SAFETY ASSESSMENT OF GRAVITY LOAD DESIGNED RC FRAMED BUILDINGS

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ABSTRACT: A large number of gravity load designed (GLD) RC buildings are found in many seismic-prone countries including India. Many of them were built before the advent of seismic codes or with the utilization of old and inadequate seismic design criteria. Engineers and decision makers need to have information on the seismic vulnerability of such buildings in a given region for mitigation planning. The present study aims at evaluating the relative seismic vulnerability of GLD building subjected to seismic hazards corresponding to various seismic zones of India. The relative seismic vulnerability of GLD buildings for various site hazard conditions is estimated in a practical 'load and resistance factor' format as per the 2000 SAC Federal Emergency Management Agency (SAC-FEMA) guidelines. The results of this study show that the relative vulnerability of the GLD building (with respect to a building designed for seismic forces) increases many folds from lower to higher seismic zone. It also indicates that the GLD buildings existing in higher seismic zones of India (IV and V) should be immediately uninhabited as an urgent mitigation measure.

Keywords: Gravity load, building, fragility, seismic hazard, confidence level

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INTRODUCTION

During past earthquakes (Bhuj 2001, Kashmir 2005, Sichuan 2008, Nepal 2015, Italy 2016) reinforced concrete (RC) buildings often displayed unsatisfactory seismic behavior, particularly when their design included only gravity loads. A large number of gravity load designed (GLD) buildings are found in many seismic-prone countries including India. Many of them were built before the advent of seismic codes or with the utilization of old and inadequate seismic design criteria. Special attention has been given (Hoffman *et al.* 1992, Masi 2003, Magenes and Pampanin 2004, Polese *et al*. 2008) to the investigation on the seismic vulnerability of such existing buildings. This special attention may be attributed to a number of socio-economic reasons. The global rural-urban balance is increasingly in favour of cities (UNFPA 2011) and gravity load designed RC buildings represent a large portion of the built environment of urban areas. The loss in terms of human lives and economy during an earthquake in the present scenario depends largely on the performance of GLD buildings in urban areas. Engineers and decision makers need to have information on the seismic vulnerability of such buildings in a given region for mitigation planning (Ramamoorthy *et al.* 2008). Thus, the evaluation of the seismic vulnerability of GLD buildings has a key role in the determination and reduction of earthquake impact. Most of the previous studies (Aktan and Bertero 1987, Aktan and Nelson 1988, Qi and Pantazopoulou 1991, Benavent-Climent *et al.* 2004, Magenes and Pampanin 2004, Laterza 2016, Lin *et al*. 2016) employed deterministic approach for the performance assessment of the GLD buildings. Almost all of the studies (Masi 2003, Ramamoorthy *et al*. 2008, Masi and Vona 2012, Bakshi and Asadi 2013, Masi *et al*. 2015, Wu *et al*. 2015) which considered the probabilistic approach do not include the site hazard for the safety assessment. As the GLD buildings perform differently in different site hazard conditions, the performance assessment is not complete without the consideration of associated site hazards. A detailed literature review revealed only a few studies (Polese *et al*. 2008, Ellingwood *et al.* 2007, Celik and Ellingwood 2009, Halder and Paul 2016) that deals with fragility of buildings along with site hazard. However, these studies do not focus on the relative vulnerability of GLD buildings in different seismic zones having low to high seismic hazards.

The present study aims at evaluating the relative seismic vulnerability of GLD building subjected to seismic hazards for various seismic zones of India (IS 1893:2002). Probabilistic seismic risk of a representative four storey GLD building is evaluated and compared with those of similar building designed for seismic loads corresponding to selected seismic zones.

RESEARCH SIGNIFICANCE

GLD buildings are expected to be more vulnerable than the seismically designed buildings but their relative vulnerability with respect to the seismically designed building may not be the same at different site hazard conditions. Previous studies on the quantification of this relative seismic vulnerability of GLD buildings are limited. Stakeholder's need to have information about this relative seismic vulnerability of GLD buildings for improved disaster mitigation planning. This paper demonstrates the relative seismic vulnerability of GLD buildings for various site hazard conditions in a practical load and resistance factor format as per the SAC-FEMA method. However, SAC-FEMA method, which is introduced initially for simplified probabilistic seismic assessment of steel buildings, is validated using a more accurate Monte Carlo simulation method in the present study. The confidence levels of existing GLD buildings in terms of demand and capacity factors are compared with respect to seismically designed buildings.

SAFETY ASSESSMENT METHODOLOGY

Probabilistic seismic safety assessment characterizes the randomness and uncertainty in both seismic demand and capacity of the structure. Following parameters are used by SAC-FEMA method (Cornell *et al.* 2002) for the evaluation of seismic risk: Probabilistic seismic demand model (PSDM), fragility curves, drift hazard curves and mean annual probability of exceedance, demand factor, capacity factor and confidence levels. This section explains the theory behind this assessment.

PSDM and Fragility Curves

A fragility function represents the probability of exceedance of the seismic demand (D) for a selected performance level (C) for a specific ground motion intensity measure (PGA). Maximum inter-storey drift demand in a building subjected to a ground motion is considered as demand parameter (D) in the present study. Fragility curve presents a cumulative probability distribution that indicates the probability that a building will be damaged to a given damage state or a more severe one, as a function of a particular intensity measure. It can be obtained for each damage state and can be expressed in closed form as follows:

$$P\left(C - D \le 0 \mid PGA\right) = \Phi\left(\frac{ln\left(\hat{D}/\hat{C}\right)}{\sqrt{\beta_{D|PGA}^{2} + \beta_{C}^{2} + \beta_{m}^{2}}}\right) \quad (1)$$

in which Φ is widely tabulated as standardized Gaussian distribution function, $\hat{D}$ is the median drift demand, $\hat{C}$ is the median of the drift at chosen performance level, $\beta_{D|PGA}$ is the dispersion in drift demand at a given PGA level, β_C is the dispersion in capacity and β_m is the dispersion in modeling. A series of nonlinear time history analysis is carried out to obtain the probabilistic representation of demand parameter. An analytical approximation of this representation is considered as per Cornell *et al.* (2002) that says, at given level of PGA, the predicted median drift demand $(\hat{D})$ can be represented approximately by the form:

$$\hat{D} = a\,(PGA)^b \tag{2}$$

where the constants, 'a' and 'b' are the regression coefficients. The drift demands (D) are assumed to be distributed log-normally about the median (Shome and Cornell 1999) with a standard deviation, $\beta_{D|PGA}$ (the dispersion in drift demand D at a given PGA level). The three parameters, a, b and $\beta_{D|PGA}$ are obtained by performing a number of nonlinear analyses and then conducting a regression analysis of $ln(D)$ on $ln(PGA)$. The power-law relationship presented in Eq. 2 represents the probabilistic seismic demand model (PSDM) for the considered frame.

Incorporating this power-law approximation, Eq. 1 can be re-written as follows:

$$P\left(C - D \le 0 \mid PGA\right) = 1 - \Phi\left(\frac{ln\left(\hat{C}\right) - ln\left(a.PGA^b\right)}{\sqrt{\beta_{D|PGA}^2 + {\beta_C}^2 + {\beta_m}^2}}\right) \tag{3}$$

A performance level (C) defines the capacity of building to withstand a specified level of damage which can be represented quantitatively. Inter-storey drift capacities $(\hat{C})$ for various performance levels, range from slight damage to complete destruction, are taken from Masi *et al*. (2015) as shown in Table 1. The value of β_C depends on the building type and construction quality, and it has been assumed as 0.25 as per ATC 58 (2012) for the moderate quality of construction in this study.

Drift hazard curves and Mean annual probability of exceedance

To provide the likelihood of unacceptable behavior of selected GLD building at a given site and the associated confidence levels (Yun *et. al*. 2002), it is important to consider the seismic hazard of the site which is not considered in the PSDM and fragility curves. The parameters, drift hazard curve and mean annual probability of exceedance permit one to assess the seismic safety of the building. The confidence level of the design of any building will provide the degree of uncertainty in its seismic safety. The methodology to obtain these parameters are discussed in this section.

This method incorporates three analytical approximations. The first approximation is the assumption of the hazard function, $H(PGA)$ which gives the annual probability of occurrence of the earthquake at any given site. The other two approximations are introduced in the form of a power law relationship (Eq. 2) between inter-storey drift demand (D) and PGA and the log normality assumption of inter-storey drift (*D*). The probabilities of the buildings exceeding any performance level are achieved by combining the probabilistic representations of the three elements in two steps. The first step couples the first two basic elements, hazard function $H(PGA)$ and drift demand function, $D(PGA)$ in terms PSDM to produce a drift hazard curve $H_D(d)$. $H_D(d)$ provides the annual probability that the drift demand (D) exceeds any specified drift value (d). The second step combines this curve with the drift capacity (C) to produce P_{PL} which is defined as the annual probability of the performance level not being met.

Using the total probability theorem (Benjamim and Cornell 1970) $H_D(d)$ can be written as

$$H_D(d) = \int P[D \geq d | PGA = x_i] |dH(x)| \tag{4}$$

Where $dH(x)$ can be obtained from standard hazard curve $H(PGA)$. Assuming that the hazard curve can be estimated in the region of interest, by the form

$$H(PGA) = \int P[PGA \geq pga] = k_0.(PGA)^{-k} \quad (5)$$

Where k_0 and k are the constant coefficients. The above expression implies that the hazard curve is linear on a log-log plot in the region of interest.

Using Eq. 2 and the log normality assumption, the first factor of Eq. 4 can be written as

$$P[D \geq d|PGA = x_i] = 1 - \Phi\left(ln\left[d / ax^b\right] / \beta_{D|PGA}\right) \quad (6)$$

Using Eq. 6 and Eq. 5, Eq. 4 for the drift hazard curve can be written in a simplified form as

$$H_D(d) = H(PGA^d) exp\left[\frac{1}{2}\frac{k^2}{b^2}\beta^2_{D|PGA}\right] \quad (7)$$

PGA^d is the peak ground acceleration corresponding to the drift demand level, *d* i.e.

$$PGA^d = \left(\frac{d}{a}\right)^{\frac{1}{b}} \quad (8)$$

where '*a*' and '*b*' are the regression coefficients (refer Eq. 2). Detailed derivation of Eq. 7 is available in Jalayer and Cornell (2003). Using the total probability theorem, the annual probability of unacceptable performance (P_{PL}) can be defined as:

$$P_{PL} = P[C \leq D] = \sum_{all\, d_i} P[C \leq D|D = d_i]P[D = d_i] \quad (9)$$

The second factor in the above equation represents the likelihood of a given drift demand level, $P[D = d]$ which can be determined from the drift hazard curve derived in Eq. 4. Eq. 9 can be represented in continuous form as

$$P_{PL} = \int P[C \leq d]|dH_D(d)| \quad (10)$$

The drift capacity (C) is assumed to be log-normally distributed with a median value $\hat{C}$ and dispersion β_C. Estimation of these parameters ($\hat{C}$ and β_C) is described by Yun and Foutch (2000) and Yun *et al*. (2002). With the log-normality assumption, the first factor in Eq. 10 becomes

$$P\left[C \le d\right] = \phi\left(\frac{In(d/\hat{C})}{\beta_C}\right) \qquad (11)$$

Substituting and carrying out the integration, P_{PL} can be written as

$$P_{PL} = H\left(PGA^{\hat{C}}\right) exp\left[\frac{1}{2}\frac{k^2}{b^2}\left(\beta^2_{D/PGA} + \beta_C{}^2\right)\right] \qquad (12)$$

where, $PGA^{\hat{C}}$ is the peak ground acceleration 'corresponding to' the median drift capacity, $\hat{C}$. In other words, it is the most likely intensity of the earthquake (PGA) at which the building will be subjected an inter-storey drift equal to the value $\hat{C}$ (limit state drift capacity) and it can be found out from Eq. 2 by substituting $\hat{D}$ as $\hat{C}$.

$$PGA^C = \left(\hat{C}/a\right)^{\frac{1}{b}} \qquad (13)$$

Eq. 12 implies that, if there is no uncertainty in D and C, the P_{PL} will be the probability of the occurrence of the ground motion having PGA of PGA^C. The dispersion in D and C increases the failure probability (P_{PL}) exponentially.

Confidence levels

In order to represent the seismic performance assessment of buildings in a practically convenient format in line with Load Resistance Factor Design (LRFD) approach, three factors are introduced by Cornel *et al.* (2002): demand factor (γ) to account for the uncertainty in drift demand, capacity factor (ϕ) to account for the uncertainty in capacity and confidence factor (λ) to account for the desired safety. The parameter γ represents here the measure of dispersion in the ground motion. Similarly, the parameter, ϕ represents the dispersion in the capacity of the structure. The parameter, λ measures the safety level considering uncertainty in both demand and capacity.

To transform Eq. 12 into a convenient format, P_{PL} is equated to the performance objective P_0 , and rearranged using Eq. 5, yielding

$$\left\{exp\left[-\frac{1}{2}\frac{k}{b}\beta_C^2\right]\right\}\hat{C} \geq \left\{exp\left[\frac{1}{2}\frac{k}{b}\beta_{D|PGA}^2\right]\right\}\hat{D}^{P_0} \tag{14}$$

$$\phi.\hat{C} \geq \gamma.\hat{D}^{P_0} \tag{15}$$

In which $\hat{D}^{P_0}$ is defined as the median drift demand under a given ground motion having a PGA level of the annual probability P_0 of being exceeded. The capacity and demand factors can be calculated as,

$$\varphi = exp\left[-\frac{1}{2}\frac{k}{b}\beta_C^2\right] \tag{16}$$

$$\gamma = exp\left[\frac{1}{2}\frac{k}{b}\beta_{D|PGA}^2\right] \tag{17}$$

By obtaining these explicit relationships, one can ensure the probabilistic performance objectives which involve the explicit nonlinear dynamic behavior of buildings based on the drift or displacements rather than forces.

In order to ensure the probability of failure of the building as low as P_0, the median drift capacity $\hat{C}$, must exceed the median drift demand $(\hat{D}^{P_0})$. By this scheme, one can find the probability of occurrence of maximum earthquake level that any designed building can resist, provided the building satisfies certain safety standards given by Eq. 15. The Eq. 15 can be used to confirm whether a building designed as per the existing design standards satisfy the performance objective P_0 in three steps. Step 1: find the ground motion intensity from the hazard curve with a probability of occurrence, P_0. Step 2: determine the median drift demand for this PGA . Step 3: compare the factored median capacity $(\hat{C})$ against the factored $(\hat{D})$ considering uncertainty, to determine the level of confidence as follows.

$$\lambda = \gamma . \hat{D}^{P_0} \Big/ \phi . \hat{C} \tag{18}$$

Where λ is the confidence factor. Higher the value of λ the lower is the level of confidence in the safety. This factor can also be expressed in terms of total uncertainty in demand and capacity as per Cornell *et al.* (2002) as follows.

$$\lambda = exp\left[-K_x \beta_T + \frac{1}{2}\frac{k}{b}\beta_T^2 \right] \tag{19}$$

Where K_x, the confidence-measuring parameter, is defined as the standard Gaussian variability associated with probability x not being exceeded and β_T, the total uncertainty is given by $\beta_T^2 = \beta_C^2 + \beta_{D|PGA}^2$. Eq. 19 can be rearranged to express K_x as follows.

$$K_x = \left[-ln(\lambda) + \frac{1}{2}\frac{k}{b}\beta_T^2 \right] \Big/ \beta_T \tag{20}$$

The confidence level (x) can be calculated from the value of K_x using the standard Gaussian table. This implies that the confidence level of probability of failure (P_{PL}) less than P_0 is about x %. This approach is used in the present study as an evaluation methodology.

FRAMES CONSIDERED

Typical RC bare frame having four storeys (uniform storey height of 3.2m) and two bays (uniform bay width of 5m) is selected for the present study. This building is designed considering only gravity forces using IS 456 (2000) and designated as ‘G’. The same building is designed for seismic force corresponding to four seismic zones as per IS 1893 (2002). The buildings designed for the seismic load of Zone II (PGA of 0.10g), III (PGA of 0.16g), IV (PGA of 0.24g) and V (PGA of 0.36g) are designated as S1, S2, S3 and S4 respectively. All the building frames are designed considering medium soil conditions (N-value in the range 10 to 30). The characteristic strength of concrete and steel are taken as 25 MPa and 415 MPa respectively. The buildings are assumed to be symmetric in plan and elevation, and hence a single plane frame is considered to be representative of the building along the loading direction. The dead load of the slab including floor finishes is taken as 3.75 kN/m^2 and live load as 3 kN/m^2. The self-weights of the partition walls (230 mm) are applied separately as the uniformly distributed load on the respective beams. The design base shear is calculated using the equivalent static method as per IS 1893 (2002).

The design parameters such as seismic zone, seismic weight (W), response reduction factor (R), natural period (T_{code}) and seismic design base shear (V_B) are given in Table 2. The design details of beams and columns of all the selected frames are presented in Table 3. It is to be noted here that the order of the frames in terms of increasing design lateral strength is G < S1 < S2 < S3 < S4.

STRUCTURAL MODELLING

Selected buildings are modelled for nonlinear time history analysis required for seismic risk assessment. The Open System for Earthquake Engineering Simulation (OpenSees) Laboratory tool developed by McKenna *et al*. (2014) is used for all the analyses. A force-based nonlinear beam-column fiber element that considers the spread of plasticity along the element is used for modeling the beams and columns for nonlinear time history analysis. Formulation of the force-based fiber element is explained in Lee and Mosalam (2004). Kunnath (2007) has studied the sensitivity due to the number of integration points in each element and suggested the use of five integration points for fiber elements, which is followed in the present study. The core concrete is modelled by considering the effect of confinement due to the special reinforcement detailing in the beams and columns using the Kent and Park (1971) model. The cover concrete is modelled as unconfined concrete. Steel reinforcing bars are modelled using uniaxial Giuffre-Menegotto-Pinto steel material model with isotropic strain hardening. More details about reinforcement modeling used in the present study can be found in Filippou *et al*. (1983). In the present study, a lumped mass approach is considered in which all the permanent weights that move with the structure is lumped at the appropriate nodes. This includes all the dead loads and part of the live loads (25%) which is expected to be present in the structure during the ground shaking. The in-plane stiffness of the floor is modelled using rigid diaphragm constraint. Damping is modelled using Raleigh damping for dynamic analysis, reported by Filippou *et al.* (1992).

The number of ground motions required for an unbiased estimate of the structural response is 3 or 7 as per ASCE/SEI 7-10. However, ATC 58 (2012) recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. ASCE/SEI 41 (2013) suggests 30 recorded ground motions to meet the spectral matching criteria for nuclear power plant structures. Celik and Ellingwood (2010) used 40 ground motions for developing fragility curves. In the present study, twenty-two pairs of ground motions (44 ground motions) are collected from Haselton *et al.* (2012) and the details of the same are available in Haran *et al.* (2015). These ground motions are converted to match with IS 1893 (2002) design spectrum using a computer program (Mukherjee and Gupta 2002) and used for the nonlinear dynamic analyses. Norm displacement increment test criteria is used for the convergence test. The nonlinear dynamic analysis performed in the present study uses three algorithms, namely Newton-Raphson method, Broyden Algorithm and Newton Line Search Algorithm to find the equilibrium at each time step. In some cases, a few (0 to 5% out of 44) computational models are found to be unconverged due to computational instability. The computational models which are failed to converge are ignored in the calculation of probabilistic seismic demand model.

Uncertainties associated with concrete compressive strength, the yield strength of reinforcing steel, and global damping ratio are considered in the probabilistic seismic risk assessment. The mean value and coefficient of variation (COV) of the normal probability distributions of the above parameters (uncorrelated) are obtained from published literature and presented in Table 4.

VALIDATION OF SAC-FEMA METHOD

The present study employs SAC-FEMA method that uses power law assumption and log normality assumption of drift demand which are originally proposed for steel frames that simplify the calculation of the probability of unacceptable performance of the selected frames considering two structure-related assumptions. In order to use this method for GLD RC frames in the present study, it has been validated with more rigorous Monte Carlo simulation (MCS) in line with previous studies (Tsompanakis 2002; Zhang and Foschi 2004; Lu *et al.* 2008; Celik and Ellingwood 2010; Shahraki and Shabakhty 2015). The validation study is carried out on a typical frame (S4) by comparing the fragility curves obtained from both SAC-FEMA and MCS methods.

The accuracy of MCS method depends on the number of samples of random variables considered for the simulation. In order to check the accuracy of the probability of exceedance values obtained from MCS, a convergence study has been conducted by increasing the number of samples (computational models of frame) and MCS was found to be converged for a sample size of 10,000. Random values of material properties are generated as per Latin hypercube sampling technique (Ayyub and Lai 1989) using the parameters given in Table 4. These values are used randomly to create different computational models for the selected S4 frame. The 44 selected ground motions are scaled linearly from 0.1g to 1.0g and each of the computational models is analyzed for a particular earthquake (randomly selected) with a particular *PG*A. A total of 60,000 nonlinear time history analyses are performed (at the final stage) and the maximum inter-storey drift (ISD) for each frame is recorded to obtain the fragility curves using MCS.

For SAC-FEMA method, a set of 44 computational models is developed for the selected frame as discussed above. These 44 computational models are analyzed for a particular earthquake (randomly selected from the set of 44 earthquakes) with a particular *PGA*. A total of 44 nonlinear time history analysis is performed for the selected frame.

The probability distributions for ISD obtained from MCS and SAC-FEMA method are compared in Fig. 1. The fragility curves are further developed using both the MCS and SAC-FEMA method. Figs. 2a and 2b show the comparison of the fragility curves obtained from the two methods at selected performance levels. It is to be noted from Fig. 1 that the log-normal assumption of inter-storey drift demand used by SAC-FEMA method is in agreement with the results obtained from MCS. Fig. 2 shows that the SAC-FEMA method is able to predict the fragility curves with reasonable accuracy. While SAC-FEMA method results in a closed form continuous expression for exceedance probabilities, MCS provides exceedance probabilities at discrete points. It shows that the simplified SAC-FEMA method can yield satisfactory results for RC framed structures with less number of sample sizes and less computational effort.

Considering the computational effort of MCS procedure, the present study uses SAC-FEMA method for all further analysis. Previous researchers (Yun *et al.* 2002, Ellingwood *et al.* 2007, Wu *et al.* 2009, Celik and Ellingwood 2009; 2010, Davis *et. al.* 2010, Haran 2014, Haran *et al.* 2015; 2016, Bhosale *et al.* 2016) have used this method for RC framed buildings.

PERFORMANCE OF GLD BUILDINGS

Probabilistic Seismic Demand Models (PSDM)

The maximum ISDs and the corresponding PGAs are plotted on a logarithmic graph as shown in Fig. 3 for all the selected frames. Each point in the plot represents the PGA values and the corresponding maximum ISD. A power law relationship (Eq. 2) for each frame is fitted using regression analysis, which represents the PSDM for the corresponding frames. The regression coefficients, '*a*' and '*b*', are found out for each frame and reported in Table 5. The PSDM model provides the most likely value of maximum ISD in the event of an earthquake of certain PGA (up to 1g) in each frame. Depending on the PSDM, the vulnerability of the particular frame can be identified. It can be seen from Fig. 3 that GLD frame (G) has the highest drift demand whereas frames designed for seismic loads have the lower drift demands for any given intensity measure (PGA).

Seismic fragility curves

Fragility curves are developed for all the selected frames at different selected performance levels and presented in Fig. 4. The exceedance probabilities of ISD are compared among the selected frames. It can be seen that the order of the frames in terms of decreasing exceedance probabilities is G > S1> S2> S3> S4 for all PGAs and performance levels. The GLD building poses the maximum failure probability among the selected frames. The failure probabilities decrease with the increase in design seismic load.

Drift Hazard curves

Drift hazard is defined as the probability of unacceptable seismic performance of a building in terms of the annual probability of exceedance of performance levels considering the probability of an earthquake at a particular site. The site seismic hazard curves of four locations representing four different seismic zones of India are obtained from NDMA (http://www.ndma.gov.in) as shown in Fig. 5. These hazard curves are used in the present study for the development of drift hazard curves. The selected seismic hazard curves are fitted into the closed form equation (Eq. 5) in a log-log format as shown in Fig. 6 and parameters k_0 and k are found out (Table 6).

The drift hazard curves for all the selected frames are developed as per the procedure discussed previously and presented in Fig. 7. Figs. 7a-7d show the comparison of drift hazard curves for GLD building and building design for seismic forces for four different seismic zones. Each of the four plots in Fig. 7 represents the drift hazard curves for the respective seismic zones of India. The GLD building is found to be more vulnerable compared to the building designed for seismic forces. The increase in vulnerabilities of GLD buildings increases with an increase in the seismic zone/hazard.

Mean Annual Probability of Exceedance

The values of the annual probability of collapse (P_{PL}) or the annual exceedance probability of all the designed frames for selected performance levels are calculated and presented in Table 7. It can be seen from the calculated P_{PL} values that, GLD building is always more vulnerable in comparison to buildings designed to seismic load. The value of P_{PL} for GLD building in Zone V is found to be unity (indicating 100% failure) for some performance levels. In order to understand the relative vulnerability of GLD building, normalized P_{PL} of this building (relative to P_{PL} of building designed to seismic load) is presented in parentheses. For example, normalized P_{PL} of two in Zone-II at the performance level of 1.0% ISD means that GLD building has twice the risk than that of a building designed to seismic load in Zone II. It can be observed from Table 7 that normalized P_{PL} of GLD building can go as high as 100 in the higher seismic zone.

Confidence Levels

The confidence levels of GLD buildings at different seismic zones are calculated for three performance objectives. Details of the selected performance objectives are presented in Table 8. The capacity factor (ϕ), demand factor (γ) and confidence factor (λ) are computed for each performance level to satisfy the condition given by Eq. 17, Eq. 18 and Eq. 19 respectively, and presented in Table 9. It is observed that the capacity factor for both GLD building and buildings designed to seismic load are almost identical as the quality of construction is assumed to be identical for both of these two categories of buildings. However, the demand and confidence factors are significantly higher for gravity load designed building. Substantially higher dispersion in the demands for GLD building (compared to buildings designed to seismic load) results in higher demand factor. Also, a higher value of λ in gravity load design building represents a lower level of confidence in the safety.

The confidence level for achieving the corresponding performance objective for both categories of frames at all seismic zones are presented in Table 9. It can be seen that the confidence levels in meeting the performance objectives for the GLD buildings are consistently lesser than that of buildings designed for the seismic load. The decrease in the confidence levels of GLD buildings increases as the seismic zone level and performance objective level increases. The decrement in the confidence level of GLD building (in comparison with buildings designed for seismic load) is found to be about 1% for Zone II at PO-I, whereas this decrement is found to be 49% for Zone V at same performance objective PO-I. Similarly, for Zone II, the decrement in the confidence level of GLD building increases from 1% in PO-I to 12% in PO-III. In general, the confidence level of GLD building is found to be relatively higher in Zone II and Zone III to achieve a lower level of performance objectives. However, it is significantly lower for higher seismic zones (Zones IV and V). This indicates that performance of GLD buildings in lower seismic zones (Zones II and III) is fairly good, whereas catastrophic performance can be expected from such buildings for higher seismic zones (Zones IV and V).

If the criterion is set such that there must be a confidence of at least 90% that the actual (but uncertain) probability of exceeding the performance level is less than the specific value of the annual probability of performance level not being met, the design requirements for IS 1893 (2002) fails to satisfy this criterion for higher seismic zones (Zones IV and V).

SUMMARY AND CONCLUSIONS

The present study evaluates the relative seismic vulnerability of GLD framed building subjected to seismic hazards corresponding to various seismic zones of India (IS 1893:2002). The vulnerability of a typical four storey GLD building is studied using a probabilistic performance-based approach in terms of fragility curves, drift hazard curves, the probability of unacceptable performance and the confidence levels. Salient conclusions of this study are listed as follows.

- The SAC-FEMA method for the probabilistic assessment of steel buildings has been verified for its applicability to GLD RC buildings through more rigorous MCS method. Results show that the SAC-FEMA method is in reasonably good agreement with the more accurate MCS method to predict the fragility curves. The Results of the MCS method are found to be supporting the log-normality assumption of SAC-FEMA method.

- The GLD building is found to be more vulnerable compared to the buildings designed for seismic forces. The increase in vulnerability of the GLD building (in comparison with buildings designed with seismic force) increases with the level of seismic hazard. For example, a GLD building has twice the seismic risk of buildings designed to seismic load at Zone-II (PGA of 0.1g) at a performance level 1.0% drift. This value of relative seismic risk can be more than 100 for a higher seismic zone (Zone V).
- Confidence levels in meeting the performance objectives for the GLD buildings are found to be consistently less in all the seismic zones than that of the corresponding buildings designed for the seismic load. The decrease in the confidence levels of GLD buildings increases as the seismic zone level and performance objective level increases.
- The results of the present study indicate that the GLD buildings existing in seismic zones IV and V of India should be immediately uninhabited to avoid devastating situations like Nepal (2015) and Kashmir (2005) earthquakes.

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