# DESIGN VERIFICATION OF AN EXISTING 8-STOREY IRREGULAR STEEL BUILDING BY 3-D DYNAMIC AND PUSHOVER ANALYSES

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## SUMMARY

Regarding that several buildings, specially irregular ones, which have been designed by using recent seismic design codes, have shown some vulnerability against earthquakes it can be said that there is still a need for modification of some of the seismic design codes for building systems. In this paper an irregular 8-story steel building, designed based on the Iranian National Seismic Standard, has been considered to verify its existing design by performing three-dimensional linear dynamic and nonlinear pushover analyses. Time History Analyses (THA) have been performed by applying the accelerograms of some local earthquakes in different configurations such as: only the principal horizontal component in that direction which leads to higher response values; the principal horizontal component in either of the principal directions and 30% of its values in the other direction, simultaneously. Pushover Analyses have been performed by using NISA-II program, which can consider both material and geometric nonlinearities. In this case the applied forces have been considered as the lateral forces of the Seismic Standard, without the effect of the response modification factor, multiplied by a coefficient of 1.7, suggested by most of well-known codes. All of nine loading categories recommended by design codes for steel building have been considered for analysis. Response quantities considered for comparison of the results include the displacements in different levels of the building, shear and axial forces and also bending moments in some corner, side, and middle columns and some bracing elements, and finally the stresses in the critical members. Numerical results show that in the case of multi-component excitations the response values can be much higher than those resulted by code recommended loadings. Furthermore, the nonlinear behavior of this building is very different from that assumed in the Code Seismic Analysis. This difference is more remarkable for the corner columns.

## INTRODUCTION

Many buildings, including steel ones, designed by even recently revised seismic design codes, have shown some vulnerability in recent earthquakes. Several researchers have reported the damages imposed on steel buildings of them some are more detailed [Engelhardt, 1996] [Nakashima, 1996]. It has been reported that in Kobe earthquake the number of damaged steel buildings has been considerable, with tilting and the complete collapse of the first floor as main modes of damages [Hassani and Takada, 1998]. The vulnerability of steel buildings, which is usually higher in the case of irregular buildings, can be partially due to the difference between the real seismic loading, applied on the building during earthquake, with the assumed code loading [Hosseini and Motamedi, 1999]; and partially due to the difference between the nonlinear behavior of the building with that assumed in the code by applying the response modification factor [Nasser-Assadi and Hosseini, 1999]. Although the aforementioned studies have clarified some main aspects of the problem, still it seems that more study is needed to make sure on the required modification of the seismic design codes for building systems.

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In this study an existing irregular 8-story steel building, designed based on the Iranian Code for Seismic Resistant Design of Buildings (issued in 1988 [1] and revised in 1997 [2]), has been considered to verify the existing design by performing three-dimensional linear dynamic and nonlinear pushover analyses.

## THE STRUCTURAL SYSTEM OF THE BUILDING

The building under study is an 8-story irregular steel building consisting of braced frames having different kinds of bracings. In E-W direction X and Z bracings have been used whereas in N-S direction Chevron or K bracing has been used because of some architectural restrictions. In the first two stories there is a circumferencial R/C wall on which the exterior column are based. Figure 1 shows the schematic skeleton of the building.

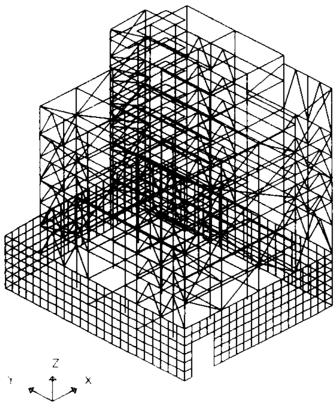



**Figure 1. The schematic skeleton of the building**

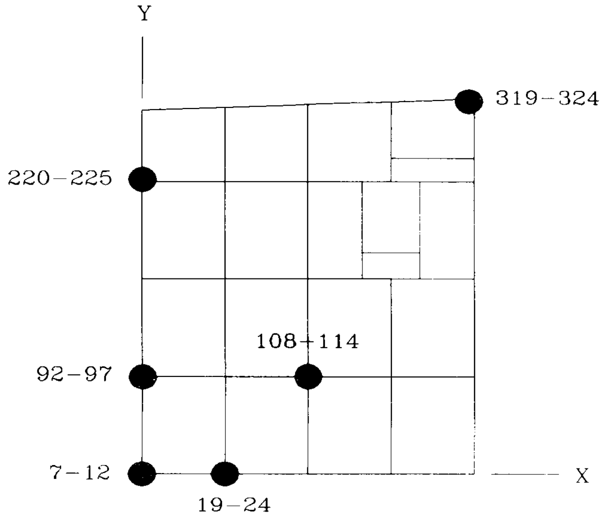



**Figure 2. Plan of the building, and the location of columns selected for detailed study**

All columns have box section and all beams are I section plate girders. Braces are made of U section profiles as well as I and box sections. All connections are simple or hinge and floors are composite section consisting of I section steel beams and R/C slabs. The plan dimensions in the first floor is 22.65 meter N-S by 23 meter E-W. In N-S direction the columns spacing is 7.55 meters and in the E-W direction is 5.75 meters, both equally. As it can be seen in Figure 1 this building has some irregularities in plan so that the equivalent static loading of the Code can not be used for its seismic design.

## THREE-DEMENSIOANL TIME-HISTORY ANALYSES

Time History Analyses (THA) have been performed, using SAP-90 computer program, by applying the accelerograms of some local earthquakes, such as Naghan and Sarkhoon, in four different configurations. 1) The longitudinal component in x direction and transverse component in y direction; 2) The longitudinal component in y direction and transverse component in x direction; 3) The longitudinal component in x direction with 30% of the transverse component in y direction; and 4 ) The longitudinal component in y direction with 30% of the transverse component in x direction. The two first cases are based on the recommendations of the original issue of Iranian Code, while the second two cases are based on its modified version. Two cases of single component excitation by using the main component in either of x and y axes has been performed as well for comparison. In all THA both importance and response modification factors have been considered.

## PUSHOVER ANALYSES

Pushover analyses have been performed by using NISA-II computer program, which can consider both material and geometric nonlinearities. Some of nonlinear models for material, which can be used in NISA-II program are shown in Figure 2. In this study the "elastic linear hardening" model has been used. The applied forces for pushover analyses have been considered as the lateral forces of the Seismic Standard, without the effect of the response modification factor, multiplied by a coefficient of 1.7, suggested by most of well-known codes. All of nine loading categories recommended by design codes for steel buildings have been considered for analysis [INBC, 1992].

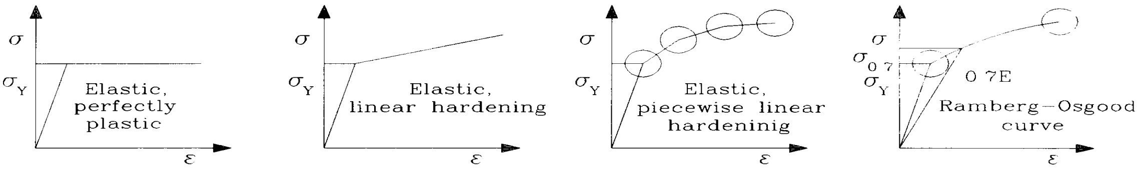



**Figure 3. Some stress-strain models used in NISA-II computer program for nonlinear analysis**

## NUMERICAL RESULTS

Response quantities considered for comparison of the results include bending moments, axial and shear forces in some corner, side, and interior columns, which their locations and member numbers are shown in Figure 2, the drift values in different levels of the building, the von Mises stresses in some of the critical members, and finally the lateral load distribution over the height of the building in both linear and nonlinear cases. The fundamental period of the building is 0.79 second without the effect of infills. Figure 4 shows the results of THA for members 7 to 12 (referring to Figure 2) in comparison with the results of Equivalent Static Loading.

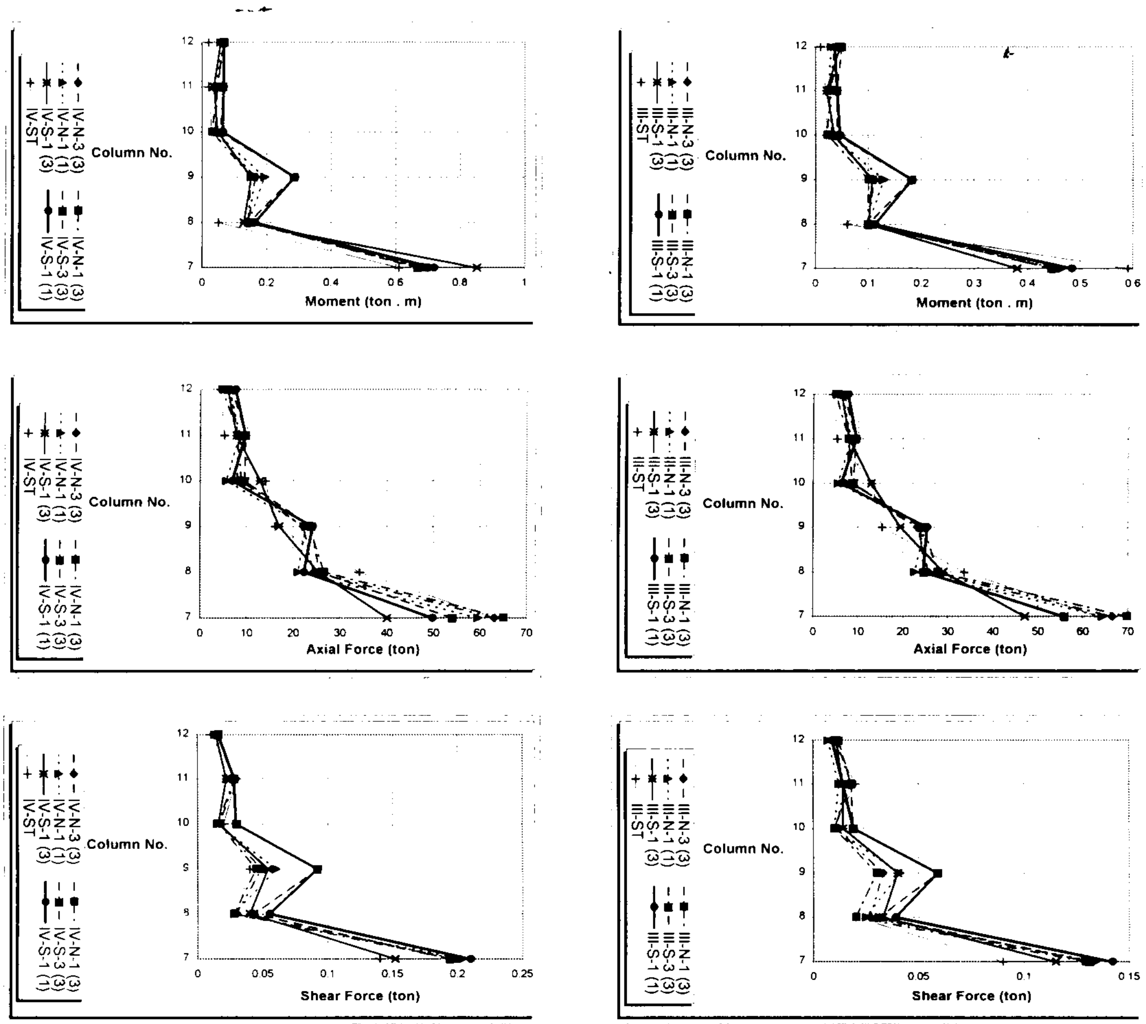
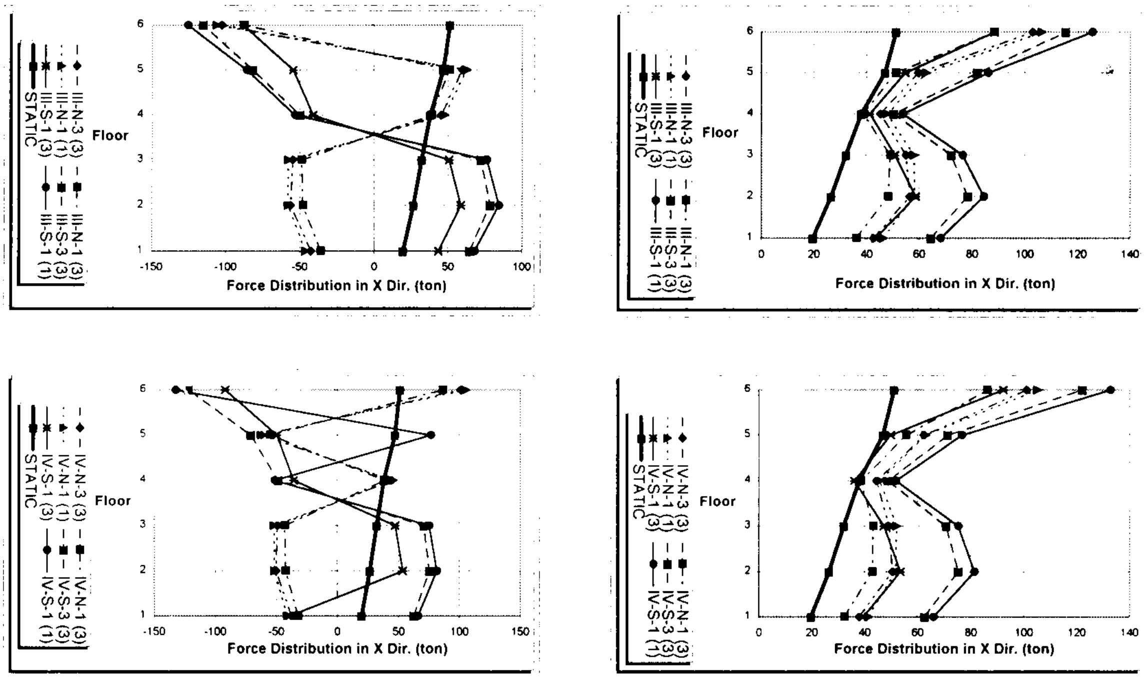



**Figure 4. Internal forces in series of columns, number 7 to 12, obtained from linear THA using single- and multi-component accelerograms of selected earthquakes compared with the equivalent static analysis**




**(a)** **(b)**

**Figure 5. Distribution of lateral forces over the height of the building obtained from dynamic and equivalent static loadings: (a) real values, (b) absolute values**

Figure 5 shows the distribution of the lateral forces over the height of the building for both dynamic and static analyses. In Figures 4 and 5, III is corresponded to the structural cross-sections introduced in technical structural drawings, while IV is related to the equivalent cross-sections, proposed and used by the builder instead of the initial cross-sections, because of the enforced restrictions on the material market in the country. In these Figures ST, N and S mean the equivalent static, Naghan Earthquake and Sarkhoon Earthquake, respectively. Finally the numbers inside the parentheses refer to the number of component(s) used in THA and the outer ones refer to the configuration number of accelerograms' components appliance. As it can be seen in Figure 4 that in most cases the response values obtained by THA are higher than the corresponding equivalent static analysis. Also it can be observed that the response to single-component accelerograms is higher than those to multi-component ones. These results are confirmed by Figure 5, which shows the lateral load distribution for different cases.

Figure 6 shows the internal forces in columns 7 and 108 obtained from pushover analyses for two different load combinations in x and y directions. The variation of von Mises stresses versus the load steps in pushover analyses for columns 7, 19 and 108 are demonstrated in Figure 7 for two different load combinations. Figure 8 shows the shear forces of columns 7 and 108 versus the inter-story drift for one of the load combinations. The variations of story shears versus the inter-story drifts are shown in Figure 9 for two different load combinations. Figure 10 shows the distribution of lateral forces over the height of the building in the case of pushover analyses for two different load combinations. More detailed results of the building analyses in both linear and nonlinear cases can be found in the main work by the second author [Yaghoobi, 1999].

It can be seen in Figures 6 and 8 that there are some disturbances in the variation of moments and shear forces in columns, which at the first look can make the implication of negative stiffness of columns. These disturbances are more remarkable in the case of column 7. In fact the main reason for these disturbances is the formation of plastic hinges in some columns, while other columns and bracing elements are still in the elastic range. These phase changes of different members in different steps of loading can result in the redistribution of corresponding lateral forces carried by different columns as well as bracing elements. It is obvious that by increasing the load step some other elements enter the plastic phase and the incorporation of formerly yielded members to carrying the lateral load increase again leading to an increase in the slope of the corresponding curves. It should be noted that these disturbances are not seen in the general force-displacement of the building as shown in Figure 9. Furthermore, it is noticeable that the geometric nonlinearity affects strongly on the aforementioned disturbances as shown in Figure 8, which is related to the column number 7 that is a member of bracing system as well. This effect is more remarkable in the few last steps of loading before the building collapse as shown in the Figure.

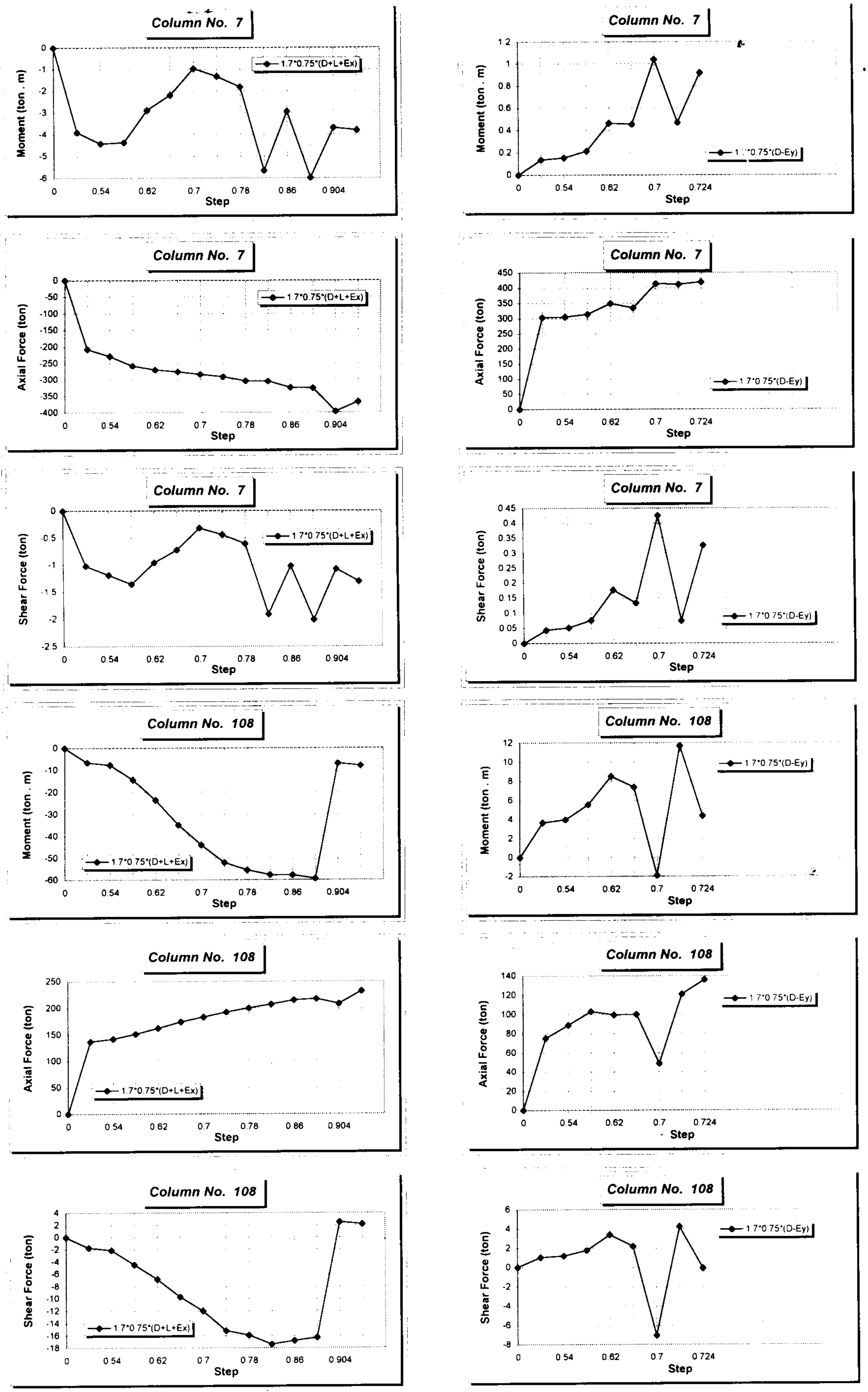



**Figure 6. Internal forces in columns 7 and 108 obtained from pushover analyses for two different load combinations in x and y directions**

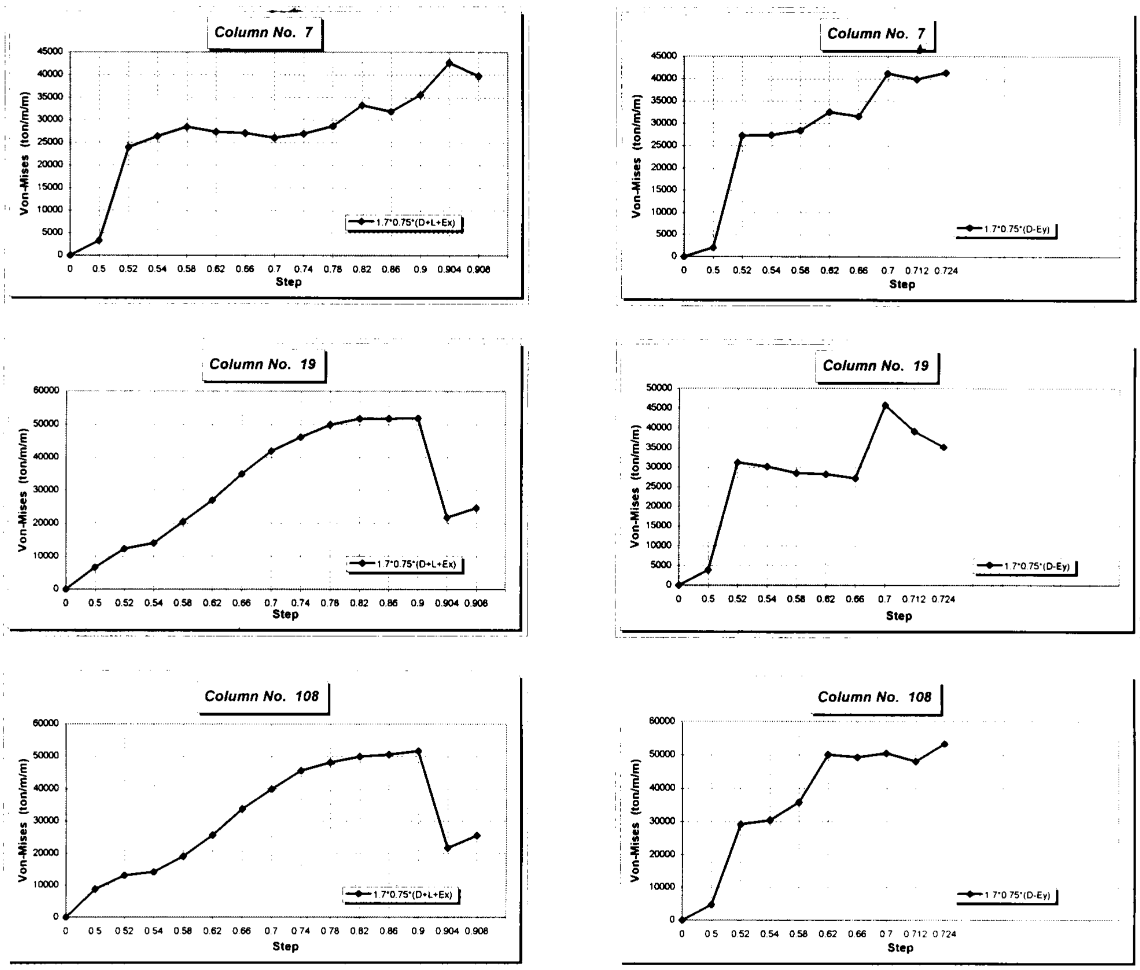



**Figure 7. The variation of von Mises stresses versus the load steps in pushover analyses for columns 7, 19 and 108 for two different load combinations**

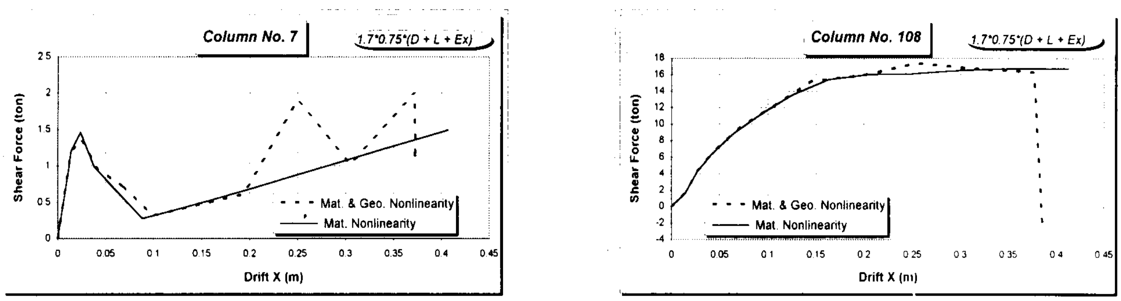



**Figure 8. The shear forces of columns 7 and 108 vs. the inter-story drift for one of the load combinations**

## CONCLUSIONS

Based on the numerical results it can be concluded that normalizing the dynamic base shear values to the equivalent static base shear lead to increase of response of single-component accelerograms comparing with the multi-component ones. Furthermore, it can be seen from numerical results that considering the geometric nonlinearity in addition to the material nonlinearity decrease the ultimate sustainable displacement of the building. Finally it can be concluded that the nonlinear behavior of this building is very different form that assumed in the Code Seismic Analysis, specially form the response modification factor point of view. This difference is much more for the corner columns. Therefore, it seems that the used seismic code needs some modification, particularly for the case of irregular buildings.

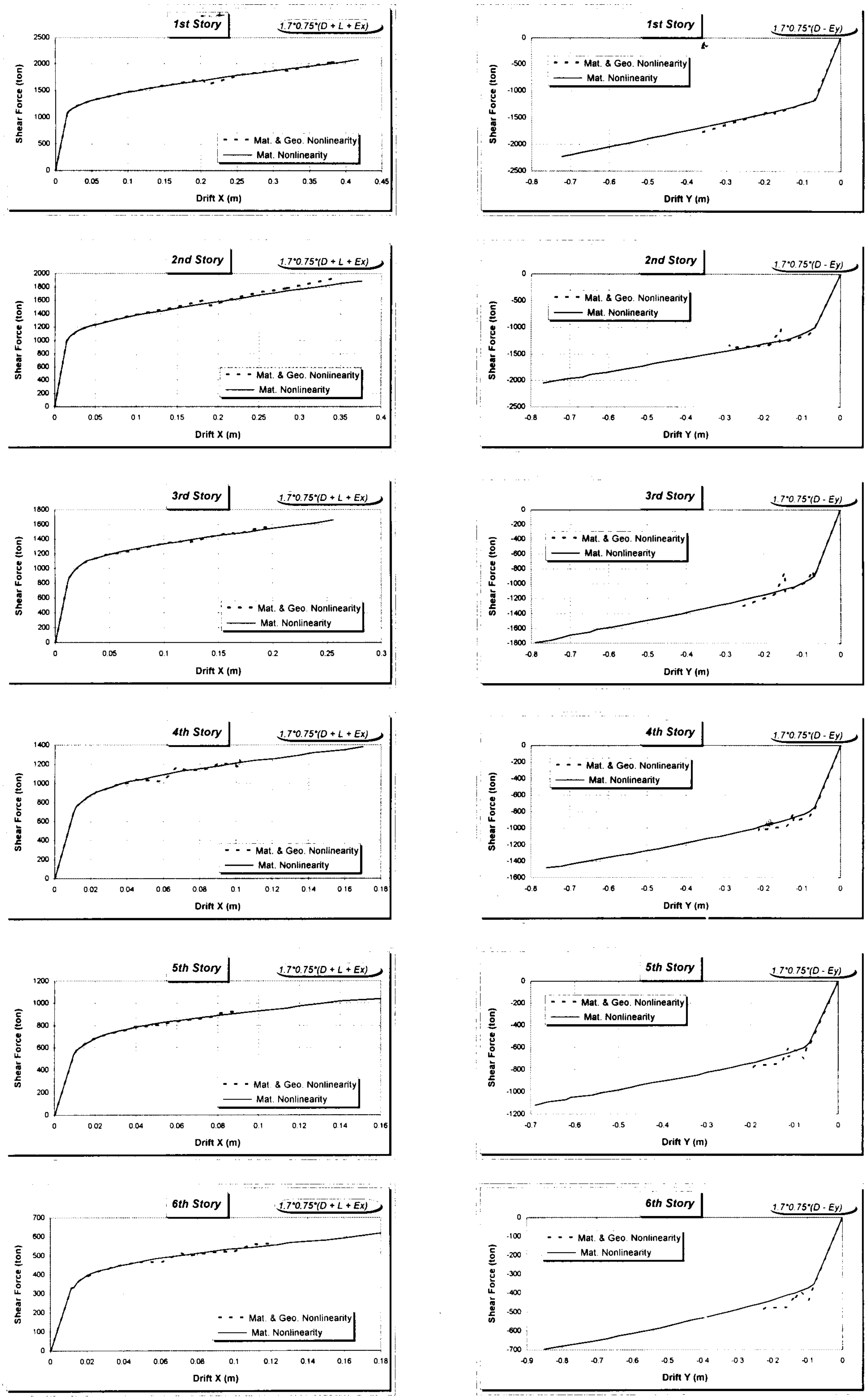
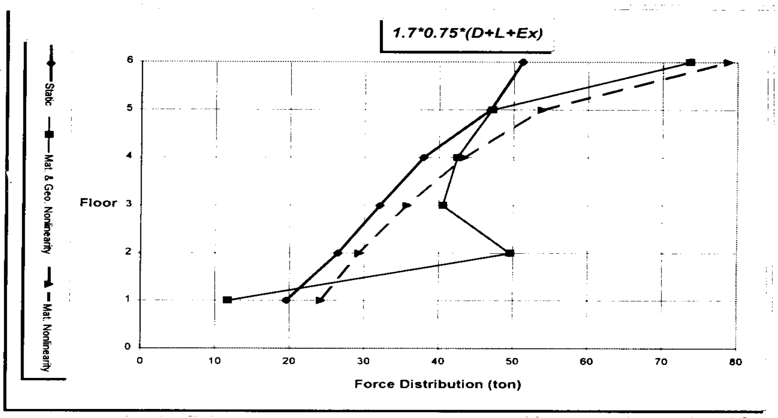



**Figure 9. The variations of story shears versus the inter-story drifts for two different load combinations**



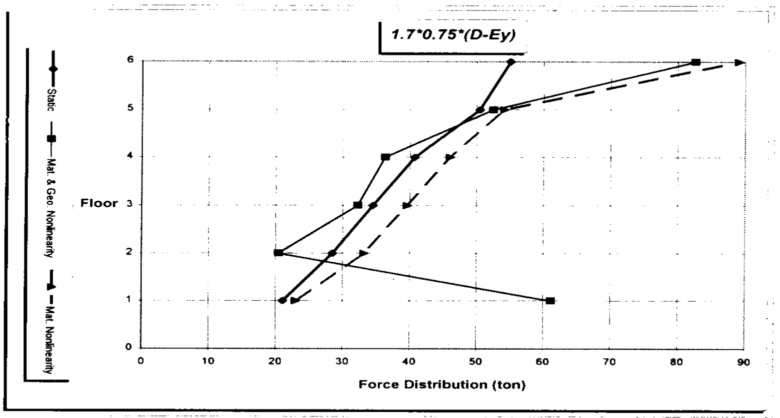




**Figure 10. The distribution of lateral forces over the height of the building in the case of pushover analyses for two different load combinations**

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