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Non-linear performance analysis of existing and concentric braced steel structures

R. Tuğrul Erdem *

Department of Civil Engineering, Celal Bayar University, Manisa, Turkey

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Abstract. Since there are several places located in active seismic zones in the world, serious damages and losses have happened due to major scaled earthquakes. Especially, structures having different irregularities have been severely damaged or collapsed during these seismic events. Behavior of existing structures under several loading conditions is not completely determined due to some uncertainties. This situation reveals the importance of design and analysis of structures under seismic effects. Several non-linear static procedures have been developed in recent years. Determination of the seismic safety of the existing structures and strengthening techniques are significant civil engineering problems Non-linear methods are defined in codes to determine the performance levels of structures more accurately. However, displacement based ones give more realistic results. These methods provide more reliable evaluation possibilities for existing structures with developing computer technology. In this study, non-linear performance analysis of existing and strengthened steel structures by X shaped bracing members with 3, 5 and 7 stories which have soft story irregularity is performed according to FEMA-356 and Turkish Earthquake Code-2007. Damage ratios of the structural members and global performance levels are determined as well as modal properties and story drift ratios after non-linear finite elements analysis for each structure.

Keywords: non-linear methods; performance analysis; steel structures; strengthening techniques

1. Introduction

Analysis of structures for different levels of earthquake intensity and determination of damage levels have been mainly investigated by scientists and engineers for the last couple of decades. As lateral forces cause horizontal displacements, damages are observed because of these displacements after earthquakes. Studies have been developed about determination of seismic safety of structures in the world (Sucuoğlu *et al*. 2007, Kalkan and Kunnath 2007, Inel *et al*. 2008, Yun *et al*. 2002, Elghazouli 2007, 2010).

Design of the structure that depends on necessary conditions under seismic effects is defined as performance based design. New approaches are presented with the development in design of earthquake resistant structures and experiences after earthquakes. Researchers determine the possible damages under seismic effects due to these approaches (Grigorian and Grigorian 2011, Dalal *et al*. 2012). Studies about performance analysis of existing structures have increased all

*Corresponding author, Assistant Professor, E-mail: tugrul.erdem@cbu.edu.tr

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around the world. Performance based design procedures are defined in FEMA-356 (DCM) and Chapter 7 of Turkish Earthquake Code-2007 (FEMA-356 2000, TEC-2007 2007).

Performance conception is developed to determine the seismic safety of existing structures and being used widely for this purpose. Main purpose of these methods is controlling of the structural performance for the significant seismic force level. Behavior of structural system is evaluated more accurately in non-linear analysis methods. Non-linear methods are deformation based ones and more parameters are required to evaluate the structural members. Deformation based methods tend to overestimate the global deformation demands with respect to the capacity spectrum method. However, sizes of the structural members, material properties and structural details shall be exactly known in non-linear performance analysis of existing structures. Joints of members are mostly suffered from damages under seismic effects. Generally, it is mentioned in codes that structures shall survive without any damage in minor scaled earthquakes, they shall provide life safety in big scaled earthquakes and they shall not be totally collapsed in major scaled earthquakes.

Irregularities happen in several structures due to some negative cases. Because irregular structures negatively effect the seismic behavior, engineers shall avoid designing them. Knowing the effects which occur from structural irregularities during an earthquake is important to predict the structural behavior. Determination of seismic safety of irregular structures in a proper way can be possible by non-linear analysis methods. Irregularities are observed both in plane and vertical directions. However, vertical irregularities make difficult to perform analyses. Damages easily occur at weak regions of irregular structures.

Height of the base floors of the structures may be higher than other ones in terms of some economic purposes. Base floors are usually used for stores, restaurants and banks. Rigidity of the base floor differs from other ones in these structures. Therefore, horizontal displacements occurring at these floors are relatively bigger than other ones. This situation causes soft story irregularity. Structures having soft story irregularity have low load carrying capacities under the effect of lateral forces. Since behavior of soft story is different from other ones, especially high structures may collapse suddenly due to the bigger displacements and non-proportional lateral stress values of soft story columns. Massive damages are mostly observed in the base floors of these types of structures during earthquakes (Stefano and Pintucchi 2008, Soni and Mistry 2006).

Steel structures are used for many purposes such as high-rise buildings and long-span bridges. Steel enables improved quality with less maintenance. Thus, steel structures provide safety and resistance against earthquake and wind effects for long period of time. Steel structural systems are homogenous, isotropic, ductile, weightless and resistant. They enable reliable and non-damages production in all weather conditions. These structures also have fast and cost effective construction process. In addition, the material can be used after the structure is disassembled.

Strengthening of existing structures is a significant issue in our day. Different techniques are used to strengthen steel structures. However, steel bracings are mostly utilized to primarily improve seismic behavior of many existing structures (Mahmoudi and Zaree 2010, Sabelli *et al.* 2003, Korkmaz *et al.* 2008, Brandonisio *et al.* 2012, Chao and Goel 2006, Chao *et al.* 2008, Merczel *et al.* 2013). Concentrically braced systems are efficient in resisting lateral forces because of providing high strength and stiffness. Braced members are usually used with two diagonal supports placed in an X shaped manner in practice. X shaped bracing members that intersect at a node are primarily preferred since they are efficient and provide complete truss action. They are also used in bridge supports, structural foundations as well as buildings.

In this paper, non-linear static analysis of existing steel structures with 3, 5 and 7 stories whose height of the first floor is different from other ones is modeled in the first place. Then, base floor of the structure is strengthened with X shaped steel bracing members which are frequently used as a strengthening technique for existing structures. Weight of the structures is calculated as well as period and effective mass ratio values. Damage ratios of the structural members are obtained according to non-linear methods of FEMA-356 (DCM) and TEC-2007 Finally, global performance levels of the structures and story drift ratios are determined. SAP2000 finite elements program is used for analyses (Computers and Structures Inc. 1995). The results are comparatively given and suggestions are proposed.

2. Soft story irregularity

Although there are several criteria about analysis of seismic safety, soft story irregularity is one of the most significant one which may cause destructive damages. Soft story irregularity can be defined as the different respond of a story compared to other ones under the effect of an earthquake. First story of many buildings that are located on the main streets and center of the cities are usually used for stores and showrooms. Windows take place instead of walls in the first stories to see the whole interior face from outside of the buildings. However, brick walls are used for all faces in other stories. In this case, while most of the deformation occurs in the soft story, energy dissipation substantially develops in the columns of this story during a possible earthquake as shown in Fig. 1.

Soft story irregularity is usually observed in apartment buildings having three or more stories located on a ground level with large openings. When soft story irregularity combines with failures in column design in a building, serious damages and losses are inevitable. Several buildings have been collapsed because of soft story irregularity in recent years. Many examples of this situation have been happened in previous earthquakes (Kirac *et al*. 2011). Generally, upper stories collapse on the soft story in the buildings which have soft story irregularity. For this reason, it is impossible to use this kind of buildings after damages.

Most of the braced frame systems are concentric which means members of the system intersect at a node which is the center point. Members in these systems are designed to work in both tension and compression in common with a truss. Concentrically braced steel members are frequently used in areas of high risk to strengthen of existing structures because of being economic to construct

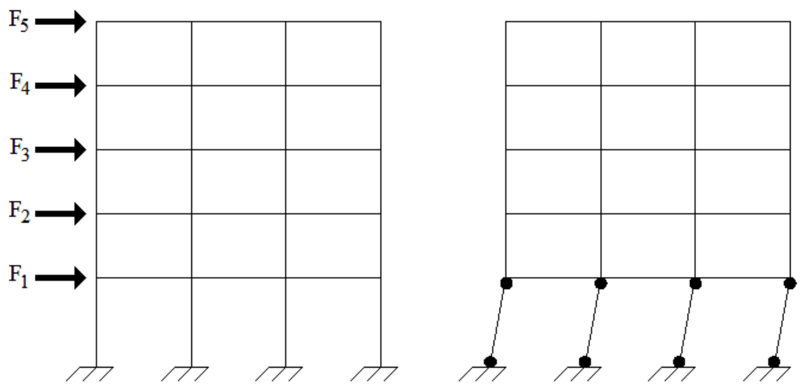
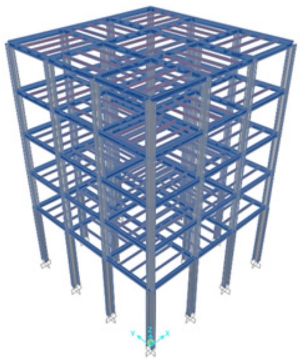
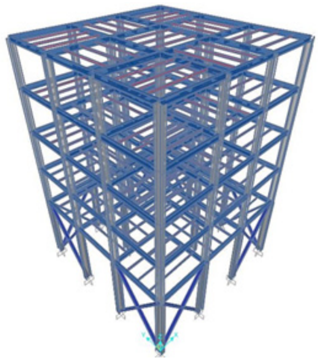


Fig. 1 Soft story irregularity



(a)



(b)

Fig. 2 Finite elements model

and simple to analyze. These members are produced to resist lateral forces and provide stability. X shaped bracing members also improve inelastic behavior and rigidity of structures. Existing structural systems concentrate damage to some extent especially in higher buildings. As X bracings intersect at the middle of the spans, they enable perfect balancing. For this reason, X braced frames improve the deformation capacity and seismic behavior of existing structures.

3. Examined cases

The structures have 3, 5 and 7 stories. While height of the first floor is 5.2 m, other floors have 3 m height for each structure. Thus, the structures are considered to represent low-rise, mid-rise and high-rise buildings that consist of typical steel sections. Structural frame systems are designed to have typical column-beam sections to obtain the ductile behavior. Soil class type is selected as Z3 according to TEC-2007 which is similar to class C of FEMA. Besides, the structures are assumed to be located in the first level seismic zone. The analyses are performed for both existing and strengthened structures by concentric X shaped steel bracing members. As soft story columns are mostly effected from lateral forces, these members are placed in outer axes of the first stories. Steel bracings also restrain torsion effects. The analyses are performed for each structure. Finite elements models of the 5 storey structures are given as an example in Fig. 2.

The structures are 12 × 12 m in plan. Steel material type is St 37 whose ultimate strength is 370 MPa, yield point is 235 MPa, and shear safety strength is 82 MPa. While column sections are HEA 500 in the first floor and HEA 400 in other floors, primary beam sections are IPE 300 in all stories. However, sections of the secondary beams that sustain the slabs by reducing their span length are IPE 120. Finally, steel bracing members are box sectioned and their section sizes are 100 × 200 × 5 mm. Plan and front views of the 5 storey structure are presented in Fig. 3.

The vertical loads consist of dead and live loads of slabs, wall loads on beams as well as self loads of structural members. While weights of the 3, 5 and 7 storey existing steel structures are 182.62 t, 304.05 t and 425.49 t, strengthened ones are 184.47 t, 305.90 t and 427.33 t respectively. Modal properties of the structures are determined after finite elements analyses. First periods of vibration modes and related effective mass ratios are given in Table 1.

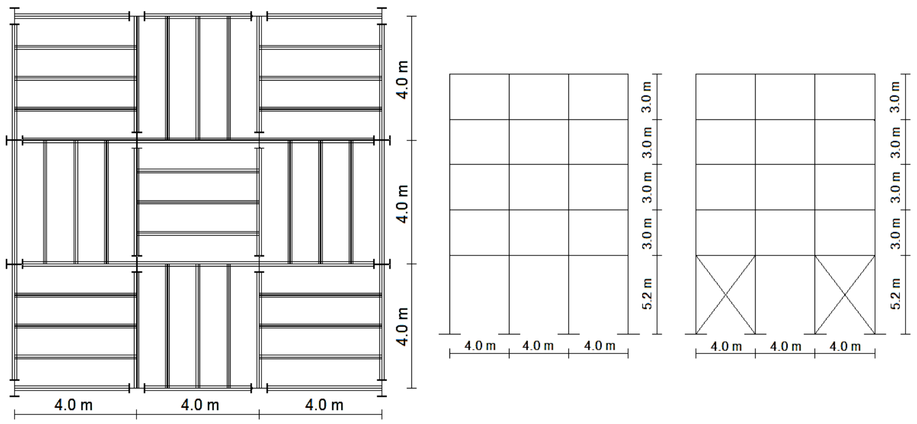


Fig. 3 Plan and front views

Table 1 Periods of vibration modes and related effective mass ratios

Structure type	Period (s)	Effective mass ratio (%)
3 Storey	0.51	94
5 Storey	0.77	88
7 Storey	1.02	83
Strengthened 3 Storey	0.33	87
Strengthened 5 Storey	0.61	76
Strengthened 7 Storey	0.82	72

After the structures are modeled and loads are applied to the members, analyses are performed according to non-linear evaluation methods. For this purpose, static pushover curves are obtained for each structure. Afterwards, damage situations and the performance levels of the structures are determined by comparing the values related to plastic rotations with the limit values defined in FEMA and TEC-2007.

4. Non-linear analysis

Non-linear pushover analysis is considered as a series of incremental analysis that is performed to determine the behavior of the structure. The main purpose of static pushover analysis is evaluating the performance level of the structure at target displacement value. Useful information about response characteristics is provided by pushover analysis which can't be obtained from elastic static or dynamic analyses (Fajfar 2000, Moghaddam and Hajirasouliha 2005).

There are available procedures for non-linear performance analysis in the literature. Non-linear evaluation methods are described in the main guidelines. Displacement based methods are mainly subjected to performance analysis and they have taken place instead of force-based methods recently. Internal forces and deformations of the investigated structural system shall be determined according to non-linear analysis methods to estimate possible damages and losses. General steps for non-linear analysis are presented in Fig. 4.

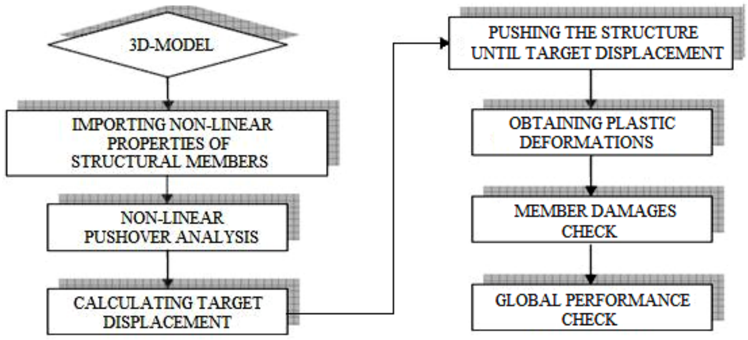


Fig. 4 Non-linear analysis steps

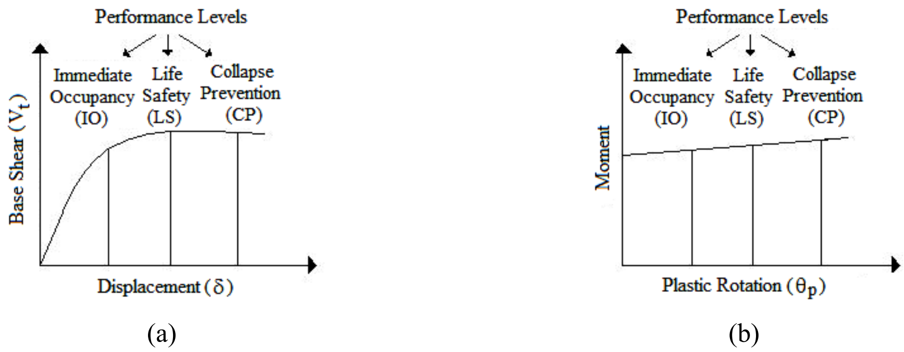


Fig. 5 Performance levels for structures

The main difference between related methods in the guidelines is the target displacement determination of the structure and the criteria employed for the acceptance which is based on the performance limit values. Performance levels which are immediate occupancy, life safety and collapse prevention are defined similarly in FEMA and TEC-2007. In immediate occupancy level, there are no damages happened in structural members under seismic effect. A few members may exceed yield point. Small cracks might be seen in non-structural members. In life safety level, damages may occur in some structural members. However, these members still keep most of lateral stiffness and rigidity. Little deformations may happen in the structure but they can't be noticed visually. In collapse prevention level, damages happen in substantial part of the structural members. Some of these members lose their strength and lateral rigidity. Some secondary members are collapsed. Permanent displacements also occur in the structure. Performance levels are seen in Fig. 5.

4.1 Performance analysis for FEMA

Target displacement value can also be calculated according to Displacement Coefficient Method which is defined in FEMA. In this method, base shear force (V_t) and displacement of the peak point ($\delta_{\max}$) is obtained in the first place after static pushover analysis. Afterwards, this curve is idealized to be formed of two lines. While slope of the first line represents elastic rigidity (K_e),

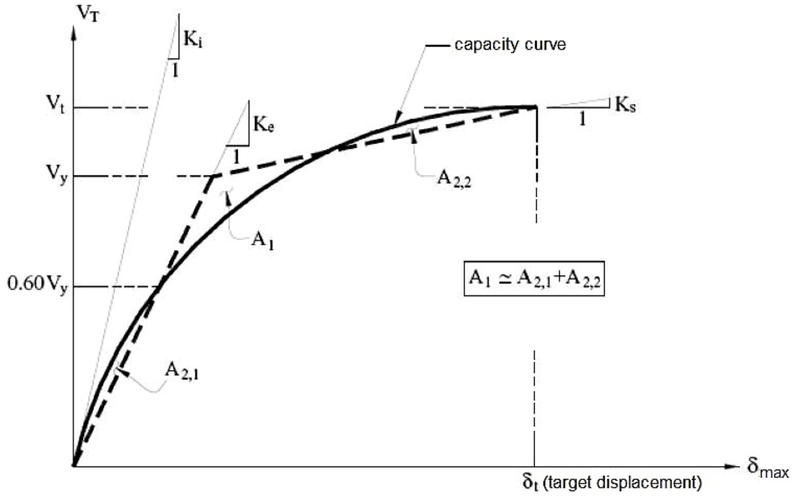


Fig. 6 Determination of target displacement according to FEMA

Table 2 Analysis steps according to FEMA

Coefficient	FEMA-356 (DCM)		
C_o	- The first modal participation factor at the level of the displacement control node. - The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement. - It is explained according to framing system and story number at the Table 3-2 of FEMA 356.		
C_1	$C_1 = 1.00$	for	$T_e \geq T_o$
	$C_1 = 1 + \frac{(R_o - 1)T_o}{T_e}$	for	$T_e < T_o$
C_2	Values for different framing systems and structural performance levels shall be obtained from Table 3-3 of Fema 356.		
C_3	$C_3 = 1.00$		$\alpha = \frac{K_s}{K_e} > 0$
	$C_3 = 1.0 + \frac{\vert\alpha\vert (R_o - 1)^{3/2}}{T_e}$		$\alpha = \frac{K_s}{K_e} \leq 0$

the second one is elasto-plastic rigidity (K_s). Areas under real and idealized capacity curves shall be equal to each other as shown in Fig. 6.

Effective period (T_e) of the structure is calculated according to Eq. (1). While T_i is the elastic period in the related direction, K_i is the elastic lateral rigidity of the structure in the equation. After determining the effective period value, target displacement is calculated according to Eq. (2). While S_a is the response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration, g is the acceleration of gravity. Analysis steps according to FEMA-356 (DCM) and parameters of Eq. (2) are given in Table 2. T_o represents characteristic period of the response spectrum in the table.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{1}$$

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2 g} \tag{2}$$

4.2 Performance analysis for TEC-2007

The coordinates of capacity curve is changed to modal response acceleration-modal response displacement to determine the target displacement value (δ_t) according to TEC-2007. This value is calculated according to initial period in TEC-2007 as shown in Fig. 7. Analysis steps for TEC-2007 are given in Table 3.

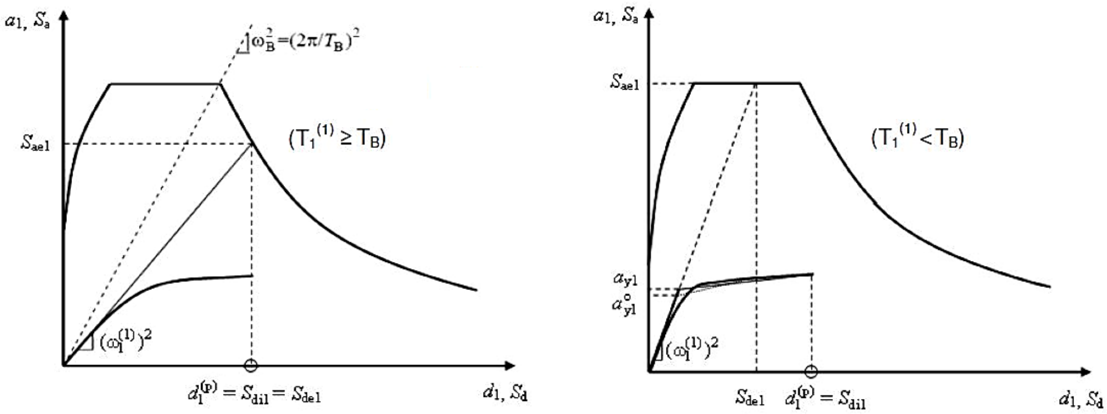


Fig. 7 Determination of target displacement according to TEC-2007

Table 3 Analysis steps according to TEC-2007

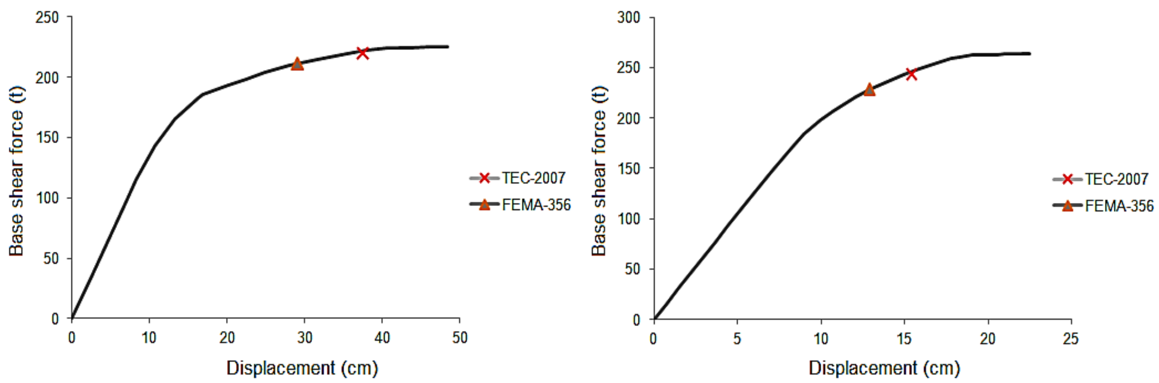
1.	Any point V_i, δ_t on the multiple degree of freedom capacity curve is converted to the corresponding point S_{ai}, S_{di} on the equivalent single degree of freedom capacity spectrum using the modal mass coefficient and participation factors equations.
2.	A point on capacity spectrum curve is estimated as performance point and spectrum curve is idealized with two linear lines.
3.	Non-linear spectral displacement, $S_{di1} = C_{R1} S_{del}$ Linear spectral displacement, $S_{del} = \frac{S_{ael}}{(\omega_1^{(1)})^2}$ Spectral displacement ratio C_{R1}, is determined by initial period $T_1^{(1)}$, $T_1^{(1)} = 2\pi / \omega_1^{(1)}$

Table 3 Continued

4.	If $T_1^{(1)}$ initial period is equal or bigger than characteristic period T_B, at acceleration spectrum $C_{R1} = 1$ is taken.
5.	If $T_1^{(1)}$ initial period is lower than characteristic period T_B at acceleration spectrum, $C_{R1} = \dfrac{1+(R_{y1}-1)T_B/T_1^1}{R_{y1}}$ R_{y1} is strength decrement coefficient in the first mode, $R_{y1} = \dfrac{S_{ael}}{a_{y1}}$
6.	After the target performance point is calculated, converted capacity curve should be made linear with equal areas rule and a_{y1}, R_{y1}, C_{R1} values shall be calculated. Target performance point is not known at first. So, a few trial and error solutions can be necessary.

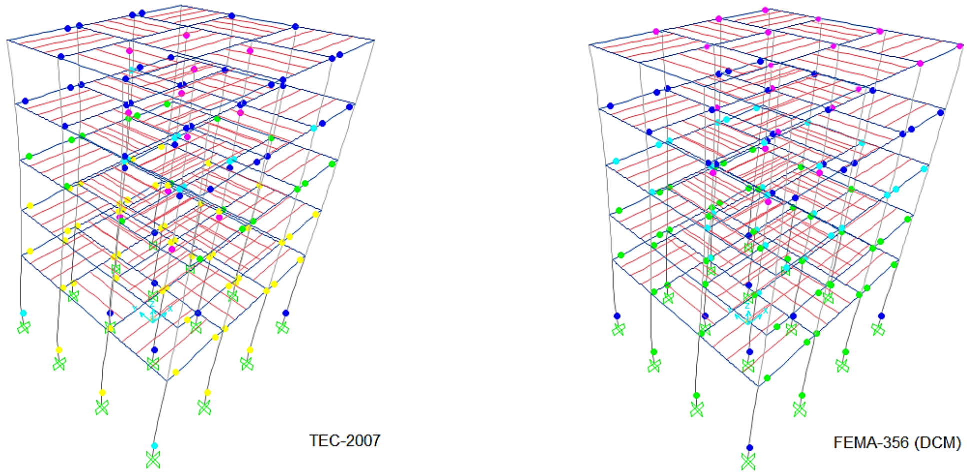
5. Analysis results

The main purpose of non-linear analysis is to determine the performance of existing structures that are affected by seismic loads. Incremental static pushover analysis is usually employed for performance evaluation. This analysis is an attempt to evaluate the useful and effective results for the performance based designs. Geometry of the structural system, sections, material properties and inelastic behavior are taken into consideration to apply lateral forces step by step. Capacity curve representing the relationship between the base shear force and the roof displacement is obtained after incremental pushover analysis. Non-linear static analysis under incrementally increasing lateral seismic forces is distributed in accordance with the dominant mode shape in the related earthquake direction until the target displacement is reached or the structure is not able to resist further forces.



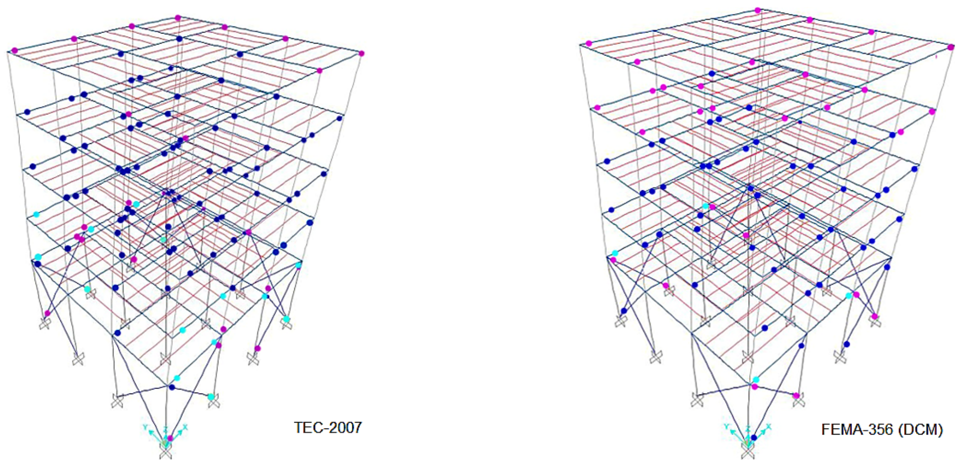
(a) (b)

Fig. 8 Target displacements of the structures



(a) (b)

Fig. 9 Plastic hinges at target displacements for the 5 storey existing structure



(a) (b)

Fig. 10 Plastic hinges at target displacements for the 5 storey strengthened structure

Damage states of the structural members are determined after the 3, 5 and 7 storey steel structures are pushed to the calculated target displacements. Target displacements according to each code are signed on the capacity curve and given for the 5 storey existing and strengthened structures as an example in Fig. 8. Target displacements which are also accepted as performance points are 28.37 cm and 37.28 cm for the existing structure according to FEMA-356 and TEC-2007. On the other hand, these values are 12.96 cm and 15.43 cm respectively after the structure is strengthened.

Non-linear behavior is confined to plastic hinges which are defined at both ends of primary structural members. These members are modeled as non-linear frame members with lumped plasticity by defining these plastic hinges. The steel structures are pushed to the calculated target displacements for each code in the first place. Afterwards, plastic hinges occur at the ends of structural members.

To avoid buckling situation, the hinges are defined to work both tension and compression cases. By this way, the hinges are observed at each end of braces. Since the most conservative results are obtained according to TEC-2007, the results are presented for 5 storey existing and strengthened structures according to this code and given in Figs. 9-10. It is seen that plastic hinges occur at braces after strengthening operation. Thus, damages of structural members are reduced. Similar solution steps are followed for the rest of the structures.

There are three performance levels such as immediate occupancy, life safety and collapse prevention defined for existing structures. These levels are decided according to damage ratios of structural members which are determined after non-linear analysis for each code. Member damages are calculated at both directions for the 3, 5 and 7 storey structures. Damage situations of structural members according to codes are given between Tables 4-6. Since the story plan of the

Table 4 Member damages for 3 storey structure

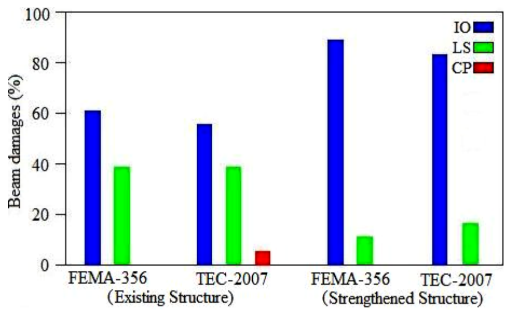
Members	Story	Existing structure						Strengthened structure					
		FEMA-356			TEC-2007			FEMA-356			TEC-2007		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	-	12	-	-	10	2	8	4	-	6	6	-
	2	10	2	-	8	4	-	12	-	-	12	-	-
	3	12	-	-	12	-	-	12	-	-	12	-	-
Columns	1	-	14	2	-	12	4	12	4	-	10	6	-
	2	16	-	-	16	-	-	16	-	-	16	-	-
	3	16	-	-	16	-	-	16	-	-	16	-	-

Table 5 Member damages for 5 storey structure

Members	Story	Existing structure						Strengthened structure					
		FEMA-356			TEC-2007			FEMA-356			TEC-2007		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	-	7	5	-	3	9	-	12	-	-	12	-
	2	-	10	2	-	8	4	6	6	-	2	10	-
	3	8	4	-	6	6	-	12	-	-	12	-	-
	4	12	-	-	12	-	-	12	-	-	12	-	-
	5	12	-	-	12	-	-	12	-	-	12	-	-
Columns	1	-	10	6	-	8	8	-	16	-	-	16	-
	2	16	-	-	16	-	-	16	-	-	16	-	-
	3	16	-	-	16	-	-	16	-	-	16	-	-
	4	16	-	-	16	-	-	16	-	-	16	-	-
	5	16	-	-	16	-	-	16	-	-	16	-	-

Table 6 Member damages for 7 storey structure

Members	Story	Existing structure						Strengthened structure					
		FEMA-356			TEC-2007			FEMA-356			TEC-2007		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	-	4	8	-	-	12	-	10	2	-	8	4
	2	-	8	4	-	4	8	3	9	-	-	12	-
	3	4	8	-	-	12	-	10	2	-	8	4	-
	4	10	2	-	8	4	-	12	-	-	12	-	-
	5	12	-	-	12	-	-	12	-	-	12	-	-
	6	12	-	-	12	-	-	12	-	-	12	-	-
	7	12	-	-	12	-	-	12	-	-	12	-	-
Columns	1	-	4	12	-	-	16	-	14	2	-	12	4
	2	16	-	-	16	-	-	16	-	-	16	-	-
	3	16	-	-	16	-	-	16	-	-	16	-	-
	4	16	-	-	16	-	-	16	-	-	16	-	-
	5	16	-	-	16	-	-	16	-	-	16	-	-
	6	16	-	-	16	-	-	16	-	-	16	-	-
	7	16	-	-	16	-	-	16	-	-	16	-	-



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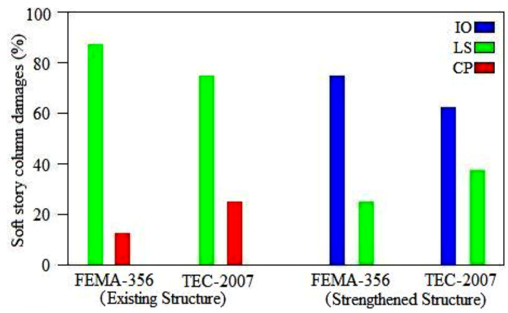
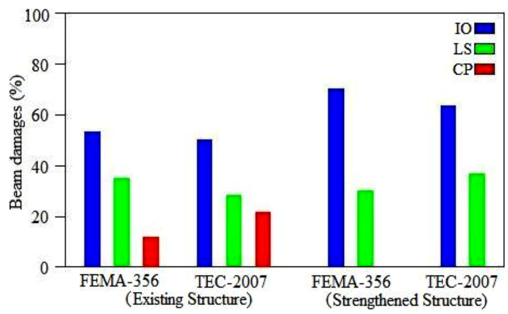


(b)

Fig. 11 Damage ratios for 3 storey structures



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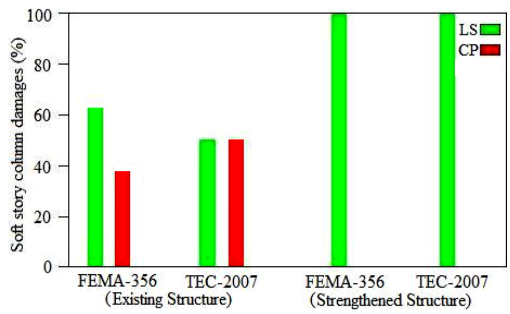
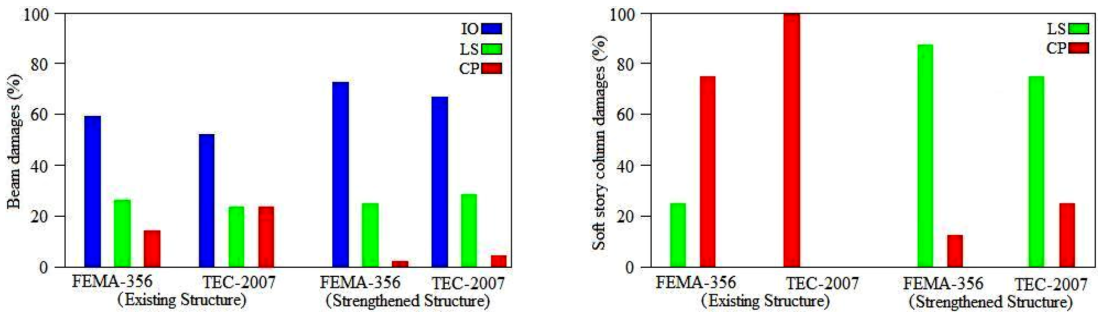


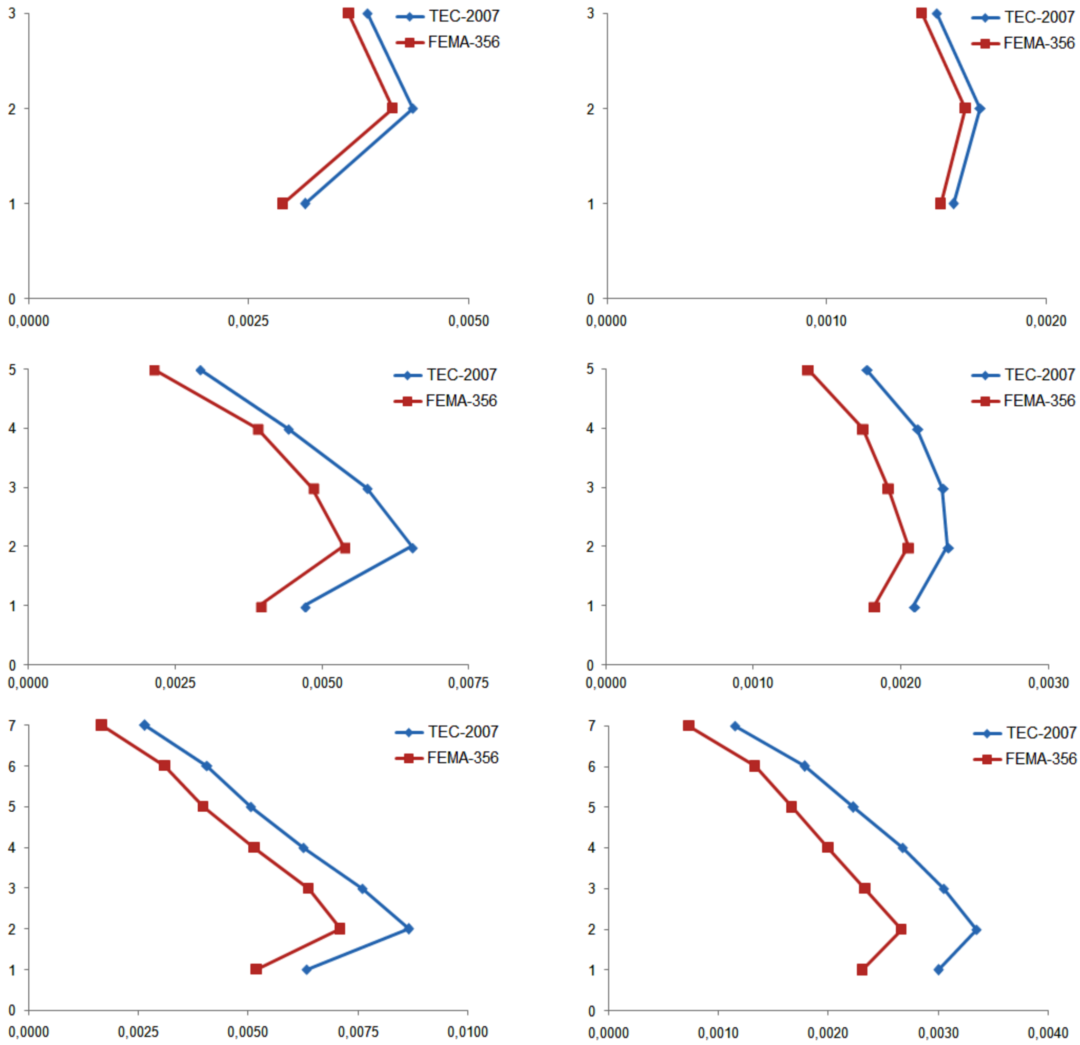
(b)

Fig. 12 Damage ratios for 5 storey structures



(a) (b)

Fig. 13 Damage ratios for 7 storey structures



(a) Existing structure (b) Strengthened structure

Fig. 14 Story drift ratios for structures according to codes structures is symmetrical in both directions and sizes of the members are same at each floor, damage situations are equal for *x* and *y* directions. So, the results are given for one direction at each story.

After damage ratios of structural members are determined for each story, total damage ratios of beams and soft story columns for the structures according to related codes are presented between Figs. 11-13.

As big displacements in structural members lead to severe damages, story drift ratios are accepted as one of the most significant parameters effecting performance results. These ratios are especially effective in higher buildings under seismic effects. For this reason, existing and strengthened steel structures are pushed to the target displacements and drift ratios for each story are also calculated according to codes. It's seen that strengthening operation has important effect on reducing the lateral displacements. Story drifts according to codes differ from each other as the height of the structure increases. The results for 3, 5 and 7 storey structures are comparatively presented in Fig. 14.

6. Conclusions

There are several performance based analyses defined in recent codes and guidelines to decide the seismic performance of existing structures. It's taken much time and work to define linear and non-linear procedures by researchers. However, performance analysis results according to these methods usually differ from each other. This situation may cause contradiction between engineers since both methods are used. On the other hand, more data about geometry, material and structure is needed for non-linear procedures to perform analyses. Thus, non-linear methods provide detailed information about seismic behavior of structures only if the data is reliable.

Different strengthening techniques are used to improve seismic safety of existing structures in applications. Concentrically braced systems develop the lateral strength and stiffness to provide serviceable structural performance during ground motions. These systems are generally efficient and economic for structures in areas of high seismicity. Concentric braces are widely used applications for strengthening of existing steel structures to improve the seismic resistance. Since configuration of braces effect system performance, multiple configurations of bracings are commonly utilized in applications. X shaped bracing members intersect and connect at middle of the sections. These members enable balancing easily. Thus, the lateral resistance in tension and compression is distributed similarly in both directions. As a consequence, engineers have increasingly turned to concentrically braced steel frames to resist seismic forces in an economic way.

Soft story which is seen in several existing structures is an undesired structural irregularity. Analysis of structures having soft story irregularity can be performed after detailed calculations by engineers and researchers. Due to non-proportional lateral stress, soft story is suffered from heavy damages which usually cause complete collapse of the entire structure. Base floors of the structures are usually used for different purposes as parking lots and stores. While the surfaces of these floors are covered by glass cases; brick walls or gas concretes are used in other floors. In this case, rigidity of the base floors is much lower than other ones. Since, bigger displacements occur in soft story columns, the structures may collapse suddenly.

Seismic performances of structures can be determined more realistically due to the displacement based methods. In this paper, non-linear analyses of steel structures with 3, 5 and 7 stories are performed according to FEMA-356 (DCM) and TEC-2007. Although span length values and sizes of the structural members are the same, height of the first floor is different from other ones for each structure. After capacity diagrams are obtained by static pushover analyses, target displacements are calculated according to each code. The structures are pushed to the target displacements to decide damage situations and story drift ratios relatively. Damage ratios of the structural members are obtained according to non-linear methods of related codes. Finally, global performance levels of the structures are determined.

Based on the results of non-linear analysis for each code, it is stated that TEC-2007 gives more conservative results than FEMA-356 (DCM). With respect to analyses of existing structures, damage ratios reach the highest values in the first floors as expected. Damage situations of the structural members increase in direct proportion to total height of the buildings. Heavy damages and bigger story drift ratios occur in 7 storey structure. While all soft story columns reach collapse prevention level according to TEC-2007, %75 of them get this damage level according to FEMA-356 for the highest existing structure. Life safety and immediate occupancy levels are provided in upper floors of the structures. Maximum beam damages are also observed according to TEC-2007. %23.8 and %14.3 of the beams stay in collapse prevention level according to TEC-2007 and FEMA-356 for 7 storey existing structure relatively.

After the structures are strengthened by using X shaped bracing members, significant improvements are observed in seismic safety according to codes. There aren't any structural members staying in collapse prevention level for 3 and 5 storey strengthened structures. All members provide immediate occupancy and life safety levels. On the other hand, although remarkable improvement is provided in performance levels, a few story columns take place in collapse prevention level for 7 storey strengthened structure. Moreover, story drift ratios considerably decrease after strengthening operation.

Due to the results of the existing and strengthened structures, it's stated that soft story irregularity is more effective for 7 storey structure. Soft story columns cannot provide immediate occupancy or life safety levels as the height of the structures increase. Moreover, collapse damage situation is seen in more than one floor of existing structures except 3 storey one according to each code. On the other hand, significant improvement is obtained in performance levels and story drift ratios after the structures are strengthened by X shaped bracing members. While immediate occupancy and life safety performance level ratios of structural members increase, collapse prevention level ratios decrease for all structures. Finally, this study can be improved by analyzing different types of existing structures with other strengthening techniques according to non-linear methods of various codes.

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