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Effects of near-fault vertical seismic accelerations on the response of steel moment-resisting frames

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ABSTRACT

Inelastic dynamic analyses are performed on a 6-story moment-resisting frame designed according to current California seismic provisions for the city of Los Angeles. The frame is subjected to historically based and artificially generated impulsive near-field time histories proposed by the S.A.C. Steel Project. In this study through extensive parametric analyses, the effects of the vertical seismic accelerations on this moment-resisting frame are investigated. It is shown that the effect of the vertical accelerations on the demand in rotational ductility and on the maximum story deflections is negligible, even though a large increase in the maximum axial loads of columns is noted. Furthermore, vertical accelerations are found to increase the strain rate substantially especially in the early part of the shaking, when the structure is in the elastic domain.

INTRODUCTION

Following the January 17, 1994 Northridge, California earthquake, where brittle weld fractures were observed in beam-column connections of more than 100 steel moment-resisting frames, many studies have investigated the possible role played by the high vertical accelerations recorded during this earthquake (Papazoglou et al. 1996, Broderick et al. 1994). In this paper, the effects of the vertical accelerations on the plastic hinge formation, the maximum story deflections, the rotational ductility demand, the strain rate, and the increase in column axial loads, are investigated for a 6-story steel building subjected to near-fault earthquake records typical of the Los Angeles region. The results presented in this paper summarize the work presented in a more extensive study on the characteristics of vertical accelerations (Christopoulos 1998).

DESCRIPTION OF ANALYZED STRUCTURE

The 6-story structure studied in this paper was first designed by Tsai and Popov (1988), and modified by Hall (1995). As shown in Fig. 1, the structure is rectangular in shape, and spans 37 by 22 meters. Lateral loads in the North-South direction are resisted by two exterior moment-resisting frames. The structure is designed according to the 1994 edition of the Uniform Building Code (ICBO 1994) for a building in Zone 4 and on soil type S2. Design loads include 3.8 kPa of dead load on the roof, 4.5 kPa of dead load on the floors, a roof live load of 1.0 kPa, and a floor live load of 3.8 kPa.

The wind loads were calculated assuming a wind speed of 113 km/h and an exposure type B. The steel grade is assumed to be A36.

CHARACTERISTICS OF GROUND MOTIONS

Out of the 20 groups of accelerograms proposed by the S.A.C. steel project, 5 groups of time-histories were chosen for the analyses. These accelerograms represent near-fault, extreme events, and are used to predict ground motions for a major earthquake in the Los Angeles area.

Horizontal and a Vertical accelerograms were used for each group. For all pairs of earthquakes, the vertical spectral acceleration peaks occur at smaller period values than those of the horizontal spectral acceleration peaks. The fundamental horizontal period of the structure is estimated at about $T_h = 1.3$ s, and the vertical one at $T_v = 0.09$ s. For several earthquake records, the spectral ordinates at these periods are significant (Table 1). Figure 2 shows the spectra of three selected records. Table 1 summarizes the characteristics of the 5 pairs of earthquake records. The records were selected to maximize the spectral accelerations at the predominant periods of the structure, in both the horizontal, S_{aH}, and vertical, S_{aV}, directions.

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TABLE 1 : Characteristics of ground motions.

Earthquake	*PGA Horiz. (g)*	*PGA Vert. (g)*	*Total Duration (s)*	S_{aH} for $T_h = 1.3$ s *(g)*	S_{aV} for $T_v = 0.09$ s *(g)*	Ratio S_{aV}/S_{aH}
NF1	0.90	0.75	50	0.85	1.30	1.53
NF13	0.62	0.85	15	1.40	2.10	1.50
NF21	0.79	0.65	40	1.55	1.90	1.23
NF23	1.68	0.72	40	4.50	1.25	0.28
NF27	0.90	0.71	40	1.90	1.25	0.66

MODELLING ASSUMPTIONS

The 2D analyses were performed using the Ruaumoko nonlinear dynamic analysis computer program (Carr. 1996). Only an exterior frame was modeled since the interior frames were simply connected. A bilinear inelastic model was introduced to model the rotational hysteretic hinging at beam and column ends. Each member was assigned a plastic hinge length of 90% of its depth at each end. Shear deformations in panel zones were ignored. and only the response of the bare frame was included. The columns were assumed fixed at the ground level. A Raleigh type damping of 5% critical was assigned based on the first two elastic modes of vibration of the structure. Axial load-moment interaction surfaces were introduced according to LRFD (AISC 1993). Rigid end offsets were defined to account for the actual size of the member's connections. The actual yield strength of the steel was taken at 290 MPa.

The analyses were performed for the total duration of the earthquake records. with a time step increment of 0.002 s. Second order P-Δ effects were included for both the frame and the interior portion of the structure. Half the building weight. and 0.5 kPa of live load were included in the seismic weights of each level. The gravity load effects caused by the interior portion of the building were taken in account by adding an external 6-story column. attached to the main building by hinged connections. At each level. the interior weight of half of the structure was added. as well as the vertical masses not included in the analyzed frame.

NUMERICAL RESULTS

♦ Maximum floor deflections :

The envelope of maximum horizontal floor deflections is an important parameter when assessing the seismic response of buildings. since excessive deflections may cause loss of stability or excessive damage to inter-story elements. Soft story behaviour can also be detected from this parameter. The results of the numerical analyses show that peak horizontal deflections were unaffected by the consideration of the vertical accelerations in the analysis. A maximum variation of 1 % was noted. but no significant effect of the vertical accelerations could be found.

♦ Rotational ductility demand in beams and columns

The rotational ductility demand. μ^{θ}, is a good indicator to characterize the severity of damage of a moment-resisting frame. Since the major energy dissipating mechanism is the formation of rotational plastic hinges near the ends of beams and columns. this parameter is a direct measure of the structural seismic performance. The parameter μ^{θ} can be transformed into a plastic rotation by using the geometric properties of the section. It then becomes a failure criterion. Experimental data indicates that a plastic rotation of the order of 0.02 to 0.03 rad is sufficient to induce failure in beam-column connections. Based on the numerical results. the near-fault vertical accelerations were found to play a negligible role on the rotational ductility demand of both beams and columns. This contradicts some conclusions reported in the literature (Broderick et al. 1994). where an increase of 10 % in rotational ductility demand was attributed to the vertical accelerations.

♦ Axial load in columns

The most important effect brought by vertical acceleration as noted in this study. is the increase of axial loads in columns. Figure 3 shows the high frequency and increase of maximum axial load in column A of the structure for the

NF1 ground motion. The maximum compressive axial load increases to 2400 kN from an initial value of 1700 kN, when the vertical accelerations are included. In fact for both maximum compression and tension values, significant increases were attributed directly to the consideration of vertical accelerations in the analyses. As illustrated in Fig. 4, compressive loads were increased by 85% on average over the 5 earthquakes for the top middle columns, reaching a maximum increase value of 115%, for earthquake NF21, where the axial compressive load reached 370 kN from an initial value of 170 kN. Tension loads were increased by as much as 26.8% on average for top exterior columns. The mean, the maximum, and minimum axial load increase for the 5 earthquakes considered are summarized in Fig. 4. The large increases noted for the compressive load of the interior columns of the frame is explained by the fact that design loads for these columns are not influenced as much by the overturning moments as are the exterior columns. Added to this, the vertically accelerated mass attributed to the interior columns is larger then that attributed to the exterior columns, since the tributary area of these columns is much larger.

The increase in maximum column axial loads is explained by the vibration of beams due to the vertical excitation. In fact, the period of the structure in the vertical direction is very close to the periods of large vertical spectral ordinates. The vibrations induced by vertical accelerations are at a high frequencies, especially at the beginning of the earthquake. As the damage to the structure progresses, the period of the structure in the vertical direction is lengthened, and in some cases after major yielding of the structure has taken place, the effect of vertical accelerations becomes insignificant.

♦ P-M interaction curves for columns

The P-M interaction curve (Fig. 5) indicates that the large increase in column axial load does not modify significantly the structural response, nor does it increase yielding in the structure. This is explained by the fact that the columns are designed for very large bending moments and have large strength reserves in the axial direction (Fig. 5). Nevertheless, the large increase in axial loads, could lead to serious inadequacies for structural systems where members are designed mainly for axial load, like for example pined-braced structures. This could be assessed in a further study.

When considering the combined effect of vertical and horizontal accelerations, the phasing between these two solicitations must be assessed. The high frequency axial load fluctuation, attributed mainly to the vertical accelerations, and the bending moment variation at column and beam ends, attributed mainly to the horizontal accelerations, interact through the P-M interaction curves of the section. As noted earlier, the rotational ductility demand is unaffected by the presence of the vertical accelerations, despite the large increases of axial loads (Fig. 7a). Examining more closely a single yielding phase, and plotting the corresponding time-history of the axial load, for both analysis cases , with, and without vertical accelerations, we can see that the axial load fluctuates at a higher frequency with the vertical acceleration (Figs 7b, 7c). The mean value of the axial load during this phase is equal to the value of the static load. In fact, during a single flexural yielding phase, the axial load oscillates around the value of the static axial load of 1450 kN, spanning from a minimum value of 1150 kN in the first half of the yielding phase, to a maximum of 1900 kN which corresponds to an increase of approximately 30 %. During the first portion, where the axial load is decreasing, we see that when vertical accelerations are not included, the section starts yielding at a lower moment. Similarly, during the increase of the axial load in the second portion of the yielding cycle, the opposite can be observed; the section is yielding at a lower value for the analysis including vertical accelerations.

Considering this, it could be expected that vertical accelerations may be more critical for structures which respond in the horizontal and vertical directions at approximately the same frequency . This phenomenon may be favored by time histories where the predominant periods of ground motion in the horizontal and vertical directions are similar.

♦ Strain Rate

One of the most important characteristics of the high frequency vertical accelerations, is the increase in strain rate. It is well known that high strain rates affect the material properties of steel, increasing the elastic limit, thus increasing the plastic moment that a given section is able to develop. In classical capacity design approach, weld connections are designed according to the plastic moment of the adjacent beam, which is in fact the maximum theoretical moment that they have to carry. The strain rate effect increases the plastic moment. This can lead to brittle fractures in the connection weld, since welds are designed based on ultimate strength F_u, and are usually of high strength brittle material.

To assess the effect of vertical accelerations on this potential failure mechanism, strain rate time-histories were computed, for typical beam element, by multiplying curvature time-histories at the beam ends, by the lever arms of the sections, and for a typical column element by using the axial strain time-history (Fig. 6). With the absolute values of the strain rate time-history, and using the equation proposed by Wakabayashi et al. (1984), an estimate of the dynamic yield strength, F_{yd} time-histories can be calculated :

$$\frac{F_{yd}}{F_{ys}} = 1 + 0.0473 \cdot \log\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right) \tag{1}$$

where F_{yd} is the dynamic yield strength at a strain rate of $\dot{\varepsilon}$, and F_{ys} is the quasi-static yield strength under a strain rate of $\dot{\varepsilon}_0 = 50 \times 10^{-6}$ /s.

The results show that on average the yield strength is increased from 290 MPa to 315 MPa with a maximum value of 320 MPa, which corresponds to a 10 % increase. This increase is more important at the beginning of the shaking, when the structure is still in the elastic range. In fact for the first 2 to 3 seconds, the yield strength is notably higher when the vertical accelerations are included. Even when only the horizontal acceleration is used, high strain rate spikes can be observed. With the vertical acceleration, high strain rates are maintained constant at high values for a longer period of time. It is thus more probable that when a peak horizontally induced solicitation occurs, the steel will exhibit a higher yield strength, thus favoring brittle weld failure.

CONCLUSIONS

Based on the results of inelastic dynamic analyses on a 6-story moment resisting frame obtained in this study, the following conclusions can be drawn :

1) The analyses show that the effect of vertical accelerations on steel moment resisting frames is more important when the structure is responding in the elastic domain. As the yielding progresses the effect of vertical accelerations is filtered out, and becomes insignificant.

2) The numerical results obtained in this study show that the effect of vertical accelerations on the seismic response of moment-resisting frames is negligible in a general sense under near-fault ground motions. In fact, the demand in rotational ductility and in maximum story deflections, which represent the major indices to assess the seismic performance of such structures, are not sensitive to vertical accelerations.

3) Substantial increases in axial loads were directly attributed to the vertical accelerations. These increases reached higher values for the interior, top story columns. These axial loads did not induce stability or failure problems for the structure, mainly because elements are designed for high moments, thus have large strength reserves in the axial direction. These increases may represent a critical factor for other types of structures, like for example pinned-braced structures, where the elements work mainly as truss elements and bare larger axial loads.

4) Examining the yield phases, it is noted that due to the high frequency of the vertical accelerations, the axial load is both slightly increased and diminished during a single flexural yielding phase when the vertical accelerations are included in the analysis. The effect of vertical accelerations may therefore be more important when the response in the vertical and horizontal directions are in phase with similar frequency content. This behaviour could be favored by ground motions with similar predominant periods in both directions.

5) The vertical accelerations substantially increase the strain rate, consequently increasing the probable yield strength of the base material, and therefore favoring the occurrence of brittle weld failures.

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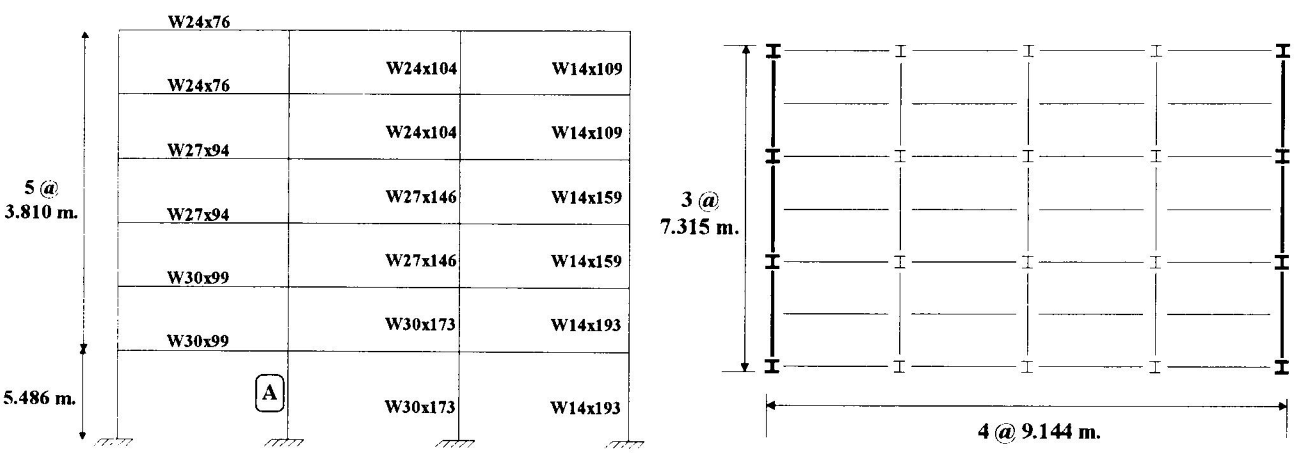

Fig. 1 : Building analyzed

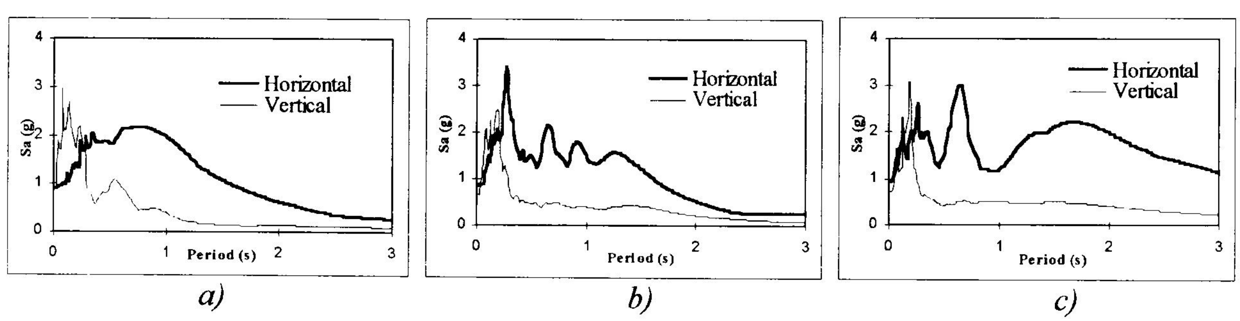

Fig. 2: Elastic response spectra (ξ =5%) : *a) NF 13, b) NF 21, c) NF 27 earthquake records*

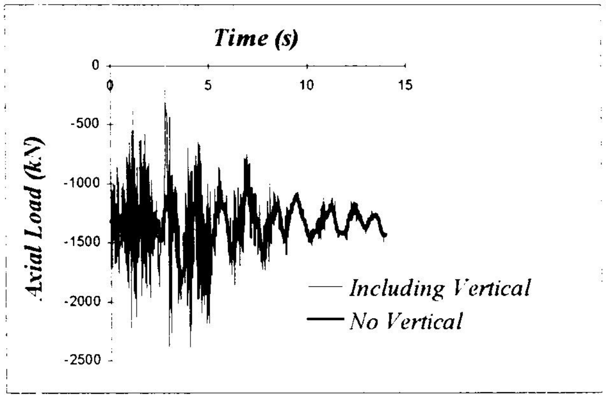

Fig. 3: Column A : axial load time-history (see fig.1) (NF 1 earthquake)

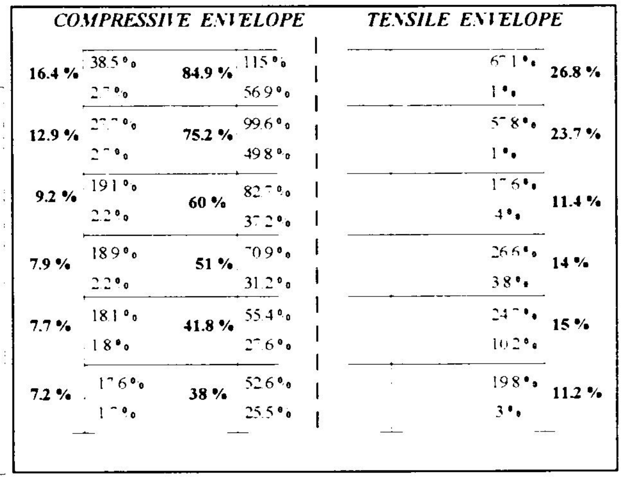

Fig. 4: Percentage of increase in axial loads
Bold Numbers : Mean value over 5 records.
Normal Numbers : Max and Min Values

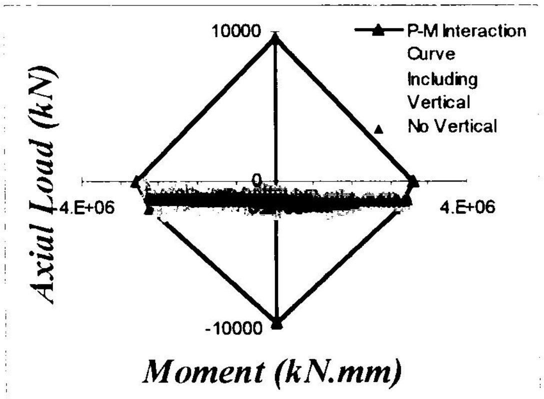

Fig. 5 : P-M interaction for column A (NF13 e.q.)

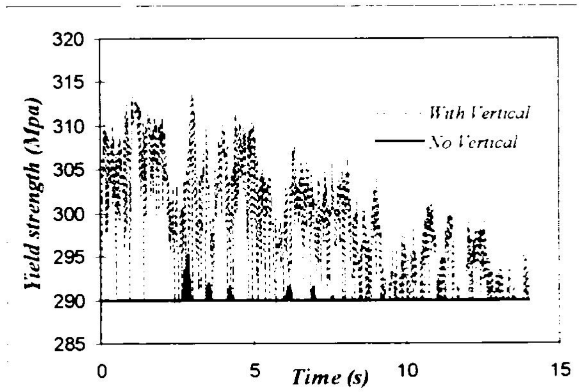

Fig. 6 : Yield Strength Column A (NF13 e.q.)

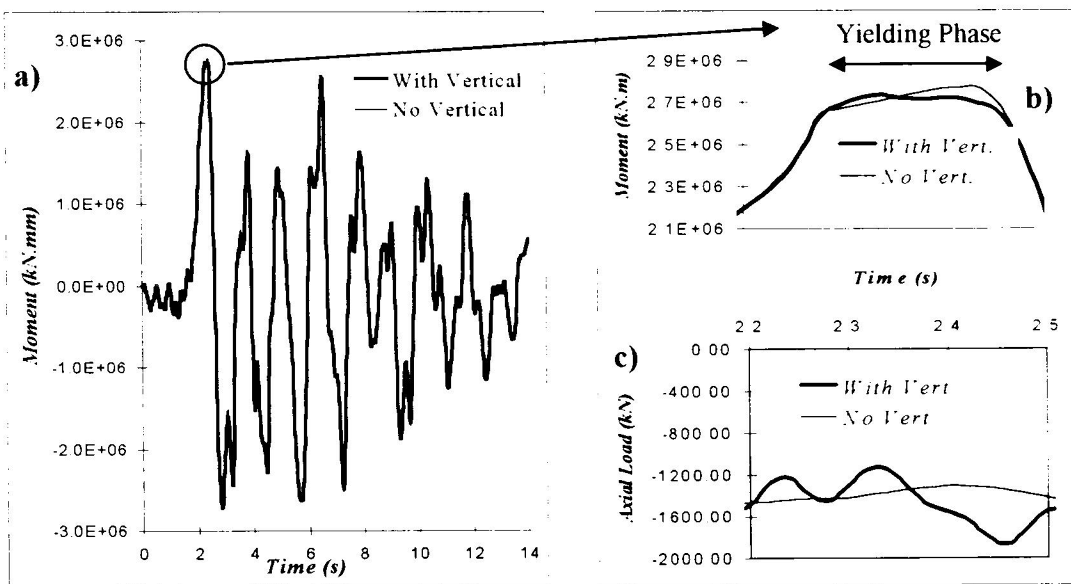

Fig. 7 : Yielding Phase : a) Moment time-history. b) Yielding moment peak. (NF 21 earthquake) c) Corresponding axial load