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Floor Acceleration Amplification Factor in Yielding of Moment Resisting RC frame Structures

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Structural elements have been designed as load-bearing as well as non-load bearing. Non-structural components (NSCs) represent the non-load-bearing elements of the structures. Many provisions have been provided for seismic design of primary components of structures, but limited prescription has been provided for seismic designing of NSCs. This paper describes the behaviour of the acceleration-sensitive NSCs for different ranges of ground motions. For this study, the four different height of moment-resisting RC frame models, fixed at the base of the structure have been considered. With 17 far-field seismic ground motions, the building models have been investigated using the incremental dynamic approach. To analyze the floor response spectra, building periods, and structures ductility parameters, based on this proposed the acceleration amplification factors of the NSCs.

Keywords: Acceleration amplification factor , Ductility ratio, Non-structural component ,Time history data

1 Introduction

Non-structural components (NSCs) in a structure (also called secondary elements) are elements that add to the building design dead load and do not subsidize the resistance against seismic actions. NSCs are essential because without them the buildings have been neither complete nor work as intended. However, the primary structural components (load-resisting components) provided structural strength and stiffness. In contrast, the NSCs provided heat and sound insulation, partition, and shielding from the sun's rays, or rain to make the building usable[1]. Some examples of NSCs are roofs, cladding, doors, windows, etc. The moveable elements of the buildings devote the live load (e.g. chairs, equipment, furniture's, appliances, etc.) are classified as contents[2].

In commercial buildings, the cost of NSCs and contents is approximately 70 to 80% of the building total construction cost[3]. In a recent earthquake, the losses of NSCs have been reported to exceed the failures of the affected buildings main structural components[4-8]. In 2011, Tohoku earthquake (Japan) caused loss of life for ceiling boards collapse[9]. Similarly, in 2010, the Darfield earthquake reported enormous losses of life and severe injury when the failure of NSCs (parapets, chimneys, canopies etc.). In order to minimize such losses during the earthquake, it is essential to study the behaviour of NSCs to withstand the design level of the earthquake.

The NSCs have been classified based on storey drift sensitive (as indoors, windows etc.) as well as acceleration sensitive (as in ducts, parapets, boilers etc.)[10]. The acceleration-sensitive NSCs are mainly caused by the inertia force obtained by horizontal or /and vertical acceleration at various levels of the supporting structures, so overturning or sliding of the components occurs. However, the displacement-sensitive NSCs have been mainly caused by inter-storey drift in the supporting structures, which causes significant distortion in the elements. For the study of these sensitive, many research studies have been done[11]. Still, the lack of information about the nature of the NSCs under seismic hazard and seismic design provision by different codes does not perform adequate results[12].

The inertia forces act on the NSCs depending on the acceleration amplification factor (the ratio between peak floor acceleration and peak ground acceleration). UBC code[13] marked that the acceleration amplification factors had counted only the height of the building, and its maximum value is observed as 4. The other code, ASCE[14], also defined that acceleration amplification factor varies following the height of the building, and its highest value is 3.

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Agrahari[15] proposed the nonlinear amplification model based on ground motion acceleration. In actuality, the amplification factor depends not only on the height of the building but also on other factors. Akhlaghi[16] marked that the amplification factor also depends on the building fundamental period (it is the period of structures for one complete cycle of oscillation). Fathali[17] observed that the ground motions intensity has also affected the acceleration amplification factor. For this, the proposed amplification model depends not only on the building period but also on the Peak Ground Acceleration (PGA). The ductility capacity (ratio between the performance limit displacement to first yield displacement) of the buildings has also been affected by the acceleration amplification factor. Wiser[18] proposed an amplification model to determine the non-structural force reduction factor.

This paper investigates the acceleration amplification factor for fixed support conditions and compares the previously proposed model. Consider the four different moment-resisting RC frame models to determine the amplification factor. These models have been analysed by the incremental dynamic method with various PGA. The fundamental period of the structures, ductility, and floor response spectra parameters proposed in the acceleration amplification model have been compared with the previously renowned model. It has been found that the previous model performed conservative results. However, the proposed model achieved better results than earlier models.

2 Materials and Methods

For the analysis of the structures, the incremental dynamic analysis method and the M25 grade of concrete have been used to determine the acceleration amplification factor. A brief discussion of the previously proposed amplification models and method of analysis are explained below.

2.1 Current Model Equations

2.1.1ASCE

ASCE/SEI 7-05[11] section 13.3.1, defines the lateral seismic force on the non-structuralcomponent as

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

$$0.3 S_{ds} a_p W_p \leq F_p \leq 1.6 S_{ds} a_p W_p \qquad \dots (2)$$

Where F_p represents the lateral seismic design force, S_{ds} is the site-specific short-period spectral acceleration, a_p denote the component amplification factor having ranged from 1.0 to 2.5, z is the height of the component to base, h is the total height of the buildingin accordance with base, Ip refer the component important factor, and R_p is the component response modification factor which shows the energy absorbed by the component andalso W_p is the weight of the component. The terms of $\left(1 + 2\frac{z}{h}\right)$ represent the floor acceleration amplification factor (Ω) of the NSCs.

2.1.2 IITK-GSDM

As per IS 1893: 2002[19], the seismic design RC frame structures were not provided the information on the non-structural components. Clause 7.12.2 states that the relation between NSCs and primary structure be designed for five times the horizontal design acceleration coefficient, multiplied by the weight of the component. The provision given by the IS 1893:2002 code was highly inadequate for the seismic design of secondary elements of the structures. IITK-GSDMA[20,21] proposed the NSCs seismic prevention in structures. The proposed equation of the "Ω" based on the height of the building was given as $\left(1 + \frac{z}{h}\right)$, where z is the height of NSCs and the h is the height of the building from the base of the structures. It observed that the maximum amplification of the structures non-structural component was equal to 2 when the z is similar to h. The above equation assumes that the relation between the peak ground motion and the peak floor acceleration is linear.

2.1.3 Akhlaghi and Moghadam

This paper observes the seismic behaviour of the rigid acceleration sensitive secondary elements linked with the main structure. It was marked 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 the building's height, connecting with the main structure. It was detected that the response of the floor or roof during the ground motion was the same as the response of the NSCs, so that it proposed the equations of the Ω based on the natural period of the structures is:

$$\Omega = 1 + (\alpha - 1)\left(\frac{h_i}{h_n}\right) \qquad \dots (3)$$

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 to the base of building and α represents the fundamental period-dependent factor, which was given as:

α =3 when T<0.5

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

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

Here, T represent the fundamental period of the structures.

2.1.4 Fathali and Lizundia

In this paper, Fathali marked the floor acceleration amplification factor not only depends on the height of the structure or fundamental period of the structure but also depends on the ground motion level. Based on it, they proposed the non-linear equation shown;

$$\Omega = 1 + \alpha \left(\frac{z}{h}\right)^{\beta} \quad \ldots (4)$$

Where z and h are the height of the NSCs and the height of the storey to the base. α and β are two parameters based on the building period and the degree of the ground motion, respectively.The values of α and β are shown in Table 1 and Table 2 respectively.

2.1.5 Joseph Wiser

This paper investigated the amplification factor of NSCs of four moment-resisting frame structures suited to far-field ground motion data. To generate the floor response spectra and observe that the ductility factor plays a significant role in determining the amplification factor of the structures. As per ATC 1996[22] define the effective period of the structures, given as:

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

$$T_{eff} = T\sqrt{\frac{\mu}{\alpha\,(\mu - 1)}}$$

With the help of this effective period, it proposed the amplification factor for any type of the ground motion data. The proposed model was given as:

$$\Omega = 1 + \left(\frac{T_{max} - T_{eff}}{T_{eff}}\right)\left(\frac{z}{h}\right) \quad \ldots (5)$$

Where

Ω=Acceleration Amplification factor

μ=Ductility factor

α=Post-yield stiffness ratio

T=Elastic Period

T_{eff}=Effective Period of the Structures

T_{max}= Maximum structural period for which the peak acceleration at the roof is not less than the PGA

2.2. Proposed model

The above discussion observed that no modal gives satisfactory results for calculating the acceleration amplification factor of the RC frame structures. The previous model represented that the floor acceleration amplification factor depends on either building height or the buildings natural period. However, the Wiser model marked that the amplification factor also affects the effective period of the structures. After the analysis, it was found that the ductility of the structure is a significant and vital aspect for defining the amplification factor. Therefore, to propose the amplification model, which depends on the floor response spectra, fundamental period of the structures and effective period of the structures, and depends on the ductility of the buildings. The proposed model is given as below:

$$\Omega = 1 + \left(\frac{T_{max} - T_{eff}}{\mu * \beta * T_{eff}}\right)\left(\frac{z}{h}\right) \quad \ldots (6)$$

Where Ω=Acceleration Amplification factor

μ=Ductility ratio

T= Elastic Period

T_{eff}=Effective Period of the Structures

T_{max}=Maximum structural period for which the peak acceleration at the roof is not less than the PGA

β=Constant which depends on the natural time period and the range of the seismic motion

Constant β values are shown in Table 3.

2.3 Building configuration

To determine the amplification factor, considered the four-moment resisting RC frame structures with different heights (Four, Six, eight and ten storey) with fixed support conditions that act above the hard soil. For all calculations presented here, chosen first and the other storey height is 4m and 3.4m, respectively. Incremental Dynamic analysis has been used for the study of the models. The two-dimensional models of the fixed supports are shown in Fig. 1. The size of the beams and columns is given in Table 4. The structures fundamental period is taken in the ranges of 0.1 to 1.5 seconds, and the damping ratio is 5%. For the analysis of all the models, finite element software is used. All four-building plan is regular.

Table 3 — Various value of β based on natural period of structures and range of ground motion

Ground Motion	β	Natural period of the structure (sec.)
0.01g to 0.067g	1.80	$T < 1.0$
	0.95	$1.0 \leq T \leq 1.5$
0.067g to 0.2g	2.00	$T < 1.0$
	1.60	$1.0 \leq T \leq 1.5$
0.2g to 0.31g	1.80	$T < 1.0$
	1.30	$1.0 \leq T \leq 1.5$

2.4 Selection of Ground Motion

For the study of moment-resisting RC frame models, 17 far-field time history data are considered with the ground motion intensity up to 0.7g. This data is obtained from the strong ground motion virtual data centre[23]. Details of recorded ground motion data are given in Table 5.

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

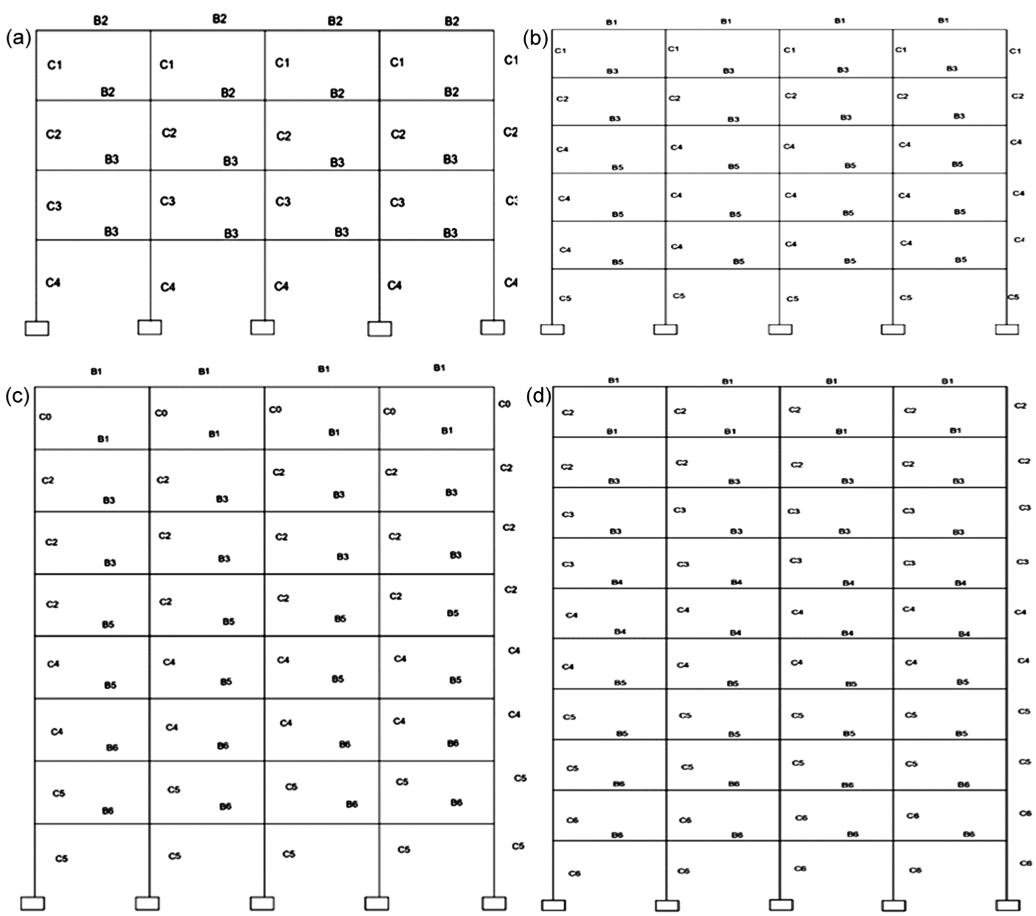

Fig. 1 — Moment resisting frame models (a) 4, (b) 6, (c) 8, and (d) 10 stories.

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

Ground Motion Name	PGA (g)	T (sec)	T_p (sec)
Chi-chi 1	0.2296	40	8.015
Chi-chi 2	0.2167	60	11.94
Chi-chi 3	02061	45	17.615
Chi-chi 4	0.2252	50	10.04
Chi-chi 5	0.2779	55	5.8
Chi-chi 6	0.2347	60	15.22
Chi-chi 7	0.2678	68	15.39
Chi-chi 8	0.2818	45	12.15
Chi-chi 9	0.2164	50	19.62
Kobe 1	0.562	30	7.7
Kobe 2	0.523	25	3.68
Northridge 1	0.33	24	4.92
Northridge 2	0.58	32	9.12
Northridge 3	0.75	30	8.89
Northridge 4	0.43	30	8.83
Sierra Madre	0.28	35	2.46
Oyama	0.44	50	18.4

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

2.5 Incremental Dynamic Analysis

For obtaining the database of floor response acceleration, incremental dynamic analysis is used. A total of 340 non-linear time history analyses were performed with varying seismic motion levels, as marked by the peak ground acceleration (PGA). The numerous response variables were supervised: peak floor accelerations (PFA), maximum roof and interstorey roof drift, and floor spectral acceleration (FSA). Multiple relationships were derived involving Interpolation and a combination of IDA curves. Counted statistics were used for generating the maximum, mean and mean+ standard deviation (Mean+SD) values. Based on spline interpolation, an IDA curve was generated. IDA Curve aligns with the piecewise polynomial function of discrete data obtained after analysing the structures. An improved hunt and fill algorithm[24]were executed to reduce the required analysis. This algorithm was improved "to hunt" for seismic motion intensity, which generates a drift response within a pre-set limit of 4% to 6% maximum inter-story drift. It is marked that these models obtained don't take into account the strength degradation factor; the collapse was expected to happen at 6% interstorey drift.

2.6 Modelling of the structure

All the building models were analyzed using finite element-based software. For the building frame, the centreline dimension is used. Gravity force-resisting components are modelled using flexible beam-column elements, whereas horizontal force-resisting components are modelled with plastic hinge elements to account for material nonlinearity. P-M2 type plasticity hinges are provided in beam elements; however, P-M2-M3 type hinges are provided in the column section. Using the co-rotational formulation, the geometric nonlinearities, P-Delta effects, were accounted for[25]. The slab of the buildings has been modelled using thin shell elements. For all models, the modal mass participation ratio (defined as the amount of building mass that participates in each mode of structural response) is set to greater than 90%[21].

3 Results and Discussions

3.1 Floor Response Spectra

The peak floor acceleration demand can be used when the weight of the acceleration-sensitive NSCs is higher than the weight of the structures. However, when acceleration-sensitive NSCs weight is low compared to the weight of the buildings, the floor spectra concept is used. Figure 2 presents the spectral acceleration's nature at various floor levels of the building model when the ground motion range is up to 0.7g. From Fig. 2, it is evident that the floor spectral acceleration is higher when the natural period of structure is lower, and it reduces with an increase in fundamental period of the structures. It marked that when the building height increases, the spectral acceleration values are decreases approximately 1.5 times with respect to the buildings lower height.

3.2 Nature of peak floor acceleration with various seismic motion

Figures 3&4 presented the comparison of Peak floor accelerations enumerated in the four buildings model to those recorded during the Kobe and Northridge earthquakes. It can be seen that the peak roof acceleration is 1.5 to 2 times higher than the recorded acceleration, which acts at the base of structure. The peak floor acceleration performed the linear behaviour up to normalized height 0.3. However, it marked nonlinear nature as the normalized height higher than 0.3. It also notices that peak floor acceleration demand decreases as the structures natural period increases.

3.3 Effect of natural period and effective period over the normalized building height

Based on incremental time history analysis, it was observed that the structures natural period is affecting

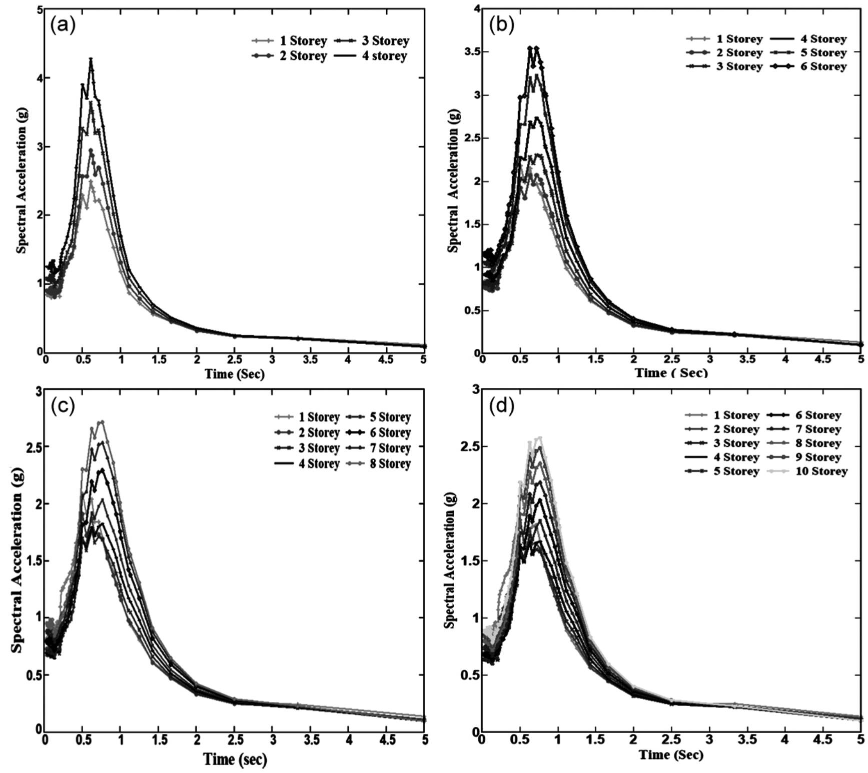

Fig. 2 — Variation of Spectral acceleration for fixed support condition (a) 4 stories, (b) 6 stories, (c) 8 stories, and (d) 10 stories.

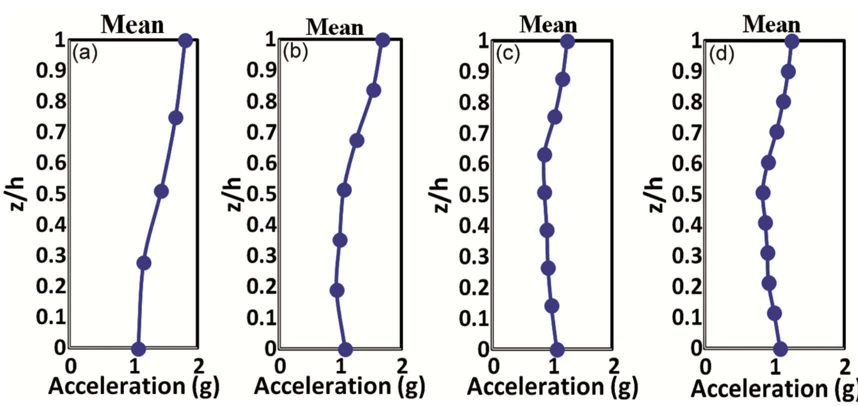

Fig. 3 — Behaviour of PFA over the normalize height for Northridge earthquake of (a) 4 stories, (b) 6 stories, (c) 8 stories, and (d) 10 stories.

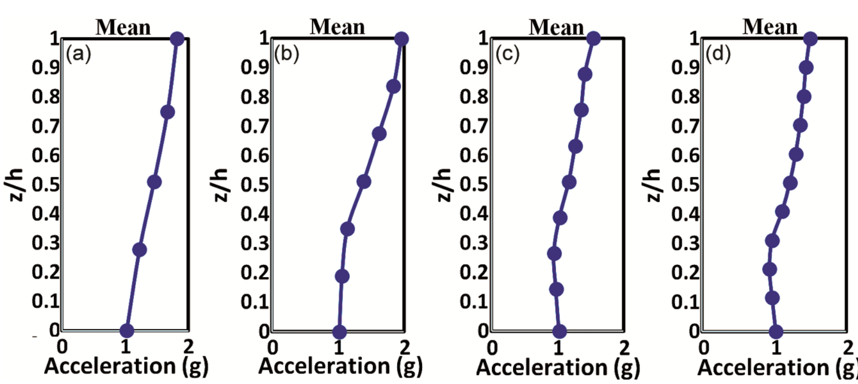

Fig. 4 — Behaviour of PFA over the normalize height for kobe earthquake of (a) 4 stories, (b) 6 stories, (c) 8 stories, and (d) 10 stories.

the building amplification factor. Figure 5 illustrates the mean and Mean+SD of PFA/PGA of four different height levels compared with the previously proposed model defined in equations 2 to 6.

It marke's that the floor amplification factor decreases with increase in building period. ASCE, Fathali and IITK model performed the constant values for natural period of the structures up to 1.5 sec. However, Wiser model notifies that as the building period increases, PFA/PGA decreases. Akhlaghi model observed that the amplification values are high as the fundamental period of the structure is low and reduced as the building period is high. Apart from the previous model, the proposed modal performed very close results to Mean+SD results.

Since the yielding of building causes prolongation of the fundamental period during the response time history, the degree of yielding experienced in the buildings influences the PFA/PGA proportion. The ductility ratio helps in measuring the global structural yielding. The ductility ratio defines, the ratio between maximum roof drift to yield roof drift. As per ATC 1996, given the concept of the effective period of the structure for assuming the elastoplastic hardening, it depends on the structure ductility ratio.

$$T_{eff} = T\sqrt{\frac{\mu}{\alpha\,(\mu-1)}}$$

Here T_{eff} is the effective period, T is the elastic fundamental period, μ represents the drift ductility, and α represents the post-yield stiffness ratio (define as the ratio between post-yield stiffness to initial stiffness). In this paper, the yield drift is obtained based on the idealizing of the first mode of pushover analysis, however, the roof drift is used to find out the ductility, respectively. When the drift ductility is less than one, the structure performed the truly elastic response corresponding to PFA/PGA ratio. Since the effective period depends on the ductility, to account for the effect of structural yielding, the acceleration amplification factor (PFA/PGA) is proposed in equation 6.

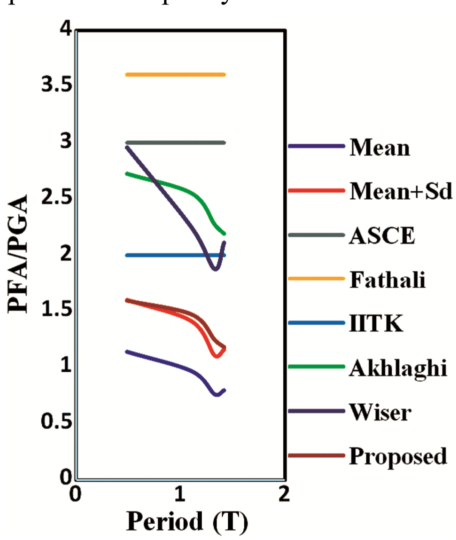

Fig 5 — Comparison ofpeak roof acceleration amplification with respect to building period.

With this improved illustration of the floor acceleration amplification factor, the seismic design forces on NSCs may be obtained by:

$$F_p = 0.4S_{ds}a_p\left(\frac{I_p}{R_p}\right)\left\{1+\left(\frac{T_{max}-T_{eff}}{\mu*\beta*T_{eff}}\right)\left(\frac{z}{h}\right)\right\}W_p \quad \ldots (7)$$

3.4 Comparison of Acceleration Amplification Factor

Figure 6 shows the nature of the PFA with respect to the height of the building for various PGA ranges. It observed that the amplification factor at the top of the building is inversely proportional to the structure natural period. Based on the shape of PFA, the outcome is that the natural period of the building over the height of the building is higher under the strong ground motions compared to moderate and minor seismic motion. When the ductility ratio of the structure is known to find out the effective period of the structure, using equation 6 defined by ATC 1996. This effective period used for determining the amplification factor of the structures. The Mean+SD amplification factor obtained after the analyses are compared with the previously proposed amplification model. The nature of the Mean+SD amplification factor of PFA is non-linear. It observed that the Fathali amplification model of PFA is approximately 2 times higher than Mean+SD results when the building period increases up to 1.5 sec. It notifies that Fathali model performed conservative results as the height of the building increases. The amplification factor of PFA observed by ASCE is approximately 1.5 times higher than the Mean+SD results. ASCE amplification model also performed obscure results than the Mean+SD results. IITK, Wiser and Akhlaghi models also performed the obscure results compared to Mean+SD amplification results and its values are approximately 70%, 80% and 85% higher than Mean+SD amplification results. The performance of the proposed acceleration amplification factor of PFA is satisfactory compared to other models.

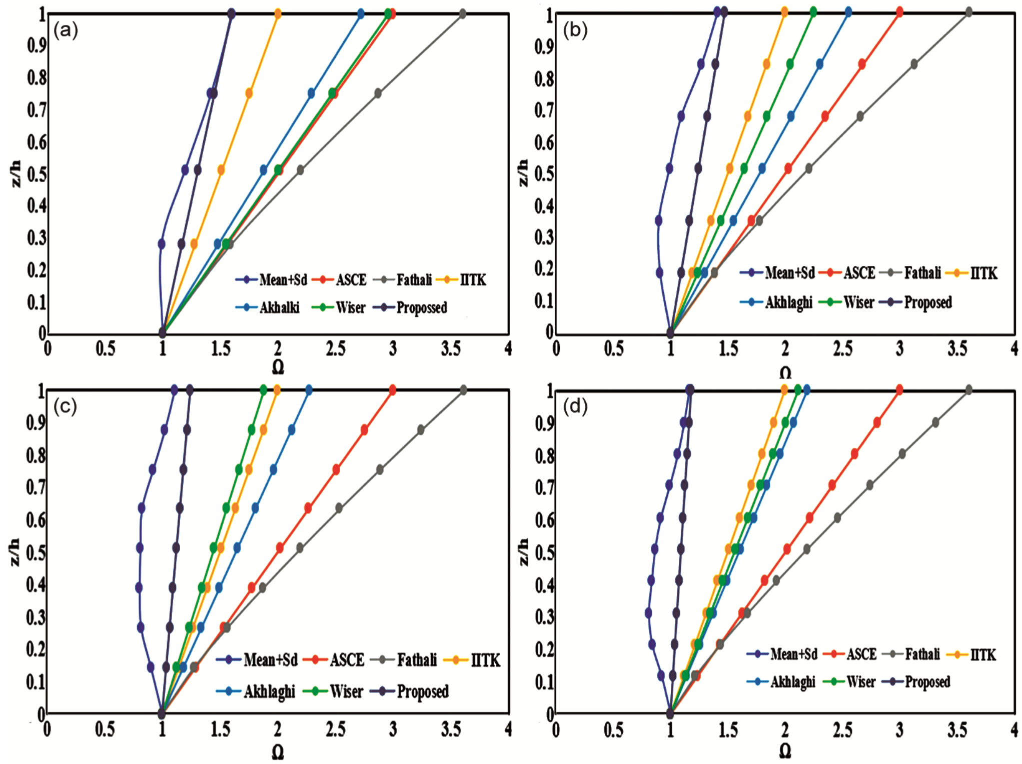

Fig. 6 — Comparison of the acceleration amplification model with respect to normalized height of (a) 4 stories, (b) 6 stories, (c) 8 stories, and (d) 10 stories.

3.5 Component Amplification Factor

For NSCs, floor response spectra (FRS) presented the peak acceleration responses on that element. Fig. 7, marked the component acceleration amplification factor to the component period. The ratio between FRS/PFA has presented the component acceleration amplification factor and is denoted as a_p. ASCE7-10 design code states that the all-elastic NSCs have a component amplification factor of 2.5. however, for rigid components (component period less than 0.06 sec), its values are 1. Fig. 4 represents the mean and Mean+SD, a_p values for the top of the six-storey building. It notifies that a_p values are not always less than 2.5 as given by ASCE code. Mean and Mean+SD component amplification factor reached 2.5 at the component period of 0.5 sec and 0.4 sec. The Mean+SD components acceleration amplification values are approximately 1.5 times higher than the ASCE code. The maximum Mean+SD amplification factor values are observed in 0.6 sec. It observed that a_p values given by the ASCE code are conservative.

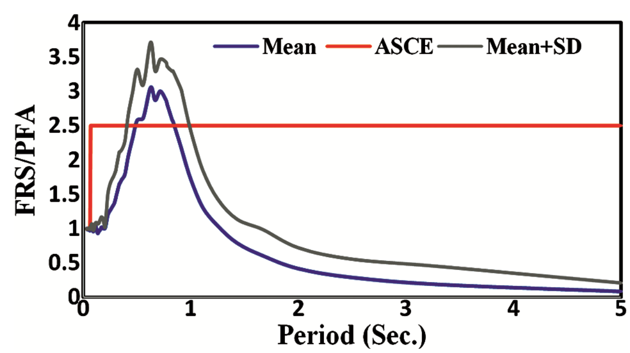

Fig. 7 — Comparison of the Mean+SD component amplification factor to ASCE model.

4 Conclusions

In this paper, four different models 4, 6, 8, and 10 stories have been considered. An incremental dynamic analysis suite of 17 far-field ground motion data has been used to examine various heights' four-moment resisting RC frame models. The acceleration amplification factor has been presented using several parameters such as building period, floor response spectra, and the structures ductility ratio. The proposed amplification factor has been compared to existing models of amplification factor. The following conclusions can be drawn:

- The ASCE code acceleration amplification values are approximately 1.5 to 2 times higher than Mean+SD results. It performed obscure results as the building period increases up to 1.5 sec.

- The component amplification factor given by the ASCE code is conservative. It shows a constant value of 2.5 when the flexible component period has been higher than 0.06 sec. However, a_p values are not constant as the component period is higher than 0.06 sec; sometimes it resulted in higher values and vice versa.
- Fathali models depend only on the building period and have not depended on another parameter. It marked that the amplification values are approximately 2 times higher than Mean+SD results. These models also performed obscure results for building periods up to 1.5 sec.
- IITK, Wiser and Akhlaghi models performed better results as compared to the ASCE and Fathali models, but it also observed conservative results compared to Mean+SD results.
- The proposed model performed satisfactory results with respect to the other models.

This investigation focused on the moment-resisting RC frame structures; hence, the results and conclusions derived herewith may not represent shear wall or steel frame structures.

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