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# DEVELOPMENT OF SEISMIC FRAGILITY CURVES FOR TALL BUILDINGS

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## ABSTRACT

This study focuses on the reliability assessment of tall buildings subjected to earthquake loadings. The vulnerabilities of these tall structures are expressed with the development of fragility curves, which provide the probability of exceeding a prescribed level of damage for a wide range of ground motion intensities. Such fragility curves are extremely important for estimating the overall risk to the civil infrastructure from potential earthquakes. The methodology to develop these curves follows a Monte Carlo simulation approach incorporating uncertainties in the ground motion and in the structural characteristics. Uncertainty of ground motion is introduced by modeling it as a stochastic process. This is accomplished by simulating power spectrum compatible stochastic processes with a prescribed duration and marginal probability distribution function. This study evaluates the effect of the assumption of Gaussianty and the role of duration of strong ground motion. Deviations from Gaussianity can have significant detrimental effects on the structural response, particularly on maximum values. Duration can also have a significant effect on the response, as a nonlinear model involving stiffness degradation is used. Both effects are studied. Uncertainty of structural characteristics is introduced by modeling different material properties as random variables at critical locations of the structure. The developed fragility curves can be very useful for emergency management agencies and insurance companies wishing to estimate the overall loss after a scenario earthquake.

**Keywords:** Tall Buildings, Seismic Fragility Curves, Non-Gaussian Stochastic Processes, Strong Ground Motion Duration, Monte Carlo Simulation.

## INTRODUCTION

The past several decades have witnessed a series of costly and damaging earthquakes. In order to be prepared for such natural disasters, it becomes essential to reasonably estimate, predict and mitigate the risks associated with these potential losses. Risk is typically defined by three components: a hazard (the earthquake), the assets (e.g. building infrastructure) and the fragility of those assets with respect to the hazard. Each component of risk is inextricably fraught with uncertainties. To accurately and reliably relate how fragile (or vulnerable) specific types of buildings may be for a given earthquake intensity may only appropriately be expressed by accounting for these uncertainties. This study is one of the first to relate the vulnerability of

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tall buildings (a specific class of infrastructure) to different earthquake intensity measures. This study is also unique in that it quantifies the effect that the duration of earthquake strong ground motion and its deviations from Gaussianity have on structural response. Although the effects of non-Gaussian loads on cantilevers with geometric nonlinearities have been studied (Antonini *et al.*, 2001), this study will incorporate geometric and material nonlinearities with energy dissipation for a structure. Most vulnerability relationships have been developed for classes of buildings below ten stories. Some assumptions and methods used to develop these relationships are not appropriate for tall buildings (i.e., pushover analyses). Tall buildings have enhanced designs (e.g. advanced lateral resistance for wind loads), are influenced by more complex structural phenomena (P-Δ effects, soil-structure interaction) and different earthquake intensity measures can dominate their response (PGV tends to govern damage rather then PGA because taller building have longer natural periods). This work was particularly inspired by an earthquake loss study for Manhattan (Tantala et al., 2001), which has a considerable number of tall buildings (approximately 1,600 buildings over 20 stories).

## SIMULATION METHODOLOGY FOR GROUND MOTION

Earthquake ground motion time histories are known to exhibit significant randomness in their amplitude, frequency content, duration, phases, etc. To account for the uncertainty in earthquake ground motion, input time histories are modeled as stochastic processes. This is accomplished by simulating sample functions using the following prescribed characteristics: a power spectrum, a marginal probability distribution function and duration of strong ground motion.

The Clough-Penzien acceleration spectrum (Clough and Penzien, 1975) was prescribed to model the power spectral density function of the acceleration time history. Appropriate parameters for deep cohesionless soil conditions for the Clough-Penzien spectrum were used (Ellingwood and Batts, 1982).

Earthquake ground motions have exhibited some deviations from Gaussianity (Grigoriu, 1995). To properly study and account for non-Gaussian characteristics of random seismic excitations, a variation of a method first developed by Yamazaki and Shinozuka (1988) was used to generate sample functions of non-Gaussian earthquake time histories (Deodatis and Micaletti, 2001). This iterative method is based on mapping a Gaussian process to a non-Gaussian process using a memoryless nonlinear transformation (Grigoriu, 1984, 1995). Several Beta distributions were prescribed in this study to compare slight to moderate deviations from Gaussianty. These Beta distributions have coefficients of skewness equal to 0.25, 0.55, 1.10 and 1.68.

Duration can also have a significant effect on the response, as a nonlinear model involving stiffness degradation is used. To study the effect of duration, sets of time histories were simulated to have prescribed durations of strong ground motion equal to 2, 7 and 12 seconds. It should be noted that the three different durations were realized by simulating stationary stochastic processes of different time lengths.

## TALL BUILDING MODEL AND MATERIAL UNCERTAINTIES

In this study, a typical three-bay, 25 story reinforced concrete moment resisting frame building (Figure 1), with appropriate beam and column designs was analyzed. The building was idealized and analyzed using a two-dimensional finite element model consisting of a series of frame elements. The column and beam elements are modeled as elastic elements with pairs of

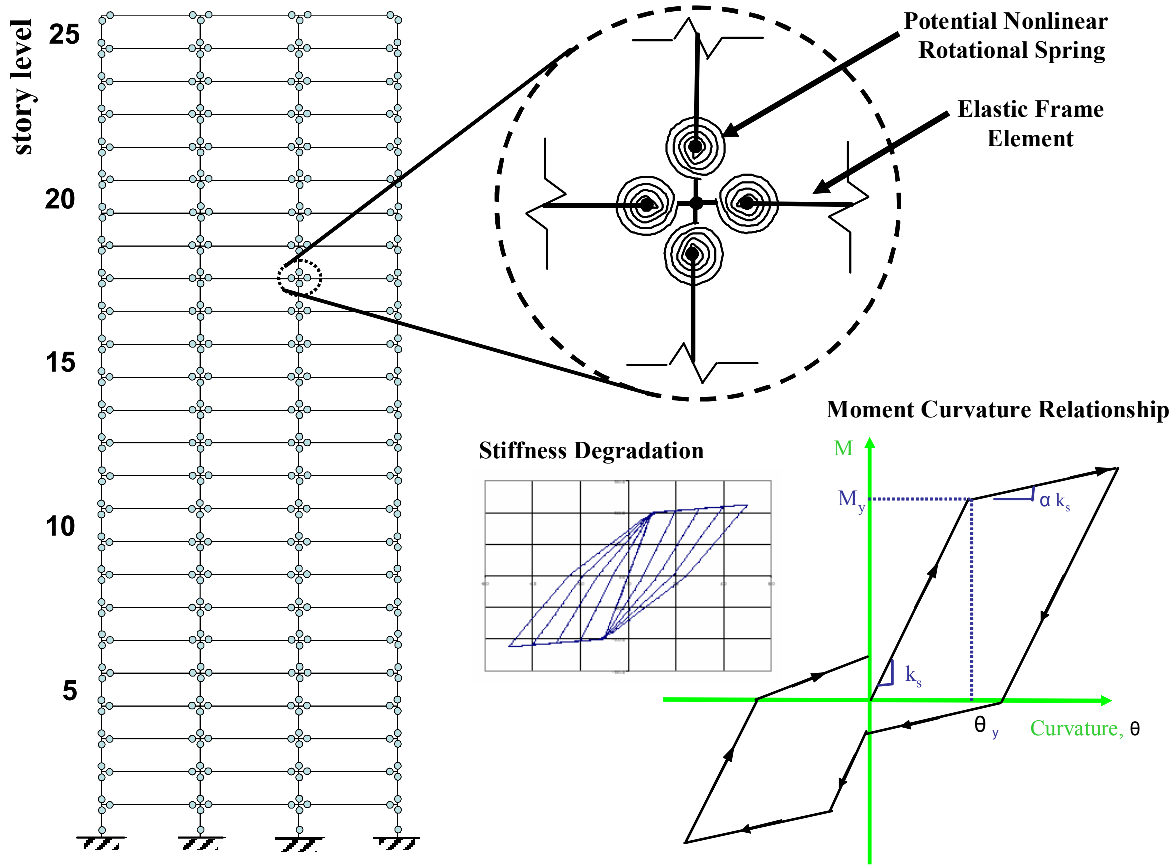


**FIGURE 1. Building model, potential plastic hinge locations and moment-curvature relationship with degradation**

plastic zones at each end. In these zones, material nonlinearity was introduced in the model using Takeda-type plastic hinges (or nonlinear rotational springs) with a bilinear moment-curvature relationship (Figure 1). This relationship also included stiffness degradation over successive cycles of the hysteresis (of particular use for studying the effects of strong motion duration). The moment-curvature envelope, which describes the changes in force capacity with deformation during analysis, was computed for each beam and column cross-section using an incremental, discretized fiber model analysis. For the fiber model parameters, material strengths were considered random variables (summarized in Table 1), while other parameters like beam dimensions were considered deterministic. It should be noted that although the two material properties in Table 1 will follow non-Gaussian distributions in general, it is assumed that they follow normal distributions to be consistent with the assumption in Ellingwood et al. (1980). The parameters of the moment-curvature envelope (yield moment, yield rotation and post yield stiffness) are therefore functions of random variables. The random material strengths were simulated for every beam and column using Latin Hypercube sampling, which is a random variable “shuffling” technique for reducing the number of simulations required to obtain reasonable results (Iman and Conover, 1980). Random variables for beam elements of the same level were considered perfectly correlated (i.e., concrete for all beam elements on a particular floor are likely to come from the same batch). Random variables for column elements (top and bottom hinges) were considered independent and therefore uncorrelated. Stiffness degradation was considered deterministic and an appropriate parameter representing hysteretic energy dissipation capacity was used. In addition to plastic hinging, the nonlinear analysis considered P-Δ effects. These dynamic, inelastic time history analyses were performed using DRAIN-2D (Kanann and Powell, 1973) with several personal enhancements. At this time, the effects of soil-structure interaction were not considered.

**TABLE 1. Statistical parameters of material properties (Ellingwood *et al.*, 1980)**

| Random Variable | Symbol | Mean | C.O.V. | Distribution |
|---|---|---|---|---|
| Concrete Compressive Strength | $f_c^{'}$ | 3390 psi | 18 % | Normal |
| Reinforcement: Grade 60 yield | $f_y$ | 67.5 ksi | 9.8 % | Normal |

The classic criterion of inter-story drift was used to establish different damage states ranging from slight (hairline cracks) to complete (collapse). More detail on these damage state descriptions are available in NIBS (1997) and on drift ratios in Kircher (1999).

## DEVELOPMENT OF FRAGILITY CURVES

It is assumed that a "suite" of fragility curves may be developed by expressing the vulnerability relationship as a series of two-parameter lognormal distribution functions. A set of parameters of lognormal distributions representing fragility curves associated with all levels of damage states for the building under consideration may be simultaneously estimated so that fragility curves do not intersect. Using the maximum likelihood method, a common log-standard deviation ($\zeta$) and medians ($c_j$) of lognormal distributions are estimated for a set of j fragility curves. For this paper, a family of 3 fragility curves may be determined where events $E_1$, $E_2$, $E_3$ and $E_4$ respectively indicate the state of no, slight, moderate and extensive damage. The estimation of the sets of parameters ($c_j$ and $\zeta$) is done by maximizing the likelihood function which is expressed as (Shinozuka *et al.*, 2000):

$$L(c_1, c_2, c_3, \zeta) = \prod_{i=1}^{n} \prod_{k=1}^{4} P_{ik}\left(a_i; E_k\right)^{x_{ik}} \tag{1}$$

where $x_{ik}$ is equal to 1 if the damage state $E_k$ occurs in the $i$-th simulation with earthquake intensity $a_i$, while $x_{ik}$ is equal to zero otherwise. In the likelihood function, $P_{ik}(a_i;E_k)$ represents the probability that a building i, selected randomly from a sample of n simulations, will be in damage state $E_k$ when subjected to motion intensity expressed by PGA = $a_i$. The maximum likelihood estimates of the set of fragility curve parameters (medians $c_j$ and common log-standard deviation $\zeta$) are obtained by solving the following equations using an optimization algorithm (Shinozuka *et al.*, 2000):

$$\frac{\partial \ln L(c_1, c_2, c_3, \zeta)}{\partial c_j} = \frac{\partial \ln L(c_1, c_2, c_3, \zeta)}{\partial \zeta} = 0 \;;\; (j = 1, 2, 3) \tag{2}$$

All fragility curves are represented by two-parameter lognormal distribution functions in Equation 3 and as part of the same "family" are related to one another in Equation 4 (Shinozuka *et al.*, 2000).

$$F_j\left(a_i;c_j,\zeta\right)=\Phi\left[\frac{\ln\left(a_i/c_j\right)}{\zeta}\right];\ (j=1,2,3) \tag{3}$$

$$\begin{aligned}
P_{i1}&=P\left(a_i,E_1\right)=1-F_1\left(a_i;c_1,\zeta\right)\\
P_{i2}&=P\left(a_i,E_2\right)=F_1\left(a_i;c_1,\zeta\right)-F_2\left(a_i;c_2,\zeta\right)\\
P_{i3}&=P\left(a_i,E_3\right)=F_2\left(a_i;c_2,\zeta\right)-F_3\left(a_i;c_3,\zeta\right)\\
P_{i4}&=P\left(a_i,E_4\right)=F_3\left(a_i;c_3,\zeta\right)
\end{aligned} \tag{4}$$

## RESULTS

An exhaustive series of parametric simulations were performed to express the response statistics of the tall building considered here with uncertain material properties and uncertain loads modeled by stochastic processes. Each parameter set of this study consisted of 1000 simulations. These studies compared the effects of the assumption of Gaussianity and of duration.

Time histories were simulated with prescribed strong ground motion durations of 2, 7 and 12 seconds. Figure 2 shows a sample scatter plot of maximum inter-story drift ratios for different PGA and durations. Figure 3 shows the corresponding fragility curves representing the probability of exceeding ‘extensive damage’ (partial collapse) for the three durations in Figure 2. The effect of duration of strong ground motion on the vulnerability of this model structure is clearly significant.

Time histories were also simulated with different prescribed marginal probability distributions to study the effect of the Gaussian assumption. The statistics of the structure’s response (maximum inter-story drift ratios) from time histories that are Gaussian (zero skewness) and non-Gaussian (Beta distributions with a 1.68 coefficient of skewness) are compared in Figures 4 and 5. It is noted that for every Gaussian time history considered, a corresponding non-Gaussian time history is generated according to the exact same power spectrum and a prescribed marginal Beta pdf. It is evident from Figure 5 that deviations from Gaussianity have significant detrimental effects on the maximum structural response.

## CONCLUSIONS

Vulnerability relationships have been developed for a specific class of tall buildings. Prior to this work, such relationships had only been appropriately developed for buildings below 10 stories. Ground motion was simulated as a stochastic process with a prescribed power spectrum, marginal probability distribution function and duration. These ground motions were used with a nonlinear finite element model with uncertain material properties to generate, via Monte Carlo simulation, the statistics of the response of a tall, reinforced concrete moment resisting frame building. The results of this study:

- Demonstrate that duration of strong ground motion is a significant factor in the vulnerability of tall buildings. This study quantifies the effect that duration has on a building’s statistical response using fragility curves. The longer the duration, the higher the probability of exceeding a certain damage level for a given PGA (as continued shaking degrades stiffness).

- Demonstrate that deviations from Gaussianity do have significant detrimental effects on the structural response (particularly on maximum values). These detrimental effects are numerically expressed by comparison of fragility curves.

These new vulnerability relationships will be very useful for researchers, emergency management agencies and engineers wishing to better estimate the overall loss after an earthquake for tall buildings (a unique infrastructure which concentrates people and value-at-risk). Improved forecasts (accounting for tall building behavior and non-Gaussian and duration effects) will provide officials with better tools to more adequately plan and stimulate efforts to reduce risk and to better allocate resources to prepare for emergency response and recovery from an earthquake. Future work is required and may include incorporation of soil-structure interaction, more sophisticated representation of tall building behavior and development of vulnerability relations for other classes of tall buildings.

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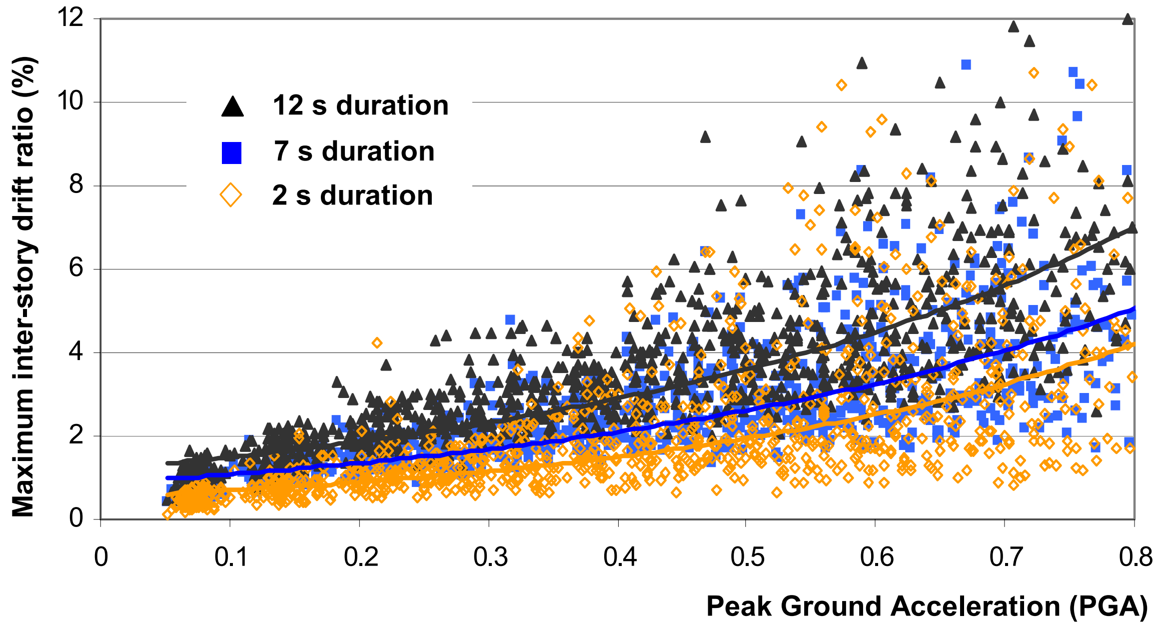


**FIGURE 2. Sample scatter plot of maximum inter-story drift ratios as a function of peak ground acceleration (PGA) for earthquakes with 2, 7 and 12 second durations. For each duration set, there are 1000 simulations.**

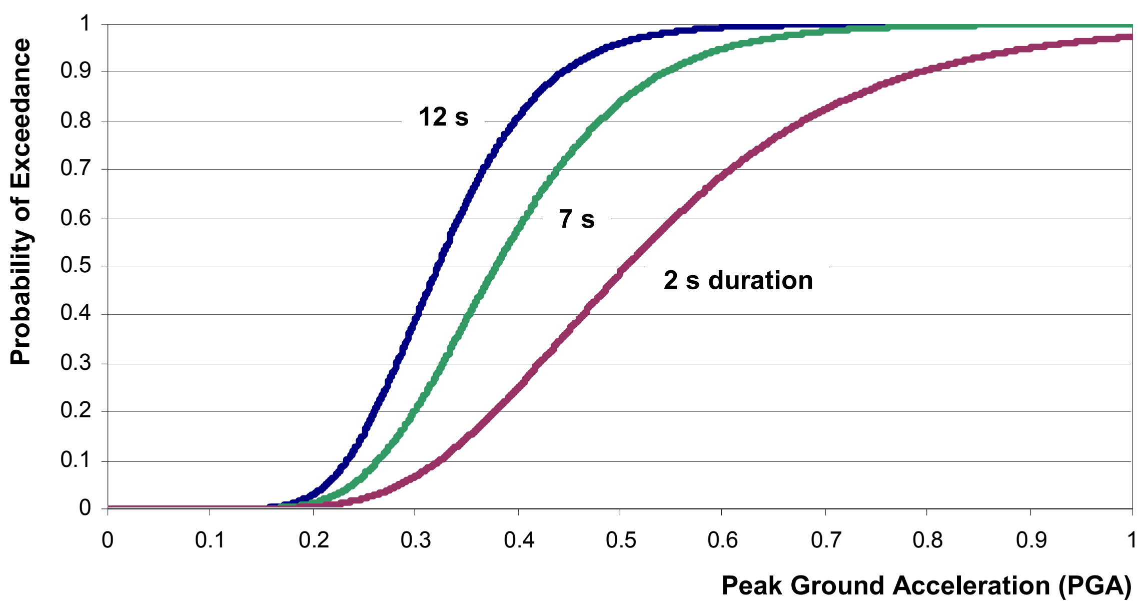


**FIGURE 3. Fragility curves for extensive damage as a function of peak ground acceleration (PGA) for earthquakes with 2, 7 and 12 second durations.**

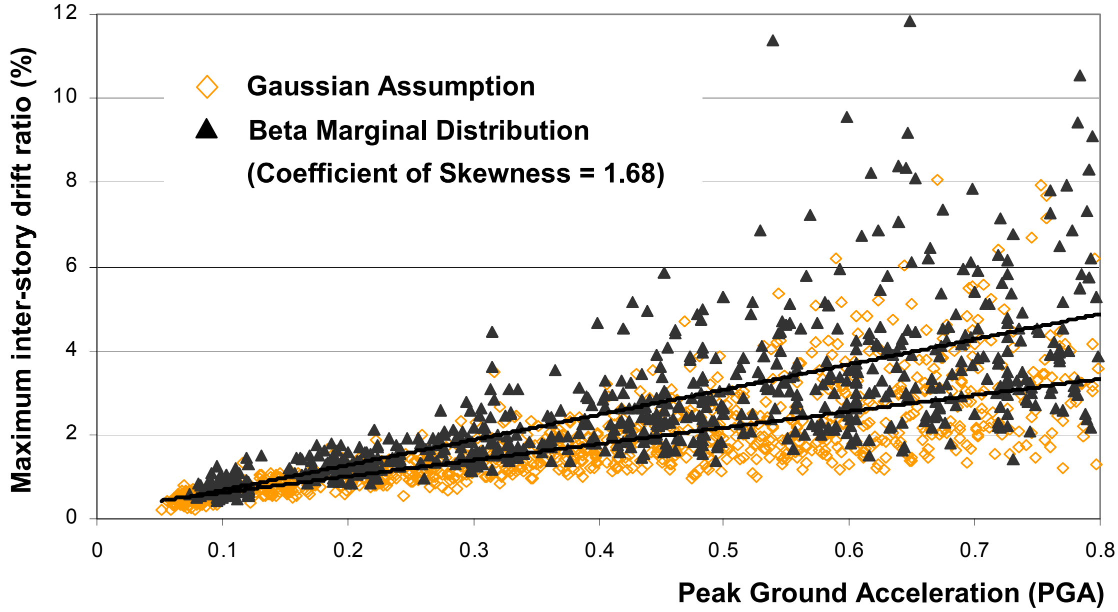


**FIGURE 4. Sample scatter plot of maximum inter-story drift ratios as a function of peak ground acceleration (PGA) for earthquakes with Gaussian and Beta marginal distribution functions.**

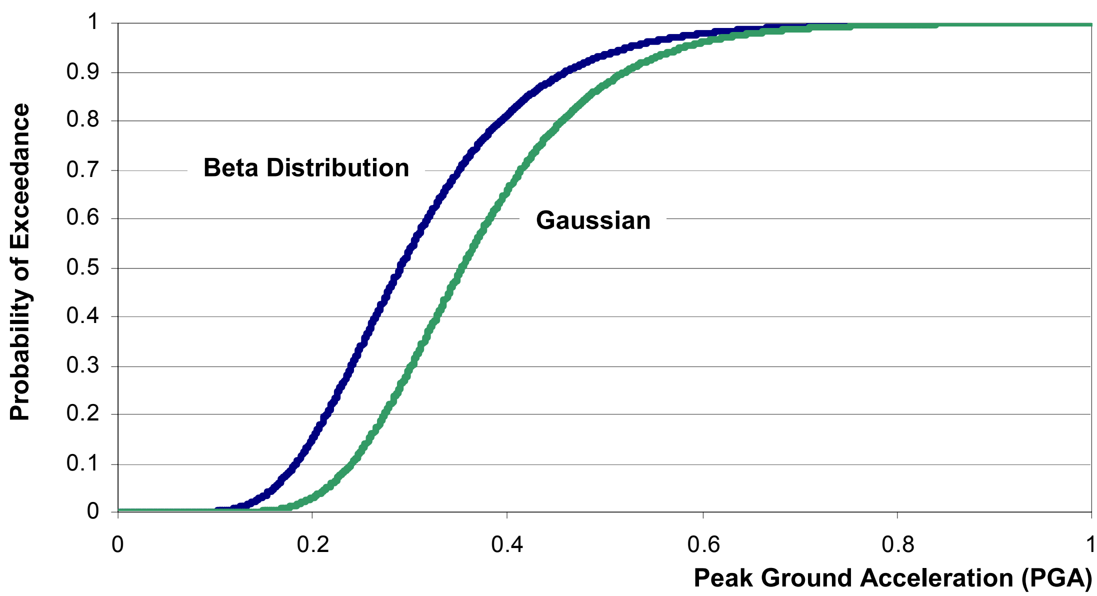


**FIGURE 5. Fragility curves for extensive damage as a function of peak ground acceleration (PGA) for earthquakes with Gaussian and Beta (skewness coefficient of 1.68) marginal distribution functions.**