A RATIONAL STRATEGY FOR SEISMIC RETROFITTING OF RC EXISTING BUILDINGS

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ABSTRACT :

The paper addresses the topic of seismic behavior of existing RC structures presenting a general strategy for combining member- and structure-level techniques to achieve a cost-effective retrofitting solution. First of all, two key vulnerability parameters are introduced; then the basic equations describing the retrofitting intervention within the mentioned strategy are presented. Finally, two sample applications point out the key features of the retrofitting solution which can be achieved through such a general strategy for two different buildings.

KEYWORDS: Existing structures, Vulnerability, Retrofitting

1. INTRODUCTION

Seismic assessment of non-seismically designed RC structures generally points out their structural deficiencies related to a general lack of strength and ductility of either the most engaged members or the structural system as a whole. Retrofitting is generally a need for such structures and various technical solutions can be considered for improving their seismic performance according to suitable objectives basically depending on the building use. Those possible technical solutions for seismic retrofitting of existing buildings can be classified into two broad classes collecting on one hand the *member-level techniques* and on the other one the *structure-level techniques* (*fib*, 2003a). The former ones can be adopted for enhancing the seismic capacity of the most deficient members by means of various technical solution such as confinement, jacketing and so on; the latter ones are rather aimed to reduce seismic demand on the existing structure as a whole by means of new members and substructures, such as shear walls or steel bracings, working in parallel with the existing one.
Although plenty of researches have been carried out to investigate the behavior of under-designed RC members and structures repaired by one of the above techniques, no general strategy has been still proposed for choosing and possibly combining those techniques to obtain the most efficient and effective solution for seismic retrofitting. Such strategy should be strictly connected with a consistent procedure for seismic vulnerability assessment of structures aimed to quantify a reasonably limited number of parameters representing the diagnosis of the structure at hand. Pushover-based procedures, such as those based on N2-Method (Fajfar, 2002), are widely accepted to reproduce the key features of the non-linear response of existing structures under seismic actions. Two complementary measures for describing such a response in terms of seismic vulnerability will be introduced and commented in the first of the following paragraphs as a preliminary step toward the formulation of a rational strategy for seismic retrofitting of structures which is the key subject of the paper.

2. VULNERABILITY ASSESSMENT: TWO KEY PARAMETERS

Performance-Based approach should be followed in seismic vulnerability assessment of existing structures and, for instance, the following Limit States (LS) can be introduced (OPCM 3362, 2004):
- Limit State of Damage Limitation (DL), to be checked under a seismic action whose Probability of Exceedance (PoE) is 50% in 50 years;
- Limit State of Severe Damage (SD), related to a seismic event whose PoE is 10% in 50 years;
- Limit State of Near Collapse (NC), related to an event with 2% PoE in 50 years.

It is worth to precise that the reference period can be even longer (or shorter) than 50 years as reported above, depending on the type of structure and its expected life time (EC8, 2004). Two complementary parameters have been proposed by Faella et Al. (2006) for describing seismic performance of buildings. The first measure is a demanded-to-available ratio in terms of displacements or forces. In particular, the Italian Code (OPCM 3362, 2004) introduces a similar parameter in terms of peak ground accelerations PGA_{LS} resulting in the structure to attain the given LS and the value $PGA_{PoE,LS}$ characterized by the required PoE:

$$\alpha_e = \frac{PGA_{DL}}{PGA_{50\%}} , \qquad \alpha_{u,SD} = \frac{PGA_{SD}}{PGA_{10\%}} , \qquad \alpha_{u,NC} = \frac{PGA_{NC}}{PGA_{2\%}} . \tag{2.1}$$

A further parameter should be introduced for pointing out whether the structural deficiency is due to either few under-designed members or widespread to a huge number of members as a result of an unfavorable 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:

$$\eta_{LS} = \frac{n_{LS}}{n_{tot}} , \tag{2.2}$$

n_{tot} being the total number of members (namely, plastic hinges in a lumped plasticity model). All the quantities in equations (2.1) and (2.2) can be easily derived through the N2-Method (Fajfar, 2002).

3. FORMULATION OF A GENERAL STRATEGY FOR SEISMIC RETROFITTING

The values of the two vulnerability parameters defined above (namely, α_{LS} and η_{LS}) can lead the choice of the best retrofitting intervention as a combination of *member-* and *structure-level* techniques. As a matter of principle, various technically feasible solutions can be generally conceived by varying the extents of the two mentioned techniques. Under the qualitative standpoint, member-level techniques lead to a cost-effective solution as the number of under-designed members is reasonably limited, namely, if η_{LS} approaches to zero. On the contrary, when η_{LS} approaches to one, retrofitting based on only member-level techniques would result too expensive and reduction in demand at the structure-level is needed to find a more cost-effective solution.

3.1 Definitions

Whatever technology is adopted (namely, concrete shear walls or steel bracings), structure-level interventions can be described by yielding strength $F_{y,b}$, lateral stiffness K_b and, optionally, overstrength factor $\phi_{b,ov}$ (Figure 1).

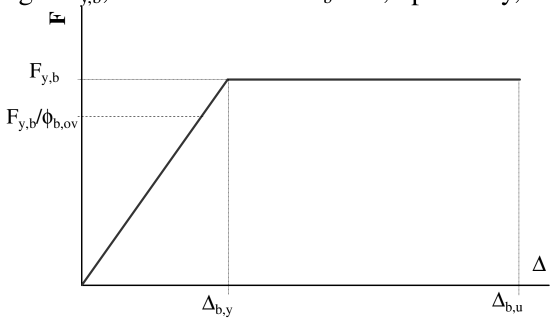


Figure 1. Idealized capacity curve for the sub-structure which realizes the member-level intervention.

Two limit states will be considered in the following (DL and SD); the properties of the existing structures are denoted with the subscript "ES", while the ones of the generic sub-structure realizing the structure-level intervention take the subscript "b". Consequently, the following non-dimensional parameters can be defined:

- $\phi = F_{y,b} / F_{y,ES,SD}$, ratio between the lateral strength of the bracing structure and the resisting base shear of the existing structure $F_{y,ES,SD}$ (both referred at the LS of SD);
- $\kappa = K_b / K_{ES,SD}$, bracing-to-structure stiffness ratio;
- $\phi_{ov,ES} = F_{y,ES,SD} / F_{y,ES,DL}$ overstrength ratio for the existing structure;
- $\kappa_{ov,ES} = K_{ES,DL} / K_{ES,SD}$ secant stiffnesses ratio for the existing structure at LS of DL and SD;
- $\mu_{ES,LS} = \Delta_{c,ES,LS} / \Delta_{y,ES,LS}$ displacement ductility of the existing structure (LS=DL, SD, NC).

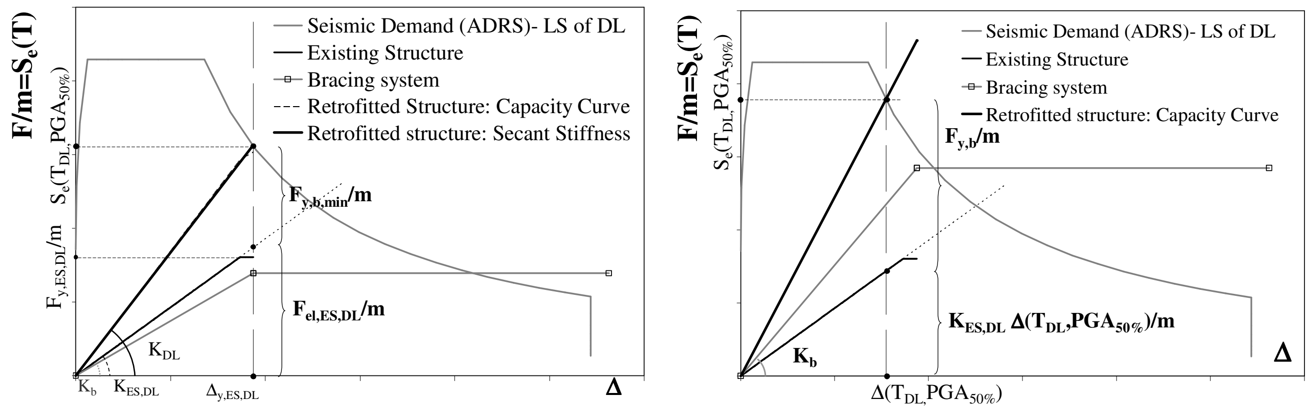
3.2 Retrofitting objective at LS of Damage Limitation

Stiffness and strength of the bracing sub-structure realizing the structure-level technique can be determined by imposing that the displacement demand $\Delta_{d,DL}$ at LS of DL is not larger than the capacity $\Delta_{c,DL}$ of the structure, which is assumed equal to $\Delta_{c,ES,DL}$, obtained by assessing the existing one. Since no significant post-elastic displacements are required at LS of DL, the stiffness K_{DL} of the retrofitted structure can be easily derived:

$$K_{DL} \approx K_{ES,DL} + K_b \,. \tag{3.1}$$

The fundamental period of vibration T_{DL} of the retrofitted structure can be determined depending on its mass m (supposed invariant after the retrofitting) and secant stiffness K_{DL}. Consequently, the displacement demand can be expressed as a function of the pseudo-acceleration $S_e(T_{DL}, PGA_{50\%})$ invoking the equal-displacement rule:

$$\left(\frac{T_{DL}}{2\pi}\right)^2 \cdot S_e\left(T_{DL}, PGA_{50\%}\right) \le \Delta_{c,ES,DL} \quad . \tag{3.2}$$



a) minimum stiffness of the bracing system b) behaviour of the retrofitted structure at LS of DL.

Figure 2. Retrofitting objective at LS of DL.

The value of peak ground acceleration PGA_{DL} leading to a displacement demand on the existing structure equal to the corresponding capacity $\Delta_{c,ES,DL}$ depends on the secant period of the existing structure at the same LS:

$$\Delta_{c,ES,DL} = \left(\frac{T_{ES,DL}}{2\pi}\right)^2 \cdot S_e\left(T_{ES,DL}, PGA_{DL}\right) \quad . \tag{3.3}$$

Introducing equation (3.3) in (3.2) and using the α_e definition in (2.1) the following relationship can be derived:

$$\frac{1}{\alpha_e} \le \sqrt{\frac{K_{DL}}{K_{ES,DL}}} \quad , \tag{3.4}$$

and the non-dimensional stiffness κ of the bracing sub-structure needs to meet the following lower limitation, as the definition of K_{DL} given by equation (3.1) is introduced:

$$\kappa \ge \kappa_{min} = \kappa_{ov,ES} \cdot \left(\frac{1}{\alpha_e^{\,2}} - 1\right) \quad . \tag{3.5}$$

The strength $F_{y,b}$ of the sub-structure conceived as structure-level intervention can be chosen to face the action at LS of DL within the elastic range. Consequently, a minimum value $F_{y,b,min}$ can be defined as follows:

$$m \cdot S_e\left(T_{DL}, PGA_{50\%}\right) = F_{el,ES,DL} + F_{y,b,min} \quad , \tag{3.6}$$

and, in non-dimensional, terms larger values of strength ϕ are required if a stiffness $\kappa > \kappa_{min}$ is chosen:

$$\phi \ge \frac{\phi_{min}}{\alpha_e} \cdot \frac{\kappa/\kappa_{min}}{\sqrt{1 + \kappa/\kappa_{ov,ES}}} \quad . \tag{3.7}$$

3.3 Retrofitting objective at LS of Severe Damage

The retrofitting objective at LS of SD can be met in terms of balance between capacity and demand:

$$\Delta_{d,SD} \le \Delta_{c,SD} \quad . \tag{3.8}$$

As a matter of principle, displacement capacity $\Delta_{c,SD}$ of structure after retrofitting can be expressed as a function of the corresponding capacity $\Delta_{c,ES,SD}$ of the existing structure through a magnification factor which takes into account the effect of member-level techniques in enhancing local capacity of members:

$$\Delta_{c,SD} = k_\mu \cdot \Delta_{c,ES,SD} \cdot \tag{3.9}$$

Following the same path which led to equation (3.3), the displacement capacity $\Delta_{c,ES,SD}$ can be expressed as a function of the PGA_{SD} resulting in a displacement demand equal to capacity:

$$\Delta_{c,ES,SD} = \left(\frac{T_{ES,SD}}{2\pi}\right)^2 \cdot S_e\left(T_{ES,SD}, PGA_{SD}\right) \ , \tag{3.10}$$

where $T_{ES,SD}$ is the fundamental period of the existing structure simply determined as a function of the lateral stiffness $K_{ES,SD}$. Under the usual hypotheses, the displacement demand after retrofitting can be expressed as a function of the expected peak ground acceleration $PGA_{10\%}$ and the fundamental period T_{SD}:

$$\Delta_{d,SD} = \left(\frac{T_{SD}}{2\pi}\right)^2 \cdot S_e\left(T_{SD}, PGA_{10\%}\right) \ . \tag{3.11}$$

Lateral stiffness of the structure after retrofitting can be determined as a function of both $K_{ES,SD}$ and K_b; since significant displacement beyond the yielding limit are expected at LS of SD, the following expression can be utilized for determining the secant stiffness of the structure after retrofitting:

$$K_{SD} = \left(1 + \frac{F_{y,b}}{F_{y,ES,SD}}\right)^2 \Bigg/ \left[\frac{1}{K_{ES,SD}} + \left(\frac{F_{y,b}}{F_{y,ES,SD}}\right)^2 \cdot \frac{1}{K_b}\right] \ . \tag{3.12}$$

The following relationship can be derived for determining the required increase k_μ in global ductility of the structure as a function of the key parameters of the structure:

$$k_\mu \ge \left(\frac{T_{SD}}{2\pi}\right)^2 \cdot S_e\left(T_{SD}, PGA_{10\%}\right) \Bigg/ \left[\left(\frac{T_{ES,SD}}{2\pi}\right)^2 \cdot S_e\left(T_{ES,SD}, PGA_{SD}\right)\right] \ , \tag{3.13}$$

and, after few transformations, the following inequality can be derived:

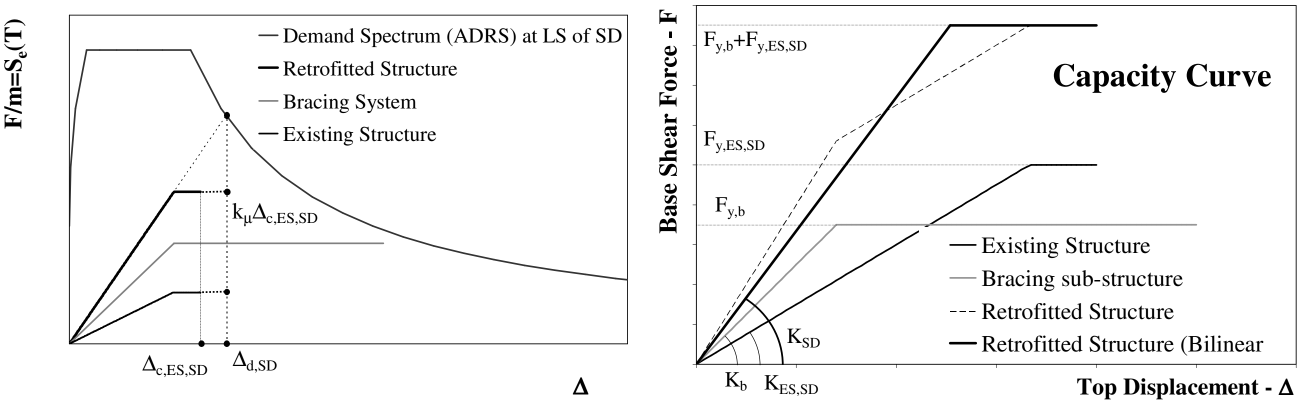
$$k_\mu \ge \sqrt{\frac{K_{ES,SD}}{K_{SD}}} \cdot \frac{1}{\alpha_u} \ . \tag{3.14}$$

Finally, introducing equation (3.12) in (3.14) the following relationship can be found for expressing k_μ as a function of the other two parameters related to the structure-level component of the retrofitting intervention as a whole:

$$k_\mu \ge \frac{\sqrt{\dfrac{\phi^2}{\kappa} + 1}}{(1+\phi) \cdot \alpha_u} \ . \tag{3.15}$$

As a matter of principle, the k_μ value ranges 1.0 e $1/\alpha_u$; in particular, the retrofitting objective represented by equation (3.8) can be satisfied without any increase in stiffness as the initial displacement capacity $\Delta_{c,ES,SD}$ is amplified by the factor $1/\alpha_u$. On the contrary, for k_μ =1.0 a sharp reduction in demand is required on $\Delta_{d,ES,SD}$ by means of a structure-level intervention described by the lateral stiffness K_b whose non-dimensional value κ can be derived by relation (3.15) imposing k_μ=1.0. Since there is no rational reason to require that displacement demand would be significantly smaller than $\Delta_{c,ES,SD}$ (obtained as k_μ=1.0), the value of κ obtained above can be intended as the maximum stiffness κ_{max} to be required to the structure-level technique for the retrofitting purposes; consequently, solving equation (3.15) with k_μ=1.0 the following expression can be obtained for κ_{max}:

$$\kappa \le \kappa_{max} = \frac{\phi^2}{(1+\phi)^2 \cdot \alpha_u{}^2 - 1} \quad . \tag{3.16}$$



a) required increase of capacity k_μ b) secant stiffness of the structure at LS of DS.

Figure 3. Retrofitting objective at LS of SD.

Equation (3.16) points out a lower-bound for strength ϕ as a function of one of the vulnerability parameters:

$$\phi \ge \frac{1}{\alpha_u} - 1 \quad , \tag{3.17}$$

and a further lower bound for ϕ for resulting in $\kappa_{max}>\kappa_{min}$:

$$\frac{\phi^2}{(1+\phi)^2 \cdot \alpha_u{}^2 - 1} \ge \kappa_{min} \quad . \tag{3.18}$$

The number n of required member-level interventions basically depends on the value η_{SD} assessed on the existing structure; the following general expression can be considered:

$$n = \left(\frac{k_\mu - 1}{1/\alpha_u - 1}\right)^h \cdot \eta_{SD} \cdot n_{tot} \quad , \tag{3.19}$$

where h depends on the global structural behavior and n_{tot} is the number of members on the structural model.

3.4 General definition of a cost function.

Retrofitting interventions obtained as a combination of the two mentioned techniques can be described in terms of the three non-dimensional parameters κ, ϕ, k_μ; such parameters can be utilized for defining a cost function:

$$C(\phi, \kappa, k_\mu) = C_\phi \cdot \phi + C_\kappa \cdot \kappa + C_\mu \cdot (k_\mu - 1) + C_0 \quad , \tag{3.20}$$

being C_φ, C_κ and C_μ related to the materials unit costs to be adopted for retrofitting and C_0 a constant term possibly related to indirect costs. In particular, C_ϕ and C_κ are directly related to the type of structure-level technique adopted for retrofitting and can be related to ϕ e κ, respectively. On the contrary, C_μ is related to the costs of the member-level interventions; since the number of such interventions can be derived by (3.19), the total cost can be obtained through the unit cost $C_{c,u}$ of such interventions:

$$C(\phi, \kappa, k_\mu) = C_\phi \cdot \phi + C_\kappa \cdot \kappa + C_{c,u} \cdot n + C_0 = C_\phi \phi + C_\kappa \kappa + C_{c,u} \cdot \left(\frac{k_\mu - 1}{1/\alpha_u - 1}\right)^h \eta_{SD} n_{tot} + C_0 \, . \tag{3.21}$$

3.5 Final remarks about the retrofitting strategy formulation.

Finally, retrofitting procedure conceived as a combination of the two basic techniques mentioned above is completely defined by the three parameters κ, ϕ and k_μ. Two relationships (equation (3.7) for LS of DL and

(3.15) for SD) have been derived to relate those quantities obtaining technically feasible and successful combinations of member- and structure-level interventions. Consequently, seismic retrofitting is an undetermined problem since it is basically described by three unknown parameters involved in only two equations. Once the existence of a proper range $[\kappa_{min}, \kappa_{max}]$ is demonstrated, infinite solutions can be found as κ ranges therein. Hence, an optimal solution can be found by minimizing an objective function, namely the cost function reported in (3.21):

$$\bar{\kappa} = \arg\min_{\kappa} \left[C\left(\phi(\kappa), \kappa, k_\mu(\kappa, \phi(\kappa))\right)\right] \ , \tag{3.22}$$

where $\phi = \phi(\kappa)$ and $k_\mu = k_\mu(\kappa, \phi(\kappa))$ are given by equation (3.7) and (3.15), respectively. The solution for the optimization problem stated in equation (3.22) has to be found within the following range:

$$\bar{\kappa} \in [\kappa_{min}, \kappa_{max}] \ , \tag{3.23}$$

defined by (3.5) and (3.16). Finally, it is worth noticing that the retrofitting strategy summarized by the two last relationship as a constrained optimization problem is organically connected to the assessment procedure outlined within the second section of this paper; indeed, the outcome of the latter procedure (namely, α_e, α_u, η_{SD}, etc.) plays a key role in determining $\bar{\kappa}$, $\bar{\phi}$ and $\bar{k}_\mu$ as solution of equation (3.22) and (3.23).

4. APPLICATIONS

Two sample applications of the strategy formulated in the previous paragraphs are proposed for pointing out how diverse can be the optimal solution for retrofitting when different structures are considered. Two existing structures, designed for only gravitational loads are considered being two and four the total number of storeys, respectively. Four-storey building is firstly considered and only the behavior in longitudinal direction is taken into account herein. The structural model is represented in Figure 4; the value a_g=0.35 g has been assumed for the expected event at LS of SD and Soil Class B is considering according to the Italian Seismic Code and Eurocode. Seismic assessment of that structure has been firstly carried out by means of pushover analyses and N2-Method. The following values have been determined for the parameters of interest in seismic assessment:

$$\alpha_e = 0.787 \qquad K_{ES,SD} = 766.44 \text{ kN/cm} \qquad \phi_{ov,ES} = 1.114$$

$$\alpha_u = 0.627 \qquad F_{y,ES,SD} = 2234.07 \text{ kN} \qquad \mu_{ES,DL} = 1.303$$

$$\eta_{SD} = 0.157 \qquad \kappa_{ov,ES} = 1.387$$

The two extreme values of stiffness κ of the sub-structure can be found through equations (3.5) and (3.16):

$$\kappa_{min} = 0.850 \ , \qquad \kappa_{max} = 1.700 \ .$$

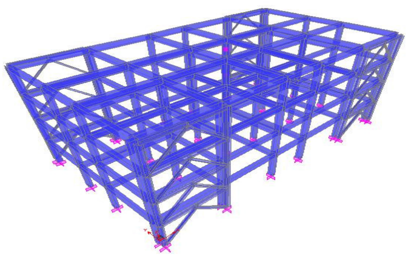
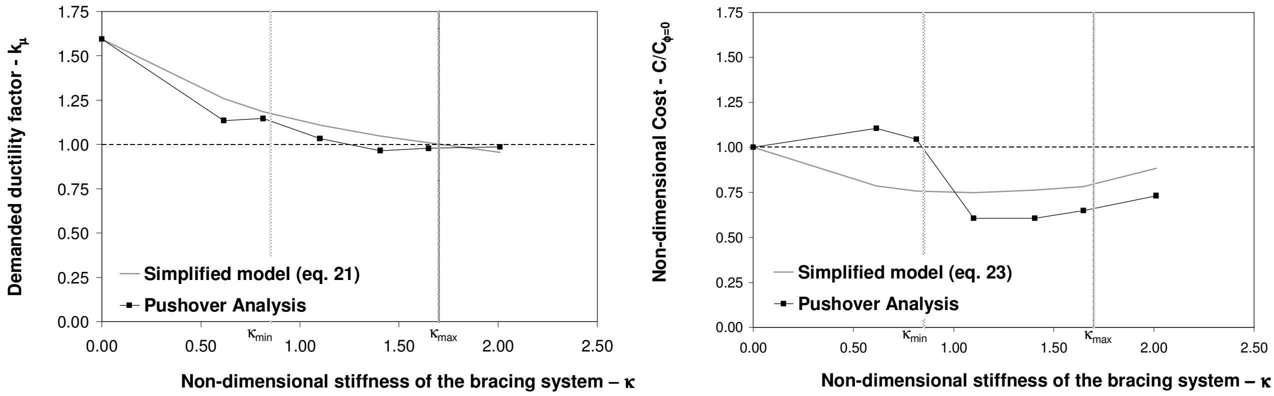


Figure 4. Model of the retrofitted structure – Four-storey building.

The strengthening intervention of the structure is conceived according to the proposed strategy considering the coupled action of Y-shaped steel bracings (as structure-level technique) and FRP-based confinements of beams and columns (as member-level technique). Four bracings can be considered on the structure as shown in Figure 4, and their stiffness can be assumed as variable within the two mentioned values κ_{min} and κ_{max} as a result of the variation of the link geometric and mechanical properties. For each value of κ within the range $[\kappa_{min}, \kappa_{max}]$ the corresponding value of the strength ϕ can be derived by equation (3.7); furthermore, for each bracing solution defined by the couple (κ, ϕ) the corresponding ductility factor k_μ can be obtained through equation (3.15).
Figure 5a shows the k_μ value derived by the simplified relationship (3.15) and compare this value to the one

derived by pushover analyses carried out on the structural model with different bracing systems. For the sake of completeness, κ values within the range [κ_{min},κ_{max}] as well as other values outside that range have been considered for the stiffness of the bracing structure. The number n of over stressed member as a function of the bracing stiffness κ as been also considered with the aim of testing the accuracy of equations (3.15) and (3.19). The continuous line is derived by the simplified relationship (3.19) (with h=1.0) adopted in the formulation of the retrofitting strategy while the points have been obtained through the same pushover analyses carried out on the structure coupled with bracing frames of variable stiffness. Retrofitting cost can be evaluated through equations (3.20) and (3.21) as a function of κ and the two related parameters ϕ and k_{μ}, after defining the constants C_0, C_k, C_j e $C_{c,u}$ on the basis of the costs related to the adopted techniques. The terms $C_{c,u}$ is the more straightforward to be determined: in the present application it is the unit cost of a FRP-based confinement of RC members. On the contrary, the constant C_{κ} can be generally related to the stiffness of the bracing system; nevertheless, in this case, stiffness variations have been obtained by changing the link length and no relevant cost variations can be assumed directly depending on that parameter. Consequently, C_{κ}=0 and, since foundations hugely affect the total cost of the structure-level intervention, the cost of bracings and foundation is assumed being proportional to the non-dimensional strength ϕ. The two following values can be assumed for $C_{c,u}$ e C_{ϕ} with the aim of pointing out the key features of the formulated strategy:

- $C_{c,u}$=2000 €/member;
- C_{ϕ}=100000 €.



a) required ductility increase k_{μ} b) retrofitting cost

Figure 5. Response of the retrofitted structure depending on κ – four-storey building.

Figure 5b shows the values of the cost-function as a ratio with respect to $C_{\kappa=0}$ which is the cost of the only member-level intervention; an optimal value of the stiffness κ can be chosen for that function and, although the cost prevision obtained by the numerical analyses and the simplified relationship (3.22) have two different shapes, the minimum is attained form both curve for similar values of $\kappa \approx 1.10$. belonging to the range [κ_{min}, κ_{max}] and, hence, for similar combination of member-level and structure-level interventions.
Finally, a two-storey building, obtained by taking the two top storeys of the first one, is also considered. The retrofitting strategy can be also applied to the two-storey structure considering assuming that the non-dimensional stiffness of the bracing structure ranges from 0,385 and 1,743, as a result of equations (3.5) and (3.16); consequently, the range [κ_{min}, κ_{max}] is wider than the one derived for the taller structure.

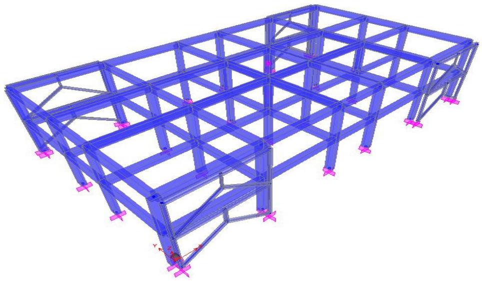
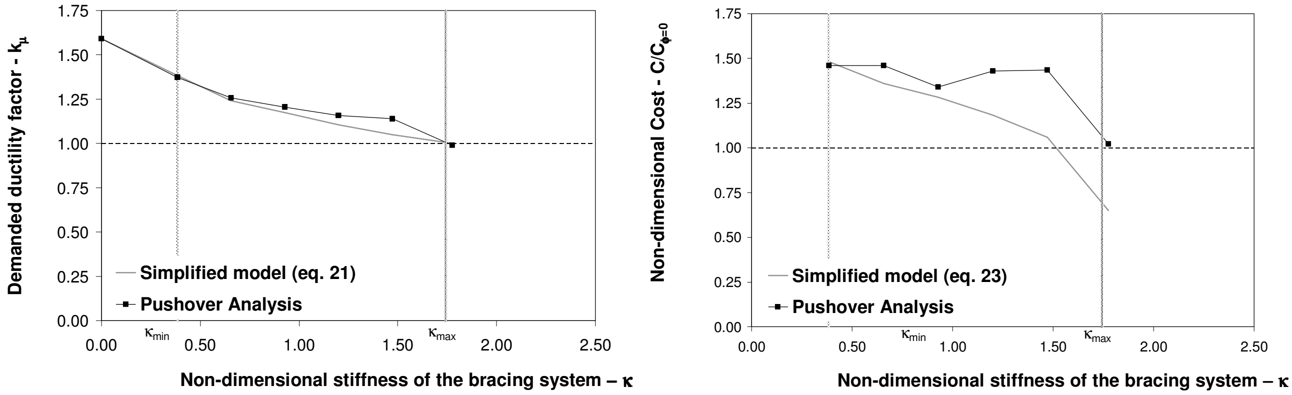


Figure 6. Model of the retrofitted structure – Two-storey building.



a) required ductility increase k_μ b) retrofitting cost

Figure 7. Response of the retrofitted structure depending on k – two-storey building.

Figure 7a shows the values of k_μ as a function of the non-dimensional stiffness κ of the bracing system; the figure confirms the accuracy of equation (3.15) in predicting the residual demanded displacement for the retrofitted structure as a result of the bracing stiffness. On the contrary, Figure 7 shows the overall intervention cost estimated through equation (3.21) considering the residual number n of overstressed members obtained by a pushover analysis (dotted line) or derived by equation (3.19).

5. CONCLUSIONS

A general strategy for conceiving seismic retrofitting of existing buildings as a combination of member- and structure-level techniques has been formulated in the present paper. The strategy takes the results of an integrated and multilevel assessment procedure as input data. Three non-dimensional parameters fully describe the general combination of a structure-level intervention and a member-level technique. Two equations relate those three parameters imposing that the structure after retrofit meet the safety checks at LSs of DL and SD. A further condition derives by minimizing the total cost of the retrofitting intervention by tuning the combination of member- and structure-level techniques even considering the initial performance of the existing structure. Further development can be thought for the present strategy to enrol other strengthening techniques within those that can be chosen and combined to obtain the optimal solution for seismic retrofitting.

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