
SEISMIC FRAGILITY ASSESSMENT AND RETROFIT OF A GOVERNMENT HOSPITAL BUILDING IN CHITTAGONG, BANGLADESH

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Abstract: Chittagong Medical College Hospital (CMCH) is one of the most important government hospitals in Bangladesh. It is located in the heart of Chittagong city, the only port city of Bangladesh. Bangladesh National Building Code (BNBC) is the only official document, which has been used since 1993 as guidelines for seismic design of buildings. As per the guidelines of BNBC, the CMCH building was designed for an earthquake ground motion having a return period of 200 years. However, the revised version of BNBC has suggested that the building structures shall be designed for an earthquake ground motion having a return period of 2475 years. It is mentioned that a single seismic performance objective, the life safety, of the building is considered in both versions of BNBC. Considering the significant importance of CMCH building in providing the emergency facilities during and after the earthquake, it is indispensable to evaluate its seismic vulnerability for the two types of earthquake ground motion records having return period of 200 (Type-I) and 2475 (Type-II) years. In this regard, this paper deals with the seismic vulnerability assessment of the existing ancillary building (AB) of CMCH. The seismic vulnerability of building is usually expressed in the form of fragility curves, which display the conditional probability that the structural demand (structural response) caused by various levels of ground shaking exceeds the structural capacity defined by a damage state. The analytical method based on elastic response spectrum analyses results is used in evaluating the seismic fragility curves of the building. To the end, 3-D finite element model of the building subjected to 18 ground motion records having PGA of 0.325g to 0.785g has been used in the response spectrum analysis in order to evaluate its inter-story-drift ratio (IDR), an engineering demand parameter (EDP) for developing fragility curves. The analytical results have shown that structural deficiencies exist in the existing ancillary building (AB) for the Type-II earthquake ground motion record, which requires the building to be retrofitted to ensure that the existing ancillary building (AB) becomes functional during and after the Type-II earthquake ground motion record.

Keywords: *Seismic Fragility, Fragility Curve, Retrofit, Inter-story-drift, Seismic vulnerability and Bangladesh National Building Code*

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1.0 Introduction

The hospital is very essential for human civilization which provides life saving medical care on a daily basis to the community people as a basic need (Hasan, 2015). The community expects from the hospital and its staff to serve with proper medical treatment in an emergency as well as when they get seriously injured or become seriously ill. All hospital facilities need to be capable of continuing operation during and after natural disasters; like an earthquake. That's why, every building code in the world considers higher important factors for seismic design of hospital building to withstand operation during and after an earthquake. The Bangladesh National Building Code (BNBC, 1993) suggests for consideration of 25% higher importance in design than the standard occupancy building. Similarly, in Euro Code 8, Australian Building Code (AS11704, 1993) and New Zealand Building Code (NZS4203, 1992) suggest for providing 75%, 20% and 30% higher importance than normal building respectively. After taking these kind of design provision the world has suffered huge loss of confidence, as well as economic losses on account of damages occurred in hospital buildings from previous earthquake disaster. Damage of existing hospital building structures was observed in 1971 San Francisco Earthquake, 1994 Northridge Earthquake and 2001 Bhuj Earthquake etc.

Bangladesh is geographically located in a high seismic risk zone. Based on the records of the Geological Survey of Bangladesh this country has experienced at least 465 earthquakes of minor to moderate size from 1971 to 2001 (Paul and Bhuiyan, 2009). Seismic experts consider recent repeated earthquakes of low to medium size is an advance warning of a massive and potentially disastrous earthquake in the near future; as these tremors fail to release large part of the stress that accumulates within fault rupture zones (Bolt, 2005 and DPF, 2002). It is also experimentally proved that cyclic deterioration is responsible for damage in the buildings that have experienced successive earthquakes. Examples of buildings that had damaged during earlier earthquakes and damaged more severely in next earthquakes mentioned in the report by Stratta and Wyllie (Stratta and Wyllie, 1979). Most of the existing hospital buildings in Bangladesh were constructed before implementation of seismic design guideline BNBC (1993). Moreover the BNBC (1993) has been revised recently to consider the 2% probability of occurrence of earthquakes (Type-II) in 50 years having return period of 2474 years. In case of BNBC-1993 the design earthquake having a 22% probability of occurrence in 50 years was considered. So it is needed to assess about the safety of the existing hospital building which was designed by following seismic design guideline provided in BNBC (1993).

In view of the importance of hospital building, it is a key contemporary issue to lower the loss of hospital functionality as much as possible against earthquake to ensure continuity of community life. So it is very important to check the seismic vulnerability of the hospital building, which is often explicitly expressed as fragility curves. Fragility curves indicate the conditional probability of a structure that sustains a particular degree

of damage when subjected to a given level of ground shaking (Billah *et al.*, 2013). In this research work the Ancillary Building (AB) of the Chittagong Medical College Hospital (CMCH) was considered. The design procedure of AB was followed as per design guideline provided in BNBC (1993).

From the study, it is found that the existing hospital building is vulnerable against different damage states for design ground shaking suggested in revised BNBC. To reduce the possibility of seismic risk of this life line structure, effective retrofitting strategy using base isolation devices was selected as well as designed according to the guideline provided in JRA, 2002. The most effective advantage of using base-isolation devices is, it not only provides safety against collapse, but also largely reduces damage, which is essential for the hospital building. It is observed from the fragility curves of hospital building (existing & retrofitted) that, the possibility of seismic risk is greatly reduced, which makes the structure safer for seismic design load as per revised BNBC (1993).

2.0 Methodology of Seismic Fragility Function

Fragility curves are mostly used for realistic seismic vulnerability assessment of a structure which was utilized here to find the vulnerability of AB. In the absence of adequate damage data, fragility function was developed using a variety of analytical methods such as elastic response spectrum analysis (Hwang *et al.*, 2001), nonlinear time history analysis (Choi *et al.*, 2004) and non-linear static analysis (Shinozuka *et al.*, 2000) etc. Towashiraporn *et al.*, 2004 proposed a method for formulating the meta-model for fragility curve generation using response surface method, where the input variables are composed of two components: random variable and a control variable. The random variable defines uncertainties in structural properties, while the control variable is deterministic with its fixed values characterizing different response prediction models. Figure 1 represents the flowchart for developing fragility curve. Firstly the selection of building type (hospital building) and the modelling of structure were done. Then a number of past ground excitations were selected for that site and structural analysis was performed. In this research work the generated models were analysed for various response spectrum loading and corresponding structural responses (IDR) were computed. The PGA of corresponding IDR values was evaluated and distributed to performance levels and damage states suggested in FEMA-273. After that probability distribution function were generated. From the probability distribution function (PDF) cumulative distribution function (CDF) is developed which is fragility function.

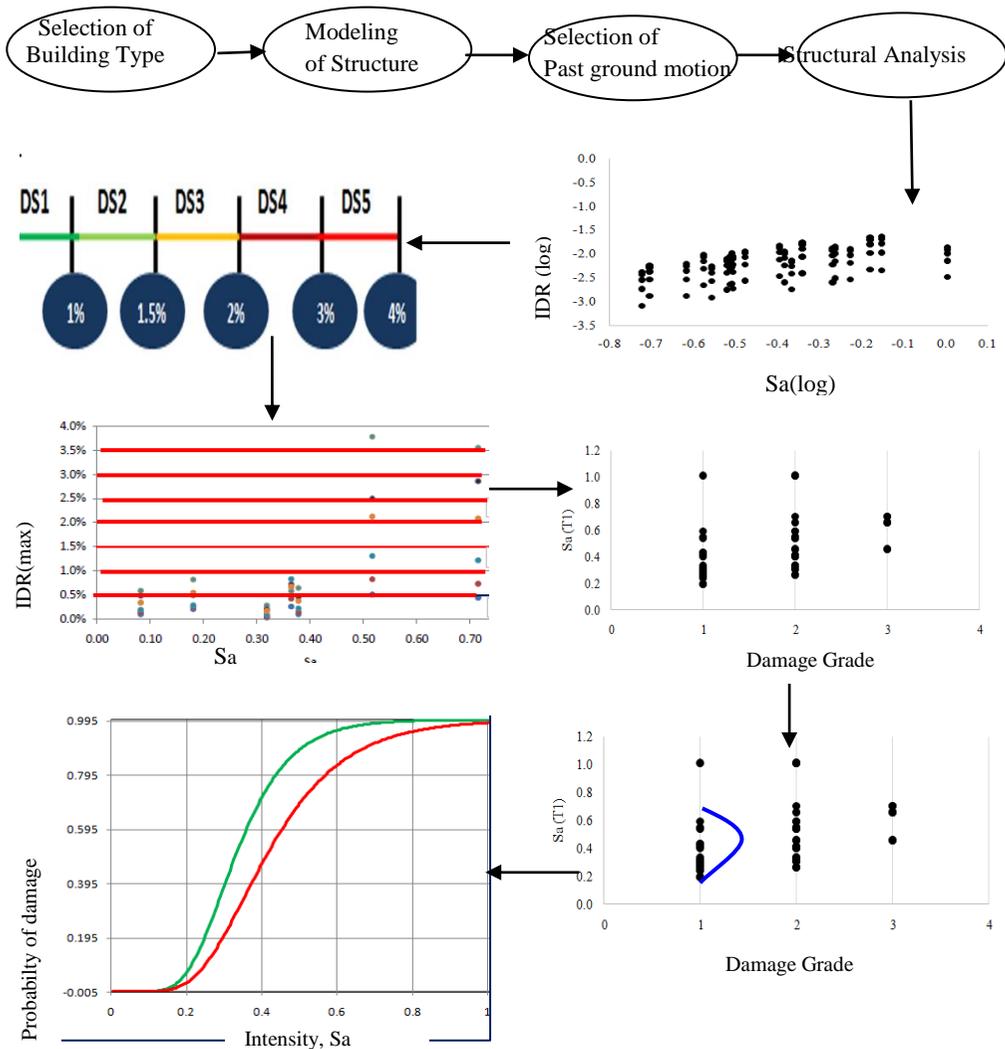


Figure 1: Flow chart diagram for developing Fragility Curve

3.0 Characteristics of Damage States

Fragility curves represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation (Shinozuka *et al.*, 2000 and Ellingwood *et al.*, 1980). The structural damage states are defined by three discrete structural performance levels and two intermediate structural performance ranges. The structural performance levels are the immediate occupancy level (DS-1), the life safety level (DS-3) and the collapse prevention level (DS-5) where structural performance ranges are the damage control range (DS-2) and the limited safety range (DS-4). Structural

performance level DS-1, immediate occupancy, indicates the post-earthquake damage state in which very limited structural damages occur. In this state, basic vertical and lateral-force-resisting systems of the structure retain nearly all of their pre-earthquake strength and stiffness as per FEMA-273. The risk of life threatening injury as a result of structural damage is very low, although some minor structural repairs may be needed, but these would generally not be required prior to pre-occupancy. Structural performance level DS-3, life safety, means the post-earthquake damage state in which significant damage to the structure occurs, but the partial or total collapse will not happen. In this damage level some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. During the earthquake, injuries may occur; however, the overall risk of life threatening injury as a result of structural damage is low. Though the damaged structure is not a near collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to pre-occupied. Structural performance level DS-5, collapse prevention, indicates that the building is on the verge of experiencing a partial or total collapse. Major damage to the structures occur, potentially including significant degradation in the stiffness and strength of the lateral force-resisting system, as well as large permanent lateral deformation of the structure and to more limited extent degradation in vertical-load-carrying capacity.

Table1: Structural performance levels and damage states as per FEMA-273

Elements	Type	Structural Performance Levels				
		Immediate Occupancy (DS-1)	Damage Control Performance Range (DS-2)	Life Safety (DS-3)	Limited Safety Performance Range (DS-4)	Collapse Prevention (DS-5)
Concrete Frame	Drift (transient)	<1%	<1.5%	<2%	<3%	<4%

However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for pre-occupancy, as the aftershock activity could induce collapse. Structural performance range DS-2, damage control, means the continuous range of damage states that entails less damage than the life safety level, but causes more damage than that is defined for the immediate occupancy level. Designing for damage control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for immediate occupancy is excessive. Structural performance range S-4, limited safety, means the continuous range of damage states between the life safety and collapse prevention levels. In this study the inter story drift ratio (IDR) of the

hospital building was adopted as damage index (DI). For different damage states and performance ranges the limit value of IDR enlisted in Table 1.

4.0 Physical Descriptions and Modelling of Hospital Building

Chittagong Medical College Hospital (CMCH) was established in the year 1957 and started functioning in the present place in 1960 with only 120 beds and few outpatient services (https://en.wikipedia.org/wiki/Chittagong_Medical_College, 25/10/2016). The Ancillary Building (AB) was built in 2001 which is ten storied with mat foundation. This building has been providing medical treatment for 24 hours in a day and all surgical units required to be functioning all time. The total area of 1st floor is 12661.11 sq. ft. Figure 2 represents the AB of CMCH. Two types of column (rectangular and circular) have been used in this building. The rectangular columns having six different physical dimensions were constructed here with large size of 750 mm X 625 mm. Three types of circular column exist there with a maximum diameter of 625 mm. From the collected data, it has been found that the compressive strength, modulus of elasticity and poisson's ratio of concrete are 25 MPa, 23670 MPa and 0.2 respectively. The tensile strength of steel is 415 Mpa with a modulus of elasticity 200000 MPa. For modelling purpose, the compressive strength, modulus of elasticity and poisons ratio of clay brick were supposed 13.7 MPa, 12930 MPa and 0.19 respectively. An analytical model of existing AB shown in Figure 3 was developed using finite element programming SAP 2000 which represents the actual condition of the hospital building.



Figure 2: 3-D view of Ancillary Building of CMCH

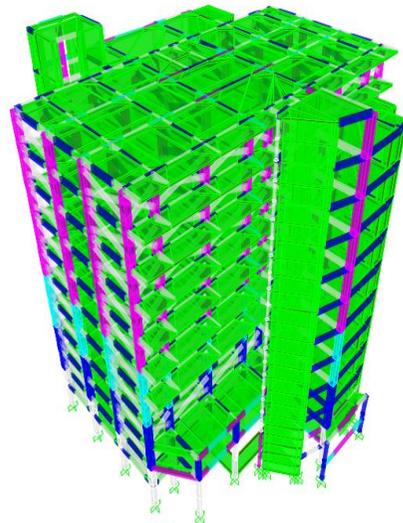


Figure 3: Analytical Model of Ancillary Building of CMCH

Beam and column elements were modelled as a frame element; floor, roof, mat foundation and shear wall were modelled as shell element. The existence of masonry infill was modelled as an equivalent strut model by stafford-Smith(Stafford, 1966).

5.0 Ground Motion Records

The location of considered hospital building is in the highly seismic zone of Bangladesh. To compute the realistic structural responses of the hospital building, eighteen ground motion records were considered. Figure 4 and 5 are the plot of the ground motions those were selected for performing structural analysis of the hospital building. The PGA of the ground motion records varied from 0.325g to 0.78g.

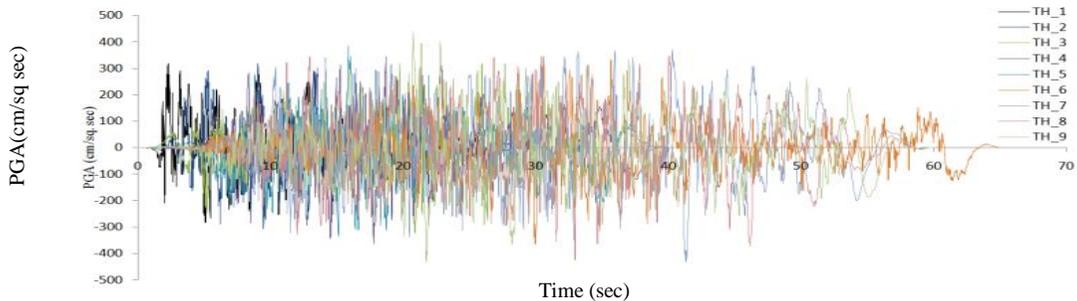


Figure 4: Earthquake Ground Motion (TH_1 to TH_9)

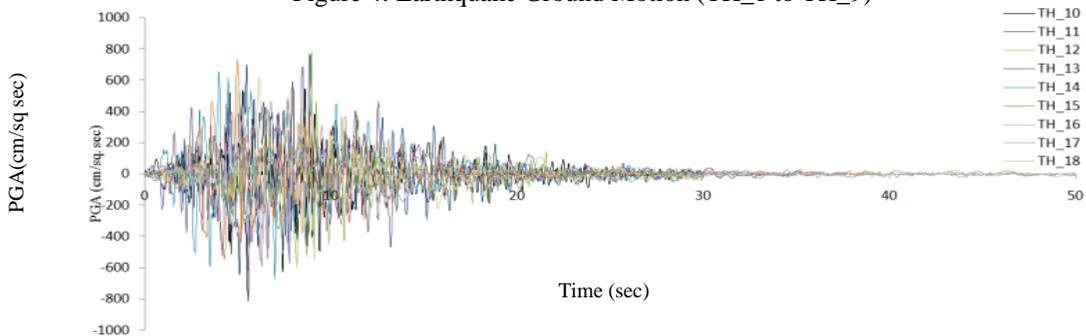


Figure 5: Earthquake Ground Motion (TH_10 to TH_18)

For performing elastic response spectrum analysis the response spectrum load were generated from the corresponding earthquake ground motion. Figure 6 and 7 are representing the eighteen response spectrum loading those were used to compute the inter-story-drift (IDR) as structural response of the structure.

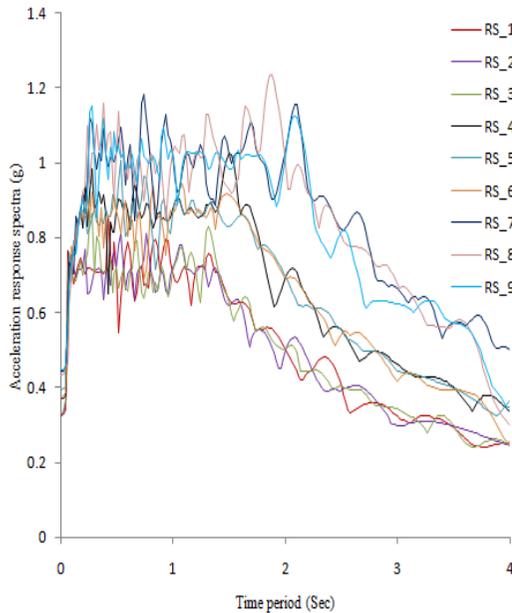


Figure 6: Response Spectrum (RS_1 to RS_9)

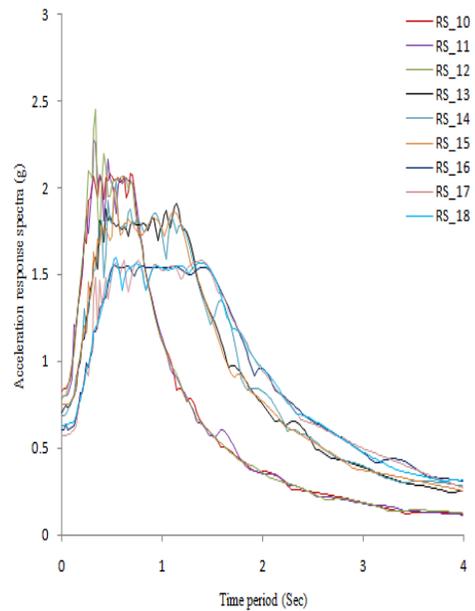


Figure 7: Response Spectrum (RS_10 to RS_18)

6.0 Elastic Response Spectrum Analysis of AB

For analysing the performance of structure under earthquake motions elastic response spectrum analyses were done in this research. In this method, it is assumed that a single degree of freedom system has to be excited by a ground motion in order to obtain the response spectrum curves for structural responses. The number of requests modes was selected in such a way that their combined participating mass is at least about 90% of the total effective mass in the structure. When the number of significant modes was established, several methods were used for the purpose of estimating the peak response values. The Square Root Some of Squares (SRSS) of the maximum modal values are one of the popular methods. The existing Ancillary Building (AB) was analysed for 18 response spectrum loadings. The results of response spectrum analysis are graphically presented in Figure 8 and 9. From the Figure 8 and 9, it is seen that the inter-story drift value initially increases up to maximum IDR values till 4th floor and after that it decreases with increase of floor levels.

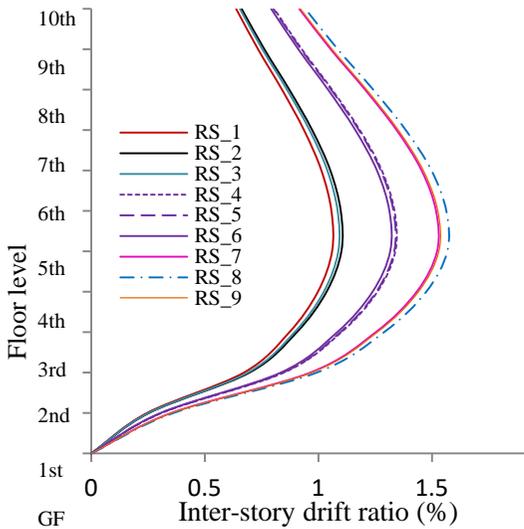


Figure 8: IDR (%) of Existing AB for RS_1 to RS_9

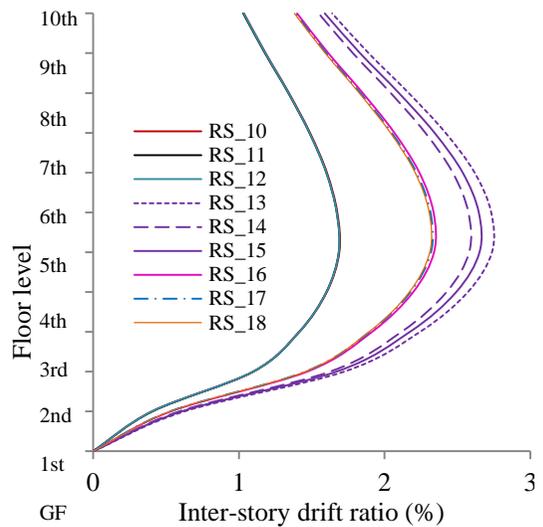


Figure 9: IDR (%) of Existing AB for RS_10 to RS_18

7.0 Fragility Assessment of AB

By using the IDR values as Engineering Demand Parameter (EDP), fragility curves were developed for three damage states (DS-1, DS-3 and DS-5) and two performance ranges (DS-2 and DS-4). The probability of exceedance of each damage state and damage range for design ground motions (Type-I and Type-II) was also evaluated. Figure 10 represents the fragility curve for immediate occupancy level; the probability of exceedance of DS-1 is 14% for Type-I earthquake and the value increases to 79% for Type-II earthquake. Figure 11, which resembles the fragility curve for DS-2 showing that there is no probability of exceedance of DS-2 for Type-I ground motion. In case of Type-II seismic excitation, there is a probability of exceedance of damage control performance range which is 19%. Figure 12 indicates a fragility curve of considered AB for life safety (DS-3) performance level, which states that there is no probability of exceedance of DS-3 level for Type-I while for Type-II earthquake the probability of exceedance of this damage state is 3%. For DS-4 and DS-5 there is no probability of exceedance for considering seismic zoning coefficient and they are plotted in Figure 13 and 14.

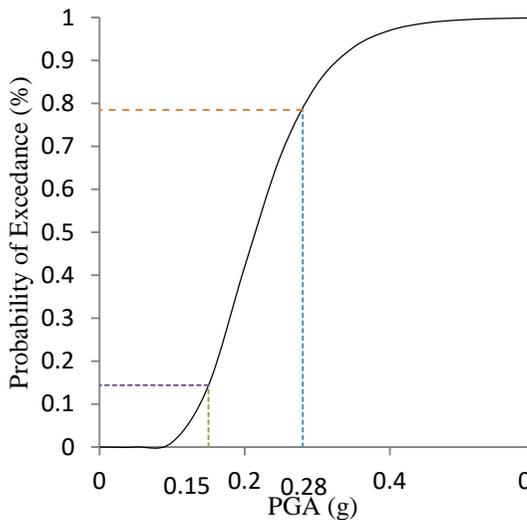


Figure 10: Fragility curve of Existing AB for DS-1

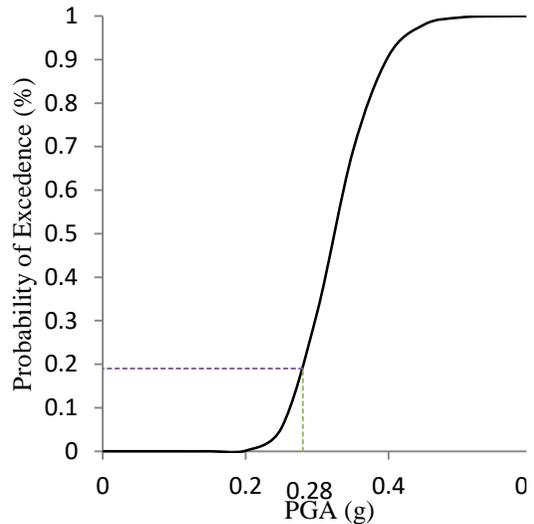


Figure 11: Fragility curve of Existing AB for DS-2

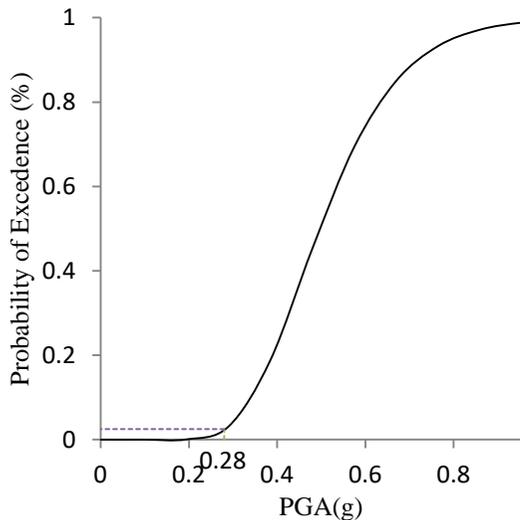


Figure 12: Fragility curve of Existing AB for DS-3

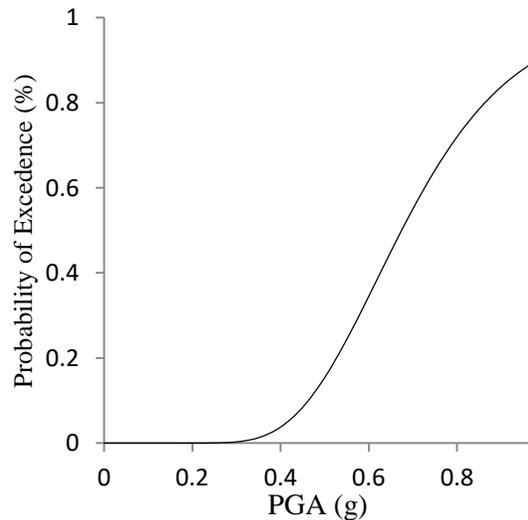


Figure 13: Fragility curve of Existing AB for DS-4

8.0 Retrofitting of AB Using Base-isolation Devices

After the detailed study of fragility curves of existing AB against different damage states it is clear that the existing AB is vulnerable for PGA of 0.28g (Type-II EQ). To reduce the risk during earthquake, proper retrofitting strategy is suggested. Among the various available retrofitting strategy, base-isolation devices are more effective for

building retrofits because it not only provides safety against collapse, but also causes damage reduction. The damage reduction is highly required for facilities that need to remain operational after severe earthquakes such as emergency response centres, hospitals and fire stations.

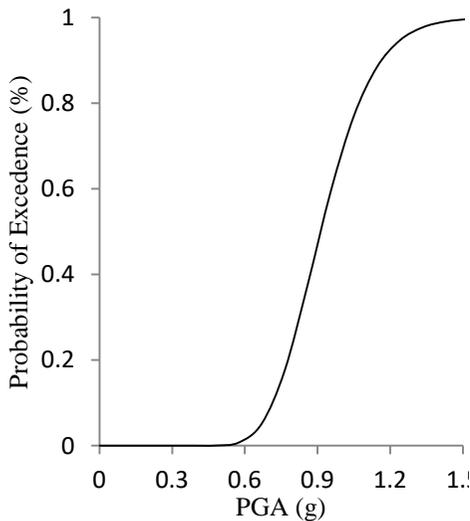


Figure 14: Fragility curve of Existing AB for DS-5

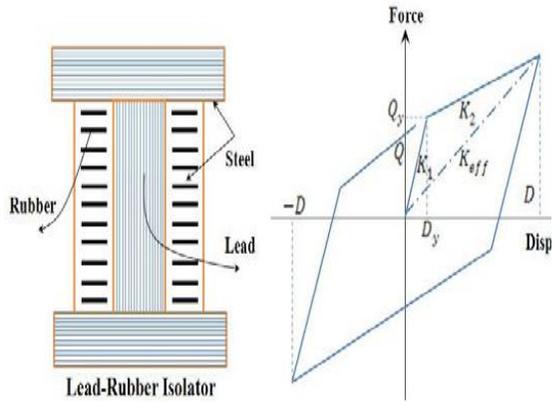


Figure 15: LRB components with Force Displacement curve

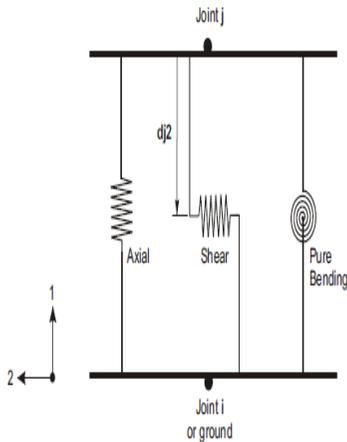


Figure 16: Three of the six independent springs in link

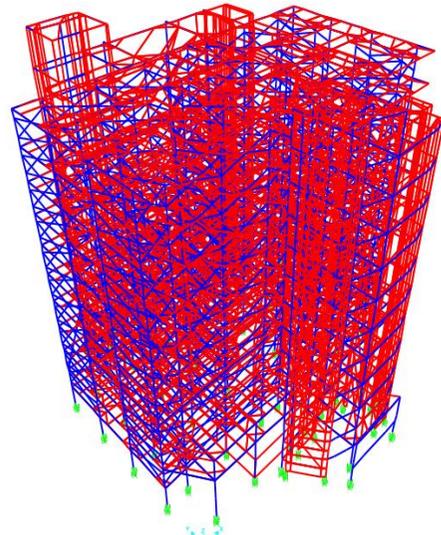


Figure 17: Analytical model of retrofitted AB of CMCH

There are various types of base-isolation devices that are widely used for retrofitting purposes. The elastomeric LRB is effectively used for building retrofitting which consist of two steel plates; one is placed at the top and remaining one is placed at the bottom of the device, with several alternating steel shims and central lead core. The purpose of the top and bottom plates is to compact the whole system. The aim to use rubber, steel shims and central lead core are to provide lateral flexibility, vertical load carrying capacity and damping accordingly. When the structure with an isolation system experiences earthquake, the rubber layers deform laterally by shear deformation, allowing the structure to translate laterally. Figure 15 represents the X-section of LRB devices with force displacement relationship. For designing of BI devices, the guideline provided by the Japan Road Association, 2002 is followed. According to the guideline, shear strength of rubber is assumed as 6 MPa. The standard value of shear strain of rubber is considered as 100% in the USA and 200% in Japan. In this research work, it was assumed the value is 175%. JRA, 2002 guideline suggests that the maximum horizontal displacement of base isolation device should lie in between 100 mm to 400 mm. Eq. (1), (2), (3), (4) and (5) have been used for computing properties of LRB.

$$k_1 = 6.5k_2 \quad (1)$$

$$k_2 = (F - q_d) / u_{(Be)} \quad (2)$$

$$q_d = q_0(\gamma_e)A_p \quad (3)$$

With,

$$q_0(\gamma_e) = b_0 + b_1\gamma_e \quad (4)$$

$$F = G_e A_e \gamma_e + A_p q_0(\gamma_e) \quad (5)$$

By using the above equations and data, 31 BI devices have been designed having a maximum size of 1300 mm*1300 mm and the minimum size of 250 mm*250 mm. In SAP 2000, Isolators are modelled by using link/support element option where the link is considered as a two-node element connected by six springs. Detailed descriptions of the link element are given in Figure 16. The shearing behaviour of base-isolation is based on the model proposed by Park *et al.*, 1986 and extended by Nagarajaiah *et al.*, 1991. For the elastomeric bearing option in the link element, nonlinear (bilinear) properties are assigned to the two horizontal shear directions, but only linearelastic behaviour is accommodated here for the remaining axis and three rotational directions. Figure 17 represents the analytical model of base-isolated ancillary building.

9.0 Fragility Assessment of Base-isolated AB and Comparison with Existing AB

By using the IDR values that are computed from the elastic response spectrum analysis of base-isolated AB, fragility curve is developed for three performance levels (DS-1, DS-3 and DS-5) and two performance range (DS-2 and DS-4) which are compared with the fragility curve of existing AB (before retrofitting) for designing earthquakes suggested by BNBC-1993 and revised BNBC-1993. Figure 18 and 19 represents the fragility curve of AB (before and after retrofitting) for immediate occupancy (DS-1) and damage control performance range (DS-2) respectively. From Figure 18, probability of exceedance of DS-1 for existing AB for Type-I EQ is 14% and it reduces to 6% when base isolation devices is used for retrofitting purposes. Similarly for Type-II EQ the probability of exceedance for

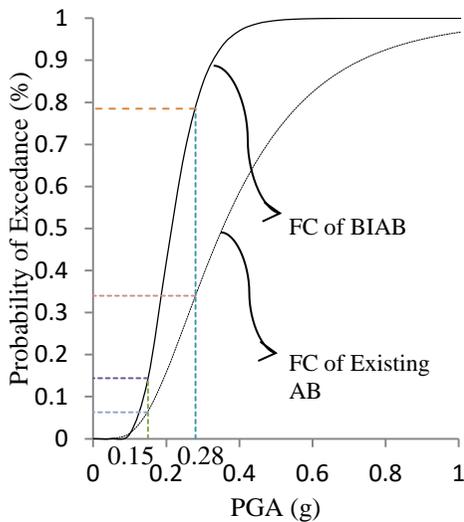


Figure 18: Fragility curve for DS-1

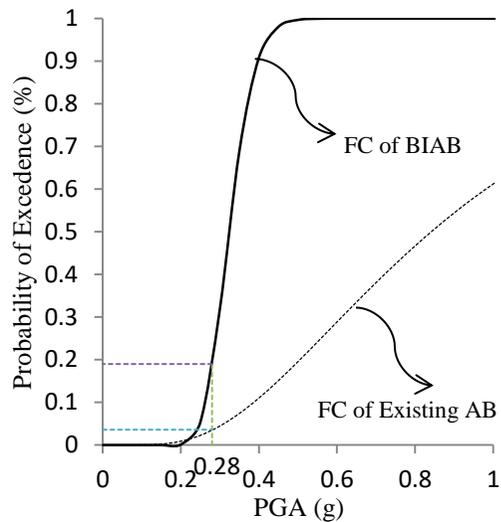


Figure 19: Fragility curve for DS-2

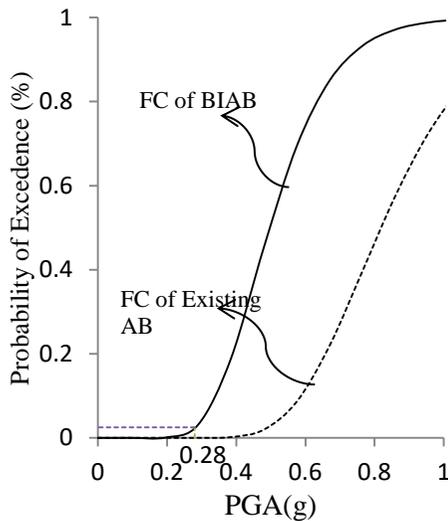


Figure 20: Fragility curve for DS-3

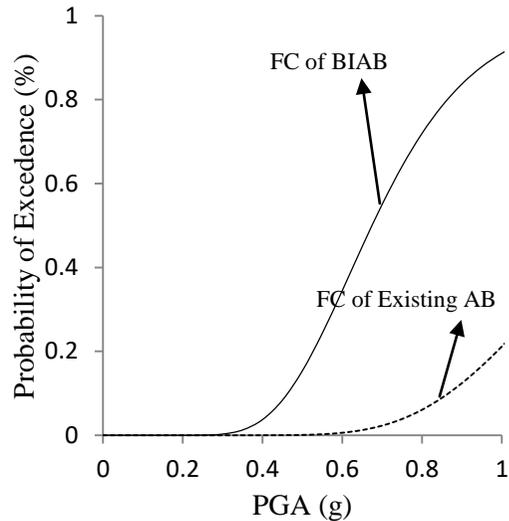


Figure 21: Fragility curve for DS-4

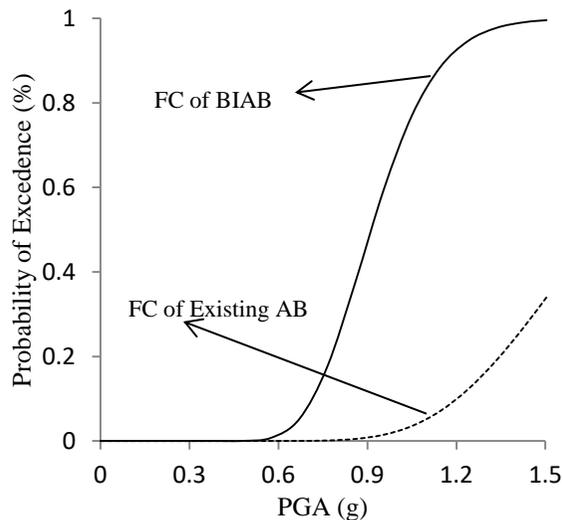


Figure 22: Fragility curve for DS-5

benchmark AB is 78%, whereas the value decreased to 34% for retrofitted AB. It has been found from Figure 19 that there is no probability of exceedance of DS-2 against Type-I EQ for hospital buildings. But for Type-II EQ the probability of exceedance of DS-2 for benchmark AB is 19%, whereas the value turned into 4% for base-isolated AB. From Figure 20 which represents the fragility curve for DS-3, it is observed that the probability of exceedance of benchmark AB for Type-I EQ is zero. For Type-II EQ, the probability of exceedance is 3% in the case of existing AB but there is no probability of exceedance for DS-3 when it is retrofitted. Figure 21&22 presents fragility curves of

hospital buildings (before and after retrofitting) for DS-4 and DS-5 respectively. From these two figures (Figure 21 & 22) it is clear that the building is completely safe for designing earthquake suggested in BNBC.

10.0 Conclusions

The fragility assessment of the considered hospital building before and after retrofitting is carried out here. From the fragility curve of DS-1, it is found that the probability of exceedance of DS-1 for benchmark hospital building (Existing AB) is 14% for Type-I EQ for this site. But due to application of base isolation devices, it reduces to 6%. Similarly, in the case Type-II EQ the probability of exceedance of immediate occupancy level is 78% of benchmark hospital building. After implementing base isolation devices, it was decreased to 34%. From the fragility curve for DS-2, it has been found that for Type-I EQ the probability of exceedance for DS-2 is zero for hospital building before and after retrofitting. But for Type-II EQ the probability of exceedance of DS-2 for benchmark hospital building was 19%, while it reduced to 4% for base isolated AB. The probability of exceedance of hospital building for DS-3 is zero for Type-I EQ. But for Type-II EQ the probability of exceedance for DS-3 is 3% for existing AB, but it becomes zero for retrofitted AB. It is clear from the fragility curve that for DS-4 and DS-5, there is no probability of exceedance of corresponding damage state of hospital buildings for considering design earthquake (PGA of 0.15g and 0.28g). So it can be said that for all considered damage states and damage ranges, the probability of exceedance is greatly reduced due to the application of base isolation devices as a retrofitting technique. From the comparative fragility assessment of AB, before and after retrofitting against various damage states, it is found that due to adopting base isolation devices as rehabilitation approaches the vulnerability is greatly reduced when compared to existing AB. It is also confirmed that the existing hospital building is vulnerable for Type-II EQ. But by implementing suggested (BI device) retrofitting strategy, it can be kept functional during and after an earthquake. So it is clear from this research that vulnerability of all hospital buildings in Bangladesh need to be evaluated and proper retrofitting strategy should be adopted to mitigate the risk for up-coming disasters.

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