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SEISMIC DAMAGE PREDICTION OF MASONRY INFILLED RC FRAME BUILDINGS IN BHUTAN BASED ON FUZZY PROBABILITY ANALYSIS

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Abstract

Geographically, Bhutan is located right on the inter-plate boundary in the Himalaya where the Indo-Australian plate is continuously being subducted under the Eurasian plate. A number of big earthquakes have occurred in Bhutan and the neighboring region causing a huge loss to properties and human lives. There are numerous masonry infilled RC buildings in Bhutan whose performance under the seismic excitation is literally unknown. The need for predicting the realistic damages of these buildings has been dearly felt in the country for quite some time. In this study, three typical masonry infilled RC buildings that represent the general building stocks in the country are considered for the prediction of damages. Unlike in many studies where the damage of buildings are assessed based on the distinct interstorey drift limits, probability and fuzzy set theory are used in this study. The ground motions predicted in Thimphu from the probabilistic Seismic Hazard Analysis are used for the prediction of structural responses. The soil-structure interaction is incorporated using the uncoupled spring support. It is found that the masonry infilled RC buildings in Bhutan could mostly experience repairable to irreparable damage under the 475 return period ground motion and severe damage to even collapse under the 2475 return period ground motion.

Keywords: Bhutan; masonry infilled frame; fuzzy probability; interstorey drift; soil structure interaction

1. Introduction

Since the start of its construction in the early 1970s, masonry infilled RC frame buildings have become the most popular form of constructions in Bhutan. As shown in Fig. 1(a), all types of buildings in urban Bhutan such as residential, commercial and institutional are mostly built of RC frames with masonry infill walls. The traditional stone masonry buildings which once dominated the urban centers in Bhutan have now reduced to a minority. Moreover, with the improvement of the living standard of people, the construction of masonry infilled RC buildings have also penetrated into the rural areas. However, tracing back the construction history of the country and also observing the current construction scenario, these buildings pose a real risk to the country. The country still has no seismic design code of its own. The Indian seismic code was adopted only in 1997 simply due to its location in the same geographical area as shown in Fig. 1 (b). As such, the buildings built prior to 1997 were either based on some thumb rules or on the intuitive judgement of the building owners and site supervisors. Although the buildings built after the adoption of Indian seismic code are supposed to have been designed for lateral load, yet even today these buildings are generally designed as bare frames. Many studies in the past such as Murty and Jain [1] and Crisafulli and Carr [2] highlighted the drastic change in structural responses with the addition of the infill wall. Moreover, the applicability of using Indian seismic code for the design of buildings in Bhutan is still in question. Hence, there exists a total uncertainty in the performance of masonry infilled RC frame buildings in Bhutan under the seismic action. This is further aggravated by the high seismicity of the country which sits right on the interplate boundary of Indo-Australian and Eurasian plates in the Himalaya. The most damaging earthquake that occurred in Bhutan in the recent memory is M6.1 Monger earthquake in 2009 which killed 13 people and damaged hundreds of rural homes, school buildings and monasteries. The 2011 M6.9 Sikkim earthquake in India was also equally disastrous, damaging hundreds of buildings in the western Bhutan.

Since these earthquakes occurred in the rural areas which are typified by stone masonry and adobe structures, no severe damages were reported on the masonry infilled RC building in the urban areas. It was also reported that the big Himalayan earthquakes such as 1897 M8.7 Shillong Plateau, 1934 M8.3 Bihar-Nepal border, 1947 M7.7 upper Assam and 1950 M8.6 Arunachal Pradesh earthquakes which occurred some epicentral distance away from Bhutan were heavily felt in Bhutan leading to the damages of numerous building structures [3]. The recent M7.8 Nepal earthquake in 2015 forced many people out of the buildings although no major damages were reported. This suggests that earthquakes of all sizes can occur in Bhutan and is just a matter of time. In fact, Bilham et al. [4] already reported an overdue of one or more big earthquakes in the Himalaya based on the evidence such as seismic gap hypothesis and GPS measurements of control points. Hence, assessing the performance of masonry infilled RC buildings has become very crucial and the only way forward for understanding the performance and effectively mitigating the seismic risk in Bhutan.

(a) (b)

Fig. 1- (a) Masonry infilled RC buildings in Thimphu and (b) Geographical location of Bhutan.

On the other hand, the performance of buildings is mainly assessed based on the distinct interstorey drift limits. Based on the interstorey drift values, many existing guidelines such as ASCE-41 [5], ATC-32 [6] and Vision 2000 [7] have specified the corresponding performance levels of the buildings. In reality, it is not logical to assess the performance of the buildings based on the single interstorey drift value and on the distinct interstorey drift limits. There are many uncertainties arising from the modelling options, ground motion, material and geometrical parameters which are either random or a fuzzy events [8]. These uncertainties result in a large variation of structural responses. For instance, the material parameters such as compressive strength of concrete and yield strength of steel and the geometrical parameters such as the size of the beams and columns inevitably vary from their design values due to quality control during construction and deterioration during service. Hence, the interstorey drift estimated from the design parameters of the buildings could vary to a large extent. Moreover, damage of the structure is a continuous process and cannot be matched to a fixed interstorey drift value. At a particular interstorey drift, the corresponding damage may or may not occur to the structure. In fact, as reported by Zhao et al. [8], damage criterion is commonly regarded as a fuzzy and more logically modelled as fuzzy event. On the other hand, material and geometrical parameters are generally considered as random and more appropriately modelled as random events. Therefore, it is more realistic and practical to assess the performance of buildings considering the randomness in structural parameters and fuzziness in damage criteria.

This study is aimed at realistically assessing the performance of the masonry infilled RC buildings in Bhutan taking into account both the randomness in structural parameters and fuzziness in damage criteria. Three typical masonry infilled RC buildings representing the general building stocks in Bhutan are considered for the performance assessment. Ground motions predicted at the generic soil sites in Thimphu from the Probabilistic Seismic Analysis (PSHA) for the return periods of 475 and 2475 years are used for the assessment. For more realistic prediction of structural responses, openings and foundation flexibility of the buildings are considered in the analysis. Rosenbluth Point Estimate Method (RPEM) [9] is used for modelling the material and geometrical parameters and the computer program Perform 3D is used for predicting the structural responses of the buildings. The Monte Carlo Simulation (MCS) technique is used for verifying the accuracy of RPEM and also for determining the statistical distribution of the random structural parameters. Finally, fuzzy probability analysis is used to estimate the damage probabilities of the buildings based on the interstorey drift limit proposed by Ghobarah [10] and triangular membership function. From this study, it is found that masonry infilled RC buildings in Bhutan could suffer repairable to irreparable damages under the 475 year return period ground motion while severe damage and even complete collapse are predicted under the 2475 year return period ground motion. As expected, the buildings designed according to Indian Seismic Code perform much better than the building built prior to the adoption of Indian Seismic Code.

2. Ground Motions

Being located on one of the most active seismic regions in the world, earthquakes of various sizes have occurred in Bhutan. As reported by Dorji [11], there were 32 earthquakes of engineering significance occurred in Bhutan during the last eight decades. However, there are no acceleration time history records available for these earthquakes. In this study, the ground motions predicted at the generic soil sites in Thimphu by Hao and Tashi [12] for 475 and 2475 year return periods are used. The ground motions were predicted from the Probabilistic Seismic Hazard Analysis (PSHA) using 18 seismic source zones located within the distance of 400 km from Thimphu, Bhutan. The response spectra of ground motions at generic soil sites for 475 and 2475 year return periods are respectively shown in Fig. 2.

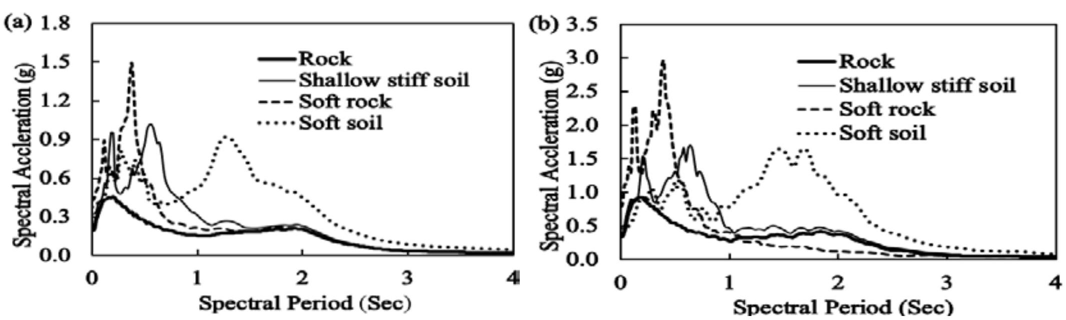


Fig. 2 – Ground motion response spectra at generic soil sites for (a) 475 and (b) 2475 year return periods.

3. Typical Buildings in Bhutan

To realize the performance of the masonry infilled RC buildings in Bhutan in general, three typical masonry infilled RC buildings namely '6 storey', '3 storey new' and '3 storey old' representing the general masonry infilled RC buildings in Bhutan are considered for the study. 6 storey buildings are very common in the core areas of Thimphu city while 3 storey buildings are common in all urban centers in Bhutan and those constructed in some pockets of the rural areas. '6 storey' and '3 storey new' buildings were built after the adoption of Indian seismic code in the country and hence were designed according to the Indian seismic code. They are the real structures currently standing in Thimphu and represent the buildings built after the adoption of Indian seismic code. The details of these buildings are obtained from the Thimphu municipal corporation in the form of structural and architectural drawings. On the other hand, '3 storey old' represents the masonry infilled RC buildings that were built prior to the adoption of Indian seismic code in 1997. Since buildings were built without any kind of design prior to the adoption of Indian seismic code, no credible structural or architectural drawings are available for those buildings. As such, the plan and elevation of '3 storey old' building are adopted identical to the '3 storey new' building. However, the structural details of '3 storey old' building are obtained from the result of the non-destructive test carried out for 15 such buildings in Thimphu [13]. It was observed that dimensions of beams and columns, concrete strength, the yield strength of reinforcement and foundation details are very similar for all 15 buildings. Similar details are assumed for the '3 storey old' building. The floor plans of the considered buildings are shown in Fig. 3. The loading and reinforcement details of the three typical buildings can be found in Thinley and Hao [14]. The size of beams and columns, the thickness of infill wall and other dimension details are given in Table 1.

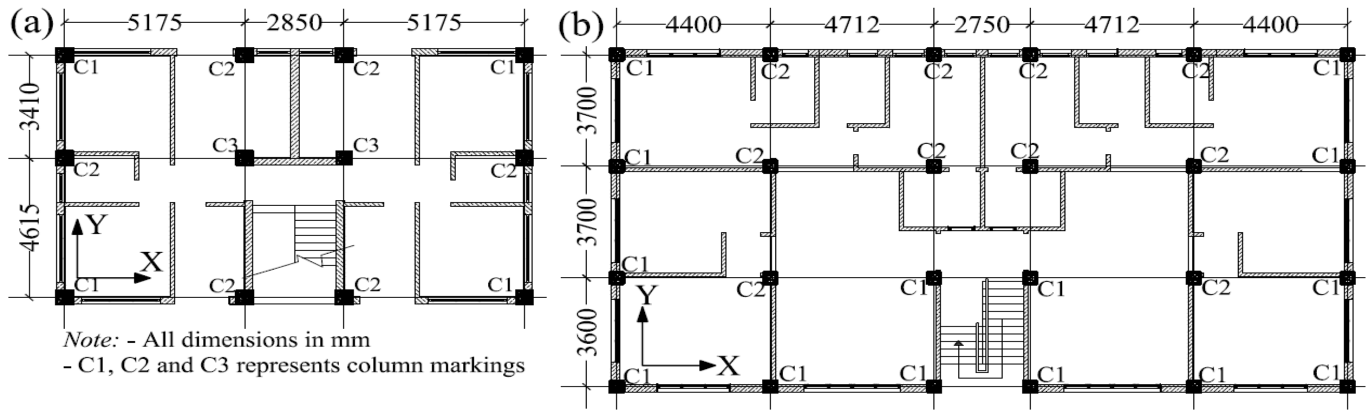


Fig. 3 – Column and masonry infill wall layout plan for (a) 6 storey and (b) 3 storey masonry infilled RC buildings

Table 1 –Member dimensions of typical buildings

Members	**6 storey**	**3 storey new**	**3 storey old**
Column, C1 (bxD) in mm	450x450	400x400	250x250
Column, C2 (bxD) in mm	450x450	400x400	250x250
Column, C3 (bxD) in mm	500x500	-	-
Beam along longer direction, (bxD) in mm	300x450	300x400	250x350
Beam along shorter direction, (bxD) in mm	300x400	300x350	250x300
Thickness of exterior infill wall, mm	250	250	150
Thickness of interior infill wall, mm	125	125	125
Slab depth, mm	150	150	100
Storey height, mm	3060	3060	3060

4. Numerical model

Under the lateral load, masonry infilled RC frames exhibit a highly nonlinear behavior resulting in a number of failure modes. This is mainly due to the interaction of infill wall and the surrounding frames which are made up of materials with varying mechanical properties. The presence of openings in the infill walls and the geometrical distribution of infill walls in the RC frames further results in a highly complex behavior. Hence, modelling the masonry infilled RC frames to simulate these complex behaviors has been a great challenge to many researchers. This is the very reason which forced design engineers to treat masonry infill wall as non-structural and in turn ignore in the design. Basically, modelling of masonry infilled RC frames consists of the masonry infill wall and the surrounding RC frames which when combined is expected to capture the nonlinear response of the masonry infilled RC frames. The numerical model used in this study was previously calibrated with the experimental results in Thinley and Hao [14] and is briefly discussed below.

Chord rotation model is used to numerically model the nonlinear behavior of RC beams and columns. It consists of stiff end zone and a plastic hinge connected to the elastic beam column/column segment. The plastic hinge together with elastic beam/column segment is referred to as FEMA beam/column component. The trilinear force-deformation (F-D) relationship implemented in Perform 3D program is used for the numerical simulation. The F-D relationship is defined by stiffness, strength and deformation capacity of the RC component as shown in Fig. 4 (a). The effective stiffness, K_e of the RC component highly influence the structural response. Many guidelines and studies [15-19] define effective stiffness as some percentage of the gross flexural stiffness. In this study, effective stiffness is obtained from the expression given by Elwood and Eberhard [19] and range from $0.2EI_g$ to $0.7EI_g$ depending on the axial load. EI_g is the gross flexural stiffness, where E is the modulus of elasticity and I_g the gross moment of inertia of the RC member. The yield strength, M_y and yield rotation, θ_y are estimated from the expression of Panagiotakos and Fardis [17] and the maximum or capping strength, M_c is taken as $1.13M_y$ as recommended in the same study based on a number of experimental test results. The residual strength, M_r is taken as $0.01M_y$ as recommended in the Perform 3D user guide and in other studies. The pre-capping, θ_y and post-capping θ_y rotations are estimated from the expressions given by Haselton et al. [18]. The exterior and interior beams are respectively approximated as L and T beam to account for the contribution of the slab in the analyses. The effective width of the beams is obtained from ACI 318R-02 [16] based on the span and overall depth of the beams.

The commonly used single diagonal strut model is employed in this study to model the masonry infill walls. It is computationally inexpensive and capable of capturing the global response of the infilled frame with sufficient accuracy. The F-D relationship of the diagonal strut model is shown in Fig. 4 (b) and is defined by strength, stiffness and deformation capacity of the infill wall. In general, the diagonal length, thickness and material of the diagonal strut are taken the same as that of the infill wall. However, there are various expressions proposed by different studies to estimate the width of the diagonal strut. The studies such Holmes [20], Paulay and Priestley [21] and Penelis and Kappos [22] empirically define width as some fraction of the diagonal length. Some studies estimate the width of the diagonal strut based on the relative stiffness of the infill wall [23-26]. In this study, the width of the diagonal strut is estimated from the expression given by Mainstone [25] which also is adopted in FEMA 274 [27]. The F-D relationship developed by Panagiotakos and Fardis [28] and modified in Dolsek and Fajfar [29, 30] is used to define force, displacement and stiffness parameters of the infill wall. The initial stiffness, K_i and the yield strength, F_y of infill wall are respectively given by Eq. (1) and Eq. (2) as given below.

$$K_i = \frac{G_w L_w t_w}{H_w} \tag{1}$$

$$F_y = f_{tp} t_w L_w \tag{2}$$

where G_w, L_w, t_w, H_w and ftp are shear modulus, length, thickness, height and cracking strength of the infill wall respectively. The maximum strength, F_{max} and residual strength, F_r of infill wall are assumed to be $1.67F_y$ and $0.01F_y$ respectively. As given in Dolsek and Fajfar [30], the displacement at maximum strength, D_{max} and at collapse, D_u are taken as 0.25% of the diagonal length and $5D_{max}$ respectively. These details are used to define the F-D relationships of RC members and infill walls which were found to provide a very good agreement with the experimental results validated in Thinley and Hao [14].

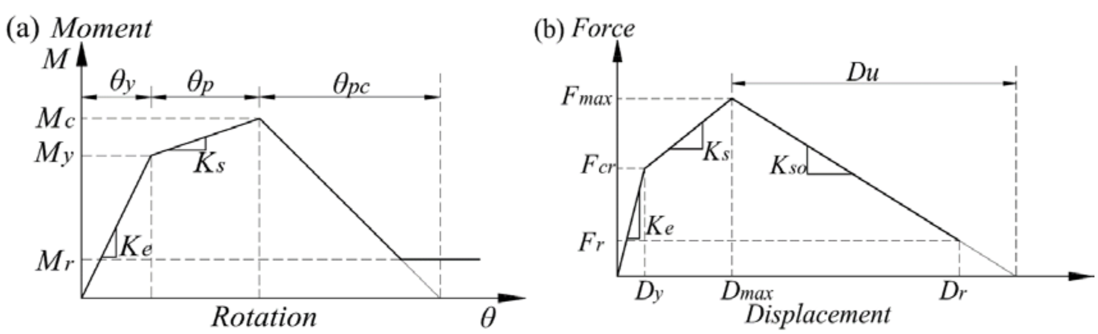


Fig. 4 – Force-deformation relationship of (a) 6 RC members and (b) 3 masonry infill walls.

The FD relationship described above is defined and validated for the solid infill wall. To assess the performance of the masonry infilled RC buildings more realistically, effects of openings and soil-structure interaction (SSI) are included in the analysis. The presence of opening results in the reduction of stiffness and strength of the infill wall. Many researchers have proposed reduction factor mostly as a function of opening and the infill wall areas [31-35]. This reduction factor is multiplied by the effective width of the diagonal strut to estimate the strength and stiffness of the infill wall with the opening. In this study, the reduction factor proposed by Durrani and Luo [33] is used for the same since it was found to be more or less the mean of reduction factors proposed by other researchers. An uncoupled spring support is introduced at the soft soil site to study the effect of SSI. The stiffness of the soil is estimated as the product of soil stiffness at the surface and the embedment correction factor as given in ASCE/SEI-41 [5]. The geometric nonlinearity in the form of P-delta effect is also included in the analysis.

5. Estimation of Probabilistic Structural Response

5.1. Consideration of uncertainties

Estimating the structural responses probabilistically is more logical owing to the consideration of a number of uncertainties. Among the number of uncertainties, only material and geometrical uncertainties are considered in this study. The other uncertainties such as modelling are not expected to have a significant effect since the numerical model used in this study was previously calibrated with the experimental results. The ground motions used in this study are considered as deterministic as they were specifically predicted for the site conditions in Bhutan.

The material uncertainties considered in this study are the compressive strength of masonry wall (f_m), the compressive strength of concrete (f_c) and the yield strength of steel (f_y) since they are found to significantly influence the structural responses amongst the other parameters. The modulus of elasticity of masonry wall and concrete are also empirically related to the compressive strength of masonry wall and concrete respectively. The depth (D) and width (b) of beams and columns and the thickness of main and partition masonry walls are the geometrical parameters considered in this study. The design values of these parameters are taken as mean while the coefficient of variation (CoV) and probability distribution have been taken from the number of past studies and guidelines. The material and geometrical uncertainties considered in this study along with the CoV and distribution types are given in Table 2.

Table 2-Mean and coefficient of variation of material and geometrical parameters

	6 storey building		3 storey new buildings		3 storey old building		
Variables	**Mean (MPa)**	**CoV**	**Mean (MPa)**	**CoV**	**Mean (MPa)**	**CoV**	**Probability Distribution**
f_m	6.07	0.20	6.07	0.20	3.77	0.24	Normal
f_c (columns)	25.00	0.20	20.00	0.20	15.00	0.23	Normal
f_c (others)	20.00	0.20	20.00	0.20	15.00	0.23	Normal
f_y	415.00	0.09	415.00	0.09	415.00	0.09	Normal
Dimension	As in Table 1	0.05	As in Table 1	0.05	As in Table 1	0.05	Normal

The compressive strength of the masonry wall is the most important parameter that determines the strength and stiffness of the wall. The compressive strength of the brick masonry walls used in this study is adopted from the results of a number of prism tests conducted by Kaushik et al. [36] on Indian bricks. They found the mean compressive strengths of the brick masonry with medium and weak cement mortars to be 6.07 MPa and 3.77 MPa respectively with the respective CoV of 0.20 and 0.24. These values are assumed to be applicable in Bhutan since Indian bricks with similar cement-sand mortar compositions are used in Bhutan.

Other studies such as Gumaste et al. [37] and Sarangapani et al. [38] also provided similar compressive strength values on the Indian brick masonry. The CoV of concrete strength are adopted from the recommendations of the Indian Standards for Plain and Reinforced Concrete, IS 456:2000 which are 0.20 for 20 MPa and 25 MPa concrete grades and 0.23 for 15 MPa concrete. The CoV of steel strength is adopted from the results of series of tests conducted on 415 grade steel by Basu et al. [39]. The CoV resulted from the test was 0.0893 which is used in this study. The CoV of geometrical dimensions is assumed by many studies in between 0.03 and 0.05 [40, 41]. The CoV of 0.05 is similarly used in this study. All these parameters are considered as random and statistically independent to one another. They are assumed to be normally distributed as in many studies [40, 41].

5.2 Estimation of structural responses

The estimation of structural responses involving a number of statistical parameters is quite complicated. In order to simplify the procedure, Rosenbluth Point Estimate Method (RPEM) is employed for the statistical estimation of the structural responses using the Perform 3D program. This method was first developed by Rosenbluth [9] and consists of considering two point estimates at one standard deviation on either side of the mean value of each variable. Depending on the number of variables considered, the number of point estimates and the number of possible combination of point estimates are respectively given by $2n$ and 2^n, where n is the number of variables. Since four statistical variables are considered in this study, there are 8 point estimates and 16 possible combinations of point estimates. The dynamic nonlinear analyses are carried out for every possible combination of these point estimates and accordingly the mean and standard deviation of response quantities are calculated from the output of 2^n analyses. This method is used to estimate the structural responses such as interstorey drift and displacements of three typical masonry infilled RC buildings considered in the study.

On the other hand, Monte Carlo Simulation Method (MCSM) is used to validate the accuracy of RPEM and also to determine the statistical distribution of the response quantities. MCSM is a very direct and reliable procedure that solves a deterministic problem number of times to build up the statistical distribution of the response quantity. However, MCSM is computationally very expensive and not practical for everyday use. Hence, it is only used to estimate the response quantity of ‘3 storey new’ building to validate the accuracy of RPEM. Normally, a large number of simulations in the order of a few hundred to a few thousands are required to arrive at the converged solution in MCSM. In this study, a variance reduction technique known as the stratified sampling is used to reduce the number of simulations. Using the stratified sampling method, it is found that the mean and standard deviation of the interstorey drift of ‘3 storey new’ building got converged at around 150 to 200 simulations. Fig. 5 shows the comparison of mean interstorey drifts estimated at generic soil sites for ‘3 storey new’ building under the 475 year return period ground motion. It can be observed from the figure that the interstorey drifts estimated by RPEM and MCSM are very close to each other indicating the accuracy of RPEM in estimating the structural responses. The interstorey drifts estimated are found to be lognormally distributed with the significance levels of 5% at rock and soft rock sites and 10% at shallow stiff soil and soft soil sites as per the Kolmogorov-Smirnov test.

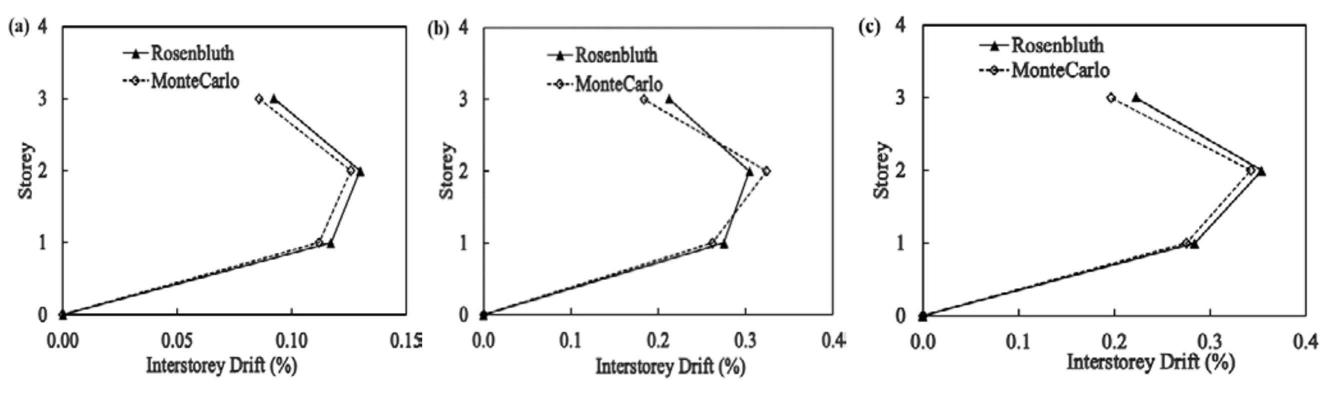


Fig. 5 – Comparison of interstorey drifts estimated from RPEM and MCSM at (a) rock, (b) shallow stiff soil and (c) soft rock sites

After validating the accuracy of RPEM and determining the statistical distribution, RPEM is further used to estimate the structural responses of the typical masonry infilled RC buildings at different soil sites under the 475 and 2475 year return period ground motions. The responses are also estimated by introducing flexible supports at the soft soil site to study the effect of soil structure interaction. The mean maximum interstorey drift and the corresponding standard deviation of three buildings are given in Tables 3-5. It is to be noted that the analysis failed to complete after the initial run for the '3 storey old' building with flexible support at soft soil site under 2475 year return period ground motion. Its mean and standard deviation are hence not shown in the Table 5.

Table 3-Mean maximum interstorey drift and corresponding standard deviation of '6 storey' building.

		475 return period			2475 return period			
Site class	Support Type	Mean (%)	Std. Dev. (%)	Floor Level	Mean (%)	Std. Dev. (%)	Floor Level	Probability distribution
Rock	Fixed	0.287	0.050	3	0.509	0.095	4	Log normal
Shallow stiff	Fixed	0.470	0.027	3	0.856	0.115	4	Log normal
Soft rock	Fixed	0.338	0.043	3	0.831	0.077	4	Log normal
Soft soil	Fixed	1.012	0.194	3	2.781	0.258	1	Log normal
Soft soil	Spring	1.413	0.243	2	2.903	0.597	2	Log normal

Table 4-Mean maximum interstorey drift and corresponding standard deviation of '3 storey new' building.

		475 return period			2475 return period			
Site class	Support Type	Mean (%)	Std. Dev. (%)	Floor Level	Mean (%)	Std. Dev. (%)	Floor Level	Probability distribution
Rock	Fixed	0.130	0.031	2	0.195	0.066	2	Log normal
Shallow stiff	Fixed	0.304	0.090	2	0.576	0.103	2	Log normal
Soft rock	Fixed	0.354	0.119	2	0.737	0.057	2	Log normal
Soft soil	Fixed	0.376	0.078	2	0.598	0.144	2	Log normal
Soft soil	Spring	0.439	0.094	1	1.933	0.483	1	Log normal

Table 5-Mean maximum interstorey drift and corresponding standard deviation of '3 storey old' building.

		475 return period			2475 return period			
Site class	Support Type	Mean (%)	Std. Dev. (%)	Floor Level	Mean (%)	Std. Dev. (%)	Floor Level	Probability distribution
Rock	Fixed	0.265	0.079	1	0.447	0.135	1	Log normal
Shallow stiff	Fixed	0.511	0.045	2	1.182	0.150	1	Log normal
Soft rock	Fixed	0.372	0.024	1	0.999	0.128	1	Log normal
Soft soil	Fixed	1.260	0.250	1	2.045	0.734	1	Log normal

Soft soil	Spring	1.981	0.330	1	Log normal

6. Fuzzy failure probability analyses and performance assessment

Based on the probabilistic information of the maximum interstorey drift given in Tables 3-5, the damage probabilities of the typical buildings can be conventionally estimated from the equation below.

$$P_f = (D \geq D_c) = \int_{D_c}^{\infty} f_D(D)dD \quad (3)$$

where D is the maximum interstorey drift demand given in Tables 3-5, f_D *(D)* is its probability density function and D_c is the critical interstorey drift value correlated to the damage. The critical interstorey drift values associated with the damages of the buildings are given by a number of guidelines [5-7]. However, there are very limited studies carried out on the correlation of interstorey drift with the damages of the masonry infilled RC buildings. In this study, the critical interstorey drift values correlated to the damages of masonry infilled RC buildings by Ghobarah [10] and shown in Table 6 are used. They were derived from a number of analytical and experimental studies on the masonry infilled RC buildings and are assumed to be applicable to Bhutan.

Table 6-Performance criteria proposed by Ghobarah (2004).

Performance levels	Damage state	Interstorey drift (IDR) limit (%)
Fully Operational (IO)	Slight	<0.1
Operational (O)	Repairable	0.1≤IDR<0.4
Life Safety (LS)	Irreparable	0.4≤IDR<0.7
Near Collapse (NC)	Severe	0.7≤IDR≤0.8
Collapse (C)	Complete	IDR>0.8

The damage probabilities predicted by Eq. (3) are based on the fixed damage boundary wherein the structure is said to be damaged if $D \geq D_c$ and not damaged if $D \leq D_c$. However, the damage is a continuous process under the action of the load and cannot have a fixed boundary. In other words, damage of a structure is not only dependent on the randomness but also on the fuzziness [8]. Hence, it is logical to define a fuzzy region between the damage boundaries given in Table 6 to estimate the realistic damage probabilities. In the fuzzy region, the structure may fail even if $D \leq D_c$ and may not fail if $D \geq D_c$. Based on the random-fuzzy probability theory, the fuzzy failure probability of structure can be obtained from

$$P_{ff} = P(D \geq D_c) = \int_{D_L}^{D_u} \mu(D) f_D(D)dD \quad (4)$$

where D_U and D_L are respectively the upper and the lower fuzzy limits and μ *(D)* is the membership function [8]. The fuzzy limits in this study are the midpoints of the interstorey drift limits in Table 6 which are depicted in Fig. 6. The membership function, on the other hand, is quite complex to define and often based on the judgement of some experts [42]. A commonly used triangular membership function shown in Fig. 6 is used in this study. It is constructed by extending the damage state to the midpoint of the next damage stage. The membership function, $\mu(D)$ =1 and μ *(D)* =0 respectively indicate 100% and 0% failure probabilities.

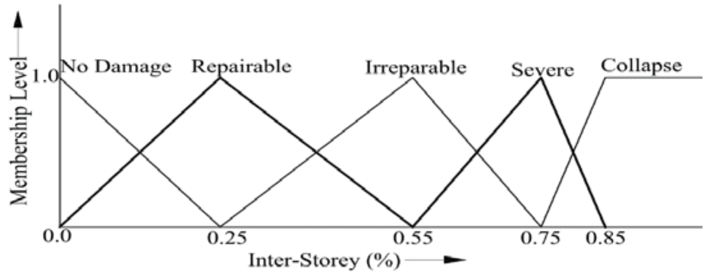


Fig. 6 – Triangular membership function constructed based on the interstorey drift limits of (Ghobarah 2004).

The failure probabilities of the typical buildings estimated from the conventional probability analyses using Eq. (3) and that from the fuzzy probability analyses using Eq. (4) at the generic soil sites in Thimphu under the 475 and 2475 year return period ground motions are given in Figs. 7-9. Since interstorey drifts are found to be lognormally distributed, lognormal probability density function is used in Eq. (3) and Eq. (4) to estimate the failure probabilities.

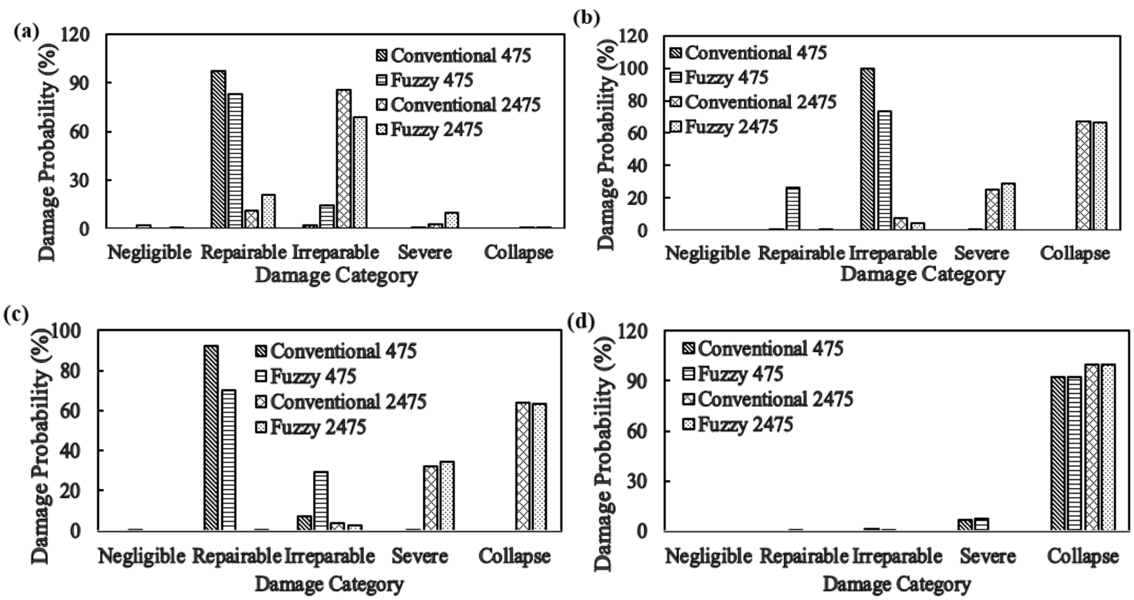
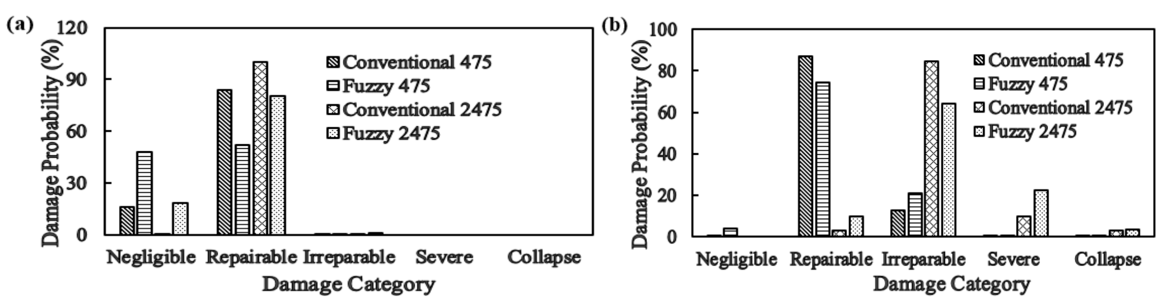


Fig. 7 – Damage probabilities of '6 storey' building for 475 and 2475 year return period ground motions at (a) rock, (b) shallow stiff soil, (c) soft rock and (d) soft soil sites.



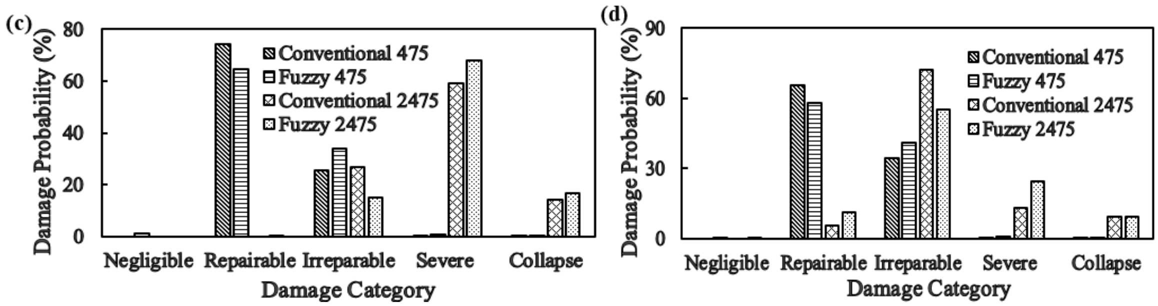


Fig. 8 – Damage probabilities of '3 storey new' building for 475 and 2475 year return period ground motions at (a) rock, (b) shallow stiff soil, (c) soft rock and (d) soft soil sites.

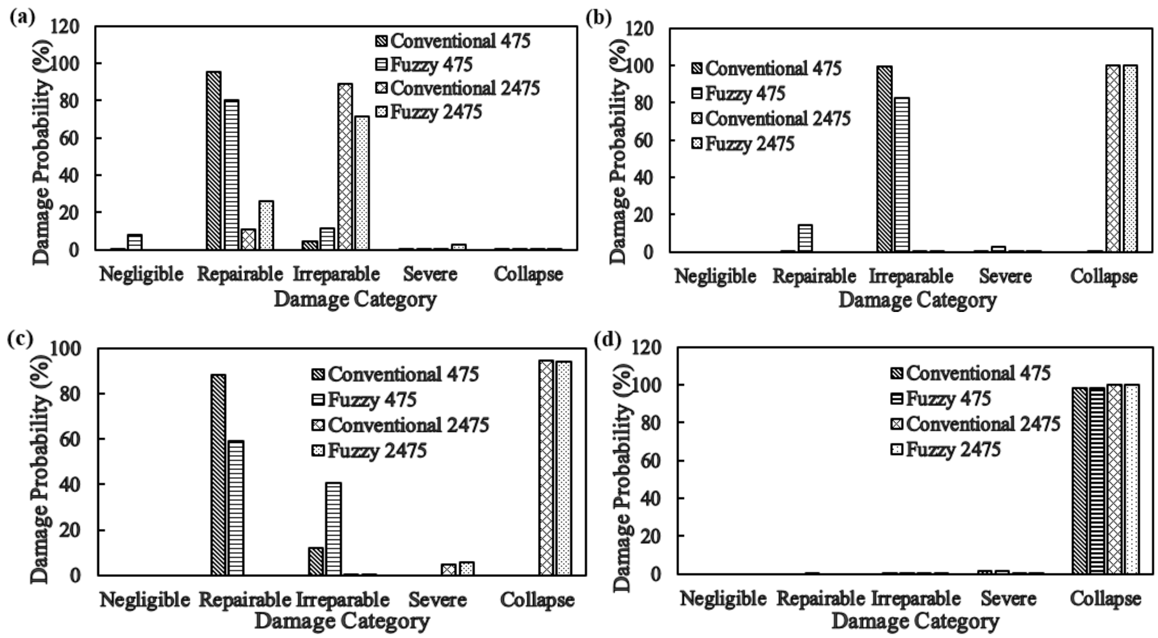


Fig. 9 – Damage probabilities of '3 storey old' building for 475 and 2475 year return period ground motions at (a) rock, (b) shallow stiff soil, (c) soft rock and (d) soft soil sites.

6. Discussion

The damage probabilities presented in Figs. 7-9 are predicted for the fixed-base buildings without considering soil-structure interaction. The figures pertaining to the SSI are not shown here owing to the page limitation. However, it is observed that the effect of SSI is not very significant for the '6 storey' and '3 storey old' buildings while it was found to be slightly beneficial under the 475 year return period ground motion and detrimental under the 2475 year return period ground motion for the '3 storey new' building. The effect of SSI is found to be dependent on the stiffness and site natural period of the soil and on the fundamental period of the buildings. Since the effect of SSI is not very significant, the damage probabilities predicted for the fixed-base buildings are expected to provide the true damage scenarios of the typical masonry infilled RC buildings in Bhutan and are hence discussed here.

From Figs. 7-9, it can be observed that the damage probabilities predicted from the conventional and fuzzy probability analyses vary from one damage state to another with no definite pattern. The variation is as small as less than 1% to as big as more than 95%. For instance, referring Fig. 7(b), the probability of collapse predicted from the conventional and fuzzy probability analyses under the 2475 year return period ground motion are 67.21% and 66.8% respectively in which the variation is less than 1%. On the other hand, referring to the same figure, the repairable damage predicted from the conventional and fuzzy analyses under the 475 year return period ground motion are 0.3% and 26.7% in which the variation is more than 98%. As discussed above, the damages predicted by the conventional analyses are based on the fixed damage boundaries and hence are not realistic. With the consideration of fuzzy region in between the damage boundaries, the damages predicted by the fuzzy probability analyses are more realistic and are only discussed in the following paragraphs.

As shown in Fig. 7, ‘6 storey’ building has the high probability of undergoing repairable damage at the rock and soft rock sites under the 475 return period ground motion. Under the same ground motion, the building could undergo irreparable damage at the shallow stiff soil site and even collapse at the soft soil site. Under the 2475 year return period ground motion, high probability of irreparable damage is predicted at the rock site while the building has more than 30% and 60% probabilities of undergoing severe damage and complete collapse respectively at the shallow stiff soil and soft rock site. At the soft soil site, 100% probability of collapse is predicted for ‘6 storey’ building under the 2475 year return period ground motion.

The damage probabilities of ‘3 storey new’ building can be observed from Fig. 8. As shown in the figure, the building has almost an equal probability of undergoing negligible and repairable damages at the rock site under the 475 year return period ground motion. High probabilities of 75%, 65% and 58% are respectively predicted at the shallow stiff soil, soft rock and soft soil sites for ‘3 storey new’ building under the same ground motion. The ‘3 storey new’ building could suffer repairable damage at the rock site and severe damages at the shallow stiff soil, soft rock and soft soil sites under the 2475 year return period ground motion. The building has a very small probability of undergoing complete collapse under both the ground motions.

It can be observed from Fig. 9 that the ‘3 storey old’ building has no chance of surviving at the soft soil site under the ground motions considered. Total collapse is also predicted at the shallow stiff soil and soft rock sites under the 2475 year return period ground motion, while the high probability of irreparable damage is predicted at the rock site under the same ground motion. On the other hand, the building has about 80% probability of undergoing repairable and irreparable damages at rock and shallow stiff soil sites respectively under the 475 year return period ground motion. At the soft rock site, the building has 60% and 40% probabilities of suffering repairable and irreparable damages respectively under the 475 year return period ground motion.

In summary, ‘3 storey new’ and ‘3 storey old’ buildings respectively exhibit the best and the worst performance under the ground motions considered. This is expected since ‘3 storey new’ buildings was designed according to the Indian Seismic Code while ‘3 storey old’ buildings was not designed to any standard. On the other hand, the performance of the ‘6 storey’ building is just better than the ‘3 storey old’ building although it was also designed according to the Indian Seismic Code. It could be either due to the fact that the building was not properly designed or that the Indian Seismic Code is not adequate enough for the design of buildings in Bhutan.

6. Conclusion

In this study, the failure probabilities of three typical masonry infilled RC buildings representing the general buildings stocks in Bhutan are predicted considering both the material and geometrical uncertainties. Unlike in many studies where damages of the buildings are normally assessed based on the conventional probability analysis, fuzzy probability analysis is employed in this study to realistically predict the damage probabilities of the typical buildings in Bhutan. Rosenbluth point estimate method is employed for the statistical variation of material and geometrical parameters leading to the estimation of the response quantities using the Perform 3D program. Monte Carlo Simulation method is used to validate the accuracy of the Rosenbluth point estimate method and also to determine the statistical distribution of the response quantities.

From this study, it is observed that the ‘3 storey new’ building designed according to the Indian Seismic Code could experience a high probability of repairable and irreparable damages under the 475 and 2475 year return period ground motions respectively conforming to the intended design objectives. The ‘6 storey’ building which was also designed to Indian Seismic Code could experience higher probabilities of repairable and irreparable damages under the 475 year return period ground motion and high probability of collapse under the

2475 year return period ground motion. The ‘3 storey old’ building which was not designed to any standard could suffer higher probabilities of irreparable damages and complete collapse under the 475 and 2475 year return period ground motions respectively.

This study provides information on the failure probabilities of masonry infilled RC buildings in Bhutan and could be very useful for loss estimation and seismic mitigation studies in the country.

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