A Procedure for the Design of Viscous Dampers to Be Inserted in Existing Plan-Asymmetric Buildings

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SUMMARY:
In this paper one of the procedures proposed in literature for the design of viscous dampers to be inserted in existing buildings is examined and extended to 3D eccentric buildings. This procedure is based on an energy criterion for the determination of the damping ratio and accounts for the nonlinear behaviour of the structure through pushover analysis. The proposed procedure has been verified through a case study characterized by a six storey RC building dimensioned only for gravity loads and rehabilitated with fluid-viscous dampers. Both plan-symmetric and plan-asymmetric configurations have been considered for comparisons. The second one has been analysed both with the procedure proposed in literature for plan buildings, and with the extended one. Moreover, the importance of considering the higher modes has been investigated. The effectiveness of the design procedure has been then evaluated through the comparison with nonlinear dynamic analyses.

Keywords: Plan Asymmetric Structures; Seismic Retrofit; Viscous Dampers

1. INTRODUCTION

The importance of seismic assessment and rehabilitation of existing buildings is due to the large number of inadequate existing structures in earthquake regions. The retrofit objective of satisfying the seismic requirements for new buildings is often economically prohibitive and very difficult to achieve. In these cases an innovative technique as the dissipation of energy by added damping devices may be very promising (Christopoulos and Filiatrault, 2006; Constantinou et al., 1998). In the rehabilitation interventions the use of fluid-viscous dampers offers some advantages as their behaviour is independent from the frequency and their dissipative density is very high. Most of the procedures proposed in literature for the design of viscous dampers to be inserted in existing buildings are referred to plan structures, or to spatial plan-symmetric structures. In particular one of these procedures (BSSC, 1997; Ramirez et al., 2000; BSSC, 2003) is based on an energy criterion for the evaluation of the damping ratio of multi-degree of freedom systems and accounts for the nonlinear behaviour of the structure. The purpose of this study is to extend the procedure to 3D plan-asymmetric buildings. In particular the extension regards the determination of the damping ratio, of the maximum damper forces and of the maximum interstorey drifts. The proposed extended procedure is then applied to two reference RC frame buildings, both characterized by six floors and by the same plan dimensions and lateral stiffness and designed only for gravity loads. One of these structures is symmetric, the other one is plan-asymmetric. The seismic assessment of the considered structures is performed by developing nonlinear models and applying pushover procedures. Initially the design of the viscous dampers is performed for the two structures considering only the translational components of the first mode. Then for the irregular structure the design is repeated considering both the translational and the rotational components of the first translational modes in the two principal directions. The maximum response parameters as the maximum damper forces and the interstorey drifts are calculated considering also the higher modes of vibration. The results of the design performed considering the spatial response are compared with the ones obtained neglecting the plan-asymmetry. The effectiveness of the different design procedures is evaluated through comparisons with nonlinear dynamic analyses, carried out considering a set of spectrum-compatible ground motions.

2. DETERMINATION OF SUPPLEMENTAL DAMPING FOR REDUCING THE SEISMIC DEMAND

The determination of the supplemental damping for reducing the seismic demand is performed according to a procedure proposed in literature and here described. This procedure is based on the comparison between capacity and demand spectrum in the acceleration-displacement graphical representation. The capacity spectrum is derived from a nonlinear static analysis, while the demand spectrum is obtained by reducing the elastic response spectrum corresponding to the considered limit state. In particular, the demand spectrum is determined as the damped response spectrum associated to the global effective damping ratio of the building. This damping ratio accounts for both contributions of dissipative devices and hysteretic behaviour of structural members. Intersection of capacity curve and demand spectrum gives the performance point and the actual displacement demand. This procedure is iterative since the global effective damping ratio depends on the displacement demand. The curve of base shear V_b versus roof displacement D_{roof} obtained from pushover analysis is transformed into capacity spectrum by applying the following relationships:

$$S_a = V_b / M_1 ; \qquad S_d = D_{roof} / \phi_{roof1}\Gamma_1 \tag{2.1}$$

where ϕ_{roof1}= 1 if the mode shape is normalized in order to have unit component at the roof. ϕ_{roof1} is the modal deformation in the direction considered in the pushover analysis, generally coincident with the direction of the seismic action. Γ_1 and M_1 are respectively participation factor and effective modal mass of the fundamental mode. The application of the procedure requires a bilinear idealization of the capacity spectrum so that elastic stiffness, yielding point and post-elastic stiffness of the equivalent single-degree of freedom structure are known.

The demand spectrum is determined by applying to the elastic response spectrum a damping modification factor B, which is a function of the global effective damping ratio. In this study the damping modification factor was defined using the tables provided by the Report MCEER (Ramirez et al., 2000). The linear elastic response is determined as the intersection between the elastic branch of the capacity curve and the reduced demand spectrum. The damping ratio provided by hysteretic response of structural member, ξ_H, is evaluated considering the energy dissipated by a loading cycle up to displacement demand:

$$\xi_H = 2q_H \cdot \left(S_{a,y}S_d - S_a S_{d,y}\right)/\left(\pi \cdot S_a S_d\right) \tag{2.2}$$

where $S_{a,y}$ and $S_{d,y}$ are spectral acceleration and displacement at yielding point, S_a and S_d are spectral acceleration and displacement corresponding to performance point and q_H is a factor equal to the ratio of the actual area of hysteresis loop to that of the assumed perfect bilinear oscillator. Some indications for defining the factor q_H may be found in the mentioned Report MCEER. According to these indications, a value of q_H equal to 0.5 was adopted for the examined building. To calculate the damping ratio given by linear dampers under nonlinear structural behaviour, the fundamental period T_1 has to be replaced with the effective period:

$$T_{1,eff} = T_1\sqrt{\mu} \tag{2.3}$$

where μ is the ductility demand. The effective period is calculated considering the secant stiffness of the structure at maximum displacement. In presence of nonlinear viscous dampers the damping ratio provided by the dampers for a linear structural response, ξ_{v1}, shall be multiplied by $\mu^{1-\alpha/2}$, where α is the exponent of velocity of the dampers. The global effective damping ratio, ξ_{eff}, is obtained by adding to the damping given by the dissipative devices the inherent damping ξ_i and the hysteretic damping ξ_H provided by the structure:

$$\xi_{eff} = \xi_i + \xi_{v1}\mu^{1-\alpha/2} + \xi_H \tag{2.4}$$

In the nonlinear case the iterative procedure starts assuming a certain displacement D_{roof}. The spectrum is then reduced according to ξ_{eff}, calculated as seen in Eqn. 2.4, and the intersection with the capacity curve is evaluated (Figure 2.1). This value has to correspond to the one initially supposed: this method is thus iterative, since it is based on an assumed value of displacement D_{roof}.

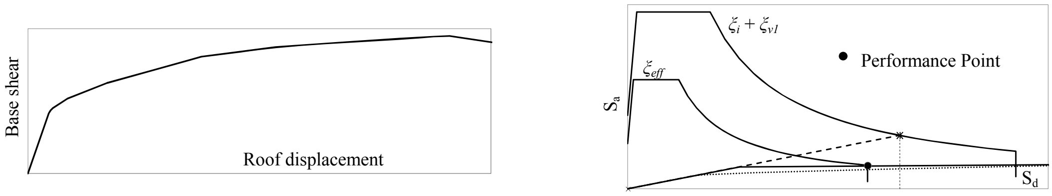

Figure 2.1. On the left: pushover curve; on the right: response evaluation

3. DESIGN PROCEDURE FOR THE VISCOUS DAMPERS

3.1. Symmetric case

The design of the damping devices is performed considering as reference documents the Italian seismic code (D.M. 14/01/2008), the Report MCEER (Ramirez et al., 2000) and the Guidelines FEMA 273 and 274 (BSSC, 1997). The behaviour of the nonlinear viscous dampers is defined by:

$$F_{Dj} = C_{Nj} \left| \dot{u}_{Dj} \right|^{\alpha_j} \mathrm{sgn}\left(\dot{u}_{Dj} \right) \tag{3.1}$$

where F_{Dj} is the damper force, C_{Nj} is the damper coefficient, α_j is the damper exponent and u_{Dj} is the relative displacement between the ends of the device. The modal damping ratio provided by the dampers in a building can be calculated as:

$$\xi_{v1} = W_D / 4\pi W_S \tag{3.2}$$

where W_D is the energy dissipated in the dampers and W_S is the elastic energy stored at the maximum displacement of the system. It is assumed that a building undergoes harmonic vibration such that:

$$\{u\} = D_{roof} \{\phi\}_n \sin(2\pi t / T_n) \tag{3.3}$$

where D_{roof} is the amplitude of roof displacement, T_n is the undamped n^{th} period of vibration and $\{\phi\}_n$ is the n^{th} undamped mode shape, normalized so that ϕ_{in}=1 for i corresponding to the roof level; then, the energy dissipated by the damping system per cycle of motion in mode n is:

$$W_D = \sum_{j=1}^{N_{Dj}} (2\pi / T_n)^{\alpha_j} C_{Nj} \lambda_j \left(D_{roof} f_j \phi_{rjn} \right)^{1+\alpha_j} \tag{3.4}$$

where λ_j is a function of the exponent α_j, N_{Dj} is the number of damping devices, f_j is the displacement modificaton factor depending on the damping system configuration and ϕ_{rjn} is the difference between the n^{th} modal ordinates associated with the degrees of freedom to which is connected the damper. The maximum strain energy W_S is calculated assuming that it is equal to the maximum kinetic energy:

$$W_S = \frac{2\pi^2}{T_n^2} \sum_{i=1}^{N} \left(\frac{w_i}{g} \right) D_{roof}^2 \phi_{in}^2 \tag{3.5}$$

where N is the number of reactive weights. So the damping ratio for mode n equal to 1 is given by:

$$\xi_{v1} = \left[\sum_{j=1}^{N_{Dj}} (2\pi)^{a_j} \cdot T_1^{2-a_j} \cdot \lambda_j C_{Nj} f_j^{1+a_j} D_{roof}^{a_j-1} \phi_{rj1}^{1+a_j} \right] / \left[8\pi^3 \sum_{i=1}^{N} \left(\frac{w_i}{g} \right) \phi_{i1}^2 \right] \tag{3.6}$$

Once the damping ratio has been defined and the roof displacement has been calculated, the determination of the damping coefficients may be obtained by inverting Eqn. 3.6 on the basis of a prefixed distribution of the damping coefficients. If a constant distribution is considered, and the same value of exponent α is adopted for all the dampers, the coefficient C_N may be calculated as follows:

$$C_N = \frac{\xi_{v1} \cdot 8\pi^3 \sum_{i=1}^{N} \left(\frac{w_i}{g} \right) \varphi_{i1}^2}{(2\pi)^{\alpha} T_1^{2-\alpha} \cdot \lambda \cdot D_{roof}^{\alpha-1} \sum_{j=1}^{N_{Dj}} f_j^{1+\alpha} \varphi_{rj}^{1+\alpha}} \tag{3.7}$$

The calculation of the damping ratio in the higher modes is complicated by the fact that Eqn. 3.6 is not applicable to higher modes in the case of nonlinear viscous dampers. To overcome this difficulty, Seleemah and Costantinou (1997) resorted a physical interpretation of the higher mode response. They viewed higher mode response as small amplitude, high frequency motion centred on the first mode response. Accordingly, one could define an effective damping constant C_{effj} for each nonlinear viscous device. This constant is taken as the slope of the force-velocity curve of the device at the calculated device velocity in the first mode, $\dot{u}_{Dj1}$:

$$C_{effj} = \alpha_j C_{Nj} \dot{u}_{Dj1}^{\alpha_j - 1} \tag{3.8}$$

Assuming now an elastic behaviour of the building, the damping ratio for the n^{th} mode can be calculated as:

$$\xi_{vn} = \left(\frac{T_n}{4\pi} \right) \cdot \left[\sum_{j=1}^{N_{Dj}} f_j^2 C_{effj} \phi_{rjn}^2 \right] / \left[\sum_{i=1}^{N} \left(\frac{w_i}{g} \right) \phi_{in}^2 \right] \tag{3.9}$$

The design of the dampers requires also the determination of the maximum forces and displacements they have to support. The maximum force in the dampers may be estimated using Eqn. 3.1. Considering the first mode the following expression is obtained:

$$F_{Dj} = C_{Nj} \left| \dot{u}_{Dj} \right|^{\alpha_j} = (2\pi / T_1)^{\alpha_j} C_{Nj} \left(f_j D_{roof} \varphi_{rj1} \right)^{\alpha_j} \tag{3.10}$$

The maximum forces and displacements can then be calculated considering also the higher modes. The constant C_{effj} and the corresponding damping ξ_{vn} are calculated using, respectively, Eqn. 3.8 and Eqn. 3.9. The roof displacement is calculated supposing for the higher modes an elastic behaviour. The contribution of a higher mode to the maximum damper force can be calculated as:

$$F_{Djn} = (2\pi / T_n) C_{effj} \left(f_j D_{roofn} \varphi_{rjn} \right) \tag{3.11}$$

These values are combined with the ones given by the first mode, using modal combination rules as SRSS. A similar procedure is followed in order to include higher mode effects in the interstorey drifts.

3.2. Asymmetric case

If a 3D plan-asymmetric structure is under study, the fundamental mode is characterized in general by translations in two orthogonal directions and by rotation at each floor. For this reason, not only dampers parallel to the direction of the seismic action are activated, but also the ones in the orthogonal direction, if any are present. Assuming N_{Dx} dampers installed parallel to the x direction and N_{Dy} dampers parallel to the y directions, Eqn. 3.6 could be rewritten by showing the contributions of all the dampers:

$$\xi_{v1} = \left[(2\pi)^{\alpha} T_1^{2-\alpha} \lambda D_{roof}^{\alpha-1} \right] \cdot \frac{\left[\sum_{j=1}^{N_{Dx}} C_{Nxj} f_{xj}^{1+\alpha} \varphi_{rxj}^{1+\alpha} + \sum_{j=1}^{N_{Dy}} C_{Nyj} f_{yj}^{1+\alpha} \varphi_{ryj}^{1+\alpha} \right]}{8\pi^3 \left[\sum_{i=1}^{N} m_i \varphi_{xi1}^2 + \sum_{i=1}^{N} m_i \varphi_{yi1}^2 + \sum_{i=1}^{N} I_i \varphi_{\theta i1}^2 \right]} \tag{3.12}$$

where I_i represents the polar moment of inertia of the floor mass m_i and the same value of exponent α is adopted for all the dampers. Once ξ_{v1} is established and D_{roof} is determined, by considering the same value of the damping coefficient for all the dampers in each direction, the coefficient of the dampers in x direction, C_{Nx}, and the coefficient of the dampers in y direction, C_{Ny}, may be calculated as follows:

$$C_{Nx} \sum_{j=1}^{N_{Dx}} f_{xj}^{1+\alpha} \varphi_{rxj}^{1+\alpha} + C_{Ny} \sum_{j=1}^{N_{Dy}} f_{yj}^{1+\alpha} \varphi_{ryj}^{1+\alpha} = \xi_{v1} \frac{8\pi^3 \left[\sum_{i=1}^{N} m_i \varphi_{xi1}^2 + \sum_{i=1}^{N} m_i \varphi_{yi1}^2 + \sum_{i=1}^{N} I_i \varphi_{\theta i1}^2 \right]}{(2\pi)^{\alpha} T_1^{2-\alpha} \lambda D_{roof}^{\alpha-1}} \tag{3.13}$$

The damping coefficients are hence obtained from a system of two equations. One of them is given by the application of Eqn. 3.13 for the first mode in x direction and considering the seismic action in x direction, the other one is given by the application of Eqn. 3.13 for the first mode in y direction and considering the seismic action in y direction (Landi and Diotallevi, 2011). The same procedure can be used in order to extend to the 3D case the Eqn. 3.9, concerning higher modes:

$$\xi_{vn} = \frac{T_n}{4\pi} \cdot \left[\sum_{j=1}^{N_{Dx}} C_{effxj} f_{xj}^2 \varphi_{rxjn}^2 + \sum_{j=1}^{N_{Dy}} C_{effyj} f_{yj}^2 \varphi_{ryjn}^2 \right] / \left[\sum_{i=1}^{N} m_i \varphi_{xin}^2 + \sum_{i=1}^{N} m_i \varphi_{yin}^2 + \sum_{i=1}^{N} I_i \varphi_{\theta in}^2 \right] \tag{3.14}$$

4. FIRST CASE STUDY: SYMMETRIC BUILDING

The symmetric building under study is a six floor building, having dimensions in plan of 15 m × 5 m, storey height of 3,2 m and square columns of 40 cm × 40 cm. The building is assumed to be located in a site characterized by a reference peak ground acceleration equal to 0.242 g, and it is supposed to be designed to resist only to vertical loads. The seismic action is applied in x direction. The dampers are installed at each floor in the three frames parallel to the x direction (Fig. 4.1).

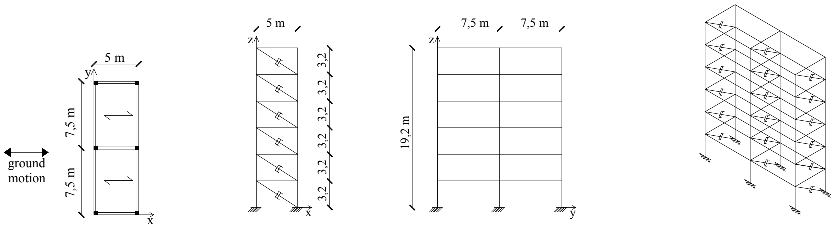

Figure 4.1. Plan, front and axonometric view of the symmetric building

First of all, the modal dynamic analysis is performed, in order to obtain the modal shapes. The modes are ordered in triplets, according to the main direction along which masses are activated in each mode. In Table 4.1 the modal participation mass ratios and the mode classification are shown. The modes characterized by a traslation of the building along the *x* direction are the second one, the fifth one and the eighth one.

Table 4.1. Modal properties for the symmetric building

	T (s)	Modal participating mass ratios			Mode classification
		x	y	θ	
1	1,597	0	0,82	0	1y
2	1,566	0,81	0	0	1x
3	1,485	0	0	0,82	1θ
4	0,509	0	0,1	0	2y
5	0,498	0,11	0	0	2x

4.1. Response evaluation and design of the damping system

The vulnerability evaluation, in particular for seismic action, performance levels and mechanical properties of materials, is performed according to the Italian seismic code (D.M. 14/01/2008), similar to Eurocode 8 (2003). Three performance levels, referred to as limit states, are considered in these codes for existing structures: Damage Limitation, Life Safety and Collapse Prevention. It is intended that each limit state is achieved by the structure when the first of its members attains the corresponding deformation capacity. The structural elements are modelled by adopting a concentrated plasticity model. The nonlinear model is implemented in a finite element computer program (SAP2000). Plastic hinges, located at the ends of each element, are characterized by the moment-rotation curve defined by assigning yielding and ultimate bending moments, elastic stiffness and ultimate hinge rotation.

The pushover analysis is performed using a lateral load distribution referred to as modal pattern. This is characterized by lateral forces proportional at each floor to the mass multiplied by the corresponding modal deformation of the dominant mode in the direction of seismic action. The assessment procedure requires the comparison, for each Limit State, between demand and capacity in terms of displacement. Under elastic conditions, the damping ratio adopted in this study is equal to 30%, which is the sum of the inherent viscous damping, called ξ_i, equal to 5%, and of the damping given by the dampers, called ξ_{v1}, equal to 25%. This is the maximum damping allowed by the Italian seismic code and suggested by the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273). The response evaluation procedure is then applied. The values obtained at convergence are S_d = 0,057 m, μ = 2,14, ξ_{eff} = 65% and roof displacement D_{roof} = 0,0836 m. Considering a constant distribution of dampers, once the velocity exponent α has been decided (in this case α = 0,5), and the roof displacement D_{roof} has been calculated, Eqn. 3.7 can be used in order to obtain the coefficient C_N of the dampers, as seen before. The coefficient of the dampers is calculated considering the fundamental mode in *x* direction, which is the second mode. The obtained value is C_{Nj} = 322 kN(s/m)^{0,5}. The maximum force in the dampers may be estimated using Eqn 3.1, while the maximum damper forces and the maximum interstorey drifts are then calculated considering also the higher mode, that is, in this case, the fifth mode (Tab. 4.1).

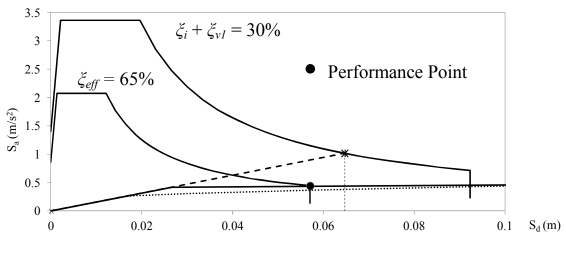

Figure 4.2. Response evaluation

4.2. Time-history analyses

The time history analyses are performed by applying a set of 5 spectrum-compatible ground motions to the structure. These are scaled so that the spectral acceleration for the fundamental period is the same as the spectral acceleration of the design spectrum, as illustrated in Fig. 4.3.

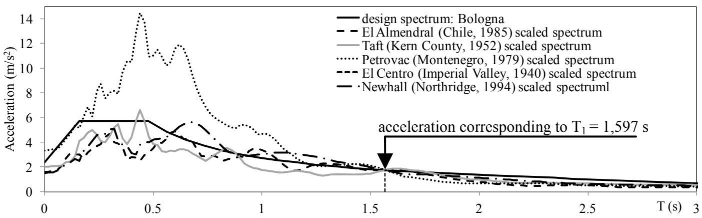

Figure 4.3. Acceleration response spectra of the scaled ground motions

The maximum damper forces and the maximum interstorey drift ratios obtained with the nonlinear dynamic analyses and with the design procedure are compared. The design procedure is applied in two ways: considering the first mode along the *x* direction or using the first and the second mode along the *x* direction. As shown in Fig. 4.4, the results concerning the maximum forces obtained with the design procedure considering two modes are more accurate, when compared to the time-history results, than the ones obtained considering only the first mode. The maximum interstorey drifts, instead, are quite accurate even with just the first mode.

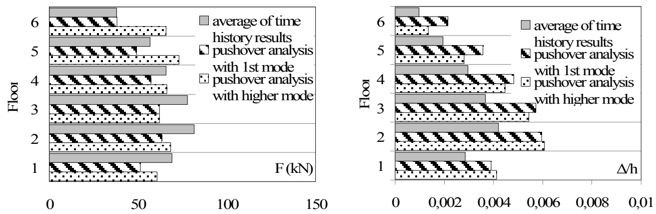

Figure 4.4. On the left: results relative to the maximum damper forces; on the right: results relative to the maximum interstory drift ratios

5. SECOND CASE STUDY: ASYMMETRIC BUILDING

The considered asymmetric building is a six floor building, characterized by two frames parallel to the *y* direction and three frames parallel to the *x* direction. This building has the same plan dimensions and the same lateral stiffness as the symmetric one, but a different position of the central frame and different column dimensions. The columns are all rectangular, sized 25 cm × 50 cm. In two frames they have the larger side parallel to the *x* direction while in the third one they have the larger side parallel to the *y* direction. The beams and columns are designed to resist gravity loads only.

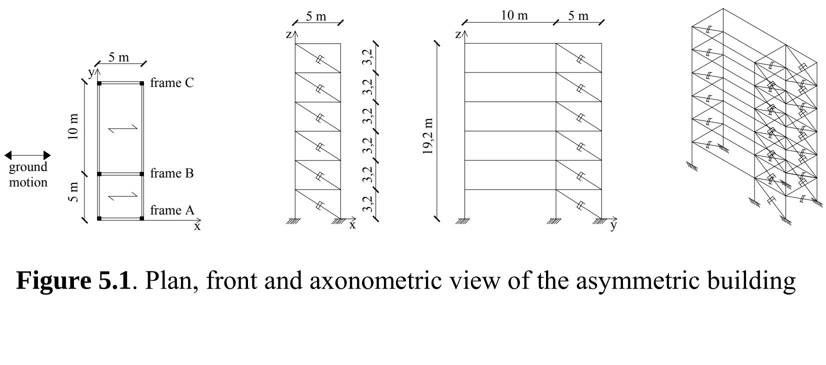

Figure 5.1. Plan, front and axonometric view of the asymmetric building

The modes are ordered in triplets, according to the main direction along which masses are activated in each mode. The modal participation mass ratios and the mode classification are shown in Table 5.1

Table 5.1. Modal properties for the asymmetric building

	T (s)	Modal participating mass ratio			Mode classification
		x	y	θ	
1	1,934	0	0,823	0	1y
2	1,9183	0,5804	0	0,25314	1x
3	1,2858	0,237	0	0,57631	1θ
4	0,6256	0,0733	0	0,02855	2x
5	0,6223	0	0,1013	0	2y

5.1. Response evaluation and design of the damping system

As in the previous case, the damping ξ_{v1} given by the dampers for elastic structural response is assumed equal to 25%. The dampers parallel to the x direction are installed as shown in Fig. 5.1. There are also two dampers in y direction at each floor. The pushover analysis along the x and y directions are performed, and the results are shown in Fig. 5.2. The values obtained at convergence of the response evaluation for the roof displacement are: in x direction D_{roof} = 0,071 m and in y direction D_{roof} = 0,093 m.

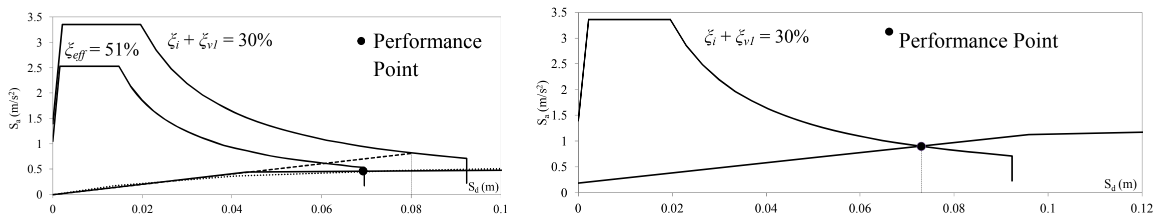

Figure 5.2. On the left: response evaluation for a seismic action in the x direction; on the right: response evaluation for a seismic action in the y direction.

In order to determine the damping coefficient, the maximum damper forces and the maximum interstorey drifts relating to the fundamental mode, the second mode properties are used. This because the second mode is the first one mainly characterized by vibration along the x direction. In order to determine the maximum damper forces and the maximum interstorey drifts relative to the higher mode, two different criteria can be used. According to the first criterion, the properties of the fourth mode are used, since it is the second mode in x direction. According to the second criterion, the properties of the third mode are used. This mode is mainly dominated by rotation, but it activates much more mass in x direction than the fourth mode. Once the damping ratio is established as ξ_{v1}=25% and once the roof displacements in the two directions are obtained, it is possible to solve the system of two equations (Eqn. 3.13) relative to the two directions. The results are: C_{Nx} = 202 kN(s/m)^{0,5} and C_{Ny} = 373 kN(s/m)^{0,5}. Then the higher mode is considered for the determination of the maximum damper forces and the maximum interstorey drifts. Following the first criterion, the fourth mode is used. In this mode the roof displacement is calculated considering an elastic behaviour. Following the second criterion, instead, the third mode is used. Since this is not an actual higher mode, but the first rotational mode, Eqn. 3.14 should not be used since the response in this mode is nonlinear. The pushover analysis relating to this mode is performed, and the roof displacement is iteratively obtained on the basis of Eqn. 3.13. Then the maximum damper forces and interstorey drifts are calculated.

A simplified application of the described procedure may be based on the assumption that the first mode is characterized only by translation in the direction of the seismic action. It is evident that this assumption is more appropriate for buildings nearly plan-symmetric. In this way all the dampers undergo the same interstorey displacement, equal to the one of the mass centre. Moreover the mode shape is characterized by components only in one direction, for example x direction, and the dampers in the orthogonal direction are not activated. The two equations relating to the two directions can be solved independently in order to obtain the damper coefficients. The results are: C_{Nx} = 248 kN(s/m)^{0,5} and C_{Ny} = 373 kN(s/m)^{0,5}. The first value is larger than the one calculated previously with the extended method.

5.2. Time-history analyses

The nonlinear time-history analyses are performed using the scaled ground motions previously described. The maximum damper forces and the maximum interstorey drift ratios obtained with the nonlinear dynamic analyses and with the design procedure are compared. The design procedure is applied in different ways: considering the first mode along the *x* direction, using the first and the second mode along the *x* direction (4th mode, I criterion) or using the first mode along the *x* direction and the first rotational mode (3rd mode, II criterion). Fig. 5.3 shows that, using the first rotational mode as higher mode, better estimates of the time-history results are obtained than using the second mode along the *x* direction. The comparison of the results of time history analyses performed with dampers designed considering (extended method) or neglecting (simplified method) the plan-asymmetry can be seen in Fig. 5.4. As a consequence of the larger values obtained for the damper coefficients, it is possible to notice that the damper forces are larger for all the three frames with the simplified than with the extended method.

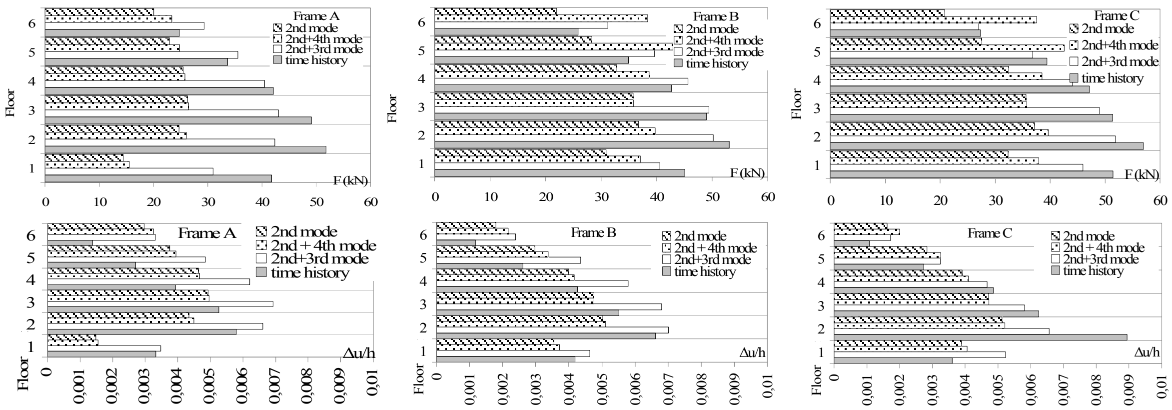

Figure 5.3. Results relative to the maximum damper forces in frames A, B and C (first line); on the right: results relative to the maximum interstory drift ratios f each frame (second line)

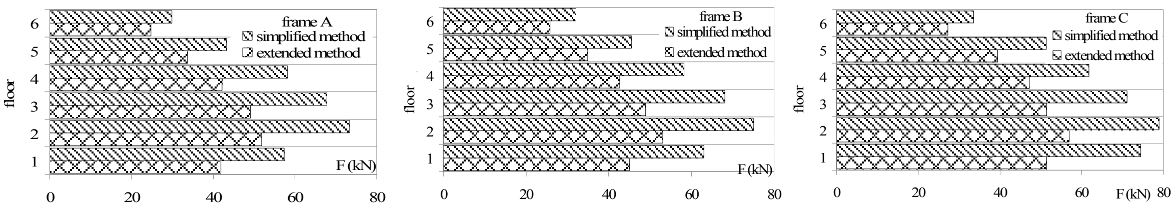

Figure 5.4. Comparison between the maximum damper forces in the three frames calculated with the simplified and with the extended method

An error rate is defined, in order to evaluate the effectiveness of each method when compared to the time-history analyses. It is defined as:

$$E = \sqrt{\sum_{i=1}^{N_l} s_i^2 / N_l} \; ; \qquad s_i = \left(r_{time-history} - r_{design} \right) / r_{time-history} \tag{5.1}$$

where N_l is the number of levels, $r_{time\text{-}history}$ is the result of time-history analysis and r_{design} is the estimate of the design procedure. The error reduces considering higher modes, as shown in Fig. 5.5. In particular, a significant reduction of the error is obtained considering as higher mode the first rotational mode, which is also characterized by a significant modal participating mass ratio in the x direction.

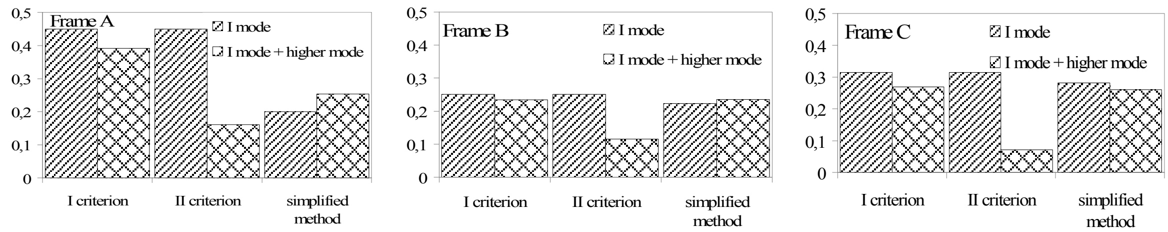

Figure 5.5. Error in the estimates of the maximum damper forces in frames A, B and C

6. CONCLUSIONS

The object of this study is to extend the procedure proposed by Ramirez et al. (2000) to 3D plan-asymmetric buildings. The proposed method has been applied with reference to two RC buildings and its effectiveness has been verified through nonlinear dynamic analyses. First of all, a symmetric building has been examined: the results underline the importance of considering the higher modes in order to estimate the maximum damper forces. Then, a plan-asymmetric building has been studied. It is characterized by the same dimensions and the same lateral stiffness as the symmetric building. Similarly to the symmetric case, the results are more accurate when higher modes are included in the design. In addition, a significant improvement of the estimates of the results of nonlinear dynamic analyses has been obtained when the rotational mode has been considered instead of the second translational mode. The simplified procedure, which neglects the plan-asymmetry, has given larger values of the maximum damper forces than the extended procedure, which accounts for the plan-asymmetry. The extended procedure seems to be more convenient since it can provide a more economic design of the damping system and it can give estimates more accurate or at least similar to the ones of the simplified procedure.

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