SEISMIC RESPONSE EVALUATION AND RETROFIT OF A FIVE-STORIED RC BUILDING DAMAGED DUE TO THE 2017 TRIPURA EARTHQUAKE

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ABSTRACT

The 2017 Tripura earthquake caused wide spread structural and nonstructural damage to some buildings in the North-Eastern part of Bangladesh. To investigate the dynamic behavior of a building, seismic response simulation was carried out. For time history analysis, synthetic ground motions were used since no ground motion was available for the Tripura earthquake. When actual damage was compared with the analytical result from the nonlinear time history analysis, reasonable correlation was obtained. It was found that low strength concrete, bare frames in the ground floor, insufficient shear reinforcement, inadequate column sizes and weak beam-column joints were mainly responsible for the damage due to this earthquake. Both strength oriented and ductility oriented retrofitting approach were considered for rehabilitation. From the nonlinear analysis, it can be concluded that strength oriented retrofitting is the best option for strengthening for local design and construction conditions in Bangladesh.

Keywords: Tripura earthquake, Seismic evaluation, Seismic rehabilitation, Non-linear analysis.

1. INTRODUCTION

On January 3, 2017, an earthquake of Magnitude (M_w) approximately 5.7 on the Richter scale hit India-Bangladesh Border region with a focal depth of 30 km. This shallow strike-slip earthquake has originated from the Indian Plate which lies beneath the Burmese Plate. While this moderate intensity earthquake occurred in Dhalai district in India, it caused some damage in some parts of Bangladesh close to India-Bangladesh Border region. At least seven buildings in this region experienced structural and nonstructural damage. One building named “Sarkari Shishu Paribar” situated in Sreemangal Upazila in Maulvibazar District in the Sylhet Division of Bangladesh suffered severe damage due to this earthquake. Damaged to this building due to moderate earthquake (MMI intensity V, according to USGS) is causing real concern. Most of the buildings in Bangladesh are constructed without following the Building Code. These buildings are very vulnerable to moderate to strong earthquakes.

Bangladesh has experienced eight major earthquakes over the last 250 years. Bangladesh is situated in the place where the Indian Plate, the Burmese Plate & the Eurasian Plate meet. Owing to the fact, Bangladesh is one of the tectonically active seismic regions in the world, probability of occurrence of moderate to strong earthquake is very high. The Bangladesh National Building Code (BNBC) was prepared in 1993 and enacted in 2006. For this reason, a large number of buildings in Bangladesh which were constructed before 2006 have not complied with the seismic provisions of BNBC. Under this circumstances, it is necessary to assess seismic vulnerability and then rehabilitate buildings according to the seismic provisions of the building code.

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2. TARGET BUILDING

2.1. Outline

The target building of this individual study which was subjected to the Tripura Earthquake, January 3, 2017 experienced some damage in the ground floor. This building is a five-storied reinforced concrete residential building named as "Sarkari Shishu Paribar". It is located in Sreemangal Upazila of Bangladesh. According to BNBC 2015 Draft, this area is in Zone IV (Z = 0.36) and the site class is SD ($\overline{V}_s \leq 180$ m/s). This building is located 40 km away from the epicenter of the 2017 Tripura earthquake. The isometric view and location of the target building are shown in Figure 1 and Figure 2 respectively.

Figure 1. Isometric view of the target building.

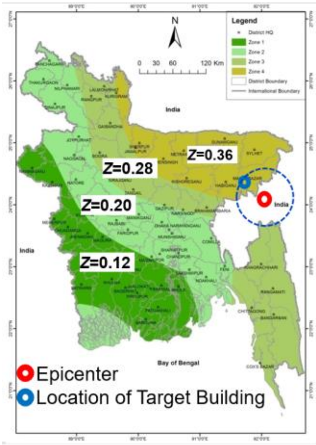

Figure 2. Location of the target building and the epicenter in the seismic zoning map.

2.2. Description of the damage

One column in the ground floor damaged severely. Big shear cracks and concrete spalling were found at the top of the column (Figure 3). Flexural Cracks were found just below beam-column Joints for most of the ground floor columns. One masonry wall in the ground floor suffered heavy damage (Figure 4). At first, diagonal cracks initiated and then sliding failure occurred. Lots of horizontal cracks were found in the masonry walls of the staircase in the ground floor.

Figure 3. Big shear crack in column.

Figure 4. Damaged masonry infill.

2.3. Damage ranking of the structural members

Damage classes based on "the Japanese Damage Evaluation Guideline" (JBDPA 2001a) were assigned to structural members of the target building according to the post-earthquake field survey after the Tripura Earthquake, 2017. Figure 5 shows the damage ranking of each structural vertical member and associated average ductility factor (μ) proposed by Maeda *et al* (2012).

Table 1. Ductility Factors based on Damage Class (Maeda *et al*, 2014).

Damage class	Average ductility factor (μ)
0	0.05
I	0.5
II	1.5
III	2.5
IV	4
V	5.5

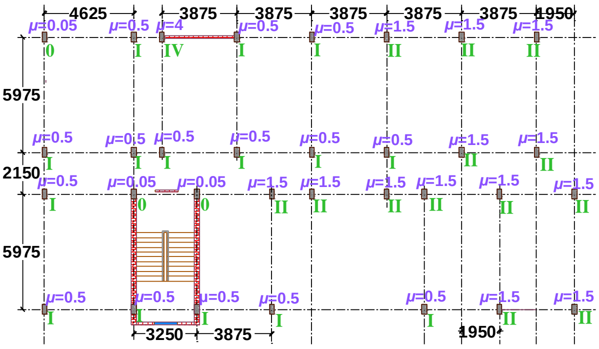

Figure 5. Damage distribution in the ground floor and associated average ductility factors.

3. SEISMIC RESPONSE SIMULATION

3.1. Input ground motion generation

No real ground motion acceleration data is available for the Tripura Earthquake, 2017. Hence, synthetic ground motion data is generated to perform simulation of seismic responses. For generation of synthetic ground motion, Kanno *et al*. (2006) attenuation relation has been used. The empirical attenuation formulas used are presented in Eq. (1) and Eq. (2):

$$\log pre = a_1 M_w + b_1 X - \log(X + d_1 10^{e_1 M_w}) + c_1 + \varepsilon_1 \quad when\ \ D \leq 30km \tag{1}$$
$$\log pre = a_2 M_w + b_2 X - \log(X) + c_2 + \varepsilon_2 \quad when\ \ D > 30km \tag{2}$$

where, *pre* is the PGA, PGV or 5% damped response spectral acceleration in cm/sec^2, M_w is Moment magnitude, X is source distance (km), D: Focal depth (km) and $\varepsilon_2, \varepsilon_2$ are errors. To take into account the local soil condition of the site, soil amplification factor is used based on BNBC 2015.

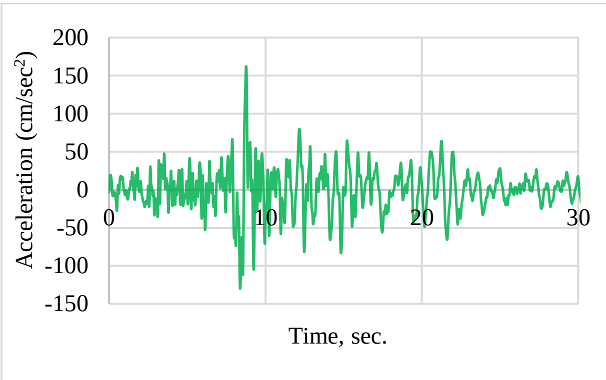

Figure 6. Synthetic ground motion.

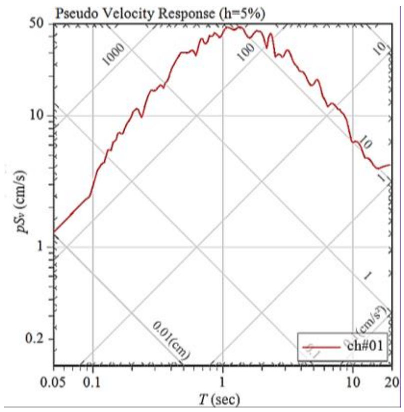

Figure 7. Pseudo velocity response spectrum (h = 5%).

3.2. Ductility factors obtained from nonlinear time history analysis

Non-linear dynamic analysis is performed using STERA 3D, a finite element method (FEM) based software developed by Dr. T. Saito. Synthetic ground motion is applied for this analysis. From the non-linear time history analysis, ductility factors for each vertical element (μ) are obtained. After that these ductility factors are compared with the average ductility factor assigned in Figure 5 to verify simulation result in terms of ductility factors.

The comparison between average ductility factors based on damage class (Figure 5) and the ductility factors (Figure 8) acquired from non-linear time history analysis using synthetic ground motion shows reasonable correlation. The values of ductility factors for vertical structural elements obtained from time history analysis are close to the average value of ductility factors depending on damage rankings of structural members.

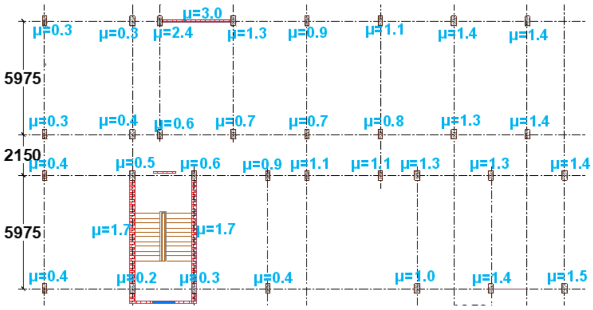

Figure 8. Ductility factors in the ground floor through nonlinear time history analysis.

4. SEISMIC EVALUATION AND REHABILITATION

4.1. Seismic evaluation by JBDPA 2nd Level Screening

For seismic evaluation of the target building, shear strength of masonry infill is considered only for the transverse direction. In the longitudinal direction, Contribution of masonry infill walls is minor in the longitudinal since most of the walls are out of the frames and the rest of the masonry walls have the big openings.

4.1.1. Seismic demand Index (I_{SO}) and minimum strength requirement

According to the seismic evaluation manual of Bangladesh, for the 2nd level and the 3rd level, the seismic demand index (I_{SO}) and the minimum strength requirement of the target building irrespective of the stories are calculated as follows:

$$I_{SO} = 0.8 \times \frac{2}{3} Z.I.C_s \;\; (= \; 0.8 \times \frac{2}{3} \times 0.36 \times 1.0 \times 3.37 \; = 0.65) \tag{3}$$

$$C_{TU}.S_D \geq 0.4.\frac{2}{3} Z.I.C_s \;\; (= 0.4 \times \frac{2}{3} \times 0.36 \times 1.0 \times 3.37 = 0.325) \tag{4}$$

where, Z: Seismic Zone Coefficient, I: Structure Importance Factor, C_s: Normalized Acceleration Response Spectrum which is a function of building natural period and site class.

4.1.2. Seismic evaluation results

The results demonstrated in Table 2 indicate seismic rehabilitation needs to be provided for all stories along the longitudinal direction. The results of seismic evaluation along the transverse direction reflect all the stories except the top story require strengthening. In case of seismic evaluation considering masonry infill for the 4th story seismic index (I_S) value is more than seismic demand index (I_{SO}). Still this story along the transverse direction requires strengthening since $C_{TU}S_D$ value is less than the minimum strength requirement.

Table 2. Seismic evaluation results.

Story	Evaluation along longitudinal direction		Evaluation along Transverse direction	
	$I_s = E_0 * S_D * T$	$C_{TU}\, S_D$	$I_s = E_0 * S_D * T$	$C_{TU}\, S_D$
5	0.51	0.19	1.11	0.38
4	0.34	0.13	0.67	0.20
3	0.21	0.11	0.47	0.47
2	0.11	0.10	0.43	0.43
1	0.10	0.10	0.13	0.13

4.2. Seismic Rehabilitation

Required seismic performance can be achieved through improvement of strength or ductility or both. For retrofit design strength dominant seismic rehabilitation is given a priority. Three retrofitting methods have been applied. Chosen retrofitting methods are steel framed brace insertion, RC column jacketing and aramid fiber sheet wrapping.

4.2.1. Seismic evaluation result after retrofitting

Seismic evaluation after the retrofit is conducted taking into account the strength of masonry infill. Seismic indices (I_s) of the stories of the target building along longitudinal direction and transverse direction before and after the retrofit are depicted in Figure 9 and Figure 10 respectively.

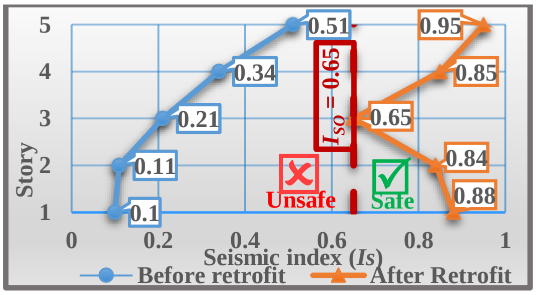

Figure 9. Seismic Index (I_s) along longitudinal direction before and after the strength oriented retrofit.

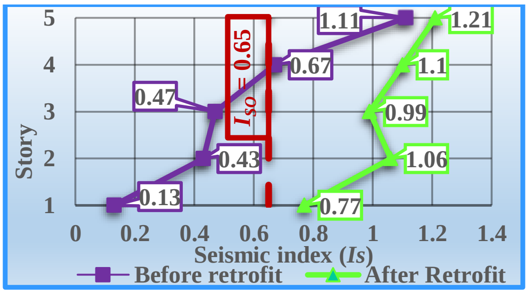

Figure 10. Seismic Index (I_s) along transverse direction before and after the strength oriented retrofit.

4.3. Non-linear analysis of the target building

4.3.1. Performance objectives

According to FEMA 356 and ATC-40, immediate occupancy (IO), life safety (LS), and collapse prevention (CP) are three discrete structural performance levels. Structural performance levels and associated drifts for concrete frames are shown in Table 3.

Table 3. Structural performance levels and associated drift limits according to FEMA 356 and ATC-40.

Type	Immediate Occupancy	Life Safety	Collapse Prevention
Primary	Very limited post-earthquake damage state, building is safe to occupy.	Significant post-earthquake damage state, still structure isn't collapsed.	Verge of partial or total collapse, however, structure can still bear gravity load.
Drift limit	4% transient or permanent	2% transient or 1% permanent	1% transient or negligible permanent

4.2.2. Inter-story drift

When strength oriented retrofit was opted, inter-story drift ratio of all the stories were (shown in Figure 11) below the immediate occupancy level. In the case of ductility dominant retrofit inter-story drift ratio (depicted in Figure 12) exceeded the immediate occupancy level. From Capacity Spectrum Method (CSM), inter-story drift ratios for all the stories for both directions are higher than those from the accurate method time history analysis.

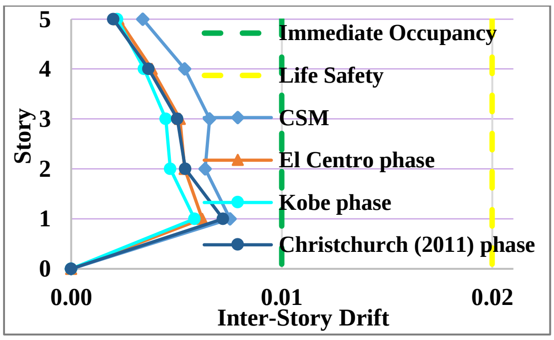

Figure 11. Inter-story drift ratio after the strength oriented retrofit along the longitudinal direction for THA and CSM.

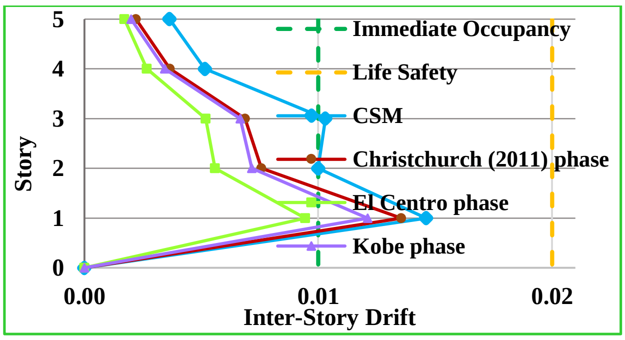

Figure 12. Inter-story drift ratio after the ductility oriented retrofit along the longitudinal direction for THA and CSM.

5. CONCLUSIONS

The target building of this study suffered damage in the ground floor due to the 2017 Tripura earthquake. To get an analytical explanation of the damage, seismic response simulation was performed using synthetic ground motion. It shows reasonable correlation between actual behavior and analytical behavior. Low strength concrete, bare frames in the ground floor, inadequate column sizes, inadequate shear reinforcement were responsible for the damage. Kanno et al. (2006) Attenuation relation proved to a good option to generate artificial ground motion when the earthquake is shallow.

Both strength oriented and ductility oriented retrofit approach were studied to choose the feasible retrofit methods. Strength dominant retrofit was found to be the best option in case of no damage is allowed. However, ductility dominant retrofit can be opted, if some damage is allowed in the structure. RC column jacketing and steel framed braces have reduced large, concentrated deformation (due to soft story) in the ground floor. For inadequate shear reinforcement and weak beam column joints, RC jacketing and aramid sheet wrapping provided effective retrofitting options. FRP wrapping increased the shear strength thus increased the ductility of the brittle columns.

6. RECOMMENDATION

The strength of columns surrounding the masonry infill, and the mortar strength of the masonry, can greatly impact the ductility and strength of the infilled frame. Research works need to be conducted varying these parameters especially to bear in mind the construction conditions of the concerned country.

Weak beam-column joint areas and inadequate or no joint-shear reinforcement inside joints exist in most of the RC framed buildings in Bangladesh. These beam-column joint areas will fail within a small deflection of the concerned columns. Especial attention should be paid when seismic evaluation is conducted if a weak-beam column joint subsists. Further research works can be carried out to find feasible retrofit methods for weak beam-column joint area.

ACKNOWLEDGEMENTS

I would first like to express my deepest gratitude to my supervisor Dr. Eng. Matsutaro Seki and Dr. Toshihide Kashima for their guidance, constant support and instruction during my study.

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