



# Prediction of inelastic response of base-isolated building frame by pushover analysis

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## Abstract

With the advancements in the analysis tools to estimate the inelastic performance of the buildings, the Pushover Analysis (POA) has been adopted as a reliable procedure for the assessment of the nonlinear performance of the buildings. The pushover analysis has a few advantages over the nonlinear dynamic analysis; (i) it is less time-consuming; (ii) it is easy to implement as it does not require to define specific earthquake time-history data. The one of the governing parameters in the seismic response prediction by the POA is the choice of the load pattern. The present study is performed to evaluate the predictions of a new lateral load pattern (LLP) which is developed to perform the pushover analysis of base-isolated buildings. A LLP is developed by modifying the original uniform load pattern; as an example, a 5-storey building frame, which resembles low rise buildings and a 10-storey building, which resembles midrise buildings are selected for the analysis. The buildings are provided with the base isolation layer at the bottom of the buildings with the lead rubber bearings' isolators (LRB). The POA is conducted by selecting three target displacements depicting three behavioral states of the building. The three states are such chosen to depict the structural behavior from elastic to plastic range. For the comparison purpose, the conventional LLP which corresponds to the fundamental mode shape is also used. The predictions of the newly proposed LLP are then compared with the accurate results obtained by conducting the Nonlinear Time History Analysis (NTHA). A suite of real far-field earthquake records is used for performing the NTHA. The study concludes that the new LLP provides better results as compared to the conventional LLP when compared to the benchmark estimates obtained by the NTHA.

**Keywords** Lateral load pattern · Elastic–plastic state · Lead rubber bearing · Elastic state · Far-field earthquakes · Pushover analysis

## Introduction

Base isolation (BI) is one of the effective, widely recognized, and implemented passive control techniques to protect the buildings from the damages caused by the earthquakes (Jangid and Datta 1995; Kelly 1986; Warn and Ryan 2012). Under this technique, horizontally flexible devices are interposed between the superstructure and the foundation, which offers the dual advantage of filtering the damaging high-frequency ground motions, and also creates a designed and a safe location for seismic energy dissipation (Kelly 1986; Warn and Ryan 2012). Thus, the buildings are made more flexible in the horizontal direction, resulting in deliberately

lengthened fundamental vibration time period that results in a reduction in the seismic responses during earthquakes. Moreover, the bearings have the capability to dissipate energy up to a certain limit. As the bearings are manufactured in a controlled factory environment, they are easily replaceable and have large deformation capacities. The base isolation systems as a strategy for earthquake hazard mitigation are globally proven for more than 3 decades. With the development of new base isolation devices, its effectiveness has further increased (Warn and Ryan 2012).

The fundamental difference between a base-isolated and fixed-base building is the behavior of the superstructure in the first mode of vibration. Almost entire deformations are concentrated at the bearing level in the base-isolated building, whereas the superstructure remains almost undeformed as against the cantilever form of deformation of the superstructure in the conventional fixed-base building in the first mode of vibration (Jangid and Datta 1995). The base

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shear and building acceleration are reduced in the building by employing the base isolators in the building as the earthquake energy is absorbed by the isolators. Though the base-isolated buildings are expected to remain in the elastic region at the design-level earthquake but some of the inelastic effects are expected in regions of high seismicity where the design-level earthquake is high (Cardone et al. 2013; Kikuchi et al. 2008; Kilar and Koren 2009; Ordonez et al. 2003). Furthermore, it can go into the significant inelastic state under extreme level earthquakes. The extent of inelastic excursion that BI structure undergoes may depend on the nature and intensity of the earthquake.

There have been considerable efforts which have been put to increase the performance of the buildings for the fixed-base building and also for base-isolated buildings. The research in this area is still going on, and different tools and techniques were developed to access the performance of the buildings. One of the recent research areas is the Performance-Based Design of the building in which the buildings are designed by considering the appropriate performance objectives to meet the desired performance levels (Ghobarah 2001; Krawinkler 1996; Krawinkler and Seneviratna 1998). With this approach, there are lot of performance assessment tools which have been developed by different renowned researchers. The nonlinear static procedure or also named as the pushover analysis (POA) has been proved as an effective tool for the evaluation of the inelastic behavior of the buildings. The POA is simplified procedure to compute the performance of the buildings at a specified target displacement or until building reaches to the collapse limit. It does not require the precise time-history records of earthquakes as required in the case of NTHA (Krawinkler and Seneviratna 1998). The POA generates a relation between load and displacement in terms of pushover curve of the building which gives an idea of the linear and nonlinear displacement capacity of the building. Therefore, a newly designed building or an existing building can be modelled with its member properties and POA can be performed to evaluate its linear as well as nonlinear response (Li et al. 2017; Nakamura et al. 2017).

The POA has gained a lot of recognition in the recent years for its effectiveness. A good quantity of literature can be found related to the research works on the evaluation of POA for fixed-base 2D and 3D buildings (Elnashai 2001; Jan et al. 2004; Kalkan and Kunnath 2004). There have been considerable efforts which have been put in the development of different pushover methods by various researchers (Liu and Kuang 2017; Soleimani et al. 2017; Uva et al. 2018). The most important parameter in the pushover over analysis is the section of lateral load pattern (LLP), which represents an approximate pattern of the inertial forces which are going to act on the building during the event of an earthquake. Thus, the selection of the LLP becomes a crucial aspect in

the process of performing POA (Providakis 2008; Tso and Moghadam 1998).

A few important research works pertaining to the evaluation of seismic evaluation of fixed-base and base-isolated buildings are discussed. Lee et al. (2001) worked out in deriving the mathematical expression to evaluate the action of earthquake forces acting on the base-isolated building. The authors have done idealization of the buildings as a two-degree freedom system and isolation system was defined to be in linear state. A new LLP was developed by integrating the first mode of base-isolated and fixed-base buildings. The numerical study was performed on a 5-storey and 15-storey RCC framed buildings. The predictions obtained by the new LLP were matched with the more accurate results of dynamics analysis and lateral load patterns as suggested by UBC-91 and UBC-97. The results indicated that the proposed pattern closely predicts the storey shear as compared to the other patterns considered in the case of 5-storey building. On the contrary, the proposed distribution underestimates the storey forces for the higher storey level building, i.e., 15-storey. This underestimation was interpreted due to the effect of the higher modes which becomes more active in case of high rise and midrise buildings as the building reaches into nonlinear state.

Doudoumis et al. (2006) performed a comparative evaluation of the prediction of seismic performance considering a 4-storey base-isolated building. The LRB were used as the isolation layer to the building. The predictions of the seismic response parameters were obtained by both POA and NTHA. The seismic responses of the two analyses methods were compared in terms of base shear, formation of plastic hinges, and roof displacement. The NTHA was conducted by employing three real earthquake time-history records. The earthquake records were matched with the long-range periods of the response spectrum as defined in the Greek seismic code. The design spectrum was scaled to two times to get the building into nonlinear range. For performing POA, the uniform load pattern was adopted to represent the seismic lateral forces. The authors concluded that the POA yields close prediction of base shear and corresponding plastic hinges with the results of NTHA.

York and Ryan (2008) suggested simplified equations for estimating the lateral forces in the base-isolated buildings. Several NTHA analyses were performed to obtain the seismic response data set, which was further used in performing the regressing analysis. For the analysis, a three-, six-, and nine-storey single bay superstructure was consider with bi-linear isolation system. For performing NTHA, a suite of 20 earthquake time-history records was employed. Regression analysis is performed to develop equations which relate to superstructure time period  $T_s$  and effective damping,  $\xi$ . The results of the study concluded that the newly developed equations by the authors were

capable of yielding lateral forces with reasonable accuracy as compared to the results of NTHA.

Kilar and Koren (2010) presented an evaluation study to estimate the seismic behavior of a four storey base-isolated building fitted with LRB isolators at bottom of the building. The authors applied the N2 pushover over analysis method to evaluate the seismic performance of the base-isolated building. They have idealized the pushover curve of the base-isolated building as a trilinear curve. The different LLPs were employed for performing POA like, triangular load pattern, a pattern proportional to the fundamental mode of the base-isolated building, and a pattern as suggested by the protective system committee (PSC) (SEAONC 1986). The NTHA is performed by employing a suite of several artificially generated earthquake time histories scaled to three PGA levels like 0.35 g, 0.525 g, and 0.70 g. Finally, the results of the POA and NTHA were compared for different cases and LLPs. It was concluded that the LLP corresponding to the shape of the fundamental mode of the base-isolated building has predicted the close results of storey drift and plastic hinges as compared to other LLPs and NTHA. There are more research studies which were conducted by the same authors for the prediction of the seismic responses for different cases of base-isolated buildings (Kilar et al. 2011; Koren and Kilar 2011). Faal and Poursha (2017) conducted a parametric investigation of evaluation of different POA methods in predicting the responses of base-isolated buildings. The three POA methods which are compared are (i) modal pushover analysis (MPA) (Chopra and Goel 2002); (ii) the N2 pushover procedure (Fajfar and Gaspersic 1996); and (iii) extended N2 procedure (Kreslin and Fajfar 2011). The NTHA was also performed for the comparison purpose at different PGA levels. The study concluded that the close predictions of different seismic responses were estimated by the extended N2 method.

It is observed from the literature survey that a considerable amount of research work has been conducted on the evaluation of seismic responses by the POA in the case of base-isolated buildings. On the contrary, the research work in the direction to evolve more load patterns specific for the response prediction of base-isolated buildings is scanty. With this motivation in view, the present study is conducted to evaluate the efficacy of new LLP which is proposed in this study for carrying out POA. For the reference and comparison purpose, the pattern proportional to the fundamental mode shape is also considered. For the numerical study, 5-storey and 10-storey base-isolated frames are considered. The buildings considered are of reinforced concrete and are base-isolated by LRB bearings. Finally, the results of the two considered patterns are compared with the exact predictions of NTHA to check the efficacy of the new LLP.

## Theoretical background

Although the procedures for POA and NTHA are well established and substantial parts have been presented in the literature, the pushover analysis carried out in this study with a special emphasis on the lateral load distribution patterns is presented for the sake of completeness.

The building is pushed in performing POA by the pre-defined invariant lateral load pattern up to a specified target displacement or until the structure reaches to the ultimate state. The lateral load pattern is the approximation of the lateral forces which are likely to be acted on the structure in the event of an earthquake. There are several lateral load patterns which have been applied in the pushover analysis for the case of the base-isolated structure. The most common LLPs are: (i) uniform force distribution; (ii) triangular profile; (iii) pattern representing the shape of 1st mode; (iv) pattern suggested by the protective systems committee of Structure Engineer Association of Northern California (SEAONC 1986) in which an inverted triangular distribution along the building height is considered with an addition concentrated force at the base as per Eq. 1.

The lateral force  $F_i$  at  $i$ th level of the structure is given by:

$$F_i = \frac{(V_b - F_b)}{R} \frac{w_i h_i}{\sum_{i=0}^n w_i h_i}, \quad (1)$$

$$V_b = K_{eff} D_b, \quad (2)$$

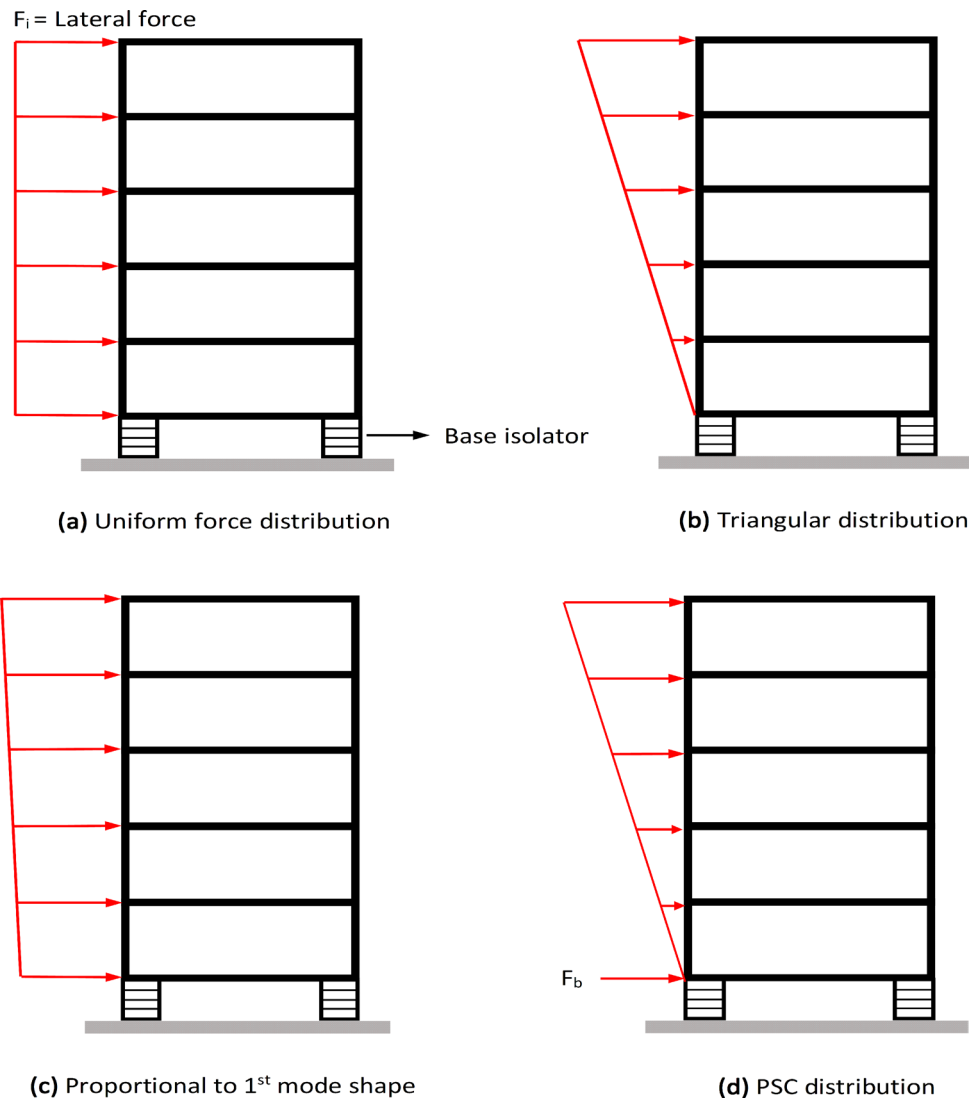
$$F_b = \frac{V_b}{W} W_b, \quad (3)$$

where  $w_i$  represents the weight of  $i$ th the storey level,  $h_i$  is the height of the  $i$ th the storey level,  $V_b$  is the base shear at the isolation level,  $K_{eff}$  equals to the effective stiffness of the isolator,  $D_b$  is the base displacement,  $F_b$  is the concentrated force at bottom level proportional to the base mass,  $W_b$ , and the total seismic weight of the structure is  $W$ . The different lateral patterns are represented in Fig. 1.

## Modelling and designing of building frames

For this study, two reinforced concrete (RCC) building frames were considered for performing analysis. One frame is 5-storey high, which represents the low rise buildings, and the another frame is 10-storey high, which represent the midrise buildings. The configuration and

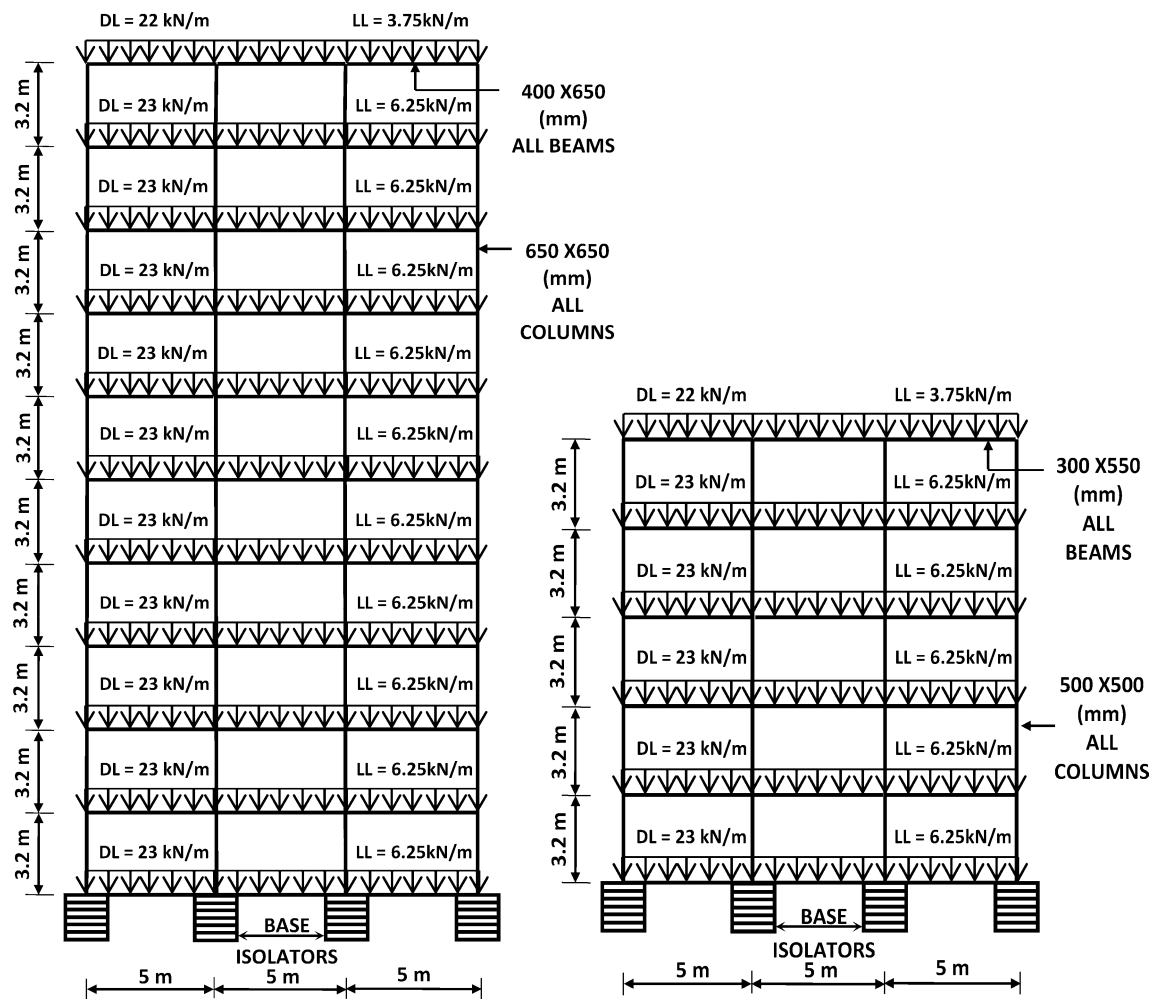
**Fig. 1** Different types of load patterns adopted for performing POA



loadings of the two building frames are depicted in Fig. 2. The modelling of the building frames and LRB isolator is done in the SAP 2000 software. This software is also used to carry out the POA and NTHA. The beams and columns are modelled as line elements in SAP 2000, with user-defined properties. The inelastic behavior in the beams and columns is introduced by introducing the plastic hinges at the ends of the members at a distance  $0.1L$  at one end and  $0.9L$  at other end ( $L$  is defined as the total length of the member). The default hinge properties are defined to the plastic hinges as governed by the FEMA-356 (2000). The default moment–rotation backbone curve for plastic hinges is shown in Fig. 3. The moment hinges are defined in beams, and combined bending and axial force hinges are defined in columns. Three performance levels as per FEMA-356 has been considered: (i) immediate occupancy, (ii) life safety, and (iii) collapse prevention, as shown in Fig. 3.

The frames are designed by adopting the guidelines of Indian standards and following earthquake code, IS-1839 (2016). For the seismic design, 100% dead load and 25% live load are considered. The concrete grade M20 having compressive strength  $40 \text{ N/mm}^2$  and having modulus of elasticity equal to  $31,620 \text{ N/mm}^2$  is adopted. The reinforcement bars of grade Fe 415 having ultimate yield strength of  $415 \text{ N/mm}^2$  are used.

The lead rubber-bearing (LRB) isolators are modelled in the SAP 2000 with the help of link element. The one node of the link element is attached to the ground and second node is attached to the building. The idealized bi-linear characteristic Bouc–Wen model (Wen 1976) is defined for the LRB, as shown in Fig. 4. The LRB isolators are designed as per guidelines given by Datta (2010). The fundamental period of 5-storey frame is 2 s and for 10-storey building frame is 3 s.



(a) 10-story test frame

(Elevation view)

(b) 5-story test frame

(Elevation view)

Fig. 2 Details of the building frames

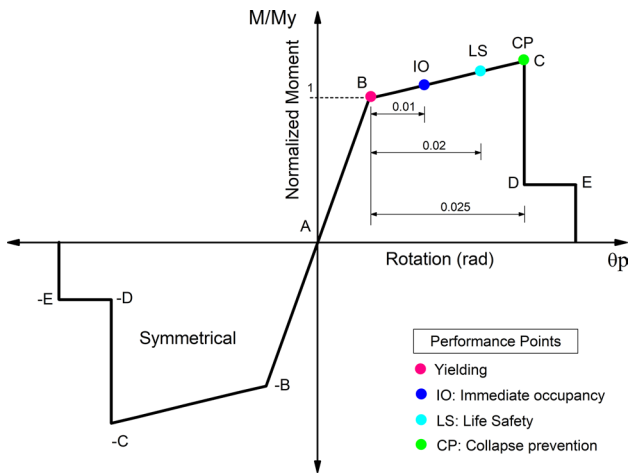
## Numerical study

For the numerical study, 5-storey and 10-storey building frames are selected for the simulations. A new load pattern is developed by the modification of the original uniform load pattern to predict the inelastic responses of the base-isolated building frames. The details of the two load patterns chosen in this study are given below:

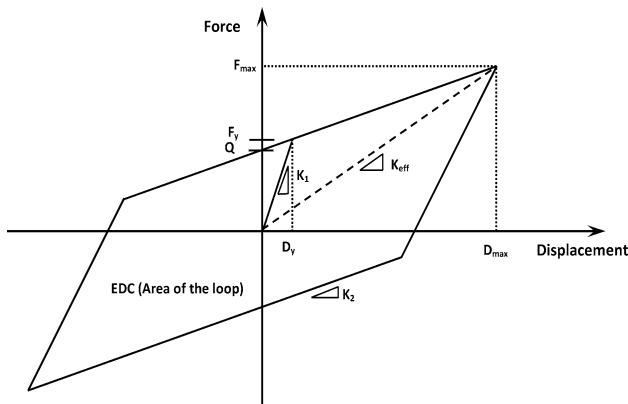
- (i) **LLP-1:** This LLP is corresponding to the pattern of the fundamental mode of the base-isolated building. This is also the conventional load pattern which has been adopted by the most of the researchers. Due to the fact that in the base-isolated buildings, most of the modal mass  $> 90\%$  is excited in the first mode. Therefore, this load pattern representing the pattern

of the fundamental mode is most appropriate for the analysis of base-isolated buildings (Doudoumis et al. 2006; Kilar and Koren 2010). Hence, it is selected LLP to compare the results of the new LLP.

- (ii) **LPP-2:** This is a LLP which is proposed in the present study. It is derived from the basic uniform load pattern which is having uniform distribution of the lateral forces over the building height. The LLP 2 is derived by assuming the uniform 50% of the load at the top half of the building and 100% uniform lateral load is defined at the bottom half of the building. The motivation of this pattern is due to the fact that in the event of an earthquake, when the building goes into the nonlinear range, the load from the upper part of the building is transferred to the lower part of the building due to the formation of plastic hinges.



**Fig. 3** Default moment–rotation curve for plastic hinges as per FEMA-356



**Fig. 4** The force–displacement curve for LRB isolator

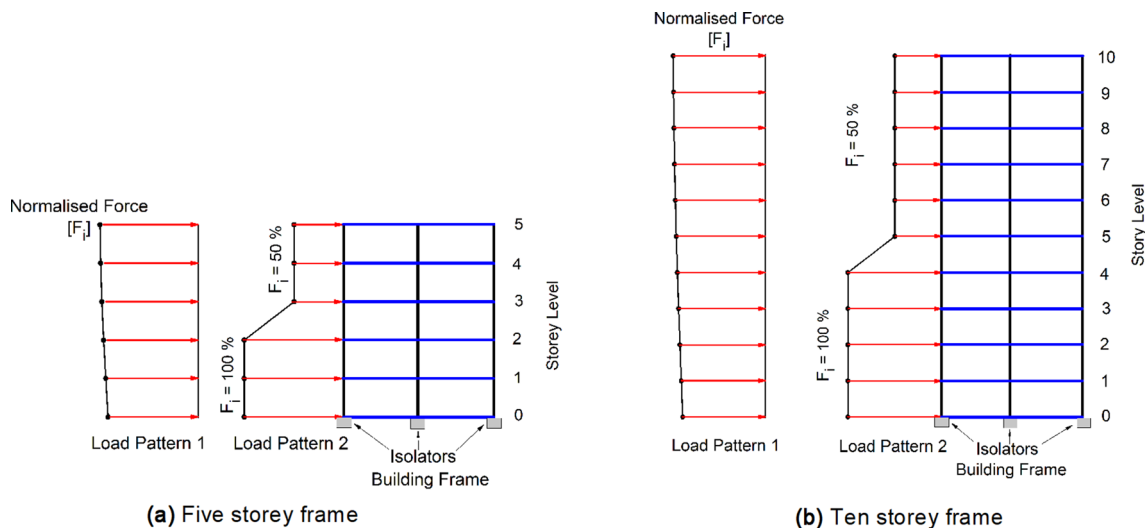
Note that the pattern of this LLP is intuitive and it is subjected to the verification the results. Figure 5 represents the pictorial representation of the LLPs.

For finding the efficacy of the LLP, the POA is performed at three assumed target displacements (TD-1 (elastic), TD-2 (elastic–plastic), and TD-3 (plastic state). The three building states as mentioned above are assumed to fall into three different states of the building as identified on the pushover curve.

The seismic responses like storey drift ratio, inter-storey drift ratio, isolator displacement, base shear, and plastic rotations have been considered as the response parameters for the analysis. The POA is conducted on both building frames by considering the two above-mentioned LLPs. The NTHA is conducted by considering a suite of five ground motions records of real earthquakes, as shown in Table 1. The earthquake ground motion records are obtained from the ground motion database of Pacific Earthquake Engineering Research Center (<https://ngawest2.berkeley.edu/>). The Peak ground acceleration (PGA) levels of each earthquake record is first normalized and then scaled to a value which will match the roof displacement value to a particular value of target displacement. This scaling procedure is also adopted in the study of (Kalkan and Kunnath (2007)).

### Results and discussion

The results of the POA is obtained in terms of capacity curves, as shown in Fig. 6. For the two buildings, the trend and the difference in the capacity curves are observed to be same at the TD-1. On the other hand, as the capacity curves



**Fig. 5** Representation of lateral load patterns selected to perform POA

progress towards the TD-2, the difference in the curves increases and this difference becomes maximum when the TD-3 is reached. The three target displacements are manually marked on the capacity curves in such a manner that they will represent three different states of the building like, elastic state (initial straight portion of the curve), elastic–plastic state (when capacity curve starts its curve path and the formation of hinges starts), and perfectly plastic state (when the capacity curve becomes straight). At the TD-3, the

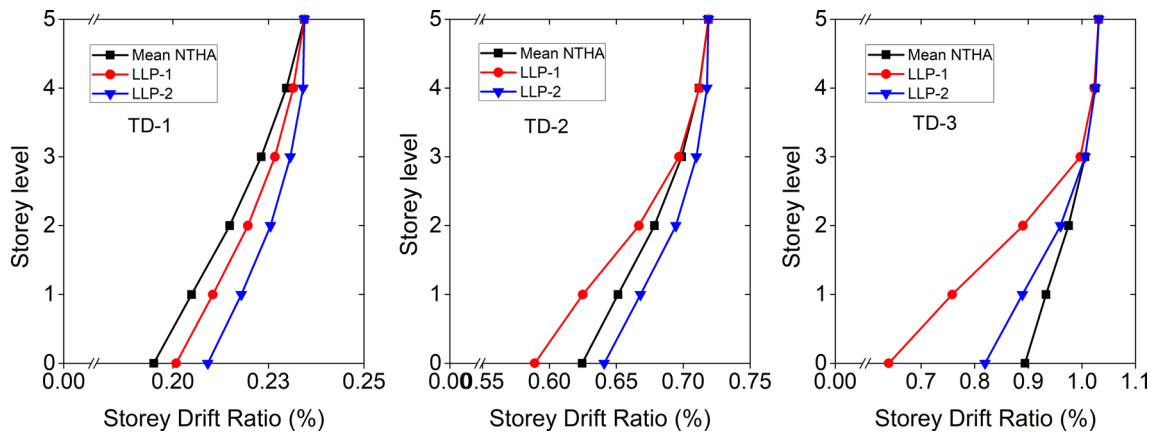
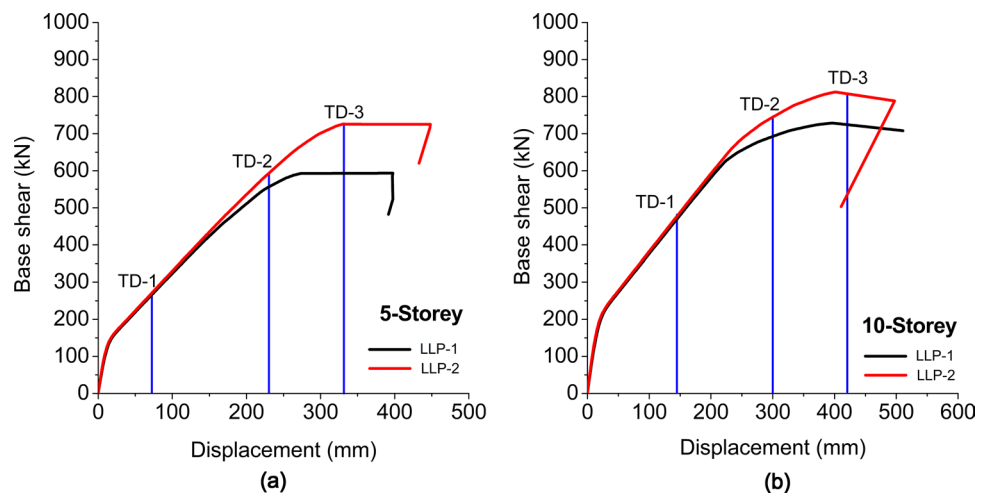
structure goes into highly inelastic state by forming large number of plastic hinges. For the 5-storey frame, the TD-1 is at 75 mm, TD-2 is at 230 mm, and TD-3 is at 330 mm. For the 10-storey frame, the TD-1 is at 144 mm, TD-2 is at 300 mm, and TD-3 is at 420 mm, as shown in Fig. 6.

The peak storey displacement profiles of the frames are represented in terms of the storey drift ratio (SDR), which is defined as the maximum storey displacement normalized by the building height. Figures 7, 8 show the comparison

**Table 1** Characteristics of far-field ground motions

Sr. No	Earthquake	Magnitude $M_w$	Recording station	Recording component	PGA (g)	PGV (cm/s)	PGD (cm)
1	Tabas (1978)	7.4	Ferdows	L	0.093	5.4	2.24
2	San Fernando (1971)	6.6	LA Hollywood stor	090	0.21	18.87	12.42
3	Kobe (1995)	6.9	Nishi-Akashi	000	0.51	37.28	9.53
4	Landers (1992)	7.3	Cool water	TR	0.42	42.35	13.84
5	Superstition hill (1987)	6.5	Poe road	270	0.45	35.72	8.81

**Fig. 6** Capacity curves of two frames: **a** 5-storey; **b** 10-storey



**Fig. 7** Variation of storey drift ratio along the height for 5-storey building frame

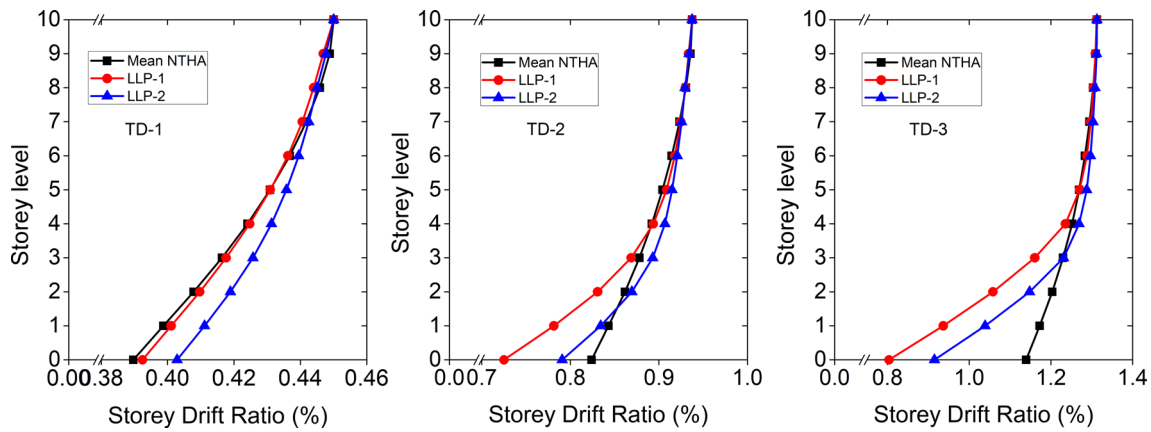


Fig. 8 Variation of storey drift ratio along the height for 10-storey building frame

of SDR along the height of 5-storey and 10-storey building frames as predicted by the two LLPs and mean NTHA. The trend in the results shows that the SDR values are very less and close to the mean NTHA values at TD-1 for both building frames. The same trend is also seen at TD-2, but the LLP-2 slightly overestimates the SDR values. On the contrary, there is large deviation from the mean NTHA values for both LLPs at TD-3, when the building goes into fully inelastic range. It is also worth noting that the LLP-1 underestimates the SDR response at TD-3 when compared to the exact mean NTHA values. The LLP-2 turns to give more close and conservative predictions as compared to the mean NTHA results.

The variation of the inter-storey ratio for the two building frames estimated by the two LLPs and mean NTHA are shown in the Figs. 9 and 10. For the TD-1, the IDR values are underestimated by the both LLPs when compared to the mean NTHA values. Furthermore, the underestimation is very less and is limited at TD-1 only. For TD-2 and

TD-3, both the LLPs overestimates IDR values and there is a large underestimation at TD-3. This is due to the fact that, at TD-3, the building goes into highly nonlinear range and it becomes difficult for the LLP to mimic the pattern of forces which are going to act at this state resulting in the difference of results from mean NTHA values. It is worth noting that by considering all the deviations, the LLP-2 predicts the closest results. This make the LLP-2 a good predictor than LLP-1.

Figure 11 shows the estimation of the base shear demands for the both building frames as estimated by the two LLPs and mean NTHA. It is evident from the figure that there is very less difference in the values of base shear as predicted by the both LLPs at TD-1 and TD-2 as compared to the mean NTHA values for both 5-storey and 10-storey building frames. This shows that both the LLPs are good enough to predict the base shear in the elastic range and elastic plastic range of the buildings. At TD-3, when there is considerable damage in the building, there is large difference in the values of base shear as estimated by the two LLPs as compared to

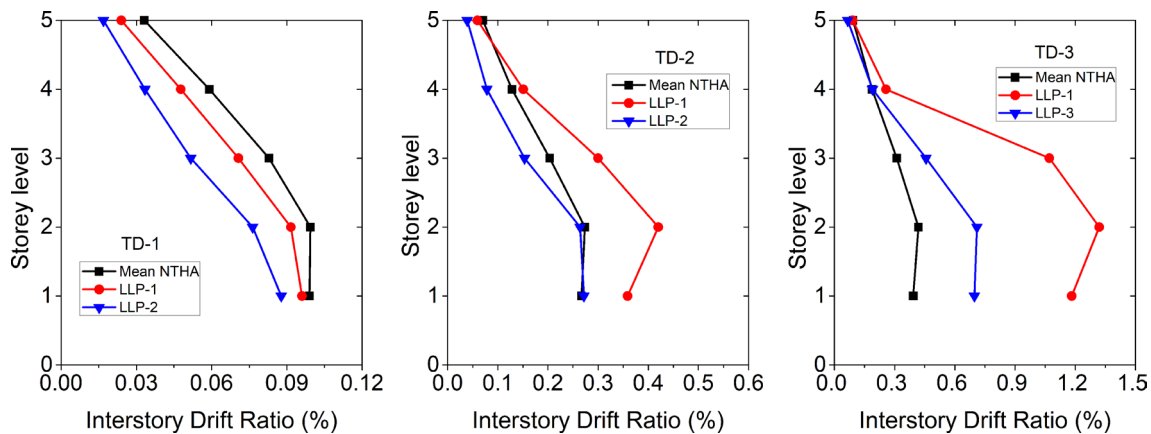


Fig. 9 Variation of inter-storey drift ratio along the height for 5-storey building frame



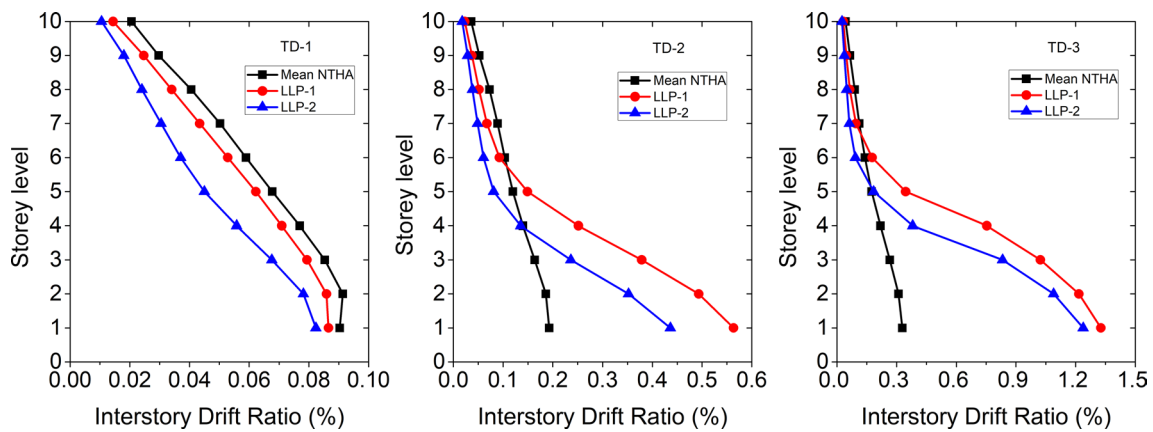


Fig. 10 Variation of inter-storey drift ratio along the height for 10-storey building frame

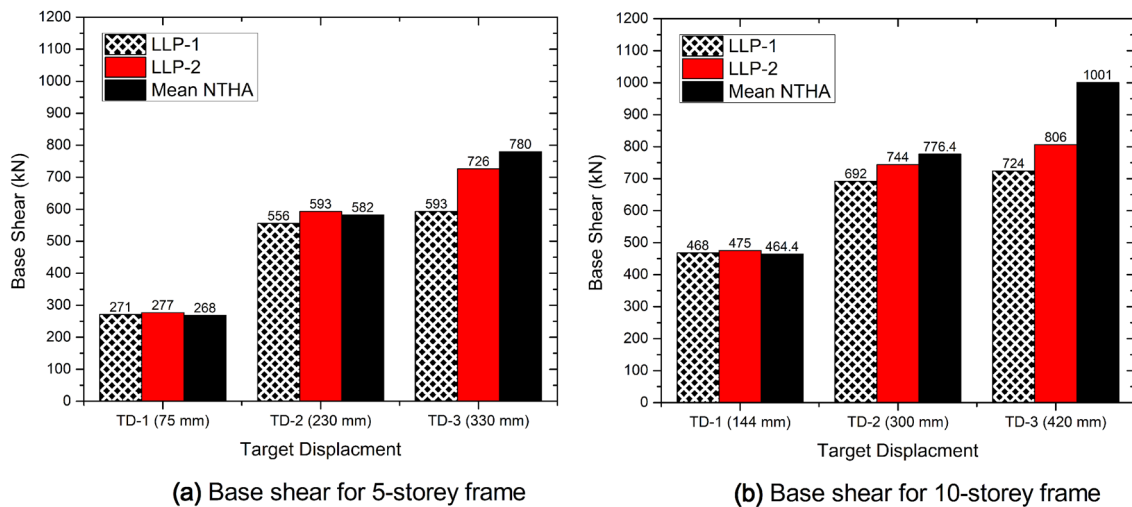


Fig. 11 Estimation of base shear by two considered LLPs

mean NTHA values. By observing both the building frames and at all target displacements, it is noted that the LLP-2 gives the closest prediction to the mean NTHA values, which proves LLP-2 a good predictor of responses.

Figure 12 shows the maximum values of the plastic hinge rotations estimated by the two LLPs for both building frames. It is noted that the values of only TD-2 and TD-3 are given as TD-1 falls in the elastic states, where no hinges are formed. It is observed from the figure that the plastic rotation values differ by significant amount in both buildings. The LLP-2 has estimated near values to the mean NTHA results. For the reference, the plastic hinge states for the 10-storey building frame are shown in Figs. 13 and 14. Also the number of plastic hinges is provided in the Table 2.

Figure 15 shows the prediction of isolator displacement in the two building frames as estimated by the two considered LLPs. It is observed from the figure that there is very less

difference in the values of both LLPs as compared to the mean NTHA values. On the contrary, there is a significant difference in the values, especially for the 10-storey frame. It is worth noting that at every target displacement, the LLP-2 provides the nearer estimation to mean NTHA values.

### Conclusions

The pushover analysis is conducted to evaluate the performance of two buildings frames considering 5-storey and 10-storey frames as a test examples. The building frames are base-isolated by lead rubber-bearing isolators. For the pushover analysis, a new lateral load pattern is derived from modification of original uniform pattern to predict the seismic responses of the frames. For the reference, the load pattern proportional to the first

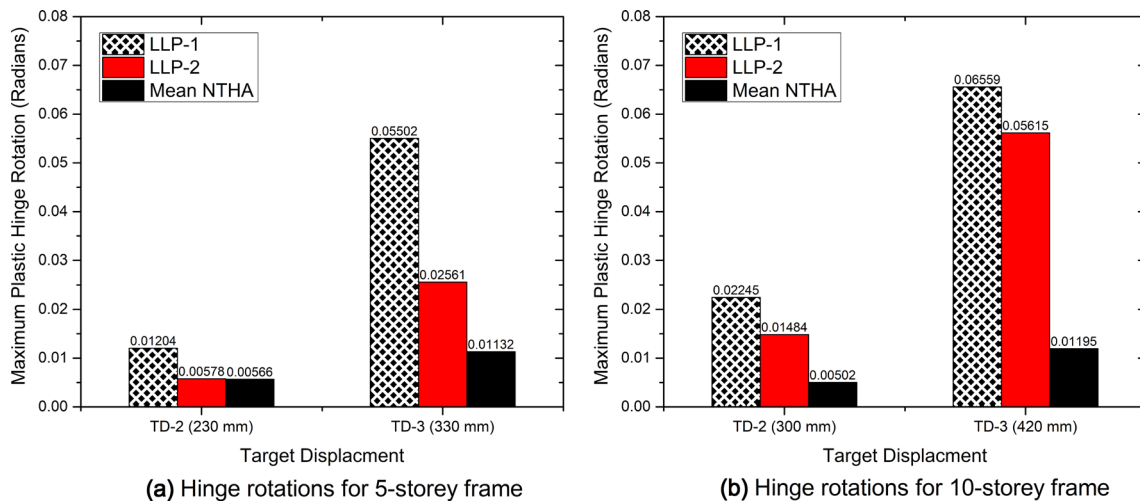
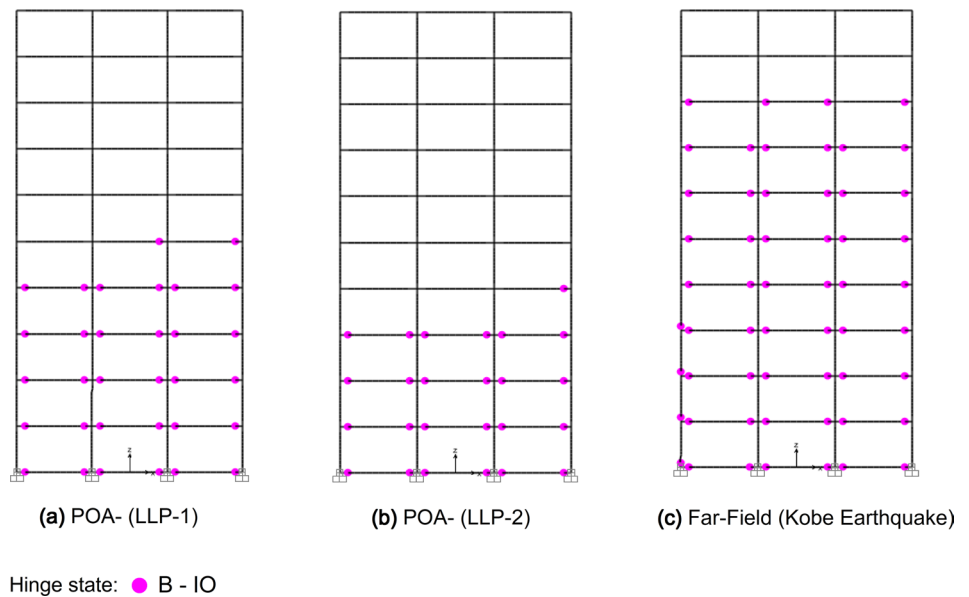


Fig. 12 Maximum plastic hinge rotations as estimated by the two considered LLPs

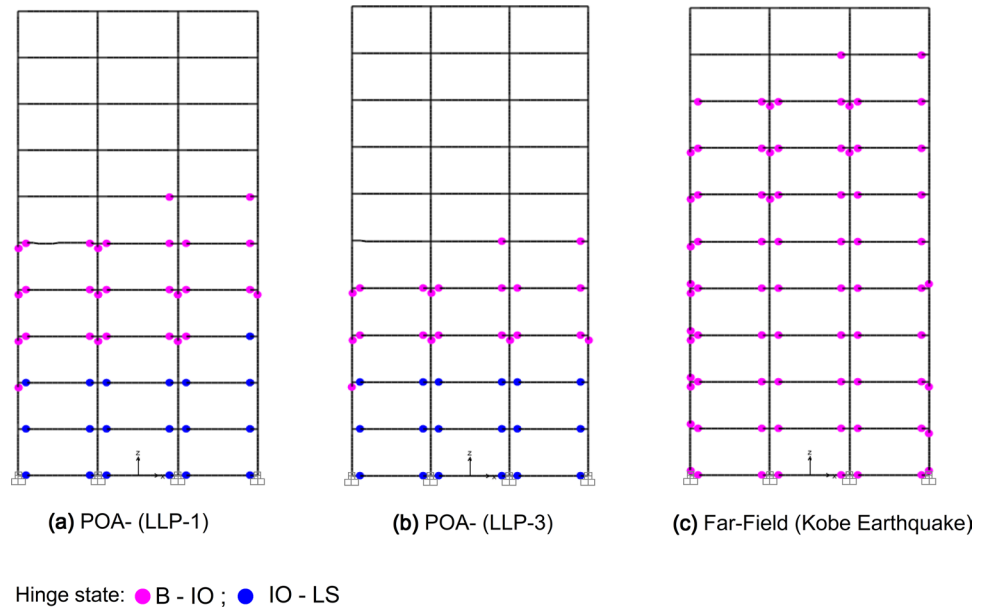
Fig. 13 Formation of plastic hinge states and pattern for 10-storey building at TD-2



mode is also used for the comparison. The efficacy of the new pattern is checked by comparing its results by mean NTHA results and to that of fundamental mode shape load pattern. The different seismic response parameters which are selected for the comparison are storey drift, inter-storey drift, base shear, isolator displacement, and plastic hinges. The important conclusion incurred from the analysis of specific buildings frames and parameters taken are appended below:

1. The new lateral load pattern (LLP-2) which is developed in the study provides a better estimation of the seismic responses as compared to conventionally used pattern for base-isolated buildings, i.e., pattern corresponding to first mode.
2. At the elastic state (TD-1), the LLP-2 estimates the seismic response of 5-storey and 10-storey buildings frame with high accuracy.

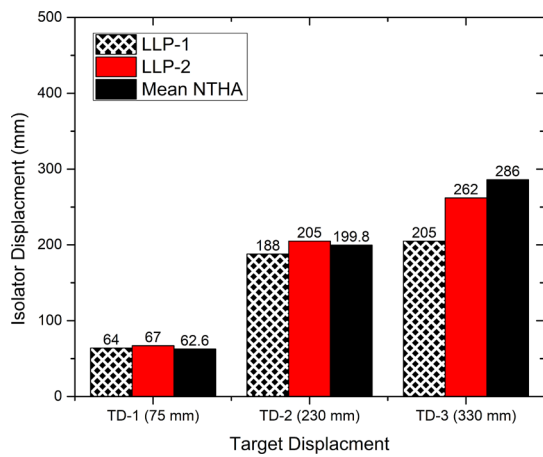
**Fig. 14** Formation of plastic hinge states and pattern for 10-storey building at TD-3



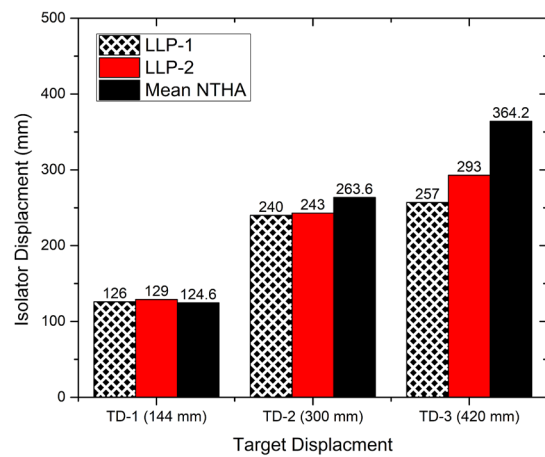
**Table 2** Comparison of number of plastic hinges formed by different LLPs and mean NTHA

Analysis type	Number of plastic hinges		
	TD-1	TD-2	TD-3
5-Storey building frame			
POA (LLP-1)	0	21	30
POA (LLP-2)	0	13	28
Mean NTHA (Far-field)	12	21	30
10-Storey building frame			
POA (LLP-1)	0	32	48
POA (LLP-2)	0	25	39
Mean NTHA (Far-field)	22	49	60

3. The accuracy of the LLP-2 decreases to estimate the inelastic effect in the building frames as the structure goes more into inelastic states.
4. The capacity curves generated by the two considered LLPs are the same for the elastic states of the building and as the building goes into the inelastic states, the difference between them increases significantly.
5. For the specific building frames and parameters considered in this study, it is found that the LLP-2 turns to be the better load pattern to estimate the seismic response of the base-isolated frames by pushover analysis.



**(a)** Isolator displacement for 5-storey frame



**(b)** Isolator displacement for 10-storey frame

**Fig. 15** Estimation of isolator displacement by the two LLPs

## Compliance with ethical standards

**Conflict of interest** On behalf of all authors, the corresponding author states that there is no conflict of interest.

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