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Seismic acceleration amplification factor under fixed and pin supported RC frame building

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Assessing non-structural elements' vulnerability to acceleration has relied on peak horizontal floor acceleration. Given the dynamic nature of structures, estimates for forces acting on elements like mechanical, electrical, or architectural components have been crucial. The amplification factor for these non-structural components has been affected not just by building height but also by the structural support conditions. This study has examined five distinct building models spanning 2, 4, 6, 8, and 10 stories. The models have been analysed using the nonlinear time history analysis method under various near-field Chi-chi earthquake scenarios (ranging from 0.01g to 0.067g, 0.067g to 0.2g, and 0.2g to 0.32g). This analysis has aimed to determine the acceleration amplification factor (Ω), defined as the ratio of peak floor acceleration to peak ground acceleration (PGA), for both fixed and pinned support conditions. A disparity has emerged after comparing the actual acceleration amplification values under fixed and pinned support conditions with those from previous models. While some models have performed adequately under fixed support conditions, others have yielded accurate results under pinned support conditions. Consequently, there has been no singular formula capable of effectively accommodating both support conditions across the varying ranges of ground motion.

Keywords: Acceleration amplification factor, Non-structural components, Peak floor acceleration, Time history analysis

1 Introduction

Among the most devastating natural calamities are earthquakes. Taiwan is found alongside the thrust amid the Eurasian and Philippine Sea Plates. This region is the world's most sensitive area, where frequent earthquakes occur[1]. The Chi-chi earthquake in 1999 affected more than 8000 buildings in various ways. This earthquake destroyed the brick masonry structures and collapsed the engineering design of the reinforced concrete (RC) frame structures. A large percentage of damage to non-structural components was also observed[2]. Limited research has been done to understand the seismic nature of non-structural components compared to the main structural elements of the structures.

From the past earthquake investigations, it has been acknowledged that most of the losses (in terms of death, injury and economy) in the structure are due to the direct or indirect corollary of the damages of the secondary element[3]. Furthermore, different seismic design techniques can prevent the deterioration of primary structural components during several earthquakes. Still, no direct provision exists to control the damage to the non-structural elements. Sometimes, the destruction cost of secondary components is more than 75% of the initial construction cost[4-6]. The non-structural components can be flexible or rigid depending on the working conditions. These components' damping properties are lower than those of the RC building. Resonance occurs when the NSCs and RC building frequency are matched, further amplifying the damage in secondary elements[7,8].

The secondary components interlink the structures' floor, roof, and wall, and they are also affected by seismic action[9]. These can be classified based on the acceleration-sensitive subsystem and the storey-drift-sensitive subsystem. Many studies have been done to investigate the behaviour of storey drift as well as acceleration-sensitive non-structural components, proposing some guidelines[10-13]. Based on the minimum equivalent force method for new buildings and improvements in the existing buildings, some provisions were recommended for the design of non-structural components[14-16]. These provisions were partially based on the analytical studies[17,18] and partly based on the data recorded by instrumental

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buildings[19-21]. However, some structural engineers criticised these provisions. The above studies show that the current provisions do not sufficiently assess the variation of floor accelerations along with the height, neither for the buildings nor for the components[22-24].

UBC code[25] evaluated that the amplification factor depends only on the height of the structure, and its maximum value is 4. However, the ASCE code provided the maximum value of the amplification factor as 3, and the IITK[26] model remarked that the maximum acceleration amplification value for the RC structure is 2 and is dependent on the normalising height of the building. Akhalkhi[27] presented the formula for the acceleration amplification factor that depends on both the height and the fundamental time period of the structures. However, Fathali[28] proposed that the amplification factor depends on the fundamental period and the range of the ground motion of the structure.

This paper investigates a comparison of the structural acceleration amplification factor under two support conditions: fixed and pin support conditions of the buildings. The deep and shallow foundations represent the fixed and pin support conditions. To determine these conditions, five different heights of the buildings are considered. These models are analysed using the linear time history method with a varied range of ground motions. On comparing the actual amplification factor with different proposed models, it is found that no single formula is present which gives good results for both support conditions. The comparison of these models with different support conditions, applicable only to reinforced concrete frame structures, is done with the assumption of non-structural components (NSCs) connected rigidly with the main structure.

2 Materials and Methods

2.1 Existing models

2.1.1 Uniform building code

The lateral seismic design force (F_p) defined in the UBC 1997[25] for non-structural components is as follows,

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_n}\right) W_p \quad \dots (1)$$

Here, the defined structural amplification factor is denoted by ap, and the response modification factor is denoted by Rp, respectively. hx is the height of the part, and hn represents the total height of the structure from the base. The important factor of the NSCs is denoted by Ip, and Wp represents the total weight of the NSCs. The coefficients a_p and R_p range between 1.0 and 2.5 and 1.0 and 4.0, respectively. The term floor acceleration amplification factor is represented by $\left(1 + 3\frac{h_x}{h_n}\right)$ in Eq. (1) of the secondary elements h_n. Furthermore, the lateral seismic design force F_p lies between

$$0.7 C_a I_p W_p < F_p < 4.0 C_a I_p W_p \quad \dots (2)$$

To evaluate the seismic design force for the elastic part of the structure, both a_p and R_p are considered as 1.0.

2.1.2 ASCE

According to ASCE/SEI 7-05[16], clause 13.3.1, the lateral seismic force on the NSCs is given as:

$$F_p = 0.4 S_{ds} a_p \left(\frac{I_p}{R_p}\right)\left(1 + 2\frac{z}{h}\right) W_p \quad \dots (3)$$

$$0.3 S_{ds} a_p W_p \leq F_p \leq 1.6 S_{ds} a_p W_p \quad \dots (4)$$

Here, the seismic design force is denotes by F_p, S_{ds} is represents as the short-period spectral acceleration, and the component amplification factor for a range from 1.0 to 2.5 indicates by a_p, z represents the height of the component from the base, and the total height of the building from the base is denoted by h. The important factor of the component is denoted by Ip, and Rp denotes the response modification factor, which is based on the energy absorbed by the NSCs, and W_p represents the weight of the NSCs. The term floor acceleration amplification factor is represented by $\left(1 + 2\frac{z}{h}\right)$ in Eq. (3) of the non-structural components.

2.1.3 IITK-GSDM

The seismic design of the reinforced concrete frame structure, as per IS 1893: 2002[29], did not provide information for the non-structural components of the structure. According to section 7.12.2, the element's weight multiplied by the horizontal acceleration coefficient allows for the design of the components and the main structure to be five times larger. The Indian Standard earthquake code 1893:2002[29] provisions for secondary structural elements are woefully insufficient. The IITK-GSDMA[30] proposes seismic prevention of NSCs of the buildings. The given equation of the floor acceleration amplification based on the height of the building and is defined by term $\left(1+\frac{z}{h}\right)$, where z represents the height of the NSCs, and h denotes the height of the building from the base of the structure. It was found that when z and h are equal, the maximum amplification of the structure's NSCs is equal to 2. Peak ground motion and peak floor acceleration are assumed to be linearly related in the aforementioned equations.

2.1.4 Akhlaghi and Moghadam

Akhlaghi[27] assessed the seismic analysis of the rigid acceleration-sensitive secondary elements connected with the main structure elements. It was observed that the peak horizontal acceleration of the floor or roof is equal, along the height of the buildings, regardless of the behaviour of the rigid NSCs. Thus, equations (5) based on the fundamental time period of the structures for the floor acceleration amplification factor (Ω) formulae are as follows:

$$\Omega = 1 + (\alpha - 1)\left(\frac{h_i}{h_n}\right) \qquad \text{... (5)}$$

Where the ratio of peak horizontal floor acceleration to PGA is denoted by Ω, h_i represents the height of the storey, and h_n is the total height of the building from the base, and the fundamental time period dependent factor is denoted by α, which is given by:

$\alpha=3$ when $T < 0.5$

$\alpha = \frac{2.5}{T^{1/4}}$ when $0.5 \leq T \leq 1.0$

$\alpha = \frac{2.5}{T^{3/4}}$ when $T > 1$

here, the defined fundamental time period of the building is represented by T.

2.1.5 Fathali and Lizundia

Fathali (2011)[28] observed that the floor acceleration amplification factor is not only governed by the height and fundamental period of the structures but also depends on the magnitude of the earthquake, and based on it, the non-linear Eq. (6) is given as:

$$\Omega = 1 + \alpha\left(\frac{z}{h}\right)^{\beta} \qquad \text{… (6)}$$

where z represents the height of the NSCs and h denotes the storey with respect to the base. α and β are parameters that depend on the fundamental time period and severity of ground motion of the structure, respectively. (Inserted Table 1 and Table 2)

Table 1 — Values of the ‘α’ parameter for seismic design of NSCs in new constructions.

Natural Period (second)	PGA < 0.067 g	PGA 0.067 ≤ < 0.20 g	PGA ≥ 0.20 g
$T_a < 0.5$	2.12	1.93	1.75
$0.5 \leq T_a < 1.5$	2.61	1.55	1.01
$T_a \geq 1.5$	2.52	1.53	0.50

Table 2 — The values of the ‘β’ parameter for seismic design of NSCs in new constructions.

Natural Period (second)	PGA < 0.067 g	PGA 0.067 ≤ 0.20 g	PGA ≥ 0.20 g
$T_a < 0.5$	0.78	1.25	0.92
$0.5 \leq T_a < 1.5$	1.16	0.75	0.69
$T_a \geq 1.5$	1.64	1.65	3.00

2.2 Buildings configuration

This study investigates the amplification factor for five RC moment-resisting frame (MRF) buildings spanning 2, 4, 6, 8, and 10 stories under both fixed and pin support conditions on hard soil type. The selected storey heights for all calculations are 4m and 3.4m. The considered two-dimensional models for fixed support conditions are depicted in Fig. 1, and the dimensions of beams and columns are detailed in Table 3. The fundamental time period of the buildings ranges from 0.1 to 1.5 seconds, with the damping ratio of 5% as per IS code[29]. All models are analysed using the finite element analysis-based software ETABs[31].

2.3 Ground motion selection

The study considers ground motion intensity ranging from 0.01g to 0.32g, with a specific focus on lower accelerations due to the recognition that buildings often collapse at lower accelerations than those typically studied. This range is divided into three categories based on Fathali's[28] classification, which is on the basis of ground motion range 0.01g to 0.067g, 0.067g to 0.2g, and greater than 0.2g. The building model analysis utilises near-field time history data within these ranges. The ground motion data used in the study is taken from the Strong Ground Motion Virtual Data Centre. Specifically, there are 28 recorded ground motion data points considered within the 0.01g to 0.067g range, 29 data points obtained within the 0.067g to 0.2g range, and 24 data points between 0.2g and 0.32g. Comprehensive information on the ground motion data can be found in Tables 4-6.

In these Tables 4-6, PGA represents peak ground acceleration, Tp denotes the time peak acceleration, and T signifies total recorded time period.

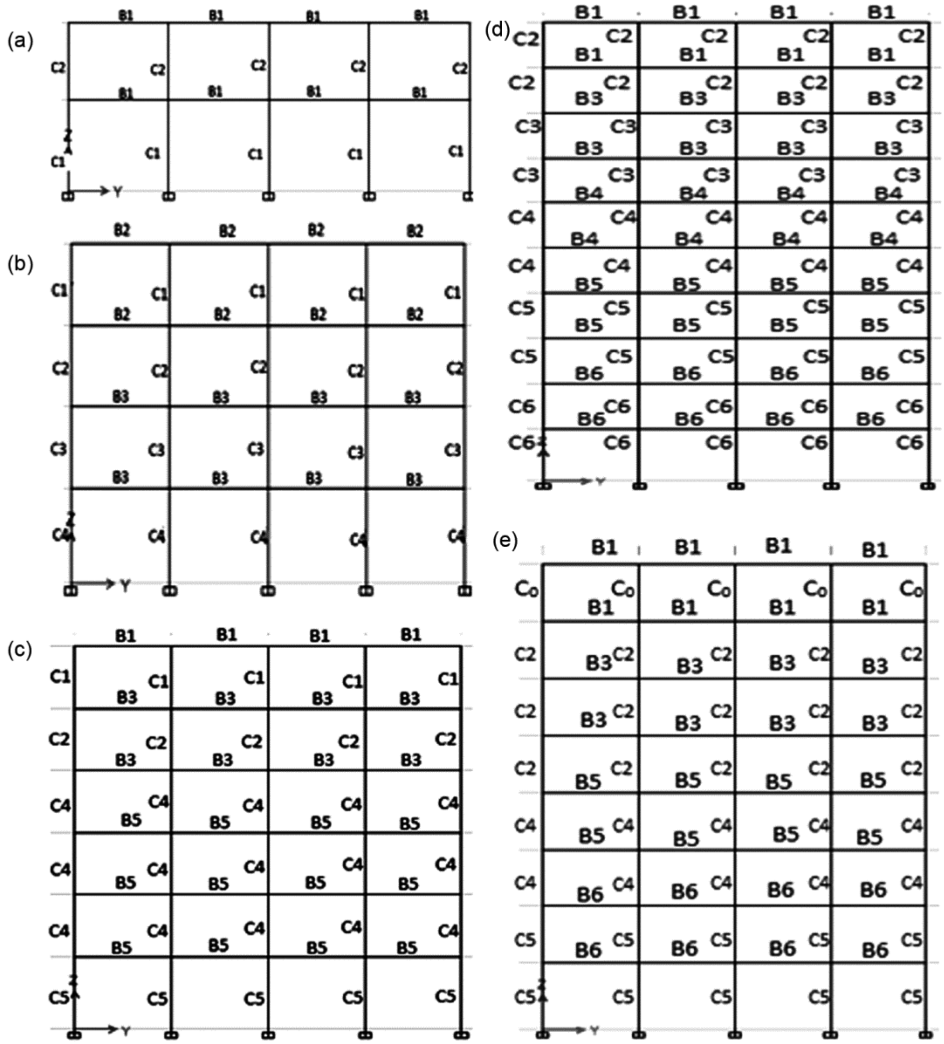

Fig. 1 — Elevation view of the building Models (a) 2-stories building, (b) 4-stories building, (c) 6-stories building, (d) 8-stories building and (e) 10-stories building.

Table 3 — Description of the size of beams and columns of the models.

	Element	Cross-section size (mm)
Beams	B1	300 x 400
	B2	300 x 450
	B3	450 x 500
	B4	450 x 600
	B5	450 x 650
	B6	450 x 675
Columns	C_0	300 x 400
	C1	300 x 450
	C2	450 x 500
	C3	525 x 550
	C4	550 x 600
	C5	600 x 700
	C6	650 x 850

2.4 Analysis with finite element method-based software

The software utilized for analyzing all the reinforced concrete models in the study was based on the Finite Element Method, specifically ETABS version 2016[31]. In the ETABS nonlinear time history analysis, the Wilson Method was employed. This method is a modern numerical integration tool that utilizes an iterative vector superposition approach. It is particularly effective for studying structures with predetermined, confined nonlinearity. The Wilson Method offers significantly reduced processing times compared to previous nonlinear analysis approaches. A linear time history technique was employed for the analysis of different support conditions of the various models, as mentioned by Agrahari *et al.* [33]. A substantial amount of seismic data ranging from

Table 4 — Details of seismic data of ranges 0.01g to 0.67g used in the analysis[34].

Ground motion	T_p (sec)	PGA (g)	T (sec)
Chi-chi 1	16.696	0.066	64.992
Chi-chi 2	17.005	0.057	52.98
Chi-chi 3	15.66	0.044	47.975
Chi-chi 4	16.115	0.047	47.975
Chi-chi 5	17.14	0.0199	45.988
Chi-chi 6	16.112	0.027	45.988
Chi-chi 7	22.875	0.028	63.98
Chi-chi 8	26.69	0.0237	63.98
Chi-chi 9	20.615	0.0372	53.98
Chi-chi 10	15.56	0.066	70.97
Chi-chi 11	16.23	0.0516	70.97
Chi-chi 12	18.42	0.0441	50.98
Chi-chi 13	17.14	0.0352	50.98
Chi-chi 14	18.1	0.0592	60.98
Chi-chi 15	13.73	0.0503	60.98
Chi-chi 16	14.305	0.0467	56.985
Chi-chi 17	16.775	0.0492	56.985
Chi-chi 18	25.205	0.0361	62.98
Chi-chi 19	17.275	0.065	62.98
Chi-chi 20	14.69	0.0662	74.98
Chi-chi 21	17.275	0.066	62.98
Chi-chi 22	15.656	0.0604	59.98
Chi-chi 23	18.694	0.0574	59.98
Chi-chi 24	19.476	0.0511	62.988
Chi-chi 25	20.66	0.0514	62.988
Chi-chi 26	16.004	0.024	49.992
Chi-chi 27	15.872	0.0185	49.992
Chi-chi 28	20.424	0.0439	56.992

0.01g to 0.3g, with a fundamental period of up to 1.5 seconds, was considered during the analysis process.

3 Results and Discussion

3.1 Floor response spectra

Different ground motion data are applied to the model's base for the dynamic analyses of the structures. A damping ratio of 5% is considered to obtain the various models' floor response spectra. A mean response spectrum of the structures is plotted in Fig. 2 for both fixed and pin support conditions, using ground motion ranging from 0.2g to 0.30g. These floor spectra provide information on the acceleration demand acting on the NSCs, connected with the main components having the fundamental time period T.

Fig. 2 illustrates that for a two-storey model, if the frequency content of the NSCs matches the frequency of the main structures, the floor response exhibits up to ten times the base acceleration. This value decreases to up to 6 times for a ten-storey model under fixed support conditions. However, for pin supports, the floor spectra for a two-storey model are 7 times higher than the base acceleration, decreasing to 4.5 times for the ten-storey model. To compare both support conditions, the amplification factor for fixed support is higher than that for pin support conditions. This suggests that the fixed support condition generally produces higher amplification factors than pin support conditions.

Table 5 — Details of seismic data for range 0.067g to 0.2g[34].

Ground motion	T_p (sec)	PGA (g)	T (sec)
Chi-chi 1	17.74	0.1374	60.98
Chi-chi 2	15.93	0.1348	60.98
Chi-chi 3	13.905	0.1217	56.985
Chi-chi 4	15.08	0.1179	56.985
Chi-chi 5	15.99	0.1092	57.975
Chi-chi 6	14.58	0.147	65.045
Chi-chi 7	14.475	0.1215	65.045
Chi-chi 8	16.555	0.1167	70.97
Chi-chi 9	17.265	0.127	70.97
Chi-chi 10	16.81	0.1913	65.045
Chi-chi 11	16.545	0.1565	65.045
Chi-chi 12	15.6	0.1654	65.045
Chi-chi 13	16.0	0.1692	65.045
Chi-chi 14	14.275	0.1439	63.985
Chi-chi 15	14.45	0.1624	65.045
Chi-chi 16	14.73	0.1517	65.045
Chi-chi 17	13.45	0.1322	66.005
Chi-chi 18	16.125	0.1234	66.005
Chi-chi 19	17.235	0.1866	74.98
Chi-chi 20	15.04	0.155	74.98
Chi-chi 21	16.72	0.157	61.98
Chi-chi 22	15.47	0.197	74.985
Chi-chi 23	11.85	0.1344	96.985
Chi-chi 24	16.635	0.1863	96.07
Chi-chi 25	12.685	0.1563	93.985
Chi-chi 26	12.02	0.1883	96.985
Chi-chi 27	10.715	0.1831	99.03
Chi-chi 28	9.66	0.1699	61.99
Chi-chi 29	14.38	0.1829	71.0

3.2 Acceleration amplification models

The profile of amplification factor (Ω) is observed to be non-linear, resembling an S-shape, as the structure's natural period increases. Under fixed support conditions, acceleration amplification values decrease as the structure's height increases. This trend indicates that taller structures experience less ground motion amplification than shorter ones when supported fixedly.

Table 6 — Details of seismic data for ranges 0.2g to 0.30g[34].

Ground motion	T_p (sec)	PGA (g)	T (sec)
Chi-chi 1	8.015	0.2296	47.99
Chi-chi 2	11.94	0.2167	74.985
Chi-chi 3	17.615	02061	86.485
Chi-chi 4	10.04	0.2252	124.06
Chi-chi 5	5.8	0.2779	65.005
Chi-chi 6	15.22	0.2347	68.03
Chi-chi 7	15.39	0.2678	68.03
Chi-chi 8	12.15	0.2818	74.98
Chi-chi 9	19.62	0.2164	77.49
Chi-chi 10	1428	0.2465	63.985
Chi-chi 11	36.02	0.2504	139.98
Chi-chi 12	37.19	0.2021	122.97
Chi-chi 13	37.53	0.2449	149.97
Chi-chi 14	31.98	0.2206	143.97
Chi-chi 15	20.14	0.201	139.98
Chi-chi 16	46.14	0.2515	149.97
Chi-chi 17	33.97	0.2828	149.97
Chi-chi 18	34.05	0.2820	149.97
Chi-chi 19	10.965	0.2656	60.03
Chi-chi 20	47.37	0.2598	149.97
Chi-chi 21	29.39	0.2229	149.97
Chi-chi 22	37.37	0.2270	149.97
Chi-chi 23	31.64	0.201	119.976
Chi-chi 24	36.688	0.2596	119.976

In the considered model of pin support conditions, as the range of ground motion increases (from 0.01g to 0.31g), the amplification value decreases with respect to the normalised height of the buildings. This suggests that as the amplitude of ground motion increases, the amplification effect diminishes with respect to the normalized height of the building. This could imply that taller structures are less susceptible to ground motion amplification under pin support conditions than shorter ones, especially when exposed to higher-intensity ground motions.

3.3 Comparison of amplification models with fixed and pin support conditions

The behaviour of the building model with fixed and pinned support conditions is shown in Fig. 3-5.

In Fig. 3, the amplification factor (Ω) shape is observed to be nonlinear with the height of the building for both support conditions. However, the nonlinearity of the amplification factor is more pronounced in the fixed support condition compared to the pin support condition, particularly for ground motions ranging from 0.01g to 0.067g.

As the fundamental time period of the structure increases from 1 to 1.5 seconds, the performance of the IITK model is satisfactory for fixed support conditions but not for pin support conditions. The UBC model yields conservative results for both support conditions. Similarly, Fathali's[28] model produces conservative results for both support conditions when the fundamental time period ranges from 0.5 to 1.5 seconds. It's worth mentioning that the ASCE code equation demonstrates superior results for fixed support when the fundamental time period of the structure is up to 0.5 seconds. However, its findings become conservative beyond that range. Conversely, the ASCE formula yields satisfactory results for all models with ground motions ranging from 0.01g to 0.067g and buildings with natural time periods ranging from 0.1 to 1.5 seconds. The Akhlaghi[27] model performs better for fixed support conditions when the fundamental time period of the building ranges from 0.10 to 1.5 seconds. However, this model is not helpful for pin support conditions when the seismic motion range is between 0.01g and 0.067g.

The shape of the actual amplification value concerning the normalized height is nonlinear, shown in Fig.4 for the seismic data range, which is taken from 0.067g to 0.2g. Under fixed support conditions, the amplification value decreases as the normalized height of the buildings increases. However, for pin support conditions, the amplification value remains approximately the same with any increase in the normalized height of the buildings.

The IITK model must yield satisfactory results in fixed and pin support conditions within this ground motion range. The UBC model produces conservative results for both support conditions when the seismic range is taken from 0.067g to 0.2g. The ASCE code formula exhibits good results when the natural time period of the building is less than 0.5 seconds. However, for natural time periods higher than 0.5 seconds, its results become conservative for the ground motion range taken from 0.067g to 0.2g. The Akhlaghi[27] model provides satisfactory results for fixed support conditions when the natural time period of the buildings is up to 1.5 seconds, while for pin support conditions, this model is unsuitable. Fathali's[28] model yields unsatisfactory results for both support conditions when the natural time period of the buildings is less than 0.5 seconds. However, as the fundamental time period of the buildings increases, their results become adequate for ground acceleration ranges between 0.067g and 0.2g.

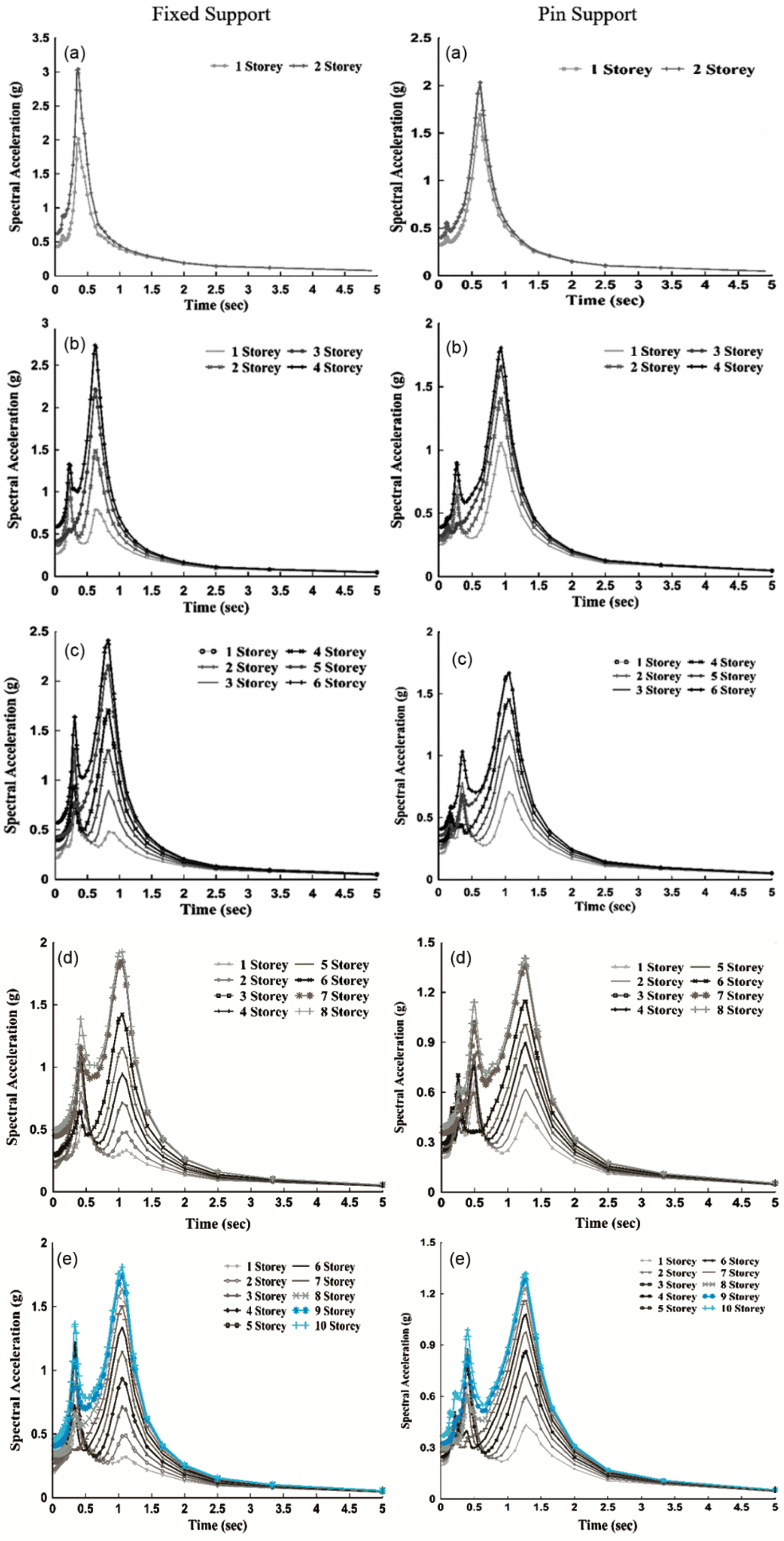

Fig. 2 — Floor response spectra of the different models for fixed (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories building and for pin support conditions (a) 2-stories building, (b) 4-stories building, (c) 6-stories building, (d) 8-stories building and (e) 10-stories building.

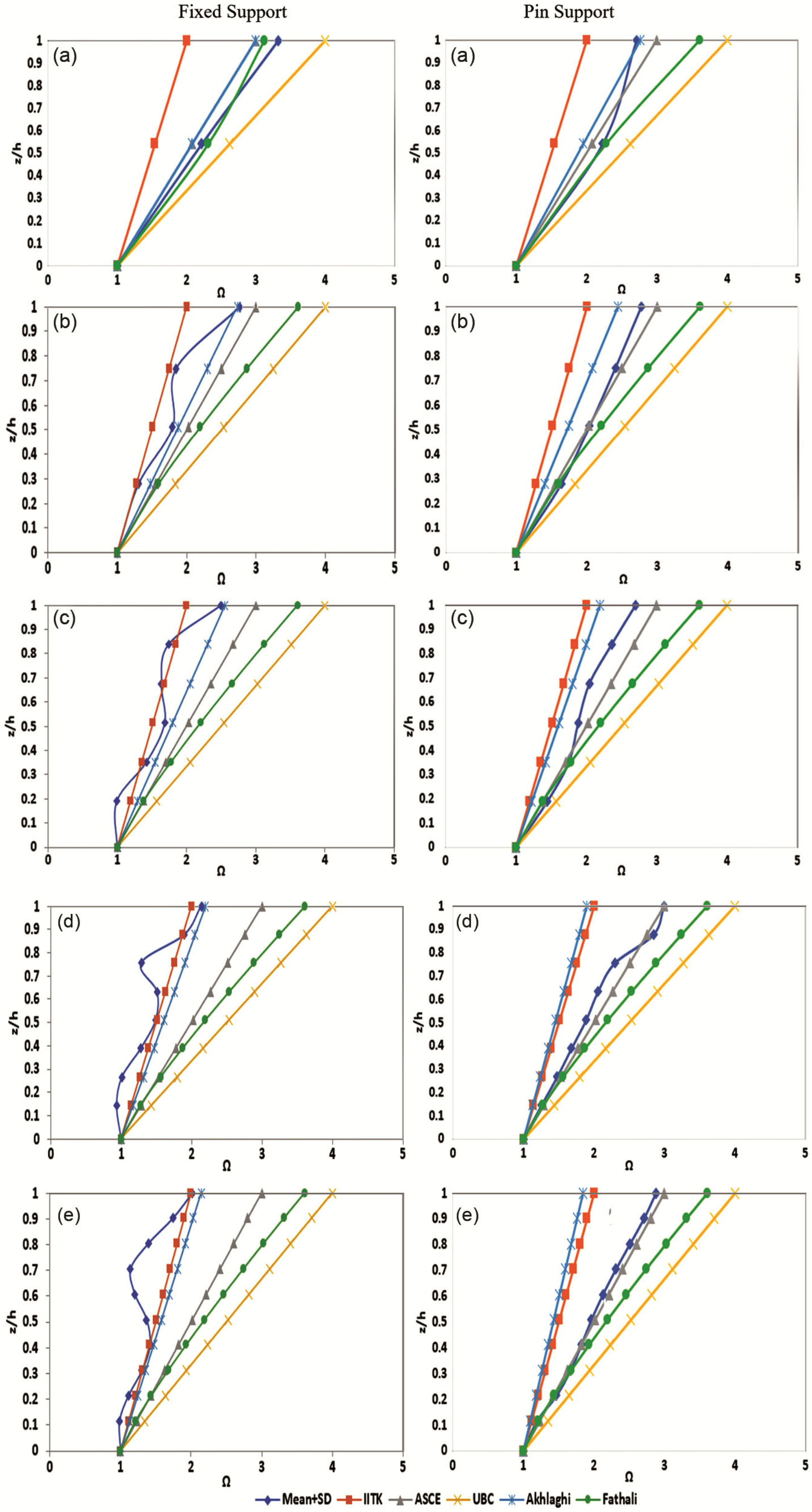

Fig. 3 — Comparison of the models for fixed (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories building and pin support conditions (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories building with acceleration 0.01g to 0.067g having 2,4,6,8 and 10 stories.

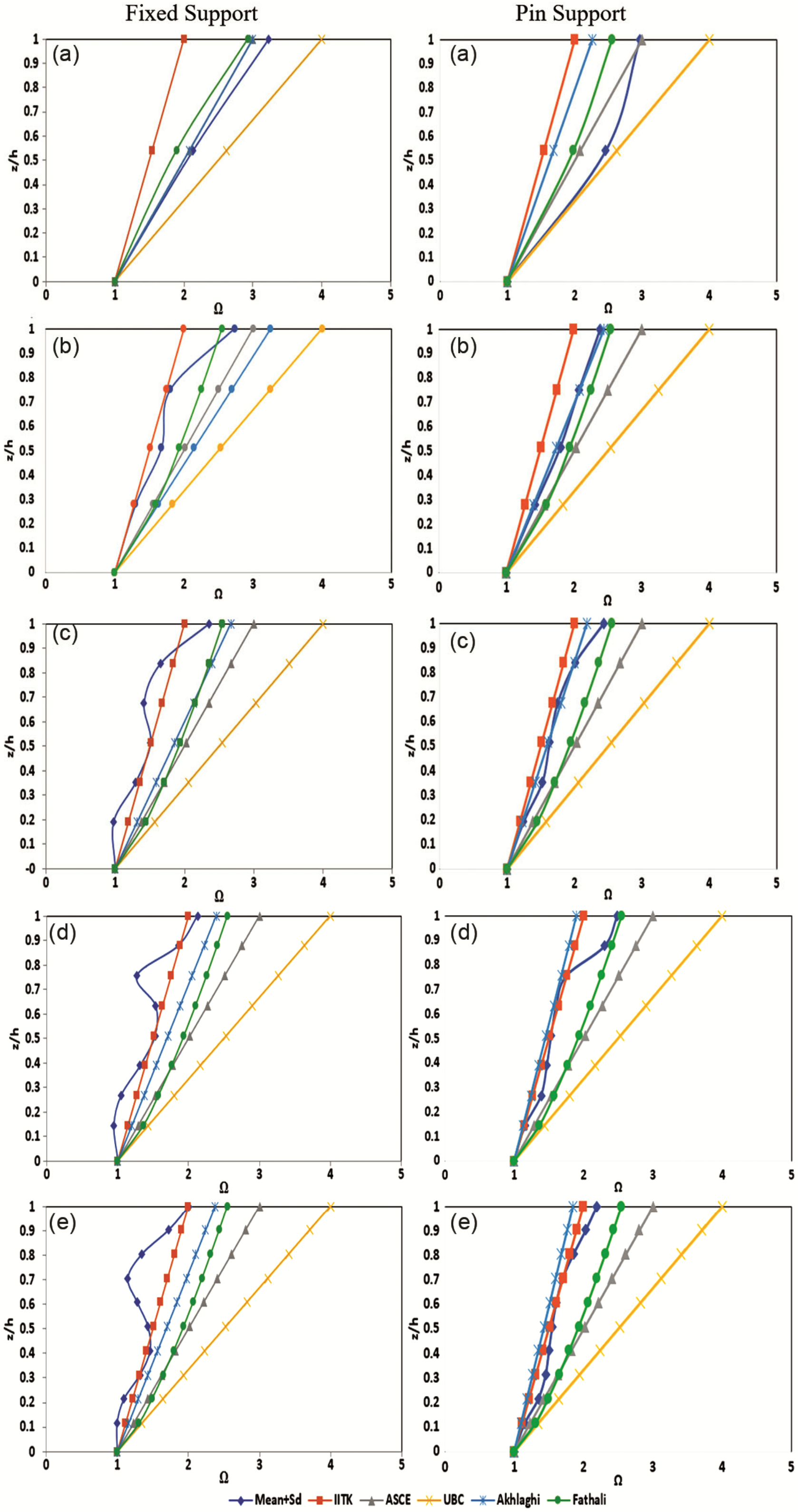

Fig. 4 — Comparison of the models for fixed (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories building and pin support conditions (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories and pin support conditions with acceleration 0.067g to 0.2g

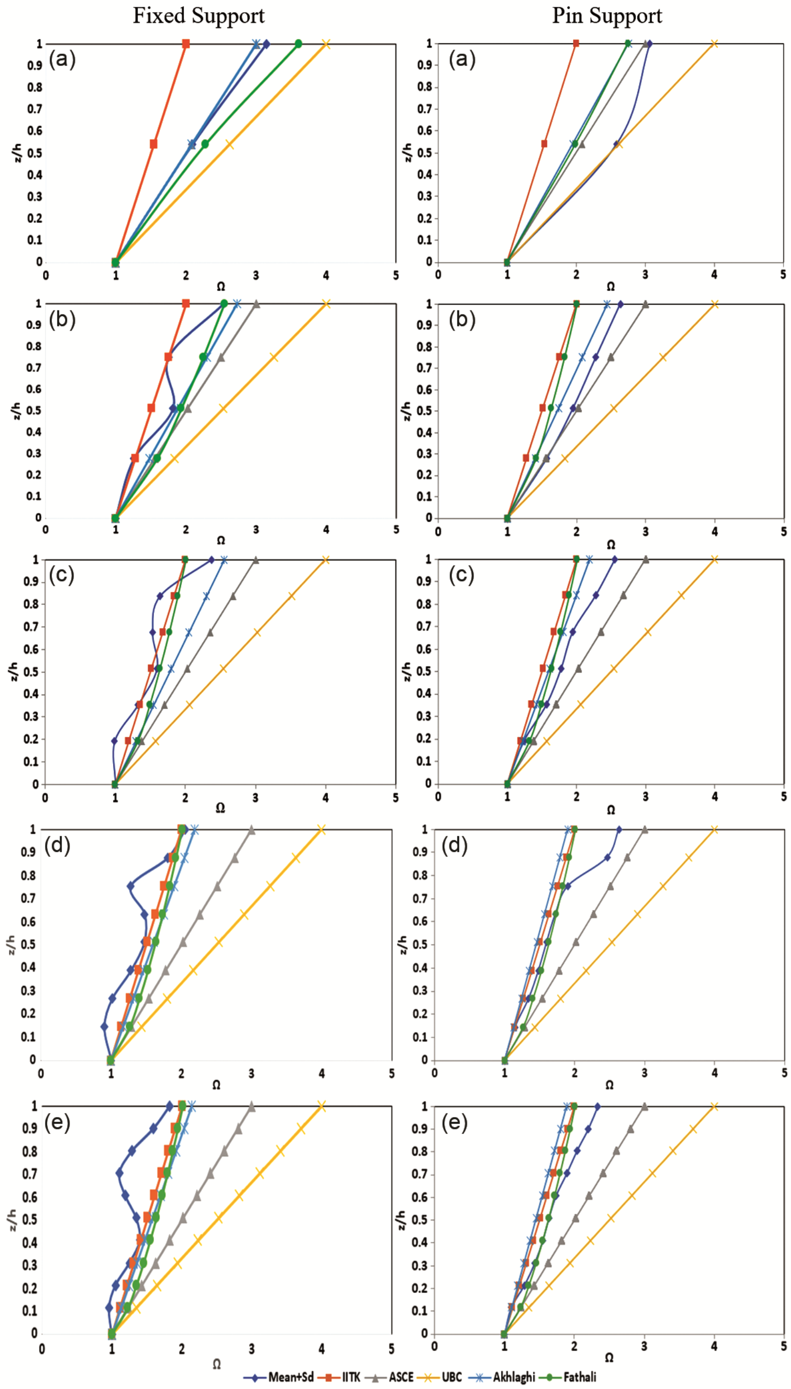

Fig. 5 — Comparison of the models for fixed (a) 2-stories building (b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories building and pin support conditions (a) 2-stories building(b) 4-stories building (c) 6-stories building, (d) 8-stories building and (e) 10-stories and pin support conditions with acceleration 0.2g to 0.31g.

For the ground motions for a range from 0.2g to 0.3g, the performance of various models differs under different support conditions shown in Fig. 5. The IITK model, when applied to fixed support conditions, produces conservative results when the natural time period of the building model is up to 1 second. However, after that threshold, it provides satisfactory results. For pin support, the IITK model does not yield satisfactory results for any building models.

The UBC formula shows conservative results for both support conditions within this ground motion range. Under fixed support conditions, the ASCE model exhibits improved results for the natural time period of less than 0.5 seconds for the buildings. However, as the fundamental time period of the buildings increases, their results become conservative. The ASCE code formula performs poorly for pin support conditions when the structure's natural period is less than 0.5 seconds. Nevertheless, its results are acceptable when the structure's natural period increases to 1.5 seconds. The Akhlaghi[27] model produces effective results for fixed support conditions but is ineffective for pin support conditions within the ground motion range of 0.2g to 0.30g. Fathali's[28] model yields acceptable results for fixed support conditions up to 1 second, after which its results become conservative. However, it demonstrates better results for pin support conditions than other renowned models within the seismic data range from 0.2g to 0.30g.

3.4 Comparison of amplification models with respect to mean plus standard deviation

From the analysis of various considered building models with fixed and pin support conditions, several observations can be made: Table 7 shows that under pin support conditions for ground acceleration in the range of 0.01g to 0.067g, with the fundamental time period of the structure between 0.5 and 1.0 seconds, the Akhlaghi[27] model demonstrates better results. However, when the fundamental time period of the building increases up to 1.5 seconds, the ASCE model outperforms the Akhlaghi model. Overall, for the seismic range of 0.01g to 0.067g, the ASCE formula exhibits satisfactory results in terms of acceleration amplification factor.

These observations suggest that the choice of model depends on various factors such as ground motion intensity, fundamental time period of the structure, and support conditions. The ASCE formula generally provides satisfactory results across different scenarios. Still, specific conditions may warrant using alternative models, such as the Akhlaghi model, especially for certain ranges of ground motion intensity and fundamental periods.

Table 7 — Amplification factor (in %) for different models at pin support with respect to actual results for the ground motion range from 0.01g to 0.067g.

Fundamental Time Period (sec.)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-	-	-	-	-
0.5<T<1.0 sec	-27	11	48	33	2
1.0<T<1.5 Sec	-33	11	48	34	-37

In Table 8, when the seismic data range lies from 0.067g to 0.2g, and the structures' time period is between 0.5 and 1 secs, all models are conservative except the UBC code. When the natural period lies in 1 to 1.5 seconds, the Fathali[28] model gives good results. Overall, for the ground motion range from 0.067g to 0.2g, for pin support, the UBC code provided satisfactory results.

In Table 9, for seismic range 0.2g to 0.32g and the time period 0.5 to 1 sec, the UBC code gives better results in comparison to other models to determine the amplification factor. On the other hand, the ASCE formula gives better results when the fundamental time period increases from 1 to 1.5 seconds. Overall, the UBC code shows satisfactory results of the acceleration amplification factor of NSCs for the seismic acceleration range from 0.2g to 0.30g.

Based on the provided information, here's a summary of the findings from Tables 10-12:

Table 8 — Amplification factor (in %) for different models at pin support with respect to actual results for the seismic data range 0.067g to 0.2g.

Fundamental Time Period (sec.)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-	-	-	-	-
0.5<T<1.0	-37	-15	35	-20	-32
1.0<T<1.5	-19	42	84	29	-24

Table 9 — Amplification factor (in %) for different models at pin support compared to actual results for the seismic data ranges from 0.2g to 0.30g.

Fundamental Time Period (sec.)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-	-	-	-	-
0.5<T<1.0	-41	-20	31	-23	-25
1.0<T<1.5	-24	29	72	-24	-28

Table 10 — Amplification factor (in %) for different models at fixed support compared to actual results for the seismic data range 0.01g to 0.067g.

Fundamental Time Period (sec.)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-28	35	76	55	35
0.5<T<1.0	-28	54	102	79	24
1.0<T<1.5	25	111	172	140	58

Table 11 — Amplification factor (in %) for different models at fixed support compared to actual results for the seismic data range 0.067g to 0.2g.

Fundamental Time Period (second)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-38	-7	24	-11	-7
0.5<T<1.0	-27	66	114	52	57
1.0<T<1.5	38	110	171	91	71

Table 12 — Amplification factor (in %) for different models at fixed support compared to actual results when the seismic data ranges from 0.2g to 0.30g.

Fundamental Time Period (second)	Amplification factor with respect to actual results (in %)				
	IITK	ASCE	UBC	Faithli	Akhlaghi
T<0.5	-36	-5	27	15	-5
0.5<T<1.0	-21	63	114	34	40
1.0<T<1.5	54	117	181	62	63

From Table 10, it is evaluated that when the seismic data ranges from 0.01g to 0.067g, the Akhlaghi[27] model provides satisfactory results compared to another renowned model across the range of seismic ground motion from 0.01g to 0.067g for fundamental natural periods ranging from 0.1 to 1.5 seconds.

In Table 11, for the seismic data range from 0.067g to 0.2g; the UBC code closely matches the mean plus standard deviation results of amplification factors for fundamental time periods up to 0.5 seconds within the range of ground motion from 0.067g to 0.2g. However, it tends to be conservative for periods beyond 0.5 seconds. Akhlaghi [27] model performs satisfactorily for fundamental periods between 0.5 and 1.5 seconds within the same ground motion range.

In Table 12, for the earthquake motion ranges from 0.2g to 0.30g, Fathali's model[28] demonstrates better results than other models for ground motions in the 0.2g to 0.30g range across different fundamental periods.

In summary, different models perform differently across various ranges of ground motion and fundamental periods. The Akhlaghi[27] model is satisfactory in lower ground motion ranges and for longer fundamental periods within the range considered, while the UBC code is more conservative for longer periods within a specific ground motion range. The Fathali[28] model outperforms others in higher ground motion ranges. These findings provide valuable insights for selecting appropriate models for seismic analysis based on specific ground motion conditions and structural characteristics.

4 Conclusion

The conclusions drawn from the paper regarding the analysis of building models with two different support conditions and ground motion ranges are as follows:

a Effect of building height and support conditions

(i) Amplification values decrease with increasing building height for fixed and pinned support conditions.

(ii) Floor spectral acceleration is higher for fixed support compared to pinned support.

b Performance of specific models

(i) The acceleration amplification factor based on the IITK model does not perform satisfactorily when the fundamental time period of the building models is taken between 1 and 1.5 seconds, regardless of the support condition.

(ii) The ASCE code provision gives satisfactory results for fixed support conditions compared to pinned support. It performs better for pinned support when the fundamental time period of the building model is higher than 1.0 seconds.

(iii) The UBC code tends to provide conservative results for all cases when the fundamental period of the structure exceeds 0.5 seconds.

c Performance of fathali model

(i) For pinned support, the Fathali model provides improved outcomes for ground motion ranges from 0.01g to 0.067g. However, it becomes conservative for higher ground motion ranges.

(ii) For fixed support, the Fathali model yields satisfactory outcomes across the entire ground motion range from 0.01g to 0.32g compared to other models.

d Performance of akhlaghi model

(i) The Akhlaghi model shows satisfactory results for all ground motion ranges in fixed support conditions but tends to give conservative results for pinned support.

e Overall assessment

(i) No single model can claim to produce acceptable acceleration amplification values for both the considered fixed and pin support conditions of the structures.

In summary, the paper underscores the importance of considering various factors such as building height, support conditions, and ground motion range when selecting models for seismic analysis. A combination of models may be necessary to predict acceleration amplification values for different scenarios accurately.

Conflict of Interest

I declare that I have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Declaration

I declare that the work does not infringe on copywriting and is free from plagiarism. I also declared that I shall not submit the paper to another journal until the editor decides.

References

1 D'Amicoa S, Caccamob D, Parrillo F & Lagana C, *Phys Solid Earth*, 46(4) (2010) 317.

2 Tsai K C, Hsiao P C & Bruneau M, *Earthq Eng & Eng Seismol*, 2(1) (2000) 93.

3 Anajafi H & Medina R A, *Proc 11th US Natl Conf Earthq Eng: Integrating Science, Eng & Policy*, Los Angeles, USA, June 25-29, 2018.

4 Sullivan T J, Calvi P M & Nascimbene R, *Earthq Struct*, 4(1) (2013) 109.

5 Reinoso E & Miranda E, *Struct Design Tall Spec Build*, 14 (2005) 107.

6 Agrahari R K & Pathak K K, *Indian J Eng Mater Sci*, 30(1) (2023) 1.

7 Azeem M A & Mohiuddin H, *Indian J Sci Technol*, 9(36) (2016) 1.

8 Devin A & Fanning P J, *Eng Struct*, 187 (2019) 242.

9 Villaverde R, *J Struct Eng (ASCE)*, 123(8) (1997) 1011.

10 Lin J & Mahin S, *J Struct Eng (ASCE)*, 111(2) (1985) 400.

11 Chen Y & Soong T T, *Eng Struct*, 10(4) (1988) 218.

12 Kingston K M, *An evaluation of floor response spectra for acceleration-sensitive non-structural components supported on regular frame structures*, Master's thesis, Univ Maryland, 2004.

13 Chaudhuri S R & Villaverde R, *J Struct Eng (ASCE)*, 134(4) (2008) 661.

14 Building Seismic Safety Commission, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA, Washington, DC, 1997.

15 Building Seismic Safety Commission, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA, Washington, DC, 2000.

16 American Society of Civil Engineers (ASCE), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA, Washington, DC, 2000.

17 Soong T T, Shen G, Wu Z, Zhang R H & Grigoriu M, *National Center for Earthquake Engineering Research Report*, Univ at Buffalo, NY, 1993.

18 Schroeder M E & Bachman R E, *Proc 5th US Natl Conf Earthq Eng*, Chicago, USA, July 10–14, 1994, Vol. 4, p. 755.

19 Bachman R E & Drake R M, *Proc 5th US Natl Conf Earthq Eng*, Oakland, CA, 1994, Vol. 4, p. 765.

20 Bachman R E & Drake R M, *Proc 6th US Natl Conf Earthq Eng*, Earthq Eng Res Inst, Oakland, CA, 1998.

21 Drake R M & Bachman R E, *Proc 64th Annual Convention, Structural Engineers Assoc California*, Sacramento, CA, 1995, p. 333.

22 Kehoe B E & Freeman S A, *ATC-29-1 Report*, Applied Technology Council, Redwood City, CA, 1998.

23 Searer G R & Freeman S A, *Proc 7th US Natl Conf Earthq Eng*, Boston, MA, Earthq Eng Res Inst, Oakland, CA (CD-ROM), 2002.

24 Horne P & Burton H, *Report ATC-29-2, Proc Seminar on Seismic Design, Performance & Retrofit of Nonstructural Components in Critical Facilities*, Applied Technology Council, Redwood City, CA, 2003.

25 Uniform Building Code (UBC), *Uniform Building Code*, Vol. 2, International Conf Building Officials, Whittier, CA, 1997.

26 Murty C V R, *Earthquake Protection of Non-Structural Elements in Buildings*, IITK-GSDMA Project Report, Govt of Gujarat, 2013.

27 Akhlaghi H & Moghadam A S, *Proc 14th World Conf Earthq Eng*, Beijing, China, October 12-17, 2008.

28 Fathali S & Lizundia B, *Struct Design Tall Spec Build*, 20 (2011) 30.

29 Bureau of Indian Standards, *IS 1893:2002 (Part 1): Criteria for Earthquake Resistant Design of Structures*, BIS, New Delhi, 2002.

30 Azeem M A & Mohiuddin H, *Indian J Sci Technol*, 9(36) (2016) 1.

31 Computers and Structures Inc., *Extended 3D Integrated Analysis, Design and Drafting of Building System, Advanced ver. 16.2.0*, CSI, Berkeley, CA, 2016.

32 Strong ground motion database, available at: https://strongmotioncenter.org/vdc/scripts/default.plx (accessed 23 Sept 2025).

33 Agrahari R K, Sharma A & Pathak K K, *J Struct Eng*, 47(3) (2020) 181.

34 Agrahari R K, Pathak K K & Jha I, *Indian J Eng Mater Sci*, 29(2) (2022) 189.