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Retrofit of a hospital through strength reduction and enhanced damping

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Abstract. A procedure to retrofit existing essential facilities subjected to seismic excitation is proposed. The main features of this procedure are to reduce maximum acceleration and associated forces in buildings subjected to seismic excitation by reducing their strength (weakening). The weakening retrofit, which is an opposite strategy to strengthening, is particularly suitable for buildings having overstressed components and foundation supports or having weak brittle components. However, by weakening the structure large deformations are expected. Supplemental damping devices however can control the deformations within desirable limits. The structure retrofitted with this strategy will have, therefore, a reduction in the acceleration response and a reduction in the deformations, depending on the amount of additional damping introduced in the structure. An illustration of the above strategy is presented here through an evaluation of the inelastic response of the structure through a nonlinear dynamic analysis. The results are compared with different retrofit techniques. A parametric analysis has also been carried out to evaluate the effectiveness of the retrofitting method using different combination of the performance thresholds in accelerations and displacements through fragility analysis.

Keywords: damping; fragility; inelastic spectral analysis; retrofit; strengthening; weakening.

1. Introduction

Recent earthquakes events in Pakistan (October 2005) where many people, especially children died, have shown how vulnerable public facilities, such as schools and hospitals, are. These facilities not only must satisfy given safety performance requirements but also must remain functional after an extreme event, such as a severe earthquake. Different retrofitting techniques intend to improve performance of structures, maintaining the response below acceptable thresholds, defined also as performance limit states. The structural response of inelastic buildings is measured in terms of displacements (deformations), although the accelerations (and stresses) are also important in order to avoid damages in the non-

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structural components and contents of structures. Performance based design strategy is usually concerned with prevention of structural damage, although it is very important to protect the contents and non-structural systems, in particular in critical facilities such as hospitals, laboratories, advanced technology centers, where these "secondary" systems can be more expensive than the structure itself. Therefore, in order to improve the performance of a building, both displacements and accelerations should be kept below acceptable limits. The most common procedures to improve the seismic performance of existing buildings are the following:

• *Strengthening* produced by adding (or by reinforcing) lateral elements, which leads to a reduction of deformations and displacements but it leads to an increase in accelerations in the yielding structures (see for example the performance of the strengthened Sylmar hospital in 1994 Northridge earthquake).

• *Base isolation* changes the dynamic properties of structures, reducing the seismic acceleration and drift but increasing the total displacement.

• *Supplemental Damping* devices reduce lateral displacements, but do not change substantially the amount of seismic acceleration in the inelastic structures (Reinhorn, *et al*. 1995).

Supplemental damping has a positive influence on structural response reducing deformations in inelastic structures and also accelerations in elastic structures. Damping devices are quite inexpensive and easy to insert in existing structures. Various damping devices - with different mechanical properties and dissipation characteristics - can be adopted (Reinhorn, *et al*. 1995). Depending on the dampers chosen, stiffness and strength of the structure might increase in addition to damping characteristics, although using viscous dampers such stiffness and strength may be avoided.

2. Description of the proposed retrofit technique

This paper presents a new retrofitting method, aimed at reducing both displacements and accelerations (Reinhorn, *et al*. 2005). The retrofit procedure consists of:

1. **Weakening the building** by releasing joints, disconnecting frames or walls in the structure, to decrease its lateral strength; this reduction is however accompanied by increased displacements;
2. **Adding damping** devices in opportune locations to reduce and control the deformations and displacements.

Fig. 1 shows the effects of the steps of the above procedure separately and then combined. The effects are shown in the plane of base shear (BS) vs. displacements and in the plane of pseudo spectral acceleration (PSA) vs spectral displacements (SD). In the plane of base shear vs. displacements the capacity of the structure has been represented by a global bi-linear curve obtained through pushover analysis. It is assumed that the response of the structure will be collocated on that curve. The distribution of maximum response of the structure due to multiple ground motions is represented in the plane of PSA vs. SD by the contour plot of the joint lognormal distribution function that has been obtained by fitting the curve with the numerical data. In the first two rows of Fig. 1 the effects of the strengthening and weakening of structures are shown using the spectral approach (plane base shear vs. displacements) proposed by Reinhorn (1997) and the more accurate non linear dynamic analysis (plane PSA-SD). The effect of weakening is shown in the 2nd row of Fig. 1, where the yield strength of the structure is reduced and a bigger displacement is obtained. The third row of Figure 1 shows the effect of damping in the non linear region. consisting of a primary reduction of the maximum displacement (which switches along the inelastic branch of the capacity curve). The fourth row of Fig. 1 shows the final result of the suggested retrofitting procedure that combines the effects of weakening and added

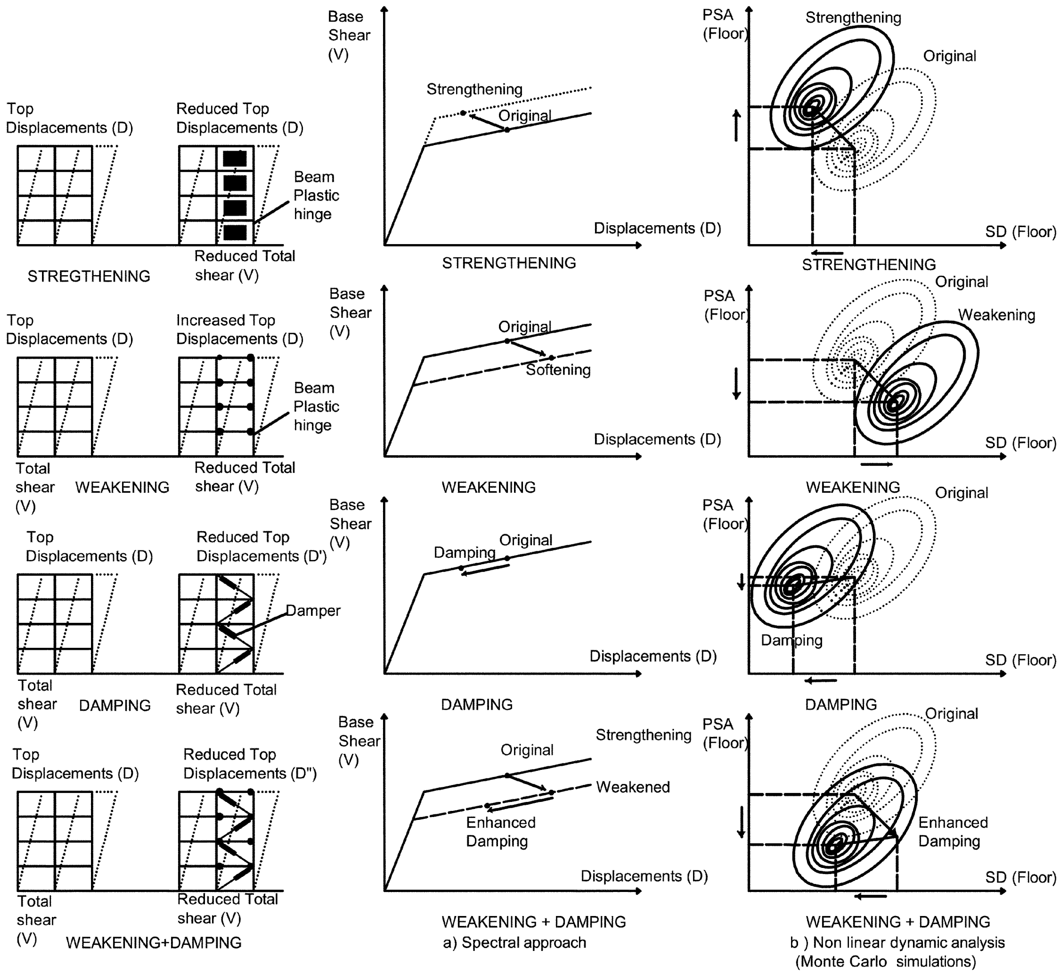

Fig. 1 Retrofit strategy: (a) spectral approach; (b) non linear dynamic analysis (Monte Carlo simulations)

damping, providing a smaller demand both in accelerations and in displacements. In this paper the effectiveness of the method and its feasibility are discussed and compared with different retrofit techniques. This method has some similarities with the base isolation method when used together with damping, since it reduces both accelerations and displacements.

3. Spectral evaluation of retrofitted structures

The analysis of the structure for different steps of the retrofitting procedure (original, weakened and damped structure) has been made through traditional non linear dynamic analysis and through a simplified spectral response approach. Such an analytical procedure, proposed by Reinhorn (1997) and Ramirez, *et al.* (2000), for low damped structures, has been specifically adapted in this work to be applied to highly damped structures. The proposed method leads to a simplified and effective evaluation of the structural response of damped structure under seismic excitation and it allows a quick evaluation of such complex retrofit. Other methodologies based on the "capacity spectrum" were developed (Deierlein, *et al.* 1991, Freeman 1994). Freeman used the inelastic capacity curve in conjunction with a combined elastic accelerations and displacements response spectra to determine both acceleration and displacement demands, while the inelastic hysteretic effects were considered through an increased period. All these methods including the proposed one are based on the observation that for a bilinear structure, the locus of all force maxima and their associated displacements are coincidental with the restoring force function $Q(t)$, having bi-linear characteristics. The suggested preliminary design method based on Reinhorn (1997) and Ramirez, *et al.* (2000), can be summarized in the following steps:

1) The strength deformation capacity curves are determined using a monotonically increasing lateral load profiles (*Static* Pushover or *Dynamic* Pushover Analysis) and a curve base shear capacity V_{MAX} versus displacements d_{MAX} are determined to have a global description of the building. This curve for MDOF systems depends on the lateral force distribution used to load it. The capacity curve is converted then to be plotted in the spectral acceleration spectral displacements plane using the following relationships:

$$S_d = d_{\max} / \varphi_i \Gamma \tag{1}$$

$$S_a = V_{\max} / (g \times \Gamma^2) \tag{2}$$

where Γ and φ are the mass normalized modal participation factor and modal shape of the dominant mode.

[1]2) The spectral demand is represented by the "composite spectrum" (Reinhorn 1997) that is a combination of the acceleration S_a and displacements S_d response spectra in the plane spectral acceleration vs. spectral displacements (Fig. 2).

3) The elastic response is obtained from the intersection of the elastic capacity curve with the elastic spectral demand (Fig. 2a).

4) For low damped structures the R-factor is determined as ratio between the elastic base shear $S_{a,el}$ and the yield base shear Q_y/W, then the elastic spectra is reduced using the R factor obtaining the "inelastic spectral demand" (Fig. 2b).

5) The effects of damping are taken into account using the B-factor adopted to scale the elastic spectra. The analysis uses the B-factors provided by NEHRP 2000 (FEMA 2001) as a function of the effective damping β_{eff}. The effective damping can be found as a function of the initial damping

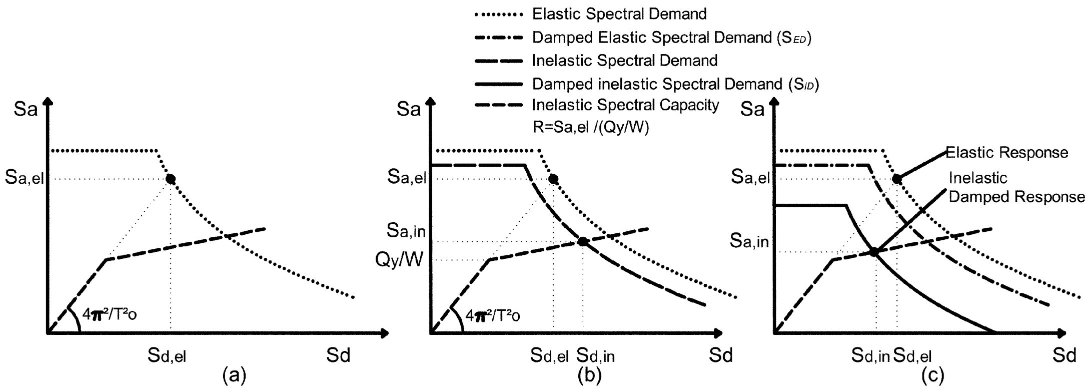

Fig. 2 Spectral evaluation of response: (a) Elastic (b) undamped inelastic and (c) damped inelastic

β_i, usually assumed to be 5% and the additional viscous damping β_V.

$$\beta_{eff} = \beta_i + \beta_v \tag{3}$$

The values of additional viscous damping, in turn is given by the following:

$$\beta_V = \frac{T_{sec}}{4\pi} \cdot \frac{\sum_j c_j \cdot \cos\alpha^2 \cdot \Delta_j^2}{\sum_i \frac{W_i}{g} \cdot D_i^2} \tag{4}$$

where T_{sec} is the secant period; Σ_j is the summation over every j-dampers; Σ_i is the summation over every i-seismic weights; c_j is the damping coefficient; $\cos\alpha$ is the displacement amplification factor; D is the modal drift obtained as a product of the maximum top displacement U_{MAX} and the eigenvector provided by the modal analysis for the first mode; Δ is the interstory drift obtained as a difference between the drifts of two consecutive stories; W is seismic weight; g the gravity constant. The secant period T_{sec} of the structure can be calculated according with the spectral procedure, with the following expression:

$$T_{sec} = 2\pi\sqrt{S_d / S_a} \tag{5}$$

where the spectral quantities S_d and S_a are, respectively, the spectral displacement and acceleration corresponding to the inelastic response of the undamped structure under the assumed (undamped) seismic input.

The elastic damped spectrum (S_{ED}) is determined from the undamped elastic spectrum dividing the spectra by the B factor.

$$S_{ED} = S_E / B \tag{6}$$

6) The inelastic damped spectral demand (S_{ID}) is obtained by dividing the elastic damped spectrum (S_{ED}) by a R factor.

$$S_{ID} = S_{ED} / R = S_E / B \cdot R \tag{7}$$

7) The inelastic response of the building is found at the intersection between spectral capacity and the inelastic damped spectral demand (Fig. 2c).

It should be noted that for inelastic spectra both spectral acceleration and displacement demands, S_E, must be adjusted, however, for most practical purposes using the equal elastic – inelastic displacement demand, only the acceleration spectral demand must be adjusted.

3.1. Preliminary design of viscous dampers

The retrofit technique described above requires the design of the viscous dampers. For example these dampers can be added to the structure in two symmetric bays at all story levels (as shown in Fig. 3). Viscous damping braces have been model using the Maxwell model that is a combination in series of a

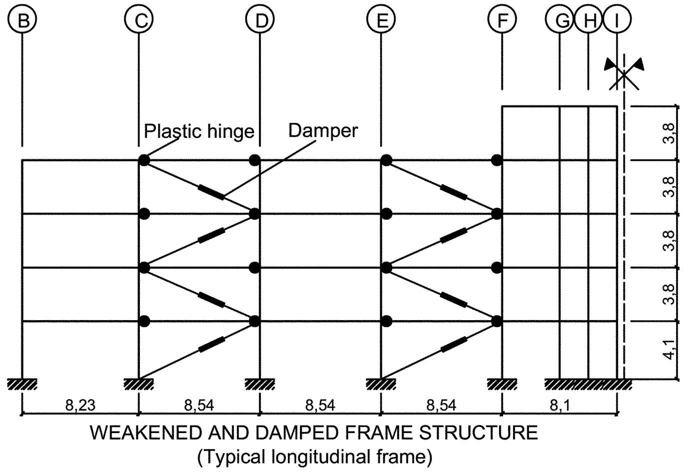

Fig. 3 Typical half longitudinal frame of the case study (symmetric half)

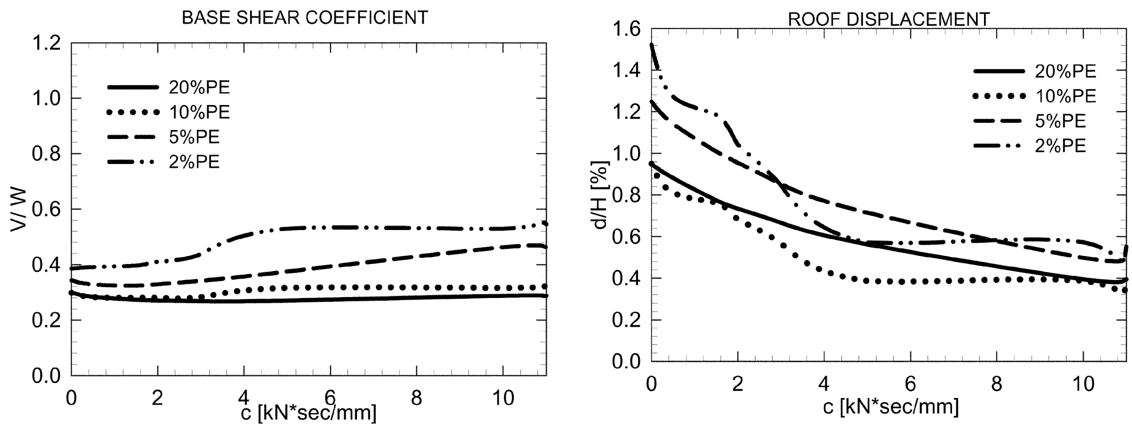

Fig. 4 Base shear coefficient and roof displacement vs. damping coefficient c

spring and a dashpot (Valles, *et al.* 1996, Ramirez, *et al.* 2000). In order to choose the optimal value of the damping coefficient c multiple non linear dynamic analyses have been performed for different values of the damping coefficient of the devices and for four different hazard levels. The normalized base shear and roof displacement for four different hazard levels corresponding to 20%, 10%, 5% and 2% of the probability of exceeding in 50 years are plotted as function of damping coefficient c (Fig. 4). Each point of these curves is obtained as average of the maximum of 25 records. In order to optimize the response a final value of $c = 5$ kN · sec/mm has been selected that corresponds to an initial *equivalent* damping ratio of 44%. The value of the equivalent damping ratio has been calculated from the logarithmic decrement (Chopra 2002) obtained from the free vibrations of the roof before the end of the motion.

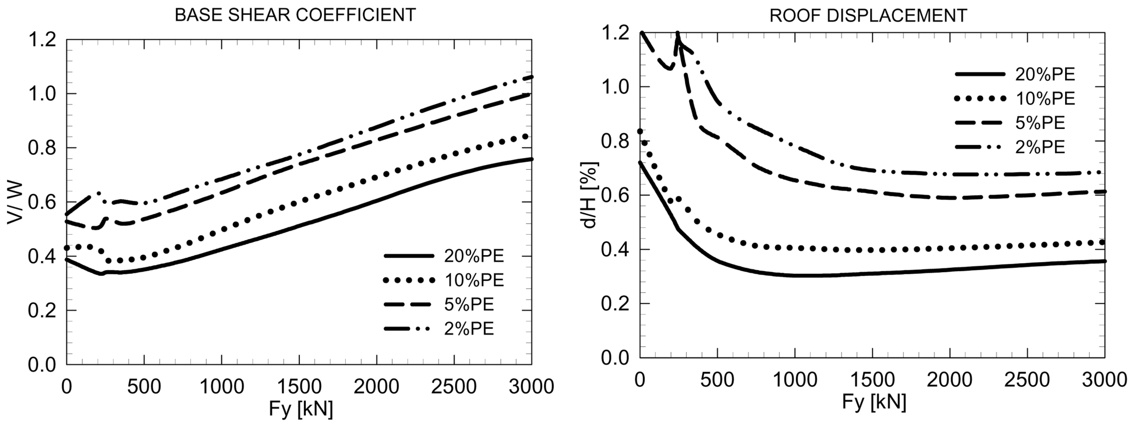

Fig. 5 Base shear coefficient and Roof displacement vs. yielding force F_y

3.2. Hysteretic dampers: unbonded braces

The hysteretic damper adopted for this part of the study is the unbonded steel brace. The damper was developed in Japan in the 1980s (Watanabe, *et al.* 1988). The unbonded steel brace is composed of a cruciform cross section of welded steel plate that is designed to yield in tension and in compression and an exterior concrete-steel tube of circular or rectangular cross section that is selected such that the buckling capacity of the tube exceeds the "squash" load of the cruciform cross section. The unbonded brace is designed to have equal strength in tension and in compression. This is conceptually superior to the concentrically (K) braced frames (known also as Chevron braces), because the beam at the intersection point of the Chevron braces does not need to be designed for large out-of balance vertical or horizontal forces (Bruneau, *et al.* 1998, Vargas 2006).

3.2.1. Preliminary design of unbonded braces

A preliminary design has been done with an energy approach (Christopoulos and Filiatrault 2006), using 25 records corresponding to 10%PE in 50 years. The brace section chosen is $HSS8 \times 8 \times 5/8$ (in inches) which corresponds to a stiffness of the brace $K_b = 226.4$ kN/mm and the preliminary yield force for the braces in each floor is $F_y \cong 1554$ kN. Using this preliminary value, multiple non linear dynamic analyses have been performed for different values of the yielding force and for four different hazard levels. The normalized base shear and roof displacement for four different hazard levels are plotted as function of yielding force F_y (Fig. 5). Each point of these curves is obtained as average of the maximum of 25 records. In order to optimize the response a final value of $F_y = 750$ kN has been selected. According to the AISC LRFD manual 3rd edition (2002) the buckling force of the brace is $F_{crb}A_g = 1178.6$ kN much smaller than the yielding force of the devices.

3.3. Infill masonry panels

Masonry infill panels are structural panels placed in the laterally resisting steel frame. They have high stiffness and are usually tightly connected to the bounding frame. Infill masonry panels are added to the structural model case study (see Fig. 6). These walls have a thickness of 250 mm. The prismatic strength of masonry adopted is $f_m = 28$ MPa and a ductility value of 32 has been used (Mander, *et al.* 1994). The

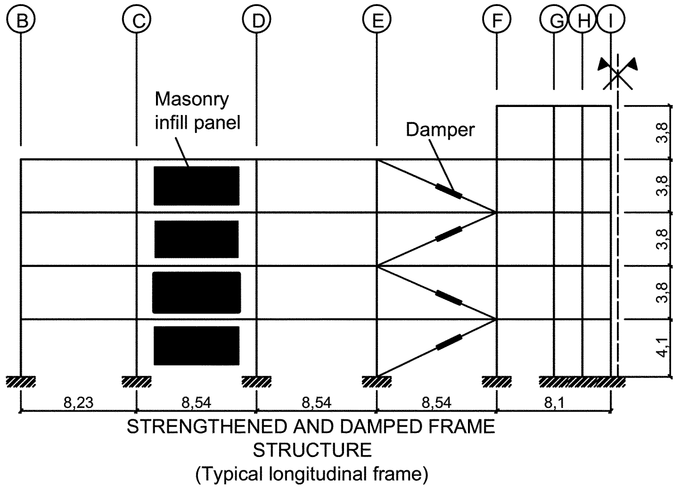

Fig. 6 Location of masonry infill panels and dampers in half longitudinal frame (symmetric half)

masonry infill panel has been modeled using a smooth hysteretic model used in IDARC2D version .6.0 (Reinhorn, *et al.* 2004). This hysteretic model, is a series of springs using the Reinhorn-Sivaselvan model (Sivaselvan, *et al,* 2001), with stiffness and strength degradation and slip lock model (Madan, *et al*. 1997). The model is able to predict the hysteretic effects of structural masonry elements subjected to cyclic loading such as “stiffness degradation”, “strength deterioration” and “pinching”.

4. Case study

A research demonstration hospital located in the San Fernando Valley in California (model W70) is used to demonstrate the proposed retrofit technique. This model, part of the Multidisciplinary Center for Earthquake Engineering Research (MCEER) critical facilities program, was selected for this study in order to ensure common base for comparison with other retrofit techniques. A series of ground motion defined as “MCEER series” (Wanitkorkul and Filiatrault 2005) has been adopted. This series consists of 100 synthetic near fault ground motions corresponding to different return periods (250, 500, 1000 and 2500 years or respectively 20%, 10%, 5% and 2% probability of exceeding in 50 years) that have been considered as white noise generated by a spectrum that is based on a physical model, the “Specific Barrier model” (Papageorgiou, *et al*. 1983a) that has been calibrated using actual near fault records.

4.1. Structural model

The hospital is a 5-stories steel moment resisting frame (Fig. 3, Fig. 6) with plan dimensions of 83.82 m in the east-west direction and 17.22 m in the north-south direction. The height of the building, from grade level to the roof is 15.54 m. The hospital was constructed in the early 1970s to meet the seismic requirements of the 1970 Uniform Building Code (ICBO 1970) and it was damaged in Northridge earthquake in 1994 although its strength was larger than 60% of its weight. Large accelerations induced strong forces in connections and damaged much of the interior of the building. The lateral force

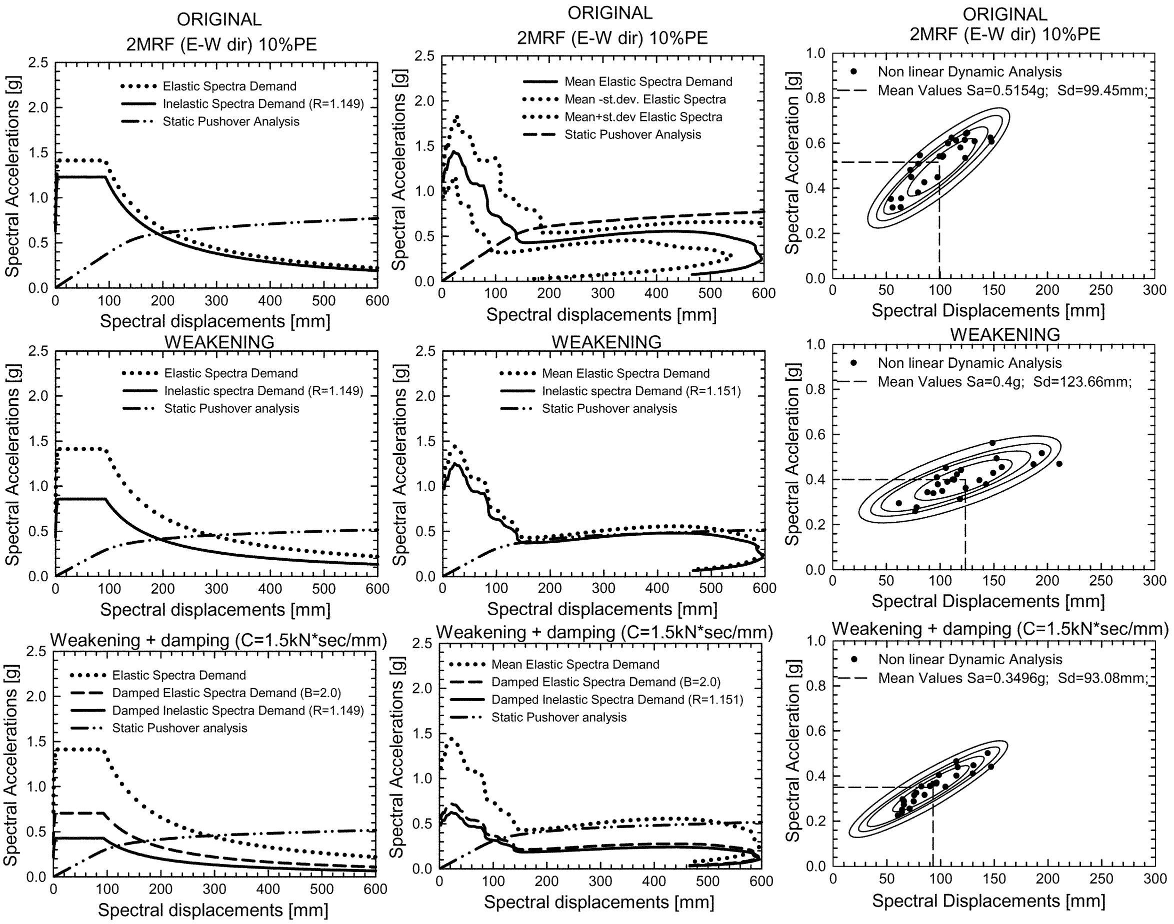

Fig. 7 Spectral approach using design spectrum (left); real spectrum (center) and non-linear dynamic analysis (right)

resisting system is comprised of four moment-resisting frames in the north-south direction and two perimeter moment-resisting frames in the east-west direction. The moment frames are constructed with ASTM A572 and A588 Grade 50 steel. ASTM A36 steel was used for the remaining steel beams, girders, and columns. A bi-dimensional non linear model of the building was developed in IDARC2D vers.6.0 (Reinhorn, *et al*. 2004) and non linear dynamic analysis and pushover analysis has been performed. The inelastic spectra demand and the inelastic spectra capacity for respectively original, weakened and weakened with damping building are plotted in Fig. 7. The curves are plotted in the spectral accelerations - spectral displacements plane. The third column shows the contour plot of the probability density function of the response of the building obtained performing non linear dynamic analysis. The analysis has been performed both in the longitudinal and transversal direction of the structure and the moment resisting frames were taken in account separately in both directions. All moment resisting frames (MRF) have been modeled as rigid beam-column connections. All other beam-column connections of the non moment resisting frames (non-MRF) were assumed to be pinned and ASTM A36 steel was used. The inelastic behavior of the beam element has be performed through the "spread plasticity model", that yield penetration along a length of the element depending from the ratio between the

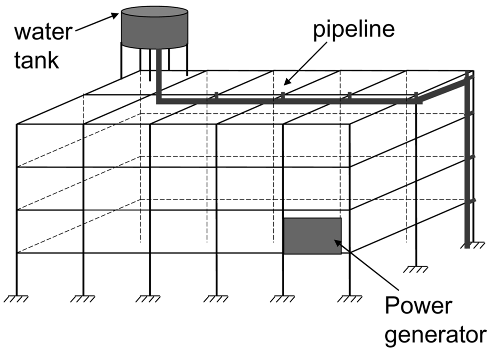

Fig. 8 Location of non structural components

ultimate and the cracking moment values (Valles, *et al*. 1996). The elements (beams and columns) in the structure have been represented through bilinear moment-curvature relationships, and a yield surface including M-N interaction. The ultimate curvature was set equal to 50 times the yield curvature, and the post-elastic stiffness was set equal to 1% to the elastic stiffness. Five different levels of damage are considered: No damage, slightly damage, moderate damage, extensive damage and complete damage. The non structural system considered consists of a water tank located at the roof level and a power generator located at the first story level (Fig. 8). It is assumed that the components are not interacting with the structure and are rigidly connected with the floor. The water tank is a drift sensitive component and the power generator is an acceleration sensitive component.

4.2. Numerical results

The different retrofit techniques analyzed in the study are compared in term of base shear coefficients, roof displacements and performance index. The damage index adopted is the modified Park and Ang damage index (Reinhorn, *et al.* 1989) that is the sum of two terms one depending on deformation and one depending on energy. Fig. 9 shows these three quantities as function of the hazard level. The weakening technique alone has the best performance in term of base shear for all hazard levels, but as expected it has the worst performance in term of displacements. However by adding damping the combined system produces lower displacements and acceleration response. It can be observed that even if the unbonded braces are able to reduce displacements to a level comparable with the retrofit technique proposed (weakening + damping), they develop bigger base shear and accelerations in the same buildings. In structures with sensitive contents, such as hospitals, this response can damage non structural components that are acceleration sensitive. However, overall in term of base shear, roof displacement and performance index the proposed retrofit technique performs better than others compared techniques.

4.3. Fragility analysis

One of the most important aspects in retrofitting is the sensitivity in the estimation and evaluation of the mechanical properties of the existing building and of the retrofit components. In the current case, a correct and reliable analysis of the structure to retrofit is extremely important in deciding the amount of weakening and the amount of additional damping devices. It also important to evaluate the uncertainties in

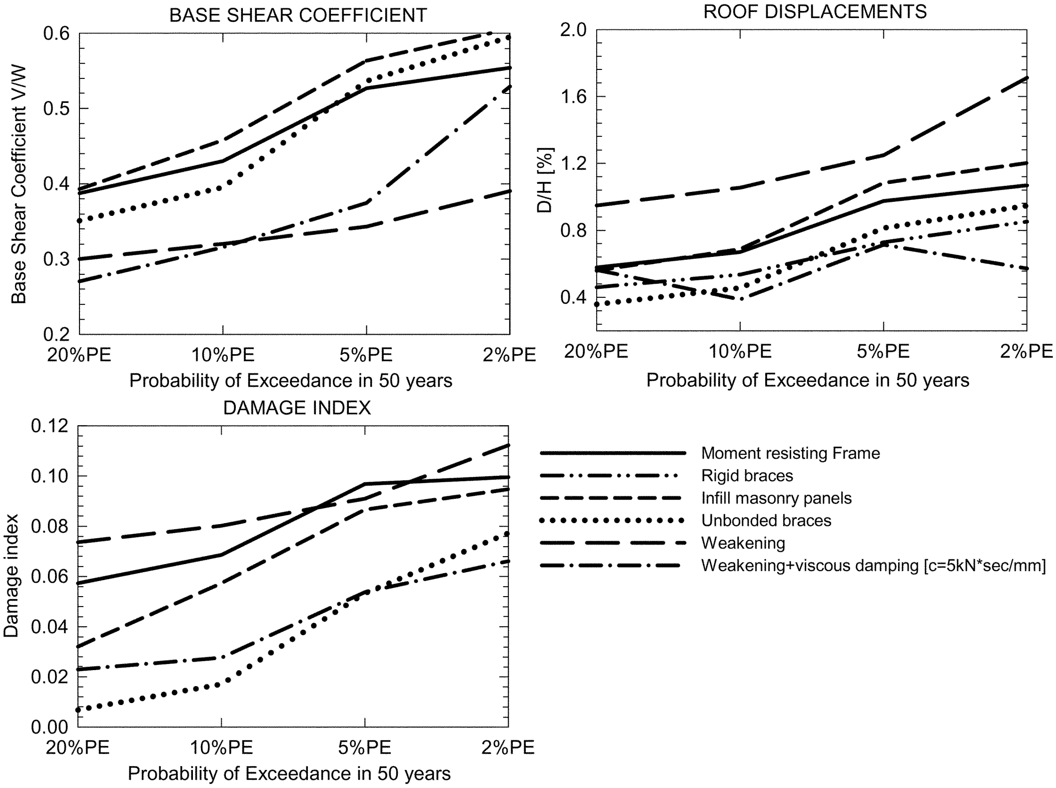

Fig. 9 Base shear, roof displacements and damage index for different retrofit strategies (non-linear dynamic analysis)

the response quantities, to assure that the retrofit will be effective and will not be overcome by the variability of response due to uncertainties. This evaluation is developed using a probabilistic approach, which implies the development of a "simplified" fragility analysis (Barron-Corverra 2000). Fragility curves are functions that represent the conditional probability that a given structure's response subjected to various seismic excitations exceeds a given performance limit state. Theoretically fragility represents

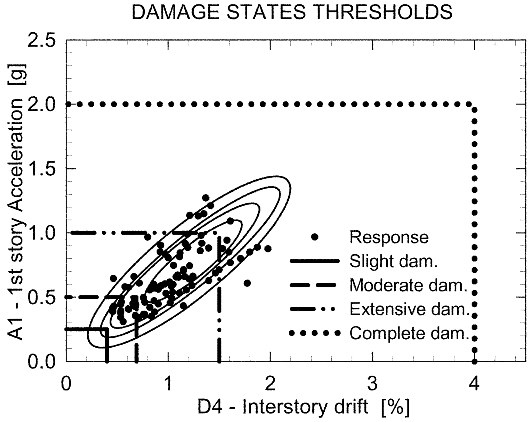

Fig. 10 Acceleration and displacements limit thresholds

the probability that the response $R = [R_1 \ldots R_n]$ of a specific structure (or family of structures) exceeds a given performance threshold $r_{\lim} = [r_{\lim 1} \ldots r_{\lim 2}]$. associated with a limit state, conditional on earthquake intensity parameter I. This definition in N dimensional form can be expressed by the following equation (Cimellaro, *et al*. 2005):

$$\text{Fragility} = P\{R_1 \geq r_{\lim 1} \cup R_2 \geq r_{\lim 2} \ldots \cup R_N \geq r_{\lim N} / I\} = P\left\{\bigcup_{i=1}^{N} R_i \geq r_{\lim i} / I\right\} \tag{8}$$

Where R_i is the response parameter in terms of a mechanical quantity (deformation, force, velocity, etc.); $r_{i\lim}$ is the response threshold parameter in terms of the above mechanical quantity that is correlated with the performance level. The calculation of fragility has been performed using a generalized formula that describes the multidimensional performance limit state threshold (MPLT) and allows considering multiple limit states related to different quantities in the same formulation (Cimellaro, *et al*. 2005). The multidimensional performance limit state function $L(R_1 \ldots R_n)$ for N-dimensional case, when N different types of limit states are considered simultaneously, is the following:

$$L(R_1, \ldots, R_n) = \sum_{i=1}^{n} \left(\frac{R_i}{r_{i\lim}}\right)^{Ni} - 1 \tag{9}$$

This model can be used to build the fragility curve of a single non structural component, or also to obtain the overall fragility curve of the entire building with non structural components, because it

Table 1 Probability of damage of drift sensitive components

	None	Slight	Moderate	Extensive	Complete
			20% in 50 years		
MRF	10.2	45.9	42.4	1.5	0.0
Unboun	58.2	40.5	1.3	0.0	0.0
Panels	18.3	59.5	22.2	0.1	0.0
W+D	9.4	85.3	5.3	0.0	0.0
			10% in 50 years		
MRF	1.3	34.2	63.1	1.4	0.0
Unboun	33.8	56.6	9.6	0.0	0.0
Panels	4.7	47.8	47.0	0.5	0.0
W+D	14.2	77.2	8.6	0.0	0.0
			5% in 50 years		
MRF	1.5	12.4	53.5	32.1	0.5
Unboun	1.8	36.5	60.4	1.3	0.0
Panels	1.3	14.3	62.8	21.5	0.1
W+D	2.1	58.3	39.5	0.1	0.0
			2% in 50 years		
MRF	0.0	0.9	56.7	42.3	0.1
Unboun	0.4	16.4	75.9	7.3	0.0
Panels	0.0	1.6	68.9	29.5	0.0
W+D	0.3	30.5	69.0	0.2	0.0

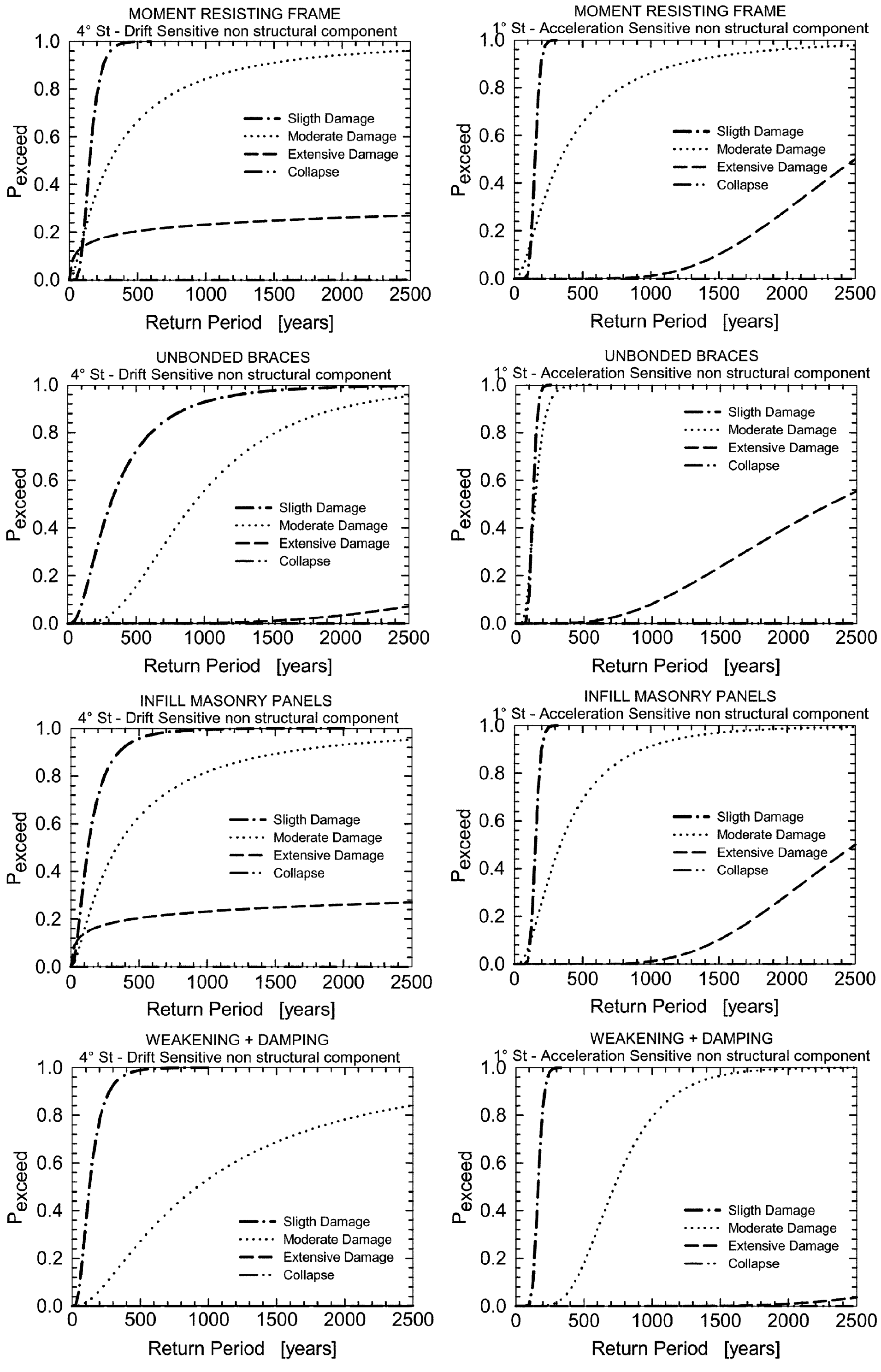

Fig. 11 Fragility curves of retrofit strategies for two non structural components: drift (left) and acceleration sensitive (right)

Table 2 Probability of damage of acceleration sensitive components

	None	Slight	Moderate	Extensive	Complete
			20% in 50 years		
MRF	0.6	71.0	28.3	0.1	0.0
Unboun	0.0	7.4	92.6	0.0	0.0
Panels	1.0	75.6	23.3	0.0	0.0
W+D	1.9	92.7	5.4	0.0	0.0
			10% in 50 years		
MRF	0.1	47.9	51.8	0.2	0.0
Unboun	0.0	13.9	85.1	1.0	0.0
Panels	0.1	46.5	53.2	0.2	0.0
W+D	0.2	83.7	16.1	0.0	0.0
			5% in 50 years		
MRF	0.0	7.8	88.5	3.7	0.0
Unboun	0.0	0.1	90.1	9.8	0.0
Panels	0.0	2.9	95.3	1.8	0.0
W+D	0.0	18.8	80.5	0.7	0.0
			2% in 50 years		
MRF	0.0	0.1	71.5	28.4	0.0
Unboun	0.0	0	45.7	54.3	0.0
Panels	0.0	0.2	72.1	27.7	0.0
W+D	0.0	2.3	94.1	3.6	0.0

Table 3 Median return period for moderate damage in the non structural components

Median return period of Moderate damage [years]	Moment resisting frames	Unbonded braces	Masonry infill panels	Weakening+ Damping
Drift sensitive non structural component	300	925	350	925
Acceleration sensitive non structural component	350	150	350	725

allows controlling different response parameters (force, displacement, velocity, accelerations etc. in the building and combine together in a unique fragility curve).

Different deterministic performance thresholds in term of acceleration and displacements have been adopted (Fig. 10) corresponding to non structural components drift and acceleration sensitive. The various retrofit techniques are compared in term of fragility curves (Fig. 11). The probability of being in a given damage state for different retrofit techniques are reported in Table 1 and 2. Inspecting the fragility curve it is possible to observe that the fragility curve of the acceleration sensitive non structural component for the weakening technique, shifts to the right more than that characterizing the unbonded brace solution, reducing the probability of reaching a given damage state. Same conclusion is observed in Table 3 where the median return period of moderate damage for different retrofit techniques is reported. Using the weakening and damping technique the return period of moderate damage for acceleration sensitive components will increase to 725 years compared to the 125 years of the unbonded braces.

5. Remarks and conclusions

In this paper a new retrofitting procedure for buildings subjected to seismic excitation is proposed, consisting in weakening the structure and adding additional damping devices. The procedure modifies both accelerations and displacements, thus improving the structural performance of structures subjected to seismic excitation. The weakening of the structure alone, it has the effect of decreasing the inelastic acceleration and the base shear response, however, it increases the ductility demand. Enhancing the structural damping alone, it produces a strong reduction in the ductility demand, without much change in the seismic acceleration. The new procedure reduces both quantities depending on the amount of strength reduction and amount of added damping. The benefits of this retrofit procedure, introduced for a general case, have been shown in detail on a case study comparing with other optimally designed retrofit techniques.

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Notation

The following main symbols are used in this paper:

I	: earthquake intensity represented by either return period, or PGA, or Modified Mercalli
Qt	: restoring force function
Q_y	: yield base shear
R	: Response parameter (deformation, force, velocity, etc.)
S_d	: spectral displacement
S_a	: spectral acceleration.
T_{sec}	: secant period
$V_{\max}$	: maximum base shear
U_{MAX}	: maximum top displacement
$a_{\lim}$	: acceleration performance threshold related to i component
$d_{\lim}$	: displacement performance threshold related to i component
$d_{\max}$	: maximum displacements
g	: acceleration of gravity
$r_{\lim}$	: performance threshold parameter correlated with damage
Δ	: interstory drift
β_V	: Viscous damping
β_{eff}	: effective damping

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