Timber-based seismic retrofit of a 5-storey RC structure built in 1955

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ABSTRACT

The paper presents a study on the seismic behaviour of a 1955 masonry-infilled reinforced concrete (RC) building subjected to two alternative timber-based interventions to reduce its seismic vulnerability. The first intervention (named RC-TPext) consists in the application of cross-laminated timber (CLT) panels from the outside of the building, while the second (RC-TP) sees the replacement of the outer masonry wythes with CLT panels. Various building configurations were analysed: i) as-built, with the masonry infills included solely as mass; ii) as-built, with the infills modelled as equivalent diagonal struts; iii) retrofitted with the RC-TPext intervention; iv) retrofitted with the RC-TP intervention. Each configuration was analysed by making two alternative hypotheses about the properties of the existing material. The numerical analyses confirmed the importance of considering the masonry infills in the evaluation of the as-built seismic response, and proved the effectiveness of the proposed retrofit interventions. The results showed that the proposed timber-based retrofits can markedly improve the seismic behaviour of existing buildings characterised by poor material properties and construction detailing.

1 INTRODUCTION

Throughout the 1900s, concrete frame buildings spread widely in many countries around the World, becoming one of the most common building types in many areas. However, a substantial part of these buildings was built with design procedures and construction techniques that are nowadays considered obsolete or even detrimental to the building safety (e.g., by considering only the vertical loads in the design phase) (Masi 2003). In addition, the common practice of underestimating the role of infill walls (both in terms of local and global effects) has often led to incorrect interpretations of the seismic behaviour of these structures (Smith 1962). In particular, in the presence of seismic actions, it is not rare to observe brittle collapse of columns or activation of soft-storey mechanisms as a consequence of the interaction between structural elements and masonry infills (Hashemi & Mosalam, 2007; Ning et al., 2019). Over the last few decades, various intervention systems have therefore been developed with the aim of reducing such vulnerabilities (Di Lorenzo et al., 2020; Gkournelos et al., 2021). More recently, the use of timber-based interventions has become an increasingly investigated topic (Sustersic & Dujic 2012; Stazi et al., 2019). The present study falls within the framework of a broader research project on an intervention technique that sees the application of CLT structural panels to the existing structural frames (Smiroldo et al., 2020, 2021a-c, 2022). The intervention can be performed either by removing one or more of the existing masonry wythes (an approach called RC-TP) or by maintaining the infills (RC-TPext).

The work presented here is in continuity with a published study (Smiroldo et al., 2021a), where a case study structure built in the 1950s and alternatively subjected to RC-TP and RC-TPext interventions was analysed in terms of local seismic response, energy consumption, and thermal behaviour. In this study, the seismic behaviour of the existing structure before and after the application of the alternative retrofit interventions was evaluated through numerical analyses of the entire building. The response of the as-built configuration was evaluated using: i) a model in which the infills were considered only in terms of mass; ii) a model in which the infills were modelled as diagonal struts. The retrofit approaches were simulated with a modelling strategy derived from previous studies (Smiroldo et al., 2020, 2021a-c). The application of interventions RC-TP and RC-TPext has led to clear improvements in the local and global response of the analysed building, reducing the seismic vulnerabilities of the existing structure.

2 CASE STUDY

The case study building (Figure 1) is located in Turin (Italy) and was built in 1955. The building, with a rectangular plan of 30.10×12.20 m, consists of an underground basement used as a garage, five storeys above ground used for commercial activities and accommodation, and an unusable attic. The inter-storey height is equal to 3.25 m. The building has a multi-storey frame structural scheme with spans of variable length made of RC beams and columns, and floors made of RC joists and hollow clay blocks. Along the building's transverse direction, four RC shear walls extend from the ground to the roof. Following a visual inspection and a literature survey (Masi et al., 2015), masonry infills were assumed as cavity walls with an internal wythe made of hollow clay blocks and an external wythe made of solid clay bricks.

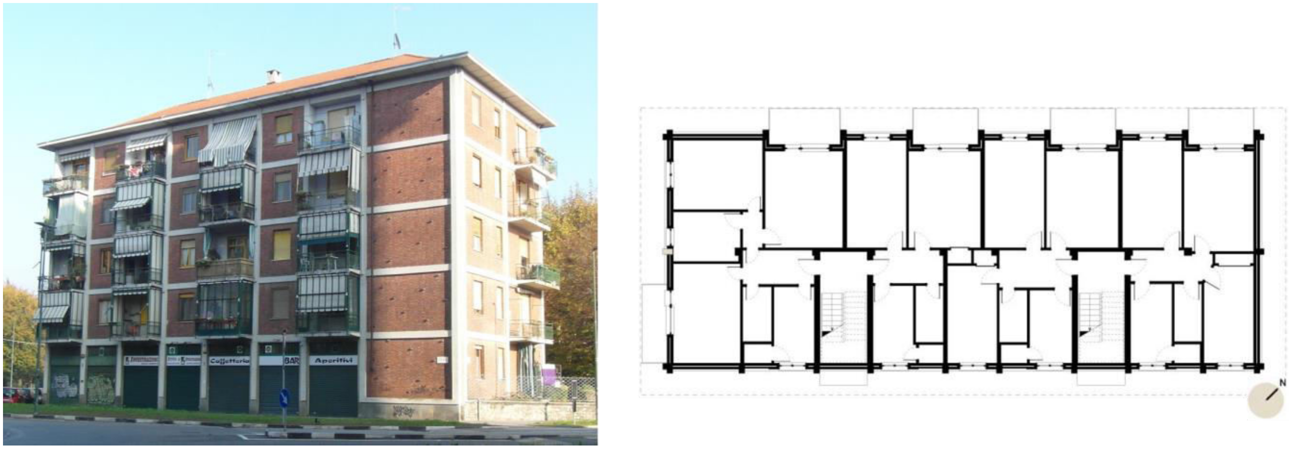

Figure 1: Case study structure

Regarding the mechanical properties of concrete and steel, two alternative hypotheses were considered. In the first hypothesis (HpA), the mechanical properties were determined from the test certificates of the construction period. In the second hypothesis (HpB), the selected materials properties represent the most probable values for a 1950s structure based on data literature (Masi and Vona, 2007; Tanganelli et al., 2011; Ricci et al., 2011; Cristofaro et al., 2012). For both hypotheses, the properties of the masonry wythes were determined based on the indications provided by the Italian code (NTC 2018 and *Circolare* 2019) and information found in the literature (Carnal 2006; Liberatore et. Al., 2018). Knowledge levels KL2 and KL3, as per Eurocode 8-Part 3, were associated with HpA and HpB, respectively. The details of HpA and HpB are reported in Table 1.

Table 1: Characteristics of the analysed structures in the two alternative hypotheses (HpA, HpB)

Characteristic/Material/Element	Description	Hp A	Hp B
Knowledge level	Eurocode 8-Part 3	KL2 (CF=1.2)	KL3 (CF=1)
Concrete*	Compressive strength f_{cm} [MPa]	26,85	12,32
	Elastic modulus E_{cm} [MPa]	29586,92	23421,00
Steel*	Yielding strength f_{ym} [MPa]	336,69	343,60
Solid bricks-hollow blocks masonry wythe*	Compressive strength f_m [MPa]	3,45-1,70	
	Shear strength (diagonal cracking) τ_0 [MPa]	0,09-0,05	
	Shear strength (bed joints sliding) f_{v0} [MPa]	0,20-0,13	
	Elastic modulus E_m [MPa]	1500-1150	
	Shear modulus G_m [MPa]	500-460	
	Weight W_m [kN/m^3]	18-8	

**mean values without confidence factor (CF) coefficients*

3 RETROFIT INTERVENTIONS

Two alternative retrofit methods with different levels of invasiveness have been previously proposed by the authors. Both methods are timber-based and see the use of CLT panels. The reason for that is to promote the adoption of sustainable/renovable materials, such as wood, in integrated strengthening interventions. The energy performance of such methods has been described in detail by Smiroldo et al. (2021a). This paper addresses the seismic aspects exclusively. Specifically, the structural components of the as-built configuration and the characteristics of the proposed retrofit strategies are described in Figure 2. The least invasive intervention is named RC-TPext (Reinforced Concrete-external Timber Panels), while the most invasive is named RC-TP (Reinforced Concrete-Timber Panels). Both retrofit strategies entail using timber structural panels made of CLT connected to the existing RC elements on the structure's perimeter. The main objective is to improve the seismic response in-plane and also out-of-plane. With RC-TP, the external masonry wythe (made of solid bricks) is removed and replaced with a CLT panel. The timber panel is inserted inside the frame and connected to the structural elements thanks to a timber subframe and metal fasteners. The RC-TPext approach, instead, provides for the application of the CLT panel from the outside, connected directly to the concrete elements. In this solution, the solid brick wythe is not removed, and vertical cuts on the wythe lateral edges are created to prevent the infills from transferring undesirable shear forces to the columns.

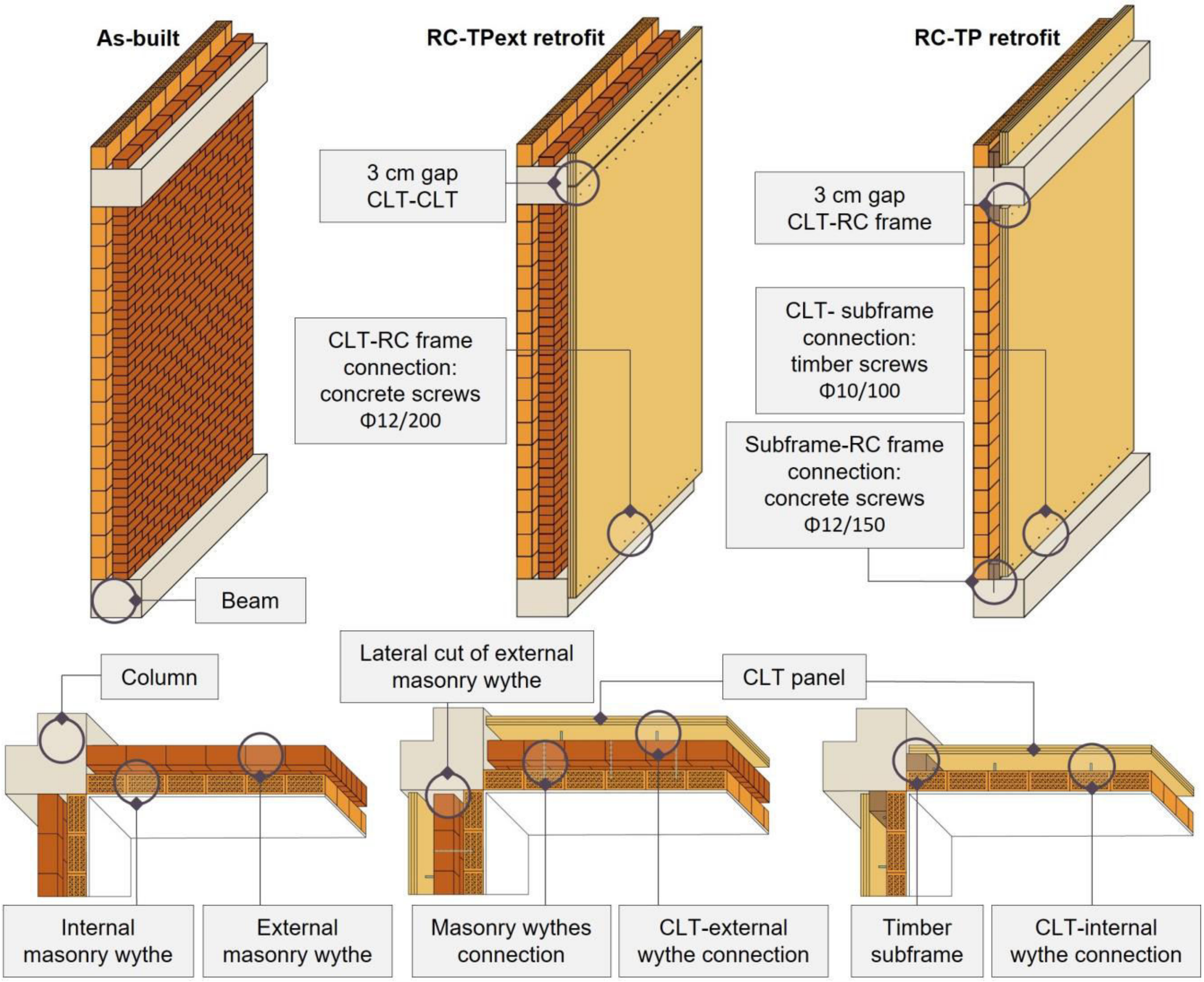

Figure 2: As-built configuration and proposed retrofit approaches

The RC-TP solution is more intrusive on the building but leads to a reduction in the seismic mass and the overall thickness of the walls. Furthermore, because the CLT structural panels are installed inside the RC frames, RC-TP can participate in resisting the vertical actions in case of collapse or damage to the existing structural elements. The RC-TPext solution is instead characterised by shorter execution times and little to no disturbance for the occupants. In both approaches, the CLT panels are connected to the remaining masonry withes (i.e., both wythes for RC-TPext and the inner wythe for RC-TP) to avoid their out-of-plane collapse. An in-depth discussion of the intervention approaches is reported in Smiroldo et al. (2021a) and Smiroldo et al. (2021b).

The retrofit intervention should address the frame bays from the upper portion of the building to the base to ensure adequate load transfer from the various storeys to the foundations. To make sure that stress is transferred from the existing structure to the CLT panels solely via mechanical fasteners, a gap of 3 cm is created all around the edges of every panel. In those frames where it was either impossible or inefficient to apply the CLT panels (e.g., due to the presence of wide openings), timber strong-backs (Cassol et al., 2020) were used to prevent the out-of-plane collapse of the infill walls. A schematic of the intervention layout as applied to the building façades is reported in Figure 3 (further details are reported in Smiroldo et al., 2021a).

Figure 3: Application of the intervention strategies

4 NUMERICAL ANALYSES

The case-study structure was invetigated via linear dynamic analyses in the bare-frame, masonry-infilled, RC-TPext, and RC-TP configuration (Figure 4). The response of the bare RC structure was analysed to show the different behaviour resulting from considering or neglecting the presence of the masonry infills in the as-built configuration. The analyses and the safety checks on the RC elements (i.e. beams, columns and shear walls) were performed in accordance with the provisions contained in the Italian code (NTC 2018 and *Circolare* 2019). The numerical model of the case-study structure was created using the finite element software SAP2000.

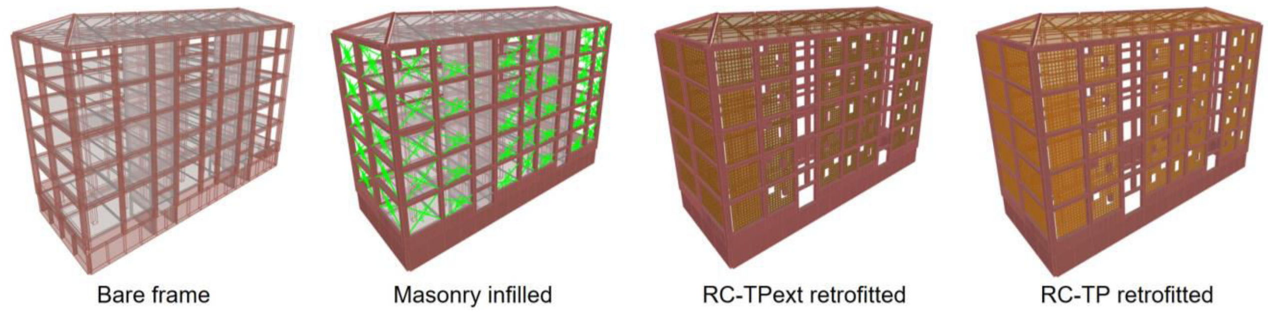

Figure 4: Numerical analysis models

RC beams and columns were modelled as *frame* elements. Floors, roof and concrete shear walls were represented using bidimensional *shell* elements. As shown in Figure 5, the masonry infills were simulated by introducing equivalent diagonal struts connected to the beam-column joints and defined according to the model proposed by Liberatore et al. (2018). Two crossed struts (modelled as *link* elements) were therefore applied to each infilled frame. The arrangement of the diagonal struts and the relative reduction of stiffness and resistance was defined according to the characteristics of the openings, following the indications of Tabeshpour et al. (2020). In the case of the diagonal struts being connected to the external beam-column joints or columns with adjacent unfilled frames or large openings, the additional shear action V_a transferred to the columns by the struts was computed *a posteriori*. It was assumed that in case of columns with infill walls on both sides, effectively confining the structural element, no significant additional shear is transferred to the column.

The additional shear V_a was evaluated tacking into account the infill-frame interaction as suggested by Celarec & Dolsek (2013) (see Equation 1).

$$V_a = \gamma_c \cdot N_{s,max} \cdot cos\vartheta \quad \text{with} \quad N_{s,max} \leq N_p \qquad (1)$$

where: γ_c = a coefficient that quantifies the additional shear as the amount of force transferred to the column by the equivalent strut, assumed equal to 0,5; $N_{s,max}$ = the maximum axial force acting on the equivalent strut obtained from the analyses; ϑ = angle of inclination of the strut; N_p = the maximum axial capacity of the strut.

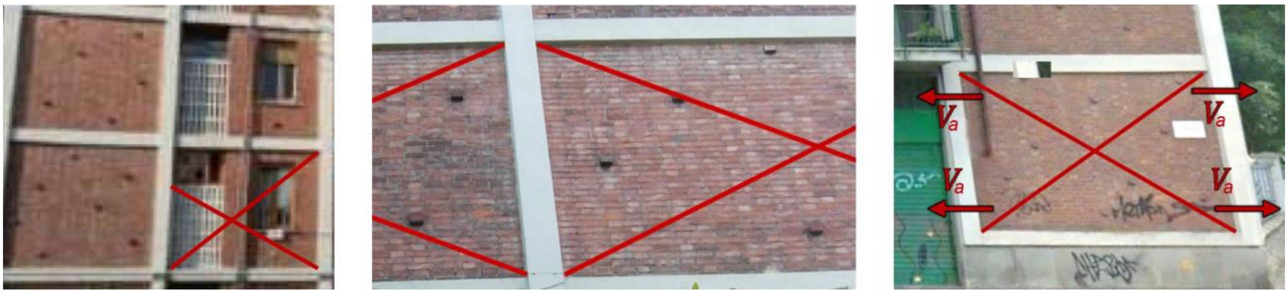

Figure 5: Diagonal equivalent strut arrangement and additional shear action

For the modelling of the retrofit, a simplified numerical model suitable for linear analysis was developed based on evidence from previous studies (Smiroldo et al., 2021a-c). The CLT panel was simulated in this simplified numerical model as a rigid body connected to the frame elements via linear links, each representing a single fastener. The accuracy of this simplification relies on the hypothesis that the panel deformation is neglectable if compared to fastener deformation. Such a hypothesis was confirmed by modelling test trials unless wide openings were present on the panel. In the case of wide openings, the fastener stiffness was calibrated to include the effect of the panel deformation. The masonry infills (i.e., both wythes for RC-TPext and the internal wythe for RC-TP) were modelled in both retrofit configurations using diagonal struts. In all the models, the effect of cracking has been taken into account according to ASCE 41-17 based on the element type (e.g., column, beam or shear wall) and on the axial load.

5 ANALYSES RESULTS

A modal response spectrum analysis was performed considering the Life-safety Limit State corresponding to 475 years return period. The analysis method (i.e., modal response spectrum) was selected because of its relevance in the engeneering practice, where more refined methods (e.g., time history analysis) are rarely used for the assessment of ordinary buildings. The two hypotheses on the material properties (i.e., HpA and HpB) were examined for all four building configurations: bare-frame, masonry-infilled, RC-TPext, and RC-TP. From the analysis results, and for all the models under HpB, very poor behaviour was observed for the beam-to-column joints with respect to code requirements. Consequently, a satisfactory performance under HpB was out of reach for all the configurations. For this reason, strengthening of the beam-to-column joints was provided for HpB, such as to satisfy the code requirements. Whereas the poor performance of the beam-to-column joints was caused by diagonal compression failure (insufficient concrete compressive strength under HpB), the preferred solution was increasing the concrete cross-section by using high-performance fibre-reinforced concrete (Beschi et al. 2011, Riva et al. 2017). Conversely, no critical issues related to beam-to-column joints were found under HpA. Therefore, the collapse of the beam-to-column joints was not included in Table 2, which shows the analysis results by reporting the number of elements (i.e., beams, columns, and shear walls) that collapsed during the analysis with ductile (i.e., flexural) or brittle (i.e., shear) behaviour. The as-built configurations (bare-frame and masonry-infilled) results for both HpA and HpB show that the infills strongly influence the overall building response. Specifically, the number of beams failing in bending decreases when moving from the bare (50 in HpA and 22 in HpB) to the infilled configuration (27 in HpA and 16 in HpB). On the other hand, the number of columns failing in shear increases significantly, going from the bare-frame configuration (8 in HpA and 0 in HpB) to the infilled one (37 in HpA and 12 in HpB). The infills' presence resulted in a stiffer building response that reduced the flexural engagement of the beams. However, the local *masonry infill-RC frame* interaction caused an additional shear transfer to the columns, increasing the number of shear failures. Such results are consistent with the outcomes from various studies available in the literature (e.g., Basha 2019; Ning et al. 2019). The RC-TP and the RC-TPext retrofits resulted in an overall reduction in the seismic vulnerability of the existing structure. Specifically, the most evident effect of the retrofit interventions was preventing brittle collapse mechanisms. Compared to the masonry-infilled configuration, the number of columns failing in shear for both retrofits went down from 37 to 3 (HpA) and from 12 to 0 (HpB).

Table 2: Ductile and brittle mechanisms activated at Life-safety Limit State - n° CE number of collapsed elements; D/C maximum demand/capacity ratio

Configuration			Ductile mechanisms			Brittle mechanisms		
			Beams	Columns	Shear walls	Beams	Columns	Shear walls
HPA	Bare-frame	n° CE	50	0	0	0	8	0
		D/C	1,66	0,87	0,43	0,62	1,26	0,85
	Masonry-infilled	n° CE	27	0	0	0	37	0
		D/C	1,45	0,64	0,37	0,59	2,24	0,76
	RC-TPext retrofit	n° CE	17	0	0	0	3	0
		D/C	1,45	0,55	0,31	0,75	1,07	0,66
	RC-TP retrofit	n° CE	14	0	0	1	3	0
		D/C	1,41	0,61	0,36	1,16	1,07	0,74
HPB	Bare-frame	n° CE	22	0	0	0	0	0
		D/C	1,40	0,71	0,39	0,91	0,95	0,74
	Masonry-infilled	n° CE	16	0	0	0	12	0
		D/C	1,21	0,58	0,32	0,84	1.84	0,63
	RC-TPext retrofit	n° CE	6	0	0	0	0	0
		D/C	1,19	0,55	0,29	0,83	0,82	0,57
	RC-TP retrofit	n° CE	9	0	0	0	0	0
		D/C	1,16	0,54	0,33	0,85	0,83	0,65

For further insight into the seismic performance of the studied building, the ratio of the maximum seismic intensity that the structure can withstand (peak ground acceleration PGA_C) to the seismic intensity prescribed for a new building (PGA_D) was calculated. Proceeding by iteration, multiple modal response spectrum analyses were performed by changing the return period (P_r) of the seismic action (in the 10-475 years range) until the PGA_C associated with each configuration was obtained. The results are reported in Table 3.

For HpA, the building in the as-built configurations (bare-frame and masonry-infilled) showed a 10 years return period and a PGA_C/PGA_D ratio of 14%. For HpB, 21 and 53 years return periods (associated with PGA_C/PGA_D equal to 30% and 49%) were obtained for the bare-frame and masonry-infilled configurations, respectively. Remarkable improvement in the seismic behaviour of the structure emerged from the analysis of the RC-TPext and RC-TP configurations. Using the masonry-infilled configuration as a benchmark, RC-TPext increases the PGA_C/PGA_D ratio by 171% in HpA and 51% in HpB, while the more invasive RC-TP makes the ratio grow by 264% and 63%. As already stated, the interventions proposed involved the most significant improvements in the activation of brittle mechanisms (e.g., column shear failure). In HpA, the return period associated with brittle failures changed from 23 years (masonry-infilled) to 357 (RC-TPext) and 316 (RC-TP), while in HpB, it increased from 43 (masonry-infilled) to 475 (RC-TPext and RC-TP).

It was also observed that the presence of extremely weak beams (e.g., with poor rebar detailing in the support area) limited the positive effect of the timber-based strengthening (e.g., see the P_r associated with RC-TPext in the HpA). To fully benefit from the retrofit methods' effectiveness, it was decided to apply external steel jacketing (ESJ, Campione et al., 2018) to the beams that showed flexural failure for gravity loads. The two additional retrofit configurations ("RC-TPext + ESJ" and "RC-TP + ESJ") were subjected to iterative modal response spectrum analysis, and the relative PGA_C/PGA_D ratio was determined. Thanks to the steel jacketing of the beams, it was possible to exploit the timber-based retrofits' potential further. The ESJ intervention was not applied to the masonry-infilled configuration because the P_r associated with the activation of brittle phenomena (23 years in HpA and 43 in HpB) limited any possible enhancement given by the flexural strengthening of the beams. Using the masonry-infilled configurations as a reference, RC-TPext + ESJ and RC-TP + ESJ resulted in the PGA_C/PGA_D ratio increasing by >400% for HpA and ≈90% for HpB. In addition, it is worth noting that for the retrofitted configurations, the P_r associated with PGA_C was determined by the activation of ductile mechanisms, with a large safety margin against the activation of brittle phenomena.

Table 3: Return period (P_r) associated with each configuration and related capacity/demand ratio (PGA_C/PGA_D)

	Configuration	P_r (years) Ductile mechanisms	Brittle mechanisms	PGA_C/PGA_D (%)
HPA	Bare-frame	10	172	14
	Masonry-infilled	10	23	14
	RC-TPext retrofit	27	357	38
	RC-TP retrofit	47	316	51
	RC-TPext + ESJ retrofit	158	357	75
	RC-TP + ESJ retrofit	158	357	75
HPB	Bare-frame	21	475	30
	Masonry-infilled	53	43	49
	RC-TPext retrofit	152	475	74
	RC-TP retrofit	202	475	80
	RC-TPext + ESJ retrofit	387	475	95
	RC-TP + ESJ retrofit	329	475	91

6 CONCLUSIONS

In the present research, two alternative timber-based seismic retrofits were applied to an existing concrete building constructed in 1955, which was analysed via numerical modelling. The study follows previous research, which studied the building from an energy-performance point of view. In this work, the structure was subjected to linear dynamic analyses with particular attention to the role of the masonry infills. The analyses showed that neglecting the infill walls in the modelling phase can lead to a response significantly different from when taking into account not just the mass but also the stiffness of the infill walls and the additional forces that they transfer to the structural elements. A marked shear vulnerability of the existing structure emerged, in fact, from considering such additional forces. The application of the interventions made it possible to reduce this vulnerability significantly. For example, shear failures of the columns were observed for seismic actions with a return period equal to 23 years in the case of the as-built condition and 357 years in the case of the timber-based retrofitted structure. The benefits brought by the interventions were further increased by strengthening with external steel jacketing a few beams that were especially vulnerable. Overall, the research has shown that applying the proposed retrofit approaches can significantly improve the seismic response of existing RC frame buildings.

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