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Seismic Assessment of Masonry Infilled Reinforced Concrete Frame Buildings in Bhutan

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ABSTRACT:

Masonry infilled Reinforced Concrete (RC) frame building constitutes more than 70% of buildings in Thimphu. More than 50% of these buildings were not designed for seismic load and the rest were designed following the Indian Seismic Code. While the capital city is being adorned with masonry infilled RC buildings, their performance during earthquakes is quite uncertain. Being located on one of the most active seismic regions in the Himalaya, devastating earthquakes such as the one that recently occurred in Nepal on 25th April 2015 cannot be ruled out in Bhutan. This paper presents the seismic assessment of three masonry infilled RC buildings which typically represent the stocks of masonry infilled RC building in Bhutan. The predicted ground motions obtained from Probabilistic Seismic Hazard Analysis (PSHA) at generic soil sites in Thimphu, Bhutan are used as input in the structural response analysis. Non-linear analysis and performance assessment software, Perform 3D is used for the numerical simulations. The accuracy of the numerical model is calibrated with the test results reported by other researchers. Soil Structure Interaction (SSI) is incorporated for different soil sites and its effect on the performance of building is discussed. The typical masonry infilled RC buildings in general exhibit life safety and collapse prevention performance levels.

1 INTRODUCTION

Masonry infilled RC buildings were the major victims of many past earthquakes such as 2000 Kocaeli earthquake in Turkey, 2001 Gujarat earthquake in India and 2008 Wenchuan earthquake in China. Even in the recent Nepal earthquake on 25th April 2015, many masonry infilled RC buildings were collapsed and many suffered major damages. In spite of their vulnerability, masonry infilled RC buildings are commonly designed as bare frame by considering only the weight of infill wall and totally neglecting the strength and stiffness of the infill wall. Introduction of masonry infilled wall into the RC frames significantly increase the strength and stiffness of the structure which in turn significantly affect the structural responses. Presence of infilled wall also modifies the failure mechanism of the structure by leaning more towards brittle failure. Seismic Codes around the world also lack the comprehensive treatment of masonry infilled RC frames due to their complex behaviour under seismic action.

In Bhutan, construction of masonry infilled RC frame buildings had started as early as 1970 and it has become the most popular form of construction today. There are hundreds of masonry infilled RC buildings in Thimphu and other parts of Bhutan and many are under construction. However, their performance under seismic action is highly questionable. The country has no seismic design code of its own and buildings built prior to 1997 were mostly built based on some thumb rules without any kind of design. Indian Seismic Code was adopted in 1997 to be used in Bhutan, but its applicability for the site conditions in Bhutan is not studied. Moreover, masonry infilled RC buildings are still being designed as bare frames by solely neglecting the strength and stiffness of the infilled wall. Hence, there are stocks of masonry infilled RC buildings whose performance under seismic action is yet to be ascertained. Past studies on the performance of masonry infilled RC buildings are very rare and none have studied the 3 dimensional real masonry infilled frame buildings in Bhutan subjected to expected ground motions in Bhutan. On the other hand, Bhutan is located right on the Himalayan arc where Indo-Australian plate is continuously subducted into Eurasian plate at an average rate of 20 ±3mm per year (Bilham et al. 2001). Based on the number of evidences such as seismic gap hypotheses and potential slip accumulation, it is reported that one or more major earthquakes of magnitude 8 or greater are already overdue along the Himalayan arc (Bilham, et al. 2001, Walling and Mohanty 2009). In the event of major earthquake in Bhutan as reported, it is highly likely that loss of life and property would be substantial since expected performance of these masonry buildings are not known beforehand. Hence, it is paramount to assess the performance of these buildings and address the mitigation measures.

In this study, three typical masonry infilled RC frame buildings that are currently standing in Thimphu are considered. They are very typical and represent the general masonry infilled RC frame building stocks in Bhutan. Dynamic nonlinear analysis and performance assessment software, Perform 3D is used for the numerical analysis. The numerical model is first calibrated with the experimental results to ensure the accuracy of structural response prediction. The ground motions predicted by Hao and Tashi (2010) for the generic soil sites in Thimphu are used for the analyses. The most commonly used equivalent diagonal strut model is used for modelling the infilled wall. The opening of the masonry infilled wall which is an integral part of masonry infilled RC buildings is also considered. An uncoupled spring support is considered at soft soil site to study the effect of soil structures interaction. The performance of typical buildings are then assessed based on the interstorey drift limit proposed by Ghobarah (2004).

In order to check the performance of typical masonry infilled RC buildings in Bhutan in the event of earthquake such as the one that occurred in Nepal on 25th April 2015, ground motions from Nepal earthquake is also applied for numerical analyses of typical buildings. From the numerical analysis of typical buildings, it is observed that masonry infilled RC buildings in Bhutan in general exhibit life safety and collapse prevention performance levels. However, none of the buildings considered in this study can be immediately occupied under the given ground motions. The response of typical buildings from Nepal ground motions also exhibit the kind of performances exhibited by masonry infilled RC buildings in Nepal during the earthquake.

2 GROUND MOTION

In absence of real recorded ground motions, the ground motions specifically predicted for the generic soil sites in Bhutan by Hao and Tashi (2010) from Probabilistic Seismic Hazard Analysis (PSHA) are used for the numerical simulation. For the PSHA, 18 seismic source zones within a distance of 400Km from Thimphu were considered. The response spectrum of ground motions predicted in Thimphu at various soil sites for the return period of 475 and 2475 years and 5% damping are shown in Figures 1 below.

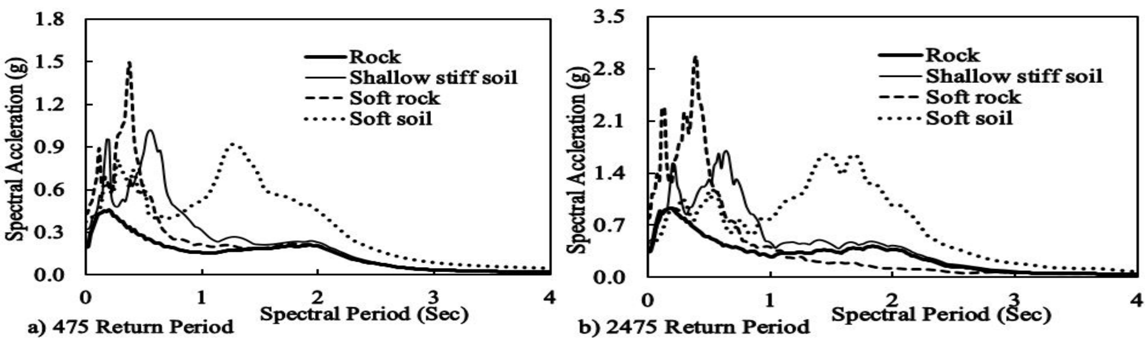


Figure 1. Ground motion response spectra at different sites for 475 and 2475 year return periods for 5% damping.

To check the performance of buildings in Bhutan in the event of real earthquake such as the one in Nepal that occurred on 25th April 2015, masonry infilled RC buildings in Bhutan are also analysed using the ground motion recorded at stiff soil site in Kathmandu on 25th April 2015. This could provide a realistic idea on the performance of buildings in Bhutan since seismicity and geographical locations of Kathmandu and Thimphu are similar. Figure 2 shows the response spectrum of 25th April Nepal earthquake and its comparison with the response spectra predicted at shallow stiff soil in Thimphu. As shown in the figure, shapes of the spectra are very much similar although amplitude and second peak periods are different.

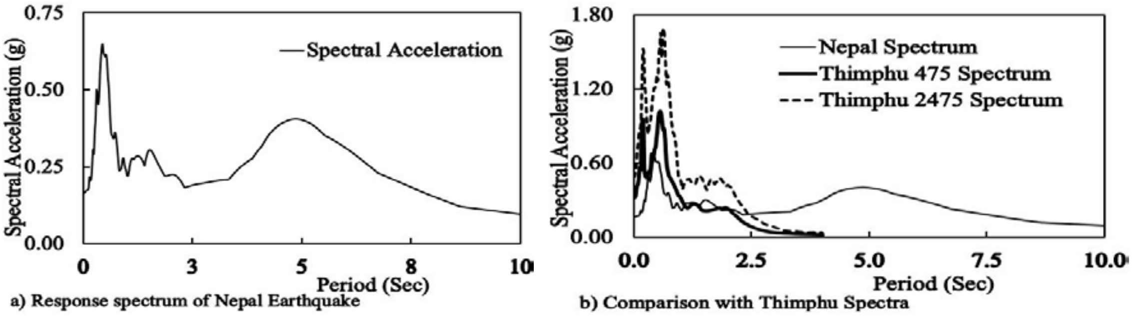


Figure 2. Response spectrum of Nepal earthquake and its comparison with response spectra at shallow stiff soil site in Thimphu for 475 and 2475 year return periods at 5% damping.

3 MODELLING OF MASONRY INFIILED FRAME AND MODEL CALIBRATION

Modelling of masonry infilled RC frame basically consists of modelling the RC frame and modelling of masonry infilled wall which when combined are expected to capture the true behaviour of infilled RC frame. The lumped plasticity model with trilinear force deformation (F-D) relationship is used for the modelling of RC members. The numerical model for the bare frame was previously calibrated with experimental results and the details of calibration can be found in Thinley et al. (2014).

Masonry infilled wall is commonly modelled by using either micro or macro models. In micro model, infilled wall is divided into number of elements to take into account the local effects, while in macro model, modelling is based on the global behaviour of masonry infilled frame (Crisafulli et al. 2000). Due to its simplicity and its ability to capture the global behaviour of masonry infilled frame with sufficient accuracy, numerous studies such as Sattar and Liel (2010), Dolsek and Fajfar (2002), Panagiotakos and Fardis (1994) and Negro and Verzeletti (1996) have used equivalent strut model for modelling the infilled wall. Similarly, a pair of compression struts are used in this study to represent each panel of the masonry infilled walls. A force deformation (F-D) relationship as shown in the Figure 3 is used to define the properties of each strut. The definition of force and corresponding deformation form the main crux of modelling the masonry infilled wall.

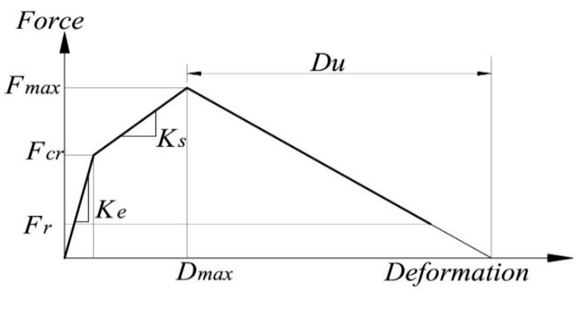


Figure 3. Force deformation relationship of equivalent strut.

In this study, the initial stiffness of the wall is estimated from the simple equation given in Dolsek and Fajfar (2008) as

$$K_e = \frac{G_w L_{in} t_w}{H_{in}} \quad (1)$$

Where G_w = shear modulus of infilled wall, L_{in}=length of infilled wall, t_w= thickness of infilled wall and H_{in}=height of the infilled wall.

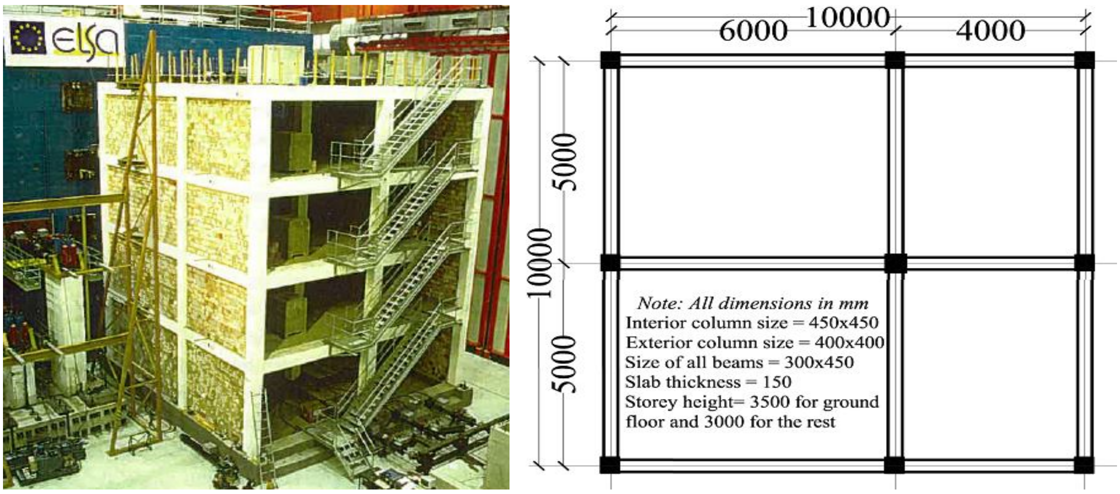
For the estimation of maximum strength of infilled wall, the following expression developed by Zarnic and Gostic (1997) is used in this study.

$$F_{max} = 0.818\frac{L_{in} t_w f_{tp}}{C_I}\left(1+\sqrt{C_I^2+1}\right), \qquad C_I = 1.925\frac{L_{in}}{H_{in}} \quad (2)$$

Where f_{tp}= cracking strength of the infill. Other parameters are same as in equation (1).

As recommended by Dolsek and Fajfar (2008), the cracking strength of masonry, F_{cr} is taken as 60% of the maximum strength. An almost the same ratio was obtained from the experimental data by Manzouri (1995). Similarly, based on the pseudo dynamic test on partially infilled frame, Dolsek and Fajfar (2008) found the occurrence of maximum strength at 0.2% drift. They assumed the displacement at collapse as 5 times the displacement at maximum strength and was found to agree well with the experimental results. Shing et al. (2009) also found the occurrence of maximum strength at 0.25% drift from the experimental test. In this study, the deformation values observed and assumed by Dolsek and Fajfar (2008) are used.

Using the strength and deformation parameters of masonry infilled wall as obtained above and using the F-D relationship of RC members previously calibrated in Thinley et al. (2014), numerical simulation is run for the four storey masonry infilled RC frame building which was pseudo-dynamically tested at the European Laboratory of Structural Assessment (SLEA). The details of the building and the test details can be found in Negro et al. (1994), Negro et al. (1996) and Negro and Colombo (1997). The test set up and plan of the building is shown in the Figure 4.



(a) (b)

Figure 4. (a) Test set up of masonry infilled RC building (Negro et *al*. 1996); (b) Plan and dimension of RC members.

Considering the entire structural details of the building such as load, member dimension, specification of materials and input ground motion, nonlinear analysis is carried out using Perform 3D. The structural response obtained numerically is compared with the experimental test results. Figure 5 shows the comparison of displacement time histories of the building obtained from the numerical analysis and that from the pseudo-dynamic test. From the figure, it can be observed that a very good match has been obtained given the fact that the simplest macro models are used for the modelling of RC members and masonry infilled wall. This indicates that numerical model is calibrated with sufficient accuracy. A little bit of mismatch is observed at first and second floors which could be due to the use of single equivalent strut model which is the simplest model and only sufficiently captures the global behaviour of the masonry infilled frame.

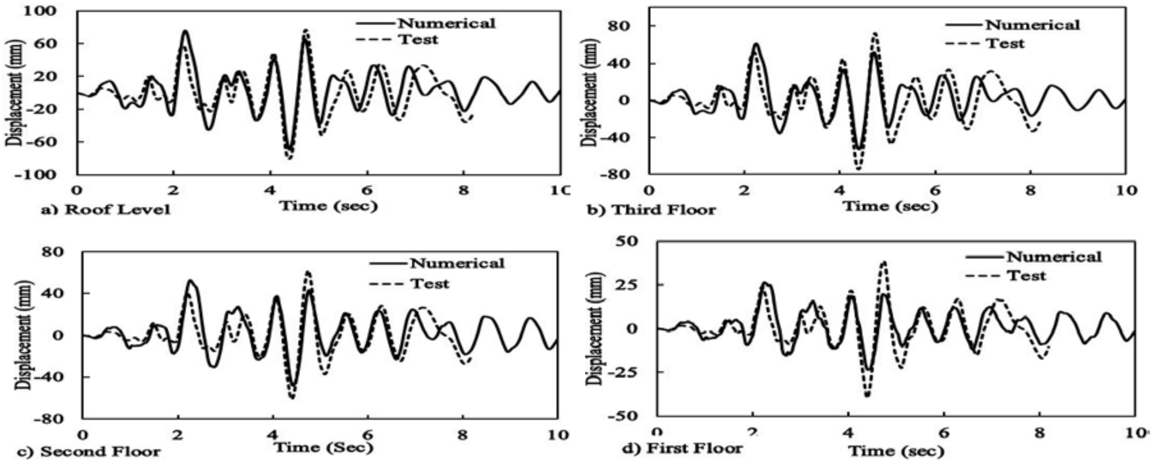


Figure 5. Comparison of displacement time histories obtained from numerical and experimental test (Negro et *al*. 1996).

4 TYPICAL BUILDINGS CONSIDERED FOR PERFORMANCE ASSESSMENT

In order to represent the performance of buildings in Bhutan more realistically, three typical masonry infilled RC buildings denoted as '6 storey', '3 storey new' and '3 storey old' buildings are considered. '6 storey' and '3 storey new' represent the masonry infilled buildings designed and built after the adoption of Indian Seismic Code in Bhutan in 1997, while '3 storey old' building represents the masonry infilled RC frame buildings built prior to the adoption of Indian Seismic Code. The structural details of '6 storey' and '3 storey new' buildings are obtained from Thimphu City Corporation. Since buildings built prior to 1997 were mostly based on some thumb rules without any kind of design, structural details of these buildings are either not at all available or not sufficient enough. Hence, structural details of '3 storey old' building are adopted from the result of non-destructive test conducted on 15 such buildings in Thimphu under the Thimphu Valley Earthquake Risk Management Project in 2005 (UNDP Report 2006). For the purpose of comparison, plan and elevation of '3 storey old' building are assumed identical to that of '3 storey new' building. Beam and column layout plan and sectional elevation with strut arrangement of these buildings are shown in Figures 6.

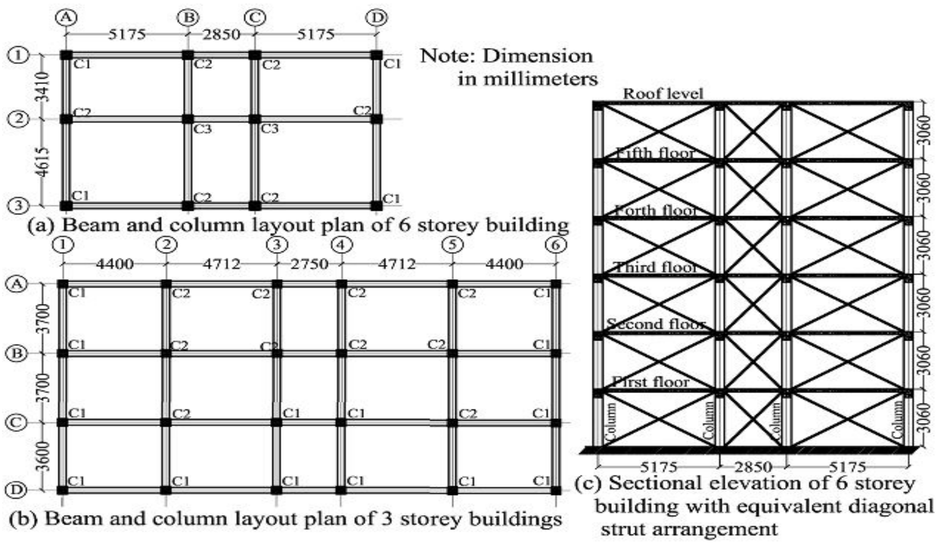


Figure 6. Typical masonry infilled RC building plans and sectional elevation of 6 storey building.

Structural details of these buildings such as loads, RC member dimensions, reinforcement and concrete details can be found in Thinley et al. (2014). In regard to the infilled walls, brick masonry with cement mortar is used for all the buildings. The structural details of infilled wall are given in Table 1.

Table 1. Structural details of brick masonry infilled wall

Structural Details	6 storey	3 storey new	3 storey old
Unit weight of clay bricks, (kN/m^3)	19.6	19.6	19.6
Cement to sand ratio of cement mortar	1:4	1:4	1:6
Compressive strength of wall, f_m (MPa)	6.6	6.6	4.1
Modulus of elasticity of wall, E_m (MPa)	3630	3630	2255
Cracking strength of wall, f_{tp} (MPa)	0.35	0.35	0.25

The unit weight of bricks and mortar composition of '6 storey' and '3 storey new' buildings are obtained from the structural drawings. In absence of the structural details, they are reasonably assumed for '3 storey old' building based on the result of non-destructive test. Length, height and thickness of the walls are obtained from the structural and architectural drawings. However, the most important parameter, compressive strength of brick masonry wall is obtained from Kaushik et *al*. (2007) who had undertaken comprehensive numerical and experimental studies on Indian brick masonry. Based on the number of tests, the mean compressive strength of brick masonry with intermediate (1:4) and weak (1:6) mortars are estimated to be 6.6 MPa and 4.1 MPa respectively. They found the modulus of elasticity of brick masonry to vary from $250f_m$ to $1100f_m$. The mean value of $550f_m$ was recommended to be used for the design and same has been adopted in this study. FEMA 273 (FEMA 1997) also recommended the modulus of elasticity of masonry equal to $550f_m$. International codes such as Eurocode6 (CEN 1996) and International Building Code (IBC 2003) recommended $700f_m$ and $1000f_m$ respectively which are higher than that recommend by Kaushik et *al*. (2007) for Indian brick masonry. Pauley and Priestley (1992) also recommended modulus of elasticity of masonry wall to be $750f_m$. The cracking strength of brick masonry obtained from the number of compression diagonal tests by Dolsek and Fajfar (2002) was found to vary from 0.28MPa to 0.4Mpa. Hendry et al. (2004) also recommended the cracking strength of brick masonry to be 0.4MPa. Based on these studies and quality of cement mortar used for the typical buildings, reasonable value of 0.35MPa and 0.25Mpa are used in this study as shown in Table 1.

Opening is an integral part of the building and is provided for functional and ventilation purposes. Both external and partition walls of typical buildings have openings of various sizes. Presence of opening significantly modifies the structural response of the building. Owing to the number of uncertainties resulting from different sizes and positions of opening, prediction of structural response of infilled frame buildings with opening is quite complex. The most general understanding is that the presence of opening reduces the stiffness and strength of the infilled panel. Numerous numerical and experimental studies were undertaken by number of researchers and many have proposed reduction factor to reduce stiffness and strength of the infilled wall with openings. Reduction factor proposed by some of the prominent studies are studied and compared as shown in Figure 7. In this study, the reduction factor proposed by Symrou et *al*. (2006) is used since it was very well validated with the experimental results.

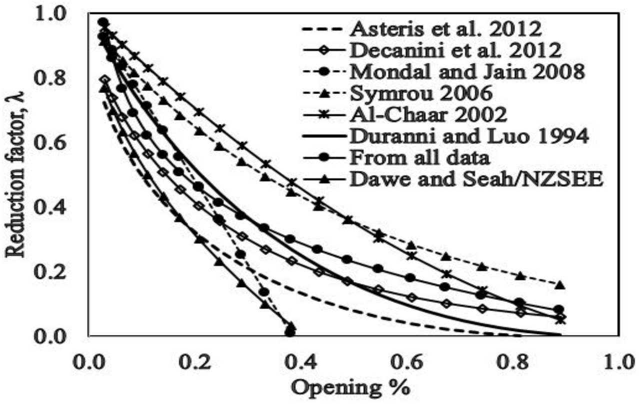


Figure 7. Comparison of reduction factors proposed by various researchers.

5 **RESPONSE OF TYPICAL BUILDINGS WITH MASONRY INFILLED WALLS**

After determining the structural details of masonry infilled typical buildings, dynamic nonlinear analyses are carried out using the ground motions predicted in Thimphu and the real recorded ground motion of 25th April 2015 Nepal earthquake. Since seismicity and geographical location of Thimphu and Kathmandu are similar, estimation of structural response based on Nepal earthquake could provide valuable information on the performance of buildings in Thimphu. The effect of soil structure interaction is studied by introducing uncoupled spring support at shallow stiff soil, soft rock and soft soil sites. The stiffness of spring is obtained from ASCE/SEI-41 (2006).

To better understand the response of masonry infilled RC structures, the predicted response is compared with the response obtained for bare frames from the previous study. Figure 8 shows the comparison of interstorey drifts of bare and infilled frames estimated from ground motions predicted in Thimphu for 475 year return period with fixed support. The interstorey drift of typical buildings for 2475 year return period follows the same pattern as that of 475 year return period although with higher drift. Similarly, interstorey drift of typical buildings considering soil structure interaction also follows the same pattern with slight increase in drift. Hence response of typical buildings for 2475 year return period and that for SSI can be visualised from Figure 8 and are not shown in this section.

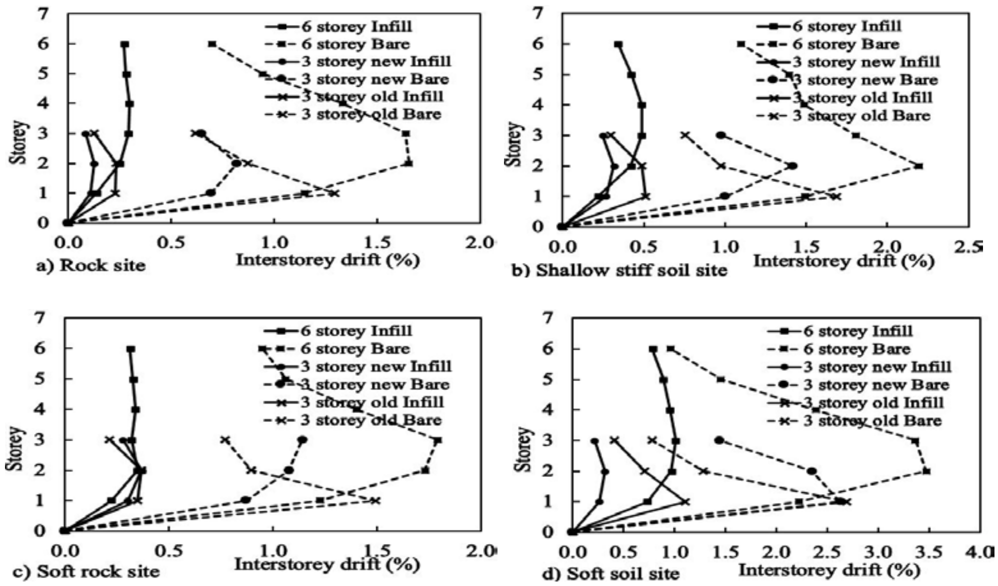


Figure 8. Comparison of interstorey drifts for infilled and bare frame buildings with fixed support and for 475 year return period ground motions.

The static pushover analyses are also carried out for both bare and infilled frames to estimate the load carrying capacity of the buildings. Figure 9 shows the comparison of pushover curves of bare and infilled frame typical buildings.

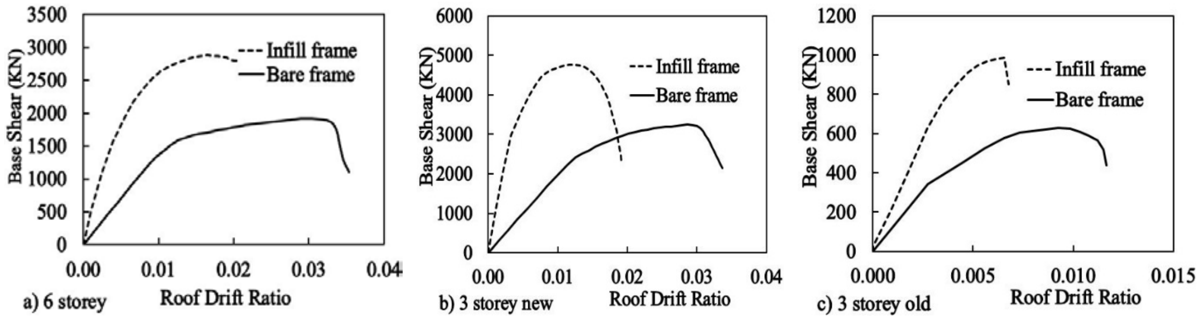


Figure 9. Comparison of static pushover curves for bare and infilled frame buildings.

The interstorey drift of typical buildings obtained from real recorded Nepal earthquake and its comparison with the interstorey drift obtained from predicted ground motion in Thimphu at shallow stiff soil site are shown in Figure 10. Since ground motion of Nepal earthquake was recorded at stiff soil with shear wave velocity of 240m/s as per the site classification of ASCE/SEI-41 (2006), the response predicted from Nepal ground motion is comparable with that obtained from ground motions predicted at shallow stiff soil in Thimphu.

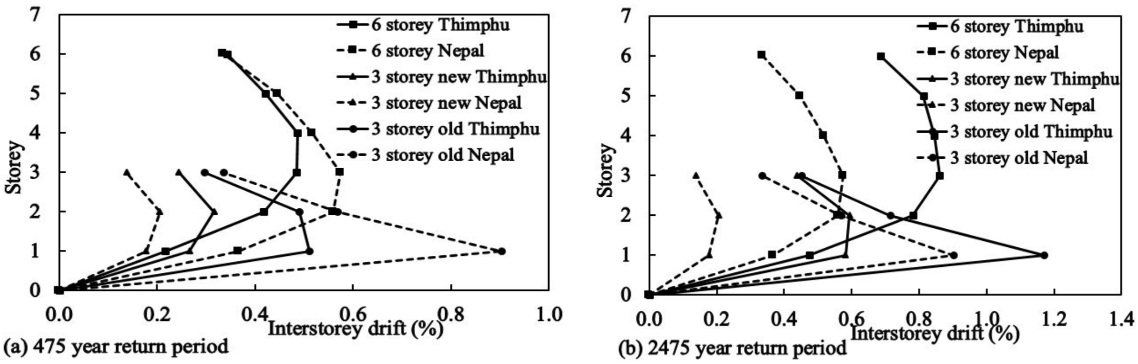


Figure 10. Interstorey drift comparison obtained from Nepal earthquake and that from ground motions predicted at shallow stiff soil site in Thimphu.

6 PERFORMANCE ASSESSMENT OF MASONRY INFILLED TYPICAL BUILDINGS

The interstorey drift is the most important response quantity associated with the damage of a building. It governs both structural and non-structural components of a building. The interstorey drifts associated with the performance levels are defined by number of publications such as ASCE 41-06, ATC-40 and Vision 2000 document (SEAOC 1995). However, there are limited studies undertaken in regard to the correlation of interstorey drift to the performance levels of masonry infilled frame buildings. Some of these studies such as ASCE/SEI-41 (2006), Ghobarah (2004) and Kalman-Sipos and Sigmund (2014) are shown in Table 2.

Table 2. Relation of performance level, damage state and interstorey drift of masonry infilled RC frame buildings

		Interstorey drift limit (%)		
Performance levels	**Damage State**	**ASCE/SEI-41 (2006)**	**Ghobarah (2004)**	**Kalman-Sipos & Sigmund (2014)**
Immediate Occupancy (IO)	Slight	0.10	0.10	0.10
Life Safety (LS)	Irreparable	0.50	0.40	0.30
Collapse Prevention (CP)	Severe	0.60	0.70	0.75

As shown in the Table 2, interstorey drifts proposed by different studies for the corresponding performance levels are very similar. In this study, interstorey drift limits proposed by Ghobarah (2004) is used since they look more rational and more or less represent the mean interstorey drifts proposed by other two studies. Moreover, they were developed from a large number of analytical and experimental data.

The performance levels and interstorey drift profiles of typical buildings at various soil sites for 475 and 2475 year return period ground motions are respectively shown in Figures 11 and 12. The same is shown in Figure 13 for the real recorded Nepal earthquake of 25th April 2015. The effect of soil structure interaction is studied at shallow stiff soil, soft rock and soft soil sites by introducing an uncoupled spring support. The solid lines in the figures represent the interstorey drift profile for fixed support (FS) and the dotted line represent the profile considering the soil structure interaction (SSI).

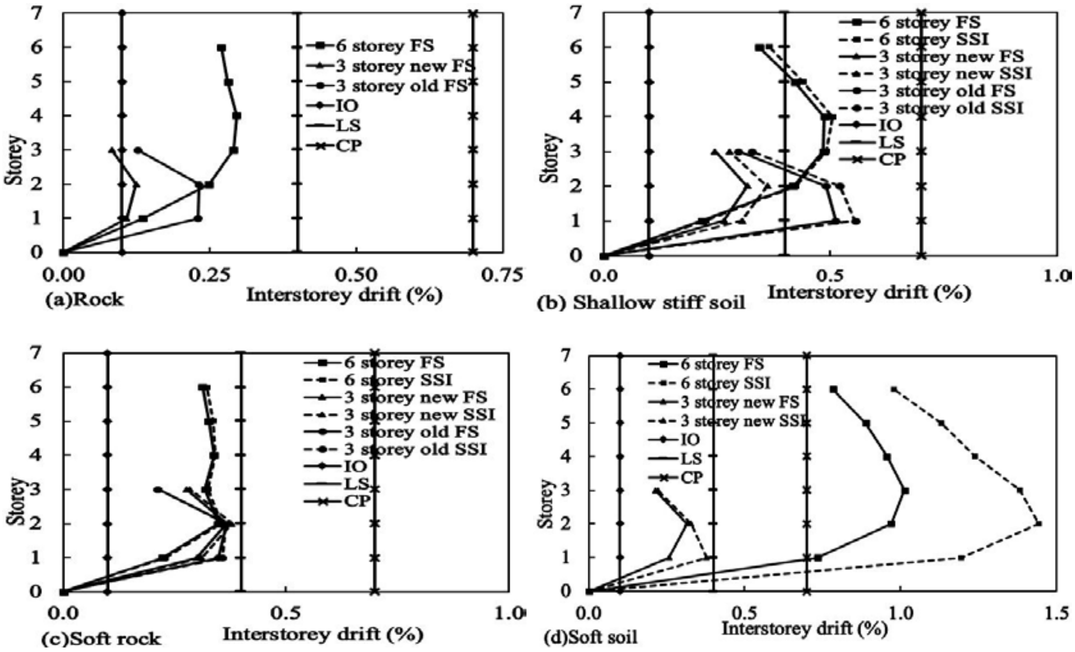


Figure 11. Performance levels and interstorey drift profiles for 475 year return period ground motion.

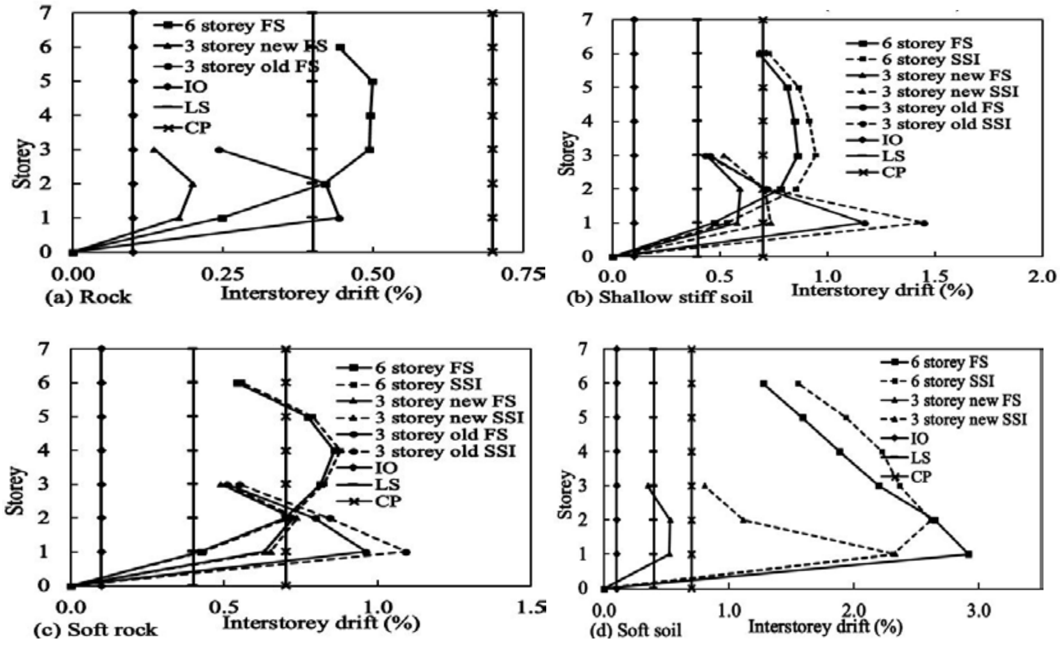


Figure 12. Performance levels and interstorey drift profiles for 2475 year return period ground motion.

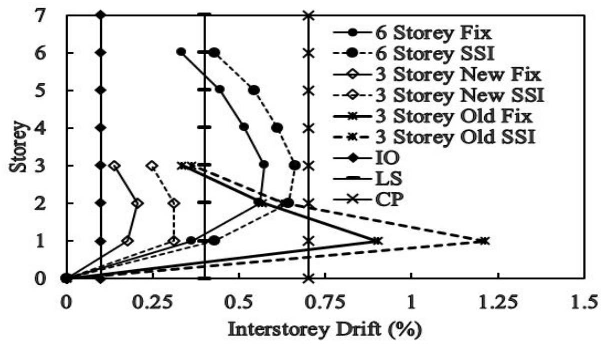


Figure 13. Performance levels and interstorey drift profiles for Nepal earthquake.

7 DISCUSSION

The structural response of masonry infilled RC frame buildings in Thimphu and their performances under the predicted earthquake ground motions and real recorded ground motion of Nepal earthquake are shown in Figures 8-13. From Figures 8 and 9, it can be clearly seen that presence of masonry infilled wall in RC frame greatly increases the strength and stiffness of the building. As shown in Figure 8, maximum interstorey drift is reduced by a factor of 2 to 6 depending on the type of site as compared to bare frame buildings. From Figure 9, it can be observed that presence of infilled wall makes the building stiffer by approximately 3 to 5 times and also increases the peak strength by approximately 1.5 times. Sattar and Liel (2010) also observed similar responses while studying the performance of pre-1975 California masonry infilled RC buildings. The figure also shows the reduction of ductility of masonry infilled building indicating the brittle behaviour as compared to bare frame.

Figure 10 shows the comparison of interstorey drift predicted from Nepal earthquake at stiff soil and that predicted from ground motions predicted at shallow stiff soil in Thimphu. It can be observed from the figure that except for ‘3 storey new’ building, interstorey drift predicted from Nepal earthquake is higher than that predicted from 475 year return period ground motion, but lower than that predicted from 2475 year return period ground motion. However, interstorey drift obtained from 475 return period ground motion in Thimphu reasonably compares well with that obtained from Nepal earthquake. Nepal earthquake ground motion is considered in this study owing to the similarity of seismicity and geographical locations. It can be seen from Figure 2 that shapes of acceleration response spectra are also very much similar. Moreover, masonry infilled RC buildings in Bhutan and Nepal are also similar which were either built without proper design or were designed according to Indian Seismic Code (Chaulagain et *al*. (2014). Hence, structural response predicted from Nepal earthquake could indicate the possible damages of buildings in Thimphu in the event of future earthquake. On the other hand, performance of typical buildings in Thimphu could also indicate the performance of masonry infilled buildings in Kathmandu during the last earthquake. As shown in Figure 13, gravity designed ‘3 storey old’ building collapses, while ‘3 storey new’ and ‘6 storey’ buildings designed according to Indian Code are within the life safety and collapse prevention performance limits respectively. A very similar kind of performances were exhibited by masonry infilled frame buildings in Nepal during the last earthquake.

From Figures 11 and 12, it is evident that none of the buildings would be fit for immediate occupancy under the given ground motions. The ‘3 storey new’ building exhibits life safety performance level under 475 year return period ground motion and exhibits the best performance among three buildings. For 2475 return period ground motion, its interstorey drift is within life safety limit at rock site, collapse prevention limit at shallow stiff soil and soft rock sites and exceeds the collapse prevention limit at soft soil site. The performance of ‘6 storey’ building is within the life safety limit at rock and soft rock sites and collapse prevention limit at shallow stiff soil for 475 year return period ground motions. Under 2475 year return period ground motion, maximum interstorey drift of ‘6 storey’ building is within the collapse prevention limit at rock site, while it crosses past collapse prevention limit at other sites indicating the collapse of building. The performance of ‘3 storey old’ building is similar to that of ‘6 storey’ buildings with slightly higher interstorey drift values. As shown by the dotted lines in the Figures 11-13, soil structural interaction has limited effect at soft rock site and slightly detrimental effect at shallow stiff soil. However, SSI has significant effect at soft soil site with detrimental and beneficial effects under 475 and 2475 year return period ground motions respectively. The effect of SSI is however found to be dependent on soil sites and period of building.

Judging by the interstorey drift values, the performance of ‘3 storey new’ and ‘3 storey old’ buildings are as expected, but the performance of ‘6 storey’ buildings was expected to be better than that predicted for being designed according to the Indian Seismic Code. It could be either due to the inadequate design or inadequacy of Indian Seismic Code to be used in Bhutan. It could also be due to the use of very low concrete grade in spite of having 6 full floors.

8 CONCLUSION

With large number of masonry infilled RC buildings combined with the high seismicity of the area, there exist a real seismic risk in Bhutan. Until 1997, these buildings were haphazardly built and no proper design procedure was followed. After 1997, the use of Indian Seismic Code has started for the design of buildings, but its applicability for the site conditions in Bhutan is still in question. The performance of these buildings under seismic action is relatively unknown with very limited studies in the past.

This study is aimed at realistically assessing the performance of masonry infilled RC buildings in Thimphu using the ground motions predicted at generic soil sites in Thimphu. Three typical masonry RC buildings that represent the masonry infilled RC buildings constructed before and after the adoption of Indian Seismic Code are considered. The nonlinear analysis and performance assessment software Perform 3D is used for the nonlinear analysis of the buildings. The numerical model is first calibrated with experimental results and then applied for the response prediction of the typical masonry infilled RC buildings. The performance of the buildings are assessed based on the interstorey drift and using the performance limit states proposed by Ghobarah (2004). The buildings are also assessed for the recorded ground motion of the recent Nepal earthquake to understand the performance of buildings under such earthquake.

The typical masonry infilled RC buildings in general are within the life safety and collapse prevention performance levels under 475 return period ground motion, while under 2475 return period ground motion, performance level shift to collapse prevention level and exceed the collapse prevention level. As expected, buildings designed according to Indian Seismic Code perform better than that of gravity designed building. SSI has significant effect at soft soil site and limited effect at other soil sites. It is interesting to note that performance of typical buildings under the ground motion of Nepal earthquake depict the similar performances exhibited by the masonry infilled RC buildings in Kathmandu during the earthquake on 25th April 2015. However, it is to be noted that the performance of the buildings predicted in this study are indicative since it is based on the performance levels proposed by Ghobarah which may not be applicable to buildings in Bhutan.

9 ACKNOWEDGEMENT

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