Seismic strengthening of masonry infills in RC frame buildings using Composite Textile Reinforced Mortar

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Abstract:

A real case study belonging to the class of reinforced concrete (RC) infilled buildings designed without anti-seismic prescriptions is examined in this paper. It is a 7-storey residential building erected in Italy in the late 1950s, with a pure RC frame structure and hollow masonry infills.
A seismic performance assessment study in current state is carried out via incremental dynamic analysis (IDA), referring to the set of nine hazard levels provided for by the Italian Technical Standards.
At a first step, the analyses are developed on the bare frame; then, infills are included in the structural model, simulated as equivalent non-linear strut elements.
The results highlight a better performance of the frame structure in infilled configuration and a significant vulnerability both of infills and RC members.
Based on these results, a retrofit strategy is proposed, consisting in the application of a thin layer of Composite Textile Reinforced Mortar on both sides of each infill panel, reinforced with a glass fiber net, which minimizes the added thickness, and thus impact and cost of the intervention.
The IDA analyses are replicated on the building in retrofitted conditions, showing a significant improvement in the seismic response of the infills and, consequently, also of the RC structure.

1 Introduction

Reinforced concrete (RC) frame structures with masonry infills represent one of the most widespread construction systems worldwide. In Italy, this typology was extensively adopted in the 1950s and 1960s for multi-storey residential buildings. At that time, they were typically designed for gravity loads only, as the first modern seismic design normative was released in Italy in 1974. The absence of dedicated lateral-load resisting systems, such as RC cores or shear walls, determines poor seismic performance of these buildings, with damage involving not only structural members but also masonry infills, as observed in past and recent earthquakes. This highlights the importance of investigating the seismic response of infill panels jointly with the RC frames, so as to identify possible vulnerability mechanisms and define effective retrofit strategies.

Among the available strengthening measures, techniques based on the use of composite materials have been increasingly adopted in recent years, owing to their mechanical properties, which guarantee significant seismic performance improvements while limiting both architectural impact and costs.

In this paper, a seismic performance assessment of a real case study building, representative of the stock of residential RC frame buildings constructed between the 1950s and 1960s, is carried out through incremental dynamic analysis (IDA), by referring to the full set of hazard levels defined by the Italian Standards [1][2].

The IDA analyses are performed on both the bare frame and the infilled structure, where masonry panels are modelled as equivalent diagonal struts. Based on the identified vulnerabilities, a retrofit strategy is proposed to enhance strength and stiffness of the infill panels. The intervention consists in applying a thin layer of Composite Textile Reinforced Mortar (CTRM), reinforced with a glass fiber mesh, on both faces of each panel. The IDA analyses are then extended to the retrofitted configuration, showing a remarkable improvement in the seismic response of the infill panels and, consequently, of the RC structure.

The following sections describe the case study building and its numerical modelling, and present the results of the IDA analyses for the bare-frame, infilled and retrofitted configurations, assessing both structural and non-structural performance.

2 Geometrical and structural characteristics of case study building

The case study building is located in the town of Tolmezzo, Friuli-Venezia Giulia, Italy. The plan has a rectangular shape, with dimensions of 20 m × 15 m, and a total above-ground height of 24.1 m. The structure includes 7 storeys – from the basement to the fifth level – extending over the entire plan area. The staircase continues to the accessible rooftop. Storey heights are equal to 3.42 m, except for the basement (3.8 m), ground (4.2 m) and fifth storeys (3.62 m).

The structural plan of the ground storey, including the X and Y axes of the Cartesian reference system adopted in the analyses (with Z as the vertical axis), and the south and east elevation views of the building are shown in Figure 1a, 1b and 1c, respectively.

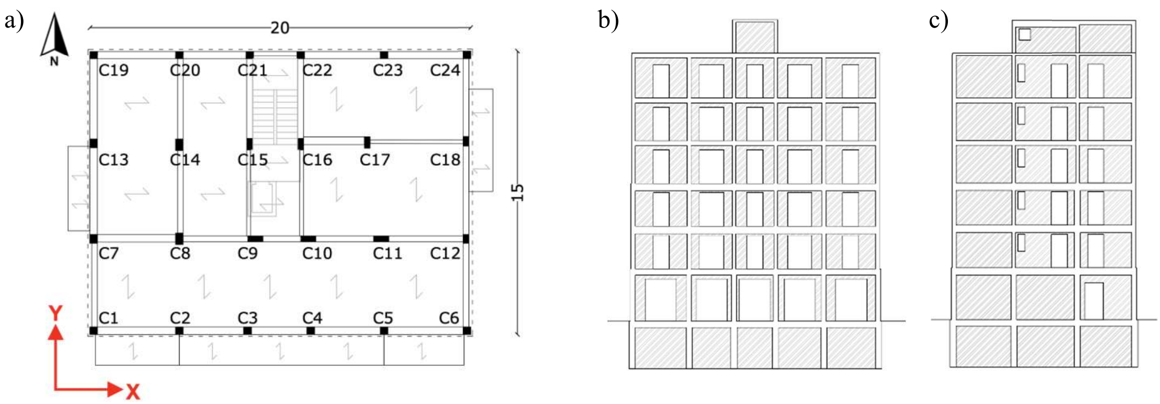


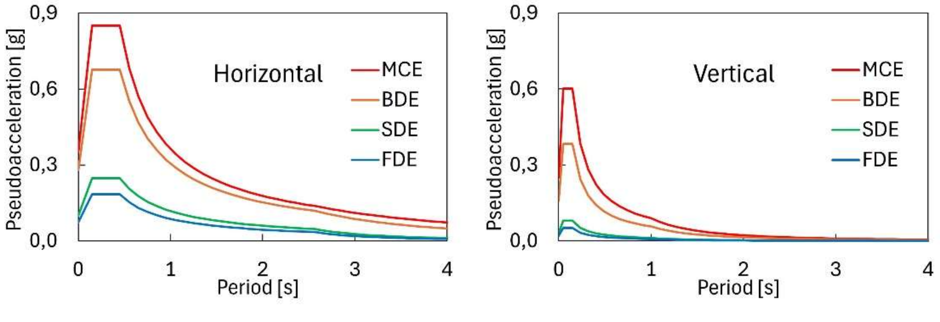
Figure 1. a) Ground floor structural plan, b) south and c) east elevation views of case study building.

The RC structure consists of a three-dimensional skeleton, with the main frames not aligned along a single principal direction in plan. Columns are tapered along the building height. Perimeter

columns have square cross section with sides of 0.5 m × 0.5 m on the basement and ground storeys, 0.4 m × 0.4 m on the first storey, and 0.3 m × 0.3 m on the second through fifth storeys. Internal columns have rectangular cross sections, with dimensions ranging from 0.25 m × 0.6 m to 0.4 m × 0.8 m; columns C9, C10 and C11 (according to the nomenclature in Fig.1a) have the longest side parallel to X, and the remaining columns to Y. Perimeter beams are sized 0.4 m × 0.5 m on the basement and ground storeys, 0.3 m × 0.6 m on the first and second storeys, and 0.3 m × 0.4 m on the third, fourth and fifth storeys. Internal beams exhibit variable section dimensions, with widths ranging from 0.25 m to 0.4 m and depths from 0.4 m to 0.6 m, except for the top storey, where the section is sized 0.7 m × 0.2 m. The staircase is supported by pairs of knee beams spanning between columns C15 and C21, and C16 and C22.
Floors consist of partially prefabricated RC joists with hollow clay bricks and a 40 mm-thick cast-in RC slab, resulting in a total thickness of 200 mm. The RC joists are predominantly oriented along Y and, in some spans, along X. According to [1], the slab provides diaphragm action, ensuring sufficient in-plane stiffness of the floors under seismic action.
The RC frame is infilled with two 80 mm-thick unconnected layers of hollow clay masonry bricks.

3 Seismic assessment analyses of case study building in current state

The seismic performance of the building in its current state is assessed for both the bare frame and the infilled configuration via time-history incremental dynamic analysis. The latter is performed for the nine increasing seismic hazard levels assumed by the Italian Standards [1][2], with the aim of capturing the progressive development of damage in the infill panels. These hazard levels correspond to probabilities of exceedance, P_{VR}, over the reference lifespan, V_R, equal to 50 years, given by P_{VR}/V_R ratios equal to 81% (Frequent Design Earthquake, FDE), 63% (Serviceability Design Earthquake, SDE), 50%, 39%, 30%, 22%, 10% (Basic Design Earthquake, BDE), 5% (Maximum Considered Earthquake, MCE), and 2%.
For each hazard level, seven time-history analyses are carried out. In each analysis, a set of three accelerograms is simultaneously applied along the X, Y and Z directions. The accelerograms are generated to match the normative pseudo-acceleration response spectra of the site in which the building is located. The horizontal and vertical spectra referred to the four hazard levels basically assumed in design and assessment analyses (FED, SDE, BDE and MCE) are shown in Figure 2, along with a table listing the values of the peak ground acceleration, a_g, corresponding to all nine levels, normalized to the acceleration of gravity, g.



P_{VR}/V_R	a_g/g
81%	0.061
63%	0.083
50%	0.100
39%	0.119
30%	0.139
22%	0.164
10%	0.238
5%	0.322
2%	0.461

Figure 2. Elastic pseudo-acceleration response spectra and relevant normalized peak ground acceleration values.

3.1 Bare frame structure

The finite element model is generated using $SAP2000_{NL}$ calculus program [3]. Columns and beams are modelled by *frame*-type elements. Plastic hinges are assigned to these elements, with moment–rotation relationships based on the yielding and ultimate moment and rotation capacities of the RC member sections [1][2], and isotropic hysteretic behaviour. Rigid diaphragms are introduced at each storey level to represent the in-plane stiffness of floors. Two views of the finite element model are shown in Figure 3.

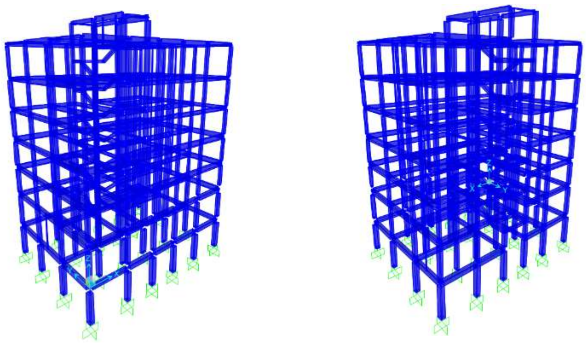


Figure 3. South-west and north-east views of the finite element model of the bare frame structure.

The first mode, with vibration period T_1 = 1.16 s, is mixed translational in Y – rotational around Z, with an effective modal mass (EMM) equal to 58% of the seismic mass of the structure in Y, and 21% around Z. The second mode, with period T_2 = 1.05 s, is mixed translational in X and Y – rotational around Z, with EMMs equal to 40% (X), 13 % (Y), 25% (Z). The third mode (T_3 = 0.98 s) is prevailingly translational in X (EMM = 39%), with a minor contribution of the rotational component (EMM = 32%). A total of five modes is required to reach a summed modal mass (SMM) exceeding 85% along X and Y, and around Z.

For the sake of brevity, the results of the incremental dynamic analysis are extrapolated here only for the BDE level. The maximum base shear is equal to 1,459 kN in X (V_X) and 2,933 kN in Y (V_Y). The peak inter-storey drift ratio (IDR), adopted as response parameter to evaluate the lateral deformation demand, reaches peak values of 1.72% in X (first storey), and 1.52% in Y (second storey) along the C6 vertical alignment, taken as a reference for the IDR control.

The response of the plastic hinges incorporated at the end sections of columns and beams is schematically represented in the four elevation views of the building drawn in Figure 4. Hinges not exceeding the yielding rotation (θ_y) are highlighted by green markers, whereas yellow markers denote hinges exceeding the yielding limit but not achieving the ultimate rotation capacity (θ_u). Red markers identify hinges exceeding θ_u, corresponding to local failure conditions. As visualized by these images, the response of the bare frame is characterized by a widespread development of ductile mechanisms associated with plastic hinge formation. Approximately 18% of the hinges exceeds the yielding rotation capacity, while about 6% reaches the ultimate rotation capacity.

3.1 Infilled structure

The in-plane behaviour of the infills is simulated by means of a trilinear force-displacement curve, shown in Figure 5, which interpolates the response of a panel subjected to cyclic horizontal loading. The maximum lateral resisting force, F_{max}, is evaluated by applying the strength evaluation criteria by Bertholdi et al. [4], while reduction factors accounting for the presence of openings are introduced according to the analytical expressions of Decanini et al. [5].

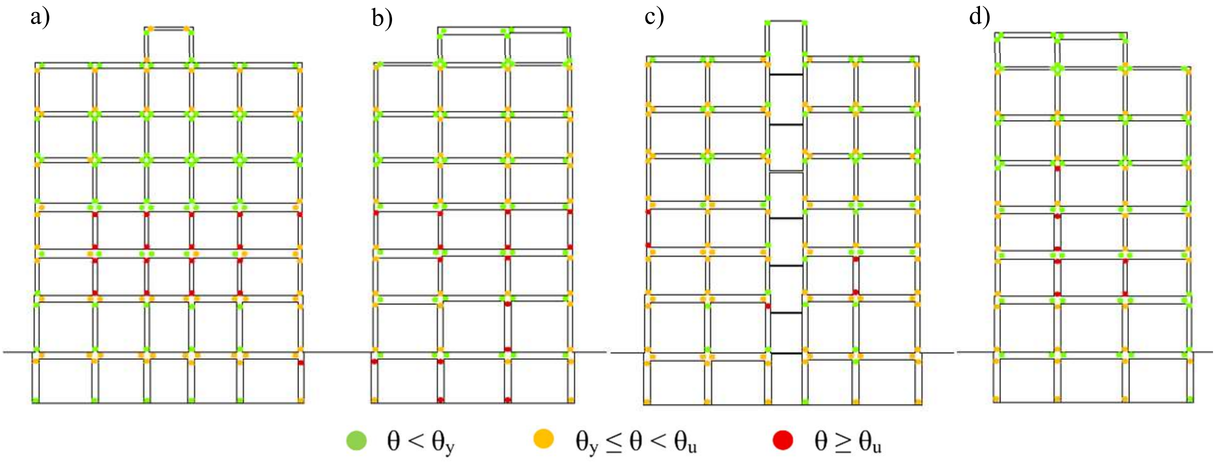


Figure 4. Plastic hinges response of the bare frame structure in current state at the BDE: a) south, b) east, c) north and d) west elevation views.

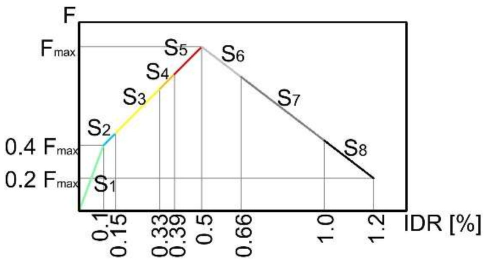


Figure 5. Backbone curve assumed in the computational analyses of infills.

The backbone curve is discretized into eight segments limited by the IDR performance thresholds defined in [6], in order to capture the progressive evolution of damage. In particular, for IDR values below 0.1% the panels are assumed to remain elastic, while damage progressively develops for higher drift levels and becomes severe in the range between 0.39% and 0.5%. At IDR = 0.5%, the peak lateral resistance is reached, followed by a strength degradation branch leading to collapse in the IDR = 1%÷1.2% range.

The infills are simulated into the finite element model of the structure by means of pairs of equivalent crossed compression-only resisting diagonal struts, implemented through *non-linear link* elements. Elevation views of the finite element model of the infilled structure are shown in Figure 6. The characteristic parameters describing the behaviour of the masonry panels are converted into equivalent axial properties of the non-linear struts using basic geometric transformations. The hysteretic behaviour is reproduced by means of a multilinear no-tension pivot-type model, as detailed in [6].

The incorporation of the infills significantly modifies the dynamic and seismic response of the structure compared to the bare frame. More specifically, the stiffening effect of the infills leads to a more regular dynamic behaviour, with a reduction in the vibration periods and a decoupling of the first modes. In particular, the first mode becomes predominantly translational in Y (T_1 = 1.02

s, EMM = 75%), while the second mode is mainly translational in X (T_2 = 0.93 s, EMM = 80%). The third mode is predominantly rotational around Z (T_3 = 0.84 s, EMM = 75%).

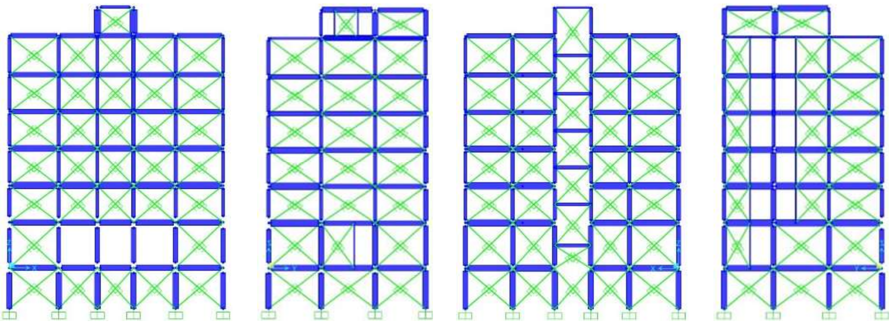


Figure 6. Elevation views of the finite element model of the infilled structure.

IDA results show that the increase in stiffness results in higher seismic demand in terms of base shear, reaching 1,990 kN in X (+36% with respect to the bare frame) and 3,455 kN in Y (+18%) at the BDE. Conversely, the lateral deformation demand is reduced, with peak inter-storey drift ratios decreasing to 1.62% in X (-5.8% with respect to the bare frame) and 1.34% in Y (-11.8%). The presence of infills is also reflected in the plastic hinge response of beams and columns, as shown in Figure 7. At the BDE level, approximately 88% of hinges remain elastic (+14% with respect to the bare frame), 8% exceed the yielding rotation, and 4% achieve the ultimate rotation capacity. The latter are mainly concentrated in the columns of the first and second storeys, highlighting the potential development of soft-storey mechanisms at these two levels.

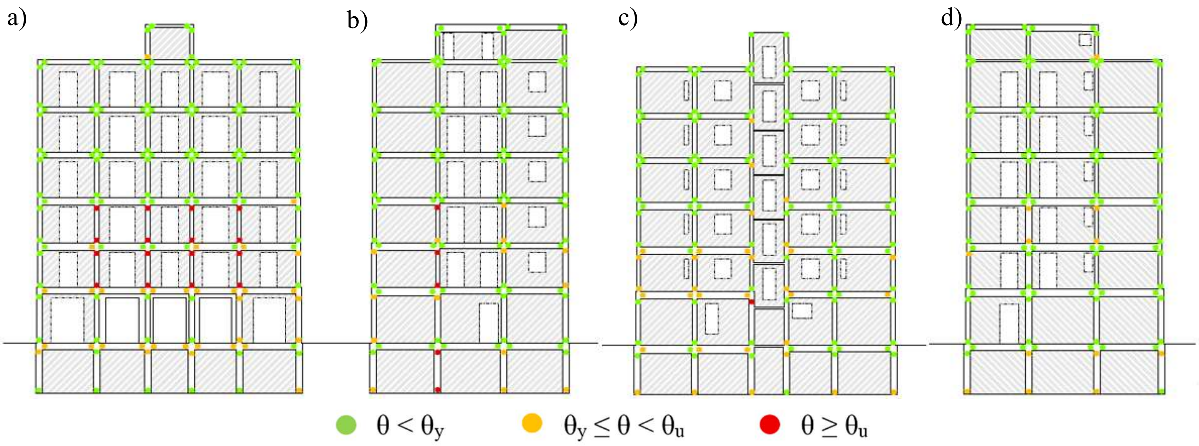


Figure 7. Plastic hinges response of the infilled frame in current state at the BDE: a) south, b) east, c) north and d) west elevations views.

The performance of the infills is summarized by colour maps identifying their damage state at the nine hazard levels, according to the chromatic scale of the backbone curve adopted in Figure 5. As representative examples, the response of the panels at the hazard levels identified by P_{VR}/V_R = 63% (SDE), 50%, 22% and 10% (BDE) are shown in Figure 8. As highlighted by these drawings, four panels (filled in red) already exhibit severe damage conditions at the SDE, with IDR values ranging from 0.39% to 0.5%. The peak lateral resistance is attained for the first time at P_{VR}/V_R =

50%. As seismic intensity further increases, the first collapse mechanisms develop at P_{VR}/V_R = 22%, with some panels (black) exceeding IDR values of 1%, while approximately 50% of the infills already exceed the 0.5% threshold (grey).

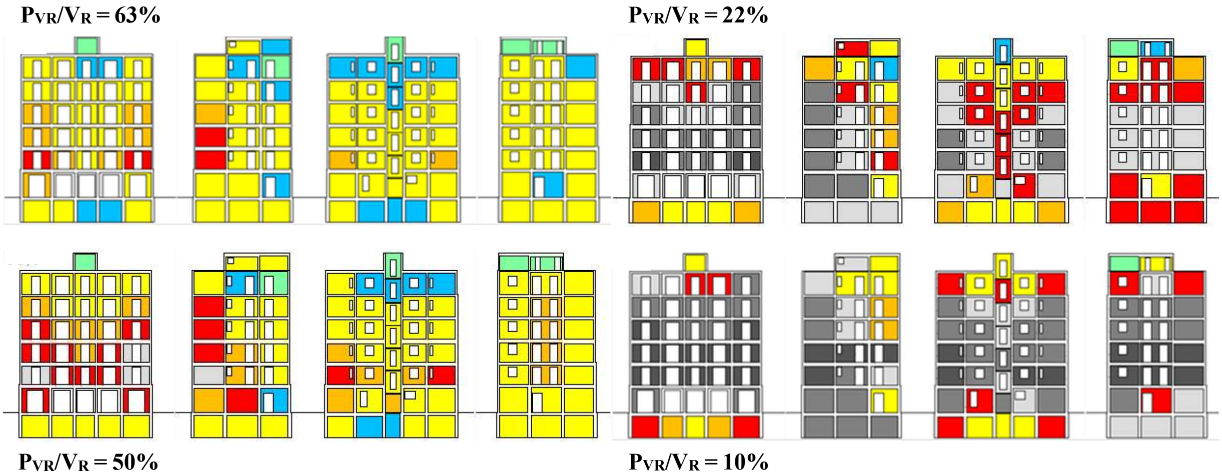


Figure 8. Colour maps assessing the response of the infills at the hazard levels with P_{VR}/V_R equal to 63%, 50%, 22% and 10% in current state.

At the BDE, damage becomes more pronounced, with over 70% of the infills exceeding the IDR limit of 0.5% and over 20% of the panels, mainly located on the first and second storeys, exceeding 1%, which identifies generalized collapse conditions. These results underline the opportunity of considering intermediate seismic intensity levels in the assessment of non-structural vulnerability, as severe damage is already observed at the SDE, and collapse mechanisms develop at P_{VR}/V_R = 22%, before the BDE.

4 Composite Textile Reinforced Mortar-based retrofit of infills

4.1 Retrofit intervention and numerical modelling

Based on the results obtained for the masonry infill panels in current conditions, a retrofit strategy is developed to enhance their seismic performance and, consequently, the overall structural response of the building, while limiting the architectural impact of the intervention.

The proposed solution consists in the application of a CTRM system, which includes a cementitious matrix mortar reinforced with a glass fiber net, with a grid spacing of 25 mm × 25 mm and thickness of 0.05 mm, on both faces of each infill panel. The total thickness of the strengthening layer is 20 mm, including a 5 mm base mortar coat applied on the masonry substrate, followed by mesh placement and a 15 mm covering mortar layer. This small total thickness is comparable to the one of a conventional plaster finishing layer, enabling a reduction in the intervention cost and causing no aesthetical impact. A schematic drawing of a CTRM-based intervention is shown in Figure 9, along with a photographic image taken during its installation.

The effects of the CTRM technology on the in-plane behaviour of the infills are derived from the experimental campaign conducted by Akhoundi et al. [7]. Based on these results, the enhancement in lateral capacity induced by the strengthening intervention is quantified by a 290% increase in the elastic limit force, a 120% increase in the peak lateral force, and a 90% increase in the ultimate force corresponding to an IDR of 1.2%.

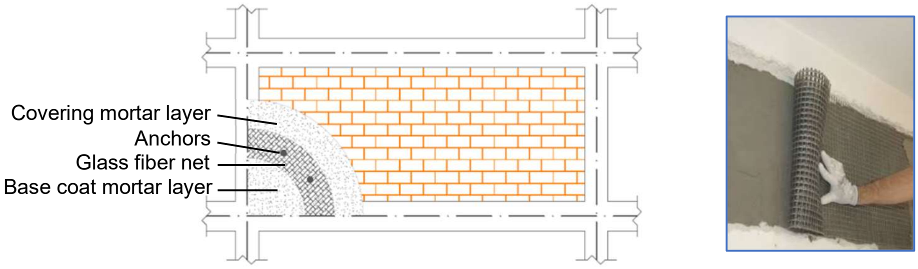


Figure 9. Scheme and photographic image of the CTRM system installation.

The retrofit intervention is simulated within the equivalent diagonal strut model of infills. The effect of the CTRM system is accounted for by updating the mechanical properties of the panels to reflect the above-mentioned increases in the elastic limit, peak lateral and ultimate force values. This is achieved by scaling the force–displacement backbone curves (Figure 5) according to the corresponding amplifying factors.
IDA analyses are repeated to evaluate the effects of the proposed retrofit on both non-structural and structural response.

4.2 Seismic response in retrofitted configuration

The application of the CTRM system significantly modifies the structural response observed in the presence of the infills, leading to further increase in global stiffness – in addition to the increase observed in passing from the bare frame to the infilled structure – and a consequent reduction in vibration periods. In particular, the first three periods pass from the T_1 = 1.02 s, T_2 = 0.93 s, T_3 = 0.84 s values for the infilled structure, to T_1 = 0.72 s, T_2 = 0.68 s, T_3 = 0.49 s in retrofitted conditions.
At the BDE level, the base shear demand rises by 21% in X (V_X = 2,412 kN) and 6% in Y (V_Y = 3,657 kN), whereas the IDR demand decreases by 31% (1.11% against 1.62%) in X, and 60% (0.54% against 1.34%) in Y. The effects on the lateral deformation demand are visualized in Figure 10, where the maximum absolute horizontal displacement profiles of the C6 vertical alignment in X and Y are plotted for the bare (black), infilled (red) and retrofitted (green) structure. The latter displays reductions of 38% and 46% in X and Y, respectively, as compared to the infilled model in current state. These drops provide a comprehensive measure of the reduced seismic demand on the non-structural elements.
In terms of structural performance, the beneficial effects of the retrofit intervention are reflected in the plastic demand on the RC members. Up to the hazard level with P_{VR}/V_R = 22%, the response of all plastic hinges is constrained within the elastic range. At the BDE level, more than 91% of the hinges remain elastic, while less than 8% exceed the yielding rotation and less than 1% reach the ultimate rotation capacity. By comparing this performance to the current state one, where up to 4% of the hinges reaches ultimate conditions and damage is concentrated in the lower storeys, the retrofitted configuration substantially mitigates the development of local failure mechanisms. These results underline that, although the retrofit measures are limited to the infill panels, they

lead to an overall improvement in the seismic performance of the building. Nevertheless, a limited number of first-storey columns still achieve the ultimate rotation capacity, which suggests to adopt additional local strengthening measures on these members, in order to completely eliminate potential collapse mechanisms.

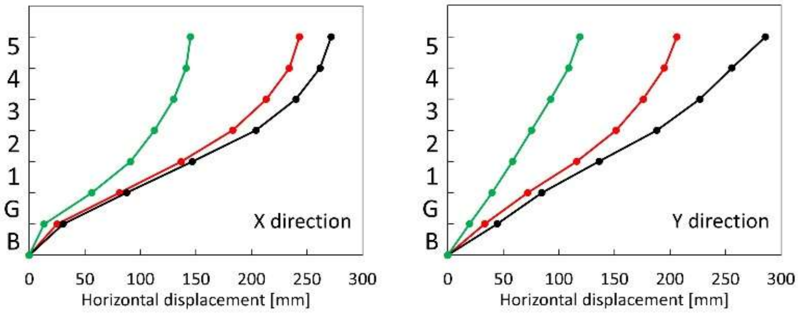


Figure 10. Maximum absolute horizontal displacements of the C6 vertical alignment for the bare frame (black), infilled frame (red) and CTRM-retrofitted configuration (green).

Figure 11 duplicates in post-intervention conditions the colour maps representing the damage state of infills drawn in Figure 8 for the current state.

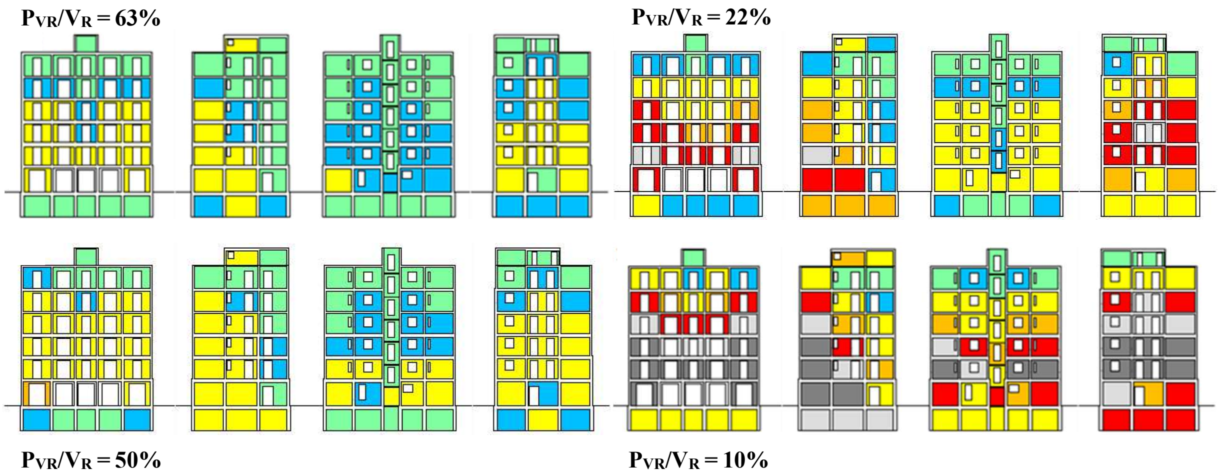


Figure 11. Colour maps assessing the response of the infills at the hazard levels with P_{VR}/V_R equal to 63%, 50%, 22% and 10% in retrofitted configuration.

At the SDE and for P_{VR}/V_R = 50%, the infill panels exhibit a predominantly elastic response, with IDRs below 0.33%. For P_{VR}/V_R = 22%, a small number of panels reach the peak lateral resistance; however, collapse mechanisms are not yet activated. At the BDE, although several panels experience severe damage, none of them reach the collapse threshold (IDR = 1%).
By comparing these data to the corresponding ones in current state, where 20% of the infill panels is found to achieve the collapse limit at the BDE (Figure 8), the retrofit intervention allows to prevent collapse activation under the same hazard level.
This results in a remarkable reduction in damage, at the same time complying with the BDE performance objective postulated by the Italian Standards, i.e. keeping life-safety conditions even in the presence of severe non-structural damage.

5 Conclusions

This paper investigated the seismic performance of a residential building representative of Italian construction practice in the 1950s and 1960s. The assessment analysis was carried out considering nine increasing hazard levels defined by the Italian Standards, in order to capture the progressive development of damage in both structural members and infill panels.

The results of the analysis highlighted a significant seismic vulnerability in current state. Both non-structural and structural elements suffered severe damage at intermediate intensity levels, with first collapses in infill panels and yielding of RC members starting from $P_{VR}/V_R = 22\%$. At the BDE level, 20% of the infills reached collapse conditions, while about 4% of the plastic hinges exceeded their ultimate rotation capacity, leading to fail the Life Safety performance objective.

This poor performance prompted to adopt a retrofit strategy based on the application of Composite Textile Reinforced Mortar reinforced with a glass fiber net system on both faces of the infill panels. The intervention was characterised by a negligible architectural and aesthetical impact due to its very limited thickness, comparable to conventional plaster finishing layers.

The results obtained for the retrofitted configuration demonstrated that the proposed intervention effectively prevents collapse of the infill panels up to the BDE hazard level. Although applied exclusively to non-structural elements, the strengthening action of the CTRM technology also produced beneficial effects on the global structural response, reducing the percentage of plastic hinges reaching the ultimate rotation capacity to less than 1%, and preventing the development of the soft-storey mechanisms observed in the current state.

6 References

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