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Performance assessment and strengthening proposal of an existing building

Authors:

Assoc.Prof. **R. Tuğrul Erdem**, PhD. CE
Manisa Celal Bayar University, Turkey
Department of Civil Engineering
tugrul.erdem@cbu.edu.tr
Corresponding author

Koçali Karal, MSc. CE
Manisa Celal Bayar University, Turkey
Department of Civil Engineering
191292017@ogr.cbu.edu.tr

Professional paper

R. Tuğrul Erdem, Koçali Karal

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Due to major-scale earthquakes, revisions of earthquake resistant structural design methods have periodically been made in seismic codes. It is known that many existing buildings are located in active seismic zones in all parts of the world. Severe damage to structural members, with partial or total collapse of buildings, has been observed in past earthquakes. Consequently, the evaluation of seismic performance and strengthening techniques of the existing buildings according to various seismic codes has become a global issue in structural engineering. With the development of computer technology, non-linear methods have been offering increasingly reliable evaluation procedures in the performance-based assessment of buildings. In this study, non-linear performance analysis of an existing typical mid-rise reinforced concrete building is first performed according to Turkish Building Earthquake Code-2018 and American Standard, ASCE. After evaluation of damage to structural members, the building is strengthened by steel braces and the analysis is performed once again. The SAP2000 finite elements analysis software is utilized in the solutions. Damage ratios of structural members as well as modal properties and storey drift ratios are determined and compared according to both codes, and it is concluded that the proposed strengthening method could be a significant alternative in such buildings.

Key words:
reinforced concrete building, performance analysis, seismic codes, strengthening

Stručni rad

R. Tuğrul Erdem, Koçali Karal

Ocjena ponašanja i prijedlog pojačanja postojeće građevine

Kao odgovor na pojavu snažnih potresa u seizmičkim se propisima provode revizije metoda za projektiranje građevina otpornih na seizmička djelovanja. Poznato je da su mnoge postojeće građevine smještene u seizmički aktivnim područjima diljem svijeta. Potresi uzrokuju znatna oštećenosti elemenata konstrukcije, te djelomično ili potpuno rušenje građevina. Zbog toga se, u okviru različitih seizmičkih propisa, diljem svijeta provodi ocjenjivanje seizmičkog ponašanja i postupaka pojačanja postojećih građevina. Zahvaljujući razvoju računalne tehnologije, nelinearne metode danas nude sve pouzdanije postupke za određivanje ponašanja građevina. U ovom se radu najprije provodi nelinearna analiza ponašanja postojeće tipične armiranobetonske građevine srednje visine prema turskom seizmičkom propisu – 2018 te prema američkoj normi ASCE. Nakon ocjene oštećenosti konstrukcijskih elementa, građevina je pojačana čeličnim razuporama te je zatim analiza ponovljena. U postupku je korišten računalni program za analizu konačnih elemenata SAP2000. Koeficijenti oštećenosti konstrukcijskih elemenata te modalna svojstva i međukatni pomaci, određeni su i uspoređeni prema oba spomenuta propisa, te je zaključeno da predložena metoda pojačanja može biti prihvatljivo alternativno rješenje za takve građevine.

Ključne riječi:
armiranobetonska građevina, analiza učinkovitosti, seizmički propisi, pojačanje

1. Introduction

An earthquake is a result of a sudden release of seismic energy stored in the crust of the earth where seismic waves are formed. Earthquakes rank among the most effective natural disasters which lead to the loss of life and property. According to the United States Geological Survey, a total of 122 earthquakes with a magnitude of 6 or above occurred worldwide in 2020. Serious damage, collapsed buildings, and loss of lives occurred as a result of these earthquakes. This situation points to the significance of the design of earthquake resistant buildings.

Reinforced concrete (RC) buildings form the majority of the building stock in the world. High ductility, durability and rigidity, long service life, resistance to fire, and ease of construction, are the main advantages of RC buildings. That is why such buildings are increasingly being built in all parts of the world. However, low quality of concrete is an important cause of structural damages to the existing RC buildings that were constructed before the advent of the ready-mixed concrete technology. So, substantial damages can be registered in RC structures after major earthquakes. Seismic safety of RC buildings has been analysed according to various codes in the literature [1-6].

The performance-based design and non-linear analysis are important subjects in structural engineering. Performance evaluation involves the use of specific techniques, from the design to the verification of analysis. Design criteria are stated in respect to specific objectives, relating to seismic effects, to be achieved in the performance-based design. In this approach, the performance of a structural system is evaluated by relating performance objectives to an appropriate damage level. Although a more detailed study is required when performing numerical analysis of a structure by non-linear analysis methods, more accurate results are obtained compared to approaches based on linear analysis [7-11]. Studies also reveal that the performance-based design is expected to become more prevalent in seismic codes in the near future.

As the existing structures have been affected by several earthquakes, their seismic performance is determined by engineers. Generally, seismic codes require that the life safety performance level shall be provided and that the total collapse must be prevented in large scale earthquakes. However, damage may occur to structural members. After evaluating the level of damage to structural members, various strengthening techniques can be applied to structural systems [12-16].

Because of some economic or constructional reasons, it is not always possible to rebuild the existing building whose seismic safety has been compromised by seismic action. In this case, the building needs to be strengthened by a proper technique. The strengthening technique should be rapid to perform, and only minor damage to the structural system of the building should be allowed. In such situations, strengthening by steel bracings has proven to be an effective procedure to improve seismic behaviour of existing buildings [17-21].

Non-linear analysis of an existing mid-rise RC building, representative of a significant part of residential buildings in the building stock, is initially performed in this study. Seismic performance of the building is evaluated by considering levels of damage to structural members according to the Turkish Building Earthquake Code-2018 (TBEC-2018) and ASCE [22, 23, 29]. Afterwards, selected axes of the building are strengthened by steel bracing members and the analysis is performed once again. The analysis is performed by SAP2000 finite elements software that is widely used for finding non-linear solutions [24]. Finally, period values, mass ratios, damage situations and drift ratios of the existing and strengthened building are compared according to both codes. It is thought that the obtained results will contribute to the literature about structural engineering.

2. Description of the building

The 5 storey RC frame building is selected as case study. The height of each storey is 3 m, and a total floor area of the building is 221 m². While the total length is 17 m in the x direction, it is 13 m in the y direction. The storey plan of the existing building is presented in Figure 1.

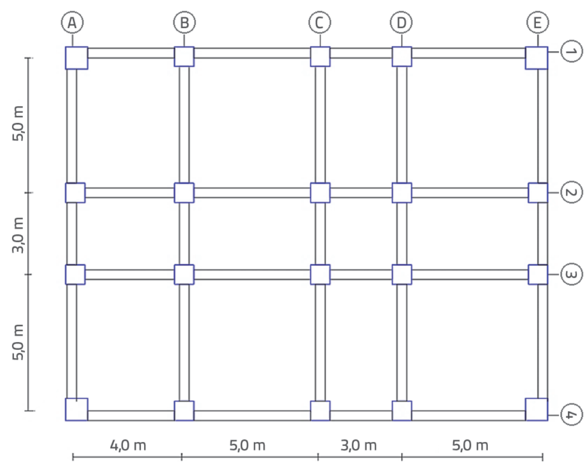

Figure 1. Plan view of the building

The existing building is formed of RC columns, beams and slabs. Two types of column sections are used in the RC frame building. On the other hand, dimensions of the beams are constant. Cracked section rigidity coefficients of 0.70 and 0.35 are used for columns and beams, respectively, as suggested in TBEC-2018 and ASCE 7-16. For that reason, bigger lateral displacements occur and higher period values are obtained after analysis. The slab thickness is taken to be 13 cm for all storeys. While the live load of 1.5 kN/m² is used for the top floor, the value of 2.0 kN/m² was adopted for other floors, as defined in TS-498 [25]. External and internal walls are 20 cm and 10 cm in thickness, respectively. Diagonal steel braces are utilized as strengthening members. The braced frames are considered for resisting lateral forces. Section sizes of columns, beams and diagonal braces are given in Table 1.

Table 1. Section sizes

Structural members	Existing building	Strengthened building
Corner columns	400 x 400 mm	
Other columns	250 x 500 mm	
Beams	250 x 450 mm	
Diagonal braces	–	CHS 177. 8 x 8 mm

In TBEC-2018, minimum concrete grade is defined as 25 MPa for the design of new RC buildings. However, lower concrete grades are observed in the evaluation of existing buildings constructed according to previous seismic codes. In this study, the compressive strength of concrete is taken to be 16 MPa. The yield strength of longitudinal and transverse reinforcement is 420 MPa. The steel grade of bracing members is S235JR in the strengthened building. Reinforcement configuration for columns and beam sections in the supports, as well as section details of the braces, are shown in Figure 2. Diameter of stirrups is 8 mm for all columns and beams.

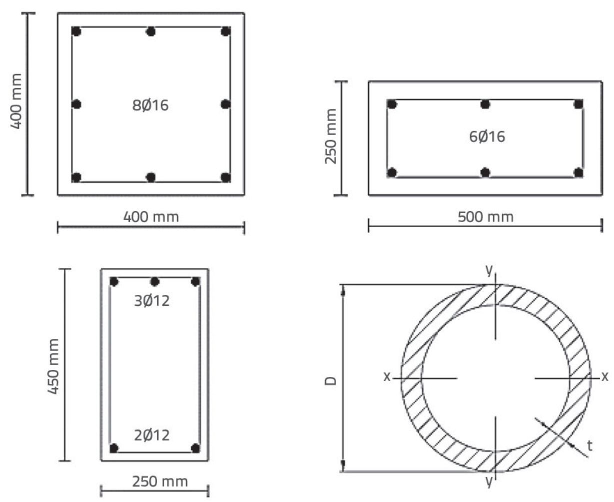

Figure 2. Section details

In practice, braced frame systems are generally concentric. So, the braces intersect at the node of the centre point. Geometrical and mechanical properties of steel diagonal braces that are used in the strengthened building are presented in Table 2. Geometrical properties such as the diameter, wall thickness, and area, are symbolized by D, t, and A, respectively. On the other hand, the moment of inertia and radius of gyration values for the related directions are given under mechanical properties.

Table 2. Properties of braces

Section	Geometrical properties	Mechanical properties
CHS 177.8x8	D =177.8 mm	$I_x = I_y = 1541$ cm^4
	t = 8 mm	$I_t = 3083$ cm^4
	A = 4270 mm^2	$i_x = i_y = 6.01$ cm

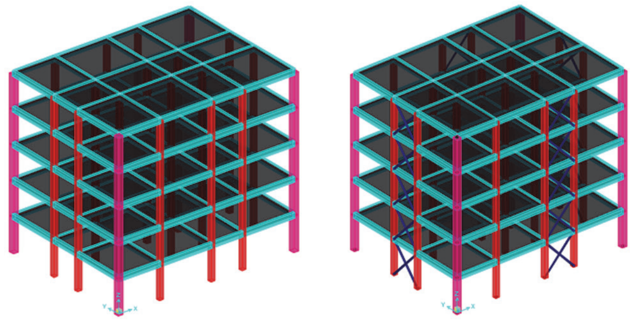

Figure 3. Existing and strengthened buildings

Three dimensional models of the 5 storey RC frame building and strengthened building are shown in Figure 3. In the strengthened building, steel braces, which intersect in the middle of the spans, are symmetrically utilized in both directions for all storeys. The aim is to improve the rigidity and deformation capacity of the existing building. After performing modal analysis of both buildings, first periods of vibration modes and effective mass ratios are obtained using an appropriate software. The results for relevant directions are given in Table 3.

Table 3. Period and effective mass ratio values

Building type	Mode number	Period [s]		Effective mass ratio [%]	
		x direction	y direction	x direction	y direction
Existing building	1	–	1.22	–	80.01
	2	1.19	–	80.03	–
Strengthened building	1	–	0.69	–	77.1
	2	0.68	–	77.2	–

3. Nonlinear analysis

After the modelling phase and definition of loads, non-linear analysis is performed using the software. The static pushover procedure is utilized to determine seismic performance of both buildings. This procedure is considered to be quite effective for performance evaluation of buildings in structural engineering [26]. The pushover technique is a series of non-linear incremental analyses that are linked to determine lateral deformation and damage situation of structural members. Structural behaviour is evaluated by the performance level of the structure at the

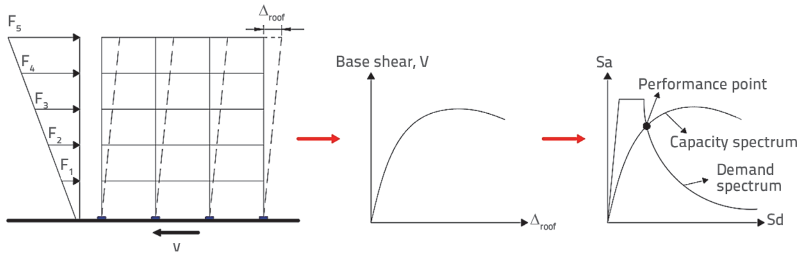

Figure 4. Pushover curve and performance point

value of target displacement, also named as performance point, according to capacity and demand spectrums, as shown in Figure 4.

The relationship between the base shear and roof displacement is presented by capacity curve in pushover analysis. This analysis is commonly used by many researchers owing to several advantages such as fast response, reliable results, and minimum computational efforts. As the relationship between performance targets and damage levels are directly obtained by displacement-based methods in performance analysis, these methods have recently taken the place of force-based methods. Main steps of non-linear analysis are presented in Figure 5.

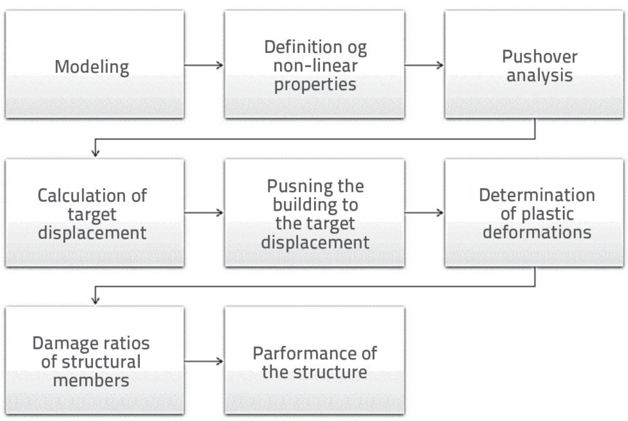

Figure 5. Analysis steps

Plastic hinge behaviour in non-linear analysis can be defined by concentrated and distributed plasticity approaches. These approaches assume that non-linear behaviour can occur at the end of the structural member in the concentrated plasticity model, or along the element and over the element cross section in the distributed plasticity model [27]. In this study, the concentrated plasticity approach is utilized in the software to define the plastic hinge behaviour of RC columns and beams in non-linear analysis. While hinge properties are automatically defined as per ASCE regulations in the software, the moment-rotation relationship of structural members is required to define plastic-hinge properties according to TBEC-2018 [28]. For this purpose, an idealized force-deformation relationship of plastic hinge is defined in the software, as shown in Figure 6. The curve is formed of certain points. There are four lines in the curve, i.e., AB, BC, CD, and DE, and they can be defined as the elastic stage, the strengthening stage, the unloading stage, and the failure stage, respectively. Point A represents the unloaded situation of hinge deformation. The yield point is reached when F_y strength value is reached. Beyond point B, deformation is effective on the force of the hinge. The plastic hinge reaches the collapsing situation as the displacement reaches point C. Generally, the slope of BC line is around 10 % of the slope of the AB line. Eventually, the hinge loses its strength, and the failure situation of the structure is defined when points D and E are reached. Their values equal 20 % of the strength value of point B as defined in ASCE 41-13 [29].

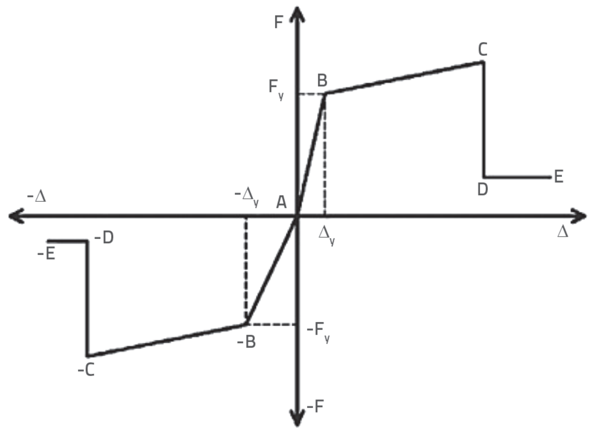

Figure 6. Plastic hinge relationship

Non-linear behaviour is confined to plastic hinges that are defined at both ends of columns and beams. The location of plastic hinges is shown in Figure 7. The plastic hinge length is represented by L_p in the figure [28]. Besides, the software enables definition of the moment-curvature relationship of sections to obtain the yield moment (M_y), ultimate moment (M_u), as well as the yield curvature (φ_y) and ultimate curvature (φ_u) values, as shown in Figure 8.

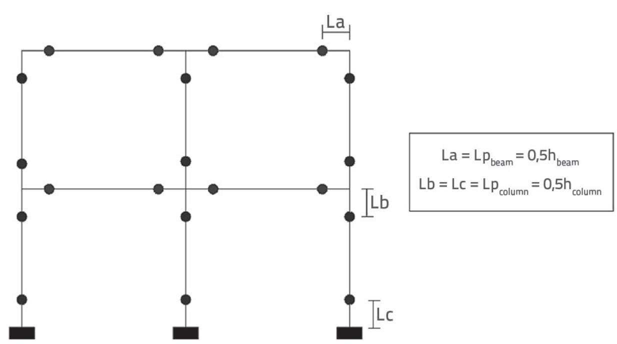

Figure 7. Plastic hinges

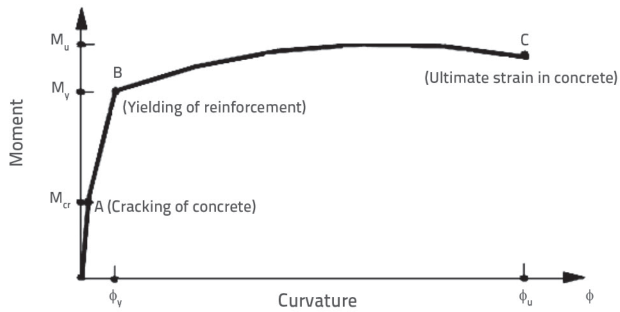

Figure 8. Moment-curvature diagram

After determining the yield curvature (φ_y) and ultimate curvature (φ_u) values of the sections, the yield rotation (θ_y) is calculated according to Eq. (1) as defined in TBEC-2018. In the equation, the value of η is taken to be 1.0 for the beams and columns. In addition, the plastic hinge length, length of shear span in the section, height of the section, average diameter of steel bars in the joint, average compression strength of concrete, and average yield strength of reinforcement, are symbolized by L_p, L_s, h, d_b, f_{ce} and f_{ye}, respectively. Furthermore, plastic rotation (θ_p) of the collapse prevention level is determined according to Eq. (2).

$$\theta_y = \frac{\phi_y L_s}{3} + 0.0015\eta\left(1 + 1.5\frac{h}{L_s}\right) + \frac{\phi_y d_b f_{ye}}{8\sqrt{f_{ce}}} \tag{1}$$

$$\theta_p = \frac{2}{3}\left[\left(\phi_u - \phi_y\right)L_p\left(1 - 0.5\frac{L_p}{L_s}\right) + 4.5\phi_u d_b\right] \tag{2}$$

Main structural performance levels are presented in Figure 9. These levels are similarly defined in seismic codes. No damage is depicted before the immediate occupancy level.

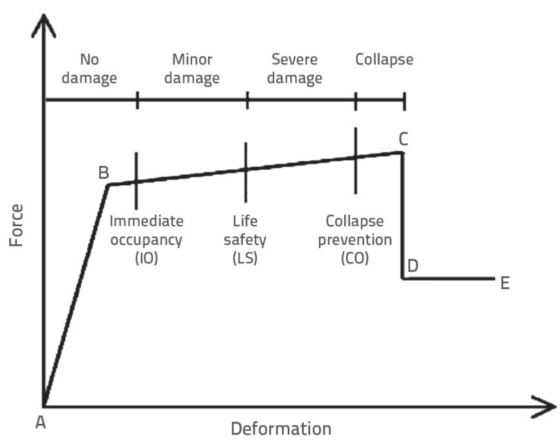

Figure 9. Performance levels

However, light damage, such as small cracks in non-structural members, may be observed at the immediate occupancy level. Low deformation and damage values are observed at the life safety level. On the other hand, the strength and lateral rigidity of structural members are still preserved. At the collapse prevention level, an extensive inelastic distortion occurs in the structural members of low strength and rigidity. Although permanent displacements may be observed, the total collapse is prevented at this performance level.

3.1. Performance analysis for TBEC-2018

In TBEC-2018, capacity curve coordinates are transformed into the modal response acceleration - modal response displacement to calculate the target displacement value, as shown in Figure 10. The non-linear spectral displacement, $S_{di}(T_1)$ is determined according to the spectral displacement ratio, C_R and elastic design spectral displacement $S_{de}(T_1)$ using equation ($S_{di}=C_R x S_{de}(T_1)$). T_A and T_B represent corner periods of horizontal elastic design spectrums. The value of C_R changes due to the relationship between the natural vibration period value of the building (T_1) and the corner period value (T_B). The modal capacity diagram shown in Figure 10.a is utilized when the value of T_1 is greater than T_B. On the other hand, the diagram in Figure 10.b is considered when the value of T_1 is lower than or equal to the value of T_B.

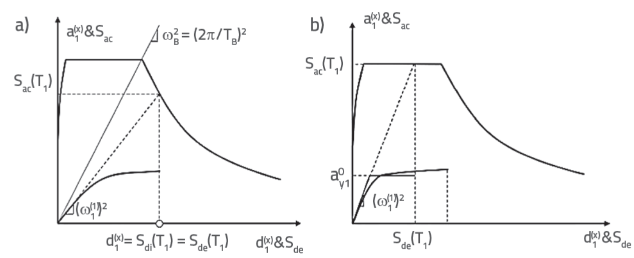

Figure 10. Modal capacity diagrams

Local soil class is taken to be D as the soil is composed of gravel or very solid clay layers. An average shear wave velocity in the upper 30 m of the depth of class D soil ranges between 180 and 360 and between 183 and 366 m/s according to TBEC-2018 and ASCE 7-16, respectively. In addition, the soil category according to EN regulations is available in the literature [30]. Design spectrum parameters determined by Eqs. (3) and (4) are given in Table 4.

In these equations, S_s and S_1 are the spectral acceleration coefficients. Besides, F_s and F_1 are local soil effect coefficients. These coefficients are obtained from the seismic hazard map. By using the coefficients, S_{DS} and S_{D1} representing the design spectral acceleration coefficients for the short period and 1.0 sec period, respectively, are calculated in the end.

$$S_s \cdot F_s = S_{DS} \tag{3}$$

$$S_1 \cdot F_1 = S_{D1} \tag{4}$$

Table 4. Design Parameters for TBEC-2018

Soil class	T_A	T_B	S_s	S_1	F_s	F_1	S_{DS}	S_{D1}
D	0.095	0.476	1.124	0.274	1.050	2.052	1.181	0.562

Table 5. Design Parameters for ASCE 7-16

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	2.0	1.225	0.548	0.817	0.365

3.2. Performance analysis for ASCE

The main steps of the analysis are quite similar in TBEC-2018 and ASCE 7-16. However, there are some differences between these codes. For instance, in TBEC-2018 live loads are multiplied with a certain coefficient, which is defined as 0.3 for residential buildings. On the other hand, effective seismic weights are calculated in ASCE 7-16 by considering dead loads. So, weights of the buildings are different in the two codes. In addition, the buildings are classified according to their risk categories in ASCE 7-16. The risk category of the existing RC buildings is II, which is defined in the standard as buildings and other structures that represent high risk to human life in the event of failure. The importance factor of the building (I_e) relating to risk category is assumed to be 1.00. S_s and S_1 are spectral acceleration parameters determined via the interactive web application. Afterwards, F_a and F_v parameters are taken from the related tables according to soil class D that is described as stiff soil in ASCE 7-16. By using these parameters, values of S_{MS} and S_{M1} for the short period and 1.0 second period are calculated using Eqs. (5) and (6). Finally, the spectral response acceleration values S_{DS} and S_{D1} are determined according to Eqs. (7) and (8). The values of these parameters are given in Table 5.

$$S_{MS} \cdot F_a = S_s \tag{5}$$

$$S_{M1} \cdot F_V = S_1 \tag{6}$$

$$(2/3)S_{MS} = S_{DS} \tag{7}$$

$$(2/3)S_1 = S_{D1} \tag{8}$$

4. Analysis results

After generating non-linear analysis in the software, performances of the both existing and strengthened buildings are determined according to TBEC-2018 and ASCE 7-16. For this purpose, the incremental static pushover analysis is taken

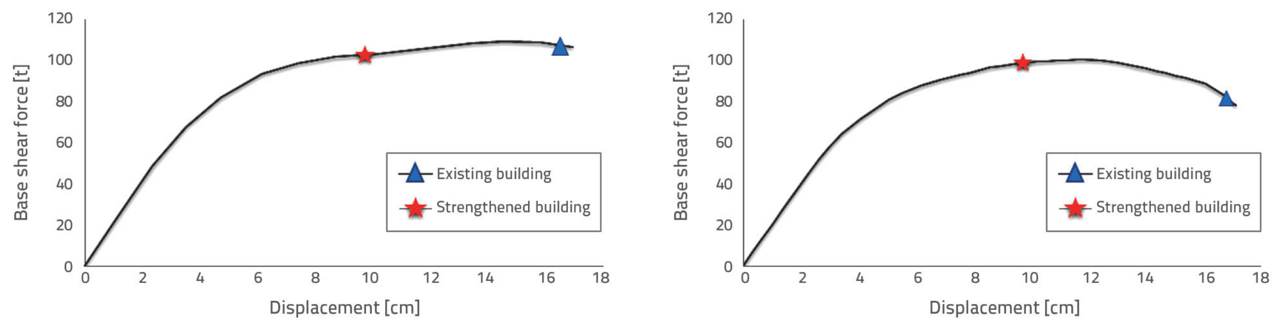

Figure 11. Target displacements in both directions according to TBEC-2018

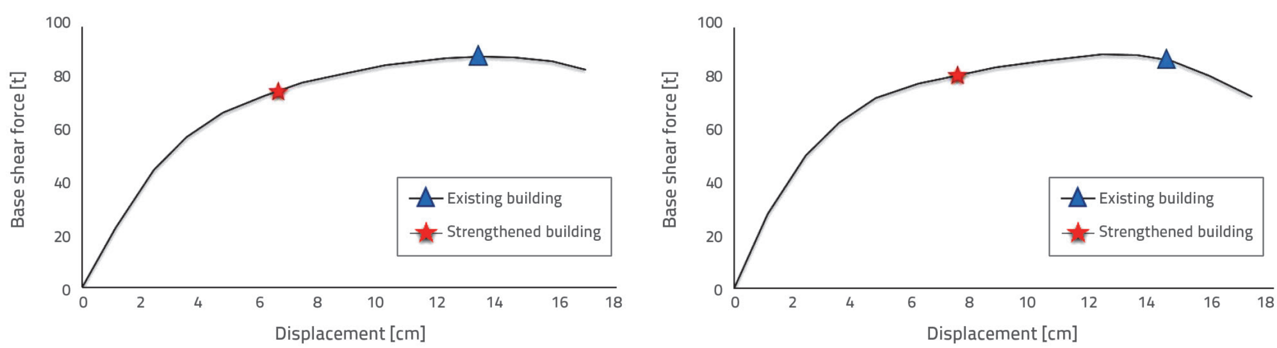

Figure 12. Target displacements in both directions according to ASCE 7-16

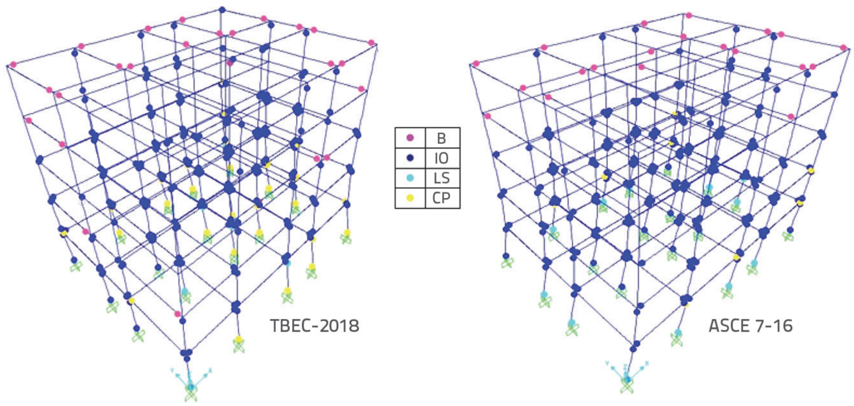

Figure 13. Plastic hinges for the existing building

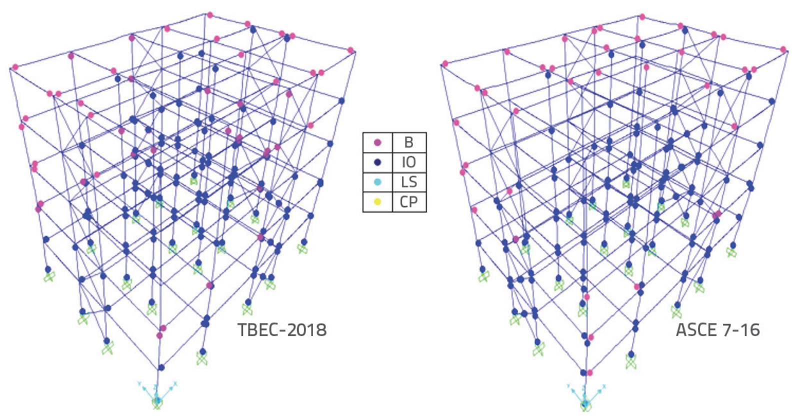

Figure 14. Plastic hinges for the strengthened building

into consideration in relation to gradual application of lateral forces. The analysis is performed in the related seismic direction until the building is no longer able to resist further forces. The capacity curve showing relationship between the base shear force and roof displacement is obtained by pushover analysis.

While target displacement values of the existing building are 16.97 and 17.13 cm according to TBEC-2018, the values according to ASCE 7-16 are 13.55 and 14.21 cm for x and y directions, respectively. After strengthening by steel braces, target displacements are reduced to 9.56 and 9.59 cm for TBEC-2018. On the other hand, target displacements of 6.52 and 7.54 cm are obtained for ASCE 7-16. The capacity curves of the existing building for x and y directions according to codes are presented in figures 11 and 12. To exhibit the differences between target displacements of the existing and strengthened buildings, both values are marked on these figures.

The buildings are pushed to the calculated target displacement values to determine the damage states of structural members. Afterwards, plastic hinges occur at the ends of the structural members. Plastic hinges in target displacements for the x direction are shown on the three-dimensional models of the buildings according to TBEC-2018 and ASCE 7-16 in figures 13 and 14.

Damage situation of structural members is evaluated by considering the plastic hinges for each direction in target displacements. Levels of damage to structural members are evaluated, and the results are presented in Table 6 according to TBEC-2018 and ASCE 7-16. Damage levels in the strengthened building reveal that the braces are capable of improving seismic capacity of the existing building for each code.

Table 6. Levels of damage to structural members in x direction

Structural members	Storey number	TBEC-2018						ASCE 7-16					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	–	12	4	8	8	–	–	13	3	10	6	–
	2	–	14	2	9	7	–	–	15	1	13	3	–
	3	–	16	–	16	–	–	–	16	–	16	–	–
	4	10	6	–	16	–	–	9	7	–	16	–	–
	5	16	–	–	16	–	–	16	–	–	16	–	–
Columns	1	–	4	16	17	3	–	–	7	13	17	3	–
	2	–	10	10	19	1	–	–	14	6	20	–	–
	3	–	14	6	20	–	–	–	15	5	20	–	–
	4	11	9	–	20	–	–	13	7	–	20	–	–
	5	20	–	–	20	–	–	20	–	–	20	–	–

Table 7. Levels of damage to structural members in y direction

Structural members	Storey number	TBEC-2018						ASCE 7-16					
		Existing			Strengthened			Existing			Strengthened		
		IO	LS	CP	IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	–	10	5	7	8	–	–	12	3	10	5	–
	2	–	12	3	9	6	–	–	14	1	13	2	–
	3	–	15	–	15	–	–	–	15	–	15	–	–
	4	12	3	–	15	–	–	10	5	–	15	–	–
	5	15	–	–	15	–	–	15	–	–	15	–	–
Columns	1	–	2	18	16	4	–	–	6	14	18	2	–
	2	–	9	11	18	2	–	–	12	8	20	–	–
	3	–	13	7	20	–	–	–	16	4	20	–	–
	4	12	8	–	20	–	–	14	6	–	20	–	–
	5	20	–	–	20	–	–	20	–	–	20	–	–

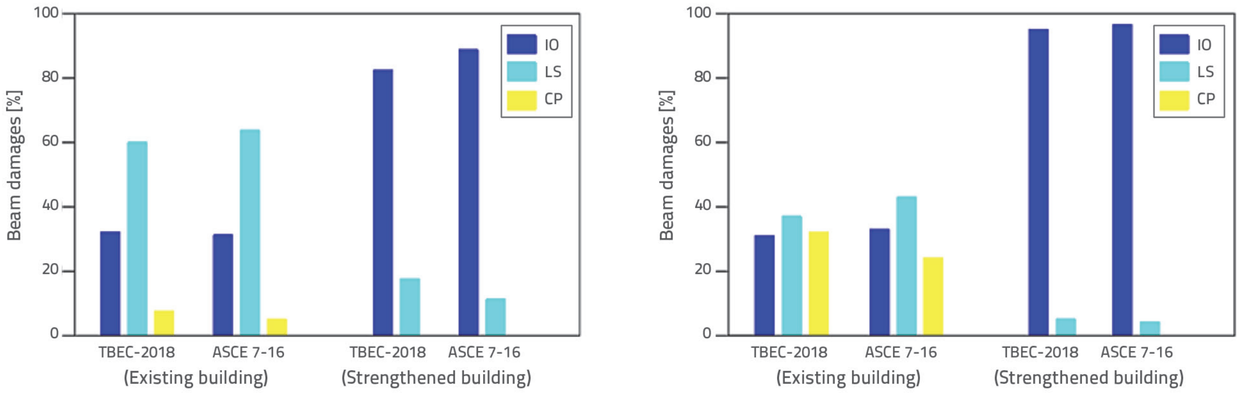

Figure 15. Damage ratios for x direction

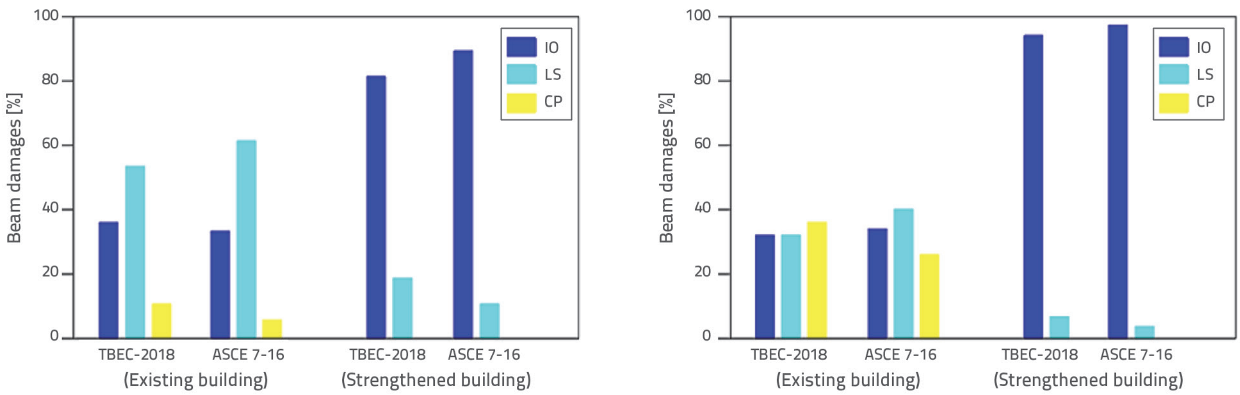

Figure 16. Damage ratios for y direction

After levels of damage to structural members have been determined for each storey, total damage ratios are calculated for the existing and strengthened buildings. The results for the related direction are shown in figures 15 and 16 so as to visually exhibit damage ratios according to both codes. It can be seen that no structural members are at CP level after strengthening.

In the final step of the analysis, storey drift ratios affecting performance of the buildings are determined at target displacement values for each code. The results are presented in figures 17 and 18 for the x any y directions according to TBEC-2018 and ASCE 7-16. The evaluation of results shows that steel braces can effectively reduce lateral displacement values.

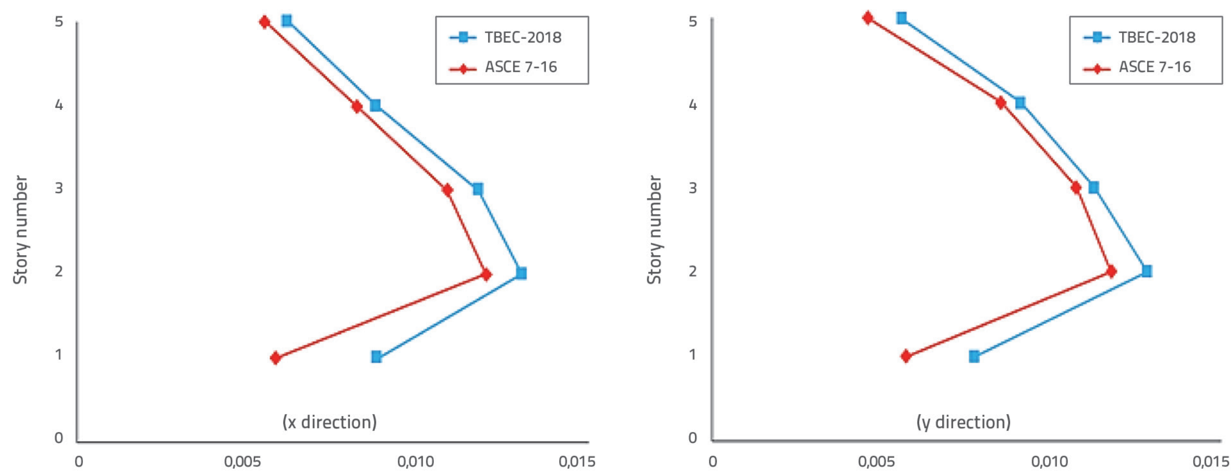

Figure 17. Storey drift ratios for the existing building

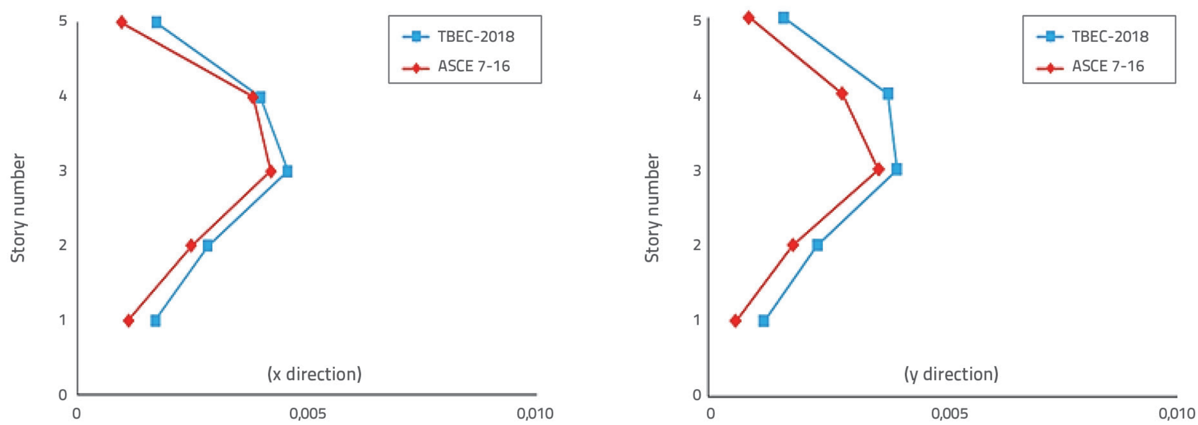

Figure 18. Storey drift ratios for the strengthened building

5. Conclusion

Various instances of structural damage and total collapse have been observed in recent earthquakes around the world. That is why appropriate performance-based design and evaluation procedures have been defined in seismic codes to minimize the related damage and losses. In this regard, studies have been performed to evaluate performance of existing buildings and to prepare strengthening projects so as to ensure the life safety performance level of such buildings. Non-linear analysis can be generated by static and dynamic methods. However, the analysis time is longer and advanced computer technology is required in dynamic solutions. Static analysis is therefore performed to evaluate seismic performance of buildings when reliable data about material properties and structural system have been obtained.

Performance based evaluation involves a combination of the design, analysis, and evaluation steps. Displacement based methods have proven to be successful in presenting the relationship between the displacement demand and the lateral force capacity of buildings. The capacity curve is obtained after pushover analysis, and damage levels of structural members are evaluated. Strengthening techniques have become prevalent for improving seismic performance of existing buildings. In this regard, concentrically braced systems have become one of the most effective alternatives due to their high strength and stiffness properties.

It is known that RC frame buildings are widely constructed and that they have quite a long service life. In this study, non-linear static analysis of an existing mid-rise RC frame building is initially performed according to two different seismic codes. After obtaining damage limits of structural members, the building is symmetrically strengthened by steel braces. The aim has been to improve seismic resistance of the existing building after the strengthening operation. Finally, the performance of both buildings and storey drift ratios are evaluated.

First periods of vibration modes and related effective mass ratios are determined after modal analysis. As expected, period values decrease after strengthening by steel braces. Afterwards, target displacements are calculated for each code and the buildings are pushed to these values to obtain damage limits of structural members. Damage ratios of structural members attain the highest values at the first floors according to analysis results of each code. On the other hand, light damage levels are obtained in the upper floors.

Based on the results of non-linear analysis, it can be stated that TBEC-2018 offers more conservative results compared to ASCE 7-16. When damage limits are evaluated, it can be seen that the life safety damage level is not provided for a number of structural members for TBEC-2018. While a total of 36 columns and 8 beams reach collapse prevention level according to TBEC-2018 in the y direction of the existing building, 26 columns and 4 beams are in the same damage level for ASCE 7-16.

The aim has been to strengthen the existing RC frame building by steel braces that improve the inelastic behaviour, provide stability and lateral stiffness. The strengthened building is analysed once again, and damage to structural members is determined. When damage ratios of structural members are evaluated according to both codes, it can be seen that most beams and columns offer the immediate occupancy performance level. In addition, there are no structural members beyond the life safety level after strengthening.

Since lateral displacements cause severe damage to structural members, storey drift ratios are considered as one of the most important parameters in the performance-based evaluation. Storey drift ratios are calculated for each direction of the existing building and the strengthened building. It has been noticed that storey drift ratios decrease considerably after the use of steel braces. In conclusion, it is thought that this study will prove beneficial to the researchers studying non-linear performance and strengthening techniques in structural engineering.

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