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MULTI-PERFORMANCE DESIGN OF DISSIPATIVE BRACING SYSTEMS THROUGH INTERVENTION COST OPTIMIZATION

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Abstract

In recent years, the need of retrofit interventions on existing buildings is increasing, also due to the high economic losses recorded after the recent seismic events. The intervention techniques based on stiffening and energy dissipation are one of the most widespread among the several available. Despite the great diffusion of this kind of interventions, there are still no clear and shared rules about design principles and, above all, about the performance objectives to be followed within the design. In particular, there is a lack of design approaches based on optimization of intervention costs and cost-benefit assessment. This paper concerns the optimal design of dissipative braces for seismic retrofitting of existing reinforced concrete buildings in the context of a multi performance design problem. Topological and dimensional optimization of the braces is achieved by minimizing the real intervention cost through an innovative formulation of the objective function. The procedure is based on elastic linear analyses, keeping in count the inelastic behavior of dissipative devices and concrete frame through linear equivalent schemes. The reliability of such linearized schemes is discussed by a comparison with structural response obtained with Non-Linear dynamic analyses on a case study structure. Further, it will be shown how the procedure allows, besides the design of the braces characteristics, a critical assessment and choice of the performance levels in order to optimize the cost benefit ratio of the intervention, thus showing its effectiveness as decision-making tool for seismic risk mitigation.

Keywords: Bracing; Energy Dissipation; Optimization; Retrofit; Multiperformance.

1 INTRODUCTION

Nowadays, the seismic protection of existing buildings is one of the most relevant topics in earthquake engineering. The recent seismic events that hit many countries all over the world (Umbria-Marche 1997, Italy; L'Aquila 2009, Italy; Christchurch 2011, New Zealand; Tohoku 2011, Japan; Emilia 2012, Italy; Centro Italia 2016, Italy) have highlighted the weaknesses of existing building heritage, causing many casualties and very high economic losses [1]–[3]. For many years politics, economists and the whole scientific community have not appropriately considered the problem of the enormous costs of reconstruction due to seismic events that have been very high [4] [5]. In the recent years, the need of a seismic risk mitigation strategy has emerged. In order to gain this goal, decision making tools are needed to obtain adequate seismic protection with the least financial investments possible.
To this aim, the existing design methodologies are often inappropriate because don't keep in count the financial aspects of a retrofit interventions [6]. Therefore, new methodologies based on seismic risk assessments are substituting traditional performance based approaches [7]–[10].
The risk-based methodologies allow to evaluate the economic losses of an asset throughout its nominal life by correlating seismic hazard, structural response and damages. The aim of retrofit interventions is to mitigate the damages and consequent economic losses by modifying the structural response. The intervention will be profitable if its cost is commensurate with the gain obtained in terms of damage reduction, in other words in the presence of an advantageous cost-benefit ratio. In this moment, there is a lack of methodologies capable of performing a cost-benefit assessment for retrofit interventions.
Among the several intervention techniques available, the most interesting in order to find an optimal cost/benefit ratio are the ones based on stiffening and energy dissipation [11]–[15]. The main advantage of this techniques is their potential low invasiveness and their broad adaptability to different performance requests.
Many design process and methodologies are available in the international literature for dissipative bracing design, some of them pursue an optimality criterion (e.g. minimum interstory drift, minimum base shear, maximum energy dissipation) [16]–[23], while others allow to find the brace characteristic and disposition just by guaranteeing some performance requirements through simplified procedures [24]–[28]. The design procedures based on an optimality criterion are commonly based on complex models that explicitly keep in count the nonlinear effects, while the simplified procedures are commonly based on the use of single degree of freedom (SDOF) equivalent models. Among the procedure previously cited, only few of them foreseen a multiperformance approach and, moreover, none of them foresee a risk-based design for retrofit intervention, even in the case the cost intervention is explicitly kept in count [20]–[22].
In this paper is adopted the optimization procedure proposed in [29], that allow a multiperformance design of braces by minimizing the intervention cost and by constraining the interstory drift ratio (IDR). The structural performance is assessed through linear analyses by adopting a model with a linearized behaviour of both existing frame and dissipative elastoplastic device. On the case study presented herein, will be shown how this procedure allows a dimensional and topological optimization of the braces by minimizing the intervention costs. Furthermore, a comparison between the intervention costs obtained for several performance requests is shown, in order to evidence the effectiveness of this procedure as a decision-making tool.

2 STRUCTURAL BEHAVIOUR AND MODELLING

The optimization procedure adopted herein is based on elastic linear analyses. The structural model keeps in count the non-linear behaviour of existing frame and dissipative braces through properly developed linearization schemes. The linearized secant stiffness of the frame is calculated on the basis of the maximum ductility demand expected on structural elements, while the braces behaviour is linearized within the procedure on the base of calculated local displacement demands. The global stiffness matrix is then assembled by adding to the un-braced frame stiffness matrix, the contribution of each brace assessed within the procedure. Similarly, the energy dissipated by each brace is assessed within the procedure and added to the energy dissipated by the frame in order to compute a global equivalent damping ratio for the braced building and reduce the seismic forces.

2.1 Existing frame

The existing frame behaviour is assumed as bilinear, as it is shown in Figure 1. The secant stiffness K_0 is calculated in correspondence of the final desired displacement of the braced structure, u_d, which can be determined on the base of the drift profile used as a constraint in the optimization procedure. Once known such final displacement, a reasonable estimate of the secant linear stiffness of each structural element can be determined on the base of expected global ductility demand through the following expressions [30]:

$$I_{ieq} = \frac{I_i}{\mu} \tag{1}$$

$$\mu = \frac{u_d}{u_y} \tag{2}$$

where: I_{ieq} is the equivalent inertia of the i-th section, I_i is the inertia of the i-th section, μ is the global ductility demand, u_d is the final displacement and u_y is the yielding displacement of the frame.

In correspondence of the final desired displacement, u_d, the energy dissipated by the frame is assessed through the following expression:

$$E_0 = 0.33 \times 4 \times \left(F_y u_d - F_y u_y \right) \tag{3}$$

where: F_y and u_y are the yielding force and the yielding displacement of the existing frame, respectively.

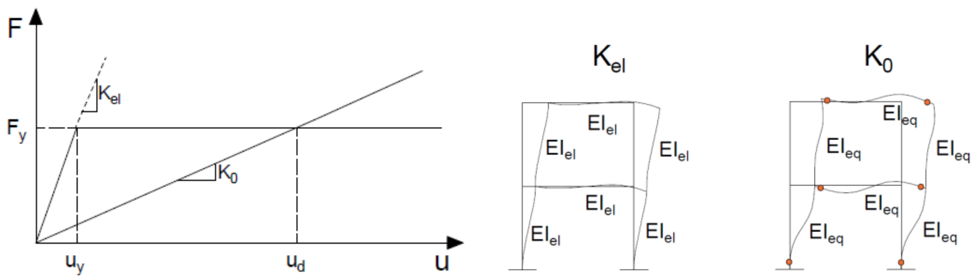


Figure 1: Existing frame behaviour, elastic stiffness K_{el} and equivalent secant stiffness K_0.

2.2 Dissipative brace

The dissipative brace is composed by a series disposition of an elastic steel truss and an elastoplastic dissipative device. The global bilinear behaviour of the brace is linearized in correspondence of its maximum displacement, u_u. Differently than for the existing frame, the linearization process of the brace is performed within the optimization procedure on the base of calculated local displacement of each brace. Given the force-displacement relationship of Figure 2, the linear secant stiffness of each brace and the associated dissipated energy can be calculated as follows:

$$K_i^B = F_{ui}^B / u_{ui}^B \tag{4}$$

$$E_i^B = 4 \times \left(F_{yi}^B u_{ui}^B - F_{ui}^B u_{yi}^B \right) \tag{5}$$

where: K_i^B, E_i^B F_{yi}^B, u_{yi}^B, F_{ui}^B, u_{yi}^B are the equivalent stiffness, the energy dissipated, the yielding force, the yielding displacement, the ultimate force and the ultimate displacement of the i-th brace, respectively.

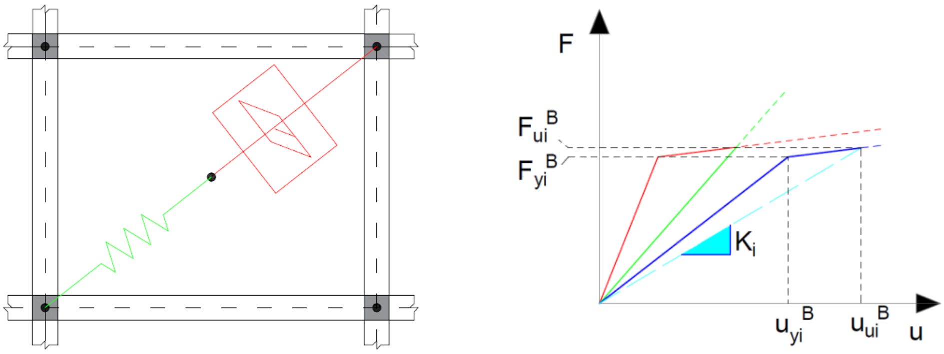


Figure 2: Brace assemblage and force-displacement relationship adopted for the brace (blue line), the steel truss (green line) and the dissipative device (red line).

2.3 Braced structure

In Figure 3 it is shown the sketch of the assembling procedure of stiffness and dissipated energies within the procedure. The global stiffness matrix of the braced structure is assembled by simply adding the contribution of the existing frame, K_0, and the contributions of each brace, represented through an influence matrix ΔK_i, using the following expression:

$$\mathbf{K} = \mathbf{K}_0 + \sum_i \mathbf{\Delta K}_i \tag{6}$$

Similarly, the total energy dissipated by the braced structure, E, is assembled as follows:

$$E_D = E_0 + \sum_i E_i^B \tag{7}$$

By using the total energy dissipated obtained through Eq.7, the equivalent damping ratio of the system can be assessed as follows:

$$\zeta_{eq} = \frac{1}{4\pi} \frac{E_D}{E_p} + \zeta_v \tag{8}$$

where E_p is the elastic energy of the braced structure and ζ_v is the added viscous damping.

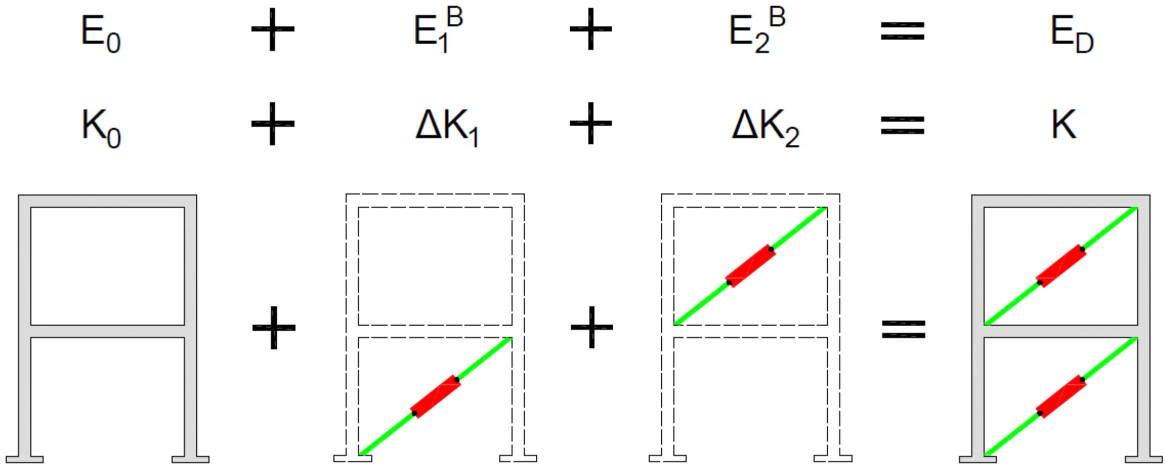


Figure 3: Assembling of stiffness and dissipated energies within the procedure.

3 OPTIMIZATION PROCEDURE

The optimization procedure is asked to provide the dimensional and topological characteristics of the dissipative braces. With this aim and given the brace characteristics exposed in section §2.2, the unknowns of the optimization problem, namely the independent variables, are defined as follows:

$$\mathbf{x} = \left[\mathbf{A}^{\mathbf{A}} \quad \mathbf{F}_{\mathbf{y}}^{\mathbf{D}} \quad \mathbf{u}_{\mathbf{y}}^{\mathbf{D}} \right] \tag{9}$$

where: $\mathbf{A}^{\mathbf{A}}$ is a vector whose generic component A_i^A is the area of the i-th steel truss, $\mathbf{F}_{\mathbf{y}}^{\mathbf{D}}$ is a vector whose generic component F_{yi}^D is the yielding force of the i-th dissipative device and $\mathbf{u}_{\mathbf{y}}^{\mathbf{D}}$ is a vector whose generic component u_{yi}^D is the yielding displacement of the i-th dissipative device.

The objective function of the optimization procedure can be defined, as follows:

$$O.F.(\mathbf{x}) = \sum_{i=1}^{n} (C_i^S(A_i^A) + C_i^D\left(F_{yi}^D\right) + C_i^M(A_i^A) + \sum_{r=1}^{n_p} C_r^F(\mathbf{x}) \tag{10}$$

where: C_i^S is the cost function of the steel elements, C_i^D is the cost function for the dissipative devices, C_i^M is the cost function for the masonry works, C_i^F is the cost function for the foundation system interventions.

A detailed description of Eq. 10 may be found in [29]. The constraints of the procedure are expressed in terms of interstory drift ratio and can be formalized as follows:

$$\mathbf{h}_{\mathbf{u}}(\mathbf{x}) = \mathbf{IDR}(\mathbf{x}) - \mathbf{IDR}^{Lim} \tag{11}$$

where: $\mathbf{h}_{\mathbf{u}}$ is the vector of constraints, **IDR** is the vector containing the interstory drift ratio obtained from structural response and $\mathbf{IDR}^{Lim}$ is the vector of maximum interstory drift ratio desired.

The optimization procedure can consequently be defined as follows:

$$\min \quad O.F.(\mathbf{x})$$
$$\left\{ \mathbf{h_u}(\mathbf{x}) = \mathbf{IDR}(\mathbf{x}) - \mathbf{IDR}^{Lim} \le 0 \right. \tag{12}$$

4 NUMERICAL EXAMPLE

The procedure has been applied to a case study which represent the typical Italian reinforced concrete building realized in the absence of seismic provisions. It has six floors with a height of 3m, three spans with a length of 5m. The considered floor mass is 112.5kNs^2/m. The beams have a 30x50cm section, external columns have a 30x30cm section and central columns are tapered along the building height: a 30x60cm has been considered for the first three floors while a 30x30cm section has been considered for the other three floors. Braces are located in the central span of the building with a diagonal disposition, as shown in Figure 4.

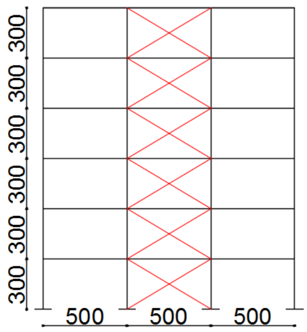


Figure 4: Case study structure and brace initial arrangement.

The optimization problem aims to find the optimal size and disposition of braces in order to obtain the desired interstory drift ratio for different action levels. The considered action levels are related to the Italian site of Reggio Calabria (LAT 38.11, LON 15.66) for the return periods (T_r) of 30, 101, 475 and 975 years, soil C and T1 site conditions as defined in the italian seismic code [31]. For each of these action levels the acceptance criteria for interstory drift ratio are defined as 0.1%, 0.4%, 0.7% and 1%, respectively. By considering these action levels and acceptance criteria, the optimization problem of Eq. (12) is extended to a multi-performance problem that can be posed as follows:

$$\min \quad O.F.(\mathbf{x})$$
$$\begin{cases} \mathbf{h}_{u-30}(\mathbf{x}) = \mathbf{IDR}_{30}(\mathbf{x}) - \mathbf{IDR}_{30}^{Lim} \le 0 \\ \mathbf{h}_{u-101}(\mathbf{x}) = \mathbf{IDR}_{101}(\mathbf{x}) - \mathbf{IDR}_{101}^{Lim} \le 0 \\ \mathbf{h}_{u-475}(\mathbf{x}) = \mathbf{IDR}_{475}(\mathbf{x}) - \mathbf{IDR}_{475}^{Lim} \le 0 \\ \mathbf{h}_{u-975}(\mathbf{x}) = \mathbf{IDR}_{975}(\mathbf{x}) - \mathbf{IDR}_{975}^{Lim} \le 0 \end{cases} \tag{13}$$

Where $\mathbf{h}_{u\text{-}j}$ is the vector of constraints for the j-th hazard level, $\mathbf{IDR}_j$ is the vector of IDR obtained for the j-th hazard level and $\mathbf{IDR}_j^{Lim}$ is the vector of acceptance criteria adopted for the j-th hazard level.

The results of the optimization problem are shown in Figure 5. Brace stiffness and strength reduces along the height with the exception of the fourth floor where the braces are sensibly stiffer than the floor below in order to balance the stiffness reduction due to the column tapering. In Figure 6a are shown the interstory drift ratio obtained with the elastic linear analyses performed within procedure, as it can be seen the drift are very regular along the height, with the exception of the last floor for events with low rate of occurrence. Furthermore, it should be observed that the IDR for T_r=101yrs are quite lower than the maximum drift allowed (i.e. IDR_{101}^{Lim}=0.4%). In order to verify the reliability of the linearization procedure, several Non-linear-dynamic analyses are performed in the software Opensees [32] by considering the ductile flexural behaviour and by using the set of natural ground motions described in [33]. Despite of several modeling possibilities of fragile and degradation phenomena [34]–[38], it has been chosen to perform analyses with a model coherent with the one adopted within the procedure that does not take into account such phenomena, further information about modeling assumptions may be found in [29] and [39]. The average interstory drift ratio obtained through NL time history analyses are in good agreement with the one obtained through the procedure, proving the reliability of linearization schemes adopted. Furthermore, a parametric study has been performed solving several single-performance problems accepted through the optimization problem herein proposed by adopting different maximum IDR. In Figure 7 is shown the variation of the intervention cost with the maximum IDR accepted. It can be noticed that maintaining the structure in the elastic field (i.e. IDR <0.4%) considerably increases the costs of intervention, while admitting slight plasticization (i.e. IDR> 0.7%) allows to contain costs considerably, as well as using a multi-performance approach analogous to the one described above (i.e. MP in Figure 7). The intervention cost thus evaluated, if compared with the expected annual losses due to the achievement of the requested performances, can therefore be used to evaluate which performance configuration guarantees the optimal cost-benefit ratio.

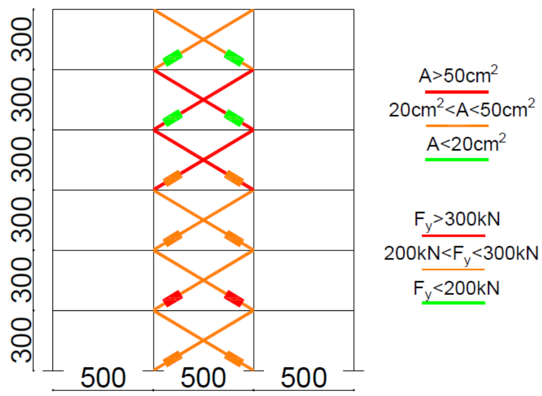
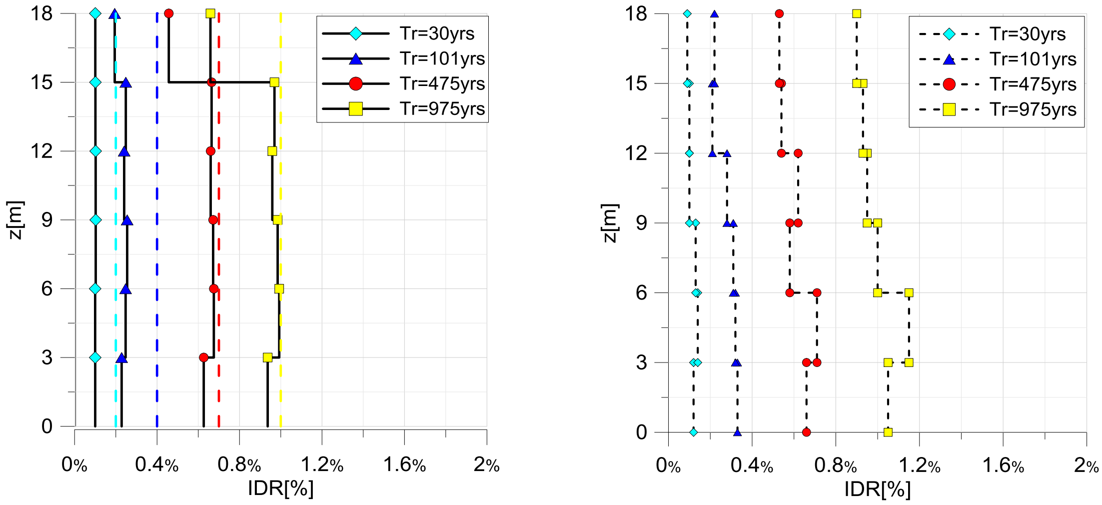


Figure 5: Brace characteristics and arrangement of the optimal solution.



(a) (b)

Figure 6: Structural response of the braced structure: IDR obtained through elastic linear analyses within the procedure (solid lines) and IDR^{Lim} imposed as constraints (dashed lines) (a); average IDR obtained through NL time history analyses.

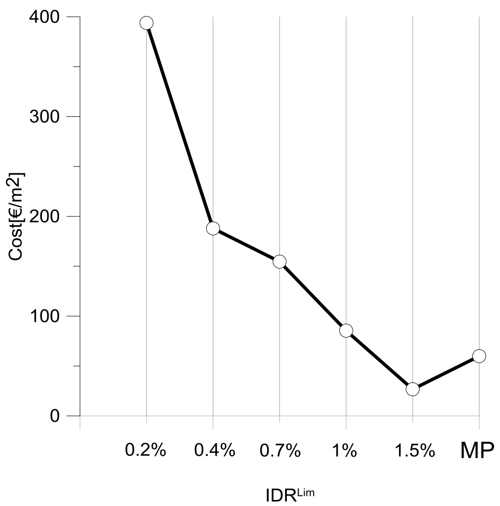


Figure 7: Total intervention cost for different maximum IDR.

5 CONCLUSIONS

In this paper is presented an optimization procedure which allows the design of bracing systems by minimizing the intervention costs through a multi-performance approach. In order to keep in count the nonlinear behaviour of frame and braces, equivalent linearization schemes are adopted. The analyses on the case study have shown how the braces designed through this procedure allow a regularization of the building behaviour along the height and the achievement of the desired performance requirements for all the considered action levels. The results obtained with NL analyses are in good agreement with the one obtained with linear analyses performed within the procedure, showing how the linearization schemes are reliable in order to describe the structural behaviour even when marked non-linear behaviour is expected. This procedure is therefore able to solve a complex multi-performance problem by linear analysis, with the great advantage of limiting the computational times and without the need to adopt complex structural models with explicit modeling of nonlinearities, thus showing itself as a useful design tool. Furthermore, the comparison of results in terms of intervention costs, once given several performance requirements, show the effectiveness of the procedure as a fast and reliable tool to estimate the intervention cost and choice the performance requirements to optimize cost-benefit ratio too.

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