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INTRODUCING A STORY-DEPENDENT RESPONSE MODIFICATION FACTOR FOR STEEL BUILDINGS WITH CONCENTRICALLY BRACED FRAMES

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SUMMARY

Almost all of the existing seismic design codes suggest a so-called “response modification factor” for taking into account the plastic behavior of the structure. This factor appears in the calculation of the building total seismic shear force rather than the lateral load distribution calculations, which is believed to be more dependent on the plastic behavior of the system in its various story levels. To find out how far the actual distribution of lateral loads is from the assumed patterns of the design codes some sets of concentrically braced steel buildings as the most common type of moderately high rise steel buildings, having up to 18 stories and up to 5 bays, have been analyzed by a nonlinear time history analysis (NLTHA) program. The maximum values of lateral loads experienced by the building in each case have been compared with the lateral load pattern of the code, used for the design of the building. Great differences are observed between these two patterns in all cases. Then, buildings have been redesigned by the average load pattern obtained in the previous stage, and the NLTHA has been repeated. This time a good agreement is observed between the actual load distribution and the average pattern used for the redesigning of buildings. By calculating the ratio of the values given by this average load pattern to those values given by the suggested linear pattern of the code a somehow new concept of “story-dependent response modification factor” can be defined. Results show that this ratio is more than unity in lower and higher stories of the building and less than unity in the intermediate stories.

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INTRODUCTION

The so-called "response modification factor" which can be seen in almost all of the existing seismic design codes of building systems is supposed to take into account the plastic behavior of the structure subjected to sever loadings. This factor appears in the base shear force formula just as a reduction factor, and since the value of lateral load at each story level is a portion of total shear force value and the distribution of lateral loads along the height of the building is assumed to be dominated by the first vibration mode of building, the same reduction factor is automatically applied to all lateral forces. Since for every building the reduction factor is, based on the code, a single value which depends just on the type of the lateral load resisting system of the building regardless of its height, aspect ratio, and other features, it is implicitly assumed that the plastic behavior of the building in its various stories is almost the same. However, this assumption can not be true at least for beams of a moment frame, as their design in lower stories is controlled by the lateral loads, while in upper stories it is controlled basically by gravity loads. Therefore, this point comes to mind that using a single-valued reduction factor for all of the lateral loads in a building can not be true, and it is much reasonable to consider some story-dependent load reduction factor.

In recent years, vulnerability analyses of several steel and r/c buildings have shown that the actual lateral loads which a building experiences during a severe earthquake do not have a linear distribution, although the building has been designed by linearly distributed lateral loads based on the code. For example, Nasser Assadi and Hosseini [1] have studied the ductility factor of common steel buildings in Iran and have shown that firstly this value is not close to the code suggested value, and secondly the lateral load pattern, which really affects the building during an earthquake is far from the rather simplified linear distribution suggested by the code. As another example Hosseini and Motamedi [2] have also shown that the distribution of lateral seismic forces in the height of R/C buildings is not similar at all to the code proposed distribution. Similar results have been obtained in some other studies by Hosseini and Yaghoobi Vayeghan in a study entitled "design verification of an existing 8-Story irregular steel building by 3-D dynamic and push-over analyses" [3], and also by Hosseini and Firoozi Nezamabadi in another recent study entitled "seismic vulnerability assessment for selected steel building by use of visual inspection and nonlinear dynamic and pushover analyses" [4].

In this study it has been tried to find out how far the actual distribution of lateral loads is from the assumed patterns of the design codes for one of the most common building systems in Iran. For this purpose three sets of concentrically braced steel buildings, having 6, 12, and 18 stories respectively, and up to 5 bays have been considered to be analyzed by a 3-demensional nonlinear time history analysis (NLTHA) program. Buildings have been assumed to be regular in plan and elevation to be designed by the equivalent static load method of the code, and it has been assumed that the effect of infills can be eliminated by some construction considerations. Normalized accelerograms of some Iranian earthquakes such as Zarrat, Zanjiran, Fin, and Shabankareh, selected based on the different soil conditions of the recording stations to be compatible with various soil types of the code, have been used for NLTHA, considering both longitudinal and transverse components [5]. The maximum values of the lateral loads experienced by the building in each case have been compared with the lateral load pattern of the code, used for the design of the building. There are great differences in all cases between the actual load distribution and the code load pattern. Then, buildings have been redesigned by the average load pattern obtained in the first set of analyses, and the NLTHA has been repeated. This time a good agreement has been observed between the actual load distribution and the average pattern used for the redesigning of buildings. By calculating the ratio of the values given by this average load pattern to those values given by the suggested linear pattern of the code a somehow new concept of "story-dependent response modification factor" has been defined. In the following parts of this paper at first the building models considered for study are introduced, then the results of NLTHA are presented and discussed, and finally the new "story-dependent response modification factor" is introduced.

BUILDINGS CONSIDERED FOR NLTHA AND THE EMPLOYED APPROACH

Three sets of buildings with 6, 12, and 18 stories, all having rectangular plan, have been considered for modeling and analysis. Numbers of bays in these three sets are 3, 4, or 5 in either direction. Each set include two concentric bracing configurations, one with bracing just in one bay along the height of the building in each direction, and the other with two braced bays in each direction. It is believed that these different bracing configurations have some effects on the plastic behaviors of the buildings, while the response modification factor, given by the code for these two configurations, are the same. To be as much as possible close to the mostly desired features recommended by the seismic design code for the buildings they all considered to be quite regular in plan and elevation and to be symmetrical in both directions. It is believed that with these features the result of NLTHA should be in good agreement with the code results. The employed approach for the study has been as follows:

- At first, each building has been designed based on the seismic design code (National Standard No. 2800) and the national regulations for design of steel buildings, which is based on the allowable stresses. All four types of soil conditions given in the code have been considered which have led to various designed sections for the frames and bracing elements. To minimize the effect of over-strength resulting from the use of available steel profiles, the auto-select option of the design program has been used so that the section properties exceed the required values obtained by calculation by just minimum values. This means that the stress ratios are mostly very close to unity.
- After achieving the appropriate design in each case the NLTHA have been performed for the building by applying the two horizontal components of selected accelerograms simultaneously. All accelerograms have been normalized to the peak ground acceleration (PGA) value of 0.35g for their dominant components to cope with the code value for the high seismic hazard zone of the country. In these analyses the vertical components of accelerograms have not been considered as the code does not consider the vertical excitation for seismic loading. The desired response values in these analyses include the time histories of shear forces as well as story drift values in various levels.
- In the next step the maximum values of lateral forces acting on the building for each earthquake have been obtained by subtracting the shear time histories of excessive floors. Furthermore, in each case the maximum shear values at various levels of the building resulted from different accelerograms have been obtained and their differences which are the virtual maxima of the lateral forces have been also calculated.
- Then all of maximum values of lateral forces obtained for various story levels have been compared to each other, and their average values have been compared with the code corresponding values. These average values from several earthquakes represent a new lateral load pattern, which is not similar to the code pattern as it can be seen in the next section of the paper.
- The new load pattern, obtained by averaging in the previous step, has been used for redesigning the building structure as it is believed that this design should be closer to the optimum design.
- To make sure that the second design of the building is better than the first one the NLTHA with the same accelerograms have been repeated. This time the lateral load pattern obtained by the same method as before are again not similar to the code load pattern, but are very similar to applied load pattern for design.
- In the final step by using the obtained load pattern by the second series of NLTHA and dividing the obtained lateral loads by their code corresponding values it has been tried to introduce a simple formula whose results are some factors, which can be called the "story-dependent response modification factors".

NUMERICAL RESULTS AND DISCUSSION

There are lots of numerical results of which because of lack of space just some samples, related to the 6-story buildings whose natural periods of the first 6 modes in both directions for both cases of basic and modified designs are shown in Table 1, can be presented here. In this table 1st building refers to the relatively stiff building with two braced bays in each direction, and 2nd building refers to the relatively flexible building with just one braced bay in each direction. It is seen that in both cases the modified design, which is based on the new load pattern obtained by NLTHA, the natural periods are a little more than their corresponding values related to the basic design. This means that the modified load pattern leads to relatively more flexible structures.

Table 1- Natural periods (in seconds) of the 6-story designed buildings in both directions

1st BLD. - ST. 2800		2nd BLD. - ST. 2800		1st BLD. - New Pattern		2nd BLD. - New Pattern	
Tx	Ty	Tx	Ty	Tx	Ty	Tx	Ty
0.7114	0.7459	1.1063	0.9949	0.731	0.7534	1.2733	1.3879
0.2159	0.2314	0.3424	0.2968	0.2277	0.2483	0.3879	0.3969
0.1131	0.2014	0.1787	0.1544	0.1248	0.2202	0.2024	0.2042
0.0851	0.1011	0.1237	0.1264	0.091	0.1109	0.1453	0.1498
0.0695	0.0651	0.0964	0.107	0.0738	0.0729	0.1119	0.1417
0.0611	0.0579	0.0803	0.0835	0.0715	0.0648	0.0924	0.1122

Table 1- The maximum shear forces (in kN) of 6-story buildings in both X and Y directions

SOIL TYPE	RECORD/CODE	BLD. 1 X-X	BLD. 1 Y-Y	BLD. 2 X-X	BLD. 2 Y-Y
(I)	1	2830	1090	2270	1010
	2	1090	475	1170	469
	3	2220	627	2030	686
	ST. NO. 2800	1951	2137	1951	2137
(II)	1	713	195	603	174
	2	900	324	711	198
	3	732	277	661	229
	ST. NO. 2800	2364	2415	2364	2415
(III)	1	2620	1110	624	338
	2	2370	1230	1610	712
	3	1130	658	704	368
	ST. NO. 2800	2415	2415	2415	2415
(IV)	1	809	567	716	544
	2	1010	520	898	551
	3	1520	722	894	561
	ST. NO. 2800	2415	2415	2415	2415

As the first sample of numerical results the maximum values of base shear force of the designed 6-story buildings, obtained by NLTHA have been considered as shown in Table 2. In this table numbers 1 to 3 in the second column refer to the three selected applied records for various soil types. It can be seen that although the PGA values of all accelerograms have been the same, the actual base shears are quite different for different earthquakes. The great differences between the shear force values in Y direction are also noticeable. Lower values of shear forces than the code corresponding values for soil types II and IV are particularly notable. Furthermore, it is seen that the base shears values for the relatively flexible buildings are generally less than their corresponding values for the relatively stiff buildings, particularly for soil types III and IV, while based on the code there is not difference in the base shear values of these two types of buildings.

As the second sample of numerical results the lateral load distribution in X direction along the height of the relatively stiff 6-story building at the moments in which the lateral load value at every story reaches its maximum value are shown in figure 1. These results are related to the 1st record on the soil type I. It is seen that none of these distribution can relate to the first mode of the building vibration alone, and they show mostly the contribution of modes 2 and 3.

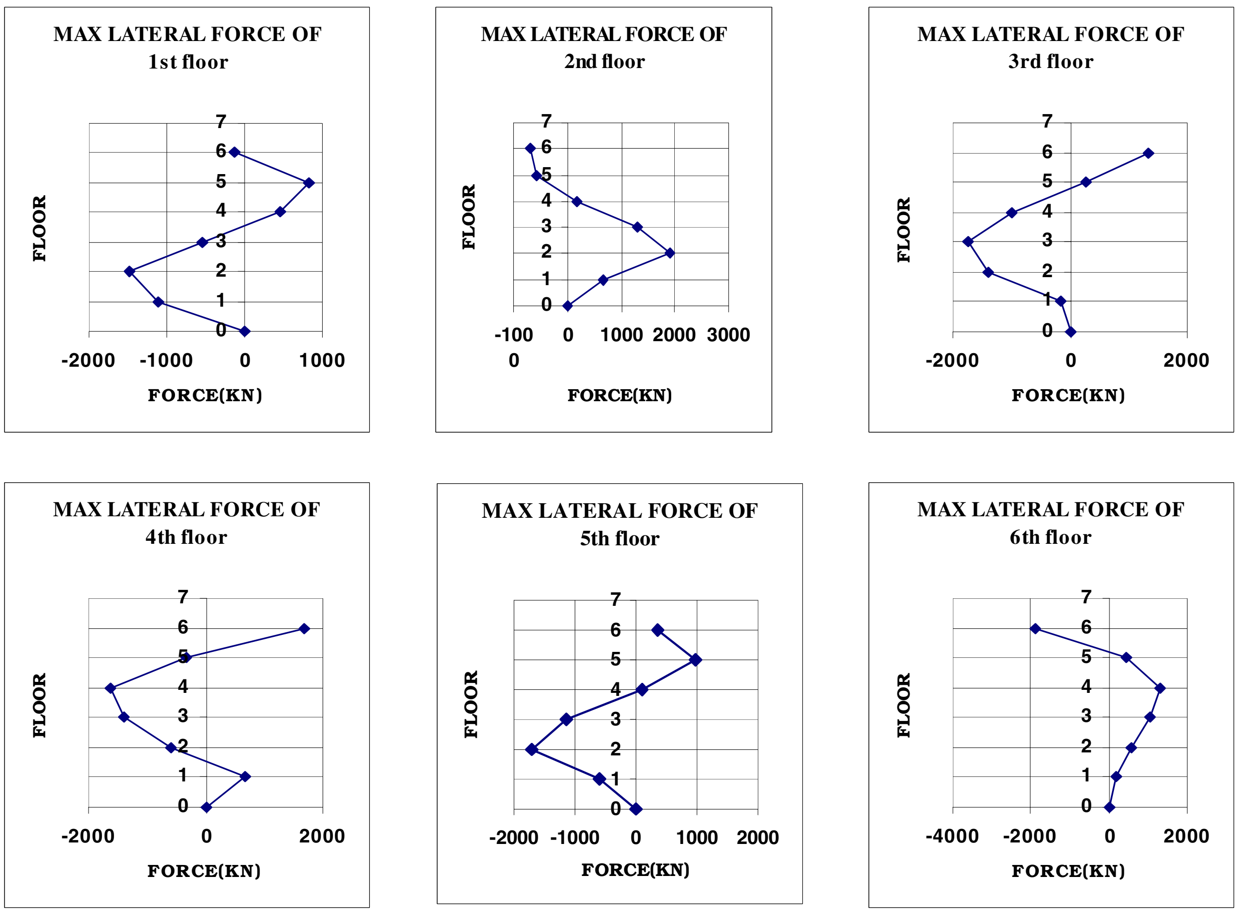

Figure 1- Samples of lateral load distribution along the height of the relatively stiff 6-story building at the moments in which the lateral load value at every story reaches its maximum value

A concise format of the graphs shown in Figure 1 is shown in Figure 2 along with the absolute maxima or the envelope of the maximum lateral loads of the building. It can be seen that the envelope is not similar to the code distribution at all. The main reason behind this fact can be the energy dissipation trend in different stories of the building, as a sample of which is shown in Figure 3. It is seen that in this sample the second floor has dissipated more energy than other stories, even the first floor. The figure shows that the 4th, 5th, and 6th floors have not contributed in energy dissipation so much. This means these floors have behaved almost elastically, while the 1st and 2nd floors have sustained great amount of plastic deformations. Therefore, it can be easily claimed that the code lateral load pattern does not lead to a uniform design of the lateral resisting system of building.

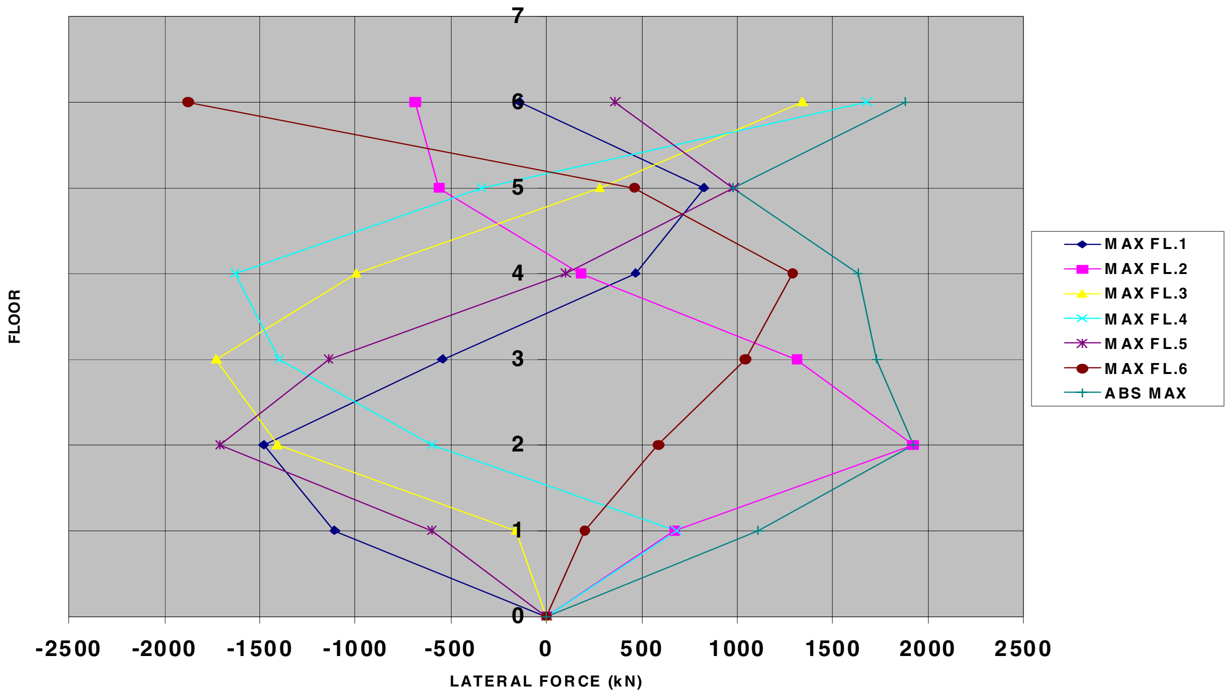

Figure 2- Lateral load distribution at the moments of story maximum forces along with the maximum forces envelope for relatively stiff 6-story building

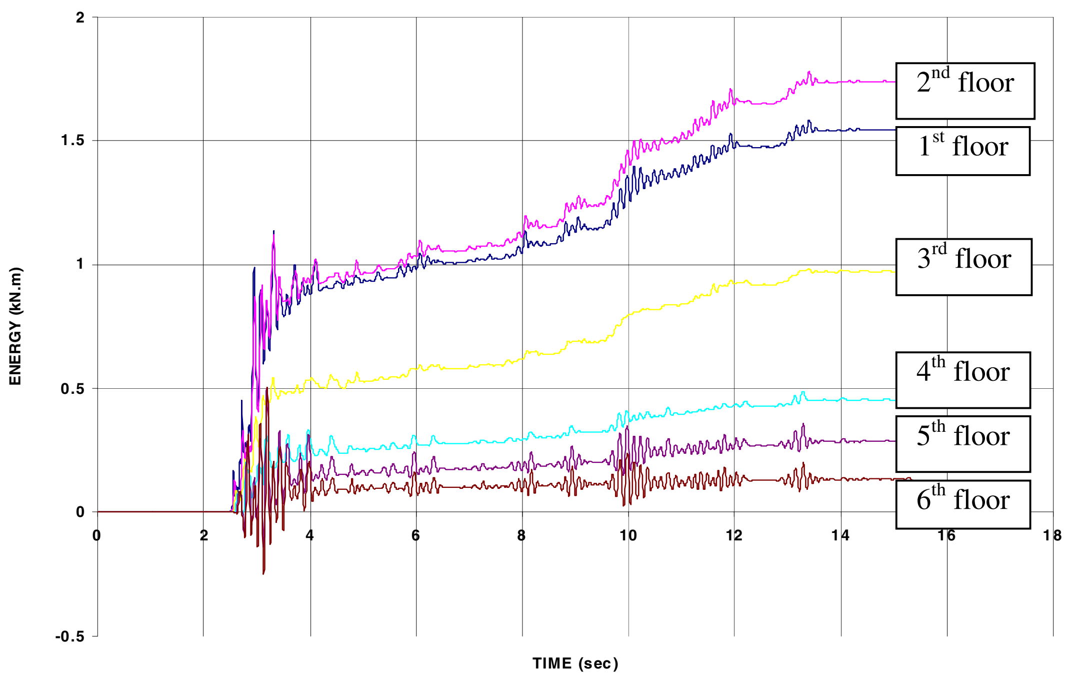

Figure 3- A sample of energy dissipation time history of the relatively stiff 6-story building in X direction

Other samples of concise results are shown in Figures 4 and 5, which shows respectively the maximum lateral forces envelopes for stiff and flexible buildings on soil type I in both X and Y directions.

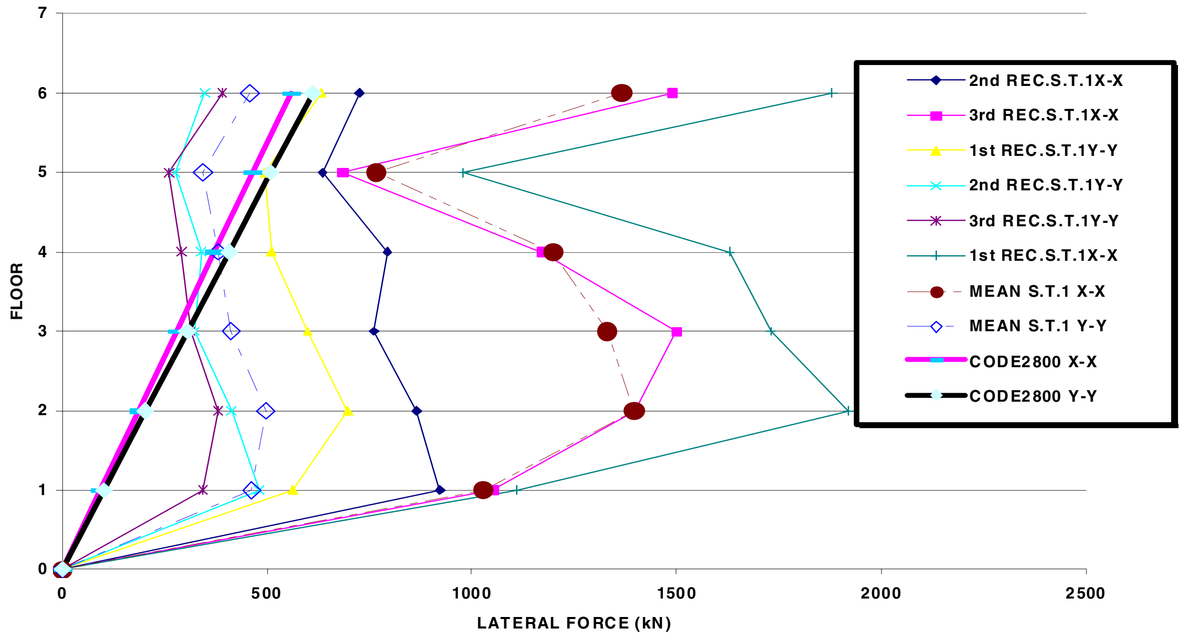

Figure 4- Maximum lateral forces envelopes for relatively stiff building on soil type I in both X and Y directions

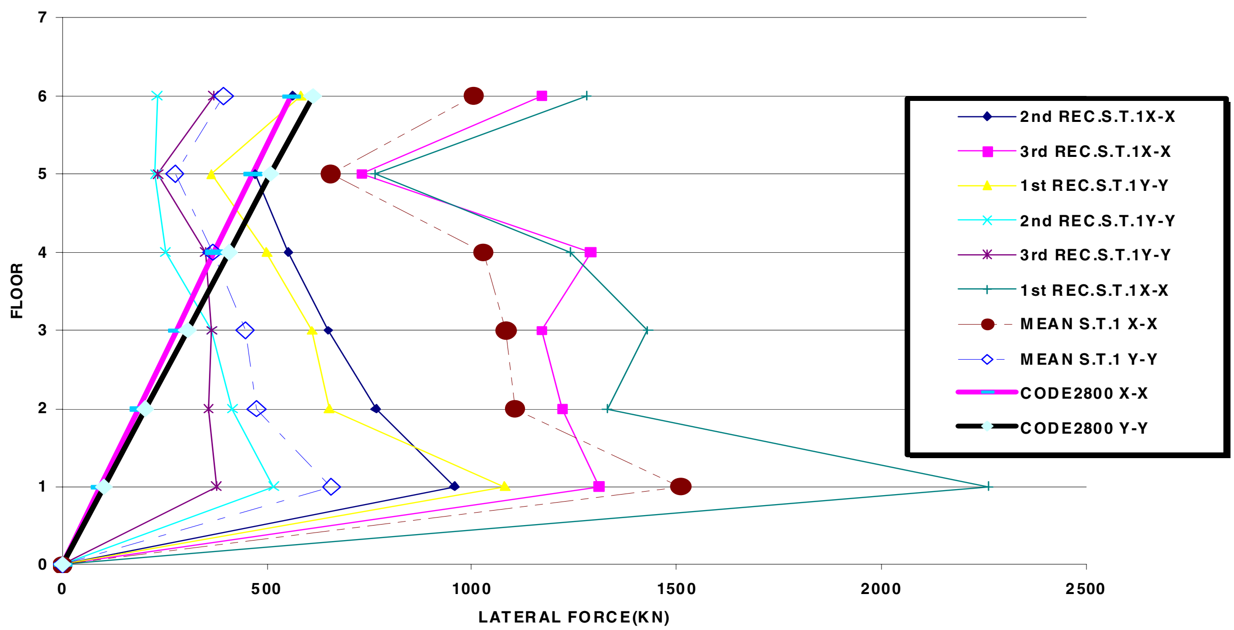

Figure 5- Maximum lateral forces envelopes for relatively flexible building on soil type I in both X and Y directions.

It is seen that although all of them have been normalized to the same PGA value, there are great differences between the lateral forces resulted from different records in both X and Y directions. It is also noticeable that the average envelopes show greater values comparing to the code loading particularly in X direction.

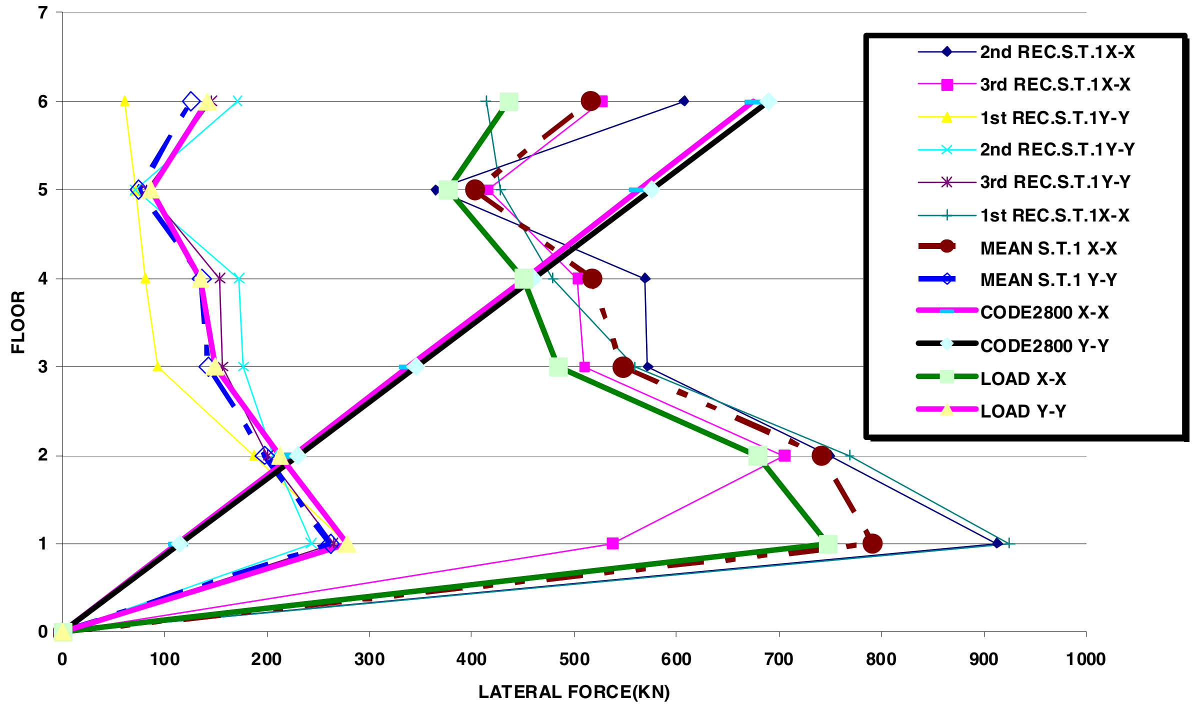

Figure 6- Maximum lateral forces envelopes for redesigned relatively stiff building on soil type II in both X and Y directions

As mentioned before, by using the mean envelope of maximum lateral loads as the new loading patterns in each case the buildings have been redesigned, and to see how close the actual lateral loads are with the new loading patterns, another series of NLTHA have been performed. A sample of results of this series of analyses is shown in Figure 6. It is seen in this figure that because of using the new load pattern the obtained actual loads are much closer to the applied design forces. Therefore, the new load pattern can be considered as a suitable pattern for lateral loading of concentrically braced frames.

INTRODUCING THE "STORY-DEPENDENT RESPONSE MODIFICATION FACTOR"

The verified lateral load pattern obtained by the analyses explained in the previous section can be used for defining some new concept, which can be called the "story-dependent response modification factor". This can be done easily by dividing the new loading values to their corresponding values given by the code. The results of these calculations are some relatively regular curves, of which a sample is shown in Figure 7. These ratios are in fact the desired coefficients which convert the conventional response modification factor to the story-dependent factors. As a first try the very simple formula given by Equation (1) can be introduced for calculating the "story-dependent response modification factors".

$$R_i = R\,(n\,/\,i) \tag{1}$$

In this equation R_i is the response modification factor of the i^{th} story, R is the basic response modification factor given by the code, and n is the total number of stories of the building. Equation (1) shows that for lower stories greater modification factors should be used. This implies that the amount of energy dissipation in lower stories should not be great comparing with the higher stories, and this will lead to more uniform plastic deformations in the building which makes it closer to the optimum design.

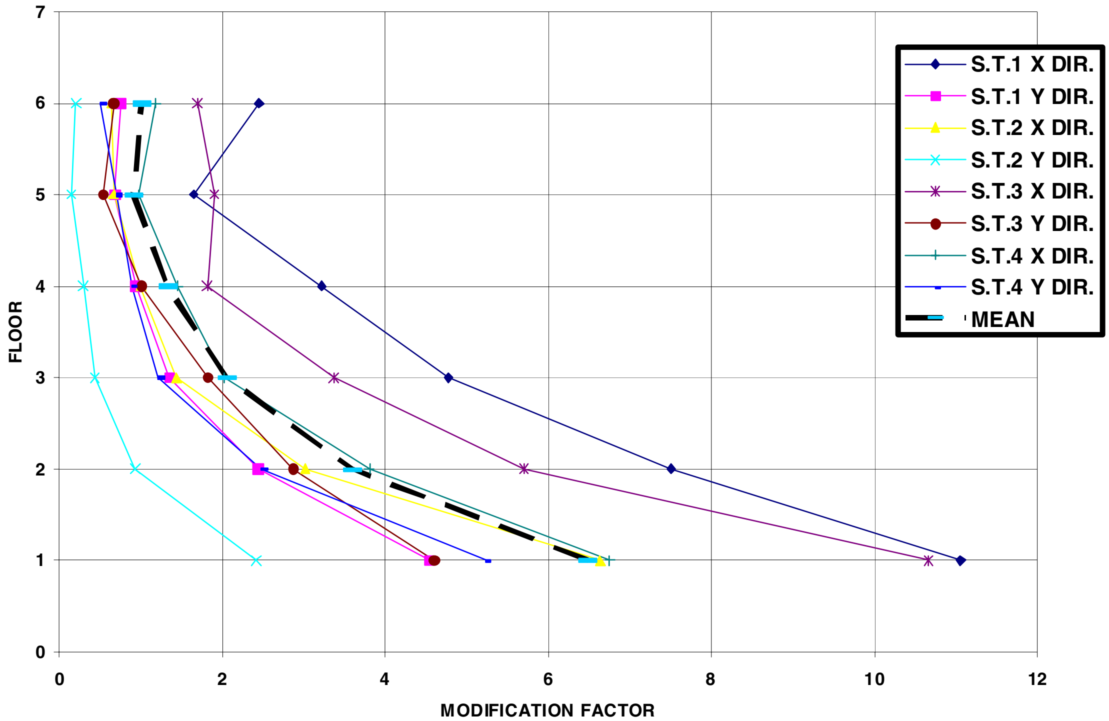

Figure 7- A sample of the ratios between the new loading pattern values to their corresponding values given by the code, which can be called the modification factors

CONCLUSIONS

Based on this study the following conclusions can be stated:

- Calculation the fundamental period of building with concentrically braced frames (CBFs) just base on the building geometry, as code suggests, is not appropriate, and a better rule which considers the configuration of the braced bays should be employed.
- The code suggested based shear value for buildings with CBFs seems not to be appropriate as there is great difference between this value and the actual values obtained by NLTHA.
- The value of response modification factor (*R*) should also be dependent on the configuration of bracings.
- Using a single value *R* for the whole system is not sufficient and a story-dependent response modification factor should be applied. The simple equation given in this paper is a preliminary suggestion in this regard.

Obviously, to be able to propose more general equations for "story-dependent response modification factors" much more research is necessary. Presently, the authors have some projects at hand in this regard.

REFERENCES

1. Nasser Assadi, K. and Hosseini, M., "A Study on the Ductility Factor of Common Steel Buildings in Iran" (in Persian), Proceedings of the 3rd International Conference on Seismology and Earthquake Engineering (SEE-3), Tehran, IRAN, May 1999.

2. Hosseini, M. and Motamedi, M., "A Study on the Distribution of Lateral Seismic Forces in the Height of R/C Buildings by Using Nonlinear Dynamic Analysis" (in Persian), Proceedings of the 1st Conference of Iranian Society of Civil Engineers (ISCE), Tehran, IRAN, October 1999.
3. Hosseini, M. and Yaghoobi Vayeghan F., "Design Verification of an Existing 8-Story Irregular Steel Building by 3-D Dynamic and Push-over Analyses", Proceedings of the 12th World Conference on Earthquake Engineering (12WCEE), NEW ZEALAND, January-February 2000.
4. Hosseini, M. and Firoozi Nezamabadi, M., "Seismic Vulnerability Assessment for Selected Steel Building by Use of Visual Inspection and Nonlinear Dynamic and Pushover Analyses" (in Persian), Proceedings of the 5th International Conference on Civil Engineering, Mashhad, IRAN, May 2000.
5. Yazsinejad, Mohammad M., "Variations of Response Modification Factors Along the Height of Common Steel Buildings" (in Persian), Masters Thesis submitted to the Science and Research Branch of the Islamic Azad University, Tehran, Iran, 2001.