



Evaluation of seismic performance of steel frames with stiffness irregularity subjected to mainshock–aftershock using equivalent single-degree-of-freedom system

Samyak D. Parekar¹ · Debarati Datta¹

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Abstract

Many buildings in the present-day scenario have irregularities due to the discontinuity of their mass, stiffness, and geometry through their height. Buildings with vertical irregularities suffer severe damage as observed in past records. Structures situated in earthquake-prone regions are subjected to multiple earthquakes, occurring at a short interval of time, due to the mainshock–aftershock (MS–AS) sequence. This study investigates the effects of stiffness irregularities on the seismic performance of the 9-storey steel moment-resisting frame (SMRF) subjected to mainshocks and MS–AS. Nonlinear time-history analysis (NTHA) was conducted to compare the height-wise variation of maximum inter-storey drift ratio (IDR) demands of the regular and stiffness irregular frames. The results show that stiffness irregularity through the height of the frame influences the height-wise variation of inter-storey drift. Also, the structural responses of regular and irregular frames subjected to MS–AS are increased compared to mainshocks. This study also evaluates the accuracy of previously proposed three different equivalent single-degree-of-freedom systems (ESDOF) for obtaining seismic demands of multi-degree-of-freedom (MDOF) systems of regular and irregular frames subjected to mainshocks and MS–AS. The mean values of the height-wise variation of IDR obtained from MDOF and ESDOF systems are compared. The effectiveness of ESDOF systems for estimation of IDR is examined by mean bias and standard deviation of the ratio of mean values of IDR determined by NTHA of MDOF systems to the values determined by ESDOF systems. The study concludes ESDOF systems are useful for estimating the height-wise variation of IDR for regular and stiffness irregular frames subjected to mainshocks and MS–AS.

Keywords Stiffness irregularity · Equivalent single-degree-of-freedom system · Mainshock–aftershock · Pushover analysis · Nonlinear time-history analysis

Introduction

An ideal multi-storey building designed to resist lateral loads in seismically active regions would consist of uniform distribution of mass, stiffness, and strength through the height of the building. However, it is very difficult to achieve such conditions because buildings with vertical irregularities constitute a large portion of modern urban infrastructure due to different usages of the specific storey with respect

to the adjacent storey. Many vertical irregular buildings have suffered severe and unexpected damage during past earthquakes. The most common irregularity that will be considered for this study is based on the stiffness that can be modified for floors along with the building height. In seismic sequence, the largest event having medium–high level intensity is called mainshock. The events which come before the mainshock are called foreshocks, and events after the mainshock are called aftershocks having comparable and smaller intensity than the mainshock [30]. In this study, the seismic sequence that contains the mainshock and aftershocks has been considered for analysing the seismic response of the SMRF building. Aftershocks may cause severe damage to the structure because any rehabilitation action between these seismic sequences cannot be complete because the time interval between mainshock and aftershocks is very less. This study aims to investigate the effects of stiffness

✉ Samyak D. Parekar
samyakparekar@gmail.com; samyak854@students.vnit.ac.in
Debarati Datta
debaratidatta@apm.vnit.ac.in

¹ Applied Mechanics Department, Visvesvaraya National Institute of Technology, Nagpur, Maharashtra, India

irregularity by comparing the height-wise variation of IDR of 9-storey SMRF regular and stiffness irregular frames subjected to mainshocks and MS–AS.

There is a lot of research work being conducted by researchers to investigate the seismic performance of vertically irregular buildings. Valmundsson and Nau [37] conducted a parametric study on seismic performance of 5, 10, and 20-storey frame with mass, stiffness, and strength irregularity. They concluded that the strength irregularity increases the response quantities compared to mass and stiffness irregularities. Al-Ali and Krawinkler [1] evaluated the effects of different types of vertical irregularities on the seismic response of a 10-storey frame. They found that the effect of combined stiffness and strength irregularity is the largest on roof displacement as compared to the mass, stiffness, and strength irregularity. Chintanapakdee and Chopra [5] compare the seismic response of regular and vertically irregular 12-storey single-bay frames and investigates the accuracy of modal pushover analysis (MPA) for estimation of seismic responses of regular and vertically irregular frames. They found that the effects of stiffness, strength, and combined stiffness and strength irregularities on the distribution of inter-storey drift through the height of the frames are similar; however, the effect is largest in the frames with combined stiffness and strength irregularity. Michalis et al. [21] studied the effect of vertical irregularity on the capacity of a 9-storey SMRF building using incremental dynamic analysis (IDA). They concluded that the effect of irregularities on the limit states is significant and depends on the intensity of earthquakes. Varadharajan et al. [38] summarized the research work regarding the plan and vertical structural irregularities. Pirizadeh and Shakib [26] used a probabilistic approach to investigate the seismic performance of vertical irregular structures and compared the various performance levels with the irregular structure. Nazri et al. [23] studied the effect of vertical irregularity on the seismic response of three-moment-resisting steel and RC frames using fragility curves. Azghandi et al. [3] studied the collapse mechanism of mass and stiffness irregular high-rise steel moment-resisting frames using a finite element model. Their results showed that the collapse resistance is adversely affected due to vertical irregularities.

A variety of research studies have taken into account the aftershock effect to investigate the structural responses. Some studies like Aschheim and Black [2] and Mahin [20] focused on the effects of MS–AS on SDOF systems. Also, some studies concentrated on inelastic response spectra for MS–AS seismic sequence. Zhai et al. [40] investigated the damage spectra considering the Park–Ang damage index as a damage parameter which accounts for the contribution of maximum displacement and hysteretic energy for the MS–AS seismic sequences. They concluded the effect of seismic sequence on damage spectra has a significant

nearly entire period region. Zhai et al. [41] investigated the response of the inelastic SDOF system taking into account three damage parameters as normalized hysteretic energy, ductility demand, and modified Park–Ang damage index with four different hysteretic models subjected to seismic sequence considering different relative intensities of aftershocks to mainshocks. Zhang et al. [42] investigated the damage-based strength reduction factor of the SDOF system under the MS–AS seismic sequences. They concluded the strength reduction factor due to the effect of aftershock increases with the increase in the intensity of aftershock. The seismic response of the MDOF system under MS–AS has been examined [7, 13, 16, 17, 19, 28, 29, 31]. Fragiaco et al. [7] performed nonlinear analysis on two low-rise steel frames and SDOF systems under four different seismic sequences characterized by the repetition of one, two, and three ground motions. Lee and Foutch [16] examined the performance of damaged two SMRF buildings subjected to ground motions having different hazard levels. They found the first and second applications of similar earthquake ground motions result in no significant damage that occurred in the second application of earthquake ground motion. Li and Ellingwood [17] investigated the seismic performance of the SMRF buildings by obtaining the randomized artificial aftershock ground motions ensembles. They determined the scale factors for simulating aftershock ground motions using the method used by Sunasaka and Kiremidjian [36]. Two extreme cases one is replicate (earthquake ground motion in the mainshock are simply repeated with a scale factor) and second is randomized with respect to mainshock were studied. They found that in the replicate case the original damage pattern remains the same as a result of aftershocks while in the case of the randomized case it is depending on the characteristics of aftershocks. Hatzigeorgiou et al. [13] studied the seismic response of regular and vertically irregular reinforced concrete frames under the forty-five MS–AS seismic sequences. They used incremental dynamic analysis (IDA) for the estimation of structural responses and concluded there is a considerable difference in the seismic sequences compared to single seismic events. Loulelis et al. [19] evaluated the seismic behaviour of SMRF using IDA under sequential ground motions and concluded that the seismic sequences lead to different responses than the corresponding to single seismic events. Ribeiro et al. [29] proposed a methodology for the robustness assessment of SMRF buildings under MS–AS using a reliability approach. Back to back IDA was performed for a combination of mainshock and aftershocks on three SMRF buildings and evaluates the probability of failure. They observed that there is an increase in the probability of failure under MS–AS compared to single events. Raghunandan et al. [28] quantified the aftershock collapse vulnerability of four reinforced concrete frame structures using IDA. They found that the collapse

capacity of RC frame structures subjected to MS–AS seismic sequence is reduced in aftershocks if the structural damage in mainshocks is significant. Ruiz-Garcia and Aguilar [31] examined the effect of modelling assumptions in the seismic response of 4-storey steel frame subjected to the aftershock of different hazard levels.

During the earthquake ground motions, structures behave inelastically, therefore nonlinear analysis is required to evaluate the behaviour of structures under seismic loading. However, the seismic behaviour of MDOF structures is carried out by sophisticated NTHA which becomes computationally heavy. Besides modelling the actual MDOF structure, it can be represented through simple ESDOF systems. The ESDOF system which is derived from modal and pushover analysis is useful for the determination of the seismic response of real MDOF structures. Chopra and Goel [6] developed an MPA procedure for estimation of seismic response which incorporates the contribution of higher modes of vibration. The seismic responses determined by this method gives good results in terms of inter-storey drifts and floor displacements. Chintanapakdee and Chopra [4] evaluated the seismic response of a wide range of steel frames and check the accuracy of MPA. Chintanapakdee and Chopra [5] evaluated the accuracy of the MPA procedure for 10-storey single-bay irregular frames. Han and Chopra [12] explored the MPA procedure in estimating the IDA curves and limit-state capacities of three SMRF buildings. Results showed that the accuracy of the approximate method is satisfactory for estimating structural capacities at different limit states. Ghosh and Collins [8] developed a probabilistic design methodology using hysteretic energy demand. Prasanth et al. [27] proposed equivalent systems by using the concept of the MPA procedure to evaluate the hysteretic energy demands of SMRF buildings. Ghosh et al. [9] proposed three ESDOF systems for evaluation of the Park–Ang damage index of SMRF buildings under various ground motions. Kashkooli and Benan [15] investigated the accuracy of the MPA procedure of the setback irregular frame in height for the estimation of target displacement, storey drift, and base shear. Lin et al. [18] developed a 2DOF system based on an approximate method for the estimation of seismic response of setback building.

It should be noted that from previous studies the effect of MS–AS seismic sequence was investigated for regular building structures only and very few researches investigated the effect of MS–AS seismic sequence on irregular structures. Oyguc et al. [24] investigated 3-dimensional three irregulars reinforced concrete buildings under Tohoku seismic sequence. Nazri et al. [22] evaluated probabilistically the effect of irregular adjacent reinforced concrete frames subjected to the repeated seismic sequence. Parekar and Datta [25] studied the effect of stiffness irregularity on the seismic response of steel moment-resisting frames subjected to artificial MS–AS seismic sequence. Therefore, there is

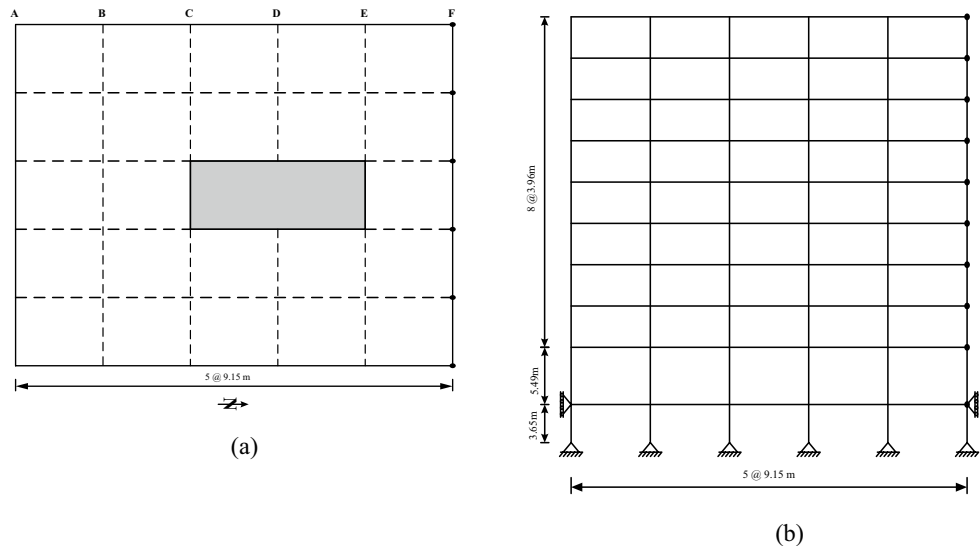
still a need for investigating the seismic behaviour of vertically irregular steel frames subjected to MS–AS seismic sequence. The first objective of this study is to investigate the effect of stiffness irregularities on the seismic performance of 9-storey SMRF building. The 9-storey steel frame is considered a reference regular frame. The modification factor of 2 is for increased stiffness and 0.5 for reduced stiffness of the reference regular frame. The Stiffness irregularities at three different locations along the height of the frame were considered. The randomized approach is used for generating artificial MS–AS seismic sequences. The comparison of seismic performance between reference and stiffness irregular frames was investigated by comparing the height-wise variation of mean inter-storey drift ratio under mainshock and MS–AS. The study also expands to evaluate the accuracy of the previously proposed three ESDOF systems for estimation of height-wise variation of IDR of stiffness irregular and reference frames subjected to mainshocks and MS–AS. The comparison of mean IDR along the height of the frame obtained from MDOF and ESDOF systems subjected to mainshock and MS–AS is presented and analysed. The accuracy of ESDOF systems in estimating IDR is obtained by representing the bias and standard deviation of the ratio of the seismic demands on irregular frames determined by exact values to the values determined by ESDOF systems computed by NTHA.

Structural models

Reference regular building frame

The 9-storey SMRF building studied in this work was designed for the SAC steel project [33]. This building with a symmetric plan as shown in Fig. 1 and the primary lateral load resistance is provided by special moment-resisting frames around the perimeter of the building. The interior system is designed to resist the gravity load only. Figure 1 shows the plan view (showing location of moment-resisting frame with dark lines) and elevation of 9-storey reference regular frame. This building was designed as a standard office building as a part of the SAC project prevailing seismic code requirement as per UBC 1994 situated on stiff soil for Los Angeles. The details of this building frame were described in the report [10]. The 2-dimensional analytical model represents each building that was prepared using a computer software SAP2000 [34]. This software makes use of finite element analysis for analysing the structure. The basic 2-dimensional centreline model (Model M1 of Gupta and Krawinkler 1998) of bare moment-resisting was modelled without considering the effect of slab contribution, interior gravity frames, and the strength of the panel zone. The frame elements

Fig. 1 Floor plan and elevation of 9-storey reference regular building



(beams and columns) were modelled using 2 noded beam elements with 6 degrees of freedom at each node. The rigid diaphragm was provided to mimic the effect of a slab while the corresponding slab loads were transferred to the frames. The frame element with concentrated plastic hinges assigned to beam or column ends were modelled by bilinear moment rotation relationship with a constant 3% strain hardening and for columns M–P interaction column with the bending strength defined by a bilinear M–P interaction diagram. The natural periods of the first three modes of the regular frame were 2.406 s, 0.90 s, and 0.52 s, respectively, and it was a good match with the building periods available in the literature [39]. The nonlinear pushover analysis is conducted corresponding to elastic modes of vibration of the MDOF system and considering IBC 2006 [14] lateral load. The nonlinear time-history analysis is conducted using SAP2000 based on Newton–Raphson direct integration method. The time steps of considered earthquake ground motions are 0.02, 0.01, and 0.005 s. A time step of 0.005 is considered to produce negligible errors. The NTHA is performed on MDOF and ESDOF systems of regular and stiffness irregular frames.

Stiffness irregular frames

The vertical irregularities of the structure are defined as the structures with irregular distribution of different properties like mass, strength, and stiffness along with the height of the structure. The vertical irregularities considered in this study are based on stiffness which is obtained by changing the stiffness at the bottom, mid-height, and top storey of the 9-storey reference regular frame. The stiffness of storey is modified by upgrading or degrading the stiffness of storey member by a single modification factor of 2. For obtaining a soft or stiff storey, the storey stiffness was divided or multiplied by a modification factor of 2, respectively. Following six stiffness irregularity cases were investigated: (1) Soft first storey (KI-0.5–1), (2) Soft mid-height storey (KI-0.5–5), (3) Soft top storey (KI-0.5–9), (4) Stiff first storey (KI-2–1), (5) Stiff mid-height storey (KI-2–5), and (6) Stiff top storey (KI-2–9). The notations used for representing stiffness irregular frames consist of three parts referring to the type, modification factor, and location of irregularity. For example, KI-0.5–1 notations refer to the irregular frame with stiffness irregularity having a modification factor of 0.5 at storey 1. Figure 2 shows the distribution of stiffness over

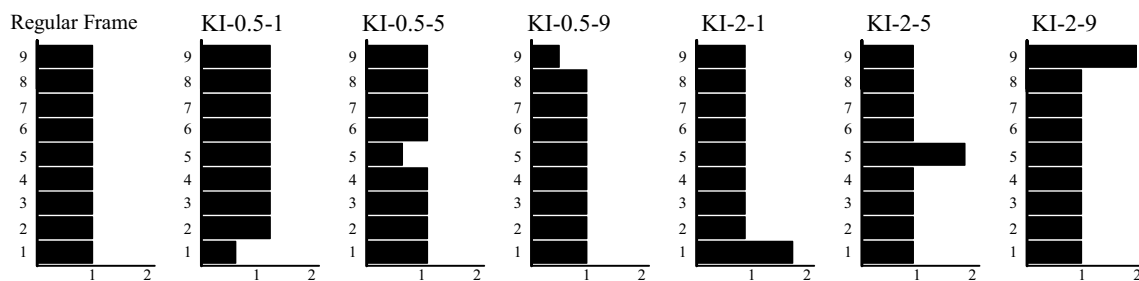


Fig. 2 Ratio of storey stiffness along the height of irregular frames to the storey stiffness of the regular frame

the height of stiffness irregular frames in comparison with the regular frame. The stiffness irregularity at the particular storey was obtained by changing the stiffness of beams and columns in that storey with the modification factor of 2. For a fair comparison between regular and irregular frames, the structural specifications like fundamental time period, yield base shear, and damping properties are kept the same. All the storey stiffnesses of irregular frames were scaled uniformly with the factor for maintaining the same fundamental time period as that of the regular frame. All the storey strength of irregular frames was scaled uniformly with the factor for maintaining the same yield base shear as that of the regular frame.

Equivalent system methodology

The ESDOF system concept of representing an actual MDOF structure is nothing new and it is capable of estimating certain responses of MDOF structure. The main objective of this study is to evaluate the accuracy of previously proposed ESDOF systems for regular and stiffness irregular frames subjected to mainshocks and MS-AS. The basic formulation of the ESDOF systems is given in Ghosh et al. [9]. The parameter of ESDOF systems used in this work is quantified by assuming mode shapes and nonlinear static pushover analysis of MDOF structure.

The development of ESDOF systems begins from the inelastic response of a multi-storey building to horizontal ground motion which is given by differential equation (Eq. 1):

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + \{R\} = -[M]\{1\} \ddot{u}_g \tag{1}$$

where [M] and [C] are the mass and damping matrix of the system, {u} is the vector of lateral displacement, and {R} is a vector of restoring force.

The lateral displacement vector {u} is replaced (Eq. 2) by roof displacement (D) and shape vector {ϕ} which has a value of 1 at roof displacement by normalization.

$$u(t) = \phi D(t). \tag{2}$$

Furthermore, pre-multiplying the differential equation (Eq. 1) with the shape vector, obtained Eq. 3.

$$\{\phi\}^T [M] \{\phi\} \ddot{D} + \{\phi\}^T [C] \{\phi\} \dot{D} + \{\phi\}^T \{R\} = -\{\phi\}^T [M] \{1\} \ddot{u}_g. \tag{3}$$

The shape vector {ϕ} is the natural vibration modes of the structure or obtained by the pushover analysis of the MDOF structure. The pushover curve represents the

variation between base shear (V) with roof displacement (D) obtained from pushover analysis. By using a bilinear approximation of pushover curves [6], the mathematical relationship between V and D can be given by Eq. 4,

$$V = KG(D). \tag{4}$$

K is the elastic stiffness obtained from the bilinear idealization of the pushover curve and G(D) is the mathematical function showing the force-displacement relation of ESDOF systems. Equation 3 can be reduced to Eq. 5,

$$M^* \ddot{D} + C^* \dot{D} + K^* G(D) = -L^* \ddot{u}_g. \tag{5}$$

After dividing by M*, Eq. 5 becomes

$$\ddot{D} + 2\xi(\omega^*)\dot{D} + (\omega^*)^2 G(D) = -P^* \ddot{u}_g, \tag{6}$$

where $M^* = \{\phi\}^T [M] \{\phi\}$, $K^* = K \{\phi\}^T \{f\}$, $C^* = \{\phi\}^T [C] \{\phi\}$, $L^* = \{\phi\}^T [M] \{1\}$, $P^* = L^*/M^*$, $(\omega^*)^2 = K^*/M^*$ and $C^*/M^* = 2\xi(\omega^*)$. Equation 6 is the governing equation of motion of an inelastic SDOF system, an SDOF system with mass M*, damping ratio ξ and linear elastic stiffness K*. {f} is the lateral load distribution vector for pushover analysis. The following three ESDOF systems methodology is selected which is proposed by Ghosh et al. [9].

ES1: This system is considered the fundamental mode instead of taking multiple modes. The mode shape vector is the first mode displacement profile obtained from the eigenvalue analysis of MDOF structures. The normalization of the mode shape is done to have unit roof displacement. The lateral load distribution is obtained for pushover analysis after normalizing to have unit base shear $\{f\} = m\phi/[1]^T m\phi$. Pushover analysis is performed up to 2.5% maximum inter-storey drift as suggested by [27]. ESDOF parameters are obtained by idealizing the pushover curve as a bilinear curve as explained by [6].

ES2: This ESDOF system uses the concept of the MPA procedure that includes the contribution of several modes of vibration. Eigenvalue analysis is performed on the MDOF structure for finding the elastic mode shapes. The pushover analysis is conducted corresponding to elastic modes of vibration of the MDOF system for finding the ESDOF parameters. The ESDOF parameters for 9-storey regular and stiffness irregular frames are calculated corresponding to the first three modes. The total responses are determined by combining modal responses using the square root of sum of square combination rule.

ES3: The pushover analysis is performed considering IBC 2006 [14] lateral load distribution. The shape vector is obtained by performing a pushover analysis of up to 1% global drift.

Description of analysis and ground motions

The earthquake records developed in SAC Steel Project for Los Angeles were used as mainshocks [35]. The 40 earthquake records were mentioned previously, i.e. LA01–LA40. For this study, 20 earthquake records as mainshocks were selected. These 20 mainshocks represent the seismic hazard level of 10% probability of being exceeded in 50 years and have an average magnitude of 6.8. One of the objectives of this investigation is to study the seismic response of 9-storey SMRF regular and stiffness irregular frames subjected to mainshock and MS–AS. For this purpose, the accurate modelling of expected mainshock–aftershock seismic sequences is very important. The MS–AS seismic sequence can be generated by the artificial and as-recorded seismic sequence. Ruiz-Garcia and Negrete-Manriquez [32] recommended to use the recorded MS–AS seismic sequences to simulate MS–AS. However, most of the studies employed the artificial MS–AS seismic sequences for seismic analysis due to the unavailability of enough number of recorded aftershocks at the seismic station [11]. Artificial MS–AS seismic sequences can be generated using the repeated and randomized approach by seeding the mainshock ground motion. In a repeated approach, the mainshock is repeated as an artificial aftershock at scaled or identical amplitude without changing the frequency content and duration of mainshock ground motion [16, 17]. The randomized approach is used to generate artificial MS–AS seismic sequences considering the ground motion features as frequency content and duration of ground motion. The randomized approach consists of an ensemble of a set of mainshocks and the artificial MS–AS seismic sequences generated by selecting mainshocks randomly and generating remaining aftershocks by scaling the ground motion intensity. For this study, the artificial MS–AS seismic sequences are generated by using a randomized approach given by Li and Ellingwood [17]. Li and Ellingwood [17] generated a randomized seismic sequence using the methodology given by Sunasaka and Kiremidjian [36] and find of the scale factor of 0.9 for representing the aftershock of the return period of 308 years (i.e. seismic hazard level of 15% probability of being exceeded in 50 years). In this study, 380 randomized seismic sequences are generated taking 0.9 as a scaling factor for the aftershock. The free vibration time of 20 s is taken between the mainshock and aftershock to ensure the structure comes to rest after the mainshock. The NTHA performed on MDOF and ESDOF systems of regular and stiffness irregular frames by using SAP2000 [34].

Results and discussion

The present study mainly focused on the seismic behaviour of regular and stiffness irregular steel moment-resisting building frames subjected to mainshocks and MS–AS seismic sequences. A total of six stiffness irregularities cases was investigated as the soft first storey, soft mid-height storey, soft top storey, stiff first storey, stiff mid-height storey, and stiff top storey. The comparison between the seismic performance of stiffness irregular and regular frames is done by comparing its mean seismic demands. Generally, the seismic response of multi-storey buildings under given ground motions is indicated by maximum roof displacement. However, the height-wise variation of inter-storey drift is the most preferred response quantity. The inter-storey drift of structure is defined as the relative lateral displacement of two successive storeys. The i^{th} storey peak inter-storey drift ($\Delta_i(t)$) during earthquake ground motion is the peak value of i^{th} inter-storey drift ratio (Δ_{mi}):

$$\Delta_i(t) = \frac{D_i(t) - D_{i-1}(t)}{h_i}, \quad (7)$$

where D_i is the i^{th} storey displacement and h_i is the height of the storey. In this investigation, nonlinear time-history analysis is performed on frames subjected to mainshocks and MS–AS seismic sequences to investigate the seismic behaviour of regular and stiffness irregular frames in terms of mean inter-storey drift demands. The basis for ESDOF systems remains the same for calculating inter-storey drift demand. For finding the i^{th} storey peak inter-storey drift (Δ_{mi}) demand for ES1 and ES3 systems, the maximum roof displacement (D_{m-ES}) is multiplied with a factor based on the mode shapes [9]:

$$\Delta_{mi} = P^* \left(\frac{(\phi_i - \phi_{i-1})}{h_i} \right) D_{m-ES}, \quad (8)$$

where ϕ_i is the mode shape element of the i^{th} storey. The peak inter-storey drift demand of the ES2 system is obtained by combining modal responses using the square root of sum of square combination rule as given in Eq. 9,

$$\Delta_{mi} = \left(\sum_{n=1}^i \left(P_i^* \left(\frac{(\phi_i - \phi_{i-1})}{h_i} \right) D_{m-ES} \right)^2 \right)^{0.5}. \quad (9)$$

The building frame is analysed for 20 mainshocks and each MS–AS seismic sequence consists of one mainshock followed by 19 aftershocks. Thereby, 20 MS–AS sequence consists of 20 mainshocks and 380 aftershocks. The structural responses in terms of maximum inter-storey drift ratio during each mainshock and MS–AS are

obtained from NTHA of ESDOF systems and the MDOF system. The mean values of IDR are the average value of the maximum inter-storey drift ratio obtained subjected to ground motions. The mean values of the distribution of IDR along the height of frames determined by NTHA of ESDOF systems and MDOF system are compared, and the bias statistics is provided in terms of mean bias and standard deviation (SD) gives an overall picture of the level of accuracy for selected ESDOF systems for estimation of IDR. The mean bias close to 1 and minimum SD values shows that ESDOF systems are very accurate for the estimation of seismic responses of the MDOF system. The mean bias values of seismic responses were taken as the ratio of mean values determined by NTHA of MDOF system to the mean values of ESDOF systems.

Basic factor, $N = \frac{D_{-MDOF}}{D_{-ES}}(10)$, where D_{-MDOF} and D_{-ES} are the seismic response based on NTHA of the MDOF system and based on ESDOF systems, respectively. Section 5.1 describes the effect of MS-AS on the 9-storey regular frame

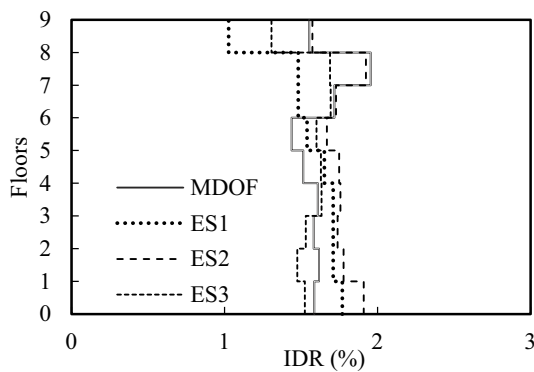


Fig. 3 Distribution of mean IDR along the height of the regular frame determined by ESDOF systems and NTHA of MDOF systems under mainshocks

and ESDOF systems are tested for regular frame subjected to mainshocks and MS-AS. The NTHA of regular and stiffness irregular frames are performed subjected to mainshocks and MS-AS for the comparison of mean IDR between regular and irregular frames. The accuracy of ESDOF systems in estimating seismic demands and detecting a weakness in stiffness irregular frames subjected to mainshocks and MS-AS are presented in Sect. 5.2.

Estimation of IDR of regular frame

Comparison with ESDOF Systems for IDR subjected to mainshocks

Figure 3 shows the mean values of the height-wise distribution of IDR subjected mainshocks obtained from three ESDOF systems and the NTHA of the MDOF system for the regular frame. The distribution of IDR along the height of the frame becomes non-uniform and as the contribution of higher mode is significant, IDR increases at upper storeys of the frame. Table 1 provides the bias statistics for the regular frame under mainshocks which gives a quick way to assess how much the overestimation and underestimation of ESDOF systems for estimation of IDR of the MDOF system. By comparing the bias statistics of the regular frame under mainshocks, there are considerable differences between ESDOF systems for estimation of IDR along the height of the frame. The estimation of IDR at the upper storeys of the reference frame shows that the ES2 system gives the best estimate of IDR as higher modes contribute to seismic demands. ES2 system is much superior to ES1 which gives nearly the same estimate of IDR as NTHA of the MDOF system by including sufficient modes. ES1 and ES3 show the underestimation of the IDR at upper storeys. ES1 and ES3 show less bias compared to ES2 at middle storeys. By comparing IDR at bottom storeys, ES1 gives better results while ES2 shows

Table 1 Bias statistics for the regular frame based on IDR under mainshock

Storey no.	Regular frame					
	Mean bias			Std. deviation		
	ES1	ES2	ES3	ES1	ES2	ES3
9	1.712	1.004	1.328	0.738	0.193	0.546
8	1.454	1.033	1.260	0.558	0.225	0.459
7	1.258	1.025	1.091	0.403	0.231	0.331
6	1.012	0.905	0.965	0.268	0.183	0.244
5	0.970	0.898	0.982	0.178	0.125	0.199
4	0.991	0.954	1.037	0.174	0.147	0.216
3	0.965	0.944	1.077	0.148	0.134	0.195
2	0.984	0.935	1.135	0.154	0.130	0.183
1	0.959	0.868	1.104	0.206	0.153	0.222

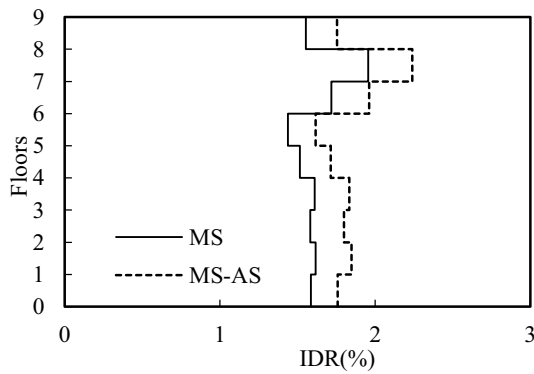


Fig. 4 Distribution of mean IDR along the height of the regular frame obtained by NTHA of MDOF system under mainshock and MS-AS

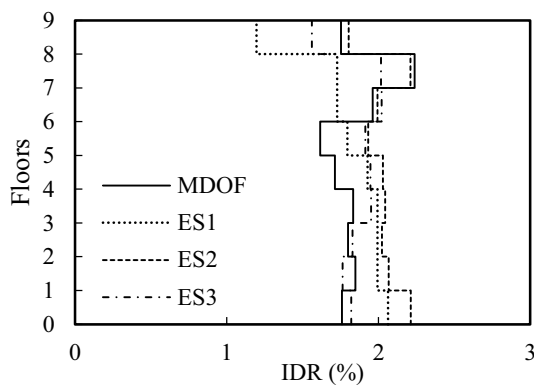


Fig. 5 Distribution of mean IDR along the height of the regular frame determined by ESDOF systems and NTHA of MDOF systems under MS-AS

overestimation, and ES3 shows an underestimation of IDR than ES1. By comparing SD values, ES1 and ES3 are less

consistent along the height of the frame compared to ES2. This demonstrates that ESDOF systems are effective in estimating the IDR of the MDOF system. There are considerable differences between ESDOF systems, still, these systems can be considered to be in practical use.

Comparison with ESDOF Systems for IDR subjected to MS-AS

Figure 4 shows the distribution of mean IDR obtained from NTHA of the MDOF system subjected to mainshocks and MS-AS seismic sequences. It is very well may be seen that under MS-AS seismic sequences the IDR at the bottom, mid-height, and top storey increases to 10%, 13%, and 13%, respectively, relative to mainshocks for the regular frame. Figure 5 shows the mean values of IDR obtained from three ESDOF systems and the NTHA of the MDOF system under the MS-AS seismic sequence. The mean bias and standard deviation are provided (Table 2) to check the accuracy of the ESDOF system for finding IDR of the regular frame subjected to the MS-AS seismic sequence. For the estimation of IDR at the top storey frame, the ES1 system has a very large bias (1.546) and has a quite large standard deviation (0.485). ES2 system that considers the effect of three modes has been effective in the estimation of IDR at the upper storey as mean bias (0.977) close to 1 and minimum standard deviation (0.158) compared to ES1. ES3 gives an underestimation at the upper storey but gives a good estimation than ES1. On the other hand, all ESDOF systems show overestimation at middle and bottom storey but have a less standard deviation. ES1 and ES3 show a good estimation of IDR at the middle and bottom storey than ES2. ES3 gives the best estimates of IDR at the bottom storey while ES2 gives the best estimates at the top storey.

Table 2 Bias statistics for the regular frame based on IDR under MS-AS

Storey no.	Regular frame					
	Mean bias			Std. deviation		
	ES1	ES2	ES3	ES1	ES2	ES3
9	1.546	0.977	1.183	0.485	0.158	0.364
8	1.355	1.018	1.159	0.396	0.182	0.332
7	1.178	0.997	1.005	0.306	0.200	0.256
6	0.930	0.852	0.871	0.174	0.133	0.159
5	0.912	0.859	0.902	0.142	0.118	0.138
4	0.942	0.914	0.962	0.148	0.135	0.153
3	0.919	0.903	1.002	0.130	0.123	0.149
2	0.942	0.905	1.067	0.139	0.127	0.167
1	0.879	0.811	0.997	0.158	0.132	0.185

Estimation of IDR of stiffness irregular frames

Influence of stiffness irregularity on IDR subjected to mainshocks

The NTHA of MDOF stiffness irregular and regular frame subjected to mainshocks is performed and the mean values of the height-wise distribution of IDR between stiffness irregular and regular frames are compared. The height-wise variation of IDR along the height of the frame is affected due to stiffness irregularity. From Fig. 6a, b, and c, it is very well may be seen that there is an increase in the IDR of the modified and adjacent storeys and a decrease in the other storeys for the soft storey cases. However, stiff storey (Fig. 6d, e, and f) decreases the IDR in the irregular storey and neighbouring storeys and increases the IDR in the remaining storeys.

Figure 7 shows the influence of stiffness irregularity by representing the ratio of the distribution of mean IDR along with the height of the frame obtained by NTHA of MDOF of stiffness irregular and regular frames. Soft storey increases IDR at the bottom, mid-height, and top storeys at a percentage of 31%, 30%, and 39%, respectively. Similarly, stiff storey reduces IDR at the bottom, mid-height, and top storeys at a percentage of 23%, 19%, and 22%, respectively. The effect of stiffness irregularity at the bottom storey on inter-storey drift demand of storey further away from the bottom storey (i.e. top storey) is significant. This effect is less when the stiffness irregularity is at the top storey. Hence, it can

be concluded that stiffness irregularity at the bottom storey has shown to be the most severe effect on the distribution of inter-storey drift demand along with the height of the frames. However, the effect of stiffness irregularity at the top storey on IDR is not significant. Hence, it can be advantageous to provide a soft storey at the top of the buildings to reduce the seismic response at the bottom storeys of the building.

Comparison with ESDOF Systems for IDR under mainshocks

Figure 8 presents the distribution of mean IDR along with the height of the frame determined by ESDOF systems and NTHA of MDOF systems for stiffness irregular frames under mainshocks. The comparison of bias statistics for estimating the height-wise variation of IDR of 9-storey stiffness irregular frames under mainshocks is presented in Table 3. The comparison of bias statistics of ESDOF systems between the reference frame and soft first storey (KI-0.5-1) frame shows that ES2 gives overestimation at the upper storey for soft first storey compared to the reference frame. Also, the accuracy of ES1 and ES3 is improved at the upper storey. There is an overestimation of IDR at bottom storeys by ES1 and ES2 systems, but ES3 gives a good estimation of IDR at bottom storeys. Considering bias statistics of all ESDOF systems for stiff first storey (KI-2-1) frame is about the same as found for the regular frame at the top and middle storey but the accuracy deteriorates by ES1 and ES3 systems at the

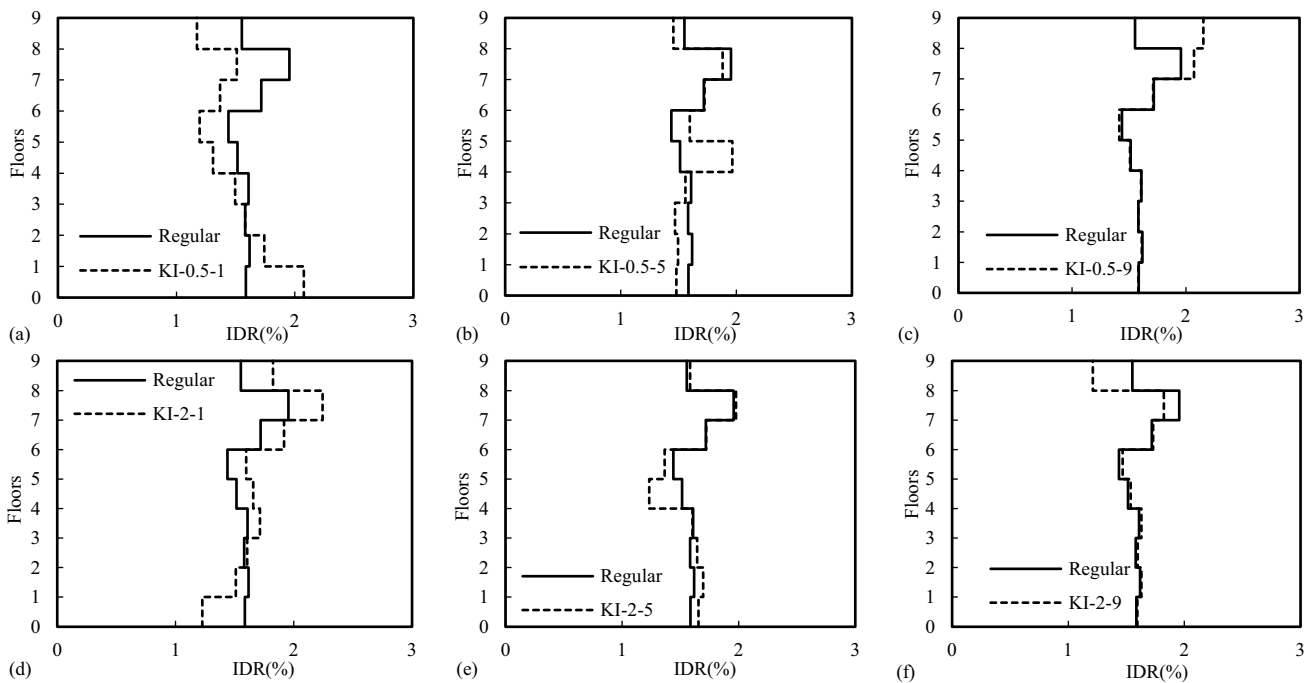


Fig. 6 Distribution of mean IDR along with the height of regular frame and stiffness irregular frames determined by NTHA of MDOF system under mainshocks

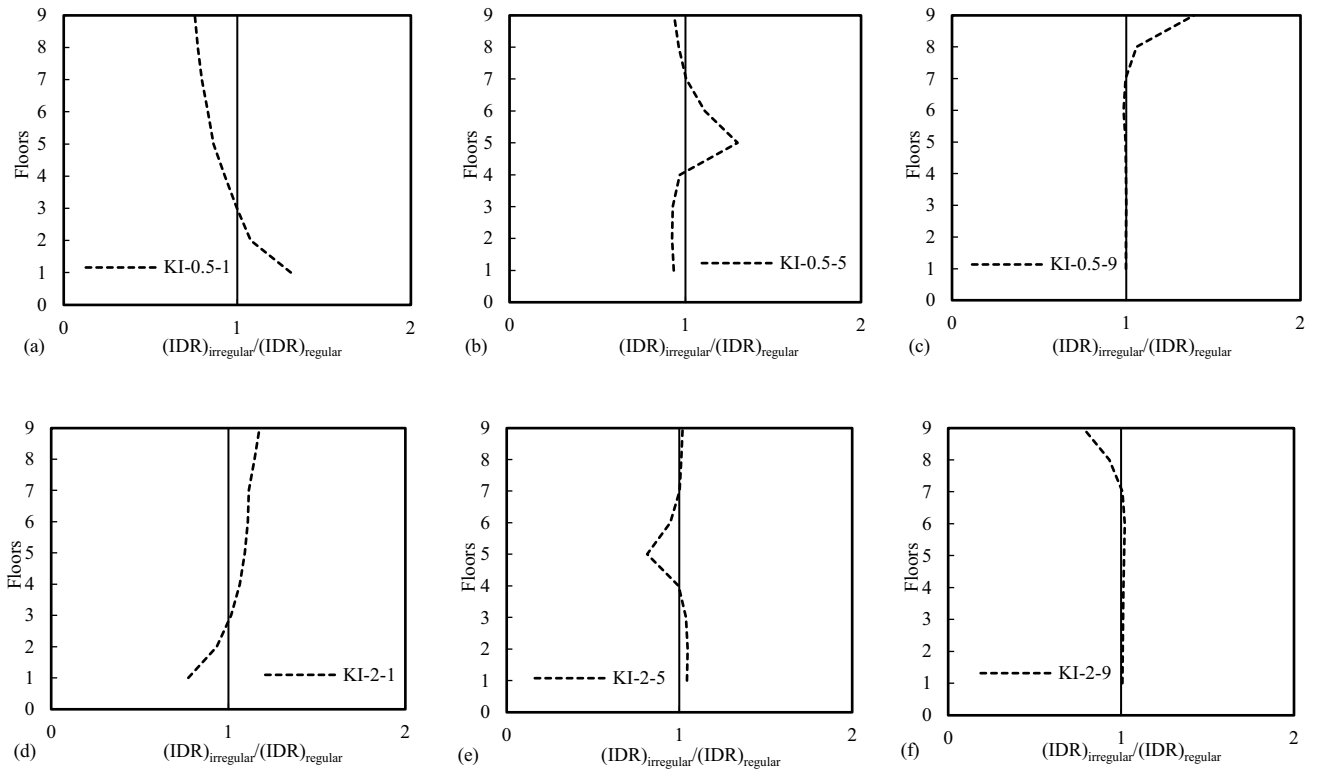


Fig. 7 Ratio of distribution of mean IDR along the height of frame obtained by NTHA of MDOF of stiffness irregular frames and regular frame under mainshocks

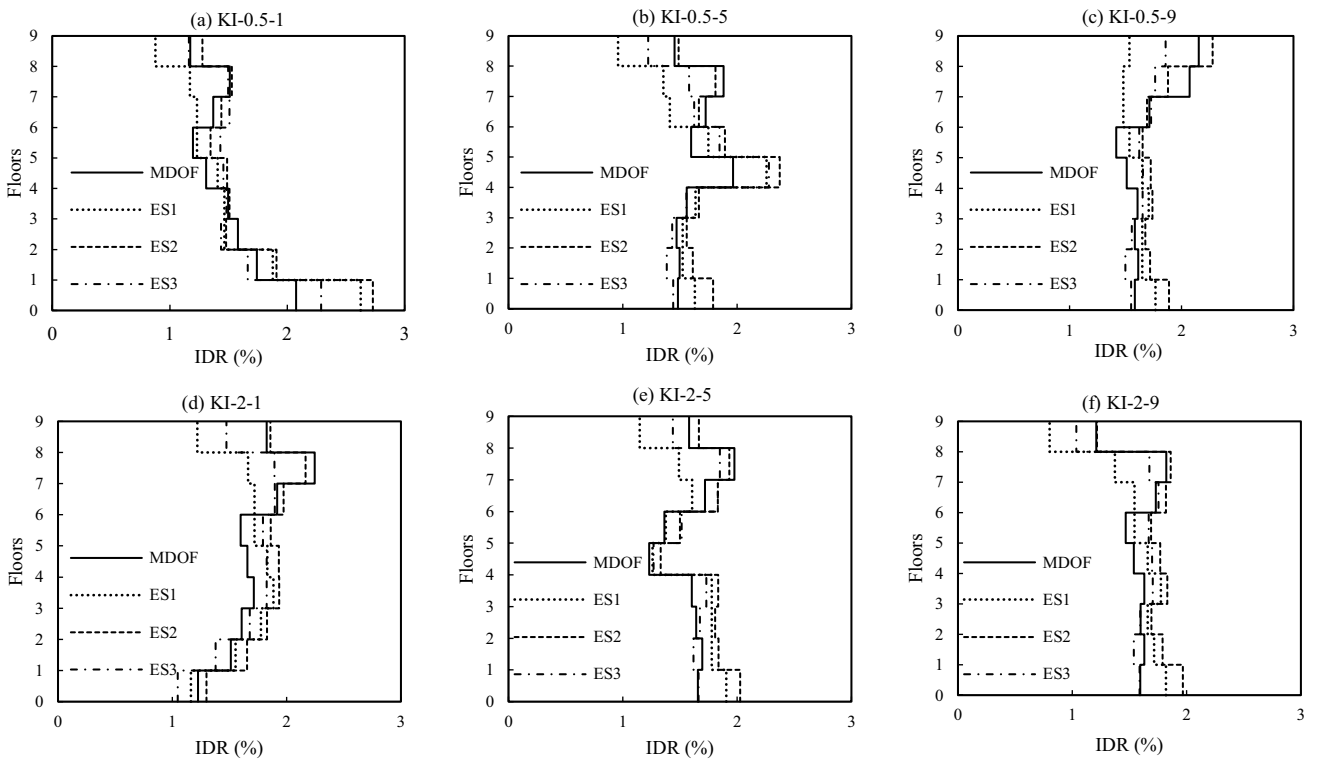


Fig. 8 Distribution of mean IDR along the height of stiffness irregular frames determined by ESDOF systems and NTHA of MDOF systems under mainshocks

Table 3 Bias statistics for the 9-storey stiffness irregular frames based on IDR under mainshock

Storey No	(a) KI-0.5–1						(b) KI-0.5–5					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.512	0.948	1.143	0.638	0.202	0.481	1.720	0.999	1.347	0.753	0.212	0.586
8	1.422	1.012	1.113	0.535	0.213	0.416	1.540	1.058	1.318	0.598	0.229	0.509
7	1.216	0.986	0.991	0.409	0.228	0.331	1.326	1.063	1.150	0.443	0.249	0.381
6	1.051	0.932	0.905	0.294	0.206	0.252	0.995	0.893	0.943	0.270	0.190	0.254
5	0.980	0.911	0.947	0.181	0.132	0.174	0.940	0.879	0.931	0.205	0.160	0.203
4	1.054	1.016	1.044	0.143	0.122	0.143	0.994	0.970	1.044	0.150	0.135	0.158
3	1.102	1.087	1.125	0.130	0.125	0.137	1.003	0.973	1.067	0.151	0.134	0.163
2	0.961	0.938	1.084	0.134	0.125	0.155	1.027	0.953	1.130	0.176	0.141	0.196
1	0.855	0.811	0.982	0.192	0.161	0.223	0.983	0.870	1.113	0.240	0.167	0.271
Storey No	(c) KI-0.5–9						(d) KI-2–1					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.600	0.968	1.323	0.720	0.199	0.593	1.699	1.003	1.406	0.753	0.200	0.621
8	1.519	1.109	1.272	0.558	0.247	0.465	1.487	1.054	1.306	0.574	0.227	0.501
7	1.245	1.039	1.065	0.371	0.227	0.315	1.207	1.004	1.093	0.372	0.222	0.335
6	1.000	0.904	0.946	0.264	0.191	0.249	1.011	0.906	0.970	0.273	0.182	0.261
5	0.972	0.916	0.968	0.202	0.163	0.200	0.967	0.899	0.970	0.210	0.154	0.210
4	0.990	0.964	1.022	0.160	0.143	0.165	0.969	0.935	1.001	0.194	0.166	0.200
3	0.996	0.974	1.055	0.143	0.131	0.153	0.954	0.917	1.010	0.170	0.144	0.180
2	1.014	0.961	1.117	0.156	0.135	0.174	1.017	0.937	1.146	0.170	0.132	0.192
1	0.956	0.874	1.090	0.202	0.156	0.230	1.109	0.966	1.229	0.221	0.170	0.246
Storey No	(e) KI-2–5						(f) KI-2–9					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.569	0.976	1.251	0.697	0.211	0.553	1.693	1.007	1.307	0.730	0.214	0.560
8	1.466	1.042	1.180	0.572	0.225	0.458	1.477	1.001	1.209	0.572	0.200	0.464
7	1.165	0.972	1.021	0.367	0.216	0.320	1.222	0.983	1.075	0.403	0.216	0.350
6	1.065	0.949	0.967	0.270	0.180	0.244	1.025	0.909	0.949	0.270	0.184	0.247
5	1.015	0.948	1.008	0.171	0.128	0.168	0.980	0.903	0.957	0.178	0.123	0.172
4	0.955	0.916	0.980	0.181	0.152	0.186	0.965	0.927	1.005	0.172	0.145	0.178
3	0.968	0.947	1.028	0.155	0.142	0.166	1.006	0.979	1.046	0.155	0.140	0.161
2	0.993	0.952	1.092	0.154	0.134	0.171	0.989	0.937	1.103	0.160	0.135	0.178
1	0.926	0.854	1.062	0.188	0.150	0.216	0.941	0.849	1.078	0.210	0.154	0.239

bottom storey. Also, ES2 gives good estimates at bottom storeys compared to the regular frame. It can also be observed that all ESDOF systems overestimate the IDR demands at bottom storeys in the case of soft first storey case (KI-0.5–1) and underestimates the IDR demands at bottom storeys in case of stiff first storey case (KI-2–1) compared to the reference frame. Considering the cases where irregularity is in the middle (KI-0.5–5 and KI-2–5) and top storeys (KI-0.5–9 and KI-2–9), the accuracy of ESDOF systems for estimation of IDR does not deteriorate.

Structural responses of stiffness irregular frames under mainshocks and MS–AS

The NTHA of stiffness irregular frames is performed subjected to MS–AS seismic sequences. Figure 9 shows the distribution of mean IDR along the height of stiffness irregular frames subjected to mainshocks and MS–AS seismic sequences. It is very well understood by the height-wise distribution of IDR that the seismic sequence gives larger IDR relative to mainshocks for stiffness irregular frames.

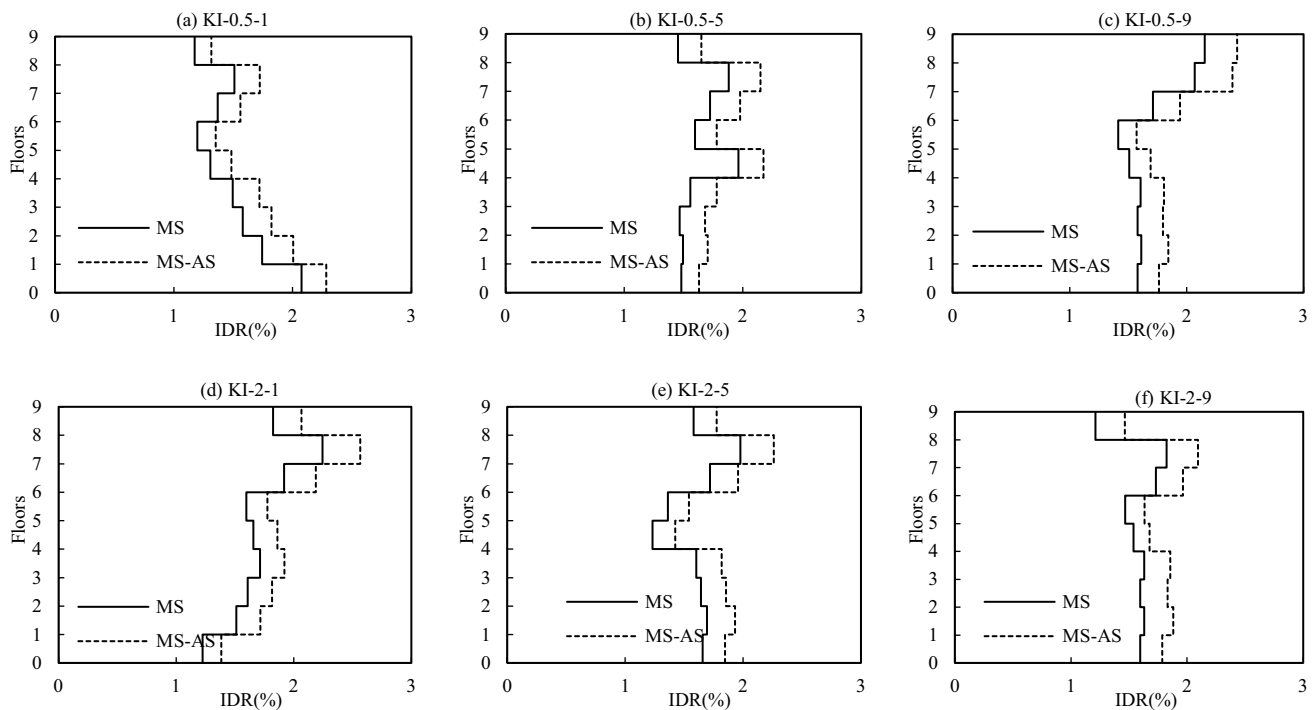


Fig. 9 Distribution of mean IDR along the height obtained by NTHA of MDOF system under mainshock and MS–AS

The increase in the percentage of IDR is in the range of 10–13% at the bottom storey, 10–15% at the middle storey, and 11–14% at the top storey for stiffness irregular cases, except for stiff top storey the percentage increase is 21% at the top storey. Figure 10 shows the distribution of mean IDR along the height of the regular and stiffness irregular frames subjected to MS–AS. The soft and stiff storeys influence the height-wise variation of IDR compared to the regular frame and which depends on the location of the irregular storey. Figure 11 shows the influence of stiffness irregularity by representing the ratio of the distribution of mean IDR along the height of the frame obtained by NTHA of MDOF of stiffness irregular and regular frames subjected to MS–AS. Soft storey increases IDR at the bottom, mid-height, and top storeys at a percentage of 29%, 27%, and 39%, respectively. Similarly, stiff storey reduces IDR at the bottom, mid-height, and top storeys at a percentage of 22%, 17%, and 17%, respectively.

Comparison with ESDOF systems for IDR under MS–AS

Figure 12 shows the mean values of the height-wise variation of IDR under the MS–AS seismic sequence obtained from three ESDOF systems and the NTHA of the MDOF system of stiffness irregular frames. The mean bias and the standard deviation in the estimation of IDR under the MS–AS seismic sequence from the three ESDOF systems for stiffness irregular frames are presented in Table 4. By

comparing the bias statistics of stiffness irregular frames under mainshocks and MS–AS seismic sequences (Tables 3 and 4), the accuracy of the ES2 system is deteriorating at the top storey (mean bias in the range of 0.9–1.05) for stiffness irregular frames under MS–AS seismic sequences. ES2 is incapable of estimating IDR at middle and bottom storeys. ES1 and ES3 give an underestimation at the top storey. All ESDOF systems give an overestimation of IDR at the middle storey (mean bias in the range of 0.82–0.977) for stiffness irregular frames. ES3 gives a good estimation of IDR at the bottom storey (mean bias in the range of 0.98–1.02) for stiffness irregular frames except for irregularity in the first storey.

Summary and conclusions

The present study mainly focused on the seismic behaviour of regular and stiffness irregular steel moment-resisting building frames subjected to mainshocks and MS–AS seismic sequences. The present study discusses three different ESDOF systems for obtaining the distribution of IDR along with the height of the real MDOF system. The study also expands to evaluate the accuracy of the ESDOF systems for estimating inter-storey drift ratios of regular and stiffness irregular frames subjected to mainshocks and MS–AS. The comparison of the distribution of IDR along the height of the frames obtained from MDOF and ESDOF systems was

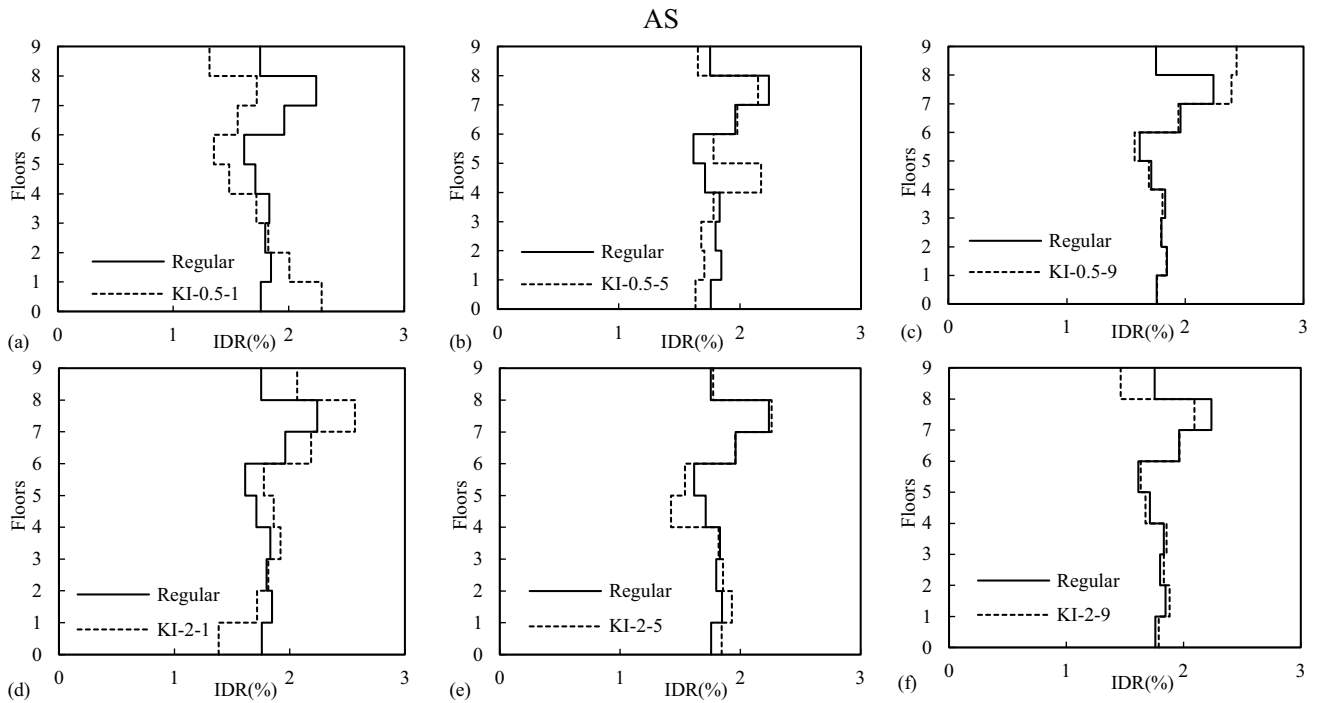


Fig. 10 Distribution of mean IDR along with the height of regular frame and stiffness irregular frames determined by NTHA of MDOF system under MS-AS

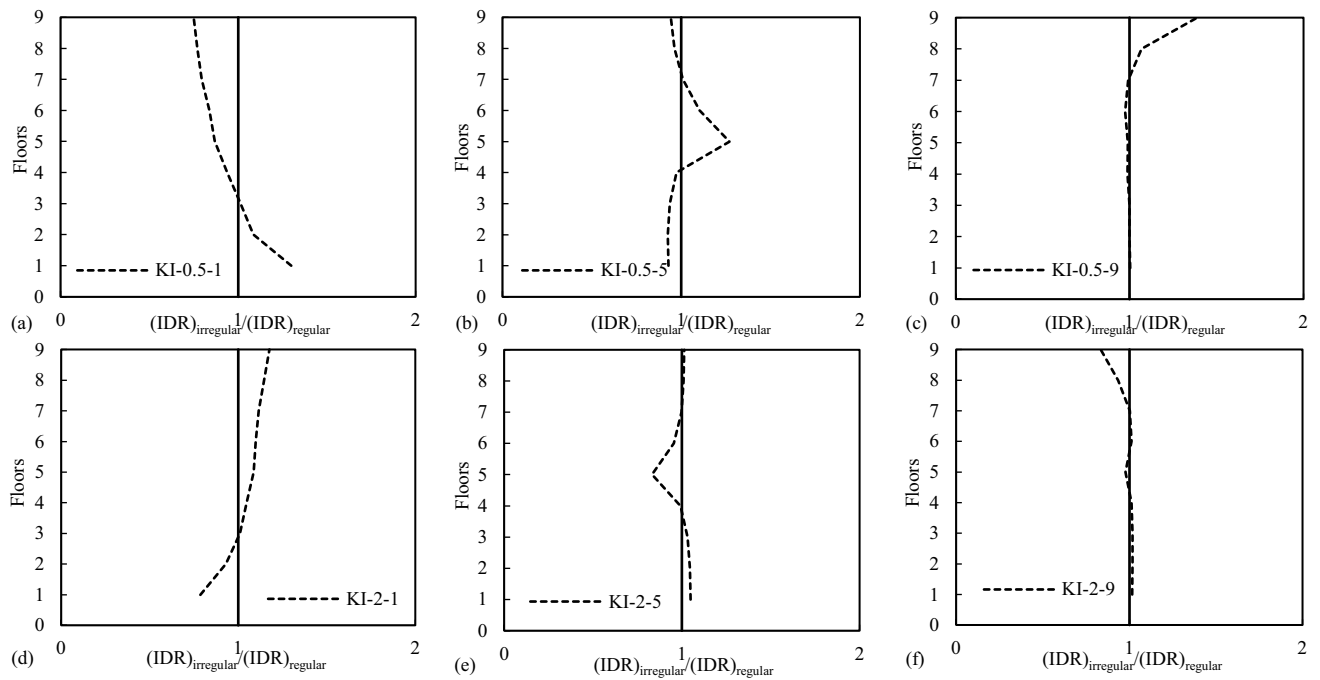


Fig. 11 Ratio of distribution of mean IDR along with the height of frame obtained by NTHA of MDOF of stiffness irregular frames and regular frame under MS-AS

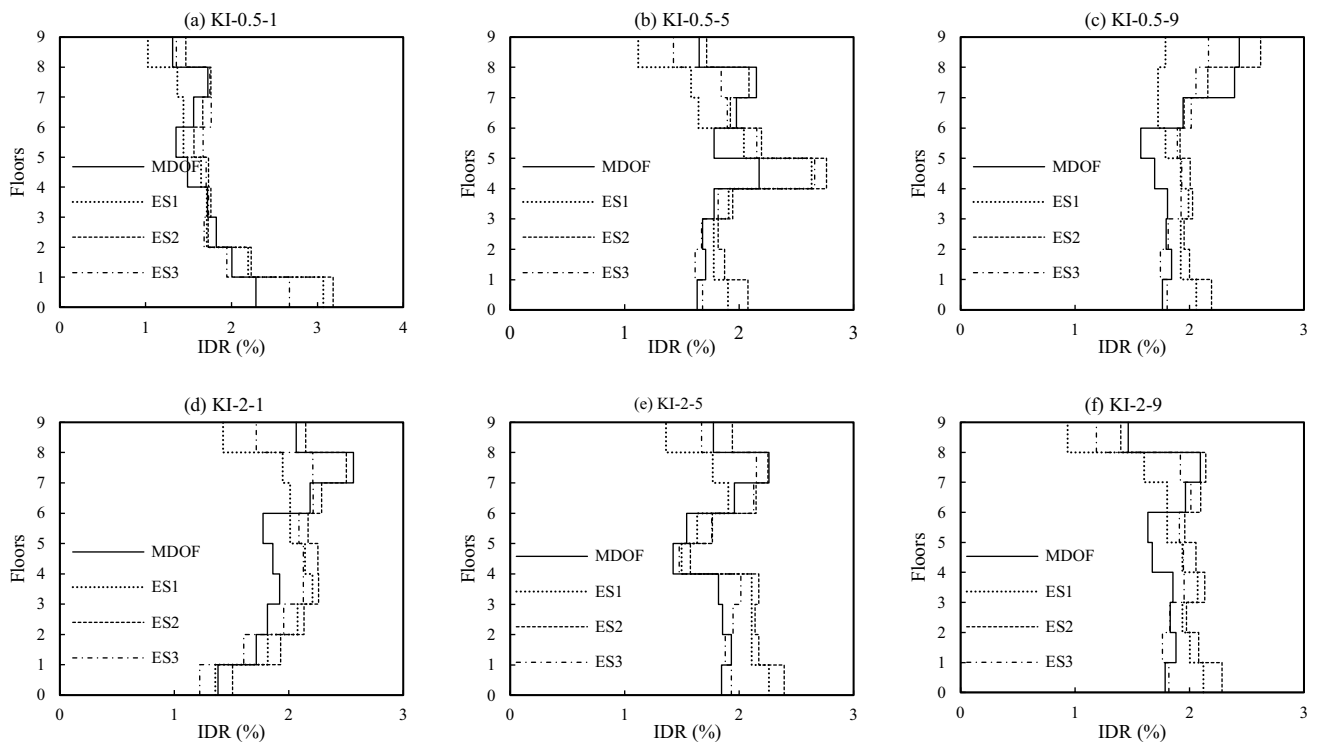


Fig. 12 Distribution of mean IDR along the height of stiffness irregular frames determined by ESDOF systems and NTHA of MDOF systems under MS-AS

given. The effectiveness of ESDOF systems for estimation of IDR is examined by mean bias and standard deviation of the ratio of mean values of IDR determined by NTHA of MDOF systems to the values determined by ESDOF systems. This investigation on seismic demands of 9-storey SMRF building has highlighted the following conclusions:

- 1 The effect of stiffness irregularity on seismic behaviour of 9-storey frame subjected to mainshocks and MS-AS is significant depending on the location of the irregular storey. IDR increases in modified and adjacent storeys and decreases in remaining storeys in the case of the soft storey. However, stiff storey decreases the IDR in the modified and neighbouring storeys and increases the IDR in other storeys.
- 2 The MS-AS seismic sequences have increased the distribution of IDR along the height of regular and stiffness irregular frames. Therefore, it is necessary to consider the MS-AS seismic sequence in the estimation of seismic response of structures rather than considering mainshocks only as structures in seismically active regions are subjected to the sequence of earthquakes.
- 3 Based on bias statistics and standard deviation of ESDOF systems for the regular and stiffness irregular frames under mainshocks and MS-AS, it can be concluded that: ES2 is recommended for the estimation of IDR at the upper storey which is similar to the exact results from the NTHA of the MDOF system. ES1 and ES3 can be recommended for the estimation of IDR at middle and bottom storeys. The accuracy of ES1 and ES2 reduces under MS-AS compared to mainshocks and gives overestimation of IDR at middle and bottom storeys, but ES3 gives good estimation at the bottom storey.
- 4 The comparison of the statistics shows that there are considerable differences between ESDOF systems for estimating the distribution of IDR along with the height of 9-storey regular and stiffness irregular frames subjected to mainshock and MS-AS. The results presented herein clearly show the ESDOF systems do a better job of estimating the height-wise variation of IDR along the height of regular and stiffness irregular frames subjected to mainshocks and MS-AS without going for computation-intensive nonlinear dynamic analysis of MDOF systems.

Table 4 Bias statistics for the 9-storey stiffness irregular frames based on IDR under MS-AS

Storey No	(a) KI-0.5-1						(b) KI-0.5-5					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.346	0.902	1.018	0.398	0.142	0.298	1.556	0.966	1.218	0.507	0.168	0.392
8	1.315	0.985	1.029	0.374	0.167	0.290	1.424	1.035	1.218	0.419	0.183	0.355
7	1.127	0.948	0.919	0.294	0.180	0.237	1.244	1.038	1.079	0.320	0.205	0.274
6	0.972	0.881	0.837	0.207	0.159	0.177	0.905	0.830	0.857	0.174	0.132	0.162
5	0.927	0.873	0.895	0.143	0.115	0.138	0.857	0.810	0.849	0.148	0.121	0.146
4	1.022	0.991	1.012	0.135	0.124	0.137	0.954	0.935	1.003	0.144	0.135	0.152
3	1.078	1.066	1.100	0.138	0.135	0.146	0.961	0.938	1.022	0.134	0.126	0.146
2	0.928	0.911	1.048	0.126	0.121	0.146	0.976	0.920	1.075	0.150	0.135	0.169
1	0.773	0.740	0.887	0.136	0.119	0.159	0.887	0.803	1.004	0.165	0.131	0.188
Storey No	(c) KI-0.5-9						(d) KI-2-1					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.439	0.938	1.191	0.440	0.146	0.363	1.526	0.966	1.268	0.493	0.164	0.404
8	1.440	1.108	1.207	0.430	0.231	0.358	1.376	1.031	1.213	0.401	0.189	0.350
7	1.164	1.006	0.997	0.302	0.218	0.257	1.124	0.968	1.022	0.276	0.190	0.248
6	0.908	0.838	0.859	0.166	0.130	0.157	0.911	0.835	0.877	0.157	0.117	0.149
5	0.906	0.862	0.902	0.159	0.134	0.157	0.893	0.841	0.899	0.147	0.121	0.150
4	0.931	0.911	0.962	0.142	0.132	0.147	0.893	0.868	0.927	0.141	0.128	0.148
3	0.949	0.932	1.006	0.132	0.126	0.142	0.892	0.865	0.947	0.135	0.122	0.142
2	0.973	0.931	1.072	0.145	0.132	0.162	0.961	0.899	1.087	0.147	0.129	0.165
1	0.882	0.820	1.007	0.158	0.132	0.181	1.038	0.925	1.154	0.189	0.165	0.210
Storey No	(e) KI-2-5						(f) KI-2-9					
	Mean Bias			Std. Deviation			Mean Bias			Std. Deviation		
	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3	ES1	ES2	ES3
9	1.397	0.929	1.123	0.444	0.163	0.350	1.646	1.051	1.310	0.548	0.221	0.605
8	1.355	1.020	1.099	0.404	0.193	0.319	1.369	0.984	1.156	0.389	0.159	0.513
7	1.082	0.937	0.955	0.285	0.198	0.245	1.134	0.949	1.028	0.287	0.174	0.449
6	0.984	0.896	0.900	0.186	0.142	0.162	0.935	0.849	0.894	0.189	0.143	0.378
5	0.976	0.921	0.977	0.155	0.138	0.145	0.884	0.825	0.892	0.156	0.128	0.379
4	0.895	0.864	0.925	0.145	0.130	0.141	0.917	0.886	0.983	0.143	0.129	0.362
3	0.910	0.895	0.973	0.140	0.133	0.141	0.964	0.944	1.034	0.137	0.129	0.383
2	0.945	0.913	1.046	0.148	0.137	0.156	0.956	0.916	1.103	0.148	0.136	0.465
1	0.852	0.797	0.984	0.156	0.135	0.174	0.869	0.799	1.026	0.159	0.135	0.422

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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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