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Seismic Fragility Assessment of RC Building Using HAZUS Methodology and Incremental Dynamic Analysis

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Abstract

The technique of coupled building strategy is more viable to protect the adjacent buildings of dissimilar in characteristics against seismic hazards. The philosophy of coupled building control in which adjacent buildings are allowed to exerts counter-acting forces one upon another. This study includes the seismic performance of various Coupled Buildings with respect to Uncoupled Buildings for which two models of Coupled Building are considered that is, first one is the two-shear type adjacent buildings connected in-line with MR dampers and second one is same as first with taller building is isolated by Resilient-Friction Base Isolator at its foundation. The seismic response analysis of RC coupled buildings are studied in terms of peak responses subjected to unidirectional excitation due to Kobe 1995 earthquake. The governing equation of motion of various coupled buildings is solved numerically by Newmark's step-by-step method. The dynamic behaviour of semiactive MR damper and R-FBI base isolation system has been predicted by modified Bouc-Wen model and Wen's model respectively. This study employed the Lyapunov direct approach as a control algorithm for the stability analysis and design of semiactive MR controller. The responses of various coupled buildings and uncoupled buildings are simulated through MATLAB® computing software. This study outlined that model of Coupled Building-2 performs more effective in controlling the seismic responses whereas model of Coupled Building-1 performs well not only in reducing the responses but also avoid pounding from the adjacent buildings. Further, there is significant reduction in responses of taller building isolated by base isolation system whereas marginal reduction takes place in shorter building which is not isolated.

Keywords: Seismic performance, responses, Uncoupled Building, Coupled Building, Semiactive MR Damper, R-FBI base isolator, Pounding

1. Introduction

Many developments and advances have occurred over the years in the approach of designing structures which are resistant to loads and actions due to various sources i.e. live load, vehicle load, loads due to natural hazard etc. The demands imposed due to dead, live and moving loads on the structure are mostly deterministic, and when not, the loads are made certain essentially by assuming an appropriate load factor. However, demands imposed due to seismic and natural hazards are mostly uncertain in nature due to their spatiotemporal variations and variations in intensity. In such cases, it is impractical and uneconomical to design structures of given service life for the maximum demand that will be imposed on them. Moreover, the design and analysis of structures for seismic loads is very different from designing for other loads. To reduce the losses and aftermath caused due to earthquake, it is important to analyse the safety and reliability of structures under such an event and assess the level of risk to the structure and its geographical region might be subjected to.

1.1 Performance-based earthquake engineering

For the design of earthquake-resistant structures, most codes rely on redundancy, over-strength and ductility properties of the structure to resist the imposed seismic demand and to maintain the structures' integrity and safety. The design philosophy thus allows for damage of a structure but disallows the complete collapse and damage which is life-threatening to the occupants. Therefore, the performance of the structure and the state it reaches under a given seismic action matters and is of significance not only for design purpose but also for damage estimation, repairability and rehabilitation purposes following a natural hazard. It is necessary to quantify the level of damage that may occur to the existing structures subjected to seismic hazard.

Further, it is important to address the probable performance of the existing structural systems and determine the level of risk they might be subjected to. Performance-based earthquake engineering (PBEE) is a paradigm that aims at ensuring that the structure achieves a desired set of performance objectives defined beforehand. These performance objectives are relevant to various types of stakeholders and should be addressed in building loss estimation procedures. The basic philosophy of performance-based engineering is to generate a mapping between the qualitatively stated performance objectives or observable damage state and the quantitatively measurable

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engineering demand or response parameters (limit states) consistent with the structural analysis principles [1]. The limits states can be defined based on any of these performance criteria of safety, serviceability and/ or repairability [2].

PBEE approach consistent with Pacific Earthquake Engineering Research (PEER) centre's framework, is divided into four core stages i.e. hazard analysis, structural analysis, damage analysis, and loss analysis [3]. The PBEE approach as applied to damage analysis involves probabilistic estimation of building response quantified by an engineering demand parameter (EDP) for a range of given intensity measure IM values and, then, estimation of building performance, defined in terms of damage state (DS) or limit state exceedance probabilities for a given building response [4].

1.2 Seismic Risk and Seismic Fragility

Seismic risk can be defined as the spatial and temporal integration of the product of seismic hazard at that location, the fragility of assets and value of assets [5]. The overall risk (i.e. annual probability of failure) for an individual hazard is evaluated by convolution of hazard curve and the corresponding fragility as following:

$$P_f = \int P_{f|\lambda} \cdot \left|\frac{dH(\lambda)}{d\lambda}\right| d\lambda \tag{1}$$

Wherein, λ is a hazard intensity parameter, $P_{f|\lambda}$ is the fragility curve, and $H(\lambda)$ represents the hazard curve [6]. Thus, fragility is a key component in seismic risk assessment, especially at the regional level. Fragility is the probability of an undesirable outcome as a function of excitation. Simply put, it can be thought of as the probability of breaking or being damaged. Whereas, vulnerability is the degree of loss as a function of excitation. Thus, fragility is measured in terms of probability, while vulnerability in terms of loss such as repair cost and fragility curves can be obtained. The general equation for developing fragility is based on the conditional probability of reaching or exceeding a specific damage state given an earthquake excitation level and is obtained by considering uncertainties in the available physical model of a component or system through the use of empirical, experimental, and/or numerical data.

$$Fragility = P[LS|IM = y] \tag{2}$$

Here, *LS* is the limit state or damage state (*DS*), *IM* is the intensity measure (ground motion), and *Y* is the realized condition of ground motion *IM*.

The quantities involved in the generation of fragility curves are assumed to follow particular distribution models, the fragility of a structural system is most prominently modelled using a two-parameter log-normal cumulative distribution function. Moreover, the variations in capacity (response) or demand (hazard intensity) quantities are computed considering them as log-normally distributed. The seismic curves are then conditioned on various engineering quantities of either demand- PGA, S_a, PGV, PGD; or response- S_d, inter-story or roof drift ratio, etc.

Based on the type and nature of structural response and demand data, fragility curves can be broadly classified as empirical fragility curves and analytical fragility curves. Empirical fragility curves are developed using actual post-disaster damage data. The development of such empirical fragility curves for buildings, following the 2014, Mae Lao earthquake in Thailand is comprehensively illustrated by Foytong and Ornthammarath [6]. While, analytical fragility curves are developed using structural response data from numerical analysis and are based on a certain quantified or assumed relation between the response and demand quantities. The analytical fragility curves are often described in parametric forms. This study focuses on analytical fragility curves.

1.3 System-level and Component-level Fragility

The above stated PBEE approach can be applied to the entire structure as a whole or on the component/s of the structure. As per the damage quantification using the PBEE concept for developing fragility curves, the probable damage can be computed given a threshold/ limit state and a measurable or computable engineering quantity. When the performance objective can be related to local or component damage, a limit state based on member or connection strength or deformation is required. This gives rise to component-level or local fragility curves. Whereas, system-level fragility deals with the conditional probability of the system as a whole being in or reaching a specified limit state or damage threshold [1]. The engineering demand parameters for a system commonly include maximum roof drift ratio or maximum inter-story drift. Since the failure of a particular component does not necessarily mean the failure or collapse of the complete system as a whole, therefore we can say that the damage curves based on components can give a pessimistic view of the capacity or strength of the structure. Thus, structural performance levels for system-level fragility and reliability analysis should not be based on a single component, it should rather be based on the system EDPs or on a proper combination of multiple components' fragility or reliability curves to obtain resultant system behaviour. The latter type of analysis, i.e., at a system-level using component-level damage information can be carried out by the fault-tree analysis under the Bayesian statistics framework [7]. In case of an integrated structural system analysis, consisting of primary and secondary structural systems, it is suggested to consider interactions and account for coupling between the various infrastructure/ building component for realistic fragility assessment and to avoid conservative outcomes [7]. Further, selection of appropriate structural system-level performance indicators which can be extended to life-cycle performance, risk, safety and reliability metrics of structural systems form a core part towards studying and analysing not only resilient infrastructure system but also communities [8]. Fragility curves are thus a robust tool to determine, predict and analyse the structures' probable performance. In addition to generating fragility curves for existing structures, they can also be derived robustly at the design phase of the structures under construction to facilitate simplified comparison of performance of various design solutions [9].

The objective of this study is to derive analytical fragility curves using the HAZUS [10] methodology and from incremental dynamic analysis (IDA) [4]. The differences, scope and applicability of these methods are presented using studies performed on a hypothetical, six-story reinforced concrete frame building. The aim is to also understand the variation in fragility curve parameters resulting from the application of the above-stated approaches, how the probability of failure is conditioned and their suitability at a given level of damage, risk analysis and loss estimation.

2. Methodology and Model Details

For the purpose for estimation of losses incurred, the degree and type of rehabilitation, repair required and to predict the suitability of the building following a hazard, a proper framework is needed that will yield the required degree of damage and reliability of the structure prone to disaster and quantify the same. Since each building and structural system is unique, there arises a need for reliability assessment of each structure in a risk-prone area. This can be considered as an impractical approach. To solve this problem, we have the HAZUS methodology [10]. This methodology generates regional and stock fragility curves for a given group of buildings with common characteristics subjected to potential earth science hazards (PESH). However, in cases of critical structures and facilities, high-rise buildings whose damage will lead to a great extent of structural and economic losses, individual building-specific fragilities or damage curves are required in addition to the information about their dynamic capacity. Moreover, to perform retrofitting and repair of structures, one needs to quantify the structure-specific probable damage that will be incurred. Therefore, asset or structure-specific fragility curves and reliability information are required. Lastly, to produce valuable information for generating pre-disaster mitigation and emergency response plans, fragility curves are needed to be developed.

2.1 Model Details

This study deals with the analytical seismic fragility curve generation using IDA methodology [4]. Both these methods are applied on a six storied reinforced concrete building with moment-resisting frame type (MRF). The building and site properties are given in Table 1 and seismic-resistant design, as per the response spectrum method as per IS 1893 (Part 1): 2016 [11] and analysis of the building is carried out in SAP-2000 software [12]. The designed member sizes (in mm) are as follows: beam 230 × 300; column 400 × 500; slab thickness 120; outer and inner wall 230 and 115, respectively. Fig. 1 shows the plan and elevation of the building considered in this study. The plan dimensions are 12 m × 10 m, the overall building height is 18 m (story height 3 m). Table 1 states the site properties and dynamic characteristics of the building. Fig. 2 shows the capacity pushover curves for this building in X and Y-direction. The model is designed as per the strong-column weak-beam design provision of the Indian Standards and to represent the recent stock of buildings designed for seismic loads, mostly located in the urban centres. Such buildings have increased collapse capacity and global ductility, which finally leads to the reduction in collapse fragility [13].

Table-1. Building and site properties used in the present investigation

Seismic Zone	Soil Type	1^{st} Fundamental Period- T_Y (s)	2^{nd} Fundamental Period T_X (s)
Zone 3	Medium Stiff	1.587	1.322

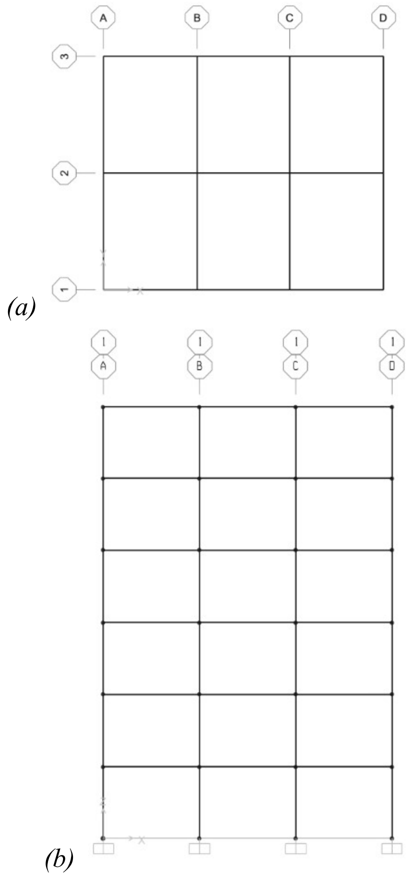

Fig. 1. Details of building used in the present analysis (a) plan and (b) elevation

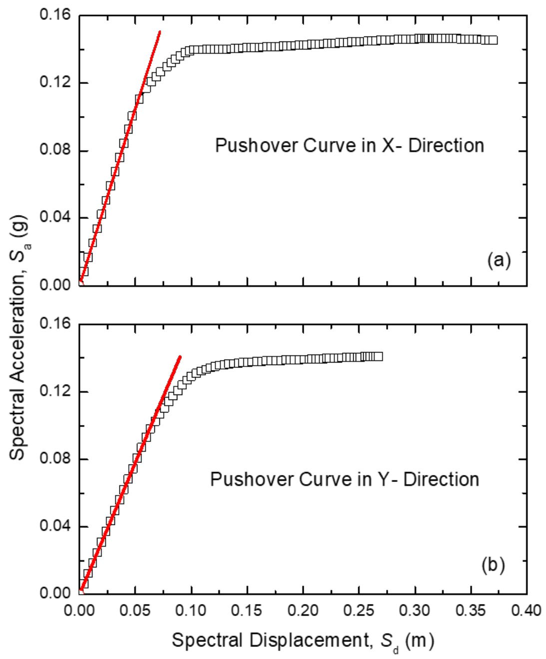

Fig. 2. First mode spectral capacity curves in (a) X-direction and (b) in Y-direction

2.2 HAZUS Methodology

The primary purpose of the HAZUS methodology is to develop a procedure for making earthquake loss estimates at a regional scale. It also presents a method for developing a set of fragility curves for a class of structures subjected to earthquake hazard. Many extensions of the HAZUS methodology have been developed to extend the applicability of this methodology at different geographical locations [14] and for the structures' asset class. One such study conducted by Tafti et al. [15] addressed the development of reliable fragility curve parameters for buildings in Iran. HAZUS earthquake damage functions consist of (1) capacity curves and (2) fragility curves as its 2 components. The capacity curve is dependent upon the nonlinear pushover behaviour, building's dynamic property and it is based on the engineering parameters that characterise the above-mentioned behaviour of the building [10]. In this study, the model corresponds to the building type C1M as per the HAZUS classification, i.e., corresponding to midrise reinforced concrete moment resisting frame. The seismic design level of the building is considered as moderate code. In this method, building damage functions are in the form of lognormal fragility curves relating the probability of being in or exceeding, a building damage state for a given demand parameter, response spectrum displacement in this case [10]. Also, HAZUS [10] uses lognormal distribution for capturing variations in capacity and demand. The fragility is quantified based on the following equation

$$P[d_s|S_d] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \quad (3)$$

Here, $\bar{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds; Φ is the standard normal cumulative distribution function; and β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state, ds describing the total variability and it is defined as

$$\beta_{ds} = \sqrt{\mathrm{CONV}(\beta_C, \beta_D)^2 + \beta_T^2} \quad (4)$$

Where β_C is the lognormal standard deviation parameter that describes the variability of the capacity curve; β_D is the lognormal standard deviation parameter that describes the variability of the demand spectrum and; β_T is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state, *ds*.

This two-parameter fragility definition requires estimation of damage state thresholds corresponding to a given damage state and the total damage state variability. In this study, the curve parameters are obtained using 2 approaches, the first one is as per HAZUS-MH MR5 [10] and the second one is as per HAZUS-MH MR1 [10]. For the purpose of nomenclature, in this study, the first method is named HAZUS (HZ) method and the second as Method M1, respectively. Method M1 is partly based on the study performed by Patel and Vasanwala [16].

Table-2. Damage state variability for M1 with $\beta_C = 0.3$ and $\beta_{T,\,ds} = 0.4$

Damage State	Degradation (k)	β_{ds}
Slight	Minor (0.9)	0.75
Moderate	Major (0.5)	0.85
Extensive	Extreme (0.1)	1
Collapse	Extreme (0.1)	1

Damage state thresholds and total variability for HAZUS method are determined according to HAZUS-MH MR5 [10]. For method M1, the total variability is determined as per HAZUS-MH MR1 [10], considering the degradation factor and other parameters as stated in Table 2 [16].

Further, for method M1 the capacity curve control points are found from HAZUS [10] and the damage state thresholds are in accordance with the study performed by Barbata et al. [17] and are given in Table 3.

2.3 Incremental Dynamic Analysis

Incremental dynamic analysis (IDA), also known as the dynamic pushover capacity analysis, uses real or synthetically generated ground motion (GM) records. The IDA can be a single record or multi-record IDA. Multi-record IDA is used for this study and these ground motions are scaled at different intensities and applied to the model under the nonlinear time history analysis regime and the analysis at different intensity levels is carried until the structure reaches the state of incipient collapse or complete collapse. The analysis is performed as per Vamvatsikos and Cornell [4] and as per the guidelines given in ASCE 7-16 for nonlinear response history analysis [18].

The method involves the following steps: (1) selection of ground motion: ASCE 7-16 recommends using a suite of minimum 11 different ground motion (GM) records. Also, according to Vamvatsikos and Cornell [4], for mid-rise buildings records between 10- 20 are sufficient for estimating seismic demands on a structure. Thus, 11 real earthquake records were selected from records obtained from the PEER Ground Motion Database. (2) Ground motion scaling: Here records were scaled using the amplitude scaling method as mentioned in ASCE 7-16 [18]. Finally, 11 suitably scaled records were obtained such that their average does not fall below 90% of the target response spectra [19]. Fig. 3 shows the design, target (MCE) spectra and the average spectra of the scaled GMs. Fig. 4 shows the spectra of the 11 unscaled GM records at 5% damping and its average.

The fragility curve plotted using this method uses maximum roof drift ratio as the response quantity and spectral acceleration (S_a (T_{1x}, 5%)) as the IM against which the curve is plotted. However, to reduce the degree of scatter inherent in the record-to-record variability of the GMs, the IM values for plotting fragility curves must be selected based on a few structure-specific criteria, degree of non-linearity experienced and the angle of incidence of the ground motion

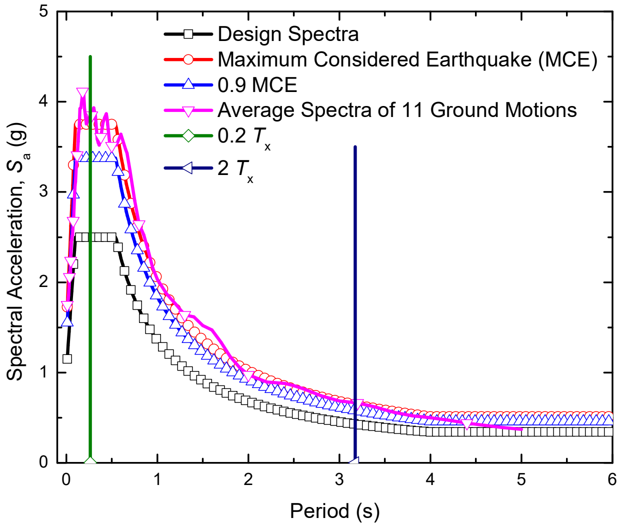

Fig. 3. Design spectra and average spectra of 11 scaled records

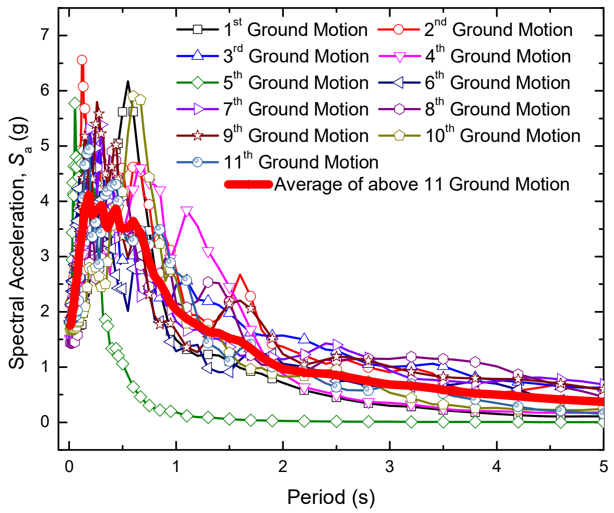

Fig. 4. Spectra of selected ground motions and its average

with respect to the structural axis [20]. In this study, the spectral acceleration- S_a (T_{1x}, 5%) can be considered as an efficient IM as per the conclusion from the study of Kostinakis and Athanatopoulou [20] and the findings of Jeong et al. [21]. Equation [5] gives the formulation of fragility used in this case i.e., for fitting the IDA response data. This is again a two-parameter definition of fragility.

$$F(x) = \Phi\left(\frac{\ln(x/\mu)}{\beta}\right) \quad (5)$$

F(x) represents the continuous fragility; Φ is the standard normal cumulative distribution function; x is the EDP (S_a in this case); μ is the limit state or performance level and β is the total standard deviation. The fragility curves are derived from the IDA response data using the definitions of the parameters of the fragility curve from Wen and Ellingwood [22]. The value of total standard deviation is given as [22].

$$\beta = \sqrt{\beta_{D|Sa}^2 + \beta_C^2 + \beta_m^2} \quad (6)$$

Here, $\beta_{D|Sa}$ is the demand uncertainty, β_C is the capacity uncertainty and β_m is the modelling uncertainty.

The qualitative performance levels as per FEMA- 356 [23] are considered. They are immediate occupancy (IO), life safety (LS) and collapse prevention (CP) [23]. In terms of drift values, these are 1%, 2% and 4%, respectively. These suggested limits are approximate and for our model reduced drift values as 0.5%, 1% and 2% are also considered [24]. Besides using this method, the probability of failure can also be determined non-parametrically as the relative frequency of the peak displacement exceeding the specified drift limit. This has the advantage of not requiring a particular distribution function be fit to the peak displacements [1].

3. Results and Discussions

3.1 Fragility curves propagated using HAZUS methodology

The fragility curves were generated using two different approaches to find the curve parameters-damage state median (threshold) and overall standard deviation. The methods are HAZUS method and Method M1. Control points according to M1 from the first mode pushover curve along X-direction are mentioned in Table 3. Whereas, Table 4 shows the curve parameters according to HAZUS methodology and method M1.

Table-3. Damage state thresholds as per Barbata et al. [17]

Control Point	**0.7D_y**	**D_y**	**D_y+ 0.25(D_u - D_y)**	**Du**
Damage State	Slight	Moderate	Extensive	Complete
Displacement S_d (mm)	0.0355	0.05067	0.1183	0.3213
Acceleration, S_a (g)	A_u	0.1065	A_y	0.1464

Table-4. Curve parameters according to M1 and HAZUS methodology

	Method M1				**HAZUS Method**			
Damage state	**Slight**	**Moderate**	**Extensive**	**Complete**	**Slight**	**Moderate**	**Extensive**	**Complete**
Threshold S_d (m)	0.0355	0.0507	0.1183	0.3213	0.0439	0.0775	0.1965	0.3699
Total Variability	0.75	0.85	1	1	0.7	0.7	0.7	0.89

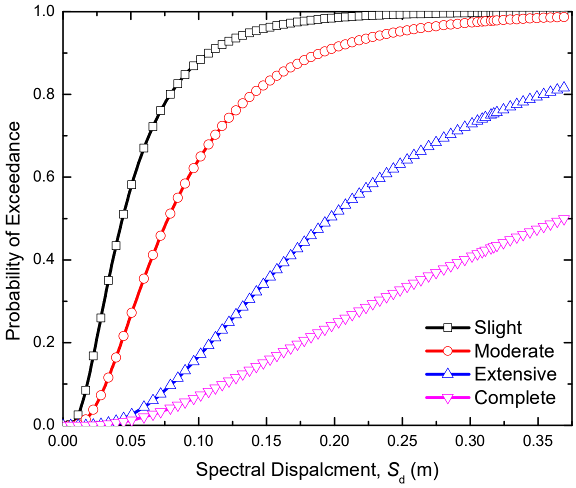

Fig. 5. Fragility curve using HAZUS (HZ) method for different damage levels

Fig. 5 shows the fragility curve for 4 damage states as per the HAZUS Method. Whereas, Fig. 6 depicts the comparison of curves obtained from the two methods for each damage state individually. It is observed that the curve parameter values- damage thresholds (limit states), and total variability are different in both cases. Both the procedures are different in that the HAZUS method is based on a more general set of methodology applicable to a group of assets, as per the HAZUS MR5 manual [10]. Whereas method M1 is as per the HAZUS MH- MR1 AEBM [10], which describes the procedure for developing building-specific damage functions by extending the more general methods of HAZUS MR5.

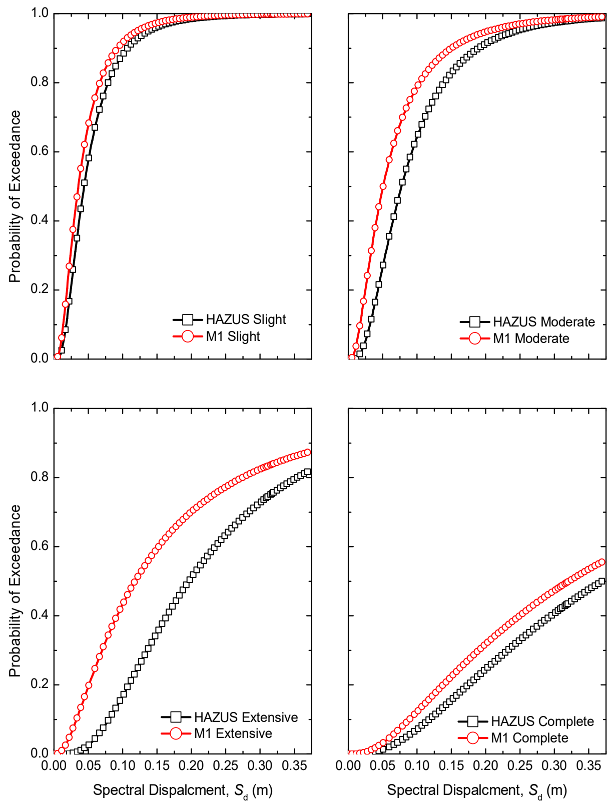

Fig. 6. Comparison between M1 and HAZUS method for different damage thresholds

3.2 Building specific fragility curves using Incremental Dynamic Analysis

Nonlinear time-history analysis along the X-direction is carried out for the building considered herein. Fig. 7 shows the IDA response curves for each GM record. Probabilistic demand models of roof drift ratio and spectral acceleration are modelled using a bivariate regression analysis using a power trend line [22]. The relationship between median drift and S_a in the log space is shown in Fig. 8. The overall uncertainty (β) from eq. (6) is found to be 0.37004. The fragility curves using the two sets of performance levels mentioned in section 2.3 are plotted and shown in Fig. 9.

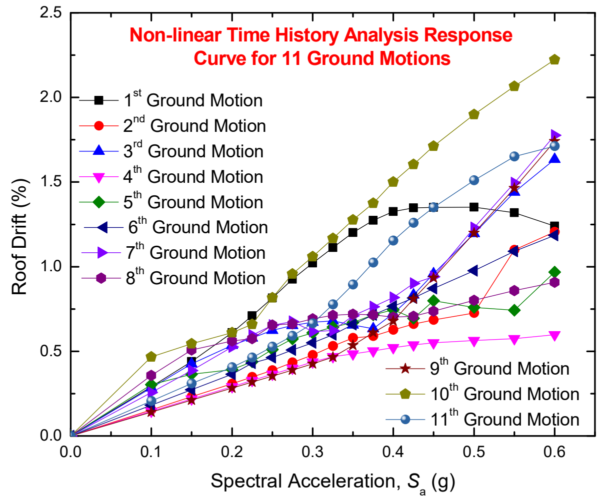

Fig. 7. 11 IDA response-demand curves for the ground motions considered in this study

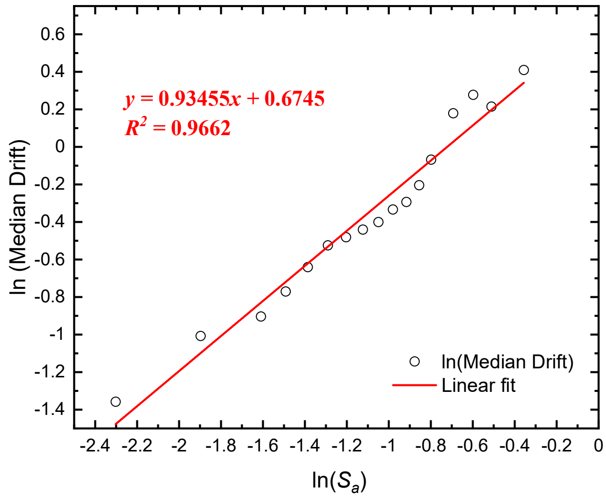

Fig. 8. Median response (roof drift) for each record vs. S_a in log-log space showing linear regression line and R^2 coefficient

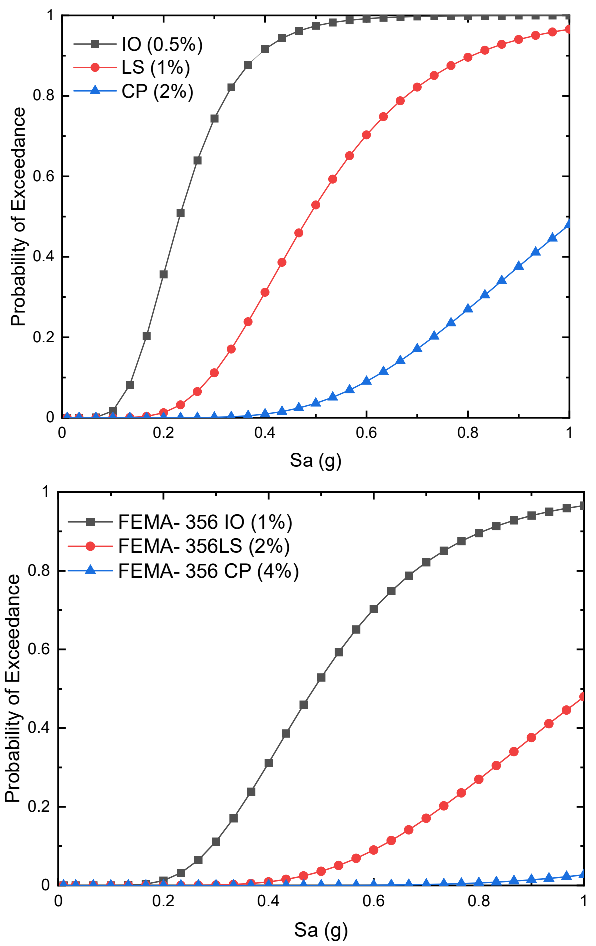

Fig. 9. Analytical fragility curve using reduced FEMA-356 performance levels [24] and FEMA- 356 levels [23]

4. Conclusions

The role of performance-based earthquake engineering is vital in damage and reliability studies of structures subjected to earthquake loads. Further, PBEE can also be used in the design stage of important structures. The fragility curves are also a fundamental and important tool used in the framework of damage, reliability, vulnerability and loss estimation. They are a basic component in seismic performance and risk assessment and consequence-based engineering. These curves were developed for a reinforced concrete building model, as these class of structures form a majority of important structures in this country, in an urban area. These curves coupled with the hazard curves provide a basis for assessing the nationwide risk of earthquake losses and for suitably mitigating the consequences due to it. Two methodologies are illustrated on the model for generating analytical fragility curve- the HAZUS methodology and incremental dynamic analysis method.

The HAZUS methodology incorporates engineering judgment and relies on observed and experimental data in finding the values of the damage thresholds and standard deviation. Also, the building capacity curves developed in HAZUS are based on engineering design parameters and judgment for a class of structures based on their material height, seismic design level, etc. The curves derived from IDA are model and site-specific fragility curves.

It was observed that the standard deviation for the curve plotted using the IDA method was lower than the values mentioned in the HAZUS manual. This because the HAZUS curves being present for stocks of buildings and assets have higher levels of uncertainties as compared to structure-specific uncertainties. The HAZUS curves are conditioned on the response of the structure as they use spectral displacement (response) as their engineering demand parameter, while the IDA curves used in the above method are conditioned on hazard and use spectral acceleration or peak ground acceleration as their engineering demand parameter. The HAZUS curves are thus applicable for a portfolio of assets and the method can be further used for propagating building-specific response conditioned fragility curves as illustrated in Method M1. These regional-level and stock loss estimates would be used primarily by local, state and regional officials to plan and stimulate efforts to reduce risks from earthquakes and to prepare for emergency response and recovery. The building-specific reliability analysis though computationally demanding generates extensively informative results about the building's seismic response and is useful for reliability and safety analysis of important structures vulnerable to natural hazards and for estimating the related losses.

Disclosures

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