Indian Journal of Engineering & Materials Sciences
Vol. 29, April 2022, pp. 189-200

Seismic acceleration amplification factor for pin supported moment resisting RC frame structures for Chi-Chi earthquake

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Received: 9 February, 2021; Accepted: 9 August, 2021

The Seismic forces, acting on the non-structural component, are different with respect to the lateral force observed by the different design codes. For remarking the behavior of the structural components during dynamic action, large numbers of research were done as compared to the secondary or non-structural components of the structures. The behavior of inertia force acting on the non-structural components changes along with the altitude of the structure. The inertia force existing on non-structural components depends on the acceleration amplification factor. In this paper, five different RC moment-resisting frame models as 2,4,6,8 and 10 stories, with pin support conditions are considered. The linear time history method is used for the analysis of all RC frame models with different range (0.01g to 0.32g) of ground motion data. Determine the acceleration amplification factor for all these models and compared with the previously models. It is observed that no previous model performed satisfactory results. Therefore, to proposed the amplification model which not only depends on the height of the building, natural period of the building but also depends on the range of the Peak Ground Acceleration (PGA).The proposed model is compared with the previous renowned models, It is detected that the proposed model performs better results with respect to other models.

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

1 Introduction

Until the worldwide acceptance of current seismic design events towards the end of the twentieth century, the structures (irrespective of the structural material) collapsed or got damaged due to moderate to high earthquake actions. The engineering reinforced building or unreinforced masonry building both suffered huge damages during recent earthquake actions. As the basic shell of structures (i.e., load carriage parts, for example, outlines, substructure, establishments, and so forth; generally meant to as structural segments) are harmed seriously to make their annihilation fundamental, the degree of harm caused by the optional structure components (i.e., components not contributed the heap way of horizontal seismic force; for the most part indicated as non-auxiliary segments) used to go totally disregarded[1].With the acceptance of seismic design practiceuniversally, damages to the structural segment of engineered buildings have been repairable during moderate to severe earthquake events. But, by evaluation, as Non-Structural Components (NSCs) are not planned and fitted to resist earthquakes, they continually get damaged to a larger extent. The devastation to non-structural segments recognized in the current Taiwan earth tremor is evidence of these aspects[2].

Taiwan is situated along with the thrust between the Eurasian and Philippine Sea Plates. This region is recognized as a sensitive area in the world where frequent earthquake occurs[3]. A magnitude of 7.3 earthquakes was hit on the central region of Taiwan, on 21 September 1999. A large percentage of building components (structural and non-structural components) got damaged due to the mainshock or strong aftershocks that cause not only loss of the economy but also loss of life[4]. Sometimes the expense of the non-auxiliary segments of a structure is 70-80% of the all-out development cost of business structures[5].

Non-Basic Segments (NSCs) are not regularly seen in design norms[6,7] and are estimated to be basically for building, mechanical, or electrical purposes[8,9]. Research and development have been deficient in design methods of NSCs on the behaviour of the building. For unsymmetrical building, the damages of NSCs are more with respect to symmetrical buildings[10]. The perception that the NSCs perform openly to the structural casing is wrong, as it has been dug in that NSCs interface with the structural framework[11 – 15].

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For the seismic action on the structure, NSCs are categorized based on storey drift sensitive and the acceleration sensitive. Some guidelines provided by the researchers, for the designing of these components[16,17], are based on the minimum equivalent static forces for the construction of a new building and retrofitting of the existing building[18,19]. These guidelines were obtained based on partial analytical studies[20] and partial experimental studies[21]. It is also observed by some researchers that these guidelines do not provide truthful results when floor acceleration varies along with the height of the building[22,23].

The inertia force is a significant parameter for the designing of NSCs, and these inertia forces act in NSC by the floor acceleration[24]. For studying the behavior of the floor acceleration effect of the structures, Gillengerten and Bachman[25] presented an approach to understands the concept of the acceleration amplification factor. It was observed that the acceleration amplification factor formula proposed by ASCE 7-10[26], bounded between the mean plus standard deviation (mean+sd) of the peak acceleration. Akhalkhi[27] suggested, that the acceleration amplification factor, was based on the normalized height (ratio between the height of the floor to the total height of the structure to the base) of the structure and the fundamental period of the structures. Agrahari[28], proposed the amplification factor for fixed supported RC frame structures which was based on the different range of the PGA. However, Fathali's[29] marked that the amplification factors of the structures were also dependent on the intensity of the seismic motion acting on the structures. In this paper, to an analysis of five different height storeys of the moment-resisting RC frame model with pin support condition. For this analysis different ranges of ground motion data are considered (based on Fathali's[29]), to determine the acceleration amplification factor. The proposed mathematical model of acceleration amplification factor after comparison with mean+sd and previous renowned amplification model is found to be better performing than previous models.

2 Materials and Methods

2.1 Current model equations

2.1.1 Uniform Building Code 1997 (UBC)

In this code[30], the horizontal force acting on the non-structural components of the floor is given as:

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

where, a_p is the structural amplification factor, R_p is the response modification factor, h_x and h_n are the altitude of the components and the over-all height of the building from the bottom of the structures, I_p represent the important factor of the components, W_p is the total weight of the components respectively. The coefficient of a_p, wary between 1.0 to 2.5 and the response modification factor (R_p) vary between 1.0 to 4.0. In this equation $\left(1 + 3\frac{h_x}{h_n}\right)$ represent the floor acceleration amplification factor of the secondary elements.

The floor horizontal force F_p

$$0.7C_a I_p W_p < F_p < 4.0C_a I_p \tag{2}$$

For determining the horizontal force on the elastic components of the floors, the response amplification and structural amplification factor is considered as 1.0.

2.1.2 ASCE

The lateral seismic force acting on the non-structural components, defined by ASCE/ SEI 7-10[26] in section 13.3.1 as

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

$$0.3S_{ds} a_p W_p \le F_p \le 1.6S_{ds} a_p W_p \tag{4}$$

where, F_p is the lateral seismic design force, S_{ds} represent the site-specific short period spectral acceleration, $a_p$is the component amplification factor having a range of 1.0 to 2.5, z and h denotes the height of the component and the height of the building with respect to base respectively, Ip is the component important factor, and R_p refers to the component response modification factor which shows the energy absorbed by the component and W_p is the weight of the component. However, the value of $\left(1 + 2\frac{z}{h}\right)$ express the floor acceleration amplification factor of the non-structural segments.

2.1.3 IITK-GSDM

For the design of non-structural components in RC frame structures, IS 1893-2002[31] code does not provide clear information. Clause 7.12.2, define the lateral force acting on the non-structural components should be five times the horizontal design acceleration multiplied by the weight of the components. The provision given by the code gave highly inadequate results for the estimating of lateral force acting on non-structural components. IITK-GSDM[32] proposed an acceleration amplification model for obtaining the amplification factor on the RC frame structures. IITK amplification model is based on the normalized height of the structures which is given as

$$\Omega = \left(1 + \frac{z}{h}\right) \tag{5}$$

where, z and h are the height of the components and the height of the building with respect to the base. This model found that the maximum amplification of the non-structural components which occurred is 2 when the z and h are equal.

2.1.4 Akhlaghi and Moghadam

Akhalaghi and Moghadam estimated that the seismic behavior of rigid acceleration sensitive secondary elements having the fundamental period was less than or equal to 0.06 sec. It was observed that the nature of the peak horizontal acceleration of the floor or roof is the same as the nature of the rigid non-structural components along with the height of the building, linked with the main structure. They concluded that the response of the floor or roof during the ground motion, was the same as the response of the non-structural components, so they proposed the equations of the floor acceleration amplification factor (**Ω**) based on the fundamental time period of the structures.

$$\Omega = 1 + (\alpha - 1)\left(\frac{h_i}{h_n}\right) \tag{6}$$

Where, Ω is the floor acceleration amplification factor, defined as the ratio between peak horizontal floor acceleration to peak ground acceleration, h_i and h_n are the height of the storey and the total height of the building with respect to the base of the building and α represent the fundamental period dependent factor, which was given as:

$\alpha = 3$ 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

where, T is the fundamental period of the structures.

2.1.5 Fathali and Lizundia

Fathali and Lizundia observed that the floor acceleration a height amplification factor to be not only dependent on the height of the components of the structure but also dependent on the level of the ground motion and proposed the non-linear equation based on it.

$$\Omega = 1 + \alpha\left(\frac{z}{h}\right)^{\beta} \tag{7}$$

where, z, h are the height of the non-structural component and height of the storey to the base. α and β are two parameters based on the natural period of the structure and the level of the ground motion respectively. The values of α and β are shown in Table 1 and 2 respectively.

2.2 Proposed mathematical model

The IITK model is based only on the normalized height of the structures and is not dependent on the fundamental period of the structures or the intensity of the seismic motion. UBC 1997 formula for amplification factor, gave obscure results when the height of the building increases. ASCE model, also based on the normalized height of the structures only. The Fathali and Akhlaghi models show that the amplification factor not only depends on the normalized height of the building but also depends on the intensity of ground motions and the fundamental period of the structures. But sometimes its results were observed to be conservative when the range of the ground motion changes. So based on these factors, the proposed amplification factor model for this study. In these models, no single maximum structural

Table 1 — Value of α is suggested for the seismic design of newly constructed NSCs

Natural period	PGA = 0.4SDS < 0.067 g	0.067 ≤ PGA = 0.4SDS < 0.20 g	PGA = 0.4SDS ≥ 0.20 g
T_a< 0.5s	2.120	1.930	1.750
0.5≤ T_a<1.5s	2.610	1.550	1.010
T_a ≥1.5s	2.520	1.530	0.500

Table 2 — Value of β is suggested for the seismic design of newly constructed NSCs

Natural period	PGA = 0.4SDS < 0.067 g	0.067 ≤ PGA = 0.4SDS < 0.20 g	PGA = 0.4SDS ≥ 0.20 g
T_a< 0.5s	0.780	1.250	0.920
0.5≤ T_a< 1.5s	1.160	0.750	0.690
T_a ≥1.5s	1.640	1.650	3.000

period is found to satisfy the actual amplification factor. To find the realistic amplification factor, two steps have been followed. Firstly, the ground acceleration has been divided into three ranges viz. 0.01-0.067g, 0.067-0.2g and 0.2-0.32g. Secondly, T_{max} is divided into three ranges in each acceleration range based on natural period. About 90 simulation studies have been carried out to arrive at the T_{max} values. The proposed models based on observed results are represented as:

$$\Omega = \frac{PFA}{PGA} = \left(1 + \frac{T_{max} - T}{a * T}\frac{z}{h}\right) \quad (8)$$

where T_{max} is the maximum structural period, and its value is recommended as 2.5 seconds[33]. T is the period of supporting structure when the peak roof acceleration is not less than PGA. Constant "a" not only depends the period of the supporting structures but also depends the nature of the ground motion, its values given in Table 3.

2.3 Configuration of buildings

For the analysis in this paper, five different RC frame building models as two, four, six, eight, and ten storeys are considered. All these models are pin supported in hard rock strata. From base to the first storey, the height is 4m and for the above storey height of 3.4m is considered. The 2D model of the pin supports shown in Fig. 1. The size of the beam and column are presented in Table 4. The damping ratio

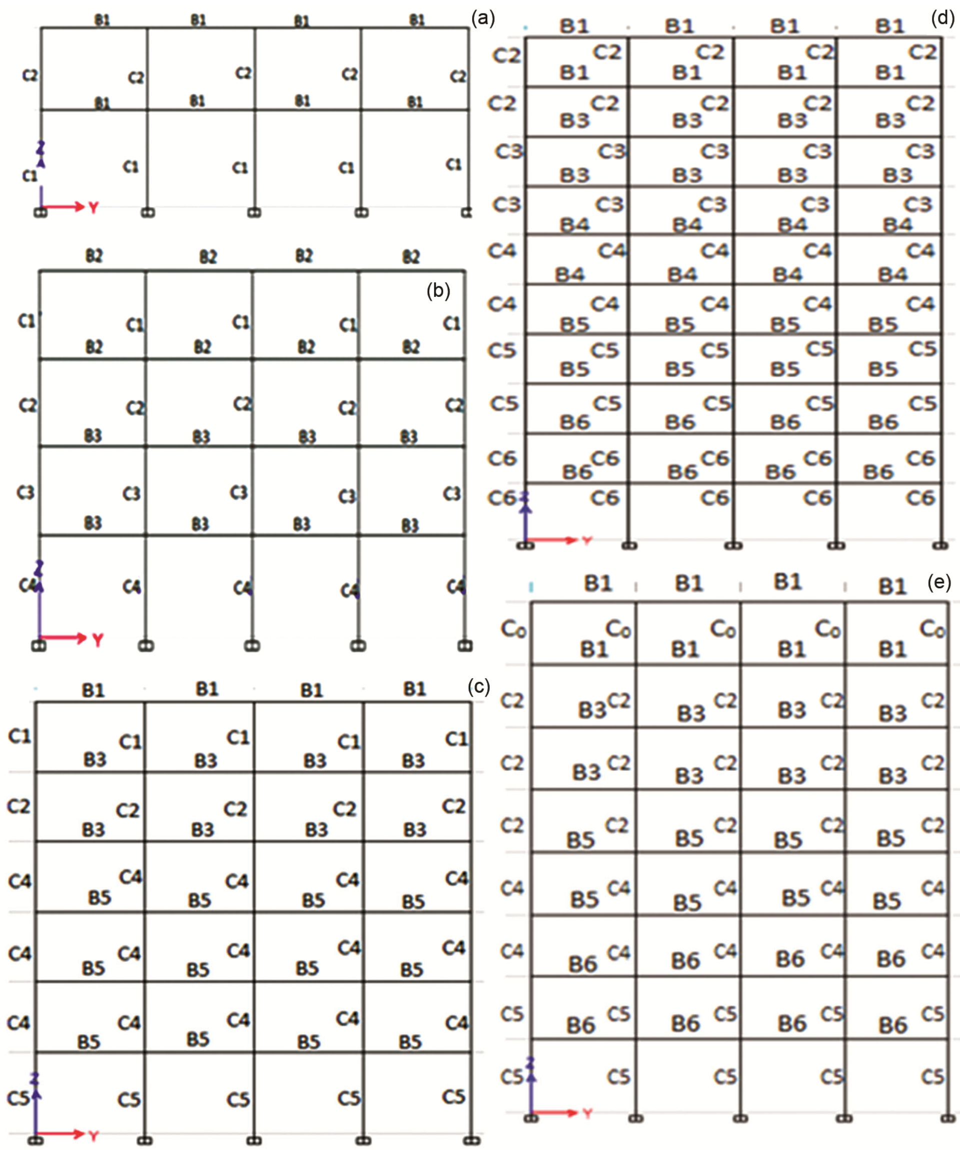

Fig. 1 — Moment resisting frame models (a) 2, (b) 4, (c) 6, (d) 8, and (e) 10 stories.

and the fundamental period of the structures are considered as 5% and up to 1.5 sec, respectively. Etabs[34] software has been used for the analysis of all these models.

2.4 Selection of time history data

The previous researches worked with the higher range of the ground motion (0.3g to 0.8g), but most of the cases the structures are damages below the low range of the ground motion. So, in this paper, considering the low range of the acceleration data. The analysis of RC frame structure, seismic data has been taken in the range of 0.01g to 0.31g. These data is divided based on the Fathali and lizundia[29] approached (0.01g to 0.067g, 0.067g to 0.02g and higher than 0.2g). The selection of ground acceleration was obtained from the strong ground motion virtual data center[35]. For the study of these models, different ranges of time history data (28 recorded ground motion data, between 0.01g to 0.067g, 29 ground motion data in the range of 0.067g to 0.2g, and 24 ground motion data between 0.2g to 0.32g) are considered. Details of the ground motion data are given in Tables (5-7) as below.

In these tables, T represents the total Recorded period and T_p represents the time that peak acceleration occurs.

2.5 Analysis of building based on finite element technique

To the examination of all the RC frame model have been done by finite element method (e.g. Etabs v2016). It is a mathematical strategy to understand the frame models into element models. For all the various models clarified in Agrahari *et al*.[36], linear time history methods are used. The Pin support condition is allocated at the joint base of the structures, and the soil type is hard. A large number of near field time history data (0.01g to 0.32g) is considered and also the fundamental period of the structure is taken up to 1.5 sec.

3 Results and Discussion

3.1 Floor spectra curve

For obtaining the spectral acceleration of the structures in each storey, different ground motion data

Table 3 — Values of “a” based on ground motion range and period of supporting structure

Ground motion acceleration	“a”	Period of supporting structure
$PGA=0.04S_{DS}<0.067g$	1.2	$0<T<0.70sec$
	0.61	$0.7sec\leq T<1.2sec$
	0.35	$1.2sec\leq T<1.5sec$
$0.067g\leq PGA=0.04S_{DS}<0.2g$	1.00	$0<T<0.70sec$
	0.75	$0.7sec\leq T<1.2sec$
	0.49	$1.2sec\leq T<1.5sec$
$0.2g\leq PGA=0.04S_{DS}<0.31g$	1.25	$0<T<0.70sec$
	0.70	$0.7sec\leq T<1.2sec$
	0.43	$1.2sec\leq T<1.5sec$

Table 4 — Proportions of beams and columns

Beam	Size in mm
B1	300x400
B2	300x450
B3	450x500
B4	450x600
B5	450x650
B6	450x675
Column	Size in mm
C_0	300X400
C1	300x450
C2	450x500
C3	525x550
C4	550x600
C5	600x700
C6	650x850

Table 5 — Recorded ground motion data having ranges 0.01g to 0.67g

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

Table 6 — Recorded ground motion data having ranges 0.067g to 0.2g

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

Table 7 — Recorded ground motion data having ranges 0.2g to 0.31g

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

on the structures is applied. Figure 2, express the mean spectral acceleration of the building for the ground motion ranges as 0.2g to 0.32g respectively.

For the analysis of all these models hard rock type soil is considered. It is noticed that when the fundamental period of the structures increases the floor spectral acceleration of the structures decreases i.e. when the height of the building is less, the amplification value is maximum. When the fundamental period of the structure increases to 1.5 sec, the floor spectra value decreases to 25% with respect to the low natural period of the structure.

3.2 Compared to peak floor acceleration compared with respect to seismic ground accelerations

To compared the peak floor acceleration of all model with respect to seismic ground acceleration are shown in Figs (3 – 5) respectively.

Figures (3 – 5), present the peak floor acceleration in five building models to those recorded during the Chi-Chi earthquake. The peak floor acceleration is approximately 4 times, 1.5 times and 2 times higher than the building base for .01g to .067g, 0.067g to 0.2g and 0.2g to 0.31g acceleration. The amplification values are significantly higher when the natural period of the structure less than 1 sec. The shape of peak floor acceleration performed non-linear, as the seismic motion increases.

3.3 Compared to the mean +sd acceleration amplification factor with previous models

To compare all models with the proposed model is shown in Figs (6 – 8), respectively.

In Fig. 6, the behaviour of acceleration amplification factors is non-linear as the height of the building increases. IITK model performed the conservative result for all the building models when the ground motion range is 0.01g to 0.067g. UBC code performed safe results on an approximate basis. It was 48% high than the mean + sd results. ASCE model gives the linear relation between the amplification factor and the normalized height of the structures. ASCE model performed the obscure results for the natural period of the structure less than 0.7 sec., after that it performed satisfactory results. In Akhlaghi model, performed better results when the natural period of the structure is less than 1.0 sec but for a higher period, it reacts obscure results and its values approximately 35% higher than mean + sd results. Fathali model is a non-linear model and its results are 34% higher than the mean + sd results. The

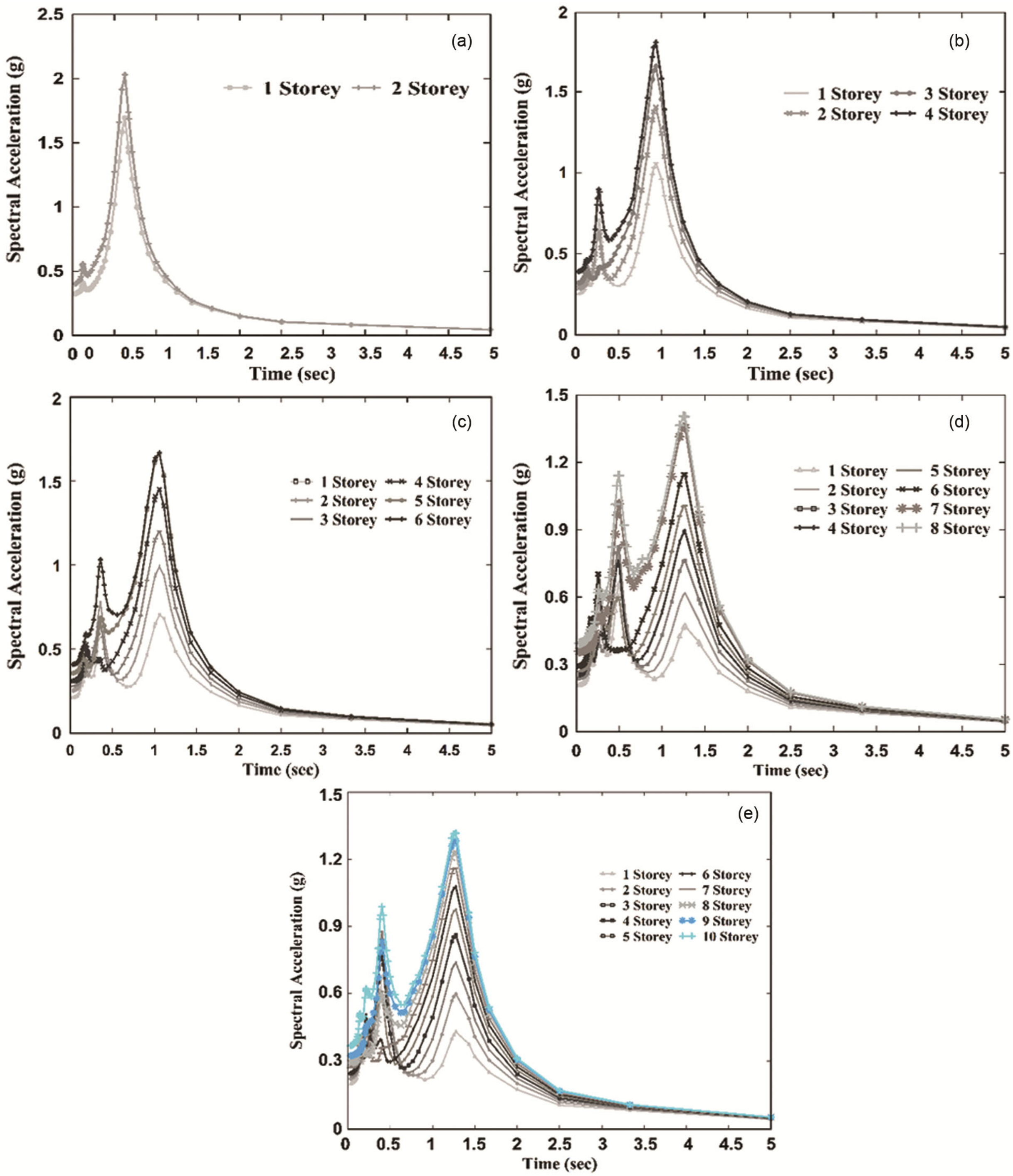

Fig. 2 — Floor spectral acceleration of different model for hinge support condition.

proposed amplification model values are nearly 20% higher than the mean + sd amplification factor value for natural period of the structure less than 0.7 sec. However, the proposed model results are much close to the mean + sd results, when the natural period of the structure higher than 0.7 sec, and its values are nearly 10% higher than the mean + sd results.

In Fig. 7, IITK and Akhlaghi models gave lower value with respect to mean + sd results for all the fundamental period of the structures. UBC code observed better results compare to the IITK model, and it is approximately 38% higher than the mean + sd results for the natural period of the structure upto 0.7 sec. For the higher natural period of the

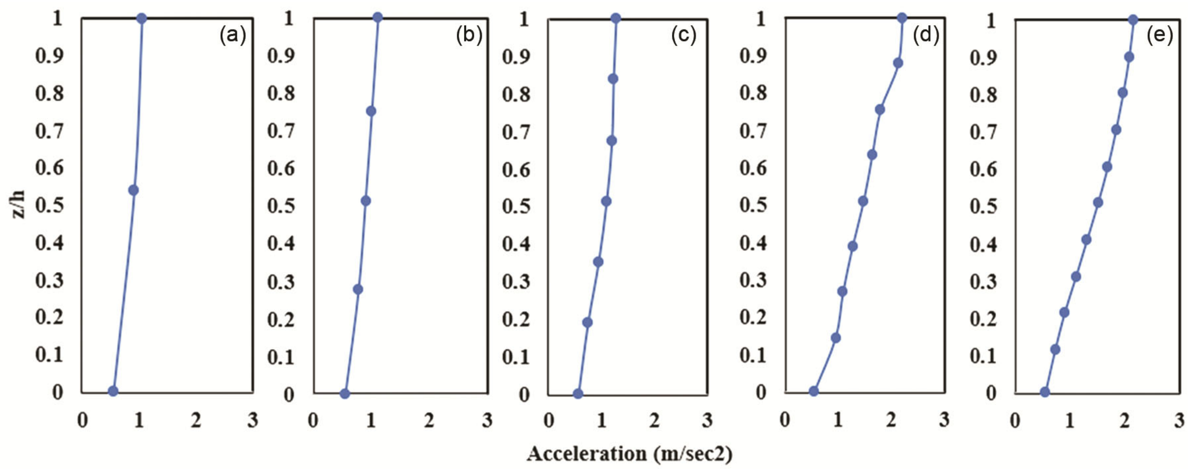

Fig. 3 — Behavior of peak floor acceleration with respect to normalized height when the chi-chi earthquake 0.06g for (a) 2 storey, (b) 4 storey, (c) 6 storey, (d) 8 storey, and (e) 10 storey.

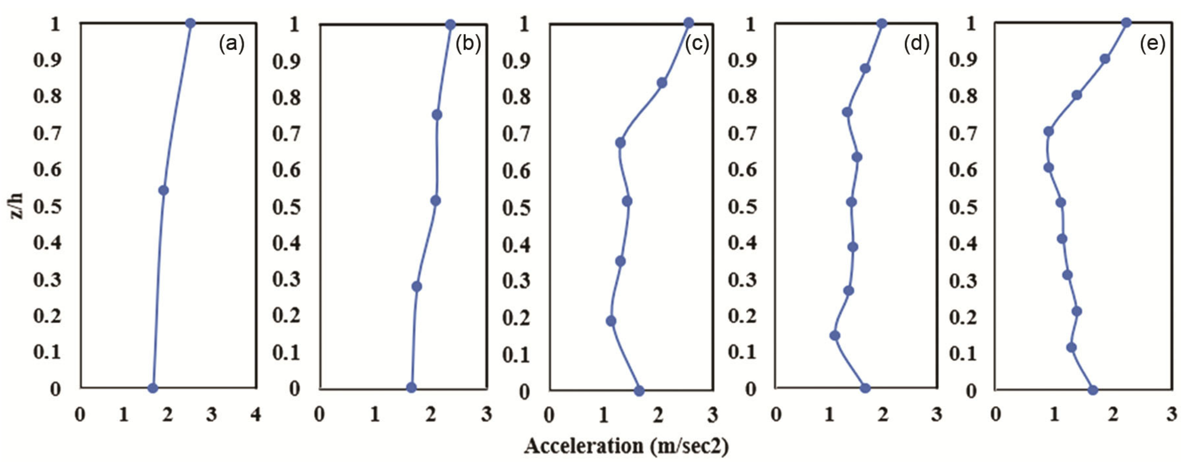

Fig. 4 — Behavior of peak floor acceleration with respect to normalisedheight when the chi-chi earthquake 0.17g for (a) 2 storey (b) 4 storey (c) 6 storey (d) 8 storey, and (e) 10 storey.

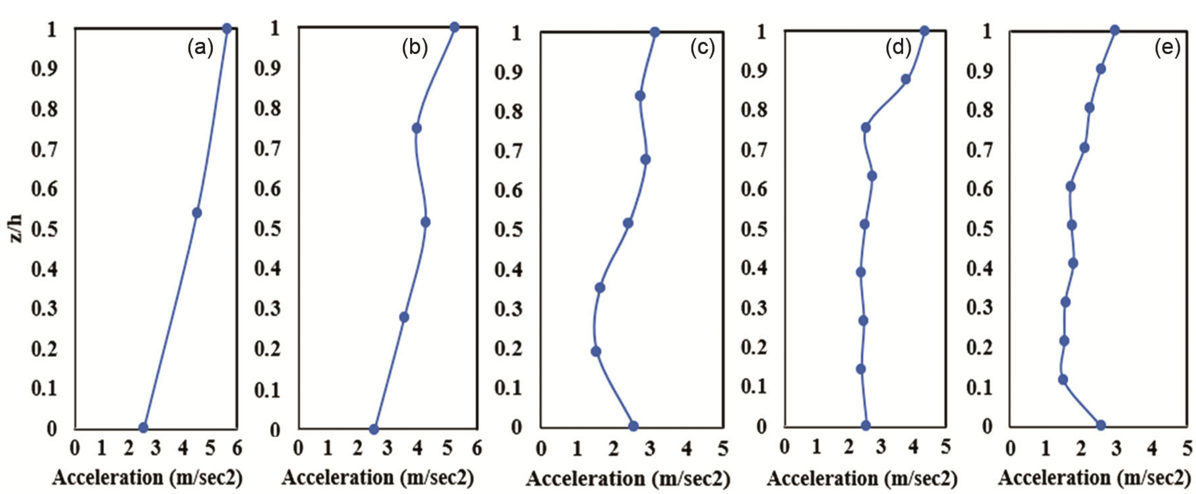

Fig. 5 — Behavior of peak floor acceleration with respect to normalized height when the chi-chi earthquake 0.26g for (a) 2 storey, (b) 4 storey, (c) 6 storey, (d) 8 storey, and (e) 10 storey.

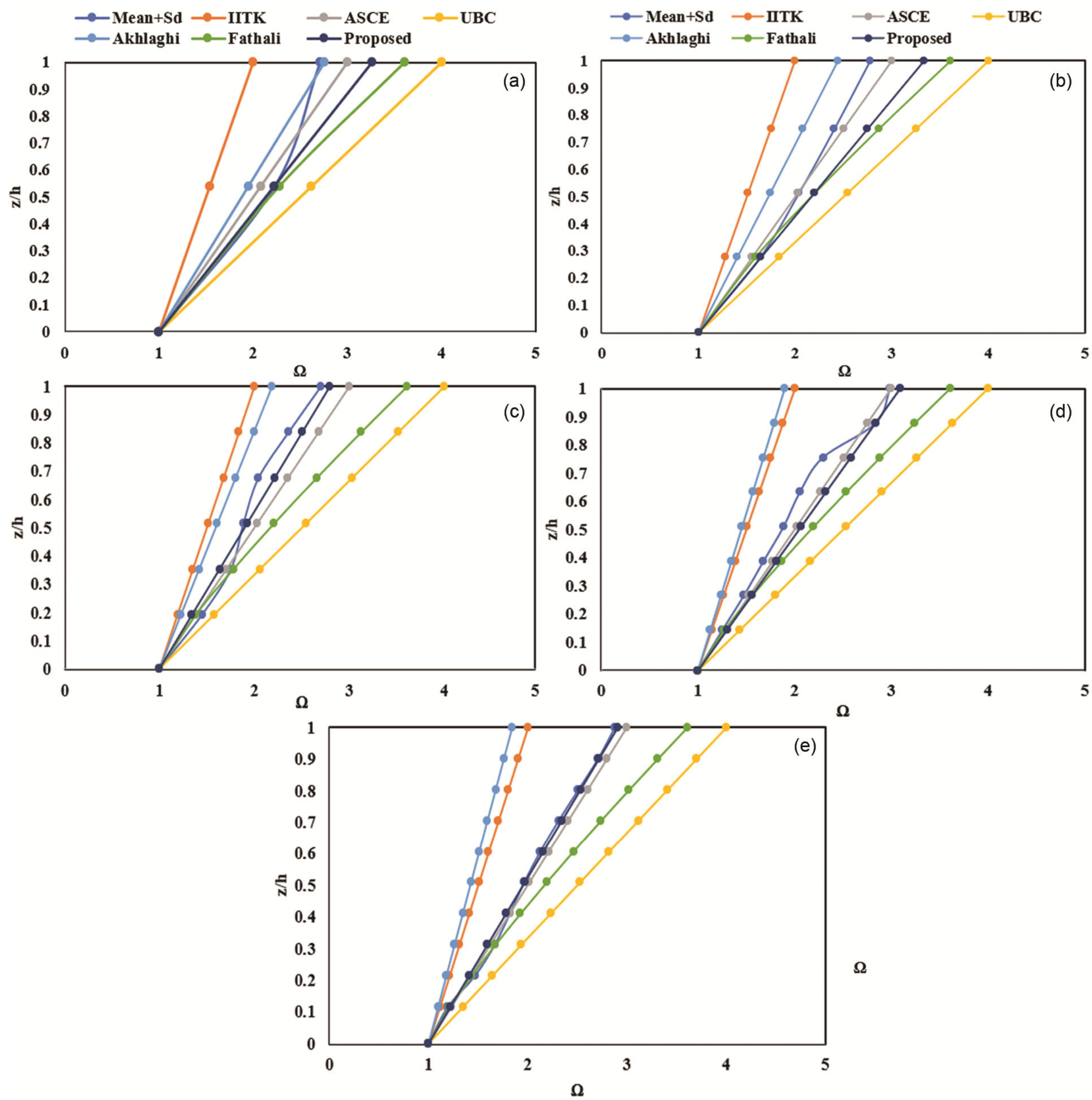

Fig. 6 — Comparition of the models with the seismic range 0.01g to 0.067g (a) 2, (b) 4, (c) 6, (d) 8, and (e) 10 stories.

structure, gaping of amplification values are increased related to mean + sd results. The Fathali model results are obscure when the natural period of the structures is less than 0.7 sec. However, its results are satisfactory when the period of the structure higher than 0.7 sec, and the amplification factor for this model is approximately 29% higher than the mean + sd results. When the natural period of the structures less than 0.7 sec, only one model (UBC code) gives the truthful results as comparing the other model, and the amplification values of this model are around 40% higher than the mean + sd amplification value. However, the amplification of the proposed model value is approximately 22% higher than the mean + sd results. The fundamental period of the structures increases from 1 to 1.5 sec where some model performed unclear results (IITK and Akhalghi model), however ASCE, UBC, and Faithli models amplification value 42%, 84%, and 30% higher than the mean + sd amplification results. The amplification value obtained by the proposed model is almost 18% higher than the mean + sd amplification results for the

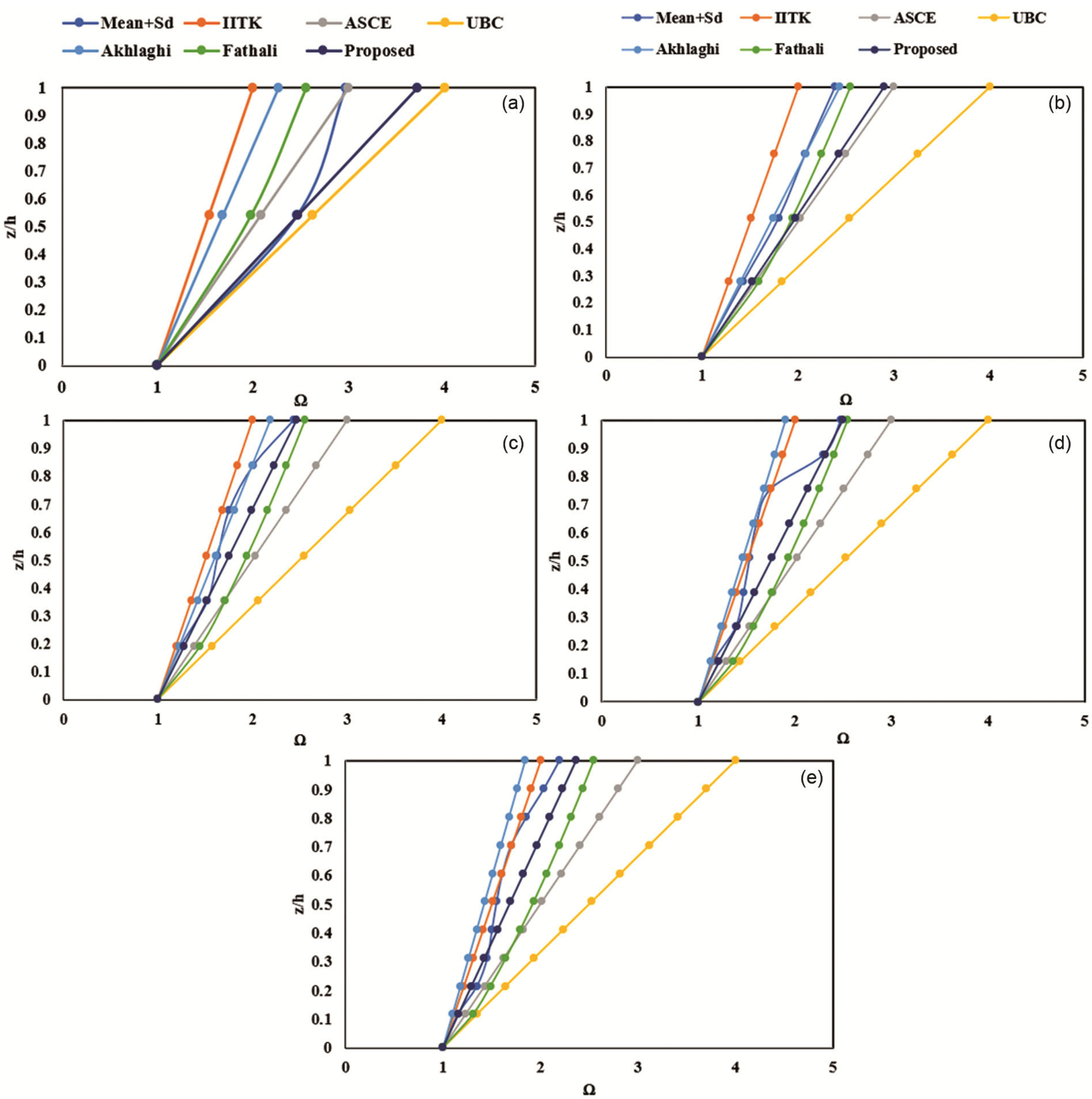

Fig. 7 — Comparition of the models with the seismic range 0.067g to 0.2g (a) 2 (b) 4 (c) 6 (d) 8, and (e) 10 stories.

natural period of the structure is 1.0 to 1.5 sec and the seismic range of ground motion is 0.067g to 0.2g, respectively.

In Fig. 8, When the ground motion range between 0.2g to 0.31g, the UBC model gives better results and is nearly 30% higher than the mean+sd results although the natural period of the structures is up to 0.7 sec. For a higher natural period, the UBC model performed obscure results, and the amplification factor values are approximately 72% higher than the mean+sd amplification factor results. Whereas, the amplification value of the proposed model is 20% higher than the mean+sd amplification results for the fundamental period of the structure up to 1.0 sec. As the natural period of the structures increases from 1.0 sec to 1.5 sec IITK and Faithli model does not observe reasonable results, whereas ASCE and UBC model observed 30% and 72% higher than the mean+sd amplification results. The amplification values of the proposed model are approximately 15% higher than the mean+sd amplification value for the period of structure lying between 1.0 sec to 1.5 sec and the seismic range are 0.2g to 0.31g, respectively.

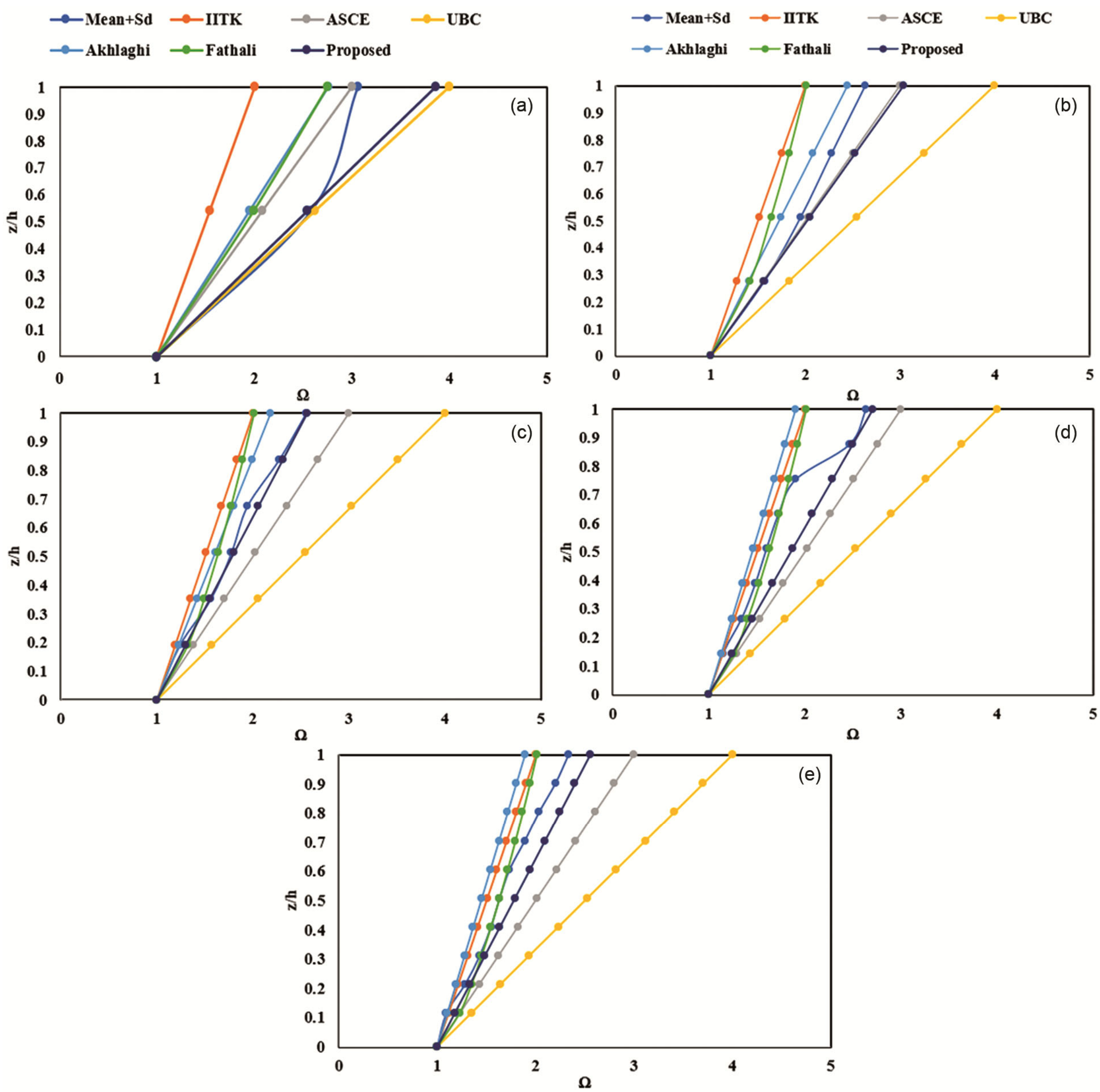

Fig. 8 — Comparition of the models with the seismic range 0.2g to 0.31g (a) 2, (b) 4, (c) 6, (d) 8, and (e) 10 stories.

4 Conclusion

In this paper, five different models as 2, 4, 6, 8, and 10 stories have been considered. The support condition of all these models is the pin and linear time history method used for the analysis of these models. A range of time history data, 0.01g to 0.32g is considered. To determine the acceleration amplification factor and comparison with the previously proposed models is done. It is found that for acceleration amplification values, no single model is performed to give satisfactory results for pin support conditions. To this overcome, the proposed acceleration amplification model is given and compared with the previous renowned models. The conclusions are summarised as:

- For pin support condition and the different range of seismic motion, IITK-GSDM model do not perform to give satisfactory results.
- UBC code formula, performed to give better results when the natural period of the structures is less than 0.7 sec., after that its results are conservative.
- Akhlaghi model depends on the fundamental period of the structures, but this model does not

perform to give truthful results when the support condition has pined.

- ASCE model is not given the adequate result when the fundamental period of the structures less than 0.7 sec for all the different ranges of seismic ground motion.
- When the natural period of the structure is higher than 0.7 sec, the ASCE model gives better results for the ground motion range of 0.01g to 0.067g. But for other ranges as 0.067g to 0.2g and 0.2g to 0.32g, its results are found to be conservative.
- Fathali model observed satisfactory results when the ground motion range is 0.067g to 0.2g. But on the higher seismic range, its results performed inadequately.
- The acceleration amplification factor of the non-structural components is not only dependent on the height of the building, but it also depends on the fundamental period of the structures and the intensity of the ground motion.

The proposed model was performed to give satisfactory results with respect to the previous renowned models for the different range of ground motion with pin support conditions. This research is focused on the pin supported RC frame Structures: hence the results and conclusion derived herewith may not presented the shear wall or braced frame structures.

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