論文 Effect of Distributed Mass and Vertical Motion on Axial Forces of Columns in R/C Moment Resisting Frames

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ABSTRACT: Floor mass is conventionally assumed to concentrate at beam-column connections. Vertical vibration due to distributed mass along slabs and girders is neglected. Response of the conventional lumped-mass model was compared with the model incorporating distributed vertical masses along girders subjected to horizontal and vertical components of near field earthquakes. It was observed that axial forces in columns were significantly affected especially when the contribution of lateral seismic load was small such as in the interior columns. The effect of distributed mass and vertical motion on axial forces of columns was more significant in low-rise buildings.

KEYWORDS: nonlinear earthquake response analysis, distributed mass, near field earthquake

1. INTRODUCTION

The effect of vertical motion is usually neglected in a design procedure. The 1995 Hyogo-ken Nanbu earthquake and the 1994 California, Northridge earthquake have been highlighted the importance of vertical motion. Buildings in a city located close to an epicenter and/or a ruptured fault may simultaneously suffer non-attenuated horizontal and vertical components of a ground motion. In case of strong vertical motion, both the vertical column modes and the vertical floor modes of a frame must be considered. Some researchers placed a lumped mass at the mid-span of the floor system in order to include the fundamental vertical mode of the floor system [1, 2]. However, under horizontal and vertical motions, the maximum vertical response may not be occurred at the mid-span of the floor system. Higher vertical modes of the floor system may significantly amplify. A distributed lumped-mass model was developed to include the contribution of all vertical floor modes of a frame. A foregoing study on single-story one-bay reinforced concrete (R/C) plane frame structures revealed that the distributed mass under vertical motion greatly affected the axial force in columns, but had little effect on lateral drift and story shear [3, 4]. This paper is a further attempt to study the effect of distributed mass and vertical motion on the earthquake response characteristics of R/C frames.

2. METHOD OF ANALYSIS

A computer program, BASIJ, was developed for nonlinear earthquake response analysis of R/C frames with distributed mass along girders. Each girder was divided into 10 segments along its axis and floor mass was lumped at the internal nodes and at the beam-column connections. Gravity loads were applied gradually at the mass nodes in 100 loading steps prior to a dynamic analysis. One component model [5] was used for girder segments and for columns. Columns are assumed to be linearly elastic in axial direction and the interaction between bending moment and axial force was neglected. Only material nonlinearity was included, and the geometrical nonlinearity was assumed

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to be negligible. A trilinear skeleton curve was assumed for a moment-rotation relation at the end of each segment. The Sugano equation [6] was used to estimate the post cracking stiffness. The post-yielding stiffness was assumed equal to 1% and 5% of the initial stiffness for a girder-segment and a column, respectively. Axial stiffness was assumed to be linearly elastic. Beam-to-column joints were assumed to be rigid with a finite length equal to a column or beam width. The members were assumed to have infinite ductility. A viscous damping proportional to the mass matrix and instantaneous stiffness matrix was assumed. Damping factors were assumed equal to 0.05 in fundamental horizontal and vertical vibration modes at the elastic stage.

3. PROTOTYPE FRAMES

Five single-story one-bay plane frames with different span lengths, and a six-story two-span plane frame were designed. These prototype frame structures were subjected to different ground motions. Assumed properties of materials are listed in **Table 1**. The modulus of elasticity for concrete was calculated based on the equation introduced by the AIJ [7].

Table 1: Mechanical properties of material

Item	Denote	Value	Unit
Compressive strength of concrete	f_c'	30	MPa
Tensile strength of concrete	f_t	3	MPa
Reliable yielding strength of steel (SD40)	f_y	400	MPa
Tensile strength of steel (SD40)	f_u	500	MPa
Initial modulus of elasticity for concrete	E_c	29.2	GPa
Young's modulus of steel	E_s	200	GPa

3.1 SINGLE-STORY FRAMES

Five R/C plane frames with different span lengths of 4, 8, 12, 16, and 20 meters were designed for a combination of dead load of 36 kN/m, live load of 12 kN/m, and seismic coefficient C_b of 0.2 (**Fig. 1**). The dimensions of members were determined to require a tensile reinforcement ratio of about 1%. Member sections and natural period of frames with distributed mass (DM) model are listed in **Table 2**. Natural lateral periods of frames with lumped mass (LM) model were nearly identical to frames with the DM model. Natural vertical periods of frames with LM model (column mode) were constant at about 0.017 second for all frames. One half of live load was considered to be effective as inertia mass in the horizontal direction. Total dynamic weight of frames were 192, 384, 576, 768, and 960 kN for span lengths of 4, 8, 12, 16, and 20m, respectively.

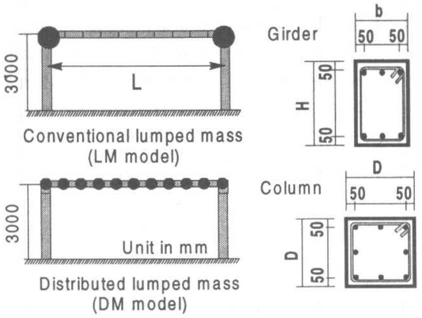

Table 2: Frame parameters and characteristics

L (m)	b (mm)	H (mm)	D (mm)	Natural periods (sec)	
				Horizontal	Vertical
4	300	400	350	0.16	0.06
8	400	550	500	0.13	0.11
12	500	700	650	0.11	0.14
16	500	700	800	0.12	0.22
20	500	700	900	0.08	0.36

Fig. 1: Analytical models and prototype frame structures

3.2 SIX-STORY FRAMES

A six-story two-bay R/C plane frame structure was designed to fulfill the requirements of the "Design Guideline for Earthquake Resistant R/C Buildings Based on Ultimate Strength Concept" introduced by the Architectural Institute of Japan in 1990 [7]. The Prototype structure was designed for a seismic coefficient C_b of 0.25. Geometry of sections were determined based on a linear analysis such that inter-story drift angle was less than 1/300 when frame was subjected to inverse triangular equivalent seismic design load. Location of intended yield hinges, weight of structure, dead and live loads at floors, and cross section of columns and girders are shown in **Fig. 2**. Member reinforcements are listed in **Table 3**. A nonlinear pushover analysis under triangular lateral seismic load was carried out to confirm the location of planned hinges (**Fig. 2**). Fundamental natural period of frame was 0.44 and 0.062 seconds for horizontal mode and vertical column mode, respectively

Table 3: Members reinforcements

Story	Member	Bottom steel	Top steel	Slab steel
6 (hinge)	Girder	2D19	2D22	4D13
6 (no hinge)	Girder	4D22	6D22	4D13
5	Girder	2D22	3D22	4D13
4	Girder	3D22	4D22	4D13
3	Girder	4D22	5D22	4D13
2	Girder	4D22	6D22	4D13
1	Girder	5D22	5D25	4D13
Hinge	Column	12D22		
No hinge	Column	12D29		

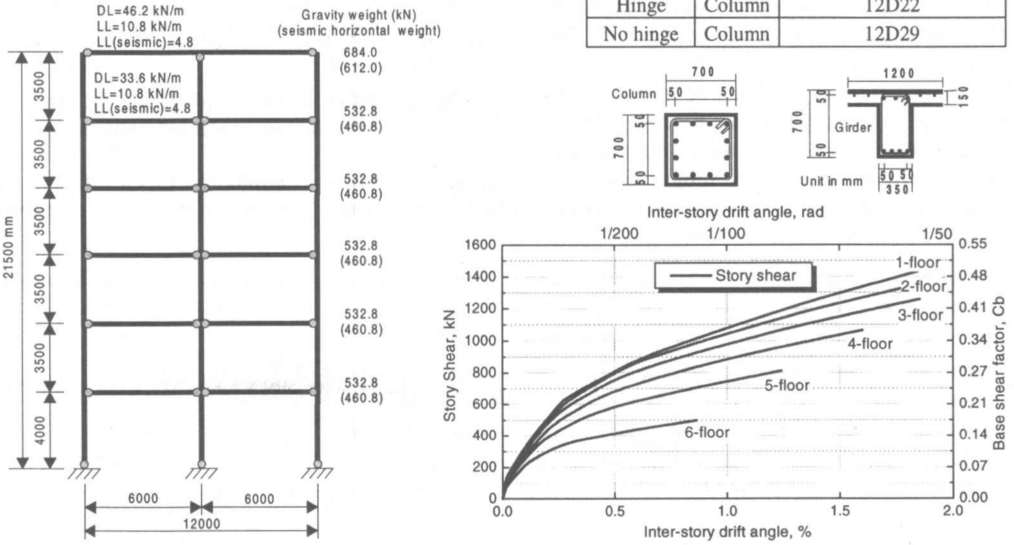

Fig. 2: **Elevation of the prototype structure, location of intended hinges, and results of pushover analysis**

4. NONLINEAR EARTHQUAKE RESPONSE ANALYSIS

The six prototype frames were subjected to horizontal and vertical components of seven renowned strong ground motion records listed in **Table 4**. The analysis was carried out for different cases of the frames with (a) distributed mass model under horizontal motion alone DM(H), (b) lumped mass model under horizontal and vertical motions LM(H + V), and (c) distributed mass model under horizontal and vertical motions DM(H + V).

Table 4: Free-field recording stations used in this study

Earthquake	Recording Station	Magnitude	Horizontal PGA (gal)	Vertical PGA (gal)	Faulting Distance (km)	Epicentral Distance (km)
Kobe	JMA	7.2	820.56	333.27	-	16.0
Northridge	Saticoy Street	6.7	444.15	785.05	0.0	2.2
Northridge	Sylmar County Hospital	6.7	826.76	524.99	1.5	15.8
Northridge	Newhall Fire Station	6.7	578.19	537.35	3.7	20.2
Northridge	Rinaldi Receiving Station	6.7	423.42	829.98	0.0	9.9
Kern County	Taft Station	7.7	152.70	102.80	-	43.7
El Centro	Imperial Valley	6.3	341.69	206.35	-	8.2

6. AXIAL FORCES IN COLUMNS

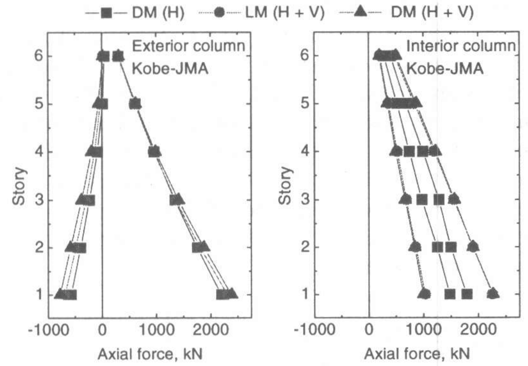

Fig. 3: **Maximum axial force in the interior and exterior columns**

Figure 3 compares the maximum axial forces in the interior and exterior columns of the six story frame subjected to the Kobe-JMA record. The axial forces in columns are significantly influenced by vertical motion, especially at interior columns. Distribution of maximum axial forces is nearly linear over the height of the building. The DM model may reduce or enlarge the maximum axial forces in columns when comparing with the results of the LM model. Axial forces in exterior and interior columns at first story of the frame are compared in **Fig. 4**. More fluctuations in column axial forces are observed when the frame is subjected to vertical motion. Interior columns are much affected by vertical motion than that of exterior columns, which were mainly influenced by the overturning moment due to horizontal motion.

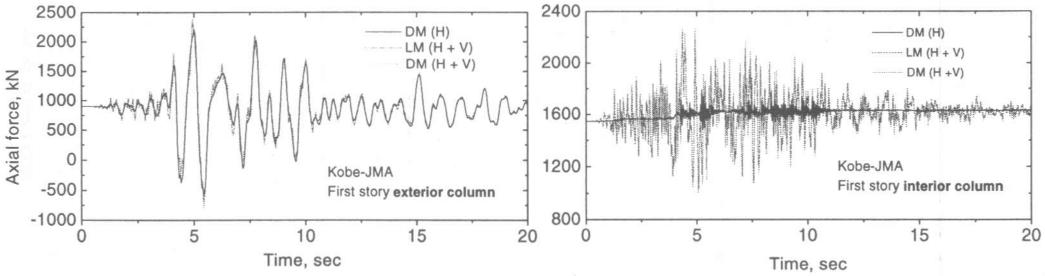

Fig. 4: Axial force in exterior and interior columns of first story

Figure 5 compares the interaction diagrams of interior and exterior columns of the six-story frame under Kobe-JMA records. It is observed that the maximum axial forces can be occurred simultaneously with the maximum bending moment. This is clearly realized in the left side of the interaction diagram of the interior columns. The interaction diagram may exceed yielding surface due to fluctuation in axial forces. Consequently a column which is designed only for the combination effects of lateral seismic load and gravity load may yield by the effect of distributed mass under strong vertical motion.

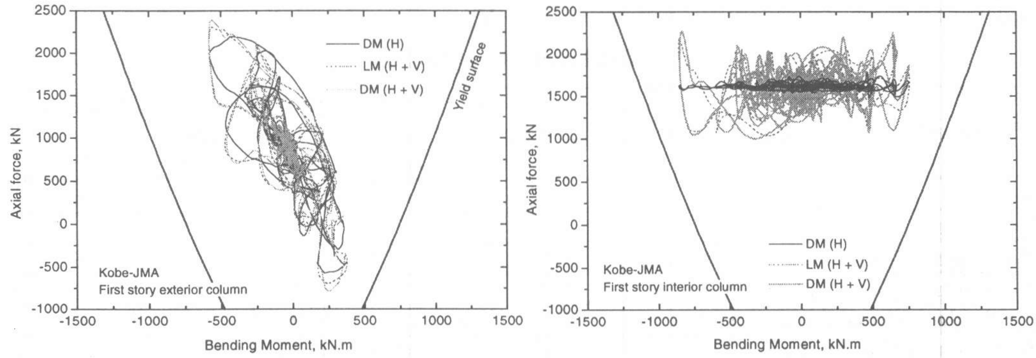

Fig. 5: Interaction response diagrams of the first story columns

The single-story frames, and the six-story frame with LM or DM model were subjected to the records of ground motions listed in the **Table 4**. Maximum compressive axial forces in columns of the frames with DM model and LM model are compared in **Fig. 6**. The differences in maximum axial forces between LM model and DM model are up to about 25% for single story frames, in which vertical vibration modes are governed by floor modes. The differences reduced to less than about 10% in six story frame, in which vertical vibration modes are governed by column modes. However, the differences between the LM and DM model in the interior columns under Northridge-Saticoy and Rinaldi records were significantly exaggerated. This may be attributed to the very strong vertical motions of the records.

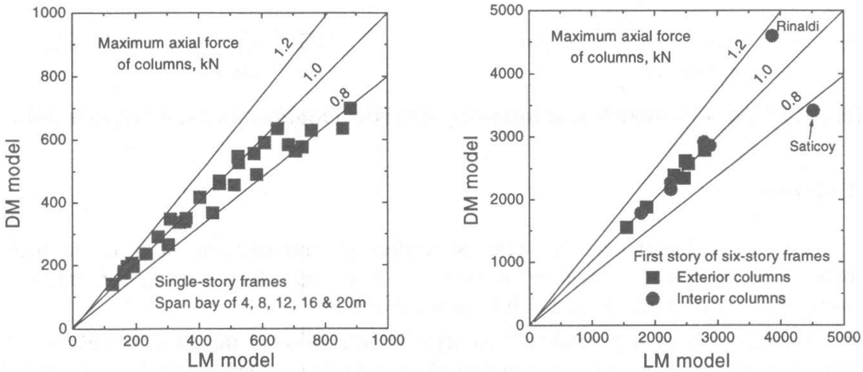

Fig. 6: Effect of DM vs. LM model on axial forces in columns of frames under various records

To study the effect of intensity of vertical motion and distributed mass on the axial forces in columns, the six-story plane frames were subjected to the strong ground motion records with vertical PGA fictitiously normalized to 0.0, 0.5, 1, and 2 times of horizontal PGA (i.e. a vertical to horizontal PGA of 0.0, 0.5, 1.0, & 2.0). In other words, Kobe-JMA record was normalized for a horizontal PGA of 820.56 gal and vertical PGA of 0.0, 410.28, 820.56, and 1641.12 gal; and the Northridge-Saticoy record was normalized for a horizontal PGA of 444.15 gal and vertical PGA of 0.0, 222.07, 444.15, and 888.30 gal. **Figure 7** shows the maximum compressive axial forces in the first story columns of the frames with DM model. The maximum axial forces are almost linearly proportional to the magnitude of vertical motion. Interior columns are more affected by increasing the magnitude of vertical motion. The maximum axial forces in columns of frames with DM and LM models under vertical motions with different intensities are compared in **Fig. 8**. It is observed that the effect of DM and LM models are negligible in most cases. However, under very large vertical motion of Northridge-Saticoy record, the LM model resulted to larger maximum axial forces in interior columns.

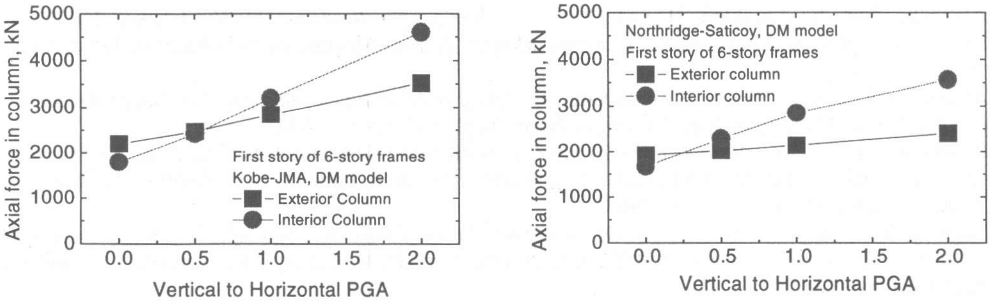

Fig. 7: Effect of increasing the magnitude of vertical motion on axial forces of columns

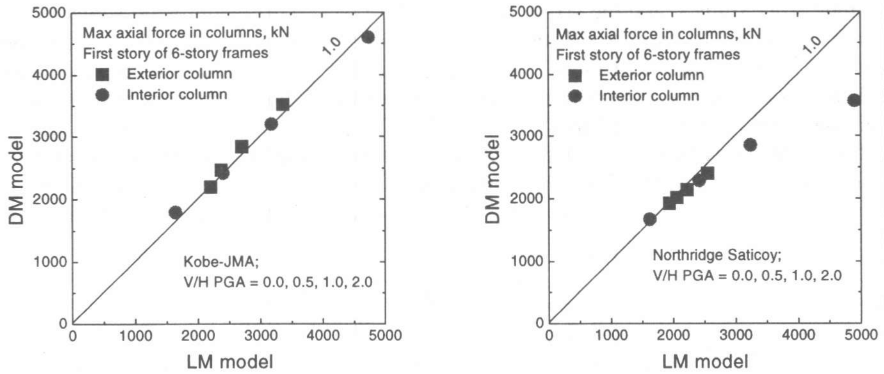

Fig. 8: Effect of DM & LM models and intensity of vertical motions to axial forces in columns

7. CONCLUSIONS

The effect of distributed mass as well as vertical ground motions of different near field earthquake records on the nonlinear dynamic response of five single-story one-bay R/C frames, and a six-story two-bay R/C frame were studied. It is concluded that:

1. The vertical component of a ground motion significantly affected the axial forces in columns. This especially happened when the contribution of lateral seismic load on axial load is small such as in a low-rise building or in the interior columns of an intermediate or a high-rise building.
2. The maximum axial forces of columns in a low-rise building, in which natural vertical period was governed by floor mode, were greatly affected by distributed mass. But axial forces of columns in an intermediate or high-rise building, in which natural vertical periods were governed by column mode, are less affected by distributed mass. Thus the effect of distributed mass can not be neglected when estimating the design load of a column in low rise buildings.
3. Vertical vibration caused more fluctuation in column axial forces. The size of maximum axial force in a column was almost linearly proportional to the intensity of vertical ground motion. Interior columns were more affected by increasing the magnitude of vertical motion.

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