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Non-linear time domain analysis of base isolated multi-storey building under site specific bi-directional seismic loading

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ABSTRACT

The incorporation of base isolation in building construction in the region of medium risk seismicity is now an important issue. Thorough investigation is needed for buildings located in those regions, to be incorporated with base isolator and then dynamic analysis to carry over. This research provides incorporation of Lead Rubber Bearing and High Damping Rubber Bearing as base isolators in addition to focussing on the changes of structural parameters for isolating effects in those vicinities. Nonlinear models of Lead Rubber Bearing and High Damping Rubber Bearing have been built up. The design of base isolators for building construction is covered along with structural feasibility. Linear static, free vibration and nonlinear dynamic time domain analyses are performed for both isolated and non-isolated buildings under site specific bi-directional earthquake. The automated Newmark-beta time integration approach has been adopted for solution in time domain. The nonlinearities, arising due to base isolated bearings and seismic forces are duly considered. The study reveals that for medium rise building construction, isolation can significantly reduce seismic response in soft to medium stiff soil. The reduction of overturning base moment due to isolation indicates that the building becomes more stable compared to the fixed base structure. Modelled non-linear bearings have been found to be suitable to cope with the precise nonlinearities. The building experiences more flexibility even when using the same structural element configuration. In addition, the flexibility of the structure envisages some sort of savings due to reduced structural responses through incorporation of the isolator. In seismic vulnerable areas where the main concern is the mitigation of the seismic instability with the support of critical components, the study shows the effectiveness of the base isolation system in terms of lessening structural responses under seismic loading.

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1. Introduction

Seismic isolation is the separation of the structure from its base to negotiate the destructive movement of the ground by providing flexibility and energy dissipation capability through the insertion of isolators between the foundation and the building structure [1]. Unlike the conventional design approach, which is based on an increased resistance (strengthening) of the structures, the seismic isolation concept is aimed at a significant reduction of dynamic loads induced by the earthquake at the base of the structures themselves [2]. Invention of lead rubber bearing (1970's) and high damping rubber (early 1980's) sets forth a new dimension for the design of base isolated structure [3–8]. A significant amount of both past and recent research in the area of base isolation has focused on the use of elastomeric bearings, such as high damping rubber bearing and lead rubber bearings [9–12]. Jangid [13] and Providakis [14] investigated seismic responses of multi-storey buildings for near fault motion isolated by LRB. Dall'Asta and Ragni [15,16] have covered experimental tests, analytical model and nonlinear dynamic behaviour of HDRB. Bhuyan [17] has developed a rheological model of high damping rubber bearing for seismic analysis identifying nonlinear viscosity. Analysis procedures to investigate the dynamic structural behaviour with the isolation have also been discussed [18,19]. Although it is a relatively recent technology, seismic isolation for multi-storey buildings has also been well evaluated and reviewed [20–35]. Base isolator with hardening behaviour under increasing load has been developed for medium-rise buildings (up to four storey) and sites with moderate earthquake risk [36]. Nonlinear seismic response evaluation was performed by Balkaya and Kalkan [37]. Resonant behaviour of base-isolated high-rise buildings under long-period ground motions was studied by Ariga et al. [38] and long period building responses by Olsen et al. [39]. Ebisawa et al. [40], Dicleli and Buddaram [41], Casciati and Hamdaoui [42], Di Egidio and Contento [43] have also given effort in the progress of isolated system. Komodromos et al. [27], Sharma and Janggid [44], and Kilar and Koren [45] focused on the seismic behaviour and responses through dynamic analyses of

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isolated buildings. Wilkinson and Hiley [46] have presented a non-linear response history model for the seismic analysis of high-rise framed buildings.

Though the implementation of isolator is going to be very familiar all over the world, there is a lack of proper research to implement the device practically for local buildings in a medium risk seismic region like Dhaka, Bangladesh as per the local requirements. So, a thorough study regarding the feasibility of implementing isolator in this locality is a must to go task. Besides, bi-directional earthquake has rarely been considered as an issue. The time domain method is relatively more time consuming, lengthy and costly. The frequency domain method, on the other hand, is relatively more rapid, concise, and economical. However, the time domain method has been employed for considering non-linearity present in the structural systems.

Time domain analyses, following bi-directional earthquake history, have been carried out. Site specific seismic record in two orthogonal directions has been selected to evaluate the bi-directional behaviour. Combined model of High Damping Rubber bearing (HDRB) and Lead Rubber Bearing (LRB) has been adopted here to explore the feasibility. Preliminary exploration for determining the suitability of isolator incorporation has been done with equivalent static analysis. Then dynamic analysis has been performed to satisfy the structural limitation executing different comparative contributions. The analysis and design of the isolators for a 10-storey sample residential building in Dhaka using SAP 2000 [47] were performed. Design parameters of the isolator for this building and several buildings varying number of storey have been evaluated. Static analysis and free vibration analysis were also performed along with dynamic analyses. Finally, the acceleration excitation behaviours for fixed and isolated buildings were discussed with the displacement patterns at different levels as well. Base shear and overturning moments were also compared to and for certain cases. Every comparison was enforced mentioning the maximum and minimum values on structural excitation.

2. Mathematical formulation

An ideal model for a multi storey building is shown in Fig. 1. The building has been considered as moment resisting concrete frame structure. The superstructure is configured as a linear elastic system. The base and floor between which the isolator is placed are assumed to be infinitely rigid. The nonlinearities arising due to base isolator bearings and seismic forces are properly considered. The superstructure and base are modelled by a consistent mass approach having six degrees of freedom at each node. The base isolator carries the vertical load without undergoing vertical deformation. Lead rubber bearing (LRB) and High damping rubber bearing (HDRB) are used as isolation devices. The structural system is excited in x and y directions by two components of earthquake ground motions at East–west and North–south directions respectively. Nonlinear dynamic analysis has been carried out for both FB and BI structure using SAP2000 [48].

2.1. Design of isolators

The HDRB and LRB isolators are designed as per the procedure mentioned by Kelly [49], Kelly et al. [50], and Naeim [51]. In this research, a computer code ISODNG09, has been generated to iteratively design both the isolators. The total seismic weight, dimensions, layer thickness and number of layers of bearings are considered as initial input. The isolator parameters such as post elastic stiffness, high initial stiffness, yield strength, post yield stiffness ratio and effective damping are computed using the above code. These parameters are then defined in SAP2000. The bearings are linked at the bottom of each column. The detailed sequential procedure for the design of both isolators is shown in a flow chart (Fig. 2). The higher shear strain limit for HDRB results in smaller plan size compared to LRB. Due to the large vertical stiffness of HDRB it can carry heavy loads from the structure [6]. For the present study, interior columns are isolated using HDRB and exterior columns are supported by LRB. Dynamic analysis of the three dimensional building has been carried out considering the associated nonlinearities.

2.2. LRB isolator

LRB is formed by force-fitting the lead plug into a preformed hole in the low damping elastomeric bearing as shown in Fig. 3. The steel plates force the lead plug to deform in shear. Performance of LRB is maintained during repeated strong earthquakes with proper durability and reliability. LRB produces the required amount of damping and has higher initial stiffness. The behaviour of LRB is influenced by yield capacity of the lead core, horizontal stiffness of the lead core and horizontal stiffness of the elastomer. The non-linear behaviour of the isolator is modelled considering the approach suggested by Nagarajaiah et al. [52]. The hysteresis loop for LRB is shown in Fig. 5a.

2.3. HDRB isolator

The HDRB isolator consists of thin layers of high damping rubber and steel plates built in alternate layers as shown in Fig. 4. Horizontal stiffness of bearing is controlled by low shear modulus of elastomer while steel plates provide high vertical stiffness as well as prevents

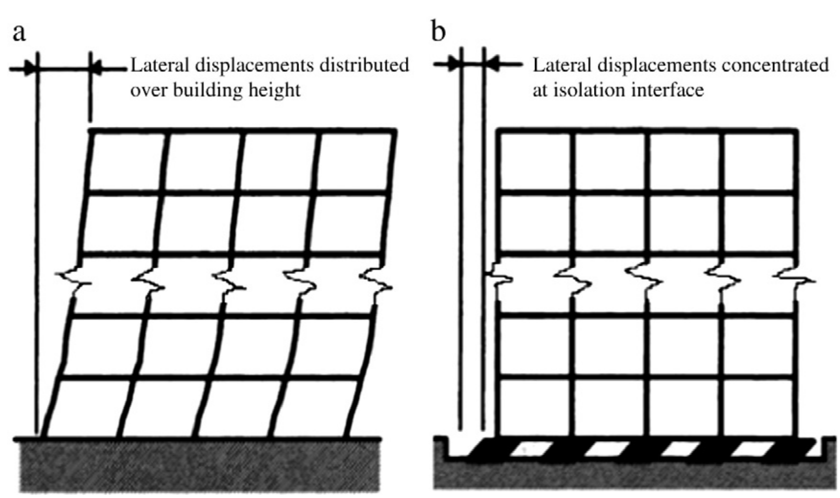

Fig. 1. Structural model of multistory building: a) FB and b) BI.

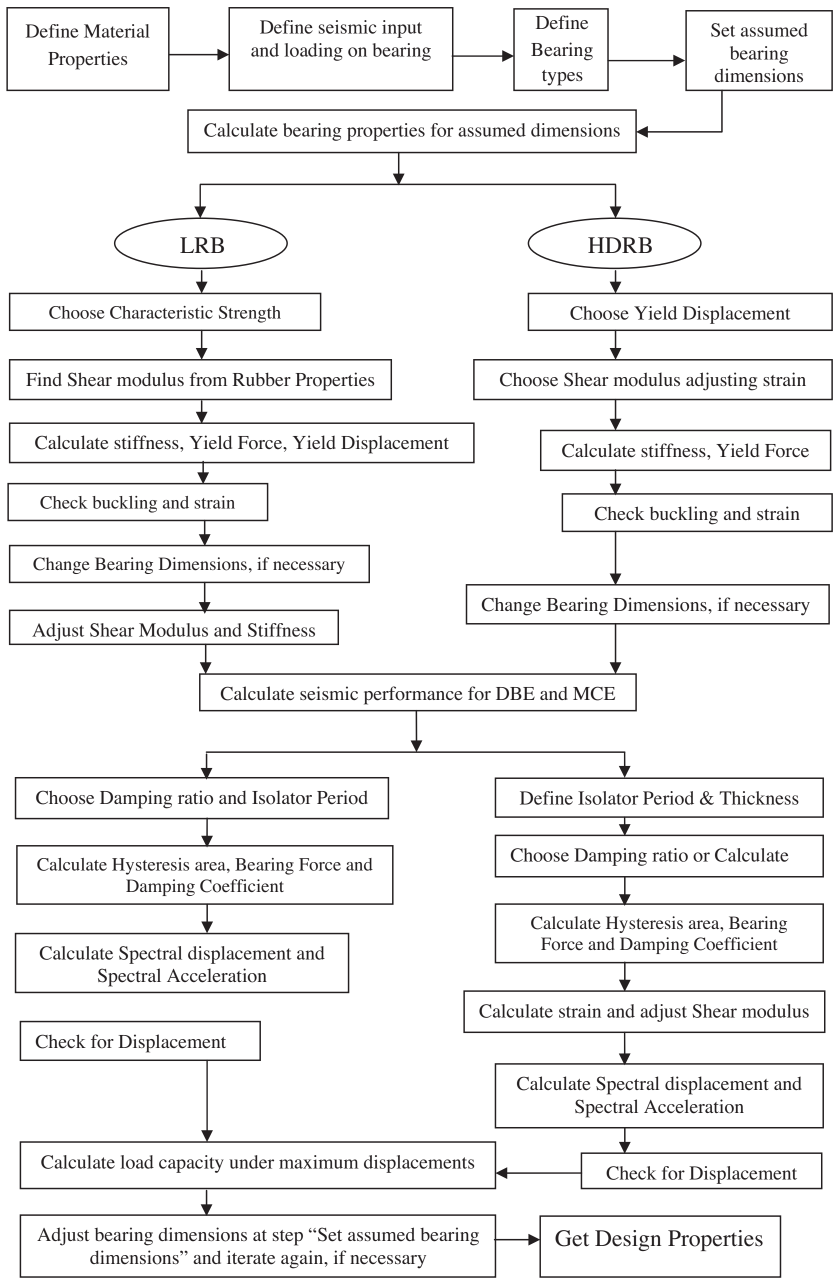

Fig. 2. Design flow chart of isolator properties.

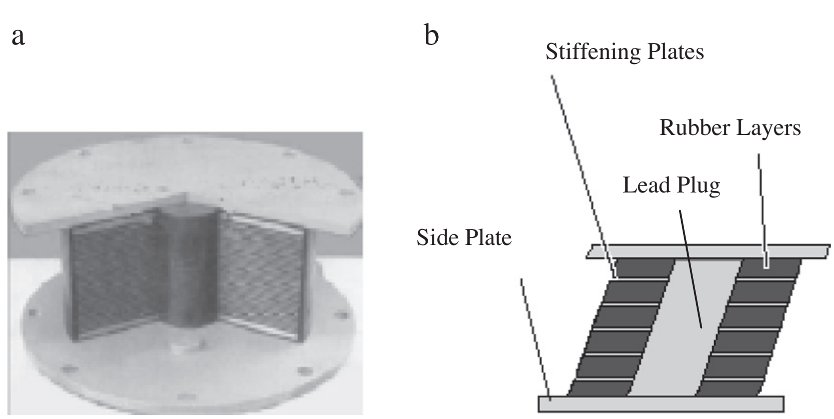

Fig. 3. Lead rubber bearing a) geometry and b) deformation due to loading.

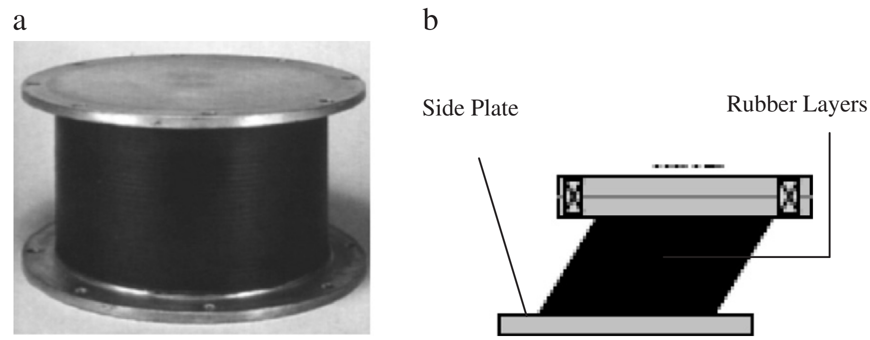

Fig. 4. High damping rubber bearing: a) geometry and b) deformation due to loading.

bulging of rubber. High vertical stiffness of the bearing has no effect on the horizontal stiffness. The damping in the bearing is increased by adding extra-fine carbon block, oils or resins and other suitable fillers. The dominant feature of HDRB system is the parallel action of spring and viscous damping. The damping in the isolator is neither viscous nor hysteretic, but somewhat in between. HDRB isolator has lower horizontal stiffness which causes higher natural period of the structure. The stiffness and energy dissipation characteristics for HDRB are highly nonlinear and depend on shear strain as shown (Fig. 5b). The force-deformation behaviour of the HDRB isolator is considered as nonlinear force displacement hysteresis. The hysteresis loop area is obtained from the shear strain corresponding to shear modulus and damping.

2.4. Static analysis

The isolators are designed considering earthquake and wind loads to be static. The procedure mentioned in Bangladesh standard BNBC, 1993 [53] to compute static loads due to earthquake and wind is considered. Lateral loads for the building located in Dhaka, Bangladesh are determined considering Z (seismic zone factor), R (response modification factor), C (coefficient for soil profile) and I (importance factor). The base shear due to earthquake and wind can be calculated using Eqs. (1) and (3) respectively.

$$\text{Base shear} = \frac{ZIC}{R}W \tag{1}$$

$$C = \frac{1.25S}{T^{2/3}} \tag{2}$$

where, S = soil structure interaction, T = time period of structure and W = effective weight of structure

$$q_z = C_C C_I C_Z {V_b}^2 \tag{3}$$

where, q_z = sustained wind pressure at height 'z' kN/m^2, C_C = velocity to pressure conversion = 47.2×10^{-6}, C_I = structure importance coefficient, C_Z = combined height and exposure coefficient, V_b = basic wind speed at km/h,

$$P_z = C_G C_p q_z \tag{4}$$

P_z = design wind pressure at height 'z' kN/m^2, C_G = gust coefficient and C_p = pressure coefficient.

2.5. Nonlinear dynamic analysis

Non-linear dynamic analysis has been carried out considering a typical bi-directional ground motion recorded at Dhaka, Bangladesh. Finite element analysis package SAP2000 is found to be more appropriate for the current study. The governing equations of motion are obtained considering equilibrium of all forces at each degree of freedom. The equations of motion for super structure and isolated base are written in Eq. (5).

$$[M]\{\dot{y} + \ddot{y}_b\} + [C]\{\dot{y}\} + [K]\{y\} = -[M]\left[T_g\right]\left\{\ddot{y}_g\right\} \tag{5}$$

where, $[M]$, $[K]$ and $[C]$ are the mass, damping and stiffness matrices of the superstructure respectively; $\{y\}$ is displacement of super structure; $\{y_b\}$ and $\{\ddot{y}_g\}$ are base displacement and acceleration relative to the ground; $[T_g]$ is the earthquake influence coefficient matrix.

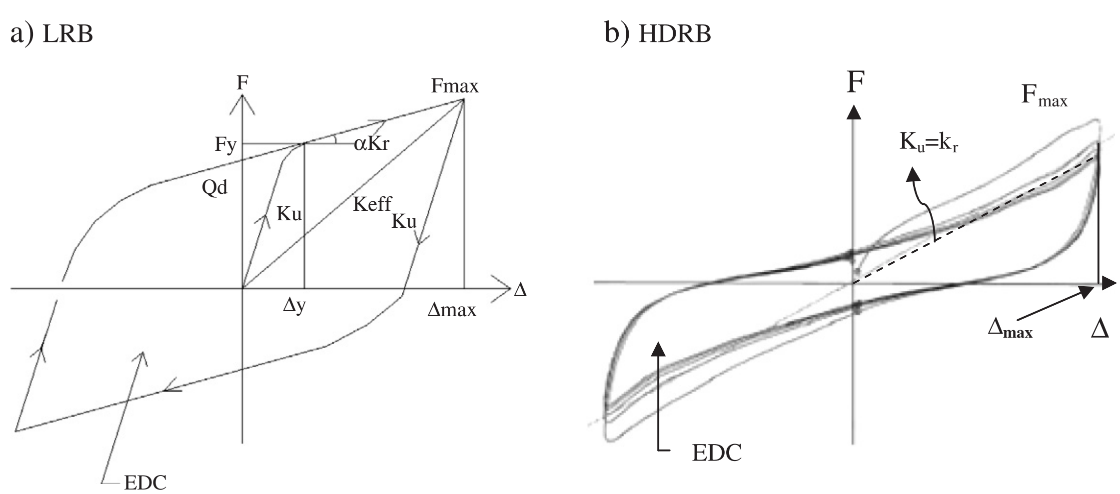

Fig. 5. Idealized non-linear force-displacement curve of bearing.

All nonlinearities are restricted to the base isolator elements only. The above dynamic equilibrium equations considering the super structure as elastic and base isolator as nonlinear can be written as:

$$M\ddot{y}(t)+C\dot{y}(t)+K_Ly(t)+r_N(t)=r(t) \tag{6}$$

where K_L is the stiffness matrix for the linear elastic super structure; C is the proportional damping matrix; M is the diagonal mass matrix; r_N is the vector of forces from nonlinear degrees of freedom in the isolator elements; y, $\dot{y}$, and $\ddot{y}$ are the relative displacement, velocity and acceleration with respect to ground; r is the vector of applied loads.

The effective stiffness at nonlinear degrees of freedom is arbitrary, but varies between zero and the maximum stiffness of that degree of freedom. The equilibrium equation can be rewritten as

$$M\ddot{y}(t)+C\dot{y}(t)+K_Ly(t)+r_N(t)=r(t)-[r_N(t)-K_Ny(t)] \tag{7}$$

where $K=K_L+K_N$; K_L = stiffness matrix of all linear elements, K_N = stiffness matrix for all of the nonlinear degrees of freedom.

2.6. Solution technique

Fast nonlinear analysis (FNA) technique suggested by Wilson [54] has been considered for solution of equilibrium equations. The method is extremely efficient as it is designed for structural systems which are primarily linear-elastic, but have limited number of predefined nonlinear elements. For the FNA method, all nonlinearities are restricted to the isolator elements. The site specific time history load is applied quasi-statically with high damping. The FNA considers a ramp type of time history function which increases linearly from zero to one over a length of time. The nonlinear modal equations are solved iteratively in each time step. The program considers that the analysis results vary during a time step. The iterations are carried out until the solution converges. If convergence cannot be achieved, the program divides the time step into smaller sub steps and tries again.

3. Numerical study

A 10 storey building situated at Dhaka, Bangladesh is considered for the present study. The building is in seismic zone 2 and soft to medium stiff soil as per Bangladesh national building code (BNBC 1993). It is a moment resisting framed structure consisting of 4 bays having spacing 7.62 m c/c in both directions as represented in Fig. 6. The material and geometric properties considered are f'c = 28 MPa, fy = 414 MPa, dead load (excluding self-weight) = 4.8 kPa, live load = 2.4 kPa, slab thickness = 150 mm, all exterior corner columns 750 mm × 750 mm, all exterior middle columns 950 mm × 950 mm, all interior columns 1000 mm × 1000 mm. LRB and HDRB have been assigned for exterior and interior columns respectively. Shear modulus, ultimate elongation, elastic modulus and material constant of rubber have been selected as 400 kPa, 650%, 1350 kPa and 0.87 respectively. 40 mm thick steel plates are attached at two sides of each bearing. Non-linear unloading stiffness is 9.39 kN/mm for LRB and 28.87 kN/mm for HDRB. Whereas the corresponding Yield strengths are 155.93 kN and 586.69 kN.

The vertical loads from static analysis are used to design the base isolators (Table 1). The earthquake loads on the bearing obtained from the dynamic analysis of base isolated (BI) building are used to check the roll-out condition of base isolators. The designed diameter of LRB and HDRB is 800 and 950 mm respectively. Their cross section is given in Fig. 7. The HDRB isolator is defined by plan size and rubber layer configuration. In addition to plan size and rubber later the LRB isolator has lead core.

For static analysis of the fixed based (FB) building seismic loads are calculated as per BNBC 1993. For isolated building response modification factor and importance coefficient is R_I = 2.0 and I = 1.0 respectively [49]. Table 1 shows results of static analysis. Since there is a lack of earthquake record of Dhaka, in the present study the Natore earthquake record is used to generate acceleration time history for the Dhaka earthquake [7,55]. The particulars for the Natore earthquake are Station ID: ALTUS S/N 2928, Channel 1, 6th Jan 2009 16:04:03 (GMT), Magnitude 4.0. The acceleration time history for the Dhaka earthquake has been illustrated in Fig. 8. Non-linear dynamic time domain analysis is performed using time history of the Dhaka EQ for both x and y directions. The local building construction codes BNBC used in this research are comparable and up to mark with the national as well as international standards. Each building model

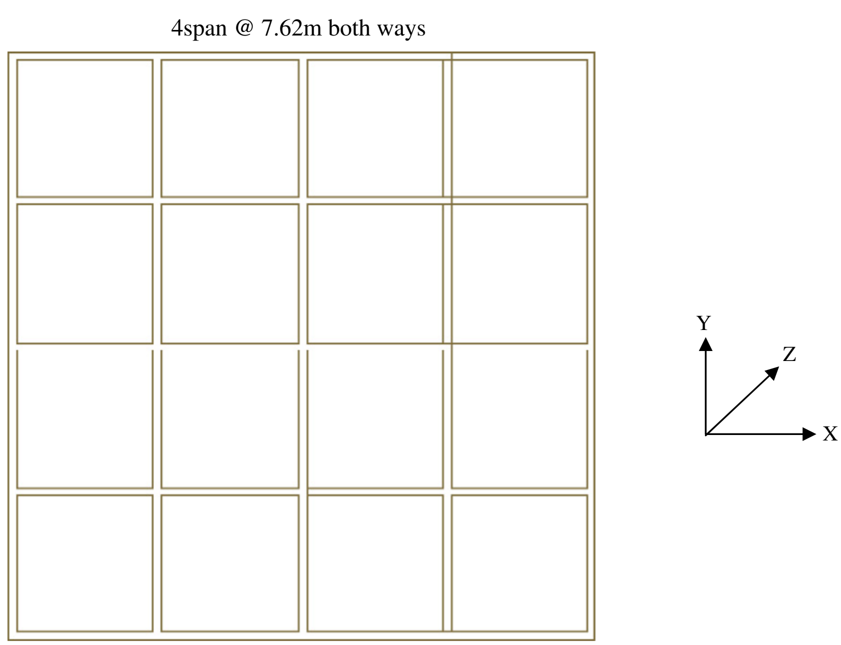

Fig. 6. Plan view of the multi storied building (FB or BI).

Table 1
Static analysis results (FB building).

Parameter	Value
Design base shear (EQ load)	3936 kN
Design base shear (Wind load)	2829 kN
Design base moment (EQ load)	89,523 kN-m
Design base shear (wind load)	48,547 kN-m
Maximum top story displacement (EQ load)	58.9 mm
Maximum Top story Displacement (Wind load)	35.6 mm
Base displacement (EQ and wind load)	0
Total weight of building	127,754 kN
Governing axial load on interior column	7215 kN
Governing axial load on exterior column	4546 kN

and damping system configuration is analysed for 30 s durational earthquake. The Newmark-beta time integration approach has been adopted for solution in time domain. The time step used for numerical integration is 0.005 s.

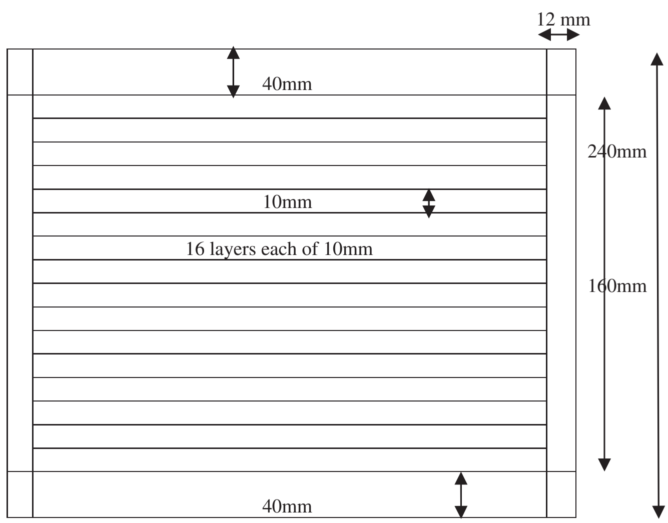

Fig. 7. Vertical section of HDRB (diameter 950 mm) and LRB (diameter 800 mm).

4. Results and discussion

4.1. Static analysis

The results of static analysis of FB and BI building are shown in Table 1 and Table 2 respectively. The design base shear of earthquake loading is greater as compared to that of wind loading. Therefore, the building and isolators are designed for seismic load and checked for wind loads. Lateral load due to wind is approximately 3% of building weight. It satisfies one of the conditions for base isolation i.e. lateral load due to wind should be less than 10% of building weight [49]. The design base shear and moment for FB building is 3936 kN and 89523 kN-m respectively. The maximum displacement at top story for BI building is 88.5 mm. In case of BI building the isolators absorbs the seismic load and displaces by 72.8 mm. The lateral loads for wind cause a relative displacement of 1.6 mm at top story. Due to wind loads, the isolator displaces by 52.2 mm using the axial loads on interior and exterior columns for which the isolators are designed. Table 3 shows the designed results of LRB and HDRB base isolators. The diameter of LRB isolator is 800 mm. The HDRB isolator is slightly larger having a diameter of 950 mm. All other parameters such as layer thickness, number of layers, height and shape of both isolators are same.

4.2. Free vibration analysis

Natural time periods or natural frequencies are the important characteristics of a structure. It can be used to analyse the results obtained by dynamic analysis. To evaluate natural frequencies, free vibration analysis of both FB and BI buildings has been carried out. The nonlinearities of base isolators have been considered appropriately for BI buildings. Tables 4–7 show the first 15 natural time periods, natural frequencies and modal accelerations of FB and BI buildings respectively. The first time period of the FB building is 0.9132 s whereas the BI building is having 2.847 s. The natural frequency associated with the building on seismic isolator is far lower than the corresponding frequency for the conventional fixed based foundation. The frequency is shifted to 0.35 Hz, which is in the target range of 0.3–0.5 Hz [2]. The first mode maximum acceleration has been reduced from 29.77 cm/s^2 to 9.768 cm/s^2 which is only one third of the former value. Results also show that the first global modes for non-isolated foundations are mainly rocking modes, whereas in the case of seismic isolation they are

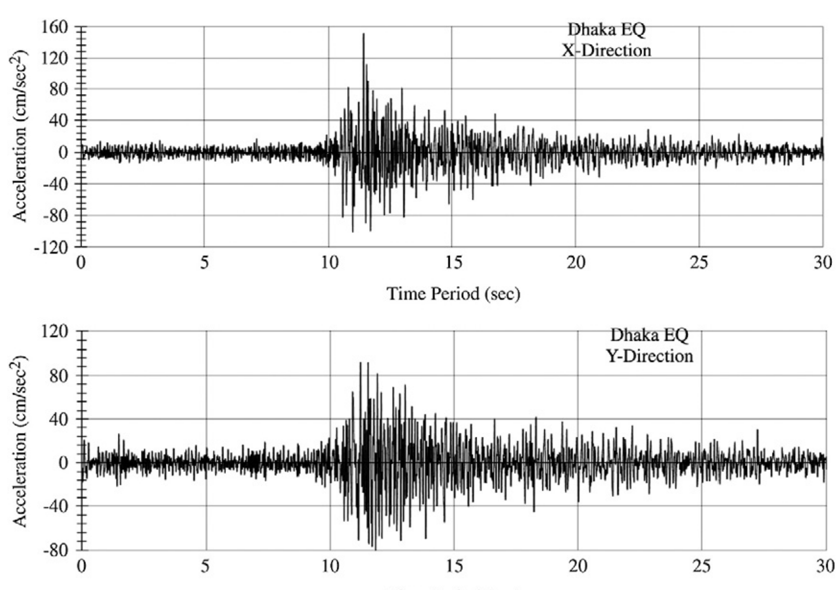

Fig. 8. Time history for Dhaka EQ.

Table 2
Static analysis results (BI building).

	Top story displacement (mm)	Isolator displacement (mm)	Total structure drift (mm)
Displacement (EQ load)	88.5	72.8	15.7
Displacement (wind load)	53.8	52.2	1.6

Table 3
Dimensions of designed isolators.

Bearing dimensions	LRB	HDR
Plan dimension (mm)	800	950
Layer thickness (mm)	10	10
No. of layers	16	16
Lead core size (mm)	150	–
Shape	Circular	Circular
Total height (mm)	240	240

Table 5
Free vibration analysis result (BI building).

Mode no.	Period	Frequency	Cicular frequency
	s	Cyc/s	Rad/s
1	2.847	0.351	2.207
2	2.847	0.351	2.207
3	2.837	0.353	2.215
4	0.478	2.090	13.135
5	0.478	2.090	13.135
6	0.416	2.407	15.121
7	0.214	4.680	29.406
8	0.214	4.680	29.406
9	0.194	5.151	32.365
10	0.169	5.913	37.153
11	0.160	6.258	39.321
12	0.160	6.258	39.321
13	0.145	6.875	43.194
14	0.141	7.084	44.511
15	0.135	7.383	46.387

associated to pure translation movements. It is also noted that the shift of natural vibration period of an isolated system points out that base isolation provides more flexible isolated system.

As the structural time period is less than 1.0 unit, seismic base shear is larger than base shear for wind and this wind induced lateral force is less than 10% of seismic weight of the building. Therefore, the incorporation of isolator is feasible for the structure.

4.3. Dynamic analysis

Dynamic analysis in time domain has been performed for FB and BI structures. Responses of both types of structures are obtained in x and y directions. The time histories obtained are maximum base shear, overturning moment and displacements. Then, the results of FB and BI structures are compared with each other. It demonstrates the advantages of the BI structure over the FB structure.

4.3.1. Responses of the FB structure

Table 8 describes the obtained result from the dynamic analysis of conventional fixed base building structure. Parameter wise comparison has been shown in the subsequent sections.

4.3.1.1. Base shear time history. Time histories of base shear in x and y directions for the FB building subjected to Dhaka earthquake are shown in Fig. 9. In the x-direction, the response reflects minor oscillations and it continues more or less in same pattern up to 10 s until a sudden shoot up occurs at 10 s. The maximum base shear peak occurs at 2365 kN in the negative direction. In the case of the y-direction, both negative and positive peaks occur between 12 and 13 s. The oscillation then reduces to 15–25% of the earthquake ground excitation. It is also noticeable that the maximum positive base shear value is 1730 kN. It occurs first, whereas the negative base shear occurs 1 s later with a 5% lower value. The analysis represents the maximum lateral base shear force as $V = 2365$ kN in the x-direction and 1730 kN in the y-direction. Here the y-directional value is 17% lower than that of the x-direction.

4.3.1.2. Overturning moment time history. Overturning moment is an important design criterion for buildings subjected to seismic loading. It is the product of base shear and the lever arm of lateral forces. Fig. 10 shows the time histories of overturning moment for the FB building. In comparison with the base shear response, the overturning moment represents a quite similar trend of oscillation. In the x direction the maximum positive base moment value is 19,740 kN-m. It occurs first whereas the maximum negative value occurs 1 s later with a 62% higher magnitude. In the y-direction, minor oscillations continue in same pattern up to 11 s and then the excitations are amplified. Within 11 to 12 s both negative and positive peaks transpire. It is also noticeable that the positive peak base moment value 24,200 kN-m, crops up first while the negative is 5% lower occurring 1–1.5 s later. In the y-direction the maximum overturning moment is 24,800 kN. It occurs in the positive direction. The analysis predicts a maximum moment of 32,120 kN-m in the x-direction and

Table 4
Free vibration analysis result (FB building).

Mode no.	Period	Frequency	Cicular frequency
	s	Cyc/s	Rad/s
1	0.913201	1.095	6.8804
2	0.913201	1.095	6.8804
3	0.820971	1.2181	7.6534
4	0.305778	3.2703	20.548
5	0.305778	3.2703	20.548
6	0.277169	3.6079	22.669
7	0.169141	5.9122	37.148
8	0.169141	5.9122	37.148
9	0.156279	6.3988	40.205
10	0.112683	8.8745	55.76
11	0.109486	9.1335	57.388
12	0.109486	9.1335	57.388
13	0.106621	9.379	58.93
14	0.106621	9.379	58.93
15	0.100209	9.9791	62.701

Table 6
Modal acceleration for varying Period (FB building).

Period	Acceleration	Acceleration
s	x-Direction (cm/s^2)	y-Direction (cm/s^2)
0.913201	29.77	8.935
0.913201	29.77	8.935
0.820971	33.044	9.917
0.305778	46.06	13.824
0.305778	46.06	13.824
0.277169	46.06	13.824
0.169141	46.06	13.824
0.169141	46.06	13.824
0.156279	46.06	13.824
0.112683	46.06	13.824
0.109486	45.87	13.767
0.109486	45.87	13.767
0.106621	44.814	13.45
0.106621	44.814	13.45
0.100209	42.449	12.74

Table 7
Modal acceleration for varying period (BI building).

Period	Acceleration	Acceleration
s	x-Direction (cm/s^2)	y-Direction (cm/s^2)
2.847212	9.768	2.932
2.847212	9.138	2.743
2.836714	9.285	2.787
0.478372	46.05	13.821
0.478372	44.588	13.382
0.41553	45.13	13.545
0.213669	46.059	13.823
0.213669	44.803	13.446
0.194135	45.23	13.575
0.169118	45.96	13.794
0.15979	45.931	13.785
0.15979	45.834	13.756
0.145465	45.93	13.785
0.14116	45.961	13.794
0.135452	45.864	13.765

Table 8
Dynamic analysis results (FB buildings).

Parameter	Time domain analysis value
Design base shear (kN) in the x-direction	2365
Design base shear (kN) in the y-direction	1730
Design base moment (kN-m) in the x-direction	32,120
Design base moment (kN-m) in the y-direction	24,800
Top story displacement in x-direction (cm)	4.3
Top story displacement in the y-direction (cm)	4.9

24,800 kN-m in the y-direction. The self-weight of the structure plays a vital role in stabilizing the building against the overturning moment. The overturning moment is larger in the y-direction because it results from maximum base shear in the x-direction.

4.3.1.3. Acceleration response. Floor acceleration plays a vital role for structural analysis as it outputs the building inertial load for different levels. Once the mass is known for a level the inertia load can be determined through simple linear calculation. In contrast to the relative displacement response, the floor accelerations are found to be more sensitive to impact occurrences. Acceleration histories for the FB building are demonstrated in Fig. 11. The transitional larger oscillation for the top floor acceleration is from the 11 to 15 second region of the excitation history in the x-direction. In the y-direction the vacillation pattern is also effective for the similar transition area but more peaks are larger here and the average of this region is high as well. The peak floor acceleration at the top is 0.215 g in the x-direction which is about 40% greater and 0.122 g in the y-direction which is about 25% greater than the input Dhaka earthquake ground excitation. This shows that with increasing height the acceleration is also increased due to the seismic impact on the building structure.

4.3.1.4. Displacement response. The structure displaces laterally due to earthquake excitation. Fig. 12 shows the displacement time histories at the top level of the FB building under the selected Dhaka earthquake. It is observed that the top floor displacement oscillates between $+2.9$ and -4.3 cm in the x-direction. In the y-direction the displacement fluctuates between -4.9 and $+3.4$ cm in the 12 to 13 second zones. This phenomenon is expected as displacement is proportional to the lateral force. It is clearly evident from the FB case that with support the lateral displacement is zero. Maximum displacement of the top floor resembles total structural drift which is 4.3 cm in the x-direction and 4.9 cm in the y-direction respectively. Displacement in the y-direction is 14% higher compared to the displacement in the x-direction. The structure deforms from zero at the base to maximum sway at the top floor in a parabolic pattern as shown in Fig. 1a.

4.3.1.5. Statistical analysis. Table 9 shows the results of statistical analysis of the FB building. The maximum, minimum, mean, standard deviation and root mean square values of base shear, overturning moment, floor acceleration and floor displacement respectively are evaluated. The maximum base shear and overturning moment values are 2365 kN and 32,120 kN-m respectively. It occurs in the negative x-direction. The values fluctuate with a standard deviation of 284.65 and 3706.58 respectively. The FB building has a maximum displacement of 4.9 mm at the top

4.3.2. Responses of BI structure

The bearings designed are linked at the bottom of the respective columns at base level of the building to ensure all the properties in the spring. The structure with isolators is then analysed again for time domain. The evidences in Tables 10–11 have been attained from dynamic analysis of the BI building.

4.3.2.1. Base shear time history. The time histories for base shear of the BI building are illustrated in Fig. 13. The analysis executes the collapse of the structure to occurred at $V = 363$ kN in the x-direction and 651 kN in the y-direction. Peak base shear reduces drastically compared to the FB structure which fulfils the demand of isolating the superstructure. It is clear that for the isolated building base shear reduces by up to 43% of the corresponding base shear of fixed value. It predicts a good amount of structural savings and economic assistances as well.

4.3.2.2. Overturning moment time history. Accordingly in Fig. 14 overturning moment corresponding to the BI building is presented. The breakdown forecasts the ultimate moment capacity $M = 12{,}510$ kN-m in the x-direction and 11,260 kN-m in the y-direction to prevent collapse. Overturning base moment reduces by up to 35% of the fixed foundational base moment. So the building becomes more stabilized compared to the FB structure. The building experiences more flexibility even when using the same structural element configuration. Allowance

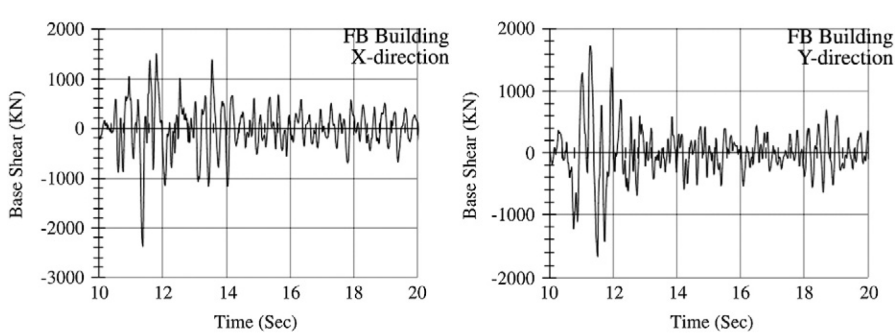

Fig. 9. Base shear in x and y directions (fixed based case).

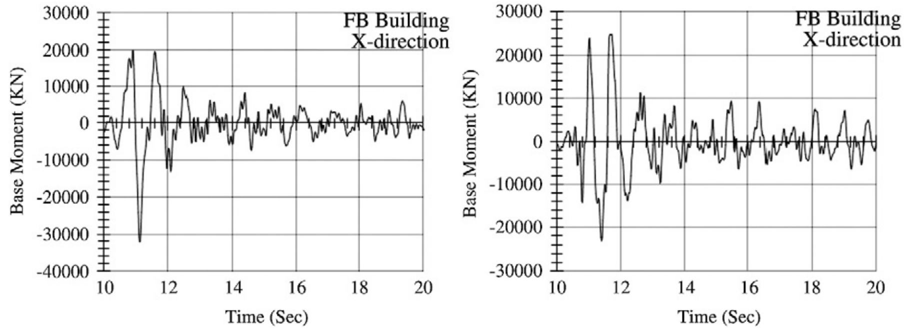

Fig. 10. Base moment in x and y directions (fixed based case).

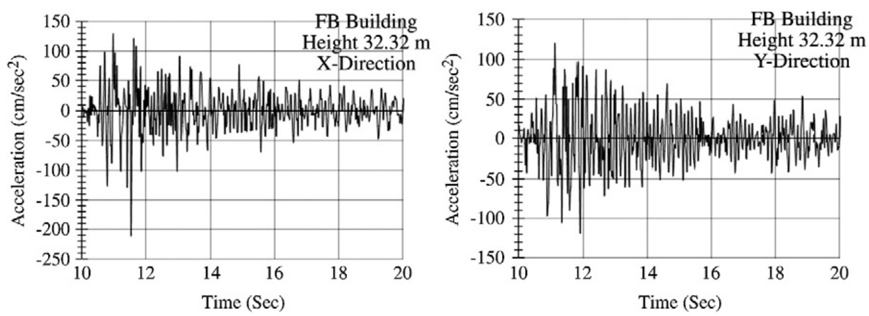

Fig. 11. Acceleration time history in x and y directions (fixed based case).

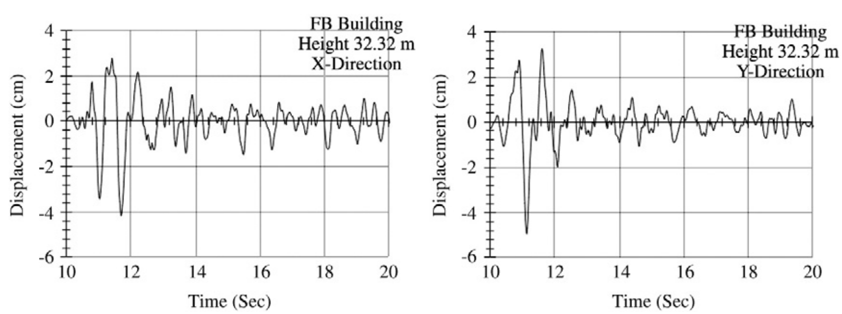

Fig. 12. Displacement time history in x and y directions (fixed based case).

of translational movements of support abruptly changes the pattern of whole structure deformation like in Fig. 1b.

4.3.2.3. Acceleration response. Acceleration time histories for the seismically isolated building are given in Fig. 15 for the top and base of the structure. The peak floor accelerations in this case at the top are 0.159 g in the x-direction which is about 5% greater and 0.095 g in the y-direction which is about 5% lower than the input ground excitation. For the isolation flexibility the structure experiences a mentionable amount of acceleration at the base also that is valued as 0.12 g in the x-direction and 0.08 g in the y-direction. For the isolated building peak, acceleration at the top floor reduces by up to 74% of the

Table 9
Statistical analysis results of time domain responses (FB building).

structural parameter	Direction	Fixed				
		Maximum	Minimum	Mean	Standard deviation	Root mean square
Base shear (kN)	x	1506	2365	−0.03957	284.6573	284.5625
	y	1730	1649	0.268015	262.4023	262.315
Base moment (kN-m)	x	19,740	32,120	−4.19639	3706.582	3705.349
	y	24,800	23,060	−6.27736	3937.748	3936.441
Top point acceleration (cm/s^2)	x	129	211	−0.00454	20.91838	20.91141
	y	120	118	−0.00148	20.56363	20.55678
Base acceleration	x	–	–			
	y	–	–			
Top point displacement (cm)	x	2.9	4.3	−0.00057	0.545929	0.545748
	y	3.4	4.9	0.00459	0.721892	0.721666
Base displacement (cm)	x	–	–			
	y	–	–			

Table 10
Displacement output in dynamic analysis (BI building).

	Isolator displacement (mm)	Total structure drift (mm)
x-Direction (static analysis)	72.8	15.7
y-Direction (static analysis)	72.8	15.7
x-Direction (time domain analysis)	16.9	8.1
y-Direction (time domain analysis)	29.8	12.2

Table 11
Base Shear and base moment in dynamic analysis (BI building).

Parameter	Time domain analysis value
Design base shear (kN) in the x direction	363
Design base shear (kN) in the y direction	651
Design Base Moment (kN-m) in the x direction	12,510
Design base moment (kN-m) in the y direction	9523

corresponding top point acceleration of the fixed one. The isolated building exerts a good amount of acceleration at the base. But for the fixed one, at base, there is a null acceleration excitation and displacement.

4.3.2.4. Displacement response. Time histories of displacement for the BI building are shown in Fig. 16 for the top and base of the structure. In case of isolation the isolator itself moves laterally. So, there is also mentionable lateral movement in the joint/support at base level. It is observed that the peak displacement of the top floor is 2.5 cm in the x-direction and 4.2 cm in the y-direction whereas the values at the base level are 0.8 cm and 1.7 cm respectively. Here total structure drifts come to 1.7 cm in the x-direction and 2.5 cm in the y-direction significantly lower than that in the FB case. In case of isolation the isolator itself moves laterally while displacement is zero for the FB building. So, total structural drift is lesser than that of the fixed based case. For the isolated structure, the displacement of both the superstructure and isolator increases as the superstructure becomes more flexible. This trend is true for static, free vibration and time domain analysis. The peak top displacement reduces by up to 35% of the fixed foundational lateral movement. Magnitudes of displacement history indicate that the base isolator absorbs most of the displacement of the superstructure.

4.3.2.5. Statistical investigation. Table 12 shows the statistical study of the BI structure for nonlinear dynamic behaviour. It includes base shear, overturning moment, floor acceleration and floor displacement with their maximum and minimum values and mean, standard deviation, root mean square values as well.

The acceleration responses reduce significantly in the base-isolated building compared to the original building as expected. The base shears in each direction are decreased compared to the fixed building. On the other hand, as regards the building response in terms of displacements, the maximum horizontal displacements evaluated at the foundation level were well below the expected static design displacement of isolators.

Peak base shear reduces significantly compared to the FB structure as expected, which fulfils the demand of isolating superstructure. The reduction of overturning base moment due to isolation indicates that the building becomes more stabilized compare to the FB structure.

4.3.2.6. Influence of nonlinear automated simulation. Consideration of nonlinearity of base isolation exerts actual behaviour of the isolating elements and their upshot on the structural responses. The selected automated simulation includes the precise modelling and analysis in a consistent manner. Automated fast nonlinear analysis is found to be efficient requiring very less time but it offers precise solution. Isolating strategy predicts well amount of structural savings and also economic assistances as well. The building experiences additional flexibility even using same structural elements configuration. Allowance of transitional movements of support abruptly changes the pattern of whole structure deformation. As a concluding remark, for applications like the medium rise building structures in soft to medium stiff soil of Dhaka, where the main concern is the mitigation of the seismic excitation at the supports of critical components, a base isolation is to be viably recommended.

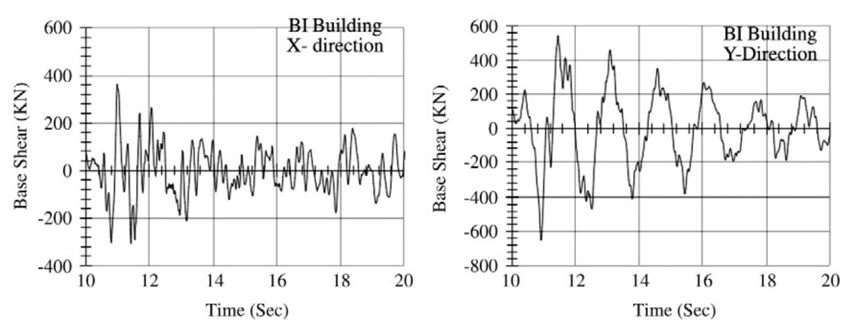

Fig. 13. Base shear in x and y directions (isolated based case).

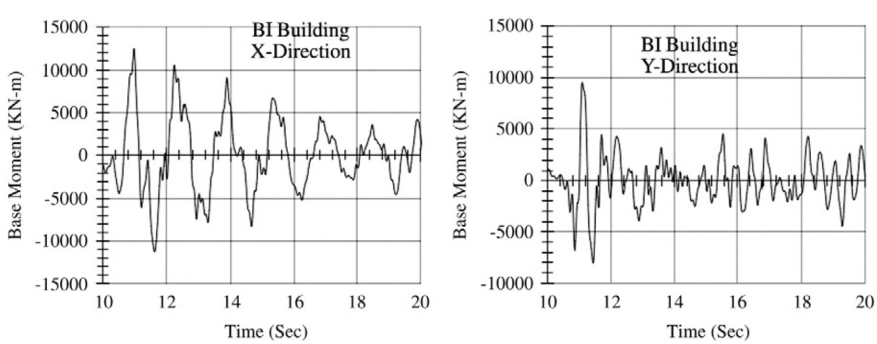

Fig. 14. Base moment in x and y directions (isolated based case).

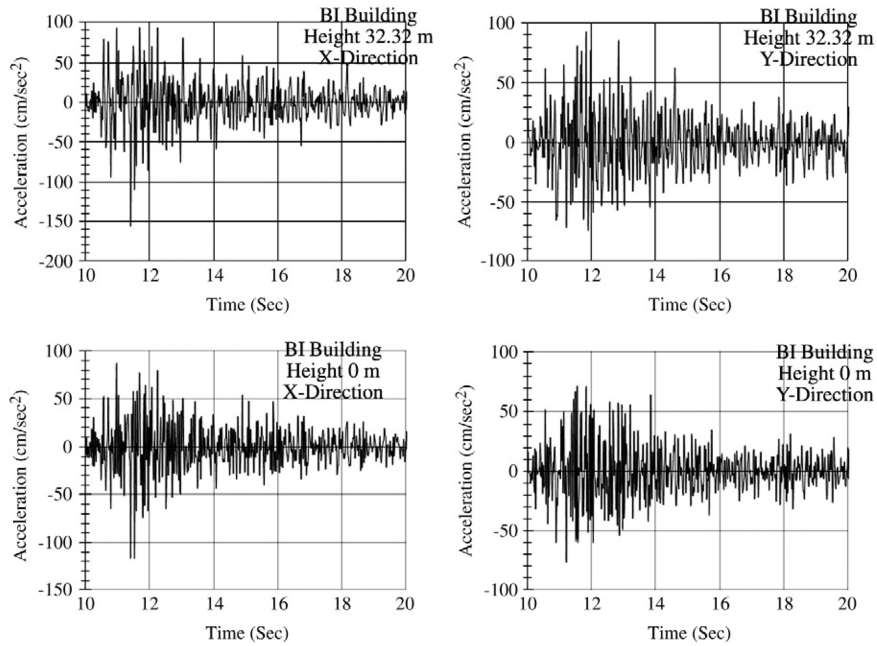

Fig. 15. Acceleration time history in x and y directions (isolated based case).

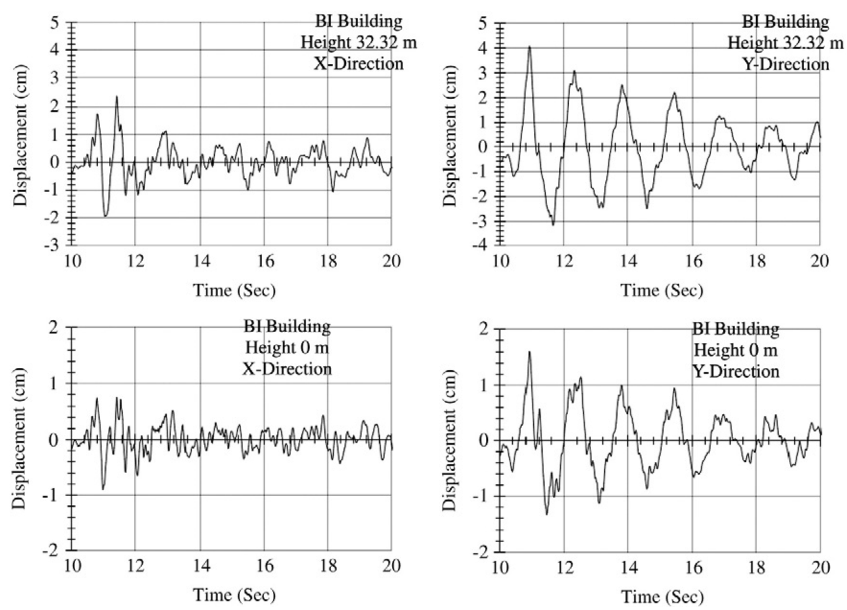

Fig. 16. Displacement time history in x and y directions (isolated based case).

Table 12
Statistical analysis results of time domain responses (BI Building).

Structural parameter	Direction	Isolated				
		Maximum	Minimum	Mean	Standard deviation	Root mean square
Base shear (kN)	x	363	306	0.7575748	69.668	69.648909
	y	546	651	−0.85059	131.0317	130.9908
Base moment (kN-m)	x	12,510	11,260	18.24944	2655.256	2654.434
	y	9523	8041	9.170651	1648.975	1648.451
Top point acceleration (cm/s^2)	x	94	156	−0.00272	16.43835	16.43287
	y	93	73	−0.01663	15.31261	15.30752
Base acceleration (cm/s^2)	x	87	117	0.006199	14.13679	14.13208
	y	71	76	−0.00627	13.34395	13.3395
Top point displacement (cm)	x	2.5	2.0	−0.00337	0.428622	0.428492
	y	4.2	3.3	0.007431	0.847827	0.847577
Base displacement (cm)	x	0.8	0.9	−0.00108	0.170408	0.170355
	y	1.7	1.3	0.003522	0.319779	0.319692

5. Conclusion

Nonlinear time domain analyses under bi-directional earthquake history have been carried out. The performances of multi storey structures isolated with the bearing systems are evaluated by Fast Nonlinear Analysis. The responses due to structural changes are accurately formulated to estimate responses of base-isolated structures. The results from the analyses, static, free vibration and time domain are obtained. The summarized findings are as follows.

1. The major effect of base isolation on the seismic response is by far the radical reduction of horizontal accelerations. It is observed that, on soft to medium stiff soil, the reduction of response peak accelerations at the support level is not far from ten times for isolators with respect to the case without base isolation. This comparison was performed for foundations on soft to medium stiff soil, but similar conclusions could be drawn for any type of soil.
2. As regards the building response in terms of displacements, the maximum horizontal displacements evaluated at the foundation level were well below the expected static design displacement of isolators.
3. Extensive sensitivity studies to find the influence of various important structural parameters of both isolator and superstructure on the behaviour of isolated structure are possible by the SAP module.
4. Both the displacement and acceleration responses of a superstructure without an isolator are much more sensitive while the displacement and acceleration responses of a superstructure with an isolator reduce significantly.
5. The base isolator is more effective to mitigate displacement than acceleration.
6. Peak base shear reduces significantly compared to the FB structure which fulfils the demand of isolating superstructure. It predicts well the amount of structural savings and also provides economic assistance.
7. There is a reliable reduction of overturning base moment than the fixed foundational base moment. So the building becomes more stabilized compared to the FB structure. The building experiences more flexibility even when using the same structural element configuration. Allowance of transitional movements of support abruptly changes the pattern of whole structure deformation.
8. For applications like the medium rise building structures in Dhaka, where the main concern is the mitigation of the seismic excitation at the supports of critical components, a base isolation is to be viably recommended.
9. This investigation was based on free-field excitations in accordance with the site specific bilateral EQ data. For applications on buildings on soft soils where more significant long period excitations are to be taken into account, the design of the base isolation needs particular care, in order to avoid resonance effects.
10. In this case, the most effective choice appears that of HDRB and LRB bearings, as resulting in a lower isolation frequency and then in lower peak accelerations, but the isolation choice should generally be based on the best compromise between the reduction of floor accelerations and the amplification of building rigid-body displacements.
11. To accurately determine the collapse loads and acceleration–displacement behaviours of the structures, in the nonlinear time history analysis $P-\delta$ effects need to be considered as does the more true sketch be accessed.

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