

# ADVANCED LIGHT-FRAME TIMBER SOLUTIONS IN ARCHITECTURAL TOP ADDITIONS

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**Abstract:** A study on architectural top additions of reinforced concrete buildings made of light-frame timber (LFT) structures is reported in this paper. Initially, the possibilities offered by traditional LFT solutions are examined. Then, a new configuration based on the incorporation of a small-sized supplemental damping system, easily hidden behind the oriented strand board sheathing panels of the light-frame structure, are proposed. A two-storey reinforced concrete residential building is selected for a demonstrative application of the two structural solutions. The single-storey top addition covers approximately 70% of the flat roof level. The paper reports: a seismic assessment analysis of the building in current conditions; the modelling criteria and the design of the light-frame timber superstructure in the traditional configuration and in the presence of the supplemental damping system, consisting of dissipative braces incorporating small-sized pressurized fluid viscous spring-dampers; and comparisons between the seismic performance of the superstructure and the entire building under the two different design hypotheses.

**Index terms:** Light-frame timber structures, Top additions, Dissipative braces, Pressurized fluid viscous dampers.

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## I. INTRODUCTION

Building superelevations made with light-frame timber structural solutions are attracting growing interest from companies in the sector, as well as from the professional world, for the considerable potential they offer. Indeed, they present several advantages, including: the greater intrinsic lightness of the wooden structure as compared, for a same structural performance, to solutions made with other structural materials, resulting in less additional masses, particularly important for interventions in seismic zones; the flexibility and speed of installation (with elements that can be assembled on site or entirely prefabricated); the equally simple possibility of future dismantling and recycling, or readaptation to new functional and usage needs, which is an essential requirement from the point of view of sustainability in the building sector; the aesthetic appeal; the versatility of finishes and integration into the plant and energy improvement project, etc.

Based on these considerations, a study on the addition of light-frame timber structures on top of reinforced concrete (RC) buildings has recently been started, in the context of the Italian ReLUIS-DPC Project 2024/2026, WP12 – “Steel, timber and composite civil and industrial constructions”, Sub-Task 12.2.2 – Timber top additions. The study aims at (a) evaluating the structural performance of traditional “platform-frame” top additions, providing contributions in terms of design and finite element modelling, and assessing the consequences of these additions on the seismic response of the underlying structures; (b) exploring the possibilities offered by the

incorporation of supplemental damping systems, with sizes suitable to hide them behind the oriented strand board (OSB) sheathing panels of the light-frame structure, with the objective of substantially limiting the increase in seismic demand on the existing structures caused by the upper extension.

This paper presents the results of the first section of this research, in which a real case study building was selected for the possible application of the two structural solutions. The building is a two-storey RC residential block designed in the 1990s, when the municipality where it is located was classified as a non-seismic zone. Subsequently, the new classification of the Italian territory provided for by the 2008 edition of the national Technical Standards placed the municipality in a moderate seismic zone, confirmed by the 2018 update of the Standards [1]. The building is characterized by an eccentric position of the RC core surrounding the stairwell, which causes a notable irregularity in plan and thus significant torsional seismic response effects, making the case examined particularly challenging. The top addition planned in the architectural project for the height expansion of the building extends to approximately 70% of the flat roof level. The supplemental damping strategy adopted for incorporation into the timber structure consists of a dissipative bracing (DB) system equipped with small-sized pressurized fluid viscous (PFV) spring-dampers. The assessment analysis of the building in current state, the modelling criteria and the design of the timber superstructure in traditional and dissipative configurations, and a comparison between the seismic performance offered by the two solutions are summarized in the following sections.

## II. GEOMETRICAL AND STRUCTURAL CHARACTERISTICS OF CASE STUDY BUILDING

The case study building is located in a medium-low seismicity site in Italy. Its plan dimensions are 36.3 m  $\times$  22.2 m, with a total height above ground of 6.55 m. The ground and first storey heights are 3.4 m and 3.15 m, respectively. Figure 1 shows the structural plan of the ground storey, including the  $X$  and  $Y$  axes of the Cartesian reference system assumed in the finite element analyses (being  $Z$  the vertical axis), and a longitudinal and a transversal section.

The structure of the first and roof floor is 280 mm thick and made of 240 mm-high partly prefabricated R/C joists, parallel to  $Y$ , clay lug bricks, and a 40 mm thick upper RC slab. The beams in longitudinal direction, parallel to  $X$ , have an in-depth cross section of 1000 mm  $\times$  280 mm, in the four internal alignments, and 350 mm  $\times$  280 mm, in the two perimeter alignments. All beams in transversal direction have in-depth section of 400 mm  $\times$  280 mm, except for the one facing the “C-shaped” RC core enclosing the flights of stairs (C-3/C-4 span), sized 1000 mm  $\times$  280 mm, and those belonging to the A alignment, with section of 300 mm  $\times$  280 mm. The 200 mm thick RC core has dimensions of 5555 mm along  $X$  and 3000 mm along  $Y$ .

All columns have rectangular section with the larger side parallel to  $Y$ , equal to 600 mm on the ground storey, except for columns located on A-2, A-3, A-4 and A-5 fixed lines (Figure 1), whose side is 400 mm long. On the first storey, the  $Y$ -parallel side is 600 mm long, for all columns belonging to the perimeter longitudinal alignments (1 and 6), and 400 mm long, in the four internal alignments (2 through 5). The  $X$ -parallel side is equal to 200 mm for all columns on both storeys. On the ground storey, the distance between the columns is equal to 6 m in the internal longitudinal alignments, and 3 m in the external ones. Columns located on B-1, E-1, H-1 and M-1 fixed lines on the south façade, and C-6, F-6, I-6 and N-6 fixed lines on the north façade do not continue on the first storey. In transversal direction, the distance between the columns is equal to 3.6 m for the four lateral spans and 6.8 m for the central span.

The foundation is constituted by a continuous 500 mm-thick RC slab, incorporating a mesh of 1500 mm-wide in-depth rib beams plotted under the longitudinal and transversal frame alignments and the C-shaped core. This solution, oversized in terms of stress states transferred to the underlying soil, was adopted in the original design of the building to protect the ground storey from the seasonal variations of water table level. This allows to design the top addition of the building without the need to enlarge the foundation slab or introduce piles under possible critical zones.

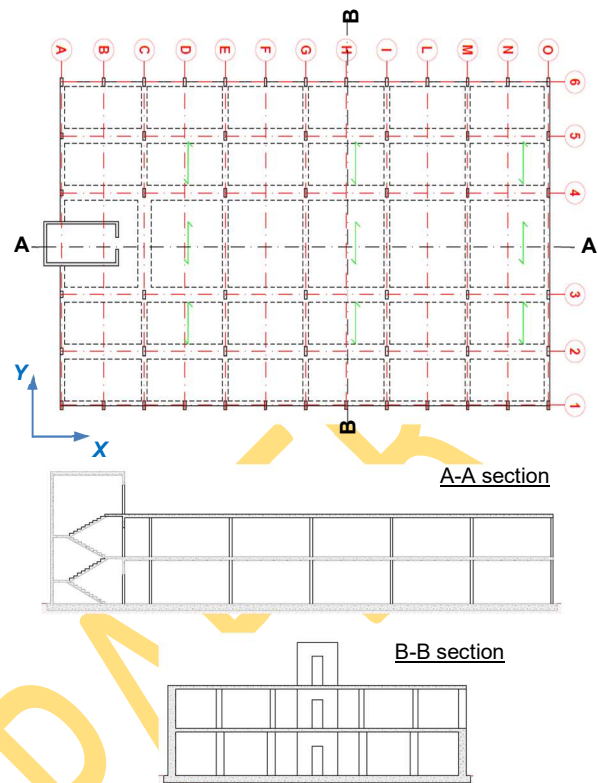


Figure 1: Ground Storey Structural Plan, Longitudinal and Transversal Sections of Case Study Building

## III. SEISMIC ASSESSMENT ANALYSIS IN CURRENT STATE

The seismic assessment study of the building was carried out, via time-history linear analysis, by using the finite element model of the structure displayed in Figure 2, generated with SAP2000NL calculus program [2]. Frame-type elements were assumed for columns and beams, and shell elements for the stairwell core.

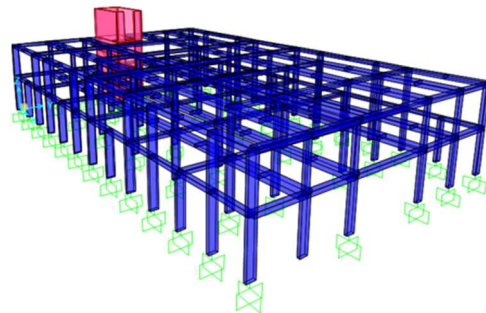


Figure 2: View of the Finite Element Model of the Structure

The analyses were performed for the Serviceability Design Earthquake (SDE, with 63% probability of being exceeded over the reference time-period,  $V_R$ ) and the Basic Design Earthquake (BDE, with 10%/ $V_R$  probability) hazard levels assumed by Italian Standards [1]. The  $V_R$  period was fixed at 50 years, obtained by multiplying the nominal structural

life  $V_N$  of 50 years by a coefficient of use  $C_u$  equal to 1, corresponding to the residential use of the building. A set of seven groups of three accelerograms each was applied as input to the time-history analyses. The artificial ground motions were generated from the elastic pseudo-acceleration response spectra, at linear viscous damping ratio of 0.05, assumed for the municipality where the building is located, drawn in Figure 3. For each group of input motions, one accelerogram was applied in  $X$  direction, one in  $Y$  and one in  $Z$ . A preliminary modal analysis of the structure highlights a first mode, mixed translational along  $Y$ –rotational around  $Z$ , with vibration period of 0.566 s and effective modal mass (EMM) equal to 57.9% of the total seismic mass of the building along  $Y$  and 24% around  $Z$ , and a first translational mode along  $X$ , with period of 0.169 s and EMM of 86.8%. A total of 10 modes is needed to activate a summed modal mass (SMM) greater than 85% both in  $Y$  and  $X$ , and 43 modes in  $Z$ , as the rotational modes are associated, with similar small EMM contributions, to several translational modes in  $X$  and  $Y$ .

Although designed with no reference to Seismic Standards, the reinforcement details of the structural members are not poor for stirrups and ties, with well-anchored 135-degree hooks in all columns and beams of the elevation structure and the foundation rib beams, as well as for longitudinal bars and electro-welded steel nets, which include appropriate lap splices and end-hooks. In addition, the 1996 edition of the Italian Standards of RC structures, according to which the building was designed, imposed minimum percentages of reinforcement in flexure and shear similar to the ones requested for structures located in low seismicity zones at the time. In view of this, and by considering the irregularity of the building in plan, according to the suggestions of the Instructions for the application of the current Italian Standards [3], a behaviour factor  $q$  equal to 2 was assumed in the stress state checks in flexure and compression-flexure (corresponding to moderately ductile mechanisms), and 1.5 in shear (brittle mechanisms).

Based on the assumptions recapitulated above, the results of the time-history analyses at the SDE show that: (a) all structural members meet the stress state checks; (b) the maximum values of the interstorey drift ratio,  $IDR$  (i.e. the ratio of the interstorey drift to the interstorey height), always attained on the first storey, are equal to 0.1% in the stiffer direction  $X$ , and 0.4% in  $Y$ . The latter value is below the  $IDR$  limit of 0.5% fixed by the Italian Standards as basic requirement for the attainment of the Immediate Occupancy (IO) seismic performance level for infilled frame buildings, like the case study one.

At the BDE, 35% of columns and 22% of beams on the ground storey, and 15% of columns and 20% of beams on the first storey, are in unsafe conditions. By way of example of the response at this

hazard level, the  $M_Y$ – $M_X$  biaxial moment interaction curves—being  $M_Y$ ,  $M_X$  the bending moments around  $Y$  and  $X$  axes—obtained from the most demanding among the seven groups of accelerograms, are plotted in Figure 4 for the most stressed columns with section  $200\text{ mm} \times 400\text{ mm}$ , located on the first storey O-2 fixed line, and  $200\text{ mm} \times 600\text{ mm}$ , located on the ground storey O-1 fixed line.

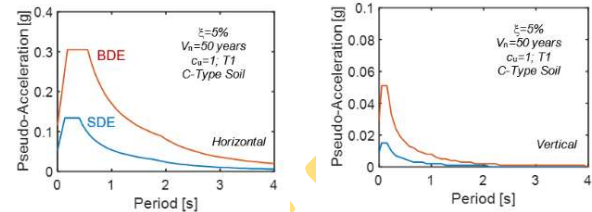


Figure 3: SDE and BDE-Scaled Pseudo-Acceleration Elastic Response Spectra

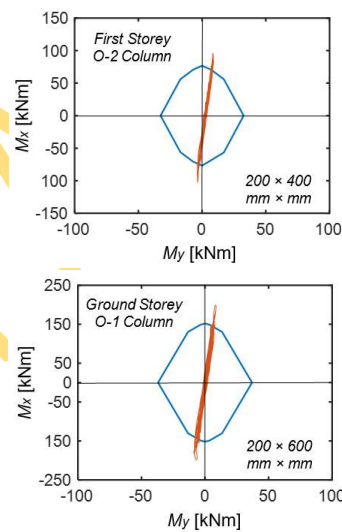


Figure 4:  $M_Y$ – $M_X$  Biaxial Moment Interaction Curves for First Storey O-2 Column and Ground Storey O-1 Column Obtained from the Most Demanding BDE-Scaled Group of Accelerograms (orange), and Relevant Biaxial Moment Safe Domains (blue).

$IDR$  reaches a peak value of 1.19% on the first storey of the O transversal alignment, which corresponds to collapse conditions of relevant infills [4], and maximum values ranging from about 0.8% to about 1% on the “I” and “M” alignments, denoting very severe and irreparable damage conditions. On the ground storey, a peak  $IDR$  value of 0.63% is achieved, assessing severe but still repairable damage.

#### IV. LIGHT-FRAME TOP ADDITION

As shown by the drawings in Figure 5, the superelevation extends across the three central transversal spans of the building, whereas the two side spans are not covered, to be left as terraces for the apartments of the new storey. The top addition is 3.8 m high. The structure is constituted by light timber frames sheathed on both sides by oriented strand board (OSB) panels nailed to the studs, as well as to the bottom, intermediate blocking and top plates of the

frames. The studs are sized 80 mm (side parallel to the frame plan) × 120 mm (transversal side), and the plates 120 mm × 120 mm, for the frames situated along the transversal and external longitudinal alignments in plan. Studs are sized 80 mm × 150 mm, top and bottom plates 150 mm × 150 mm, and blocking plates 120 mm × 150 mm in the internal longitudinal alignments and the top extension of the stairwell core. All OSB sheathing panels are 15 mm thick.

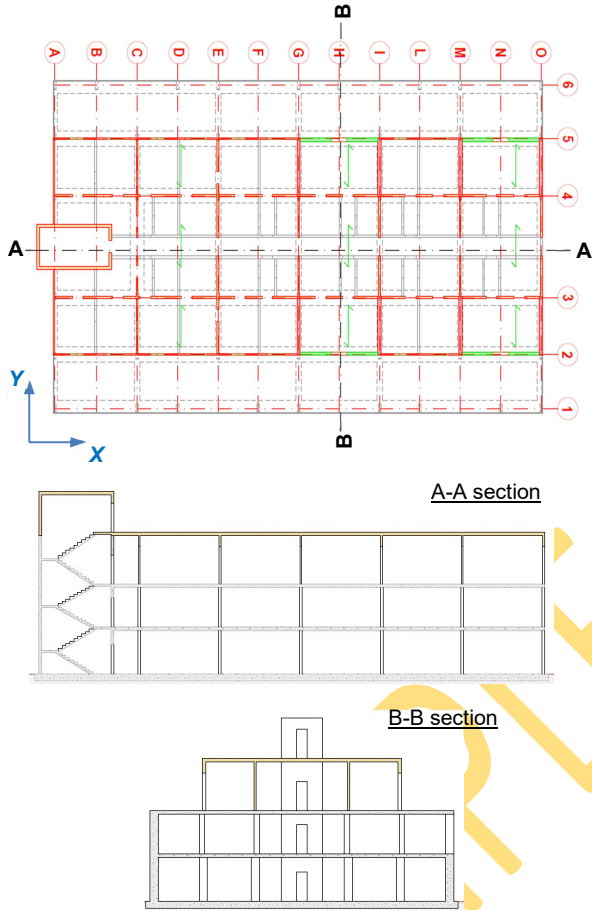


Figure 5: Top Storey Structural Plan, and Longitudinal and Transversal Cross Sections Including the LFT Top Addition

The structure of the flat roof is made of wooden beams parallel to Y, with sections of 160 mm × 180 mm in the two lateral spans and 160 mm × 180 mm in the central span, 25 mm-thick roof boards and 25 mm-thick OSB sheathing panels.

Views of the finite element model of the structure in the presence of the superelevation are shown in Figure 6. As visualized by these images, the LFT structure was simulated by using equivalent shell-type elements, the elastic properties of which were calibrated by means of careful equivalence criteria—not discussed here for brevity’s sake—aimed at exactly reproducing the racking stiffness of the framed panels.

Like in current state, the modal analysis highlights a first mode, mixed translational along Y–rotational around Z, with vibration period of 0.697 s

and EMM equal to 60.2% along Y and 22.5% around Z, and a first translational mode along X, with period of 0.275 s and EMM of 82.9%. Four modes are needed to activate a SMM greater than 85% in Y, 24 modes in X, and 44 in Z.

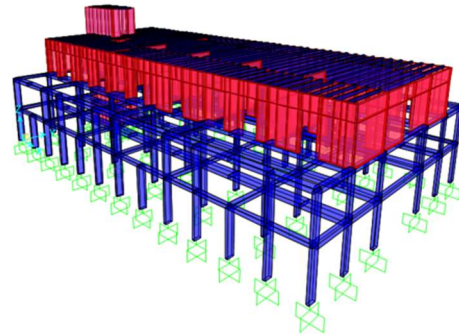


Figure 6: View of the Finite Element Model of the Structure Including the LFT Top Addition

At the SDE, all structural members meet the stress state checks also in the presence of the top addition; the maximum IDR values are equal to 0.16% in X, and 0.46% in Y, still below the IO-related limit of 0.5%. At the BDE, the percentage of columns in unsafe conditions rises from the 35% amount in current state to 41%, on the ground storey, and from 15% to 30%, on the first storey. The number of beams not meeting the stress state checks remains practically unchanged. By way of example of these results, the  $M_Y$ – $M_X$  biaxial moment interaction curves plotted in Figure 4 in as-built conditions are duplicated in Figure 7 in the presence of the top addition.

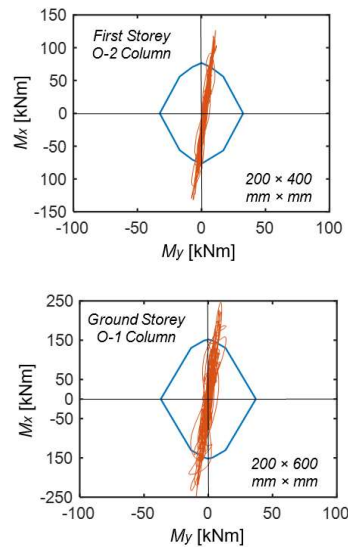


Figure 7:  $M_Y$ – $M_X$  Biaxial Moment Interaction Curves for First Storey O-2 Column and Ground Storey O-1 Column Obtained from the Most Demanding BDE-Scaled Group of Accelerograms (orange), and Relevant Biaxial Moment Safe Domains (blue), in the Presence of the LFT Top Addition.

IDR increases by about 20% as compared to current conditions, reaching 1.42% in the O longitudinal alignment on the first storey, and exceeding 1% on the

“I” and “M” alignments of the same storey. A peak IDR value of 0.77% is found on the O alignment of the ground storey, assessing irreparable damage conditions of the infills belonging to this level too.

## V. SUPPLEMENTALY DAMPED LIGHT-FRAME TOP ADDITION

As mentioned in the Introduction, the supplemental damping technology adopted for incorporation in the LFT top addition is based on the incorporation of a dissipative bracing system equipped with small-sized pressurized fluid viscous spring-dampers. The mechanical behaviour of these devices, whose working principle is based on the flow of a pressurized highly viscous fluid through a thin annular space between piston head and tank casing [5], is characterized by the following damping,  $F_d$ , and elastic,  $F_{ne}$ , force components:

$$F_d(t) = c \operatorname{sgn}[\dot{x}(t)] |\dot{x}(t)|^\gamma \quad (1)$$

$$F_{ne}(t) = k_2 x(t) + \frac{(k_1 - k_2)x(t)}{\left[1 + \left|\frac{k_1 x(t)}{F_0}\right|^5\right]^{1/5}} \quad (2)$$

where:  $t$  = time variable;  $c$  = damping coefficient;  $\operatorname{sgn}(\cdot)$  = signum function;  $\dot{x}(t)$  = velocity;  $|\cdot|$  absolute value;  $\gamma$  = fractional exponent, ranging from 0.1 to 0.2;  $F_0$  = static pre-load;  $k_1, k_2$  = stiffness of the response branches situated below and beyond  $F_0$ ; and  $x(t)$  = displacement.

Drawings illustrating the installation details of the system are presented in Figure 8, according to the general mounting scheme adopted in several early [6] through recent [7] applications of this technology. Views of the finite element model of the structure including the supplementally damped LFT (hereafter named SD-LFT) top addition are shown in Figure 9.

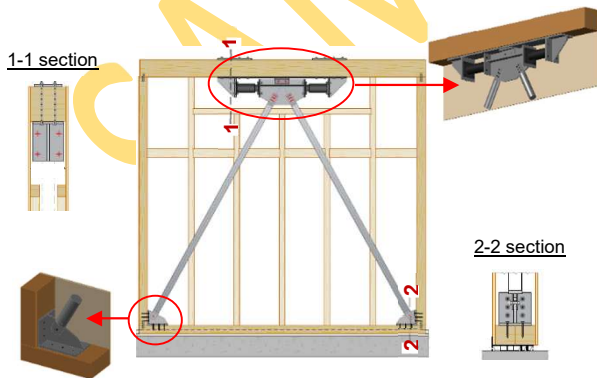


Figure 8: Installation Details of the DB System into the LFT Structure

The basic design objective of the DB system consisted in nearly annulling, or at least significantly mitigating, the increase in member stress states and drifts induced by the superelevation, at the same time limiting the incorporation of the system in the

perimeter frames and few internal frames, thus constraining the architectural impact on the interiors of the new apartments.

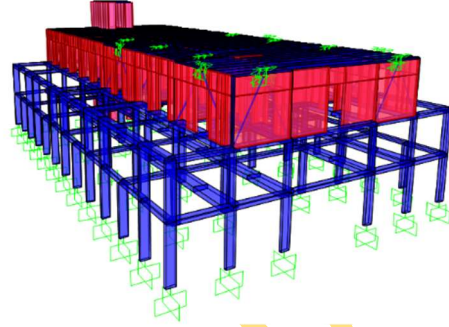
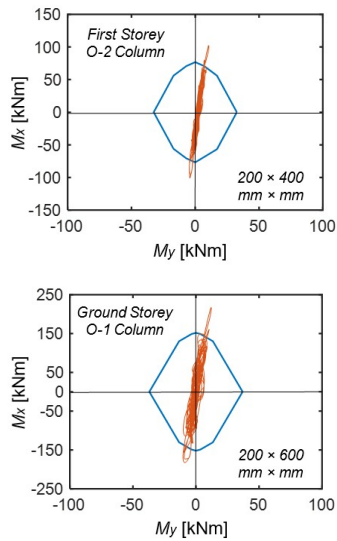


Figure 9: Views of the Finite Element Model of the Structure the SD-LFT Top Addition

According to the design criteria formulated in [6], one of the smallest types of PFV spring-dampers currently in production was adopted, with the following mechanical properties, taken from the manufacturer’s catalogue [8]:  $c = 9.9 \text{ kN}\cdot(\text{s}/\text{mm})^\gamma$ , with  $\gamma = 0.15$ ,  $F_0 = 17 \text{ kN}$ ,  $k_2 = 1.74 \text{ kN}/\text{mm}$ , nominal energy dissipation capacity  $E_n = 7 \text{ kJ}$ , stroke  $s_{max} = \pm 30 \text{ mm}$ , maximum response force  $F_{max} = 150 \text{ kN}$ . By referring to these properties, eight spring-damper pairs were installed in transversal direction, and namely in spans G-2/G-3, G-4/G-5, I-2/I-3, I-4/I-5, M-2/M-3, M-4/M-5, O-2/O-3 and O-4/O-5, and four pairs in longitudinal direction, in spans G-2/I-2, G-5/I-5, M-2/O-2, M-5/O-5. Due to the very low lateral stiffness increase produced by the DB system, as determined by the in-series connection of the diagonal trusses and the PFV devices, and the low axial stiffness of the elastic component of the latter, the modal parameters remain nearly unchanged as compared to the conventional top addition solution. Indeed, the first mixed mode has a period of 0.694 s and EMMs of 60.6% along  $Y$  and 21.4% around  $Z$ , and the first translational mode in  $X$  has a period of 0.276 s and EMM of 83%. Furthermore, the same numbers of modes are needed to activate a SMM greater than 85% in  $Y$  (4),  $X$  (24), and  $Z$  (44).

At the SDE, all structural members pass the stress state checks; the maximum  $IDR$  values are equal to 0.14% in  $X$  and 0.43% in  $Y$ , i.e. practically halfway the values computed in current state and for the conventional top addition solution. In comparison with the latter, at the BDE the percentage of columns in unsafe conditions drops to 33% on the ground storey and to 24% on the first storey, and the percentage of beams to 20% (ground) and 13% (first). These percent values are also lower than the ones in current state, except for first storey columns, four of which passes to unsafe conditions as compared to current state, although with demand/capacity ratios only slightly greater than 1. Figure 10 shows the  $M_Y$ - $M_X$  biaxial moment interaction curves for the same columns

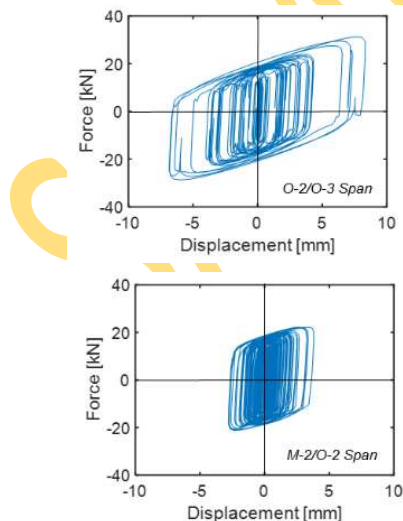
which Figures 4 and 7 refer to, highlighting a similar response to as-built conditions.



**Figure 10:**  $M_x$ - $M_y$  Biaxial Moment Interaction Curves for First Storey O-2 Column and Ground Storey O-1 Column Obtained from the Most Demanding BDE-Scaled Group of Accelerograms (orange), and Relevant Biaxial Moment Safe Domains (blue), in the Presence of the SD-LFT top addition

$IDR$  drops from 1.42% to 1.23% on the first storey O transversal alignment, and from 0.77% to 0.67% on the 1 longitudinal alignment, as compared to the conventional top addition design, highlighting a very small increase on the above-mentioned values computed in current state (1.19% and 0.63%).

By way of example of the DB system performance, Figure 11 shows the response cycles of the spring-damper pairs installed in the O-2/O-3 and M-2/O-2 spans.



**Figure 11:** Response Cycles of the Pairs of PFV Spring-Dampers Installed in the O-2/O-3 and M-2/O-2 Spans of the SD-LFT Top Addition Obtained from the Most Demanding BDE-Scaled Group of Accelerograms.

These graphs highlight that the devices positioned in the stiffer  $X$  direction achieve maximum displacements approximately equal to 45% of those reached by the dampers installed along  $Y$ . Therefore, thanks to their early activation, also the former devices offer an appreciable contribution to the seismic protection of both the superstructure and the underlying structure.

## VI. CONCLUSIONS

Due to its intrinsic lightness, the conventional light-frame timber structure adopted as first design hypothesis for the architectural addition to the top of the reference building examined in this study produced an acceptable growth in the seismic demand on the underlying RC structure. This was assessed at the SDE, with maximum  $IDR$  values increased by 15% as compared to as-built conditions, but still below the 0.5% IO-related normative drift limit, as well as at the BDE, with the percentage of columns in unsafe conditions rising from 35% to 41% at the ground storey, and from 15% to 30% at the first storey, and a virtually unchanged number of beams failing the stress state checks.

The incorporation of a dissipative bracing system equipped with small-sized PFV spring-dampers into the timber structure allowed to meet the basic design objective of nearly annulling the growth in demand caused by the superelevation, thus leaving practically unchanged the seismic performance of the RC structure as compared to its current state. This was achieved by limiting the incorporation of the DB system only in eight perimeter frames and four internal frames, and by hiding the dissipative braces behind the OSB sheathing panels of the light-frame structure, thus constraining the architectural intrusion into the interiors of the new apartments.

In view of this, the supplemental damping-based design solution appears to represent a promising alternative for building top additions, to be explored further in the next steps of the research programme recently began on this topic.

## ACKNOWLEDGEMENTS

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