Seismic improvement of a hospital in Milan: a sustainable choice among alternative solutions.

Lorenzo Radice[1] and Marco di Prisco[1]

[1]DSC-Erba s.r.l. via Fiume, 9 - 22036, Italy

Summary

Following recent seismic events in Emilia Romagna, current regulations prescribe seismic accelerations significantly higher than those considered before 2003 for non-seismic areas in Northern Italy. This study focuses on a reinforced concrete framed structure with cast-in-place shear walls. A comprehensive diagnostic campaign, including material characterization and dynamic tests with micro-seismic accelerometers, was conducted to assess the structural behavior. The tests revealed partial effectiveness of the expansion joints between structural bodies.

A finite element model was developed and validated based on dynamic test results. Numerical simulations considering independent body behavior under seismic loads highlighted a pounding issue. To mitigate this, a retrofitting solution involving reinforced concrete links at the upper floor was designed. This intervention effectively reduces pounding effects and internal forces in columns while maintaining building functionality during construction. However, additional shear issues in the main walls have emerged, which led to further evaluation of alternative solutions in terms of sustainability, costs and operational impact on healthcare activities.

1 INTRODUCTION

Seismic vulnerability assessment of strategic buildings, such as hospitals and civil protection facilities, is essential to ensure their functionality during and after seismic events. Many of these buildings, as well as high-occupancy structures like hospitals, were designed without adequate seismic regulations, making them particularly vulnerable to earthquake damage. Numerous studies on the Italian building stock have highlighted the high vulnerability of strategic and school buildings in the event of an earthquake [1].

A recurring problem in strategic buildings is the presence of narrow expansion joints (typically 30–50 mm) that separate independent structural blocks. These joints, designed to compensate for thermal variations and shrinkage, are often insufficient to prevent seismic hammering [2,3,4]. Furthermore, many of these buildings, originally designed to withstand only gravitational and wind loads, do not meet current seismic design standards. Post-seismic observations have documented significant damage due to hammering between adjacent buildings during high-intensity earthquakes, while moderate earthquakes have caused minor, but relevant damage. To mitigate this problem, recent seismic regulations prescribe a minimum distance between adjacent buildings or independent blocks within the same structure, to allow for asynchronous movement of masses in the event of an earthquake.

This study analyzes the seismic vulnerability of a 14-story reinforced concrete (RC) building, consisting of multiple independent blocks, and proposes a simple and cost-effective seismic retrofitting strategy. This proposal is the result of sustainability, cost, and operational impact assessments on healthcare activities for various intervention solutions.

The case study concerns a strategic structure located in Italy, originally designed without considering seismic actions, highlighting its structural deficiencies. Given its classification as a strategic building, it is associated with a high seismic risk. The retrofitting intervention must ensure the full operability of the building, making the proposed solution particularly relevant for similar structures in high-seismicity areas.

2 BUILDING DESCRIPTION

The building, constructed between 1967 and 1968, consists of 14 levels: a tunnel foundation floor (PC), a basement floor (S2), a semi-basement floor (S1), a mezzanine floor (R), and eleven above-ground floors (PT to P9). Designed with an H-shaped layout (refer to Figure 1), the structure is divided into three segments (Block A, B and C), spanning from floor S2 to floor P9, separated by expansion joints. Notably, for the first five floors, the building is connected to a neighboring structure.

The structural documentation details nominal joints, each 50 mm wide, positioned between the different building blocks and specifically between block C and the adjacent buildings. These joints were intended to accommodate thermal expansion in the materials due to their limited size. However, it is crucial to note that these joints were not initially designed to withstand displacement caused by seismic activity.

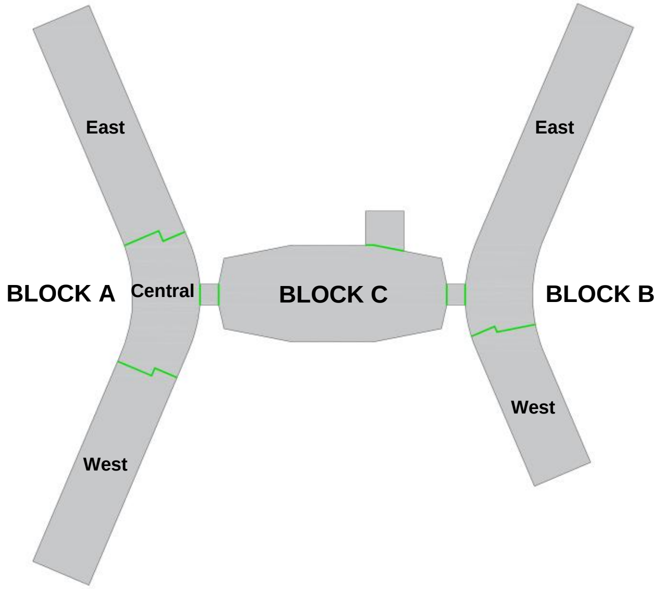

Fig. 1Plan view of the building, showing expansion joints in green and independent blocks.

Under the columns we have plinth foundations. In correspondence with the stair and elevators we have slab foundations.

The concrete floors for Block A and B are made of brick and concrete, with a distance between the beams of 50 cm; they are arranged transversally to the building and they are continuous on two spans, respectively 5.45 m and 7.85 m long. The floors have different thicknesses at the different floors from a minimum of 28 cm (24 + 4 cm) to a maximum of 43 cm (36 + 7 cm). The elevation structure consists of columns and beams in cast-in-situ reinforced concrete, predominantly characterized by irregular shapes. The columns have a rectangular cross-section with a recess at the drainpipe or, for those on the façade, a T-shaped section with a 10 cm flange. Their reinforcement consists of longitudinal bars with diameters ranging from 10 to 24 mm and stirrups generally Φ6/20, with some exceptions where Φ8 are used at a reduced spacing of up to 15 cm. The slab-thickened beams, continuous over multiple spans, run longitudinally along the building parts and have a regular span of 3.75 m per bay. In the central area, the main beams are arranged both transversely and longitudinally, depending on the floor section considered, with a maximum span of approximately 6 m.

For block C, however, the elevation structure is a mixed steel and reinforced concrete system. The floors are made with a reinforced concrete slab with a thickness of 10 cm or 16 cm cast on site on the secondary steel beams, the columns are made of steel and consist of paired H or I-shaped profiles, while the elevator cores, which serve as bracing elements, are made of reinforced concrete.

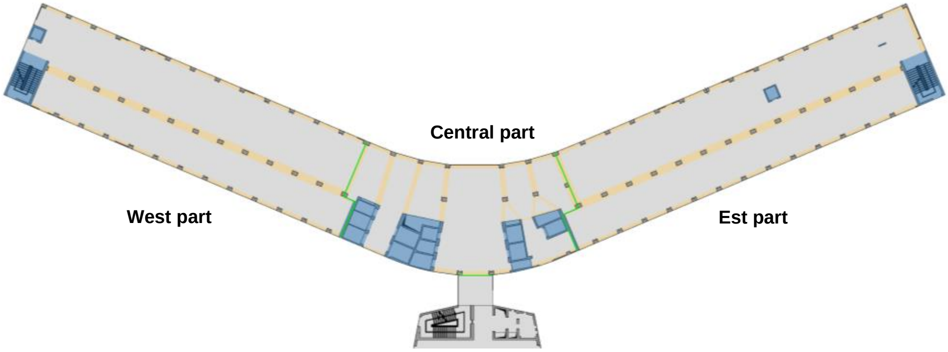

Fig. 2 Structural plan of block A: in blue stairwells-lift cores, in orange beams, and in green expansion joints.

3 SURVEY CAMPAIGN

3.1 Experimental investigation on materials

In structural drawings, concrete with $\sigma_R \geq 300$ kg/cm² has been specified. To define the concrete strength, reference is made to the Italian code, which specify that, for existing constructions, the average compressive strength value of n samples ($R_{cm(n)is}$) should be considered [5], reduced by the confidence factor (FC). For safety purposes, a medium level of knowledge (LC2) is assumed, with an associated confidence factor FC = 1.20. The compressive strength of the concrete is calculated as:

$$R_c = \frac{R_{cm(n),is}}{FC} = \frac{R_{cm(n),is}}{1.20} \quad (1)$$

During the first phase of the investigation, 26 concrete cores were extracted from the walls and columns of Block A. The average strength of the concrete $R_{cm(n),is}$ was 25.52 MPa, with a standard deviation (SD) of 8.11 MPa, and the compressive strength R_c adopted to evaluate the capacity was assumed to be 21.27 MPa according to Eq. (1) divided by the safety coefficient γ_c in case of brittle mechanisms.

On the design drawings, a "ribbed steel σ_s = 4400 kg/cm²" is specified, corresponding to a FeB44k steel with f_y > 440 MPa and f_t > 540 MPa [6]. The results of hardness tests on bars at 60 examined positions are in agreement with the type of steel specified in the design phase.

3.2 Ambient vibration tests of the building

Dynamic tests were performed by means of micro-seismic accelerometers, detecting minimum oscillations deriving from seismic background noise or from wind thrust. The tests allowed the definition of the natural frequencies and vibration mode shapes. A sketch of the first and the second mode shape detected experimentally is shown in Figure 3. The first seven experimental natural frequencies are collected in Table 1. In the same table, the vibration mode type is also indicated.

Table 1 Experimental natural frequencies

Mode No.	Frequency (Hz)	Period (sec)	Type
1	1.12	0.89	Translational in x of all Blocks
2	1.18	0.85	Block A - Torsional
3	1.30	0.77	Block A + Block C - Torsional
4	1.33	0.75	Translational in y of all Blocks
5	1.44	0.69	Block A – Translational in y
6	1.75	0.57	Block A – Translational in x
7	2.56	0.39	Block C – Translational in y

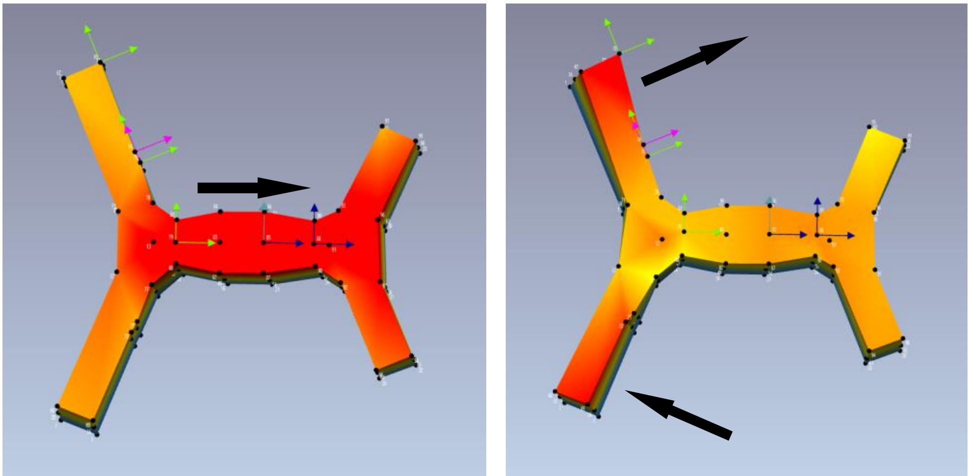

Fig. 3 (a) first and (b) second modal deformations experimentally detected (Courtesy of 4 EMME Service S.p.a.).

4 STRUCTURAL MODELLING

Finite element (FE) structural models are created using Midas Gen software (2024; [7]) to assess the vibration modes, displacements, and forces/moments acting on the structural elements. Columns and beams are modeled with one-dimensional beam elements, while two-dimensional "wall" elements are used for the reinforced concrete walls of the elevator shaft and stairwell, and plate elements are applied to the retaining walls. A 3D view of the building FE model is shown in Figure 4. For seismic analysis, infinite stiffness is assumed in the floor plane (i.e., horizontal diaphragm). Each floor is divided into three separate diaphragms, one for each part. Fixed-end conditions are applied at the base of the walls and columns and out-of-plane displacements are prevented for the retaining walls. The structural permanent load is calculated based on the slab type, while the non-structural permanent load accounts for the weight of screeds, pavements, and vertical partitions. The live load is determined from the technical drawings.

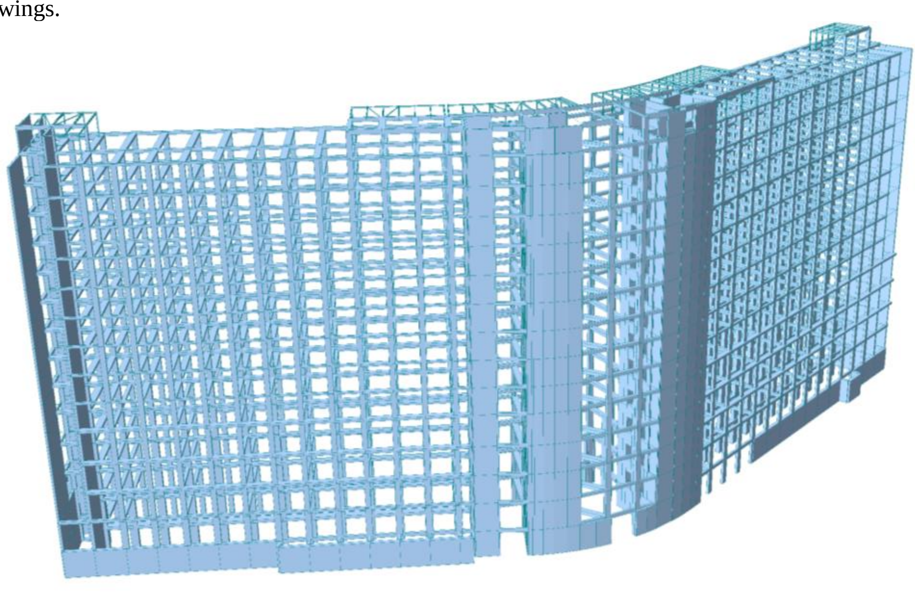

Fig. 4 FE model of the building.

The seismic action on the building is represented by a response spectrum, taking into account the site seismic hazard and soil type, in accordance with the Italian Building Code (NTC 2018; [8]) and Eurocode 8 (CEN, 2004; [9]). With a nominal life span of V_N = 50 years and a usage coefficient C_u = 2, the reference period $V_R = C_u \times V_N$ results in 100 years. The building is located on soil class C, with a topographic amplification factor of 1.0. Due to its irregularity in both plan and elevation, the building experiences a horizontal peak ground acceleration (PGA) of 0.059 g for the Life Safety Limit State (LSLS). A behaviour factor of q = 1.5 is applied. The response spectrum analysis (RSA) focuses on the horizontal components of seismic motion, considering 32 load combinations derived from four positions of the center of mass and eight possible combinations of seismic action components in both horizontal directions.

Table 2 presents the first eight vibration modes obtained from the modal analysis, along with the corresponding percentage of participating mass for each mode. Figure 5 illustrates the deformation associated with the primary numerical vibration modes of each Block.

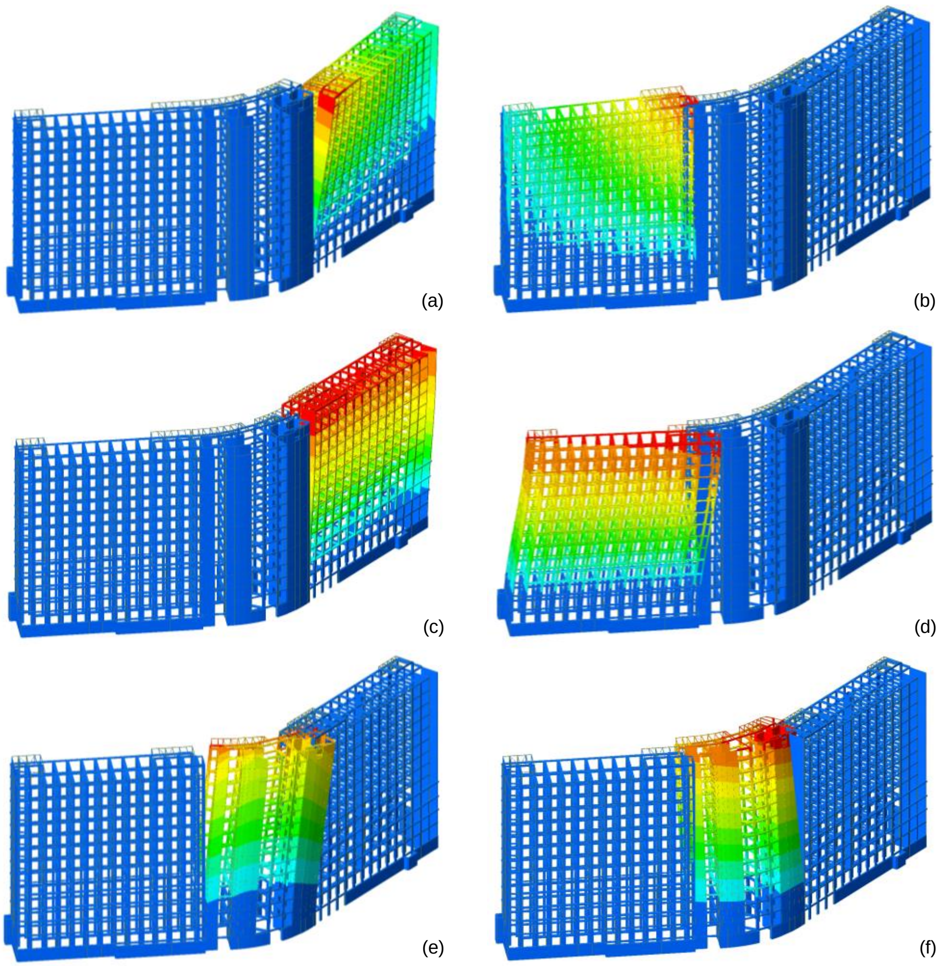

Fig. 5 (a,b,e) First and (c,d,f) second numerical mode shapes, for each structural block, before strengthening interventions.

Table 2 Numerical modes of vibration

Mode No.	Frequency (Hz)	Period (sec)	TRAN-X		TRAN-X		ROTN-Z	
			MASS (%)	SUM (%)	MASS (%)	SUM (%)	MASS (%)	SUM (%)
1	0.4062	2.462	7.6688	7.6688	11.5069	11.5069	7.5497	7.5497
2	0.408	2.4512	6.9428	14.6117	10.3084	21.8154	7.5707	15.1204
3	0.4324	2.3125	15.5715	30.1832	9.0141	30.8294	6.8768	21.9973
4	0.4513	2.2156	14.7737	44.9569	7.924	38.7534	5.6572	27.6544
5	0.6124	1.6329	1.4074	46.3643	2.8564	41.6098	14.0169	41.6713
6	0.7874	1.2701	1.2391	47.6034	3.8045	45.4144	15.5707	57.2421
7	0.9431	1.0603	16.3159	63.9193	0.1241	45.5385	0.4928	57.7349
8	1.0144	0.9858	0.2172	64.1365	17.5962	63.1346	0.007	57.7419

The experimental results suggest that, while structural joints are present between the building constituent blocks, they exhibit only partial effectiveness. Dynamic tests indicate that the blocks do not behave independently; rather, their displacements are influenced by the movement of adjacent blocks. As a result, directly comparing modal deformations between experimental and numerical analyses proves challenging.

In terms of primary mode frequencies, numerical values range from 0.41 Hz to 1.01 Hz (for modes 1 to 8 with significant mass participation), whereas experimental values fall between 1.12 Hz and 2.56 Hz. Despite the finite element model incorporating fixed-end constraints and assuming uncracked concrete, it exhibits lower stiffness than observed experimentally. This discrepancy can be attributed to several factors:

(i) the exclusion of the stiffening effect from unmodeled infill walls;

(ii) the introduction of live loads that may exceed those present during experimental tests;

(iii) the simplified representation of joints, which are assumed fully active in the numerical model, but only partially effective in reality;

(iv) the lack of constraints at connections with adjacent buildings.

5 ASSESSMENT OF SEISMIC VULNERABILITY

To examine the potential seismic interaction between blocks arising from restricted joint openings, horizontal displacements of points near these joints are computed, considering the LSLS (NTC 2018):

$$d_E = \mu_d d_{Ee} \tag{2}$$

$$\mu_d = q \qquad \text{if } T_1 \geq T_c \tag{3}$$

$$\mu_d = 1 + (q-1)\frac{T_c}{T_1} \qquad \text{if } T_1 < T_c \tag{4}$$

$$\text{in any case } \mu_d \leq 5q - 4 \tag{5}$$

In eq. (2) d_{Ee} represents the horizontal displacement obtained from the linear dynamic analysis.

Table 4 Horizontal displacement d_E at LSLS obtained from the FE analysis.

	West – Central joint		Central – East joint	
	$d_{E,x}$ (mm)	$d_{E,y}$ (mm)	$d_{E,x}$ (mm)	$d_{E,y}$ (mm)
S	111.63	233.07	116.07	234.44

For all joints, the relative displacement exceeds the nominal joint width (50 mm). This indicates a potential issue of pounding between the building constituent blocks.

RC columns and RC shear walls are checked with regard to the LSLS design spectrum considering combined moment and axial force (MN) and shear force (V). The rate of seismic action ξ_E carried by the i-th element is defined in bending coupled with axial load and in shear as the ratio between capacity X_{Rd} and demand X_{Ed}.

In Figure 6 a view of the model is proposed with highlighted in red RC walls that resulted not verified. In addition, the lower value obtained for the rate of action is shown for each block.

The rate of action obtained considering shear is equal to 0.29, for the West part, 0.39, for the Central part, and 0.33, for the East part.

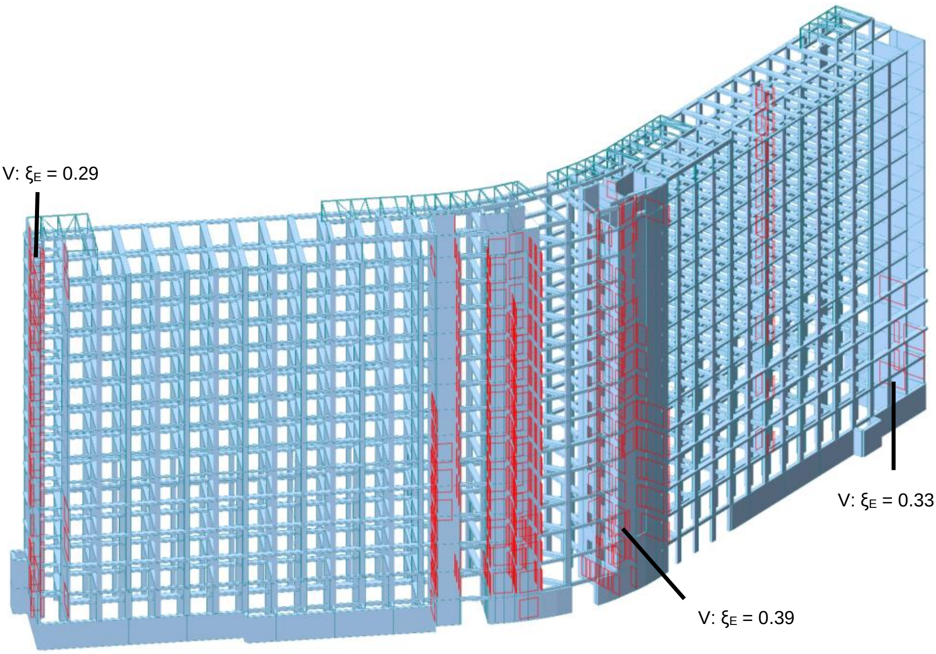

Fig. 6 Building view with not verified walls highlighted in red (rate of action reported for the most critical elements of each block).

6 RETROFITTING PROPOSAL

In order to evaluate the best intervention solution for the seismic improvement of the building, assessments were carried out in terms of costs, environmental impact, and interference with medical activities. The three proposed solutions offer different approaches, each with specific advantages and disadvantages related to operational and safety needs. The first option involves the demolition and reconstruction of the building, ensuring the highest level of seismic safety and improved energy efficiency. However, this choice implies high costs and a significant environmental impact. Nevertheless, it does not compromise hospital activities, as the construction of the new building and the demolition of the old one can be carried out in phases, allowing for the transfer of operations to part of the new structure, before demolishing the existing one. The second strategy involves adding reinforced concrete (RC) walls or new steel braces to the existing structure, improving the building stiffness and seismic resistance. However, this solution requires invasive interventions that necessitate the temporary redistribution of hospital activities to other areas, causing operational disruptions and reducing available space. Additionally, this solution is not feasible from the outside due to the lack of space in the immediate vicinity of the blocks. The third option involves the use of advanced composite materials (FRCM), providing effective and minimally invasive reinforcement, along with the introduction of rigid connections between blocks on the P9 floor to eliminate the issue of pounding (Fig. 7). This approach allows localized interventions, significantly reducing the impact on hospital activities and minimizing operational downtime compared to the other solutions.

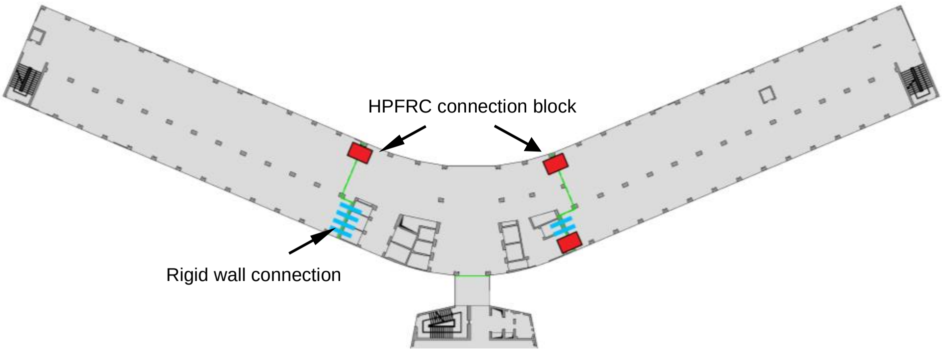

Fig. 7 Sketch of the strengthening interventions at 9Th floor

The introduction of connections between the different parts of block A at floor P9 aims to obtain a unified structural response, minimising significant relative displacements between the slabs of the unconnected floors.

In the finite element model, the connection between blocks is implemented by introducing translational springs at discrete points, linking the end nodes of columns and walls. The assigned stiffness is uniform in both directions and considers a delamination length equal to four times the bar diameter, with a specific value of:

$$k = \frac{EA}{4\phi} = \frac{190000 \cdot 14130}{120} = 2.24 \cdot 10^{7} \frac{\text{N}}{\text{mm}} \qquad (6)$$

The design of the connections is based on the maximum force experienced by the translational springs. Specifically, two types of connections have been designed: the first consists of a post-tensioned HPFRC block attached to the intrados of the slab, while the second involves connecting two adjacent walls of separate blocks using post-tensioned HPFRC sleeves.

Table 4 reports the results of the modal analysis for the strengthened building, showing the percentage of participating mass for each vibration mode. Figure 6 depicts the deformed shapes corresponding to the first three vibration modes.

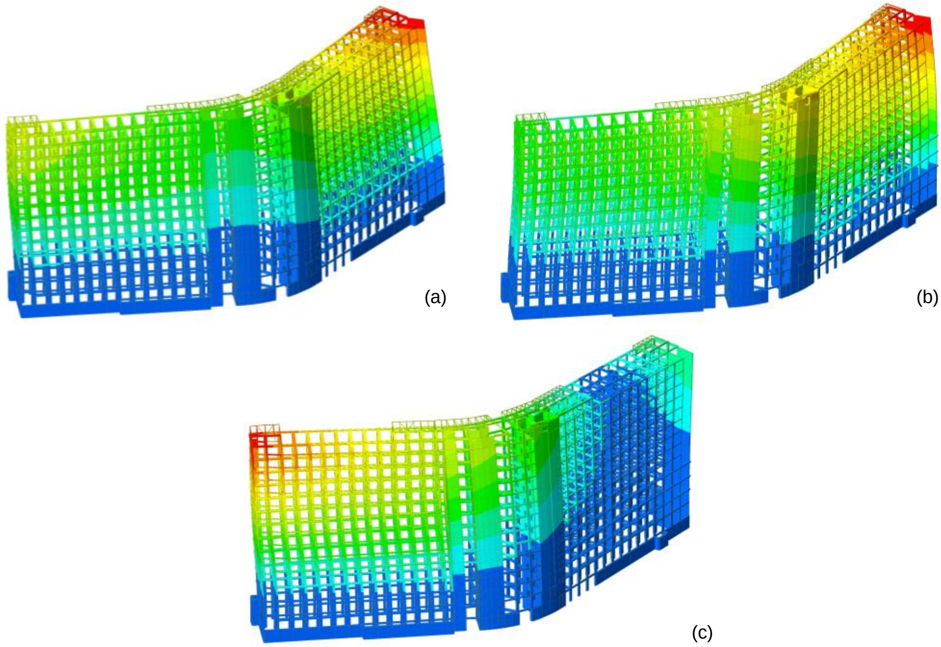

Fig. 8 (a) First, (b) second and (c) third numerical mode shapes after strengthening interventions.

Table 4 Post-strengthening first three numerical natural frequencies

Mode No.	Frequency (Hz)	Period (sec)	TRAN-X MASS (%)	TRAN-X SUM (%)	TRAN-X MASS (%)	TRAN-X SUM (%)	ROTN-Z MASS (%)	ROTN-Z SUM (%)
1	0.5652	1.7693	37.7161	37.7161	10.9541	10.9541	16.1081	16.1081
2	0.6188	1.6159	30.0847	67.8008	18.1758	29.1299	18.3113	34.4195
3	0.669	1.4948	0.1642	67.965	37.1897	66.3196	27.6819	62.1013

The pounding verification, previously conducted for the building composed of separate blocks, is now repeated for the connected structure (Table 5). In extracting displacements from the model, those associated with the first two vibration modes are excluded, as they exhibit the same sign and lack inflection points along the deformed shape.

Summing the absolute values of the displacements in the remaining modes the value of 50 mm is not exceeded, confirming that they do not contribute to pounding issues between blocks on the unconnected floors. This approach remains conservative, as even for higher-order modes, some displacements share the same sign and would not necessarily sum as absolute values.

Table 5 Post-strengthening horizontal displacement d_E at LSLS obtained from the FE analysis.

	West – Central joint		Central – East joint	
	$d_{E,x}$ (mm)	$d_{E,y}$ (mm)	$d_{E,x}$ (mm)	$d_{E,y}$ (mm)
S	30.18	25.79	29.69	39.21

To enhance the strength of the existing walls, a 15 mm thick FRCM reinforcement has been adopted. This system allows for a significant increase in the shear strength of the walls; however, according to current regulations, the strength improvement attributable to this reinforcement is limited to 50% of the existing capacity.

To overcome this limitation, metal profiles consisting of horizontal plates have been installed, anchored to both faces of the wall using through bars and epoxy resin (Fig. 10), with the aim of further enhancing the transverse reinforcement. In cases where an increase in flexural strength is also required, the metal profiles have been arranged vertically.

The entire design was developed to ensure a safety index for seismic loads equal to 60% of the values prescribed by the regulations.

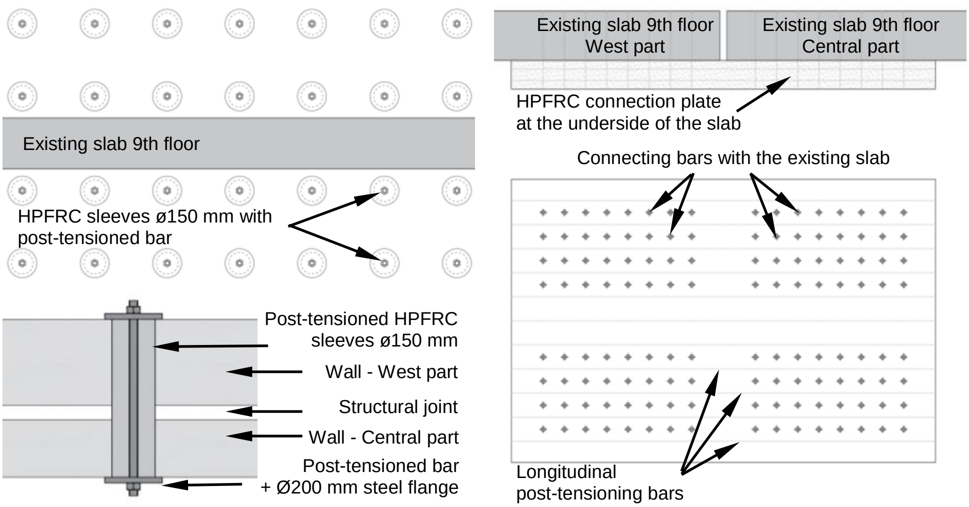

Fig. 9 Detail of the connections designed on the 9th floor

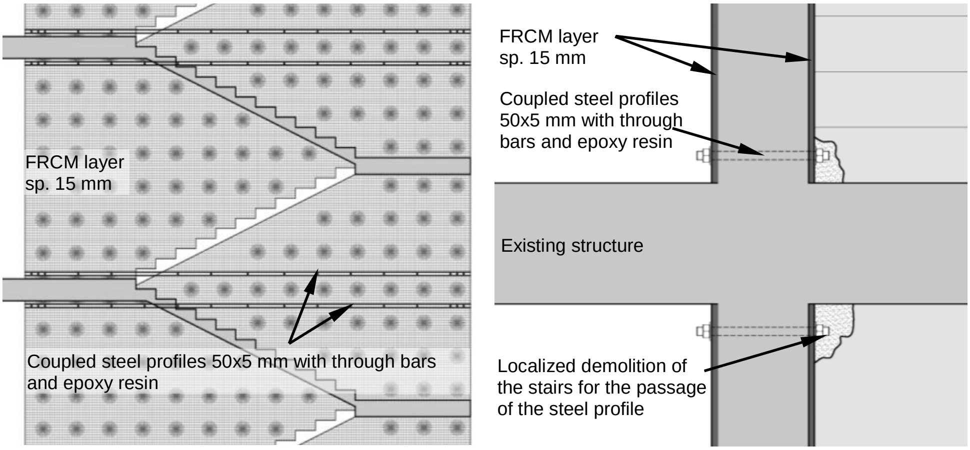

Fig. 10 Detail of the reinforcements designed on the walls

7 CONCLUDING REMARKS

A sustainable approach to improve the seismic behaviour of a 14^{th} floor hospital located in Milan is proposed: it solves the pounding problem and the lack of shear resistance of the existing structural braces. The monitoring of the slow displacements due to thermal variations of the existing joints is in progress and the solution should be further detailed to allow these displacements.

Acknowledgements

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