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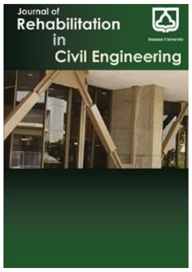


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# Experimental Study of Reinforced Concrete Frame Rehabilitated by Concentric and Eccentric Bracing

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**ABSTRACT**

Adding steel braces to reinforced concrete frames is a common way for seismic rehabilitation of these structures. Due to ease of installation and the possibility of creating openings in the braced bays, this method of rehabilitation has been more preferred than using shear walls. In this paper, three experimental specimens including a reinforced concrete frame, a reinforced concrete frame with concentric bracing and a reinforced concrete frame with eccentric bracing are constructed and their cyclic behavior investigated and compared with each other. Results show that the ultimate loads of the both concrete frames with concentric and eccentric braces are about 2.11 and 1.9 times more than that of reinforced concrete frame, respectively. Ductility of rehabilitated frame by eccentric bracing is more than that of reinforced concrete frame and rehabilitated frame by concentric bracing too. Moreover, the absorbed energy of the rehabilitated frames with eccentric and concentric bracing is about 1.98 and 1.63 times more than that of concrete frame.

## 1. Introduction

Using steel bracing systems for seismic rehabilitation of reinforced concrete frames is a common technique which many researches have been carried out about it. This method of retrofitting provides some advantages including minimal added weight to the structure, increasing the stiffness, decreasing the lateral displacement of concrete frame and the ability to accommodate openings in the braced bays. In 1987, the inelastic seismic response of reinforced concrete frames with concrete bracing members arranged in X and K patterns were investigated by Desai et al. [1]. In 1991, Badux and Jirsa studied the use of steel bracing systems for seismic retrofitting of inadequate reinforced concrete frames. It was concluded that the diagonal bracing provided

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an excellent approach for strengthening and stiffening the existing buildings under lateral loads [2]. In 1997, using steel bracing system in concrete-framed structures was investigated by Maheri and Sahebi. The test results indicated a significant increase in the strength and stiffness of the braced frame. Moreover, with proper connection between the brace and the frame, the steel bracing could be a suitable alternative or supplement to shear walls in concrete framed buildings [3]. In 2001, the seismic performance of a three-story reinforced concrete building retrofitted by eccentric steel bracing was analyzed by Ghobarah and Elfath. A three story office building was analyzed using various ground motion records and subsequently, effectiveness of the EBFsteel bracing in retrofittingof thisbuilding was studied. Results showed that performance of the building wasenhanced in terms of story drifts and damage indices [4]. In 2003, Shan and Di tested some experimental specimens including seven reinforced concrete braced frames, one reinforced concrete frame and one reinforced concrete shear wall under cyclic loading. It was concluded that the lateral resistance, stiffness and energy dissipation of the braced frames were more than that of the others [5]. Results of pushover experiments conducted on scaled models of ductile reinforced concrete frames, directly braced by steel X and knee braces was presented by Maheri et al. Results indicated that the yield and the strength capacity of a ductile RC frame increased and theultimate displacements decreased by adding either an X or a knee-bracing system to the frame. It was concluded that that both X and kneebracing systems could be used for designing or retrofittingagainst a damagelevel earthquake and for a collapselevel earthquake, kneebracing is a more effective system [6]. Maheri and Hadjipour studied the experimental investigation and design of steel brace connection to RC frame. Three types of connections were investigated. Results showed a good agreement with the design strength predictions of individual elements in each connection [7]. In 2006, seismic rehabilitation of a 7-story reinforced concrete residential building with concentric steel bracing system was investigated by Hemmati and the optimal arrangement of the bracing system presented too [8]. Ghaffarzade and Maheri investigated the cyclic behavior of internally braced reinforced concrete frame. Results indicated that the bracing system enhanced the strength capacity of the RC frame while maintaining adequate ductility [9]. Mechanical compression release device in steel bracing system for retrofitting of the RC frames were studied by these authors too. Results showed thatthis device could be used in braced systems to prevent buckling failure of the bracing members and subsequently ductility of these systems increased [10]. In 2007, Youssef et al. evaluated the seismic performance of a reinforced concrete frame with concentric steel bracing and compared the results with that of reinforced concrete frame. Test results showed that the braced frame resisted higher lateral loads than that of the moment frame and provided adequate ductility [11]. In 2008, Kheyroddin presented some failure patterns for strengthened reinforced concrete frames with steel braces [12]. Ghaffarzade and Maheri studied the connection overstrength in steel-braced RC frames by experimentaland numerical investigations and the level of capacity interaction between the two systems was discussed. It was concluded that the capacity interaction is primarilydue to the connections overstrength and some guidelines for the seismic design of the internally cross-braced RC frames with direct connections were presented [13]. Massumi and Tasnimi also studied different connection details of steel bracing systems and reinforced concrete frames. Experimental tests were carried out on eightRC frames with different details including bolts and nuts, steel jackets around thecolumns and embedded plates in frame corners. Results indicated considerableincrease in the lateral strength andductility of strengthened frames upon bracing details [14]. In the same year, Ghodrati Amiri and Gholamrezatabar studied the capacity of energy dissipation of a shear link in a reinforced concrete building rehabilitated by eccentric steel bracing. The results showed that shear link energy dissipation capacity can improve seismic performance of RC buildings [15]. Said and Nehdi also proposed a new beam-column joint rehabilitation technique using local steel brace members. Two specimens including a standard joint and a rehabilitated joint were made and tested under cyclic loading. It was concluded that the rehabilitation technique enhanced the overall performance of the deficient joint [16]. In 2009, Mazzolani et al. investigated the use of two steel dissipative steel bracing systems including eccentric bracing and buckling restrained bracing, for seismic upgrading of the structures [17]. In 2013, Massumi and Absalan studied the interaction between the steel bracing system and the reinforced concrete frame. For evaluation of interaction between bracing and reinforced concrete frame, a new numerical model was developed. The results showeda considerable interaction effects in enhancement of seismic characteristics of compound system especially on increasing of energy damping [18]. Hemmati et al. evaluated the behavior of large scale bracing systems in tall buildings and the effective influence of this system on behavior of these structures was studied [19]. In 2014, Umesh and Shivaraj examined the seismic response of reinforced concrete structures with various steel bracing systems. 7 models with different steel bracing types were selected and analyzed. It was concluded that X steel bracing system was more suitable case for enhancing the capacity of buildings [20]. Karthic and Vidyashree in 2015 presented the effect of the steel bracing on seismic behavior of vertically irregular reinforced concrete buildings. X, V and inverted V/K types of bracing were added to the reinforced concrete frame and it was found that the X type was the best option for enhancing the performance of building [21]. Huang et al. examined the seismic behavior of Chevron bracing in reinforced concrete frames. Test results indicated that reinforced concrete frames withbraces exhibited better performance than plain reinforced concrete framesin terms of strength, stiffness degradation, hysteresisloop, and energy dissipation [22]. Ince et al. also investigated the seismic behavior of one story reinforced concrete frame rehabilitated by an eccentric bracing with vertical link element. Link element was designed and used as a shear element to evaluate the effect of the change at the length ofthis link on the behavior of system by applying an eccentrically braced system in the shape of “Y” connected vertically to thebeam. It was concluded that using this system improved the energy dissipation and lateral load bearingcapacities of the lean RC specimen [23]. In 2017, Gong et al. presented the different methods of rehabilitation of reinforced concrete buildings with steel bracing and reviewed the research status of strengthening RC structures with braces [24]. In 2019, Seismic evaluation of reinforced concrete moment frames retrofitted with steel braces using IDA and Pushover methods in the near-fault field were investigated by Kheyroddin et al. Two ten-story concrete frames with five spans were designed and analyzed. The results indicated that using of EBF braces in a concrete frame reduces up to 7 times the amount of base shear applied to the building relative to the CBF frame [25].

As it mentioned, there are two main categories for using steel braces in RC frames including external and internal bracing systems. In the external bracing system, RC buildings are rehabilitated byadding a steel bracing system to the exterior or interior frames. Architectural problems and design of an appropriate connection of the steel bracing to the RC frames are two of the main shortcomings of this method. In the second method, the RC frames are retrofitted by positioning a braced frame system inside the bays of the RC frames and the transfer of load between the steel bracing and the concrete frame is performed indirectly through the steel frame. Technical difficulties in fixing the steel frame to the RC frame are the main problems of this system.

In direct connection systems, the steel braces are directly connected to the RC frames without the use of an intermediary steel frame. In the first technique, the concrete beams and columns around the connections are jacketed with steel plates and the gusset is subsequently welded to the steel jacket. In the second manner, a steel plate is bolted to the connection face of the concrete member and the gusset is welded to this plate.As it seen, a great amount of research has been carried out on using various steel bracing systems in a reinforced concrete frame. But, because of the technical difficulties in determining the length of reinforced concrete link beam, the seismic behavior of a steel braced concrete frame with direct connection between the eccentric braces and the frame (beams and columns) has been less investigated. Although, concentric bracing provides the reinforced concrete frame with the required stiffness and strength, it exhibits rapid degrading behavior due to buckling of the brace members. In eccentric braced frames, the lateral loads are transmitted to the braces through bending and shear forces developed in link elements. In this paper, the cyclic behavior of three specimens, including a reinforced concrete frame (RC), a reinforced concrete frame with concentric steel bracing (CBF) and a reinforced concrete frame with eccentric steel bracing (EBF) are investigated and their cracking and failure patterns are studied. The concrete beams and columns around the connections are jacketed with steel plates and the braces attached to the welded gusset plates. Different bracing systems with direct connection to RC frame are evaluated and the results discussed.

## 2. Materials

Coarse aggregate with maximum size of 19 *mm*, fineness modulus of 7.38, water absorption ratio of 0.6% and specific gravity of 2.61 was used for experimental work. Natural river sand with fineness modulus of 2.69, water absorption ratio of 0.8% and specific gravity of 2.55 was used too. Portland cement type II was used for this experimental study. Particle size curves of the used aggregates associated with the permitted ranges of ASTM are shown in Figures 1 and 2.

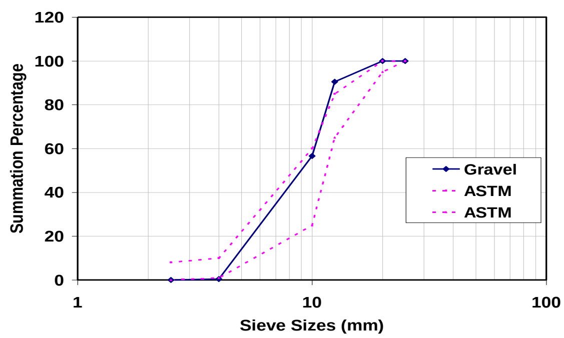


**Fig. 1.** Particle size curve of coarse aggregates.

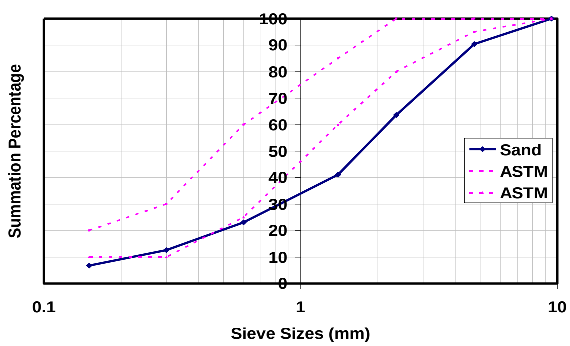


**Fig. 2.** Particle size curve of fine aggregates.

The Concrete mix design is presented in Table 1. The concretewas poured and cured for 28 days. For each frame, three cubic specimens were tested after 28 days of curing todetermine the compressive properties of the concrete. Compressive strength of the used concrete was determined about 26 *MPa*. Yield stress of steel reinforcements and steel plates are measured 400 and 240 *MPa* respectively.

**Table 1.** Concrete mix design ( $kg/m^3$ ).

| Cement (type II) | gravel | Sand | Water |
|---|---|---|---|
| 370 | 802 | 980 | 185 |

## 3. Experimental Specimens

The size of the experimental specimens was selected in consideration of laboratory and practicalengineering requirements. The test specimenswere 1/2.5 scale in size.Shear hinges have a high level of energydissipation capacity. Selectingan appropriate length concludes tolarge plasticdeformation and does not involve inelastic instability such as buckling according to Eq. 1. Where, e, $V_p$ and $M_p$ are the length, plasticshear and plastic moment of the link beam respectively. This equation is used to provide the conditions for the formation of shearhinge before the flexural hinge in the horizontal link beam.

$$e \le 1.6\frac{M_p}{V_p} \tag{1}$$

This equation has been proposed for steel beams and columns and it stated that a link beam with length of less than 1/5 of beam span was appropriate for eccentric bracing. Hence, because of practical requirements and existence of adequate space for welding process, the length of the link beam was selected about 250 *mm* (measured on centerline of the beam). Some parameters for seismic design of reinforced concrete frames retrofitted by steel bracings have been proposed by researchers [26, 27]. The RC frame was located in highly seismic area and designed according to Standard No. 2800 with modification factor of 5 (moment frame with moderate ductility). But the reinforcement ratio of the columns ( $\rho = \frac{A_s}{b \cdot d} = \frac{314}{200 \times 200} = 0.00785$ ) was less than minimum reinforcement ratio ( $\rho_{\min} = 0.01$ ). In the other hand, the columns of this frame were weak and must be retrofitted [28].

Three test specimens were selected for this experimental study. The clear span of the whole frames is 800 mm, with a total span of 1200 mm. The cross section of the beams is 150 mm deep by 200 mm wide. The total height of the frames was 1450 mm, and the cross section of the all columns was 200 mm deep by 200 mm wide. Structural details of the experimental specimens including a reinforced concrete frame (reference specimen), a reinforced concrete frame with concentric bracing and a reinforced concrete frame with eccentric bracing are shown in Figures 3, 4 and 5 respectively. The frames were formed and casted in reclined position. After the required curing period and removing the form works, the frames were lifted to their upright position. The mat base of the each frame was bolted to the laboratory strong floor, thus giving an essentially fully-fixed condition.

As it shown in these figures, the distances between the stirrups in the beams and columns of the whole frames were 100*mm*. But, these distances were reduced to 50*mm* in the critical areas of the beams and columns. Each of the bracing members consisted of two equal angles. At the connection between the bracing members and the frames, some gusset plates were used. Four angles with a thickness of 6 *mm* were placed at the corners of the frame members (beam, columns and foundation) and four steel plates with a thickness of 8 mm were welded to these angles according to Figures 4 and 5. These steel plates surrounding the frame members were welded to each other too. Then the gusset plates were welded to the surrounding plates. In the other word, the connection between the bracing members and the frames provided by means of these steel plates attached to the frame members and gusset plates. The plates attached to the end of the columns provided some confinement to the concrete. The length and dimensions of the welding are also indicated in these figures.

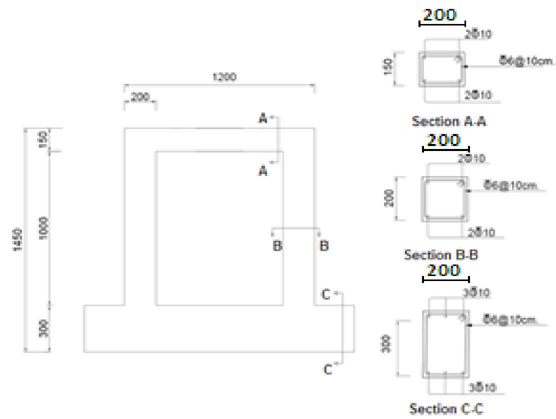


**Fig. 3.** Dimensions and details of RC specimen (*mm*).

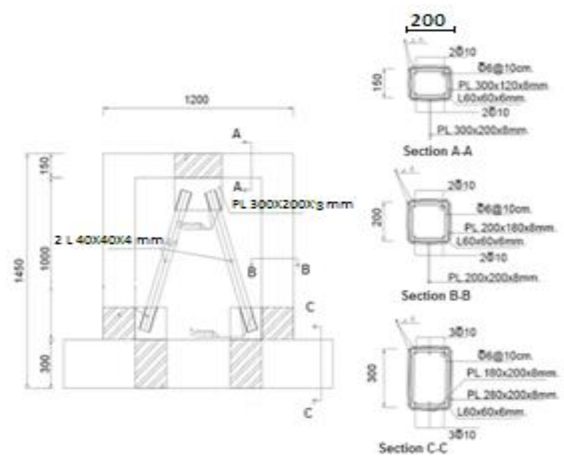


**Fig. 4.** Dimensions and details of CBF specimen (*mm*).

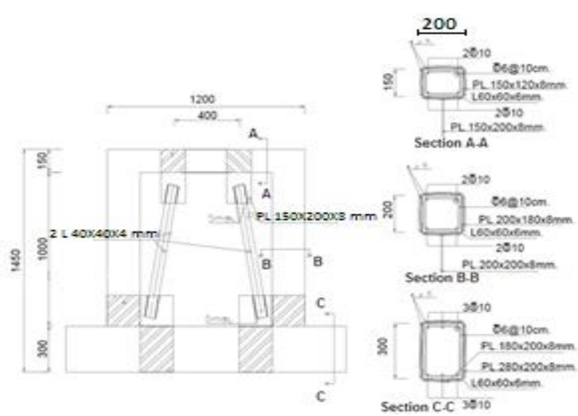


**Fig. 5.** Dimensions and details of EBF specimen (*mm*).

Test set up is shown in Figure 6. The loading protocol is also presented in Figure 7 [29].

**Fig. 6.** Test set up.

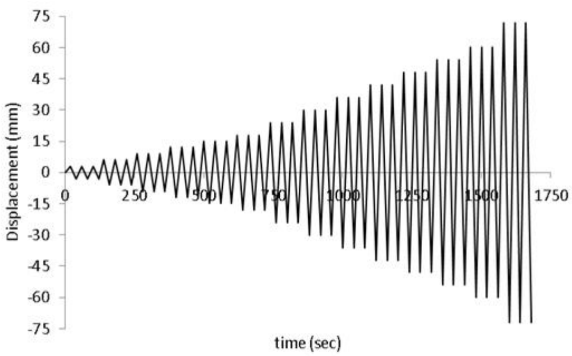


**Fig. 7.** Loading protocol [29].

The steel plates surrounding the beam and columns are welded together and attached to these members by epoxy adhesive.

## 4. Observations and Discussion

### 4.1. RC Frame

The first cracks were observed at the lateral load and displacement of 30 *kN* and 7.1 *mm* respectively. These initial cracks occurred at the external parts of the right column, near the beam-column junction. The second and third cracks appeared at the both columns top near the column-foundation connections. The fourth inclined crack was observed at the external parts of the left column, within the beam-column joint. These initial cracks are shown in Figure 8. Further loading caused the cracking to spread at the beam-column junctions and column-foundation connections of the reinforced concrete frame, as shown in Figure 9. With increasing lateral load, most parts of the concrete around the critical sections (around the connections) cracked and crushed and finally, the frame carried the lateral load of about 104 *kN* and displacement of about 44 mm. The condition of the RC frame at the end of loading is shown in Figure 10.

The hysteresis curve of this frame is shown in Figure 11. As it shown, when the lateral displacement of the frame exceeds than 30 mm, the lateral load is slightly reduced. In the other hand, after displacement of 30 mm, a slightly softening behavior is observed in the hysteresis curve of this frame.

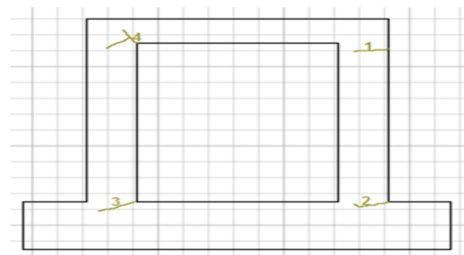

**Fig .8.** The first cracks in RC frame.

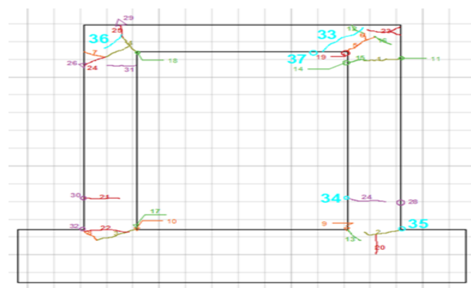

**Fig. 9.** Cracking in RCframe at the end of test.

**Fig. 10.** RC frame at the end of loading.

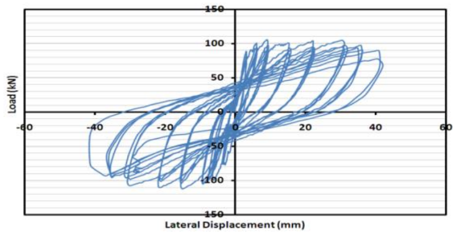


**Fig. 11.** Hysteresis curve of the RC frame.

## 4.2. CBF Frame

The first and second cracks were appeared at the lateral load and displacement of 41.6 *kN* and 7 *mm* respectively. These initial cracks occurred at the external parts of the left and right columns, near the beam-column junction. The third and fourth inclined cracks appeared at the both columns top near the column-foundation connections. These initial cracks are shown in Figure 12. Further loading caused the cracking to spread at the beam-column junctions and column-foundation connections of the concentric braced frame as shown in Figure 13. With increasing lateral load, most parts of the concrete around the critical section (around the connections) cracked and crushed and finally, the frame carried the lateral load of about 220 *kN* and displacement of about 42.8 *mm*. The condition of the CBF frame at the end of loading is shown in Figure 14.

The hysteresis curve of this frame is shown in Figure 15. As it shown, a slightly hardening behavior is observed in the hysteresis curve of this frame. But, pinching phenomena occurs in the hysteresis curve too. It seems that, buckling of concentric braces concludes to this effect.

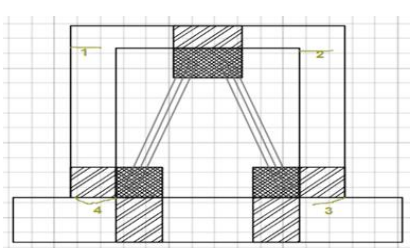

**Fig. 12.** The first cracks in CBF.

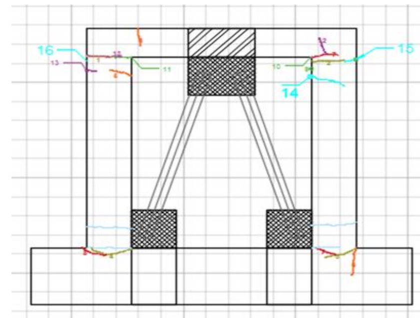

**Fig. 13.**Cracking in the CBF at the end of test.

**Fig. 14.**CBF specimen at the end of loading.

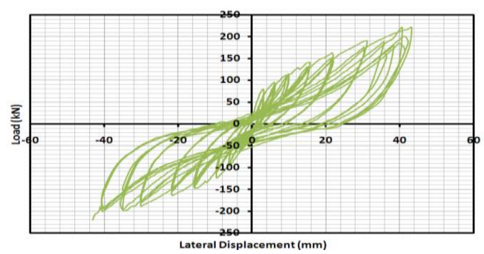


**Fig. 15.** Hysteresis curve of CBF.

The force in the steel bracing member is transmitted to the reinforced concrete frame through the steel plates attached to the ends of the beam and columns as shown in Figure 4. The load of the braces is transferred to the beam and columns by these gusset plates. Moreover, these gusset plates have a confinement effect on the concrete. This bracing system resists the lateral loads by truss action and consequently, significant axial force transfers to the columns. The axial force which is developed in the beam is small compared to that of columns.

Initial crack pattern of the concentric braced frame is similar to that of reinforced concrete frame. But, further loading causes the cracking to spread at the beam-column junctions and some new cracks are observed in the bottom and top of the columns. This cracking pattern is different from that of the reinforced concrete frame. Moreover, the number of cracks in this frame is about 16 cracks which is less than half of the cracks in the reinforced concrete specimen. Ultimate lateral load of this rehabilitated frame is about 220 *kN* which is 2.11 times more than that of reference frame. After removing the surrounding steel plates from the beam and columns, some cracks are observed at the bottom of the columns (about 100*mm* above the column-foundation connection). These cracks may be due to significant axial forced which are developed in columns. Few cracks are observed in the beam too.

### 4.3. EBF Frame

The first and second inclined cracks were appeared at the lateral load and displacement of 40 *kN* and 7.2 *mm* respectively. These initial cracks occurred at the beam-column joint and spread toward the beam. The third and fourth inclined cracks appeared at the external part of the right column, near the beam-column junction. These initial cracks are shown in Figure 16. Further loading caused the cracking to spread at the beam-column junctions and column-foundation connections of the EBF frame, as shown in Figure 17. With increasing lateral load, most parts of the concrete around the critical section (around the connections) cracked and crushed and finally, the frame carried the lateral load of about 198 *kN* and displacement of about 51 mm. The condition of the EBF frame at the end of loading is shown in Figure 18.

The hysteresis curve of this frame is shown in Figure 19. As it shown, a hardening behavior is observed in the hysteresis curve of this frame. Pinching phenomena is less than that of concentric braced frame.

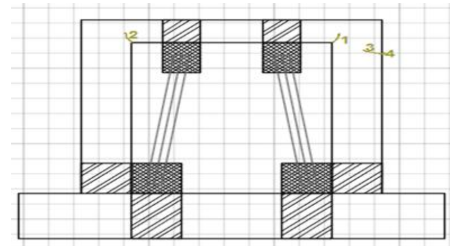

**Fig. 16.** The first cracks in EBF.

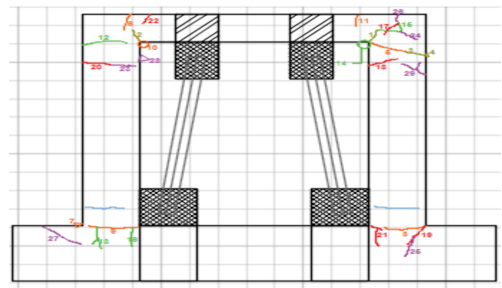

**Fig. 17.** Cracking in the EBF at the end of loading.

**Fig. 18.** EBF at the end of loading.

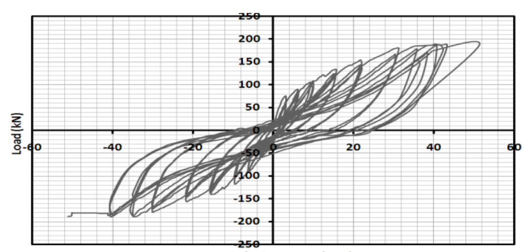


**Fig. 19.** Hysteresis curve of EBF.

The force in eccentric steel bracing is transmitted to the frame by bearing action on reinforced concrete members. The load of braces is transferred to the beam and columns by some gusset plates. This bracing system resists the lateral load by developing axial forces in the beam and columns. Crack pattern of the EBF is similar to that of RC frame. The number of cracks in this frame is about 30 cracks which is less than that of reinforced concrete specimen and more than that of concentric braced frame. Ultimate lateral load of this rehabilitated frame is about 198 *kN* which is 1.9 times more than that of reference frame. After removing the surrounding steel plates from the beam and columns, some cracks are observed at the bottom of the columns (about 100 *mm* above the column-foundation connection). These cracks may be due to axial forced which are developed in columns. Some cracks are observed in the beam too. A summary of the results of these three specimens is shown in Table 2. Where, $P_{cr}$ =cracking load, $P_y$ =yielding load, $P_u$= ultimate load, $\Delta_{cr}$ =cracking displacement, $\Delta_y$ =yielding displacement, $\Delta_u$ =ultimate displacement and $\mu = \frac{\Delta_u}{\Delta_y}$ (ductility ratio).Yielding loads and displacements were measured through the envelope curves of the specimens and based on idealized bilinear capacity curves of them [30].

**Table 2.** Summary of results.

| Specimens | $P_{cr}$ $(kN)$ | $\Delta_{cr}$ $(mm)$ | $P_y$ $(kN)$ | $\Delta_y$ $(mm)$ | $P_u$ $(kN)$ | $\Delta_u$ $(mm)$ | $\mu$ |
|---|---|---|---|---|---|---|---|
| RC | 30 | 7.1 | 85 | 9.1 | 104 | 44 | 4.84 |
| CBF | 41.6 | 7 | 144 | 9.5 | 220 | 42.8 | 4.5 |
| EBF | 40 | 7.2 | 111 | 9.2 | 198 | 51 | 5.54 |

As it presented in Table2, the racking loads of the concentric and eccentric braced frames are about 1.39 and 1.33 times more than that of the reference specimen respectively. Regarding the ultimate loads, these values are about 2.11 and 1.9 times respectively. It seems that using steel bracing concludes to more stiffness and subsequently, less lateral displacement compared to the moment RC frame. Ultimate load of the concentric braced frame is more than that of other frames. But, because of buckling of the braces, the ductility decreases. Hysteresis curves of these three specimens are shown in Figure 20. Initial stiffness of CBF is more than that of EBF and RC frame. As shown in this table, the cracking loads of the CBF and EBF frames are about 1.39 and 1.33 times more than that of the reference specimen respectively. Regarding the ultimate loads, these values are about 2.11 and 1.9 times respectively.

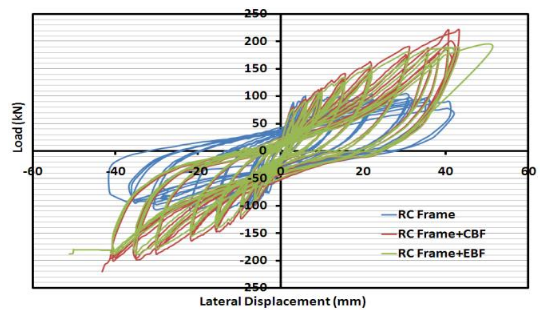


**Fig. 20.** Hysteresis curves of the experimental specimens.

The area under the hysteresis curves of these frames which is defined as absorbed energy is calculated and compared with each other too. Absorbed energy of the CBF and EBF frames is about 1.63 and 1.98 times more than that of reference frame. No tearing or buckling is observed in gusset plates of connections between the bracing and frames. Load capacity of the CBF is more than that of other frames. But, because of buckling of the braces, the ductility decreases. No buckling is observed in eccentric braced frame and subsequently the ductility increases and the ductility of this rehabilitated frame is more than that of others. It seems that in the case of one story one bay RC frame, the influence of using concentric and eccentric steel bracing systems on the ultimate loads of the braced frames are more than that of ductility. However, the ductility of EBF frame is about 1.14 times more than that of RC frame and this bracing system increases the ductility of the RC frame. In the other hand, adding steel bracing to low rise frames have small effect on ductility of them. Moreover, steel jacketing especially around the columns provides a confinement effect on concrete and decreases the damages of concrete parts.

## 5. Conclusion

The maximum number of cracks and the minimum ultimate lateral load occurred in the RC frame. The minimum number of cracks and the maximum ultimate lateral load occurred in the CBF frame too. The ductility of EBF frame is more than that of other frames. The cracking patterns of RC and EBF frames are similar to each other and are different from that of CBF frame. The ultimate loads of CBF and EBF frames are 2.11 and 1.9 times more than that of RC frame, respectively. Moreover, the absorbed energy of rehabilitated frames with EBF and CBF bracings is about 1.98 and 1.63 times more than that of RC frame. It seems that using steel bracing for rehabilitating of a one bay one story RC frame concludes to more ultimate loads and the variation of ductility is less than 15 %. But, adding steel bracings results in different failure mechanisms of the retrofitted specimens and concludes to more absorbed energy. Adding eccentric bracing system concludes to more ultimate load and ductility compared to RC frame. But concentric bracing results in more ultimate load than that of RC frame.

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