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DESIGN VERIFICATION OF STEEL BUILDINGS WITH ECCENTRICALLY BRACED FRAMES BY USING PUSHOVER ANALYSES

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ABSTRACT

Almost in all seismic design codes, the “response modification factor” appears in the calculation of the building total seismic shear force rather than the lateral load distribution calculations. This is the case while the latter is believed to be more dependent on the plastic behavior of the system in its various story levels. It has been proven that this plastic behavior is dependent on not only the height of the building, but also on the height to width ratio and the number as well as the location of braced bays, or more generally, the configuration of the bracing system in the building. To verify the design of Eccentrically Braced Frames (EBFs) by using the response modification factor concept, some sets of buildings with EBFs have been designed and studied by nonlinear pushover analyses. The buildings include 6 various ones varying from 6 up to 11 stories. For pushover analyses the target displacement method has been used. Based on the push-over analyses the formation of plastic hinges has been studied. The results show that for a building with more than 9 stories the allowable stress design leads to excessive lateral drifts, and if the drift values are to be limited in the allowed range of the code, the design will not be economical. It seems that the code provisions for seismic design of EBFs, particularly using a single value response modification factor, do not guarantee the formation of plastic hinges in link beams. Furthermore, the EBFs, designed by the EBF provisions of the UBC-97, do not show necessarily the expected ductility. It can be claimed that the code provisions in this case need some modifications.

Introduction

Eccentrically Braced Frames (EBFs) as a seismic resisting system were introduced to the structural engineering field in 70s by Popov, who claimed that EBFs are very stiff and can easily satisfy story drift limitations, and they can be designed to provide excellent inelastic behavior and energy dissipation characteristics. He also claimed that EBFs appear to be very economical structures, indicating savings for some framing arrangements on the order of 30% in weight of steel over un-braced frames (Popov and Roeder 1978). A few years later a plastic design method of EBFs was proposed (Kasai 1983). Malley and Popov (1983) also presented the design considerations for shear links in EBFs based on experimental and analytical studies, which was modified later to be Chapter 11 of the "ASCE Manual on Beam-to-Column Building Connections" (Popov and Malley 1983). After that the basic characteristics and the available methods for seismic design of EBFs were also described (Kasi and Popov 1984; Popov et al. 1987). Also the capacity design procedures for ductile eccentrically braced steel frames were also

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presented (Clifton 1987; Engelhardt and Popov 1989). Clifton discussed the need to increase member sizes to control lateral drift.

The possibility and the advantages of using EBFs observing the Italian and European codes were also studied (Scibilia 1990). Popov and his colleagues (1992) reported that EBFs designed improperly can develop undesirable large inelastic link deformations where energy is dissipated at only a few floors. They claimed that the mentioned unsatisfactory behavior can be caused by ill proportioning of the links, and proposed a methodology for optimum EBF link design. A guide was also developed to assist practicing engineers in the design of eccentrically braced frames (Ishler 1992). However, the studies continued and a year later the relations between link length design and EBF seismic performance were studied (Kasai and Goyal 1993). The continued studies resulted in a revised seismic design practice for EBFs based on the 1994 UBC (Becker and Ishler 1996). Also a step-by-step seismic design method for EBFs was proposed including link sizing to avoid concentration of link rotation and drift at a particular story, beam and brace sizing considering special moment redistribution, and column sizing considering the effect of moment (Kasai, Kazuhiko; Han, Xueming, 1997). The development of an inelastic two-dimensional analytical model for the 3D EBFs was also described (Qi et al. 1997). In spite of revision of design procedures it was shown again that for a number of ground motions, for example those corresponding to Zone 5 of Canada, forces and deformations developed in the upper storeys of the frames exceeded those anticipated in the design process, and a modification to the design process to overcome this deficiency was proposed, which uses an iterative process incorporating the evaluation of dynamic response in the selection of members (Koboevic and Redwood 1997).

Design of a 3-D EBF system for seismic loading of a building in Taiwan, with moderate to high seismicity has been performed as well (Qi et al. 1998). They designed a thirteen-story office building by employing three-dimensional eccentrically braced space frames as the primary earthquake load-resisting system. They tried to follow the strong-column-weak-girder principle by combining the floor truss as girders with strong columns. In their work the eccentrically braced frames and moment-resisting frames were designed and detailed following the requirements prescribed in the Uniform Building Code. In order to gain insight into the structural behavior and to verify the post-yielding performance of the structure under earthquake loading, Qi and his colleagues conducted a series of nonlinear pseudo-static pushover and nonlinear dynamic response history analyses, and claimed that the structure has adequate stiffness and strength to resist strong earthquakes.

Interaction of Eurocode 8 and Eurocode 3 in EBFs design was also discussed (Causevic 1998). An iterative design procedure approach to seismic design of EBFs was presented as well (Koboevic and Redwood, 1999). AISC-LRFD design and optimization of EBFs was also studied (Home et al. 2001). Recently, Shear-moment interaction in plastic design of EBFs was studied as well (Mastrandrea et al. 2003). Finally, very recently again capacity design of EBFs was taken into consideration (Richards and Uang 2006). Based on their study brace and column sizes in eccentrically braced frames (EBFs) depend on the capacity of the links. As braces are designed for the maximum deliverable forces from the adjacent link, column demands, however, stem from links in all the stories above. The extent of simultaneous yielding and hardening of links determines the maximum axial force that can develop in the columns. They proposed some new guidelines for determining appropriate loads for EBF columns.

It is seen that in spite of several studies and analytical as well as experimental investigations on EBFs, still there are some uncertainties about their proper seismic behavior and the adequacy of codes' provisions, particularly the use of "response modification factor". This paper presents the results of a study conducted to verify the design of EBFs by using the response modification factor concept. For this purpose some sets of buildings have been designed and studied by nonlinear pushover analyses. The buildings include 6 various ones varying from 6 up to 11 stories. For pushover analyses the target displacement method has been used. Based on the push-over analyses the formation of plastic hinges has been studied. The results of this study are presented in the following sections of the paper.

The Studied Buildings

The buildings considered for the study include six buildings having 6 to 11 stories with similar plans and similar locations of braced bays, as shown in Figs. 1 and 2, and their structural system consist of CBFs in one direction and EBFs in the other, of which just the EBFs have been studied. The steel material has been assumed to have modulus of elasticity of 2.1E6 kgf/cm^2, and yielding and ultimate stresses of respectively 2400 kgf/cm^2 and 4000 kgf/cm^2. The floor systems have been considered as R/C joists with hollow blocks. The story height has been considered as 3.2 m in all cases.

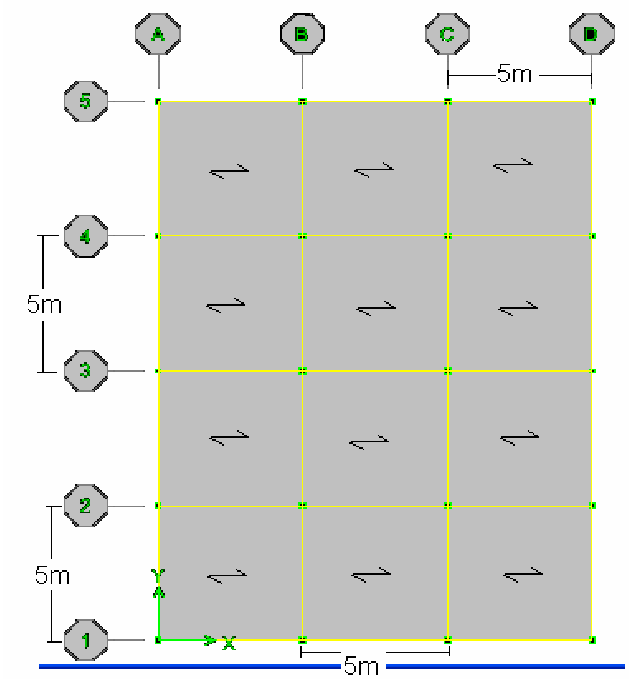

Figure 1. Plan of the studied buildings.

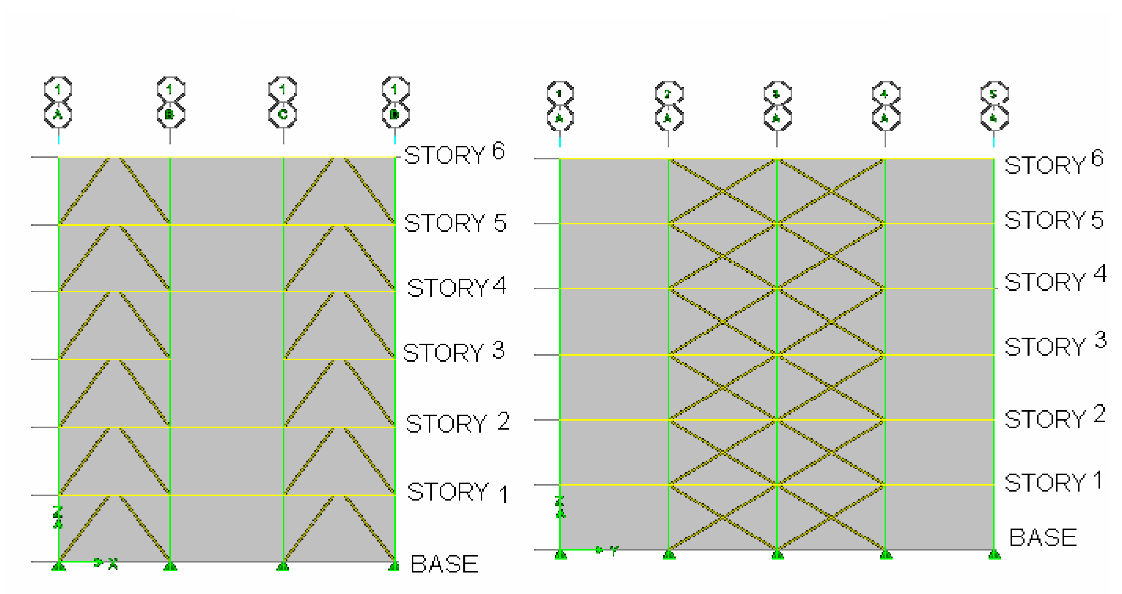

Figure 2. Elevation views of the 6-story studied building.

The amount of dead and live loads for the initial design of frames have been assumed as 380 kgf/m^2 and 200 kgf/m^2 (150 kgf/m^2 for roofs) respectively. Also to account of the weight of external walls a dead load of 800 kgf/m has been assumed for the outer girders in floors and 200 kgf/m in roofs. Seismic loads calculations have been performed based on the Iranian Standard No. 2800 (the equivalent static force procedure), assuming the importance factor of 1.0 for buildings, soil type II for their sites, and design acceleration of 0.35g for the region. Based on this Standard the values of response modification factors in CBFs and EBFs are respectively 6.0 and 7.0. The steel profiles for beams and columns have been IPE and Box Section respectively. The EBFs have been designed by the EBF provisions of the UBC- 97.

Numerical Results

It this section at first the basic results of the study with regard to the code drift limitation provision are presented and then the results of push-over analysis are presented.

Basic Results

The design of EBFs has been done once without consideration of drift ratio limitation of the code and once with that limitation, and the results have been compared. In the first case the design criteria has been just the stress ratios in structural members. The out-puts which have been considered in design of buildings include: the fundamental period, story drift, and stress values. Table 1 shows the maximum values of drift ratios for the EBFs designed without consideration of drift limitation.

Table 1. The maximum values of drift ratios for the EBFs designed without considering the drift limitation.

Story No.	6-Story Building	7-Story Building	8-Story Building	9-Story Building	10-Story Building	11-Story Building
11	-	-	-	-	-	0.00559
10	-	-	-	-	0.005204	0.005795
9	-	-	-	0.004641	0.005249	0.005555
8	-	-	0.003980	0.004677	0.005071	0.005225
7	-	0.003400	0.004009	0.004346	0.004618	0.004859
6	0.002833	0.003413	0.003713	0.003892	0.004150	0.004366
5	0.002840	0.002948	0.003288	0.003517	0.003726	0.003643
4	0.002778	0.002640	0.002876	0.003073	0.002922	0.003122
3	0.002150	0.002394	0.002426	0.002247	0.002384	0.002578
2	0.001664	0.001798	0.001940	0.001680	0.001815	0.001924
1	0.001238	0.001318	0.000994	0.001140	0.001221	0.001207

It should be mentioned that the value of allowable drift ratio (0.03 / R) for these frames is 0.004286. It is seen in Table 1 that for frames up to 8 stories the drift ratios in all stories are less than the allowable value, while in frames with more stories these values exceed the allowable value in higher stories, and as the number of stories increases the number of stories in which the drift ratio exceeds the allowable value increases as well. Furthermore, it can be realized from Table 1 that the values of drift ratio in lower stories are much lower that the values in upper stories of the buildings, while the code provisions imply that the drift ratios are almost the same for various stories in the case of low-rise buildings which deform in shear mode. To have a better insight into the variation of drift ratios along the height of buildings their variation has been shown in Fig. 3.

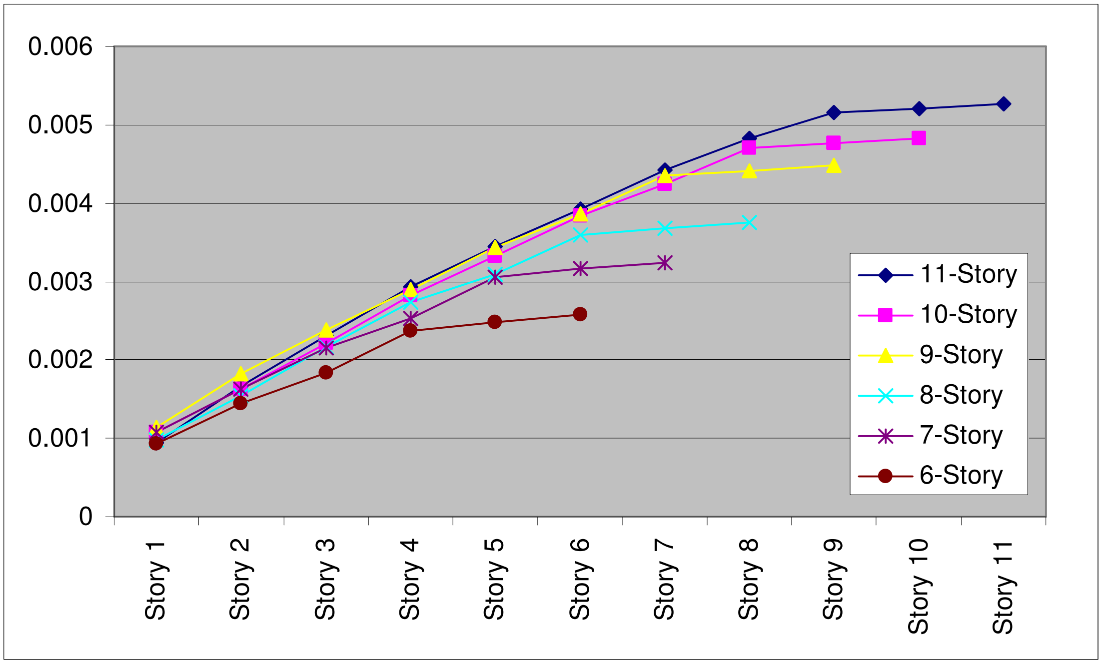

Figure 3. Variation of drift ratios along the height of buildings with various numbers of stories.

It can be seen in Fig. 3 that almost in all cases the values of drift ratios in the upper three stories of buildings are almost the same, and in lower stories their variation is almost linear. By this observation It can be concluded that the rate of stiffness reduction of stories along the height of the buildings (except for the few higher stories) is higher than the rate of lateral loads increase obtained by code formulas. In other words, using a single value of "response modification factor" (R) for design does not lead to a uniform stiffness in various stories of buildings. Obviously, to meet the code requirement for the drift ratios in the upper stories of 9-, 10-, and 11-story buildings it is necessary to use larger sections for beams, columns, and/or bracing elements in these buildings, and this increases the total amount of the consumed steel profiles in corresponding frames. To find out the differences in the weight of consumed steel profiles in the two cases their total weights for the cases of 9-, 10-, and 11-story buildings are shown in Table 2.

Table 2. The total weight of consumed steel profiles in 9-, 10-, and 11-story frames in both cases of design without consideration of drift ratio limitation and with that limitation.

Design Case	No. of Stories		
	9	10	11
Without drift ratio limitation	20906	25018	30025
With drift ratio limitation	22465	30254	39223

It can be found out from Table 2 that the steel weight is increased by 7.5%, 20.9%, and 30.6% for the cases of 9-, 10-, and 11-story frames, respectively, if the drift ratio limitation provision is applied. This implies that using EBFs as the lateral resisting system for buildings with more than 8 stories is not economical in high seismic areas like Iran, and the higher buildings will be more uneconomical.

Results of Push-Over Analyses

Push-over analyses have been performed by considering the target displacements of 0.02 of the buildings heights in all cases. Before each case of push-over analysis a static analysis has been done for the vertical load distribution pattern used in the initial design of each frame. It has been tried to keep the stability of the frame till reaching the target displacement in each case by changing the structural members' cross-sectional properties as necessary, and checking the amount of dissipated energy in various elements of the structures. Push-over analyses showed that in all cases the initial design of EBFs does not provide the required conditions for frames to reach the target displacement. Therefore, again the values of structural members' cross-sectional properties were increased to postpone the buckling of critical members and increase the energy dissipation capacity of plastic hinges. Obviously, this again caused an increase in the amount of the consumed steel profiles in corresponding frames as shown in Table 3.

Table 3. The total weight of consumed steel profiles in building frames in three design cases.

No. of Stories	Stress-Based Design	Drift-Based Design	Energy-Based Design
6	10076	10076	12134
7	14167	14167	15192
8	16933	16933	19512
9	20906	22465	22731
10	25018	30254	28064
11	30025	39223	33816

It is seen in Table 3 that although the energy-based design for reaching the target displacement leads to increase in the weight of frames for all buildings, the amount of increase for higher buildings in not as much as the increase due to drift-based design in these buildings. The amount of increase due to the energy-based design, in which the criteria adopted for modifying the members has been choosing a member with higher plastic moment capacity, varies from around 20% for 6-story building to around 12% for 11-story buildings. It is seen that the weight increase in this case decreases as the number of stories increases, while the increase due to drift-based design increases as the number of buildings stories increases as observed in the previous section.

The other point which is worth mentioning with regard to push-over analyses is the trend of plastic hinge formation in frames. Fig. 4 shows the Locations and types of plastic hinges in 6- and 7-story frames in the last step of their push-over analyses. Fig. 5 shows the same for 8- and 9-story frames, and Fig. 6 for 10- and 11-story frames. In these figures the colors used for plastic hinges show how much percent of the maximum plastic capacity of the sections has been employed in each case; where light blue, dark blue, green, yellow, and pink means respectively less than 30%, between 30% and 50%, between 50% and 70%, between 70% and 90%, and more than 90%.

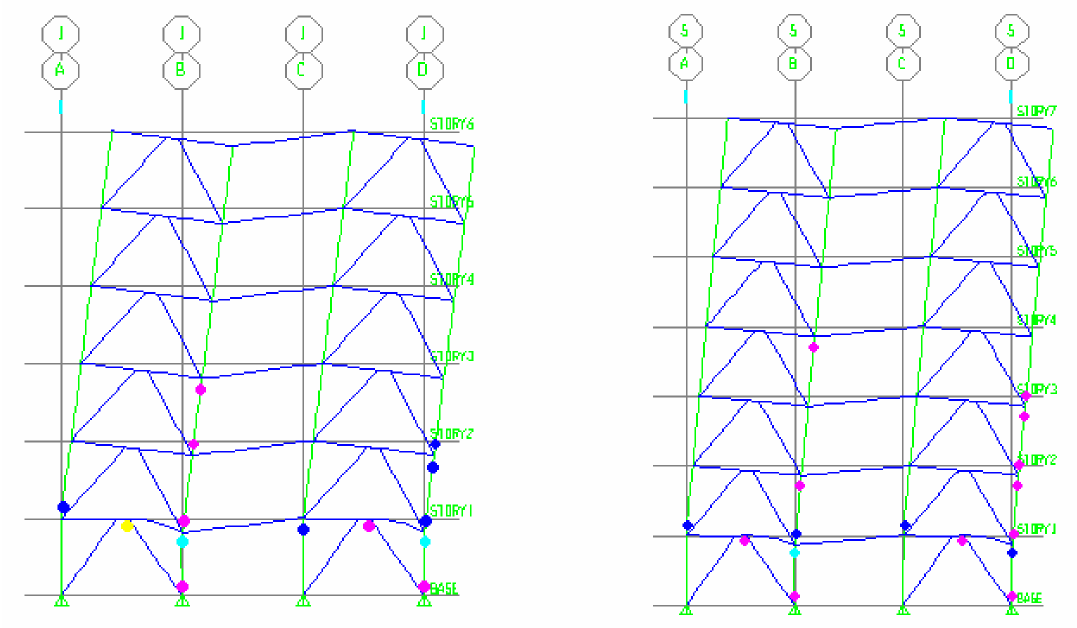

Figure 4. Locations and types of plastic hinges in 6- and 7-story frames in the last step of push-over analysis.

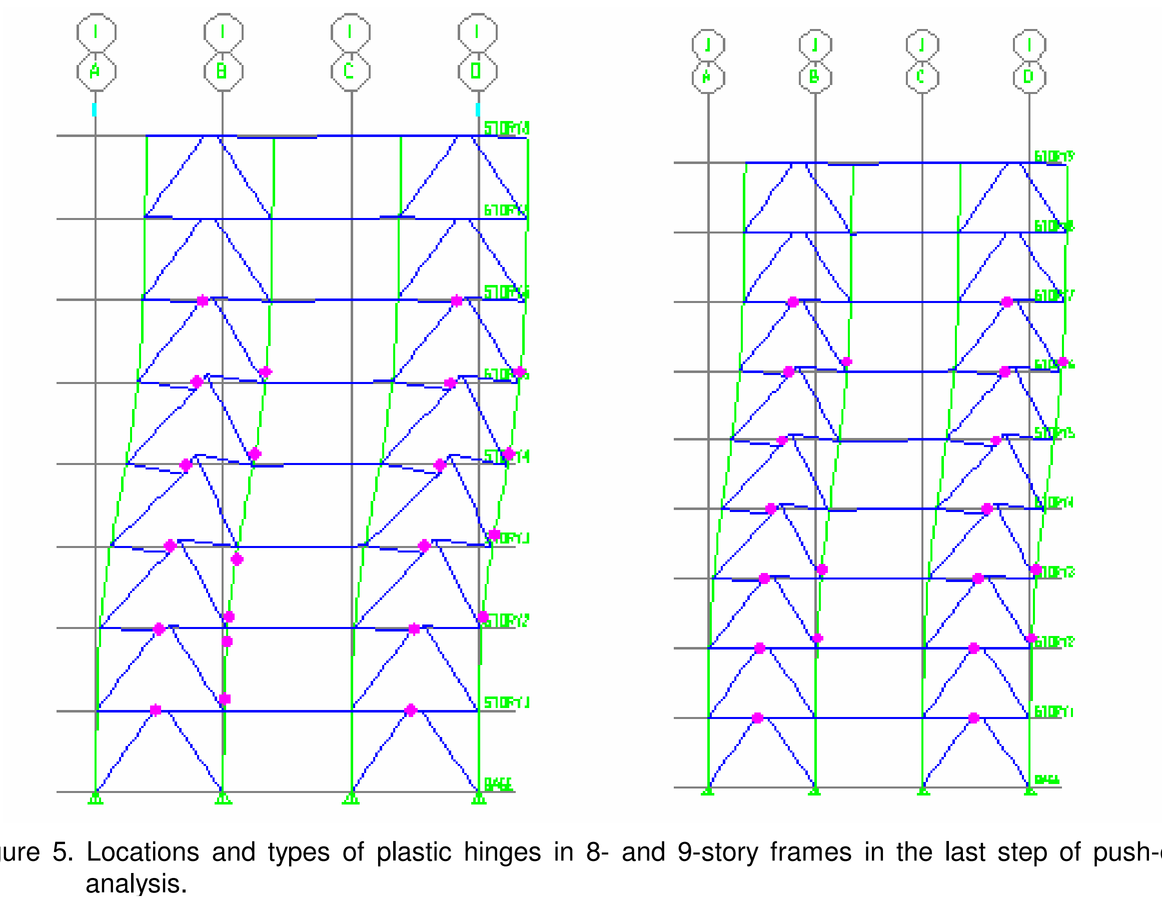

Figure 5. Locations and types of plastic hinges in 8- and 9-story frames in the last step of push-over analysis.

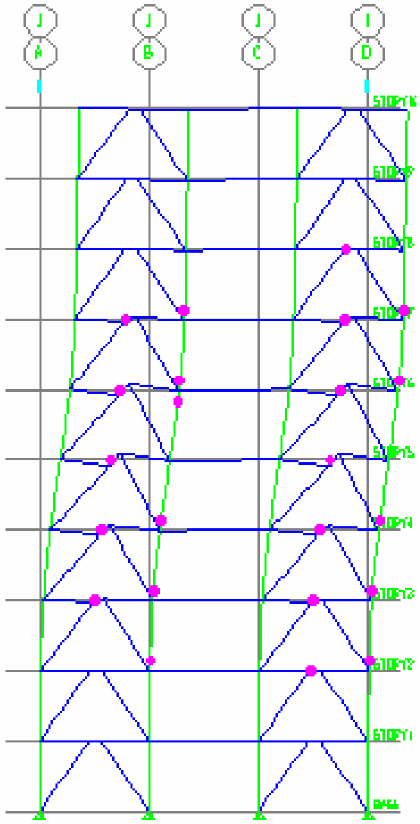
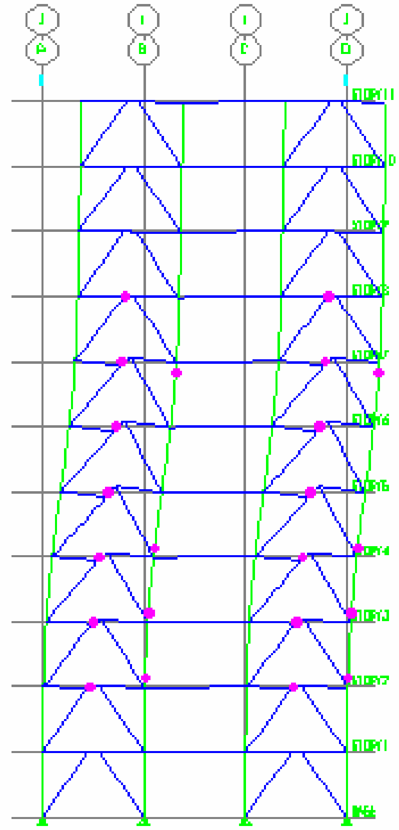

Figure 6. Locations and types of plastic hinges in 10- and 11-story frames in the last step of push-over analysis.

The extensive plastic hinging which have formed in some columns is because of applying the full capacity design procedure as prescribed in AISC provisions or as described in the work of Richards and Uang (2006). It is seen in Figs. 4 to 6 that in the case of 6- and 7-story frames the plastic hinges have been formed mostly in columns and bracing elements, while in the case of 10- and 11-srtory frames they have been formed mainly in link beams. This means that the philosophy behind the use of EBFs does not completely come to reality for the case of low-rise buildings, and furthermore, that philosophy can come to reality for the case of higher buildings if the drift limitation is not considered in their design.

Conclusions

Based on the numerical results it can be concluded that:

- If stress-based design is used, the drift ratios for low-rise buildings in all stories are less than the allowable value, while in relatively high buildings (9-story or more) these values exceed the allowable value in higher stories, and as the number of stories increases the number of stories in which the drift ratio exceeds the allowable value increases as well.
- With stress-based design the values of drift ratio in lower stories are much lower that the values in upper stories of the buildings, while the code provisions imply that the drift ratios are almost the same for all stories.
- With stress-based design almost in all cases the values of drift ratios in the upper three stories of buildings are almost the same, and in lower stories their variation is almost linear. This may mean that the rate of stiffness reduction of stories along the height of the buildings (except for the few higher stories) is higher than the rate of lateral loads increase obtained by code formulas.

- To meet the code requirement for the drift ratio in the upper stories of relatively high buildings it is necessary to use larger sections for beams, columns, and/or bracing elements in these buildings, and this increases the total amount of the consumed steel profiles in corresponding frames. The value of this increase can reach 30% or more. This implies that using ordinary EBFs as the lateral resisting system for buildings with more than 8 stories is not economical in high seismic areas like Iran, and for higher buildings it will be more uneconomical.
- Push-over analyses showed that in all cases the initial design of EBFs does not provide the required conditions for frames to reach their target displacements. This means that the value of R, recommended by the code for these buildings, has not been appropriate.
- The energy-based design for reaching the target displacement leads to increase in weight of frames for all buildings, the amount of increase for higher buildings in not as much as the increase due to drift-based design.
- The amount of increase due to the energy-based design varies from around 20% for low-rise buildings to around 12% for relatively high buildings. In fact, the weight increase in this case decreases as the number of stories increases, while the increase due to drift-based design increases as the number of buildings stories increases.
- In the case of low-rise buildings the plastic hinges are formed mostly in columns and bracing elements, while in the case of relatively high buildings they are formed mainly in link beams. This means that the philosophy behind the use of EBFs does not completely come to reality for the case of low-rise buildings, and furthermore, that philosophy can come to reality for the case of relatively high buildings if the drift limitation is not considered in their design.

Based on the above conclusions it can be claimed that the UBC-97 provisions in the case of EBFs, particularly using a single value response modification factor (R), need some modifications.

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