Modeling and Nonlinear Seismic Analysis of Framed Structures Equipped with Damped Braces

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Abstract: - The supplementary energy dissipation represents an efficient technique for the seismic protection of structural systems. Specifically, the insertion of steel braces equipped with damping devices proves to be effective in order to enhance the performance of a framed building under horizontal seismic loads. In the last decades several applications were experienced in many countries, adopting damping systems with different characteristics, depending on both arrangement of the damped braces and kind of damping devices. In this paper, the attention is focused on the modeling and nonlinear seismic analysis of framed structures equipped with friction, metallic yielding, viscoelastic and viscous dampers. To check the effectiveness of the damped braces, a six-storey reinforced concrete (r.c.) structure, representative of a symmetric framed building, designed in a medium-risk seismic region, is supposed to be retrofitted as in a high-risk seismic region.

Key-Words: - Framed buildings, damped braces, friction dampers, hysteretic dampers, viscoelastic dampers, viscous dampers.

1 Introduction
Among the techniques of passive control that have had in the last two decades real application for the control of the seismic response of buildings, that based on the dissipation of a large portion of the energy transmitted by the earthquake to the structure can be considered very effective. Actually a wide variety of energy dissipating devices is available for the passive control of vibrations [1-6]. The extra cost of the damped braces is widely recovered by the achievable advantages: high level of seismic protection of a framed structure, considerable reduction of the reparations required after a strong earthquake, functionality and practicability of the buildings even after a such earthquake.

In the case of seismic retrofitting the properties of the framed structure are known previously, while for the seismic design of a new structure it is necessary to select, starting from the level of protection assigned for the unbraced frame, the properties of the frame itself as well as those of braces and dampers. For a widespread application of supplemental dampers, comprehensive analysis, design and testing guidelines should be available. New seismic codes (e.g., Italian code 2008, NTC08, [7]) allow for the use of dissipative braces for the seismic retrofitting of framed buildings, while only few codes provide simplified criteria for their design (e.g., FEMA 356, 2000, [8]).

After a brief description of the arrangements of damped braces to be inserted in framed buildings, features and limitations of the considered damper models are discussed. Then, the results of an analytical study are presented comparing the nonlinear dynamic responses of a six-storey r.c. framed building before and after retrofitting by hysteretic damped braces.

2 Modeling
The damped braces available in literature differ for the features of the supplemental damping devices and/or for the particular arrangement of the braces supporting them, using classical (e.g. cross braces, Fig. 1a; single diagonal brace, Fig. 1b; chevron braces, Fig. 1c), geometrically amplified (e.g. toggle-brace, Fig. 1d; scissor-jack, Fig. 1e). The supplemental damping devices may be classified in: displacement-dependent (e.g., friction damper, FR; metallic-yielding damper, YL), velocity-dependent (e.g., viscoelastic damper, VE; viscous damper, VS) and self-centring (e.g. shape memory alloys, SMA). Apart from the system in Fig. 1a, where the braces are assumed to be slender enough to buckle elastically (practically negligible buckling load), in the other systems depicted in Figs. 1b-1d the braces are designed not to buckle. For clarity, referring to the schemes in Fig. 1, the properties of the system at a storey are indicated in what follows: $K_B$ is the
2.1 Displacement-dependent damping
FR and YL dampers have a stable hysteretic behavior and present a mechanism of energy dissipation depending on the storey drifts; their activation happens when preset stress levels are reached or overcome. Using the cross-bracing system shown in Fig. 1a, either FR device in Fig. 2a or YL device in Fig. 3a, which are based on the same mechanism, have been adopted. The single diagonal system in Fig. 1b can be adopted for axially stressed devices, such as: the friction damper in Fig. 2b; the buckling-resistant unbonded brace (Fig. 3c), consisting of a core steel plate encased in a concrete filled steel tube (metallic-yielding of the interior component under reversal axial loads provides stable energy dissipation, while the exterior concrete-filled steel tube prevents local and member buckling). Moreover, damping devices can be also located between the top of chevron braces and beam mid-span (Fig. 1c), e.g.: a FR device of the kind in Fig. 2c; a YL device, consisting of multiple X-shaped steel plates (Fig. 3b).

Figure 1. Typical arrangements of damped braces.

In addition, the models should be relatively simple to carry out the analysis with a reasonable computational effort.

In what follows aspects of the analytical modeling are discussed only with reference to devices with displacement- and velocity-dependent damping [9, 10].

Figure 2. Typical friction (FR) dampers and their modeling and idealized response.

As shown in Fig. 3d, the behaviour of a YL device can be idealized by a bilinear law, which specializes as a rigid-plastic one for a FR device (Fig. 2d).

The design parameters of the FR and YL dampers are synthetically reported below:
(a) FR damper: \( N_y = \text{slip load} \);
\( N_{\text{max}} \) = tension-brace force at frame-yielding onset;
\( N^* = N_y / N_{\text{max}} \) = slip-load ratio;

(b) YL damper: 
\( N_y \) = yield load;
\( N^* = N_y / N_{\text{max}} \) = yield-load ratio;
\( K_D \) = initial damper stiffness;
\( r_D \) = damper stiffness ratio.

VE and VS devices dissipate energy, depending on the velocity of motion, due to viscoelasticity and/or viscosity of elastomers or fluids; for these devices the dissipated energy is a linear or nonlinear function of the load frequency and temperature. A typical viscoelastic device consists of layers of polymers or glassy substances bonded with steel plates (Figs. 4a,b); it dissipates energy through heat loss when subjected to direct shearing of the viscoelastic material layers. Viscoelastic or pure viscous fluid dampers consist of a moving piston immersed in a more or less viscous (compressible or incompressible) fluid (Fig. 5a).

The design parameters of the velocity-dependent dampers are synthetically reported below:

(a) VE damper: 
\( C_D \) = effective damping coefficient;
\( K'_D = G'A/h \) = storage stiffness;
\( K''_D = G''A/h \) = loss stiffness;
\( G' \) = shear-storage modulus;
\( G'' \) = shear-loss modulus;
\( A/h \) = shear area/total thickness;
\( \tan \delta = K''_D/K'_D \) = loss factor (generally within the range 0.8÷1.5)

(b) VS damper: 
\( C_D \) = damping coefficient;
\( \beta = 0 \) (\( \beta = 1 \) for linear fluid damper);
\( N_D = C_D \left[ \Delta_D \right]^\beta \) sign \( \left[ \Delta_D(t) \right] \)

As shown in Fig. 4c, the behaviour of a VE device can be simulated by a six-element generalized model (GM), which is a combination of the classical Kelvin and Maxwell models (KM and MM); the GM allows, in comparison to KM and MM, a better description of the material properties depending on the frequency. Finally, a VS device (Fig. 5b) can be considered as a specialization of a VE device assuming \( K'_D = 0 \).
the solution of Eq. (1), the following residual iteration scheme is used [11]:

\[ r^{(j)} = q^{(j)} - q_0 + \left( \frac{1}{2} \alpha \right) \Delta t (s_0 - p_0) + \left( \frac{1}{2} \alpha \right) \Delta t (s^{(j)} - p_j) \]

\[ u^{(j+1)} = u^{(j)} - H r^{(j)} \]  

(2a,b)

in which the indexes 0 and 1 refer, respectively, to the beginning and the end of the generic time step, \( q = M \dot{u} \) is the momentum vector, \( s = f[u] + C \ddot{u} \), while \( \alpha \) and \( \beta \) are suitable functions of the time step \( \Delta t \). The convergence of the iterative process is assured adopting the iteration matrix

\[ H = \left( M + \left( \frac{1}{2} + \alpha \right) \Delta t^2 K_E + \left( \frac{1}{2} + \alpha \right) \Delta t C \right)^{-1} \]  

(3)

where \( K_E \) is the elastic stiffness matrix.

4 Design and seismic analysis of the unbraced and damped braced frames

A typical six-storey residential building with a r.c. framed structure (Fig. 6a) is considered as primary structure. The primary framed building is designed according to the Italian Seismic Code in force in 1996 [12], for a medium-risk seismic region (seismic coefficient: \( C = 0.07 \)) and a typical subsoil class (main coefficients: \( R = \varepsilon = \beta = 1 \)). The gravity loads are represented by a dead load of 4.2 kN/m\(^2\) at the top floor and 5.0 kN/m\(^2\) at the other floors, and a live load of 2.0 kN/m\(^2\) at all the floors. Masonry infill walls, regularly distributed in elevation along the perimeter (see Fig. 6a), are considered assuming an average weight of about 2.7 kN/m\(^2\). The design is carried out to comply with the ultimate limit states. Detailing for local ductility is also imposed to satisfy minimum conditions for the longitudinal bars of the r.c. frame members. Further detail can be found in a previous work by the authors [13].

For the purpose of retrofitting the test structure from a medium-risk region up to a high-risk seismic region, diagonal steel braces equipped with dampers are inserted, at each storey, as indicated in Figs. 6b and 6c. For brevity, only the case of YL dampers (HY damped braces) is considered. The design of the damped braces is carried out considering seismic loads provided by NTC08 for a high-risk seismic region (peak ground acceleration on rock: \( a_g = 0.27 \text{g} \); maximum spectrum amplification coefficient: \( F_0 = 2.5 \)) and subsoil class B on a level ground (site amplification factor: \( S = S_h S_t = 1.13 \); PGA = 1.13x 0.27g = 0.31g). The following values are assumed for design: ductility of the frame, \( \mu_u = 1.5 \), and stiffness hardening ratio, \( r_l = 0\% \); ductility of the damper, \( \mu_D = 10 \), and stiffness hardening ratio, \( r_D = 0\% \). Details on the design procedure can be found in previous
works [11, 13]. Finally, the distribution of the global stiffness between the braces of a storey is obtained assuming the contribution of the dissipative braces as proportional to the ratio between ultimate shear and design shear, calculated for the weakest column of the considered plane frame at that storey. Then, the strength distribution is assumed proportional to the stiffness distribution.

To check the effectiveness of the YL damped braces for controlling the local damage undergone by the r.c. frame members, in Fig. 7 the ductility demands attained by girders are shown along the frame height, in the cases of unbraced frame (UF) and damped braced frame (DBF) subjected to sets of real ground motions. More precisely, sets of seven real motions, selected according to the procedure proposed by Iervolino et al. [14], are considered in order to match (on the average) the design spectra assumed by NTC08 for different limit states (damage, SLD; life safety, SLV; collapse, SLC).

Figure 6. R.c. framed building (dimensions in cm).

(a) Unbraced framed structure (UF)
(b) Damped braced structure (DBF).
(c) Damped braced plane frame.

Figure 7. Maximum ductility demand to girders.

The above results are obtained as an average of those separately obtained for the sets of real motions corresponding to a limit state. As can be observed, the reduction of the ductility demand due to insertion of damped braces, in comparison with the case of the unbraced frame, is even more than 100% for both serviceability (i.e. SLD) and ultimate (i.e. SLV and SLC) limit states. Analogous results, omitted for the sake of brevity, have been obtained also for the columns. However, under strong ground motions (as for SLV and SLC) some columns can undergo ductility demand greater than that in the unbraced structure, due to high variation of the axial force inducing a reduction of the flexural capacity (e.g., for corner columns, subjected to rather low gravity loads).
5 Concluding remarks
A step-by-step procedure for nonlinear seismic analysis of framed structures equipped with damped braces was presented. For this purpose, suitable models were adopted for simulating the response of a frame member and that of different kinds of dampers (friction, metallic yielding, viscoelastic, viscous) under strong ground motions. To show the effectiveness and potentiality of the above procedure, the results of nonlinear dynamic analyses were presented comparing the responses of a six-storey r.c. framed building, primarily designed in a medium-risk seismic region, and successively retrofitted as in a high-risk seismic region by insertion of braces equipped with YL dampers.

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References: