Probabilistic Models for Critical Building Responses of High-rise Building

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Abstract

Probabilistic performance-based design and assessment of structures take into account the uncertainty in the estimation of seismic hazard, structural reponse (as a function of the ground motion intensity level), and structural capacity. The objective of this study is to develop statistical models for critical building responses (such as roof drift, roof acceleration, base shear, etc) which might help to develop and/or assess guidelines for seismic design of high-rise buildings.

One 64-story diagrid high-rise building is selected for this study. A total of 435 non-linear time-history analyses are conducted in OpenSEES utilizing 145 recorded earthquake motion from various magnitude and distance bins. The results indicated that roof drift ratio correlates well to spectral acceleration at the 1st mode period, roof acceleration correlates better to PGA, base shear correlates better to spectral acceleration in the 2nd mode period and base moment correlates more to spectral acceleration at the 1st mode structural period. Further result shows that, if a structure with a fundamental period of 5.0 second is designed for LA area then a 0.6% roof drift will have a probability of exceedance of 10% for a 100-year lifetime. And base shear of 0.25W (W is the seismic weight of the structure) has an annual rate of exceedance of 3.2e-3 or a return period of roughly 300 years.

1. Introduction

The objectives of this paper are: (i) To conduct a very large number of nonlinear dynamic analyses of tall building utilizing ground motion selected from various magnitude and distance bins; (ii) To characterize key building responses to these ground motions; (iii) To develop statistical models for these critical building responses which might help to develop and/or assess guidelines for seismic design of high-rise buildings. For example, the kind of answer sought are:

- What is the annual rate (probability) that the roof drift ratio will exceed 1%?
- What should be the median roof drift ratio if one is designing the structure for a life time of 75 years?

The work of this chapter is motivated by the work of the PEER Tall Building Initiative [2] where studies were performed with similar objectives for several concrete high-rise buildings. Section 2 describes the theoretical foundation for the derivation of a closed-form expression for the mean annual frequency of exceeding specified limit state. Section 3 presents the results for hundred of non-linear time history analyses (NLTHA) and develops statistical models for critical building responses.

2. Theoretical Foundation for Developing Statistical Models

The theoretical development of this section is based on the methodology developed by Jalayer and Cornell [1]. The probabilistic foundation developed in this section involves the derivation of a closed-form expression for the mean annual frequency of exceeding a specified limit state. The term “limit state frequency” will be used from now on for “the mean annual frequency of exceeding a specified limit state”.

2.1 Limit State Frequency H_{LS}, and General Solution Strategy

H_{LS} is defined as the product of the mean rate of occurrence of events with seismic intensity larger than a certain “minimum” level, ν, and the probability that demand D exceeds capacity C, when such an event occurs.

$$H_{LS} = \nu . P[D > C]$$

In order to determine H_{LS}, the strategy is to decompose the problem into more tractable pieces and then re-assemble it. First, a ground motion intensity measure IM (such as the spectral acceleration, S_a, at the 1st mode structural period) is introduced because (a) the level of ground motion is the major determinant of the demand D, and (b) this permits to separate the problem into a seismological part and a structural engineering part. To do this, a standard tool in applied probability theory, known as the “total probability theorem” (TPT), is used. This theorem permits the following decomposition of the expression for limit state frequency with respect to an interface variable (here, the spectral acceleration):

$$H_{LS} = \nu . P[D > C] = \nu . \sum_{\text{all x}} P[D > C \mid S_a = x] . P[S_a = x] \tag{1}$$

In simple terms, the problem of calculating the limit state frequency has been decomposed into two problems. The first problem is to calculate the term P[Sa = X] or the likelihood that the spectral acceleration will equal a specified level, x. This likelihood (together with ν) is a number we can get from a probabilistic seismic hazard analysis (PSHA) of the site. The second problem is to estimate the term P[D>C|Sa = x] or the conditional limit state probability for a given level of ground motion intensity, here represented by, Sa = x.

2.2 Ground Motion Intensity Measure: Specral Acceleration Hazard

The hazard corresponding to a specific value of the ground motion intensity measure (here spectral acceleration S_a) is defined as the mean annual frequency that the intensity of future ground motion events are greater than or equal to that specific value x and denoted by $H_{Sa}(x)$. The spectral acceleration hazard referred to as $H_{Sa}(x)$ can be defined as the product of the rate parameter ν (defined in Section 2.1) and the probability of exceeding the spectral acceleration value, x, denoted by $G_{Sa}(x)$:

$$H_{S_a}(x) = \nu . G_{S_a}(x)$$

The spectral acceleration hazard values $H_{Sa}(x)$ are usually plotted against different spectral acceleration values, x; this results in a curve that is usually referred to as a spectral acceleration hazard curve. It is advantageous to approximate such a curve in the region of interest by a power-law relationship [3]:

$$H_{S_a}(S_a) = P[S_a \geq x] = k_0 . x^{-k} \qquad (3)$$

where k_0 and k are parameters defining the shape of the hazard curve. Figure 1 shows a typical hazard curve for a southern California site that corresponds to a period of 1.8 seconds. As it can be seen from the figure, a line with slope k and intercept k_0 is fit to the hazard curve (on the two-way logarithmic paper) around the region of interest (e.g., mean annual frequencies between 1/475 or 10% frequency of exceedance in 50 years, and 1/2475 or 2% frequency of exceedance).

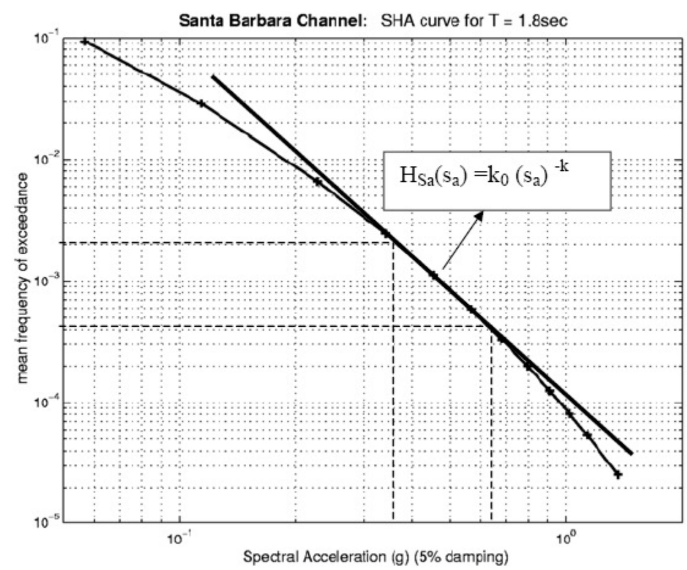


Figure 1: A typical hazard curve for spectral acceleration corresponding to a structural fundamental period of 1.8 seconds [1]

2.3 Median Relationship between Specral Acceleration and Roof Drift Demand

Observations of demand values are normally obtained from the result of structural time history analyses performed for various ground motion intensity levels. Figure 2 shows such results, e.g. maximum roof drift, D versus S_a. This figure shows data points from the analyses described later in this paper. For a given level of ground motion intensity, there will

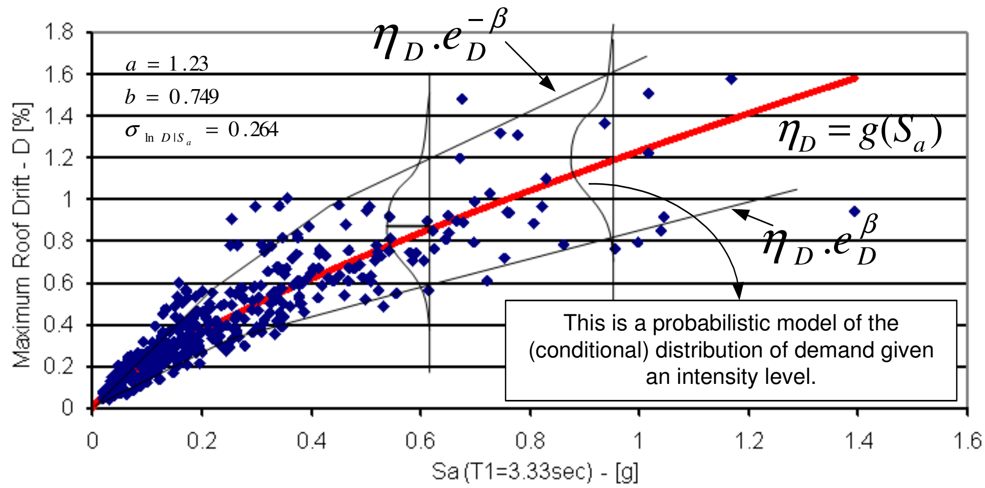


Figure 2: A set of spectral acceleration and demand data pairs and the regression model fit to these data points

be variability in the demand results over any suite of ground motion records applied to the structure. It is convenient to introduce a functional relationship between the ground motion intensity measure and a central value, specifically the median η_D of the demand parameter based on the data available from such time history analyses. In general, for a spectral acceleration equal to x, the functional relationship will be:

$$\eta_D(x) = g(x)$$

This is called the conditional median of D given Sa (more formally denoted by $\eta_{D|S_a}(x)$, but the simpler notation shall kept). A full conditional probabilistic model can be constructed with the variability displayed in Figure 2 by writing:

$$D = \eta_D(x).\varepsilon = g(x).\varepsilon$$

where ε is a random variable with a median equal to unity and a probability distribution to be lognormal. Linear regression is used in logarithmic space (i.e, $\ln \eta_D(x_a) = \ln a + b \ln x$). Such regression will result in the following relationship between spectral acceleration and (median) roof drift response:

$$\eta_D(x) = a.x_a^b \tag{4}$$

2.4 Mean Annual Frequency of Exceeding Demand

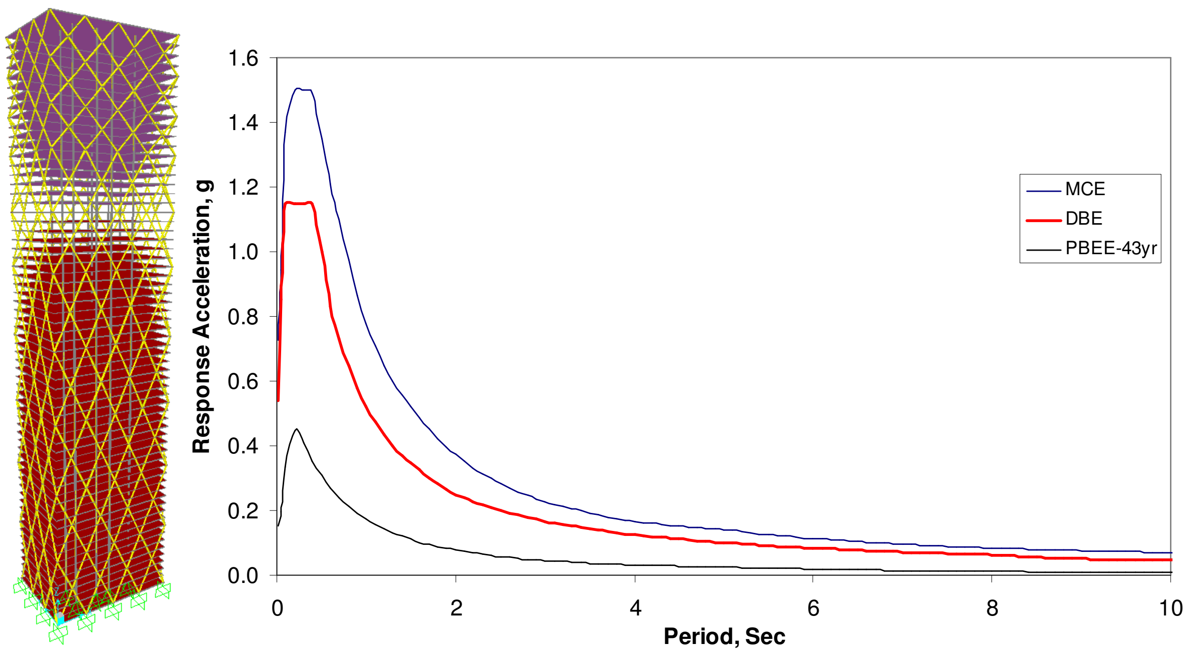
A closed-form expression for the mean annual frequency of exceeding a certain demand value d, also known as the "drift hazard, $H_D(d)$", was derived by Jalayer and Cornell [1]:

$$H_D(d) = k_0 \left(\frac{d}{a}\right)^{\frac{-k}{b}} . e^{\frac{1}{2}.\frac{k^2}{b^2}.\beta_{D|S_a}^2}$$

where all the variables are defined in Section 2.1 to 2.3.

3. Development of Statistical Models for Critical Building Responses

Statistical models for different critical building responses such as roof drift, roof acceleration, base shear will be developed based on the theory presented in Section 2 using the 64-story structure shown in Figure 3a. One hundred forty five ground motions are used for this purpose. These ground motions were provided by a research team from the PEER Center at University of California Berkeley [5]. Three-component input ground motions are used in the 3D non-linear time history analyses (NLTHA). Ground motion scaling factors of 1, 2 and 4 are used for all the motions; this mean that a total of 435 NLTHA are conducted in OpenSees for the 64-story diagrid building.



(a) (b)

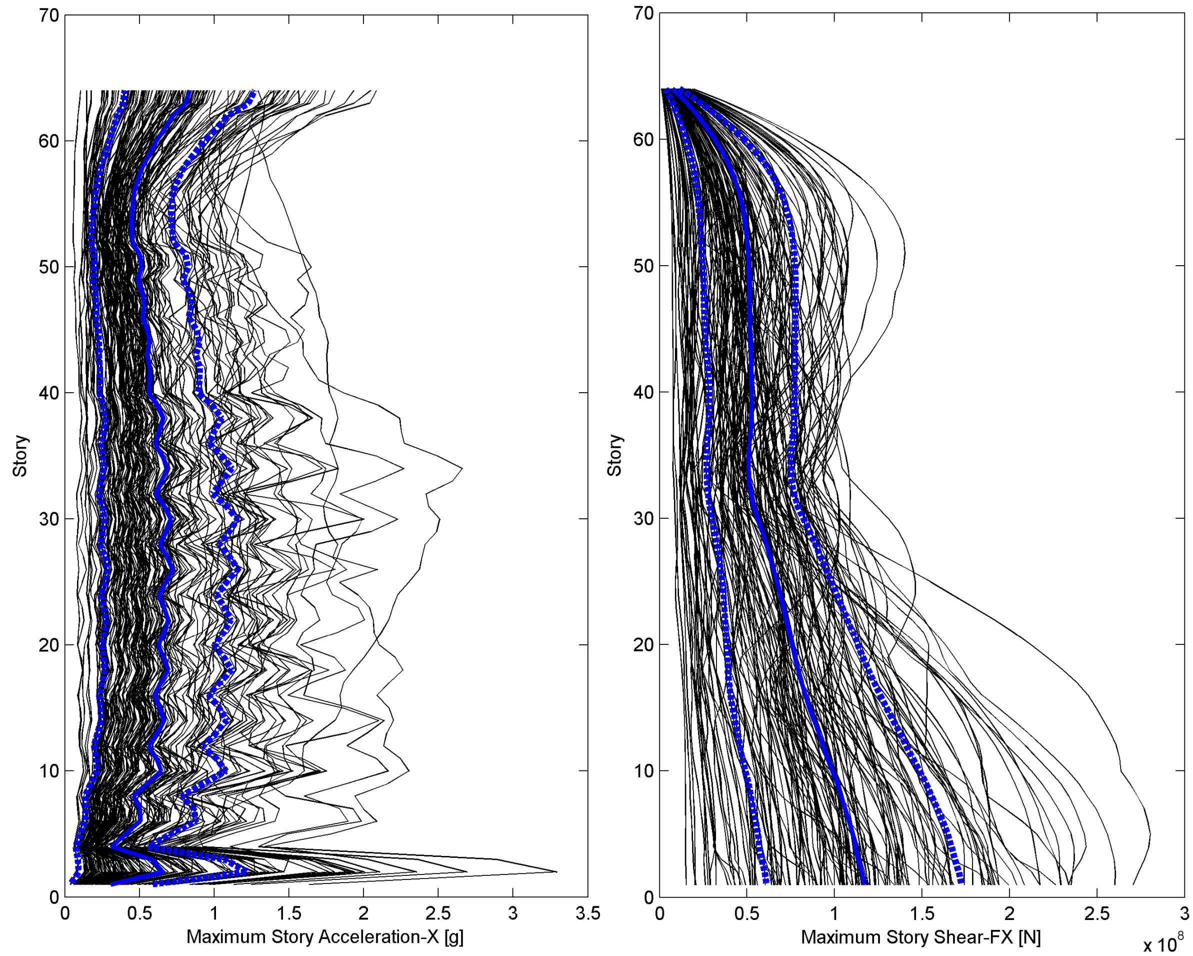
Figure 3: (a) 3-D view of the 64-story structure; (b) Response spectrum used in this study

3.1 Variation in the Structural Responses

Variations in the structural responses are presented in Figure 4. In Figures 4a and 4b, responses are shown only for the ground motion with scaling factor (SF) of one. A large variability in the responses is observed from Figures 4a and 4b, which means that the structural responses are very sensitive to the ground motions. Higher mode effects are visually noticeable from these figures. Mean response and mean $\pm$ one standard deviations are also shown in the Figures 4a and 4b. A particular observation can be made from Figure 4a, where maximum story accelerations are plotted. It is quite interesting to see that story accelerations are high throughout the height of the building in the case of high-rise building.

3.2 Effect of the Scaling Factor

Figure 5 illustrate the effects of the scaling factor. The mean of the responses for each



(a) (b)

Figure 4: (a) Variation in Story Acceleration; (b) Variation in Story Shear; [motions with SF=1 are shown only]; mean and mean ± std are also shown

scaling factor are shown in those figures. These figures shows the relative magnification in the responses when the input ground motions are scaled by a factor of 1, 2 and 4.

3.3 Correlation between EDP and Ground Motion Parameters

It is necessary to find out the appropriate ground motion parameter which correlates better with the Engineering Design Parameter (EDP) of interest such as roof drift, roof acceleration, base shear or other. Let's first consider maximum roof drift in the X-direction. For each NLTHA, one value of absolute maximum roof displacement in X-direction from the time series and ground motion parameters for that particular input motion in X-direction are calculated. Figure 6 presents a plot of maximum roof drift in X-direction vs Spectral acceleration at the 1st mode structural period in X-direction (T_{1X} = 3.33 sec). Figure 7a shows a plot of maximum roof drift in the X-direction vs Spectral acceleration in the 2nd mode structural period in X-direction (T_{2X} = 0.91 sec). Similarly, plots of roof drift vs other ground motion parameters are presented elsewhere [4]. After a close observation it is evident that

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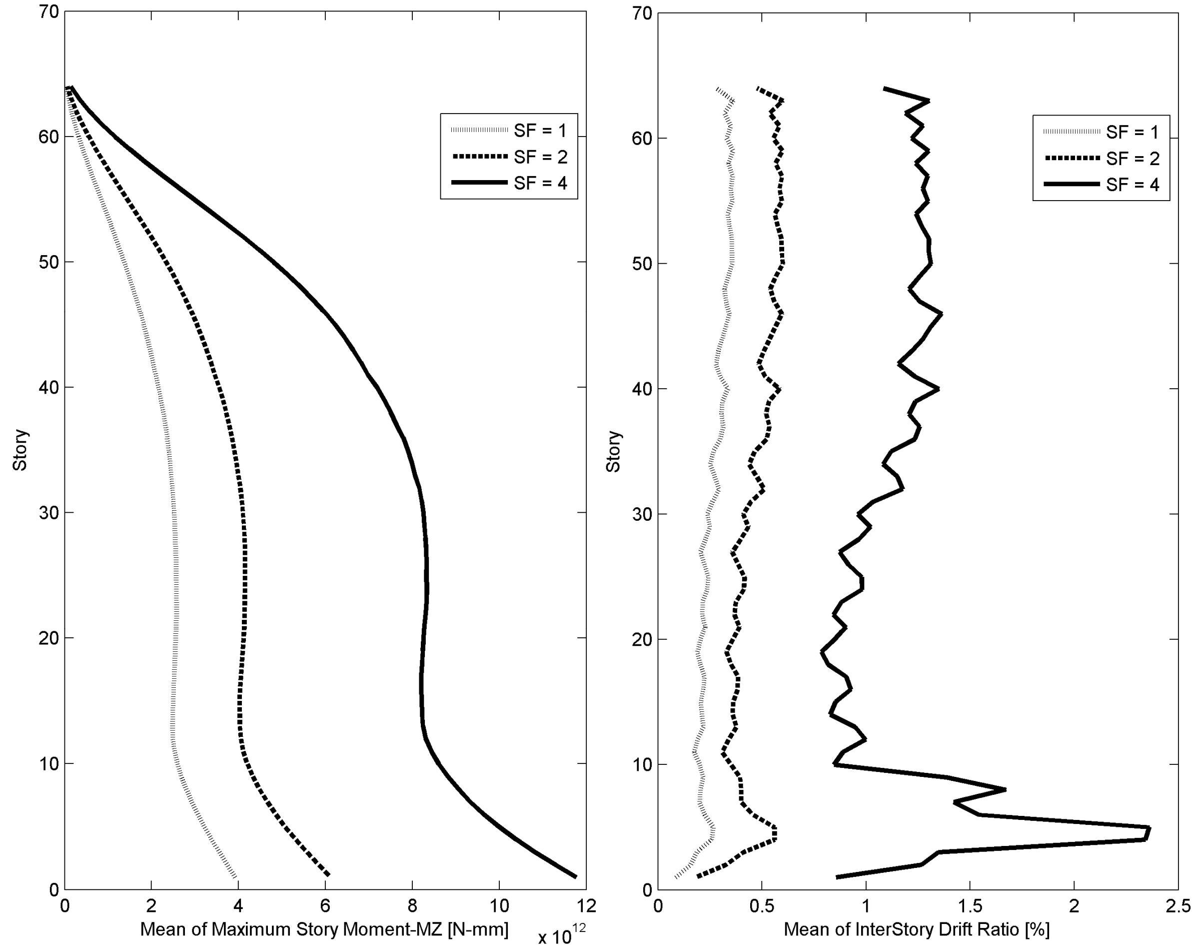

(a) (b)

Figure 5: Effects of the Scaling Factor in (a) Story Moment (b) Inter-story drift ratio – mean of the responses for each SF are shown

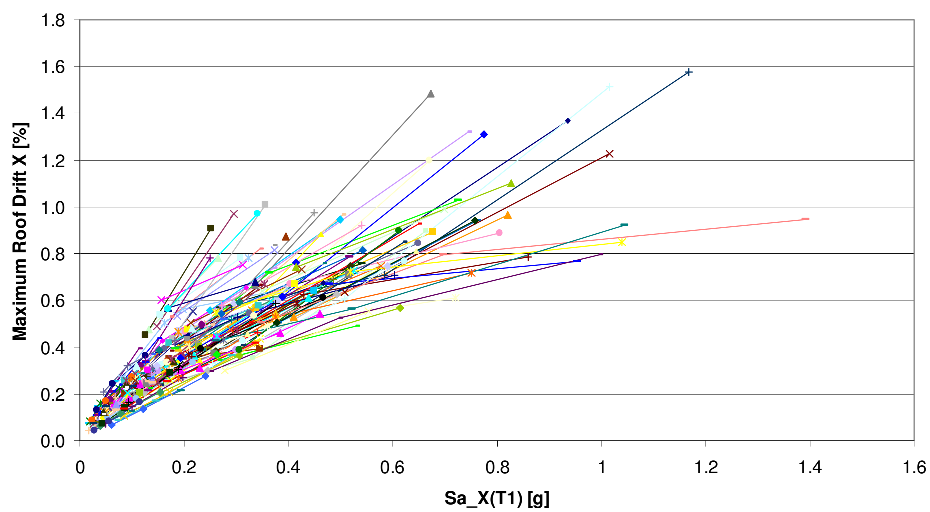

Figure 6: Roof drift ratio vs Spectral acceleration at the 1st Mode structural period

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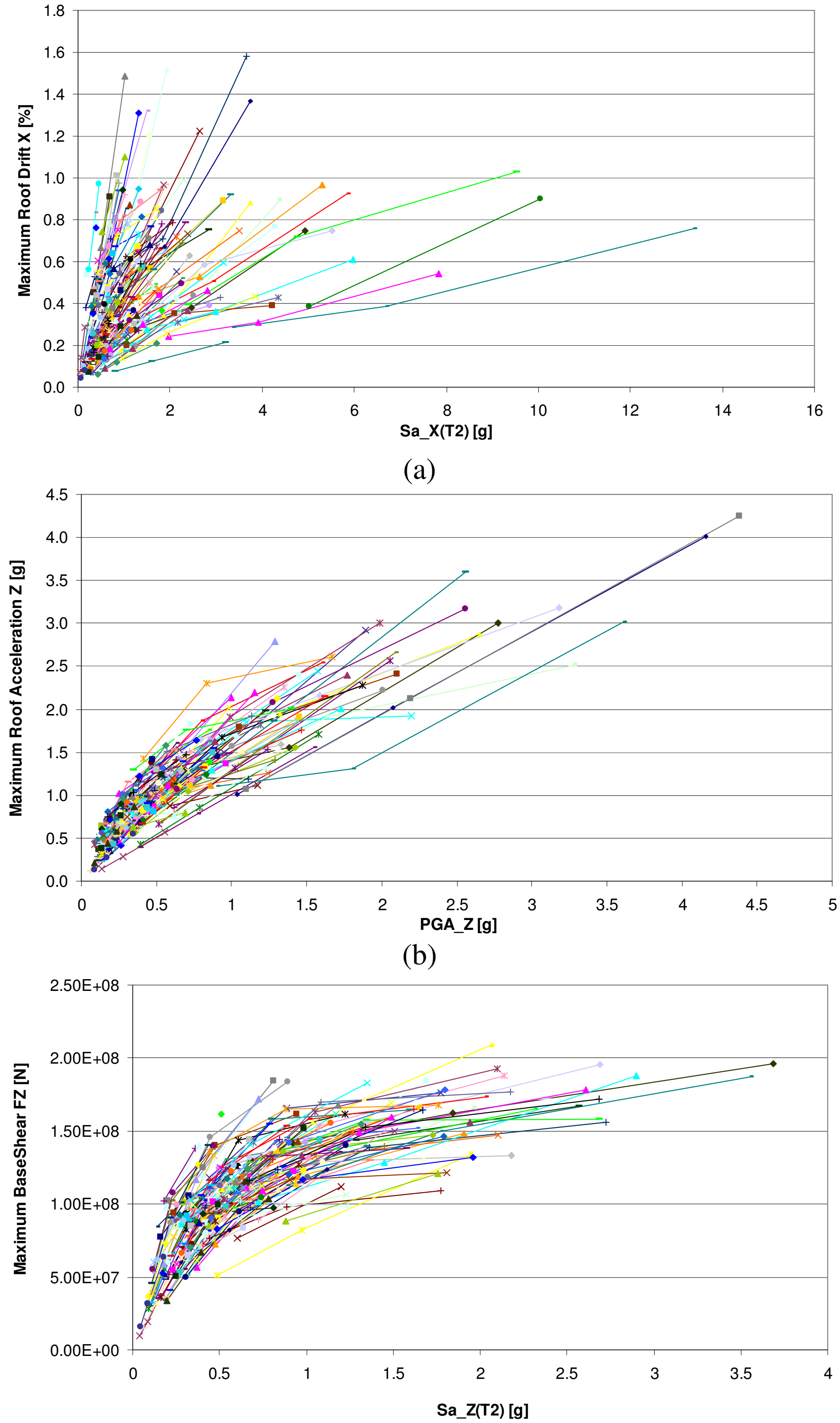


Figure 7: (a) Roof drift ratio vs Spectral acceleration at the 2nd Mode structural period; (b) Roof acceleration vs PGA; (c) Base Shear vs Spectral acceleration at the 2nd Mode structural period

roof drift ratio correlates well to Spectral acceleration at the 1st mode period (Sa[T1]). Similarly, it is found that roof acceleration correlates better to PGA (Figures 7b), base shear correlates better to Spectral acceleration in the 2nd mode period (Figures 7c) and base moment correlates more to Spectral acceleration at the 1st mode structural period. Therefore, it is evident that, for a high-rise building, it might not be a good idea to characterize EDP with a single ground motion parameter.

These findings can be used in performance-based design for high-rise buildings to answer the questions such as (i) what is the annual rate (probability) that the roof drift ratio will exceed 1%? (ii) what should be the target median roof drift ratio a lifetime of 75 years? The next section presents probabilistic model of EDP responses.

3.4 Probabilistic Model of EDP Responses

Findings from Section 3.3 and the theoretical background presented in Section 2 are used to develop probabilistic models for critical building responses. First of all, it is necessary to determine the constants k and k_0 (Equation 3; Figure 1) to represent the spectral acceleration hazard curve with a power-law relationship. Section 2.2 described the technique to obtain these constant by using spectral acceleration value for 475-years and 2475-years return periods. The spectral acceleration values for 475-year (10% probability of exceedance in 50 year – DBE) and 2475-year (2% probability of exceedance in 50 year – MCE) return periods can be obtained from Figure 3b, where MCE and DBE response spectra are plotted. The value of k may be calculated as:

$$k = \frac{\ln\left(\dfrac{H_{s(10/50)}}{H_{s(2/50)}}\right)}{\ln\left(\dfrac{S_{(10/50)}}{S_{(2/50)}}\right)} = \frac{1.65}{\ln\left(\dfrac{S_{(10/50)}}{S_{(2/50)}}\right)}$$

where $S_{(10/50)}$ = spectral amplitude for 10/50 hazard level; $S_{(2/50)}$ = spectral amplitude for 2/50 hazard level; $H_{S(10/50)}$ = probability of exceedance for the 10/50 hazard level = 1/475 = 0.0021; and $H_{S(2/50)}$ = probability of exceedance for the 2/50 hazard level = 1/2475 = 0.00404.

(a) Roof Drift Ratio. Figure 8 shows the plot of roof drift ratio (in the Z-direction) from NLTHA and a fitted regression curve for the 1st mode period (5.0 seconds). Following the methodology described in Section 2, the hazard curve for roof drift ratio is presented in Figure 9a. This curve shows the annual rate of exceedance of roof drift, but it extremely important to point out that it is applicable only for the Los Angeles area and for a building of the same 1st mode structural period of 5.0 second. From the figure, it can be seen that a 1% roof drift ratio has an annual rate of exceedance of 4e-5 or a return period of about 25000 years. Figure 9b illustrates the Poissonian probability of exceedance for roof drift values for a life time of the structure of 50-, 75-, and 100-years. From the figure, if a structure with a fundamental period of 5.0 second is designed for LA area then a 0.6% roof drift will have a probability of exceedance of 10% for a 100-year lifetime.

Similarly, Figure 10 shows the hazard curve and Figure 11a depicts the Poissonian probability of exceedance of roof drift for a 1st mode structural period of 3.33 sec. Now from Figure 9b and Figure 11a, roof drift values corresponding to 2% probability of exceedance in 50- and 100-years can be extracted and are plotted in Figure 11b. The horizontal axis of the Figure 11b represents the fundamental period of vibration of a structure whose performance is sought. The concept shown in this figure is similar to that of a response spectrum. The trend in Figure 11b can be predicted with few more points with different fundamental period of vibration.

(a) Base Shear. Figure 12 shows the plot of base shear (in Z-direction) from NLTHAs and a fitted regression curve for the 2nd mode structural period of 1.429 seconds. The hazard curve for base shear is presented in Figure 13, which shows the annual rate of exceedance of base shear. From the figure, it can be seen that base shear of 0.25W (W is the seismic weight of the structure) has an annual rate of exceedance of 3.2e-3 or a return period of roughly 300 years. Figure 14 illustrates the Poissonian probability of exceedance for base shear values for a lifetime of the structure of 50-, 75-, and 100-years.

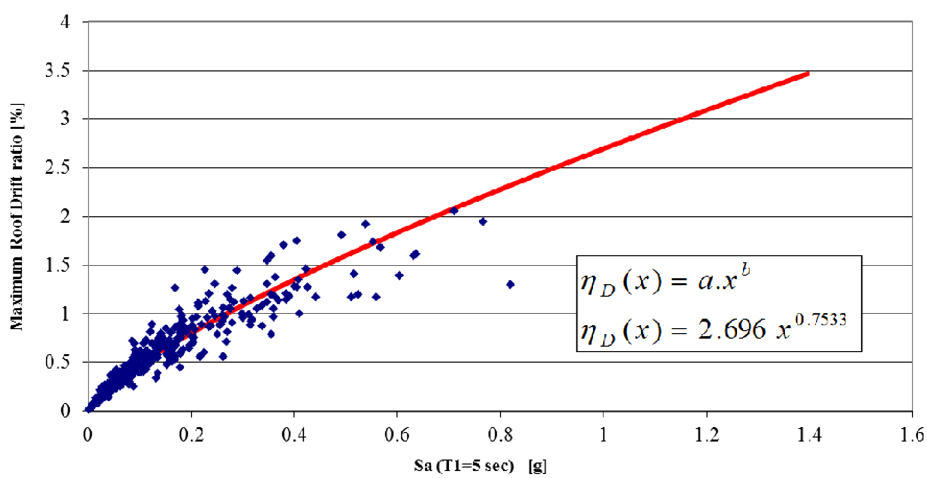
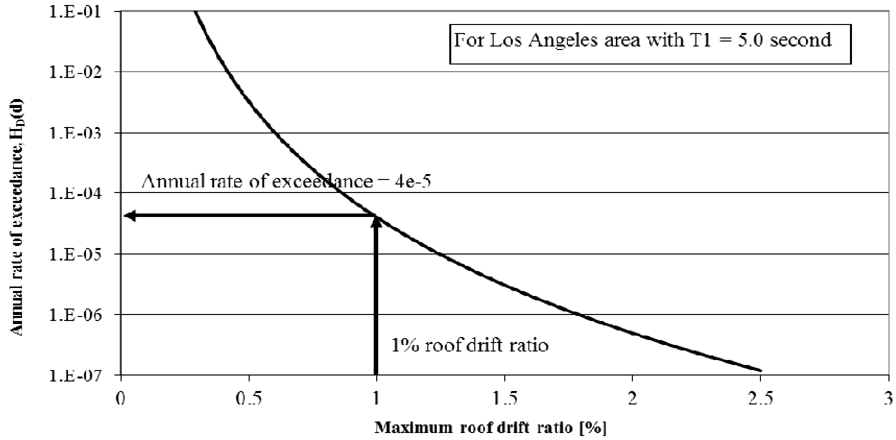
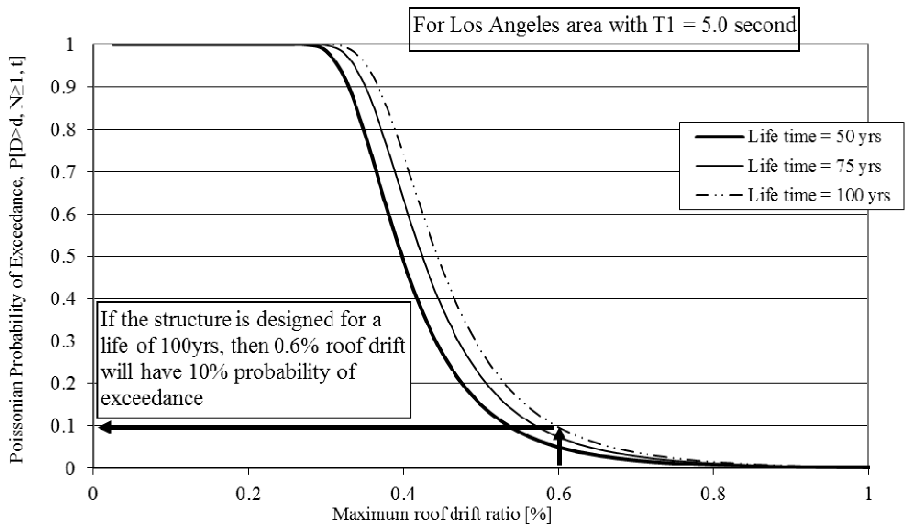


Figure 8: Roof drift ratio from NLTHA and fitted regression curve for 1st Mode structural period of 5 sec



(a)



(b)

Figure 9: (a) Hazard curve for roof drift ratio for 1st mode structural period of 5 sec; (b) Poissonian probability of exceedance of roof drift for 1st mode structural period of 5 sec

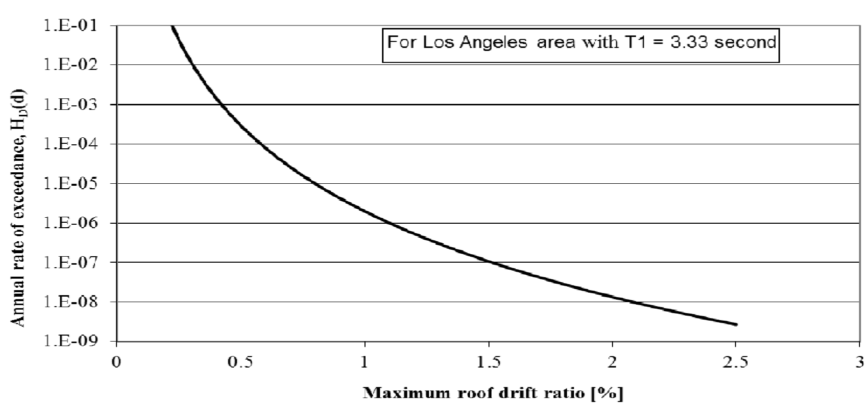


Figure 10: Hazard curve for roof drift ratio for 1st mode structural period of 3.33 sec

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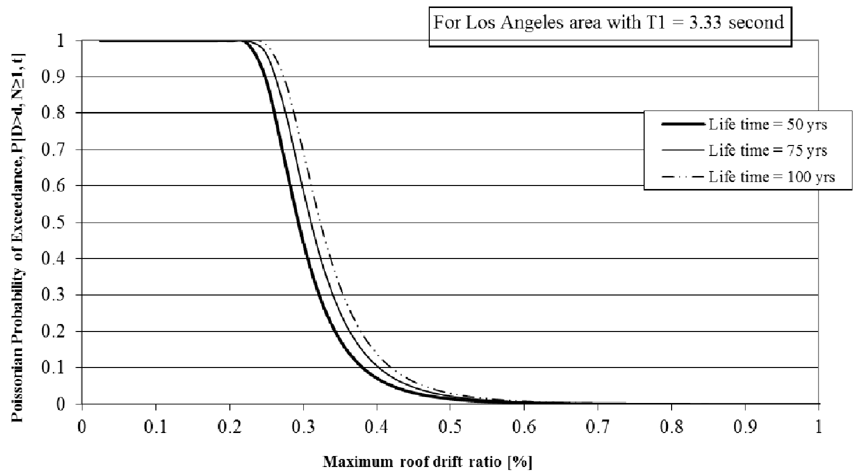

(a)

(b)

Figure 11:(a) Poissonian probability of exceedance of roof drift for 1st mode structural period of 3.33 sec;(b) Performance of a Structure (defined by fundamental period of vibration)

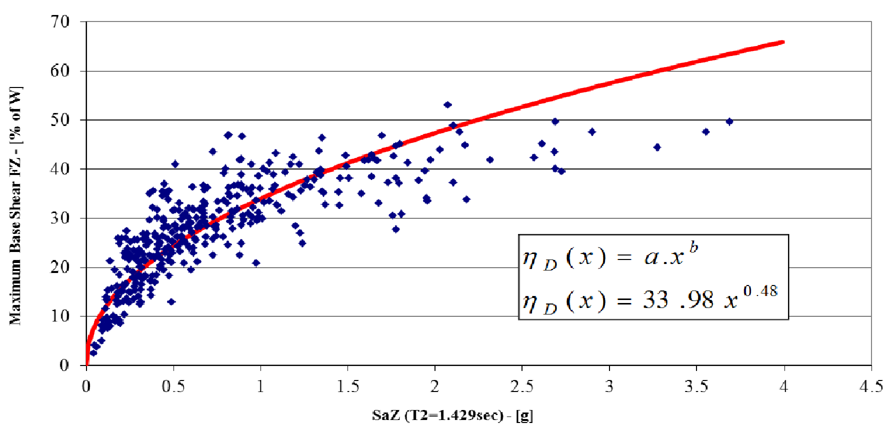

Figure 12: Base Shear from NLTHA and fitted regression curve for 2nd Mode structural period of 1.429 sec

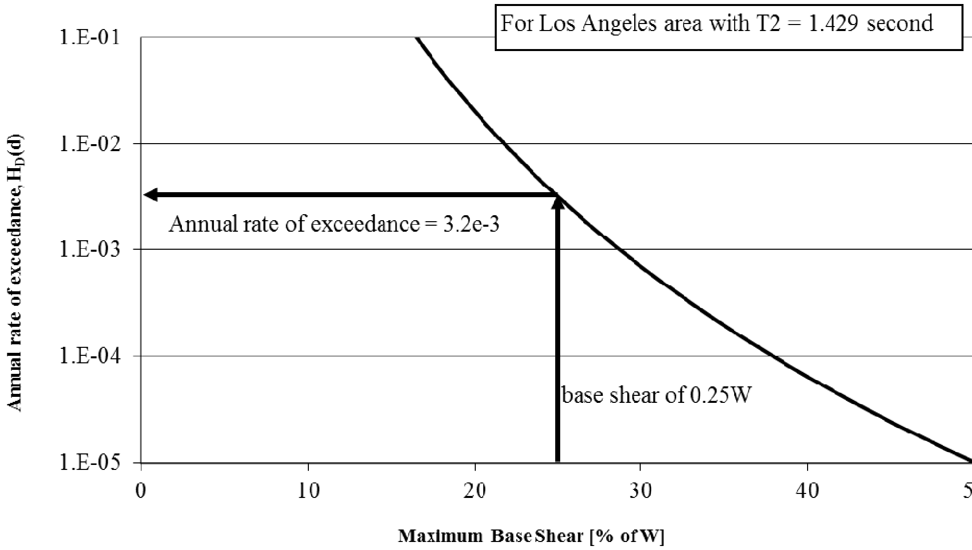

Figure 13: Hazard curve for Base shear for 2nd mode structural period of 1.429 sec

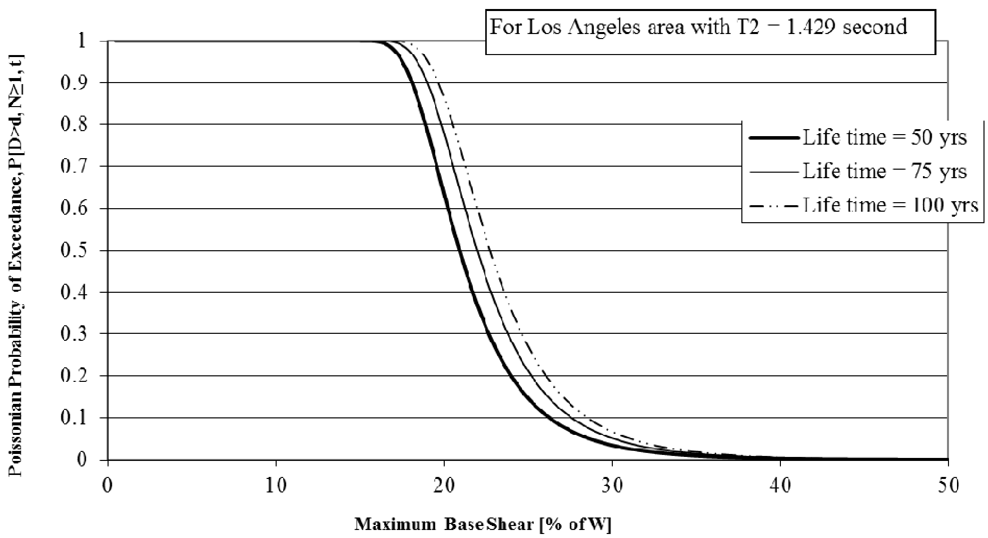

Figure 14: Poissonian probability of exceedance of Base shear for 2nd mode structural period of 1.429 sec

4. Conclusion

Statistical models for several critical building responses were developed using diagrid tall building. These probabilistic characterization will help to develop and/or assess guidelines for seismic design of high-rise buildings. A total of 435 3D-NLTHA are conducted to develop probabilistic models of critical building responses (such as roof drift, base shear, etc). Mathematical model between EDP and ground motion intensity are developed from regression analysis. Then annual rate of exceedance and Poissonian probability of exceedance for each EDP are calculated and ploted. Results indicate that the annual rate of exceedance for roof drift for a building with 1st mode structural period of 5.0 seconds is roughly 4e-5 or a return period of 25000 year. Similarly, a structure with a fundamental period of 5.0 second and designed for the Los Angeles area for a 0.6% roof drift limit will have a probability of exceedance of 10% for a 100-year lifetime.

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