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# Implication of rubber- steel bearing nonlinear models on soft storey structures

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**Abstract.** Soft storey buildings are characterised by having a storey that has a large amount of open space. This soft storey creates a major weak point during an earthquake. As the soft stories are typically associated with retail spaces and parking garages, they are often on the lower levels of tall building structures. Thus, when these stories collapse, the entire building can also collapse, causing serious structural damage that may render the structure completely unusable. The use of special soft storey is predominant in the tall building structures constructed by several local developers, making the issue important for local building structures. In this study, the effect of the incorporation of an isolator on the seismic behaviour of tall building structures is examined. The structures are subjected to earthquakes typical of the local city, and the isolator is incorporated with the appropriate isolator time period and damping ratio. A FEM-based computational relationship is proposed to increase the storey height so as to incorporate the isolator with the same time period and damping ratio for both a lead rubber bearing (LRB) and high-damping rubber bearing (HDRB). The study demonstrates that the values of the FEM-based structural design parameters are greatly reduced when the isolator is used. It is more beneficial to incorporate a LRB than a HDRB.

**Keywords:** rubber-steel bearing, nonlinear model; soft storey structure; floor acceleration; masonry infill; inter-storey drift, seismic isolation

## 1. Introduction

Isolation techniques for structures are being popular in structural engineering at the recent decades. The first patent for an isolation system was filed in the early 20th century, and the strategy gained popularity in structural engineering during the 1970s (Islam *et al*. 2013a). For building structures, base isolation has been implemented since the late 1970s (Hussain *et al*., 2010; Islam *et al*. 2012c). Rubber foundation elements can help to minimise earthquake damage to buildings under even the tremendous forces that the buildings must endure in a major earthquake (Islam *et al*. 2012b). The incorporation of such a device may alter the structural behaviour of a soft

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storey building. A soft storey building is defined as a structure that has a floor with a 70% lower stiffness than that of the adjacent floor. Whereas the unobstructed space of the soft storey might be aesthetically or commercially desirable, there is less opportunity to install walls to distribute lateral forces so that the building can handle the swaying caused by an earthquake. A seismic isolation system has not been adequately considered in research for buildings with soft storeys. The effects of the increased use of soft storeys on structural feasibility in a base-isolated (BI) building are still unknown. Therefore, thorough research on this topic is crucial. Unlike the conventional design approach, which is based on an increased resistance (strengthening) of the structures, the seismic isolation concept reduces the dynamic loads induced by the earthquake at the base of the structures themselves (Ates 2012; Sharma and Jangid 2012; Soni *et al*. 2011). Seismic isolation separates the structure from the harmful motions of the ground by providing flexibility and energy dissipation capability through the insertion of the isolated devices, i.e., isolators, between the foundation and building structure (Ates and Yurdakul 2011; Khoshnoudian and Azad 2011).

The development of the lead rubber bearing (LRB 1970s) and high-damping rubber bearing (HDRB, early 1980s) has played an indispensable role in isolating superstructures, as these materials provide excellent seismic response lessening (Hussain *et al*., 2010; Islam *et al*., 2013b). A significant amount of research in the field of BI has focused on the use of elastomeric bearings, such as HDRBs and LRBs, and their effects on buildings (Islam *et al*. 2011d). Jangid (Jangid, 2007) investigated the seismic responses of multistorey buildings to near fault motion isolated by LRBs. Dall'Asta and Ragni (Dall'Asta and Ragni 2006; Dall'Asta and Ragni 2008) presented experimental tests and an analytical model and studied the nonlinear dynamic behaviour of HDRBs. Islam *et al*. (Islam *et al*. 2012a) studied RB isolation in multistorey buildings in a low-to-medium risk seismicity region. Sharma and Jangid (Sharma and Jangid 2011) evaluated the influence of a high initial isolator stiffness on the seismic response of a BI benchmark building. Wilkinson and Hiley (Wilkinson and Hiley 2006) presented a non-linear response history model for the seismic analysis of multistorey framed buildings. The resonant behaviour of BI high-rise buildings under long-period ground motions was analysed by Ariga *et al.* (Ariga *et al*. 2006).

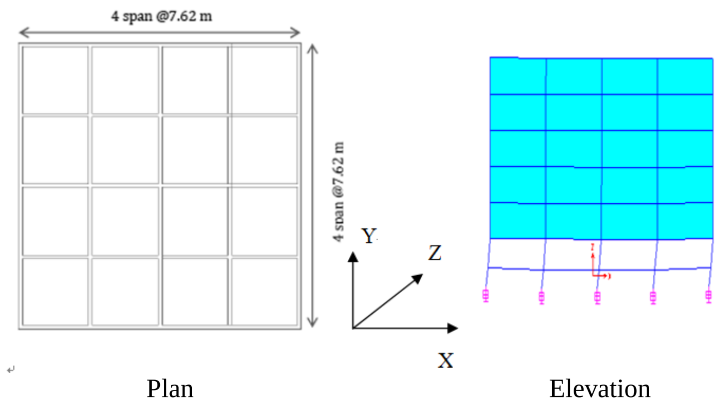
Islam *et al*. (Islam *et al*., 2011a) explored the corollary of seismic BI systems on soft storey buildings. They observed that the structural parameters are significantly reduced when using isolators. The structure experiences abrupt, potentially severe structural responses at the soft storey level that may cause immature failure. Yoshimura (Yoshimura 1997) performed a nonlinear analysis of a reinforced concrete (RC) soft storey building that collapsed during the Hyogoken-Nanbu earthquake in 1995. Mo and Chang (Mo and Chang 1995) studied the application of BI and its modification (Chen and Constantinou 1990) to a building with a soft first storey. Chen and Constantinou (Chen and Constantinou 1992) introduced the use of Teflon sliders to advance the soft first storey concept. Kirac *et al*. (Kirac *et al*. 2011) and Wilbowo *et al*. (Wibowo *et al*. 2010) performed failure and collapse modelling analyses of weak storey buildings.

Despite the worldwide popularity of the incorporation of RBs, there is little research on the practical implementation of such devices in local buildings in Bangladesh, especially in such a medium-risk seismicity region as Dhaka, and following local requirements. The non-linear model of the rubber-steel bearing (RSB) is an innovative model that can handle the precise behaviour of isolation devices. The bidirectional nature of the earthquake is also very important to consider but has largely overlooked in the literature. Time-history and response spectrum methods have only rarely been implemented simultaneously in buildings to model isolated behaviours. The time-history method is relatively more time consuming, lengthy, and costly. The response spectrum analysis is relatively more rapid, concise, and economical. However, the time-domain method has been employed to include nonlinearities present in the structural systems.

Hence, the objective of the study is to incorporate rubber-steel bearing nonlinear models on soft storey structures and investigate its influence in the behavioural changes of structural parameters. This feasibility study adopted nonlinear models of HDRBs and LRBs. The suitability of incorporating the isolators was initially evaluated using equivalent static analysis. Then, dynamic analysis was performed to satisfy the structural limitations when executing different comparative contributions. The structural design implications of incorporating the isolators on the seismic behaviour of multistorey buildings subjected to a typical earthquake for the city of Dhaka, Bangladesh, were determined using RSBs with variable properties. The response behaviours for fixed and isolated buildings are discussed for various types of evaluations. The incorporation of RSBs results in notable decrease in the structural responses.

## 2. Structural model

The prototype building of six-storey with soft bottom storey shown in Fig. 1 was used for this study. First, its bearing properties were analysed. Using the same plan, the building was then analysed with three, four, and five stories to establish an important relationship to be discussed later in this section. Masonry walls were used for the upper stories of the tall buildings, and soft storey configuration was considered for the bottom floor. All walls were defined in the SAP 2000 finite element program (Habibullah 2005) as shell elements of 250-mm-thick brick wall. The brick wall properties are shown in Table 1. The evaluations in this section provide the overall response characteristics of a tall building with a special soft storey utilising a LRB or HDRB system. The isolators have been incorporated in the prototype building with a soft bottom storey.




Plan Elevation

Fig. 1 Plan and elevation of the isolated-based building with soft bottom story

Table 1 Masonry infill properties

| Parameters | Values with unit | Comment |
|---|---|---|
| Compressive strength | $f'_m$ =7.5 MPa | Mix Proportion 1:4 |
| Modulus of Elasticity | $E_m$ =5625 N/mm$^2$ | 750*$f'_m$ |
| Shear Modulus | G = 2250 N/mm$^2$ | 0.4*$E_m$ |
| Unit Weight | $\gamma$=18.85 KN/m$^3$ | -- |

## 3. Computer-aided design of the isolator

Four variations of LRBs and HDRBs were evaluated. The designs were completed using the program DESBEA12 earthquake (Islam *et al*., 2012b) which implements the design procedures according to the UBC 1997 standards (Uniform Building Code (UBC), 1997). The design considers an S3-type soil profile for Dhaka with a seismic zone coefficient of Z = 0.15 (Bangladesh National Building Code 1993) and more than 15 km of a Type A fault.

### *3.1 Modelling of the isolators*

A hysteresis model provides the stiffness and resistance under any displacement history. In addition, the basic characteristics are defined through member geometry and material properties. The effective stiffness ($K_{eff}$) and equivalent viscous damping, derived from the isolator's energy dissipated per cycle (EDC), are essential for the response spectrum analysis. The force-deformation behaviour of the isolators in this study is modelled as a nonlinear link element for both the LRB and HDRB cases.

### *3.2 Non-linear LRB model*

A LRB is formed by force-fitting a lead plug into a preformed hole in a low-damping elastomeric bearing as shown in Fig. 2a. The basic components of such a bearing are rubber and steel plates in alternating layers. The steel plates force the lead plug in the bearing to deform in shear. The LRB system offers the parallel action of a linear spring and damping. The system decouples the structure from the horizontal components of earthquake ground motion by interjecting a layer of low horizontal stiffness between the foundation and superstructure. The LRB provides the required amount of damping, horizontal flexibility, and vertical stiffness. The large difference in the damping of the structure and isolation device creates a non-classically damped system. Such physiognomies lead to a coupling of the equations of motion. An elastic-perfectly plastic hysteretic model was used to handle the essential isolation characteristics in the non-linear model. The behaviour is varied throughout the parameters, i.e., yield point load for the lead core, horizontal stiffness (lead core contribution), and horizontal stiffness (elastomer contribution). The non-linear force deformation behaviour of the RSB is modelled through three parameters: i) characteristic strength, ii) post-elastic stiffness, and iii) yield displacement. An idealised hysteresis for the bearing is shown in Fig. 2b. The force intercept at zero displacement in a hysteresis, $Q_d$, is the characteristic strength, which is related to the yield strength as follows

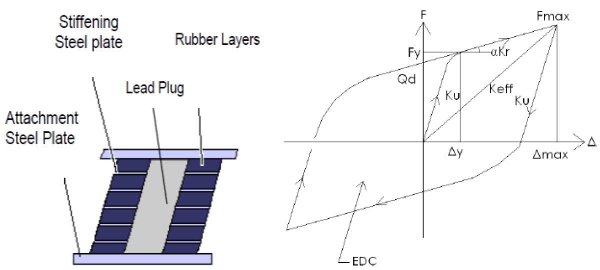



Fig. 2 LRB deformation pattern (left) and idealized non-linear hysteretic model (right)

$$Q_d = \sigma_y A_{pl} \tag{1}$$

where the yield strength, $\sigma_y$, is dependent on the vertical load and lead core confinement. $A_{pl}$ is the area of lead core.

The post-elastic stiffness is defined as follows

$$K_r = \frac{G_\gamma A_r}{T_r} \tag{2}$$

The elastic (or unloading) stiffness (Kilar and Koren 2009) is defined as follows

$$K_u = 6.5 K_r (1 + \frac{12 A_{pl}}{A_r}) \tag{3}$$

The weight of the structure, *W*, is used to define a bilinear model. The ratio of the post-elastic stiffness to the yield stiffness varies within 0.08-0.12 for the LRB. When the peak displacement of a bilinear model is larger than the yield displacement, the lateral shear force, *F*, and effective stiffness, $K_{eff}$, (secant stiffness) at peak displacement for a bilinear system can be calculated from the following equations.

Effective stiffness

$$K_{eff} = \frac{F_m}{\Delta} \tag{4}$$

$F_m = F_{max}$ is the maximum force can be derived as

$$F_m = Q_d + K_r \Delta \tag{5}$$

Effective period

$$T_e = 2\pi \sqrt{\frac{W}{g \sum K_{eff}}} \tag{6}$$

Equivalent viscous damping

$$\beta = \frac{1}{2\pi} (\frac{A_h}{K_{eff} \Delta^2}) \tag{7}$$

$A_h$ is the area of the hysteresis loop.

LRB isolators are strongly nonlinear, i.e., the parameters $K_{eff}$ and $\beta$ are valid only for the design displacement, $\Delta_{max.}$ The maximum isolator displacement is calculated as follows

$$\Delta_m = \frac{S_a T_e^{\ 2}}{4\pi^2 B} \tag{8}$$

where $S_a$ is the spectral acceleration at $T_e$, $\Delta_m$ is the maximum displacement, $B$ is damping coefficient.

### 3.3 Non-linear HDRB model

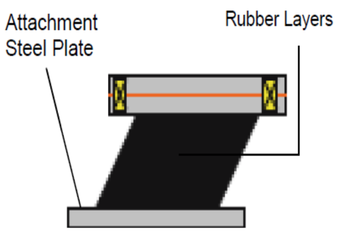
A HDRB consists of thin layers of high-damping rubber and steel plates fabricated in alternating layers, as illustrated in Fig. 3a. The low shear modulus of the elastomer controls the horizontal stiffness of the bearing. The steel plates provide high vertical stiffness and prevent bulging of the rubber. Horizontal stiffness is not affected by the high vertical stiffness for such a RSB. Damping in the isolation system is increased by adding extra-fine carbon blocks, oils, resins, and other proprietary fillers. The parallel action of the linear spring and viscous damping is the dominant feature of the HDRB system. Furthermore, the damping in this bearing model is neither viscous nor hysteretic. HDRB uses a lower stiffness to obtain a higher natural period. An equivalent linear elastic viscous damping model was chosen to configure the HDRB (Fig. 3b). The non-linear force-deformation characteristic of the RSB is swapped through effective elastic stiffness and effective viscous damping. In this model:

- Instead of Kr, stiffness is expressed as the effective horizontal stiffness $K_{eff}$.
- Damping is considered as effective viscous damping.

The equations required to model the HDRB follows Eqs. (2) and (4)-(8). The elastic (or unloading) stiffness is defined as follows

$$K_u = K_r \tag{9}$$

The recent earthquake was scaled to produce the desired earthquake load for the Dhaka buildings. The time history (Islam *et al*. 2011c) and corresponding response spectrum (Islam *et al*. 2011b) for 5% damping for this record are shown in Figs. 4 and 5, respectively. This load is the design basis earthquake (DBE) used to evaluate the structural response. The maximum capable earthquake (MCE), which is a function of the DBE, is used to obtain maximum isolator displacements. Each isolation system was defined with effective periods of 1.5, 2.0, 2.5, and 3.0 s, spanning the typical range of isolation system periods. Table 2 lists the variations evaluated and hysteresis parameters used in the model.



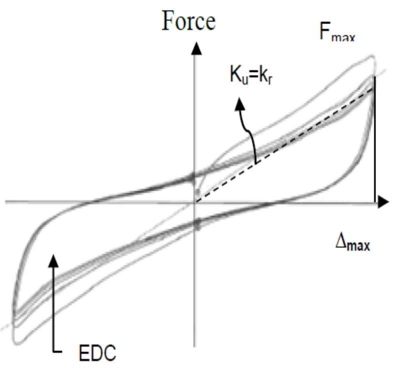
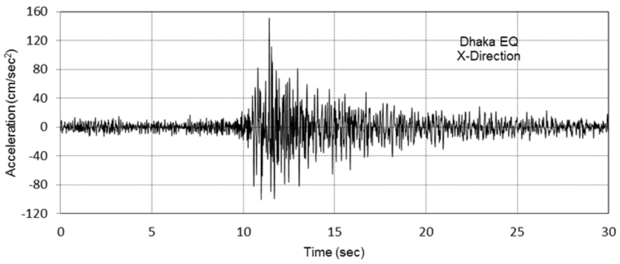



Fig. 3 HDRB deformation pattern (left) and idealized non-linear hysteretic model (right)



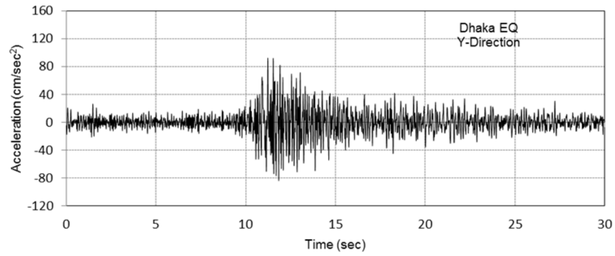




Fig. 4 Selected time history for dhaka EQ in X-direction (top) and Y-direction (bottom)

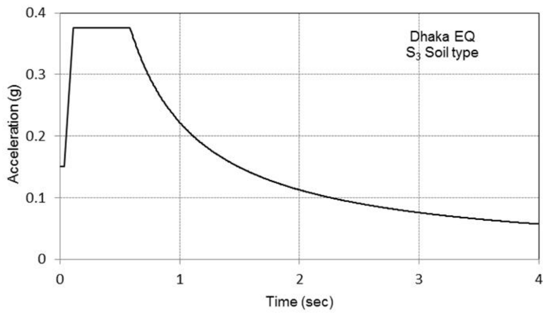



Fig. 5 Selected acceleration response spectrum for Dhaka EQ

Table 2 Isolation system variations

| Rubber-steel bearing | Characteristic Strength ($Q_d$) | Period of Isolator, $T_i$ (sec) | Initial Stiffness, $K_1$ (KN/mm) | Post-yield stiffness, $K_2$ (KN/mm) | Yield force, Fy (KN) |
|---|---|---|---|---|---|
| LRB1 | 0.050 | 2 | 12.20158 | 1.11331 | 328 |
| LRB2 | 0.075 | 2 | 12.20158 | 1.11331 | 492 |
| LRB3 | 0.100 | 2 | 12.20158 | 1.11331 | 656.1 |
| HDR | -- | 2 | 10.97653 | 2.45166 | 223 |

## 4. Structural analysis technique

The procedures for evaluating isolated structures are, in increasing order of complexity, (1) static analysis, (2) response spectrum analysis, and (3) time-history analysis. The dynamic cases: response spectrum and time-history analyses are considered in this study. Isolator design is based on an effective stiffness formulation and is typically an iterative process. The effective stiffness is estimated based on estimated displacements and then adjusted depending on the analysis results. For every analysis, the structural parameters at each level were saved and then processed to provide further investigation.

For this investigation, two types of isolators were chosen for comparison. The effects are compared for both the LRB and HDRB with an isolator period of Ti=2.0 s and 16% damping. Masonry infills of 83%, 67%, and 50% were tested. Exterior, middle, and interior columns were investigated to determine the variation in the isolator's characteristics with a fixed base structure.

## 5. Moment behavior of vertical member

The column moment values for the case of 83% masonry infill are shown in Figs. 6, 7, and 8 for an exterior corner, exterior middle, and interior column of the selected building. It is seen that for fixed based building there is a large moment in the ground level whereas for BI building, column moment at zero level is null. It is obvious that the rubber steel bearing made the structure flexible ensuring no moment at the base. Due to the soft storey at the ground to first floor, the column moment reduces a lot for the fixed building and again increases when masonry wall is existent. Yet the maximum moment is well below for both LRB and HDRB than the fixed based building at the fifth floor. The results demonstrate that the column moment decreases with decreasing storey height for the LRB up to 50-55% and for the HDRB up to 40-50%. It ensures the influence of RSB at the structural base. Furthermore, from the zero moment there is a gradual increment of moment with building height for BI building whether it shows ups and downs of moment magnitude for FB building showing possibility of sudden collapse. Therefore, the nonlinear model of RSB improves the structural behaviour in good fashion.

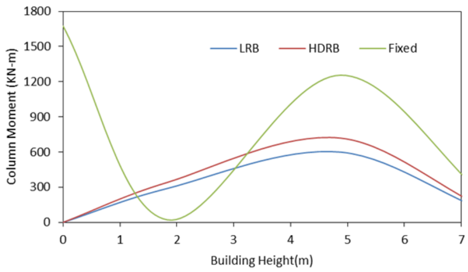



Fig. 6 Exterior corner column moment

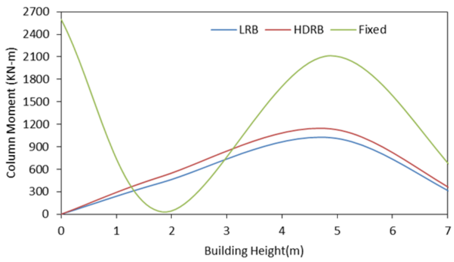



Fig. 7 Exterior middle column moment

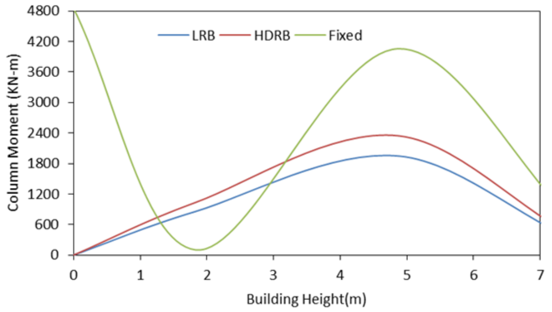



Fig. 8 Interior column moment

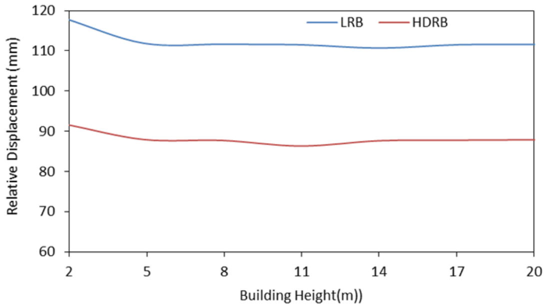



Fig. 9 Relative displacement of isolated with fixed base building

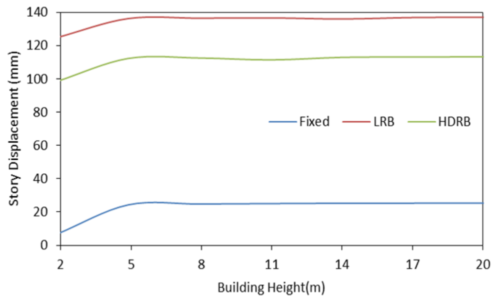



Fig. 10 Story displacement of isolated and fixed base building

## 6. Relative displacement and drift

Figs. 9, 10, and 11 present the variations of the relative displacements, storey displacements, and inter-storey drifts, respectively, for the 83% percentage of masonry infill. Behaviour changes for the masonry infill variations are discussed below. The relative displacement for the LRB is 25-35% greater than that for the HDRB, as compared with the fixed building (Fig. 9). The storey displacement increases 4.1-4.3 times and 5.2-5.5 times for the LRB and HDRB, respectively (Fig. 10). The storey drifts exhibit a peculiar characteristic. The results indicate a relatively large value for the first storey and a 74-80% and 78-82% decrease in the storey drift for the storey

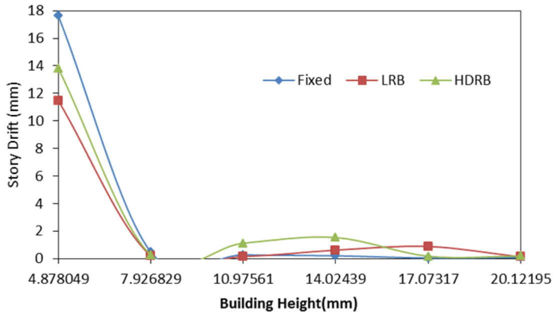



Fig. 11 Story drift of isolated and fixed base building

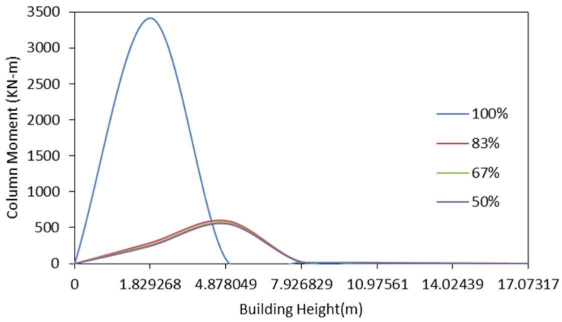



Fig. 12 Exterior corner column moment for varying infill (LRB)

immediately above when using the LRB and HDRB, respectively (Fig. 11). Such structural behaviour shows that the nonlinear RSB model at base gives allowances of significant lateral sway maintaining flexibility of superstructure.

## 7. Effect of infill variation

The percentage of infill in the structure changes the behaviour of different parameters. In this study, values for 83%, 67%, and 50% masonry infill were compared with 100% infill, as shown in Figs. 12-14 for column moments. It has been revealed that there is radical reduction of column moments for BI building when the masonry infill amount is decreased than cent percent. This amount comes to around 18% value of the column moment at 100% infill. With decreasing infill percentages, the column moment also increases which is true for both FB abs BI building. However, for the fixed structural case, 100% infill shows less column moment than lower infill percentage responses as the moment pattern is different for the fixed base case than isolated base.

Figs. 15 and 16 show inter-storey drift and relative displacements respectively for varying infill. The storey drift muscularly increases when the masonry infill is reduced. This phenomenon is applicable for both LRB and HDRB based building as well as FB structures. Similarly, the relative displacement also show significant upturn for lessening of infill percentage. The structural weight came from the more infill supress the superstructure to retain its position and so less displacement occur for higher infill. From the comparative evaluation, it is observed that the inter-storey drift is less BI building for non-cent percent masonry infill. This is desirable for flexible structure as isolation device shakes the superstructure from ground and the relative displacement among the adjacent floor ought to less.

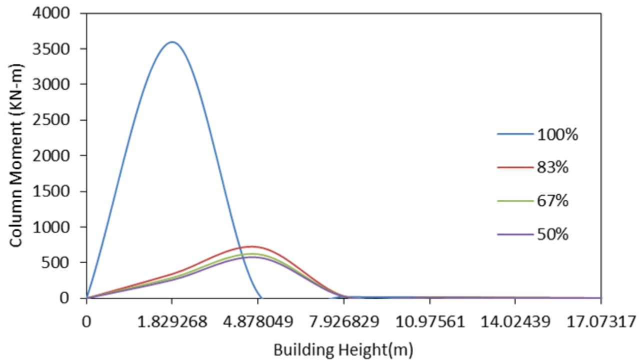



Fig. 13 Exterior corner column moment for varying infill (HDRB)

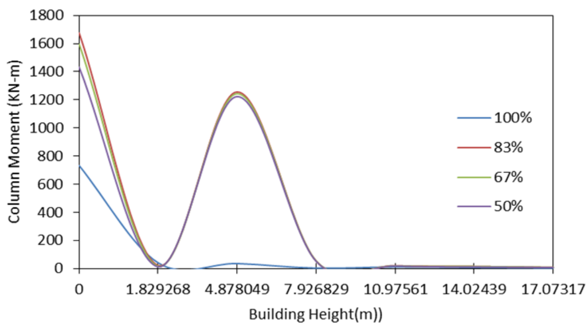



Fig. 14 Exterior corner column moment for varying infill (Fixed)

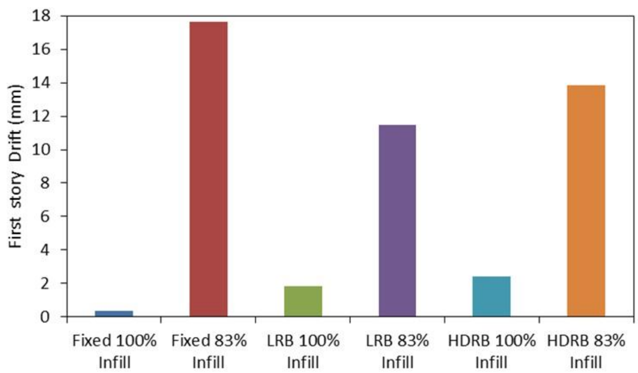



Fig. 15 Story drift at first story of isolated and fixed base building for varying infill

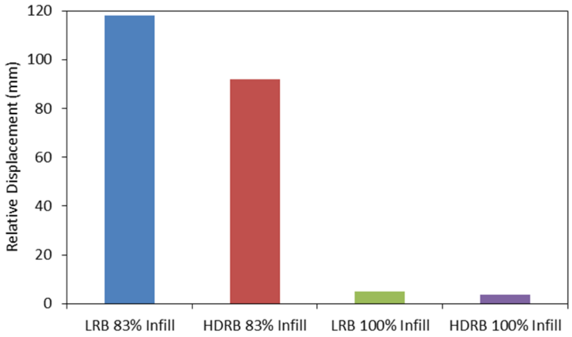



Fig. 16 Relative displacement at first story of Isolated with fixed base building for varying Infill

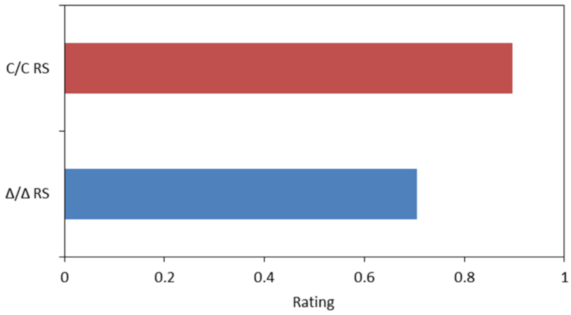



Fig. 17 Spectrum results for LRB1

## 8. Displacement in simulation and isolation design scheme

Fig. 17 displays a plot of the response spectrum analysis results divided by the design estimates. Displacements coefficients Δ/Δ are consistently lower than the design procedure results by a relatively small amount. A plot of the ratio of the response spectrum analysis results to the design procedure values for the first LRB (LRB 1) is shown in the illustration. The results demonstrate that the range of ratios for this system is 70% of the design parameter. However, in the time-history analysis, the values are significantly lower (Fig. 18). As the building period increases, the effects of building flexibility become more important and the response spectrum values diverge from the design procedure results. Because of this effect, the base displacement and base shear coefficient are lower for the buildings with longer periods.

## 9. Shear in simulation and isolation design scheme

The ratios of the shear coefficients c/c from the response spectrum and time-history analyses to the values predicted by the design procedure are plotted in Figs. 17 and 18 for LRB1. The time-history results varied from the design procedure predictions by a lesser degree than did the response spectrum analysis results. The range of ratios for this system is around 90 % of the design value. The shear increases and equivalent viscous damping decreases as the period of the hysteresis systems increases. Fig. 18 presents the ratios based on the maximum values from the earthquake history compared to the design procedure values for lead rubber bearing case. The results are relatively insensitive to the period of the structure above the isolators. The mean time-history results demonstrate that the design procedure generally provides a conservative estimate of isolation system performance except for the elastic isolation system, where the design procedure underestimates the displacement and shear force, especially for short-period isolation systems.

## 10. Assessment of peak floor acceleration

Reducing earthquake damage by seismic isolation includes not only the structural system but also non-structural items, such as building parts, components, and contents. Lessening of peak floor acceleration (PFA) is of particular importance in reducing non-structural damage. The PFA in response spectrum analysis and time history analysis have been evaluated in the succeeding section.

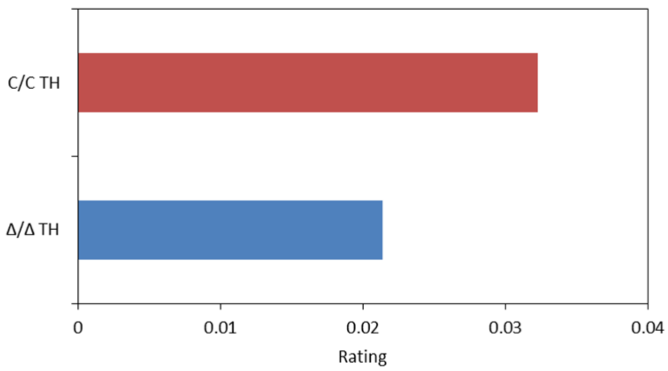



Fig. 18 Time history result for LRB1

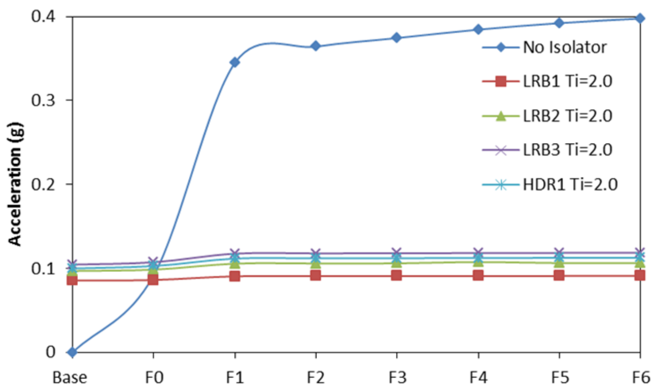



Fig. 19 Response spectrum floor accelerations

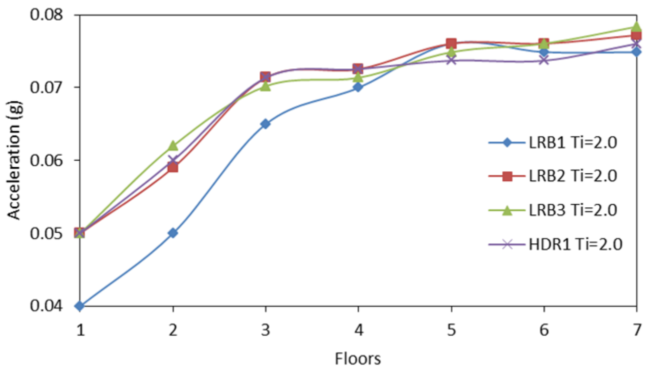



Fig. 20 Time history floor accelerations

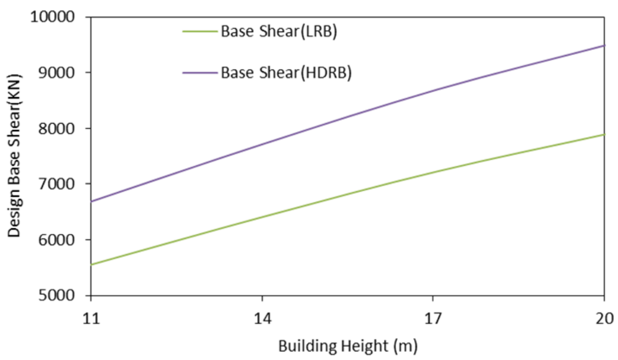



Fig. 21 Design base shear with building height for LRB and HDRB system

## 11. PFA in response spectrum analysis

The floor accelerations from the response spectrum analysis are proportional to the floor inertia forces, as shown in Fig. 19. The accelerations for the building without devices increase approximately linear with height, from a base level equal to the maximum ground acceleration (0.127 *g*) to values from three to four times this value at the roof (0.421 *g*). The isolated displacements in all cases are lower than the 0.114-*g* ground acceleration and remain relatively stable with height.

## 12. PFA in time-history analysis

Plots of the maximum floor accelerations for the building configurations are provided in Fig. 20. These building and isolation system configurations are also used for the inertia forces plotted in Fig. 18. All plots are the maximum values from the time-history analysis and include the accelerations in the building with no isolation as a benchmark. The acceleration increases linearly from the lower value at the base to the upper floors but exhibits values lower than 0.1 *g*. The most discernible feature of the plots is that most isolation systems do not provide the nearly constant floor accelerations developed from the response spectrum analysis in Fig. 19. There are differences between isolation systems, but the trends for each system tend to be similar for each building.

## 13. Design base shear with building elevation

The soft storey building of the plan described in Fig. 1 was analysed for different numbers of stories with an isolator time period Ti of 2.0 and 16% damping for both the LRB and HDRB. The most important parameter, design base shears are evaluated for incorporation of both the RSB in structural base. The comparative data for design base shear are summarized graphically in Fig. 21. With the same damping and isolator period, the LRB system displays a decrease in design forces, i .e., base shear of 20-25% compared with the HDRB system. The rate of change of these forces is high with increasing number of storey. Both the RSB systems are advantageous depending on the site condition and requirement of maximum allowable movement of the isolation devices.

## 14. Conclusions

The investigation demonstrates that the column moment decreases by incorporation of nonlinear rubber-steel bearing model up to 50-55% for the LRB and up to 40-50% for the HDRB. The relative displacement for the LRB is 25-35% greater than that for the HDRB as compared with the fixed tall building structure. Storey displacement increases for the LRB and HDRB by 4.1-4.3 times and 5.2-5.5 times, respectively. The storey drift parameter exhibits peculiar characteristics. At the first storey, the storey drift increases, and for the storey immediately above, it decreases by 74-80% for the LRB and 78-82% for the HDRB. With the same damping and isolator period, the LRB system exhibits a decrease in design forces, i.e., base shear of 20-25% compared with the HDRB system. The rate of change of these forces is high. The floor accelerations from the response spectrum structural analysis are proportional to the floor inertia forces. The accelerations for the tall building structure without devices increase approximately linear with height, from very low (≈0) maximum ground acceleration at the base level to 0.421 g at the roof. The acceleration in all cases is lower than the 0.114-g ground acceleration and remains relatively stable with height. For the time-history analysis, the accelerations in the building structure increase linearly from the base to the upper floor but remain below 0.1 g. For the LRB, the accelerations decrease as the period of the isolator increases.

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