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Assessing seismic risk in typical Italian school buildings: From in-situ survey to loss estimation

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ABSTRACT

The seismic assessment of existing school buildings is of paramount importance to protect the lives of occupants and manage post-earthquake response and recovery. The collapse of a primary school during the 2002 Molise earthquake in Southern Italy increased the awareness of the need to better understand the potential vulnerability of the existing Italian school building stock. To this end, a recent research project entitled "Progetto Scuole", whose main objective was to assess the seismic risk of a number of representative school buildings, was carried out at the Eucentre Foundation, in collaboration with the University School for Advanced Studies IUSS, in Pavia, Italy. The results of this project and of the subsequent research conducted by the authors are discussed in this paper, starting with the compilation of a school building database, focusing on the structural typology, the geometrical configuration and the time of construction. Three schools, representative of the Italian school building stock, were then analysed in detail through advanced numerical models developed using information collected during in-situ inspections and calibrated with the results of ambient vibration measurements. Two site locations were chosen to perform probabilistic seismic hazard analysis and select hazard-consistent ground motion record sets adopting the seismicity model used for the calculation of the Italian national seismic hazard map currently in place. Expected annual loss was used as a performance parameter to quantify the seismic vulnerability of the school buildings. Furthermore, the results of the detailed loss estimation were compared with the outcome of the seismic risk classification guidelines, recently introduced in Italy, applied to these same buildings. Finally, this paper presents a preliminary extension of the results to estimate the seismic risk of school buildings of the same typologies if located throughout the Italian territory.

1. Introduction

Italian seismic provisions were updated and refined several times in the last few decades, as the understanding of the effects of earthquakes on structures and the knowledge of regional seismic hazard improved. In Italy, the first attempt to provide seismic design provisions was made in 1909 after the Messina Earthquake [1] but the modern seismic design code era began in 1974 when the building code was updated to reflect new seismic design concepts [2]. Until 2003, two-thirds of the Italian territory were not considered by any seismic zonation hence buildings that were designed for gravity loads only are nowadays classified as being in seismic areas. It was only in 2008 that the first probabilistic seismic hazard assessment (PSHA) was introduced in the Italian building code, providing a detailed map of ground motion shaking with respect to different return periods (i.e. hazard map).

The census data [3] provided by the Italian National Institute of Statistics (ISTAT) highlighted that the highest percentage of Italian buildings were built before the introduction of the seismic provisions or in the transition period (Fig. 1). This leads to potentially high seismic vulnerability of the Italian building stock, a fact that has unfortunately been often confirmed following major earthquakes in past decades [4–7].

Analysing the reported damage in reinforced concrete (RC) buildings during past seismic events [8], one of the most representative building typologies in Italy, a common feature is the lack of adequate seismic detailing and design philosophies now included in modern design standards around the world. The columns were generally designed only for gravity loads with low shear and flexural capacity. The lack of shear reinforcement in the joints, combined with the increment of forces due to the interaction between the RC frame and masonry infills, often

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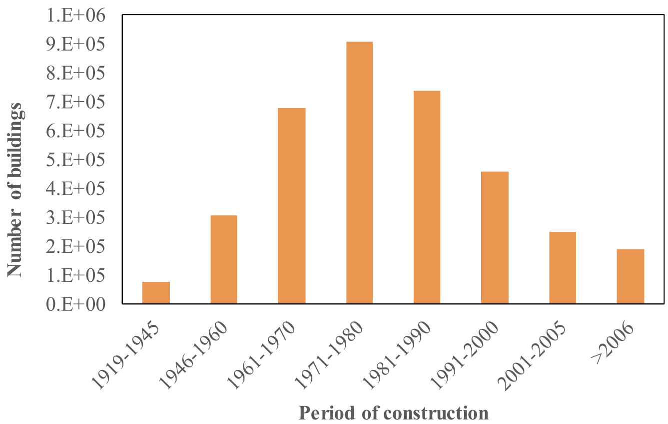


Fig. 1. Period of construction of Italian buildings [3].

caused the shear failure of beam-to-column joints [4]. As a consequence of the lack of shear reinforcement, as well as of the presence of storeys without masonry infill, the development of soft storey mechanisms was often reported [4].

When referring to unreinforced masonry buildings (URM), another very common typology, their partial or complete collapse was observed during many past earthquakes in Italy [5,7,9]. During the 2016 Central Italy earthquake, URM buildings suffered significant damage due to some typical construction features, such as rubble masonry walls with poor mud mortar or two to three unconnected layers across the thickness that make the use of steel ties inefficient. Also, the poor connections between orthogonal walls, the presence of vaults without confining ties and the presence of floors providing a poor diaphragm restraining effect have all contributed to the poor seismic response of historical URM buildings [5].

In terms of seismic response of precast (PC) buildings, the 2012 Emilia-Romagna earthquake in Italy demonstrated the lack of shear and ductility capacity in simply supported beam-to-joist and beam-to-column connections [6]. These connections generally consisted of vertical steel dowels or solely relying on shear friction, with or without neoprene pads. The beam-to-column connections were designed for gravity loads only and their premature failures were observed due to the high relative movement involved in the loss of beam supports and consequently in the partial collapse of the buildings, primarily at the roof levels [6]. Furthermore, the high structural deformability resulted in either the in-plane or out-of-plane failures of precast cladding panels for which the presence of fragile connections was often observed [6].

Finally, it is worth noting that a high percentage of earthquake-induced losses are related to the damage of non-structural elements [7]. The poor seismic performance of non-structural elements is generally the consequence of the omission of proper seismic design and detailing, and expertise on how to effectively perform it. For example, significant damage to ceiling systems, partitions, shelves and ornaments in heritage URM buildings was reported by Perrone et al. [7] following the 2016 Central Italy earthquake.

Based on these considerations, the high seismic vulnerability of the Italian building stock becomes evident, an issue that is particularly important for high-priority buildings, such as schools and hospitals. Following the 2002 Molise earthquake in the south of Italy, which resulted in the collapse of a school building with 27 fatalities, the seismic vulnerability assessment of school buildings in particular has received more attention. The importance of the seismic vulnerability of school buildings and the need of a seismic rehabilitation program based on a risk-based management framework was pointed out by Grant et al. [10]. The Ministry of Education, University and Research (MIUR) carried out a systematic survey of the Italian school buildings and developed a database called "Anagrafe Edilizia Scolastica" [11]. The database collects information regarding period of construction, geometrical configuration, structural typology and other information not only related to the structural response. The database was made accessible to the Italian community to increase awareness of school buildings safety, which is not just related to the seismic performance of the school building stock but also to their maintenance. Damage due to the lack of maintenance, such as partial collapse of slabs or damage to non-structural elements (i.e. doors and windows), sometimes results in fatalities and injuries [12]. In this context, a research project funded by the "Centro di Geomorfologia per l'Area del Mediterraneo" (CGIAM) entitled "Progetto Scuole" has been recently carried out. Its main objective was to identify the main features of the existing school building stock and to analyse the seismic performance of representative school buildings, using state-of-the-art methods and risk metrics, in order to provide useful indications for their seismic prioritisation and retrofit. The identified schools were instrumented with a monitoring system to measure their modal properties from ambient vibrations as well as their dynamic response during earthquakes. The loss estimation assessment of the school buildings according to the procedure proposed by FEMA P58 [13] was also performed. This methodology, which follows the developments by the Pacific Earthquake Engineering Research Center (PEER) on performance-based earthquake engineering (PBEE) [14], is one of the most comprehensive methods available in the literature for seismic loss estimation studies. With this method, the expected annual losses (EAL) can be computed by integrating the expected direct economic losses expressed as a function of the intensity measure (IM) over the site hazard curve obtained from the probabilistic seismic hazard analysis (PSHA). At the same time, guidelines for simplified seismic risk classification were approved and introduced by the Italian government in 2017 [15,16]. These guidelines were developed in order to increase the knowledge on the seismic risk of existing buildings, as well as to incentivize seismic mitigation programmes via tax deduction plans for building owners should they choose to invest and upgrade the resilience of their buildings. The document provides a simplified procedure to estimate the expected annual losses and to define a seismic risk class [15,16], in a similar fashion to the energetic efficiency scheme adopted in Europe. Based on the identified seismic risk class and the improvement that can be achieved through seismic retrofit, specific tax deduction ratios are foreseen.

In this article, the complete seismic loss assessment of three school buildings belonging to the most common typologies of the Italian existing school building stock is presented. These were selected from a database of over 49,000 buildings. A detailed inventory of structural and non-structural elements was developed during in-situ surveys. The results of the ambient vibrations measurements were used to identify the modal properties of the buildings and to help in the development of detailed numerical models. The results of the seismic loss assessment performed according to the FEMA P58 procedure were compared with those provided by the recent risk classification guidelines in Italy. Finally, the results are extended to estimate the seismic risk of school buildings of the same typologies located throughout the Italian territory. These extended results provide further insight into the more vulnerable regions and building typologies in terms of both economic losses and collapse safety.

2. School buildings in Italy: selection of case studies

A database of the Italian school building stock was developed by the Eucentre Foundation (www.eucentre.it) within the aforementioned "Progetto Scuole" as well as from previous research projects made in collaboration with the Italian Department of Civil Protection [17]. The database relies on data available for about 49,000 buildings [11]. Data related to structural behaviour as well as other features concerning the school organization (e.g. presence of public transport and parking, etc.) was collected through a specifically developed survey form. The following information of interest to this study was available from the census form: 1) location coordinates of each school building; 2) general information about the school location; 3) period of construction; 4) number of storeys; 5) structural typology and main characteristics of vertical and horizontal structural elements; 6) maintenance status of the buildings; 7) services and installations in the buildings; and 8) other general observations. The availability of this data provides a clear idea on the main characteristics of the school building stock in Italy. However, the survey forms were partially filled in many cases and some assumptions were made to have a complete picture of the building stock. For instance, if the structural typology was not available, it was assumed to be related to the period of construction. If the building was built before 1940 or if the number of storeys was less than five, it was assumed to be constructed in URM. All buildings with more than five storeys were supposed to be of RC. Fig. 2a shows the Italian map with the location of the 49,503 georeferenced school buildings, while Fig. 2b reports the percentages of structural typologies.

Approximately 80% of the existing school buildings in Italy are made of URM and RC, whereas the remaining 20% are characterised by other typologies, such as precast, steel or mixed typology structures. The number of storeys is one of the main parameters used for the building's classification. This is because rapid visual screening procedures, as well as regional scale approaches, provide safety indices and fragility curves as a function of the number of storeys and generally classify buildings in three categories: low-rise, medium-rise and high-rise [18,19]. Fig. 3 reports the classification of the school building stock based on the number of storeys for URM (Fig. 3a) and RC (Fig. 3b) buildings. Two-storey typologies are the most common both for URM (47.5%) and RC (41.9%) school buildings. Approximately 20% of the URM school buildings are characterised by one (24.6%) or three (21.3%) storeys while for RC buildings, the percentage of one-storey buildings (32.2%) is higher than that of three-storey school buildings (17.4%). The tallest URM school buildings are five storey buildings (1.0%) while for RC typologies, the percentage of buildings with five or more storeys (13.8%) is definitely more significant.

The existing school building stock was also classified according to the year of construction (Fig. 4a); this is a fundamental information to understand if the buildings were seismically designed or not. The highest percentage of school buildings were built after 1976 (31.5%); in this period, many changes were made to the Italian building code with a completely new design approach starting from 2003. A significant percentage of buildings was built between 1960 and 1975 (28.0%) - the period in which a high percentage of the Italian territory was not classified as earthquake-prone. In order to provide information on the seismic hazard at the sites of the school buildings, a preliminary classification provided by the Italian Law Decree "Ordinanza n.3274 del Presidente del Consiglio dei Ministri 20 marzo 2003 (OPCM 2003)" [20] was made. The OPCM 2003 is one of the first documents that provide a complete seismic zonation of Italy. In particular, four seismic areas were defined based on the expected peak ground acceleration (PGA): Area 1: PGA $>$ 0.25 g; Area 2: 0.25 g $<$ PGA $<$0.15 g; Area 3: 0.15 g $<$ PGA $<$ 0.05 g; and Area 4: PGA $<$ 0.05 g. Fig. 4b reports the percentages of URM and RC buildings distributed across the seismic areas defined by the OPCM 2003. The highest percentage of RC buildings is observed in the seismic area characterised by the highest seismicity (Area 1), while in Area 4 the percentage of URM buildings is higher than that of RC buildings.

Based on the compiled database, the main features of the existing school building stock could be identified and some representative school buildings could be selected. Figs. 2b and 3 show that the majority of school buildings in Italy have three or less storeys and commonly consist of RC frames with masonry infills or URM. Even though the PC buildings are not directly represented in the statistical analysis, it was observed that these buildings represent a small portion of the more recent school building stock. Furthermore, approximately 67% of the existing school buildings in Italy have been constructed before 1975, which precedes the introduction of modern seismic design provisions in the country. According to the above considerations, three school buildings were selected in this study to perform a systematic loss estimation assessment. Table 1 lists the structural typology, the number of storeys and the period of construction of each selected building.

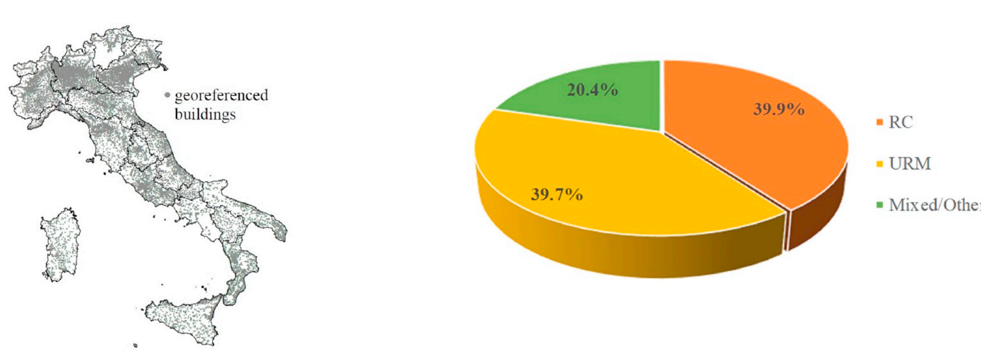
The selected school buildings analysed in this study were built prior to the introduction of modern seismic design provisions in Italy. Therefore, it is reasonable to consider the existence of school buildings of similar constructions at different locations throughout Italy. It was thus decided that each school building analysed may exist at any given location throughout Italy to provide further insight into the relative behaviour of such typologies throughout the country, for different levels of seismicity.

3. IN-SITU survey and modal identification

In-situ surveys were conducted to gather the information required for the seismic loss assessment and to define the seismic risk class according to the Italian guidelines [16]. Furthermore, each school building was equipped with a structural monitoring system to evaluate its modal properties and dynamic responses in case of a seismic event. In the following sections, a brief overview of the main information collected during the surveys and of the results of the ambient vibration analysis is provided.

3.1. Building survey

The main objective of the in-situ surveys was to create a detailed inventory of all structural and non-structural elements for each school building. The agreement between the architectural and structural blueprints provided by the local administrations and the structural layout observed during the surveys was verified in each case. All the information on the material certificates and the results of non-destructive tests was also collected.



a) Georeferenced school buildings

b) Classification by structural typology

Fig. 2. General classification existing school building stock in Italy.

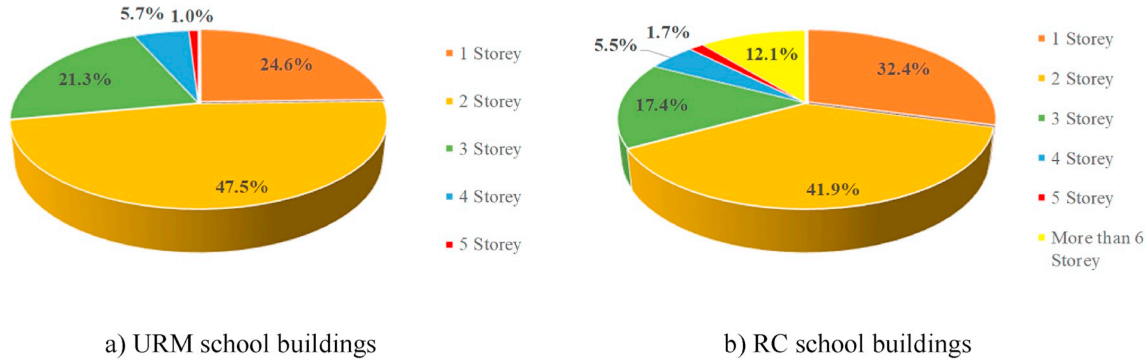


Fig. 3. School buildings on Italy classified by number of storeys.

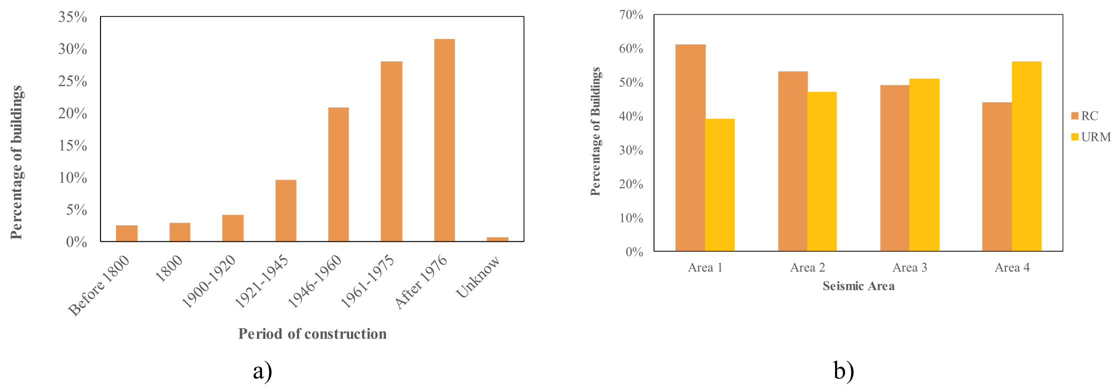


Fig. 4. School buildings in Italy classified according to: (a) Period of construction and (b) OPCM 2003.

Table 1
General information for case study school buildings.

Typology	Label	No. of Storeys	Construction Period
Reinforced concrete frame with masonry infill	RC	3	1960s
Unreinforced masonry	URM	2	1900s
Precast RC frame	PC	2	1980s

The RC school building, shown in Fig. 5, is a three-storey RC frame building constructed in the 1960s. As such, the design requirements at the time of construction did not contain any seismic design specifications and the structure was designed for gravity loading only. Survey data provided the structural members layouts and sample reinforcement contents. The typical features of buildings from this period were observed, such as the lack of proper stirrups in the beams, columns and joints and the use of smooth bars. The floor system was observed to be a *laterizio* floor system [21] of adequate thickness to consider the floor diaphragm as rigid in numerical models. The RC members typically consist of 30 × 30cm columns and 30 × 50cm beams. Infill panels were identified as double-leaf 12 cm hollow clay brick with 5 cm wide internal cavity. The results of the in-situ tests reported that the mean compressive strength of the concrete varies between 8.7 and 14.4 MPa, while the yield strength of the reinforcement bars was reported to be equal to 381 MPa.

The URM school building, illustrated in Fig. 6, was built in the early 1900s and is characterised by two storeys with interstorey height of

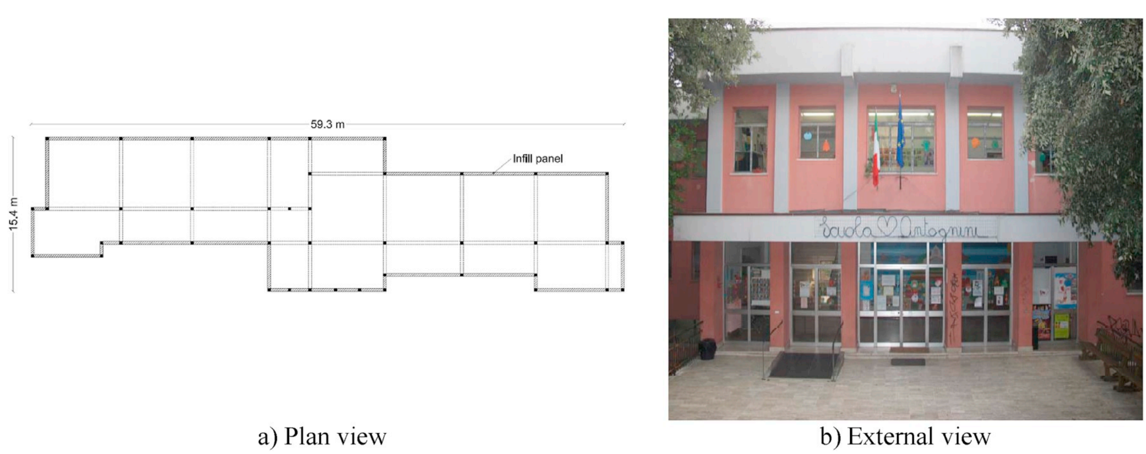


Fig. 5. RC school building selected for detailed analyses.

3.95 m and 4.35 m, respectively. The in-plan dimensions are approximately 47 × 12 m and the masonry walls are made of limestone with thickness varying between 0.40 and 0.85 m. As reported in Fig. 6a, the structural layout comprises four thick perimeter masonry walls. The structural configuration is not symmetric, as on the left side of the building more masonry walls are present, while in the central and right side of the building, the reinforced concrete floor is supported by reinforced concrete beams connected to masonry columns. These features make the building highly vulnerable seismic actions due to the lack of seismic structural details, particularly regarding the connections between structural elements.

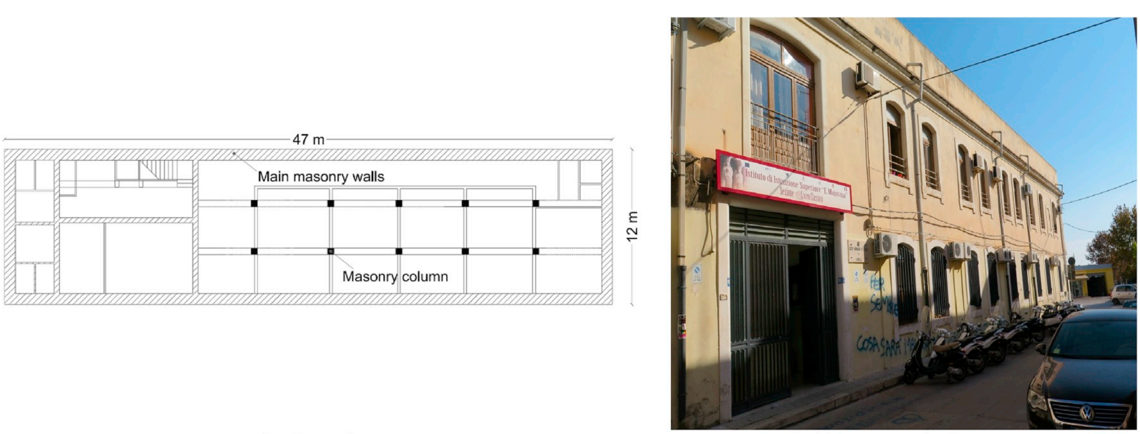
The third building selected, depicted in Fig. 7, is a two-storey PC frame building, which was constructed in 1987. The structural system comprises of a precast hollowcore floor system supported on precast frames, which are in-turn supported on shallow foundations. The frame system consists of precast beams in only one direction between precast columns, which are supported in pocket-type foundations. Although not considered part of the primary structural system, it should be noted that there are a number of masonry infill partitions and the precast concrete panels make up the exterior cladding. The precast concrete beams are seated on column corbels without any horizontal restraint (other than friction) and there is a gap between the column face and beams ends of typically 20 mm. This means that there is no moment transfer between the beams and columns until the gap closes as the frames displace laterally. The maximum moment that can be developed at the joint is then limited by sliding at the interface of the beam and column corbel.

The inventory of non-structural elements, for all three school buildings, was performed according to two levels of detail. A first general survey was carried out in order to establish the typologies and quantities of non-structural elements installed in each school. A more detailed survey was then performed and was aimed at the evaluation of a safety index calculated by filling specific assessment forms. The forms were developed following the suggestions contained in FEMA E−74 [22], which is a US guideline specifically aimed at the reduction of the seismic risk of non-structural elements. The forms were developed only for some non-structural element typologies and are divided in four main sections: 1) identification, description and quantification of the non-structural element; 2) evaluation of a safety index based on the configuration of the non-structural element; 3) proposal of mitigation details; and 4) fragility data to perform the loss estimation study. Based on all the collected data, a complete inventory of structural and non-structural elements was developed for the three schools, as reported in Table 2.

3.2. Ambient vibration measurements

The results of experimental and in-situ measurements can be very important to improve the prediction capability of numerical models . Among others, the uncertainty related to the material properties as well as to the construction details leads to high variability in the results. For this reason, many numerical models proposed in the literature to simulate the response of particular buildings' features, such as the beam-to-column shear behaviour or the influence of the masonry infills, are developed referring to the results of previous experimental campaigns in which the response of buildings designed with similar characteristics as that of the analysed buildings are investigated [23–25]. The ambient vibration measurements are a useful tool to investigate the modal properties of a building and to better calibrate the global behaviour of the structure in the numerical model, in particular when detailed information on the cracking of concrete, the mechanical properties of the masonry, or the quality of the connections are not available. Based on these considerations, the three selected school buildings were instrumented with an advanced monitoring system aimed at providing continuous dynamic system identification through ambient vibration acquisitions. The monitoring system was also designed in order to record possible seismic events. A detailed description of the dynamic identification results for the case study buildings, and of their implication in the seismic response, is provided by O'Reilly et al. [26,27]. Fig. 8 shows an illustrative example of single transfer functions from ambient vibrations recorded in the RC and URM school buildings. The natural periods of the buildings are easily identified by the peaks of the transfer functions.

Based on the results of the ambient vibration measurements, a parametric analysis was carried out to identify the parameters that affect the most the modal properties of the buildings [27]. A description of the numerical model developed for each building is provided in Section 4.1. It was observed, for all the buildings, that the assumption of cracked section stiffness for flexural behaviour model significantly affects the results. This assumption is widely adopted in design and assessment but can lead to discrepancy when comparing the experimentally assessed modal properties for undamaged buildings with the results of numerical analyses. For RC buildings, a second parameter that significantly affect the modal properties is the presence of masonry infills, both in terms of mechanical properties and cracked/uncracked sections. In the case of URM building, the Young's modulus of the masonry was observed to be a salient parameter in the evaluation of its fundamental frequency. Finally, for the PC school building, the cladding panels and their connections were also founded to be very important. Table 3 lists the natural frequencies obtained from the ambient vibration measurements and those predicted by the revised numerical models addressing all the issues described above. The matching is quite good, although some differences are still observed. One of the reasons for these discrepancies can be attributed to neglecting the internal partitions and other non-structural element stiffness contributions in the numerical models, which would slightly increase the buildings' stiffness and in turn their



a) Plan view b) External view

Fig. 6. URM school building selected for detailed analyses.

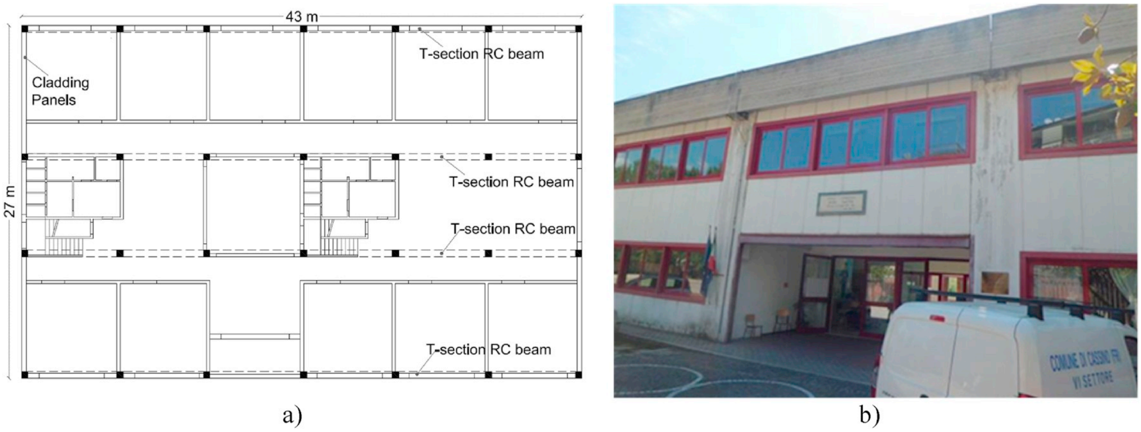


Fig. 7. PC school building selected for detailed analyses.

Table 2
Building inventory.

Element	Demand Parameter	Unit	Quantities						
			RC building			URM Building		PC Building	
			Ground	1st Floor	2nd Floor	Ground	1st Floor	Ground	1st Floor
Structural Elements									
Exterior Be am−Column Joints	Drift [%]	each	20	20	20	–	–	–	–
Interior Beam−Column Joints	Drift [%]	each	23	23	22	–	–	–	–
Non−Ductile Columns	Drift [%]	each	44	44	44	2	2	–	–
Exterior masonry infill	Drift [%]	m^2	454.4	454.4	447.3	–	–	–	–
Staircase	Drift [%]	each	1	1	1	1	1	2	2
Precast Columns	Chord Rotation [%]	each	–	–	–	–	–	28	28
Precast Panles	Panel Mom ent [kNm]	each	–	–	–	–	–	6	6
Unreinforced Masonry Piers	Chord Rotation [%]	m^2	–	–	–	212.4	225.2	–	–
Unreinforced Masonry Spandrels	Chord Rotation [%]	m^2	–	–	–	77	87.5	–	–
Non−Structural Elements									
Interior Gypsum Partitions	Drift [%]	m^2	317.8	291.9	268.1	176	191.4	786.9	537.9
Interior Masonry Infill	Drift [%]	m^2	198.9	198.9	195.7	–	–	–	–
Doors	Drift [%]	each	18	13	15	15	13	35	20
Windows	Drift [%]	each	23	50	53	23	26	26	23
Desks	Drift [%]	each	110	145	182	82	104	410	393
Chairs	Drift [%]	each	140	182	182	108	134	509	469
Ceiling System	PFA [g]	m^2	560	588	566	365	365	1651	1490
Fan coils	PFA [g]	each	28	30	30	8	10	50	35
Lighting	PFA [g]	each	66	48	48	52	56	176	142
Piping − Water Distribution	PFA [g]	m	452	452	452	232	232	763	496
Piping − Heating Distribution	PFA [g]	m	476	476	476	140	140	1330	967
Bookcases	PFV [m/s]	each	16	22	14	8	12	41	43
Mobile Blackboards	PFA [g]	each	3	3	4	11	12	29	12
Electronic Blackboards	PFA [g]	each	0	3	3	0	6	0	7
Computer and Printers	PFA [g]	each	6	20	0	3	28	3	46
Projectors	PFA [g]	each	0	3	3	0	6	0	8
Switchboards	PFA [gl	each	1	3	3	2	7	4	12

Note: PFA: Peak Floor Acceleration, PFV: Peak Floor Velocity.

natural frequencies. For the PC school building, despite the use of gross section properties and the incorporation of cladding panels, which resulted in a significant increase of the stiffness, the discrepancies are still relatively large. The lack of quality information from the data acquisition system is noted as a limiting factor also.

Based on the obtained results, the importance of the modal system identification is pointed out. In particular, discrepancies in the initial range of the structural response may have a significant impact on the computation of expected annual losses, which tend to be heavily affected by contributions from frequent, low-intensity earthquake events.

It is worth noting that the ambient vibration technique adopted in this study allows the improvement of the prediction capability of the numerical models in terms of mass and stiffness distribution, as well as to identify the parameters that mostly affect the stiffness of the buildings. However, it does not allow one to extrapolate on the nonlinear response range, which the analysed structures might undergo if subjected to strong earthquake. To this aim, advanced structural health monitoring techniques with mechanical-based nonlinear finite element models, which also include methodologies for accounting for unknown input excitation, would enable the reconstruction of the nonlinear response experienced by the structure during a damage-inducing event [28].

4. Seismic assessment

The data gathered during the surveys and ambient vibration measurements was used to develop a detailed numerical model for each

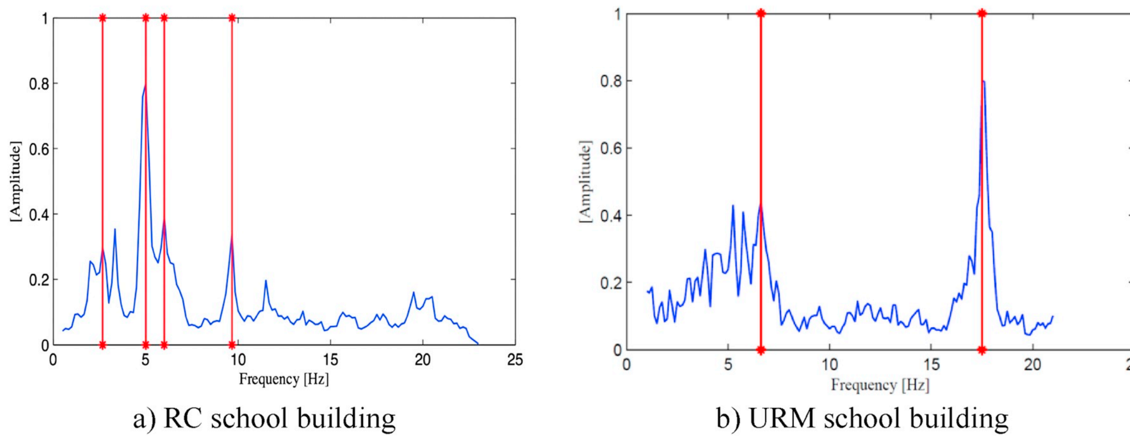


Fig. 8. Single transfer function from ambient vibrations recorded at school buildings.

Table 3
Measured vs Predicted natural frequencies.

School	Mode	Natural Frequencies [Hz]		Ratio
		Predicted	Measured	
RC	1	3.66	5.25	1.4
	2	5.28	6.38	1.2
	3	6.50	9.63	1.5
URM	1	5.13	5.21	1.0
	2	9.86	17.5	1.7
PC	1	2.48	7.5	3.0
	2	2.66	17.8	6.7

school building and to assess their seismic response both in terms of structural behaviour and seismic risk, quantified in terms of expected annual losses. In this section, a brief overview of the numerical modelling approach, the hazard characterisation and the structural responses is provided. More details regarding all of these aspects can be found in O'Reilly et al. [27,29].

4.1. Numerical modelling

For each school building, numerical models were developed using the information collected during the survey as well as following the measurements and details provided in the structural drawings, when available. The RC building was modelled using the structural analysis software Opensees [30] following the recommendations of O'Reilly and Sullivan [23] for existing RC buildings in Italy. The modelling approach accounted for the typical features of RC buildings designed for gravity loads, such as the presence of smooth bars and the lack of transverse reinforcements in columns and beam-to-column joints that could involve brittle shear failures. The influence of the masonry infills was also considered assuming the equivalent diagonal strut approach proposed by Crisafulli et al. [31] and the mean properties proposed for hollow clay masonry proposed by Hak et al. [32]. These properties were assumed for the double leaf infill panels described by Hak et al. [32] that were observed during in-situ inspection of the case study building. These material properties were used with the numerical modelling approach of Crisafulli et al. [31], where the maximum force capacity, F_{max}, of each equivalent diagonal strut can be determined based on the most critical force among the potential diagonal tension failure, shear sliding, corner crushing, or compressive failure, as described by Decanini et al. [33]. For simplicity, the influence of the openings in the infill panels was neglected in the numerical modelling. The floor slab system was assumed to be rigid, following the examination of the actual floor system in place. Fig. 9 illustrates the numerical model for the RC school building (more details are provided in O'Reilly et al. [29]).

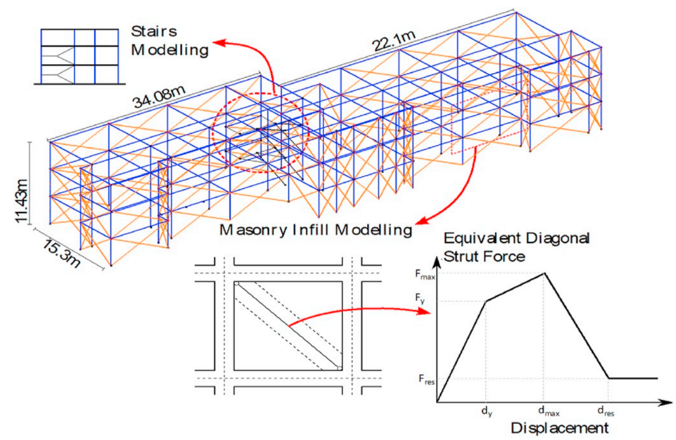


Fig. 9. Numerical model of the RC school building.

The numerical model for the URM school building was developed in the structural analysis software TreMuri [34] using an equivalent frame approach (see Fig. 10). Considering the regular structural configuration and the stiffness of the floor slab, it was assumed that a global failure mechanism governs the seismic response; the possible local mechanisms were separately verified and are not detailed here. Using the equivalent frame modelling approach, the masonry building is represented by two main structural element types: vertical piers and horizontal spandrels. The vertical and lateral capacity of the building is provided by the constitutive law assigned to the piers, while the spandrels couple the response of adjacent piers. The bi-linear relationship describing the piers' behaviour is reported in Fig. 10. The initial elastic branch is computed according to the geometric and mechanical properties of the panel and accounting for shear and flexural stiffness. Two in-plane failure mechanisms were accounted for to calculate the maximum shear capacity (V_u): 1) flexural failure due to rocking and crushing of the walls; and 2) shear failure due to shear sliding or diagonal cracking. The drift limit states provided by Eurocode 8 were assumed [35].

Finally, the numerical model of the PC school building was developed in the structural analysis software RUAUMOKO [36] using a lumped plasticity approach to simulate the behaviour of the columns, while a more detailed approach was followed to predict the dynamic response of the beam-to-column connections. This issue is one of the most important parameters governing the response of PC buildings, as past earthquakes [6] have demonstrated. For modelling the beam-to-column joints, gap and friction elements were adopted. Due to the lack of seismic restraints, the friction element simulates the sliding of the beams on the column corbels. Each joint reduced the moment

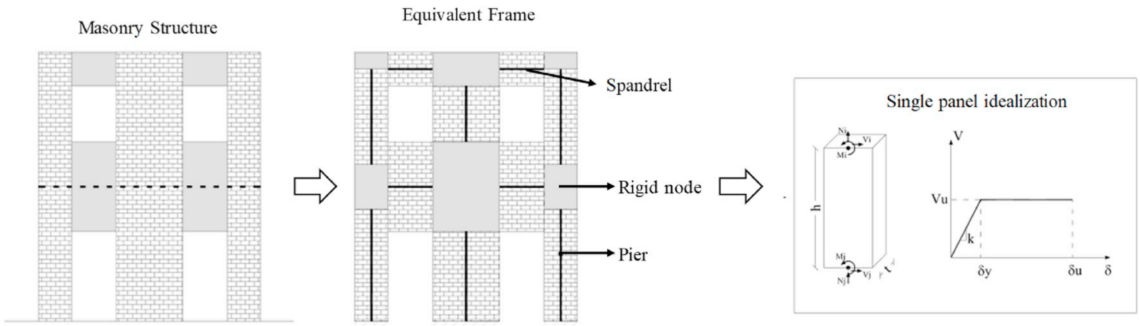


Fig. 10. Numerical model of the URM school building.

capacity based on the frictional resistance of the beam seated on the column corbel multiplied by the lever arm. The stiffness of the slab was directly modelled, based on the real properties obtained during the in-situ survey.

4.2. Characterisation of seismic hazard

As discussed in Section 2, the selection of the school building typologies was done in order to be representative of the existing Italian school building stock. Furthermore, the considered structures were designed without specific seismic provisions. Based on these considerations, it was deemed reasonable to assume that the selected school buildings may exist in any location throughout Italy hence two sites, characterised by different seismic hazard levels (medium-low and medium-high), corresponding to the cities of Ancona and Cassino were selected and the PSHA performed. The first site (Ancona) is characterised by a PGA on stiff soil equal to 0.16 g for a 10% probability of exceedance in 50 years (or 475 years return period), while the corresponding PGA value for the second site (Cassino) is 0.21 g. Fig. 9 reports the hazard curves for the two sites showing the relationship between the mean annual frequency (MAF) of exceedance and level of spectral acceleration for different return periods. Hazard-consistent record selection was based on spectral compatibility with a conditional mean spectrum, according to the methodology of Jayaram et al. [37], which also considers the conditional variance given return period of spectral acceleration at the vibration period of interest. The selection procedure considers spectral acceleration as the maximum between the two as-recorded horizontal components. This is consistent with the definition of spectral acceleration adopted within the ground motion prediction equation employed for the hazard calculations [38]. Seismic hazard calculations and derivation of the conditional mean spectra were performed using the REASSESS software [39], using the correlation model among spectral acceleration ordinates suggested by Baker and Jayaram [40]. Eleven return periods, varying from 30 to 9975 years, were considered. For each return period, a set of 20 ground motion pairs were selected from the PEER NGA-West database [41]. Each hazard curve shown in Fig. 11 refers to a different conditioned period. The conditional periods, to be used for the nonlinear dynamic analysis of each school building, were selected based on the results of eigenvalue analyses. In particular, for each building, the average of the periods in the two main directions was computed and used to choose the closest conditional period.

4.3. Structural response

Nonlinear static (pushover) analyses were initially conducted to characterise the seismic response of the three school buildings. The performance points representing the exceedance of the severability and ultimate limit states were identified in each capacity curve for both principal directions of the school buildings, as detailed in O'Reilly et al. [29]. According to the Italian Code [42], four limit states must be considered:

- Operational Limit State (SLO): following the earthquake, the building, including structural and non-structural elements, maintains its function without damage and interruption of its usage;
- Damage Control Limit State (SLD): following the earthquake, the building, including structural and non-structural elements, does not suffer damage that could put at risk the life of occupants and compromise the strength capacity and stiffness to the vertical and horizontal loads. The loss of functionality for some systems is allowed;
- Life Safety Limit State (SLV): following the earthquake, the building suffers the failure of non-structural elements and significant damage to the structural elements that involve loss of lateral stiffness.

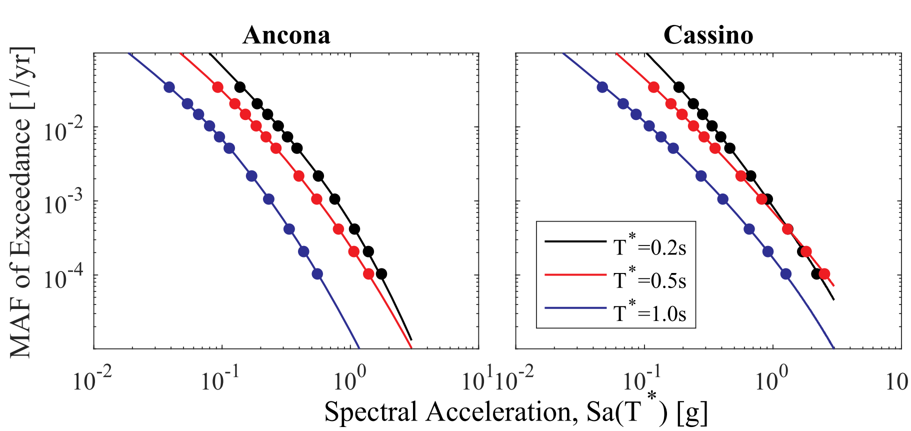


Fig. 11. Seismic hazard curves for selected building sites.

However, the building maintains the gravity load carrying capacity and partial safety with respect to seismic loads;

- Collapse Prevention Limit State (SLC): following the earthquake, the building suffers heavy damage to structural and non-structural elements but still maintains a partial gravity load carrying capacity.

As will be further explained in Section 6, the identification of limit state exceedances is paramount for the seismic risk classification according to the Italian guidelines [16]. From these pushover analyses, the resulting capacity curves indicated a higher lateral stiffness of the URM building with respect to the RC and PC school buildings (Fig. 12). The RC school building is characterised by a low ductility and a soft-storey mechanism occurring in both directions. As expected, the presence of the masonry infill results in a large initial stiffness and maximum base shear coefficient for the RC building, which degrades as a result of the combined effects of the exhaustion of the masonry infill capacity, the post-capping degradation of beam and column member capacities in addition to geometrical nonlinearity (P-Δ) effects. The pushover analysis of the URM school building showed its high seismic vulnerability. Due to the presence of a reduced number of masonry walls, the base shear coefficient in the transverse direction is lower than in the longitudinal direction. In particular, the URM school building already reached its maximum capacity in the transverse direction at the SLD limit state. In both directions, the collapse mechanism is related to the shear failure of piers and spandrels. In the case of the PC school building, a similar behaviour was observed in the longitudinal and transverse directions, both in terms of base shear coefficient and displacement capacity. However, the collapse mechanisms in the two main directions differ. In the longitudinal direction, the failure of the structure is mainly related to the cantilever behaviour of the columns and in the transverse direction, after an initial cantilever behaviour of the columns, the closure of the column-beam gap was observed. This mechanism induced the immediate increase of the base shear coefficient and, then, the sliding of the beam at the second level, causing the general failure of the structure.

With the aim of calculating risk-based measures of performance (e.g. annual probability of collapse, expected annual losses) a multiple-stripes analysis (MSA) was conducted. The MSA consists of conducting nonlinear dynamic analyses at a number of different intensity levels, or 'stripes', where at each intensity a different set of ground motions is used. The results of the MSA were used to compute the collapse fragility function for each building, which was estimated by taking the number of collapsing cases at each intensity level and fitting a lognormal distribution using the maximum likelihood method [43]. The intensity measure used herein was the spectral acceleration (S_a) at the conditioned period T*, where T* is chosen in a way to be consistent with the hazard information and also be representative of the period range of the building's modes of vibration. Fig. 13 reports the collapse fragility functions for each building considering both locations. The collapse fragilities for each school building at the two site locations are relatively consistent between the two different ground motions ensembles but do not coincide, which denotes the importance of the building's location in its seismic performance.

5. Loss estimation

As discussed above, the FEMA P-58 [13] methodology is the most complete procedure currently available to perform the probabilistic seismic assessment of a building performance. It allows to perform loss analysis using four main steps (Fig. 14). The first step consists in the identification of all facility information including structural and architectural details, as well as the hazard model. Once the structural model is developed, the structural response is assessed through nonlinear dynamic analysis. The output of the structural analysis results in probabilistic distributions of engineering demand parameters (EDPs), such as peak inter-storey drift and peak floor acceleration for a given level of seismic intensity. These quantities are combined with fragility functions of both structural and non-structural elements to estimate the level of observed damage. Finally, the results of the damage analysis are used to estimate the performance of the building in terms of expected annual losses (EAL), fatalities and downtime.

The Performance Assessment Tool (PACT), developed as part of FEMA P58 [13], was utilised to conduct the loss estimation for the three school buildings. The structural and non-structural inventory developed during the in-situ surveys was used to create a building performance model of all elements that could be damaged following a seismic event. For each element typology, quantities, EDPs, fragility functions and consequent functions were identified. The adopted fragility functions for structural elements refer to existing sources [45,46] or to experimental data [47]. For masonry infills, the fragility functions proposed by Sassun et al. [24] were adopted while the data provided by FEMA P58 [13] were utilised for the other typologies of drift-, acceleration- and velocity-sensitive non-structural elements. For elements for which specific fragility functions were not available, some assumptions were made using engineering judgment. The repair cost functions were taken from existing literature [45,46,48], Italian cost manuals [49] or adapting to the Italian context the repair cost functions from the PACT fragility library [13].

To conduct the loss estimation study, the replacement cost of the buildings must be defined. Available information following the 2012 Emilia-Romagna earthquake was used to estimate the replacement cost as well as the costs of demolition and removal of debris [50]. The average replacement cost used in this study is equal to 1805.75€ per m^2 in addition to 95.50€ per m^2 for demolition and removal of debris. Three criteria were introduced to decide if the building ought to be replaced: 1) structural collapse, 2) ratio between the expected direct loss and the replacement cost, and 3) residual lateral drift limit. The threshold for the ratio between expected direct loss and replacement cost for which the

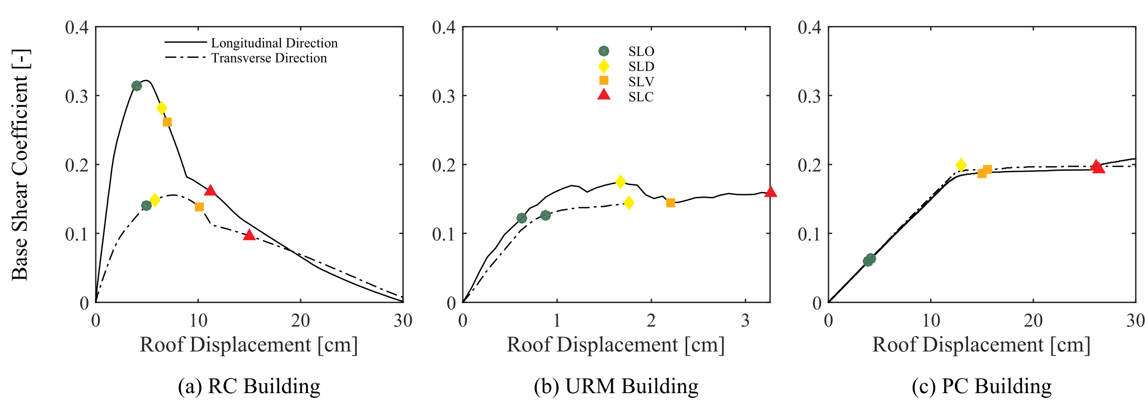


Fig. 12. Static pushover curves and limit states for each school building.

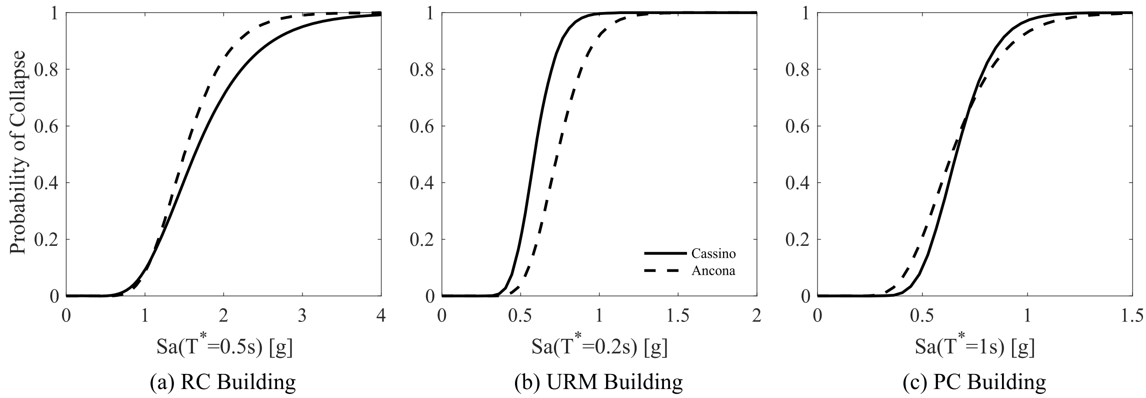


Fig. 13. Collapse fragility curves for each school building.

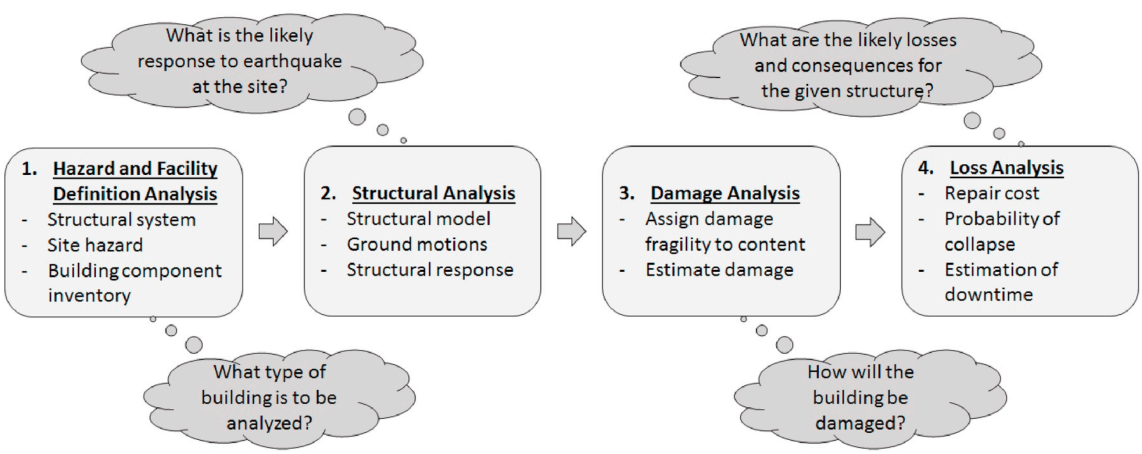


Fig. 14. Overview of the four stages of PEER PBEE framework, after [44].

stakeholders would decide to demolish instead of repairing the building was assumed as 60%, according to Cardone and Perrrone [51], while the residual drift fragility function for RC and PC buildings was characterised by a median storey drift equal to 1.5% and a dispersion equal to 0.30 [52]. The residual drift limit criterion was not adopted for the URM building because, to the authors' knowledge, such a criterion is not available in the literature.

The expected losses were calculated using PACT for each of the 11 ground motions return periods considered. Fig. 15 reports the results of the loss estimation study in terms of vulnerability curves (Fig. 15a) and EAL ratios (Fig. 15b). From Fig. 15a, it can be seen that the URM school building has higher vulnerability than the RC and PC school buildings. The expected losses of the URM building equal the replacement cost for the 475-year return period if the building is located in Cassino, while for RC and PC buildings a 2475-year return period is required to reach an expected loss ratio equal to unity. For the RC and PC buildings, a more gradual increment of the expected loss ratio is observed, when compared to the URM school building. The influence of the site location is also

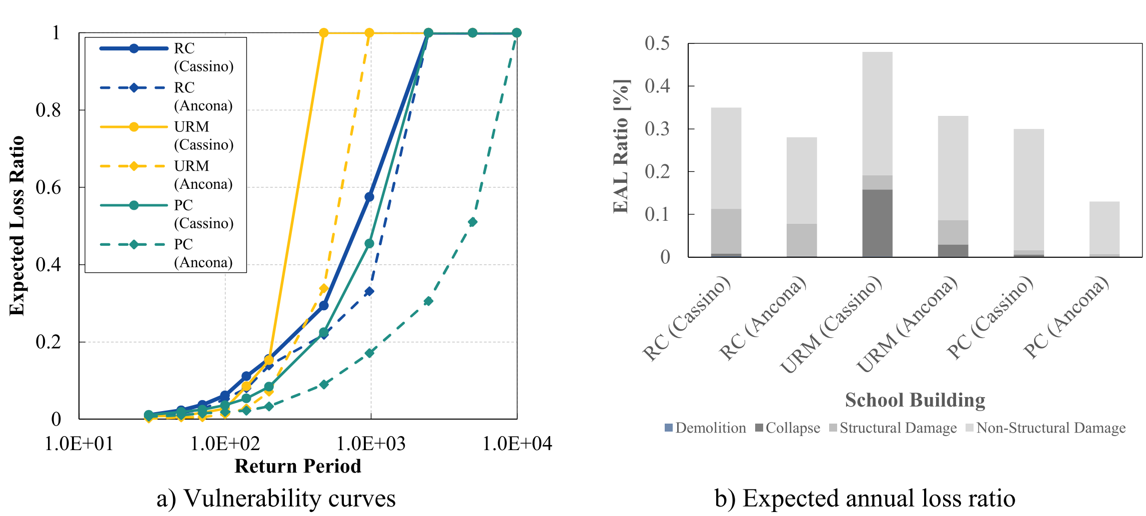


Fig. 15. Results of loss estimation analysis for the three school buildings and at both site locations.

observed, as the Ancona site generally leads to higher return periods to achieve an expected loss ratio of one.

Fig. 15b shows the EAL ratios (i.e. EAL divided by the building replacement cost) distinguishing between the losses due to structural and non-structural damage, as well as EAL ratios associated to structural collapse and demolition. The difference between structural collapse and demolition is in the state of the building at the end of each realization. If excessive residual drifts, which require the demolition of the building, were computed during the analyses, the expected loss was associated with the demolition scenario, where the building would need to be demolished and removed. The same occurs if the ratio between the expected direct loss and the replacement cost is higher than 60%. The expected loss was related to the collapse case if the building achieved the structural collapse during the analyses, which means that just the removal costs were considered. In all the other cases, the structural and non-structural losses were separately computed. The highest contribution to the EAL ratios is provided by damage to non-structural elements for all case study school buildings, particularly for lower and more frequent return periods. For RC and PC buildings the damage to non-structural elements account for approximately 70% and 90% of the total losses, respectively. A lower percentage is observed for the URM typology due to the lower structural performance, as demonstrated by the higher EAL ratios associated to the structural collapse. These results pointed out the importance of considering the non-structural elements in the performance assessment as well as of detailed surveys to gain their typologies and quantities.

The EAL ratios for the medium-high seismicity Cassino site are equal to 0.35%, 0.48% and 0.30%, respectively, for the RC, UMR and PC school building, and equal to 0.28%, 0.33% and 0.13% for the medium-low seismicity Ancona site. These values are comparable with those available in the literature for similar building typologies [48,51,53,54].

In addition to the loss estimation analysis, the collapse safety of the buildings was also analysed to determine the return period associated to the collapse prevention limit state. According to the Italian building code, the return period associated to the collapse prevention limit state for school buildings is equal to 1463 years. The performance of the school buildings was assessed by verifying that the probability of collapse at the collapse prevention limit state is lower than 10%, as suggested by FEMA P695 [55]. The results pointed out that the probability of collapse is higher than 10% for all case study school buildings if the schools are located in Cassino (medium-high seismicity), while only the URM school building does not satisfy the criteria for the Ancona site (medium-low seismicity). These results point out the importance of structural retrofit to improve the seismic performance of the analysed school buildings. The best retrofit strategy should be selected focusing not only on the improvement of the structural performance but also analysing the retrofit scheme that does not negatively affect the EAL [56].

6. Seismic risk classification

The FEMA P58 methodology [13] represents a suitable tool to calculate EAL of buildings but it is often difficult to be applied by practitioners. In February 2017, the Italian "*Guideline for the seismic risk classification of constructions*" was approved [16], proposing a methodology to define a seismic risk classification of buildings based on a simplified calculation of their seismic performance and EAL. In this section, this methodology is initially applied to the three case study school buildings for the two considered sites, followed by a preliminary extension of the results to obtain seismic risk estimates of the same school buildings located throughout the Italian territory.

6.1. The Italian seismic risk classification

The Italian "*Guidelines for the seismic risk classification of constructions,*" commonly known as SISMABONUS, defines a technical procedure to obtain tax deductions by improving the seismic performance of buildings through strengthening interventions. The proposed procedure is simple and allows practitioners to deal with the evaluation of EAL without having to perform sophisticated probabilistic seismic assessment. A letter-based classification is used to define the seismic class to within which a building performs. The seismic class is defined as the worst between two classes: the first is related to the life safety of the building's occupants under a severe event, associated to the structural performance at SLV (IS-V), while the second is related to economical features, represented by EAL. Fig. 16 illustrates the steps involved in the procedure. The parameter IS-V is defined as the ratio PGA_D/PGA_C, where PGA_D is the design peak ground acceleration of the code spectrum according to the hazard map, while PGA_C is the capacity peak ground acceleration at which SLV is achieved. The PGA_C is assessed using nonlinear static pushover analysis and then converting it to a single-degree-of-freedom (SDOF) system. The PGA required to achieve the four limit states defined in Section 4.3 are also computed in order to evaluate the mean annual frequency of exceedance (MAFE). Given the PGAs, the return period of the relevant seismic event can be computed using the seismic hazard maps provided by the reference building code. For each limit state, the guidelines prescribe corresponding expected loss ratios, which can then be integrated with the MAFE to calculate the EAL as the area under the loss curve. A detailed description of the procedure is provided by Cosenza et al. [15].

6.2. Seismic risk classification of the case study school buildings

The Italian SISMABONUS was applied to the three school buildings and the results were compared with those obtained following the detailed FEMA P58 (PACT) methodology [13]. The PGAs of the code spectrum required to exceed each limit state described in Section 4.3, in both building directions, were identified using the N2 method [57] and following the prescriptions provided by Eurocode 8 [34]. The lower of the two limit state intensities in either direction was used for each limit state in order to compute the MAFE and EAL.

The overall seismic classification of the three buildings yielded that the RC and URM school buildings are classified in the seismic class C for Cassino site and in seismic class B for Ancona site. The PC school building falls in the seismic class A and A+ for the Cassino and Ancona site, respectively. For all analysed school buildings, the safety index criteria (IS-V) is governing the overall seismic classification. Fig. 17 shows the EAL ratios computed from detailed analysis (see Section 5) and those estimated from the simplified SISMABONUS method. Despite the EALs computed using the simplified method being much higher with respect to those calculated using the detailed analysis, the general trends are similar and confirm the typical conservative nature of the simplified procedure. The relative differences between the EALs computed for the two sites are also consistent between the two methods. O'Reilly et al. [29] investigated the reasons of the discrepancies and suggested two main issues. The simplified procedure relies on the assumption that the loss ratios for each limit state are fixed regardless of building typology and occupancy. While simplifying the computation of EAL, this assumption could lead to different results with respect to more advanced methodologies. A second important issue is related to the assumption that the damage begins at a fixed return period of 10 years and small changes to this threshold significantly impact the estimated EAL. Finally, some discrepancies could be also related to the analysis methodology used to assess the structural behaviour. It is generally recognised that the pushover analysis leads to results that are different than the ones obtained with nonlinear time history analyses, in particular for those structures for which the structural behaviour is not dominated by the fundamental mode. In those cases, adaptive pushover analysis, which is generally not considered in most of seismic design provisions, could be used to improve the prediction capability with respect to nonlinear time history analyses, which also account for the record-to-record variability.

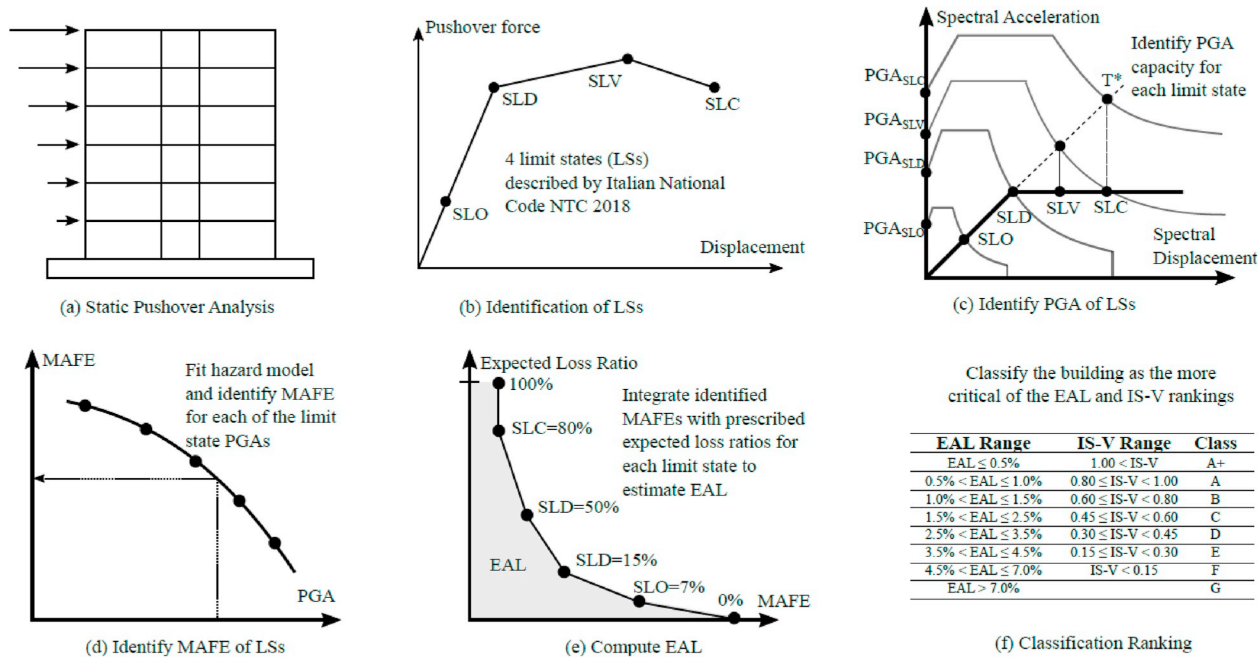


Fig. 16. Steps of the methodology proposed by the Italian Guidelines for the seismic risk classification of constructions [16].

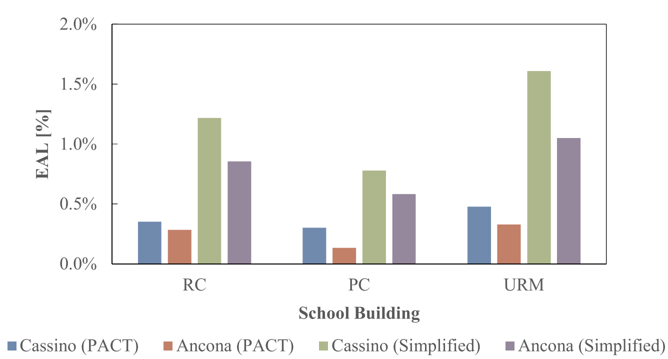


Fig. 17. Comparison of the EAL ratios from detailed analysis and those estimated from simplified analysis.

Although some discrepancies between the simplified and detailed methods are observed, the capability of the simplified method to predict the overall trend and the improvement of the seismic performance between different structural typologies encourage the adoption of this method into practice and in performing large scale risk mapping.

6.3. *Preliminary extension to the Italian territory*

Using the Italian seismic risk classification guidelines illustrated in Fig. 16, the relative performance of each school building typology throughout the entire Italian peninsula may also be quantified. Such a comparison relies on a number of simplifying assumptions, so any conclusions drawn should be treated as indicative rather than accurate calculations. It is noted, however, that this exercise is still completely consistent with the current implementation of the seismic risk guidelines in Italy.

As previously discussed, a static pushover analysis was conducted for each building and the exceedance of each limit state was identified. Converting these pushover curves to equivalent SDOF systems, the seismic intensity levels required to exceed these limit states were identified as per Fig. 16c and (d). It is at this point of the procedure that some information regarding the seismic hazard was required. Until now in this study, just two locations were examined and the results illustrated in Fig. 17. Knowing the hazard curves at each grid point around the Italian peninsula available from Ref. [2], the relative seismic risk of each school building typology was calculated. These were quantified in terms of the life safety index (IS-V) and the EAL, as per the SISMABONUS framework.

Fig. 18 shows this comparison for each school building in terms of both life safety and EAL indices and the more critical of the two for each location in Italy. As an overall observation, it can be clearly seen how the URM typologies tend to be the most critical among the three typologies considered, followed by the RC buildings and lastly the PC buildings. Comparing the different plots on the third row for the URM building (Fig. 18 (g), (h) and (i)), for example, the life safety index at each location tends to be the dominating factor meaning that for these typologies, structural interventions should be prioritised. Doing the same comparison for the PC buildings on the second row (Fig. 18 (d), (e) and (f)), the EAL rating is seen to be dominant meaning that more advanced methods of protecting the vulnerable structural and non-structural elements in these structures could be sought, considering that the life safety index rating in most cases is already rated sufficiently high. The RC typology (Fig. 18 (a), (b) and (c)) tend to show a mix of these two previous situations, with the life safety index tending to govern around concentrated central areas of the country near Umbria, Abruzzo and Calabria, for example, and also the northeast in Friuli, but with the EAL rating highlighting potential issues on a more broader scale in more moderate zones of seismicity like Emilia-Romagna and stretching across large parts of Veneto into Lombardy.

Such a comparison allows the relative comparison of each school building typology's rating for different regions of the country. Expressed in this simplified manner, a broader perspective on the relative risk of the school building stock in Italy can be gained. With this, the more vulnerable structural typologies may be identified and the regions of the country in which more investment is needed may also be recognised. This kind of information is of particular use to governmental decision makers who need to decide and justify the distribution of limited financial resources that aim to reduce the overall seismic risk of the Italian building stock. Again, due to specific nature of the Italian seismic risk classification guidelines, it is also possible to identify whether a specific school requires structural reinforcements due to excessively high collapse risk, or whether the excessive accumulation of economic

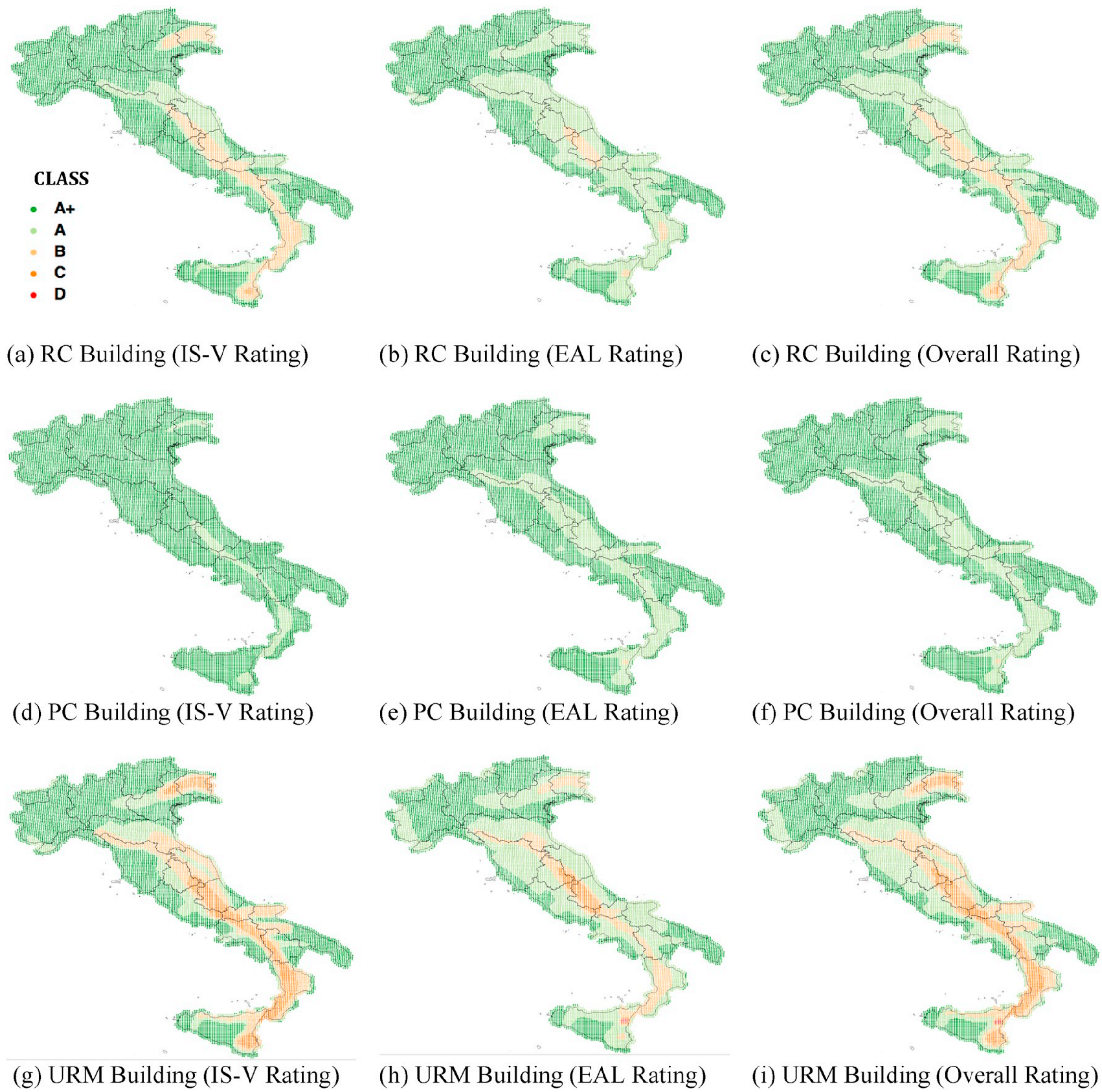


Fig. 18. Relative comparison of life safety index (IS-V) ratings, EAL ratings and overall (more critical) ratings for school buildings located in Italian peninsula.

losses ought to be reduced. The first aspect typically involves the provision of any number of structural interventions in order to improve the structure's lateral strength and ductility capacity in order to encourage a ductile and stable mechanism under severe ground shaking. The last aspect regarding the reduction of economic losses can be achieved in a number of ways, with the most familiar approach being through the structural retrofitting of the building to improve its performance. Alternatively, O'Reilly and Sullivan [56] have also shown how if the building's life safety performance is satisfactory but the economic losses need to be reduced, retrofitting the non-structural elements can give very effective results in this regard, such as the PC buildings in the example presented in Fig. 18. This study also noted that the excessive structural retrofitting of a building can results in instances where the EAL actually increases, essentially meaning that the collapse risk is reduced but the dominant problem of excessive EAL has actually been worsened by this intervention through increase seismic demands on acceleration-sensitive non-structural elements.

7. Conclusions

An overview of the main results obtained during a recent research project entitled "Progetto Scuole", whose main objective was to investigate the seismic risk of existing Italian school building stock, have been presented in this paper. In particular, the seismic loss assessment of three school buildings representative of the existing Italian school building stock was presented. The detailed seismic loss assessment framework, following the developments by the Pacific Earthquake Engineering Research Center on performance-based earthquake engineering, and recent simplified Italian guidelines developed for seismic risk classification of existing buildings were applied to three school buildings in Italy to identify the most vulnerable structural typology. The three school buildings were selected from an Italian school database developed within the "Progetto Scuole" as well as during previous research projects made in collaboration with the Italian Department of Civil Protection. Each building had a different construction typology: 1) Unreinforced Masonry (URM), Reinforced Concrete (RC) and Precast (PC). Detailed in-situ surveys were performed to gather the information required for the numerical modelling as well as for the loss estimation assessment. The importance of the in-situ surveys, and of ambient vibration measurements, was pointed out by using such data to improve the prediction capability of the numerical models. Two sites characterised by different seismic hazards were initially selected to compare the results of the detailed analyses with those of the simplified methodology. The simplified Italian guidelines were then applied throughout

Italy knowing the hazard curves at each point around the Italian peninsula in order to provide a broader perspective on the relative risk of the school building stock in Italy.

The results of this study highlighted the seismic vulnerability of existing school buildings in Italy both in terms of collapse capacity and expected annual loss (EAL). The URM was identified as the most vulnerable structural typology. The damage to non-structural elements tends to dominate the EAL for all analysed school typologies, in particular at more frequent return periods of the seismic intensity. Comparing the findings of the detailed loss assessment with those of the recent seismic risk classification guidelines, introduced in Italy in 2017, a similar trend in terms of identification of more vulnerable typologies was observed. However, due to the simplified nature of the guidelines, the overall magnitude of the EAL was significantly overestimated with respect to the EAL evaluated using the detailed approach. Despite this, this simplified methodology has been found to be a useful tool to investigate the seismic risk of school buildings at regional scale. The application of the simplified method throughout Italy pointed out that URM school buildings similar to that one analysed in this study are potentially at high risk in many Italian regions, in particular along the Apennine regions and in Friuli. A lower, but always considerable, seismic risk has been observed for RC school buildings, while the lowest seismic risk has been observed for PC school buildings. The results of this preliminary regional scale application are considered particularly useful to governmental decision makers who would need to decide and justify the distribution of limited financial resources that aim to reduce the overall seismic risk of the Italian school building stock. It is worth noting that the results could be significantly affected by the variability in materials and design practices often observed in different regions of the same country. For this reason, the considerations made in this study should not be generalised but should be only related to the analysed building typologies.

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Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.ijdrr.2019.101448.

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