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Research Article

Performance evaluation and strengthening of reinforced concrete buildings

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Abstract: In view of the past earthquakes, it is stated that several structures are located in active seismic zones around the world. So, improvement of the earthquake performance levels of the existing buildings by using various strengthening methods has been the major interest in structural engineering. Non-linear analysis procedures that are defined in several seismic codes exhibit reliable results in the evaluation of seismic performances of existing buildings. In this study, seismic performances of three, five and eight storey existing and strengthened reinforced concrete buildings having the common floor plan, material and section properties of the structural members are investigated according to Turkish Building Earthquake Code-2018 and American Standard, ASCE. To obtain the earthquake performance results of reinforced concrete buildings, displacement demands of the buildings have been obtained and utilized in nonlinear analyses. SAP2000 structural analysis software is used in the solutions. The determined strengthening techniques are provided by adding concentric steel bracing members to existing reinforced concrete buildings and jacketing of the determined columns. As a result of non-linear analyses applied to the existing and strengthened buildings, damage situations of the structural members are determined, seismic performances of the buildings are evaluated according to both codes and the results are interpreted in the end.

Keywords: existing buildings, non-linear analysis, seismic codes, seismic performance, strengthening.

1. Introduction

Earthquake is the event of instant release of stored seismic energy in earth's crust, spreading as waves and shaking the environments as they pass through. Earthquake is one of the most effective natural disasters on the planet we live in. It is known that many people have been affected by the earthquakes of various magnitudes since the existence of the world. Especially, several earthquakes with a magnitude of 6 or above have been happened in the year of 2020 as it is reported by United States Geological Survey. Considering the losses and damages in the recent earthquakes, it becomes evident that the performance evaluation and strengthening of the existing buildings is as important as the design of new buildings.

As reinforced concrete (rc) structures provide several advantages such as long service life, high rigidity, ductility and durability, these structure make up the big majority of the building stock in all around the world. However, seismic performance evaluation of the existing rc buildings shall be investigated due to the recent improvements in the seismic codes.

Assessment of these buildings under various levels of seismic intensity and determination of damage levels have been in the focus of structural engineers and researchers (Mosleh et al.; 2016; Yurdakul et al., 2021).

Due to the advancing computer technology and improvements in the seismic codes, non-linear methods have commonly used by researchers rather than linear approaches (Erdem, 2016; Halder & Paul, 2016). As a result of this situation, non-linear analysis procedures according to different seismic codes or guidelines are utilized to investigate the structural performances of the existing buildings in recent studies (Aksoylu et al.; 2020; Masi et al.; 2019; Vijay et al.; 2012). Especially, the essential component of these procedures is the pushover analysis technique in which a series of non-linear incremental static analyses which are performed to obtain the lateral deformation and damage situations of structural members (El-Betar, 2018; Gunes et al., 2019; Mazza, 2014; Zou & Chan, 2005).

The most common view of the seismic codes is providing the life safety performance level and preventing the total collapse after evaluating the performance levels of the existing buildings. However, seismic performances of these buildings may be low after performance analyses. Rebuild of the existing buildings may not be applicable due to constructional and economical points. In this case, strengthening comes forward as a significant approach in terms of providing life safety level of the existing buildings. Thus, studies have been performed by applying different strengthening methods on the existing buildings [Formisano et al., 2020; Karki et al., 2020; Sayed et al., 2020]. After the buildings are strengthened, performance analyses are performed to check the global performance level.

In this study, performance levels of the 3, 5 and 8 storey existing rc buildings which covers the important part of the residential buildings in the building stock are investigated at first. Damage situations of the structural members are evaluated for each building according to TBEC-2018 and ASCE [TBEC-2018, 2018; ASCE 7-16, 2017; ASCE 41-13, 2014]. Sap2000 structural analysis software is used in the analyses [Sap2000, 1998]. After deciding the performance levels of the existing buildings, two different strengthening approaches such as jacketing of the specific columns and adding steel bracings are utilized. Non-linear analyses are performed once again for the strengthened buildings and change in the damage ratios of the structural members and performance levels of the structures are determined. The results are comparatively presented by figures and tables for each seismic code. Finally, it is considered that results of this study will contribute to the literature about performance assessment of the rc buildings and strengthening approaches.

2. Materials and methods

2.1. *Properties of the buildings*

The rc frame buildings have 3, 5 and 8 storeys respectively. Floor plans are common for the buildings as presented in Figure 1. The frame systems have typical column-beam sections without any shear-walls. Each story height is taken to be 3.0 m. While total span length is 21.0 m in the x direction, it is 14.5 m in the y direction (Karal, 2021).

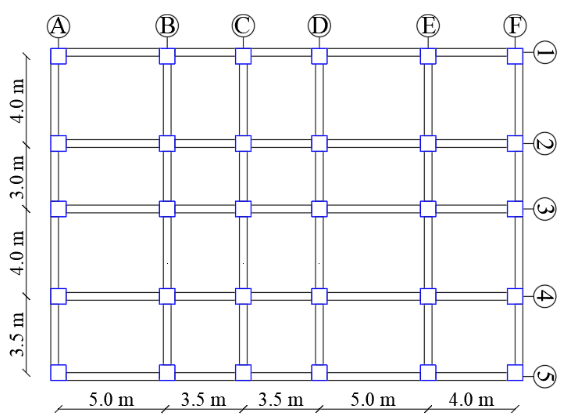


Figure 1. Floor plan of the buildings (Karal, 2021).

Three column sections are used in the existing buildings. Column section sizes and steel bars are different for corner, front and inner columns. However, beam section sizes are taken as constant. Cracked section rigidity coefficients are considered as 0.70 and 0.35 for the columns and beams respectively as defined in both codes. Slab thickness is 13 cm for the buildings. In addition, live load is applied as 1.5 kN/m^2 on the slabs in the top floor and it is considered as 2.0 kN/m^2 for the rest of the floors according to Turkish Standard-498 [TS-498, 1997]. Furthermore, thicknesses of the external and inner walls are 20 and 10 cm respectively. To improve the seismic behavior of existing buildings, two strengthening procedures are applied to the existing rc frame buildings. While jacketing technique is used for inner columns of the existing buildings, steel braces are symmetrically placed in both directions. Section sizes of the structural members for the existing and strengthened buildings are given in Table 1.

Table 1. Properties of the specimens.

Structural members	Existing buildings	Strengthened buildings
Beams	250x450 mm	
Corner columns	500x500 mm	
Front columns	300x500 mm	
Inner columns	400x400 mm	
Inner columns after jacketing	–	600x600 mm
Diagonal braces	–	CHS 219.1x8 mm

After the latest improvement, minimum longitudinal reinforcement ratio of the rc columns is 0.01 and minimum compressive strength of concrete is defined as 25 MPa in TBEC-2018. However, lower compressive strengths and reinforcement ratios are encountered in the assessment of existing buildings constructed in previous years. For instance, longitudinal reinforcement ratio is lower than 0.01 for some columns in the existing buildings. In this study, while compressive strength of concrete is taken as 20 MPa, yield strength of the steel bars and stirrups is 420 MPa. While diameter of the stirrups is 8 mm, stirrup spacing is 15 and 20 cm for columns and beams respectively. Furthermore, steel grade of bracing members is taken to be S235JR whose yield strength value is 235 MPa. While details of the beam sections in the supports and column sections of the existing buildings are shown in Figure 2, details of the inner column after jacketing and bracing member are presented in Figure 3.

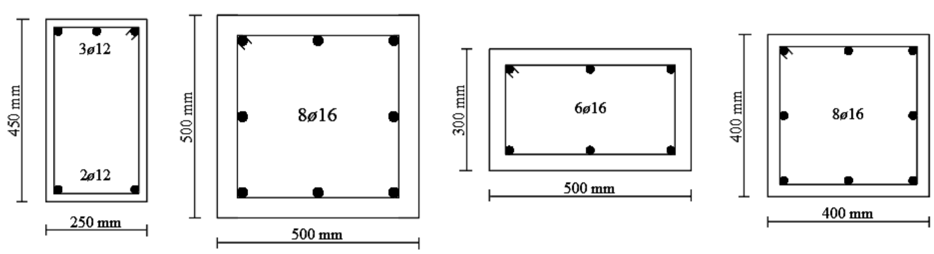


Figure 2. Section details for the existing buildings.

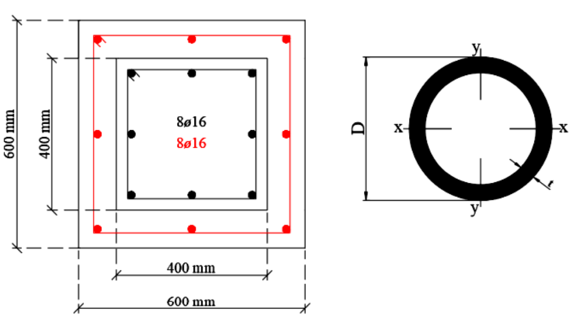


Figure 3. Section details for the strengthened buildings.

Concentrically braced frame systems that are efficient in in resisting lateral forces due to high stiffness and rigidity are usually utilized in practice. In these systems, the steel braces intersect at the node of center point and they provide complete truss action. Braced members are generally utilized with two diagonal supports placed in an X shaped manner. In addition, these members can be used in bridge supports, structural foundations as well as buildings. While geometrical properties are diameter (D), wall thickness (t) and area (A), mechanical properties are moment of inertia (I) and radius of gyration (i) given in Table 2.

Table 2. Characteristics of the braces.

Section	Geometrical properties	Mechanical properties
CHS 219.1x8 mm	D = 219.1 mm	$I_x = I_y = 2959.63$ cm^4
	t = 8 mm	$I_t = 5916.26$ cm^4
	A = 5306 mm^2	$i_x = i_y = 7.5$ cm

Three dimensional models of the both existing and strengthened buildings are presented in Figures 4 and 5 respectively. Inner columns after jacketing operation and symmetrically placed bracing members in both directions are shown in the strengthened buildings. Thus, the lateral deformation capacity and seismic performances of the existing buildings are improved.

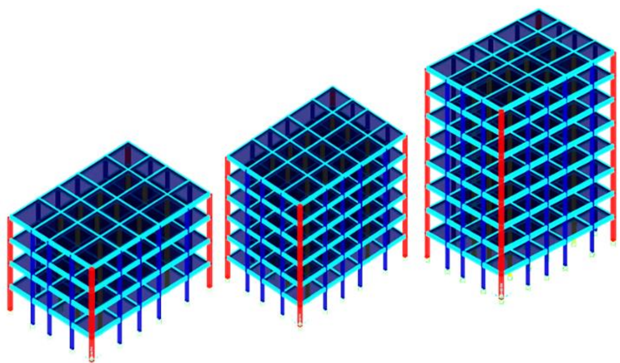


Figure 4. Existing buildings (Karal, 2021).

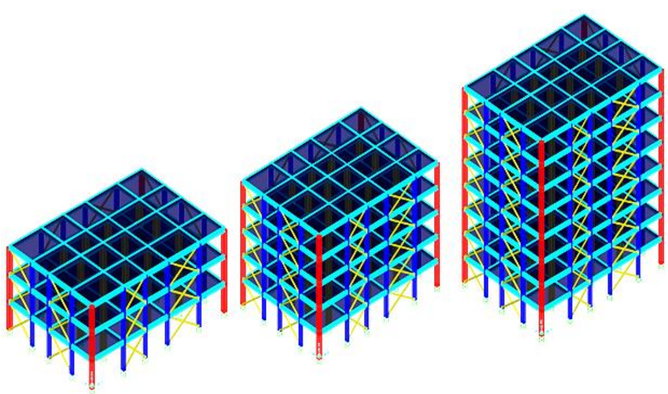


Figure 5. Strengthened buildings (Karal, 2021).

Modal analysis is generated for each of the existing and strengthened buildings in the software. Afterwards, first period values of the vibration modes and effective mass ratios are determined. The analysis results for both directions are presented in Table 3.

Table 3. Modal analysis results.

Story number	Building type	Mode	Period (sec)		Effective mass ratio (%)	
			x direction	y direction	x direction	y direction
3 Storey	Existing	1	0.57	–	84	–
		2	–	0.49	–	81
	Strengthened	1	0.23	–	85	–
		2	–	0.22	–	84
5 Storey	Existing	1	0.98	–	82	–
		2	–	0.86	–	80
	Strengthened	1	0.42	–	79	–
		2	–	0.40	–	78
8 Storey	Existing	1	1.59	–	81	–
		2	–	1.41	–	79
	Strengthened	1	0.76	–	75	–
		2	–	0.74	–	75

3. Non-Linear analysis

Seismic performances of the existing and strengthened buildings are evaluated by non-linear static pushover procedure that is considered as an effective way to investigate the performance levels of the buildings. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern and it is utilized as a series of incremental analysis which is generated to obtain the structural behavior. The behavior is characterized by a capacity curve that exhibits the relationship between the base shear and the roof displacement. As, fast and reliable responses can be determined by static pushover analysis, it is commonly used in several studies instead of performing elastic static or dynamic solution procedures (Golghate et al., 2013). By considering the capacity and demand spectrum curves as shown in Figure 6, the lateral deformation and damage situations of the structural members are evaluated at the value of target displacement which is also entitled as performance point.

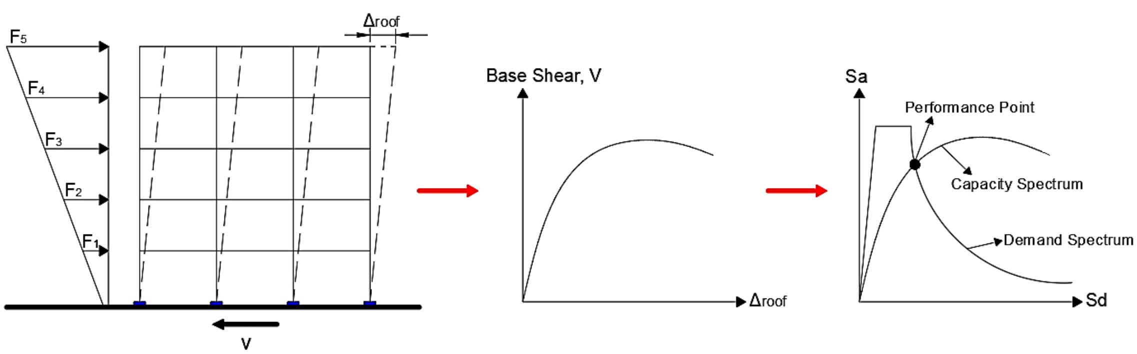


Figure 6. Determination of the performance point.

The global performance level of the structure depends on the damage ratios of the structural members. Recently, displacement based methods have been preferred to force-based methods owing to direct relationship between performance objectives and structural damage levels. The general analysis steps from modelling phase to deciding performance level of the structure is given in Figure 7.

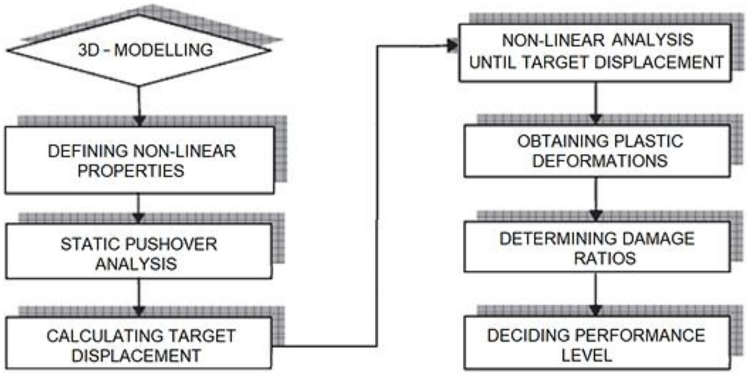


Figure 7. Analysis steps.

The lumped plasticity approach is adopted and an inelastic behavior is determined by plastic hinges at two ends of columns and beams in the software. Properties of the plastic hinges are defined to obtain the moment-rotation relationship of the primary structural members (İnel & Özmen, 2006). The certain points in the idealized force-deformation relationship of a plastic hinge are seen in Figure 8. First of all, unloaded situation of the hinge deformation is presented by point A. Yield of a structural member occurs as the strength value of F_y in a hinge is reached. Beyond point B, deformation is effective on the force of the hinge. Collapse situation is reached by the plastic hinge when the displacement reaches point C. Finally, the plastic the plastic hinge loses its strength and the remaining capacity is considered on point D and then it continues deforming until reaching point E with the remaining capacity.

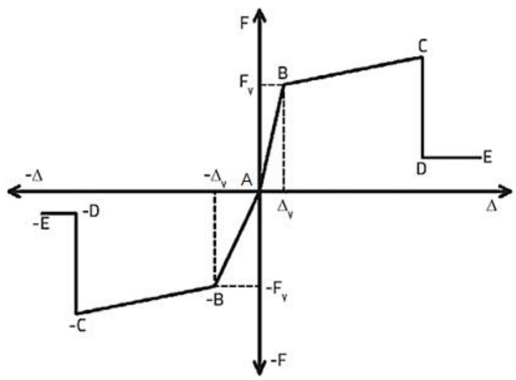


Figure 8. Properties of the plastic hinge (Erdem, 2016).

Location of the plastic hinges which are defined at two ends of the structural members is presented in Figure 9. In addition, the length of the plastic hinge is symbolized by Lp in the figure.

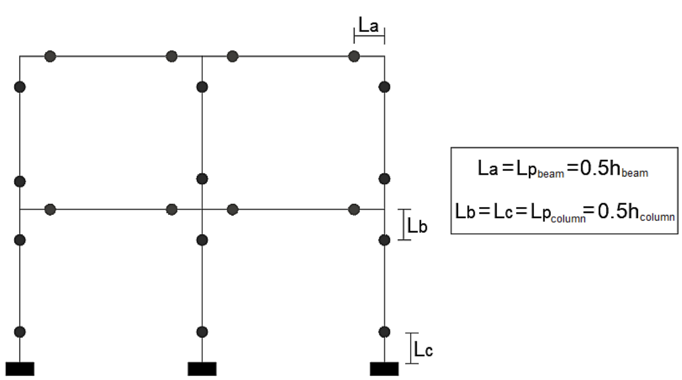


Figure 9. Plastic hinges in the frame system.

There are three global performance levels that are described in the seismic codes or guidelines as shown in Figure 10. In immediate occupancy (IO) performance level, small light damages such as cracks may be observed in non-structural members. However, there are no damages happened in structural members under seismic effect. In life safety (LS) performance level, some limited damages can be seen in structural members. Nevertheless, the lateral stiffness and rigidity of all structural members are still preserved. Although little deformations may occur, they are not visually noticed. In collapse prevention (CP) performance level, permanent deformations are observed in the structural members that lose their strength and lateral stiffness. However, total collapse is still restrained. Finally, total collapse occurs beyond CP level.

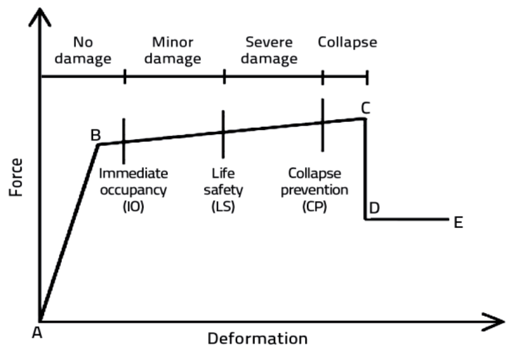
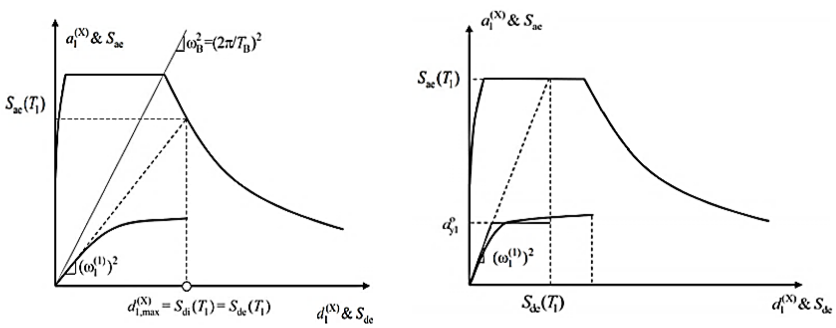


Figure 10. Performance levels (Erdem, 2016).

3.1. *Performance analysis according to TBEC-2018*

The coordinates of the capacity curve are converted into modal response acceleration - modal response displacement to determine the value of target displacement according to TBEC-2018 as presented in Figure 11. By using the equation of ($S_{di}=C_R x S_{de}(T_1)$), non-linear spectral displacement, $S_{di}(T_1)$ is calculated. C_R and $S_{de}(T_1)$ represent the spectral displacement ratio and the elastic design spectral displacement respectively. Natural vibration period value of the building (T_1) and the corner period value (T_B) are effective on the value of C_R. There are two cases for modal capacity diagram as presented in Figure 11. While Figure 11 (a) is used when value of T_1 is bigger than T_B, Figure 11 (b) is utilized as value of T_1 is less than or equal to value of T_B.



(a) (b)

Figure 11. Modal capacity diagrams (TBEC-2018, 2018).

Local soil class is utilized as D which is comprised of dense sand, gravel or very solid clay layers. Eqs. (1) and (2) are used to calculate the design spectrum parameters that are given in Table 4. While S_s and S_1 are the spectral acceleration coefficients, F_s and F_1 are the local soil effect coefficients in the equations. All of these coefficients are determined from seismic hazard map. Finally, S_{DS} and S_{D1} which symbolize the design spectral acceleration coefficients for the short period and 1.0 sec period are calculated according to spectral acceleration and local soil effect coefficients. In addition, T_A and T_B represent the corner periods of the horizontal elastic design spectrums.

$$S_{DS} = S_S \times F_S \quad (1)$$

$$S_{D1} = S_1 \times F_1 \quad (2)$$

Table 4. Design parameters according to TBEC.

Soil class	T_A	T_B	S_S	S_1	F_S	F_1	S_{DS}	S_{D1}
D	0.095	0.476	1.120	0.273	1.052	2.054	1.178	0.561

Deformation limits due to concrete and steel strains (ε_c and ε_s) are utilized to determine the damage situations of the columns and beams in the view of Figure 10. The equations for the performance levels are given between Eqs. (3) and (9). While ω_w represents the mechanical reinforcement ratio of the effective confinement reinforcement, ε_{su} is the ultimate strain of the steel reinforcement in the equations below. In addition, specific coefficients are used to determine the damage limits of the steel bracing members under compression and tension according to TBEC-2018.

$$\varepsilon_c^{(CP)} = 0.0035 + 0.04\sqrt{\omega_{we}} \leq 0.018 \quad (3)$$

$$\varepsilon_s^{(CP)} = 0.4\varepsilon_{su} \quad (4)$$

$$\varepsilon_c^{(LS)} = 0.75\varepsilon_c^{(CP)} \quad (5)$$

$$\varepsilon_s^{(LS)} = 0.75\varepsilon_s^{(CP)} \quad (6)$$

$$\varepsilon_c^{(IO)} = 0.0025 \quad (7)$$

$$\varepsilon_s^{(IO)} = 0.0075 \quad (8)$$

$$\Delta = \frac{Pl}{EA} \quad (9)$$

3.2. *Performance analysis according to ASCE*

Generally, analysis steps in ASCE are similar to those in TBEC-2018. Yet, there are still some differences. For example, a certain coefficient with a value of 0.3 is multiplied with live loads for residential buildings in TBEC-2018. However, effective seismic weights are determined by taking account of the dead loads in ASCE. Thus, weights of the buildings are differently considered according to both codes. Besides, the buildings are classified according to their risk categories in ASCE. Risk category of the existing rc buildings is II that is defined as buildings and other structures representing high risk to human life in the event of failure in the standard. The importance factor of the building (I_e) which is associated with the risk category is considered as 1.00. Interactive web application is used to determine the S_S and S_1 that are the spectral acceleration parameters. In addition, the parameters as F_a and F_v are obtained for soil class D which is defined as stiff soil in ASCE. S_{MS} and S_{M1} that represent the short period and 1.0 second period coefficients are calculated by using these parameters according to Eqs. (10) and (11). Eventually, spectral response acceleration values symbolized by S_{DS} and S_{D1} are determined by Eqs. (12) and (13). The values of these parameters are seen in Table 5.

$$S_S = S_{MS} \times F_a \quad (10)$$

$$S_{D1} = S_1 \times F_1 \quad (11)$$

$$S_{DS} = \frac{2}{3} S_{MS} \quad (12)$$

$$S_{D1} = \frac{2}{3} S_1 \quad (13)$$

Table 5. Design parameters according to ASCE.

Soil class	T_0	T_s	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.089	0.447	1.124	0.274	1.09	2.00	1.225	0.548	0.817	0.365

4. Results

The non-linear analyses are performed for each of the existing and strengthened buildings with respect to TBEC and ASCE in the software. Afterwards, target displacements are calculated for both directions as given in Table 6. It is seen that the calculated target displacement values of the existing buildings are bigger than the values for the strengthened buildings as expected. In addition, bigger displacement values are obtained for TBEC. To give an example, target displacements for the 5 storey existing and strengthened buildings are shown on the capacity curves according to TBEC for both directions in Figures 12 and 13.

Table 6. Target displacements according to codes.

Story number	Building type	Direction	TBEC		ASCE	
			Base shear force (kN)	Target displacement (mm)	Base shear force (kN)	Target displacement (mm)
3 Storey	Existing	x	2218.7	8.0	2136.2	6.7
		y	1750.2	7.1	1708.8	5.9
	Strengthened	x	8554.5	3.2	8527.3	2.9
		y	9145.0	2.9	9085.4	2.5
5 Storey	Existing	x	1834.9	15.4	1790.8	12.7
		y	1642.2	14.7	1549.1	12.3
	Strengthened	x	7157.0	5.3	7243.7	4.2
		y	7208.1	5.3	7334.2	4.5
8 Storey	Existing	x	1802.7	22.1	1776.7	17.3
		y	1454.3	19.7	1389.3	16.0
	Strengthened	x	7444.5	10.5	7562.4	7.8
		y	7601.7	10.3	7743.6	7.5

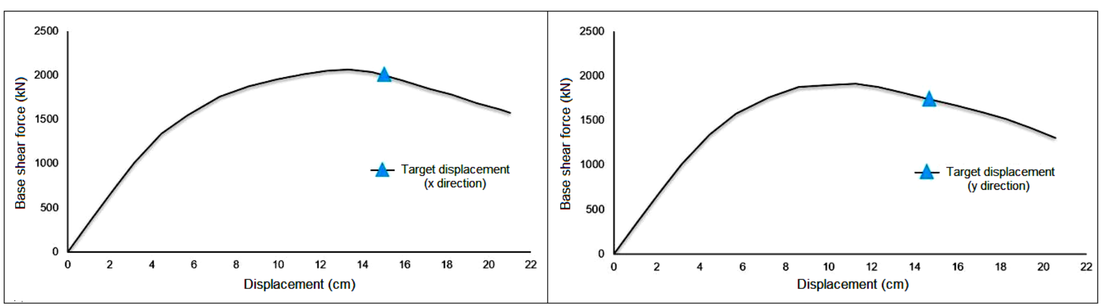


Figure 12. Target displacements for the 5 storey existing building according to TBEC-2018.

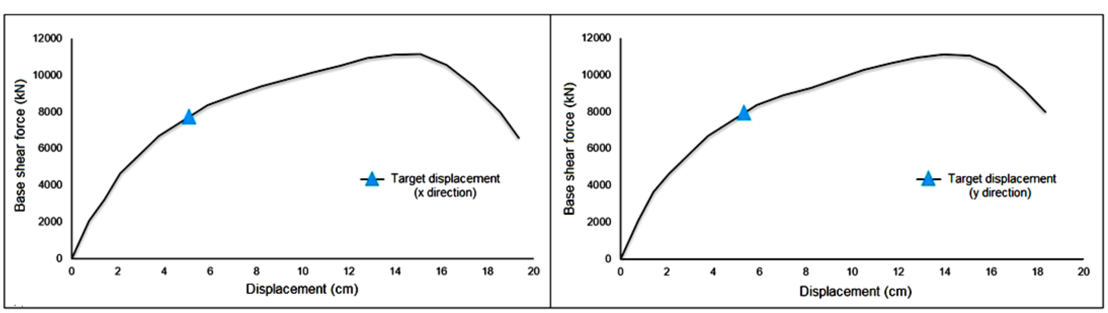


Figure 13. Target displacements for the 5 storey strengthened building according to TBEC-2018.

Both existing and strengthened buildings are pushed to the calculated target displacement values to obtain the damage situations of the structural members according to TBEC and ASCE. Thus, plastic hinges in the structural members are observed. After applying rc jacketing and steel bracing members to the existing buildings, there aren't any damages occurred in structural members beyond LS performance level. To give an example, plastic hinges at target displacements for 5 storey existing and strengthened buildings are exhibited for the related directions according to seismic codes in Figures 14 and 15.

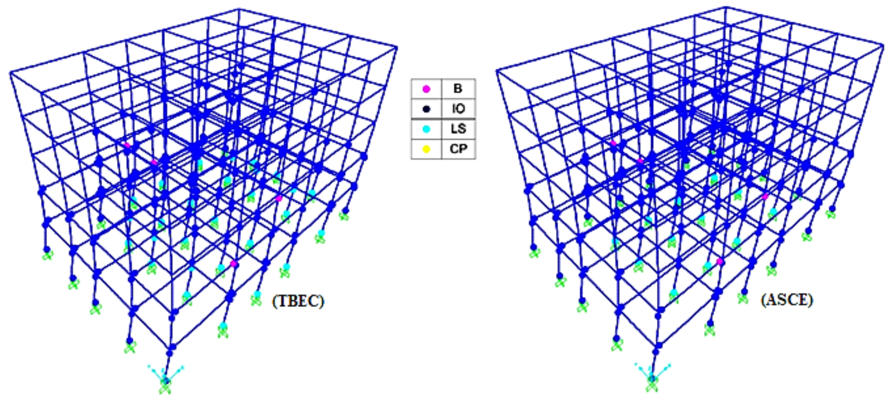


Figure 14. Plastic hinge distributions for the 5 storey existing building at x direction.

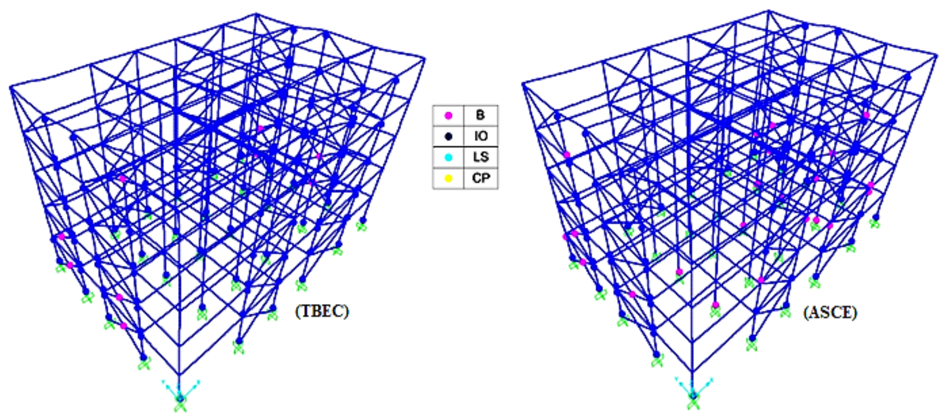


Figure 15. Plastic hinge distributions for the 5 storey existing building at y direction.

After considering the plastic hinges at the calculated target displacements, damage situations of the structural members are determined for both codes. Damage levels of the beams and columns for the 3, 5 and 8 storey existing and strengthened buildings are given between Tables 7-12. Due to the bigger target displacement values, more damages occur in the structural members according to TBEC for each building.

Table 7. Damage levels for 3 storey buildings at x direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	19	6	–	25	–	–	21	4	–	25	–	–
	2	25	–	–	25	–	–	25	–	–	25	–	–
	3	25	–	–	25	–	–	25	–	–	25	–	–
Columns	1	–	30	–	–	30	–	3	27	–	10		–
	2	3	27	–	22	8	–	27	3	–	28		–
	3	19	11	–	30	–	–	25	5	–	30	–	–

Table 8. Damage levels for 3 storey buildings at y direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	19	6	–	25	–	–	21	4	–	25	–	–
	2	25	–	–	25	–	–	25	–	–	25	–	–
	3	25	–	–	25	–	–	25	–	–	25	–	–
Columns	1	–	30	–	–	30	–	3	27	–	10		–
	2	3	27	–	22	8	–	27	3	–	28		–
	3	19	11	–	30	–	–	25	5	–	30	–	–

Table 9. Damage levels for 5 storey buildings at x direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	14	11	–	25	–	–	17	8	–	25	–	–
	2	19	6	–	25	–	–	22	3	–	25	–	–
	3	25	–	–	25	–	–	25	–	–	25	–	–
	4	25	–	–	25	–	–	25	–	–	25	–	–
	5	25	–	–	25	–	–	25	–	–	25	–	–
Columns	1	–	8	22	6	24	–	–	18	12	7	23	–
	2	–	30	–	22	8	–	–	30	–	24	6	–
	3	12	18	–	26	4	–	16	14	–	28	2	–
	4	18	12	–	30	–	–	22	8	–	30	–	–
	5	30	–	–	30	–	–	30	–	–	30	–	–

Table 10. Damage levels for 5 storey buildings at y direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	21	3	–	24	–	–	22	2	–	24	–	–
	2	22	2	–	24	–	–	23	1	–	24	–	–
	3	24	–	–	24	–	–	24	–	–	24	–	–
	4	24	–	–	24	–	–	24	–	–	24	–	–
	5	24	–	–	24	–	–	24	–	–	24	–	–
Columns	1	–	25	5	–	30	–	–	28	2	8	22	–
	2	–	30	–	18	12	–	–	30	–	20	10	–
	3	4	26	–	20	10	–	8	22	–	23	7	–
	4	17	13	–	20	10	–	19	11	–	25	5	–
	5	28	2	–	22	8	–	30	–	–	30	–	–

Table 11. Damage levels for 8 storey buildings at x direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	15	10	–	25	–	–	20	5	–	25	–	–
	2	17	8	–	23	2	–	21	4	–	21	2	–
	3	21	4	–	21	4	–	23	2	–	20	5	–
	4	25	–	–	19	6	–	25	–	–	19	6	–
	5	25	–	–	20	5	–	25	–	–	21	4	–
	6	25	–	–	21	4	–	25	–	–	21	4	–
	7	25	–	–	21	4	–	25	–	–	25	–	–
	8	25	–	–	23	2	–	25	–	–	25	–	–
Columns	1	–	–	30	3	27	–	–	5	25	6	24	–
	2	–	30	–	22	8	–	–	30	–	22	8	–
	3	5	25	–	22	8	–	9	21	–	22	8	–
	4	18	12	–	22	8	–	20	10	–	24	6	–
	5	18	12	–	26	4	–	24	6	–	28	2	–

	6	28	2	–	28	2	–	30	–	–	30	–	–
	7	30	–	–	30	–	–	30	–	–	30	–	–
	8	30	–	–	30	–	–	30	–	–	30	–	–

Table 12. Damage levels for 8 storey buildings at y direction.

Structural members	Story number	TBEC						ASCE					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	20	4	–	24	–	–	20	4	–	24	–	–
	2	20	4	–	24	–	–	21	3	–	24	–	–
	3	23	1	–	22	2	–	24	–	–	22	2	–
	4	24	–	–	20	4	–	24	–	–	22	2	–
	5	24	–	–	20	4	–	24	–	–	20	4	–
	6	24	–	–	20	4	–	24	–	–	20	4	–
	7	24	–	–	20	4	–	24	–	–	20	4	–
	8	24	–	–	22	2	–	24	–	–	22	2	–
Columns	1	–	15	15	–	30	–	–	17	11	6	24	–
	2	–	30	–	14	16	–	–	30	–	18	12	–
	3	–	30	–	16	14	–	–	30	–	20	10	–
	4	4	26	–	18	12	–	22	8	–	24	6	–
	5	19	11	–	19	11	–	26	4	–	24	6	–
	6	23	7	–	24	6	–	30	–	–	30	–	–
	7	30	–	–	30	–	–	30	–	–	30	–	–
	8	30	–	–	30	–	–	30	–	–	30	–	–

The damage ratios of the beams and columns are visually presented for both codes at x and y directions between the Figures 16-21. It is seen that the proposed strengthening techniques have improved the seismic capacity of the existing buildings. There aren't any structural members in CP performance level for the strengthened buildings.

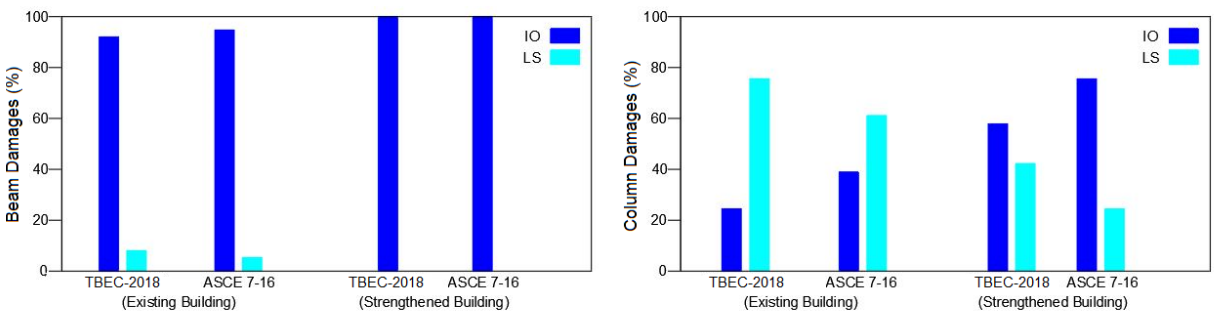


Figure 16. Damage ratios for 3 storey buildings at x direction.

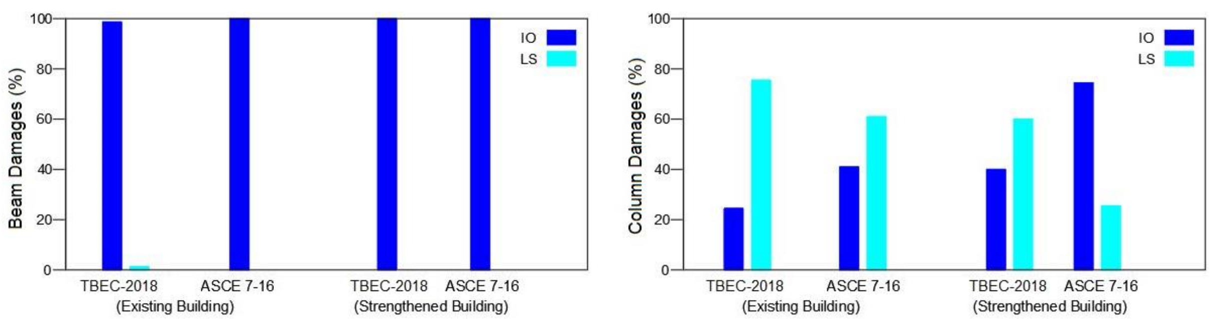


Figure 17. Damage ratios for 3 storey buildings at y direction.

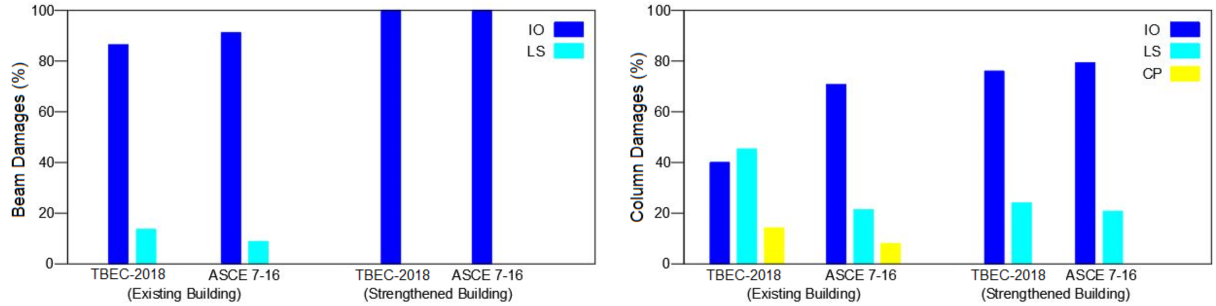


Figure 18. Damage ratios for 5 storey buildings at x direction.

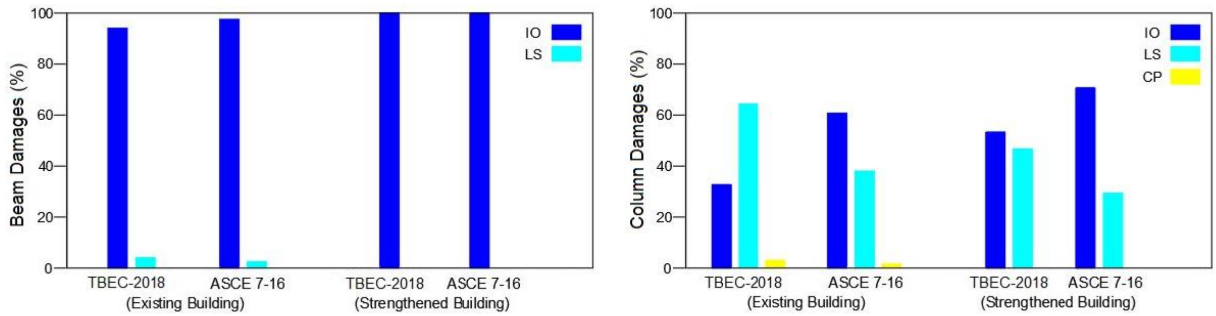


Figure 19. Damage ratios for 5 storey buildings at y direction.

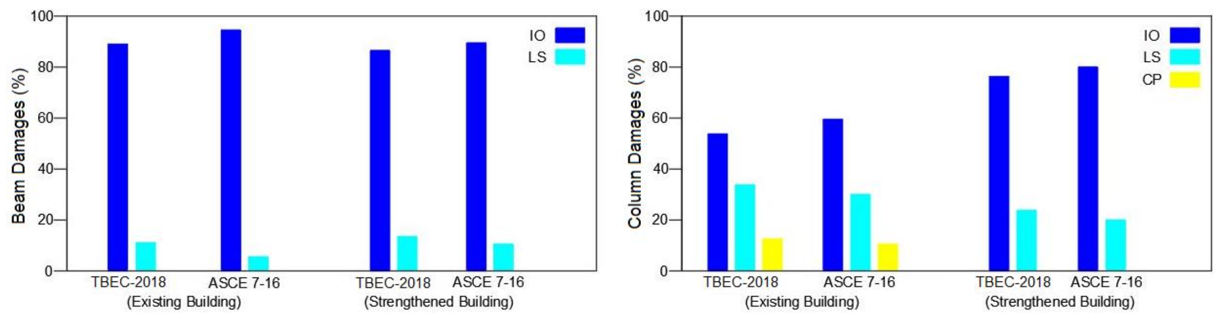


Figure 20. Damage ratios for 8 storey buildings at x direction.

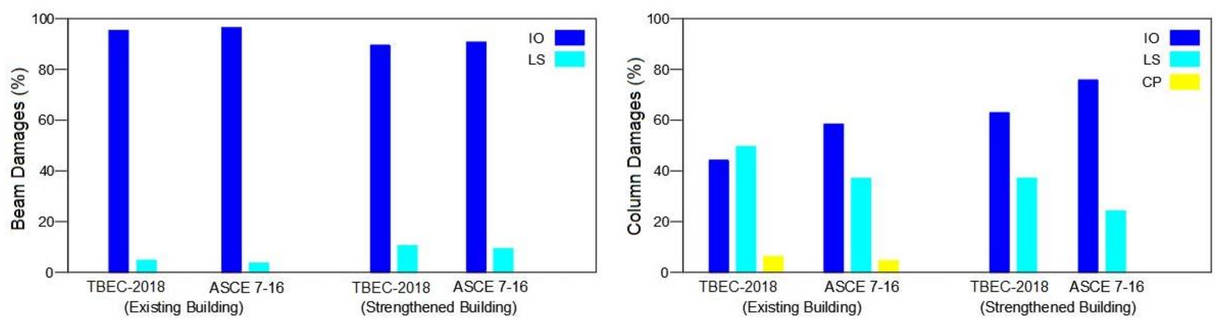


Figure 21. Damage ratios for 8 storey buildings at y direction.

Finally, story drift ratios are determined for each building in the critical direction where bigger values are obtained and shown in the figures below. When the results are evaluated, lateral displacement values are reduced after applying strengthening procedures according to both codes.

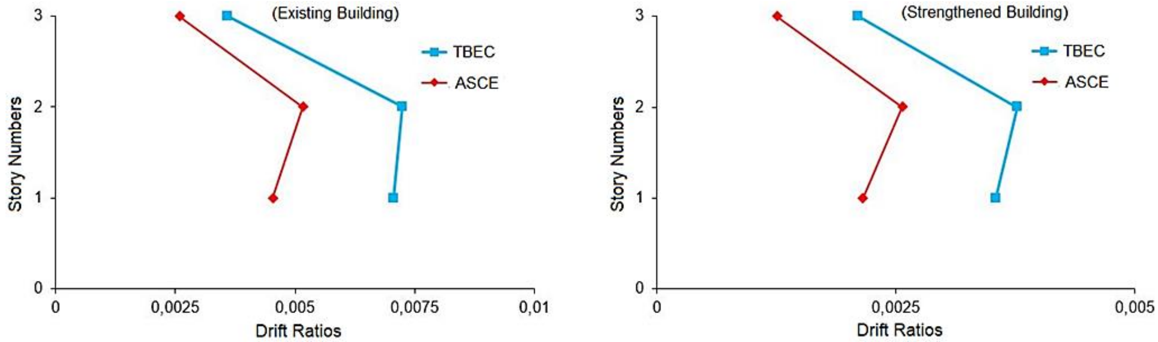


Figure 22. Story drift ratios for 3 storey buildings.

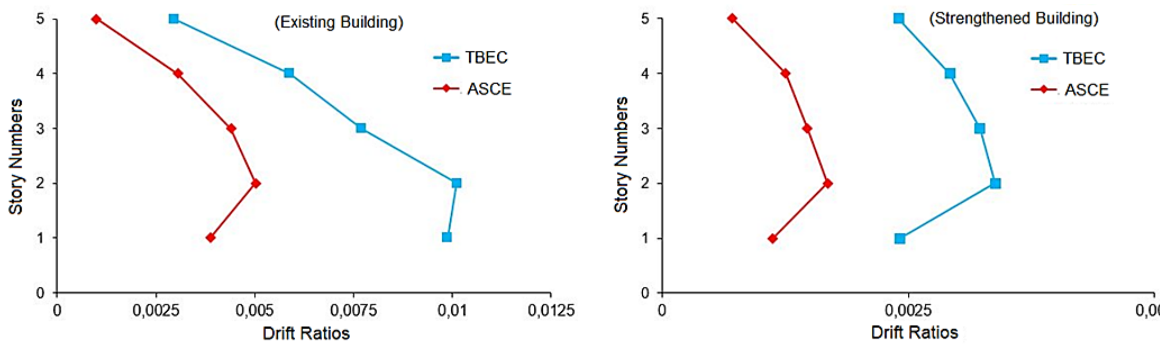


Figure 23. Story drift ratios for 5 storey buildings.

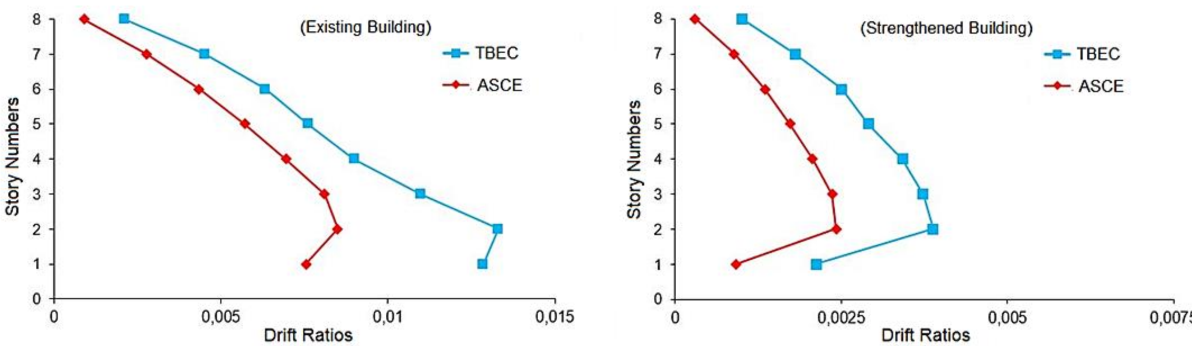


Figure 24. Story drift ratios for 8 storey buildings.

5. Conclusions and comments

As many buildings have experienced different levels of damages due to the past earthquakes in the world, assessing the seismic capacities of the existing buildings in accordance with the present seismic codes has become a significant task in performance based evaluation that consists of the combination of design, analysis and evaluation steps. The non-linear static analysis represents the current trend in structural engineering in terms of evaluating the seismic performances of the existing buildings according to seismic codes. The analysis provides adequate information on the structural systems and yields reasonable prediction of the seismic behavior.

It may not be possible to provide life safety performance level of the existing rc buildings that have not been constructed by modern seismic codes. Generally, these buildings are not able to provide adequate strength and lateral resistance. In this situation, strengthening methods have become important alternatives to improve the seismic performance of the existing rc buildings. The main purpose of the strengthening is to ensure that the displacement demand of the building is to be kept below its displacement capacity. To achieve this situation, displacement capacity of the building is improved or the expected displacement demand of the building is reduced during the earthquake.

In this study, non-linear static analyses have been performed for 3, 5 and 8 storey rc buildings according to TBEC and ASCE in the first place. Although the existing buildings have different story numbers, material properties, section details and floor plans are taken same for each building. Damage situations of the structural members are determined and performance levels of the buildings are decided according to both codes. It is stated that insufficiency performance levels result from the flexural damages in the structural members. Afterwards, the buildings are strengthened by two different techniques such as adding symmetrical steel braces and rc jacketing of inner columns. By this way, it is aimed to increase the lateral stiffness and reduce lateral displacements. Main conclusions are listed below.

1. When damage ratios of the 3 storey existing rc building is investigated, it is seen that there are no beams and columns in the CP performance level for both directions. However, the same strengthening procedures are also applied to the 3 storey to obtain the changes in damage ratios of the structural members. After performing the analysis for the strengthened building, it is seen that all beams provide IO performance level. In addition, while number of the columns in the LS level has decreased, the columns in the IO level have increased according to both seismic codes;
2. As the story number of the existing buildings increases to 5 and 8, damage ratios of the structural members in the CP level have increased. Although the beams of the 5 and 8 storey existing buildings provide IO and LS performance levels, many columns in the first storey are in CP level according to seismic codes. When the first story columns of the 5 storey existing building are investigated in the critical direction, 73% and 40% of the columns are in CP performance level according to TBEC and ASCE. On the other hand, the ratios of the first story columns in CP level rises to 100% and 83% for the 8 storey building according to TBEC and ASCE respectively. However, all columns provide IO and LS performance levels after applying strengthening procedures;
3. Because one of the important targets of the strengthening is limiting the lateral displacement, story drift ratios are also evaluated for each building. It is seen that story drifts have significantly decreased in the strengthened buildings for both codes. Bigger target displacements are calculated according to TBEC. Thus, more conservative results are after performing analyses. Consequently, it is considered that this study will contribute to the engineers and researchers investigating the seismic performance analysis and strengthening methods of rc buildings. Besides, it is suggested for the future studies that different strengthening techniques may be applied to improve the seismic performances of the existing rc buildings according to other seismic codes.

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