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# SEISMIC RESPONSE ASSESSMENT OF AN EXISTING RC STRUCTURE RETROFITTED WITH STEEL EXOSKELETONS

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**Abstract**

*Several reinforced concrete (RC) structures built in Europe in the first decades after WWII do not meet the current safety standards, as those older buildings were built without any regard to the seismic actions and the structural details for ductility. Moreover, to reduce land use and environmental impact due to the construction of new buildings, governments are encouraging the upgrading and the reuse of older ones, rather than their demolition. Consequently, nowadays seismic retrofitting of existing structures is a very frequent request that civil engineers must handle. In general, such a challenge could be addressed combining member- and structure-level techniques. In principle, global (or structure-level) techniques may represent a feasible retrofit solution not only for structures characterized by low lateral stiffness, but also for buildings exhibiting a significant number of under-designed members with respect to the design seismic action suggested by modern Codes. Furthermore, these techniques could represent a more cost-effective strategy rather than the upgrading of existing members, especially when the potential disruption of occupancy and the replacement of non-structural elements are considered in the design process. More specifically, this paper aims to show how existing RC buildings can be seismically upgraded through retrofitting by adding external steel bracing systems referred to as "exoskeletons" as they are placed along the outer surface of the building. Although such a technique has a significant impact on the structural dissipation capacity and allows avoiding soft-storey mechanisms, its effectiveness is generally affected by the detailing of the braces and connections against buckling and post-buckling fracture. Consequently, an accurate model of the nonlinear response of the exoskeleton is as essential as representing the existing RC structure response in case of rare or very demanding earthquakes. In this context, this paper describes the modelling and analysis of an existing RC frame structure for which an exoskeleton was designed according to the current Italian code.*

**Keywords: Steel Exoskeletons, Seismic Upgrading, Existing RC structures, Strengthening, SCBF**

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# 1 INTRODUCTION

The European Environmental Agency recently stated that Europe has the highest proportion of land use on the globe, with respect to the other continents [1]. However, since land is a finite resource, governments are strongly encouraging the upgrading and the reuse (rather than the demolition) of older structures. Moreover, recent estimates of the Italian society of engineers outlined that huge investments are needed to enhance the safety of many existing Italian buildings, since only 2% of them were built after the year 2000, when technical standards began to impose more restrictive criteria [2].

To address this issue, the effectiveness of the available retrofit intervention techniques has been tested in several occasions during the last decades, resulting into the *fib* bulletin 24, which divides such interventions in two broad classes: on one side, member-level techniques are adopted to enhance both strength and ductility of deficient members , whereas structure-level ones result to be essential to increase the overall structural strength and stiffness [3].

Focusing on the structure-level side, a potential retrofit solution consists in providing some bays of the existing structure with steel braces. The new braces could be either placed inside the existing RC frames ("endoskeletons") or outside them creating completely new steel frames ("exoskeletons"). For instance, in case of endoskeletons steel bracing systems have been proposed exploiting buckling-restraining features of buckling-restrained braces or eccentric bracing ([4],[5],[6],[7]) so that the new braces could be placed close or even inside existing masonry walls.

The steel exoskeleton solution might be feasible when the building is sufficiently distant from other structures and it is particularly feasible to create integrated structural and energy consumption upgrading [8]. When it is feasible, the steel exoskeleton may represent a cost-effective strategy, because it avoids the potential disruption of the building occupancy and the partial demolition/reconstruction of existing non-structural elements.

When a steel exoskeleton is selected as a solution, then the new steel bracing system could be also made of a standard concentrically braced frame (CBF). In such a case, an adequate detailing of the braces and their connections against buckling and post-buckling fracture should be considered, because those phenomena can be detrimental for the global behaviour of the upgraded structure [9]. Despite the fact that several authors have been dealing with the design of such upgrading interventions [10], current seismic design codes do not yet provide specific design rules for a combined RC frame and steel CBF structure.

Therefore, the present paper aims at outlining how different design criteria can influence the effectiveness of the steel exoskeletons, considering some relevant issues that may arise in modelling their non-linear behaviour. After a summary of the models adopted for the braces and their connections (section 2), section 3 illustrates two alternative design procedures and, finally, section 4 reports a numerical application, comparing the outcomes deriving from those design criteria.

# 2 STEEL EXOSKELETON MODEL

The exoskeletons studied in the present work comprise special concentrically braced frames (SCBFs) placed parallel to the facades of the existing building. Due to their strength and stiffness, the added exoskeleton results into a significant modification of the seismic response of the existing building. Consequently, a detailed modelling of the nonlinear behaviour is required to capture accurately their role in withstanding the seismic action. More specifically, a proper model is required to predict the braced frame plastic mechanism and their ultimate failure mode, including the capability of the gusset plate connections to accommodate the inelastic deformations and end rotations due to brace buckling. The following subsections are devoted to explaining how such aspects have been implemented within the OpenSees framework [11].

## 2.1 Diagonal braces

Since the dissipative elements of a SCBF are represented by its braces, particular attention has been devoted to catching the correct way to represent their buckling and post-buckling behaviour. The braces are modelled adopting force-based elements with a fiber discretization of cross sections ([12]). Moreover, the brace members are discretized into a suitable number of “sub-elements”, to apply an equivalent geometrical imperfection with a sinusoidal shape, as reported in Figure 1. The maximum amplitude $e_0$ has been calibrated to obtain a brace buckling load from the numerical model that is close to that evaluated according to the buckling curves of Eurocode 3 [13].

The results from a mesh-sensitivity analysis for the elastic response are summarized in Table 1. The reported results suggest that the adoption of two elements might be enough to model the elastic response of the SCBFs. However, in the present work the braces are modelled using 4 elements to reproduce a sinusoidal shape of the equivalent geometrical imperfections and for a better representation of the nonlinear behavior. This is currently considered a sufficient compromise between accuracy and cost of the numerical analysis for the global system response. However, further study is going to be carried out from this perspective.

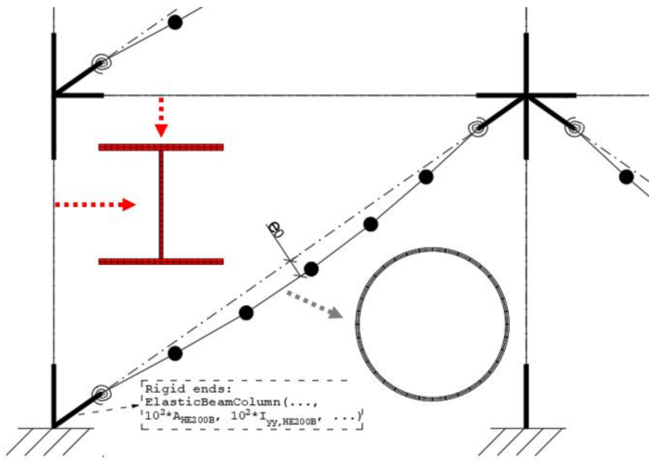



Figure 1 : Brace discretization

| *Number of elements for each brace member* | **Periods [sec]** | | |
|---|---|---|---|
| | **UX** | **UY** | **RZ** |
| *2* | 0.266 | 0.275 | 0.171 |
| *4* | 0.265 | 0.274 | 0.169 |
| *6* | 0.264 | 0.273 | 0.169 |
| *8* | 0.264 | 0.273 | 0.169 |

Table 1: Influence of the number of elements for each brace member on the structural periods

### 2.2 Gusset connections

Although typical modeling approaches for brace members assume that the gusset plate connection is fully restrained or fully pinned, experimental tests have shown that the real restraint condition is intermediate between those two simplified assumptions. Furthermore, an inadequate prediction of the bracing restraint condition can significantly affect the buckling capacity, tensile yielding, distribution of yielding, and post-buckling behavior of the dissipative elements of the frame [14].

Consequently, to enhance the model accuracy, the brace boundary condition were simulated through a nonlinear out-of-plane rotational spring, accounting for the gusset plate stiffness and strength in bending. The remaining part of the connection was assumed to have a rigid behaviour, as reported in Figure 1. Figure 2 shows a brace end connection. A knife plate was adopted in order to accommodate the buckling behavior in the same plane of the exoskeletons.

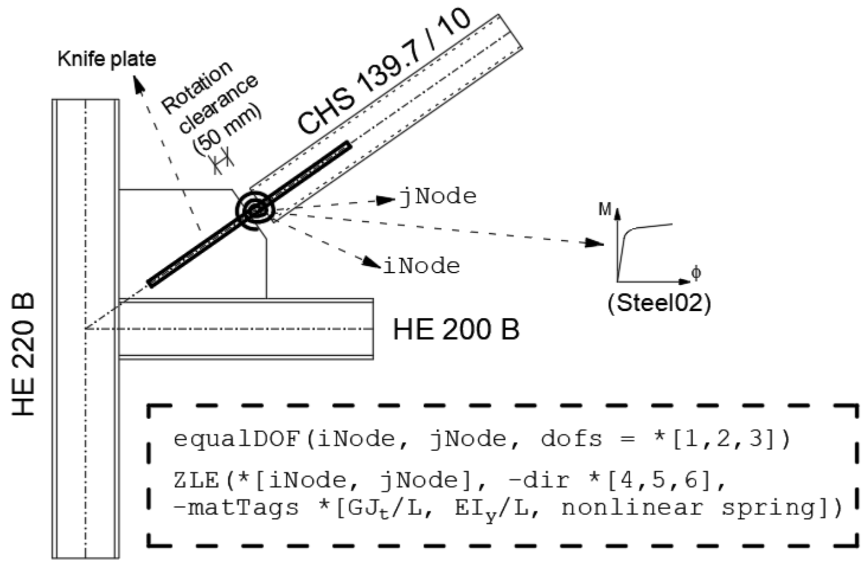



Figure 2 : Gusset to beam-column joint

## 3 DESIGN OF THE UPGRADING INTERVENTIONS

In the present paper two different scenarios are considered with the aim of pointing out how the adoption of a more refined method of designing the exoskeletons can influence the efficiency of the upgrading interventions. For the sake of simplicity, only a height-wise linear distribution of horizontal forces has been considered during the design phase.

### 3.1 Scenario 1: simplified Force-Based Design

Firstly, a simplistic code-compliant design approach is adopted, imagining that the SCBFs are the only structural components devoted to withstanding the seismic actions. This methos is intended to mimic what a practitioner would likely do while solving the design problem. To this aim, the design base shear force applied to the SCBF, is obtained from the design pseudo-acceleration at the “Severe Damage” limit (SDL) state at the SCBF fundamental period of vibration ($T_{1,SCBFs}$). Then, the design spectral ordinate is reduced by means of a q-factor equal to

4, which has been recently demonstrated to be an appropriate assumption [15]. Therefore, the design base shear force is calculated as follows:

$$V_{\text{b, d, Scenario1}} = (\lambda m_{tot}) \cdot \delta \cdot \frac{S_a(T_{1,SCBFs}, PGA_{SD})}{q_{SCBFs}} \tag{1}$$

where $\lambda$ = 0.85 is a correction factor included in the Italian code, $m_{tot}$ is the total mass of the structure for the seismic load combination, $\delta$ = 1.3 is an amplification coefficient representing the effect of accidental torsional effects, $S_a$ is the spectral ordinate, $q_{SCBFs}$ is the behaviour factor.

Secondly, after a simple linear elastic analysis, the dissipative zones (diagonal braces) are designed with respect to this load combination, neglecting the contribution of the braces in compression, while the non-dissipative zones (beams and columns) are subsequently designed according to the resistance hierarchy criteria.

### 3.2 Scenario 2: Design according to a pushover analysis

A more sophisticated design method might be sought and found suitable for a better performance assessment and/or saving the costs of the new structure. To this aim, a nonlinear static (pushover) analysis of the exiting RC structure could be exploited. In fact, the design of a seismic upgrading system is normally preceded by an assessment of the as-built structure. Then, a more rational design strategy can be defined, based on the following hypothesis:

- the overall displacement capacity of the existing structure is not affected by the introduction of the exoskeletons;
- the secant period $T^*$ of the structure equipped with the exoskeleton is still larger than $T_C$, meaning that the so-called “equal displacement rule” can be applied;
- the existing RC structure and the new steel exoskeletons (SCBFs) are described by two independent capacity curves, working in parallel, so that the total yielding strength of the upgraded structure can be expressed as $F_{y,tot} = F_{y,RC} + F_{y,SCBFs}$ .

Consequently, a different design criterion, as depicted in Figure 3, might be defined:

$$V_{\text{b, d, Scenario2}} = \alpha \cdot \Gamma_1 \cdot V_{\text{b, el, SCBFs}} = \alpha \cdot \Gamma_1 \cdot [m^* \cdot S_a(\delta^*_{\text{u, RC}}, PGA_{SD}) - F_{y,RC}] \tag{2}$$

where $\alpha$ represents the fraction of the total elastic base shear ( $\Gamma_1 \cdot V_{\text{b, el, SCBFs}}$ ) that the SCBFs have to withstand, $\Gamma_1$ is the modal participation factor for the 1st mode, $S_a$ is the spectral ordinate corresponding to $\delta^*_{u,RC}$ , that is the displacement capacity of the existing structure, $m^*$ is the mass of an equivalent SDOF system [16] and $F_{y,RC}$ is the yielding strength of the existing structure. It is straightforward that, under the above reported assumptions, a design equation can be expressed in terms of the minimum stiffness required to the equivalent SDOF, $k^*_d$:

$$k^*_{\text{d, Scenario2}} = \left(\frac{2.5 \cdot \eta \cdot S \cdot PGA_{SD} \cdot T_C}{2\pi}\right)^2 \cdot \frac{m^*}{(\delta^*_{\text{u, RC}})^2} \tag{3}$$

where $\eta$ is the damping correction factor, $S$ is the soil factor, $T_C$ is the upper bound of the constant acceleration portion of the design spectrum.

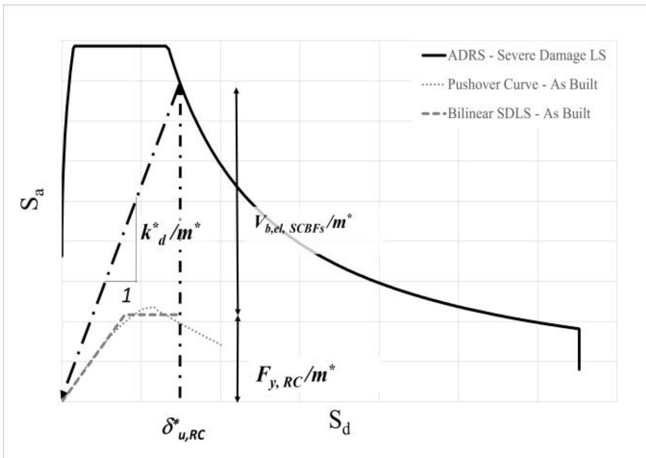



Figure 3 : Graphical representation of the design criterion based on the pushover response of the existing RC structure

# 4 APPLICATION

## 4.1 Structure presentation

The design of an archetype existing 4-storey building has been simulated, using the Codes and practice adopted in Italy during the 1970s. The building is assumed to be in a high seismic hazard region in Italy (Sant'Angelo dei Lombardi, AV, Italy). It has been consequently designed to withstand a minimum value of horizontal forces. The structure is characterized by 4 frames of 5 bays in the longitudinal (X) direction and by 4 frames of 3 bays in the transversal one (Z), having uniform bay width of 5.00 m and uniform interstorey height of 3.50 m. Table 2 summarizes the cross sections of the RC members.

| *Storey* | Columns [$cm^2$] | Beams - X [$cm^2$] | Beams - Y [$cm^2$] |
|---|---|---|---|
| *1st* | 35 x 35 | 30 x 60 | 30 x 60 |
| *2nd* | 30 x 30 | 30 x 60 | 30 x 60 |
| *3rd* | 30 x 30 | 30 x 60 | 30 x 60 |
| *4th* | 30 x 30 | 30 x 60 | 30 x 60 |

Table 2: RC members cross sections

To represent the typical materials utilized in the 1970s, a cylindrical compressive strength $f_{cm}$ = 25 MPa is assumed for concrete, while the average tensile strength of reinforcing bars is assumed equal to $f_{sm}$ = 220 MPa, representative of steel type FeB22k. The construction site is assumed to be Sant'Angelo dei Lombardi (AV, Italy) having a value of $PGA_{SD}$ = 0.266 g for the expected event at SD limit state and $PGA_{DL}$ = 0.082 g for the expected event at "Damage Limitation" (DL) limit state, considering a type "C" soil.

Firstly, a nonlinear static (pushover) analysis was carried out, adopting the N2-Method [16] to assess the performance of the existing RC structure. To this aim, a nonlinear FE model was implemented, utilizing *force-based elements* (distributed plasticity) for all the existing RC members, accounting for both mechanical and geometrical non-linearities. Fiber sections are associated to each member, considering the uniaxial materials *Concrete01* and *Steel02* available in the OpenSees library [11]. The brittle shear failures of RC members and joints were not explicitly included in the FE model, but the corresponding shear resistances were evaluated in the post-processing phase according to the current Italian Seismic Code [17].

### 4.2 Exoskeleton configurations

Figure 4 shows the FE model, highlighting that the connection between the RC and the steel structures was represented by a kinematic constraint (*equalDOF* in OpenSees) applied to the horizontal displacement components of the connected nodes in the two plan directions X and Y. To avoid interactions between the steel frames and the existing structure in bearing the vertical loads, the corresponding nodes are disconnected for the vertical displacement component. An appropriate design of the steel-to-RC connection has to be developed for the applicability if these kinematic constraints, but such a detail is out of the scope of this paper.

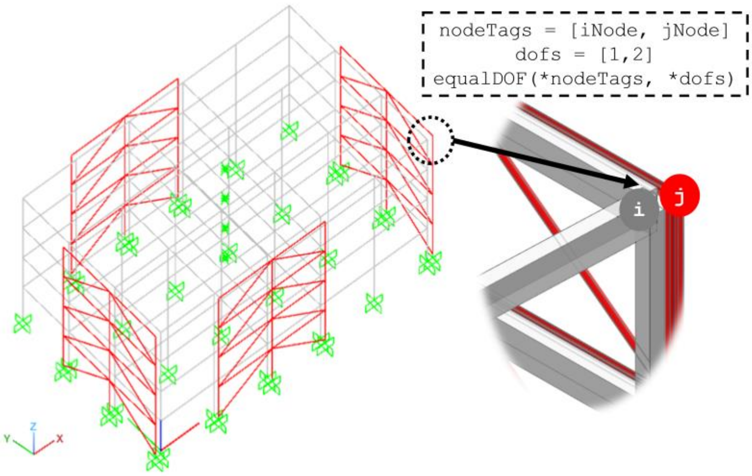



Figure 4 : 3D view of the FE model of the existing structure equipped with the steel exoskeletons

Table 3 summarizes the cross sections adopted for the exoskeletons. It is stressed that the design of the diagonal braces takes into account the design criteria reported in Sections 3.1 and 3.2, as well as code limitations to the brace slenderness $\lambda$ and overstrength ratios $\Omega$ in the frame elevation [18], as expressed by means of the following two equations:

$$\bar{\lambda} = \sqrt{\frac{N_{pl}}{N_{cr}}} = \frac{\lambda}{\pi}\sqrt{\frac{f_y}{E}} \leq 2.0 \tag{4}$$

$$\frac{\Omega_{\max} - \Omega_{\min}}{\Omega_{\min}} \leq 0.25 \tag{5}$$

where $N_{pl}$ is the plastic tension resistance, $N_{cr}$ the Eulerian buckling load and $\Omega$ the ratio between $N_{pl}$ and the corresponding axial force demand deriving from a linear analysis.

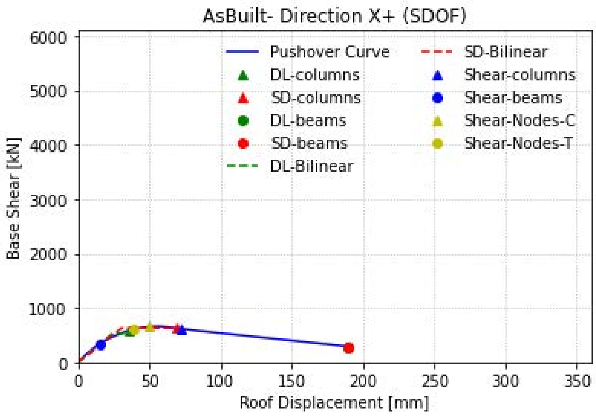
| *Storey* | **Brace** cross sections (**CHS type**) | | **Column** cross sections (**HE type**) | | **Beam** cross sections (**HE type**) | |
|---|---|---|---|---|---|---|
| | Scenario 1 | Scenario 2 | Scenario 1 | Scenario 2 | Scenario 1 | Scenario 2 |
| *1st* | 193.7 / 16 | 139.7 / 12.5 | 360 B | 220 B | 300 B | 200 B |
| *2nd* | 193.7 / 12.5 | 139.7 / 12 | 360 B | 220 B | 300 B | 200 B |
| *3rd* | 193.7 / 10 | 139.7 / 10 | 360 B | 220 B | 300 B | 200 B |
| *4th* | 193.7 / 6 | 139.7 / 5 | 360 B | 220 B | 300 B | 200 B |

Table 3: Exoskeleton members cross sections

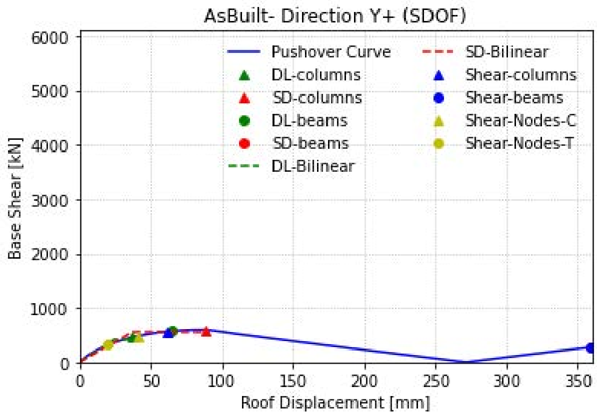
It is worth mentioning that the ratio between the design base shear forces associated to the two possible upgrading solutions is approximately equal to 1.50, having assumed $\alpha = 0.50$.

## 4.3 Resulting structural response

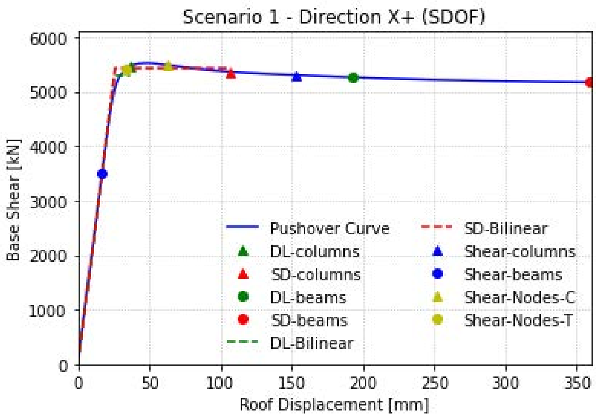
For the sake of simplicity only a linear (height wise) distribution of horizontal forces has been adopted for the pushover analysis, consistent with the design Eqs. (1) and (2). More specifically, Figure 5 (a-f) outlines the position of the various capacity points on the pushover curves of the existing structure (a-b) and the upgraded ones for Scenario 1 (c-d) and Scenario 2 (e-f). For the sake of brevity, only the results in the X+ and Y+ direction are reported herein.



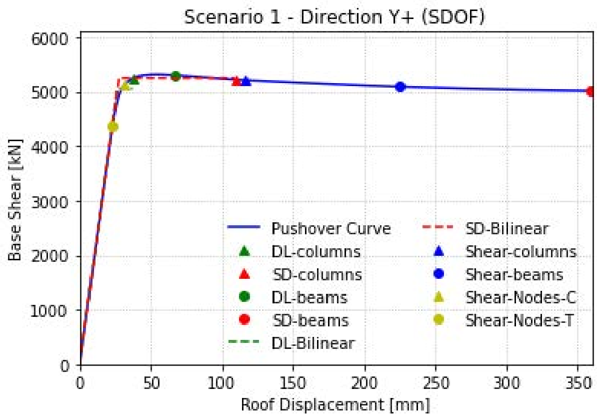

(a)




(b)




(c)

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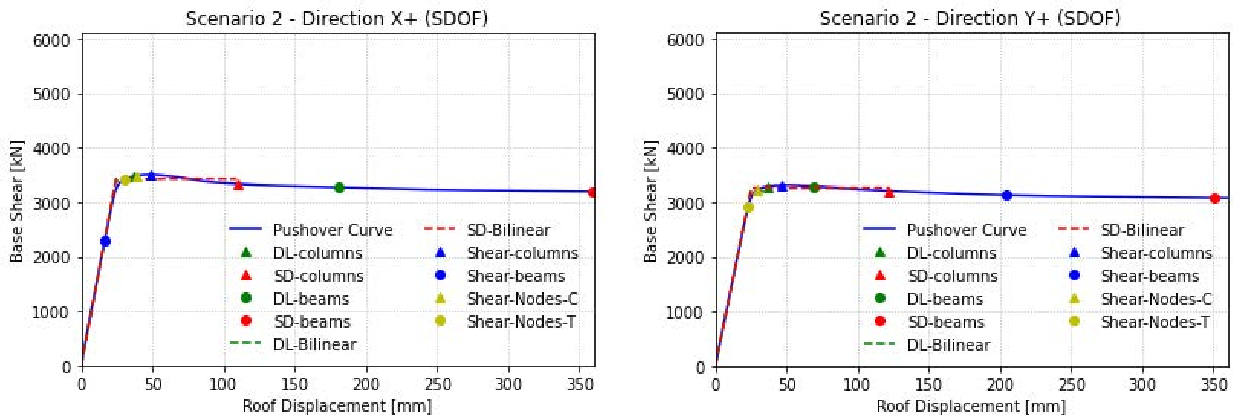
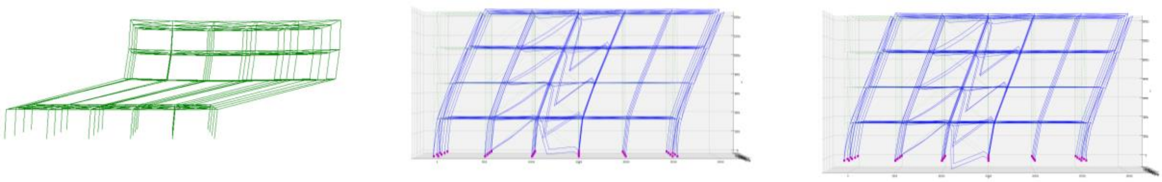



(e) (f)

Figure 5 : Pushover Curves, bilinearizations and Capacity Points of the existing RC structure (a-b) and the upgraded structures for Scenario 1 (c-d) and Scenario 2 (e-f)

Figure 5(a-b) shows that the existing structure is characterized by a global behaviour with a post-peak negative slope in the force-displacement relationship. This response corresponds to a soft-storey collapse mechanism involving the 2nd storey, as shown in Figure 6(a). The introduction of the exoskeletons leads to a more uniform interstorey drift distribution (Figure 6(b-c)), regardless of the adopted design procedure. This benefit derives also from the fulfillment of the braces design criterion expressed in Eq. (5), which tends to provide a uniform brace inelastic deformation along the frame elevation.



(a) (b) (c)

Figure 6 : Deformed shapes corresponding to the last analysis step for the X+ pushover analysis of the existing RC structure (a) and the upgraded structures for Scenario 1 (b) and Scenario 2 (c)

## 4.4 Capacity evaluation

The safety checks were performed with respect to the current Italian provisions for existing buildings [19]. Two relevant parameters were evaluated: the **safety index $\zeta_E$**, consisting in the capacity-over-demand ratio in terms of PGA, and the **Damage Extension Index (DEI)** accounting for the actual number of under-designed members (or unfavourable mechanisms) for each considered Limit State. More specifically, the elements chord rotation capacity was evaluated as suggested in the Eurocode 8, Part 3, Section A.3.2.2 [20]. Conversely,

the shear resistance of members was evaluated according to the Italian code provisions, considering proportionality to the chord rotation ductility ($\mu_\Delta^{pl}$). In particular, when $\mu_\Delta^{pl} \geq 3$ the expression of the shear cyclic resistance (Eurocode 8, Part 3, Section A.3.3.1) is adopted; when $\mu_\Delta^{pl} \leq 2$ the shear resistance is assumed equal to the maximum between the shear cyclic resistance and the shear resistance under vertical loads; for intermediate values of $\mu_\Delta^{pl}$ an interpolation is performed. Moreover, the following relationships have been utilized for checking shear failures of beam-to-column joints:

$$\sigma_{jc} = \frac{N}{2A_j} + \sqrt{\left(\frac{N}{2A_j}\right)^2 + \left(\frac{V_j}{2A_j}\right)^2} \le 0.5 f_{cd} \tag{6}$$

$$\sigma_{jt} = \left| \frac{N}{2A_j} - \sqrt{\left(\frac{N}{2A_j}\right)^2 + \left(\frac{V_j}{2A_j}\right)^2} \right| \le 0.3\sqrt{f_{cd}} \tag{7}$$

where $\sigma_{jc}$ and $\sigma_{jt}$ are the maximum compressive and tensile stress, $f_{cd}$ is the concrete compressive strength assumed for the brittle mechanisms, $A_j$ is the beam-to-column joint area, $N$ is the axial load acting on the column above the joint, $V_j$ is the joint shear demand (depending on the bending moments acting at the joint sides and on the shear force acting on the column above the joint).

Table 4, Figure 7 and Table 5, Figure 8 respectively report the structural capacities in terms of safety index $\zeta_E$ and Damage Extension Index (DEI). To summarize, only the worst values per each analysis direction are reported. For each safety check the minimum safety index and the maximum DEI between the two plan directions are highlighted with a bold character.

| **$\zeta_E$ = PGA$_D$ / PGA$_C$ [-]** | | | | | | |
|---|---|---|---|---|---|---|
| *Limit State* **Structure** | **As Built** | | **Scenario 1** | | **Scenario 2** | |
| | **X+ / X-** | **Y+ / Y-** | **X+ / X-** | **Y+ / Y-** | **X+ / X-** | **Y+ / Y-** |
| *DL - Ductile* | 0.91 | **0.83** | 3.30 | 3.15 | 2.29 | 2.32 |
| *SD - Ductile* | **0.51** | 0.56 | 2.86 | 2.72 | 2.43 | 2.16 |
| *SD - Brittle Beams* | **0.12** | 2.30 | **0.50** | 5.34 | **0.37** | 5.66 |
| *SD - Brittle Columns* | 0.54 | **0.39** | 3.57 | 2.88 | 1.01 | **0.93** |
| *SD - Joints Eq. (6)* | 0.81 | **0.27** | 1.34 | **0.93** | 0.79 | **0.59** |
| *SD - Joints Eq. (7)* | 0.30 | **0.14** | 0.96 | **0.70** | 0.60 | **0.48** |

Table 4: Comparison between the upgrading solutions in terms of safety index $\zeta_E$

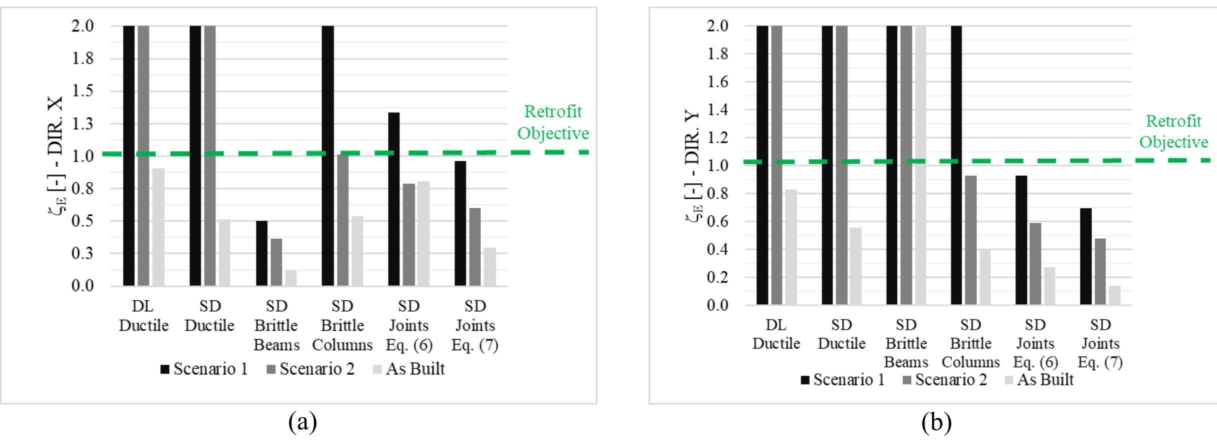
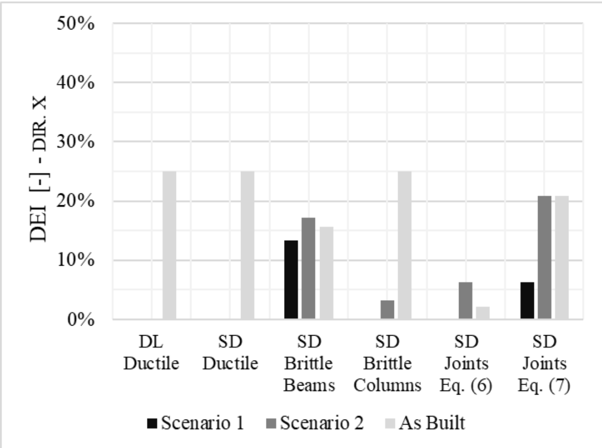


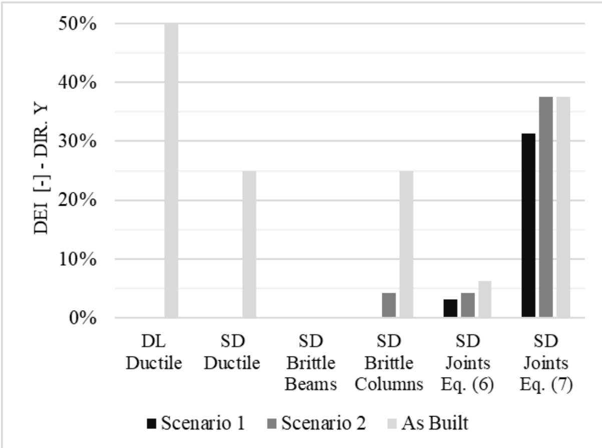

Figure 7 : Comparison between the upgrading solutions in terms of safety index $\zeta_E$ for X (a) and Y (b) direction

| Limit State\ **Structure** | **As Built** | | **Scenario 1** | | **Scenario 2** | |
|---|---|---|---|---|---|---|
| | **X+ / X-** | **Y+ / Y-** | **X+ / X-** | **Y+ / Y-** | **X+ / X-** | **Y+ / Y-** |
| *DL - Ductile* | 25 | **50** | 0 | 0 | 0 | 0 |
| *SD - Ductile* | **25** | **25** | 0 | 0 | 0 | 0 |
| *SD - Brittle Beams* | **16** | 0 | **13** | 0 | **17** | 0 |
| *SD - Brittle Columns* | **25** | **25** | 0 | 0 | 3 | **4** |
| *SD - Joints Eq. (6)* | 2 | **6** | 0 | **3** | **6** | 4 |
| *SD - Joints Eq. (7)* | 21 | **38** | 6 | **31** | 21 | **38** |

Table 5: Comparison between the upgrading solutions in terms of Damage Extension Index (DEI [%])




(a)




(b)

Figure 8 : Comparison between the upgrading solutions in terms of Damage Extension Index for X (a) and Y (b) direction

Observing the results, it is clear that the advantages of the steel exoskeletons are both in the enhancement of the structural capacity (Figure 7) and the reduction of the number of deficient members (Figure 8), except for the brittle mechanisms involving the beams and the tensile beam-column joint failure.

The exoskeleton modifies the global behaviour (Figure 6), both regularizing the drift distribution and modifying the RC frame internal actions. As a result, not only the number of failing members is generally reduced (Table 5), but also the location of plastic hinges is modified.

Finally, comparing the design criteria, the results show that Scenario 1 is generally characterized by higher values of the safety index and lower values of DEI. However, it is yet not sufficient to fully achieve the retrofit objective. Consequently, it may be advisable to design a less robust exoskeleton (as in Scenario 2) combined with a proper number of member-level techniques. Smaller braces will also limit the costs at the foundation level, which usually have a large impact on the entire cost of the retrofit system.

## 5 CONCLUSIONS

A proper nonlinear model of the steel exoskeleton is required to estimate properly the effectiveness of the examined upgrading technique in enhancing the structural performance of the existing structure under seismic loading. The combined effect of the capacity increase in terms of acceleration and the modification of the collapse mechanism in the existing RC structure will result in a synergistic effect, significantly enhancing the structural performance.

The outcomes deriving from the adoption of two different design criteria have been presented, demonstrating that the adoption of a more rational design criterion based on the assessment of the existing structure can help in avoiding an overestimation of the seismic demand, thus also reducing the cost of the required upgrading at the foundation level.

Nevertheless, it is apparent that the adoption of this structure-level technique alone is not sufficient to fulfill all the retrofit objectives, because of the need to consider those brittle mechanisms representing the main weaknesses of the existing RC structure, i.e. shear failure of beams and shear tensile failure of joints for the considered case study.

Consequently, the results lead to conclude that a complete seismic retrofit of the existing structure requires collaborative efforts, with some member-level interventions in addition to the steel exoskeleton.

Further developments of the present work will be devoted to further explore the effectiveness of a combined intervention, by both refining the design and improving the numerical model.

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