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BEHAVIOUR OF REINFORCED CONCRETE BUILDINGS UNDER SIMULTANEOUS HORIZONTAL AND VERTICAL GROUND MOTIONS

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ABSTRACT

This paper is concerned with the study of the effect of combined horizontal and vertical accelerations on the seismic response of reinforced concrete structures. To achieve this objective, three reinforced concrete buildings representative of rigid, semi-rigid and flexible structures were analyzed in the nonlinear range using lumped mass and distributed mass models. The results obtained indicate that the inclusion of the vertical component has little effect on the storey drifts and base shears but can greatly influence the axial forces in the columns and the vertical displacements of girders.

Keywords: Reinforced concrete structures; vertical and horizontal excitations; nonlinear analysis; distributed mass; lumped mass

1. IINTRODUCTION

For a long time, little or no consideration has been given to vertical accelerations in the aseismic design of structures due to the following reasons: -a) it was believed that the vertical component is smaller than the horizontal one and that structures are very rigid in the vertical direction compared to the horizontal direction, -b) aseismic analyses for buildings have been considered to be fairly satisfactory even if they use only the horizontal component. Actually, the trend is towards giving vertical accelerations more attention because : -c) observations of strong motions earthquake records and reports on destructive earthquakes show that the effect of vertical accelerations can no longer be ignored, -d) there are problems arising in the design of structures that cannot be solved only by considering the horizontal component alone, -e) investigations of previous earthquake records, showed that even if the peak horizontal accelerations may not occur at exactly the same time as the peak vertical accelerations, they do occur within the same general time. Thus, for some special important structures such as nuclear power plants, as a conservative assumption, the horizontal and vertical accelerations could be assumed to act simultaneously. Relatively,

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little has been published concerning the effect of vertical accelerations. Iyengar and Shinozuka [1] investigated the effect of self weight and vertical accelerations on the behaviour of tall structures. The structures have been idealized as cantilevers and the ground motion as a random process. Their main conclusions are; the consideration of self weight and vertical accelerations might increase or decrease the peak responses. However, the difference either way seems to be considerable in most cases. The effect of vertical accelerations could be more pronounced particularly in beam response, if a frame structure is considered. Iyengar and Sahia [2] investigated the effect of vertical ground motion on the response of cantilever structures using the mode superposition method; their main conclusion is that the consideration of the vertical component is essential in analyzing towers. The non inclusion of this makes the design err on the unsafe side at some sections. Anderson and Bertero [3] evaluated using numerical methods the inelastic response of a ten story unbraced steel frame subjected to a horizontal component of earthquake and to combinations of this component with the vertical one, they deduced the following points; the inclusion of the vertical motion on one hand does not increase the displacements but on the other hand increases the girder ductility requirement by 50% and induces plastic deformations in columns. Mostghel and Ahmadi [4, 5, and 6] studied the effect of vertical motion on columns and tall buildings which have been idealised as cantilevers, using the mathematical theory of stability of Liapunov. Their main conclusion is : in the elastic range, for an initially straight column, if the total maximum loading during an earthquake is less than the Euler buckling load, then the column will be always be stable irrespective of the time history of the earthquake to which it is subjected. But this is unlikely to be the case for reinforced concrete columns because of the crushing of concrete in compression and the buckling of the yielded reinforcement. Javed et al. [7] studied the inelastic seismic performance of a 12- story reinforced concrete building under a combination of horizontal and vertical ground motions. This analysis showed a slight increase in the maximum deformation when the vertical ground motion is included. The formation patterns revealed that vertical accelerations induced a slightly different hinge formation pattern and hinge rotation magnitudes, and the response of the frame-wall system did not show sensitivity to the vertical acceleration in this case. Antoniou [8] studied the effect of vertical accelerations on RC buildings by analyzing an 8 storey RC building designed to high ductility class in EC8 with a design acceleration of 0.3 g. This analysis showed that the vertical of ground motion has almost no effect on roof displacements and interstorey drift, but can increase the compressive forces by 100 % or even more and led to the development of tensile forces in columns. These fluctuations in axial forces can result in shear failure in columns. Ghobarah and Elnashai [9] analyzed a 3 story non ductile RC building and an 8 storey RC building designed according to EC8. The results obtained indicated that vertical excitation did not affect considerably roof displacements and interstorey drifts, while it led to accumulated damage by 10 % to 20 %.. Collier and Elnashai [10] proposed simplified procedures to combine vertical and horizontal ground motions, and analyzed a 4-storey RC frame of typical 1960s European construction. Their emphasis was placed on the effect on the vertical period of structural vibration considering various V/H ratios and time intervals between horizontal and vertical peak acceleration. They concluded that the vertical period of vibration can be affected significantly by the amplitude of both vertical and horizontal accelerations. It was also identified that the interaction effect of the horizontal motion as a function of the time interval is significant when the time interval between peaks is less than 2 sec. Shakib et al [11] conducted a study to evaluate the effects of the vertical component of earthquake on the response of pure-friction base isolated asymmetric building for a single story building resting on sliding supports. The results of this analysis showed that the vertical component of the earthquake highly influences the lateral response of the pure-friction base-isolated, and the torsional response of the structure in the moderate range of eccentricities increases considerably when the system is subjected to both horizontal and vertical ground motions. Mwafy and Elnashai [12] conducted extensive analyses on three different RC frame groups including 8-storey irregular frame, 12-storey regular frame, and 8-storey frame-wall building, where each ground has 4 different design levels, resulting in total 12 RC buildings. They concluded that the effect of vertical ground motion increases when the contribution of the lateral seismic action is relatively small, such as in low-rise buildings and interior columns of taller structures at higher stories. They indicated that global response parameters may increase by more than 20% at the design PGA and the interstory drift of collapse limit state was frequently reached at lower earthquake intensities when the effect of vertical ground motion was included. It was also observed that the axial compressive forces in columns increased by up to 45% and tensile forces were detected only when vertical ground motion was included. It was reported that vertical ground motion increased the curvature ductility demand by nearly 60%.

2. INELASTIC MODELS

Over the years many inelastic models have been proposed to model inelastic deformations which occur in reinforced concrete members. Due to the complexity of the problem several simplifications have to be made. One of the major simplifications is the assumption which consists of lumping inelastic deformations at the member ends, instead of considering the spread of inelastic deformations along the member length. The most widely nonlinear models used in most analyses are: the one component model and the dual component model.

The one component model which has been first generalized by Gibson [13], has been developed on the assumption that inelastic deformations concentrate at some critical locations. A major feature of the model is that inelastic member-end deformation is assumed to depend only on the moment at the end. On one hand this is an advantage because it means that the deformation at one end is independent of the deformation at the other end, but on the other hand this is unrealistic because the member end-rotation should be dependent on the curvature distribution along the member and consequently on the other end. The model consists of a flexible line with one rotational spring at its end and two rigid zones outside the rotational spring.

The dual component which has been first introduced by Clough [14] assumes that every member consists of two components, an elasto-plastic component which simulates the yielding phenomenon and a completely elastic one which represents the strain hardening acting in parallel. The sum of the two results in a bilinear moment-curvature relationship for the member. The stiffness of the second component **p**EI is a specified fraction of the total stiffness and corresponds to the second slope of the bilinear moment-curvature relationship. In practice **p** is taken as equal to 0.05. It is worth noting that in this model, a member end rotation depends on both member end moments. At yield, the elasto-plastic component is assumed to have yielded completely, while the second component can continue to take increase in moment while remaining elastic. The model assumes that reduction in stiffness applies along the entire length of the beam when yielding occurs at both ends. The limitations of the concentrated plasticity models are discussed in Charney and Bertero [15], and Bertero et al. [16]. The distributed inelasticity describes more accurately the continuous structural characteristics of reinforced concrete members, requiring simply geometrical and material characteristics as input data. The constitutive behaviour of the cross-section can be either formulated according to the classical plasticity theory in terms of stress and strain resultants, or explicitly derived by discretising the cross section into fibres. The latter approach known as fibre modelling represents the spread of material inelasticity both along the member length and across the section area, which allows an accurate estimation of the structural damage distribution even in the highly inelastic range. Further details concerning this approach can be found in Kaba and Mahin [17], Zeris and Mahin [18].

3. STRUCTURAL MASS MODELS

For an earthquake in the horizontal direction, the modes excited are associated with very small vertical displacements compared to the horizontal ones, thus we do not introduce big errors by concentrating the masses in the principal nodes as shown in Figure 1(a). In the vertical direction, the horizontal displacements are negligible compared to the vertical ones, so we can concentrate at the principal nodes the masses of the vertical elements. However, this rule cannot be applied to the masses of the floors because the flexural stiffness of the horizontal elements is in general less than the compressive stiffness of the vertical elements, thus the vertical displacement of a mass depends upon its position along the horizontal elements. Therefore, the masses of the slabs must be distributed between the principal nodes and one or several nodes within the slabs as shown in Figure 1(b).

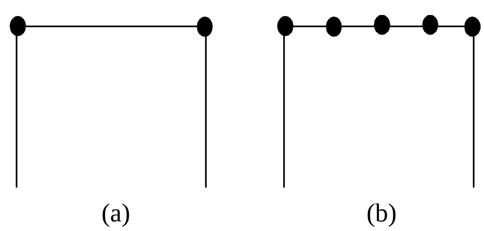


Figure 1. Mass models (a) Lumped (b) Distributed

4. DESCRIPTION OF THE STRUCTURES

Three structures with two, five and eight stories representing low, medium and high rise reinforced concrete frame structures with lumped and distributed masses as shown in Figure 2 are considered in this study. These structures are designed according to the Algerian code RPA 2003 [19] and are located in high seismicity region with a peak ground acceleration of

0.32g. The three buildings are 12mx12m in plan. Typical floor to floor height is 3.06 m. The dimensions of the beams and the columns for the three structures are shown in Table 1 and the reinforcement details are shown in Table 2.

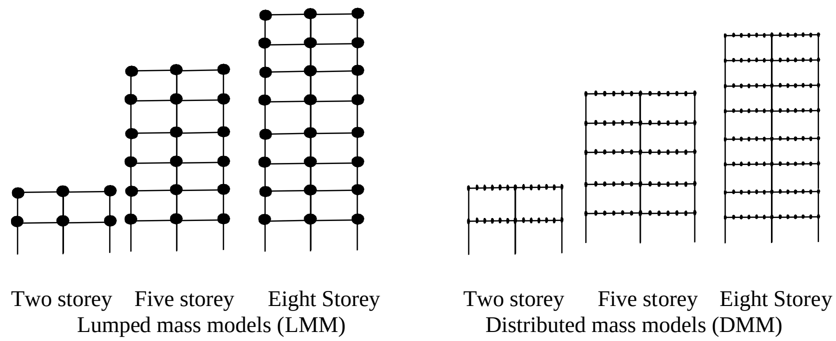


Figure 2. Structural models

Table 1: Dimensions of beams and columns

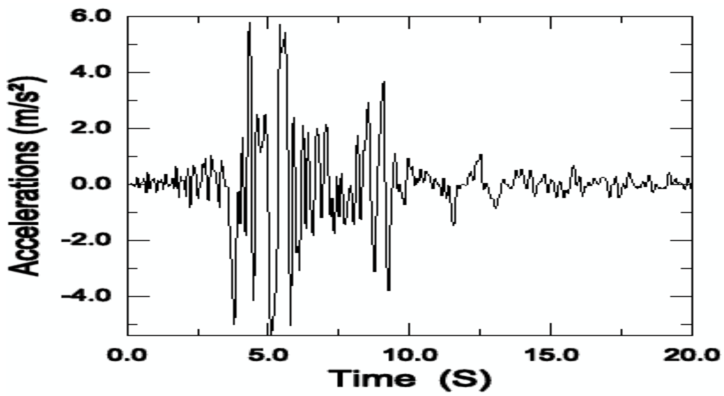
Beam and column dimensions in cm			
Structure	Beams	Level	Columns
Two storey	30X60	1-2	40X40
Five storey	30X60	1-3 4-5	50X50 40X40
Eight storey	30X60	1-5 6-8	70X70 50X50

Table 2: Reinforcement details

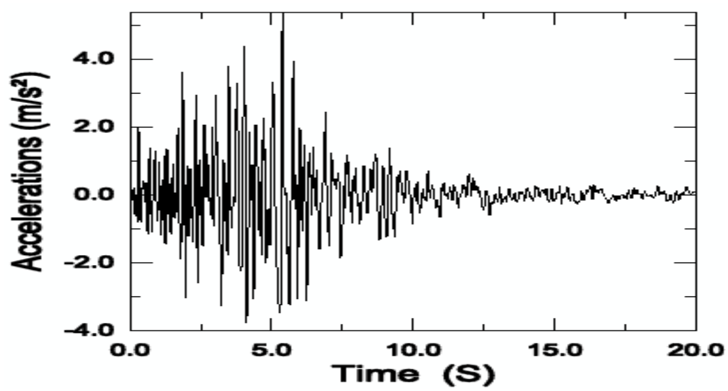
Steel reinforcement in beams and columns				
Structure	Beams			Columns
	Level	Top	Bot	
Two storey	1-2	6T16	3T20	8T16
Five storey	1-3 4-5	6T20 4T25	3T20	8T32+4T25 12T25
Eight storey	1-5 6-8	6T20 4T25	3T20	12T32 12T25

5. EARTHQUAKE GROUND MOTIONS

The accelerograms used in this investigation are the horizontal and vertical components of the New Hall earthquake record, Figure 3. These records are believed to be representative of strong earthquake. Studies by Clough and Benuska [20] indicate that structural response depends primarily on the peak acceleration impulse in the ground motion and that continuing motions of smaller amplitude have only a small effect on the maximum response. Therefore the duration of the earthquake used in this analysis was primarily limited to the first fifteen seconds of the earthquake. The peak ground acceleration of the horizontal component is 0.578 g and that of the vertical one is 0.537g.



(a) New Hall accelerogram (north-south component)



(b) New Hall accelerogram (vertical component)

Figure 3. Earthquake records

6. PRELIMINARY ANALYSES

Prior to dynamic analysis, frequency and static analyses were conducted and the results obtained for each structure using the lumped mass model and the distributed mass model were identical. The first three periods of each structure are shown in Table 3.

Table 3: Periods of the structures

Structure	**Mode 1**	**Mode 2**	**Mode 3**
Two storey	0.30	0.11	0.049
Five storey	0.79	0.27	0.14
Eight storey	1.03	0.33	0.18

7. DYNAMIC ANALYSIS METHOD

In transient dynamic analysis, the following system of dynamic equilibrium equations is solved at each time t:

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+\left(f_{\text{int}}\left(\{u\},\{\dot{u}\},\{\varepsilon\},\{\sigma\},t,......\right)\right)=\{f_{ext}(t)\} \tag{1}$$

where $[M]$ and $[C]$ are the mass and damping matrices respectively, and $\{f_{ext}(t)\}$ is the vector of external forcing functions. The vectors $\{\ddot{u}(t)\}$, $\{\dot{u}(t)\}$, and $\{u(t)\}$ are the resulting accelerations velocities and displacements, respectively. The vector $\{f_{\text{int}}(t)\}$ is the internal set of forces opposing the displacements and is usually dependant on the displacements, velocities, and field of strains $\{\varepsilon\}$ and stresses $\{\sigma\}$.

In this study, a Rayleigh type damping is adopted. It is based on a linear combination of the mass matrix $[M]$ and the linear elastic stiffness matrix or the tangent stiffness $[K]$ as follows:

$$\alpha[M]+\beta[K] \tag{2}$$

where the coefficients α et β are determined to provide for two selected damping ratios for two specific modes of vibration, ie the first two modes.

The direct integration of the above equations is required. One of the most widely used methods is the Hilbert-Hughes-Taylor method [21]. This method has superior numerical dissipation characteristics over the widely used Newmark method. For large number of degrees of freedom, the numerical damping feature is essential to suppress undesirable higher modes.

8. RESULTS AND DISCUSSIONS

The analysis was carried out for different cases of frames subjected to (a) horizontal motion alone and (b) to a combination of horizontal and vertical motions with lumped mass model and distributed mass model using the free software Seismosoft [22].

8.1 Axial Forces

Figures 4-6 compare the axial forces in the columns of the two storey building. The axial forces in the exterior columns are increased by 15% and 11 % at the base and first level respectively for the lumped mass model and by 6% and 26 % for the distributed mass model. For the interior columns this rate is about 5% for the two mass models. The lumped mass model may overestimate the columns axial forces.

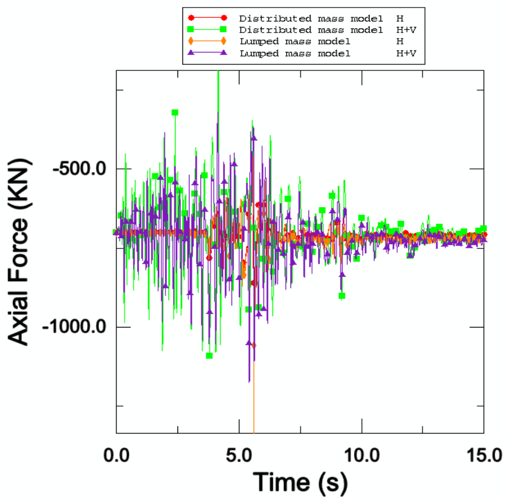


Figure 4. Two storey structure base exterior column

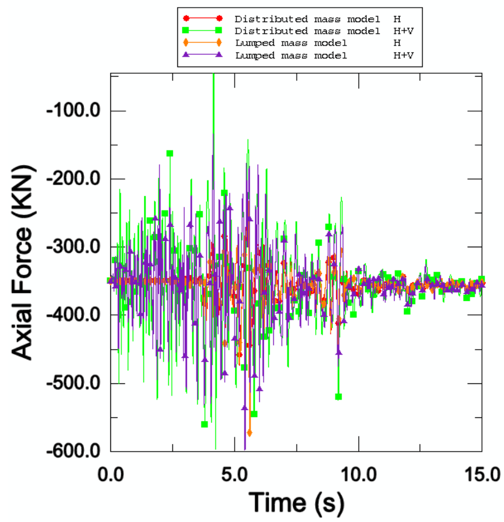


Figure 5. Two storey structure base interior column

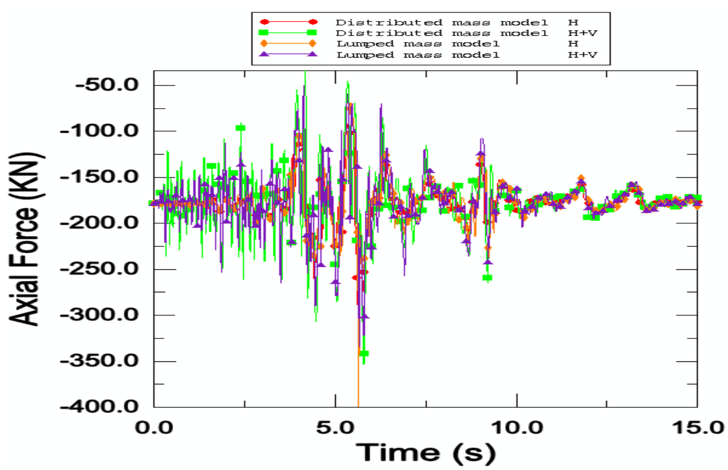


Figure 6. Two storey structure exterior column level 1

For the five storey building, the inclusion of the vertical component increases the axial forces in columns especially interior columns where the rate of increase can be as high as 84%

for the distributed mass model and 70% for the lumped mass model, Figures 7 and 8. For the exterior columns, Figures 9 and 10, this rate is about 12% for both mass models. In some circumstances, the compressive forces are decreased and can even change to tensile forces. Again for this building, the lumped mass model may overestimate the columns axial forces.

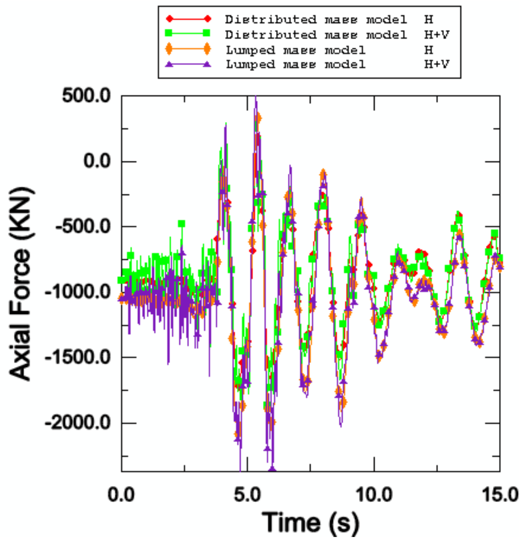


Figure 7. Five storey structure base exterior column

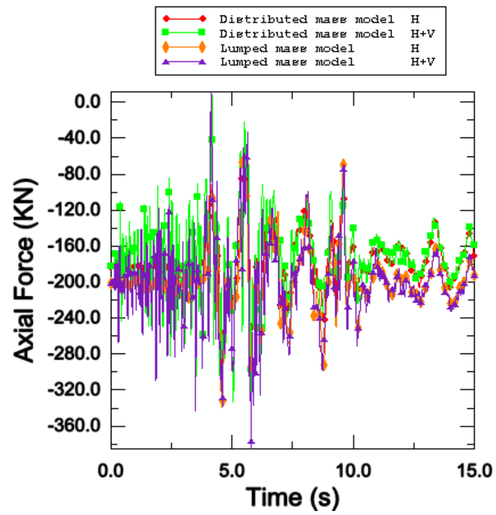


Figure 8. Five storey structure exterior column level 5

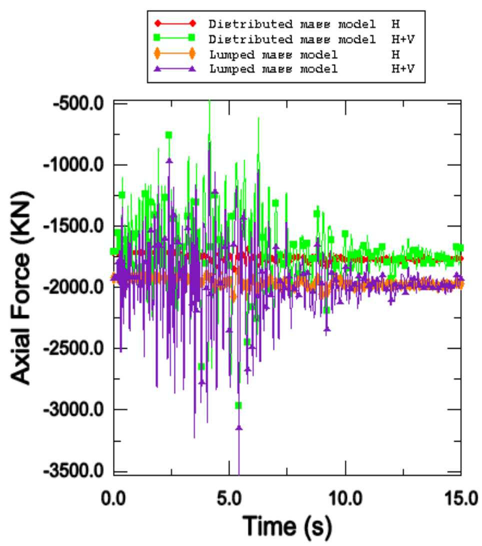


Figure 9. Five storey structure base interior column

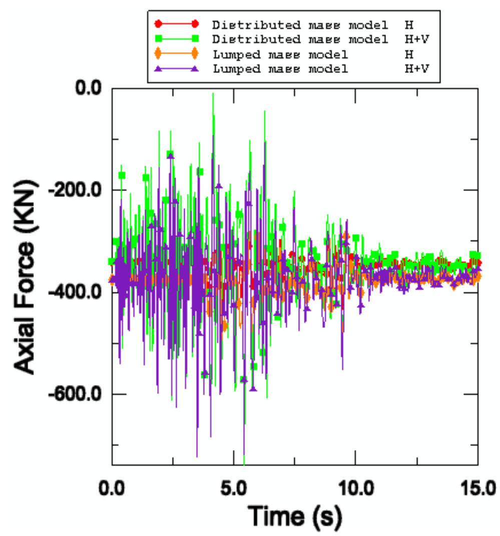


Figure 10. Five storey structure interior column level 5

For the eight storey building, Figures 11, 12, 13 and 14, the results obtained are similar to those obtained for the five storey building suggesting that the dynamic characteristics of the buildings can affect considerably the axial forces in the columns

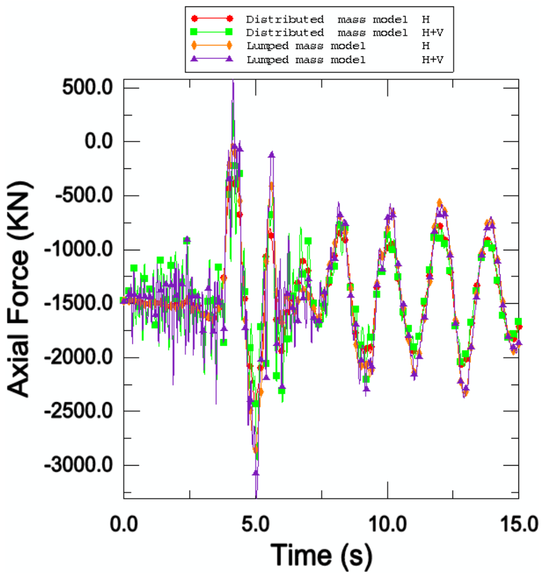


Figure 11. Eight storey structure exterior column level 1

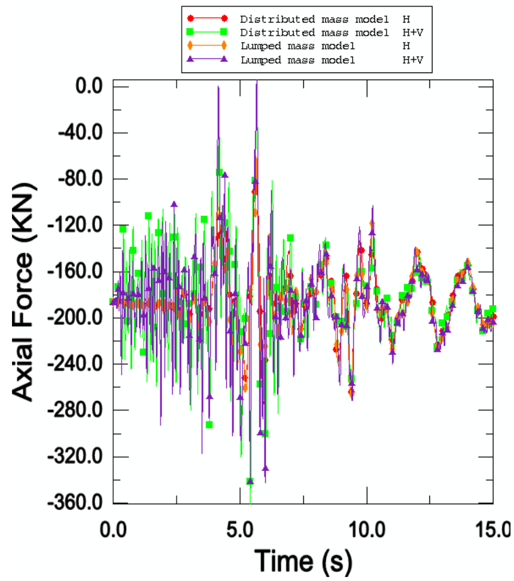


Figure 12. Eight storey structure exterior column level 8

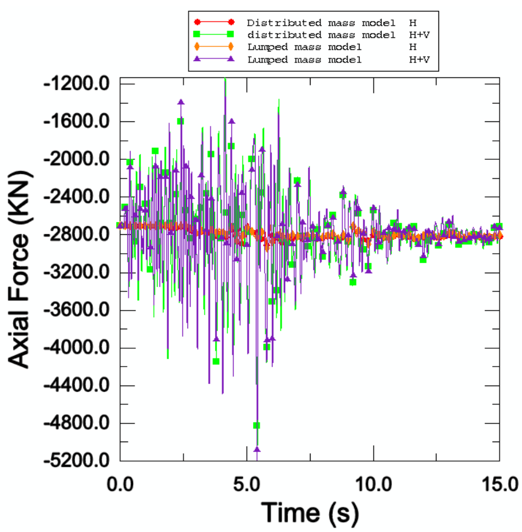


Figure 13. Eight storey structure interior column level 1

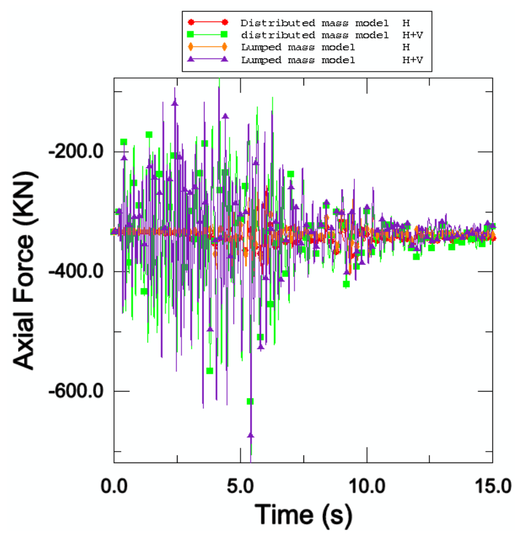


Figure 14. Eight storey structure interior column level 8

8.2 Shear Forces

Shear forces are not affected by the combination of vertical and horizontal earthquakes even though the values predicted by the two mass models present some differences, Figures 15, 16 and 17.

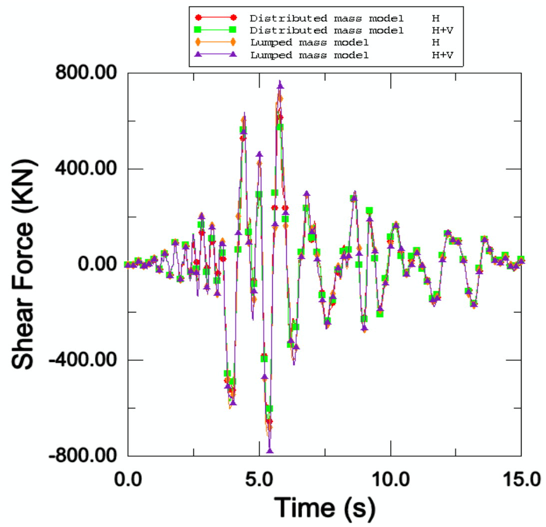


Figure 15. Two storey structure base shear

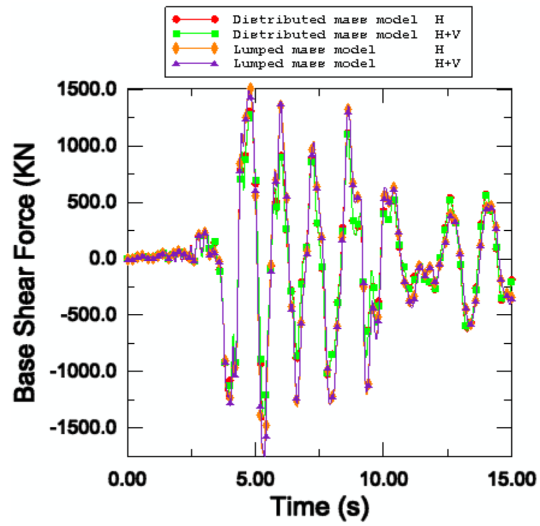


Figure 16. Five storey structure base shear

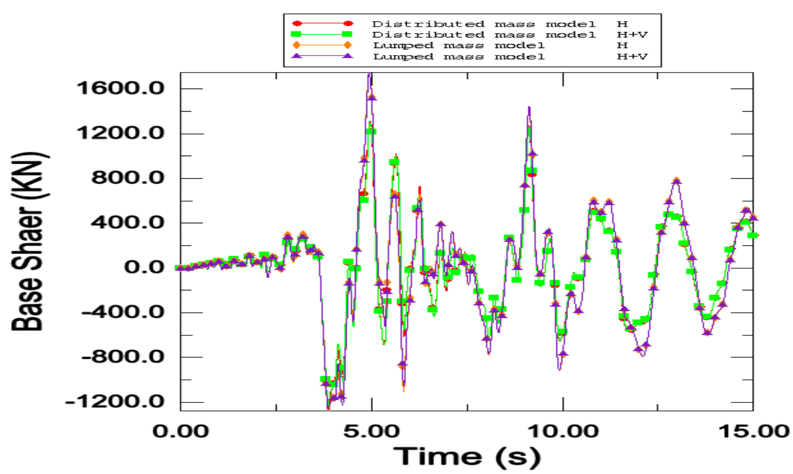


Figure 17. Eight storey structure base shear

8.3 Horizontal Displacements

The same conclusions reached for the floor shear forces apply to horizontal displacements, Figure 18.

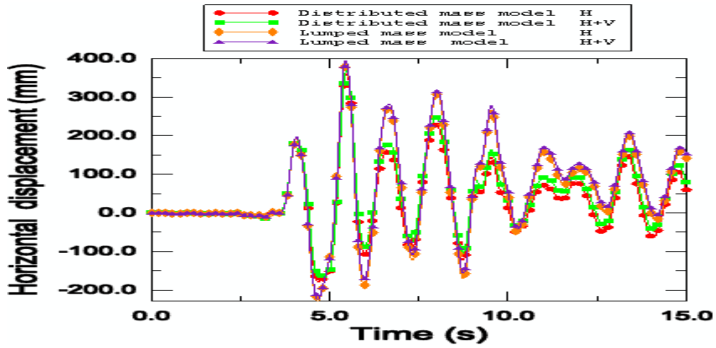


Figure 18. Horizontal displacement top node eight storey structure

8.4 Girder Vertical Displacement

The distributed mass under vertical motion affects considerably the vertical displacements of girders. The lumped mass model can consider only the vertical displacements associated with columns elongations, while the distributed mass model, results in larger vertical displacements by including the vertical vibration of floor systems into the response especially for the five and eight storey buildings which are flexible. Thus, the dynamic characteristics of the buildings tend to influence the vertical displacements which are magnified at floor levels, Figures 19 and 20.

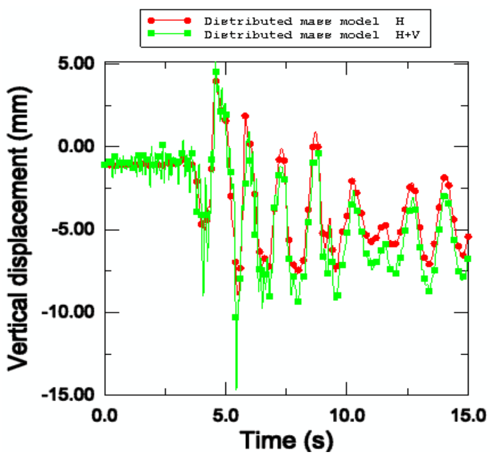


Figure 19. Vertical girder displacement five storey structure

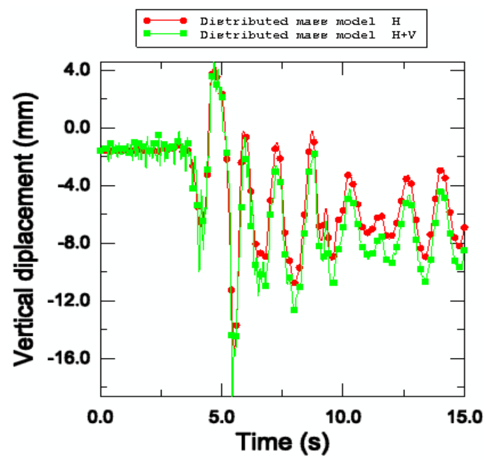


Figure 20. Vertical girder displacement eight storey structure

9. CONCLUSIONS

In this study, both the effect of vertical accelerations and distributed mass on the nonlinear dynamic response of reinforced concrete framed structures were investigated and it can be concluded that:

1. The axial forces in columns are significantly affected by the vertical motion, especially the interior columns.
2. The vertical accelerations can induce tensile forces in columns which can enhance the overturning moment.
3. The dynamic characteristics of the buildings can affect considerably the seismic response of reinforced concrete buildings when considering the effect of vertical motion since this effect is more pronounced in high rise buildings than in low rise buildings.
4. The lumped mass model may overestimate the columns axial forces.
5. Vertical displacements at floor level are magnified by the vertical component of ground motion.
6. Vertical accelerations within floors can be amplified and in seismic areas where high vertical accelerations are expected, this phenomenon can be very dangerous.
7. Vertical ground motion and distributed mass model do not have a great influence on horizontal displacements and story shears.

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