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Effect of Building Characteristics on Vibration-Induced Acceleration of Non-Structural Components (pp.69-87)

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Abstract

The effective accelerations for non-structural components located at different heights in a building are amplified due to the vibrations of the building and the components themselves. The amplifications along the height of buildings having different heights, structural systems and stiffness were studied. Four, eight and sixteen storied, frame and frame-shear wall buildings with similar plan and storey heights, designed as per relevant Indian codes of practice, were considered in the study. The effect of the building characteristics on the amplification of motion was studied and the observed amplification was compared with the various design practices. It was observed that the design practices overestimate the floor amplification and underestimate the component amplification. However, for eight storey bare frame building, the overall amplification having the combined effect of floor amplification and component amplification was observed to be close to that specified by FEMA.

Introduction

Nonstructural components are important operational and functional parts of a building before and after an earthquake. In most buildings the cost of nonstructural components is greater than the cost of the main structure. For example, it is known that in new hospitals and healthcare facilities about 80 percent of the cost is for nonstructural elements and equipment. The response of nonstructural components can affect the functionality of the main structure after an earthquake. They may act as the weakest link in a building, and in the event of a major earthquake, create a significant life safety hazard. In past earthquakes, poor performance of nonstructural components has led to the loss of life, economic losses and evacuation of buildings.

Acceleration sensitive nonstructural components are those components which are sensitive to and subject to damage from inertial loading. Examples are bookcases, hazardous materials, computer and communication racks, parapets and appendages in the out of plane direction, boilers, light fixtures, tanks and vessels, elevators, electrical equipment, chimneys and stacks, mechanical equipment, etc.

For the analysis and design of acceleration sensitive nonstructural building components, design accelerations are specified by different codes. In the Uniform Building Code of ICBO (1994), provisions are based on allowable stress design while the provisions of ICBO (1997) are based on strength design. The strength based seismic design force is also given in FEMA356 (2000), in the recommendations by the Building Seismic Safety Council (2000) and in the International Building Code by ICC (2000). A horizontal force formula to be applied on nonstructural components is also given by Eurocode 8. The lateral inertia force is also defined in the National Building Code of Canada (NBCC, 1995). The amplification of motion along the height in NBCC (1995) and Eurocode 8 is defined by (1+x/h). FEMA 356, the NEHRP guidelines by the Building Seismic Safety Council (2000) and IBC (2000) define this amplification as (1+2x/h) whereas the Uniform Building Code of ICBO (1997) defines it as (1+3x/h), where x is the floor height and h is the building height. The maximum component amplification given by FEMA 356, the NEHRP guidelines by the Building Seismic Safety Council (2000), IBC (2000) and the Uniform Building Code of ICBO (1997) is 2.5.

Modeling, Analysis and Design of Buildings

The buildings' plan has 8 bays in the longitudinal direction and 3 bays in its transverse direction. The ground storey height is 4.73m including the depth of foundation below ground level and the upper storey heights are 3.35m. The buildings are symmetrical in plan and regular along height. The plan and elevations of the bare frame buildings are shown in the Figs. 1& 2. The plans of frame-shear wall buildings with two cases of shear wall arrangements in longitudinal and transverse direction are shown in Fig. 3. These two cases were considered for each of the three building heights.

The three dimensional frame buildings were modeled using beam and column elements. The floor slab was modeled as rigid diaphragms. The supports were modeled as fixed. In case of frame shear wall buildings, shear walls were modeled as wide columns along the centerline of the actual shear wall. To simulate the finite width of shear walls, strong rigid beams are provided across the width of the shear walls at each floor level.

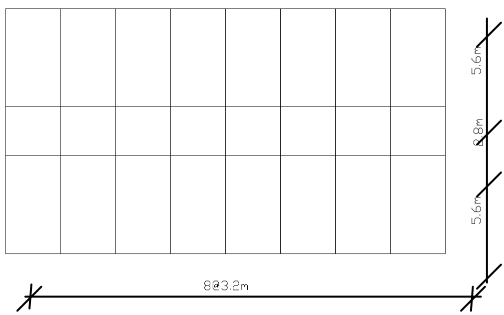
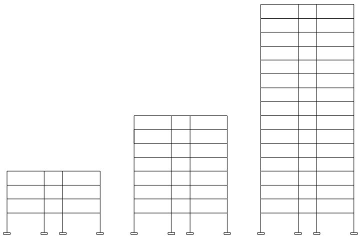
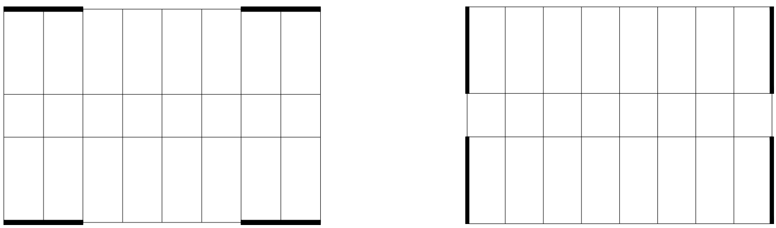


Fig. 1 Floor plan of the bare frame buildings



(a) (b) (c)

Fig. 2 Elevations of the bare frame buildings; (a) 4-storey (b) 8-storey (c) 16 storeys



Case-1 Case-2

Fig. 3 Floor plan of frame shear wall buildings

Modeling, linear dynamic analysis, and design were done using structural analysis software, SAP 2000. The buildings were designed as hospital buildings located in seismic zone V, on medium soil. The necessary loads were applied as per the relevant IS codes. For infill load calculations 230mm external and 150mm internal unreinforced masonry partition walls were considered. The materials used in the study are Fe-415 Steel and M-25 Concrete. The live loads considered are 2.5KN/m^2 in the wards and 4KN/m^2 in the corridor.

Modal analysis was performed to study the various modes of free vibration of the buildings. These modes are useful to understand the behavior of the buildings. The linear dynamic analysis was carried out using mode superposition based on response spectrum analysis. For the purpose of dynamic analysis, the value of damping was taken as 5 percent of the critical. The complete quadratic combination (CQC) method was used to combine the peak response quantities. The worst load combination was used for the subsequent design. The design base shears (Vb) obtained from the analysis results were compared with the base shears ($\bar{V}b$) calculated using the empirical fundamental periods given in IS 1893. The calculated base shears were scaled up to the empirical value by multiplying the base shear with $\bar{V}b/Vb$.

Direct integration time history analysis was carried out for frame and frame-shear wall buildings using generated response spectrum compatible time histories (Ashok, 2004) as shown in Fig. 4. Direct integration of the full equation of motion without the use of modal superposition is available in the software used (Wilson, Undated). The Hilber-Hughes-Taylor time integration method was used for the time history analysis. Three time histories (Fig. 4) were selected and applied in the longitudinal and transverse directions and the average results were taken for the study.

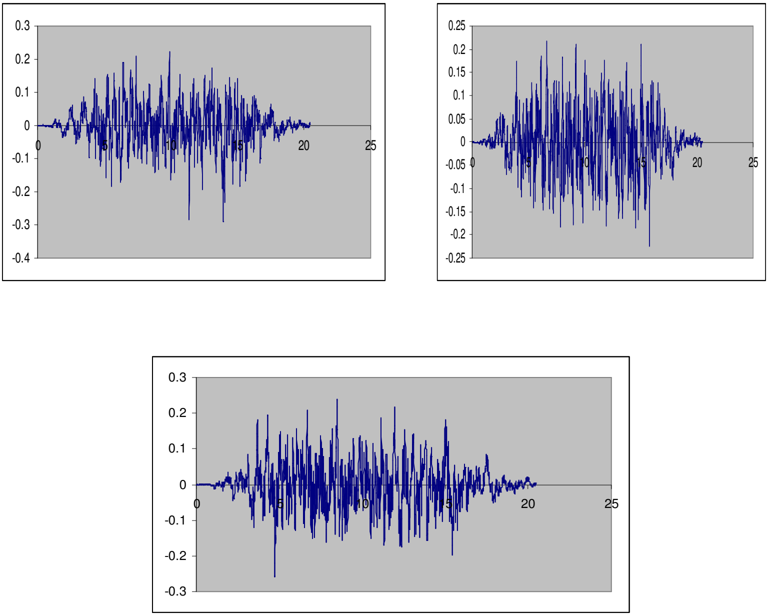


Fig. 4 Generated response spectrum compatible time histories

Using the input time histories, the absolute acceleration time histories and the acceleration floor response spectra at the geometric center of different floor levels were studied. The corresponding Peak Floor Accelerations (PFA) and Peak Component Accelerations (PCA) were taken and compared with the Peak Ground Accelerations (PGA) and short period Peak ground Spectral Acceleration (PSA).

Floor Responses

Floor Time Histories

Sample floor time histories at 1st, 4th and 8th storey levels of the 8 storey building are shown in Fig. 5. From the figure the amplification of the motion along the height and change in the frequency content is clearly visible.

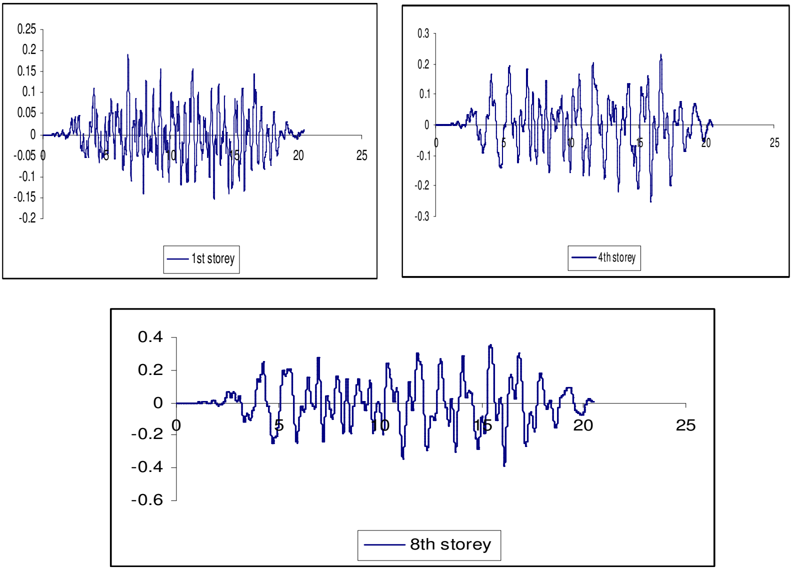


Fig.5 Sample floor acceleration time histories (for 8 storey frame building)

Frequency Content

Sample Fourier Amplitude Spectra of a ground time history and the corresponding floor time histories of the 8-storey frame building in longitudinal direction were compared and shown in the Fig. 6. The frequency content of the floor motion is significantly different than that of the ground motion. The floor motion spectras show peaks corresponding to the 1st and 2nd modes of vibration of building. The building has a tendency to focus the energy corresponding to these frequencies. As we go along the height, the focusing is more and more pronounced.

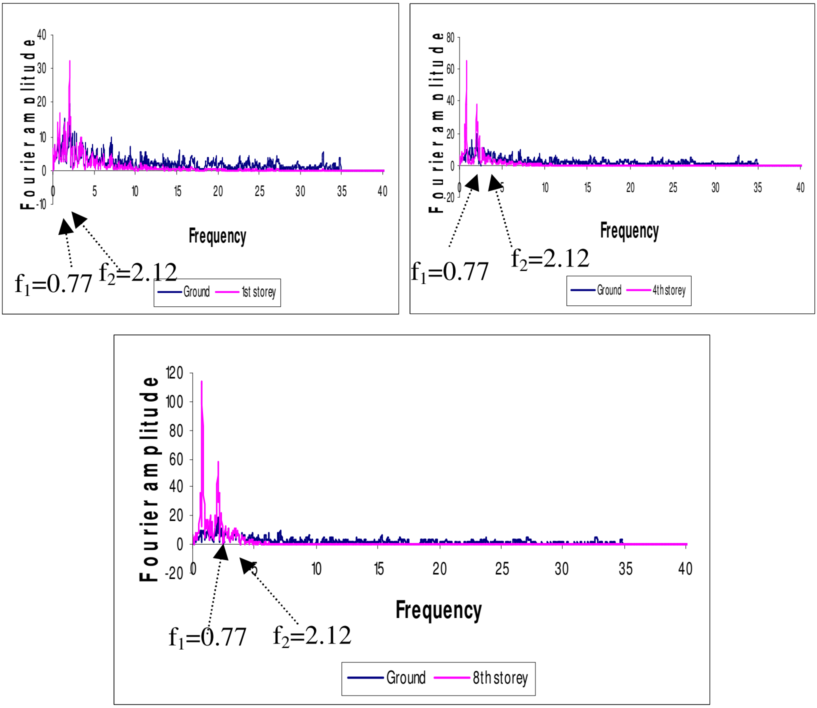
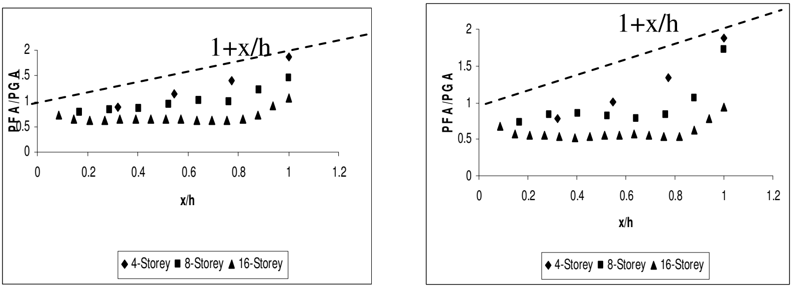


Fig. 6 Fourier amplitude spectra for ground time history and the corresponding floor time histories

Peak Floor Response

The peak value of a ground acceleration time history is defined as Peak Ground Acceleration (PGA) and the corresponding peak value of floor acceleration time history is called Peak Floor Acceleration (PFA). The ratio PFA/PGA is the amplification of the ground motion along the height. Comparisons between PFA and PGA for the three bare frame buildings in the transverse and longitudinal directions at each floor level are shown in Fig. 7. In the figures x/h is the ratio of floor height, x to the building height, h.

From Fig. 7 we can see generally, the ratio of PFA/PGA increases along the height of the buildings. The results show that compared to the 8 and 16 storey buildings, the 4 storey building exhibit large amplification of acceleration along the height. All the values of three buildings fall below the line (1+x/h).

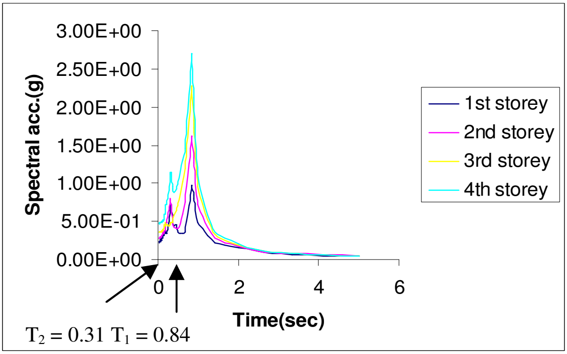


(a) (b)

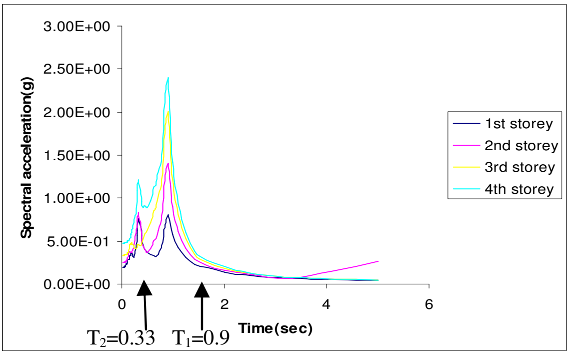
Fig. 7 PFA/PGA along the height of the three frame buildings: (a) in longitudinal direction, and (b) in transverse direction

Floor Response Spectra of Frame Buildings

Average floor response spectra for 5% damping at each floor level are shown from Figs. 8-10 for the three bare frame buildings. These floor response spectra were used to evaluate the maximum acceleration response of nonstructural components and provide significant insight into their dynamic behavior. The peaks in the floor acceleration response spectra are corresponding to the initial modes of vibration of the supporting structure in the direction considered. In case of four storey building there is a single peak in the response spectrum corresponding to the fundamental period of vibration. The contribution of the second mode in floor response is small. In the eight storey building there are two peaks corresponding to the first and second time periods of the building. Unlike the four storey building, the contribution of the second mode is significant. For 16 storey building the peaks were observed corresponding to the first three modes of vibration and the highest spectral acceleration corresponds to the second mode. The results indicate that the acceleration response of a nonstructural component is strongly dependent on how its period compares to the modal periods of the supporting structure. We can see that if the natural periods of the nonstructural component are close to those of the structure, the component can experience Peak Component Acceleration (PCA), which is the peak acceleration value of the floor response spectra.

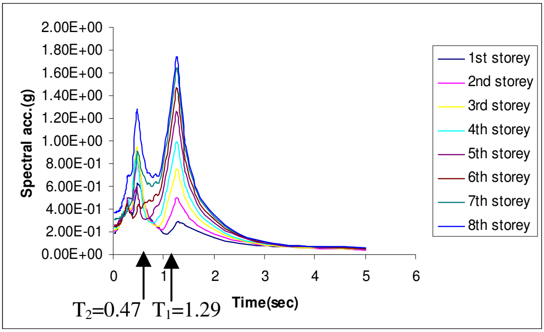


(a)

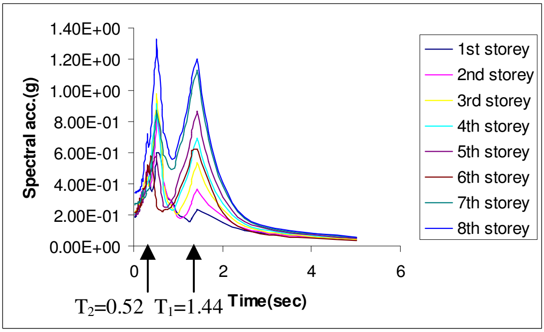


(b)

Fig. 8 Average floor response spectra (4-storey building)
(a) in the longitudinal direction (b)in transverse direction

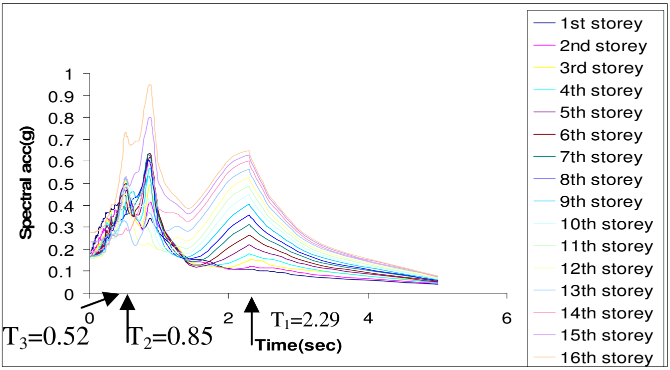


(a)

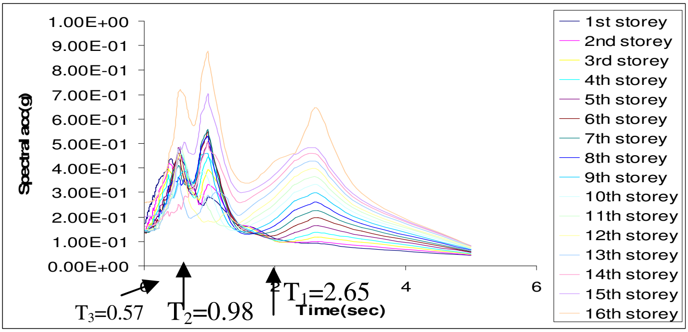


(b)

Fig. 9 Average floor response spectra (8-storey building)
(a) in the longitudinal direction (b)in transverse direction



(a)

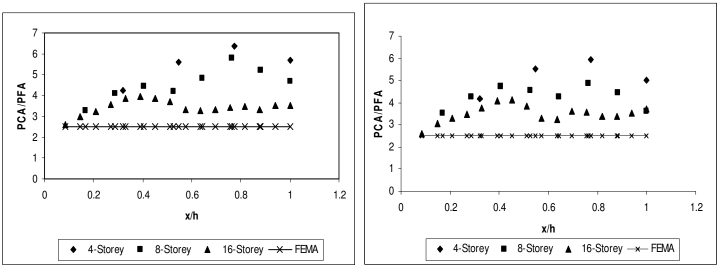


(b)

Fig. 10 Average floor response spectra (16-storey building)
(a) in the longitudinal direction (b)in transverse direction

Comparisons between PCA and PFA and between PCA and PSA are given in Figs. 11 and 12, respectively for the bare frame buildings, where PSA is the short period peak ground spectral acceleration which is 2.5 times the effective peak ground acceleration, in accordance with the provisions of IS 1893.

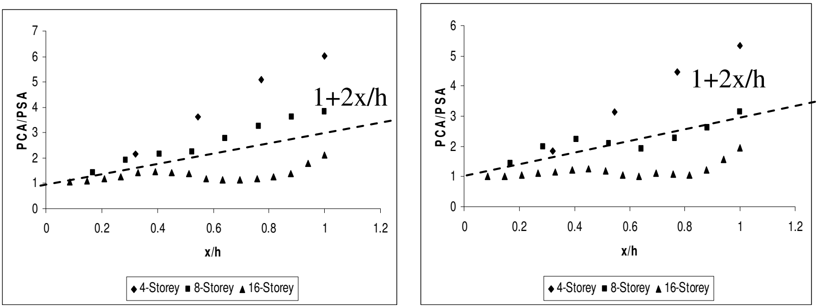
The plots of the Component Amplification Factor, PCA/PFA in Fig. 11 show that the amplifications at different floor levels in the two directions decrease as the building height increases. Compared to the eight and sixteen storey buildings the four storey building exhibits higher component amplifications. The amplifications observed in the three buildings are greater than the maximum component amplification, which is 2.5, specified by FEMA 356. From the results we can observe that, in all cases, the maximum component amplification factor of current seismic design provisions severely underestimates the maximum acceleration response of NSCs.



(a) (b)

Fig..11 PCA/PFA along the height of the three frame buildings (a) in longitudinal and (b) in transverse direction

PCA/PSA represents the overall effect of amplification due to building and the nonstructural components. The results in the Fig.12 show how much the peak component accelerations at each floor level are greater than the peak spectral ground acceleration. The ratio, PCA/PSA generally increases along the height of each building except for some stories in the 16 storey building. PCA/PSA values at different floor levels decrease as the building height increases. The four storey building exhibits larger PCA/PSA values than the eight and sixteen storey buildings. For the 16 storey building the variation along the height is not much significant. It is almost constant. The line (1+2x/h) is the value of PCA/PSA in FEMA 356 corresponding to maximum component amplification factor of 2.5. In case of four storey building PCA/PSA values are above the line (1+2x/h). In case of eight storey building most of the values are close to the line whereas the PCA/PSA values in the sixteen storey building are below the line.



(a) (b)

Fig. 12 PCA/PSA along the height of the three frame buildings (a) in longitudinal and (b) in transverse direction

Effect of Shear Walls

The effect of building stiffness on the floor responses was studied by considering shear walls along the two directions of the buildings as shown earlier in Fig. 3.

PFA/PGA, PCA/PFA and PCA/PSA for the four storey, eight storey and sixteen storey frame-shear wall buildings were compared with those for bare frame buildings in Fig.13-21. Due to the addition of shear walls, it was observed that at all storey levels of the three building heights, the values of PFA/PGA increase, but PCA/PFA values decrease at the lower storey levels and increase in most of the top storey levels. It was also observed that the values of PCA/PSA increase at all storey levels of the three building heights due to the addition of shear walls in longitudinal and transverse directions. These results generally show that increasing the stiffness of buildings by addition of shear walls will exert more acceleration on nonstructural building components. Due to the addition of shear walls PFA/PGA values at the top storey levels in the four and eight storey buildings are observed above the line (1+x/h), but the values in the sixteen storey building are all below the line. Fig.16-18 show that all results of PCA/PFA in both bare frame and frame-shear wall cases were above the maximum value given by FEMA 356, which is 2.5. In case of four storey and eight storey buildings, all values of PCA/PSA at each storey level in bare frame and frame-shear wall buildings are above the line (1+2x/h) whereas PCA/PSA values of the sixteen storey building due to the addition of shear were close to the line.

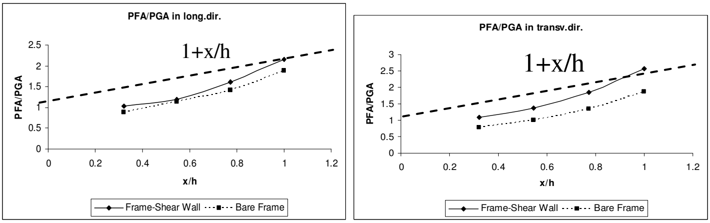


Fig. 13 Effect of shear wall on PFA/PGA of the 4-storey building

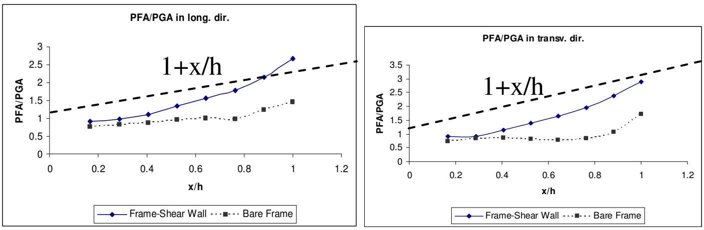


Fig. 14 Effect of shear wall on PFA/PGA of the 8-storey building

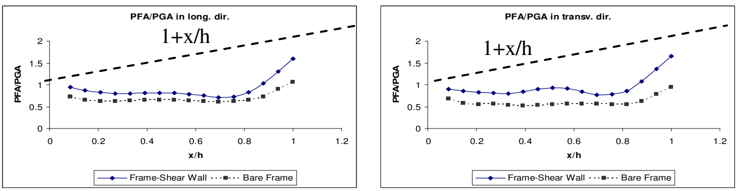


Fig. 15 Effect of shear wall on PFA/PGA of the 16-storey building

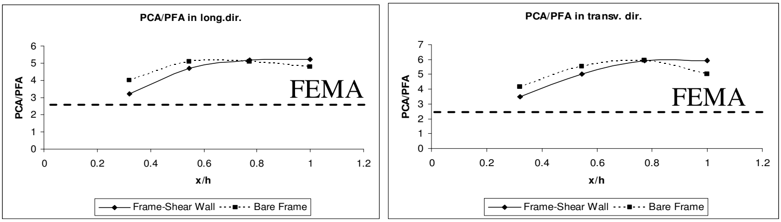


Fig. 16 Effect of shear wall on PCA/PFA of the 4-storey building

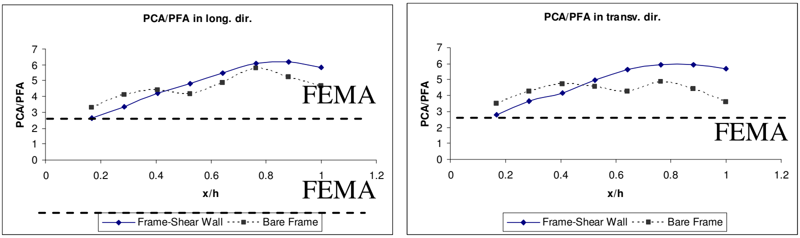


Fig. 17 Effect of shear wall on PCA/PFA of the 8-storey building

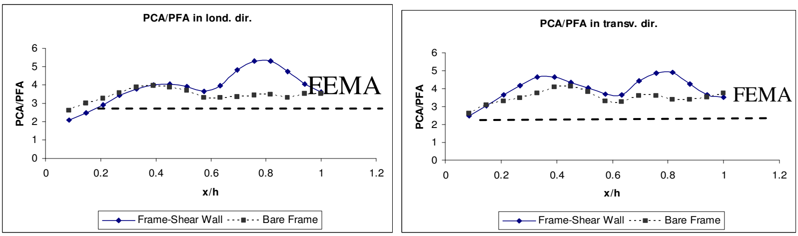


Fig. 18 Effect of shear wall on PCA/PFA of the 16-storey building

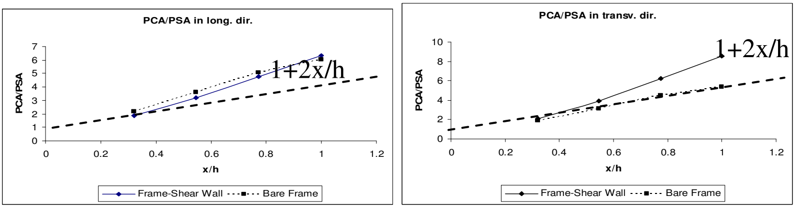


Fig. 19 Effect of shear wall on PCA/PSA of the 4-storey building

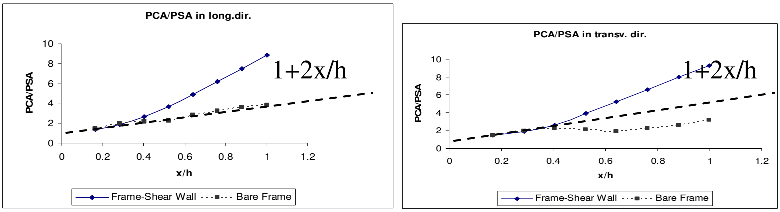


Fig. 20 Effect of shear wall on PCA/PSA of the 8-storey building

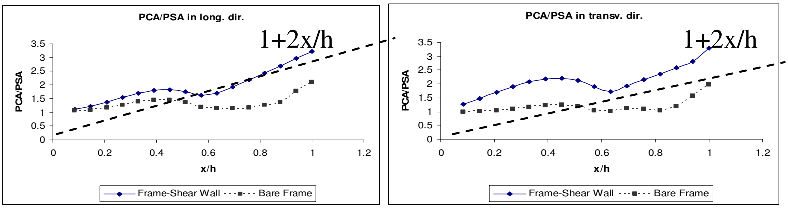


Fig. 21 Effect of shear wall on PCA/PSA of the 16-storey building

Conclusions

- The buildings have a tendency to focus the energy into the bands around the frequencies corresponding to the initial modes of vibration. As we go along the height, the focusing becomes more and more pronounced.
- The peaks in the floor acceleration response spectra are correspond to the initial time periods of the supporting building structures in the direction under consideration. In

case of the 4 storey building studied, the peak corresponds to the first mode but as the building height increases the contribution of second mode increases. For the 16 storey building the peaks were observed to correspond to the first three modes of vibration and the highest spectral acceleration corresponds to the second mode.

- PFA/PGA, PCA/PFA and PCA/PSA for the three frame buildings at different floor levels decrease as the building height increases. Short period buildings result in higher amplification of ground motion along height.
- The amplification of ground motion along the height defined by different codal provisions overestimate the values of PFA/PGA obtained at different floor levels for the frame buildings.
- The maximum Component Amplification Factor given by current seismic design provisions underestimates the values of PCA/PFA obtained at different floor levels for the bare frame as well as the frame-shear wall buildings.
- For the eight storey frame building the PCA/PSA representing the effect of amplification due to building as well as due to nonstructural component was observed to be close to the values specified by FEMA. However, it is above and below the value specified by FEMA for the four and sixteen storey frame buildings respectively.
- PFA/PGA, PCA/PFA, and PCA/PSA at different floor levels of the frame-shear wall buildings are higher than the corresponding bare frame buildings. The increment due to provision of shear walls is higher at the upper floors.

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