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INFLUENCE OF THE CONNECTIONS BETWEEN THE EXISTING BUILDING AND THE STRENGTHENING EXOSKELETON

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Abstract

Holistic retrofit solutions for RC buildings carried out from outside by means of structural exoskeletons have been widely investigated in the last years. The effectiveness of such a renovation approach emerges both in the construction time, when addressing the barriers to the renovation such as the inhabitant relocation and the building disruption, and when broadening the time frame of the analyses when shifting from the construction time to a life cycle perspective. The potential of the holistic approach becomes clear in reducing costs, impacts on the inhabitants and impacts on the environment over the building life cycle. Within such a new perspective, new technology options are needed, and their performance has to be investigated.

This work addresses the influence on the global response of the connections between the existing building and the seismic retrofit solution carried out from outside. A parametric evaluation of the retrofitted structure is conducted by means of nonlinear time history analyses on a simplified 2 DOFs system considering the existing building, the additional external retrofit, and their mutual connection. The results show that the connections play a fundamental role in the calibration of both stiffness and strength of the retrofit exoskeleton and must be therefore considered in the design phase. Indeed, considering the sole stiffness of the retrofit system leads to a substantial underestimation of the required exoskeleton stiffness compared to the case where the stiffness of both the exoskeleton and the connections are considered.

Finally, a case study resembling a RC building typical of the European post-WWII building stock is considered for the validation of the proposed design procedure by means of nonlinear time history analyses.

Keywords: Connections; seismic retrofit from outside; exoskeletons; holistic renovation;

1. Introduction

It has been recognized that the existing building stock is obsolete, massively energy consuming, and vulnerable to natural and man-induced hazards [1]; a systematic intervention on the built environment has to be undertaken to foster environmental, economic and social sustainability among the European communities.

In spite of this scenario, according to BPIE [2], the average European renovation rate of reinforced concrete (RC) buildings is very low (about 1%) because of the major barriers to the renovation consisting in: the need to relocate the occupants, the extended downtime required during the construction works, the high cost of the interventions, and the lack of adequate business models fostering the renovation [2, 3, 4]. To overcome these barriers, holistic retrofit solutions for RC buildings carried out from outside by means of structural exoskeletons have been widely investigated in the last years.

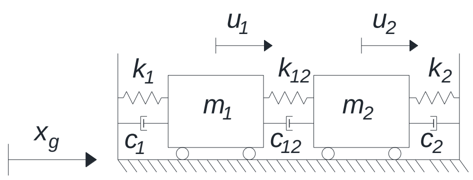
When addressing building renovation barriers, working from outside the building may avoid the relocation of the occupants and the damage on the finishing. This way, the feasibility of the intervention increases while reducing indirect costs of the renovation. Moreover, the potential of the holistic approach becomes clear in reducing costs, impacts on the occupants and impacts on the environment over the building life cycle. Within such a new perspective, new technology options are needed, and their performance has to be investigated. This work, considering seismic retrofit solution carried out from outside, addresses the influence on the global response of the connections between the existing building and the new seismic exoskeleton. In this context, connections are considered both as elastic (*EL*) and as non-linear (*NL*). In the former case, they may represent those sacrificial elements that enable localizing damage in the case of an earthquake, thereby reducing the repairing costs and construction time after the seismic event.

A parametric evaluation of the retrofitted building is conducted by means of nonlinear time history analyses carried out on simplified 2 Degrees of Freedom (2DOF) models considering the existing building, the additional external retrofit and their mutual connection. The objective of such evaluation is to investigate how the connections may affect the response of the retrofitted building and to provide some preliminary insights on the parameters governing the design of these systems.

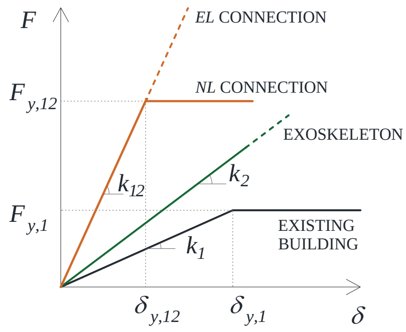
Finally, a design method for the retrofit solution carried out from outside is proposed and validated by means of a case study resembling a RC building typical of the European post-WWII building stock.

2. Definition of the 2DOF system

The interaction between the existing building (DOF_1) and the retrofit solution (from outside) (DOF_2) is described by means of a 2 DOF system that consists of 2 masses connected by springs (k_1, k_2, k_{12}) and viscous dampers (c_1, c_2, c_{12}) and subjected to an external acceleration $\ddot{X}_g = \frac{d^2 x_g}{dt^2}$ (Fig. 1).



a)



b)

Fig. 1 – a) Simplified 2DOF model; b) Response curves of the retrofitted structure with 2 degrees of freedom (2DOF) working in parallel

The parameters needed to define the structural response of the DOF_1 (indicated with the subscript 1 in Fig. 1) are the effective mass (m_1), the initial elastic stiffness (k_1), the damping coefficient (c_1), and the yielding force ($F_{y,1}$). Given the elastic stiffness (k_1) and the yielding force ($F_{y,1}$), the yielding displacement ($\delta_{y,1}$) can be derived ($\delta_{y,1} = \frac{F_{y,1}}{k_1}$).

The retrofit exoskeleton (DOF_2 indicated with the subscript 2 in Fig. 1) is defined by means of the elastic stiffness (k_2) and the damping coefficient (c_2); for these preliminary considerations, the mass of the exoskeleton is assumed equal to 1/10÷1/20 of the mass of the existing building (m_1) [5].

The 2 masses are connected with a general link, which models the connection between the existing structure and the exoskeleton (which is indicated with subscript 12 in Fig. 1). The structural response of the connections is described by the elastic stiffness (k_{12}), and the damping coefficient (c_{12}). In the case of non-linear connections, to describe the system, the connection yielding force ($F_{y,12}$) and the associated yielding displacement ($\delta_{y,12}$) are introduced (Fig. 1).

The structural response is analyzed with reference to a set of fundamental parameters:

- η: represents the yielding strength of the existing building, adimensionalized with respect to the mass (m_1) multiplied by the ground acceleration $Sa(T_1)$ as in (1).

$$\eta = \frac{F_{y,1}}{[m_1 \cdot S_a(T_1)]} \quad (1)$$

- μ: represents the "ductility demand" of the existing building after the retrofit. μ is defined as the ratio between the maximum displacement (δ_{MAX}) experienced by the DOF_1 during a seismic event and the yielding displacement ($\delta_{y,1}$) of the DOF_1 (Fig. 1).

$$\mu = \frac{\delta_{MAX}}{\delta_{y,1}} \quad (2)$$

- λ: represents the ratio between the elastic stiffness of the retrofit (k_2) and the stiffness of the existing building (k_1) (Fig. 1).

$$\lambda = \frac{k_2}{k_1} \quad (3)$$

2.1 Equations of motion and non-linear model

For each mass, the Newton's second law of motion yields:

$$\begin{cases} m_1(\ddot{x}_G + \ddot{u}_1) + k_1 u_1 + c_1 \dot{u}_1 = k_{12}(u_2 - u_1) + c_{12}(\dot{u}_2 - \dot{u}_1) \\ m_2(\ddot{x}_G + \ddot{u}_2) + k_2 u_2 + c_2 \dot{u}_2 + k_{12}(u_2 - u_1) + c_{12}(\dot{u}_2 - \dot{u}_1) = 0 \end{cases} \quad (4)$$

The inelastic behavior of the existing building and, in the *NL* case, of the non-linear connection is described by the Bouc-Wen hysteresis law [6]:

$$P(t) = \alpha \cdot k_i \cdot u_i + (1-\alpha) k_i \cdot \delta_{y,i} \cdot Z(t) \quad (5)$$

Where the subscript *-i* is equal to 1 for the DOF_1 and equal to 12 for the connection, α is the post yielding stiffness ratio, and Z is an internal variable whose behavior is described by its derivative:

$$\frac{dZ}{dt} = \left(\frac{1}{\delta_{y,i}}\right) \cdot (\dot{u}_\iota - \gamma \cdot |\dot{u}_\iota| \cdot Z(t) \cdot |z(t)|^{n-1} - \upsilon \cdot \dot{u}_\iota \cdot |z(t)|^n) \quad (6)$$

n, v, and γ are dimensionless quantities (assumed, in this case, as equal to 1, 0.5, 0.5, respectively); n governs the smoothness of the curve in the proximity of the yielding point, v and γ control the size and the shape of the hysteretic loop ($|v| + |\gamma| = 1$).

2.2 Input parameters

The input parameters are selected to be representative of the ordinary post-WWII RC buildings according to Marini et al. [7]. As for the yielding force, different values of η were considered to represent weak (η=0.30), medium (η=0.50-0.60) and strong (η=0.85) buildings as proposed in [8] [9].

The parameters considered for the reference case are reported in Table 1.

Table 1 – Inputs used in the sensitivity analysis of the elastic SDOF system

Parameter	**Symbol**	**Value**	
Elastic period	T_1	0.85	[s]
Effective mass	m_1	640	[kN/g]
Elastic stiffness	k_1	34	[kN/mm]
Strength parameter	η	0.30-0.50-0.60-0.85	[-]

The influence of the connections on the global response of the retrofitted system is investigated for an inter-story drift target (ϑ) equal to 0.5%. As for the ground acceleration ($\ddot{X}_g$), 7 accelerograms compatible with the code spectrum were determined by adopting the software Rexel 2.2beta [10]. The structural system is supposed located in L'Aquila (Italy), on a flat surface made of deposit of sand or medium-dense sand gravel or stiff grave (soil category C and T1 topography) [11]. A maximum scale factor equal to 2 and upper and lower tolerance equal to 10% and 15%, were imposed.

3. Preliminary considerations on the 2DOF system

3.1 Elastic connection

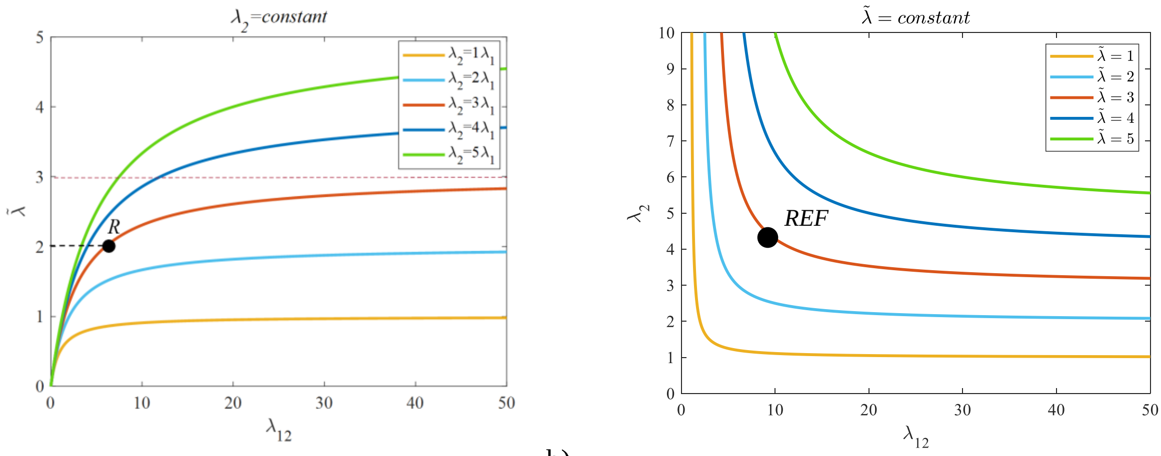
As demonstrated in [12], when the connections between the 2 DOFs are elastic and the mass of the exoskeleton (m_2) is negligible if compared with the mass of the existing building (m_1) (m_2=1/10-1/20 m_1) the elastic system can be simplified in the SDOF system shown in Fig. 2, in which the exoskeleton and the connections can be considered as 2 spring in series and lumped together (Fig. 2b).

Fig. 2 – a) Simplified SDOF system; b) Simplified SDOF system with equivalent spring and damping.

For the simplified SDOF system the following quantities can be defined:

$$\text{a)} \quad \tilde{k} = \frac{k_2 k_{12}}{k_2 + k_{12}} \qquad \text{b)} \quad \tilde{c} = \frac{c_2 c_{12}}{c_2 + c_{12}} \qquad \text{c)} \quad \tilde{\lambda} = \frac{\tilde{k}}{k_1} \tag{7}$$

From (7) it can be seen that the equivalent stiffness of the retrofit ($\tilde{k}$) is a function of the stiffness of the exoskeleton (k_2) and of the stiffness of the connection (k_{12}). To give some preliminary considerations about the influence of the connections on the global response of the retrofitted system, in Fig. 3a, $\tilde{k}$ is plotted by varying the elastic stiffness of the connections (k_{12}). To eliminate the dependency on the DOF$_1$ parameters, the results are expressed in terms of normalized stiffnesses $\tilde{\lambda} = \frac{\tilde{k}}{k_1}$, $\lambda_{12} = \frac{k_{12}}{k_1}$, and $\lambda_2 = \frac{k_2}{k_1}$.



a) b)

Fig. 3 – Evaluation of the series of pairs: a) $(\lambda_{12};\tilde{\lambda})$ obtained from varying k_2; b) $(\lambda_{12};\lambda_2)$ obtained from varying $\tilde{k}$. Note: *REF* refers to the Reference Case in section 4

Fig. 3a highlights the influence of the connection stiffness on the global response the retrofit solution; supposing that the global stiffness of the retrofit (exoskeleton + connections) is equal to the sole stiffness of the exoskeleton, may result in an erroneous estimation of the actual stiffness of the retrofit, therefore leading to higher displacements, and so damage, in the existing building. In fact, taking as reference the case of $k_2=3k_1$ (i.e. red line in Fig. 3a), the global stiffness ratio of the retrofit ($\tilde{\lambda}$) is equal to 3 only if the connection stiffness tends to infinity (i.e. $\lambda_{12}\rightarrow\infty$), while an actual stiffness of the connection lower than infinity leads to lower values of λ_{12} and consequently of $\tilde{\lambda}$; for example, if $k_{12}=7k_1$, the corresponding λ_{12} value is 7 and the global stiffness of the intervention is 2, i.e. 33% lower than expected. It is worth noting that the value of λ_{12} over which there is a good correspondence with the case of connection with infinite stiffness changes with the stiffness of the exoskeleton, the lower the stiffness of the exoskeleton compared to the building stiffness, the lower such value.

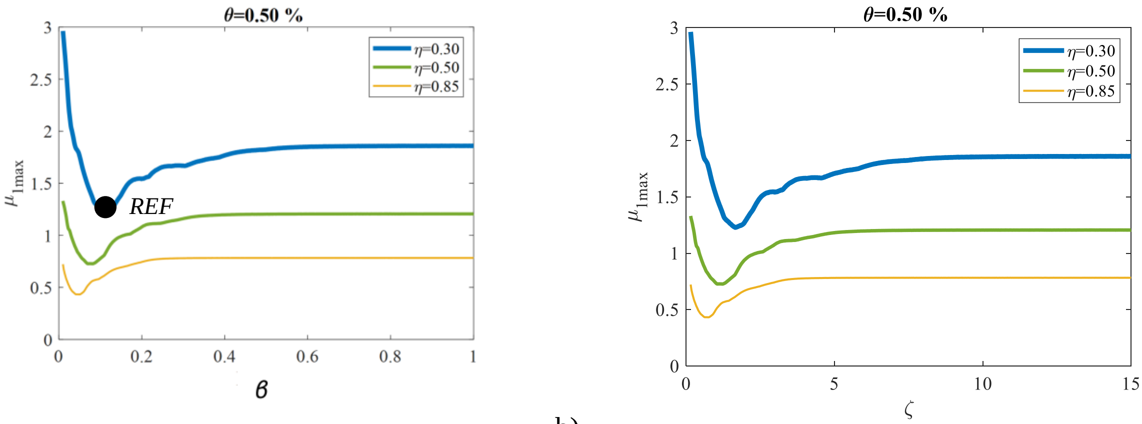
In Fig. 3b, for a given $\tilde{\lambda}$, i.e. for a given global stiffness of the retrofit intervention, infinite pairs of $(\lambda_{12};\lambda_2)$ that satisfy equation Eq. (7a) are possible. By decreasing λ_2 (i.e. decreasing k_2), the stiffness ratio of the connection (λ_{12}) increases. As expected, by increasing $\tilde{\lambda}$, both stiffness ratios increase. Fig. 3b highlights the presence of regions in which both λ_{12} and λ_2 are minimized, however, the optimal combination is related to many aspects, such as the technological aspects of both exoskeleton and connections.

3.2 Non-linear connection

With reference to the simplified 2 DOFs system described in Fig. 1, in this section, a non-linear connection is considered. Parametric analyses are carried out to investigate the influence on the global response of the system of the yielding displacement of the connections ($\delta_{y,12}$). In particular, the maximum displacement of DOF$_1$ when subjected to 7 GMs is evaluated for the reference case by varying the yielding displacement of the connections within the range [0: δ_{y1}], given the elastic stiffness of the connections (k_{12}) and the target drift (θ=0.5%). In this preliminary stage, the parametric analysis is carried out taking the reference parameters reported in Table 1, therefore, only η has been varied.

To evaluate the influence of the yielding displacement of the connections ($\delta_{y,12}$) on the global response of the retrofitted system, the required ductility μ (obtained as the mean of the results of 7 accelerograms) is plotted as a function of: a) the yielding ratio β that represents the ratio between the yielding displacement of the connections ($\delta_{y,12}$) and the yielding displacement of the existing building ($\delta_{y,1}$) (Fig. 4a) and b) the ratio

ζ that represents the ratio between the yielding force of the connections ($F_{y,12}$) and the yielding force of the existing building ($F_{y,1}$) (Fig. 4b).



a) b)

Fig. 4 – Ductility demand (μ) on the existing building (DOF_1) as a function of a) yielding displacement ratio of the non-linear connections (β) and b) yielding force ratio of the non-linear connections (ζ).

In Fig. 4 it can be seen that the 2 plots lead to the same results; in particular, it can be observed that, for the same value of yielding displacement (or force), the higher η, the smaller the required ductility on the DOF_1 after the retrofit. Increasing the value of β (and, consequently, of ζ), the ductility demand μ approaches to the elastic response; e.g. for β higher than 0.3, the connections can be considered as elastic. However, the most important consideration is that all curves show a stationary point (minimum). For specific values of β, the ductility demand on the existing building (DOF_1) can be minimized; this entails minimum displacement demand and minimum seismic action on the existing building. The design yielding displacement of the connections should be identified around these values.

It is worth noting that, the smaller the strength parameter (η), the higher the stationary point of the curves; moreover, it can be observed that, increasing the value of η, the optimal yielding significantly shift to the left. Basing on these considerations, it can be supposed that also the other parameters that describe the 2 DOFs may affect the position of the stationary point of the curves. However, the influence of other parameters on the position of the stationary point of the curves is postponed in other future research work.

4. Proposed design procedure and application to a case study

In this section, based on the results discussed above, a design procedure for non-linear retrofit solution carried out from outside is proposed and applied to a case study representative of post-World War II. In this work, the main focus is the influence of the connections between the existing building and the retrofit solution, for this reason, the retrofit solution is designed and modelled as a generic and elastic retrofit solution carried out from outside; at this regard, additional shear walls are introduced.

The proposed design procedure is made by the following steps:

Step 1: Vulnerability analysis of the existing building and definition of the equivalent SDOF parameters;

Step 2: Definition of performance targets: (e.g. maximum inter-story drift ϑ and top displacement d_{TOP});

Step 3: Design of the retrofit solution: Application of the design spectra developed in [13][1] to define the equivalent stiffness of the retrofit (exoskeleton and connections - $\tilde{k}$). Definition of the stiffness of the connections (k_{12}) and the exoskeleton (k_2), and, in case of non-linear solution, of the yielding displacement of the connections ($\delta_{y,12}$) by means the curves plotted in Fig. 3b and Fig. 4a.

Step 4: SDOF to MDOF transformation: considering, for example, a constant drift profile for the deformed shape of the retrofitted building;

Step 5: Verify the design through non-linear analyses (NLA).

4.1 Application to a case study

The selected case study is a four-story rectangular building (24.00m x 10.64m) featuring three one-way longitudinal frames and two infilled lateral frames. The inter-story height is 3.15m, and the structural system is made of RC frames in the longest direction designed to withstand static loads only. The main features and the structural details of the reference building are described in Feroldi [8] and Passoni [9]. The floors are made of a composite RC beam and clay block systems featuring a 4 cm RC overlay with enough, as assumed, in-plane capacity (i.e. able to develop an in-plane tied-arch resistant mechanism [8, 9]). The staircase core is not designed to withstand seismic loads. The geometry and the materials of the mainframe are reported in Fig. 5.

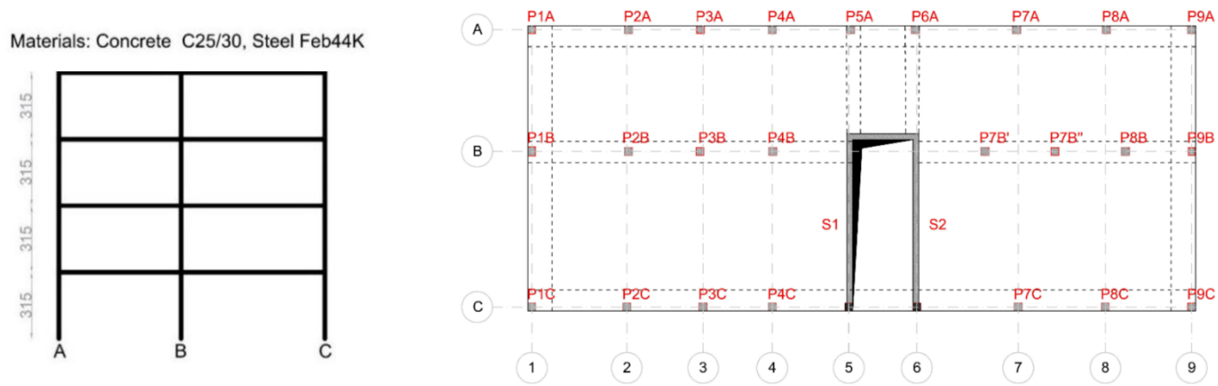


Fig. 5 – Geometry and materials of the reference building mainframe. Characteristics and features are representative of ordinary RC buildings

The finite element model was developed with the software MidasGen 2020. Structural elements such as beams, columns and staircase core were modeled like beam elements with lumped plasticity in accordance with the European Building Code [11]. In particular, the beam flexural plastic hinge has been defined according with Eurocode 8:2004 constitutive law [14]. About columns, both shear and bending behavior have been considered by introducing Eurocode 8:2004 plastic hinges [14]; as concern the flexural plastic hinges, the P-M interactions have been accounted for. Columns are assumed fixed at the base. The floors are modeled as rigid diaphragms.

In the finite element model, the non-linear behavior of staircase walls was modeled with lumped plastic hinges on each floor of the building. At the base of the staircase walls, flexural plastic hinges were introduced considering the minimum value between the plastic moment of the soil and the yielding moment of the walls. For the reference case, the minimum value was given by the soil ultimate capacity considering a maximum soil stress (σ_s) of 0.3 MPa. At the base of each wall a rotational spring was introduced in order to simulate the stiffness of a spread footing. The properties of the rotational hinge were determined considering a unit soil

[1] in which, starting from the consideration made for bracing system by Ciampi et al. [15], a set of design spectra are defined in order to simplify the design procedure of elastic seismic retrofit solutions carried out from the outside.

stiffness (k_s) equal to 0.1 (N/mm)/mm [8]. As for the non-structural elements, infills were modeled as two non-linear equivalent trusses converging in the beam-column joints. Cracking and peak forces were evaluated according to Decanini et al. [15], while the selected cracking and peak drifts were set, respectively, to 0.5% for moderate damage and 1.5% for collapse [16]. Non-linear static analyses were performed in order to evaluate the structural performance of the reference building in the As-Is conditions (Fig. 6).

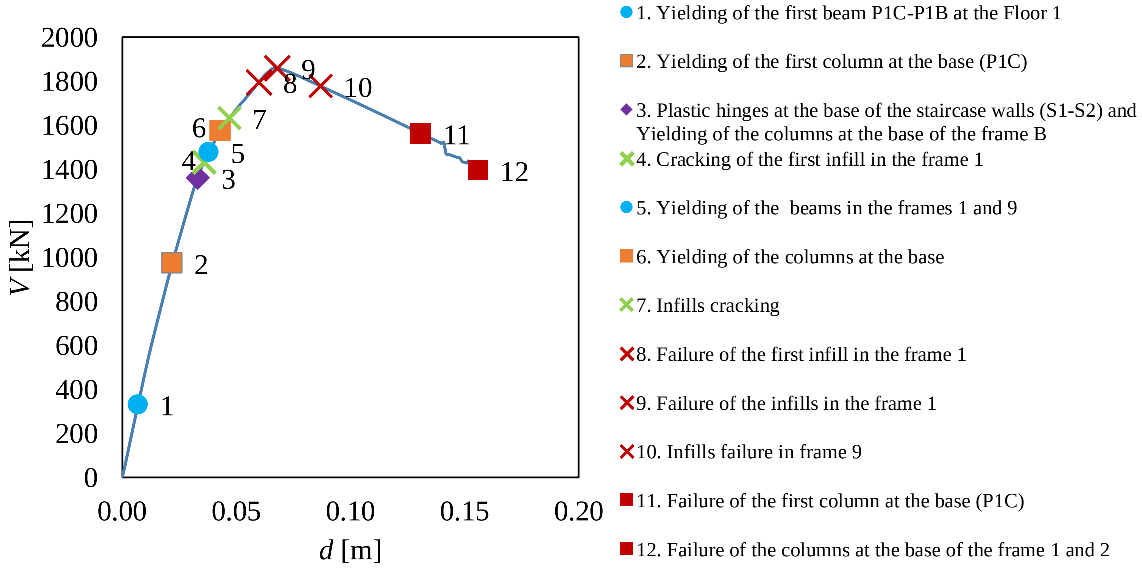


Fig. 6 – Capacity curve of the existing building: dots are related to beams; squares are related to columns; rhombuses are related to the staircase walls; crosses are related to infills

The proposed design procedure is applied to the considered case study following the aforementioned steps:

Step 1: Vulnerability analysis of the existing building

A vulnerability analysis is conducted according to the current building code [11]. The reference building is supposed located in L'Aquila (Italy), with soil category C and T1 topography. The bilinearized capacity curve and the displacement demands, in ADRS terms, are plotted in Fig. 6; the parameters of the equivalent SDOF system used for the vulnerability analysis are reported in Table 2. The Life Safety Limit State (*LSLS*) and the Collapse Prevention Limit State (*CPLS*) are indicated in the capacity curve [11]. The *CPLS* is considered in correspondence to the failure of the first column at the base (ultimate rotational capacity); the *LSLS* limit is considered in correspondence to the ¾ of the ultimate rotational capacity of the columns at the ground floor [11].

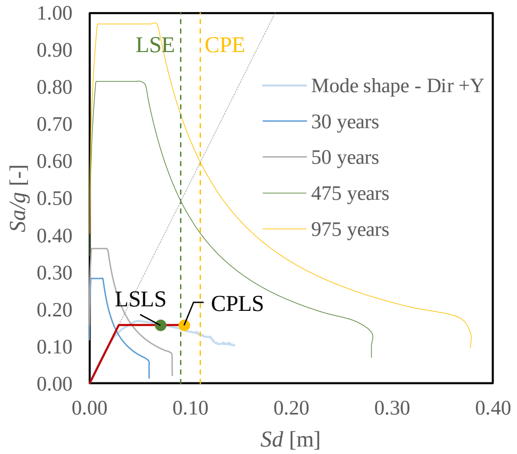


Fig. 7 – Vulnerability analysis in the ADRS [11]

Table 2 – Equivalent SDOF system parameters.

Equivalent SDOF [11]		
m^*	640.42	[kN/g]
k_1	34.1	[kN/mm]
F_{y1}	988.15	[kN]
d_{y1}	29	[mm]
Γ	1.39	[-]
η	0.30	[-]

As shown in Fig. 7, the existing building does not satisfy the displacement demands related to Life Safety Earthquake (*LSE*)[2] and Collapse Prevention Earthquake (*CPE*)[3] and, for this reason, a structural retrofit of the existing building is provided.

Step 2: Definition of performance targets

In this case, a maximum drift for the *LSE* is imposed. In particular, the target limit corresponds to the inter-story drift (θ) allowed by the infill panels (i.e. 0.5%) which corresponds (considering a linear deformed shape of the retrofitted building) to a maximum top displacement (d_{TOP}) equal to 0.063 m.

Step 3: Design of the retrofit solution

According to Labò et al. [13] the parameters of the SDOF reported in Table 2 are considered and an equivalent stiffness of 100 kN/mm (λ=3) is derived.

Considering the proof-of-concept nature of this work, the stiffnesses of the connections (k_{12}) and of the exoskeleton (k_2) are chosen without dealing with their technological aspects; in particular, with reference to Fig. 3b, k_{12} and k_2 are chosen in order to minimize their values while satisfying the equivalent stiffness required. To obtain an equivalent stiffness ratio (λ) equal to 3, an exoskeleton and connection stiffnesses equal to 4 and 10, respectively, are chosen; the reference value is indicated with a full black dot in Fig. 3b.

In the case of non-linear connections between the existing building and the external exoskeleton, the non-linear parameters were derived based on the results plotted in Fig. 4a,b. Considering an inter-story drift target (θ_{MAX}) equal to 0.5%, for a dimensionless yielding force (η) of 0.3, the resulting optimal yielding ratio of the connection (β) is 0.1. By multiplying β for the yielding displacement of the reference building ($\delta_{y,1}$), the yielding displacement of the connection ($\delta_{y,12}$) was derived.

Step 4: SDOF to MDOF

The equivalent stiffness (k_{12}) and the optimal yielding displacement ($\delta_{y,12}$) must be translated to a MDOF system (k_i, d_i). Herein, the stiffness and the optimal yielding displacement of the connections are distributed considering a linear deformed shape with constant drift.

[2] e.g. with a return period equal to 475 years corresponding to a probability of exceedance of 10% in 50 years [10]

[3] e.g. with a return period equal to 975 years corresponding to a probability of exceedance of 5% in 50 years [10]

Step 5: Verify the design through NLA:

Non-linear Time History analyses were carried out considering (a) the elastic retrofit solution (*EL*) in which the inelastic behavior of the sole existing building is considered, and (b) the retrofit with dissipative connections (*NL*). The non-linear connections are modeled by introducing hysteretic links between the existing building and the external exoskeleton at each floor. In the reference case, the same parameters used in the Bouc-Wen hysteretic model were considered. The retrofit solution is modeled as an elastic system; in particular, 2 shear walls are introduced in adhesion to the existing building (only the *y*-direction is considered). The wall cross-section has a thickness of 0.3 m and length of 2.5 m; this leads to a retrofit exoskeleton stiffness (k_2) equal to $4k_1$.

Fig. 8 shows the time history results expressed in terms of inter-story drift and top displacement of the elastic (*EL*) and of the non-linear (*NL*) solutions: the design top displacement and the drift targets are satisfied.

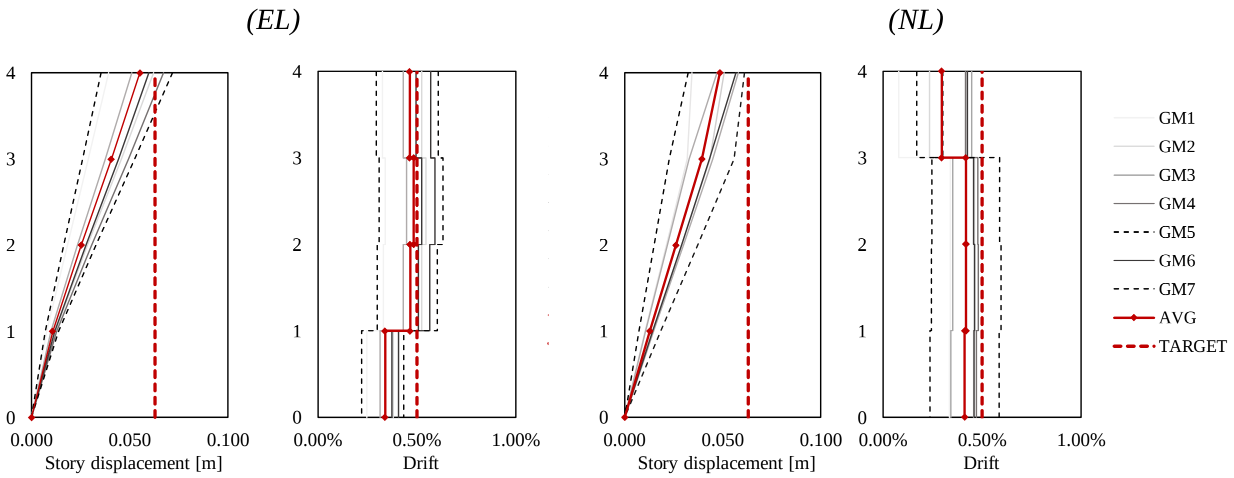


Fig. 8 – Story displacement and inter-story drift: a) In case of elastic connections (*EL*); b) In case of non-linear connections (*NL*)

The results show a 18% reduction of the top displacement (d_{TOP}) when non-linear connection between the existing building and the exoskeleton (*NL*) are introduced (form 0.058m to 0.049m). As well, the spread of the results in the non-linear retrofit (*NL*) is more limited than in the elastic retrofit solution (*EL*). It is interesting noting that in *EL* the top displacement (d_{TOP}) corresponds to a ductility demand (μ) equal to 1.55 while in the *NL* case it corresponds to a ductility demand equal to 1.3. Therefore, it can be seen that the average results of the time histories are well in agreement with the result of the simplified system reported in Fig. 4 (the reference case with non-linear connections is indicated on the curve with the back full dot). Moreover, the introduction of dissipative connections leads to a 33% reduction of total base shear (V_{MAX}) (from 6993 *kN* to 5261 *kN*) and, consequently, of the floor actions over the building height; this aspect becomes relevant when the seismic forces need to be limited to not exceed the existing floor capacity.

5. Discussion: limitation and future developments

In this paper, the optimal values of the yielding displacement of the connections were derived through parametric analyses on a simplified 2DOF system. The results were validated by means of non-linear time history analyses carried out on a 3D Finite Element Model resembling a typical existing European RC building. However, to critically evaluate the results, the following considerations must be drawn:

First, it must be noted that the Finite Element Model is characterized by several uncertainties that may affect the response of the existing building. Many uncertainties are connected to the existing materials and the structural detailing (such as the beam-column joints). In many cases, it is difficult to find the original construction drawings, furthermore, variations to the planned detailing were frequently decided on the construction site without updating the project documentation. Moreover, other uncertainties are connected to modeling issues such as the inelastic behavior of the infill panels and of the staircase core [5].

Second, the optimal yielding displacement is significantly affected by the existing building structural parameters. To make this clearer, parametric analyses are carried out to investigate how the variation of some parameters may affect the optimal yielding displacement of the connections. The parameters considered herein are the stiffness of the connections (k_{12}), the elastic period of the existing building (T_1), the strength ratio (η), and the set of ground motion considered (*GM*) (in the same seismic zone - zone 1 -). The sensitivity analysis is carried out by changing only one parameter at a time (P_i) - with respect to the reference case described in the previous section (Table 1) - in the range $\pm 0.30 P_i$. Fig. 9 shows the variation of the median values represented by means of tornado plots; the optimal yielding displacement can vary up to 79% of the expected value in the case in which the values of the yielding displacement of the connection ($d_{y,12}$) are not normalized (Fig. 9a), and 49% of the expected value in the case in which the yielding displacements are normalized (θ) (Fig. 9b).

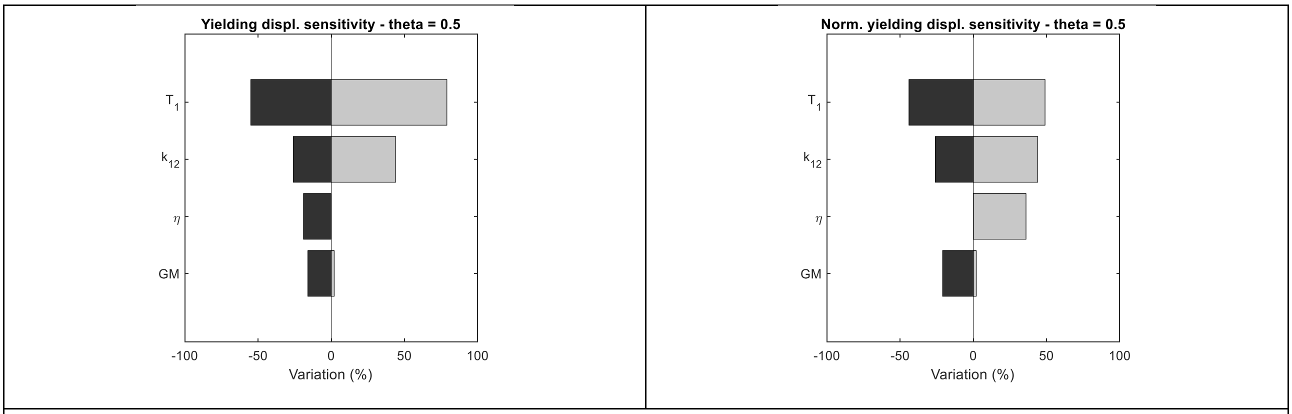


Fig. 9 – Tornado plots resulting from the sensitivity analysis: a) variation of the yielding displacement; b) variation of the normalized yielding displacement by varying the ordinate parameters

Given the above, it is worth noting that some uncertainties on the building characteristics may lead to an erroneous design of the retrofit solutions, with structural responses very far from the expected targets. Ignoring these issues may result in an erroneous expectation of the building capacity. The influence of the connections between the existing building and the exoskeleton must be further investigated; connections represent a crucial aspect in the design of retrofit solutions carried out from outside. However, in the design of the optimal connections between the existing building and the external exoskeleton, also the uncertainties related to the existing building must be considered.

6. Summary

This work is part of an ongoing research on the holistic renovation of the post-WWII RC buildings. In particular, this paper investigates how the connection non-linearity between the existing building and the external exoskeleton may affect the global response of the retrofitted system. A sensitivity analysis is carried out considering a simplified 2DOF system representative of ordinary post World War II RC building (DOF_1) and the strengthening exoskeleton carried out from outside (DOF_2); the 2 DOFs are connected by a general link. In the sensitivity analyses the connection between the 2DOF system is considered both as elastic (*EL*) and as non-linear (*NL*); as far as the non-linear retrofit is concerned, a hysteretic connection was considered. Through the sensitivity analyses, the parameters that govern the retrofitted building response are investigated and their optimal setting values are derived, as for instance the minimum required elastic stiffness (k_{12}) and the optimal yielding displacement of the non-linear connection ($d_{y,12}$). The results obtained from the sensitivity analysis have then been applied to a reference building representative of ordinary RC post World War II European buildings; in this context, a design method for the retrofit solution carried out from outside is proposed. Finally, through non-linear time-history analyses, the effectiveness of the design method and the accuracy of the results have been validated.

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