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Towards a rational strategy for seismic retrofitting of RC frames by combining member- and structure-level techniques

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ABSTRACT: Existing reinforced concrete (RC) frames are generally affected by deficiencies related to the lack of capacity of either single members or the entire structural system. Seismic assessment procedures are not only needed for unveiling the key weaknesses of the seismic response, but they can also be helpful in choosing the most efficient retrofitting solutions. As a matter of principle, such solutions can be grouped into two broad. On the one hand, the former are primarily aimed at enhancing strength and/or ductility of single deficient members by means of various technical solutions. On the other hand, the latter are intended at reducing the seismic demand by introducing new members or sub-structures. This paper describes the key features of a rational strategy for the possible combination of member- and structure-level techniques to achieve the optimal retrofitting solution. A parametric application is also proposed to highlight the potential of such a strategy with respect to the current practices, often based on using either of the two classes of retrofitting techniques.

1 INTRODUCTION

Vulnerability assessment of existing reinforced concrete (RC) structures generally unveil the weaknesses of their seismic response and represents the basis for designing appropriate retrofitting solutions (Kaplan et al, 2010). In principle, retrofitting techniques can be grouped into two broad classes: *member-level* and *structure-level techniques* (*fib* Bulletin 24, 2003). On the one hand, the former are aimed at enhancing strength and/or ductility of single members of the existing structures by means of alternative technical solutions, such as confinement, jacketing and so on forth. On the other hand, the latter are intended at reducing the seismic demand on the existing structure by introducing new members and/or sub-structures, such as shear walls or steel bracings that work in parallel with the existing structural system (Faella et al, 2014). This paper formulates a conceptual strategy for obtaining optimal seismic retrofitting solutions by combining member- and structure-level techniques. A case study dealing with the seismic retrofitting of a RC frame originally designed for gravitational loads only is presented: the results of a series of nonlinear time history analyses aimed at pointing out different aspects of the seismic response regarding the aforementioned alternative retrofitting solutions. A series of concluding remarks are reported in conclusions.

2 OUTLINE OF A RATIONAL RETROFITTING STRATEGY

According to the European and Italian codes (Eurocode 8, NTC 2008), various Limit States (LS) should be checked in seismic retrofitting of RC structures. In this paper, the LS of Damage Limitation (SLD), related to a seismic event whose Probability of Exceedance (PoE) equal to

63% in 50 years, and the Limit State of Life Safety (SLV), corresponding to an event with 10% PoE in 50 years, are taken into account. The two following parameters are firstly considered:

$$\alpha_e = \frac{PGA_{SLD}}{PGA_{63\%}}, \qquad \alpha_u = \frac{PGA_{SLV}}{PGA_{10\%}}, \tag{1}$$

in which PGA_{LS} is the peak ground acceleration resulting in the structure to attain a LS and $PGA_{63\%}$ and $PGA_{10\%}$ characterise the design spectra for the given LS and the corresponding PoE.

A further parameter is introduced herein for pointing out whether the structural deficiency is limited to few under-designed members or widespread to a wide number of members as a result of an unfavourable failure mechanisms. This two opposite situations, as well as all the intermediate ones, can be described by a parameter based on the number of members n_{LS} in which demand exceeds capacity as a global displacement $\Delta_{d,LS}$ is imposed on the structure; a non-dimensional measure called *Damage Extension Index* (DEI) can be introduced:

$$DEI = \frac{n_{LS}}{n_{tot}} \tag{2}$$

where n_{tot} is the total number of members. All the quantities in eq. (1) and (2) can be easily determined through the N2-Method (Fajfar, 1999). A rational retrofitting strategy is outlined in the following subsections starting from these parameters and introducing two main assumptions: (i) structure-level techniques do not modify the global displacement capacity of structures, and (ii) the displacement capacity can only be increased by means of member-level techniques.

2.1 Key parameters and assumptions

The structural behaviour can be generally identified by the elastic-plastic relationship shown in Figure 1 that outlines the response of the structural systems under consideration. The behaviour of the bracing structure used as a structure-level technique can be described through its yielding strength $F_{y,b}$, lateral stiffness K_b and, optionally, its overstrength factor $\Phi_{b,ov}$ (Figure 1).

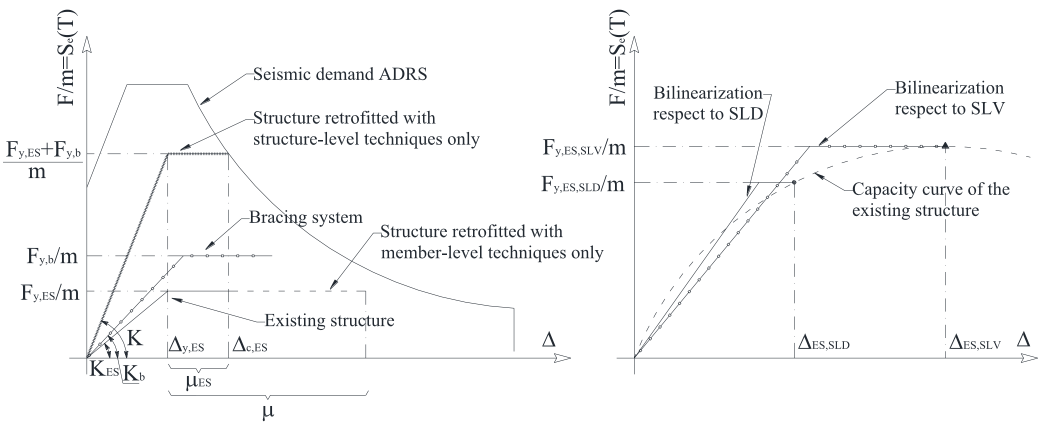


Figure 1. Idealised capacity curves for the structural systems involved in the rational retrofitting strategy.

The symbols referring to properties of the existing structures are denoted with the subscript "ES", while the ones of the generic sub-structure realising the structure-level intervention take the subscript "b". Consequently, the following non-dimensional parameters can be defined:

- $\Phi = F_{y,b}/F_{y,ES,SLV}$ ratio between the lateral strength of the bracing structure and the resisting base shear of the existing structure $F_{y,ES,SLV}$ (both referred to the SLV);

- $\kappa = K_b/K_{ES,SLV}$, bracing-to-existing-structure stiffness ratio;
- $\Phi_{ov,ES} = F_{y,ES,SLV}/F_{y,ES,SLD}$ overstrength ratio for the existing structure;
- $\kappa_{ov,ES} = K_{ES,SLV}/K_{ES,SLD}$ secant stiffnesses ratio for the structure at LS of SLD and SLV;
- $\mu_{ES,LS} = \Delta_{c,ES,LS}/\Delta_{y,ES,LS}$ displacement ductility of the existing structure (LS=SLD, SLV).

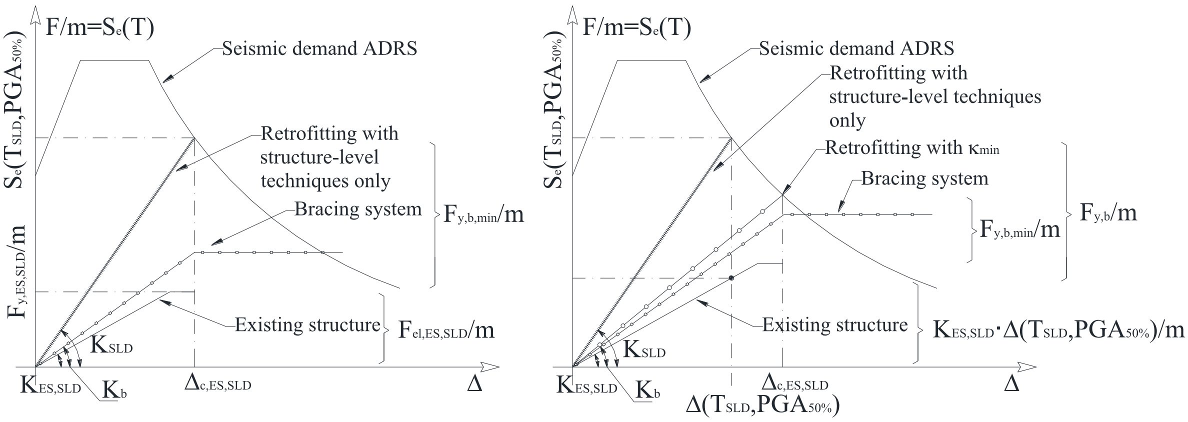
2.2 Retrofitting objective at SLD

The retrofitting objective at SLD can be achieved through structure-level techniques by reducing the demand on the existing structure. The stiffness parameter κ of the bracing sub-structure can be determined by imposing that the displacement demand $\Delta_{d,SLD}$ at SLD is not higher than the capacity $\Delta_{c,SLD}$ of the structure, that is assumed equal to $\Delta_{c,ES,SLD}$. Figure 2 represents this condition and after some analytical transformations, the following relationships can be determined for κ (Faella et al., 2008):

$$\kappa \geq \kappa_{\min} = \kappa_{ov,ES} \cdot \left(\frac{1}{\alpha_e^2} - 1 \right). \tag{3}$$

Moreover, the following limitation should be imposed to the strength parameter Φ for the retrofitted structure to respond elastically at the LS of SLD (Faella et al., 2008):

$$\Phi \geq \frac{\Phi_{\min}}{\alpha_e} \cdot \frac{\kappa/\kappa_{\min}}{\sqrt{1+\kappa/\kappa_{ov,ES}}}. \tag{4}$$



a) minimum stiffness of the bracing system κ_{min} b) stiffness of the bracing system $\kappa > \kappa_{min}$

Figure 2. Retrofitting objective at SLD.

2.3 Retrofitting objective at SLV

The retrofitting objective at SLV can be met as a balance between capacity and demand:

$$\Delta_{d,SLV} \leq \Delta_{c,SLV}. \tag{5}$$

As a matter of principle, the displacement capacity $\Delta_{c,SLV}$ of the retrofitted structure can be expressed as a function of the corresponding capacity $\Delta_{c,ES,SLV}$ of the existing structure through a magnification factor k_μ related to the adopted member-level technique:

$$\Delta_{c,SLV} = k_\mu \cdot \Delta_{c,ES,SLV}. \tag{6}$$

Figure 3 shows the effects of member-level techniques on the ductility of members and the structure-level one on the structural demand.

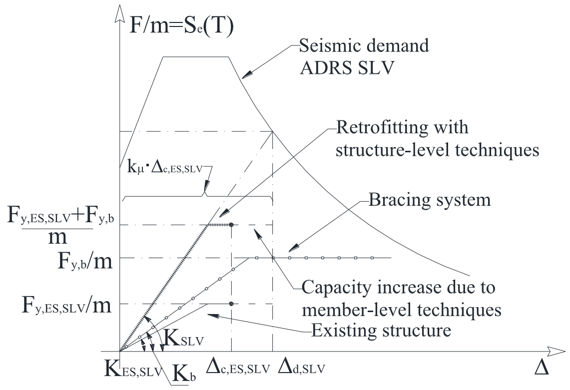


Figure 3. Retrofitting objective at SLV with required increase of capacity k_μ.

Then, some analytical transformations lead to these relationships (Faella et al, 2008):

$$k_\mu \geq \frac{\sqrt{\frac{\Phi^2}{\kappa}+1}}{(1+\Phi)\cdot\alpha_u} \tag{7}$$

which emphasises the relationship between the required increase in displacement capacity k_μ and the properties of the bracing system represented by κ and Φ. As it can be seen, a value of $k_\mu=1/\alpha_u$ is simply obtained in the case of retrofitting with member-level techniques only. Conversely, a retrofitting intervention based on structure-level techniques only should not require further increase in displacement capacity and, then, an upper bound κ_{max} can be defined for the stiffness parameter κ by imposing that the right member of eq. (7) is equal to one:

$$\kappa \leq \kappa_{max} = \frac{\Phi^2}{(1+\Phi)^2\cdot\alpha_u^2-1}. \tag{8}$$

The number n of required member-level interventions basically depends on the value of the DEI assessed on the existing structure. A possible analytical expression of the relationship between n and DEI is proposed in (Faella et al., 2008) and omitted herein for the sake of brevity.

2.4 Definition of a rational retrofitting strategy

In principle, a retrofitting strategy conceived by combing the two techniques mentioned above can be conceptually defined by the three parameters κ, Φ and k_μ. Since two relationships (eq. (4) for SLD and eq. (7) for SLV) were derived, seismic retrofitting is an undetermined problem, as it is described by three unknowns and only two equations: if proper range [κ_{min}, κ_{max}] exists, infinite solutions can be found as κ ranges therein. Hence, an optimal one can be found by minimizing an objective function, namely the cost function conceptually described in eq. (9):

$$C(\Phi,\kappa,k_\mu) = C_\Phi\cdot\Phi + C_\kappa\cdot\kappa + C_\mu\cdot(k_\mu-1)+C_0, \tag{9}$$

where C_Φ and C_κ are related to the type of structure-level technique adopted for retrofitting and, then, to Φ e κ, respectively. Conversely, in principle, C_μ is related to the cost of the single member-level intervention. The optimization problem can be formally stated as follows:

$$\bar{\kappa} = \arg\min\left[C\left(\Phi(\kappa),\kappa,k_\mu\left(\kappa,\Phi(\kappa)\right)\right)\right] \tag{10}$$

in which the solution has to be found within the range [κ_{min}, κ_{max}].

3 PARAMETRIC ANALYSIS

An existing four-storey structure, designed for only gravitational loads, was considered as a case study: A simulated design procedure was carried out according to the codes and practice in force in Italy in '60s and '70s of the past century (Regio Decreto, 1939; Santarella, 1966). To this end, a cylindrical compressive strength f_{cm}=16 MPa was taken into account for concrete, while steel type FeB22k (medium tensile strength f_{sm}=220 MPa) was considered for rebars. Further details about the structure considered as a case study can be found in Lima et al. (2013).

The structure under consideration was supposed to be located in L'Aquila (Italy) and, hence, a value of a_g=0.30g was derived for the expected event at SLV and Soil Class B was considered according to the Italian Seismic Code (NTC, 2008). The seismic assessment of the existing structure was firstly carried out by means of pushover analyses and N2-Method (Fajfar, 1999). To this end, a non-linear structural model was developed in OpenSEES (Mazzoni et al, 2007) by modelling beam and column through distributed plasticity elements for taking into account both geometric and mechanical non linearity: Concrete01 and Steel01 models were adopted for materials (Mazzoni et al, 2007). For the sake of brevity, only the response in the longitudinal direction and the seismic performance for the LS of SLV are reported in the following.

The assessment phase led to quantifying the parameters α_u=0.392 and DEI=0.500 related to the SL of SLV. Therefore, the strengthening intervention of the structure was conceived by considering X-shaped steel bracings (as a structure-level technique) and FRP-based confinement of columns (as a member-level technique). Three combinations were examined in this study:

- Solution N.1 was based on introducing steel bracings with a lateral stiffness $\kappa = \kappa_{max}$;
- Solution N.2 was based on combining member- and structure-level techniques: the steel bracing was designed for a stiffness $\kappa \cong 0.5 \cdot \kappa_{\max}$ a number of columns were confined;
- Solution N.3 was only based on FRP-confinement intended at obtaining a capacity increase factor $k_\mu=1/\alpha_u$ as obtained from eq. (7) for $\kappa \rightarrow 0$.

Table 1 reports the details of steel members used in the retrofitting intervention N.1 and N.2.

Table 1. Steel bracings profiles of the sub-structure in the interventions N.1 and N.2

Retrofitting intervention	1^{st} storey	2^{nd} storey	3^{rd} storey	4^{th} storey
N.1 (Structure-level techniques)	HE 180 A	HE 180 A	HE 140 A	HE 100 A
N.2 (combined member- and structure-level techniques)	HE 100 A	HE 100 A	HE 100 A	HE 100 A

High modulus (390000 MPa) unidirectional carbon fibres web (600 g/m^2) with thickness 0.329 mm is considered for confinement in Solution N.2 and N.3: 68 weak columns were confined for the N.3 solution with variable numbers of layers, while only 32 ones were actually confined in N.2 with one layer. Steel bracings were modelled through force-based distributed plasticity finite elements (Mazzoni et al., 2007) and an equivalent geometric imperfection EN 1993-1-1 (2005) was assigned to steel bracing for simulating their non symmetric structural response in tension and compression. The effects of the FRP and steel confinement were taken into account increasing the ductility of concrete according to CNR DT 2000 (2004) and Paulay and Priestley (1992), respectively. Nonlinear static analysis and N2-Method (Fajfar, 1999), as well as Incremental Dynamic Analysis (IDA) were carried out on the retrofitted structures. For the latter, 20 natural signals were selected from the PEER Database (2010) and duly scaled to obtain a pseudo-acceleration acceleration $S_a(T)$ at the fundamental period T of the three structures ranging between 0 and 5 m/s^2.

4 RESULTS IN TERMS OF RELEVANT RESPONSE PARAMETERS

The seismic analyses of the retrofitted structures, carried out according to the N2 Method, demonstrate that all the three solutions result in a seismic demand smaller than the corresponding capacity. Figure 4 testifies this result for the structures retrofitted according to the solution N.1 (on the left) and N.3 (on the right), while the N.2 is omitted herein due to space constraints. The retrofitting solution N.1 provides stiffness higher than the one of the existing structure and results in lower displacement demand. The retrofitting solution N.3 increases the displacement capacity of the structure while stiffness does not change. For both solution N.1 and N.3 the Limit Demand Spectrum is greater than the demand one demonstrating the achievement of the seismic retrofitting.

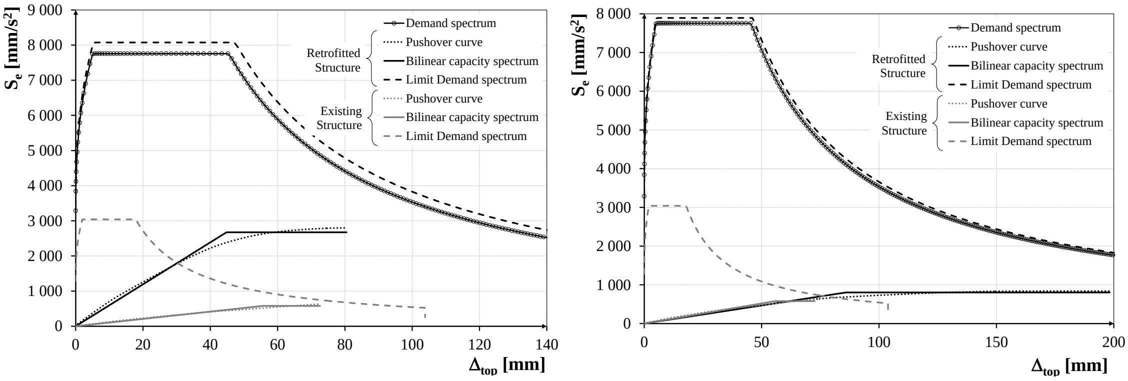


Figure 4. Seismic analysis of the retrofitted structure (solution N.1 on the left and N.3 on the right).

Moreover, Figure 5,a shows the unit costs determined for the three solutions: it shows that the Solution N.2 results in the lower unit cost, due to the synergetic action of the two techniques. Secondly, nonlinear static analysis and the N2 Method were applied for determining the value of α_u for the three retrofitting structures (Figure 5,b). As expected, all three solutions led to α_u higher than the unity (which means that the corresponding displacement demand does not overcome the available capacity); however, a significantly higher value (which implies a higher safety margin) was determined for the so-called Solution N.2.

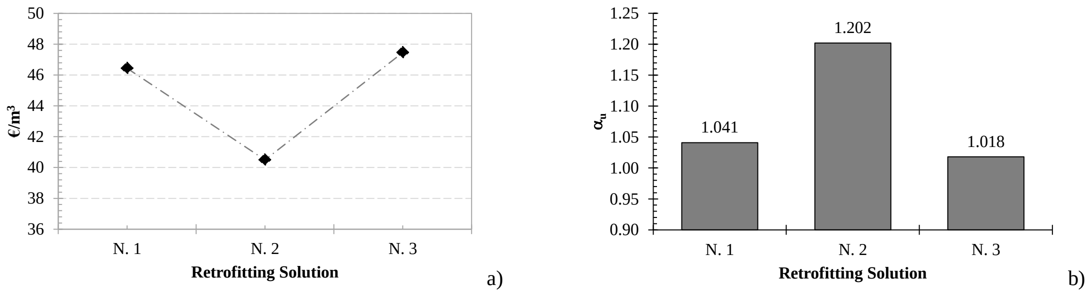


Figure 5. Retrofitted structures: a) initial unit costs and b) resulting values of α_u

Beyond this preliminary assessment of the considered retrofitting solutions, a wider set of parameters should be considered to have a more comprehensive idea about the multifaceted aspects of their seismic response. Such parameters can be better defined by analysing the output of the nonlinear time-history analyses which were carried out for determining the so-called IDA curves. The maximum floor acceleration is the first parameter considered in this study, as it is supposed to control the damage possibly induced in “non-structural” (acceleration sensitive) components. Figure 6,a shows the maximum acceleration at the roof level for the solution N. 1

and depicts both the response obtained for the single accelerograms and the resulting median values. Moreover, the comparison in terms of floor acceleration corresponding to a pseudo-acceleration $S_a(T)=4.00\ m/s^2$ of the three retrofitting solutions is, then, depicted in Figure 6,b.

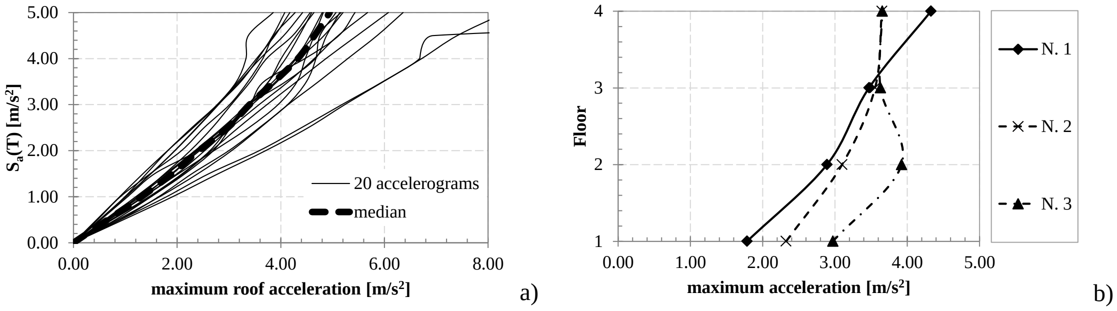


Figure 6. a) Maximum roof acceleration from IDA of the retrofitting solution N.1; b) Maximum floor accelerations of the three retrofitting solutions

The axial load at basis of the columns is another relevance response parameter, as it is related to the actions transferred at the foundation level. Figure 7 reports the values of the axial loads at the basis of both a central (namely, the ones adjacent to the steel bracing in solutions N. 1 and N. 2) and a corner column. As expected, the retrofitting solutions based on using the structure-level techniques (i.e. N.1 and N.2) led to higher values of axial forces which would likely require significant (and expensive) interventions at the foundation level. Also in this case, Figure 7,a shows the significant reduction in terms of forces transferred to the foundations in the case of the combined member- and structure-level retrofitting.

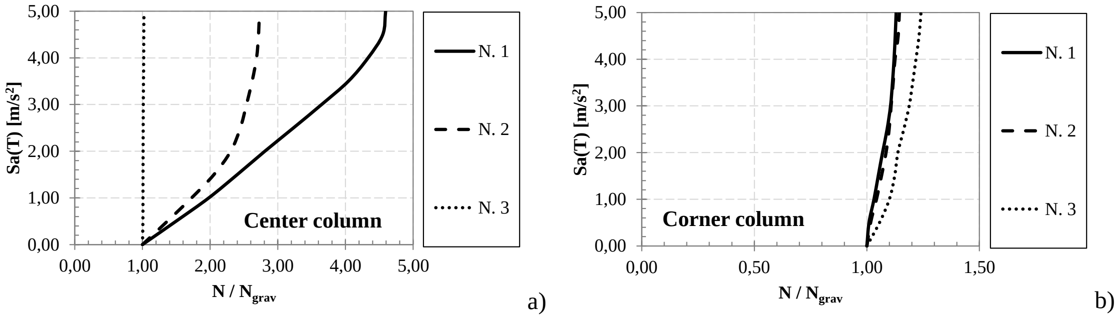


Figure 7. Maximum axial loads: a) central columns and b) corner column

A further aspect of the dynamic response is related to its record-to-record variability quantified via the dispersion coefficient β_D of the displacement D, which controls the seismic reliability of structures (Pinto et al., 2004). Table 2 reports the numerical values obtained for the three retrofitting solution under consideration and highlights the optimal ones in bold fonts: as a matter of fact, in most cases the combined solution (N.2) results in the optimal performance.

Table 2. Values of the relevant parameters obtained in this study

Retrofitting solution	α_u	Initial unit cost (€/m³)	Roof acceleration (m/s²) [$S_a(T)$=4.00 m/s²]	Axial load at foundation (kN) [$S_a(T)$=4.00 m/s²]	β_D [$S_a(T)$=4.00 m/s²]
N.1	1.041	46.44	4.33	2924.90	**0.34**
N.2	**1.202**	**40.50**	**3.65**	1812.87	0.48
N.3	1.018	47.46	3.66	**687.14**	0.59

5 CONCLUSIONS

This paper presented a general strategy for seismic retrofitting of existing buildings as a combination of member- and structure-level techniques. Particularly, the strategy consists of an optimisation problem aimed at quantifying the three parameters conceptually describing the retrofitting solution in terms of stiffness and strength of a structure-level technique (i.e. steel bracings) and the increase in global displacement capacity induced by member-level techniques (i.e. confinement of critical RC sections).

The proposed application to a relevant case-study highlighted that combining member- and structure-level techniques can be more effective and efficient than realising the seismic retrofitting intervention by either of the two techniques.

Finally, this paper was intended as a contribution to highlighting some aspects, often neglected or disregarded, of the multifaceted seismic response of retrofitted structures. The quantities considered in this study can be considered as objective functions in a possible multi-criteria optimisation for retrofitting. In this light, the possible coupling of member- and structure-level technique revealed its potential in optimising the resulting seismic response in terms of the various quantities relevant features of structural response analyses in this study.

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