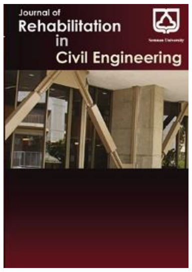

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Seismic Performance Assessment of Steel Moment Frames with Non-parallel System Irregularity

Payam Tehrani[1*], Ariya Eini[2]

1. Assistant Professor, Department of Civil & Environmental Engineering, AmirKabir University of Technology (Tehran Polytechnic), Tehran, Iran
2. Former graduate student, Department of Civil & Environmental Engineering, AmirKabir University of Technology (Tehran Polytechnic), Tehran, Iran
Corresponding author: payam.tehrani@aut.ac.ir

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ABSTRACT

Many structures exhibit non-orthogonal systems irregularity based on architectural design, which is a type of torsional irregularity. This paper evaluates the inelastic response of multi-story steel moment resisting frames with this type of irregularity. A parametric study is carried out on six building models exhibiting coupled behavior in lateral and torsional response with various degrees of torsional irregularity. Current code regularity limits for structures appear to be based on engineering judgment rather than on quantitative analyses, which indicates that these limits need to be investigated for different structural systems with different types of irregularities. Another goal of this paper is the evaluation and comparison of the response modification factor values of steel moment resisting frames with non-parallel systems irregularity derived by pushover analysis, as well as nonlinear time history analysis. A new torsional irregularity coefficient is proposed based on the response spectrum analysis results. It is shown that it is essential to undertake nonlinear dynamic analysis to design some structures with high irregularity in plan and to capture nonlinear mechanisms due to non-parallel systems irregularity.

1. Introduction

Seismic code provisions typically classify structural irregularity into irregularity in plan and elevation. Irregularity in plan typically occurs because of uneven distributions of stiffness, strength or mass, in the plan, which can result in severe damage during

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earthquakes mainly due to the floor rotations (i.e., due to torsion) coupled with floor translations. This torsional behavior typically can result in higher seismic demands than those expected in regular structures, which can cause a concentration of damage in some structural elements with the highest seismic demands.

The presence of irregularities in the lateral load resisting systems in structures may inhibit the uniform distribution of dynamic loads and plastic deformations within the structure[1]. Depending on the degrees of irregularity and the internal forces, the irregularity may localize the vibration modes [2]. It was shown that a building that was strengthened only at one side of the structure was significantly damaged due to irregularity effects resulting from partial rehabilitation of the building when subjected to an earthquake [3].

The impact of irregularity in plan and elevation on the seismic behavior of irregular single- or multi-story structures has been investigated in some studies [4–8].

Different aspects of torsional irregularity such as geometric asymmetry have been investigated in a number of studies [9–11], using different analysis methods such as nonlinear time history dynamic (NTH) and nonlinear static analyses (i.e., pushover analysis), etc.[12–14]. Similarly, some studies investigated the code provisions regarding the definition of irregularity [15,16], using both experimental and analytical studies [17,18].

Anagnostopoulos investigated several reinforced concrete buildings with torsional irregularity and observed increased displacements and increased displacement ductility factors at the flexible edges of the building [18,19]. Teddy investigated the effect of architectural geometry in non-parallel systems irregularity configuration [20]. There is a lack of research on this type of torsional irregularity [21].

Sirsikal et al. found critical dimensions of structural members for different performance levels in RC frames with different types of plan irregularities [22]. Kheyroddin et al. studied the effect of the concentrically braced frame position in steel structures with a triangular plan. They considered different configurations and used the response spectrum and pushover analyses. it was shown that the least torsion is caused by the continuous braces in the middle of the triangle sides [23].

In accordance with FEMA-451B [24], an irregular configuration in non-parallel system is defined for the cases that the lateral force-resisting elements are neither parallel nor symmetric with the main orthogonal axes of the system. The purpose of this study is to determine the effect of angled frames on the torsional irregularity of non-orthogonal steel frames and to investigate the adequacy of code provisions for this type of irregularity.

2. Regular and irregular in plan steel structures

The efficiency of the methods of analysis considered is evaluated using six multi-story buildings, consisting of one basic regular model and five irregular models. In order to study different degrees of torsional irregularity, the west side moment frame (frame A) is rotated about the middle column at different angles, including 15, 20, 25, 30, and 35 degrees, as shown in Fig.1. Choosing greater degrees of rotation was avoided because it changed the design sections and the seismic mass in a way that the results could be of no interest. The models under investigation are three-by-two bay five-story steel moment resisting frames. The bay width and story height of each model are kept constant. Lateral resistance in both directions is provided by special moment resisting frames, and each floor is 3.3 m high. No stairwell is considered in the plan of the structure so that non-parallel system irregularity is isolated from other irregularities. Therefore, using this setup any change in the seismic response of the structures as a result of a change in the plan irregularity can be assessed. The loading conditions have been adopted from ASCE7-16 [25] and are shown in Table 1. St37 steel with a yield strength of 240 MPa was used in this study.

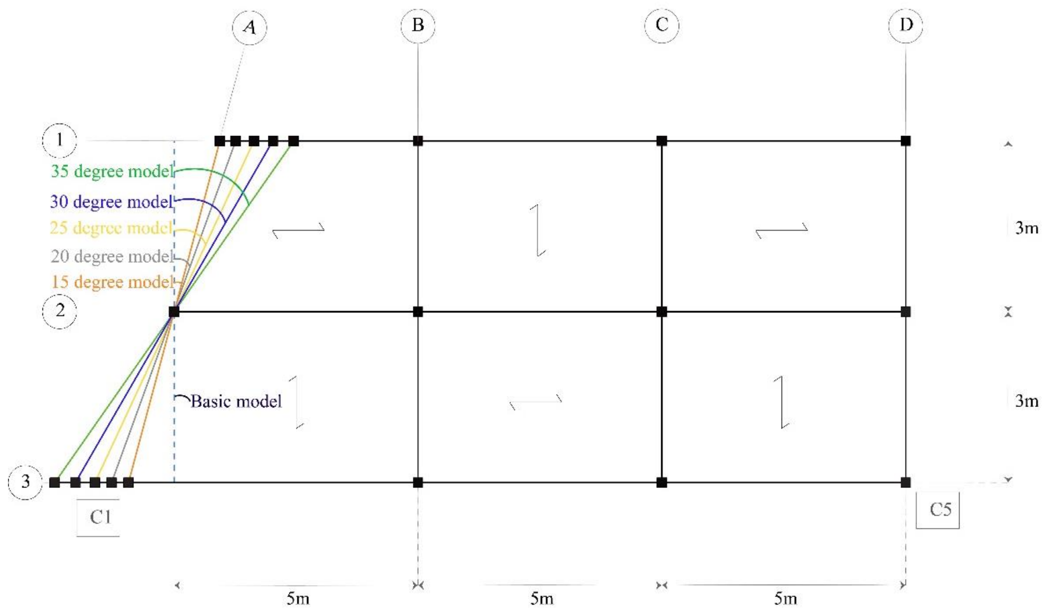


Fig. 1. The basic model plan and five irregular models, including 15, 20, 25, 30, 35 degrees models.

Table 1. Gravity Loads applied to all models.

Dead loads		Live Loads	
Exterior	6500N/m	Interior*	$1000N/m^2$
Floors	$500\ N/m^2$	Floors	$2000N/m^2$
Roof	$570\ N/m^2$	Roof	$1500N/m^2$
		Snow	$500\ N/m^2$

* The lightweight partition load in this research was considered as the live load. It is noted that the build-in partitions are defined as dead loads in ASCE7-16.

In ASCE 7-16 [25], when the maximum story drift at one corner of the story is more than 1.2 times the average story drift on any floor, the structure is considered to be torsionally irregular. A three-dimensional analytical model is required for the analysis of plan-irregular structures to consider the torsional effects. In this study, the response spectrum analysis (RSA) method, which has no limitations according to the level of seismic design categories is used. For these irregular structures, both inherent and accidental torsions should be appropriately considered in structural analysis and design. The initial design is carried out through the equivalent static method (ESM) with a 100-30 combination rule [25]. RSA is performed with 100% of the scaled spectrum acting in one direction, concurrent with the application of 30% of the scaled spectrum acting in the orthogonal direction. A 5% eccentricity of the center of mass along the direction with the greater effect is also applied. For steel special moment frames (SMFs), considered in this study, the response modification factor (R), displacement amplification factor (C_d), overstrength factor (Ω_0), and importance factor (I_e) are 8.0, 5.5, 3.0, and 1.0, respectively [25]. Special moment frames are used in seismic regions where higher ductility capacity is needed. The ETABS16 software [26] is used for analysis and design. The strong column and weak beam requirements and drift limitations are also considered in the design. The site is class B, and SS and S1 coefficients for a return period of 475 years are 1.5g and 0.634g, respectively. The member specifications are listed in Table 2.

Story drifts are calculated using both modal RSA and ESM. Firstly, the results of the story drifts obtained using the ESM are shown in Fig.2. It is evident in Fig. 2 that due to the rotation of frame A, in the y-direction, the inter-story drifts increases as the angle of rotation increases. The drifts in the y-direction were greater than those in the x-direction. The ratios of δ_{max} (Maximum story drift) to δ_{CM} (average story drift) are shown in Fig.3.

Table 2. Member sections for designed models

	Columns	Beams	
Story	All frames	X-dir	Y-dir
1	Box 250-250-20	IPE 300	IPE 270
2	Box 250-250-20	IPE 300	IPE 270
3	Box 250-250-20	IPE 300	IPE 300
4	Box 200-200-15	IPE 270	IPE 270
5	Box 200-200-15	IPE 270	IPE240

* These sections change to IPE 330 in 25, 30, and 35 models.

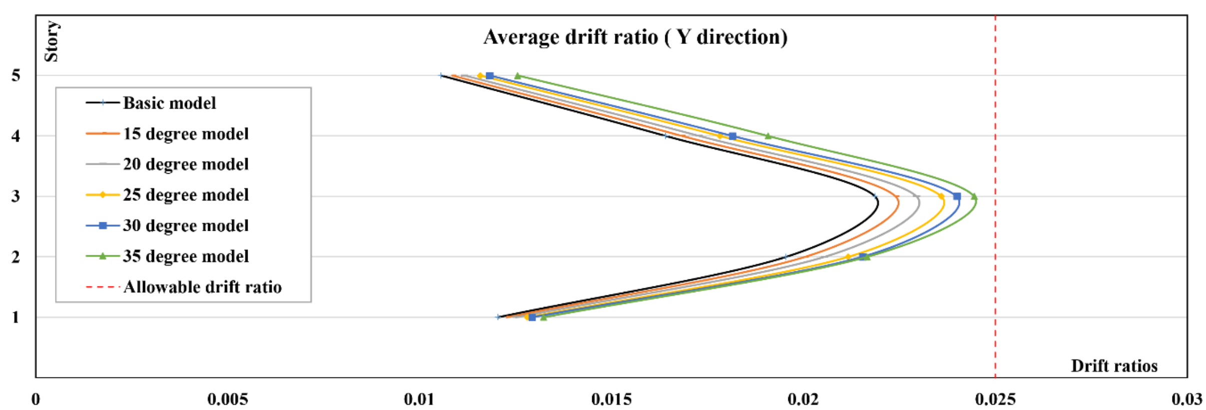


Fig. 2. Average drift ratios in y – direction using ESM for all models and the allowable limit.

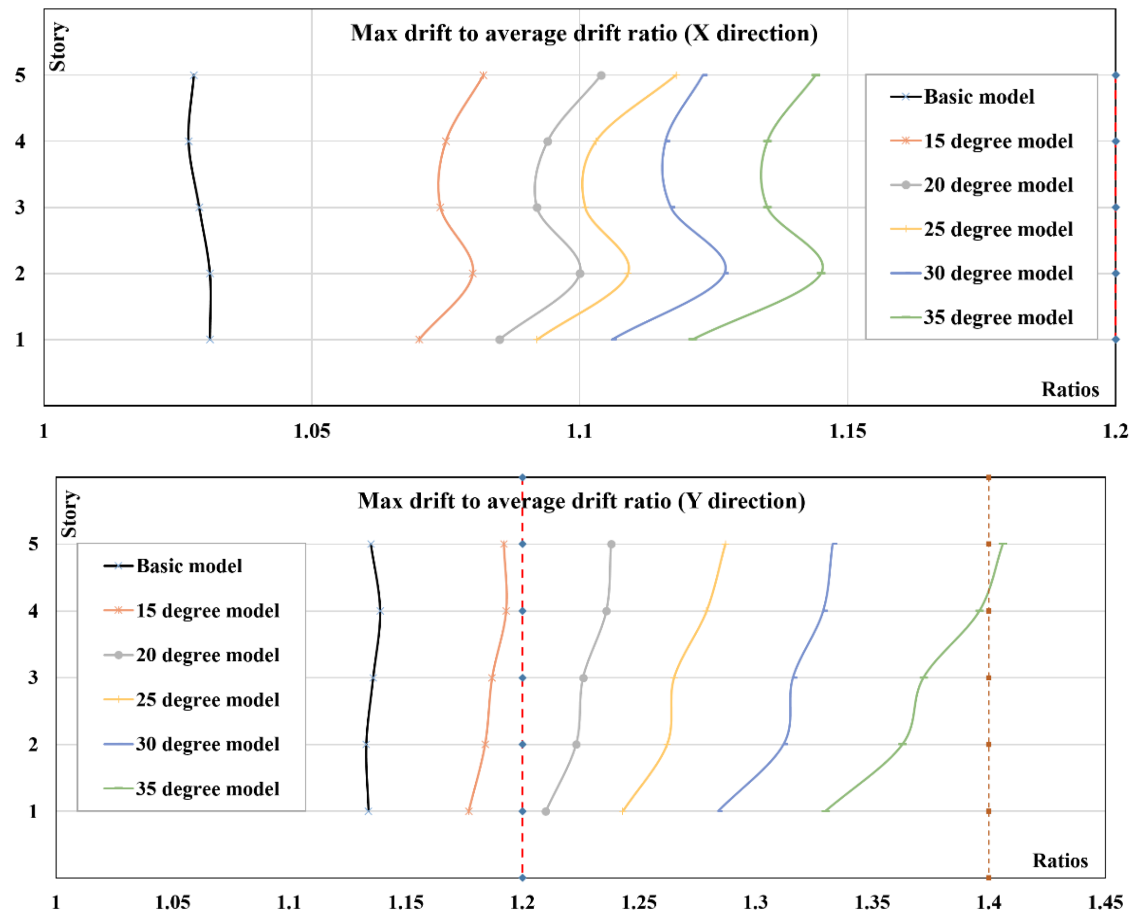


Fig. 3. Maximum drift to average drift ratio of each floor in all models in x and y - directions.

The plan irregularity exists because of the variations in the y-direction rather than the x-direction (Fig.2). The basic model and the 15 degrees model do not have any irregularity according to the code provisions, the 20, 25, 30 degrees models have torsional irregularity and the 35 degrees model, due to the drifts in its fifth floor, is characterized as extremely irregular. It is noted that the amplification factor for the accidental torsion (A_x) was applied only to the 35 degrees model. In the other models the A_x factor was assumed to be 1.0, since the maximum drift to the average drift ratio was always less than 1.2. The distribution of stiffness in these models in both directions is in a way that the fundamental periods (T) of the structures in x and y-directions are almost identical. The first modes in both directions of the basic model are purely transitional, while, in other cases, because of the asymmetry in the y-direction, the first and the second modes are coupled, while the third one is acting more torsional.

As the degree of irregularity increases from the basic structure to the 35 degrees model, the higher mode effects and also the need to implement dynamic analysis increases.

Since the eigenfrequencies for the structures under study are close, the complete quadratic combination (CQC) rule was used for combining the effects of different modes. When three-dimensional multi-story buildings are studied, it is assumed that each floor has a rigid diaphragm with three in-plan degrees of freedom.

In this study, all fifteen modes of vibration are considered in the RSA analysis. When the response spectrum is applied in each direction, the periods and the normalized base shear forces associated with the mode shapes are calculated. For all models except the basic model in each mode, there are base shears both parallel to the principal direction and perpendicular to it. After combining the response of all modes, there still remains a smaller base shear perpendicular to the direction that the spectrum was applied. If this total smaller force perpendicular to the principal direction is expressed as the percentage of the base shear in the principal direction, it can serve as a new definition of plan irregularity (Table 3).

For design purposes, according to ASCE7-16 [25,27], base shears should be scaled to ensure the minimum strength of a structure designed using the RSA is similar to the strength that would be required if the structure was designed using the ESM. Reduced base shear should be scaled to 85%, 90%, and 100% of the value calculated using ESM for the case of regular, irregular, and extreme torsional irregular structures, respectively. Average drift ratios based on the scaled base shears in the critical case are shown in Fig.4.

Table 3. The ratios of the base shear perpendicular to the principal direction, to that in the parallel direction.

model / Direction	Basic	15°	20°	25°	30°	35°
X	0.00	0.31	0.42	0.46	0.47	0.47
Y	0.00	0.30	0.40	0.45	0.49	0.50

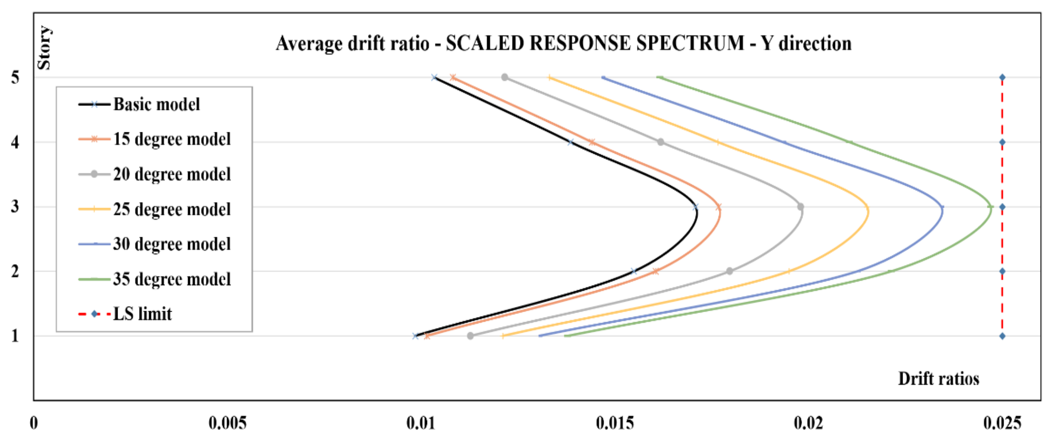


Fig. 4. Average drift ratio of scaled response spectrum analysis for all six models.

3. Nonlinear static analysis

In order to predict the target displacements of the structures, the structures are pushed based on a predetermined load pattern and the structural capacity curves are determined. The seismic performance of the structural elements is then evaluated at the target displacements. In pushover analysis, the geometric nonlinearities are taken into account according to the P-Delta formulation, and material nonlinearity is modeled through a concentrated plasticity method. The plastic hinge response curves and the relevant acceptance criteria were derived according to the ASCE 41-17 provisions [27]. The flexibility of the beam-column joints was neglected in the modelling. Since a comparative study is carries out to evaluate the seismic response of buildings with different degrees of irregularity, this assumption is deemed appropriate for the sake of this research.

A coupled P-M2-M3 hinge that yields based on the interaction of axial force and bending moments at the hinge location was assigned to columns. Also, moment and shear hinges were used for beams. All hinges were located at 5% of the length of elements from the connection joints.

In pushover analysis, the basic assumptions are that the distribution of inertial forces (i.e., the load pattern) will be constant throughout the earthquake. However, in reality the structural response is affected by higher mode effects and therefore, the structure does not have a single yielding mechanism. A load pattern derived from the response spectrum analysis that accounts for elastic higher mode effects is recommended to perform the pushover analysis [28]. The analysis procedure starts with the application of the gravity loads, and an incremental step-by-step analysis in which the load pattern is applied in increments corresponding to stiffness changes in each component.

The target displacement for the multi-degree of freedom (MDOF) structure can be estimated as the displacement demand for the corresponding equivalent single degree of freedom (SDOF) system using Eq. (1) [27].

$$\delta_t = C_0 C_1 C_2 Sa \frac{T^2}{4\pi^2} g \qquad \text{Eq.(1)} \qquad (1)$$

Where in Eq. (1), the modification factor for the MDOF, C_0, transforms the equivalent SDOF displacement to the building roof displacement. If the elastic first mode shape is used in each direction, C_0 becomes the first mode participation factor which is calculated as 1.32 for y-direction and 1.34 for x-direction. The modification factor C_1 accounts for the difference between the inelastic and elastic displacement of SDOF, and C_2 is the modification factor for stiffness and strength deterioration. For the first iteration, the δ_t was calculated as 30 cm in both directions for the basic structure. The nonlinear relation between the base shear and roof displacement is idealized with a bilinear relation using FEMA 365 and ASCE 41-17 procedure [29], according to which the effective stiffness of the structure K_e and the yielding force V_y are determined (Fig.5).

For seismic assessment of asymmetric-plan buildings under bi-directional ground motions, it is essential to define load cases to account for the directional combination of load patterns. One option is to use the 100/30 percent rule in each of the major directions. In y orientation which was the critical case, considering 100 percent of the target displacement in the negative direction of the y-axis plus 30 percent of the target displacement in the negative direction of the x-axis resulted in the most critical case for all six models. The comparison between the basic model and the 35 degrees model is shown in Fig. 5. The final failure point and base shear–roof displacement relation for different steel frame models are also presented in Fig.6.

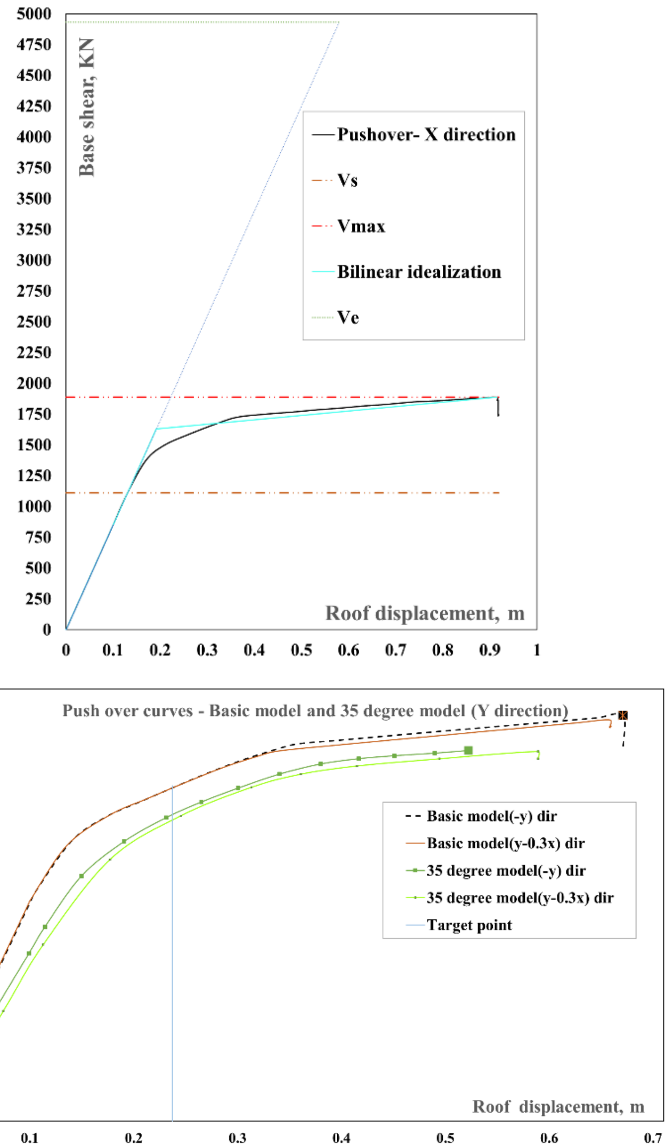


Fig. 5. Bilinear idealization of basic model in x-direction (Top), comparison of the 100-30 rule in the basic model and the 35degree model in the y-direction (Bottom).

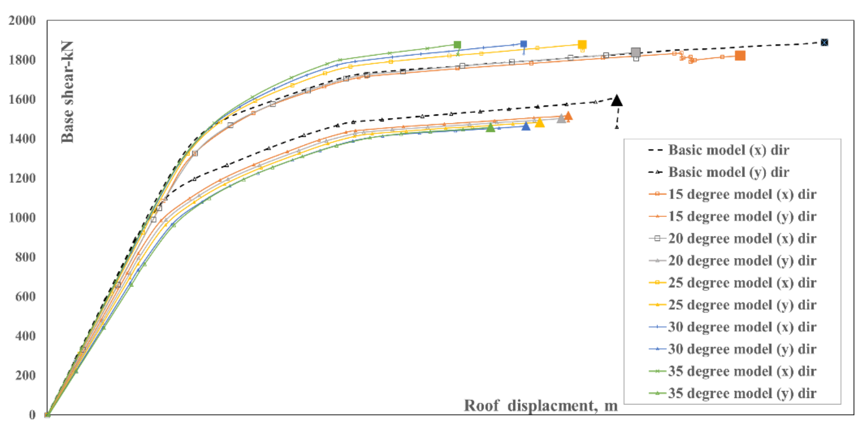


Fig. 6. Pushover curves of all models in x and y-direction.

4. Response modification factor

Pushover analysis with spectral load patterns was carried out to obtain over strength, ductility, and response modification factors. According to the definition of the R factor provided by Chopra and Goel [30], a bilinear force-deformation relationship of an inelastic system and the corresponding elastic system should have the same area under their pushover curves. After the idealization of the bilinear relation, the maximum base shear seismic demand for elastic response (V_e) was calculated. The base shear corresponding to the first yield point of the system (V_s), the maximum base shear of the inelastic system (V_{max}) and the yield base shear of the overall system (V_y) were defined using the pushover curves. The ductility factor (R_μ), is defined by Eq. (2) and the overstrength factor (Ω_0), that is the proportion of the yield base shear to design base shear was calculated [31]. In this case, the response modification factor is determined as the product of the overstrength factor and the ductility factor and it is calculated in Table 4 for all models. It is evident in Table 4 that as the irregularity increases, the ductility capacity and the corresponding modification factor decrease. For example, the predicted R value for the regular structure in the y-direction is 8.83, while for the structure with extreme irregularity (35° model) the predicted R value is 5.19. This indicates around a 40% reduction in R value due to the irregularity effects. Although this reduction in ductility capacity of the structure may not be an issue in a design level earthquake, this can be an important issue when the capacity of the structures is evaluated at higher seismic intensity levels such as those leading to structural collapse.

$$R_\mu = \frac{V_E}{V_y} = \frac{D_E}{D_y} \tag{2}$$

Table 4. Response modification factor in both principal axes.

Models	X- direction			Y- direction		
Basic	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.92	3.03	8.83	2.35	2.90	6.83
15°	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.82	2.75	7.78	2.13	2.64	5.63
20°	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.75	2.52	6.93	2.08	2.61	5.44
25°	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.70	2.39	6.45	2.00	2.58	5.16
25°	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.60	2.25	5.85	1.92	2.53	4.85
35°	Ω_0	R_μ	R	Ω_0	R_μ	R
	2.48	2.09	5.19	1.99	2.43	4.82

5. Nonlinear Time History analysis (NTH)

The NTHs were performed accounting for both geometrical and material nonlinearities, using the Hilber-Hughes-Taylor alpha method [32] with alpha, beta and gamma coefficients equal to 0, 0.25, and 0.5, respectively. A time step of 0.01s for the direct integration method and damping ratio of $\xi = 5\%$ was considered. Discrete concentrated hinge models were used with nonlinear behavior of the kinematic hysteretic type.

This study conducts NTH under orthogonal pairs of horizontal ground motions, using seven ground motion records selected from the PEER database[33]. Table 5 shows the characteristics of the selected records. These records are selected on the basis of the near unity scale factor with a maximum acceptable scaling factor of 2.75, when matched to the target response spectrum [25]. The ground motion scaled combined response spectra along with the average response spectrum are shown in Fig. 7. All records are scaled for the periodic range between 0.13 to 1.1 seconds corresponding to 0.2 and 2 times the fundamental period of the structural models considered, respectively.

To further explore the influence of the frequency content of ground motions on structural response, the predominant period (T_p) of the ground motion records was chosen as the main parameter. The Fourier transform decomposes a signal into its constituent frequency components. As a result, the dominant frequencies and periods of earthquakes were extracted by referring to the Fourier spectrum. The set of ground motions selected was classified into two groups: a) short-period predominant records, b) long-period predominant records. This classification is shown in Table 6.

For instance, "Darfield " record in group “a” of Table 6, affected each model differently regarding the distribution of drifts over the building height. Higher modes contribution in the 15 degrees model, with shorter higher mode periods, leads to a decrease in the middle story drift ratio. In this condition, the contribution of the higher modes to the seismic response is smaller in the 25, 30 and 35 degrees models (Fig.8).

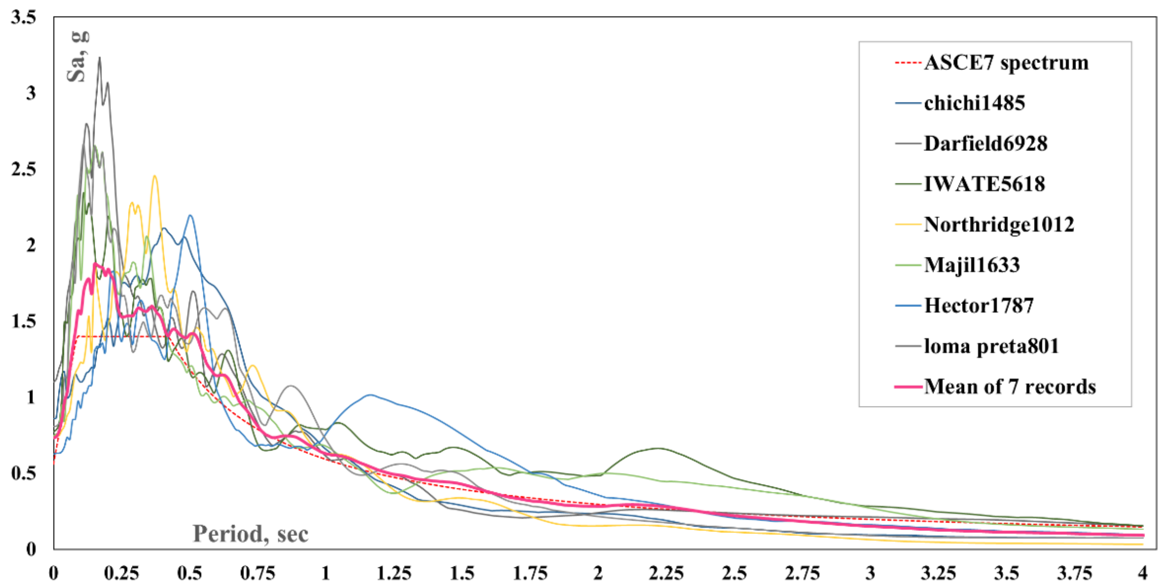


Fig. 7. Scaled combined response spectrum of records, mean and ASCE-7 [25] spectrum.

Table 5. List of records used for NTH [33]

Earthquake Name	N#	5-95% time (sec)	Year	Station Name	M	Mechanism	Rjb (km)	Vs30 (m/sec)
"Loma Prieta"	801	10.1	1989	"San Jose	6.93	R- Oblique	14.18	771.77
"Northridge"	1012	10.8	1994	"LA 00"	6.69	Reverse	9.87	706.22
"Manjil "	1633	29.1	1990	"Abbar"	7.37	strike slip	12.55	723.95
"Hector Mine"	1787	11.7	1999	"Hector"	7.13	strike slip	10.35	726.00
"Chi-Chi "	1485	11.3	1999	"TCU045"	7.62	R- Oblique	26	704.64
"Iwate "	5618	22.6	2008	"IWT010"	6.9	Reverse	16.26	825.83
"Darfield "	6928	12.9	2010	"LPCC"	7	strike slip	25.21	649.67

Table 6. Classification of records

	Earthquake Name	T- predominant(sec)
a)	"Darfield "	0.22
	"Northridge"	0.23
	"Loma Prieta"	0.61
	"Chi-Chi "	0.64
b)	"Hector Mine"	1.15
	"Iwate "	2.20
	"Manjil "	*

*"Manjil" record has multiple predominant periods, including long periods

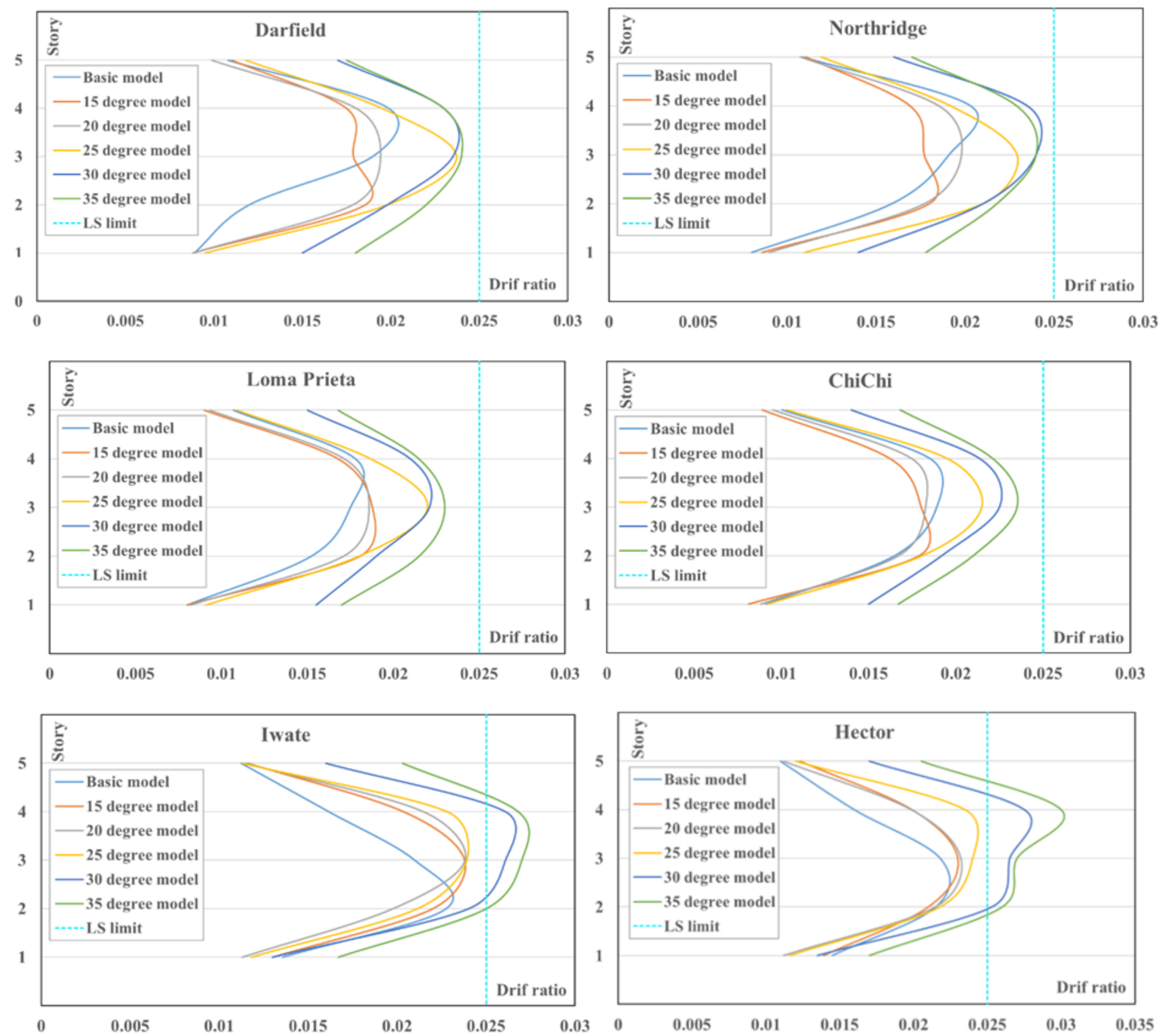


Fig. 8. Inter-story drift ratios of Group “a” of records for all models and the code drift limit.

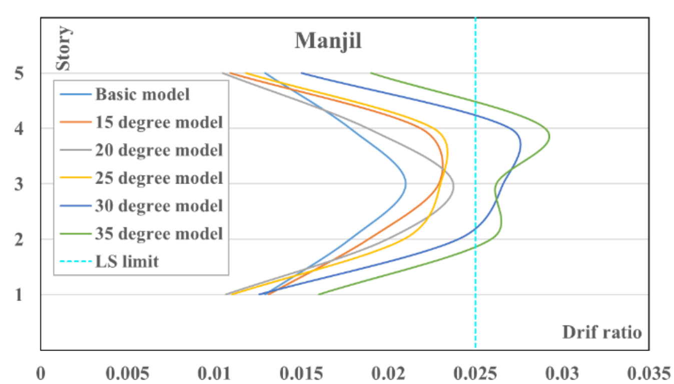


Fig. 9. Inter-story drift ratios of Group “b” of records for all models and the Life safety limit.

On the other hand, "Hector Mine", "Iwate " and "Manjil" (Fig.9) records, with longer predominant periods, suppress higher modes for the basic model, 15 and 20 degrees models so that they mainly vibrate at their first modes. However, the 30 and 35 degrees models will be affected by the higher modes with longer periods (close to 1.0 sec.) [34]. This indicates that the response of such irregular structures can be sensitive to the characteristics of the records used in structural analysis and care should be taken in selecting appropriate ground motion records for analysis.

To calculate R-factor using dynamic analysis, linear time history analysis was also carried out. The R factor was obtained using the ratio of the base shear obtained using linear and nonlinear time history analyses [35]. The results indicate that the ductility demands have increased in the flexible edges of the plan (i.e., frame A and frame 1) [36].

6. Results

In Fig. 10, at the failure point of the structure, the percentage of the plastic hinges created in different models at different performance levels (e.g., range of immediate occupancy (IO) to life safety (LS) and range of life safety to collapse prevention (CP)), derived from pushover analysis have been shown. It is evident in Fig. 10 that as the irregularity of structures increases, the number of plastic hinges in the structure exceeding the LS criteria increases. This is due to the concentration of seismic demands in a few elements in irregular sutures that can cause the failure of such elements before the structure can attain the intended performance levels.

In Fig. 11, the axial force of the perimeter columns, one at the intersection of the orthogonal frames (C5) and the one at the intersection of the non-orthogonal frames (C1) were investigated for all cases. It is observed that as the irregularity level increases, axial forces in both columns increase.

The residual deformations of structures can be effectively utilized to evaluate the post-earthquake performance level of steel structures. The mean residual displacement of the roof is presented in Fig.12, and the residual displacement for each record is shown in Fig.13. It is evident in Fig. 12 that by increasing the irregularity of the models, larger residual deformations are observed. This is again due to larger plastic deformations concentrated in a few elements in the case of irregular structures. According to Fig. 13, the trend is almost similar for different ground motion records used in the analysis. The effects of residual deformations are more pronounced in the y-direction, since the non-parallel irregularities considered in this study had larger impacts on the response of the structures in the y-direction.

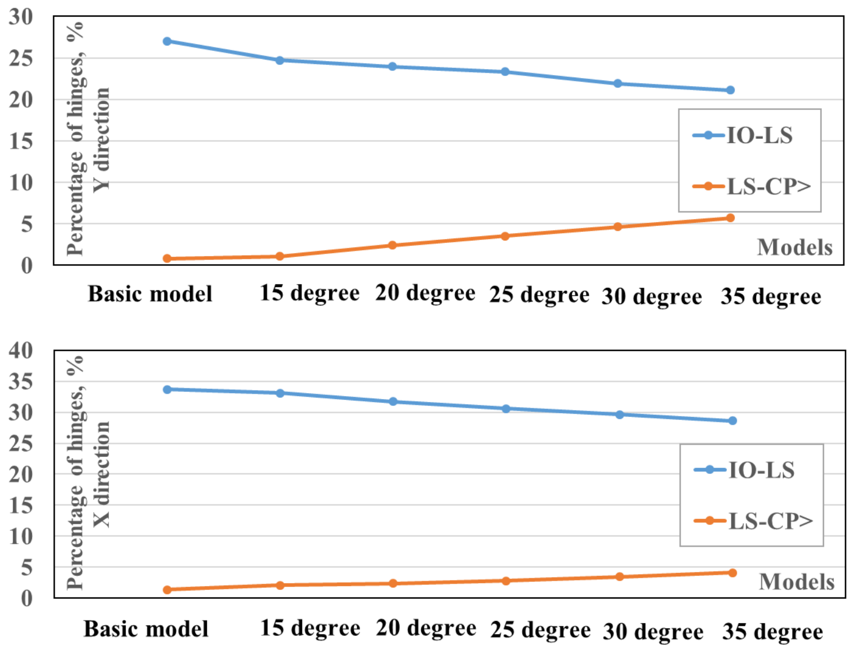


Fig. 10. The percentage of plastic hinges in IO to LS limit states and in LS to CP or above limit states for y-direction (Top) and x-direction (Bottom).

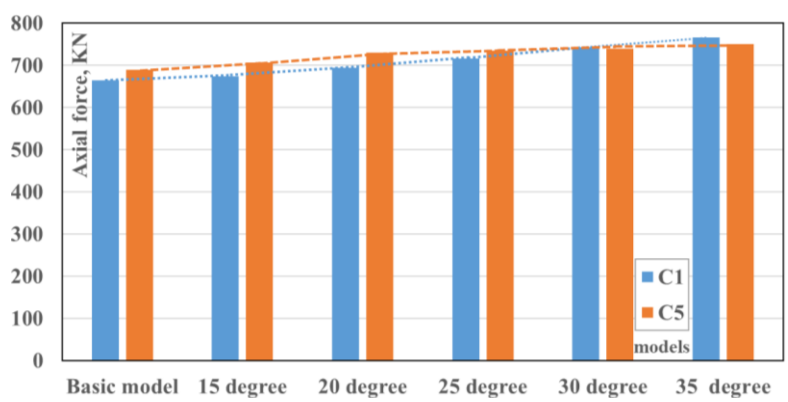


Fig. 11. Mean Axial force of all models using seven ground motion records (column C1 and C2 in the first story, shown in Fig.1. All columns have the same cross-section).

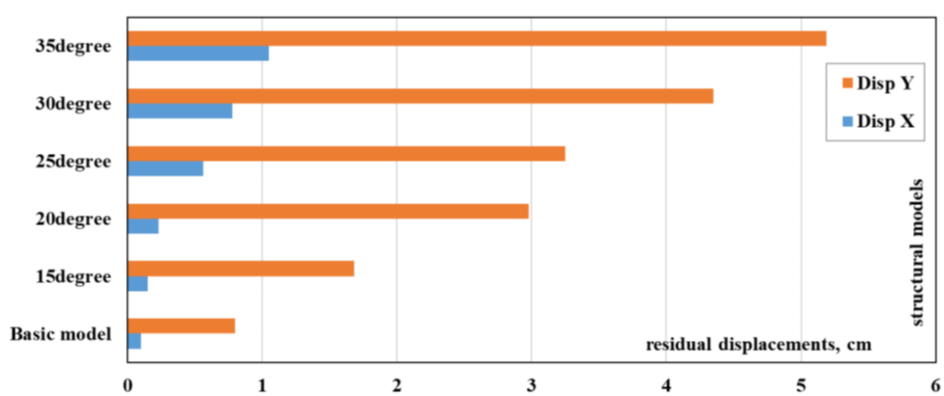
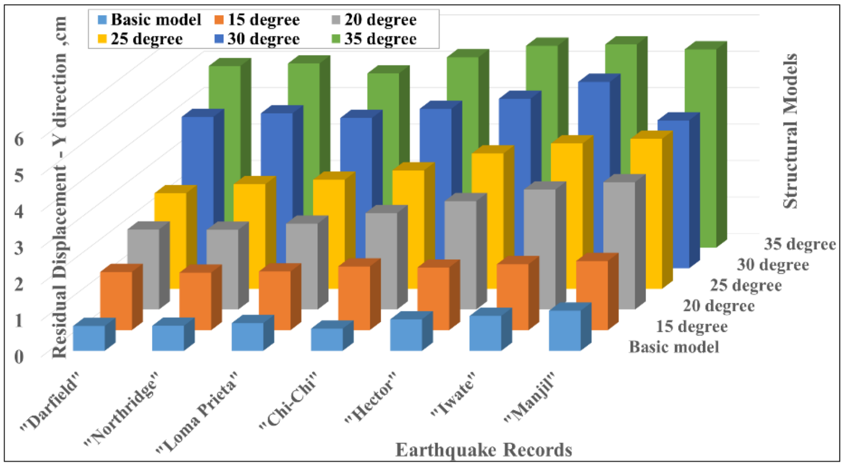


Fig. 12. Mean residual displacements of the roof in both principal directions of the six models.



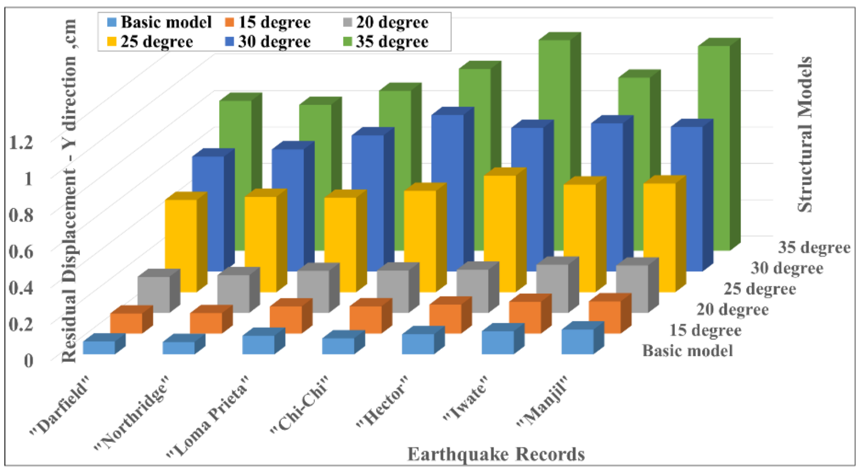


Fig. 13. Roof residual displacement in each model for different records in x-direction (Top) and y-direction (Bottom).

Fig.14 demonstrates all response modification factors (R) from six different pushover analyses required in the derivation of ductility factor, R_μ and overstrength factor, Ω_0. In addition, R factors calculated from nonlinear time history analysis are shown in table 7. As explained before, the R factors decreased as the non-parallel irregularity in the structural models increased. Similar trends were also observed when the NTH analysis was used. This reduction in the ductility capacity of irregular structures can affect the behavior of such structures, especially in the case of earthquakes with larger intensities. Currently, in the codes, similar R factors are used for structures with different regularity levels that can adversely affect the capacity of such structures, especially near collapse.

The maximum inter-story drift ratios in all floors of all six models derived from ESM, Pushover and NTH analysis are presented in Fig.15. A comparison of different analysis methods in Fig.15 reveals that while the use of the ESM and Pushover analysis may be sufficient for structural models with non-parallel irregularity less than 25 degrees, significant deviations of the results from elastic and inelastic analyses are observed for models with larger irregularity (i.e., angles larger than 25^{o}).

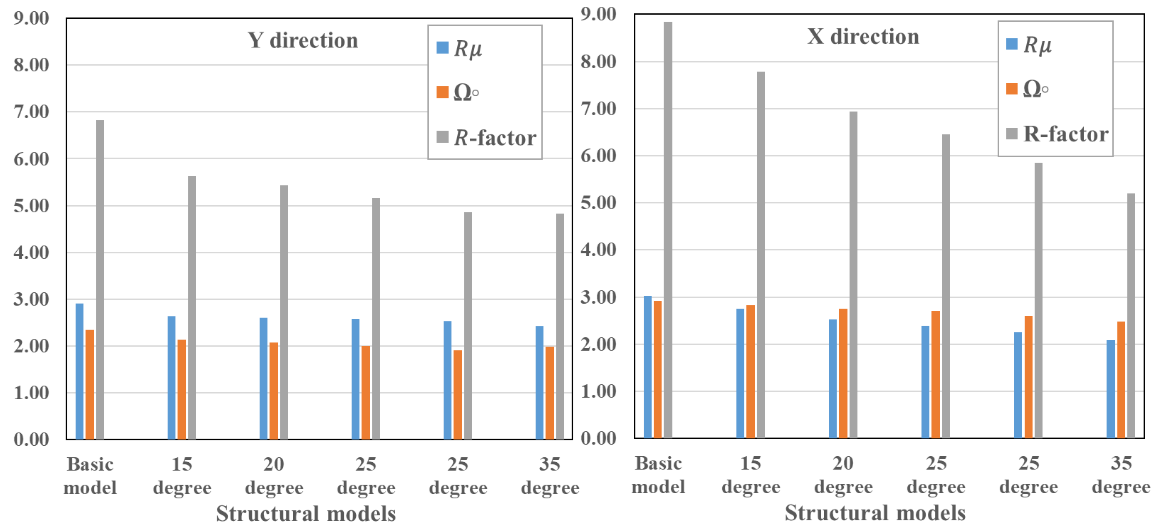


Fig. 14. Response modification factor, R_μ and Ω_0 factors obtained from Pushover analysis in both directions.

Table 7. Comparison of R-factor in Pushover and NTH

Models	X- direction		Y- direction	
	PUSHOVER	NTH	PUSHOVER	NTH
Basic	8.83	8.56	6.83	6.68
15◦	7.78	7.7	5.63	6.05
20◦	6.93	7.26	5.44	5.61
25◦	6.45	6.81	5.16	5.27
30◦	5.58	6.44	4.85	5.09
35◦	5.19	5.9	4.82	4.89

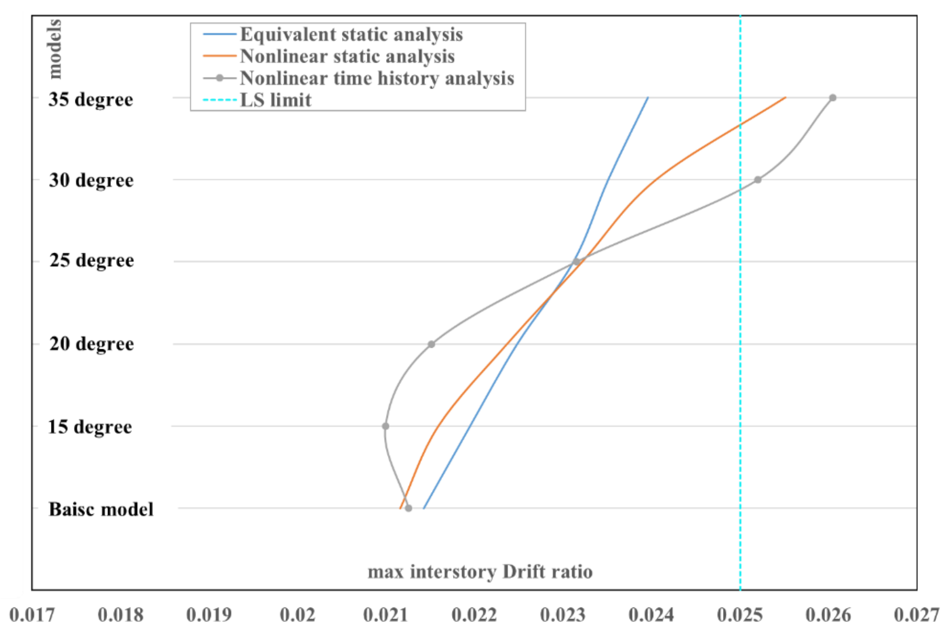


Fig. 15. Maximum inter-story drift ratio in all six models (comparioson between methods of analysis).

7. Conclusions

In this study, five-story steel special moment frames with non-parallel systems irregularity were designed according to ASCE 7-16 [25]. Nonlinear time history and pushover analyses were performed along with the conventional analysis methods such as equivalent static method (ESM) and elastic dynamic analysis used in design codes to evaluate the seismic behavior of the structures. The conclusions and recommendations of this study are as follows:

When the degree of torsional irregularity was increased, smaller response modification factors (between 5.0 to 8.0) were obtained, and the ductility factors also decreased. The suggested R factor of ASCE 7-16 provisions [25] recommends using an R factor of 8.0 for the steel special moment frames. As a result, for some highly irregular cases, the use of the R factors in the codes may lead to unsafe designs. More research is needed to investigate this issue.

In both directions, the percentage of plastic hinges in IO-LS limit states decreases, and the percentage of plastic hinges in LS-CP> limit states increase, as the irregularity of the structural models increases. When the degree of irregularity increases, the plastic hinge formation concentrates in one edge of the plan, and just those fewer hinges contribute to the overall ductility of the structure. In contrast, all hinges almost uniformly add to the ductility capacity of the structure in the basic regular model.

Lateral force induced by earthquakes in irregular models has a significant effect on the perimeter columns by adding to their axial force that can decrease the capacity of columns. Columns show an increase in their axial force, when the models become more irregular. The axial force of Column (C1) appears to increase more due to being located at the intersection of non-orthogonal frames.

Higher mode effects in plan caused some models with a smaller degree of irregularity to have smaller maximum drifts and caused others with higher irregularities to have greater drifts in comparison with drifts obtained from pushover analysis. As a result, code provisions appear to be unsafe in defining torsional irregularity of non-parallel systems. More investigations are required.

Post-earthquake conditions are more severe for irregular models, as the mean residual displacement of the roof has increased by increasing the irregularity in the models.

It is noted that the conclusions made here may be limited to the cases studied in this paper. More research is needed to further investigate the effects of non-parallel irregularity for structures with different configurations and different lateral force resisting systems.

Symbols

E	Modulus of Elasticity, Pa
g	Gravitational acceleration, m/s2
I	Moment of inertia, m4
π	Pi number
$p-\Delta$	P Delta
θ_y	Plastic rotation
θ	Stability factor
δ	Maximum drift ratio

δ_t	Target displacement
Ω_0	Overstrength factor
ξ	Damping ratio
*	Description for Tables

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