SEISMIC ASSESSMENT OF AN EXISTING RC HOSPITAL BUILDING: STUDY FOR THE REHABILITATION WITH SUPPLEMENTAL FLUID-VISCOUS DAMPERS

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ABSTRACT:

This paper describes a study for the seismic assessment and rehabilitation of an existing RC hospital building using fluid-viscous dampers. The structure, located in Italy, was built in the earlier 70s and it was designed without considering the seismic action. Due to a change in the seismic classification of the territory, the location is now classified as a seismic zone. The building is characterized by eight storey and by an asymmetric "T" shaped plan. The lateral resisting system is made of frames and shear walls in correspondence to stairs and elevators. This structure was analyzed by developing a non-linear model and by applying pushover procedures. The assessment was carried out in the framework of multi-level performance based approach. The adopted retrofit strategy is based on introduction of non-linear fluid-viscous dampers at each storey. Two options were examined for the rehabilitation: introduction of only supplemental dampers and combination of dampers and new RC shear walls. Design of dampers was performed considering the non-linear behaviour of the structure.

KEYWORDS: RC Structures, Pushover Analysis, Seismic Assessment, Existing Buildings, Seismic Rehabilitation, Fluid-Viscous Damper

1. INTRODUCTION

The importance of seismic assessment and rehabilitation of existing buildings is even more evident for structural engineers. This is due to the large number of inadequate existing structures in earthquake regions. In the last years a lot of research activity (Fib, 2003) has been focused on this topic and various guidelines and seismic codes (BSSC, 1997; CEN, 2003) have given great attention to such buildings. In Italy up to 70s the design was often performed neglecting seismic actions even for structures in regions characterized by medium-to-high seismicity. The need to assess and retrofit existing structures becomes particularly stringent for public and strategic buildings, which have to maintain their functionality for stronger earthquakes than ordinary buildings. In practical applications the evaluation of the effects of seismic action is usually based on linear elastic analysis and on application of a force reduction factor, called also behaviour factor, for reducing design strength. This factor depends on ductility of the structure, which for new buildings is implicitly assured by design rules. In the assessment of existing structures non-linear methods of analysis seem to be more appropriate than conventional force-based approach. Despite its approximations, pushover analysis seems to be more useful than non-linear dynamic analysis. This is due to more simplicity and independence on the input motion.

The retrofit objective of satisfying the seismic requirements of new structures is often economically prohibitive and very difficult to reach, especially for strategic buildings. In these cases an innovative technique as the dissipation of energy by added damping devices may be very promising in improving the seismic performance. The introduction of supplemental dampers allows to limit the energy to be dissipated by the structural elements and to obtain a reduction of their damage (Constantinou et al., 1998; Christopoulos and Filiatrault 2006). In the rehabilitation interventions the use of fluid-viscous dampers offers some advantages (Miyamoto et al., 2002) as their behaviour is independent from the frequency and their dissipative density is very high. Moreover the only addition of dampers does not require in general significant interventions on the elements of existing structure.

This paper illustrates a study regarding an existing RC hospital building located in Italy. The performance of the structure was evaluated by means of pushover analyses. A rehabilitation intervention based on introduction of non-linear fluid-viscous dampers at each storey was proposed. Both introduction of only dampers and combination of dampers and new RC shear walls were examined. Design of dampers was performed according to a procedure based on energy criteria and considering the non-linear behaviour of the structure (BSSC, 1997; Ramirez et al. 2000; BSSC, 2003). Finally non-linear dynamic analyses were performed to verify the procedure.

2. ASSESSMENT OF THE EXISTING BUILDING

The structure under study was built in the earlier 70s and it was designed neglecting the seismic action. Due to a change in the seismic classification of the Italian territory the location of the building is now classified as a Seismic Zone 2, characterized by a reference peak ground acceleration (PGA) equal to 0.25g. The structural configuration of the building is characterized by eight storeys and by an asymmetric "T" shaped plan. Figure 1 shows plan of building with number of columns at the end of wings while Figure 2 shows the model of the structure. The lateral resisting system is made of frames and shear walls in correspondence to stairs and elevators. The structure is not adequate to sustain seismic actions since frames are present mainly in one direction and shear walls are characterized by inadequate dimensions.

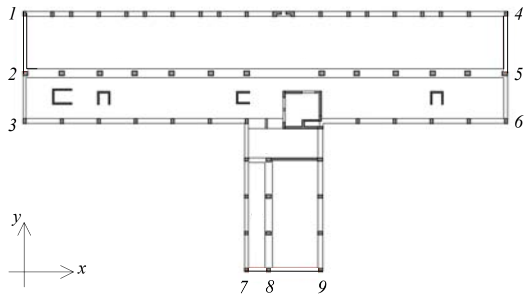

Figure 1 Plan of the existing structure

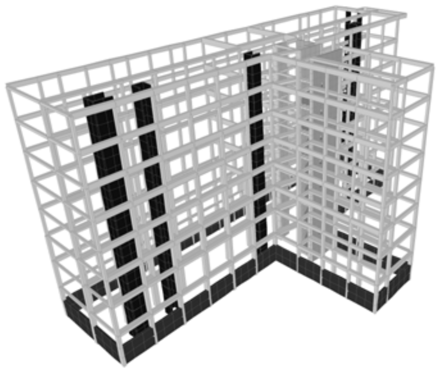

Figure 2 Model of the existing structure

The vulnerability evaluation, for what concerns in particular seismic action, performance levels and mechanical properties of materials, was performed according to Italian Seismic Code (2003), inspired to Eurocode 8 (2003). Three performance levels, referred as Limit States (LS), are considered in these codes for existing structures: Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). It is intended that each Limit Sates is achieved by the structure when the first of its members attains the corresponding deformation capacity. Thanks to the knowledge of reinforcement detailing from original drawings it was possible to develop a non-linear model of the structure and to perform pushover analyses. The structural elements were modelled by adopting a concentrated plasticity model. The non-linear model was implemented in a finite element computer program (SAP2000). Plastic hinges, located at the ends of each element, were characterized by a bilinear moment-rotation curve (Fig. 3) which was defined by assigning yielding and ultimate bending moments, elastic stiffness and ultimate hinge rotation. In general flexural rotation capacity of members may be calculated from ultimate curvature and plastic hinge length. Alternatively, some authors (Panagiotakos and Fardis, 2001) proposed expressions derived from regression of test data. Both approaches are adopted by Italian Seismic Code and EC8. In this study the ultimate rotation θ_u was calculated using the empirical expression:

$$\theta_u = \frac{1}{\gamma_{el}} 0.016 \cdot (0.3^{\nu}) \left[\frac{\max(0.01, \omega')}{\max(0.01, \omega)} f_c \right]^{0,225} \left(\frac{L_V}{h} \right)^{0,35} 25^{\left(\alpha \rho_{sx} \frac{f_{yw}}{f_c} \right)} \left(1.25^{100 \rho_d} \right) \tag{2.1}$$

where γ_{el} is equal to 1.5 for primary elements and to 1 for secondary elements, ν is the axial force ratio, ω and ω' are the mechanical reinforcement ratios of tension and compression reinforcements respectively, h is depth of section, f_c is the cylinder compressive strength of concrete (in MPa), f_{yw} is the yielding stress of transverse reinforcement, ρ_{sx} is the ratio of transverse reinforcement, ρ_d is the ratio of diagonal reinforcement, L_V is the shear span ratio and α is an efficiency factor of confinement which can be calculated as illustrated in code. The deformation capacity of members for Limit State of NC corresponds to the ultimate rotation given by Equation 2.1, while the one for Limit State of SD is set equal to $0.75\theta_u$. The deformation capacity for Limit State of DL is defined from yielding rotation θ_y, which was calculated according to the expression provided by code. As for ultimate rotation, the expression of θ_y is inspired to the one proposed by Panagiotakos and Fardis (2001):

$$\theta_y = \Phi_y \frac{L_V}{3} + 0.0013\left(1 + 1.5\frac{h}{L_V}\right) + 0.13\Phi_y \frac{d_b f_y}{\sqrt{f_c}} \tag{2.2}$$

where d_b is the medium diameter of longitudinal bars and Φ_y is the curvature at steel yielding. In Equation 2.2 the contributions of shear and bar slip are added to the flexural one. Ultimate rotation of columns of building under study was included between 0.016 and 0.03 rad, yielding rotation between 0.004 and 0.008 rad. According to code provisions, the non-linear static analysis has to be performed considering two lateral load distributions. One, referred as modal pattern, is characterized by lateral forces proportional at each floor to mass multiplied by corresponding modal deformation of dominant mode in the direction of seismic action. Although each mode is characterized by displacements in two orthogonal directions and by rotations, the load vector for conventional pushover analysis should include only lateral forces in the direction of seismic action. The other prescribed load pattern, referred as uniform, is characterized by lateral forces proportional at each floor to the mass. For each load pattern the pushover analysis was conducted in longitudinal (x) and transversal (y) directions. Figure 4 shows the pushover curves obtained considering modal load pattern. The three points corresponding to achievement of the considered LS are indicated along the pushover curves. As expected, curve in y direction is characterized by much lower stiffness and strength than curve in x direction. In terms of displacement capacity there are not significant differences between the two directions. In y direction the ultimate displacement is slightly lower than in x direction, while displacement capacity for LS of DL is slightly larger.

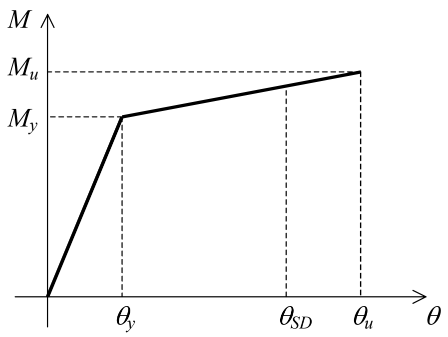

Figure 3 Moment-Rotation curve for plastic hinges

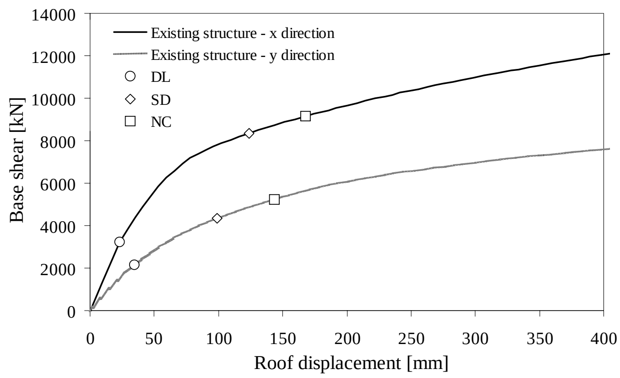

Figure 4 Pushover curves of the existing structure

The assessment procedure requires the comparison, for each Limit State, between demand and capacity in terms of displacement. Displacement demand was calculated by transforming pushover curve into the force-displacement curve of the equivalent single degree of freedom system (SDF) and by idealizing it as a bilinear curve. The demand was then obtained on the basis of design elastic response spectrum. Different levels of earthquake intensity in terms of PGA, corresponding to different values of return period or probability of occurrence, are associated to each Limit State. The reference value of PGA, depending on Seismic Zone, is the design value for LS of SD, corresponding to a return period of 475 years. This value is reduced by a factor 2.5 for LS of DL (return period of about 70 years) and it is amplified by a factor 1.5 for LS of NC (return period of about 1000 years). These PGA values has to be modified according to subsoil class, which influences also spectrum shape. Subsoil of the building under study is classified as type D, associated to an amplification of PGA equal to 1.35. PGA has to be multiplied also by another factor, depending on the importance of building. For strategic buildings, increased values of earthquake intensity, corresponding to increased values of return period, has to be adopted. For hospital buildings this amplification is equal to 1.4. As a consequence, the values of PGA adopted in this study are: 0.189g for LS of DL, 0.4725g for LS of SD, 0.7085g for LS of NC. The low values of capacity to demand ratio (Tab. 2.1), significantly lower than one, indicate that the building is seismically inadequate. This is related to the high values of PGA prescribed by new design criteria and to the low stiffness and lateral strength of the building, especially in the *y* direction.

Table 2.1 Roof displacement demand D_{roof} and capacity $D_{roof,lim}$ for considered Limit States

Limit State	PGA [g]	Pushover in *x* direction			Pushover in *y* direction		
		D_{roof} [mm]	$D_{roof,lim}$ [mm]	$D_{roof,lim}$ / D_{roof}	D_{roof} [mm]	$D_{roof,lim}$ [mm]	$D_{roof,lim}$ / D_{roof}
DL	0.189	180	23	0.13	250	35	0.14
SD	0.4725	450	124	0.27	625	99	0.16
NC	0.7085	675	168	0.25	937	144	0.15

3. EVALUATION OF DAMPING GIVEN BY FLUID-VISCOUS DEVICES

The design procedure for retrofit with dissipative devices is based on the definition of supplemental damping that has to be given to the structure for achieving the desired performance. The target damping ratio is then correlated with damping coefficients of the devices introduced at each storey (Ramirez et al., 2000). To this purpose the displacement u_{Dj} of the device at storey j is expressed as a function of roof displacement D_{roof} and of modal deformations normalized in order to have unit component at the roof:

$$u_{Dj} = f_j D_{roof} \phi_{rj} \tag{3.1}$$

where f_j is a constant that considers geometric configuration of the device and ϕ_{rj} is the inter-storey modal deformation of the considered mode, which is, in the case under study, the fundamental mode in the direction of seismic action. Mode shapes of undamped structure are considered in the calculation of damping ratio. Maximum force of devices is estimated from constitutive force-velocity law of fluid-viscous dampers:

$$F_{Dj} = C_{0j} \left|\dot{u}_{Dj}\right|^{a_j} = \left(2\pi / T_1\right)^{a_j} C_{0j} \left(f_j D_{roof} \phi_{rj}\right)^{a_j} \tag{3.2}$$

where T_1 is the undamped fundamental period of vibration, C_{0j} is the damping coefficient and α_j is the damper exponent. If α_j is equal to 1 the behaviour of damper is linear, if α_j<1 the behaviour is non-linear. In this study α_j was set equal to 0.3 for all dampers in order to improve the dissipative capacity and to reduce the strain of device. Considering an harmonic vibration, the energy W_D dissipated per cycle of motion by the devices is:

$$W_D = \lambda_j F_{Dj} u_{Dj} = \sum_{j=1}^{N_D} \left(2\pi / T_1\right)^{a_j} C_{0j} \lambda_j \left(f_j D_{roof} \phi_{rj}\right)^{1+a_j} \tag{3.3}$$

where λ_j is a coefficient dependent on α_j and N_D is the number of dampers. Considering that maximum strain energy W_S of the system is equal to maximum kinetic energy W_K, it is possible to obtain:

$$W_S = (1/2) \sum_{i=1}^{N} m_i \dot{u}_i^2 = \left(2\pi^2 / T_1^2\right) \sum_{i=1}^{N} m_i D_{roof}^2 \phi_{i1}^2 \tag{3.4}$$

where m_i is the lumped mass at degree of freedom i, ϕ_{i1} is the corresponding modal deformation and N is the number of degrees of freedom. On the basis of the energy criterion the damping ratio ξ_{v1} of the fundamental mode is calculated as follows:

$$\xi_{v1} = \frac{1}{4\pi} \frac{W_D}{W_S} = \left[\sum_{j=1}^{N_D} (2\pi)^{a_j} T_1^{2-a_j} \lambda_j C_{0j} f_j^{1+a_j} D_{roof}^{a_j - 1} \phi_{rj}^{1+a_j}\right] / \left[8\pi^3 \sum_{i=1}^{N} m_i \phi_{i1}^2\right] \tag{3.5}$$

If the damping ratio is known, it is possible to use this equation for determining the damping coefficients C_{0j}. Equation 3.5 is based on linear elastic behaviour of the structure. To calculate the damping ratio under non-linear behaviour, period T_1 has to be replaced with effective period $T_{1,eff} = T_1\mu^{1/2}$, which is dependent on ductility demand μ and is calculated considering secant stiffness of the structure at maximum displacement. The global effective damping ratio ξ_{eff} is obtained by adding to the damping given by dissipative devices the inherent damping ξ_i and the hysteretic damping ξ_H of the structure, which undergoes cyclic inelastic deformations:

$$\xi_{eff} = \xi_i + \xi_{v1}\mu^{1-\alpha/2} + \xi_H \tag{3.6}$$

where it is assumed that all devices have the same exponent $\alpha_j = \alpha$.

4. DESIGN OF REHABILITATION WITH FLUID-VISCOUS DEVICES

The adopted procedure is characterized by the comparison between capacity and demand spectrum in the acceleration-displacement graphical representation. The capacity spectrum is derived from the pushover curve, while the demand spectrum is obtained by reducing the elastic response spectrum related to the considered Limit State. In particular, demand spectrum is determined as damped response spectrum for the global effective damping ratio, accounting for the contributions of dissipative devices and hysteretic behaviour of the structure. Intersection of capacity and demand spectrum gives the performance point and the actual displacement demand. This procedure is iterative since global effective damping ratio depends on 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 following relations:

$$S_a = V_b / M_1 ; \qquad S_d = D_{roof} / \phi_{roof1}\Gamma_1 \tag{4.1}$$

where $\phi_{roof1} = 1$ if mode shape is normalized in order to have unit component at the roof. Γ_1 and M_1 are respectively participation factor and effective modal mass of the fundamental mode:

$$\Gamma_1 = \sum_{i=1}^{N} m_i\phi_{i1}s_i \Big/ \sum_{i=1}^{N} m_i\phi_{i1}^2 ; \qquad M_1 = \Gamma_1\left(\sum_{i=1}^{N} m_i\phi_{i1}s_i\right) \tag{4.2}$$

where s_i is the deformation component at degree of freedom i corresponding to a unit horizontal ground displacement. Design of damping devices was performed considering as reference documents, in addition to Italian Seismic Code, also Report MCEER 00-0010 (Ramirez et al., 2000) and Guidelines FEMA 273 and 274 (BSSC, 1997). The application of the procedure requires a bilinear idealization of capacity spectrum so that elastic stiffness, yielding point and post-elastic stiffness of equivalent SDF structure are known. The hysteretic damping ξ_H is evaluated considering energy dissipated by a loading cycle up to displacement demand:

$$\xi_H = 2q_H\left(S_{a,y}S_d - S_aS_{d,y}\right)/\pi S_aS_d \tag{4.3}$$

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. The demand spectrum is determined by applying to the elastic response spectrum a damping modification factor B, which is a function of global effective damping ratio. In this study the damping modification factor was defined using Tables provided by the mentioned Report MCEER. The design of dissipative devices is illustrated here with reference to the Limit State of Significant Damage. To start the procedure, an initial value of the damping ratio given by the dissipative system under elastic condition has to be defined. Due to low seismic capacity of the existing building, a value of ξ_{v1} equal to 0.25 was adopted for both x and y directions. A first estimate of displacement demand was obtained considering elastic behaviour of the structure. Putting μ= 1 and ξ_H= 0 in Equation 3.6, a value of the global effective damping ratio ξ_{eff} equal to 0.3 was determined. This value is associated to a damping modification factor B= 1.8. The intersection between the extension of the elastic branch of the equivalent bilinear SDF system and the damped demand spectrum gave the displacement demand for elastic behaviour of the structure. Considering this value and the corresponding spectral acceleration on the capacity curve, the ductility demand μ and the hysteretic damping ξ_H were evaluated and used in Equation 3.6 for calculating new values of the global effective damping ratio ξ_{eff} and of the damping modification factor B. From the intersection of new damped demand spectrum and capacity curve a new value of displacement demand was obtained and compared with the previous value. The procedure was repeated up to convergence, characterized by negligible difference between two subsequent values of displacement demand. The graphical representation of the procedure is illustrated in Figure 5.

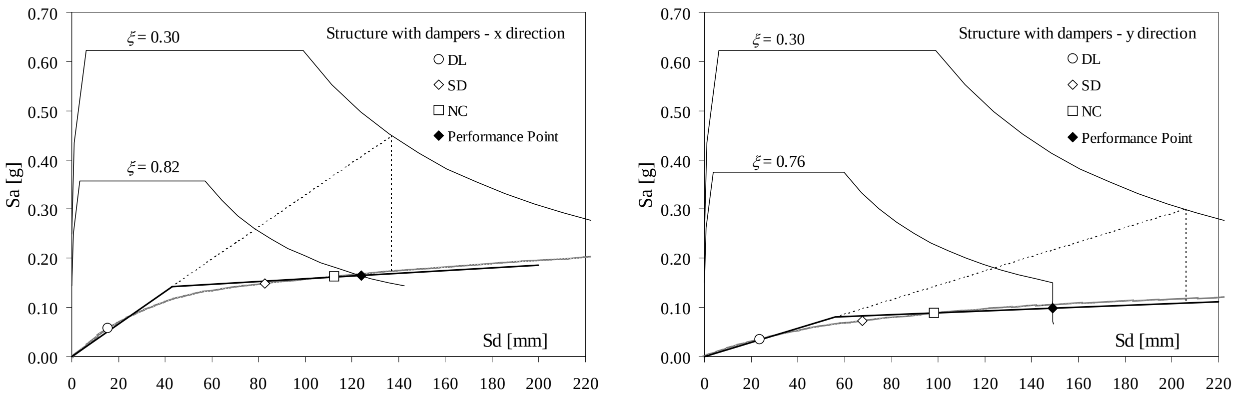

Figure 5 Determination of displacement demand for the structure with dampers

The values of damping and response parameters determined at convergence for the x and y directions are shown in Table 4.1. S_d and $S_{d,lim}$ are respectively displacement demand and capacity of the equivalent SDF system for Limit State of Significant Damage. The results indicate that added dampers produced a significant improvement of the behaviour of building. The capacity to demand ratio became about 2÷3 times larger than that of existing building. However the objective of satisfying the seismic requirements of new structures was not achieved.

Table 4.1 Values of damping and response parameters at performance point

	T_1 [s]	ξ_{v1}	μ	ξ_H	ξ_{eff}	B	S_d [mm]	$S_{d,lim}$ [mm]	$S_{d,lim}/S_d$	C_{0j} [kN(s/m)$^{0.3}$]	F_{Dj} [kN]
x direction	1.66	0.25	2.86	0.16	0.82	3.30	123	83	0.67	17781	7780
y direction	1.10	0.25	2.66	0.14	0.76	3.18	149	68	0.45	13018	4751

The damping coefficient which provides ξ_{v1}= 0.25 was calculated from Equation 3.5, assuming that it is constant for all dampers and considering the roof displacement associated to the estimated displacement demand. Design of dampers requires also the determination of maximum forces and deformations they have to sustain. Forces in dampers may be evaluated using Equation 3.2 and replacing the fundamental period with the effective period. A better estimate of maximum damper forces should includes contribution of higher modes, which may be accounted for using the same Equation 3.2 and considering the corresponding values of period, modal deformations and roof displacement. The global values for a single storey of damping coefficient C_{0j} and of maximum damper force F_{Dj} are shown in Table 4.1. These values has then to be divided by the number of dampers of each storey. In addition to design of damping system it is necessary to check structural members of rehabilitated building. Maximum actions in the building frame should be calculated at the stages of maximum drift, maximum velocity and maximum acceleration (BSSC, 1997). Particular attention should be paid to axial forces in the columns connected to dampers.

5. COMBINATION OF FLUID-VISCOUS DEVICES WITH NEW RC SHEAR WALLS

The possibility of realizing another type of intervention was examined with the purpose of giving the ability to sustain design seismic actions of new structures. In particular a combination of fluid-viscous dampers and new reinforced concrete shear wall was considered. The introduction of new RC shear walls allows to improve the low strength and stiffness of the building while the addition of fluid-viscous dampers provides a further reduction of seismic induced displacements. Three new shear walls were considered for the building under study. Two of them were placed parallel to the *y* direction at the ends of the long wings, one between columns 1 and 2, the other between columns 4 and 5. The purpose was to improve the low stiffness of building in *y* direction. One shear wall was placed parallel to the *x* direction at the end of the short wing between columns 7 and 9 in order to reduce stiffness plan-asymmetry in this direction. The new walls were included in the non-linear model and the pushover analyses were repeated for the two principal directions. Applied load vectors were based on mode shapes of building with new walls. Pushover curves of the building with and without shear walls are shown in acceleration-displacement format in Figure 6. Shear walls produced an evident increase of strength and stiffness. They became about two times larger in *x* direction and more than three times larger in *y* direction. The displacement capacity of the three Limit States did not undergo significant variations due to shear walls except a slight increase of capacity for the LS of SD and NC in the *y* direction. Due to improvements obtained with new shear walls, a value of the damping ratio ξ_{v1} lower than the previous one, and equal to 0.18 for both principal directions, was adopted. The calculation of the global effective damping ratio and the iterative procedure for the assessment of displacement demand were applied again (Fig. 6).

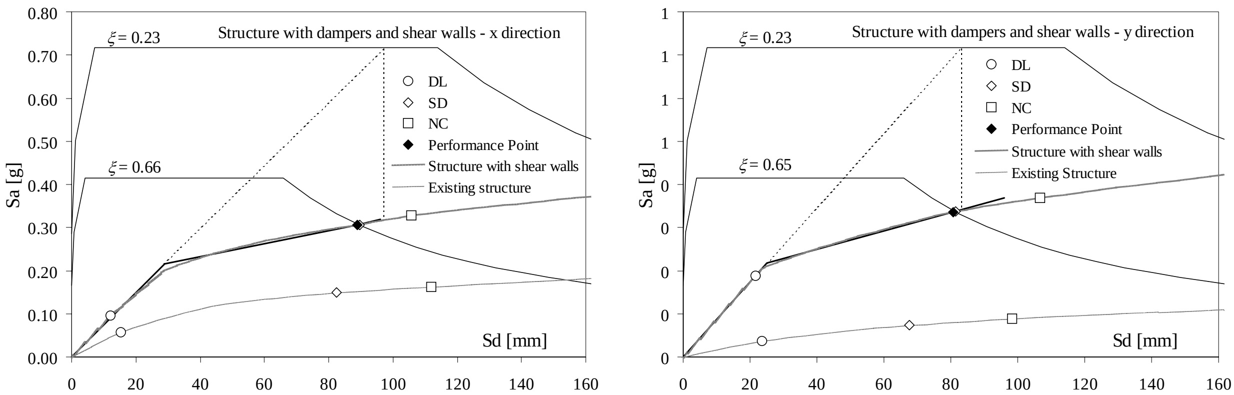

Figure 6 Determination of displacement demand for the structure with dampers and shear walls

The values of damping and response parameters determined at convergence for the *x* and *y* directions are shown in Table 5.1. The results indicate that introduction of new shear walls in addition to damping devices allowed to achieve the design objective since displacement demand was almost equal to displacement capacity in both *x* and *y* directions. Also in Table 5.1 the global values for a single storey of C_{0j} and F_{Dj} are shown. As a consequence of larger stiffness of the building, larger forces in dampers were obtained than without shear walls.

Table 5.1 Values of damping and response parameters at performance point

	T_1 [s]	ξ_{v1}	μ	ξ_H	ξ_{eff}	B	S_d [mm]	$S_{d,lim}$ [mm]	$S_{d,lim}/S_d$	C_{0j} [kN(s/m)$^{0.3}$]	F_{Dj} [kN]
x direction	0.74	0.18	3.00	0.12	0.66	2.85	86	88	1.02	21525	10143
y direction	0.68	0.18	3.20	0.11	0.65	2.84	59	61	1.03	21219	9738

Non-linear dynamic analyses were performed in order to verify the simplified procedure based on pushover analysis. The results obtained by applying a ground motion record of the Friuli earthquake in Italy in the *x* direction are described here. Maximum displacement calculated during the non-linear dynamic analysis was 55.3 mm. This value was compared with the one estimated with the procedure based on pushover analysis and on spectrum of applied earthquake record. A value equal to 44 mm was calculated for the equivalent SDF structure and a value equal to 66 mm was determined at the roof. The comparison shows that the illustrated procedure was able to give a good estimate of displacement demand despite some approximated assumptions as the consideration of only the first mode lateral load distribution for the analysis. Values of displacement were lower than those obtained in design since spectrum of considered earthquake record is characterized by lower acceleration ordinates than design spectrum.

6. CONCLUSIONS

The seismic assessment of an existing RC hospital building located in Italy and the study for the rehabilitation with fluid-viscous devices were carried out. The performance of the structure was evaluated by means of pushover analyses. A rehabilitation intervention based on introduction of non-linear fluid-viscous dampers at each storey was proposed. The results indicate that the existing structure is not adequate to sustain design seismic actions due to its low stiffness and strength, especially in one of the two principal directions. In the examined case the procedure for designing the retrofit with fluid-viscous dampers was useful and effective. The dampers introduced at each storey provided a significant improvement of the seismic behaviour of the building thanks to reduction of displacement demand. However dampers alone were not enough to resist design actions of new structures. The combination of dampers and new shear walls gave the building the ability to sustain such actions, but this option is much more onerous in terms of interventions on existing structure.

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