CHAPTER 7

FLOOR ACCELERATION AMPLIFICATION FACTOR FOR NON-LINEAR MOMENT RESISTING RC FRAME STRUCTURES

This chapter describes the behaviour of the nonlinear acceleration-sensitive NSCs for different intensities of ground motions. For this study, the four different height of moment-resisting RC frame models fixed at the base of the structure is considered. All these models have been analysed by incremental dynamic method with 17 far-field seismic ground motions. The acceleration amplification factors were proposed to illustrate the floor response spectra, building periods, and structure ductility parameters. The proposed amplification factors are compared with the previously proposed models

7.1 PROPOSED MODEL

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 is also affected by the effective period of the structures. After the analysis, it was found that the ductility of the structure is a significant and important aspect for defining the amplification factor. Therefore, the proposed amplification model, which not only depends on the floor response spectra, fundamental period of the structures and effective period of the structures but also depends on the ductility of the buildings. The proposed model is given below:

$$\Omega = 1 + \left(\frac{T_{max} - T_{eff}}{\mu * \beta * T_{eff}}\right) \left(\frac{z}{h}\right) \tag{7.1}$$

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 roof acceleration is greater than or

equal to the PGA

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

Table 7.1 β Value based on the ground motion and the natural period of the building

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

7.2 BUILDING CONFIGURATION

Four-moment resisting RC frame structures with different heights (Four, Six, eight and ten storey) are considered for fixed support condition above the hard soil to determine the amplification factor. 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 Figure 7.1. The size of the beams and columns are given in Table 3.1, respectively. The structures' fundamental period is taken in the ranges of 0.1 to 1.5 seconds, and the damping ratio is 5%.

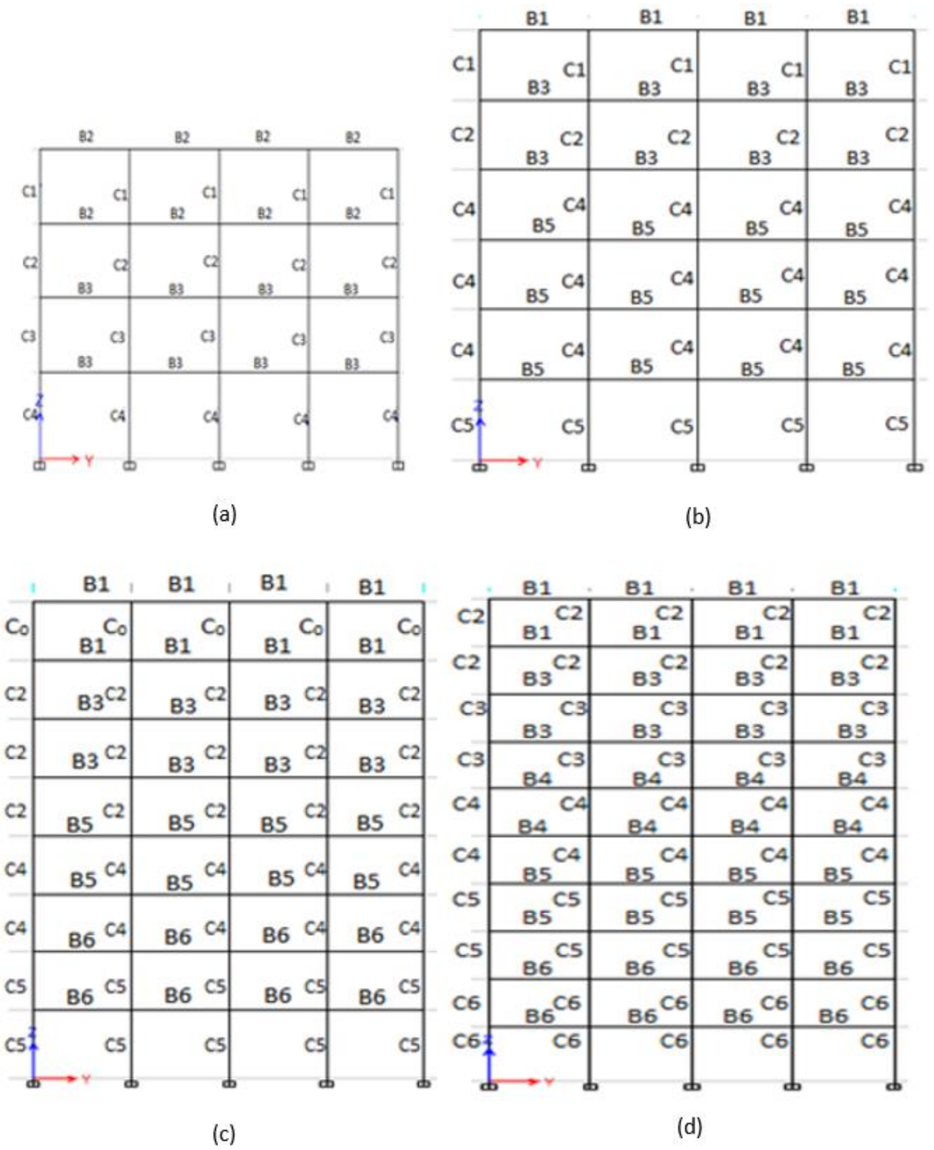

Figure 7.1 Moment resisting frame models (a) 4 (b) 6 (c) 8 and (d) 10 stories

7.3 SELECTION OF GROUND MOTIONS

For the study of moment-resisting RC frame models, 17 far-field time history data is considered with the ground motion intensity up to 0.7g. All these data are obtained from the strong ground motion virtual data centre [140]. Details of the ground motion data are given in Table 7.1.

Table 7.2 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

7.4 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 intensity levels, as marked by the peak ground acceleration (PGA). The numerous response parameters peak floor accelerations (PFA), maximum roof and inter-storey roof drift and floor spectral acceleration (FSA) were supervised. Numerous relationships were derived with the help of interpolation and combination of the IDA curve. Counter statistics were used for generating the maximum, mean and mean+ standard deviation (mean + sd) values. Based on spline interpolation, to generate the IDA curve. In the spline interpolation method to create the piecewise polynomial function of discrete data got after the analysis of the structures.

IDA curve was created using spline interpolation, which fits piecewise polynomial functions to the discrete data points obtained from the analysis runs. To reduce the number of the required analysis, a modified hunt and fill algorithm [28] was executed. This algorithm was improved “to hunt” for the seismic motion intensity, generating a drift response within a predetermined window of 4% to 7% maximum inter-story drift. It is marked that strength degradation is not considered for this study, and it assumes that when the inter storey drift reached 7%, collapse occurred.

7.5 RESULTS AND DISCUSSION

7.5.1 Floor Response Spectra

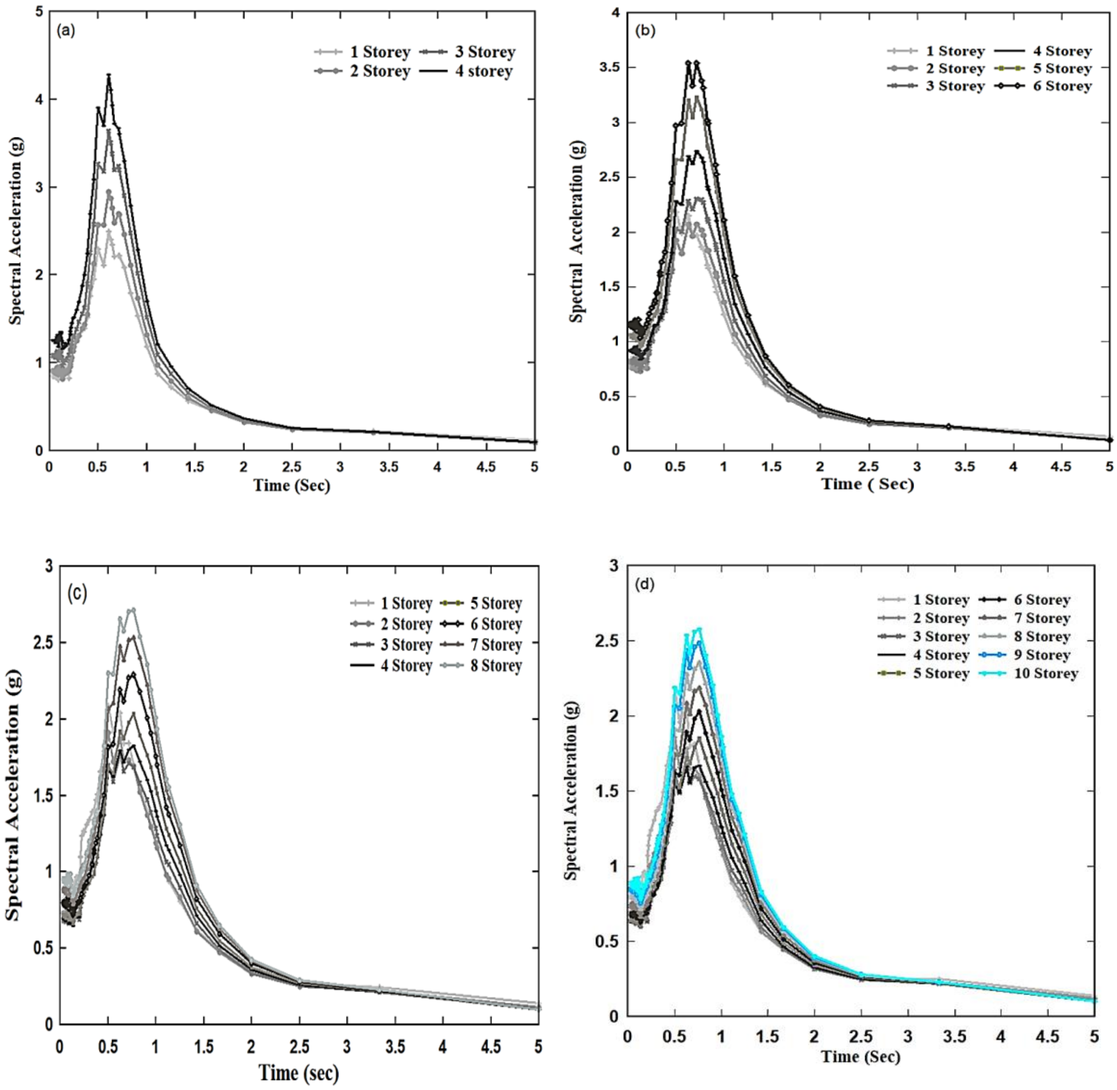

Figure 7.2 Nature of Spectral acceleration of 4,6,8 and 10 storey

The peak floor acceleration demand can be used when the weight of the acceleration 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 7.2 present the nature of the spectral acceleration at various floor levels of the building model when the ground motion range is up to 0.7g. It observed that the floor spectral acceleration is higher when the building period is low and decreases as the fundamental period of the structures increases. It marked that when the building height increases, the spectral acceleration values decrease approximately 1.5 times with respect to the buildings lower height.

7.5.2 Nature of peak floor acceleration with various seismic motion

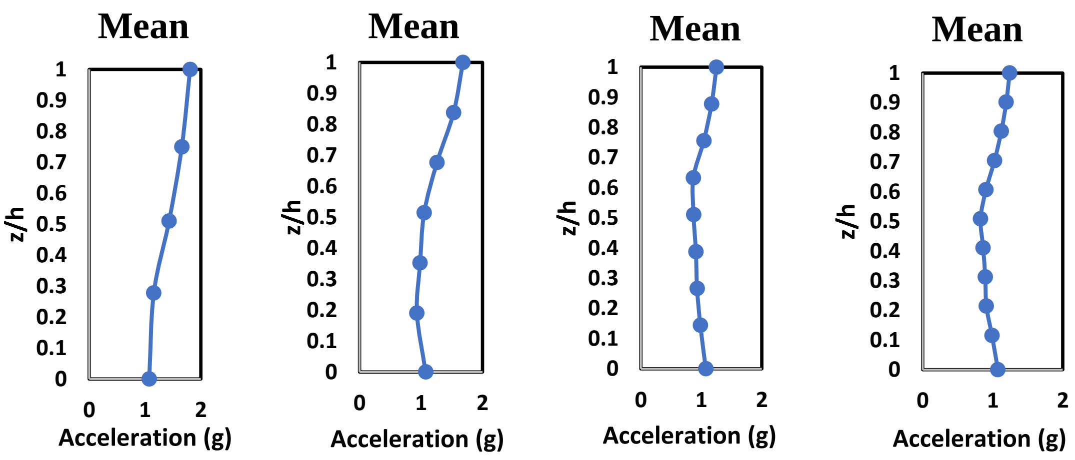

Figure 7.3 Behaviour of PFA over the normalize height for Northridge earthquake

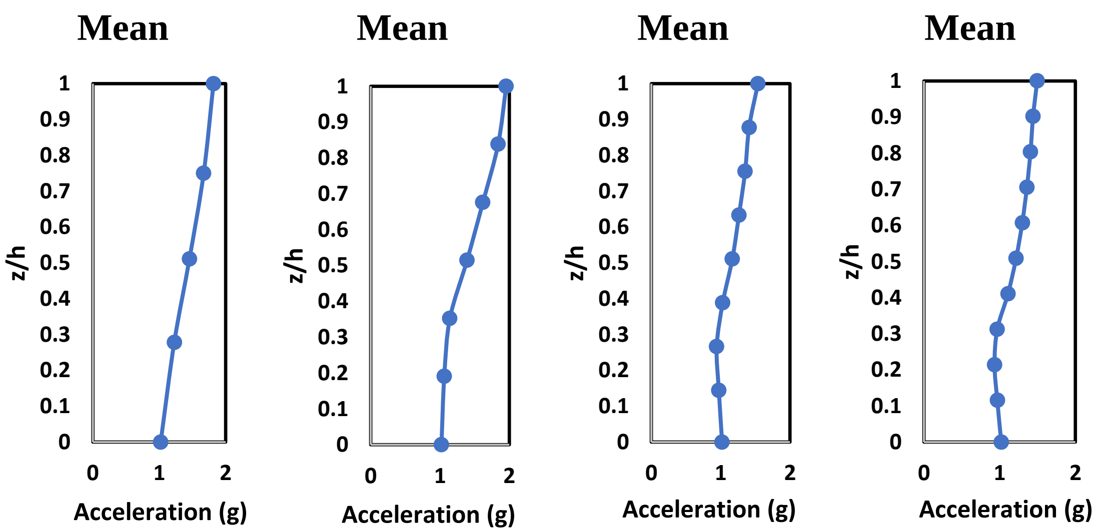

Figure 7.4 Behaviour of PFA over the normalised height for Kobe earthquake

Figure 7.3 and 7.4 present 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 the buildings. The peak floor acceleration performed the linear behaviour up to normalized height 0.3. However, it marked nonlinear nature as the normalised height was higher than 0.3. It also notices that peak floor acceleration demand decreases as the natural period of the structures increases.

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

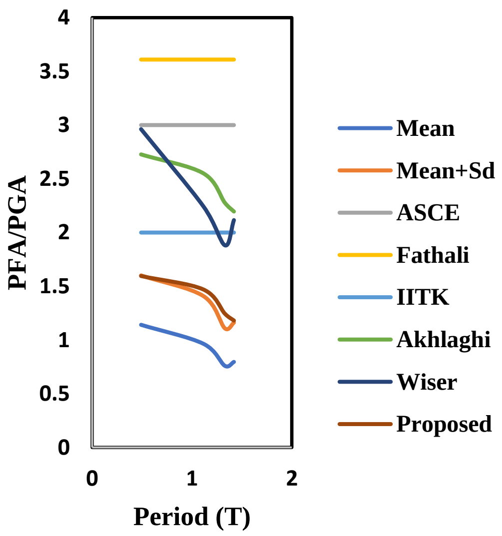

Figure 7.5 Comparison of peak roof acceleration amplification with respect to building period

Based on incremental time history analysis, it is observed that the structure's natural period is affected by the building amplification factor. Figure 7.5 illustrates the mean and Mean+Sd of PFA/PGA of four different height levels compared with the previously proposed model as defined in equations 2 to 6.

It is marked that the floor amplification factor decreases as the building period increases. ASCE, Fathali and IITK model performed the constant values for the natural period of the structures up to 1.5 sec. However, the 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 decrease as the building period 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. For measuring the global structural yielding, the ductility ratio is used. 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 structures ductility ratio.

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

Here T_{eff} is the effective period, T is elastic fundamental period, μ represents the drift ductility, and α represents the post-yield stiffness ratio. In this chapter, the yield drift is obtained based on the idealizing of the first mode of push over analysis; however, the roof drift is used to determine 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 10.

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

$$F_p = 0.4 S_{ds} a_p \left(\frac{I_p}{R_p}\right)\left\{1 + \left(\frac{T_{max} - T_{eff}}{\mu * \beta * T_{eff}}\right)\left(\frac{z}{h}\right)\right\} W_p \qquad (7.2)$$

7.5.4 Comparison of Acceleration amplification factor

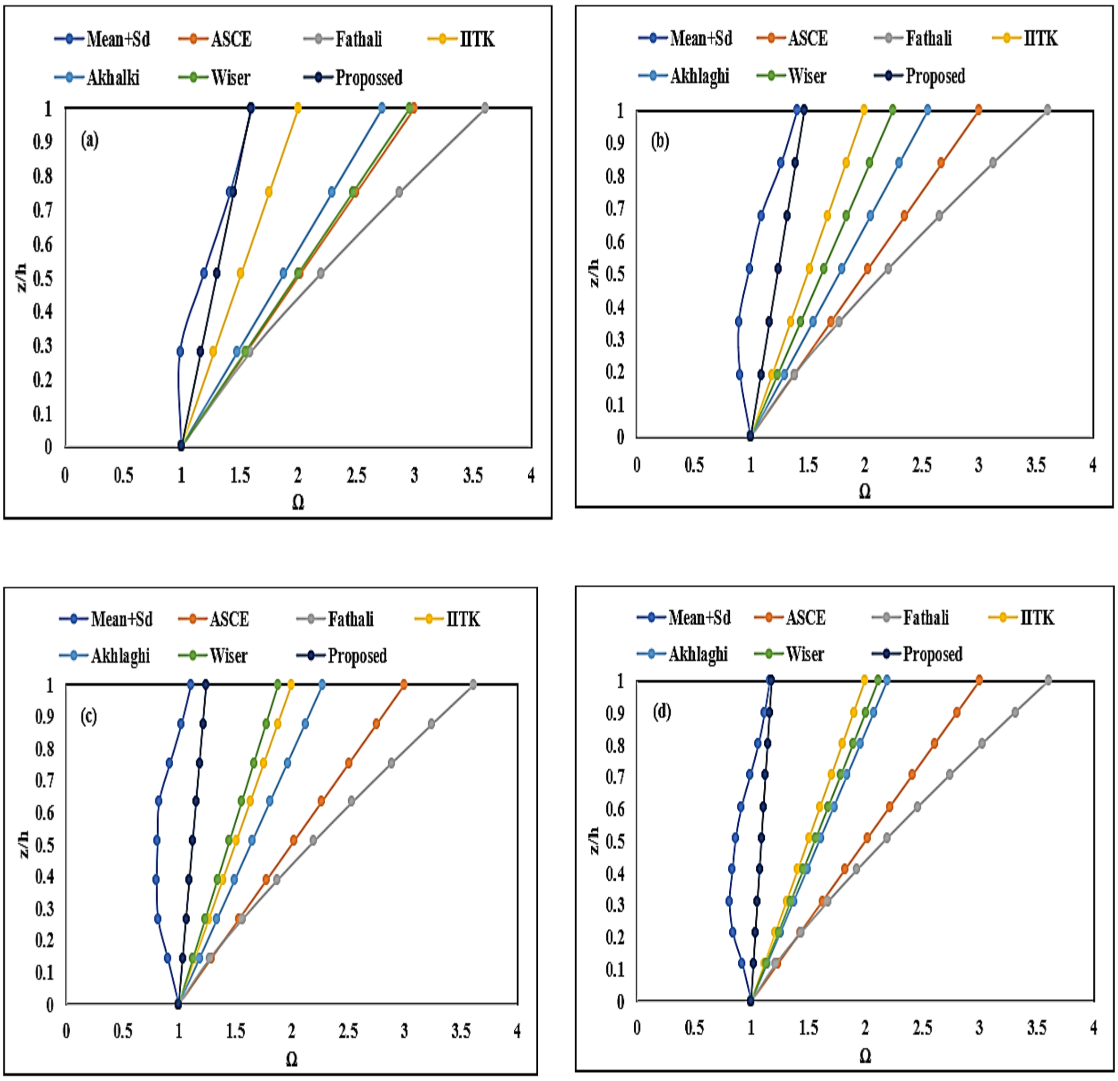

Figure 7.6 Comparison of the acceleration amplification model with respect to normalised height of the building

Figure 7.6 shows the shape of the PFA over 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 structures' 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 is 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 two 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 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 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.

7.5.5 Component Amplification Factor

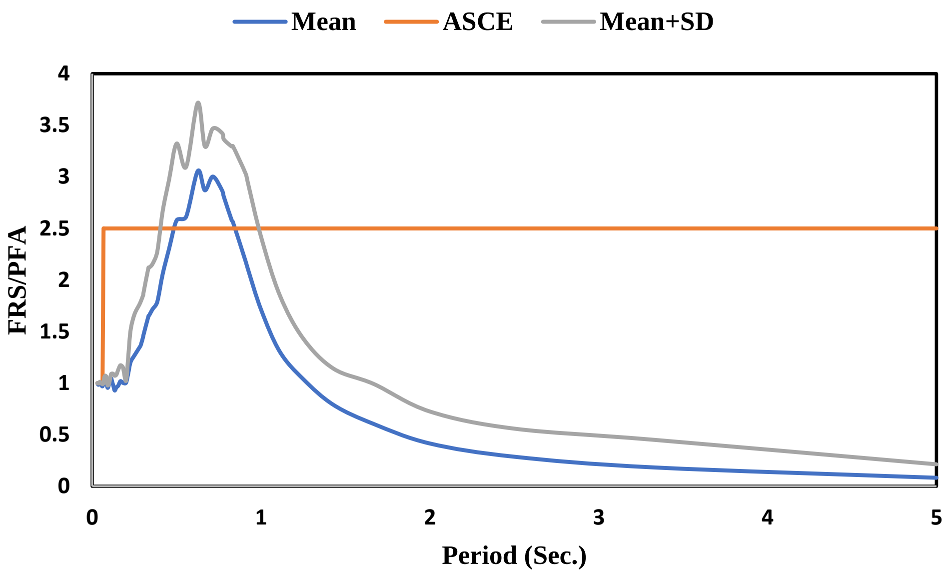

Figure 7.7 Comparison of the mean +sd component amplification factor to ASCE model

For NSCs, floor response spectra (FRS) presented the peak acceleration responses on that element. Figure 7.7 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 state that the all-elastic NSCs having component amplification factor is 2.5. however, for rigid components (component period less than 0.06 sec), its values are 1. Figure 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. The 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 ASCE code. The maximum mean +sd amplification factor values are observed in 0.6 sec. It observed that a_p values given by ASCE code is conservative.

7.6 CONCLUDING REMARKS

This chapter studies floor acceleration behaviour using the typical parameters for determining the acceleration demand on NSCs. This took four moment resisting RC frame models with different height and analysed by an incremental dynamic analysis suite of 17 far-field ground motion data. Using the various parameters as; building period, floor response spectra, and the ductility ratio of the structure, proposed the acceleration amplification factor. The proposed amplification factor is compared to the previously proposed amplification models. The following conclusions are given:

- 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 increased up to 1.5 sec.
- The component amplification factor given by the ASCE code also performed unsatisfactory results. It shows a constant value of 2.5 when the flexible component period is higher than 0.06 sec. However, a_p values are not stable as the component period is higher than 0.06 sec; sometimes, it performed higher values and sometimes performed lo values.
- Fathali models depend only on the building period and not considered the other parameters. It marked that the amplification values are approximately two 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 compared to the ASCE and Fathali models, but it also observed conservative results compared to mean+sd results.
- The proposed model performed satisfactory results to the other models.

This investigation focused on the moment-resisting RC frame structures; therefore, the results and recommendations presented herein may not represent shear wall or braced frame structures.