ANALISI SISMICA E RINFORZO DI UN EDIFICIO IN ACCIAIO MULTIPIANO NON CONFORME ALLE NORMATIVE VIGENTI

SEISMIC ASSESSMENT AND STRENGTHENING OF A NON-CODE CONFORMING MULTI-STOREY STEEL BUILDING

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ABSTRACT

Existing steel structures are characterized by a wide variability in terms of structural conception and adopted constructional details. Moreover, older steel buildings are often non-conforming with normative provisions currently in force. Such unconformity can possibly lead to severe structural shortages. Within this framework, an existing six-storey steel building located in Naples is investigated as a case-study. The selected building features both concentrically braced frames (CBF) along the transversal direction and moment resisting frames (MRF) in the longitudinal direction, in which non-conforming beam-to-column joints were adopted. Multiple on-site surveys carried out by the Authors allowed the complete characterization of the structure. Global behavior of the investigated structure was inspected by means of finite element simulations. Non-linear analyses showed that the case-study has a poor seismic performance in both directions. Hence, a seismic strengthening intervention was designed and numerically checked. Efficiency of the proposed solution is presented by comparing the global structural behavior in both ante- and post-operam configurations.

SOMMARIO

Le strutture esistenti in acciaio sono caratterizzate da un'ampia variabilità in termini di concezione strutturale e dettagli costruttivi adottati. Inoltre, gli edifici in acciaio più datati risultano spesso non conformi con le prescrizioni normative attualmente in vigore. Suddetta non conformità può potenzialmente portare a gravi inadeguatezze strutturali. A tal proposito, nel presente lavoro un edificio esistente in acciaio di sei piani situato a Napoli è analizzato quale caso di studio. L'edificio selezionato presenta sia controventi concentrici (CBF) in direzione trasversale che telai momento resistenti (MRF) in direzione longitudinale, rea-lizzati impiegando nodi trave colonna di tipo non conforme.

Diversi rilievi in loco effettuati dagli Autori hanno permesso la completa caratterizzazione della struttura. La risposta globale della struttura indagata è stata esaminata mediante simulazioni agli elementi finiti. Le analisi non lineari hanno dimostrato che il caso studio ha un comportamento sismico scadente, con prestazioni insoddisfacenti in entrambe le direzioni. Pertanto, un intervento di rinforzo sismico che tenesse conto delle peculiarità strutturali rilevate è stato progettato e verificato numericamente. L'efficienza della soluzione proposta è presentata confrontando la risposta globale della struttura nelle configurazioni ante- e post-operam.

1 INTRODUCTION

Existing multi-storey steel constructions display a substantial variability in terms of distribution and typology of resisting systems, structural conception and detailing. Furthermore, owing to the absence of adequate seismic provisions, older steel structures were often conceived to endure only gravity and wind loads, and without proper detailing compliant with principles of capacity design [1]. Hence, the investigation of the seismic behaviour of existing steel constructions is undoubtedly a critical task in order to design suitable retrofit interventions to achieve an adequate structural performance. Within this framework, the seismic assessment and upgrade of an existing six-storey steel building located in Naples is presented in this paper. The investigated structure was designed in accordance with Italian normative requirements that were in force during the 1960s. Hence, only gravity loads and moderate wind loads were considered in the design process. Moreover, several non-code conforming details were adopted (e.g., the beam-to-column joints) [2].
The aim of the present work is *i)* to inspect the global seismic behaviour of the selected case study and *ii)* to check the effectiveness of low impact retrofit solutions. This paper is divided in four parts; in the first section, main features of the selected structure are introduced; hence, modelling assumptions adopted for finite element analyses (FEAs) are briefly described in the second part. The global seismic performance of the as-built structure is presented in the third section. Finally, the efficiency of designed retrofit interventions is checked.

2 THE CASE STUDY

The considered case study was erected between 1960 and 1961 to function as a public office and depository of documents. The structure was realized starting from a former two-storeys masonry construction, which was hollowed out from the inside to make place for a six-storeys steel building, preserving only existing façades and the original V-shaped footprint (1400 m^2) of the building [2]. An average inter-storey height equal to 3.7 m was adopted, with the only exception of the first inter-storey height (3.9 m), for a total height of 22.4 m. The design was carried out in accordance with Italian normative provisions that were in force during the 1960 (R.D. 2105/1937). Hence, only gravity and moderate wind loads were accounted for. Notably, no P-Delta effects were considered despite of the height of the building. Frontal and lateral views of the selected case study as reported in original design report are depicted in Fig. 1 [2]. Moment resisting frames (MRFs) were located along the longitudinal direction of the structure, while different typologies of concentrically braced frames (CBFs) were adopted in the transverse direction (i.e., both Y- and X-shaped braces).
With reference to MRF systems, IPN 320 profiles were adopted for main beams at all storeys, while columns were made by means of hollow squared profiles with constant external footprint along the height of the building (SHS 140x140 mm). Notably, tubes' thickness decreases from 18 to 6 mm storey by storey. CBFs were made using different types of single or coupled members. Namely, 80x40x4mm Ls were used for Y-shaped CBFs in both coupled and single configurations, while both single angle (L 75x50x5mm) and tubular (CHS 80x10 mm) profiles were adopted for X-shaped braces. On-site surveys were carried out by the Authors to check the compliance between original design drawings and the as-built structure. Results of inspections are depicted in Fig. 2.

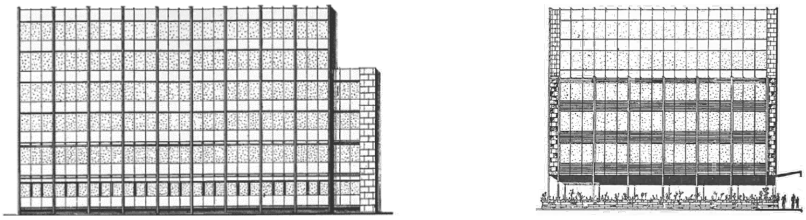


Fig. 1. Front (a) and lateral (b) view of the building according to the original design report [2].

Main properties of adopted structural materials were drawn from the original design report. Accordingly, Aq 42 structural steel was used for all members (maximum allowable stress σ_{adm} = 160 N/mm^2) with the only exception of hollow columns, which were realized using Aq 55 steel (σ_{adm} = 200 N/mm^2). In accordance with European and Italian codes in force [3-4], these materials can be assimilated to modern S235 and S275 steel grades, respectively. In compliance with Italian provisions for existing buildings [4-5], the highest level of structural knowledge (i.e. "KL3 – exhaustive knowledge") was reached for the selected case study in light of all collected data. Hence, characteristic values of material properties were used for FEAs accounting for no reductions.

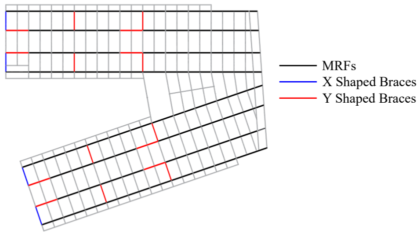


Fig. 2. Planar distribution of adopted resisting systems.

3 MODELLING ASSUMPTIONS

Numerical modelling of the investigated structure was made using SAP2000 v.23 [6] (see Fig. 3). Beams, columns and braces were modelled using frame elements introduced in correspondence of profiles' centroidal axes. The column-to-foundation joints were modelled as fixed restraints. In-plane rigidity of the floors, which is ensured by the presence of concrete slab and floor bracings, was modelled by means of diaphragm constraints at each floor. Flexural releases were introduced to model beam-to-beam and brace-to-beams connections owing to their negligible flexural stiffness. In compliance with the original design report, the yielding strength f_y of all existing members was set equal to 240 N/mm^2, with the exception of columns, for which f_y = 300 N/mm^2 was assumed. With respect to new CBFs adopted for upgrading, an S355 steel grade (f_y = 355 N/mm^2) was used. Non-linear behaviour of steel elements was accounted for introducing lumped plastic hinges at members' ends, for which moment-rotation curves were defined according to ASCE-13 [7].
For instance, M_x-M_y plastic hinges were introduced in case of main beams, while the influence of the axial force was explicitly considered for columns by means of N-M_x-M_y lumped hinges. Non-symmetric axial hinges were adopted to model the axial plastic hinges of concentrically bracings.
A uniform area load $g_{2k,f}$ = 2.5 kN/m^2 was assumed to model the composite floor system (i.e. realized by steel sheeting + concrete slab + screed + pavement), while a unitary weight $g_{2k,c}$ = 2.3 kN/m^2 was assumed for perimetral claddings. Live loads due to crowding (q_{ck} = 3.0 kN/m^2) and snow (q_{sk} = 0.48 kN/m^2) were also introduced in accordance with current Italian provisions [4].

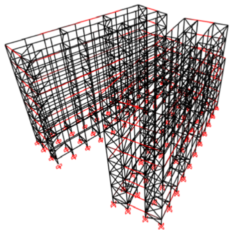


Fig. 3. Global numerical modelling of the selected case study.

The global behaviour of the selected case study under ultimate limit state (ULS) [8] gravity loads was preliminary assessed by means of static linear analyses. Thus, resistance and stability checks were carried out for each steel member to detect the highest demand/capacity (D/C) ratios.
Subsequently, non-linear static (i.e. pushover) analyses were performed according to both NTC2018 [4] and EN1998:3 [9] prescriptions. Namely, both modal and mass displacements distributions were considered, assuming two orthogonal directions for the application of lateral loads.

4 PERFORMANCE OF THE AS-BUILT STRUCTURE

4.1 Structural behaviour under gravity loads

The structural performance of the as-built structure under gravity loads is summarized in Table 2 in terms of highest stress characteristics and highest D/C ratios for each kind of structural member.

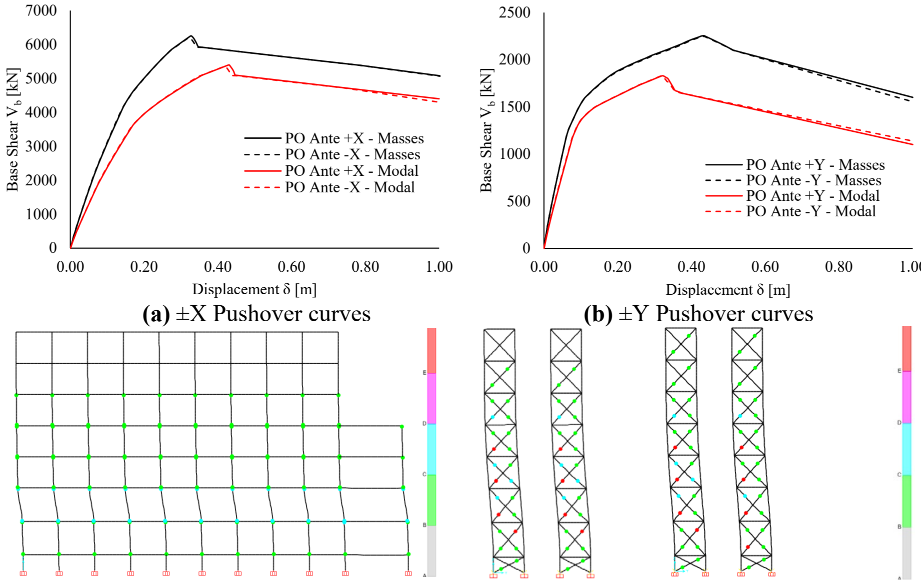
Table 2. Safety checks for the as-built structure under gravity loads.

Member	Check (Worst condition)	Design Demand	Design Capacity	D/C
Beams	Lateral Torsional Buckling	123.8 kNm	219.8 kNm	0.56
Col-umns	Lateral Torsional + Global Flexural Buckling	1156 kN + 36.4 kNm	2340.5 kN + 108 kNm	0.85
X-CBFs	Global Flexural Buckling	30.5 kN	38.2 kN	0.80
Y-CBFs	Global Flexural Buckling	29.1 kN	14.2 kN	2.05

It can be noticed that the as-built structure does not meet safety requirements under ULS gravity loads. For instance, Y-shaped CBFs exhibit significant instability problems (D/C = 2.05) in spite of the low design demand. Indeed, since the braces were originally designed as tension-only systems for lone wind loads, the buckling resistance of adopted angle profiles (L 80x40x4 mm) is rather low. Conversely, beams, columns and X-shaped braces are able to resist gravity loads, although significant D/C ratios are attained in case of columns (D/C_{max} = 0.85).

4.2 Structural behaviour under seismic loads

The global seismic behaviour of the as-built structure is summarized in Fig. 4 in terms of *i*) base shear vs. top displacement curves (i.e. pushover curves, hence also referred as “PO”) and *ii*) deformed structural configurations in correspondence of the peak base shear. Namely, while a total of eight PO curves are reported in Fig. 4a-b for the as-built structure (i.e., ±X, ±Y directions, Mass/Modal distributions) only the results related to worst cases (i.e. -X and -Y modal POs) are shown in Fig. 4c-d for the sake of brevity. Notably, the seismic behaviour of the as-built structure is unsatisfactory in both directions as the displacement capacity is unsufficient. Namely, a minimum imposed top displacement $\Delta_{top,min,x}$ of 0.43 m (-X Modal PO, drift: 1.9%) is attained in X-direction for the maximum base shear $V_{b,max,x}$ (5322 kN). Conversely, in Y-direction $\Delta_{top,min,y}$ = 0.32 m is reached (-Y Modal PO, drift: 1.4%) for $V_{b,max,y}$ = 1788 kN. Moreover, as shown in Fig. 4c, an undesirable global failure mechanism occurs in X-direction, i.e. a “soft-storey” collapse at the third floor.



(a) ±X Pushover curves **(b)** ±Y Pushover curves

(c) Deformed configuration (-X Modal PO) **(d)** Deformed configuration (-Y Modal PO)

Fig. 4. Results of non-linear static analyses for the as-built structure.

This outcome depends on the absence of any beam-to-column hierarchy. Indeed, main beams were made with the same deep hot-rolled IPN 320 profile at each storey, while hollow columns become thinner along the height of the building, as also described by the Authors in [1]. With respect to Y-direction structural behavior, it can be observed from Fig. 4d that a non-uniform distribution of plasticity along the building height is attained in correspondence of the peak base shear. This result is mainly due to the non-uniform variation of brace overstrength Ω_i (i.e. the ratio among the plastic axial resistance of a given brace $N_{pl,Rd,i}$ and the relative design seismic demand $N_{Ed,E,i}$) along the building height, which was uncontrolled at design stage. Indeed, in accordance with current seismic provisions [3,4,9], when designing new steel buildings featuring CBFs, for braces in tension it shall result that $\Omega_{max} \leq 1.25\ \Omega_{min}$ to promote simultaneous brace yielding. In the considered case study, braces always feature the same cross-section along the building height. Thus, the same excessive ratio $\Omega_{max}/\Omega_{min} = 4.61 > 1.25$ is attained for all types of CBFs. It is worth remarking that, although overstrength checks are not mandatory for existing buildings [9], their unfulfillment justifies the observed non-uniform yielding along the height. Finally, one can observe that, when the peak shear resistance is achieved, almost all braces subjected to compressive forces have already failed due to global buckling. This outcome is compliant with the original design philosophy, which accountend only for tensile braces to resist wind loads.

5 PERFORMANCE OF THE UPGRADED STRUCTURE

As reported in Section 4, the structural behaviour of the investigated case study is poor due to several structural inadequacies. Thus, proper global interventions were designed and numerically checked. As for ULS gravity loads, expressions for resistance and stability checks were used to derive minimum section properties X_{min} required to meet EN1993:1-1 [8] prescriptions (Eq. 1):

$$X_{min} = \max\left(\frac{S_{max,R} \cdot \gamma_{M0}}{f_{yk}}; \frac{S_{max,S} \cdot \gamma_{M1}}{\chi \cdot f_{yk}}\right) \tag{1}$$

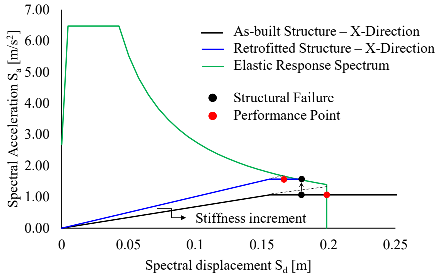
with $S_{max,R}$ and $S_{max,S}$ being the maximum stress characteristics which can induce collapse due to resistance and stability issues, respectively, $\gamma_{M0} = \gamma_{M1} = 1.05$ being the safety factors adopted for resistance and stability checks, respectively and $\chi \leq 1$ being the global instability reduction factor. Notably, as Eq. (1) ensures a satisfactory response only under gravity loads, further changes may be needed to satisfy seismic requirements. The seismic strengthening interventions were designed to enhance the capacity curve of the building by means of the N2 method as codified in EN1998:3 [9], thus meeting the design seismic demand. For instance, starting from as-built PO curves, equivalent bi-linear curves were derived by equating *i*) the ultimate displacement capacities and *ii*) the area underneath the curves. According to [7,9], the ultimate displacement capacity is achieved if at least one of two failure criteria is fulfilled, namely:

1. The ultimate rotation/displacement capacity is reached;
2. The base shear force drops below 0.80 $V_{b,max}$ on the degrading branch of the PO curve.

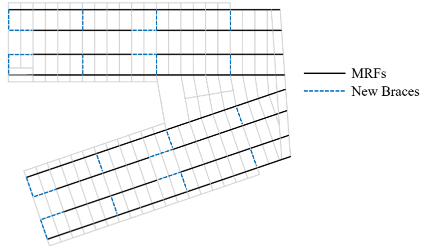
Consistently with assumptions of the current version of EN1998-1 [3], the achievement of ultimate displacement capacity in compressed axial hinges was not considered as a possible source of failure. Hence, bi-linear curves were reported in the Acceleration-Displacement Response Spectrum (ADRS) domain. For this purpose, a preliminary modal analysis was required. Indeed, with respect to translational mode shapes in X- and Y- direction, mass participation factors Γ_i were estimated (i.e., $\Gamma_{1,y} = 1.40$, $\Gamma_{2,x} = 1.32$) and used to scale bi-linear curves, thus reducing the structural response to the equivalent behaviour of a SDOF system. Performance points (PPs) for all considered combinations were obtained by intersecting the elastic branches of bi-linear curves with the site-depending elastic response spectrum (ERS). According to the N2 method, a given structure is safe against design seismic loads if the displacement demand at PP (D_{PP}) is equal or lower than the relative ultimate displacement capacity D_u. Contrariwise, in case of $D_{PP} > D_u$, one can derive the target increment of stiffness by imposing the occurrence of failure at the intersection with ERS (Fig. 5a). This condition ensures that $D_{PP} < D_u$ in the upgraded configuration, although no information are provided about changes in collapse mechanisms (i.e., new PO analyses are still needed).
For the selected case study, an unsatisfactory global seismic performance is shown in X-direction due to insufficient displacement capacity and high lateral deformability (see Fig. 5a, $D_{PP,X} = 0.20$ m > $D_{u,X} = 0.18$ m). Therefore, a target lateral stiffness increment of about 11200 kN/m was required. Such increment was supplied by means of new X-CBFs made with S355 RHS profiles (90x50x*t* mm, with $t = 3 \div 5$ mm). Conversely, Y-direction response meets EN1998:1 [3] requirements, although existing Y-CBFs are unable to resist gravity loads and have high lateral deformability. Thus, new X-CBFs were also located in Y-direction to increase both stiffness and resistance. The dynamic response of the upgraded structure was hence further regularized by symmetrically placing the new braces in both directions of the building plan, thus mitigating the influence of higher torsional modes of vibration (see Fig. 5b). Moreover, as the performance of the as-built structure proved to be poor due to MRFs "soft-storey" mechanisms, in the upgraded configuration, local interventions on MR joints were accounted for in compliance with results reported in [1]. Namely, introducing rib stiffeners and small cuts on beams' lower flanges allowed to restore local hierarchy criteria, with a reduced resistance equal to 0.55 times the pristine beams' plastic moment.

5.2. Structural behaviour under gravity loads

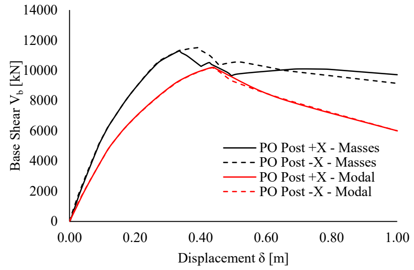
Table 2 summarizes safety checks for the upgraded structure in terms of highest design demands and D/C ratios. Notably, the strengthened structure now meets EN1993:1-1 [8] requirements against ULS gravity loads. For new X-CBFs an increment of compressive forces is obtained (+88%). Nevertheless, upgrading of cross-sections ensures ULS checks fulfilment ($D/C_{max} = 0.47$).



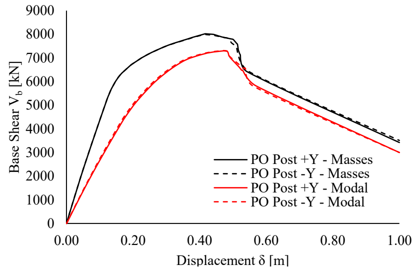
(a) Target stiffness increment (X-direction)



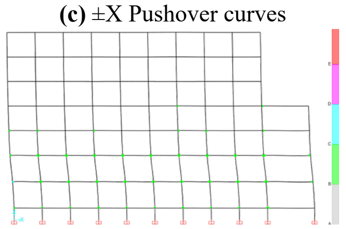
(b) Planar disposition of new X-CBFs.



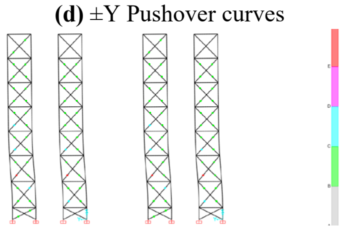
(c) ±X Pushover curves



(d) ±Y Pushover curves



(e) Deformed configuration (-X Modal PO)



(f) Deformed configuration (-X Modal PO)

Fig. 5. **(a)** ADRS checks, **(b)** planar view and **(c-f)** numerical results for the upgraded structure.

Table 3. Safety checks for the upgraded structure under gravity loads.

Member	Check (Worst condition)	Design Demand	Design Capacity	D/C
Beams	Lateral Torsional Buckling	124.1 kNm	219.8 kNm	0.56
Columns	Lateral Torsional + Global Flexural Buckling	1137.9 kN + 36.4 kNm	2340.5 kN + 108 kNm	0.84
New CBFs	Global Flexural Buckling	54.6 kN	117.4 kN	0.47

5.3. Structural behaviour under seismic loads

Figure **5c-f** depicts the global seismic performance of the upgraded structure in terms of PO curves and deformed structural configurations for $V_b = V_{b,max}$. Designed interventions induce a significant increment in terms of both lateral stiffness and resistance. Namely, $V_{b,max,x}$ = 10144 kN is attained in the worst case (i.e., -X Modal PO, +91% with respect to the as-built structure), while $V_{b,max,y}$ is at least equal to 7085 kN (i.e., -Y Modal PO, +296% with respect to the as-built structure). Moreover, N2 safety checks are now met in both directions. Indeed, the increased stiffness enables to highly reduce D_{PP}. In addition, upgraded MR joints display an enhanced rotational capacity, resulting in a beneficial effect in terms of $D_{u,X}$ (0.25 m, +39% with respect to the as-built structure). A considerable number of plastic hinges also forms in beams in upgraded MRFs (Fig. 5e), allowing an effective seismic energy dissipation. Nevertheless, although soft-storey collapses are now prevented, local interventions are not able to induce a global mechanism owing to P-Delta effects and lack of structural regularity along the height. As for the seismic behaviour in Y-direction, adopting variable cross-sections for new CBFs ensures a more uniform yielding of braces (Fig. 5f).

CONCLUSIONS

An existing non-conforming multi-storey steel building was selected as case-study. The global behaviour of the structure was assessed via refined FEAs. The following conclusions can be drawn:

1. The existing structure does not meet EN1993:1-1 [8] requirements for ULS gravity loads. Namely, Y-shaped CBFs exhibit premature failure due to global instability phenomena.
2. Seismic performance of the structure is poor due to insufficient displacement capacity.
3. Poor collapse mechanisms occur in both directions (i.e., soft-storey collapse and non-uniform braces yielding) as local hierarchy criteria and overstrength checks are not met.

The seismic strengthening interventions were designed and numerically checked. The following remarks can be pointed out:

4. New X-CBFs made with RHS profiles ensure a suitable behaviour under gravity loads.
5. New CBFs were designed with an extension of N2 method [3,9]. Accordingly, an increment of ≈11200 kN/m in terms of lateral stiffness is needed to meet safety requirements.
6. New CBFs improve stiffness and resistance in both directions (i.e., $V_{b,max,x}$ and $V_{b,max,y}$ increase of 91% and 296%, respectively). Simultaneously, $D_{PP,X}$ highly reduces (-20.8%);
7. MR joints were also upgraded to restore hierarchy criteria [1]. Global performance is positively affected, as $D_{u,X}$ increases (+39%) and a larger set of beams dissipate energy;
8. Further studies will be carried out to explore alternative upgrading strategies.

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KEYWORDS

Existing Structures, Steel Buildings, Global Seismic Analysis, FEM, Seismic Retrofitting.