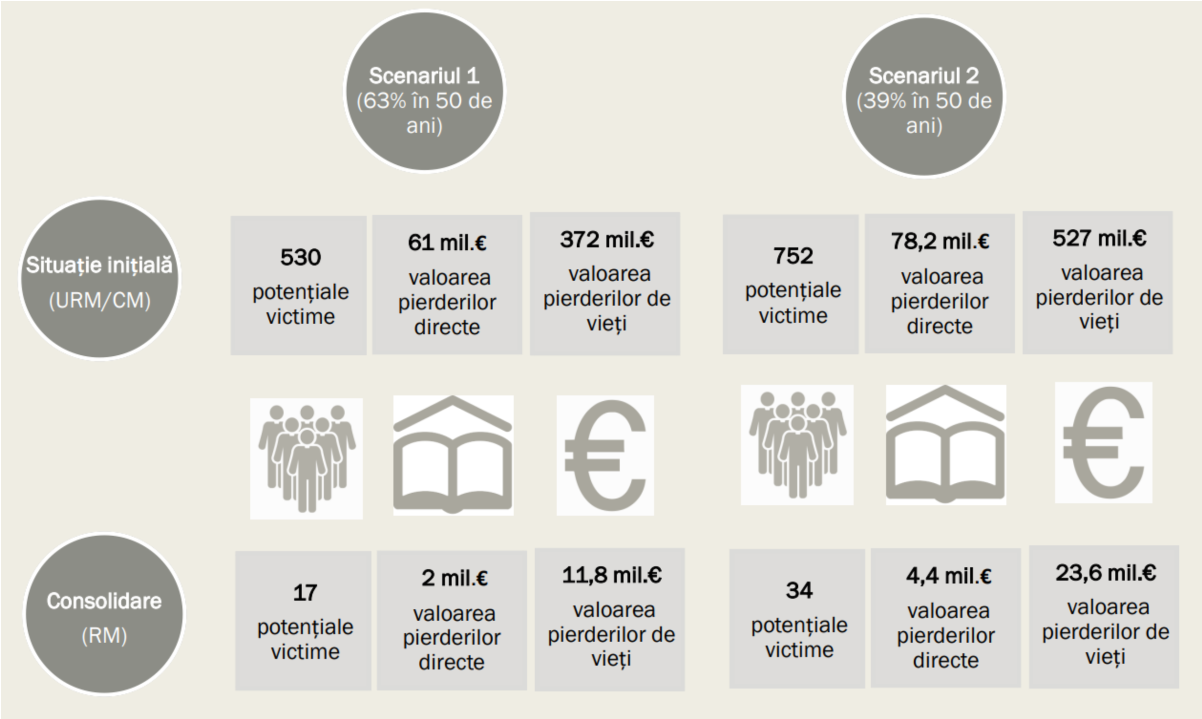


Fig. 32 Expected looses in case of the two scenarios: initial situation and post-retrofitting situation for both earthquake scenarios

6. CONCLUSIONS

6.1. Conclusions

Starting from the motivation to investigate the behavior of historic masonry buildings in Romania, this paper proposes a simplified method of assessing the structural vulnerability for such buildings, so that the results can be used later for seismic risk analysis. Starting from numerical models of representative buildings of the proposed taxonomy (unreinforced masonry with flexible or rigid floors, confined masonry, reinforced masonry retrofitted by RC jacketing and confined masonry retrofitted by RC jacketing), the evaluation of damage levels is made using the level II procedure proposed in within the RISK-EU project. This methodology allows to establish the parameters for the fragility functions expressed according to spectral displacements or accelerations. In order to validate the fragility functions proposed in the present work, comparisons were made with results from the literature for similar structural systems and also the post-earthquake damage recorded for the real structures was compared with the failures obtained from numerical models.

The results obtained from the non-linear static analyses for the initial model of the Geology Museum could be compared with the damages recorded in the seismic evaluation report, so that the proposed calculation model can be validated. The significant degradations observed at the level of the post-earthquake URM walls are confirmed by the high percentages of damage at the level of the walls and by the stages in which the macro-elements are at the end of the Pushover analyses. Diagonal cracks were present in the elements that present shear failures, while horizontal cracks caused by bending occurred in the case of slender piers. The curved walls connecting the two building parts were approximated in the calculation model with orthogonal walls, but their behavior in the calculation model places them in the first places in the list of the most damaged walls, given their slenderness and differences in stiffness between the two building parts of the Museum. The maximum relative displacements recorded for the numerical model are found at the level of the portico, an area that also underwent severe damages, partial collapses and remanent rotations as a result of the earthquake from 1977. Therefore, the structural behavior captured in the initial model of the Geology Museum is in accordance with the post-earthquake observations mentioned in the seismic evaluation report.

The ultimate displacements resulted from the global Pushover analyses proved to be smaller than the displacements of the walls generated by local failures. Thus, the comparison of relative displacements with different drift limit values at wall and floor level proved to be in line with the thresholds suggested in the literature. For the initial model, the maximum drift indicates exceeding the performance criterion for life protection, while the limits of relative displacement of the walls in the portico area exceed in some cases the criterion for collapse prevention.

Strengthening work carried out in 1980 were assessed by comparing the initial model in terms of maximum lateral forces, relative displacements of damaged walls and fragility functions. The increase in load-bearing capacity reaches up to 80% in the transverse direction where the retrofitting works are concentrated, but the ultimate displacement decreases in the case of the retrofitted model. For this reason, the seismic dissipation capacity is only increased by 25% for the retrofitted model. However, when comparing the relative displacements at wall level, it is observed a significant decrease in local displacements, where the walls that previously indicated exceeding the pre-collapse limit (1.01%) decrease to the stage of damage control (0.46%). The overall values of relative displacements used in the fragility analysis, both in terms of spectral displacements and in terms of spectral accelerations, do not lead to conclusions that support improvements in behavior at the local level. From the fragility functions expressed in terms of spectral displacements for the retrofitted model, probabilities of exceeding the damage states are higher than for the initial model. In order to highlight the contribution of the jacketing interventions, fragility functions were also expressed in spectral accelerations. Reductions in the failure probability were more evident, but only for the state of slight and moderate damage and partially for the state of extensive and complete damage, only for SA values below 0.3g.

Since the effectiveness of the retrofitted method could not be demonstrated through fragility analyses, it was decided to consider an alternative method: reinforcing the URM walls with FRP reinforcement. The FRP-retrofitted model involved replacing the RC jackets with FRP strips. For capacity increases similar to those recorded in the retrofitted model in 1982, the ultimate displacement increases are in this case over 40%, resulting in a seismic energy dissipation capacity increased by over 160% compared to the initial model. These results were also reflected in the fragility curves obtained for the FRP retrofitted model, where the probabilities of failure are significantly reduced compared to the initial, not retrofitted model.

For the second structural model, the school building in Bucharest was chosen to allow the analysis of the confined masonry structure, later strengthened by adding concrete columns and the jacketing masonry walls. The mixed structural system required a different approach for the calculation model, in a software able to capture the interaction between the reinforced concrete elements (columns and tie beams) and the URM masonry panels.

Comparing the different structural changes over time, there were increases of 50-60% in capacity, comparing the URM masonry model with the retrofitted one, for the longitudinal direction where the strengthening interventions were concentrated. However, the values of the ratio of the lateral force to the weight of the building larger than 1, obtained in the analysis of the retrofitted model indicate excessive strengthening. The intermediate model with confined masonry with only a few RC columns already shows significant improvements when compared to the URM structure, since reinforced concrete elements manage to ensure the redistribution of efforts and avoid local failures, as was the case with the Geology Museum building. Numerical analyses indicate for the URM a threshold of 0.06% of the lateral displacements for the initiation of failures, which corresponds to the damage control limit for such structures.

When assessing the structural capacity according to the global displacements, it is found that the unreinforced masonry model reaches values of lower final displacements even lower than the target displacement for the immediate occupation requirement, suggesting the need for strengthening interventions due to premature masonry panel failures. In order to highlight the influence of retrofitting measures done by RC jacketing, the most important walls were evaluated in terms of relative displacements before and after retrofitting. The increases recorded for the base shear force reached 70%, while the ultimate lateral displacements reached values four times higher.

The results of the fragility analyses performed for the school case study are proposed as representative for the typology of school buildings with masonry structure in Romania. Comparative analyses were performed between the fragility functions obtained and those used in the literature to characterize a similar structural system in Europe, in particular the "Placa" buildings in Lisbon, the equivalent of the confined masonry buildings in Romania.

The calculation models elaborated in the thesis start from two buildings representative of the building stock of historical masonry buildings in Romania: the building of a museum and a school, both located in Bucharest and listed as heritage buildings. The damage suffered by the earthquakes led to structural changes aimed at improving their behavior, interventions that marked the transition from an URM structural system to a structural system retrofitted masonry. The comparative analyses carried out between the behavior of the initial and the strengthened structures intend to highlight the benefits of the retrofitting interventions, but also the situations in which the applied strengthening measures are excessive. As an alternative, less invasive methods of reinforcement are also briefly presented, such as the replacement of reinforced concrete jackets with FRP strips. This thesis aims to highlight the importance of numerical analysis in the assessment of existing masonry structures, which can determine the most vulnerable areas where local failure could occur. With their help, local strengthening measures can be proposed, thus avoiding the application of excessive interventions.

The importance of fragility and seismic risk analyses at the typological level is highlighted by the application of the proposed methodology in order to estimate the expected losses for masonry buildings in the education sector. Based on the information on the exposure and vulnerability of school buildings collected through the Integrated Information System of Education in Romania (SIIIR), the expected losses for masonry buildings were estimated and criteria for prioritizing retrofitting interventions were proposed. Such instruments can contribute to the planning of the necessary investments for actions aiming to reduce the seismic risk associated with the building stock from Romania. Taking into account the large share of masonry buildings, the present work represents only an initial step for analyzing the behavior of such structures. Further research is needed in the field, as well as for other different structural typologies, for a better assessment of the Romanian building stock exposed to earthquakes.

6.2. Dissemination of results

Scientific papers published in index scientific journals: Web of Science WoS (ISI):

Pavel F., Scupin A., Văcăreanu R. (2021) Analysis of the seismic risk of low-code masonry and large panels structures in Romania, Iranian Journal of Science and Technology, Transactions of Civil Engineering, https://doi.org/10.1007/s40996-021-00736-2

Pavel, F., Văcăreanu, R., Arion, C., Aldea, A., Scupin, A. (2021). Seismic risk assessment of lifelines in Bucharest, International Journal of Disaster Risk Reduction, 66 (2021) 102629, https://doi.org/10.1016/j.ijdrr.2021.102629

Pavel, F., Văcăreanu, R., Scupin, A. (2020). Seismic fragility assessment for post-1977 high-rise reinforced concrete structures in Romania, Bulletin of Earthquake Engineering, https://doi.org/10.1007/s10518-020-01014-8

Scientific papers published in proceedings volumns from scientific conferences:

Scupin A, Văcăreanu R., Pavel F. (2020) Evaluation of invasive retrofitting interventions on an unreinforced masnory heritage building, 12th International Conference on Structural Analysis of Historical Constructions, pp. 3462-3473, ISBN 978-84-123222-0-0

Scupin A, Văcăreanu R., Pavel F. (2020) Vulnerability assessment of masonry walls by using capacity curves obtained from experimental testing, 19th National Technical-Scientific Conference on Modern Technologies for the 3rd Millenium, Oradea, Romania, 2020, pp. 247-254, ISBN 978-88-87729-68-9 (în curs de indexare WoS, conferința anterioară 2019 indexată WoS)

Scupin A., Văcăreanu R. (2021) Numerical simulation of the non-linear response of historic masonry, *10th International Conference on energy and environment (CIEM)*, Bucuresti, Romania, pp. 1-5, DOI: 10.1109/CIEM52821.2021.9614765

Scupin A, Văcăreanu R., Pavel F. (2021) Seismic performance assessment based on displacement capacity of unreinforced masonry structures, IOP Conference Series, Earth and Environmental Science, Bucuresti, Romania, 664, DOI:10.1088/1755-1315/664/1/012087

6.3. Personal contributions

Buildings with masonry structures, in particular the historical ones, built before the existence of seismic design provisions, represent an important part of the Romanian building stock. Over time, such buildings have benefited from strengthening works, local repairs or structural changes. Evaluation of existing buildings is a topic of interest for areas earthquake prone and concerned with reducing the impact of a potential seismic event in the future. Having this goal, the thesis includes the following personal contributions in the field of vulnerability assessment for masonry structures:

- A literature review was conducted on methods for assessing structural fragility and seismic risk for masonry buildings, as well as methods for assessing the effectiveness of retrofitting interventions for such buildings.
- A methodology for assessing the seismic risk associated with masonry buildings was presented, which can be used at the building level or at the typology level, for estimating potential losses. This was based on the examples analysed in the literature review, adapting the methods according to the availability of input data needed for such analyses.
- Numerical models were developed for two case study buildings, calibrated using data from seismic evaluation reports and 1977 post-earthquake damage assessments.
- For each case study, comparative analyses were presented between the initial condition of the building and the progressive changes in the structural systems, carried out in order to remedy the deficiencies and improve the structural behavior.
- The applicability of the global and local relative displacement limits proposed in the literature or in design regulations for existing buildings was studied, in order to establish correlations between them and the damages registered in the numerical models.
- The applicability of the relative global and local displacement limits for masonry buildings was verified and validated, establishing a link between them and the expected damage states.
- The results recorded in the numerical model for an alternative retrofitting solution based on the use of FRP fibers instead of RC jacketing of masonry walls for the Geology Museum building were analyzed.
- Typological fragility curves have been proposed for URM structures with rigid floors, URM structures with flexible floors, CM structures with RC tie beams and columns, CM structures strengthened by RC jacketing and URM structures strengthened by RC jacketing.
- For both buildings analyzed as a case study, the expected annual losses were calculated, expressed by annual failure probabilities associated with each damage state
- The expected annual losses for the five types of masonry buildings were calculated, namely the exceedance probabilities of certain damage states, using data from the most recent probabilistic seismic hazard analyses done for Romania.
- A database of pre-university public education units has been created with heritage buildings listed on the official List of Monuments.

- A sample of 29 284 buildings from 15 929 educational units in the pre-university education sector in Romania was analyzed. The processing of input data involved the correction of inconsistencies observed between the structural parameters of interest for the analysis (structural system, year of construction, built-up areas, height regime).
- For the entire sample analyzed in the education sector, exposure maps were made at county level, with information on the number of students enrolled in 2018-2019 and the number of buildings with educational units.
- A seismic risk analysis was performed for a reduced sample of the education sector, consisting only of masonry buildings. It contained 15084 buildings belonging to 10170 educational units, for which exposure maps were overlapped on seismic risk maps, for annual probabilities of exceeding the stages of damage DS3 (extended damage) and DS4 (complete damage).
- In order to optimize investments in retrofitting works, a set of parameters and a matrix of prioritization for buildings in the education sector was proposed.
- Average retrofitting costs have been proposed, estimated based on previous investments in the rehabilitation and retrofitting of buildings in the education sector.
- A cost-benefit analysis was developed for the sample of 15 084 masonry buildings, in order to highlight the losses avoided by investments in retrofitting the building stock from the education sector, using the HAZUS methodology for loss estimations.
- Based on the results of the loss analysis for two earthquake scenarios, maps were made illustrating the distribution of potential victims in the education sector.
- In order to highlight the importance of the prioritization stage, the results of two scenarios for cost-benefit analysis were analyzed in parallel: random investment planning and investment planning according to the prioritization process.
- A cost-benefit analysis has been developed for masonry buildings in the education sector that have already benefited from strengthening interventions to highlight the economic viability of such projects, the expected benefits and losses avoided through such seismic risk reduction actions.

In conclusion, the research results and personal contributions of this thesis are relevant for analyzing the behavior of historic masonry buildings. The proposed methods can be applied to seismic risk analyses on a national scale and the education sector database developed for this purpose can be an initial step in the preparation of public policies aiming to reduce seismic risk for masonry buildings.

6.4. Perspectives for future research activities

The steps initiated in this thesis can be complemented by research in the field of evaluation of existing buildings in order to reduce the seismic risk associated with them, as follows:

- Research on the behaviour of historic masonry buildings in other sectors, such as: health, residential, administrative, etc.
- Building databases for the entire building stock with the key parameters necessary for seismic risk analysis and proposals for prioritization criteria, similar to the ones presented for the education sector.
- Creation and analysis of several numerical models in order to define the behaviour of buildings at typology level, in particular for masonry buildings.
- Calibration of numerical models and material characteristics through laboratory tests.

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