Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2017) 4088-4098

ECCOMAS
Proceedia

COMPDYN 2017
6th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15–17 June 2017

SEISMIC PROGRESSIVE COLLAPSE OF MRF-EBF DUAL STEEL SYSTEMS

Alireza Faroughi[1], **Abdolreza S.Moghadam**[2], **Mahmood Hosseini**[3]

1 Assistant Professor
Department of Civil Engineering, College of Engineering, East Tehran Branch, Islamic Azad University, Tehran, Iran
e-mail: faroughi@iauet.ac.ir
2 Associate Professor
International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran
e-mail: moghadam@iiees.ac.ir
3 Associate Professor
International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran
e-mail: hosseini@iiees.ac.ir

Keywords: Columns' Failure, Progressive Collapse, Eccentric Steel Bracing, Seismic.

Abstract.*In structural engineering, it is always attempted to predict a set of events which greatly contribute to lifecycle of the structure. These factors must be considered by design engineer and ensure resistance of the structure against those events. Seismic progressive collapse is one of the causes of serious damages and losses which may happen in events such as earthquakes. In this paper, it is intended to introduce more appropriate brace configuration based on an upper-bound estimation of the probability of seismic progressive collapse in MRF-EBF dual systems designed based on the 3rd edition of Iranian Seismic Code. In fact, in that standard there are only provisions for capacity of moment frames and no detail is given for bracings elements. The study has been concentrated on low to mid-rise buildings with residential and office usage. Results show that for 5-story buildings the best number of bracing bays is as many as 10-20% of the number of building's circumferential bays, and for 8-story buildings best number of bracing bays is as many as 20-30% of the number of building's circumferential bays. Results also show that regular bracing in internal bays are more appropriate for more resistance of the structure against progressive collapse.*

doi: 10.7712/120117.5707.18345

1. INTRODUCTION

Advances in methods of analysis and design of structures and development of performance-based concepts together with the observation of collapse in many structures as a result of natural and non-natural events, directed researchers toward investigation of structures in conditions far beyond the service conditions which is considered in conventional designs. In recent years, natural disasters such as Northridge earthquake (1994) and Kobe (1995), as well as non-natural events such as explosion of Mora federal building (1995), collapse of the Ronan famous building (1996) and terrorist attacks to buildings of world trade center (2001) resulted in structural failure of members progressively and finally, progressive collapse of the structure whose consequence was a major life loss and considerable economic losses. For this reason, progressive collapse has been taken into consideration during past decade. Progressive collapse is the failure of overall structure or an extensive part of it as a result of local events such as failure of one or more load bearing members and failure of adjacent members to redistribute the excessive loads through a path which can maintain the resistance and overall integrity. According to the definition of ASCE-7, progressive collapse is diffusion of the initial failure from one element into another whose consequence is the overall collapse of the whole structure or a large portion of it. More recently some researchers have worked on progressive collapse of steel frames using pushdown analysis [1].

Braced frames with the ability of eccentricity were introduced first in 1978 by Popov et.al. This system was initially designed for confronting lateral loading and its effect on large deformation and relative displacement particularly in high-rise buildings. Therefore, these researches presented eccentric bracing frame in regulations as a new system resisting against earthquake. According to extensive researches performed in this regard, systems of eccentric bracing have the capacity to combine high stiffness in inelastic range. Eccentric systems are in fact combined systems and the purpose of making eccentric bracing frames is to direct shear yield in a small portion of the main beam which is referred to as linking beam [2 , 3, 4, 5].

Although several researchers have worked on Seismic design of MRF-EBF dual systems [6], very few studies have been conducted with regard to their progressive collapse. This paper tries to suggest a rational approach for identifying the columns that should be removed for "seismic progressive collapse" studies in MRF-EBF dual systems, and then tries to suggest the most appropriate placement of the eccentrically braced bays to achieve the minimum potential of seismic progressive collapse. The columns have been selected to represent the most vulnerable columns to seismic damage, as identified by pushover analysis. Thus, instead of removing certain preset and predefined columns, the potential of progressive collapse at the end of earthquake is evaluated by removing the seismically damaged column. Details of the study are presented in the following sections.

2. PROBABILITY OF SEISMIC PROGRESSIVE COLLAPSE

Investigation of the progressive collapse is performed with elimination of some columns due to extensive loads such as explosion and then assessing the probability of progressive failure of other main structural elements caused by the intensification of gravity load of floors. However, in this paper, the issue of progressive collapse has been looked at from a different viewpoint. In this viewpoint it is assumed that the primary members which lose their load bearing capability (have passed point E in the performance curve) due to earthquake are removed from the structure, and then the structure is analyzed under the amplified effect of gravity loads (to account for the dynamic effect of gravity loads due to the sudden elimination of vertical load bearing elements). In this way an upper-bound estimation of the progressive collapse probability of the building's structure is obtained.

For this purpose, first, structure is analyzed under the gravity and lateral loads using pushover analysis method and in this stage, hinges with different levels of performance will be created, which can represent the probability of initiation of the seismic progressive collapse in the structure. On this basis some columns are selected for removal and stating the progressive collapse analysis. These are often at the lowest floor and connected to a braced and moment frame. Finally, it is investigated whether structure is able to bear gravity loads resulted from removal of the column and transfer loads to its other members.

This is a new trend in the field of progressive collapse in which removal of column occurs as a result of earthquake and analysis is performed on a structure which is damaged by earthquake which is known as 'seismic progressive collapse'. This issue is studied in models which have most hinges in performance level near collapse. Therefore, two models of low-rise five floor buildings with distribution of damage in columns adjacent to the bracing bay (models 7 and 9 in Figure 1) and four models of mid-rise buildings with distribution of damage in bays of moment frame as well as bracing bays (models 20, 22, 23 and 24 in Figure 1) will be provided.

3. PROCESS OF DESIGN RESISTANT AGAINST PROGRESSIVE COLLAPSE

3.1. Instructions of the US department of defense (DoD)

US Department of Defense published a document entitled "unified facilities criteria (UFC)" [7] which must take the progressive collapse for new buildings which have three floors or more. DoD instruction can be applied for reinforced concrete, steel, wooden and cold-rolled steel buildings.

3.1.1. Alternative logical method

In this method, structure must be able to bridge damaged portions. Analyses must be applied for linear and nonlinear statistical analyses by taking into account the appropriate portion for removal of columns and load bearing walls according to UFC4-023, 2009 [7] and ASCE7, 2010 [8] as well as maximum emergency load with load multipliers. These criteria must be applicable in alternative logical analysis.

4. MAIN CHALLENGES

Many challenges facing the system of bracing in dual systems for this research is conceivable and number and location of braces and its effect on seismic capacity and probability of the progressive collapse will be determined as a main challenge of selection.

5. RESEARCH METHODOLOGY

To start, the first step is to select the models and their characteristics for investigation and performing intended analyses of this work. For this end and understanding the requirements of the model, the method and recommendations of FEMA P 695 – 2009 [9] will be used.

5.1. Statistical population and volume of the sample

a. Statistical population: common residential and official buildings
b. Method of sampling: using professional software
c. Volume of sample: investigation, analysis and design of 28 different plans of the structure with afore mentioned characteristics

d. Method of calculation: based on regulation ASCE 41-13 [10] and instruction of seismic improvement 360 [12] and design using allowable stress and nonlinear analysis (pushover).

5.2. Design considerations

For the purposes of this research, in this stage, certain structural models were selected. At the beginning, with a test pattern of design, number of required bracing bays of the structure was assumed as 10-30% of the building's circumferential bays. Regarding the number of structural models, due to importance of the models, 28 models were considered. Sections used for software modeling were all box section with the size varying from 8-40 according to their function. Beams are mainly I-shaped (From No. 16 to 27). The material used in all cases is steel ST37. (The following models were analyzed as numerated from 1-28 from left to right).

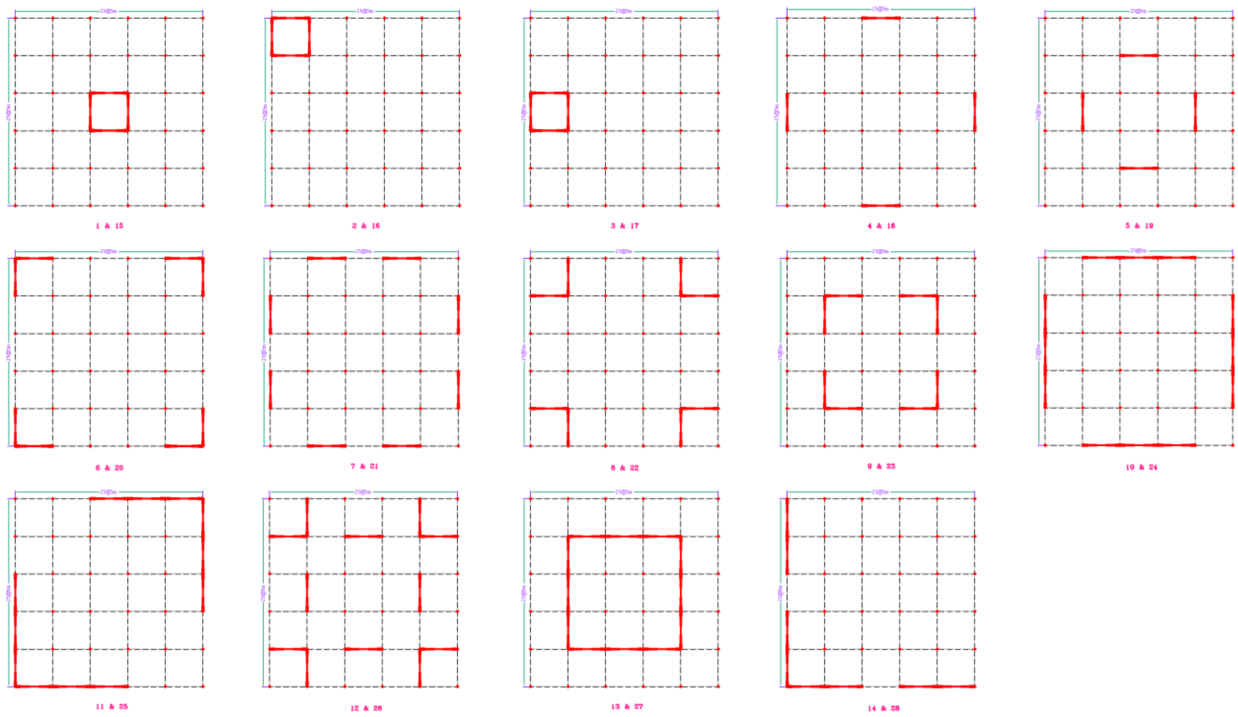

Fig. 1: Bracing patterns considered in the study for two sets of 5- and 8-story buildings

To meet the criteria listed in the AISC 341-10 [11] Section F3, as the length of all link beams are shorter than 1.6Mp/Vp therefore shear mode governs their behavior.

6. MODELS OF DESIGN AND ANALYSIS

6.1. Design of structure

Structural design was performed according to design regulations and 3rd edition of the Iranian Standard No. 2800 [13] of earthquake as linear and elastic with allowable stress method.

6.2. Analysis method: Pushover nonlinear static

Nonlinear models are defined for models which have the highest probability of the nonlinear – inelastic behavior. In this method in order to evaluate a structure, it has been analyzed under the effect of constant loading pattern (statistic) and this analysis carries on till displacement of point of structure (displacement of control point) .This displacement which is evaluated with a specific improvement goal is called target displacement and will be the basis of investigation of structural components so that deformation, rotation and forces of the members will be investigated if they approach such limit (calculation of the required displacement of members is impossible, [14]).

7. ANALYSIS OF RESEARCH FINDINGS

In this section, results of models will be analyzed and divergence of analytical models and loading pattern using redistribution of the secondary stiffness is used.

Here, results will be analyzed and models are classified into two parts; low-rise with 5 floors and mid-rise with 8 floors.

7.1. Classification and analysis of data

Model	No. of floors	No. of CP hinges in braces		No. of CP hinges in beams		No. of CP hinges in columns connected to MR bays		No. of CP hinges in columns connected to BF bays	
		Push Y	Push X	Push Y	Push X	Push Y	Push X	Push Y	Push X
1	5	0	0	8	8	0	0	0	0
2	5	0	0	10	6	0	3	0	0
3	5	0	0	10	10	0	0	0	0
4	5	0	0	10	10	0	0	0	0
5	5	0	0	8	8	0	0	0	0
6	5	0	0	12	12	0	0	4	2
7	5	0	0	12	12	0	0	4	4
8	5	0	0	12	12	0	0	4	3
9	5	0	0	12	12	0	0	4	4
10	5	0	0	12	12	0	0	0	0
11	5	0	0	12	12	0	0	0	0
12	5	0	0	12	12	0	0	0	0
13	5	0	0	12	12	0	0	0	0
14	5	0	0	20	20	0	3	0	0
15	8	0	0	16	16	2	0	0	0
16	8	0	0	16	22	12	43	0	4
17	8	1	0	10	14	1	0	0	0
18	8	0	0	16	14	1	2	0	0
19	8	0	0	14	14	0	0	0	0
20	8	0	0	28	28	2	6	0	0
21	8	0	0	24	24	1	0	0	4
22	8	0	0	24	24	8	4	0	0
23	8	0	0	24	24	0	0	2	2
24	8	0	0	36	36	2	0	3	2
25	8	0	0	30	30	1	1	1	0
26	8	0	0	36	36	0	0	0	0
27	8	0	0	30	30	2	2	0	2
28	8	0	0	32	32	8	3	0	0

Table 1: Characteristics of hinges in collapse of the different structural members

8. ANALYSIS OF THE RESULTS OF PROBABILITY OF PROGRESSIVE COLLAPSE

8.1. Probability of progressive collapse in mid-rise and low-rise models

Model 7: in this model, number of hinges passing from the threshold of collapse as a result of pushover analysis in X-direction and in braced frames will be studied.

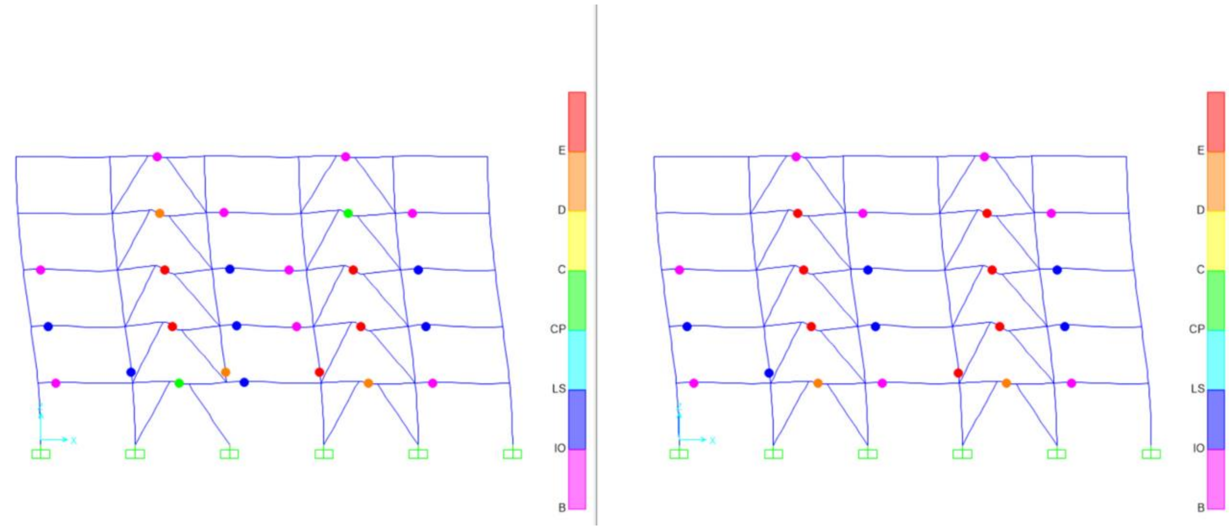

Fig. 2: hinges of pushover analysis (left), removal of column in progressive collapse (right)

After pushover analysis, it is observed that in linking beams connected to bracing bays, shear hinges are formed. Moreover, in other bracings, bending hinges are formed in beams and columns with LS and IO performance levels which can be seen in Fig. 2. After the pushover analysis and removal of column according to Fig. 2, it is observed that the number of hinges formed in linking beams remained constant while some of them pass the CP performance level and in beams bear the bracing span in which no hinge was formed before, two hinges are formed now with IO performance level in 2nd and 3rd floors and bending hinges are formed with LS performance level in 1st floor.

Model 9: in this model, number of hinges passing from the threshold of collapse in pushover analysis in X direction is bracing frame is studied.

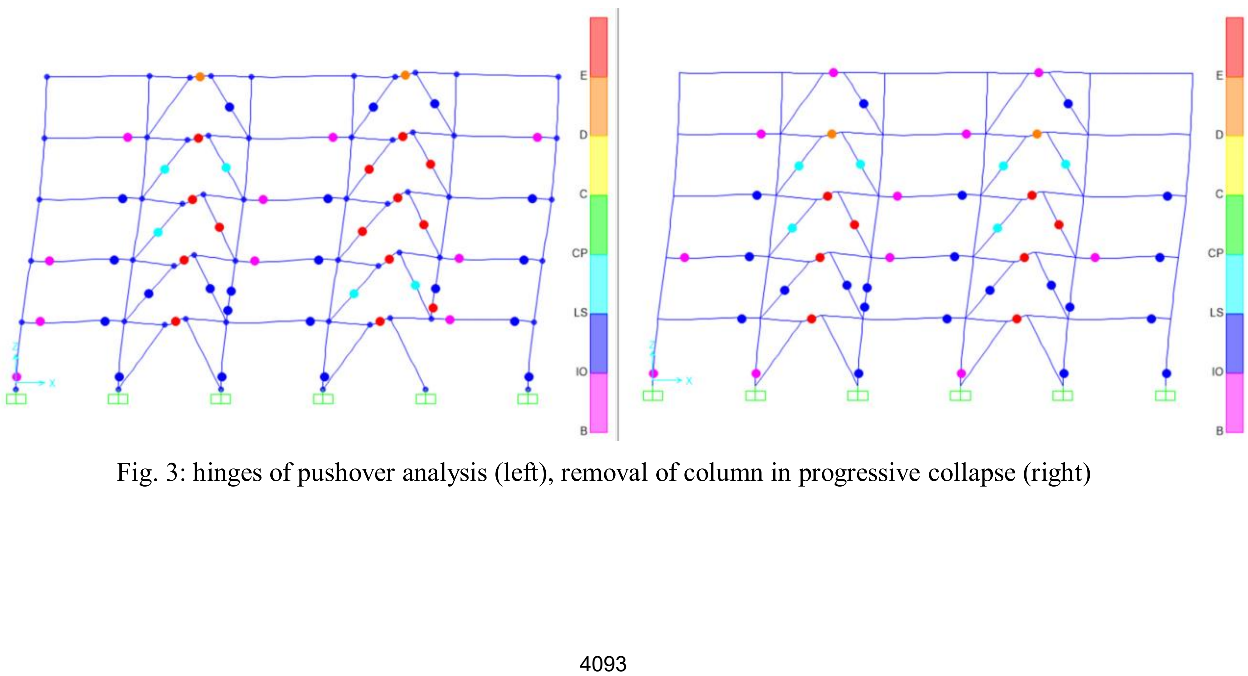

Fig. 3: hinges of pushover analysis (left), removal of column in progressive collapse (right)

After pushover analysis and removal of the intended column (Fig. 3), a number of shear hinges are formed in all floors of the structure and in D-E range. In first stage, hinges are formed only in four floors. Furthermore, some of the bracing members in which CP hinges are formed pass now the CP level and are in critical limit. In what follows, it can be seen that column of the top floor which had a bending hinge with CP level before, now passes this performance level. In addition, the beam connected to the top floor is removed in which no column was formed before, has now a bending hinge with IO performance level.

Model 20: in this model, number of hinges passing the threshold of collapse as a result of pushover analysis in X direction and in moment frame is studies.

In this model, probability of progressive collapse of columns of moment frame is investigated. In first stage of the pushover analysis, it is observed that in linking beams shearing hinges with CP performance level are formed. Furthermore, in the column of the second floor of a moment frame, a bending hinge is formed in the range of passing the point E and beams connected to this column in higher floors have mostly bending hinges with LS performance level. This issue can be seen in Fig. 4 and after removal of aforesaid column according to Fig. 4, following results were obtained:

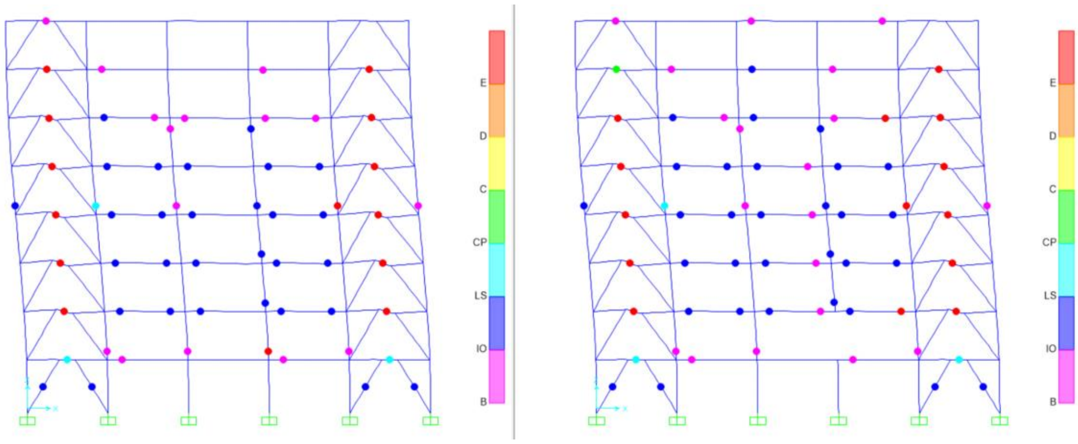

Fig. 4: hinges of pushover analysis (left), removal of column in progressive collapse (right)

Beams connected to this column from 2nd to 5th floor in which no hinge was formed before, have now bending hinges with IO performance level. In addition, in beams near to the bracing bays which had bending hinges with IO and LS performance levels before removal of the column, pass these performance levels at this stage and are now in the range of point E. Of course, other bending hinges are formed in other floors and frames now which are not important from changes in performance level point of view and don't contribute to the probability of progressive collapse.

Model 22: in this model, number of hinges passing the threshold of collapse as a result of pushover analysis in Y direction in moment frame is studied.

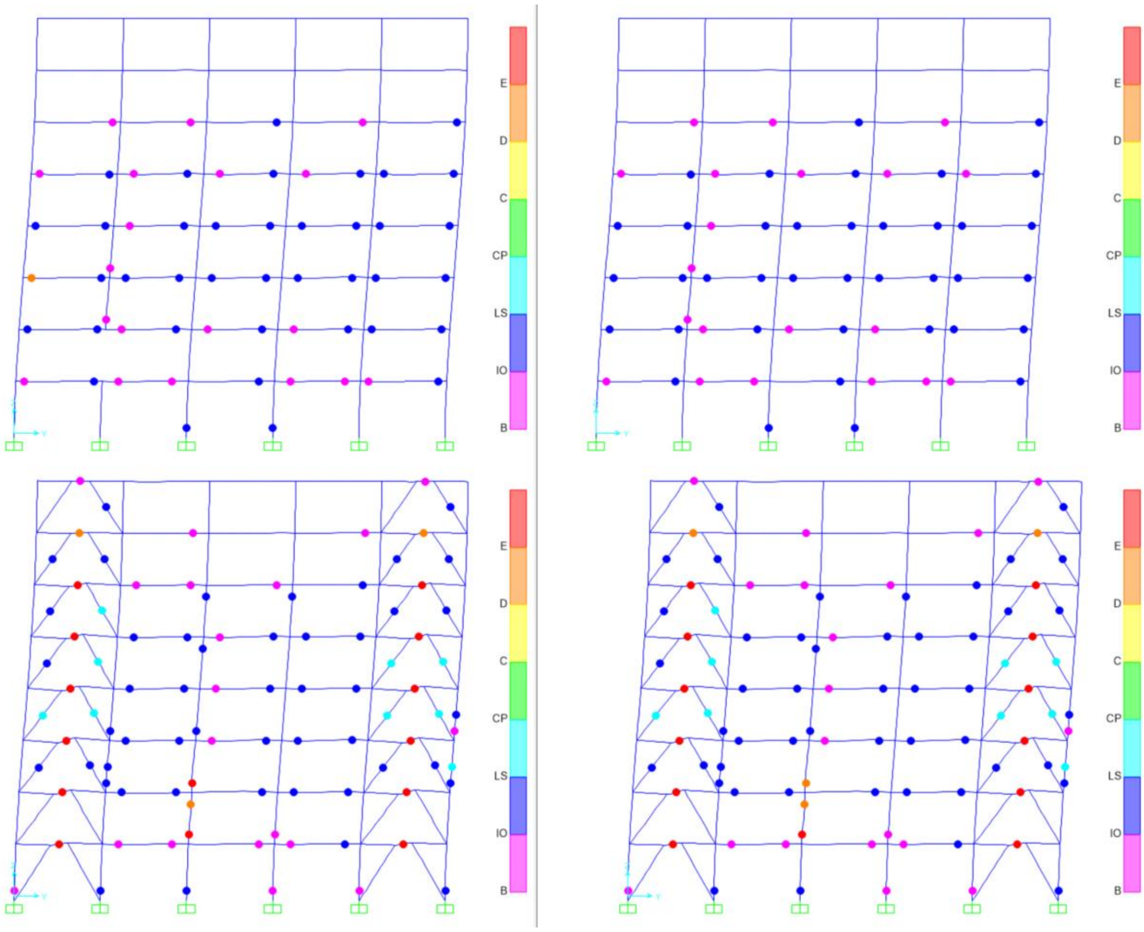

Fig. 5: hinges of pushover analysis (left), removal of column in progressive collapse (right)

In this model, column of the moment frame bays is investigated again. In first step and according to Fig. 5, after the pushover analysis in bracing frames of this model, linking beams have all shear hinges in D-E range and in some columns, bending hinges are formed in the same region. Bracing members have axial force hinges with CP performance level in worst conditions and regarding the moment frames which are the main topic of this model, as can be seen in Fig. 5, bending hinges are formed in beams and columns with IO and LS performance levels. Now, according to Fig. 5, after removal of the column, a lot of bending hinges which were in IO and LS levels before remains unchanged now and only a bending hinge is formed in one of the beams of the third floor in E level which has no considerable importance for change in performance level and no contribution to the probability of seismic progressive collapse.

In addition, as shown in Fig. 5, no change is observed in bracing bays as a result of removal of a column in a moment frame and only one of the columns of the third floor which had bending hinge with LS performance level before, passes now the point E.

Model 23: in this model, number of hinges passing the threshold of collapse as a result of pushover analysis in X direction in moment frame is studied.

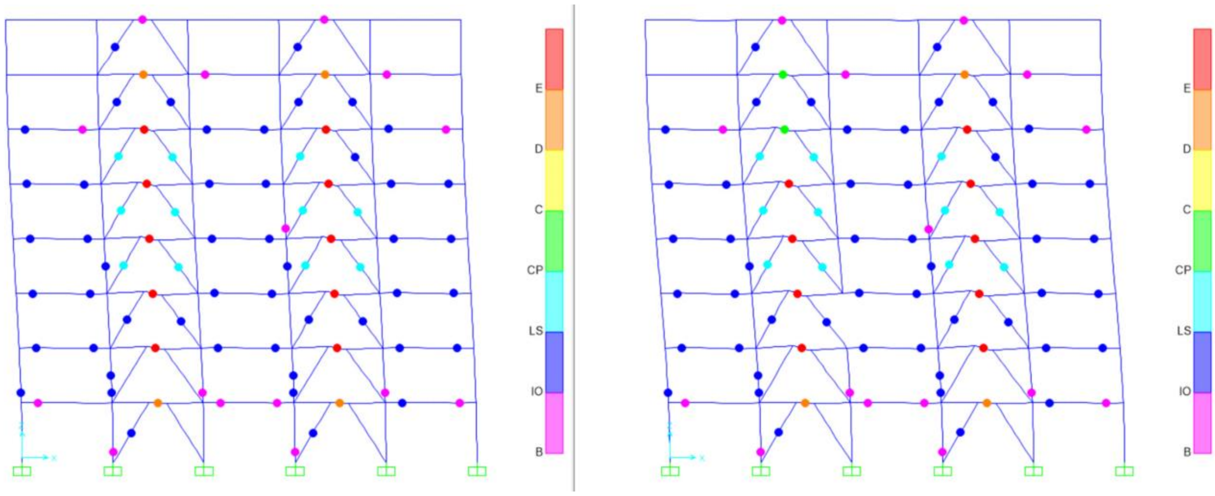

Fig. 6: hinges of pushover analysis (left), removal of column in progressive collapse (right)

In this model, column of the moment frame bays is investigated again. In first step and according to Fig. 6, after the pushover analysis in bracing frames of this model, linking beams have all shear hinges in D-E range and in some columns, bending hinges are formed in the same region. In addition, as shown in Fig. 6, no change is observed in bracing bays as a result of removal of a column in a moment frame and only one of the columns of the third floor which had bending hinge with LS performance level before, passes now the point E.

Model 24: in this model, number of hinges passing the threshold of collapse as a result of pushover analysis in Y direction in moment frame is studied.

The last model in which progressive collapse is studied is model 24. This model has 30% bracing span with respect to perimeter. According to Fig. 7, in the next stage after the pushover analysis, in linking beams, shearing hinges which pass the point E are formed. Moreover, bracing members have mainly the hinges of axial forces with LS performance level or passing point E. now, after removal of column of the fourth floor of one of the bracing bays shown in Fig. 7, linking beams remained in the same state and only a number of linking beams of the last floor in which no hinge was formed before now have hinges with IO performance level and many hinges which pass the point E. in addition, bracing members which are close to the removed column and had hinges with LS performance level before, approach the CP performance level now.

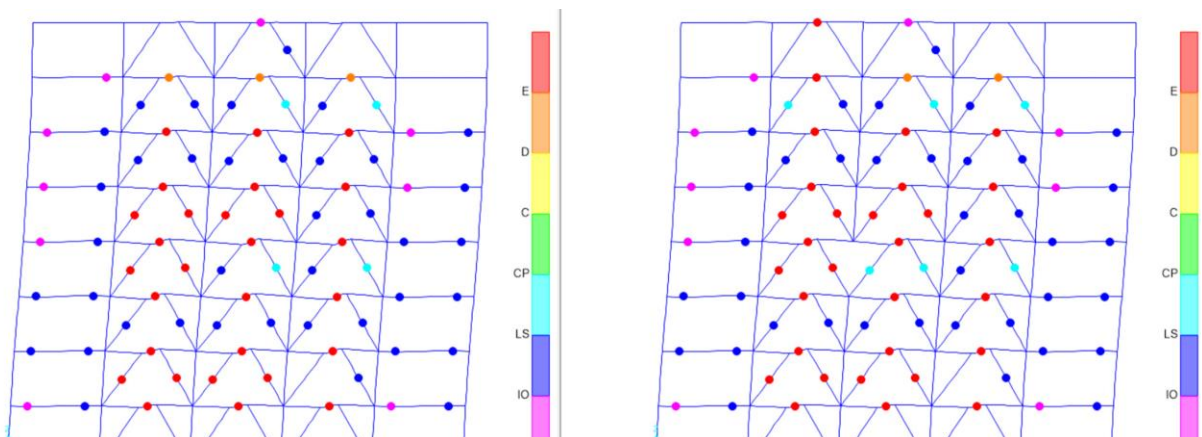

Fig. 7: hinges of pushover analysis (left), removal of column in progressive collapse (right)

9. CONCLUSIONS

- For 5 story buildings, configuration of the model 6 has a low level of risk of progressive collapse. Furthermore, except models 2 and 14, other models are not considerably different and it can be inferred that for 5-soty buildings with conditions of the models, with respect to the number of bracing bays, 10-20% of the building perimeter is an acceptable level and in these models, regular position of bracings has no considerable effect on the probability of progressive collapse. In 5-story models, it was proved that structures with 1-2 bracing bays will be appropriate models and more bays have no contribution to the reduction of weight and probability of the progressive collapse.
- For 8-story buildings models; 23 and 25 are the best. In these models, location of bracing are more effective than their number. With respect to the number of bracing bays, 20-30% of the building's perimeter is an acceptable level. Moreover, irregular models such as 16 and 28 are not appropriate because of contribution of the moment frame, secondary torsional effects and consequently, more hinges in threshold of collapse. In 8–story models, it was proved that structures with 2-3 bracing bays will be suitable models.
- As it can be seen, in low-rise 5-story structures, such as models 3-5 and 10-13, no hinge is formed in moment frames with exceeding collapse prevention performance level and it means that number and location of bracing bays in these models is more appropriate.
- Comparing models 7 and 9, it can be said that in model 9 since columns are connected to braces from both directions and seismic force is applied from both directions to the column, number of failure hinges and effect of progressive collapse increases more than model 7.
- Regarding mid-rise 8-storystructures, it can be observed that as number of floors increases, the structure needs more bracing bays since in model 24, despite of the sufficient number of columns, enough number of bracing bays leads to preventing collapse of the roof and adjacent beams and consequently preventing the seismic progressive collapse in the structure performance level changes significantly. Now, according to conclusion 2, 20-30% of the building's perimeter is of great importance and will be a good factor for maintaining stability of the structure and transfer of force to all of the structure.
- In models such as model 23 in which braces are inside the structure and in middle bays, if the column connected to the bracing bay is removed, no considerable change occurs in efficiency and performance level of other hinges since gravitational loads are transferred to braces and distribution of forces to other members takes place well and it seems that removal of the column connected to the bracing bay has no significant effect on other structural frames and the seismic progressive collapse. Model 23 is considered as the ideal model.
- In general, in structural systems which have bracings, probability of progressive collapse is not high since braces contribute to bearing gravitational loads and if hinges passing from the collapse threshold are not formed in vertical or diagonal bracing members, they can greatly resist against seismic progressive collapse.

REFERENCES

[1] Ferraioli, M., Avossa, A.M., Mandara, A. ,(2014) "Assessment of progressive collapse capacity of earthquake-resistant steel moment frames using pushdown analysis", Open Construction and Building Technology Journal 8(1): 324-336.

[2] Merovich, A.T., Nicoletti, J.P. and Hartle, E. (1982). “Eccentric Bracing in Tall Buildings,” ASCE Journal of the Structural Division, 108(ST9):2066-2079

[3] Roeder, C.W. and Popov, E.P. (1978). “Eccentrically Braced Steel Frames for Earthquakes,” ASCE Journal of the Structural Division 104(ST3): 391-411

[4] Libby, J.R. (1981). “Eccentric Braced Frame Construction—A Case History,” AISC Engineering Journal, 4th Quarter:149-153

[5] Engelhardt, M.D. and Ricles, J.M. (1989). “Eccentrically Braced Frames: U.S. Practice,” AISC Engineering Journal, 26(2): 66-80.

[6] Montuori R., Nastri E., Piluso V. (2015) "Seismic design of MRF-EBF dual systems with vertical links: EC8 vs plastic design", Journal of Earthquake Engineering 19(3): 480-504.

[7] DOD (Department of Defense) (2009), UFC (Unified Facilities Criteria), UFC 4-023-03: Design of buildings to resist progressive collapse, Washington ,USA

[8] ASCE (American Society of Civil Engineers) (2010), ASCE7-10: Minimum Design Loads for Buildings and Other Structures , Virginia, USA

[9] FEMA (Federal Emergency Management Agency) (2009), FEMA P695-2009: Quantification of Building Seismic Performance Factors, USA

[10] ASCE (American Society of Civil Engineers) (2013), ASCE41-13: Seismic Evaluation and Retrofit of Existing Buildings , Virginia ,USA

[11] AISC (American Institute of Steel Construction (2010),AISC341-10 Seismic Provisions for Structural Steel Buildings, Chicago, Illinois, USA

[12] Permanent committee of revision, (2013), instruction of seismic optimization of the available buildings, no. 360, deputy of planning and strategic monitoring of the president.

[13] Permanent committee of revision, (2005), third edition of the earthquake standard 2800, regulation of the building design, center of housing and building researches.

[14] Richards, Paul.W. (2010). “Estimating the stiffness of eccentrically braced frames (EBFs).” ASCE Practice Periodical on Structural Design and Construction 15(1): 91-95.