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SEISMIC DAMAGE CONTROL OF DAMPED BRACED RC FRAMES

Alfonso VULCANO and Fabio MAZZA

Dipartimento di Strutture, Università della Calabria, Arcavacata di Rende (Cosenza), Italy

ABSTRACT

A simplified yet reliable procedure is proposed for a practical design of a friction-damped bracing to control the seismic damage of a reinforced concrete (r.c.) framed structure. Supposing the properties of the frame are known, suitable distribution laws are assumed for the stiffness of the braces and for the slip load in the damping devices; the optimum level of the slip load ratio is obtained minimizing the global damage of the framed structure by using a suitable *damage index*. To check the effectiveness and reliability of the design procedure a numerical investigation is carried out considering the nonlinear seismic response of a single-degree-of-freedom (SDOF) system representing a damped braced frame and that of three-, five- and ten-storey structures. Either an elastic-perfectly plastic model (EPM) or an evolutionary-degrading model (DM) are adopted to simulate the hysteretic behaviour of the r.c. frame members; an EPM simulates the hysteretic response of the friction-damped bracing at each storey. Design charts are obtained which, for a designated damage level of the framed structure, enable the selection of the stiffness distribution of the braces once the optimum slip-load distribution has been selected. The results show the effectiveness of the adopted design procedure and the importance of considering mechanical degradation for r.c. frame members.

KEYWORDS

Passive control of vibrations; energy dissipation; damping devices; damped steel braces; damped braced frames; damped bracing design; r.c. frame retrofitting; degrading models; seismic damage; damage indexes.

INTRODUCTION

The traditional aseismic design - based on the use of the inelastic design spectrum obtained according to the average ductility level accepted for the structure - needs additional design and detailing rules such that the average ductility level be representative of the ductility demand for the dissipating zones of the structure and these zones be provided with a good dissipating capacity. An elastic-perfectly plastic model (EPM) can be considered reliable enough to predict the inelastic behaviour.

A novel technique aims to reduce the ductility demand of framed structures through supplemental damped steel bracing (for a synthetic discussion about different typologies and dissipation mechanisms, see, e.g., papers by Vulcano, 1993 and 1994). This can lead to use, for the frame members, light additional design and detailing rules, less conservative than those adopted in traditional design approach; on the other hand, these rules often turn out to be not satisfied for existing framed building to be retrofitted by using damped bracing. Therefore, whether inelastic deformations of a reinforced concrete (r.c.) framed structure occur during severe earthquakes, the mechanical degradation can become very important and the inelastic design spectrum should account for it. Both a suitable damage measure more conservative than that by displacement ductility factor and a degrading model (DM) more accurate than EPM should be adopted for a reliable design.

In previous papers (Vulcano, 1993 and 1994; Vulcano and Mazza, 1995) a simplified yet reliable procedure was developed for a practical seismic design of a damped bracing to attain a designated protection level (corresponding to an accepted damage level) for a framed structure. An EPM was assumed for both the frame members and the damped bracing; the effectiveness of the damped bracing was evaluated as based on some *performance index* (e.g., a *generalized performance index* accounting for the energy area and the maximum energy in the time, including both the contributions of absorbed and dissipated energy in the unbraced and damped braced frames).

In this paper the problem of the design is reconsidered assuming for the r.c. frame members also a DM with evolutionary stiffness and strength. To evaluate the effectiveness of the damped bracing, *damage indexes* are introduced on the basis of a suitable *normalized damage functional*, which is also adopted for simulating the mechanical degradation by the DM. To compare the effectiveness and reliability of the design procedure when adopting either EPM or DM for r.c. frame members, a numerical investigation is carried out considering the nonlinear seismic response of a single-degree-of-freedom (SDOF) system representing a damped braced frame and that of three-, five- and ten-storey r.c. frames with a friction-damped bracing.

TEST STRUCTURES AND DESIGN CRITERIA

For sake of clearness, in the following discussion the three-, five- and ten-storey test structures schematically shown in Fig. 1 are considered. They consist of r.c. frames with damped steel braces equipped with the friction-device of the kind proposed by Pall and Marsh, 1982 and shown in Fig. 2a. It should be mentioned that, although a friction-damped steel bracing is considered, the qualitative conclusions which will be drawn apply also to other kinds of damped bracing systems.

The r.c. frames of the test structures have been designed, according to C.E.B. Seismic Code (1987), as "weak beam-strong column" structures in a high-risk seismic zone, assuming soil profile S_2; the friction-damped bracing system has been designed according to the criteria which are illustrated below. Details on design and properties of the r.c. frames and the friction-damped bracing can be found in a previous paper by the authors (Vulcano and Mazza, 1995). However, it is useful to remind the fundamental vibration periods of the three-, five- and ten-storey frames, i.e., 0.493, 0.902 and 1.135 sec, respectively, as well as the main equilibrium equation in the friction device with reference to the phase in which slippage occurs (Fig. 2b):

$$N_g = 2N_l - N_{cr} \tag{1}$$

where N_g is the global slip load in the tension brace, N_l is the local slip load in each slip joint and N_{cr} is the buckling load of the compression brace. Assuming practically $N_{cr}=0$, the global response of the friction-damped bracing at the generic storey can be idealized by the elastic-perfectly plastic law shown in Fig. 2c (F=horizontal force; Δu=interstorey drift).

The design of damped braces for the seismic protection of framed structures has been treated by other authors following different approaches (Austin and Pister, 1985; Braga and D'Anzi, 1994; Ciampi *et al.*, 1992; Filiatrault and Cherry, 1990). In this paper, supposing the properties of the framed structure are known, the damped bracing system is designed according to criteria analogous to those proposed in the papers already mentioned (Vulcano, 1993 and 1994; Vulcano and Mazza, 1995) for achieving a designated protection level (corresponding to an accepted damage level) of the framed structure.

These design criteria are summarized below with reference to an assumed lateral load pattern, e.g. that corresponding to the first vibration mode of the elastic braced frame:
- the elastic-stiffness distribution of the braces is similar to that of the bare frame, that is, the same value of the stiffness ratio $K^*=K_b/K_f$ is assumed at each storey between the two lateral elastic stiffnesses provided, respectively, by the damped bracing system (before the slippage in the friction device) and by the bare frame;
- the distribution law of the global slip load N_g in the damping devices at different stories is similar to that of the elastic axial force induced by the lateral loads in the tension braces before the slippage, so that the totally dissipated energy be as large as possible;
- the selection of the N_g value at the generic storey is restricted to the range (N_{min}, N_{max}), where the lower bound, reasonably assumed as $N_{min}=0.5N_{max}$, should ensure that the device does not slip under normal service loads and moderate earthquakes, whereas the upper bound should avoid any yielding of the frame members before slippage in the device as well as the occurrence of undesirable phenomena in the frame columns (e.g., buckling, brittle failure in r.c. columns, etc.);
- due to the last two assumptions, the slip load can be characterized at each storey by the same value of the slip-load ratio $N^*=N_g/N_{max}$;
- the optimum slip-load distribution is selected by a criterion of minimization with reference to some *damage index* intending to evaluate the effectiveness of the damped bracing system as the ratio of the damage level for the damped braced frame to that for the unbraced frame.

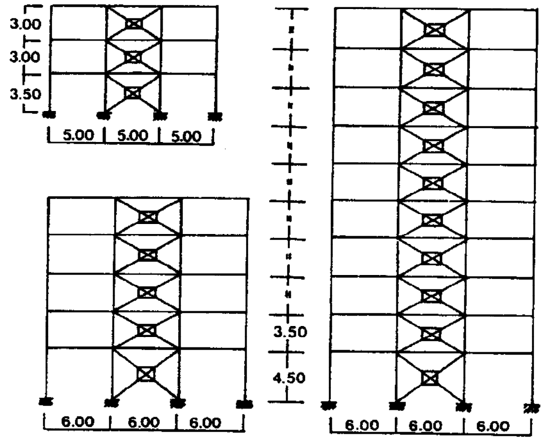


Fig.1. Test structures.

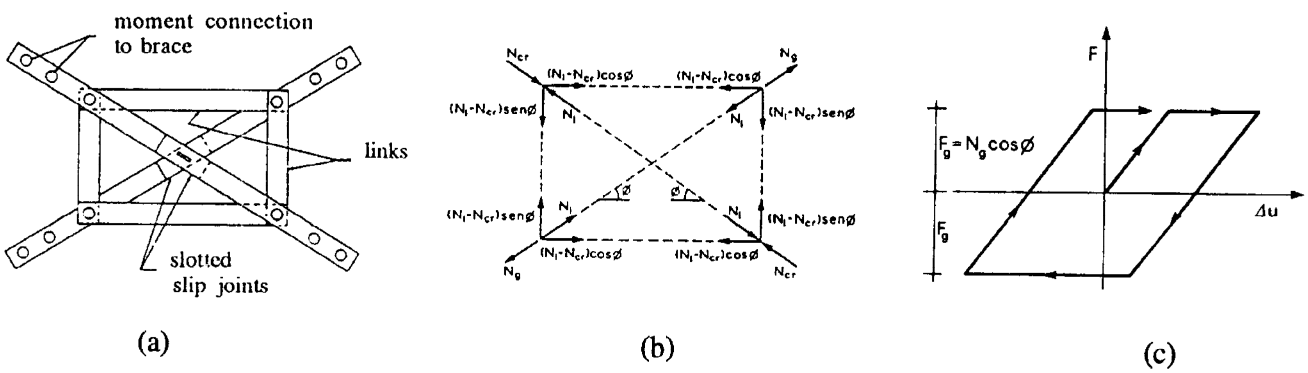


Fig. 2. Friction device (a), forces under slippage (b) and idealized response (c) of the friction-damped bracing.

DAMAGE INDEXES AND MODELING OF THE FRAMED STRUCTURE

As seen in the previous Section, the optimum slip-load distribution is selected by a criterion of minimization with reference to some *damage index* related to the damage level for the unbraced and damped braced frames. The damage level D ($0 \leq D \leq 1$) can be evaluated by a suitable *normalized damage functional* among those available in the literature (for a general discussion see, e.g., Cosenza and Manfredi, 1992). For instance, with reference to the moment-curvature (M-χ) law in Fig. 3, D can be evaluated as

$$D = \frac{\mu - 1}{\mu_{u,mon} - 1} \qquad (\mu: \mu_{max}, \mu_c, \mu_h) \qquad (2a)$$

if μ is a *ductility factor*, e.g. in turn:

$$\mu_{max} = \frac{\chi_{max}}{\chi_y} \ \textit{(maximum)}; \qquad \mu_c = \frac{\max(\chi_{p,i})}{\chi_y} + 1 \ \textit{(cyclic)}; \qquad \mu_h = \frac{E_h}{M_y \chi_y} + 1 \ \textit{(hysteretic)}$$

being χ_{max} and $\chi_{p,i}$ the maximum absolute values attained by the actual curvature and the plastic curvature in the ith loading cycle, respectively, while M_y and χ_y correspond to the yielding point and E_h is the hysteretic energy. When using a degrading law as that in Fig. 3, μ_c is calculated accounting for $\chi_{p,i}$ also with reference to a reloading branch; however, to make the results comparable to those by an elastic-perfectly plastic law, a net value is calculated subtracting χ_y from the total value (one time until the reloading branch is completed).

In alternative to Eq. 2a, D can be evaluated as

$$D = \frac{\mu_F}{\mu_{Fu,mon}} \qquad (\mu_F: \mu_{PA}, \mu_{BV}, \mu_{PF}) \qquad (2b)$$

if μ_F is a suitable *damage functional*, e.g. in turn:

$$\mu_{PA} = \mu_{max} + \beta_h (\mu_h - 1); \qquad \mu_{BV} = \sqrt{(\mu_{max}-1)^2 + \{a[2(\mu_h-1)]^b\}^2}; \qquad \mu_{PF} = \sum_{i=1}^{n} (\mu_{c,i} - 1)^c$$

respectively proposed by: Park and Ang, 1985; Banon and Veneziano, 1982; Banon *et al.*, 1981 (*plastic fatigue*). With reference to r.c. structures, the following average values suggested in the literature can be assumed for the parameters appearing in the above expressions: β_h=0.15; a=1.1 and b=0.38; c=1.8. The values $\mu_{u,mon}$ (see Eq. 2a) and $\mu_{Fu,mon}$ (see Eq. 2b) represent the ultimate ones which can be attained in a monotonic test for μ and μ_F, respectively (in particular, it may be simply assumed $\mu_{PAu,mon}=\mu_{u,mon}$ and $\mu_{PFu,mon}=\{\mu_{u,mon}-1\}^b$).

In the following discussion the performance of the damped braced frames is evaluated with reference to *damage indexes* defined as a ratio:

$$\mathrm{DPI} = \frac{D_m^{(BF)}}{D_m^{(UF)}} \tag{3}$$

where $D_m^{(..)}$, representing in turn the level of the damage for damped-braced (BF) or unbraced (UF) frame, is calculated as mean (e.g., arithmetic or weighted) of the values assumed by a suitable *normalized damage functional* D (see Eqs. 2) in all the end (critical) sections of the girders and columns. In spite of the general symbol DPI, the following more appropriate symbols are adopted when using μ_{max}, μ_c, μ_h, μ_{PA}, μ_{BV} and μ_{PF} for evaluating the local damage D: KDI, CDI, HDI, PAI, BVI and PFI, respectively.

To investigate about the effects of the mechanical degradation, the frame members are modeled using, besides the EPM already adopted in a previous work (Rega *et al.*, 1990), the degrading beam model (DM) represented in Fig. 4 and characterized by the M-χ degrading law shown in Fig. 3. To compare EPM and DM, the same elastic-perfectly plastic skeleton curve is assumed for the M-χ law. As shown in Fig. 3, the mechanical degradation is simulated by suitable strength and stiffness evolutionary laws depending on the damage level D and various parameters (further detail can be found in the paper by Cosenza and Manfredi, 1992). The lengths z_i and z_j of the critical zones of the beam model in Fig. 4 are assumed on the basis of a suitable increasing law depending on the maximum curvature ductility μ_{max} at the corresponding end sections.

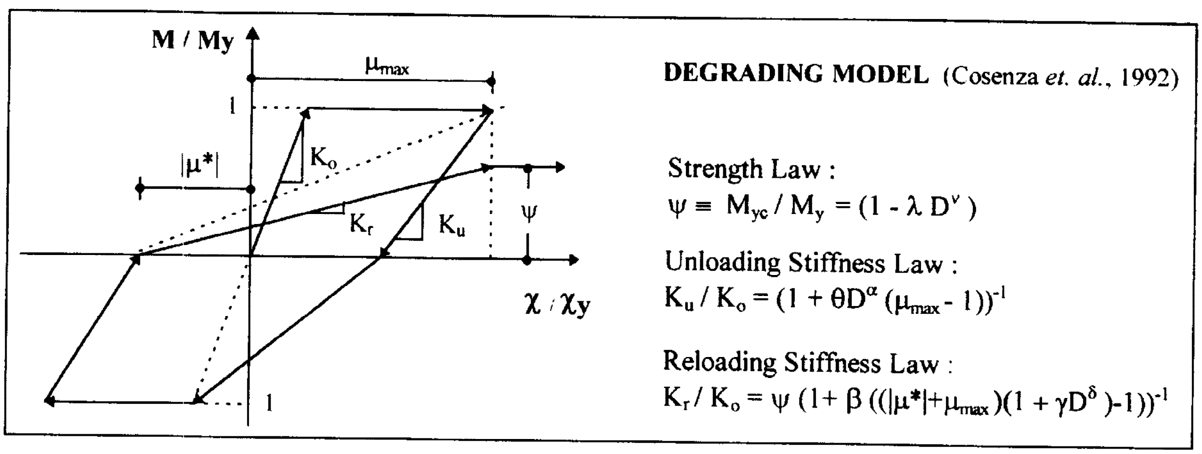


Fig. 3. Moment-curvature (M-χ) evolutionary-degrading law.

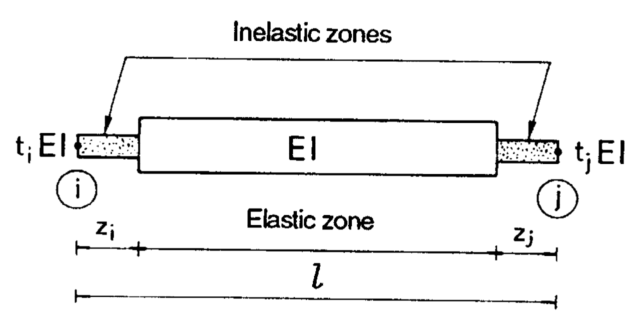


Fig. 4. Degrading beam model (DM).

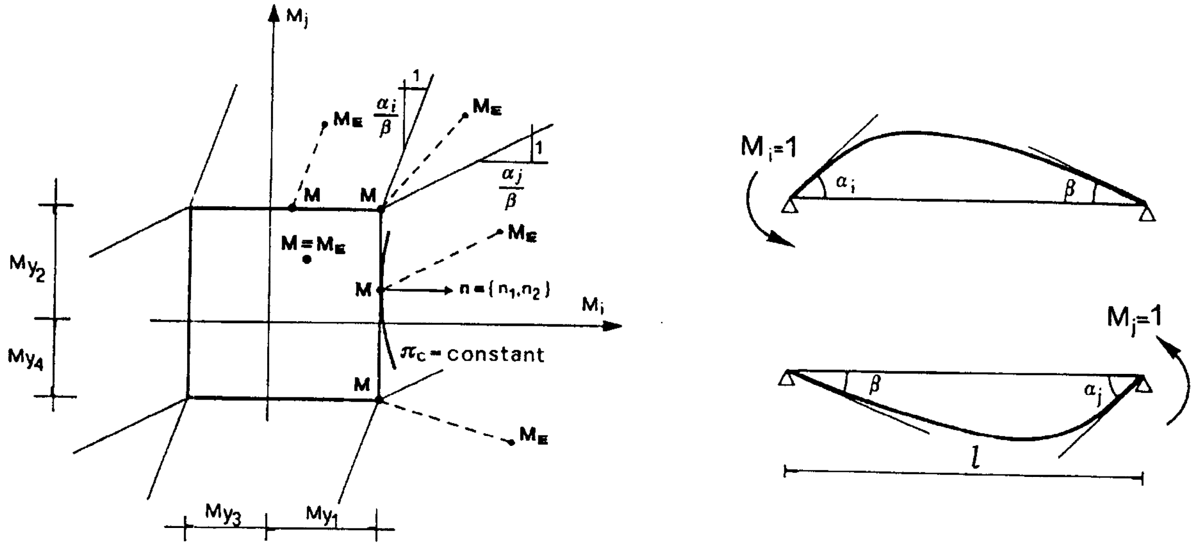


Fig. 5. Elastic-plastic solution for degrading beam model in Fig. 4.

To carry out the nonlinear analysis a procedure is adopted analogous to that described in the paper by Rega *et al.*, 1990 with reference to the EPM. This procedure is based on an implicit two-parameter time integration scheme and an initial stress-like iterative procedure. A crucial point is searching the elastic-plastic solution at each step of the analysis. More precisely, once the incremental elastic moments $M_E=\{M_{E_i},M_{E_j}\}^T$ at the end of a step are determined as due to the incremental nodal displacements, the elastic-plastic solution $M=\{M_i, M_j\}^T$ is obtained, according to Haar-Kàrmàn principle, as the point of the elastic domain with minimum distance in terms of complementary energy from the point M_E. The solution procedure is schematically shown in Fig. 5, where, more in general, the flexibility coefficients (α_i, α_j, β) of the DM can be taken into account.

RESULTS

To check the effectiveness and reliability of the design procedure a numerical investigation is carried out considering the nonlinear seismic response of a SDOF system and that of the test structures in Fig. 1. Many time-step dynamic analyses have been carried out using the procedure mentioned in the previous Section and adopting the EPM or, in alternative, the DM for simulating the hysteretic behaviour of the frame members. In particular, when using the DM, it is simply assumed: ν=0.7, $K_u=K_o$ (see Fig. 3). Three artificial motions matching the C.E.B. response spectrum for soil profile S_2 have been considered, assuming a peak ground acceleration PGA=0.35g. The results discussed below have been obtained as an average of those corresponding to these motions.

Before considering the test structures the response of a SDOF system representing a damped braced frame with different properties is studied. The damage level of this system, evaluated in terms of μ_{PA} when adopting either EPM or DM (λ=0.2), is represented in Fig. 6 against the vibration period T_{uf} of the unbraced frame for different values of the stiffness ratio K* (in particular, the curves for K*=0 refer to the unbraced frame itself). All the curves have been obtained with reference to the optimum value $N^*_{opt.}$ of the slip-load ratio and assuming the same value for the strength ratio η of the unbraced frame ($\eta=A_y$/PGA=0.50, being A_y the yielding acceleration); the value 10 has been reasonably assumed as ultimate for μ_{PA}, considering that the damage is evaluated with reference to the critical sections. It is evident that the effectiveness of the damped bracing is greater for larger values of K* and that the use of the DM produces a greater damage, especially for stiffer unbraced frames (T_{uf} lower). However, it should be noticed that, as expected, the assumption η=constant is more favourable for less stiff structures.

Smoothed curves of the kind in Fig. 6 may be adopted for design purpose once both the strength level η and the vibration period T_{uf} of the unbraced frame are given: for a designated protection level (or, better, damage level) of the frame, the stiffness of the braces can be easily selected. As shown in previous papers (Vulcano, 1993 and 1994) using *performance indexes*, different than *damage indexes* according to Eq. 3, the selection of a value practically optimum for the slip-load ratio does not seem so restrictive, especially for relatively high values of K* (K*≥1); relatively high values of N* (about 0.8÷1.0) seem to be a good choice also for relatively low values of K* (≤0.5).

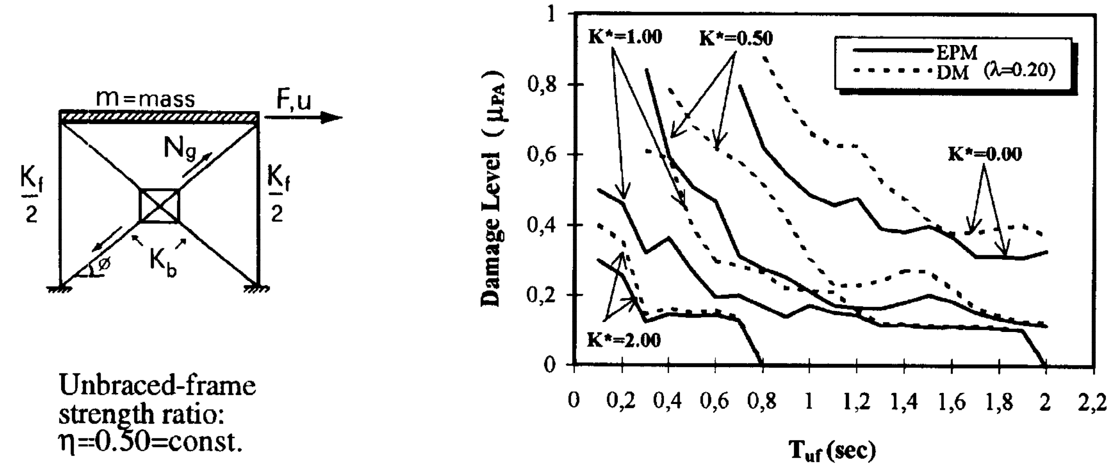
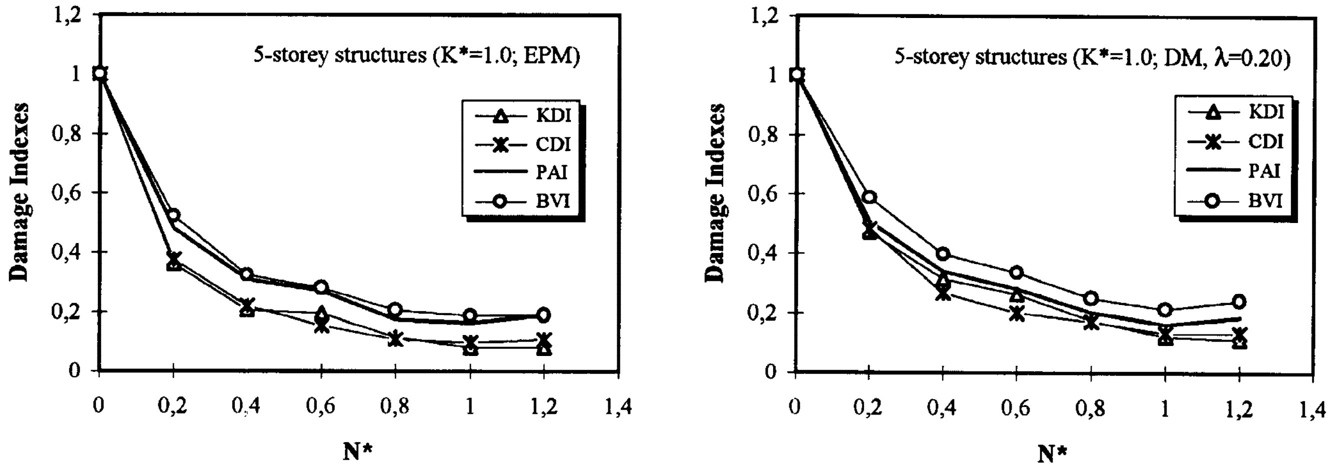
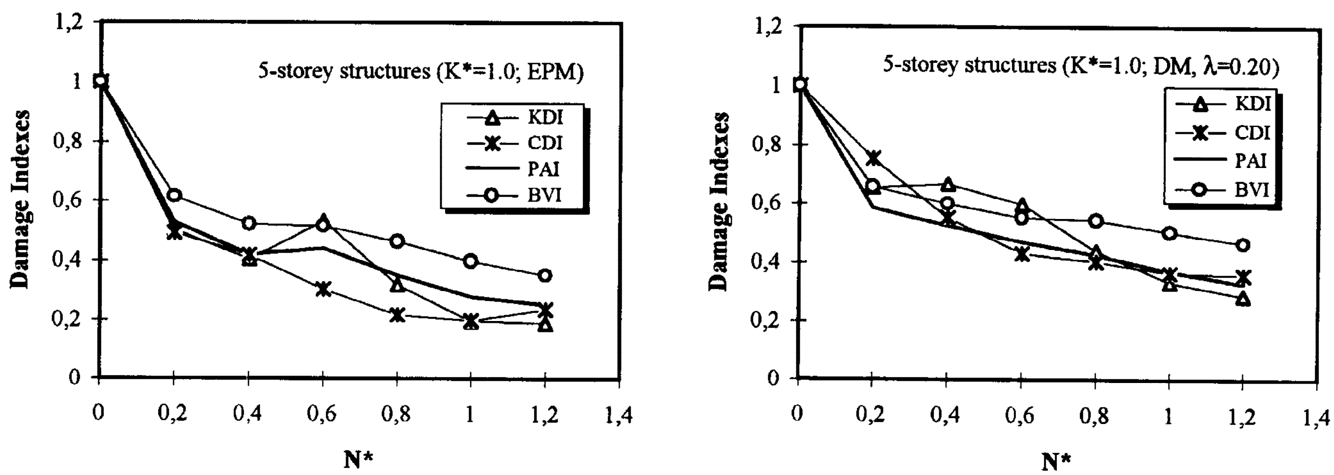


Fig. 6. Curves for SDOF system obtained adopting EPM and DM (λ=0.2) for frame vertical members.



(a, b) With reference to the arithmetic mean of the local damages.



(c, d) With reference to the weigthed mean of the local damages.

Fig. 7. Damage indexes against slip-load ratio.

To check whether this last result is valid also when the damage level is measured by different *damage indexes* (e.g., KDI, CDI, PAI, BVI according to Eq. 3), in Fig. 7 only curves obtained for five-storey test structures assuming K*=1 are reported. Exactly, the curves, which have been obtained adopting EPM and DM (λ=0.2), are separately shown when evaluating the arithmetic and weighted mean of the local damage in all the critical sections of the frame members. However, the curves for three- and ten-storey structures, which are omitted for sake of brevity, were similar. Although the curves corresponding to the weighted mean of the local damages are little more irregular (e.g., the curves corresponding to KDI), the result above mentioned about the selection of N* is substantially confirmed independently of the *damage index* which is adopted.

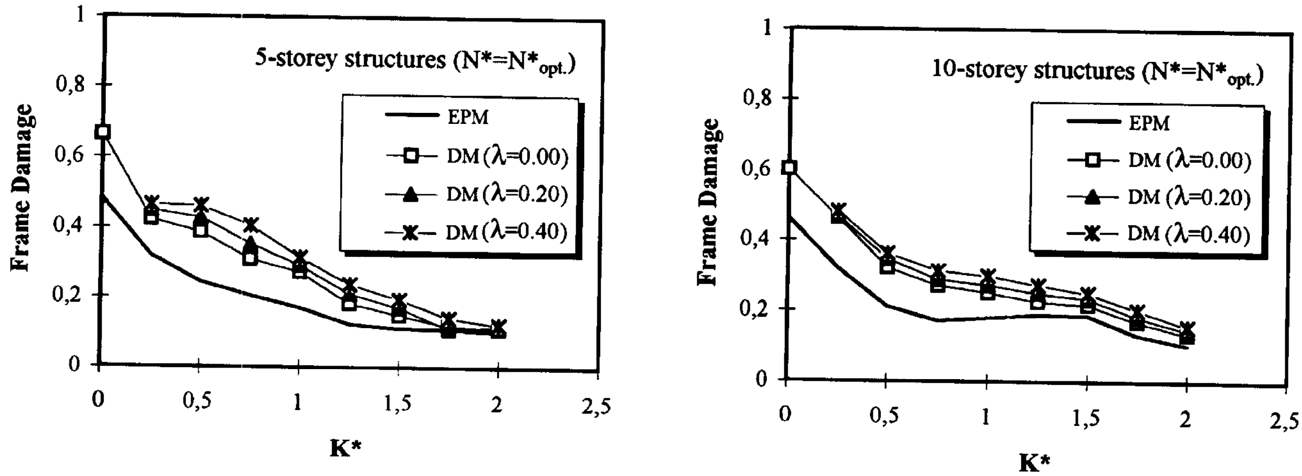


Fig. 8. Frame damage according to μ_{PA} (weigthed mean) against stiffness ratio.

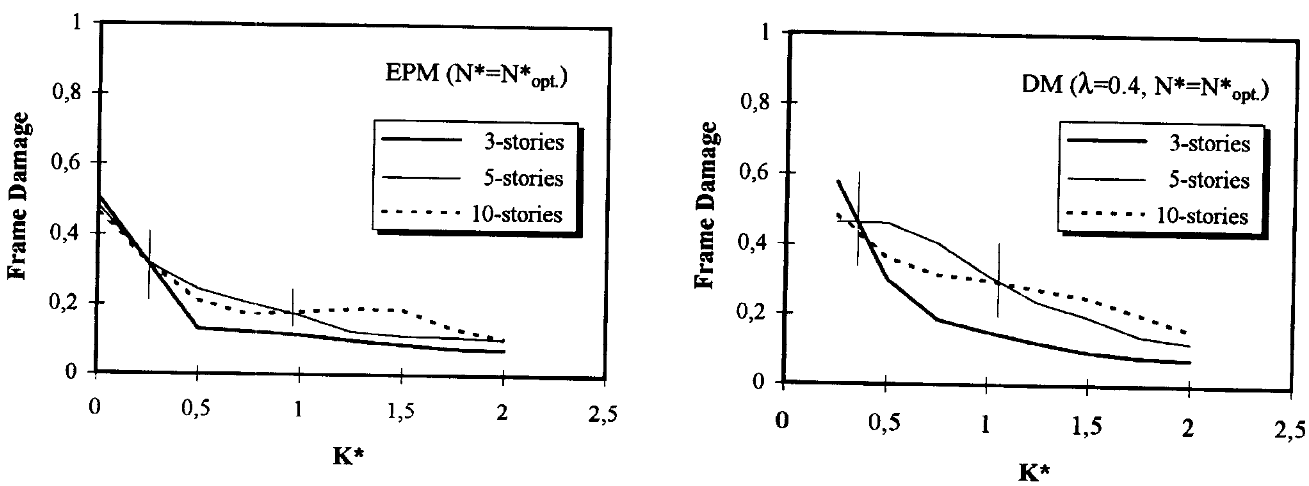


Fig. 9. Comparison of results for test structures adopting different frame-member models.

From the results shown above it comes out the importance of selecting a suitable value of the stiffness ratio K* for controlling the frame damage level. In Fig. 8 results for test structures are illustrated which have been obtained for EPM and DM, assuming for this last one different values of the parameter λ describing the strength degradation (see Fig. 3). As it can be observed, the DM produces, for a same value of K*, damage levels greater than those obtained by EPM, particularly for lower values of K*, with a little increase of damage when strength degradation is included ($\lambda>0$). As expected, for increasing values of K* the damage decreases.

A comparison between results for three-, five- and ten-storey structures is illustrated in Fig. 9 with reference to EPM and DM (λ=0.4). The results are similar for both the frame-member models, but the difference betweeen the curves for the above test structures is more marked when using the DM. It is interesting to notice that the effectiveness of the damped bracing for the structures with different number of stories depends on the range of K*. In fact, the damped braces, except for very little values of K*, are more effective for the three-storey structures; the comparison between the curves for five- and ten-storey structures shows that the damped braces are more effective in ten-storey structures for lower values of K* (less than about 1), while the contrary occurs for higher values of K*.

These last results can be substantially explained with reference to the shape of the C.E.B. spectrum adopted for generating the artificial accelerograms used for the analysis. Indeed, the spectral acceleration (Sa) corresponding to the fundamental vibration period of the elastic structure (whose corresponding vibration mode gives the prevalent contribution to the maximum response) is practically the same for unbraced and braced three-storey frames (K*≤2). On the contrary, Sa increases when using stiffer braces (i.e., higher values of K*) for five- and ten-storey structures; however, the increase of Sa is more marked for five-storey structures when using relatively low values of K* (<1).

CONCLUDING REMARKS

On the basis of the results shown in the previous Section the following conclusions can be drawn:

- the design procedure adopted for proportioning the damped braces prove to be effective and reliable for controlling the seismic damage of the framed structure;
- the choice of the optimum slip-load distribution does not seem so restrictive, being adequate to assume a relatively high value of the slip-load ratio (N*=0.8÷1.0);
- by using suitable charts (analogous to design spectra) with reference to an assigned strength ratio of the frame, an appropriate value of the stiffness ratio K* can be selected such that an accepted damage level of the framed structure is achieved;
- the use of a reliable degrading model for r.c. frame members is very important for the reliability of the design procedure, particularly when light additional design and detailing rules are adopted for the frame.

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REFERENCES

Austin, M.A. and K.S. Pister (1985). Design of seismic-resistant friction-braced frames. *Journal of Structural Engineering, ASCE*, **111**, No. 12, 2751-2769.

Banon, H., J. Biggs and H. Irvine H. (1981). Seismic damage in reinforced concrete frames. *Journal of the Structural Division, ASCE*, **107**, No. ST9, 1713-1728.

Banon, H. and D. Veneziano (1982). Seismic safety of reinforced concrete members and Structures. *Earthquake Engineering and Structural Dynamics*, **10**, 179-193.

Braga, F. and P. D'Anzi (1994). Steel braces with energy absorbing devices: a design method to retrofit reinforced concrete existing buildings. *Procs. Italian-French Symp. on Strengthening and repair of structures in seismic area*, Nice (France), Ouest Éditions, 145-154.

Ciampi, V., A. Paolone and M. De Angelis (1992). On the seismic design of dissipative bracings. *Procs. 10th World Conf. on Earth. Eng.*, Madrid (Spain), **VII**, 4133-4138.

Comité Euro-International du Beton (1987). *Seismic Design of Concrete Structures*, based on *CEB Bull. No. 160bis and 165*, Gower Technical Press Ltd, Aldershot (England).

Cosenza, E. and G. Manfredi (1992). Seismic analysis of degrading models by means of damage functions concept. In: *Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings* (P. Fajfar and H. Krawinkler, edts.), Elsevier Applied Science, 77-94.

Filiatrault, A. and S. Cherry S. (1990). Seismic design spectra for friction-damped structures. *Journal of Structural Engineering, ASCE*, **116**, No. 5, 1334-1355.

Pall, A.S. and C. Marsh (1982). Response of friction damped braced frames. *Journal of the Structural Division, ASCE*, **108**, No. ST6, 1313-1323.

Park, Y.J. and A. H.-S. Ang (1985). Mechanistic seismic damage model for reinforced concrete. *Journal of the Structural Division, ASCE*, **111**, No. 4, 722-739.

Rega, G., F. Vestroni and A.Vulcano (1990). Effectiveness of design prescriptions of CEB seismic code for satisfactory inelastic behaviour of reinforced concrete frames. *European Earthquake Engineering*, **2**, 17-26.

Vulcano, A. (1993). Design criteria of damped steel bracing systems for earthquake protection of framed structures. *Procs. Int. Post-SMiRT Conf. Seminar on Isolation, Energy Dissipation and Control of Vibrations of Structures*, Capri (Italy), 709-720.

Vulcano, A. (1994). Design of damped steel bracing systems for seismic control of framed structures. *Procs. 10th European Conf. on Earth. Eng.*, Vienna (Austria), **3**, 1567-1572.

Vulcano, A. and F. Mazza (1995). Aseismic design of damped braced frames (in italian). *Report No. 168*, Dipartimento di Strutture, Università della Calabria, Arcavacata di Rende, Italy.