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RINTC-E PROJECT: TOWARDS THE SEISMIC RISK OF RETROFITTED EXISTING ITALIAN URM BUILDINGS

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Abstract

This paper presents the results of the work carried out in an ongoing Research Project, funded by the Italian Civil Protection Department, aimed at computing the risk of collapse of existing masonry buildings. Other papers submitted to this conference describe the overall Research Project [1], its different areas of application ([2][3][4][5][6]), the overall seismic risk calculation procedure, the seismic hazard assessment and the ground motion selection process followed to identify the recorded ground motions used for nonlinear dynamic analyses. Several unreinforced masonry buildings were considered, characterized by stone and clay masonry, regular and irregular geometries and number of storeys varying from two to seven. These buildings were retrofitted according to the prescriptions of different Italian seismic codes, applying commonly adopted retrofit techniques (e.g. floor stiffening, injection of masonry, addition of tie beams, etc). The paper presents the results for a single case study building. Equivalent frame models are used to simulate the in-plane response, whereas out-of-plane failure modes are prevented by the selected strengthening interventions. Pushover analyses are performed to estimate the capacity in terms of a properly selected engineering demand parameter, whereas the demand is obtained by multi-stripe nonlinear dynamic analyses for ten different earthquake's return periods. The results allow to understand the different level of risk implicit in buildings retrofitted according to various codes and strategies.

Keywords: Unreinforced masonry, Nonlinear static and dynamic analysis, Equivalent frame models, Multi-stripe analysis, Seismic retrofit.

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1 INTRODUCTION

RINTC-e is an ongoing research project, funded by the Italian Civil Protection Department, aimed at computing the risk of collapse of existing buildings. As a continuation of the previous work, which concerned new buildings [7], [8], [9], this paper focuses on understanding the different level of risk implicit in existing unreinforced masonry (URM) buildings retrofitted according to the prescriptions of different Italian seismic codes.

Several building configurations, realised with stone and clay masonry, with regular and irregular geometries, a number of storeys ranging from two to seven, and located in three different towns (L'Aquila, Naples and Rome) were analysed within the project. These buildings were retrofitted adopting common retrofit techniques aiming at improving wall-to-wall and wall-to-diaphragm connections, at increasing the in-plane stiffness of diaphragms and, in some cases, enhancing the strength of masonry by grout injections or other techniques.

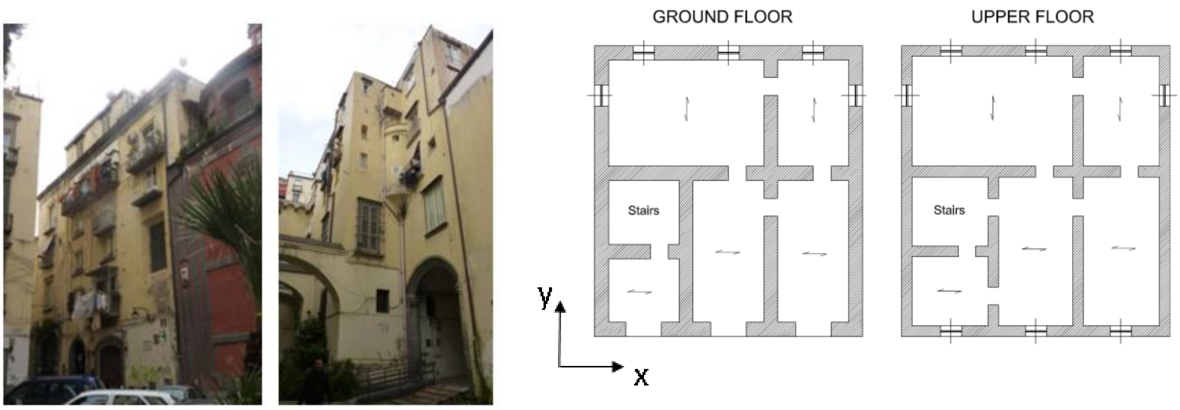
The risk assessment in terms of global failure was performed by multi-stripe nonlinear dynamic analyses for ten different earthquake's return periods. The equivalent frame model implemented in the Tremuri program [10] was adopted, using the available macroelement models ([11], [12]). Pushover analyses were performed to estimate the capacity in terms of maximum wall inter-storey drift. Out-of-plane failure modes were not considered, as strengthening interventions are supposed to be appropriately designed to prevent their activation.

The results shown in this paper refer to one of the case study structures analysed, located in Naples and representative of a typical typology of the late nineteenth century.

2 IDENTIFICATION OF THE SELECTED CASE STUDY

The case study discussed in this paper consists of a 5-storey unreinforced yellow tuff masonry structure, with flexible wooden floors and different wall thicknesses and inter-storey heights (Table 1).

Figure 1 (a) shows two photos of the existing building and the corresponding plans of the ground and upper floors (Figure 1b), whereas Figure 2 shows the elevations views, obtained from a slight simplification of the real situation.



(a) (b)

Figure 1: Photos of the considered existing building prototype (a) and corresponding plans (b), obtained from a simplification of the real building. X and Y indicate the directions of analysis.

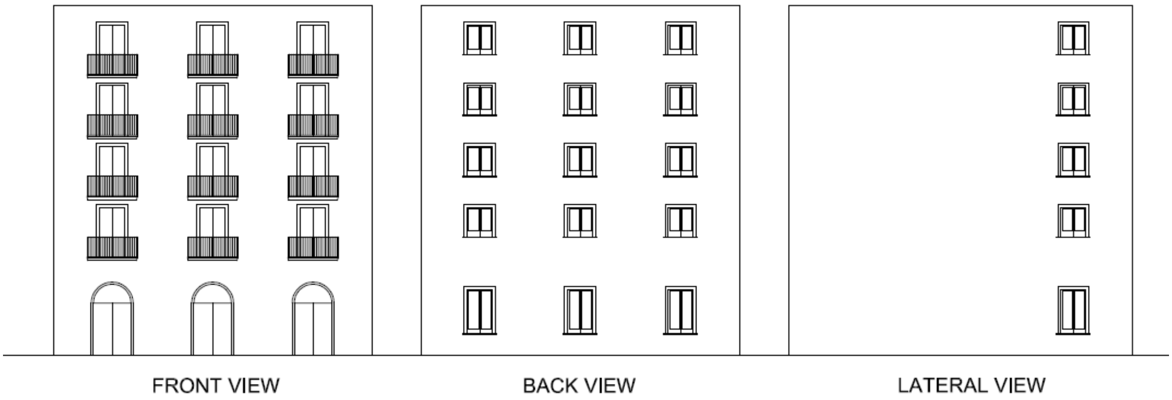


Figure 2: Elevations of the considered building, obtained from a simplification of the real building.

	Ground Floor	1st - 2nd Floor	3rd Floor	4th Floor
Wall thickness [m]	0.8	0.6	0.4	0.4
Inter-storey height [m]	4.5	2.7	2.7	3
Total Length [m]	15.2			
Total Height [m]	15.6			
Total Width [m]	16.4			

Table 1: Main geometrical properties of the different floors of the considered case study building

3 DESIGN OF RETROFIT SOLUTIONS USING DIFFERENT CODES

The retrofit interventions on the considered URM buildings were designed according to the "Code for repair and strengthening of buildings damaged by earthquakes," issued by the Ministry of Public Works in 1981 (D.M. 1981, [13]), and the associated guidelines [14], incorporating the POR method, originally proposed by Tomaževic [15] for the seismic analysis of retrofitted URM buildings. Such documents were published after the 1976 (Friuli), 1979 (Norcia) and 1980 (Irpinia) earthquakes and extensively used in the reconstruction phase. The procedures presented in this code were then included, with minor modifications, in the national seismic codes and largely implemented in the retrofit interventions designed in the last two decades of the 20th century.

Interventions were also alternatively designed according to the Italian building code issued in 2008 (NTC08, [17]) and its guidelines [18], which came into force after the 2009 L'Aquila earthquake. Eventually, retrofit interventions were assessed according to the code update published in 2018 (NTC18, [19]) and its guidelines [20].

3.1 D.M. 1981 (POR method)

The POR approach is an equivalent static, simplified nonlinear assessment method, proposed and developed by Tomaževič [15]. The method, which has undergone several refinements in the subsequent years, is based on the so-called "storey-mechanism" approach, which assumes that failure occurs only in the piers (shear failure), without any damage in the spandrels. Each masonry pier is modelled by a linear elastic-perfectly plastic shear spring, with limited ductility and effective height equal to the net height of the openings, as indicated in [14]. Alternative definitions of the effective height aiming at accounting for the orientation of cracks at the pier edges and for the deformability of spandrels were proposed for example by Dolce [16].

Applying the POR method, according to the guidelines in force at the time of construction of the building [14], the effects of the following seismic retrofit strategies were investigated and verified:

- realisation of a reinforced concrete ring beam at the storey levels;
- stiffening of the floors by casting collaborating reinforced concrete rigid slabs.

Figure 3 shows the definition of the equivalent frame mesh of two significant walls of the numerical model adopted for the analyses according to the POR method.

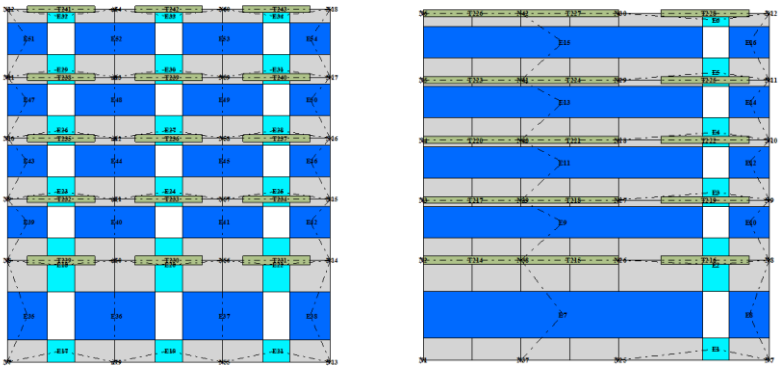


Figure 3: Equivalent frame mesh for two significant walls of the numerical model adopted in the analyses according to the POR method for the selected building.

The mechanical properties of masonry were assumed based on the values proposed in the guidelines [14] for tuff masonry. The adopted values are summarized in Table 2, in which f_m is the mean compressive strength of masonry, τ_0 is the mean shear strength of masonry, E and G the mean values of the elastic and shear moduli of elasticity and w is the masonry specific weight.

f_m [MPa]	τ_0 [MPa]	E [MPa]	G [MPa]	w [kN/m^3]
2.5	0.1	660	110	16

Table 2: Adopted mechanical properties of tuff masonry, assumed according to [14].

To be consistent with the assumptions of the POR method, the maximum shear strength of the structure was calculated under the hypothesis of fixed floor rotations, performing nonlinear static analysis (pushover), with a force distribution proportional to the first vibration mode. The seismic demand was computed as a percentage of the building weight, depending on the degree of seismicity of the considered area (Table 3).

Seismic Zone	**Building weight, W_{tot} [kN]**	**Seismic Force, F_t [kN]**	**F_t / W_{tot} [-]**
3	11489	1838	0.16

Table 3: Seismic forces computed according to [14].

Figure 4 shows the results of the analyses, in terms of shear strength normalized with respect to the weight of the structure (V/W_{tot}) versus top displacement. Since the strength in both the directions of analysis clearly exceeds the seismic demand (dashed red line), the adopted retrofit strategies satisfy the code requirements of the period.

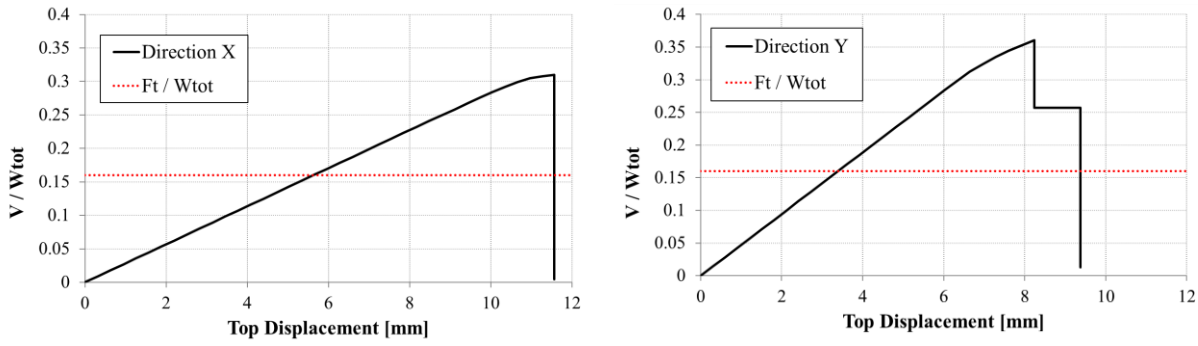


Figure 4: Results of the POR method, in terms of shear strength normalized to the weight of the structure versus top displacement: comparison between the strength of the structure in both the directions of analysis (longitudinal (left) and transversal (right)) and the seismic demand (dashed red line).

3.2 NTC08

For the same building prototype, different retrofit solutions were designed and analysed to fulfil the prescriptions of NTC08.

In particular, the effects of the following seismic retrofit strategies were investigated:

- stiffening of the floor diaphragm by adding a reinforced concrete collaborating slab, with lightweight concrete;
- improvement of the masonry quality by grout injections;
- enhancement of the wall-to-diaphragm connections by means of L-shaped steel ring beams at each floor level, except for reinforced concrete ring beams at the roof level;
- steel framing of all the openings of the longitudinal wall at the back (Figure 1b) by S275 HEB100 profiles.

These retrofit strategies were verified by applying the adopted equivalent-frame modelling strategy implemented in the 3Muri program and based on an elastic-perfectly plastic constitutive law of the structural element [10]. Figure 5 shows the 3D views of the numerical model of the retrofitted building.

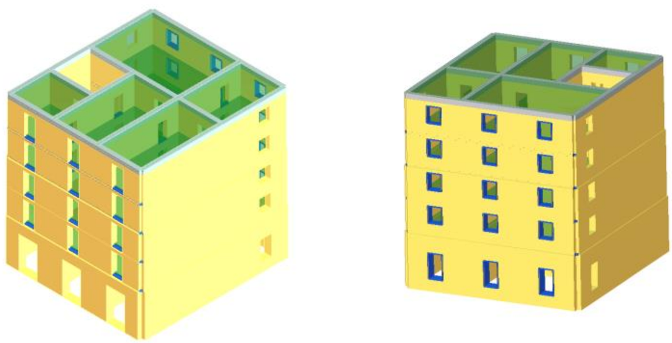


Figure 5: 3D views of the numerical models of the structures retrofitted according to NTC08.

The mechanical properties of masonry were defined based on the values proposed by the guidelines for NTC08 [18] for tuff masonry, assuming a confidence factor of 1.2 (knowledge level 2). The selected building was analysed in Naples, soil type A and C (topographic category T1). Figure 6 reports the pushover curve and damage pattern of one significant wall, obtained in one of the critical analyses (minimum capacity over demand ratio) carried out according to NTC08 (soil type C). A damage concentration at the ground storey can be observed, with shear failure and flexural damage of the masonry piers of the longitudinal wall at the back (the one with steel profiles around all the openings).

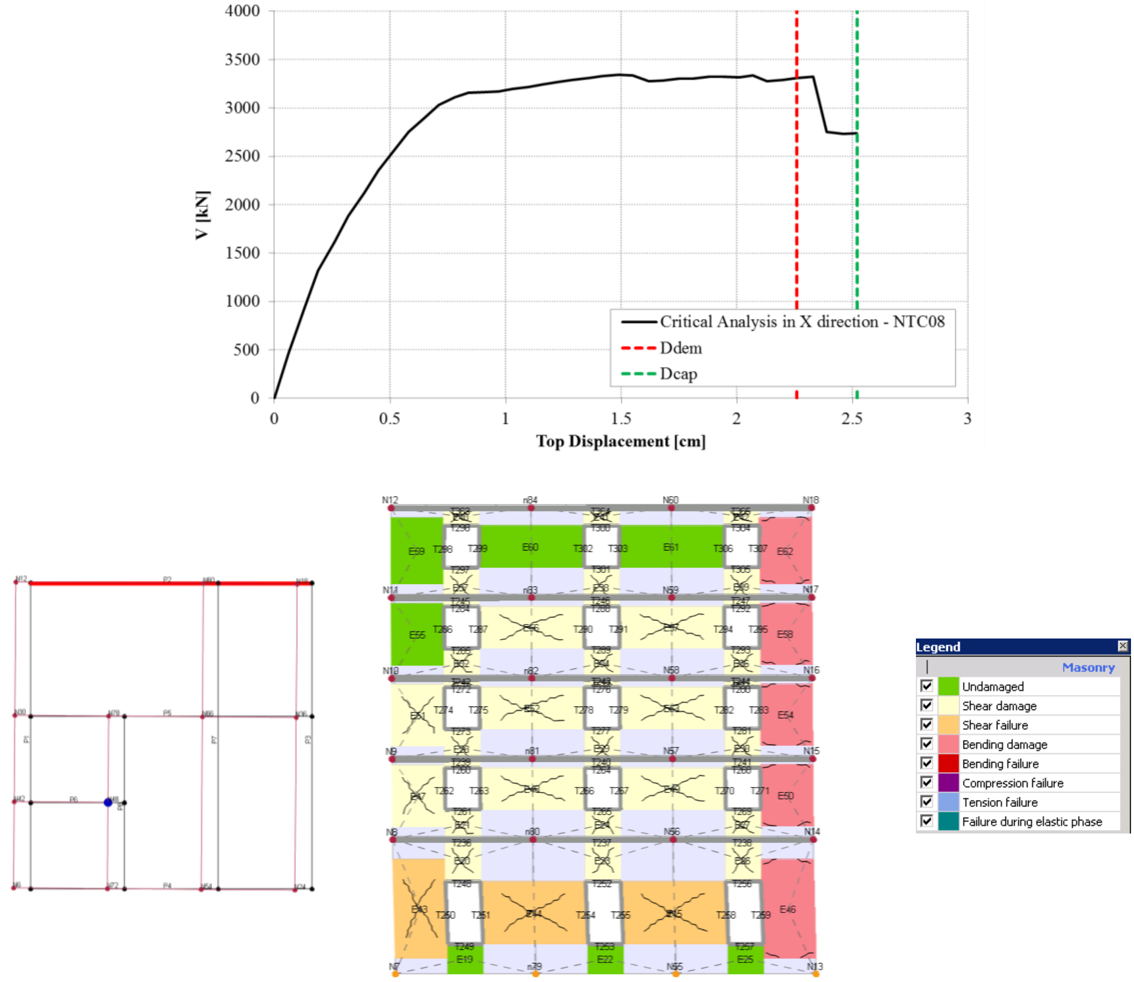


Figure 6: Results obtained for the critical analysis in the X direction (Figure 1), soil type C, for the building retrofitted according to NTC08: damage pattern of one significant wall (bottom) and pushover curve with indication of the demand (D_{dem}) and capacity (D_{cap}) displacement thresholds (top).

The retrofitted model was checked to fulfil also the prescriptions of NTC18 for both soil type A and C, for the two ultimate limit states of life safety and collapse. In particular, for the analyses carried out for soil type C, the maximum ratio between the maximum seismic capacity of the structure and the maximum seismic demand used for the design of new buildings (ζ_E) resulted equal to 90%, satisfying the requirements of the code ($\zeta_E \geq 80\%$).

4 SEISMIC PERFORMANCE OF RETROFITTED BUILDINGS

4.1 D.M. 1981

The seismic vulnerability of the retrofitted building was assessed by means of nonlinear static (pushover) and dynamic analyses, using the equivalent frame macroelement approach implemented in the computer program Tremuri [10] and adopting for the piers an improved version [21] of the macroelement proposed by Penna *et al.* [11].

The mechanical properties of the macroelement model were calibrated starting from experimental tests on masonry of similar characteristics [22], consisting of in-plane cyclic shear-compression tests carried out on specimens made of cement mortar and tuff units obtained from demolished buildings erected in Naples in the last two centuries.

Figure 7 reports the comparison between the experimental results (black curve) and the numerical simulation (red curve), obtained for two of the unreinforced masonry piers tested.

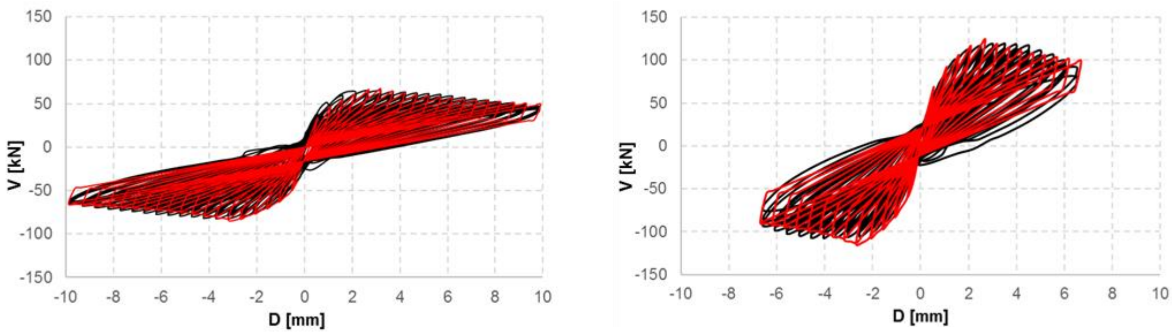


Figure 7: Comparison between the experimental results (black curve) and the numerical simulation (red curve), obtained for two of the unreinforced masonry piers tested by [22]: specimen T1-1 (left) and T1-3 (right).

Based on the comparison with experimental results, it was possible to calibrate only the parameters related to shear failure modes, since all experimental specimens failed in shear. The missing flexural parameters were derived from indications reported in the literature [24].

The seismic vulnerability was assessed in terms of frequency of exceedance of given thresholds of a selected Engineering Demand Parameter (EDP). As in [9], the maximum inter-storey drift (among all walls and all stories), accounting for the average rotations of the nodes of the storey, was selected as EDP. The EDP thresholds were computed by pushover analyses with an inverse triangular load pattern, considering the different behaviour of the building in the two directions of analysis, X and Y (positive and negative). The maximum inter-storey drift was evaluated separately for each direction and the EDP was calculated as the minimum value between the positive and negative direction for X and Y separately.

Two different limit states were considered: usability-preventing damage, UPD, evaluated according to the multi-criteria approach discussed in [1], and global collapse, GC, corresponding to a total base shear loss equal to 50% of the maximum base shear.

Figure 8 shows the pushover curves in the two directions of analysis, with the identification of the thresholds for the considered limit states (Table 4).

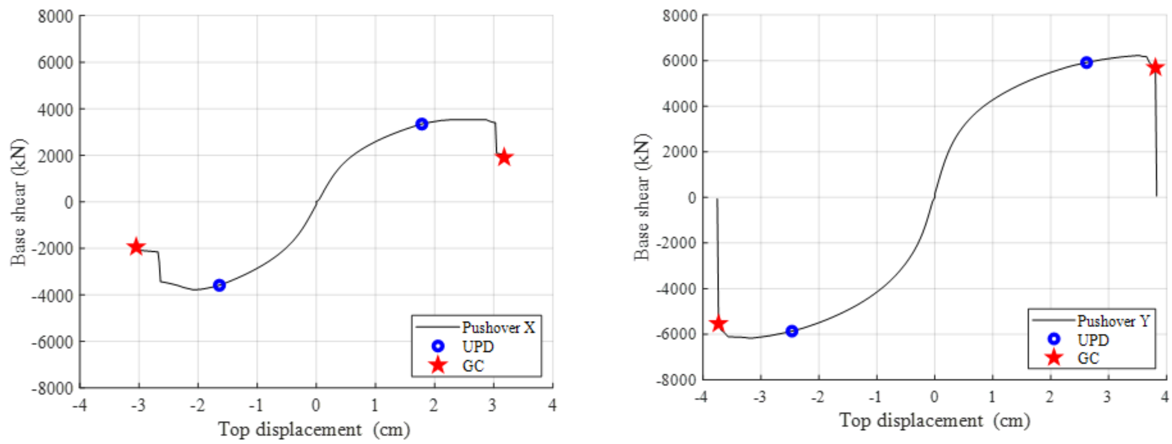


Figure 8: Pushover curves in the two directions of analysis for the building retrofitted according to D.M. 1981, with the identification of the thresholds for the considered limit states.

UPD threshold [%]		GC threshold [%]	
X direction	**Y direction**	**X direction**	**Y direction**
0.125	0.193	0.525	0.516

Table 4: Values of EDP thresholds corresponding to usability-preventing damage (UPD) and global collapse (GC) limit states, for the building retrofitted according to D.M. 1981.

The criterion governing the UPD limit state was the one associated with the attainment of 95% of the building lateral strength, corresponding to a condition in which the structure is still fully capable of withstanding horizontal forces. Figure 9 shows the damage in the structural elements at the end of analyses (GC limit state).

By means of nonlinear dynamic analyses, it was possible to calculate the percentage of exceedance of the two considered limit states, for increasing values of the return period. The time history analyses were carried out with a set of accelerograms (representative of the seismicity of Naples) selected for buildings with a fundamental period close to 0.25s, since the building fundamental vibration periods in undamaged conditions is about 0.28s in one direction and 0.2s in the other. A set of 20 ground motions was selected for each of the 10 return periods considered. The results in terms of frequency of exceedance of the two limit states are reported in Figure 10.

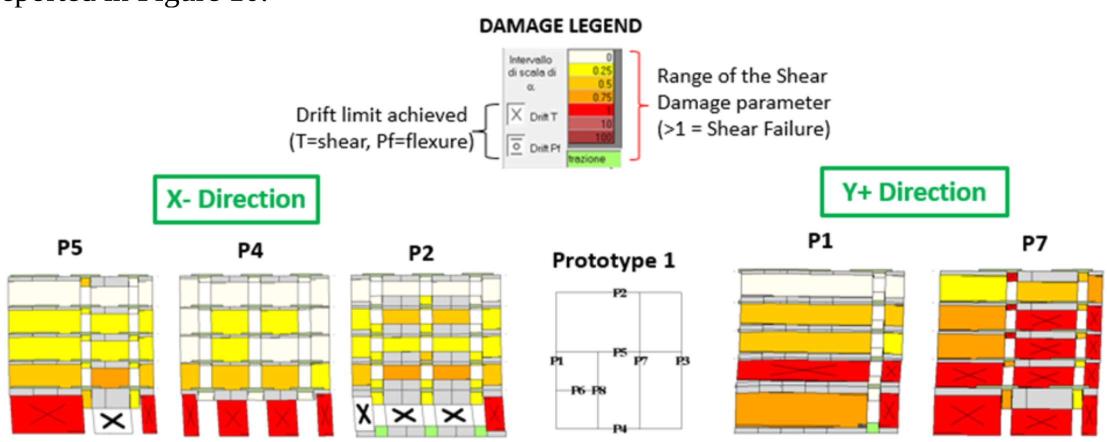


Figure 9: Damage in meaningful structural walls of the building retrofitted according to D.M. 1981 at the end of the pushover analyses carried out in the two directions of loading.

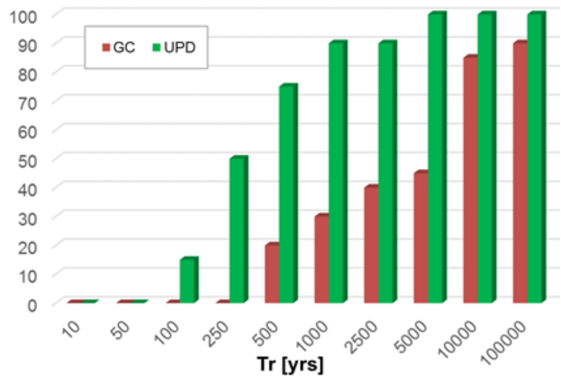


Figure 10: Frequency of occurrence (%) of UPD and GC limit states, as a function of the return period, for the building retrofitted according to D.M. 1981 (soil type C).

4.2 NTC08

The same approach described before was followed to assess the seismic performance of the building retrofitted according to NTC08. The mechanical properties of the macroelement were calibrated starting from the experimental campaign already considered for D.M. 1981, considering only the specimens strengthened by grout injections [22].

Figure 11 reports the comparison between the experimental results (black curve) and the numerical simulation (red curve), obtained for two of the considered walls.

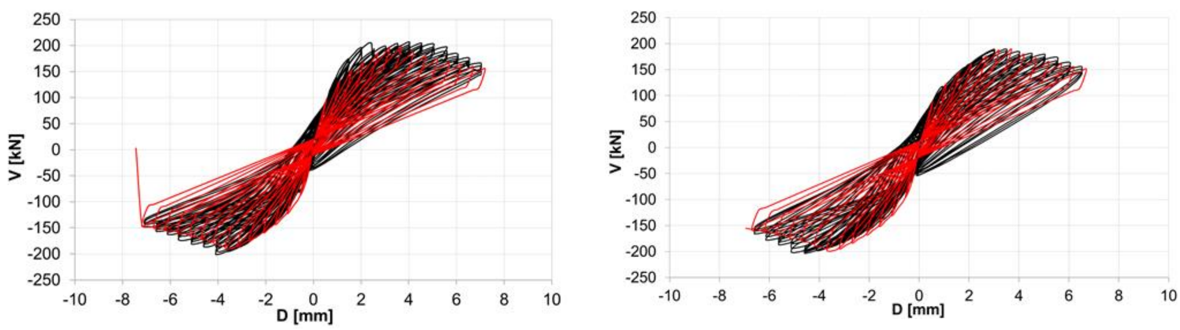


Figure 11: Comparison between the experimental results (black curve) and the numerical simulation (red curve), obtained for two of the retrofitted masonry piers tested by [22]: specimen T3-7 (left) and T3-9 (right).

The in-plane and out-of-plane stiffness of the floors was modelled assuming equivalent mechanical properties, calibrated on experimental shaking table tests on full scale stone masonry buildings with floor diaphragms and ring beams of similar characteristics [25].

Figure 12 shows the pushover curves obtained in the two directions of analysis, with the identification of the thresholds for the considered limit states (Table 5). The higher displacement capacity exhibited, with respect to the building retrofitted according to D.M. 1981 (Figure 8), is mostly due to the development of a global behaviour and failure mechanism different from what observed with the retrofit strategies designed in that case.

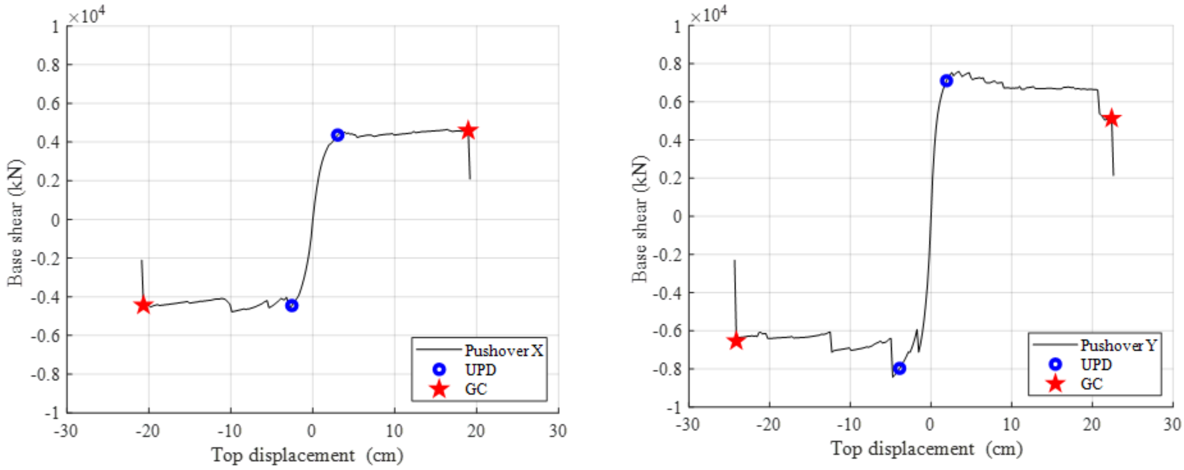


Figure 12: Pushover curves in the two directions of analysis, for the building retrofitted according to NTC08, with the identification of the thresholds of the considered limit states.

UPD threshold [%]		GC threshold [%]	
X direction	**Y direction**	**X direction**	**Y direction**
0.153	0.135	1.125	0.943

Table 5: Values of EDP threshold corresponding to the considered limit states for the building retrofitted according to NTC08

Also in this case, the dominant criterion identifying the attainment of UPD was associated with the attainment of 95% of the peak shear strength. The damage occurred in the structural elements at the end of the analyses (GC limit state) is reported in Figure 13.

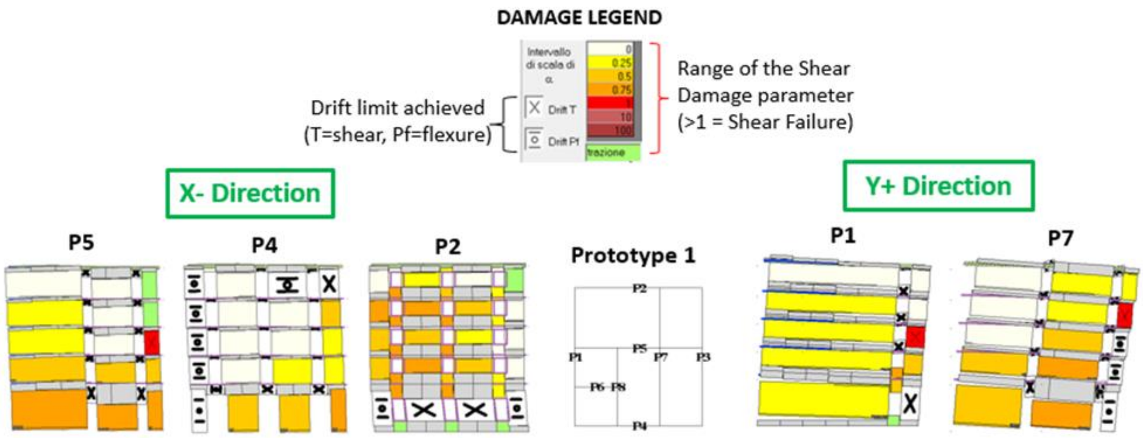


Figure 13: Damage observed in significant structural walls of the building retrofitted according to NTC08 at the end of the pushover analyses carried out in the two directions of loading.

Nonlinear time history analyses were then performed using the same two sets of ground motions of D.M. 1981, to obtain the percentage of exceedance of the two considered limit states (UPD and GC), for increasing values of the return period. Figure 14 shows the results for a reference period of 0.25 s and soil type C.

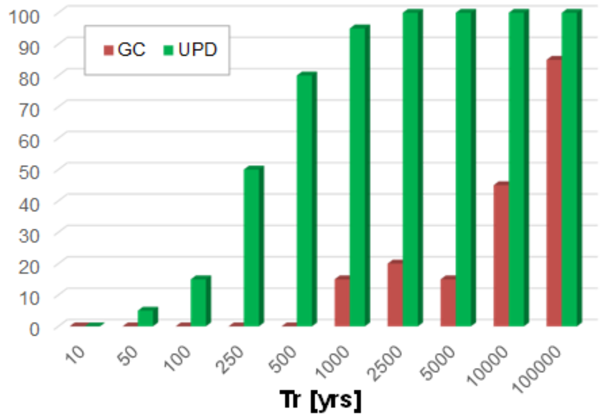


Figure 14: Frequency of occurrence (%) of UPD and GC limit states, as a function of the return period, for the building retrofitted according to NTC08.

5 DISCUSSION AND CONCLUSIONS

The assessment of the seismic performance of the building retrofitted according to the prescriptions of the two considered codes and associated hazard, allows to conclude that, for the considered case study, the vulnerability at the global collapse (GC) limit state is lower in the case of retrofit strategies designed according to NTC08, with respect to D.M. 1981. On the contrary, similar performances were obtained at the usability-preventing damage (UPD) limit state. This difference could be in part explained by the different global behaviour and failure mechanisms exhibited by the structure: a global rocking behaviour with failure of most of the spandrels occurred in case of NTC08, whereas the activation of an evident soft-storey mechanism, with shear failure of the piers, was observed in case of D.M. 1981. Observation of the global displacement capacity of the pushover curves confirms these results, showing in the two cases similar EDP thresholds for the usability preventing damage limit state and different thresholds for the global collapse limit state.

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