FLOOR ACCELERATION SPECTRUM FOR RC MOMENT FRAME BUILDINGS ON SLOPES

A.R. Vijayanarayanan[(1)], and R. Goswami[(2)]

[(1)] *Former Doctoral Student , Indian Institute of Technology Madras, India, arvn.iitm@gmail.com*

[(2)] *Associate Professor, Indian Institute of Technology Madras, India, rg@iitm.ac.in*

Abstract

It is prudent to build critical buildings (such as hospitals, power grids, and fire stations) on flat competent ground. Nevertheless, in hilly terrain, such buildings are built on hill slopes resulting in the height of the building changing along the slope (Fig 1). Therefore, it is essential to develop *floor acceleration spectra* (FAS) for such buildings to aid the seismic design of acceleration sensitive non-structural elements. This study considers reinforced concrete buildings with special moment frame to assess the applicability of code recommended equations to estimate *peak floor acceleration* (*PFA*) for the design of non-structural elements. The study buildings are designed conforming to the existing Indian seismic design standard. Subsequently, *FAS* and *PFA* for the said buildings are estimated using Linear and Nonlinear Time History Analyses (*LTHA* and *NTHA*) using 20 natural earthquake ground motions. Also, quantified in this study is the effect of unreinforced masonry infill walls on the *FAS* and *PFA*. Results of *NTHA* indicate that the *FAS* and *PFA* vary within a given floor - the *PFA* near the flexible frame is about 30-50% more than the same near the stiff frame. This difference is due to the twisting of the building while resisting ground motions perpendicular to the slope along which it is built. Further, comparison between models with and without infill indicates the said difference is less by about 10% for buildings in which unreinforced masonry infills are present. Finally, a comparison is presented of the results obtained with those recommended in seismic design codes – the code recommended amplification is found to be adequate for low-rise reinforced concrete buildings on slope.

Keywords: Non-structural elements; seismic design; torsional irregularity; infill walls

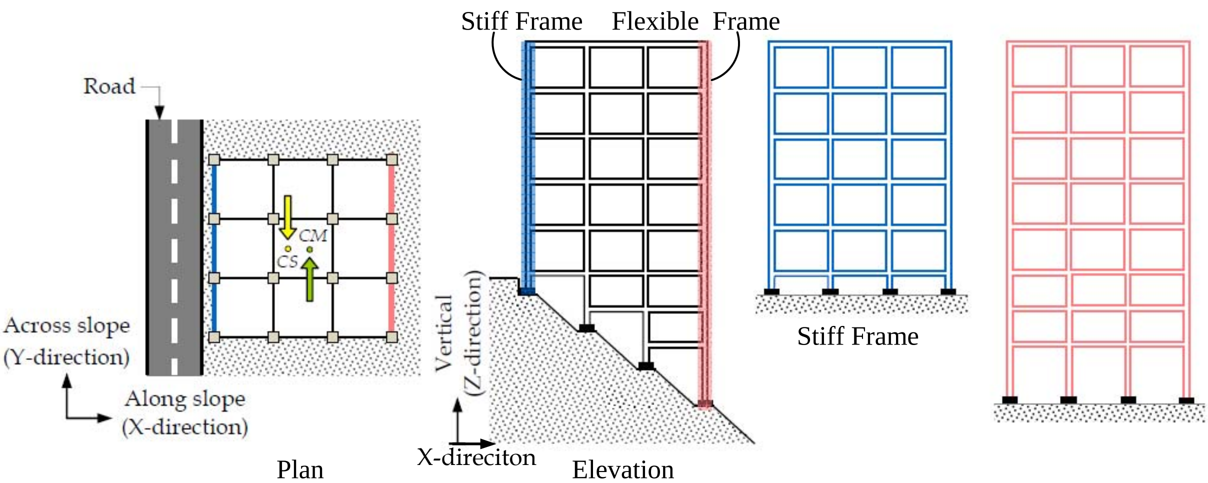

Fig. 1 – Plan and Elevation of a typical RC frame building on slope

1. Introduction

Non-structural elements (*NSE*s) are generally classified into two categories, namely *acceleration sensitive* and *displacement sensitive* elements for seismic design. As the names suggest, floor accelerations and relative displacements govern the demand of acceleration and displacement sensitive *NSE*s, respectively. For acceleration sensitive *NSE*s, code recommended design lateral force (F_{NSE}) is:

$$F_{NSE} = \frac{(S_g/g)a_{NSE}}{(R_{NSE}/I_{NSE})}\left(1+\alpha\frac{z}{h}\right)W_{NSE}, \tag{1}$$

where (S_g/g) is the expected intensity of ground shaking at the location where the building is present; a_{NSE}, R_{NSE}, I_{NSE} and W_{NSE} are the amplification factor, response modification factor, importance factor and seismic weight of *NSE*s, respectively; z is the height of storey, in which the *NSEs* are present, measured from the base of the building; h the height of the building; and α is a constant with values ranging between 1 and 3 [1-3]. In Eq. (1), the term $(1+\alpha(z/h))$ represents the amplification in *Peak Floor Acceleration* (*PFA*) along the building height.

In the past, several studies quantified the amplification of *PFA* for buildings, (a) that remain elastic while resisting earthquake shaking, (b) that sustain varying degrees of damage while resisting earthquake shaking, (c) in the presence of *Un-Reinforced Masonry* (URM) infills, (d) with different lateral load resisting characteristics and (e) that resists ground motions with different characteristics (such as near-field and far-field motions) [4-8]. One of the limitations of those studies is that the variation of *PFA* is determined using numerical (or analytical) models of buildings founded on flat ground. Hence, the reported variation of *PFA* is not best suited for buildings that are founded on a *hill slope*. The lack of suitability is because, unlike buildings founded on flat ground, buildings founded on hill slope have inherent torsional irregularity (Fig. 1). This torsional irregularity cases twisting of such buildings about the vertical axis during earthquake shaking oriented across the hill slope. Given this, there is a need to estimate the variation of *PFA* for the said building typology.

2. Numerical Study

2.1 Details of Study Buildings

Twelve buildings are considered in this study. The buildings are dividend into two sets of six buildings; the first set of six buildings *consider* the contribution of lateral translational stiffness offered by *URM* infills while the other set of six buildings *does not consider* the said contribution (called the bare frame buildings). Further, the six buildings in each set are subdivided into three pair of buildings; each pair of buildings are considered present on one of the three hill slopes ($30°$, $40°$ and $50°$). Furthermore, among the two building on the same hill slope, one has 3-bays along the hill slope while the other has 4-bays along the hill slope. Thus, to denote each building,

(a) mentioned first is the steepness of hill slope on which the building is present,
(b) next, the number of bays along the hill slope is mentioned, and
(c) lastly, suffix “IN” and “F” are used to denote buildings which consider and does not consider the lateral translational stiffness contribution of URM infills, respectively.

For example, the nomenclature 404F denotes a building (a) on a hill slope of $40°$ angle, (b) with 4-bays along the hill slope, and (c) that does not consider the lateral stiffness contribution of URM infills, *i.e.*, it is a bare frame building. In addition to the 12 buildings, two benchmark buildings are considered, namely the BMF and BMIN. The BMF does not account for the lateral translational stiffness offered by URM infills; in contrast, the BMIN accounts for the lateral translational stiffness offered by infills. Notwithstanding the said difference, the BMF and BMIN (a) have 5-storeys, (b) have 3-bays in both X- and Y-directions and (c) are founded on flat ground. Lastly, to quantify the influence of inelastic action, both *elastic* and *inelastic* models of the said buildings are considered.

Typical plan and elevation of a building (*303F*) are shown in Fig. 1. All buildings are assumed located in Zone V as defined in Indian Seismic Design Code [2]. Following are standard features of all the buildings: (a) bay length is 4m, (b) storey height is 3m, (c) uniform design dead load in all storeys except the top storey is 24kN/m^2 (in top storey the same is 10.4kN/m^2), (d) uniform design live load is 3kN/m^2, (e) beams are of same size, (f) columns have square cross-section, and (g) column bases are assumed fixed. Further, outer and interior frames, in both X-and Y-directions, are considered with full thick (*i.e.*, 230mm) and half thick (*i.e.*, 110mm) URM infills, respectively. Also, the URM walls are assumed to have a compressive strength of 3MPa. The lateral force demand on all buildings is estimated using the Indian Seismic Design Code [2]. All buildings are designed following the Indian Concrete Design Code [9]; all members are designed considering M30 concrete and Fe415 reinforcing steel. Other key details, such as (a) member sizes, (b) longitudinal reinforcement in members, and (c) transverse reinforcement in members, are present in literature [10].

2.2 Modelling of Buildings and Method of Analyses

Both *PFA* and *FAS* of designed buildings is assessed using a commercially available structural analysis software Perform 3D (version 6) [11]. Key details of *elastic* models of the buildings are: (a) members are modelled using lineal elements, (b) beam-column joints are considered stiff and strong (*i.e.*, no damage is expected in the beam-column region), (c) seismic mass is lumped at the beam-column joints, (d) slabs are not modelled and integral action of slab, and beams is not considered when estimating the flexural strength and stiffness of beams, and (e) the ratio of *effective rigidity* to *gross rigidity* of beams and columns are considered to be 0.4 and 0.7, respectively [12]. In addition, following are the key details of the *inelastic* models of the buildings: (f) only flexural yielding of members is considered, assuming all other modes of failure to be avoided through capacity design and detailing of members, (g) inelasticity in beams is modelled using lumped plasticity model (more specifically, a moment-curvature relationship with 0.5D as the assumed length of plastic hinge), and (h) inelasticity in columns is modelled by considering fiber section [10].

URM infills are modelled as diagonal struts in buildings that consider the contribution of lateral translational stiffness offered by *URM* infills (Fig. 2). The cross-sectional area of a diagonal strut and its modulus of elasticity are estimated using the recommendations in Indian Seismic Design Code (considering a masonry strength of 3MPa) [2]. In the *elastic* models of buildings, an *elastic bar* element is used to model *URM* infills. Also, each elastic bar element is sized with half the cross-sectional area of the diagonal strut; the cross-sectional area is halved as both diagonal struts contribute to lateral translational stiffness while resisting lateral force. Alternatively, in the *inelastic* model of buildings, *inelastic* bar element is used to model *URM* infills. Also, each inelastic bar is modelled to have (a) near zero tensile capacity (=10N) but with large displacement capacity, and (b) a tri-linear simplified compression load-deformation response as specified in literature [13, 14]. Due to the low tensile capacity, the diagonal strut disengages while sustaining tensile force and does not contribute to the lateral translational stiffness; for this reason, each inelastic bar element is sized with full cross-sectional area of the diagonal strut.

Modal analysis is used first to identify the dynamic characteristics of buildings. *Linear Time History Analysis* (LTHA) and *Nonlinear Time History Analysis* (NTHA) are used then to obtain *PFA* and *FAS* of all the buildings. A total of 20 components of 10 natural earthquakes are considered in this study (Table 1). The elastic response spectra of the considered unscaled ground motions are shown in Fig. 3. Further, all buildings are subjected to earthquake shaking along Y-direction only (*i.e.*, across the slope); this is done to quantify the influence of torsional irregularity in buildings on *PFA* and *FAS*. Furthermore, in both *LTHA* and *NTHA*, all ground motions are scaled using the *Peak Ground Acceleration Scaling* method. In *NTHA*, crushing of confined concrete in columns is the limit state that is used to stop the analysis. Lastly, both *LTHA* and *NTHA* is performed considering 5% *Rayleigh damping* at 0.25 and 0.90 of the fundamental natural period of the a building in the direction of shaking [11].

Table 1 – Characteristics of the suite of 10 ground motions considered in this study

No.	*Event*	M_w	*PGA (g)*	*Components*	*Fault Type*	*Recording Station*	
						Name	*Epicentral Distance (km)*
1	1976 Fruili	6.4	0.35	EW, NS	Thrust	Tolmezzo	28
2	1979 Montenegro	6.9	0.27	EW, NS	Thrust	Veliki	143
3	1985 Algarrobo	8.0	0.22	EW, NS	Thrust	Rapel	108
4	1988 Armenia	6.7	0.18	EW, NS	Thrust	Gukasian	36
5	1992 Big Bear	6.4	0.16	90, 180	Strike-slip	Snow creek	37
6	1999 Chamoli	6.6	0.36	NE, NW	Thrust	Gopeshwar	17
7	1999 Chi-Chi	7.6	0.04	EW, NS	Strike-slip	Chiyai	45
8	1999 Hector Mine	7.1	0.08	90,180	Strike-slip	Heart Bar	69
9	2002 Denali	7.9	0.08	EW, NS	Strike-slip	Fairbanks	139
10	2011 Sikkim	6.8	0.16	EW, NS	Thrust	Gangtok	71

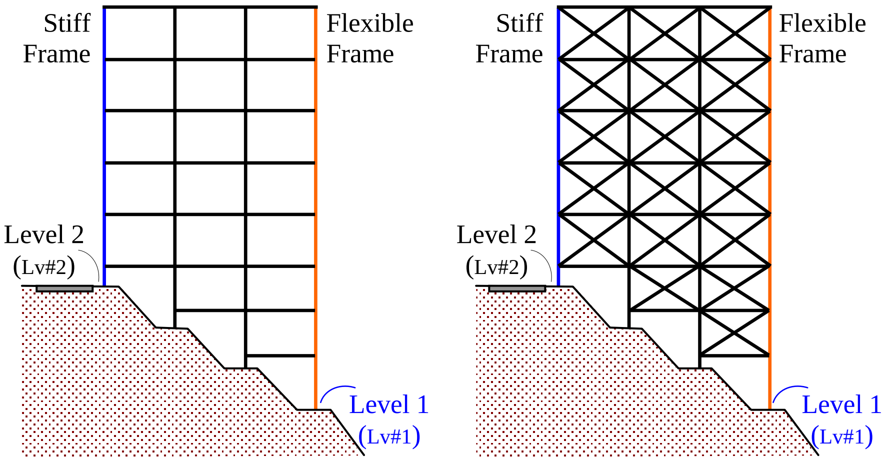

Fig. 2 – Elevation of building 303F and 303IN (with infills modelled)

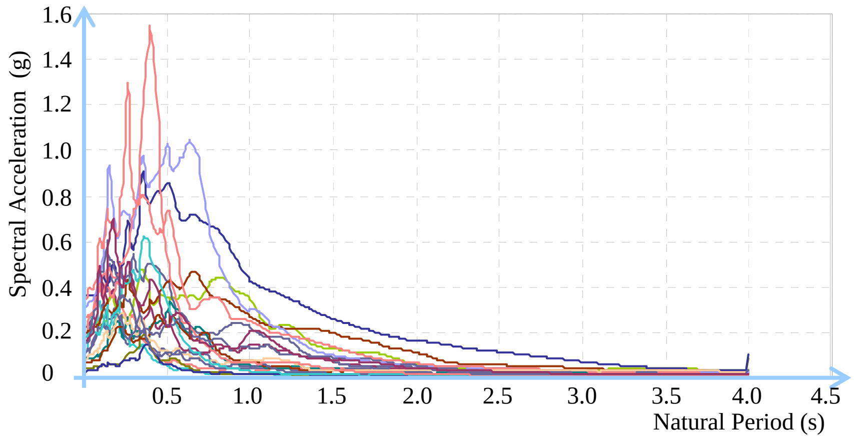

Fig. 3 – Elastic Acceleration Response Spectra (un-scaled) of ground motions considered in this study

3. Results and Discussion

3.1 Dynamic Characteristics of Buildings

The fundamental lateral translational mode of buildings founded on hill slope is torsionally coupled along Y-direction (*i.e.*, the direction across the hill slope); the natural periods of the buildings are listed in Table 2. In such buildings, the flexible frame moves more than the stiff frame (Fig. 4a). Consequently, mode shape ordinates of the flexible frame ($\phi_{i,1F}$) are more than the corresponding ordinates of the stiff frame ($\phi_{i,1S}$) in each storey (Fig. 4b).

Table 2 – The fundamental lateral translational periods of the building along Y-direction

Building	T_1 *(s)*	*Building*	T_1 *(s)*	*Building*	T_1 *(s)*
303F	1.18	403F	1.25	503F	1.24
303IN	0.66	403IN	0.68	503IN	0.69
304F	1.30	404F	1.29	504F	1.28
304IN	0.72	404IN	0.73	504IN	0.74
BMF	1.10	BMIN	0.64		

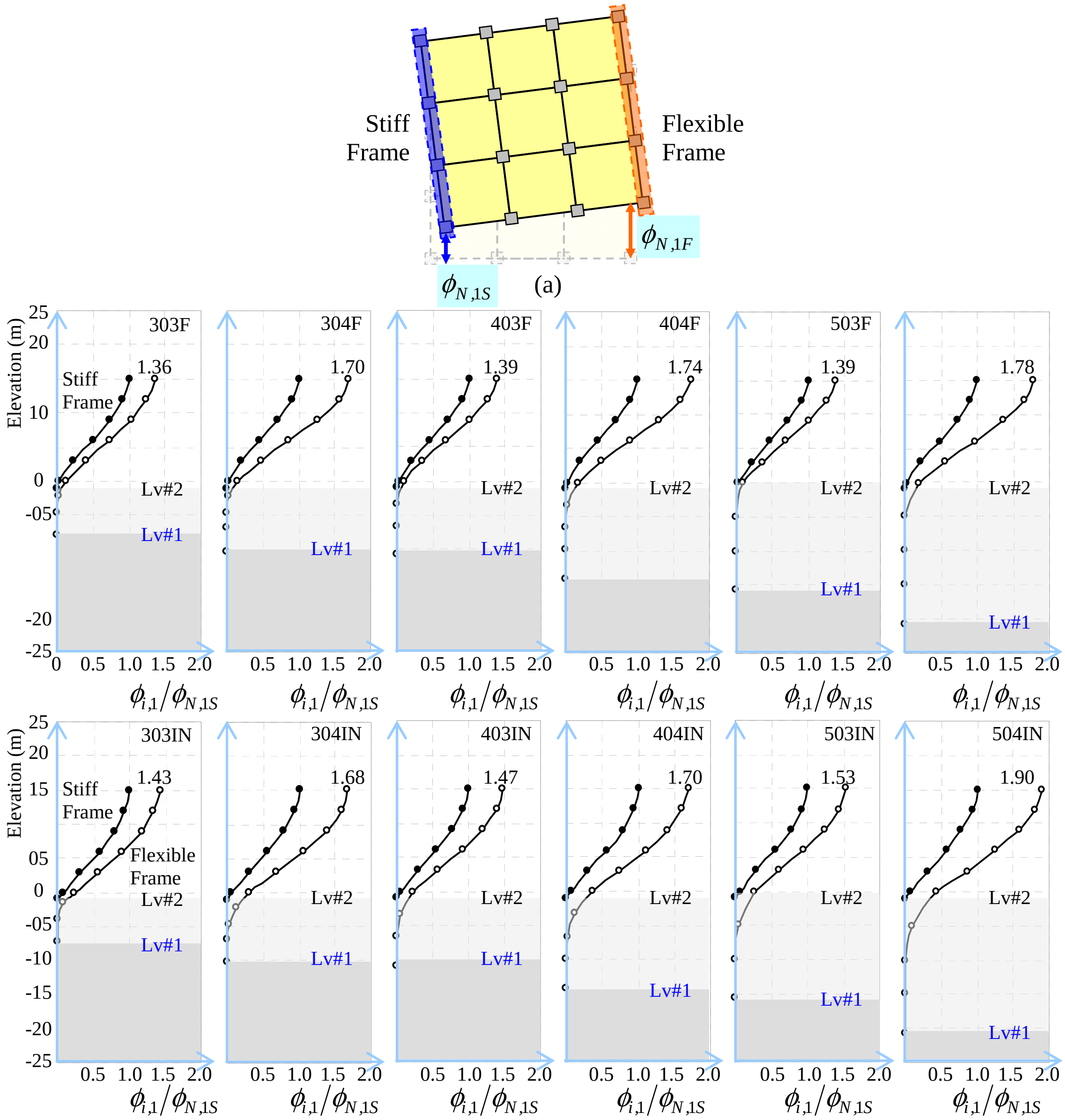

Fig. 4– (a) Plan of the top storey (storey N) of buildings founded on hill slope and (b) Normalised lateral translational mode shape of the buildings (represented using stiff and flexible frames alone) along Y-direction
(Note: values specified in the graph denotes the ratio of ordinates of the mode shape (at top storey) of flexible to stiff frame)

The ratio of mode shape ordinates $\phi_{i,1F}$ and $\phi_{i,1S}$ at the top storey signifies the extent of the torsional irregularity of the building. Thus, a quantitative measure of torsional irregularity is presented. The following are inferred from the lateral translational mode shape of study buildings presented in Fig. 4b:

(a) between the pair of bare frame buildings that are founded on same steepness of hill slope (say 303F and 304F), the torsional irregularity is more in buildings with 4-bays along the hill slope (*i.e.*, 304F) compared to the same in buildings with 3-bays along the hill slope (*i.e.*, 303F); similar results are observed in buildings with URM infill as well;
(b) the torsional irregularity is nearly the same among bare frame buildings that are founded on increasingly steep ground (*i.e.*, 303F, 403F and 503F); and
(c) unlike bare frame buildings, marginal increase in the torsional irregularity is observed for URM infill frame buildings that are founded on increasingly steep ground.

3.2 Peak Floor Acceleration

Normalized *PFA*s of each building obtained from *LTHA* and *NTHA* are shown in Fig. 5a and 5b, respectively; in addition to the average response, 10 and 90 percentile responses are shown too. The ratio of the *PFA* (estimated at the top storey) of stiff and flexible frames is shown as inset in Fig 5a and 5b. Additionally, *PFA*s of benchmark buildings (BMF and BMIN) are shown in Fig. 5c. Following are the inferences drawn from results presented in Fig. 5a, 5b and 5c:
(a) as expected, *PFA*s of *flexible frames* are more than the *PFA*s of *stiff frames* by an average value of 47% (when building remains elastic while resisting earthquake shaking); the said value reduces to about 36% when buildings resist earthquake shaking through inelastic action;
(b) between the pair of bare frame buildings that are founded on same steepness of hill slope (say 303F and 304F), the *PFA* is more in buildings with 4-bays along the hill slope (*i.e.*, 304F) compared to the same in buildings with 3-bays along the hill slope (*i.e.*, 303F); similar results are observed in buildings with *URM* infill as well;
(c) distributions of *PFA* (along the building height) are different for buildings considered with and without the lateral translational stiffness offered by *URM* infills; in general, *PFA* of intermediate storeys is more in buildings that considers the lateral translational stiffness offered by *URM* infills,

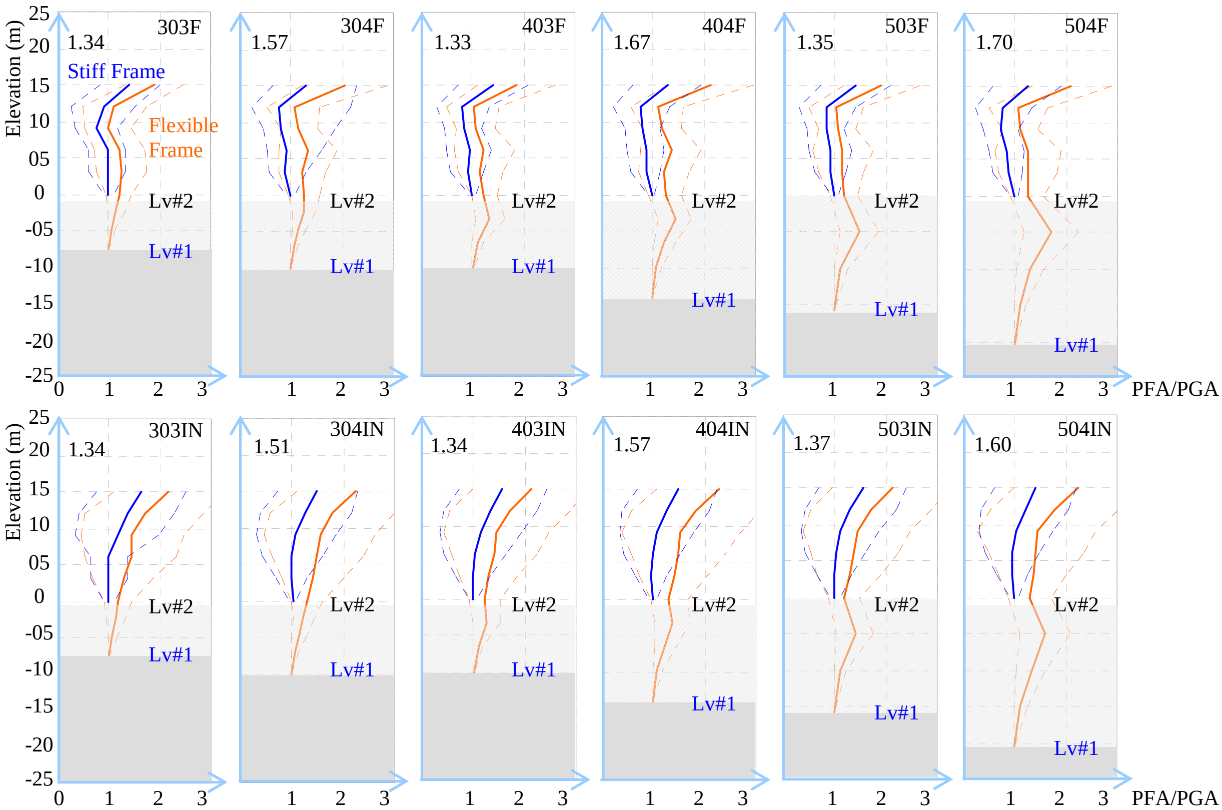

Fig. 5(a) – Distribution of *PFA/PGA* along the stiff and flexible frames of study buildings obtained from *LTHA*
(Note: values specified inside the graph (top-left) denote the ratio of *PFA* (top storey) of flexible frame to stiff frame)

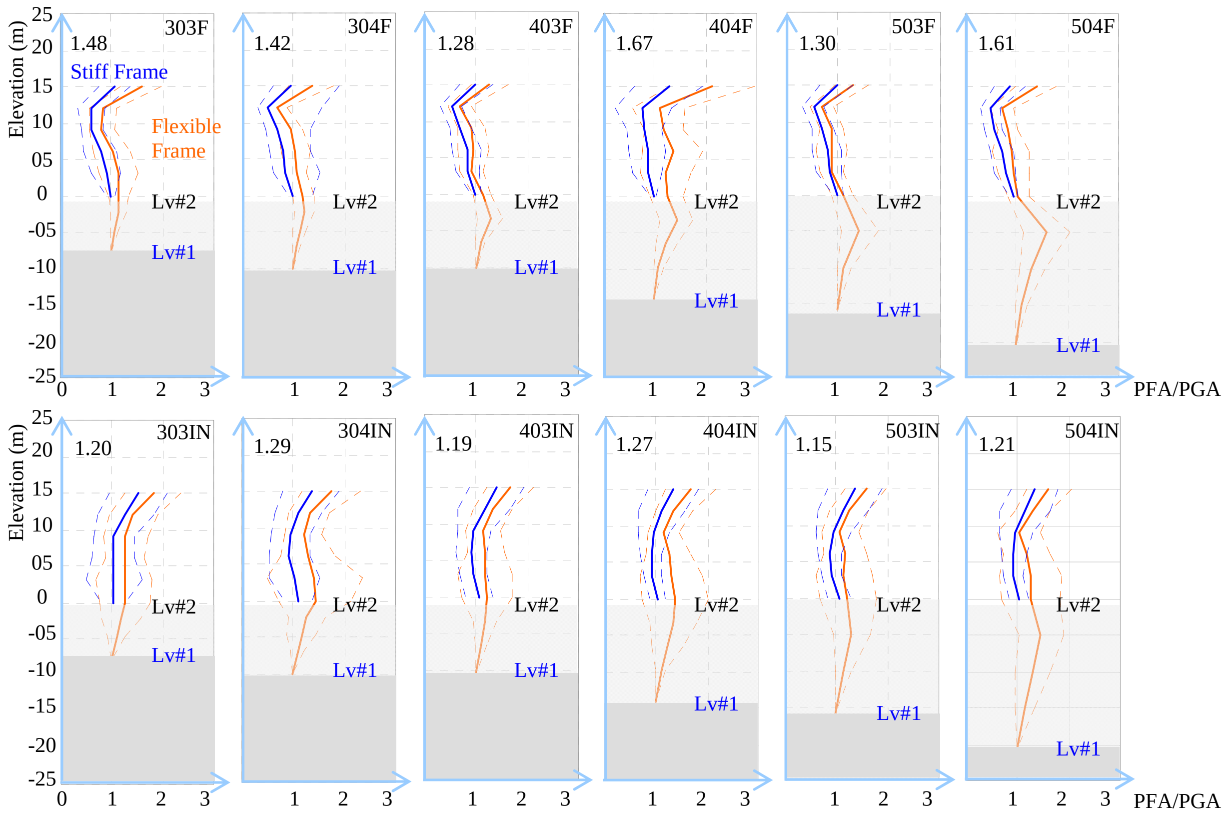

Fig. 5(b) – Distribution of *PFA/PGA* along the stiff and flexible frames of study buildings obtained from *NTHA*
(Note: values specified inside the graph (top-left) denote the ratio of *PFA* (top storey) of flexible frame to stiff frame)

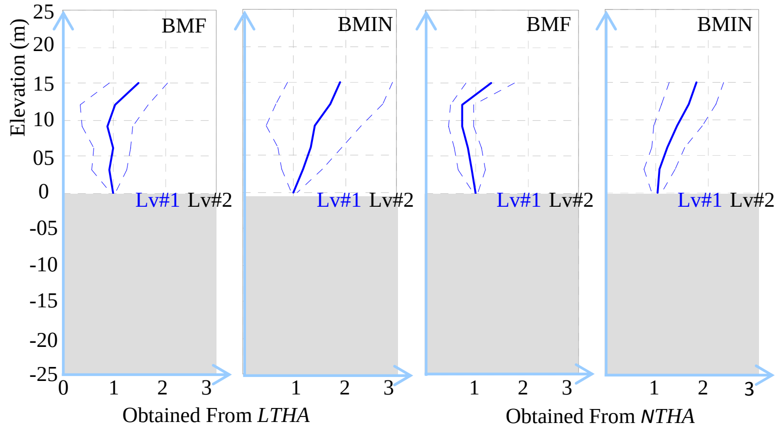

Fig. 5(c) – Distribution of *PFA/PGA* along the benchmark buildings obtained from *LTHA* and *NTHA*

(d) whiplash effect (*i.e.*, significant increase in *PFA* at the top storey) is observed in buildings with bare frame (*e.g.*, 303F, 404F); but, the same is not observed in buildings with URM infills modelled;

(e) in general, *PFA*s in buildings that sustain inelastic action is less compared to the buildings that remain elastic while resisting earthquake shaking; and

(f) for buildings that remain elastic while resisting earthquake shaking, the amplification factor of $1+(z/h)$ (as in Eq.1) reasonably estimates the *average PFA* of the top storey while the amplification factor $1+2(z/h)$ reasonably estimates the 90% percentile *PFA* of the top storey; in contrast, for buildings that sustain inelastic action while resisting earthquake shaking, amplification factors of $1+0.5(z/h)$ and $1+(z/h)$ reasonably estimate average and 90% *PFAs* of the top storey, respectively. Thus, the factor α (in Eq. 1) is seen to vary between 0.5 and 2.0, in contrast to 1.0 to 3.0 recommended in design codes.

3.3 Floor Acceleration Spectrum

In a given storey, *FAS* developed using the estimated acceleration response at a node that is the part of the stiff frame is different from that developed using the estimated acceleration response at a node that is part of the flexible frame (Fig 6). Consequently, acceleration sensitive *NSEs* that are placed near the flexible frame sustains more acceleration demand compared to the same that are placed near the stiff frame. Also, it is observed that acceleration amplification factor (a_p in Eq. 1) is more than 2.5, which is the recommended value of a_p in most seismic design codes.

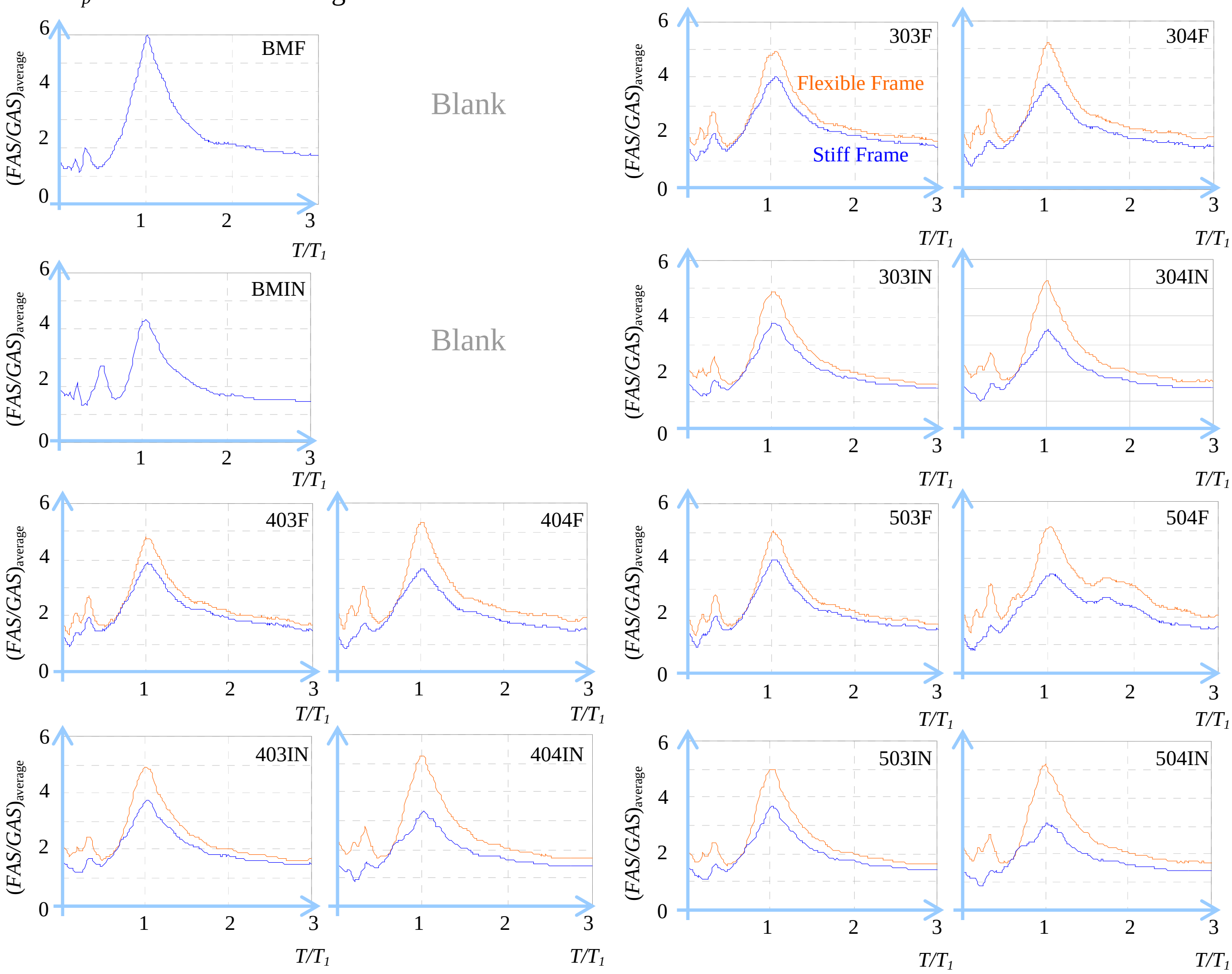

Fig. 6 –Average *FAS* (normalized with response spectrum of scaled ground motion) of top storey in buildings that remain elastic when resisting earthquake shaking

4. Summary and Conclusions

Most buildings along hill slope have torsional irregularity. Therefore, there is a necessity to estimate the *Peak Floor Acceleration* and *Floor Acceleration Spectrum* of such buildings to aid the design of acceleration sensitive nonstructural elements. For the purpose, the study assesses the applicability of the existing provisions, in the seismic design codes, to estimate *PFA* of buildings on the hill slope. The assessment is based on *linear* and *nonlinear time history analysis* of typical reinforced concrete buildings. The results indicate that the amplification of *PFA* along the building height is best represented using the function ($1+\alpha z/h$), where α is in the range 0.5 to 2.0. Also, results show that the amplification of *PFA* in the flexible frame is more than that in a stiff frame. And, the presence of *URM* infills alters the magnitude and distribution of *PFA* along the building height (by about 10%).

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