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REVIEW ARTICLE

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SEISMIC RESPONSES OF TWENTY-STOREY REINFORCED CONCRETE FRAME STRUCTURE IN AKHALIA, SYLHET BANGLADESH

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ABSTRACT

In the high earthquake era, the efficient performance evaluation of reinforced concrete building structures has become a key issue. In this paper, basedon the nonlinear dynamic evaluation method of structural seismic performance atwenty-storey reinforced concrete building structure seismic assessment was carry out. the building structure was considered to be built in Akhalia, Sylhet Bangladesh. Four recorded ground motions were matched with Bangladesh National Building Code (BNBC) 2017response spectrums and scaled to PGA of 0.25g,0.30g and 0.35g. Therefore, nonlinear dynamic time history analyses were carry out using the matched and scaled ground motions and the seismic performance of the structure was analyzed. It can be seen that the seismic performance of the selected project of twenty-storey reinforced concrete frame structure is excellent. Thus, the platform based on the concept of seismic evaluation structures demonstrates a general agreement that an efficient and accurate assessment of reinforced concrete frame structures can be achieved.

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INTRODUCTION

Occurrences of recent large earthquakes have demonstrated that many properly designed and constructed structures were no longer functional after the earthquake and were later demolished rather than being repaired (Takagi& Wada, 2018). Reinforced concrete (RC) structural systems subjected to strong earthquakes can undergo sufficiently large deformations so as to cause structural members and connections to respond in the nonlinear range (Wang & Zhao, 2018). An accurate prediction of this behavior is essential to facilitate the development of design guidelines and to ensure satisfactory performance of structures subjected to expected design ground motions level (Villaverde, 2007). While structural damage under an extreme event (equal to or exceeding the design load) is permissible, the degree of damage allowed by design depends on several factors: the type of structure, the "frequency" (or return period) and severity of the loading (CPNI, 2011; Bungale, 2016). Some critical structures may not be allowed to suffer any damage whatsoever in order to ensure its continued functionality after the event. On the other hand, life-safety issues assume greater importance for regular structures (Mimura *et al*., 2011). Bangladeshi tectonic framework shows that Sylhet city is located in the seismic zone three which means Sylhet city is a zone of high seismic risk (Apu& Das, 2020; Raihanul *et al*.,2016). Experience from past earthquake events has demonstrated earthquakes pose a significant risk in the region. This is due to several factors like 1) concentration of populations living or settling in that hazardous areas;2) design methods, detailing, construction, and maintenance issues;3) The historical earthquake record is largely incomplete;4) updating mapping of seismogenically active faults is still an on-going program, among others (Al-Hussaini *et al*.,2015). So there is an urgent need evaluation of existing structures in terms of seismic performance and to continuously upgrade the seismic codes for the design of new structures. Most of the damages caused by earthquakes are mostly a consequence of the collapse and damage of existing structures not the earthquake itself – meaning harm-reduction measures can make an impact. This study aims at filling the lack of research on seismic assessment of reinforced concrete structure in Sylhet city.

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For this purpose, this paper evaluates the performance to inelastic seismic response of twenty-storey RC moment frame in Sylhet city. Therefore, modal and nonlinear dynamic time history analyses are carried out on the twenty-storey RC buildings designed with current Bangladeshi design specifications subjected to design level earthquake ground motions. Finally, Seismic performance evaluation was also carried out through nonlinear dynamic time history analyses.

Structural Performance Levels: Performance-based seismic design (PBSD) approach refers to the methodology as a means to determine the likely performance of an existing building structure during an earthquake. However, this concept is as well applicable to new buildings. Performance objective defines the related desired performance level to a perceived hazard level. Different performance requirements are proposed for various types of structural components. Generally, comprehensive numerical analysis is carried out to evaluate the seismic performance of the target building. For high rise and complex buildings exceeded the usual design experience, full structural testing and modeling is highly recommended to conduct in order to carry out the structural behavior and check the seismic performance directly. Furthermore, If the predefined seismic performance objectives cannot be satisfied, design iteration should be performed till satisfied. Life Safety (LS) represent the minimum Performance Level according to most current seismic design codes for building systems. Therefore, PBSD defined a key parameter as performance objective, which represents the acceptable level of damage selected for a specified seismic level. Various seismic codes have concluded goals for building performance levels during earthquakes in order to confirm life safety and to limit property damage (Fajfar,2018; Mousavi *et al*.,2008). These performance levels represent those codes prediction of both level of damage to a facility and the ability to continue operations. The levels concern both structural and non-structural members. The structure may be designed based on single or multiple performance objectives. The selected performance objectives will depend on the expected use of the structure. The performance objective for a design should be accurately define as a certain level of confidence (i.e. 95%) that the structure will present Collapse Prevention or better performance for seismic hazards. The method usually uses fixed target drift and yield mechanisms as important performance objectives, and these two limit states are dependent on the degree and distribution of structural damages.

Nonlinear Time History Analysis

Description of case-study structures: To carry out the seismic performance evaluation, a three-dimensional structural model is used. The basic characteristics of the model are as follows; the considered structure is a moment resisting frame. Hence, the lateral force resistant elements consist of beams, columns, and joints. Here below on figure 2is presented the building plans, modeled and analyzed with SAP2000 program. The slab was designed with gravity loads only using one-way slab system. It was assumed that the diaphragm is rigid. Frame structure is modeled as 20 stories plane frames with 5 bays and the total building height is 72.9 m. The total length of the building is 30 m long in the north-south (N-S) direction and in the east-west (E-W) direction which divide into 5 typical bays and two core walls. The typical bays are 6 m long in the north-south (N-S) direction and in the east-west (E-W) direction. The selected location of this study was Akhalia, Sylhet Bangladesh with soil site class c classification. The compressive strength of concrete, fc' of 30 MPafor the columns and 25 MPa for the beams, the yield strength of the reinforcement bars, fy is 400 MPa. Modulus of elasticity E = 25GPa, Poisson ratio μ = 0.2. The density of the structures is 24 kN/m^3. The structural analysis was carried out following the Bangladesh National Building Code (BNBC) 2017, using a response modification factor, R, of 8. The frame is subjected to dead, live, and seismic loads. In order to perform the nonlinearity of the structure, the Modified Takeda Trilinear model is employed as the hysteretic model for beams and columns. Figure1presents the stiffness degradation factors valued by

$$\sim_1^{(\pm)} = K_1^{(\pm)} / K_0 = 0.5 \text{ and} \sim_2^{(\pm)} = K_2^{(\pm)} / K_0 = 0.1$$

The Rayleigh's damping is applied and the second-order effect is considered. All elements are assumed to be fully fixed in foundation, the soil foundation interaction and foundation flexibility effects are ignored. Frames and walls are connected together by means of rigid diaphragms at each floor level. The typical height of all floors, cross sections of columns and beams, thickness of slabs and core walls are listed as follows: the height of the first storey is 4.5 m, and that of the rest, respectively, is 3.6m; the section size of the column from the first to the sixth storey is 0.80X0.80m; from the seventh to the fifteen storey is 0.75X0.75m; and that from the sixteen to the twenty storey is 0.65X0.65m.The beams are 0.35X0.65m, the roof slab is 0.15m thickness and 0.2m for the remaining floors. The core walls are of 0.4m thickness. It was assumed that the diaphragm is rigid.

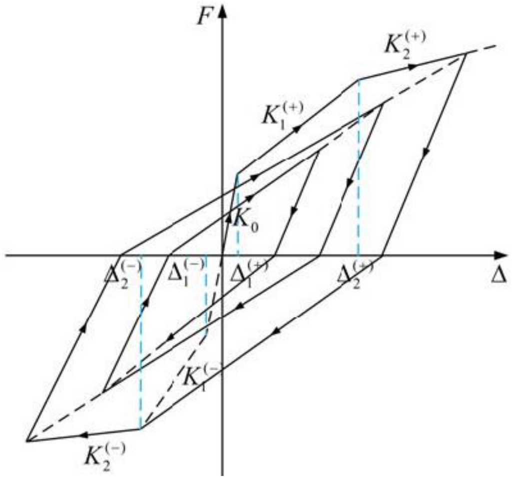

Figure 1. The restoring force model (modified Takeda trilinear model)

Ground Motion Selection

The codes for seismic design of building stand in needs of dynamic time-history analysis to be performed for high-rise buildings and other important structures seismic design. The ground motion selection represents a fundamental point in defining the seismic load input during the analysis in structure design, the selection of variable ground motion records also represents a critical aspect.

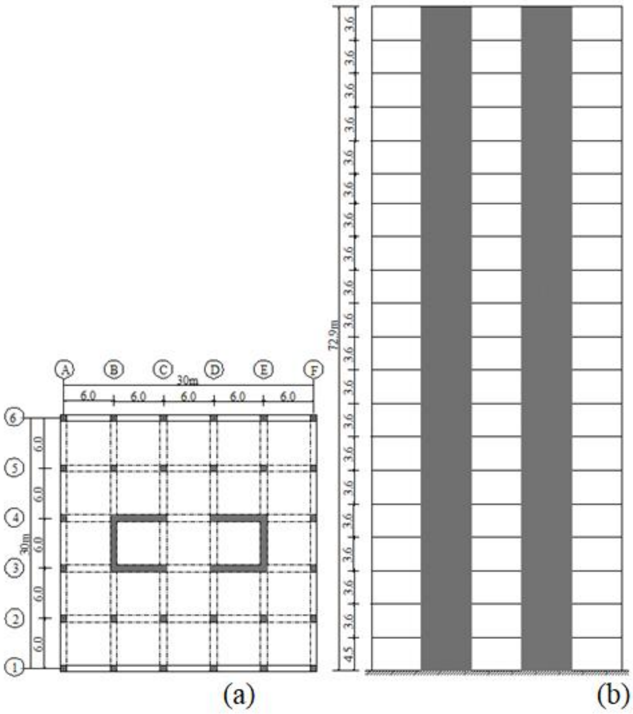

Figure 2. Building plans: (a) plan view ;(b) elevation view

The recorded or simulated ground motions are generally required to match a target mean response spectrum, moreover the recorded ground motions can provide more seismic information as compared with the simulated ground motions. Therefore, it is necessary to select ground motion records matching target response spectrum for seismic assessment of structure. The target response spectrum along with code spectrum, uniform hazard spectrum and condition mean spectrum, symbolizes the demand of structure design. For a given specific building site, four ground motions were selected from the PEER database. The selected ground motions shear velocities range from 200 to 300m/s at the recording station to make consistent for designed model frames bases, equivalent ground types, soil c in the Bangladesh National Building Code (BNBC) 2017.

The selected recorded ground motions have also horizontal acceleration values in the range of 0.28 g to 0.61g (Table 1). Matching and scaling mechanism was carried out to match and scale the recorded ground motions with elastic response spectrums of PGA =0.25g,0.30g and 0.35g with a 5% viscous damping ratio. Table1shows the four selected ground motions. Figure 3shows the ground motion spectrums matched with the code spectrum for moment resisting frame structures. The matched and scaled accelerograms are then applied to carry out the structural seismic performance evaluation

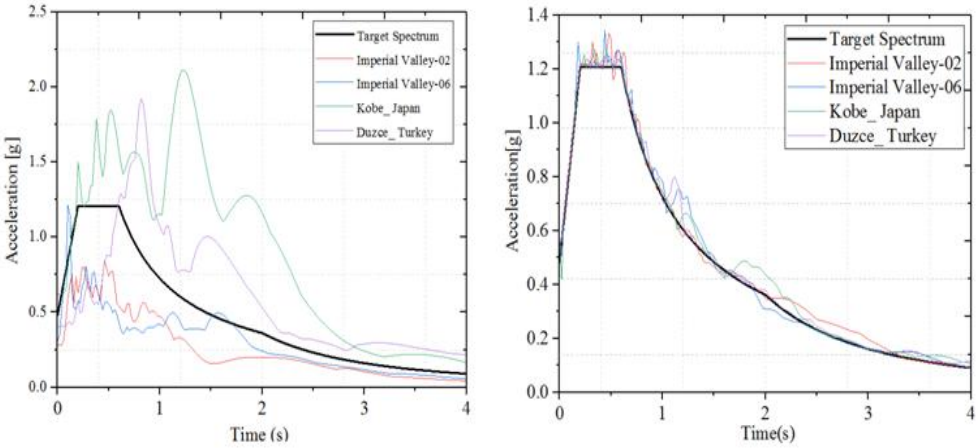

Figure 3. Design response spectrum (BNBC-2017) and matched records

Table 1. Unscaled ground motions

No	Earthquake name	Year	Station name	Magnitude	PGA(g)
1	Imperial Valley-02[RSN6]	1940	El Centro Array #9	6.95	0.28
2	Imperial Valley-06[RSN158]	1979	Aeropuerto Mexicali	6.53	0.30
3	Kobe_ Japan[RSN1120]	1995	Takatori	6.9	0.61
4	Duzce_Turkey[RSN 1605]	1999	Duzce	7.14	0.40

RESULTS AND DISCUSSION

Free Vibration Analysis Results: Modal analysis was performed for the ground excitation in horizontal directions. Figure 4 and Table 2denote the modal analysis results.

Figure 4illustrate the modal shapes of the first twelve modes from which different translation and torsion order can be observed. As can be seen the structure moves toward left due to lateral loading and forward significantly. The twelve modes frequencies of structures are shown in Table 2.

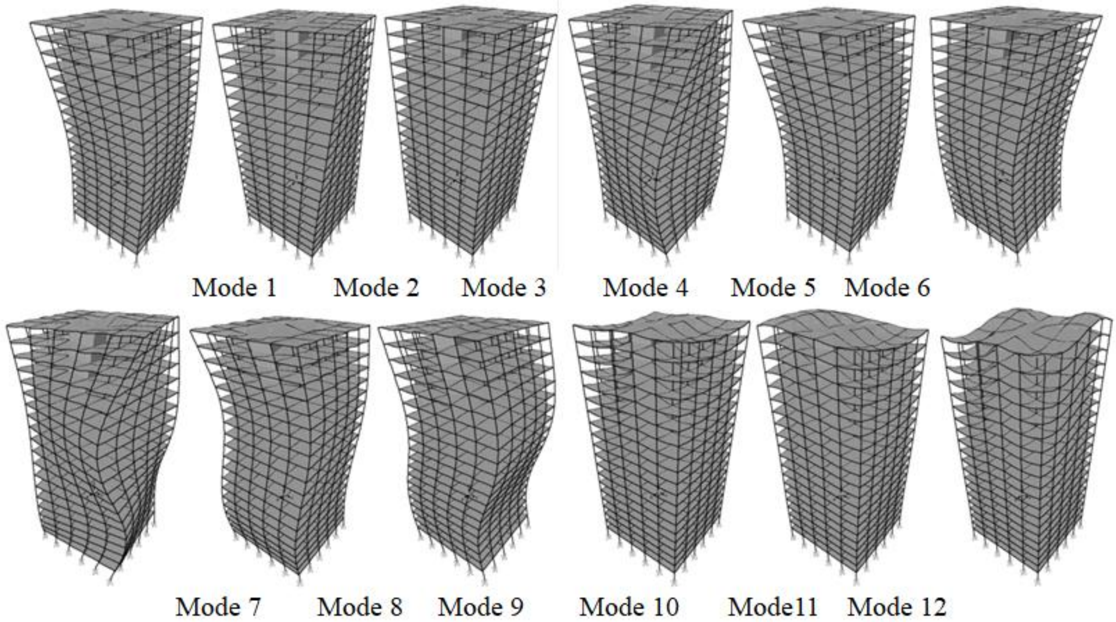

Figure 4. Mode shapes

Table 2. Natural frequency of the building

Mode	Frequency [Hz]
Mode 1	0.39449
Mode 2	0.41266
Mode 3	0.44022
Mode 4	1.63703
Mode 5	1.65802
Mode 6	1.84385
Mode 7	3.54614
Mode 8	3.94729
Mode 9	4.11541
Mode 10	4.65599
Mode 11	4.74912
Mode 12	4.99889

Nonlinear Time History Analysis Responses: Nonlinear time history analysis has been carried out in multiple degrees of freedom and response of building in longitudinal and transverse directions are obtained. Analysis responses such as the storey displacement, storey drift, storey acceleration, storey axial force, storey shear and moment have been obtained. The complete set of maximum responses of the structure are presented in Table 3.

Figure 5,6 and 7 show the maximum displacement, inter-storey drift and acceleration responses, respectively. By comparison displacement response from figure5, it can be observed that maximum displacements at top floor from RSN1120 and RSN158 are having greater values at negative and positive axis, respectively. Peak displacement of -0.279 m and 0.211m are obtained under 0.25g scaled ground

Table 3. Maximum responses of the structure

Ground motions excitation					
Seismic Responses		RSN6 0.25g 0.3g 0.35g	RSN158 0.25g 0.3g 0.35g	RSN1120 0.25g 0.3g 0.35g	RSN1605 0.25g 0.3g 0.35g
Displacement(m)		0.177 0.2080.236 -0.238-0.262 -0.288	0.2110.2550.30 -0.162-0.166 -0.174	0.1790.200.226 -0.279 -0.318 -0.342	0.1980.2420.291 -0.185-0.208 -0.244
Inter-storey drift(%)		0.0028 0.0033 0.0037 -0.0038-0.0042 -0.0046	0.0034 0.0041 0.0047 -0.0026 -0.0026 -0.0027	0.003 0.0033 0.0036 -0.0044 -0.005 -0.0053	0.0033 0.004 0.00468 -0.0029-0.0033 -0.0038
Acceleration(m/s^2)		4.02 4.825.61	3.19 3.73 4.35	4.11 4.81 5.48	3.72 4.31 4.97
Axial force(KN)		3165.592899.52621.6	2931.52668.7 2449.74	2748.97 2621.6 2537.8	2971.172712.5 2478.3
Storey shear(KN)		90.61139.01180.58	85.63 144.48161.84	61.34 84.38152.39	136.69179.12213.40
Moment(KN.m)		460.64660.5838.35	478.95 627.81763.31	328.75 435.63707.95	638.39814.89954.74

motions. For the case of 0.3g scaled ground motions, peak displacement of -0.318m and 0.255m are observed. Regarding case of 0.35g scaled ground motions, it can be seen that the peak displacements are about -0.342m and 0.30m.Compare these three analyses show a significantly increase in displacement magnitude. Percentage increase of 22.58%[0.25g] and 7.54%[0.3g] for RSN1120 responses can be observed compare with the 0.35g ground motions excitation response, while 42.18%[0.25g] and 17.64%[0.3g] are obtained for RSN158. Inter-storey drift response of this structure was evaluated. The analysis of the results on the variation of the inter-storey drift ratio (IDR) under 4 ground accelerations in three groups of scaling are given in Figure 6. It was obtained that the maximum IDR were resulted from RSN1120 and RSN158 which clearly follow the displacement response law. Peak values corresponding to -0.0044,0.0034,-0.005,0.0041,-0.0053 and 0.0047 are obtained from 0.25g,0.3g and 0.35g scaled ground motions, respectively.

Also by comparison between ground motions scaled cases, it can be observed that percentage increase of 20.45%[0.25g] and 5.99%[0.3g] for RSN1120 when 0.35g is considered, 38.23%[0.25g] and 14.63%[0.3g] can be obtained for RSN158.Considering the IDR limit of 0.010h□□ of the Bangladesh National Building Code (BNBC) 2017, where h□□ represents the story height below Level x. Thus, the inter-story drift responses mentioned above were observed to be less than the corresponding value. Figure 7. shows the acceleration response under the effect of different scaled ground motions. As it can be seen under the effect of 0.25g and 0.3g scaled ground motions RSN1120 shows greater responses, while RSN6 is having highest acceleration response under 0.35g scaled ground motion excitation as indicated in Figure 7.Peakaccelerations of 4.117m/s^2 ,4.58m/s^2 and 5.61$m/s^2$are observed from, 0.25g, 0.3g and 0.35g scaled ground motion excitations, respectively. One can see that from these comparison, the peak accelerations are observed in the top floor. However, maximum collapse acceleration for the structure is assumed to be occurred at that point of structure. Compare the peak global responses, corresponding percentage increases of 36.26%[0.25g] and 22.48%[0.3g] are observed when 0.3g scaled ground motion action is applied.

Figure 8 collects the set of earthquake-induced compressive loads experienced by the columns. As illustrated in the figure 8, the structures subjected to RSN6 ground motion are shown to have greater axial force. A large redistribution of forces in the columns took place (Figure 8); the corresponding peak forces in the columns under RSN6 excitations are of 3165.59 KN, 2899.46KN and 2621.54KN for 0.25g,0.3g and 0.35g scaled ground motion excitation. In detail, an approximately 17.18% [0.25g] and 9.58% [0.3g] decrease is computed at the base of the structures when 0.35g ground motion action is considered; as the height of the structure increases, this effect tends to decrease, showing a roughly linear piecewise decaying slope with the structure height. Also by comparison the reimaging excitations, same law with RSN6 can be observed. As shown in figure 9, the evaluation of reinforced concrete frame structure storey shear force response based on different scaled ground motion excitations is compared. A great redistribution of shear forces took place; the shear force significantly increased in floor columns. Compare the different ground motion excitation scenarios, RSN1605 shows highest shear forces in both scaled ground excitations. Eventually, the peak responses rest at136.69KN,179.12KN and 213.406KN for 0.25g, 0.3g and 0.35g scaled ground motions, respectively. The shear force response percentage increase finally rested at 56.12%[0.25g] and 19.14%[0.3g] which confirm a significantly increase in both scenarios. Furthermore, by comparison the responses between the scaled ground motion excitations, it can be observed that the greater scaled ground motion the closer responses from different ground motion.

A similar consideration can be drawn in terms of moment peaks for both scaled ground motions (see Figure. 10). Highest responses are obtained from RSN1605 excitations. As can be observed from figure 10, moment responses eventually rested at 638. 39KN.m,814.89KN.m and 954.74KN.m under 0.25g, 0.3g and 0.35g scaled ground motion actions, respectively.

Thus, the peak moment responses show considerable percentage increase of 49.55%[0.25g] and 17.16%[0.3g] when subjected to 0.35g scaled ground motion excitation. It can be seen from the global performance of the structure that the maximum value (bolded values, see table 3) of each specific response does not necessarily occurs with one single earthquake. Moreover, the ground motions with maximum scaled PGA value do not necessarily result in the maximum responses in the structure.

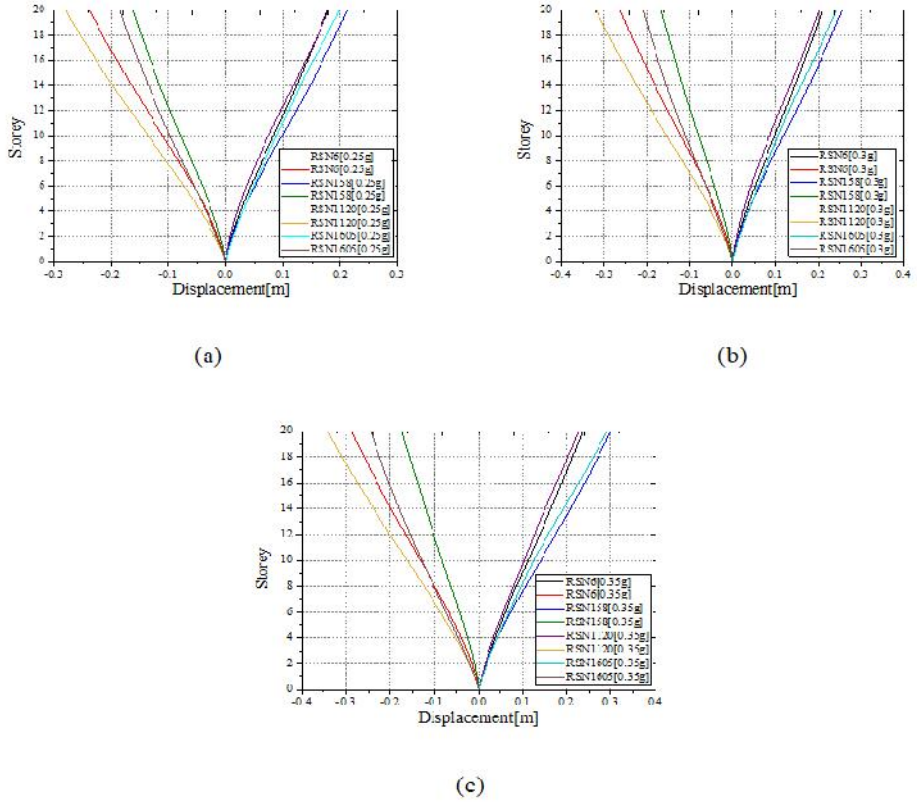

Figure 5. Maximum displacements at different floors: (a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

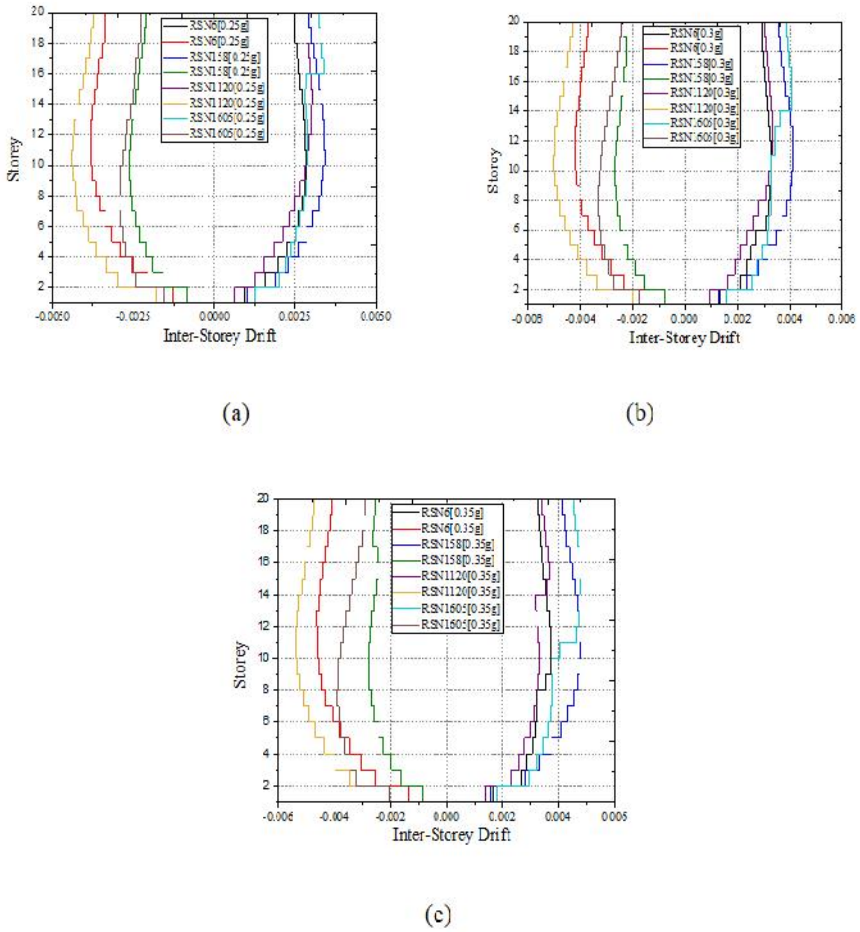

Figure 6. Maximum storey drift at different floors:(a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

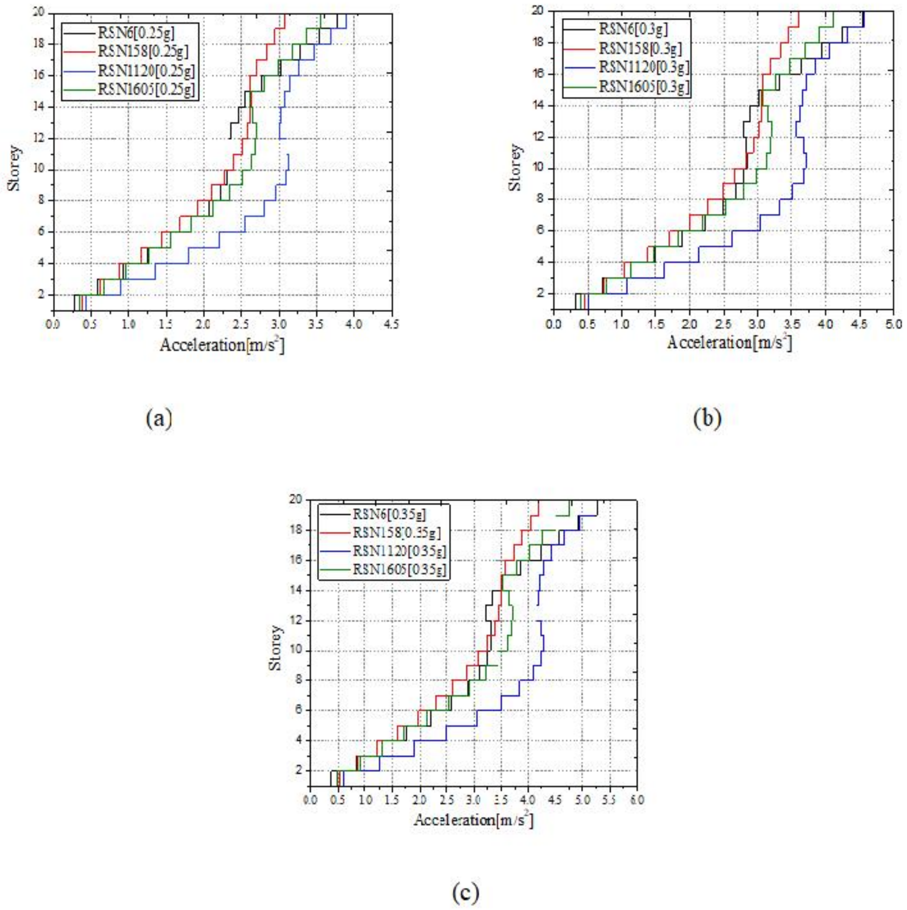

Figure 7. Maximum acceleration at different floors:(a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

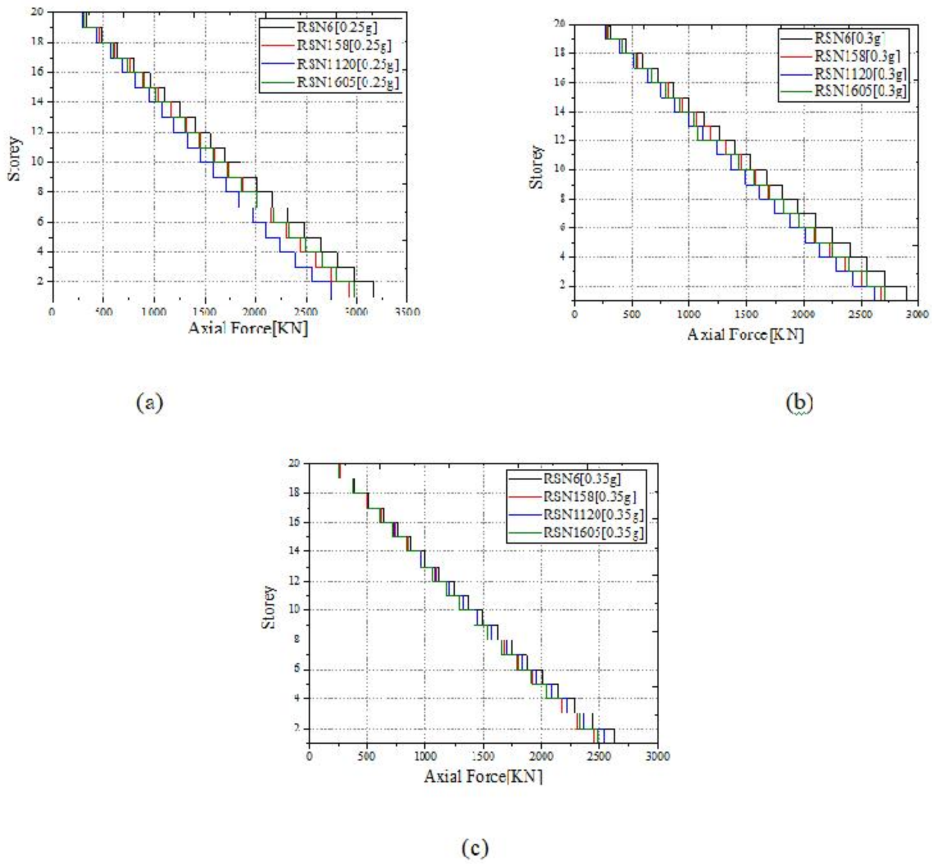

Figure 8. Maximum axial force at different floors:(a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

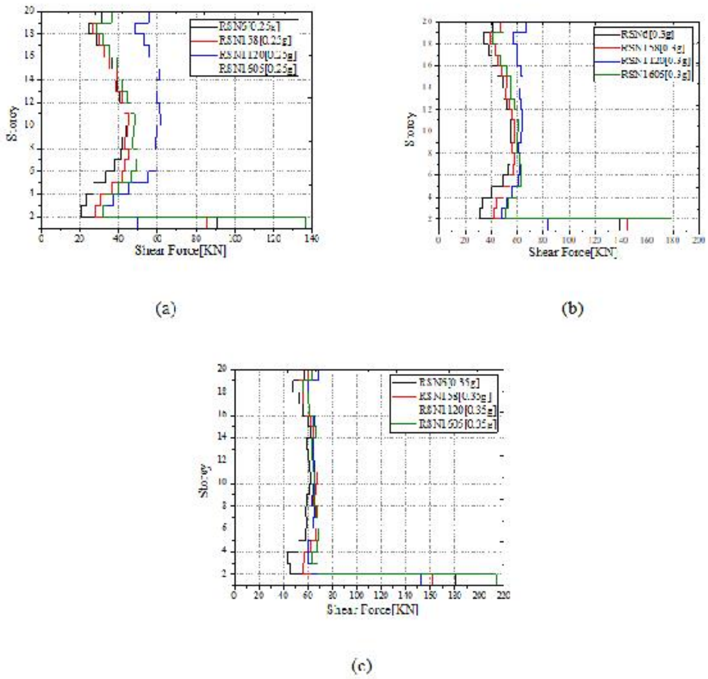

Figure 9. Maximum shear force at different floors:(a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

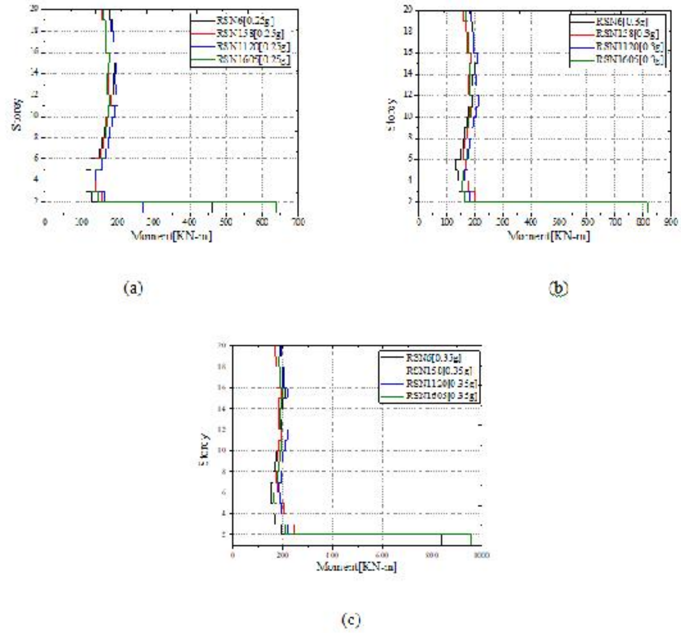

Figure 10. Maximum moment at different floors:(a):0.25g excitation;(b):0.3g excitation; (c):0.35g excitation

Conclusion

this study represents a performance investigation for the seismic behavior of a twenty-storey RC moment frame in Sylhet city. The complex seismic response of the structure is estimated by representing it by a 3D FE model. Modal and nonlinear time-history analysis analyses methods are used to analyze the structure based on different scaled ground motion excitations and compare the results. It is observed that the structure shows good behavior in the global performance and distribution of maximum responses comes from different ground motion action not necessarily with maximum scaled earthquake.

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