Seismic vulnerability assessment of reinforced concrete buildings having nonlinear fluid viscous dampers

Mehdi Mokhtari1 · Hosein Naderpour1

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Abstract
Seismic resilience is the residual capacity of a damaged system for recovering and sustaining an acceptable level of performance as a consequence of a seismic event. Firstly, this paper summarizes the current calculation framework of seismic resilience using fragility analyses. After that, a new metric based on vulnerability analyses is developed to calculate seismic resilience. Finally, a typical hospital building is adopted as a single building and then retrofitted with supplemental Energy Dissipation Systems (EDS) to show the applicability of the new metric and compare it with the current calculation model of the seismic resilience. The considered supplemental EDS is the Nonlinear Fluid Viscous Damper (NFVD) located as the diagonal damper-brace system. The seismic performance of the supplemental EDSs is to enhance structural performance by reducing induced lateral displacements and as a result the level of physical damages and losses. To this end, Incremental Dynamic Analysis (IDA) is implemented to achieve the damage fragility and vulnerability curves. A comparison of the fragility, vulnerability, and functionality curves indicates that NFVDs are potentially useful for attenuating seismic damages to structural and non-structural components.

Keywords RC hospital buildings · Nonlinear fluid viscous damper · Fragility · Vulnerability · Seismic resilience

1 Introduction

Significant induced damages to critical facilities such as hospitals have occurred around the World following large ground motions despite better detailing in modern buildings leading to increased damages endurance and decreased collapse capability. Such damages need extensive repair or complete replacement of the building. In recent years, some researches have been directed towards developing and using innovative materials, supplemental EDS, and seismic response modification methods for new and existing buildings to assure their post-earthquake serviceability and functionality, as well as reducing the repair costs (Guo et al. 2014; Ras and Boumechra 2016). To this end, recently the seismic-reduction methods
such as seismic isolation, supplemental EDS, and active control systems are used rather than using conventional strengthening methods. Nevertheless, some general requirements for damper installation should be considered for reducing expenses that are: 1- placing the dampers at the stories with a relatively large story drifts, 2- the effect of damper configuration for minimizing the architectural appearance, and 3- placing the dampers for minimizing the torsion deformation of the building under an earthquake (Guo et al. 2014).

The rapid growth of supplemental EDS led to the development of some guidelines and specific reports for designing, construction, and testing different dampers (FEMA 273 1997). NFVD is one of the supplemental EDSs for the improvement of seismic performance and damage control. In these systems, mechanical devices are installed into the frame throughout the height of the building. The input energy from the ground acceleration is dissipated by the movement of a piston within a viscous fluid. Some types of damper-brace systems commonly used are diagonal, chevron, toggle, and scissor damper-brace systems. However, the toggle and scissor systems are more complex than the diagonal and chevron systems and are not widely used in practice.

So far numerous experimental and analytical research studies have been conducted on NFVDs (Chopra and Chintanapakdee 2004; Lee and Taylor 2001; Sorace and Terenzi 2008) because they can be used easily in practice and modeled simply by a linear or nonlinear dashpot. According to these researches, applications were implemented in seismic upgrading of existing buildings such as office, industrial, and hospital buildings (Guo et al. 2014; Miyamoto et al. 2007; Ras and Boumechra 2016; Sarkisian et al. 2013).

The seismic resilience concept is the ability of a damaged system especially critical facility due to a severe earthquake for recovering to the desirable performance. In essence, four basic characteristics should be considered for seismic resilience including robustness, rapidity, redundancy, and resourcefulness (Bruneau et al. 2003). During the past decades, research efforts were made on seismic resilience regarding the general framework of the resilience evaluation (Bruneau and Reinhorn 2007), the quantitative estimations of the recovery times, economic losses and damages (Bruneau et al. 2003), and the quantitative framework for analyzing the disaster resilience of different structural systems (Cimellaro, Reinhorn, and Bruneau 2005, 2010; Titi and Biondini 2013) by using different retrofitted schemes.

Although there are several types of quantitative research on resilience regarding systems and communities such as water and gas distribution systems, electric power networks, infrastructures, and hospital networks (Bocchini et al. 2011; Chandrasekaran and Banerjee 2015; Cimellaro et al. 2005; Jennings 2015; Miles and Chang 2011; Waller, Ph, and Waller 2001), only a few quantitative studies have been conducted on the seismic resilience of a single building recently (Anwar and Dong 2020; Jiang et al. 2022; Mokhtari and Naderpour 2020; Rezaei Ranjbar and Naderpour 2020; Samadian et al. 2019). A single building is the main part of a community and if the resilience of a single building is quantified firstly, the resilience of the community will be better studied. On the other hand, a single building is composed of structural and non-structural components, so the resilience assessment of a single building will give decision-makers beneficial information about the resilience of these components in more detail. Therefore, this paper at first reviews the conventional quantitative evaluation of seismic resilience for a single building in terms of fragility analyses, and then develops a new calculation model of seismic resilience based on vulnerability analyses obtained from fragility analyses. For this purpose, the developed calculation model of seismic resilience is put forward and applied to a portion of the Olive View Hospital’s main building, which is named as the original model (OM), at different seismic hazard levels. This new metric is also applied to two different upgraded models with NFVDs: (1) retrofitted OM with NFVD, named
as the upgraded model (UM); (2) weakened OM by removing shear walls and retrofitted with NFVD, named as the upgraded weakened-model (UWM). In other words, this paper investigates the effect of implementing NFVDs on the seismic resilience of two conventional types of single buildings: (a) a single building with shear walls and moment-resisting frames; (b) a single building with moment-resisting frames. In this regard, at first, IDAs are conducted on each model and after that, the probability of exceedance of a given damage state for structural and nonstructural components is computed based on fragility analyses. Finally, the seismic resilience index is calculated based on the fragility and vulnerability analyses.

2 Research significance

According to different definitions of resilience, it can be observed that they are divided into two major groups. The first group insists on the post-event situation and the recovery process, while the second group takes into account the state of the system before the occurrence of a disaster. In the seismic resilience evaluation, it is vital to consider both groups (i.e. pre and post-event), because a thorough analysis of the state of a system before the earthquake occurrence can reveal the weak points, leading measures which can decrease the seismic risk and increase the rapidity of the recovery of the system. To this end, the concept of seismic resilience is very useful especially for making post-disaster management plans or conceiving a strategy of action that may increase the resilience of the system to a seismic event.

This work aims to enhance the seismic resilience of typical hospital buildings as special buildings that should be structurally and non-structurally safe during an extreme earthquake, through improved engineering and management tools. For achieving this goal, it is necessary to understand seismic resilience concepts and dimensions, as well as create different models to calculate seismic resilience. To this end, this paper evaluates the seismic resilience of RC hospital buildings retrofitted with NFVD. The research trend is to use NFVD as a new strategy for improving and enhancing the seismic resilience of hospital buildings against extreme earthquakes. This retrofit technique will be able to reduce displacement and acceleration (Viti et al. 2006). The reduction of accelerations is important for hospitals because many of the building contents (nonstructural components) are acceleration sensitive. In this regard, the significant novelty of this research study is to assess the seismic resilience index based on the vulnerability approach and develop loss function based on vulnerability analyses for the comparative seismic resilience of existing RC hospital buildings before and after improving them by the incorporation of NFVDs that has not been investigated in previous studies. Besides, the results obtained from the developed calculation model for the evaluation of the seismic resilience of single buildings can be incorporated into decision and management systems. Indeed, decisions created before an earthquake can considerably have effects on the seismic resilience of a system, as well as its robustness and capability of fast recovery.

3 NFVD-brace system

Figure 1 shows a typical schematic of an FVD that consists of a piston head with orifices contained in a cylinder filled with a highly viscous fluid. Energy is dissipated in the damper through the movements of the piston head and fluid orifices. The nonlinear force–velocity behavior of an FVD is given by Eq. (1).
where $C_D$ is the damping coefficient in terms of force per velocity raised to the $\alpha$ power; $\alpha$ is a damping exponent in a typical range of 0.35 to 1.0 for seismic cases; $\dot{u}$ is the relative velocity between both ends of the damper; $\text{sgn}()$ is the signum function.

Considering $\alpha = 1.0$, Eq. (1) becomes $F_D = C_D \dot{u}$, which represents the linear force–velocity behavior of a linear FVD. Therefore, $\alpha$ indicates the non-linearity of FVDs.

The mechanical system of an FVD can be mathematically depicted by the simple Maxwell model in Fig. 2b, which consists of a damper and an equivalent spring with the stiffness of $K_D$. According to Fig. 2a, the spring with the stiffness of $K_b$ is used to simulate the brace and the spring with the stiffness of $K_d$ is due to the inherent stiffness of the FVD. In the Maxwell model, the damper and the brace are connected in series.

For a multi-degree of freedom (MDOF) system with NFVDs, the damping coefficient can be calculated using the process mentioned in the flowchart of Fig. 3. In this flowchart, considering only the first vibration mode, Eq. (2) is the effective damping ratio of a building structure contributed by the NFVDs that was derived by (Seleemah and Constantinou 1997; Soong and Constantinou 1994). According to the study conducted by (Ras and Boumechra 2016), it would be convenient to distribute $C$ values identically in each story. They also demonstrated that the efficiency of dampers on the up floors is smaller than the bottom floors (Ras and Boumechra 2016).

$$\zeta_{\text{eff}} = \zeta_0 + \frac{T^{2-\alpha} \sum \eta_j \times C_j \times \lambda \times \cos^{1+\alpha} \frac{\theta_j - \theta_{j-1}}{2\pi} \times (\phi_j - \phi_{j-1})^{1+\alpha}}{(2\pi)^{3-\alpha} \times A^{1-\alpha} \sum M_i \phi_i^2}$$  

where: $T$ is the natural period of the first vibration mode; $C_j$ is the damping coefficient of the NFVDs at the $j$th story; $\eta_j$ is the number of identical NFVDs with the same $C_j$ in each story; $\phi_j$ and $\phi_{j-1}$ are the first modal displacements of the $j$th and $j-1$th story, respectively; $\theta_j$ is the inclination angle of the NFVDs at the $j$th story; $A$ is the amplitude of roof response corresponding to $\phi_j$ normalized to a unit value at the roof; $M_i$ is the mass of the $i$th floor; $\lambda$ is a factor which can be calculated by Eq. (3):
where \( \Gamma \) is the gamma function.

4 Calculation model of seismic resilience based on vulnerability

The graphical flowchart illustrated in Fig. 4 presents the seismic resilience assessment of typical buildings that was developed based on the general methodology proposed by Cimellaro et al. (Cimellaro et al. 2010). In the first step, the nonlinear model of the building is built and is subjected to a set of ground motions. These earthquake records are selected based on the geotechnical profile and intensity of ground motion at the building location as per ASCE/SEI 7–10 (ASCE 2006) standard. In the second step, the nonlinear building response is examined to record the maximum inter-story drift ratio among floors. Then, IDA curves are driven using spectral acceleration corresponding to the maximum inter-story drift ratio. After the IDA curves were derived, fragility analysis is conducted considering four different damage levels (slight, moderate, extensive, and complete) as per HAZUS (HAZUS 2003). Finally, vulnerability curves are calculated for each of the structural and non-structural components and consequently, the seismic losses and the recovery function are determined. The last step is to evaluate the seismic resilience of the studied building.

Fragility is the probability of being in or exceeding a demand parameter \( (D) \) from a given performance limit state or capacity \( (C) \), conditional on ground motion intensity \( (IM) \) parameter such as peak ground acceleration \( (PGA) \), peak ground velocity \( (PGV) \), spectral acceleration \( (S_a) \), spectral displacement \( (S_d) \), etc. The response and limit state parameters can be deformation, drift, acceleration, stresses, strains, mechanical properties, or other functionality measures. To this end, the fragility function, \( F \) can be defined as Eq. (4) (C. Lee and Su 2012):

\[
F = P(D \geq C|IM) = 1 - \Phi \left( \frac{\ln C - m_X(IM)}{\sigma_X} \right)
\]  

where \( \Phi \) is the cumulative distribution function of a standard normal variable, with a mean of 0 and a standard deviation of 1. Respectively, \( m_X \) and \( \sigma_X \) are the mean and the standard deviation of the variable \( X \) that can be estimated from the regression analysis for each component (C. Lee and Su 2012). In this method, \( D \) is obtained from IDAs and is assumed.
to follow a power law of \( D = a(IM)^b \) and therefore has the logarithmic transformation which is related to variable \( X \) as Eq. (5):

\[
X = \ln(D) = \ln(a) + b \ln(IM)
\]

where the unknown regression coefficients \( a \) and \( b \) can be easily computed through a linear regression analysis of the \( D \) in the transformed space. \( D \) is obtained from IDAs. Therefore, \( m_X \) and \( \sigma_X \) can be expressed as Eq. (6) and (7):

\[
m_X(IM) = \ln \left( aIM^b \right)
\]

\[
\sigma_X = \sigma_{\ln D} = \sqrt{\frac{1}{n-2} \sum_{i=1}^{n} \left[ \ln \left( \frac{\delta_i}{aIM^b_i} \right) \right]^2}
\]

In HAZUS, the limit value of inter-story drift ratio corresponding different damage states has been considered for different types of buildings. For instance, Table 1 shows the limit value of the inter-story drift ratio at four damage levels for a hospital building as an essential facility based on HAZUS’s recommendation.

Since many of hospital building contents are acceleration- and drift-sensitive components, therefore it is worth mentioning that the consideration of non-structural damages is necessary for loss and resilience assessment. To this end, HAZUS proposes the damage fragility curves for both acceleration- and drift-sensitive components of different building types (HAZUS 2003). HAZUS also presents the median and lognormal standard deviation (Beta) values corresponding to each damage state. For instance, Table 2 gives the values of median and lognormal standard deviation for each non-structural component of two hospital building types. Using these values and the fragility function mentioned in HAZUS the fragility curves for the acceleration- and drift-sensitive components at different damage states can be plotted according to Fig. 5.
In order to assess the vulnerability of single buildings and loss evaluation before or after an earthquake, the structural and non-structural vulnerability analyses can be conducted corresponding to spectral accelerations for different hazard levels such as 2%, 5%, 10%, and 20% probability of exceedance in 50 years. For this purpose, according to Eq. (8) discrete probabilities regarding each damage state $ds$, are calculated using cumulative probabilities obtained from fragility functions firstly, and then vulnerability function can be obtained by Eq. (9).

$$P[ds = \text{Complete}] = P[ds \geq \text{Complete}]$$
$$P[ds = \text{Extensive}] = P[ds \geq \text{Extensive}] - P[ds \geq \text{Complete}]$$
$$P[ds = \text{Moderate}] = P[ds \geq \text{Moderate}] - P[ds \geq \text{Extensive}]$$
$$P[ds = \text{Slight}] = P[ds \geq \text{Slight}] - P[ds \geq \text{Moderate}]$$

(8)

$$V = \sum_{ds} [P(DS = ds) \times \text{MDF}]_{ds}$$

(9)

where $MDF$ is the mean damage factor that is defined as the central value of the damage factor range which introduces the conditional probability of a damage state for an earthquake intensity in accordance with Table 3 (HAZUS 2003).

### Table 1 Limit value of inter-story drift ratio at different damage levels for high-rise hospital buildings (HAZUS 2003)

| Type of hospital building | Description                              | Inter-story drift ratio at damage limit states  |
|---------------------------|------------------------------------------|-----------------------------------------------|
|                           |                                          | Slight | Moderate | Extensive | Complete |
| C1H                       | High-rise RC building with moment resisting frames | 0.0031 | 0.0054   | 0.0146    | 0.0375   |
| C2H                       | High-rise RC building with shear walls    | 0.0025 | 0.0053   | 0.0145    | 0.0375   |

### Table 2 Median and lognormal standard deviation for non-structural components of the Hospital (HAZUS 2003)

| Component Type                        | Type of Hospital Building | Slight Median | Slight Beta | Moderate Median | Moderate Beta | Extensive Median | Extensive Beta | Complete Median | Complete Beta |
|---------------------------------------|---------------------------|---------------|-------------|-----------------|---------------|------------------|---------------|-----------------|---------------|
| Acceleration-Sensitive with Median Spectral Acceleration (g) and Lognormal standard Deviation | C1H                       | 0.38          | 0.65        | 0.75            | 0.65          | 1.50             | 0.65          | 3.00            | 0.65          |
|                                       | C2H                       | 0.38          | 0.65        | 0.75            | 0.64          | 1.50             | 0.64          | 3.00            | 0.64          |
| Drift-Sensitive with Median Spectral Displacement (inches) and Lognormal standard Deviation | C1H                       | 3.46          | 0.71        | 6.91            | 0.71          | 21.60            | 0.79          | 43.20           | 0.93          |
|                                       | C2H                       | 3.46          | 0.70        | 6.91            | 0.71          | 21.60            | 0.77          | 43.20           | 0.87          |
The seismic resilience index can be graphically defined as the normalized area underneath the performance function $Q(t)$ (see Fig. 6) which is mathematically expressed by Eq. (10).

$$RI = \int_{t_{OE}}^{t_{OE} + T_{RE}} \frac{Q(t)}{T_{RE}} dt \quad (10)$$

where $T_{RE}$ is the recovery time, the time required to recover the performance of a system to attain a desirable level of functionality, and $t_{OE}$ is the occurrence time of the event $E$. Performance function $Q(t)$ is defined as Eq. (11).

$$Q(t) = 1 - L(I, T_{RE}) \times F_{RE}(t, t_{OE}, T_{RE}) \times [H(t - t_{OE}) - H(t - (t_{OE} + T_{RE}))] \quad (11)$$

where $L(I, T_{RE}), F_{RE}(t, t_{OE}, T_{RE})$, and $H(\cdot)$ are the loss function, the recovery function, and the Heaviside function respectively.

The recovery function is related to how the resilience procedure extends in time. At the first step of a recovery process, a period of time is considered for the organization of restoring the activities and resources. Recovery time depends on the level of seismic resilience within the society that will be smaller for a prepared community. Following this idea, there are three types of empirical recovery functions including 1- linear, which is usually implemented when there is no particular information as to the way in which the recovery process is developing. In this case, the society has a medium preparedness deal with a seismic event, 2- Exponential, which is used for a fast societal response to a disaster, but a high initial recovery rapidity, that decreases as the intended

| Damage States | Damage Factor Range (%) | MDF (%) |
|---------------|--------------------------|--------|
| None          | 0                        | 0      |
| Slight        | >0–4                     | 2      |
| Moderate      | 4–16                     | 10     |
| Extensive     | 16–84                    | 50     |
| Complete      | 100                      | 100    |
According to Eq. (12), the total direct economic loss function can be divided into two physical direct economic losses \( L_{D,C} \): 1- structural losses \( L_{D,S} \) which happen immediately during an event, and 2- non-structural losses \( L_{D,NS} \) which have temporary dependencies.

![Graphical schematic of resilience](image)

**Fig. 6** Graphical schematic of resilience

\[
L(I, T_{RE}) = \sum_C L_{D,C} = L_{D,S}(I) + L_{D,NS}(I, T_{RE})
\]  

(12)

Structural and non-structural losses for an essential facility are mainly defined by the physical direct economic loss and can be calculated in terms of the fragility function and the ratio between building repair cost \( C_s \) and replacement building cost \( I_s \). In HAZUS the physical direct economic loss function has been proposed in terms of the fragility function according to Eq. (13). As can be seen from Eq. (13), the physical direct economic loss for each component (structural or non-structural component) is the summation of the losses obtained at each damage state.

\[
L_{D,C} = \sum_{ds} \left\{ \left( \frac{C_s}{I_s} \right)_{ds} \times P[DS \geq ds] \right\}
\]  

(13)

where \( P[DS \geq ds] \) is the fragility function at a specific damage state, \( ds \).

As mentioned before, in this study a new calculation model is developed for evaluating the seismic resilience in which the physical direct economic loss function is obtained based on the vulnerability function (Eq. (9)) instead of the fragility function. To this end, firstly the loss function is calculated at each damage state separately according to Eq. (14).
After that, the following equations are considered using Eq. (14):

\[
\begin{align*}
L_{\text{Slight}} &= P[DS \geq \text{Slight}] \times \left( \frac{C_s}{I_s} \right)_{\text{Slight}} \\
L_{\text{Moderate}} &= P[DS \geq \text{Moderate}] \times \left( \frac{C_s}{I_s} \right)_{\text{Moderate}} \\
L_{\text{Extensive}} &= P[DS \geq \text{Extensive}] \times \left( \frac{C_s}{I_s} \right)_{\text{Extensive}} \\
L_{\text{Complete}} &= P[DS \geq \text{Complete}] \times \left( \frac{C_s}{I_s} \right)_{\text{Complete}}
\end{align*}
\] (14)

Finally, using Eq. (15) the physical direct economic loss, based on vulnerability, for each component (structural or non-structural component), is obtained according to Eq. (16).

\[
\Delta L_{\text{Slight}} = |L_{\text{Slight}} - L_{\text{Moderate}}| \\
\Delta L_{\text{Moderate}} = |L_{\text{Moderate}} - L_{\text{Extensive}}| \\
\Delta L_{\text{Extensive}} = |L_{\text{Extensive}} - L_{\text{Complete}}| \\
\Delta L_{\text{Complete}} = L_{\text{Complete}}
\] (15)

where \( L'_{D,C} \) is the new loss function based on vulnerability function that can be replaced with \( L_{D,C} \) in Eq. (12) and subsequently Eq. (11).

Table 4 shows structural and non-structural repair cost ratios in percent of building replacement costs regarding the hospital occupancy as per HAZUS (HAZUS 2003). It is noteworthy that the values of complete damage state in Table 4 should sum to 100 since the complete state mentions that the building must be replaced (e.g. for acceleration-sensitive non-structural components \( C/I_s = 151.3\% \)).

| Components                                | \( C_s/I_s \) (%) | Slight | Moderate | Extensive | Complete |
|-------------------------------------------|-------------------|--------|----------|-----------|----------|
| Structural components                      | 0.2               | 1.4    | 7.0      | 14.0      |          |
| Acceleration-sensitive non-structural components | 1.0               | 5.1    | 15.4     | 51.3      |          |
| Drift-sensitive non-structural components  | 0.8               | 3.5    | 17.4     | 34.7      |          |
5 Case study

As mentioned before, in this paper a portion of the Olive View Hospital’s main building is selected as the case study to apply the conventional and developed calculation models of seismic resilience considering four seismic hazard levels. These calculation models are also applied to two different upgraded models to investigate the effect of NFVDs on the seismic resilience of two conventional types of single buildings: (a) a single building with shear walls and moment-resisting frames; (b) a single building with moment-resisting frames.

5.1 Models characteristics

The Olive View medical center was the largest of Los Angeles’s medical complex. As shown in Fig. 7, the main building of the Olive View medical center was a relatively huge RC building with four wings located around a central courtyard. This building had 6 stories and was built in 1969 based on Uniform Building Code (UBC 1997). The lateral load systems were shear walls and moment-resisting frames (see (Mahin et al. 1976) for details). The shear walls had been used only in four upper stories and had not been continued to two bottom stories. The hospital building sustained major structural and non-structural damages during the San Fernando earthquake (February 9, 1971), and was subsequently rebuilt. While there was relatively minor damage in the four upper stories due to the presence of the shear walls, the damages had considerably occurred in the two bottom stories due to discontinuation of the shear walls that led to large displacements and story drifts and triggered a soft story mechanism failure type (Mahin et al. 1976).

Fig. 7 Plan of the main building of Olive View Hospital
Regarding the complexity of the structural system and many special details of the main building, a complete physical description is impossible. Thus, for understanding the purposes of this study, only the essential characteristics of the building are presented. Due to the size of the building and the unavailability of the structural details for realistic three-dimensional (3D) modeling, a 2D model corresponding to one-quarter of the building (Wing D) is selected. Fig. 8a is the plan view of Wing D including five frames and is assumed to be completely isolated from the rest of the building at the boundaries. These frames have been connected at the floor levels by diaphragms that have been assumed rigid in their plane and rotation of the diaphragms has not been allowed. Thus, all of the frames have the same lateral displacement along the N-S direction, and deformations in the E-W direction have not been allowed (Mahin et al. 1976). Frames 26, 27, and 28 are completely identical in the strength and stiffness properties. So, by the appropriate transformation of mass, strength, and stiffness, these three frames can be modeled as an equivalent frame with the same geometrical and section dimensions but three times the gravity loads, modulus of elasticity, and the floor level masses.

Fig. 8  Schematic view of the Wing D model a Plan b Elevation
of frame 28. The equivalent frame will be called frame 28*. Fig. 8b indicates schematically the 2D frame representation of the OM used in nonlinear analysis. In this model, frames are connected at each floor level by rigid axial links with pin-ended that are analytically treated as elastic elements with nearly zero flexural stiffness.

As said before, this paper study the effect of using NFVDs on the seismic resilience of two upgrade models of the OM: 1- UM; 2- UWM. Due to the fact that FVDs are relatively expensive in comparison with conventional retrofitting methods and may influence the architectural space more or less, therefore they should be placed most effectively. For this purpose, based on the general requirements proposed by (FEMA 273 1997) and (Guo et al. 2014) for practical design of damper-brace systems for seismic retrofitting of the OM and also due to the severe damages occurred in the two bottom stories of the main building of the Olive View Hospital, the diagonal-brace NFVDs are suggested and installed in these stories according to Fig. 9a. Figure 9b shows the UWM in which the OM was weakened by removing shear walls and retrofitted with NFVD at all height of the building.
5.2 Nonlinear modeling of the buildings

As before mentioned in Fig. 4, at the first step the nonlinear model of the building is made. To this end, the OpenSees software is employed for the simulation of the seismic response of structural systems (Mazzoni et al. 2006). The flowchart shown in Fig. 10 summarizes the nonlinear modeling procedure for this case study. Fiber elements are used to model the RC elements for making the strain compatibility between the rebar and the adjacent concrete without any relative movement between both. The fiber element includes concrete cover, concrete core, and steel rebar. In the fiber model, effects such as the confinement variation of concrete, spalling of concrete cover as well as buckling of steel rebar have not been considered. In this study, the Steel01 and Concrete02 materials are used to model reinforcing steel and concrete material, respectively in OpenSees software as shown in Fig. 10. Based on the field tests conducted on the concrete specimens and reinforcing bars after the San Fernando earthquake (Breasler and Bertero 1973), the characteristics of the concrete and steel materials are selected according to Table 5. By performing the eigenvalue analysis, the fundamental period of vibration of the OM and the WM (weakened model) was $T = 0.63\mbox{ s}$ and $T = 1.20\mbox{ s}$, respectively. NFVDs are simulated using the twoNodeLink element. 

![Flowchart of the nonlinear modeling procedure](image)
### Table 5 Properties of concrete and steel used in the nonlinear modeling (Breasler and Bertero 1973)

| Concrete | Young’s modulus (ksi) | Ultimate compressive strength (ksi) | Yield stress (ksi) | Critical cracking stress (ksi) | Tension softening modulus (ksi) | Concrete | Poisson’s ratio, ν |
|----------|-----------------------|------------------------------------|--------------------|-------------------------------|--------------------------------|----------|-------------------|
| 3080     | 3080                  | 5.2                                | 1.73               | 0.52                          | 1026.67                         | All concrete, except for the ground and first story column of the main building | 0.1     |
| 3540     | 3540                  | 7                                  | 2.33               | 0.7                           | 1180                           | Ground and first story columns of the main building | 0.1     |

| Steel    | Reinforcing steel     | Young’s modulus (ksi) | Yield stress (ksi) | Ultimate tension strength (ksi) | Poisson’s ratio, ν |
|----------|-----------------------|-----------------------|--------------------|-------------------------------|-------------------|
| All reinforcing steel, except for the vertical column reinforcement | 29,100                | 52.60                 | 79.50                          | 0.3               |
| vertical column reinforcement | 28,540                | 71.70                 | 112.50                         | 0.3               |
element in OpenSees. In the present study, 20 and 24 same diagonal-brace dampers are suggested to be used for the UM and the UWM, as shown in Fig. 9a and b, respectively in which the dampers were placed in the stories with a relatively large inter-story drift. According to the procedure depicted in Fig. 3, by considering the damping exponent ($\alpha$) of 0.50, the inherent damping ratio ($\xi_0$) of 5%, and the desired adding damping ratio ($\xi_d$) of 30%, the damping coefficient ($C$) of the UM and the UWM shall be equal to 101.016 kips. in/s and 63.097 kips.in/s, respectively.

5.3 Selection of ground motion records

According to the soil type of the construction site, type D according to NEHRP classification (Council 1997), 11 far-field ground motions from 22 far-field earthquake records and 8 near-field ground motions from 28 near-field earthquake records proposed by FEMA P695 (FEMA P695 2009) were selected for nonlinear dynamic analyses. Tables 6 and 7 tabulate the properties of far and near-field earthquake records respectively including magnitude ($M$), maximum horizontal peak ground acceleration ($PGA_{\text{max}}$), maximum peak ground velocity ($PGV_{\text{max}}$), and normalization factors as proposed in FEMA P695. Thereafter, according to the scaling method suggested in ASCE/SEI 7–05 standard both ground motion earthquake records are scaled such that the median spectrum matches the maximum considered earthquake (MCE D) over the period range $0.2T_1$-1.5$T_1$, where $T_1$ is the fundamental period of the studied building due to dynamic analysis.

6 Seismic response results

In this study, four seismic hazard levels, from the frequent lower magnitude to the frequent higher magnitude, have been considered according to the earthquake intensity at the site of the Olive View Hospital (i.e. 20, 10, 5 and 2% probability of exceedance within 50 years). Fig. 11, and Fig. 12 indicate the seismic hazard curves in terms of the spectral acceleration and the spectral displacement, respectively. As can be seen, an increase in the effective damping ratio leads to a significant reduction in both spectral acceleration and displacement.

With respect to the OM and the upgraded models, the $S_a$ and the $S_d$ corresponding to either of seismic hazard levels have been listed in Tables 8 and 9, respectively.

6.1 IDA Results

The IDA is conducted for each model using the nonlinear time history analyses (Vamvatsikos and Cornell 2002) under all 19 earthquake records. For obtaining the IDA curves, maximum inter-story drift ratio and spectral acceleration at the first mode period are selected as the demand parameter ($D$) and the intensity measure parameter ($IM$), respectively. Figures 13, 14 and 15 show the IDA curves for each model under all far and near-field earthquake records.

As reported in (Mahin et al. 1976), although the main building of the Olive View Hospital was designed for lateral forces substantially higher than those required by then-existing and current code requirements, the main failure of the building was due to a “soft” story type of response as a consequence of discontinuing the shear walls in the bottom two stories which resulted in a concentration of the high drifts and inelastic
| ID. no. | Earthquake | M | Recording station name | Recorded motions | Normalized motions |
|---|---|---|---|---|---|
| 1 | Duzce, Turkey 1999 | 7.1 | Bolu | 0.82 | 0.62 |
| 2 | Imperial Valley 1979 | 6.5 | Delta | 0.35 | 1.3 |
| 3 | Imperial Valley 1979 | 6.5 | El Centro Array #11 | 0.38 | 1 |
| 4 | Kobe, Japan 1995 | 6.9 | Shin-Osaka | 0.24 | 1.09 |
| 5 | Kocaeli, Turkey 1999 | 7.5 | Duzce | 0.36 | 0.68 |
| 6 | Landers 1992 | 7.3 | Yermo Fire Station | 0.24 | 0.98 |
| 7 | Landers 1992 | 7.3 | Coolwater | 0.42 | 1.14 |
| 8 | Loma Prieta 1989 | 6.9 | Capitola | 0.53 | 1.08 |
| 9 | Loma Prieta 1989 | 6.9 | Gilroy Array #3 | 0.56 | 0.87 |
| 10 | Superstition Hills 1987 | 6.5 | El Centro Imp. Co | 0.36 | 0.86 |
| 11 | Superstition Hills 1987 | 6.5 | Poe Road (temp) | 0.45 | 1.16 |
Table 7  Properties of the selected near-field earthquake records

| ID. no. | Earthquake          | Recorded Motions | Normalized Motions |
|--------|---------------------|-------------------|--------------------|
|        | Name M Recording Station Name | $\text{PGA}_{\text{max}} (g)$ | $\text{PGV}_{\text{max}} (\text{cm/s})$ | Normalization Factor | $\text{PGA}_{\text{max}} (g)$ | $\text{PGV}_{\text{max}} (\text{cm/s})$ |
|        | NF-Pulse Record Subset |                  |                    |                    |                             |                       |
| 1      | Imperial Valley-06, 1979 6.5 El Centro Array #6 | 0.45 | 113.55 | 0.997 | 0.45 | 113.21 |
| 2      | Imperial Valley-06, 1979 6.5 El Centro Array #7 | 0.47 | 113.14 | 1 | 0.47 | 113.14 |
| 3      | Superstition Hills-02, 1987 6.5 Parachute Test Site | 0.43 | 134.29 | 0.843 | 0.36 | 113.21 |
| 4      | Erzican, Turkey, 1992 6.7 Erzincan | 0.50 | 107.14 | 1.057 | 0.52 | 113.25 |
| 5      | Duzce, Turkey, 1999 7.1 Duzce | 0.51 | 84.23 | 1.344 | 0.69 | 113.21 |
|        | NF-no pulse record subset |                  |                    |                    |                             |                       |
| 6      | Imperial Valley-06, 1979 6.5 Bonds Corner | 0.78 | 46.75 | 1 | 0.78 | 46.75 |
| 7      | Imperial Valley-06, 1979 6.5 Chihuahua | 0.27 | 29.90 | 1.564 | 0.42 | 46.76 |
| 8      | Kocaeli, Turkey, 1999 7.5 Yarimca | 0.32 | 71.89 | 0.651 | 0.21 | 46.80 |
displacements in these floors. Therefore, by comparing Figs. 13 and 14, it can be seen that by using the dampers in the bottom two stories of the OM, the building will be able to withstand large drifts and displacements under severe ground motions.
6.2 Fragility and vulnerability results

After deriving the IDA curves, fragility analyses were performed based on the method that was introduced in Sect. 4. In this regard, the linear regression analysis was conducted on the IDA data of each model (see Fig. 16) to obtain the regression coefficients $a$ and $b$. Afterward, using Eq. (6) and Eq. (7), the values of $\sigma_X$ and $m_X$ were calculated, respectively and finally, the fragility curves were developed for each model at four damage states according to Fig. 17. In Eq. (4), the parameter $C$ was considered as the limit value of the inter-story drift ratio at each damage state mentioned in Table 1. As shown in Fig. 17, the fragility curves shift towards the right with increasing damage limit state and become flattered. It is also observed that for all damage states, the
Fig. 14  IDA curve for the UM subjected to far- and near-field earthquake records

Fig. 15  IDA curve for the UWM subjected to far- and near-field earthquake records
physical improvement of the seismic fragility of the OM becomes evident through adding NFVDs to it so that the fragility curves moved towards the right when plotted.

In order to assess the effect of NFVDs on the vulnerability of the UM and UWM, vulnerability analyses were conducted on each model based on Eqs. (8) and (9). Table 10 compares the structural vulnerability of the models at the four seismic hazard levels. Furthermore, Table 11 compares the vulnerability of non-structural components sensitive to acceleration and drift at the four seismic hazard levels.

According to Table 10, the structural vulnerability of the OM at the highest seismic hazard level (i.e. 2% probability of exceedance within 50 years) was calculated about 100% that means the building will be collapsed at this seismic scenario, as was seen that the main building of the Olive View Hospital was entirely demolished during the San Fernando earthquake. Moreover, by implementing the damper systems in the OM, the vulnerability of the UM was considerably improved so that the vulnerability at all
Concerning the spectral acceleration curves shown in Fig. 11, the increase in the effective damping ratio and the natural period ($T$) has led to a substantial decrease in the spectral acceleration and consequently had a great effect on the structural and the non-structural acceleration-sensitive vulnerability of the UWM.

As can be seen from Table 11, the non-structural vulnerability of the OM was significant, especially at high seismic hazard levels, and after retrofitting the model with NFVDs, damage to both non-structural components of the UM was greatly reduced. Although the increase in the natural period of the UWM has led to the increase in the spectral displacement of the UWM, the increase in the vulnerability of the non-structural drift-sensitive components was slight in comparison to the OM because of the increase in the effective damping ratio.

![Fig. 17 Fragility curves of all models](image)

**Table 10** Comparison of the structural vulnerability of the models at different seismic hazard levels

| Probability of exceedance in 50 years (%) | Vulnerability (%) |
|------------------------------------------|-------------------|
|                                           | OM ($T=0.63$ s & $\zeta_{eff}=5\%$) | UM ($T=0.63$ s & $\zeta_{eff}=35\%$) | UWM ($T=1.20$ s & $\zeta_{eff}=35\%$) |
| 20                                       | 49.48             | 10.00            | 1.88            |
| 10                                       | 49.78             | 10.01            | 2.03            |
| 5                                        | 76.82             | 41.84            | 9.90            |
| 2                                        | 99.95             | 50.03            | 10.71           |
Table 11  Comparison of the non-structural vulnerability of the models at different seismic hazard levels

| Probability of Exceedance in 50 Years (%) | Vulnerability (%) | Acceleration-sensitive | Drift-sensitive |
|------------------------------------------|-------------------|------------------------|-----------------|
|                                           |                   | OM ($T=0.63$ s & $\zeta_{eff}=5\%$) | UM ($T=0.63$ s & $\zeta_{eff}=35\%$) | UWM ($T=1.20$ s & $\zeta_{eff}=35\%$) | OM ($T=0.63$ s & $\zeta_{eff}=5\%$) | UM ($T=0.63$ s & $\zeta_{eff}=35\%$) | UWM ($T=1.20$ s & $\zeta_{eff}=35\%$) |
| 20                                       | 15.44             | 3.97                   | 0.80            | 2.54           | 0.53            | 2.55            |
| 10                                       | 16.39             | 4.39                   | 0.90            | 2.73           | 0.58            | 2.74            |
| 5                                        | 34.30             | 11.74                  | 3.10            | 6.68           | 1.89            | 6.81            |
| 2                                        | 49.59             | 20.58                  | 6.39            | 11.11          | 3.60            | 11.38           |
6.3 Seismic resilience assessment

After obtaining the structural and non-structural responses, based on the new calculation model of evaluating the seismic resilience developed in Sect. 4, the seismic resilience of three case models is assessed. To this end, the physical direct economic loss curves are derived according to Eqs. 14, 15 and 16 firstly. Figs. 18 and 19 show the structural loss curves for each model in terms of fragility and vulnerability analyses. According to Eq. (12), the total loss is comprised of the structural and non-structural losses that have been shown in Fig. 20.

Regarding four seismic hazard levels, the chart shown in Fig. 20 represents the structural and non-structural direct economic losses for the case models based on vulnerability and fragility analyses. According to Eq. (12), the total loss is comprised of the structural and non-structural losses that have been shown in Fig. 20.

As can be seen from Fig. 20, in all case models and all seismic hazard levels, the total loss obtained from fragility analyses was more than that of the vulnerability analyses. However, the significant effect of NFVDs on loss reduction (both structural and non-structural losses) is quite obvious, so that compared to the OM’s losses, the total
losses of the UM and UWM, even at high seismic hazard levels, has been considerably reduced.

Because there is no particular information as to how the recovery process is developing and given that the society has a medium preparedness in the deal of a seismic event, so the linear recovery function (see Eq. 17) is used in this study.

\[
F_{RE}(t, T_{RE}) = \left( 1 - \frac{t - t_{OE}}{T_{RE}} \right)
\]

In this study, according to (Cimellaro et al. 2010), the recovery time \(T_{RE}\) is chosen based on the location of the Olive View Hospital in Los Angeles County and is assumed the building is fully recovered to its initial functionality. Moreover, the recovery time at the 2%PE hazard level is adopted as the control time for seismic resilience quantification, which is 297 days. The recovery times related to the seismic hazard levels, as well as the seismic resilience index calculated by Eq. (10) are summarized in Table 12 corresponding to each model.

Based on fragility and vulnerability analyses, Figs. 21 and 22 illustrate the comparison of the functionality curves for the existing and the upgraded models by NFVFDs at four different hazard levels, respectively. Regarding the functionality curves, there is a drop with increasing intensity of the earthquake due to the increasing losses as well as the time of recovery, as expected.

The seismic resilience indices indicate that at low hazard levels (e.g. the 10 and 20%PE hazard levels), the seismic resilience of all UM and UWM is very high based on both fragility and vulnerability analyses. What’s more, the structural components of the upgraded models experience no damage and the damage of nonstructural
components is very small compared to the OM. At these seismic hazard levels, the upgraded models can be reoccupied immediately. At high hazard levels (e.g. the 5 and 2%PE hazard levels), the damage of the OM tends to manifest as a failure of the nonstructural components especially the acceleration-sensitive components, and the effect of nonstructural components on system functionality is larger than that of structural components (see Fig. 20). However, the NFVDs have a significant effect on the improvement of the seismic resilience of the upgraded models.

Table 12  Seismic resilience index and recovery time corresponding to the seismic hazard levels

| Probability of Exceedance in 50 Years (%) | Recovery Time (days) | Seismic Resilience Index (%) |
|------------------------------------------|----------------------|------------------------------|
|                                          |                      | OM  | UM  | UWM | OM  | UM  | UWM |
| 20                                       | 71                   | 92.88 | 98.76 | 99.00  | 97.46 | 99.80 | 99.73 |
| 10                                       | 94                   | 92.18 | 98.62 | 98.90  | 97.03 | 99.76 | 99.69 |
| 5                                        | 228                  | 77.71 | 95.65 | 96.13  | 84.27 | 98.76 | 98.59 |
| 2                                        | 297                  | 58.88 | 90.75 | 93.06  | 64.18 | 95.46 | 96.75 |

Fig. 21  Comparison of functionality curves based on fragility analyses for (a) OM (b) UM (c) UWM

Fig. 22  Comparison of functionality curves based on vulnerability analyses for (a) OM (b) UM (c) UWM
7 Final remarks and conclusions

Generally, assessment of the seismic resilience of hospital buildings using new retrofitting strategies helps the community to limit the impact of business disruption after earthquake events and accelerate decisions and restore them fast. In this paper, the conventional quantitative evaluation of seismic resilience for a single building in terms of fragility analyses was reviewed and then a new calculation model of seismic resilience based on vulnerability analyses was developed. At different seismic hazard levels, the mentioned calculation models of seismic resilience were applied to a typical RC hospital building, located in a high seismic zone, and two different retrofitted models with NFVDs. For this purpose, firstly the fragility analyses of the OM and the upgraded models were derived through the IDAs under a set of far and near-field earthquake records. After that, the developed metric based on the vulnerability analyses were used to determine the potential direct economic losses and finally evaluate the effectiveness of the upgrading strategies on the functionality and seismic resilience.

The results showed that NFVDs had a significant effect on reducing both structural and non-structural damages and consequently improving the functionality and the seismic resilience of the upgraded models. It is also observed that there is a difference in the losses obtained from the fragility and vulnerability analyses at all seismic hazard levels so that the losses calculated by the fragility analyses were larger than that of the vulnerability analyses for all case models. Moreover, the computed analytical functionality curves corresponding to all hazard levels appear to make intuitive sense relative to RC hospital building’s design, upgrade, and performance in future earthquakes. In other words, the simulated loss curves based on vulnerability analyses result in reducing both structural and nonstructural damages and an excellent improvement in the seismic resilience of the building compared to the seismic resilience obtained from fragility analyses.

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Declarations

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