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FRAGILITY CURVES FOR CONCRETE FRAME BUILDINGS WITH PASSIVE CONTROLLERS

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ABSTRACT

Fragility curves present the probability of a structure exceeding a certain limit state in terms of a seismic intensity parameter function. The seismic demand varies from very low to maximum intensity. The fragility curves are developed from the adjustment of the function distribution of the results obtained. This article presents a set of fragility curves for concrete frame buildings of mid rise and high rise elevations based on the HAZUS classification in order to obtain the level of damage of a structure. The frames have the principal characteristic of having installed passive controllers of variable orifice in each level. Different dampers configurations were considered in the analysis, such as Diagonal, Chevron, Lower Toggle, among others. To develop the fragility curves the maximum drift of the top floor was obtained for each frame based on a nonlinear time history analysis using records of four earthquakes, i.e., El Centro, San Fernando, Loma Prieta, and an earthquake artificially generated. The earthquakes used were escalated from 0.1g to 1.5g with of an increase of 0.1g based on their maximum PGA. To simulate the nonlinearity of the structure the Bouc-Wen model was used. Finally, the curves illustrate the advantages of using dampers to reduce the level of damage in a structure, in which the results demonstrate that the most efficient configurations are the Lower Toggle and the Scissor-Jack.

Key words: Fragility Curves, Damping, Concrete Framed Buildings, Nonlinear Dynamic Analysis, Bouc-Wen Model.

Introduction

During the last two decades many researchers have develop investigations in order to find different types of measures, devices, and mechanisms to reduce the response of the structural system subjected to dynamic loads, e.g., earthquakes. Due to their efforts several passive, semi-active, and active devices have been developed. Throughout these investigations, different configurations, such as Diagonal, Chevron, Lower Toggle, among others, have been studied in order to determine the most effective passive dampers configuration that can be implemented (Constantinou et al 2001). On the other hand, variable orifice dampers have been proposed by Symans and Constantinou (1997), and Kurata et al. (1999, 2000). Research on fragility curves have been developed by Mieses (2007), Taylor and Dike (2007), Arjomandi et al (2009), Cortez (2006), Cundumi and Laboy (2009), and among others.

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Based on the aforementioned research, it has been found that design procedures lack the ability to relate the structural state limits, and their occurrence probability. Throughout the last few years, fragility curves have been widely used to forecast the probability of a structure exceeding a certain limit state in terms of a seismic intensity parameter function. Fragility curves can be developed empirically or analytically. FEMA 273 defines the four analytical procedures accepted that predict the performance of a structure as, linear static, linear dynamic, nonlinear static and dynamic nonlinear, identifying the nonlinear dynamic analysis as the most accurate procedure.

Upon the lack of existence of this type of curves for certain structures, this paper presents fragility curves for typical concrete framed buildings (beams and columns) within the range based on their period, i.e., frames subjected to period less than 1 or greater than 1. These frames have the characteristic of having additional damping other than the one commonly used. The damping added to the structure is generated by passive energy dissipating, commonly known as passive controllers.

The frames used were subjected to acceleration at the base by using artificially generated and historical earthquakes. To obtain the maximum drift for each of the frames a program utilizing Mathlab applications was developed. The program performs a nonlinear dynamic analysis based on the Bouc-Wen Model theory in order to capture the nonlinearity of the material. The amount of damage was quantified by using HAZUS parameter, where the damage index was obtained based on the ratio between the maximum displacement and the story height.

Finally, a series of fragility curves are presented for each group analyzed in order to represent the seismic vulnerability in terms of probability. Fragility curves were developed for each damaged state as defined by FEMA, i.e., no damage, slight, moderate, extensive, and complete damage. Developing new fragility curves for different types of structures will facilitate engineers in the design process, and in the assessment of the structural system.

Methodology

A nonlinear time history analysis was realized for the structures. There are different methodologies to model the nonlinearity of a structure. One of these methodologies is the constitutive model that was proposed by Wen (1976). The Bouc-Wen model is extremely versatile and can exhibit a wide variety of the hysteretic behavior. Wen proposed to define the behavior of the material as two elements in parallel, in which one element has elastic behavior and the other an inelastic behavior. The hysteretic behavior is included through a nonlinear relationship between $v(x)$ and x. $v(x)$ is an auxiliary displacement dependent on x and associated with the inelastic behavior. For this, Wen proposed the use of Bouc's endochronic law (Bouc, 1971) as described in equation 1:

$$\dot{v}(x) = \lambda\dot{x} - \beta\dot{x}\left|v(x)\right|^{\eta} - \gamma v(x)\left|\dot{x}\right|\left|v(x)\right|^{\eta-1} \quad (1)$$

where λ, β, γ are positive numbers and η is a odd number. By adjusting the parameters λ, β, and γ, the linearity in the unloading and the smoothness of the transition from the pre-yield to the post-yield region can be control. The equations of motion and the constitutive law are presented in equation 2:

$$\begin{cases} [M]_{nxn}\{\ddot{x}(t)\}+([C_s]+[C_1])_{nxn}\{\dot{x}(t)\}+\alpha[K]_{nxn}\{x(t)\}+(1-\alpha)[K]_{nxn}\,v(x)=-[M]\{r\}\ddot{x}_g(t) \\ \dot{v}(x)=\lambda\dot{x}-\beta\dot{x}|v(x)|^{\eta}-\gamma v(x)|\dot{x}||v(x)|^{\eta-1} \end{cases} \tag{2}$$

The matrix $[C_1]$ is the damping added by the damper. Using these equations, the controlled response of the Bouc-Wen hysteretic structural system is computed. The numerical response was calculated by means of a computer program that was developed in MATLAB using the *ode* subroutines available in this platform to numerically integrate the equations. The parameters for the model are $\lambda=1$, $\beta=0.3$, $\gamma=0.5$, and $\alpha=0.2$, α is the weighting constant ($0 < \alpha < 1$) representing the relative participation of the elastic and inelastic terms.

For the analysis, thirty concrete frame buildings were selected and divided into two groups based on the classification by Hazus (see Table 2), i.e., (1) Mid Rise Building (all building selected with a period less than 1, and (2) High Rise Building (with period greater than 1). The uncontrolled and controlled response are computed for the El Centro (Imperial Valley - May 1940) earthquake with a peak ground acceleration (PGA) of 0.348g, the San Fernando earthquake of February 9, 1971 with a PGA =1.226g, and the Loma Prieta earthquake of October 17, 1984 with a PGA = 0.799g. In addition, an artificial earthquake was generated by using the SIMQKE program. The earthquake is compatible with the design spectra of the Uniform Building Code (UBC) for a soil type of Sc, with a PGA =0.3g. The earthquakes used were escalated from 0.1g to 1.5g with of an increase of 0.1g based on their maximum PGA. Figure 1 shows the acceleration of the artificial earthquake.

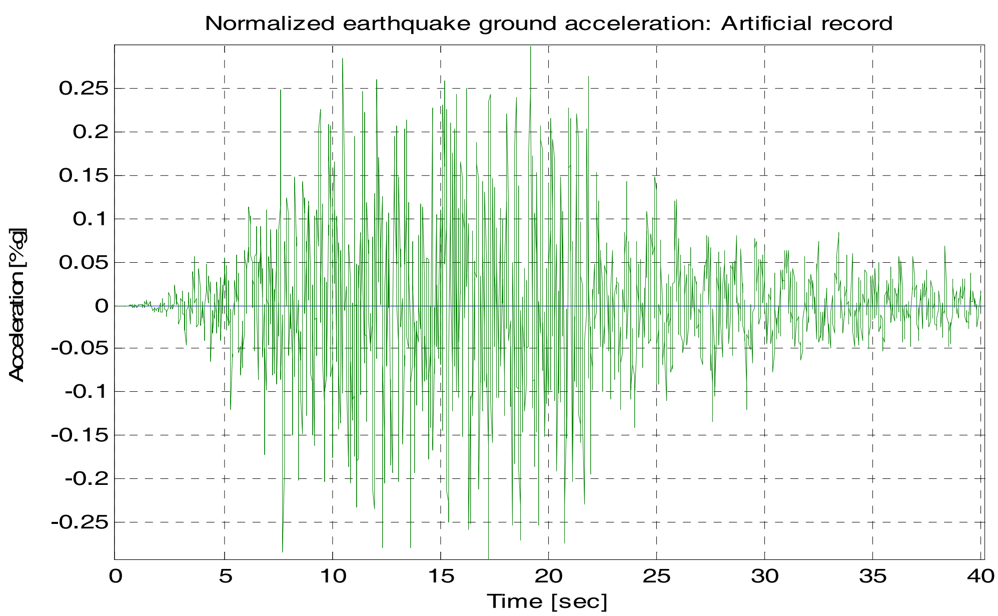

Figure 1. Ground acceleration of the Artificial record.

To obtain the maximum drift for each one of the frames a program utilizing Mathlab applications was developed for the uncontrolled model, and for each of the passive controller configurations considered for the analysis, e.g., diagonal, chevron, lower toggle, among others (see Figure 2). The damping added was limited to a maximum of 30% as established by FEMA 273 as the proportional ratio.

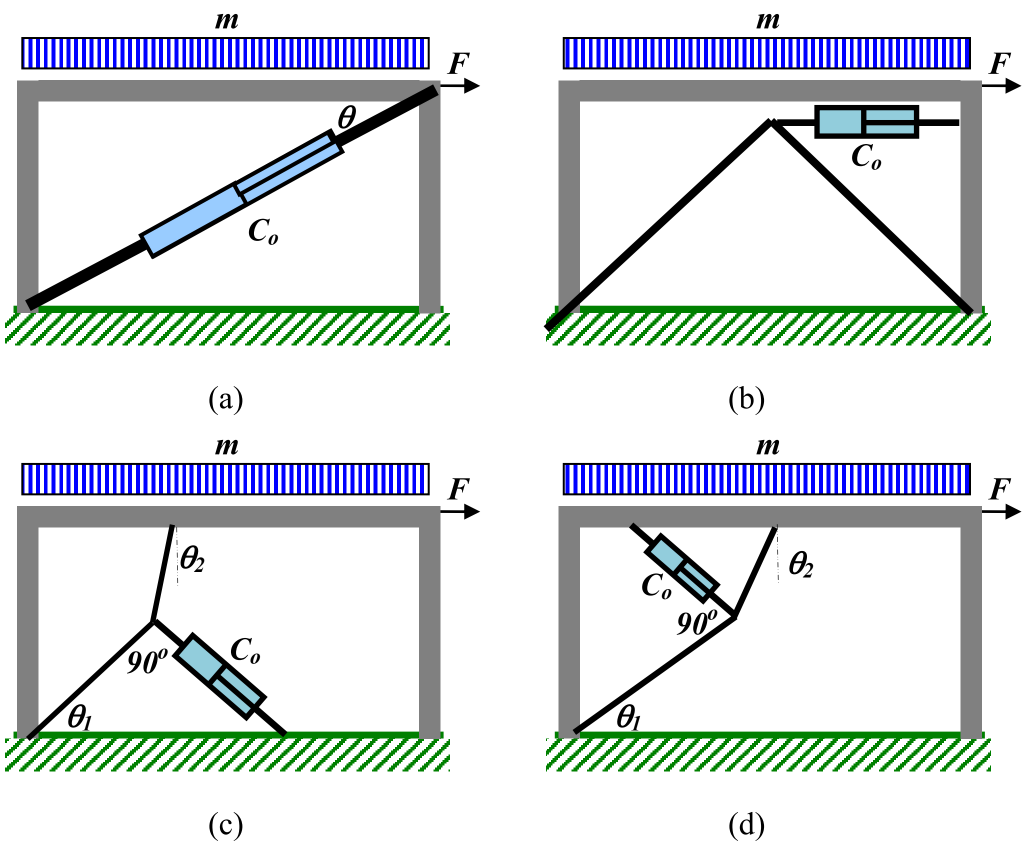

Figure 2. Damper Configurations: (a) Diagonal, (b) Chevron, (c) Lower Toggle, and (d) Upper Toggle.

F is the force exerted on the frame ($F=f \times F_D$), where F_D is the force along the axis of the damper, and *f* is the magnification factor. The magnification factors are presented in Table 1.

Table 1. Factor de Magnification for the configurations.

Configurations	Expressions
Diagonal	$f = \cos\theta$
Chevron	$f = 1.00$
Lower-Toggle	$f = \dfrac{\sin\theta_2}{\cos(\theta_1+\theta_2)}$
Upper-Toggle	$f = \dfrac{\sin\theta_2}{\cos(\theta_1+\theta_2)} + \sin\theta_1$

The damage index was calculated using the drift ratio which is the most practical index as presented Eq.3.

$$DI_{Drift} = \frac{\Delta m}{H} \tag{3}$$

where Δm is the drift, and H is the story height.

After determining the damage index, the amount of damage was quantified using the parameters established by HAZUS for the pre-code design level as presented in Table 2.

Table 2. Hazus Interstory Drift Ratio of Structural Damage States for Pre-Code Design Level.

STRUCTURAL DAMAGE STATES			
Slight	**Moderate**	**Extensive**	**Complete**
Mid Rise Buildings (4-7 levels)			
0.0027	0.004	0.011	0.027
High Rise Buildings (8+ levels)			
0.002	0.003	0.008	0.02

When a damage index is categorized into a damage state a value of 1 is assigned; otherwise a value of 0 is assigned in order to indicate that the index has not reached the specific damage state. Once the damage index is categorized for each frame, the values are added for each earthquake individually. The cumulative values are used to calculate a probability of occurrence for each damage states.

To represent the data a two parameter log normal distribution was used as described mathematically by Eq. 4. To obtain the two parameters that involve the log-normal distribution, i.e., μ, σ, the optimization code was performed by using the Microsoft Excel solver tool in order to find the best distribution possible. This process was repeated for each damage state to develop a fragility curve for each case.

$$f(x/\mu,\sigma) = \frac{1}{\sigma\sqrt{2\pi}} \int_0^x \frac{e^{-(\ln(x)-\mu)^2}}{t} dt \tag{4}$$

where x is the value at which the function is evaluated, μ is the PGA median value, and σ is the log standard deviation.

Fragility Curves for Mid Rises Buildings (T<1)

Fifteen frames were considered under this category in order to create fragility curves for the uncontrolled structures and for each case considered. The fragility curves were developed following the methodology described previously. The procedure which defines the fragility curves for all the cases considered can be found in the report prepared by Cundumi and Laboy, in which a detail summary of the results is presented. In this section only some of the results are reported.

Figures 3 to 7 present the fragility curve for the uncontrolled, diagonal, chevron, lower toggle, and scissor jack cases, respectively for the San Fernando and the Loma Prieta earthquake. These curves indicate significant damage reduction when the structure is controlled with passive dampers of variable orifice. Among the configurations considered it can be noticed that the best configurations are the Lower Toggle and Scissor-Jack since significant reduction was identified for all the earthquakes.

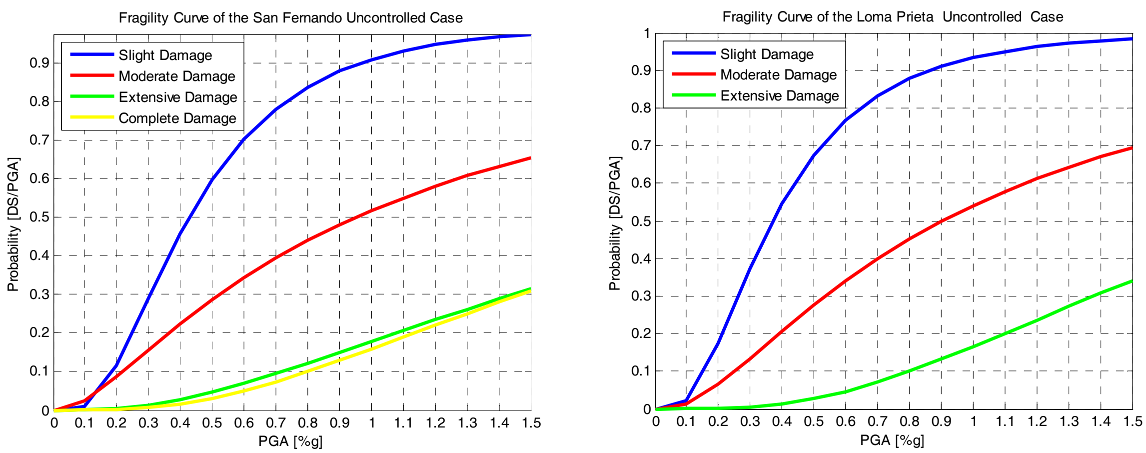

Figure 3. Fragility Curve for the San Fernando and Loma Prieta record - Uncontrolled Case.

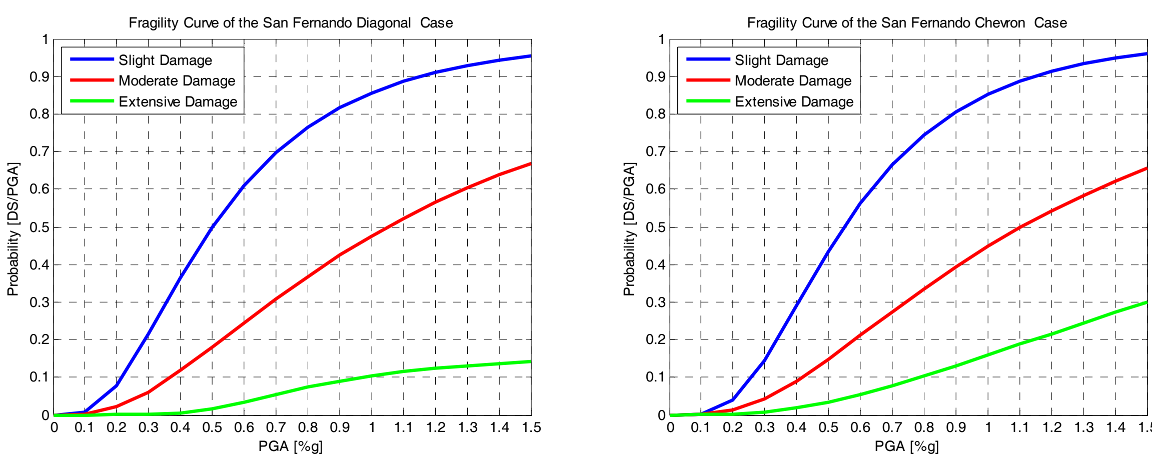

Figure 4. Fragility Curve for the San Fernando record – Diagonal and Chevron Cases.

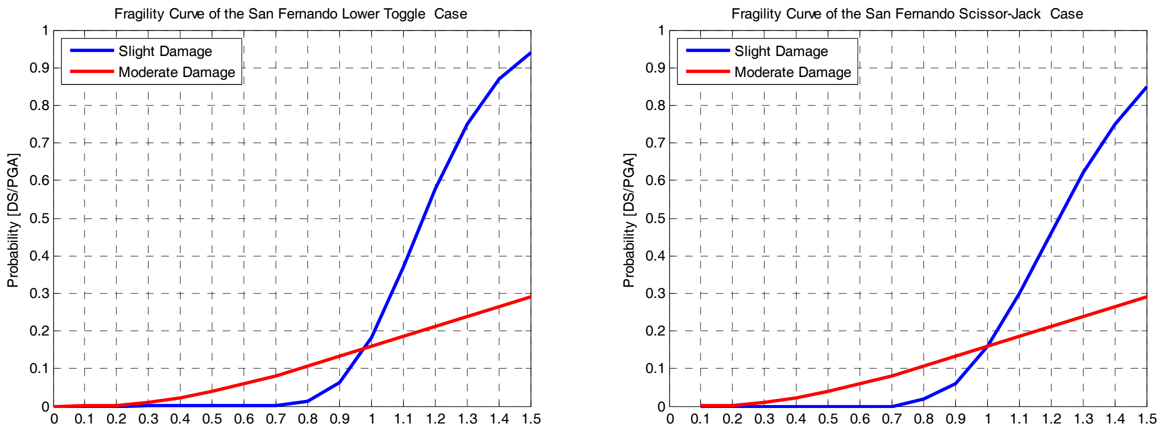

Figure 5. Fragility Curve for the San Fernando record - Lower Toggle, and Scissor-Jack Cases.

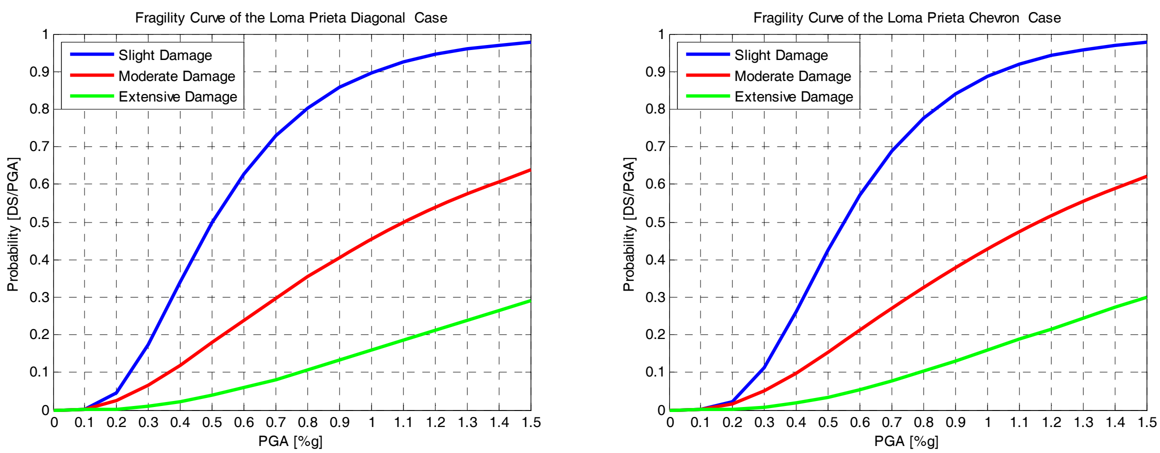

Figure 6. Fragility Curve for the Loma Prieta record – Diagonal and Chevron Cases.

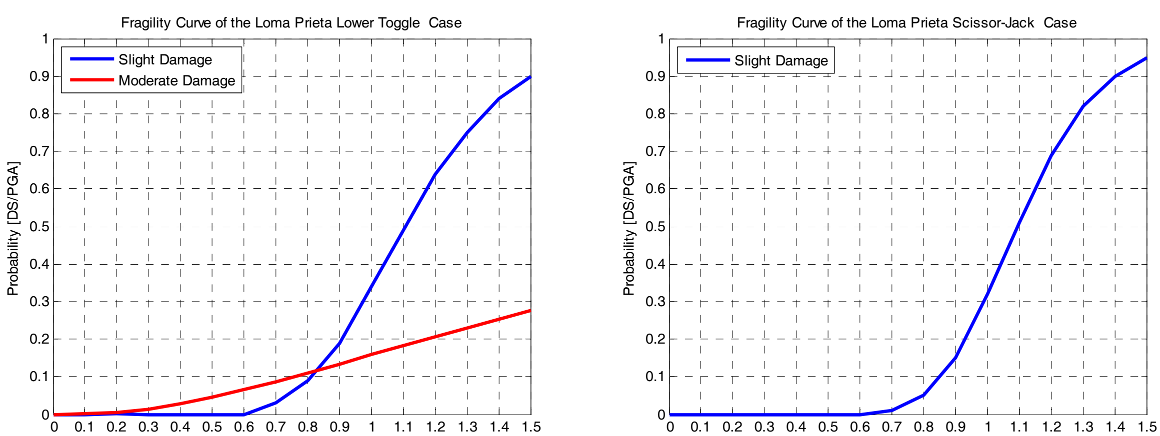

Figure 7. Fragility Curve for the Loma Prieta record - Toggle and Scissor-Jack Cases.

Fragility Curves for High Rise Buildings (T>1)

Fifteen frames were also considered under this category. Figures 8 through 12 display the last set of results of the fragility curve for the uncontrolled, diagonal, chevron, lower toggle, and scissor jack cases, respectively for the El Centro and the Artificial earthquake.

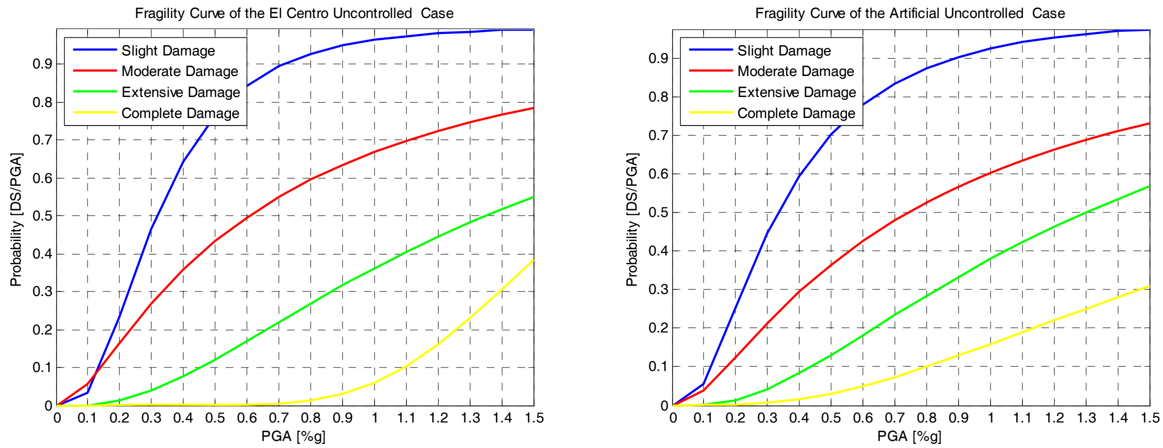

Figure 8. Fragility Curve for the El Centro and Artificial record - Uncontrolled Case.

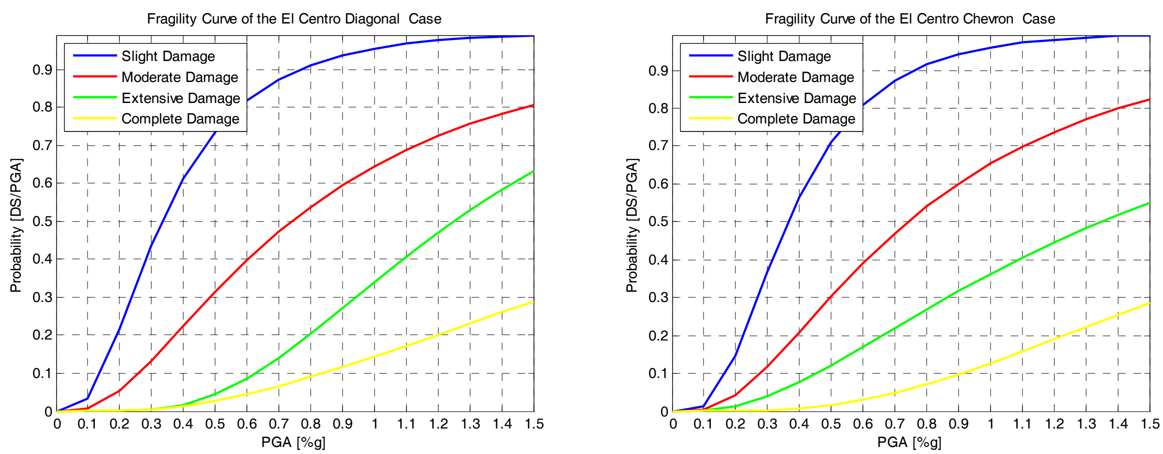

Figure 9. Fragility Curve for the El Centro record – Diagonal and Chevron Cases.

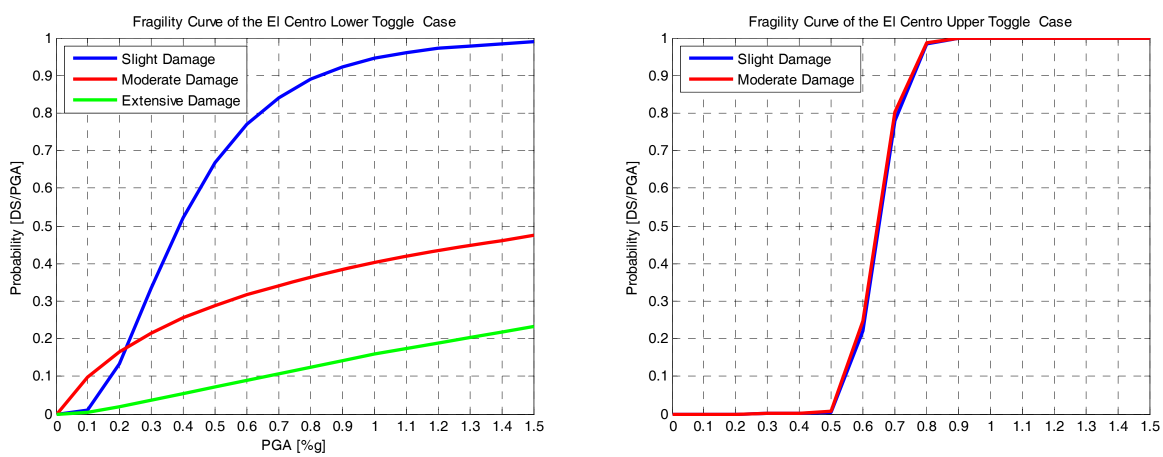

Figure 10. Fragility Curve for the El Centro record - Lower Toggle and Scissor-Jack Cases.

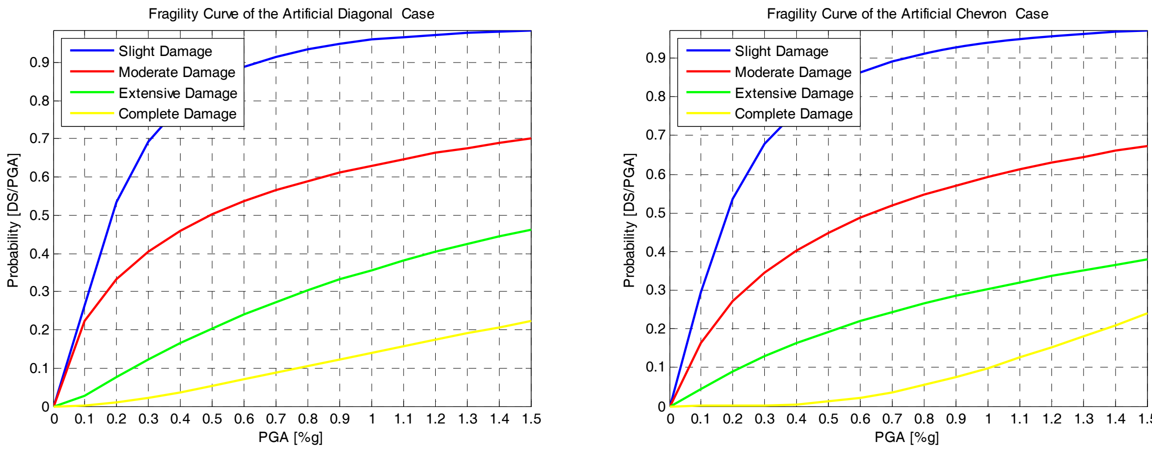

Figure 11. Fragility Curve for the Artificial record – Diagonal and Chevron Cases.

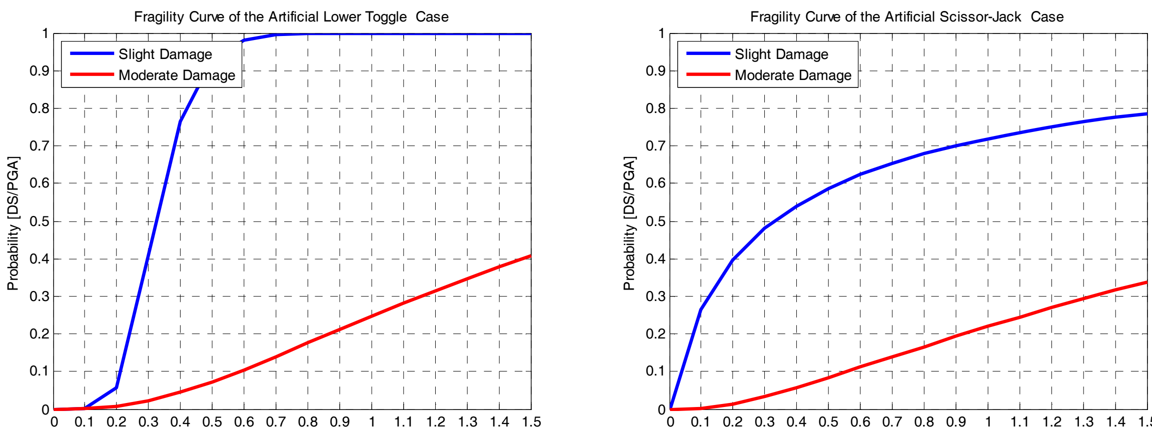

Figure 12. Fragility Curve for the Artificial record - Lower Toggle and Scissor-Jack Cases.

Conclusions

This paper presented the development of fragility curves for concrete frame buildings with passive controllers which will facilitate engineers in the design process, and in the assessment of the structural system. For the analysis the frames used were from existing construction and some were generated. The results obtained indicate that the best configurations are the Lower Toggle and Scissor-Jack since a significant reduction was noticed. Further studies are recommended in order to develop fragility curves considering the uncertainties that the damper position might have since this paper only considered the installation of the dampers in each floor. It is also recommended to perform an analysis varying the angles of the configuration.

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