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Seismic behavior of concentrically steel braced frames and their use in strengthening of reinforced concrete frames by external application

Alptug Unal [*1], and Mevlut Yasar Kaltakci [2a]

[1] *Department of Civil Engineering, Selcuk University, Engineering Faculty, Konya, Turkey*
[2] *Department of Civil Engineering, Hasan Kalyoncu University, Engineering Faculty, Gaziantep, Turkey*

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Abstract. There are many studies in the literature conducted on the subject of ensuring earthquake safety of reinforced concrete and steel structures using steel braced frames, but no detailed study concerning individual behavior of steel braced frames under earthquake loads and strengthening of reinforced concrete structures with out-of-plane steel braced frames has been encountered. In this study, in order to evaluate behaviors of "Concentrically Steel Braced Frames" types defined in TEC-2007 under lateral loads, dimensional analysis of Concentrically Steel Braced Frames designed with different scales and dimensions was conducted, the results were controlled according to TEC-2007, and after conducting static pushover analysis, behavior and load capacity of the Concentrically Steel Braced Frames and hinges sequence of the elements constituting the Concentrically Steel Braced Frames were tested. Concentrically Steel Braced Frames that were tested analytically consist of 2 storey and one bay, and are formed as two groups with the scales 1/2 and 1/3. In the study, Concentrically Steel Braced Frames described in TEC-2007 were designed, which are 7 types in total being non-braced, X-braced, V- braced, Λ- braced, \- braced, /- braced and K- braced. Furthermore, in order to verify accuracy of the analytic studies performed, the 1/2 scaled concentrically steel X-braced frame test element made up of box profiles and 1/3 scaled reinforced concrete frame with insufficient earthquake resistance were tested individually under lateral loads, and test results were compared with the results derived from analytic studies and interpreted. Similar results were obtained from both experimental studies and pushover analyses. According to pushover analysis results, load-carrying capacity of 1/3 scaled reinforced concrete frames increased up to 7,01 times as compared to the non-braced specimen upon strengthening. Results acquired from the study revealed that reinforced concrete buildings which have inadequate seismic capacity can be strengthened quickly, easily and economically by this method without evacuating them.

Keywords: concentrically steel braced frames; reinforced concrete frames; design and construction mistakes; pushover analysis; strengthening

1. Introduction

The lateral load-carrying capacity, strength and stiffness of a building might be increased by shear walls used in the system. These shear walls may be either reinforced concrete (RC) or made

∗Corresponding author, Research Assistant, E-mail: alptugunal@gmail.com
[a] Professor, E-mail: mykaltakci@gmail.com

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of "Concentrically Steel Braced Frames (CSBF)" indicated in (TEC2007). Steel braced frames are systems constituting of hinged joint or moment resisting frames and braced bound to these frames as centric and eccentric.

Such systems are generally used in order to supply stiffness and strength against lateral loads in low and medium height buildings. In addition to saving of material in these systems, it is possible to restrain storey drift effectively by providing high lateral stiffness. The braced frames provide energy consumption under the effect of big lateral loadings with changing direction by pressure-wrenching and by flowing under tensile loads (Dogan 2007, TEC-2007 2007). In this study, CSBFs present in TEC-2007 were investigated and are given in Fig. 1 (TEC-2007 2007).

In the study, "CSBF Types" defined in TEC-2007 were designed (TEC-2007 2007), which are 7 types in total being non-braced, X- braced, V- braced, Λ- braced, \- braced, /- braced and K- braced. For this purpose, 7 pieces of 1/2 scaled CSBF models made up of 100×100×3 mm cross-section box profile, 7 pieces of 1/2 scaled CSBF models made up of 80×80×6 mm cross-section box profile, and 7 pieces of 1/3 scaled CSBF models made up of 60×60×4 mm cross-section box profile were formed. In the analytic study, pushover analyses of the CSBF and of the RC frames strengthened with these CSBFs were conducted by the SAP2000 program (SAP2000a, b), and according to the results of the analyses conducted, load-displacement curves of the CSBFs were tested and compared (Unal 2013).

A number of scientists have studied the strengthening of RC frame with external shear wall addition to the existing RC frame (SAP2000a, b, Kaltakci *et al*. 2010, Kaplan *et al*. 2011). Also there are some analytical and experimental studies related to concentrically and eccentrically steel braced frames in the literature (Bahadir 2012, Unal 2012, Annan *et al*. 2009, Brandonisio 2012, Grande and Rasulo 2013, Hajirasouliha and Doostan 2010, Jazany *et al*. 2013, Lumpkin *et al*. 2012, Metelli 2013, Ozel and Guneyisi 2011, Roeder *et al*. 2011, Maheri *et al*. 2003, Qu *et al*. 2015, Khampanit *et al*. 2014, Görgülü *et al*. 2012, Korkmaz 2007). To our knowledge, there are very limited studies on the strengthening of RC frame with external CSBF addition to the existing RC frame (Tama *et al*. 2005). Based on the analysis results of the CSBF under lateral loads, strengthening of RC frames with insufficient earthquake resistance can be done through bonding CSBF externally with steel bond beams on their plane. Thus, thanks to a strengthening method to be applied only externally to the buildings, the buildings will not be out of use during installation of this method, and a lot of additional processes (coating, plaster, painting, etc.) that would be required as a result of strength inside the building will be avoided. By this way, it is aimed to strengthen the RC frame buildings comprising design and construction defects that can be widely seen in many of them within the existing stock of buildings constructed in the previous years. In order to experimentally investigate the performance of this system strengthening was performed by bonding 1/3 scaled CSBF with two story and one bay (Kaltakci *et al*. 2010, Kaplan *et al*. 2011,

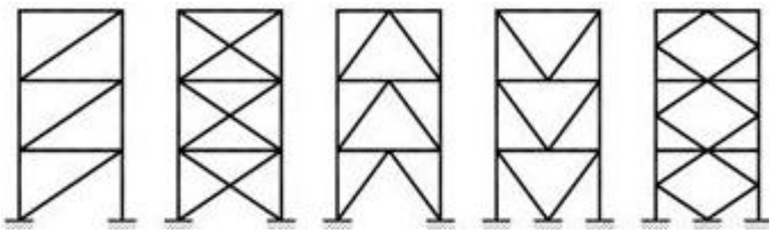


Fig. 1 Concentrically steel braced frames (TEC-2007 2007)

Wang *et al*. 2013, Yoo *et al*. 2008), formed in 7 different types, as described in (TEC-2007 2007), to the 1/3 scaled RC frame with two storey and one bay, with steel bond beams externally on its plane, and behavior of the strengthened specimens were tested, and pushover analyses were conducted, and load-displacement curves, resistance, ductility, the results were compared with each other and with those of the strengthened frame.

Furthermore, in order to verify accuracy of the analytic studies conducted, the 1/2 scaled CSBF-X braced frame test element made up of 100x100x3mm cross-section box profiles and 1/3 scaled RC frame with insufficient earthquake resistance were tested individually under lateral loads, and test results were compared with the results derived from analytic studies and interpreted (Unal *et al*. 2014).

2. Materials and methods

In this study, 1/2 scaled two groups were formed in order to compare CSBFs designed according to TEC-2007 with CSBFs not designed according to TEC-2007 and the differences between them were investigated (TEC-2007 2007). For this reason, the verifications of box cross-sections with high ductility level are given as follows

$$\frac{b}{t} \leq 0.7\sqrt{\frac{E_s}{\sigma_a}} \tag{1}$$

$$\lambda \leq 4.0\sqrt{\frac{E_s}{\sigma_a}} \tag{2}$$

In Eqs. (1) and (2), the cross-section which is not compatible according to TEC-2007 was designed as 100×100×3 mm and the cross-section compatible with TEC-2007 was designed as 80×80×6 mm (TEC-2007 2007). This is due to moments of resistance resulting in more close values in both cross-sections. The profile properties of these cross-sections are given in Table 1.

According to cross-section dimensions mentioned above, internal force values in cross-sections were determined by using linear analysis in SAP 2000 program (SAP2000a, b). The control of relative storey drifts of the system was performed according to Eqs. (3)-(5) and their second-degree effects were performed according to Eq. (6). Moreover, stress controls of all elements (column, joist and braced) in the shear wall were performed and their compatibility with TEC-2007 was investigated (TEC-2007 2007). In cross-section controls, Centric Steel X braced specimen was accepted as a reference.

$$\Delta_i = d_i - d_{i-1} \tag{1}$$

Table 1 Profile properties which are and which are not compatible with TEC-2007

	A_x (cm^2)	I_x (cm^4)	I_y (cm^4)	i_x (cm)	i_y (cm)	W_{elx} (cm^3)	W_{ely} (cm^3)	W_{plx} (cm^3)	W_{ply} (cm^3)
100×100×3 (mm)	11.4	177	177	3.94	3.94	35.4	35.4	41.2	41.2
80×80×6 (mm)	16.8	149	149	2.98	2.98	37.3	37.3	45.8	45.8

$$\delta_i = R \ \Delta_i \tag{1}$$

$$\frac{(\delta_i)_{\max}}{h_i} \leq 0.02 \tag{1}$$

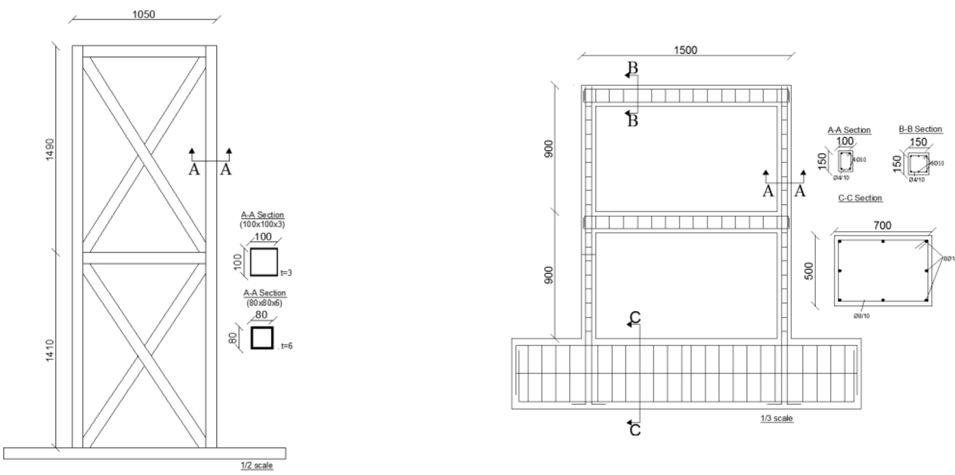
$$\theta_i = \frac{(\Delta_i)_{ort} \sum_{j=1}^{N} w_j}{V_i h_i} \leq 0.12 \tag{1}$$

In order to represent the buildings in literature, the buildings were modeled by generally forming 1/2 and 1/3 scaled systems. For this purpose, a 1/3 scaled group was dimensioned in CSBF compatible with TEC-2007 in addition to the performed two groups of studies (TEC-2007 2007). The cross-sections in 1/3 scaled CSBF group were determined as 60×60×4 mm.

CSBFs can be used to resist lateral loads in current building systems, to limit storey drifts and to improve stiffness of the building as well as they can be added to the system as strengthening elements afterwards. For this purpose, it was considered to strengthen 1/3 scaled RC frame with designed 1/3 scaled CSBFs by means of transverse beam.

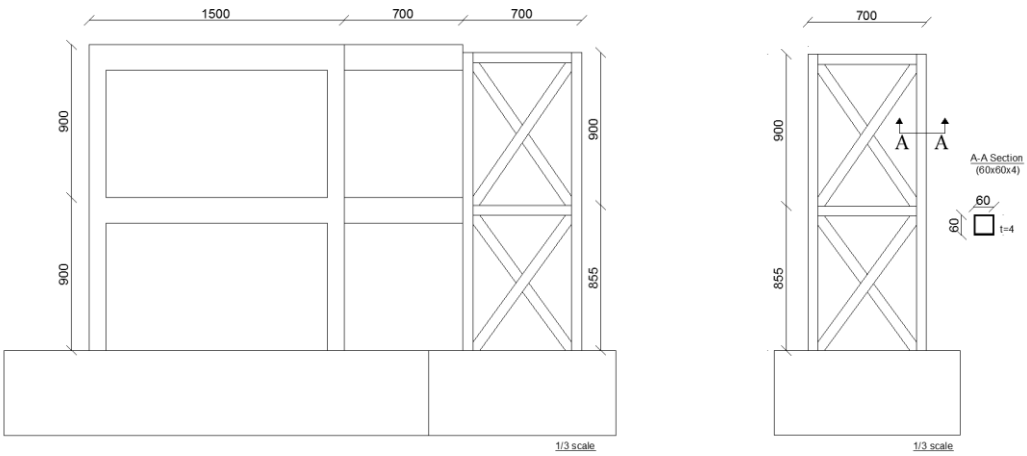
Pushover analysis of the frame strengthened with three groups of designed CSBFs and 1 group CSBF were performed. Moreover, a RC frame of 1/2 scaled with 100x100x3mm cross-section and X braced specimens were tested under lateral loads in order to prove the accuracy of the study performed.

According to dimensioning with respect to (TEC-2007 2007), CSBFs constitute of 2-storey single-bay. The dimensions of 1/2 and 1/3 scaled CSBFs and steel elements were selected among dimensions that are frequently used in literature and application. For 1/2 scaled CSBF, bay was 970 mm, total height was 2860 mm where the bay and total height for 1/3 scaled steel shear walls were 640 mm and 1728 mm, respectively. In Fig. 2, the dimensions of 1/2 scaled CSBF, dimensions of 1/3 scaled CSBF, dimensions of bare RC frame and dimensions of RC frame strengthened with CSBF are given as an example for X braced specimen.



(a) (b)

Fig. 2 Dimensions and properties of specimens (a) 1/2 scaled CSBF; (b) 1/3 scaled base RC frame; (c) 1/3 scaled RC frame strengthened with CSBF; (d) 1/3 scaled CSBF



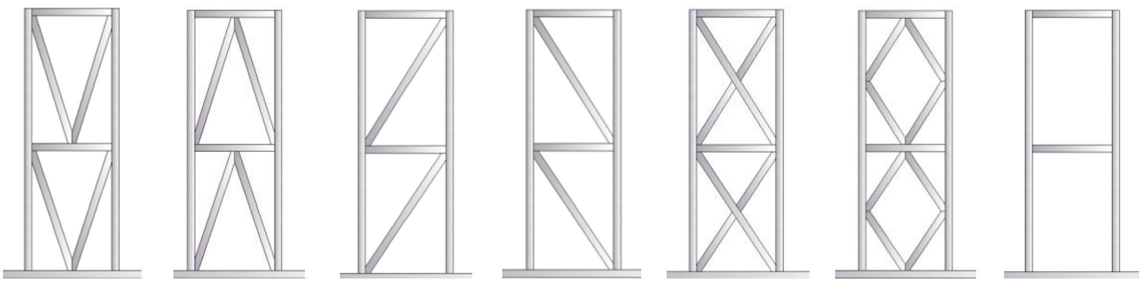
(c) (d)

Fig. 2 Continued

2.1 Pushover analysis

In this study, pushover analyses were carried out for various types of "Concentrically Steel Braced Frames" defined in TEC-2007 and the results were evaluated (TEC-2007 2007). In CSBFs having high ductility level, K braced arrangement was not allowed. However, all types of braced frames were modeled in SAP2000 program (SAP2000a, b) in order to compare them with each other and they were compared by making pushover analysis. Moreover, a RC frame without braced element in the same size with concentric steel braces shear walls was also investigated in order to examine the effect of the results obtained on non-braced frames. Correspondingly, pushover analyses of totally 18 CSBF specimens and 1 non-braced specimen including 6 for each of 1/2 and 1/3 scaled ones were performed with SAP2000 program (SAP2000a, b) (Fig. 3).

Moreover, pushover analysis of 1/3 scaled RC frame with inadequate earthquake resistance and 7 RC frames strengthened with CSBF were carried out with SAP2000 program (SAP2000a, b) (Fig. 4).



(a) (b) (c) (d) (e) (f) (g)

Fig. 3 The specimens whose pushover analysis were performed with SAP2000 program: (a) V braced; (b) Inverted V braced; (c) Diagonal braced; (d)Inverse diagonal braced; (e) X braced; (f) K braced; (g) Non-braced

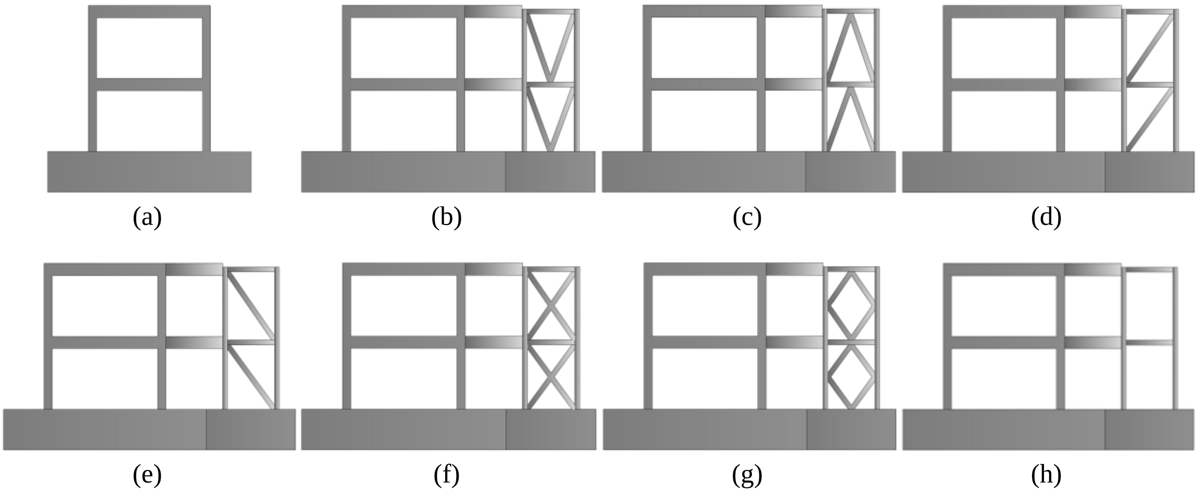


Fig. 4 Strengthened specimens the pushover analysis of which were carried out with SAP2000 program: (a) RC base frame; (b) V braced; (c) Inverse V braced; (d) Diagonal braced; (e) Inverse diagonal braced; (f) X braced; (g) K braced; (h) Non-braced

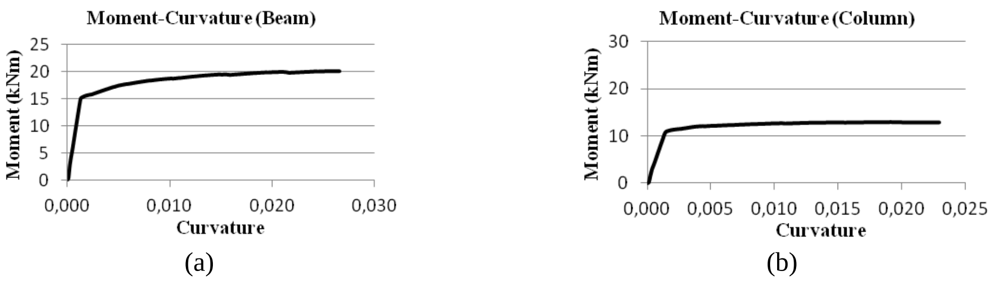


Fig. 5 Moment-curvature relationship for RC frame

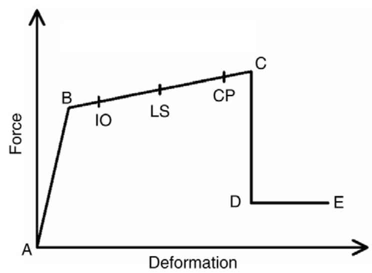


Fig. Force–deformation relationship of a typical plastic hinge (Inel and Ozmen 2008)

In pushover analysis performed with SAP2000 program, FEMA 356 American Regulations were applied (SAP2000a, b, FEMA 1996). It is necessary to make different definitions for these columns, beams and braces. The length of plastic hinge was taken as Lp = 0.5 h in FEMA 356 and TEC-2007 (TEC-2007 2007, FEMA 1996). In SAP2000 program (SAP2000a, b), the hinge choices for column, beam and braces were presented. P-M3, M3 and P hinge definitions were given for steel columns, steel beams and steel braces, respectively. Moreover, moment curvature value was calculated for RC frame and it was defined in SAP2000 (SAP2000a, b). Moment curvature curves for RC frame are given in Fig. 5.

SAP2000 implements the plastic hinge properties described in FEMA-356 (SAP2000a, b, FEMA 1996). As shown in Fig. 6, five points tagged as A, B, C, D, and E define the force–deformation behavior of a plastic hinge (Inel and Ozmen 2008).

Immediate Occupancy (IO)
– yielding of steel, significant cracking of concrete and nonstructural damage arises (TEC-2007 2007).

Life Safety (LS)
– damage of structural and nonstructural components start. We have to make essential circulation routes accessible to minimize risk of injury and causality for this stage (TEC-2007 2007).

Collapse Prevention (CP)
– This point ensure a small risk of partial or complete building collapse by limiting structural deformations and forces to the onset of significant strength and stiffness degradation (TEC-2007 2007).

Point C is the indication of ultimate capacity of the structure and Point D indicate residual strength for the structure. Complete failure will occur at point E.

In TEC-2007, Immediate Occupancy (IO) limit is given as (εcu)MN = 0.0035, Life Safety (LS) limit is given as (εcg)GV = 0.0135 and Collapse Prevention (CP) limit is indicated as (εcg)GC = 0.018 and pushover analysis was carried out with SAP2000 program based on these data (TEC-2007 2007, SAP2000a, b).

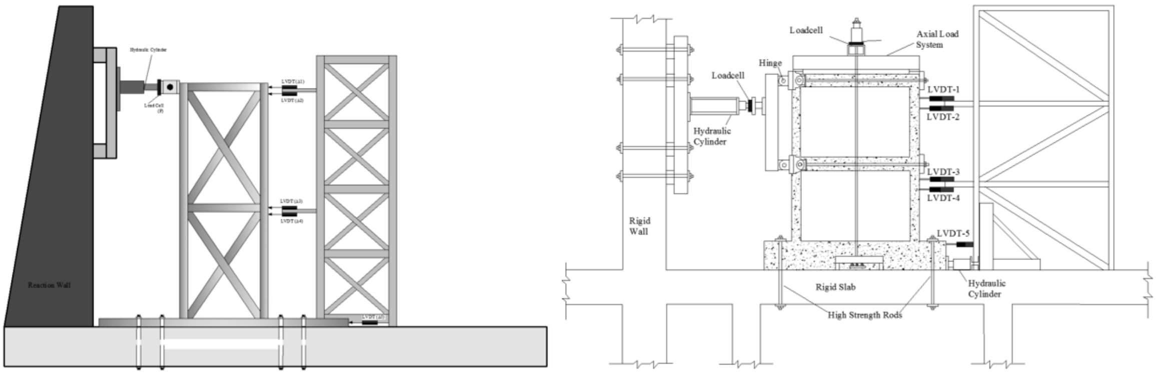
Since equivalent seismic load was considered in pushover analysis and the weights of upper and ground storey were equal, 2 units of lateral loads were defined for upper storey and 1 unit of lateral load was defined for ground storey.

2.2 Experimental study

Two experimental studies were performed within the context of this research. The first study was about 1/2 scaled CSBF and it was tested under static lateral loading by giving a load on the upper connection point of upper storey of the test specimens. The second experimental study was about 1/3 scaled bare RC frame with inadequate earthquake resistance including various structural defects and it was tested under earthquake resembling cyclic lateral loading by applying 2 units of load to the upper storey and 1 unit of load to the ground storey.

During the experiments, load and displacement readings were recorded by a computer-aided data logging system. The specimens were loaded with a hydraulic cylinder having 1000 kN compression and 500 kN tensile capacity. Transfer of loading data to the computer was provided by connecting a load cell at the end of hydraulic cylinder for load measurement. The capacity of this load cell was 500 kN. The specimens were placed to the loading system the height of which can be adjusted according to the experiment.

In the experiments, displacement controls were carried out in the middle of storey beams. For this reason, a scaffolding made of box profile was produced in order to prevent movement of Linear Variable Displacement Transducer (LVDT) placed in the system and this scaffolding was



(a) (b)

Fig. 7 Lay-out of loading system and LVDT (a) CSBF; (b) base RC frame

fixed on the ground via holes present in laboratory slab.

The load measurements were taken by load cells and the displacements in specimens were measured with LVDT's during the experiments. The data measured by these devices were transferred to the computer via data logger system and they were recorded.

One load cell for lateral load and two load cells for axial load having 500 kN capacity were used. The loads reached in compression and tension were measured by means of rebates present in load cells and transferred to the computer. The load cells were connected to data logger and the load measurements were transferred to the computers.

Totally 4 LVDTs being two for each storey were placed at the level of each storey beam. The other LVDT was placed to the ground in order to measure ground motions. LVDTs connected to the upper storey, mid-storey and ground had a stroke of 300 mm, 200 mm and 150 mm respectively. The lay-outs of LVDTs and loading system can be seen in Fig. 7.

2.3 1/3 scaled reinforced concrete frame

This frame represents defected constructions in existing buildings. In buildings having inadequate earthquake resistance, a core sampling study was performed from the buildings in order to determine the quality of the concrete. For this reason, the concrete strength was tested by coring from each specimen after testing them in order to determine the concrete strength of frames that were produced. 2 core samplings were carried out for this RC frame. The cores were sampled as 98 mm in diameter and about 100 mm height. As a result of tests, concrete compressive strength was found approximately as 19 MPa.

In order to determine characteristic values of the reinforcement used, 3 samples having 400 mm length were taken and tested. According to the tests, average yield strength and average tensile strength of reinforcements with 10 mm diameter were determined as 394 MPa and 643 MPa, respectively.

While preparing the RC frame, first of all, the formworks were prepared, then the reinforcements were prepared and placed in the formworks and finally the specimens were prepared by pouring concrete. The specimens were manufactured laterally on the ground and were up righted via lifting jack.

The columns in specimens were dimensioned as 100×150 mm and the beams were dimensioned as 150×150 mm. The dimensions of foundation were selected as quite big to prevent any damage occurrence during experiment.

In the foundations, 3 ϕ 14 mm reinforcement in the top, 3 ϕ 14 mm reinforcement in the bottom and 2 ϕ 14 mm web reinforcement were used. The stirrups were arranged as ϕ 8/100 mm in the foundation. Column longitudinal reinforcement was prepared as 4 ϕ 10 mm and ϕ 6/100 mm stirrup was used. Confinement zone was not formed in columns and the stirrups were not continued at the connection zones of foundation and beams. In the beams, 3 ϕ 10 mm reinforcement at the top, 3 ϕ 10 mm longitudinal and ϕ 6/100 mm lateral reinforcements at the bottom were used. The stirrups were not continued in the zones where beams enter to the columns and stirrup compaction was not performed at the places close to connection zones (Bahadir 2012, Unal 2012, Unal *et al*. 2014, Balik 2012).

Specimen was fixed on the ground with high-strength shafts which were passed through the holes present on the rigid laboratory slab. By means of the holes present on the retaining wall, load transfer to the specimen was performed by fixing there action wall.

While carrying out experiment, it is necessary to calculate minimum load according to Eq. (7) determined as axial load in TEC-2007 (TEC-2007 2007). It was aimed to provide predicted column behavior by applying axial load to the specimen as twice the value found. According to this, approximately an axial load of 45 kN was applied onto each column during experiments.

$$N_d \leq 0.1 \times A_c \times f_{ck} \tag{7}$$

While applying lateral load to the specimen, 2 units of load to the upper storey and 1 unit of load to the ground floor should be applied according to equivalent seismic load calculation. For this reason, hydraulic cylinder was placed 1 unit of distance to the upper floor and 2 units of distance to the ground floor.

2.4 1/2 scaled CSBF

In this study, one X braced specimen was tested under static lateral loading which was formed in order to investigate “Concentric Steel Braced Frames” presented in (TEC-2007 2007). Lateral load was applied to the specimen from the upper connection point of upper storey.

Lateral, vertical and braced elements in X braced specimen used in this study were produced from box profile with 100×100×3 mm dimensions. The connection zones of steel elements were connected via welding to provide a rigid connection.

In order to investigate the strength properties of concentric steel braced frame specimen, steel tensile test was carried out according to TS EN ISO 6892-1st of January 2010 standard and its results were investigated (TS EN ISO 6892 2010).

According to test results, it was determined that maximum loading at the cross section was 20.5 kN, while the tensile stress and yield strength were 400.2 N/mm^2 and 337.9 N/mm^2 respectively. The elongation at rupture was 29.1%.

Tested specimen was produced in lateral laid on laboratory slab. The parts of the specimen were prepared by cutting them in required sizes and they were fixed with each other by welding. The specimen produced in lateral direction was up righted via lifting jack present in the laboratory and placed in the experimental set-up.

Tested specimen was fixed on the ground with high-strength shafts which were passed through the holes present on the rigid laboratory floor covering. By means of the holes present on the reaction wall, load transfer to the specimen was performed by fixing the hoist system on the wall. Computer-aided data reading system was used throughout the experiments, required load and displacement readings were performed and recorded.

3. Results and discussion

In this part, the results of analytical and experimental studies are presented and compared.

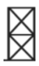
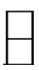
3.1 1/2 scaled CSBF

A meaning of the specimens were given below:

1/2CSBF$^{(-)}$: 1/2 Scaled Concentrically Steel Braced Frames not compatible according to TEC2007
1/2CSBF$^{(+)}$: 1/2 Scaled Concentrically Steel Braced Frames compatible according to TEC2007
1/3CSBF$^{(+)}$: 1/3 Scaled Concentrically Steel Braced Frames compatible according to TEC2007
1/3RC+CSBF$^{(+)}$: 1/3 Scaled Reinforced Concrete Frames Strengthened with Concentrically Steel Braced Frames compatible according to TEC2007

Base shear-top displacement graphs obtained after pushover analysis of 1/2CSBF$^{(-)}$, 1/2CSBF$^{(+)}$, 1/3CSBF$^{(+)}$ and 1/3RC+CSBF$^{(+)}$ types are given in Fig. 8. In Table 2, on the other hand, maximum

Table 2 Lateral load-carrying capacities and displacements of CSBF types

		a	b	c	d	e	f	g	h	i
	Max. load (kN)	144.98	1	260.43	1	127.18	1	216.10	1	7,01
	Top disp. (mm)	13.29	1	24.69	1	7.65	1	15.50	1	1,05
	Max. load (kN)	124.35	0,86	189.38	0,73	104.47	0,82	169.20	0,78	5,49
	Top disp. (mm)	45.17	3,40	33.53	1,36	21.91	2,86	40.61	2,62	2,74
	Max. load (kN)	170.21	1,17	258.80	0,99	144.78	1,14	180.07	0,83	5,84
	Top disp. (mm)	15.81	1,19	15.00	0,61	8.54	1,12	15.38	0,99	1,04
	Max. load (kN)	97.12	0,67	164.51	0,63	89.66	0,70	156.61	0,72	5,08
	Top disp. (mm)	19.59	1,47	72.01	2,92	46.36	6,06	31.72	2,05	2,14
	Max. load (kN)	126.88	0,88	194.06	0,75	107.37	0,84	142.56	0,66	4,62
	Top disp. (mm)	14.67	1,10	13.17	0,53	8.00	1,05	8.92	0,58	0,60
	Max. load (kN)	183.34	1,26	254.11	0,98	147.28	1,16	175.86	0,81	5,70
	Top disp. (mm)	61.85	4,65	50.32	2,04	26.37	3,45	33.24	2,14	2,24
	Max. load (kN)	23.08	0,16	27.77	0,11	18.04	0,14	44.69	0,21	1,45
	Top disp. (mm)	76.33	5,74	64.96	2,63	42.83	5,60	24.49	1,58	1,65

***a:** 1/2CSBF$^{(-)}$, **b:** $\frac{1/2CSBF^{(-)}}{1/2CSBF^{(-)}-X}$, **c:** 1/2CSBF$^{(+)}$, **d:** $\frac{1/2CSBF^{(+)}}{1/2CSBF^{(+)}-X}$, **e:** 1/3CSBF$^{(+)}$,
f: $\frac{1/3CSBF^{(+)}}{1/3CSBF^{(+)}-X}$, **g:** 1/3RC+CSBF$^{(+)}$, **h:** $\frac{1/3RC+CSBF^{(+)}}{1/3RC+CSBF^{(+)}-X}$, **i:** $\frac{1/3RC+CSBF^{(+)}}{1/3RC}$

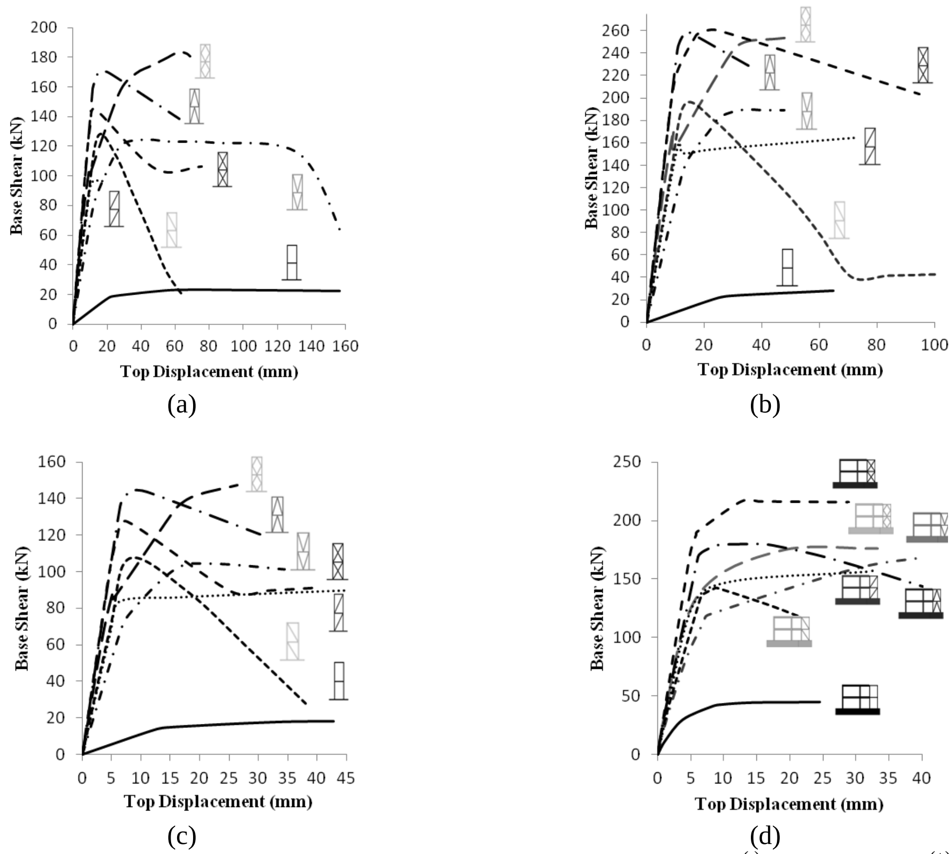


Fig. 8 Lateral load-carrying capacities of CSBF types: (a) 1/2CSBF$^{(-)}$; (b) 1/2CSBF$^{(+)}$; (c) 1/3CSBF$^{(+)}$; (d) 1/3RC+CSBF$^{(+)}$

load and top displacement values for each CSBF type are given. According to these data and graphs, it was observed that 1/2CSBF$^{(-)}$ types had more load-carrying capacity than 1/2CSBF$^{(+)}$ types at the same scale. Moreover, 1/3RC+CSBF$^{(+)}$ types constituting of CSBF added to RC frames had higher load-carrying capacity and top displacement values than 1/3CSBF$^{(+)}$ types just constituting of CSBFs.

According to the results of pushover analyses, the hinge sequences were investigated, it was observed that load-carrying capacities decreased after occurrence of hinge at the ends of columns. The number of hinges vary depending on the types of braced and hinge sequences for 1/2CSBF$^{(-)}$, 1/2CSBF$^{(+)}$, 1/3CSBF$^{(+)}$ were approximately same.

3.2 Experimental study

In order to confirm the accuracy of the results of our studies, one 1/2CSBF$^{(-)}$-X specimen and one 1/3 scaled RC frame were tested under lateral loads. Moreover, pushover analyses of these experimental studies were performed, experimental and analytical studies were compared by indicating them on the same graph (Fig. 9). As it can be seen from both graphs, the values of lateral load-carrying capacities obtained from pushover analyses and experimental studies were

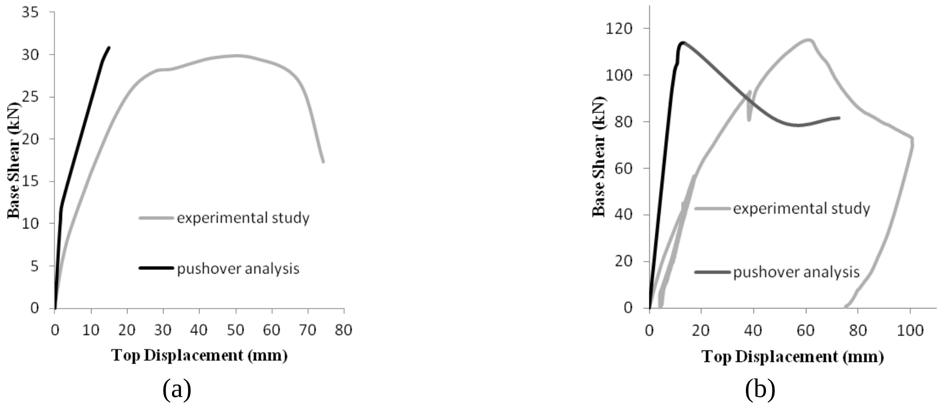


Fig. 9 Lateral load-carrying capacities of analytical and experimental studies: (a) RC frame; (b) 1/2CSBF$^{(-)}$-X

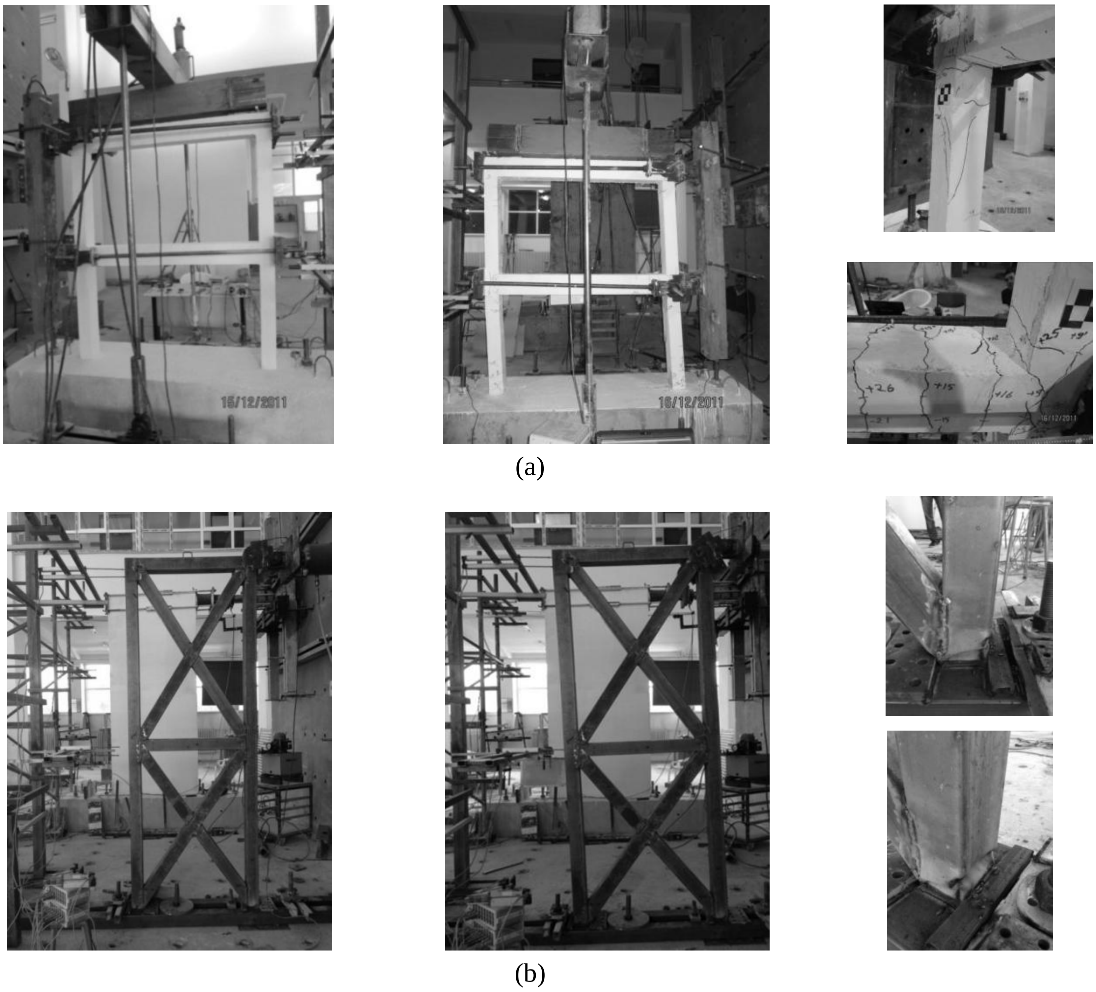


Fig. 10 Appearances before and after experiments, damages occurred during experiments: (a) RC frame; (b) 1/2CSBF$^{(-)}$-X

very close to each other. However higher values of displacements were measured at the point of maximum lateral load during experiments as compared to the pushover analyses. In RC frame, maximum lateral load obtained in experimental study was 29.67 kN and displacement corresponding to this load was 44.22 mm; maximum lateral load obtained in pushover analysis was 30.83 kN and displacement corresponding to this load was measured as 14.82 mm (Unal 2012). The difference in measured and calculated displacements is attributed to the post elastic (plastic) behavior of the frame section (due to tiny cracks, invisible deformations and etc…) during loading which cannot be modeled by pushover analyses. But the high accuracy of pushover analyses in estimating the lateral load carrying capacity of the modeled frames reveal that the load bearing mechanism was simulated successfully. For 1/2 CSBF$^{(-)}$-X specimen, maximum lateral load obtained in experimental study was 115.1 kN and displacement corresponding to this load was 60.81 mm; maximum lateral load obtained in pushover analysis was 113.60 kN and displacement corresponding to this load was measured as 13.84 mm.

In the experimental study performed for RC frame, first cracks occurred at the connection zone of column with the foundation and then the cracks were centered on column-beam connection zone. After reaching maximum load, the gaps between cracks rifted, resulting in big damages and the experiment was terminated when it has lost its stability. In the experiment performed for 1/2CSBF$^{(-)}$-X, twisting/buckling/local buckling occurred at the lower end of the column in 1/2CSBF$^{(-)}$-X under maximum lateral load and the experiment was terminated when it has lost its stability. The appearances of specimens before and after experiments are given in Fig. 10.

In 1/2CSBF$^{(-)}$specimens, lateral load-carrying capacities are listed in descending order as 1/2CSBF$^{(-)}$-K, 1/2CSBF$^{(-)}$-Λ, 1/2CSBF$^{(-)}$-X, 1/2CSBF$^{(-)}$-\, 1/2CSBF$^{(-)}$-V, 1/2CSBF$^{(-)}$-/and 1/2CSBF$^{(-)}$-non-braced. In 1/2CSBF$^{(+)}$specimens, lateral load-carrying capacities are given in descending order as 1/2CSBF$^{(+)}$-X, 1/2CSBF$^{(+)}$-Λ, 1/2CSBF$^{(+)}$-K, 1/2CSBF$^{(+)}$-\, 1/2CSBF$^{(+)}$-V, 1/2CSBF$^{(+)}$-/ and 1/2CSBF$^{(+)}$-non-braced. In 1/3CSBF$^{(+)}$specimens, lateral load-carrying capacities are ordered in descending order as 1/3CSBF$^{(+)}$-K, 1/3CSBF$^{(+)}$-Λ, 1/3CSBF$^{(+)}$-X, 1/3CSBF$^{(+)}$-\, 1/3CSBF$^{(+)}$-V, 1/3CSBF$^{(+)}$-/ and 1/3CSBF$^{(+)}$-non-braced. In 1/3RC+CSBF$^{(+)}$specimens, lateral load-carrying capacities are ordered in descending order as 1/3RC+CSBF$^{(+)}$-X, 1/3RC+CSBF$^{(+)}$-Λ, 1/3RC+CSBF$^{(+)}$-K, 1/3RC+CSBF$^{(+)}$-V, 1/3RC+CSBF$^{(+)}$-/, 1/3RC+CSBF$^{(+)}$-\ and 1/3RC+CSBF$^{(+)}$-non-braced.

1/2CSBF$^{(+)}$ braced designed in appropriate to TEC-2007 (2007) carried more load with 28% and 44% (except the non-braced specimen) rate than 1/2CSBF$^{(-)}$ braced in terms of lateral load-carrying capacity.

1/3CSBF$^{(+)}$ specimens carried more load than 1/3RC+CSBF$^{(+)}$specimens with the rate varying between 16%and 43% (except the non-braced specimen) in terms of lateral load-carrying capacity.

When load vs. displacement graphs were investigated, it was observed that V-braced specimen of braced with 1/2CSBF$^{(-)}$ had more ductile behavior than other specimens. At the same time, it can be said that this specimen had higher energy consumption capacity. V and K-braced specimens among the braced ones with 1/2CSBF$^{(+)}$ and with 1/3CSBF$^{(+)}$ had more ductile behavior than other specimens. The energy consumption capacities of these specimens were also higher than other specimens. For all specimens with1/3RC+CSBF$^{(+)}$, an increase in load-carrying capacity, ductility and energy consumption capacities was observed when compared to the ones with 1/3CSBF$^{(+)}$.

When hinge sequences for 1/2CSBF$^{(-)}$, 1/2CSBF$^{(+)}$, 1/3CSBF$^{(+)}$ were investigated, it was determined that a decrease was observed in lateral load as well as occurrence of hinge at the ends of columns. For 1/3RC+CSBF$^{(+)}$, the hinge first started in RC frame and then occurred in CSBFs. Moreover, decreases occurred in the lateral loads after occurrence of these hinges at the end of columns.

The load-carrying capacity obtained in the experiments performed for RC frame was 3.76% lower than load-carrying capacity obtained from pushover analysis. The load-carrying capacity obtained in the experiments performed for 1/2CSBF$^{(-)}$ was 1.3% higher than load-carrying capacity obtained from pushover analysis. This indicates that analytical study resulted in very close results to the experimental study.

The load-carrying capacity of 1/3 scaled RC frame strengthened with CSBFs substantially increased when compared to its first situation. When the results of pushover analyses were investigated, it was determined that 1/3RC+CSBF$^{(+)}$-X specimen carried 7.01 times more load, 1/3RC+CSBF$^{(+)}$-V specimen carried 5.49 times more load, 1/3RC+CSBF$^{(+)}$-Λ specimen carried 5.84 times more load, 1/3RC+CSBF$^{(+)}$-/ carried 5.08 times more load, 1/3RC+CSBF$^{(+)}$-\ carried 4.62 times more load, 1/3RC+CSBF$^{(+)}$-K specimen carried 5.70 times more load and 1/3RC+CSBF$^{(+)}$-non-braced specimen carried 1.45 times more load with respect to bare RC frame.

Hinge of RC column ends among experimental specimens and occurrence of big damages in this zone resulted in collapse failure position. In the experiment of 1/2CSBF$^{(-)}$-X, on the other hand, collapse failure position was reached with twisting/buckling/local buckling occurred at the lower end of the column.

4. Conclusions

Experimental studies and pushover analyses revealed similar results for the analyzed systems throughout this study. The load carrying capacity of RC frames with inadequate earthquake resistance were increased significantly by adding CSBFs. Specimens designed in accordance with TEC-2007 have more load-carrying capacities compared to those that are not designed conveniently with TEC-2007 (2007). The specimens having different scales (1/2 and 1/3) revealed similar behavior. One of the most significant findings of the study is that, structures can be strengthened by the suggested method, without complete evacuation and/or partial closure. For buildings to be strengthened via this technique, it is primarily suggested to add foundations outside of buildings, and connect them rigidly between each other and also with the main structure. Additionally, strengthening of RC buildings with Λ- braced and X braced gives better results in terms of ductility, energy consumption capacity and load-carrying capacity. Therefore, using Λ-brace and X brace is recommended for strengthening with CSBF.

In the light of these results, it was concluded that the seismically deficient RC frames can be strengthened by adding external concentric steel braced shear walls rapidly, easily and economically. Another advantage of this method is that, the structure can be strengthened without destroying the plasters, paintings and other finishings.

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