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Seismic analysis and fragility estimate of a mixed masonry-r.c. school building.

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Abstract

Nowadays, there is still very little scientific understanding of the seismic response of mixed masonry-r.c. structures, despite their unneglectable frequency, in particular in public building assets. This contribution presents the investigations carried out on a representative school, built at the turn of 1950s and 1960s. It is a two-storey building, characterised by central longitudinal load-bearing masonry walls, as well as transverse masonry panels, coupled with r.c. frames on major façades, and isolated columns in halls. This structural configuration is commonly found in public buildings, specifically schools. R.c. elements were exploited to build open-space environments, as well as to increase openings on façades, thus ensuring a better natural lighting.
In this study, masonry components were modelled through an equivalent frame model (EFM). Half-height infills, interacting with frames, were simulated through a single-strut macro-model. To simulate the nonlinear response of the structure, lumped plasticity hinges were implemented for both load-bearing masonry and infills, while a fibre model was chosen for r.c. frames. The presence of non-seismic joints among structural units was also considered. The relative contribution of masonry and r.c. components was investigated through parametric linear dynamic analyses. Preliminary nonlinear static analyses (NLSA) were carried out to identify thresholds of damage. Moreover, the structure was analysed through nonlinear time history analyses (NLTHA) by applying a large number of natural unscaled ground motion records. Lastly, fragility curves were estimated from outcomes of NLTHA. The derived fragility model represents a key instrument in seismic risk evaluations for the analysed macro-class, being one of the few examples of fragility sets specific for mixed masonry-r.c. buildings.

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1. Introduction

Historically, masonry had been the most common material for buildings. After the Second World War, the spreading of reinforced concrete (r.c.) technology led to the rise of mixed structures, in which the vertical structural system was given by a combination of r.c. and masonry elements. Nonetheless, despite the spreading of mixed structures both in private and public assets, to date, far too little attention has been paid to the seismic behaviour of this type of buildings (Magenes, 2006).

The first laboratory tests on mixed structures were presented in the works by Tomazevic and Modena (Modena and Tomazevic, 1990; Tomazevic and Modena, 1988a, 1988b). The dynamic behaviour of a mixed configuration (perimetral masonry walls with a central r.c. column) was compared with purely masonry and r.c. structures; in this case, due to the high difference in stiffness between masonry walls and the r.c. column, the latter had no influence on the global response under lateral loads. Other experimental tests on mixed structures were carried out by Paparo & Beyer (2014), aimed to assess the structural capacity in terms of ultimate lateral drift.

Some numerical investigations were carried out on various configuration of mixed buildings. Augenti & Parisi (2009) evaluated the distribution of lateral forces on coupled systems through linear analyses, while Cattari and Lagomarsino (2006, 2013) focused on nonlinear modelling of r.c. elements coupled with the equivalent frame model (EFM) of masonry components. A specific structural configuration was studied by Ferrito et al. (2016) and Milosevic et al. (2018), who analysed a case-study building with masonry load-bearing walls with strengthening r.c. beams (belting external walls), for which the fragility assessment was also carried out (Milosevic et al., 2020). Lastly, the case of masonry buildings retrofitted with novel r.c. walls was investigated by Paparo & Beyer (2018). However, available numerical results are highly influenced by the modelling assumptions (Paparo & Beyer, 2012) and the large variety of possible structural combinations. Moreover, numerical models cannot be fully validated since paucity of experimental evidence (Magenes, 2006). Thus, a full and clear comprehension of the seismic response of mixed structures has yet to be achieved. This topic is also of great interest for lawmakers and professionals, as national and international codes often do not include specific guidelines for mixed structures (NTC, 2018, EN 1998-3:2005).

In the framework of a research agreement between the University of Padova and the Municipality of Padova, visual inspections were carried on an urban stock of school buildings (Saler et al., 2019). Investigations carried out on the surveyed urban stock highlighted how significant mixed masonry-r.c. buildings can be in similar inventories (e.g., public buildings in urbanised centres of the Po Valley).

In this contribution, typological and structural features of the subset of mixed schools in Padova are illustrated. Then, a representative mixed masonry-r.c. school was selected from the Padova school inventory. Numerical simulations were carried out on the prototype school, with the aim of evaluating the seismic behaviour of this building type, and the relative contribution of masonry and r.c. members. Furthermore, fragility curves were estimated by processing outcomes of non-linear time history analyses (NLTHA) for a suite of unscaled ground motions. The derived fragility model represents an important contribution in the field of risk evaluations for existing buildings in Italy. Indeed, this is one of the first study to provide a fragility set for a macro-class of mixed masonry-r.c. buildings.

2. Selection of a representative mixed masonry-r.c. building

The prototype mixed masonry-r.c. school analysed in this contribution was selected based on typological and structural characteristics observed for mixed masonry-r.c. schools of the Padova urban stock of school buildings (Figure 1). This subset of schools was mainly built between 1945 and 1975, in the years of Post-World War economic expansion. Two thirds of mixed school are two-storey buildings, while almost a third has one storey. For most of the dataset, clay bricks with lime mortar were identified for load-bearing walls, through archive documentation or direct on-site observation. The use of modern clay blocks with cement mortar was observed for a portion of 24% of the subset, in the most recent buildings. For all the cases for which information was available, the presence of ring-beams was observed. The organisation of structural system for the observed subset of schools was also analysed. The main types of mixed masonry-r.c. structures observed were the following, characterised in terms of position of r.c. elements: *i*) central frames (24%); *ii*) r.c. frames on façades, with or without isolated r.c. columns in halls (66%); and *iii*) single columns in halls (6%). In cases *i*) and *iii*) r.c. elements can be expected to be secondary in the seismic response of the building, as masonry panels are often more stiff and more distant from the centre of rotation.

The selected prototype school has two storeys, and it was built at the turn of 1950s and 1960s in subsequent construction phases. For this reason, three structural units (s.u.) separated by non-seismic joints were identified. The overall building appears irregular in plan, with a C-shape plan arrangement; however, each s.u. present a more regular rectangular shape. Its structural system is characterised by r.c. frames on longitudinal façades, coupled with central longitudinal masonry walls, as well as transverse masonry panels. In addition, single columns were used in halls, to create large spaces. This arrangement corresponds to the above-mentioned type *ii*), in which a significant contribution of r.c. elements to the global seismic response is expected. A scheme of plan arrangement of the case study is illustrated in Figure 2a.

The analysed school has masonry walls made of clay bricks with lime mortar, which mean mechanical properties were assumed according to Italian Code (Circ 21/01/2019 N.7, 2019). The same type of masonry was plausibly adopted for infills walls in r.c. frames. Results of compression tests on concrete cores, which were carried out at the time of construction, were retrieved, and used to evaluate a mean compressive strength of concrete, equal to 27.3 MPa. On the contrary, no information was available for the class of reinforcing steel used. The class of smooth rebars (AQ42) was adopted on the basis of the construction period, according to a literature study (Verderame et al., 2011). Values of material properties are summarised in Table 1.

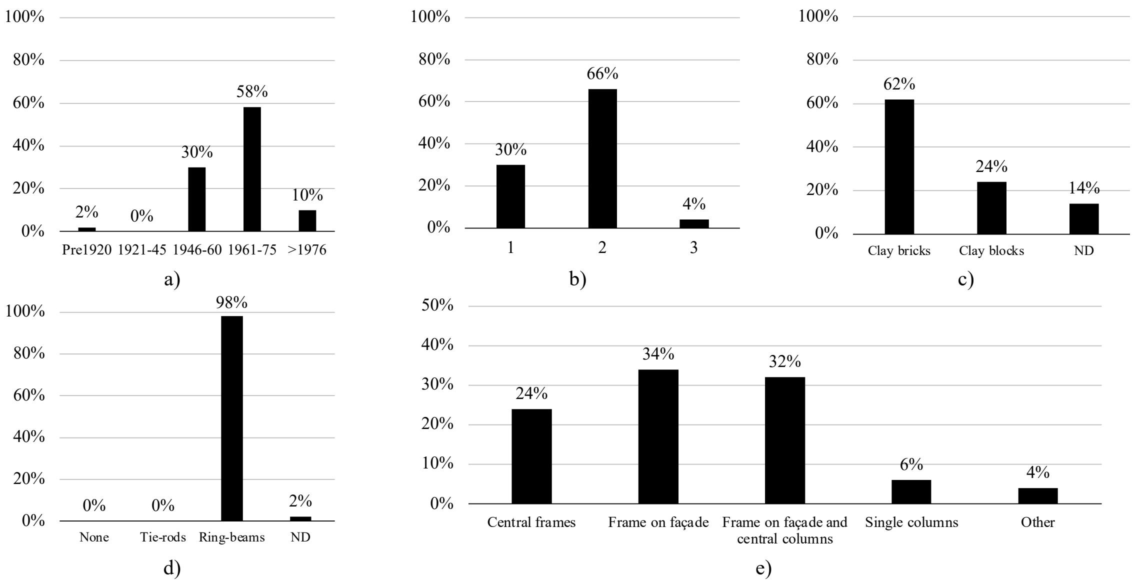
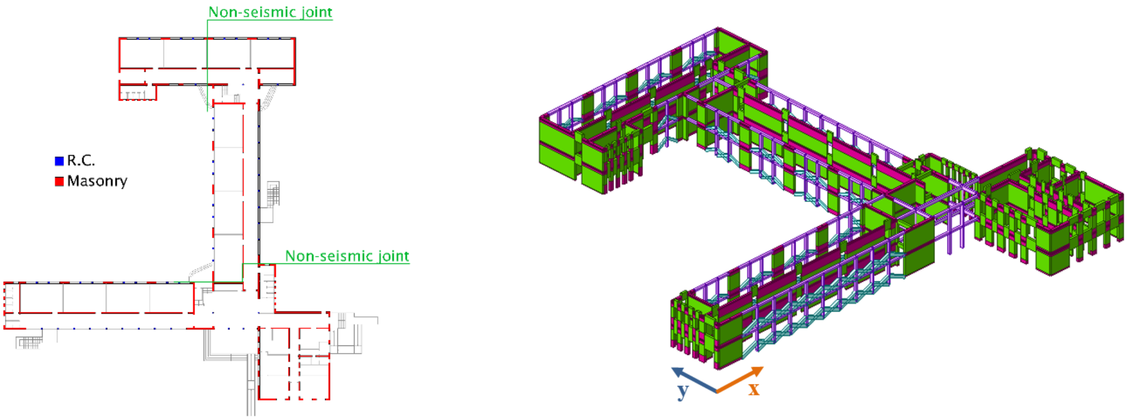


Figure 1. Typological and structural features of mixed masonry-r.c. schools in Padova.

Table 1. Mechanical properties of materials.

Masonry		*Concrete*		*Steel*	
Mean compressive strength (f_m) [MPa]	3.45	Mean compressive strength (f_{cm}) [MPa]	27.32	Mean strength (f_{ym}) [MPa]	322.3
Shear strength (τ_0) [MPa]	0.09	Elastic modulus (E) [GPa]	29.74	Elastic modulus (E) [GPa]	210
Shear strength w/o vertical loads (f_{vk0}) [MPa]	0.2				
Elastic modulus (E) [GPa]	1.5				
Shear modulus (G) [GPa]	0.5				
w [kN/m^3]	18				



a) b)

Figure 2. Scheme of plan arrangement (a), and F.E. model (b) of case study.

The school presents r.c. floors, with various type of clay lightening elements, typical of the age of construction (i.e., SAPAL floor and N-Rex floor), all endowed with a rigid reinforced slab.

3. Structural modelling

Numerical simulations on the selected school were carried out by using the software Midas Gen (MIDAS Information Technology Co., 2020). Numerical modelling of the structure is shown in Figure 2b.

Equivalent frame model (EFM) was implemented for masonry components, by identifying deformable elements (i.e., piers and spandrel) and rigid nodes, according to (Dolce, 1989). Nonlinear behaviour of both piers and spandrels was characterised through lumped plasticity models, defined according to provisions of the U.S. Federal Emergency Management Agency (FEMA, 1997). Reinforced concrete elements were modelled through a distributed plasticity (i.e., fibre) model (Spacone et al., 1996), adopting the constitutive law for concrete proposed by (Kent and Park, 1971). Different laws were defined for cover and core concrete, respectively, considering for the latter a slight effect of confinement (Mander et al., 1988).

A single-strut macro-model was implemented to simulate the effect of half-height infills, on the global seismic response. The equivalent strut parameters were defined (Mainstone, 1971; Stafford Smith, 1967) considering two types of infill panels, characterised by different geometry (height and thickness) which were surveyed in classrooms and corridors, respectively. To overcome the absence of shear deformations in fibre elements, non-linear lumped hinges simulating shear failure were adopted, in series with fibre columns. Indeed, the presence of half-height infills might induce shear failure in columns, due to reduced effective length. Lumped shear hinges were defined having brittle behaviour, with an initial elastic branch up to the maximum shear strength, and a sudden loss of resistance beyond it. The maximum shear strength was calculated according to Italian Code (NTC, 2018) for r.c. section with no reinforcement.

Non-linear contact points were implemented at interfaces among s.u. for non-linear dynamic analyses. Indeed, non-linear elements cannot be included in eigenvalue analysis for modal evaluations.

Linear dynamic analyses were carried out on the implemented model by considering the following configurations: *i*) whole building with no separation in s.u., and *ii*) separate models of each structural unit. The relative contribution of masonry and r.c. components was thus investigated. Moreover, the structure was analysed through nonlinear time history analyses (NLTHA) by applying a suite of 84 unscaled ground motion records, covering a large range seismic intensity.

4. Contribution of masonry and r.c. components

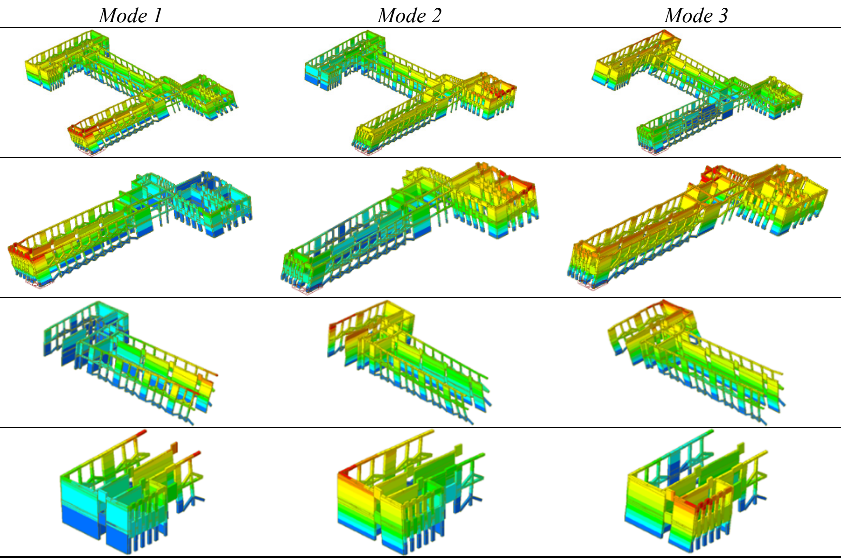
Results of linear dynamic analyses are discussed in this section, to evaluate the relative contribution of masonry and r.c. frames to the global seismic response for the analysed school building.

Table 2 shows results of eigenvalue analyses carried out on four models, listed in the previous section. In the numerical model of the whole building, torsional component appeared significant in the first two modal shapes, to become prevalent in the third mode. For each s.u., the percentage of participant rotational mass in first modes increased, suggesting a greater torsional deformability of structural units.

The portion of base shear for masonry and for reinforced concrete, respectively, is compared in Figure 3, normalised on masonry base shear. Indeed, a structural system - in this case, r.c. frames - can be considered secondary towards seismic actions whether its contribution to the total stiffness do not exceed 15% of the analogous stiffness of the main system - in this case, masonry walls - (NTC, 2018). Thus, the thresholds value of 15% is indicated in graphs.

Table 2. Dynamic properties for first three vibration modes.

Mode	T [s]	m_{Tx} [%]	m_{Ty} [%]	m_{Rz} [%]
Whole building				
1	0.204	-	62.9	18.3
2	0.185	56.0	10.4	16.4
3	0.172	25.7	9.2	47.9
s.u. 1				
1	0.254	-	45.8	35.5
2	0.196	-	34.0	43.6
3	0.176	78.9	-	-
s.u. 2				
1	0.440	34.2	-	43.0
2	0.178	33.7	27.5	28.1
3	0.166	12.1	56.3	14.9
s.u. 3				
1	0.342	-	48.3	32.0
2	0.161	41.3	18.3	22.3
3	0.147	40.7	11.7	28.9



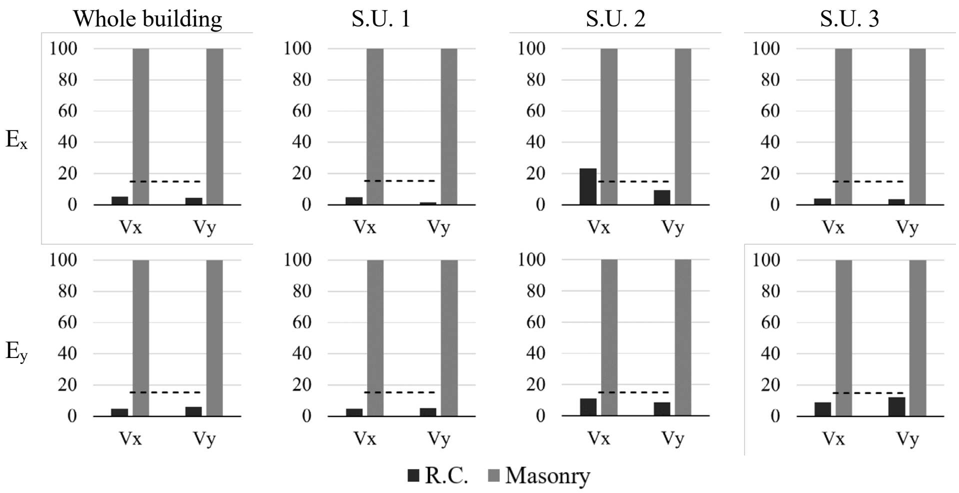


Figure 3. Comparison of r.c. and masonry relative contribution to base shear, normalised on masonry base shear.

Results from the numerical model of the entire building suggested that r.c. frames could be considered secondary. However, when each structural unit was specifically analysed, the central units showed the exceedance of the threshold value for one horizontal components. Therefore, through a more detailed evaluation that considers the specific dynamic characteristics of each s.u., the seismic response of r.c. elements appeared not negligible. These considerations, deduced based on a specific case study, highlight the need to carefully analyse this type of structure, case by case, to take into account the specific characteristics of each building and better simulate the actual behaviour.

5. Fragility assessment

A large number of nonlinear time history analyses were carried out on the building's model, with the aim of deriving fragility curves, expressed as lognormal cumulative distribution functions. A ground motion suite comprised of 84 natural unscaled ground motions (Paolucci et al., 2020; Manfredi et al., 2022) was adopted in this study. All ground motion records were bidirectional, and referred to soil types A and B. The peak ground acceleration (PGA) was adopted as intensity measure for the fragility assessment.

The proposed fragility set was defined for four damage states (DS): slight (DS_1), moderate (DS_2), severe (DS_3) and very heavy (DS_4) damage (Grünthal, 1998).

A key point in fragility estimate through numerical simulations is the definition of the demand parameter. In this study, the interstorey drift ratio (IDR) was adopted, by maximising its values among storeys and the two main horizontal directions. IDR thresholds, describing the transitions between subsequent DS were estimated for the case study, adopting criteria defined by Rota et al. (2010). Preliminary nonlinear static analyses (NLSA) were implemented, and performance levels (PL) were identified on pushover curves according to the following definition:

- PL1 (between DS_0 and DS_1): first attainment of yield displacement in a masonry pier (FEMA, 1997; NTC, 2018);
- PL2 (between DS_1 and DS_2): first shear cracking in a masonry pier (FEMA, 1997; NTC, 2018);
- PL3 (between DS_2 and DS_3): maximum shear resistance in the pushover curve;
- PL4 (between DS_3 and DS_4): attainment of 80% of maximum shear resistance in the pushover curve.

The first two thresholds were thereby defined locally, considering masonry mechanisms only, while the other values were related to the global response of the structure. Indeed, NLSA showed the first attainment of local failure in masonry piers, followed by the first failure of spandrels and lastly by the yielding of r.c. columns. The first two damage states were thus controlled by masonry piers, as critical elements.

Then, IDR thresholds were estimated as median value of each PL (Figure 4a).

Cloud plot of results of NLTHA is displayed in Figure 4b, in terms of (natural logarithm of) maximum IDR, associated with (natural logarithm of) the intensity measure (i.e., PGA of the event). Results were thereby associated with the attained damage state and processed to directly estimate the parameters of fragility functions – the mean value (μ) and the logarithmic standard deviation (β) – for each DS, as also proposed in other studies (Masi et al., 2021; Saler et al., 2021).

Dispersion related to record-to-record variability was thereby directly included in the estimate of logarithmic standard deviation (β_D). However, other sources of uncertainty should be included to derive fragility curves suitable for large scale risk evaluations. Hence, dispersions associated with structural capacity and threshold estimate were adopted equal to 0.3 and 0.4, respectively, according to HAZUS for pre-code buildings (FEMA, 2020). Dispersion values were then combined through SRSS (*Square Root of Sum of Squares*) combination, obtained the total dispersion (β_{TOT}). Mean values and standard deviations for the derived fragility set are finally showed in Table 3.

Lognormal fragility curves were thus defined for the analysed case study and illustrated in Figure 4c.

6. Conclusions

This contribution has presented the investigations carried out on a representative mixed masonry-r.c. school building, with irregular plan shape and r.c. frames on façades.

Table 3. Estimated parameters (median value and logarithmic standard deviation) of log-normal fragility curves.

	μ_{Dsi}	β_D	β_{TOT}
DS_1	0.094	0.350	0.610
DS_2	0.170	0.281	0.574
DS_3	0.310	0.336	0.602
DS_4	0.500	0.270	0.568

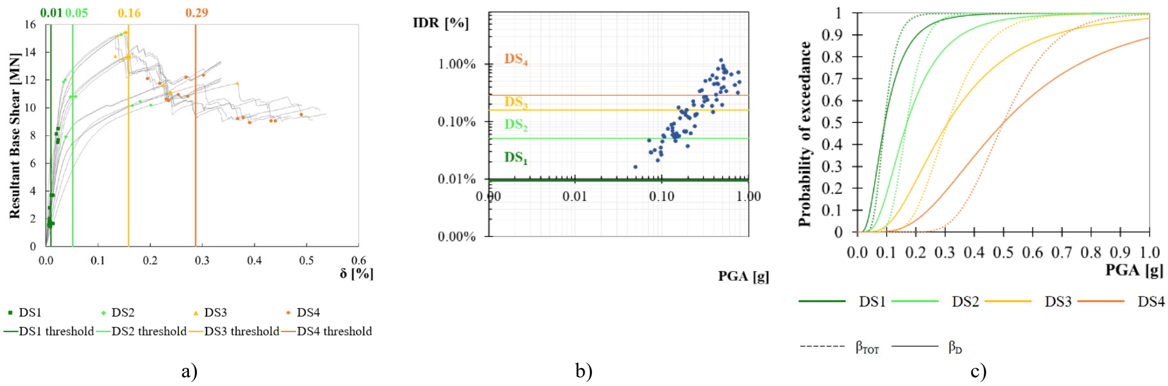


Figure 4. IDR thresholds identified on pushover curves (a), cloud plot (b), and derived fragility set (c) for case study

Results of linear dynamic analyses have been discussed to illustrate the relative contribution to base shear of the masonry and r.c. components, respectively. Results from modelling of the entire building suggested that r.c. frames could be considered secondary, but a deepen analysis of each s.u. highlighted the importance of r.c. contribution in the seismic response.

Preliminary nonlinear static analyses showed that low levels of damage were driven by the failure of masonry piers, which were observed to be critical elements in the structural response. Moreover, a fragility set for four damage states was derived through nonlinear time history analyses, by applying a suite of 84 natural ground motion records.

This contribution has provided one of the first fragility model for mixed masonry-r.c. schools, representing the expected seismic response of a mixed building with two storeys, built in the period 1950s-1960s, characterised by r.c. frames on façades, coupled with longitudinal and transverse masonry walls.

The findings of this research provide insights for the seismic vulnerability assessment of mixed buildings, which represents a significant portion of the building heritage in Italy, dated back to the reconstruction after the Second World War.

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