SEISMIC UPGRADING OF RC BUILDINGS BY ROCKING WALLS AND VISCOUS DAMPERS

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Abstract

Retrofitting of old RC framed structures is drawing the attention of the scientific community because of the inadequacy of these structures to withstand the seismic action. In fact, a large part of the existing buildings was designed for gravity loads only. In many other cases, buildings were designed with an unconservative evaluation of the seismic hazard or according to old seismic design provisions. This paper proposes a design procedure for seismic retrofitting of RC framed buildings by means of rocking walls and viscous dampers.

The proposed design procedure follows a displacement-based approach and makes use of overdamped elastic response spectra. Rocking walls are added to the existing system to ensure an almost uniform distribution of the inter-story displacement in elevation. Viscous dampers are added so that the displacement demand caused by the design peak ground acceleration be lower than the displacement capacity.

The top displacement capacity of the building is evaluated by a pushover analysis of the structure connected to the rocking wall. In particular, it is given by the top displacement at the attainment of the ultimate chord rotation in either beams or columns of the existing structure. The required equivalent viscous damping ratio is evaluated based on the response of an equivalent SDOF system so that the displacement demand be equal to the displacement capacity. The equivalent viscous damping ratio of the RC structure with rocking walls is calculated based on semi-empirical relationships available in literature. Finally, the design internal forces of the rocking walls are evaluated taking into consideration the contributions of more than one mode of vibration.

The proposed design procedure is applied to two case studies that are representative of RC buildings characterized by different levels of seismic deficiency. The considered RC frames are first retrofitted to sustain a target value of peak ground acceleration. Then, nonlinear dynamic analyses are performed to evaluate the seismic response of the upgraded frames and to compare the obtained response with that of the existing frames.

Keywords: design procedure, seismic retrofit, existing buildings, rocking walls, viscous dampers.

1. Introduction

Many buildings in the world are extremely vulnerable to earthquakes for several reasons. First of all, a large part of the building heritage has been designed when seismic regulations were not in force yet. This is a common problem in Italy, where a large part of structures was realized during the 1960s or 1970s when only gravity loads were considered for design. Second, even buildings designed to sustain seismic loads may need to be retrofitted because (1) an explicit reference to design capacity principles has been included only in recent seismic codes and (2) some seismically active areas have been recently classified with a level of seismic hazard higher than that considered at the time of design of the building. Furthermore, the natural decay of the mechanical properties of the materials reduces in time the strength and ductility of members. Consequently, several studies focus on the proposal of techniques or design procedures finalized to the seismic upgrading of existing RC buildings.

Retrofitting interventions generally aim to modify the structural strength, stiffness and/or damping by applying local modification to structural components and/or global modification to the structural system.

The local intervention on isolated components of the structural and non-structural system, e.g. epoxy repair, concrete jacketing, steel jacketing and application of fiber-reinforced polymer composites, aims to increase the deformation capacity of deficient components so that they will not reach their limit state when the building is subjected to a seismic event characterized by the expected intensity level [1]. In the case of very flexible structural systems or in the case of systems prone to form soft story mechanism, global intervention techniques are considered [2]. Out of these interventions, traditional solutions, such as the addition of shear RC walls and conventional braces, or more innovative techniques, such as the addition of metal shear panels [3] or Buckling Restrained Braces [4] embedded in in the RC framed structure, may be used.

Recently, retrofit techniques which involve structural systems that are built outside the existing structure are gaining the attention of the scientific community because they minimize the disruption to the functionality of the building. Out of these techniques, steel exoskeleton [5] and rocking walls [6] are becoming very popular. While the first technique is both an efficient structural solution and a support for energy efficient devices; the second is attractive because rocking walls do not increase the global lateral stiffness of the structure but, at the same time, they provide high lateral story stiffness thus ensuring almost uniform inter-story displacements. This feature promotes a significant exploitation of the deformation capacity of the dissipative members of the whole building and is of outmost interest for the retrofit of all the structures that are prone to concentration of damage at a few stories.

In this paper a design procedure is proposed to retrofit RC framed structures by means of RC pinned rocking walls and viscous dampers. Post-tensioned tendons are embedded in the wall to increase the crack resistance of concrete. The procedure is an extension of that previously proposed by the same authors for the retrofit of steel braced structures [7]. Rocking walls are added to make the inter-story displacement of the existing structure uniform in elevation. This allows the development of ductile collapse mechanisms because soft story collapse mechanisms are prevented and the yielding is spread in members belonging to all the stories. When additional damping is needed, viscous dampers are added between the sides of the rocking wall and adjacent reaction columns.

2. The proposed design procedure

The retrofitted system is designed to fulfill the Near Collapse (NC) limit state for seismic events characterized by a peak ground acceleration corresponding to a given probability of exceedance in 50 years ($a_{gd,NC}$). The design procedure is iterative and consists of five steps. In the first step, the top displacement capacity Δ_u of the multi-story structure coupled to a rocking wall (MRF+RW structure) is estimated and the secant stiffness K_{sec} of the same structure at the achievement of Δ_u is evaluated. Second, a SDOF system equivalent to the multi-story retrofitted structure is defined. This system is characterized by a displacement capacity equal to $\Delta_u / \Gamma^{(1)}$ and an effective period T_{sec}, where $\Gamma^{(1)}$ is the modal participation factor corresponding to the fundamental mode of vibration of the MRF+RW structure. Overdamped elastic response spectra are used to evaluate the equivalent viscous damping ratio ξ_{req} that is required so that the displacement demand corresponding to $a_{gd,NC}$ equals the displacement capacity of the SDOF system. Third,

ξ_{req} is compared to the equivalent viscous damping ratio of the existing structure ξ_{str}, which is determined based on analytical equations available in literature. If the required equivalent viscous damping ratio is larger than ξ_{str}, additional damping should be provided by properly designed viscous dampers. Fourth, internal forces in the RW are predicted and, finally, the required RW cross-section is defined. The above-mentioned steps of the design procedures are discussed in detail in the following sections.

3. Displacement capacity and secant stiffness of the retrofitted multi-story structure

In order to evaluate the displacement capacity and the secant stiffness of the retrofitted multi-story structure a pushover analysis of the MRF structure coupled to the rocking wall is carried out. Nonlinear elements with distributed plasticity and fiber discretization of the cross-sections are used to model beams and columns. The adopted distribution of forces is proportional to the first mode of vibration $\Phi^{(1)}$ of the MRF+RW structure. At each step of the pushover analysis the demand to capacity ratio is determined for each end of beams and columns in terms of chord rotation and shear forces. Specifically, for the generic element characterized by a length L and a number of integration points equal to n, the chord rotation demand θ_d at the two ends of each member is determined as:

$$\theta_{d,1} = \sum_{i=1}^{m} \chi_i W_i \left(L_{V1} - z_i \right) \qquad \theta_{d,2} = \sum_{i=m+1}^{n} \chi_i W_i \left(L - z_i - L_{V2} \right) \tag{1}$$

where L_{V1} and L_{V2} are the distances of the point of contraflexure to the ends of the element, χ_i and W_i are the curvature and the weight of the i-th integration point located at a distance z_i from the first end of the member and m is the number of integration points characterized by z_i lower than L_{V1}.

The chord rotation capacity is calculated as reported in Eurocode8 – Part 3, i.e. as

$$\theta_u = 0.016 \frac{1}{\gamma_{el}} 0.3^{\nu} \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c \right]^{0.225} \left[\min\left(9; \frac{L_V}{h}\right) \right]^{0.35} 25^{\left(\alpha \rho_s f_{yw}/f_c\right)} \tag{2}$$

where γ_{el} is equal to 1.5 (primary seismic elements), ν is the ratio of the axial force in the element at the generic step of the pushover to the axial strength provided by the concrete section, f_c and f_{yw} are the concrete compressive strength and the stirrup yield strength, h is the depth of the cross-section, ω and ω' are the mechanical reinforcement ratios of the tension and compression longitudinal reinforcements, α is the confinement effectiveness factor and ρ_s is the ratio of the transverse steel parallel to the direction of loading.

With reference to the shear demand to capacity ratio, instead, the shear resistance V_{Rd} is calculated according to the truss model with variable compressive strut inclination angle (Eurocode 2).

The displacement capacity Δ_u is achieved when the maximum value θ_d/θ_u or V_{Ed}/V_{Rd} is equal to unity (Fig. 1a). Note that, when degradation occurs or P-Δ effects are significant, the pushover curve exhibits a softening branch. In this case, Δ_u is assumed as the minimum between the value previously determined and the top displacement corresponding in the softening branch to a base shear Q_b equal to 0.8 Q_{max} (Fig. 1b). Once the displacement capacity and the corresponding base shear $Q_{b,Rd}$ have been determined, the secant stiffness is

$$K_{sec} = Q_{b,Rd} / \Delta_u \tag{3}$$

The story-shear forces $Q_{i,Rd}$ sustained by the columns of the framed structure at the achievement of the displacement capacity Δ_u are also determined. These forces will be used in the fourth step of the design procedure to evaluate the internal forces of the rocking wall.

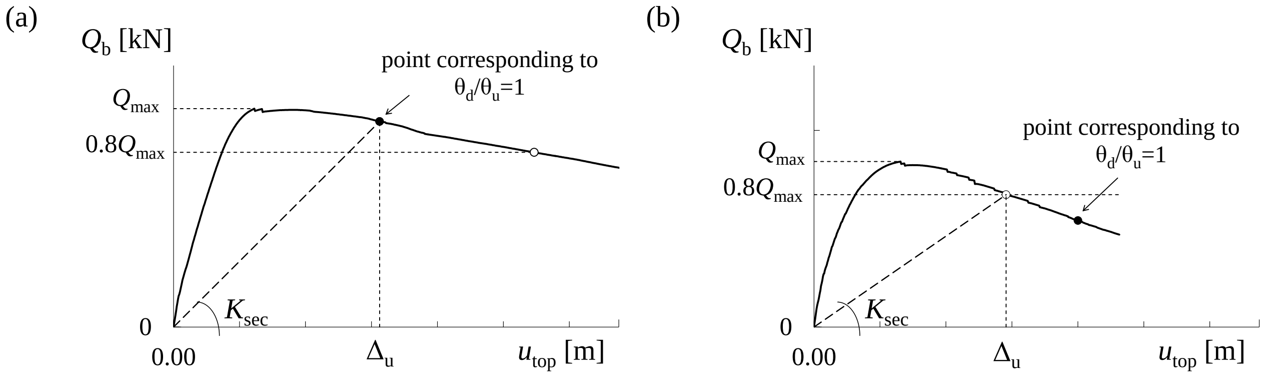

Fig. 1 – Evaluation of the displacement capacity and secant stiffness of the multi-story retrofitted structure in the case of collapse due to: (a) the achievement of θ_d/θ_u =1, (b) 20% strength loss.

4. Equivalent SDOF system and required equivalent viscous damping ratio

The SDOF system equivalent to the multi-story retrofitted structure is characterized by a displacement capacity equal to Δ_u / $\Gamma^{(1)}$, a stiffness equal to K_{sec} and a mass m^* equal to

$$m^* = \frac{\sum_{i=1}^{ns} m_i \Phi_i^{(1)}}{\Phi_{ns}^{(1)}} \tag{4}$$

where n_s is the number of stories and m_i is the mass at the i-th story of the MRF+RW structure. The effective period T_{sec} of the system is calculated as

$$T_{sec} = 2\pi\sqrt{m^*/K_{sec}} \tag{5}$$

To evaluate the equivalent viscous damping ratio ξ_{req} that is required to limit the displacement demand caused by $a_{gd,NC}$ at the displacement capacity of the SDOF system, overdamped elastic response spectra are used (Fig. 2a). A suite of 30 artificially generated accelerograms that will be adopted in numerical analyses (Section 9) is considered to build the spectra. The accelerograms are scaled so that their peak ground accelerations are equal to $a_{gd,NC}$ and the response spectra are obtained assuming equivalent viscous damping ratios in the range from 5% to 50%.

5. Equivalent viscous damping ratio provided by the structure

The equivalent viscous damping ratio of the MRF+RW system (ξ_{str}) is the sum of the inherent damping (ξ_0) and the contribution provided by the energy dissipated in beams and columns of the framed structure.

Different expressions are available in literature to estimate the structural damping. The first studies devoted to the estimate of ξ_{str} were based on the Jacobsen approach [8]. Kowalsky and Dwairi [9] concluded that the Jacobsen approach frequently overestimates the damping and that the fundamental period of the system, the characteristics of the ground motion and the ductility level are critical variables for the equivalent damping concept. Based on this result, Blandon and Priestley [10] proposed the following expression to evaluate ξ_{str} as a function of the secant period of vibration of the system, the ductility demand μ, and the cyclic response of the structure.

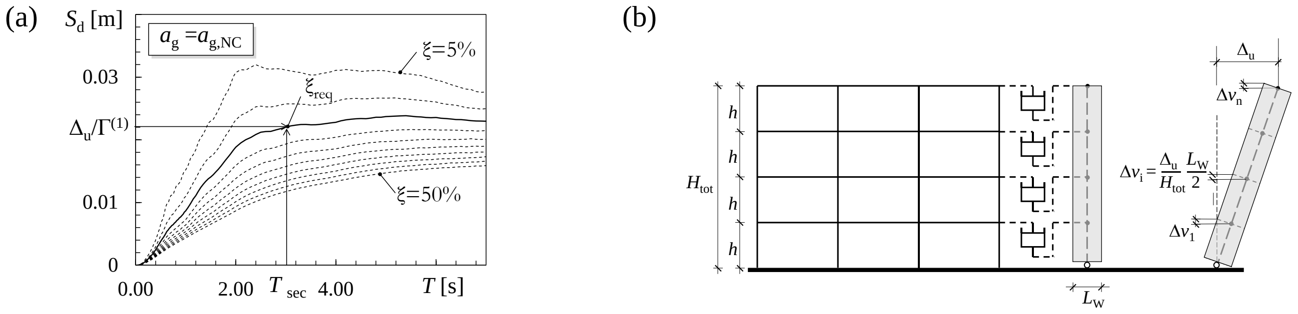

Fig. 2 – (a) Evaluation of the required equivalent viscous damping ratio; (b) vertical displacements for the evaluation of the energy dissipated by dampers

$$\xi_{str} = \frac{a}{\pi} \cdot \left(1 - \frac{1}{\mu^b}\right) \cdot \left[1 + \frac{1}{(T_{sec} + c)^d}\right] \cdot \frac{1}{N} \tag{6}$$

where a, b, c and d are coefficients defined in reference [10] for different hysteretic models and N is a normalizing factor equal to

$$N = 1 + \frac{1}{(0.50 + c)^d} \tag{7}$$

Here, the above-mentioned coefficients are set equal to a = 130, b = 0.5, c = 0.85 and d = 4 as suggested in reference [10] in the case of "Takeda fat" hysteretic model.

To evaluate the ductility demand μ, the capacity curve of the MRF+RW structure is idealized within the relevant range of displacements (i.e. for top displacement not higher than Δ_u) by a bi-linear elastic-perfectly plastic relationship. The yield displacement and the corresponding base shear are determined forcing the areas under the actual and the idealized curves to be equal and the intersection between the bi-linear curve and the capacity curve to be at a base shear equal to $0.6Q_{max}$.

6. Equivalent viscous damping ratio provided by viscous dampers

If ξ_{str} is smaller than ξ_{req}, an additional source of equivalent viscous damping ratio ξ_d has to be provided by viscous dampers. ξ_d can be expressed as $\xi_d = W_D/(2\pi\, W_S)$, where W_D is the energy dissipated by the dampers and W_S is the external work produced by lateral loads. The energy dissipated by the dampers can be easily determined assuming that (i) dampers dissipate energy because of the relative vertical displacement Δv between the rocking wall and the adjacent columns and (ii) the rocking wall is able to provide a uniform distribution of inter-story displacement in elevation (Fig. 2b). A similar assumption is made to evaluate the external work W_S

$$W_D = n_d n_s\, c \frac{2\pi^2}{T_{sec}} \left(\frac{\Delta_u}{H_{tot}} \frac{L_W}{2}\right)^2 \qquad W_S = \left(\frac{2\pi}{T_{sec}}\right)^2 \left(\frac{\Delta_u}{H_{tot}}\right)^2 \sum_{i=1}^{ns} m_i\, H_i^2 \tag{8}$$

where n_d is the number of dampers at each story, c is the viscous damping coefficient, H_{tot} is the height of the building, L_W is the length of the cross-section of the rocking wall. By combining the definition of ξ_d and Eq. (8), the viscous damping coefficient c is determined as follows

$$c = \frac{16\pi}{T_{sec}} \frac{\sum_{i=1}^{ns} m_i\, H_i^2}{n_s\, n_d\, L_W^2} \xi_d \tag{9}$$

7. Internal forces in the RW and design of the RW cross-section

Internal forces in rocking walls are determined as the sum of three contributions, which take into account the effect of the first, the second and the upper modes of vibration, respectively.

The first contribution is produced by a lateral distribution of forces F_1 that is proportional to the first mode of vibration $\Phi^{(1)}$ of the retrofitted structure. The scaling factor of the forces is adjusted so that the overturning moment at the base produced by external loads be balanced by the base resisting moment $M_R^{(k)}$ produced by four prefixed distributions (k =1,..,4) of the story-shear $Q_{i,col}$ sustained by the columns of the RC framed structure. The force $F_{i,1}^{(k)}$ at the i-th story is evaluated as

$$F_{i,1}^{(k)} = \frac{\Phi_i^{(1)}}{\sum_{i=1}^{ns} \Phi_i^{(1)} H_i} M_R^{(k)} \qquad k = 1,..,4 \tag{10}$$

where H_i is the height of the i-th floor with respect to the base. In any case, the story-shear $Q_{i,col}$ sustained by columns of the single story is equal to the story-shear resistance $Q_{i,Rd}$ determined by the pushover analysis at the achievement of the top displacement Δ_u or is equal to a null value. In particular, in the first distribution, at each story $Q_{i,col}$ is fixed equal to the story-shear resistance $Q_{i,Rd}$. In the second distribution, the story shear forces $Q_{i,col}$ sustained by the columns of the RC frame are equal to $Q_{i,Rd}$ at the story under examination and at all the levels above; in the third distribution, the story shear forces sustained by the columns are equal to $Q_{i,Rd}$ at all the levels above the story under examination; in the fourth distribution, the story shear forces sustained by the columns are equal to $Q_{i,Rd}$ at all the levels below the story under examination (Fig. 3a).

The base resisting moment corresponding to the k-th distribution of story shear $M_R^{(k)}$ is

$$M_R^{(k)} = \sum_{i=1}^{ns} \left(Q_{i,col}^{(k)} - Q_{i+1,col}^{(k)} \right) H_i \tag{11}$$

The shear force and the bending moment sustained at each story by the rocking wall are

$$V_{i,RW,1}^{(k)} = \sum_{j=i}^{ns} F_j^{(k)} - Q_{i,col} \qquad M_{i,RW,1}^{(k)} = \sum_{j=i+1}^{ns} \left(V_{j,RW,1}^{(k)} - V_{j+1,RW,1}^{(k)} \right) \left(H_j - H_i \right) \tag{12}$$

The maximum internal forces in rocking walls obtained by the previously considered distributions of lateral forces and story shears sustained by the columns provide the effects of the first mode of vibration.

A second set of lateral forces is considered to simulate the effects of the second mode of vibration. These forces are the sum of two subsets of forces, $F_{i,2a}$ and $F_{i,2b}$. The distribution of the first subset of forces $F_{i,2a}$ is proportional to the second mode of vibration whereas that of the second subset $F_{i,2b}$ is proportional to the first

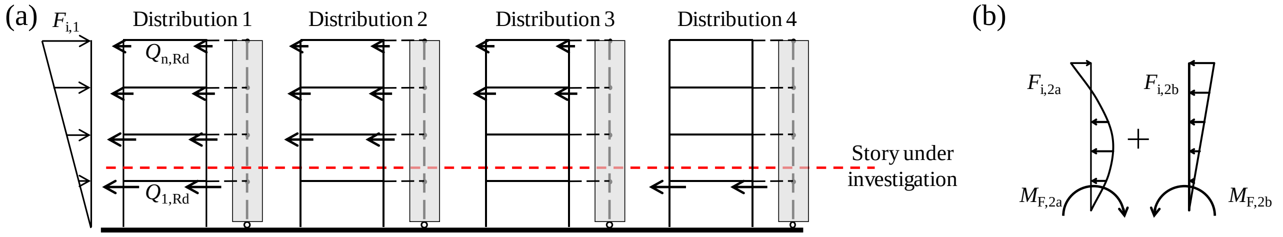

Fig. 3 – Estimation of internal forces in RW: (a) distributions of story shear sustained by columns (effect of the 1st mode of vibration); (b) external lateral forces to take account of the 2nd mode of vibration.

mode of vibration (Fig. 3b). Further, forces $F_{i,2a}$ + $F_{i,2b}$ are such that their overturning moment at the base of the system is null. The seismic forces $F_{i,2a}$ proportional to the second mode of vibration are calculated as

$$F_{i,2a} = \Gamma^{(2)} S_{a,el}\left(T_2, \xi_d + \xi_0, a_{gd,NC}\right) \Phi_i^{(2)} m_i \tag{13}$$

where T_2 is the period of the second mode of vibration, $\Phi^{(2)}$ is the shape of the second mode, $\Gamma^{(2)}$ is the modal participation factor of the second mode and $S_{a,el}$ is the pseudo-acceleration calculated at T_2 for an equivalent viscous damping ratio $\xi_d+\xi_0$ and a peak ground acceleration $a_{gd,NC}$. Lateral forces $F_{i,2b}$ are scaled so as to balance the overturning moment $M_{F,2a}$ given by $F_{i,2a}$. Forces $F_{i,2a}$ + $F_{i,2b}$ act on a structure where the internal forces of the dissipative members are null, because already considered to balance the overturning moment produced by F_1. Hence, the story shear given by $F_{i,2a}$ + $F_{i,2b}$ is entirely sustained by the rocking wall, i.e.

$$V_{i,RW,2} = \left|\sum_{j=i}^{ns} F_{j,2a} + \sum_{j=i}^{ns} F_{j,2b}\right| \qquad M_{i,RW,2} = \sum_{j=i+1}^{ns} \left(F_{j,2a} + F_{j,2b}\right)\left(H_j - H_i\right) \tag{14}$$

For high-rise structures, the contribution of higher modes of vibration is considered. The seismic forces proportional to the *m*-th mode of vibration, the shear forces and the bending moments on the RW are

$$F_{i,m} = \Gamma^{(m)} S_{a,el}\left(T_m, \xi_d + \xi_0, a_{gd,NC}\right) \Phi_i^{(m)} m_i \qquad V_{i,RW,m} = \sum_{j=i}^{ns} F_{j,m} \qquad M_{i,RW,m} = \sum_{j=i+1}^{ns} F_{j,m}\left(H_j - H_i\right) \tag{15}$$

Predicted values of shear forces and bending moments in rocking wall are finally determined adding the above-mentioned contributions as follows

$$V_{i,RW} = \left|V_{i,RW,1}\right| + \left|V_{i,RW,2}\right| + 0.3\sqrt{\sum_{m=3}^{ns} V_{i,RW,m}^2} \qquad M_{i,RW} = \left|M_{i,RW,1}\right| + \left|M_{i,RW,2}\right| + 0.3\sqrt{\sum_{m=3}^{ns} M_{i,RW,m}^2} \tag{16}$$

The dimensions of the rocking wall cross-section and the magnitude of the longitudinal prestressing force are assigned to satisfy four requirements: 1) the prestressing force required to avoid cracking of concrete should be limited so that the concrete compressive stress resulting from forces acting at the time of tensioning be lower than 60% of the characteristic compressive strength of concrete; 2) the principal stress due to the combined effect of axial and shear forces in rocking wall should be lower than the characteristic axial tensile strength of concrete; 3) the top horizontal displacement due to the shear and flexural deformability of the rocking wall should be smaller than 20% of the top displacement capacity; 4) the length of the rocking wall should be able to limit the damping coefficient of the dampers to 6 kNs/mm. Further details can be found in reference [7].

8. Case-studies

Two 6-story RC framed buildings were designed as case studies: the first one (Fig. 4a) represents Italian buildings designed to sustain gravity loads only (GL building), while the second (Fig. 4b) exemplifies the existing buildings designed according to old Italian seismic standards for low seismicity areas (SR building). The cross-sections of beams and columns of the GL building were designed according to the Italian regulations in force during the 1970s [11-13] while building SR was designed according to the seismic code in force in Italy during the 1990s [14]. The characteristic compressive cubic strength R_{ck} of concrete is assumed equal to 25 MPa (f_{ck} equal to 20 MPa), while steel grade Feb38K (characteristic yield strength f_{yk} = 375 MPa) and steel grade Feb44k (f_{yk} = 430 MPa) are used for rebars of building GL and SR, respectively. Details about the design of the two structures and the adopted cross-section can be found in reference [4]. A full knowledge of the buildings is achieved according to the requirements of Eurocode 8 – part 3, based on construction drawings, comprehensive in-situ inspection of structural details and in-situ tests on materials.

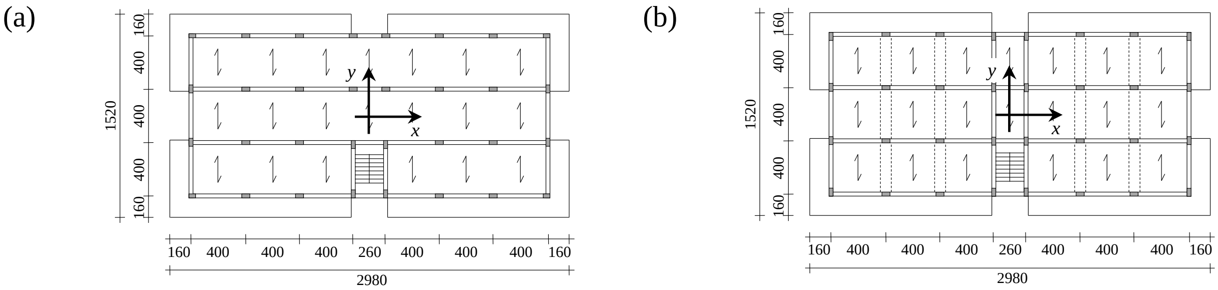

Fig. 4 – Plan view of (a) GL building; (b) SR building.

9. Seismic assessment of the case studies

The perimetric frame arranged along the y-direction was extracted from each of the considered structures and the seismic response of the frame has been determined by incremental nonlinear dynamic analysis. The single numerical analysis is carried out by the OpenSees computer program.

The seismic input consists of 30 artificially generated accelerograms compatible with the Eurocode 8 elastic spectrum for soil type C characterized by 5% equivalent viscous damping ratio. The SIMQKE computer program is used to generate the ground motions. Each ground motion is defined by a stationary random process modulated by means of a compound intensity function. The earthquake rise time is 4 s, the duration of the stationary part is 7 s and the total duration 30.5 s.

The response is expressed in terms of the chord rotation demand-to-capacity ratio θ_d/θ_u for beams and columns. The maximum values of the response parameter are determined for each time-history and then averaged over the 30 accelerograms $(\theta_d/\theta_u)_m$. The peak ground acceleration of the ground motions is scaled in steps of 0.05g up to the first achievement of an average chord rotation demand-to-capacity ratio equal or larger than 1.0.

9.1 Numerical model

Each beam and column of the existing structure is modelled by two force-based with distributed plasticity elements to have different reinforcements at the two ends of the member. The Gauss-Lobatto integration method is used and three integration points are assigned to each element. Since Gauss-Lobatto integration method places an integration point at each end of the element, five integration points are displaced along each beam and column. The cross-section of each member is discretized into fibers. The uniaxial material model used for concrete is present in OpenSees as “Concrete04” while the response of the longitudinal steel bars is simulated by means of the uniaxial material “Steel02”. As required in Eurocode 8 – part 3, mean values of material properties are used in the structural model. In particular, the yield strength of steel is f_{ym}=400 MPa and f_{ym}=450 MPa for steel grade Feb38K and Feb44K, respectively. The compressive strength of concrete fibers is f_{cm}=20 MPa and thus smaller than that considered in design in order to simulate the effect of degradation of materials in time. The same strength is assigned to fibers belonging to the core and to the cover of the cross-section even in the case of the SR structure. Indeed, the seismic code in force in 1990s did not include specific provisions to promote a ductile response of the members and thus did not require restrictive details for stirrups. Note that the confinement effectiveness factor α due to stirrups is assumed null not only when evaluating the strength of concrete, but also in the estimation of the chord rotation capacity of members.

The rigid diaphragm effect of the concrete deck is simulated by constraining all the nodes belonging to the same floor to have the same horizontal displacement. To prevent the development of fictitious axial force in beams due to combined presence of the rigid diaphragm and the fiber modelling, a “ZeroLength Element” characterized by a small axial stiffness and a large shear and flexural stiffness is added at one end of each beam [15]. Masses are lumped at the floor levels. In the case of the GL building, a mass equal to 30% of the mass of the deck of the building is assigned to the considered frame. In the case of the SR frame, instead, the percentage of the floor mass assigned to the frame is 25%. Furthermore, the actual seismic weight of the frame SR has been increased by 25% with respect to the original value, to simulate the occurrence of modifications in the non-structural elements and/or type of occupancy of the building after its construction. Viscous damping forces are obtained through the formulation proposed by Rayleigh, assuming that the stiffness proportional damping coefficient is applied to the tangent stiffness matrix of the elements. An equivalent viscous damping ratio equal to 0.05 is fixed for the first and second modes of vibration. *P*-Δ effects are considered by adding a leaning column.

9.2 Response of the considered structures

The chord rotation demand-to-capacity ratios $(\theta_d/\theta_u)_m$ of beams and columns of GL and SR structures are plotted in Fig. 5a and 5b, respectively. In the case of the structure designed to sustain gravity loads only, a soft-story mechanism is recorded. Indeed, the chord rotation demand equals the capacity at the NC limit state at the ends of the columns of the 4th story for a peak ground acceleration lower than 0.15g, while beams suffer negligible damage. Instead, in the case of the structure designed to sustain also seismic actions, the NC limit state is achieved for a_g greater than 0.50g and lower than 0.55g. On the average, the damage is spread at all the stories; however, even if it is not shown in figure, for a given accelerogram, the damage is not uniform along the height of the building.

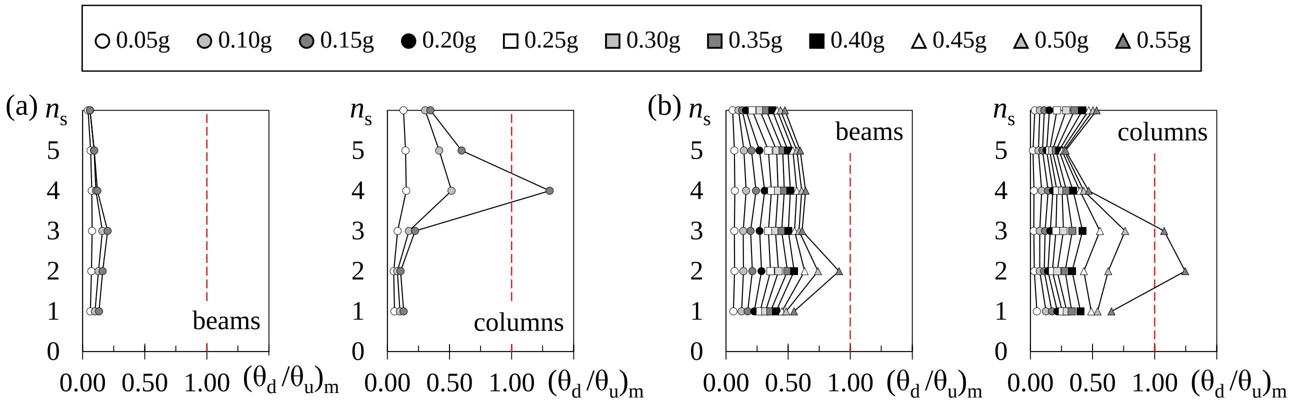

Fig. 5 – Chord rotation demand-to-capacity ratios of beams and columns of the structure: (a) GL; (b) SR

10. Design of the seismic upgrading and assessment of the retrofitted structures

The GL structure is retrofitted to achieve the NC limit state for a target peak ground acceleration $a_{gd,NC}$ = 0.45g, while the SR structure is retrofitted to sustain $a_{gd,NC}$ = 0.60g. Two rocking walls are arranged along the y-axis to upgrade the structures. In particular, rocking walls are located close to the central span of the perimetric frames. Table 1 summarizes some design parameters of the retrofitted framed structure, i.e. the top displacement capacity, the secant period of vibration of the structure, the ductility obtained by the bi-linearization of the pushover curve, the equivalent structural damping, the equivalent viscous damping ratio required to keep the top displacement demand below the displacement capacity and the viscous damping coefficient. It is worth noting that the top displacement capacity Δ_u is achieved because of a 20% strength loss (Fig 1b) in the case of the GL frame, and because of a chord rotation demand equal to the chord rotation capacity at the base of the columns of the first story (Fig 1a) for the SR frame. Based on the predicted internal forces, the rocking wall cross-section is 30x300 for both the considered structures. To obtain this cross-section, the characteristic compressive strength f_{ck} of concrete is assumed equal to 40 MPa, while the prestressing losses are quantified as 20% of the prestressing forces.

The seismic response of the retrofitted structures has been determined by nonlinear dynamic analysis. To this end, the 30 artificial accelerograms previously described are scaled to $a_{gd,NC}$.

Table 1 – Design parameters of the retrofitted

Structure	$a_{gd,NC}$ (g)	Δ_u (m)	T_{sec} (s)	μ	ξ_{str} (%)	ξ_{req} (%)	c (kNs/mm)
GL	0.45	0.302	4.73 s	4.918	17.48 %	33.27 %	1.588
SR	0.60	0.324	2.90 s	3.786	15.54 %	36.66 %	3.774

The numerical model used for the RC structure is described in Section 9.1. The rocking wall is pinned at the base and is modelled at each story by a vertical (elastic) element and two rigid beams having length equal to $L_W/2$. A horizontal zero-length element is considered at each story to replicate the shear deformability of the wall. Further details on the modelling of this member can be found in [7]. All the elements used to simulate the rocking wall are elastic to force no limit on the internal forces of this member and thus to allow the comparison between the internal forces obtained by numerical analyses and those predicted in the design phase. Viscous dampers are modelled as zero-length elements and connect the reaction columns to the closest end of the rigid beams. The viscous uniaxial material implemented in OpenSees is used and linear damping is considered.

The seismic responses of the GL structure retrofitted to sustain $a_{gd,NC}$= 0.45g and that of the SR structure retrofitted to sustain $a_{gd,NC}$= 0.60g are shown in Figs. 6 and 7, respectively. White dots pinpoint the maximum values of the response parameters obtained in each time-history analysis. The values of the response parameters are also averaged over the number of accelerograms (solid black line) and compared to the predicted values (red dashed line). For both the considered structures the distribution of floor displacements along the height of the building is almost linear (Fig. 6a and 7a), thus the stiffness of the rocking wall is adequate. Further, the top displacement is lower than the displacement capacity (i.e. lower than the predicted values). However, while in the case of the GL structure the obtained and predicted values are very close, in the case of the SR structure the differences between the obtained and predicted top displacement are not negligible. This means that the structural damping in the case of the SR structure is underestimated. The analysis of the distribution of story drift angles along the height of the building (Fig. 6b and 7b) confirms the conclusions derived based on the floor displacements. Finally, Fig. 6c-d and 7c-d show the effectiveness of the proposed procedure in the prediction of the internal forces of the rocking wall. Both shear forces and bending moments are well predicted in the case of GL structure and overestimated in case of the SR structure, especially at the first and second floor. This result is mainly caused by the internal forces determined by the third distribution of story-shear sustained by the columns of the RC framed structure described in Section 7.

To investigate further on the effectiveness of rocking walls in ensuring a prefixed collapse mechanism, the response of the existing SR structure at a_g=0.50g (i.e. the upper value of peak ground acceleration leading to the fulfillment of requirements given in Eurocode 8 at the NC limit state) and the response of the retrofitted structure at a_g=0.60g are compared in Fig. 8. In the left side of the figure, for each accelerogram, a grey bar represents the variability of the ratio θ_d/θ_u along the height of the structure, while the black solid line gives the average value of the same parameter over the number of stories. In the right side of the same figure, the values θ_d/θ_u are averaged at each story over the number of accelerograms to obtain $(\theta_d/\theta_u)_m$. Referring to the existing SR structure, the figure shows that, despite the uniformity of $(\theta_d/\theta_u)_m$ along the height, there is a large scattering in the distribution of θ_d/θ_u for a fixed accelerogram. This means that the story with the maximum damage strongly depends on the selected accelerogram and that it is difficult to predict the seismic response of the structure. Instead, when analyzing the response of the retrofitted structure, the response is almost independent of the considered accelerogram. Indeed, the damage in beams is spread at all stories as shown by both the uniformity of $(\theta_d/\theta_u)_m$ along the height and the low height of the grey bars for each accelerogram. The damage in columns is limited to the base of the first story and to the top of the upper story, as confirmed by both the distribution of the values of the ratio $(\theta_d/\theta_u)_m$ along the height of the structure and the height of the grey bars for each accelerogram.

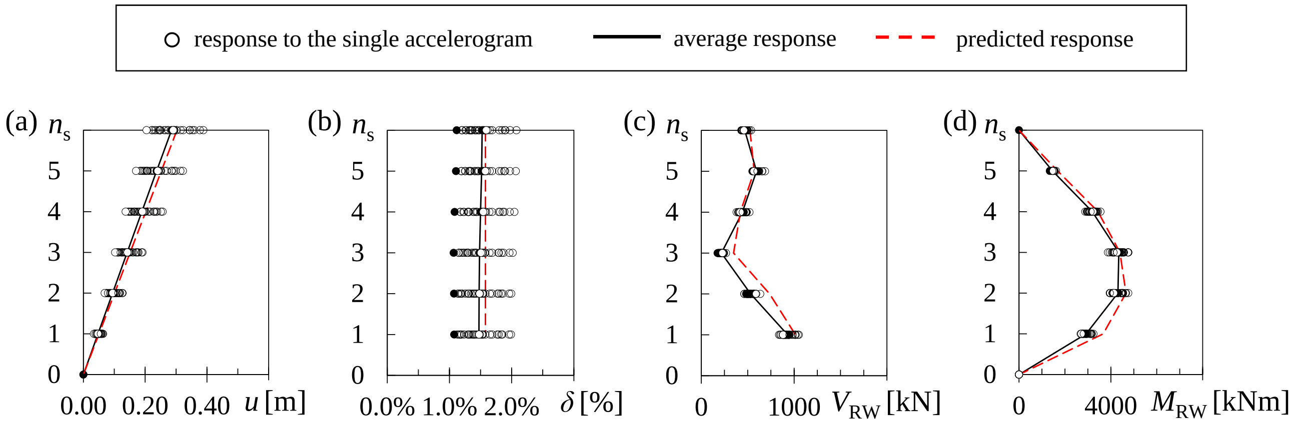

Fig. 6 – Response of the retrofitted GL structure: (a) floor displacements; (b) story drift angle; (c) shear forces and (d) bending moments in rocking wall.

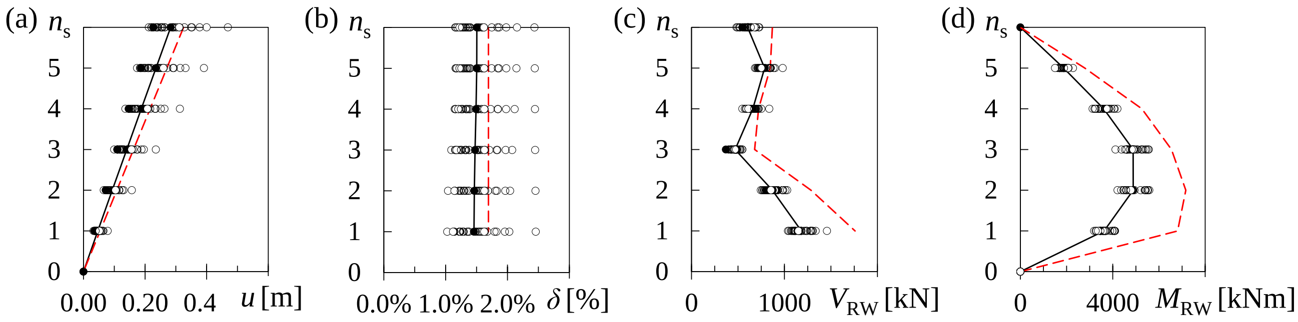

Fig. 7 – Response of the retrofitted SR structure: (a) floor displacements; (b) story drift angle; (c) shear forces and (d) bending moments in rocking wall.

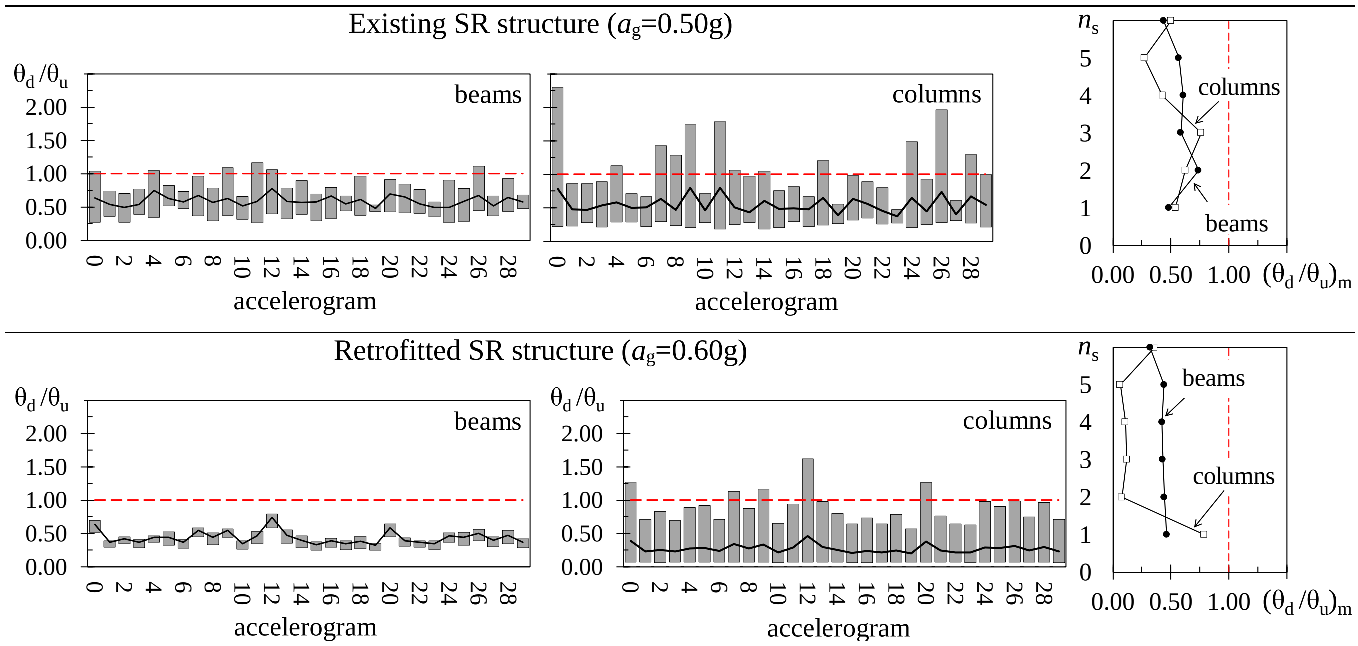

Fig. 8 – Variability of the response over the story level and over the number of accelerograms

12. Conclusions

The paper proposes a displacement-based design procedure for the retrofit of RC framed structures by RC prestressed rocking walls and viscous dampers. The procedure is validated with reference to two case studies suffering of different seismic deficiencies. The main conclusions of this study are: (1) the rocking wall is able to ensure a uniform distribution of the story drifts along the height of the structure, thus promoting a global collapse mechanism; (2) the proposed design procedure is able to limit the displacement demand of the structure below the displacement capacity; (3) the adopted formulation to predict the structural damping slightly underestimates the structural damping of the building characterized by plastic deformations in most beams (4) internal forces in rocking walls are generally well predicted.

13. References

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